



## **Geotechnical Investigation for the Proposed Development**

**6515 145A Street NW  
Edmonton, Alberta**

Submitted to:  
Birkholz Homes Inc.  
8872 – 48 Avenue  
Edmonton, Alberta, T6E 5L1

Submitted by:  
LWL Engineering Ltd.  
4207 Ramsay Crescent  
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File No. A-0277  
February 29, 2016



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Kevin Birkholz  
Birkholz Homes Inc.  
8872 – 48 Avenue  
Edmonton, Alberta, T6E 5L1

Dear Mr. Birkholz:

**Re: Geotechnical Investigation for the Proposed Development at 6515 145A Street NW in Edmonton, Alberta**

We are pleased to submit a geotechnical report for the proposed development located at 6515 145A Street NW in Edmonton, Alberta. This report provides a review of available geotechnical information, a summary of subsurface soil and groundwater conditions, and a slope stability analysis. Based on the existing slope stability conditions, the setback line for the proposed development has been defined. Furthermore, recommendations for foundation design, grading, drainage, and maintenance of the setback zone and the slope abutting the lot area are included.

We thank you for the opportunity to complete this project in your behalf. Should you have any questions or require additional information, please do not hesitate to contact us.

Sincerely,

**LWL Engineering Ltd.**

A handwritten signature in black ink, appearing to read 'M Petrov', is written in a cursive style.

Mikhail Petrov, B.Sc., E.I.T.  
Geotechnical Engineer



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## 1 INTRODUCTION

LWL Engineering Ltd. (LWL) has conducted a geotechnical investigation for the property located at 6515 145A Street NW in Edmonton, Alberta (Lots 1 and 2, Block 10, Plan 2478MC). The subject lots are situated adjacent to three slopes: the slope to the north overlooks the Fort Edmonton Park area along the North Saskatchewan River, the slope to the east is towards the Whitemud Drive on-ramp from Fox Drive, and the slope to the southeast is towards Whitemud Drive. Henceforth, these slopes will be referred to as the “North Slope,” “East Slope,” and “South Slope,” respectively.

The existing house was constructed on the property in 1971. It is understood that the existing house will be demolished, and a new house will be constructed.

The objectives of this investigation were to determine the soil stratigraphy and groundwater level and to perform a slope stability analysis of the site. Based on the analysis, a minimum safe setback distance from the Top-of-Bank (TOB) of each abutting slope was established in accordance with the City of Edmonton Policy C542. Recommendations regarding foundation, site grading, drainage and maintenance of the setback zone and the slope abutting the subject area are also provided.

## 2 METHOD OF INVESTIGATION

The geotechnical investigation of the site included the following:

- Review of available geotechnical data and aerial photographs;
- Site reconnaissance;
- Field drilling program and laboratory testing;
- Slope stability analysis; and
- Geotechnical evaluation and foundation considerations.

### 2.1 Review of Available Geotechnical Data and Aerial Photographs

Kathol and McPherson’s Urban Geology of Edmonton (1975) was reviewed to assist in the determination of expected local soil stratigraphy.

Aerial photographs from 1978, 1990, 1999 and 2009 were reviewed to assist in the finding of any relevant historical features or topographical changes within the subject area since 1978. More recent photographs were not available in an appropriate scale to assess local topographic features.

No geotechnical reports were available for nearby areas from the City of Edmonton archives.



## 2.2 Site Reconnaissance

Site reconnaissance was conducted by Ljubo Bijeljanin, P. Eng. on November 12, 2015 to assess the subject area and existing slope conditions.

## 2.3 Field Program

Two test-holes were originally planned, but three were drilled to accurately represent the stratigraphy of each of the three slopes abutting the subject lots. Test-holes TH-01 and TH-03 were drilled to 19 m, while TH-02 was drilled to 24 m, which was 5 m into bedrock. The test-hole locations are shown in Figure 1.0 in Appendix A.

The test-holes were advanced using a track-mounted, 150 mm diameter solid stem auger operated by Canadian Geological Drilling Ltd. on December 1, 2015. The soil profile was visually examined and logged.

Standard Penetration Tests (SPT) were conducted at 1.5 m intervals to obtain an estimate of the consistency and strength of the various soil strata encountered. The SPT values (N) are shown on the attached test-hole logs in Appendix B. Soil samples from the SPT's split spoon sampler and grab samples from the auger were collected at regular intervals and submitted to the laboratory for index testing.

Standpipe piezometers of 25 mm diameter slotted PVC pipe were installed in each of the three test-holes to facilitate further groundwater monitoring. Details of the piezometer installations are shown on the test-hole logs in Appendix B. Groundwater was visually monitored during drilling, and measured at drilling completion and on January 18, 2016, 48 days after drilling.

## 2.4 Laboratory Testing

The laboratory testing program included the determination of the moisture content of each collected sample. Gradation, Atterberg Limits, and water soluble sulphate concentration tests were also performed on selected samples. The laboratory test results are summarized on the test-hole logs in Appendix B and presented in their original form in Appendix C. Based on the laboratory results, soils were classified as per the Modified Unified Soils Classification System (MUSCS). A description of the Modified USCS is presented in Appendix B.

## 2.5 Slope Profile Survey

A contour map was obtained from the City of Edmonton to establish slope profiles. It was found that the North Slope was not well represented on the contour map as it appears to have been cut to accommodate the construction of the existing house. Delta Land Surveys Ltd. were contracted to perform a profile survey of the South Slope and the upper portion of the North Slope to determine the geometry of these slopes and aid in establishing the top-of-bank lines. The contours were modified based on the results of the survey and additional site reconnaissance to



accurately reflect the existing slopes and site conditions and are shown on the plan view in Figure 1.0 in Appendix A.

## 2.6 Slope Stability Considerations

A slope stability analysis was performed using GeoStudio 2007 Slope/W software on four cross sections of the overall site deemed to govern the slope stability for each of the three slopes in the area under consideration (one cross section per slope). The sections of the North Slope and the South Slope were joined together and are presented as one cross section (A-A') in Figure 2.0 in Appendix A, while the East Slope is sectioned as B-B'. Additional slope stability analysis for the North Slope was conducted on section C-C'. The cross sections are also shown in plan view in Figure 1.0. A comprehensive limit equilibrium method (Morgenstern-Price) was used for the slope stability analysis. Soil parameters and groundwater conditions were defined based on the investigation results and available data.

A target factor of safety of 1.50 was adopted to establish the setback line as the minimum safe distance from the TOB for the proposed development. The TOB was determined from the contour map obtained from the City of Edmonton and modified based on the survey of the North and South slopes.

## 3 REVIEW OF AVAILABLE DATA

### 3.1 Geological Maps

Based on Kathol and McPherson's Urban Geology of Edmonton (1975), from a surface elevation of 668 m, the expected soil profile within the area consists of the following:

- Glaciolacustrine deposits: sediments deposited in glacial lakes, comprised of bedded sands, silts, and clays to an approximate depth of 8.0 mBGS;
- Glacial till: unsorted, unstratified sediment deposited by a glacier, composed of clay, silt, sand, and gravel with pebbles and boulders to an approximate depth of 25.0 mBGS;
- Edmonton Formation bedrock: interbedded bentonitic shales and sandstones with numerous coal seams, at an approximate elevation of 643 m above sea level.

### 3.2 Aerial Photographs

Aerial photographs (stereo pairs) of the site area from 1978, 1990, 1999, and 2009 were reviewed to assess topographical or other relevant changes. The following was noted:

1978 - The North Slope was well vegetated with mature trees in 1978. The East Slope was not vegetated, smooth, and appeared to have been recently cut or graded to accommodate the off-ramp from Fox Drive to Whitemud Drive. The South Slope was treed with smaller



trees and was likely previously cut as well for the construction of Whitemud Drive. No signs of instability or previous failures were evident. The top-of-bank of the East Slope was well defined approximately 30 m from the main structure of the subject house. The top-of-bank lines of the North and South slopes were obscured by tree canopies and not clearly discernable from the air photos, but the retaining wall structure running along the South Slope was clearly visible approximately 10 m from the southeast corner the subject house. The surrounding area was mostly developed, but a few empty lots remained. The construction of Fort Edmonton Park on the river terrace beneath the North Slope was well underway, with much of the area deforested and graded.

1990 - A slope instability became visible on the face of the North Slope, approximately 400 m west of the subject site. The noted instability was not covered in the 1978 aerial photos, so it is unclear whether this movement occurred between 1978 and 1990 or in fact prior to that. It is possible that it is a very old, dormant slide, predating 1978. No exposed earth was visible from the air photos due to the dense vegetative cover. Based on the shape of the visible depression, the main scarp appears to have been approximately 30 m from 66 Avenue NW, which runs near the top-of-bank. This instability appears to have been shallow in nature.

With the exception of this observation, only minor changes occurred to conditions observed in 1978. Vegetation was visibly denser on the South Slope, and numerous young trees were apparently planted along ditches by the Whitemud Drive, as well as all around the Fort Edmonton Park area.

1999 - No significant changes. Vegetation in the area continued spreading, including considerable growth near the toe of the East Slope.

2009 - No significant changes. Vegetation in the area continued spreading. Growth near the toe of the East Slope became considerable, with young to mature spruce occupying the area.

## **4 EXISTING SITE CONDITIONS**

### **4.1 Site Reconnaissance**

The following observations were recorded during site reconnaissance conducted by Ljubo Bijeljanin, P. Eng., on November 12, 2015, when no snow cover was on the ground:

The subject area consists of a relatively flat top-of-bank area surrounded on three sides by slopes: a slope to the north, towards Fort Edmonton Park; a slope to the east, towards the Whitemud Drive on-ramp from Fox Drive; and a slope to the southeast, towards Whitemud Drive. No signs of settlements or cracking were noted within the top of bank area. The three slopes abutting the subject property are described separately as follows.





### 1. North Slope:

- The North Slope spans approximately 115 m horizontally from T.O.B. to the toe at Fort Edmonton Park in 3 stages: a short slope approximately 35° steep for 8 to 12 m from the top-of-bank (about 5-8 m drop), to a flat area for approximately 20 m, to a long, relatively smooth slope approximately 20° steep to the bottom of the slope.
- The 'flat area,' a small terrace on the face of the North Slope, appears to be an area of ponding, with a visible drainage channel likely conveying water to it in the spring and summer. This area is clearly identifiable on the contour map, which is superimposed on the plan view in Figure 1.0.
- The rest of the slope towards fort Edmonton Park, beyond the aforementioned ponding area, is mostly consistent in grade and moderately covered with brush and trees. Local, steeper areas; slight humps; and uneven areas could suggest old erosion channels or ongoing shallow sloughing, but there is no evidence of any large scale instability.
- The upper portion of this slope was likely modified with the construction of the existing house. The area between the house and the top of bank was probably cut to accommodate construction of the existing walkout basement, changing the geometry of the face of the slope and the top of bank line. The ground elevation adjacent to the house is approximately 2-3 m lower than adjacent areas of the lots, as is the top-of-bank line. The existing house is positioned approximately 8 m from the top-of-bank line.
- A small concrete retaining wall runs beside the house and backyard area alongside this slope. The retaining wall appears to be in decent condition. Other retaining structures have been built along this slope, including several small cinder-block retaining walls near the top of bank area to the east and west of the existing house. These retaining walls have not fared well over time, leaning and toppling over in places.
- Vegetation consists of a mix of brush; younger trees; and some large, older trees.

### 2. East Slope:

- The East Slope spans approximately 60 m horizontally from T.O.B. to the toe at the Fox Drive off-ramp to Whitemud Drive at an incline of approximately 18°.
- This slope has only grass growing on it, except near the toe where a large cluster of 15-25 year old spruce trees now grows, as noted on the historical air photos.
- This slope appears stable as no evidence of any movements was found.

### 3. South Slope:

- This slope is about 25 m long measured horizontally from the top-of-bank, at an angle of approximately 34°. The elevation gain is approximately 16 m.
- A large, curved, concrete retaining wall structure exists along the slope, situated approximately along the top-of-bank.
- Vegetation on this slope consists primarily of brush and young deciduous trees. A few larger trees are present as well. Some trees are leaning downslope, but this may be attributable to their tendency to lean towards the sun rather than a definite movement of soil on the slope.



- It is possible that a shallow failure occurred on this slope a long time ago, and some sloughing could occur in the future. Any such local failures would not affect the overall stability of the slope.

## 4.2 Subsurface Conditions

The soil stratigraphy encountered was fairly consistent across all three of the drilled test-holes and generally consisted of topsoil overlying silty sand, overlying clay till, overlying sand, overlying bedrock. A brief description of each soil type encountered is included below, while a detailed soil description is provided on the test-hole logs in Appendix B.

### 4.2.1 Topsoil

A layer of topsoil was encountered in the upper 0.3 m of soil. The topsoil was a mixture of clay and silt; contained a trace of rootlets; was firm to stiff, damp to moist, and dark brown in colour.

### 4.2.2 Upper Sand

A layer of sand termed “upper sand” was noted beneath the topsoil to a depth of approximately 6 mBGS. This sand was silty; contained some clay, traces of gravel, coal, and rust; was fine grained, firm to stiff, damp to moist, and light brown in colour.

The moisture content of the upper sand ranged from 3.9% to 12.3%, with an average moisture content of 8.8%. Atterberg Limits were tested on one sample with more visible fines and cohesive behaviour and showed a liquid limit of 27.0%, plastic limit of 12.8%, and a plasticity index of 14.2%. The SPT values (N) for the upper sand ranged from 9 to 34 with an average of 21 blows.

### 4.2.3 Clay Till

Clay till was encountered from approximately 6 mBGS to 13 mBGS. The clay till was silty; contained some sand, traces of gravel, rust, and coal; was very stiff to hard, damp to moist, and light brown to light grey in colour. Numerous sand lenses were encountered.

The moisture content of the clay till ranged from 8.8% to 20.2%, with an average of 13.9%. Atterberg Limits were tested on one sample and showed a liquid limit of 33.1%, plastic limit of 12.6%, and a plasticity index of 20.5%, indicating medium to low plasticity. The SPT values (N) for the clay till ranged from 25 to 59 with an average of 34 blows.

### 4.2.4 Lower Sand

A sand layer named “lower sand” to differentiate it from the distinct upper sand layer was encountered from approximately 13 mBGS to 19 mBGS. This sand contained some fines; traces of gravel, coal, and rust; was fine to medium grained, compact to dense, damp to moist, and light brown to light yellow in colour. It should be noted that the depth of this sand approximately corresponds with the typical depth of Saskatchewan Sand and Gravel deposits in this area. As such, this sand may be a part of Saskatchewan Sands and Gravel deposits.



The moisture content of the lower sand ranged from 2.2% to 18.1%, with an average of 9.9%. The SPT values (N) ranged from 28 to 93 with an average of 58 blows.

#### 4.2.5 Bedrock

Sandstone and clay shale bedrock was encountered at 18-20 mBGS at all three test-holes, at an elevation of approximately 646 m. The sandstone was very dense, damp to moist, and light grey. The clay shale was weathered, silty; contained some sand, traces of gravel, coal, and rust; was very hard, damp to moist, and dark brown in colour.

Moisture contents ranged from 11.1% to 19.9%, with an average of 16.0%. SPT values (N) ranged from 72 to 82, with an average of 80 blows.

### 4.3 Groundwater Conditions

No seepage was noted at any of the test-holes, and all were dry at completion. Groundwater levels were monitored again 48 days after drilling, on January 18, 2016. All test-holes were dry, signifying that groundwater is likely deeper than the piezometer installations in TH-01 and TH-03 at 15 mBGS. It should be noted that groundwater levels can vary in response to seasonal factors and precipitation. The actual groundwater conditions encountered during construction could vary from those recorded during the investigation. For this reason, the groundwater level should be checked prior to starting any excavation work for the proposed development.

### 4.4 Frost Penetration Depth

The effects of frost must be considered in the design of foundations. The maximum frost penetration at this site is estimated to be 2.5 mBGS in the surficial silty sand deposit, based on an expected 50 year return period and Annual Freezing Index (AFI) of 2,250°C-days. This estimated depth is applicable for the areas of the site where organic soils and snow have been removed or are not present on the surface.

## 5 SLOPE STABILITY ANALYSIS

The soil parameters and groundwater conditions for the slope stability analysis were defined based on investigation results and available data. Table 1 provides the effective soil parameters used in this analysis.

For the purposes of the analysis, it was assumed that no major movements had previously occurred in the area of the subject lots. The entrance/exit slip surface option was used to determine the slip surface with a factor of safety of at least 1.50. This was used to define the setback line.



**Table 1: Soil Parameters used in Slope Stability Analysis**

Soils	Unit Weight (kN/m <sup>3</sup> )	Internal Friction Angle (degrees)	Cohesion (kPa)
Upper Sand	19.0	33	0
Clay Till	19.5	26	0
Lower Sand	20	36	0
Bedrock	22	20	25

Based on the site investigation and review of available data, no signs of cracking, sinkholes or other indications of possible weak bedrock layers were evident on the site. The geometry and stratigraphy of the slopes, however, suggests that the steeper North and South slopes could experience local sloughing due to changes in weather conditions. Any such localised movements would not impact the overall slope stability.

The presence of retaining walls on both slopes should improve their resistance to local sloughing, but the depths of these retaining structures are not known. The depths of the retaining walls shown on the slope stability plots in Appendix D are not intended to be representative of their actual dimensions. Any effect on retained soil potentially provided by these structures was not incorporated into the slope stability analysis.

As no evidence of seepage was observed on any of the slope faces and groundwater was not observed in any of the drilled test-holes, the depth and shape of the groundwater table was estimated based on the profile of the slope. Groundwater is unlikely to cause adverse effects on the overall stability of any of the slopes at the subject site. For the purposes of the slope stability analysis, it was set at approximately 15 mBGS, the lowest detectable depth from the installed standpipes. Even at this conservative estimate of groundwater elevation, however, it does not interact with the governing slip surfaces.

The slope stability analysis was performed on four cross sections of the site area deemed representative of the three slopes discussed in this report. Sections of the North and South slopes were joined together to form one section called A-A', while the East Slope was sectioned as B-B'. An additional section, C-C', was analyzed to reflect the variable geometry of the North Slope. Figure 1.0 in Appendix A shows the location of the cross sections on the overall site plan, while the stratigraphic and surface profiles along the sections are presented on Figure 2.0 in Appendix A. The soil profile along section A-A', where it occurs beneath the existing house, was approximated as a smooth incline.

Slope stability analyses were performed on the existing site profiles assuming a house footprint load of 20 kPa. As the footprint or location of the new house is not available, an allowable building envelope was established, bounded by the setback distance from each of the three slopes. Note that if the footprint load or earth profile differs substantially from those used in the analysis, a new slope stability analysis may be required.



The slip surfaces selected for the establishment of the setback line from the top-of-bank for each of the three investigated slopes are shown on the plots in Appendix D. The factors of safety for each of these cases exceeds 1.5, and therefore satisfies the criteria for the Estimated Long Term Line of Stability, as defined in the City of Edmonton Top-of-Bank Policy C542. The governing factors of safety obtained with the slope stability analysis for each of the three slopes, as well as the corresponding setback distances, are shown in Table 2.

**Table 2: Slope Stability Analysis Results**

Slope	Governing Case	Governing Factor of Safety	Setback Line from TOB
North Slope (A-A')	1-1 – Failure Surface through Clay Till	1.504	12 m
South Slope (A-A')	2-2 – Failure Surface through Lower Sand	1.503	14 m
East Slope (B-B')	3-1 – Failure Surface through Clay Till	1.552	8 m
North Slope (C-C')	4-1 – Failure Surface though Clay Till	1.509	13 m

Based on this analysis, the setback line for foundation walls of the proposed development from the TOB perimeter was established for the existing site conditions and is shown on Figure 1.0 in Appendix A. The setback distance from the East Slope is superseded by the legal property limit, which is farther away from the TOB than the established setback line (it is approximately 12 m from the top of bank at its closest point). This is shown on the plan view in Figure 1.0 in Appendix A.

Plots 2 and 6 are attached only for reference and are not relevant to the establishment of the setback lines for their respective slopes.

## 6 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

The soil stratigraphy encountered on the site can be summarized as silty sand, overlying clay till, overlying sand, overlying sandstone and shale bedrock.

Groundwater levels were monitored on January 18, 2016. Groundwater was not observed in any of the installed standpipes to a depth of 15 mBGS. However, the groundwater level at the time of construction could vary from that recorded during the investigation. Supplemental monitoring of groundwater levels should be performed prior to excavation for the new foundation as a precaution, although no groundwater related issues are expected for construction at this site. If LWL is contracted to perform supplemental groundwater monitoring, additional costs will be incurred.



## 6.1 Slope Stability

The slope stability analysis was performed on four sections, covering each of the three prominent slopes adjacent to the subject site. The factors of safety used for the determination of the setback line along the North Slope were 1.504 and 1.509 based on two sections (A-A' and C-C'). The factors of safety used for the determination of the setback line along the South and East slopes were 1.503 and 1.552 based on sections A-A' and B-B', respectively.

Based on the slope stability analysis, the setback line for the rear foundation of any future primary dwelling or structure should be set as shown in Figure 1.0 in Appendix A. The setback distance along the North Slope varies between 12 and 13 m due to the variable geometry of the slope. The setback distance from the South Slope is 14 m, and from the East Slope it is 8 m. The actual setback distance from the East Slope is governed by the legal limit and will be greater than the stability analysis derived setback of 8 m.

Even though the minimum setback distances for the foundation walls of the proposed development should be sufficient under the existing site conditions, the client must be aware of the inherent risk of development at this site, and the possible consequences of building adjacent to the top of bank.

Also, although the existing slope conditions can be considered acceptable with respect to local and overall stability, further erosion and shallow sloughing may occur on the slopes, especially those to the north and south. Evidence of past sloughing was observed during site reconnaissance. Shallow failures of this type, however, would not impact the overall stability of the slopes or the position of the established setback line.

The TOB is well defined on the site plan, Figure 1.0 in Appendix A, so no additional survey is required unless the area is going to be re-graded or altered during construction of the proposed residence. If this is the case, then an additional slope stability analysis should also be carried out to reflect the new slope geometry.

The following measures for the slope and setback area have to be applied to reduce local sloughing and to maintain the integrity of the slope and safe conditions for the proposed development.

1. Grading of the development area should not block any existing drainage courses.
2. Grading of the site and construction activity should not allow any ponding of water or the concentrated discharge of water towards the slopes. Surface run-off should be directed away from the slopes and into the storm drainage system, where possible.
3. Fill soils should not be placed within the setback zone, near the crest of the slope or on the slope. If placement of fill is required within the backyard area, it should be approved by a geotechnical engineer.



4. Slope vegetation should not be disturbed to reduce surface erosion, local instability, and sloughing. Further vegetation of the setback zone and slope is encouraged and would reduce erosion in this area.
5. Underground sprinklers, in-ground storage systems, or similar structures should not be placed within the backyard area on the north, east, or southeast facing sides of the proposed development. Underground sewer or water utilities should not be installed within the setback area to avoid potential leakage into the surrounding soil.
6. No pools should be constructed or installed anywhere within the setback zone or the backyard area on the eastern part of the lots. This includes swimming pools, ornamental ponds, and other water retention structures.
7. A modestly sized hot tub or similar structure within the front yard of the property (southwest area of the lots) may be constructed provided it is equipped with a water proof liner and water tight connections to ensure no seepage into the ground. This amenity and the connections must be designed to provide for leakage detection monitoring.
8. Sump-pump, weeping tile, and any runoff discharges from the proposed building should be directed to the storm sewer system and must not be discharged down the slopes or within the setback area.
9. Roof leaders, downspouts and sump-pump discharge spouts should not be allowed to discharge onto ground surface. They should be connected to a storm drainage system where possible.
10. Only natural surface run-off from the backyard may be allowed to drain onto the slope.

Failure to comply with any of the above development and site maintenance measures could either hasten or increase the severity of any slope instability, which could ultimately threaten the proposed residence. The owner and resident should be aware that inappropriate development can have significant adverse effects on slope stability conditions.

It is recommended to provide design drawings of the new foundation structure, when they become available, to a geotechnical engineer for review and approval to ensure that all geotechnical conditions related to slope stability and setback distance are met.

Regardless of the established setback distance and all above measures being applied, the resident and the owner must be aware that the subject is surrounded on three sides by slopes with some impact on the property value due to risk of slope instability.

## 6.2 Site Preparation

All topsoil, organics and other deleterious material should be removed from beneath the proposed building areas. Topsoil was encountered in the upper 0.3 metres or so in LWL's field investigation, but depths could vary across the site. Engineered fill, where required below slabs on



grade, pavements and sidewalks, should be non-expansive, medium plastic inorganic clay compacted to 98% of Standard Proctor Maximum Dry Density (SPMDD) at  $\pm 2\%$  of optimum moisture content. The site's surficial silty sand deposit can be used for engineered fill construction. Alternatively, the engineered fill can comprise granular material compacted to the same level as clay fill.

Native clay soils at the grade level underneath the house area should be scarified to 150mm and compacted to 98% of SPMDD at  $\pm 2\%$  of optimum moisture content. The final grade of landscaped areas should be compacted to no less than 92% of the SPMDD at  $\pm 2\%$  of optimum moisture content.

As local areas of exposed soil may be sensitive to excessive moisture contents, prolonged exposure to wet weather might cause soft soil conditions, workability and trafficability problems in these areas.

Additionally, a review of the suitability of the existing retaining walls should be completed by a geotechnical engineer prior to deciding whether they should be left in place or removed.

## 6.3 Foundations

Based on building loads and local soil and groundwater conditions, the following foundations types are considered suitable for the proposed development:

- Strip and spread footings (shallow foundations) if a basement structure is included;
- Cast-in-place concrete piles (deep foundations) if no basement is included.

### 6.3.1 Strip and Spread Footings

Shallow foundations should be founded on the native, undisturbed, non-organic upper sand layer. Some exceptions to this may be considered where incompetent material would be subcut and replaced with engineered fill. Footings placed within the surficial upper sand deposit to a minimum depth of 2.5 m below finished grade may be designed on a factored bearing capacity of 160 kPa and 200 kPa for strip and spread footings, respectively. Minimum footing widths of 400 mm for strip footings and 600 mm for spread footings are recommended.

The minimum depth of 2.5 m below the finished grade is recommended for shallow footings to avoid the effects of frost heave. The shallow subsurface soil present across the site is silty sand, which could be susceptible to volume changes due to frost penetration. If footings shallower than 2.5 m below finished grade are required, frost protection must be provided.

The base of the excavations for the footings must be thoroughly cleaned of all loose or softened soil prior to pouring the concrete. Any loose soil encountered at the footing elevations should be over-excavated to expose the underlying more competent soil and replaced with compacted





engineered fill. Footing excavations should be protected from rain, snow, wetting, drying, and inflow of surface water at all times.

When construction is to occur during the winter or spring months, footings should not be cast on frozen soil. Footings founded on soil frozen during the construction period may settle when the foundation soils are weakened by thawing. It is also essential that foundation soils do not freeze after the concrete for the footings has been poured. Frost should not be allowed to penetrate beneath the footings prior to, during, or following construction.

The base of the excavation for the footings should be inspected by a geotechnical engineer to verify soil conditions and allowable bearing capacity.

### 6.3.2 Cast-in-Place Concrete Piles

Cast-in-place concrete piles embedded to a minimum depth of 6 m below finished grade are also considered suitable for the proposed development, provided that there is adequate resistance to uplifting frost heave forces for piles of this length. Cast-in-place piles should be designed using the bearing capacity values provided in Table 3.

**Table 3: End Bearing and Skin Friction Values for Cast-in-Place Piles**

Depth Below Grade (m)	End Bearing (kPa)		Skin Friction (kPa)	
	Ultimate	Factored	Ultimate	Factored
0 – 1.5	N/A	N/A	N/A	N/A
1.5 – 6.0	N/A	N/A	38	15
≥ 6.0	1000	400	50	20

Skin friction should not be included in the upper 1.5 m of the pile below grade level due to the possibility of soil drying and shrinking away from the pile shaft.

Factored bearing capacities and skin frictions were determined using the recommended geotechnical resistance factor for deep foundations using limit state design, 0.4, as per NBCC (2005), as referenced in Table 8.1 in the Canadian Foundation Engineering Manual, 4<sup>th</sup> edition (2006).

For end-bearing piles, additional pile capacity due to skin friction can be added to end-bearing capacity to calculate the total pile bearing capacity. All piles should be inspected by a geotechnical engineer to document each pile installation and to confirm soil conditions and bearing capacity values.

Piles should be reinforced along their full length. Concrete for piles should be poured immediately after drilling and inspection of the pile hole to reduce the amount of soil sloughing of any sand lenses or layers. Casings should be available on the site during pile installation to prevent



sloughing as required. Groundwater seepage is not expected and is not likely to cause issues during pile casting up to 15 mBGS.

Although cobbles and boulders were not found in the clay till during the drilling program, they may be encountered during drilling for cast in place pile installation.

Where cast-in-place concrete piles are to be used, grade beams are generally required to transfer wall loads to the top of the piles. To prevent heaving of the grade beams due to frost penetration of the underlying soil, a void form with a minimum thickness of 100 mm should be placed under the grade beams where the grade beams will lie less than 2.5 m below the final grade.

Vertical piles may resist uplift due to frost heave by developing adfreeze bond stress along the pile shaft. An adfreeze bond stress of 100 kPa can be assumed for the surficial silty sand deposits to calculate uplift loads due to frost heave. The frost heave is resisted by the shaft friction along the unfrozen portion of the pile combined with the weight of the pile, plus dead load on the pile.

## 6.4 Basement

If a basement is to be included in the proposed development, basement walls should be designed to resist lateral earth pressures in an “at-rest” condition, assuming a triangular pressure distribution with no surcharge on the walls. Lateral pressures may be calculated using Equation 1:

$$P_o = K_o (\gamma H) \quad (1)$$

Where:

- $P_o$  = lateral “at-rest” earth pressure at depth (H), (kN/m<sup>2</sup>)  
 $K_o$  = co-efficient of “at-rest” earth pressure (use 0.5 for clay backfill and 0.45 for sand and gravel backfill)  
 $\gamma$  = bulk unit weight of backfill soil (use 19 kN/m<sup>3</sup> for clay or 21 kN/m<sup>3</sup> for granular backfill)  
H = depth below final grade (m)

Weeping tiles should be installed at (or slightly below) the base of the basement wall footings or grade beams if shallow footings or piles are used, respectively. The weeping tile should consist of perforated pipe wrapped in a geotextile fabric and surrounded by a 150 mm thick layer of free-draining granular material. Alternatively, the weeping tile pipe may be surrounded by coarse drainage rock wrapped in geotextile. The weeping tile should drain to a sump-pump system and discharge into the storm water system or other system available on the site.

A vertical drainage layer should be placed against the basement walls and connected to the weeping tile system. This would assist in reducing any pore water pressures generated against the walls by groundwater and/or infiltrating surface water. The drainage layer could consist of either free draining gravel or a proofed vertical sheet drain.



If a weeping tile system is not installed, the basement walls may be subjected to hydrostatic pressure. The groundwater table should be assumed to be at the ground surface for design purposes. The hydrostatic pressure may be calculated using Equation 2:

$$P_w = \gamma_w H_w \quad (2)$$

Where:

$P_w$	=	hydrostatic pressure (kPa)
$\gamma_w$	=	unit weight of water (9.8 kN/m <sup>3</sup> )
$H_w$	=	depth below the groundwater table (m)

Backfilling around concrete foundation walls should not commence before the concrete has reached a minimum two-thirds of its 28-day strength or the walls should be laterally braced. Only hand operated compaction equipment should be used within 600 mm of the concrete walls. Caution should be used when compacting backfill to avoid high lateral loads caused by excessive compaction. The backfill should be brought up evenly around the walls. A minimum 600 mm thick clay cap should be placed at the top of the backfill to the ground surface to minimize the infiltration of surface water.

## 6.5 Slab-On-Grade

In general, preparation for slab-on-grade areas should be carried out according to the site preparation procedures specified previously.

Natural and subcut ground should not be exposed to prolonged drying or wetting conditions prior to floor slab placement to reduce wet soil conditions or the possibility of shrinking and cracking of sandy layers after the slab-on-grade is completed.

All subgrade support areas should be inspected by a qualified geotechnical engineer prior to slab-on-grade construction to detect any soft or weak areas. Soft or weak areas should be subcut and replaced with engineered, compacted fill.

The slab-on-grade should rest on at least 100 mm of well-graded, free draining granular fill. This granular material should be compacted to 98% of Standard Proctor Maximum Dry Density (SPMDD).

The slab-on-grade should be constructed independently of all walls, columns, and grade beams and should be reinforced. The reinforcement can extend through construction joints. Construction joints are necessary to prevent shrinkage cracks as the concrete hydrates.

Heating ducts beneath slab-on-grade are not recommended as the hot air may cause drying and shrinking of the soil. If possible, water lines should not be placed beneath slab-on-grade floors.



## 6.6 Settlement Considerations

The magnitude of post-construction settlement is difficult to assess; however, it is anticipated that settlements of the proposed development will be in the acceptable limits for such a building.

An estimate of settlement values based on the existing soil strength is as follows:

- For a spread and strip footing, the value should not be more than 1% of the footing width.
- For a single cast-in-place concrete pile, the value should not be more than 10 mm.

Minor settlement of compacted fill material and subgrade soils beneath fill areas will likely occur. Maximum magnitude settlements will occur in the areas of thickest fill and/or most compressible subgrade soils. It is anticipated that these settlements will be within 1% of the fill thickness, which is acceptable for the nature of the proposed development.

## 6.7 Excavation and Backfill

### 6.7.1 Excavation

All excavations should be executed in accordance with all applicable Alberta Occupational Health and Safety Regulations. It is expected that excavations will be required for shallow foundations, basements, grade beams and underground utility installations.

In general, the groundwater table throughout most of the site is sufficiently far below the ground surface such that it should not interfere with excavations. If any seepage occurs during excavation, a temporary sump-pump system will be sufficient to keep water away from the trench bottom.

Excavation of side slopes to a depth of 1.5 m should be cut back to a minimum of 1H:1V (short term) in the silty sand deposits. Where local conditions are soft or wet, excavations should be cut back to slopes of 1.5H:1V or flatter.

Temporary surcharge loads, such as construction materials and equipment, should not be allowed within 1.5 m of an unsupported excavated face. When an excavation is greater than 1.5 m depth, the setback distance should be the same as the excavation depth. Vehicles delivering materials should be kept back from the edge of excavations by at least 1.0 m. All excavations should be checked regularly for signs of sloughing, especially after periods of rain. Sloughing of the side-slopes is a potential source of danger to workers and caution must be taken to avoid this potential risk.

### 6.7.2 Backfill

Generally, backfilling inspection and testing should be carried out as per the specifications of the City of Edmonton. However, the following procedures can be used as guidelines.



Where backfill comprises native material, fill should be placed in lifts not exceeding 150 mm and compacted to 98% of Standard Proctor Maximum Dry Density (SPMDD) at  $\pm 2\%$  of optimum moisture content. Where backfill comprises granular material, fill should be placed in lifts not exceeding 300mm and compacted to 98% of Standard Proctor Maximum Dry Density (SPMDD) at  $\pm 2\%$  of optimum moisture content.

### 6.7.3 Utility Installation

Trenches or excavations greater than 1.5 m will likely require bracing or shoring. Bracing or shoring of trenches is recommended wherever seepage is encountered.

The soil conditions found across most of the site will likely provide adequate support to utility pipes. Where soil strengths are inadequate or soft material is encountered, over-excavation of the pipe trench and replacement with compacted engineered fill may be required.

Trench backfill should consist of selected native material, compacted to a minimum 97% SPMDD at optimum moisture content, with the compacted thickness of each lift not exceeding 300 mm. Additionally, the upper 1.5 m of service trenches below paved areas or floor slabs should be cut at a maximum slope of 1H:1V to avoid an abrupt transition between backfill and in-situ soil. Trench backfill should be free from frozen, organic and other deleterious material.

## 6.8 Drainage

As the silty sand encountered across the site is susceptible to erosion from concentrated surface water flows, water from roof leaders, pavement and other collection structures should be collected and directed to a storm sewer system or an adequate discharge point at least 1.8 m away from the building walls.

A clay cap should be placed over any backfill adjacent to buildings to reduce water infiltration into building foundations and basements. Surface grades within 2.0 m of building areas should slope away from the building edges at a minimum of 10% grade to direct water away from the building foundations. Gravel surfaces should have a minimum grade of 2% to promote runoff.

## 6.9 Frost Protection

Where utility pipelines lie below roads or sidewalks, a minimum cohesive soil cover of 1.8 m is recommended for frost protection. A greater depth of cover (2.4 m) is required where backfill is comprised of granular soil. Insulation should be provided to protect the pipe from frost when the recommended soil cover is not feasible. Frost protection is also required for any shallow foundations placed less than 2.5 m below finished grade.

## 6.10 Concrete Type

Soluble sulphate tests conducted at various depths showed concentrations of dissolved sulphates ranging from negligible to severe. Therefore, sulphate resistant Portland cement (CSA Type 50/HS) is recommended for all concrete work in contact with the native soil at this site.



## 7 LIMITATIONS

The findings and recommendations contained in this report were based on the subsurface investigation at the subject site. Since the possibility of variation of soil conditions exists, this office should be contacted if, at any stage of development, soil or groundwater conditions are encountered which appear to be different from those described or assumed for analysis in this report.

This report has been prepared by LWL Engineering Ltd. for the benefit of Birkholz Homes Inc. Except as where required by law, this report and the information and data contained herein are to be treated as confidential and may be used and relied upon only by Birkholz Homes Inc. LWL Engineering denies any liability whatsoever to other parties who may obtain access to this report for any injury, loss or damages suffered by such parties arising from their use of, or reliance upon, this report or any of its contents without the express written consent of LWL Engineering and Birkholz Homes Inc.

This report has been prepared in accordance with generally accepted professional engineering principles and practices. No other warranty, expressed or implied, is given.

Respectfully submitted by,

**LWL Engineering Ltd.**

Mikhail Petrov, B.Sc., E.I.T.  
Geotechnical Engineer

Reviewed by,

Ljubo Bijeljanin, MSc., P. Eng.  
Senior Geotechnical Engineer

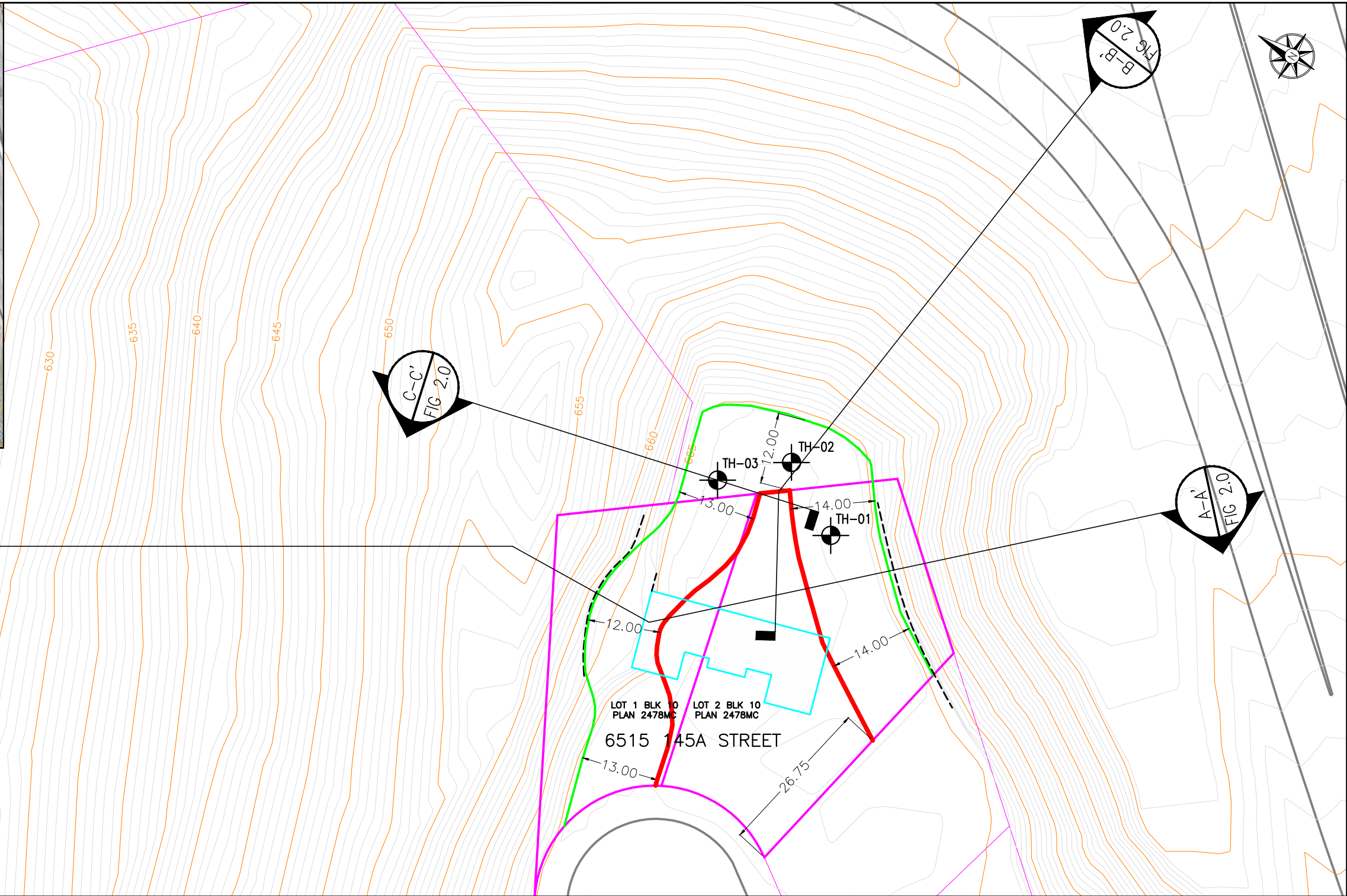
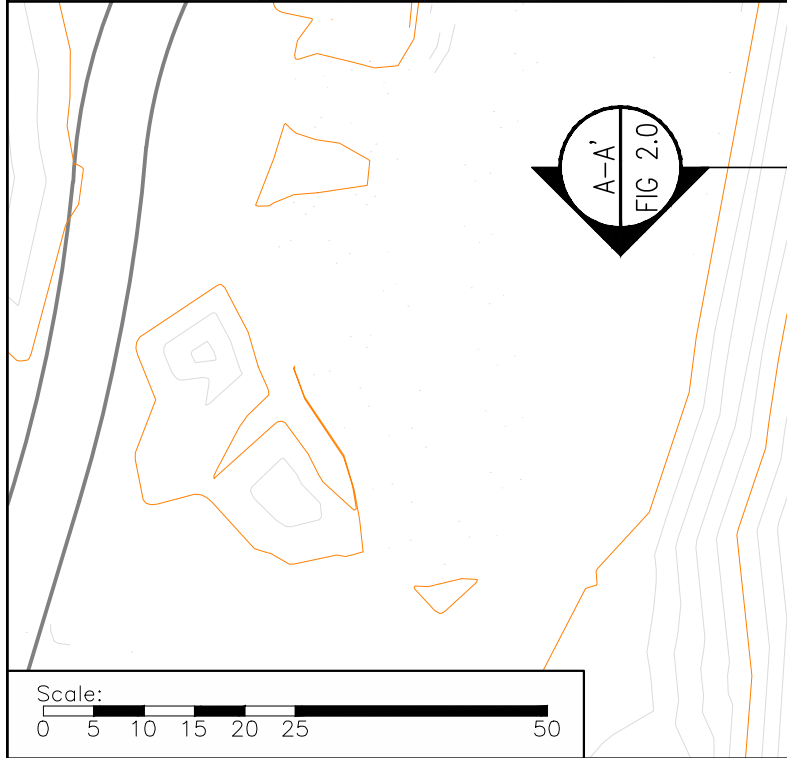
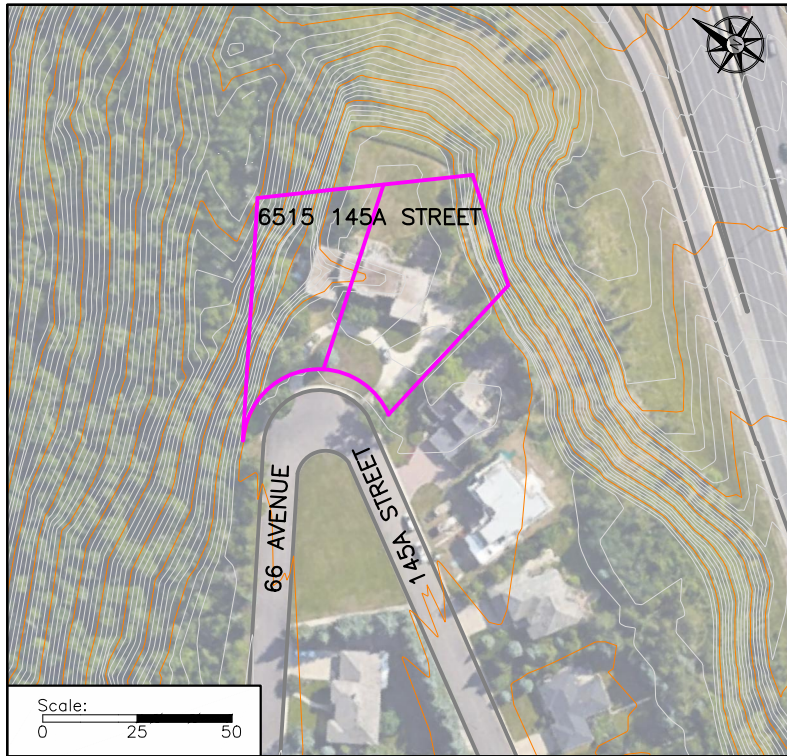
APEGA Permit to Practice No.: P10323



## Appendix A

Figure 1.0: Site Location Plan

Figure 2.0: Soil Profile Sections A-A', B-B', C-C'



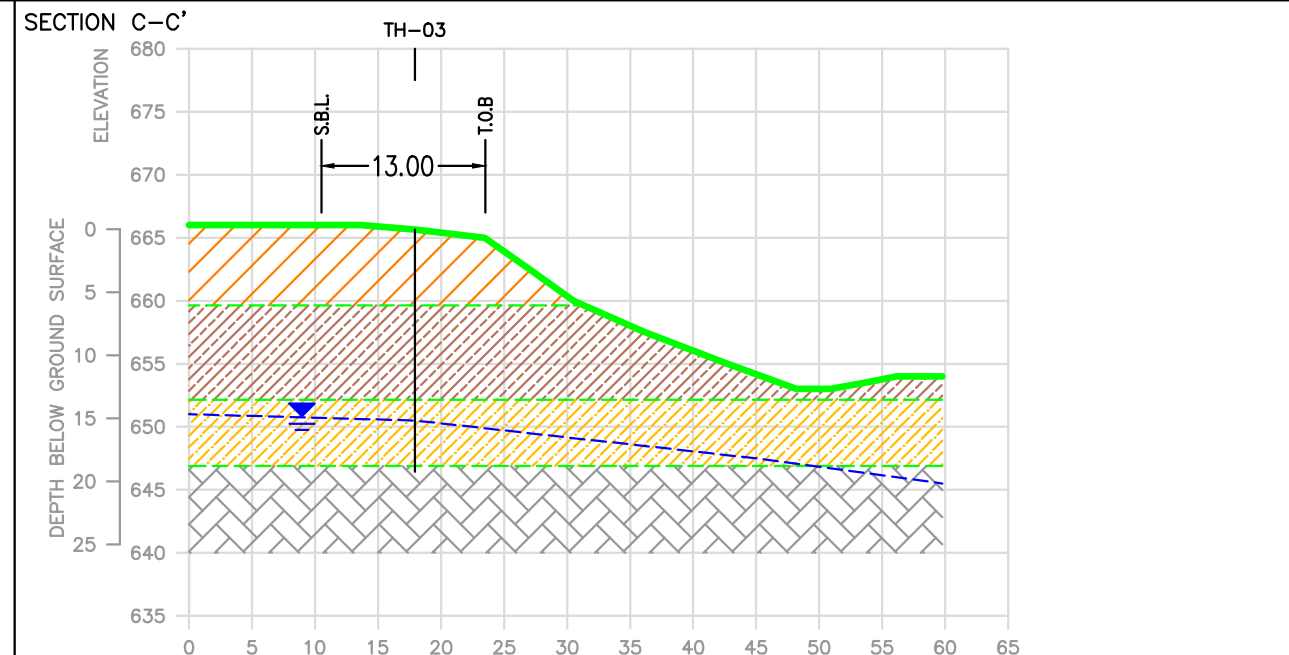
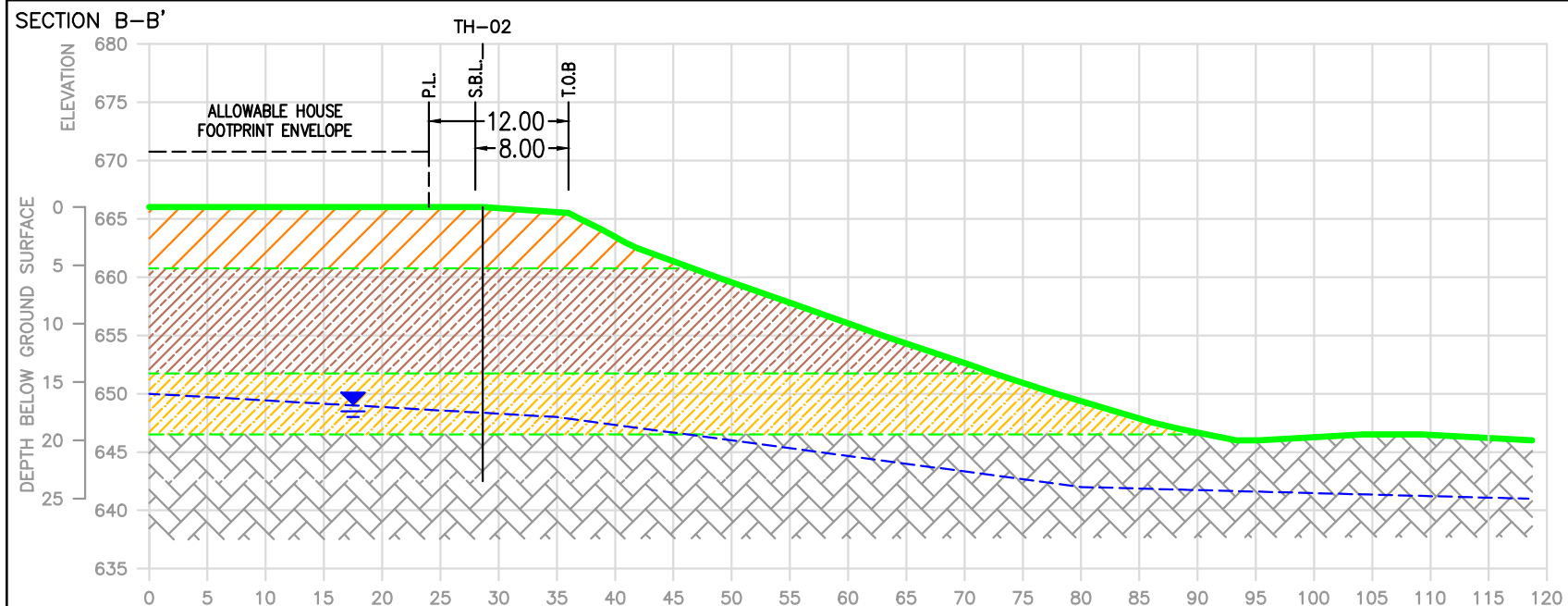
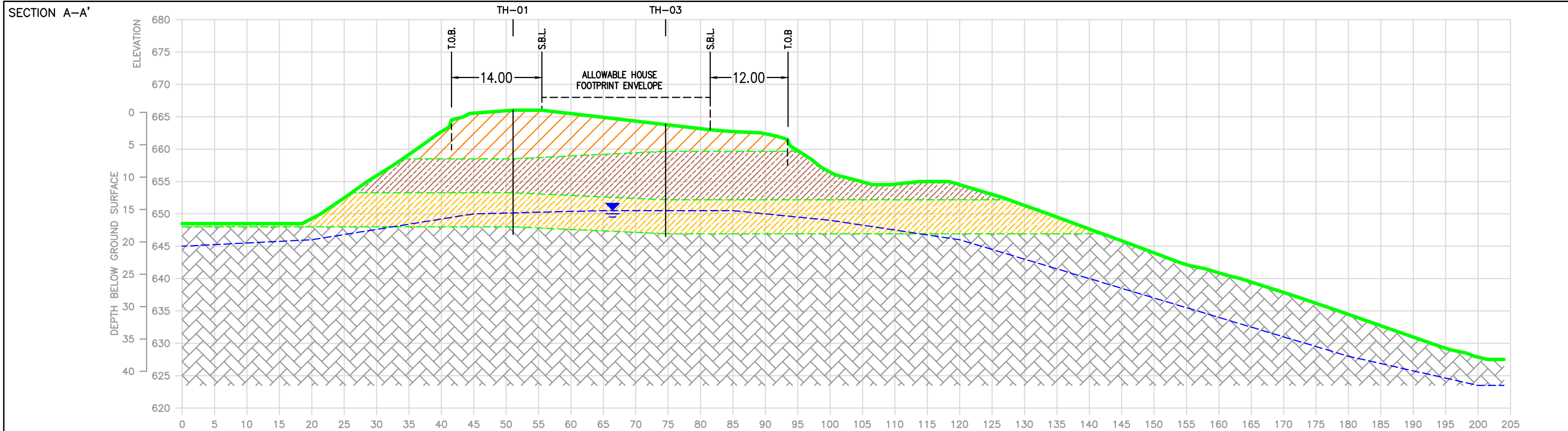
LEGEND:	
	TESTHOLE LOCATION
	PROPERTY LINE (PL)
	ROAD
	ELEVATION CONTOURS (0.5 m)
	ELEVATION CONTOURS (2.5 m)
	TOP-OF-BANK
	SETBACK LINE
	RETAINING WALL
	EXISTING HOUSE FOOTPRINT

**NOTE:**  
 UNITS ARE IN METERS UNLESS OTHERWISE SPECIFIED.  
 TEST-HOLE, TOP-OF-BANK, RETAINING WALL, AND EXISTING HOUSE FOOTPRINT LOCATIONS ARE APPROXIMATE.  
 ELEVATION CONTOURS OBTAINED FROM THE CITY OF EDMONTON.  
 SATELLITE IMAGE FROM GOOGLE MAPS.

APPROVALS	DATE
DRAWN MP	2016/2/25
ENGINEER MP	2016/2/25
APPROVED LB	2016/2/25

<b>LWL ENGINEERING LTD.</b> Geotechnical Engineering Consultant			
		TITLE GEOTECHNICAL INVESTIGATION - 6515 145A Street NW SITE PLAN	
CLIENT BIRKHOLZ HOMES INC.		SCALE AS SHOWN	
PROJECT NO. A-0277		DRAWING NO. FIGURE 1.0	





**LEGEND:**

	EXISTING GROUND SURFACE		UPPER SAND		BEDROCK
	INFERRED SOIL BOUNDARY		CLAY TILL	T.O.B.	TOP-OF-BANK LINE
	ASSUMED GROUNDWATER LEVEL		LOWER SAND	S.B.L.	SETBACK LINE
				P.L.	PROPERTY LINE

**NOTE:**  
 UNITS ARE IN METERS UNLESS OTHERWISE SPECIFIED.  
 TEST-HOLE AND TOP-OF-BANK LOCATIONS ARE APPROXIMATE.  
 ELEVATIONS OBTAINED FROM CITY OF EDMONTON CONTOUR MAP.  
 GROUNDWATER PROFILE ASSUMED FOR SLOPE STABILITY ANALYSIS ONLY.

APPROVALS		DATE
DRAWN	MP	2016/2/25
ENGINEER	M.P	2016/2/25
APPROVED	LB	2016/2/25

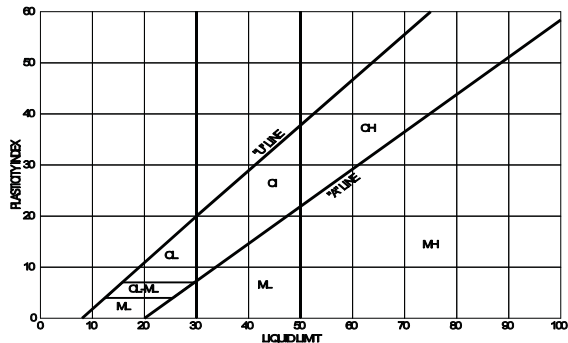
		<b>LWL ENGINEERING LTD.</b> Geotechnical Engineering Consultant
TITLE GEO TECHNICAL INVESTIGATION – 6515 145A Street SOIL PROFILE SECTION A-A'		
CLIENT BIRKHOLZ HOMES INC.		SCALE AS SHOWN
PROJECT NO. A-0277		DRAWING NO. FIGURE 2.0



## Appendix B

1. Modified Unified Classification System for Soil
2. Test-hole Logs

MAJOR DIVISION		LOG SYMBOLS	MUCS	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA			
COARSE GRAINED	GRAVELS (MORE THAN HALF COARSE GRAINS LARGER THAN 4.75 mm)	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL GRADED GRAVELS, LITTLE OR NO FINES	$C_u \cdot \frac{D_{60}}{D_{10}} > 4$ $C_c \cdot \frac{(D_{30})^2}{D_{10} \cdot D_{60}} \cdot 1 \text{ to } 3$			
			GP	POORLY GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS			
		DIRTY GRAVELS (WITH SOME FINES)	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE $W_p$ LESS THAN 4		
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE $W_p$ MORE THAN 7		
	SANDS (MORE THAN HALF COARSE GRAINS SMALLER THAN 4.75 mm)	CLEAN SANDS (LITTLE OR NO FINES)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u \cdot \frac{D_{60}}{D_{10}} > 6$ $C_c \cdot \frac{(D_{30})^2}{D_{10} \cdot D_{60}} \cdot 1 \text{ to } 3$			
			SP	POORLY GRADED SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS			
		DIRTY SANDS (WITH SOME FINES)	SM	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE $W_p$ LESS THAN 4		
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE $W_p$ MORE THAN 7		
		FINE GRAINED SOILS	SILTS (BELOW 'A' LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 50$	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)	
$W_L > 50$	MH			INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS				
CLAYS (ABOVE 'A' LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 30$		CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS				
	$30 < W_L < 50$		CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS				
	$W_L > 50$		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS				
ORGANIC SILTS & CLAYS (BELOW 'A' LINE)	$W_L < 50$		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINE CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER 'F'. E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY			
	$W_L > 50$		OH	ORGANIC CLAYS OF HIGH PLASTICITY				
HIGHLY ORGANIC SOILS				Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE		
BEDROCK			BR	SEE REPORT DESCRIPTION				



NOTE:  
1. BOUNDARY CLASSIFICATION POSSESSING CHARACTERISTICS OF TWO GROUPS ARE GIVEN GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL MIXTURE WITH CLAY BINDER BETWEEN 5% AND 12%

#### SOIL COMPONENTS

FRACTION	SIEVE SIZE (mm)		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
	PASSING	RETAINED	PERCENT	IDENTIFIER
GRAVEL	COARSE	75	50 - 35	AND
	FINE	19		
SAND	COARSE	4.75	35 - 20	Y
	MEDIUM	2.00		
	FINE	0.425		
SILT (non plastic) or CLAY (plastic)	0.080		20 - 10	SOME
			10 - 1	TRACE

#### OVERSIZE MATERIALS

ROUNDED OR SUBROUNDED COBBLES 75 mm to 200 mm BOULDERS > 200 mm	ANGULAR ROCK FRAGMENTS > 75 mm ROCKS > 0.75 m <sup>3</sup> IN VOLUME
---	--

## MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

JUNE, 1995



## 1.0 Explanation of Field and Laboratory Test Data

The field and laboratory test results, as shown on the logs, are briefly described below.

### 1.1 NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS

The relationship between the natural moisture content and depth is significant in determining the subsurface moisture conditions. The Atterberg Limits for a sample should be compared to the natural moisture content and should be on the Plasticity Chart in order to determine their classification.

### 1.2 SOIL PROFILE AND DESCRIPTION

Each soil strata is classified and described noting any special conditions. The Modified Unified Soils Classification System (MUSCS) is used. The soil profile refers to the existing ground level. When available, the existing ground elevation is shown. The soil symbols used are shown in detail on the soil classification chart.

### 1.3 TESTS ON SOIL SAMPLES

Laboratory and field tests on the logs are identified by the following:

**N** (Standard Penetration Test (SPT) Blow Count) - The SPT is conducted in the field to assess the in situ consistency of cohesive soils and the relative density of non-cohesive soils. The N value recorded is the number of blows from a 63.5 kg hammer dropped 760 mm which is required to drive a 51 mm split spoon sampler 300 mm into the soil.

**SO<sub>4</sub>** (Water Soluble Sulphate Content) - Conducted primarily to determine requirements for the use of sulphate resistant cement. Further details on the water soluble sulphate content are given in Section 1.6.

**$\gamma_D$**  (Dry Unit Weight)  $\text{kN/m}^3$  and  **$\gamma_T$**  (Total Unit Weight)  $\text{kN/m}^3$ .

**Q<sub>U</sub>** (Unconfined Compressive Strength) kPa - May be used in determining allowable bearing capacity of the soil.

**C<sub>U</sub>** (Undrained Shear Strength) kPa - This value is determined by an unconfined compression test and may also be used in determining the allowable bearing capacity of the soil.

**C<sub>PEN</sub>** (Pocket Penetrometer Reading) kPa - Estimate of the undrained shear strength as determined by a pocket penetrometer.

The following tests may also be performed on selected soil samples and the results are given on the borehole logs: Grain Size Analysis; Standard or Modified Proctor Compaction Test; California Bearing Ratio; Unconfined Compression Test; Permeability Test; Consolidation Test; Triaxial Test

#### 1.4 SOIL DENSITY AND CONSISTENCY

Table 1.1 Cohesive Soils		
N	Consistency	C <sub>u</sub> (kPa) (approx.)
0 - 1	Very Soft	<10
1 - 4	Soft	10 - 25
4 - 8	Firm	25 - 50
8 - 15	Stiff	50 - 100
15 - 30	Very Stiff	100 - 200
30 - 60	Hard	200 - 300
>60	Very Hard	>300

The SPT test described above may be used to estimate the consistency of cohesive soils and the density of cohesionless soils. These approximate relationships are summarized in the following tables:

Table 1.2 Cohesionless Soils	
N	Density
0 - 5	Very Loose
5 - 10	Loose
10 - 30	Compact
30 - 50	Dense
>50	Very Dense

#### 1.5 SAMPLE CONDITION AND TYPE

The depth, type, and condition of samples are indicated on the borehole logs by the following symbols:

■	Grab Sample	▨	A-Casing
◻	Shelby Tube	▤	No Recovery
⊗	SPT Sample	▥	Core Sample



## 1.6 WATER SOLUBLE SULPHATE CONCENTRATION

The following table from CSA Standard A23.1-94 indicates the requirements for concrete subjected to sulphate attack based upon the percentage of water soluble sulphate as presented on the borehole logs. CSA Standard A23.1-94 should be read in conjunction with the table.

Class of Exposure	Degree of Exposure	Water-Soluble Sulphate (SO <sub>4</sub> ) in Soil Sample %	Sulphate (SO <sub>4</sub> ) in Groundwater Samples mg/L	Minimum Specified 28 d Compressive Strength MPa†	Maximum Water/Cementing Materials Ratio†	Portland Cement to be Used‡
S-1	Very severe	over 2.0	over 10,000	35	0.40	50
S-2	Severe	0.20 - 2.0	1,500 - 10,000	32	0.45	50
S-3	Moderate	0.10 - 0.20	150 - 1,500	30	0.50	20§, 40, or 50

\* For sea water exposure see Clause 15.4

† See Clause 15.1.4

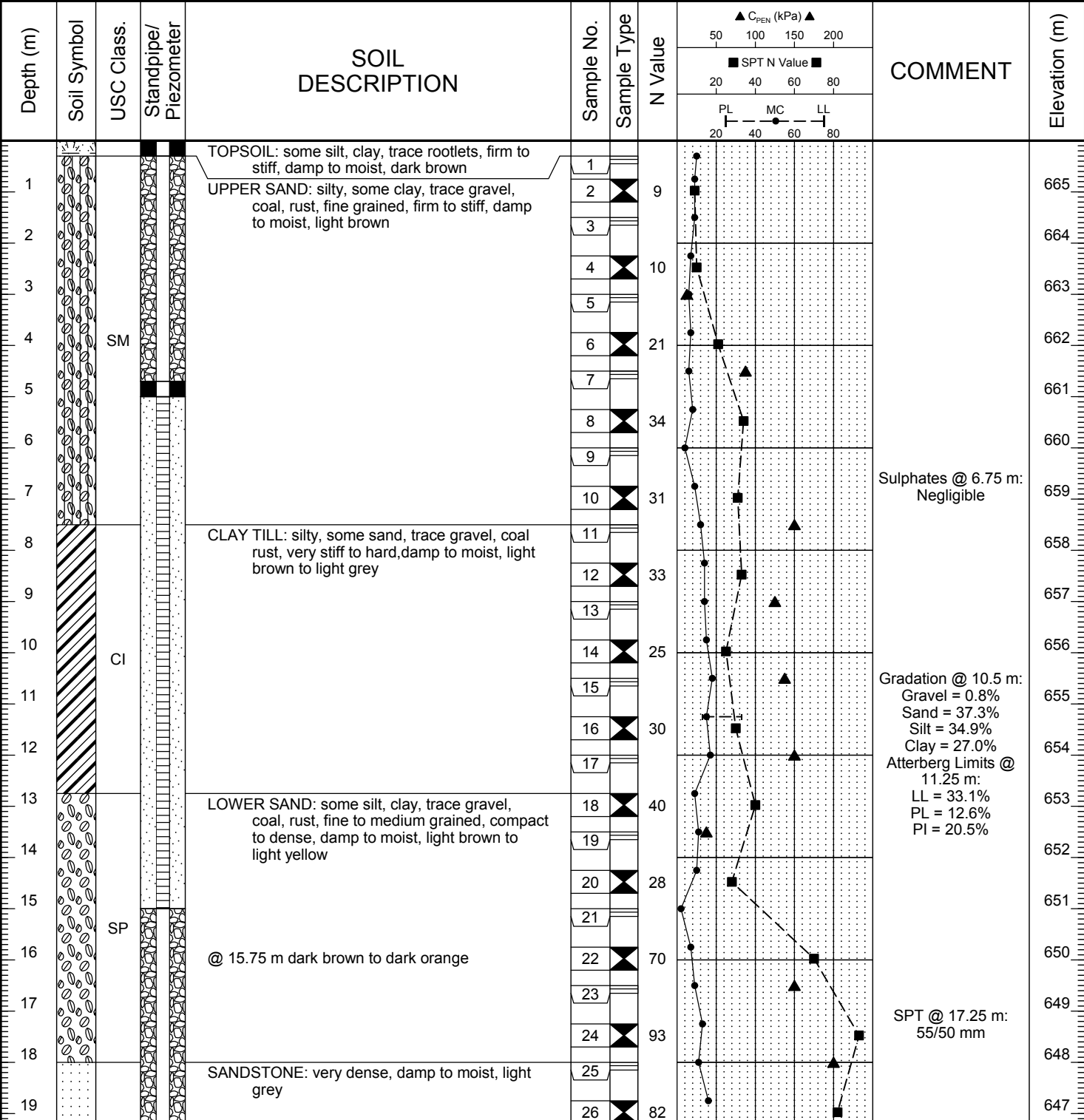
‡ See Clause 15.1.5

§ Type 20 cement with moderate sulphate resistance (see Clause 3.1.2).

## 1.7 GROUNDWATER TABLE

The groundwater table is indicated by the equilibrium level of standing water in a standpipe installed in a borehole. This level is generally taken at least 24 hours after installation of the standpipe. The groundwater level is subject to seasonal variations and its highest level usually occurs in spring. The symbol on the borehole logs indicating the groundwater level is an inverted solid triangle (▼).

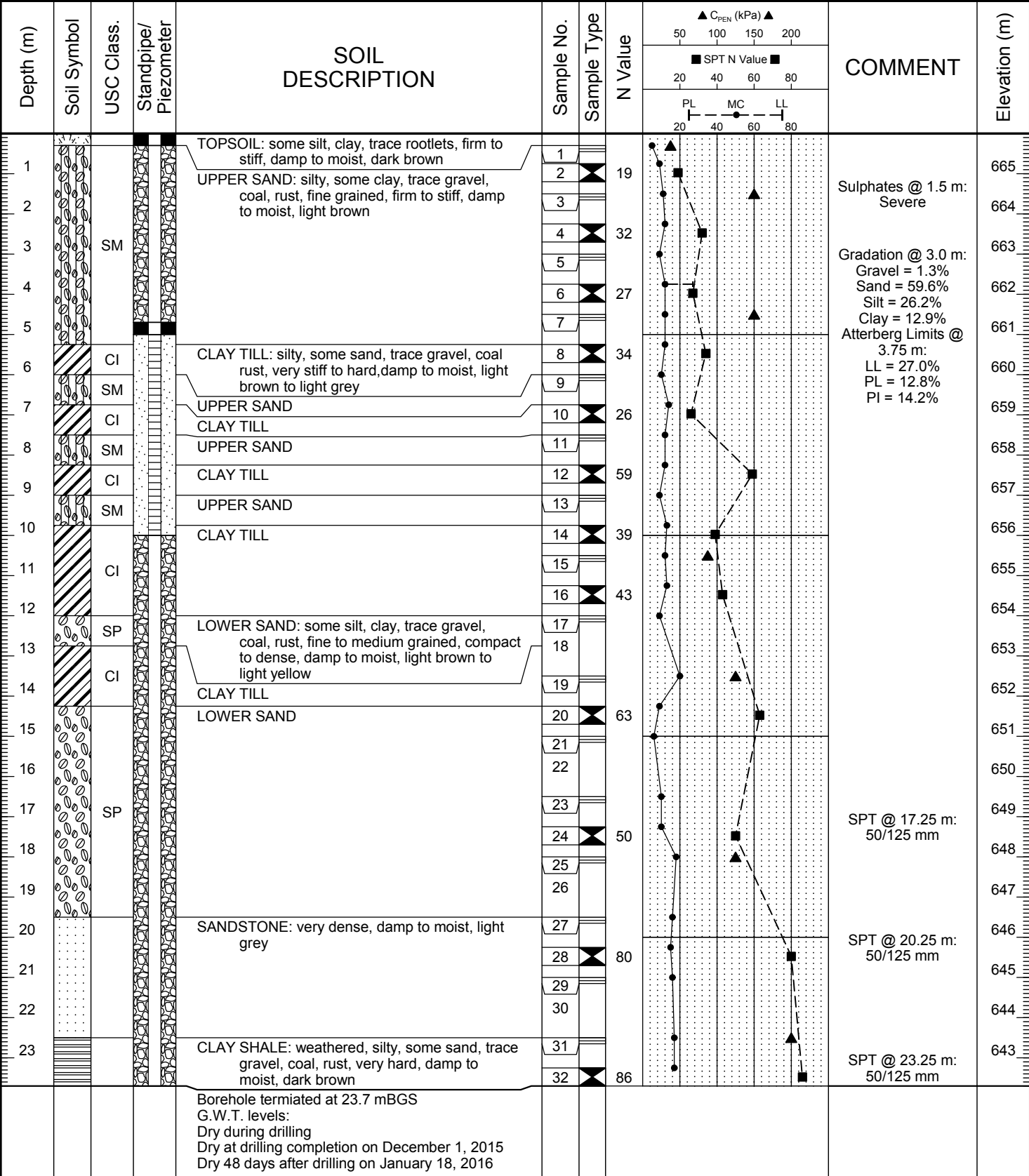
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<b>Client:</b> Birkholz Homes Inc.	<b>Location:</b> 6515 145A Street NW	
<b>Contractor:</b> Canadian Geological	<b>Method:</b> Solid Stem	<b>Elevation:</b> 666 m
<b>Sample Type:</b> <input checked="" type="checkbox"/> Shelby Tube <input type="checkbox"/> Core <input checked="" type="checkbox"/> SPT <input type="checkbox"/> Grab <input type="checkbox"/> No Recovery		



Borehole terminated at 19.2 mBGS  
 G.W.T. levels:  
 Dry during drilling  
 Dry at drilling completion on December 1, 2015  
 Dry 48 days after drilling on January 18, 2016

Logged By: Rasie, M.E.	Completion Date: 15/12/1
Reviewed By: Bijeljanin, L.	Completion Depth: 19.2m
Project Eng: Rasie, M.E.	Page 1 of 1

<b>Project:</b> Geotechnical Investigation	<b>Project No.:</b> A-0277	<b>TestHole No.</b> TH-02
<b>Client:</b> Birkholz Homes Inc.	<b>Location:</b> 6515 145A Street NW	
<b>Contractor:</b> Canadian Geological	<b>Method:</b> Solid Stem	<b>Elevation:</b> 666 m
<b>Sample Type:</b> <input checked="" type="checkbox"/> Shelby Tube <input type="checkbox"/> Core <input checked="" type="checkbox"/> SPT <input type="checkbox"/> Grab <input type="checkbox"/> No Recovery		



LWL BORING LOG 2 A-0277.GPJ LWL TEMPLATE.GDT 16/2/25



**LWL ENGINEERING LTD.**

Logged By: Rasie, M.E.	Completion Date: 15/12/1
Reviewed By: Bijeljanin, L.	Completion Depth: 23.7m
Project Eng: Rasie, M.E.	Page 1 of 1



**Project:** Geotechnical Investigation

**Project No.:** A-0277

**TestHole No.** TH-03

**Client:** Birkholz Homes Inc.

**Location:** 6515 145A Street NW

**Contractor:** Canadian Geological

**Method:** Solid Stem

**Elevation:** 666 m

**Sample Type:**

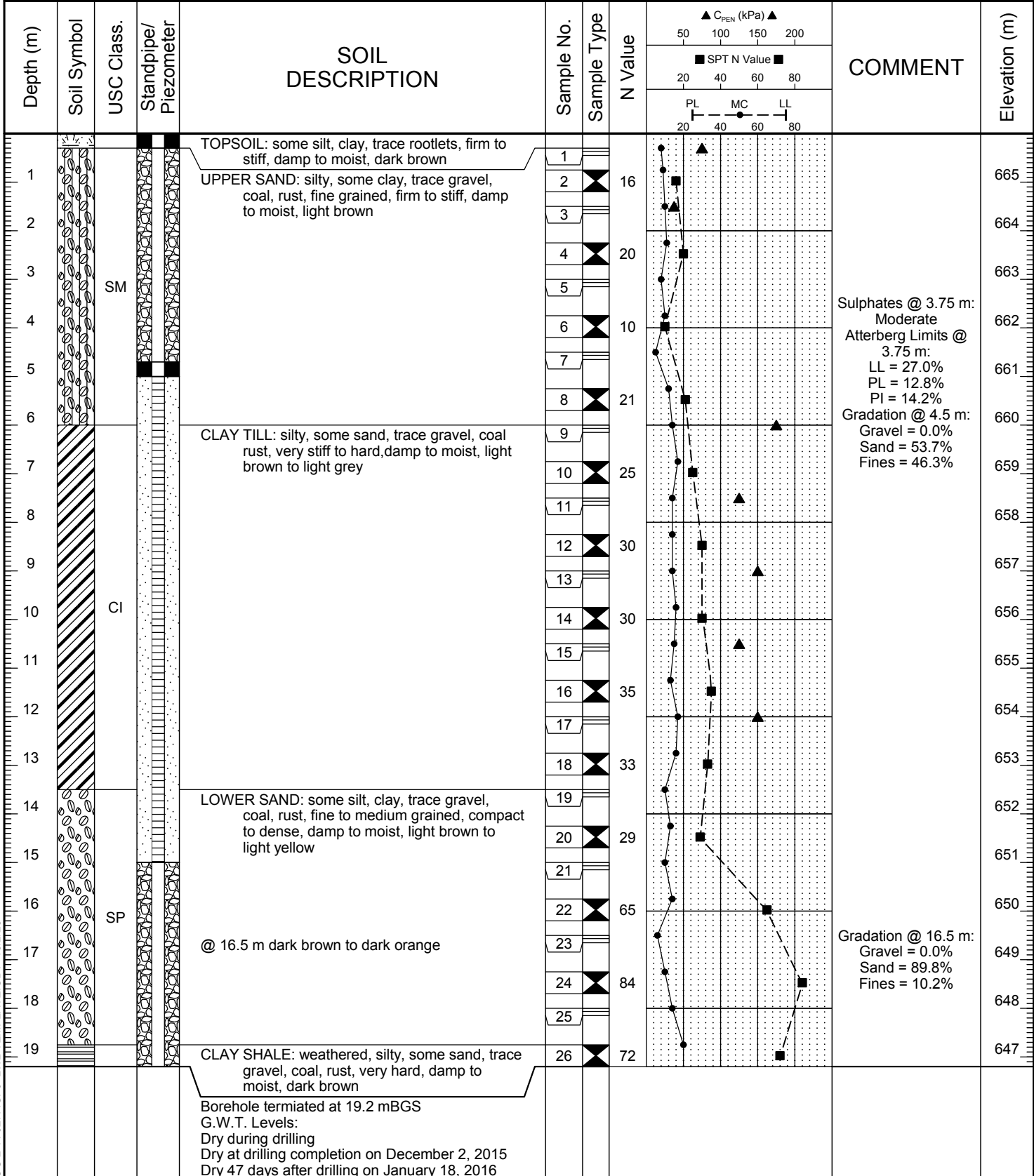
■ Shelby Tube

□ Core

⊗ SPT

▨ Grab

○ No Recovery



LWL BORING LOG 2 A-0277.GPJ LWL TEMPLATE.GDT 16/2/25



**LWL ENGINEERING LTD.**

Logged By: Rasie, M.E.

Completion Date: 15/12/2

Reviewed By: Bijeljanin, L.

Completion Depth: 19.2m

Project Eng: Rasie, M.E.

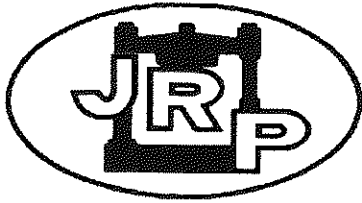
Page 1 of 1



## Appendix C

### Laboratory Testing Results





## J.R. Paine & Associates Ltd.

Client: LWL Engineering Ltd.  
 Project: 6515 - 145A Street NW  
 Edmonton, Alberta  
 Project No.: A-0277  
 Attention: Michael Rasie

JRP File No.: 4192-1

Date: December 3, 2015

### Moisture Content Summary Sheet

<b>Testhole #</b>	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02
<b>Sample #</b>	1	2	3	4	5	6	7	8	9
<b>Depth (m)</b>	0.30	0.75	1.50	2.25	3.00	3.75	4.50	5.25	6.00
<b>M.C. %</b>	5.4	9.4	10.8	12.1	9.3	12.1	11.8	11.7	10.2
<b>Testhole #</b>	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02
<b>Sample #</b>	10	11	12	13	14	15	16	17	18
<b>Depth (m)</b>	6.75	7.50	8.25	9.00	9.75	10.50	11.25	12.00	12.75
<b>M.C. %</b>	14.1	12.2	12.3	8.8	12.6	12.0	13.4	9.0	No Sample
<b>Testhole #</b>	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02
<b>Sample #</b>	19	20	21	22	23	24.A	24B	25	26
<b>Depth (m)</b>	13.50	14.25	15.00	15.75	16.50	17.25	17.25	18.00	18.75
<b>M.C. %</b>	20.2	9.4	5.8	No Sample	9.6	15.3	9.5	18.1	No Sample
<b>Testhole #</b>	TH-02	TH-02	TH-02	TH-02	TH-02	TH-02	Two Samples Labelled TH-02 24. We Labelled the Samples 24A and 24B. 24A Was Sandstone and 24B Was Silty Sand, Possible Trace of Clay, Occasional Pebbles.		
<b>Sample #</b>	27	28	29	30	31	32			
<b>Depth (m)</b>	19.50	20.25	21.00	21.75	22.50	23.25			
<b>M.C. %</b>	15.6	No Sample	15.7	No Sample	17.4	17.0			















# J.R. Paine & Associates Ltd.

CONSULTING AND TESTING ENGINEERS  
EDMONTON - GRANDE PRAIRIE - PEACE RIVER

## SCREEN ANALYSIS

Sample: 7 Depth: 4.50m  
Location: TH-03

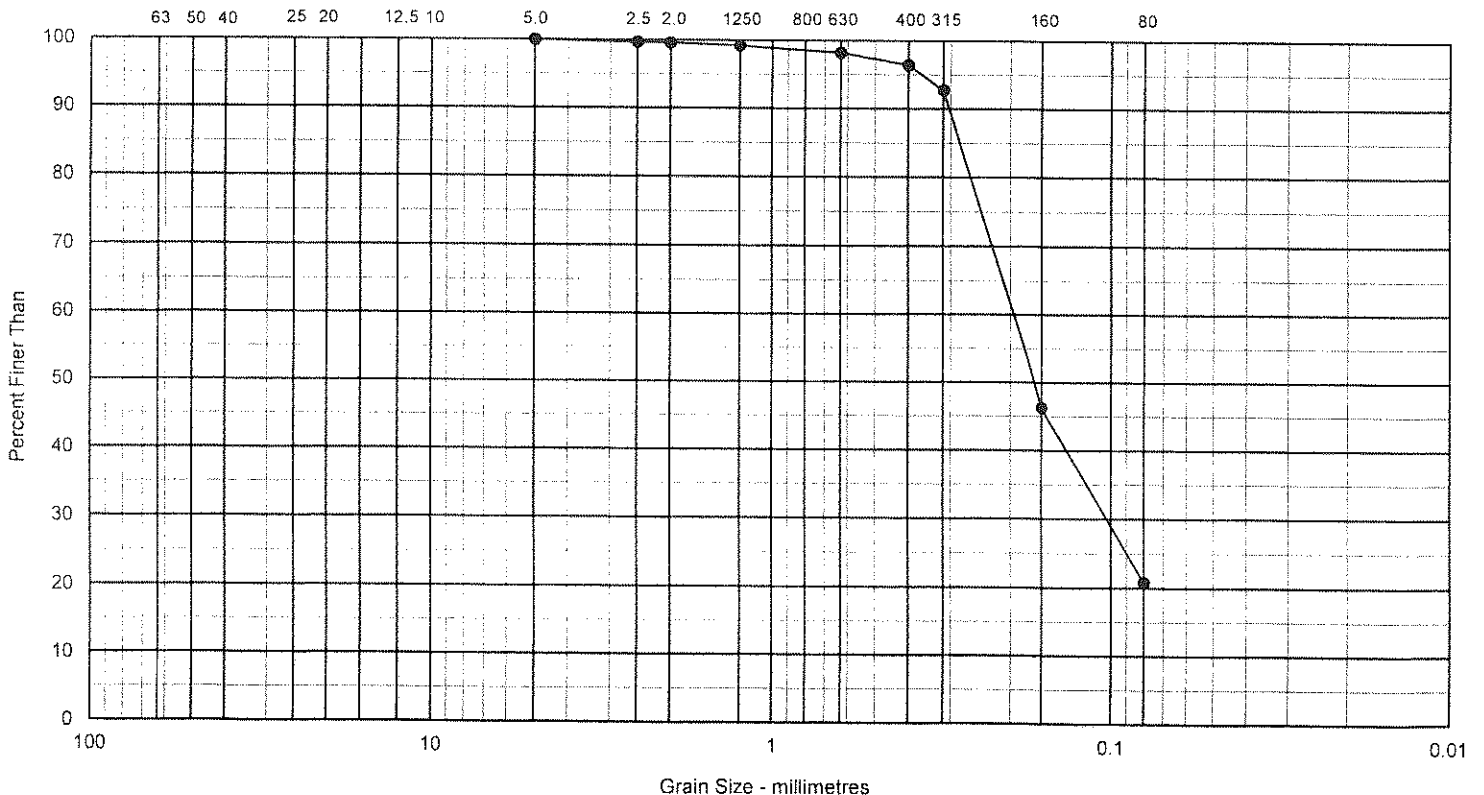
Client: LWL Engineering Ltd.  
Project: A-0277, 6515 - 145A Street NW, Edmonton, Alberta  
Made By: TK File: 4192-1  
Ck'd By: \_\_\_\_\_ Date: December 3, 2015

Sieve No.	Size of Opening millimetres	Specifications		Percent Finer Than	Percent Finer Than Basis Original Sample
		Minimum	Maximum		
80,000	80.0				
63,000	63.0				
25,000	25.0				
20,000	20.0				
16,000	16.0				
12,500	12.5				
10,000	10.0				
5,000	5.0				100.0
2,500	2.5				99.7
2,000	2.0				99.6
1,250	1.250				99.2
630	0.630				98.2
400	0.400				96.4
315	0.315				92.7
160	0.160				46.3
80	0.080				20.8

Description of Sample: \_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_

Method of Preparation: Dry  Washed   
Moisture Content: 5.1 % \_\_\_\_\_ %  
Remarks: \_\_\_\_\_  
\_\_\_\_\_

Time of Sieving: 15 minutes





# J.R. Paine & Associates Ltd.

CONSULTING AND TESTING ENGINEERS  
EDMONTON - GRANDE PRAIRIE - PEACE RIVER

## SCREEN ANALYSIS

Sample: 23 Depth: 16.5m  
Location: TH-03

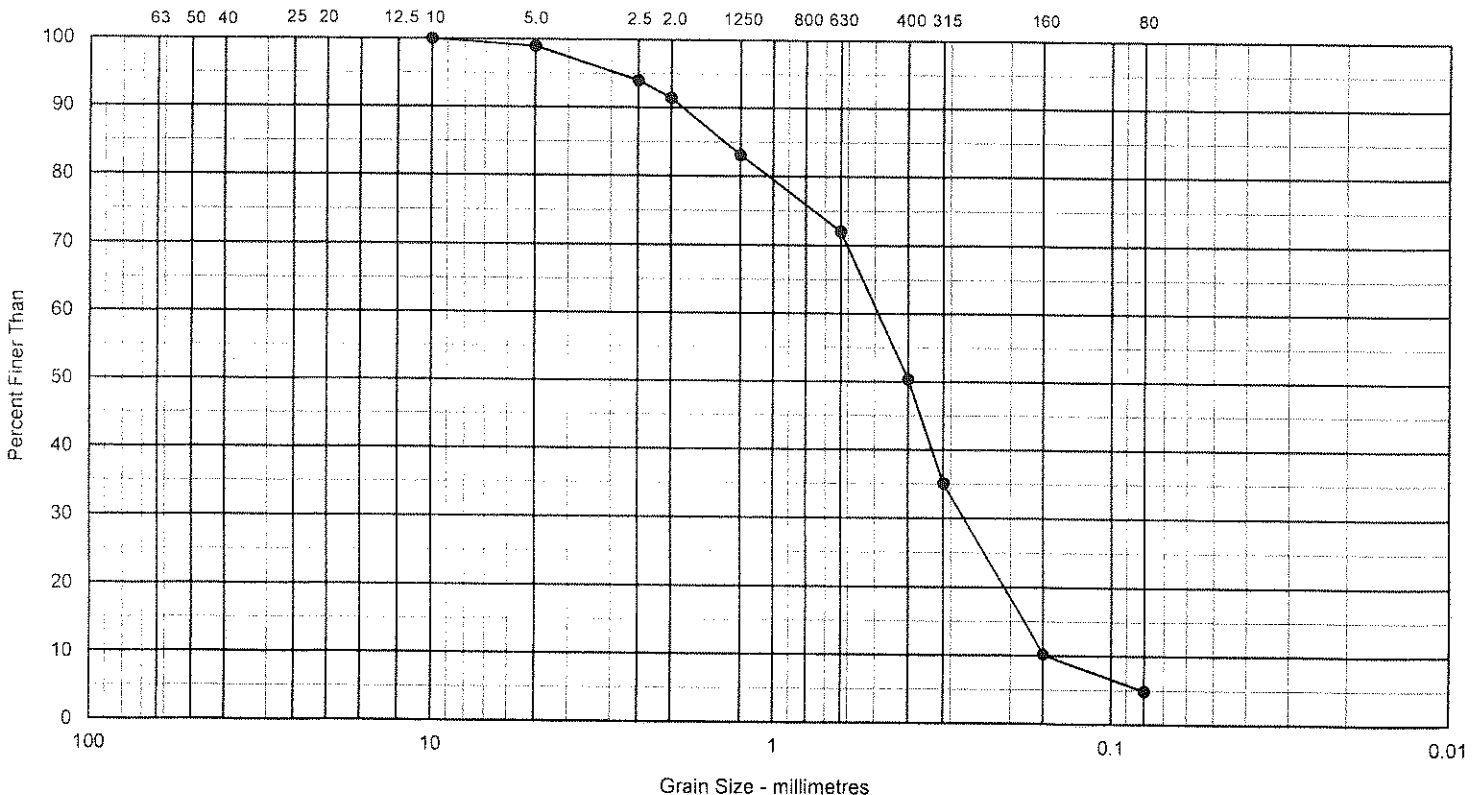
Client: LWL Engineering Ltd.  
Project: A-0277, 6515 - 145A Street NW, Edmonton, Alberta  
Made By: TK File: 4192-1  
Ck'd By: \_\_\_\_\_ Date: December 3, 2015

Sieve No.	Size of Opening millimetres	Specifications		Percent Finer Than	Percent Finer Than Basis Original Sample
		Minimum	Maximum		
80,000	80.0				
63,000	63.0				
25,000	25.0				
20,000	20.0				
16,000	16.0				
12,500	12.5				
10,000	10.0				100.0
5,000	5.0				99.0
2,500	2.5				94.0
2,000	2.0				91.4
1,250	1.250				83.1
630	0.630				72.0
400	0.400				50.4
315	0.315				35.1
160	0.160				10.2
80	0.080				4.9

Description of Sample: \_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_

Method of Preparation: Dry  Washed   
Moisture Content: 5.1 % \_\_\_\_\_ %  
Remarks: \_\_\_\_\_  
\_\_\_\_\_

Time of Sieving: 15 minutes





## Appendix D

### Slope Stability Plots

# SLOPE STABILITY ANALYSIS

Project No. A-0277

6515 145A Street, Edmonton, AB

## Plot 1: Section A-A' - North Slope

### Case 1-1: Failure Through Clay Till

#### LEGEND:

T.O.B. - Top of Bank Line

S.B.L. - Setback Line

TH-01/TH-02 - Approximate Test-Hole Location

#### NOTES:

Soil layers assumed continuous and horizontal.

Retaining walls not simulated in stability analysis.

Groundwater assumed for stability analysis only.

#### MATERIAL PROPERTIES:

Name: UPPER SAND

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 0 kPa

Phi: 33 °

Name: CLAY TILL

Unit Weight: 19.5 kN/m<sup>3</sup>

Cohesion: 0 kPa

Phi: 26 °

Name: LOWER SAND

Unit Weight: 20 kN/m<sup>3</sup>

Cohesion: 0 kPa

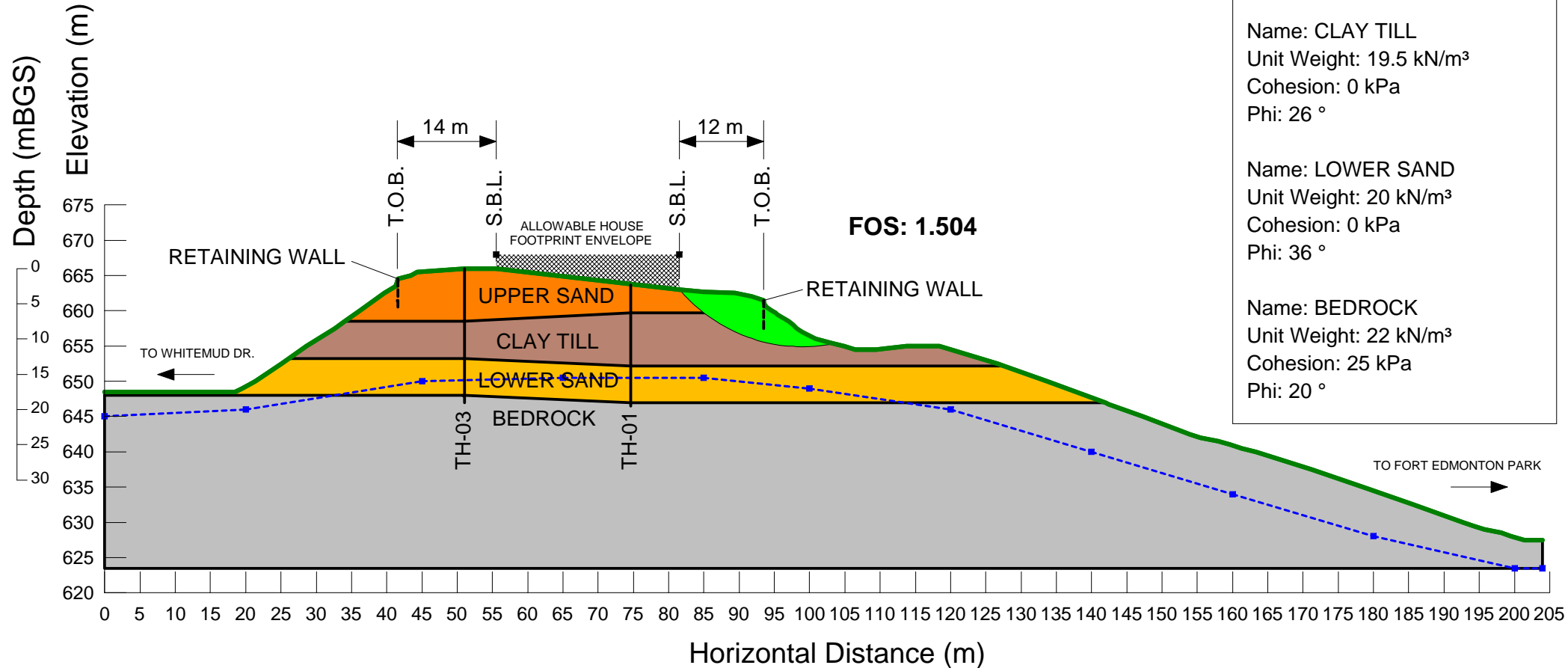
Phi: 36 °

Name: BEDROCK

Unit Weight: 22 kN/m<sup>3</sup>

Cohesion: 25 kPa

Phi: 20 °



# SLOPE STABILITY ANALYSIS

Project No. A-0277

6515 145A Street, Edmonton, AB

## Plot 2: Section A-A' - North Slope

### Case 1-2: Failure Through Lower Sand

#### LEGEND:

T.O.B. - Top of Bank Line

S.B.L. - Setback Line

TH-01/TH-02 - Approximate Test-Hole Location

#### NOTES:

Soil layers assumed continuous and horizontal.

Retaining walls not simulated in stability analysis.

Groundwater assumed for stability analysis only.

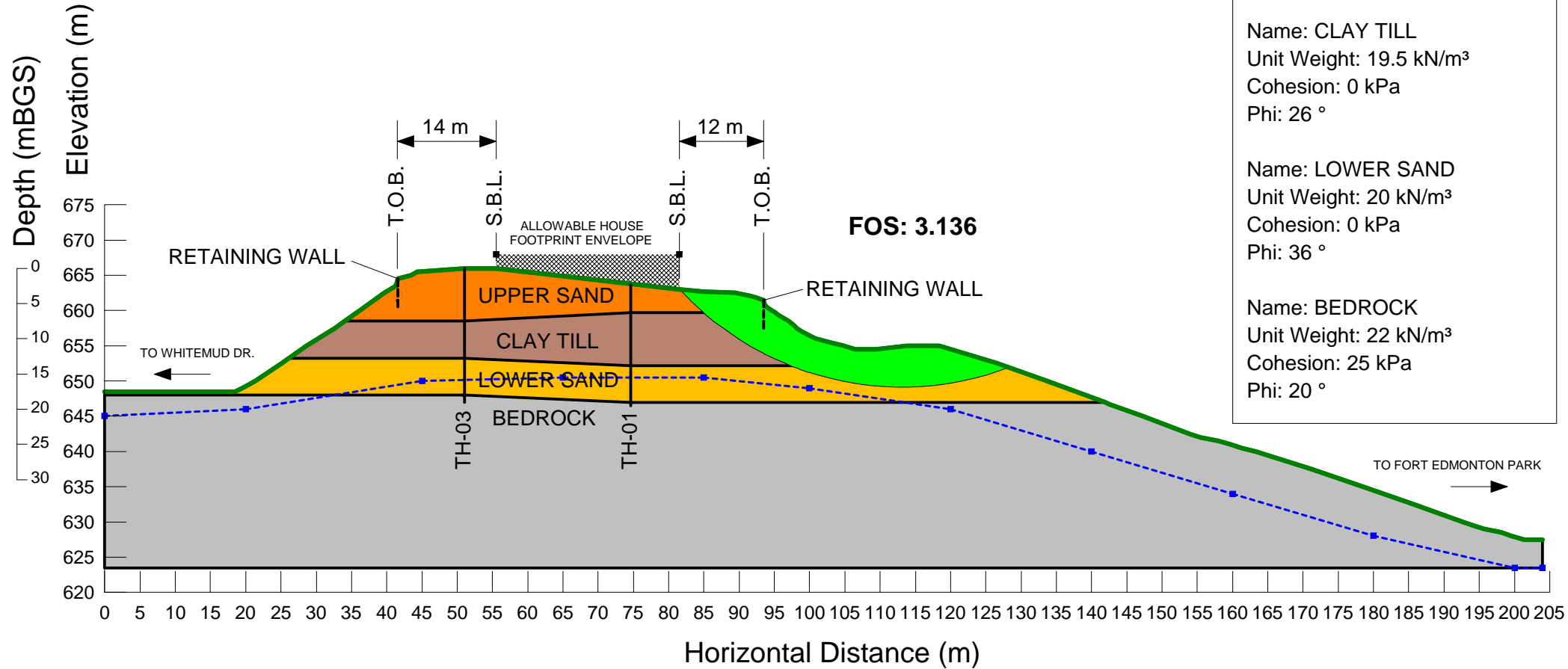
#### MATERIAL PROPERTIES:

Name: UPPER SAND  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Phi: 33 °

Name: CLAY TILL  
Unit Weight: 19.5 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Phi: 26 °

Name: LOWER SAND  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Phi: 36 °

Name: BEDROCK  
Unit Weight: 22 kN/m<sup>3</sup>  
Cohesion: 25 kPa  
Phi: 20 °



# SLOPE STABILITY ANALYSIS

Project No. A-0277

6515 145A Street, Edmonton, AB

## Plot 3: Section A-A' - South Slope

### Case 2-1: Failure Through Clay Till

#### LEGEND:

T.O.B. - Top of Bank Line

S.B.L. - Setback Line

TH-01/TH-02 - Approximate Test-Hole Location

#### NOTES:

Soil layers assumed continuous and horizontal.

Retaining walls not simulated in stability analysis.

Groundwater assumed for stability analysis only.

#### MATERIAL PROPERTIES:

Name: UPPER SAND

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 0 kPa

Phi: 33 °

Name: CLAY TILL

Unit Weight: 19.5 kN/m<sup>3</sup>

Cohesion: 0 kPa

Phi: 26 °

Name: LOWER SAND

Unit Weight: 20 kN/m<sup>3</sup>

Cohesion: 0 kPa

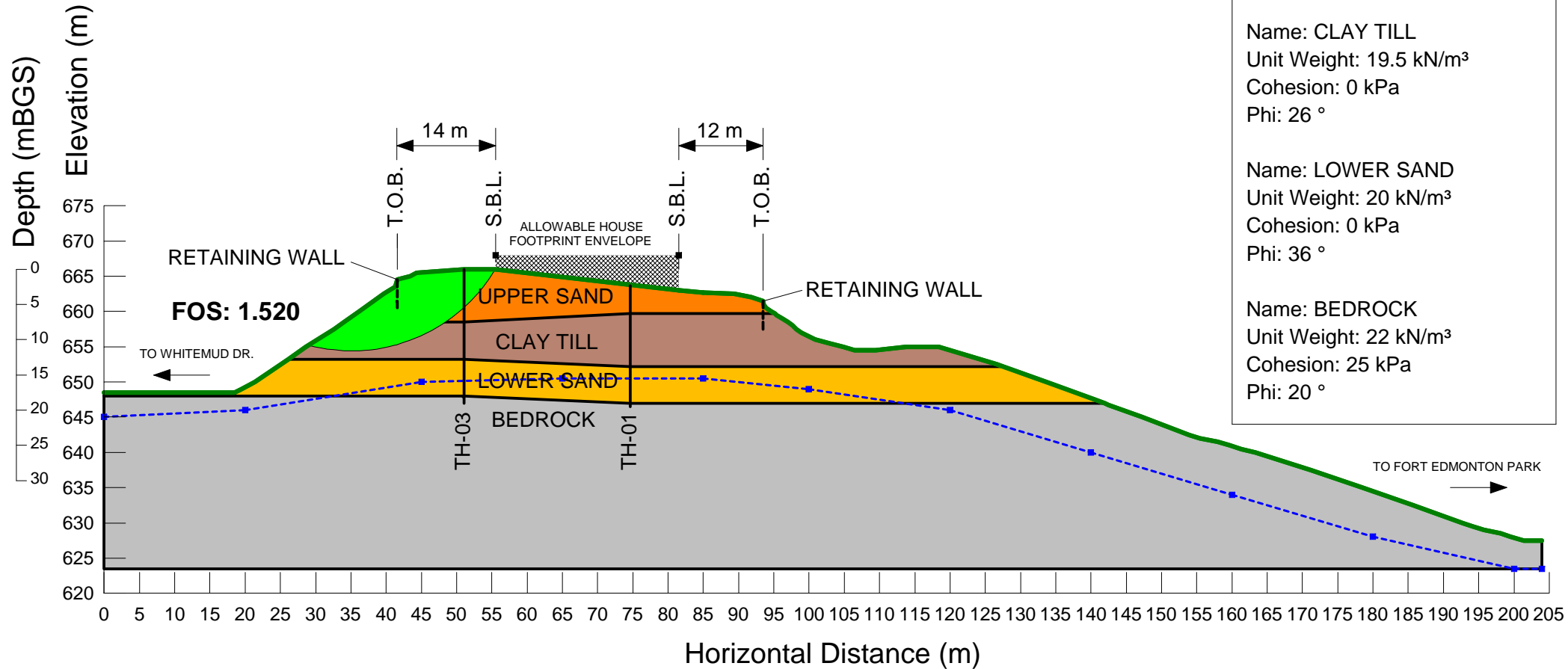
Phi: 36 °

Name: BEDROCK

Unit Weight: 22 kN/m<sup>3</sup>

Cohesion: 25 kPa

Phi: 20 °



# SLOPE STABILITY ANALYSIS

Project No. A-0277

6515 145A Street, Edmonton, AB

## Plot 4: Section A-A' - South Slope

### Case 2-2: Failure Through Lower Sand

#### LEGEND:

T.O.B. - Top of Bank Line

S.B.L. - Setback Line

TH-01/TH-02 - Approximate Test-Hole Location

#### NOTES:

Soil layers assumed continuous and horizontal.

Retaining walls not simulated in stability analysis.

Groundwater assumed for stability analysis only.

#### MATERIAL PROPERTIES:

Name: UPPER SAND

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 0 kPa

Phi: 33 °

Name: CLAY TILL

Unit Weight: 19.5 kN/m<sup>3</sup>

Cohesion: 0 kPa

Phi: 26 °

Name: LOWER SAND

Unit Weight: 20 kN/m<sup>3</sup>

Cohesion: 0 kPa

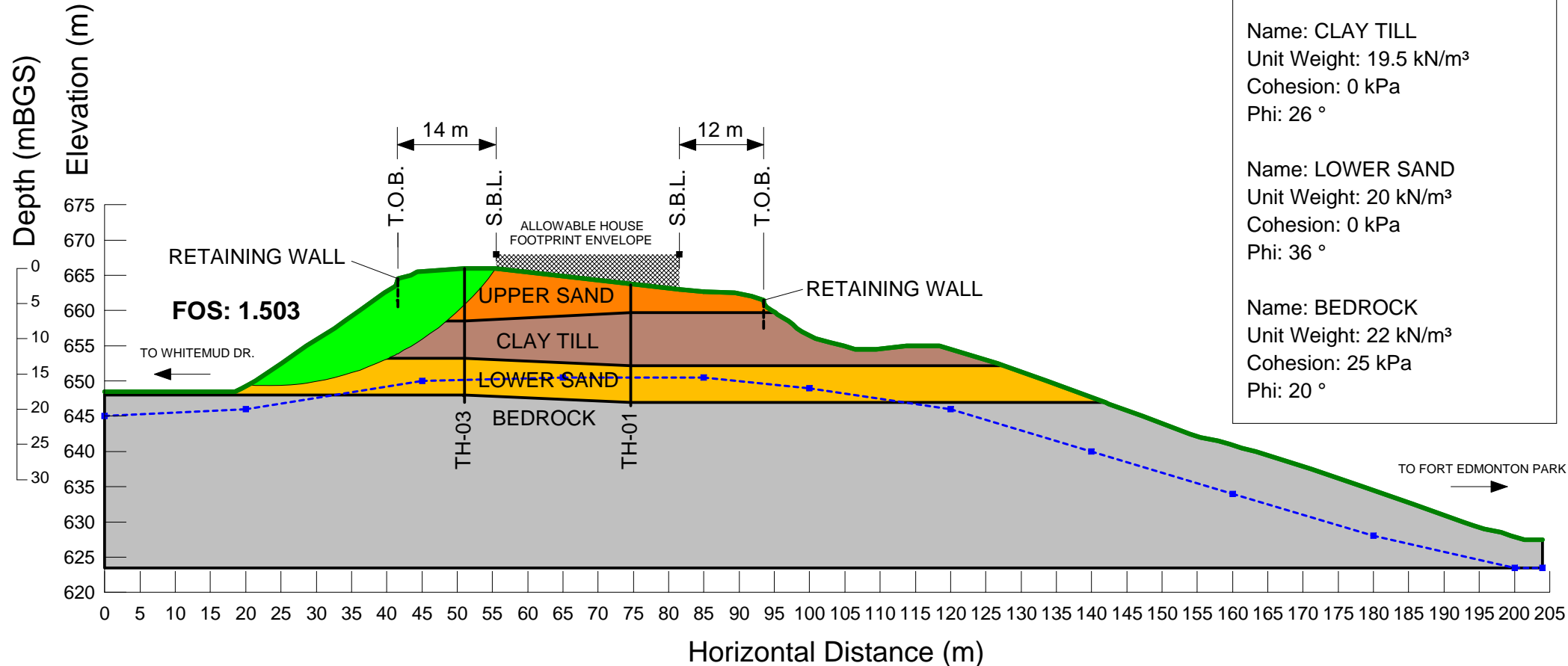
Phi: 36 °

Name: BEDROCK

Unit Weight: 22 kN/m<sup>3</sup>

Cohesion: 25 kPa

Phi: 20 °





**SLOPE STABILITY ANALYSIS**  
 Project No. A-0277  
 6515 145A Street, Edmonton, AB

**Plot 5: Section B-B' - East Slope**  
**Case 3-1: Failure Through Clay Till**

**LEGEND:**

- T.O.B. - Top of Bank Line
- S.B.L. - Setback Line
- P.L. - Property Line
- TH-02 - Test-Hole TH-02 Approximate Location
- FOS - Factor of Safety for Shown Slip Surface

**NOTES:**

Soil layers assumed continuous and horizontal.  
 Groundwater assumed for stability analysis only.

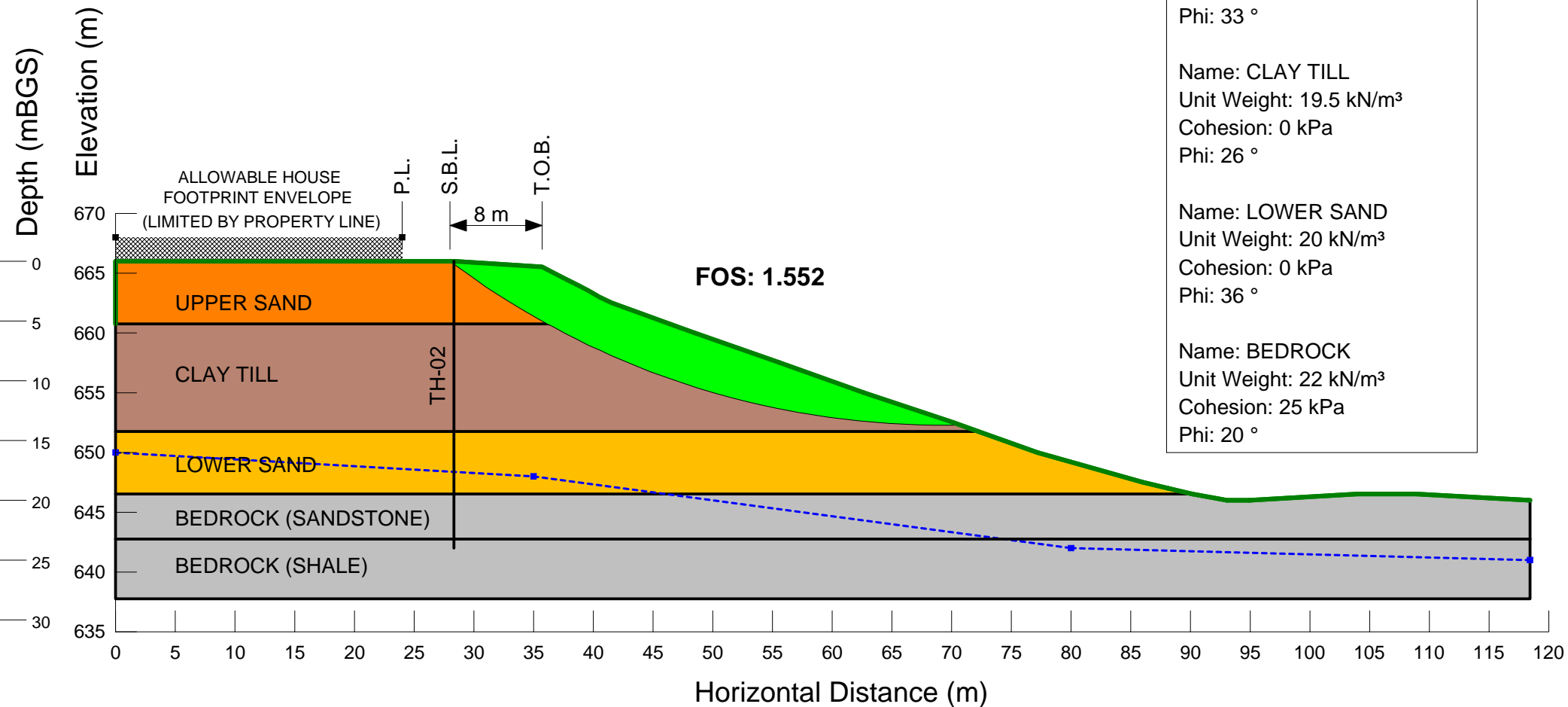
**MATERIAL PROPERTIES:**

Name: UPPER SAND  
 Unit Weight: 19 kN/m<sup>3</sup>  
 Cohesion: 0 kPa  
 Phi: 33 °

Name: CLAY TILL  
 Unit Weight: 19.5 kN/m<sup>3</sup>  
 Cohesion: 0 kPa  
 Phi: 26 °

Name: LOWER SAND  
 Unit Weight: 20 kN/m<sup>3</sup>  
 Cohesion: 0 kPa  
 Phi: 36 °

Name: BEDROCK  
 Unit Weight: 22 kN/m<sup>3</sup>  
 Cohesion: 25 kPa  
 Phi: 20 °



**SLOPE STABILITY ANALYSIS**  
 Project No. A-0277  
 6515 145A Street, Edmonton, AB

**Plot 6: Section B-B' - East Slope**  
**Case 3-2: Failure Through Clay Till**

**LEGEND:**

- T.O.B. - Top of Bank Line
- S.B.L. - Setback Line
- P.L. - Property Line
- TH-02 - Test-Hole TH-02 Approximate Location
- FOS - Factor of Safety for Shown Slip Surface

**NOTES:**

Soil layers assumed continuous and horizontal.  
 Groundwater assumed for stability analysis only.

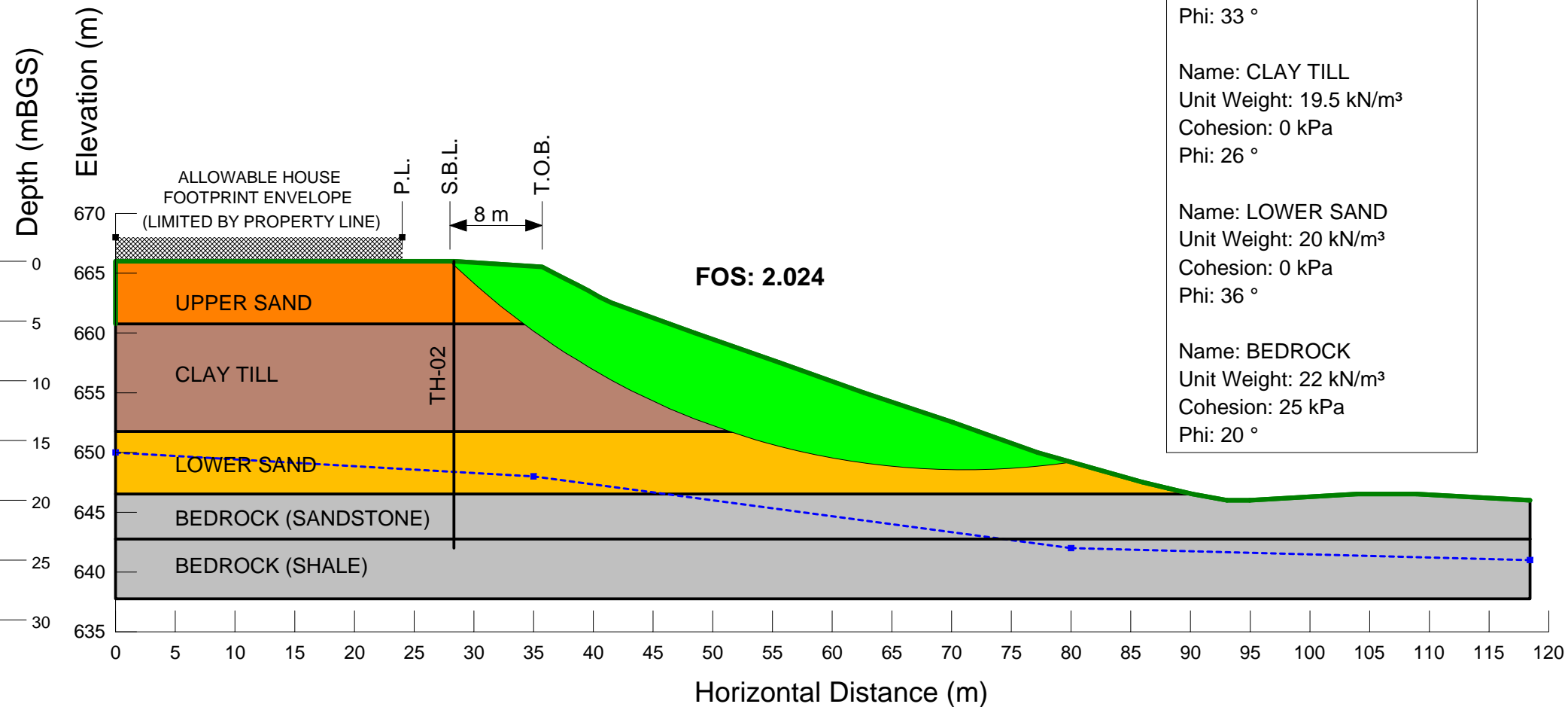
**MATERIAL PROPERTIES:**

Name: UPPER SAND  
 Unit Weight: 19 kN/m<sup>3</sup>  
 Cohesion: 0 kPa  
 Phi: 33 °

Name: CLAY TILL  
 Unit Weight: 19.5 kN/m<sup>3</sup>  
 Cohesion: 0 kPa  
 Phi: 26 °

Name: LOWER SAND  
 Unit Weight: 20 kN/m<sup>3</sup>  
 Cohesion: 0 kPa  
 Phi: 36 °

Name: BEDROCK  
 Unit Weight: 22 kN/m<sup>3</sup>  
 Cohesion: 25 kPa  
 Phi: 20 °



# SLOPE STABILITY ANALYSIS

Project No. A-0277

6515 145A Street, Edmonton, AB

## Plot 7: Section C-C'

### Case 4-1: Failure Through Clay Till

#### LEGEND:

T.O.B. - Top of Bank Line

S.B.L. - Setback Line

TH-02 - Approximate Test-Hole Location

#### NOTES:

Soil layers assumed continuous and horizontal.

Groundwater assumed for stability analysis only.

#### MATERIAL PROPERTIES:

Name: UPPER SAND  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Phi: 33 °

Name: CLAY TILL  
Unit Weight: 19.5 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Phi: 26 °

Name: LOWER SAND  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Phi: 36 °

Name: BEDROCK  
Unit Weight: 22 kN/m<sup>3</sup>  
Cohesion: 25 kPa  
Phi: 20 °

