April 18, 2016 File: 2016-2179

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Attention: Mr. Mitch Braun

Re: Geotechnical Investigation
Hamptons Golf Course Redevelopment
Calgary, Alberta

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation and slope stability assessment undertaken by E2K Engineering Ltd. (E2K) for the proposed residential development at the above noted address in Calgary, Alberta. It is understood that the proposed development will consist of two separate residential parcels located within existing golf course lands.

The objective of the geotechnical investigation would be to determine the subsurface soil and groundwater conditions at the site in order to provide geotechnical recommendations affecting the construction of the proposed development. The geotechnical investigation consisted of drilling nine (9) boreholes in Site A and three (3) boreholes in Site B within the proposed developments, installing standpipes, soil sampling and laboratory testing.

2.0 SITE DESCRIPTION

The project sites (Sites A and B) are located in the Hamptons Golf Course in NW Calgary. Site A is located south of Hamptons Drive NW and Site B is located north of Hamptons Way NW. The current drainage pattern varies significantly across the golf course due to the hilly terrain. A manmade pond was located within Site A which was frozen at the time of drilling. Site plans of the proposed developments are shown on Figures 1 and 2.

3.0 DETAILS OF THE INVESTIGATION

The geotechnical investigation consisted of drilling twelve (12) boreholes within the two proposed developments. The drilling was performed on March 7 and March 8, 2016 by All Service Drilling Inc. of Airdrie, Alberta. The boreholes were drilled to a maximum depth of 6.6 m below ground surface. All boreholes were drilled at the marked locations consulted by the client. The borehole location plans are shown on Figure 1 and Figure 2.

The subsurface soil conditions encountered were continuously logged using the Modified Unified Soil Classification System which includes soil types, depths, moisture contents and soil descriptions. Disturbed soil samples were obtained at regular intervals from the auger and tested in the laboratory for moisture, Atterberg and sulphate content. Standard Penetration Tests (SPT's) were conducted at regular intervals (approximately 1.5 m), in all boreholes. Pocket penetrometer readings were taken on all cohesive soils encountered.

A standpipe was installed to a depth of approximately 6.6 m in all boreholes for future groundwater table measurements. Following drilling, the boreholes were backfilled to the surface with drill cuttings, sand and a bentonite cap. As water levels will need to be measured at each borehole on a periodic basis, flush-mounted road boxes were installed at the surface in all boreholes. The borehole logs are shown in the Appendix of this report.

The laboratory testing program included visual classifications and moisture content determinations for all soil samples. Atterberg Limit tests were carried out on select soil samples. Six samples were tested for water soluble sulphate to determine the type of concrete that will be required during the construction. Unconfined compressive strength tests were conducted on the undisturbed samples. Two hydrometer tests were performed to determine the gradation of particle sizes in the soil. The results of the laboratory program are presented on the borehole logs in the Appendix of this report.

The locations of the twelve (12) boreholes were recorded with a handheld GNSS receiver. The coordinates and existing ground elevations are shown on the boreholes logs.

4.0 SUBSURFACE CONDITIONS

4.1 Soil Stratigraphy (Site A)

The geologic profile within Site A (Borehole 4 to 12) consisted of stiff to hard clay till below topsoil.

Detailed soil descriptions are provided on the borehole log in the Appendix and are discussed in the following section. Variations in the thickness and condition of materials observed in the boreholes could be encountered in areas of the site not investigated.

4.1.1 Topsoil

Topsoil was encountered in all boreholes drilled at the site. The thickness of the topsoil was observed to be 0.2 m and was described as clay containing trace sand, trace roots, trace organics, damp to moist and dark brown to black in colour.

4.1.2 Clay Till

Clay till was encountered in all boreholes below the topsoil at a depth of 0.2 m and extended to the maximum depth investigated at 6.6 m except in Boreholes BH-11 and BH-12 where rafted bedrock was encountered. The clay till was described as being silty containing sand (trace to some sand), and varied in gravel content (trace gravel to gravelly). Traces of oxides and coal specks were also generally observed in the till. The clay till was found to be low to medium plastic, damp to moist and light brown to grey in colour.

Moisture content tests performed on the disturbed samples of the clay till resulted in a moisture contents ranging from 13.1% to 25.4%, indicative of damp to moist in-situ moisture content. The SPTs performed in the clay till material resulted in blow counts ranging from 4 to 64 blows for 300 millimeters (mm) of penetration, indicative of firm to hard, cohesive material. Pocket penetrometer readings taken within the clay till ranged between 150 kPa and 400 kPa, indicative of a stiff to hard cohesive material. Five Atterberg limits were performed on samples of the clay and are shown in the following table.

Summary of Atterberg Limit Tests				
Borehole No.	**Depth (m)**	**Liquid Limit (%)**	**Plastic Limit (%)**	**Plasticity**
BH-05	2.3	39	18	Medium
BH-07	0.8	38	16	Medium
BH-09	2.3	38	16	Medium
BH-10	1.5	35	16	Medium
BH-11	0.8	37	15	Medium

The results indicate a medium plastic, cohesive soil.

A grain size distribution analysis was performed on the disturbed sample from BH-12 at a depth of approximately 6.1 meters. The result indicates a 39% clay content, a 54% silt content and 7% sand. The result indicates that the material primarily consists of clayey silt. It should be noted that even though the material consists primarily of silt, the material behaves as a cohesive material.

4.1.3 Sandstone (Bedrock)

An interbedded sandstone bedrock was observed below the clay till in the Borehole BH-11 at 3 meters and in the Borehole BH-12 at a depth of 2.1 m respectively. In BH-12, the 2.4 m thick rafted bedrock was terminated in clay till at a depth of 4.6 m. This material was described as being very weak, weathered and light brown in colour. Moisture content tests performed on the disturbed samples of the bedrock material resulted in moisture contents ranging from 6.2% to 11.4%, indicative of a dry to damp in-situ moisture content.

4.2 Soil Stratigraphy (Site B)

The geologic profile within Site B (Boreholes 1 to 3) consisted of stiff to hard clay till below topsoil and overlying sandstone.

Detailed soil descriptions are provided on the borehole log in the Appendix and are discussed in the following section. Variations in the thickness and condition of materials observed in the borehole could be encountered in areas of the site not investigated.

4.2.1 Topsoil

Topsoil was encountered in all boreholes drilled at the site. The thickness of the topsoil was observed to be 0.2 m and was described as clay containing trace sand, trace roots, trace organics, damp to moist and dark brown to black in colour.

4.2.2 Fill

A silt fill with a thickness of 1.3 m was observed below the topsoil in Borehole BH-02. The fill was described as being sandy, contained some gravel and trace clay, firm, damp and light brown in colour. The material was considered a fill material due to its loose density and variation from soil conditions at elevation depths.

4.2.3 Clay Till

Clay till was encountered below the topsoil in the Boreholes BH-01 and BH-03 at a depth of 0.2 m and below the fill in Borehole BH-02 at a depth 1.5 m. The thickness of the clay till was observed between 5.5 and 6.0 m and was terminated in the sandstone bedrock at depths of 5.6 and 6.1 m. This material was described as being silty containing sand (trace to some sand), and varied in gravel content (trace to some gravel). Traces of oxides and coal specks were also generally observed in the till. The clay till was found to be low to medium plastic, damp to moist and brown to grey in colour.

Moisture content tests performed on the disturbed samples of the clay till resulted in a moisture content ranging from 13.7% to 22.5%, indicative of damp to moist in-situ moisture content. The SPTs performed in the clay till material resulted in blow counts ranging from 9 to 34 blows for 300 millimeters (mm) of penetration, indicative of stiff to hard, cohesive material. Pocket penetrometer readings taken within the clay till ranged between 150 kPa and 350 kPa, indicative of a stiff to hard cohesive material. One UCS test conducted on the undisturbed clay sample recovered at 3 m below grade in Borehole BH-01 resulted in a compressive strength of 379 kPa. One Atterberg limit was performed on a sample of the clay till in BH-02 at a depth of 3.8 m and resulted in a liquid limit of 37% with a corresponding plastic limit of 15%, indicating a medium plastic, cohesive soil.

A grain size distribution analysis was performed on the disturbed sample from BH-03 at a depth of approximately 3 meters. The result indicates a 34% clay content, a 48% silt content and 18% sand.

4.2.4 Sandstone (Bedrock)

Sandstone bedrock was observed in all three boreholes at Site B at depths below 5.6 and 6.1 m. This material was described as being very weak, weathered and light brown to brown in colour. Auger refusal occurred at depths of 6.2 m and 6.3 m below ground surface in Boreholes BH-02 and BH-03, respectively.

Moisture content tests performed on the disturbed samples of the bedrock material resulted in moisture contents ranging from 8.6% to 9.8%, indicative of a damp in-situ moisture content.

4.3 Groundwater

During drilling, seepage or saturated conditions were encountered in the Boreholes BH-04, BH-05, BH-08 and BH-09. A standpipe was installed in all boreholes to a depth of approximately 6.6 m. Upon completion of the drilling, the groundwater table was measured and is shown on the table below. Groundwater monitoring in the standpipes was also performed on March 18, 2016 by E2K personnel. The groundwater depths in these standpipes are provided below.

Static Groundwater Depth Below Surrounding Grade				
Borehole	**Ground Elevations (m)**	**Groundwater Table at Drill Completion (m)**	**Static Groundwater Table March 18, 2016 (m)**	**Static Groundwater Table March 18, 2016 Elevation (m)**
BH-01	1211.29	dry	dry	-
BH-02	1203.70	dry	dry	-
BH-03	1210.67	dry	dry	-
BH-04	1195.43	1.5	0.9	1194.53
BH-05	1195.26	dry	6.4	1188.86
BH-06	1193.71	dry	dry	-
BH-07	1190.98	dry	dry	-
BH-08	1194.47	4.2	3.9	1190.57
BH-09	1197.95	dry	3.0	1194.95
BH-10	1202.47	dry	dry	-
BH-11	1197.91	dry	dry	-
BH-12	1199.20	dry	dry	-

Perched water may be present in sand lenses. During the spring months and times of heavy precipitation the long term groundwater table elevation is anticipated to fluctuate. It should be noted that the groundwater varies with seasonal conditions including, precipitation, temperature, site drainage characteristics, etc. Groundwater measurements will be conducted once a month for six month to determine the seasonal variation.

5.0 COMMENTS AND RECOMMENDATIONS

Based on the results of the investigation, the testing carried out, and our understanding of the proposed development, we submit the following comments and recommendations related to the geotechnical aspects of the site that may affect the development.

5.1 Site Preparation

Any vegetation, topsoil, disturbed or organic soil should be stripped away from any portions of the site where fill is needed to bring the subject site to final grade. Variations of the above noted quantities and conditions may occur in areas of the site not investigated. Areas of the site requiring subgrade support (ie, Roads, Sidewalks, Lanes, etc) should be inspected for bearing capacity at the time of construction. The exposed subgrade should be proof-rolled to identify any localized soft areas within the development footprint. These soft areas should be sub-excavated and replaced with a material approved by E2K. This will allow for an evenly prepared subgrade at the time of construction, and reduce the risk of any differential settlement resulting from poorly compacted fills.

The pond located within Site A is anticipated to be pumped out and filled with suitable soil. The fill required to bring the subject site to design grade may consist of locally available cohesive soil of low to medium plasticity or well-graded gravel-sand mixtures with little or no fines. The encountered clay material is suitable for reuse as backfill materials. Where fill is required, the fill material should be within 2% of the optimum moisture content with the degree of compaction of each lift being at least equal to 98% of the Standard Proctor Maximum Dry Density (ASTM Method D-698). All fill material must not contain rocks over 200 mm in diameter. Fill should be placed in lifts such that the maximum thickness of any lift, before compaction, does not exceed 200 mm.

It should be expected that settlement of new or replaced fill will occur due to "self-weight", particularly where thick fills are placed. For granular fill soils compacted to a minimum of 100% of Standard Proctor Maximum Dry Density (SPMDD), the fill settlement is expected to be 0.5% or less of the fill height. For clay fill compacted to 98% of SPMDD, the fill settlement is expected to be in the range of 0.5 to 1% of the fill height, with the majority of the settlement occurring during the first freeze thaw cycle.

5.2 Site Grading and Drainage

At the time of this report, no cut/fill plan was available for review. It is anticipated that minimal grading will occur on the sites. The finished grade in the vicinity of structures or roads should be sloped away from these elements. The upper 0.5 m of backfill around any proposed structure should consist of compacted clay to act as a seal against the ingress of surface runoff. The clay should extend a distance of 3 meters from the structure and should be graded away at a minimum slope of 2% in landscaped areas and 1.5% in paved areas.

Site grading should be provided in paved areas, both during and following construction, such that water is rapidly shed from the surface of the pavement to a positive drainage system. Water should not be allowed to pond on, or be adjacent to any proposed pavement areas.

Due to site development, the static groundwater elevation across Site 'A' could rise to near the foundation depth of future residences. For this reason, a suitable weeping tile system installed along the perimeter of shallow foundations should be considered for any structures that will extend below grade with half or full basements. On Site 'B', no indication of groundwater was encountered and it is not expected that development on this site will cause groundwater to rise to within foundation depths. For this reason, weeping tile systems would not be required to control groundwater around proposed structures, however, weeping tile is generally recommended to promote dry foundation conditions in the event of poor surface drainage, prolonged precipitation or flooding conditions.

Upon establishment of a site grading plan, E2K should be notified to provide comment on stripping requirements prior to fill. Inspections should be performed prior to fill placement to verify that all organic and deleterious materials have been removed. In general, the existing site soils are suitable for reuse as fill material for rough grading activities. Grading of slopes will require a 5H:1V backslope in building areas prior to placing fill.

5.3 Foundation System

Based on our evaluation of the site conditions and the proposed development, shallow foundations are considered suitable foundation systems for structures within the proposed development. the preferred option, provided the shallow groundwater table is accounted for in Site A.

5.3.1 Spread and/or Strip Footings

Based on the field investigation performed, the factored geotechnical resistance at ULS (utilizing a 0.5 reduction factor) for the design of spread and strip footings are to be taken as 110 kPa for footings founded within native materials at depths of either 1.5 m or 3.0 m below current surface grade elevation. Isolated soft areas were encountered on each site, therefore bearing inspections of the foundation soils **must** be completed and verified prior to the construction of any proposed footing foundations.

5.3.1.1 Footing Serviceability Limit States Design

In addition to the assessment of ULS foundation bearing resistance, SLS must be addressed. SLS is an assessment of settlement experienced under unfactored structural loading conditions.

The exact calculation of settlement is complex and difficult without significant laboratory soil testing and a complete understanding of foundation loading conditions. The following expression can be used to estimate the settlement of shallow foundations (footings and pads) under SLS conditions.

$$S = KP / LE$$

Where:

S	=	Foundation settlement (m)
K	=	[0.453 x ln (L / B)] + 0.788
L	=	Footing length (m)
B	=	Footing width (m)
P	=	Unfactored load at the base of the footing (kN)
E	=	Elastic Modulus of the foundation soil, use 8,000 - 15,000 kPa

An elastic modulus, E of 8,000 kPa should be used for footings placed upon the stiff clay or 15,000 kPa for very stiff clay material encountered during the investigation of the subject site.

The minimum footing widths should be 0.45 m for strip footings and 0.9 m for spread footings. The perimeter footings for a heated structure should be founded at least 1.5 m below finished grade to be below the frost line, reducing the potential harmful effects that may occur from frost. In the case of unheated structures outside the influence of any beneficial heat transfer, the minimum footing depth should be 2.5 m below finished grade.

The footings must not be founded on un-compacted fill, loosened or disturbed native or fill soils, or organic soils. The base of the footing excavations should be thoroughly cleaned of all loosened or disturbed soil prior to pouring concrete. Soft or weak areas should be removed and replaced with a more suitable material.

5.3.2 Swell Potential

Clay soils are known to experience swelling and shrinkage when exposed to variances in moisture content. Medium plastic clay near the plastic limit was encountered in most areas of this site. To minimize the risk for such movements the following recommendations should be followed:

- When preparing the clay subgrade at the foundation elevation, it is **critical to avoid excessive drying** as rehydration of the clay subgrade may result in swelling pressures at the base of the shallow foundation system.

- The potential for swelling can be reduced by implementing proper surface drainage measures around the exterior of the foundation and limiting potential sources of external water infiltrating beneath the footings (please see Section 5.2 for details on drainage).

Foundation excavations should not be exposed to rain, snow, freezing temperatures and/or ponded water prior to footing construction. In the instance where seepage is encountered within the footing excavation, the base should be graded to a low point and groundwater must be removed prior to casting the footing. **It is anticipated that excavations may extend below the groundwater elevation in Site A. It should be anticipated that groundwater will need to be controlled during construction at the subject site.**

If the construction of a foundation system is taking place during winter conditions, steps should be taken to insulate and heat the foundation elements, as well as protect them from the elements to prevent frost from developing underneath the footings. If frost develops underneath the footings, substantial foundation movement is to be expected.

The bearing surface of footings is to be inspected by a geotechnical engineer to verify that the required bearing support is attained. Footings may also be placed on engineered fill provided the foundation soil conditions are confirmed by a qualified geotechnical engineer prior to footing construction.

5.3.3 Frost Protection and Penetration

Protection against the effects of frost action will likely be a concern at this site due to the inherent frost susceptibility of the near surface soil.

Based on the 1 in 25 year return period winter, the depth of frost penetration in the Calgary area is approximately 1.8 m assuming no snow cover and a soil profile consisting of native silty clay material or silty clay fill with typical moisture contents of 15%. In the case of granular backfill soils with an average moisture content of 10%, the expected depth of frost penetration would be about 3.1 m for the same return period.

It is assumed that buildings within the subject development will be heated. Accordingly, footings must be placed at a minimum depth of 1.5 m below finished grade to prevent frost related movements.

If it is deemed necessary to reduce the frost related forces acting on the building, one option would be to install insulation adjacent to frost walls. For an unheated building, the insulation should be 80 mm thick and extend 2.44 m out from all edges of the slab. Insulation extending out from the slab should be placed at a depth of 300 mm below final grade and should be installed on top of 300 mm of clean granular fill. Multiple layers of insulation should be used to achieve the required thickness, if possible. Consecutive layers of insulation should be staggered to cover gaps between insulation boards and further reduce frost heave effects.

5.4 Foundation Settlement

To minimize foundation settlement and differential settlement, E2K is to be onsite to inspect the subgrade of any footing foundation systems to ensure that no soft spots, loose fill or organic soils are within the building footprint, and to address such issues should they arise. In general, it is not anticipated at this time that settlement in excess of 25 mm will occur at this site, based on the borehole information obtained during the site investigation.

5.5 Burial Depths for Water Lines

The burial depths for water lines should be established on the basis of the 25-year return period with an added embedment depth as a safety margin since the trench backfill may not consist entirely of clay. Where the water lines will be covered with primarily clay backfill, the minimum burial depth should be taken as 2.75 m and increased to 3.2 m where granular backfill is used. These burial depths should at least meet City Standards, which are 2.75 m in a clay soil and 3.05 m in a granular soil.

5.6 Pipe Support

For installation of utilities across the subject sites, we do not anticipate any difficulty regarding pipe support while using conventional methods. To prevent the migration of fines into the bedding gravel, the installation of plugs consisting of compacted clay or lean concrete is recommended at frequent intervals around the pipe and manholes. In addition, weepers should be connected into the storm system upstream of the plugs. This will reduce water flow through the bedding gravel and minimize migration of fine grained soils. In some cases, a non-woven geotextile filter fabric may be required to separate fine grained silt and sand from bedding gravel. E2K can provide further recommendations for plug frequency and filter fabric requirements upon request.

5.7 Excavations

Excavations are expected to be required for utility trenches and footing preparation. Based on the boreholes advanced at the site, excavations will be completed primarily within native clay soils. Conventional open excavations with cut slopes are considered to be appropriate for short term excavations (periods of 1 to 2 months) to depths of about 4 m below grade.

For the typical excavations anticipated at the site, short-term trench slopes through the clay soils may be cut back at slopes no steeper than 1H (Horizontal):1V (Vertical). Flatter slopes in the order of 2H:1V or flatter will be required if sand layers, soft soils, or poor quality fill are encountered. The degree of stability of excavated trench walls typically decreases with time and therefore construction should be directed at minimizing the length of time that service trenches are left open.

Where groundwater seepage is encountered, flatter excavation slopes in the order of 2 to 2.5H:1V or flatter may be required. Where seepage is encountered, grading the bases of the excavations towards low points and pumping of the groundwater accumulations should suffice. Groundwater seepage was encountered during the field investigation and groundwater elevations indicate a potentially high spring groundwater elevation in Site A. The construction crews should be prepared to deal with groundwater issues during construction. As a result of the localized high water table, shoring may be required within Site A.

Stockpiles of materials and excavated soil should be placed away from the slope crest by a distance equal to the depth of excavation. Similarly, wheel loads should be kept back at least 1 m from the crest of the excavation. The applicable sections in the Occupational Health and Safety Act must be followed.

5.8 Trench Backfill

In areas where backfilled service trenches coincide with areas that subgrade support is required, the backfill should consist of an engineered fill and be placed according to Section 5.1. It should be expected that settlement of new or replaced fill will occur due to "self-weight", particularly where thick fills are placed. For granular fill soils compacted to a minimum of

100% of SPMDD, the fill settlement is expected to be 0.5% or less of the fill height. For clay fill compacted to 98% of SPMDD, the fill settlement is expected to be in the range of 0.5 to 1% of the fill height, with the majority of the settlement occurring during the first freeze thaw cycle.

In areas where trench backfill coincide with pavement or foundation structures, construction of these elements should be delayed as much as practical following completion of the trench backfill, allowing for settlement of the fill soils to occur.

In landscaped areas or other areas not requiring subgrade support, the degree of compaction and fill type would be less critical. In these areas generally all inorganic soils from the excavation could be re-used as backfill for trenches and the degree of compaction could be reduced. However, it should be recognized that poorly compacted fill soils would be subject to higher degrees of post construction settlement and could disrupt the desired surface drainage scheme. Where the backfill is placed at 90 to 95% of SPMDD, the settlement in the backfill is estimated to be in the order of 2 to 4% of the fill thickness.

5.9 Groundwater Consideration

During drilling, saturated soils were encountered in three boreholes. Groundwater monitoring in the standpipes was performed on March 18, 2016 and was observed to be as high as 0.9 m below grade within Site ‘A’. The level of the hydrostatic water table is anticipated to fluctuate throughout the year, depending upon variations in precipitation, infiltration, site topography and drainage. Groundwater table at shallow depth could affect development within Site A. We recommend that de-watering systems be considered during construction in order to control any seepage that may be encountered. In addition, foundation weeping tile, waterproofing, and buoyancy needs to be considered for foundation design of structures on Site A. No groundwater was encountered on Site ‘B’ and it is not expected that groundwater control during construction will be required for development in this area.

5.10 Requirements for Concrete

To determine the potential of sulphate attack on any concrete in contact with soils at the site, six (6) soil samples were taken from the boreholes to test for water-soluble sulphate concentrations. The results of the chemical tests are summarized in the following table.

Summary of Water-Soluble Sulphate Concentration			
Borehole No.	**Depth (m)**	**Sulphate Concentration (%)**	**Degree of Exposure**
BH-02	1.5	0.16	Moderate
BH-04	1.5	0.11	Moderate
BH-08	0.8	0.14	Moderate
BH-09	2.3	0.10	Moderate
BH-11	1.5	0.13	Moderate
BH-12	1.5	0.12	Moderate

The sulphate content revealed a "moderate" potential for sulphate attack. Therefore as per CSA guidelines, all concrete in contact with native soils at this site should be made from CSA Type 20E, 40 or 50 (moderate sulphate) Portland cement possessing a minimum compressive strength of 30 MPa at 56 days. This would apply to all concrete foundations, utilities and sidewalks in contact with the site soils. The maximum water cement ratio should be 0.5 and an air entrainment agent is recommended for improved workability and durability. For foundation elements in contact with imported fill materials, the fill should be tested for soluble sulphate content and concrete type chosen accordingly.

5.11 Seismic Classification

Seismic design for various structures is based on the 2005 National Building Code of Canada (NBCC). The primary objective of the NBCC earthquake resistant design requirements is to protect the life and safety of the public in response to strong ground shaking. Structures designed in conformance to the code may undergo structural damage but should not collapse as a result of the ground shaking.

The 2005 NBCC seismic design procedures are based on ground motion parameters (e.g. peak ground acceleration, (PGA) and spectral acceleration, Sa values) having a 2% probability of excedance in 50 years; ie. the 2,475 year return period earthquake event.

Based on the results of the E2K field investigation, it is appropriate to classify the ground conditions at the subject site as a Class C site, in accordance with the 2005 NBCC.

5.12 Preliminary Pavement Sections

Subgrade preparation for paved driveways and roads should be carried out as recommended in Section 5.1. The pavement design recommendations for residential roads are provided based on the assumption that the traffic conditions will consist primarily of cars and light trucks.

The completed subgrade for pavement areas should be proof rolled to confirm that the surface deflections are minimal under the influence of construction traffic and to verify that an acceptable degree of compaction has been obtained. Any weak subgrade soils should be removed and replaced with engineered fill. Proof-rolling should be performed by 2 passes of a single axle, dual wheel truck with an 80 kN axle load. Additional details for placement of fill soils can be provided at the time of the detailed design and construction. The degree of compaction in the upper 150 mm of the engineered backfill beneath pavement should be increased to 100% of SPMDD.

Pavement design for the subject residential roads should be performed according to City of Calgary "Roads Construction – 2015 Standard Specifications". The required minimum pavement section is provided below. This pavement section is based on an adequately prepared subgrade, an assumed California Bearing Ratio (CBR) of 3, maximum axle loads of 80 kN (18 kips), and traffic loading conditions consisting of $5x10^4$ repetitions of an Equivalent Single Axle Load (ESAL).

Minimum Pavement Section (Residential Roads)	
Pavement Component	**Minimum Thickness (mm)**
Minimum Asphalt Concrete Thickness	80
Minimum Granular Base Thickness	100
Minimum Sub-base Thickness	200

Pavement materials should be provided and constructed in accordance with City of Calgary road construction specifications. Surface runoff should not be allowed to accumulate on or adjacent to the proposed roadway alignment.

6.0 SLOPE STABILITY ANALYSIS

Based on our drilling program the site's soils consisted of clay till with a firm to hard consistency. During drilling, saturated conditions were observed in the boreholes on Site A. The overall drainage characteristics of the sites were good, directing surface water towards the manmade pond in Site A. However, this will be removed as part of redevelopment. The majority of the landscaped area across Sites 'A' and 'B' consist of moderate slopes. From site reconnaissance and visual inspection, there were no signs of unstable slopes observed within the subject sites. The stability analyses and the results shown in this report comply with all the requirements of The City of Calgary guidelines for slope stability.

6.1 Slope Assessments

Eight slope sections were selected as worst case representative sections for the slope stability assessment from Site A and three from Site B, as shown in Figure 3. Finished slopes in other areas of the sites are relatively flat and should not experience any slope stability issues.

Limit-Equilibrium Analyses were performed using the representative sections and worst case groundwater elevation within Sites A and B, as shown on (Figures 4-7). The soil parameters used in the analysis are summarized in the following table.

Soil Parameters		
Soil Type	**Unit Weight (kN/m^3)**	**Friction Angle (^{o})**
Fill	19	25
Clay Till	20	29
Sandstone	21	32

The representative sections were analyzed using the slope stability analysis software "SLOPE/W" with the encountered soil conditions and assumed worst-case groundwater elevations. On March 18, 2016 the groundwater level was measured to be 0.9 m below grade at BH-04 in Site A. This was not selected as a worst case due to its localized nature and relatively flat surrounding grades. The worst case groundwater elevation in areas where groundwater was encountered during this monitoring period were taken as the seasonally adjusted water level. In areas where no groundwater was encountered, a Ru value of 0.1 was used to depict surface saturation in the slope model.

6.2 Results

Slope stability analyses were performed at twelve critical slopes within the subject sites. A summary of the factor of safety are provided below:

Slope Stability Analysis - Site A		
Sections	**Required Minimum Factor of Safety**	**Calculated Factor of Safety**
A-1	1.5	1.83
A-2	1.5	1.59
A-3	1.5	2.30
A-4	1.5	2.39
A-5	1.5	1.80
A-6	1.5	1.73
A-7	1.5	1.98
A-8 Lower	1.5	1.98
A-8 Upper	1.5	2.01

Slope Stability Analysis - Site B		
Sections	**Required Minimum Factor of Safety**	**Calculated Factor of Safety**
B-1	1.5	1.57
B-2	1.5	2.68
B-3	1.5	1.77

Based on the results of our analyses, Section B-1 within Site B was observed to have the lowest factor of safety of 1.57, which is higher than that required by the City of Calgary. Based on the slope stability analyses completed, existing grades within the subject site are stable against instability. It should be noted that if significant grading is required for development, new slopes will need to be analyzed at that time.

In their current configuration, slopes on the subject sites do not require setback lines or limits to development area. Upon establishment of a rough grading plan, E2K can re-assess the proposed slopes to confirm that the minimum global stability safety factor of 1.5 will be maintained. To maintain a safety factor in excess of 1.5, the proposed development must promote proper surface drainage. In general, development on the subject sites will be located near the bottom or below the existing slopes. Provided that slopes are not steepened or the toe of a slope is not cut out, a global stability factor of safety in excess of 1.5 is expected to be maintained.

7.0 LIMITATIONS

Recommendations made within this report are based on the interpreted findings encountered in twelve (12) boreholes. It should be noted that natural conditions are innately variable. Should conditions other than those reported herein, be identified at any stage of development, E2K should be notified and given the opportunity to re-evaluate current information, if required.

The recommendations presented herein, are subject to an adequate level of inspection during construction. Levels of inspection and material testing set out by the City of Calgary "Field Services Guidelines, 6th Edition" must be adhered to. This report has been prepared with accepted soil and foundation engineering practices for the project specified in Section 1.0 of this report. No other warranty is expressed or implied.

8.0 CLOSURE

We trust the information contained herein meets your present requirements. Should you require inspection services, or further information regarding the geotechnical aspects of this project, please do not hesitate to contact our office.

Yours truly,
E2K Engineering Ltd.
APEGA Permit to Practice: P9582

Prepared By:	Reviewed By:
Muhammad A. Shad, P.Eng.	Josh Clark, P. Eng.
Intermediate Geotechnical Engineer	Intermediate Geotechnical Engineer

Attachments: Site Plans
Slope Stability Analysis
Borehole Logs
Explanation of Terms and Symbols

Figure 1
Site Plan
Borehole Locations

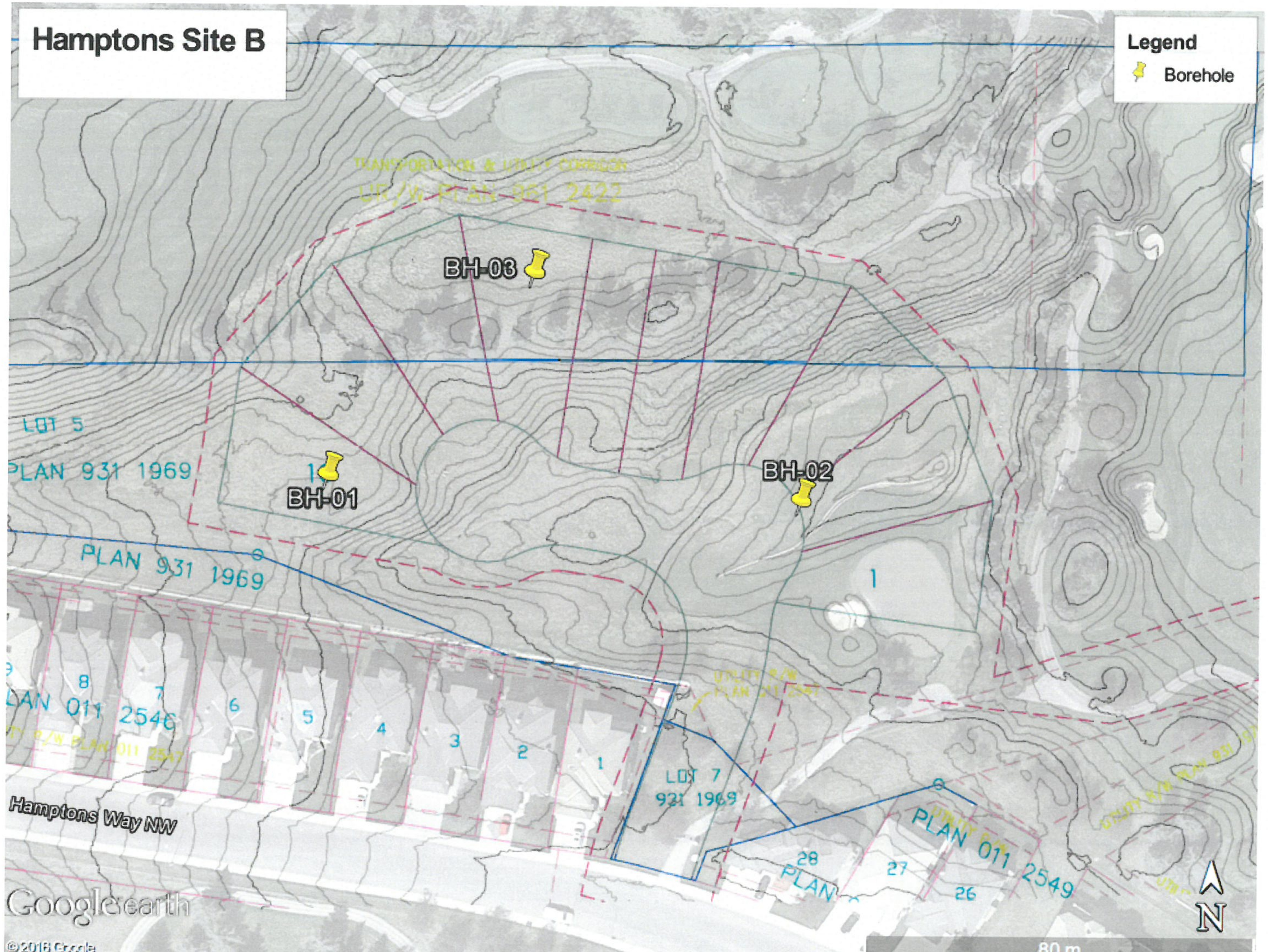
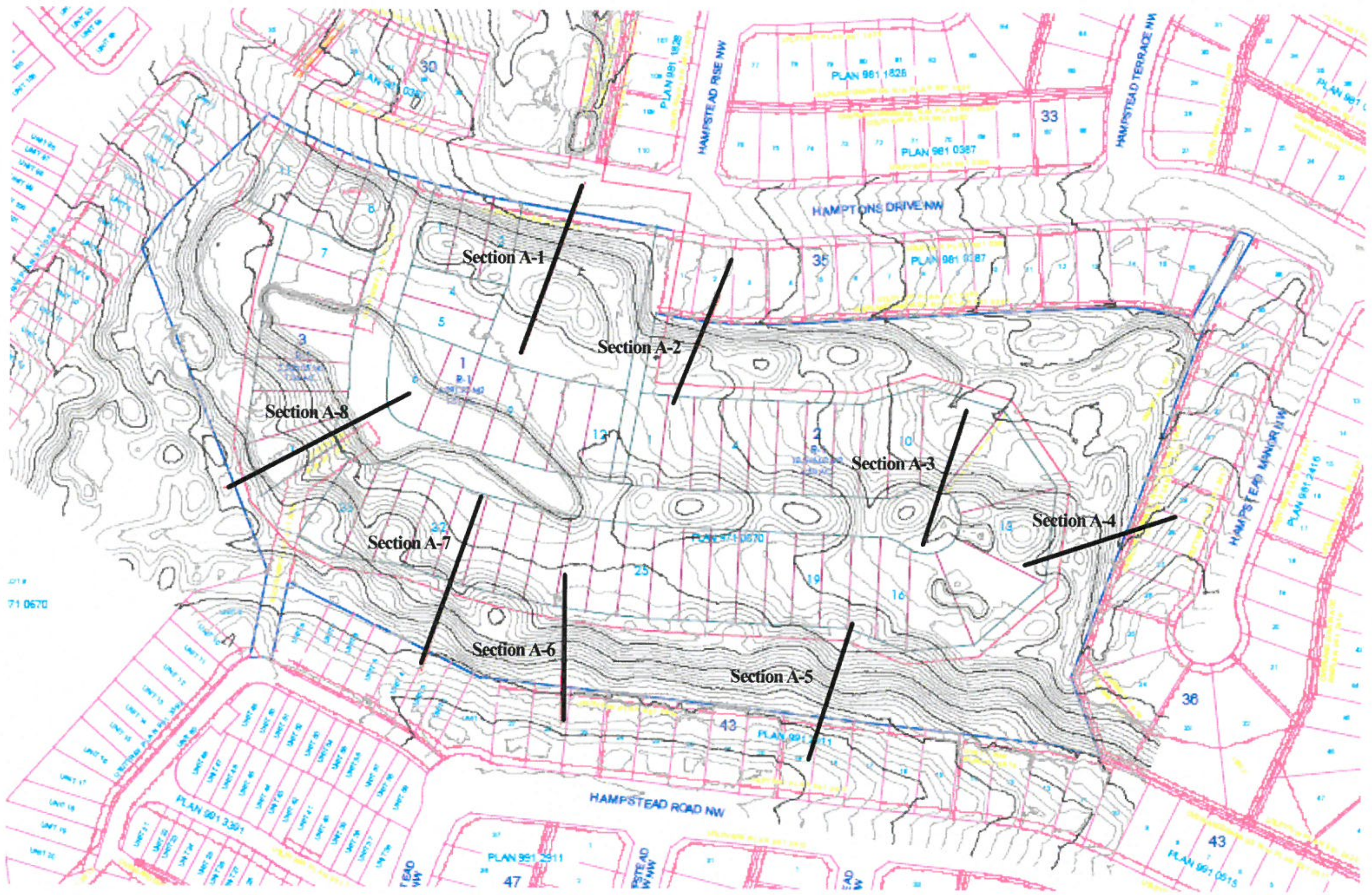


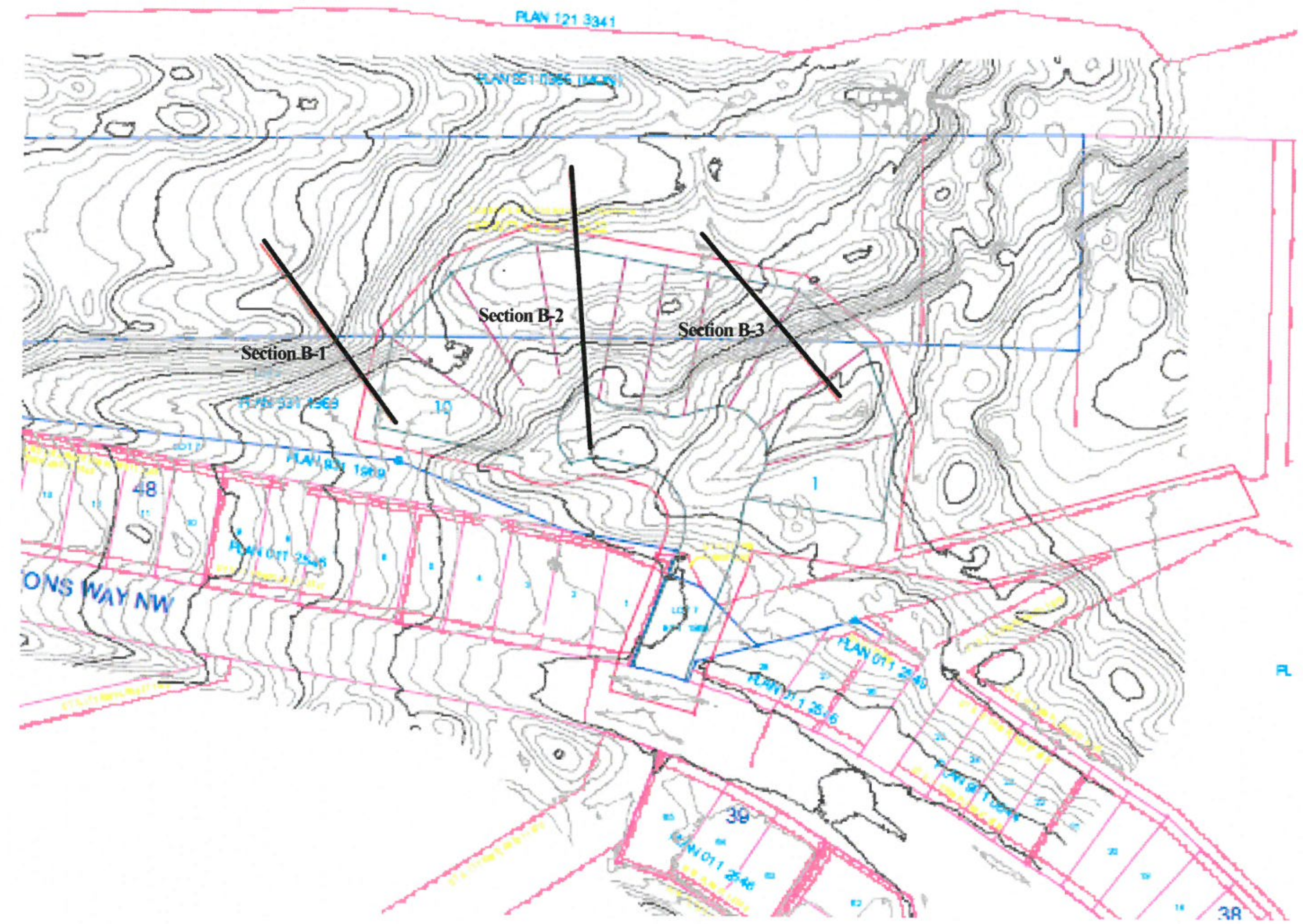
Figure 2
Site Plan
Borehole Locations

Slope Stability Assessment
Hamptons Golf Course Redevelopment
Calgary, Alberta

Project: 2016-2179



Site - A



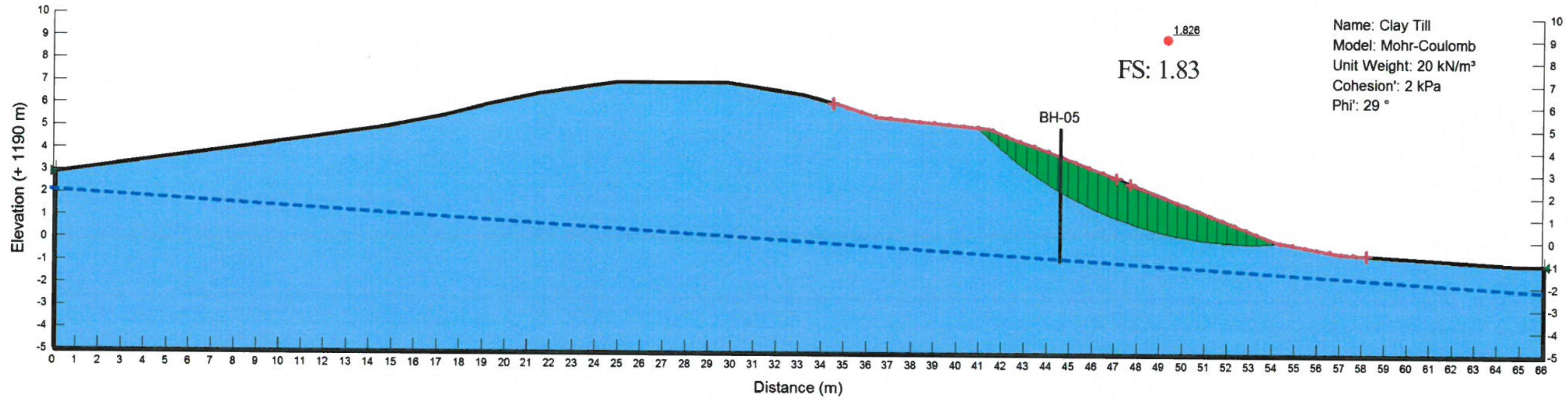
Site - B

Figure 3

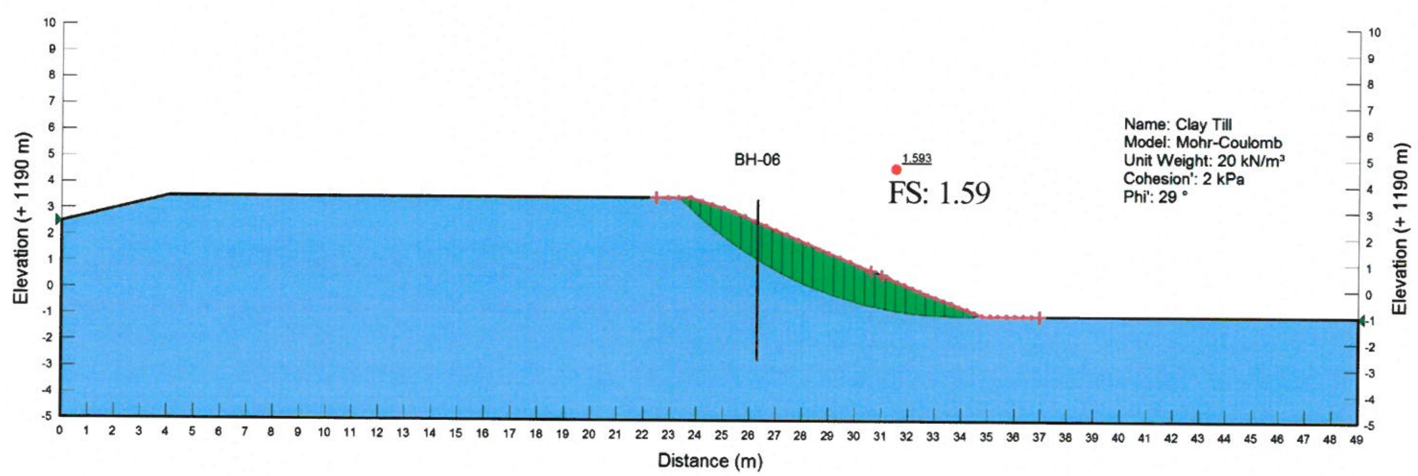
Slope Sections

Slope Stability Assessment
Hamptons Golf Course Redevelopment
Calgary, Alberta

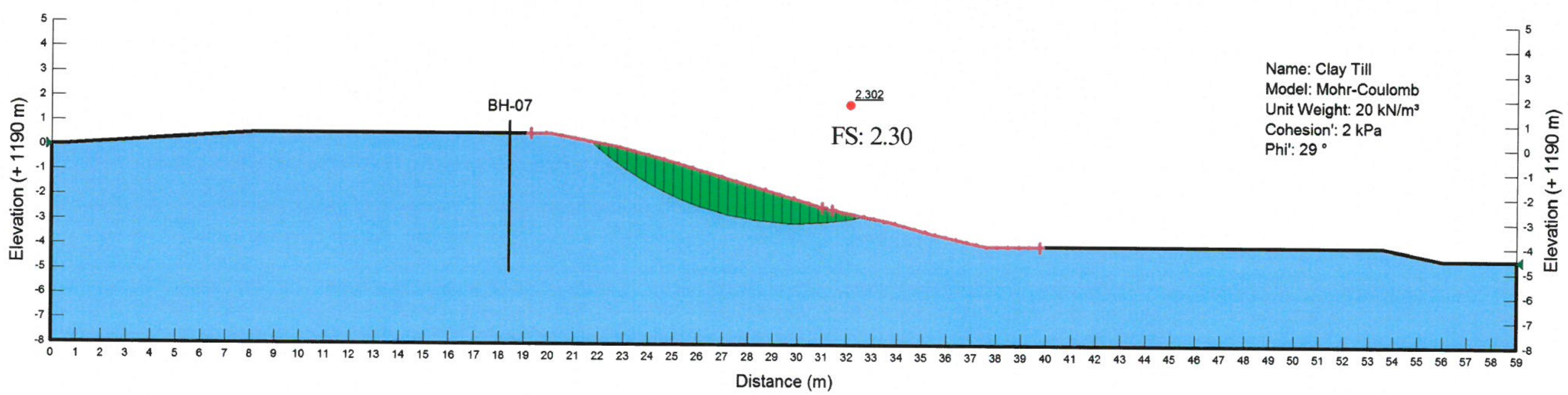
Project: 2016-2179



Section A-1



Section A-2

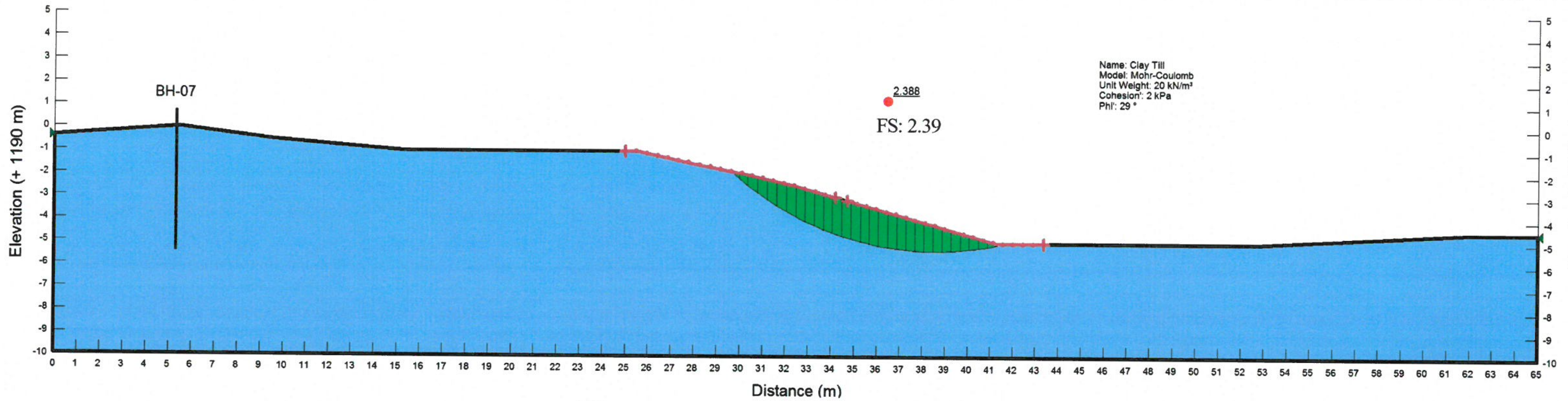


Section A-3

Figure 4
Slope Stability Sections

Slope Stability Assessment
Hamptons Golf Course Redevelopment
Calgary, Alberta

Project: 2016-2179



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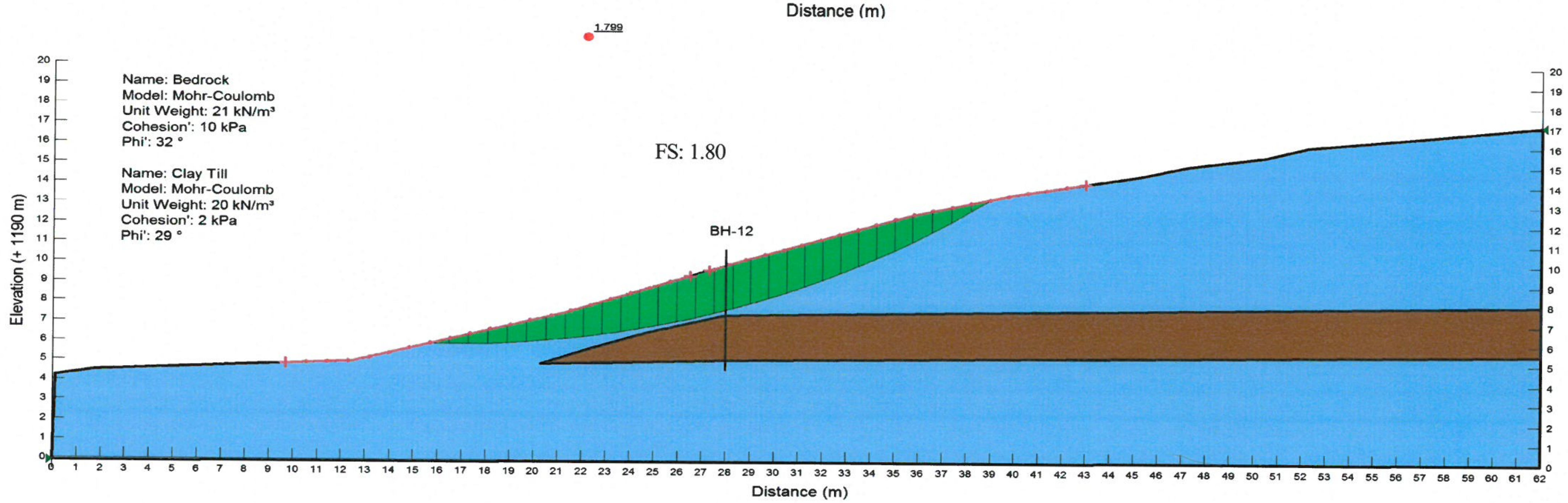
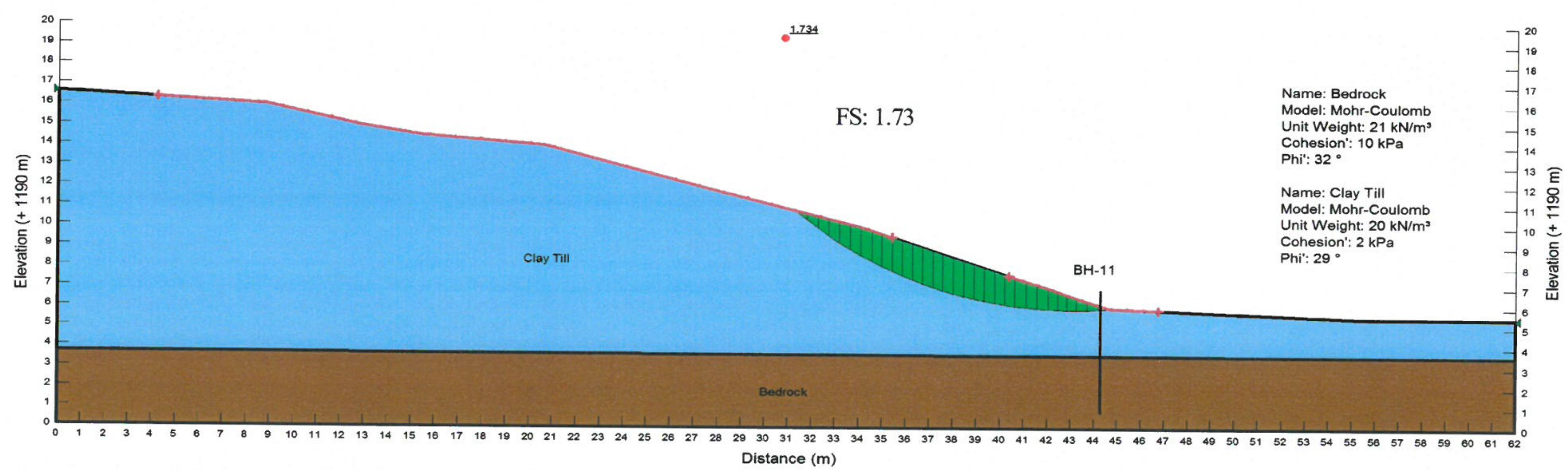


Section A-5



Section A-6

Figure 5

Slope Stability Sections

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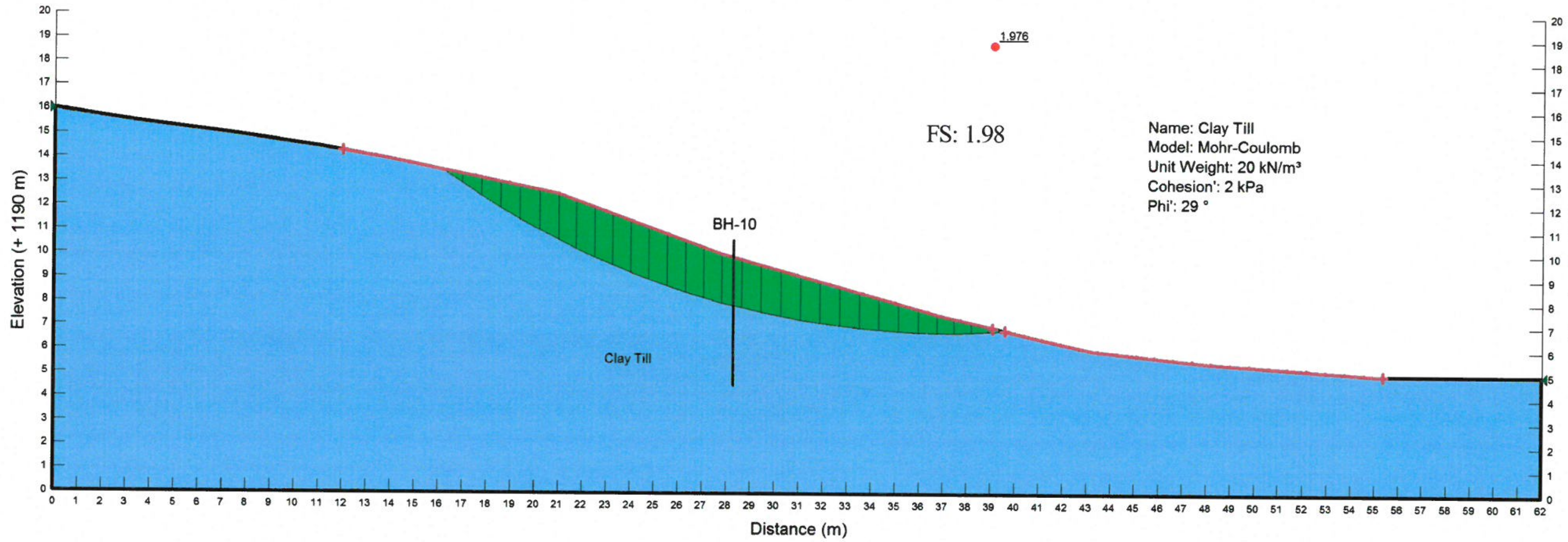
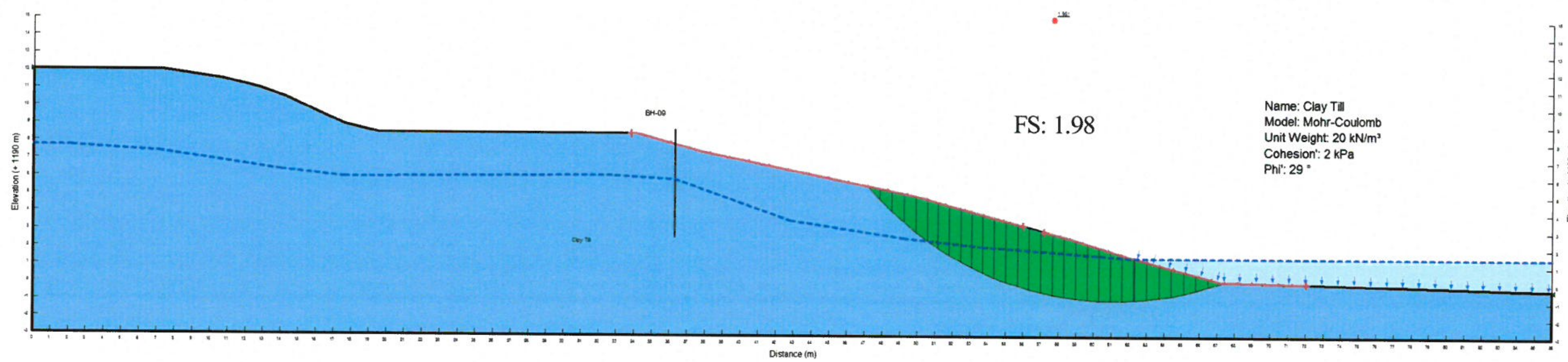
Slope Stability Assessment
Hamptons Golf Course Redevelopment
Calgary, Alberta

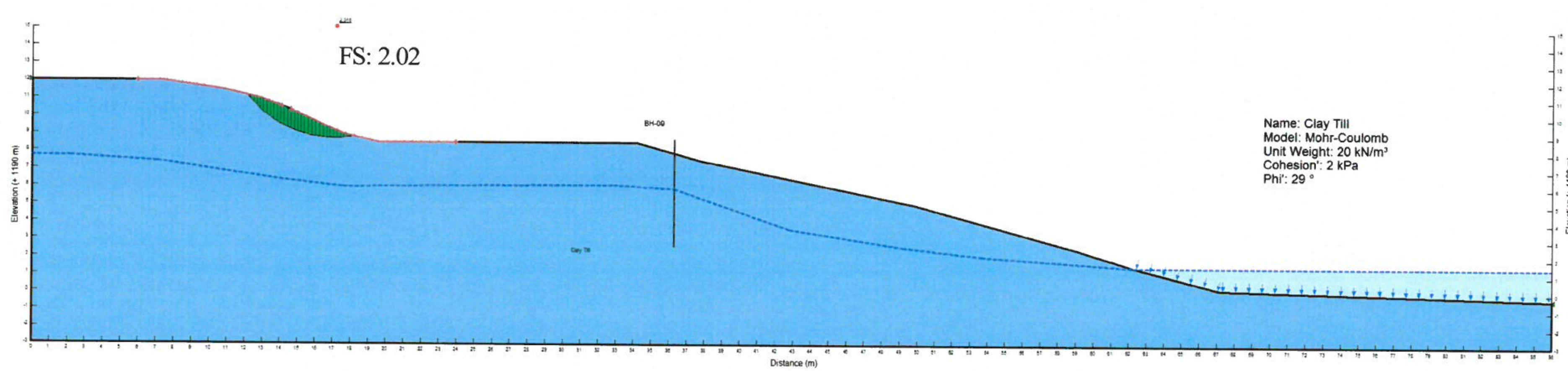
Project: 2016-2179



Section A-7

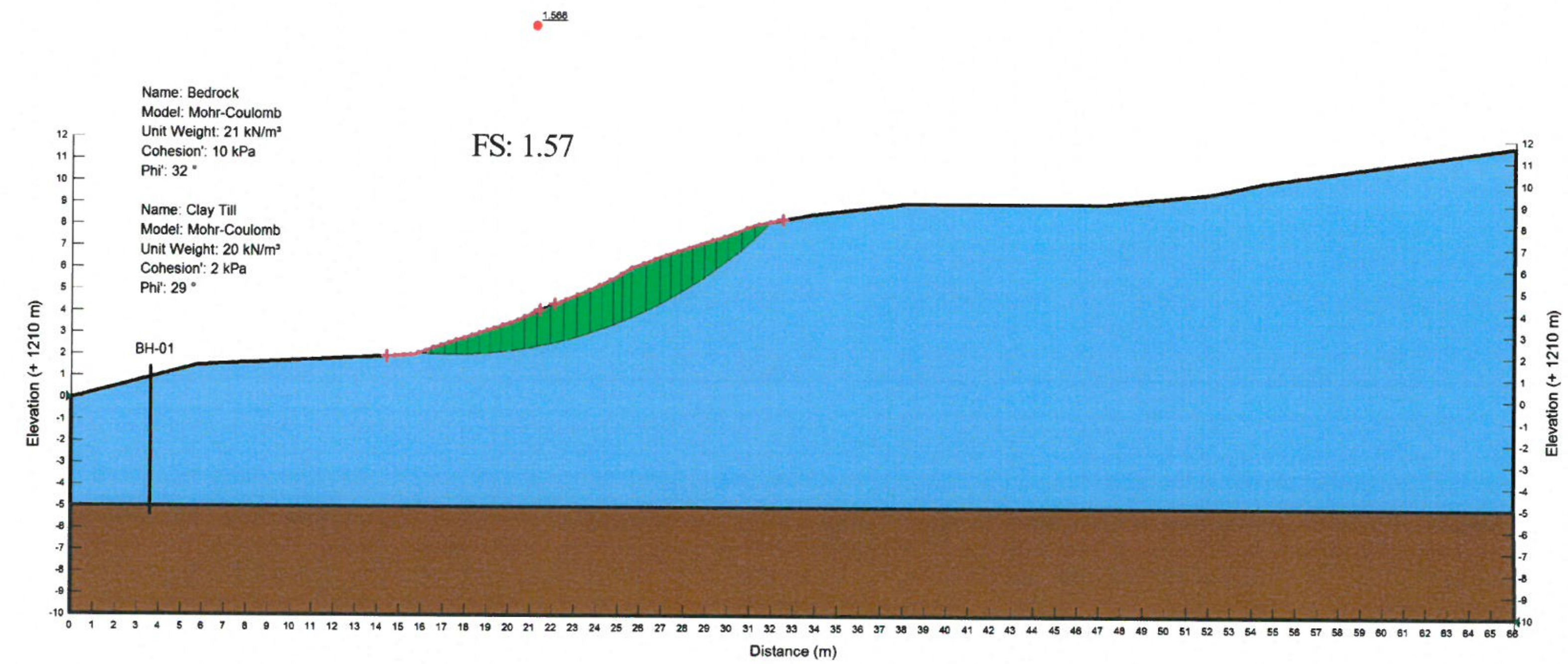


Lower Section A-8

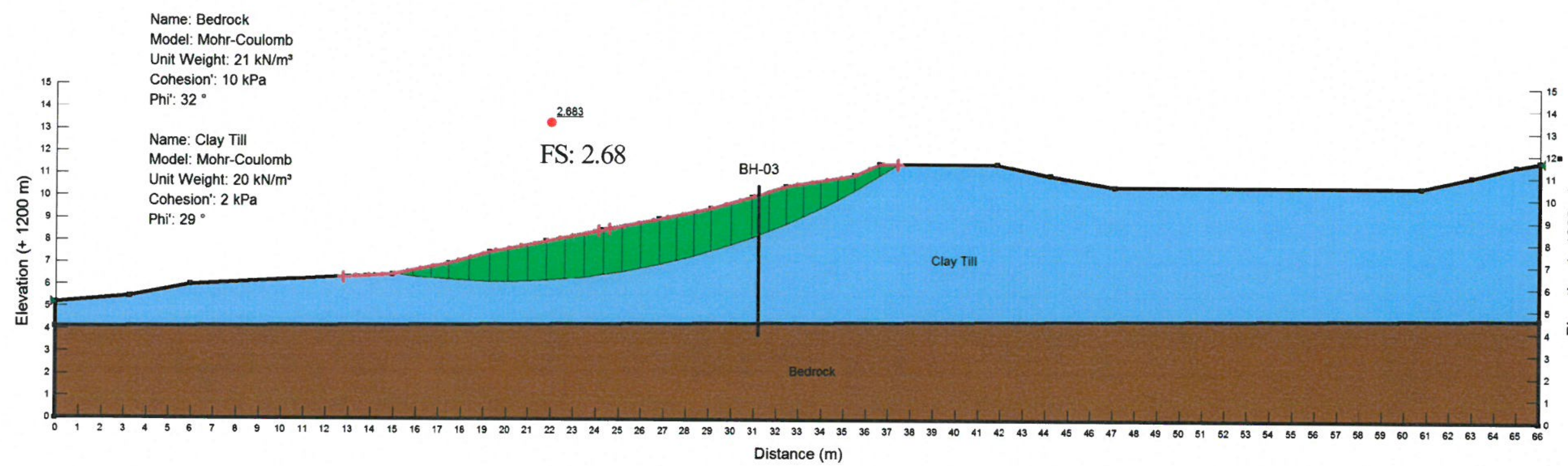


Upper Section A-8

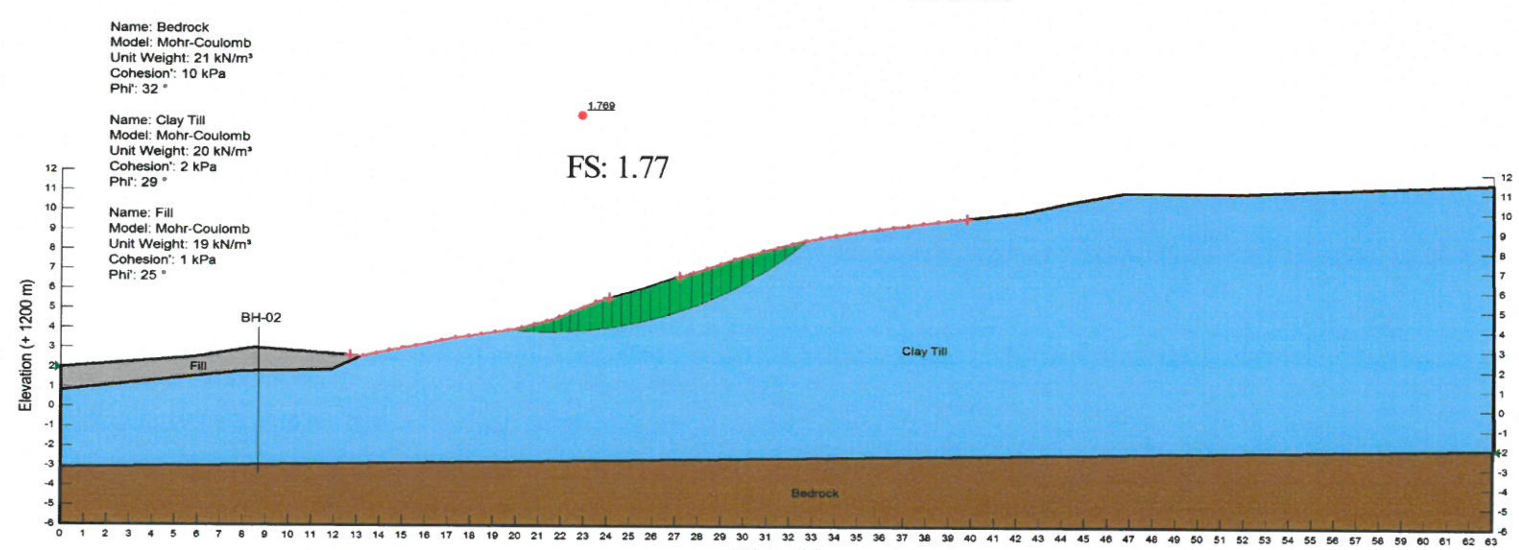
Figure 6
Slope Stability Sections



Section B-1



Section B-2



Section B-3

Figure 7

Slope Stability Sections

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