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COVER LETTER

Project No.:	WF54331	**Date:**	July 24, 2026
To:	Appin Developments Inc.	**T.**	403-831-73355
Company:		**E.**	Bruce bmac@yahoo.ca
From:	Jason Thai, P.Eng.	**T.**	587-557-4545
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Project:	Carstairs Subdivision – NE ¼ 24-29-01 W5M		
Subject:	Sealed Geotechnical Report Cover Letter		
Prepared by: JASON THAI ID 91533 2026-07-24 Jason Thai, P.Eng.		PERMIT TO PRACTICE METRO TESTING & ENGINEERING LTD. RM SIGNATURE RM APEGA ID # 95290 DATE 2026-07-24 PERMIT NUMBER: P12584 The Association of Professional Engineers and Geoscientists of Alberta (APEGA)	
Geotechnical Engineer Geotechnical Division			

1.0 Introduction

Metro Testing & Engineering Ltd. (Metro) has been retained by Bruce McIntosh to provide a cover letter regarding the attached geotechnical report (AE37269) for the Carstairs Subdivision at NE ¼ 24-29-01 W5M dated May 25, 2021.

Metro attests that the aforementioned geotechnical report was signed and sealed according to the best practices at the time.

2.0 Closure

We appreciate the opportunity to be of service to you and trust that this cover letter meets your current requirements. If you have any questions regarding the contents of this letter, or if we can be of further assistance to you on this project, please do not hesitate to contact us.

Sincerely,

Metro Testing & Engineering Ltd.

This letter has been prepared for the exclusive use of the Client, their consultants, and representatives for the specific application of the development described within this memorandum. Any use of this letter by third parties, or any reliance on or decisions made based on it are the responsibility of such third parties. Metro accepts no responsibility for loss or liability, if any, suffered by any third party as a result of decisions made or actions taken based on this letter.

JT

File: U:\c. WG1 & WF Abbotsford\WF54000 - WF54999\WF54331 Carstairs Subdivision Cover Letter - Appin Consruction\5. Schedules, Reports, Letter, Memos & Field Reviews\WF54331 2026-07-24 Cover Letter R0 JT.docx

May 25, 2021
Project No.: AE37269

Appin Developments Inc.
203 – 16 Midlake Boulevard SE
Calgary, Alberta
Canada T2X 2X7

Attn: Mr. Bruce McIntosh
Email: bruce_bmac@yahoo.ca
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GEOTECHNICAL ASSESSMENT

Carstairs Subdivision
NE ¼ 24-29-01 W5M
Mountain View County, Alberta

Submitted By:
Metro Testing + Engineering Ltd.
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Rev	Description	Date	By	Check	Approve
A	Issued for Internal Review	2021-05-17	LCK	BLJM	
B	Issued as Final	2021-05-25	LCK	BLJM	BLJM

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APPENDICES

1.0 INTRODUCTION

Metro Testing + Engineering Ltd. (Metro) presents our geotechnical engineering recommendations for the proposed development located entirely within the NE ¼ 24-29-01 W5M approximately 10 km southeast of the Town of Carstairs, Alberta. Figure 1 in Appendix A shows the approximate location of the site.

Alberta Materials Testing and Engineering Ltd. (AMTE) previously provided a report submitted to Mr. Robert Weston of ERW Consulting Inc., titled *Geotechnical Assessment Carstairs Subdivision NE SEC 24-29-1 W5M, Mountain View County, AB* dated April 19, 2019. In addition, a supplemental drilling program was undertaken to determine the extent of potential gravel resources located west of the property located along the north boundary of the proposed development. The results of the supplemental geotechnical program are summarized in the memorandum titled *Memorandum – Geotechnical Investigation Gravel Pit Locations, NE 24-29-01 W5M, Mountain View County, AB* dated May 1, 2019. The information provided in these reports will supplement the information provided herein.

Use of this report is subject to conditions outlined in Important Information and Limitations of This Report which follows the main text and forms an integral part of this document.

2.0 SCOPE OF WORK

The proposed scope of work was to provide updated geotechnical recommendations and supply of a report based on the previous geotechnical work conducted in 2019. An additional field drilling program was recommended to confirm existing site soil and groundwater conditions primarily adjacent to the steep slopes within the proposed development. The current field drilling program consisted of advancing four (4) boreholes drilled to depths varying between approximately 10 m and 15 m below existing grade. The purpose of the supplemental geotechnical study was to confirm and assess the subsoil and groundwater conditions so that necessary engineering properties of the subject site subsurface soil conditions could be determined. This report summarizes the results of field and laboratory testing programs and presents preliminary geotechnical recommendations for the design and construction of the proposed development. It should be noted that archaeological and environmental considerations are not part of the current scope of work.

The recommendations provided in this report are intended for the guidance of the design engineer. Where comments are made on the general site conditions and construction, they are provided to highlight aspects that could affect the design of the project. Those requiring information on geotechnical aspects of the site beyond the scope of this report must make their own interpretation of the subsurface information, particularly as it affects their proposed construction, methods, costs, equipment selection, scheduling and the like.

3.0 METHODOLOGY

3.1 Geological Setting

Part of the Western Alberta Plain, the project area mostly comprises gently to moderately sloping, locally undulating to hummocky topography, with slopes typically in the 5 to 10 degree range. According to Hamilton *et al*., nonmarine bedrock strata of the Paleocene era, Paskapoo Formation underlie the area of interest. Sandstone, mudstone, thin limestone, coal and tuff beds are the dominant lithologies.

The surficial geology of the project area has been mapped by Shetson. Till, a heterogeneous mixture of clay to cobble and boulder sized material, is the dominant deposit of glacial origin but locally includes sand, silt, clay or gravel of lacustrine origin. Ground moraine and till plains dominate, with lesser amounts of large, flat areas of former proglacial lakes, which are largely peat-covered. The topography of the area is described as level to undulating.

The soils encountered during the geotechnical field program were generally consistent with published information.

3.2 Topography and Drainage

The proposed development is located entirely within the NE ¼ 24-29-01 W5M approximately 10 km southeast of the Town of Carstairs, Alberta. The proposed development covers an overall area that is roughly 160 acres. A review of available information and aerial imagery, shows that the project site would be described as undeveloped pasture. The north central portion of the site was used as an aggregate source prior to 1985.

The regional topography comprises level to undulating terrain. However, local moderately to steep slopes, with slopes in excess of 30 to 40 degree range, are present throughout the central portion of the development. Slopes within the development area generally range between 5 and 10 degree, with local relief between 10 m and 15 m.

The proposed development is bisected by an unnamed creek channel providing drainage to the northwest. The drainage of the soil deposits is variable and directly related to the topographic position and texture of the material. In general, the deposits encountered during the current geotechnical field program tend to be poorly drained. Small hilltops and sloping terrain tend to be moderately well drained, with the exception of seepage areas characterized by imperfect to poor drainage. Flat, low-lying terrain and depressional areas tend to be imperfectly to poorly drained.

3.3 Field Program

Alberta Materials Testing and Engineering Ltd. previously conducted a field drilling program in 2019. This program included drilling twenty three (23) boreholes throughput the development boundaries. The locations of the boreholes were governed by Mountain View County required grid of approximately

150 m by 150 m. The boreholes were advanced to a maximum depth of 6.6 m below existing ground surface (mbeg). Due to the limited size of the drilling equipment used, the majority of previous boreholes were advanced to 2.0 mbeg before practical refusal was encountered.

More recently, four (4) boreholes, designated as BH21-01 thru BH21-04 were drilled within the proposed development boundaries between April 12 and 13, 2021. Approximate borehole locations are shown on the Borehole Location Plan provided in Appendix A. The borehole locations were selected by Metro personnel. Location and identification of buried utilities at the site was coordinated and overseen by Metro personnel. Underground utility locates were retained through an Alberta One Call submission prior to mobilization to site. Underground utility locations in the area were confirmed prior to commencement of the geotechnical drilling program. Final borehole locations were selected based on known utility locations and to accommodate drill rig access.

The boreholes were drilled using a truck mounted drill rig capable of rock coring techniques in addition to continuous flight, solid and hollow stem augers. The drilling equipment was operated by Mobile Augers & Research Ltd. of Calgary, Alberta. Borehole Logs are provided in Appendix B. The boreholes were advanced to depths varying between 12.3 m and 15.3 m below existing ground surface. The boreholes were logged and samples classified in accordance with the Modified Unified Soil Classification System described in Appendix C. Pocket penetrometer and Standard Penetration Tests (SPTs) were carried out at selected intervals. Disturbed soil and rock samples were returned to Metro's office for further detailed examination, classification and laboratory testing. Upon completion, all boreholes were backfilled with drill cuttings supplemented with bentonite chips.

Surface elevations at the borehole locations were determined utilizing a handheld GPS unit. Surface coordinates (i.e. Northings and Eastings) were referenced to geodetic datum (NAD 83) as established using a handheld GPS unit. Approximate borehole locations, as drilled, are shown on the drawings provided in Appendix A. UTM coordinates and geodetic elevations of the boreholes are provided below in Table 1. It should be noted that the accuracy of the handheld GPS unit is approximately ±10 m.

TABLE 1: UTM (NAD 83 ZONE 11U) COORDINATES AND ELEVATIONS OF BOREHOLES

Borehole	Northing	Easting	Approx. Ground Elevation (m)
BH21-01	5709956	707366	1023.7
BH21-02	5709853	707481	1025.5
BH21-03	5709341	707700	1032.2
BH21-04	5709545	707863	1027.0

Select site photographs taken during the field drilling program are provided in Appendix F.

3.4 Laboratory Program

A laboratory testing program meeting applicable ASTM and/or CSA standards was undertaken on selected samples secured in the field. The laboratory testing consisted of the following:

- Determination of the natural moisture content;
- Atterberg limits;
- Unconfined Compressive tests; and
- Unit weight determination.

The results of the laboratory program are provided in Appendix D and presented graphically on the borehole logs. All soil and rock samples will be stored for 60 days following issuance of this report. The samples will then be discarded, unless Metro is instructed otherwise.

4.0 SUBSURFACE CONDITIONS

4.1 Soil Conditions

The general soil profile within the proposed development area consisted of surficial topsoil overlying silty clay which is in turn underlain by bedrock. The soil profile at the majority of the borehole locations was dominated by bedrock deposits. The presence of silt and sand lenses within the silty clay soils were highly variable and no discernable pattern was observed in the distribution of these deposits. The following is a general description of the soil units encountered. Detailed descriptions of the soil strata encountered are provided on the borehole logs in Appendix B.

The soil descriptions provided in this report are based on accepted standard methods of classification and identification routinely used in current geotechnical practice. It should be noted that the transitions between the classified soil units are gradual, rather than the distinct unit boundaries as shown on the borehole logs.

4.1.1 Topsoil

Within the project boundaries, all boreholes advanced encountered topsoil. This material is weak and highly compressible. Subjected to loading, these soils will display prolonged secondary consolidation effects. The topsoil was described as fibrous, dark brown to black and typically 400 mm to 600 mm thick. It is possible that there may be localized areas where the thickness of organic deposits is greater than those observed at the borehole locations and may be more prevalent than implied by the borehole logs.

4.1.2 Clay Till

The predominant soil encountered was silty clay till and was noted in all boreholes advanced during the current geotechnical program. It should be noted that clay till is a heterogeneous mixture of soil particles from fine grained to boulder sized material. Coarser grained material may be more prevalent within the till at the site than implied by the borehole logs. Oxides, coal fragments and numerous sand partings were also noted. Where encountered, this deposit was noted below the topsoil and extended to a maximum depth of 3.8 m, where drilling was terminated.

The till was typically described as olive to olive in colour, and in a moist condition. Standard Penetration Test “N” values in the till ranged between 12 and 17, averaging 42, indicating a stiff to very stiff consistency; the clay till increased consistency with increasing depth. Fine grained sand and silt layers were noted within this soil group. Atterberg Limit Index Property tests performed on these soils indicated Liquid Limits averaging 37 and Plastic Limits averaging 17, which result in an average Plasticity Index 20. The tests classify the soil as medium plastic clay (CI). The unit weight of the clay till generally increased with depth and decreased moisture content.

4.1.3 Bedrock

The bedrock encountered during the current geotechnical program was comprised of a series of interbedded layers of shale, mudstone and sandstone which is typical of the Paskapoo Formation. Generally, this deposit was encountered underlying the clay till deposits and extended beyond the depth of exploration. The layered bedrock formation consisted of “dense” (completely weathered) and “very weak to medium strong” materials. The bedrock has been shown to be extremely variable in its physical properties, changing from a soil like consistency (reworked and highly weathered) to “very weak” as evidenced by the SPT “N” values, which ranged from 31 to 88. Using rock coring methods, the drill equipment was capable of penetrating a maximum of 12.9 m into the bedrock in Borehole BH21-02. Other than Borehole BH21-01, bedrock was encountered in all boreholes and drilling was terminated after confirmation of bedrock stratigraphy.

According to the International Society for Rock Mechanics (ISRM), the estimated range of compressive strengths for the “dense (completely weathered)” to “very weak” bedrock materials are typically less than 1 to 5 MPa. More indurated and “weak” layers may be encountered which would be expected to exhibit compressive strengths of 10 MPa or greater. Rock core compressive strengths were determined at various depths and are summarized in the following table.

TABLE 2: SUMMARY OF ROCK CORE COMPRESSIVE STRENGTHS

Borehole	Depth (mbeg)	Sample Description	Compressive Strength
BH21-02	11.3	Medium Strong Sandstone	36.3 MPa
BH21-02	13.1	Medium Strong Sandstone	26.5 MPa
BH21-03	8.5	Very Weak Mudstone	3.6 MPa
BH21-03	11.9	Extremely Weak Shale (bentonitic)	357 kPa
BH21-04	5.8	Extremely Weak Mudstone	401 kPa
BH21-04	9.0	Strong Sandstone	64.8 MPa
BH21-04	11.9	Extremely Weak Mudstone	486 kPa

A review of information available on the A*lberta Groundwater Wells* website indicates that there are existing groundwater supply wells located within approximately 1 km of the proposed development. The information from the water supply well logs and our general knowledge of the area indicates that the bedrock deposits in the proposed development area extends to depths in excess of 30 m below the existing ground surface.

4.2 Groundwater Conditions

Monitoring of the groundwater conditions was conducted during the current drilling operations. After completion of drilling, all boreholes were dry with no observed groundwater seepage. A review of the information provided in the previous geotechnical studies performed in 2019 indicates that all boreholes were dry at the end of drilling operations. Groundwater conditions are provided on the borehole logs in Appendix B. It should be noted, that the clay contains numerous saturated sand lenses which may result in perched groundwater conditions at the site. Additionally, groundwater levels fluctuate seasonally and in response to climatic conditions.

5.0 DISCUSSIONS & RECOMMENDATIONS

5.1 General

The site is considered suitable for the proposed commercial/industrial development.

The following sections of this report provide the interpretation of the geotechnical data obtained during the current geotechnical assessment and information obtained during the geotechnical assessment performed in 2019. The recommendations provided in this report are intended for the guidance of the design engineer. Where comments are made on the general site conditions and construction, they are provided to highlight aspects that could affect the design of the project. Those requiring information on the geotechnical aspects of the site beyond the scope of this report must make their own interpretation of the subsurface information, particularly as it affects their proposed construction, methods, costs, equipment selection, scheduling, and the like.

5.2 General Site Grading

Where the subgrade will support roadways, all organic material and uncontrolled fill, if encountered, should be completely removed down to the native mineral soils. Care should be taken to minimize disturbance to the subgrade during stripping of unsuitable materials. Any soft or loose areas identified during subgrade preparation should be removed and replaced with suitable subgrade fill, as approved by the geotechnical consultant.

In general, where clay soils are to be used as general engineered fill, moisture conditioning may be required. Extensive fill placement required for general site grading should not be performed during freezing conditions or using frozen soils. The native inorganic soil encountered in the boreholes would likely be suitable material for use as general engineered fill. All engineered fill material should be free of any organic and other deleterious material, and should be tested and approved prior to use. General engineered fill should be compacted to a minimum of 98% Standard Proctor Maximum Dry Density (SPMDD) as per ASTM D698 at a moulding moisture of optimum to 2% above optimum moisture content (OMC) for cohesive soils and ±2% OMC for cohesionless soils. All engineered fill should be placed in lift thicknesses compatible with the compaction equipment being used, to a maximum compacted lift thickness of 300 mm and 150 mm for granular and cohesive fill soils, respectively. Where satisfactory subgrade conditions exist, the subgrade should be scarified to a minimum depth of 300 mm and re-compacted to a minimum of 98% SPMDD, prior to placing additional fill. The use of alternative fill placement methods should be reviewed by the geotechnical engineer prior to construction activities.

The exposed subgrade should be reviewed under the supervision of a qualified geotechnical engineer, prior to placement of granular fill to identify any soft, loose or non-uniform areas. Recommendations for mitigation of any soft and/or wet areas will be provided upon review of the subgrade. The method of remediation will be dependent upon the size, severity, and composition of the unsuitable material, and will be specified on-site by the geotechnical engineer responsible for the review. Potential methods of remediation include the use of geosynthetics, scarification and re-compaction (typically upper 300 mm of subgrade), or removal and replacement. Any organic or other deleterious material removed during subgrade reviews should be replaced with engineered fill placed and compacted as noted above. For planning and costing purposes, it would be prudent to assume some form of subgrade enhancement will be required in localized areas on site.

Final subgrade grades should direct surface water to areas away from proposed structures and promote rapid drainage of surface runoff. Surface gradients of at least 2% are recommended to reduce the potential for ponding in localized areas and potential for saturation and degradation of the subgrade.

5.3 Temporary Excavation Stability

Excavations are expected to be required for foundation, utility installations, road reconstruction. Based on the soil encountered during the field investigation program, the excavations will be within the native clay and /or silt soils and bedrock. It should be noted that the design of the temporary excavation and shoring is typically the responsibly of the contractor. The following outlines the general guidelines which should be used for preliminary planning and should be confirmed by a geotechnical engineer during construction.

Temporary (i.e. less than one week) excavated faces in the clay materials, above groundwater level, should be cut no steeper than 2 Horizontal to 1 Vertical (2H:1V). Excavations in the bedrock materials may be cut 1H:1V. Sloughing should be expected and periodic cleaning of debris at the base of the excavated slope may be required. Care will be required to avoid sloughing and failure of the sidewalls. The actual slope inclination should be evaluated during construction and factors such as groundwater, soil layering, and weather conditions should be considered. A qualified geotechnical engineer should also assess cut slopes inclination in areas where soil is saturated or groundwater seepage is encountered at the time of construction.

Excavation planning should be reviewed by qualified geotechnical personnel if specific situations of the planned excavations require adjustments to the cut slope angles (i.e. steeper) from the general recommendations provided herein. Excavations more than 1.2 m in depth should be reviewed prior to construction in accordance with Alberta Occupational Health and Safety Act. Temporary excavations should be assessed and reviewed during construction to confirm that the soil and groundwater conditions are as discussed herein. If unexpected conditions are encountered, Metro should be allowed to review the excavation plan considering the soil conditions at the time. Should deeper excavations be required and/or temporary excavation slopes need to be steeper due to site constraints, temporary shoring may be required. The latest edition of the Alberta Occupational Health and Safety Act should be followed for all excavations.

Based on the results of the subsurface investigation, groundwater is not expected to be a significant concern with respect to temporary excavations. If groundwater is encountered, flows will likely be manageable with conventional trenching and sump pumps in isolated areas. Collected water from excavation pumping should be discharged a sufficient distance away from the excavation to minimize the potential for water to return to the excavated area. Water pumped from the trench will likely contain silt-sized particles; therefore, direct discharge into natural drainage courses should be avoided. A sediment control plan should be considered where dewatering is required near these drainage features.

Stockpiles of materials, fill, and excavated soil should be placed away from the crest of the excavation slopes by a distance equal to at least the depth of the excavation (minimum 3.0 m). Similarly, wheel and track loads should be kept away from the excavation sides for a distance equivalent to the depth of the excavation (minimum 3.0 m) while workers are present within the excavation. These measures will reduce the potential for trench sidewall failure.

5.4 Underground Utility Trenches

Trenching and backfilling in advance of pipe placement should be carried out in accordance with local municipal specifications, the current Red Deer County Design Guidelines and Red Deer County General Construction Specifications. If groundwater is encountered during construction, it is recommended that the excavation proceed in short sections not far in advance of pipe laying, and that backfilling of the pipe be carried out at least above any perched or regional groundwater levels before the end of each day.

Excavation and site preparation are expected to be required for utility installations. Recommendations for site preparation and temporary excavation are included in Sections 5.2 and 5.3, respectively. For utilities that require protection against freezing, a minimum soil cover of 2.7 m for clayey soil backfill and greater soil cover if granular backfill is used over the utility lines. It is recommended that all utilities should be installed below the expected frost penetration depth. Actual frost penetration is subject to change based on environment and depends on soil type. For instance, frost penetration depth will be deeper if granular backfill is used rather than clay backfill in areas such as utility trenches. This should be considered particularly if utility trenches will be backfilled with granular soil. In areas where the minimum ground cover cannot be achieved, a properly designed rigid insulation system may be incorporated. Additional comments and/or recommendations can be provided upon request.

The minimum thickness of pipe bedding below the pipe should be 100 mm. Bedding material for utility trenches should be Class "B" bedding in accordance with current specifications and should be placed and compacted in lifts to at least 98% of SPMDD around the pipe, including underneath the haunches. Hand-tamping equipment should not directly contact the pipe and should not be allowed to compact above the pipe until the full 300 mm thick layer of pipe bedding has been placed above it.

The native clay soils encountered at the borehole locations will likely be suitable as trench fill materials if properly compacted within ±2% of OMC and compacted to 98% of SPMDD. Adverse weather conditions, which could raise the moisture of the soils during construction, could make the recommended compaction criteria difficult to achieve. Moisture conditioning of the native soils will be required in order to achieve necessary compaction specifications.

Should construction activities take place during cold weather (temperature below +5˚C), it is recommended that the excavation proceed in short sections and that the time span between start of excavation and backfilling be kept to a minimum to reduce the potential for the soils to freeze. It should be noted that the placement of fill during freezing conditions will be difficult and the specified degree of compaction may not be achieved. In addition, adverse weather conditions, which could affect the moisture content of the soils during construction, could make the recommended compaction criteria difficult to achieve. Moisture conditioning of the native soils will be required in order to achieve necessary compaction specifications.

5.5 Pavement Structure Design Alternatives

5.5.1 General

Based on the results of the geotechnical drilling assessment and subsequent laboratory testing program, the subsurface soil is generally considered suitable for the proposed road construction. Actual subgrade conditions should be confirmed during construction of the road to verify that the design assumptions included herein are valid.

The structural Asphalt Concrete Pavement (ACP) analysis and design methodology was developed based on guidelines provided in the AASHTO *Guide for Design of Pavement Structures 1993* and *1998 Supplement*, Mountain View County Bylaws and Red Deer County Design Guidelines.

In order to estimate the subgrade resilient modulus, M_r, utilized in commercially available software and the design procedure provided by AASHTO (1993), the following relationship may be used to estimate the subgrade modulus based on soaked CBR testing for fine grained soils encountered during the geotechnical field program:

$$M_r = 10.34 \times CBR$$

5.5.2 Asphalt Concrete Pavement Alternatives

Based on our assessment in the field by conducting SPT and PP tests and subsequent laboratory testing to determine soil index properties, test results on the native clay samples are reasonably representative of the subgrade material. Based on the field and laboratory test results, a soaked CBR value was determined to be approximately 4.0%.

The following preliminary pavement sections provided in Table 3 may be used in the planning stages of this development.

TABLE 3: RECOMMENDED MINIMUM PAVEMENT STRUCTURE

Material	Road Classification				
	Industrial Collector	Residential Collector	Industrial Local	Residential Local	Pathways
Asphalt (mm)*	100	100	90	75	75
Granular Base (mm)	200	150	150	100	150
Granular Sub-base (mm)	300	300	300	250	n/a
Subgrade	Reviewed by geotechnical engineer				
Total Thickness (mm)	600	550	540	425	225

*The maximum depth of a single lift of asphalt shall be 75 mm. The minimum initial depth of asphalt shall be 50 mm. The minimum depth of successive lifts shall be 40 mm.

5.5.3 Construction Recommendations

The performance and maintenance requirements for these designs are highly dependent upon proper subgrade preparation and subgrade drainage, and these must constitute part of the design. Movements due to frost heaving must be anticipated, unless mitigation of frost is undertaken by utilization of insulation or granular structures thicker than those proposed above.

It should be noted that all pavements eventually crack, and proper and timely maintenance of the pavement will be required to achieve maximum life expectancy. Generally, pavement maintenance will involve crack sealing and repair of areas exhibiting pavement distress. Should thinner pavement structures than those recommended, the owner should be prepared to accept the risk of premature deterioration (rutting, cracking, etc.) and maintenance (crack repair, asphalt resurfacing, etc.) of the roadway. The severity of these potential risks is difficult to quantify, although increased deterioration and maintenance should be expected with decreasing pavement structure thickness.

The recommendations for site grading provided in Section 5.2 should be followed in the preparation of the subgrade beneath the roadways. Prior to gravel placement, the exposed subgrade should be proof rolled, using suitably sized equipment, to identify any soft, loose, or non-uniform areas. Any soft, loose or non-uniform areas detected should be over-excavated and replaced with approved material. If extensive, deep, soft soil deposits are encountered, additional geogrid and/or geotextile may be incorporated to improve the condition of the subgrade soils. The use of such methods to improve poor subgrade conditions will need to be made at the time of construction. The final subgrade elevation and any sub cut sections are to permit positive subgrade drainage to the edges of the roadway and into local storm system. Implementation of these measures will significantly reduce the moisture content ingress into the subgrade following construction minimizing saturation and degradation of the subgrade.

All engineered fill material should be free of any organic and other deleterious material, and should be tested and approved by the geotechnical consultant prior to use. In general, where clay soils are to be used as general engineered fill, moisture conditioning may be required. Extensive fill placement should not be performed during freezing conditions or using frozen soils. The native inorganic soil encountered in the boreholes is suitable material for use as general engineered fill. General engineered fill should be compacted to a minimum of 98% SPMDD at a moulding moisture of optimum to 2% above optimum moisture content (OMC) for cohesive soils and ±2% OMC for cohesionless soils. All fill should be placed in lift thicknesses compatible with the compaction equipment being used, to a maximum compacted lift thickness of 300 mm and 150 mm for granular and cohesive fill soils, respectively. Where satisfactory subgrade conditions exist, the subgrade should be scarified to a minimum depth of 150 mm and re-compacted to a minimum of 100% SPMDD, prior to placing granular fill.

Granular sub-base and base materials are to be compacted to a minimum of 100% SPMDD at a moulding moisture of ±2% OMC for cohesionless soils. The use of alternative fill placement methods should be reviewed by the geotechnical engineer prior to construction activities. If the roadways are to be constructed during the winter months (temperature below +5˚C), the required degree of compaction can only be achieved by using unfrozen gravel.

The gradations of the imported granular materials should be within the limits specified in Table 4. Other gradations may be acceptable but should be reviewed by the geotechnical engineer prior to use. All granular materials supplied must comply with the minimum requirements identified in project specifications. Other gradations may be acceptable but should be reviewed by the geotechnical engineer prior to use.

TABLE 4: RECOMMENDED GRAIN SIZE DISTRIBUTION FOR SUB-BASE AND CRUSHED BASE GRAVEL

Sieve Designation	Granular Sub-base Gravel Percent Passing* (%)	Granular Base Gravel Percent Passing (%)
80 mm	100	-
50 mm	55 - 100	-
25 mm	38 - 100	-
20 mm	-	100
16 mm	32 - 85	84 - 94
10 mm		63 - 86
5 mm	20 - 65	40 - 67
1.25 mm	-	20 - 43
0.630 mm	-	14 - 34
0.315 mm	6 - 30	9 – 26
0.160 mm	-	5 - 18
0.080 mm	2 - 10	2 - 10
Plasticity Index (%)	NP - 8	NP - 6
% Fracture (2 faces)	-	60+
CBR (%)	40 min.	80 min

5.6 Seismic Design Considerations

The 2015 National Building Code of Canada (NBCC) Seismic Hazard Calculation values have been provided in Table 5 below for the site as derived from the online National Building Code of Canada Seismic Hazard Calculator available through the following link: https://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/calc-en.php. A copy of the site-specific seismic hazard calculation is provided in Appendix E.

The seismic hazard calculation is performed based on the seismic activity in the region. The seismicity of the area of interest is derived from the earthquake catalogues collated in the region. A pre-requisite for the analysis is the process of de-clustering and unification of magnitude scales for all the earthquakes in the catalogues. The regional geological setting, local geology, geotectonic settings and local seismic characteristics of the area seismic sources are also very important elements of the Probabilistic Seismic Hazard Assessment (PSHA). Given the site is in a relatively low seismic activity zone and located within Stable Canada, a comprehensive PSHA for the proposed facility has not been performed.

The 2015 National Building Code of Canada (NBCC) is intended to be used for standard structures. Pertinent seismic data for the proposed development are provided below and the values are for *very dense soil and soft rock* (2015 NBCC Site Class C).

The proposed structures should be designed under the seismic provisions of the 2015 NBCC. Horizontal Peak Ground Acceleration (PGA) and 5% damped spectral response acceleration values $S_a(T)$ for different periods (ie. 0.2 s, 0.5 s, 1.0 s, 2.0 s, 5.0 s and 10.0 s) are outlined in Table 7. As per current standards, the proposed structure should be designed for a seismic event with a 2% probability of exceedance in 50 years (1 in 2,475 year event).

TABLE 5: NBCC INTERPOLATED SEISMIC HAZARD VALUES (SITE CLASS C)

Return Period (years)	Sa (0.2)	Sa (0.5)	Sa (1.0)	Sa (2.0)	Sa (5.0)	Sa (10.0)	PGA (g)	PGV (g)
2475	0.159	0.106	0.062	0.031	0.010	0.004	0.085	0.067

Based on the current level of geotechnical data, the development's Site Classification for Seismic Response is Site Class C with a corresponding average shear wave velocity of 360 m/s < Vs_{30} < 760 m/s and undrained shear strengths greater than 100 kPa. As such, the design spectral values provided in the 2015 NBCC Seismic Hazard Calculation may be utilized to aid in determining the effects of seismic generated loadings and do not need to be modified to account for the variance in the Site Classification. The site-specific design spectral acceleration values of S(T), are provided in Table 5.

5.7 Liquefaction Assessment

Seismic liquefaction refers to a sudden loss in stiffness and strength of the soil due to cyclic loading effects of an earthquake. The loss arises from a tendency for soil to contract under cyclic loading, and if such contraction is prevented by the presence of water in the pore spaces that cannot escape, it leads to rise in pore pressure and a resulting decrease in effective stress. If the effective stress decreases sufficiently, the strength and stiffness will also decrease to a point that the soil will behave as a liquid. However, unless the soil is very loose it will dilate and regain some stiffness and strength, as it strains. The post-liquefaction strength is called the residual strength and be appreciably lower than the static strength.

If the residual strength is sufficient, it will prevent a bearing failure for level ground conditions, but may still result in excessive settlements. For sloping ground conditions, if the residual strength is satisfactory, it will prevent a flow slide, but displacements commonly referred to as lateral spreading, could be excessive. Further, even for level ground conditions, where there is no possibility of a flow slide and lateral movements may be tolerable, very significant settlements may occur due to the dissipation of excess pore water pressures during and after the seismic event.

The soil response depends on the characteristics of the soil layers, the depth of the water table and the intensity and duration of the ground shaking. If the soil consists of deposits of loose granular materials, it may be compacted by the ground vibrations induced by the earthquake, resulting in large settlement and differential settlements of the ground surface. This compaction of the soil may cause liquefaction of the soil, resulting in settlement, tilting and rupture of structures.

The following factors influence the liquefaction potential of a given site.

- Soil type – saturated granular soils, especially fine-grained loose sands and silts, with poor drainage conditions are susceptible to liquefaction.
- Relative density – loose sands are more susceptible to liquefaction, i.e. sand with Dr > 80% is not likely to liquefy.
- Confining pressure – the confining pressure increases the resistance to liquefaction.
- Stress due to earthquake – as the intensity of the ground shaking increase, the shear stress ratio increases and the liquefaction is more likely to occur.
- Duration of earthquake – as the duration of the ground shaking increases, the number of stress cycles increases leading to an increase in the excess pore pressure, and consequently liquefaction.
- Drainage conditions – poor drainage allows pore pressure to build up and consequently liquefaction.

Based on the current Scope of Work, a comprehensive liquefaction assessment has not been performed. If required, Metro can provide additional comments and recommendations regarding liquefaction. However, based on the soil and groundwater conditions encountered during the current and previous geotechnical programs, and our assessment of these conditions, the site has a low probability of seismic liquefaction. It should be noted that although the structure may be damaged beyond operational use under the DGM (design ground motion), it is expected catastrophic failure of the proposed structure would not occur. This performance level is termed ‘extensive damage’ because, although the structure may be heavily damaged and may have lost a substantial amount of its initial strength and stiffness, it retains some margin of resistance against collapse (APEGBC, 2010).

5.8 Additional Construction Recommendations

5.8.1 General

Selected samples of the native soils were submitted to an analytical laboratory to determine the resistivity/salinity of the soils encountered during the field investigation. The results of the resistivity/salinity are included in Appendix D and discussed below.

5.8.2 Resistivity/Corrosivity

Soil resistivity is typically a function of soil moisture and concentrations of ionic soluble salts and is used to indicate the potential level of soil corrosiveness. Generally, the lower the resistivity, the more corrosive the soil will be.

Soil resistivity testing was performed on six representative soil samples secured during the initial geotechnical assessment completed in 2019. These soil samples were used to determine the potential for corrosion of steel structures in contact with native soils. Measured soil resistivity values ranged between 131 and 2130 ohm-cm. After comparison with the US FHWA (2010) standards, the resistivity of the soil satisfies the criteria for strong corrosion potential. In addition, based on a review of *Corrosion Basics: An Introduction by Pierre R. Roberge (2006)*, suggests an extreme degree of corrosion potential in the native soils tested at the site. Consideration should be given to the provision of corrosion protection of steel pipes.

5.8.3 Sulphate Attack Potential

Sulphates have a detrimental effect on concrete and can adversely affect the performance and/or service life of these materials. Water soluble sulphate testing was performed on six representative soil samples secured during the initial geotechnical assessment completed in 2019. The test results indicate the potential degree of sulphate attack may be considered *very severe* (CSA A23.1-19, Table 3). Accordingly, it is recommended that concrete in contact with native soils be designed for an "S-1" class exposure, as per CSA A23.1-19, Table 3. In addition, all concrete must be designed in accordance with the CSA A23.1-19 e.g. air-entraining agents are required in freeze/thaw zones. Should any fine grained soils be imported to the site, it should be tested for the presence of sulphates and the above recommendations modified, if required.

All concrete must be supplied in accordance with the current Alberta and National Building Code requirements. All concrete mix design, and construction, should be carried out in accordance with the CAN/CSA A23.1-19 and A23.2-19 specifications.

6.0 BUILDING FOUNDATIONS

6.1 General

The geotechnical recommendations for foundation design in this report are presented in terms of limit states design. The basic equation is:

$$\Phi R_n \geq \sum \alpha_i S_{ni}$$

Where: ΦR_n = factored geotechnical resistance
Φ = geotechnical resistance factor
R_n = ultimate geotechnical resistance
$\Sigma\alpha_i S_{ni}$ = summation of the factored overall load effects for a given load combination condition
α_i = load factor corresponding to a particular load
S_{ni} = specified load component of the overall load effects (i.e. dead load due to the weight of the structure or live load due to wind)
i = represents various types of loads such as dead load, live load, wind load etc.

There are several foundation alternatives suitable for the subsurface conditions encountered during the current geotechnical program. Spread and continuous strip foundations are considered feasible for support of proposed building foundations. However, it should be noted that the soils encountered have the potential for relatively large frost related movements and frost heave pressures if allowed to freeze in proximity to groundwater.

Two general design scenarios are evaluated corresponding to the ultimate capacity and serviceability requirements.

1) Ultimate Limit States (ULS) is concerned with ensuring that the maximum structural loads do not exceed the nominal (ultimate) capacity of the foundation elements. The ULS foundation resistance is obtained by multiplying the nominal (ultimate) capacity by a resistance factor (Φ), which is then compared to the factored (increased) structural loads. The ULS capacity must be greater or equal to the maximum factored load. ULS resistance factors that can be used for the design of foundations are outlined in the current Canadian Foundation Engineering Manual (CFEM 2006) depending on the method of analysis and verification testing completed during construction.
2) Service Limit States (SLS) is concerned with limiting deformation or settlement of the foundation under service loading conditions such that the integrity of the structure will not be impacted. The SLS should generally be analyzed by calculating the deformation resulting from applied service loads and comparing this to the tolerance of the structure. However, the settlement tolerance of the structure is typically not yet defined at the preliminary design stage. As such, SLS resistance (or unit resistance) are provided that are developed on the basis of limiting deformation to approximately 25 mm or less, unless otherwise specified.

The geotechnical factors believed to be pertinent for the design and construction of the proposed development are presented below. The recommended design values are subject to engineering observations and approval by a qualified geotechnical engineer.

6.2 Shallow Foundations

6.2.1 Bearing Resistance

Shallow foundations and slabs-on-grade (including thickened edge slabs) may be considered as foundation elements for the proposed structures at the site. However, to reduce the potential negative impacts associated with swelling clays and frost heave, supplemental construction measures may be required.

Based on the results of the geotechnical program, it is likely that continuous and spread footings would be founded within the stiff to very stiff native clay deposits or bedrock. The recommended ultimate and factored ultimate limit states (ULS) bearing resistances for shallow foundations and slab-on-grade construction are provided below.

TABLE 6: ULTIMATE AND FACTORED BEARING RESISTANCE

Type of Material	Ultimate Geotechnical Resistance (kPa)	Factored Geotechnical Resistance at ULS (kPa)*
Engineered Fill/Stiff Silty Clay	200	100
Bedrock	300	150

*A geotechnical resistance factor, Φ, of 0.5 has been applied for the recommended bearing pressures provided above in accordance with the Canadian Foundation Engineering Manual (CFEM, 4th Edition, 2006).

The estimated bearing pressures for strip and spread foundations bearing on the silty clay deposits, based on serviceability limit states (SLS), are provided below in Table 7. These values are based on settlements not exceeding 25 mm for foundations placed at a depth of 2.0 m below grade, for strip and spread footings, with widths ranging between 1.0 m and 2.5 m. If settlement or differential settlement is of potential concern, it is recommended that precision elevation surveys be performed during and after the structures are put into service.

TABLE 7: RECOMMENDED SLS BEARING PRESSURE

Footing Width (m)	1.0	1.5	2.0	2.5
Strip footing (kPa)	100	80	60	40
Spread footing (kPa)	150	100	80	60

The recommended design coefficient of sliding friction between concrete foundations and silty clay subgrade soils may be taken as 0.35. If the foundation is on granular soils, the friction factor may be taken as 0.55.

The recommended bearing pressures presented above are for vertical, concentric loading conditions as described in the Canadian Foundation Engineering Manual (CFEM, CGS 2006). For loadings subject to eccentric loads, the use of equivalent footing width, as outlined in the CFEM, should be followed.

If the footings are to be stepped, this should be done so that a line connecting the closest edges of two footings is no steeper than 2H:1V. Where this cannot be achieved, the lower footing should be designed to accommodate the footing surcharge. The base of the footing should be below a 1H:1V line projected up from the base of any adjacent excavation undertaken for installation of buried utilities.

The bearing surfaces must be cleaned of all loosened or softened soils. Foundation excavations and bearing surfaces should be protected before, during and after footing construction from rain, snow, freezing temperatures, excess drying and the ingress of free water. Footings must not be constructed on frozen soils. In addition, a qualified geotechnical engineer should observe bearing surfaces, prior to placement of foundation concrete, to confirm that the design bearing parameters are appropriate.

Footings founded on frozen soil will settle after thawing. Therefore, all footing subgrades must not be allowed to freeze prior to and after casting of the concrete foundation. Any frozen soil should be removed and replaced with additional footing concrete. Alternatively, footings may be extended down to a suitable depth below the frost level. If construction activities are scheduled during cold weather (less than +5°C, cold weather construction practices should be followed.

6.2.2 Concrete Slab-on-Grade

Generally, concrete slabs-on-grade are governed by consideration of tolerable settlements rather than bearing capacity failure. The modulus of subgrade reaction, K_v, is commonly used to characterize the vertical stiffness of the soil below a slab-on-grade foundation.

The modulus of subgrade reaction for a foundation on soil is defined as follows:

$$K_v = {}^{q}/_{\delta}$$

Where K_v = Modulus of vertical subgrade reaction (kPa/mm)
q = Applied bearing or contact pressure acting on the footing (kPa)
δ = Settlement under the applied pressure q (mm)

The modulus of subgrade reaction is not a fundamental soil property. A typical value for the modulus of vertical subgrade reaction, K_v, is approximately 30 kPa/mm for the stiff silty clay encountered on the site. The value is applicable to a 300mm by 300mm (1 ft^2) plate, the adopted standard reference basis. For footings with larger dimensions, the following formulae are recommended:

For clayey soils:

$$K_{vb} = \frac{K_{v1}}{b}\frac{[m + 0.15]}{1.5m}$$

For sandy soils:

$$K_{vb} = K_{v1}\frac{[b + 0.3]^2}{2b}$$

Where K_{v1} = Modulus of vertical subgrade reaction (kPa/mm) for a 1 ft^2 plate
K_{vb} = Modulus of vertical subgrade reaction (kPa/mm) for actual footing
b = Foundation width (m)
m = Foundation length (m)

The above settlement estimates are for individual elements and do not consider the influence of adjacent foundations or settlement loads. These influences should be incorporated into the design.

6.2.2.1 Construction Recommendations

Grade supported concrete slabs, supported by engineered fill prepared in accordance with Section 6.2, are expected to perform adequately at this site. The following recommendations should be followed:

- The excavated subgrade beneath the slabs-on-grade should be protected from the ingress of free water before, during and after footing construction from rain, snow, freezing temperatures and excess drying. Foundation excavations for footings and slabs-on-grade (including thickened edge slab) foundations should be inspected and approved by qualified geotechnical personnel prior to placing granular fill or concrete to confirm that the design bearing parameters are appropriate.
- The upper 300 mm of subgrade should be compacted to 100% of SPMDD at a moulding moisture of optimum to 2% above optimum moisture content (OMC) for cohesive soils and ±2% OMC for cohesionless soils prior to placement of crushed gravel.
- Lightly loaded (less than 10 kPa) grade supported concrete slabs should be underlain with 150 mm crushed gravel. Moderately loaded (greater than 10 kPa) grade supported concrete slabs in the building area should be underlain with 300 mm of crushed gravel. The gravel should be well graded, free draining and compacted uniformly to 100% SPMDD. Alternatively, the thickness of the concrete slab can be increased by 25 mm (nominal thickness of 150 mm) or as specified by the structural engineer. All granular materials supplied must comply with the minimum requirements identified in the project specifications. Other gradations may be acceptable but should be reviewed by the geotechnical engineer prior to use.
- Differential settlements of slab foundations are possible, particularly if non-uniform support conditions exist or if non-uniform load intensities are applied. Differential movements of the slabs are also possible where the thickness of structural fill beneath the slab varies considerably. Some relative movement between the slab and the walls and columns should be anticipated. Generally, if the recommendations outlined in this report are followed, these movements should be within acceptable limits. Where vertical movements of equipment or facilities on grade supported concrete slabs will be critical to operations, consideration should be given to the installation of a structural floor system supported on separate foundations. To minimize differential settlements, a well compacted granular fill is recommended. Control joints should be provided at regular intervals within the concrete slabs to control random cracking. It is not recommended that the slab be tied to the walls or columns. Mechanical equipment supported on the floor slab should also contain provision for re-levelling.
- Concrete slabs should be provided with construction joints or saw cuts in accordance with local practice or as required by the structural engineer. The concrete slab should be reinforced with steel bars or equivalent wire mesh. Steel reinforcement should be in accordance with the structural engineer's requirements.
- Concrete slabs should be constructed independently of the walls, columns and grade beams. Slab-on-grade floors should be tied into the grade beams with dowels at doorways. Alternatively, the slab may be tied to grade beams if a construction joint is placed parallel to the wall at a distance of about 2.0 m or as per structural engineer's requirements.

- Non-load bearing partitions on slab-on-grade floors should be designed to accommodate slight vertical movements of at least 50 mm. Service connections should provide flexibility to allow for differential movements.
- Piping and electrical conduit connections should be laid out to permit some flexibility, as vertical movement of such equipment as water meters, furnaces and electrical equipment may cause significant stress in the pipes. The provision is particularly important where there are short pipe runs between mechanical equipment and points where pipes pass through walls. Heating ductwork beneath floors should be insulated with at least 50 mm of rigid insulation to prevent drying of subgrade soils.
- A moisture barrier, such as 6 mil polyethylene sheeting, should be installed underneath the slab to minimize potential for slab dampness. Consideration should be given to the effect of any vapour barrier on concrete slab curling. A thin layer of sand can be placed beneath the polyethylene to avoid puncture when clear crushed material is used.

6.2.3 Frost Protection for Shallow Foundations and Concrete Slab-on-Grade

Based on the soil conditions encountered during soil investigation and the estimated thermal conductivity provided in Figure 13.7 of the *Canadian Foundation Engineering Manual (CFEM)* (CGS 2006), the frost penetration depth in the project area has been estimated, assuming clay type soil, to be approximately 2.6 m below the existing ground surface. Frost penetration may vary with soil composition, consistency, and snow cover. Actual frost penetration is subject to change based on environment and depends on soil type. Alternatively, insulation and/or void form materials can be utilized to reduce adverse frost effects. Otherwise, frost heave and adfreeze effects should be considered in the foundation design. Interior footings in a permanently heated structure may be constructed at any depth, provided suitable soils with the design bearing pressure are available.

Slabs-on-grade and thickened edge slabs constructed without rigid insulation and/or void form materials should be supported on at least 150 mm of compacted, free draining, well graded, 20 mm minus crushed gravel and sand placed over properly prepared, competent subgrade soils. The crushed gravel base should be compacted uniformly to 100% SPMDD. The gradation for well graded, 19 mm crushed gravel should be within the limits specified in Table 4. Other gradations may be acceptable but should be reviewed by the geotechnical engineer prior to use.

Slabs-on-grade and thickened edge slabs that are placed directly on the silty clay subgrade materials would be subject to seasonal vertical movements due to frost heave and thawing effects. Frost related movements are due to the in-situ water content and capillary action of the native soils, and potential near surface water bearing soil units. Frost related movements in the order of 50 mm to 75 mm for perimeter footings (including thickened edge slabs) for heated structures and 75 mm to 100 mm for unheated structures placed at or near final ground surface, may be anticipated. If these movements are not acceptable, slabs-on-grade should be placed on thick pads of granular material and/or insulating and void-form materials can be used to reduce adverse frost effects. Rigid insulation may be used to limit the effects of frost action. Metro can provide additional comments regarding the use of insulation, if required.

Specific installation details should be verified with the manufacturer/supplier of the particular product selected to confirm their bearing capacity and compressibility characteristics. Care must be taken to select types of insulation sheets or foam that do not deteriorate in the presence of hydrocarbons and other chemical contaminants.

6.2.4 Perimeter Foundation Drainage

Due to the possibility of high plastic clay soils and to mitigate the potential for movements associated with swelling soils, it is recommended that a suitably designed permanent perimeter drainage system be installed to minimize the potential for water infiltration into the soil surrounding foundations. The weeping tile should be provided around the exterior of the basement and frost walls, with adequate connections to the storm sewer system. It should be installed with positive slope away from foundation elements and in accordance with the current Alberta Building Code requirements.

Perimeter drainage should consist of a perforated rigid PVC pipe, and should be placed around the external sides of the building below the base of the foundations. Drainage pipes should be provided with permanent cleanouts and surrounded by a minimum of 150 mm of clean drain rock. A non-woven geotextile (Nilex 4551 or approved equivalent) should completely encapsulate the drainage gravel to act as a filter against the migration of fines from the general backfill and surrounding soil. The perimeter drainage system should be designed to direct water by gravity flow into a storm sewer or permanent drain located away from the building. The roof and surface runoff should be collected and directed to a storm sewer or permanent drain in solid wall pipes separate from the perimeter drainage.

6.3 Retaining Walls

There are three categories of lateral earth pressure and each depends upon the movement experienced by the wall on which the pressure is acting. The three categories are:

1. <u>At rest</u> earth pressure – develops when the wall experiences no lateral movement. Typically occurs when the wall is restrained from movement such as along a basement wall that is restrained at the bottom by a slab and at the top by a floor framing system prior to placing soil backfill against the wall.
2. <u>Active</u> earth pressure – develops when the wall is free to move outward such as a typical retaining wall and the soil moves sufficiently to mobilize its shear strength.
3. <u>Passive</u> earth pressure – develops when the wall structure moves into the soil sufficiently to mobilize its shear strength. This will not be discussed.

In order to develop the full active or passive pressures, the wall must move. If the wall does not move a sufficient amount, then the full active or passive pressures will not develop and the wall will experience higher than the expected pressure.

The results of the subsurface investigation indicate that underground structures are feasible within the proposed development. It should be noted that earth pressures on underground structure depends on a number of factors including the backfill material, hydrostatic pressure, surcharge loads, backfill slope, drainage, rigidity of the structure or retaining wall, and method of construction including sequence and degree of compaction. For the proposed development, Metro assumes that construction practices will involve backfilling the foundation wall prior to the floor system being in place. Therefore, an active pressure condition will exist during construction. However, long term wall stability will be provided by lateral restraint at both the top and bottom of the wall creating an at rest condition.

Assuming that the wall is frictionless and the soil backfill against the vertical foundation wall is horizontal, the following Rankine earth pressure coefficients, are presented below. For purposes of preliminary design, Table 8 provides earth pressure coefficients assuming a granular soil backfill with the following properties:

- Friction Angle, Ø = 33 degrees;
- Unit Weight, ϒ = 21 kN/m^3; and,

TABLE 8: RANKINE EARTH PRESSURE COEFFICIENTS

Description	Coefficient
Static Active Earth Pressure Coefficient, K_a	0.29
At-Rest Earth Pressure Coefficient, K_o	0.46
Passive Earth Pressure Coefficient, K_p	3.39

The value provided above assume that the underground structures will be backfilled with clean, free draining granular material. Also, it is important to ensure that any groundwater entering the backfill area is free to drain vertically into the drainage system. If it is not possible to provide the granular materials and drainage behind the wall, then increased earth pressures and hydrostatic pressures must be assumed to act on the wall and the hydrostatic pressures would be additive to the static design earth pressures. If the assumptions made are incorrect, Metro must be notified to prepare appropriate parameters for design.

7.0 SLOPE STABILITY ASSESSMENT

7.1 General

The Mountain View County Municipal Development Plan Bylaw 20/20 indicates that a Slope Stability Geotechnical Report is required for all sites where slope stability is a concern. Further, slopes of 10% or more shall require a geotechnical report demonstrating stability and suitability for development along with the standards for development.

Slope setbacks are required for developments located near ravines or rivers where the slopes may be unstable. A setback represents what is considered a safe distance based on information provided about the physical characteristics of a site. Several factors are considered when establishing setbacks, these include:

- Variability of river breaks;
- Variability of ground stability;
- History of slope stability in the area;
- Potential mitigation measures; and
- Other information specific to the site.

The primary objective of seismic design is to provide an acceptable level of safety for building occupants and the general public as the building responds to ground motions to minimize loss of life. Although there will likely be extensive structural and non-structural damage during the Design Ground Motion (DGM), there is a reasonable degree of confidence that the building will not collapse nor will its attachments break off and fall on people near the building. Although the structure may be heavily damaged and may have lost a substantial amount of its initial strength and stiffness, it retains some margin of resistance against collapse.

Metro has conducted a slope stability assessment pertaining to the proposed development. The purpose of the slope stability assessment was to perform long term static assessments, determine the effects of seismic induced loadings and to ensure the safety and functionality of proposed structures. The overall slope configurations are provided in Appendix A. The following sections of this report provide guidance and recommendations for geotechnical aspects pertaining to the overall stability of the development.

7.2 Methodology

For the purpose of this slope assessment, deep seated rotational slope failures and isolated block failures were considered for the purpose of establishing safe slope setbacks. In other words, shallow surficial slope failures are not considered. The slope stability analyses were carried out using soil parameters, groundwater conditions and a slope profile that attempt to model the slopes in question but do not exactly represent the actual conditions. The configurations are based on the layout provided by Appin Construction personnel. Based on the proposed layout of the development, five cross sections were determined to be critical and are presented in Appendix A.

Slope stability Factor of Safety (FOS) is defined as the ratio of forces that tend to resist slope movement to forces that tend to drive slope movement. For the purposes of this study, a computed factor of safety of less than 1.0 to 1.3 is considered to represent a slope bordering on failure to marginally stable, respectively; a factor of safety of 1.3 to 1.5 is considered to indicate a slope that is less likely to fail in the long term and provides a degree of confidence against failure ranging from marginal (1.3) to adequate (1.4 and greater) should conditions vary. A factor of safety of 1.5 and 1.0, or greater, is considered to indicate adequate long term stability under static and pseudo-static loading conditions, respectively.

The slope stability of the proposed containment area has been assessed using the two dimensional limited equilibrium software SLOPE/W (Geo-studio International) using the Morgenstern-Price method of analysis for both undrained (short-term) and drained (long-term) conditions. The software calculates factor of safety, for a user specified geometry using a number of analytical methods, against sliding or sloughing. For each case, we evaluated the slope stability factor based on a judicious search of circular slip planes and assumed block failure surfaces. Graphical representation of the results of each analysis is shown in Appendix E and present the minimum calculated FOS for each analysis. Other failure surfaces are possible but provide higher FOS.

7.3 Seismic Effects

Slope movements are a secondary effect of an earthquake and it is anticipated that the soils will experience a significant, albeit transient, reduction in shear strength. During a seismic event, there are two modes or mechanisms of slope instability:

- Flow sliding develops when the driving forces exceed shear strength of soil along a failure surface.
- In lateral spreading, driving forces do not exceed shear strength of the soil along a slip surface. Instead, the driving forces only exceed resisting forces during those portions of earthquake that impart net inertial forces in the downward direction. This results in progressive and incremental lateral movement.

Pseudo-static analysis is one of the simplest approaches used in earthquake engineering to assess the seismic response of soil embankments and slopes. This method of analysis ignores the cyclic nature of an earthquake and assumes that additional static forces are applied on the slope due to the earthquake. In earthquake prone areas, horizontal and vertical pseudo-static (seismic) coefficients, k_h and k_v, respectively are used to estimate the horizontal and vertical forces caused by the potential earthquake. These forces are, in turn, added to the overall equilibrium calculations for the individual slices composing the failure surface. The selection of an appropriate seismic coefficient to be used in the analysis is the most important, and difficult, aspect of a pseudo-static analysis. Recommended horizontal seismic coefficients differ depending on the Authority and vary between 0.05 – 0.15 (in the United States) to 0.5 for catastrophic earthquakes (Melo, 2014).

7.4 Design Inputs

7.4.1 General

The following design inputs and assumptions have been considered in the stability assessment pertaining to the proposed development.

- For the design earthquake, it is considered the soils and bedrock materials encountered during the geotechnical assessment are unlikely be subject to liquefaction behavior. However, if liquefaction occurs, the resulting ground movements will be tolerable. Therefore, liquefaction effects have not been considered.
- Assumed horizontal seismic coefficient, k_h, equal to 1.0 (conservative).
- Assumed vertical seismic coefficient, k_v, equal to 0 (design seismic event would not be expected to have a vertical acceleration component).
- A comprehensive PHSA has not been completed for the facility. Seismic loading analyzed for Annual Exceedance Probability of 0.000404, or a return period of 2475 years (conservative).
- No load (or material) factors have been applied for the stability assessments.
- Porewater coefficient, Ru, assumed to be approximately 0.15 for soils above seasonal groundwater levels. This coefficient is used to denote the extent to which cracks are filled with water and is the ratio of pore pressure to total vertical overburden stress at any point.
- Seasonal increase in groundwater levels assumed to be 1.0 m from results of the groundwater levels measured in March 2021.

7.4.2 Soil and Groundwater Parameters

The short term, end of construction design of the project should be based on undrained (total) shear strength parameters. The undrained shear strength (S_u) of cohesive soils can be determined using unconfined compression tests of undisturbed samples and is generally accepted to be equal to one half of the unconfined compressive strength. Typically, the total internal friction angle (ϕ) is negligible and assumed to be zero in cohesive materials.

The long term design of the project should be based on drained (effective) strength parameters. The use of either peak or residual effective shear strengths are based on the expected or tolerable soil displacements. For normally-consolidated cohesive soils, the cohesion intercept (apparent cohesion) should be zero. For over-consolidated cohesive materials, such as many glacial tills, the cohesive intercept can be non-zero.

This section provides guidance on the interpreted values of engineering properties adopted in the stability analysis. Soil properties including unit weight, effective friction angle, effective cohesion and undrained shear strength for the anticipated near surface soils and underlying bedrock used in performing the sensitivity and stability analysis are presented below in Table 9. The soil and bedrock parameters provided are based on the current laboratory testing results, and our experience with similar soils and bedrock. It is assumed that each soil or bedrock unit is characterized as having the same geologic depositional history and stress history and generally has similarities throughout the stratum in terms of density, source material, etc.

TABLE 9: SOIL AND BEDROCK PARAMETERS ADOPTED FOR SLOPE STABILITY ANALYSIS

Soil Type	Unit Weight (kN/m^3)[1]	Friction Angle, ϕ' (degrees)[2]	Cohesion, c´ (kPa)[3]	Undrained Shear Strength, Su (kPa)[4]
Topsoil	14.0	0	0	0
Upper Silty Clay Till	17.5	18 - 22	5	50
Lower Silty Clay Till	18.5	22 - 26	10	75
Sandstone	23.5	28 - 32	500	2000
Mudstone	22.0	24 - 28	150	300
Shale/Siltstone	23.0	26 - 28	400	1000
Shale (Bentonitic)	21.5	18 - 22	100	200

Note:
1. Natural Condition, above groundwater table.
2. Friction angle to be used in tong-term (drained/effective stress) loading conditions only.
3. Designer shall determine whether fully softened strength values (c=0) are appropriate.
4. Undrained shear strength to be used in short-term (undrained/total stress) loading conditions. Total internal friction angle (ϕ) is negligible and assumed to be zero (ϕ=0).

At the time of this report, it is assumed that groundwater is located at the base of the slopes and may be expected to increase some 1.0 m from current levels due to seasonal fluctuations.

7.5 Slope Stability Results

7.5.1 Long Term Static Stability

The long term static stability reflects a finished condition for the development and a sufficient period of time is allowed for drained conditions to exist and analysis should be based on drained (effective) strength parameters. For long term conditions under static loading conditions, a minimum calculated Factor of Safety (FOS) of 1.5 is recommended. The results of the slope stability analysis for the development indicate that the calculated FOS values under static loading conditions were consistently greater than 1.5 for all models. Graphical representation of the results of each analysis is shown in Appendix F and present the minimum calculated FOS for each analysis. Other failure surfaces are possible but provide higher FOS.

The results of the slope stability analysis indicate that current development configuration will be stable under long term static loading conditions.

7.5.2 Long Term Pseudo-Static Stability

The seismic case was developed based on the 2475 year return interval event with the application of horizontal seismic coefficients (k_h) for the pseudo-static seismic assessment. In widely accepted analytical methods, a value equal to one-half of the peak horizontal acceleration would be suitable for the site. However, since the peak horizontal acceleration is minimal, a pseudo-static coefficient equal to 1 was used for the analysis (conservative). The results of the overall slope stability analysis for the development indicate that under pseudo-static loading conditions the calculated FOS values exceeded a minimum factor of safety of 1.0. Graphical representation of the results of each analysis is shown in Appendix F and present the minimum calculated FOS for each analysis. Other failure surfaces are possible but provide higher FOS.

The results of the slope stability analysis indicate that current development configuration will be stable under long term pseudo-static loading conditions.

7.5.3 Setback Requirements

At the time of preparing this report, the information regarding the subdivision development details, and applicable structural loadings at areas adjacent to the slopes, were unavailable. Therefore, the setback values in the report are considered as preliminary. The following recommendations shall be verified by additional slope stability assessment in a later phase of the project when the above-mentioned information regarding the subdivision development is available.

In general, the following considerations should be addressed as part of the design and construction of this subdivision, in order to prevent or mitigate any potential future slope stability concerns:

- A minimum 5 m setback from the crest of existing slopes is required for the development. The setback should be established (along the top of the slope) from that location where the slope transitions (breaks) from horizontal (or near horizontal) to steeper than 3 Horizontal to 1 Vertical (3H:1V).
- Stockpile of fill material shall not be allowed in the setback area as mentioned above, unless it is approved by the geotechnical engineer and after the completion of additional slope stability analyses.
- To mitigate saturation and degradation of the slopes from surface run-off and erosion, buffer zones and slopes should be maintained in a vegetated state that preserves the condition of the slope faces and crests to ensure stability of the slope is not affected. Typically, the combination of landscaping and well as run-off from roads, buildings and other landscape areas increase runoff rates and lead to concentrations of storm water. At a minimum, such concentrations should be lead away from slope crests, preferably towards a system of ditches or storm drains. If storm water is to be discharged towards sloping areas because of localized grade conditions, the site should be evaluated by the Geotechnical Engineer to develop appropriate recommendations to mitigate possible slope instability. Such recommendations may include retaining wall installations, dry well installation and/or use of rip rap armour. Such measures normally require accounting for design levels of risk required for protection of nearby structures including extreme storm events and seismic action.

- As part of the overall site grading for any future development, no additional surface water should be directed towards the slope unless adequate erosion control measures are incorporated. This could cause erosion of the slope and could also negatively affect the stability of the slope. At a minimum, final plan and finished grades for any proposed development adjacent to the slope should be reviewed by a geotechnical engineer to ensure that the guidelines provided on this report have been interpreted as intended.

8.0 FIELD REVIEW

It is recommended that geotechnical field review is carried out to assess the actual soil conditions encountered. Should the conditions differ significantly from those assumed for design, Metro should be provided with the opportunity to review the design assumptions and modify the design, as appropriate. Construction monitoring must be performed, and should be undertaken by qualified personnel to ensure that the minimum requirements, contained in this report, are achieved. Specific recommendations in this regard can be provided when details of site grading and earthwork operations are known.

9.0 CLOSURE

Geotechnical engineering recommendations presented herein are based on Metro's interpretation and evaluation of the findings of the geotechnical investigation completed on April 8 and 9, 2017, review of available information and recognized foundation engineering principles and practice. The materials in this report reflect Metro's best judgment based on the information that was available to Metro at the time of preparation of this report. If conditions other than those are noted during subsequent phases of the development, Metro should be given the opportunity to review and revise the recommendations included in this report, as necessary. Construction monitoring is required, and should be undertaken by qualified personnel to verify that at least the minimum requirements contained in this report, as well as the project specifications, are achieved. Recommendations presented herein may not be valid if an adequate level of inspection is not provided during construction, or if relevant building code requirements are not met.

Although subsurface conditions are not expected to vary significantly from those encountered during the geotechnical investigation program, it should be appreciated that extrapolation of subsurface conditions between boreholes is subject to interpretation, and could be at variance with actual field conditions. Similarly, ground conditions below the depth of exploration are expected to be generally as discussed herein. However, any extrapolation of data does carry a risk that actual ground conditions could differ from those shown.

This report has been prepared for the exclusive use of Appin Developments Inc., their consultants, and representatives for the specific application of the development described within this report. Any use of this report by third parties, or any reliance on or decisions made based on it are the responsibility of such third parties. Metro accepts no responsibility, if any, suffered by any third party as a result of decisions made or actions taken based on this report.

We appreciate the opportunity to be of service to you. If you have any questions regarding the contents of this report, or if we can be of further assistance to you on this project, please feel free to contact the undersigned at (250) 261-6615 or lking@metrotesting.ca.

Sincerely,

Metro Testing + Engineering Ltd.

Prepared By

L.C. KING
MAY 25/21

L.C. (Lorne) King, P.Eng.
Senior Geotechnical Engineer

Reviewed By

Brian L.J. Mylleville, Ph.D., P.Eng.
Senior Geotechnical Engineer

APEGA Permit to Practice Number: P12584

LCK/BLJM

File: AE37269_GeotechReport_Final_20210525_IFC

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care

Metro Testing + Engineering Ltd. (Metro) has prepared this report in a manner consistent with that level of care ordinarily exercised by members of the engineering and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the limits and physical constraints applicable to this report. No other warranty, expressed or implied, is made.

Basis and Use of the Report

This report has been prepared for the specific site, design objective, development and purpose described to Metro by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in the report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of this report may alter the validity of the report. Metro cannot be responsible for use of this report, or potions thereof, unless Metro is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Metro's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the Client, Metro may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Metro. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Metro are considered its professional work product and shall remain the copyright property of Metro, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Metro. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Metro's report or other products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Metro by the Client, communications between Metro and the Client, and to any other reports prepared by Metro for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Metro cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of boreholes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their own work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions

Classification and identification of soils, rocks and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgement, and boundaries between different soil, rock or geologic types or units may be translational rather than abrupt. Accordingly, Metro does not warrant or guarantee the exactness of the descriptions, associated soil characteristics or parameters.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, assessment, sampling and testing program may fail to detect all certain subsurface conditions. The environmental, geological, geotechnical, geochemical, and hydrogeological conditions that Metro interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, groundwater level lowering, pile driving, blasting etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Follow-Up and Construction Services

All details were not known at the time of submission of Metro's report. Metro should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Metro's report.

During construction, Metro should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Metro's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Metro's report. Adequate field review, observation and testing during construction are necessary for Metro to be able to provide letters of assurance, in accordance with requirements of many regulatory authorities. In cases where this recommendation is not followed, Metro's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage

Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Metro be notified of any changes and be provided with the opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Metro be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Metro takes no responsibility for the aspects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

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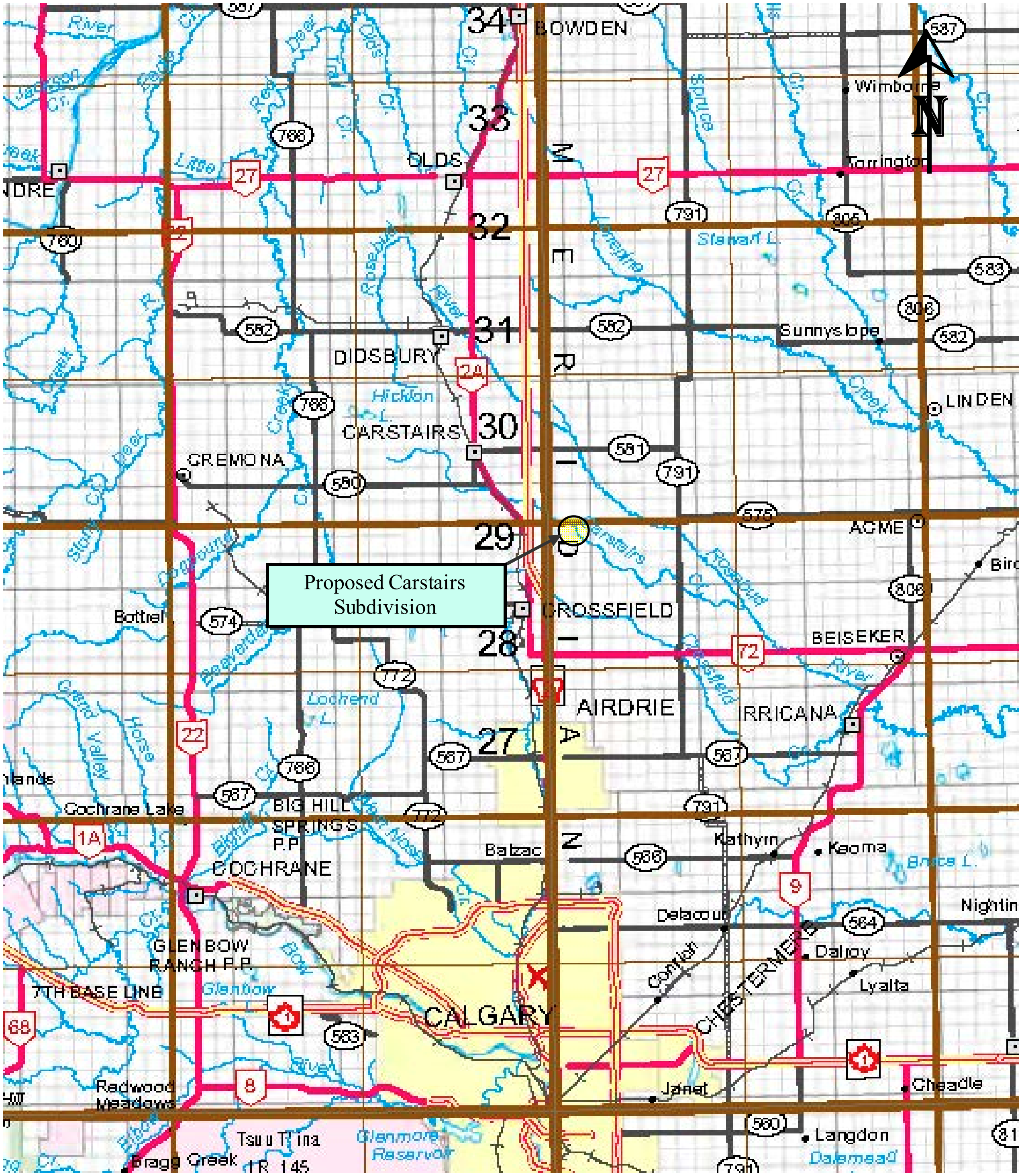
Appendix A

Site and Borehole Location Plans

Cross Section Plans



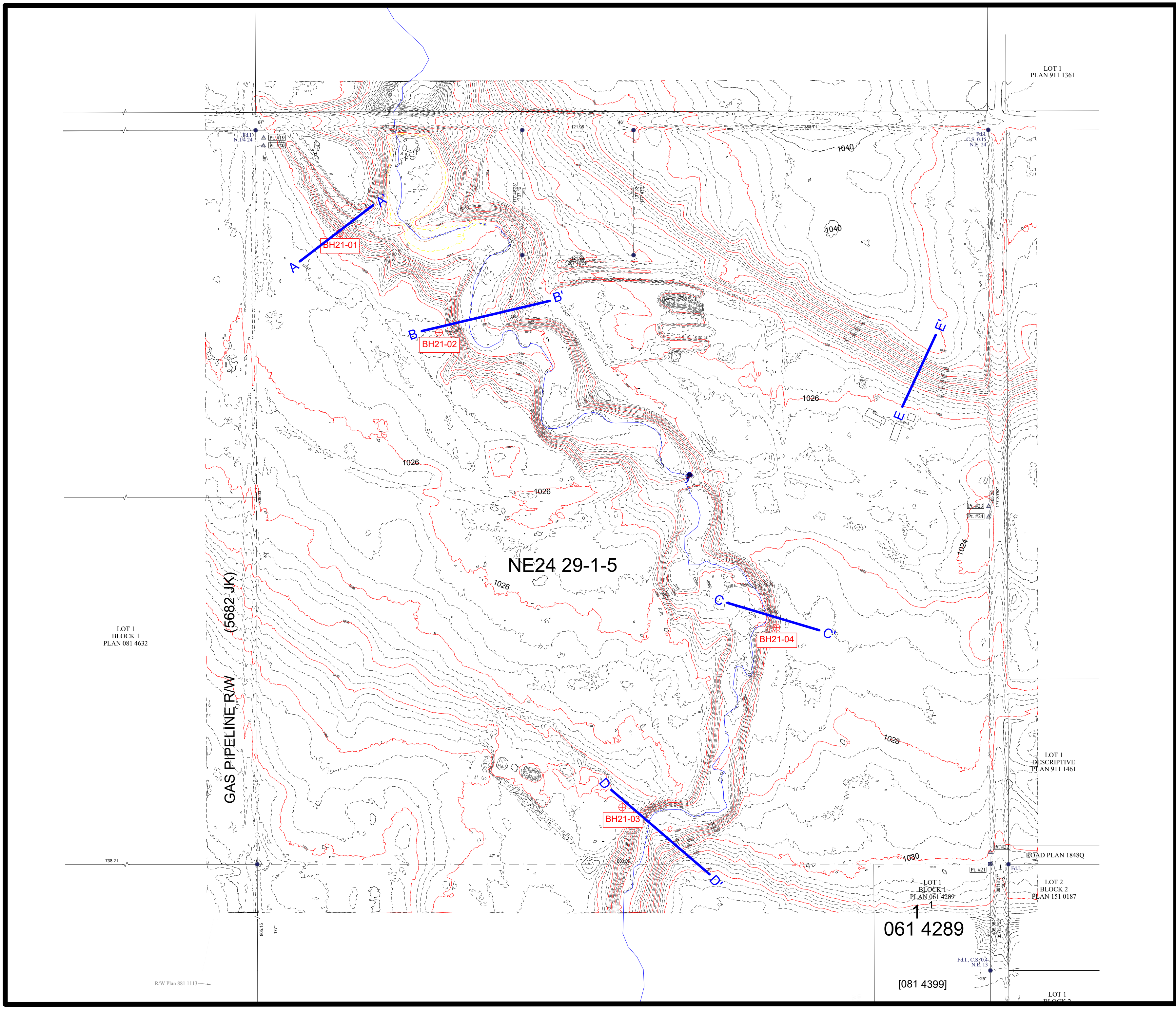
NOTE:

Portion of Government of Alberta Provincial Base Map (2016).

APPIN DEVELOPMENTS INC.

PROPOSED CARSTAIRS SUBDIVISION
(NE 1/4 24-029-01 W5M)
LOCATION MAP

SCALE N.T.S.	Project No. AE37269	FIGURE 1



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LEGEND

PROFESSIONAL SEAL

SITE PLAN

APPIN DEVELOPMENTS INC.

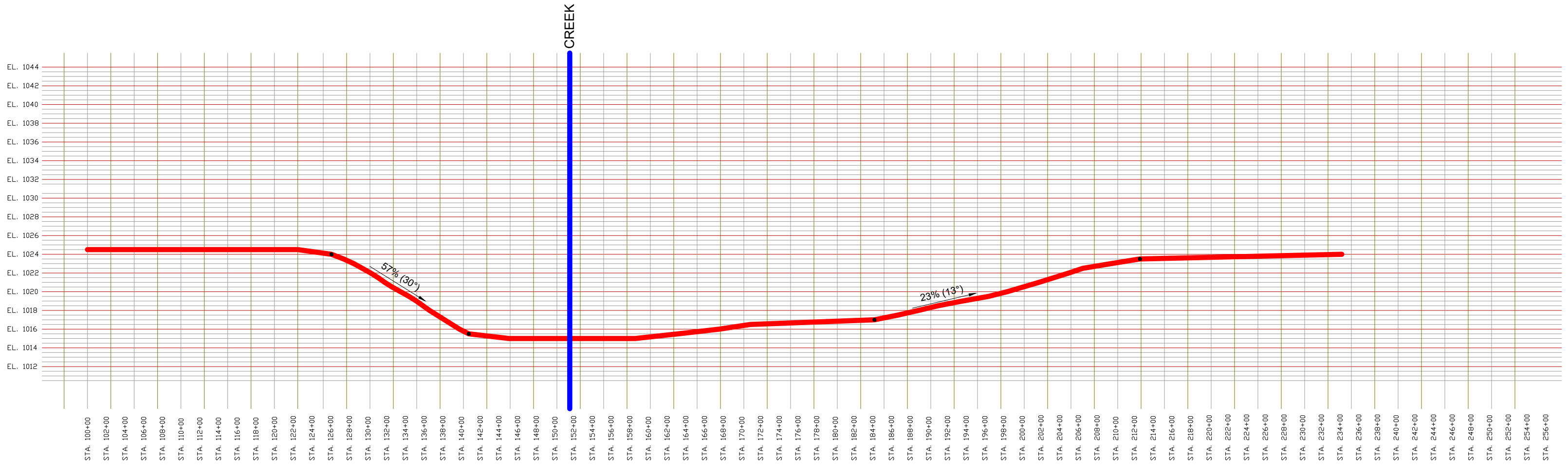
DESIGNED BY:
DRAWN BY: GL
REVIEWED BY: (AS PER OQM) LK
PROJECT NUMBER: AE37269

REV	DESCRIPTION	BY	DATE
00	SLOPE STABILITY ANALYSIS	GL	2021-05-07

SHEET NO: G-01

SCALE: NTS

SECTION A-A'



SECTION B-B'

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LEGEND

PROFESSIONAL SEAL

SECTIONS A & B

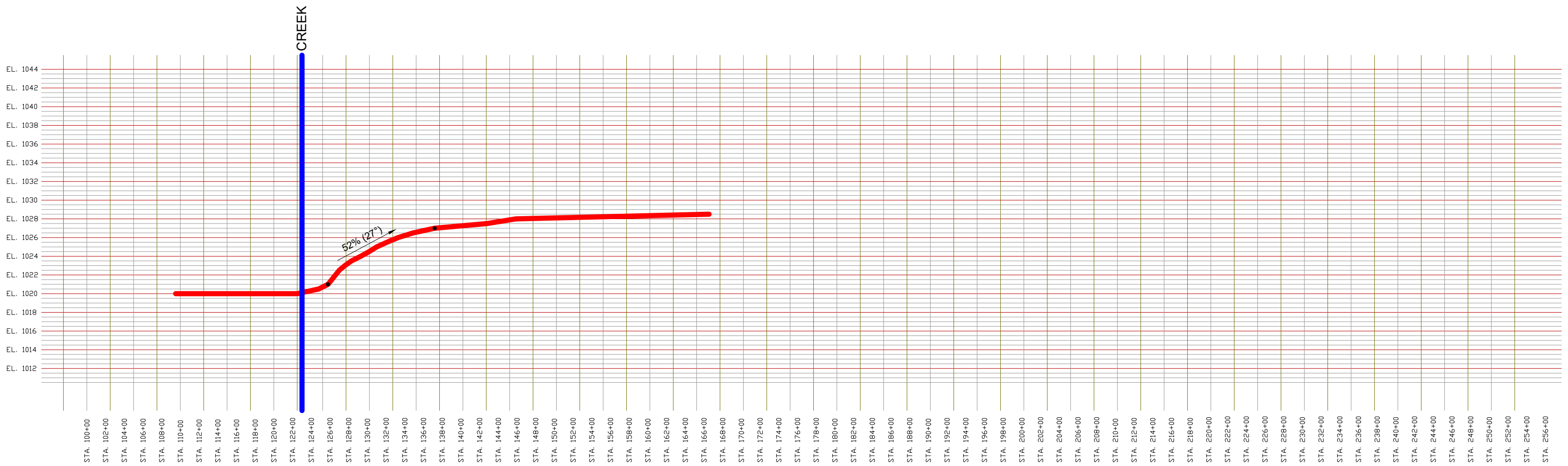
APPIN DEVELOPMENTS INC.

DESIGNED BY:
DRAWN BY: GL
REVIEWED BY: LK
(AS PER OQM)
PROJECT NUMBER: AE37269

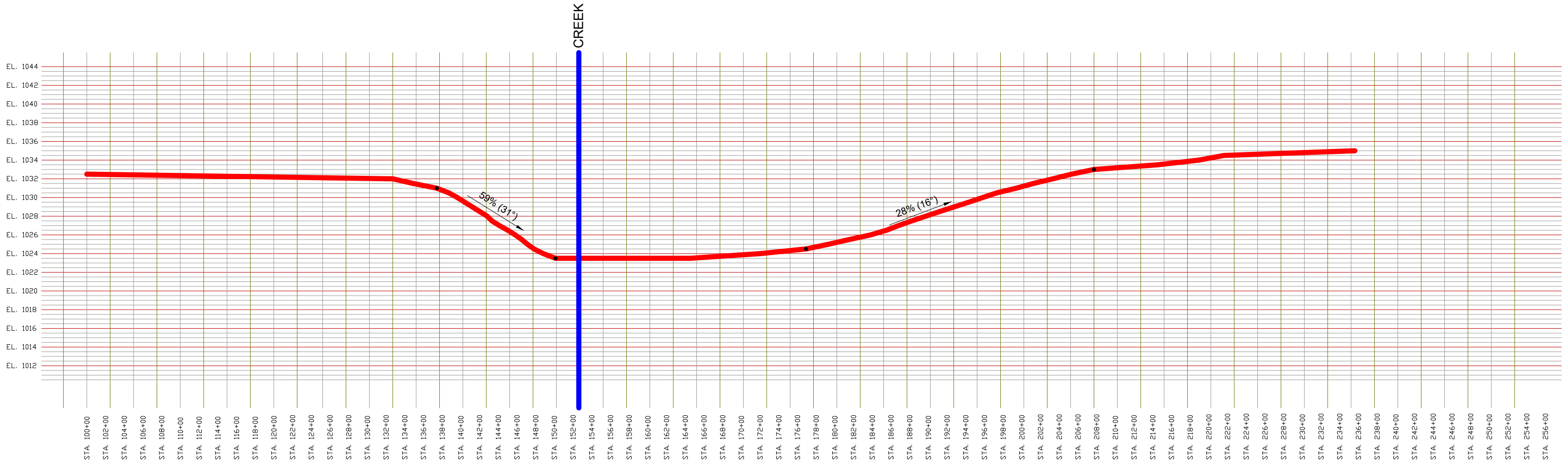
00	SLOPE STABILITY ANALYSIS	GL	2021-05-07
REV	**DESCRIPTION**	**BY**	**DATE**

SHEET NO: G-02

SCALE: NTS



SECTION C-C'



SECTION D-D'

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LEGEND

PROFESSIONAL SEAL

SECTIONS C & D

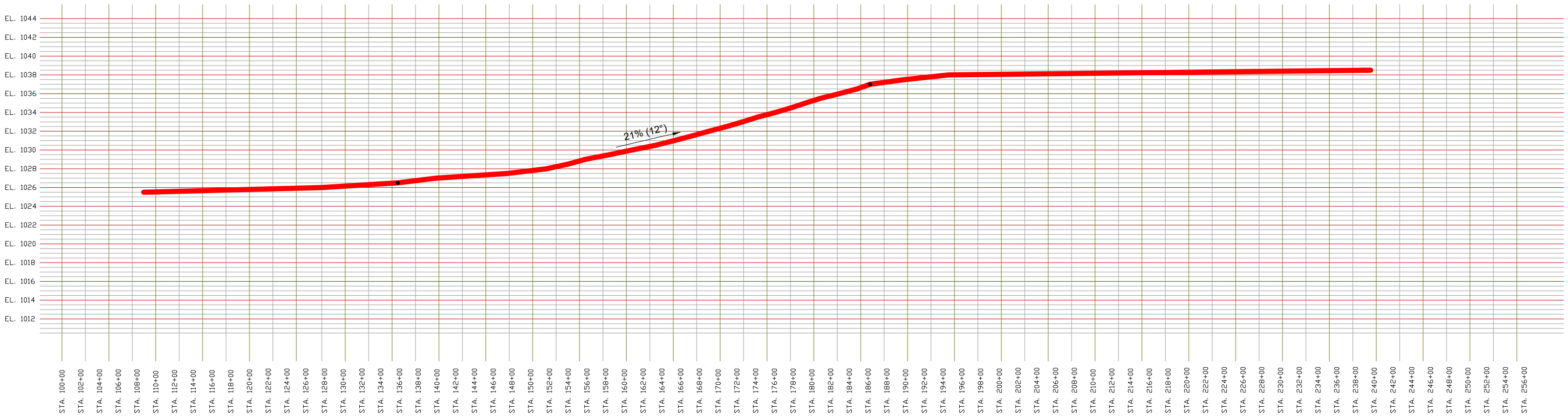
APPIN DEVELOPMENTS INC.

DESIGNED BY:
DRAWN BY: GL
REVIEWED BY: LK
(AS PER OQM)
PROJECT NUMBER: AE37269

00	SLOPE STABILITY ANALYSIS	GL	2021-05-07
REV	**DESCRIPTION**	**BY**	**DATE**

SHEET NO: G-03

SCALE: NTS



SECTION E-E'

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LEGEND

PROFESSIONAL SEAL

SECTIONS E

APPIN DEVELOPMENTS INC.

DESIGNED BY:	
DRAWN BY:	GL
REVIEWED BY: (AS PER OQM)	LK
PROJECT NUMBER:	AE37269

00	SLOPE STABILITY ANALYSIS	GL	2021-05-07
REV	**DESCRIPTION**	**BY**	**DATE**

SHEET NO: G-04

SCALE: NTS

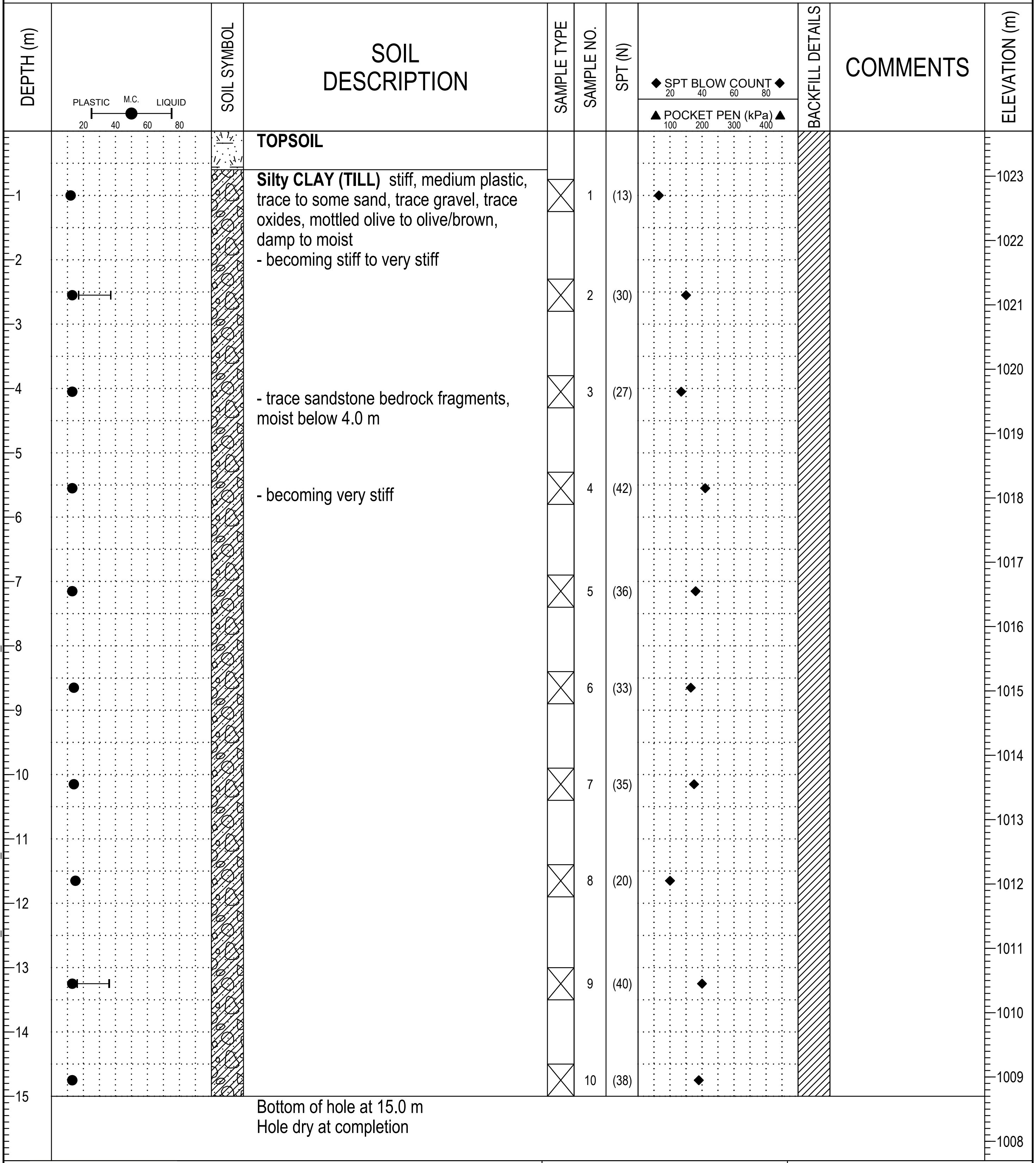
Appendix B

Borehole Logs

PROJECT: Carstairs Subdivision	CONTRACTOR: Mobile Augers & Research Ltd.	BOREHOLE NO: BH21-01
CLIENT: Appin Construction Ltd.	EQUIPMENT: Truck Mounted Drill Rig	PROJECT NO: AE37269
LOCATION: Crossfield, AB (NE 24-029-01 W5M)	METHOD: Solid Stem Auger	ELEVATION: 1023.7 m (approx.)

SAMPLE TYPE: ■ GRAB SAMPLE ◿ NO RECOVERY ☒ SPT ⊟ A-CASING ▥ SHELBY TUBE ◫ ROCK CORE

BACKFILL TYPE: ■ BENTONITE ◘ PEA GRAVEL ▥ SLOUGH ⁙ GROUT ▨ DRILL CUTTINGS ░ SAND (10/20)



TOPSOIL

Silty CLAY (TILL) stiff, medium plastic, trace to some sand, trace gravel, trace oxides, mottled olive to olive/brown, damp to moist
- becoming stiff to very stiff

- trace sandstone bedrock fragments, moist below 4.0 m

- becoming very stiff

SAMPLE NO.	SPT (N)
1	(13)
2	(30)
3	(27)
4	(42)
5	(36)
6	(33)
7	(35)
8	(20)
9	(40)
10	(38)

Bottom of hole at 15.0 m
Hole dry at completion

METRO REPORT B/F WITH DRILL CUTTINGS AE37269_2021 REVIEW_CARSTAIRS SUBDIVISION.GPJ GEER_ENG.GDT 11-5-21

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Abbotsford, British Columbia
Canada V2S 7W8
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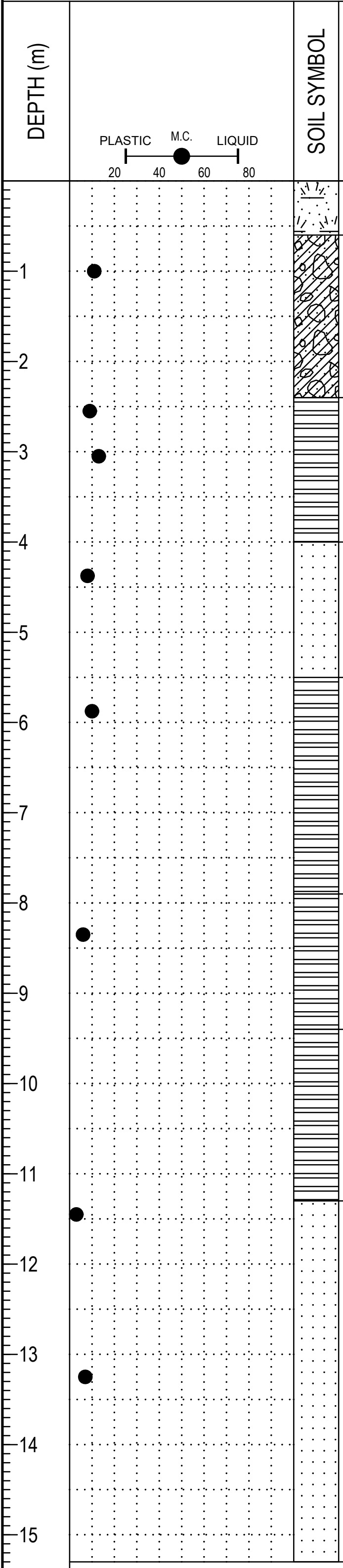
BOREHOLE LOG

LOGGED BY: Lorne King	COMPLETION DEPTH: 15.00 m
CHECKED BY: Brian Mylleville	COMPLETION DATE: 12-4-21
REVISION NO.: Draft (Rev 1)	

PROJECT: Carstairs Subdivision	CONTRACTOR: Mobile Augers & Research Ltd.	BOREHOLE NO: BH21-02
CLIENT: Appin Construction Ltd.	EQUIPMENT: Truck Mounted Drill Rig	PROJECT NO: AE37269
LOCATION: Crossfield, AB (NE 24-029-01 W5M)	METHOD: Solid Stem Auger	ELEVATION: 1025.5 m (approx.)

SAMPLE TYPE: ■ GRAB SAMPLE ▨ NO RECOVERY ☒ SPT ▤ A-CASING ▥ SHELBY TUBE ▮ ROCK CORE

BACKFILL TYPE: ■ BENTONITE ◂ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ░ SAND (10/20)



SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO.	SPT (N)	CORE REC (%)	CORE CONDITION	CORE RQD (%)	BACKFILL DETAILS	COMMENTS
TOPSOIL							▨	
Silty CLAY (TILL) stiff, low plastic, some sand to sandy, trace gravel, trace cobble, trace oxides, olive, moist	☒	1	(12)				▨	
SHALE (BEDROCK) "extremely weak to very weak" (R0-R1), low plastic, highly weathered, fractured, tan to brown, damp - interbedded mudstone layers (<300 mm thick) below 3.7 m	☒	2	(46)				▨	
	■	3					▨	- set casing, switched to HQ coring
	▮	4		98		24	▨	
SANDSTONE (BEDROCK) "very weak to weak" (R1-R2), fine grained, fractured, moderately to highly weathered, olive, damp	▮	5		85		10	▨	
MUDSTONE (BEDROCK) "extremely weak" (R0), medium plastic, highly weathered, olive to olive/brown, damp to moist	▮						▨	γ_w = 24.3 kN/m^3
	▮	6		81		52	▨	
SHALE (BEDROCK) "extremely weak" (R0), high plastic (bentonitic), highly weathered, grey to grey/blue, damp to moist	▮						▨	γ_w = 22.5 kN/m^3
MUDSTONE (BEDROCK) "extremely weak to very weak" (R0-R1), medium plastic, highly weathered, olive/brown, damp to moist - lost water circulation	▮	7		100		72	▨	
SANDSTONE (BEDROCK) "very weak to weak" (R1-R2), fine grained, fractured, moderately weathered, salt and pepper, damp - lost water circulation	▮						▨	Q$_u$ = 36.3 MPa
	▮						▨	Q$_u$ = 26.5 MPa
- interbedded siltstone layers (<300 mm thick), brown/black below 14.0 m	▮	8		98		67	▨	

Bottom of hole at 15.3 m
Hole wet at completion

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ROCK CORE LOG

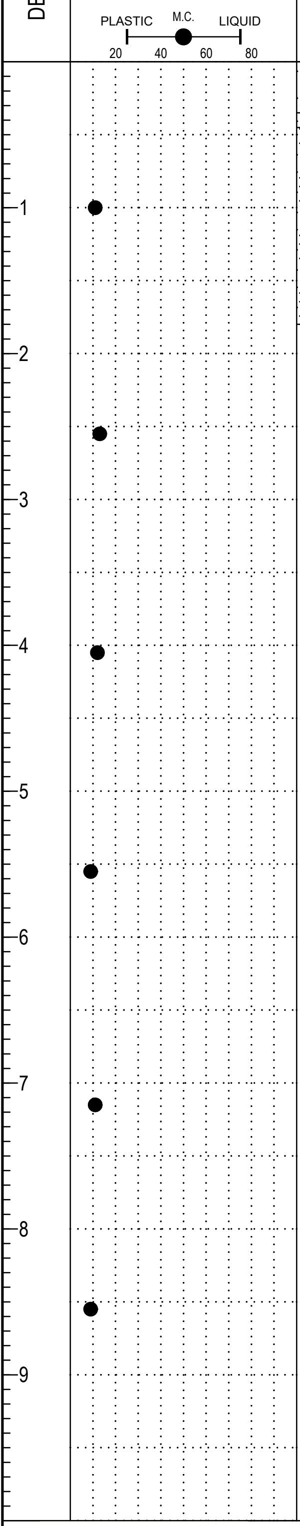
LOGGED BY: Lorne King	COMPLETION DEPTH: 15.3 m
CHECKED BY: Brian Mylleville	COMPLETION DATE: 12-4-21
REVISION NO.: Draft (Rev 1)	Page 1 of 1

METRO ROCK CORE LOG B/F DRILL CUTTINGS AE37269_2021 REVIEW_CARSTAIRS SUBDIVISION.GPJ GEER (ROCK CORE).GDT 11-5-21

PROJECT: Carstairs Subdivision	CONTRACTOR: Mobile Augers & Research Ltd.	BOREHOLE NO: BH21-03
CLIENT: Appin Construction Ltd.	EQUIPMENT: Truck Mounted Drill Rig	PROJECT NO: AE37269
LOCATION: Crossfield, AB (NE 24-029-01 W5M)	METHOD: Solid Stem Auger	ELEVATION: 1032.2 m (approx.)

SAMPLE TYPE: ■ GRAB SAMPLE ◸ NO RECOVERY ☒ SPT ⊟ A-CASING ▥ SHELBY TUBE ▮ ROCK CORE

BACKFILL TYPE: ■ BENTONITE ◂ PEA GRAVEL ▥ SLOUGH ⁙ GROUT ▨ DRILL CUTTINGS ░ SAND (10/20)



SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO.	SPT (N)	CORE REC (%)	CORE CONDITION	CORE RQD (%)	BACKFILL DETAILS	COMMENTS
TOPSOIL							▨	
Silty SAND compact to dense, fine grained, poorly graded, olive, damp	☒	1	(29)				▨	
SANDSTONE (BEDROCK) "extremely weak" (R0), trace clay, fine grained, highly to completely weathered, olive, damp	☒	2	(88)				▨	
	☒	3	(82)				▨	
	☒	4	(86)				▨	
	☒	5	(88)				▨	
- auger grinding							▨	
- interbedded mudstone layers (<300 mm thick) below 8.8 m	▮	6		78		51	▨	- set casing, switched to HQ coring Q_u = 3.6 MPa

ELEVATION (m): 1032, 1031, 1030, 1029, 1028, 1027, 1026, 1025, 1024, 1023

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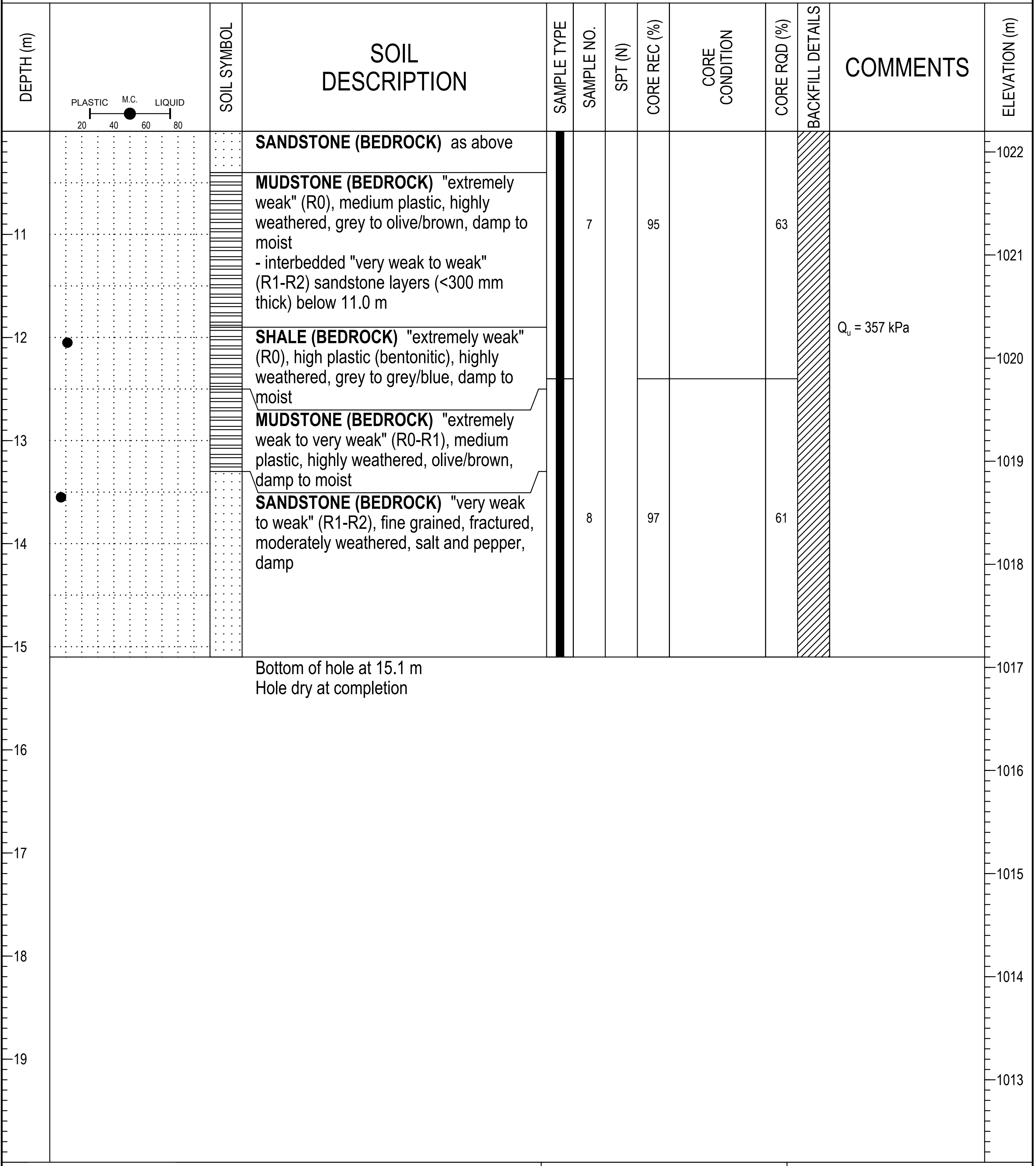
ROCK CORE LOG

LOGGED BY: Lorne King	COMPLETION DEPTH: 15.1 m
CHECKED BY: Brian Mylleville	COMPLETION DATE: 13-4-21
REVISION NO.: Draft (Rev 1)	

PROJECT: Carstairs Subdivision	CONTRACTOR: Mobile Augers & Research Ltd.	BOREHOLE NO: BH21-03
CLIENT: Appin Construction Ltd.	EQUIPMENT: Truck Mounted Drill Rig	PROJECT NO: AE37269
LOCATION: Crossfield, AB (NE 24-029-01 W5M)	METHOD: Solid Stem Auger	ELEVATION: 1032.2 m (approx.)

SAMPLE TYPE: ■ GRAB SAMPLE ◩ NO RECOVERY ☒ SPT ⊟ A-CASING ▥ SHELBY TUBE ◧ ROCK CORE

BACKFILL TYPE: ■ BENTONITE ◂ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ░ SAND (10/20)



SOIL DESCRIPTION	SAMPLE NO.	CORE REC (%)	CORE RQD (%)	COMMENTS
SANDSTONE (BEDROCK) as above				
MUDSTONE (BEDROCK) "extremely weak" (R0), medium plastic, highly weathered, grey to olive/brown, damp to moist - interbedded "very weak to weak" (R1-R2) sandstone layers (<300 mm thick) below 11.0 m	7	95	63	
SHALE (BEDROCK) "extremely weak" (R0), high plastic (bentonitic), highly weathered, grey to grey/blue, damp to moist				Q_u = 357 kPa
MUDSTONE (BEDROCK) "extremely weak to very weak" (R0-R1), medium plastic, highly weathered, olive/brown, damp to moist				
SANDSTONE (BEDROCK) "very weak to weak" (R1-R2), fine grained, fractured, moderately weathered, salt and pepper, damp	8	97	61	

Bottom of hole at 15.1 m
Hole dry at completion

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Unit 20, 3275 McCallum Road
Abbotsford, British Columbia
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T: (604) 385-4244

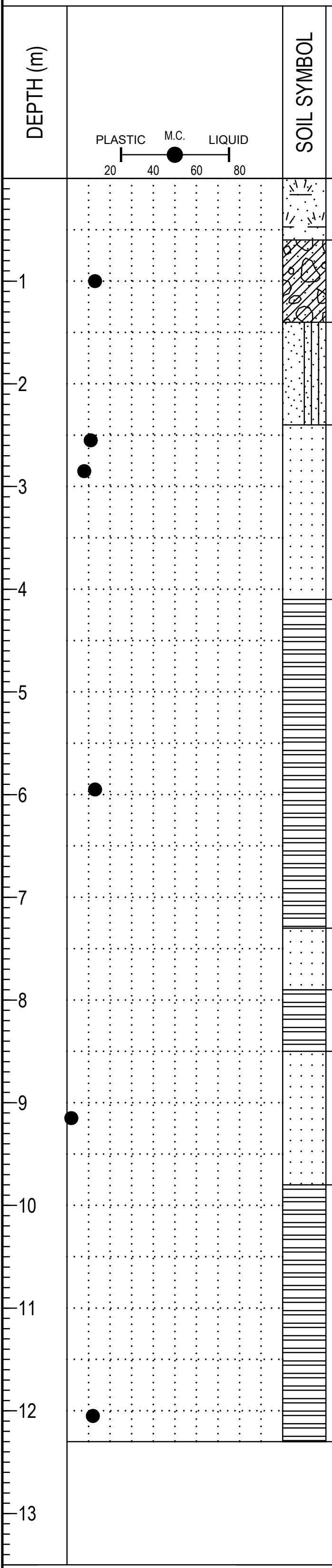
ROCK CORE LOG

LOGGED BY: Lorne King	COMPLETION DEPTH: 15.1 m
CHECKED BY: Brian Mylleville	COMPLETION DATE: 13-4-21
REVISION NO.: Draft (Rev 1)	Page 2 of 2

PROJECT: Carstairs Subdivision	CONTRACTOR: Mobile Augers & Research Ltd.	BOREHOLE NO: BH21-04
CLIENT: Appin Construction Ltd.	EQUIPMENT: Truck Mounted Drill Rig	PROJECT NO: AE37269
LOCATION: Crossfield, AB (NE 24-029-01 W5M)	METHOD: Solid Stem Auger	ELEVATION: 1027 m (approx.)

SAMPLE TYPE: ■ GRAB SAMPLE ◪ NO RECOVERY ☒ SPT ⊟ A-CASING ▥ SHELBY TUBE ▮ ROCK CORE

BACKFILL TYPE: ■ BENTONITE ◂ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ░ SAND (10/20)



SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO.	SPT (N)	CORE REC (%)	CORE CONDITION	CORE RQD (%)	BACKFILL DETAILS	COMMENTS
TOPSOIL							▨	
Silty CLAY (TILL) stiff, medium plastic, some sand to sandy, trace gravel, trace coaly pieces, olive, damp to moist	☒	1	(14)				▨	
Silty SAND compact to dense, trace clay, fine grained, poorly graded, olive, damp							▨	
SANDSTONE (BEDROCK) "extremely weak to very weak" (R0-R1), fine grained, highly weathered, brown to tan, damp - auger grinding	☒ ■ ▮	2 3 4	(31)	 25		 0	▨	- set casing, switched to HQ coring
MUDSTONE (BEDROCK) "extremely weak" (R0), medium plastic, highly weathered, grey to olive/grey, damp to moist - interbedded highly to completely weathered brown layers below 6.2 m	▮	5		96		55	▨	Q_u = 401 kPa
SANDSTONE (BEDROCK) "very weak to weak" (R1-R2), fine grained, moderately to higly weathered, grey, damp	▮	6		100		66	▨	
SHALE (BEDROCK) "extremely weak" (R0), high plastic (bentonitic), highly weathered, grey to grey/blue, damp to moist	▮						▨	Q_u = 64.8 MPa
SANDSTONE (BEDROCK) "very weak to weak" (R1-R2), fine grained, fractured, moderately weathered, grey, damp	▮						▨	
MUDSTONE (BEDROCK) "extremely weak" (R0), medium to high plastic, highly weathered, grey to olive/brown, moist - interbedded "very weak to weak" (R1-R2) sandstone layers (<300 mm thick) below 11.3 m	▮	7		93		66	▨	Q_u = 486 kPa

Bottom of hole at 12.3 m
Hole dry at completion

ELEVATION (m): 1026, 1025, 1024, 1023, 1022, 1021, 1020, 1019, 1018, 1017, 1016, 1015, 1014

METRO TESTING + ENGINEERING

Metro Testing + Engineering Ltd.
Unit 20, 3275 McCallum Road
Abbotsford, British Columbia
Canada V2S 7W8
T: (604) 385-4244

ROCK CORE LOG

LOGGED BY: Lorne King	COMPLETION DEPTH: 12.3 m
CHECKED BY: Brian Mylleville	COMPLETION DATE: 13-4-21
REVISION NO.: Draft (Rev 1)	Page 1 of 1

Appendix C

Terms & Symbols

Modified Unified Classification System for Soils

EXPLANATION OF TERMS AND SYMBOLS

The terms and symbols used on the borehole logs to summarize the results of the field investigation and subsequent laboratory testing are described in these pages.

It should be noted that materials, boundaries, and conditions have been established only at borehole locations at the time of investigation and are not necessarily representative of subsurface conditions elsewhere across the site.

Test Data

Data obtained during the field investigation and from laboratory testing are shown at the appropriate depth interval.

Abbreviations, graphic symbols, and relevant test method designations are as follows:

*C	Consolidation Test	TV	Torvane Shear Strength
D_R	Relative Density	VS	Vane Shear Strength
k	Permeability Coefficient	w	Natural Moisture Content
*MA	Mechanical Grain Size Analysis & Hydrometer	w_l	Liquid Limit
N	Standard Penetration Test (SPT)	w_p	Plastic Limit
N_d	Dynamic Cone Penetration Test	E_f	Unit Strain at Failure
NP	Non Plastic Soil	ϒ	Unit Weight of Soil or Rock
PP	Pocket Penetrometer Test	$ϒ_d$	Dry Unit Weight of Soil or Rock
*q	Triaxial Compression Test	ρ	Density of Soil or Rock
q_u	Unconfined Compressive Strength	ρ_d	Dry Density of Soil or Rock
*SB	Shearbox Test	C_u	Undrained Shear Strength
SO_4	Concentration of Water-Soluble Sulphate	!	Seepage
*ST	Swelling Test	▼	Observed Water Level

*The results of these tests are generally reported separately.

Soils are classified and described according to their engineering properties and behaviour.

The soil of each stratum is described using the Unified Soil Classification Sytem[1] which has been modified slightly to recognize inorganic clay of medium plasticity.

The use of modifying adjectives may be employed to define the actual or estimated percentage range by weight of minor components. This is similar to a system developed by D. Burmister[2].

Relative Density and Consistency

Cohesionless Soils

Relative Density	*SPT 'N' Values*
Very Loose	0 – 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	> 50

Cohesive Soils

Consistency	*SPT 'N' Value*	*Undrained Shear Strength, C_u (kPa)*
Very Soft	0 – 2	0 – 10
Soft	2 – 4	10 – 25
Firm	4 – 8	25 – 50
Stiff	8 – 15	50 – 100
Very Stiff	15 – 30	100 – 200
Hard	> 30	> 200

Standard Penetration Resistance ('N' Value)

The number of blows by a 63.6 kg hammer dropped 760 mm to drive a 50 mm diameter open sampler attached to "A" drill rods for a distance of 300 mm.

---

[1] Corps of Engineers, U.S. Army. 1953. "The Unified Soil Classification System." Vicksburg, MS: Waterways Experiment Station.

[2] Burmister, D. 1970. "Suggested Methods of Test for Identification of Soils," in *Special Procedures for Testing Soil and Rock for Engineering Purposes: 5th Ed.*, p 311-323. West Conshohocken, PA: ASTM International.

MODIFIED UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISION			GROUP SYMBOL	GRAPH SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS (> 50% BY WEIGHT LARGER THAN NO. 200 SIEVE)	GRAVELS >50% OF COARSE LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (<5% FINES)	GW	●	WELL-GRADED GRAVELS, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} \geq 4 \quad C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})} = 1\ to\ 3$	
			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	FINES EXCEED 12%	ATTERBERG LIMITS BELOW 'A' LINE OR PI < 4
			GC	▨	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS BELOW 'A' LINE, PI > 7
	SANDS >50% OF FINES SMALLER THAN NO.4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)	SW	⁘	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} \geq 6 \quad C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})} = 1\ to\ 3$	
			SP	⁘	POORLY GRADED SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY SANDS (WITH SOME FINES)	SM	⁘	SILTY SANDS, SAND-SILT MIXTURES	FINES EXCEED 12%	ATTERBERG LIMITS BELOW 'A' LINE OR PI < 4
			SC	▨	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS BELOW 'A' LINE, PI > 7
FINE-GRAINED SOILS (>50% BY WEIGHT PASSES NO. 200 SIEVE)	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 BASED ON 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 FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER 'F'. (E.G. SF FOR 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, OFTEN FIBROUS TEXTURE	

SPECIAL SYMBOLS

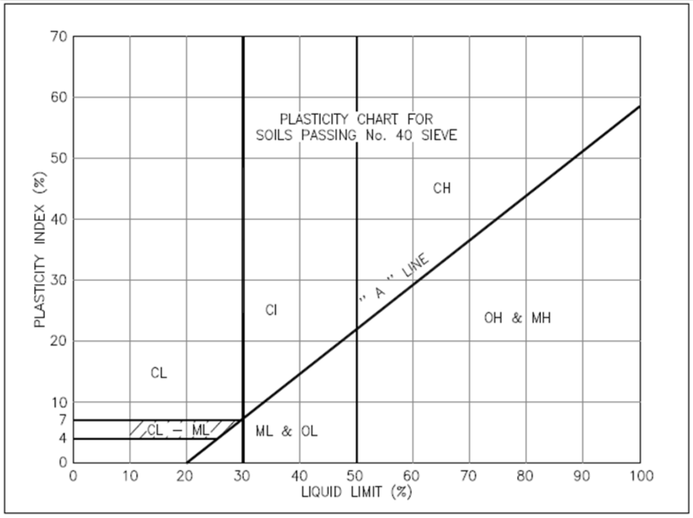
LIMESTONE	▦	OILSAND	▩
SANDSTONE	⁘	SHALE	▤
SILTSTONE	▤	FILL	▩

SOIL DESCRIPTIONS

SOIL COMPONENTS	*SIZE*	PERCENT RANGE DEFINITIONS FOR MINOR COMPONENTS	
GRAVEL			
COARSE	76mm - 19mm	*PERCENT*	*DESCRIPTOR*
FINE	19mm - 4.75mm	50 - 35	AND
SAND		35 - 20	SOME
COARSE	4.75mm - 2.0mm	20 - 10	LITTLE
MEDIUM	2.0mm - 425µm	10 - 1	TRACE
FINE	425µm - 75µm		
SILT (NON PLASTIC)	75µm - 2µm		
CLAY (PLASTIC)	< 2µm		

OVERSIZED MATERIAL

ROUNDED OR SUBROUNDED	*ANGULAR OR SUBANGULAR*
BOULDERS > 200mm	ROCK FRAGMENTS > 75mm
COBBLES 200mm - 75mm	ROCKS > 0.75 m^3



1. ALL SIEVE SIZES MENTIONED ON THIS PAGE ARE STANDARD AS PER ASTM E11

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Appendix D

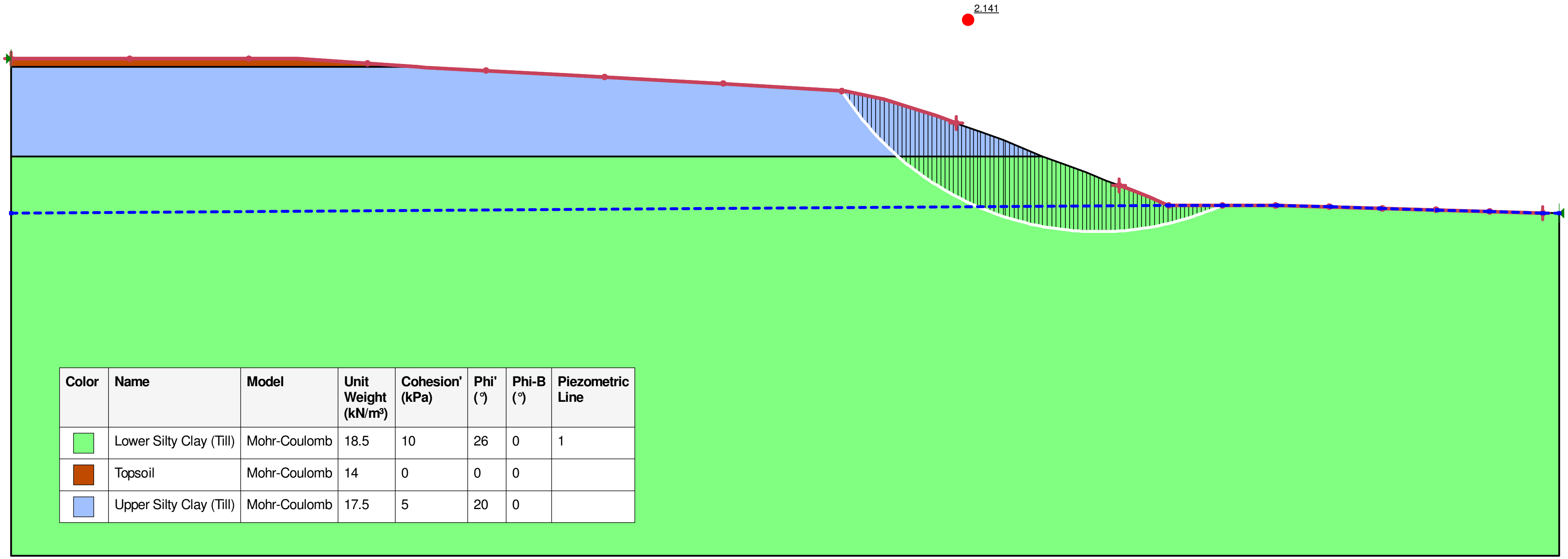
Laboratory Test Results

Appendix E

NBC Seismic Hazard Calculation

Appendix F

Summary of Slope Stability Plots

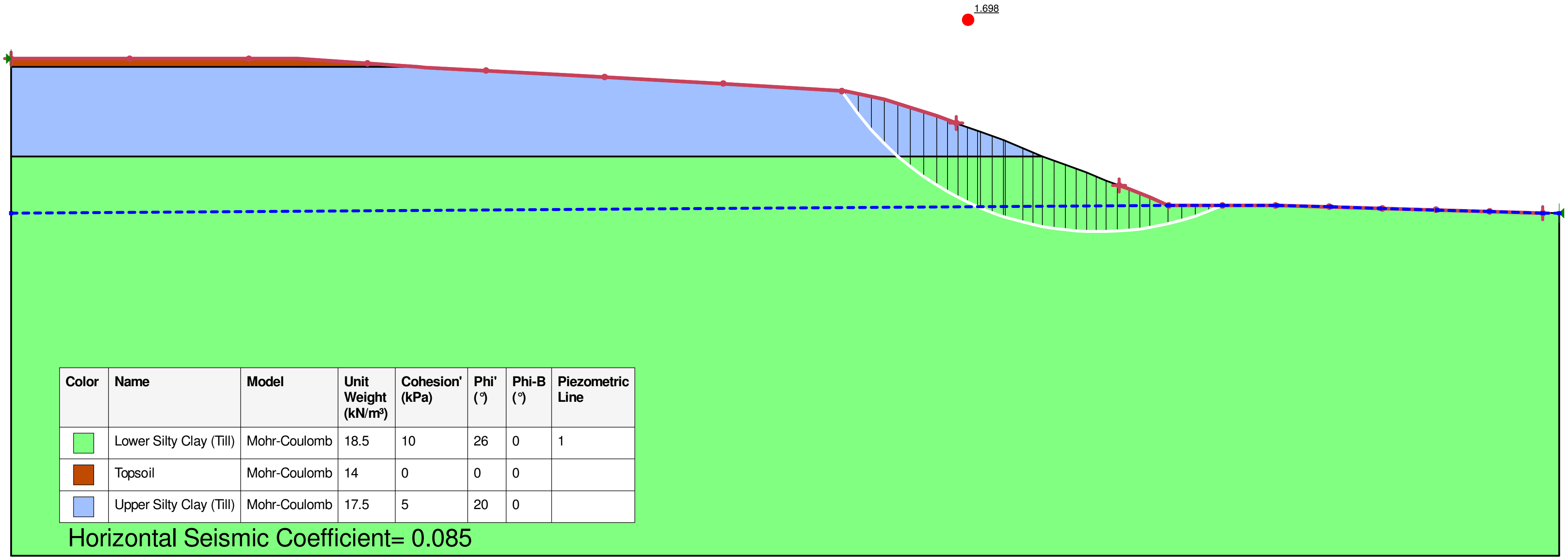


Color	Name	Model	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
🟩	Lower Silty Clay (Till)	Mohr-Coulomb	18.5	10	26	0	1
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	
🟦	Upper Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	

Static Analysis

Section A-A'

05/17/2021



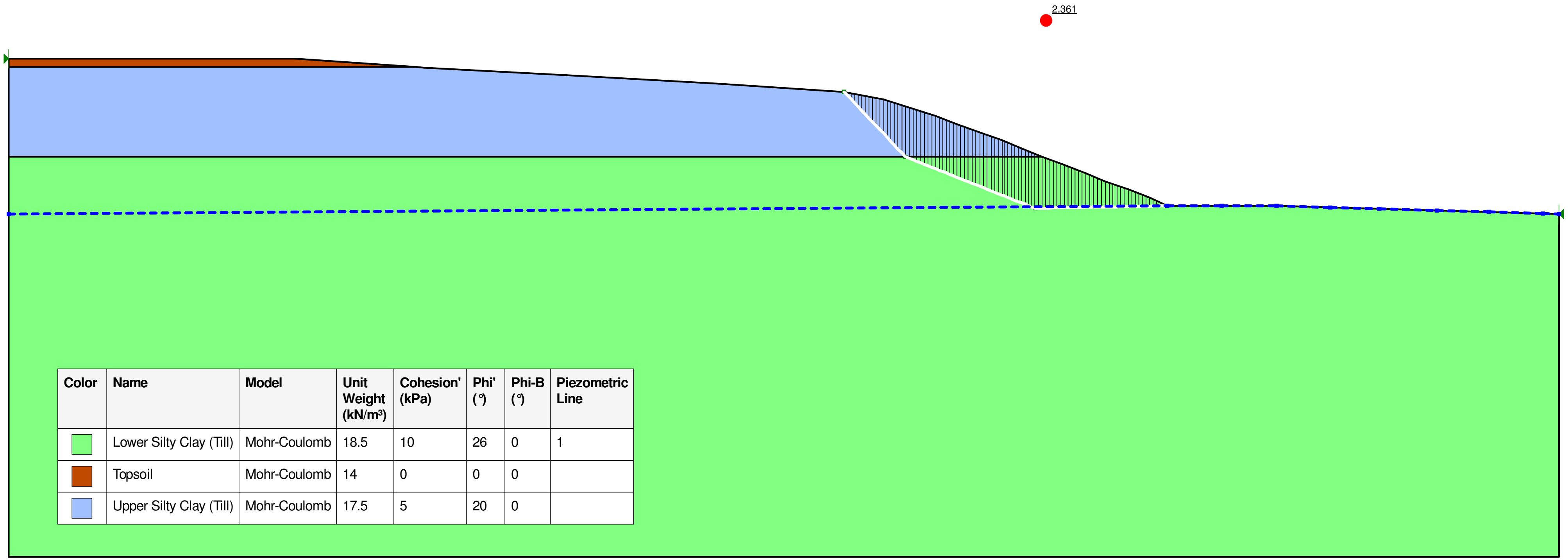
Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
🟩	Lower Silty Clay (Till)	Mohr-Coulomb	18.5	10	26	0	1
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	
🟦	Upper Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	

Horizontal Seismic Coefficient= 0.085

Seismic Analysis

Section A-A'

05/17/2021



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
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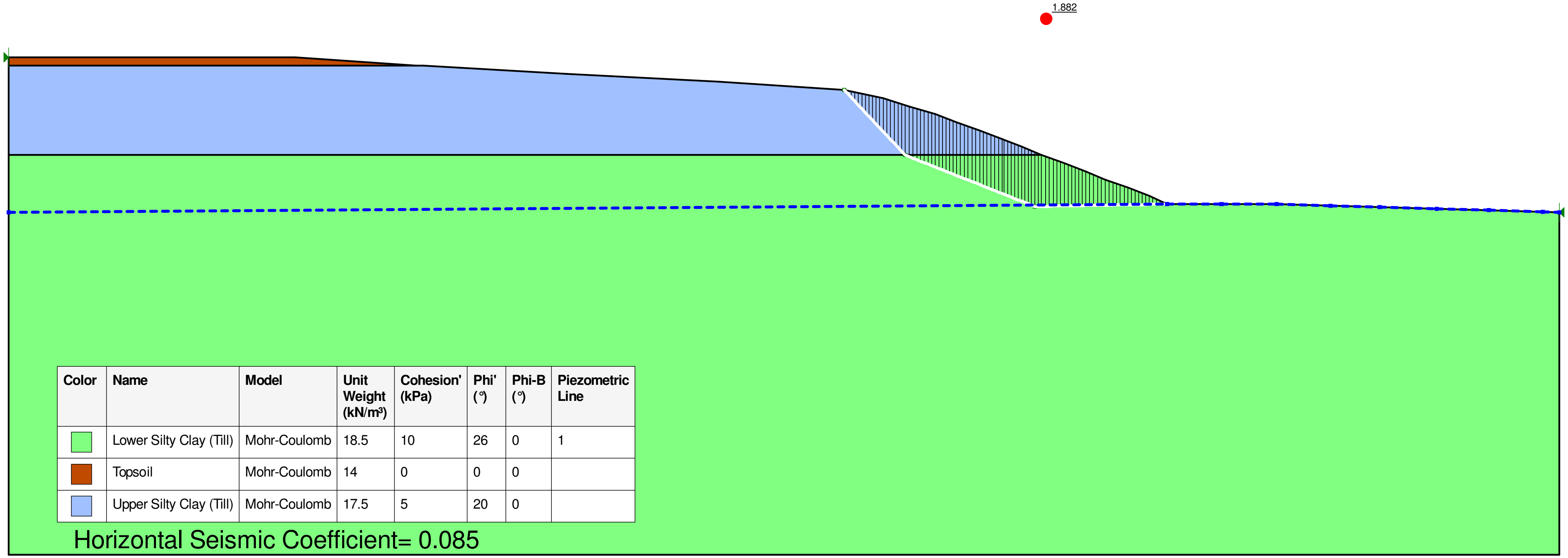
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	
🟦	Upper Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	

Static Analysis - Block Failure

Section A-A'

05/17/2021



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
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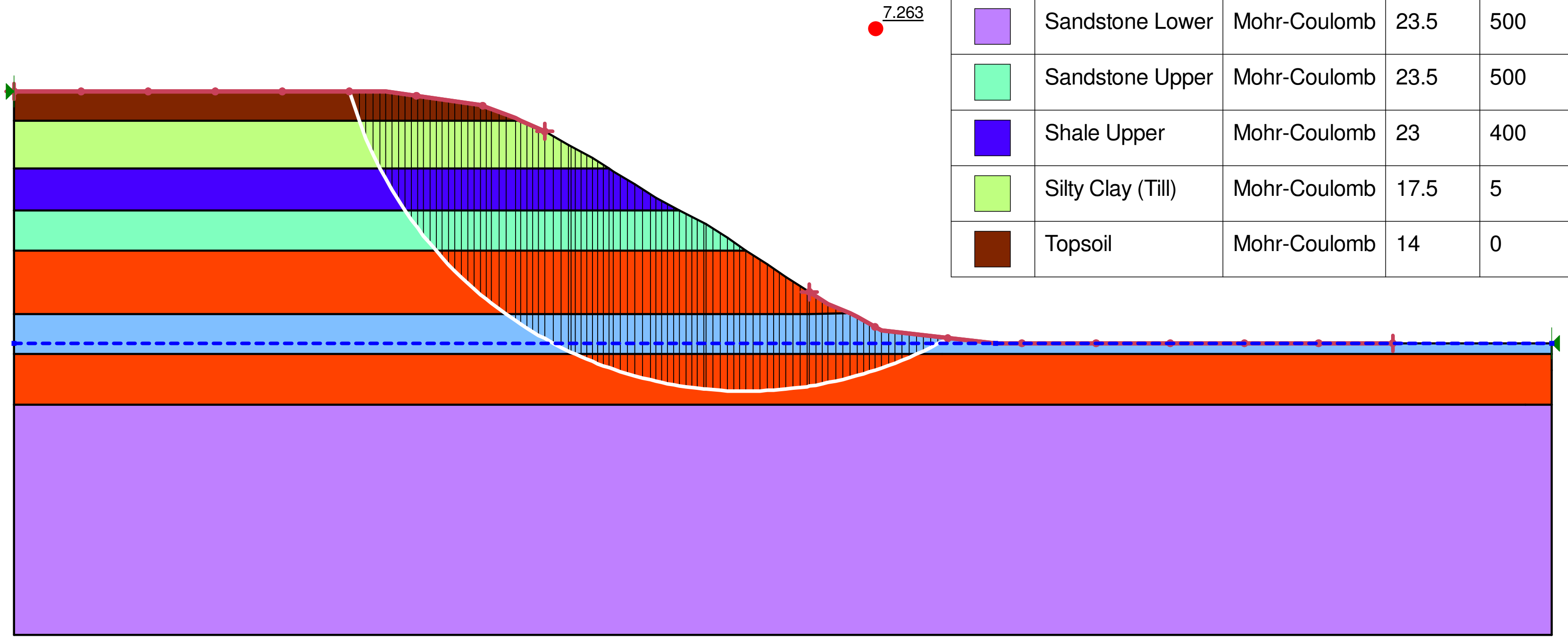
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	
🟦	Upper Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	

Horizontal Seismic Coefficient= 0.085

Seismic Analysis - Block Failure

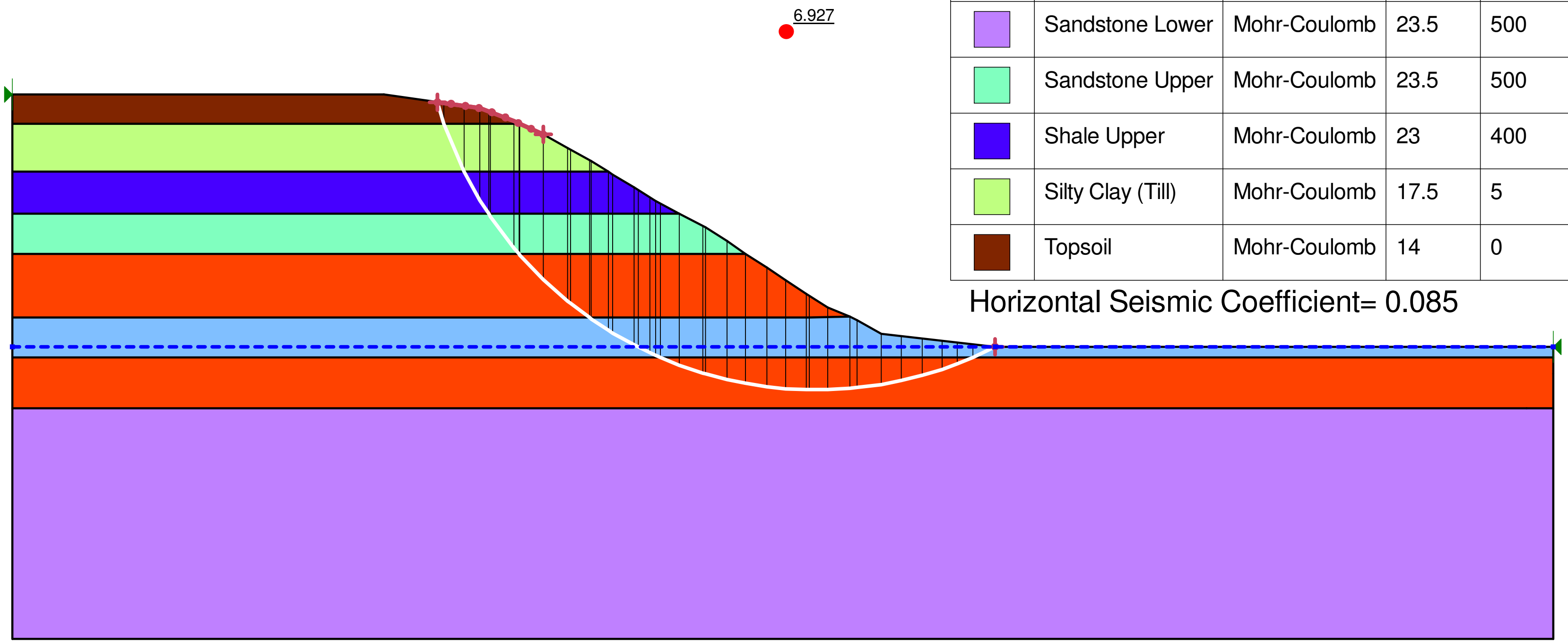
Section A-A'

05/14/2021



Color	Name	Model	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
🟦	Bentonitic Shale	Mohr-Coulomb	21.5	100	20	0	1
🟥	Mudstone	Mohr-Coulomb	22	150	26	0	1
🟪	Sandstone Lower	Mohr-Coulomb	23.5	500	30	0	1
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	
🟦	Shale Upper	Mohr-Coulomb	23	400	26	0	
🟩	Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Static Analysis
Section B-B'
05/17/2021



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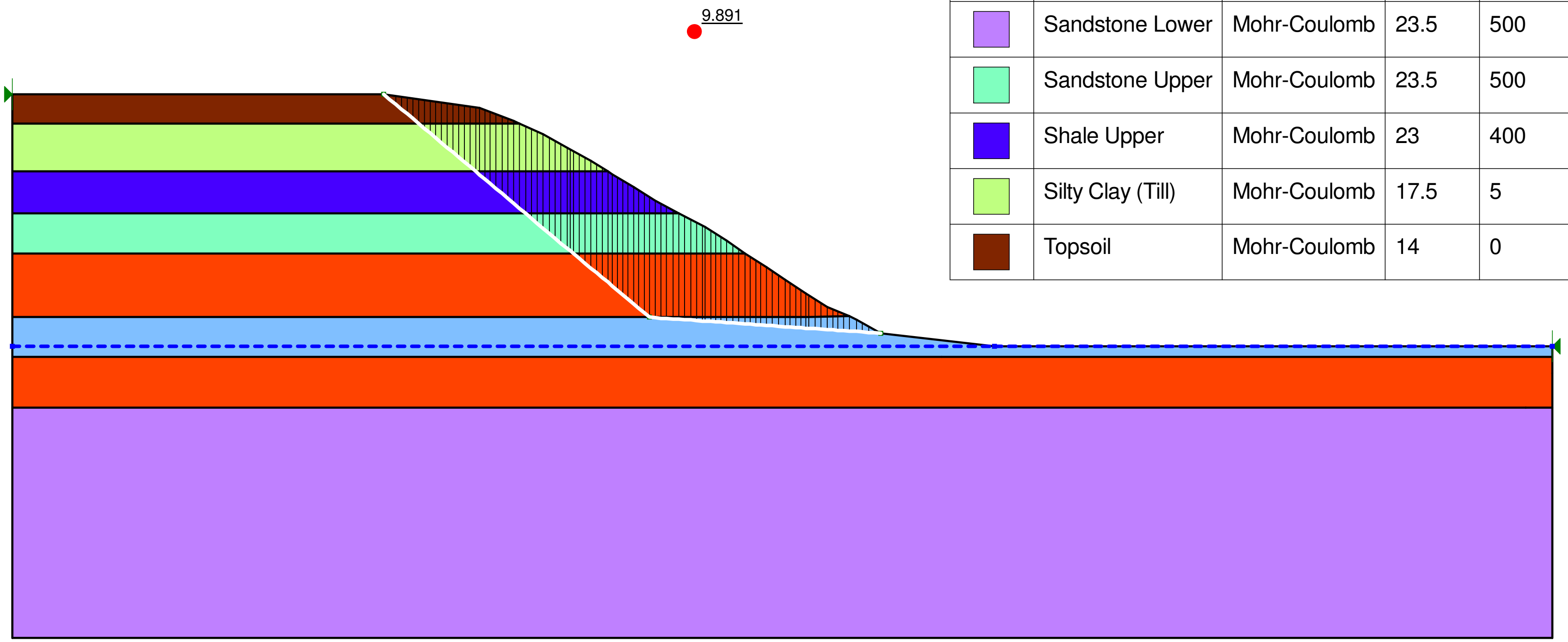
🟪	Sandstone Lower	Mohr-Coulomb	23.5	500	30	0	1
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	
🟦	Shale Upper	Mohr-Coulomb	23	400	26	0	
🟩	Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Horizontal Seismic Coefficient= 0.085

Seismic Analysis

Section B-B'

05/17/2021



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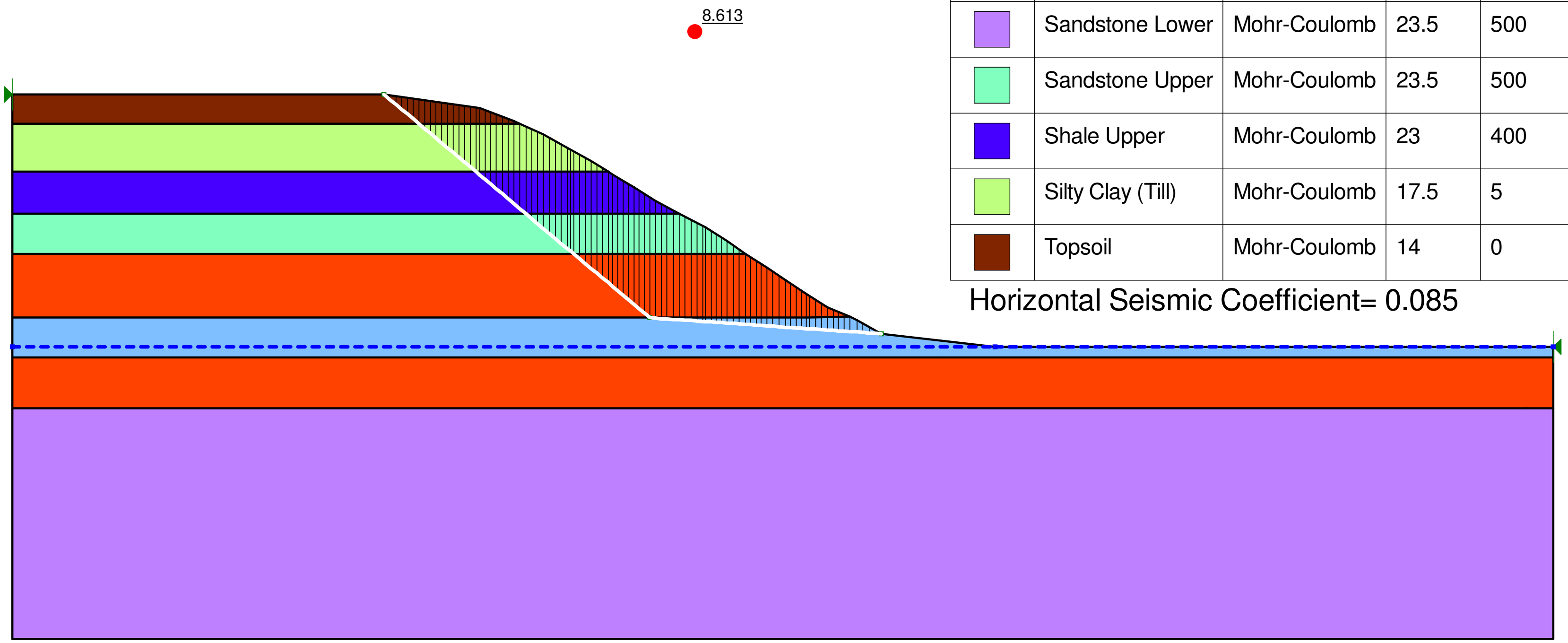
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	
🟦	Shale Upper	Mohr-Coulomb	23	400	26	0	
🟩	Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Static Analysis - Block Failure
Section B-B'
05/17/2021



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🟦	Shale Upper	Mohr-Coulomb	23	400	26	0	
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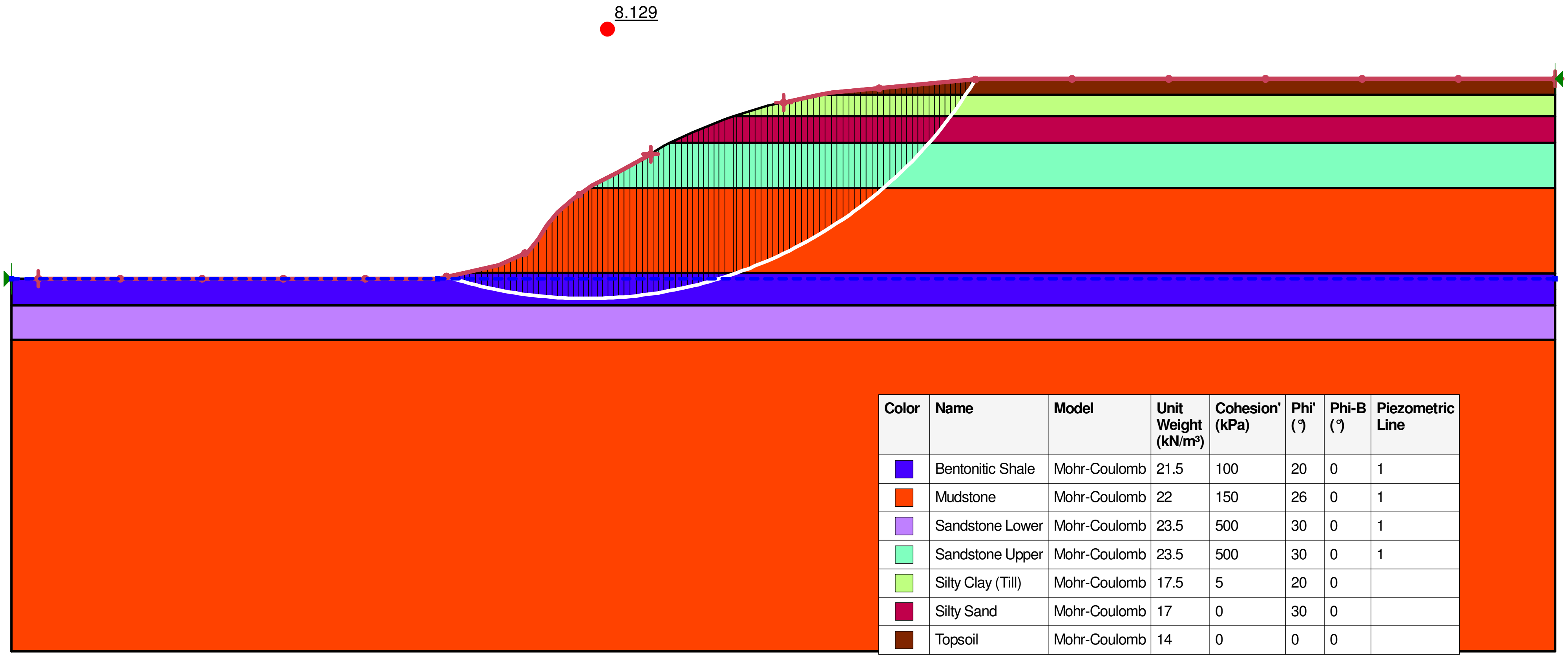
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Horizontal Seismic Coefficient= 0.085

Seismic Analysis - Block Failure

Section B-B'

05/17/2021



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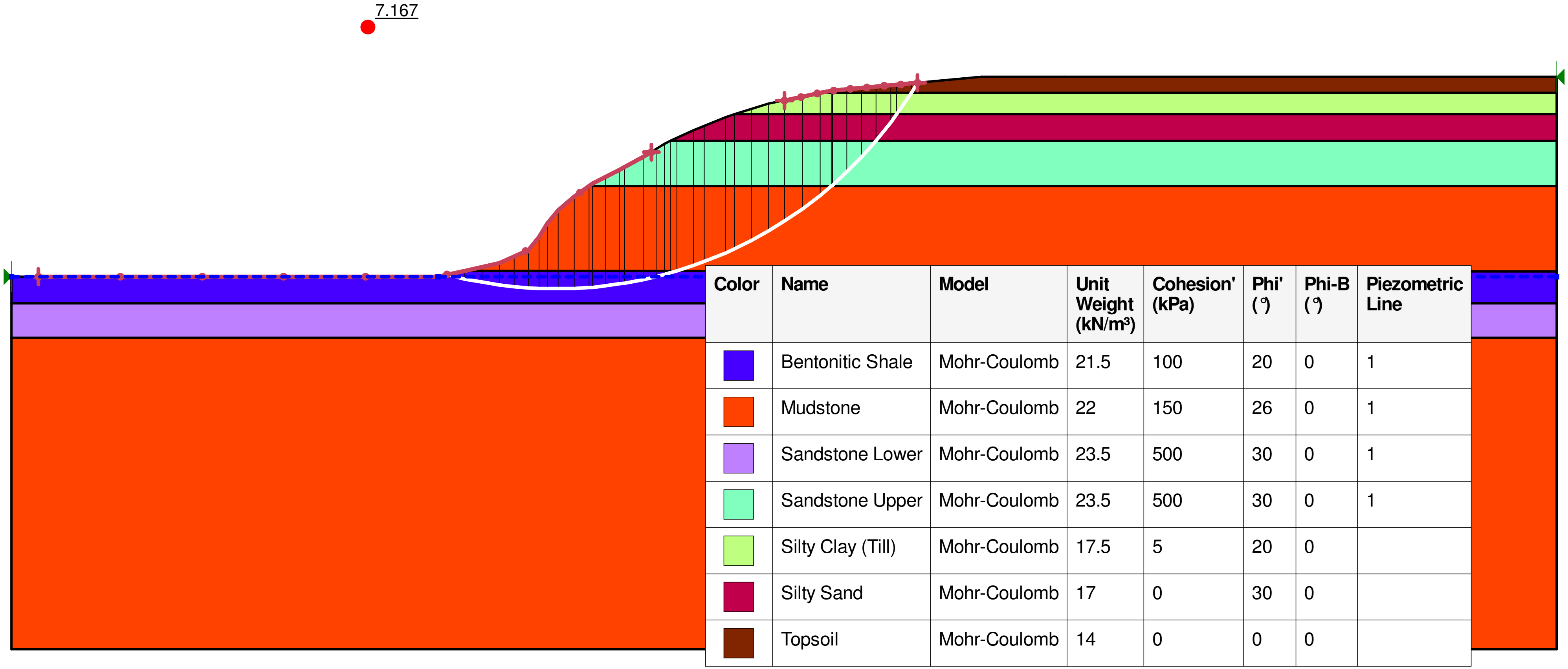
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	1
🟩	Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	
🟥	Silty Sand	Mohr-Coulomb	17	0	30	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Static Analysis

Section C-C''

05/17/2021



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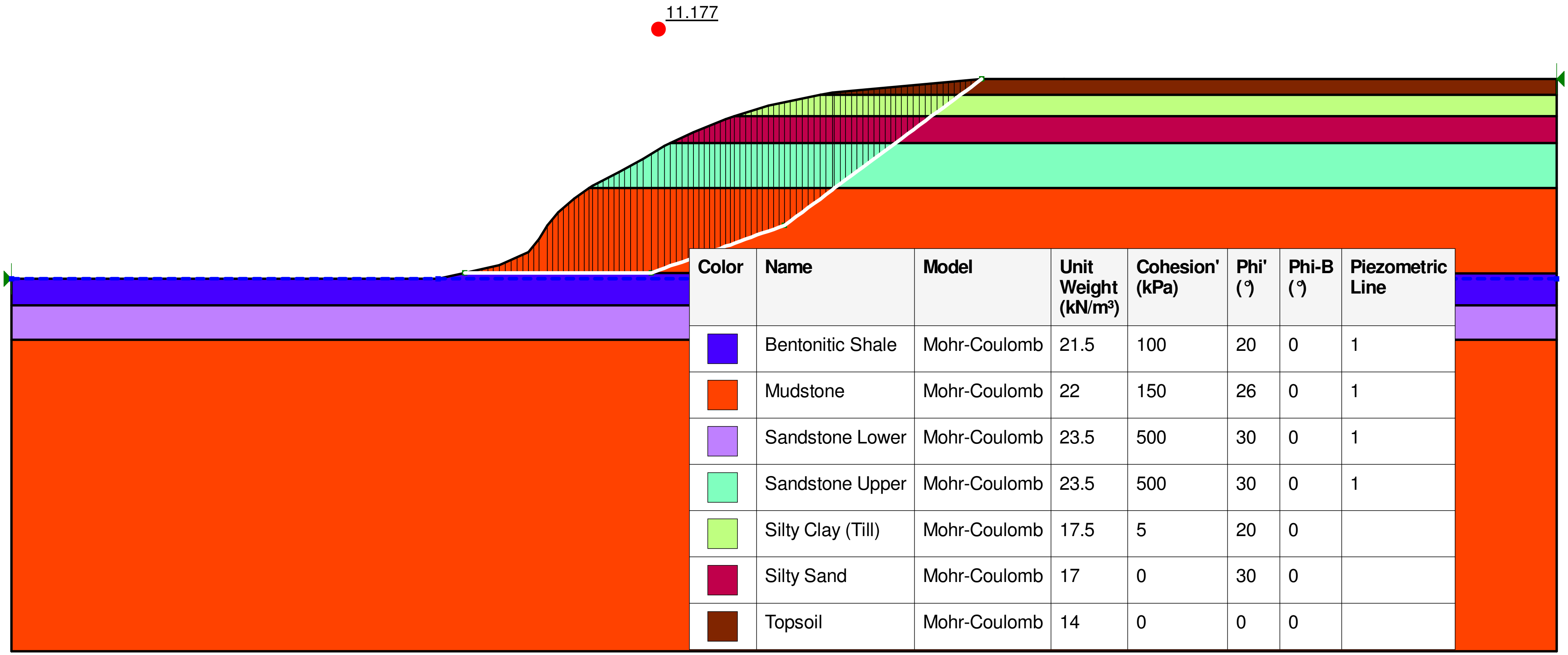
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	1
🟩	Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	
🟥	Silty Sand	Mohr-Coulomb	17	0	30	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Horizontal Seismic Coefficient= 0.085

Seismic Analysis

Section C-C''

05/17/2021



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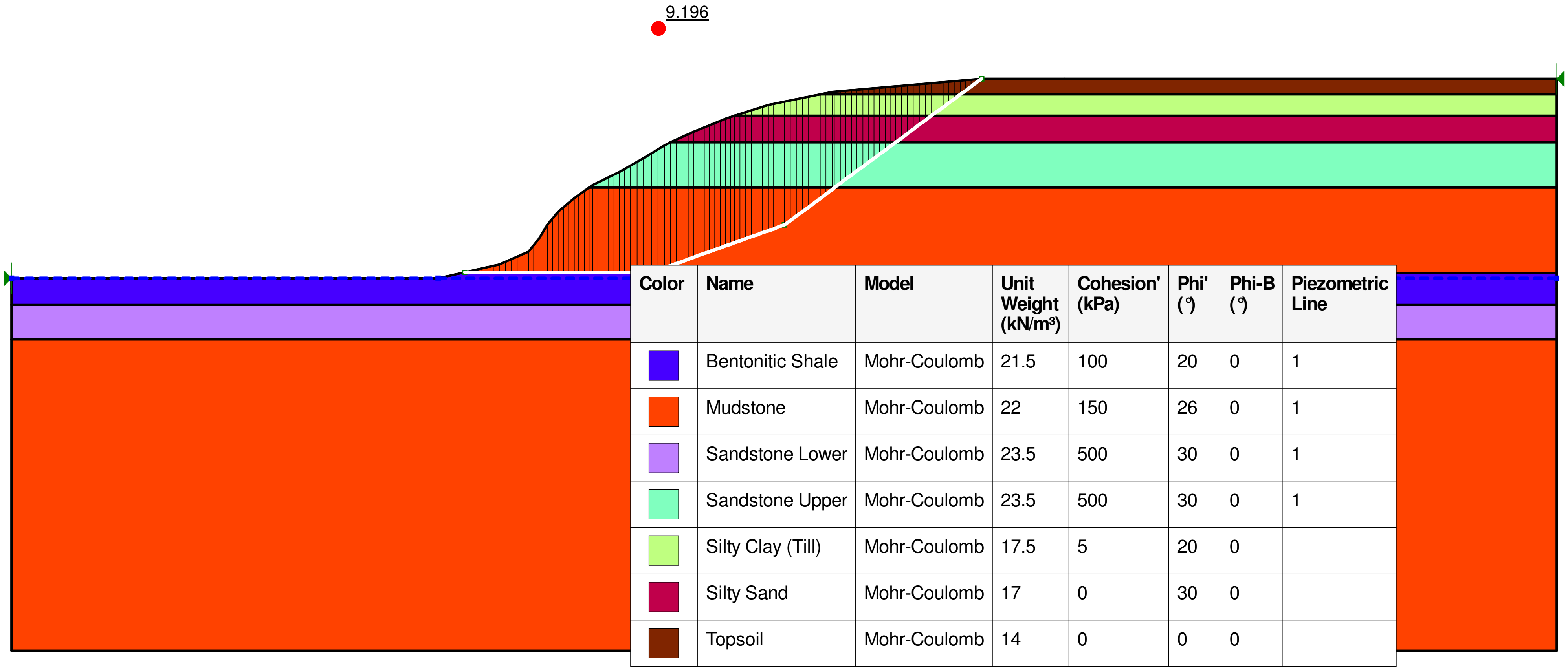
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	1
🟩	Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	
🟥	Silty Sand	Mohr-Coulomb	17	0	30	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Static Analysis Block Failure
Section C-C''
05/17/2021



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
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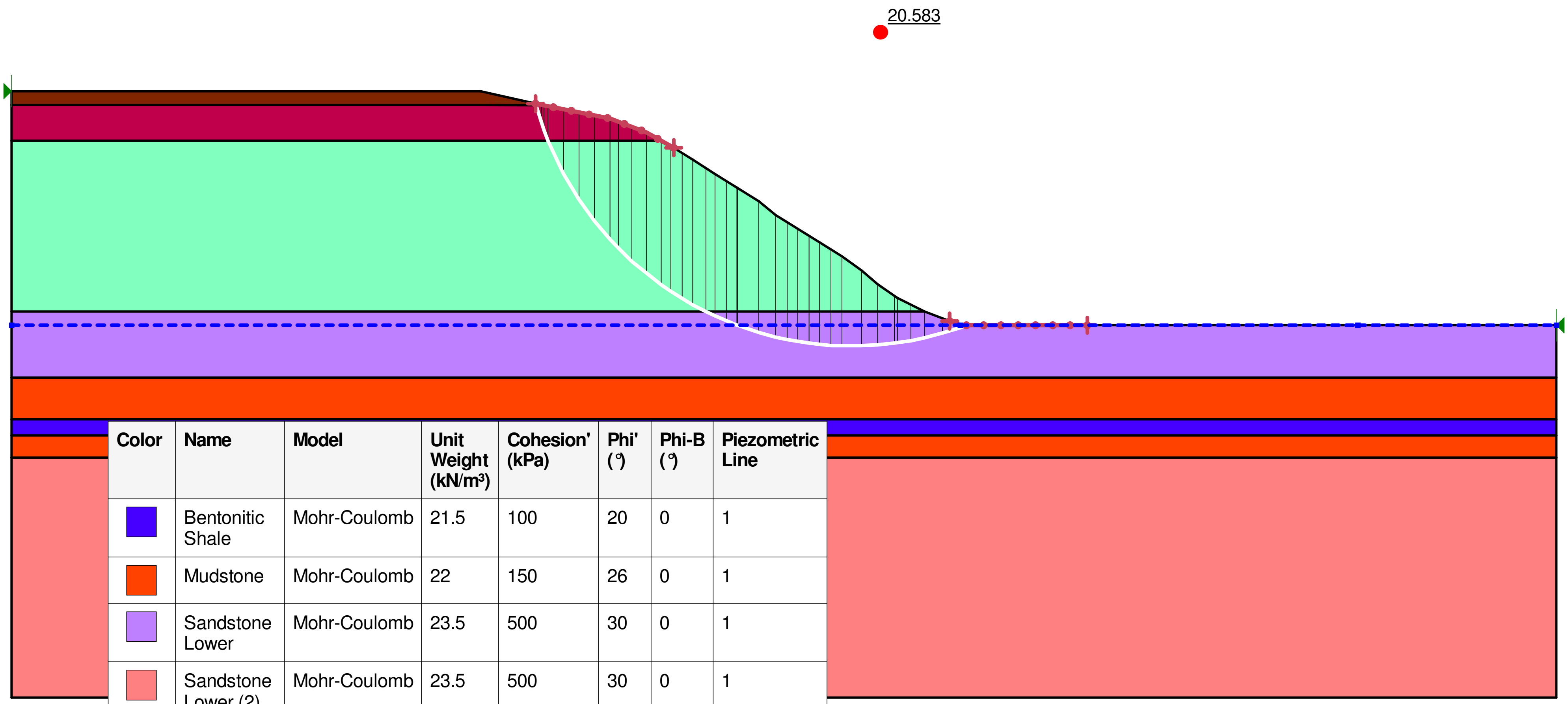
🟪	Sandstone Lower	Mohr-Coulomb	23.5	500	30	0	1
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	1
🟩	Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	
🟥	Silty Sand	Mohr-Coulomb	17	0	30	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Horizontal Seismic Coefficient= 0.085

Seismic Analysis Block Failure

Section C-C'

05/17/2021



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
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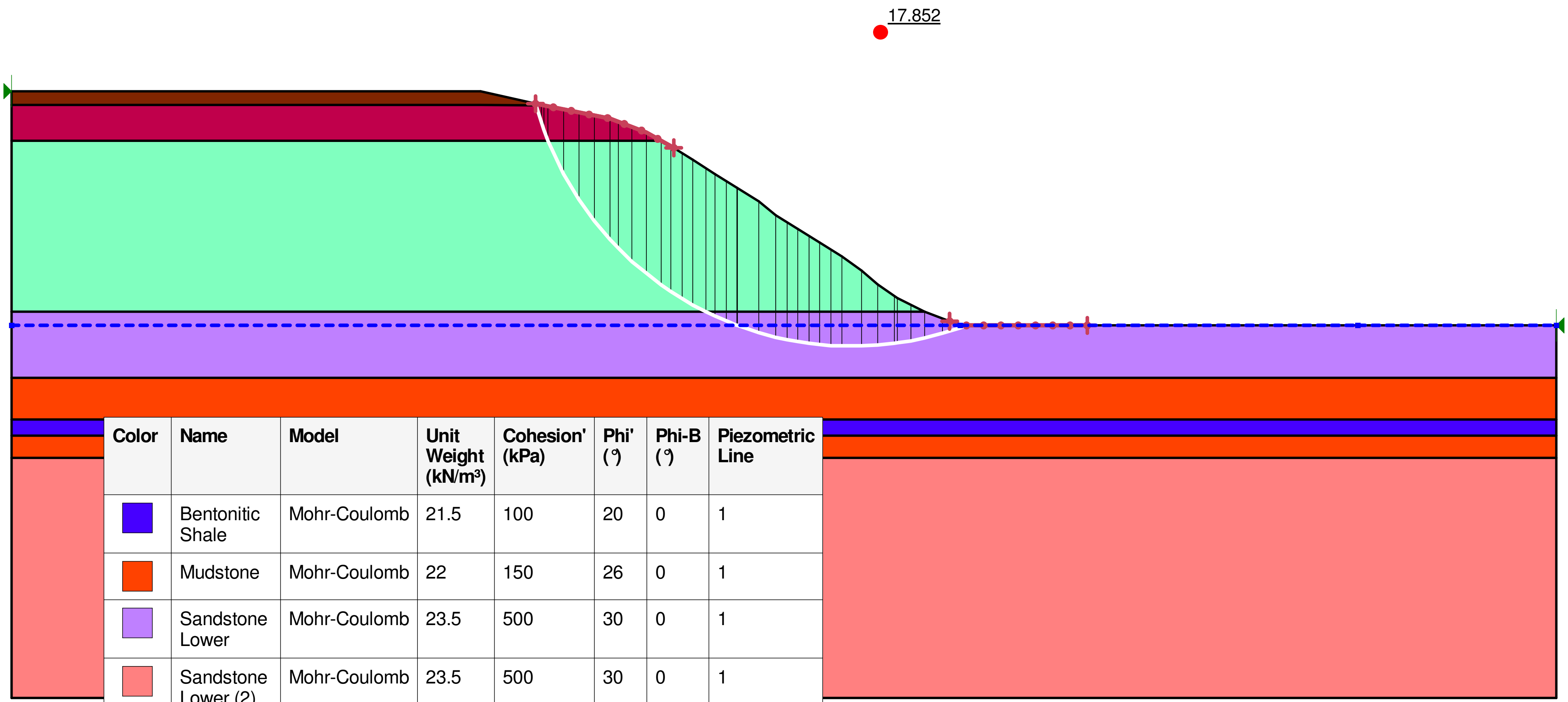
🟧	Mudstone	Mohr-Coulomb	22	150	26	0	1
🟪	Sandstone Lower	Mohr-Coulomb	23.5	500	30	0	1
🟥	Sandstone Lower (2)	Mohr-Coulomb	23.5	500	30	0	1
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	
🟥	Silty Sand	Mohr-Coulomb	17	0	30	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Static Analysis

Section D-D'

05/17/2021



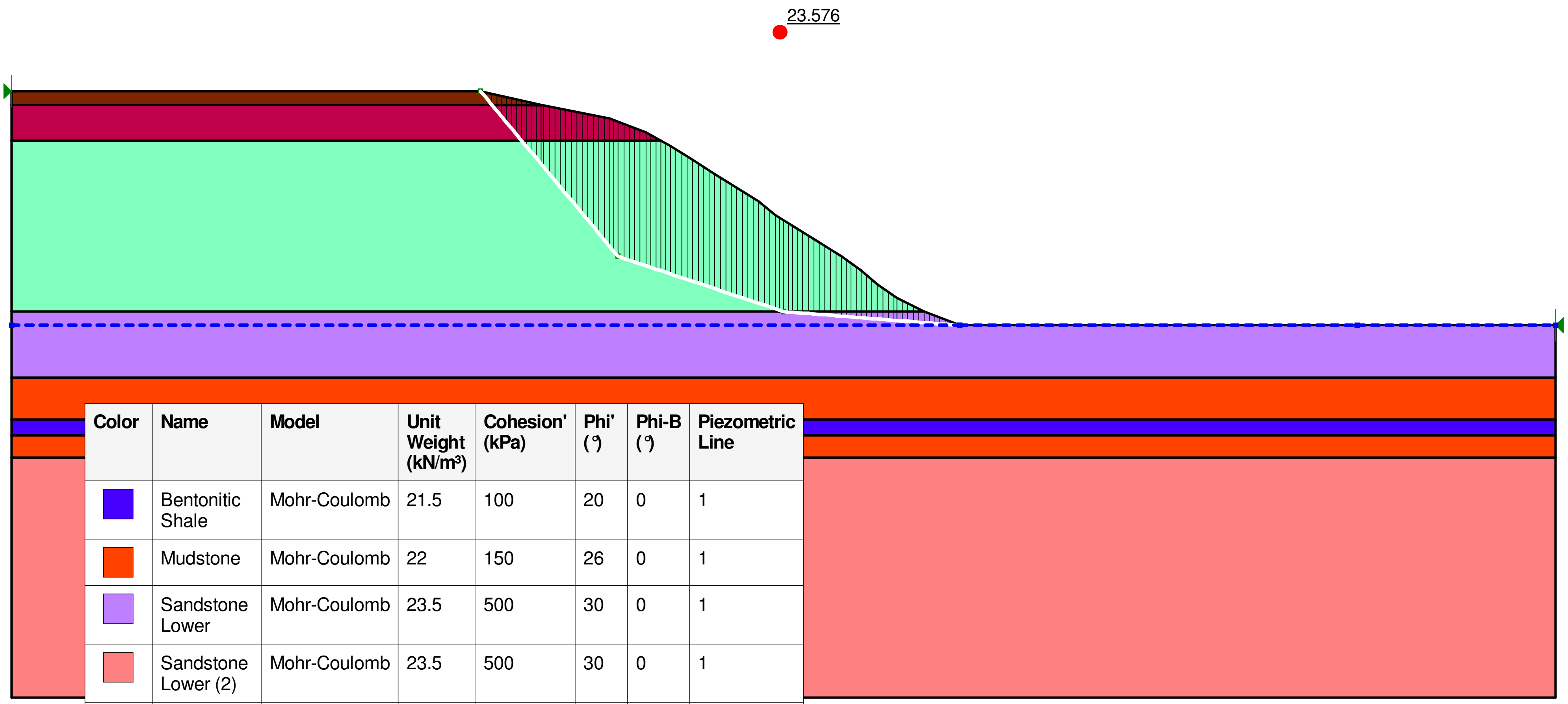
Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
🟦	Bentonitic Shale	Mohr-Coulomb	21.5	100	20	0	1
🟧	Mudstone	Mohr-Coulomb	22	150	26	0	1
🟪	Sandstone Lower	Mohr-Coulomb	23.5	500	30	0	1
🟥	Sandstone Lower (2)	Mohr-Coulomb	23.5	500	30	0	1
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	
🟥	Silty Sand	Mohr-Coulomb	17	0	30	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Horizontal Seismic Coefficient= 0.085

Seismic Analysis

Section D-D'

05/17/2021

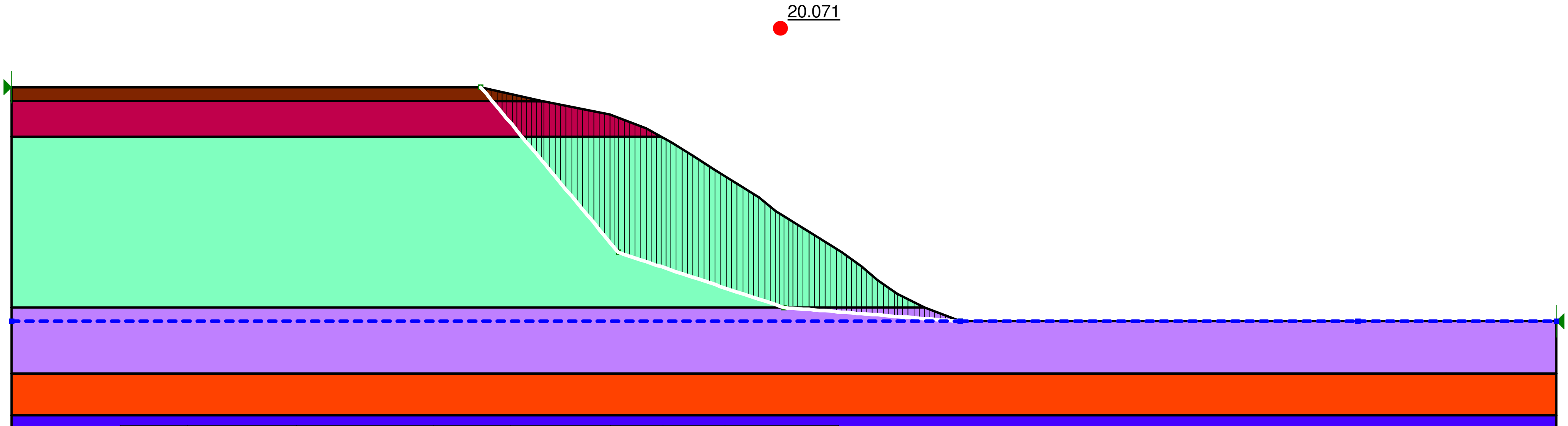


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
🟦	Bentonitic Shale	Mohr-Coulomb	21.5	100	20	0	1
🟧	Mudstone	Mohr-Coulomb	22	150	26	0	1
🟪	Sandstone Lower	Mohr-Coulomb	23.5	500	30	0	1
🟥	Sandstone Lower (2)	Mohr-Coulomb	23.5	500	30	0	1
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	
🟥	Silty Sand	Mohr-Coulomb	17	0	30	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Static Analysis Block Failure

Section D-D'

05/17/2021



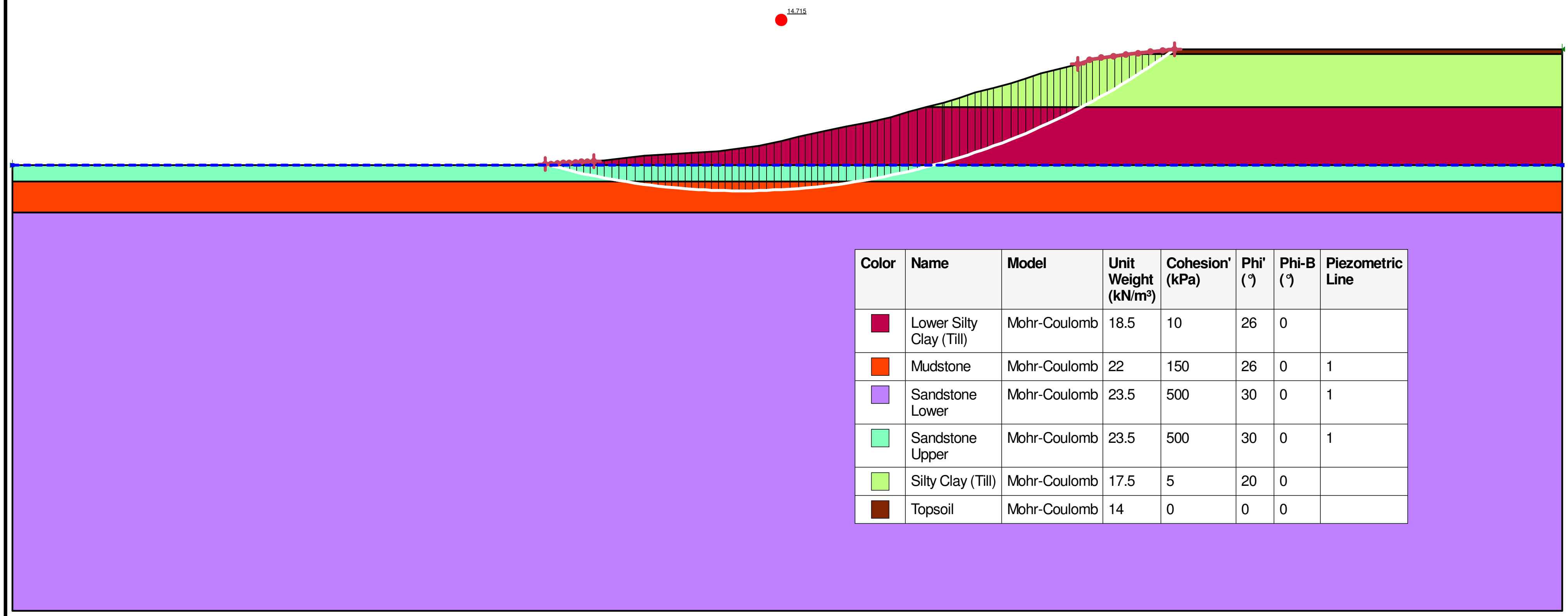
Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
🟦	Bentonitic Shale	Mohr-Coulomb	21.5	100	20	0	1
🟧	Mudstone	Mohr-Coulomb	22	150	26	0	1
🟪	Sandstone Lower	Mohr-Coulomb	23.5	500	30	0	1
🟥	Sandstone Lower (2)	Mohr-Coulomb	23.5	500	30	0	1
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	
🟥	Silty Sand	Mohr-Coulomb	17	0	30	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Horizontal Seismic Coefficient= 0.085

Seismic Analysis Block Failure

Section D-D'

05/17/2021

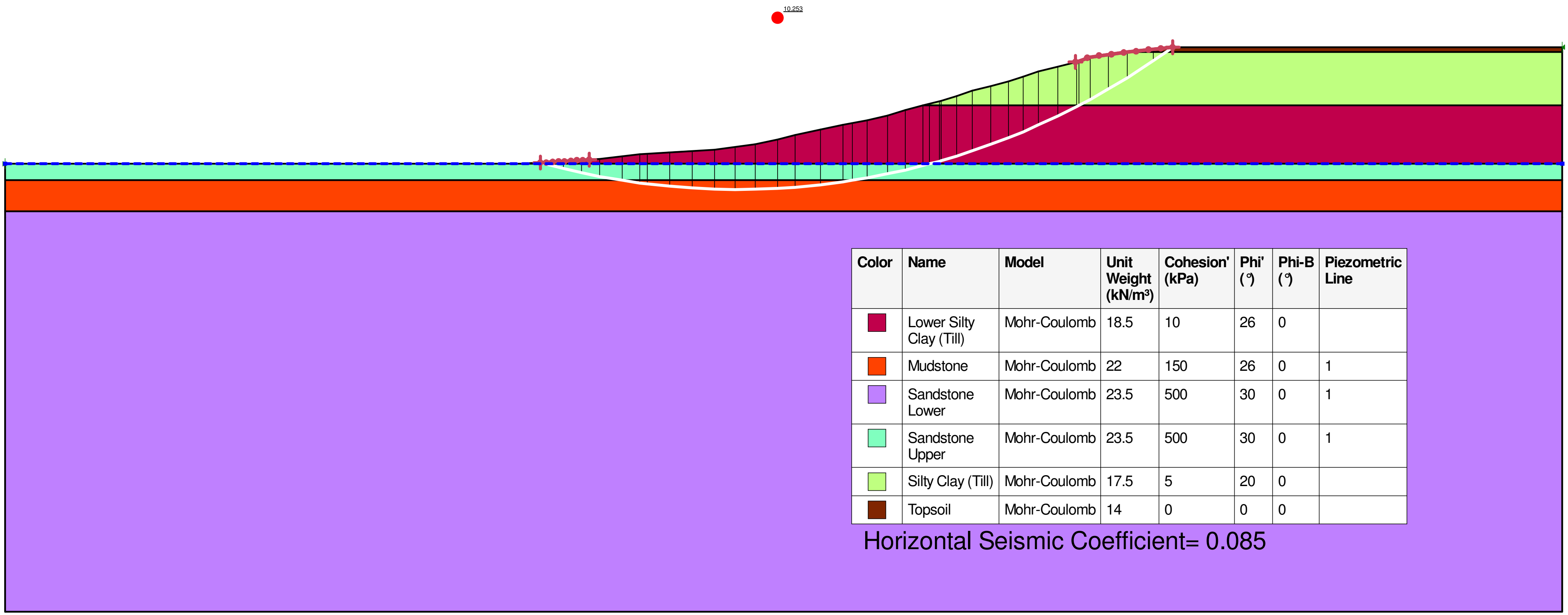


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
🟥	Lower Silty Clay (Till)	Mohr-Coulomb	18.5	10	26	0	
🟧	Mudstone	Mohr-Coulomb	22	150	26	0	1
🟪	Sandstone Lower	Mohr-Coulomb	23.5	500	30	0	1
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	1
🟩	Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Static Analysis

Section E-E'

05/17/2021



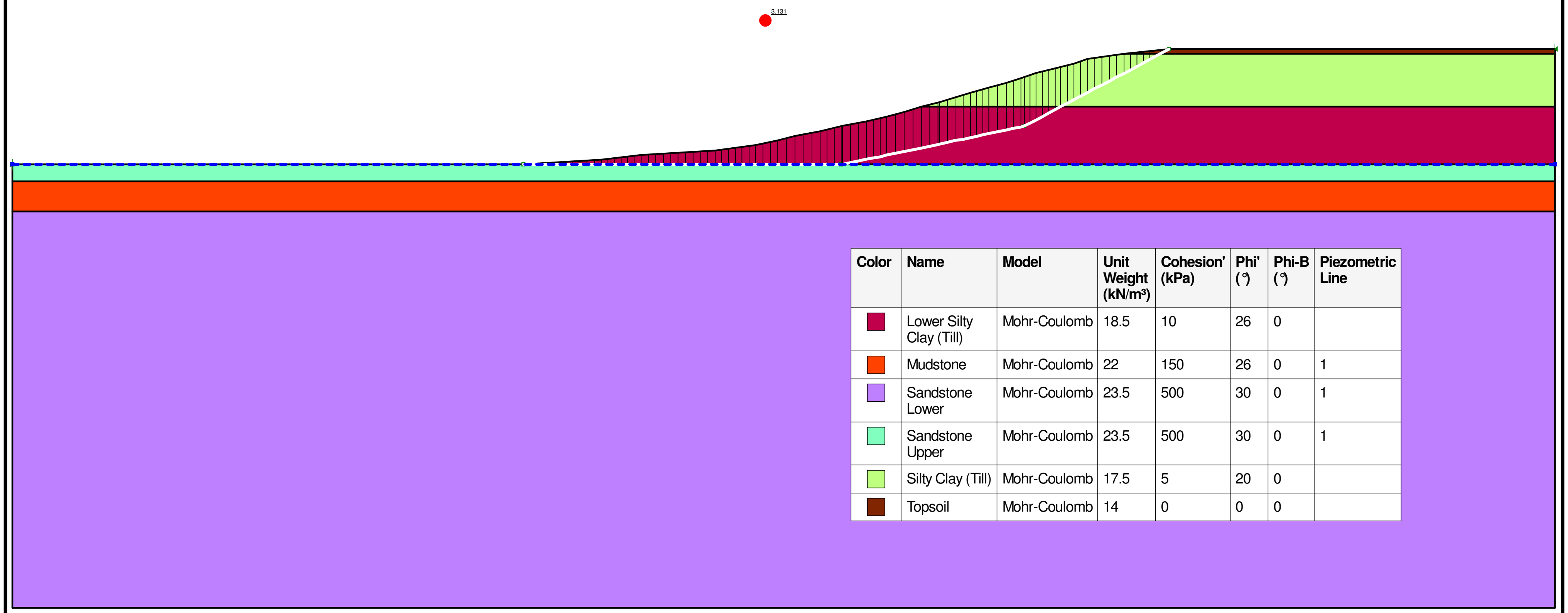
Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
🟥	Lower Silty Clay (Till)	Mohr-Coulomb	18.5	10	26	0	
🟧	Mudstone	Mohr-Coulomb	22	150	26	0	1
🟪	Sandstone Lower	Mohr-Coulomb	23.5	500	30	0	1
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	1
🟩	Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Horizontal Seismic Coefficient= 0.085

Seismic Analysis

Section E-E'

05/17/2021

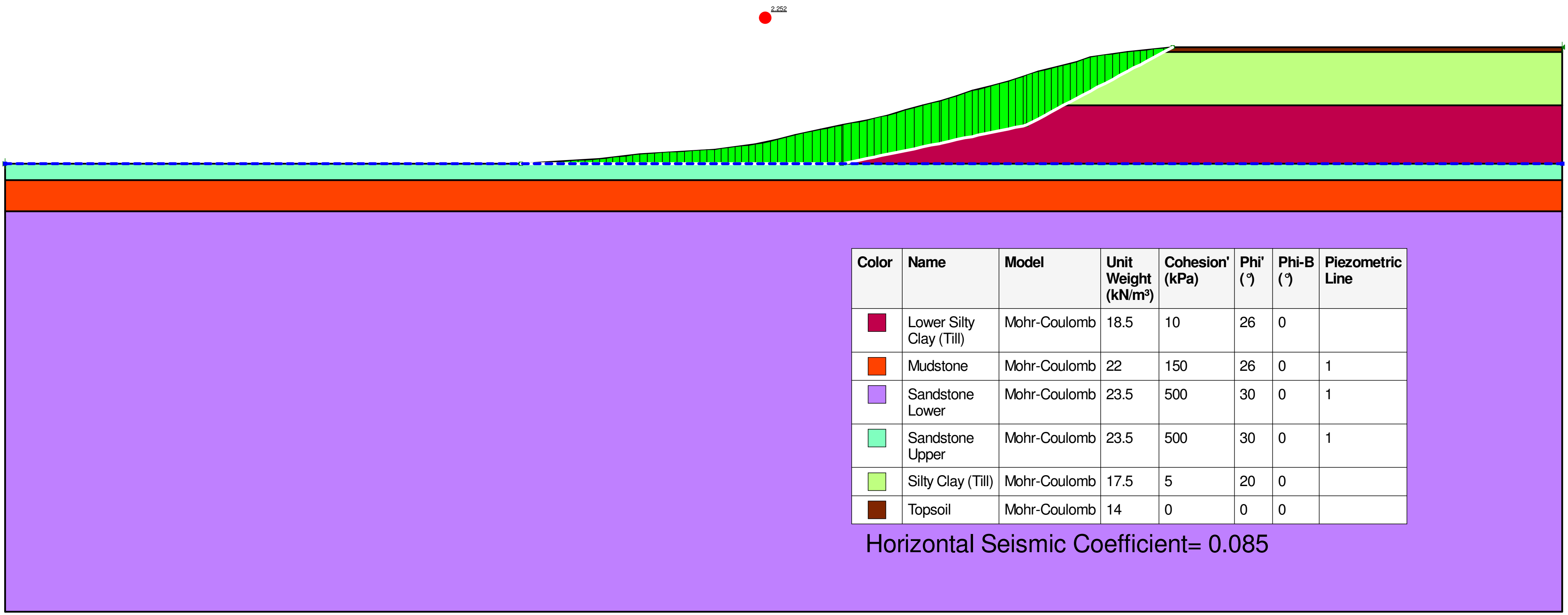


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
🟥	Lower Silty Clay (Till)	Mohr-Coulomb	18.5	10	26	0	
🟧	Mudstone	Mohr-Coulomb	22	150	26	0	1
🟪	Sandstone Lower	Mohr-Coulomb	23.5	500	30	0	1
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	1
🟩	Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Static Analysis Block Failure

Section E-E'

05/17/2021



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
🟥	Lower Silty Clay (Till)	Mohr-Coulomb	18.5	10	26	0	
🟧	Mudstone	Mohr-Coulomb	22	150	26	0	1
🟪	Sandstone Lower	Mohr-Coulomb	23.5	500	30	0	1
🟩	Sandstone Upper	Mohr-Coulomb	23.5	500	30	0	1
🟩	Silty Clay (Till)	Mohr-Coulomb	17.5	5	20	0	
🟫	Topsoil	Mohr-Coulomb	14	0	0	0	

Horizontal Seismic Coefficient= 0.085

Seismic Analysis Block Failure

Section E-E'

05/14/2021

Appendix G

Select Site Photographs

Photograph 1 – Borehole BH21-01 Location

Photograph 2 – Borehole BH21-02 Location

Photograph 3 – Ravine located at north end of development adjacent to Borehole BH21-02 Location

Photograph 4 – Ravine located at south end of development adjacent to Borehole BH21-03 Location

Photograph 5 – looking north from Borehole BH21-03 Location

Photograph 6 – Ravine located adjacent to Borehole BH21-04 Location

Photograph 7 – Central ravine area looking towards Borehole BH21-03

Photograph 8 – Central ravine area looking towards Borehole BH21-04