

Geotechnical Investigation Sinclair Canyon Site Rehabilitation

Sinclair Canyon Site
Rehabilitation



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GEOTECHNICAL REPORT

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June 20, 2016

GEOTECHNICAL INVESTIGATION SINCLAIR CANYON SITE REHABILITATION

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1.0 Introduction

Stantec Consulting Ltd. (Stantec), acting in accordance with the terms of reference provided in our proposal file number 527748 dated December 21, 2015, has carried out a geotechnical investigation for the Sinclair Canyon Site Rehabilitation project in Kootenay National Park, British Columbia.

The scope of work for the geotechnical investigation was carried out in accordance with the above referenced proposal and as discussed within the text of this report. The scope of work for this investigation included the following:

- Review of the existing geotechnical reports and design drawings by AECOM Canada Ltd. (AECOM) to evaluate and verify if the recommendations are adequate for the design of the project;
- Coordinate public underground utility locates, drilling contractor and traffic control services to facilitate a geotechnical drilling investigation;
- Conduct a geotechnical investigation consisting of three (3) boreholes to obtain additional subsurface soil and groundwater information in order to confirm previous recommendations;
- Conduct slope stability analyses and provide remedial design options for Station 1+650 including options for various retention structures;
- Prepare a geotechnical evaluation report that presents findings from the AECOM reports, site investigation and laboratory testing programs, and provide design recommendations for site rehabilitation from the geotechnical engineering perspective.

It should be noted that the scope of this slope stability assessment included only Station 1+650 (as described further below) where slope movements have been observed. The stability of the adjacent slopes as well as the overall slopes in other areas within Kootenay National Park is not part of the present scope.

1.1 SITE AND GEOLOGY

The project site is in Sinclair Canyon, covering an area from the West Park Gate to Iron Gate Tunnel in the Kootenay National Park. The site is located approximately 2.5 km northeast of the Village of Radium Hot Springs, British Columbia (see Figure 1.0, **Appendix B**). The location of the existing retaining wall is referenced as Station 1+650 within the context of this report. The existing retaining wall runs parallel along the south side of Highway 93.

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Within this area of Highway 93, the road is gently inclined from east to west, and curved to fit the surrounding mountainous peaks and valleys. The project site is bordered to the south by steep slopes. A river runs near the base of the slope, and flows from east to west. A relatively large tunnel is located upstream of Station 1+650 which allows water to flow through the adjacent mountainside. At the time of the field investigation, the site was primarily snow covered; however based on photos provided with the AECOM Report the existing wall may have undergone lateral displacements in some areas, causing it to lean outwards towards the slope. A depression in the asphalt was also noticeable at this location. Several trees along the hillside have been observed to be sloping at an angle towards the river below. It should be noted that no details regarding the original design and construction of the existing retaining wall have been provided for review prior to preparation of this report.

Based on review of published surficial geology¹ and AECOM's findings, the subsurface soils were expected to comprise of Quaternary cover of alluvium, glaciofluvial gravels and sand, till and exposed bedrock. Sedimentary bedrock was expected to comprise of the upper Cambrian and Ordovician Formations, generally consisting of dolomite, limestone, shale, and quartzite².

1.2 BACKGROUND

The following documents were provided by Public Works and Government Services Canada (PWGSC) for review and consideration in preparation of this report:

- AECOM Canada Ltd., *Roadway and Sidewalks – Sinclair Canyon Stage 1 Kootenay National Park*, Report Dated September, 2010.
- AECOM Canada Ltd., *Sinclair Canyon Tunnel and Retaining Wall Rehabilitation, Radium Kootenay National Park British Columbia*, Report Dated March, 2011.
- AECOM Canada Ltd., *Geotechnical Investigation and Retaining Wall Option Analysis Report – Sinclair Canyon Stage 1 Kootenay National Park, BC*, Report Dated April, 2011.

According to the information provided by PWGSC and from the AECOM reports, it is understood that the subject slope was reportedly stable until a watermain break in December of 2002 or 2003. AECOM noted that surface water and water from damaged storm drains are likely the primary factors contributing to the destabilized slope at this location. It is our understanding that to stabilize the slope movements PWGSC is proposing to build a new retaining wall at this location.

AECOM had completed a site reconnaissance in June, 2010. AECOM's observations at Station 1+650 indicated surface water drainage issues and evidence of slope movements such as a dip in the asphalt and cracking of the curb. Settlement of the asphalt at this location has compounded the issue of surface water drainage, and water was observed to pond around the settled area. The existing catch basin at this location was reported to be ineffective at draining surface water.

¹ Little., H.W., 1962, British Columbia Geological Survey, Geological Survey of Canada

² Baird., D.M., 1964, Kootenay National Park: Wild Mountains and Great Valleys, Geological Survey of Canada

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A borehole investigation was completed by AECOM in January 2011. The investigation consisted of advancing four (4) boreholes throughout the site using a Becker Hammer rig. Two (2) of the four (4) boreholes were advanced within the vicinity of Station 1+650. The boreholes generally indicated a subsurface stratigraphy of asphalt overlying gravel and weathered bedrock. Bedrock elevation was noted to range from approximately 997 m and 1000 m within Station 1+650 (approximately 3 m to 4 m below existing grade). Groundwater levels measured within the standpipes on March 14, 2011 were noted to be dry.

Strength tests within the boreholes consisted of Becker Hammer blow counts (blows/300mm) are summarized below in **Table 1-1**. Results of three (3) grain sizes within the gravel strata are summarized in **Table 1-2**.

Table 1-1 Results of AECOM Becker Hammer Blows at Station 1+650

Gravel			Clay			Bedrock
Min	Max	Average	Min	Max	Average	50+
33	188	97	44	75	56	

Table 1-2 Results of AECOM Grain Size Analysis at Station 1+650

BH and Sample No.	% Gravel	% Sand	% Fines (Silt and Clay)
TH11-02A - Sample 3	60	31	9
TH11-02B - Sample 1	58	33	9
TH11-02B - Sample 3	63	28	9

AECOM indicated the primary cause of slope instability at Station 1+650 was due to poor drainage. AECOM also evaluated the advantages and disadvantages of various retention structures. For ease of reference, these options are summarized in **Table 1-3**.

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Table 1-3 AECOM's Proposed Retaining Wall Options

Retaining Wall Options		
Wall Type	Advantages	Disadvantages
Soil Nail Wall with Shotcrete Face	Deep piles not required, low cost, readily available, little disruption to surrounding infrastructure	Self-supporting soil required, difficult site access for equipment, unaesthetic appearance
Soldier Piles and Lagging with permanent tie back anchors	More stable at greater heights, quick installation, low cost, readily available	Difficult to drive piles in dense gravel, shallow bedrock, pre-drilling may be required, difficult site access for specialized equipment, difficult to maintain vertical tolerance in hard ground, unaesthetic appearance
Cast in Place (CIP) Piles with Concrete Panels	Anchors may not be required, low cost, readily available, durable, no excavation required	Difficult pile installation, difficult site access for specialized equipment, unaesthetic appearance
Bin-Wall	Easy and quick installation, no specialized equipment, low cost, aesthetic appearance, existing Bin-Wall at Station 1+650 is in good condition	Imported backfill material required, excavation required
Reinforced Soil Slope (RSS)	Easy installation, no specialized equipment, low cost, less backfill material required, use of native granular soils, aesthetic appearance, vegetated slope	Vegetated soil face may require maintenance, possibility for geosynthetic reinforcement degradation in some environments.

No slope stability analyses were carried out by AECOM to verify the existing condition of the slope or the suitability of each option from a slope stability perspective. Recommended earth pressure parameters in design of retaining walls are summarized in **Table 1-4**.

Table 1-4 AECOM's Earth Pressure Coefficients for Retaining Walls

Backfill Type	γ (kN/m ³)	ϕ (°)	K_a	K_p	K_o
Retained Soil (Native Sandy Gravel)	20	34	0.3	3.3	0.44
Bin-Wall Backfill (Structural Fill)	22	34	0.3	3.3	0.44

The gravity Bin-Wall or reinforced soil slope was the preferred gravity retaining wall option for Station 1+650. Additional recommendations provided by AECOM included drainage, backfill and compaction requirements, minimum Factors of Safety (FOS) for resistance to sliding, overturning, and bearing capacity.

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2.0 Method of Investigation

2.1 FIELD INVESTIGATION

Prior to the start of the borehole investigation, Stantec personnel made arrangements to verify the locations of underground utilities at and near the proposed borehole locations. The fieldwork for the geotechnical investigation was carried out on January 25 and 26, 2016 using a truck mounted rig owned and operated by Core Drilling Ltd. of Calgary, AB. All boreholes were advanced using an Overburden Drilling EXcentric (ODEX) system.

Borehole locations were selected by Stantec personnel and are shown on the Borehole Layout Plan (Figure 2.0) in **Appendix B**. A total of three (3) boreholes were advanced along Highway 93 and ranged in depths from 7.4 m to 11.4 m below existing grade.

Borehole locations were established in the field using a handheld GPS with a known accuracy of ± 3 m. Boreholes were drilled within range to the proposed locations but were shifted to ensure a safe distance from underground utilities. The three borehole locations were surveyed on January 27, 2016 and are as shown in Figure 2.0 and summarized in **Table 2-1**.

Table 2-1 NAD 83 UTM Zone 12N Borehole Coordinates

Borehole	Easting (UTM 12N)	Northing (UTM 12N)	Depth (m)
BH-01	143429	5621330	7.4
BH-02	143472	5621338	11.4
BH-03	143529	5621337	10.3

The subsurface stratigraphy encountered in the boreholes was recorded by Stantec personnel as the boreholes were advanced, using the Unified Soils Classification System (USCS). The consistency of the soil was assessed during the performance of Standard Penetration Tests (SPTs). Undrained shear strength of cohesive soils was also assessed in the field using pocket penetrometer tests. Bulk samples were obtained at regular intervals. Representative samples of each stratum encountered were collected, stored in moisture tight containers and returned to our Calgary laboratory for detailed classification and testing.

One 25 mm PVC standpipe piezometer was installed in borehole BH-02 in order to permit groundwater level monitoring. Water levels were completed one day after installation on January 26 2016, and again on February 5 2016, approximately two (2) weeks after completion of drilling activities. The groundwater levels are shown in **Table 3-2** in **Section 3.6**, and on the Borehole Records in **Appendix C**.

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2.2 LABORATORY TESTING

All samples recovered from the drilling program were stored in moisture tight containers and were transported to our Calgary laboratory for detailed classification and testing. Laboratory testing was performed on selected samples, including:

- Natural moisture content determinations;
- Atterberg Limits;
- Grain size analyses;
- Soluble Sulphate Content.

The results of the laboratory testing are provided on the Borehole Records in **Appendix C**, presented in **Appendix D**, and/or discussed in the text of this report. Samples remaining after testing will be stored for a period of three (3) months after issuance of the final report. Samples will be discarded after this period unless otherwise directed.

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3.0 Results of Investigation

3.1 GENERAL

The subsurface strata and groundwater conditions encountered at the borehole locations are described in detail on the Borehole Records, with additional and supplementary information provided in this section. The Borehole Records, along with an explanation of the symbols and terms used in its description, are provided in **Appendix C**.

The subsurface soil conditions encountered generally consisted of a surficial layer of asphalt, measuring approximately 100 mm to 150 mm in thickness, overlying sand and gravel. Clay till was encountered within boreholes BH-02 and BH-03. Bedrock was also encountered beneath the clay till in borehole BH-03. The subsurface conditions observed at the borehole locations are described in detail in the following sections.

3.2 SILTY SAND (SM)

Sand was encountered beneath the asphalt in all boreholes, and extended to depths ranging from approximately 2.3 m to 3.0 m below existing grade (to elevations 1000.7 m to 1004.3 m). The sand was generally silty and contained varying quantities of gravel. It was generally brown in colour and dry to moist.

Grain size analysis completed on a sample of the sand (BH-01, BS1) indicated 30% gravel, 50% sand, and 20% fines (silt and clay sized particles). The results of the laboratory testing indicated moisture contents for the sand ranging from 3% to 8%, with an average moisture of approximately 5%. Based on field observations, laboratory testing, and the USCS, the sand may be classified as silty sand (SM) with gravel.

Results of two (2) SPTs conducted on the sand indicated N-values of 22 and over 50. Note that the sample with over 50 blow counts (BH-03, SS2) was likely due to the split spoon bouncing on a rock (cobble or boulder). Based on the SPT N-values, the sand can be described as compact.

3.3 CLAYEY GRAVEL (GC)

Gravel was encountered underlying the sand layer within all boreholes drilled. The gravel extended beyond the termination depth of 7.4 m (elevation 995.7 m) in BH-01 and 11.4 m (elevation 993.4 m) BH-02, and was underlain by clay till in BH-03. Within borehole BH-02 the gravel was interbedded by a layer of clay till at an approximate depth of 5.3 m (elevation 999.5 m). It was generally clayey with sand, and contained frequent cobbles and boulders. It was described as dry to moist.

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Grain size analysis was completed on two gravel samples (BH-01, SS-7 and BH-03, SS-6), as summarized in Table 3-1 below. Results of moisture content testing on samples of the gravel indicated moistures from 2% to 15%, with an average of 5%. Based on the USCS, this material may be classified clayey gravel (GC) with sand.

Table 3-1 Results of Grain Size Analysis – Clayey Gravel

Borehole & Sample	% Gravel	% Sand	% Fines (Silt and Clay)
BH-01, SS7	53	32	15
BH-03, SS6	47	24	29

Results of SPTs conducted on the gravel indicated N-values between 11 and over 50, with an average N-value of 30. Based on the Standard Penetration Test N-values, the gravel can be described as compact to very dense.

3.4 CLAY TILL (CL-ML)

A layer of brown silty clay till was interbedded within the clayey gravel in borehole BH-02 at an approximate depth of 5.2 m (999.5 m), and beneath the gravel in borehole BH-03 at approximately 8.4 m (elevation 998.8 m). The clay till was generally noted to be sandy and gravelly. Rootlets were observed within this stratum, likely due to the boreholes' proximity to the nearby trees along the slope.

Atterberg Limits testing was performed on one (1) sample of the silty clay till (BH-02 SS8), and indicated a Liquid Limit of 23 and a Plasticity Index of 7. Results of a grain size analysis performed on this same sample indicated a group distribution of 26% gravel, 39 % sand and 35% fines (silt and clay sized particles). Results of moisture content testing on samples of the silty clay till indicated moistures between 17% and 26%, with an average of 21%. Based on the USCS, the till may be classified as sandy silty clay (CL-ML) with gravel.

Results of two (2) SPTs conducted on the till stratum indicated N-values of 10 and 16. Based on these N-values the till may be described as stiff to very stiff.

3.5 SEDIMENTARY BEDROCK

Grey sedimentary bedrock was encountered in BH-03 at an approximate depth of 9.9 m (approximate elevation 997.3 m). The bedrock was predominantly claystone and was noted to be extremely weak and highly weathered. Results of one SPT conducted on the bedrock indicated an N-value of over 50 blows per 150mm. Note that SPTs were advanced mainly to obtain more representative samples of the bedrock and do not correlate directly to rock strength; however, the ability to penetrate the bedrock with the split spoons confirms its weak, weathered nature. No coring was completed or deemed necessary, as part of the current work scope.

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3.6 GROUNDWATER

A standpipe piezometer was installed in borehole BH-02. Groundwater was measured upon drilling completion, one day after drilling completion, and again on February 5, 2016, ten (10) days after drilling completion. Measured groundwater levels are presented below in **Table 3-2** and on the Borehole Records in **Appendix C**.

Table 3-2 Summary of Groundwater Depths

Borehole Number	Groundwater Depth at Drilling Completion (m)	Groundwater Depth One Day After Completion (m)	Groundwater Depth February 5, 2016 (m)
BH-01	dry	N/A	N/A
BH-02	dry	dry	dry
BH-03	dry	N/A	N/A

Groundwater levels vary from year to year and from season to season, and depend on many factors including surface and subsurface drainage, precipitation, and the hydrogeology of the area. Fluctuations in the groundwater levels should be anticipated.

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4.0 Discussion and Recommendations

4.1 GENERAL GEOTECHNICAL CONSIDERATIONS

Based on the soil and groundwater conditions encountered at this site, installation of a new retaining wall is considered to be suitable for the purpose of protecting the existing road and surrounding infrastructure from potential slope movements, provided that the recommendations outlined within this report are adhered to.

Based on the findings of the preliminary investigation and our current understanding, the main issues of geotechnical concern for the proposed rehabilitation works at Station 1+650 consists of:

- The existing steep slopes in the order of approximately 1.2H:1V to 1.5H:1V within predominantly granular soils;
- Allowance for only partial road closure during construction, resulting in a limited work area;
- Unknown present condition and as-built construction details of the existing retaining wall; and
- Possible undulating bedrock elevation and differences in the subsurface soil conditions noted between AECOM and Stantec's investigations.

Our design assumptions and analysis have considered the above items, and are discussed in the following section below.

Note that site drainage improvements along this section of the highway will be provided under separate cover and/or design drawings.

4.2 SLOPE STABILITY ANALYSES

4.2.1 Existing Slope Condition

The existing slope profile was generated using surveyed data from Stantec's Geomatics group (as shown on Figure 2.0 in **Appendix B**) and the subsurface soil and groundwater information obtained from this investigation. Based on the contour information and site observations, the existing slope gradients ranged from approximately 1.2H:1V to 1.5H:1V.

Differences in subsurface soil conditions were noted between AECOM and Stantec's investigations, including the thickness of the gravel layer and depth to bedrock. As Stantec's investigation included a borehole within close proximity to the steepest section of slope and our drilling methods included soil sampling, visual assessments and laboratory testing, the slope profiles have been developed primarily based on the findings from Stantec's investigation.

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Based on results of our investigation, our experience with similar soil conditions, and typical published values, the following shear strength parameters have been assumed for the slope stability analyses, as summarized in **Table 4-1** below.

Table 4-1 Soil Parameters

Material	Bulk Unit Weight	Shear Strength Parameters	
	(kN/m ³)	c' (kPa)	φ (°)
Compact to Dense Gravel	22	0	36
Very Dense Gravel	22	0	40
Silty Sand	19.5	0	30
Clay Till	19	1	30

Bedrock elevations were assumed based on Stantec's findings, review of the surveyed contours, and existing site features. Based on field observations, this stretch of Highway 93 was subject to frequent traffic consisting of vehicles ranging from standard passenger vehicles and pickup trucks, to large multi-axle logging trucks. Due to the proximity of Highway 93 to the slope, our analysis has also considered an assumed surcharge of 20 kPa due to the frequent and heavy traffic loads.

The existing stability of the slope was analyzed along four (4) cross sections (see Figure 2.0) to identify the existing Factor of Safety (FOS) values for the slope, based on the above assumptions. Results of our analyses indicated an existing global FOS ranging from approximately 1.0 to 1.3, as summarized below and shown on Figures 3.0 to 6.0 in **Appendix B**.

Table 4-2 Existing Factors of Safety Along Slope

Section	Calculated Global FOS	Figure No.
A-A	1.3	3.0
C-C	1.1	4.0
D-D	1.0	5.0
E-E	1.2	6.0

The calculated FOS values correspond to potential slip surfaces that may extend into the existing highway. Results of our analyses indicate that the current slope is a global stability issue, and that potential slip surfaces may be deep-seated (i.e., 3 m or greater in depth) and may extend to encompass an area which could affect the current road in the event of a failure.

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It should be noted that the design and construction details of the existing retaining wall are unknown at the time of this report and as such, our back-calculation analysis has not taken into consideration any potential slope support from this existing wall. It is also likely that the existing vegetation and trees' root structure has had a positive stabilizing effect throughout the slope, which has not been accounted for in the analyses. It is important to note that specific details regarding the watermain break, such as amount of water leakage or the period of time that the watermain break occurred are also unknown. However, the additional water introduced into this slope is likely to have reduced its overall stability (especially along the more critical area at Section D-D), contributing to movements observed either as a result of erosion, or through increased pore water pressures resulting in decreased stability in the soil.

4.2.2 Remediation Options Considered

As discussed above, our analyses indicate that the existing slope is currently below a required FOS of 1.5 for global stability. Potential slip surfaces may be deep-seated and may extend into the existing roadway and surrounding infrastructure. Based on the back-calculation analyses, retention structures will need to intercept these potential slip surfaces in order to stabilize the slope to a minimum FOS of 1.5.

Stantec has reviewed the retention options proposed by AECOM (as summarized in **Table 1-3** above). Based on the results of our analyses it is Stantec's opinion that a reinforced soil slope is not expected to be feasible for use. A soil nail wall with shotcrete facing may be feasible, however difficulties should be expected during soil nail installation, as discussed in **Section 4.2.3.4** below.

Reinforced soil slopes require use of horizontal reinforcement (such as geosynthetics) to extend a sufficient distance into the slope in order to intercept potential slip surfaces and increase the slope's FOS. Since potential slip surfaces may be deep-seated and extend a significant distance into the slope, reinforced soil slopes will require significant excavations in order to install the layers of horizontal reinforcement required. Since only partial road closures are allowed during construction a reinforced soil slope is not expected to be feasible. Furthermore, this method will require removal and re-planting of the existing trees and vegetation along the slope during construction.

Potential remedial options considered to be feasible for the current site and soil conditions are discussed in the following section below.

4.2.3 Proposed Remedial Options

With the exception of the piled wall (as discussed in **Section 4.2.3.2**), the following remedial options should be considered conceptual design only, and a detailed design will be required depending on the option selected by PWGSC. The intent is to provide PWGSC with some viable options based on the current geotechnical constraints of the project, as discussed in the following sections. Note that detailed design drawings of the piled wall will be provided under separate cover.

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4.2.3.1 Gravity Bin-Wall

The principle behind the use of a gravity retaining structure such as the bin-wall to support the slope may be a feasible alternative, however it should be noted that the bin-wall will need to be founded within the native very dense gravel or bedrock at a minimum depth of 10 m below existing grade at the steepest section of slope. Therefore installation of the bin-wall may have logistical challenges that prove to be impractical. Installation will likely require large open excavations and temporary support (as discussed in **Section 4.3.2**). The large excavation footprint may also lead to additional lane closures which can be disruptive to local traffic.

If a bin-wall is considered, the manufacturer and/or contractors experienced with bin-wall assembly and installation in similar soil conditions should be consulted to discuss a safe and proper construction procedure. Note that internal stability analyses of the bin-wall have not been completed at this time. The bin-wall should be considered as a proprietary design and as such, the manufacturer should be contacted for design input.

4.2.3.2 Piled Wall

Based on the current limitations and subsurface conditions, Stantec is of the opinion that the most feasible retaining wall option will consist of a system of piles founded within the very dense gravel or bedrock in order to intersect the potential slip surfaces. The pile system will retain the roadway and surrounding infrastructure from any potential deep-seated movements at the slope. This system will also be less disruptive to traffic, as large excavations are not expected to be required, reducing construction footprint.

Preliminary slope stability analyses were completed at four (4) cross sections with a proposed pile wall in order to confirm the feasibility of this option. Results of our stability analyses are shown on Figures 7.0 to 10.0, and indicate global FOS values in excess of 1.5 for potential slip surfaces that may affect the existing highway. It should be noted that potential slip surfaces below an FOS of 1.5 were calculated in front of the pile wall which may result in some movements at the slope in front of the wall; however these potential movements will not affect the overall stability of the road and surrounding infrastructure.

Tie-back anchors may be required to provide additional lateral support for potential loss of ground that may occur due to shallow slope movements. Based on the findings of the investigation, the use of driven piles is not advised due to the presence of cobbles and boulders that may damage the pile and/or lead to early refusal. Drilled cast-in-place concrete piles are considered to be more suitable. Due to the presence of cobbles and boulders a minimum pile diameter of 900 mm is recommended. Difficulties during installation should be anticipated, and casing or other means to control sloughing conditions may be required. The piles should be founded within the very dense gravel or bedrock stratum. Based on the results of our current analyses, minimum pile lengths range between 7 m to 11 m below existing grade as summarized in **Table 4-3** below and shown on Figure 11.0 in **Appendix B**.

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Table 4-3 Minimum Pile Lengths

Section	Minimum Pile Length (m)
A-A to C-C	7.0
C-C to E-E	11.0
West of E-E	7.0
Note: Minimum pile lengths are with respect to existing grades.	

Note that the structural design of the piled wall, including detailed design drawings are provided under separate cover.

Installation of the piles is anticipated to require preparation of a level working surface within the partially closed off road. The pile walls should be installed by contractors experienced with similar soil conditions. Note that actual pile lengths may vary depending on site conditions encountered during construction. In order to confirm design assumptions and ensure the piles will satisfy design requirements, full time quality assurance and quality control (QA/QC) will be required during construction.

Lateral Earth Pressures for Retaining Wall Design

For consideration in design the following earth pressure parameters provided in **Table 4-4** may be used for design of the retaining wall. It is important to note that the earth pressure coefficients are based on the assumption that the retaining wall will be installed near the edge of the existing roadway, such that a permanent horizontal back slope will be maintained behind the retaining wall.

Table 4-4: Lateral Earth Pressure Parameters

Parameter	Clay Till	Sand	Compact to Dense Gravel	Very Dense Gravel
Total Unit Weight (kN/m ³)	19	19.5	22	22
Angle of Internal Friction, ϕ (°)	30	30	36	40
Active Earth Pressure Coefficient (K_a)	0.33	0.33	0.26	0.22
At-Rest Earth Pressure Coefficient (K_o)	0.50	0.50	0.41	0.36
Passive Earth Pressure Coefficient (K_p)	3.0	3.0	3.8	4.6

For retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied structures, the at-rest pressure should be used for design, unless the wall can deflect enough (approximately 0.4% of the wall height) to establish the active pressure.

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Passive soil resistance in front of the piled wall should be accounted for in the pile design. However, due to potential loss of ground in front of the wall (from potential slip surfaces below a FOS of 1.5), the design should neglect passive soil resistance from the following minimum distances below the top of pile as shown on **Table 4-5** below.

Table 4-5 Minimum Depths to Passive Soil Resistance

Section	Minimum Depth to Developing Passive Soil Resistance (m)
A-A to C-C	3.0
C-C to E-E	5.0
West of E-E	3.0

Note: Minimum depths are with respect to top of pile.

4.2.3.3 Road Re-Alignment

As an alternative to using retention structures, re-alignment of the existing road along this section of the highway may be considered. Results of our preliminary analyses indicates that for the road to be outside existing potential slip surfaces up to a FOS of 1.5, a minimum setback distance of 15 m from the edge of the existing slope will be required. Note that this option would likely require adjustments to the existing parking lot and/or re-grading of the adjacent slope. If re-alignment of the road is considered, further slope stability analyses will need to be completed in order to determine potential impacts of grading activities on the northern slopes.

4.2.3.4 Soil Nails

Use of soil nails to reinforce the existing slope may be a feasible alternative to stabilize the slope and surrounding infrastructure. However, due to the presence of the cobbles and boulders at this site, difficulties should be expected during soil nail installation as the cobbles and boulders may lead to obstructions that could lead to early refusal, damage to the nail, and/or damage to the drilling equipment. The existing steep slopes and limited work area may also present logistical challenges. The existing trees and vegetation throughout the rehabilitation area along the slope may obstruct equipment and personnel, and may need to be removed prior to construction and re-planted upon construction completion. Furthermore, it should be noted that the combination of the existing soil conditions (dry granular soils with cobbles and boulders) and steep slopes are generally considered to be unfavorable conditions for use of soil nails.

If considered, the design and installation of soil nails should only be completed by specialized contractors with sufficient experience in similar soil conditions.

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4.3 GENERAL EARTHWORKS

4.3.1 Site Grading and Preparation

Some site grading and subgrade preparation is expected to be required for the rehabilitation of the existing road within this stretch of Highway 93. The extent of these required activities will depend on the slope remediation option chosen, however cuts into the existing slope and roadway within the closed off lanes are anticipated to be required. Any organic soil, loose, soft or deleterious materials encountered must be removed from beneath the outline of the road and retaining wall areas.

Upon removal of the existing asphalt pavement structure, exposed subgrade surfaces within the road alignment should be scarified to a minimum depth of 200 mm, moisture conditioned within the range of $\pm 3\%$ of optimum moisture content (optimum to be determined in accordance with ASTM D698), and recompacted to a minimum of 98% of Standard Proctor Maximum Dry Density (SPMDD).

Following preparation, exposed subgrade surfaces should be proof-rolled using heavy equipment such as a loaded tandem dump truck. Proof rolling should be conducted under the direct supervision of qualified geotechnical personnel, to ensure subgrade will have adequate and consistent strength required for the support of design loads. All soft areas must be sub-excavated to competent material and replaced with approved Engineered Fill. To promote subgrade uniformity, soft area repair should be carried out using soil of a similar nature to the in-situ subgrade soils and include provisions for drainage of the repaired areas.

Where construction is carried out during winter conditions, subgrade surfaces should be protected from freezing. In addition, the subgrade should be protected from wetting or drying, both before and after the placement of granular base material and concrete. Subgrade surfaces that are allowed to dry or become wet must be scarified, moisture conditioned, and recompacted.

4.3.2 Temporary Excavation and Dewatering

All temporary excavations undertaken at the retaining wall location should be in accordance with the applicable WorkSafe BC Occupational Health and Safety regulations. For temporary stability of excavations within the predominantly cohesionless sand and gravels, excavations should be conducted with side slopes no steeper than 2H:1V. Excavations should be inspected regularly for signs of seepage and instability and be flattened as required. Sloped excavations with more than 3.0 m depth should be subject to a detailed slope stability assessment.

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The existing road is likely to be sensitive to the nearby construction and care should be taken to protect the road from construction activities. Additional temporary support such as shoring during construction may be required. Depending on the remedial option chosen and the construction methodology, a monitoring system may be warranted. Typically, monitoring systems for excavations and shoring walls consist of the installation of slope inclinometers and survey points along the perimeter of the excavation.

No groundwater seepage was encountered during the investigation, however dewatering of excavations will be dependent upon weather conditions and the time of year of construction. If encountered during construction, it is expected that groundwater may be controlled by a system of sump pumps and shallow trenches. The groundwater level should be maintained a minimum of 0.5 m below excavation grade at all times.

Control of surface water run-off will be required throughout the duration of the work. Prepared surfaces should be protected to minimize the amount of degradation during wet weather. We recommend sloping any prepared surface at approximately 2% to direct water away from construction areas and sealing the surfaces of fill zones at the end of each work day to help keep water out.

4.3.3 Fill Materials and Compaction Requirement

The in-situ silty sand and clayey gravel may be suitable for reuse as Engineered Fill with proper moisture conditioning, provided that large cobbles and boulders larger than 150 mm are removed. Based on field observations, the moisture condition of the gravel was generally dry, therefore it is expected that moisture conditioning may be required in order to achieve proper compaction. Drying or wetting of the site soils may be required during periods of heavy rain, hot weather, or in the event that excavated material is allowed to dry excessively prior to reuse. Some difficulties should be anticipated when reusing the in-situ silty sand, as this material may dry quickly and can be easily disturbed and loosened when dried.

Engineered Fill should be placed in lifts to a maximum thickness of 300 mm. Thinner lifts may be required depending on the size and compactive effort of the equipment being used on site. Engineered Fill consisting of coarse granular soils should be moisture conditioned to within $\pm 3\%$ of optimum moisture content and compacted to a minimum of 98% of SPMDD.

All imported fill materials should be tested and approved by a geotechnical engineer prior to delivery to the site.

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4.3.4 Site Drainage

Since movements of this slope were likely caused by excess water as a result of the watermain break, surface water management will be critical in order to direct surface water run-off away from the slope and retaining walls. Excess water should not be allowed to pond on or adjacent to the road embankment and should be drained as quickly as possible both during and after construction.

The prepared subgrade and subsequent layers in the road structure should be shaped to prevent ponding of water. The finished road surface should have a minimum of 2% grade whenever possible, sloped away from the road surface and retaining wall, toward the catch basins to prevent ponding. For the road, a minimum cross slope of 1.5% is recommended.

It is understood that drainage improvements will be addressed under separate cover and/or design drawings; however, a curb and gutter system may be required to adequately control surface water run-off.

4.4 LIGHT STANDARDS

It is understood that proposed foundations for light standards will consist of shallow cast-in-place concrete piles founded at a depth of approximately 2 m below finished ground surface. It should therefore be noted that the piles are not founded at a sufficient depth and may not attain adequate soil resistance to resist bending moments and lateral forces. Furthermore, the proposed embedment depth is near the estimated frost depth in the area (frost depth estimated to be approximately 1.7 m) and may be subject to soil uplift due to adfreeze.

Increased foundation embedment may be considered to achieve adequate resistance against loads induced on the piles. Typically, a minimum pile length of 6.0 m is required to resist uplift forces due to adfreeze. It should be noted that when determining the required pile length under frost conditions, the weight of the pile, the unfactored loads on the pile, and the shaft resistance from the soil below the frost zone should be considered in the calculation. Insulation may also be considered for resistance against adfreeze stresses.

Alternatively, if the designer has sufficient past experience for foundations with similar dimensions, loading conditions, and soil type in the area, then it is within their judgement to consider past foundation performance and accept a level of risk associated with the shallow embedment depths.

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4.5 SEISMIC CONSIDERATIONS

The CAN/CSA-S6-14 Canadian Highway Bridge Design Code seismic design procedures are based on ground motion parameters (e.g., peak ground acceleration (PGA)) having a 10% probability of exceedance in 50 years; i.e., the 475 year return period earthquake event. According to information provided by the Geological Survey of Canada, the Peak Ground Acceleration (PGA) at this site is 0.041 g. The CAN/CSA-S6-14 Canadian Highway Bridge Design Code refers to the PGA as the peak horizontal ground acceleration (PHA).

Based on the results of the investigation, it is appropriate to classify the ground conditions at the subject site as a **Site Class C**, in accordance with Table 4.1 under Section 4.4.3.2 of CAN/CSA-S6-14.

Liquefaction potential of the native materials is not considered to be a concern, based on results of Stantec's investigation.

4.6 CONCRETE

Water-soluble sulphate (SO_4) content testing completed on a sample of the silty sand (BH-02, BS3) indicated a SO_4 content of 0.17%. It is noted that AECOM also conducted SO_4 content on samples of the native soil, as summarized in **Table 4-6** below.

Table 4-6 Water Soluble Sulphate Contents – Stantec and AECOM

Borehole and Sample No.	Material	Depth (m)	SO_4 Content %
BH-02, BS3 (Stantec)	Silty Sand	2.0	0.17
TH11-01B, Sample 1 (AECOM)	Sand and Gravel	1.5	< 0.1
TH11-02A, Sample 2 (AECOM)	Gravel	2.3	< 0.1

Based on AECOM's test results, the degree of exposure should be considered moderate (CSA23.1-14, Table 3, Type S-3). Therefore, Type MS, LH, or HS cement with a maximum water-to-cementing materials ratio of 0.5 and minimum compressive strength of 30 MPa at 56 days should be used for concrete in contact with site soils.

Air entrainment to the requirements CSA A23.1-14 should be specified for all concrete in contact with freezing temperatures. Stricter specifications may be required for structural requirements, other exposure conditions, or other considerations.

GEOTECHNICAL INVESTIGATION SINCLAIR CANYON SITE REHABILITATION

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5.0 Closure

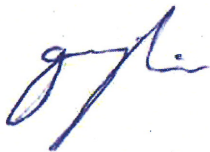
This report has been prepared for the sole benefit of Public Works and Government Services Canada (PWGSC) and their agents, and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and PWGSC. Any use, which a third party makes of this report, is the responsibility of such third party. Use of this report is subject to the Statement of General Conditions provided in **Appendix A**. It is the responsibility of PWGSC, who is identified as "the Client" within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec Consulting Ltd. should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying or unexpected site conditions
- Planning, design or construction

We trust the above information meets with your present requirements. Should you have any questions or require further information, please contact us. This report has been prepared by Gary Liu, P.Eng., and reviewed by Patrick Doyle, P.Eng., and Nigel Denby, P.Eng.

Respectfully submitted,

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APPENDIX A

Statement of General Conditions

GEOTECHNICAL INVESTIGATION
SINCLAIR CANYON SITE REHABILITATION

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Stantec's present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Stantec is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

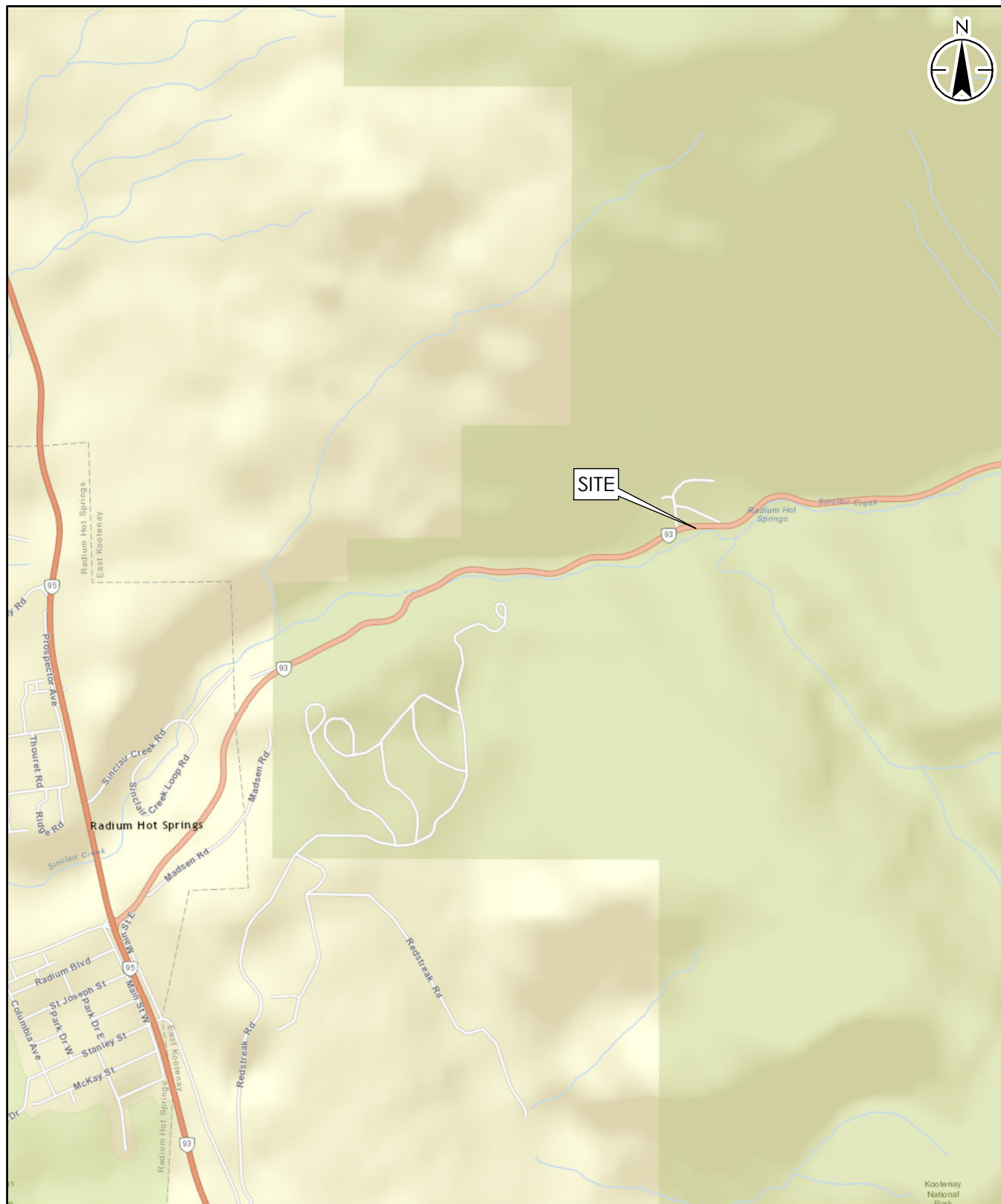
VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec will not be responsible to any party for damages incurred as a result of failing to notify Stantec that differing site or sub-surface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec, sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc.), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-surface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Stantec cannot be responsible for site work carried out without being present.

APPENDIX B

Figures

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Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand), MapmyIndia, © OpenStreetMap contributors, and the GIS User Community

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HIGHWAY 93, RADIIUM HOT SPRINGS, BC

Figure No.

1.0

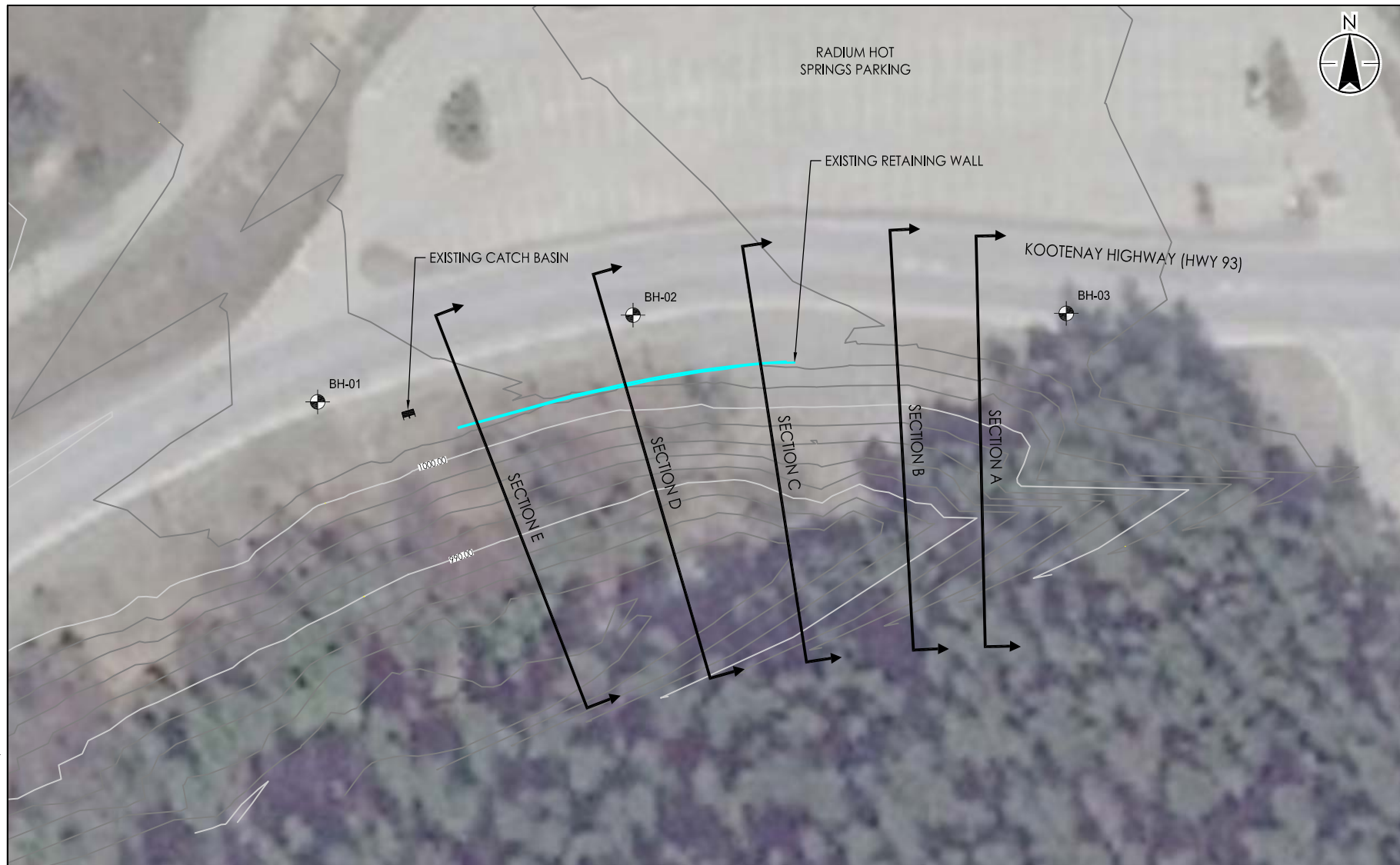
Title

Site Location Plan



200 - 325 25th St. SE Calgary, AB T2A 7H8
Stantec does not certify the accuracy of the data.
This map is for reference only and should not be used for construction.

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2016/06/20 5:12 PM By: Loewen, Caroline



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Legend:

Notes: ESRI Aerial Imagery; Digital Globe 5/6/2012

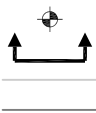
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Borehole

Cross Section

Topographical Line (10 m)

Topographical Line (5 m)



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Figure No.

2.0

Title

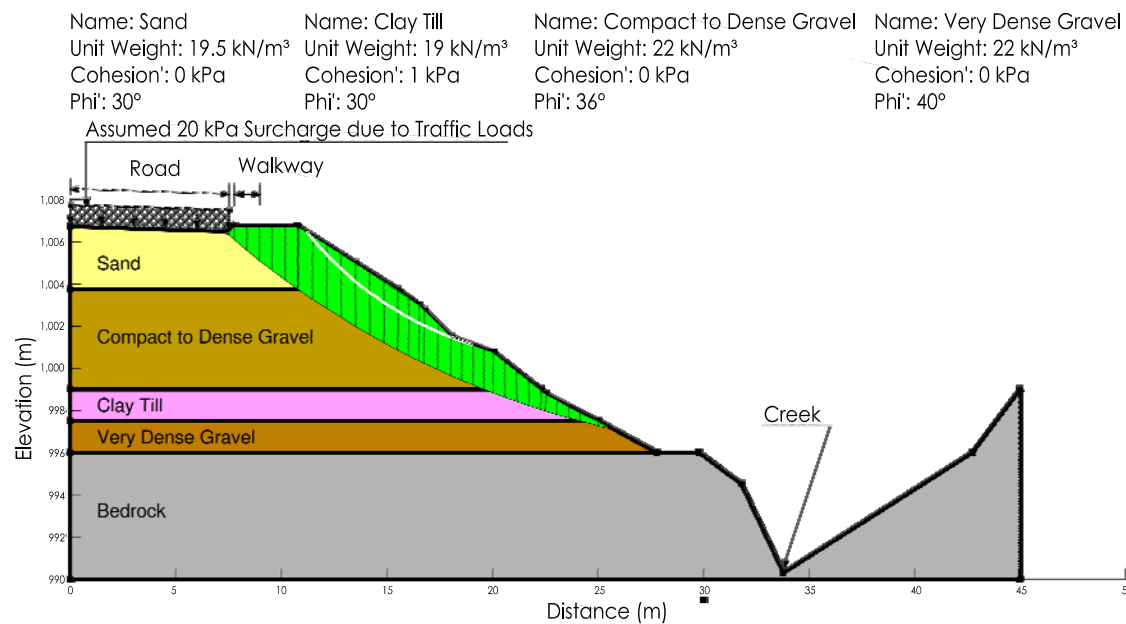
Site Plan



200 - 325 25th St. SE Calgary, AB T2A 7H8
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FOS = 1.3



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SINCLAIR CANYON SITE REHABILITATION
HIGHWAY 93, RADIUM HOT SPRINGS, BC

Figure No.

3.0

Title

Station 1+650
Section A: Existing Conditions



200 - 325 25th St. SE Calgary, AB T2A 7H8
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2016/06/20 5:04 PM By: Loewen, Caroline

$$\text{FOS} = 1.1$$

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Unit Weight: 19.5 kN/m³

Cohesion': 0 kPa

Phi': 30°

Name: Clay Till

Unit Weight: 19 kN/m³

Cohesion': 1 kPa

Phi': 30°

Name: Compact to Dense Gravel

Unit Weight: 22 kN/m³

Cohesion': 0 kPa

Phi': 36°

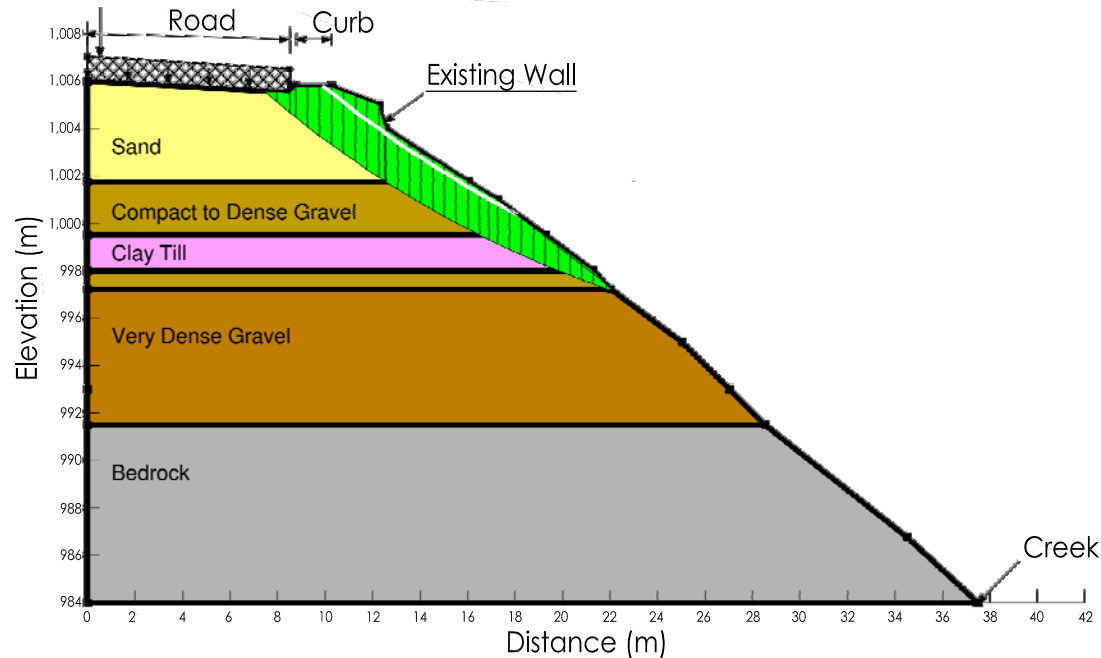
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Unit Weight: 22 kN/m³

Cohesion': 0 kPa

Phi': 40°

Assumed 20 kPa Surcharge due to Traffic Loads



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SINCLAIR CANYON SITE REHABILITATION
HIGHWAY 93, RADIUM HOT SPRINGS, BC

Figure No.

4.0

Title

Station 1+650
Section C: Existing Conditions

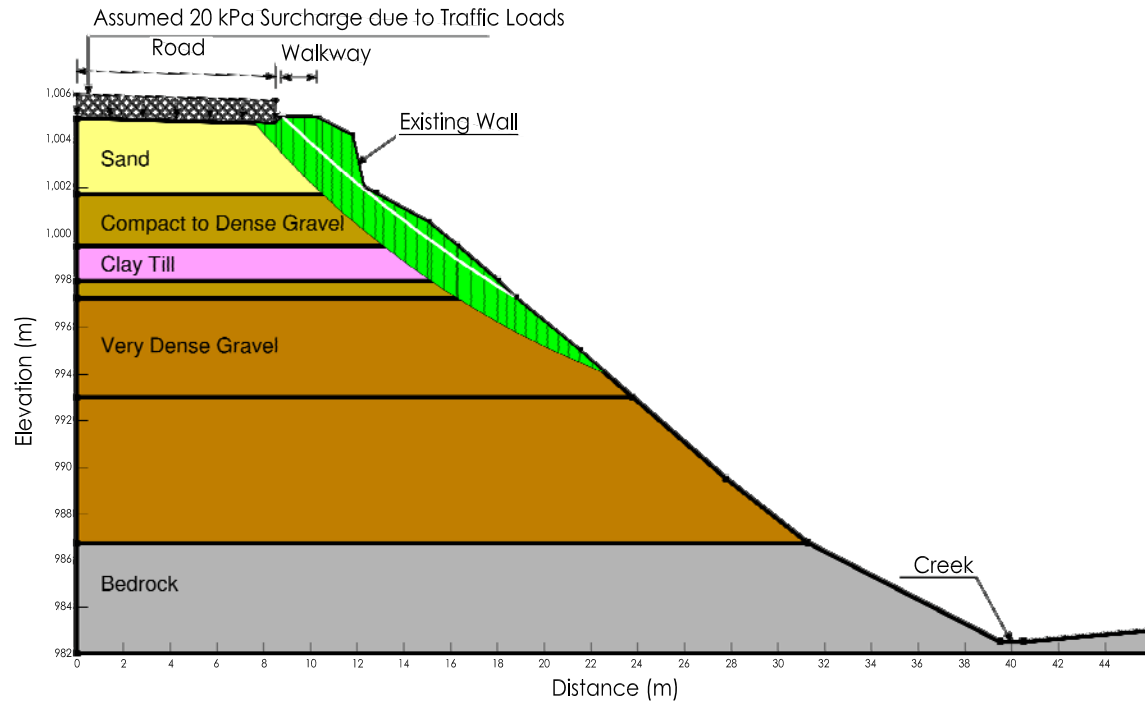


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2016/06/20 5:00 PM By: Loewen, Caroline

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Phi': 30°	Phi': 30°	Phi': 36°	Phi': 40°



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SINCLAIR CANYON SITE REHABILITATION
HIGHWAY 93, RADIUM HOT SPRINGS, BC

Figure No.

5.0

Title

Station 1+650
Section D: Existing Conditions



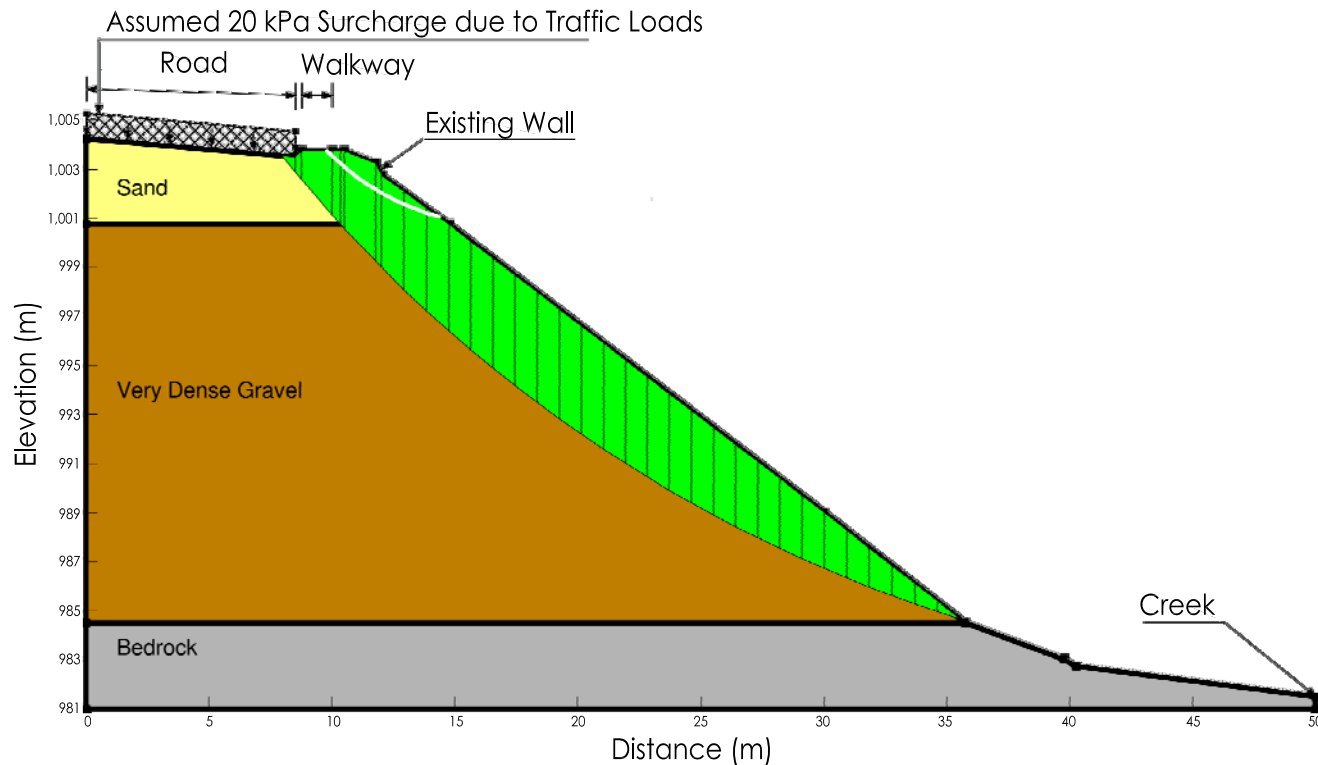
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2016/06/20 5:01 PM By: Loewen, Caroline

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Name: Very Dense Gravel
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Cohesion': 0 kPa
Phi': 40°



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Figure No.

6.0

Title

Station 1+650
Section E: Existing Conditions



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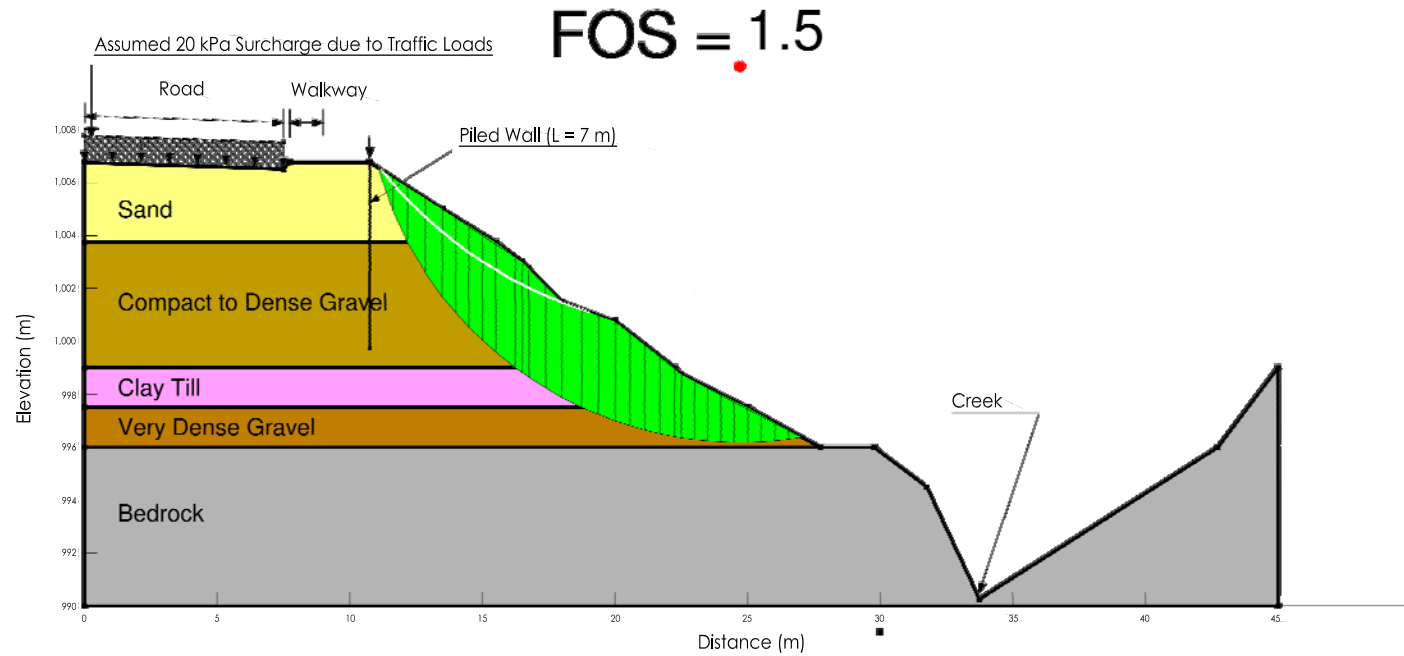
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Phi: 30°

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Unit Weight: 19 kN/m³
Cohesion: 1 kPa
Phi: 30°

Name: Compact to Dense Gravel
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 36°

Name: Very Dense Gravel
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 40°



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HIGHWAY 93, RADIUM HOT SPRINGS, BC

Figure No.

7.0

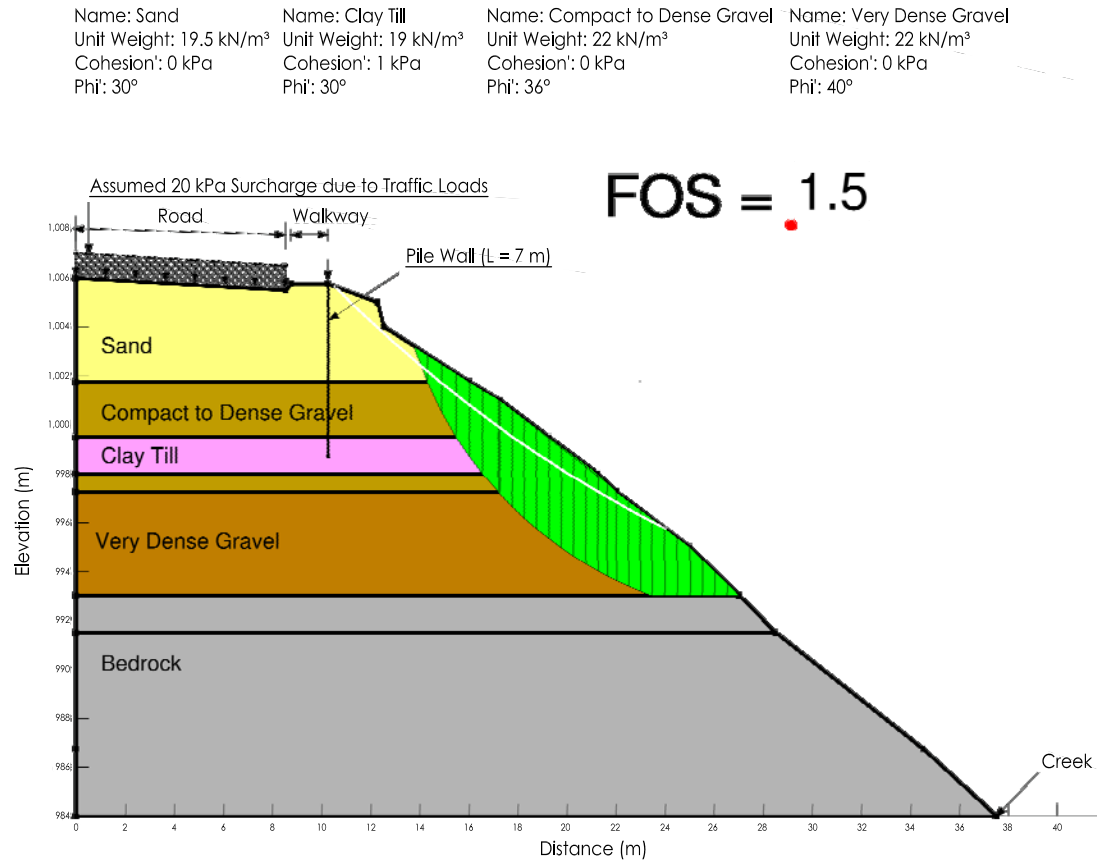
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Station 1+650
Section A: Pile Wall



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SINCLAIR CANYON SITE REHABILITATION
HIGHWAY 93, RADIUM HOT SPRINGS, BC

Figure No.

8.0

Title

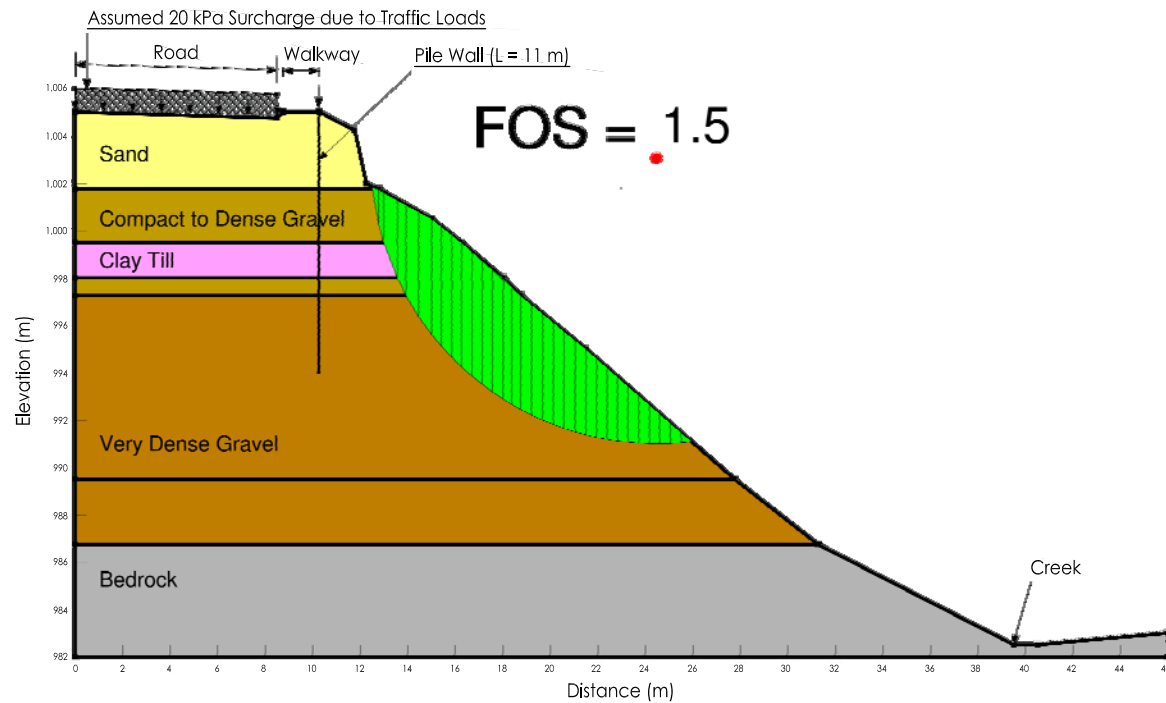
Station 1+650
Section C: Pile Wall



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2016/06/20 5:07 PM By: Loewen, Caroline

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Cohesion: 0 kPa	Cohesion: 1 kPa	Cohesion: 0 kPa	Cohesion: 0 kPa
Phi: 30°	Phi: 30°	Phi: 36°	Phi: 40°



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SINCLAIR CANYON SITE REHABILITATION
HIGHWAY 93, RADIIUM HOT SPRINGS, BC

Figure No.

9.0

Title

Station 1+650
Section D: Pile Wall

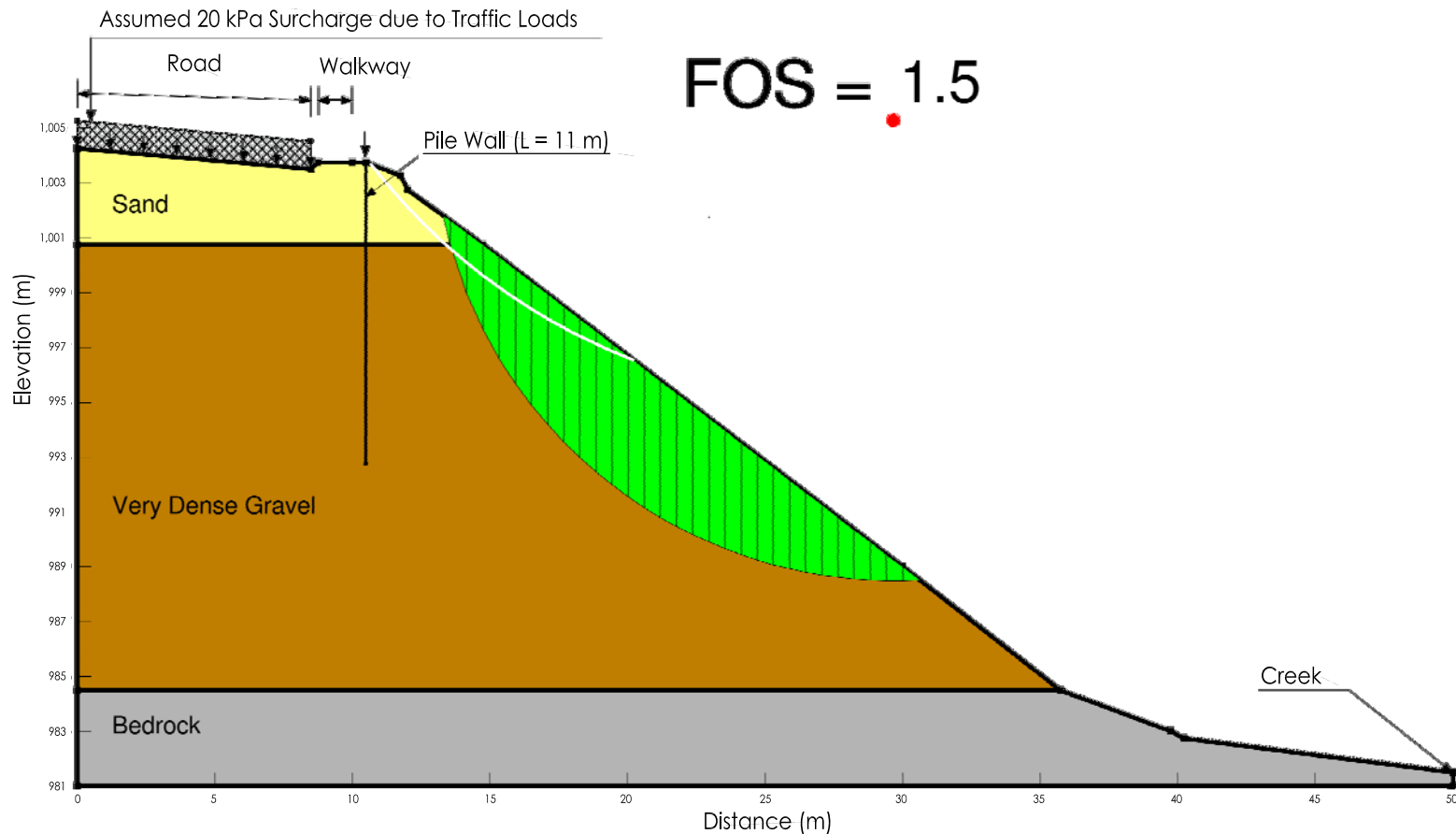


200 - 325 25th St. SE Calgary, AB T2A 7H8
Stantec does not certify the accuracy of the data.
This drawing is for reference only and should not be used for construction.

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Cohesion: 0 kPa
Phi: 30°

Name: Very Dense Gravel
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 40°

FOS = 1.5



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HIGHWAY 93, RADIIUM HOT SPRINGS, BC

Figure No.

10.0

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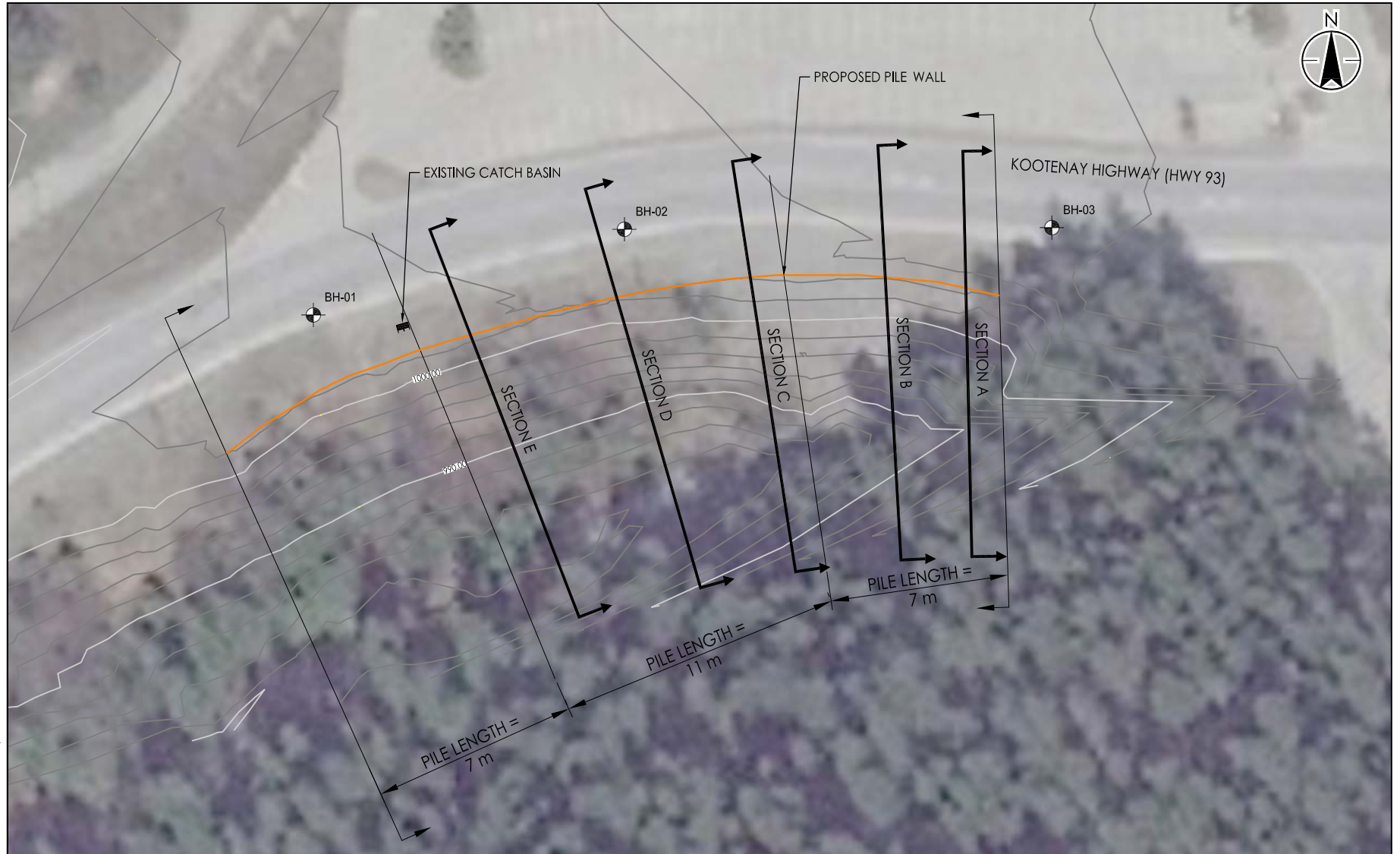
Station 1+650
Section E: Pile Wall



200 - 325 25th St. SE Calgary, AB T2A 7H8
Stantec does not certify the accuracy of the data.
This drawing is for reference only and should not be used for construction.

\\Cal1002-f04\shared_projects\115302913 - Sinclair_canyon\Geotechnical\drawings\slope_profiles.dwg
2016/06/20 5:08 PM By: Loewen, Caroline





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2016/06/20 5:10 PM By: Loewen, Caroline



ORIGINAL SHEET - ANSI A - COLOR

June, 2016
115302913

Legend:

Borehole 
Cross Section 
Topographical Line (10 m) 
Topographical Line (5 m) 

Notes: ESRI Aerial Imagery; Digital Globe 5/6/2012

Client/Project

PUBLIC WORKS & GOVERNMENT SERVICES CANADA
SINCLAIR CANYON SITE REHABILITATION
HIGHWAY 93, RADIIUM HOT SPRINGS, BC

Figure No.

11.0

Title

Pile Wall Dimensions



200 - 325 25th St. SE Calgary, AB T2A 7H8
Stantec does not certify the accuracy of the data.
This drawing is for reference only and should not be used for construction.



APPENDIX C

Borehole Records

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

<i>Rootmat</i>	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
<i>Topsoil</i>	- mixture of soil and humus capable of supporting vegetative growth
<i>Peat</i>	- mixture of visible and invisible fragments of decayed organic matter
<i>Till</i>	- unstratified glacial deposit which may range from clay to boulders
<i>Fill</i>	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

<i>Desiccated</i>	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
<i>Fissured</i>	- having cracks, and hence a blocky structure
<i>Varved</i>	- composed of regular alternating layers of silt and clay
<i>Stratified</i>	- composed of alternating successions of different soil types, e.g. silt and sand
<i>Layer</i>	- > 75 mm in thickness
<i>Seam</i>	- 2 mm to 75 mm in thickness
<i>Parting</i>	- < 2 mm in thickness

Terminology describing soil types:

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

<i>Trace, or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>Frequent</i>	> 20%

Terminology describing compactness of cohesionless soils:

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
<i>Very Loose</i>	<4
<i>Loose</i>	4-10
<i>Compact</i>	10-30
<i>Dense</i>	30-50
<i>Very Dense</i>	>50

Terminology describing consistency of cohesive soils:

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
<i>Very Soft</i>	<0.25	<12.5	<2
<i>Soft</i>	0.25 - 0.5	12.5 - 25	2-4
<i>Firm</i>	0.5 - 1.0	25 - 50	4-8
<i>Stiff</i>	1.0 - 2.0	50 - 100	8-15
<i>Very Stiff</i>	2.0 - 4.0	100 - 200	15-30
<i>Hard</i>	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	Very Poor Quality
25-50	Poor Quality
50-75	Fair Quality
75-90	Good Quality
90-100	Excellent Quality

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	Crushed
Severely Fractured	Shattered or Very Blocky
Fractured	Blocky
Moderately Jointed	Sound
Intact	Very Sound

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	Extremely Wide	-
2000-6000	Very Wide	Very Thick
600-2000	Wide	Thick
200-600	Moderate	Medium
60-200	Close	Thin
20-60	Very Close	Very Thin
<20	Extremely Close	Laminated
<6	-	Thinly Laminated

Terminology describing rock strength:

Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.

Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Igneous Bedrock	Meta- morphic Bedrock	Sedi- mentary Bedrock

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

measured in standpipe, piezometer, or well

inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
y	Unit weight
G _s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q _u	Unconfined compression
I _p	Point Load Index (I _p on Borehole Record equals I _p (50) in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer



BOREHOLE RECORD

BH-01

CLIENT Public Works and Government Services Canada
 LOCATION Sinclair Canyon, British Columbia
 DATES (mm/dd/yy): BORING 1/26/16 WATER LEVEL Dry on 01/26/16

PROJECT No. 115302913
 BH SIZE 150 mm
 DATUM Geodetic

DEPTH(m)	ELEVATION(m)	SOIL DESCRIPTION	STRATA PLOT	SAMPLES				MONITOR WELL/ PIEZOMETER	UNDRAINED SHEAR STRENGTH - kPa		WATER CONTENT & ATTERBERG LIMITS		Pocket Penetrometer kPa	STANDARD PENETRATION TEST, BLOWS/0.3m	
				TYPE	NUMBER	RECOVERY	N-VALUE OR RQD %		50	100	150	200			W _P
0	1003.07	ASPHALT				mm									
1	1002.90	Brown silty sand (SM) with gravel - dry		BS	1										
2	1000.77	Dense to very dense brown clayey GRAVEL (GC) with sand		BS	2										
3		- frequent cobbles and boulders, angular to sub angular, dry		SS	3	230	41								
4		- some clay below 3.8 m		BS	4										
5		- trace clay below 4.4 m		SS	5	200	50+								
6		- trace coal fragments below 4.4 m		BS	6										
7		- gravelly clay seam at 6.1 m		SS	7	300	50+								
8	995.67	- some clay, dry to moist at 7.3 m		BS	8										
9		Borehole terminated at 7.4 m due to difficult drilling conditions. Borehole sloughed to 7.3 m and was dry upon completion. Borehole backfilled with cuttings. Borehole sealed with bentonite from 0.2 m to 0.4 m and capped with concrete at surface.		SS	9	180	50+								
10															
11															
12															
13															
14															
15															

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BOREHOLE RECORD

BH-02

CLIENT Public Works and Government Services CanadaPROJECT No. 115302913LOCATION Sinclair Canyon, British ColumbiaBH SIZE 150 mmDATES (mm/dd/yy): BORING 1/25/16WATER LEVEL Dry on 01/26/16DATUM Geodetic

DEPTH(m)	ELEVATION(m)	SOIL DESCRIPTION	STRATA PLOT	SAMPLES				MONITOR WELL/ PIEZOMETER	UNDRAINED SHEAR STRENGTH - kPa									
				TYPE	NUMBER	RECOVERY	N-VALUE OR RQD %		50	100	150	200	WATER CONTENT & ATTERBERG LIMITS					
						mm												
0	1004.81	ASPHALT																
1	1004.66	Compact brown silty sand (SM) with gravel - dry		BS	1													
2				SS	2	300	22											
3	1001.81			BS	3													
4		Dense greyish brown clayey GRAVEL (GC) with sand - frequent cobbles and boulders, angular to sub angular, dry - trace roots at 3.4 m		SS	4	200	44											
5	999.51			BS	5													
6		Stiff to very stiff brown sandy silty clay TILL (CL-ML) with gravel - moist - trace roots at 6.1 m		SS	6	150	14											
7	997.91			BS	7													
8		- some sand and gravel below 6.1 m		SS	8	300	10											
9		Dense brown clayey GRAVEL (GC) with sand - angular to sub angular, dry - some sand at 8.4 m		BS	9													
10				SS	10	300	50+											
11				BS	11													
12				SS	12	250	50+											
13				BS	13													
14		- trace oxidation at 10.6 m		SS	14	140	50+											
15	993.41			BS	15													
16				SS	16	100	50+											
17		End of borehole at 11.4 m. Borehole sloughed to 11.3 m. Dry upon completion. 25 mm PVC standpipe installed; handslotted from 7.3 m to 11.3 m. Flush mount install at surface within concrete cap. Backfilled with cuttings and sand. Bentonite seal from 0.3 m to 0.9 m.																
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BOREHOLE RECORD

BH-03

CLIENT Public Works and Government Services CanadaPROJECT No. 115302913LOCATION Sinclair Canyon, British ColumbiaBH SIZE 150 mmDATES (mm/dd/yy): BORING 1/26/16WATER LEVEL Dry on 01/26/16DATUM Geodetic

DEPTH(m)	ELEVATION(m)	SOIL DESCRIPTION	STRATA PLOT	SAMPLES				MONITOR WELL/ PIEZOMETER	UNDRAINED SHEAR STRENGTH - kPa	
				TYPE	NUMBER	RECOVERY	N-VALUE OR RQD %		50	100
0	1007.24									
	1007.14	ASPHALT								
1		Brown silty sand (SM) with gravel - dry		BS	1					
				SS	2	90	50			
2										
3	1004.34	Compact brown clayey GRAVEL (GC) with sand - frequent cobbles and boulders, angular to sub angular, dry		BS	3					
				SS	4	150	12			
4				BS	5					
				SS	6	200	11			
5		- clay seam at 5.3 m		BS	7					
6		- some clay below 5.9 m		SS	8	100	12			
7				BS	9					
				SS	10	50	19			
8	998.84			BS	11					
9		Very stiff greyish brown gravelly silty clay TILL (CL-ML) with sand - angular to sub angular gravel, moist		SS	12	300	16			
10	997.34	- trace roots from 9.1m to 9.2m		BS	13					
	996.94	- silt seam at 9.4 m		SS	14	20	50			
11		Weak grey CLAYSTONE - highly weathered								
12		End of borehole at 10.3 m. Borehole sloughed to 10.1 m and was dry upon completion. Borehole backfilled with cuttings and sand. Sealed with bentonite from 0.2 m to 0.7 m and capped with concrete at surface.								
13										
14										
15										

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APPENDIX D

Laboratory Testing



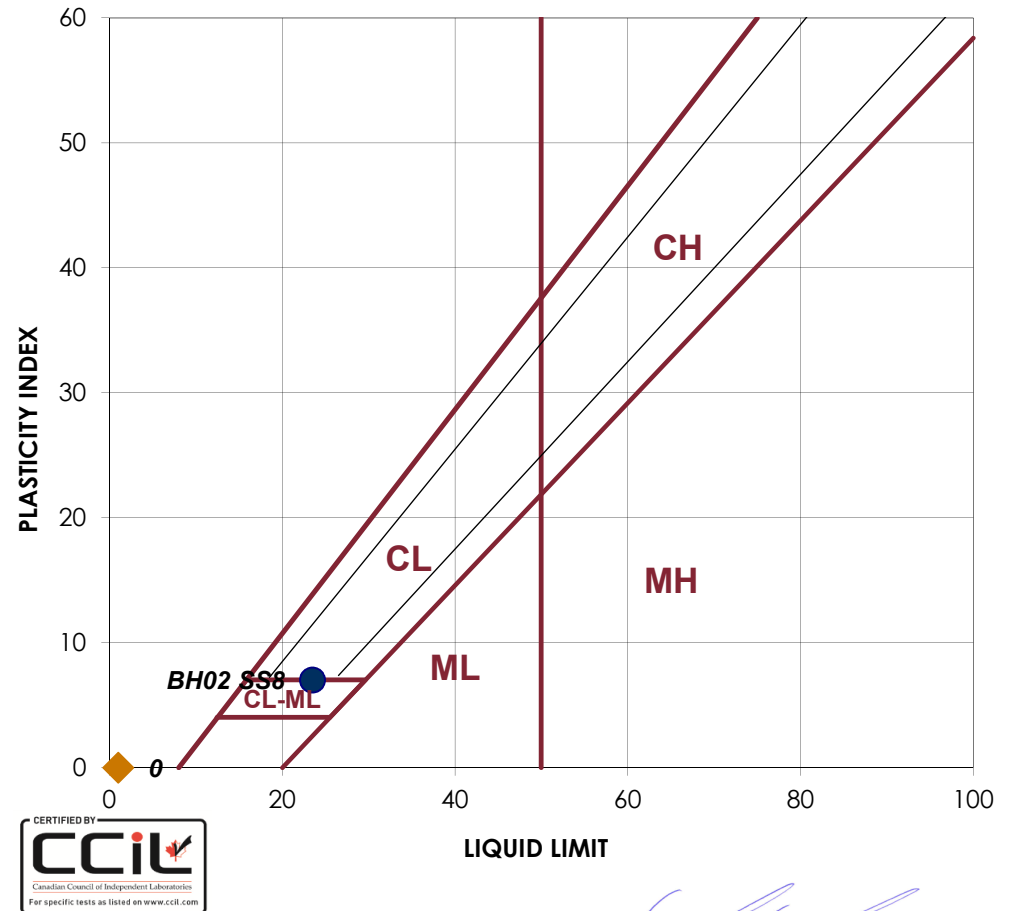
Atterberg Limits
ASTM D4318
Method B- One Point

Client: Public Works & Govt. Services
Project Name: Sinclair Canyon Elec. Upgrade
Project No: 115302913
Date Received: January 26, 2016
Date Tested: February 1, 2016
Tested By: B.Pelkey

OFFICE
325 - 25th Street SE
Suite 200
Calgary, Alberta
Canada T2A 7H8
Tel: (403) 716-8000

LABORATORY
10830 - 46th Street SE
Calgary, Alberta
Canada T2C 1G4
Tel: (403) 253-7876

Sample:		Sample:	
BH-02 SS8			
LIQUID		LIQUID	
1	2	Trial No.	1 2
28	26	Number of Blows	
		Container Number	
40.68	43.34	Wt. Sample (wet+tare) (g)	
33.26	35.37	Wt. Sample (dry+tare) (g)	
1.28	1.25	Wt. Tare (g)	
32.0	34.1	Wt. Dry Soil (g)	
7.4	8.0	Wt. Water (g)	
23.2%	23.4%	Water Content (%)	
23.5%	23.5%	Corrected Water Content (%)	
PLASTIC		PLASTIC	
1	2	Trial No.	1 2
		Container Number	
10.09	11.41	Wt. Sample (wet+tare) (g)	
8.88	10.06	Wt. Sample (dry+tare) (g)	
1.45	1.69	Wt. Tare (g)	
7.4	8.4	Wt. Dry Soil (g)	
1.2	1.4	Wt. Water (g)	
16.3%	16.1%	Water Content (%)	
AVERAGE VALUES		AVERAGE VALUES	
1	2	1	2
LL	23	LL	
PL	16	PL	
PI	7	PI	
CLASSIFICATION		CLASSIFICATION	
CL-ML		NON-PLASTIC	



Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request. The data presented above is for the sole use of the client stipulated above. STANTEC is not responsible, nor can be held liable, for the use of this report by any other party, with or without the knowledge of STANTEC.

Reviewed By: 



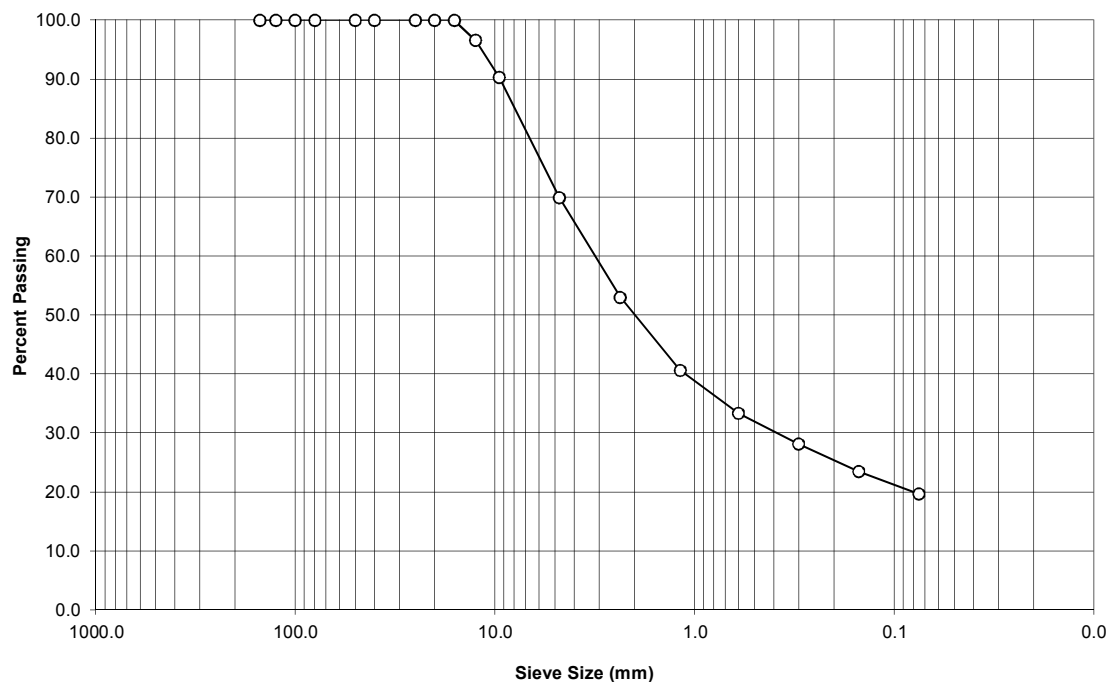
Grain Size Analysis
ASTM C136, ASTM C117

Client: Public Works & Govt. Ser.
Project Name: Sinclair Canyon Elec. Upg.
Project No: 115302913

OFFICE	LABORATORY
325 - 25th Street SE	10830 - 46th Street SE
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Calgary, Alberta	Canada T2C 1G4
Canada T2A 7H8	
Tel: (403) 716-8000	Tel: (403) 253-7876

SAMPLE #: BS1
SOURCE: BH-01
TESTED BY: B.Pelkey

DATE RECEIVED: January 26, 2016
DATE TESTED: January 29, 2016
SAMPLE DESCRIPTION: Sand with Gravel and Fines



—○— % Passing —◇— Upper Limit —☆— Lower Limit

Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
80.0	100.0	-	-
50.0	100.0	-	-
40.0	100.0	-	-
25.0	100.0	-	-
20.0	100.0	-	-
16.0	100.0	-	-
12.5	96.6	-	-
9.5	90.3	-	-
4.75	69.9	-	-
2.36	53.0	-	-
1.18	40.6	-	-
0.600	33.3	-	-
0.300	28.1	-	-
0.150	23.5	-	-
0.075	19.6	-	-

Comments:

Sample Description(USCS) Derived from Grain Size Analysis Only.
Gravel-30.1%
Sand-50.3%
Fines-19.6%

Reviewed by:



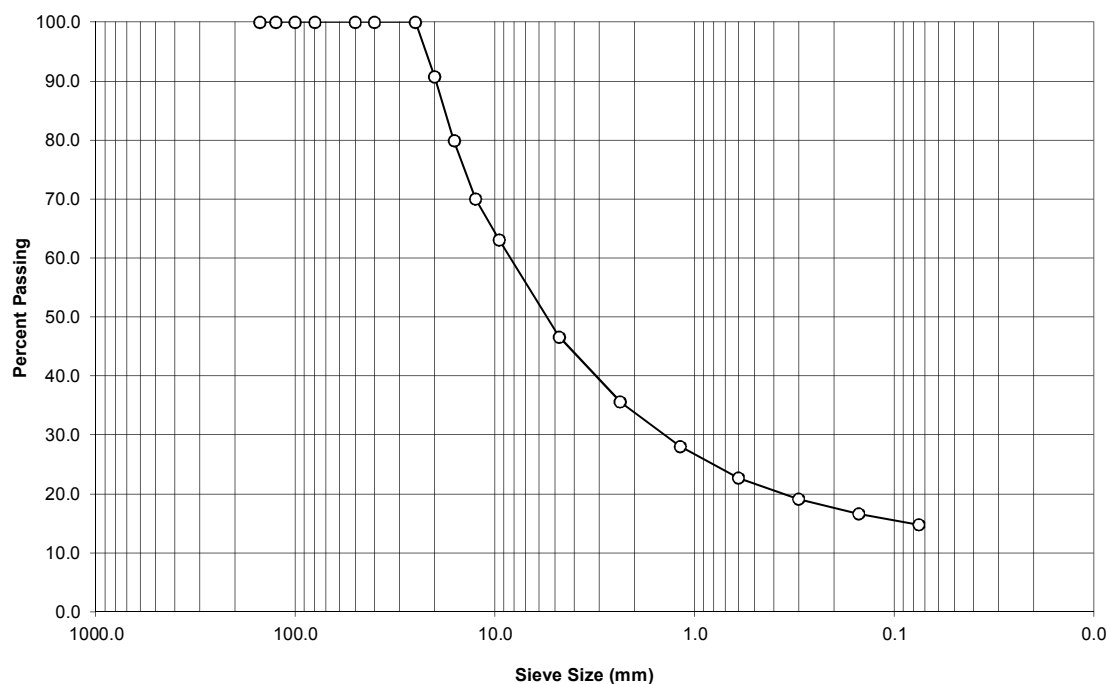
Grain Size Analysis
ASTM C136, ASTM C117

Client: Public Works & Govt. Ser.
Project Name: Sinclair Canyon Elec. Upg.
Project No: 115302913

OFFICE	LABORATORY
325 - 25th Street SE	10830 - 46th Street SE
Suite 200	Calgary, Alberta
Calgary, Alberta	Canada T2C 1G4
Canada T2A 7H8	
Tel: (403) 716-8000	Tel: (403) 253-7876

SAMPLE #: SS7
SOURCE: BH-01
TESTED BY: B.Pelkey

DATE RECEIVED: January 26, 2016
DATE TESTED: January 29, 2016
SAMPLE DESCRIPTION: Gravel with Sand and Fines



—○— % Passing —◇— Upper Limit —☆— Lower Limit

Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
80.0	100.0	-	-
50.0	100.0	-	-
40.0	100.0	-	-
25.0	100.0	-	-
20.0	90.7	-	-
16.0	79.9	-	-
12.5	70.0	-	-
9.5	63.0	-	-
4.75	46.6	-	-
2.36	35.6	-	-
1.18	28.1	-	-
0.600	22.7	-	-
0.300	19.1	-	-
0.150	16.6	-	-
0.075	14.8	-	-

Comments:

Sample Description(USCS) Derived from Grain Size Analysis Only.
Gravel-53.4%
Sand-31.8%
Fines-14.8%

Reviewed by: 

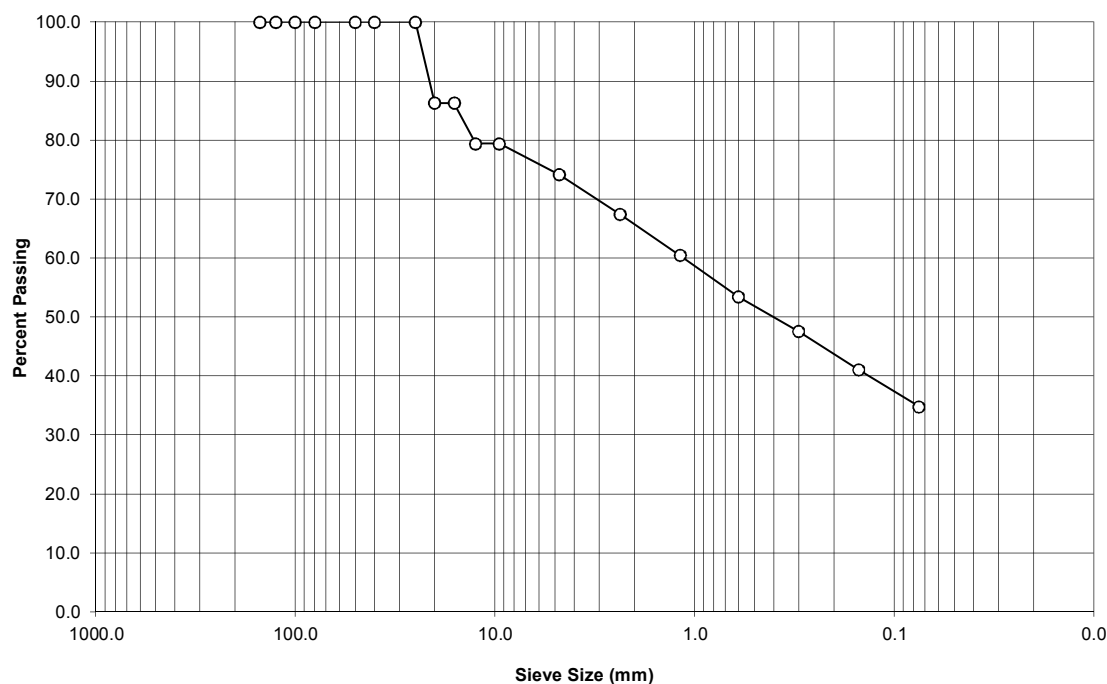


Grain Size Analysis
ASTM C136, ASTM C117

Client: Public Works & Govt. Ser.
Project Name: Sinclair Canyon Elec. Upg.
Project No: 115302913

OFFICE	LABORATORY
325 - 25th Street SE	10830 - 46th Street SE
Suite 200	Calgary, Alberta
Calgary, Alberta	Canada T2C 1G4
Canada T2A 7H8	
Tel: (403) 716-8000	Tel: (403) 253-7876

SAMPLE #: SS8	DATE RECEIVED: January 26, 2016
SOURCE: BH-02	DATE TESTED: January 29, 2016
TESTED BY: B.Pelkey	SAMPLE DESCRIPTION: Silty, Clayey Sand with Gravel



—○— % Passing —◇— Upper Limit —☆— Lower Limit

Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
80.0	100.0	-	-
50.0	100.0	-	-
40.0	100.0	-	-
25.0	100.0	-	-
20.0	86.3	-	-
16.0	86.3	-	-
12.5	79.4	-	-
9.5	79.4	-	-
4.75	74.1	-	-
2.36	67.4	-	-
1.18	60.4	-	-
0.600	53.4	-	-
0.300	47.5	-	-
0.150	41.0	-	-
0.075	34.8	-	-

Comments:

Sample Description(USCS) Derived From Both Grain Size & Atterberg Analysis.
Gravel-25.9%
Sand-39.3%
Fines-34.8%

Reviewed by:



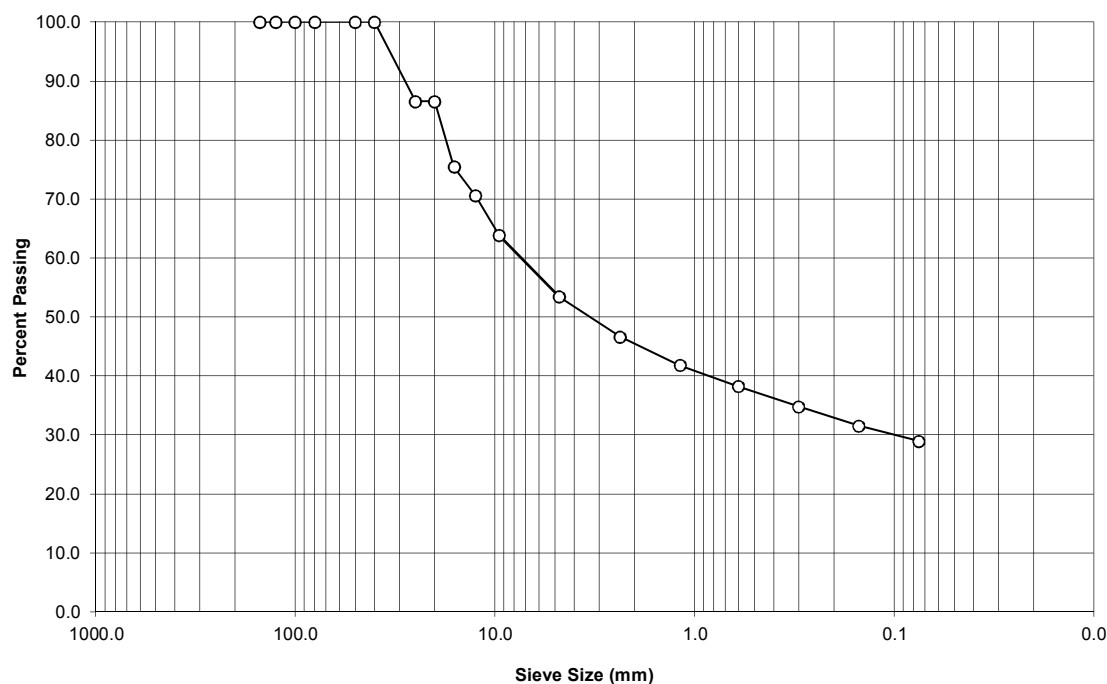
Grain Size Analysis
ASTM C136, ASTM C117

Client: Public Works & Govt. Ser.
Project Name: Sinclair Canyon Elec. Upg.
Project No: 115302913

OFFICE	LABORATORY
325 - 25th Street SE	10830 - 46th Street SE
Suite 200	Calgary, Alberta
Calgary, Alberta	Canada T2C 1G4
Canada T2A 7H8	
Tel: (403) 716-8000	Tel: (403) 253-7876

SAMPLE #: SS6
SOURCE: BH-03
TESTED BY: B.Pelkey

DATE RECEIVED: January 26, 2016
DATE TESTED: January 29, 2016
SAMPLE DESCRIPTION: Gravel with Sand and Fines



—○— % Passing —◇— Upper Limit —☆— Lower Limit

Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
80.0	100.0	-	-
50.0	100.0	-	-
40.0	100.0	-	-
25.0	86.5	-	-
20.0	86.5	-	-
16.0	75.4	-	-
12.5	70.5	-	-
9.5	63.8	-	-
4.75	53.4	-	-
2.36	46.6	-	-
1.18	41.8	-	-
0.600	38.2	-	-
0.300	34.8	-	-
0.150	31.5	-	-
0.075	28.9	-	-

Comments:

Sample Description(USCS) Derived from Grain Size Analysis Only.
Gravel-46.6%
Sand-24.5%
Fines-28.9%

Reviewed by: 



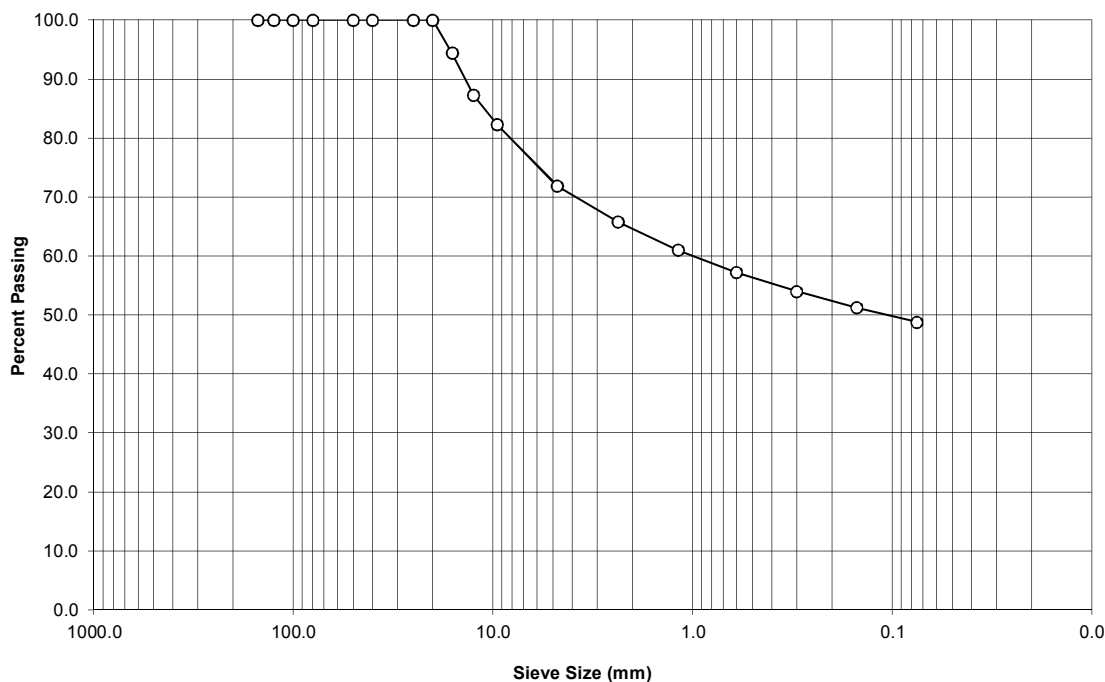
Grain Size Analysis
ASTM C136, ASTM C117

Client: Public Works & Govt. Ser.
Project Name: Sinclair Canyon Elec. Upg.
Project No: 115302913

OFFICE	LABORATORY
325 - 25th Street SE	10830 - 46th Street SE
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Calgary, Alberta	Canada T2C 1G4
Canada T2A 7H8	
Tel: (403) 716-8000	Tel: (403) 253-7876

SAMPLE #: SS12
SOURCE: BH-03
TESTED BY: B.Pelkey

DATE RECEIVED: January 26, 2016
DATE TESTED: January 29, 2016
SAMPLE DESCRIPTION: Gravelly Fines with Sand



—○— % Passing —◇— Upper Limit —☆— Lower Limit

Sieve (mm)	Sample % Passing	Specifications	
		Lower	Upper
150.0	100.0	-	-
125.0	100.0	-	-
100.0	100.0	-	-
80.0	100.0	-	-
50.0	100.0	-	-
40.0	100.0	-	-
25.0	100.0	-	-
20.0	100.0	-	-
16.0	94.4	-	-
12.5	87.3	-	-
9.5	82.3	-	-
4.75	71.9	-	-
2.36	65.8	-	-
1.18	61.0	-	-
0.600	57.2	-	-
0.300	54.0	-	-
0.150	51.2	-	-
0.075	48.8	-	-

Comments:

Sample Description(USCS) Derived from Grain Size Analysis Only.
Gravel-28.1%
Sand-23.1%
Fines-48.8%

Reviewed by: 