CITY OF VANCOUVER

Vancouver
CITY CLERK'S OFFICE
Access to Information & Privacy Division

File No.: 2026-173

May 25, 2026

s.22(1) [redacted]

Dear s.22(1) [redacted]

Re: **Request for Access to Records under the Freedom of Information and Protection of Privacy Act (the "Act")**

I am writing regarding your request dated February 26, 2026 under the ***Freedom of Information and Protection of Privacy Act*** for:

Record of the following reports:

1. **Geotechnical Factual Report - Dunbar Slope Assessment, Apr 26, 2024, Horizon Engineering;**
2. **Geotechnical Conceptual Design Report - Dunbar Slope Assessment, Jan 27, 2025, RAM;**
3. **Archaeological Letter Report for the Dunbar Slope Assessment Project, November 11, 2023, Inlailawatash; and**
4. **Tree Risk and Windthrow Assessment - Point Grey Slope Remediation, May 29, 2024, Blackwell.**

All responsive records are attached*. Some information in the records has been severed (blacked out) under s.15(1), s.17(1), s.18, and s.22(1) of the Act. You can read or download these sections here:
http://www.bclaws.ca/EPLibraries/bclaws_new/document/ID/freeside/96165_00.

*Please note that the requested Archaeological Letter Report has been withheld in its entirety under s.18 of the Act. And, due to the size of the responsive package it has been uploaded to a City of Vancouver secured SharePoint site, see email for access details.

*Please note, due to the file size of the responsive records package it has been uploaded to a City of Vancouver secured SharePoint site. Access details can be located in the email received alongside this letter.

Under Part 5 of the Act, you may ask the Information & Privacy Commissioner to review any matter related to the City's response to your FOI request by writing to: Office of the Information & Privacy Commissioner, info@oipc.bc.ca or by phoning 250-387-5629.

Yours truly,

Kevin Tuerlings, FOI Case Manager, for

[Signed by Kevin Tuerlings]

Siân Madsen, MA, MAS
Acting Director, Access to Information & Privacy

If you have any questions, please email us at foi@vancouver.ca and we will respond to you as soon as possible. You may also contact 3-1-1 (604-873-7000) if you require accommodation or do not have access to email.

:Encl. via SharePoint (Response Package)

:ma

HORIZON
ENGINEERING
A RAM COMPANY

GEOTECHNICAL FACTUAL REPORT
DUNBAR SLOPE ASSESSMENT
DUNBAR AND CAMERON AVENUES, VANCOUVER, BC

CLIENT: CITY OF VANCOUVER

PREPARED BY:
HORIZON ENGINEERING INC
PART OF RAM

220 – 18 GOSTICK PLACE
NORTH VANCOUVER, BC
V7M 3G3
604.990.0546

APRIL 26, 2024
OUR FILE: 122-5174

Table of Contents

Appendices

1.0 INTRODUCTION

Horizon Engineering was retained by the City of Vancouver, referred to as the COV hereafter, to undertake a geotechnical assessment of the existing slopes situated below the north and east termini of Dunbar Street and Cameron Avenue, respectively.

It was understood that failure of the slope located below the east terminus of Cameron Avenue and to the north of the existing beach access staircase occurred during November, 2021, following a period of significant rainfall. The exact date of the failure is unknown. A preliminary geotechnical assessment of the slopes was previously undertaken by Golder Associates of Vancouver, BC, immediately following the aforementioned slope failure. Monitoring of the slopes was also previously carried out on a near-monthly basis by Golder Associates from December 2021 to March 2022 following the slope failure and by the COV during the period of April to September 2022. Horizon Engineering has been undertaking on-going monitoring of the slopes since December 31, 2022, as per our proposed scope of services dated August 18, 2022 for this project.

The purpose of this assessment is to provide recommendations to stabilize the aforementioned slopes and staircase, if determined to be required. The following document is a factual geotechnical report presenting the findings and results of our field investigations which comprised a slope reconnaissance, carried out on October 12, 2022 and subsurface investigations, carried out on January 27 and February 3, 2023. The observations and findings from the previous geotechnical assessment and subsequent slope monitoring site visits carried out by both Horizon Engineering and others are also discussed and referenced herein for background and supplementary information. Our interpretation of the findings from these investigations as well as a summary of the engineering analyses and our recommendations pertaining to risk management of the subject slopes and staircase will be provided in a forthcoming Conceptual Design Report.

This report has also been prepared in general conformance with the aforementioned proposed scope of services.

2.0 SITE DESCRIPTION

2.1 General

The subject site encompasses an approximately 45 meter long portion of the coastal slope which overlooks the south shore of English Bay and extends approximately between Jericho and Kitsilano Beach in the COV. This portion of the slope is bordered by the shore of English Bay to the northeast, the northernmost terminus of Dunbar Street to the south, the residential property of 1509 Dunbar Street to the southeast, the cul-de-sac at the east terminus of Cameron Avenue to the west and another residential property, having a civic address of 3607 Cameron Avenue to the northwest. A site location pan is provided as Figure 1, attached to this report in Appendix A.

The subject portion of the slope faces the northeast direction and is approximately bisected by an existing concrete staircase, oriented in the east-west direction, which provides pedestrian access to the shore of English Bay from Dunbar Street and Cameron Avenue. For description purposes, the portions of the subject slope located to the north and south of this staircase will be referred to as the

"North Slope" and "South Slope", respectively, herein this report. The layout of the site is presented in the Site and Test Hole Locations Plan attached as Figure 2 and appended in Appendix A.

2.2 Beach Access Staircase

The existing concrete staircase is estimated to be approximately 9 metres in height and extend a horizontal distance of approximately 24 metres. The subject staircase spans between four concrete landing pads seated at the upper and mid-slope portions of the subject slope. The lower portion of the staircase had been constructed in between two adjacent, sloped concrete walls. This portion of the staircase and the adjacent walls are supported on top of an approximately 200 millimetre thick concrete slab, which also serves as the lower landing for the staircase.

2.3 North Slope

The topographic LIDAR data provided by the COV online mapping system, VanMap Viewer, indicates that the existing grades at the North Slope are steeply sloping down towards the northeast from geodetic elevations of approximately El. 12 metres at its crest to El. 3 metres at its toe, over horizontal distances of approximately 12 to 14 metres. This equates to slope gradients of approximately 33 to 37 degrees from horizontal. The subject slope is generally overlain by low-lying vegetation comprising shrubs including Himalayan blackberry.

A municipal boulevard is located between the cul-de-sac of Cameron Avenue and the crest of this slope. This boulevard is relatively flat and is currently landscaped with grass and a hedge situated along its southern and western edges. A BC Hydro kiosk, supported on a concrete pad, has been constructed at the western portion of this landscaping area.

2.4 South Slope

Topographic data from VanMap Viewer generally indicates that the South Slope is characterized by steep gradients of approximately 50 to 55 degrees from horizontal. The existing grades at this slope are indicated to slope down from geodetic elevations ranging from approximately EL. 12.5 to 13.0 metres at its crest to approximately El. 3.0 metres at its toe. The cross sectional length of this slope is estimated to vary from approximately 8 to 11 metres based on the aforementioned map. At the time of preparation of this report, the subject slope was covered by low vegetation (i.e. shrubs and bushes) and several trees.

The portion of the site situated between the crest of the South Slope and the roadway at the northern terminus of Dunbar Slope comprises a lawn area. This lawn area encompasses an area of approximately 53 square metres in plan and features two park benches, installed near the crest of the subject slope. The existing grades within this lawn area are generally gently sloping down towards the northwest. A pedestrian pathway is situated to the west of this lawn and between the crest of the South Slope and the residential property of 1509 Dunbar Street. This pathway is oriented parallel to the crest of the subject slope and extends from the aforementioned lawn area to the cul-de-sac of Cameron Avenue. The longitudinal profile of the pathway slopes gently down from the lawn area to the uppermost landing of the beach access staircase. From this landing, the existing grades of the pathway flatten towards the cul-de-sac of Cameron Avenue. The existing finished grades of the pathway are elevated approximately 600 to 750 millimetres above the crest of the South Slope and are currently retained by a concrete retaining wall. This retaining wall is estimated to be approximately 600 to 750 millimetres in height and is constructed at the crest of the subject slope. There is another retaining wall bordering the pathway to the south which is currently managing the grade differences between the pathway and the backyards and driveways of the adjacent residential properties located above and to the south.

2.5 Beach Front

The beach deposits overlying the shoreline of English Bay were generally observed to consist of a mixture of sand, gravel, cobbles and scattered boulders. The topography at the shoreline is gently sloping down towards the north.

3.0 BACKGROUND INFORMATION

3.1 Document Review

For the purpose of preparing this report, we have received, read and interpreted the following documents, maps and email correspondence related to the project and site:

- Surficial Geology Map (Map 1486a), dated 1979 and prepared by Geological Survey of Canada (GSC),
- Slope monitoring field review report dated December 9, 2021, prepared by Golder Associates,
- Email correspondences, dated December 22 and 28, 2021, January 4, February 1 and March 9, 2022, between Golder Associates and the COV,
- "Geotechnical Slope Assessment Report for 1500 Block Dunbar Street, Vancouver, BC", dated April 7, 2022 and prepared by Golder Associates Ltd.,
- Slope monitoring field reports dated March 9, April 29, July 12, August 17 and September 23, 2022, prepared by the COV,

3.2 Surficial Geology

The subject site is indicated in the above-listed GSC map to be situated near the interface in surface expression of Vashon Drift and Capilano Sediments, and Tertiary Bedrock.

The Vashon Drift deposits are expected to comprise glacial drift including lodgement and minor flow till, lenses and interbeds of substratified glaciofluvial sand to gravel and lenses and interbeds of glaciolacustrine laminated stony silt. These deposits are expected to be overlain by Capilano Sediments comprising marine and glaciomarine stony (including till-like deposits) to stoneless silt loam to clay loam with minor sand and silt.

The Tertiary Bedrock is indicated to include "sandstone, siltstone, shale, conglomerate and minor volcanic rocks". Where bedrock is not exposed it is it is expected to be overlain by up to 10 metres of the previously described glacial deposits.

3.3 Previous Slope Assessment (by Golder Associates)

A synopsis of the findings from the preliminary assessment completed by Golder Associates is provided below.

Based on our review of the aforementioned geotechnical assessment report, it is understood that failure of the portion of the slope located to the north of the existing staircase had occurred in November 2021, following a period of significant rainfall. A visual assessment of the slope was subsequently carried out by Golder Associates on December 9, 2021. The slope failure was identified to be in the form of a shallow landslide which initiated at a setback of 10 metres to the east of the cul-de-sac of Cameron Avenue. The landslide scarp was indicated in the report to be approximately 15.2 metres in length and approximately 4.8 and 8.2 metres wide at the top and bottom, respectively. The landslide was also identified to have occurred within the surficial vegetation and mineral soil layers mantling the North Slope. Other indicator signs of slope movement/instability documented in the report included the development of tension cracks at the crest of the failed portion of the slope as well as a shallow linear depression located to the north of the slide area and parallel to the crest of the slope.

The report states that no indicator signs of slope instability were evident at the portion of the slope located to the south of the staircase. Two cavities, understood to have been anthropogenically developed, were observed at the toe of this portion of the slope. The cavities were indicated to be approximately 1.5 metres wide and extend 2 metres into this portion of the slope which has resulted in the undermining of the toe of the slope. Furthermore, the cavities were found to be connected at the back. It was noted in the assessment that wave action during periods of high tide was impacting the toe of the slope and the root mass of several trees at the toe of the south slope was currently providing support to the slope at its base (i.e. at the time that the Golder report was authored). However, the toe of the slope was indicated to be susceptible to tidal-induced erosion which would also potentially result in expansion of the existing cavities and in turn, trigger slope instability.

The report indicates that the failure of the north slope was likely induced by a heavy rainfall event which occurred during November, 2021. However, the depression at the north side of the slope was inferred to be an indicator sign of previous slope movement. Furthermore, as the slope was determined to be subject to erosion at its toe due to wave action, this could have resulted in over-steepened and unstable geometries overtime.

Recommendations pertaining to both short and long-term slope stability management for the subject site were also provided in the report. These recommendations included the following options:

Short Term Measures

- Restricting public access to beach access staircase,
- Periodic slope monitoring to assess the conditions of the slopes for any changes and additional signs of slope movement/instability,
- Monitoring the toe of the slope for any impactful human activity, and
- Slide debris which had deposited at the toe of the North Slope was recommended to be maintained in place in order to provide short term protection of the slope against tidal erosional action.

Long Term Measures

- Development and implementation of a long-term slope monitoring plan,
- Permanent closure of the existing beach access staircase, and/or
- Re-grading the failed portion of the North Slope and installing rip-rap armouring at the toe of this slope for retention of loose material and protection against wave action. The recommended remedial measures to address the undermined portion of the south slope included backfilling the existing cavities and installation of rip-rap armouring.

3.4 Previous Slope Monitoring Observations

As previously mentioned, the conditions of the subject slopes were monitored by Golder Associates during the period of December, 2021 to March, 2022 and by the COV from April to September, 2022. The observations from this monitoring period, documented by others, are summarized in Table 1, included in Appendix B of this document for reference. This table and a summary of the slope monitoring observations made by others and Horizon Engineering were previously provided in the slope monitoring memorandum dated March 16, 2023, however, it is also presented herein as it is relevant factual information for the project.

As summarized in the aforementioned table, additional signs of slope deterioration in the form of soil ravelling and sloughing were observed at the North Slope during the February 1 and March 9, 2022 site visits and at the South Slope during the March 9, 2022 site visit. It is understood that no discernable changes in the conditions of the North Slope, relative to the conditions documented in the March 9, 2022 site visit have been observed by the COV throughout the period of April to September, 2022. No comments regarding the conditions of the South Slope were included in the field review reports prepared by the COV. As summarized in Horizon Engineering's slope monitoring memorandum dated March 16, 2023, the conditions of both slopes were generally noted to have remained unchanged, relative to the conditions observed by the COV during their March 9, 2022 site visit, throughout Horizon Engineering's monitoring period of December 31, 2022 to March 6, 2023.

4.0 FIELD INVESTIGATIONS

4.1 General

The field investigations undertaken for this slope assessment comprised the following:

- An initial slope reconnaissance carried out on October 7, 2022 to visually assess the conditions of the slopes and staircase,
- Subsurface investigation, comprising two exploratory test holes advanced with a sonic drill rig on January 27, 2023. These test holes were advanced at the portions of the roadways situated at the north and east termini of Dunbar Street and Cameron Avenue, respectively, and
- Subsurface investigation, comprising a total of five WildCat Dynamic Cone Penetrometer Test Soundings advanced near the accessible portions of the crests of the North and South Slopes on February 3, 2023.

It should be noted that the landscaping and lawn areas situated adjacent to the crests of these slopes were previously identified during the initial site reconnaissance as being ideal test hole locations for providing optimal representative stratigraphic profiles of the slopes. However, the existing hedge planted at the east end of Cameron Avenue and an existing road barrier at the terminus of Dunbar Street precluded drill rig access to these test hole locations. Furthermore, based on email correspondence dated October 24, 2022 from the COV, it was understood that these features could not be removed to accommodate the investigation due to schedule and budgetary constraints. Although the test holes were not advanced at the ideal locations, as previously described, it is envisaged that actual test hole locations were still suitable for the purposes of this assessment.

As required, a Street-Use Permit was obtained from the COV to facilitate the aforementioned subsurface investigations. The locations of traceable underground services underlying the work areas were delineated by a utility locate subcontractor (FJM Utility Locate of Coquitlam, BC) on January 24, 2023 to reduce the risks of damaging them during the subsurface investigations. Ground Penetrating Radar (GPR) and Electromagnetic (EM) techniques were used to locate the underground utilities.

s.18

[redacted]

4.2 Slope Reconnaissance

On October 7, 2022, Mr. Robert Ng, P.Eng. and Mr. Jason Tam, P.Eng., both of Horizon Engineering, attended the subject site to carry out a slope reconnaissance. Representatives from the COV were also in attendance for a portion of this reconnaissance. The reconnaissance consisted of visual site observations with respect to slope stability and measurements of existing slope geometries at locations that were assessed to be practical.

This field assessment was completed under sunny weather conditions and during a period of low tide. It should be noted that the photographs referenced below were taken during this initial reconnaissance.

4.2.1 North Slope

At the time of our slope reconnaissance, the North Slope was observed to be densely vegetated with shrubs. During this site visit, visual observations of the slope conditions were made from the crest, toe and beach access staircase and measurements of the slope geometry and landslide failure surface were also taken. Our observations and measurements for the North Slope are summarized below:

- The subject slope was estimated to have an overall gradient of approximately 35 degrees from horizontal based on slope angle measurements taken with an inclinometer, which is generally consistent with the slope angles estimated from the available topographic data. The overall slope length was measured to be approximately 18.5 metres.
- The footprint of the landslide was inferred to encompass the upper portion of the subject slope based on the geometry of the portion of the landslide channel which was not covered with vegetation and visible at the time of this reconnaissance.
- The landslide was measured to have initiated at a setback distance of 5.1 metres from the closest edge of the BC Hydro kiosk located within the western portion of the landscaping area situated between the North Slope and the cul-de-sac of Cameron Avenue.
- As shown in Photograph 1 of Figure 3, the headscarp of the landslide was approximately arc-shaped in plan and measured to be 1.1 meters in height. The headscarp area was observed to be

moderately vegetated with small shrubs and exhibited a near-vertical profile. Where exposed, the soils at the headscarp area of the landslide generally consisted of brown silty sand with some to gravelly. No obvious indicator signs of seepage were observed at the headscarp area of the landslide.

- Where visible/exposed, the landslide surface which had developed below the headscarp was estimated to be inclined at gradients varying from approximately 33 to 40 degrees from horizontal.
- A traffic barrier was located mid-slope and within the landslide channel. The traffic barrier was measured to be situated at a distance of approximately 4.3 metres, measured parallel to the slope, from the crest of the slope.
- Mixed landslide debris, consisting of uprooted vegetation and brown silty sand, with some gravel had deposited at the mid and downslope portions of the slope. The distance between the top of the debris deposit and the crest of the subject slope was approximately 7.5 metres (measured parallel to the slope). This landslide deposit is approximately delineated in Photograph 2 of Figure 3 for reference. As shown in Photograph 3 of Figure 4, the landslide deposit at the bottom of the slope was measured to be approximately 900 millimetres in thickness.
- As also indicated in Photograph 2 of Figure 3, the landslide channel was measured to be approximately 4.2 and 6.0 metres wide at the top and bottom, respectively.

Based on the above observations, the landslide was inferred to have been triggered within the loose, surficial, sandy deposits mantling the subject slope.

4.2.2 South Slope

The South Slope was observed to be overlain by low lying vegetation (i.e. shrubs and bushes) and small trees. The subject slope was estimated to be generally steeply sloping at gradients of approximately 50 degrees from horizontal. The section of this slope situated below the lawn arealocated at the north terminus of Dunbar Street was noted to steepen to near-vertical at the bottom 3 metres where bedrock outcrops comprising sandstone/siltstone were present. A photograph of the South Slope, looking upslope from the beach is provided as Photograph 4 in Figure 4.

A ground traverse of the subject slope completed during this site visit indicated that the subject slope is overlain by a thin deposit of inferred fill materials comprising light grey to brown sandy silt mixed with trace to some organics (i.e. roots and rootlets) and trace amounts of various debris, which included plastic bags and beverage containers. A photograph of the typical surficial materials overlying the slope are shown in Photograph 5. An exposure of light brown to grey silt, inferred to be a hard, natural material was also observed at the mid-slope portion of this slope, as shown in Photograph 6 of Figure 5. No obvious indicator signs of slope instability were observed at the South Slope, however, as previously documented in our slope monitoring memorandum dated March 16, 2023, an approximately 38 millimetre wide gap was present between the existing pathway and backside of the retaining wall that is currently supporting it, located at the crest of this slope. As discussed in the memorandum, it is possible that the presence of this gap could be consistent with previous movement of the slope and/or retaining wall.

The bottom 1.8 metres of the sandstone/siltstone outcrops situated at the bottom of the South Slope were observed to be wet at the time of our site reconnaissance and exhibited some local seepage. The sandstone/siltstone exposed at this outcrop was generally noted to be light brownish grey in colour and exhibited some prominent, weathered, dark brown discontinuities. These discontinuities were noted to be striking in the approximate north-south direction and dipping down to the east. It is envisaged that these discontinuities may be potential slip or erosional surfaces. A photograph showing a portion of the bedrock outcrop and its discontinuities is provided in Photograph 7 of Figure 6.

The geometries of the existing cavities situated at the bottom portion of the south slope located near the staircase were generally noted to be consistent with those previously described in Section 3.3 and documented in the preliminary assessment report prepared by Golder Associates.

4.2.3 Beach Access Staircase

At the time of our slope reconnaissance, we did not observe any obvious indicator signs of structural distress of the staircase or undermining of its landing pads. However, as shown in Photograph 8 of Figure 6, the lower portion of the staircase is abutted by an approximately 900 millimetre thick concrete slab located to the north of the adjacent concrete sidewall. It is possible that this concrete slab was previously placed to buttress this portion of the staircase.

During our slope reconnaissance, a 1.0 metre long soil probe was also advanced through the accessible portions of the slope located to the north of and adjacent to the second and third landing pad foundations, numbered from the top of the slope, in an effort to qualitatively assess the relative denseness/consistency of the surficial slope materials and the subgrade supporting the staircase. Loose/soft soil conditions were inferred to have been encountered from surface at the soil probe test locations, based on the resistance of these surficial slope materials to penetration with the probe. These loose soil conditions generally extended to at least the maximum depths of the soil probe tests (i.e. 1.0 metre). Where exposed, the surficial slope materials encountered at the soil probe test locations generally comprised brown, fine to medium grained sand with trace to some gravel.

It should also be noted that the minimum horizontal setback distance between the southern sidewall of the landslide channel of the North Slope and the closest edge of the landing pad foundations was measured to be approximately 1.2 metres. The founding depths of the landing pads could not be confirmed at the time of this site reconnaissance.

The observed conditions of the staircase are shown in Photograph 8 of Figure 6.

4.3 Sonic Drilling and Piezometer Installation

A track-mounted sonic drill rig, supplied and operated by Blue Max Drilling of Surrey, BC, was mobilized to site on January 27, 2023 to advance test holes at the roadways situated at the North and East termini of Dunbar Street and Cameron Avenue, respectively. The test holes advanced at the roadways of Cameron Avenue and Dunbar Street are denoted as “SH23-01” and “SH23-02”, respectively, in the site and test hole locations plan attached as Figure 2. SH23-01 was advanced at a setback distance of approximately 6.0 metres east of the crest of the North Slope whereas SH23-02 was advanced at a setback distance of approximately 10.0 metres south of the crest of the South Slope.

A sonic drill is a rotary vibratory drill casing capable of extracting continuous cores. The sonic drill head works by sending high frequency resonant vibrations down the drill casing string to the drill bit. The frequency ranges between 50 and 150 Hertz (cycles per second) and is varied to optimally suit the subsurface conditions. These vibrations cut and cause the surrounding soil at the drill string and bit to ‘fluidize’ which allows for relatively easy penetration into the soil layers and samples to be removed from the borehole. The sonic drill holes were approximately 200 millimetres in diameter.

The sonic drill holes were advanced to depths of 12.1 metres below the finished grades of the aforementioned roadways. These investigation depths were assessed to be sufficient for slope stability analyses purposes in consideration of the geometries of the slopes, the subsurface conditions encountered within each test hole and the observations from our slope reconnaissance. In order to evaluate the in-situ relative densities of the subsurface materials encountered at each test hole, Standard Penetration Test (SPT) soundings were advanced within the sonic drill holes. The SPT involves driving a split spoon sampler, having a diameter of 50 millimetres, into the soil with a 63.5 kilograms drop hammer falling through a height of 750 millimetres. The number of blows required to drive the split spoon sampler 450 millimetres is recorded and the number of blows required to advance the rod from 150 to 450 millimetres is reported as the “N” value, which can then be correlated to the relative denseness of the soil. SPT soundings were advanced at depth intervals of 1.5 metres within the upper 4.5 metres of the subsurface at the test hole locations. At depths below 4.5 metres, the SPT soundings were advanced at irregular depth intervals, varying between 1.5 and 3.0 metres.

Two piezometers, comprising 25 and 50 millimetre diameter PVC pipes, were installed in nested configurations at each borehole to facilitate monitoring of groundwater levels within different depths and strata. The nested piezometers installed in borehole SH23-01 are denoted as “PZ23-01” whereas the nested piezometers installed in SH23-02 are denoted as “PZ23-02” in Figure 2. The 25 millimetre and 50 millimetre diameter piezometers within each borehole were installed to respective depths of approximately 3.0 and 9.0 metres below the existing roadway grades at each test hole location. The intake zones of the shallow and deeper piezometers were developed with respective screen lengths of 1.5 and 3.0 metres. The annuli between the screened intake zones of the PVC standpipe piezometers and boreholes were backfilled with filter sand. The annular spaces located below, between and above the filter media of the nested piezometers were sealed with bentonite chips. The upper bentonite seals of the nested piezometers were developed up to approximately 450 millimetres below the finished roadway grades. Filter sand was placed as backfill within the annular spaces above the upper seals of the piezometers.

Upon completion of the piezometer installations, the tops of the piezometers were capped with ‘J-plugs’. The boreholes and piezometers within were covered with flush-mount well covers embedded in concrete placed between the well cover and adjacent pavement of the roadways. The aforementioned details regarding piezometer depths, configurations and backfill materials are also shown schematically in the test hole logs attached to this report. We recommend that the piezometers remain capped and secured during the period of service and be decommissioned as per the British Columbia Groundwater Protection regulations under Water Sustainability Act (February 29, 2016) at the end of their service lives.

The sonic drilling investigation was directed and supervised by a field engineer from our office. The soil stratigraphy was logged in the field and select SPT and grab soil samples were collected for further soil characterization.

4.4 WildCat Dynamic Cone Penetrometer Tests

WildCat Penetrometer Test soundings, which are carried out utilizing portable and hand operated equipment, were advanced near the crests of the subject slopes as these portions of the site were not accessible with drill rigs. A total of two and three WildCat Penetrometer Test soundings were advanced near the crests of the North and South Slopes, respectively, and within the municipal landscaping and lawn areas situated adjacent to these slopes. The approximate locations of the WildCat Penetrometer Test soundings are indicated as WC23-01 to WC23-05 in Figure 2. WC23-01 through WC23-03 were advanced near the crest of the South Slope whereas WC23-04 and WC23-05 were advanced near the crest of the North Slope.

The WildCat Cone Penetrometer Test entails driving a steel cone, attached to a series of steel rods, below the subsurface with a 16 kilogram drop hammer raised to a height of 380 millimetres. Blow counts are logged for every 100 millimetres of penetration of the rods below ground surface. Correlations of blow counts are made to equivalent SPT N values which are then used to assess the in-situ density/consistency of the subsurface materials. Soil samples are not retrieved during the WildCat Penetrometer Tests.

The WildCat Cone Penetrometer Test soundings were advanced to delineate the thicknesses of any loose deposits that may be underlying the crests of the aforementioned slopes which would inform the development of the slope stability analyses models for this assessment. As further described in Section 5.2 below, these test holes were terminated at depths ranging from 1.6 to 2.3 metres as effective refusal was encountered at these depths.

The WildCat Penetrometer Test soundings were carried out by two field technicians from our office.

5.0 SUBSURFACE CONDITIONS

Summaries of the subsurface conditions encountered at the sonic and WildCat Penetrometer Test hole locations are provided below. It should be noted that the depths referenced below are relative to the existing site grades at the respective test hole locations.

Detailed descriptions of the subsurface materials encountered at each sonic drill hole are provided in the test hole logs appended to this report. As indicated in the logs, the collar elevations of SH23-01 and SH23-02 were estimated to be at approximately El. 12.0 and 14.2 metres based on available topographic data for the subject site.

5.1 Soil Stratigraphy

The sonic test hole locations were found to be surfaced with approximately 300 and 75 millimetre thick layers of concrete and asphalt pavement at SH23-01 and SH23-02, respectively. The surficial pavement layers were found to be underlain by fill materials, comprising sand with varying gradations and silt and gravel contents. These fill materials were inferred to be compact and extended to depths of approximately 0.9 metre at SH23-01 and 2.0 metres at SH23-02. At SH23-01, these fill materials were underlain by an approximately 1.1 metre thick layer of reddish to light brown sand which was inferred to be a natural and compact material. At SH23-02, the fill materials were underlain by a 700 millimetre thick layer of light grey mottled brown to brown silt, which was inferred to be natural and firm to stiff in consistency. The aforementioned sand and silt were in turn underlain by a hard/very dense, grey to light brown silt and sand to sand and silt which was approximately 1.2 and 3.1 metres in thickness at SH23-01 and SH23-02, respectively, and found to be slightly lithified at SH23-01. This sand was found to grade into a hard, slightly lithified, grey sandy silt at depths of approximately 3.2 metres at SH23-01 and 5.8 metres at SH23-02. A discrete, 200- to 500-millimetre-thick stratum of dark brown silt with some fine-grained sand, inferred to be very stiff, and inferred to comprise decomposed organic matter was encountered below the aforementioned sandy silt at depths of approximately 5.3 and 7.6 metres at SH23-01 and SH23-02, respectively. Bedrock, characterized as extremely weak to weak siltstone, was found to underlie the aforementioned dark brown silt at depths of approximately 5.8 and 7.7 metres below the existing grades at SH23-01 and SH23-02, respectively. Both test holes were terminated at depths of 12.1 metres within the siltstone stratum.

A generalized stratigraphic profile of the subsurface materials encountered at the drill hole locations is summarized below in order of increasing depth:

5.1.1 Concrete/Asphalt Pavement

Approximately 300 and 75 millimetre thick layers of concrete and asphalt pavement were encountered at the existing ground surfaces of SH23-01 and SH23-02, respectively.

5.1.2 Fill

At SH23-01, the fill materials comprised, brown, moist, fine grained silty sand which graded into a reddish brown, moist, fine to medium grained sand at depths of approximately 900 millimetres. The reddish brown sand in turn, graded into a light brown, moist, well graded sand at depths of 1.2 metres. The fill materials at this test hole extended to a depth of approximately 2.0 metres.

At SH23-02, the fill materials comprised from top, 525 millimetres of a grey, dry, well graded sand with some gravel which graded into a brown, dry to moist, fine to medium grained sand. This well graded sand was underlain by 300 millimetres of brown to dark brown, moist, fine to medium grained sand. At depths below 900 millimetres, the inferred fill materials comprised grey, dry to moist, well graded sand with trace to some gravel contents which was in turn underlain by light grey to light brown, moist, silt with some sand to sandy, trace gravel, occasional cobbles, trace clay, and trace to no organic contents (i.e. rootlets). The fill materials at this test hole extended to a depth of approximately 2.7 metres.

The previously described fill materials were inferred to be compact/stiff based on the SPT soundings advanced within these materials and the observed drilling efforts.

5.1.3 Sand

At SH23-01, the fill materials were underlain by inferred natural materials comprising reddish brown, moist, fine to medium grained sand with trace gravel at depths of approximately 900 millimetres. The reddish brown sand was found to grade into a light brown, moist, well graded sand at depths of 1.2 metres. This sand layer was inferred to be compact to dense and found to be extend to depths of approximately 2.0 metres below existing site grades at SH23-01.

5.1.4 Silt

At SH23-02, the fill materials were found to be underlain by inferred natural materials comprising light grey mottled brown, moist, sandy silt with fine grained sand, trace gravel, and occasional cobbles which was inferred to be stiff. Below depths of 2.3 metres, the silt was observed to be light brown in colour and contain some sand and trace clay contents.

5.1.5 Silt and Sand to Sand and Silt

At both test hole locations, the previously described sand and silt were underlain by a moist, silt and sand to sand and silt which was inferred to be hard/very dense as the SPT soundings encountered effective refusal within this stratum.

At SH23-01, this stratum was found to be light brown in colour, slightly lithified and approximately 1.2 metres in thickness. The thickness of this stratum was found to increase to 3.1 metres at SH23-02, and the colour of the upper 1.7 metres of this stratum encountered at SH23-02 was noted to be grey. The colour of these materials encountered at SH23-02 was noted to grade into light brownish grey at a depth of approximately 4.4 metres.

5.1.6 Sandy Silt

A grey, moist, sandy silt (with fine grained sand) was encountered below the previously described sand stratum. This material was inferred to be natural, slightly lithified and hard in consistency based on the SPT soundings. This layer was found to be approximately 2.1 metres and 1.8 metres thick at SH23-01 and SH23-02, respectively.

5.1.7 Silt

The aforementioned sandy silt to silt and sand was found to be underlain by a relatively thin layer of dark brown, moist silt with trace to some fine grained sand. It should be noted that the dark brown coloration of this material may be an indication that it consists of decomposed organic matter. These materials were inferred to be partially petrified and very stiff to hard in consistency based on the SPT sounding advanced through these materials at SH23-02. This stratum was found to be approximately 500 and 200 millimetres thick at SH23-01 and SH23-02, respectively.

5.1.8 Bedrock (Siltstone)

Bedrock, comprising siltstone, was encountered at depths of approximately 5.8 and 7.7 metres below existing site grades at SH23-01 and SH23-02, respectively. At SH23-01, the siltstone was inferred to be very weak to weak and generally comprised grey, moist silt with some fine grained sand. At SH23-02, the siltstone was generally inferred to be weak and comprise light brownish grey to grey, moist, sandy silt with 600 millimetre thick interbeds of grey, moist, slightly plastic silt containing trace fine grained sand and trace clay. These interbedded layers of siltstone, comprising slightly plastic silt, were inferred to be extremely weak in strength and encountered at depths of 9.1 and 10.7 metres.

Both test holes were terminated within this stratum.

5.2 WildCat Dynamic Cone Penetrometer Test Soundings

WildCat Cone Penetrometer Test logs for WC23-01 to WC23-05 are attached to this document in Appendix B. WC23-01 to WC23-03 were advanced adjacent to the crest of the south slope and within the lawn field situated above this slope. WC23-04 and WC23-05 were advanced adjacent to the crest of the North Slope and within the landscaping area located above this slope. A summary of the soil conditions encountered at these test hole locations is provided below and is based on the correlation between the blow counts recorded for the WildCat Penetrometer Test soundings and SPT "N" values.

5.2.1 South Slope (WC23-01 to WC23-03)

WC23-01 to WC23-03 encountered very loose to loose/soft to firm soil conditions from the existing ground surface at these test hole locations to depths of 1.3 metres. Compact/stiff soil conditions were in turn encountered below these depths which extended to 1.5 to 1.6 metres below the existing site grades at which dense to very dense/hard soil conditions were encountered. Effective refusals of these soundings were encountered at depths of 1.8 metres at WC23-01 and 1.6 metres at WC23-02 and WC23-03.

5.2.2 North Slope (WC23-04 and WC23-05)

Very loose to loose soil conditions were encountered from ground surface to depths of 2.0 and 1.7 metres at WC23-04 and WC23-05, respectively. The relative denseness/consistencies of the subsurface materials at these test holes were found to transition to compact to dense/stiff to very stiff below the aforementioned depths and effective refusals were encountered at depths of 2.3 and 2.1 metres at WC23-04 and WC23-05, respectively.

5.3 Groundwater Conditions

The groundwater levels at the piezometer locations were measured on March 9, 2023, 41 days following piezometer installation. At PZ23-01, installed at the east terminus of Cameron Avenue, the groundwater levels measured within the shallow and deeper piezometers were situated at respective depths of approximately 2.1 and 6.2 metres below the existing roadway grades at this test hole location which corresponds to respective geodetic elevations of approximately El. 10.0 and El. 5.9 metres. At PZ23-02, installed at the north terminus of Dunbar Street, the shallower piezometer was found to be dry on the subject date whereas the groundwater level within the deeper piezometer was measured at a depth of 4.7 metres below the existing roadway grades at this piezometer location. This groundwater depth corresponds to a geodetic elevation of El. 9.5 metres.

The shallow groundwater level measured at PZ23-01 is interpreted to be indicative of a perched groundwater table situated near the interface of the upper, compact to dense sand and the underlying natural materials comprising very dense sand encountered at this test hole location. The deeper groundwater levels measured at the piezometers are inferred to be representative of ambient/static groundwater levels.

6.0 Closure

This report has been prepared for the sole use of the City of Vancouver and other consultants for this project, as described. Any use or reproduction of this report for other than the stated intended purpose is prohibited without the written permission of Horizon Engineering Inc.

We are pleased to be of assistance to you on this project and we trust that our comments and recommendations are both helpful and sufficient for your current purposes. If you would like further details or require clarification of the above, please do not hesitate to call.

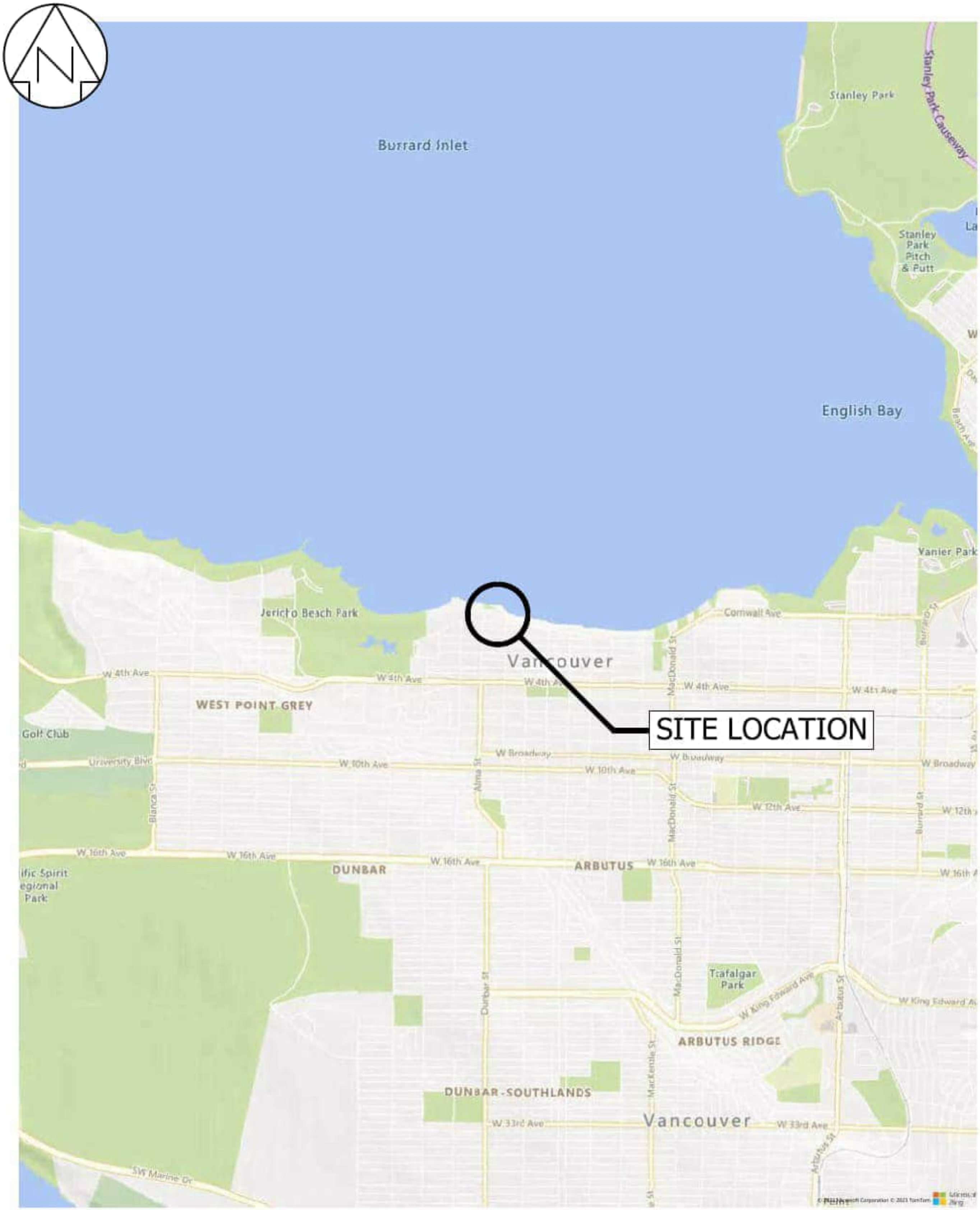
For:
HORIZON ENGINEERING INC.
s.15(1), s.22(1)
Jason Tam, P.Eng.
Geotechnical Engineer

For:
HORIZON ENGINEERING INC.
s.15(1), s.22(1)
Karen Savage, P.Eng., FEC
President

PERMIT TO PRACTICE No.1001550
ENGINEERS AND GEOSCIENTISTS BRITISH COLUMBIA

Appendix A

FIGURES



13-Apr-23, 1:48:07 PM, Leda, N:\2022 Projects\122-5174 VAN COV Dunbar Slope\Drawings\122-5174 F1 & F2 Site and Test Location Plans.dwg

CITY OF VANCOUVER
453 West 12th Ave, Vancouver, BC

GEOTECHNICAL ASSESSMENT
Dunbar Slope, Vancouver, BC

SITE LOCATION PLAN

HORIZON
ENGINEERING INC

Scale: NTS	File No: 122-5174	Date: APR/2023	FIGURE: 1
Des: JT	Dwn: LF	Chk: KS	Rev:

Reference: City of Vancouver Aerial Photography

CITY OF VANCOUVER
453 West 12th Ave, Vancouver, BC

GEOTECHNICAL ASSESSMENT
Dunbar Slope, Vancouver, BC

TEST HOLE LOCATION PLAN

HORIZON
ENGINEERING INC

Scale: NTS	File No: 122-5174	Date: APR/2023		FIGURE: 2
Des: JT	Dwn: LF	Chk: KS	Rev:	

Photograph 1: Headscarp of North Slope (looking North from top of staircase)

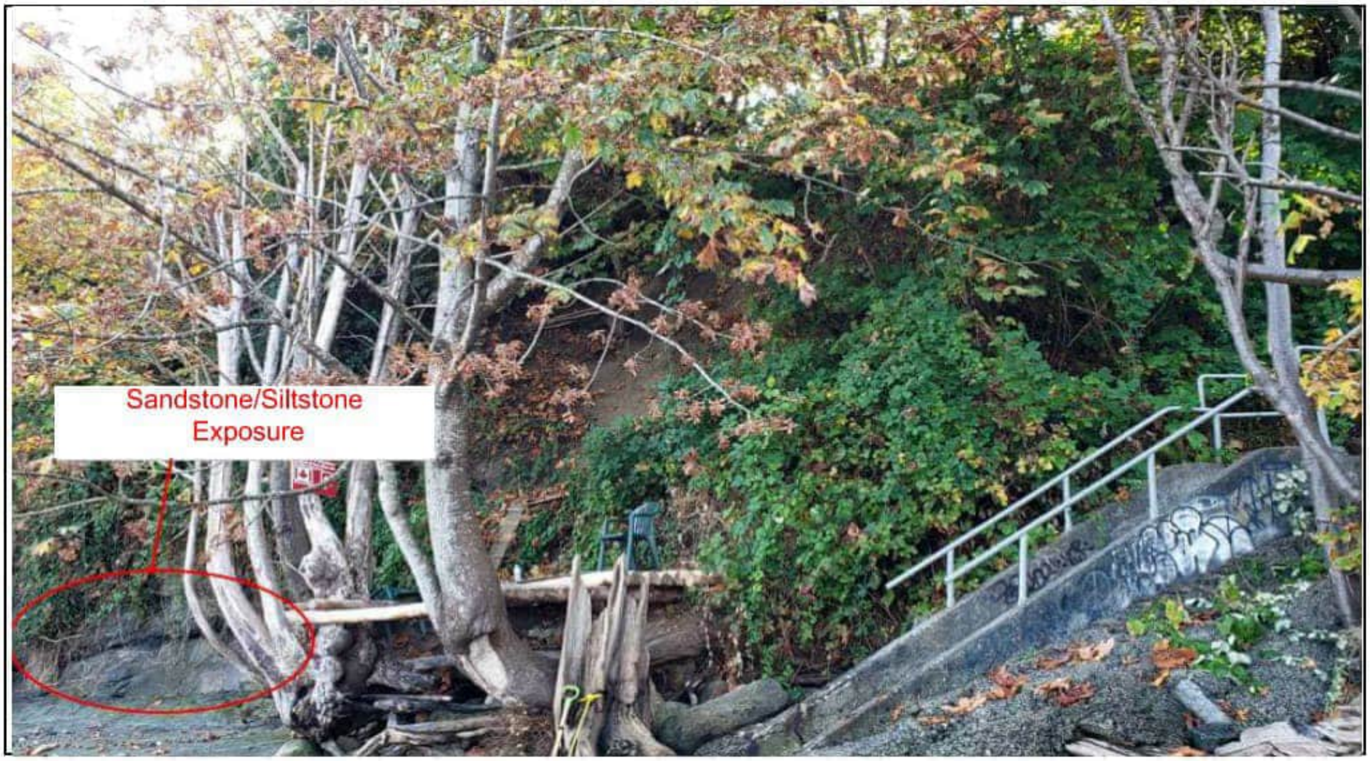
Photograph 2: Landslide scarp at North Slope (looking upslope from bottom of slope)

City of Vancouver	Figure 3: Photographs 1 and 2	HORIZON ENGINEERING INC
Dunbar Slope Assessment at Cameron Ave. and Dunbar St., Vancouver, BC		

Scale:	File No:	Date:	FIGURE:
NTS	122-5174	March/2023	3

Des:	Dwn:	Chk:	Rev:
JT	JT	KS	

Photograph 3: 900 millimetre thick deposit of colluvium/landslide debris at bottom of North Slope



Photograph 4: Existing Condition of South Slope (looking upslope from beach)

City of Vancouver

Dunbar Slope Assessment at
Cameron Ave. and Dunbar St.,
Vancouver, BC

Figure 3:
Photographs
3 and 4

HORIZON
ENGINEERING INC

Scale:	File No:	Date:	FIGURE:
NTS	122-5174	March/2023	4

Des:	Dwn:	Chk:	Rev:
JT	JT	KS	

Photograph 5: Inferred fill materials, mixed with debris overlying South Slope

Photograph 6: Existing Condition of South Slope (looking south and upslope from beach)

City of Vancouver

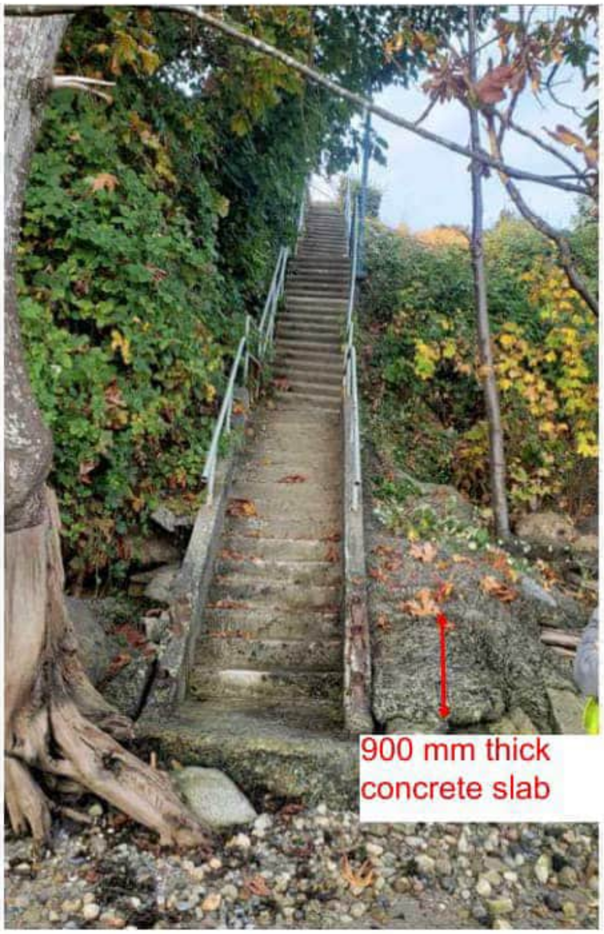
Dunbar Slope Assessment at
Cameron Ave. and Dunbar St.,
Vancouver, BC

Figure 4:
Photographs
5 and 6

HORIZON
ENGINEERING INC

Scale: NTS	File No: 122-5174	Date: March/2023		FIGURE: 5
Des: JT	Dwn: JT	Chk: KS	Rev:	

Photograph 7: Bedrock exposure at bottom of South Slope



Photograph 8: Existing condition of beach access staircase, looking up from bottom

City of Vancouver

Dunbar Slope Assessment at
Cameron Ave. and Dunbar St.,
Vancouver, BC

Figure 4:
Photographs
7 and 8

HORIZON ENGINEERING INC

Scale:	File No:	Date:	FIGURE:
NTS	122-5174	March/2023	6

Des:	Dwn:	Chk:	Rev:
JT	JT	KS	

Appendix B

SLOPE MONITORING OBSERVATIONS

Table 1: Previous Slope Monitoring Observations

Date of Site Visit	Correspondence Type	Author	Summary Observations and Recommendations by Others
December 9, 2021 (Initial site visit by Golder Associates, accompanied by COV representative)	Field Review Report	Golder Associates	The landslide scarp developed within the North Slope was documented to be approximately 15 metres in length and approximately 4.8 and 8.2 metres wide at the top and bottom, respectively. The slope failure was observed to have occurred within a relatively thin layer of vegetation and mineral soil mantling the sandstone slope. No signs of structural distress were observed at the existing concrete staircase. Access to the staircase from the upper landing had been cordoned off with survey tape and a street barricade at the time of this site visit. Temporary closure signage had also been installed. Indicator signs of slope movement, in the form of tension cracks, were observed at the crest of the North Slope. Slide debris, comprising soil and gravel, as well as uprooted vegetation, had been deposited at the bottom of the North slope from the failed portion of the slope. Recommendation provided to restrict public access to the staircase and to install signage to notify the public to maintain a setback distance from the above-mentioned deposit of slide debris in consideration of the hazards associated with tidal induced erosion/destabilization of this debris deposit. Recommendation provided to establish survey monitoring points on the staircase as well as on concrete curbs located within the vicinity of the slopes. Weekly visual monitoring of the site was also recommended as an alternative if survey monitoring was not possible.

Date of Site Visit	Correspondence Type	Author	Summary Observations and Recommendations by Others
December 22 and 28, 2021	Email Correspondence	Golder Associates	No discernible change in conditions of the slope and staircase was observed between the two site visits. Slope was judged to be stable at the time of the site visit. Barricade previously placed at the top of the staircase was now located within the failed portion of the slope located to the north of the staircase and the survey tape had been removed. Additional measures to deter public access to staircase not deemed to be necessary.
January 4, 2022	Email Correspondence	Golder Associates	No discernible change in conditions of the slope and staircase observed relative to the previous site visit. Two traffic delineators had been placed at the top of the staircase and the closure signage had remained in place Recommendations provided to reduce frequency of slope monitoring to monthly site visits and in response to public reporting or significant rainfall events. Public access to staircase recommended to remain restricted.
February 1, 2022	Email Correspondence	Golder Associates	Indication of slope ravelling observed adjacent to the staircase, near the toe of the North Slope Public access to staircase recommended to remain restricted.

Date of Site Visit	Correspondence Type	Author	Summary Observations and Recommendations by Others
March 9, 2022	Email Correspondence	Golder Associates	The barricades, delineators and survey tape which were previously placed to deter public access had been relocated to the side or removed. Additional sloughing of the north slope was apparent and had resulted in the deposition of the sloughed materials at the mid-slope portion of the slope. Slumping of slope materials was noted to have occurred at the toe of the north slope. Some vegetation and soil at the subject portion of the slope was also indicated to have washed away. Additional movement of soil was apparent at the South Slope. The soil was indicated to have sloughed into a void located at the base of the trees. The specific location of the sloughed portion of the slope was not indicated in the email correspondence.
April 29, 2022	Field Review Memo	COV	Hoarding panel was installed at the upper staircase landing. The hoarding was secured to the staircase railings with chains and locks in an effort to deter public access. Conditions of the North Slope indicated to be unchanged relative to the previous site visit conducted by Golder Associates on March 9, 2022.
July 12, 2022	Field Review Memo	COV	Conditions of the North Slope were observed to be unchanged from those documented in previous site visits carried out on June 9, 2022, by the COV, and on March 9, 2022 by Golder Associates, with the exception of an increase in vegetation.
August 17, 2022	Field Review Memo	COV	Conditions of the North Slope were noted to be unchanged from those documented in previous site visits carried out on July 12, 2022 by the COV, and on March 9, 2022 by Golder Associates, with the exception of further increase in vegetation.
September 23, 2022	Field Review Memo	COV	Conditions of the North Slope were noted to be unchanged from those documented in previous site visits carried out on August 17, 2022 by the COV, and on March 9, 2022 by Golder Associates, with the exception of further increase in vegetation.

Date of Site Visit	Correspondence Type	Author	Summary Observations and Recommendations by Others
December 31, 2022, January 27 and March 6, 2023	Field Review Memo	Horizon Engineering	Baseline measurements for monitoring points, established near the crest of the North Slope and at the top of existing retaining wall located at the crest of the South Slope, were taken on December 31, 2022. Conditions of the North and South Slopes were generally observed to be unchanged relative to those documented from previous site visits completed by Golder Associates on March 9, 2022 and by the COV on September 23, 2022.

Appendix C

TEST HOLE LOGS

Test Hole Log No. SH23-01

LOGGED BY: **JT**
REVIEWER: **KS**

DATE: **January 27, 2023**
METHOD: **Track-mounted Sonic Drill Rig**

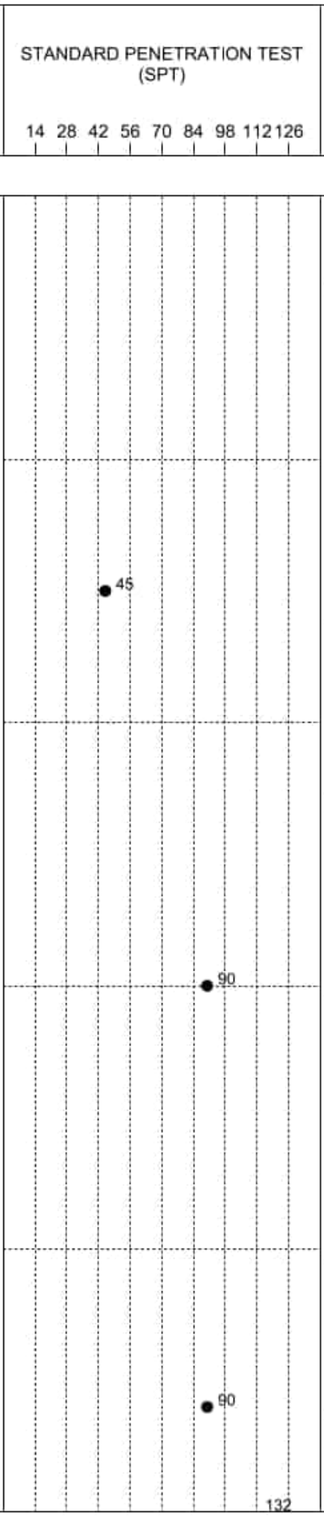
ADDRESS: **North and East Termini of Dunbar Street and Cameron Avenue**

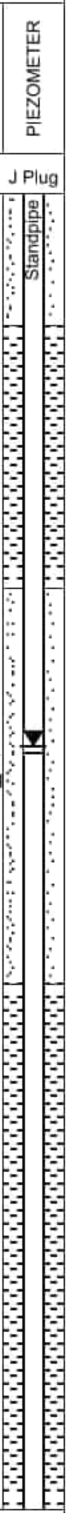
COORDINATES: **UTM**
LAT / EASTING (X): **0**
LON / NORTHING (Y): **0**
ELEVATION: **12.1 m**

TYPE OF SAMPLE:
- SPT - Split spoon
- S - Shelby tube
- G - Grab
- O - Other (Specify)

NOTES:
Test Hole advanced at east terminus of Cameron Avenue

DEPTH M	DESCRIPTION		SPT	SAMPLE NO.	COMMENTS/ADDITIONAL NOTES
	Ground Surface	EL 12.1 m			
0	**ASPHALT** 300 millimetre thick layer of asphalt	0.3 m EL 11.8 m			0-0.45 m: Filter Sand Backfill
	NO RECOVERY no sample recovery	0.6 m EL 11.5 m			0.45-1.5 m: Bentonite Seal
	FILL-SILTY SAND (brown) fine grained, moist - inferred to be compact	0.9 m EL 11.2 m		G1	
1	**SAND (reddish brown)** fine to medium grained, trace gravel, moist - inferred to be compact	1.2 m EL 10.9 m	45	G2	
	SAND (light brown) well graded sand, moist - inferred to be compact to dense	2 m EL 10.1 m			1.5-3.3 m: Sand Backfill
2	**SILT AND SAND (light brown)** fine grained sand, moist - slightly lithified - inferred to be hard/very dense	3.2 m EL 8.9 m	90	G3 G4	Inferred perched groundwater level measured at depth of 2.1 metres on March 9, 2023 SPT effective refusal: 90 blows/150 mm
3	**SANDY SILT (grey)** fine grained sand, moist - slightly cemented - inferred to be hard *(continued)*		90	G5 G6 G7	3-5.2 m: Bentonite Seal SPT effective refusal 90 blows/50 mm





PROJECT: **VAN COV DUNBAR SLOPE**
PROJECT NO.: **122-5174**
CLIENT: **City of Vancouver**
SHEET: **1 of 6**

HORIZON ENGINEERING INC

Test Hole Log No. SH23-01

LOGGED BY: **JT**
REVIEWER: **KS**
DATE: **January 27, 2023**
METHOD: **Track-mounted Sonic Drill Rig**
ADDRESS: **North and East Termini of Dunbar Street and Cameron Avenue**

COORDINATES: **UTM**
LAT / EASTING (X): **0**
LON / NORTHING (Y): **0**
ELEVATION: **12.1 m**

TYPE OF SAMPLE:
SPT - Split spoon
S - Shelby tube
G - Grab
O - Other (Specify)

NOTES:
Test Hole advanced at east terminus of Cameron Avenue

DEPTH FT	DEPTH M	DESCRIPTION	SPT	SAMPLE NO.	STANDARD PENETRATION TEST (SPT) 14 28 42 56 70 84 98 112 126	COMMENTS/ADDITIONAL NOTES
17	5	*(continued)* **SANDY SILT (grey)** fine grained sand, moist - slightly cemented - inferred to be hard 5.3 m EL 6.8 m		G8 G9		5.2-8.8 metres: Filter Sand Backfill
19–20	6	**SILT(dark brown)** trace to some fine grained sand, moist - possible inclusions of quartz and feldspar - inferred to be very stiff - inferred to be possible decomposed, organic-rich stratum 5.8 m EL 6.3 m		G10		Ambient/static groundwater level measured at depth of 6.2 metres on March 9/2023
21		27 Jan 2023 ▽				
22		**SILTSTONE (grey)** some fine grained sand, moist - inferred to be very weak to weak		G11		
23–24	7			G12		
25			132		132 ●	SPT effective refusal: 132 blows/225 mm
26–28	8					
29–32	9					8.8-12.2 m: Bentonite Seal
	10				100	

PROJECT: **VAN COV DUNBAR SLOPE**
PROJECT NO.: **122-5174**
CLIENT: **City of Vancouver**
SHEET: **2 of 6**

HORIZON ENGINEERING INC

Test Hole Log No. SH23-01

LOGGED BY: **JT**
REVIEWER: **KS**

DATE: **January 27, 2023**
METHOD: **Track-mounted Sonic Drill Rig**

ADDRESS: **North and East Termini of Dunbar Street and Cameron Avenue**

COORDINATES: **UTM**
LAT / EASTING (X): **0**
LON / NORTHING (Y): **0**
ELEVATION: **12.1 m**

TYPE OF SAMPLE:
SPT - Split spoon
S - Shelby tube
G - Grab
O - Other (Specify)

NOTES:
Test Hole advanced at east terminus of Cameron Avenue

DEPTH FT	DEPTH M	DESCRIPTION	SYMBOL	SPT	SAMPLE NO.	STANDARD PENETRATION TEST (SPT) 14 28 42 56 70 84 98 112 126	COMMENTS/ADDITIONAL NOTES	PIEZOMETER
33	10	*(continued)* **SILTSTONE (grey)** some fine grained sand, moist - inferred to be very weak to weak			G13			
35		10.7 m EL 1.4 m		100		● 100	SPT effective refusal: 100 blows/150mm	
35		**SILTSTONE (grey)** silt and sand, moist - inferred to be very weak			G14			
40		12.2 m EL -0.1 m						
40		End of Test Hole at 12.1 (m)						

DEPTH scale: FT 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49; M 10, 11, 12, 13, 14, 15

HORIZON ENGINEERING INC

PROJECT: **VAN COV DUNBAR SLOPE**

PROJECT NO.: **122-5174**

CLIENT: **City of Vancouver**

SHEET: **3 of 6**

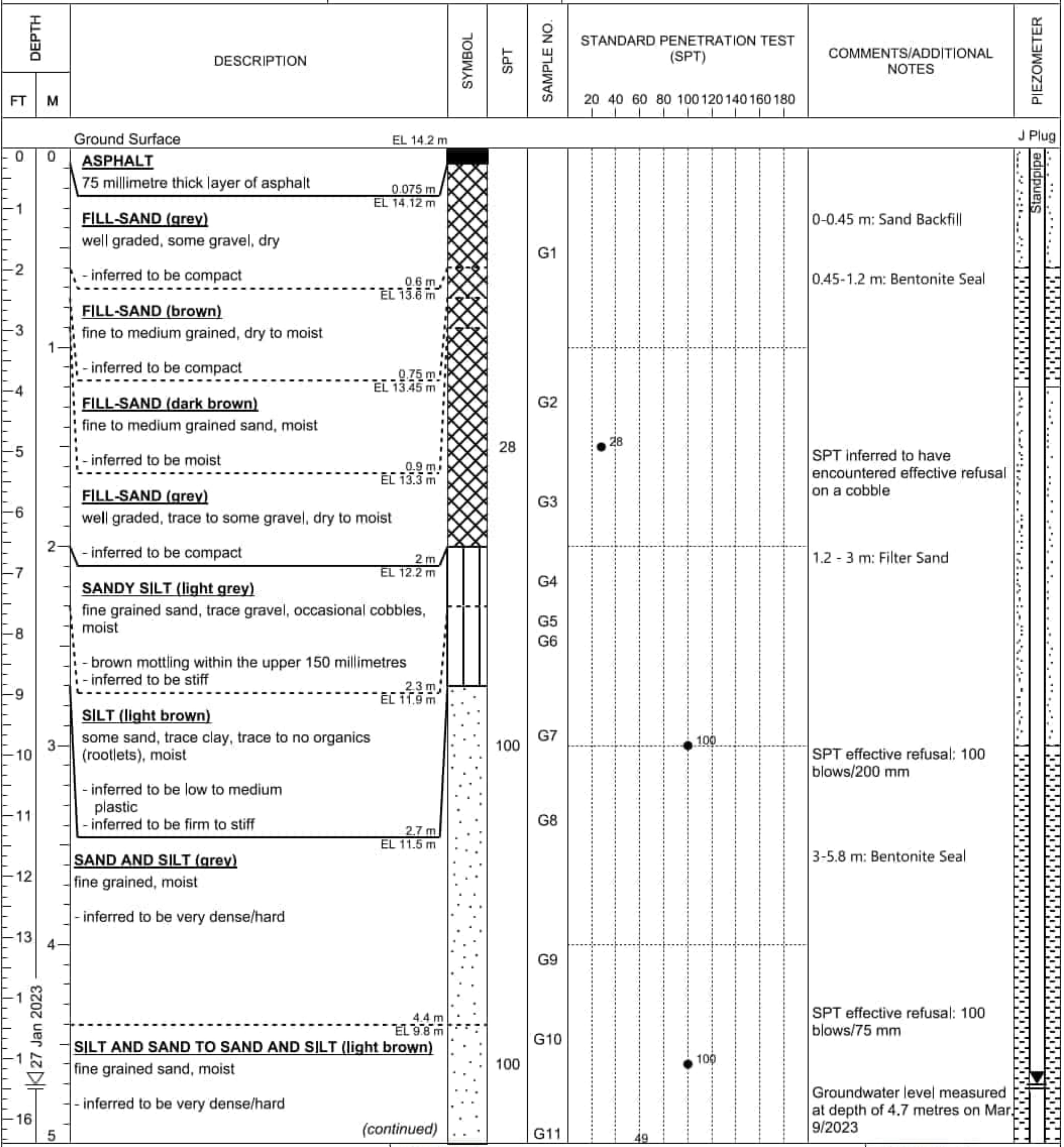
Test Hole Log No. SH23-02

LOGGED BY: **JT**
REVIEWER: **KS**
DATE: **January 27, 2023**
METHOD: **Track-mounted Sonic Drill Rig**
ADDRESS: **North and East Termini of Dunbar Street and Cameron Avenue**

COORDINATES: **UTM**
LAT / EASTING (X): **0**
LON / NORTHING (Y): **0**
ELEVATION: **14.2 m**

TYPE OF SAMPLE:
SPT - Split spoon
S - Shelby tube
G - Grab
O - Other (Specify)

NOTES:
North terminus of Dunbar Street



Ground Surface EL 14.2 m

ASPHALT
75 millimetre thick layer of asphalt
0.075 m EL 14.12 m

FILL-SAND (grey)
well graded, some gravel, dry
- inferred to be compact
0.6 m EL 13.6 m

FILL-SAND (brown)
fine to medium grained, dry to moist
- inferred to be compact
0.75 m EL 13.45 m

FILL-SAND (dark brown)
fine to medium grained sand, moist
- inferred to be moist
0.9 m EL 13.3 m

FILL-SAND (grey)
well graded, trace to some gravel, dry to moist
- inferred to be compact
2 m EL 12.2 m

SANDY SILT (light grey)
fine grained sand, trace gravel, occasional cobbles, moist
- brown mottling within the upper 150 millimetres
- inferred to be stiff
2.3 m EL 11.9 m

SILT (light brown)
some sand, trace clay, trace to no organics (rootlets), moist
- inferred to be low to medium plastic
- inferred to be firm to stiff
2.7 m EL 11.5 m

SAND AND SILT (grey)
fine grained, moist
- inferred to be very dense/hard
4.4 m EL 9.8 m

SILT AND SAND TO SAND AND SILT (light brown)
fine grained sand, moist
- inferred to be very dense/hard

(continued)

SPT	SAMPLE NO.
	G1
	G2
28	
	G3
	G4
	G5
	G6
	G7
100	
	G8
	G9
	G10
100	
	G11

COMMENTS/ADDITIONAL NOTES:

0-0.45 m: Sand Backfill

0.45-1.2 m: Bentonite Seal

SPT inferred to have encountered effective refusal on a cobble

1.2 - 3 m: Filter Sand

SPT effective refusal: 100 blows/200 mm

3-5.8 m: Bentonite Seal

SPT effective refusal: 100 blows/75 mm

Groundwater level measured at depth of 4.7 metres on Mar. 9/2023

RSLog / ** SPT Drill Hole Log 11m / horizon / Engineer / April 11, 2023 03:45 PM

HORIZON
ENGINEERING INC

PROJECT: **VAN COV DUNBAR SLOPE**
PROJECT NO.: **122-5174**
CLIENT: **City of Vancouver**
SHEET: **1 of 8**

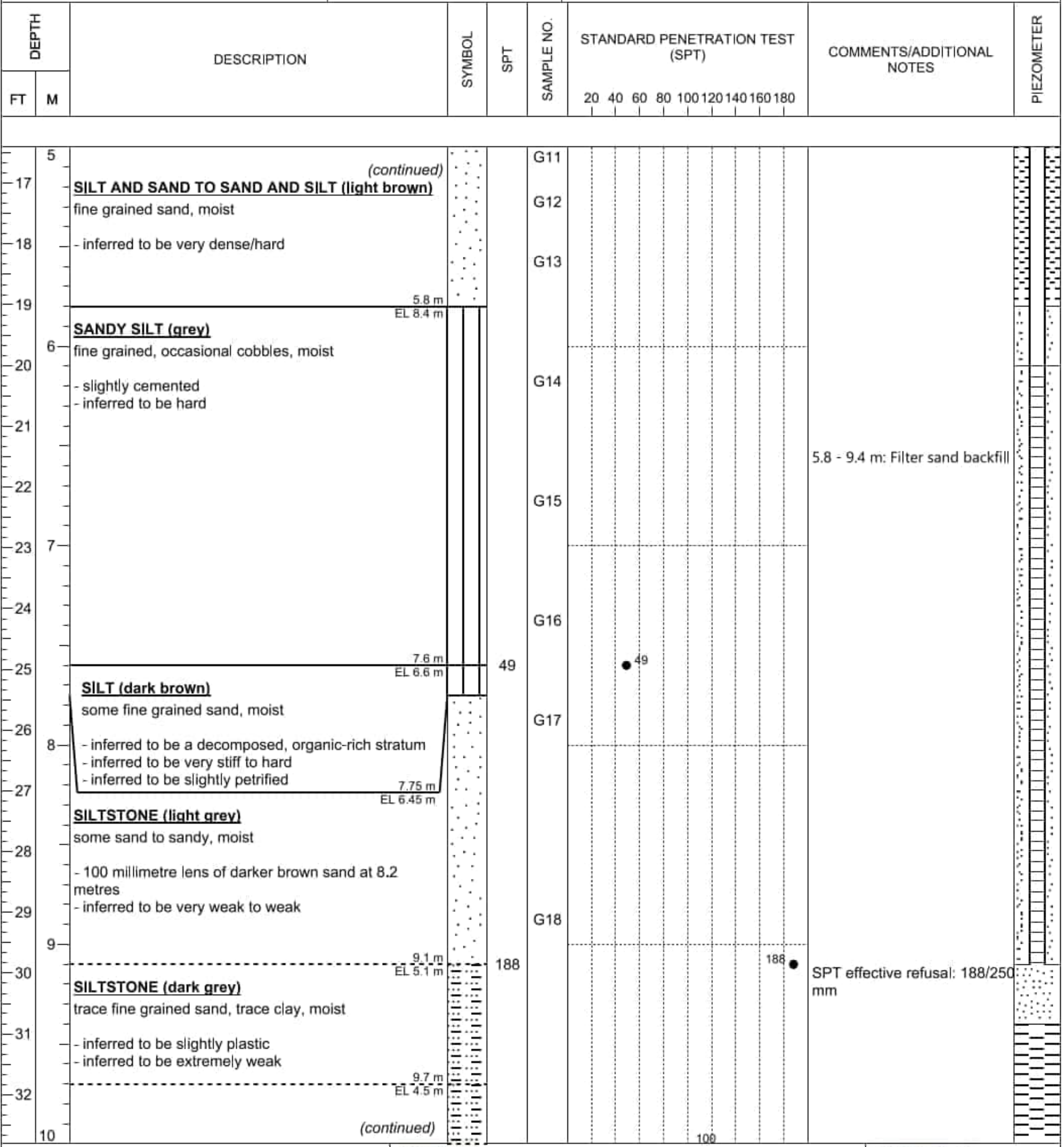
Test Hole Log No. SH23-02

LOGGED BY: **JT**
REVIEWER: **KS**
DATE: **January 27, 2023**
METHOD: **Track-mounted Sonic Drill Rig**
ADDRESS: **North and East Termini of Dunbar Street and Cameron Avenue**

COORDINATES: **UTM**
LAT / EASTING (X): **0**
LON / NORTHING (Y): **0**
ELEVATION: **14.2 m**

TYPE OF SAMPLE:
SPT - Split spoon
S - Shelby tube
G - Grab
O - Other (Specify)

NOTES:
North terminus of Dunbar Street



(continued)

SILT AND SAND TO SAND AND SILT (light brown)
fine grained sand, moist
- inferred to be very dense/hard

5.8 m
EL 8.4 m

SANDY SILT (grey)
fine grained, occasional cobbles, moist
- slightly cemented
- inferred to be hard

7.6 m
EL 6.6 m

SILT (dark brown)
some fine grained sand, moist
- inferred to be a decomposed, organic-rich stratum
- inferred to be very stiff to hard
- inferred to be slightly petrified

7.75 m
EL 6.45 m

SILTSTONE (light grey)
some sand to sandy, moist
- 100 millimetre lens of darker brown sand at 8.2 metres
- inferred to be very weak to weak

9.1 m
EL 5.1 m

SILTSTONE (dark grey)
trace fine grained sand, trace clay, moist
- inferred to be slightly plastic
- inferred to be extremely weak

9.7 m
EL 4.5 m

(continued)

5.8 - 9.4 m: Filter sand backfill

SPT effective refusal: 188/250 mm

HORIZON
ENGINEERING INC

PROJECT: **VAN COV DUNBAR SLOPE**
PROJECT NO.: **122-5174**
CLIENT: **City of Vancouver**
SHEET: **2 of 8**

RSLog / ** SPT Drill Hole Log 11m / horizon / Engineer / April 11, 2023 03:45 PM

Test Hole Log No. SH23-02

LOGGED BY: **JT**
REVIEWER: **KS**
DATE: **January 27, 2023**
METHOD: **Track-mounted Sonic Drill Rig**
ADDRESS: **North and East Termini of Dunbar Street and Cameron Avenue**

COORDINATES: **UTM**
LAT / EASTING (X): **0**
LON / NORTHING (Y): **0**
ELEVATION: **14.2 m**

TYPE OF SAMPLE:
SPT - Split spoon
S - Shelby tube
G - Grab
O - Other (Specify)

NOTES:
North terminus of Dunbar Street

DEPTH (M)	DESCRIPTION	SPT	SAMPLE NO.	STANDARD PENETRATION TEST (SPT)	COMMENTS/ADDITIONAL NOTES
10	*(continued)* **SILTSTONE (light grey)** sandy, moist - inferred to be weak		G20 G21		9.4 - 12.1 m: Bentonite Seal
10.7 m EL 3.5 m	**SILTSTONE (grey)** trace fine grained sand, trace clay, moist - inferred to be slightly plastic - inferred to be extremely weak		G22		
11.3 m EL 2.9 m	**SILTSTONE (grey)** sandy, moist - slightly cemented - inferred to be weak		G23		
12.1 m EL 2.1 m	End of Test Hole at 12.1 (m)	100	G24	● 100	SPT effective refusal: 100 blows/75 mm

PROJECT: **VAN COV DUNBAR SLOPE**
PROJECT NO.: **122-5174**
CLIENT: **City of Vancouver**
SHEET: **3 of 8**

HORIZON ENGINEERING INC

HORIZON ENGINEERING INC

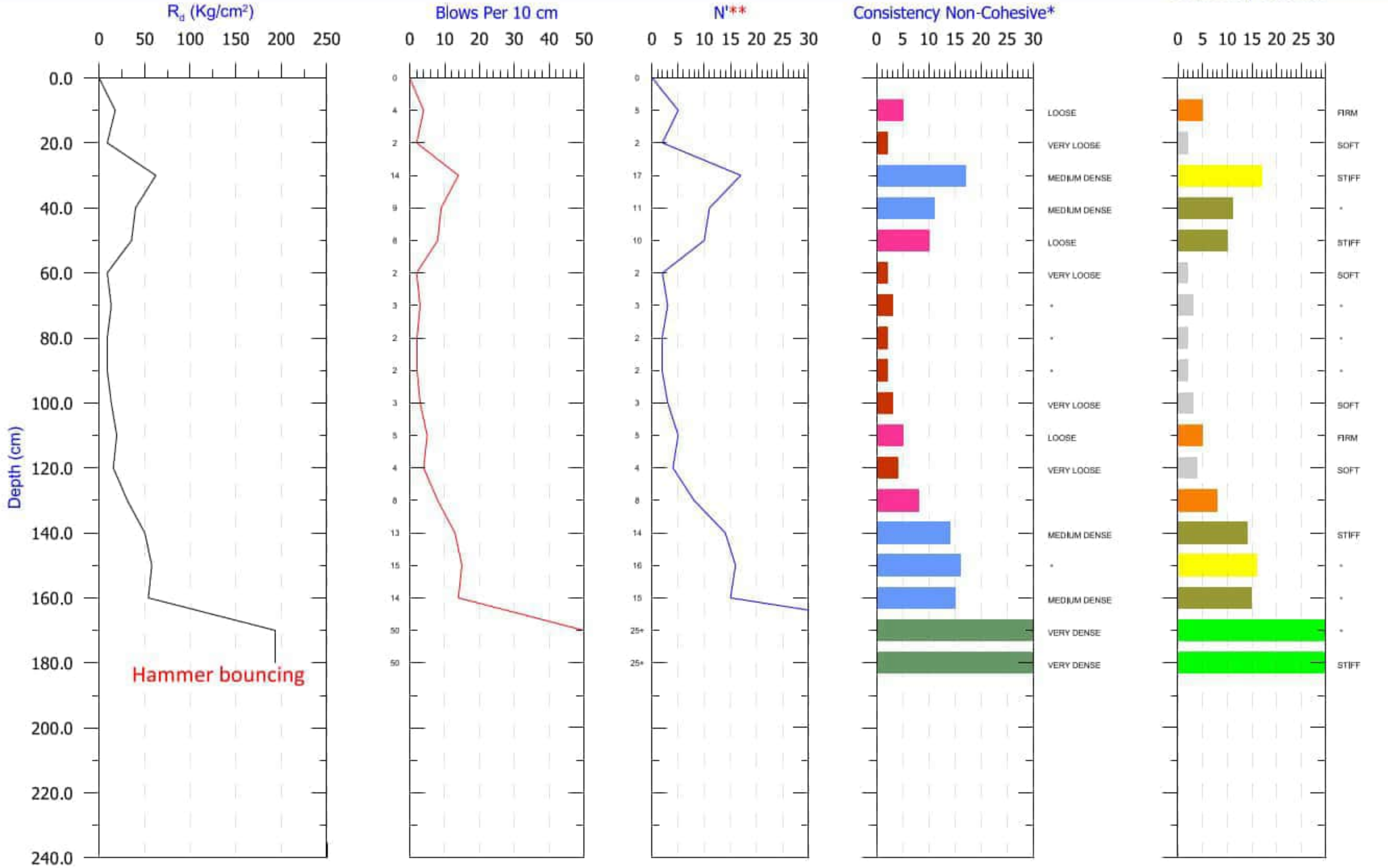
Wildcat Cone Penetration

Job No: 122-5174
Date: Feb. 1-23
Site: Dunbar Slope
Vancouver, BC

Operator: CC/DS
Test Hole: WC23-1
Max Depth (cm): 180
Predrill (cm): 0

Comments: Edge of Slope - Dunbar
0.8 m W of East Neighbor PL Fence

Data & Results



* Inferred Soil Description as per Manufacturer's Manual

** For dynamic cone resistances (R_d) above 106.7 kg/cm² the correlating SPT N' value is indicated as 30+

HORIZON
ENGINEERING INC

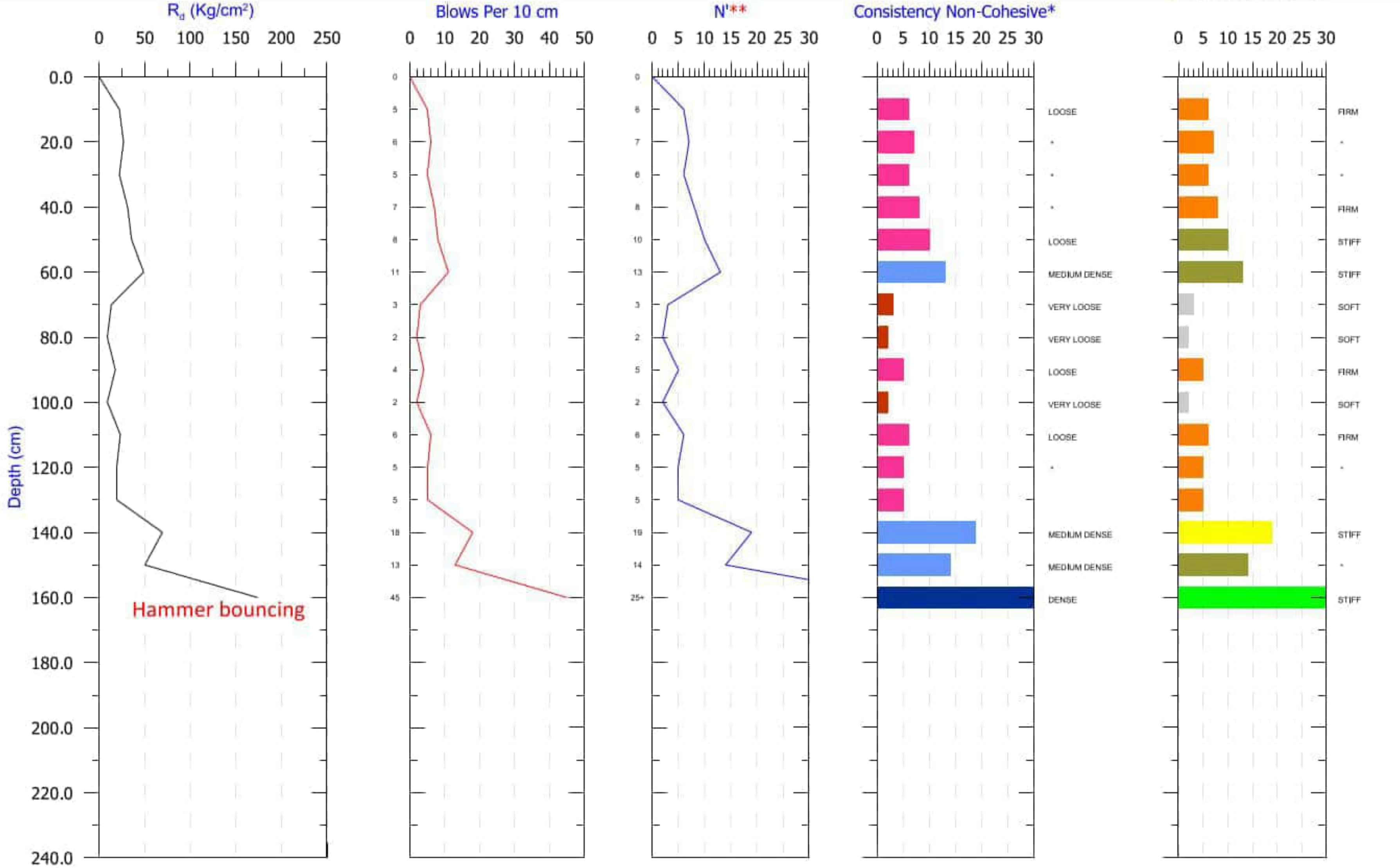
Wildcat Cone Penetration

Job No: 122-5174
Date: Feb. 1-23
Site: Dunbar Slope
Vancouver, BC

Operator: CC/DS
Test Hole: WC23-2
Max Depth (cm): 160
Predrill (cm): 0

Comments: Edge of Slope - Dunbar
8.8 m W of WC-1

Data & Results



* Inferred Soil Description as per Manufacturer's Manual
** For dynamic cone resistances (R_d) above 106.7 kg/cm² the correlating SPT N' value is indicated as 30+

HORIZON
ENGINEERING INC

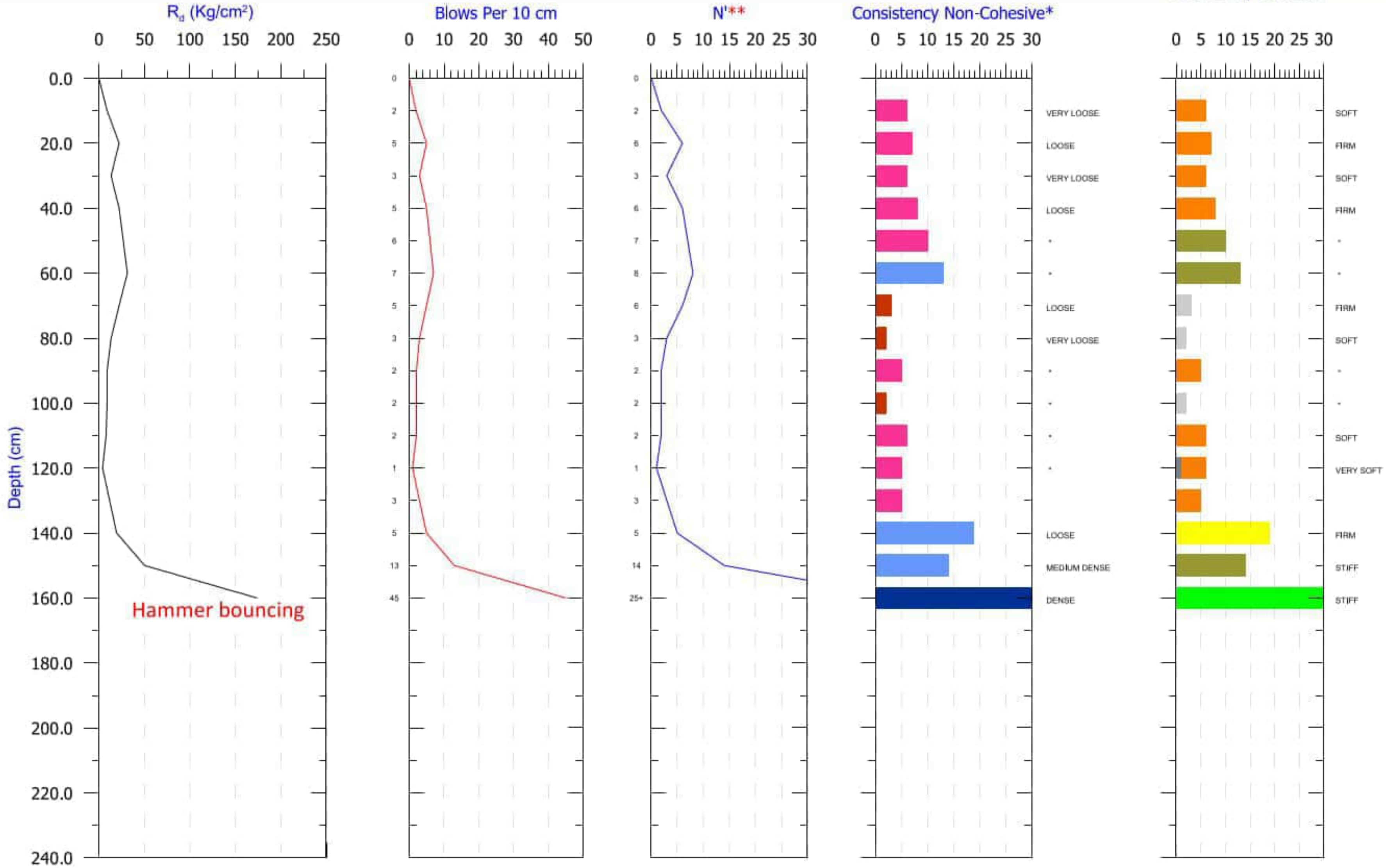
Wildcat Cone Penetration

Job No: 122-5174
Date: Feb. 1-23
Site: Dunbar Slope
Vancouver, BC

Operator: CC/DS
Test Hole: WC23-3
Max Depth (cm): 160
Predrill (cm): 0

Comments: Edge of Slope - Dunbar
4.7 m W of WC-2

Data & Results



* Inferred Soil Description as per Manufacturer's Manual
** For dynamic cone resistances (R_d) above 106.7 kg/cm² the correlating SPT N' value is indicated as 30+

HORIZON
ENGINEERING INC

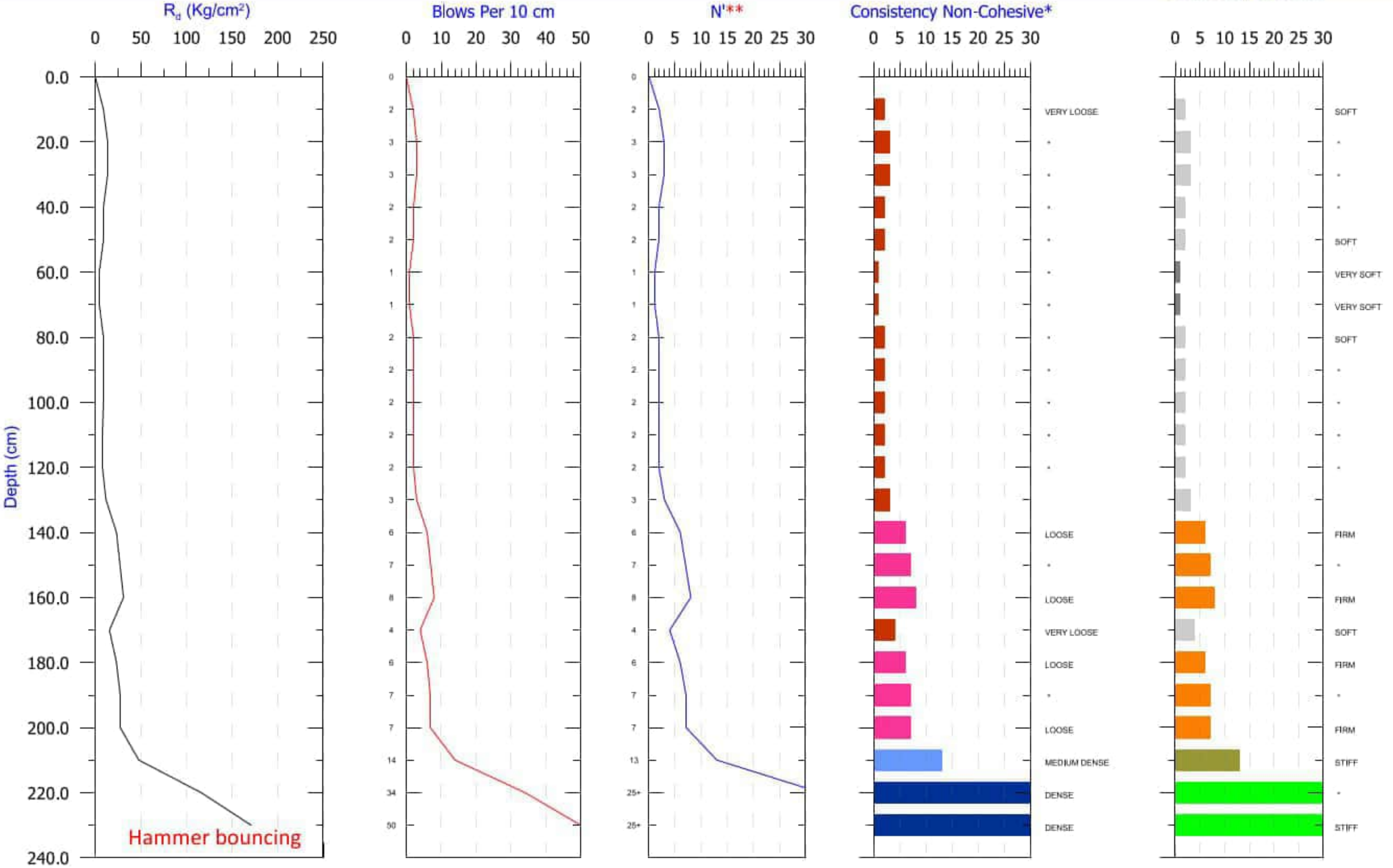
Wildcat Cone Penetration

Job No: 122-5174
Date: Feb. 1-23
Site: Dunbar Slope
Vancouver, BC

Operator: CC/DS
Test Hole: WC23-4
Max Depth (cm): 230
Predrill (cm): 0

Comments: Edge of Slope - Cameron
Middle of Landing

Data & Results



* Inferred Soil Description as per Manufacturer's Manual
** For dynamic cone resistances (R_d) above 106.7 kg/cm² the correlating SPT N' value is indicated as 30+

Horizon Engineering Inc.
Unit 220 - 18 Gostick Pl.
North Vancouver, BC V3M 3G3

HORIZON
ENGINEERING INC

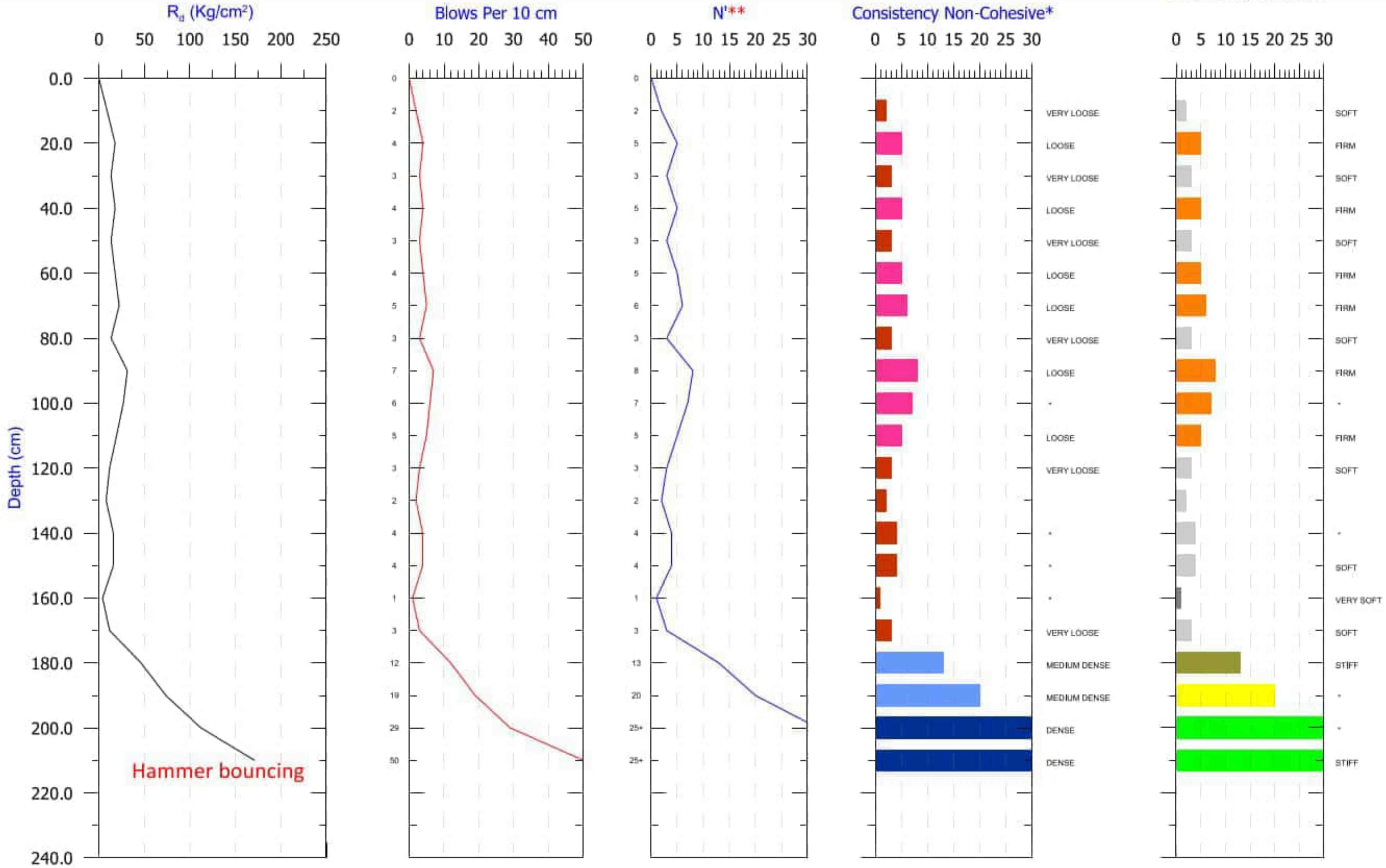
Wildcat Cone Penetration

Job No: 122-5174
Date: Feb. 1-23
Site: Dunbar Slope
Vancouver, BC

Operator: CC/DS
Test Hole: WC23-5
Max Depth (cm): 210
Predrill (cm): 0

Comments: Edge of Slope - Cameron
3.7 m W of WC-4

Data & Results



* Inferred Soil Description as per Manufacturer's Manual
** For dynamic cone resistances (R_d) above 106.7 kg/cm² the correlating SPT N' value is indicated as 30+

GEOTECHNICAL CONCEPTUAL DESIGN REPORT
DUNBAR SLOPE ASSESSMENT
DUNBAR AND CAMERON AVENUES, VANCOUVER, BC

CLIENT: CITY OF VANCOUVER

PREPARED BY:
RAM CONSULTING

220 – 18 GOSTICK PLACE
NORTH VANCOUVER, BC
V7M 3G3
604.990.0546

JANUARY 27, 2025
OUR FILE: 122-5174

Executive Summary

Following a following a period of heavy precipitation in November, 2021, a slope failure, in the form of a localized, shallow landslide, was triggered within an approximately 9 metre high portion of the coastal slope that overlooks Burrard Inlet and extends between Kitsilano Beach and Jericho Beach in the Point Grey area of Vancouver, BC. The subject portion of the slope is located below the east terminus of Cameron Avenue and to the north of an existing concrete staircase that provides access from Cameron Avenue and Dunbar Street to the beach front located below the slope. The foundations of this staircase are seated atop the portion of the slope located adjacent to the landslide area.

Following the November, 2021 failure, Golder Associates was retained by the City of Vancouver to undertake a preliminary assessment of the subject portion of the slope and the adjacent section of the slope located to the south of the staircase and below the north terminus of Dunbar Street. The findings from this preliminary assessment and slope remediation recommendations were summarized in a report, dated April 7, 2022, prepared by Golder Associates. Monthly slope monitoring was also undertaken by Golder during the period of December 2021 to March, 2022 and by the COV during March to September, 2022. Additional signs of slope movement, in the form of soil raveling and sloughing were observed at the portion of the slope located to the north of the staircase, referred to herein as the "North Slope", during site visits completed on February 1 and March 9 2022 and at the portion of the slope located to the south of the staircase, referred to as the "South Slope" during the March 9, 2022 site visit. No other changes in the conditions of the slope, relative to the March 2022 site visit were observed by the CoV during the monitoring period of March to September, 2022. It should be noted that public access to the staircase and the portion of beach located below the aforementioned slopes has been restricted since early December 2021, due to concerns of potential instability of the slopes and staircase.

Ram Geotechnical Engineering Ltd., referred to as Ram GE herein, was subsequently retained to undertake a detailed assessment of the slopes and continue the monitoring program. The objectives of this assessment were to provide potential slope and staircase remediation options to the CoV in order to inform future risk management decisions related to the assessment area. This assessment encompassed a subsurface investigation and the preparation of Factual and Conceptual Design reports summarizing the findings from the investigation and recommendations for slope remediation and provision of a "Class D" cost estimate for the design and construction of each proposed remediation option. An assessment of the trees and vegetation overlying the slopes was also undertaken by B.A. Blackwell & Associates, of North Vancouver, BC, and the findings and recommendations from their assessment are summarized in a separate report, dated May 21, 2024.

As part of this assessment, Ram GE has undertaken monitoring of the slopes and staircase since December 2022, with the exception of a pause in monitoring for the period of May 2023 to February 2024. No discernable changes in the conditions of the North Slope and beach access staircase, relative to those observed during our initial site visit completed in December, 2022, have been observed throughout the monitoring periods. However, an approximately 10 metre long section of the sandstone exposure, which forms the lower portion of the South Slope, was observed from site visits on May 10, 2023 and March 22, 2024 to have been subjected to spalling-type failure. The spalled portion of the slope was measured to be approximately 2.1 metres high.

The subsurface investigation program for this assessment entailed advancing two, 12.1 metre deep sonic boreholes at the portions of the roadways situated at the north and east termini of Dunbar Street and Cameron Avenue and a series of shallower, WildCat Penetrometer Test Soundings at the landscaping areas situated near the crests of the slopes. The boreholes and soundings were advanced in January and February 2023. In general, at the boreholes, the surficial fill materials were underlain by natural materials, which generally comprised layers of compact sand in turn underlain by hard/very dense, slightly lithified silt and sand, which graded into a sandy silt with increasing depth and a discrete, 200 to 500 millimetre thick layer of very stiff to hard silt that was inferred to comprise decomposed and partially petrified organic matter. Bedrock, comprising lightly cemented siltstone, inferred to be very weak to weak, was encountered below the aforementioned overburden materials and at depths of 5.8 metres and 7.7 metres at the boreholes advanced at Dunbar Street and Cameron Avenue, respectively. The boreholes were terminated within this bedrock stratum. Very loose to stiff soil conditions were encountered from the existing ground surface at the WildCat Penetrometer soundings, which encountered refusal at depths ranging from 1.6 to 1.8 metres at the soundings advanced near the crest of the North Slope and 1.7 to 2.3 metres at the soundings advanced near the crest of the South Slope.

Based on our observations of the conditions of the slopes from the slope monitoring site visits, our findings from the subsurface investigation program and slope stability analyses, it was identified that the slopes are generally susceptible to localized, shallow failures which could include sloughing/unravelling of loose, surficial materials. Spalling of the lower portion of the South Slope, where siltstone is currently exposed and steepened to overhanging slope geometries are present, was also identified to be a potential failure mode based on our observations from the slope monitoring site visits. The slopes were also assessed to be potentially susceptible to retrogressive slope failures which could result from the gradual, tidal-induced erosion of the lower portions of the slopes. These retrogressive failures could potentially impact the areas and existing infrastructure located above the slopes. Furthermore, the South Slope was found to be undermined at two locations due to the presence of cavities at the toe of the subject slope; the expansion of these cavities could result from tidal erosion and could destabilize this slope. However, based on the current, observed conditions of the slopes and the results of our analyses, the slopes were not identified to be susceptible to global, deep-seated failures under static loading conditions due to the relatively high shear strengths of the subsurface materials that are inferred to underlie the surficial, loose deposits of the slopes. The Factor of Safety (F.S.) criteria prescribed in Table 6.2b of the BC Ministry of Transportation and Infrastructure (BC MOTI) document titled "Bridge Standards and Procedures Manual – Volume 1 Supplement to Canadian Highway Bridge Design Code (CHBDC) S6:19, was considered for assessing the adequacy of the F.S. values that were calculated for the global stability of the slopes. The range of F.S. values calculated for the global stability of the slopes was generally found to be consistent with those prescribed in the aforementioned document for a "typical" degree of understanding and a consequence factor of "low", as determined by Ram GE to be suitable.

In its current condition, the staircase was inferred to be susceptible to ongoing settlement as its foundations were found to be supported on loose/soft subgrade materials. This settlement could adversely impact the performance of the staircase and exacerbate an already present potential tripping hazard to its users. Further potential slope failures/movements could also result in the destabilization of the staircase.

In consideration of the above slope failure mechanisms, the remediation options presented in this conceptual report include the following options for each portion of the slope:

North Slope Remediation Options

- Installation of rip-rap armouring for erosion protection, partial re-grading of the upper portion of the slope to a more gentle inclination and installation of subdrains and seepage collars near the crest of the slope to intercept and divert surface water away from the slopes;
- Construction of a Mechanically Stabilized Earth (MSE) wall buttress to stabilize the lower portion of the North Slope and partial re-grading of the slope, as described above; or
- Installation of a soil nail and mesh system to retain the slope.

South Slope Remediation Options

- Installation of rip-rap armouring for erosion protection; or
- Installation of a soil nail and mesh system to retain the slope.

As an alternative to the above, the slopes could be actively maintained in their current conditions and public education (e.g. installation of signage) and a slope monitoring program could be undertaken to manage risks. The slope monitoring would generally entail documenting changes in the conditions of the slopes such that hazard and risk mitigation measures could be implemented, if deemed to be required due to further deterioration of the slopes. As an additional risk management measure, signage could be installed to inform the public of the hazards associated with the slopes and to deter/discourage access to the beach area located below the slopes, especially during or following unfavourable weather or tide conditions, as subject to the CoV's risk management policies. This option, as well as the proposed remediation options provided above, would also entail filling the existing cavities located at the toe of the South Slope with concrete to reduce the rate of erosion of this slope. Scaling of the loose, surficial slope materials overlying the South Slope could also be implemented for the remediation options of this slope to further reduce the risk of ravelled or sloughed materials impacting members of the public.

The staircase remediation options presented herein include the following:

- Actively maintaining the staircase and re-levelling it via slab-jacking, installing FLEX-MSE earth retention systems below its existing pad foundations to provide lateral confinement of the subgrade supporting these foundations, and undertaking monitoring of the staircase;
- Underpinning the existing staircase foundations using a system of fully-grouted, vertical threadbar anchors;
- Demolition of the existing staircase and construction of a new staircase; or
- Partial or full demolition of the existing staircase in order to inhibit beach access from this location.

The slope and staircase remediation options, summarized above, are discussed in greater detail in Sections 8.0 and 9.0 of this report.

In order to further inform the CoV's risk management decisions regarding the subject site, the hazards, potential consequences and risks of allowing public access to the beach front and staircase for select, slope remediation options as well as the current, existing conditions (i.e. a “do nothing approach”), were also assessed and are summarized in Section 12.0 of this report and Tables 11 and 12 attached in Appendix E. The risks associated with the slopes were assessed based on the following risk algorithm:

$$\text{Risk} = P_{\text{Hazard}} \times P_{S:H} \times P_{T:S} \times V \times E$$

Where:

P_{hazard}= probability of hazard occurrence,
$P_{S:H}$=spatial probability of impact to public
$P_{T:S}$=temporal probability of impact to public
V=occupant vulnerability (level of consequence)
E=number of occupants (or elements) at risk (taken to be 1 for this assessment

The probability of hazard occurrence would be indicative of the likelihood that the hazard would occur whereas the spatial probability of impact represents the likelihood that a pedestrian would be situated within the impact zone of a slope failure. The temporal probability of impact would represent the probability that a member of the public would be situated within the area of impact at the time of the failure.

With regards to the level of consequence above, a low level of consequence would indicate that the hazard is unlikely to result in an injury to those impacted, a moderate consequence level would indicate that the hazard could potentially result in injury and a high consequence level would indicate that there is a potential for fatality.

In general, the Slope Hazards related to the slopes, based on current conditions, were identified to include:

- Raveling of surficial, loose materials overlying both slopes;
- Shallow landslide failures triggered within the surficial loose materials overlying the North Slope following extreme weather events;

- Rockfall/spalling hazards at the South Slope, posed by the existing overhanging and steepened geometries at the lower portion of the South Slope;

It was also identified that antecedent events, such as intense/prolonged rainfall, tidal erosion of the slopes, seismic loading conditions and/or removal of vegetation due to fires, would also exacerbate the above-noted hazards and increase the risk levels of these hazards.

In general, the current risk levels associated with raveling failures were assessed to be “low” to “moderate”. whereas the risks associated shallow landslide and rockfall/spalling hazards were assessed to be “high” and “moderate to high” respectively. These risk levels were assessed using the above algorithm and based on the potential consequences of the impact of these hazards, the likelihood of these hazards occurring and the probability that these hazards would impact the public. The risk levels of these hazards could be lowered with the implementation of the slope monitoring and remediation options summarized above.

The existing staircase hazards, based on its current conditions, are envisaged to include:

- Potential tripping hazard to users due to settlement-induced distortion of the staircase;
- Additional movement/failures of the North Slope which could result in the staircase foundations being undermined if the subgrade materials supporting these foundations were to be mobilized.

The tripping hazard of the staircase was assessed to have a risk level of “moderate” with respect to public safety, based on the current condition of the staircase. However, it is envisaged that the risk level would increase if the staircase was to be subjected to further structural distortion.

The tripping hazard and risk to public hazard could be reduced if the staircase was to be re-levelled. The risk of staircase foundation undermining could be reduced if the FLEX-MSE earth retention system was installed to laterally confine the subgrade supporting these foundations provided that this earth retention system is founded on competent, very dense/hard subgrade materials and at sufficient depths such that shallow landslide failures would not intersect the FLEX-MSE system.

Contents

Appendices

1.0 INTRODUCTION

Ram Consulting (Ram) was retained by the City of Vancouver, referred to as the COV hereafter, to undertake a geotechnical assessment of the existing slopes situated below the north and east termini of Dunbar Street and Cameron Avenue, respectively.

It was understood that a localized failure of the slope located below the east terminus of Cameron Avenue and to the north of the existing beach access staircase occurred during November, 2021, following a period of significant rainfall. A preliminary geotechnical assessment of the slopes was previously undertaken by Golder Associates of Vancouver, BC, immediately following the aforementioned slope failure. Monitoring of the slopes was also previously carried out on a near-monthly basis by Golder Associates from December 2021 to March 2022 following the slope failure and by the COV during the period of April to September 2022. Ram has been undertaking monitoring of the slopes since December, 2022. However, it should be noted that there was a pause in the monitoring between May, 2023 to February, 2024 as the initial monitoring period proposed by the COV in the original Request for Services document (RFS # PS20220727) had elapsed. Additional slope monitoring was requested by the COV via email correspondence on February 2, 2024. Ram has resumed the slope monitoring program since March, 2024.

As further described herein, spalling of the siltstone/sandstone exposure situated at the lower portion of the South Slope was documented to have occurred during slope monitoring site visits completed on April 21, 2023 and most recently, on March 22, 2024. The conditions of the North Slope have generally remained unchanged relative to those observed from our initial slope monitoring site visit completed on December 30, 2022.

The purpose of this assessment is to provide recommendations to stabilize the aforementioned slopes and staircase, if determined to be required. The following document is a conceptual design geotechnical report presenting the results of the slope analysis carried out for the subject slopes and the proposed slope remediation and staircase reparation options. The findings and results of our field investigations which comprised a slope reconnaissance, carried out on October 12, 2022 and subsurface investigations, carried out on January 27 and February 3, 2023, were previously summarized in a factual report, dated April 16, 2024. This information is also presented in this report and discussions regarding our interpretation of these findings with respect to the stability of the slopes and staircase is also presented herein. The observations and findings from the previous geotechnical assessment and subsequent slope monitoring site visits carried out by both Ram and others are also summarized and referenced in this report as background and supplementary information. These findings were also interpreted and taken into consideration for the provision of the proposed risk management strategies for the subject slopes and staircase. Recommendations to address the above-noted spalling and related considerations are also provided herein.

As required per the original Request for Services document (RFS # PS20220727), the advantages and disadvantages of each of the proposed slope remediation and staircase reparation options are presented herein. Class D cost estimates and preliminary schedules for the design and construction of each of the proposed remediation options are also presented in this report.

This report has also been prepared in general conformance with the aforementioned scope of services.

2.0 SITE DESCRIPTION

2.1 General

The subject site encompasses an approximately 45 meter long portion of the coastal slope which overlooks the south shore of Burrard Inlet and extends approximately between Jericho and Kitsilano Beaches in the COV. This portion of the slope is bordered by the shore of Burrard Inlet to the northeast, the northernmost terminus of Dunbar Street to the south, the residential property of 1509 Dunbar Street to the southeast, the cul-de-sac at the east terminus of Cameron Avenue to the west and another residential property, having a civic address of 3607 Cameron Avenue to the northwest. A site location plan is provided as Figure 1, attached to this report in Appendix A.

The subject portion of the slope faces the northeast direction and is approximately bisected by an existing concrete staircase, oriented in the east-west direction, which provides pedestrian access to the shore of Burrard Inlet from Dunbar Street and Cameron Avenue. For description purposes, the portions of the subject slope located to the north and south of this staircase will be referred to as the “North Slope” and “South Slope”, respectively, herein this report. The layout of the site is presented in the Site and Test Hole Locations Plan attached as Figure 2 and appended in Appendix A.

The November 2021 landslide was triggered within the North Slope whereas the spalling which was documented to have occurred during our May 10, 2023 and March 22, 2024 site visits encompassed the lower portion of the South Slope that fronts the north terminus of Dunbar Street.

2.2 Beach Access Staircase

The existing concrete staircase is estimated to be approximately 9 metres in height and extend a horizontal distance of approximately 24 metres. The subject staircase spans between four concrete landing pads seated at the upper and mid-slope portions of the subject slope. The lower portion of the staircase had been constructed in between two, sloped concrete walls. This portion of the staircase and the adjacent walls are supported on top of an approximately 200 millimetre thick concrete slab, which also serves as the lower landing for the staircase. A light post was also observed to be mounted on the northern edge of a landing located at the mid-span portion of the staircase.

2.3 North Slope

The topographic LIDAR data provided by the COV online mapping system, VanMap Viewer, indicates that the existing grades at the North Slope are moderately steep to steeply sloping down towards the northeast from geodetic elevations of approximately El. 12 metres at its crest to El. 3 metres at its toe, over horizontal distances of approximately 12 to 14 metres. This equates to slope gradients of approximately 33 to 37 degrees from horizontal. The subject slope is generally overlain by low-lying vegetation comprising shrubs including Himalayan blackberry.

A municipal boulevard is located between the cul-de-sac of Cameron Avenue and the crest of this slope. This boulevard is relatively flat and is currently landscaped with grass and a hedge situated along its southern and western edges. A BC Hydro kiosk, supported on a concrete pad is located at the western portion of this landscaping area. As further described in Section 7.2 of this report, there are also underground utilities located below this landscaping area. These utilities include gas and electrical distribution lines.

2.4 South Slope

Topographic data from VanMap Viewer generally indicates that the South Slope is characterized by steep gradients of approximately 50 to 55 degrees from horizontal. The existing grades at this slope are indicated to slope down from geodetic elevations ranging from approximately EL. 12.5 to 13.0 metres at its crest to approximately El. 3.0 metres at its toe. The cross sectional length of this slope is estimated to vary from approximately 8 to 11 metres based on the aforementioned map. At the time of preparation of this report, the subject slope was covered by low vegetation (i.e. shrubs and bushes) and several trees.

The portion of the site situated between the crest of the South Slope and the roadway at the northern terminus of Dunbar Slope comprises a lawn area. This lawn area encompasses an area of approximately 53 square metres in plan and features two park benches, installed near the crest of the subject slope. The existing grades within this lawn area are generally gently sloping down towards the northwest. A pedestrian pathway is situated to the west of this lawn and between the crest of the South Slope and the residential property at 1509 Dunbar Street. The northern edge of the pathway and lawn area is delineated by a chain link fence. The pathway is oriented parallel to the crest of the subject slope and extends from the aforementioned lawn area to the cul-de-sac of Cameron Avenue. The longitudinal profile of the pathway slopes gently down from the lawn area to the uppermost landing of the beach access staircase. From this landing, the existing grades of the pathway flatten towards the cul-de-sac of Cameron Avenue. The existing finished grades of the pathway are elevated approximately 600 to 750 millimetres above the crest of the South Slope and are currently retained by a concrete retaining wall. This retaining wall is estimated to be approximately 600 to 750 millimetres in height and is constructed near the crest of the subject slope. There is another retaining wall bordering the pathway to the south which is currently managing the grade differences between the pathway and the backyards and driveways of the adjacent residential properties located above and to the south.

As further described in Section 7.2 of this report, there are also underground utilities located below the aforementioned pathway and lawn area. These utilities include telecommunications and electrical distribution lines.

2.5 Beach Front

The beach deposits exposed the shoreline of the adjacent portion of Burrard Inlet were generally observed to consist of a mixture of sand, gravel, cobbles and scattered sandstone boulders. The topography at the shoreline is gently sloping down towards the north.

3.0 BACKGROUND INFORMATION

3.1 Document Review

For the purpose of preparing this report, we have received, read and interpreted the following documents, maps and email correspondence related to the project and site:

- Surficial Geology Map (Map 1486a), dated 1979 and prepared by Geological Survey of Canada (GSC),
- Slope monitoring field review report dated December 9, 2021, prepared by Golder Associates,
- Email correspondence, dated December 22 and 28, 2021, January 4, February 1, and March 9, 2022, between Golder Associates and the COV,
- “Geotechnical Slope Assessment Report for 1500 Block Dunbar Street, Vancouver, BC”, dated April 7, 2022 prepared by Golder Associates Ltd., and
- Slope monitoring field reports dated March 9, April 29, July 12, August 17 and September 23, 2022, prepared by the COV,
- COV comments and recommendations, provided on February 2, 2024, related to the original draft conceptual report, dated June 23, 2023.

3.2 Surficial Geology

The subject site is indicated in the above-listed GSC map to be situated near the interface in surface expression of Vashon Drift and Capilano Sediments, and Tertiary Bedrock.

The Vashon Drift deposits are expected to comprise glacial drift including lodgement and minor flow till, lenses and interbeds of substratified glaciofluvial sand to gravel and lenses and interbeds of glaciolacustrine laminated stony silt. These deposits are expected to be overlain by Capilano Sediments comprising marine and glaciomarine stony (including till-like deposits) to stoneless silt loam to clay loam with minor sand and silt.

The Tertiary Bedrock is indicated to include “sandstone, siltstone, shale, conglomerate and minor volcanic rocks”. Where bedrock is not exposed it is it is expected to be overlain by up to 10 metres of the previously described glacial deposits.

3.3 Previous Slope Assessment (by Golder Associates)

A synopsis of the findings from the preliminary assessment completed by Golder Associates is provided below.

Based on our review of the aforementioned geotechnical assessment report, it is understood that a localized failure of the portion of the slope located to the north of the existing staircase had occurred in November 2021, following a period of significant rainfall. A visual assessment of the slope was subsequently carried out by Golder Associates on December 9, 2021. The slope failure was identified to be in the form of a localized, shallow landslide which initiated at a setback of 10 metres to the east of the cul-de-sac of Cameron Avenue. The landslide scarp was indicated in the report to be approximately 15.2 metres in length and approximately 4.8 and 8.2 metres wide at the top and bottom, respectively. The landslide was also identified to have occurred within the surficial vegetation and mineral soil layers mantling the North Slope. Other indicator signs of slope movement/instability documented in the report included the development of tension cracks at the crest of the failed portion of the slope as well as a shallow linear depression located to the north of the slide area and parallel to the crest of the slope. These observed indicator signs of slope instability were inferred by Golder Associates to be indicative of previous slope instabilities unrelated to the November, 2021 failure.

The report states that no indicator signs of slope instability were evident at the portion of the slope located to the south of the staircase. Two cavities, understood to have been anthropogenically developed, were reported to have been observed at the toe of this portion of the slope. The cavities were indicated to be approximately 1.5 metres wide and extend 2 metres into this portion of the slope which had resulted in the undermining of the toe of the slope. Furthermore, the cavities were found to be connected at the back. It was noted in the assessment that wave action during periods of high tide was impacting the toe of the slope and that the root mass of several trees at the toe of the south slope was currently providing support to the slope at its base (i.e. at the time that the Golder report was authored). However, the toe of the slope was indicated to be susceptible to tidal-induced erosion which would also potentially result in expansion of the existing cavities and in turn, trigger additional slope instability.

The report indicates that the failure of the north slope was likely induced by a heavy rainfall event which occurred during November, 2021. However, the depression at the north side of the slope was inferred to be an indicator sign of previous slope movement unrelated to the November, 2021 event. Furthermore, as the slope was determined to be subject to erosion at its toe due to wave action, this could have resulted in over-steepened and unstable geometries over time.

Recommendations pertaining to both short and long-term slope stability management for the subject site were also provided in the report. These recommendations included the following options:

Short Term Measures

- Restricting public access to beach access staircase,
- Periodic slope monitoring to assess the conditions of the slopes for any changes and additional signs of slope movement/instability,
- Monitoring the toe of the slope for any impactful human activity, and
- Slide debris which had deposited at the toe of the North Slope was recommended to be maintained in place in order to provide short term protection of the slope against tidal erosional action.

Long Term Measures

- Development and implementation of a long-term slope monitoring plan,
- Permanent closure of the existing beach access staircase, and/or,
- Re-grading of the failed portion of the North Slope and installing rip-rap armouring at the toe of this slope for retention of loose material and protection against wave action,
- The recommended remedial measures to address the undermined portion of the south slope included backfilling the existing cavities and installation of rip-rap armouring.

3.4 Previous Slope Monitoring Observations

As previously mentioned, the conditions of the subject slopes were monitored by Golder Associates during the period of December, 2021 to March, 2022 and by the COV from April to September, 2022. The observations from this monitoring period, documented by others, are summarized in Table 1, appended to the factual report.

As summarized in the aforementioned table, additional signs of slope deterioration in the form of soil ravelling and sloughing were observed at the North Slope during the February 1 and March 9, 2022 site visits and at the South Slope during the March 9, 2022 site visit. It is understood that no discernable changes in the conditions of the North Slope, relative to the conditions documented in the March 9, 2022 site visit were observed by the COV throughout the period of April to September, 2022. No comments regarding the conditions of the South Slope were included in the field review reports prepared by the COV. As summarized in Horizon Engineering's slope monitoring memoranda dated March 16, and May 10, 2023, the conditions of the North Slope and the staircase have generally remained unchanged relative to the conditions observed by the COV during their March 9, 2022 site visit and throughout Horizon Engineering's monitoring period of December 31, 2022 to May 24, 2023. However, as summarized in the aforementioned memorandum, dated May 10, 2023, it was observed on our April 21, 2023 site visit that an approximately 10 metre long, localized portion of the siltstone/sandstone exposure which forms the lower portion of the South Slope had spalled. The most significantly spalled portion of the slope was measured to be approximately 2.1 metres high, 1.5 to 1.8 metres wide and 600 millimetres thick (i.e. into the slope). This spalling failure had resulted in the formation of an overhanging slope geometry at this portion of the subject slope. Additional details regarding this spalling failure of the south slope and recommendations for managing risks associated with this slope deterioration were previously provided in the aforementioned memorandum. It was inferred from a site visit completed on March 22, 2024, that a wedge of sandstone/siltstone that was overhanging the previously spalled area, described above, had detached from the slope. The detached debris was estimated to be approximately 1.0 metre in width, 1.2 metres in length and 1.0 metre in depth.

4.0 FIELD INVESTIGATIONS

4.1 General

The field investigations undertaken for this slope assessment comprised the following:

- An initial slope reconnaissance carried out on October 7, 2022 to visually assess the conditions of the slopes and staircase,
- Subsurface investigation, comprising two exploratory test holes advanced with a sonic drill rig on January 27, 2023. These test holes were advanced at the portions of the roadways situated at the north and east termini of Dunbar Street and Cameron Avenue, respectively, and
- Subsurface investigation, comprising a total of five WildCat Dynamic Cone Penetrometer Test Soundings advanced near the accessible portions of the crests of the North and South Slopes on February 3, 2023.

It should be noted that the landscaping and lawn areas situated adjacent to the crests of these slopes were previously identified during the initial site reconnaissance as being ideal test hole locations for providing optimal representative stratigraphic profiles of the slopes. However, the existing hedge planted at the east end of Cameron Avenue and an existing road barrier at the terminus of Dunbar Street precluded drill rig access to these test hole locations. Furthermore, based on email correspondence dated October 24, 2022 from the COV, it was understood that these features could not be removed to accommodate the investigation due to schedule and budgetary constraints. Although the test holes were not advanced at the ideal locations, as previously described, it is envisaged that actual test hole locations were still suitable for the purposes of this assessment.

As required, a Street-Use Permit was obtained from the COV to facilitate the aforementioned subsurface investigations. The locations of traceable underground services underlying the work areas were delineated by a utility locate subcontractor (FJM Utility Locate of Coquitlam, BC) on January 24, 2023 to reduce the risks of damaging them during the subsurface investigations. Ground Penetrating Radar (GPR) and Electromagnetic (EM) techniques were used to locate traceable underground utilities.

s.18

[redacted]

The services of an arborist consultant, B.A. Blackwell & Associates Ltd. of North Vancouver, BC, were also retained to undertake an assessment of the trees and vegetation overlying the North and South Slopes. The purposes of this arboreal assessment were to assess the health of the existing trees and the impacts of the vegetation cover on the stability of these slopes as well its role on shading the foreshore and creating a nesting habitat. The findings and recommendations from this assessment are provided in a separate report, dated May 21, 2024, and prepared by B.A. Blackwell & Associates Ltd.

4.2 Slope Reconnaissance

On October 7, 2022, Mr. Robert Ng, P.Eng., and Mr. Jason Tam, P.Eng., both of Ram, attended the subject site to carry out a slope reconnaissance. Representatives from the COV were also in attendance for a portion of this reconnaissance. The reconnaissance consisted of visual site observations with respect to slope stability and measurements of existing slope geometries at locations that were assessed to be practical.

This field assessment was completed under sunny weather conditions and during a period of low tide. It should be noted that the photographs referenced below were taken during this initial reconnaissance. All photographs referenced below are attached to this report in Appendix A.

4.2.1 North Slope

At the time of our slope reconnaissance, the North Slope was observed to be densely vegetated with shrubs. During this site visit, visual observations of the slope conditions were made from the crest and toe of the slope and from the beach access staircase. Measurements of the slope geometry and landslide failure surface were also taken. Our observations and measurements for the North Slope are summarized below:

- The subject slope was estimated to have an overall gradient of approximately 35 degrees from horizontal based on slope angle measurements taken with an inclinometer, which is generally consistent with the slope angles estimated from the available topographic data. The overall slope length was measured to be approximately 18.5 metres.
- The footprint of the landslide was inferred to encompass the upper portion of the subject slope based on the geometry of the portion of the landslide channel which was not covered with vegetation and visible at the time of this reconnaissance.
- The landslide was measured to have initiated at a setback distance of 5.1 metres from the closest edge of the BC Hydro kiosk located within the western portion of the landscaping area situated between the North Slope and the cul-de-sac of Cameron Avenue.

- As shown in Photograph 1 of Figure 3, the headscarp of the landslide was approximately arc-shaped in plan and measured to be 1.1 meters in height. The headscarp area was observed to be moderately vegetated with small shrubs and exhibited a near-vertical profile. Where exposed, the soils at the headscarp area of the landslide generally consisted of brown silty sand with some to gravelly. No obvious indicator signs of seepage were observed at the headscarp area of the landslide.
- Where visible/exposed, the landslide surface which had developed below the headscarp was estimated to be inclined at gradients varying from approximately 33 to 40 degrees from horizontal.
- A traffic barrier was located mid-slope and within the landslide channel. The traffic barrier was measured to be situated at a distance of approximately 4.3 metres, measured parallel to the slope, from the crest of the slope.
- Mixed landslide debris, consisting of uprooted vegetation and brown silty sand, with some gravel had deposited at the mid and downslope portions of the slope. The distance between the top of the debris deposit and the crest of the subject slope was approximately 7.5 metres (measured parallel to the slope). This landslide deposit is approximately delineated in Photograph 2 of Figure 3 for reference. As shown in Photograph 3 of Figure 4, the landslide deposit at the bottom of the slope was measured to be approximately 900 millimetres in thickness.
- As also indicated in Photograph 2 of Figure 3, the landslide channel was measured to be approximately 4.2 and 6.0 metres wide at the top and bottom, respectively.

Based on the above observations, the landslide was inferred to have been triggered within the loose, surficial, sandy deposits mantling the subject slope.

s.22(1)

[redacted]

4.2.2 South Slope

The South Slope was observed to be overlain by low lying vegetation (i.e. shrubs and bushes) and small trees. The subject slope was estimated to be generally steeply sloping at gradients of approximately 50 degrees from horizontal. The section of this slope situated below the lawn area located at the north terminus of Dunbar Street was noted to steepen to near-vertical at the bottom 3 metres where bedrock outcrops comprising sandstone/siltstone were present. A photograph of the South Slope, looking upslope from the beach is provided as Photograph 4 in Figure 4.

A ground traverse of the subject slope completed during this site visit indicated that the upper portion of the subject slope is overlain by a thin deposit of inferred fill materials comprising light grey to brown sandy silt mixed with trace to some organics (i.e. roots and rootlets) and trace amounts of various debris, which included plastic bags and beverage containers. A photograph of the typical surficial materials overlying the slope are shown in Photograph 5. An exposure of light brown to grey silt, inferred to be a hard, natural material was also observed at the mid-slope portion of this slope, as shown in Photograph 6 of Figure 5. No obvious indicator signs of slope instability were observed at this slope at the time of our reconnaissance, however, as previously documented in our slope monitoring memorandum dated March 16, 2023, an approximately 38 millimetre wide gap was present between the existing pathway and backside of the retaining wall that is currently supporting it, located at the crest of this slope. As discussed in the aforementioned memorandum, it is possible that the presence of this gap could be consistent with previous movement of the slope and/or retaining wall.

The bottom 1.8 metres of the sandstone/siltstone outcrops situated at the bottom of the South Slope were observed to be wet at the time of our site reconnaissance and exhibited some local seepage. The sandstone/siltstone exposed at this outcrop was generally noted to be light brownish grey in colour and exhibited some prominent, weathered, dark brown discontinuities. These discontinuities were noted to be striking in the approximate north-south direction and dipping down to the east. It is envisaged that these discontinuities may be potential slip or erosional surfaces. A photograph showing a portion of the bedrock outcrop and its discontinuities is provided in Photograph 7 of Figure 6.

The geometries of the existing cavities situated at the bottom portion of the south slope located near the staircase were generally noted to be consistent with those previously described in Section 3.3 and as documented in the preliminary assessment report prepared in 2021 by Golder Associates.

As noted in Section 3.4 above and documented in detail in our slope monitoring memorandum dated May 10, 2023, it was observed during a site visit completed on April 21, 2023 that spalling had occurred within an approximately 10 metre long, localized portion of the siltstone/sandstone exposure which forms the lower portion of the South Slope. Further spalling of this area, which comprised the detachment of an overhanging portion of the siltstone/sandstone exposure located above the subject area, was inferred to have occurred based on a more recent site visit completed on March 22, 2024. The observations from this site visit are documented in a memorandum dated March 30, 2024.

4.2.3 Beach Access Staircase

At the time of our 2022 slope reconnaissance, we did not observe any obvious indicator signs of significant structural distress (i.e. cracking of the concrete) of the staircase or undermining of its landing pads. However, the steps of the staircase appeared to exhibit some distortion. It was understood that this distortion was evident prior to the slope failure that occurred during November, 2021. Furthermore, as shown in Photograph 8 of Figure 6, the lower portion of the staircase is abutted by an approximately 900 millimetre thick concrete slab located to the north of the adjacent concrete sidewall. It is possible that this concrete slab was previously placed to buttress this portion of the staircase.

During our slope reconnaissance, a 1.0 metre long soil probe was also advanced through the accessible portions of the slope located to the north of and adjacent to the second and third landing pad foundations, numbered from the top of the slope, in an effort to qualitatively assess the relative denseness/consistency of the surficial slope materials and the subgrade supporting the staircase. Loose/soft soil conditions were inferred to have been encountered from surface at the soil probe test locations, based on the resistance of these surficial slope materials to penetration with the probe. These loose soil conditions generally extended to at least the maximum depths of the soil probe tests (i.e. 1.0 metre). Where exposed, the surficial slope materials encountered at the soil probe test locations generally comprised brown, fine to medium grained sand with trace to some gravel.

It should also be noted that the minimum horizontal setback distance between the southern sidewall of the landslide channel of the North Slope and the closest edge of the landing pad foundations was measured to be approximately 1.2 metres. The founding depths of the landing pads could not be confirmed at the time of this site reconnaissance nor at any time thereafter.

The observed conditions of the staircase are shown in Photograph 8 on Figure 6.

4.3 Sonic Drilling and Piezometer Installation

A track-mounted sonic drill rig, supplied and operated by Blue Max Drilling of Surrey, BC, was mobilized to site on January 27, 2023 to advance test holes at the roadways situated at the North and East termini of Dunbar Street and Cameron Avenue, respectively. The test holes advanced at the roadways of Cameron Avenue and Dunbar Street are denoted as “SH23-01” and “SH23-02”, respectively, in the site and test hole locations plan attached as Figure 2. SH23-01 was advanced at a setback distance of approximately 6.0 metres east of the crest of the North Slope whereas SH23-02 was advanced at a setback distance of approximately 10.0 metres south of the crest of the South Slope.

A sonic drill is a rotary vibratory drill casing capable of extracting continuous cores. The sonic drill head works by sending high frequency resonant vibrations down the drill casing string to the drill bit. The frequency ranges between 50 and 150 Hertz (cycles per second) and is varied to optimally suit the subsurface conditions. These vibrations cut and cause the surrounding soil at the drill string and bit to ‘fluidize’ which allows for relatively easy penetration into the soil layers from which a continuous soil core is extracted. The sonic drill holes were approximately 200 millimetres in diameter.

The sonic drill holes were advanced to depths of 12.1 metres below the finished grades of the aforementioned roadways. These investigation depths were assessed to be sufficient for slope stability analyses purposes in consideration of the geometries of the slopes, the subsurface conditions encountered within each test hole and the observations from our slope reconnaissance. In order to evaluate the in-situ relative densities of the subsurface materials encountered at each test hole, Standard Penetration Test (SPT) soundings were advanced within the sonic drill holes. The SPT involves driving a split spoon sampler, having a diameter of 50 millimetres, into the soil with a 63.5 kilograms drop hammer falling through a height of 750 millimetres. The number of blows required to drive the split spoon sampler 450 millimetres is recorded and the number of blows required to advance the rod from 150 to 450 millimetres is reported as the “N” value, which can then be correlated to the relative denseness of the soil. SPT soundings were advanced at depth intervals of 1.5 metres within the upper 4.5 metres of the subsurface at the test hole locations. At depths below 4.5 metres, the SPT soundings were advanced at irregular depth intervals, varying between 1.5 and 3.0 metres.

Two piezometers, comprising 25 and 50 millimetre diameter PVC pipes, were installed in nested configurations at each borehole to facilitate monitoring of groundwater levels within different depths and strata. The nested piezometers installed in borehole SH23-01 are denoted as “PZ23-01” whereas the nested piezometers installed in SH23-02 are denoted as “PZ23-02” in Figure 2. The 25 millimetre and 50 millimetre diameter piezometers within each borehole were installed to respective depths of approximately 3.0 and 9.0 metres below the existing roadway grades at each test hole location. The intake zones of the shallow and deeper piezometers were developed with respective screen lengths of 1.5 and 3.0 metres. The annuli between the screened intake zones of the PVC standpipe piezometers and boreholes were backfilled with filter sand. The annular spaces located below, between and above the filter media of the screens for the nested piezometers were sealed with bentonite chips. The upper bentonite seals of the nested piezometers were developed to within approximately 450 millimetres below the finished roadway grades. Filter sand was placed as backfill within the annular spaces above the upper seals of the piezometers.

Upon completion of the piezometer installations, the tops of the piezometers were capped with 'J-plugs'. The boreholes and piezometers within were covered with flush-mount well covers embedded in concrete placed between the well cover and the adjacent pavement of the roadways. The aforementioned details regarding piezometer depths, configurations and backfill materials are also shown schematically in the test hole logs attached to this report in Appendix C. We recommend that the piezometers remain capped and secured during the period of service and be decommissioned as per the British Columbia Groundwater Protection regulations under the Water Sustainability Act (February 29, 2016) at the end of their service lives.

The sonic drilling investigation was directed and supervised by a field engineer from our office. The soil stratigraphy was logged in the field and select SPT and grab soil samples were collected for further soil characterization.

4.4 WildCat Dynamic Cone Penetrometer Tests

WildCat Penetrometer Test soundings, which are carried out utilizing portable and hand operated equipment, were advanced near the crests of the subject slopes as these portions of the site were not accessible with drill rigs. A total of two and three WildCat Penetrometer Test soundings were advanced near the crests of the North and South Slopes, respectively, and within the municipal landscaping and lawn areas situated adjacent to these slopes. The approximate locations of the WildCat Penetrometer Test soundings are indicated as WC23-01 to WC23-05 in Figure 2. WC23-01 through WC23-03 were advanced near the crest of the South Slope whereas WC23-04 and WC23-05 were advanced near the crest of the North Slope.

The WildCat Cone Penetrometer Test entails driving a steel cone, attached to a series of steel rods, below the subsurface with a 16 kilogram drop hammer raised to a height of 380 millimetres. Blow counts are logged for every 100 millimetres of penetration of the rods below ground surface. Correlations of blow counts are made to equivalent SPT N values which are then used to assess the in-situ density/consistency of the subsurface materials. Soil samples are not retrieved during the WildCat Penetrometer Tests.

The WildCat Cone Penetrometer Test soundings were advanced to delineate the thicknesses of any loose deposits that may be underlying the crests of the aforementioned slopes which would inform the development of the slope stability analyses models for this assessment. As further described in Section 5.2 below, these test holes were terminated at depths ranging from 1.6 to 2.3 metres as effective refusal was encountered at these depths.

The WildCat Penetrometer Test soundings were carried out by two field technicians from our office.

5.0 SUBSURFACE CONDITIONS

Summaries of the subsurface conditions encountered at the sonic and WildCat Penetrometer Test hole locations are provided below. It should be noted that the depths referenced below are relative to the existing site grades at the respective test hole locations.

Detailed descriptions of the subsurface materials encountered at each sonic drill hole are provided in the test hole logs appended to this report. As indicated in the logs, the collar elevations of SH23-01 and SH23-02 were estimated to be at approximately El. 12.0 and El. 14.2 metres based on available topographic data for the subject site.

5.1 Soil Stratigraphy

The sonic test hole locations were found to be surfaced with approximately 300 and 75 millimetre thick layers of concrete and asphalt pavement at SH23-01 and SH23-02, respectively. The surficial pavement layers were found to be underlain by fill materials, comprising sand with varying gradations and silt and gravel contents. These fill materials were inferred to be compact and extended to depths of approximately 0.9 metre at SH23-01 and 2.0 metres at SH23-02. At SH23-01, these fill materials were underlain by an approximately 1.1 metre thick layer of reddish to light brown sand which was inferred to be a natural and compact material. At SH23-02, the fill materials were underlain by a 700 millimetre thick layer of light grey mottled brown to brown silt, which was inferred to be natural and firm to stiff in consistency. The aforementioned sand and silt were in turn underlain by a hard/very dense, grey to light brown silt and sand to sand and silt which was approximately 1.2 and 3.1 metres in thickness at SH23-01 and SH23-02, respectively, and found to be slightly lithified at SH23-01. This sand was found to grade into a hard, slightly lithified, grey sandy silt at depths of approximately 3.2 metres at SH23-01 and 5.8 metres at SH23-02. A discrete, 200- to 500-millimetre-thick stratum of dark brown silt with some fine-grained sand, inferred to be very stiff, and inferred to comprise decomposed organic matter was encountered below the aforementioned sandy silt at depths of approximately 5.3 and 7.6 metres at SH23-01 and SH23-02, respectively. Bedrock, characterized as extremely weak to weak siltstone, was found to underlie the aforementioned dark brown silt at depths of approximately 5.8 and 7.7 metres below the existing grades at SH23-01 and SH23-02, respectively. Both test holes were terminated at a depth of 12.1 metres within the siltstone stratum.

A generalized stratigraphic profile of the subsurface materials encountered at the drill hole locations is summarized below in order of increasing depth:

5.1.1 Concrete/Asphalt Pavement

Approximately 300 and 75 millimetre thick layers of concrete and asphalt pavement were encountered at the existing ground surfaces of SH23-01 and SH23-02, respectively.

5.1.2 Fill

At SH23-01, the fill materials comprised brown, moist, fine grained silty sand which graded into a reddish brown, moist, fine to medium grained sand at depths of approximately 900 millimetres. The reddish-brown sand, in turn, graded into a light brown, moist, well graded sand at depths of 1.2 metres. The fill materials at this test hole extended to a depth of approximately 2.0 metres.

At SH23-02, the fill materials comprised from top, 525 millimetres of a grey, dry, well graded sand with some gravel which graded into a brown, dry to moist, fine to medium grained sand. This well graded sand was underlain by 300 millimetres of brown to dark brown, moist, fine to medium grained sand. At depths below 900 millimetres, the inferred fill materials comprised grey, dry to moist, well graded sand with trace to some gravel. The fill materials at this test hole extended to a depth of approximately 2.0 metres.

The previously described fill materials were inferred to be compact/stiff based on the SPT soundings advanced within these materials and the observed drilling efforts.

5.1.3 Sand

At SH23-01, the fill materials were underlain by inferred natural materials comprising reddish brown, moist, fine to medium grained sand with trace gravel at a depth of approximately 900 millimetres. The reddish brown sand was found to grade into a light brown, moist, well graded sand at a depth of 1.2 metres. This sand layer was inferred to be compact to dense and found to extend to a depth of approximately 2.0 metres below existing site grades at SH23-01.

5.1.4 Silt

At SH23-02, the fill materials were found to be underlain by inferred natural material comprising light grey mottled brown, moist, sandy silt with fine grained sand, trace gravel, and occasional cobbles which was inferred to be stiff. Below a depth of 2.3 metres, the silt was observed to be light brown in colour and to contain some sand and trace clay.

5.1.5 Silt and Sand to Sand and Silt

At both test hole locations, the previously described sand and silt were underlain by a moist, silt and sand to sand and silt which was inferred to be hard/very dense as the SPT soundings encountered effective refusal within this stratum.

At SH23-01, this stratum was found to be light brown in colour, slightly lithified and approximately 1.2 metres in thickness. The thickness of this stratum was found to increase to 3.1 metres at SH23-02, and the colour of the upper 1.7 metres of this stratum encountered at SH23-02 was noted to be grey. The colour of these materials encountered at SH23-02 was noted to grade into light brownish grey at a depth of approximately 4.4 metres.

5.1.6 Sandy Silt

A grey, moist, sandy silt (with fine grained sand) was encountered below the previously described sand stratum. This material was inferred to be natural, slightly lithified and hard in consistency based on the SPT soundings. This layer was found to be approximately 2.1 metres and 1.8 metres thick at SH23-01 and SH23-02, respectively.

5.1.7 Silt

The aforementioned sandy silt to silt and sand was found to be underlain by a relatively thin layer of dark brown, moist silt with trace to some fine grained sand. It should be noted that the dark brown coloration of this material may be an indication that it consists of decomposed organic matter. These materials were inferred to be partially petrified and very stiff to hard in consistency based on the SPT sounding advanced through these materials at SH23-02. This stratum was found to be approximately 500 and 200 millimetres thick at SH23-01 and SH23-02, respectively.

5.1.8 Bedrock (Siltstone)

Bedrock, comprising lightly cemented siltstone, was encountered at depths of approximately 5.8 and 7.7 metres below existing site grades at SH23-01 and SH23-02, respectively. At SH23-01, the siltstone was inferred to be very weak to weak and generally comprised grey, moist silt with some fine grained sand. At SH23-02, the siltstone was generally inferred to be weak and comprise light brownish grey to grey, moist, sandy silt with 600 millimetre thick interbeds of grey, moist, slightly plastic silt containing trace fine grained sand and trace clay. These interbedded layers within the siltstone, comprising slightly plastic silt, were inferred to be extremely weak in strength and encountered at depths of 9.1 and 10.7 metres.

Both test holes were terminated within this stratum. As previously described in Section, this bedrock stratum outcrops at the lower portion of the South Slope.

5.2 WildCat Dynamic Cone Penetrometer Test Soundings

WildCat Cone Penetrometer Test logs for WC23-01 to WC23-05 are attached to this document in Appendix C. WC23-01 to WC23-03 were advanced near the crest of the south slope and within the lawn situated above this slope. WC23-04 and WC23-05 were advanced adjacent to the crest of the North Slope and within the landscaping area located above this slope. A summary of the soil conditions encountered at these test hole locations is provided below and is based on the correlation between the blow counts recorded for the WildCat Penetrometer Test soundings and SPT "N" values.

5.2.1 South Slope (WC23-01 to WC23-03)

WC23-01 to WC23-03 encountered very loose to loose/soft to firm soil conditions from the existing ground surface at these test hole locations to depths of 1.3 metres. Compact/stiff soil conditions were in turn encountered below these depths which extended to 1.5 to 1.6 metres below the existing site grades at which depth dense to very dense/hard soil conditions were encountered. Effective refusals of these soundings were encountered at depths of 1.8 metres at WC23-01 and 1.6 metres at WC23-02 and WC23-03.

5.2.2 North Slope (WC23-04 and WC23-05)

Very loose to loose soil conditions were encountered from ground surface to depths of 2.0 and 1.7 metres at WC23-04 and WC23-05, respectively. The relative denseness/consistencies of the subsurface materials at these test holes were found to transition to compact to dense/stiff to very stiff below the aforementioned depths and effective refusals were encountered at depths of 2.3 and 2.1 metres at WC23-04 and WC23-05, respectively.

5.3 Groundwater Conditions

The groundwater levels and estimated, corresponding groundwater elevations, at the piezometer locations were measured on March 9, 2023 and May 23, 2023. A summary of the groundwater levels measured at these piezometers, to date, is provided in Table 1 below:

Table 1 Measured Groundwater Levels at Piezometers

Date	PZ23-01 (Cameron Ave)		PZ23-02 (Dunbar Street)	
	G.W. Depth/Elevation in Shallow Piezometer (metres)	**G.W. Depth/Elevation in Deep Piezometer (metres)**	**G.W. Depth/Elevation in Shallow Piezometer (metres)**	**G.W. Depth/Elevation in Deep Piezometer (metres)**
March 9, 2023	2.1 m/El. 10.0 m	6.2 m/El. 5.9 m	-*	4.7 m/El. 9.5 m
May 23, 2023	2.3 m/El. 9.8 m	5.7 m/El. 6.4 m	-*	5.4 m/El. 8.8 m

* Notes: - denotes dry conditions encountered within piezometer

The shallow groundwater level measured at PZ23-01 is interpreted to be indicative of a perched groundwater table situated near the interface of the upper, compact to dense sand and the underlying natural materials comprising hard/very dense silt and sand encountered at this test hole location. The deeper groundwater levels measured at the piezometers are inferred to be representative of ambient/static groundwater levels. It is possible that the deeper groundwater levels measured at PZ23-02 are perched on top of the siltstone stratum based on the depth of this groundwater table with respect to the subsurface conditions.

It is expected that the above aforementioned groundwater levels would be susceptible to seasonal fluctuations.

5.4 Stratigraphic Cross-Sections of Slopes

Cross sectional profiles of the North and South Slopes, indicated as Cross Section A-A and B-B, respectively, on the site plan provided in Figure 2, have been prepared and are attached as Figures 7 and 8 in Appendix A. These profiles show the approximate geometries of these slopes and generalized interpretations of the stratigraphic profiles of the slopes which have been developed based on the findings from our site investigations; geometric changes due to recent spalling are not shown. The geometries of these slope cross-sectional profiles have been developed based on our interpretation of the topographic data accessed via Van Map. These slope cross sectional profiles are estimated to be representative of the steepest portions of the North and South Slopes, as interpreted from the existing topographic data.

6.0 SLOPE STABILITY ANALYSIS

Slope stability analyses of the North and South Slopes were carried out to assess the susceptibility of these slopes to both global and localized failures as it is envisaged that this would be beneficial to the project team with respect to informing future risk management decisions regarding these slopes.

A commercially available limit equilibrium slope stability analysis program (GeoSlope, version 9.0.0.15234) was used to carry out the analyses for this project under static-loading conditions. It should be noted that the stability of these slopes under seismic loading conditions were not analyzed as it is understood that this loading condition is not a design consideration for this project.

The Morganstern-Price method of analysis was used to search for the most critical potential circular failure surface for each slope.

6.1 Modeling Criteria

For the purpose of communicating the comparative stability of a slope, a Factor of Safety (F.S.) may be determined for a given slope condition. Within the context of slope stability, the F.S. is the ratio of available shear resistance to driving forces along a potential slip surface, where the resisting forces stabilize a slope and the driving forces contribute to instability. A F.S. greater than 1.0 would indicate that the slope is more likely to be stable, while a F.S. less than 1.0 would indicate that the slope is likely to be unstable.

6.1.1 Slope Stability Model Set-up

To assess the stability of the subject slopes, the aforementioned slope cross sections, Sections A-A and B-B were modelled in Geoslope. The slope profiles were extended a sufficient distance beyond the crest and toe of both slopes for the purpose of assessing the F.S. of potential global slip surfaces. The slope stability analysis for the North Slope was completed for both the inferred pre-failure slope geometry and existing, post failure geometry. The analysis for the pre-failure slope geometry was completed in order to corroborate our understanding of the November, 2021 slope failure mechanisms and to allow back-analysis of this failure to be carried out. The stability of the existing, post slope failure geometry, comprising the landslide channel, was analyzed to assess the current F.S against failure of this slope.

The slope stability analysis models for each slope section were developed based on the interpolated stratigraphic cross sections of the slopes, as previously discussed in Section 5.4.

6.1.1.1 South Slope

Our observations of the existing slope conditions were taken into consideration for the calibration of the slope model and assigning soil parameters. As the F.S. of this slope was found to be largely governed by the strength parameters assigned to the silt and sand and the slightly lithified sandy silt layers encountered between depths of 2.7 and 7.6 metres at SH23-02, the calibration of the slope model was completed by varying the strengths of these materials until the critical slip surface approached a value of approximately 1.0 which would be indicative of marginally stable to potentially unstable slope conditions. This critical failure surface was found to initiate near the existing fence located near the crest of this slope and daylight near the top of the siltstone exposure located at the lower portion of the slope. The friction angle and cohesion values for these soil layers which yielded a F.S. of approximately 1.0 for this slip surface were found to be 34° and 17 kPa, respectively. In consideration of the observed stability of this slope (e.g. no indicator signs of global or shallow slope instabilities within the soil overlying the bedrock have been observed) and that the relative denseness/consistencies of these materials were inferred to increase below depths of 5.8 metres at SH23-02 based on the SPT blow counts and drill efforts, the cohesion of the slightly lithified sandy silt was increased to 27 kPa to carry out the initial analysis. Due to uncertainty in the soil parameters, a sensitivity analysis of the F.S. of this slope was also completed by calculating the F.S. using a range of cohesion values assigned to the aforementioned silt and sand and slightly lithified sandy silt layers. The F.S. was found to be highly dependent on this parameter. The strength parameters assigned to each stratum for the analysis of the South Slope are tabulated in Table 2 and shown on Figures 9 to 17, attached in Appendix D, which present the analysis results.

The groundwater level modelled in the analysis for the South Slope is situated at a depth of approximately 4.7 metres below existing grades at SH23-02. This groundwater level is based on the higher of the groundwater level measurements taken within piezometer PZ23-02 on March 9, and May 9, 2023. Conversatively, the groundwater level within this slope has been modelled to be level (i.e. flat) across the profile of the slope.

Table 2: Soil Parameters for Slope Stability Analysis of South Slope

Layer	Unit Weight (kN/m^3)	Friction Angle ()	Cohesion (kPa)
Inferred Fill/Loose Surficial Deposits	18	33	0
Sandy Silt to Silt	18	33	0
Silt and Sand	19	34	17-27 (varied for sensitivity analyses)
Sandy Silt (slightly lithified)	20	34	27-32 (varied for sensitivity analyses)
Silt (possible old topsoil layer)	18	35	0
Siltstone (Bedrock)	23	0	200

Of course, the stability of the slope should be expected to be highly sensitive to loss of toe support from the underlying bedrock. If the toe of the slope is eroded by wave action or spalling/detachment of the bedrock resulting in steeper slope gradients, this will adversely impact the slope stability.

6.1.1.2 North Slope

The pre-failure geometry of the North Slope was modelled based on the topographic data accessed via VanMap. The existing, post-failure slope geometry modelled in the analysis is based on the inferred failure surface of the slope and field measurements taken from our October 7, 2022 reconnaissance of the existing slope geometry and landslide area. The soil and bedrock strength parameters assigned for each stratum for this analysis are tabulated in Table 3 below and also presented on the figures showing the analyses results. The stratigraphy encountered within SH23-01 advanced near the North Slope on Cameron Avenue was generally noted to be consistent with the stratigraphy encountered at SH23-02. Thus, the same soil properties were used for the soil layers encountered below the surficial fill and loose materials. Sensitivity analysis of the soil properties for this slope was not completed as the slip surfaces were generally found to only encompass the upper loose/surficial deposits. Back analysis of this slope was completed, based on the inferred geometry of the shallow slip surface of the November, 2021 failure, in order to calibrate the strength values assigned to the surficial slope materials and the slope stratigraphy modelled in the analysis. The extreme rainfall event that was inferred to have triggered the slope failure (which is the subject of this study) was considered in this back analyses by modelling the phreatic surface of the slope to be situated near the interface of the upper loose, surficial deposits and the underlying natural silt and sand layer.

It should be noted that the surficial slope deposits mainly comprised silty sand with some gravel based on the soils exposed at the landslide area.

The colluvium (i.e. landslide debris) which had deposited on top of the lower portion of this slope was also included in the post-failure analysis model.

The groundwater levels modelled in the analyses for the subject slope were varied based on both the shallow, and deep groundwater levels, measured within PZ23-01 to assess the sensitivity of the analysis results to the different groundwater levels. As previously described in Section 5.1, the shallower and deeper groundwater levels measured within this piezometer were inferred to be perched on top of the upper hard/very dense silt and sand and bedrock (siltstone) layers, respectively. The higher of the groundwater levels as measured within the deep piezometer (i.e. 5.7 metres) was modelled in the analysis.

Table 3: Soil Parameters for Slope Stability Analysis of North Slope

Layer	Unit Weight (kN/m^3)	Friction Angle ()	Cohesion (kPa)
Fill/Loose Surficial Deposits (Sand)	18	34	0
Landslide Debris (Post Failure Conditions)	11	30	0
Silt and Sand	19	34	17
Sandy Silt (slightly lithified)	20	34	27
Silt (possible old topsoil layer)	18	35	0
Siltstone (Bedrock)	23	0	200

6.2 South Slope Analysis Results

The results of the slope stability analysis, carried out for local failure conditions of the South Slope, but not accounting for the spalled geometry, generally indicated that the upper 2.0 metres of the slope, which was inferred to be underlain by up to approximately 1.8 metres of loose soil based on test holes WC23-01 to WC23-03, is susceptible to localized, surficial instabilities. The envelope of slip surfaces which correlate to F.S. of 1.0 or less is presented in Figure 9 of Appendix A. These slip surfaces are generally representative of localized, surficial raveling of loose slope materials. The slip surface corresponding to a F.S. of 1.0 was found to initiate at a setback of approximately 1.0 metre to the south of the slope crest and daylight at a depth of approximately 2.0 metres at the face of the slope.

The slope stability analysis carried out for the subject slope for global failure conditions, based on the initial strength values summarized in Table 2, indicated that the critical slip surface, correlating to a F.S. of 1.16, encompasses a slip surface that initiates near the existing fence line located at the top this slope and daylights near the top of the bedrock exposure located at the lower portion of this slope. This slip surface is presented in Figure 10 of Appendix A. Potential slip surfaces which initiate from the existing road barrier on Dunbar Street and encompass the northernmost roadway of Dunbar Street were generally found to have a minimum F.S. of 1.37 based on the initial soil strengths used in the analysis. The critical slip surface which encompasses the road barrier located at the northern terminus of Dunbar Street is presented in Figure 11 of Appendix A for reference.

As previously discussed, the F.S. of the global slip surfaces summarized above were found to be sensitive to the cohesion values assigned to the silt and sand and sandy silt encountered between depths of 2.7 and 7.6 metres. Accordingly, a sensitivity analysis was carried out to assess the variance in the F.S. with respect to varying values of cohesion of these soil layers. The results of the sensitivity analysis are summarized in Table 4 below.

Table 4: Results of Sensitivity Analysis – South Slope

C'(kPa) Silt and Sand	**C'(kPa) Sandy Silt (slightly lithified)**	**F.S. of Critical Slip Surface**	**F.S. of Slip Surfaces Encompassing Roadway**
17	27	1.16	≥1.32
22	27	1.22	≥1.38
27	27	1.29	≥1.42
27	32	1.37	≥1.49

The analysis results generally indicate that surficial sloughing and raveling of the loose soils overlying the upper 2 metres of the slope may occur based on its relatively steep geometry. However, it is envisaged that the root mass from the existing vegetation cover of this slope is currently providing apparent cohesion of the surficial soils and, in turn, contributing to its observed stability against shallow and surficial slope failures. As indicated in the above table, the F.S. of the critical failure plane of the slope and the critical slip surface encompassing the roadway of Dunbar Street are expected to vary between 1.16 to 1.37 and 1.32 to 1.49, respectively, for the above-noted range of cohesion values assigned to the Silt and Sand and Sandy Silt.

As noted in Section 6.1.1.1 above, the F.S. will also be sensitive to retrogression of the toe of the slope, such as that associated with the spalling observed from the May, 2023 and March, 2024 site visits. Accordingly, the above-noted analyses were re-run with the revised (i.e. current geometry). The analyses generally indicated negligible impacts on the F.S. presented in the table above for the subject slope, however, it is inferred that this is partially attributed to the condition that the spalling failures have not yet resulted in the portion of the slope overlying the siltstone/sandstone being undermined. Accordingly, it is envisaged that any further spalling of this slope that results in the development of steepened slope geometries would adversely impact slope stability and the F.S. values summarized above.

6.3 North Slope Analysis Results

6.3.1 Pre-failure Slope Conditions

The analysis for the pre-failure geometry of the north slope, based on the deeper groundwater condition (i.e. perched on top of the siltstone layer), indicated that all potential slip surfaces which correlate to a F.S. of 1.0 or less would be triggered within the loose deposits mantling this slope. As presented in Figure 12 of Appendix C, the envelope of these potential slip surfaces was found to initiate at a maximum setback of approximately 2.0 to 2.5 metres to the west of the inferred pre-failure slope crest and daylight near the toe of this slope. The potential failure surfaces correlating to F.S. ranging from 1.0 to 1.3 were found to initiate within a comparatively larger setback of 5 to 5.5 metres from this crest and would also be triggered within the surficial slope materials. A potential, shallow slip surface encompassing the location of the existing BC Hydro kiosk, situated within the westernmost portion of this landscaping area, was also identified and found to correlate to a F.S. of 1.45. This slip surface is presented in Figure 13, attached in Appendix C. It is estimated that F.S. of any slip surfaces that intersect the roadway of Cameron Avenue would exceed 1.45.

The analysis results for the pre-failure geometry of the slope based on the shallower, perched groundwater conditions were generally consistent with those previously summarized for the deeper, groundwater condition. This indicates that potential instabilities associated with this slope are expected to be shallow and confined to the upper, surficial deposits for both groundwater conditions modelled in the analysis.

6.3.2 Existing (Post Failure) Slope Conditions

The slope stability analysis for the existing condition of the North Slope, based on its post-failure geometry, generally indicates that the steepened headscarp area located at the upper portion of the slope and the colluvium (landslide debris) previously deposited at the lower portion of this slope are susceptible to localized, sloughing/unravelling of loose, surficial materials as slip surfaces having F.S. of less than 1.0 were confined to these portions of the slopes. These localized slip surfaces are shown in Figures 14 and 15 of Appendix C for the headscarp area and colluvium, respectively.

The slip surfaces which correlate to F.S. between 1.0 and 1.5 for the post-failure slope conditions were generally noted to be consistent with those previously described for the pre-failure condition. However there is a slight reduction in the F.S. of the slip surface which intersects the location of the BC Hydro kiosk to 1.40 from 1.45, as was estimated for the pre-failure condition.

Based on the previously summarized slope stability analyses results, it is envisaged that shallow slope failures extending to the location of the BC Hydro kiosk and beyond are expected to be unlikely based on the current geometry and condition of this slope. However, the F.S. of this slope would be reduced if the lower portion of the slope is steepened (i.e. due to possible tidal erosion) as further discussed in Section 7.1 below. Furthermore, slope instabilities encompassing localized and shallow failures through the surficial, loose soils may occur at this slope based on the results of the analyses.

7.0 DISCUSSIONS

7.1 North Slope Failure Mechanisms and Risks

As previously summarized, the November, 2021 slope failure that was triggered at the North Slope was in the form of a shallow landslide through the surficial, loose sand deposits mantling the subject slope. This landslide occurred following a period significant rainfall. Based on the exposed slope failure surface, inferred stratigraphic profile of the slopes, slope stability results and the weather conditions preceding the failure, it is hypothesized that the failure of the North Slope could be attributed to a combination of the following slope and site conditions:

- Presence of loose, surficial slope deposits, observed to comprise brown, silty sand with some gravel where exposed, at the subject slope, and/or
- Overly steep slope geometries with respect to the shear strengths of the aforementioned surficial slope deposits, and/or
- Inflow and infiltration of surface water (i.e. runoff from the heavy rainfall event preceding the November, 2021 failure), towards the landslide area resulting in elevated perched groundwater levels within the slope and reduction in shear strengths of the surficial slope materials.

No additional obvious indicator signs of slope deterioration and instability have been observed at the subject slope throughout Ram's monitoring period. However, it is inferred that this slope could be subject to further instabilities in its current condition in consideration of the existing geometry, the presence of loose, surficial slope materials and tidal erosion at the toe of the slope, which may contribute to development over-steepened geometries at its lower portion and further destabilization. As shown in Cross Section A-A of Figure 7, this slope is generally inferred to be underlain by very dense/hard, natural materials and in turn, by very weak to weak siltstone based on the subsurface conditions encountered at the sonic and WildCat test hole locations. Based on the relatively high shear strengths of these materials, it is envisaged that the subject slope is not susceptible to global failures, occuring along deep-seated slip surfaces. However, based on our analyses and due to the presence of the loose, surficial slope materials and the relatively steep geometry of this slope, shallow and localized failures confined to the upper, surficial deposits, potentially triggered by extreme weather events or erosion at the toe of the slope, are expected to be a likely failure mode.

It is inferred that the landscaping area located at the top of this slope is underlain by up to approximately 2.0 metres of loose surficial soils based on the subsurface conditions encountered at test holes WC23-04 and WC23-05. Based on the observed materials exposed at the headscarp area of the North Slope, it is possible that these loose soils are sandy in composition. The utility locate carried out prior to the subsurface investigations also indicated that several underground utilities are located below the landscaping area situated at the top of this slope. These utilities include three BC Hydro distribution lines as well as a gas line and an electrical line. The approximate alignments of these and other utilities that were identified to be located below the assessment area are shown in the site plan provided in the utility locate report, dated January 24, 2023 and prepared by FJM Utility Locating. This report is attached in Appendix D for reference. The gas line was inferred to be a lateral connection that services the property of 1509 Dunbar Street whereas the electrical line was found to service an existing light post that is mounted atop a landing pad of the beach access staircase. The segments of the aforementioned gas and electrical lines that traverse below the landscaping area were found to be oriented sub-parallel and perpendicular to the crest of the North slope, respectively. It should be noted that the segment of the electrical line that extends to the east of the landscaping area and towards the light post is affixed to the north sides of the staircase landing pads. As shown in the site plan provided in the utility locate report, the segments of the three distribution lines that extend below the landscaping area are located to the west of the aforementioned electrical and gas lines. Two of these distribution lines extend from a vault located within the east boulevard of Dunbar Street and traverse below the lawn area and pedestrian pathway located near the crest of the South Slope to tie into the aforementioned BC Hydro kiosk. The third distribution line was inferred to extend from this kiosk to the property of 3624 Dunbar Street, located to the south of the landscaping area. The depths of the previously described utilities were not confirmed during the utility locate.

The loose soils conditions and the backfill materials previously placed within the trenchlines of the aforementioned utilities are envisaged to have greater hydraulic conductivities (i.e. permeabilities) than the underlying natural, very dense/hard materials. These natural, very dense/hard materials underlying the surficial loose materials are inferred to comprise silt and sand based on the subsurface materials encountered at SH23-01 and are expected to have relatively low hydraulic conductivities. Therefore, it is possible that surface water could infiltrate into the surficial soils during rainfall events and perch on the interface between the surficial soils and underlying hard/very dense natural materials. This could result in elevated perched groundwater levels during and immediately following significant rainfall events. For the landscaping area located above the slope, it is envisaged that the direction of seepage of the perched groundwater would generally follow the local topography; thus, perched groundwater would be preferentially directed towards the North Slope, along with surface water. Therefore, it is possible that the existing subsurface conditions underlying the landscaping area and the existing site grades promoted concentrated inflows of perched groundwater and surface water towards the slope during and immediately following the rainfall event which preceded the November, 2021, slope failure. The water conveyed towards the slope could have resulted in a reduction in shear strengths of the loose, surficial slope materials and in turn, contributed to the destabilization of these materials and the triggering of the landslide.

It is envisaged that tidal erosion at the toe of this slope during periods of high tide could also result in the development of oversteepened geometries at the lower portions of this slope. This oversteepening of the slope geometry could potentially result in a series of retrogressive slope failures which would be characterized by the formation of a series of new, localized failure surfaces upslope from the eroded and steepened portions of the slope. Thus, it is possible that retrogressive failure of this slope may impact the existing landscaping area situated above this slope. As it is expected that steepening of the lower portion of this slope could induce further slope instabilities, it is recommended that the landslide debris that has deposited at the lower portion of the slope be maintained for the proposed slope remediation works as this debris deposit is currently serving to buttress this slope and to temporarily protect it from tidal erosion.

It should be noted that raveling of the slope materials and localized slide failures associated with this slope may impact the beach front area depending on the extent of the slope failures. Accordingly, these slope failures could potentially pose a hazard to the public accessing the beach area and could also have environmental impacts on the habitat at the beach front.
At the time of preparation of this report, it is understood that the COV does not have an adopted level of landslide risk or accepted level of F.S. with respect to slope stability considerations. However, the estimated F.S. values for the slopes, as previously summarized in Section 6.0, could be compared to those presented in Table 6.2b of the BC Ministry of Transportation and

Infrastructure (BC MOTI) document titled "Bridge Standards and Procedures Manual – Volume 1 Supplement to Canadian Highway Bridge Design Code (CHBDC) S6:19" in order to evaluate the adequacy of the estimated F.S. values with respect to criteria set forth by provincial jurisdiction. The F.S. values specified in this table are for the global stability of embankments, geotechnical systems and natural slopes that affect the performance of highways or bridges. These F.S. values have been specified for varying degrees of understanding and consequence factors ranging from "low" to "high" for both permanent and temporary slope stability.

Based on our understanding of the subsurface conditions underlying the subject slopes and the method of analyses that was implemented to evaluate the F.S. of the slopes, the degree of understanding of the site would be considered to be "typical" for the purposes of this assessment. It is envisaged that the areas situated below the slopes (i.e. the beach front) and above, which includes the landscaping areas located at the termini of Dunbar Street and Cameron Avenue, as well as the pedestrian pathway that extends between these two streets, are typically accessed by the public for relatively limited durations. Furthermore, access to the beach front is generally restricted to periods of low to mid tide and it is expected that weather conditions would also influence public access to this area. Specifically, usage of the beach area would likely be less desirable to the public during periods of precipitation, and this also coincides with periods during which there would be a higher likelihood of slope instability/movement. Accordingly, the likelihood of any global failures of the slopes impacting life safety would be relatively remote in consideration of the low probability that members of the public would be present within the assessment area or the area of impact in the event of a landslide. Thus, it is judged that a F.S. value of 1.34, as specified for a "typical" degree of understanding in the aforementioned table of the CHBDC, and a consequence factor of "low" for permanent conditions, would generally be considered an appropriate target F.S. for global stability of the subject slopes for the purposes of this assessment. We note, that based on the usage of the beach front, the assessment area does not strictly meet the criteria set forth in Section 6.5.1. of the CHBDC for a consequence classification of "low" as this classification is indicated in this document to be suitable for geotechnical systems that are designed for applications where life safety is not a concern. However, in conjunction with appropriate signage and as previously discussed, the probability of any global instabilities of the slopes impacting life safety is expected to be relatively low and thus, we consider this consequence classification to be suitable for the subject site for the purposes of evaluating the global F.S. of the slopes based on the CHBDC criteria.

The estimated F.S. values for the global slip surfaces that intersect the BC Hydro Kiosk and the roadway of Cameron Avenue, located to the west, were generally found to exceed the F.S. value of 1.34. Thus, the F.S. against global failure of this slope could be considered to be sufficient if compared to the values prescribed in the aforementioned document. However, as previously discussed, the F.S. of the subject slope could be expected to be reduced if tidal-induced erosion of the slope were to result in retrogressive failure of the slope and steepened slope geometries.

Based on the results of the slope stability analyses that was completed for the existing conditions of the North Slope, it is estimated that the critical slip surfaces of this slope do not intersect the aforementioned gas and distribution lines. As the electrical line which services the light post is affixed to the staircase landing pads, it is envisaged that any surficial slope failures that may intersect this line would not necessarily compromise its stability. Accordingly, it is envisaged that the stability of each of these utilities is not compromised by the existing condition of the slope. However, retrogressive slope failures that could be induced by the gradual erosion of the lower portion of this slope could potentially impact all of these utilities.

7.2 South Slope Failure Mechanisms and Risks

It is envisaged that the lower portion of the South Slope is susceptible to spalling-type failures, as evident from observations made during our April 21st, 2023 slope monitoring site visit. This slope failure mode is related to the presence of the weathered, discontinuities observed at the siltstone/sandstone outcrop that forms this portion of the subject slope as these weathered planes would inherently have lower shear strengths in comparison to the intact rock mass between these discontinuities. This is consistent with our slope monitoring observations, as this portion of the slope was noted to have spalled along a plane that exhibited a dark brown colouration, which was inferred to be an existing weathered plane of discontinuity within the bedrock. Tidal action would also contribute to the destabilization of this lower portion of the slope by elevating water pressures within the discontinuities and further weakening the existing weathered planes. In its current condition, as observed during our April 21st and May 24, 2023 site visits, the lower portion of this slope which is currently exhibiting an overhanging geometry is expected to be prone to further instabilities and currently poses a rock-fall hazard to the public accessing the beach. Similar to the North Slope, it is envisaged that the South Slope is also susceptible to gradual, retrogressive failure if tidal erosion of this slope results in the development of further, unstable slope geometries as this could trigger localized failure surfaces upslope from the eroded or failed downslope portions of the slope. Furthermore, expansion of the existing cavities located at the lower portion of this slope (i.e. due to tidal erosion), could also result in destabilization of this slope.

Based on the relatively steep geometry of the South Slope, it is expected that this slope is currently susceptible to surficial slope movements associated with raveling of the variable fill materials overlying this slope. Upslope instabilities above the area that had been subjected to spalling have note yet occurred but should be expected. These forms of slope movement can also pose a hazard to the public accessing the beach front as well as a potential environmental hazard to the beach front area, depending on the contents of the existing fill materials. However, the vegetative cover of this slope is expected to be providing some root mass cohesion of the surficial slope materials and contributing to its resistance against shallow slope failures. Accordingly, this vegetative cover should be retained as part of the slope remediation works. As summarized in Table 4 of the Tree Risk and Windthrow Assessment report for this project, dated May 29, 2024, prepared by B.A. Blackwell & Associates Ltd, the vegetation cover of this slope should be retained for the slope remediation works, where possible. However, individual trees may have to be removed to accommodate the construction of the proposed slope remediation systems.

The range of estimated F.S. for the global slip surfaces of the South Slope that intersect the roadway of Dunbar Street were generally found to be similar to the F.S. value specified for slopes with a "typical degree of understanding" and "low" consequences of failure in Table 6.2b of the aforementioned document. Thus, the F.S. against global failure of this slope could also be considered to be acceptable if compared to the values prescribed in the aforementioned document. As previously summarized in Section 6.2, the lower range of soil cohesion values used in the sensitivity analyses for the South Slope yielded critical F.S. values of less than 1.3 for the subject slope. However, the slip surface associated with these F.S. values is not expected to intersect the roadway of Dunbar Street. Specifically, this slip surface was found to be confined to the portion of the subject slope that approximately intercepts extends from the fenceline, situated near the crest of this slope, to the bedrock exposure located at the lower portion of this slope. Accordingly, the range of critical F.S. summarized in Table 4 for the subject slope could be considered to be acceptable based on the consequences of failure of this slope and provided that suitable risk management measures are implemented.

As indicated in the site plan provided in the utility locate report, there are two BC Hydro distribution lines and a telecommunications line that traverse below the pedestrian walkway and lawn area located at the top of the subject slope. However, based on the results of the slope reconnaissance and stability analyses, it is envisaged that these utilities are not adversely impacted by the current conditions of this slope.

In consideration of the above, the slope remediation options should address the above-noted risks related to potential spalling of this slope and the potential for raveling of its loose, surficial materials.

7.3 Beach Access Staircase

It is inferred that the beach access staircase is founded on loose/soft subgrade materials of unknown composition based on our findings from the soil probe tests advanced at the portions of the slope located adjacent to select landing pad foundations. In consideration of these findings and our observations of the existing structural conditions of the staircase, it is inferred that the observed distortion of the staircase has been induced by differential settlement of the loose/soft subgrade materials supporting the staircase foundations.

In its current condition, future potential slope movements which encompass the subgrades of the staircase foundations could compromise its stability. Furthermore, as the staircase is inferred to be founded on loose/soft subgrade, ongoing settlement, including differential, of this staircase should be expected. This ongoing settlement is envisaged to adversely impact the performance of the staircase and could present potential hazards (i.e. tripping hazards) for the public accessing the staircase.

7.4 Risks to Nearby Residence

Based on our observations and information provided by VanMap, it is estimated that the footprint of the existing single-family dwelling at s.22(1) [redacted] is set back from the crest of the South Slope by a distance of at least 10 metres, as measured between the slope crest and the closest edge of this building. It is further noted that that the section of the South Slope that fronts this property comprises gradients of approximately 1V:1H and a maximum height of approximately 9 to 10 metres. Thus, this dwelling is estimated to be seated below a 1V:2H plane projected up from the toe of the slope. Based on an infinite slope model that accounts for the effects of groundwater seepage on the shear-strength along a potential failure plane, it is estimated that the aforementioned setback of this dwelling from the crest of the south slope is sufficient such that the critical slip surfaces of this slope would not intersect its building footprint under static-loading conditions. Accordingly, we do not expect the stability of this dwelling to be adversely impacted by the existing slope conditions. However, if an assessment of the F.S. associated with any potential slip surfaces that intersect the house is required, then a more detailed slope stability analyses should be completed and this analyses may require a review of the architectural and structural drawings and details related to this dwelling.

s.22(1) [redacted]

8.0 SLOPE REMEDIATION OPTIONS

Based on our review of background information, the findings from the field investigations and the inferred failure mechanisms of the slopes, it is envisaged that suitable slope remediation options for the subject site should address and mitigate the following risks and hazards:

- Existing steep slope geometries and presence of loose, surficial deposits overlying both the North and South slopes, rendering these slopes susceptible to shallow slope instabilities such as the ravelling of loose materials,
- Infiltration of rainwater and potential concentration of perched groundwater flows directed towards the North Slope (i.e. from heavy rainfall), resulting in the development of destabilizing forces within this slope,
- Tidal-induced erosion at the lower portions of both the North and South Slopes which could potentially result in gradual, retrogressive failure of these slopes,
- Potential rock-fall hazards associated with the development of overhanging geometries at the South Slope.

The F.S. for the global stability of the slopes, based on the current conditions, were generally found to be satisfactory when compared to the F.S. values prescribed in the aforementioned BC MOTI document for slopes with a "typical" degree of understanding and "low" consequences of failure. Thus, it is envisaged that the slope remediation options provided below would serve to manage the above-noted risks. However, slope remediation options that could be designed to improve the global stability of the slopes have also been included below, if F.S. values higher than those estimated for the current global stability of the slopes are desired by the project team.

The proposed slope remediation options are provided below. As requested by the COV, options to maintain the slopes in their existing conditions have also been included for discussion.

8.1 North Slope Remedial Options

8.1.1 Maintain Existing Slope Conditions

If the existing conditions of the North Slope were to be maintained, it is envisaged that the slope would be subject to the risks and hazards previously discussed in Section 7.1 regarding surficial slope movements and tidal erosion which could be expected induce potential retrogressive slope failures. Accordingly, it is envisaged that this would pose a hazard to the public accessing the beach as well as a potential environmental hazard to the habitat of the beach and intertidal zone. In consideration of these hazards, signage could be installed within the vicinity of the assessment area to educate the public regarding these potential hazards and discourage public access to the beach area subject to the City's risk assessment and policy. The signage could also inform the public that the risk of slope/soil movement is greater during/following periods of intense precipitation, freeze-thaw and king-tide events. The risk to the environment should be assessed by a suitably qualified professional.

Based on the current slope geometry and the results of the slope stability analyses, it is envisaged that the slope is not susceptible to deep-seated global slope failures and thus, the existing BC Hydro kiosk located within the landscaping area and the roadway of Cameron Avenue are not expected to be impacted by any potential instabilities associated with this slope under static-loading conditions for the foreseeable future. However, overtime, if tidal erosion of the lower portion of this slope were to result in the development of over steepened slope geometries, gradual retrogressive failure of the slope may result in slip surfaces impacting the landscaping area which may in turn comprise any existing utilities/infrastructure (i.e. the gas line and BC Hydro kiosk) located within this area; climate change could exacerbate this risk.

In consideration of the above, if the COV prefers to maintain the subject slope in its current conditions, then an extensive, long-term monitoring plan could be developed and carried out by Qualified Professionals to document changes to the existing slope conditions or any observations of slope deterioration. This may include (but not be limited to) the following:

- Development of tension cracks at the crest of the slope or within the landscaping area located above this slope,
- Sloughing of the surficial slope materials,
- Zones of increased or concentrated seepage at the face of the slope,
- Deposition of newly sloughed soil/debris at the bottom of the slope,
- Erosion/undermining at the toe of the slope
- Erosion of the existing landslide debris previously deposited at the bottom of the slope, and/or
- Changes in geometry of the slope (may require installation of survey monitoring stakes/points along the existing profile of the slope).

If any of the above are observed, then additional recommendations to mitigate risks and hazards to the public and adjacent infrastructure may be required depending on the extent of the observed slope deterioration.

The expected advantages and disadvantages of maintaining this slope in its current condition include the following:

<u>Advantages</u>

- Envisaged to be the least expensive option to implement,
- Minimal disturbance to the existing vegetation.

<u>Disadvantages</u>

- Will require extensive, long-term slope monitoring plan and more frequent monitoring of the slope(s) by qualified personnel,
- May require implementation of more frequent slope maintenance measures such as scaling of loosened materials,
- May be difficult to enforce pedestrian setback at beach area,
- Further erosion at the lower portion of the slope will result in increased potential for retrogressive failure of the slope,
- Possible environmental impacts on the beach front from further slope failures,
- Potential slip surfaces may undermine staircase foundations,
- Further instabilities, comprising mainly surficial/localized movements, can be expected which would pose a potential hazard to the public.

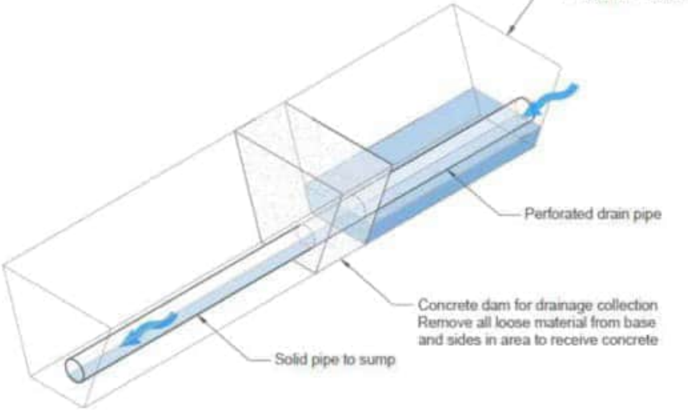
8.1.2 Re-grading of Slope and Installation of Rip-Rap and Interception Drain

Based on the existing site conditions, a suitable remediation option for the North Slope could entail a combination of partial re-grading of the existing slope geometry and placement of rip-rap. Specifically, the upper portion of the slope could be re-graded to a gentler gradient in order to mitigate risks of shallow failures and the lower portion of the slope could be armored with rip-rap to provide protection against tidal erosion. The rip-rap armoring would also serve to buttress the slope and improve its stability. Suitable sizes, gradation and geometries of the rip-rap armoring for the proposed application are expected to be largely dependent on the tidal characteristics of English Bay (e.g. tidal currents and design wave heights), including for the effects of climate change. These details should be determined during the design phase of the project and it is envisaged that input from a hydraulic, hydrotechnical or coastal engineer may be required to inform the design of the rip-rap system. The portion of the slope that will be armored with rip-rap should also be lined with a layer of non-woven geotextile, to be placed between the slope and rip-rap, to minimize the risk of migration of slope materials into the rip-rap and erosion of the slope. As previously discussed, it is recommended that the landslide debris which had deposited at the lower portion of the slope be maintained as this debris is currently buttressing the slope and excavation carried out at the lower portion of the slope could potentially induce further slope instabilities.

If the lower portion of this slope is armored, re-grading of the balance of the slope to a maximum gradient of approximately 1V:2H would reduce the potential risks of surficial slope failures. It should be noted that the extent of slope re-grading would also be constrained (i.e. with respect to equipment access and re-grading geometries) by the existing utilities located within the landscaping area, the adjacent residential development to the north and the beach access staircase to the south. Accordingly, any modifications to the slope geometry will have to accommodate these constraints unless they are relocated. Alternatively, as discussed in Section 8.2 of this report, a type of earth retention system could be installed adjacent to and below the staircase foundations to provide lateral confinement of the subgrade supporting these foundations. The upper, re-graded portion of the slope should be vegetated upon completion of the earthworks as this would further enhance slope stability and reduce erosion. It may be difficult to retain the existing staircase in its current condition if this option were to be undertaken depending on the final geometry of the re-graded slope. Thus, replacement of the staircase may be required.

An interceptor trench, incorporating subdrains and seepage collars, should be installed sub-parallel to and adjacent to the crest of this slope in order to manage the concentration of near surface groundwater towards the slope. It is envisaged that the interceptor trench should be installed to the east of the existing underground gas line which traverses the landscaping area situated above this slope. The interceptor trench should be a minimum 0.6 metre wide and extend a minimum 450 millimetres below the interface of the surficial loose soils and the underlying, natural, low permeability, very dense/hard stratum in order to suitably intercept perched groundwater and surface water flows. Based on the results of the WildCat soundings, WC23-04 and WC23-05, it is expected that this would correspond to depths of approximately 2.5 metres below the current site grades at the landscaping area. The interceptor trench should be installed with a subdrain comprising a nominal 100 millimeter diameter, PVC pipe. It is envisaged that the segment of the subdrain fronting the slope should be perforated in order to intercept water flowing downslope from the landscaping area. The invert of this subdrain pipe should be set at an elevation of approximately 150 millimetres above the trench invert and be suitably sloped to direct water away from the North Slope. This perforated segment of the subdrain pipe should be bedded on and covered with 19 millimetre clear-crushed gravel. The seepage collar for this interceptor trench should be installed at the transition between the perforated and solid segments of the PVC subdrain pipe. The seepage collar for the interceptor trench could comprise controlled density fill (i.e. low strength, unreinforced concrete) poured between the subdrain and the surrounding, natural, undisturbed, low permeability soil. Water intercepted by this interceptor trench system will have to be diverted and discharged to a suitable location, possibly the beach front. Depending on the subsurface conditions encountered within the interceptor trench, it may be recommended to line the upslope sidewall of the interceptor trench with a layer of non-woven filter fabric to mitigate the migration of fines into the gravel of the trench.

A conceptual detail of the interceptor trench and seepage collar is provided in Schematic 1 below. It should be noted that the depths of the proposed interceptor trench and subdrains, as described above, could be subject to modifications depending on the actual subsurface conditions encountered during installation if this option is to be incorporated into the slope remediation works.



Schematic 1: Proposed Interceptor Trench and Seepage Collar Detail (reference: Builder Guide to Site and Foundation Drainage Best Practices for Part 9 Buildings in British Columbia)

At this time, it is envisaged that the advantages and disadvantages associated with this slope remediation option could comprise the following:

Advantages:

- Installation of rip-rap at lower portion of the slope provides protection of this slope against tidal-induced erosion. This would reduce the risks of retrogressive slope failures,
- Removal of loose, surficial slope materials and re-grading of slope geometry would mitigate the risks of shallow-slope instabilities to which the subject slope is susceptible in its current condition,
- Lowering the elevation of the perched groundwater table and controlling the ingress of surface with the interceptor trench would contribute to slope stability,
- This slope remediation option is considered to be relatively simple to construct and expected to be less expensive than the other options described herein,
- Rip-rap could be configured to enhance habitat for some species,
- Design of this slope remediation option expected to be relatively simple with respect to technical requirements,

- Re-grading of the slope could improve F.S. for global stability, and
- The re-graded portion of the slope could be re-vegetated.

Disadvantages:

- May require significant earthworks,
- Extents of re-sloping at the upper portions of the slope would be constrained by the adjacent residential property to the north, and staircase to the south (unless removed) and some utilities. Accordingly, any modifications to the slope geometries will have to accommodate these constraints,
- Water intercepted by the proposed groundwater management system will have to be conveyed and discharged in a controlled manner to a suitable location (i.e. possibly towards the beach area),
- Periodic maintenance of upper portion of slope may be required due to possible erosion,
- Maintenance of the interceptor trench required to ensure that silt, and other debris does not clog the trench in a manner that could impair its performance,
- May be difficult to advance the excavation for the interceptor trench due to the presence of existing underground utilities within the landscaping area,
- Will require construction equipment access to beach front which may be difficult due to existing site conditions and constraints,
- Possible significant loss of vegetation during construction,
- Removal of existing trees located at the lower portion of the slope will be required to accommodate placement of rip-rap,
- As indicated in the Tree Risk and Windthrow Assessment report, installation of interceptor trench within the drip lines of trees could have potential impacts on tree health, thus, consultation with arborist will be required to determine a suitable trench location,
- Permitting with multiple jurisdictions envisaged to be required,
- Possible re-routing of existing utilities located below the landscaping area may be required if the existing alignments conflict with the proposed geometry of re-graded slope,
- Coastal engineering consultant will have to be retained to provide input regarding rip-rap
- Environmental assessment expected to be required, and

s.18 [redacted]

It is envisaged that the placement of the rip-rap armoring and re-grading of the subject slope, as described above, are feasible from a constructability perspective based on the existing site conditions. However, it is expected that the equipment (i.e. excavators) and material required to undertake this slope remediation option may have to be transported to site via barging as the beach front is not envisaged to be accessible with typical construction equipment (i.e. excavators and dump trucks) from inland. Alternatively, access to the beach front with smaller excavators may be possible if the hedge located at the top of the North Slope could be temporarily removed and a temporary ramp is constructed with fill materials placed atop the North Slope to facilitate this access. It is also expected that the construction of this slope remediation option will have to be scheduled to be completed during periods of low-tide. Thus, this condition would restrict construction to a specific period of the day and tide schedule which could potentially introduce additional challenges to the construction schedule. As previously indicated, the existing utilities located below the landscaping area at the top of the North Slope may be required to be re-routed or temporarily decommissioned to facilitate these works. The utilities may include the gas line that services the property of 1509 Dunbar Street, and the electrical line that extends from the BC Hydro kiosk to the existing street light mounted atop a landing pad of the staircase. We expect that this slope remediation option would be the simplest interventive option to construct and would require the shortest construction timeline out of the options proposed herein.

If re-grading of the slope is not preferred by the project team for this option (i.e. only placement of rip rap and installation of a interceptor trench is to be undertaken), then it should be understood that the subject slope could potentially still be susceptible to surficial and localized slope movements due to the presence of loose slope materials and steepened geometries. The risks that these remaining hazards could pose to the public could be managed by the installation of signage to deter public access within a designated setback from the toe of this slope.

8.1.3 Construction of Lower Buttress and Re-grading of Upper Portion of Slope

A type of buttress, such as a Mechanically Stabilized Earth (MSE) wall, could be constructed to stabilize the lower portion of the North Slope and the balance of the slope could be re-graded to a gentler gradient (i.e. 1V:2H), as previously described. It is envisaged that this option would improve both local and global stability of the slope and also provide protection against tidal erosion. As the buttress will be partially submerged below the waters of English Bay during periods of high tide, the effects of these submerged conditions must be considered in the design of the MSE buttress. It is also envisaged that input from a coastal, hydrotechnical or hydraulic engineer may be required to inform the design of this MSE buttress.

A MSE wall utilizes a type of wall facing with geogrids incorporated into the backfill zone and at a specified vertical spacing and length, to reinforce the soil mass behind the wall facing. The reinforced soil mass, in conjunction with the wall facing, forms the MSE wall/buttress that will stabilize the slope and resist any translational and overturning forces that would be induced by potential movements of the slope. It is envisaged that suitable types of wall facings for the proposed application could comprise Lock-Blocks or Gabion-baskets. The backfill materials to be placed in the reinforced zone between the wall facing and the existing slope should consist of suitably compacted Engineered Fill. Within the context of this report, Engineered Fill should consist of select, clean, free draining, well-graded granular material with less than 5% fines content by mass and 100% passing a 75 millimeter sieve. However, as further described below, the effects that submerged conditions would have on the reinforced backfill zone of the MSE buttress should also be considered in the selection of suitable fill materials.

Depending on the extents of slope re-grading, it is expected that the MSE buttress would extend up to approximately the mid-slope portion of this slope. This would require a MSE wall height of approximately 4 to 5 metres. If required or preferred, the MSE wall buttress could also be designed and constructed with a tiered configuration to satisfy site, design, access or aesthetic constraints or to allow habitat development. The facing for the MSE buttress should be sufficiently embedded below the potential scour depth of the beach front to reduce risks of undermining of the MSE wall due to tidal erosion of its subgrade. The length of the reinforced backfill zone of the MSE wall will have to be determined during the detailed design phase of the project. This length can typically be taken to be a minimum of 80% of the design wall height for global stability considerations. However, as the MSE structure for this site will be partially submerged below a body of water during periods of high tide, it is envisaged that geogrid lengths exceeding these typical lengths will be required to account for the reduction in pullout capacity of the geogrids which would be induced by the buoyant forces imposed on the structure under submerged conditions. This longer reinforced backfill zone would also be required to offset the potential loss in interface friction between the geogrid and backfill under submerged and saturated conditions. However, it is envisaged that this loss in frictional resistance of the backfill could also be reduced or mitigated if a suitably sized clear gravel is used as backfill for the portion of the MSE wall that will be submerged below the high-water level.

Suitable drainage features (i.e. subdrains and drainage blankets/chimneys) should also be incorporated within the backfill zone of the MSE retaining wall in order to divert water away from the wall and to minimize the risk that the wall is subjected to a build-up of hydrostatic pressures. It is envisaged that a Gabion wall may require less extensive drainage features as this type of facing is inherently porous and would facilitate drainage, although its susceptibility to corrosion and damage from debris impact would need to be considered.

The drainage and stability of the re-graded portion of the slope that will be exposed above the buttress would be enhanced if the previously described interceptor trench, with seepage collars, were to be installed near its crest.

The expected advantages and disadvantages associated with this slope remediation option are summarized below. The advantages and disadvantages associated with each type of wall facing are not described, however, this information could also be provided to inform the selection of a suitable facing if the design and construction of a buttress is to be pursued.

Advantages:

- Proposed buttress system could also serve as a rock/debris fall catchment area which would reduce the risk of sloughed, raveled slope materials impacting the beach front,
- Proposed buttress system could be designed to meet specified factors of safety against global/local slope instability,
- Proposed buttress system could provide protection of slope against tidal action and erosion-induced instability,
- Where feasible, removal of loose, surficial slope materials and re-grading of slope geometry above the proposed buttress would reduce the risks of shallow-slope instabilities to which that the subject slope is susceptible in its current condition,
- Lowering the elevation of the perched groundwater table and controlling the ingress of surface water via the installation of the upslope interceptor trench would contribute to slope stability, and
- The re-graded portion of the slope could be re-vegetated.

Disadvantages:

- Extensive engineering design and analysis will be required as the design of the buttress will have to account for the effects of being partially submerged below Burrard Inlet during high tides,
- Will require significant earthworks and import of MSE wall backfill materials,
- Construction of the buttress will require the most significant encroachment onto the beach front,
- Will require mobilization of construction equipment to beach front which may be difficult due to existing site conditions and constraints,
- Extent of re-sloping at the upper portions of the slopes is constrained by the adjacent residential property to the north, underground utilties and staircase to the south. Accordingly,

any modifications to the slope geometries will have to accommodate these constraints and mitigate the risks of negatively impacting the stability of the stairs and the adjacent property,
- Water intercepted by the proposed groundwater management system of the MSE buttress and interceptor trench will have to be conveyed and discharged to a suitable location (possibly towards the beach area),
- Periodic maintenance of the upper portion of slope may be required,
- Permitting with multiple jurisdictions is envisaged to be required,
- Significant loss of existing vegetation expected during construction, including removal of existing trees located at the lower portion of the slope,
- Environmental impacts associated with the construction of a proposed buttress on the beach front and shoreline habitat will have to be assessed by a suitably Qualified Professional,
- Coastal engineering consultant will have to be retained to provide input regarding scour depths, wave heights, current velocities, debris impact forces and climate change considerations
- s.18 [redacted]
- Hydrotechnical considerations of Burrard Inlet would have significant implications on buttress design, construction sequence, and
- May require some form of scour protection for the buttress.

Construction of the proposed MSE buttress is expected to be feasible based on the existing site conditions. However, the challenges associated with construction equipment access to the beach front, as described above in Section 8.1.2, are also expected to be relevant for this slope remediation option. Furthermore, tidal interaction with the work area during periods of high tide may pose challenges to construction and may warrant considerations for the construction sequence of the buttress and layout of the staging and work areas. It is envisaged that additional constructability considerations could be identified following the hydrotechnical assessment that should be carried out to inform the design of this slope remediation option.

8.1.4 Soil Nails and Mesh

Alternatively, a passive earth retention system, comprising soil nails and mesh could be installed to stabilize this slope. This slope remediation strategy would be suitable for improving both the shallow/surficial and global stability of the slope, if required/preferred by the COV based on the results of the slope stability analysis. Construction of this earth retention system would entail the installation of fully grouted soil nails, comprising threadbar anchors, at a design on-centre spacing, through the face of the slope and at a design downward inclination (typically 15 – 25 degrees below horizontal). The soil nails would be installed with lengths extending beyond the critical slip surfaces of the slope and embedded into the “passive” zone of the slope which is expected to comprise natural, competent, hard/very dense soil. Resistance to slope failure and deformation is provided by the segments of the soil nails embedded within the passive zone of the slope as tensile forces develop along these segments of the soil nails to resist slope movements. A flexible, high tensile strength mesh would be installed at the face of the reinforced slope and secured to the soil nails in order to provide resistance against surficial slope movement and erosion protection. Upwardly inclined sub-horizontal drains, comprising perforated PVC pipes, should be installed through the slope to facilitate drainage and to reduce hydrostatic pressures imposed on the earth retention system.

It should be noted that this slope remediation option is envisaged to be the most expensive to construct. The advantages and disadvantages associated with this slope remediation option include the following:

Advantages:

- Installation of mesh would be expected to significantly minimize surficial slope ravelling/instabilities and also serve as debris-fall protection system,
- The earth retention system could be designed to meet specified factors of safety for global/shallow slope stability if required by the COV,
- For aesthetic considerations, slope vegetation could grow through the mesh and conceal the earth retention system,
- Will require relatively less maintenance efforts in comparison to the other slope remediation options provided herein,
- Mesh could be installed around trees which are to be retained at the slope, and
- The lengths of the soil nails are expected to be relatively short for this slope based on the depths of the potential slip surfaces presented in Section 6.2.

Disadvantages:

- Will require significant engineering design and analysis as well as quality control during construction,
- Expected to be the most expensive option with respect to both engineering design and construction,
- May require lengthy duration of construction depending on design of soil nail system,
- Access to the slopes with the equipment to install the soil nails may be difficult due to existing site conditions and constraints. Unconventional/specialized equipment may have to be mobilized to install the anchors,
- As indicated in the Tree Risk and Windthrow Assessment report, this remediation option is expected to have significant impact to trees. Specifically, vegetation (i.e. trees) growing through the mesh could become problematic over time because openings would not be large enough to accommodate secondary growth. Thus, it is expected that all vegetation would have to be removed to accommodate the installation of the soil and mesh system,
- Tree growth through mesh will need to be managed (weeded) as regeneration occurs,
- May require some clearing and grubbing of slopes to facilitate installation of soil nails, and
- Permitting with multiple jurisdictions envisaged to be required.

It is envisaged that the installation of the proposed soil and mesh system is feasible for the North Slope based on the existing site conditions, however, the challenges posed by site constraints for equipment access, as previously described, are also relevant for this remediation option. The locations and depths of the existing utilities situated below the landscaping area located at the top of this slope and at the easternmost terminus of Cameron Avenue would also have to be considered in the design and installation of the soil nails in order to ensure that the soil nail locations are not in conflict with these utilities.

8.2 SOUTH SLOPE REMEDIATION OPTIONS

8.2.1 Maintain Current Slope Conditions with Scaling of Loose Materials

If preferred by the COV, it is envisaged that the current slope conditions can be maintained with the installation of signage to educate the public regarding the potential for rockfall hazards related to this slope subject to the City's risk assessment and policy. As previously recommended in our slope monitoring memorandum, dated May 10, 2023, a minimum setback distance of 3.0 metres should be maintained between pedestrians and the spalled/failed portion of this slope currently exhibiting overhanging geometries. A recommendation was also provided to install signage and snow fencing, to be affixed to the spalled portion of the bedrock exposure, to direct the attention of the public to this hazard.

If the existing slope conditions were to be maintained, it is recommended that the surficial, loose fill deposits of this slope be scaled in order to reduce risks of ravelling of these materials. It is envisaged that the scaling can be completed manually (i.e. by hand) and that disturbance to the existing vegetation overlying this slope will not be required. It is also recommended that the existing cavities located at the toe of this slope be backfilled with concrete to restore the undermined portion of the slope in order to reduce the rate of erosion-induced undermining of this slope. Slope monitoring should be undertaken by qualified personnel to assess the conditions of the slopes for any indicator signs of slope movement or deterioration which could include but not be limited to the following:

- Further spalling of the existing siltstone exposure located at the lower portion of the slope,
- Development of tension cracks at the crest of the slope,
- Expansion in width of the existing gap between the sidewalk located at the top of this slope and the 0.6 to 1.0 metre high retaining wall located below (refer to our slope monitoring memorandum, dated March 16, 2023 for more details pertaining to this observation),
- Zones of increased or concentrated seepage at the face of the slope, and/or
- Deposition of sloughed soil/debris at the bottom of the slope which could be indicative of ravelling of slope materials.

It is recommended that the monitoring of this slope be carried out at a frequency of once every month during the wetter months of October to February, as well as following any extreme precipitation events or reported slope failures in the general vicinity and once every two months during the drier season.

The advantages and disadvantages related to this option are summarized below:

Advantages

- Envisaged to be the least expensive option to implement,
- Minor reduction of risks of sloughing/movement of surficial, loose slope materials,
- Loose, surficial slope materials can be scaled by hand.
- Backfilling of existing cavities would restore the undermined portions of the slope and reduce the potential for and rate of slope erosion above, and
- Minimal disturbance to the existing vegetation.

Disadvantages

- Will require more frequent periodic monitoring of slope by qualified personnel,

- May require implementation of more frequent slope maintenance measures,
- May be difficult to enforce recommended pedestrian setback at beach area,
- Further erosion at the lower portion of the slope may result in development of overhanging geometries at other portions of this slope and potential for retrogressive failure of the slope, and
- Possible environmental impacts on the beach front due to sloughing of loose debris.

8.2.2 Placement of Rip-Rap

The lower portion of the South Slope could be armored with rip-rap to reduce potential and rate of further tidal-erosion and possible spalling failures. The recommendations for placement of rip-rap for this slope would generally be consistent with those previously provided for the North Slope remediation options. It is envisaged that re-grading of this slope is infeasible due to the current geometry of the slope and the existing constraints located at the crest of this slope (i.e. sidewalk, lawn area and roadway of Dunbar Street).

The recommendations previously provided in Section 8.2.1. pertaining to scaling of loose, surficial fill materials and backfilling of the existing cavities located at the toe of this slope are also applicable to this remediation option.

The advantages and disadvantages related to this slope remediation option are summarized below:

Advantages:

- Placement of rip-rap would provide protection of slope against tidal-induced erosion and reduce potential and rate of retrogressive slope failure,
- Backfilling of existing cavities would restore the undermined portions of the slope and reduce rate of further slope movement,
- Reduce risks of sloughing/movement of surficial, loose slope materials,
- Scaling of loose slope materials can be completed by hand,
- Not expected to require removal of trees/vegetation, and
- Rip-rap could be configured to enhance habitat for some species.

Disadvantages:

- Will require construction equipment access to the beach front which may be difficult due to existing site conditions and constraints,
- Placement of rip-rap is not expected to significantly improve global stability of the subject slope,
- Will require periodic monitoring of slope by qualified personnel and periodic slope maintenance,
- Removal of trees may be required to accommodate placement of rip-rap,
- Environmental assessment expected to be required,
- s.18 [redacted]
- Permitting with multiple jurisdictions expected to be required.

8.2.3 Soil Nails and Wire Mesh

The soil and mesh remediation option, previously discussed as a slope remediation option for the North Slope, could also be implemented for the South Slope to mitigate risks of surficial slope movement and rock-fall as well as to improve the global stability of the slope, if required, based on the estimated F.S. summarized in Table 4. The disadvantages and advantages previously summarized for this option would also be applicable to this slope.

We envisage that installation of the soil nails and mesh system at the south slope would also be feasible from a construction perspective; however, the installation of any soil nails required within the upper portion of the slope may be challenging due to the geometry of this slope and site constraints. Thus, unconventional drilling equipment, having sufficient reach, may be required to install the soils nails for this slope. As previously discussed in Section 8.1.4 for the North Slope, any utilities located below the areas located at the northern terminus of Dunbar Street will have to be considered in the design and construction of the soil nail wall system.

9.0 STAIRCASE REMEDIATION OPTIONS

As previously discussed, the distortion of the staircase is currently inferred to be settlement-induced based on the loose/soft soil conditions encountered at the soil probe test locations, which were advanced at the portions of the slope located adjacent to select staircase landing-pads. Furthermore, it is envisaged that the landing pad foundations supporting the staircase are potentially at risk of being undermined if future slope movements were to encompass the subgrade materials of these foundations.

Suitable staircase remediation to address the risks associated with settlement of the staircase and potential undermining of its foundations are provided below. As requested by the COV, an option to maintain the staircase in its current condition has also been included for discussion.

9.1 Maintain Current Condition of Staircase

If the staircase were to be maintained in its current conditions, further settlement of the staircase foundations may be expected as they are inferred to be founded on a loose/soft subgrade. Thus, the potential settlement mechanism would be compression of the loose/soft fill subgrade and potential sloughing of the subgrade materials due to localized instabilities of the slope materials located below the staircase foundations. Furthermore, decomposition of entrained organics within the subgrade would include volumetric changes of the soil, resulting in settlement of the foundations. This settlement should be expected to be differential and further adversely impact the performance of the staircase and pose a potential hazard (i.e. tripping) to the pedestrians. This could possibly be addressed by carrying out periodic slab-jacking of the staircase foundations to re-level the staircase. If it is preferred to mitigate excess settlement of the foundations, then an underpinning strategy should be employed to address the presence of the loose/compressible subgrade materials as described in Section 9.2 below. In consideration of the aforementioned risks associated with undermining of the staircase due to any future potential slope movements, it is recommended that some form of earth retention system be installed adjacent to and below the north edge of the staircase to provide lateral confinement of the subgrade supporting the staircase and to address the potential for sloughing of the subgrade materials supporting the staircase foundations. It is envisaged that a suitable earth retention system could comprise interlocked Flex MSE geomodular bags placed below and adjacent to the landing pad locations of the staircase. The suitable geometry and configuration of the earth retention system would be dependent on the remediation option pursued for the North Slope as modifications to the existing grades of this slope may be inherent.

If the staircase were to remain in its current condition, it is also recommended that survey monitoring of the staircase for any signs of movement be undertaken by a British Columbia Land Surveyor (BCLS). The survey monitoring results should be forwarded for review to the professional undertaking responsibility for the stability of the staircase. It is envisaged that the monitoring could be carried out at a frequency of once per month during periods of the year that are expected to experience heavier precipitation (i.e. October to February) and once every two months during the drier season. It is envisaged that a suitable monitoring period could be determined based on the observations from the visual monitoring that has been undertaken to date and the results from the first 4 months of the proposed surveying monitoring.

It is envisaged that the advantages and disadvantages associated with this option comprise the following:

<u>Advantages</u>

- Flex MSE system is relatively simple to construct and can be completed by hand,
- It is expected that the Flex MSE earth retention system could be constructed by the earthworks contractor that would be mobilized to site to carry out the remedial works for the slopes (if remediation is pursued),
- Reduces risks of undermining of the staircase foundation,
- Flex MSE geomodular bags are relatively inexpensive,
- Flex MSE bags can be vegetated, therefore concealing the presence of the earth retention system for aesthetic considerations, and
- Flex MSE bags are relatively light in weight and would not be expected to impact the stability of the slope that they will be placed upon (to be verified during detailed design).

<u>Disadvantages</u>

- As the staircase is inferred to be founded on loose soils, the foundations may be subjected to further settlement, including differential, which would adversely impact the performance of the staircase,
- The staircase may require periodic re-levelling (i.e. slab jacking) to remediate the impacts of settlement on the performance of the staircase (i.e. to mitigate tripping hazards to the public) if it were to remain accessible,
- Periodic survey monitoring of the staircase required for any additional indicator signs of structural distress/movement, as discussed above, and
- Possible periodic maintenance of Flex MSE earth retention system required,

We expect that the slab-jacking of the staircase foundations and construction of the FLEX MSE retention systems to provide for confinement of the foundation subgrades to be feasible from a constructability perspective as these remedial measures can be implemented without the use of typical construction equipment (i.e. excavators).

9.2 Underpinning of Staircase Foundations

If it is preferred to mitigate further settlement of the staircase as well as the risks of undermining due to slope instabilities, the foundations could be underpinned with fully-grouted, vertical threadbar anchors (i.e. micropiles), embedded into competent, very dense/hard subgrade. It is envisaged that these micropiles could be installed adjacent to the landing pad foundations, using hand-operated equipment. The anchors would have to be structurally connected to the existing land pad foundations. At this time, it is expected that a minimum of four underpinning anchors, to be installed at each corner of the landing pad foundations, would be required. The size of the threadbar anchors would be dependent on the structural loads imposed by the foundations and would be determined during the detailed design phase of the project.

In order to inform the detailed design of this option, a supplementary subsurface investigation utilizing hand operated equipment (i.e. WildCat Penetrometer Test Soundings) is recommended to delineate and confirm the depth to suitable bearing adjacent to the landing pad foundations.

Advantages

- Improved performance of staircase foundations with respect to settlement,
- Required threadbar lengths are expected to be relatively short based on the inferred subsurface conditions,
- Threadbar anchors could likely be installed using hand-operated, portable equipment,
- Potential for undermining of staircase foundations due to surficial slope movement would be mitigated,
- Reduced maintenance requirements of staircase,
- Battered anchors could be installed to provide resistance to lateral movements, if required,
- Improved long-term performance of staircase,
- Installation of underpinning anchors will not require significant disturbance to the existing vegetation of the slope, and

Disadvantages

- Would require collaboration with structural engineer to structurally connect existing foundations to the proposed threadbar underpinning anchors,
- Potential damage to tree roots should be considered in the selection of suitable anchor locations
- May require specialized sub-contractors to install dowels, and
- Permitting with multiple jurisdictions envisaged to be required.

The underpinning system, described above, is expected to be feasible from a constructability perspective as it is envisaged that the threadbar anchors could be installed with hand-operated equipment, negating the requirement for drill rig access to the existing staircase foundation locations.

9.3 Construction of New Staircase Foundations

Alternatively, the existing foundations of the staircase can be demolished and new foundations, bearing on a competent (i.e. very dense/hard) subgrade can be constructed. It is envisaged that this option will require significant earthworks as well as disturbance to the slope. However, this option would also serve to mitigate settlement of the foundations and the risk of undermining of these foundations due to surficial slope ravelling.