THURBER ENGINEERING LTD.

PENDER ISLAND – CANAL ROAD DIP SLIDE

GEOTECHNICAL DESIGN REPORT

Report

to

BC Ministry of Transportation and Infrastructure

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1. INTRODUCTION

Following the atmospheric river events of November 2021, the BC Ministry of Transportation and Infrastructure (MoTI) identified possible worsening of a known active landslide and is now proposing to stabilize this approximately 300 m segment of Canal Road on South Pender Island, BC.

At the request of MoTI, Thurber Engineering Ltd. (Thurber) has prepared this Geotechnical Design Report to aid the project team in the detailed design stage.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions. This work was completed under Contract Numbers 861CS1180 and 861CS1195.

2. BACKGROUND INFORMATION

2.1 Project Description

The Canal Road dip slide is located approximately 3 km from the Pender Canal bridge, immediately to the east of the Mt. Norman Access Road and bordering the Beaumont-Gulf Islands National Park Reserve. The road is located at the crest of a steep slope, with an average slope angle of about 40° down to ocean (north) and exposed bedrock or colluvium on the upslope (south) side. Crown land and private property are located downslope of the slide and Parks Canada (Mt Norman) on the upslope (south) side.

This section of roadway has been showing signs of movement for several years, noted by the existence of tension cracks along and across the roadway that have been observed by island residents and BC MoTI staff.

A field review carried out by MoTI staff on November 23, 2021, observed potential signs of worsened stability due to the recent atmospheric river rainfall events. Thurber was subsequently retained to carry out a field review / emergency call out to provide geotechnical engineering site review and recommendations for this area which was completed on November 25, 2021.

On December 1, 2021, the project transitioned into a Recovery phase, which included conceptual design of remedial measures for the landslide, and installation of monitoring instruments to facilitate quantitative review of the slide deformation behaviour.

Traffic is currently being diverted to the upslope lane in a single lane alternating fashion to minimize the exposure of travelers to the actively moving landslide. A risk management approach to load restrictions during and following periods of heavy rainfall is also in place. It is understood that these measures will likely remain in place until the recovery phase is complete.

The tension cracks at the northwest end of the slide are significant and extend diagonally across the entire roadway. We understand that the maintenance contractor has been filling these cracks with cold patch asphalt at the recommendation of the MoTI geotechnical team to reduce water infiltration.

2.2 Previous Work

MoTI conducted a geotechnical investigation of the site in 2020 to investigate the slide location and subsurface soil conditions. Five geotechnical test holes were drilled to depths up to 13.4 m within the roadway. One monitoring well was installed at TH20-03. Draft test hole logs were provided to us by MoTI. The locations of these test holes are included on the test hole location plan in Appendix A.

Thurber subsequently carried out geotechnical investigations, installed SAA monitoring equipment and prepared a Conceptual Design Report (issued May 18, 2022) which included discussion of potential rehabilitation options to address the roadway stability. Following review of the conceptual options, MoTI selected the realignment option (Concept 1) for detailed design. Details of the geotechnical investigations, instrumentation monitoring, options development and geotechnical design recommendations are discussed in the following sections of the report.

3. GEOTECHNICAL INVESTIGATION

Based on Thurber's Geotechnical Work Plan, dated January 19, 2022, and a Supplementary Investigation Plan, dated April 7, 2022, four holes have been drilled and the installation of three Shape Acceleration Arrays (SAA's), one standpipe piezometer and a datalogger have been completed on site.

3.1 Investigation Methodology

In accordance with Thurber's ground disturbance procedures, a BC One Call was initiated to obtain records of buried underground utilities in the vicinity of each borehole location. BC Hydro was contacted by the project team for the coordination of TH22-01 and TH22-02 which were located within 3 m of the existing overhead power lines.

BC Hydro confirmed that the lines would need to be shut off during the initial investigation and advance notice was given to residents of South Pender Island by BC Hydro for the drilling dates. Power was shut down during daytime hours to accommodate the drilling schedule for TH22-01 and TH22-02. Test holes TH22-03 and TH22-04 were located away from the existing powerlines and no shut downs were required for the supplementary investigation.

TH22-01 and TH22-02 are located within the slide area in the westbound lane of Canal Road. TH22-03 is located on the east end of the project just outside of the active slide area within the westbound lane. TH22-04 is located within the eastbound lane, upslope of the presumed active slide area.

All test holes were drilled using a track mounted sonic rig operated by Drillwell Enterprises of Duncan, BC. Soils were logged and sampled by a Thurber representative in the field. A 50 mm PVC pipe was installed in TH22-01, TH22-03 and TH22-04 with a flush mount road box to allow for installation of the SAA's. A 50 mm diameter standpipe piezometer was installed at TH22-03 to allow for groundwater monitoring and completed with a flush mount road box.

The SAA's were installed approximately 1 week following the drilling to allow time for the grout to set. TH22-01 was drilled on January 19th, 2022, and the SAA was installed on January 28th, 2022. TH22-02 was drilled on February 15th and 16th, 2022 and the SAA was installed on February 24th, 2022. TH22-03 was drilled on May 2nd, 2022. TH22-04 was drilled on May 3rd, 2022 and the SAA was installed on May 16th, 2022.

Thurber test hole location plan, test hole logs and the drilling investigation photo log are provided in Appendices A, B, and C.

3.2 Laboratory Testing

Visual identification and moisture content determination was conducted on all soil samples which were returned to our laboratory. Seven Atterberg limits were completed on samples that exhibited signs of plasticity based on visual identification. Two grain size analysis tests were completed on select samples to assess for material reuse. It should be noted that the samples selected for grain size analysis appeared to break down during testing and the results may be finer than the original samples. Fines content sieve tests (Passing No. 200) were conducted on 2 select samples.

Point Load Index Strength Tests (PLTs) were completed in the lab using a testing apparatus from RocTest (model PIL-7) as per ASTM D5731-08. They were carried out on 26 irregular lump samples from the siltstone sonic rock core in TH22-04 to determine the general range of rock strength at this location.

All lab testing results are provided in Appendix D.

3.3 Landslide Activity and Monitoring

The installation of SAA's allows for remote monitoring of the deformation with readings being recorded twice daily at noon and midnight.

The observed deformation in the test holes appears to be between depths of 7 m and 10 m below road grade near the transition from colluvial soils to dense glacial till in TH22-01 and within the glacial till in TH22-02. The nature of the movement appears to be tilting and/or sliding in the downslope (north) direction. The stratigraphy is discussed in more detail in Section 4.1 below. Plots of cumulative deflection and incremental deflection for TH22-01, TH22-02 and TH22-04 are attached in Appendix E. Rate plots of the total deformation in the downslope direction are also attached for TH22-01 and TH22-02. No significant deformation has been observed in TH22-04 and no rate plot has been prepared. Discussion of results is provided regularly in SAA Monitoring Memos.

Deformation of the slide is occurring; however, historical rate information is not available to assess if the observed rate is consistent with historical patterns. We understand that the slide has been moving for more than a decade, and therefore, the observed deformation is interpreted as being a creep type deformation pattern. It is not known if the movement is episodic (seasonal, or weather related) or if deformation will occur continuously. Brittle failure of the slide is considered possible, and it could be triggered by external factors or simply accumulated strain along the shear plane.

Based on the active movement of the landslide as demonstrated by the tension cracks and monitoring results, the current slope stability at the roadway has a Factor of Safety (FoS) of about 1.0 and does not meet the required MoTI geotechnical stability criteria.

3.4 Groundwater and Precipitation Monitoring

Groundwater seepage was not observed in any of the test holes during the geotechnical investigation. No groundwater table was observed in the colluvium slope above the site. Seasonal or episodic flows of water through the colluvium may occur but a continuous flow of groundwater is not expected. Where a stratigraphic change / reduction in permeability occurs (i.e., Colluvium – Till – Bedrock contact) the likelihood of encountering groundwater flows is increased.

The weather conditions during the drilling investigation, which followed the atmospheric river event of November 2021, seemed to be consistent with seasonal trends including moderate precipitation; however, no specific comparison with historic weather data was carried out.

One standpipe piezometer was installed by MoTI at TH20-03 during their 2020 investigation. An additional standpipe piezometer was installed at TH22-03 during the 2022 supplementary investigation. Water level readings were taken in both piezometers when Thurber was on site, as shown in Table 3.1 below.

Table 3.1: Groundwater Monitoring Results

Well ID	Install Date	Depth of Well (mbgs*)	Reading Date	Depth of Water (mbgs*)	Notes
TH20-03 (MoTI)	9/10/2020	5.68	18/1/2022	Dry	
			28/1/2022	Dry	
			24/2/2022	Dry	
			16/5/2022	Dry	
TH22-03 (Thurber)	2/5/2022	7.62	2/5/2022	6.00	Reading taken upon well completion
			3/5/2022	Dry	
			16/5/2022	Dry	

* metres below ground surface

An automated rain gauge was installed on site on August 30, 2022 and is currently recording the rainfall amounts on the same schedule as the SAA readings. Automated emails have been sent out daily since October 21, 2022 providing 24 hour precipitation amounts. Load restrictions as detailed in our memo dated July 21, 2022, are being implemented based on the precipitation data.

4. OVERVIEW OF GEOTECHNICAL SITE CONDITIONS

4.1 Soil Conditions

The slide area at the road level is approximately 110 m long (Figure 2A). At the west end of the project area the upslope side of the roadway is comprised of bedrock consisting of Nanaimo group sedimentary conglomerate, shale, siltstone, and sandstone. The bedrock ridge above the roadway trends at a skew to the roadway in a southeast direction. Small bedrock outcrops are present within the upslope ditch throughout the east end of site extending to the extent of the project boundary. Further details of bedrock outcrop and rock type are provided below in Section 4.2.

The slope below the roadway is very steep and exhibits a series of scarps and benches. The slope is about 30 m to 35 m high extending from Canal Road down to the beach with an overall slope of around 40°. The scarps are sloping at about 45° to 55° and the benches are flatter. Some larger trees (0.5 m to 0.8 m diameter) growing on the upper slope show pistol butting or outward lean however this is not universal or dominant. At the east end of the visible cracking, the slopes become somewhat flatter with an increased setback from the crest of the steep slopes.

Boulders are observed on the slopes, beneath trees and adjacent to trees. It is not known if the boulders are part of the natural deposit or were side cast down the slope at the time of roadway construction. Given the presence of the rock fill landing adjacent to the site, it is not anticipated that the roadway is built on a significant thickness of sidecast rock fill and the rock excavated for roadway construction may have been used for the landing located at the ocean level.

Existing fill materials encountered within our test holes were generally comprised of a widely graded mixture of gravel and sand, with trace to some silt. The fill was typically compact and ranged in thickness from 0.3 m to 1.5 m.

Underlying the fill at all test hole locations, a widely graded silt, sand and gravel colluvium with some clay (low plasticity) was encountered. This unit was loose to compact and contained trace amounts of organics (rootlets) throughout the entire unit. It typically ranged from approximately 5.0 m to 9.1 m in thickness at TH22-01, TH22-02 and TH22-04. At TH22-03 only a thin veneer (~0.3 m) of colluvium was observed overlying shallow bedrock. In TH22-02, till-like inclusions were encountered within a deeper portion of this deposit, bordering the glacial till. Sieve and grain size analyses resulted in fines contents ranging from 55% to 65% within TH22-03 and TH22-04. We noted that during the grain size analysis testing, the weaker gravels and coarse aggregate appeared to have broken down which resulted in more fines than visually assessed in the field. As observed from the road, cobbles to boulders reaching sizes of 1.5 m and possibly larger are visible within the colluvium slopes above.

Glacial till-like deposits typically underlay the colluvium and are comprised of dense and widely graded mixtures of silt, sand, and gravel with varying clay content from some clay to clayey. The till was only fully penetrated in TH22-02 and TH22-04 and was measured to be between 1.1 m and 10.4 m thick. Test hole TH22 01 terminated within the deposit and TH22-03 did not encounter it below the thin veneer of colluvium overlying bedrock. Atterberg testing was completed on higher fines content samples with the majority testing as low-plastic silts and clays. It is our interpretation that this deposit behaves in a non-cohesive manner. No cobbles or boulders were encountered in the till layer during drilling, but it is considered possible that they are present within the layer.

Upslope of the roadway, surficial observations indicate that the soils may be comprised of a colluvial apron (gravity and water transported particles) beneath sedimentary bluffs of Nanaimo Group bedrock (Mount Norman). The gradation of the colluvial apron has not been explicitly investigated. As evidenced by the test holes completed below the road and surface observations (boulders), it is expected that particle sizes will range from fines (material passing the 0.075 mm screen) up to large boulders.

4.2 Bedrock Conditions

Bedrock was encountered in three of the test holes completed at the site. Test hole TH22-02 encountered bedrock at a depth of 17.4 m, TH22-04 encountered rock at 6.4 m and TH22-03 encountered rock at 1.2 m depth. Bedrock was not encountered in TH22-01. The test hole depths were planned to accommodate the existing length of the MoTI owned SAA instruments and could not be drilled deeper due to time and logistical restrictions (BC Hydro shutdown).

GSC Map 1553A of Victoria identifies this site as having both De Courcy Formation and Cedar District Formation within the Nanaimo Group which consists of sandstones, conglomerates, shales, and siltstones. The available detailed bedrock geology map does not delineate the contacts between individual units across the subject site.

At the road level the existing upslope bedrock outcrop consists of predominantly conglomerate and sandstones of the De Courcy Formation. In TH22-02 the bedrock was identified as shale and siltstone at a depth of 17.4 m below existing road grade. We expect this siltstone is a part of the Cedar District Formation and underlies the De Courcy Formation at depth. In TH22-03 and TH22-04, approximately 1.2 m to 5.8 m of weathered rock overlies the more competent bedrock at depth. The bedrock was identified as a mix of conglomerate, sandstone and siltstone at these locations.

At approximately Sta. 100+240, small siltstone outcrops were identified within the upslope ditch above the road and were persistent to the eastern boundary of the project. We expect this siltstone may be part of the Cedar District Formation or within a siltstone lens of the De Courcy Formation. Outcrops are shown on the test hole location plan in Appendix A.

Point load testing on the siltstone samples within TH22-04 gave approximate point load strength index values (Is(50)) of between 0.7 MPa and 4.6 MPa. This results in correlated UCS strengths (based on an assumed K value of 24.5) of between approximately 17 MPa and 113 MPa with an average strength of 48 MPa within the siltstone. This approximate rock strength was considered in our geological model. Conglomerate rock was not encountered in the test holes and point load testing was not carried out due to the lack of samples.

The location of the bedrock contact (collected by handheld gps) is shown on Figure 2A. Along the current road alignment to the east of the contact we anticipate the rock encountered to be siltstone, west of this location we anticipate conglomerate. The structural measurements of the contact could not be determined from the visible exposure, therefore the contact location to the south of the current road cannot be confirmed.

The exposed surface of the siltstone appears to be weathered and very friable. It is possible the unweathered rock is more competent; however, an unweathered surface could not be exposed by hand. Siltstone exposures are intermittent though the vegetation and have a toe of siltstone colluvium veneer in select areas.

The conglomerate is matrix supported with clasts of varied provenance, ranging from fine sand to cobbles up to approximately 200 mm in diameter. Clasts are generally subrounded to rounded and vary from well graded at the east end of the exposure to gap graded at the west extent of the site.

The test hole location plan and test hole logs are provided in Appendix A and Appendix B respectively. Lab testing results are provided in Appendix D.

Rock structure data consisting of predominantly bedding data was mapped on site to check discontinuity orientations and carry out kinematic stability checks for the rock cut slope design. Table G1 in Appendix G presents the individual joint measurements.

Two large wedges are observed above the crest of the design rock cuts. These wedges are discussed in additional detail in Section 8.1 of this report. Photos of the wedges are provided in Photos 23 and 24 in Appendix C.

5. OPTIONS DEVELOPMENT & ASSESSMENT

The roadway design options were assessed, modified, and refined through a team-based multi-disciplinary approach which included design workshops and interim correspondence (emails and memoranda) between Thurber, MoTI, McElhanney, Hemmera and Wood. A Multiple Account Evaluation (MAE) was developed by McElhanney which weighted each concept against 10 different evaluation criteria consisting of Environmental/First Nations Impacts, Constructability, Construction Cost, Construction Schedule, Geotechnical Risk, Impacts to Parks Canada and Other Property, Maintenance/Lifecycle Cost, Road Geometry/Safety, Structural Risk and Traffic Impacts (including detour).

The following three design concepts were considered and discussed further in our Design Options Letter dated May 18, 2022:

- Concept 1 (Rock Cut Design): Shifting the road alignment to the south to result in the road structure being supported on bedrock through the main segment of the slide. This concept includes a significant encroachment into the adjacent national park reserve.
- Concept 2 (Bridge): Construction of a bridge across the slide area which reduces upslope cut requirements. The premise of the option is to avoid any construction outside of the current ROW.
- Concept 3 (Slope Stabilization System): Construction of a downslope pile stabilization system to retain existing soils. The premise of the option is to avoid any construction outside of the current ROW.

After deliberation by the team and completion of a risk register to evaluate each concept, Concept 1 was chosen by the design team.

Concept 1 consists of a full shift of the Canal Road alignment south into the existing upslope bedrock by drilling and blasting a rock cut. This alignment includes 3 m rockfall catchment ditches and assumes 0.25H:1V rock cut slope angles. The rock cut will extend from approximately Sta. 100+140 to Sta. 100+315 which is about 5 m beyond the extent of the existing cracking.

6. GEOTECHNICAL DESIGN CRITERIA AND LEVEL OF UNDERSTANDING

Consistent with the BC Ministry of Transportation (BC MoTI) Geotechnical Design Criteria (Technical Circular T-01-15 and T-04-17), the following recommendations have been made with the consideration of the following design guides and codes:

- MoTI Technical Circulars
- CSA S6-14 (Canadian Highway Bridge Design Code, CHBDC)
- MoTI Supplement to CHBDC S6-14
- Publication No. FHWA-NHI-10-024 “Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes” November 2009
- MoTI Technical Bulletin GM02001 Rock Slope Design
- AASHTO 1993 Guide for the Design of Pavement Structures or AASHTO (2004) ME Pavement (Mechanistic Empirical Pavement Design Method Guide)
- Canadian Foundation Engineering Manual (CFEM), Fourth Edition, 2006
- Letter of Agreement between the Ministry of Transportation and Islands Trust dated October 20, 1992
- Canadian Foundation Engineering Manual (CFEM)
- FHWA-IF-99-015 – Geotechnical Engineering Circular No. 4 – Ground Nails and Nailed Systems
- FHWA-NHI-14-007 – Soil Nail Walls Reference Manual
- PTI DC35.1-14

6.1 Degree of Understanding and Consequence Factor

Currently we have completed a geotechnical investigation at this site consisting of four test holes and no bedrock investigation (coring, UCS strength testing). The newly completed test holes supplement the previous completed drilling by MoTI in 2020.

We have considered the available information to be acceptable to declare a ‘Typical’ degree of understanding for the landslide and rock cut. A high degree of understanding may be achievable with additional testing; however, it is not necessary since the bedrock stability exceeds the target stability for both typical and high degree of understanding. MoTI has currently assigned a typical consequence to this section of Canal Road.

The proposed shotcrete and anchor wall is in a location which has not undergone any subsurface investigation at the exact wall location due to difficult access, park property and project schedule constraints. Anticipated conditions at the wall location are discussed in Section 8.3.3, however, a ‘Low’ degree of understanding has been achieved at this location.

7. SLOPE STABILITY ANALYSIS

7.1 General

Limit equilibrium slope stability analyses were completed for this project using the commercial software Slope/W 2021 (GeoSlope International). The Morgenstern-Price method was used for calculating factors of safety with the method of slices, which includes both force and moment equilibrium.

Thurber assessed the stability of the slopes based on a ‘Typical’ degree of understanding and a ‘Typical’ consequence factor as per Table 6.2b provided in MoTI’s Supplement to the CHBDC S6-14 dated October 28, 2016. This equates to a minimum Factor of Safety of 1.54 for global stability of permanent slopes.

Under seismic conditions, Major-Routes are subject to a minimum pseudo-static Factor of Safety of 1.1 under the 1:975-year (5% probably of exceedance in 50 years) seismic hazard values for global stability of permanent slopes and embankments according to Table 4.4.6.4 of MoTI’s Supplement to the CHBDC S6-14. Retaining walls are subject to seismic performance requirements including non-collapse under the 1:975-year seismic hazard.

All concepts described above intend to achieve the geotechnical design criteria.

7.2 Stratigraphy

The modelled soil and bedrock stratigraphy is based on engineering interpretation of the available geotechnical data from the site investigations.

The slopes were generally modelled as ‘dry’ as the result of the investigation indicates that the existing fills and overburden are well drained by surface runoff and by infiltration into the underlying fractured bedrock. Further the overburden soils are typically silty with inferred slow infiltration rates which promotes runoff on steep slopes. Groundwater may periodically drain though the surficial colluvium which would be addressed by the recommended drainage (Section 8.3.4).

Bedrock was generally modelled as infinitely strong to model soils sliding along the bedrock contact or using rock mass strength parameters. This is generally considered appropriate due to the friable / weathered nature of the bedrock encountered in the test holes. No attempt has been made to consider anisotropic behaviour of the bedrock or individual bedrock fractures.

7.3 Material Parameters

Back analysis of the existing slopes was completed to confirm that the strength parameters used in the design were appropriate. Due to the current slide conditions, the existing stability condition of the sections within the slide zone (extent of cracking) was assumed to have a factor of safety (FoS) of about unity under static (non-seismic) conditions.

Linear Mohr-Coulomb strength envelopes are a commonly used simplification in slope stability modelling. However, it is well documented that the shear strength envelopes of coarse granular soils are commonly non-linear with respect to vertical effective stress[1]. At low effective stresses, the friction angle (slope of the strength envelope) can be significantly higher which is reflected in the performance of steep slopes built of coarse angular soils. Therefore, Thurber modelled the colluvium on site using a bilinear approximation of non-linear failure envelope. This was required to satisfy the back analysis assumptions above.

The bedrock was modelled using RocScience's RocLab software to develop a Shear/Normal Function for the bedrock using the averaged PLT data and estimation of Geological Strength Index (GSI) and mi values onsite. In order to conservatively model the lower bound estimate of slope stability, siltstone was used for the bedrock model inputs across the site.

Table 7.1: Static Stability Analysis Strength Parameters for Key Materials

Stratigraphic Unit	Strength Model	Unit Weight (kN/m^3)	Effective Friction Angle (deg)	Effective Cohesion (kPa)	Phi 1 (deg)	Phi 2 (deg)	Bilinear Normal (kPa)
Existing Fill	Mohr-Coulomb	20	36	0			
New Fill	Mohr-Coulomb	21	36	0			
Colluvium	Bilinear	20		0	45	36	50
Glacial Till	Mohr-Coulomb	22	42	0			
Bedrock	Shear/Normal Function	26					

7.4 Slope Stability Analysis Results

Nine sections in total were assessed along the road alignment to review geometry in relation to the interpreted geological model. Of the nine sections, five were selected as representative for the site and three stability conditions were analyzed for each. These include a back analysis

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[1] EPRI, 1990, *Manual on Estimating Soil Properties for Foundation Design* (EL-6800).

(existing conditions), proposed alignment under static conditions, and proposed alignment under seismic conditions.

Soil strength models were developed for back analyses models only. Due to limitations of the software, the soils below the existing roadway exhibit low stability when modelling the proposed highway alignment under static and seismic conditions. As the majority of the road alignment is founded on bedrock, and the model relies exclusively on the strength of the bedrock, the soils were ignored where the road alignment was entirely on bedrock to produce a more accurate model.

A traffic surcharge of 16 kN/m^3 was implemented for static loading cases. A horizontal PGA of 0.359 g (which corresponds to 100% of the 5% in 50-year seismic event) was implemented for the seismic loading cases. No reduction of PGA was required to achieve the target stability in the proposed alignment condition.

The following Table 7.2 includes the results of the analysis. Examples of Section 2, Section 6 and Section 8 are provided in Appendix F:

Table 7.2: SlopeW Stability Analysis Results

Section	Stationing	Existing Conditions	Proposed Alignment		Proposed Alignment with GRS Reinforced Fill	
		Back Analysis (FoS)	Static Loading (FoS)	Seismic Loading (FoS)	Static Loading (FoS)	Seismic Loading (FoS)
2	100+165	1.02	3.33	1.86		
4	100+210	1.11	3.38	2.44		
6	100+240	1.04	1.28	0.71	2.19	1.11
8	100+265	1.12	3.94	2.40		
9	100+300	1.47	>5	>5		

Section 6 located at Sta 100+240, containing an existing pullout / lower area on the upslope side of the roadway, was modelled conservatively as bedrock was not confirmed in the upslope ditch as per the other sections. Therefore, half of the westbound (northern) lane is modelled as being founded partially on the native colluvium soils. It is our interpretation that this section applies over about 10 m of the alignment. We have included a design for a small geogrid reinforced fill to pass over this segment and is discussed in additional detail in section 8.4.

8. DESIGN RECOMMENDATIONS

8.1 Rock Cut Slopes

We recommend that rock cuts be sloped at 0.25H:1V with 3 m wide rockfall catchment ditches. For rock cuts greater than 8 m in height the catchment will not achieve the required catchment performance (85% of rockfall emanating from the slope) without slope mesh. It would generally be preferred to construct sufficiently wide catchments to avoid the use of slope mesh; however, the geometry was selected to provide sufficient catchment as well as to limit encroachment into the park above Canal Road.

Thurber completed a kinematic stability analysis utilizing the structural data collected on site during the investigations. Analysis shows that the proposed rock cut is in the general orientation of the bedding orientation in the siltstone. The Conglomerate bedrock tends to have a more massive, isotropic character due to the variation of grain-sizes within the unit. Some steep, natural fractures were mapped, and the existing slope angles are generally similar or steeper than the proposed cut slope angle. Two kinematically possible wedges have been identified where J4 intersects J1 or J3. Although these kinematically wedges may form, they are considered unlikely to require substantial stabilization due to the joint roughness on the conglomerate joints (high friction angle) and minor presence of the J4 joint set.

Since the rock cut angle is close to or flatter than the major joint orientation, we do not expect significant planar sliding, or toppling failures to occur. We anticipate that some stabilization of the backslope will be necessary as the rock cuts are carried out in benches to address wedges, random joints or fractured zones that may be encountered.

The results of the kinematic analyses are provided in Appendix G.

8.1.1 Cut Slope Stabilization Measures

The new roadway will be shifted closer to the base of existing rock bluffs that form Mt. Norman. The maintenance contractor rockfall reporting records for Canal Road from 1993 to Dec 2022 were provided by MoTI (Appendix J). A total of nine rockfall events were recorded, none of which are believed to be within the project site. Rockfall is considered possible emanating from above the new cuts, within the new cuts and from newly exposed natural rock faces (which are currently buried). We have considered that rock bolt stabilization will be necessary for new rock cuts and newly exposed rock faces. The need for slope mesh is also discussed in additional detail below.

For budgeting purposes, we recommend including an allowance of 0.1 m of rock bolt for each square metre of rock face (new cut and newly exposed faces) this corresponds with one 6 m long bolt for each 60 m^2 of rock face. The quantity of bolts that will be needed is not known and is based on judgement for the purpose of inclusion in the contract.

The bolt locations, length and installation angles will need to be determined in the field by the geotechnical EOR as each bench is excavated and the backslope condition can be reviewed. It is considered likely that the bolts will be installed in groups or small patterns depending on the rock slope condition and structure. The rock bolts should be installed in accordance with SS206 of the MoTI Standard Specifications for Highway Construction.

Two large wedge features were observed above the crest of cut as described in section 4.2 of this report. To reduce the probability of future issues with the wedges it is recommended that they are both stabilized with four (total of eight bolts) – 6 m long rock bolts installed in accordance with SS206.

To address rockfall emanating from the newly exposed and new rock cuts, the design 3 m wide catchment ditch is considered adequate for rock slopes up to 8 m high. For rock cuts / newly exposed slopes greater than 8 m in height, we recommend installing slope mesh in accordance with SS207.

We have also reviewed the condition at about Sta. 100+240 to Sta. 100+270 where the new roadway may be exposed to a higher rockfall hazard from above the new cuts than currently exists due to the shifted alignment. We analyzed the existing and proposed geometry using assumed parameters to assess the difference between the rockfall hazard before and following construction. The actual parameters used are not particularly important since they are consistent in both analysis cases.

The results of the analysis indicate that the post construction condition is sensitive to the actual final rock slope configuration and the use of slope mesh. Theoretical falling rocks will bounce off of irregularities on the slope and care will have to be taken to shape the new cuts to minimize irregularities that would act as launch features.

Where slope mesh is installed in accordance with the current design, the predicted number of rocks that would reach the roadway is reduced. Since some rockfall trajectories would not be intercepted by the slope mesh (1 m high at crest), the maximum energy at the edge of the road is higher in the new roadway configuration. Reduction of the impact energy at the roadway is not considered feasible due to bouncing rock trajectories (described above) that project above the slope mesh.

In this location, the new slope mesh is anticipated to intercept sufficient theoretical falling rocks from above the cuts and result in a similar risk to the travelling public. Given that no significant rockfall history is known in this location, large rockfall events are expected to be rare and are not considered for the design assessment.

We recommend that the slope mesh be installed at the crest of new cuts (overburden and rock) in this area to decrease the frequency of rockfall impacting the roadway. The mesh extent is shown on the plan drawings in the Civil design package.

8.1.2 Blasting In Proximity to the Active Slide

Since the slide is actively moving and the travelling public is accessing the top of the slide during construction, it is considered critical that the blast energy associated with rock excavation be controlled to minimize the risk of increased slide deformation. We carried out a parametric analysis using assumed shear wave velocities and a fundamental period of the critical failure surface slide mass. We used shear wave velocities in the range of 200 to 400 m/s and the resulting fundamental periods associated with the critical slip surface were in the range of 0.05 s to 0.09 s.

Using the fundamental period of the critical slip surface and varying levels of Kh, we were able to calculate an equivalent peak particle velocity (PPV) associated with an applied Kh. We used the back analyzed material parameters and then considered the decrease in stability of the slope when applying the varying levels of Kh. We also considered the possible increased allowable PPV if slope stabilization soil nails were to be installed in the slide mass.

Since the blast vibrations dissipate at an exponential rate based on the distance from the blast, the PPV at the crest of the slide could be higher depending on the rate of attenuation. This would have to be calculated based on scaled distance and characteristic values for the attenuation. These factors could be estimated based on typical values or could be based on a site-specific correlation that could be developed through blast monitoring. Based on our preliminary check, the PPV at the centre of the slide mass could be three to ten times less than that measured at the crest of the slide. We used the US Bureau of Mines square root scaled distance of the estimate of vibration dissipation. The centre of the slide mass is determined in cross section since slide may consist of individual lobes and it extends laterally along most of the site.

The most significant factor on the attenuation is the distance to the actual blast. This means that no specific limit should be set for PPV at the edge of the slide as it will need to be determined for each blast individually. Table 8.1 below presents the results of the preliminary analysis carried out for a range of PPV values at the centre of the slide mass. The benefit of installation of soil nails is also presented in Table 8.1.

Table 8.1: PPV – Stability Summary

Kh (g)	PPV lower bound (mm/s)	PPV upper bound (mm/s)	Change in Stability	Change in Stability with Soil Nails Providing 200 kN/m of Stabilizing Force
0.01	1	1	-3.00%	+4.00%
0.025	2	4	-6.00%	+1.00%
0.05	4	7	-10.00%	-3.00%
0.1	7	14	-18.00%	-11.00%

It should be noted that it is not currently known if the blast-induced vibrations / temporary accelerations could result in brittle slide deformation, sustained increases to the deformation rate or if they would have a negligible effect on the overall slide deformation pattern and rate. Accordingly, to mitigate the potential for increased slide deformation during blasting we recommend the following:

- The contractor be required to retain a blasting vibration specialist that is also a geotechnical engineer who will be responsible for analyzing the slide mass and determine PPV’s that will not trigger changes in the slide deformation pattern and rate.
- Each blast has a hold point for the review of the blast monitoring information and slide deformation data by the Contractor’s blast vibration specialist / geotechnical engineer and the Ministry Representative prior to taking the next blast.
- The contractor be responsible for completing blasting and excavation in a manner that limits the slide deformation rate in the SAA to 50% of the historic high (SAA1 = 0.6 mm/day x 50% = 0.3 mm/day and SAA2 = 1.0 mm/day x 50% = 0.5 mm/day) that cannot be exceeded during construction.
- Installation of temporary stabilization measures such as soil nails could be considered to maintain the current stability condition during blasting.

A vibration limit that represents a risk management approach is preferred to balance constructability for blasting and limit the risk of triggering movement of the slide has been requested by the project team.

For the purposes of setting guidelines at the outset of the project we believe it would be reasonable to limit the blast vibration at the centre of the slide to PPV ≤ 3 mm/sec. Measurement of the vibration at the centre of the slide may not be possible due to access constraints and may need to be measured at the edge of the slide. The tolerance for blast vibrations at the edge of slide will need to be calculated by the vibration specialist for each blast due to the square root distance attenuation relationship discussed above. An example of the configuration for blast monitoring is provided in Figure 8.1.

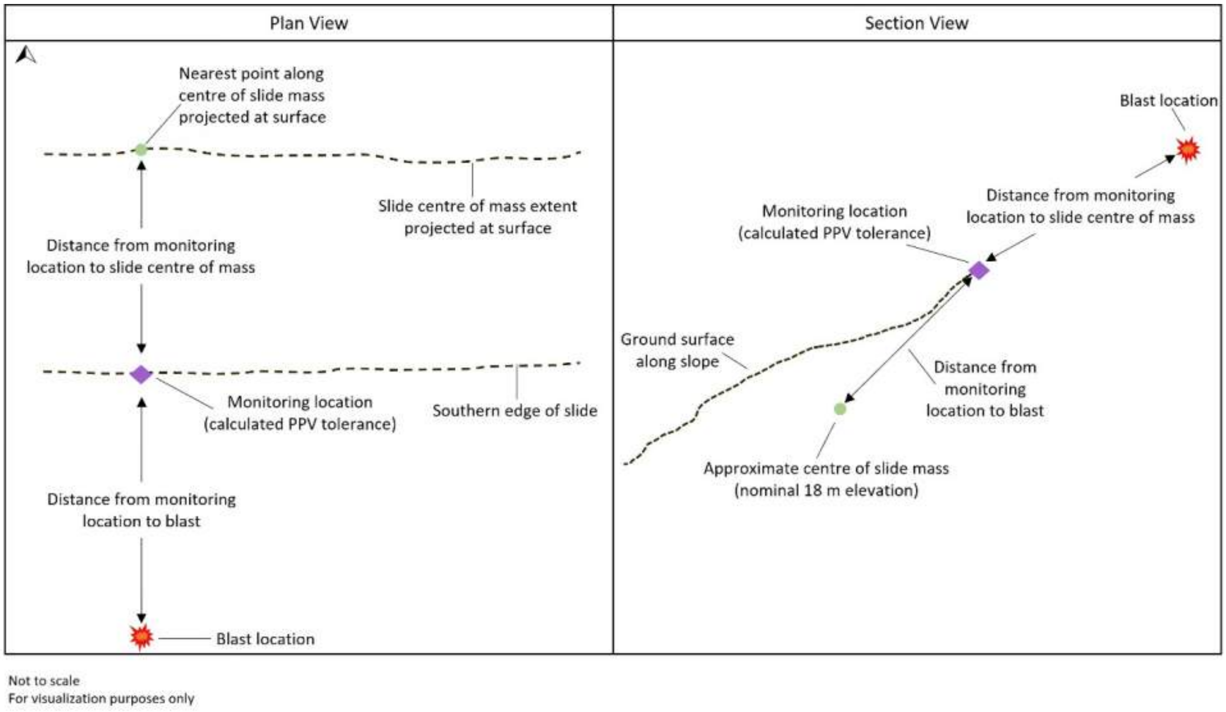

Figure 8.1: Blast Monitoring Configuration

The centre of the slide in cross section is generally 15 m north of the downslope edge of pavement (north side of Canal Road). The slide is continuous between Sta. 100+145 to 100+315. The location of the centre of the slide mass is sketched on Figure A2 in Appendix A.

The distance from the blast to the centre of the slide must be measured at the nearest points along each extent and cannot account for horizontal distance along the project chainage. The definition of the centre of the slide in section and maximum PPV may need to be updated based on monitoring and tension crack observations during construction.

We are aware of recent blasting projects on Vancouver Island that carried out many blasts with charges per delay in the range of 1 kg to 4 kg. Smaller blasts may be less productive and more expensive than larger blasts. The installation of stabilization to allow increased blast energies could be considered by the contractor if it is more efficient.

8.2 Overburden Cut Stabilization

We recommend that permanent overburden cuts on the upslope side of the roadway should generally be sloped at 1.5H:1V.

Where bedrock is encountered in the overburden cut slopes, it can be exposed following its natural profile. Exposed rock surfaces should be reviewed by the geotechnical engineer during construction to confirm stabilization requirements.

8.3 Upslope Retaining Wall

A retaining wall is required on the upslope side of the roadway from approximately Sta. 100+295 to approximately Sta. 100+335. The wall has been named Mt. Norman Wall for the project. The retaining wall would have a maximum height of about 11 m. The available geotechnical data indicates that bedrock is relatively shallow below the proposed wall toe (approximately 1.2 m deep at TH22-03).

A shotcrete and anchor wall was selected for detailed design by the design team. No subsurface investigations were undertaken at the location of the upslope retaining wall. Design recommendations are based on site observations and geologic mapping. A ‘Low’ degree of understanding has been achieved for the design of the Mt. Norman Wall.

8.3.1 Shotcrete and Anchor Wall Geometry and Construction Sequence

The shotcrete and anchor wall was selected for due to the flexibility of reduced extents where the depth to bedrock is uncertain. The following section details the geometry of the wall, constructability, and staging.

Given that the risk tolerance for failure of a permanent wall is low we recommend Class A double corrosion protected anchors as recommended in FHWA NHI 14-007 Section 7. We recommend that the geometric design of the shotcrete and soil anchor wall include a 0.25H:1V slope angle with a 2 m offset between the toe of the wall and the crest of the rock cut. We recommend a maximum soil nail spacing of 2 m.

Figures 8.1 and 8.2 below provide constructability details for a shotcrete and soil anchor wall adapted from FHWA.

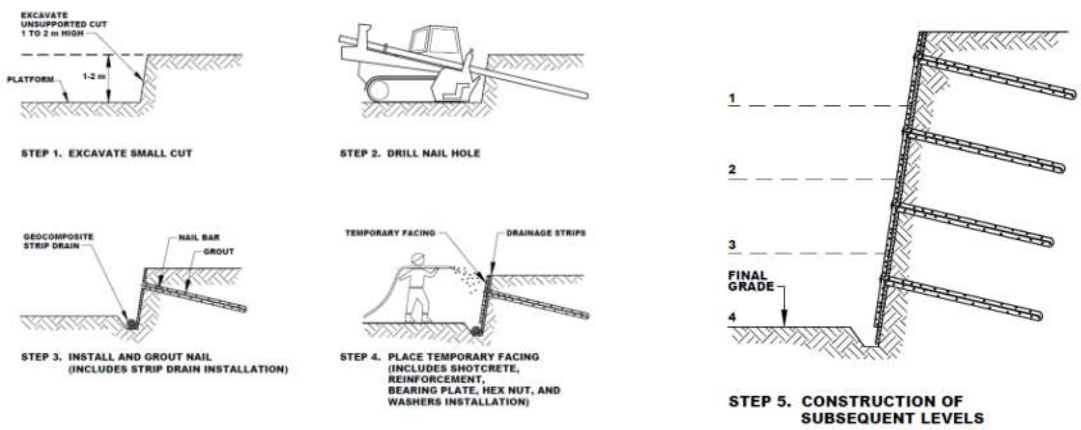

Figure adapted from FHWA0-IF-03-017 Figure 2.2 and 4.7 (b)

Figure 8.2: Shotcrete Cuts – Temporary Excavation Sequence

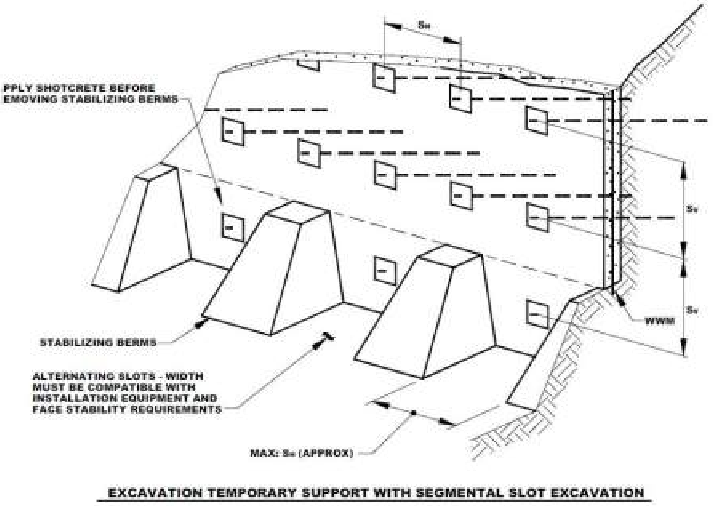

Figure 8.3: Temporary Excavation Support for Shotcrete and Anchor Temporary Wall

The wall should be excavated in panels to allow the installation of anchors and shotcrete in a controlled fashion. The responsibility for temporary excavation stability, safety and constructability should be assigned to the contractor.

It is considered likely that boulders will be encountered in the excavated face. Where boulders are pulled out such that a void is exposed behind the neat line, it will be necessary to infill the void with additional shotcrete. Alternatives to additional shotcrete could include spot bolting boulders to remain in place and / or trimming of boulders through controlled blasting or breaking to remove the projection beyond the wall face. This should be carried out with direction from the Ministry Representative. Where bedrock is encountered within the cut, it is intended that the excavation is stepped out to allow a minimum 2 m horizontal bench and then excavating the rock slope.

8.3.2 Design Recommendations

Earth Pressure Recommendations

Lateral active earth pressure coefficients were calculated using the Coulomb active pressure equations outlined in the CHBDC. These equations were used to account for both the wall batter and the slope above the wall. Due to the heigh seismic accelerations and the steep slope above the wall, a Mononobe Okabe analysis does not converge. Seismic earth pressures were calculated using the Generalized Limit Equilibrium approach using the software SlopeW assuming a rectangular distribution.

The following table provides a summary of assumed soils parameters and the resulting horizontal pressure coefficients:

Table 8.2 – Earth Pressure Coefficients for Upslope Wall

Soil Unit	Colluvium
Unit Weight Ɣ, kN/m^3	20
Friction Angle, Φ	36°
PGA (g)	0.359
Static Active (K_a)	0.291
Seismic Earth Pressure, kPa	38

The parameters given in Table 8.2 are based on the following assumptions:

- ~32° ground surface above the wall (on average),
- Wall batter angle of 0.25H:1V

- No wall friction,
- 975-year return period earthquake PGA, and
- The wall facing is fully drained (no hydrostatic water pressure).

The active earth pressure should be applied to the wall in a triangular distribution with the load acting at 1/3 up from the base of the wall. For seismic loading, the seismic earth pressure should be applied to the wall in a rectangular distribution with the load acting at 1/2 of the wall height.

Wall drainage should be comprised of regularly spaced weep holes or geocomposite strip drains to achieve fully drained conditions at the back of wall.

Global Stability

Based on our geological model, we anticipate the shotcrete and anchor wall will be constructed on a subgrade of bedrock and will meet the MoTI global stability specification (FoS > 1.71). A global stability check is provided in Appendix F (Section 9).

Anchor Design and Testing

The following recommendation presents the geotechnical design parameters for the soil anchors. We understand that the structural team (Parsons) will specify the anchor size and spacing based on the applied earth pressures, shotcrete facing design and the following anchor recommendations. It is recommended that the design be completed in general accordance with the FHWA Soil Nail Walls reference manual.

For gravity grouted soil anchors, PTI DC34.1-14 recommends an unfactored ultimate bond strength of between 70 kPa and 140 kPa. A bond reduction factor of 0.5 should be applied as per CHBDC S6 14 and therefore we recommend using an allowable bond value of 50 kPa. This bond value could be increased if low pressure grouting techniques are used; however, it has not been included at this time. Sacrificial anchors should be installed and pull tested to bond failure to verify bond strength as per FHWA NHI 14-007 Section 9.4.3 which recommends at least two verification tests are conducted in each major soil strata. This may allow optimization of the soil anchor bond length required during construction.

In addition, we recommend proof testing on five percent of the soil nails (or minimum of one in each row). The proof tests should consist of increasing loading in increments, to 120 percent of the proof test load. Deflection measurements should be taken at each load increment. A 10 minute creep test should be carried out at the last load increment.

The remaining soil nails that were not proof test should be pull tested to 120 percent of the design load and held for a 10 minute creep test.

We recommend a maximum soil anchor spacing of 2 m and a minimum anchor length in soil of 14 m to achieve sufficient bond beyond the active wedge of soil (~5 m horizontal for 9 m high wall). Soil anchors should be installed at 15° below horizontal. Limited information is available about the subsurface conditions on the colluvium slope. It is possible that large boulders and resulting voids are present. Casing is likely needed to ensure hole integrity during installation and proper bond between the soil - grout interface. Depending on the anchor installation and grouting methodology secondary grout tubes may need to be installed to allow pressure grouting and improved bond resistance. Grout socks may also be needed to control grout loss during installation.

If bedrock is encountered during the drilling, the anchors should be extended at least 2 m into competent rock.

8.3.3 Retaining Wall Drainage

A high groundwater table is not anticipated at this site. Groundwater may flow may occur through the surficial colluvium episodically during or following precipitation events, however no sustained groundwater table in the colluvium is anticipated. It is important to allow drainage behind the shotcrete facing to ensure that no hydrostatic pressures act on the wall and allow temporary flows to dissipate. This can be provided using regularly spaced weep holes or geocomposite drainage strips between the retained soil and the shotcrete. The spacing of the weep holes or strip drains should match the midpoint of anchors such that there are no conflicts.

8.4 Downslope Slope Stabilization

Shifting the road alignment to the south, results in the road structure being supported on bedrock through the main segment of the slide. Based on our geological model and slope stability analysis as described in Section 7.4, we anticipate the majority of the new road structure to meet the MoTI global stability specification (FoS > 1.54) and will be constructed on a subgrade of bedrock.

A small portion of the roadway (Section 6) may encounter compact colluvium at the base of the pavement structure between approximately Sta. 100+240 and Sta 100+250. We interpolated between surface exposures and nearby test holes and estimate that bedrock is approximately 3 m deep at this location at the proposed edge of the pavement. We recommend a sub-excavation and replacement with geogrid reinforced fill to support the portion of the westbound lane and achieve the specified stability. The sub-excavation should be down to bedrock and geogrid should overlap with the bedrock over a minimum width of 3 m. Geogrid should be placed perpendicular to the road alignment with the wrapped face in the upslope direction. All geogrid joints shall be overlapped by 300 mm and tied together at 1 m spacing.

The geogrid reinforced engineered fill pad should consist of 150 mm minus gradation with maximum of 15% fines (Passing No. 200 Sieve) or as per the MSEW backfill requirements outlined in FHWA A-NHI-10-024. For example, SGSB subbase aggregate as described in MOTI's Standard Specifications (2020 Volume 1), would exceed the backfill requirement and would be an acceptable option. The granular fill will be compacted to 95% Standard Proctor Maximum Dry Density (SPMDD). The geogrid should comprise Tensar UX1500 (or approved equivalent), should be spaced 300 mm vertically and will have variable lengths to achieve the 3 m overlap with bedrock. The backfill used within the geogrid reinforced zone should meet FHWA/AASHTO electrochemical requirements for backfill in contact with geogrid from Table 3-4 in FHWA NHI-10-024 for MSE walls and RSS.

We recommend that a 3 m deep, 3 m wide, and up to 10 m long geogrid reinforced fill from Sta. 100+240 to Sta. 100+250 can be anticipated for costing purposes. See Figure 8.3 below:

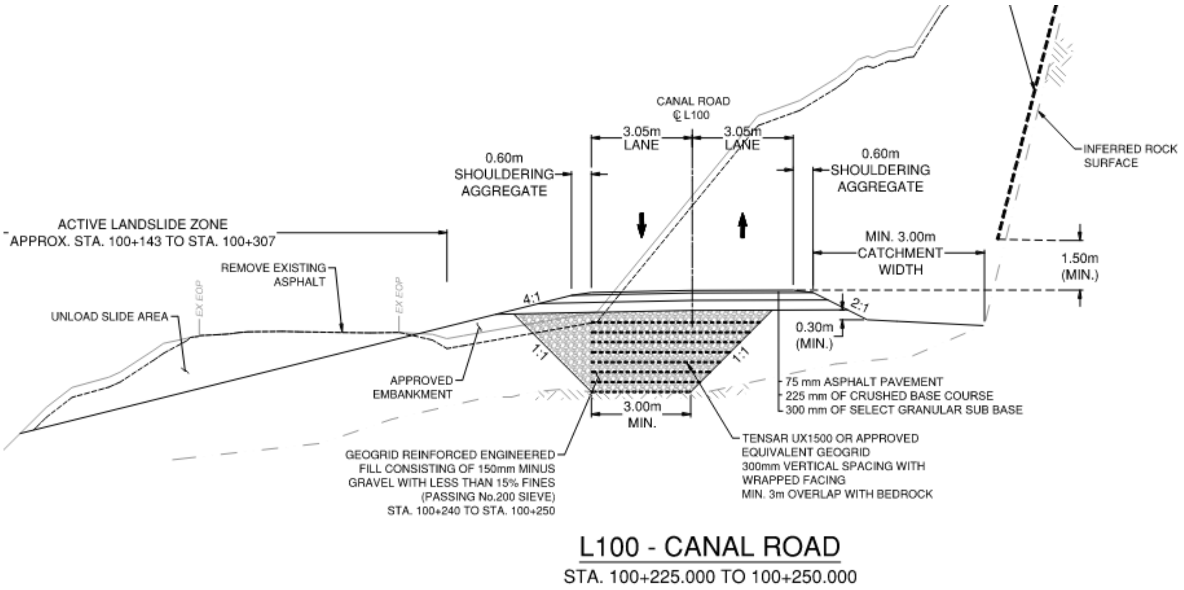

Figure 8.4: Proposed Geogrid Reinforced Fill (from Drawing R1-1025-302)

8.5 Seismic Design

Seismic hazard spectral acceleration values obtained from the National Resources Canada NBC2015 online calculator are attached in Appendix H. The peak ground acceleration (PGA) for the 2%, 5% and 10% in 50 year probability of exceedance hazards are 0.494 g, 0.359 g, and 0.266 g, respectively. Based on the site investigation data, Seismic Site Class C is generally appropriate for structures at this site founded on dense soil or weathered bedrock.

8.6 Stripping Requirements

We anticipate that the majority of the proposed alignment will be founded on bedrock subgrade. Any topsoil, organic and deleterious material exposed during excavation should be sub-excavated. For costing purposes, we suggest a minimum assumed stripping depth of 300 mm in areas of new pavements or fills not founded on bedrock subgrade.

8.7 Material Reusability

MoTI has requested durability testing on existing rock samples within the conglomerate and the siltstone for assessment of reusability for structural backfill, rip-rap or pavement gravels. Regardless of the results of durability testing, the blast rock is considered acceptable for reuse as embankment fill.

It was noted during the Grain Size Analysis testing of the overburden samples that the gravel, likely originally sourced from local bedrock, tended to break down during the wash stage of the testing, thus producing more fines than in the initial visual identification.

Additional lab testing of rock samples from site has been carried out and the results are presented in the Rock Durability Testing Memo (Revision 1) attached to this report in Appendix I.

8.7.1 ML-ARD

The re-use of blasted rock should consider the potential for metal-leaching and acid rock drainage (ML-ARD). The bedrock on site is noted to be sedimentary and based on initial visual assessment was considered low risk for ML-ARD. Thurber has collected rock samples from the rock cuts and test holes for screening level testing.

Management of potentially acid generating rock is required throughout this project. Our ML-ARD assessment, testing results and management recommendations have been reported separately.

8.7.2 Shrinkage and Swell

We anticipate for the re-use of rock fill generated from the proposed rock cuts that the bedrock will swell after blasting and shrink slightly during placement and compaction. Table 8.3 below provides an estimate of swell or shrinkage factors for the reused rock fill:

Table 8.3 – Estimated Shrinkage and Swell Factors

Material	In-Situ	Loose (Excavated)	Compacted
Bedrock	1.0	1.3	1.2

8.8 Drainage

The site is relatively free draining. However, all gully features in the slope above Canal Road should be anticipated to carry surface flow during heavy precipitation or following snow melt events. Culverts may be required where these features intersect the proposed alignment. Existing culverts should be maintained or replaced as appropriate. Suitable erosion protection measures are required at discharge locations to mitigate erosion of steep slopes.

8.9 Pavement

No analysis of the pavement structure has been carried out. The pavement structure at this site will generally be constructed within the new rock cut and the subgrade is expected to be suitable for construction of the new pavement structure.

We recommend that the pavement structure consist of Pavement Structure Type D for low volume roads as per the MoTI Technical Circular T-01/15:

- 75 mm Hot Mix Asphalt (HMA)
- 225 mm Crushed Base Course (CBC)
- 150 mm Select Granular Sub Base (SGSB)

8.10 Concrete Durability

Concrete exposure class can be determined in accordance with CSA A23.1 - 19. The exposure class determination takes into account potential concrete degradation caused by environmental conditions and/or the presence of chlorides and sulphates. At this site the presence of sulphates was below the minimum levels for a sulphate exposed classification in accordance with CSA A23.1-19 Table 3, as inferred from corrosion study which utilized the method AASHTO T 290-95 2020. The presence of chlorides is likely due to the use of de-icing salts or brines of MoTI roadways. A C1 exposure class should be considered for the specification of the concrete structures adjacent to the roadways in this location.

The corrosion study for the site is provided in a separate memorandum.

8.11 Climate Change Resiliency

We utilized the University of British Columbia Climate NA Map with the 1981 to 2010 climactic normals and several general circulation models to evaluate potential changes in future climate at the site. Historically the site receives a mean annual precipitation of 731 mm (18 mm falling as snow), the winter receives 62 frost free days, and the mean annual temperature is 13.2 °C. For the future period of 2041 to 2070 predictions of mean annual precipitation range from 795 mm to 801 mm, winter frost free days range from 71 to 77, and mean annual temperatures range from 12.0 °C to 13.2 °C.

Predictions for future climate conditions at the site are warmer and wetter. However, the new alignment lies in the order of 30 m to 40 m in elevation and will have bedrock beneath the road subgrade. Therefore, we do not anticipate climate change induced factors such as sea level rise or heavier rainfalls to have a significant effect on the geotechnical performance of the new roadway.

9. CLOSURE

We trust this provides you sufficient information for your needs at this time. If you have any questions or would like to discuss these updated recommendations, please contact us.

STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client, the BC Ministry of Transportation and Infrastructure (MoTI) and Authorized Users as defined in the MoTI Special Conditions Form H0461d. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT THURBER'S WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS THURBER MAY EXPRESSLY APPROVE. Any use which an unauthorized third party makes of the Report, is the sole responsibility of such third party. Thurber accepts no responsibility whatsoever for damages suffered by any unauthorized third party resulting from use of the Report without Thurber's express written permission.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.

d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

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APPENDIX A DRAWINGS

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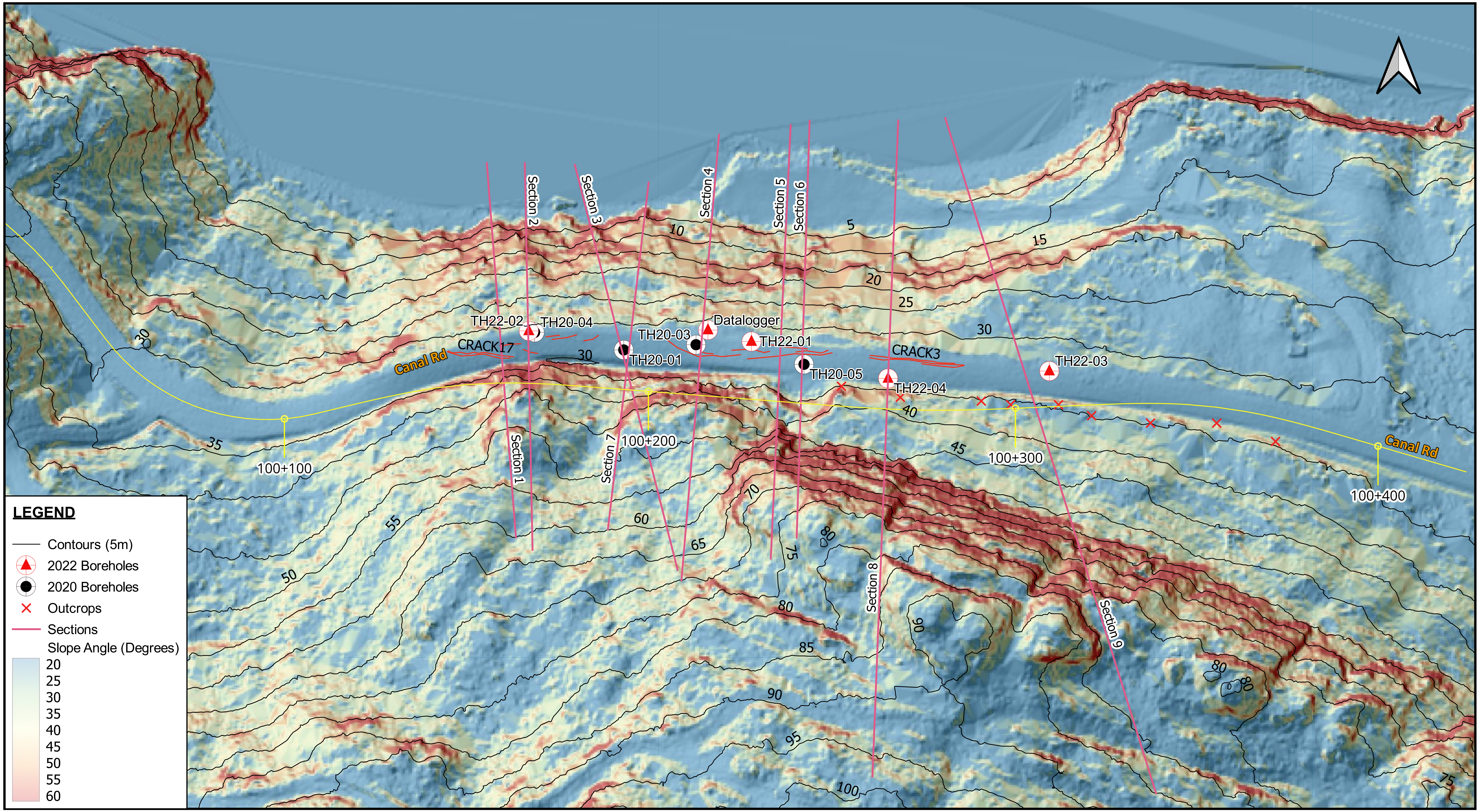
Pender Island - Canal Road Slide
Test Hole Location Plan



0 10 20 m

1:1,050

Notes:
Test hole locations based on survey by McElhanney
Contours and slope are based on bare Earth Lidar Provided to us by McElhanney January 2022

FIGURE A1

Client: BC MoTI
File No: 33450

Date: December 16, 2022

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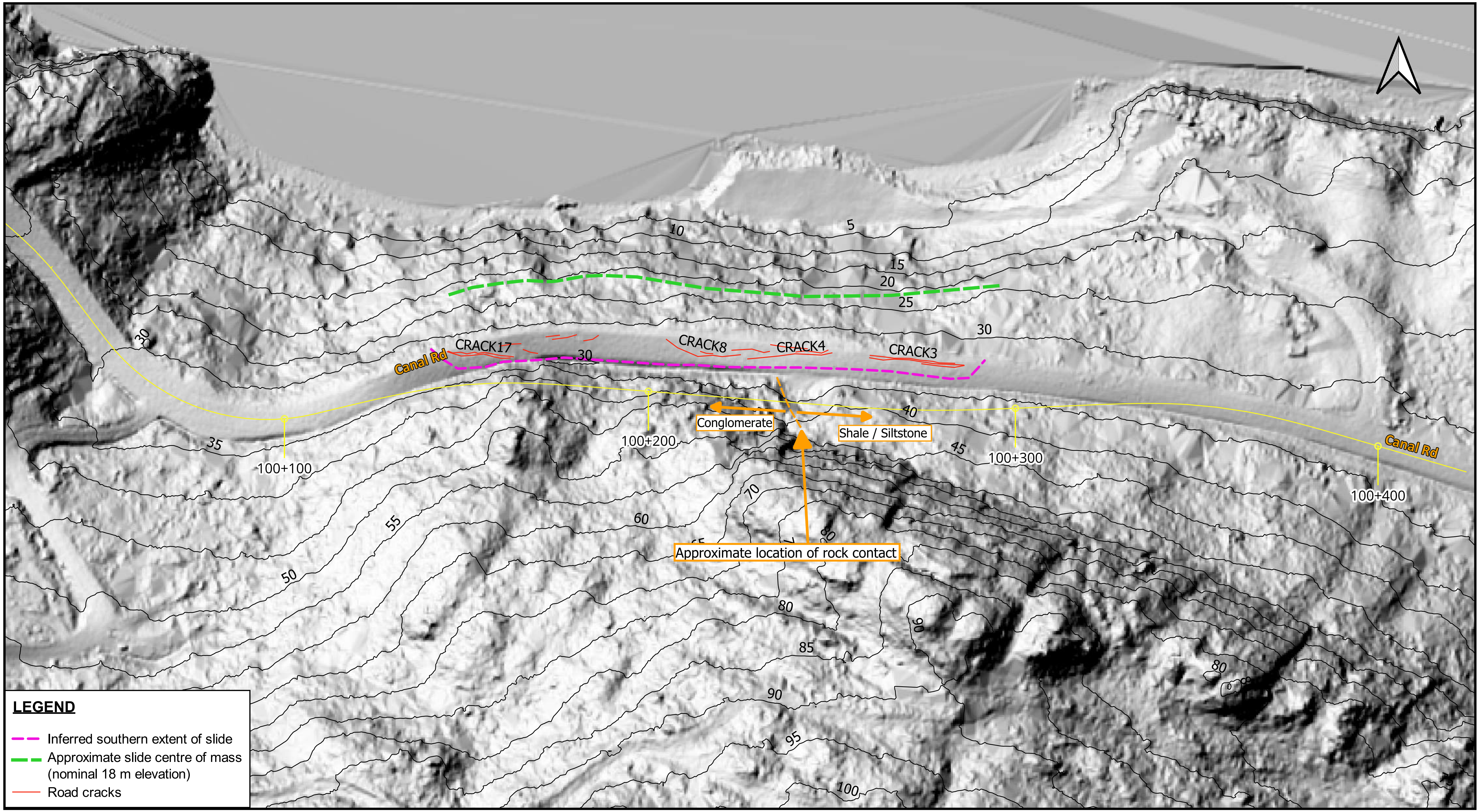
PENDER ISLAND - CANAL ROAD SLIDE
APPROXIMATE SLIDE EXTENT



0 10 20 m

1:1,050

Notes:
Crack locations based on survey by McElhanney
Contours and hillshade are based on bare Earth Lidar Provided to us by McElhanney January 2022

FIGURE A2

Client: BC MoTI
File No: 33450

Date: January 25, 2023

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APPENDIX A

APPENDIX B

APPENDIX B TEST HOLE LOGS

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

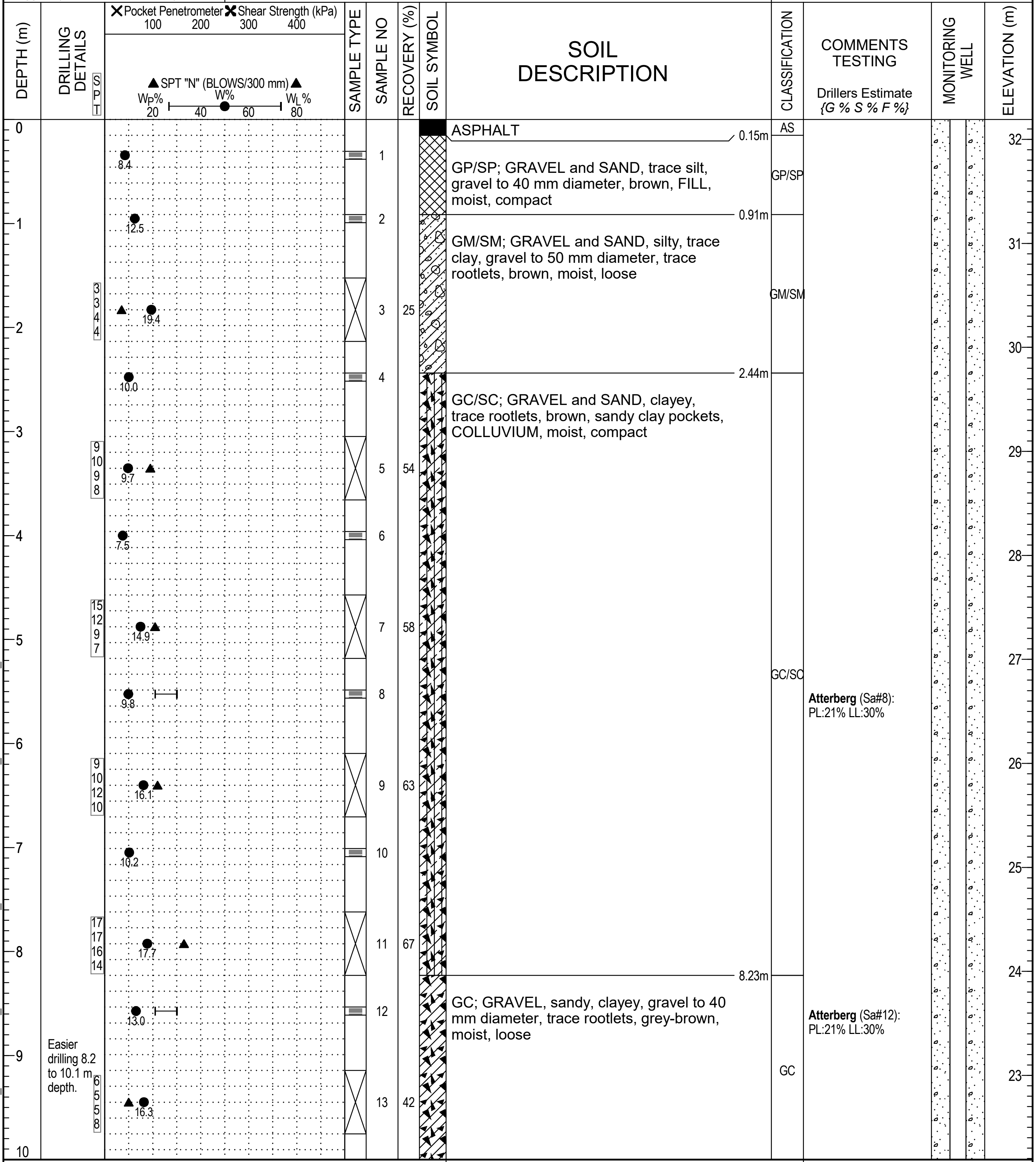
SUMMARY LOG

Drill Hole #: **TH22-1**

Project: **Canal Road - Pender Island Slide**
Location: Canal Road, South Pender Island, B.C.
Datum: UTM NAD 83 ZONE 10U
Alignment:
Northing/Easting: 5400722.13 , 483462.54
Station/Offset:
Elevation: 32.19 m
Coordinates Surveyed

Prepared by: 33450
Thurber Engineering Ltd.
Logged by: JLU
Reviewed by: WRW

Date(s) Drilled: January 19, 2022
Company: Drillwell Enterprises Ltd.
Driller: Tyler Parkhouse
Drill Make/Model: Boart LS250 Sonic
Drilling Method: Sonic



ASPHALT — 0.15m

GP/SP; GRAVEL and SAND, trace silt, gravel to 40 mm diameter, brown, FILL, moist, compact — 0.91m

GM/SM; GRAVEL and SAND, silty, trace clay, gravel to 50 mm diameter, trace rootlets, brown, moist, loose — 2.44m

GC/SC; GRAVEL and SAND, clayey, trace rootlets, brown, sandy clay pockets, COLLUVIUM, moist, compact — 8.23m

GC; GRAVEL, sandy, clayey, gravel to 40 mm diameter, trace rootlets, grey-brown, moist, loose

Easier drilling 8.2 to 10.1 m depth.

Atterberg (Sa#8):
PL:21% LL:30%

Atterberg (Sa#12):
PL:21% LL:30%

SAMPLE NO	SPT	RECOVERY (%)	W%
1			8.4
2			12.5
3	3 3 4 4	25	19.4
4			10.0
5	9 10 9 8	54	9.7
6			7.5
7	15 12 9 7	58	14.9
8			9.8
9	9 10 12 10	63	16.1
10			10.2
11	17 17 16 14	67	17.7
12			13.0
13	6 5 5 8	42	16.3

Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, Bentonite, Drill Cuttings, Slotted, Slough, Piezometer

Final Depth of Hole: 19.8 m
Depth to Top of Rock:

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

SUMMARY LOG

Drill Hole #: **TH22-1**

Prepared by: 33450
Thurber Engineering Ltd.

Logged by: JLU Reviewed by: WRW

Project: **Canal Road - Pender Island Slide**
Location: Canal Road, South Pender Island, B.C.
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Station/Offset:
Elevation: 32.19 m
Coordinates Surveyed

Date(s) Drilled: January 19, 2022
Company: Drillwell Enterprises Ltd.
Driller: Tyler Parkhouse
Drill Make/Model: Boart LS250 Sonic
Drilling Method: Sonic

DEPTH (m)	DRILLING DETAILS	SPT	✕ Pocket Penetrometer ✕ Shear Strength (kPa) 100 200 300 400 / ▲ SPT "N" (BLOWS/300 mm) ▲ / W_P% W% W_L% 20 40 60 80	SAMPLE TYPE	SAMPLE NO	RECOVERY (%)	SOIL SYMBOL	SOIL DESCRIPTION	CLASSIFICATION	COMMENTS TESTING Drillers Estimate {G % S % F %}	MONITORING WELL	ELEVATION (m)
10							▨	10.06m				22
			● 12.1	G	14		▨	SC/SM; SAND, gravelly, silty, clayey, gravel to 50 mm diameter, grey-brown, sandy clay pockets, TILL-like, dry to moist, dense		**Atterberg** (Sa#14): PL:19% LL:30%		
		65	● 17.3	S	15	100	▨					
11	65 blows / 150 mm penetration.						▨					21
			● 4.5	G	16		▨					
12		52	● 19.6	S	17		▨					20
	50 blows / 25 mm penetration.		● 10.0	G	18		▨					
13			● 4.7	G	19		▨					19
14			● 11.6	G	20		▨					18
15							▨		SC/SM			17
			● 12.8	G	21		▨					
16			● 5.0	G	22		▨					16
17							▨					15
			● 7.4	G	23		▨					
18							▨					14
			● 8.5	G	24		▨	End of Hole at required depth (19.8 m). Upon completion of drilling: 50 mm PVC casing installed for SAA installation, backfilled with bentonite-grout. SAA installation on January 28, 2022; Serial Number 247582, active segments from 1.8 m to 19.8 m depth.				
19							▨					13
			● 10.4	G	25		▨	19.81m				
20												

Legend Sample Type: A-Auger B-Becker C-Core G-Grab V-Vane L#-Lab Sample S-Split Spoon O-Odex (air rotary) W-Wash (mud return) T-Shelby Tube

Legend Installation: Sand Grout Cement Bentonite Drill Cuttings Slotted Slough Piezometer

Final Depth of Hole: 19.8 m
Depth to Top of Rock:
Page 2 of 2

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

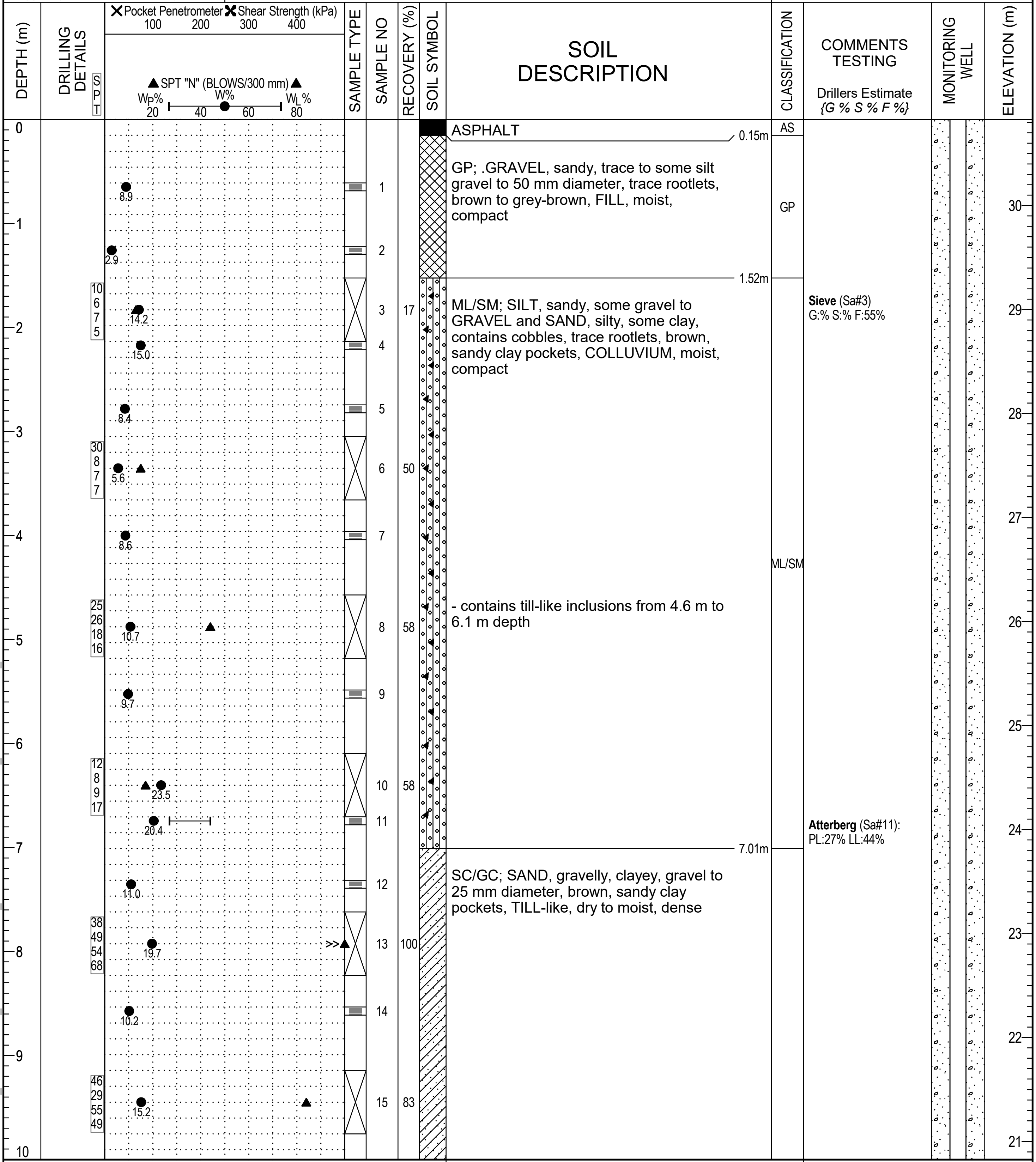
SUMMARY LOG

Drill Hole #: **TH22-2**

Prepared by: 33450
Thurber Engineering Ltd.
Logged by: JLU Reviewed by: WRW

Project: **Canal Road - Pender Island Slide**
Location: Canal Road, South Pender Island, B.C.
Datum: UTM NAD 83 ZONE 10U Alignment:
Northing/Easting: 5400724.36 , 483401.89 Station/Offset:
Elevation: 30.83 m *Coordinates Surveyed*

Date(s) Drilled: February 15-16, 2022
Company: Drillwell Enterprises Ltd.
Driller: Tyler Parkhouse
Drill Make/Model: Boart LS250 Sonic
Drilling Method: Sonic



1				8.9
2				2.9
3	17	10 6 7 5		14.2
4				15.0
5				8.4
6	50	30 8 7 7		5.6
7				8.6
8	58	25 26 18 16		10.7
9				9.7
10	58	12 8 9 17		23.5
11				20.4
12				11.0
13	100	38 49 54 68	>>	19.7
14				10.2
15	83	46 29 55 49		15.2

SOIL DESCRIPTION

ASPHALT — 0.15m — AS

GP; .GRAVEL, sandy, trace to some silt gravel to 50 mm diameter, trace rootlets, brown to grey-brown, FILL, moist, compact — GP

1.52m

ML/SM; SILT, sandy, some gravel to GRAVEL and SAND, silty, some clay, contains cobbles, trace rootlets, brown, sandy clay pockets, COLLUVIUM, moist, compact — ML/SM

- contains till-like inclusions from 4.6 m to 6.1 m depth

7.01m

SC/GC; SAND, gravelly, clayey, gravel to 25 mm diameter, brown, sandy clay pockets, TILL-like, dry to moist, dense

COMMENTS TESTING

Drillers Estimate
{G % S % F %}

Sieve (Sa#3)
G:% S:% F:55%

Atterberg (Sa#11):
PL:27% LL:44%

Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, Bentonite, Drill Cuttings, Slotted, Slough, Piezometer

Final Depth of Hole: 22.9 m
Depth to Top of Rock: 17.4 m

British Columbia
Ministry of Transportation and Infrastructure

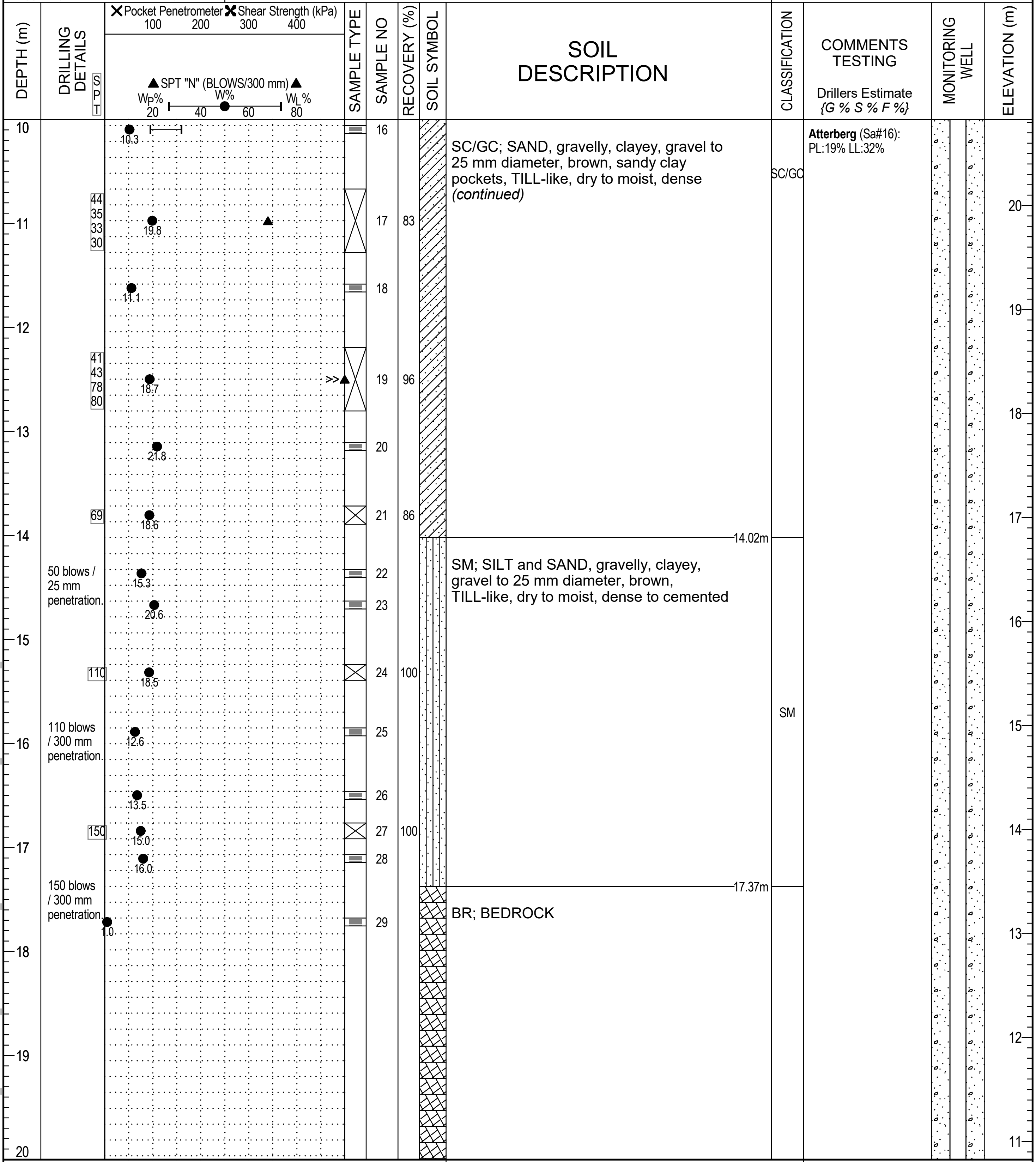
SUMMARY LOG

Drill Hole #: **TH22-2**

Prepared by: 33450
Thurber Engineering Ltd.
Logged by: JLU Reviewed by: WRW

Project: **Canal Road - Pender Island Slide**
Location: Canal Road, South Pender Island, B.C.
Datum: UTM NAD 83 ZONE 10U
Alignment:
Northing/Easting: 5400724.36 , 483401.89
Station/Offset:
Elevation: 30.83 m
Coordinates Surveyed

Date(s) Drilled: February 15-16, 2022
Company: Drillwell Enterprises Ltd.
Driller: Tyler Parkhouse
Drill Make/Model: Boart LS250 Sonic
Drilling Method: Sonic



Sample No	SPT	W%	Recovery (%)
16		10.3	
17	44 35 33 30	19.8	83
18		11.1	
19	41 43 78 80	18.7	96
20		21.8	
21	69	18.6	86
22		15.3	
23		20.6	
24	110	18.5	100
25		12.6	
26		13.5	
27	150	15.0	100
28		16.0	
29		1.0	

Drilling details:

50 blows / 25 mm penetration.

110 blows / 300 mm penetration.

150 blows / 300 mm penetration.

Soil description	Base depth	Classification
SC/GC; SAND, gravelly, clayey, gravel to 25 mm diameter, brown, sandy clay pockets, TILL-like, dry to moist, dense *(continued)*	14.02m	SC/GC
SM; SILT and SAND, gravelly, clayey, gravel to 25 mm diameter, brown, TILL-like, dry to moist, dense to cemented	17.37m	SM
BR; BEDROCK		

Atterberg (Sa#16):
PL:19% LL:32%

Legend Sample Type: ◫ A-Auger ⊥ B-Becker ▮ C-Core ▤ G-Grab ⊔ V-Vane ● L#-Lab Sample ☒ S-Split Spoon ⁙ O-Odex (air rotary) ▩ W-Wash (mud return) ▥ T-Shelby Tube

Legend Installation: ⁙ Sand ⋰ Grout ▨ Cement ■ Bentonite ▨ Drill Cuttings ⊞ Slotted ▩ Slough ◈ Piezometer

Final Depth of Hole: 22.9 m
Depth to Top of Rock: 17.4 m

SUMMARY LOG

Drill Hole #: TH22-2

Prepared by: 33450 Thurber Engineering Ltd.	Project: **Canal Road - Pender Island Slide**	Date(s) Drilled: February 15-16, 2022
	Location: Canal Road, South Pender Island, B.C.	Company: Drillwell Enterprises Ltd.
	Datum: UTM NAD 83 ZONE 10U Alignment:	Driller: Tyler Parkhouse
	Northing/Easting: 5400724.36 , 483401.89 Station/Offset:	Drill Make/Model: Boart LS250 Sonic
Logged by: JLU Reviewed by: WRW	Elevation: 30.83 m *Coordinates Surveyed*	Drilling Method: Sonic

DEPTH (m)	DRILLING DETAILS / SPT	✕Pocket Penetrometer ✕Shear Strength (kPa) 100 200 300 400 ▲SPT "N" (BLOWS/300 mm)▲ Wp% W% WL% 20 40 60 80	SAMPLE TYPE	SAMPLE NO	RECOVERY (%)	SOIL SYMBOL	SOIL DESCRIPTION	CLASSIFICATION	COMMENTS TESTING Drillers Estimate {G % S % F %}	MONITORING WELL	ELEVATION (m)
20							BR; BEDROCK *(continued)*	BR			
21											10
22			G	30							9
22.86m							End of Hole at required depth (22.9 m). Upon completion of drilling: 50 mm PVC casing installed for SAA installation, backfilled with bentonite-grout. SAA installation on February 24, 2022; Serial Number 247587, active segments from 1.6 m to 22.9 m depth.				8
23											
24											7
25											6
26											5
27											4
28											3
29											2
30											1

Legend Sample Type: A-Auger; B-Becker; C-Core; G-Grab; V-Vane; L#-Lab Sample; S-Split Spoon; O-Odex (air rotary); W-Wash (mud return); T-Shelby Tube

Legend Installation: Sand; Grout; Cement; Bentonite; Drill Cuttings; Slotted; Slough; Piezometer

Final Depth of Hole: 22.9 m
Depth to Top of Rock: 17.4 m

British Columbia | Ministry of Transportation and Infrastructure

SUMMARY LOG

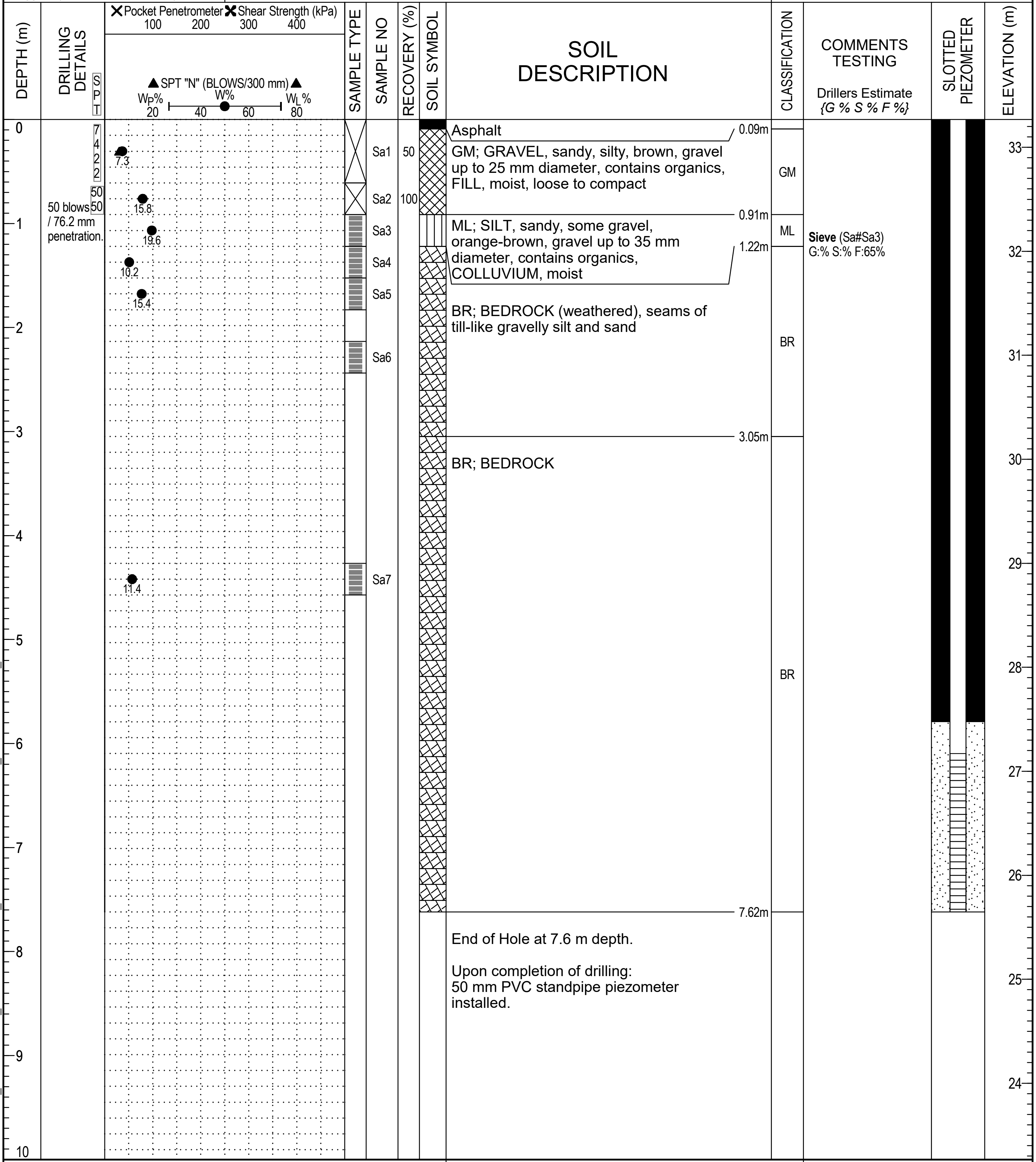
Drill Hole #: **TH22-3**

Prepared by: 33450
Thurber Engineering Ltd.

Logged by: KPJ Reviewed by: WRW

Project: **Canal Road - Pender Island Slide**
Location: Canal Road, South Pender Island, B.C.
Datum: UTM NAD 83 ZONE 10U
Alignment:
Northing/Easting: 5400714.18 , 483543.4
Station/Offset:
Elevation: 33.27 m
Coordinates Surveyed

Date(s) Drilled: May 2, 2022
Company: Drillwell Enterprises Ltd.
Driller: Tyler Parkhouse
Drill Make/Model: Boart LS250 Sonic
Drilling Method: Sonic



Depth	Soil Description	Classification	Comments / Testing
	Asphalt		
0.09m	GM; GRAVEL, sandy, silty, brown, gravel up to 25 mm diameter, contains organics, FILL, moist, loose to compact	GM	
0.91m	ML; SILT, sandy, some gravel, orange-brown, gravel up to 35 mm diameter, contains organics, COLLUVIUM, moist	ML	**Sieve** (Sa#Sa3) G:% S:% F:65%
1.22m	BR; BEDROCK (weathered), seams of till-like gravelly silt and sand	BR	
3.05m	BR; BEDROCK	BR	
7.62m	End of Hole at 7.6 m depth. Upon completion of drilling: 50 mm PVC standpipe piezometer installed.		

Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, Bentonite, Drill Cuttings, Slotted, Slough, Piezometer

Final Depth of Hole: 7.6 m
Depth to Top of Rock: 1.2 m

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

SUMMARY LOG

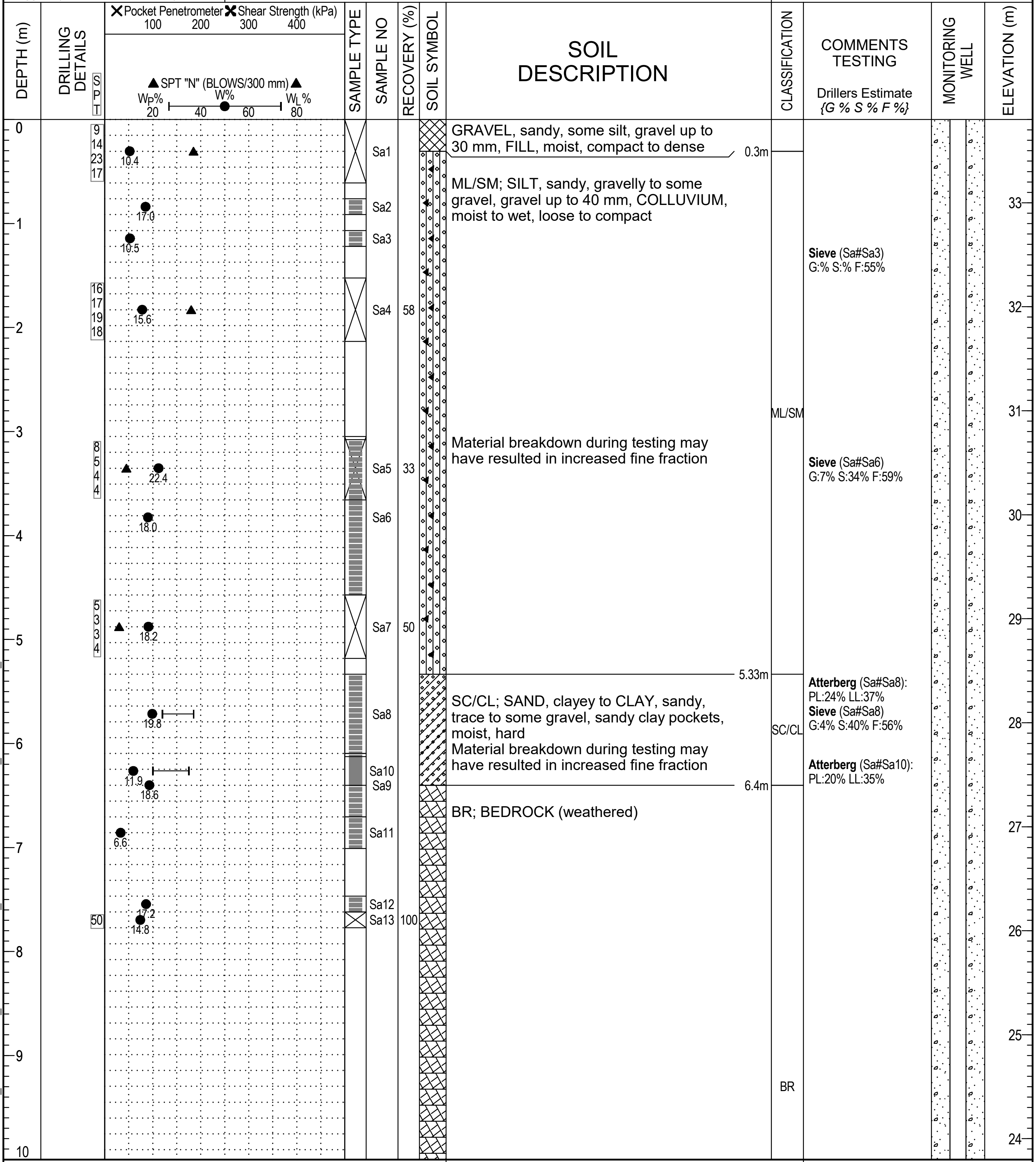
Drill Hole #: **TH22-4**

Project: **Canal Road - Pender Island Slide**
Location: Canal Road, South Pender Island, B.C.

Prepared by: Thurber Engineering Ltd. 33450
Logged by: KPJ Reviewed by: WRW

Datum: UTM NAD 83 ZONE 10U
Alignment:
Northing/Easting: 5400712.24 , 483499.68
Station/Offset:
Elevation: 33.8 m
Coordinates Surveyed

Date(s) Drilled: May 3, 2022
Company: Drillwell Enterprises Ltd.
Driller: Tyler Parkhouse
Drill Make/Model: Boart LS250 Sonic
Drilling Method: Sonic



GRAVEL, sandy, some silt, gravel up to 30 mm, FILL, moist, compact to dense — 0.3m

ML/SM; SILT, sandy, gravelly to some gravel, gravel up to 40 mm, COLLUVIUM, moist to wet, loose to compact

Material breakdown during testing may have resulted in increased fine fraction — 5.33m

SC/CL; SAND, clayey to CLAY, sandy, trace to some gravel, sandy clay pockets, moist, hard
Material breakdown during testing may have resulted in increased fine fraction — 6.4m

BR; BEDROCK (weathered)

Classification: ML/SM; SC/CL; BR

Sieve (Sa#Sa3)
G:% S:% F:55%

Sieve (Sa#Sa6)
G:7% S:34% F:59%

Atterberg (Sa#Sa8):
PL:24% LL:37%
Sieve (Sa#Sa8)
G:4% S:40% F:56%

Atterberg (Sa#Sa10):
PL:20% LL:35%

Legend Sample Type: A-Auger; B-Becker; C-Core; G-Grab; V-Vane; L#-Lab Sample; S-Split Spoon; O-Odex (air rotary); W-Wash (mud return); T-Shelby Tube

Legend Installation: Sand; Grout; Cement; ■ Bentonite; Drill Cuttings; Slotted; Slough; Piezometer

Final Depth of Hole: 16.0 m
Depth to Top of Rock: 6.4 m

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

SUMMARY LOG

Drill Hole #: TH22-4

Prepared by: 33450
Thurber Engineering Ltd.
Logged by: KPJ Reviewed by: WRW

Project: **Canal Road - Pender Island Slide**
Location: Canal Road, South Pender Island, B.C.
Datum: UTM NAD 83 ZONE 10U
Alignment:
Northing/Easting: 5400712.24 , 483499.68
Station/Offset:
Elevation: 33.8 m
Coordinates Surveyed

Date(s) Drilled: May 3, 2022
Company: Drillwell Enterprises Ltd.
Driller: Tyler Parkhouse
Drill Make/Model: Boart LS250 Sonic
Drilling Method: Sonic

✕ Pocket Penetrometer ✕ Shear Strength (kPa) 100 200 300 400
▲ SPT "N" (BLOWS/300 mm) ▲
WP% W% WL% 20 40 60 80

DEPTH (m)	DRILLING DETAILS	SPT	SAMPLE NO	RECOVERY (%)	SOIL DESCRIPTION	CLASSIFICATION	COMMENTS TESTING Drillers Estimate *{G % S % F %}*
10					BR; BEDROCK (weathered) *(continued)*		
	50 blows / 101.6 mm penetration.	50	Sa14	167			
			Sa17				
12.19m					BR; BEDROCK	BR	
			Sa16				
			Sa15				
			Sa18				
16.0m					End of Hole at 16.0 m depth. Upon completion of drilling: 50 mm PVC casing installed for SAA installation, backfilled with bentonite-grout. SAA installation on May 16, 2022; Serial Number 247601, active segments from 1.7 m to 16.0 m depth.		

DEPTH (m): 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

ELEVATION (m): 23, 22, 21, 20, 19, 18, 17, 16, 15, 14

MONITORING WELL

Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, ■ Bentonite, Drill Cuttings, Slotted, Slough, Piezometer

Final Depth of Hole: 16.0 m
Depth to Top of Rock: 6.4 m

THURBER

APPENDIX C PHOTOS

PHOTO 1: Looking east at the lane closure enacted in the downslope lane (19/1/2022) – Photo by Jill Usher

PHOTO 2: SPT sample Sa 5 at 3.0 m depth in TH22-01 (Colluvium deposit) (19/1/2022) – Photo by Jill Usher

PHOTO 3: Sonic drill rig set up at TH22-01 (19/1/2022) – Photo by Jill Usher

PHOTO 4: SPT sample Sa 15 at 10.7 m in TH22-01 (Till-like deposit) (19/1/2022) – Photo by Jill Usher

PHOTO 5: Installation of SAA 1 in TH22-01 (28/1/2022) – Photo by Jill Usher

PHOTO 6: Installation of the datalogger locker for remote readings (2/2/2022) – Photo by Liam Costerton

PHOTO 7: TH22-01 (flush mount) in relation to the datalogger locker (15/2/2022) – Photo by Jill Usher

PHOTO 8: BC Hydro shutting down power prior to drilling TH22-02 (15/2/2022) – Photo by Jill Usher

PHOTO 9: Sonic drill set up at TH22-02 (15/2/2022) – Photo by Jill Usher

PHOTO 10: Sonic sample from 0.0 m to 1.5 m depth in TH22-02 (Fill) (15/2/2022) – Photo by Jill Usher

PHOTO 11: SPT sample Sa 8 at 4.6 m depth in TH22-02 (Colluvium) (15/2/2022) – Photo by Jill Usher

PHOTO 12: SPT sample Sa 17 at 10.7 m depth in TH22-02 (Till-like) (15/2/2022) – Photo by Jill Usher

PHOTO 13: Sonic sample from 18.3 m to 22.9 m in TH22-02 (Bedrock) (16/2/2022) – Photo by Jill Usher

PHOTO 14: Shale sample from TH22-02 (16/2/2022) – Photo by Jill Usher

PHOTO 15: Example of cracking at the west end of site (24/2/2022) – Photo by Jill Usher

PHOTO 16: Compression clamp installed at the top of SAA 2 within the flush mount (TH22-02) – Photo by Jill Usher

PHOTO 17: Sonic sample from 1.5 m to 3.0 m depth in TH22-03 (Weathered Bedrock) (2/5/2022) – Photo by Khal Joyce

PHOTO 18: Sonic sample from 3.0 m to 4.6 m depth in TH22-03 (Bedrock) (2/5/2022) – Photo by Khal Joyce

PHOTO 19: SPT sample Sa 4 at 1.5 m depth in TH22-04 (Colluvium) (3/5/2022) – Photo by Khal Joyce

PHOTO 20: Sonic sample from 10.7 m to 12.2 m depth in TH22-04 (Weathered Bedrock) (3/5/2022) – Photo by Khal Joyce

PHOTO 21: Siltstone bedrock outcrop in the upslope ditch at ~Sta. 100+300 (4/5/2022) – Photo by Khal Joyce

PHOTO 22: SAA 3 installation at TH22-04. (16/5/2022) – Photo by Jill Usher

PHOTO 23: West Conglomerate Wedge (16/12/2022) – Photo by Warren Wunderlick

PHOTO 24: East Conglomerate Wedge. (16/12/2022) – Photo by Warren Wunderlick

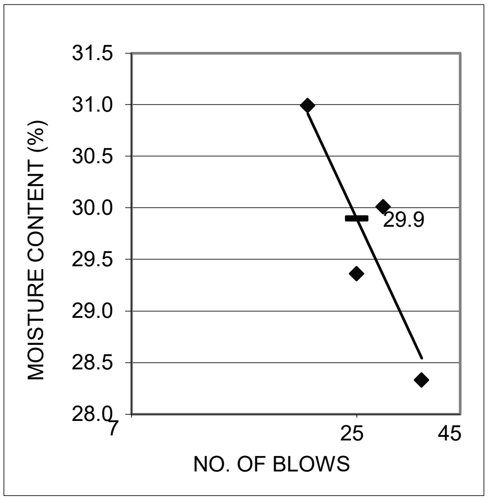
APPENDIX D LAB TESTING

Liquid Limit, Plastic Limit & Plasticity Index of Soils
ASTM D4318

Client:	BC MOTI			
Project:	Canal Road			
Project No:	33450		**Date Tested:**	7-Mar-22
Test Hole:	TH22-1	Depth: 5.49 m	**Tested By:**	BTS
Sample No:	8		**Checked By:**	JSH

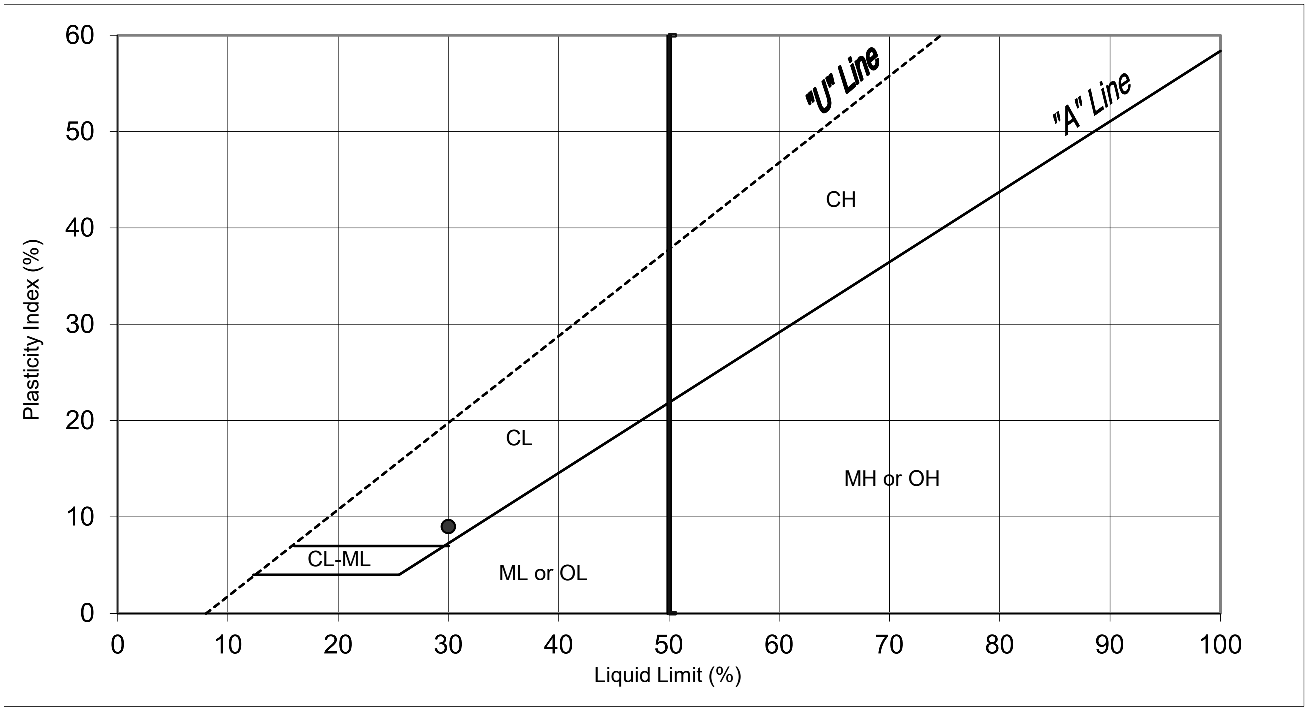
LIQUID LIMIT

Trial No:	1	2	3	4
No of Blows:	36	29	25	19
Container No.	253	207	220	223
Wet Soil + Container	25.08	27.02	25.8	27.24
Dry Soil + Container	22.55	24.13	23.04	23.93
Wt. Of Container	13.62	14.5	13.64	13.25
Moisture Content	28.3	30.0	29.4	31.0



PLASTIC LIMIT

	1	2		**AVERAGE**
Container No.	212	249		
Wet Soil + Container	21.21	20.69		
Dry Soil + Container	19.94	19.45		
Wt. Of Container	13.95	13.68		
Moisture Content	21.2	21.5		**21.3**



REMARKS	As received MC % =	9.8

Liquid Limit:	**30**
Plastic Limit:	**21**
Plasticity Index:	**9**
Liquidity Index:	**-1**
USC Classification:	**ML**

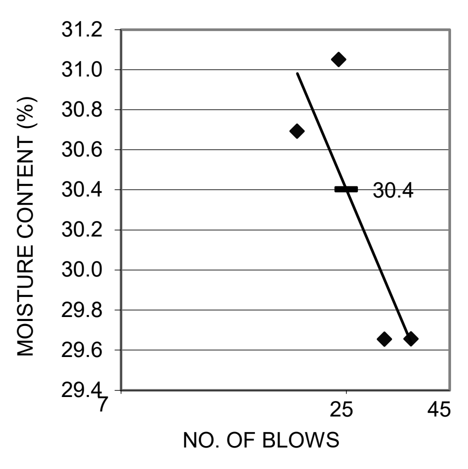
THURBER ENGINEERING LTD.

Liquid Limit, Plastic Limit & Plasticity Index of Soils
ASTM D4318

Client:	BC MOTI			
Project:	Canal Road			
Project No:	33450		**Date Tested:**	7-Mar-22
Test Hole:	TH22-1	Depth: 8.53 m	**Tested By:**	BTS
Sample No:	12		**Checked By:**	JSH

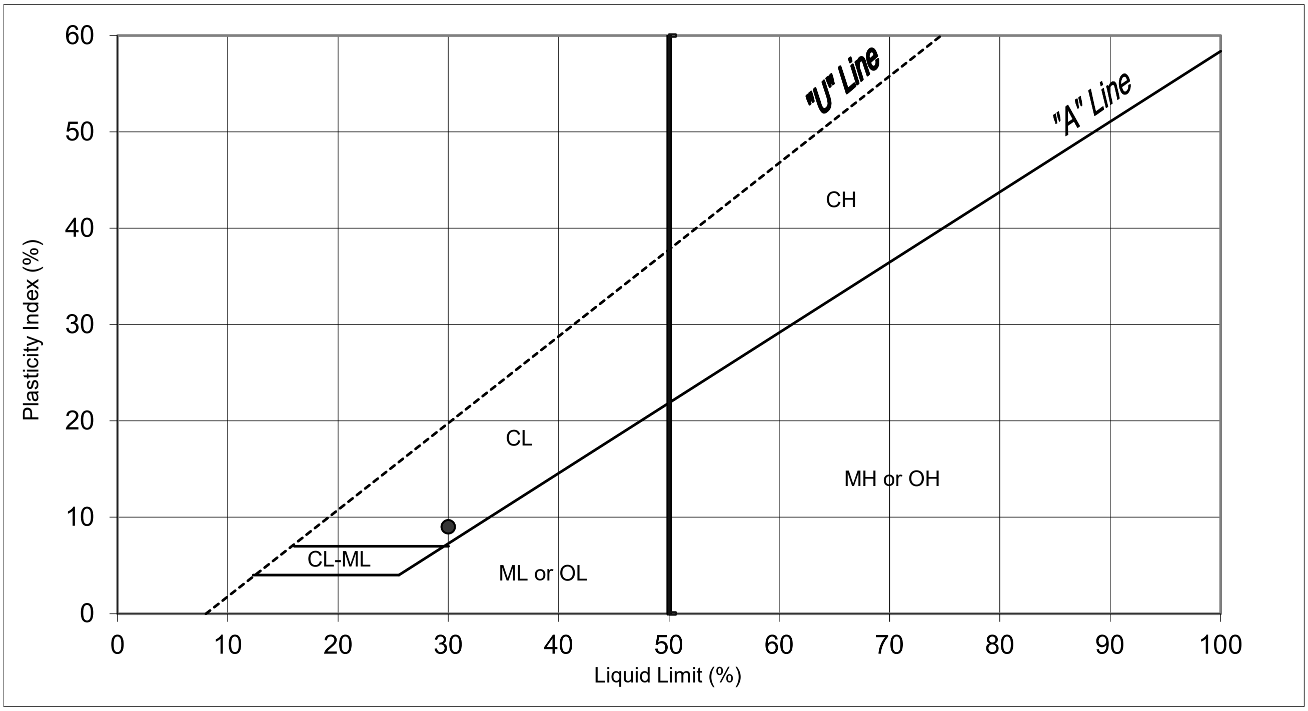
LIQUID LIMIT

Trial No:	1	2	3	4
No of Blows:	36	31	24	19
Container No.	226	227	245	206
Wet Soil + Container	26.85	24.39	27.55	24.62
Dry Soil + Container	24	21.81	24.24	22.14
Wt. Of Container	14.39	13.11	13.58	14.06
Moisture Content	29.7	29.7	31.1	30.7



PLASTIC LIMIT

	1	2		**AVERAGE**
Container No.	233	219		
Wet Soil + Container	19.39	21.57		
Dry Soil + Container	18.4	20.35		
Wt. Of Container	13.76	14.37		
Moisture Content	21.3	20.4		**20.9**



REMARKS As received MC % = 13.0

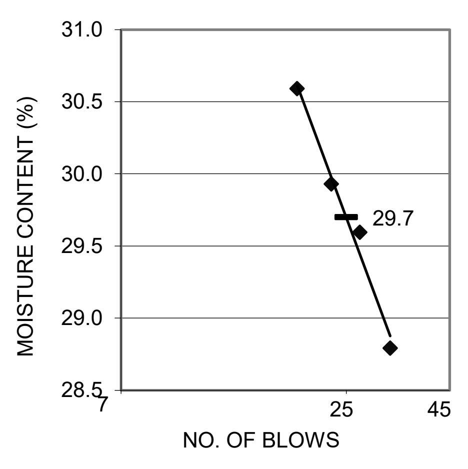
Liquid Limit:	**30**
Plastic Limit:	**21**
Plasticity Index:	**9**
Liquidity Index:	**-1**
USC Classification:	**ML**

Liquid Limit, Plastic Limit & Plasticity Index of Soils
ASTM D4318

Client:	BC MOTI			
Project:	Canal Road			
Project No:	33450		**Date Tested:**	7-Mar-22
Test Hole:	TH22-1	Depth: 10.36 m	**Tested By:**	BTS
Sample No:	14		**Checked By:**	JSH

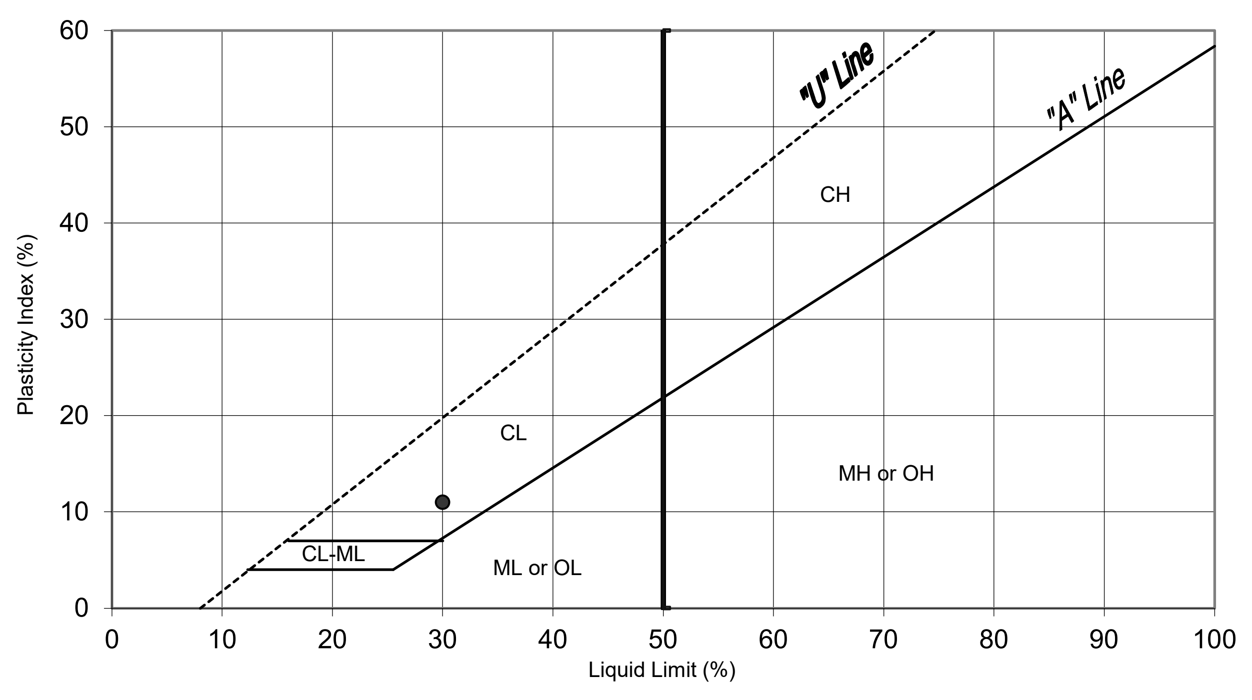
LIQUID LIMIT

Trial No:	1	2	3	4
No of Blows:	32	27	23	19
Container No.	251	252	221	208
Wet Soil + Container	25.85	27.79	25.19	25.21
Dry Soil + Container	23.04	24.57	22.64	22.57
Wt. Of Container	13.28	13.69	14.12	13.94
Moisture Content	28.8	29.6	29.9	30.6



PLASTIC LIMIT

	1	2		AVERAGE
Container No.	218	280		
Wet Soil + Container	22.23	21.59		
Dry Soil + Container	20.96	20.33		
Wt. Of Container	14.06	13.59		
Moisture Content	18.4	18.7		**18.6**



REMARKS As received MC % = 12.1

Liquid Limit:	**30**
Plastic Limit:	**19**
Plasticity Index:	**11**
Liquidity Index:	**-1**
USC Classification:	**ML**

THURBER ENGINEERING LTD.

Liquid Limit, Plastic Limit & Plasticity Index of Soils
ASTM D4318

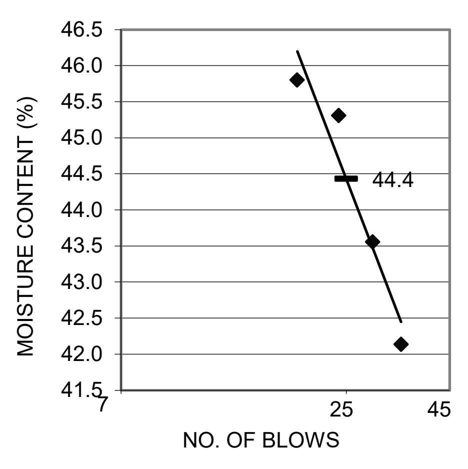
Client: BC MOTI
Project: Canal Road
Project No: 33450
Test Hole: TH22-2 Depth: 6.71 m
Sample No: 11
Date Tested: 7-Mar-22
Tested By: BTS
Checked By: JSH

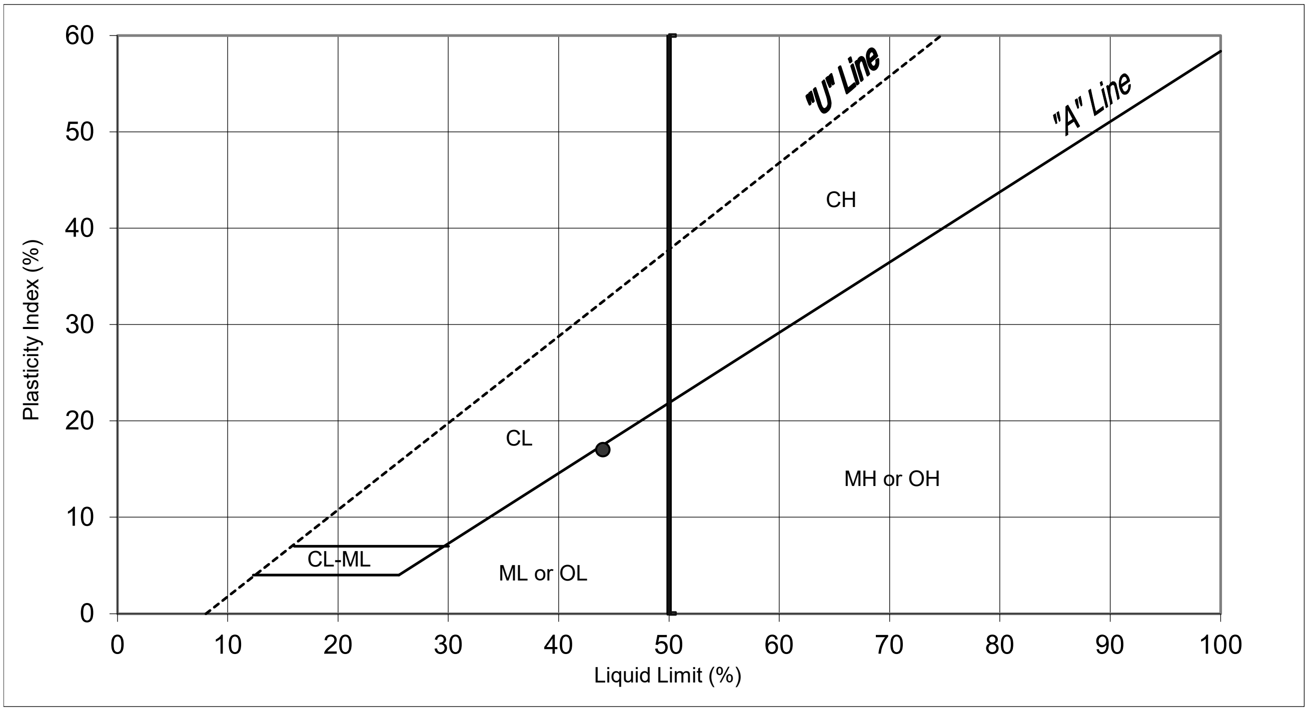
LIQUID LIMIT

Trial No:	1	2	3	4
No of Blows:	34	29	24	19
Container No.	241	215	242	232
Wet Soil + Container	24.86	26.62	25.67	25.53
Dry Soil + Container	21.51	22.97	21.95	21.82
Wt. Of Container	13.56	14.59	13.74	13.72
Moisture Content	42.1	43.6	45.3	45.8

PLASTIC LIMIT

	1	2		**AVERAGE**
Container No.	203	216		
Wet Soil + Container	20.21	19.8		
Dry Soil + Container	18.82	18.45		
Wt. Of Container	13.69	13.48		
Moisture Content	27.1	27.2		**27.1**





REMARKS As received MC % = 20.4

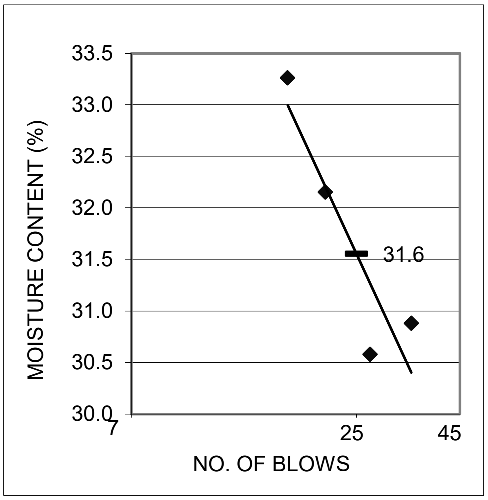
Liquid Limit: 44
Plastic Limit: 27
Plasticity Index: 17
Liquidity Index: 0
USC Classification: ML

Liquid Limit, Plastic Limit & Plasticity Index of Soils
ASTM D4318

Client: BC MOTI
Project: Canal Road
Project No: 33450
Test Hole: TH22-2 Depth: 10.06 m
Sample No: 16
Date Tested: 7-Mar-22
Tested By: BTS
Checked By: JSH

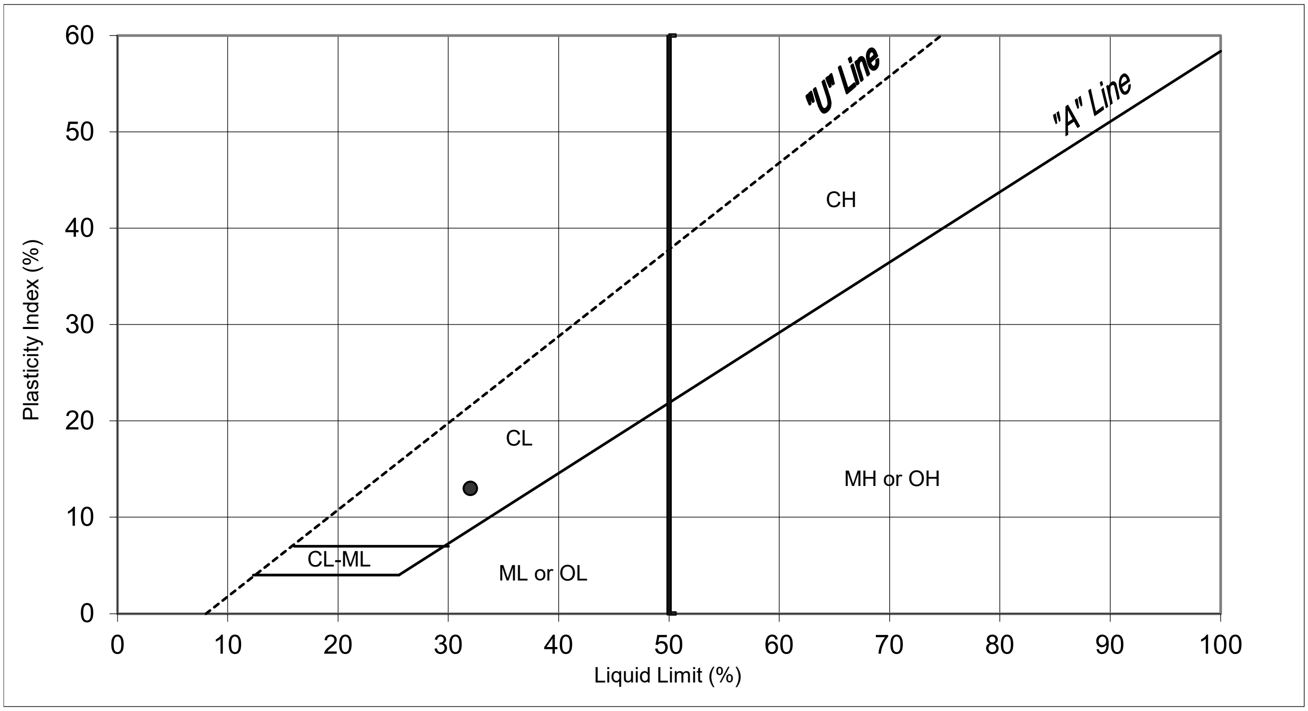
LIQUID LIMIT

Trial No:	1	2	3	4
No of Blows:	34	27	21	17
Container No.	234	258	222	256
Wet Soil + Container	26.13	24.69	28.97	26.21
Dry Soil + Container	23.15	22.1	25.52	23.02
Wt. Of Container	13.5	13.63	14.79	13.43
Moisture Content	30.9	30.6	32.2	33.3



PLASTIC LIMIT

	1	2		AVERAGE
Container No.	255	259		
Wet Soil + Container	18.24	19.38		
Dry Soil + Container	17.47	18.45		
Wt. Of Container	13.52	13.53		
Moisture Content	19.5	18.9		**19.2**



REMARKS As received MC % = 10.3

Liquid Limit: 32
Plastic Limit: 19
Plasticity Index: 13
Liquidity Index: -1
USC Classification: CL

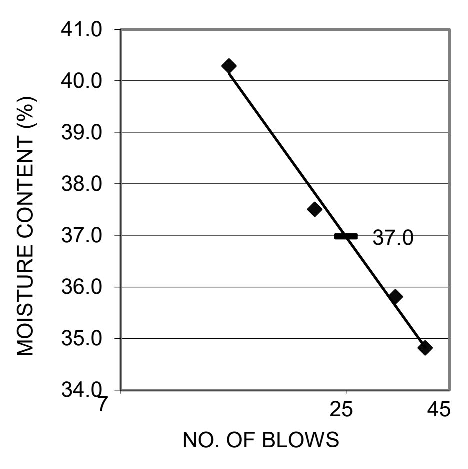
THURBER ENGINEERING LTD.

Liquid Limit, Plastic Limit & Plasticity Index of Soils
ASTM D4318

Client: BC MOTI
Project: Canal Road
Project No: 33450
Test Hole: TH22-4 Depth: 5.33 m - 6.10 m
Sample No: 8
Date Tested: 30-May-22
Tested By: JCE
Checked By: JSH

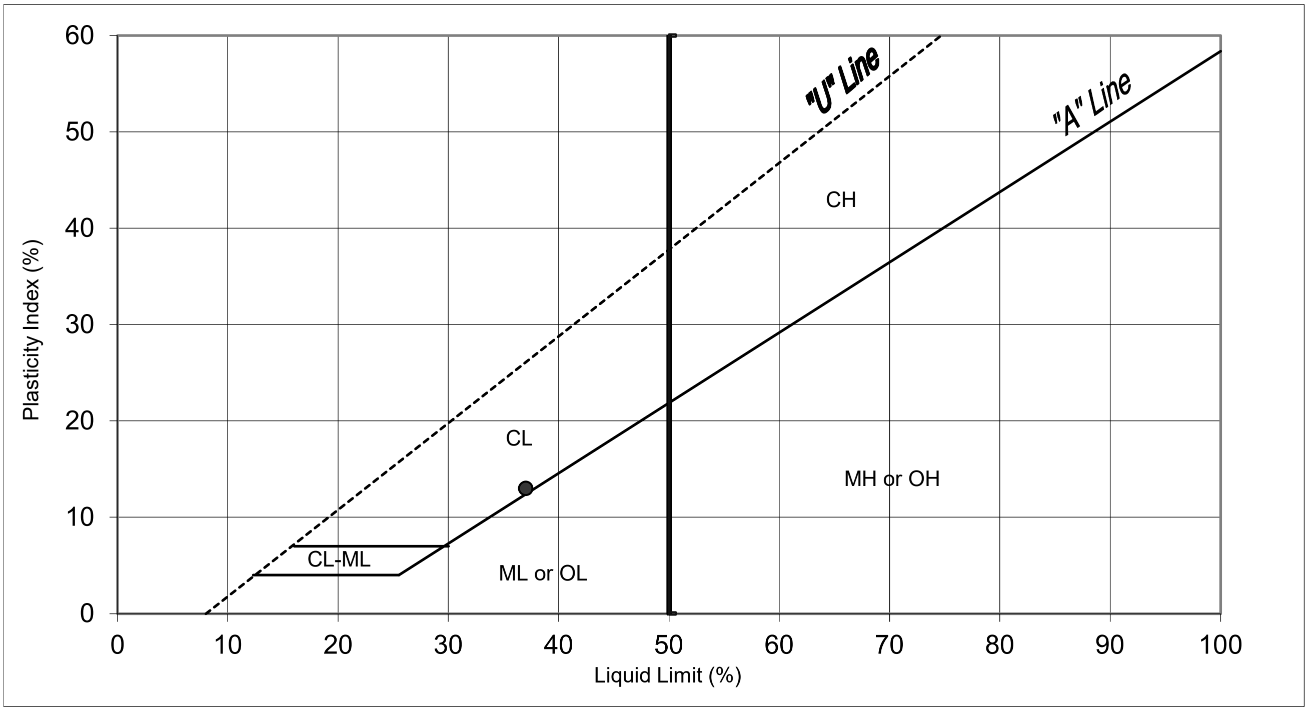
LIQUID LIMIT

Trial No:	1	2	3	4
No of Blows:	39	33	21	13
Container No.	256	243	227	232
Wet Soil + Container	32.79	32.43	33.93	34.14
Dry Soil + Container	27.79	27.52	28.24	28.27
Wt. Of Container	13.43	13.81	13.07	13.7
Moisture Content	34.8	35.8	37.5	40.3



PLASTIC LIMIT

	1	2		**AVERAGE**
Container No.	245	247		
Wet Soil + Container	25.05	25.96		
Dry Soil + Container	22.83	23.63		
Wt. Of Container	13.57	13.74		
Moisture Content	24.0	23.6		**23.8**



REMARKS As received MC % = 19.8

Liquid Limit:	**37**
Plastic Limit:	**24**
Plasticity Index:	**13**
Liquidity Index:	**0**
USC Classification:	**CL**

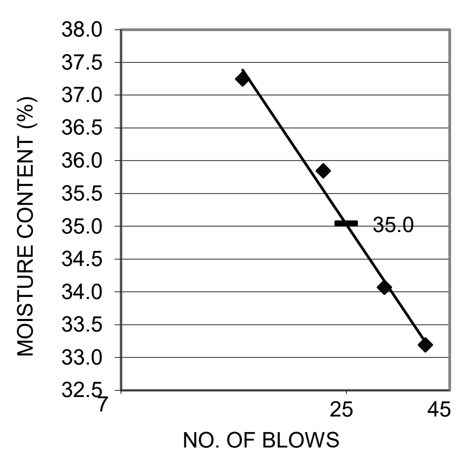
THURBER ENGINEERING LTD.

Liquid Limit, Plastic Limit & Plasticity Index of Soils
ASTM D4318

Client: BC MOTI
Project: Canal Road
Project No: 33450
Test Hole: TH22-4 Depth: 6.10 m - 6.40 m
Sample No: 10
Date Tested: 27-May-22
Tested By: JCE
Checked By:

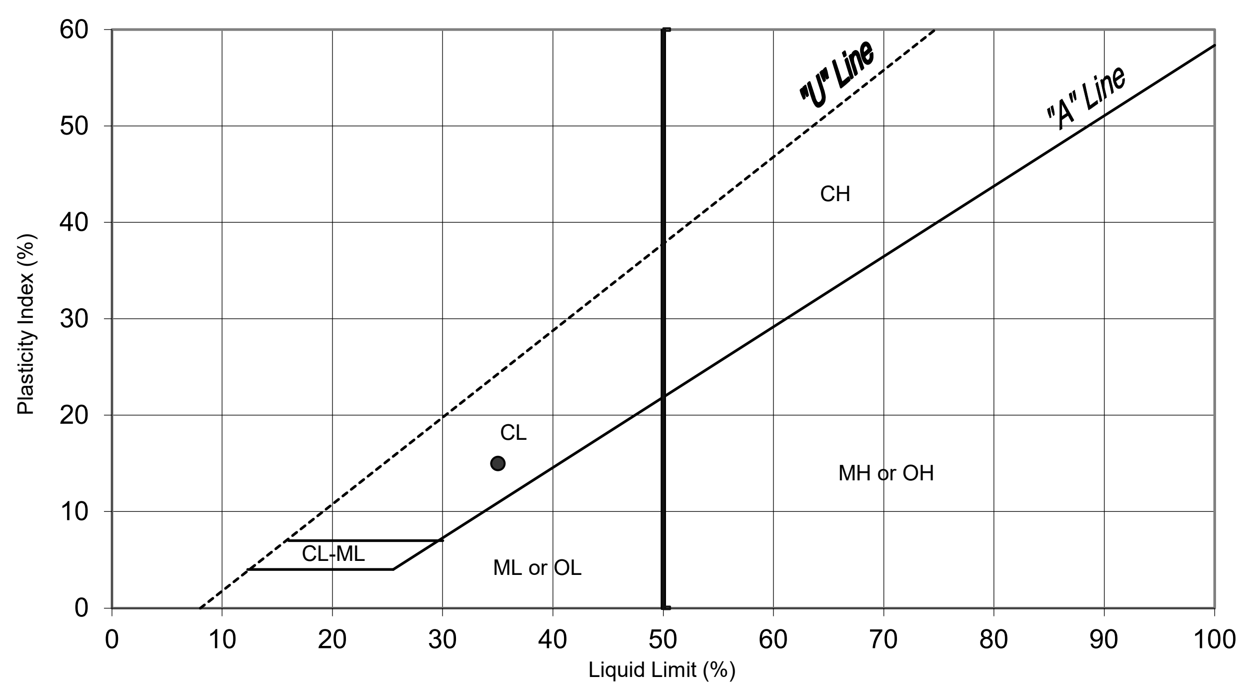
LIQUID LIMIT

Trial No:	1	2	3	4
No of Blows:	39	31	22	14
Container No.	223	246	244	236
Wet Soil + Container	34.73	31.58	32.5	34.08
Dry Soil + Container	29.37	26.94	27.51	28.4
Wt. Of Container	13.22	13.32	13.59	13.15
Moisture Content	33.2	34.1	35.8	37.2



PLASTIC LIMIT

	1	2		AVERAGE
Container No.	293	619		
Wet Soil + Container	29.5	29.99		
Dry Soil + Container	26.89	27.25		
Wt. Of Container	13.72	13.61		
Moisture Content	19.8	20.1		**20.0**



REMARKS As received MC % = 11.9

Liquid Limit:	**35**
Plastic Limit:	**20**
Plasticity Index:	**15**
Liquidity Index:	**-1**
USC Classification:	**CL**

GRADATION ANALYSIS

THURBER ENGINEERING LTD.
Suite 2302, 4476 Markham Road, Victoria, BC V8Z 7X8   Phone (250) 727-2201

Client: BC MOTI
Project Number: 33450
Project: Canal Road - Pender Island
Date: 8-Jun-22

Sample Source: TH22-04, Sa 6, 3.05 m - 4.57 m
Material Type: Grab sample
Specification:
Sample Description: grey, moist, sandy SILT and CLAY, with a trace of gravel
Water Content As Received: 15.5%

Date Tested: 26-May-22
Sampled by: KPJ
Date Sampled:
Test Method: ASTM
Series No.: 22-1

GRAVEL (FROM SIEVE)

Sieve No.	Opening (mm)	Percent Passing	Gradation Limits Max	min
	75			
	50			
	37.5			
	25			
	19			
	12.5	100.0		
	9.5	99.8		
	4.75	93.1		

Gravel: 6.9%
Sand: 34.5%
Fines: 58.6%

Percent Crush: N/A
Faces Counted: 0

SAND & FINES (FROM SIEVE & WASH)

Sieve No.	Opening (mm)	Percent Passing	Gradation Limits Max	min
	2.36	78.0		
	1.18	68.2		
	0.6	63.7		
	0.3	61.8		
	0.15	60.5		
	0.075	58.6		

SILT AND CLAY (FROM HYDROMETER)

Silt			
Clay	-		
Total Fines:	58.6%		

Comments: Material was weak/weathered. Particles continued to break down as GSA was being performed.

Checked By: JSH

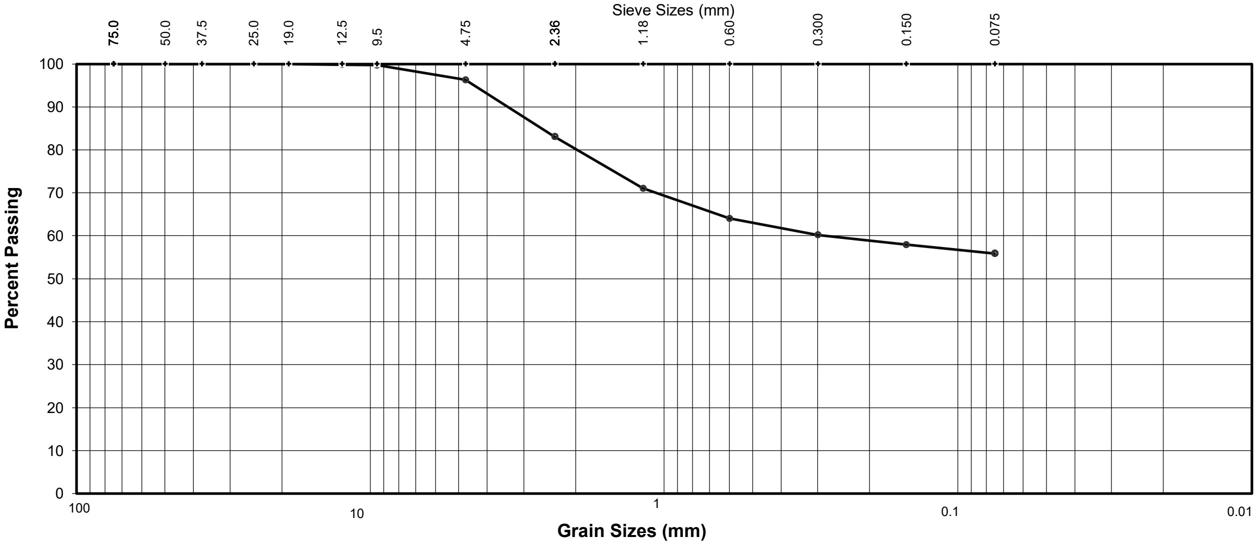
THURBER ENGINEERING LTD.
Suite 2302, 4476 Markham Road, Victoria, BC V8Z 7X8   Phone (250) 727-2201

GRADATION ANALYSIS

Client: BC MOTI
Project Number: 33450
Project: Canal Road - Pender Island
Date: 9-Jun-22

Sample Source: TH22-04, Sa 8, 5.33 m - 6.10 m
Material Type: Grab sample
Specification:
Sample Description: grey, moist, SAND, SILT and CLAY, with a trace of gravel
Water Content As Received: 20.9%

Date Tested: 26-May-22
Sampled by: KPJ
Date Sampled:
Test Method: ASTM
Series No.: 22-2



GRAVEL (FROM SIEVE)

Sieve No.	Opening (mm)	Percent Passing	Gradation Limits Max	min
	75			
	50			
	37.5			
	25			
	19	100.0		
	12.5	99.8		
	9.5	99.7		
	4.75	96.3		

SAND & FINES (FROM SIEVE & WASH)

Sieve No.	Opening (mm)	Percent Passing	Gradation Limits Max	min
	2.36	83.1		
	1.18	71.0		
	0.6	64.1		
	0.3	60.2		
	0.15	57.9		
	0.075	55.9		

SILT AND CLAY (FROM HYDROMETER)

Silt			
Clay	-		
Total Fines:	55.9%		

Gravel:	3.7%	Percent Crush:	N/A
Sand:	40.4%	Faces Counted:	0
Fines:	55.9%		

Comments: Material was weak/weathered. Particles continued to break down as GSA was being performed.

Checked By:

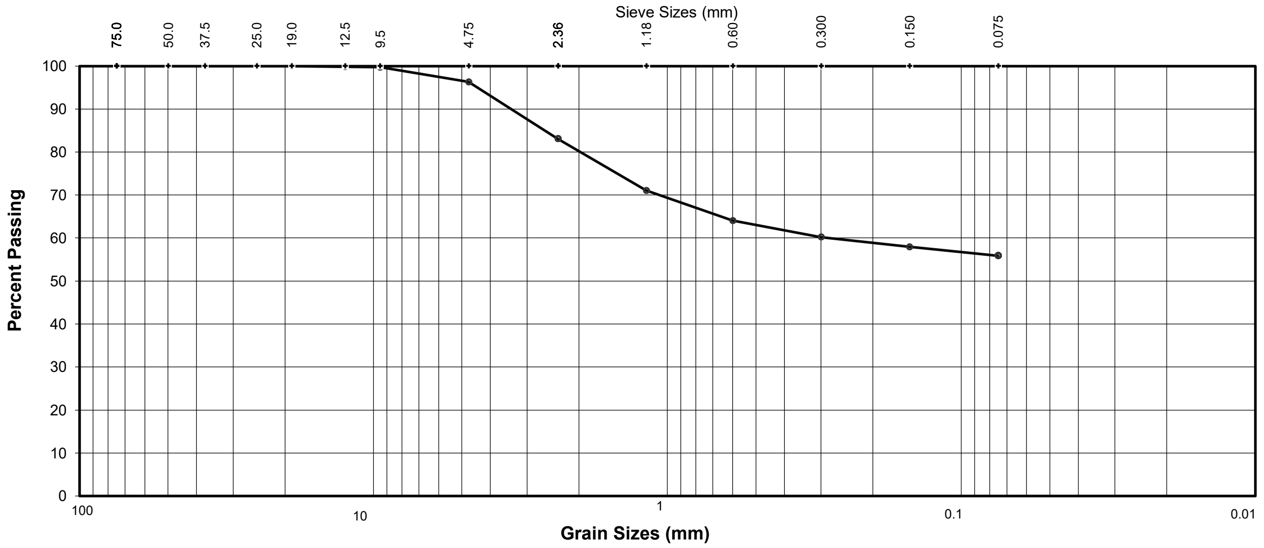
THURBER ENGINEERING LTD.
Suite 2302, 4476 Markham Road, Victoria, BC V8Z 7X8   Phone (250) 727-2201

GRADATION ANALYSIS

Client: BC MOTI
Project Number: 33450
Project: Canal Road - Pender Island
Date: 9-Jun-22

Sample Source: TH22-04, Sa 8, 5.33 m - 6.10 m
Material Type: Grab sample
Specification:
Sample Description: grey, moist, SAND, SILT and CLAY, with a trace of gravel
Water Content As Received: 20.9%

Date Tested: 26-May-22
Sampled by: KPJ
Date Sampled:
Test Method: ASTM
Series No.: 22-2



GRAVEL (FROM SIEVE)

Sieve No.	Opening (mm)	Percent Passing	Gradation Limits Max	min
	75			
	50			
	37.5			
	25			
	19	100.0		
	12.5	99.8		
	9.5	99.7		
	4.75	96.3		

SAND & FINES (FROM SIEVE & WASH)

Sieve No.	Opening (mm)	Percent Passing	Gradation Limits Max	min
	2.36	83.1		
	1.18	71.0		
	0.6	64.1		
	0.3	60.2		
	0.15	57.9		
	0.075	55.9		

SILT AND CLAY (FROM HYDROMETER)

Silt			
Clay	-		
Total Fines:	55.9%		

Gravel: 3.7%
Sand: 40.4%
Fines: 55.9%

Percent Crush: N/A
Faces Counted: 0

Comments: Material was weak/weathered. Particles continued to break down as GSA was being performed.

Checked By: JSH

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request.

THURBER

Project:	**Canal Road - Pender Island (Job #33450)**
Apparatus:	RocTest Telemac Point Load Tester, Model PIL-7 (Recommended Operating Range: 0 to 30 MPa)
Jack Piston Effective Area	9.48 cm2
Test Reference:	ASTM D5731-08

Sample #	**Sample Source**	**Thickness (D)**	**Length (L)**	**Width (W)**	**Dimension Checks**			**Peak Gauge Pressure**	**Validity**	**Failure Load (P)**	**De^2**	**De**	**Is**	**F**	**Is(50)**	**K***	**Estimated UCS**
					D/W [0.3,1]	L>0.5D	Pass/Fail										
		mm	mm	mm	-	mm		MPa		kN	mm^2	mm	MPa	-	MPa	-	MPa
1	TH22-04 ~14.2 m depth	47	62	84	0.6	24	Pass	8.88	Valid	8.42	5027	71	1.7	1.2	2.0	24.5	48
2	TH22-04 ~14.2 m depth	52	53	83	0.6	26	Pass	N/A	Invalid	-	5495	74	-	1.2	-	24.5	-
3	TH22-04 ~14.2 m depth	47	61	83	0.6	24	Pass	5.38	Invalid	5.10	4967	70	1.0	1.2	1.2	24.5	-
4	TH22-04 ~14.2 m depth	34	64	56	0.6	17	Pass	3.60	Valid	3.41	2424	49	1.4	1.0	1.4	24.5	34
5	TH22-04 ~14.2 m depth	36	52	49	0.7	18	Pass	5.41	Valid	5.13	2246	47	2.3	1.0	2.2	24.5	55
6	TH22-04 ~14.2 m depth	37	59	48	0.8	19	Pass	5.08	Valid	4.82	2261	48	2.1	1.0	2.1	24.5	51
7	TH22-04 ~14.2 m depth	31	48	38	0.8	16	Pass	3.74	Invalid	3.55	1500	39	2.4	0.9	2.1	24.5	-
8	TH22-04 ~14.2 m depth	32	71	45	0.7	16	Pass	5.90	Valid	5.59	1833	43	3.1	0.9	2.8	24.5	70
9	TH22-04 ~14.2 m depth	31	66	60	0.5	16	Pass	2.58	Invalid	2.45	2368	49	1.0	1.0	1.0	24.5	-
10	TH22-04 ~14.2 m depth	31	60	56	0.6	16	Pass	2.41	Valid	2.28	2210	47	1.0	1.0	1.0	24.5	25
11	TH22-04 ~14.2 m depth	39	52	47	0.8	20	Pass	3.48	Valid	3.30	2334	48	1.4	1.0	1.4	24.5	34
12	TH22-04 ~14.2 m depth	36	46	38	0.9	18	Pass	3.54	Valid	3.36	1742	42	1.9	0.9	1.8	24.5	44
13	TH22-04 ~13.6 m depth	62	137	72	0.9	31	Pass	18.39	Invalid	17.43	5684	75	3.1	1.2	3.7	24.5	-
14	TH22-04 ~13.6 m depth	60	137	72	0.8	30	Pass	22.34	Valid	21.18	5500	74	3.9	1.2	4.6	24.5	113
15	TH22-04 ~13.6 m depth	60	80	72	0.8	30	Pass	7.10	Valid	6.73	5500	74	1.2	1.2	1.5	24.5	36
16	TH22-04 ~13.6 m depth	48	69	61	0.8	24	Pass	8.45	Valid	8.01	3728	61	2.1	1.1	2.4	24.5	58
17	TH22-04 ~13.6 m depth	43	69	67	0.6	22	Pass	11.02	Valid	10.45	3668	61	2.8	1.1	3.1	24.5	76
18	TH22-04 ~13.6 m depth	32	63	49	0.7	16	Pass	2.11	Valid	2.00	1996	45	1.0	1.0	1.0	24.5	23
19	TH22-04 ~13.6 m depth	49	70	60	0.8	25	Pass	9.55	Valid	9.05	3743	61	2.4	1.1	2.6	24.5	65
20	TH22-04 ~15.7m depth	46	74	54	0.9	23	Pass	3.09	Valid	2.93	3163	56	0.9	1.1	1.0	24.5	24
21	TH22-04 ~15.7m depth	31	73	51	0.6	16	Pass	2.58	Valid	2.45	2013	45	1.2	1.0	1.2	24.5	28
22	TH22-04 ~15.7m depth	31	60	37	0.8	16	Pass	3.72	Valid	3.53	1460	38	2.4	0.9	2.1	24.5	52
23	TH22-04 ~11.4 m depth	30	70	47	0.6	15	Pass	1.84	Valid	1.74	1795	42	1.0	0.9	0.9	24.5	22
24	TH22-04 ~11.4 m depth	29	66	39	0.7	15	Pass	7.11	Valid	6.74	1440	38	4.7	0.9	4.1	24.5	101
25	TH22-04, unknown depth	32	63	49	0.7	16	Pass	2.11	Valid	2.00	1996	45	1.0	1.0	1.0	24.5	23
26	TH22-04, unknown depth	31	80	56	0.6	16	Pass	1.68	Valid	1.59	2210	47	0.7	1.0	0.7	24.5	17

THURBER

APPENDIX E MONITORING RESULTS

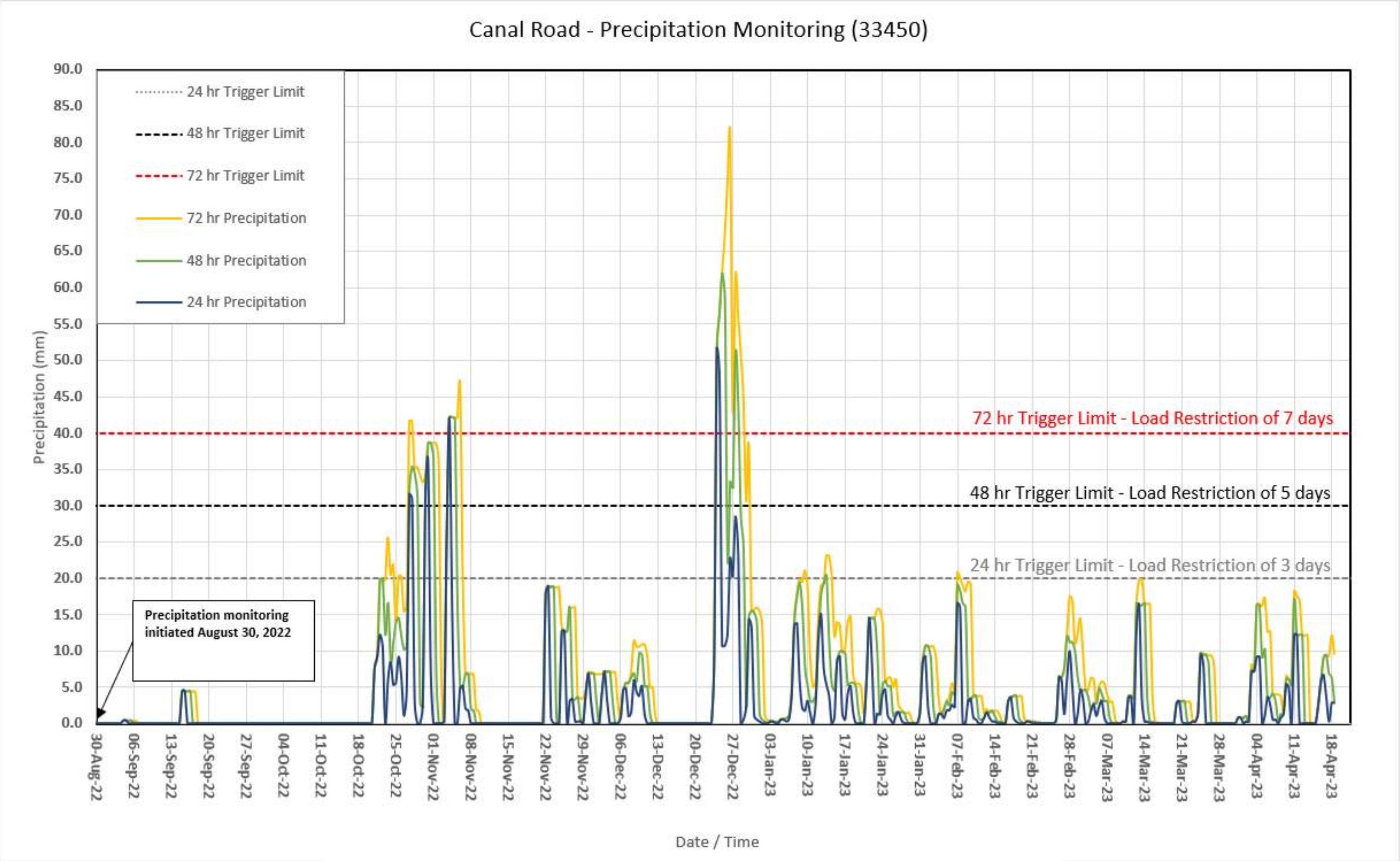


FIGURE E.1 RAIN GAUGE PRECIPITATION PLOT

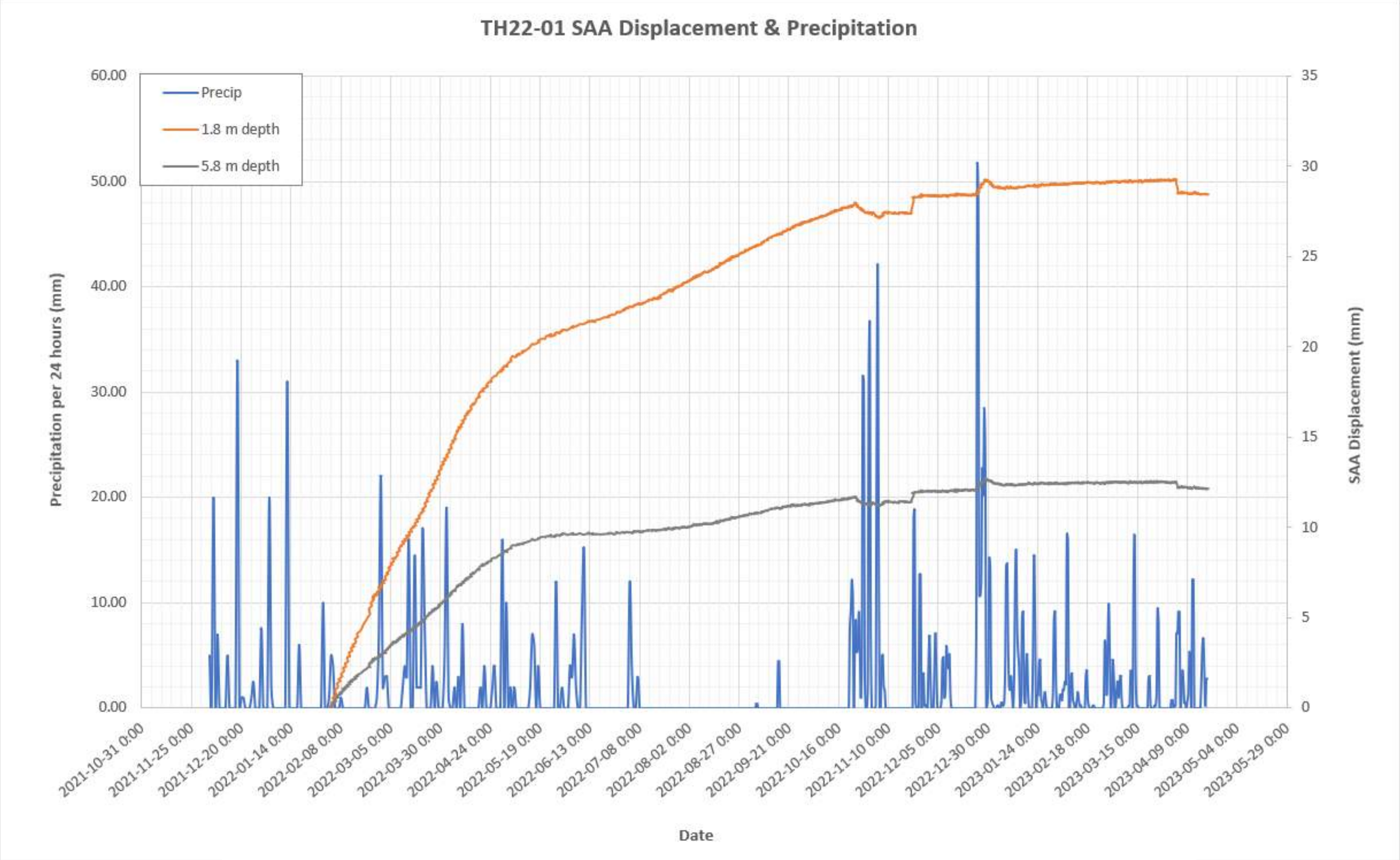


FIGURE E.2 TH22-01 DEFORMATION AND PRECIPITATION PLOT

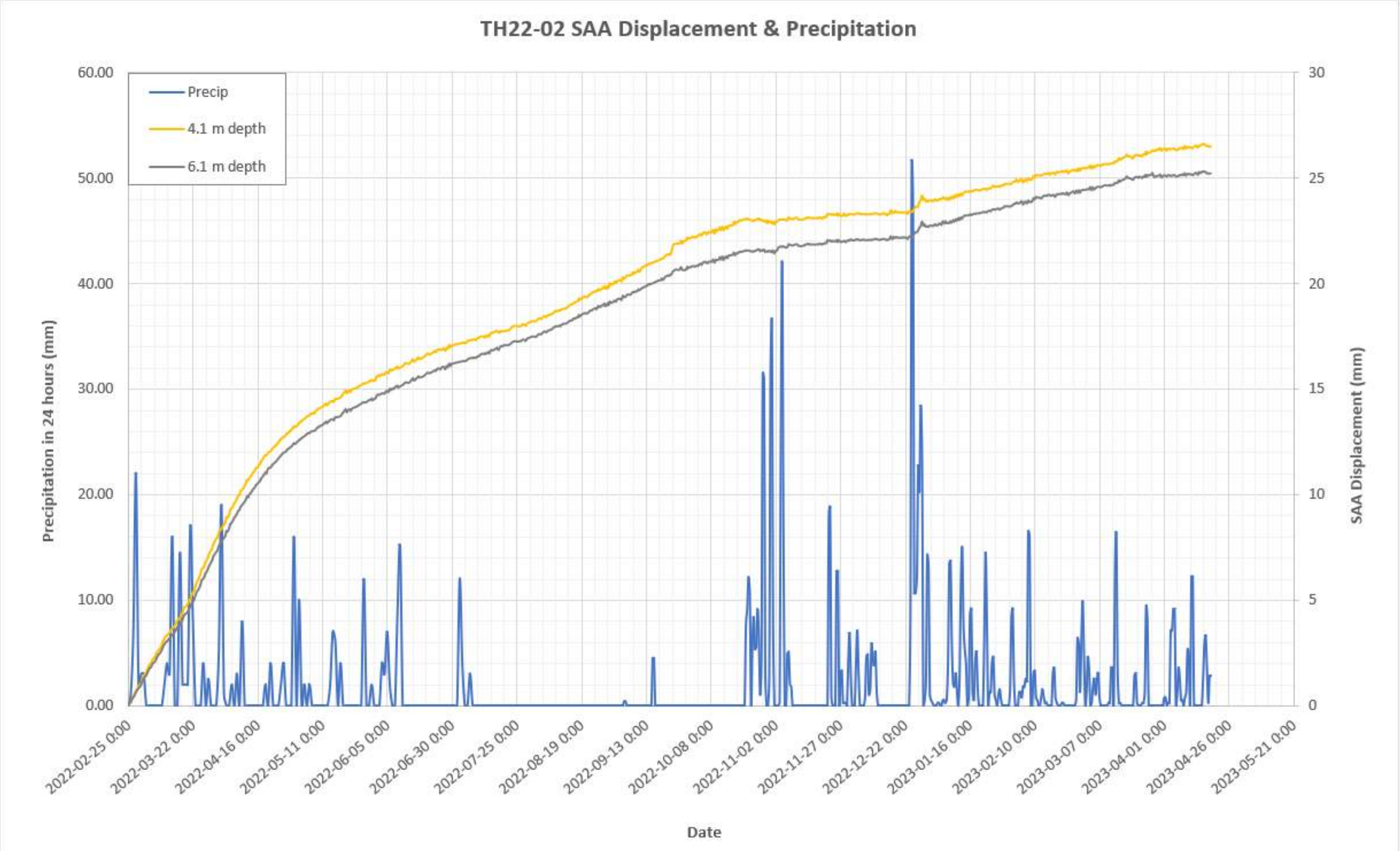
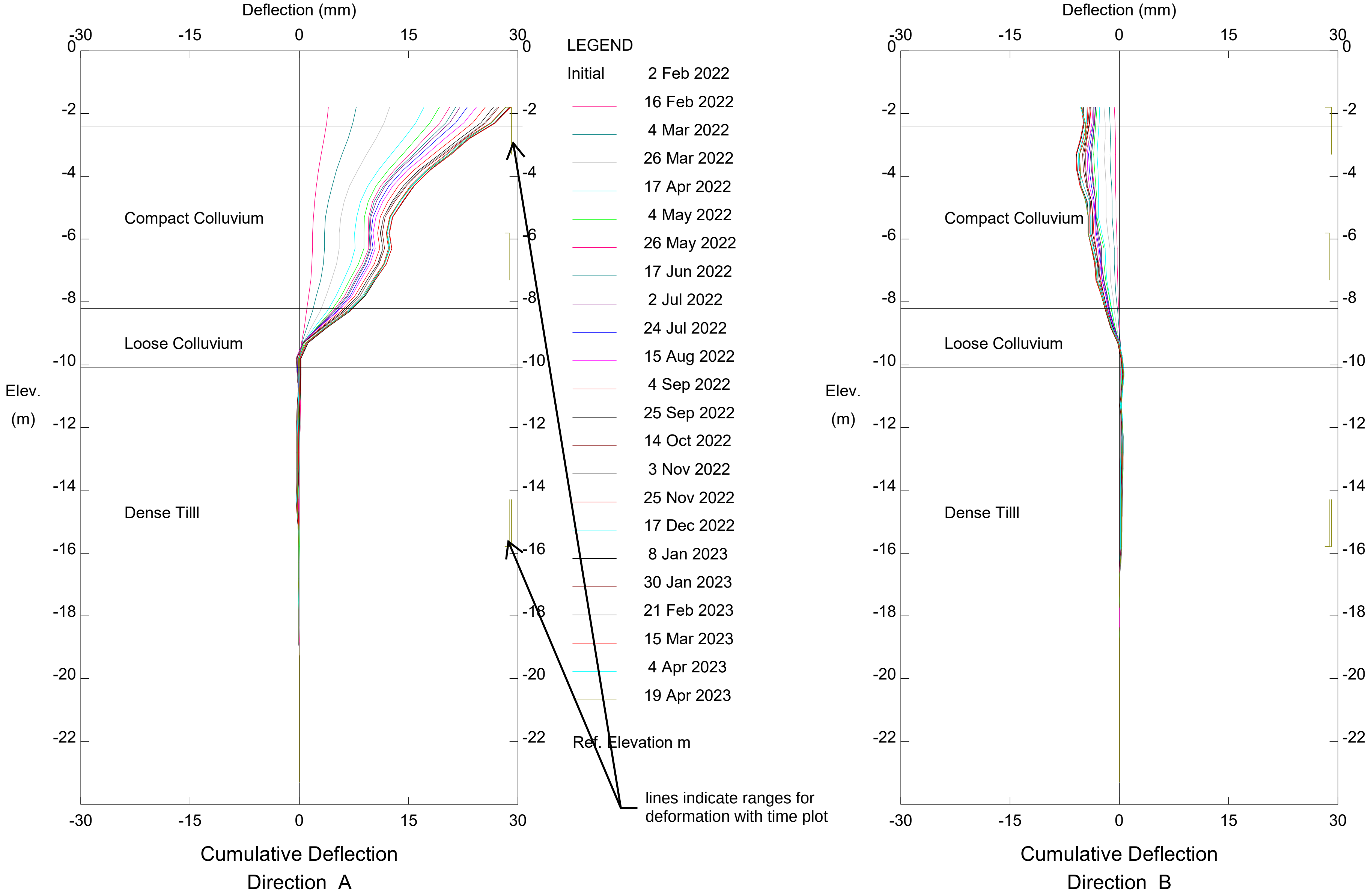


FIGURE E.3 TH22-02 DEFORMATION AND PRECIPITATION PLOT

Thurber Engineering - Victoria

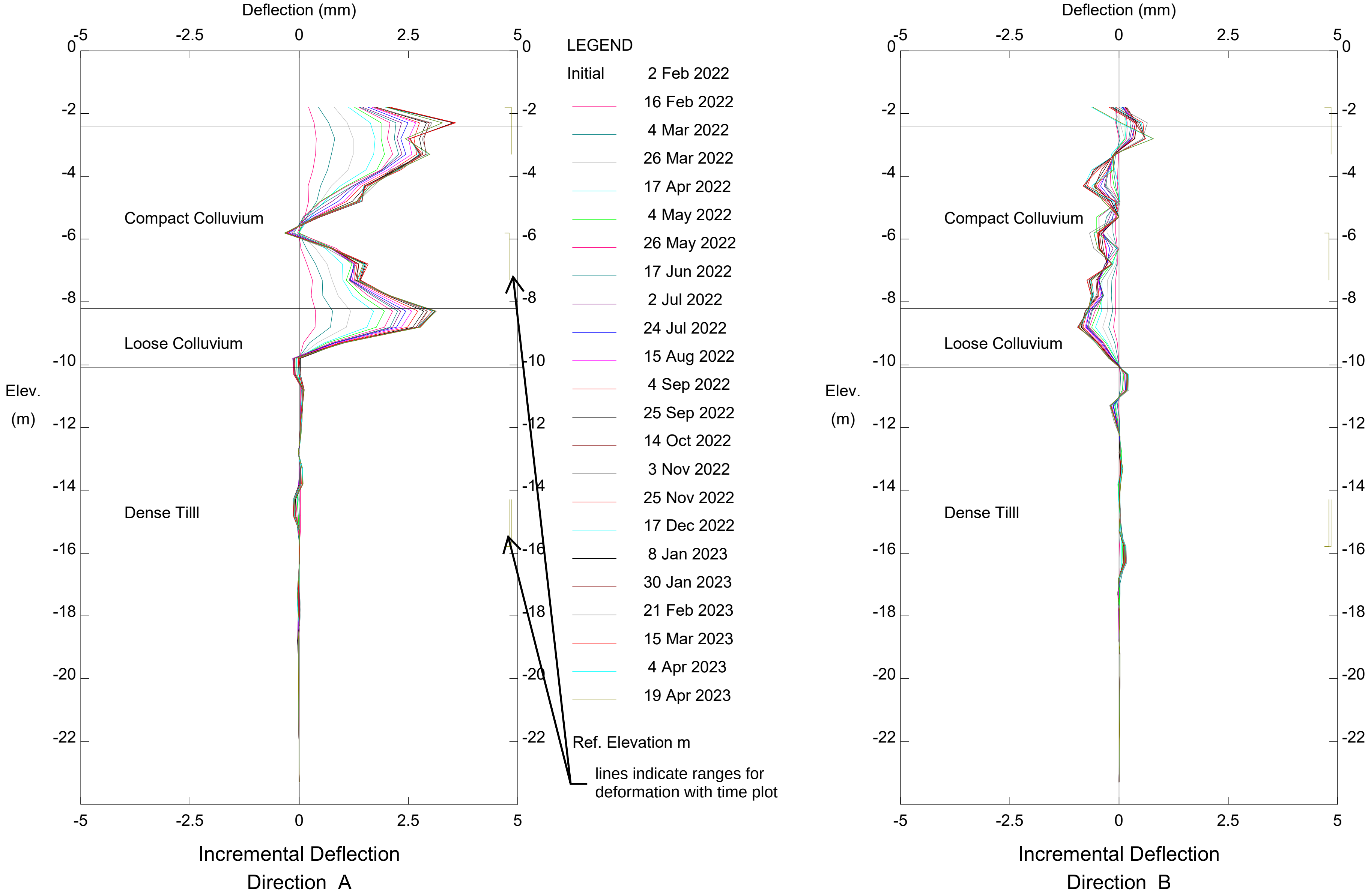


Canal Road Slide, Inclinometer TH22-01

BC MoTI

S:\Data\BST Projects\33xxx\33450\08_Work in Progress\03_SAA Data Files (Automated Collection)\wrw_TH22-01_SAA_33450.gtl

Thurber Engineering - Victoria



Canal Road Slide, Inclinometer TH22-01

BC MoTI

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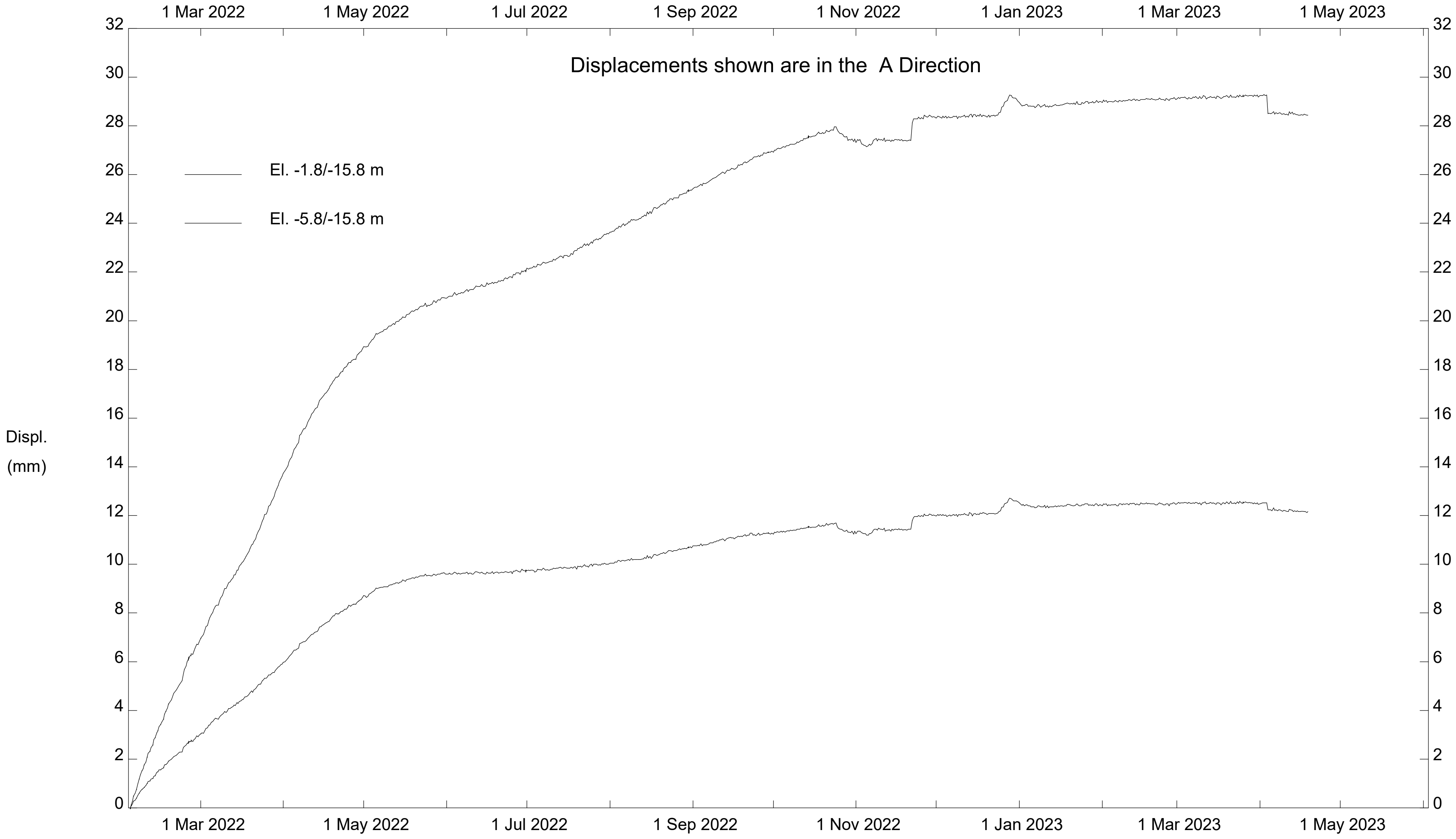
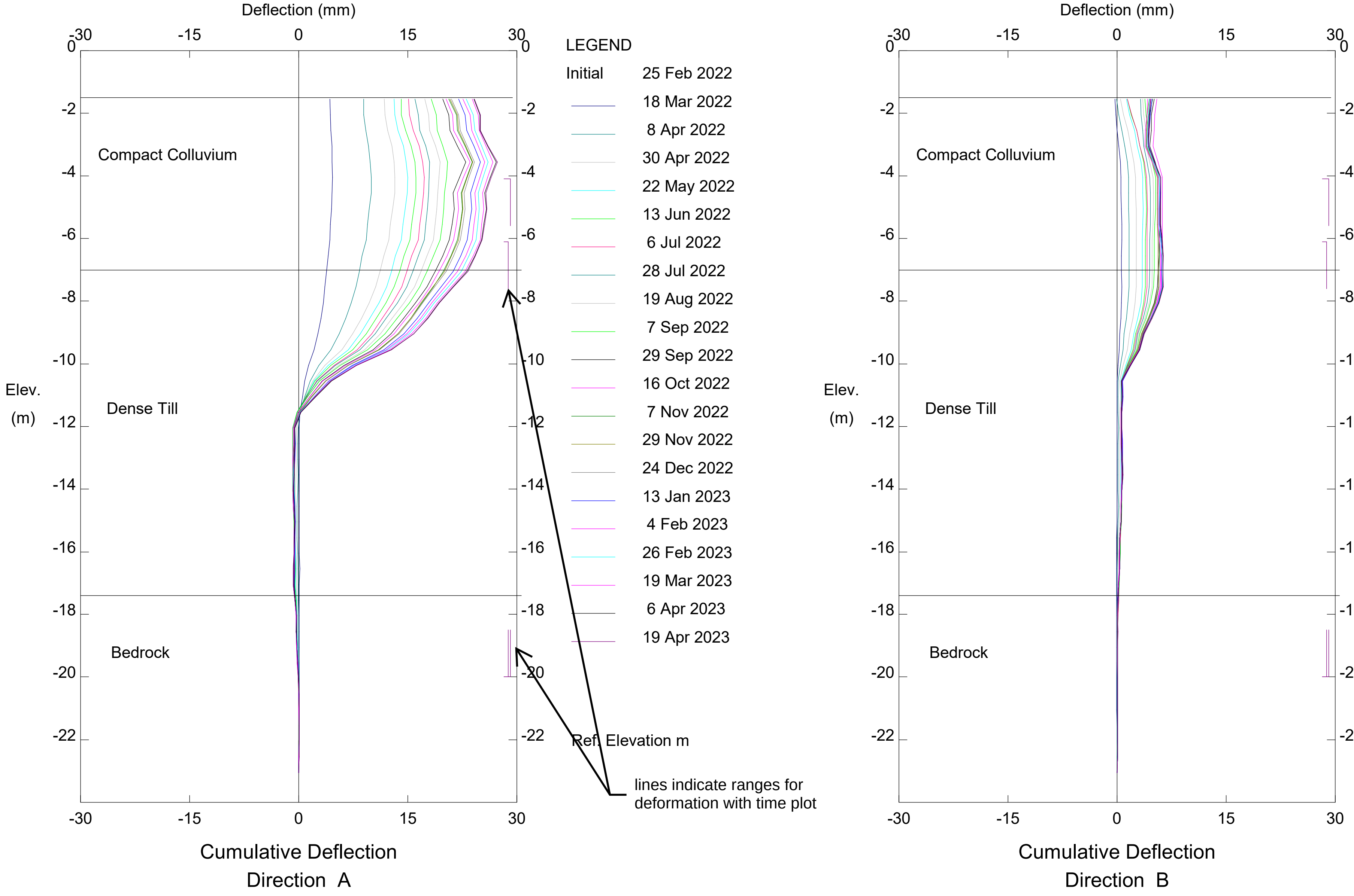
Thurber Engineering - Victoria



Canal Road Slide, Inclinometer TH22-01

BC MoTI

Thurber Engineering - Victoria



Canal Road Slide, Inclinometer TH22-02

BC MoTI

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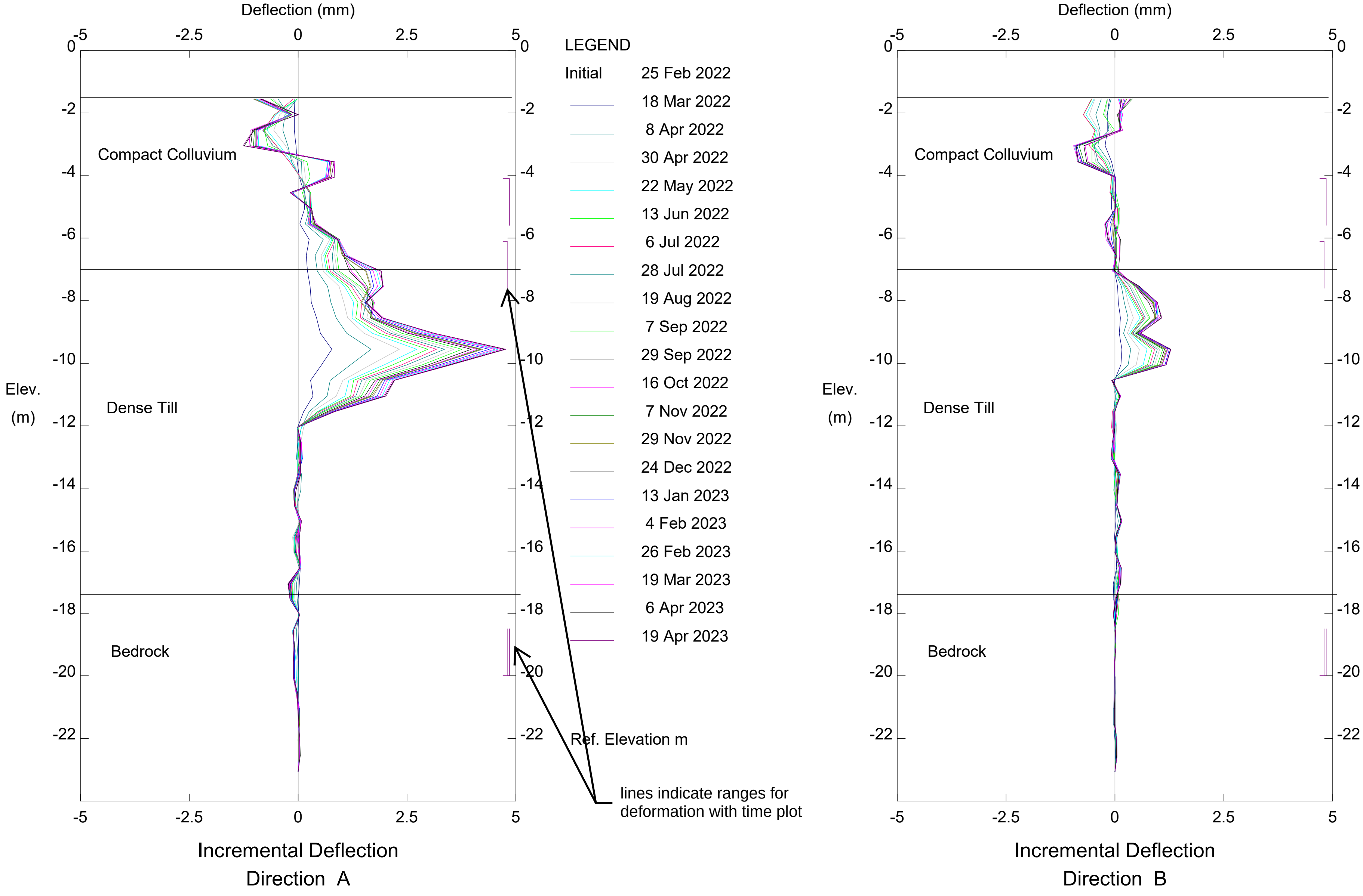
Thurber Engineering - Victoria



Canal Road Slide, Inclinometer TH22-02

BC MoTI

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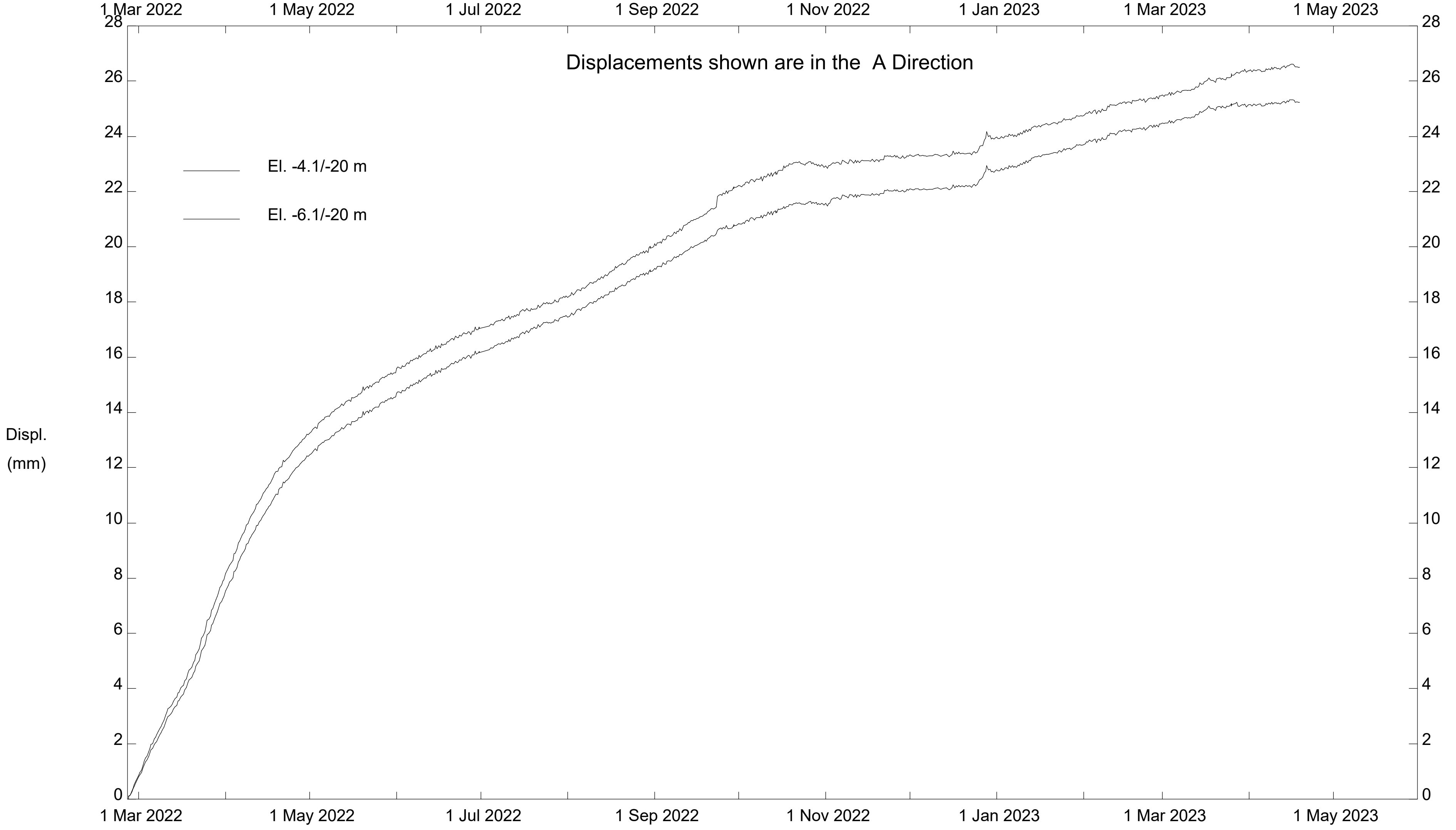
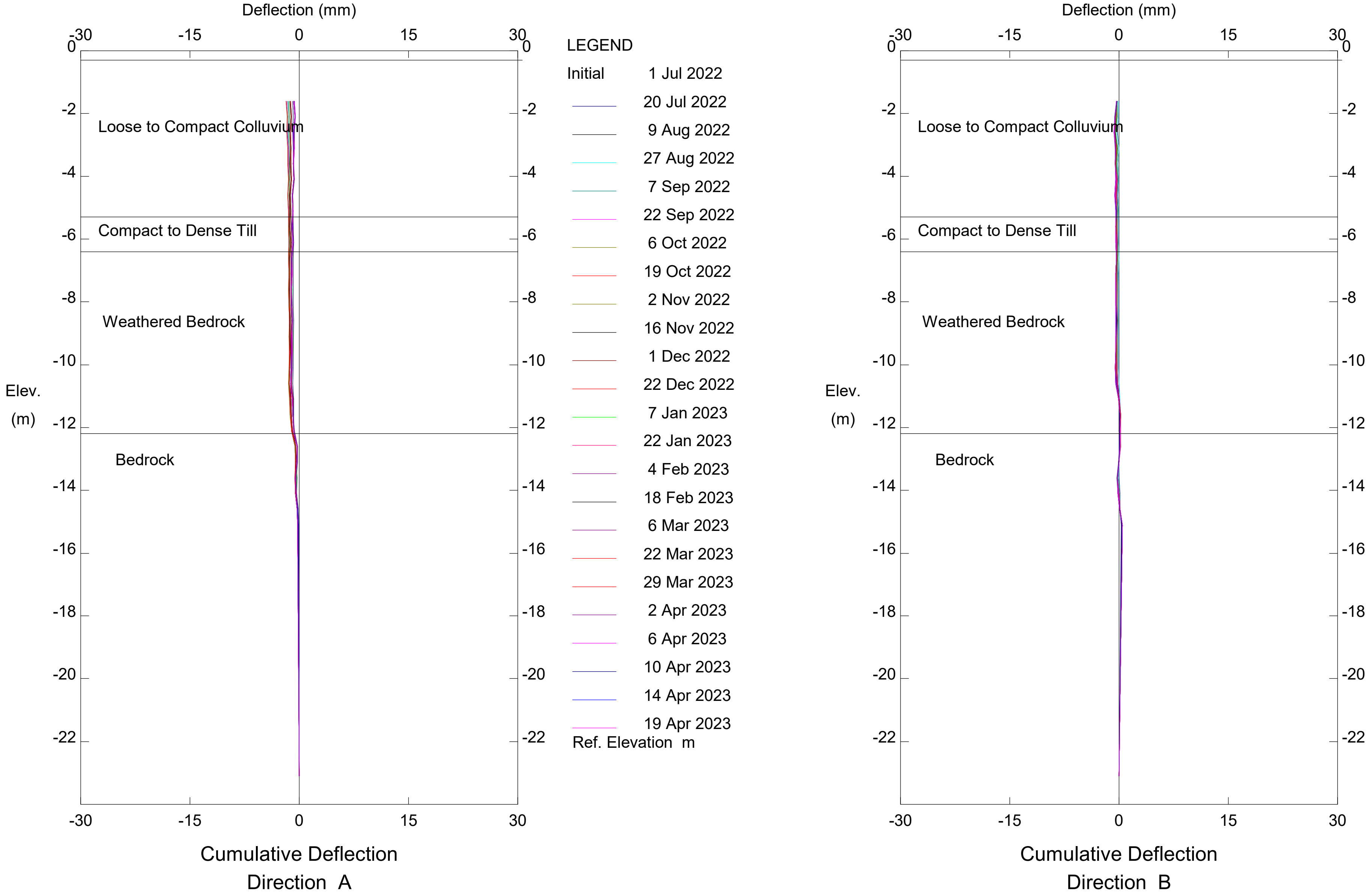
Thurber Engineering - Victoria



Canal Road Slide, Inclinometer TH22-02

BC MoTI

Thurber Engineering - Victoria



Canal Road Slide, Inclinometer TH22-04

MoTI

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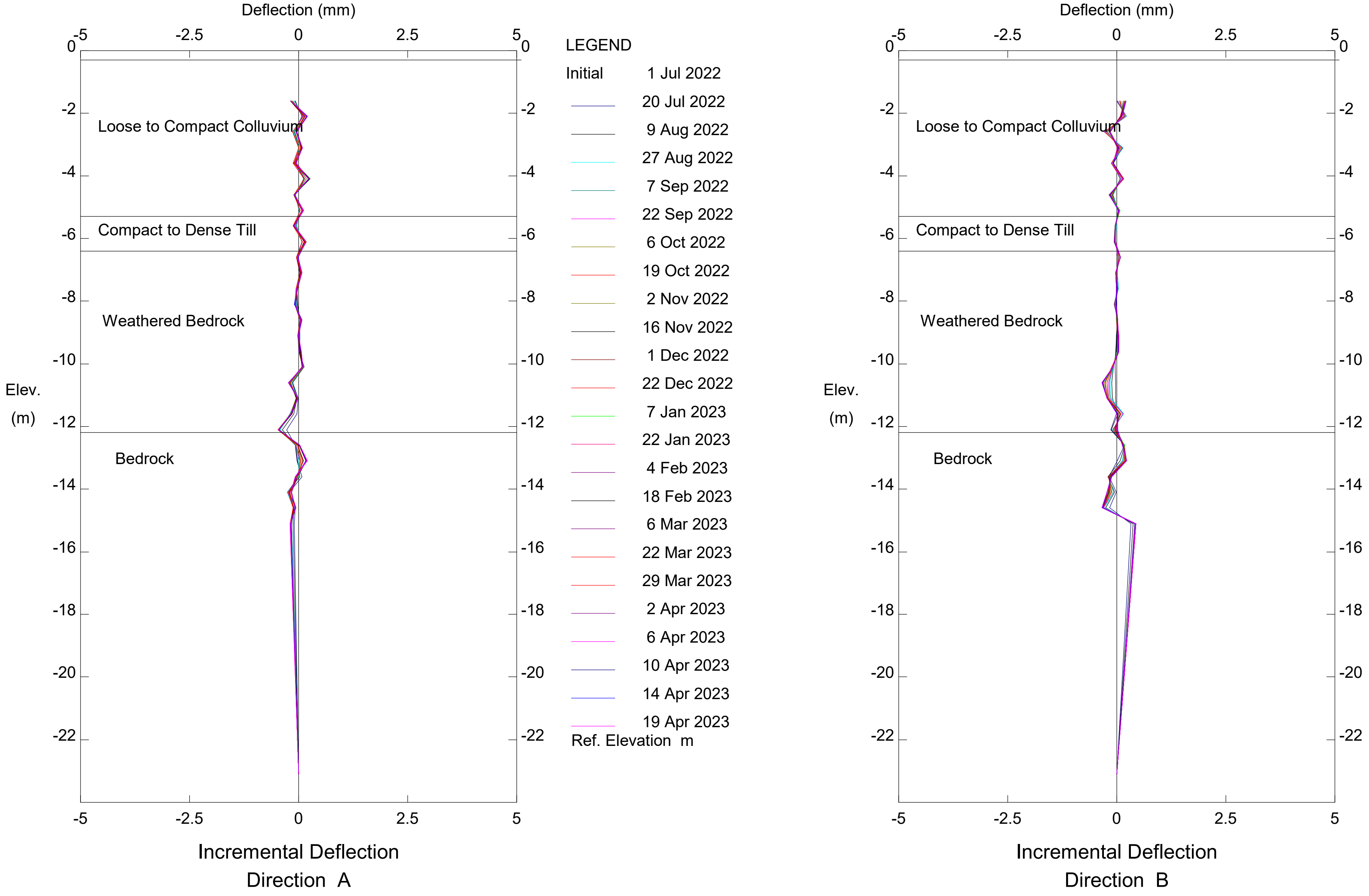
Thurber Engineering - Victoria



Canal Road Slide, Inclinometer TH22-04

MoTI

S:\Data\BST Projects\33xxx\33450\08_Work in Progress\03_SAA Data Files (Automated Collection)\jlu_TH22-04_SAA_33450.gtl

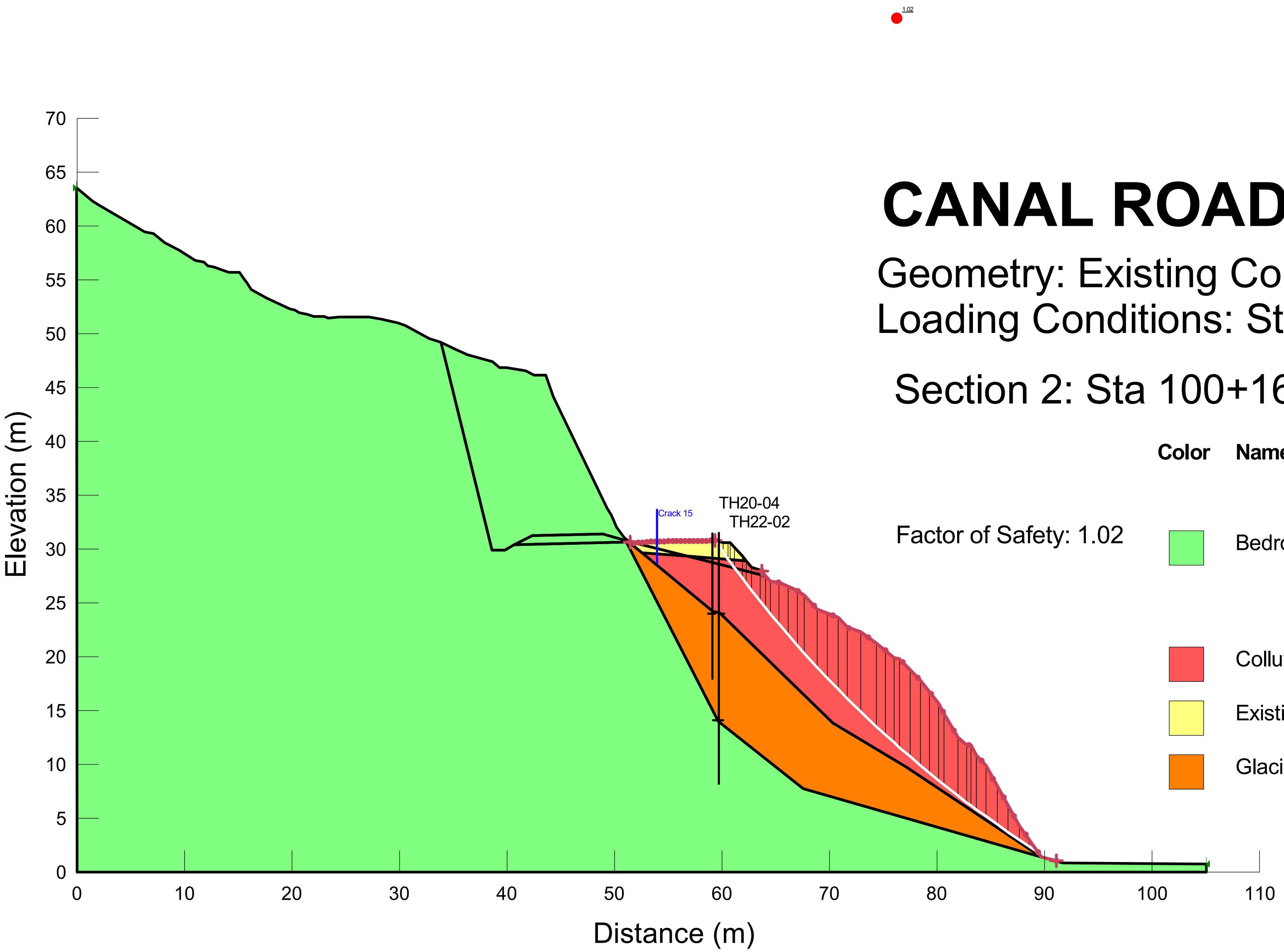
THURBER

APPENDIX F SLOPE STABILITY PLOTS

CANAL ROAD - PENDER ISLAND

Geometry: Existing Conditions (Back Analysis)
Loading Conditions: Static

Section 2: Sta 100+165



Factor of Safety: 1.02

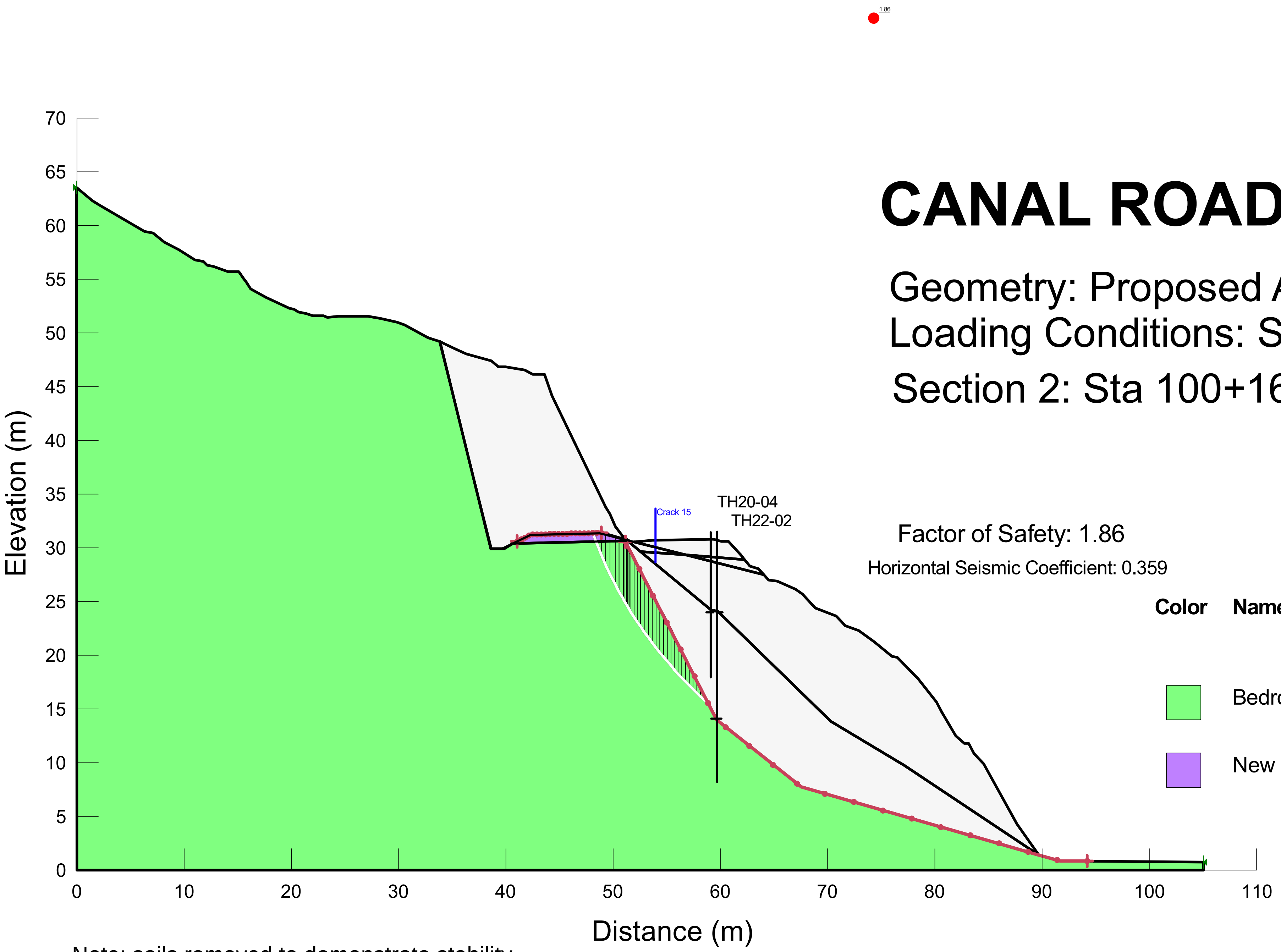
Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (kPa)	Strength Function
🟩	Bedrock	Shear/Normal Fn.	26						Siltstone (South Pender Island)
🟥	Colluvium	Bilinear	20	0		45	36	50	
🟨	Existing Fill	Mohr-Coulomb	20	0	36				
🟧	Glacial Till	Mohr-Coulomb	22	0	42				

Sta 100+165 Existing (Back Analysis)
jlu_Canal Road Slope Stability_33450.gsz
2022-06-09 1:500

CANAL ROAD - PENDER ISLAND

Geometry: Proposed Alignment
Loading Conditions: Seismic

Section 2: Sta 100+165



Factor of Safety: 1.86

Horizontal Seismic Coefficient: 0.359

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Strength Function
🟩	Bedrock	Shear/Normal Fn.	26			Siltstone (South Pender Island)
🟪	New Fill	Mohr-Coulomb	21	0	36	

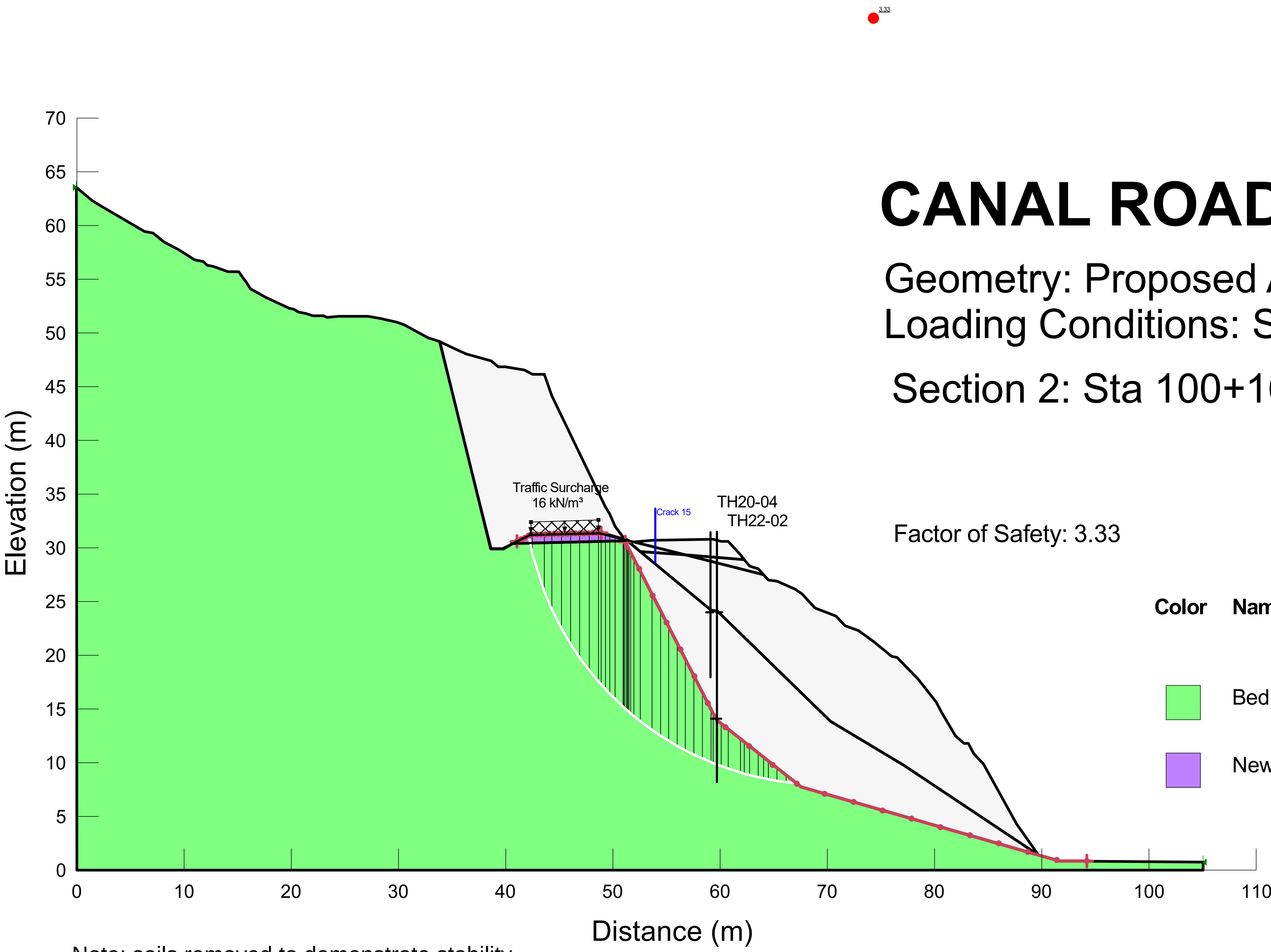
Note: soils removed to demonstrate stability
of the new alignment only

Sta 100+165 Proposed Seismic - Soils Removed	
jlu_Canal Road Slope Stability_33450.gsz	
2022-06-09	1:500

CANAL ROAD - PENDER ISLAND

Geometry: Proposed Alignment
Loading Conditions: Static

Section 2: Sta 100+165



Factor of Safety: 3.33

Color	**Name**	**Model**	**Unit Weight (kN/m³)**	**Effective Cohesion (kPa)**	**Effective Friction Angle (°)**	**Strength Function**
🟩	Bedrock	Shear/Normal Fn.	26			Siltstone (South Pender Island)
🟪	New Fill	Mohr-Coulomb	21	0	36	

Note: soils removed to demonstrate stability
of the new alignment only

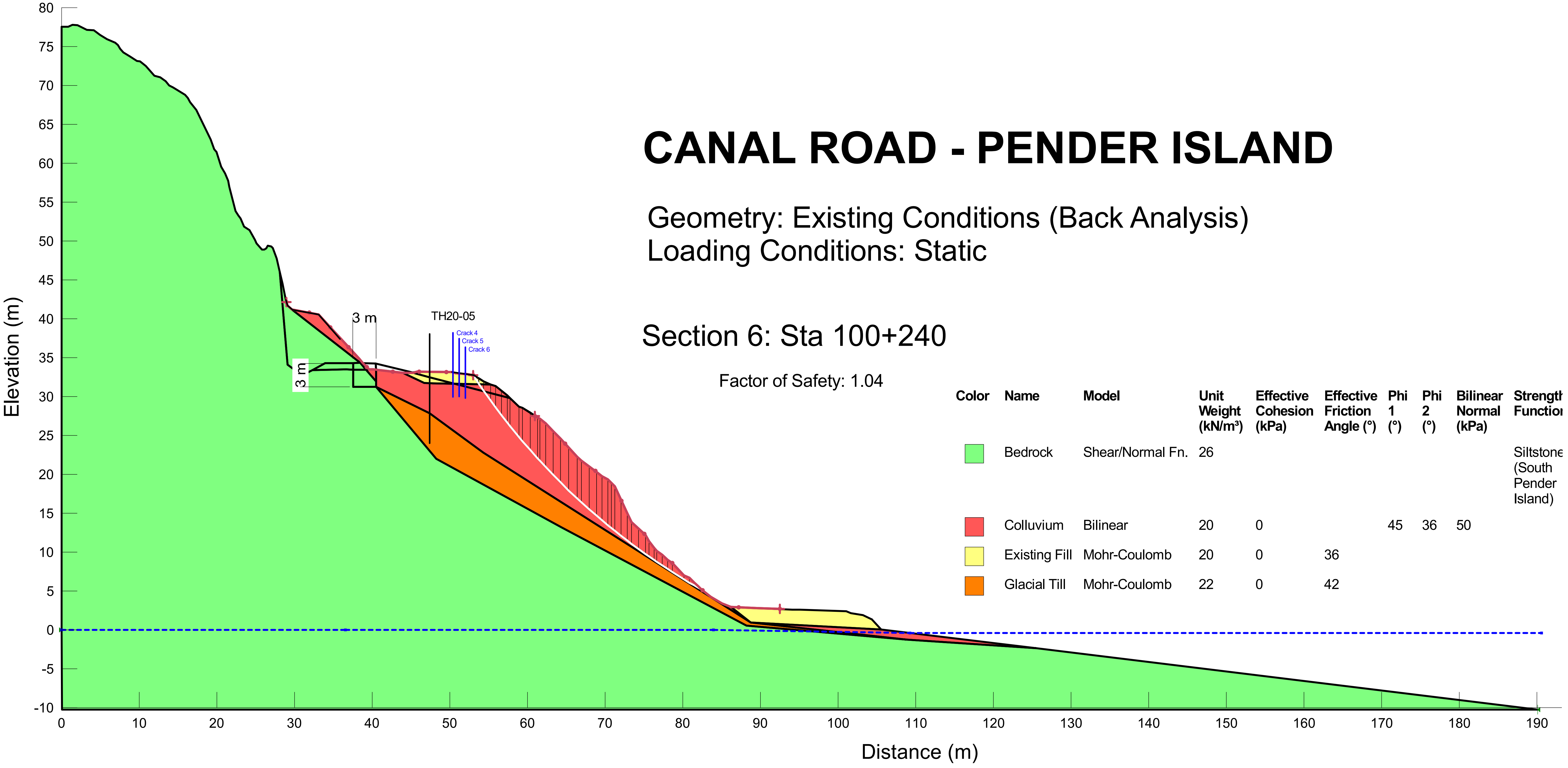
Sta 100+165 Proposed Static - Soils Removed
jlu_Canal Road Slope Stability_33450.gsz
2022-06-09 1:500

CANAL ROAD - PENDER ISLAND

Geometry: Existing Conditions (Back Analysis)
Loading Conditions: Static

Section 6: Sta 100+240

Factor of Safety: 1.04



Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (kPa)	Strength Function
🟩	Bedrock	Shear/Normal Fn.	26						Siltstone (South Pender Island)
🟥	Colluvium	Bilinear	20	0		45	36	50	
🟨	Existing Fill	Mohr-Coulomb	20	0	36				
🟧	Glacial Till	Mohr-Coulomb	22	0	42				

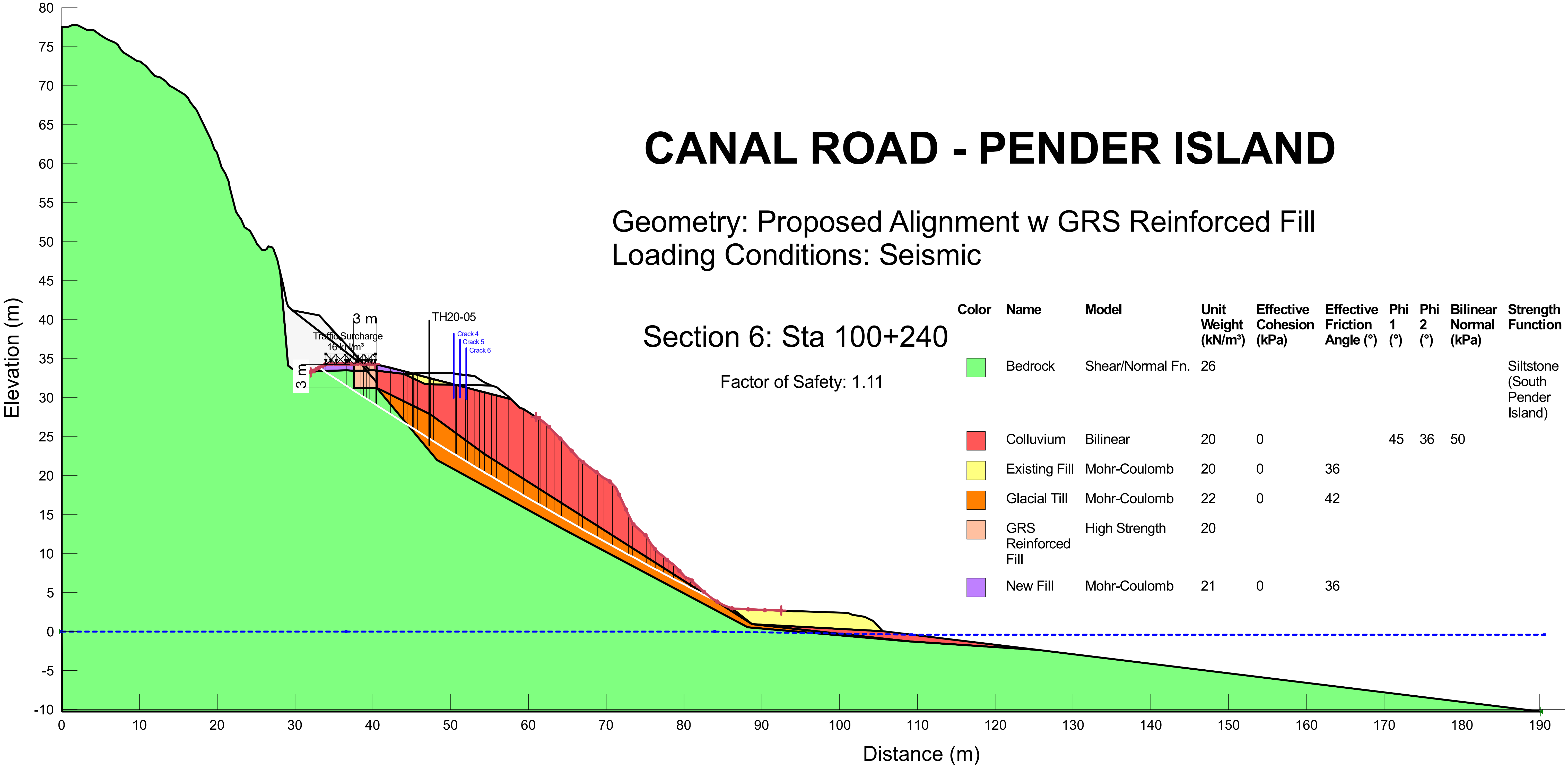
Sta 100+240 Existing (Back Analysis)
jlu_Canal Road Slope Stability_33450.gsz
2022-06-09 1:500

CANAL ROAD - PENDER ISLAND

Geometry: Proposed Alignment w GRS Reinforced Fill
Loading Conditions: Seismic

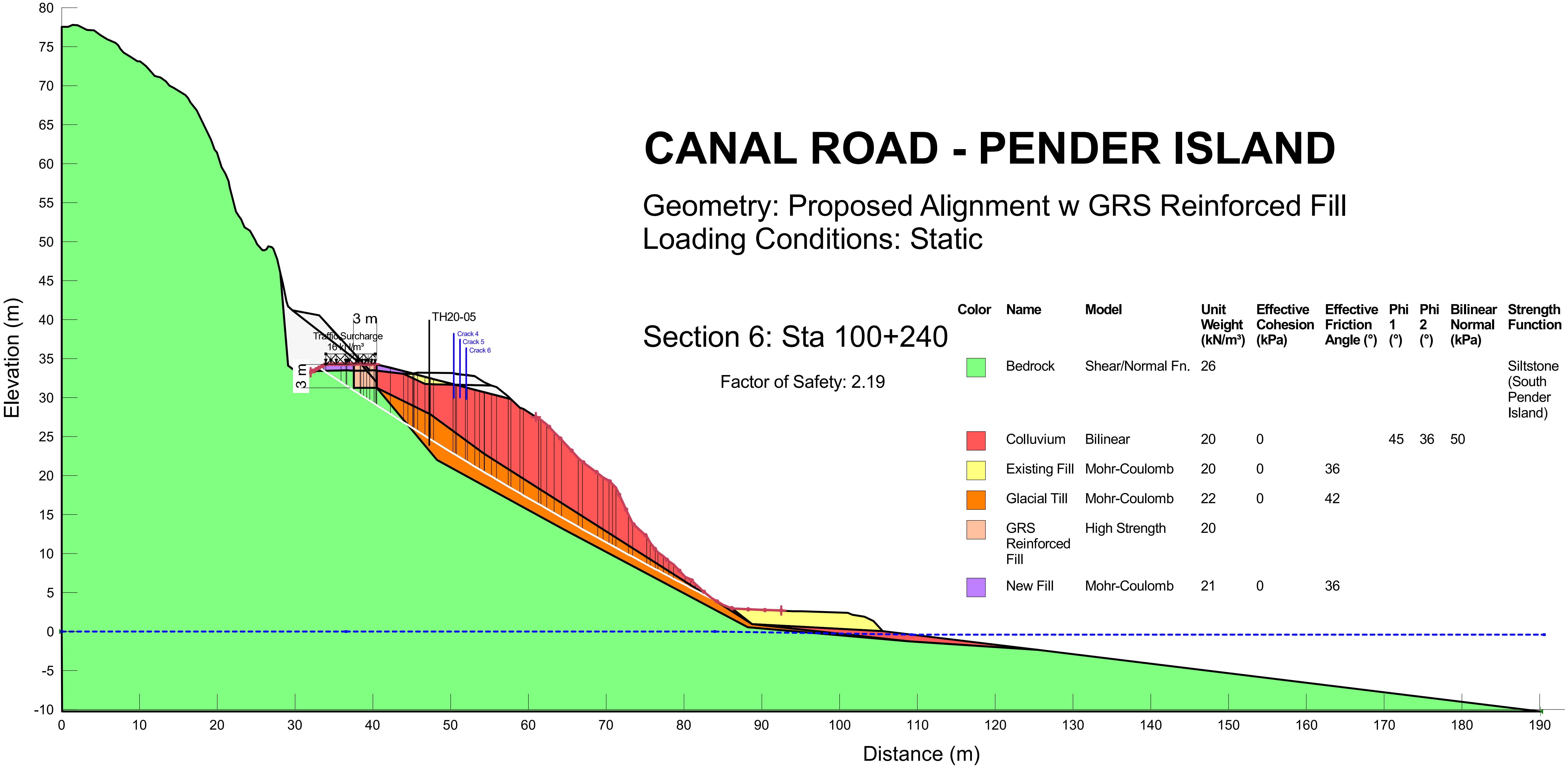
Section 6: Sta 100+240

Factor of Safety: 1.11



Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (kPa)	Strength Function
🟩	Bedrock	Shear/Normal Fn.	26						Siltstone (South Pender Island)
🟥	Colluvium	Bilinear	20	0		45	36	50	
🟨	Existing Fill	Mohr-Coulomb	20	0	36				
🟧	Glacial Till	Mohr-Coulomb	22	0	42				
▨	GRS Reinforced Fill	High Strength	20						
🟪	New Fill	Mohr-Coulomb	21	0	36				

Sta 100+240 Proposed Seismic - GRS Reinforced Fill
jlu_Canal Road Slope Stability_33450.gsz
2022-06-09 1:500



CANAL ROAD - PENDER ISLAND

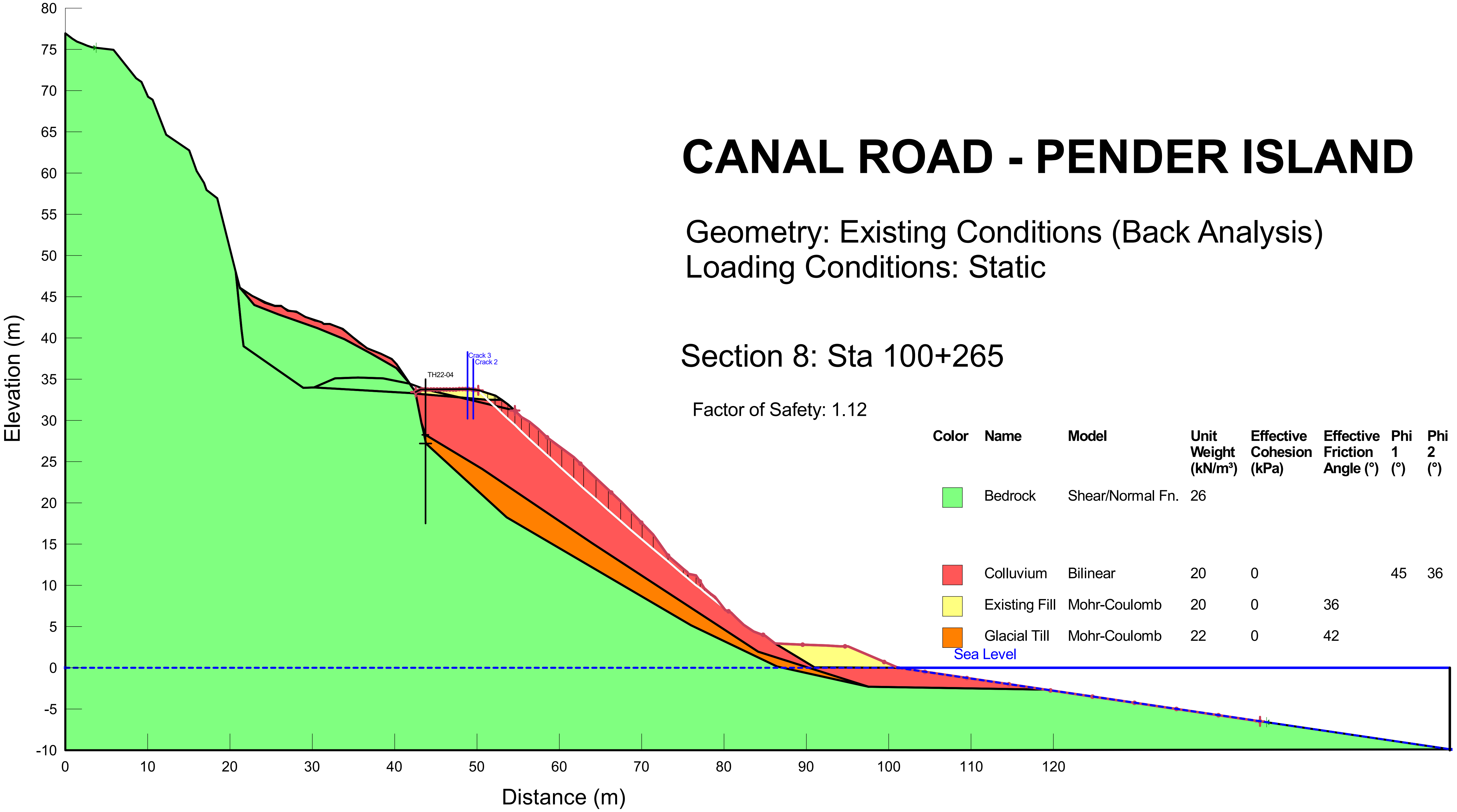
Geometry: Proposed Alignment w GRS Reinforced Fill
Loading Conditions: Static

Section 6: Sta 100+240

Factor of Safety: 2.19

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (kPa)	Strength Function
🟩	Bedrock	Shear/Normal Fn.	26						Siltstone (South Pender Island)
🟥	Colluvium	Bilinear	20	0		45	36	50	
🟨	Existing Fill	Mohr-Coulomb	20	0	36				
🟧	Glacial Till	Mohr-Coulomb	22	0	42				
🟫	GRS Reinforced Fill	High Strength	20						
🟪	New Fill	Mohr-Coulomb	21	0	36				

Sta 100+240 Proposed Static - GRS Reinforced Fill	
jlu_Canal Road Slope Stability_33450.gsz	
2022-06-09	1:500



CANAL ROAD - PENDER ISLAND

Geometry: Existing Conditions (Back Analysis)
Loading Conditions: Static

Section 8: Sta 100+265

Factor of Safety: 1.12

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (kPa)	Strength Function
🟩	Bedrock	Shear/Normal Fn.	26						Siltstone (South Pender Island)
🟥	Colluvium	Bilinear	20	0		45	36	50	
🟨	Existing Fill	Mohr-Coulomb	20	0	36				
🟧	Glacial Till	Mohr-Coulomb	22	0	42				

Sta 100+265 Existing (Back Analysis)
jlu_Canal Road Slope Stability_33450.gsz
2022-06-09 1:500

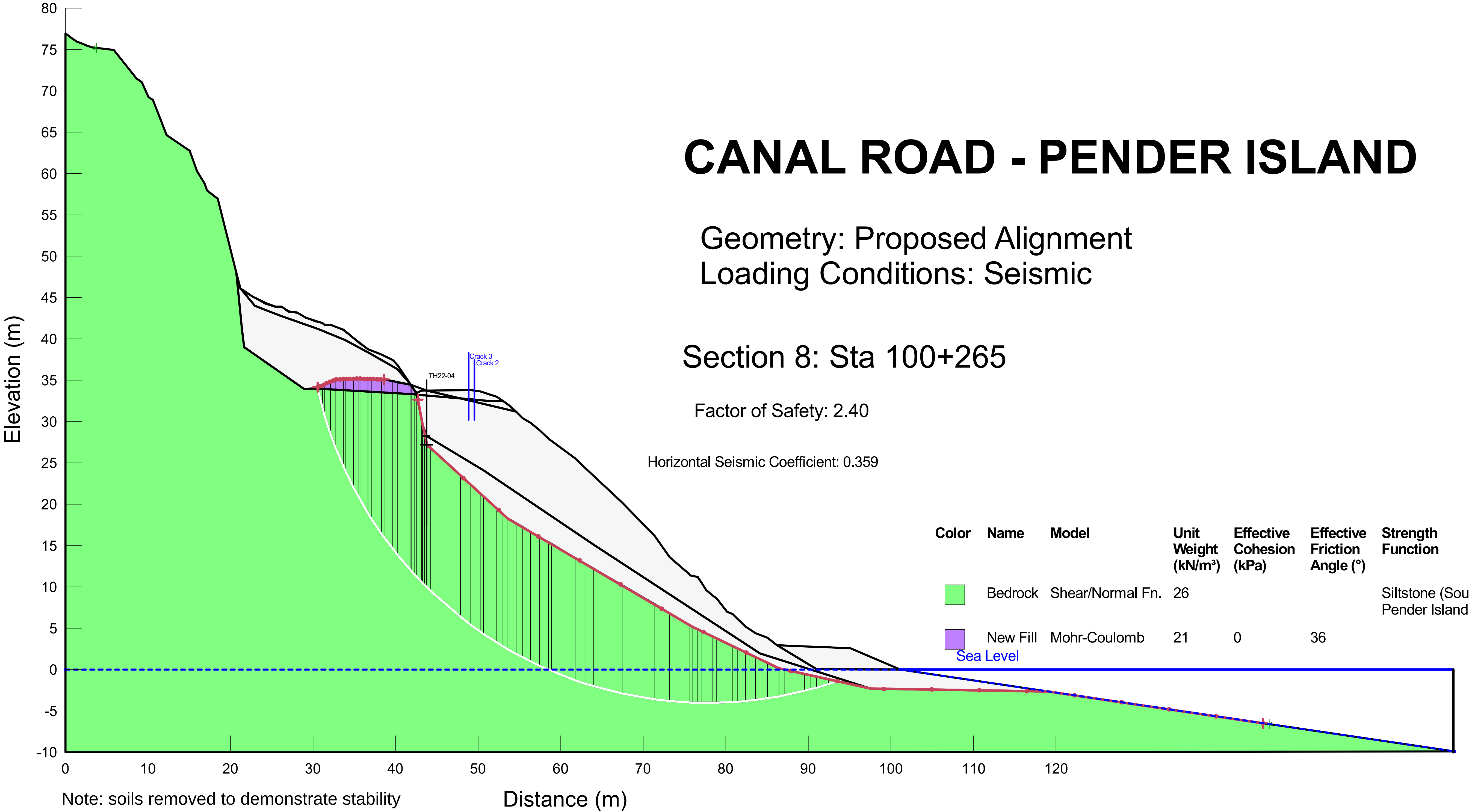
CANAL ROAD - PENDER ISLAND

Geometry: Proposed Alignment
Loading Conditions: Seismic

Section 8: Sta 100+265

Factor of Safety: 2.40

Horizontal Seismic Coefficient: 0.359



Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Strength Function
🟩	Bedrock	Shear/Normal Fn.	26			Siltstone (South Pender Island)
🟪	New Fill	Mohr-Coulomb	21	0	36	

Note: soils removed to demonstrate stability
of the new alignment only

Sta 100+265 Proposed Seismic - Soils Removed	
jlu_Canal Road Slope Stability_33450.gsz	
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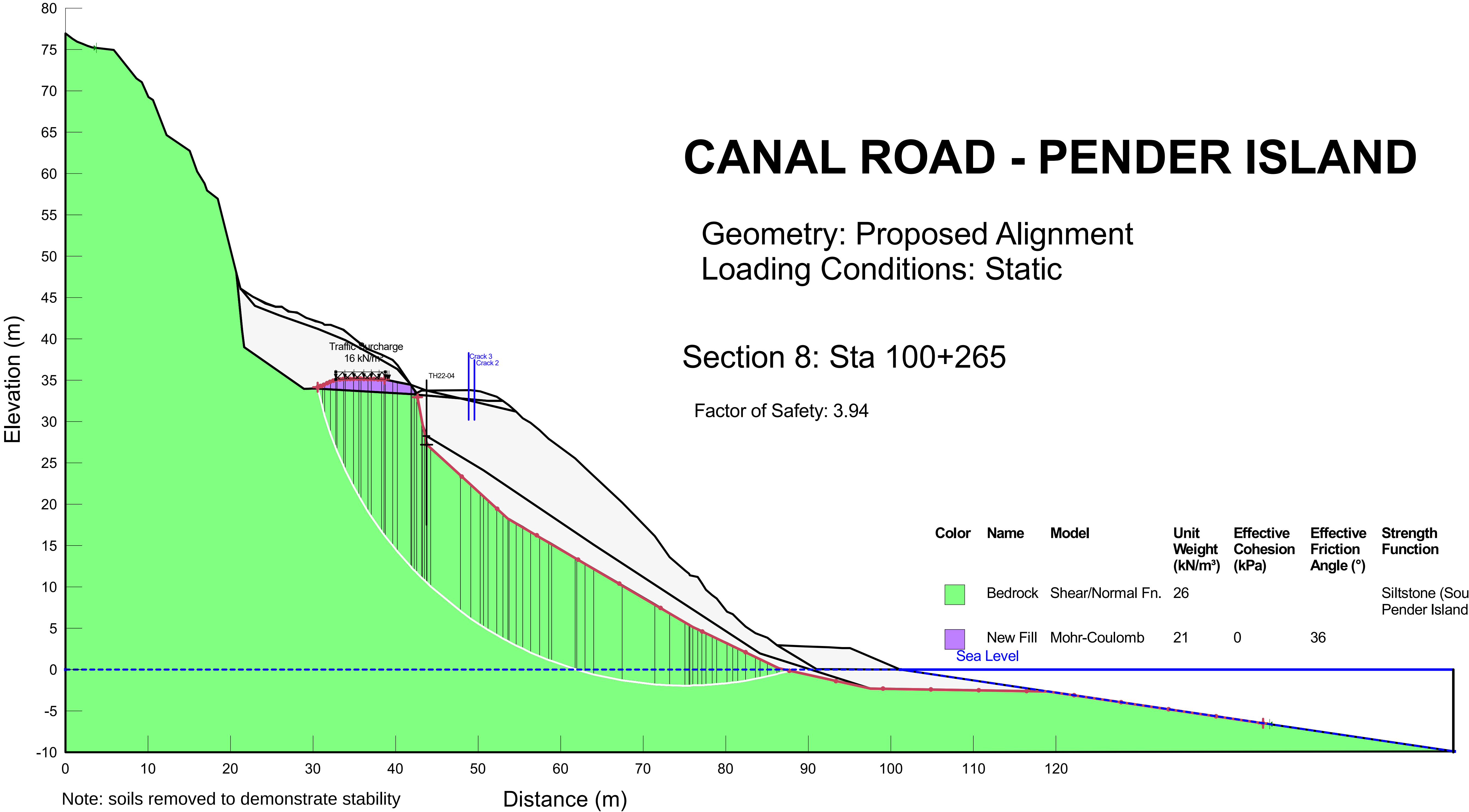


CANAL ROAD - PENDER ISLAND

Geometry: Proposed Alignment
Loading Conditions: Static

Section 8: Sta 100+265

Factor of Safety: 3.94

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Strength Function
🟩	Bedrock	Shear/Normal Fn.	26			Siltstone (South Pender Island)
🟪	New Fill	Mohr-Coulomb	21	0	36	

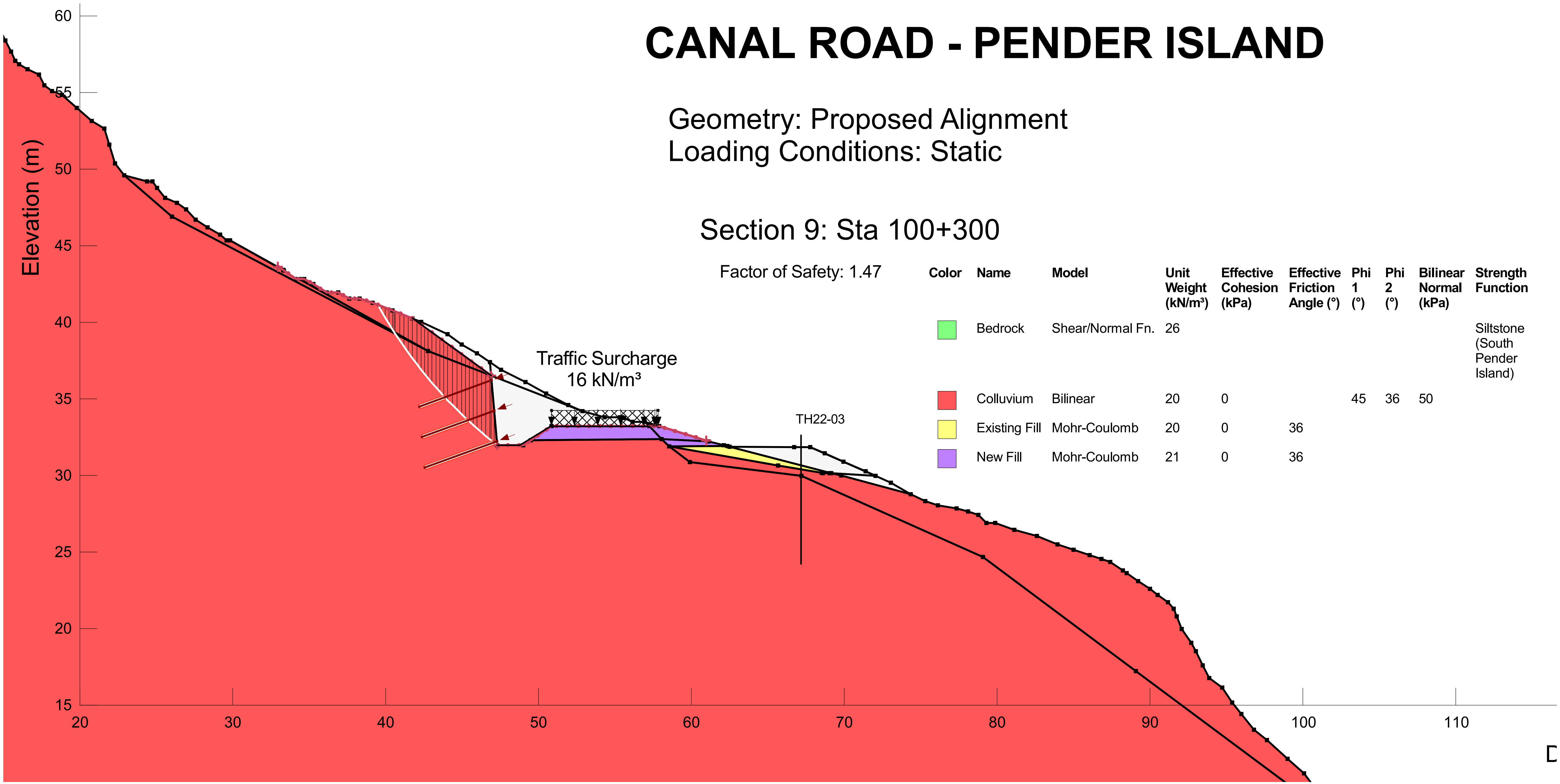
Note: soils removed to demonstrate stability
of the new alignment only

Sta 100+265 Proposed Static - Soils Removed	
jlu_Canal Road Slope Stability_33450.gsz	
2022-06-09	1:500

CANAL ROAD - PENDER ISLAND

Geometry: Proposed Alignment
Loading Conditions: Static

Section 9: Sta 100+300



Factor of Safety: 1.47

Color	Name	Model	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (kPa)	Strength Function
🟩	Bedrock	Shear/Normal Fn.	26						Siltstone (South Pender Island)
🟥	Colluvium	Bilinear	20	0		45	36	50	
🟨	Existing Fill	Mohr-Coulomb	20	0	36				
🟪	New Fill	Mohr-Coulomb	21	0	36				

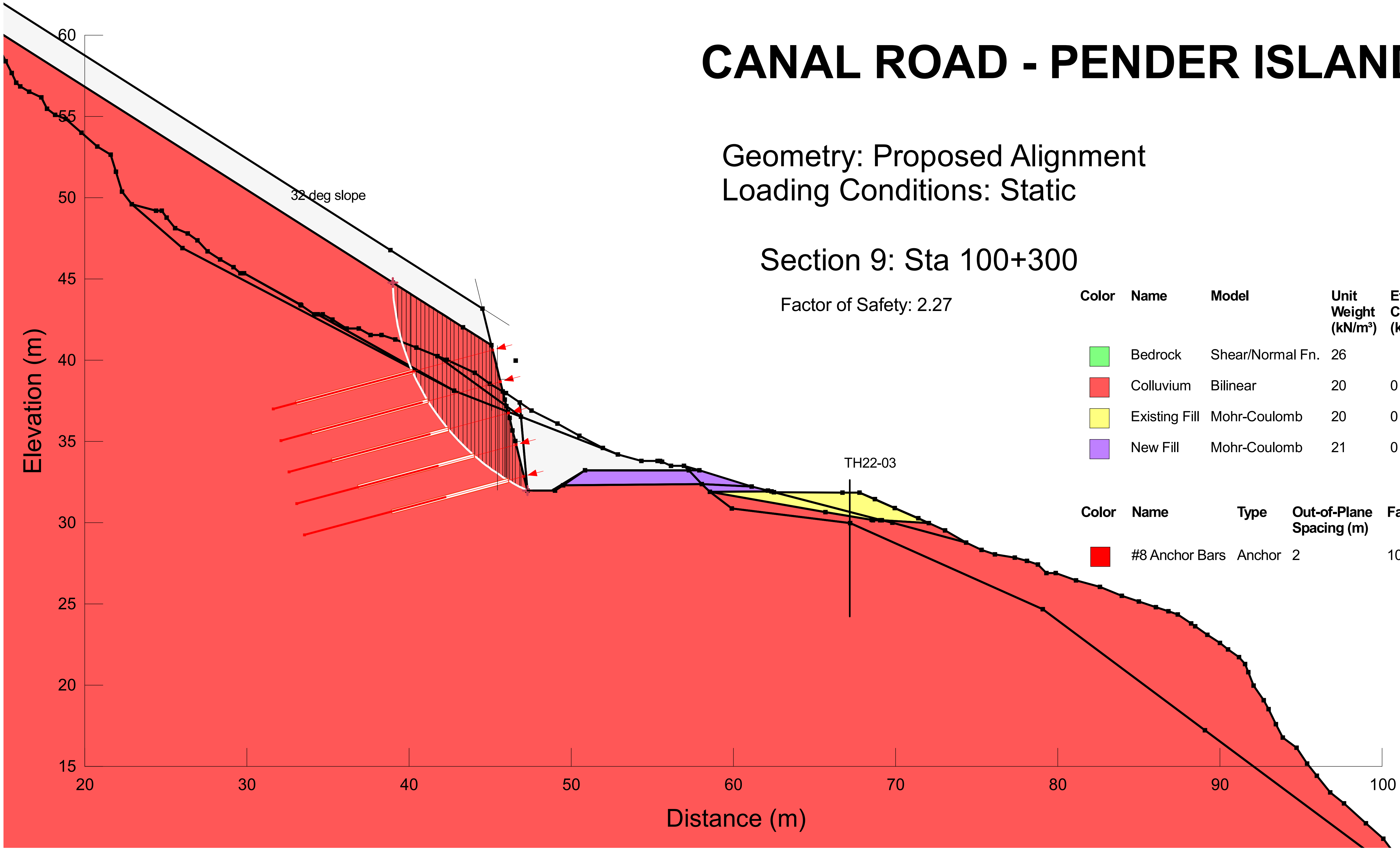
Sta 100+300 Upslope Wall Static	
jlu_Canal Road Slope Stability_33450.gsz	
2022-07-20	1:250

CANAL ROAD - PENDER ISLAND

Geometry: Proposed Alignment
Loading Conditions: Static

Section 9: Sta 100+300

Factor of Safety: 2.27



Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (kPa)
🟩	Bedrock	Shear/Normal Fn.	26				
🟥	Colluvium	Bilinear	20	0	45	36	50
🟨	Existing Fill	Mohr-Coulomb	20	0			
🟪	New Fill	Mohr-Coulomb	21	0			

Color	Name	Type	Out-of-Plane Spacing (m)	Factored Tensile Capacity
🟥	#8 Anchor Bars	Anchor	2	105.09 kN/m

Sta 100+300 Upslope Wall - Soil Nails
jlu_Canal Road Slope Stability_33450.gsz
2022-12-19 1:250

THURBER

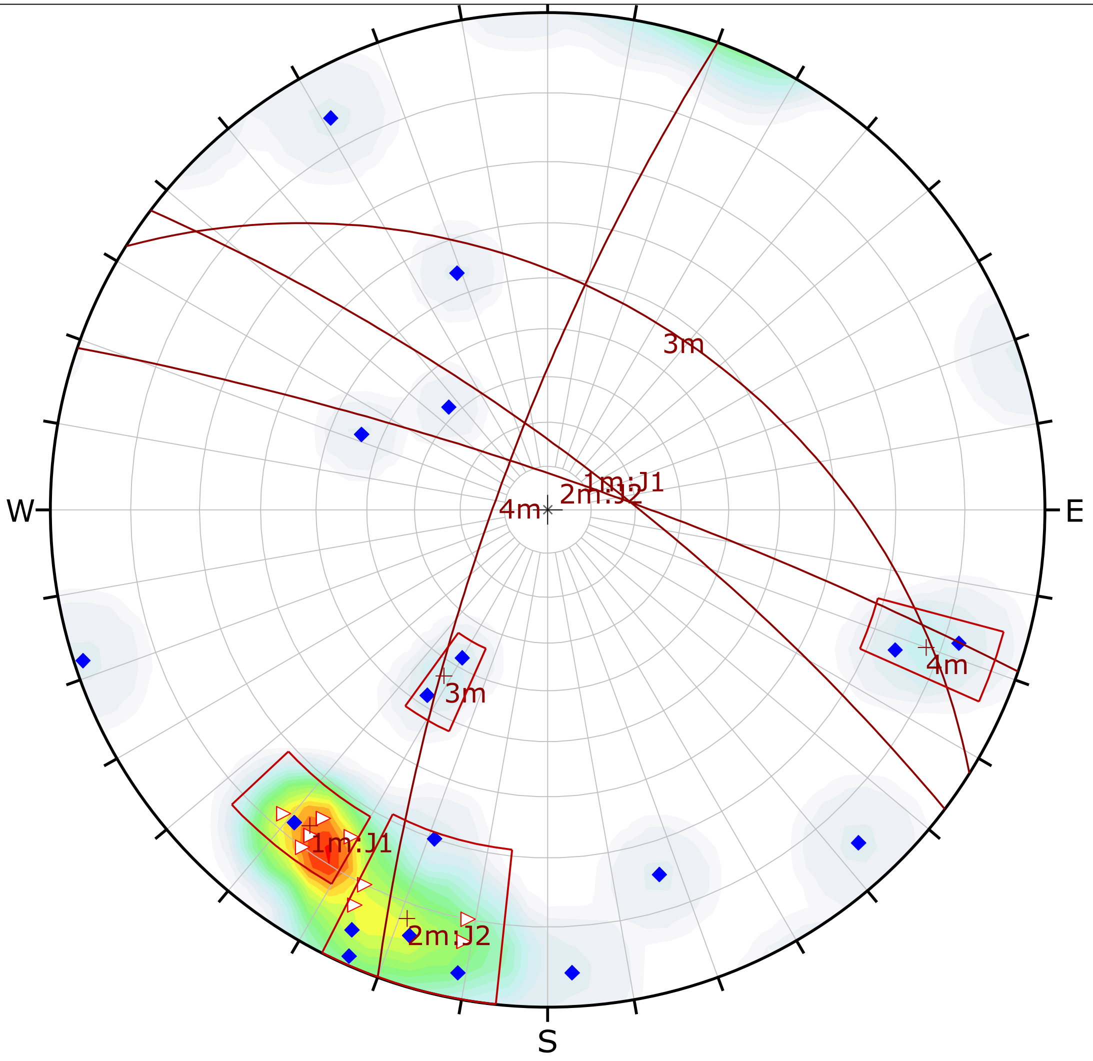
APPENDIX G ROCK SLOPE ANALYSIS

THURBER

TABLE G1 - Rock Discontinuity Measurements

Dip	Dip Direction*	Rock Type
80°	026°	Siltstone
83°	026°	Siltstone
75°	031°	Siltstone
75°	036°	Siltstone
78°	036°	Siltstone
80°	036°	Siltstone
78°	041°	Siltstone
80°	371°	Siltstone
83°	371°	Siltstone
82°	288°	Conglomerate
86°	357°	Conglomerate
44°	112°	Conglomerate
84°	151°	Conglomerate
89°	024°	Conglomerate
86°	025°	Conglomerate
38°	030°	Conglomerate
84°	018°	Conglomerate
32°	136°	Conglomerate
54°	159°	Conglomerate
89°	072°	Conglomerate
87°	011°	Conglomerate
75°	343°	Conglomerate
74°	292°	Conglomerate
78°	039°	Conglomerate
48°	033°	Conglomerate
85°	317°	Conglomerate
70°	019°	Conglomerate

*Corrected for declination of 15.8° east



Symbol	ROCK TYPE	Quantity
◆	Conglomerate	18
▷	Siltstone	9

Color	Density Concentrations
⬜	0.00 - 1.20
⬜	1.20 - 2.40
⬜	2.40 - 3.60
🟦	3.60 - 4.80
🟦	4.80 - 6.00
🟦	6.00 - 7.20
🟦	7.20 - 8.40
🟩	8.40 - 9.60
🟩	9.60 - 10.80
🟩	10.80 - 12.00
🟩	12.00 - 13.20
🟩	13.20 - 14.40
🟨	14.40 - 15.60
🟨	15.60 - 16.80
🟨	16.80 - 18.00
🟨	18.00 - 19.20
🟧	19.20 - 20.40
🟧	20.40 - 21.60
🟥	21.60 - 22.80
🟥	22.80 - 24.00

Contour Data	Pole Vectors
Maximum Density	23.03%
Contour Distribution	Fisher
Counting Circle Size	1.0%

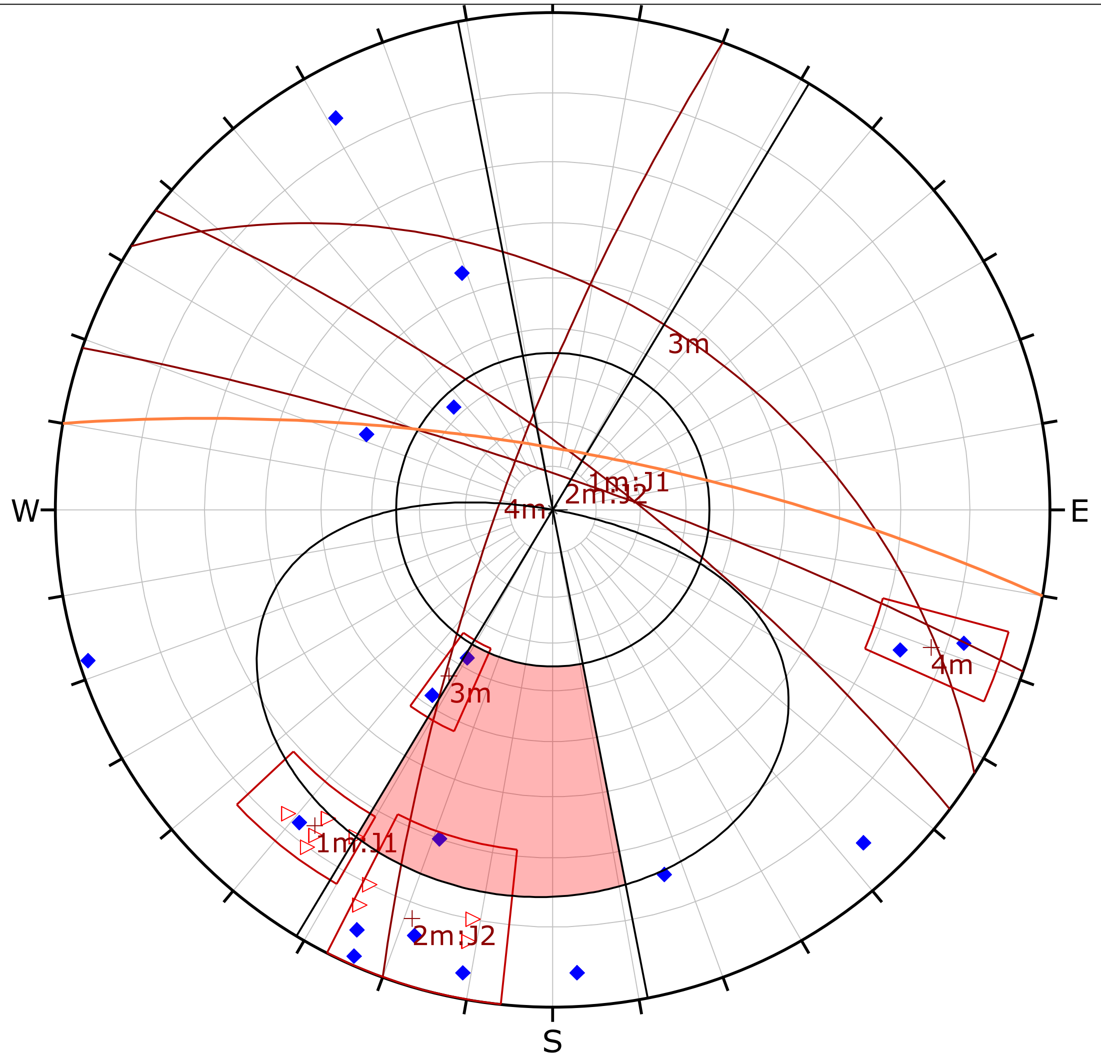
	Color	Dip	Dip Direction	Label
		Mean Set Planes		
1m	🟥	77	37	J1
2m	🟥	82	19	J2
3m	🟥	43	32	
4m	🟥	78	290	

Plot Mode	Pole Vectors
Vector Count	27 (27 Entries)
Hemisphere	Lower
Projection	Equal Angle

THURBER

Project	CANAL ROAD DIP SLIDE - PENDER ISLAND, BC		
Client	BC Ministry of Transportation and Infrastructure	*Title*	STEREONET 1
Project Number	33450		
Date	2023-01-05	*File Name*	20230105_Canal Road Outcrops_33450.dips8

DIPS 8.009



Symbol	ROCK TYPE	Quantity
◆	Conglomerate	18
▷	Siltstone	9

Kinematic Analysis	Planar Sliding
Slope Dip	76
Slope Dip Direction	10
Friction Angle	35°
Lateral Limits	21°

	Critical	Total	%
Planar Sliding (All)	3	27	11.11%
Planar Sliding (Set 1: J1)	1	6	16.67%
Planar Sliding (Set 2: J2)	1	9	11.11%
Planar Sliding (Set 3)	1	2	50.00%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1m	🟥	77	37	J1
2m	🟥	82	19	J2
3m	🟥	43	32	
4m	🟥	78	290	

Plot Mode	Pole Vectors
Vector Count	27 (27 Entries)
Hemisphere	Lower
Projection	Equal Angle

THURBER

DIPS 8.009

Project
CANAL ROAD DIP SLIDE - PENDER ISLAND, BC

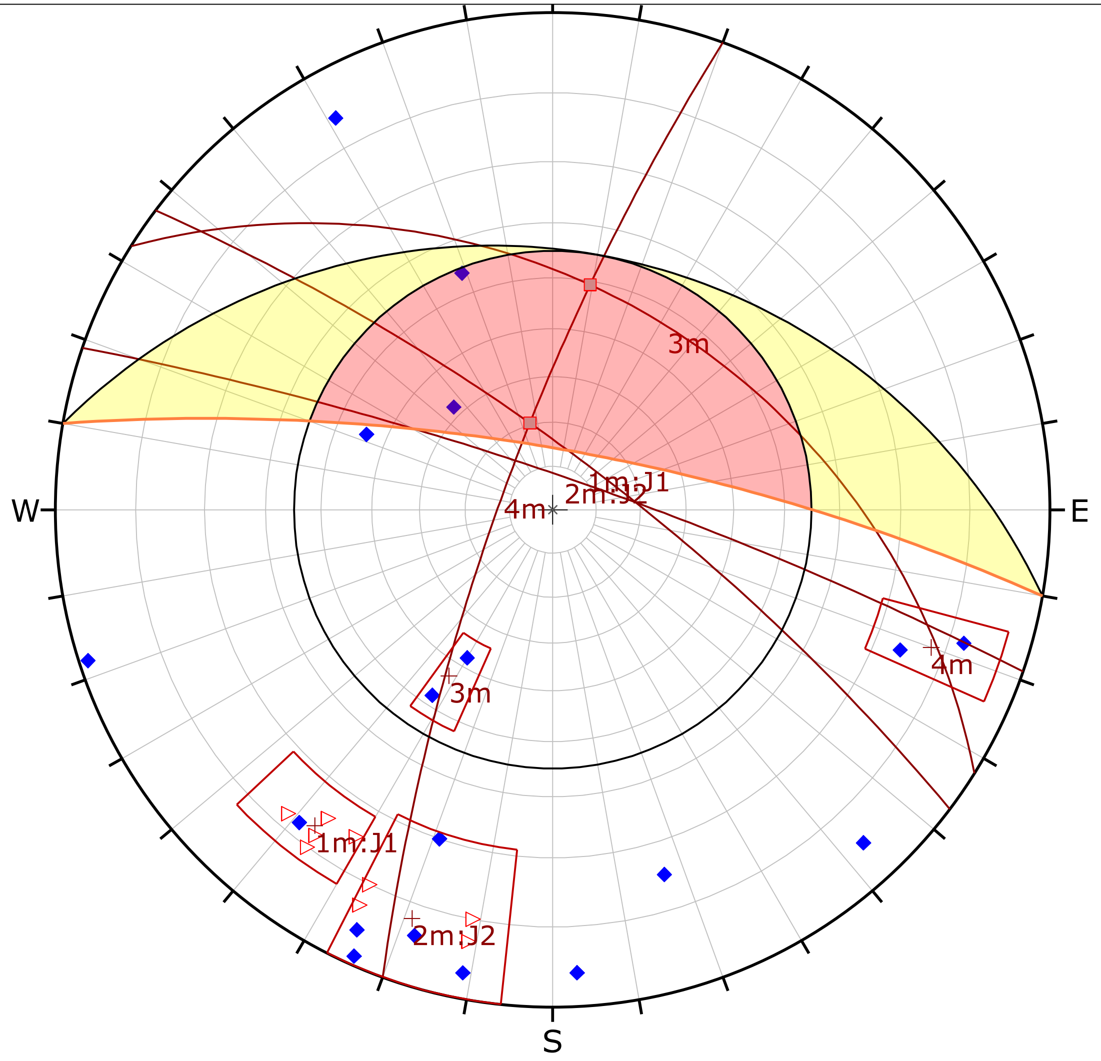
Client BC Ministry of Transportation and Infrastructure

Project Number 33450

Date 2023-01-05

Title STEREONET 2 - Planar Sliding

File Name 20230105_Canal Road Outcrops_33450.dips8



Symbol	ROCK TYPE	Quantity
◆	Conglomerate	18
▷	Siltstone	9
Symbol	**Feature**	
▨	Critical Intersection	

Kinematic Analysis	Wedge Sliding		
Slope Dip	76		
Slope Dip Direction	10		
Friction Angle	35°		
	Critical	**Total**	**%**
Wedge Sliding	2	6	33.33%

	Color	Dip	Dip Direction	Label
		Mean Set Planes		
1m	🟥	77	37	J1
2m	🟥	82	19	J2
3m	🟥	43	32	
4m	🟥	78	290	

Plot Mode	Pole Vectors
Vector Count	27 (27 Entries)
Intersection Mode	Mean Set Planes
Intersections Count	6
Hemisphere	Lower
Projection	Equal Angle

THURBER

DIPS 8.009

Project
CANAL ROAD DIP SLIDE - PENDER ISLAND, BC

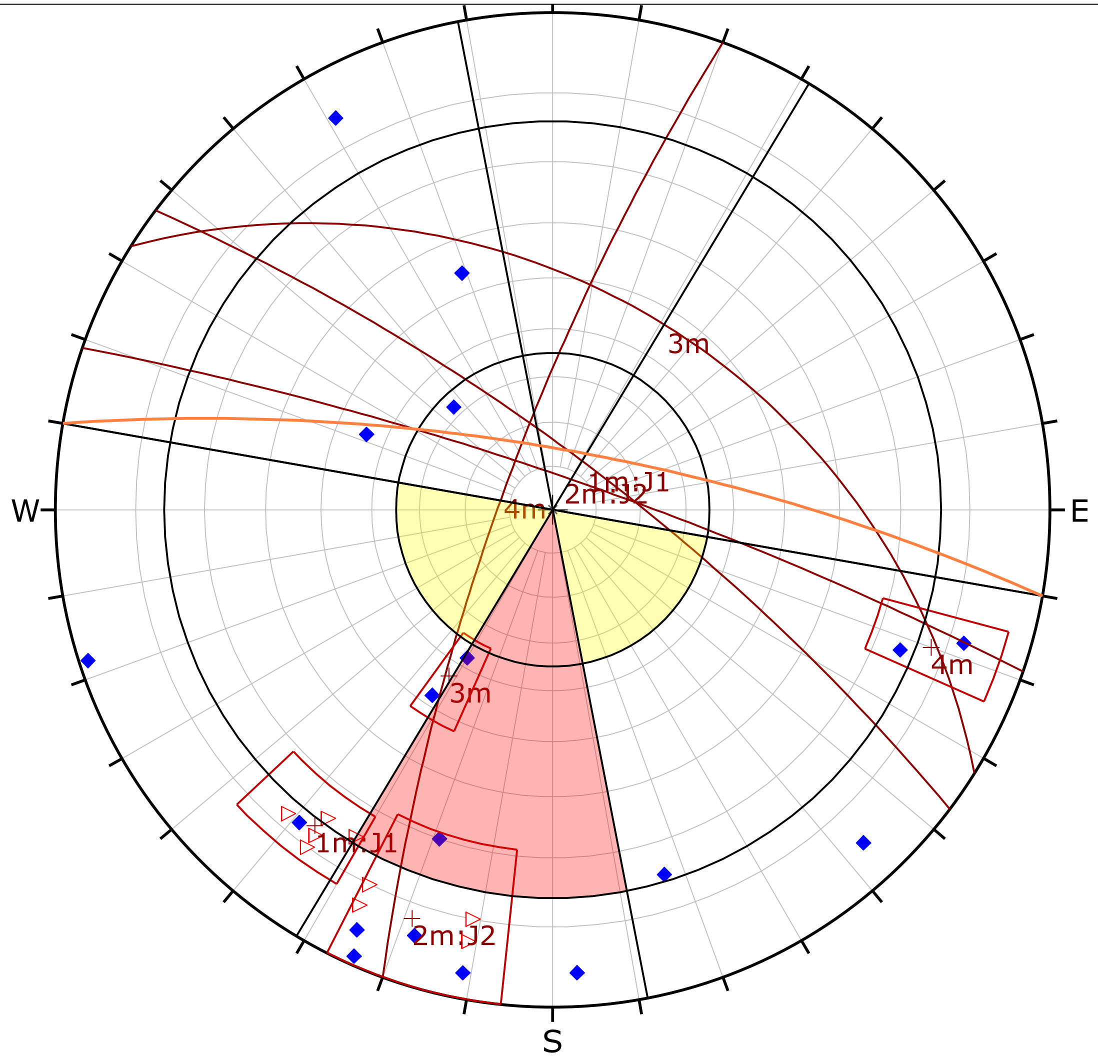
Client BC Ministry of Transportation and Infrastructure

Project Number 33450

Date 2023-01-05

Title STEREONET 3 - Wedge

File Name 20230105_Canal Road Outcrops_33450.dips8



Symbol	ROCK TYPE	Quantity
◆	Conglomerate	18
▷	Siltstone	9
Symbol	**Feature**	
▨	Critical Intersection	

Kinematic Analysis	Direct Toppling
Slope Dip	76
Slope Dip Direction	10
Friction Angle	35°
Lateral Limits	21°

	Critical	Total	%
Direct Toppling (Intersection)	0	6	0.00%
Oblique Toppling (Intersection)	0	6	0.00%
Base Plane (All)	3	27	11.11%
Base Plane (Set 1: J1)	1	6	16.67%
Base Plane (Set 2: J2)	1	9	11.11%
Base Plane (Set 3)	1	2	50.00%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1m	🟥	77	37	J1
2m	🟥	82	19	J2
3m	🟥	43	32	
4m	🟥	78	290	

Plot Mode	Pole Vectors
Vector Count	27 (27 Entries)
Intersection Mode	Mean Set Planes
Intersections Count	6
Hemisphere	Lower
Projection	Equal Angle

THURBER

Project
CANAL ROAD DIP SLIDE - PENDER ISLAND, BC

Client BC Ministry of Transportation and Infrastructure

Project Number 33450

Date 2023-01-05

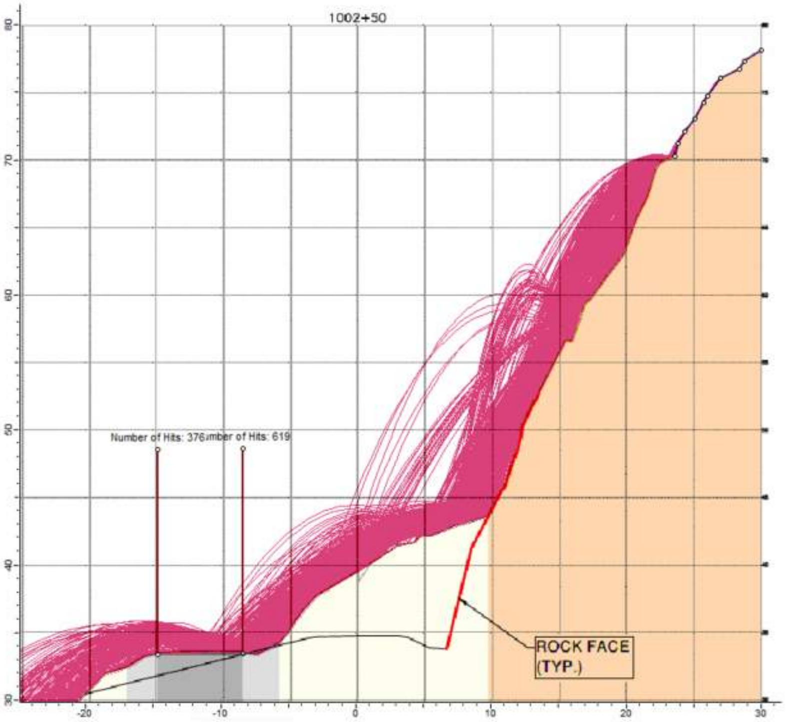
Title STEREONET 4 - Direct Toppling

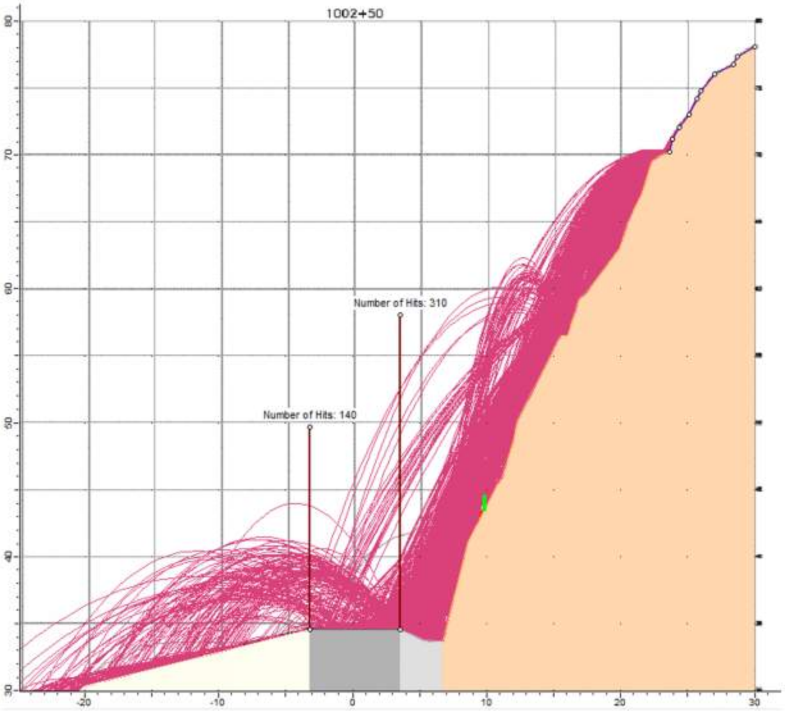
File Name 20230105_Canal Road Outcrops_33450.dips8

DIPS 8.009

THURBER

CANAL ROAD
SOUTH PENDER ISLAND





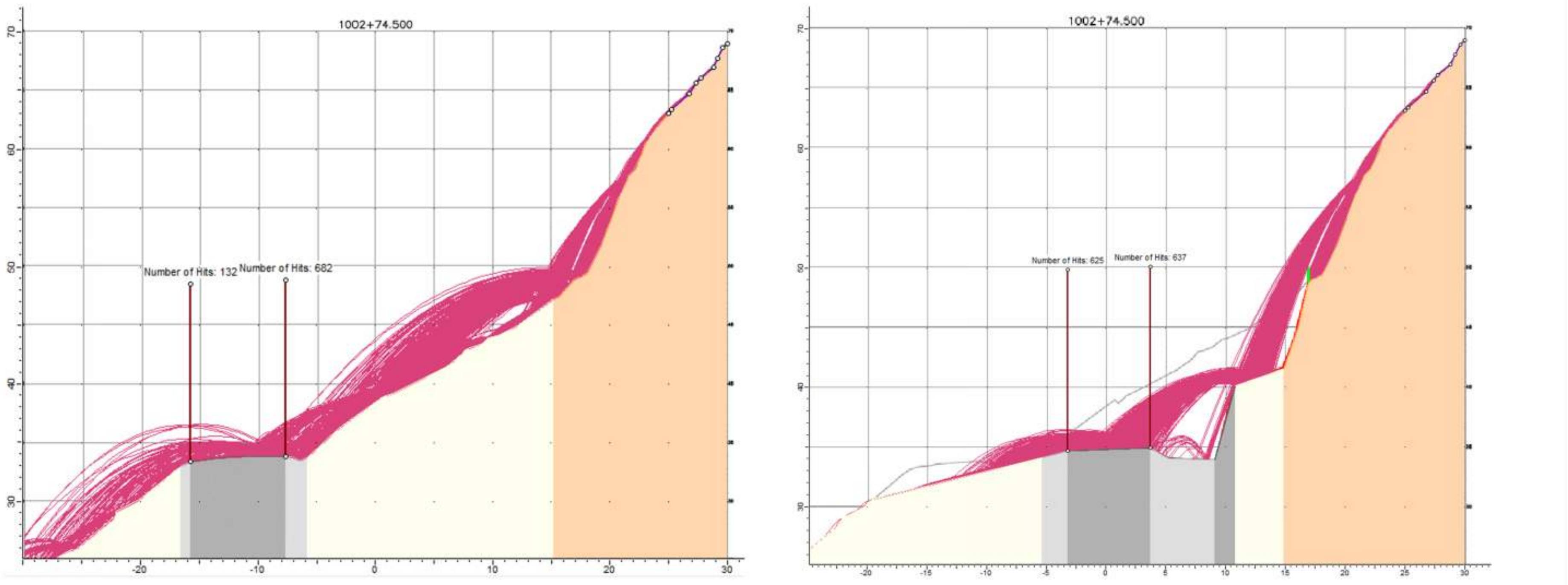
	Rock crossings Pre-construction (%)	Rock crossings post-construction with Mesh (%)	Relative Reduction of rock crossings (%)
Upslope EOP	62	31	**50**

Rockfall Analysis Stn 100+250

Client: MoTI
File No.: 33450
E-File: jbd_Stn100+250_Rockfall_Analysis.pdf

January 5, 2023

CANAL ROAD
SOUTH PENDER ISLAND



	Rock crossings Pre-construction (%)	Rock crossings post-construction with Mesh (%)	Relative Reduction of rock crossings (%)
Upslope EOP	68	63	**7**

Rockfall Analysis Stn 100+275

Client: MoTI
File No.: 33450
E-File: jbd_Stn100+275_Rockfall_Analysis.pdf

January 5, 2023

THURBER

APPENDIX H SEISMIC HAZARD

2015 National Building Code Seismic Hazard Calculation

INFORMATION: **Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836**
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 48.759N 123.225W **User File Reference:** Canal Road - Pender Island 2022-03-16 15:31 UT

Requested by: Jillian Usher, Thurber Engineering Ltd

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.608	0.437	0.322	0.149
Sa (0.1)	0.922	0.667	0.492	0.227
Sa (0.2)	1.134	0.823	0.612	0.280
Sa (0.3)	1.142	0.831	0.618	0.279
Sa (0.5)	1.016	0.731	0.532	0.230
Sa (1.0)	0.570	0.395	0.279	0.110
Sa (2.0)	0.336	0.226	0.154	0.057
Sa (5.0)	0.104	0.061	0.035	0.011
Sa (10.0)	0.036	0.021	0.012	0.004
PGA (g)	0.494	0.359	0.266	0.121
PGV (m/s)	0.733	0.506	0.356	0.138

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Natural Resources Canada Ressources naturelles Canada

Canada

APPENDIX I DURABILITY MEMO

THURBER ENGINEERING LTD.

MEMORANDUM

To: Ryan Gustafson, P.Eng
BC Ministry of Transportation and Infrastructure

Date: November 4, 2022

From: Warren Wunderlick, P.Eng, Jessica Dhami, M.A.Sc., GIT

File: 33450

Review: J. Suzanne Powell, Ph.D., P.Eng.

PENDER ISLAND – CANAL ROAD DIP SLIDE ROCK DURABILITY TESTING MEMO (REVISION 1)

This memorandum provides the results of durability testing undertaken on rock samples from the Canal Road dip slide site on Pender Island, BC. We have also summarized the suitability for re-use of both rock types based on the lab results. A previous version of this memorandum was issued on August 25, 2022. Since issuance, additional samples have been collected and analysed for durability. This memorandum summarises both the original results and the results of the additional samples.

It is a condition of this memorandum that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

1. BACKGROUND

Following the atmospheric river events of November 2021, the BC Ministry of Transportation and Infrastructure (MoTI) identified possible worsening of a known active landslide and is now proposing to stabilize this approximately 300 m long segment of Canal Road on South Pender Island, BC.

This section of Canal Road is located approximately 3 km from the Pender Canal Bridge, immediately to the east of the Mt. Norman Access Road and bordering the Beaumont-Gulf Islands National Park Reserve. The road is located at the crest of a steep slope, with an average slope angle of about 40° down to ocean (north). Bedrock or colluvium is exposed on the upslope (south) side of the road. Crown land and private property are located downslope of the slide and Parks Canada (Mt Norman) on the upslope (south) side.

Thurber has undertaken geotechnical investigations and provided recommendations for the road realignment; these deliverables have been provided separately. Two bedrock types have been identified at the site: a conglomerate of the De Courcy Formation and a siltstone/shale which may be from the Cedar District Formation or may be a siltstone lens of the De Courcy Formation.

The proposed future road realignment will require blasting into bedrock and will produce a significant surplus of blast-rock material. MoTI requested that Thurber conduct durability analysis on the two rock types to assess suitability for reuse as structural backfill, riprap, or pavement gravels.

The civil design team (McElhanney) is developing quantity estimates for the rock excavation separately.

2. SAMPLE COLLECTION

On July 21, 2022, Maggie Cramb, EIT of Thurber visited the Canal Road site to collect rock samples for durability testing. Samples of both rock types were collected by breaking off outcrop fragments using a rock hammer. Approximate locations of sample collection are shown on Figure 1 (attached).

At request of MoTI, On September 13, 2022, Jessica Dhami, GIT of Thurber returned to the site accompanied by Alex Hutter, GIT of MoTI, to collect additional samples of the conglomerate. A hand operated hammer drill, rock hammer, and mallet were used to collect conglomerate samples at six discrete locations along the outcrop. Each sample was approximately 18 to 19 L in volume (one 5-gallon pail). Sample locations were marked in the field and surveyed by McElhanney Ltd. Locations of each sample are shown on Figure 1 (attached).

3. LABORATORY TESTING

3.1 Initial Samples

Samples collected in July were sent to our Edmonton Asphalt and Advanced Aggregate Laboratory for testing. The *Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus* (ASTM D6928-17) test was run on both rock types.

3.2 Secondary Samples

Samples collected in September were sent to Golder Associates Inc.’s (Golder) laboratory in Burnaby, BC for analysis. Due to the small sample size, samples 3 and 4 as well as samples 5 and 6 were combined to obtain volumes required for ASTM testing. *Testing Rock Slabs to Evaluate Soundness of Riprap by Use of Sodium Sulfate or Magnesium Sulfate* (ASTM D5240) was run on samples 1, 2, and 3/4, sample 5/6 did not have any rock slabs large enough for testing. *Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus* (ASTM D6928-17) and *Specific Gravity And Absorption of Rock For Erosion Control* (ASTM D6473) were run on all samples.

4. LABORATORY RESULTS

Laboratory results are presented in Table 1 below.

Table 1. Summary of laboratory results for durability testing.

		Magnesium Sulphate Soundness (ASTM D5240) Loss (%)	Micro-Deval (ASTM D6928) Loss Factor (%)	Specific Gravity (ASTM D6473) (Dry Basis)	Specific Gravity (ASTM D6473) (SSD Basis)	Apparent Specific Gravity (ASTM D6473)	Absorptivity (ASTM D6473) (%)
July Samples	**Siltstone**	-	90.7	-	-	-	-
	Conglomerate	-	29.3	-	-	-	-
September Samples	**Sa.1**	2.0	12.2	2.579	2.613	2.670	1.32
	Sa.2	4.3	19.5	2.515	2.574	2.673	2.36
	Sa.3/4	0.8	22.9	2.577	2.620	2.691	1.66
	Sa.5/6	-	21.2	2.478	2.547	2.662	2.79

5. MATERIAL RE-USE SUITABILITY

To determine material suitability for re-use as aggregate or riprap, we have compared laboratory results to requirements in Sections 202 and 205 of the *2020 Standard Specifications for Highway Construction Volume 1 of 2* (BC MoTI) (Standard Specs).

5.1 Re-Use as Aggregate

Based on the initial Micro-Deval testing, the siltstone material does not meet the criteria for re-use as any aggregate type. The conglomerate material may be suitable for use as Surfacing Aggregate, 25 mm or 50 mm Base Course, Subbase Aggregates, or Bridge End Fill, all of which have maximum Micro-Deval loss factors of ≤ 25% or ≤ 30%. Sa. 1 also meets the criteria for 75 mm Base Course which has a Micro-Deval loss factor of ≤ 17%.

5.2 Re-Use as Riprap

Based on the initial Micro-Deval testing, the siltstone material does not meet the criteria for re-use as Riprap. The allowable values for use as Riprap, as stated in Table 205-A of the Standard Specs are summarized in Table 2.

Table 2. Allowable values for Riprap from Table 205-A of the Standard Specs.

Property	Allowable Value
Specific Gravity	≥ 2.50
Absorption	≤ 2%
Soundness by use of Magnesium Sulphate	≤ 10%
Micro-Deval Abrasion Loss Factor	≤ 20%

Based on the results received from Golder, the conglomerate meets the allowable value for Specific Gravity except for Sa. 5/6 which marginally fails with a value of 2.478 (Dry Basis) but passes for SDD basis and Apparent Specific Gravity. Samples Sa.1 and Sa. 4/3 meet the criteria for absorptivity, while Sa. 2 and Sa. 5/6 have results above the minimum allowable value. Two of the samples, Sa. 1 and Sa. 2 have Micro-Deval loss factors within the allowable value, while Sa. 3/4 and Sa. 5/6 both have results higher than the allowable value. All samples are within the allowable value for Soundness by Magnesium Sulphate.

6. CONCLUSION

The siltstone is not suitable for re-use as either Aggregate or Riprap. The conglomerate durability results are acceptable for use as most Aggregate types and on the margin of acceptability for use as Riprap. Since material re-use is not proposed for this project, the results should be provided to any future users of the material to assess acceptability of the product.

The conglomerate appears visually, relatively uniform across the site. Variability within the laboratory results is likely a reflection of variability within the conglomerate, and not representative of discrete regions within the rock. Therefore, we consider averaging the laboratory results for the conglomerate across all samples would be acceptable to represent the material as a whole and inform any future users.

Regardless of the results of durability testing, blast rock is considered acceptable for reuse as embankment fill.

7. CLOSURE

We trust this provides you sufficient information for your needs at this time. If you have any questions or would like to discuss these recommendations, please contact us.

Attachments:
- Statement of Limitations and Conditions
- Figure 1 - Rock Sample Location Plan
- Thurber Lab Test Reports
- Golder Lab Reports

SIGNED ORIGINAL
SENT SEPARATELY

THURBER

STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client, the BC Ministry of Transportation and Infrastructure (MoTI) and Authorized Users as defined in the MoTI Special Conditions Form H0461d. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT THURBER'S WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS THURBER MAY EXPRESSLY APPROVE. Any use which an unauthorized third party makes of the Report, is the sole responsibility of such third party. Thurber accepts no responsibility whatsoever for damages suffered by any unauthorized third party resulting from use of the Report without Thurber's express written permission.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.

d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

THURBER

Pender Island - Canal Road Slide
Rock Sample Location Plan

Notes:
2021 base imagery obtained from CRD

FIGURE 1

Client: BC MoTI
File No: 33450

Date: October 27, 2022
Drawn by: JBD

THURBER ENGINEERING LTD.

4127 Roper Road
Edmonton, Alberta T6B 3S5
P: 780 438 1460 F: 780 437 7125

TEST REPORT

Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus

Client :	BC MOTI	Sampling Date :	July 21, 2022
Project :	Pender Island-Canal Road	Receiving Date :	July 26, 2022
Project No :	33450-20.22	Testing Date :	August 4, 2022
Series No :	N/A	Source :	N/A
Material Type :	Conglomerate	Standard :	ASTM D6928-17

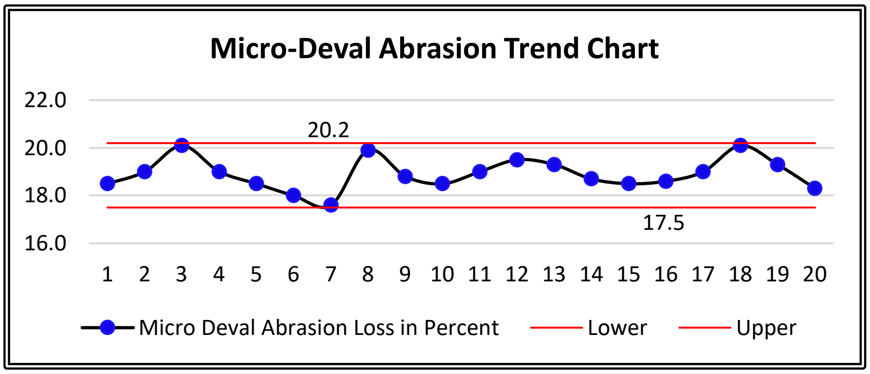
Sample Test Report

Description	Result
Grading Used	**8.2**
% Loss of Fines	**29.3**

Calibration Aggregate Test Report

Description	Result
% Loss of Fines	**19.5**
% Mean Loss	**18.9**

Sample : Reference Material (MTO-RM CA2)



Comments :

Tested By : NR

Reviewed By:

Note: The testing servicesare for the sole use of the designated client only. This report constitutes a testing service only and does not represent any results interpretation or opinion regarding specification compliance or material suitability. Engineering interpretation will be provided by Thurber upon request.

THURBER ENGINEERING LTD.

4127 Roper Road
Edmonton, Alberta T6B 3S5
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TEST REPORT

Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus

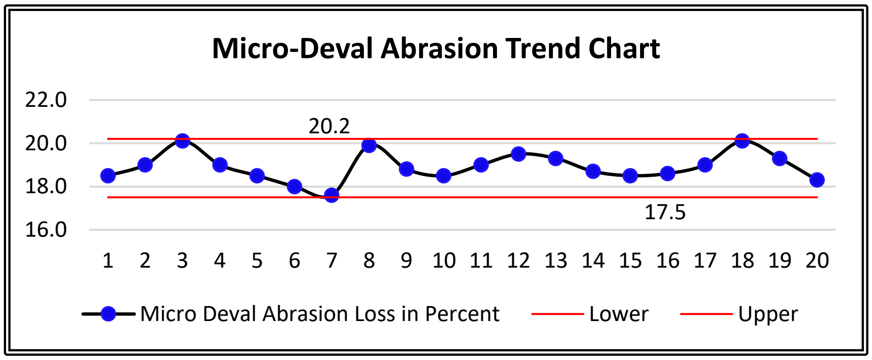
Client : BC MOTI
Project : Pender Island-Canal Road
Project No : 33450-20.22
Series No : N/A
Material Type : Siltstone/Shale

Sampling Date : July 21, 2022
Receiving Date : July 26, 2022
Testing Date : August 4, 2022
Source : N/A
Standard : ASTM D6928-17

Sample Test Report	
Description	Result
Grading Used	**8.2**
% Loss of Fines	**90.7**

Calibration Aggregate Test Report	
Description	Result
% Loss of Fines	**19.5**
% Mean Loss	**18.9**

Sample : Reference Material (MTO-RM CA2)



Comments :

Tested By : NR

Reviewed By:

Note: The testing servicesare for the sole use of the designated client only. This report constitutes a testing service only and does not represent any results interpretation or opinion regarding specification compliance or material suitability. Engineering interpretation will be provided by Thurber upon request.

GOLDER
MEMBER OF WSP

EVALUATION OF DURABILITY OF ROCK FOR EROSION CONTROL USING MAGNESIUM SULFATE

ASTM D5240

Client:	Ministry of Transportation and Infrastructure	Project No.:	20396349
Project:	Canal Road, Pender Island	Phase No.:	22009
Location ID:	Northing: 5400717, Easting: 483466	Date received:	September 19, 2022
Sample ID:	SA1	Sampled by:	Client
Date tested:	October 3 - 12, 2022	Tested by:	FF/KS

1.0. OBJECTIVE AND METHOD

The objective of this laboratory test was to evaluate the resistance to breakdown of rock samples upon repeated exposure to magnesium sulfate solution. This report comprises a testing service only.

Individual rock samples were reviewed to enable the selection of a sample suitable for the preparation of slab specimens that satisfy the requirements of ASTM D5240. Since much of the bulk rock material submitted testing was found to be too weak to produce the required number of specimens per clause 8.2.1 and 8.2.2 of the ASTM D5240, only a single slab specimen 65 ± 5 mm thick was cut using a water-cooled diamond saw; it was larger than 125 mm in length and width. The single specimen was prepared, and labelled as "1".

The test was conducted such that the specimen was subjected to five cycles of immersion and drying. The immersion portion of each cycle consisted of storing the specimen in a solution of magnesium sulfate for 16 hours, followed by 8 hours of drying in an oven at 110°C. The specimen was left in the oven when the test was interrupted.

The condition of the slab specimen was evaluated before and after the five cycles through macroscopic and microscopic examination using a binocular microscope with magnifications up to 50x. Observations were made of material dislodgement, pre-existing cracks, and newly developed deterioration. At each inspection, a photo and inspection log were generated for the specimen. The mass of the slab specimen was determined both prior to and at the end of the test procedure, and the percent loss by mass was calculated.

2.0. QUANTITATIVE ANALYSIS

The individual percent loss for the single specimen, by mass, is given in Table 1.

Table 1: Percent Mass Loss

Specimen ID	Measured Mass Loss *(%)*
1	2.0

The sample remained intact and did experience some loss of material. The percent loss was 2.0%.

3.0. QUALITATIVE ANALYSIS

The specimen was examined macroscopically and microscopically before and after the test.

3.1. Summary of Initial Observations

The specimen was a clastic sedimentary rock with grain size ranging from fine sand to gravel. The sample was moderately competent and exhibited oxidation/rusty staining on the weathered surfaces.

Description of the slab is provided below.

Slab 1: Dark grey, grey, buff-brown, locally rusty brown on weathered surface, fine to medium grained poorly sorted lithic sandstone. The sample has one 4 cm-thick layer of coarse pebbly sandstone. The matrix is silty. Sand grains have various shapes and lithological composition (volcanics, sandstone, coal, chert, minor carbonate, many rusty clasts) and range from angular to subrounded. The sample can be scratched with a steel knife with various degrees of effort (generally medium effort, locally harder, locally softer). Minor debris comes off upon touch. Locally the sample is porous and vuggy, with the pores/vugs visible under the microscope. Locally the vugs are coated with iron oxide. Reaction to HCl is localized to the carbonate grains. No response to the magnet was observed. The sample shows fractures and hairline fractures. Face I shows fractures and hairline fractures at random orientations, Face II shows fractures along the boundary between the fine sandstone and the gravelly sandstone.

3.2. Observation of Individual Specimens

Specimen ID	Initial Observation	Final Observation after 5 Cycles
1	Sample shows fractures and hairline fractures. Face I shows fractures and hairline fractures at random orientations. Face II shows fractures along the boundary between the fine sandstone and the gravelly sandstone.	Sample remained intact. One large piece came off and some fine-grained material dislodged upon touch. Face I some clasts came off from the gravelly layer. Face II no significant change.

3.3. Summary of Deterioration

The specimen remained intact and experience some material loss. The slab became more fragile with material loss upon slight touch. Fractures became wider and deeper. One cm-size chip detached form one edge.

4.0. OVERALL SUMMARY

The specimen was a grey, rusty brown, poorly sorted sandstone moderately weathered. Some loss of material and moderate deterioration was observed.

The data contained in this report pertain to the samples provided for testing and are not applicable to material from other locations or trial periods.

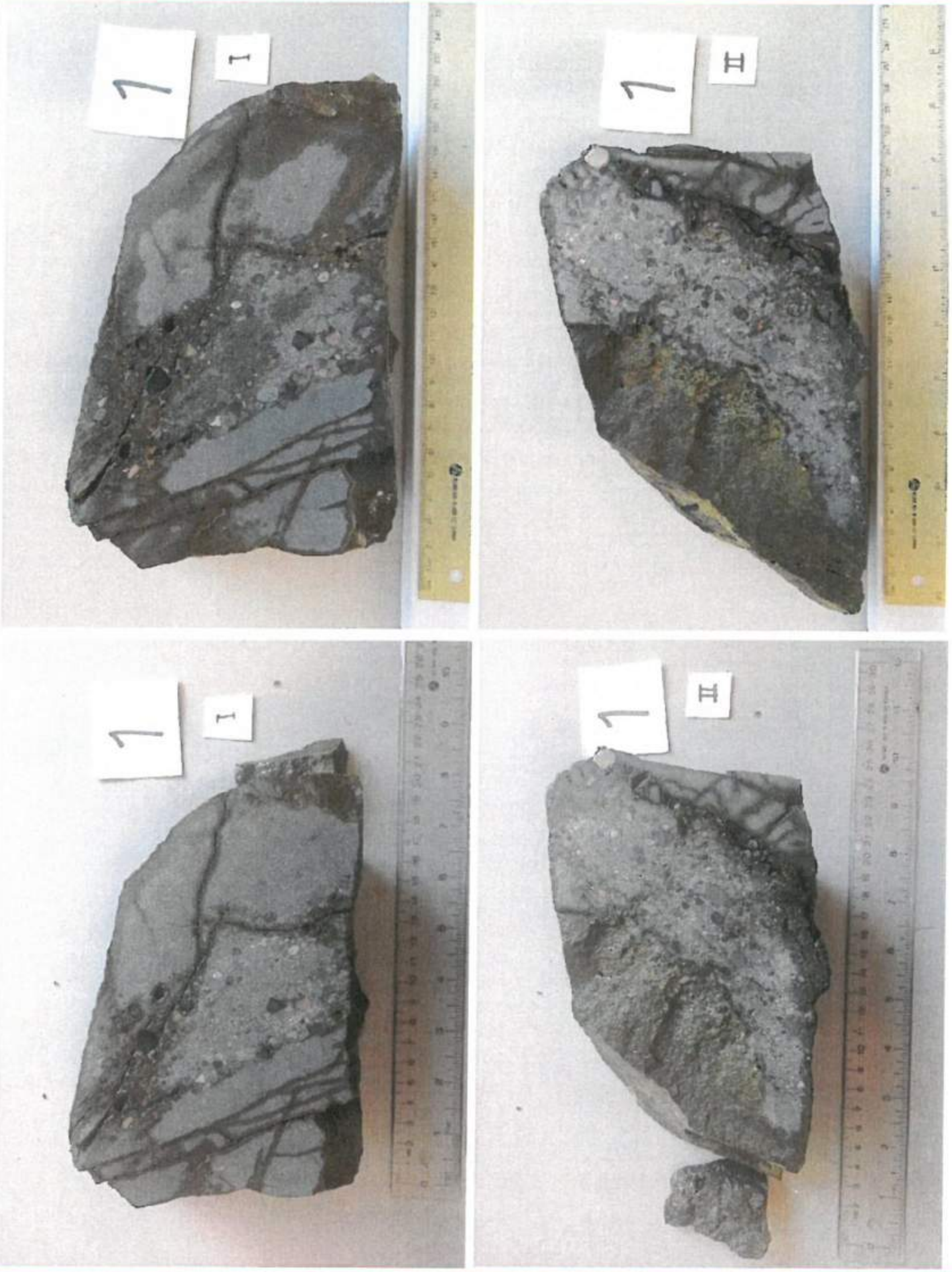
Reported by: F. Furlanetto, Ph.D., P.Geo.

Reviewed by: ________________________
F. Shrimer, P.Geo.

APPENDIX A: SPECIMEN SURFACE PHOTOGRAPHS

Photo 1.

Slab 1 prior to soundness cycles (upper photos) and after cycles (lower photos).



GOLDER
MEMBER OF WSP

EVALUATION OF DURABILITY OF ROCK FOR EROSION CONTROL USING MAGNESIUM SULFATE
ASTM D5240

Client:	Ministry of Transportation and Infrastructure	Project No.:	20396349
Project:	Canal Road, Pender Island	Phase No.:	22009
Location ID:	Northing: 5400716, Easting: 483445	Date received:	September 19, 2022
Sample ID:	SA2	Sampled by:	Client
Date tested:	October 3 - 12, 2022	Tested by:	FF/KS

1.0. OBJECTIVE AND METHOD

The objective of this laboratory test was to evaluate the resistance to breakdown of rock samples upon repeated exposure to magnesium sulfate solution. This report comprises a testing service only.

Individual rock samples were selected to produce slab specimens that satisfy the requirements of ASTM D5240. Bulk rock specimens submitted for this test were too weak and brittle to meet the minimum number of specimens required per clause 8.2.1 and 8.2.2 of the ASTM D5240 standard. Slab specimens 65 ± 5 mm thick were cut using a water-cooled diamond saw, with most specimens larger than 125 mm in length and width. Three lab specimens were prepared and labelled as 2A, 2B, 2C.

The test was conducted such that the specimens were subjected to five cycles of immersion and drying. The immersion portion of each cycle consisted of storing the specimens in a solution of magnesium sulfate for 16 hours, followed by 8 hours of drying in an oven at 110°C. Specimens were left in the oven when the test was interrupted.

The condition of each slab specimen was evaluated before and after the five cycles through macroscopic and microscopic examination using a binocular microscope with magnifications up to 50x. Observations were made of material dislodgement, pre-existing cracks, and newly developed deterioration. At each inspection, a photo and inspection log was generated for each specimen. The mass of each slab specimen was measured prior to and at the end of the test, and the percent loss by mass was calculated for each specimen. In cases of specimens breaking into smaller sub-specimens during testing, the largest remaining piece of the slab was used for mass loss calculations.

2.0. QUANTITATIVE ANALYSIS

The individual percent losses by mass for the specimens are summarized in Table 1.

Table 1: Percent Mass Loss

Specimen ID	Measured Mass Loss *(%)*
2A	5.6
2B	1.5
2C	5.8
Total	**4.3**

All samples remained intact and experienced some loss of material. The cumulative average percent loss by mass for the three specimens was 4.3 %.

3.0. QUALITATIVE ANALYSIS

Each specimen was examined macroscopically and microscopically before and after the test.

3.1. Summary of Initial Observations

The specimens are clastic sedimentary rocks with grain sizes ranging from sand to coarse gravel. They range in colour from brownish-grey, grey, and dark grey to buff and beige, with some rust-coloured zones. Some samples broke during preparation. The samples are weakly to moderately weathered and exhibit oxidation/rusty staining on the weathered surfaces.

Descriptions of the slabs are provided below.

Slab 2A: Light brownish grey, grey, beige, reddish orange, dark grey conglomerate. Reddish brown on the weathered surface. Pebble clasts are subrounded and have various shapes (some are subprismatic, some are oval, some are flattened). Grain size ranges from 2 mm to 4 cm. The sandy matrix is rust coloured, and the sand grains are angular to subangular. The lithological composition of the clasts includes volcanic, metamorphic, quartz, sandstone, siltstone, chert. Grains have a limonite-hematite coating. The sample can be scratched with a steel knife. Reaction to HCl is absent and no magnet response was observed. A few pits and vugs are visible under the microscope. The sample is fractured. Face I shows one fracture and one pit (a gravel clast that was removed during sample preparation), and one piece of slab is about to come off. Face II shows a few shallow pits.

Slab 2B: Light brownish grey, grey, beige, sand-supported conglomerate. Light rusty brown on weathered surface. Pebble clasts are subangular to subrounded and have various shapes (some are subprismatic, some are oval, some are flattened). Grain size ranges from a few mm to 5.5 cm, and average 1 cm. The sandy matrix is rust coloured, and the sand grains are angular to subangular. The lithological composition of the clasts includes volcanic, granitic, metamorphic, quartz, sandstone, siltstone, and chert. Grains have a limonite-hematite coating. The sample can be scratched with a steel knife. Reaction to HCl is absent and no magnet response was observed. The sample is fractured and a few pits and vugs are visible.

Face I shows a few pits and a couple of small fractures around the gravel clasts. Face II shows three discontinuous fractures and a few pits, and one gravel clast about to detach.

Slab 2C: Light brownish grey, grey, beige, reddish orange, dark grey, sand-supported conglomerate. Reddish brown on weathered surfaces. Pebble clasts are subrounded and have various shapes (some are subprismatic, some are oval, some are flattened). Grain size ranges from 2 mm to 4 cm, and average 4-5 mm. The sandy matrix is rust coloured, and the sand grains are angular to subangular. The lithological composition of the clasts includes volcanic, metamorphic, quartz, sandstone, siltstone and chert. Grains have a limonite-hematite coating. The sample can be scratched with a steel knife. Reaction to HCl is absent and no magnet response was observed. A few pits and vugs are visible under the microscope. Sample is fractured. Face I shows few pits (a result of gravel clasts that came off during sample preparation). Face II shows few shallow pits and three fractures.

3.2. Observation of Individual Specimens

Specimen ID	Initial Observation	Final Observation after 5 Cycles
2A	Few pits and vugs are visible under the microscope. Sample is fractured. Face I shows one fracture and one pit (a gravel clast came off during sample preparation), and one piece of slab is about to come off. Face II shows a few shallow pits.	Sample remained intact. The edges of the slab are more rounded. Minor material loss and some fine-grained material dislodges upon touch. Both faces became more porous and substantially rougher. Some gravel clasts came off.
2B	Sample is fractured and few pits and vugs are visible. Face I shows few pits and a couple of small fractures running around the gravel clasts. Face II shows three discontinuous fractures and a few pits, and one gravel clast about to detach.	Sample remained intact. Some gravel clasts came off, and the cracks around the pebbles became wider. Face II became more porous and rougher.
2C	Few pits and vugs are visible under the microscope. Sample is fractured. Face I shows few pits (as result of gravel clasts that came off during sample preparation). Face II shows few shallow pits and three fractures.	Sample remained intact. Some debris can be dislodged upon touch. The edges of the slab are rounded, and the overall porosity is much higher. The sample exhibits a more intense rusty color on the edges. Face I became more porous and rougher. The sandy matrix eroded, and the gravel clasts stand out. The fractures became wider and deeper. Face II one triangular piece came off from the right side. The surface is rougher and more porous. The gravel clasts stand out against the more eroded sandy matrix. Many gravel clasts came off.

3.3. Summary of Deterioration

All specimens remained intact and experienced variable material loss. The slabs became significantly more porous and more brittle, with material loss upon slight touch, and the edges became rounded.

The gravelly samples experienced differential loss, with the sandy matrix more degraded and the coarse clasts standing out or absent.

4.0. OVERALL SUMMARY

The specimens were brownish grey, grey, rusty brown, beige, and dark grey poorly sorted sandstone and conglomerate. The samples were weakly to moderately weathered and showed pits and fractures.

Some loss of material and moderate deterioration was observed.

The data contained in this report pertain to the samples provided for testing and are not applicable to material from other locations or trial periods.

Reported by: F. Furlanetto, Ph.D., P.Geo.

Reviewed by: ______________________
F. Shrimer, P.Geo.

APPENDIX A: SPECIMEN SURFACE PHOTOGRAPHS

Photo 1.

Slab 2A prior to soundness cycles (upper photos) and after cycles (lower photos).

Photo 2.

Slab 2B prior to soundness cycles (upper photos) and after cycles (lower photos).

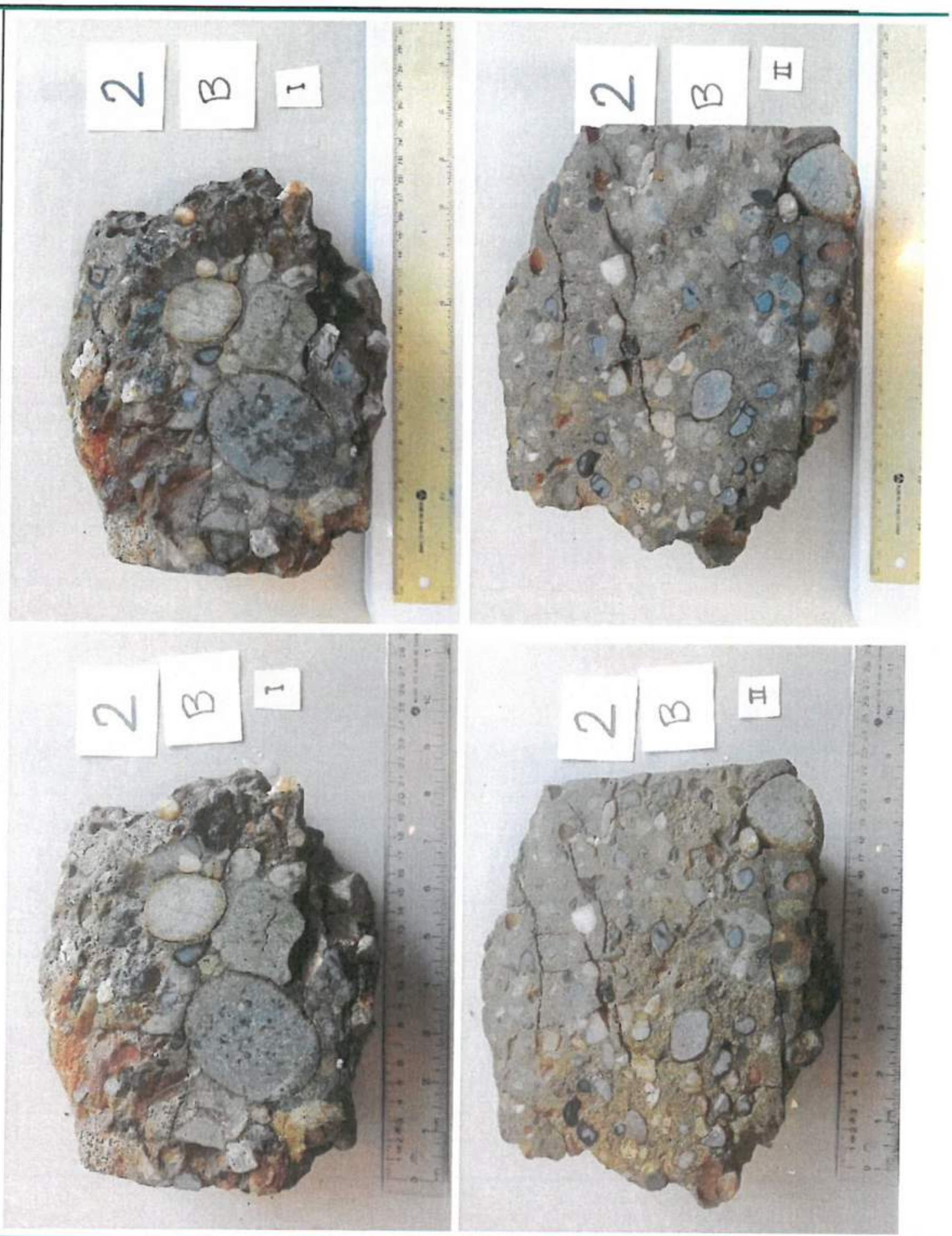
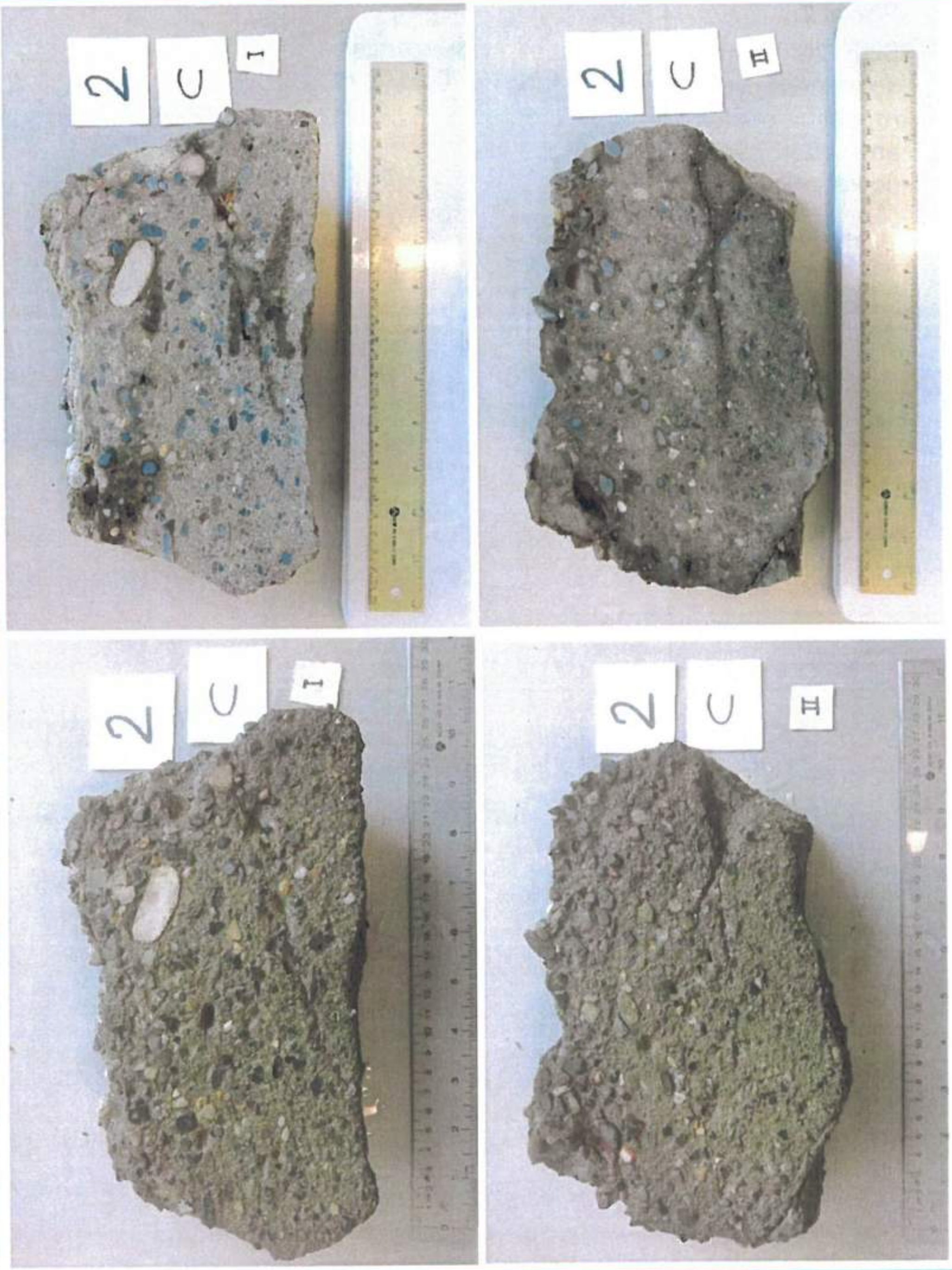


Photo 3.

Slab 2C prior to soundness cycles (upper photos) and after cycles (lower photos).



EVALUATION OF DURABILITY OF ROCK FOR EROSION CONTROL USING MAGNESIUM SULFATE

ASTM D5240

Client:	Ministry of Transportation and Infrastructure	Project No.:	20396349
Project:	Canal Road, Pender Island	Phase No.:	22009
Location ID:	Northing: 5400715, Easting: 483429 & Northing: 5400717, Easting: 483412	Date received:	September 19, 2022
Sample ID:	SA3 and SA4 Combined	Sampled by:	Client
Date tested:	October 3 - 12, 2022	Tested by:	FF/KS

1.0. OBJECTIVE AND METHOD

The objective of this laboratory test was to evaluate the resistance to breakdown of rock samples upon repeated exposure to magnesium sulfate solution. This report comprises a testing service only.

Individual rock samples were selected to enable the preparation of slab specimens that satisfy the requirements of ASTM D5240. Some of the bulk rock specimens that were submitted for testing were too weak to be able to produce a sufficient number of specimens as required by clause 8.2.1 and 8.2.2 of the ASTM D5240 standard. Slab specimens 65 ± 5 mm thick were cut using a water-cooled diamond saw, with most specimens larger than 125 mm in length and width. Four laboratory specimens were prepared and labelled as "3/4A", "3/4B", "3/4C", and "3/4D".

The test was conducted such that the specimens were subjected to five cycles of immersion and drying. The immersion portion of each cycle consisted of storing the specimens in a solution of magnesium sulfate for 16 hours, followed by 8 hours of drying in an oven at 110°C. Specimens were left in the oven when the test was interrupted.

The condition of each slab specimen was evaluated before and after the test through macroscopic and microscopic examination using a binocular microscope with magnifications up to 50x. Observations were made of material dislodgement, pre-existing cracks, and newly developed deterioration. At each inspection, a photo and inspection log were generated for each specimen. The mass of each slab specimen was determined both prior to and at the end of the test procedure, and the percent loss by mass was calculated for the individual specimens. In cases of specimens breaking into smaller sub-specimens during testing, the largest remaining piece of the slab was used for mass loss calculations.

2.0. QUANTITATIVE ANALYSIS

The individual percent losses by mass for the specimens are summarized in Table 1.

Table 1: Percent Mass Loss

Specimen ID	Measured Mass Loss *(%)*
3/4A	0.8
3/4B	0.4
3/4C	0.3
3/4D	1.4
Total	**0.8**

All samples remained intact and experienced minimal loss of material. The cumulative average percent loss by mass for the five specimens was 0.8 %.

3.0. QUALITATIVE ANALYSIS

Each specimen was examined macroscopically and microscopically before and after the test.

3.1. Summary of Initial Observations

The specimens are clastic sedimentary rocks with grain size ranging form sand to cobble. They are all dark grey, grey, buff, beige, and brownish grey poorly-sorted pebbly sandstone and conglomerate. The samples are weakly to moderately weathered and exhibit slight oxidation/rusty staining on the weathered surfaces.

Descriptions of the slabs are provided below.

Slab 3/4A: Dark grey, grey, beige, and brownish grey, medium-grained pebbly sandstone. Rusty brown on weathered surface. The sand grains are angular to sub-angular and have variable shapes. The pebble clasts are subrounded to rounded. The lithological composition of the grains comprises chert, quartz, volcanic, minor carbonate grains and a trace of shale/schist. The sample can be scratched with steel knife with effort, locally with minor effort, locally cannot be scratched. Reaction to HCl is localized on the carbonate grains, and no response to magnet was observed, except for some rare gravel clasts where there was a very weak attraction. The sample is pitted from grains that came off during sample preparation and shows a few fractures. Face I shows a few pits and one curved hairline fracture.

Slab 3/4B: Dark grey, grey, beige, and brownish grey, medium-grained pebbly sandstone. Rusty brown on weathered surface. The sand grains are angular to sub-angular and have varying shapes. The pebble clasts are subrounded to rounded. The lithological composition of the grains comprises chert, quartz, volcanic, minor carbonate grains and a trace of shale/schist. The sample can be scratched with steel knife with effort, locally with minor effort, locally cannot be scratched. Reaction to HCl is localized on the carbonate grains, and no response to magnet was observed, except for some rare gravel clasts where there was a very weak attraction. The sample is pitted as a result of grains that were removed during sample preparation and shows a few fractures. Face I shows a few pits, a few hairline fractures, and two small and one long fractures. Face II shows two small hairline fractures.

Slab 3/4C: Dark grey, grey, beige, and brownish grey, medium-grained pebbly sandstone. Rusty brown on weathered surfaces. The sand grains are angular to sub-angular and have varying shapes. The pebble clasts are subrounded to rounded. The lithological composition of the grains comprises chert, quartz, volcanics, minor carbonate grains and a trace of shale/schist. The sample can be scratched with steel knife with effort, locally with minor effort, locally cannot be scratched. Reaction to HCl is localized on the carbonate grains, and no response to magnet was observed, except for some rare gravel clasts where there was a very weak attraction. The sample is pitted as a result of the removal of grains during sample preparation and shows a few fractures. Face I shows a few pits. Face II shows the large rounded pebble clasts with microcracks.

Slab 3/4D: Grey, dark grey, light grey, greenish grey, brownish grey on weathered surface conglomerate. Matrix-supported. Clasts are subrounded to subangular and have various shapes and sizes up to 5 cm. The sand grains of the matrix are angular to subangular. The lithological composition of the clasts includes volcanic, siltstone, sandstone, chert, quartz, altered and weathered granitoids. The sample can be scratched with steel knife with variable effort. No response to the application of HCl was observed. Magnetis response was localized only on one volcanic clast of basaltic composition and was otherwise absent. The sample is pitted and fractured. Minor debris came off upon touch. Face I of the slab shows a few pits near the edges and hairline fractures, some around the harder clasts, some softer grains were fractured. Face II is pitted and shows abundant fractures, many going around the larger and harder clasts, and some fractured softer clasts.

3.2. Observation of Individual Specimens

Specimen ID	Initial Observation	Final Observation after 5 Cycles
3/4A	Sample is pitted (from grains that came off during sample preparation) and shows a few fractures. Face I shows a few pits and one curved hairline fracture.	Sample remained intact. Minimal loss of debris upon touch. Face I the top right corner became more porous and one new hairline fractured developed. Face II loss of material from the edges of the polished face, and the edges became rounded and more porous. Loss of some gravel clasts.
3/4B	Sample is pitted (from grains that came off during sample preparation) and shows a few fractures. Face I shows few pits, few hairline fractures, two small and one long fractures. Face II shows two small hairline fractures.	Sample remained intact. No change.
3/4C	Sample is pitted (from grains that came off during sample preparation) and shows a few fractures. Face I shows few pits. Face II shows the large rounded pebble clasts with microcracks.	Sample remained intact. No change.
3/4D	Sample is pitted and fractured. Minor debris came off upon touch. Face I of the slab shows a few pits near the edges and hairline fractures, some around the harder clasts, some softer grains were fractured. Face II is pitted and shows abundant fractures, many going around the larger and harder clasts, and some fractured softer clasts.	Sample remained intact. Some debris coming off upon touch. Both faces became slightly more porous, and some gravel clasts came off. The cracks around the larger pebbles became wider and deeper.

3.3. Summary of Deterioration

All specimens remained intact and experienced minimal material loss. Two slabs did not undergo any change. Two slabs became slightly more fragile with material loss upon slight touch. Few new hairline fractures developed and some increase in porosity was observed.

The coarser-grained conglomeratic samples experienced differential erosion with the sandy matrix more degraded and the coarse clasts standing out or absent. Some slabs became more porous, and the edges became rounded.

4.0. OVERALL SUMMARY

The specimens were brownish grey, grey, rusty brown, beige, and dark grey poorly sorted sandstone and conglomerate. The samples were weakly to moderately weathered and showed pits and fractures.

Minimal loss of material and minor deterioration was observed.

The data contained in this report pertain to the samples provided for testing and are not applicable to material from other locations or trial periods.

Reported by: F. Furlanetto, Ph.D., P.Geo.

Reviewed by: ____________________
F. Shrimer, P.Geo.

APPENDIX A: SPECIMEN SURFACE PHOTOGRAPHS

Photo 1.

Slab 3/4A prior to soundness cycles (upper photos) and after cycles (lower photos).

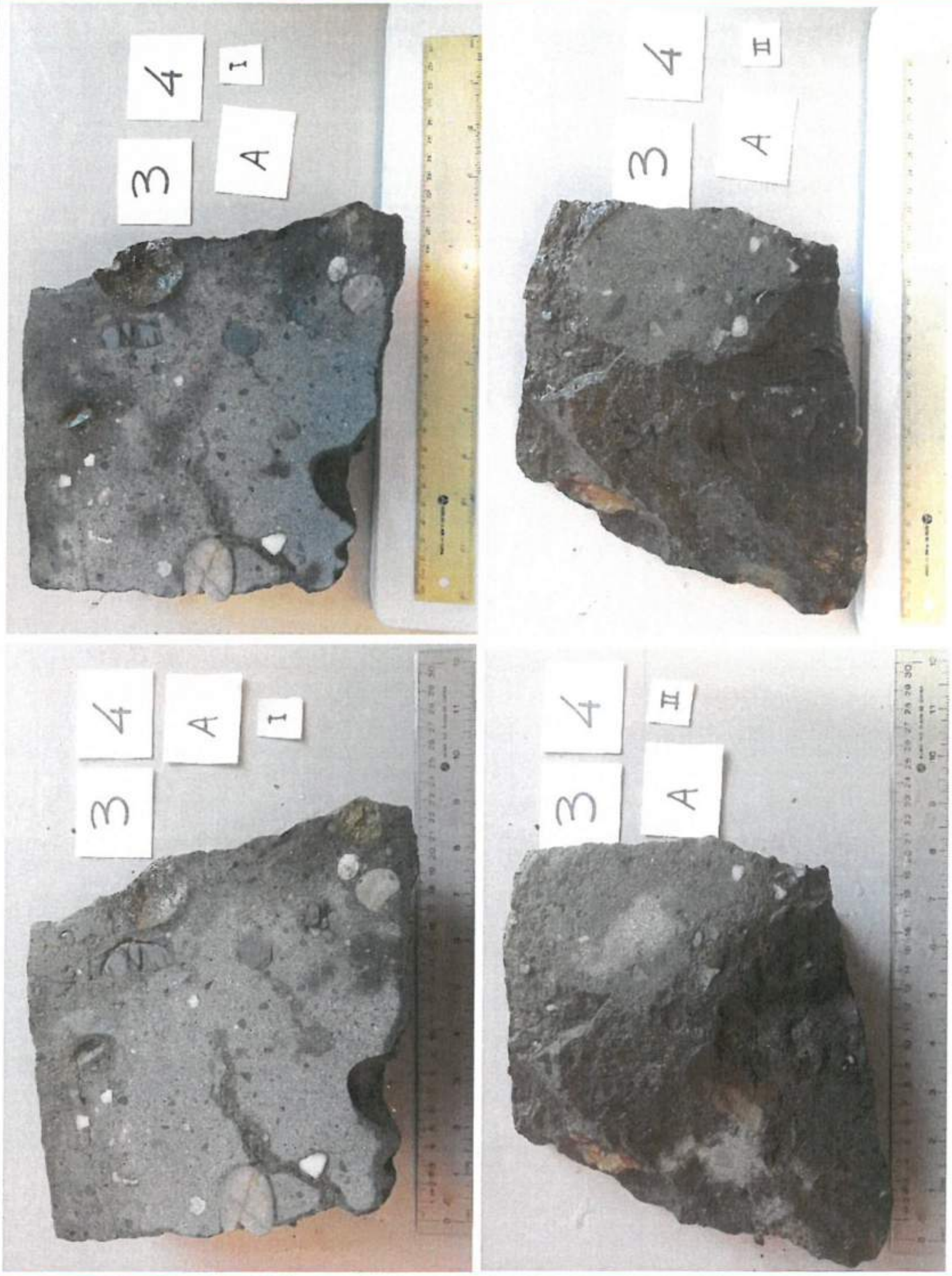


Photo 2.

Slab 3/4B prior to soundness cycles (upper photos) and after cycles (lower photos).

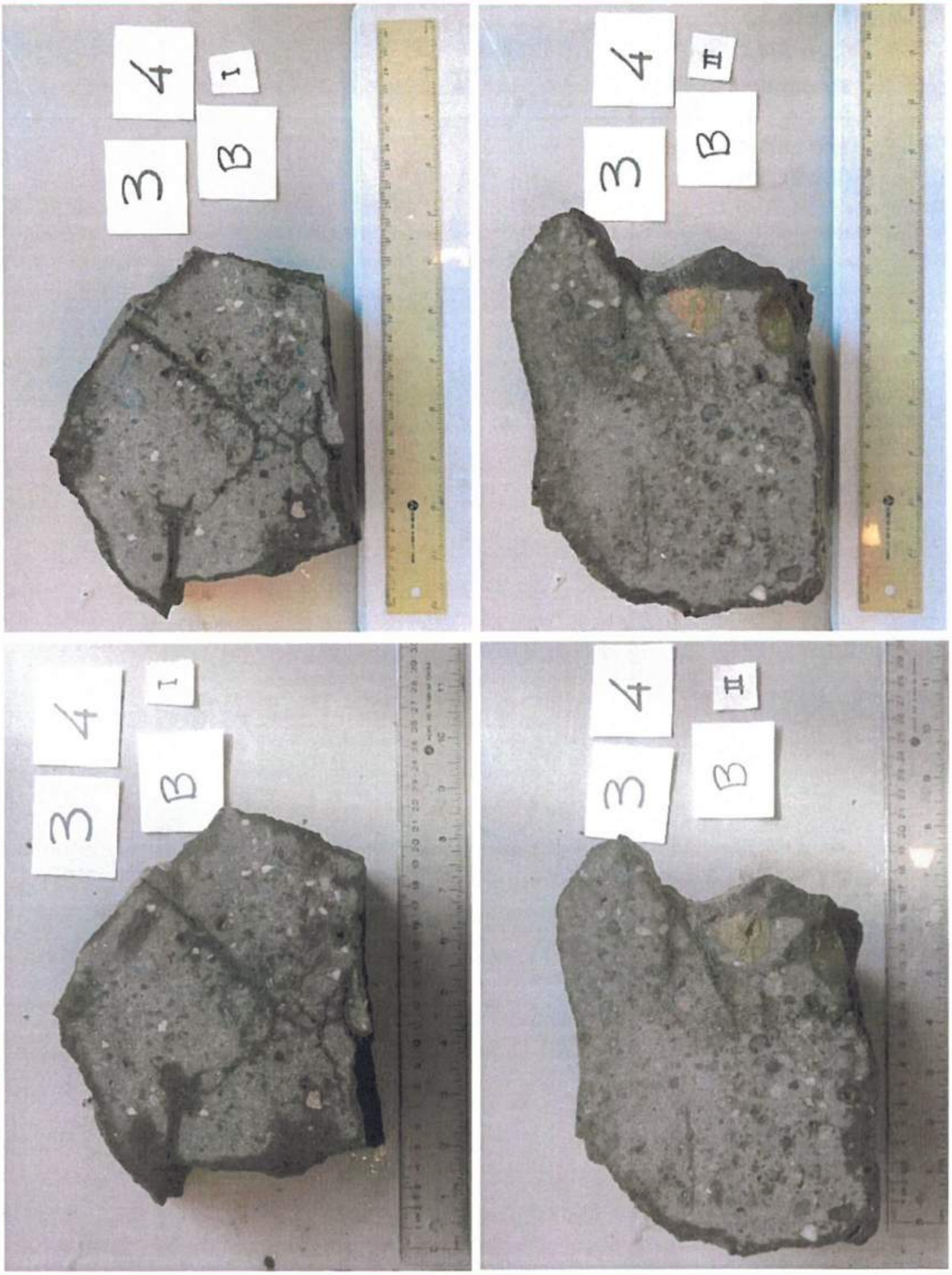


Photo 3.

Slab 3/4C prior to soundness cycles (upper photos) and after cycles (lower photos).

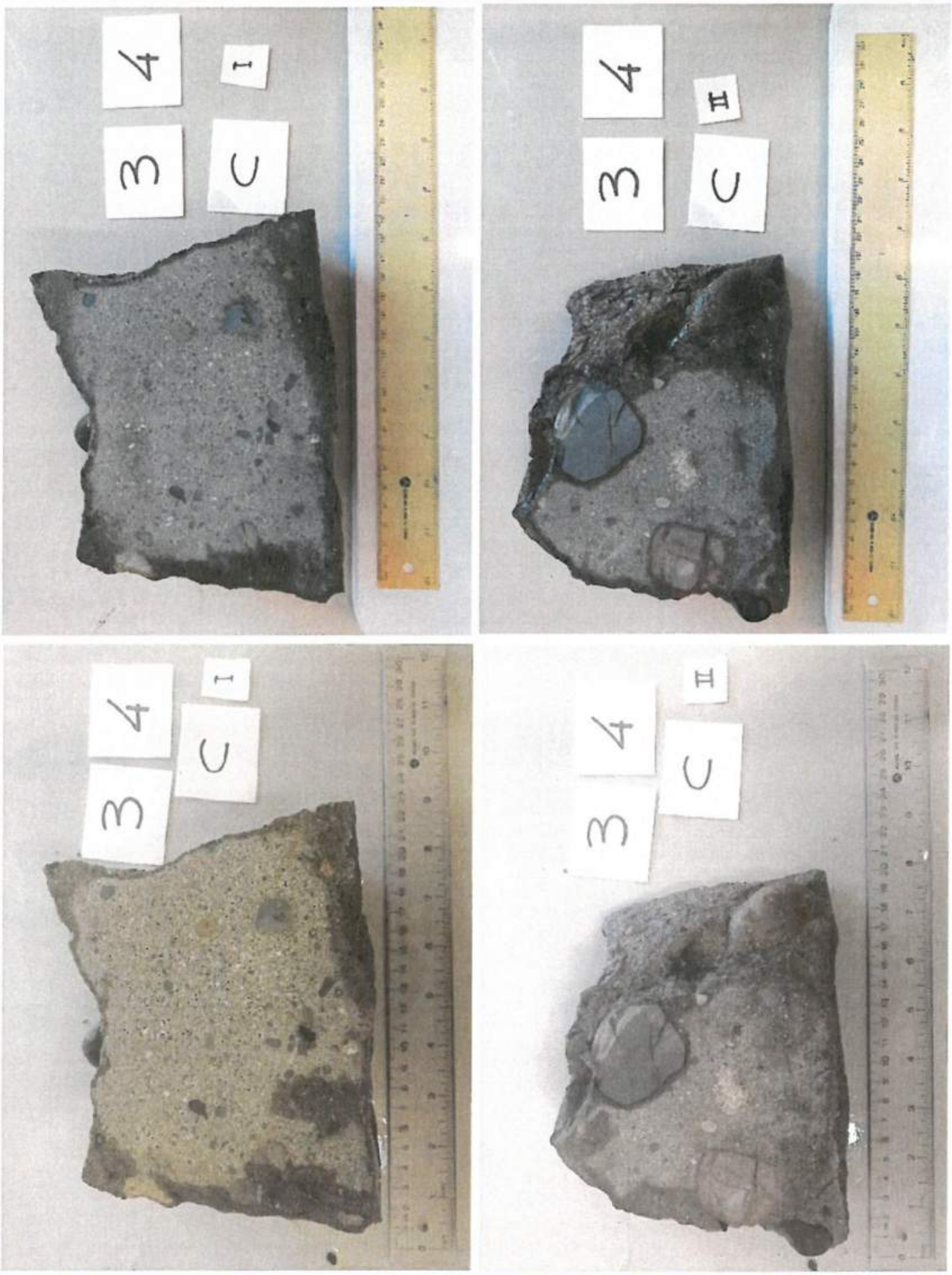


Photo 4.

Slab 3/4D prior to soundness cycles (upper photos) and after cycles (lower photos).

RESISTANCE OF COARSE AGGREGATE TO DEGRADATION BY ABRASION IN THE MICRO-DEVAL APPARATUS

ASTM D6928

Client:	Ministry of Transportation and Infrastructure	Project No.:	20396349
Project:	Canal Road, Pender Island	Phase No.:	22009
Location ID:	Northing: 5400717, Easting: 483466	Date Sampled:	September 13, 2022
Sample ID:	SA1	Sampled by:	Client
Date tested:	October 3, 2022	Tested by:	KS

Grading	**Section 8.2** 19 x 16 mm, 16 x 12.5 mm, and 12.5 x 9.5 mm Sieve Fractions
Loss at Conclusion of Test (%)	**12.2**

Notes:
Sample laboratory crushed to minus 19 mm prior to testing.
MTO RM CA2 Reference Aggregate loss was 13.0 %, tested on October 3, 2022. Valid range is between 11.4 - 14.8 %.

Reported by: K. Scribner

Reviewed by: ____________________
S. John, AScT

GOLDER
MEMBER OF WSP

RESISTANCE OF COARSE AGGREGATE TO DEGRADATION BY ABRASION IN THE MICRO-DEVAL APPARATUS

ASTM D6928

Client:	Ministry of Transportation and Infrastructure	Project No.:	20396349
Project:	Canal Road, Pender Island	Phase No.:	22009
Location ID:	Northing: 5400716, Easting: 483445	Date Sampled:	September 13, 2022
Sample ID:	SA2	Sampled by:	Client
Date tested:	October 3, 2022	Tested by:	KS

Grading	**Section 8.2** 19 x 16 mm, 16 x 12.5 mm, and 12.5 x 9.5 mm Sieve Fractions
Loss at Conclusion of Test (%)	**19.5**

Notes:
Sample laboratory crushed to minus 19 mm prior to testing.
MTO RM CA2 Reference Aggregate loss was 13.0 %, tested on October 3, 2022. Valid range is between 11.4 - 14.8 %.

Reported by: K. Scribner

Reviewed by: ______________
S. John, AScT

RESISTANCE OF COARSE AGGREGATE TO DEGRADATION BY ABRASION IN THE MICRO-DEVAL APPARATUS
ASTM D6928

Client:	Ministry of Transportation and Infrastructure	Project No.:	20396349
Project:	Canal Road, Pender Island	Phase No.:	22009
Location ID:	Northing: 5400715, Easting: 483429 Northing: 5400717, Easting: 483412	Date Sampled:	September 13, 2022
Sample ID:	SA3 and SA4 Combined	Sampled by:	Client
Date tested:	October 3, 2022	Tested by:	KS

Grading	**Section 8.2** 19 x 16 mm, 16 x 12.5 mm, and 12.5 x 9.5 mm Sieve Fractions
Loss at Conclusion of Test (%)	**22.9**

Notes:
Sample laboratory crushed to minus 19 mm prior to testing.
MTO RM CA2 Reference Aggregate loss was 13.0 %, tested on October 3, 2022. Valid range is between 11.4 - 14.8 %.

Reported by: K. Scribner

Reviewed by: ____________________
S. John, AScT

RESISTANCE OF COARSE AGGREGATE TO DEGRADATION BY ABRASION IN THE MICRO-DEVAL APPARATUS

ASTM D6928

Client:	Ministry of Transportation and Infrastructure	Project No.:	20396349
Project:	Canal Road, Pender Island	Phase No.:	22009
Location ID:	Northing: 5400713, Easting: 483390 Northing: 5400710, Easting: 483362	Date Sampled:	September 13, 2022
Sample ID:	SA5 and SA6 Combined	Sampled by:	Client
Date tested:	October 3, 2022	Tested by:	KS

Grading	**Section 8.2** 19 x 16 mm, 16 x 12.5 mm, and 12.5 x 9.5 mm Sieve Fractions
Loss at Conclusion of Test (%)	**21.2**

Notes:
Sample laboratory crushed to minus 19 mm prior to testing.
MTO RM CA2 Reference Aggregate loss was 13.0 %, tested on October 3, 2022. Valid range is between 11.4 - 14.8 %.

Reported by: K. Scribner

Reviewed by: ____________
S. John, AScT

GOLDER
MEMBER OF WSP

SPECIFIC GRAVITY AND ABSORPTION OF ROCK FOR EROSION CONTROL

ASTM D6473

Client:	Ministry of Transportation and Infrastructure	Project No.:	20396349
Project:	Canal Road, Pender Island	Phase No.:	22009
Location ID:	Northing: 5400717, Easting: 483466	Date Sampled:	September 13, 2022
Sample ID:	SA1	Sampled by:	Client
Date tested:	October 4, 2022	Tested by:	KS

Specimen ID	Mass (g)	Specific Gravity (Dry Basis)	Specific Gravity (SSD Basis)	Apparent Specific Gravity	Absorption (%)
1	1511.9	2.553	2.592	2.657	1.53
2	1937.6	2.605	2.634	2.683	1.11
AVERAGE		**2.579**	**2.613**	**2.670**	**1.32**

Note: Insufficient amount of sample provided for testing; minimum five rock specimen requirement could not be met.

Reported by: K. Scribner

Reviewed by: ____________________
S. John, AScT

Notice: The test data given herein pertain to the sample provided and may not be applicable to material from other production zones/periods. This report constitutes a testing service only. Interpretation of the data given here may be provided upon request.

GOLDER
MEMBER OF WSP

SPECIFIC GRAVITY AND ABSORPTION OF ROCK FOR EROSION CONTROL

ASTM D6473

Client:	Ministry of Transportation and Infrastructure	Project No.:	20396349
Project:	Canal Road, Pender Island	Phase No.:	22009
Location ID:	Northing: 5400716, Easting: 483445	Date Sampled:	September 13, 2022
Sample ID:	SA2	Sampled by:	Client
Date tested:	October 4, 2022	Tested by:	KS

Specimen ID	Mass (g)	Specific Gravity (Dry Basis)	Specific Gravity (SSD Basis)	Apparent Specific Gravity	Absorption (%)
1	1159.5	2.438	2.518	2.650	3.27
2	1848.5	2.564	2.610	2.689	1.81
3	1472.4	2.570	2.614	2.688	1.71
4	1104.4	2.489	2.555	2.665	2.65
AVERAGE		**2.515**	**2.574**	**2.673**	**2.36**

Note: Insufficient amount of sample provided for testing; minimum five rock specimen requirement could not be met.

Reported by: K. Scribner

Reviewed by: ____________________
S. John, AScT

GOLDER
MEMBER OF WSP

SPECIFIC GRAVITY AND ABSORPTION OF ROCK FOR EROSION CONTROL
ASTM D6473

Client:	Ministry of Transportation and Infrastructure	Project No.:	20396349
Project:	Canal Road, Pender Island	Phase No.:	22009
Location ID:	Northing: 5400715, Easting: 483429 Northing: 5400717, Easting: 483412	Date Sampled:	September 13, 2022
Sample ID:	SA3 and SA4 Combined	Sampled by:	Client
Date tested:	October 4, 2022	Tested by:	KS

Specimen ID	Mass (g)	Specific Gravity (Dry Basis)	Specific Gravity (SSD Basis)	Apparent Specific Gravity	Absorption (%)
1	1360.0	2.599	2.631	2.685	1.23
2	2303.7	2.544	2.595	2.682	2.02
3	2245.2	2.632	2.656	2.697	0.92
4	1227.7	2.632	2.656	2.696	0.91
5	1551.5	2.479	2.559	2.695	3.24
AVERAGE		**2.577**	**2.620**	**2.691**	**1.66**

Reported by: K. Scribner

Reviewed by: ____________________
S. John, AScT

Notice: The test data given herein pertain to the sample provided and may not be applicable to material from other production zones/periods. This report constitutes a testing service only. Interpretation of the data given here may be provided upon request.

SPECIFIC GRAVITY AND ABSORPTION OF ROCK FOR EROSION CONTROL

ASTM D6473

Client:	Ministry of Transportation and Infrastructure	Project No.:	20396349
Project:	Canal Road, Pender Island	Phase No.:	22009
Location ID:	Northing: 5400713, Easting: 483390 Northing: 5400710, Easting: 483362	Date Sampled:	September 13, 2022
Sample ID:	SA5 and SA6 Combined	Sampled by:	Client
Date tested:	October 4, 2022	Tested by:	KS

Specimen ID	**Mass (g)**	**Specific Gravity (Dry Basis)**	**Specific Gravity (SSD Basis)**	**Apparent Specific Gravity**	**Absorption (%)**
1	1130.3	2.487	2.552	2.661	2.64
2	1297.8	2.460	2.540	2.675	3.26
3	1645.3	2.518	2.573	2.664	2.18
4	1871.8	2.447	2.525	2.653	3.16
5	2034.2	2.475	2.543	2.655	2.73
AVERAGE		**2.478**	**2.547**	**2.662**	**2.79**

Reported by: K. Scribner

Reviewed by: ________________

S. John, AScT

APPENDIX J HISTORICAL ROCKFALL DATA

Maintenance Contractor Rockfall Reporting (MCRR)

Pender Island - Canal Rd (Rd 470)

Data Time Range: 1993 to current (Dec 5, 2022)

Part 1										Part 2																											Part 3		Maintenance Contractor			
General Information and Location							Travelled Distance (km)			Estimated Time & Date				V1 Site Conditions					V2 Site Conditions					Estimated Rockfall (m³)			V2 Max. Distance Rock Travelled							General								
Form Version	Incident Number	S A M s	Report For Month	Hwy / Road	Landmark	Direction from Landmark	From (km)	To (km)	Location	Estimated Date	Estimated Year	Estimated Month	Estimated Time	Precip. Last 48 Hrs.	Below Freezing Last 48 Hrs.	Above Freezing Last 48 Hrs.	Ditch Filled Snow/Ice	Ditch Filled Rock	Heavy Precip. Last 48 Hrs.	Freeze/Thaw Last 48 Hrs.	Dithch Filled With Compact Snow Or Ice	Vehicle Damage *(See Note)	Site Condition Comments	V1 Estimated Volume (m³)	V2 Ditch Estimated Volume (m³)	V2 Travelled Lanes Estimated Volume (m³)	1	2	3	4	5	6	7	Part 2 Comments	Verified By	Verified By Date	Frequent Rockfalls	Part 3 Comments	Name	Date	Phone No.	Name of Maintenance Contractor
1	0008019	01BQ	Jul-14	R-470	80380	E	0.18	0.18	Canal RD. South Pender Isl.	7/15/2014	2014	07 (Jul)				Y		Y						0.5 to 1.0 m³										No precip for 2 weeks / Steep cut / Rock on SIDE OF RD. + SURFACE			N		JOHN BRADLEY	7/15/2014		
1	0008006	01BP	May-07	R-470	80400	E	1.82	1.82	CANAL RD, STEEP SLOPE	5/8/2007	2007	05 (May)		Y		Y		Y						0.3 to 0.5 m³										STEEP SLOPE HIGH ROCK FALL AREA, LAST REPORT JAN 26/07			Y	SEE ABOVE	JOHN BRADLEY	5/10/2007		
1	0008005	01B0	Jan-07	R-470	80400	E	1.86	1.86	CANAL STEEP SLOPE	1/26/2007	2007	01 (Jan)				Y		Y						0.5 to 1.0 m³										Steep Slope, Rock and Debris found in ditch 3 m 4 times a year with in 50 meters each way			Y	See above	JOHN BRADLEY	1/29/2007		
1	AK470384277	01BP	May-13	R-470	80400	E	1.97	1.97	Canal RD. South Pender	5/26/2013	2013	05 (May)		Y										0.1 to 0.3 m³										Steep Slope, fractured rock			N	Rock slope with large fractures on rock above road.	JOHN BRADLEY	5/28/2013		
1	AK470384110	01BP	May-13	R-470	80400	E	1.97	1.97	Canal RD South Pender	5/26/2013	2013	05 (May)		Y										0.1 to 0.3 m³										Steep Slope, fractured rock			N	Rock slope with large fractures above road	JoHN BRADLEY	5/28/2013		
1	0008701	01BP	Nov-00	R-470	80400	E	2.20	2.20	CANAL RD. JUST E OF MT. NORMAN PARK ENTR.	11/16/2000	2000	11 (Nov)												0.3 to 0.5 m³										LARGE ROCK ONTO SHOULDER OF ROAD.			N		MARK STEVENS	11/24/2000		
1	0008003	01BP	Nov-04	R-470	80400	E			Canal Rd	11/7/2004	2004	11 (Nov)	3:00 PM	Y		Y								0.03 to 0.1 m³										Heavy rain day before, steep bank			Y	Steep bank, small slides quite Frequent	JOHN BRADLEY	11/8/2004		
1	008701	01BP	Nov-00	R-470		E	2.20		Canal Rd. just E of Mt. Norman Park Entrance	11/16/2000	2000	11 (Nov)												0.3 to 0.5 m³													Y	Large rock on shoulder of road.	Mark Stevens	11/24/2000		
1	AK470384061	01BP	May-13	R-470		E			Canal RD South Pender	5/26/2013	2013	05 (May)		Y										0.1 to 0.3 m³										Steep slope, fractured rock			N	Rock slope with large fractures in rock above road.	JoHN BRADLEY	5/28/2013		