Cottonwood River Slide – North Slide Stability Assessment

DRAFT

Prepared by BGC Engineering Inc. for:

R.F. Binnie & Associates Ltd.

May 23, 2024

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Attention: Maurizio Ponzini, P.Eng., Senior Vice President, Engineering, Senior Principal

Cottonwood River Slide – North Slide Stability Assessment

Please find the report attached. We appreciate the opportunity to collaborate with you on this challenging and interesting project.

All quality assurance and quality control processes have been completed as verified and signed off by the BGC Project Manager & Lead Geotechnical Engineer, Martin Devonald.

Should you have any questions, please do not hesitate to contact the undersigned.

Yours sincerely,

BGC Engineering Inc.
per:

Martin Devonald, M.Sc., LL.B., P.Eng.
Principal Geotechnical/Geological Engineer

EXECUTIVE SUMMARY

BGC Engineering Inc. (BGC) is undertaking a mitigation design for the Cottonwood River Slide which is impacting Highway No. 97, approximately 20 km northeast of Quesnel, British Columbia (BC). Approximately 400 m north of the Cottonwood River Slide, another landslide, referenced to as the North Slide, has encroached on the west (southbound) lane of Highway No. 97 causing slumping of the embankment and subsidence and cracking of the road surface. The design for Phase 2 of the Cottonwood River Slide stabilization project includes widening the existing embankment adjacent to the North Slide to accommodate the proposed highway realignment. This report presents a stability assessment of the North Slide considering the impact of the proposed embankment widening and highway realignment.

To support the assessment, BGC has reviewed lidar data and the site investigation data collected between December 2023 and March 2024. The site investigations included: drillholes, cone penetration testing, test pits, installation of geotechnical instruments to monitor slope movements and porewater pressures, and laboratory soil index testing.

The surficial soils within the landslide extents generally comprise of clay-dominant glaciolacustrine sediments underlain by an interbedded Diamicton and glaciolacustrine complex, followed by the Fraser Bend Formation, a predominately gravel-rich unit of both clast and matrix supported gravels.

The landslide morphology of the site indicates there to be three credible landslide mechanisms:

1. Local rotational landslide. A relatively shallow failure of the local embankment slope.
2. Translational landslide with a sliding layer at approximately elevation 744 m.
3. Deeper-seated translational landslide with a sliding layer at approximately elevation 735 m.

The stability assessment of the North Slide involved comparing baseline analysis Factor's of Safety (FoS's), i.e., of the existing highway configuration, to FoS's of subsequent forward-looking analyses, i.e., of the configuration following the proposed embankment widening and highway realignment. The assessment considered the three interpreted landslide mechanisms.

The stability assessment indicated that the proposed Phase 2 embankment widening and highway realignment does not negatively impact the stability of the North Slide (i.e., the FoS values calculated through the forward-looking analyses are equal to, or greater then, the baseline FoS values).

Based on the assessment completed herein, the following recommendations are provided:

- All recommendations relating to the embankment construction provided in the Phase 2 design report (BGC, March 20, 2024) are still valid.
- During construction of the proposed embankment widening, data should be collected from the SI installed in TH23-36 at least monthly. After construction of Phase 2, it is recommended that the SI casing be monitored at least once per year.

TABLE OF REVISIONS

Date	Revision	Remarks
May 23, 2024	Draft	

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1.0 INTRODUCTION

The Cottonwood River Slide complex is located approximately 20 km northeast of Quesnel, British Columbia along Highway No. 97. Approximately 400 m north of the Cottonwood River Slide, another landslide has encroached on the west (southbound) lane of Highway No. 97. This landslide is referred to as the 'North Slide' herein. The location of the North Slide is shown on Drawing 01. Construction of stabilization measures for Cottonwood River Slide will be completed in 3 Phases as follows:

1. Phase 1 – Riverbank erosion protection (constructed),
2. Phase 2 – Crest unloading and highway realignment (in tendering),
3. Phase 3 – stabilization works below the highway (in detailed design).

The design for Phase 2 of the Cottonwood River Slide stabilization project includes widening the existing embankment adjacent to the North Slide to accommodate the proposed highway realignment. Based on R.F. Binnie & Associates Ltd.'s (Binnie's) Phase 2 Drawing Package (Binnie, April 4, 2024), the highway adjacent to the North Slide is to be shifted approximately 10 m to 14 m east (away from the North Slide crest) and up to approximately 5 m of fill is required to bring the grade up to the pavement wearing surface level. An additional surcharge fill thickness of 2 m above the pavement wearing surface level is included in the Phase 2 construction tender to reduce post construction settlement of the underlying foundation soil. An annotated cross-section of the proposed realignment adjacent to the North Slide (Station 209+40) is provided as Figure A1 in Appendix A. This report presents a stability assessment of the impact of the proposed highway realignment and temporary surcharge loading on the North Slide, located west of the highway embankment.

1.1 Previous Work

The Cottonwood River Slide Phase 2 design was presented in BGC's Geotechnical Design Report (BGC, March 20, 2024). Although the March 20, 2024 design report does not characterize or assess the stability of the North Slide, information contained in the report, including the site geology, geotechnical soil units, and material parameters, were referred to in this assessment.

BGC completed an embankment stability assessment for the proposed Phase 2 embankment widening (BGC, April 18, 2024d). Information from the embankment stability assessment, including material parameters used in the stability analysis, were used in this assessment.

BGC has completed two site investigations in the vicinity of the North Slide, one in December 2023, and one in February and March 2024. The site investigation activities at the site included:

- Advancing two sonic drillholes, TH23-36 and TH23-36B. TH23-36 was drilled to a depth of 70.1 m and was completed with a slope inclinometer (SI) casing installation and two grouted-in vibrating wire piezometers (VWPs). TH23-36B (adjacent to TH23-36) was drilled to a depth of 9.8 m for the purpose of collecting Shelby tube samples. The drilling of TH23-36 and TH23-36B were completed in December 2023.

- Excavating one test pit (TP23-03), completed December 2023.
- Cone penetration testing (CPTu) with pore pressure measurement at five locations, CPT-BGC24-08 to CPT-BGC24-12, completed February 2024.
- Drilling of six geotechnical drillholes to depths between 5.4 m and 12 m. The drillholes were advanced with hollow stem augers and completed between February and March 2024. The following was completed in the drillholes:
  - Six Shelby tube samples were collected, with two samples from each of drillholes TH24-37A, TH24-38B, and TH24-39.
  - Nilcon vane shear testing (VST) at 1 m depth intervals from 2 m to 12 m depth, for a total of 11 tests at TH23-38A.
  - Installation of a single vibrating wire piezometer (VWP) at a depth between 5 m and 6 m within drillholes TH24-37, TH24-38C and TH24-39. A single channel datalogger was connected to each VWP.
  - Installation and development of a standpipe piezometer (SP) at one location, TH24-38B.

The locations of the site investigation activities listed above are shown on Drawing 02. Further details regarding the 2023 and 2024 site investigation activities are included in the following reports, respectively: the 2023 Geotechnical Data Report (BGC, March 22, 2024) and the Q1 2024 Geotechnical Data Report - DRAFT (BGC, April 18, 2024b).

A site reconnaissance to observe the slide and slide-affected portions of Highway 97 was completed by Martin Devonald (BGC) on September 26, 2023.

1.2 Scope of Work

The scope of the services for this assessment was presented in BGC's work plan dated April 18, 2024 (BGC, April 18a, 2024). The scope includes:

- Using the results of the previous site investigations completed in the vicinity of the North Slide to develop a geological model on one cross-section which includes geologic units, groundwater conditions, and the interpreted slide surface geometry for the North Slide.
- Based on the interpreted geologic model, slide mechanism, and slide surface geometry, complete two-dimensional (2D) limit equilibrium slope stability analysis to assess the impact of the proposed embankment widening to the stability of the North Slide. Design of slope stabilization measures are not included in BGC's scope of work.
- Complete a lidar change detection analysis of the North Slide using two data sets.
- Prepare a report that provides the items listed above (this report).

1.3 Terms of Reference

Work carried out by BGC is governed by the November 2, 2022 sub-consultant agreement between Binnie and BGC.

2.0 SITE CONDITIONS

2.1 Physiography

The North Slide is located on the east approach slope of the Cottonwood River Valley and is approximately 400 m north of the Cottonwood River Slide within the Faser Plateau. Within the valley reach the river has cut down through a Pleistocene glacial till plateau, the underlying glaciolacustrine sediments, and in some locations, the underlying bedrock.

The North Slide is evidenced by the arcuate headscarp, highway subsidence, and the hummocky ground surface within the landslide body (Drawings 01 and 02). Highway 97 is located east of the North Slide's headscarp. The landslide has encroached on the west (southbound) lane of Highway 97 causing slumping of the embankment and subsidence and cracking of the road surface (Site Photos 1 and 2, Appendix B). It is understood that the B.C. Ministry of Transportation and Infrastructure (MoTI) has re-surfaced the highway adjacent to the North Slide to repair subsidence and cracking of the existing pavement structure. The date that this work was completed is not presently known to BGC.

2.2 Site Geology

The interpreted geological units encountered at the North Slide are based on the geotechnical investigation data at the North Slide (BGC, March 22, 2024; BGC, April 18, 2024b) and a literature review (Clague, 1988; Rouse and Mathews, 1979), and considers the data gathered and reviewed to date from the proximate Cottonwood River Slide (BGC, March 20, 2024). It is noted that limited geotechnical investigation data has been collected at the site downslope of the highway, and given that the glacial history at the site is notably complex, the geology model described below is presented as an interpretation and may be subject to change.

Cross-section NS1, which illustrates BGC's interpretation of the geological units at the site, is provided as Drawing 03; the location of the cross-section is shown on Drawing 02. The geology of the site is described from lowermost to uppermost units encountered in the following subsections.

2.2.1 Bedrock Geology

Fraser Bend Formation

This is a gravel-rich unit of both clast and matrix supported gravels. The matrix is predominantly sandy and silty, uncemented, and with zones of oxidation. The Fraser Bend Formation is described in detail in BGC (March 20, 2024).

At TH23-36 (drillhole log in BGC, March 22, 2024), the Fraser Bend Formation contact is interpreted to be at 57.5 metres below ground surface (mbgs) corresponding to elevation 694.3 m. The soils in this unit were gravelly silty clays to silty clays with some sand and gravel. This unit was uncemented to moderately cemented with zones of oxidation present.

2.2.2 Surficial Geology

The interpreted surficial geological units have been correlated to unit descriptions presented by Clague (1988).

Lower Glaciolacustrine and Diamicton; Clague (1988) Unit 10

Clague (1988) describes Unit 10 as comprising interbedded glaciolacustrine and diamicton sediments. Beds of Diamicton are theorized to have been deposited by subaqueous debris flows. This unit is termed the Lower Glaciolacustrine (Lower GLU) and Diamicton in this report.

This unit was encountered in drillhole TH23-36 between 15.0 and 57.5 mbgs (elevation 736.8 to 694.3 m). Diamicton subunits were encountered between 15.0 and 33.5 mbgs and 39.2 and 42.5 mbgs, while glaciolacustrine subunits were encountered between 33.5 and 39.2 mbgs and 42.5 and 57.5 mbgs. The diamicton soils were heterogeneous sandy and gravelly clays and silts to clays with trace sand and gravel. Although not encountered in TH23-36, cobbles and boulder sized particles may be present in the diamicton. The glaciolacustrine soils were homogeneous high plastic clays with some silt and sand to low plastic silty clay with some sand and trace fine to coarse grained gravel.

Upper Glaciolacustrine; Clague (1988) Unit 12

The uppermost glaciolacustrine sediments encountered at the North Slide have been aligned with Unit 12 from Clague (1988). This unit comprises predominantly homogeneous clays and silts with interbedded sands and gravels. This unit is termed the Upper Glaciolacustrine (Upper GLU) in this report.

This unit was encountered in drillhole TH23-36 between 3.6 and 15.0 mbgs (elevation 748.2 to 736.8 m). The upper portion of this unit was mainly a sandy or gravelly silty clay with low plasticity. This unit was interpreted as homogeneous high plastic clay below 12.8 mbgs (elevation 739.0 m).

The CPT data collected along the alignment of the proposed embankment widening (Cross-Section NS2 location shown on Drawing 02) was used to assess the thickness of the Upper GLU. The CPT corrected cone tip resistance (q_t) profiles, collected at five locations along Cross-Section NS2, are shown on Drawing 04. The geological interpretations shown on Drawing 04 are based on the CPT q_t profiles and surficial soil units logged at TH23-36. Upper GLU is interpreted to consist of a weathered crust, with a thickness of approximately 3.0 m to 4.0 m, where q_t values ranged between 1 and 5 MPa, underlain by relatively homogeneous glaciolacustrine clay layer, with q_t values increasing with depth from approximately 0.5 to 2.0 MPa.

The contact of the underlying Lower GLU and Diamicton is interpreted by the abrupt increase in q_t (greater than 5 MPa) and eventual CPT refusal. The thickness of the Upper GLU is shown to vary spatially along Cross-Section NS2 (Drawing 04), being thickest at CPT-BGC24-11 (18.5 m), and thinnest at CPT-BGC24-08 (8.5 m).

Colluvium

This unit comprises thin surficial landslide deposits which have been deposited by flow-style movement or slope wash processes. In drillhole TH23-36, this unit is soft to firm silty clay to silty sand, with trace gravel encountered between 0 and 3.6 mbgs.

Engineered Fill

This unit comprises the anthropogenic materials placed to construct the highway, including the asphalt pavement, compacted granular road base, and the embankment fill.

2.3 Groundwater Conditions

Piezometric monitoring data at the site includes the data from three VWPs and one SP. There is no long-term piezometric monitoring data at the site, as the instrumentation was installed between December 2023 and March 2024. A summary of the piezometers installed at the site, including the most recently recorded piezometric levels, are provided in BGC's Geotechnical Data Reports (BGC, March 22, 2024; BGC, April 18, 2024b). The piezometric levels obtained from instruments along Cross-Section NS1 are provided on the Drawing 03 cross-section.

Along the proposed embankment widening alignment, the relatively shallow SP and VWP installed in TH24-38B and TH24-38C, respectively, indicate the groundwater level to be near the ground surface in March 2024 (elevation 759.6 to 758.5 m). The two deeper VWPs installed at TH23-36, with tip elevations of 736.8 and 716.3 m, measured piezometric elevations of 743.9 and 717.4 m, respectively, in February 2024.

The piezometric data indicates that the groundwater flow regime is characterized by downward flow, i.e., that the piezometric level declines with declining elevation. Although not detected, perched or confined groundwater conditions may be present in the more permeable strata.

2.4 Ground Movement Monitoring

2.4.1 Slope Inclinometer Data

The slope has been instrumented with one SI installed in TH23-36. The slope movement monitoring data from this instrument is presented in Appendix B of BGC's Instrumentation Reading's Summary Report (BGC, April 18, 2024c). It is noted that this SI was installed and baselined in December 2023 and only one subsequent reading was taken in February 2024. The SI data collected to date does not show any movement that can be attributed to landslide movement. It is noted that SI casing is relatively new and continued monitoring of the SI installed in TH23-36 is recommended.

2.4.2 Lidar change detection

BGC used Airborne Lidar Scanning (ALS) data collected in 2006 and 2022 to complete change detection analysis. ALS change detection is performed by computing the topographical difference between two 3-dimensional (3D) models of a given site collected at different points in time. Analysis of topographical change between ALS datasets involves aligning datasets and determining the limit of detectable change ($LoD_{95\%}$) where 95% of the cumulative alignment error distribution is considered noise or instrument error. The change detection results are presented as a colour-contoured image illustrating the 3D shortest distance measurements of differences greater than the $LoD_{95\%}$ between the two datasets. Noise and/or errors may be present in the results where there are significant gaps or differences in point resolution between the two ALS datasets.

Results from the change detection are presented in Appendix C. The change detection results are also available via Cambio™ (https://cambio.bgcengineering.ca/MoTi).

The ALS change detection between 2006 and 2022 show no significant changes in the topography within the landslide mass, or proximate to the identified headscarp, which would indicate active ground movement related to the identified landslide hazard. The zones of change (primarily negative) along the slope are likely erroneous, and due to differences in the data quality between the two ALS data sets. These zones of change within the landslide mass do not correlate with the interpreted landslide mechanism, and are of comparable magnitude to zones of change observed in the flat-lying and stable uplands, notably to the east of the railway. An area of local topographic change is identified at the southern edge of the North Slide extents, at the cut slope of an access path (Figure C1).

2.5 Seismicity

Refer to BGC's Phase 2 Design Report (BGC, March 20, 2024) for a summary of the site's seismicity and the details relating to the provided seismic design parameters.

The pseudo-static analyses conducted for this assessment used a horizontal seismic coefficient of 0.013, which is equivalent to the half the PGA value associated with the 1:475 year return period event.

2.6 Interpretation of Landslide Mechanism

The landslide morphology of the site together with the ground movement monitoring data to date indicate there to be three credible landslide mechanisms (shown schematically on Drawing 03):

1. Local rotational or translational landslide. A relatively shallow failure of the local embankment slope (Slip Surface 1).
2. Translational landslide with a sliding layer at approximately elevation 744 m (Slip Surface 2). Translational or planar landslides are characterized by movement of a block of material on a plane which is typically formed by a pre-sheared weak layer (Hunger et al., 2014). Through interpretation of observations, this mechanism is constrained by the apparent mid-slope toe bulge (shown on Drawings 02 and 03) and a potential shear zone passing through a high plastic clay zone within the Upper GLU at elevation 744 m.
3. Deep-seated translational landslide with a sliding layer at approximately elevation 735 m (Slip Surface 3) within the Upper GLU. Evidenced by the broad, arcuate-shaped

headscarp and stepped and hummocky topography along the upper slope below the highway. Based on the landslide morphology, the sliding plane for this mechanism is interpreted to exit the slope face at the break in slope at approximately elevation 735 m.

Based on the SI slope movement monitoring data collected to date, and the ALS change detection analysis conducted, the landslide mechanisms associated with Slip-Surfaces 2 and 3 are interpreted to be dormant or moving very slowly, beyond the detectable limits of the ground movement monitoring methods analyzed.

The base (toe) of the broad, deep-seated landslide (Mechanism 3) is interpreted to be controlled by a change in stratigraphy from an upper fine-grained dominant, landslide susceptible soil (Upper GLU), to a more competent stratum below (Lower GLU and Diamicton). The change in stratigraphy is evidenced by the slope break observed mid-slope (approximate elevation 735 m, Drawings 02 and 03), where the upper slope changes from an approximately 10° slope to a steeper approximately 20° slope.

The lateral extents of the broad, deep-seated landslide are interpreted to be controlled by the extents of an infilled paleo valley which intersects the Cottonwood River Valley at the location of the North Slide. The infilled paleo valley is evidenced by the CPT q_t profiles along Cross-Section NS2 (Drawing 04). It is shown that the interpreted Lower GLU and Diamicton contact (base of the paleo valley) has a valley-shaped profile along the cross-section and that the thickest Upper GLU sediments (paleo valley infill material) occur towards the center of the landslide mass. The infilled paleo valley interpretation is further evidenced by the valley slope angles inside versus outside the slide mass. Within the slide mass, the upper slope (the slope above the slope break) is consistent and relatively flat (10°) compared to that of the upper slope outside the slide mass, where it is significantly steeper (a 30° to 40° slope). The steeper slopes, outside of the landslide extents, are interpreted to consist of glacial till material, while within the landslide extents, the glacial till is interpreted to have been cut down through the formation of a paleo valley. During subsequent glaciation and glacial retreat, the paleo valley was infilled with glaciolacustrine sediments, within which the landslide is occurring.

Within the North Slide extents, the lower slope appears to be draped with colluvium. Local instability is observed within this stratum, evidenced by the arcuate scarp visible in the lidar imagery (Drawing 02). To the north of the North Slide extents, the slope break is more pronounced, as the lower slope is significantly steeper (approximately 35°); at this location it is inferred that less colluvium is present.

3.0 SLOPE STABILITY ANALYSIS

3.1 Methodology

Two-dimensional limit equilibrium slope stability analyses were carried out with the computer program Slope/W (Version 23.1.2.11) using the GLE Morgenstern-Price method of slices (Morgenstern and Price, 1965). The stability analysis factor of safety (FoS) values were computed using the Slope/W slip surface optimization feature. The shear strength and density parameters used in the stability analyses were selected based on calibration back-analysis completed for the proximate Cottonwood River Slide (BGC, March 20, 2024), North slide site investigation data, empirical correlations and laboratory test results.

The assessment of the impact of the proposed Phase 2 embankment widening on the stability of the North Slide involved establishing a stability reference point (i.e., baseline) for subsequent forward-looking analyses. The forward-looking analyses considered the following three stages:

1. Stage 1: Immediately after the placement of the proposed embankment and surcharge fill and before the dissipation of embankment placement induced pore pressures (undrained strength analysis). This analysis assumes rapid fill placement with 0% consolidation of the foundation soil below the new fill.
2. Stage 2: After the dissipation of pore pressures induced by embankment placement, but before the removal of the surcharge fill (effective stress analysis). This analysis assumes 100% consolidation is achieved during the surcharge period.
3. Stage 3: After the dissipation of pore pressures induced by embankment placement and after the removal of the surcharge fill, ditch construction, and highway realignment (effective stress analysis). This analysis assumes 100% consolidation is achieved during the surcharge period.

The forward-looking analyses were completed using the proposed Phase 2 embankment cross-section provided as Figure A1 in Appendix A; the baseline analyses considered the existing embankment and slope geometry.

The assessment considered the three landslide mechanisms identified in Section 2.6, which were analyzed along Cross-Section NS1 (Drawing 03). The location of Cross-Section NS1 was chosen such that it coincided with the location where the headscarp is nearest to the highway, the area of maximum fill heights, and close to the center of the slide mass, where the Upper GLU paleo valley infill sediments are thickest. The orientation of Cross-Section NS1 was chosen such that it follows the fall line of the slope. The relative change between the FoS values from the baseline analyses and the forward-looking analyses were compared to assess the impact of the proposed Phase 2 embankment widening on North Slide stability.

The impact of seismic loading was considered by applying a horizontal seismic loading coefficient to both the baseline and forward-looking analyses and computing the relative change in FoS values between them. A horizontal seismic coefficient of 0.013 was used in this assessment, equal to one half the PGA value associated with the 1:475 year return period event (Section 2.5).

3.2 Model Setup

3.2.1 Slip Surface Geometry

Potential slip surfaces were defined using Slope/W's Entry-Exit search function. The software produced various potential slip surfaces based on the entry and exit ranges specified.

A summary of the slip surfaces considered as part of this study is provided in Table 3-1. Slip Surfaces 1 through 3 are intended to correspond to the three identified landslide mechanisms outlined in Section 2.6. For the translational landslide mechanisms (Slip Surfaces 2 and 3), a basal sliding layer was incorporated into the models to represent a weak, pre-sheared layer within the GLU. To force the slip surface search to follow the geometry of the basal sliding layer, the materials below the sliding layer were modelled as impenetrable. The baseline and forward-looking analyses completed as part of this assessment consider the critical slip surface identified using the Entry-Exit search function (i.e., lowest calculated FoS).

Table 3-1 Slip surface geometries analyzed.

Slip Surface ID	Landslide Mechanism	Search Method
1	Local rotational or translational(1)	Entry-Exit
2	Translational, sliding layer at approximately elevation 744 m	Entry-Exit
3	Translational, sliding layer at approximately elevation 735 m	Entry-Exit

Note:
1. A minimum slip surface depth of 3 m was assigned for this slip surface search to eliminate very shallow slip surfaces associated with surficial sloughing.

3.2.2 Geotechnical Units

The geotechnical units in the Slope/W model were generally based on the geological units described in Section 2.2. An overview of the geotechnical units and material parameters used in the Slope/W model is provided below:

<u>Embankment and Surcharge Fill:</u> The existing embankment fill, proposed embankment fill (for embankment widening), and the proposed surcharge fill are all assumed to consist of suitable Type D material, per MoTI Standard Specification 201 (MoTI, November 1, 2020). The shear strength parameters assigned to these units are based on engineering judgement considering typical highway embankment construction practices. A lower effective friction angle (34°) was assigned to the existing fill, while a higher friction angle of 36° was assigned to the proposed fills, to account for uncertainty in composition and construction methods of the existing embankment.

<u>Glaciolacustrine Unit (GLU)</u>: A unit dominated by silt- and clay-sized particles, containing variable amounts of sand, and trace amounts of gravel, with plasticity ranging from low to high. The GLU was divided into two subunits: the Upper GLU (the GLU of Clague (1988) Unit 12) and

the Lower GLU, which is interbedded at greater depth with the Diamicton (the GLU of Clague (1988) Unit 10). For the undrained strength analysis (Stage 3), the Upper GLU was subdivided into three layers comprising: the weathered crust, the upper softest portion of the unweathered clay, and the lower stiffer, unweathered clay at depth.

The effective stress parameters assigned to this unit were carried forward from BGC's previously completed stability analysis for the Cottonwood River Slide (BGC, March 20, 2024) and were based on back-analysis calibration and the results of laboratory triaxial strength testing. The undrained strength parameters assigned to the Upper GLU are based on in-situ CPT and shear vane data (BGC, April 18, 2024b) and are consistent with those used in BGC's Phase 2 Updated Embankment Widening Stability Assessment (BGC, April 18, 2024d).

Glaciolacustrine Sliding Layer: A thin, pre-sheared layer within the Upper GLU (the GLU of Clague (1988) Unit 12) was included in the model as a zone with depth matching the interpreted sliding depths associated with the translation failure modes (Slip Surfaces 2 and 3, Section 3.2.1). The shear strength parameters assigned to this unit were carried forward from BGC's previously completed stability analysis for the Cottonwood River Slide (BGC, March 20, 2024) and are based on back-analysis calibration, the results of laboratory direct shear and ring shear strength testing, and empirical correlations.

Diamicton: This unit was generally composed of a mixture of coarse-grained soil (sands, gravels) and fine-grained soil (silts and clays with trace sand and gravel). The unit was subdivided into units termed the Upper Diamicton and Lower Diamicton in order to model lower piezometric levels in the Lower Diamicton subunit. The shear strength parameters assigned to this unit were carried forward from BGC's previously completed stability analysis for the Cottonwood River Slide (BGC, March 20, 2024) and are based on the soil unit termed 'Undifferentiated Soil'.

Fraser Bend Formation: The Fraser Bend Formation is generally gravel-rich within a matrix of sand, silt, or clay. Occasional layers of clay are present within the formation, as well as possible cobbles and boulders. The gravels are cemented in some areas. This is the lowermost unit considered in the models. It was assigned an impenetrable strength model to force all slip surfaces to pass above this unit.

3.2.3 Material Parameters

3.2.3.1 Effective Stress Analysis

For analyses considering effective stress strength (Stages 2 and 3), Mohr-Coulomb shear strength parameters were assigned to the geotechnical units. The unit weight and shear strength parameters selected for use for long-term conditions (i.e., after the dissipation of excess pore pressures) are summarized in Table 3-2.

Table 3-2 Mohr-Coulomb strength parameters used in the Slope/W models.

Geotechnical Unit	Unit ID	Modelled Colour	Bulk Unit Weight (kN/m^3)	Shear Strength Model	Shear Strength Parameters
Existing Embankment Fill	Fill-1		20	Mohr-Coulomb	ϕ’ = 34° c’ = 0 kPa
Proposed Embankment Fill	Fill-2		20	Mohr-Coulomb	ϕ’ = 36° c’ = 0 kPa
Proposed Surcharge Fill	Fill-3		20	Mohr-Coulomb	ϕ’ = 36° c’ = 0 kPa
Upper GLU (Unit 12 from Clague, 1988)	GLU-U		18	Mohr-Coulomb	ϕ’ = 30° c’ = 0 kPa
Lower GLU (Unit 10 from Clague, 1988)	GLU-L		20		
GLU Sliding Layer (Unit 12 from Clague, 1988)	GLU-SL		20	Mohr-Coulomb	ϕ_r’ = 14° c’ = 0 kPa
Upper Diamicton (Unit 10 from Clague, 1988)	DMT-U		20	Mohr-Coulomb	ϕ’ = 35° c’ = 5 kPa
Lower Diamicton (Unit 10 from Clague, 1988)	DMT-L				
Fraser Bend Formation	FRB		N/A	Impenetrable	N/A

Notes:

1. ϕ’, - effective friction angle
2. ϕr’ - effective, residual friction angle
3. c’ - effective cohesion

3.2.3.2 Undrained Strength Analysis

For the Stage 1 analyses, undrained shear strength parameters were assigned to the Upper GLU soils. The Upper GLU was subdivided into three units comprising: the weathered crust (CRUST), the softest portion of the unweathered clay (GLU-U1), and the stiffer, unweathered clay at depth (GLU-U2). The unit weight and shear strength parameters assigned to each subunit are summarized in Table 3-3. The Upper GLU subunits and undrained strength parameters presented below are based on in-situ testing and are consistent with those provided in BGC’s Phase 2 Updated Embankment Widening Stability Assessment (BGC, April 18, 2024d). For details regarding the subunits and their assigned shear strength parameters, refer to BGC (April 18, 2024d).

Table 3-3 Undrained strength parameters assigned to the Upper GLU in the Slope/W models.

Geotechnical Unit	Subunit ID	Modelled Colour	Bulk Unit Weight (kN/m^3)	Shear Strength Model	Peak undrained shear strength (kPa)
Upper GLU	CRUST		18	Undrained	40
	GLU-U1		18	Undrained	30
	GLU-U2		18	Undrained	35 + 5z, where z is depth below top GLU-U2 in metres.

3.2.4 Groundwater Conditions

To approximate the groundwater flow regime described in Section 2.3, three piezometric lines were utilized in the Slope/W models. The configuration of the piezometric lines were based on the available VWP and SP data together with assumptions based on engineering judgement. As noted in Section 2.3, the piezometric monitoring data is limited and the monitoring period of the piezometers installed is relatively short (one to three months); as such, the piezometric levels used to develop the piezometric lines in the Slope/W model may not necessarily represent peak porewater pressures that may occur during significant precipitation events or wetting trends.

The piezometric lines utilized in the Slope/W models are shown on the slope stability outputs in Appendix D and are summarized below (from uppermost to lowermost):

<u>Piezometric Line 1:</u> Represents porewater pressures associated with the near surface piezometric levels measured in the Upper GLU at drillholes TH24-38B and TH24-38C. The near surface piezometric level measured at the location of the proposed embankment widening was carried further down the slope. Piezometric Line 1 was applied to the Upper GLU (GLU-U), the GLU Sliding Layer (GLU-SL) associated with Slip Surface 2, and the Embankment and Surcharge Fill.

<u>Piezometric Line 2:</u> Represents porewater pressures measured by the VWP installed in TH23-36 (VWP tip A) near the base of the Upper GLU. Piezometric Line 2 was applied to the GLU Sliding Layer (GLU-SL) associated with Slip Surface 3 and the Upper Diamicton (DMT-1).

<u>Piezometric Line 3:</u> Represents porewater pressures measured by the VWP installed in TH23-36 (VWP tip B) within the Lower GLU. Piezometric Line 3 was applied to the Lower GLU (GLU-L) and Lower Diamicton (DMT-2).

3.3 Baseline Analyses

Slope stability analyses were carried out to establish a stability reference point (i.e., baseline) for the subsequent forward-looking analyses. The baseline FoS values are summarized in Table 3-4 and figures illustrating the results of the analyses are provided in Appendix D.

For Stage 1 (i.e., undrained strength analysis), a local rotational or translational failure within the Upper GLU was analyzed (Slip Surface 1).

Although the base of the landslide is interpreted to be above the Lower GLU and Diamicton contact surface (approx. El. 735 m, see Drawing 03), analyses were also conducted to assess the FoS values of deeper-seated rotational or translational slip surfaces which extend into the Lower GLU and Diamicton using Slope/W's Entry-Exit search function. Weaker sliding layers were also modelled within the Lower GLU clay zones at various elevations. The GLU sliding layers within the Lower GLU were modelled with material parameters consistent with the GLU Sliding Layer of the Upper GLU (Table 3-2). The slip surfaces which extended into the Lower GLU zones had relatively high FoS values compared to the FoS values of slip-surfaces contained to the Upper GLU. This result indicates that these deeper-seated slip surfaces likely do not represent a credible landslide mechanism; this finding is corroborative with the interpreted landslide mechanisms in Section 2.6.

Table 3-4 Calculated baseline FoS values for interpreted slip surfaces.

Analysis	Slip Surface ID	Landslide Mechanism	Baseline FoS[1,2]	Figure Number
Undrained Strength	1	Local rotational or translational	1.25	D1
Effective Stress	1	Local rotational or translational	1.42 (1.38)	D2
	2	Translational, sliding layer at approximately elevation 744 m	1.00 (0.94)	D3
	3	Translational, sliding layer at approximately elevation 735 m	1.15 (1.08)	D4

Note:

1. The critical slip surface (i.e., lowest calculated FoS) identified using the Entry-Exit search function.
2. The unbracketed values represent the static analyses, the bracketed values represent the pseudo-static seismic analyses.

3.4 Effect of Phase 2 Embankment Widening on the North Slide

The results of the forward-looking stability analyses are presented on Figures D5 to D8 and summarized in Table 3-5. Table 3-5 includes the baseline FoS values from Table 3-4 for comparison purposes.

Table 3-5 Calculated relative change values between baseline analyses and forward-looking analyses.

Analysis	Slip Surface ID	Landslide Mechanism	Baseline FoS(4,6)	Forward-Looking Analysis FoS(5,6)	Relative Change (%)(6)	Figure Number
Stage 1 – Undrained Strength(1)	1	Local rotational or translational	1.25	1.25	0%	D5
Stage 2– Effective Stress(2)	1	Local rotational or translational	1.42	1.42	0%	-
	2	Translational, sliding layer at approximately elevation 744 m	1.00	1.00	0%	-
	3	Translational, sliding layer at approximately elevation 735 m	1.15	1.15	0%	-
Stage 3 – Effective Stress(3)	1	Local rotational or translational	1.42 (1.38[6])	1.43 (1.38[6])	+1% (0%[6])	D6
	2	Translational, sliding layer at approximately elevation 744 m	1.00 (0.94[6])	1.03 (0.97[6])	+3% (+3%[6])	D7
	3	Translational, sliding layer at approximately elevation 735 m	1.15 (1.08[6])	1.17 (1.09[6])	+2% (+1%[6])	D8

Notes:
1. Stage 1: Immediately after the placement of the proposed embankment and surcharge fill.
2. Stage 2: After the dissipation of pore pressures but before the removal of the surcharge fill.
3. Stage 3: After the dissipation of pore pressures, after the removal of the surcharge fill, and after the ditch construction and highway realignment.
4. From Table 3-4.
5. The critical slip surface (i.e., lowest calculated FoS) identified using the Entry-Exit search function.
6. The bracketed values represent the pseudo-static seismic analyses.

The FoS values calculated through the forward-looking analyses are equal to, or greater then, the baseline FoS values, for both static and pseudo-static loading conditions (refer to Table 3-5). This result indicates that the proposed Phase 2 embankment widening and associated surcharge loading does not negatively impact the stability of the North Slide. It is noted that the FoS values calculated for the final embankment geometry (Stage 3) are relatively low, ranging from 1.03 to 1.43 for static loading conditions (and 0.97 to 1.38 for pseudo-static loading conditions). Although there are no design plans to improve these FoS values, removing the existing embankment fill material at the crest of the North Slide, after realigning the Highway No. 97 to the east, may improve the FoS values in Table 3-5 for Stage 3. The design of slope stabilization measures for the North Slide is not included in BGC's current scope of work; however, additional stability analyses could be completed by BGC under a separate scope of work to assess possible options for improving the stability of the North Slide.

4.0 RECOMMENDATIONS

Based on the assessment completed herein, the following recommendations are provided:

- All recommendations relating to the embankment construction provided in the Phase 2 design report (BGC, March 20, 2024) are still valid.
- During construction of the proposed embankment widening, data should be collected from the SI installed in TH23-36 at least monthly. After construction of Phase 2, it is recommended that the SI casing be monitored at least once per year.

By comparing the baseline FoS values against those calculated through forward-looking analyses for various scenarios, this assessment concludes that the proposed Phase 2 embankment widening and highway realignment work does not negatively impact the stability of the North Slide. The FoS values presented herein are estimated based on currently available information and may be refined depending on the results of future monitoring of the SI casing installed at TH23-36. The values are relatively low and consideration should be made as to whether this is acceptable or if additional steps to refine the analysis or improve the stability are warranted.

5.0 CLOSURE

We trust the above satisfies your requirements. Should you have any questions or comments, please do not hesitate to contact us.

Yours sincerely,

BGC Engineering Ltd.
per:

Johnathan Cholewa, Ph.D., P.Eng.
Principal Geotechnical Engineer

Reviewed by:

Rod Kostaschuk, M.Eng., P.Eng.
Principal Geotechnical Engineer

MD/RK/sa/mm

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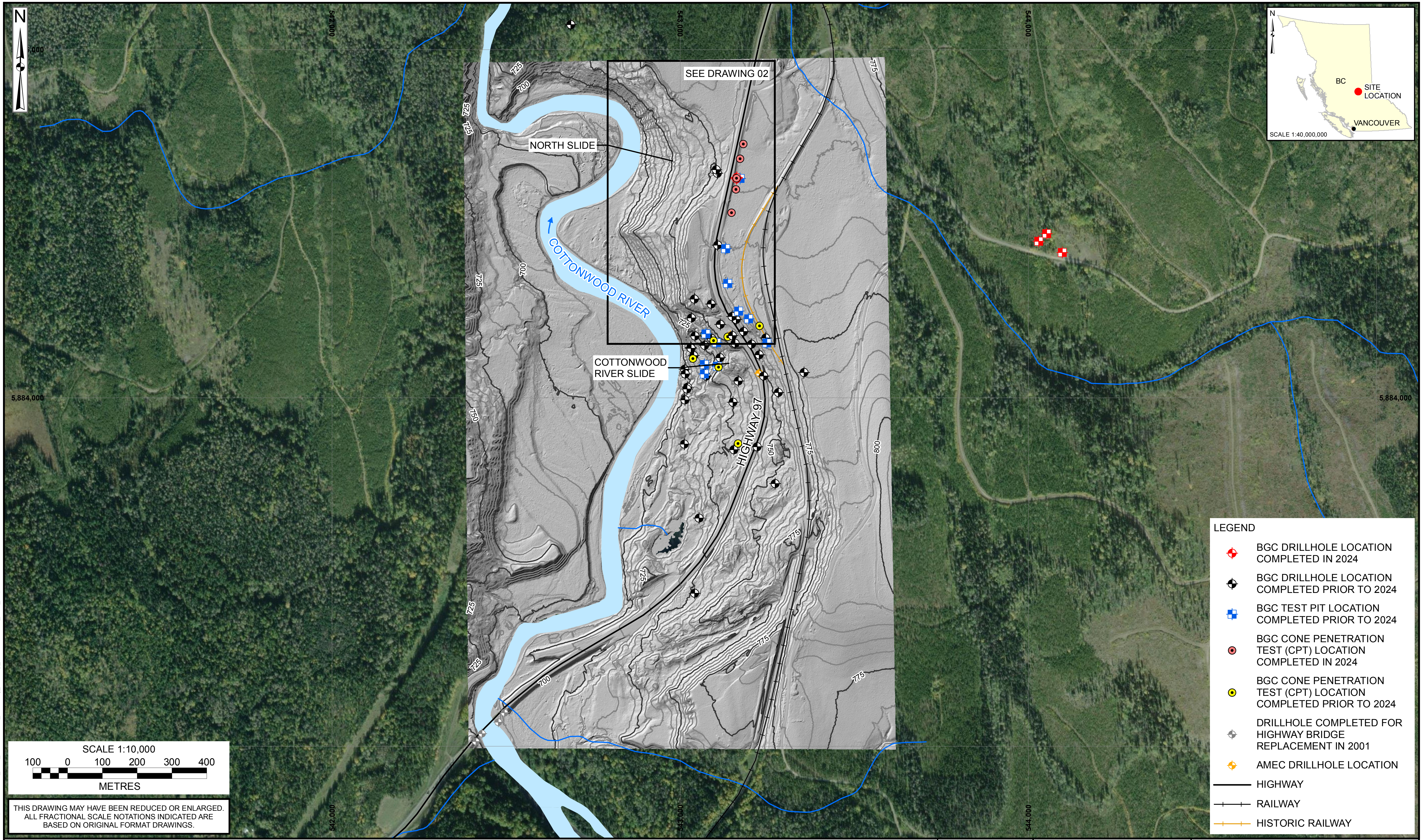
DRAWINGS



NOTES:
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3. BASE TOPOGRAPHIC DATA BASED ON LIDAR PROVIDED BY MOTI, DATED OCTOBER 2023 AND AUGUST 2021. CONTOUR INTERVAL IS 5 m.
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APPROVED:	JC

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PROJECT: COTTONWOOD RIVER SLIDE NORTH SLIDE STABILITY ASSESSMENT

TITLE: PLAN MAP

PROJECT No.: 1262102

DWG No: 01

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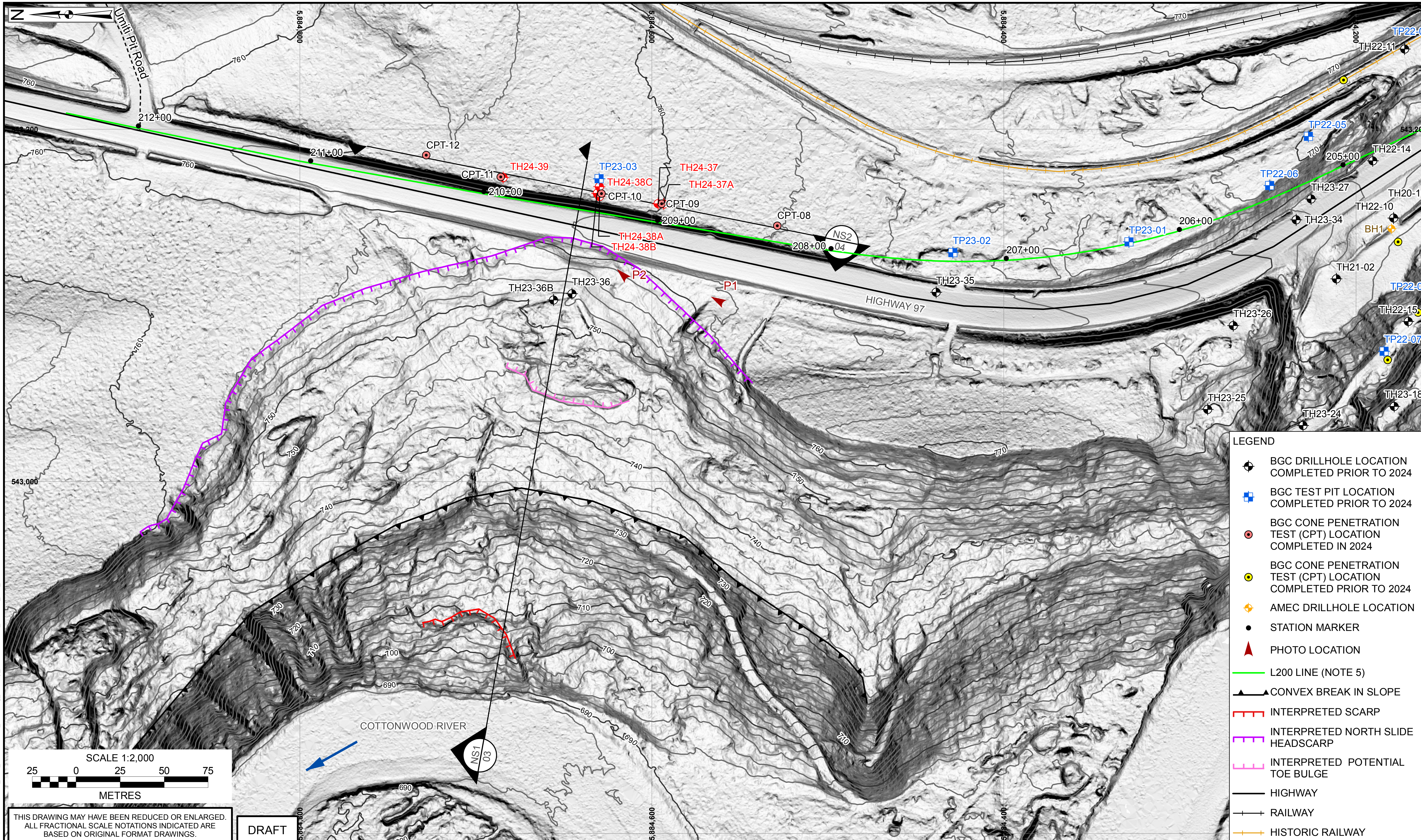

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4. DRILLHOLE AND CPT COORDINATES ARE BASED ON SURVEY DATA RECEIVED FROM BINNIE, DATED MAY 7, 2024.
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PROJECT: COTTONWOOD RIVER SLIDE NORTH SLIDE STABILITY ASSESSMENT

TITLE: SITE OVERVIEW

PROJECT No.: 1262102

DWG No: 02

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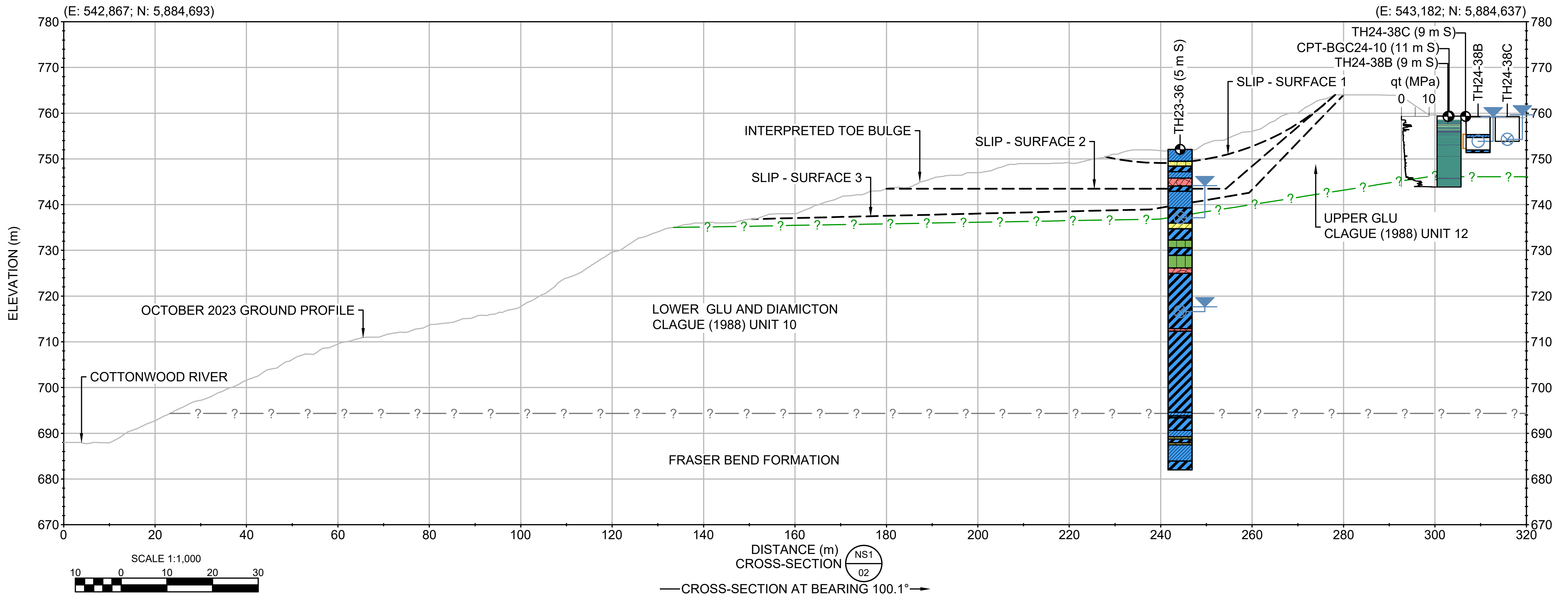

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LEGEND

- BGC DRILLHOLE LOCATION
- BGC TEST PIT LOCATION
- BGC CONE PENETRATION TEST (CPT) LOCATION
- GROUTED IN VWP
- MANUAL READING WITHIN STANDPIPE PIEZOMETER
- PIEZOMETRIC LEVEL
- STANDPIPE SCREEN INTERVAL
- CPT CORRECTED TIP RESISTANCE, q_t
- UPPER CONTACT OF CLAGUE (1988) UNIT 10 - INTERPRETED
- UPPER CONTACT OF FRASER BEND FORMATION - INTERPRETED
- INTERPRETED SLIP SURFACE

INTERPRETED LITHOLOGY LOGGED IN DRILLHOLES

- HIGH PLASTICITY CLAY
- LOW PLASTICITY CLAY, SILTY CLAY
- SILT, CLAYEY SILT
- SANDY CLAY
- CLAYEY SAND
- CLAYEY GRAVEL
- COBBLES
- PEAT/TOPSOIL

CPT SOIL BEHAVIOUR TYPE (NOTE 5)

- SENSITIVE, FINE GRAINED
- ORGANIC SOILS
- CLAYS
- SILT MIXTURES
- SAND MIXTURES
- SANDS
- GRAVELLY SAND TO SAND
- STIFF SAND TO CLAYEY SAND
- VERY STIFF FINE GRAINED
- UNDEFINED ZONE

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4. COORDINATE SYSTEM IS NAD83 UTM ZONE 10N. VERTICAL DATUM IS CGVD28.
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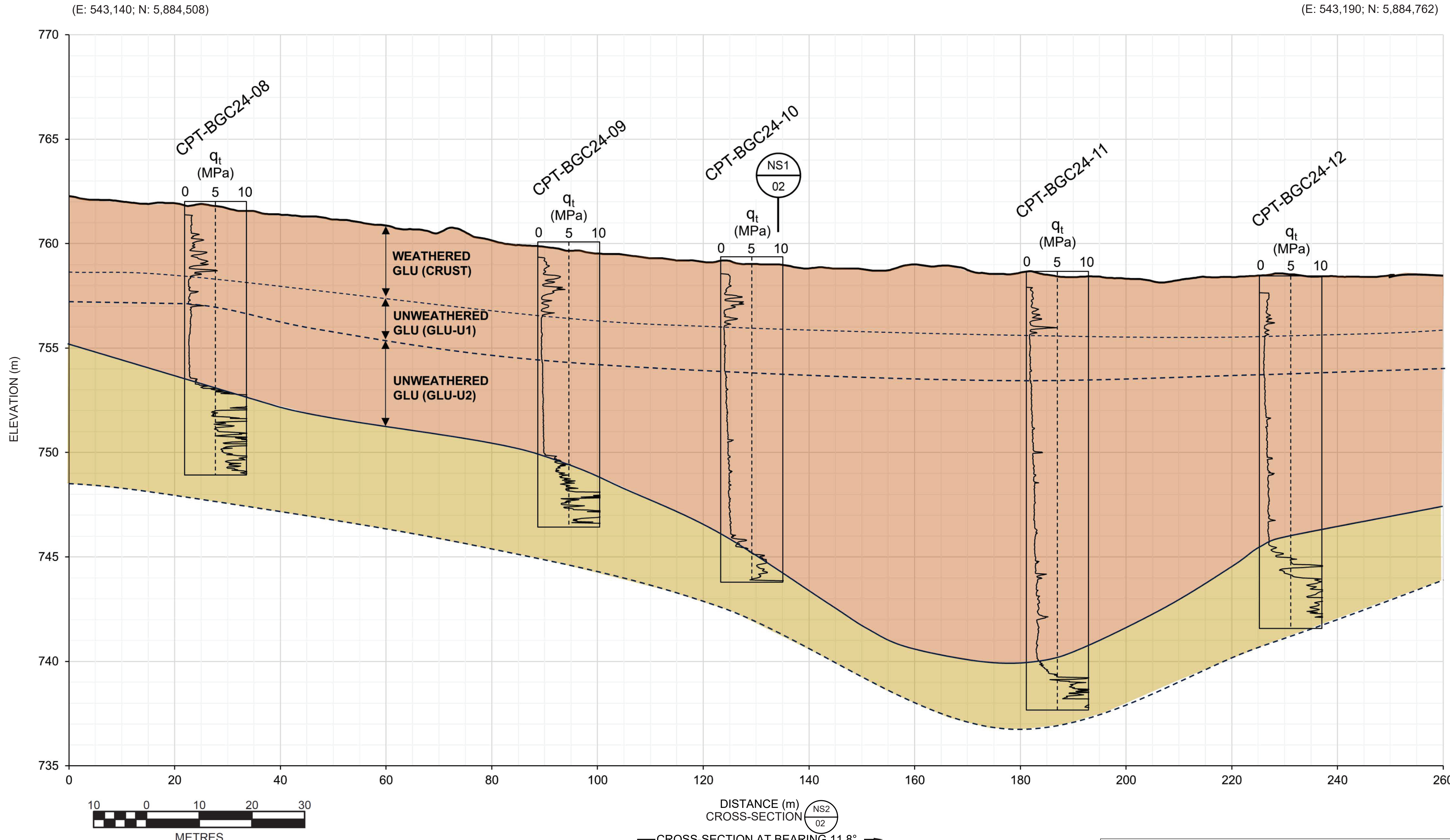
PROJECT: COTTONWOOD RIVER SLIDE NORTH SLIDE STABILITY ASSESSMENT

TITLE: CROSS-SECTION NS1

PROJECT No.: 1262102

DWG No.: 03



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LEGEND - GEOLOGICAL INTERPRETATION

- UPPER GLU, CLAGUE (1988) UNIT 12
- LOWER GLU AND DIAMICTON, CLAGUE (1988) UNIT 10

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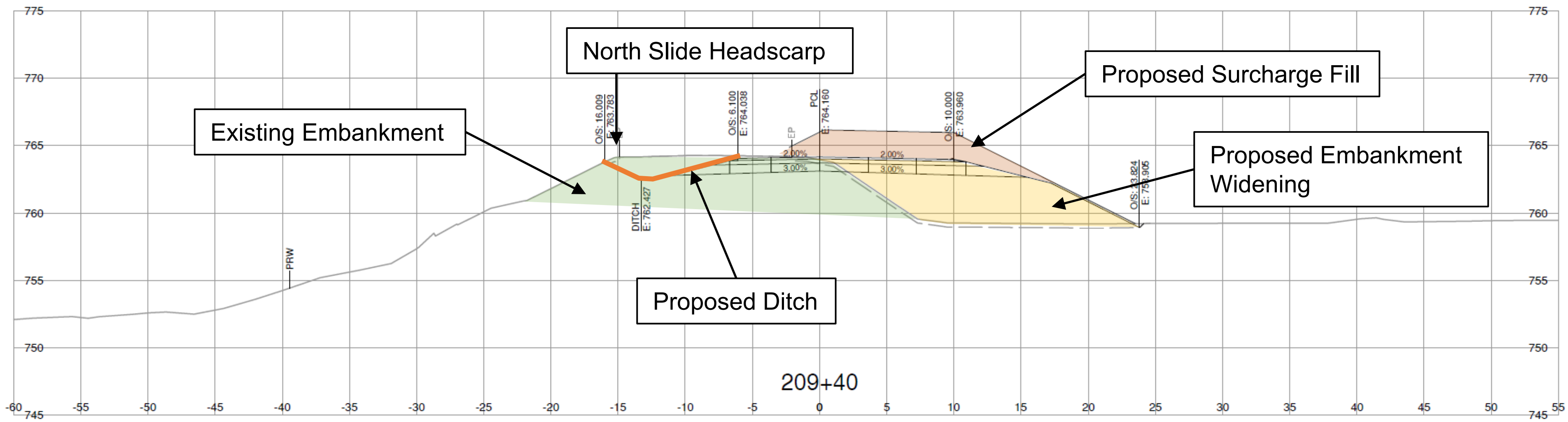
PROJECT: COTTONWOOD RIVER SLIDE NORTH SLIDE STABILITY ASSESSMENT

TITLE: CROSS-SECTION NS2 - CPT q_t PROFILES AND GEOLOGICAL INTERPRETATION

PROJECT No.: 1262102

DWG No.: 04

APPENDIX A
FIGURES



NOTES:

1. This Figure should be read in conjunction with BGC's report titled Cottonwood River Slide – North Slide Stability Assessment, and dated May 2024.
2. This annotated drawing is an excerpt from Binnie's Phase 2 Drawing Package (Binnie, April 4, 2024), Drawing number R2-1249-L200XS-63.

PREPARED BY:	FIGURE TITLE:		
SLB	ANNOTATED CROSS-SECTION OF THE PROPOSED HIGHWAY REALIGNMENT ADJACENT TO THE NORTH SLIDE		
CHECKED BY: RK	CLIENT: R.F. Binnie & Associates Ltd.		
APPROVED BY: JC	SCALE: N/A	PROJECT NO: 1262102	FIGURE NO: A1

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APPENDIX B
SITE PHOTOS

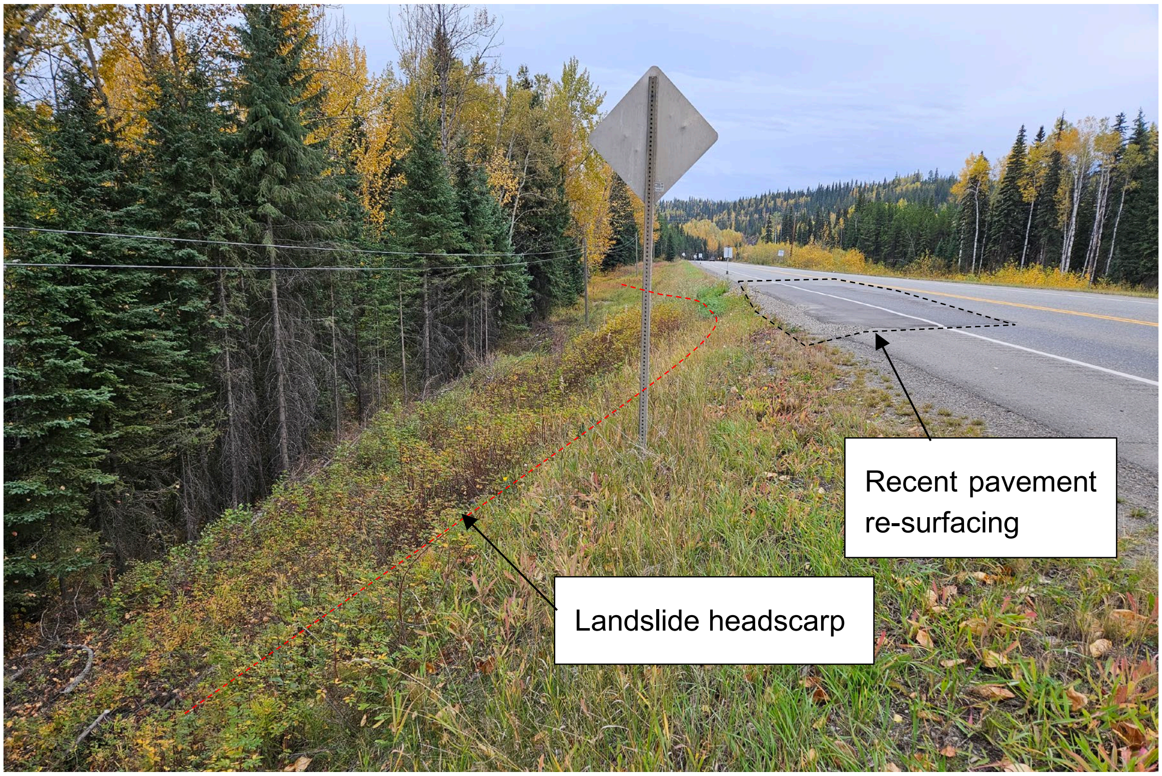
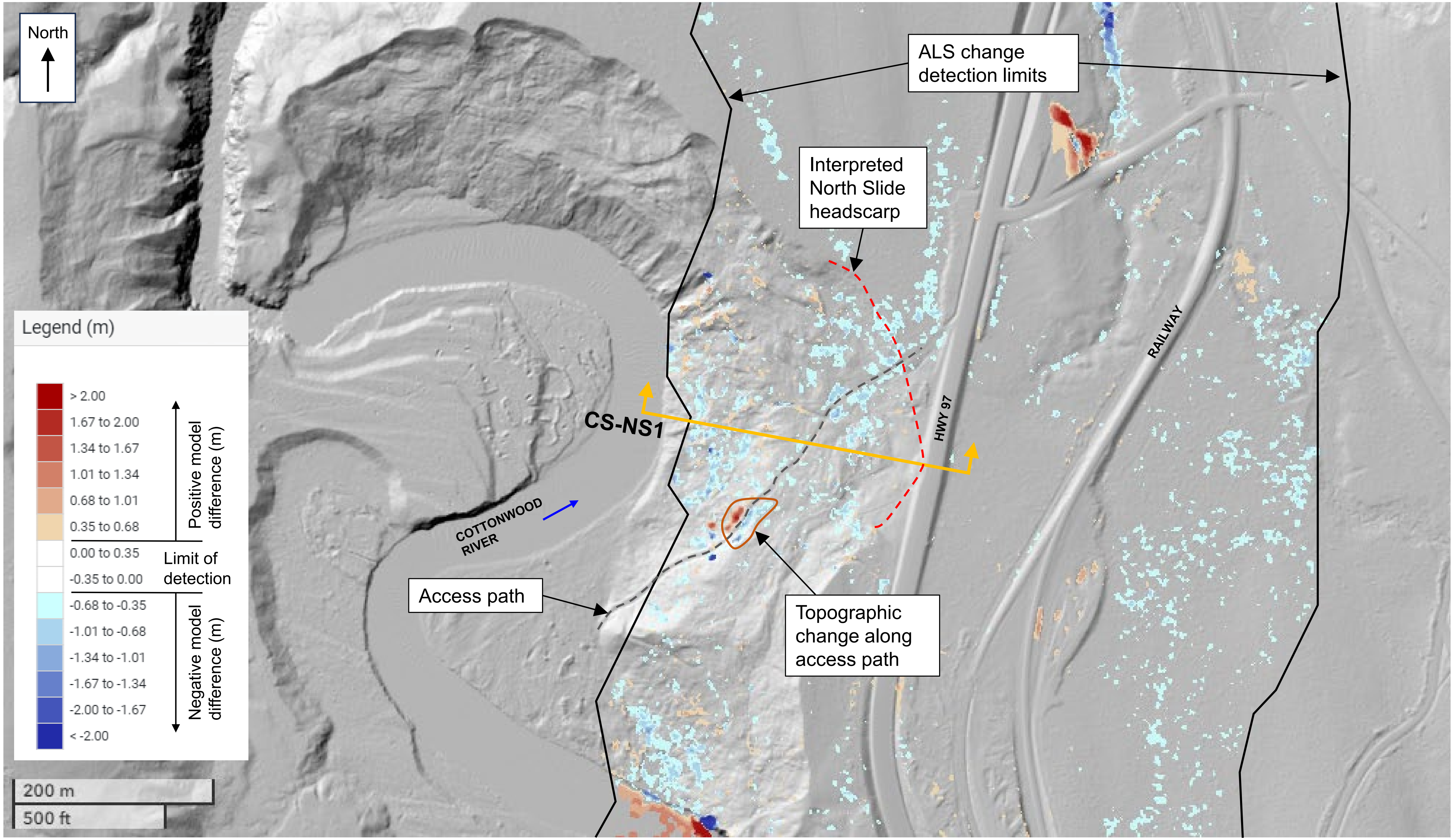


Photo 1 Landslide headscarp encroachment on highway, looking north, Sept. 25, 2023.

Photo 2 Cracks in pavement adjacent to the landslide headscarp, looking north, Sept. 25, 2023.

APPENDIX C
LIDAR CHANGE DETECTION ANALYSIS



NOTES:

1. This figure should be read in conjunction with BGC's report title Cottonwood River Slide – North Slide Stability Assessment, and dated May 2024.
2. This figure was generated from Cambio™ (https://cambio.bgcengineering.ca/MoTi).
3. ALS change detection was performed using bare-earth lidar data from 2006 and 2022 with a limit of detection of -0.35 m to +0.35 m.

PREPARED BY:	FIGURE TITLE:		
SLB	2006 VS. 2022 ALS CHANGE DETECTION RESULTS		
CHECKED BY:	CLIENT:		
RK	R.F. Binnie & Associates Ltd.		
APPROVED BY:	SCALE:	PROJECT NO:	FIGURE NO:
JC	AS SHOWN	1262102	C1

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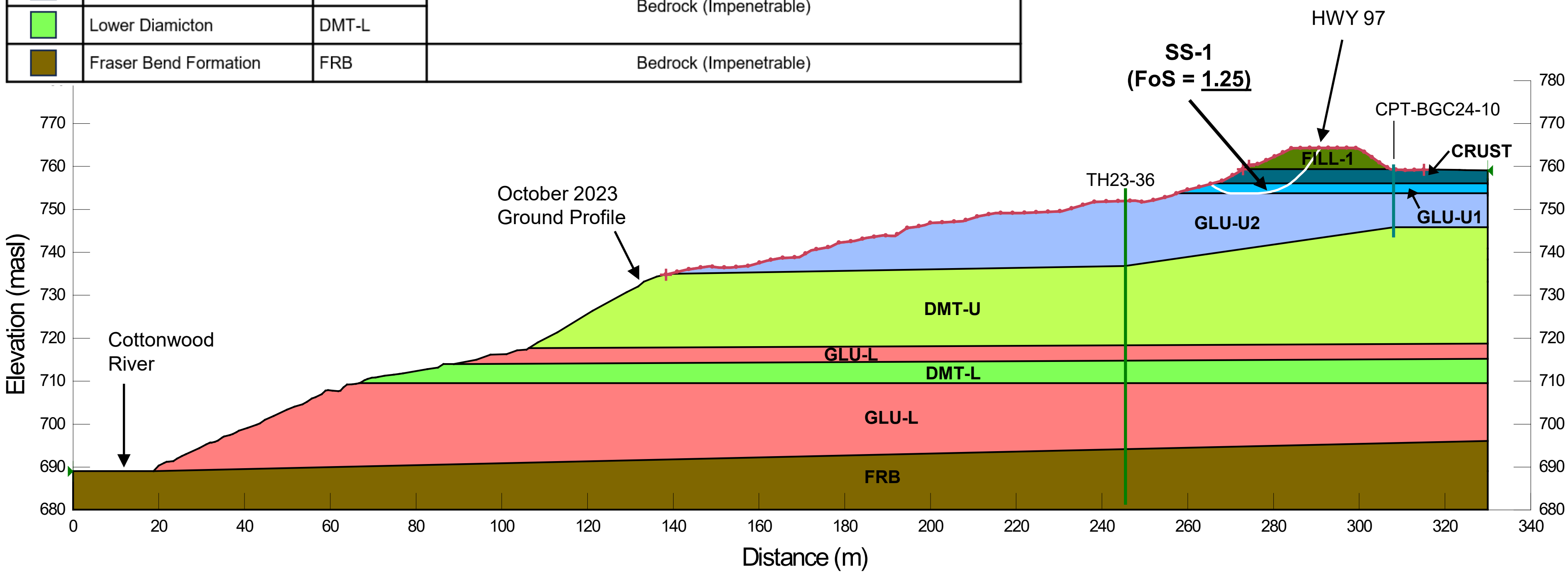
APPENDIX D
STABILITY ANALYSIS RESULTS

Colour	Material Type	Soil Unit Name	Unit Weight (kN/m^3)	Total Cohesion (kPa)	Total Cohesion Rate of Change (kPa/m)	Effective Cohesion (kPa)	Friction Angle
	Existing Embankment Fill	FILL-1	20	-	-	0	34
	Weathered Crest	CRUST	18	40	-	-	-
	Upper GLU	GLU-U1	18	30	-	-	-
		GLU-U2	18	35	5	-	-
	Lower GLU	GLU-L	Bedrock (Impenetrable)				
	Upper Diamicton	DMT-U	Bedrock (Impenetrable)				
	Lower Diamicton	DMT-L					
	Fraser Bend Formation	FRB	Bedrock (Impenetrable)				



File Location: https://ibgcengineering.sharepoint.com/:f:/s/1262-CottonwoodRiverSlide/Er60_U9wlThAnDK0WUiKILEB-N-M83D-lIJ1rzv0hlL4-A?e=6p7CB9
File Name: CS-NS1.gsz

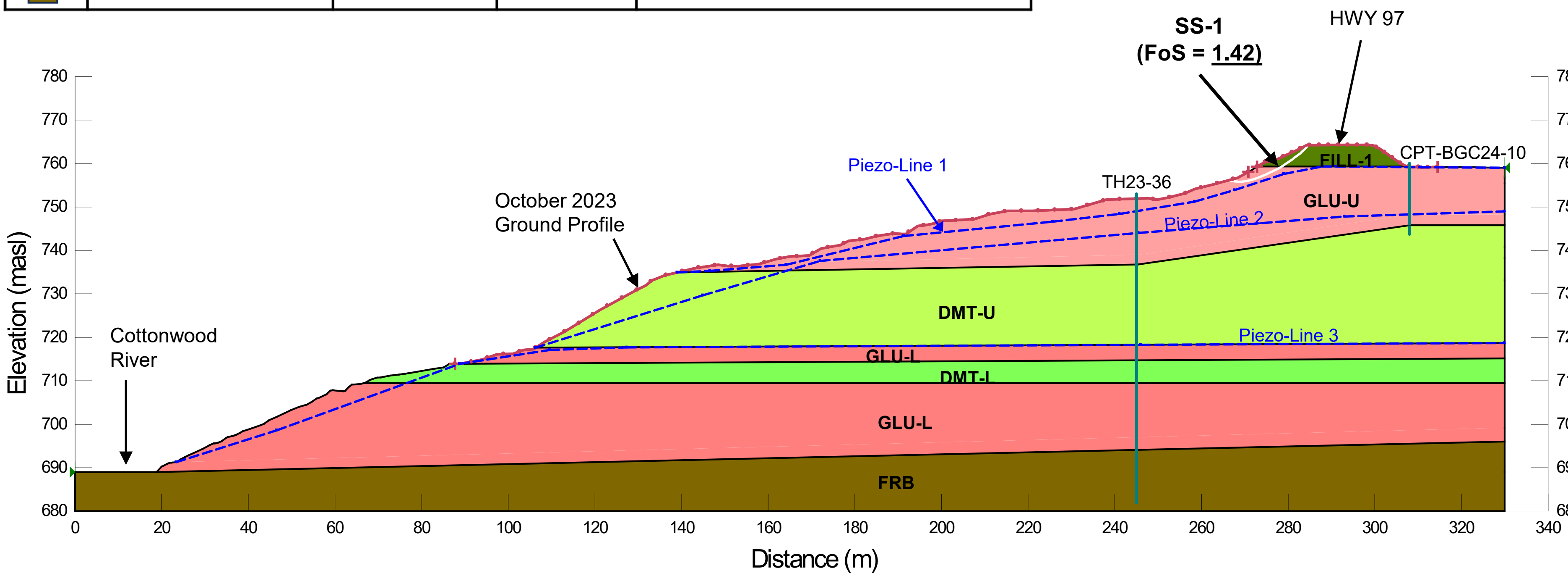
NOTES:

1. This Figure should be read in conjunction with BGC's report titled "Cottonwood River Slide – North Slide Stability Assessment", and dated May 2024.

PREPARED BY: SLB	FIGURE TITLE: Section NS1 Baseline Conditions Undrained Strength Analysis, Slip-Surface 1
CHECKED BY: RK	CLIENT: R.F. Binnie & Associates Ltd.
APPROVED BY: JC	SCALE: NTS · PROJECT NO: 1262102 · FIGURE NO: D1

0 5 10 mm in ANSI A sized paper

Colour	Material Type	Soil Unit Name	Piezometric Line	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Friction Angle
	Existing Embankment Fill	FILL-1	1	20	0	34
	Upper GLU	GLU-U	1	18	0	30
	Lower GLU	GLU-L	3	20	0	30
	Upper Diamicton	DMT-U	2	20	5	35
	Lower Diamicton	DMT-L	3			
	Fraser Bend Formation	FRB	None	Bedrock (Impenetrable)		



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File Name: CS-NS1.gsz

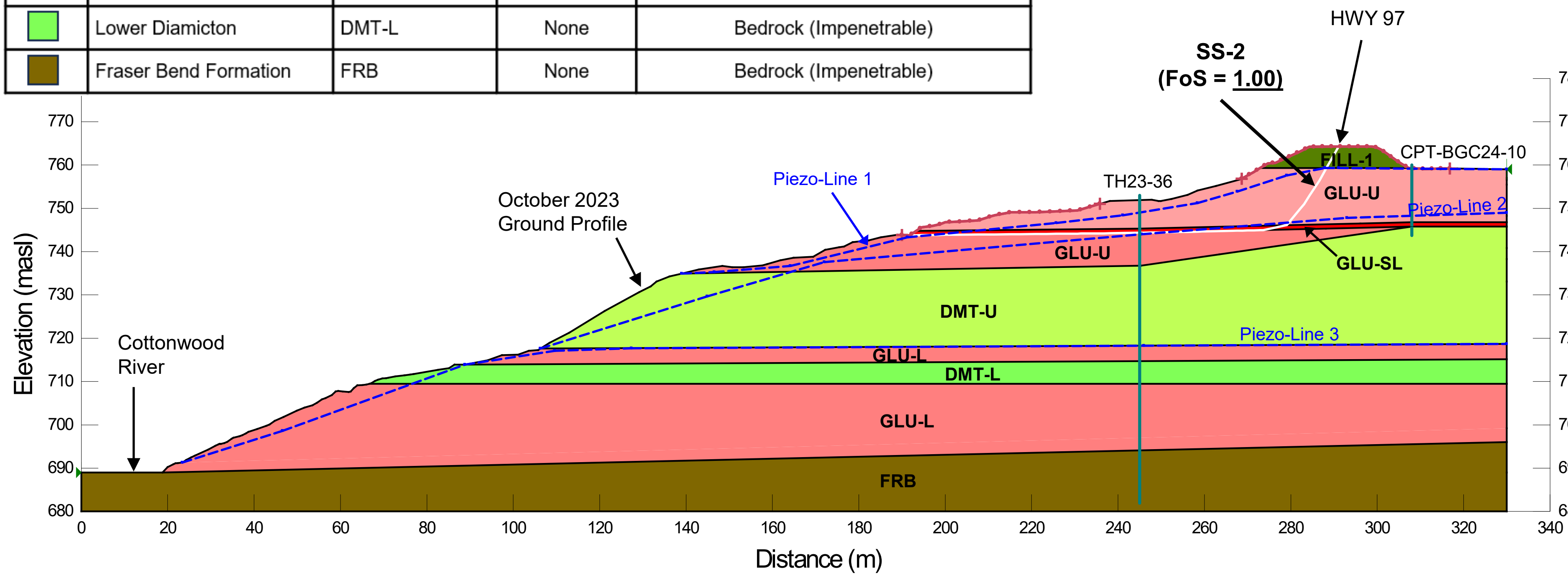
NOTES:

1. This Figure should be read in conjunction with BGC's report titled "Cottonwood River Slide – North Slide Stability Assessment", and dated May 2024.

PREPARED BY: SLB	FIGURE TITLE: Section NS1 Baseline Conditions Effective Stress Analysis, Slip-Surface 1		
CHECKED BY: RK	CLIENT: R.F. Binnie & Associates Ltd.		
APPROVED BY: JC	SCALE: NTS	PROJECT NO: 1262102	FIGURE NO: D2

0 5 10 mm in ANSI A sized paper

Colour	Material Type	Soil Unit Name	Piezometric Line	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Friction Angle
	Existing Embankment Fill	FILL-1	1	20	0	34
	Upper GLU	GLU-U	1	18	0	30
	GLU - Sliding Layer	GLU-SL	1	18	0	14
	Upper GLU	GLU-U	None	Bedrock (Impenetrable)		
	Lower GLU	GLU-L	None	Bedrock (Impenetrable)		
	Upper Diamicton	DMT-U	None	Bedrock (Impenetrable)		
	Lower Diamicton	DMT-L	None	Bedrock (Impenetrable)		
	Fraser Bend Formation	FRB	None	Bedrock (Impenetrable)		



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File Name: CS-NS1.gsz

NOTES:

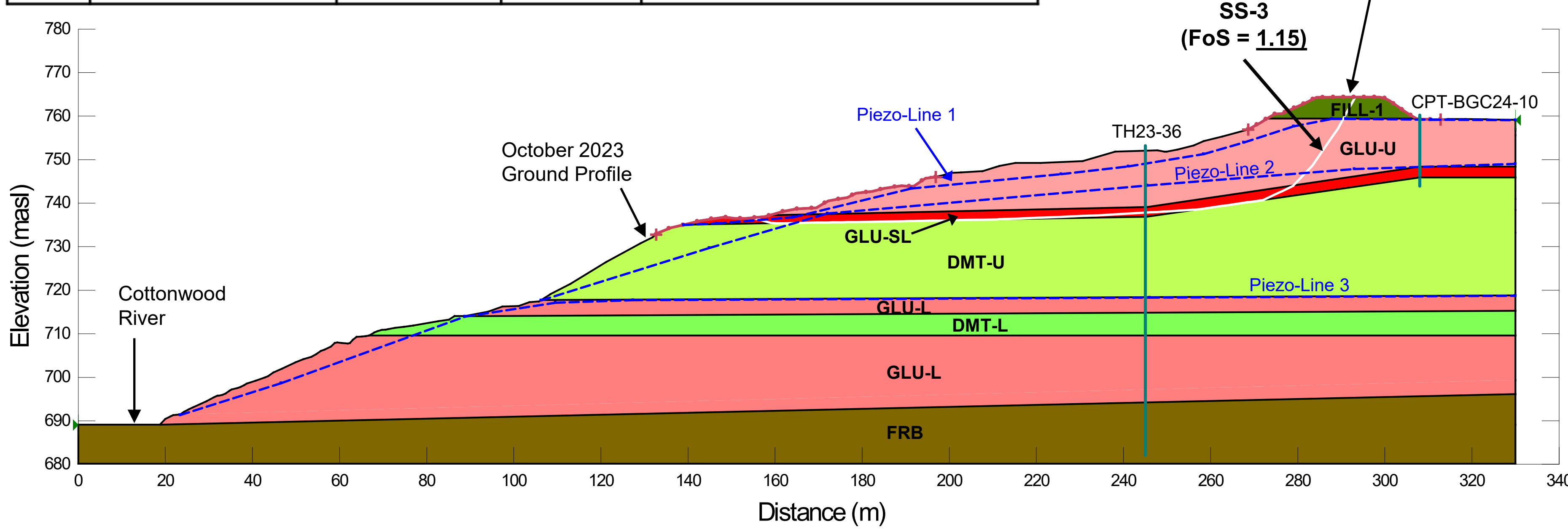
1. This Figure should be read in conjunction with BGC's report titled "Cottonwood River Slide – North Slide Stability Assessment", and dated May 2024.

PREPARED BY: SLB	FIGURE TITLE: Section NS1 Baseline Conditions Effective Stress Analysis, Slip-Surface 2
CHECKED BY: RK	CLIENT: R.F. Binnie & Associates Ltd.
APPROVED BY: JC	SCALE: NTS; PROJECT NO: 1262102; FIGURE NO: D3

0 5 10 mm in ANSI A sized paper

05-2024

Colour	Material Type	Soil Unit Name	Piezometric Line	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Friction Angle
	Existing Embankment Fill	FILL-1	1	20	0	34
	Upper GLU	GLU-U	1	18	0	30
	GLU - Sliding Layer	GLU-SL	2	18	0	14
	Lower GLU	GLU-L	None	Bedrock (Impenetrable)		
	Upper Diamicton	DMT-U	None	Bedrock (Impenetrable)		
	Lower Diamicton	DMT-L	None	Bedrock (Impenetrable)		
	Fraser Bend Formation	FRB	None	Bedrock (Impenetrable)		



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File Name: CS-NS1.gsz

NOTES:

1. This Figure should be read in conjunction with BGC's report titled "Cottonwood River Slide – North Slide Stability Assessment", and dated May 2024.

PREPARED BY: SLB

FIGURE TITLE: Section NS1
Baseline Conditions
Effective Stress Analysis, Slip-Surface 3

CHECKED BY: RK

CLIENT: R.F. Binnie & Associates Ltd.

APPROVED BY: JC

SCALE: NTS

PROJECT NO: 1262102

FIGURE NO: D4

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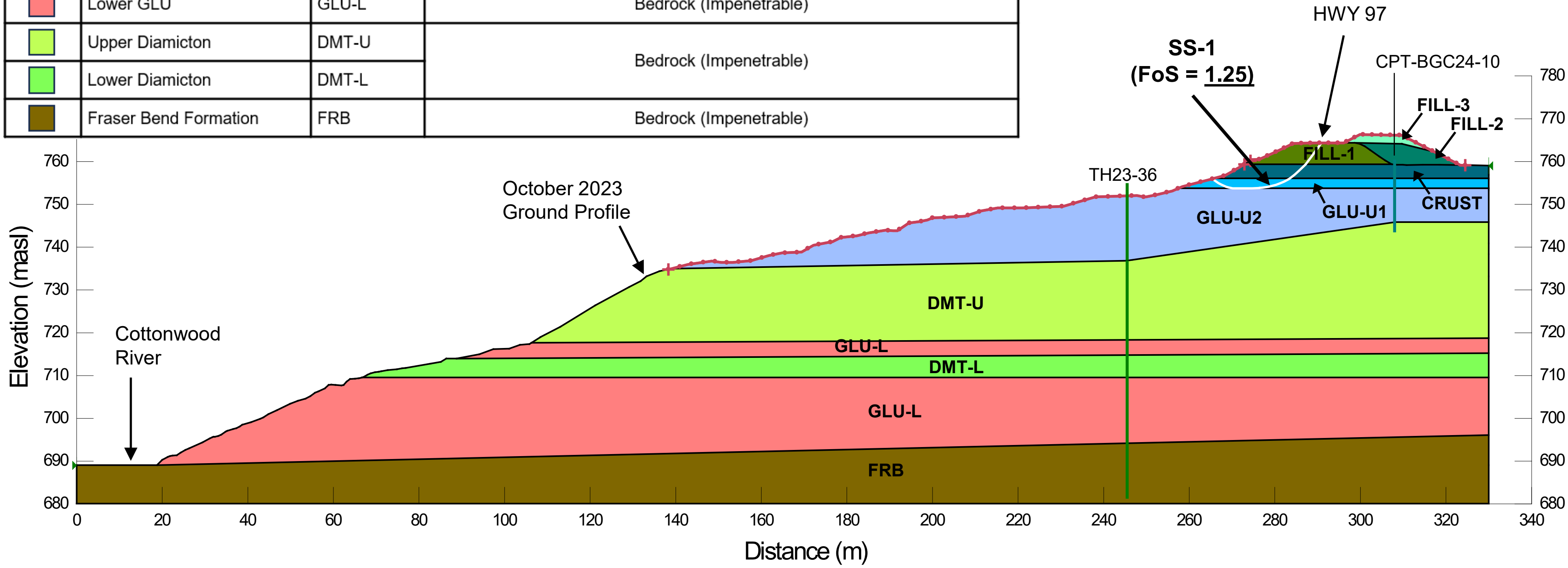
Colour	Material Type	Soil Unit Name	Unit Weight (kN/m³)	Total Cohesion (kPa)	Total Cohesion Rate of Change (kPa/m)	Effective Cohesion (kPa)	Friction Angle
	Existing Embankment Fill	FILL-1	20	-	-	0	34
	Proposed Embankment Fill	FILL-2	20	-	-	0	36
	Proposed Surcharge Fill	FILL-3	20	-	-	0	36
	Weathered Crest	CRUST	18	40	-	-	-
	Upper GLU	GLU-U1	18	30	-	-	-
		GLU-U2	18	35	5	-	-
	Lower GLU	GLU-L	Bedrock (Impenetrable)				
	Upper Diamicton	DMT-U	Bedrock (Impenetrable)				
	Lower Diamicton	DMT-L					
	Fraser Bend Formation	FRB	Bedrock (Impenetrable)				



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File Name: CS-NS1.gsz

NOTES:

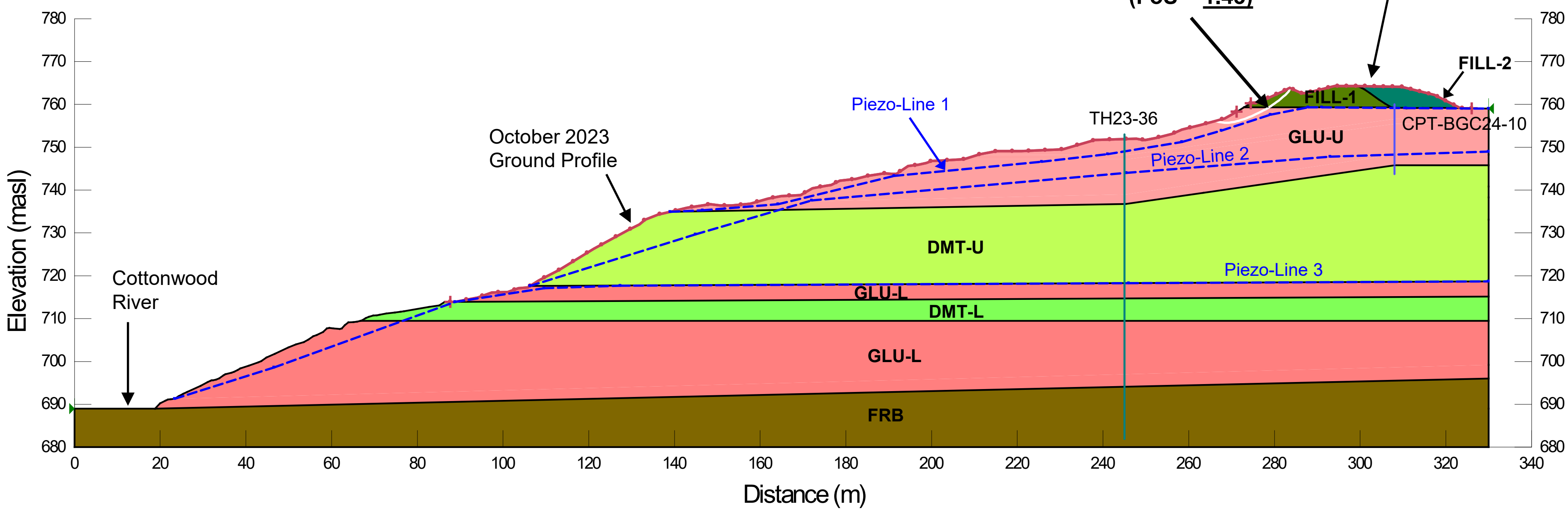
1. This Figure should be read in conjunction with BGC's report titled "Cottonwood River Slide – North Slide Stability Assessment", and dated May 2024.

PREPARED BY: SLB	FIGURE TITLE: Section NS1 Forward-Looking Analysis Stage 1 Undrained Strength Analysis, Slip-Surface 1	
CHECKED BY: RK	CLIENT: R.F. Binnie & Associates Ltd.	
APPROVED BY: JC	SCALE: NTS	PROJECT NO: 1262102 · FIGURE NO: D5

0 5 10 mm in ANSI A sized paper

05-2024

Colour	Material Type	Soil Unit Name	Piezometric Line	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Friction Angle
	Existing Embankment Fill	FILL-1	1	20	0	34
	Proposed Embankment Fill	FILL-2	1	20	0	36
	Upper GLU	GLU-U	1	18	0	30
	Lower GLU	GLU-L	3	20	0	30
	Upper Diamicton	DMT-U	2	20	5	35
	Lower Diamicton	DMT-L	3			
	Fraser Bend Formation	FRB	None	Bedrock (Impenetrable)		



File Location: https://ibgcengineering.sharepoint.com/:f:/s/1262-CottonwoodRiverSlide/Er60_U9wlThAnDK0WUiKILEB-N-M83D-IlJ1rzv0hlL4-A?e=6p7CB9
File Name: CS-NS1.gsz

NOTES:

1. This Figure should be read in conjunction with BGC's report titled "Cottonwood River Slide – North Slide Stability Assessment", and dated May 2024.

PREPARED BY: SLB	FIGURE TITLE: Section NS1 Forward-Looking Analysis Stage 3 Effective Stress Analysis, Slip-Surface 1		
CHECKED BY: RK	CLIENT: R.F. Binnie & Associates Ltd.		
APPROVED BY: JC	SCALE: NTS	PROJECT NO: 1262102	FIGURE NO: D6

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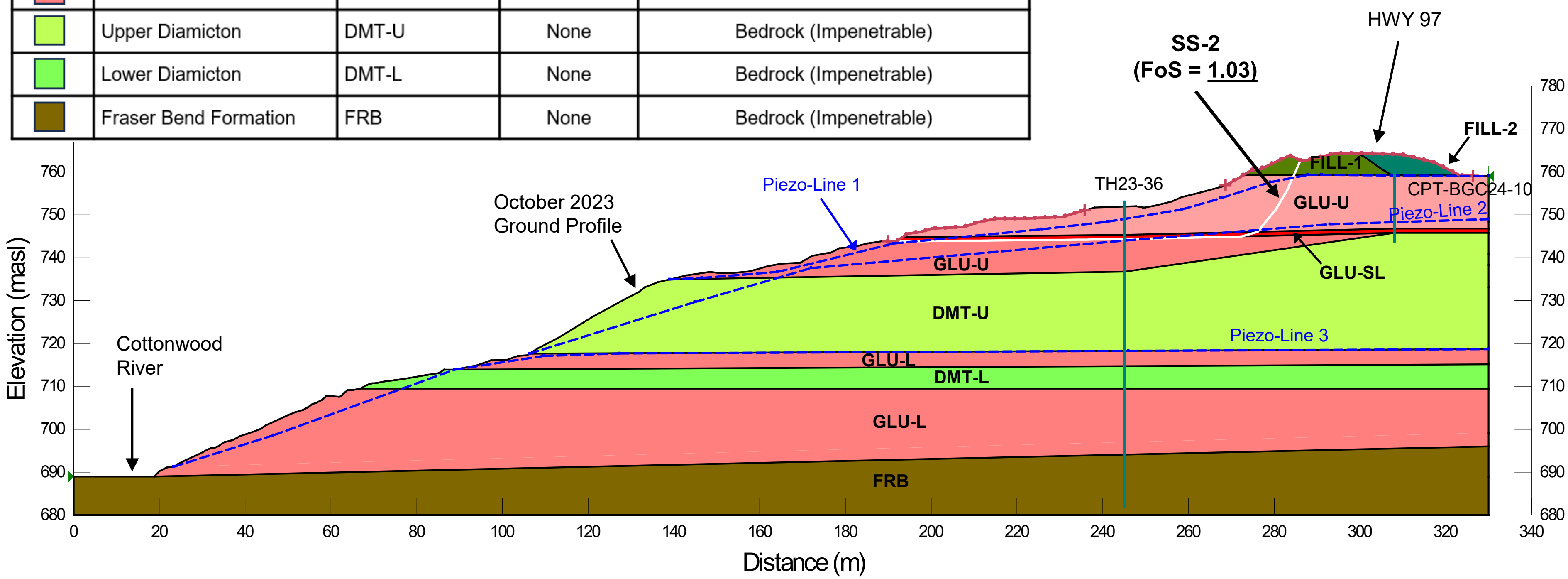
05-2024

Colour	Material Type	Soil Unit Name	Piezometric Line	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Friction Angle
	Existing Embankment Fill	FILL-1	1	20	0	34
	Proposed Embankment Fill	FILL-2	1	20	0	36
	Upper GLU	GLU-U	1	18	0	30
	GLU - Sliding Layer	GLU-SL	1	18	0	14
	Upper GLU	GLU-U	None	Bedrock (Impenetrable)		
	Lower GLU	GLU-L	None	Bedrock (Impenetrable)		
	Upper Diamicton	DMT-U	None	Bedrock (Impenetrable)		
	Lower Diamicton	DMT-L	None	Bedrock (Impenetrable)		
	Fraser Bend Formation	FRB	None	Bedrock (Impenetrable)		



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File Name: CS-NS1.gsz

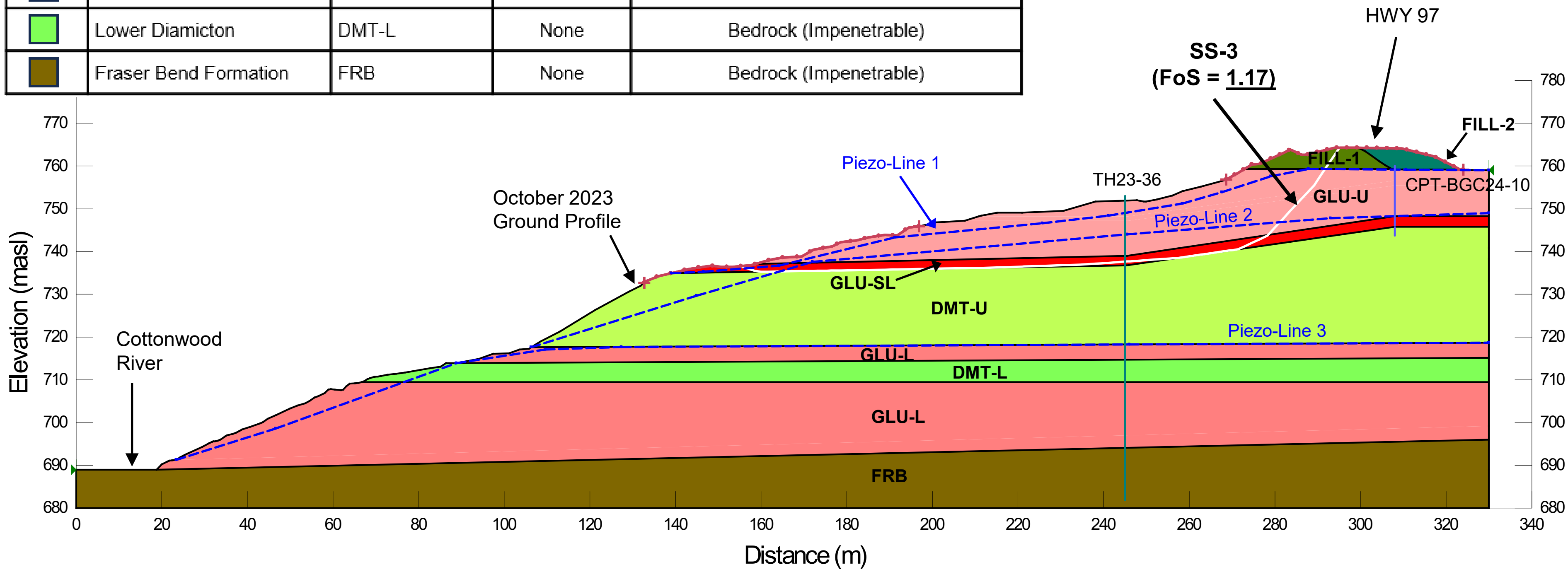
NOTES:

1. This Figure should be read in conjunction with BGC's report titled "Cottonwood River Slide – North Slide Stability Assessment", and dated May 2024.

PREPARED BY: SLB	FIGURE TITLE: Section NS1 Forward-Looking Analysis Stage 3 Effective Stress Analysis, Slip-Surface 2			
CHECKED BY: RK	CLIENT: R.F. Binnie & Associates Ltd.			
APPROVED BY: JC	SCALE: NTS	PROJECT NO: 1262102	FIGURE NO: D7	

0 5 10 mm in ANSI A sized paper

Colour	Material Type	Soil Unit Name	Piezometric Line	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Friction Angle
	Existing Embankment Fill	FILL-1	1	20	0	34
	Proposed Embankment Fill	FILL-2	1	20	0	36
	Upper GLU	GLU-U	1	18	0	30
	GLU - Sliding Layer	GLU-SL	2	18	0	14
	Lower GLU	GLU-L	None	Bedrock (Impenetrable)		
	Upper Diamicton	DMT-U	None	Bedrock (Impenetrable)		
	Lower Diamicton	DMT-L	None	Bedrock (Impenetrable)		
	Fraser Bend Formation	FRB	None	Bedrock (Impenetrable)		



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File Name: CS-NS1.gsz

NOTES:

1. This Figure should be read in conjunction with BGC's report titled "Cottonwood River Slide – North Slide Stability Assessment", and dated May 2024.

PREPARED BY: SLB	FIGURE TITLE: Section NS1 Forward-Looking Analysis Stage 3 Effective Stress Analysis, Slip-Surface 3		
CHECKED BY: RK	CLIENT: R.F. Binnie & Associates Ltd.		
APPROVED BY: JC	SCALE: NTS	PROJECT NO: 1262102	FIGURE NO: D8

0 5 10 mm in ANSI A sized paper