May 23, 2024

Project 1262102
Document 1262101-LT-240010

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

Cottonwood River Slide - Phase 2 Updated Embankment Widening Stability Assessment

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 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 Binnie’s, Phase 2 100% Detailed Design Drawings (Binnie, April 4, 2024), 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 being considered as an option to reduce post construction settlement. The proposed surcharge extent spans a length of 320 m from Station 207+80 to Station 211+00, and a typical section is provided on drawing number R2-1249-303 (Binnie & Associates Ltd., April 4, 2024).

A preliminary embankment stability assessment was completed and reported in the Geotechnical Design Report – Phase 2 (BGC, March 20, 2024). Since then, additional geotechnical investigation has been completed (BGC, May 23, 2024a). This letter report presents an updated embankment widening stability assessment based on the data collected from the recent geotechnical investigation. This stability assessment considers local stability of the highway embankment during and after its construction. Assessment of global stability of North Slide including the potential impact from the highway widening embankment are addressed in a separate report (BGC, May 23, 2024b).

2.0 SLOPE STABILITY ANALYSIS

2.1 Design Criteria

The minimum design factor of safety (FoS) for the embankment widening stability assessment was adopted from Table 6.2b of the BC MoTI design supplement to the CSA S6:19 (July 2022) and is summarised in Table 2-1. Given the recent geotechnical investigation which included cone penetration testing (CPTu) and vane shear testing (VST), a high degree of understanding and a typical consequence factor were used to select the minimum FoS for slope stability assessment.

Table 2-1 Summary of factor of safety design criteria.

Case	Loading Condition	Minimum FoS[1]
FOS for Global Stability – permanent	Static	1.43
FOS for Global Stability – permanent	Pseudo-Static	1.30
FOS for Global Stability – temporary	Static	1.25

Note: Minimum design FoS assumes a high degree of understanding and a typical consequence factor.

2.2 Geotechnical Profile

For the updated embankment widening stability assessment, the geotechnical profile provided in Table 2-2 was adopted for the native ground, based on the data from the recent geotechnical investigation as summarized in Appendix A and shown on Drawing 01. The geotechnical profile is also shown in Appendix A on Figure A-2. The descriptions provided in Table 2-2 are based on previous information reported in the Phase 2 design report, classification testing on Shelby tube samples from the recent hollow stem boreholes (BGC, May 23, 2024a) and interpretation of the CPTu data.

Table 2-2 Interpreted geotechnical profile of native ground.

Material name	Geotechnical description for stability analysis	Base of unit (mbgs)	Unit thickness (m)
Weathered crust	Over-consolidated Silty CLAY – stiff to very stiff, with pockets of firm clay	2.5	2.5
Upper GLU (lower q_t values)	Over-consolidated Silty CLAY – firm	6.0	3.5
Upper GLU	Over-consolidated Silty CLAY – firm becoming stiff	13.0	7.0
Lower Clay	Over-consolidated CLAY and SILT – stiff to very stiff	Not proven	N/A

2.3 Geotechnical Parameters

A summary of the geotechnical parameters used to complete the updated stability assessment is provided in Table 2-3. Selection of the geotechnical parameters considers the available data collected in the vicinity of the proposed embankment widening, as discussed in Appendix A. The undrained shear strength parameters and profiles provided in Table 2-3 and Appendix A represent the lower bound of interpreted values across the five CPTu locations.

It is noted that the geotechnical parameters assigned to the fill and foundation soil apply only to the models developed to complete this assessment; alternate parameters may be used in future analyses to assess other possible loading scenarios or combinations.

Table 2-3 Geotechnical parameters used in this assessment.

Material name	Bulk unit weight (kN/m^3)	Peak undrained shear strength (kPa)	Effective Stress shear strength (Mohr-Coulomb)
Existing embankment	20	N/A	ϕ' = 34°, c' = 0 kPa
Proposed embankment and surcharge fill	20	N/A	ϕ' = 36°, c' = 0 kPa
Weathered Crust	18	40	ϕ' = 30°, c' = 0 kPa
Upper GLU (lower q_t values)	18	30	
Upper GLU	18	35 + 5z, where z is depth below top of the Upper GLU unit in metres.	

Note: N/A notes that the design parameter is not applicable to the soil unit.

2.4 Piezometric levels

Limited groundwater monitoring data exists for the embankment widening location; however, piezometric levels were measured at ground surface level in March 2024 from the instrumentation installed during the recent geotechnical investigation (BGC, May 23, 2024a), which coincided with observed local spring thaw and melting snow. A piezometric level at the native ground surface below the embankment fills was adopted for the analyses, as shown on the stability analysis results provided in Appendix B.

2.5 Methodology

Two-dimensional limit equilibrium slope stability analyses were carried out with the computer program Slide2 (Version 9.027) using the GLE Morgenstern-Price method of slices. Slide2's auto refine search for non-circular failure surfaces was used to search for the critical slip surface. The stability considered the cases summarized in Table 2-4, using the profile and parameters provided in Table 2-2 and Table 2-3 respectively, and with the following assumptions:

- The deposits below the existing embankment fill have reached 100% consolidation.
- The proposed embankment and surcharge fill is placed at a maximum embankment slope angle of 2H:1V (horizontal:vertical) and up to a maximum height of 7 m above the native ground surface, as per the typical surcharge section shown on drawing number R2-1249-303 (Binnie, April 4, 2024).

The stability analyses consider local stability of the highway embankment only and the results presented are for a left to right sliding direction (away from the North Slide). A check of local stability was also completed on the right to left sliding direction (towards the North Slide) for Case 1 and the critical slip direction was assessed as left to right (away from the North Slide). The influence of the proposed embankment on the global stability of the North Slide is reported separately (BGC, May 23, 2024b).

The embankment and surcharge geometry selected for analyses is considered by BGC to be representative of the maximum fill thicknesses to be placed between Stations 207+80 and 211+00, per the Phase 2 100% Detailed Design Drawings (Binnie, April 4, 2024).

The slope stability analyses were conducted for both static and pseudo-static loading conditions. The pseudo-static analyses 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 (BGC, March 20, 2024).

Table 2-4 Summary of cases.

Case ID	Case description	Adopted strength model	Loading Condition	Additional loading
1	Immediately after placement of proposed embankment and surcharge fill.	Undrained shear strength (where applicable).	Static	10 kPa added to the top of the proposed surcharge fill to account for construction traffic
2	After dissipation of pore pressures but before removal of surcharge fill.	Drained shear strength.	Static	10 kPa added to the top of the proposed surcharge fill to account for construction traffic
3	After dissipation of pore pressures and after removal of surcharge fill.	Drained shear strength.	Static	None
4	After dissipation of pore pressures and after removal of surcharge fill.	Drained shear strength.	Pseudo-static	None

2.6 Results

The results of the stability assessment are provided in Appendix B and summarised in Table 2-5. For the assessed cases, the minimum FoS criteria were achieved.

As discussed in BGC's detailed design report for Phase 2 (BGC, March 20, 2024), climate change may result in more frequent and severe precipitation events. These events could potentially increase the porewater pressures within the embankment fill and foundation soil, lowering the FoS value presented in Table 2-5 for Case 3. To increase climate change resilience, the design of the proposed embankment incorporates the following components:

- To minimize surface water ponding at the toe of the embankment, the proposed cross culverts were designed to accommodate the climate change adjusted design flow.
- The proposed embankment will be founded on a 0.3 m thick layer of granular, free-draining material (Binnie, April 4, 2024). The drainage layer will help reduce the magnitude of porewater pressures that may develop within the embankment fill.

- The results of the slope stability analyses indicate that the final geometry of the embankment is robust (Case 3), as indicated by a calculated FoS 1.52 compared to a minimum required FoS of 1.43 (refer to Table 2-5).

It is noted that more frequent and severe precipitation events may results in increased surficial sloughing of the embankment side slopes; increased roadway maintenance efforts and the future installation erosion control measures may be required in response to climate change.

Table 2-5 Summary of stability results.

Case ID	Case description	Minimum FoS (from Table 2-1)	Modelled FoS	Relevant figure
1	Immediately after placement of proposed embankment and surcharge fill.	1.25	1.25	B-1
2	After dissipation of pore pressures but before removal of surcharge fill.	1.25	1.49	B-2
3	After dissipation of pore pressures and after removal of surcharge fill (static).	1.43	1.52	B-3
4	After dissipation of pore pressures and after removal of surcharge fill (pseudo-static).	1.30	1.48	B-4

3.0 RECOMMENDATIONS

Geotechnical recommendations for the embankment construction are provided in the Phase 2 design report (BGC, March 20, 2024). Based on the updated stability analysis, the following updates to the geotechnical recommendations are provided:

- The proposed embankment and surcharge fill can be constructed in one continuous stage, without a Hold Point during fill placement.

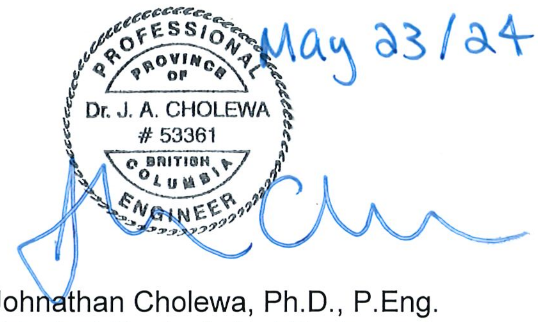
All other recommendations relating to the embankment construction provided in the Phase 2 design report (BGC, March 20, 2024) are still valid.

4.0 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 Inc.
per:



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EGBC Permit to Practice, BGC Engineering Inc. 1000944

MD/RK/sa/mm

Attachment(s): Limitations
References
Drawings
Appendix A – Geotechnical parameter interpretation
Appendix B – Stability analysis results

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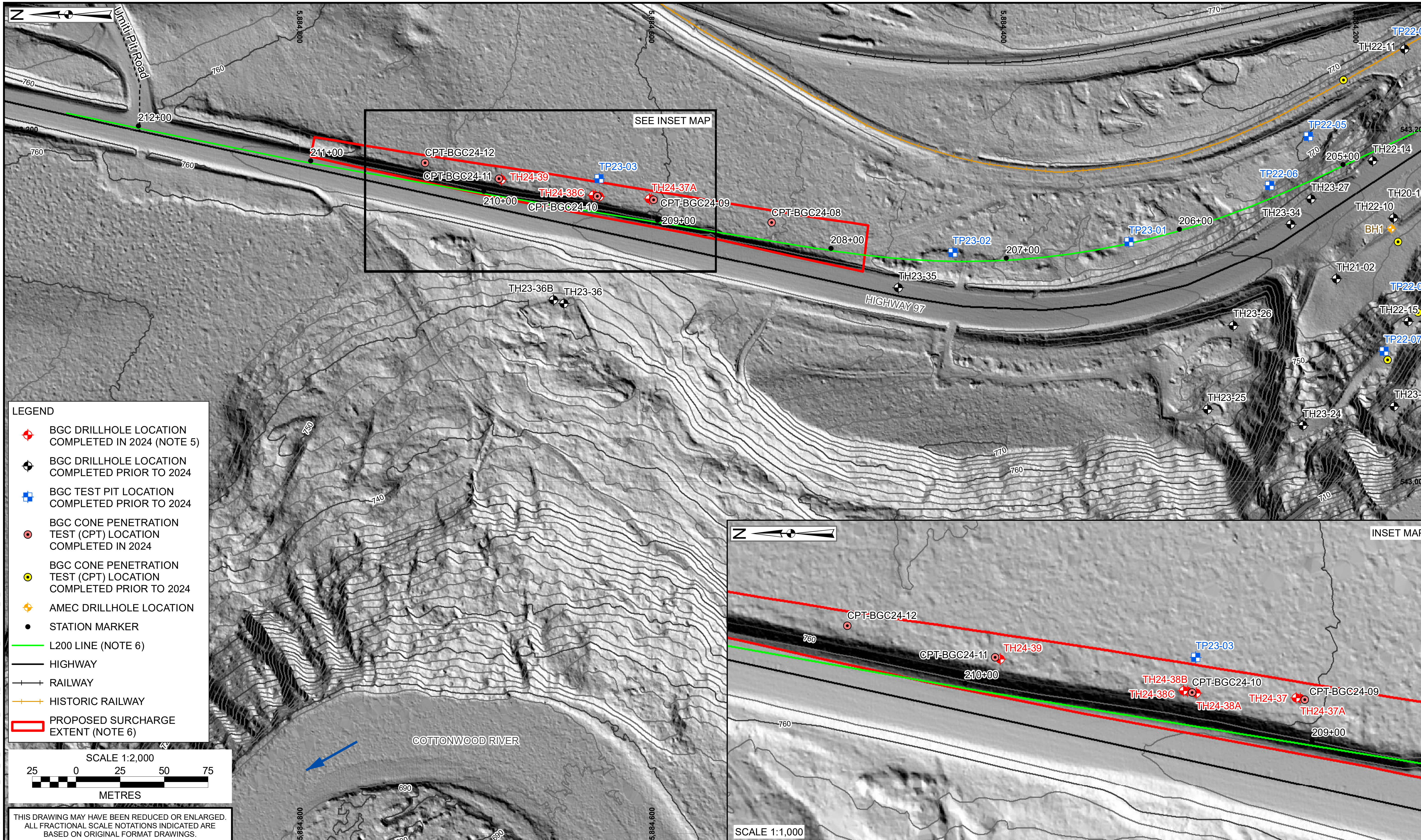
DRAWINGS



NOTES:

1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2. THIS DRAWING MUST BE READ IN CONJUNCTION WITH BGC'S REPORT TITLED "COTTONWOOD RIVER SLIDE - PHASE 2 UPDATED EMBANKMENT STABILITY ASSESSMENT", AND DATED MAY 2024.
3. BASE TOPOGRAPHIC DATA BASED ON LIDAR PROVIDED BY MOTI, DATED OCTOBER 2023 AND AUGUST 2021. CONTOUR INTERVAL IS 5 m.
4. HYDROGRAPHIC BASE DATA FROM CANVEC (NATIONAL RESOURCES CANADA).
5. Q1 2024 DRILLHOLE COORDINATES ARE BASED ON THEIR RELATIVE POSITIONS TO THEIR CORRESPONDING Q1 2024 CPT LOCATIONS. Q1 2024 CPT COORDINATES ARE FROM THE CONETEC REPORT.
6. L200 LINE AND SURCHARGE EXTENTS WAS PROVIDED BY BINNIE ON APRIL 4, 2024 AS PART OF THE 100% DETAILED DESIGN PHASE 2 DRAWINGS.
7. COORDINATE SYSTEM IS NAD 1983 UTM ZONE 10N. VERTICAL DATUM IS UNKNOWN.
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PROJECT: COTTONWOOD RIVER SLIDE
PHASE 2 UPDATED EMBANKMENT STABILITY ASSESSMENT

TITLE: GEOTECHNICAL SITE INVESTIGATION OVERVIEW

PROJECT No.: 1262102

DWG No: 01

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APPENDIX A
GEOTECHNICAL PARAMETER INTERPRETATION

A-1 AVAILABLE GEOTECHNICAL DATA

The geotechnical test locations are listed in Table A-1 and shown on Drawing 01 along with the outline of the proposed embankment widening. The geotechnical data interpreted below is provided in BGC's previous geotechnical data reports (March 20, 2024 and May 23, 2024).

Table A-1 Summary of test locations at the embankment widening.

Test cluster location	Location ID	Test type	Termination depth (mbgs[1])	Observed groundwater depth range[2] (mbgs)
1	CPT-BGC24-08	CPTu	13.1	
2	CPT-BGC24-09	CPTu	13.7	
	TH24-37	VWP	5.5	0.0 to 0.7
	TH24-37A	Shelby tube sampling	7.7	
3	TP23-03	Test pit	3.2	
	CPT-BGC24-10	CPTu	15.5	
	TH24-38A	Vane shear testing	12.0	
	TH24-38B	Monitoring well and Shelby tube	7.9	0.1 to 0.8
	TH24-38C	VWP	5.4	0.0 to 0.8
4	CPT-BGC24-11	CPTu	20.9	
	TH24-39	VWP and Shelby tube sampling	6.6	0.0 to 0.6
5	CPT-BGC24-12	CPTu	16.9	

Note:
1. mbgs = metres below ground surface.
2. Groundwater depth range observed from March 3 to 20, 2024.

A-2 SUMMARY OF LABORATORY TESTING

The results of the laboratory tests carried out on soil samples collected at the proposed embankment widening location are summarised in Table A-2.

Table A-2 Summary of relevant laboratory testing.

Location ID	Sample depth (m)	Water content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Silt content (%)	Clay content (%)
TP23-03	1.1-1.2	31	48	23	25		
TP23-03	3.1-3.2	41	44	23	21		
TH24-37A	3.96-4.53	60	41	21	20	68	32
TH24-37A	7.01-7.71	50	59	25	34	36	64
TH24-38B	3.96-4.58	35	35	20	15	62	34
TH24-38B	7.32-7.90	47	59	24	35	36	63
TH24-39	4.00-4.61	72	48	21	27	74	26
TH24-39	6.00-6.61	43	56	25	31	36	63

A-3 INTERPRETATION OF GEOTECHNICAL DATA

The available geotechnical data was used to estimate soil shear strength parameters for use in the slope stability analyses. These parameters include the peak undrained shear strength (s_u) assigned to the weathered crust and Upper GLU, and effective stress parameters assigned to the weathered crust, Upper GLU, and embankment/surcharge fills. The over-consolidation ratio (OCR) for the weathered crust and Upper GLU was also estimated based on the undrained shear strength data obtained from the CPTu and vane shear tests. Interpretation of the geotechnical parameters is discussed in the subsections below.

A-3.1 Undrained Shear Strength

The TH24-38A peak undrained shear strength values from the vane shear tests (s_u(FV)) were corrected using Bjerrum's (1972) empirical correction factor (μ) for circular arc stability analyses. The plasticity index of tested samples varied from 15% to 35% (Table A-2), which results in a range of (μ) from 0.9 to 1.0. A value of 0.9 was adopted for the assessment. The corrected values of undrained shear strength from vane shear testing (s_u(ave)) are shown on Figure A-1.

Undrained shear strength from CPTu data was estimated as follows: $s_u = (q_t - \sigma_v)/N_{kt}$, (Lunne et al, 1997) where N_{kt} is a cone factor. The corrected undrained shear strength values from the vane shear testing (s_u(ave)) were used to calibrate the N_{kt} cone factor. An N_{kt} value of 14 was adopted which provided a good correlation with the vane shear testing values and is within the expected range for this parameter. The calibration between the vane shear test and the adjacent CPTu (CPT-BGC24-10) is shown on Figure A-1.

Finally, the undrained shear strength was estimated for all five CPTu, using the same calibration (N_{kt} = 14) and is shown on Figure A-2. The undrained shear strength design profile adopted for the slope stability analyses for embankment widening is also shown on Figure A-2.

A-3.2 Over-Consolidation Ratio

From the estimated undrained shear strength, an OCR was estimated from the SHANSEP equation (Ladd et al, 2004), as follows:

$$\left(\frac{s_u}{\sigma'_v}\right) = \left(\frac{s_u}{\sigma'_v}\right)_{NC} OCR^{0.8} = 0.22 * OCR^{0.8}$$

Where the normally consolidated undrained shear strength to vertical effective stress ratio typically ranges between about 0.15 and 0.3. A value of 0.22 was adopted for this analysis as recommended for low and high plasticity clays that plot above the A-line (Ladd, 1991).

A-3.3 Effective Stress Parameters - GLU

The effective stress parameters for the GLU were estimated using empirical correlations relating clay fraction, liquid limit, and effective normal stress to fully softened, drained shear strength (Stark and Hussien, 2013). For the index properties presented in Table A-2, and the magnitude of normal stress acting on potential slip surfaces, a friction angle, ϕ′ = 30° and effective cohesion, c' = 0 kPa was assigned to the GLU using Stark and Hussain (2013). This strength also corresponds to the strength assigned to the glaciolacustrine unit within the Cottonwood River Slide complex, south of the embankment widening area, which was estimated through back-analysis of the Cottonwood River Slide complex and considering the results of the consolidated isotopically undrained triaxial tests and drained direct shear tests (BGC, March 20, 2024).

A-3.4 Effective Stress Parameters – Embankment and Surcharge Fill

The Undifferentiated Soil (defined in BGC, March 20, 2024) will be the source of the proposed embankment fill. The Phase 2 design report (BGC, March 22, 2024) provided effective stress parameters of ϕ′ = 35° and c' = 5 kPa for the Undifferentiated Soil. A friction angle, ϕ′ = 36° and effective cohesion, c' = 0 kPa was assigned to the new embankment and surcharge fill since it provided a lower strength resistance for the expected stress conditions within the proposed embankment and surcharge fill.

For the existing embankment fill, a reduced friction angle, ϕ′ = 34° and effective cohesion, c' = 0 kPa was used due to uncertainty in composition and construction methods.

A-4 NOTATIONS

Notations used in the above discussion are as follows:

c’	Effective cohesion
N_{kt}	cone factor
OCR	over-consolidation ratio
q_t	corrected cone resistance
s_u	undrained shear strength
s_u(FV)	peak undrained shear strength from vane shear testing
s_u(ave)	corrected peak undrained shear strength from vane shear testing (μ * s_u(FV))
σ_v	total vertical stress
σ'_v	effective vertical stress
ϕ’	effective friction angle
μ	Bjerrum’s empirical correction factor.

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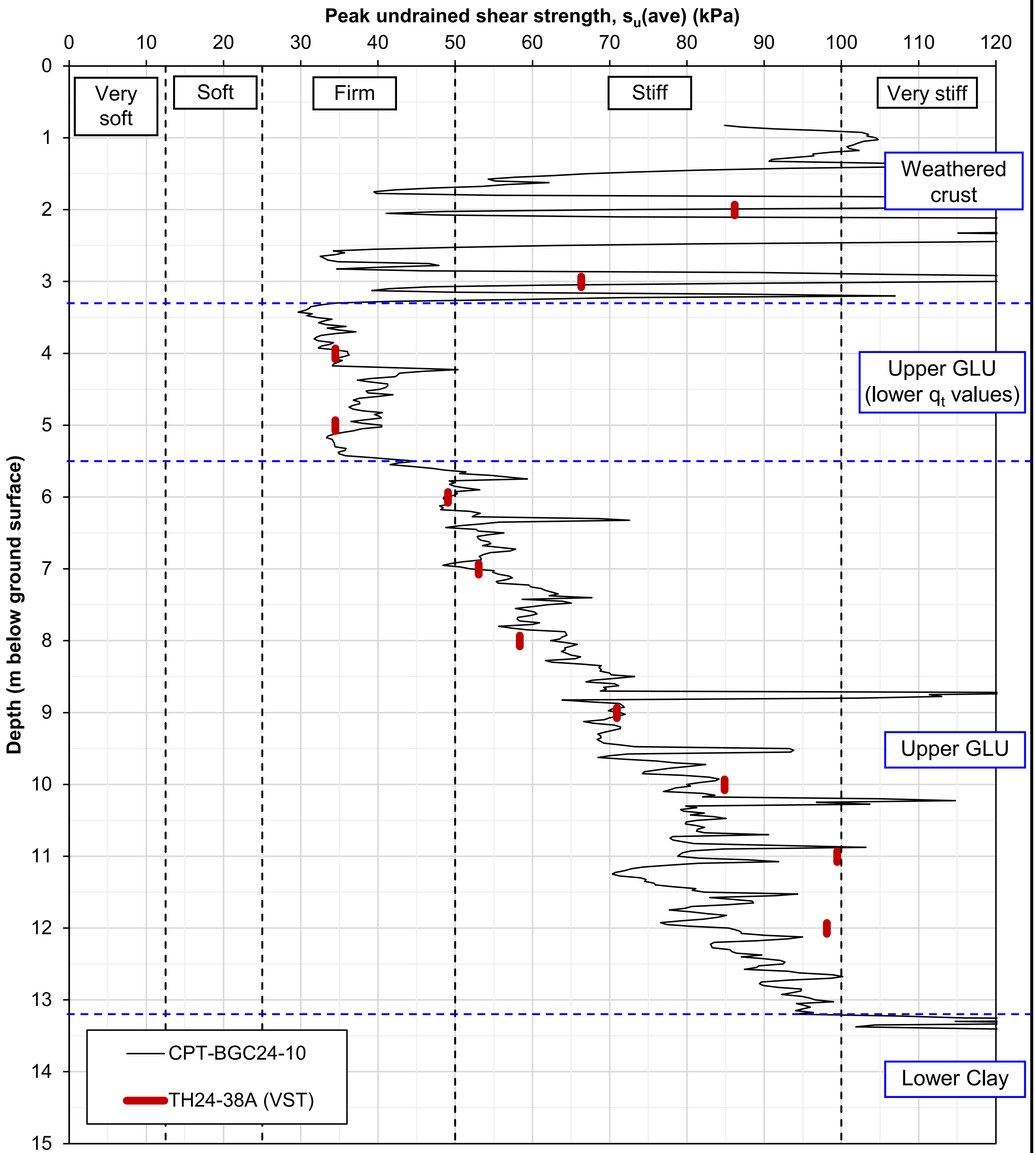
FIGURES



NOTES:	PREPARED BY: JT	FIGURE TITLE: UNDRAINED SHEAR STRENGTH VERSUS DEPTH (VST and CPT-BGC24-10)		
	CHECKED BY: RK	CLIENT: R.F. BINNIE & ASSOCIATES Ltd.		
	APPROVED BY: JC	SCALE: NTS	PROJECT NO: 1262102	FIGURE NO: A-1

NOTES:

1. This Figure should be read in conjunction with BGC's report titled Cottonwood River Slide - Phase 2 Updated Embankment Widening Stability Assessment, and dated May 2024.
2. Field vane shear strengths are plotted over the 15 cm vane length and have been corrected by subtracting the torque rod friction measured by the slip coupling.
3. The corrected undrained shear strength s_u(ave) from vane shear testing is the field vane shear strength multiplied by Bjerrum's empirical correction factor ($\mu * s_u(FV)$).
4. Adopted N_{kt} cone factor = 14.

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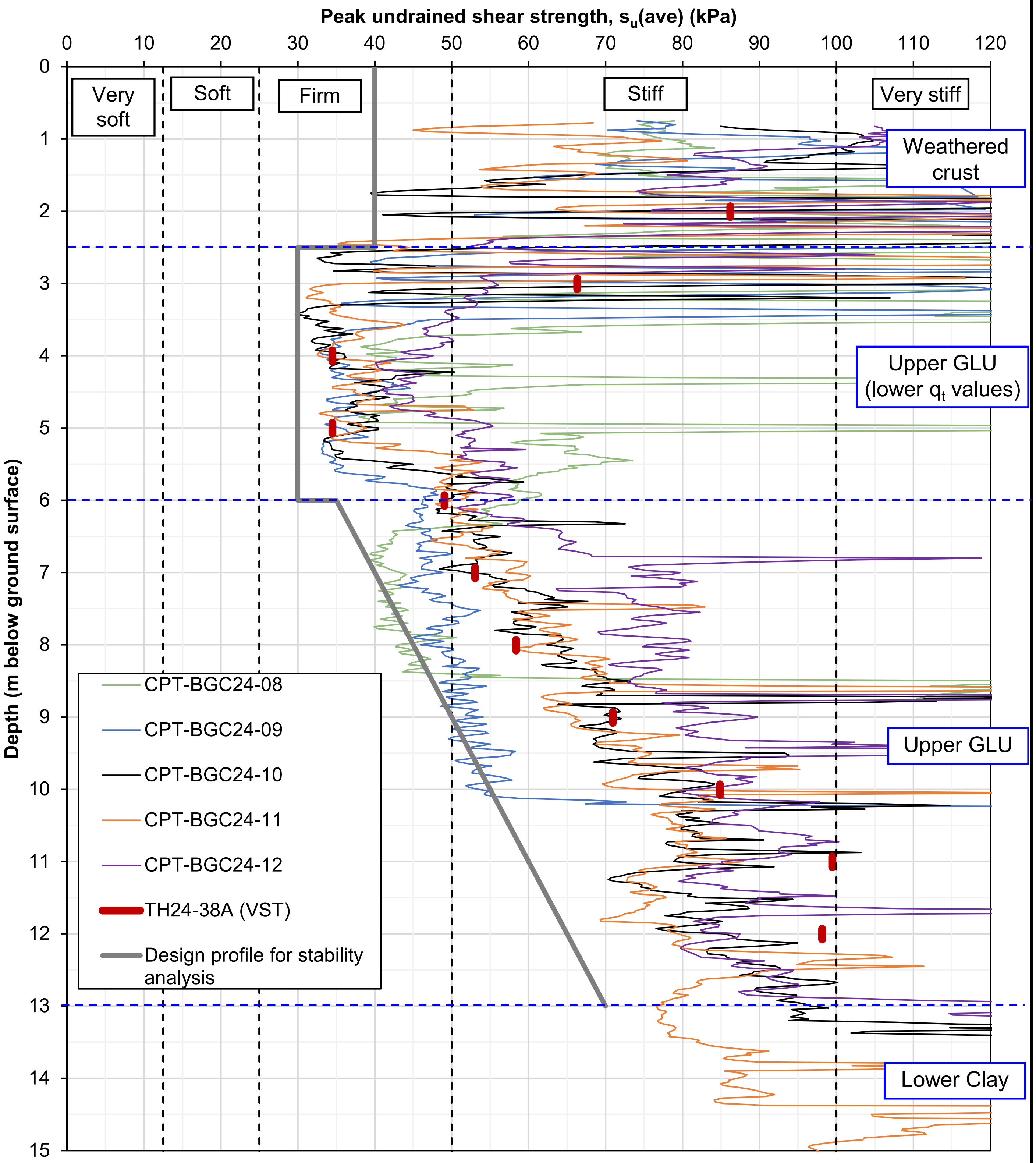

NOTES:

1. This Figure should be read in conjunction with BGC's report titled Cottonwood River Slide - Phase 2 Updated Embankment Widening Stability Assessment, and dated May 2024.
2. Field vane shear strengths are plotted over the 15 cm vane length and have been corrected by subtracting the torque rod friction measured by the slip coupling
3. The corrected undrained shear strength s_u(ave) from vane shear testing is the field vane shear strength multiplied by Bjerrum's empirical correction factor (μ * s_u(FV)).
4. Adopted N_{kt} cone factor = 14.

PREPARED BY: JT	FIGURE TITLE: UNDRAINED SHEAR STRENGTH VERSUS DEPTH (VST AND ALL CPTu)	
CHECKED BY: RK	CLIENT: R.F. BINNIE & ASSOCIATES Ltd.	
APPROVED BY: JC	SCALE: NTS	PROJECT NO: 1262102 · FIGURE NO: A-2

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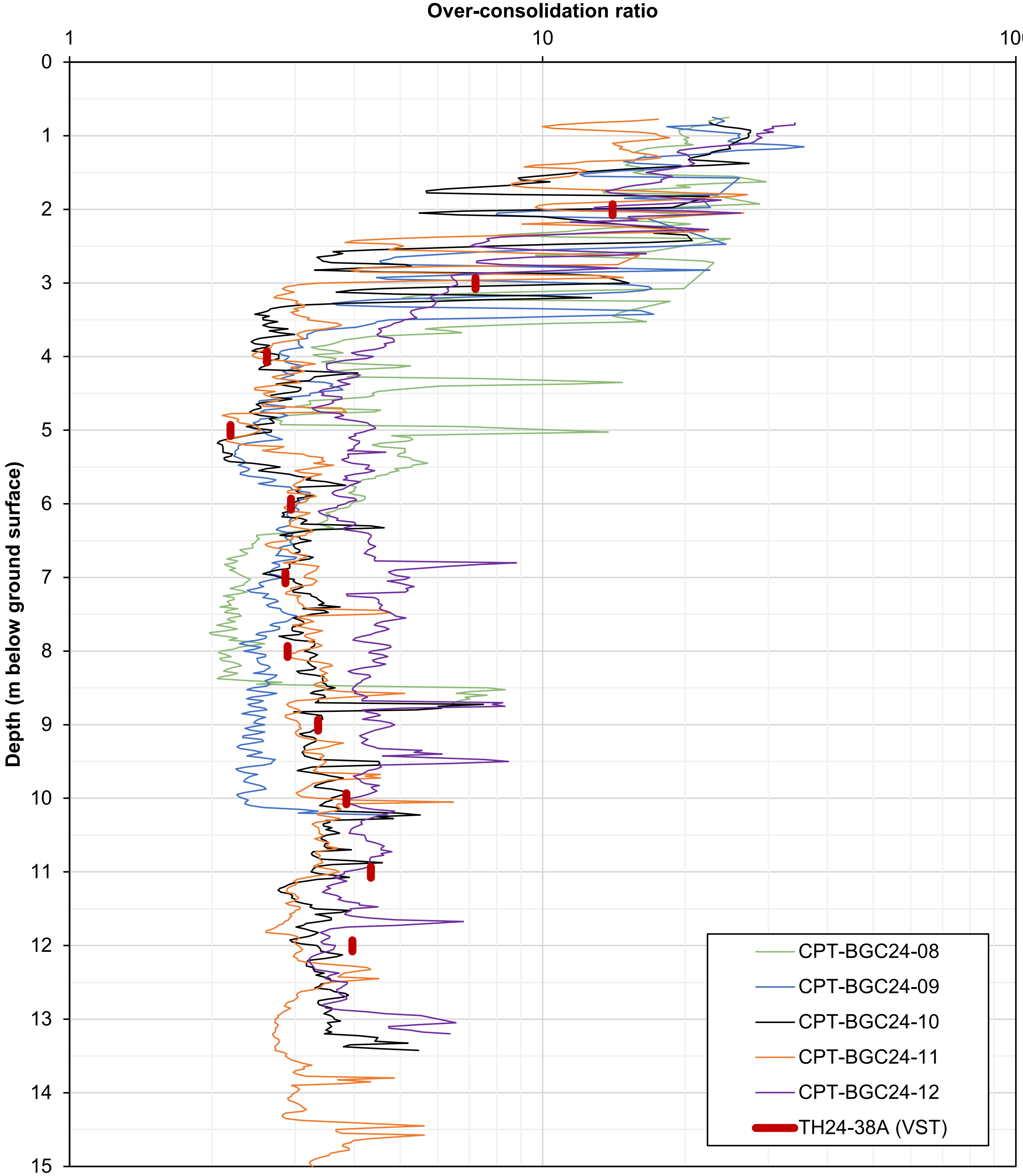
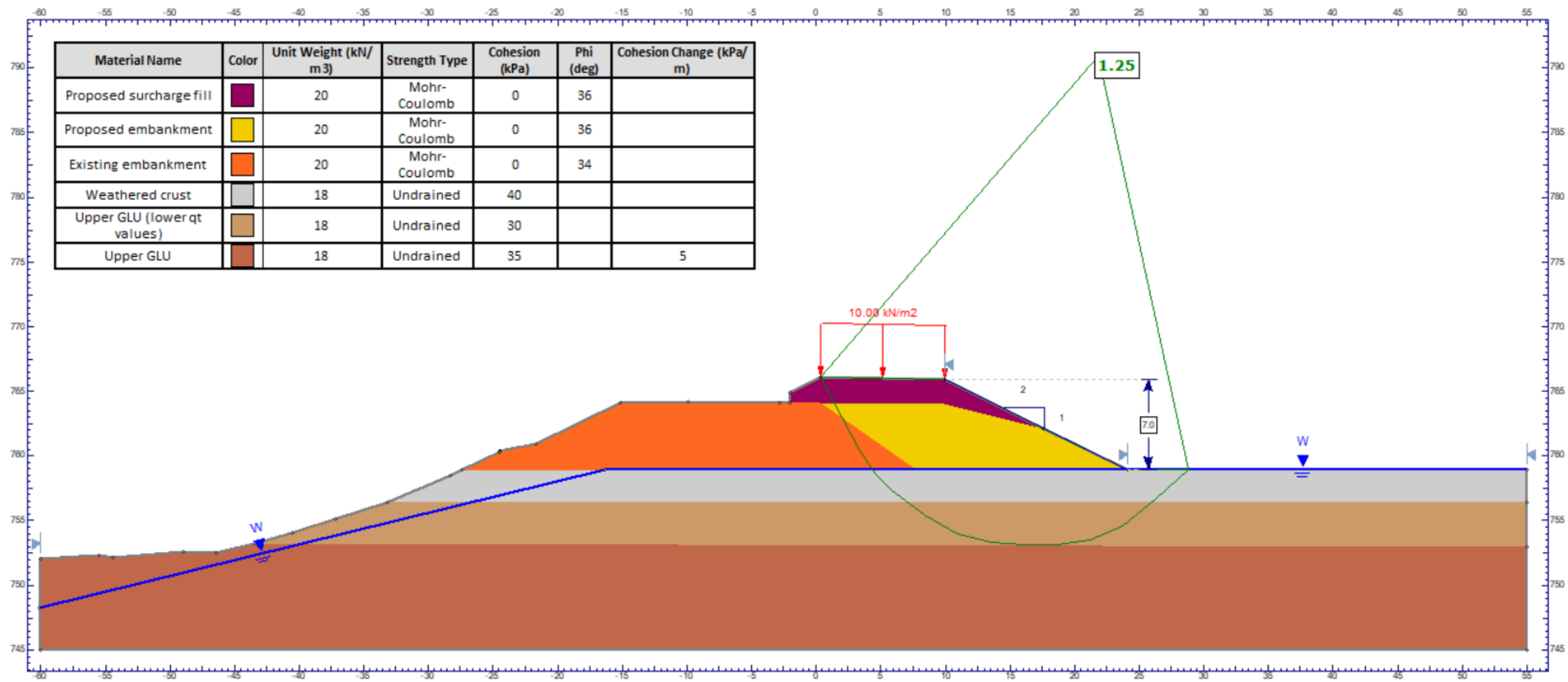


NOTES:

1. This Figure should be read in conjunction with BGC's report titled Cottonwood River Slide - Phase 2 Updated Embankment Widening Stability Assessment, and dated May 2024.
2. Field vane shear strengths are plotted over the 15 cm vane length and have been corrected by subtracting the torque rod friction measured by the slip coupling.
3. The corrected undrained shear strength s_u(ave) from vane shear testing is the field vane shear strength multiplied by Bjerrum's empirical correction factor ($\mu * s_u$(FV)).

PREPARED BY: JT	FIGURE TITLE: OVER-CONSOLIDATION RATIO VERSUS DEPTH		
CHECKED BY: RK	CLIENT: R.F. BINNIE & ASSOCIATES Ltd.		
APPROVED BY: JC	SCALE: NTS	PROJECT NO: 1262102	FIGURE NO: A-3

APPENDIX B
STABILITY ANALYSIS RESULTS



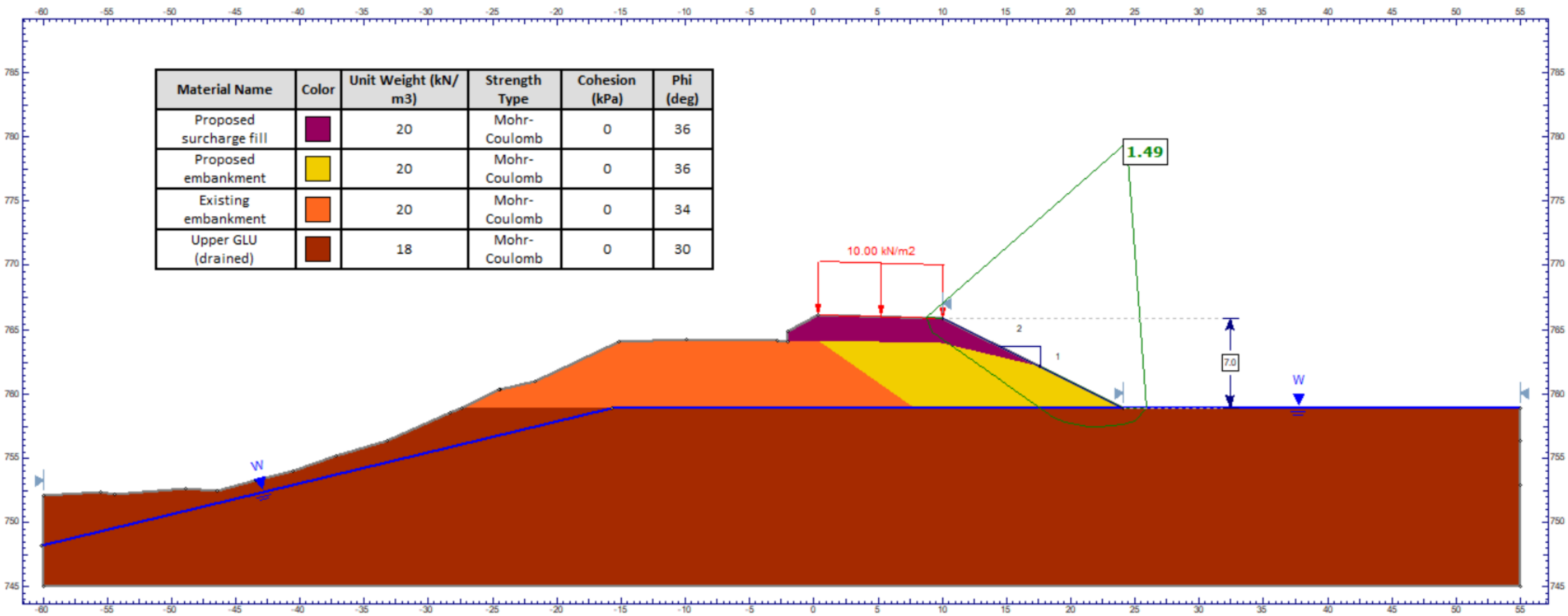
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Change (kPa/m)
Proposed surcharge fill		20	Mohr-Coulomb	0	36	
Proposed embankment		20	Mohr-Coulomb	0	36	
Existing embankment		20	Mohr-Coulomb	0	34	
Weathered crust		18	Undrained	40		
Upper GLU (lower qt values)		18	Undrained	30		
Upper GLU		18	Undrained	35		5

NOTES:

1. This Figure should be read in conjunction with BGC's report titled Cottonwood River Slide - Phase 2 Updated Embankment Widening Stability Assessment, and dated May 2024.

PREPARED BY: JT	FIGURE TITLE: STABILITY ASSESSMENT – CASE 1
CHECKED BY: RK	CLIENT: R.F. BINNIE & ASSOCIATES Ltd.
APPROVED BY: JC	SCALE: NTS; PROJECT NO: 1262102; FIGURE NO: B-1

0 5 10 mm in ANSI A sized paper



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Proposed surcharge fill		20	Mohr-Coulomb	0	36
Proposed embankment		20	Mohr-Coulomb	0	36
Existing embankment		20	Mohr-Coulomb	0	34
Upper GLU (drained)		18	Mohr-Coulomb	0	30

NOTES:

1. This Figure should be read in conjunction with BGC's report titled Cottonwood River Slide - Phase 2 Updated Embankment Widening Stability Assessment, and dated May 2024.

PREPARED BY: JT

FIGURE TITLE: STABILITY ASSESSMENT – CASE 2

CHECKED BY: RK

CLIENT: R.F. BINNIE & ASSOCIATES Ltd.

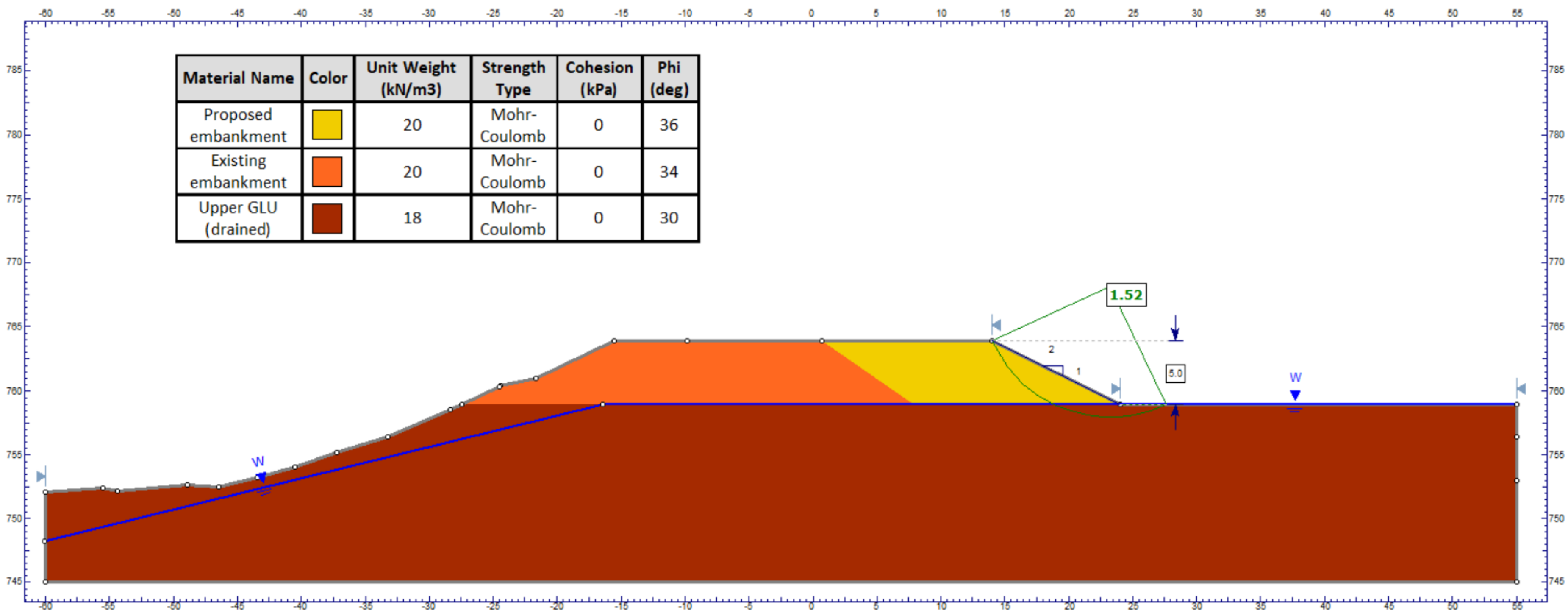
APPROVED BY: JC

SCALE: NTS

PROJECT NO: 1262102

FIGURE NO: B-2

0 5 10 mm in ANSI A sized paper



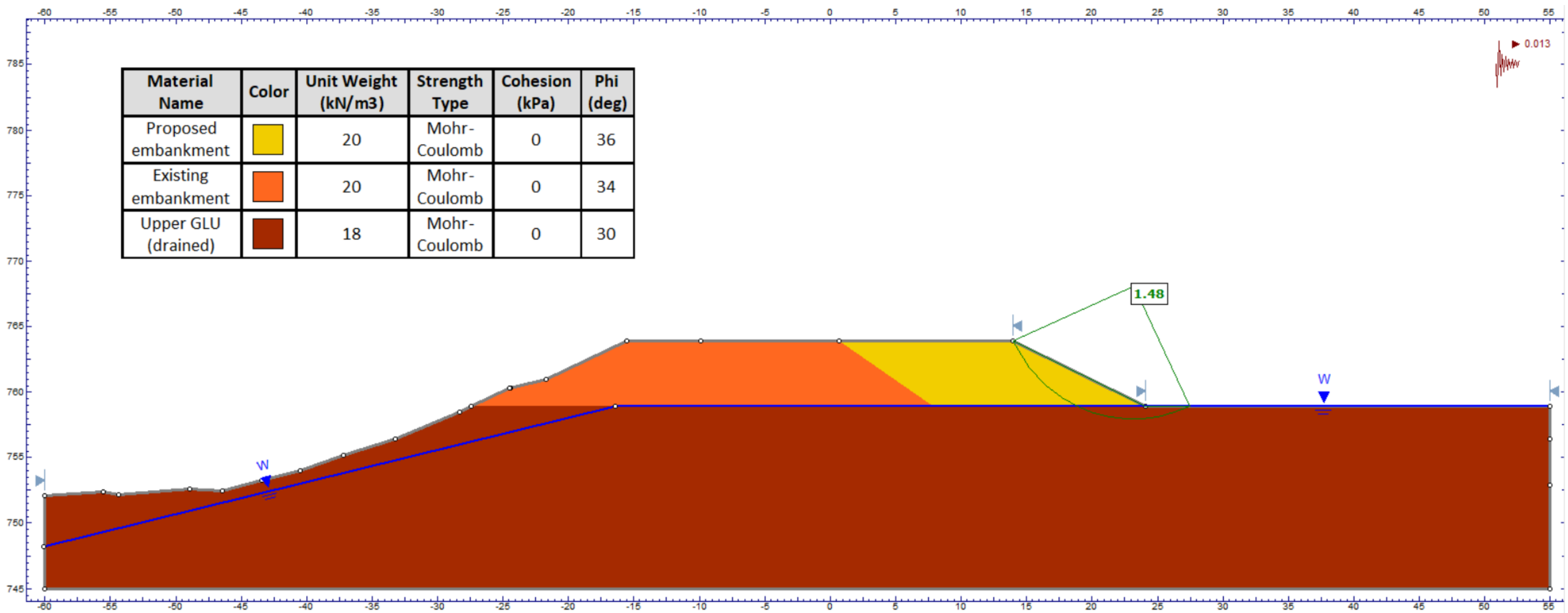
NOTES:

1. This Figure should be read in conjunction with BGC's report titled Cottonwood River Slide - Phase 2 Updated Embankment Widening Stability Assessment, and dated May 2024.

PREPARED BY: JT	FIGURE TITLE: STABILITY ASSESSMENT – CASE 3		
CHECKED BY: RK	CLIENT: R.F. BINNIE & ASSOCIATES Ltd.		
APPROVED BY: JC	SCALE: NTS	PROJECT NO: 1262102	FIGURE NO: B-3

0 5 10 mm in ANSI A sized paper

BGC Engineering



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Proposed embankment		20	Mohr-Coulomb	0	36
Existing embankment		20	Mohr-Coulomb	0	34
Upper GLU (drained)		18	Mohr-Coulomb	0	30

NOTES:

1. This Figure should be read in conjunction with BGC's report titled Cottonwood River Slide - Phase 2 Updated Embankment Widening Stability Assessment, and dated May 2024.

PREPARED BY: JT	FIGURE TITLE: STABILITY ASSESSMENT – CASE 4
CHECKED BY: RK	CLIENT: R.F. BINNIE & ASSOCIATES Ltd.
APPROVED BY: JC	SCALE: NTS
PROJECT NO: 1262102	FIGURE NO: B-4

0 5 10 mm in ANSI A sized paper