
Geotechnical Assessment A Drive Borrow Pit, Stockton Mine

✦ Prepared for

Bathurst Resources Limited

✦ March 2026



PATTLE DELAMORE PARTNERS LTD



Web www.pdp.co.nz



solutions for your environment

Quality Control Sheet

TITLE Geotechnical Assessment- A- Drive Borrow Pit, Stockton Mine

CLIENT Bathurst Resources Limited

ISSUE DATE 25 March 2026

JOB REFERENCE C052290003

Revision History					
REV	Date	Status/Purpose	Prepared By	Reviewed by	Approved
	09/09/2025	Draft	Jason Grieve	Andrew Smith	Andrew Smith
V 1	26/09/2025	Final	Jason Grieve	Andrew Smith	Andrew Smith
V 2	18/11/25	Final	Jason Grieve	Andrew Smith	Andrew Smith
V 3	5/12/2025	Final	Jason Grieve	Andrew Smith	Andrew Smith
V 4	28/01/2026	Final	Jason Grieve	Andrew Smith	Andrew Smith
V 5	25/03/2026	Final	Jason Grieve	Andrew Smith	Andrew Smith

DOCUMENT CONTRIBUTORS

Prepared by

SIGNATURE

Jason Grieve

Reviewed and approved by

SIGNATURE

Andrew Smith

Limitations:

This report has been prepared by Pattle Delamore Partners Limited (PDP) on the basis of information provided by Bathurst Resources Limited. PDP has not independently verified the provided information and has relied upon it being accurate and sufficient for use by PDP in preparing the report. PDP accepts no responsibility for errors or omissions in, or the currency or sufficiency of, the provided information.

This report has been prepared by PDP on the specific instructions of Bathurst Resources Limited for the limited purposes described in the report. PDP accepts no liability if the report is used for a different purpose or if it is used or relied on by any other person. Any such use or reliance will be solely at their own risk.

© 2026 Pattle Delamore Partners Limited

Executive Summary

Pattle Delamore Partners Ltd (PDP) was engaged by Bathurst Resources Limited to undertake a geotechnical assessment of the A-Drive Borrow Pit and proposed coal fines storage (CFS) stages 2 and 3, located at Stockton Mine, in support of the Buller Plateaux Continuation Project (BPCP) Fast Track Application (FTA). The borrow pit, which has historically supplied granitic material primarily for roading and minor amounts for rehabilitation, is proposed to be extended by approximately 60 m to the north-northeast. Following excavation, the site is intended to be repurposed for the storage of Coal Handling and Processing Plant (CHPP) tailings.

The assessment incorporated a comprehensive review of historical investigations, geological mapping, drilling and geophysical data, laboratory testing, and site walkovers. Limit equilibrium slope stability analyses were conducted using Rocscience Slide2 under both static and seismic conditions, supported by kinematic assessments and sensitivity assessments.

Key findings include:

- ✧ Geological Conditions: The borrow pit comprises overburden fill and BCM sandstone overlying granitic basement rock of the Karamea Suite. UCS and GSI data indicate increasing rock strength with depth.
- ✧ Slope Stability: Highwalls are stable under static conditions across all configurations. Under seismic loading, slopes generally meet design acceptance criteria, though shallow failures (1–2 m deep) in fill material may occur during 1-in-250 and 1-in-1000-year seismic events.
- ✧ Kinematic Risk: Wedge failure is identified as the primary structural risk, particularly on the southern highwall, driven by intersecting joint sets.
- ✧ Sensitivity Analysis: Slope yield is expected at a peak ground acceleration (PGA) of approximately 0.63 g. Displacement modelling indicates negligible movement (<0.1 cm), suggesting acceptable performance under extreme seismic conditions.
- ✧ Residual Risks: Potential inundation from a nearby coal fines pond, rockfall hazards from fill near highwall crests, and the need for updated seismic hazard modelling were identified. Mitigation measures and monitoring strategies are recommended.

No fatal flaws were identified that would preclude the proposed expansion of the A-Drive Borrow Pit and the future proposed CFS Stage 2 and 3.

Continued structural mapping, targeted investigations, and proactive risk management will be essential to support safe borrow pit development and future use of the site as a coal fines storage facility.

Table of Contents

SECTION	PAGE
Executive Summary	ii
1.0 Introduction	1
1.1 Background	1
1.2 Scope of works	3
2.0 Previous Reports and Supplied Data	3
2.1 Previous Reports	3
2.2 Supplied Information	4
3.0 Geology	4
3.1 Brunner Coal Measures (BCM) – (Eocene)	4
3.2 Karamea Suite - (Devonian to Cretaceous)	4
3.3 Greenland Group - (Late Cambrian – Early Ordovician)	5
4.0 Drilling Investigations	5
5.0 Downhole Geophysics	6
6.0 Regional Seismicity and Seismic Hazard at Stockton Mine	7
7.0 Basis of Design	8
7.1 Background	8
7.2 Laboratory Testing Review	9
7.3 Discontinuity Analysis	11
7.4 Faults	12
7.5 Kinematics	12
7.6 Kinematic Assessment Summary	14
8.0 Site Walkover/Observations	15
8.1 General	15
8.2 Historical Aerial Imagery Review	15
8.3 Existing Pit Condition	15
8.4 Surface Water and Groundwater	16
9.0 Rock Mass Parameters	16
9.1 Geological Strength Index (GSI)	16
9.2 Blast Disturbance Factor (D)	19
9.3 Estimated Blast Damage Zone	20
9.4 Rock Strength Criterion	20
10.0 Slope Stability Analysis	22
10.1 Introduction	22
10.2 Cross Sections	22

10.3	Design Acceptance Criteria	24
11.0	Slope Stability Results	25
12.0	Sensitivity Analysis	28
13.0	Geotechnical Summary / Residual Risks	30
14.0	Conclusions	32
15.0	References	32

Table of Figures

Figure 1: The current footprint and proposed expansion extent of the A-Drive Borrow Pit indicated by the wireframe overlay (Image Supplied by BRL).	2
Figure 2: BRL Supplied VULCAN image indicating the proposed locations of the final A-Drive Borrow Pit shell and CFS stages 2 and 3.	2
Figure 3: Visual distribution of UCS of Intact granite rock in the A-Drive Borrow Pit	10
Figure 4: A-Drive Borrow Pit UCS data split into a shallow and deep grouping.	11
Figure 5: Results of the Geological Strength Index (GSI) determination from Hoek and Marinos (2000) of basement rock in A-Drive Borrow Pit.	18
Figure 6: Example of cross-section through the western highwall of the A-Drive Borrow Pit, lithologies and rock strength parameters are indicated in the table. The contact between the fill and BCM is inferred to represent the BCM coal floor.	23
Figure 7: Estimated yield peak ground acceleration for failures in fill materials - Factor of Safety 1.0, Horizontal Seismic Coefficient 0.34g.	28
Figure 8: Sensitivity analysis of seismic coefficient for overall slope failure – Factor of Safety 1.0, Horizontal Seismic Coefficient 0.63g.	29

Table of Tables

Table 1: Summary of Drillholes in Close Proximity to A-Drive Borrow Pit Footprint.	5
Table 2: Approximate A Drive Highwall Orientations	8
Table 3: Approximate CFS stages 2 and 3 Highwall Orientations	8
Table 4: Basis of Design (BOD) A-Drive Borrow Pit from Tesoro (2012)	9
Table 5: Summary of Laboratory Testing Basement – Karamea Suite	10
Table 6: Summary of Discontinuities from Phillips, 2010.	11
Table 7: Joint Sets Identified by Phillips, 2010	12
Table 8: Summary Kinematic Analysis (Phillips 2010)	13
Table 9: Joint Set Identified on the Southern Highwall (Teroso 2012)	14
Table 10: Blast Disturbance (D) Values from Hoek 2012	19
Table 11: Geotechnical Rock Strength Parameters	21
Table 12: Design Acceptance Criteria for Slope Stability Analysis	24
Table 13: Slope Stability Results – Proposed Highwall Configurations A-Drive Borrow Pit	25
Table 14: Slope Stability Results – CFS Stage 2 and 3	27
Table 15: Seismic Displacement Threshold – Bray and Travarasrou (2007)	29
Table 16: Geotechnical Risk and Data Gaps	31

Appendices

Appendix A: Site Plans
Appendix B: Site Photos
Appendix C: Historical Site Imagery (Google Earth)
Appendix D: Slope Stability Outputs
Appendix E: GSI Assessment From Core Photos
Appendix F: A-Drive Borrow Pit – UCS Test Results

1.0 Introduction

1.1 Background

Pattle Delamore Partners (PDP) Limited was engaged by Bathurst Resources Limited (BRL) to undertake a geotechnical assessment as part of the Buller Plateaux Continuation Project (BPCP) - Fast Track Application (FTA) for the proposed expansion of the A-Drive Borrow Pit (CFS Stage 1) and future proposed Stages 2 and 3 coal fines storage facilities (CFS).

Since 2013, A-Drive Borrow Pit has supplied granitic basement materials for roading and rehabilitation at the Stockton Mine. The borrow pit is currently estimated to contain approximately [REDACTED]. The proposed expansion involves a 60 m pushback of the northern highwall, extending the borrow pit towards the north-northeast (NNE).

Upon completion of excavation, the site is proposed to be repurposed for the storage of Coal Handling and Processing Plant (CHPP) tailings, with an estimated capacity of [REDACTED].

The proposed CFS stages 2 and 3 are conceptual and are estimated to have a combined excavation volume of approximately [REDACTED] (MBCM) inferred to be dominated by granite resource. It is understood that the excavated pits will be repurposed for coal fines storage and will have a combined storage capacity of approximately [REDACTED].

The locations and extent of A-Drive Borrow Pit are shown in Figures A1 to A5 (Appendix A) and the BRL supplied images in Figures 1 and 2.



Figure 1: The current footprint and proposed expansion extent of the A-Drive Borrow Pit indicated by the wireframe overlay (Image Supplied by BRL).

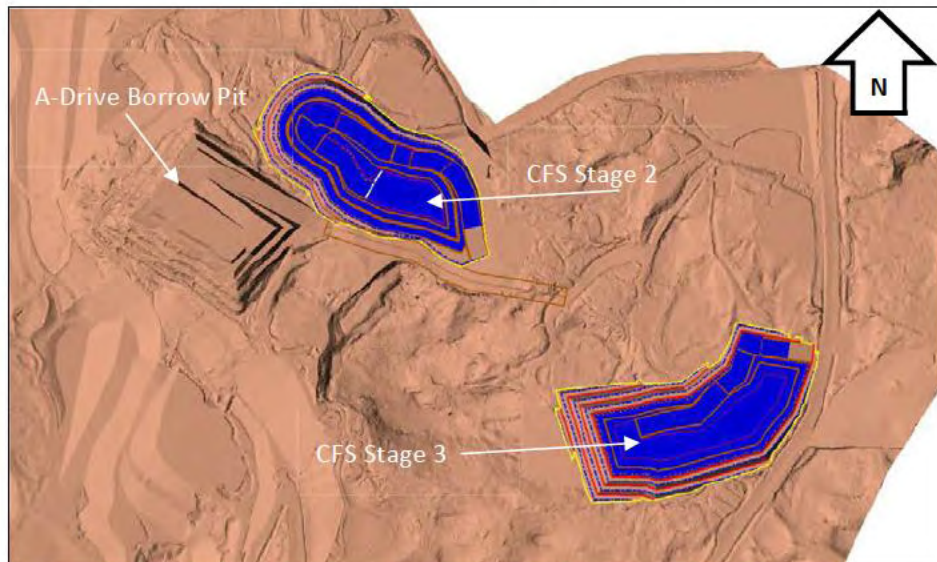


Figure 2: BRL Supplied VULCAN image indicating the proposed locations of the final A-Drive Borrow Pit shell and CFS stages 2 and 3.

1.2 Scope of works

To support the geotechnical assessment of the A Drive Borrow Pit expansion and proposed CFS stages 2 and 3, the following scope of work (SoW) has been undertaken and is discussed in the following sections:

- ✧ **Site Walkover:** A field inspection was conducted to assess the condition of existing highwalls and bench cuts, and to map geological features in relation to the surrounding terrain.
- ✧ **Data Review & Basis of Design (BOD) Development:** Existing geotechnical reports, drillhole data, geological models, and relevant Stockton Mine datasets were reviewed to develop a ground model and define geotechnical rock strength parameters for slope stability analysis.
- ✧ **Slope Stability Analysis:** Stability of both existing and proposed highwalls was assessed using Rocscience SLIDE2, considering static seismic loading conditions and, worst credible groundwater.
- ✧ **Geotechnical Reporting:** Findings were compiled into a comprehensive report, including design recommendations, a gap analysis, a risk register, and commentary on the suitability of the borrow pit for future CHPP tailings storage.

2.0 Previous Reports and Supplied Data

A review of the reports and data provided by BTM has been completed to support the development of a geotechnical model and the determination of strength parameters for use in the slope stability assessment of the A-Drive borrow pit highwalls. The geotechnical model will serve to characterize subsurface conditions, inform design assumptions, and guide the evaluation of potential failure mechanisms to ensure safe and effective borrow pit operations.

2.1 Previous Reports

The following reports were supplied by BTM:

- ✧ Geotechnical Factual Report - Results of A-Drive Quarry investigation, Stockton Mine. Stockton Alliance, (Phillips, A, December 2010).
- ✧ Geotechnical Report – A-Drive Quarry Development, Stockton Mine. Stockton Alliance, (Tesoro, Julius, September 2012).
- ✧ Tulia, Kurtis (2020). Investigation on the Suitability of Granitic Rock Aggregate for Mine Haul Roads, Stockton, New Zealand. Professional master's in engineering Geology Dissertations 2020, School of Earth & Environment, University of Canterbury, New Zealand (Tulia, Kurtis. 2020).
- ✧ TM#2 Seismic Hazard Assessment, Stockton Opencast Mine. URS Limited (2007).

2.2 Supplied Information

The following data and information were also supplied by BRL:

- ✧ BRL rock test data Excel spreadsheet for A-Drive Borrow pit
- ✧ Drillhole core photographs
- ✧ A-Drive Borrow pit and CFS stages 2 and 3- Vulcan Files - Triangulations

The results of these reviews are discussed in Sections 4.0 to 7.0.

3.0 Geology

The geological units anticipated in the vicinity of the A-Drive Borrow Pit and CFS stages 2 and 3 are based on published data from the 1:50,000 GNS Science geological map *Geology of the Buller Coalfield*, as well as subsurface investigations conducted by Phillips (2010) and Tulia (2020). The mapped geology in the vicinity of the A-Drive Borrow Pit and proposed CFS stages 2 and 3 are indicated on Figure A2 (Appendix A). The GNS map indicates that the site is underlain by the Brunner Coal measures (BCM). The BCM at the location of A- Drive Borrow Pit has been mined out exposing the underlying basement rock, inferred to be primarily comprised of Karamea Suite granites with lesser entrained Greenland Group greywacke. The geological units expected to be encountered in the vicinity of A-Drive Borrow Pit are discussed in more detail below.

3.1 Brunner Coal Measures (BCM) – (Eocene)

The Brunner Coal Measures (BCM) typically comprise pebble conglomerates, high-carbonaceous mudstones, coarse sandstones, and interbedded coal seams. A distinct, thin conglomerate layer commonly marks the unconformity between the BCM and the underlying basement lithologies. Within the mudstone units, bedding plane shears are frequently observed, indicating zones of structural weakness and deformation that may influence both slope stability and groundwater movement.

3.2 Karamea Suite - (Devonian to Cretaceous)

The Karamea Suite granites show notable compositional variability. At the A-Drive Borrow Pit, the basement is mapped as Berlins Quartz Porphyry (BQP), characterized by large anhedral quartz phenocrysts, biotite, thin pegmatite veins, and hornfels lenses—interpreted as entrained Greenland Group metasediments. With increasing mafic content, BQP grades into granodiorite. The upper 8–25 m is typically weathered, with orthoclase feldspar altered to kaolinite, likely due to hydrothermal processes.

3.3 Greenland Group - (Late Cambrian – Early Ordovician)

Geological maps of the Buller Coalfield show that the Greenland Group basement rocks mainly consist of unaltered greywacke and argillite. However, near the northern margin of the Karamea granitic batholith, these metasedimentary units undergo progressive thermal alteration due to contact metamorphism. This has transformed the original lithologies into biotite-rich hornfels, reflecting elevated temperatures from the intrusive event. Alteration intensifies toward the intrusion, forming a distinct thermal aureole. Near the A-Drive Borrow Pit, the Greenland Group typically appears as xenoliths entrained within the Karamea Suite granitic rocks.

4.0 Drilling Investigations

Drilling investigations by Phillips (2010) and Tulia (2020) were conducted to assess subsurface conditions near the A-Drive Borrow Pit. Drillhole depths ranged from approximately 32.7 to 50.3 m below ground level (BGL). From these investigations, fifteen (15) drillholes within the borrow pit footprint were selected to support rock mass characterization for the slope stability assessment. Of these drillholes, nine (9) included downhole geophysical surveys. A summary of the drillholes used in this analysis are presented in Table 1 and the locations indicated on Figure A4 (Appendix A).

Table 1: Summary of Drillholes in Close Proximity to A-Drive Borrow Pit Footprint.

Drillhole ID	Coordinates (NZTM)		Drill Collar RL (m)	Drilled Depth (m bgl)	Drillhole Type	Downhole Geophysical Testing (Y/N)
	Easting	Northing				
BH4453 ¹	1505322.55	5385262.64	876.02	50.3	Geotechnical	Y
BH4455 ¹	1505151.43	5385166.11	909.08	49.3	Geotechnical	Y
BH4456 ¹	1505117.15	5385236.12	906.64	32.7	Geotechnical	Y
BH4457 ¹	1505209.72	5385217.46	894.53	50.2	Geotechnical	Y
BH4458 ¹	1505415.75	5385126.56	887.14	50.2	Geotechnical	N
BH4269 ¹	1505162.70	5385351.86	929.99	69.8	Exploration	Y
BH4270 ¹	1505138.29	5385401.55	922.97	62.8	Exploration	Y
BH4271 ¹	1505292.40	5385399.19	903.354	56.1	Exploration	Y
BH4277 ¹	1505232.85	5385396.51	912.418	59.3	Exploration	Y
BH4285 ¹	1505259.64	5385366.95	910.667	62.1	Exploration	N

Table 1: Summary of Drillholes in Close Proximity to A-Drive Borrow Pit Footprint.

Drillhole ID	Coordinates (NZTM)		Drill Collar RL (m)	Drilled Depth (m bgl)	Drillhole Type	Downhole Geophysical Testing (Y/N)
	Easting	Northing				
BH4286 ¹	1505178.06	5385603.17	906.057	59.2	Exploration	Y
BH4307 ¹	1505189.32	5385380.79	914.879	61.0	Exploration	N
BH7090 ²	1505263.49	5385288.76	880	44.8	Exploration	N
BH7091 ²	1505329.17	5385343.38	875	45.8	Exploration	N
BH7092 ²	1505341.30	5385256.02	862	45.8	Exploration	N

Notes:

1. Data source: Phillips, Alex (2010). *Geotechnical Factual Report - Results of A-Drive Quarry investigation, Stockton Mine. Stockton Alliance, December 2010. Coordinates from Phillips 2010 have been converted from Buller 1949 to NZTM. Tulia, Kurtis (2020). Investigation on the Suitability of Granitic Rock Aggregate for Mine Haul Roads, Stockton, New Zealand. Professional master's in engineering Geology Dissertations 2020, School of Earth & Environment, University of Canterbury, New Zealand. Drillhole locations estimated from Figure 3 and drill collar Reduced Levels (RL) estimated from Figure 9 (Tulia, 2020).*
2. Coordinates of the Drillhole locations have been New Zealand Trans Mercator (NZTM) from Buller Circuit 1949 using the Land Information New Zealand coordinate conversion tool.

5.0 Downhole Geophysics

As discussed in Section 4.0, nine of the fifteen drillholes undertaken within the A-Drive Borrow Pit footprint underwent downhole geophysical testing, conducted by Weatherford International. The purpose of this testing was to obtain detailed subsurface data to support rock mass characterisation and inform the slope stability assessment. The testing programme included the following:

- ✧ Dummy Calliper: Used to confirm borehole integrity and ensure tool clearance.
- ✧ Verticality: Measured borehole deviation from vertical to assess drilling accuracy.
- ✧ Acoustic/Dip/Fracture Log: Provided high-resolution imaging of borehole walls for structural and fracture analysis.
- ✧ Gamma, Long and Short Spaced Density (in rods): Acquired radiometric and density data through drill rods.
- ✧ Gamma, Long and Short Spaced Density (open hole): Collected enhanced lithological and density data in open borehole conditions.

The downhole geophysics has not been included in this analysis.

6.0 Regional Seismicity and Seismic Hazard at Stockton Mine

The Stockton Mine, located on the West Coast of New Zealand's South Island, lies within a tectonically active region shaped by the interaction of the Australian and Pacific plates. This area is subject to frequent seismic activity, primarily driven by major fault systems such as the Alpine Fault and its associated structures. As a result, seismic hazard is a critical consideration for infrastructure and geotechnical design at the mine.

Several seismic hazard assessments have been undertaken for Stockton Mine:

- ✧ The URS (2007) study provided both probabilistic and deterministic estimates of Peak Ground Acceleration (PGA) for a Class A (rock) site:
- ✧ Probabilistic PGAs ranged from 0.24g (150-year return period) to 0.82g (10,000-year event).
- ✧ Magnitude-weighted values were slightly lower, reflecting reduced damage potential from smaller earthquakes.
- ✧ Deterministic PGAs, based on nearby faults such as Mt William and Kongahu, reached up to 0.79g.
- ✧ For design purposes, probabilistic PGAs were recommended for low to medium hazard structures, while deterministic PGAs were advised for high hazard structures to ensure seismic loads reflect fault proximity and potential impact.
- ✧ Earlier models by Stafford (2003) and Mould (2004) estimated: Operating Basis Earthquake (OBE) PGA values between 0.24–0.32g and Maximum Considered Earthquake (MCE) PGA values up to 0.97g, later factored down to 0.21g (OBE) and 0.65g (MCE).
- ✧ These assessments are now considered outdated following the release of the updated National Seismic Hazard Model (NSHM) in October 2022. Current seismic hazard estimates are based on updated guidance from Ministry of Business, Innovation, and Employment (MBIE). New Zealand Geotechnical Society. November 2021. Earthquake Geotechnical Engineering Practice – Module 1: Overview of the Guidelines.

Globally, highwalls in seismically active regions typically experience only minor failures (e.g., shallow slides or rock falls) during strong shaking, which results in only minor disruption to mining operations.

7.0 Basis of Design

7.1 Background

The A-Drive Borrow Pit has four primary highwall orientations. The proposed highwall orientations are summarised in Table 2. The basis of design (BOD) for pit highwall slope profiles is informed by existing slopes within the A-Drive Borrow Pit and operational experience from other pits across the Stockton Mine site excavated into Brunner Coal Measures (BCM) and basement rocks. The BOD establishes the design assumptions and parameters used to guide slope geometry, stability analysis, and risk management. The cut geometries presented in Table 4 are based off the descriptions outlined in Teroso (2012) which are consistent with the site observations made during the site visit. The proposed CFS stages 2 and 3 comprised two elongate pits, the approximate strike and dip direction of the CFS stages 2 and 3 highwalls is summarised in Table 3. CFS Stages 2 and 3 have been designed using the same BOD parameters used for the A-Drive borrow pit.

Table 2: Approximate A Drive Highwall Orientations

Highwall	Strike (°)	Dip Direction (°)
Northern	307	217
Eastern	035	125
Southern	127	037
Western	215	125

Table 3: Approximate CFS stages 2 and 3 Highwall Orientations

CFS Stage 2	Strike (°)	Dip Direction (°)
Northern	120	210
Eastern	170	260
Southern	120	030
Western	215	125
CFS Stage 3		
Northern (1)	082	172
Northern (2)	050	140
Northern (3)	100	190

Table 3: Approximate CFS stages 2 and 3 Highwall Orientations

CFS Stage 2	Strike (°)	Dip Direction (°)
Eastern	020	290
Southern	080	170
Western	160	070

Table 4: Basis of Design (BOD) A-Drive Borrow Pit from Tesoro (2012)

Borrow Pit Element	Overall Slope (°) ¹	Bench Height (m) ²	Bench Width (m) ¹	Batter Angle (°) ¹
Pit highwalls	42°	10 -20	8.5 – 9.5	63-67
Notes: 1. A-Drive Basis of Design from Tesoro, 2012. 2. Bench heights estimated from cross sections.				

7.2 Laboratory Testing Review

Previous investigations at the A-Drive Borrow Pit by Phillips (2010) and Tulia (2020) included laboratory testing to determine the bulk density and unconfined compressive strength (UCS) of the granitic rock. Bulk density provides insight into the material's weight and its response to loading, while UCS measures the strength of intact rock under axial compression.

Phillips (2010) reported that weathering of the basement rock ranges from fresh, un-weathered (UW) and unaltered rock to highly weathered (HW) material, generally separated by with varying amounts of slightly weathered (SW) and moderately weathered (MW) granite, which retains its original fabric but exhibits reduced strength. A spatial trend is evident across the A-Drive site: eastern boreholes show shallower transitions to fresh rock (3–8 m bgl), while western boreholes transition at greater depths (20–30 m bgl). Core strength follows a similar pattern, with harder rock encountered at 5–18 m bgl in the east and 17–30 m bgl in the west.

A summary of the laboratory test results (bulk density and UCS), along with weathering classifications, is presented in Table 5. The distribution of UCS data is shown in Figure 3.

Table 5: Summary of Laboratory Testing Basement – Karamea Suite

Laboratory Test	Geotechnical Domain	No. of Samples	Min	Max	Mean
Bulk Density (t/m ³)	HW - granite	4	2.02	2.53	2.31
	SW - granite	7	1.41	2.57	2.25
	UW - granite	13	1.88	2.57	2.30
Unconfined Compressive Strength (UCS) MPa	HW - granite	4	2.41	34.4	15.7
	MW - granite	8	16	55	34.75
	SW - granite	11	4.99	82	36
	UW - granite	26	49.6	152.2	110

Notes:
1. Data source: Phillips, (2010). and Tulia, Kurtis (2020).

The A-Drive Borrow Pit Unconfined Compressive Strength (UCS) data plotted in Figure 3 indicate an average UCS of 75 MPa.

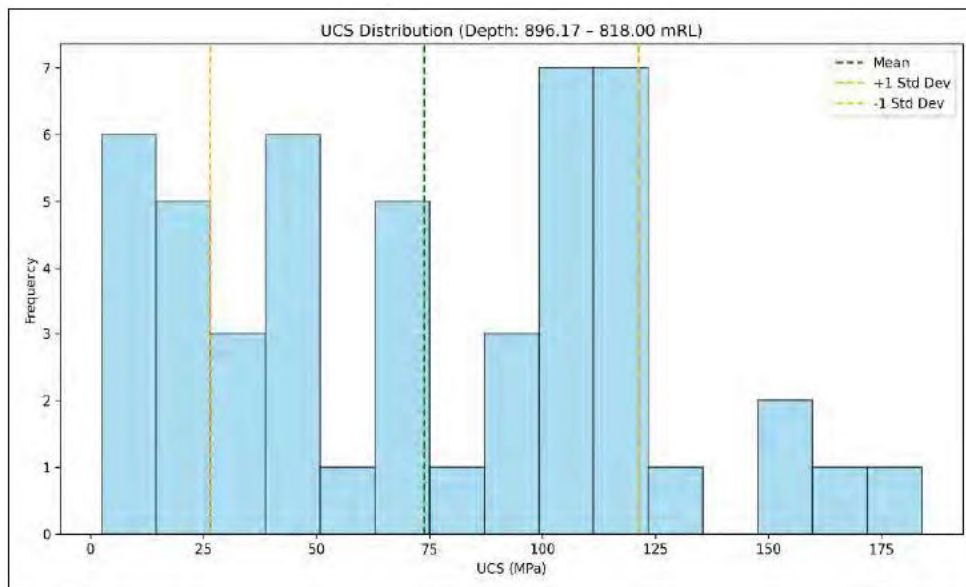


Figure 3: Visual distribution of UCS of Intact granite rock in the A-Drive Borrow Pit

Further analysis of the granitic basement at A-Drive Borrow Pit was conducted by dividing the UCS data into shallow and deep groupings (Figure 4). The shallow group has a mean UCS value of 46 MPa for intact rock, while the deep group averages 98 MPa, indicating an increase in rock strength with depth. To account

for the large variability in the data, a conservative UCS value of 50 MPa has been adopted for the slope stability analysis.

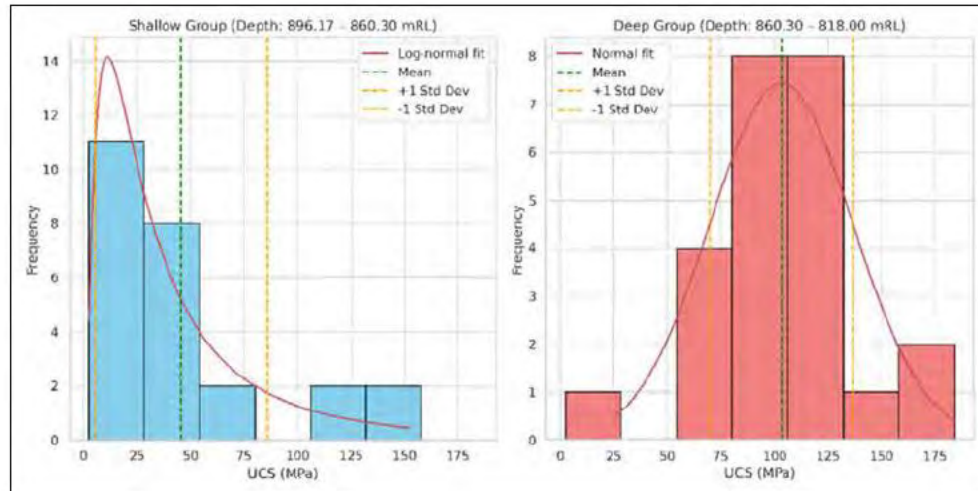


Figure 4: A-Drive Borrow Pit UCS data split into a shallow and deep grouping.

7.3 Discontinuity Analysis

Discontinuity analysis involves evaluating the orientation, spacing, and physical characteristics of bedding, joints, fractures etc within rock masses. These features significantly influence rock mass behaviour and are critical for assessing slope stability and informing geotechnical design. A summary of joint-related discontinuity data from Phillips (2010) is presented in Table 6.

Table 6: Summary of Discontinuities from Phillips, 2010.	
Feature	Summary
Joint Surface	Predominantly rough (72%) and planar (77%); some smooth and irregular joints also observed.
Aperture	99% of joints have apertures between 1–10 mm; tighter joints 1mm-0mm may be under reported.
Joint Fill	96% of joints are either clean or mineralised.
Water Flow Evidence	8% of joints show mineralisation or staining, indicating possible water flow.
Notes: 1. Source: Phillips, Alex (2010). Geotechnical Factual Report - Results of A-Drive Quarry investigation, Stockton Mine. Stockton Alliance, December 2010.	

7.4 Faults

Faults and shear zones significantly affect highwall stability by creating planes of weakness that can lead to planar, wedge, or toppling failures, their orientation, persistence, and interaction with joint sets are key to slope design. Geotechnical logging of core samples by Phillips (2010) identified three potential faults within the A-Drive Borrow Pit, observed in drillholes BH4453, BH4455, and BH4275. However, no evidence of these faults was observed in the A-Drive pit during the site walkover. The dataset was too limited to establish any regional fault trends. Mapped faultlines in the vicinity of the A-Drive Borrow Pit are indicated on Figure A3 (Appendix A). It should be noted that eastern highwall of CFS stage 3 intercepts a mapped fault trace, Figure A5 (Appendix A).

7.5 Kinematics

Kinematic assessments of the A-Drive Borrow Pit were conducted by Phillips (2010) and Teroso (2012). The following section summarises the key findings from these kinematic assessments. These assessments have been undertaken only on the A-Drive borrow pit and have not been applied to CFS Stages 2 and 3.

7.5.1 Phillips (2010)

Phillips (2010) performed a bench-scale stability assessment using data from downhole geophysical surveys, geotechnical logs, and acoustic televiewer imagery. Analysis was conducted using Rocscience Dips v4, with statistical contouring of defect orientations. Four joint sets were identified and are summarised in Table 7. The assessment indicated potential planar, wedge, and toppling failure modes across multiple borrow pit walls.

Table 7: Joint Sets Identified by Phillips, 2010

Set ID	Average Strike (°)	Average Dip (°)	Average Dip Direction (°)
Set 1	304	72	034
Set 2	152	61	242
Set 3	323	31	053
Set 4	189	60	099

Notes:

- Source: Phillips, Alex (2010). *Geotechnical Factual Report - Results of A-Drive Quarry investigation, Stockton Mine. Stockton Alliance, December 2010*

A summary of the key failure orientations identified by Phillips (2010) are presented in Table 8.

Table 8: Summary Kinematic Analysis (Phillips 2010)			
Borrow Pit Wall	Planar Failure	Toppling Failure	Wedge Failure
North-East Wall	Defects with poles falling in and around the western edge of Set 2 (61/242) have the potential to cause plane failure	Defects falling in and steeper than Set 3 (31/053) have the potential to cause Toppling Failure. Set 1 (72/034) lies just outside the failure envelope.	Intersections of any plane with an orientation between Set 4 (60/099) and approximately 60/120 have the potential to cause wedge failure.
East Wall	Defects with poles falling in and around the western edge of Set 2 (61/242) and to the south of this have the potential to cause plane failure	Defects falling between Set 4 (60/099) and Set 3 (31/053) have the potential to cause Toppling Failure	Intersections of any plane with an orientation between 60/300) and Set 1 (72/034) has the potential to cause wedge failure.
Southwest Wall	Defects falling in and steeper than Set 3 (31/053) have the potential to cause Toppling Failure. Set 1 (72/034) lies just outside the failure envelope.	Defects with poles falling in and around the western edge of Set 2 (61/242) have the potential to cause plane failure.	Intersections of any plane with an orientation between 70/080 and 70/020, incorporating Sets 1-4 has the potential to cause wedge failure.
Notes: 1. Source: Phillips, Alex (2010). <i>Geotechnical Factual Report - Results of A-Drive Quarry investigation, Stockton Mine. Stockton Alliance, December 2010</i>			

7.5.2 Tesoro (2012)

Following blasting and excavation at the A-Drive Borrow Pit, several structural discontinuities were observed daylighting from the southern highwall slope. A laser scan of the first and second benches was conducted to determine the orientations of these features. Eight discontinuity sets were identified and designated JS1 to JS8, with JS1, JS2, and JS6 recognised as dominant. The average orientation of each joint set is presented in Table 9.

Table 9: Joint Set Identified on the Southern Highwall (Teroso 2012)

Joint Set	Average Strike (°)	Average Dip (°)	Average Dip Direction (°)
JS1	345	41	075
JS2	346	67	076
JS3	161	28	251
JS4	172	78	262
JS5	208	72	298
JS6	095	77	185
JS7	294	72	024
JS8	063	78	153

Notes:

1. Source: Tesoro, Julius (2012). *Geotechnical Report – A-Drive Quarry Development, Stockton Mine*. Stockton Alliance, September 2012.

The following summarises the recommendations from Teroso (2012):

- ✧ Based on its orientation (overall slope: 213/41; batter: 213/63), kinematic analysis indicates potential wedge failure due to intersections involving joint sets JS1, JS2, JS5, JS7, and JS8.
- ✧ Maintaining the current overall slope angle of 42° and a bench width of 8–8.5 m is recommended for optimal borrow pit performance.
- ✧ Increasing the batter angle to 65–67° slightly increases failure risk but improves rock recovery. At 68°, instability increases significantly due to additional joint intersections.
- ✧ Minor wedge failures remain possible even with reduced slope angles.
- ✧ Batter angles may be adjusted as excavation progresses, and more structural data becomes available.

7.6 Kinematic Assessment Summary

Phillips (2010) conducted a broad, site-wide kinematic assessment with limited structural resolution due to sparse surface exposure, concluding that while failure mechanisms are theoretically possible, observed instability is minimal. In contrast, Teroso (2012) provided a focused, data-rich analysis of the southern highwall, identifying specific joint sets contributing to wedge failure and recommending design refinements.

Both studies highlight wedge failure as the primary kinematic risk, driven by intersecting joint sets. Maintaining current slope angles and bench widths is advised, along with ongoing structural mapping to support safe and efficient Borrow Pit development.

8.0 Site Walkover/Observations

8.1 General

A site visit was undertaken by PDP on 1 July 2025 to inspect the condition of the existing borrow pit, site photos taken during inspection of A-Drive Borrow Pit are presented in Appendix B.

8.2 Historical Aerial Imagery Review

To supplement the site walkover and understand the evolution of the existing borrow pit a review of available satellite imagery available on Google Earth was completed and are presented in Appendix C. The following observations were made from the satellite imagery:

- ✧ Excavation of the A-Drive Borrow Pit began Circa 2011.
- ✧ An inferred coal fines storage pond is located due north of the current A-Drive Pit and is in the proximity of the proposed northern highwall extension. Discussions with BRL indicate coal fines storage pond is still located in this location. Based on previous site experience with coal fines ponds, the base of the pond is not expected to extend beyond the BCM coal floor. As the base of the BCM is visible within the first bench of the highwall cut it is our opinion, there is a potential risk of breaking into the coal fines pond during push back of the northern highwall. Caution is advised when extending the northern highwall to mitigate this risk. It is anticipated that the coal fines pond will be excavated as part of the CFS Stage 2 earthworks.
- ✧ Significant extraction of aggregate from A-Drive Borrow Pit was halted circa 2019 based on the formation of pit lake.
- ✧ May 2024: Satellite Imagery of A-Drive Borrow Pit shows a fully flooded borrow pit.

8.3 Existing Pit Condition

The A-Drive highwalls were observed to generally consist of a veneer of overburden fill, likely comprising BCM sandstone overlying in situ Buller Coal Measures (BCM) sandstone and granitic basement rock of the Karamea Suite. The geological units observed in the A-Drive borrow pit and highwalls are generally equivalent with the geological descriptions discussed in Section 3.0.

The southern, western, and eastern highwalls were observed to be in good condition, with no large-scale (overall slope) failures. The highwall faces are moderately to highly jointed, with evidence of previous bench-scale wedge block failures. The western and northern highwalls exhibit a higher degree of weathering than the southern and eastern walls, which is inferred to result from

their aspect, being more exposed to prevailing weather, and surface water runoff across these slopes. Extensive coal extraction has been undertaken in areas to the north and west of the A-Drive Pit. Site photos of the key features in the A-Drive Borrow Pit are presented in Appendix B. As discussed, earlier CFS Stages 2 and 3 are still conceptual and no pits exist currently.

8.4 Surface Water and Groundwater

During the site walkover, surface water runoff was observed entering the existing A-Drive pit, particularly at the northern and western corners of the borrow pit. The inflow was observed to enter the pit along the surface inferred as the BCM coal floor and the base of the overlying fill material. This was observed following several days of heavy rainfall at the site and is considered to represent the worst credible groundwater conditions at the site. No seeps or indications of seepage (e.g. algal growth) was observed on any of the highwalls.

The runoff is currently being redirected toward the lowest point of the A-Drive pit, resulting in the formation of the A-Drive Pit Lake. Based on recent survey data, the water level in the pit lake was estimated to be approximately 850 mRL. This estimate was consistent with observations made during the site walkover.

The lake level is not considered to represent the actual groundwater level at the site. However, anecdotal evidence indicates that lake water may be naturally draining from the pit through rock fractures, with preferential flow likely occurring toward the eastern end of the pit.

9.0 Rock Mass Parameters

The rock mass strength parameters used in the slope stability analysis were derived using a combination of methods: the Geological Strength Index (GSI) approach (Hoek and Marinos, 2000), laboratory-determined Unconfined Compressive Strength (UCS) values, and estimates of blast-induced disturbance (D factor) following guidelines from Hoek (2012). The following sections outline each of these components and explain how they were integrated to approximate the rock mass properties of the granitic basement at the A-Drive Borrow Pit. It should be noted that the rock mass parameters derived for the A-Drive Borrow Pit analysis have been adopted at this conceptual stage to represent the highwalls for the CFS Stage 2 and 3 Pits.

9.1 Geological Strength Index (GSI)

GSI is a practical system for estimating rock mass strength based on its structure and surface condition. Developed by Hoek and Marinos (2000), it combines observations of blockiness and joint quality to support design calculations, especially when used with the Hoek-Brown failure criterion in slope, tunnel, and foundation analysis.

GSI is influenced by two main factors:

- ✧ Structure of the rock mass – This refers to the degree of fracturing, blockiness, and interlocking of the rock. Intact, massive rock masses have higher GSI values, while heavily fractured or blocky masses have lower values.
- ✧ Condition of discontinuity surfaces – This includes the roughness, weathering, and infilling of joints or fractures. Clean, rough, and unweathered surfaces contribute to higher GSI values, whereas smooth, weathered, or infilled joints reduce the GSI.

Together, these factors are assessed visually or through field mapping to assign a GSI value, which is then used to estimate rock mass strength and deformation properties in engineering design.

AGSI assessment has been carried out to estimate the rock mass strength of the granitic basement. This assessment was informed by site observations of the current highwall conditions and core photographs from the Phillips (2010) investigation presented in Appendix E. The results, along with a comparison to the original Phillips (2010) assessment, are presented in Figure 5.

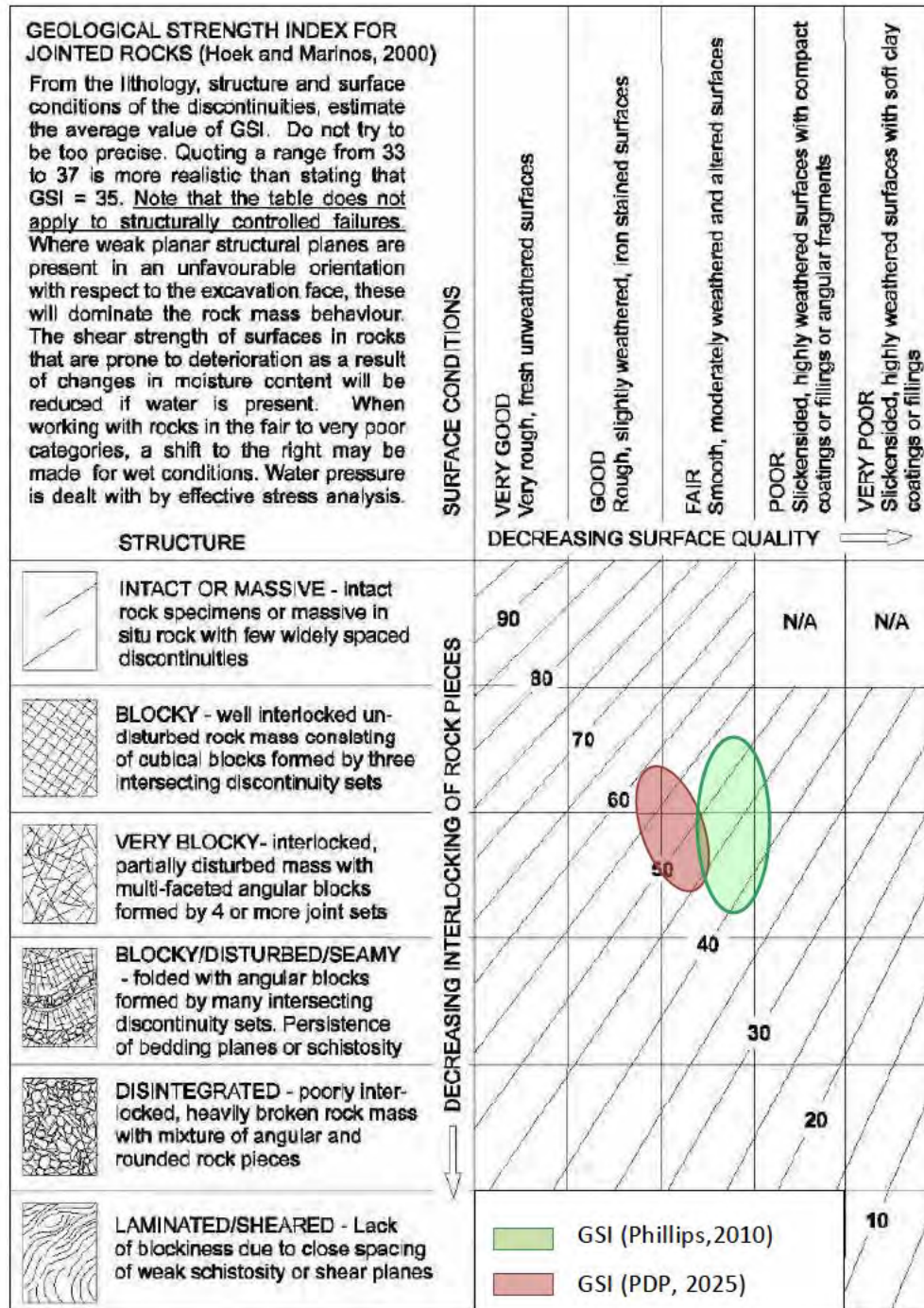


Figure 5: Results of the Geological Strength Index (GSI) determination from Hoek and Marinos (2000) of basement rock in A-Drive Borrow Pit.

Based on the GSI assessment a GSI value of 50 indicating a blocky to very blocky rock mass with joints in are fair to good condition has been adopted in the rock strength classification to be used in the slope stability analyses.

9.2 Blast Disturbance Factor (D)

Blast-induced disturbance can significantly reduce the strength and integrity of a rock mass, thereby affecting slope stability. The D factor accounts for this degradation by modifying the input parameters in the Hoek-Brown failure criterion, typically resulting in lower estimated rock mass strength and increased deformation potential. Higher D values (indicating greater disturbance) lead to more conservative stability assessments, as they reflect reduced cohesion and friction within the rock mass. Accurately estimating the D factor is therefore critical in borrow pit environments, where blasting is common, to ensure safe and reliable slope design. Table 10 provides a summary modified from Hoek, 2012 and illustrates common ranges of blast disturbance factor relative to visible damage and the quality of the excavation method.

Table 10: Blast Disturbance (D) Values from Hoek 2012		
Excavation	Typical Rock Mass Condition	Suggested D Value
Controlled Blasting	Minor damage, good joint preservation	0.2-0.4
Production Blasting	Moderate damage, visible fracturing near blasting surfaces	0.4-0.7
Poor Blast Practice	Extensive damage, loose block, poor joint condition	0.7-1.0
Notes: 1. Source: Hoek, E. (2012). Blast Damage Factor D. Technical note for RocNews – February 2, 2012.		

Discussions with BRL indicate that pre-split blasting was utilised on some of the A-Drive borrow pit benches. Pre-split blasting is a controlled blasting technique used to create a clean, stable final rock face and to limit blast-induced damage beyond the design excavation line. However, the specific benches constructed using this technique could not be confirmed.

Highly fractured rock was observed in the southern highwall (third bench) of A-Drive pit during the site walkover. For the purpose of this assessment a conservative blast damage factor of $D = 1.0$ has been adopted, in accordance with Hoek (2012), this is considered to be the worst-case scenario and has been applied to the whole slope in the slope stability analyses.

9.3 Estimated Blast Damage Zone

According to Hoek (2012), blast damage zone estimation involves applying a disturbance factor (D) to account for the reduction in rock mass quality due to blasting. The factor ranges from 0 (undisturbed) to 1 (heavily disturbed) and is used to adjust the Geological Strength Index (GSI) and rock mass parameters in the Hoek-Brown failure criterion. The depth of the blast damage zone is typically estimated empirically, considering factors like bench height, explosive energy, and confinement, and is critical for accurate slope stability analysis in blasted rock environments.

9.4 Rock Strength Criterion

A rock strength criterion for the A-Drive Borrow Pit rock mass has been determined using parameters previously applied at Stockton Mine for BCM sandstone and fill materials generally comprising BCM sandstone and mudstone overburden, as supplied by BTM. The strength criterion for the granitic basement has been developed using unconfined compressive strength (UCS) data (Section 7.1) and the Geological Strength Index (GSI) (Section 9.1).

For the rock mass, the Generalised Hoek-Brown failure criterion is considered suitable for slope stability analysis, particularly for highwall design and stormwater sump excavations. In contrast, Mohr-Coulomb drained strength parameters are deemed more appropriate for fill materials.

The selected strength criteria and associated parameters for each geotechnical unit are presented in Table 11.

Table 11: Geotechnical Rock Strength Parameters

Formation	Rock Type	Strength Criterion Model	Generalised Hoek-Brown Parameters						Mohr Coulomb Parameters			
			UCS (MPa)	mb	s	a	GSI	mi	D	Bulk Density (kN/m³)	Cohesion, c' (kPa)	Friction Angle, ϕ (°)
ELF Fill	Fill (Sandstone)	Mohr Coulomb	-	-	-	-	-	-	-	18	2	39
Brunner Coal Measures (BCM)	Insitu Sandstone	Generalised Hoek Brown	30	1.39544	0.00041894	0.522344	-	-	-	24.76	-	-
Karamea Suite	Granite	Generalised Hoek Brown	50	-	-	-	50	32	1	23	-	-

Notes:

- Generalised Hoek-Brown parameters supplied by BTM based on similar lithologies at the Stockton Mine Site – MFS south geology and rock parameters memo. The parameters (mb, s, a) supplied are based on GSI (Geological Strength Index) and Disturbance factors (D) not provided. PDP undertook a back analysis using Rocscience proprietary software RocLab and derived similar parameters using the UCS given, D = 1.
Mohr-Coulomb strength parameters supplied by BTM.
- Basement rock parameters have been inferred using UCS data supplied by BTM. The GSI (Geological Strength Index) and Disturbance factors (D) are discussed in Section 9.0. Effective stress parameters are (drained) for cohesion and friction angle

10.0 Slope Stability Analysis

10.1 Introduction

The following section presents the results of the slope stability analysis conducted using Rocscience Slide2 (Version 9.038). Deterministic analyses were performed using the Morgenstern-Price and Spencer methods to evaluate the global minimum Factor of Safety (FoS) for the overall slope, individual benches, and multiple bench configurations for each cross-section profile. The approximate locations of the cross sections are present on Figure A4 and A5 in Appendix A. These analyses were carried out under both static and seismic conditions to assess highwall performance. The groundwater condition used in the model is representative of the worst credible groundwater scenario as discussed in Section 8.4.

10.2 Cross Sections

A geological model of the A-Drive Borrow Pit was developed using the VULCAN triangulation. The triangulation surfaces included the existing topography, coal floor and top of basement. Six cross-sections were cut, one through each highwall (north, east south, west) of the A-Drive Borrow Pit and a representative cross section through the largest highwall of CFS stages 2 and 3. The sections are to be used to assess the slope stability of each of the pits.

Due to a lack of borehole data in the vicinity of CFS stages 2 and 3 the BT Mining supplied geological triangulations have been used to infer the thickness of the underlying stratigraphy at the locations of CFS Stages 2 and 3. It is expected geology at the highwall locations of CFS Stages 2 and 3 will be confirmed in future drilling rounds. An example of the cross section is shown in Figure 6. The results of the slope stability analyses are discussed in Section 11.0.

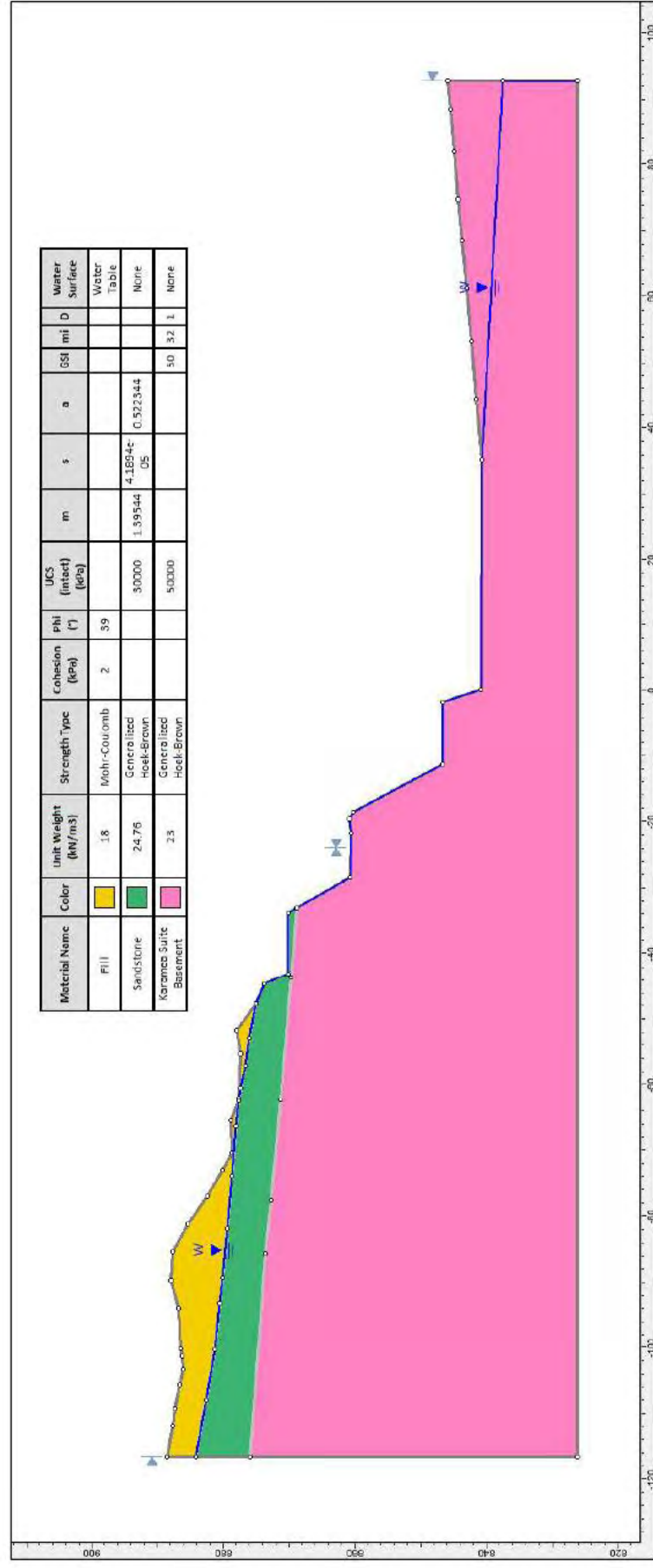


Figure 6: Example of cross-section through the western highwall of the A-Drive Borrow Pit, lithologies and rock strength parameters are indicated in the table. The contact between the fill and BCM is inferred to represent the BCM coal floor.

10.3 Design Acceptance Criteria

Design acceptance criteria for slope stability define minimum safety thresholds, typically factor of safety and probability of failure, based on slope scale and potential consequences, guiding risk-informed decisions. The design acceptance criteria to be used in the slope stability analysis of the A-Drive Borrow Pit and CFS Stages 2 and 3 are presented in Table 12. The Peak Ground Accelerations for Westport (ID 81) from Table A1 of Ministry of Business, Innovation, and Employment (MBIE). New Zealand Geotechnical Society. November 2021. Earthquake Geotechnical Engineering Practice – Module 1: Overview of the Guidelines, have been used in these analyses as they are considered to be comparable to the values expressed for Operating Basis Earthquake (OBE) and Maximum Considered Earthquake (MCE) expressed in the Stockton seismic model discussed in Section 6.0.

Table 12: Design Acceptance Criteria for Slope Stability Analysis		
Design Life (Final Landform)		100 years
Static Loading DAC	Bench	FoS ≥ 1.3
	Multiple Bench	FoS ≥ 1.2
	Overall Slope	FoS ≥ 1.2
Seismic Loading ¹ DAC	Operating Basis Earthquake (OBE)	1:250-year (PGA 0.41g) FoS ≥ 1.0
	Maximum Considered Earthquake (MCE)	1:1000-year (PGA 0.72g) FoS ≥ 1.0
Notes: 1. Earthquake Magnitude (Mw) taken as 6.0 2. Peak Ground Accelerations derived from Table A1 - Ministry of Business, Innovation, and Employment (MBIE). New Zealand Geotechnical Society. November 2021. Earthquake Geotechnical Engineering Practice – Module 1: Overview of the Guidelines.		

11.0 Slope Stability Results

As discussed in Section 10.2, four cross-section profiles were developed, one through each of the A-Drive Borrow Pit highwalls under static and seismic conditions. A single representative cross section through the highest part of the highwalls has been undertaken on both CFS stages 2 and 3 pits. Each cross-section was analysed based on the proposed final highwall configurations using the geotechnical rock strength parameters outlined in Table 11 and assessed against the design acceptance criteria in Table 12. The results of these analyses are presented in Tables 13 and 14.

Table 13: Slope Stability Results – Proposed Highwall Configurations A-Drive Borrow Pit

Highwall	Slope Element	Factor of Safety (FoS) – Global Minimum	
		Static	Seismic
			1:1000 Year (PGA – 0.72g) (MCE)
Northern	Overall	3.5	1.3
	Bench 1	3.4	0.9
	Bench 2	3.6	1.2
	Bench 3	5.3	2.1
	Multiple Bench (1-2)	3.4	1.2
	Multiple Bench (2-3)	3.5	1.3
Eastern	Overall	3.3	1.0
	Bench 1	3.9	1.3
	Bench 2	3.7	1.3
	Bench 3	3.5	1.3
	Multiple Bench (1-2)	3.6	1.1
	Multiple Bench (2-3)	3.4	1.1
Western	Overall	4.0	1.4
	Bench 1	1.7	0.6
	Bench 1 (Shallow Failures in fill excluded)	5.5	1.9

Table 13: Slope Stability Results – Proposed Highwall Configurations A-Drive Borrow Pit

Highwall	Slope Element	Factor of Safety (FoS) – Global Minimum	
		Static	Seismic
			1:1000 Year (PGA – 0.72g) (MCE)
Highwall	Bench 2	3.7	1.4
	Bench 3	3.7	1.4
	Multiple (1-2)	1.7	0.6
	Multiple (1-2) (Shallow Failures in fill excluded)	4.3	1.4
	Multiple (2-3)	3.7	1.4
	Overall	3.3	1.1
Southern	Bench 1	5.2	2.0
	Bench 2	4.1	1.5
	Bench 3	3.8	1.4
	Bench 4	4.1	1.5
	Bench 5	4.9	1.9
	Multiple (1-2)	4.1	1.4
	Multiple (2-3)	3.5	1.2
Notes: 1. Factor(s) of Safety (FoS) are expressed as global minimum values. 2. Slope failure occurred as shallow failure in ELF fill.			

Table 14: Slope Stability Results – CFS Stage 2 and 3

Feature	Slope Element	Factor of Safety (FoS) – Global Minimum	
		Static	Seismic
			1:1000 Year (PGA – 0.72g) (MCE)
CFS Stage 2	Overall	3.0	1.0
	Bench 1 (granite and sandstone)	3.5	0.7²
	Bench 2 (granite)	3.6	1.2
	Multiple Bench 1 (granite and sandstone)	3.4	1.1
	Multiple Bench 2 (granite)	3.4	1.2
CFS Stage 3	Overall	2.7	0.9
	Bench 1 (granite and sandstone)	3.4	1.1
	Bench 2 (granite)	3.5	1.2
	Multiple Bench 1 (granite and sandstone)	3.1	1.0
	Multiple Bench 2 (granite)	3.4	1.2
Notes: 1. Factor(s) of Safety (FoS) values are expressed as global minimum. 2. Slope failure occurred as shallow failure (less than 0.5m depth in BCM sandstone).			

The highwall slopes of the A-drive borrow pit and CFS stages 2 and 3 are stable under static conditions for overall, bench, and multiple-bench configurations. Under MCE seismic loading, the slopes generally remain stable; with the exception of the overall slope profile of CFS Stage 3. Bench 1 of the Northern and western highwalls (A-Drive) and CFS stages 2. The analyses indicate shallow bench scale failures of less than 1.0m depth within the steep batter faces of the BCM sandstone and shallow failures of up to 0.75m depth within the fill material placed at the top of the western highwall (Figures D30 and D39 – Appendix D).

12.0 Sensitivity Analysis

This sensitivity analysis evaluates the stability of the overall highwall pit slope under varying seismic loading conditions. It aims to identify the peak ground acceleration (PGA) threshold at which slope failure may initiate, providing essential input for seismic design and risk mitigation.

Sensitivity analysis of the shallow failures in the fill material placed at the top of the highwalls are also likely to occur under seismic loadings exceeding a peak ground acceleration (PGA) of 0.34g (Figure 7), reprofiling or removing the fill slope back from the highwall crests will reduce the risk of rock fall during smaller seismic event.

Sensitivity analysis of seismic coefficient for the overall slope of the CFS Stage 3 highwall was undertaken. The results indicate of the analysis indicate a yield threshold of the slope is likely to occur at peak ground accelerations (PGA) exceeding 0.63g. The sensitivity analysis graph is presented in Figure 8.

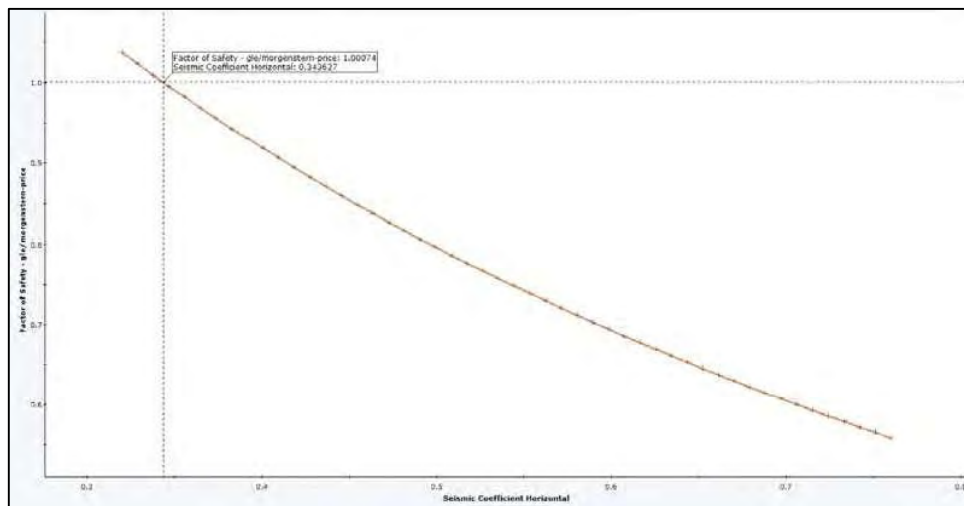


Figure 7: Estimated yield peak ground acceleration for failures in fill materials - Factor of Safety 1.0, Horizontal Seismic Coefficient 0.34g.

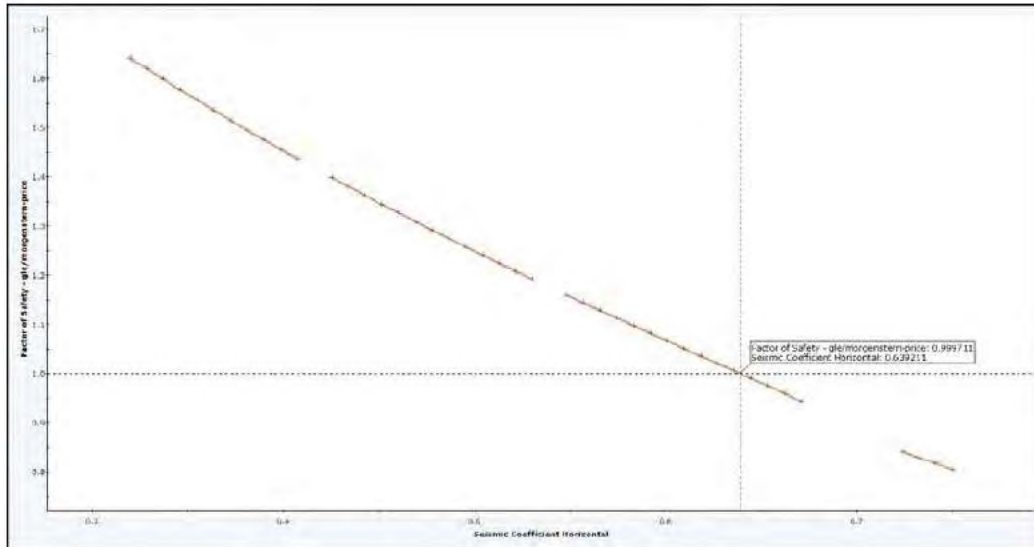


Figure 8: Sensitivity analysis of seismic coefficient for overall slope failure – Factor of Safety 1.0, Horizontal Seismic Coefficient 0.63g.

The yield-threshold PGA determined from the sensitivity analysis was used to estimate potential seismic-induced slope displacements (D) using a sliding block approach following Jibson (2007). The analysis indicates that displacements of less than 0.1 cm may occur during an MCE event. Displacements of this magnitude are considered stable based on the criteria outlined by Bray and Travasarou (2007) in Table 15. Accordingly, only minor bench-scale failures are expected in the highwalls of the A-Drive Borrow Pit and CFS Stages 2 and 3 during an MCE level earthquake.

Table 15: Seismic Displacement Threshold – Bray and Travasarou (2007)

Displacement (D)	Performance Level	Implication
$D < 1 \text{ cm}$	Stable	Negligible, no structural impact.
$1 \text{ cm} \leq D < 10 \text{ cm}$	Minor	Small permanent displacement, minor cracking or settlement.
$10 \text{ cm} \leq D < 100 \text{ cm}$	Moderate	Noticeable deformation, potential loss of functionality for infrastructure.
$D > 100 \text{ cm}$	Severe/Failure	Large displacements, slope failure likely, major damage to structures.

13.0 Geotechnical Summary / Residual Risks

During this geotechnical assessment, only small surficial failures of the A-Drive Borrow Pit or CFS stages 2 and 3 highwalls were indicated with no bench or global slope failures being indicated. Any surficial failures are expected to be contained within the proposed bench configurations and should not be problematic during expansion and or operation of the A-Drive borrow pit and CFS stages 2 and 3.

Even though only minor slope failures were indicated during this assessment several geotechnical data gaps were noted, these gaps are summarised in Table 16 along with recommended mitigation measures. These should be addressed in the next stages of design to support more detailed analysis and informed decision-making.

Table 16: Geotechnical Risk and Data Gaps

A-Drive Borrow Pit			
Identified Data Gap	Geotechnical Risk	Mitigation Measure	
Limited information regarding the coal fines dump (pond) identified in the satellite imagery located north of the northern highwall.	Inundation of A-Drive Borrow Pit During expansion of the northern highwall.	Determine the extents of the dump/pond and allow sufficient separation between the highwall and dump when designing Northern highwall. Assess the geotechnical properties of the coal fines if present.	
Rock Fall Risk -Highwalls and Fill.	Rock fall from highwall and fill areas.	Implement monitoring plan to periodically assess the highwall condition – for fill remove any fill material located at the crest of the highwall slopes and implement a suitable setback distance for fill placement from the crest of any highwall.	
Pit wall failure	Risk of collapse / slope failure.	Implement a daily highwall condition monitoring plan to be used during operation of A-Drive Borrow Pit. Undertake regular mapping and assessment of discontinuity structures on all highwalls as the borrow pit is expanded.	
Seismic hazard model requires updating to align with recent NHSM update.	Seismic parameters used in design may not be sufficient relative to the latest seismic model.	Update site seismic hazard model using the updated New Zealand Seismic Model (NZSM).	
Insufficient monitoring systems in place.	Compliance requirements may not be met.	Installation of standpipe piezometers to monitor groundwater quality	
Use of finished borrow pit as Coal Fines Storage Facility (CFSF).	Effects of filling borrow pit with coal fines has not been assessed. Potential for contamination of groundwater systems from pond leakage.	Stormwater diversions be put in place to divert runoff away from the A-Drive Borrow Pit. Update Trigger Action Response Plan (TARP) to include recommendations and monitoring requirements specific to A-Drive Borrow Pit. Detailed assessment of the interaction between coal waste and the borrow rock walls to determine any effects potentially weakening the rock. A drainage assessment of the pit needs to be undertaken to understand where any leakage from the pit goes and to evaluate the effects of drainage from the CFSF through fractures in the rock. It is recommended consideration should be given to options for lining the pit to prevent leakage when it is used as a CFSF.	
CFS Stages 2 and 3			
Old coal fines storage pond located in the southern part of CFS Stage 2.	Coal fines act as non-Newtonian fluids, risk of fluidised flow.	All previous coal fines should be removed/relocated to another storage facility.	
Lack of borehole data in the vicinity of CFS Stages 2 and 3	Geology and rock mass parameters have been inferred and may not be representative.	Undertake geotechnical drilling at the highwall locations of CFS Stages 2 and 3 to confirm ground conditions.	

14.0 Conclusions

This geotechnical assessment has confirmed that the proposed expansion of the A-Drive Borrow Pit and CFS stages 2 and 3 as part of the Bathurst Resources Limited's, Buller Plateaux Continuation Project (BPCP) Fast Track Approval (FTA) application is feasible from a slope stability and structural integrity perspective. The highwall slopes are stable under static conditions, and generally perform acceptably under seismic loading, with minor shallow failures anticipated in fill materials during higher return period events. Subsequent design stages will enable implementation of measures to address potential risks related to seismic hazard modelling, coal fines pond proximity, and highwall monitoring. Continued structural mapping, targeted investigations, and proactive risk mitigation will be essential to support safe borrow pit development and potential future use of the site as a coal fines storage facility.

15.0 References

- Barry, J.M.; Duff, S.W.; Macfarlane, D.A.B. 1994: Coal Resources of New Zealand. Resource Information Report 16, Energy & Resources Division, Ministry of Commerce, New Zealand.
- Bray, J.D., and Travarasrou, T. (2007). "Simplified procedure for estimating earthquake-induced deviatoric slope displacements.
- Bray, J.D. (2017). "Seismic slope stability procedures for geotechnical practice."
- Hoek, E. (2012). Blast Damage Factor D. Technical note for RocNews – February 2, 2012.
- Marinos, P and Hoek, E. (2000). GSI: A Geologically Friendly Tool for Rock Mass Strength Estimation.
- Mould (2004). Estimation of Seismic Hazard at the Cypress Mine.
- Ministry of Business, Innovation, and Employment (MBIE). New Zealand Geotechnical Society. November 2021. Earthquake Geotechnical Engineering Practice – Module 1: Overview of the Guidelines.
- Nathan, Simon, 1996. Geology of the Buller Coalfield, scale 1:50,000. Institute of Geological & Nuclear Sciences geological map 23. Lower Hutt, New Zealand: Institute of Geological & Nuclear Sciences Limited.
- Phillips, Alex (2010). Geotechnical Factual Report - Results of A-Drive Quarry investigation, Stockton Mine. Stockton Alliance, December 2010.
- Read, J., & Stacey, P. (2009). *Guidelines for Open Pit Slope Design*, CSIRO Publishing.
- Stafford, P. (2003). Revised Estimate of Seismic Hazard for Solid Energy.

Tesoro, Julius (2012). Geotechnical Report – A-Drive Quarry Development, Stockton Mine. Stockton Alliance, September 2012.

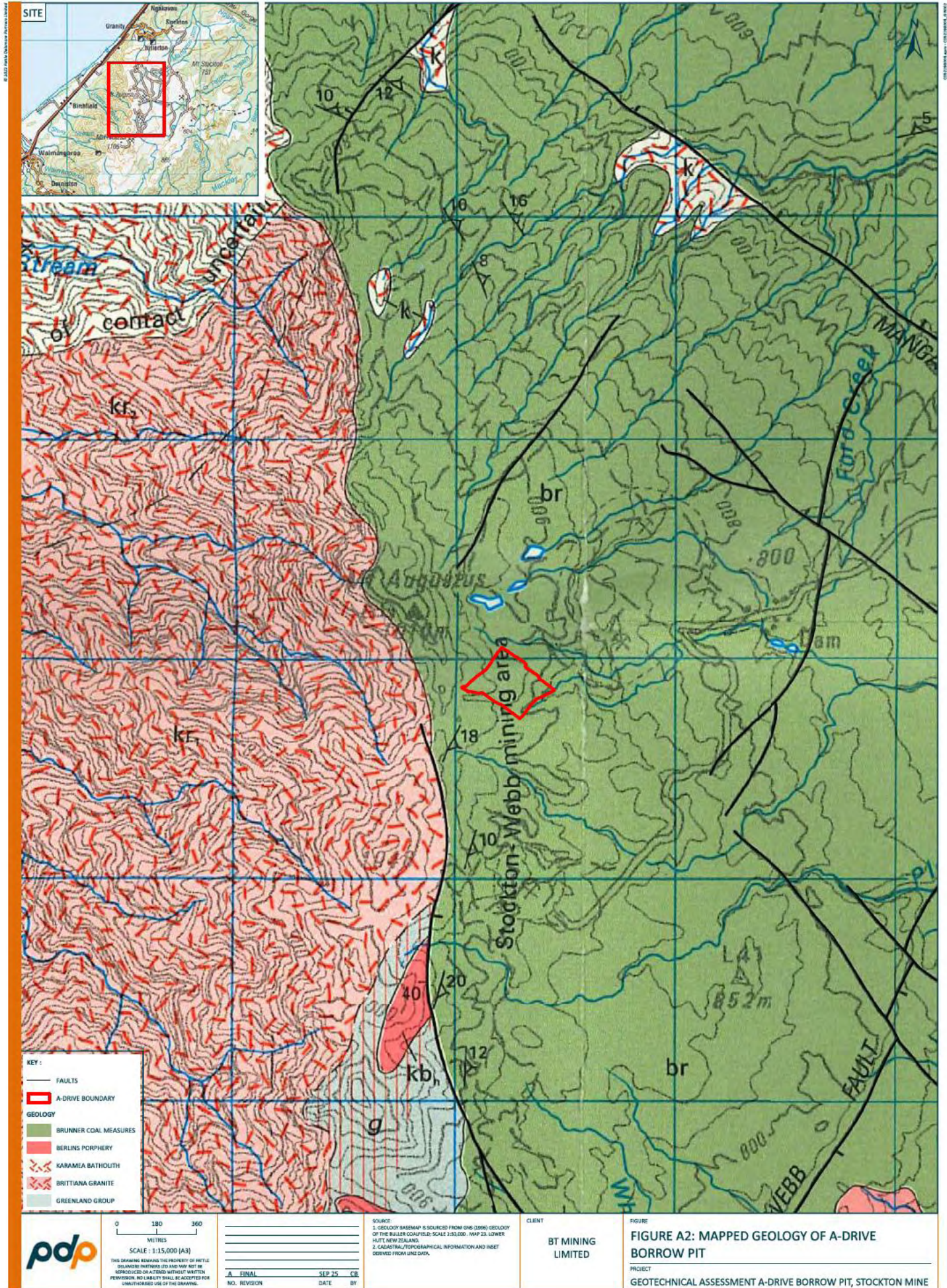
Tulia, Kurtis (2020). Investigation on the Suitability of Granitic Rock Aggregate for Mine Haul Roads, Stockton, New Zealand. Professional master's in Engineering Geology Dissertations 2020, School of Earth & Environment, University of Canterbury, New Zealand.

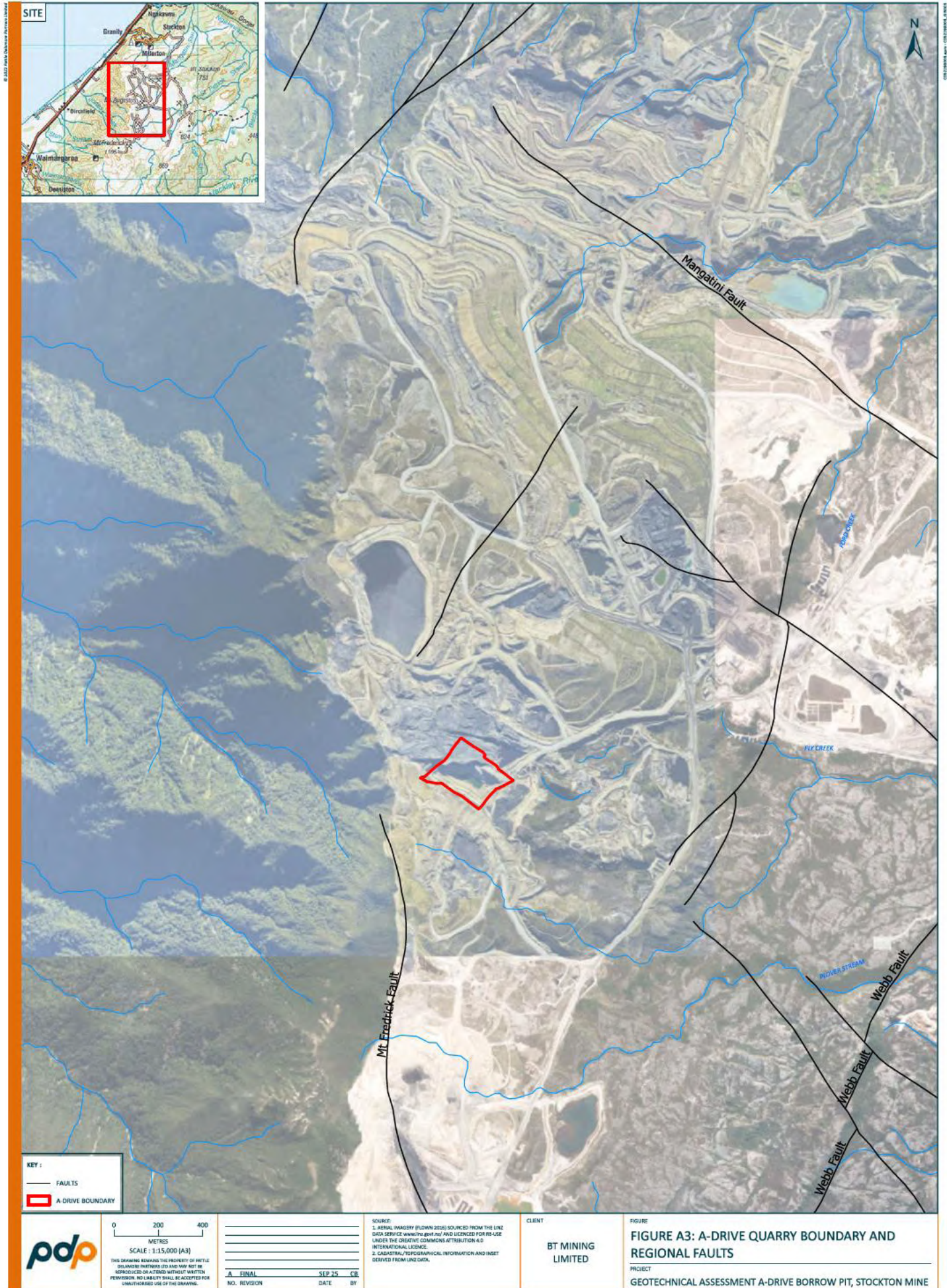
URS Limited (2007). TM#2 Seismic Hazard Assessment, Stockton Opencast Mine.

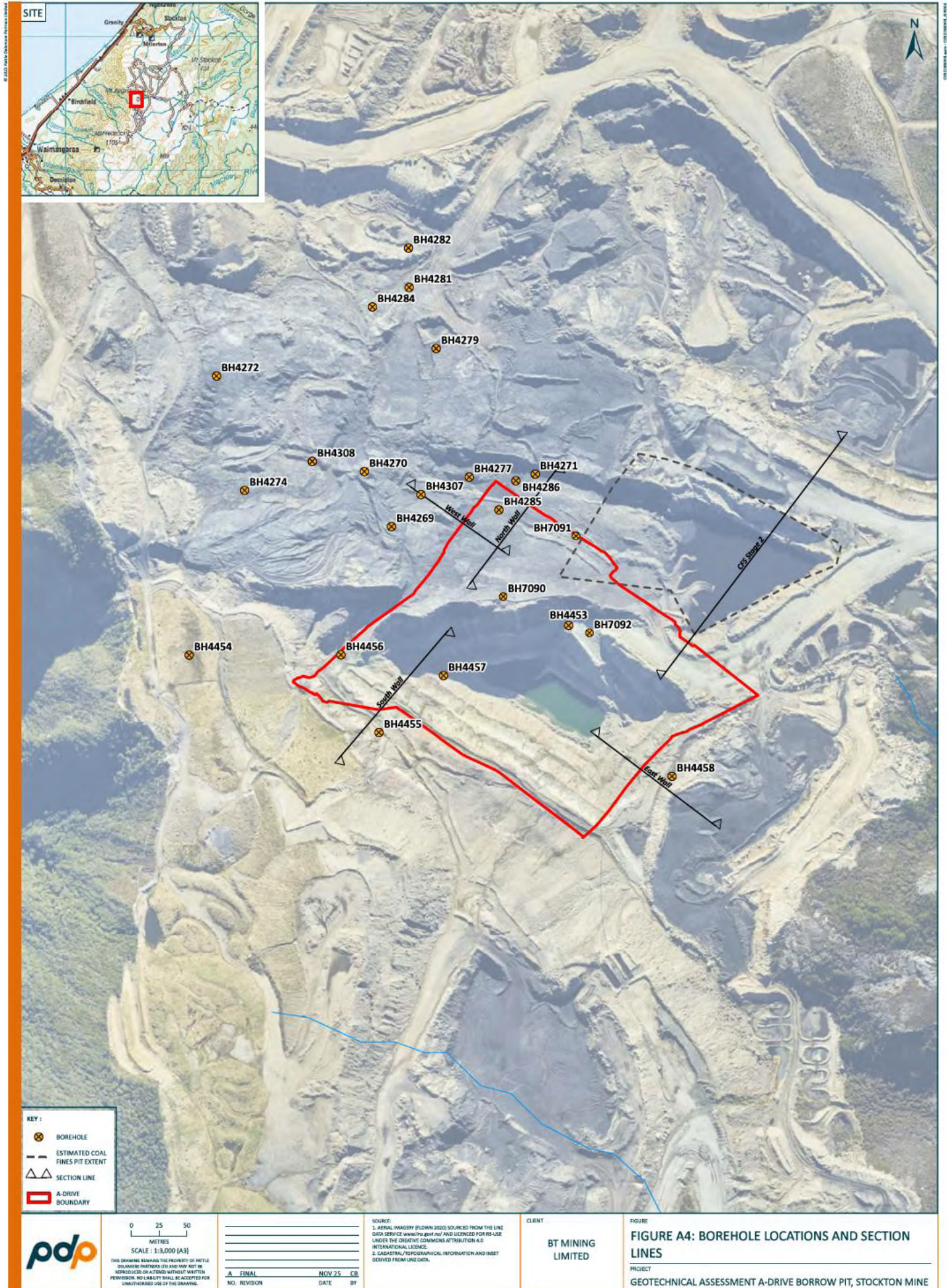
Worksafe – Mahi Haumaru Aotearoa (2025) Good Practice Guidelines - Health and Safety at Opencast Mines, Alluvial Mines and Quarries, June 2025.

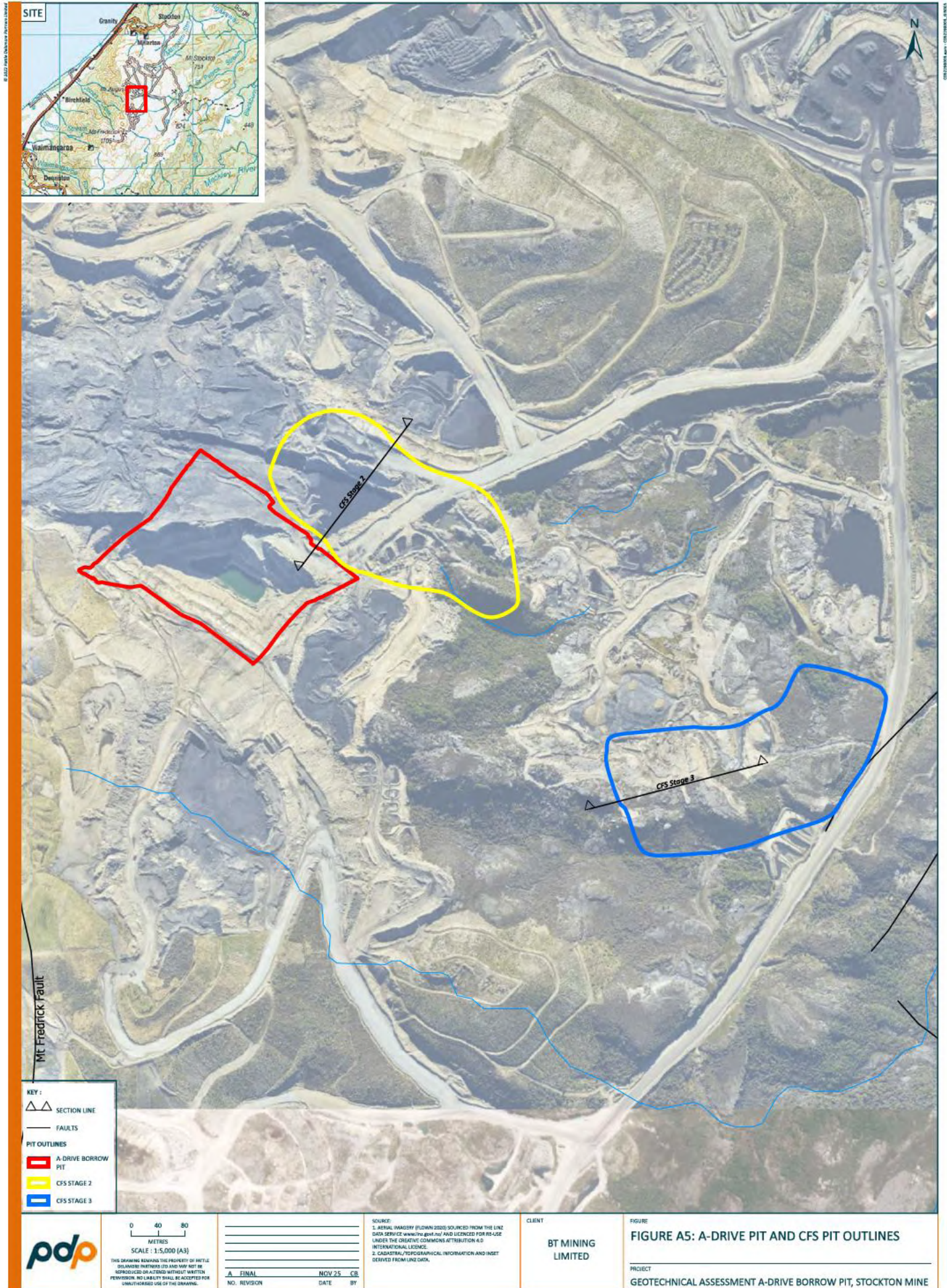
Appendix A: Site Plans











Appendix B: Site Photos

APPENDIX B – SITE PHOTOGRAPHS



Figure B1: General site photograph looking west across the existing A-Drive borrow pit.



Figure B2: General site photograph looking northwest across A-Drive borrow pit.



Figure B3: Photograph looking south at face of the southern highwall – 1 July 2025 - Pit Lake visible in foreground.

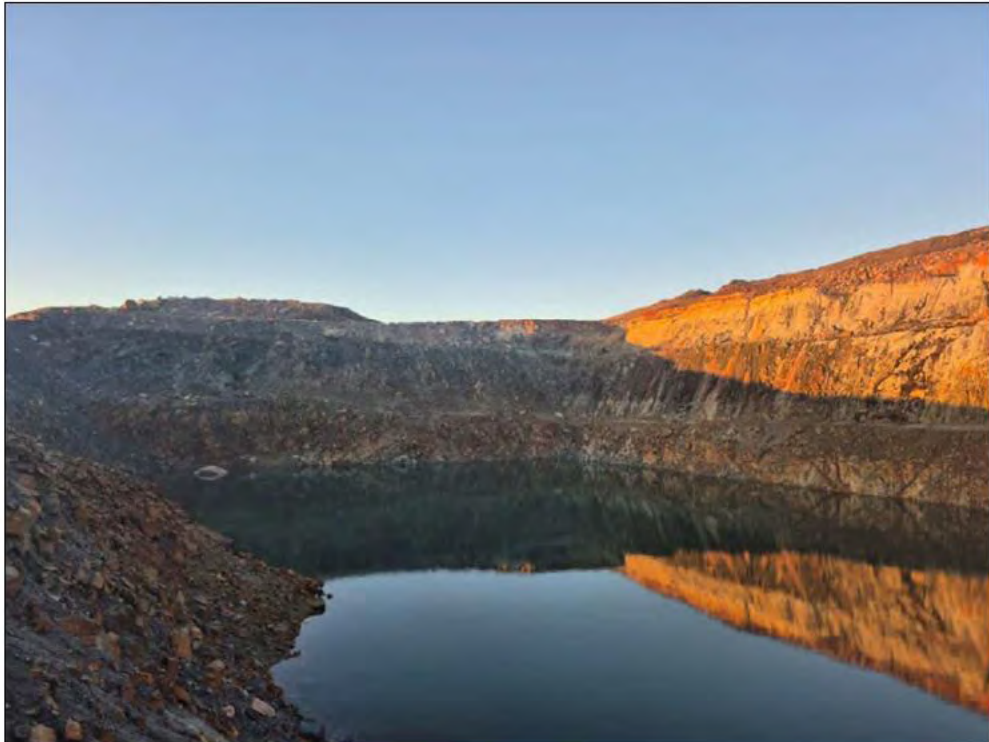


Figure B4: Photograph looking east across the A-Drive pit lake at the Eastern Highwall – 1 -July 2025.

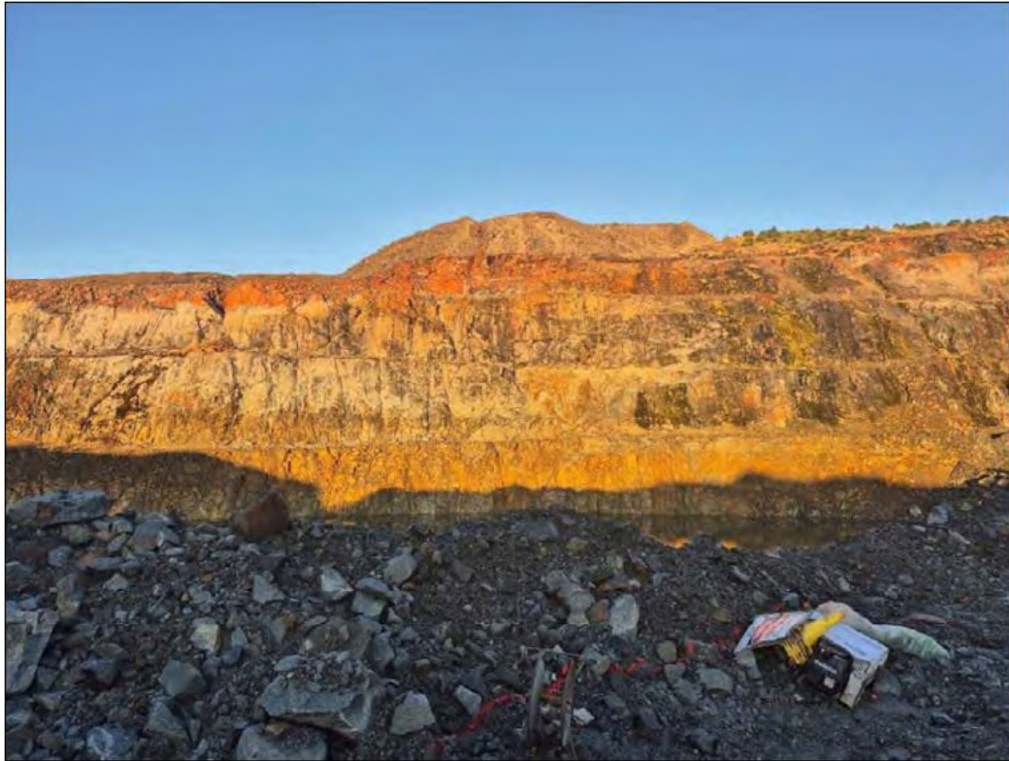


Figure B5: Photograph looking south at the face of the southern highwall – 1 July 2025.



Figure B6: Photograph of the northern highwall of the A-Drive borrow pit. Note: Surficial layer (approximately 1.0 m thick) of completely weathered to extremely weathered granite on highwall batter face.



Figure B7: Example of previously blasted granite aggregate.



Figure B8: Surface water runoff entering the A-Drive borrow pit at the intersection of the western and northern pit walls

Appendix C: Historical Site Imagery (Google Earth)

APPENDIX C – HISTORICAL AERIAL IMAGERY



Figure C1: Google Earth – 23/12/2011 – Coal Fines Dump north of proposed northern highwall extension.



Figure C2: Google Earth – 17/8/2013 – A-Drive Borrow Pit with well-developed southern highwall.



Figure C3: Google Earth – 25/1/2015 - Continued development of borrow pit and coal fines dump visible to the north.



Figure C4: Google Earth – 7/3/2017 - Continued development of borrow pit and coal fines dump visible to the north.



Figure C4: Google Earth 7/1/2018 – Water beginning to accumulate in base of A-Drive borrow pit.



Figure C5: Google Earth - 12/2018 – A-Drive borrow pit filling with water.



Figure C6: Google Earth - 8/1/2019 – A-Drive borrow pit filling with water.



Figure C7: Google Earth – 11/8/2020 - A-Drive borrow pit filling with water, coal fines dump visible to the north of the A-Drive pit.



Figure C8: Google Earth – 23/2/2023 - A-Drive borrow pit full of water.



Figure C9: Google Earth - 5/2/2024 - A-Drive borrow pit flooded.

Appendix D: Slope Stability Outputs

A-Drive - North Wall - Overall

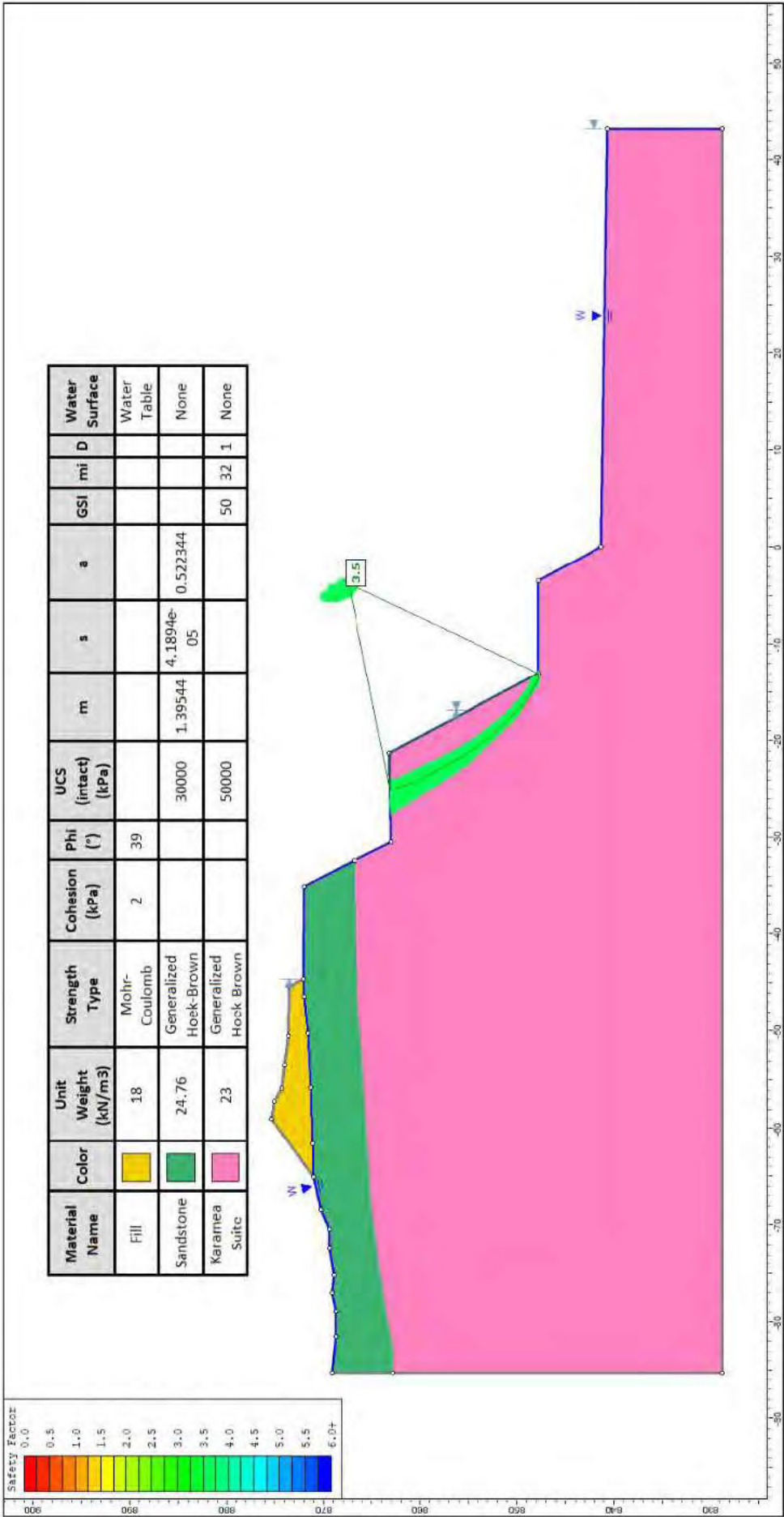


Figure D1: North Wall - Static - Overall Slope

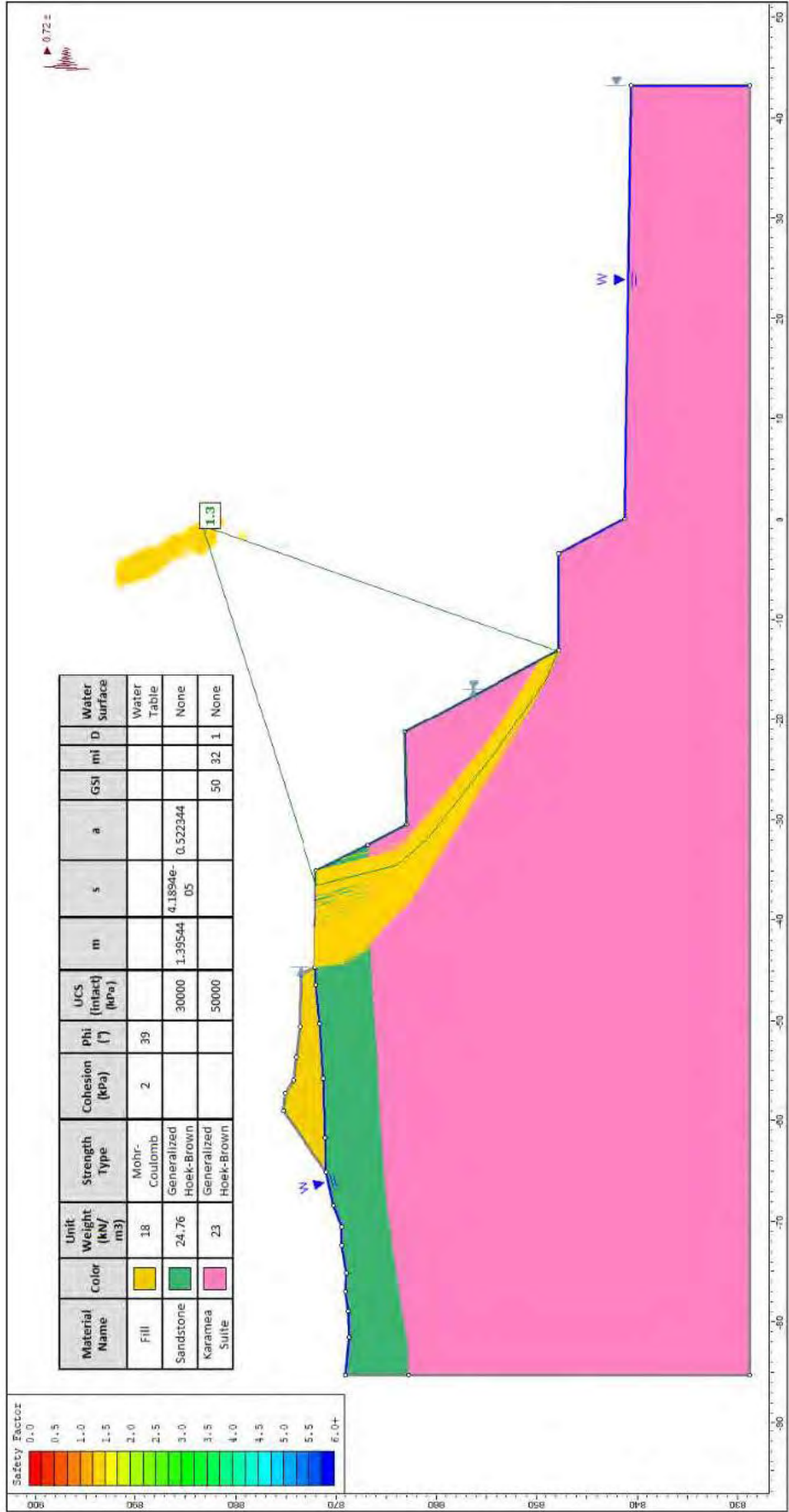


Figure D2: North Wall - Overall Slope - Seismic PGA = 0.72g.

A-Drive North Wall - Bench 1

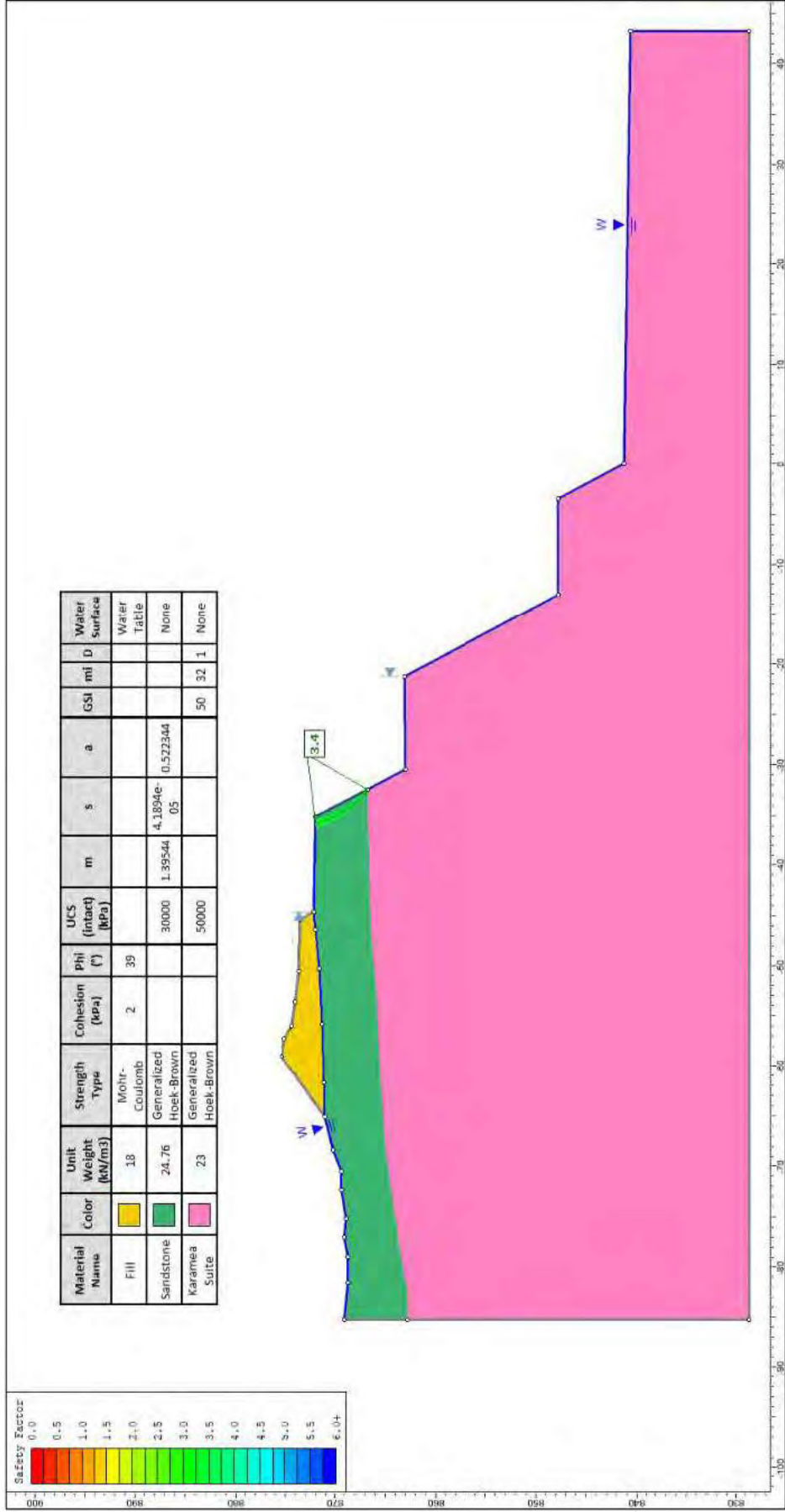


Figure D3: North Wall – Bench 1 – Static

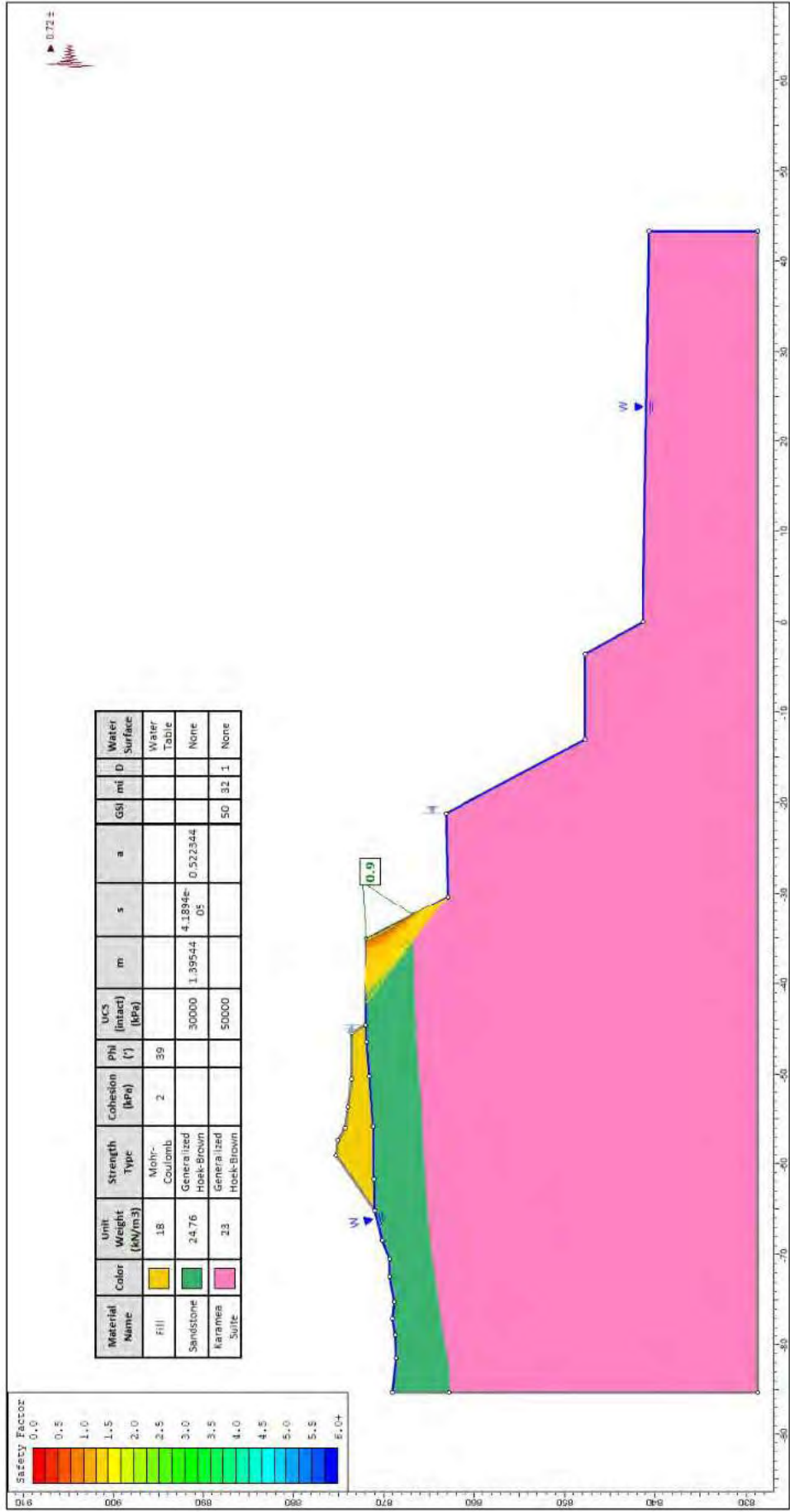


Figure D4: North Wall - Bench 1 - Seismic - PGA - 0.72g.

A - Drive North Wall Existing - Bench 2

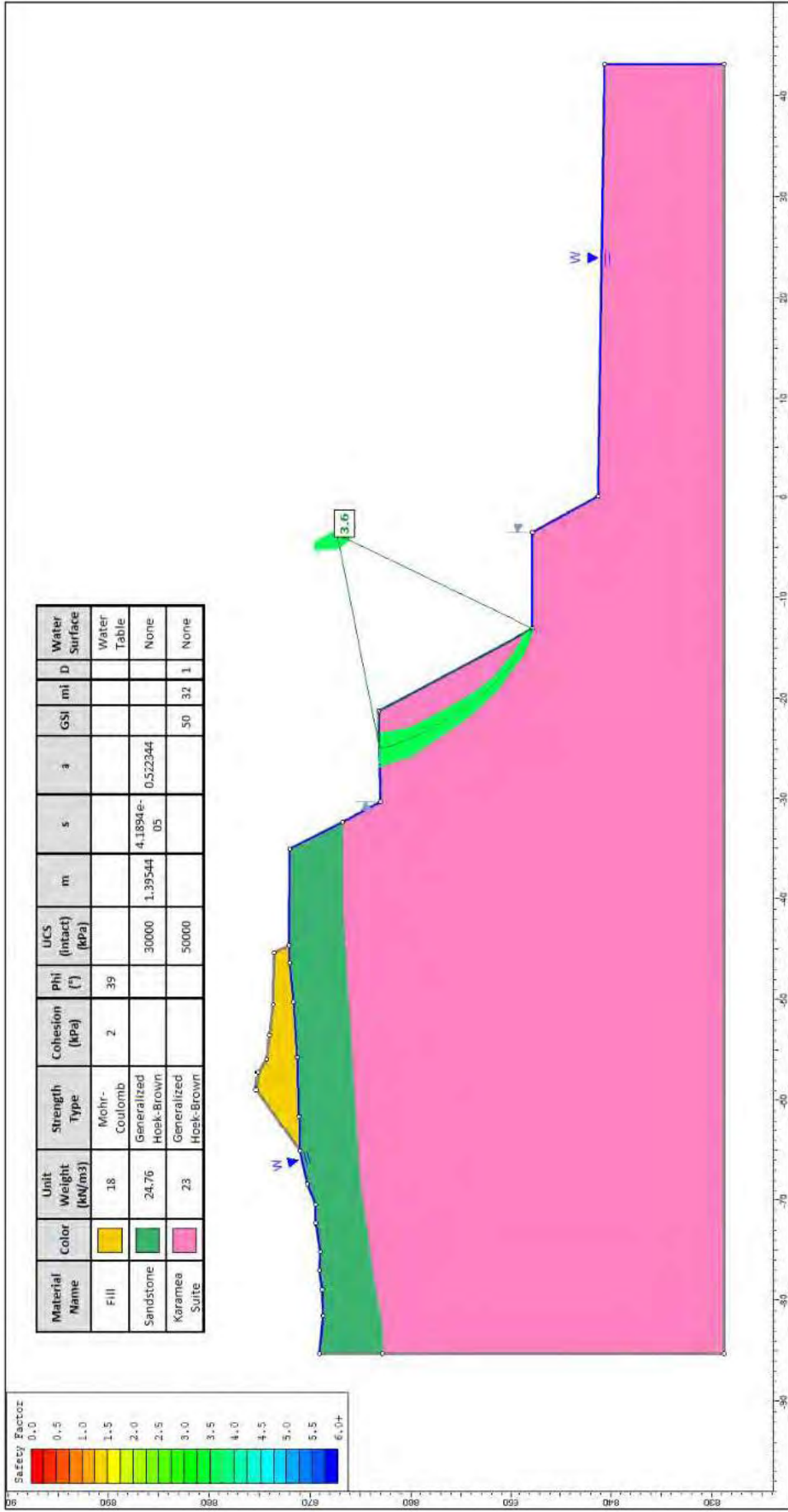


Figure D5: North Wall - Bench 2 - Static.

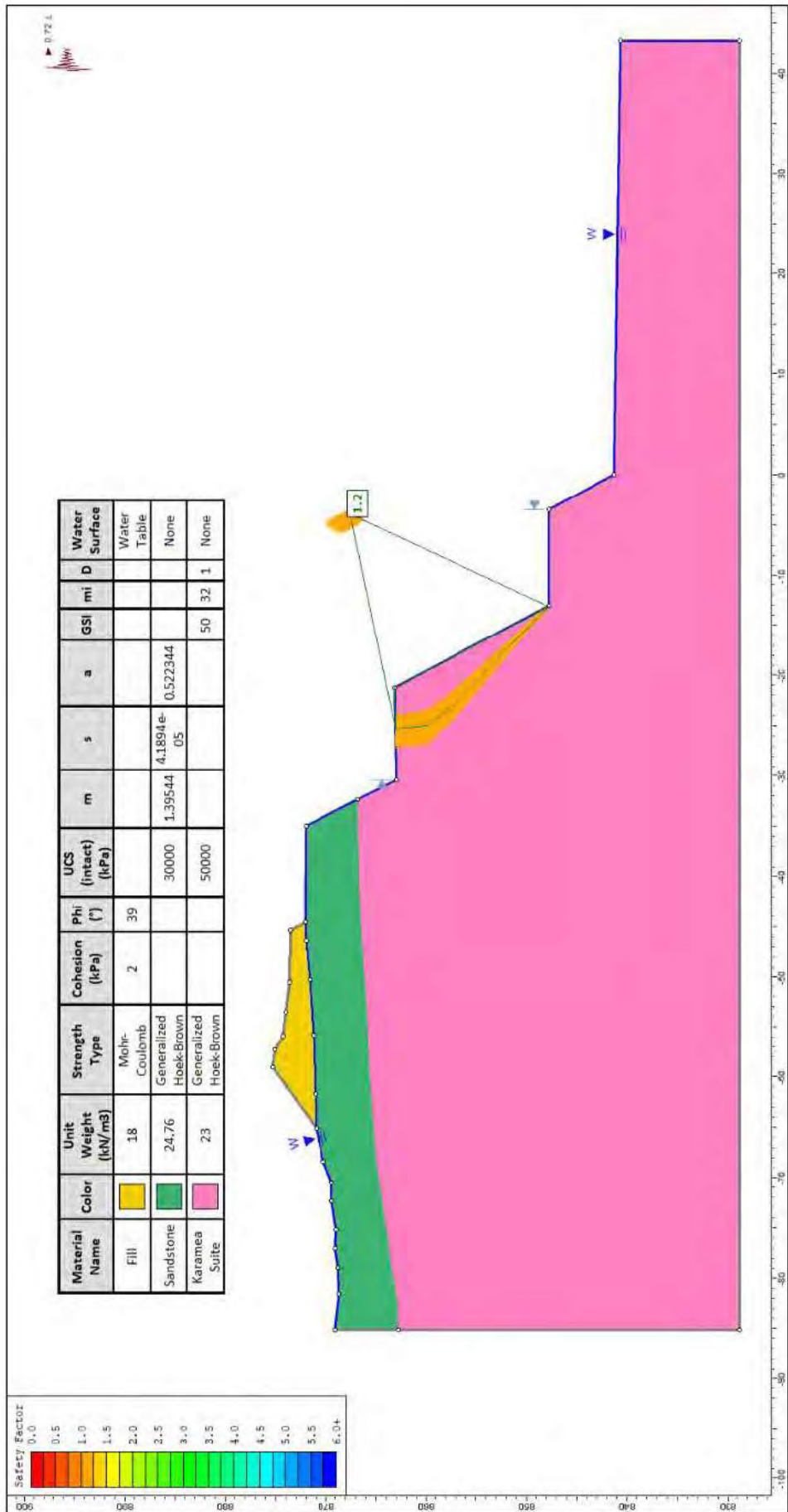


Figure D6: North Wall- Bench 2 - Seismic PGA 0.72g.

A-Drive North Wall - Bench 3

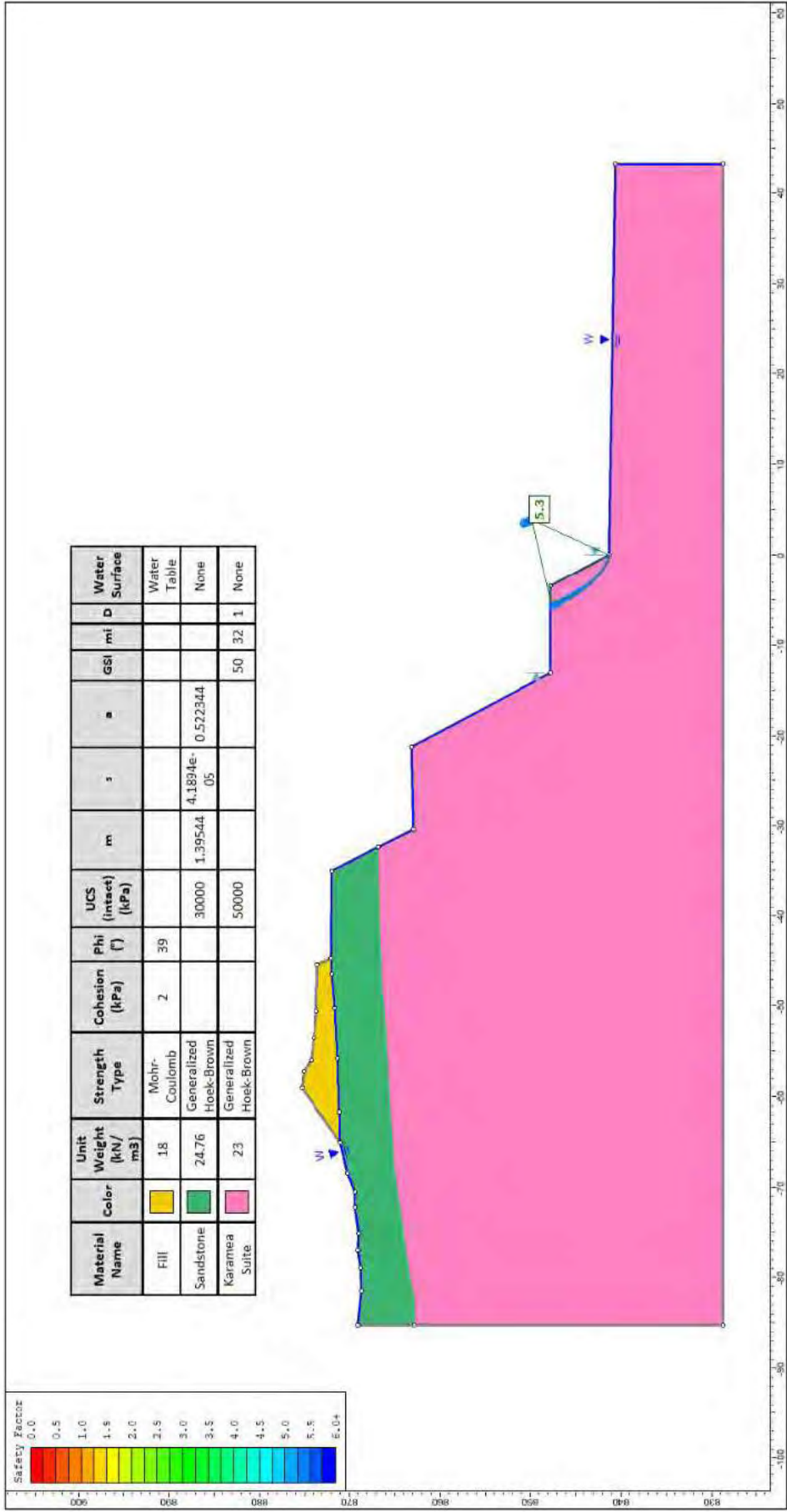


Figure D7: North Wall - Bench 3 - Static.

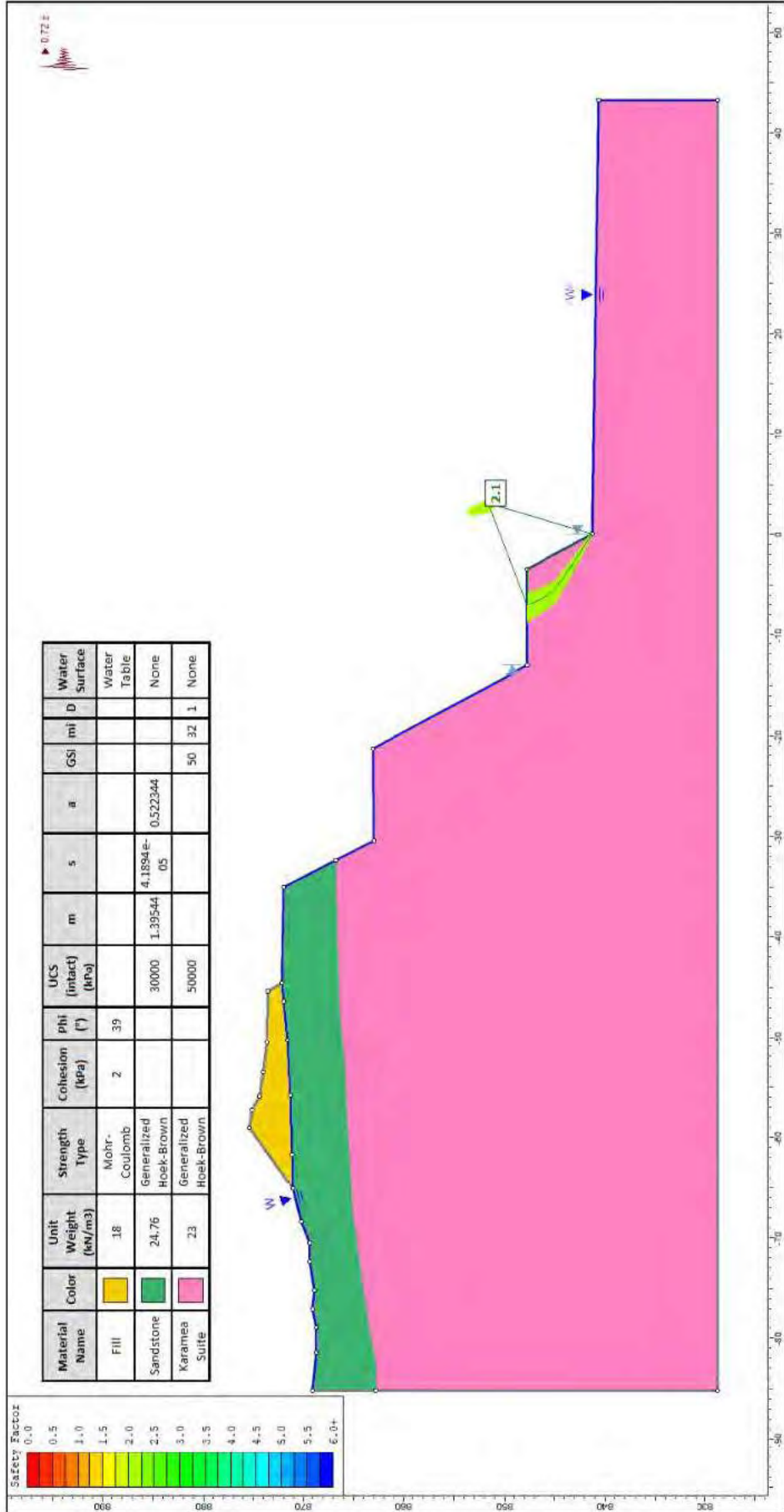


Figure D8- North Wall- Bench 3 - Seismic PGA 0.72g.

A-Drive North Wall - Multiple Bench (1-2)

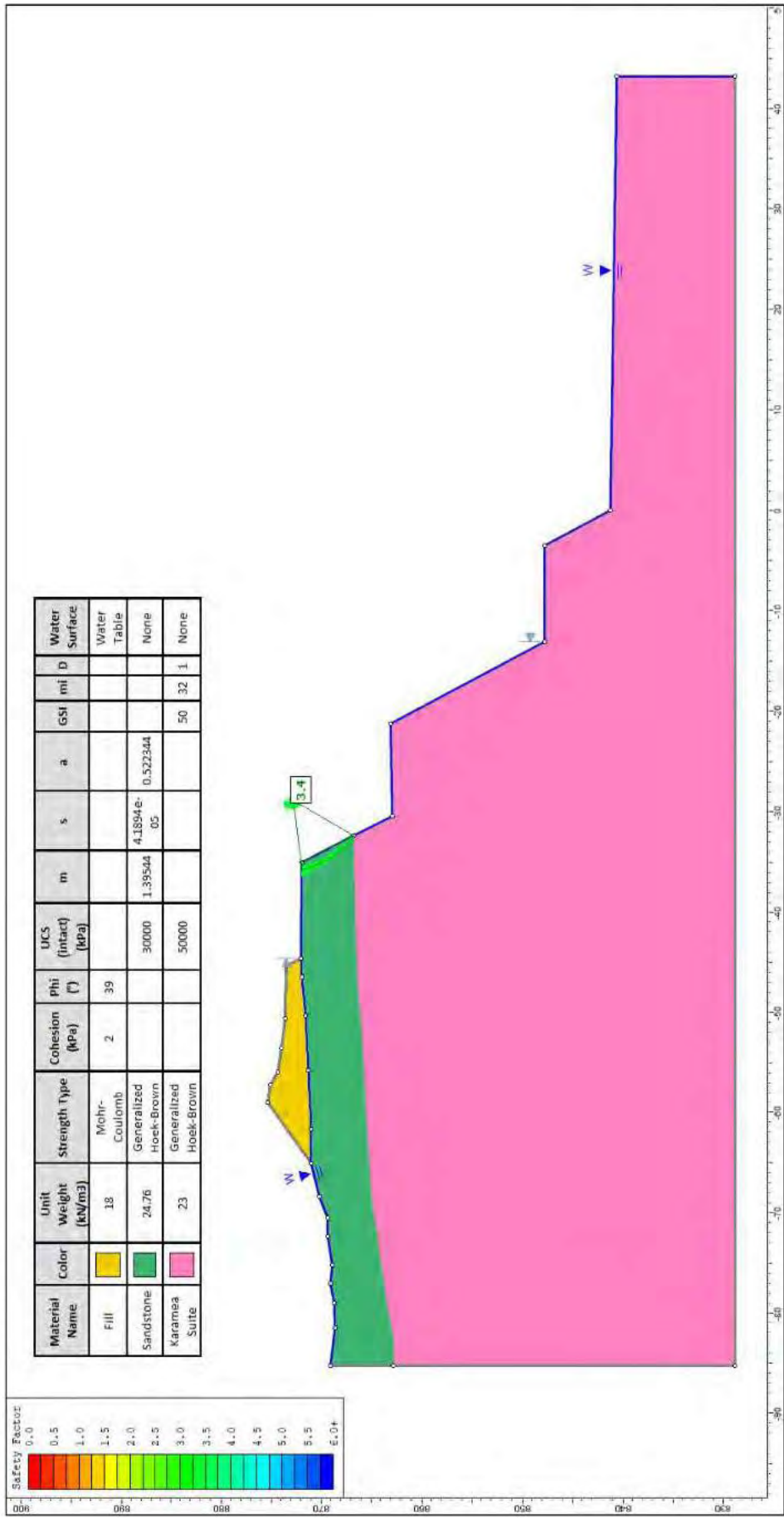


Figure D9: North wall - Static

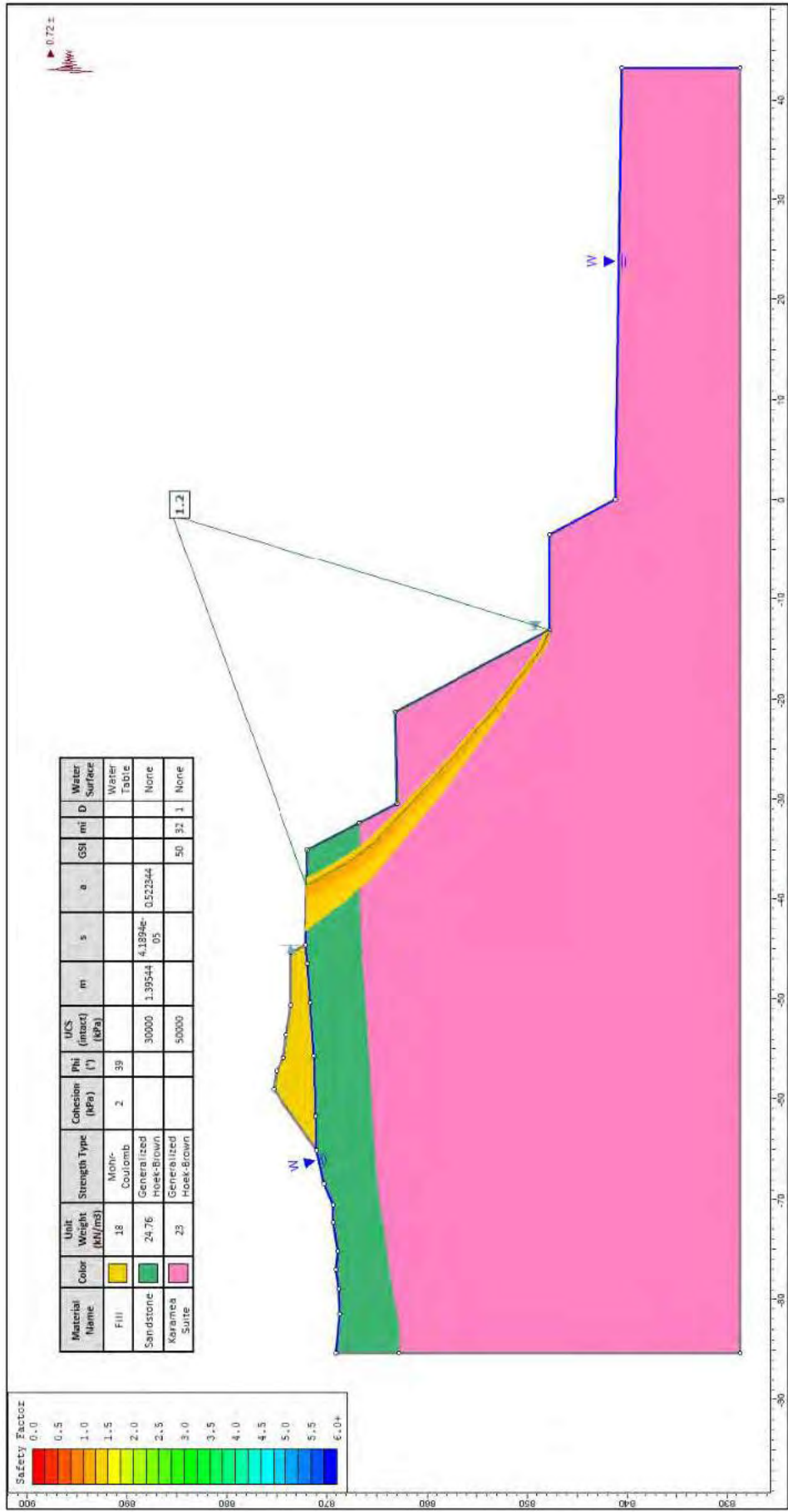


Figure D10: North Wall - Seismic PGA - 0.72g.

A-Drive North Wall - Multiple Bench (2-3)

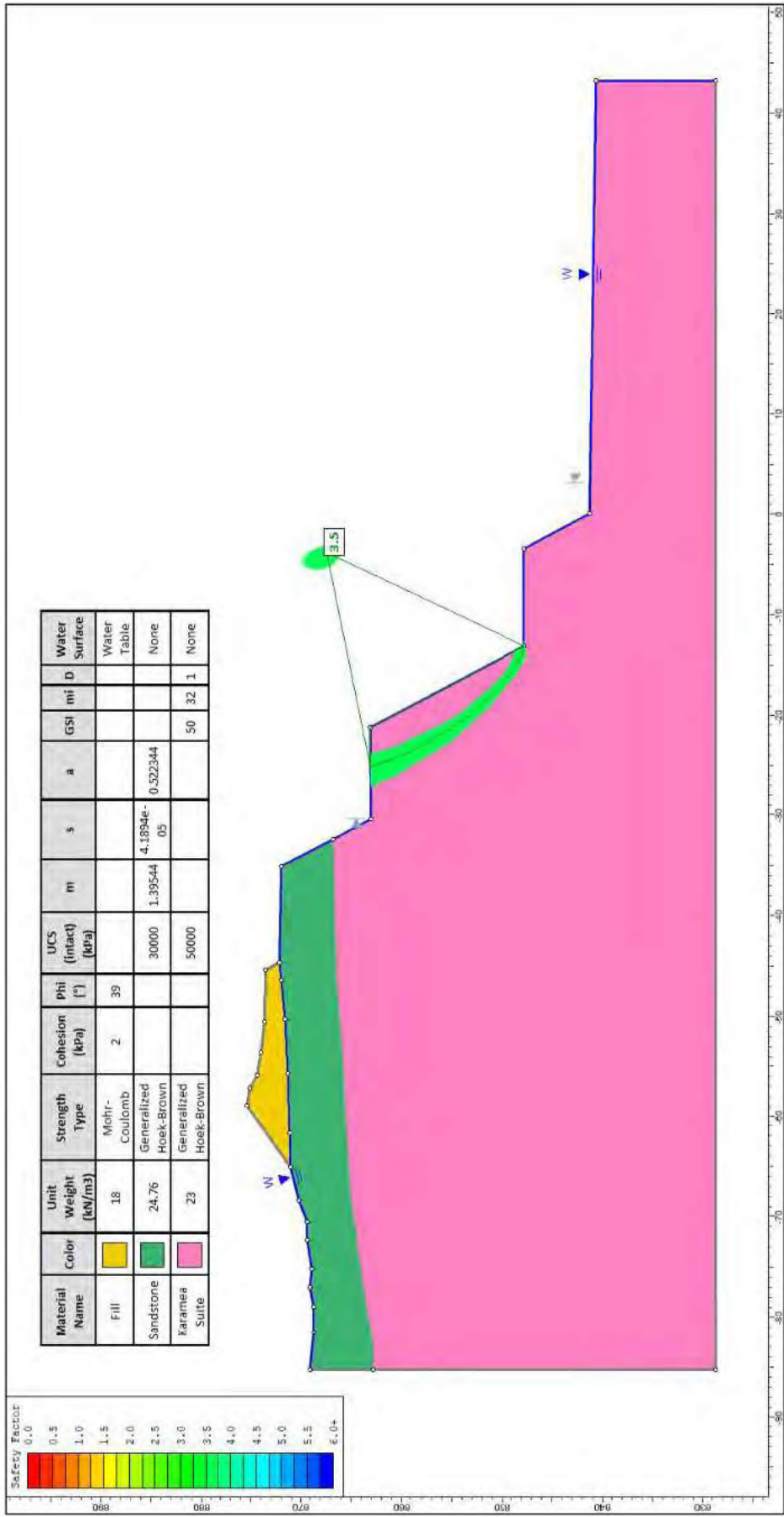


Figure D11: North Wall -Static – Multiple Bench (2-3)

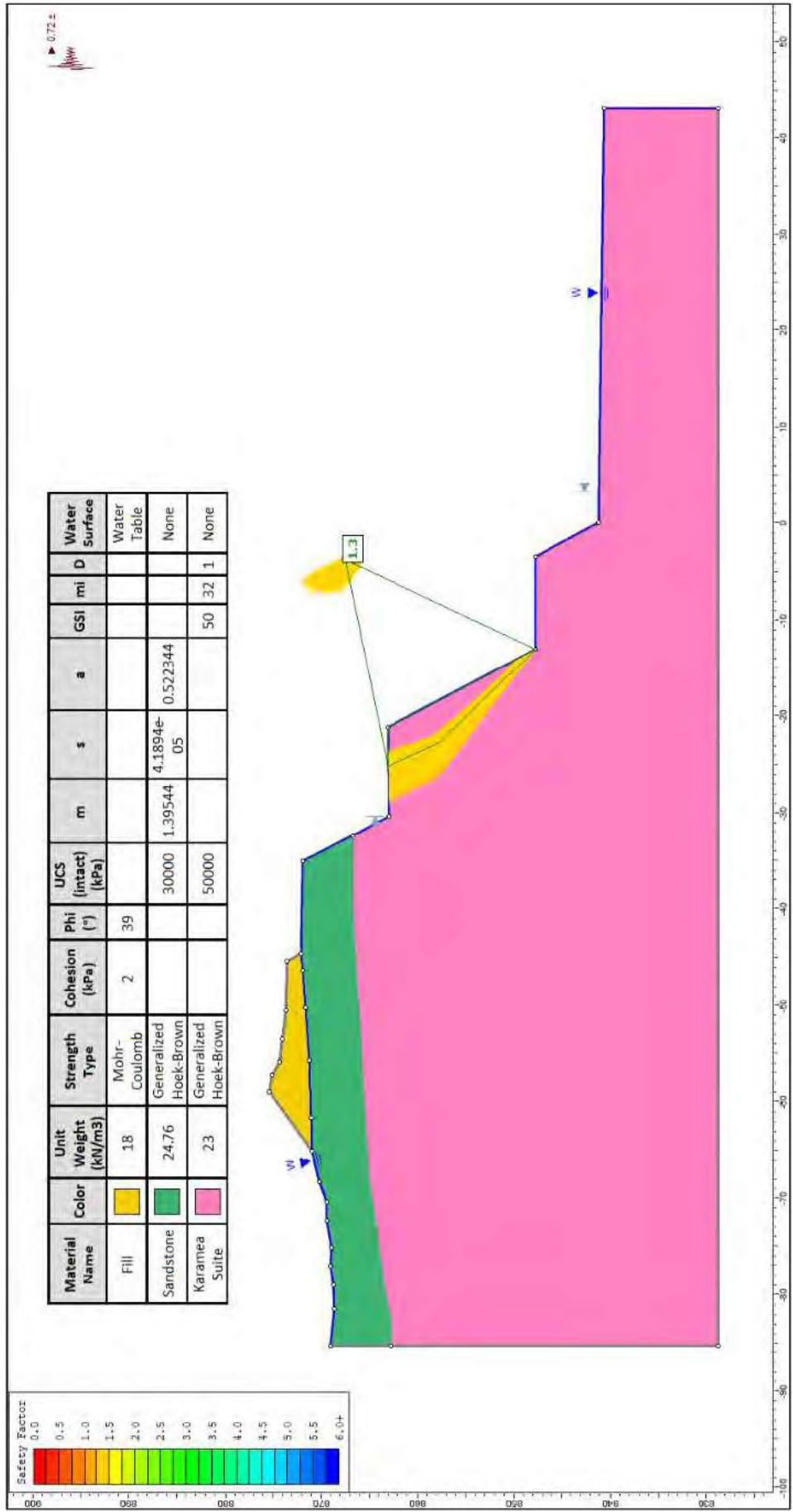


Figure D12: North Wall - Multiple Bench (2-3) - Seismic PGA - 0.72g.

A-Drive East Wall - Overall Slope

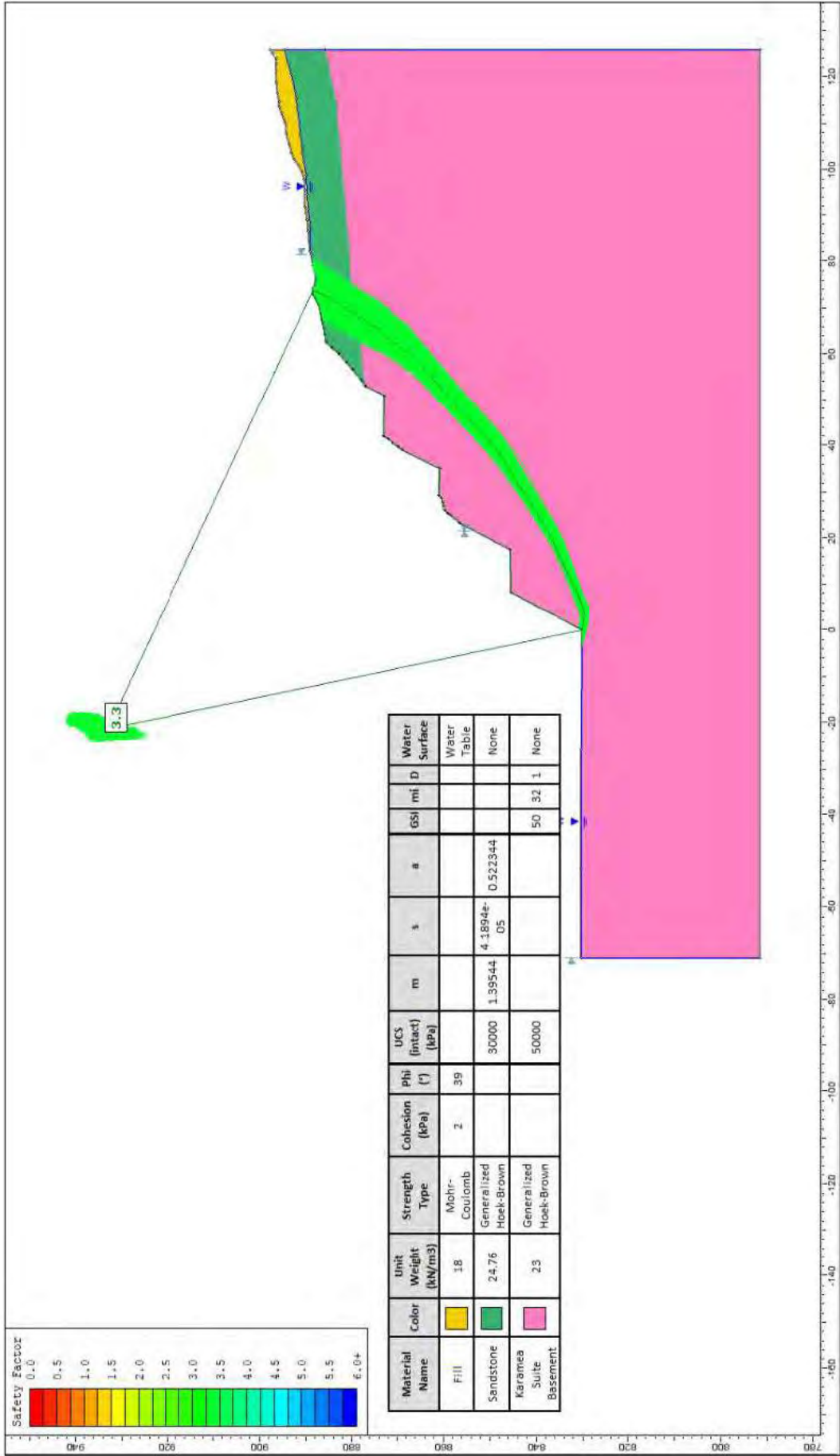


Figure D13: East Wall - Overall - Static.

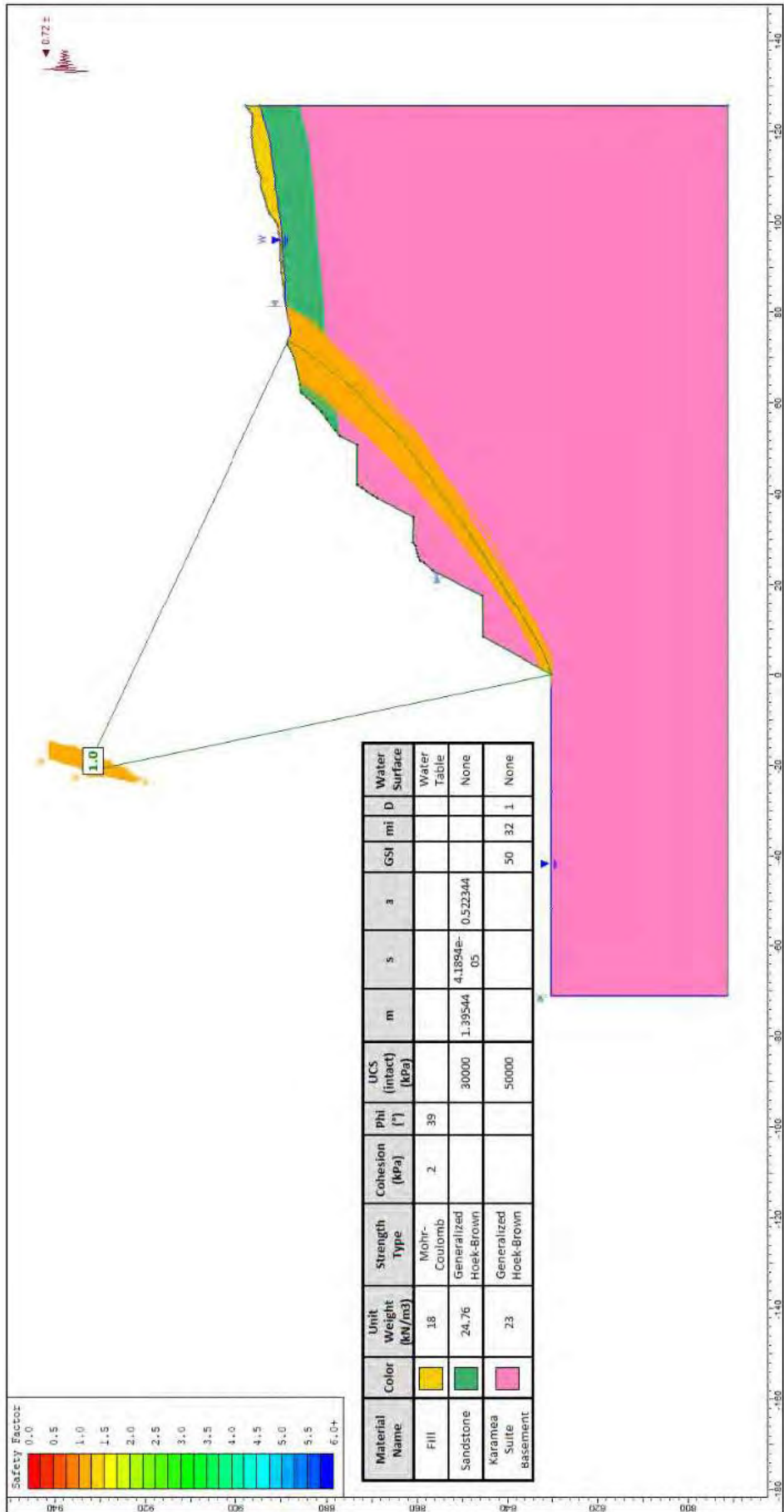


Figure D14: East Wall - Overall - Seismic PGA - 0.72g.

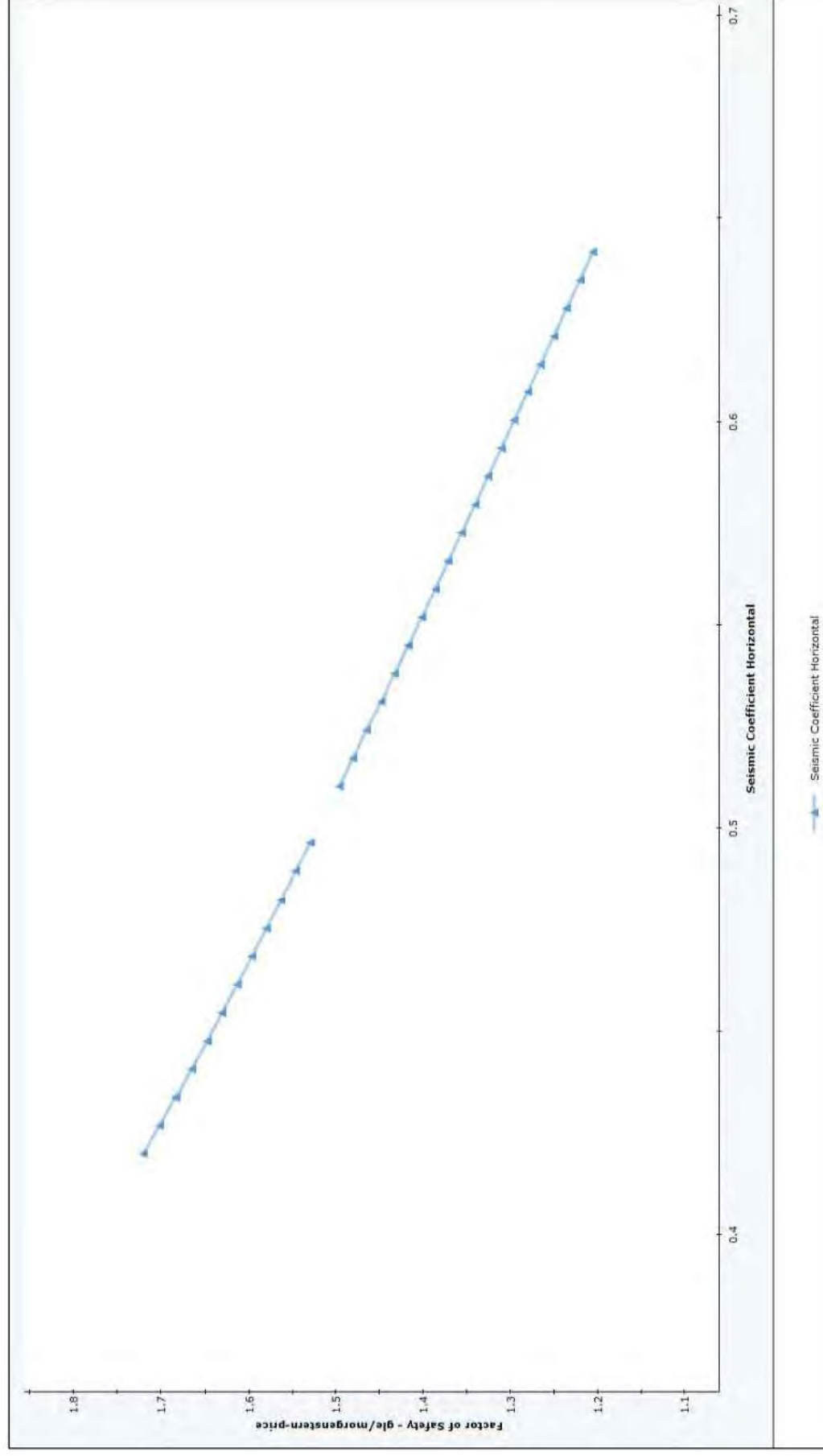


Figure D15: - Sensitivity Analysis East Wall Overall Slope

A-Drive East Wall - Bench 1

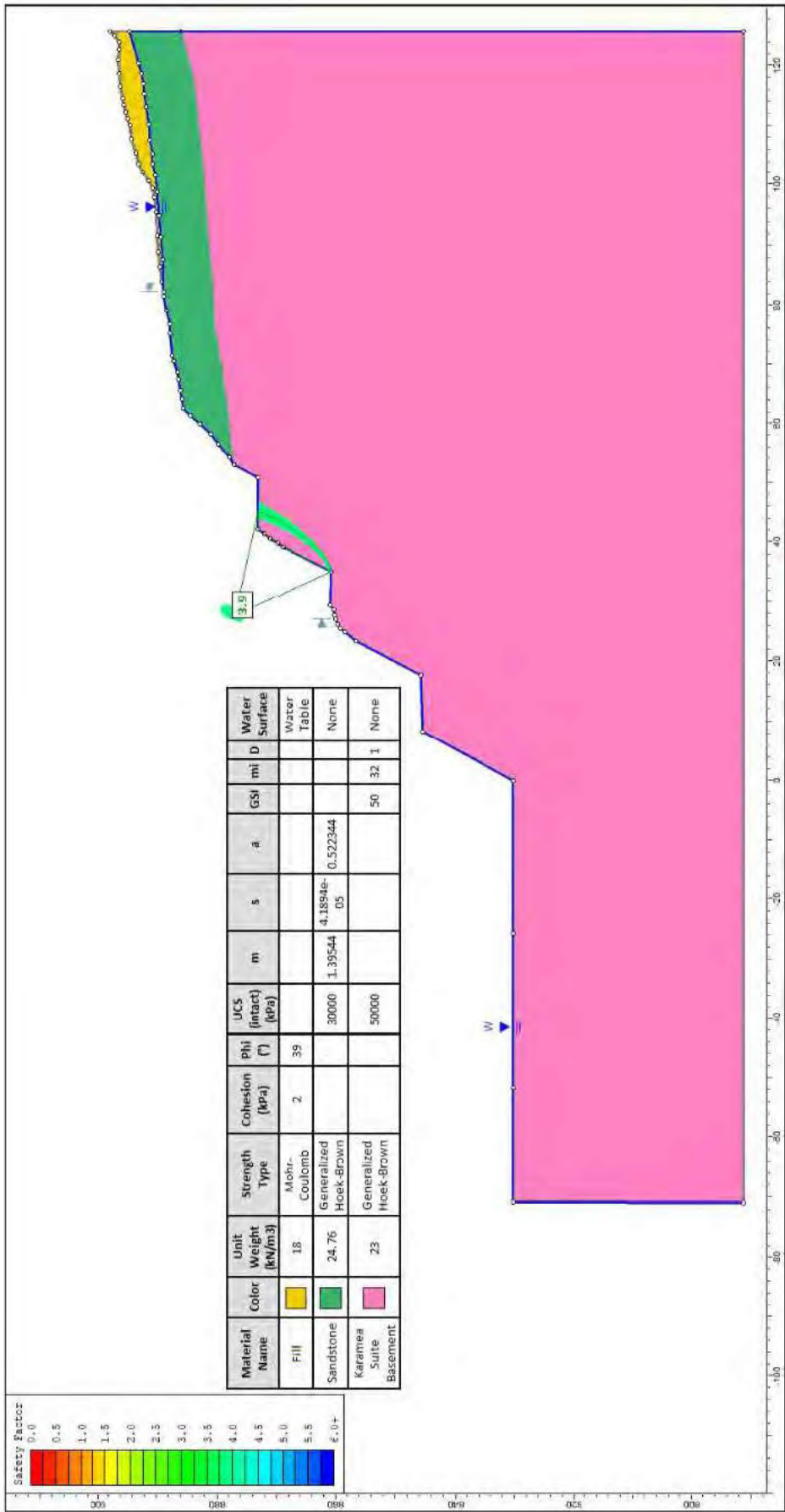


Figure D16: East Wall - Bench 1 - Static.

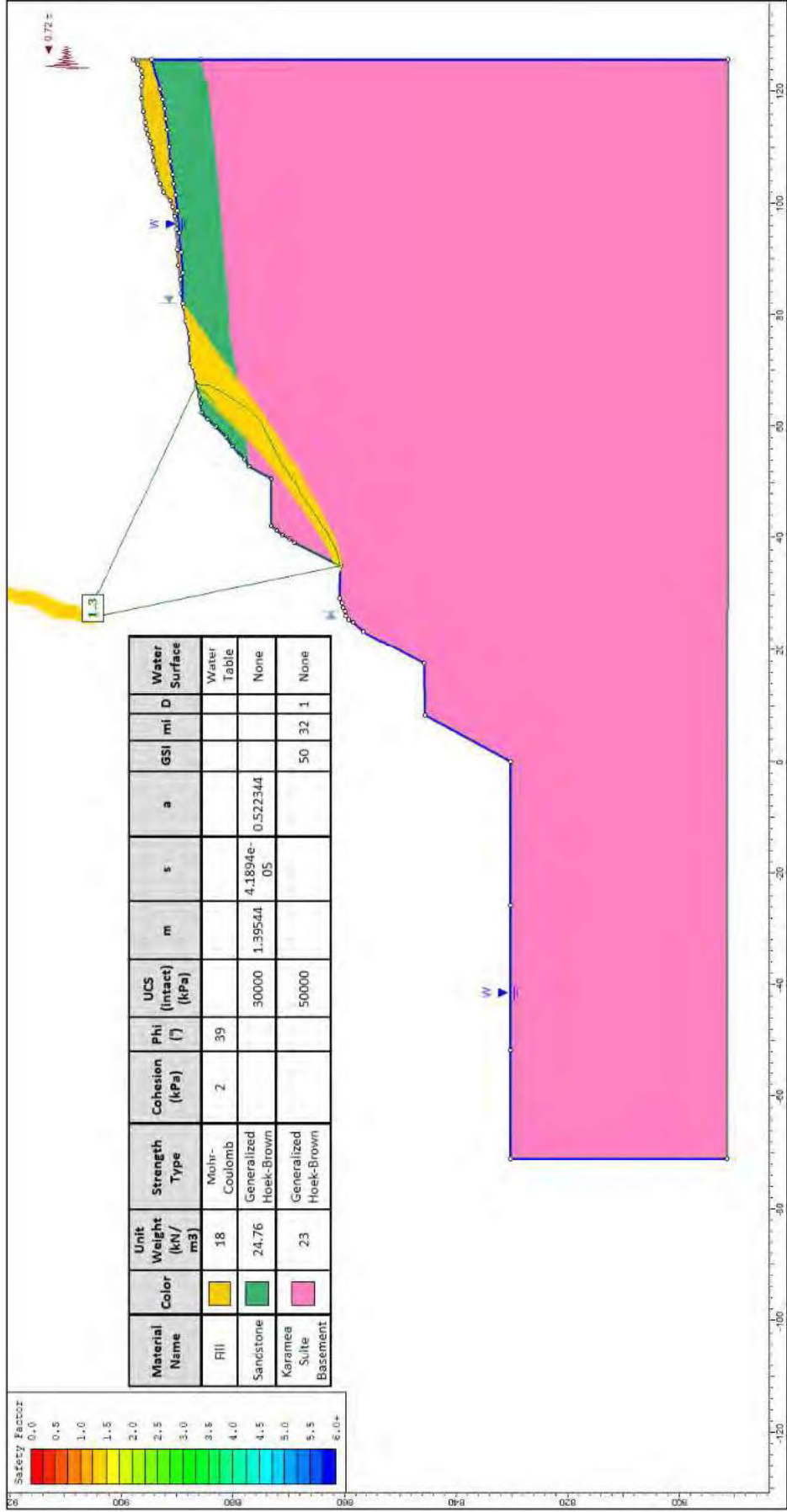


Figure D17: East Wall-Bench 1 - Seismic PGA - 0.72g.

A-Drive East Wall - Bench 2

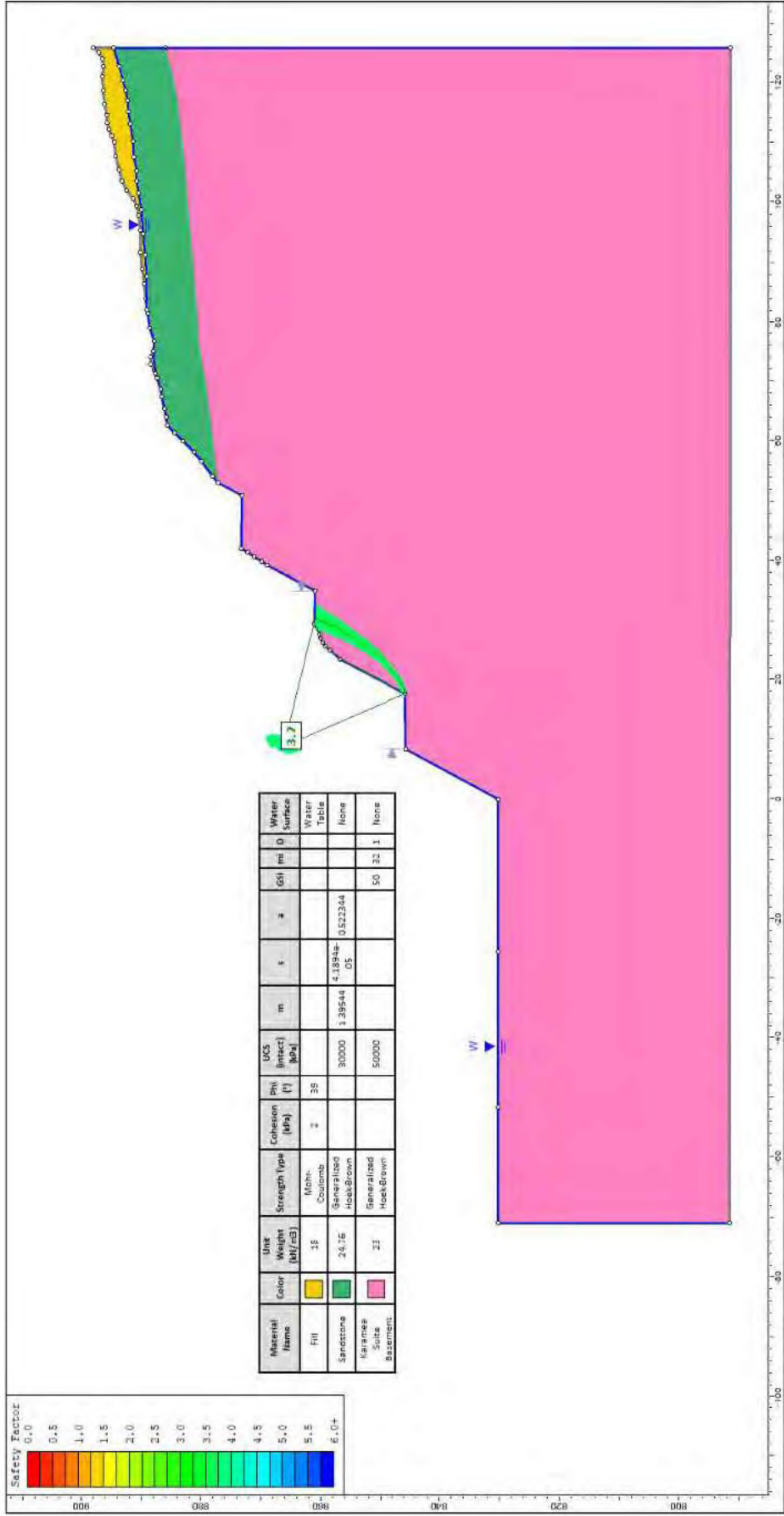


Figure D18: East Wall - Bench 2 - Static.

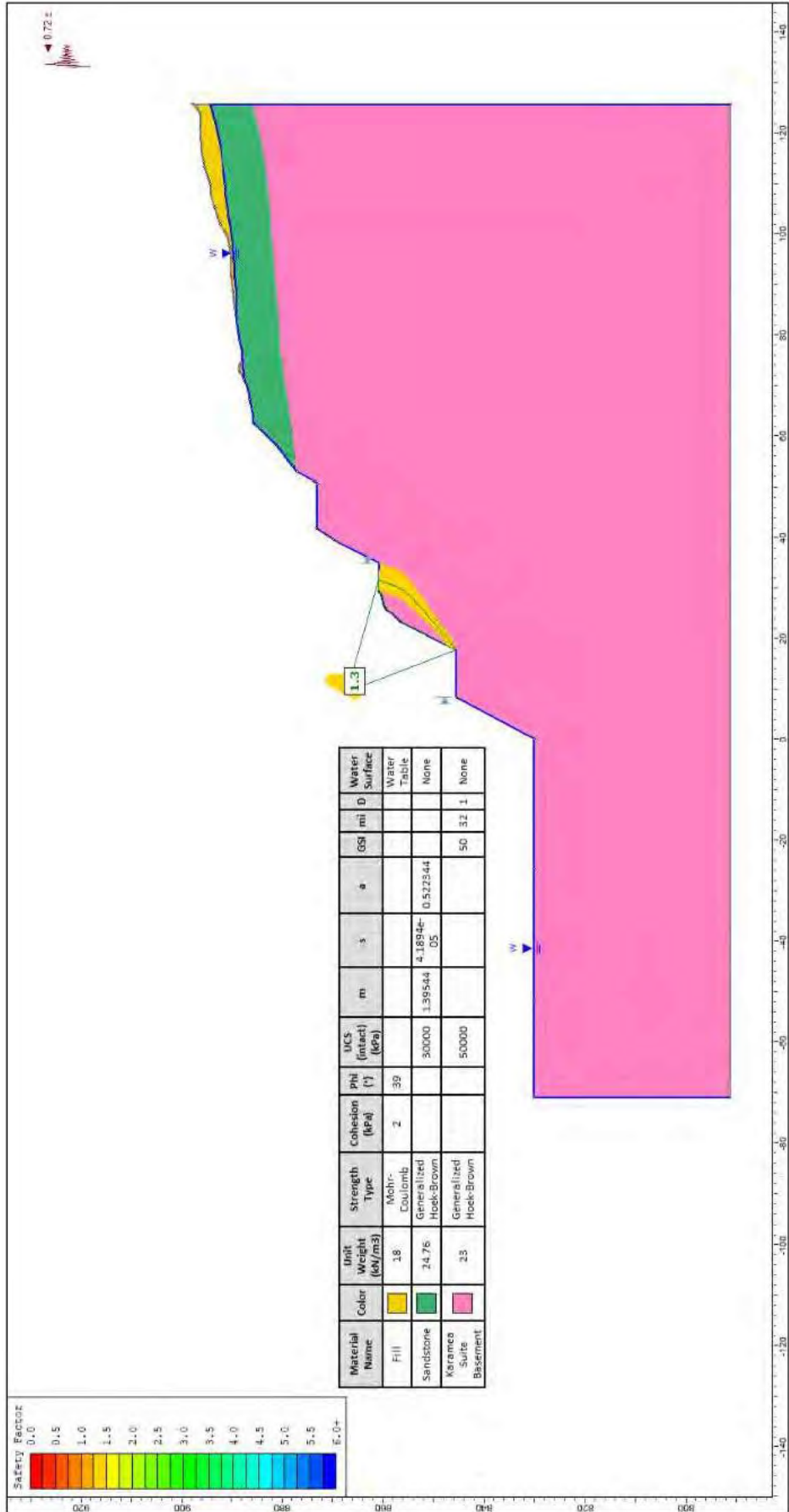


Figure D19: East Wall - Bench 2 - Seismic PGA - 0.72g.

A-Drive East Wall Proposed - Bench 3

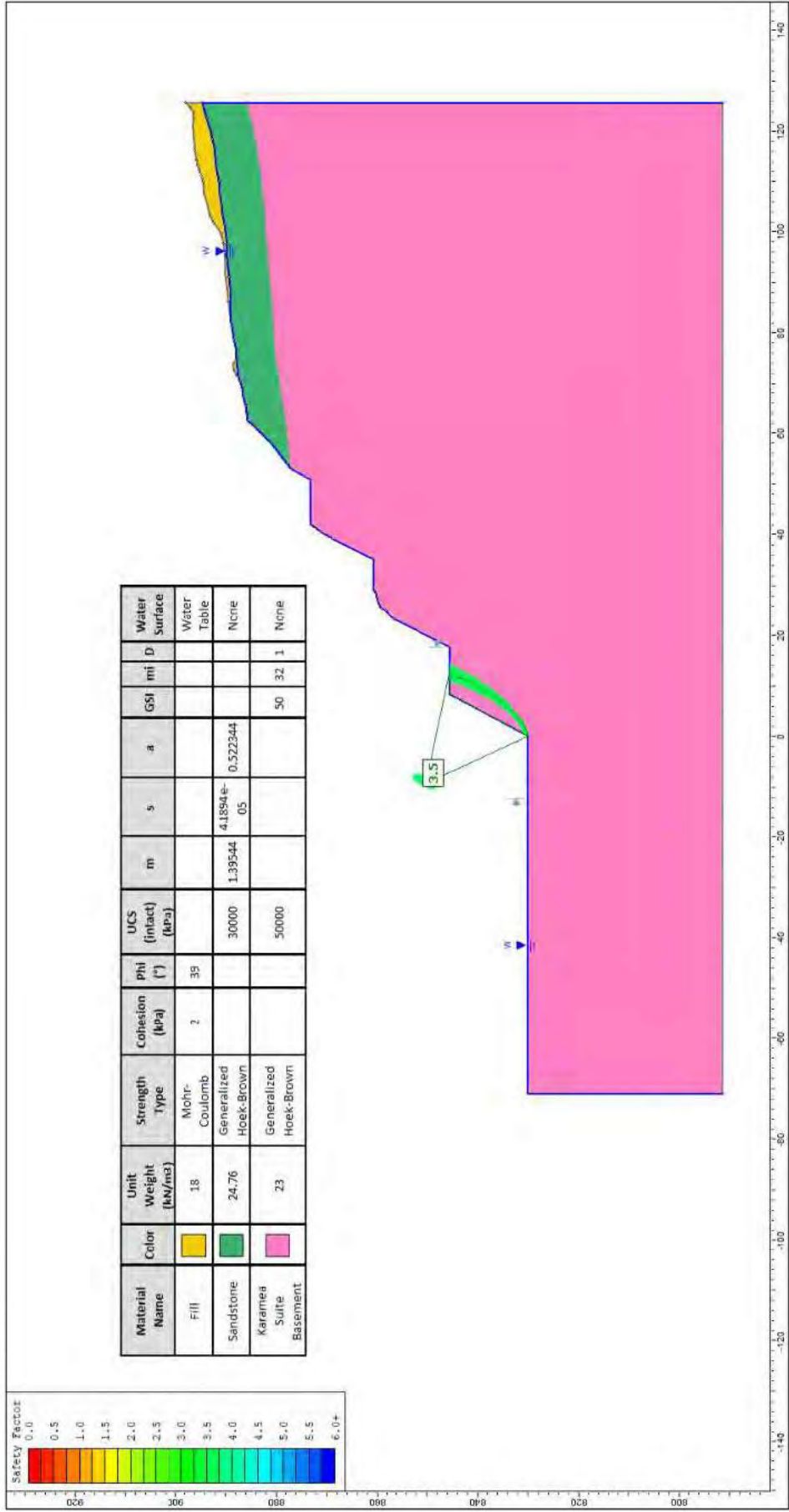


Figure D20: East Wall - Bench 3 - Static.

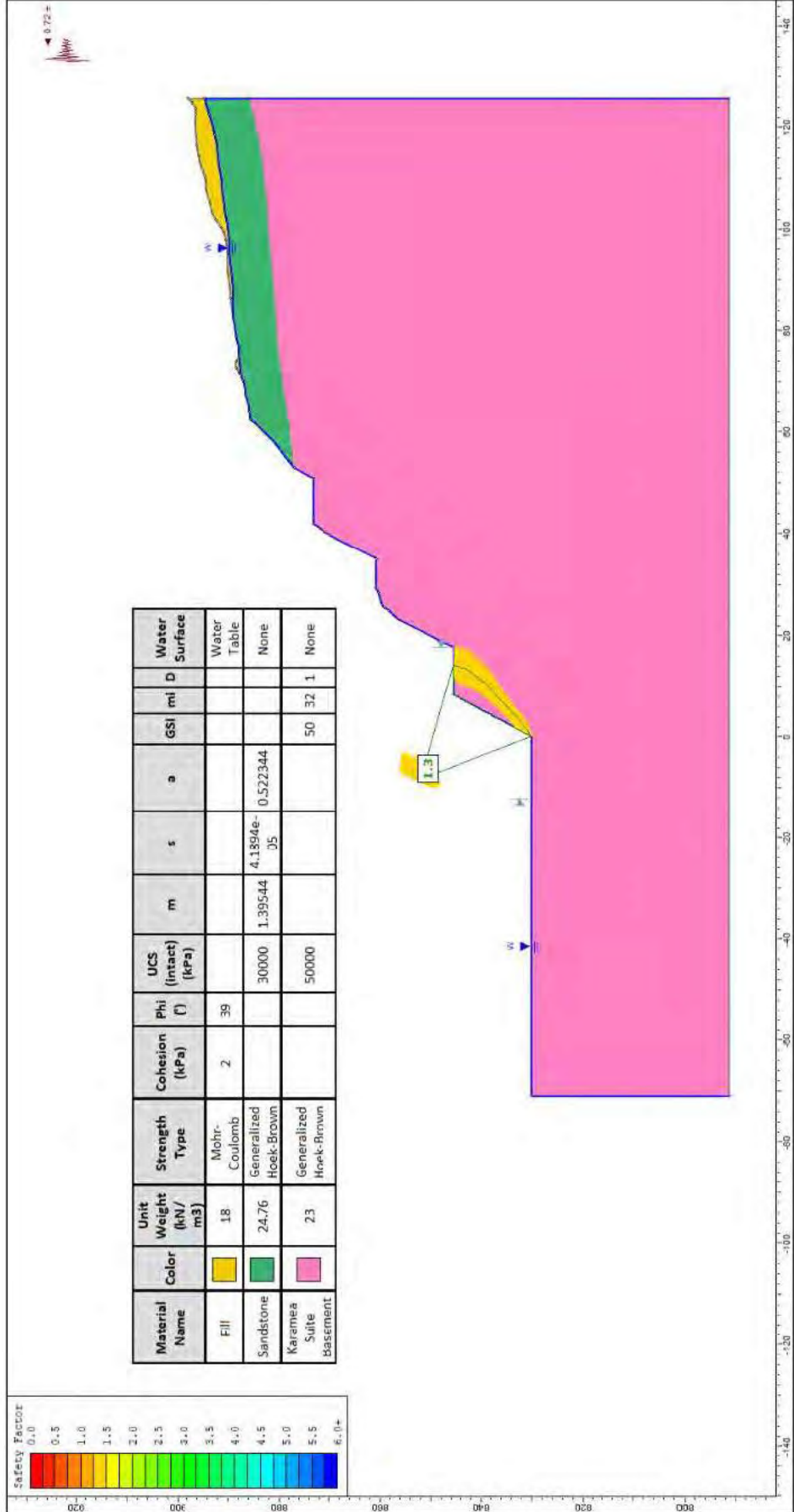


Figure D21: East Wall - Bench 3 - Seismic PGA - 0.72g.

A-Drive East Wall - Multiple Bench (1-2)

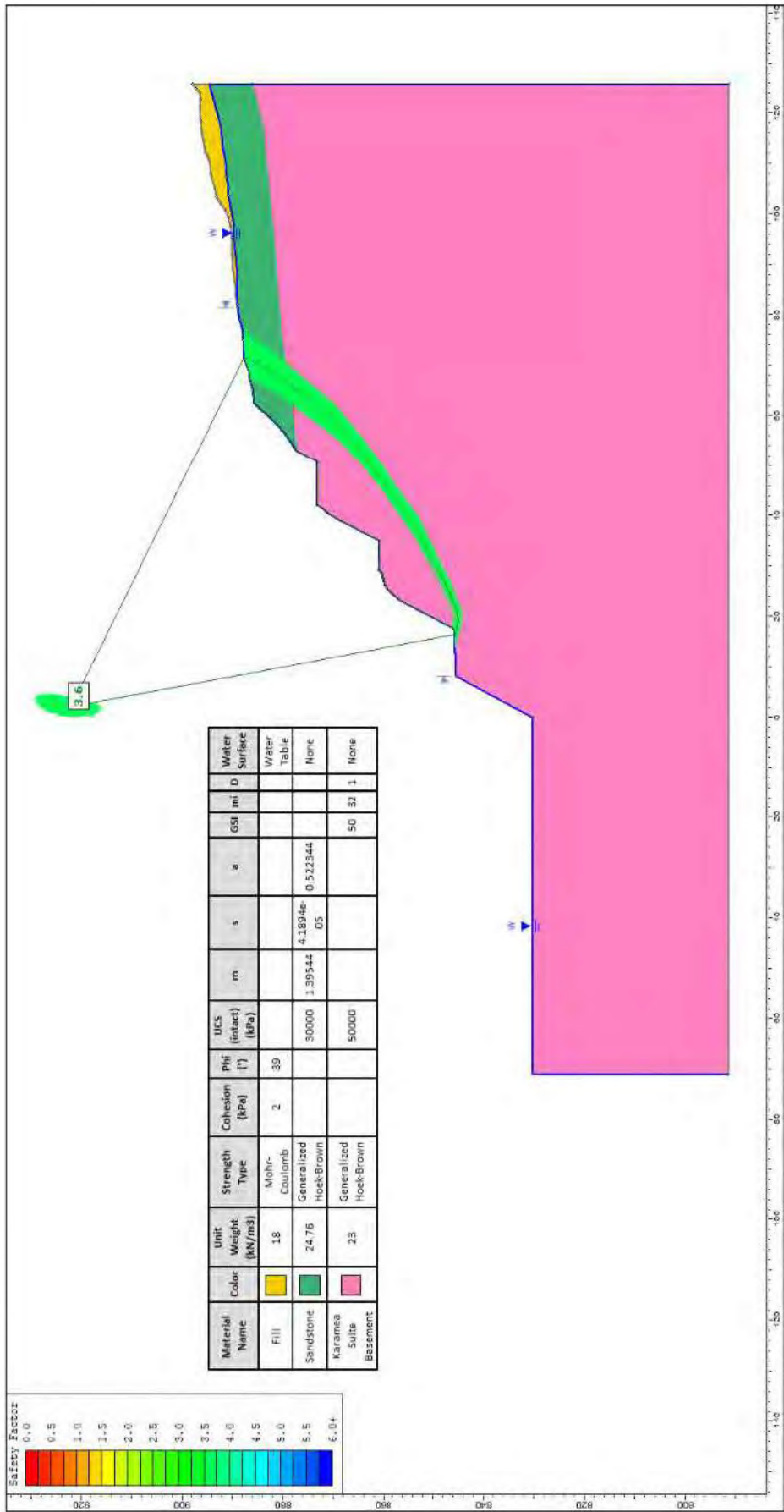


Figure D22: East Wall - Multiple Bench (1-2) - Static.

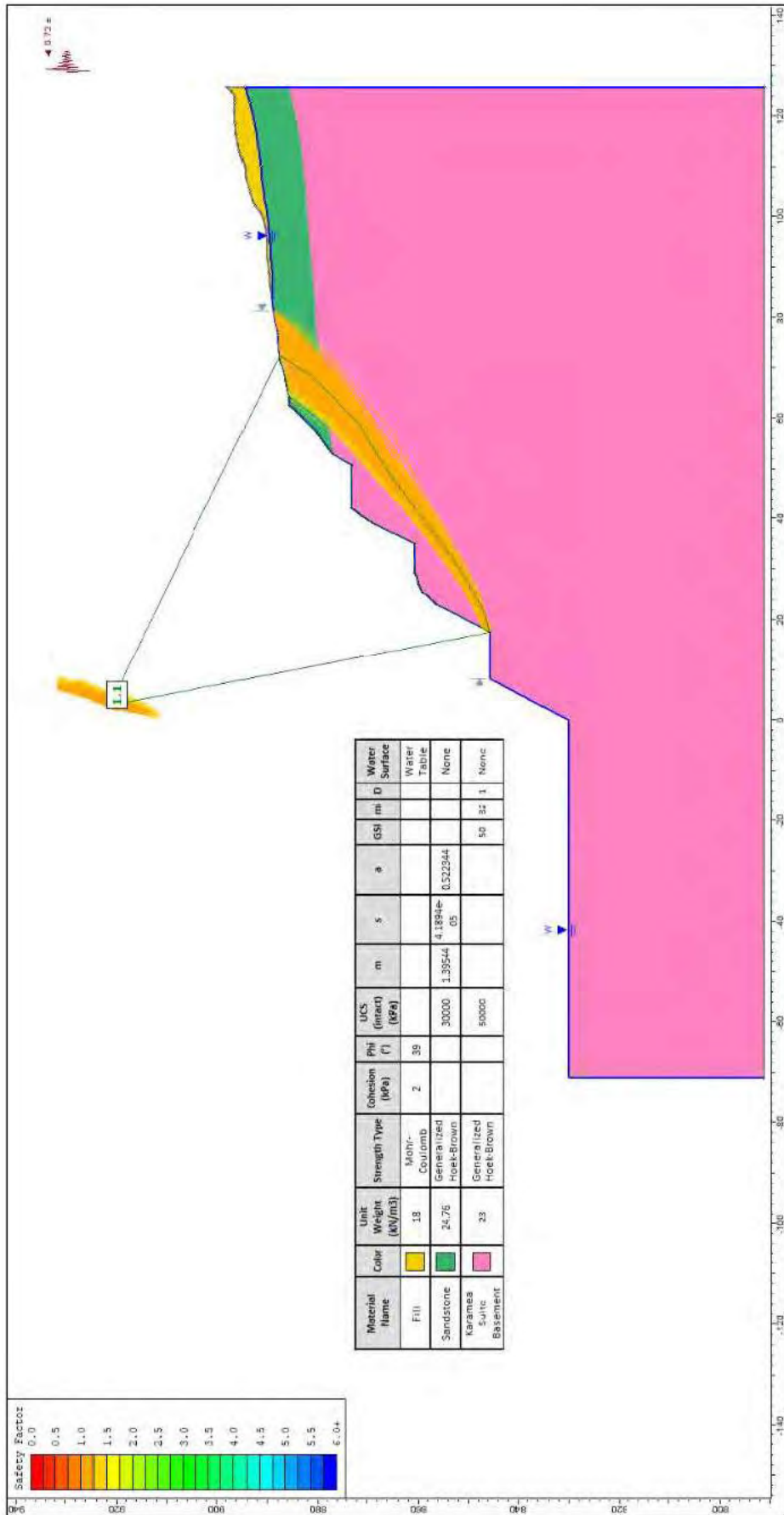


Figure D23: East Wall -Multiple Bench (1-2) - Seismic PGA - 0.72g.

A-Drive East Wall - Multiple Bench (2-3)

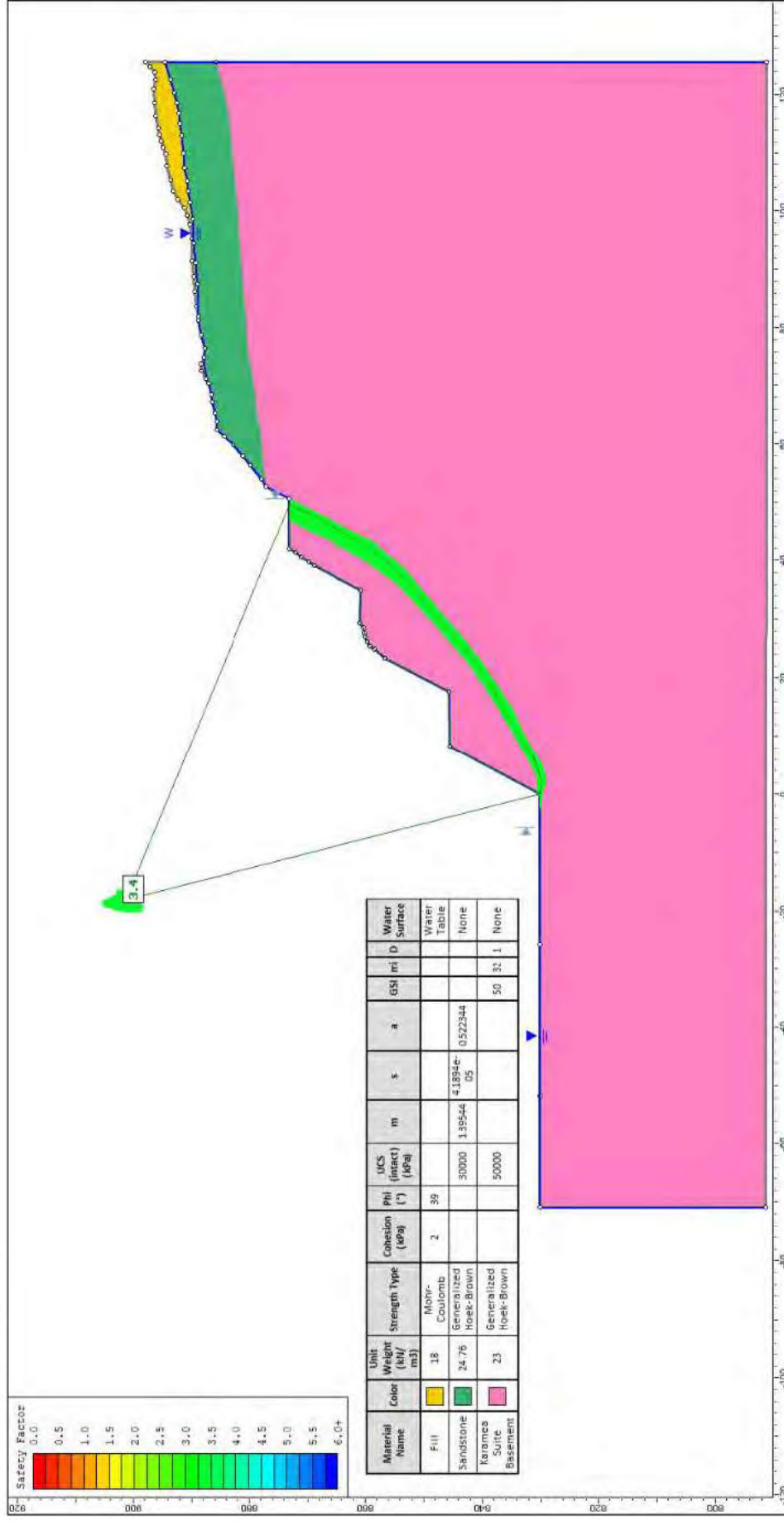


Figure D24: East Wall - Multiple Bench (2-3) - Static.

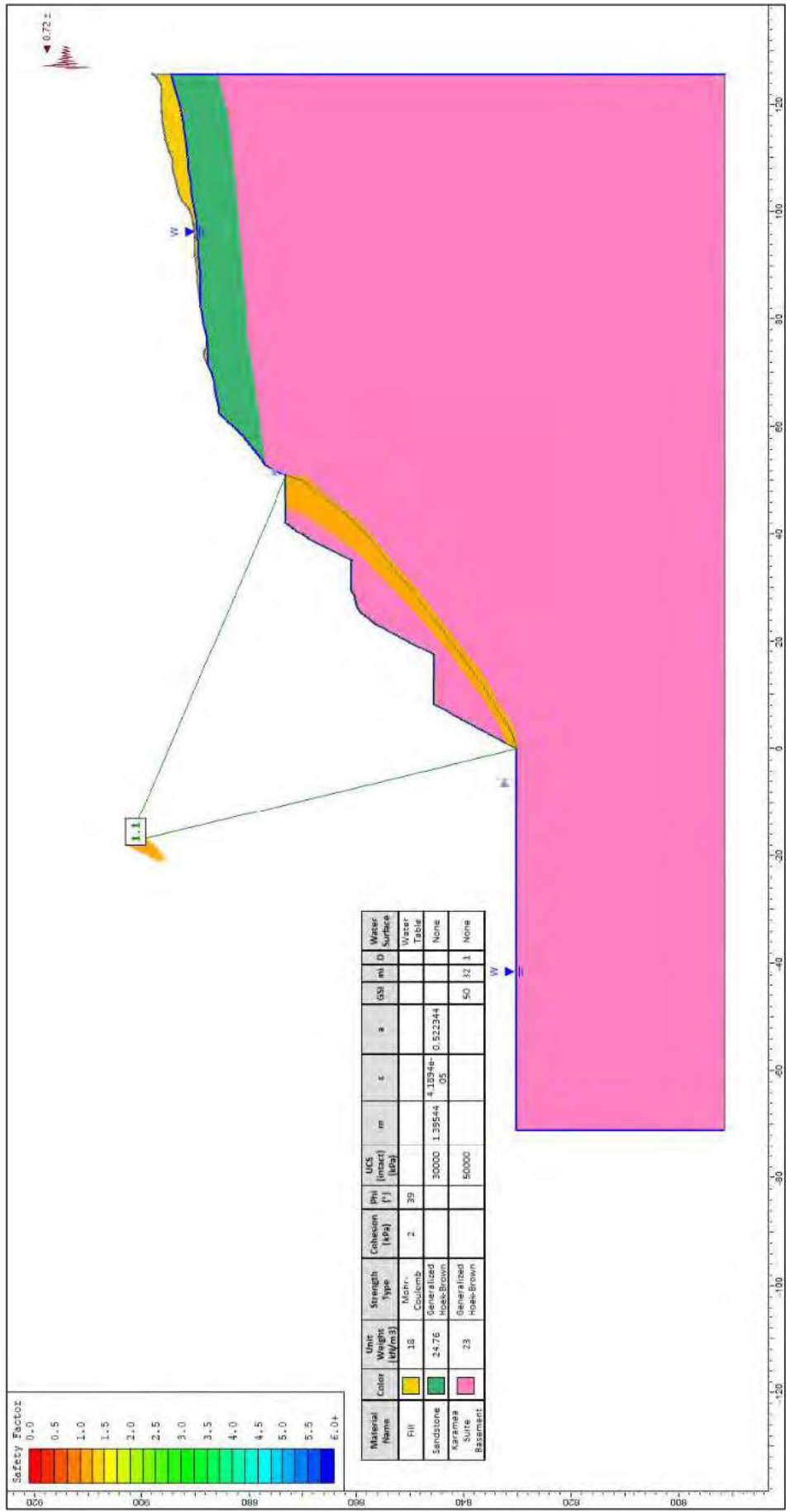


Figure D25: East Wall - Multiple Bench (2-3) - Seismic PGA - 0.72g.

A-Drive West Wall - Overall

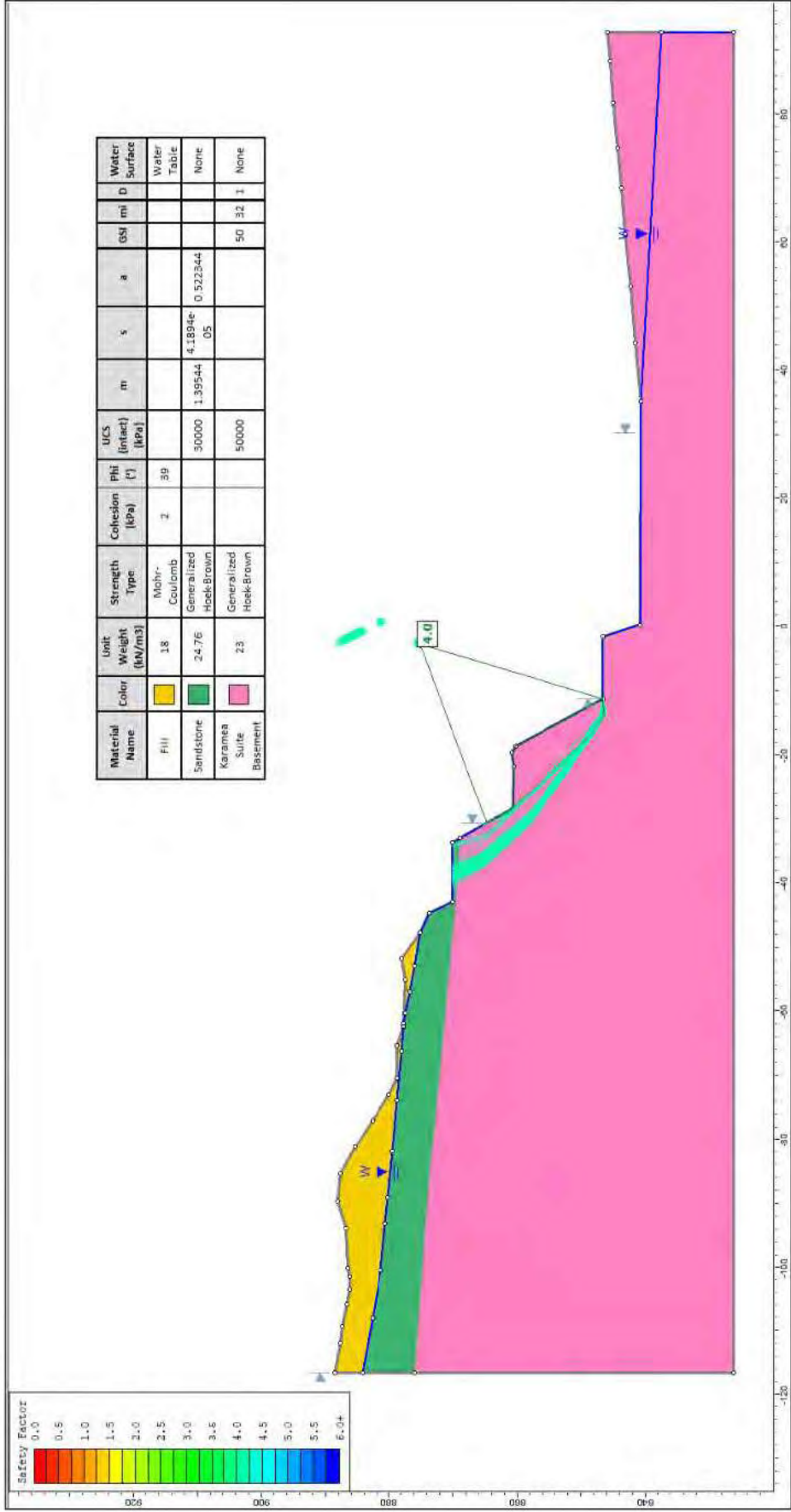


Figure D26: West Wall - Overall - Static.

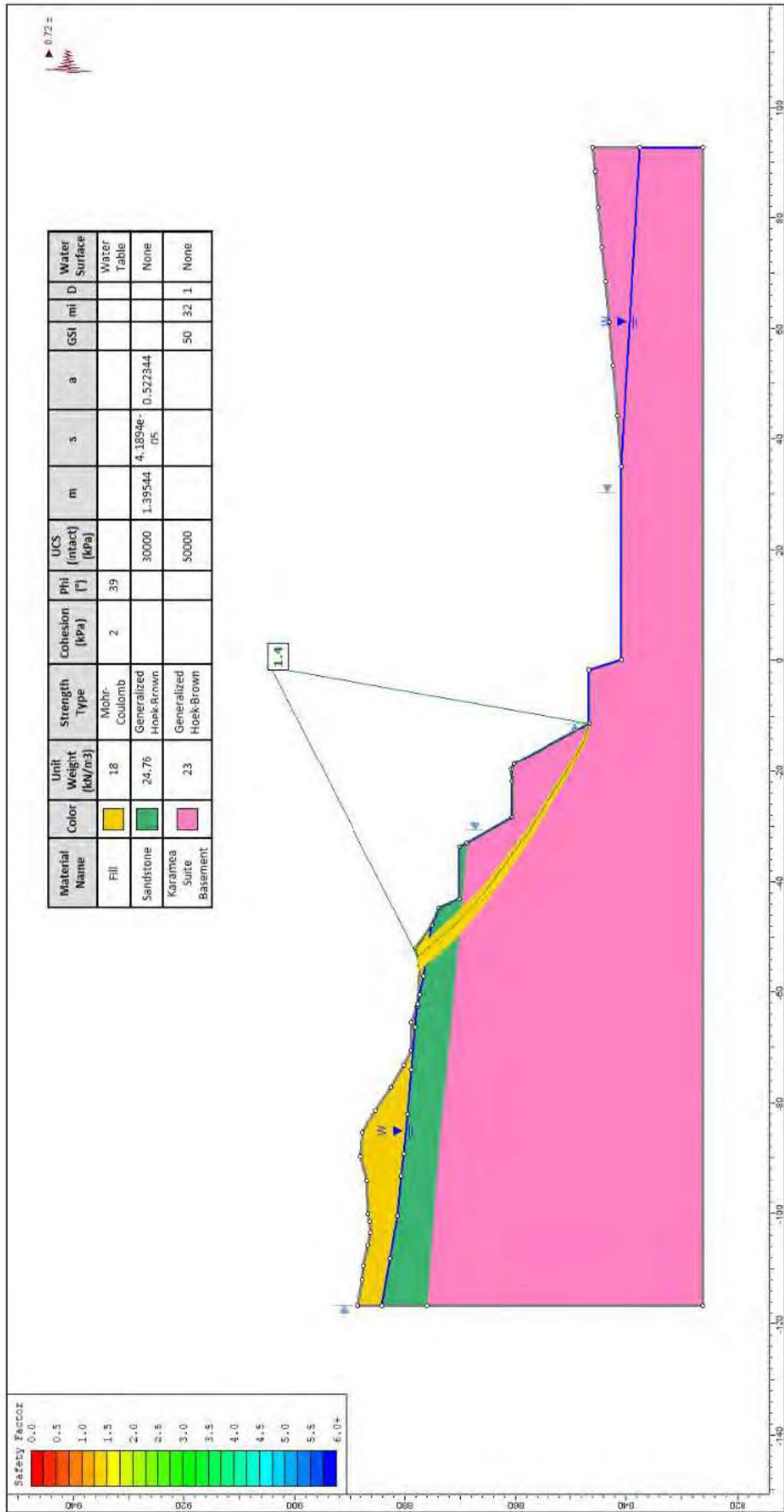


Figure D27: West Wall - Overall - Seismic - PGA - 0.72g.

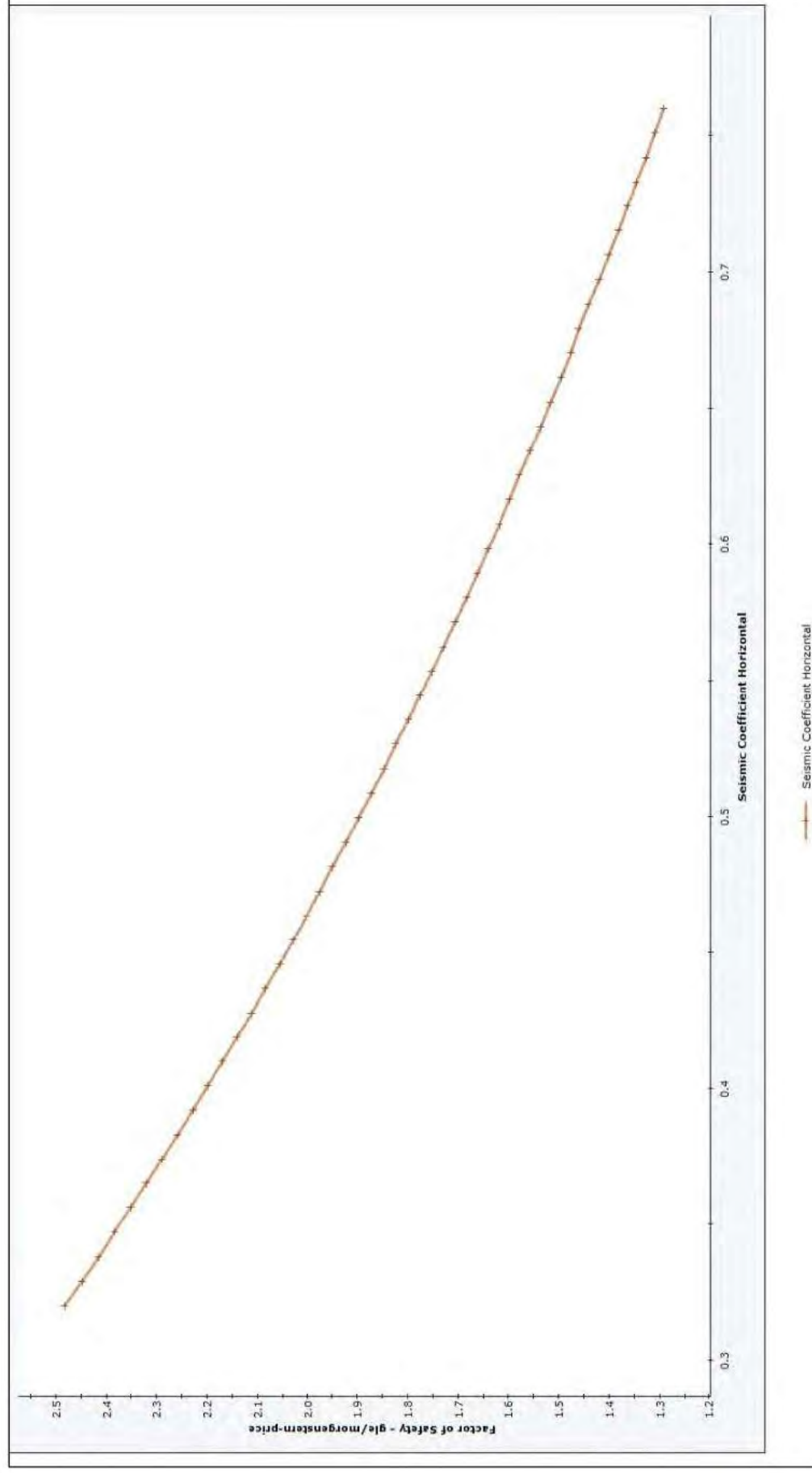
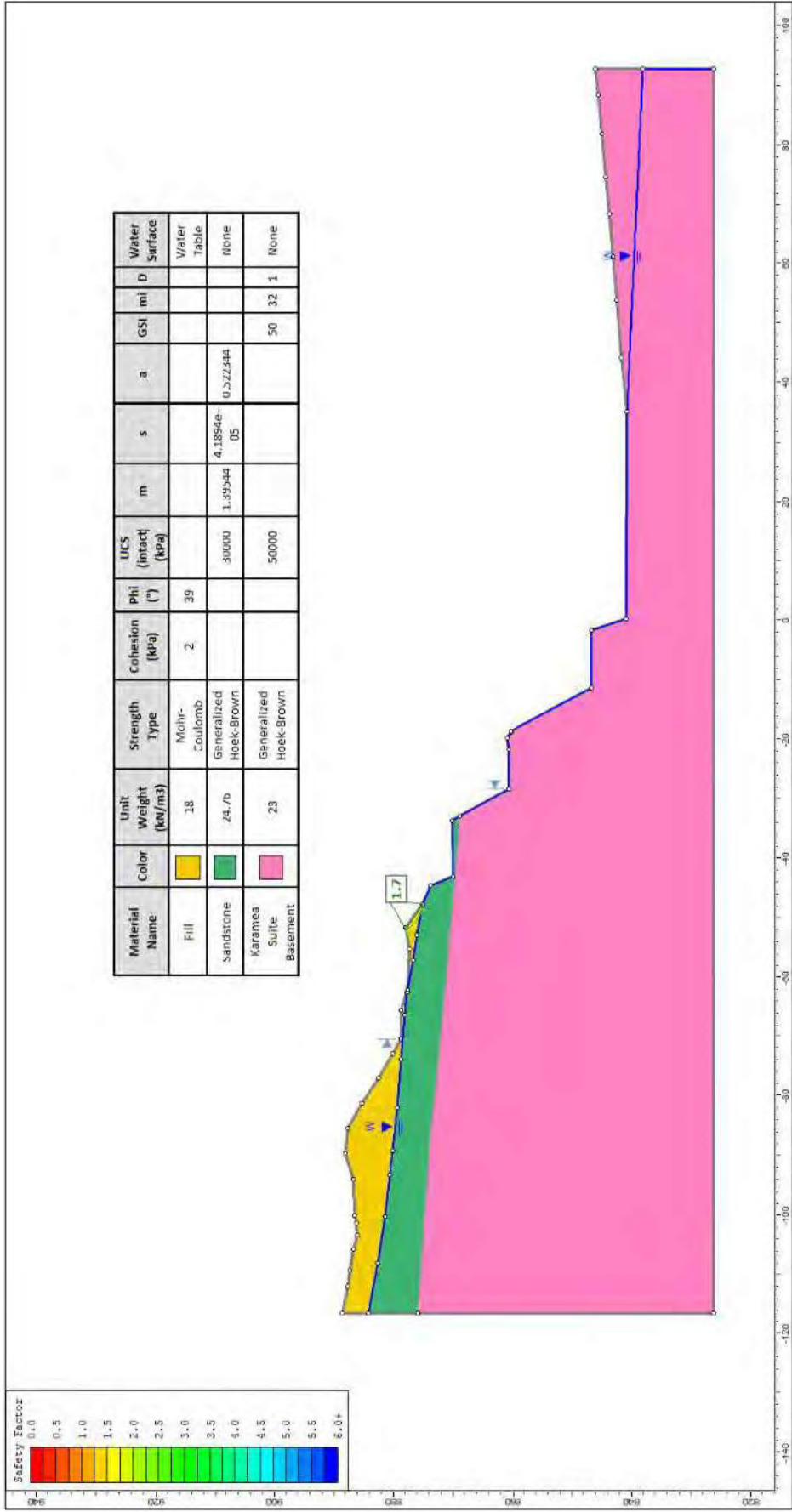
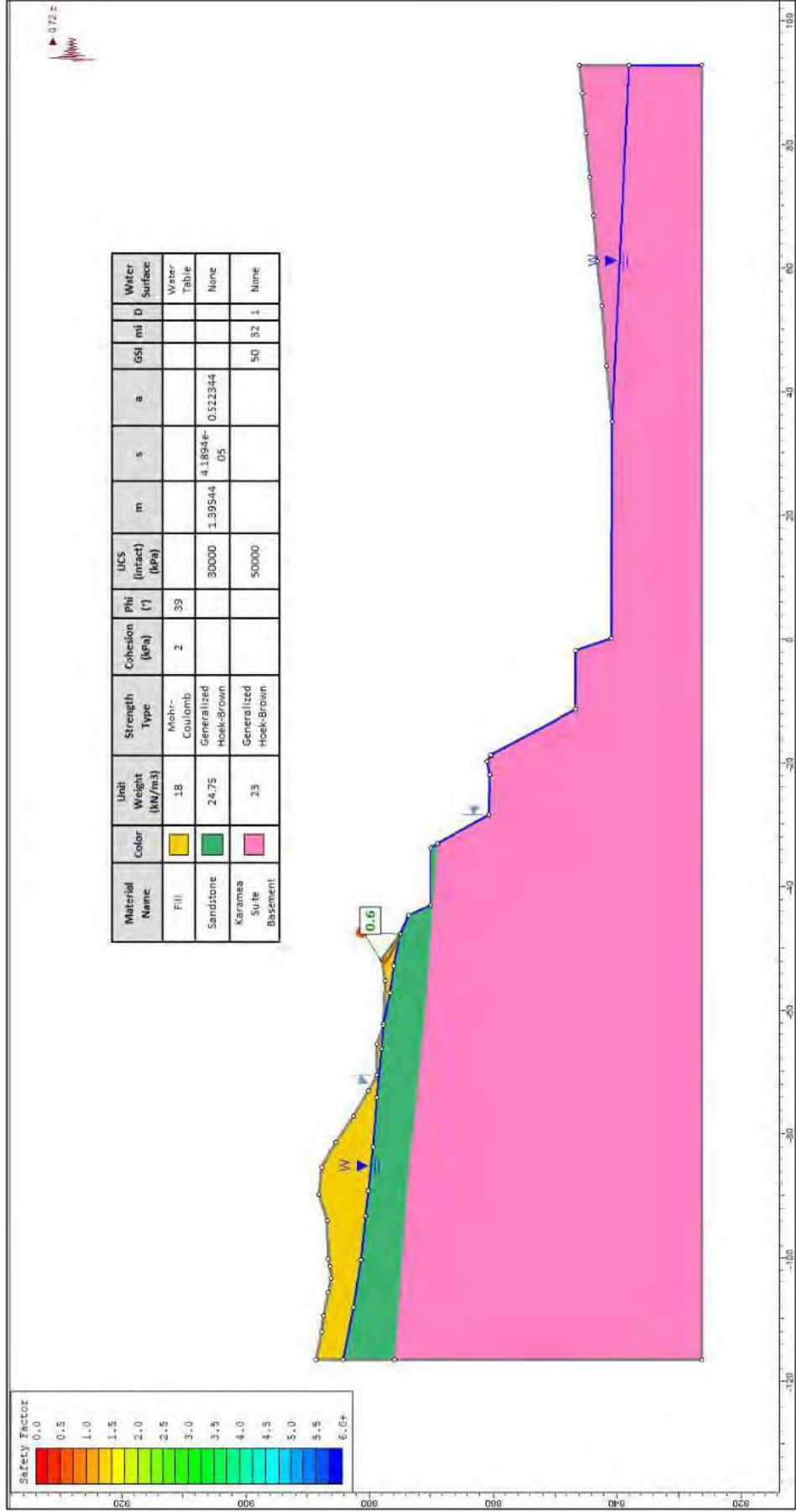


Figure D28: Sensitivity Analysis – West Wall - Overall Slope.

A-Drive West Wall - Bench 1





D30: West Wall - Bench 1 - Seismic - PGA - 0.72g.

Figure

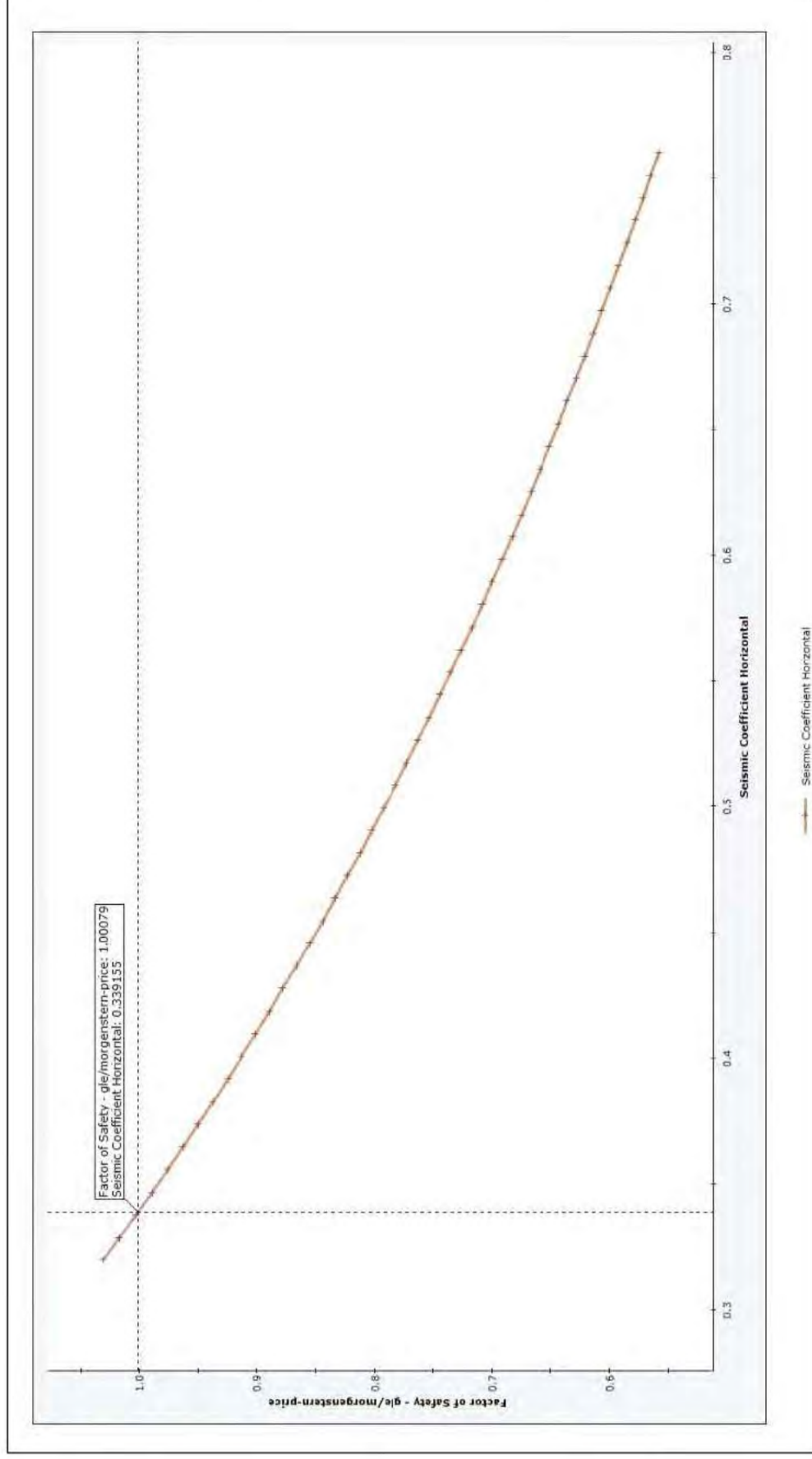


Figure D31: Sensitivity Analysis – Bench 1 – Failure in Fill.

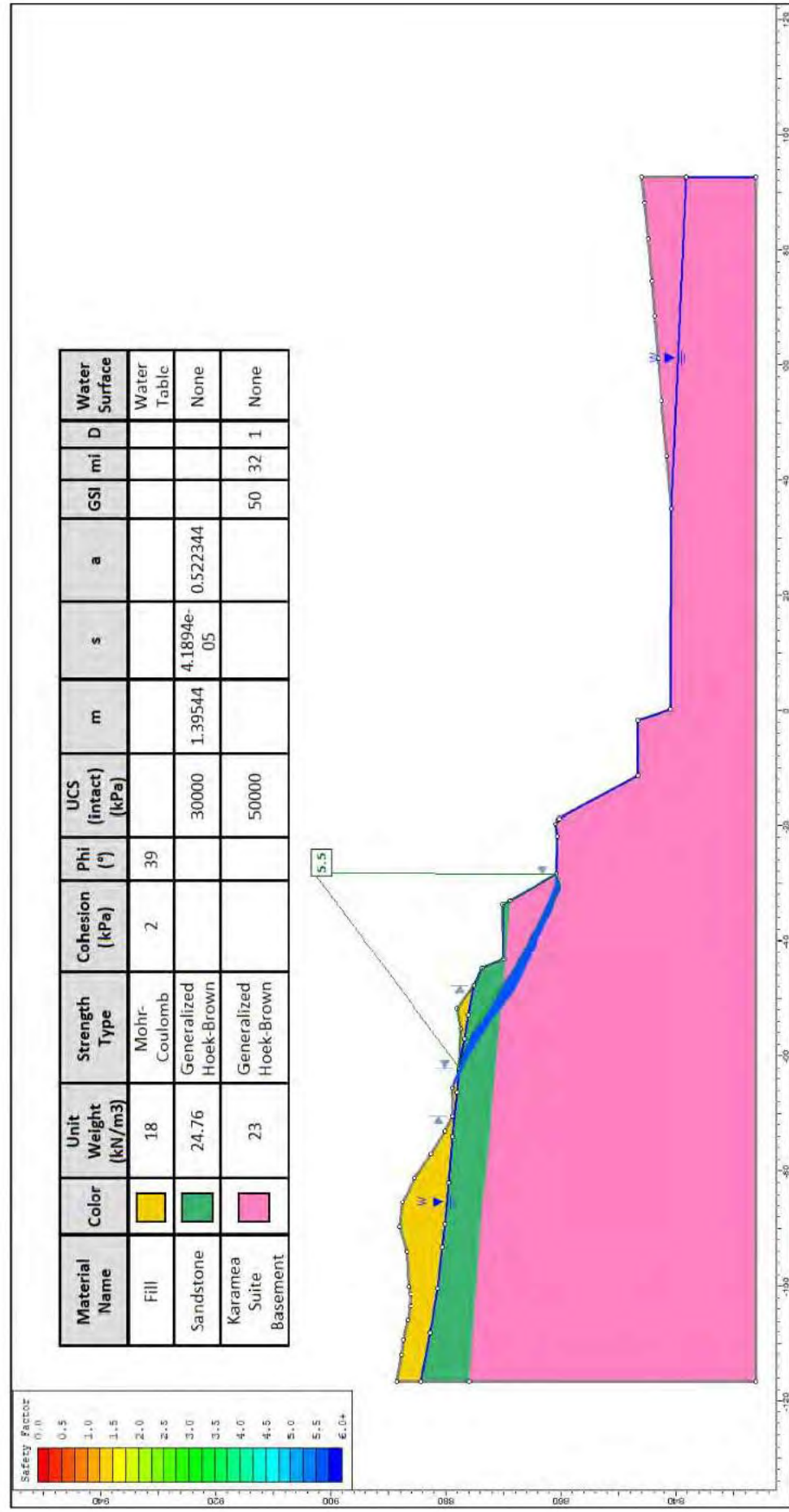


Figure D32: West Wall (Shallow Failure in Fill Ignored) — Bench 1 — Static

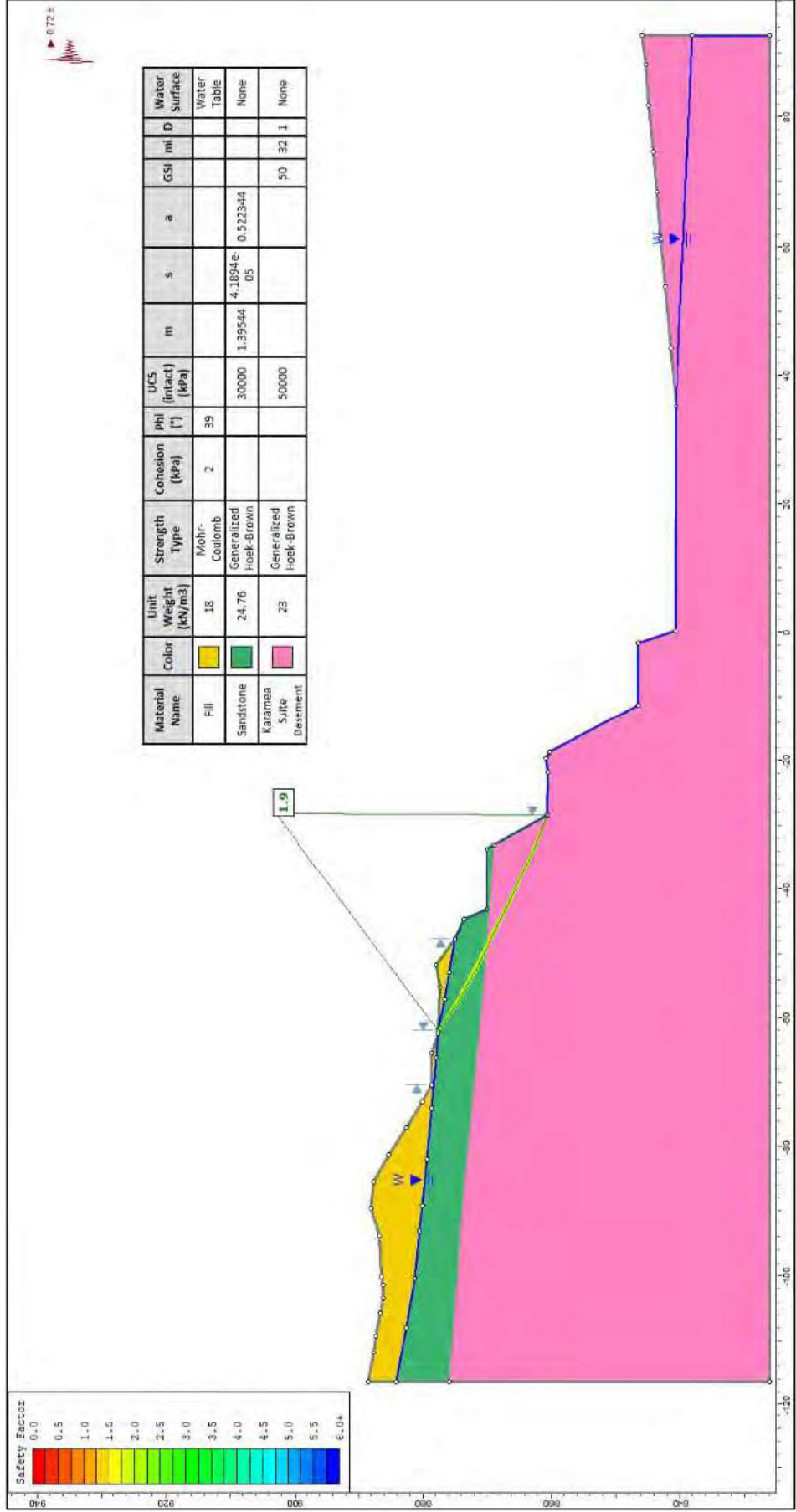


Figure D33: West Wall (Shallow Failure in Fill Ignored) - Bench 1 - Seismic - PGA - 0.72g.

A-Drive West Wall - Bench 2

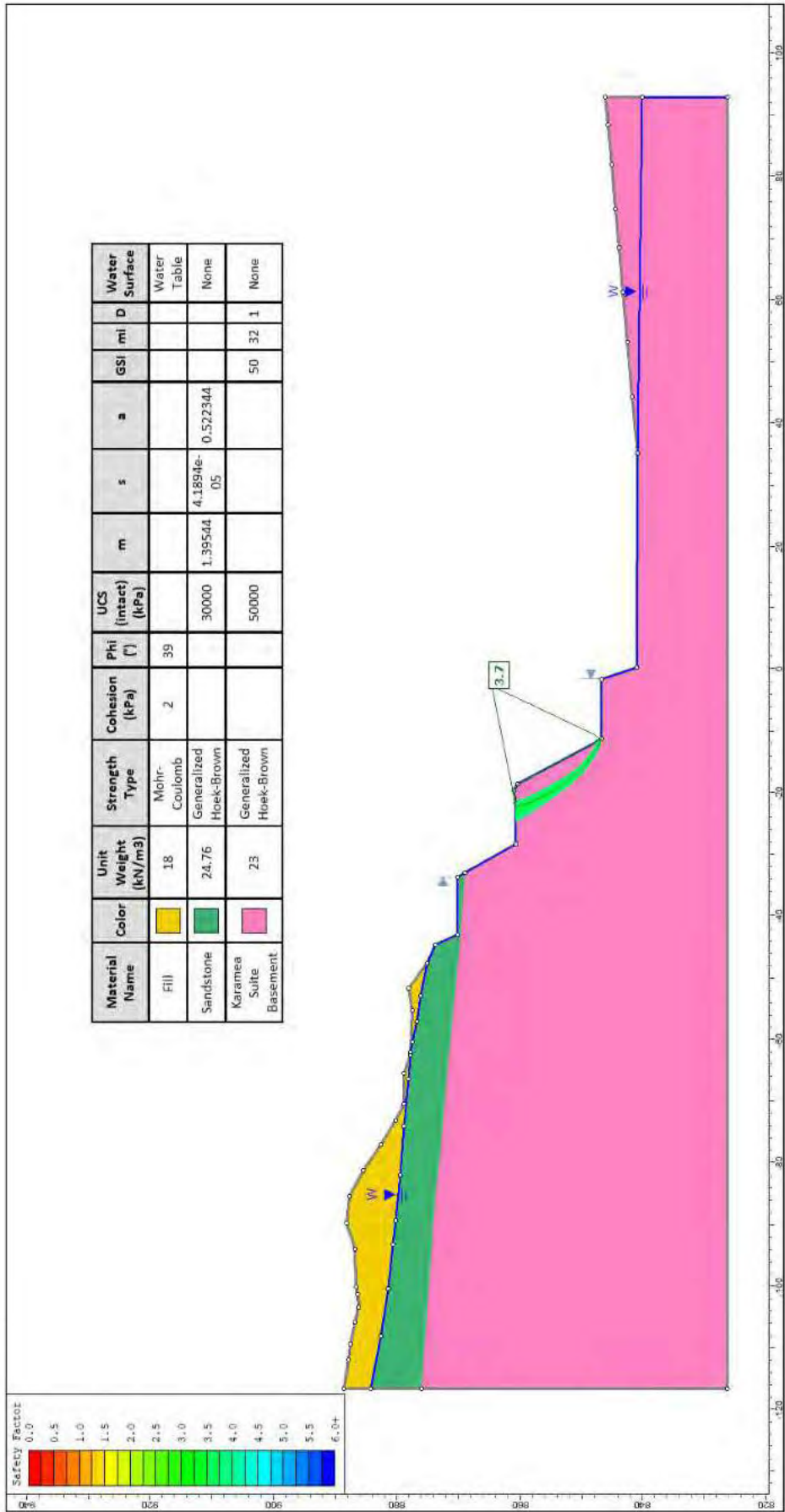


Figure D34: West Wall - Bench 2 - Static.

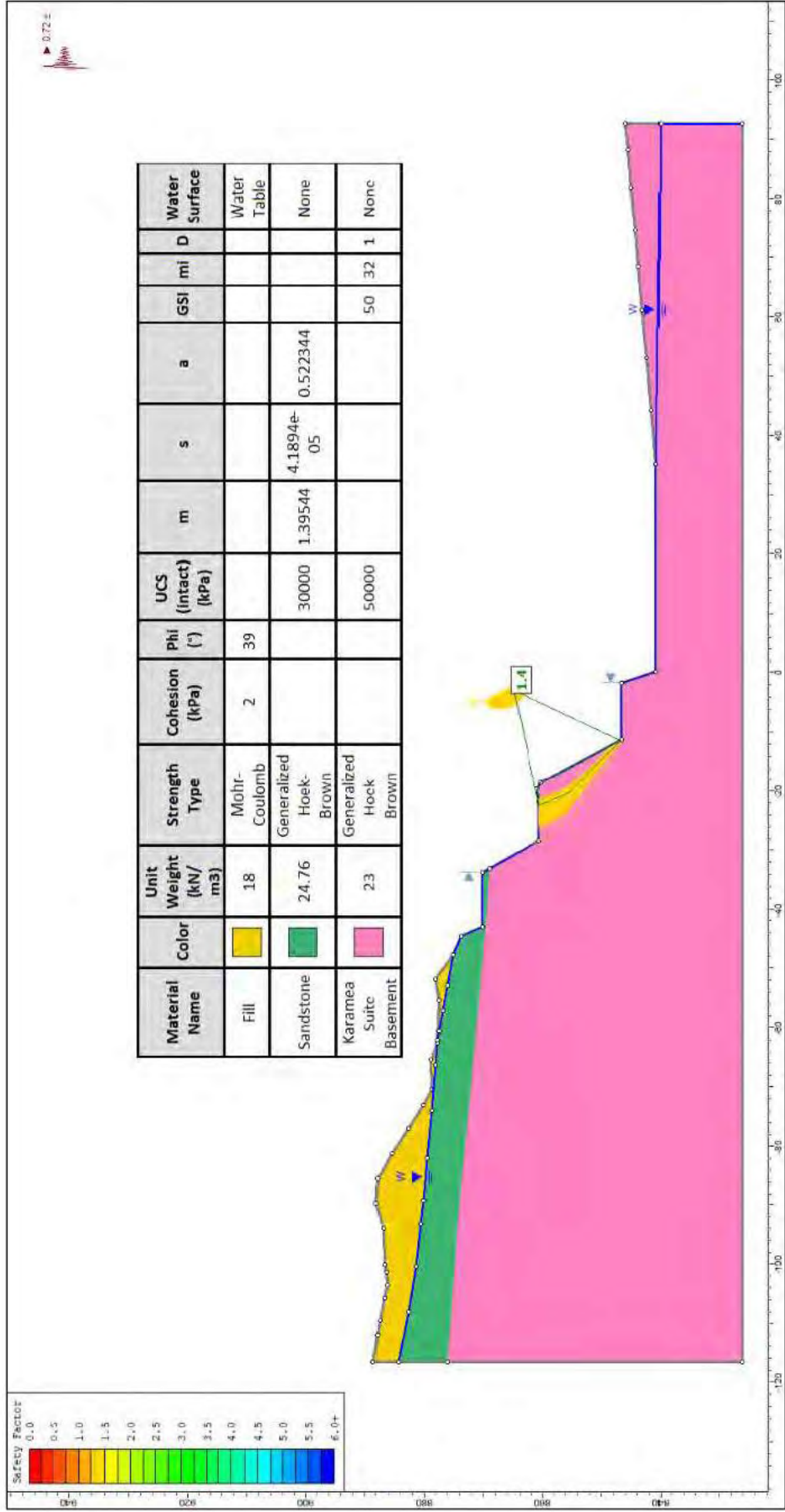


Figure D35: West Wall Existing – Bench 2 – Seismic – PGA – 0.72g.

A-Drive West Wall - Bench 3

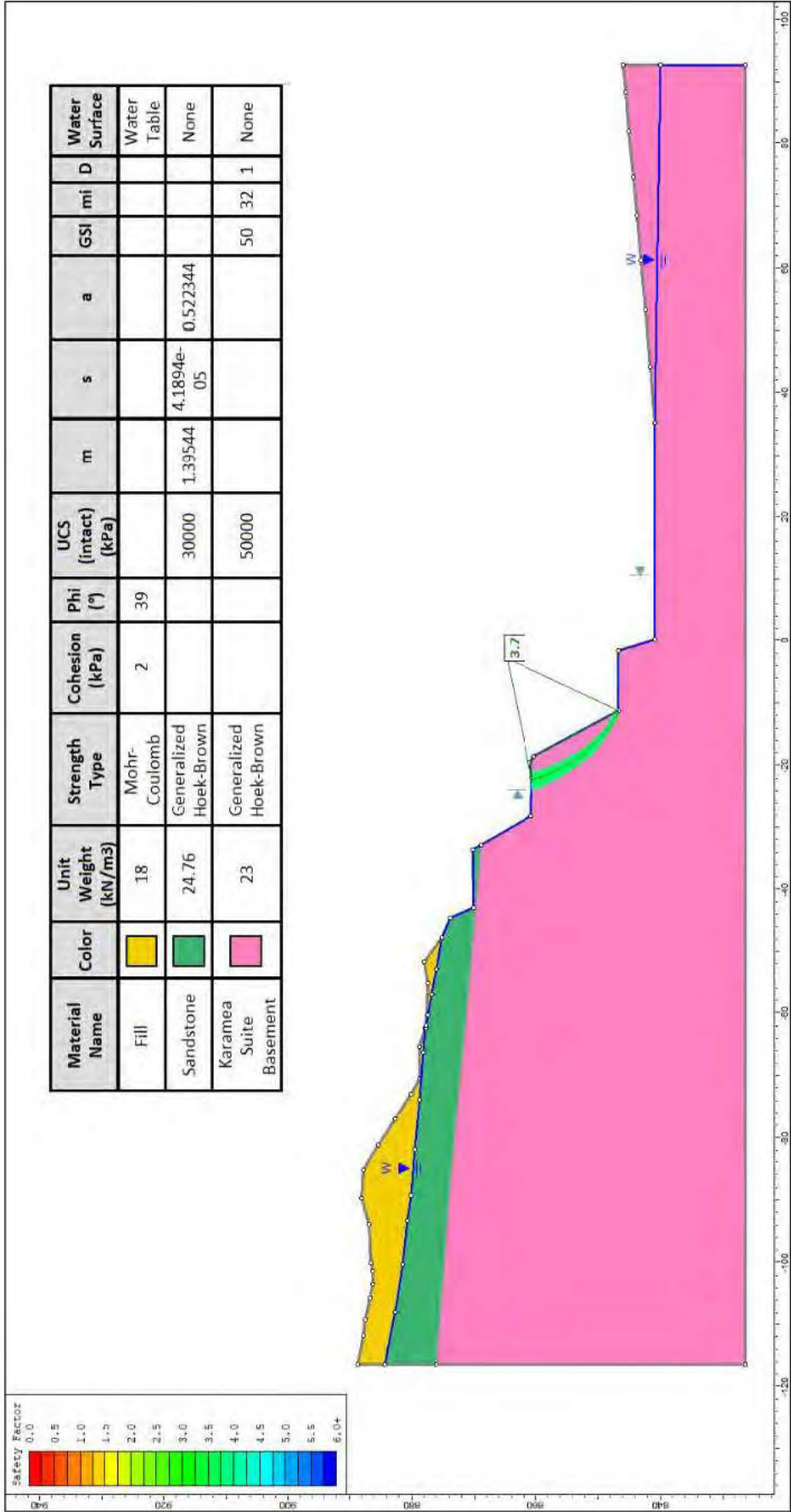


Figure D36: West Wall - Bench 3 - Static

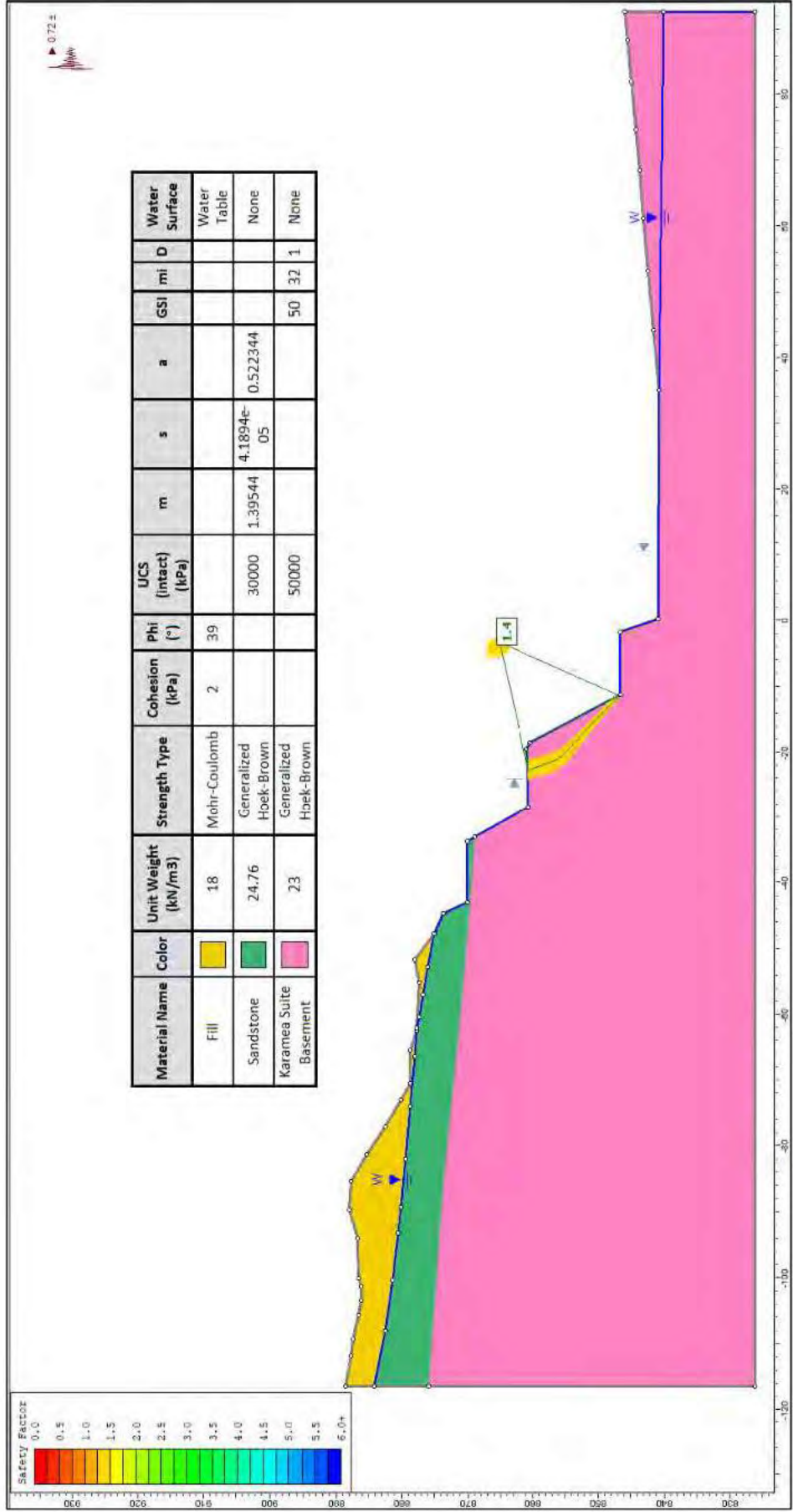


Figure D37: West Wall - Bench 3 - Seismic - PGA - 0.72g.

A-Drive West Wall - Multiple (1-2)

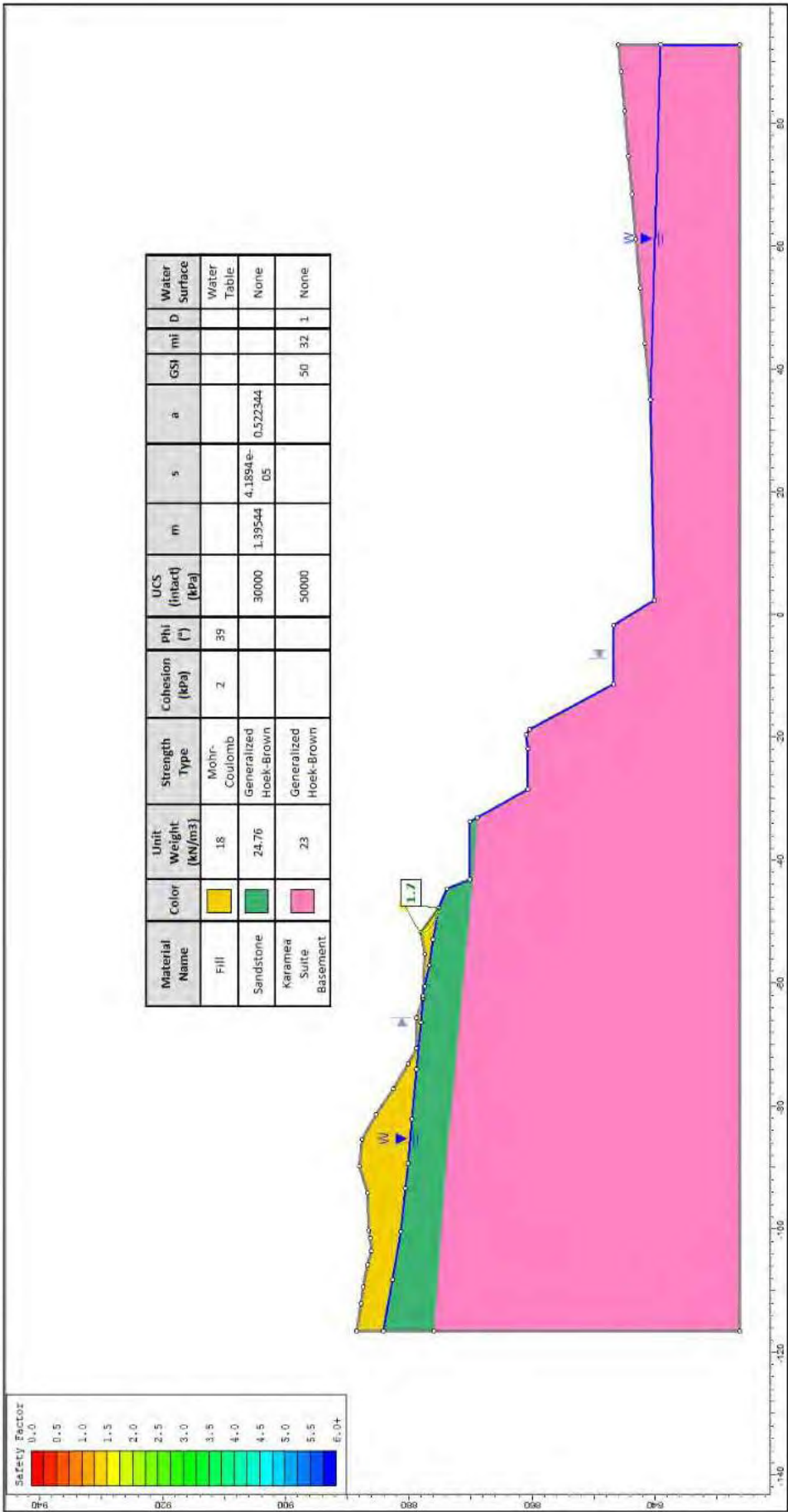




Figure D39: West Wall – Multiple Bench (1-2) – Seismic – PGA – 0.72g.

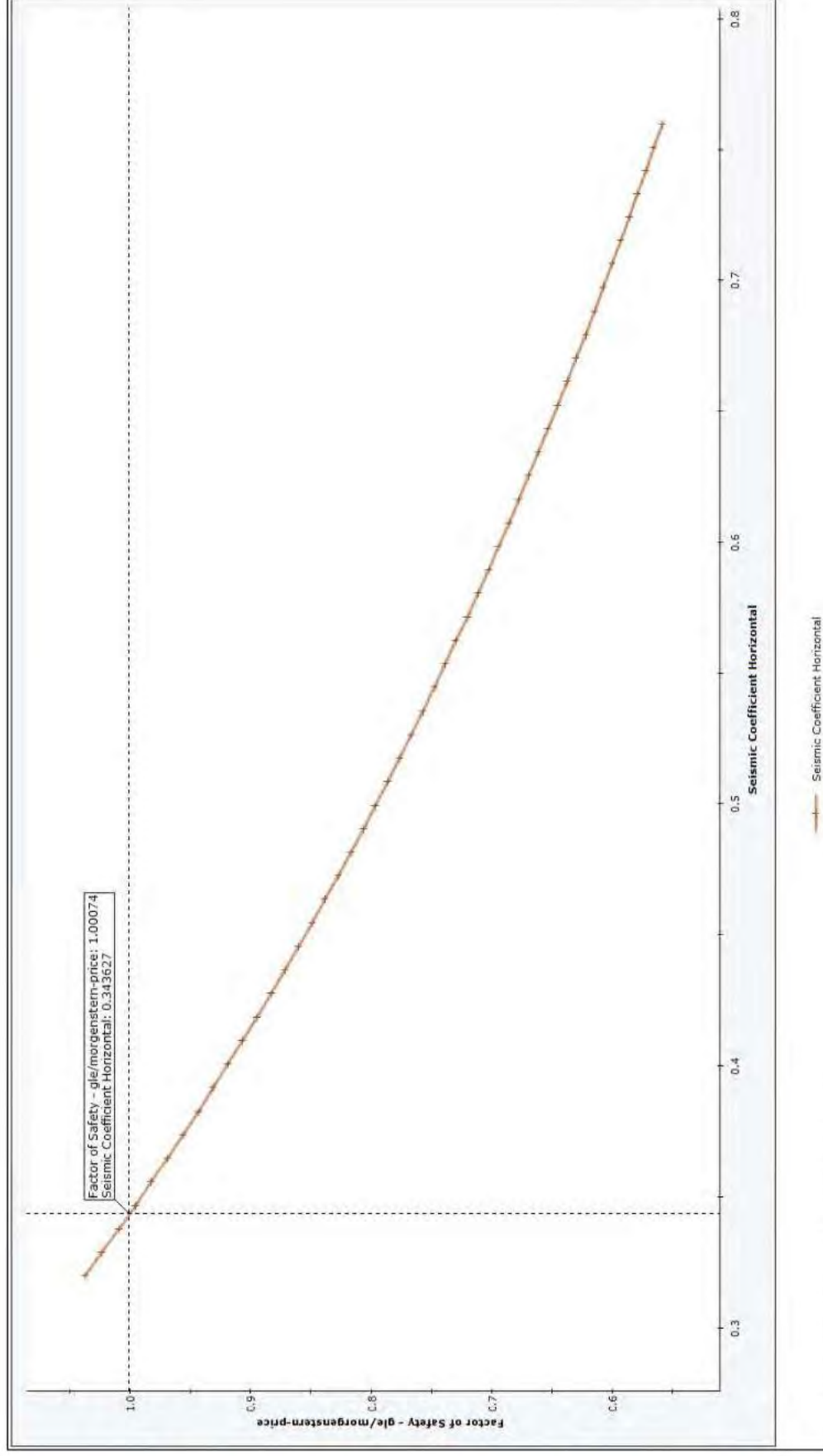


Figure D40: Sensitivity Analysis- Multiple 1-2 - Failure in Fill.

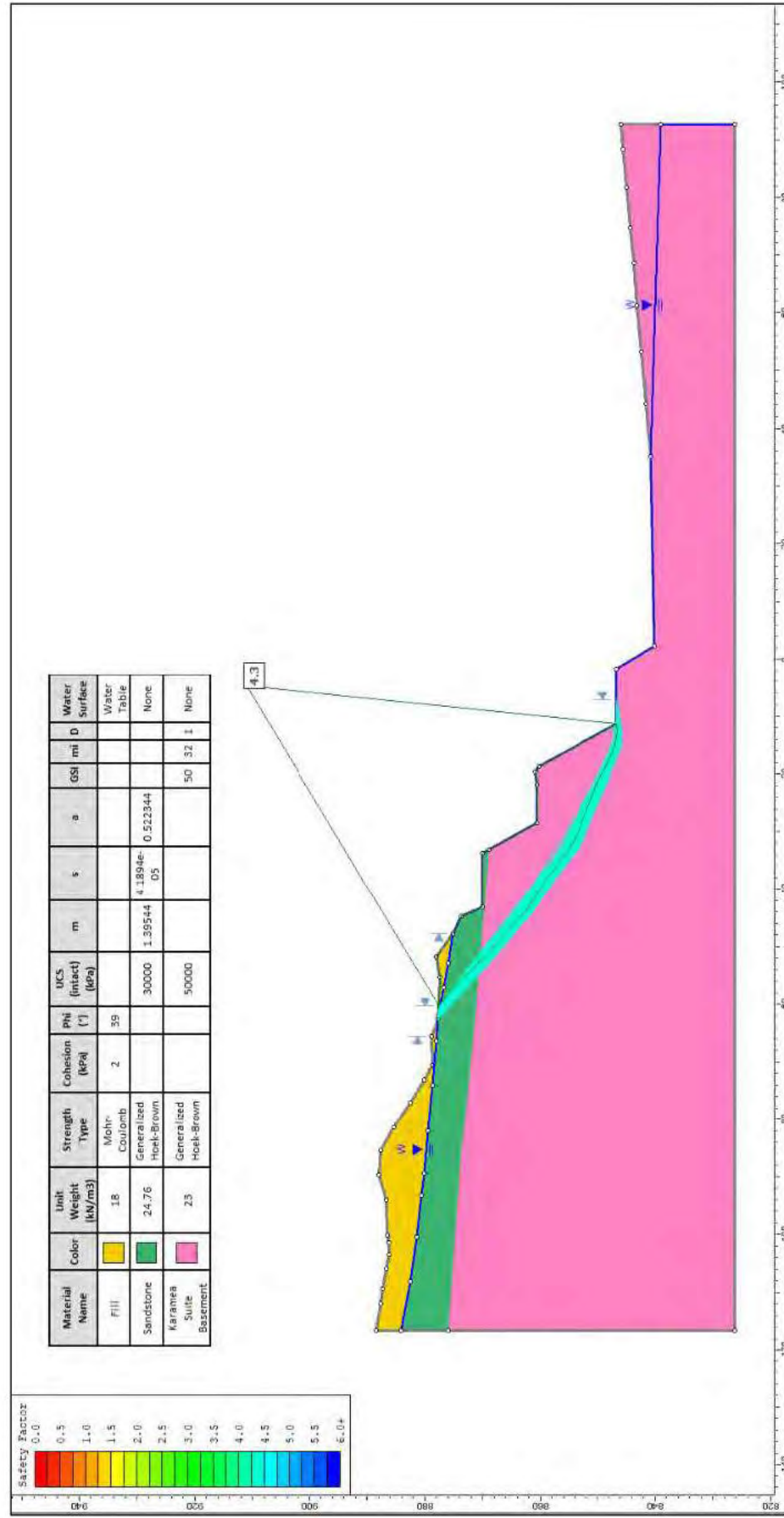


Figure D41: West Wall – Multiple Bench (1-2) (Shallow Failures Excluded) – Static

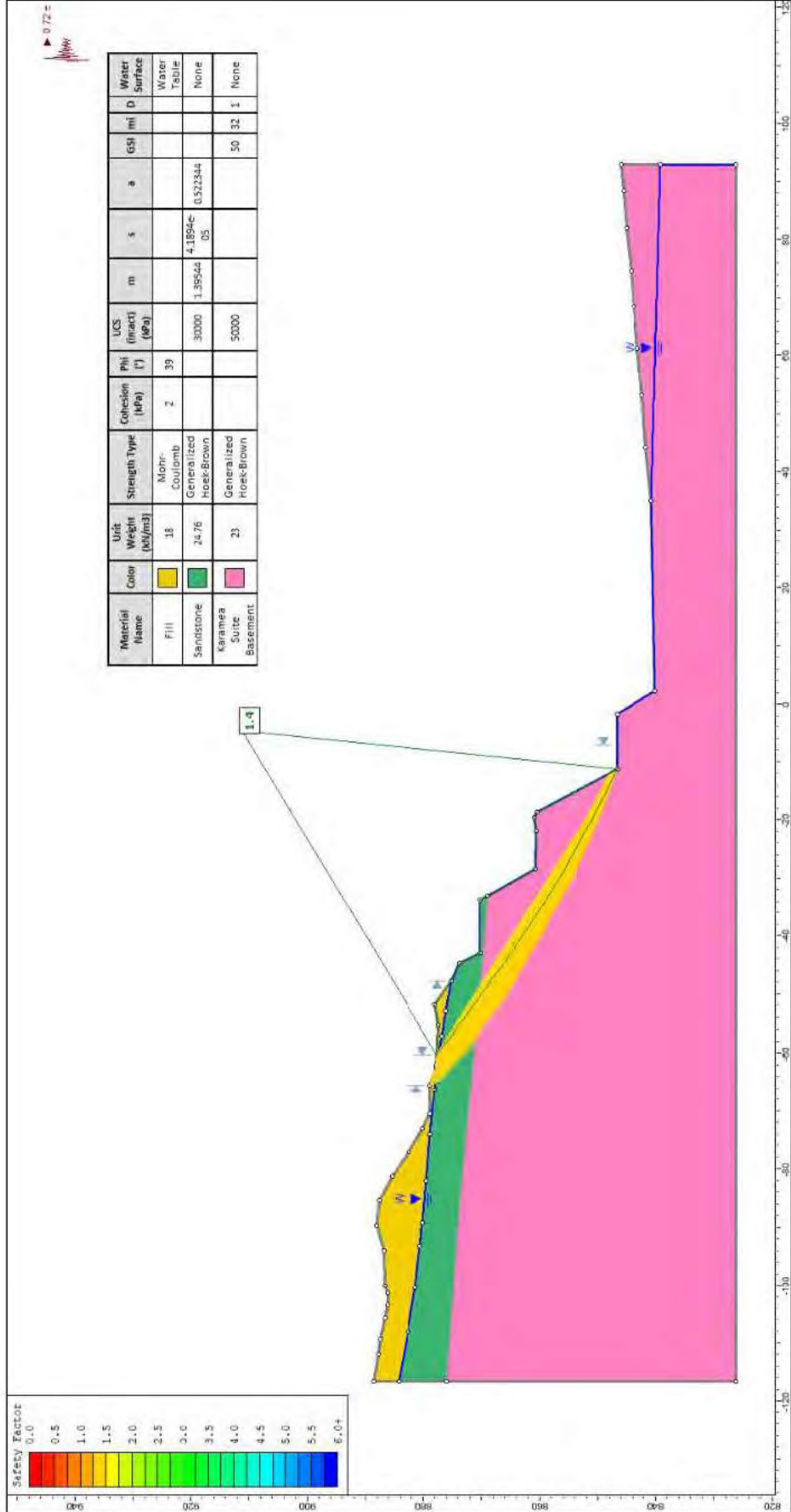


Figure D42: West Wall - Multiple Bench (1-2) (Shallow Failures Excluded) - Seismic - PGA - 0.72g.

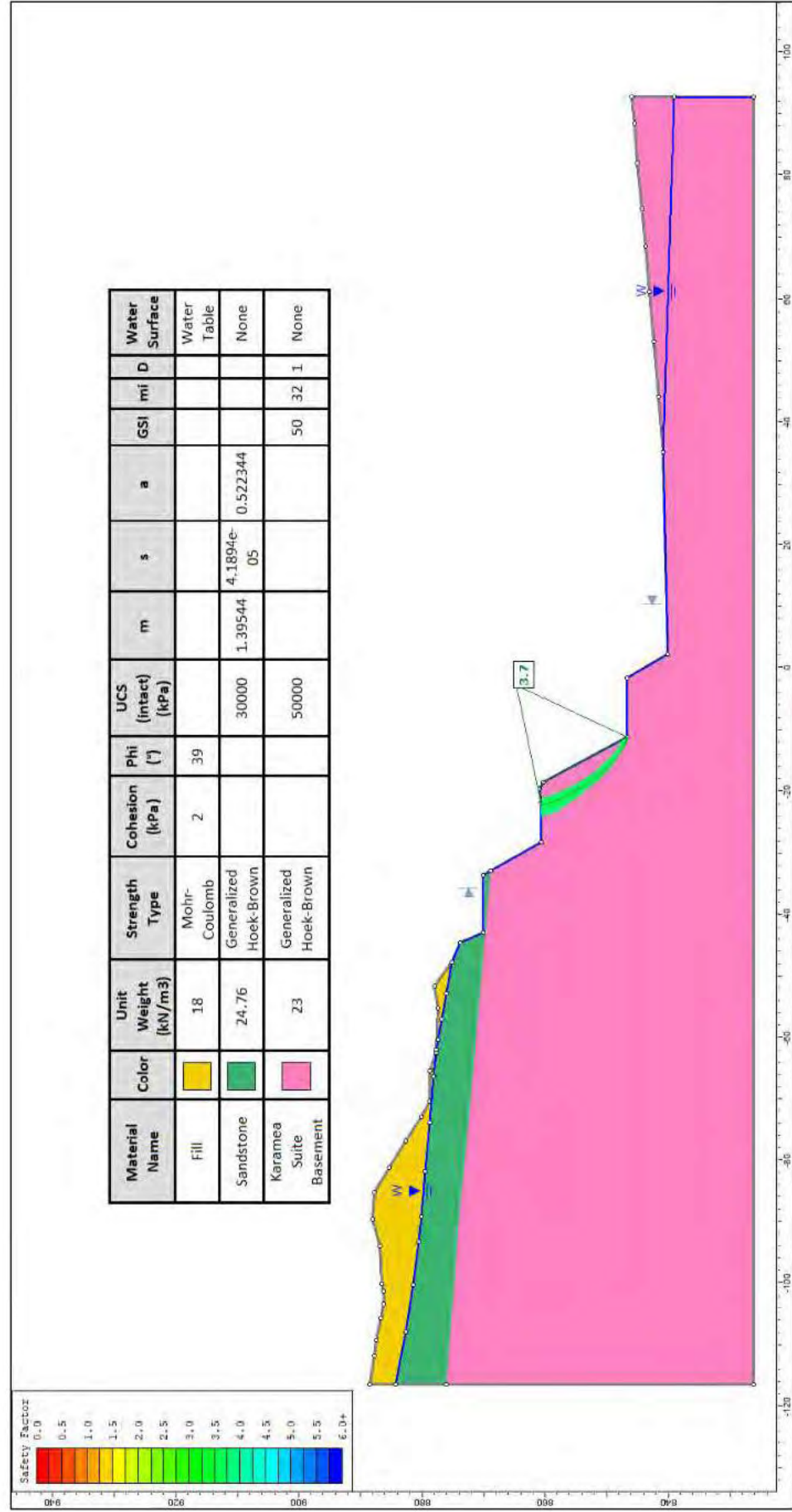


Figure D43: West Wall – Multiple Bench (2-3) – Static.

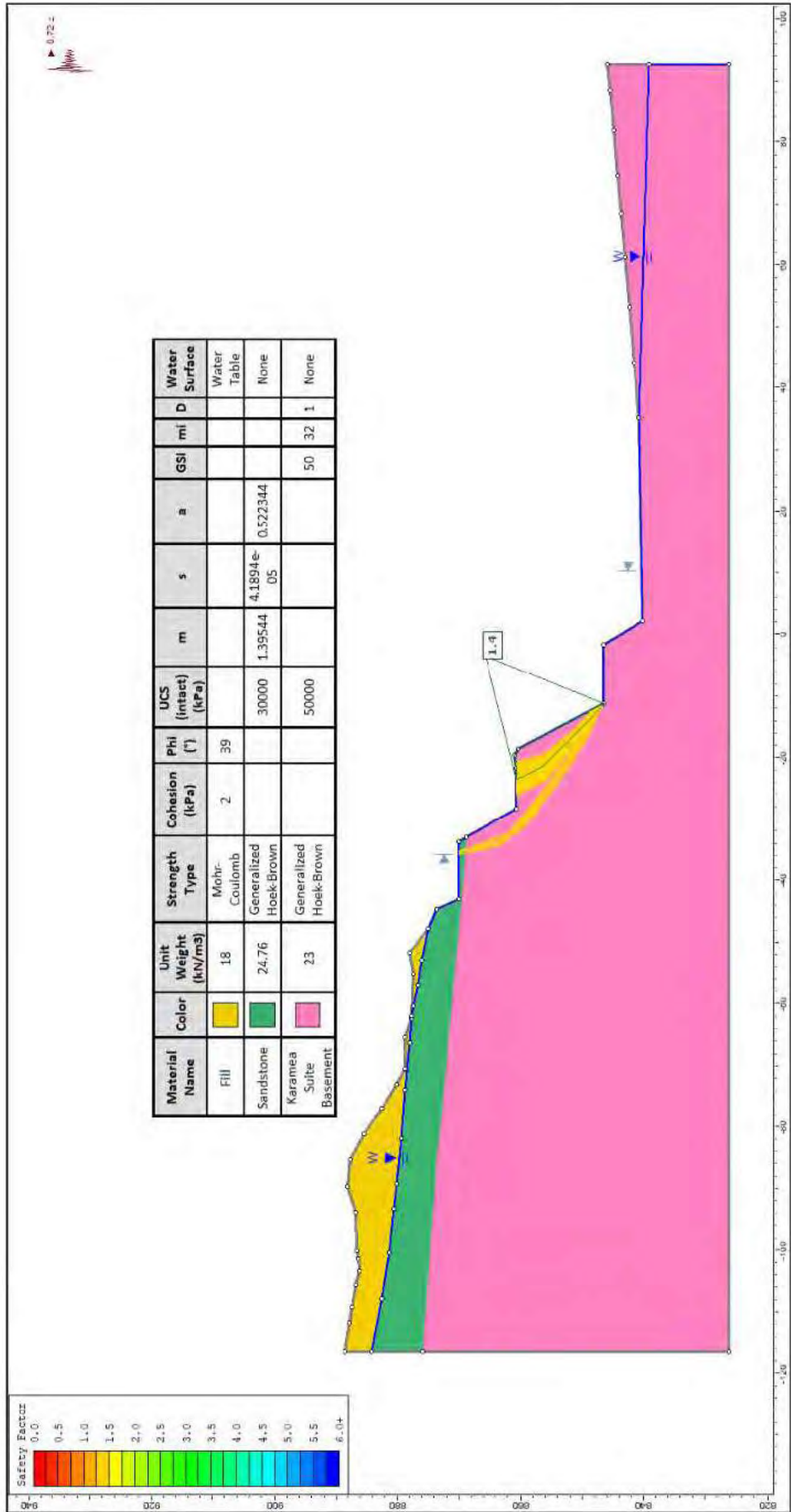


Figure D44: West Wall - Multiple Bench (2-3) - Seismic - PGA - 0.72g.

South Wall - Overall

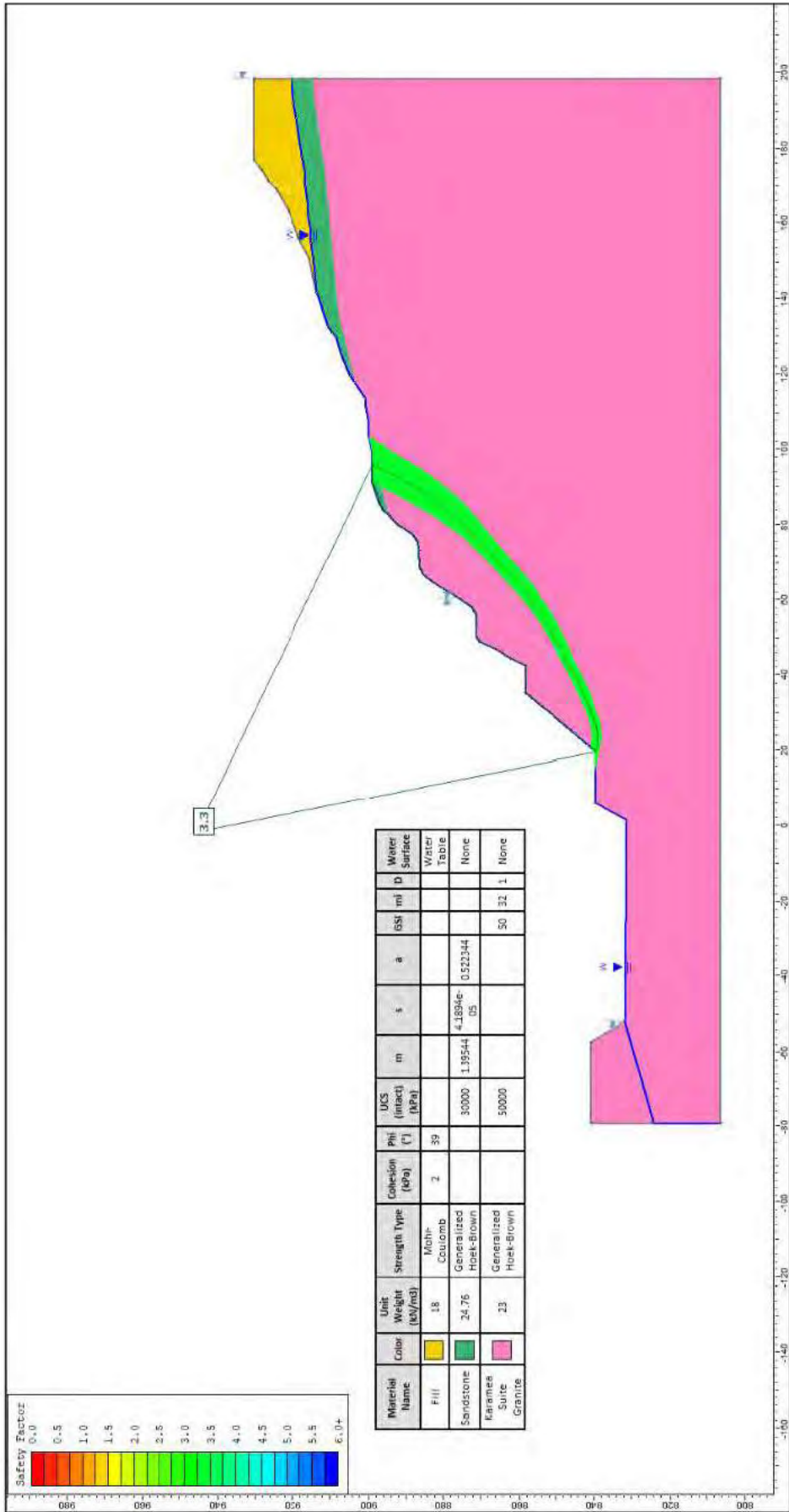


Figure D45: South Wall - Overall - Static.

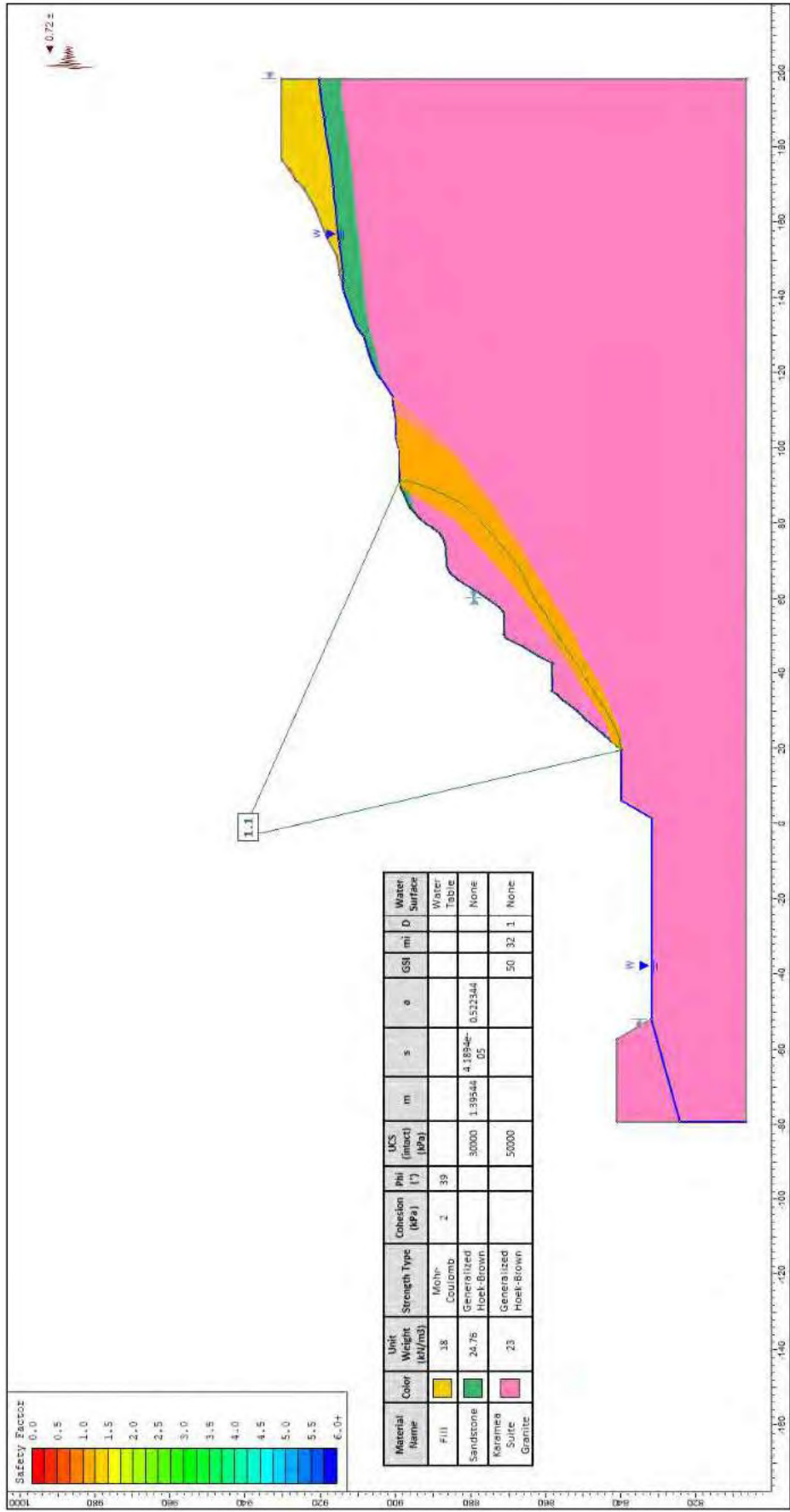


Figure D46: South Wall - Overall - Seismic - PGA - 0.72g.

A-Drive South Wall - Bench 1

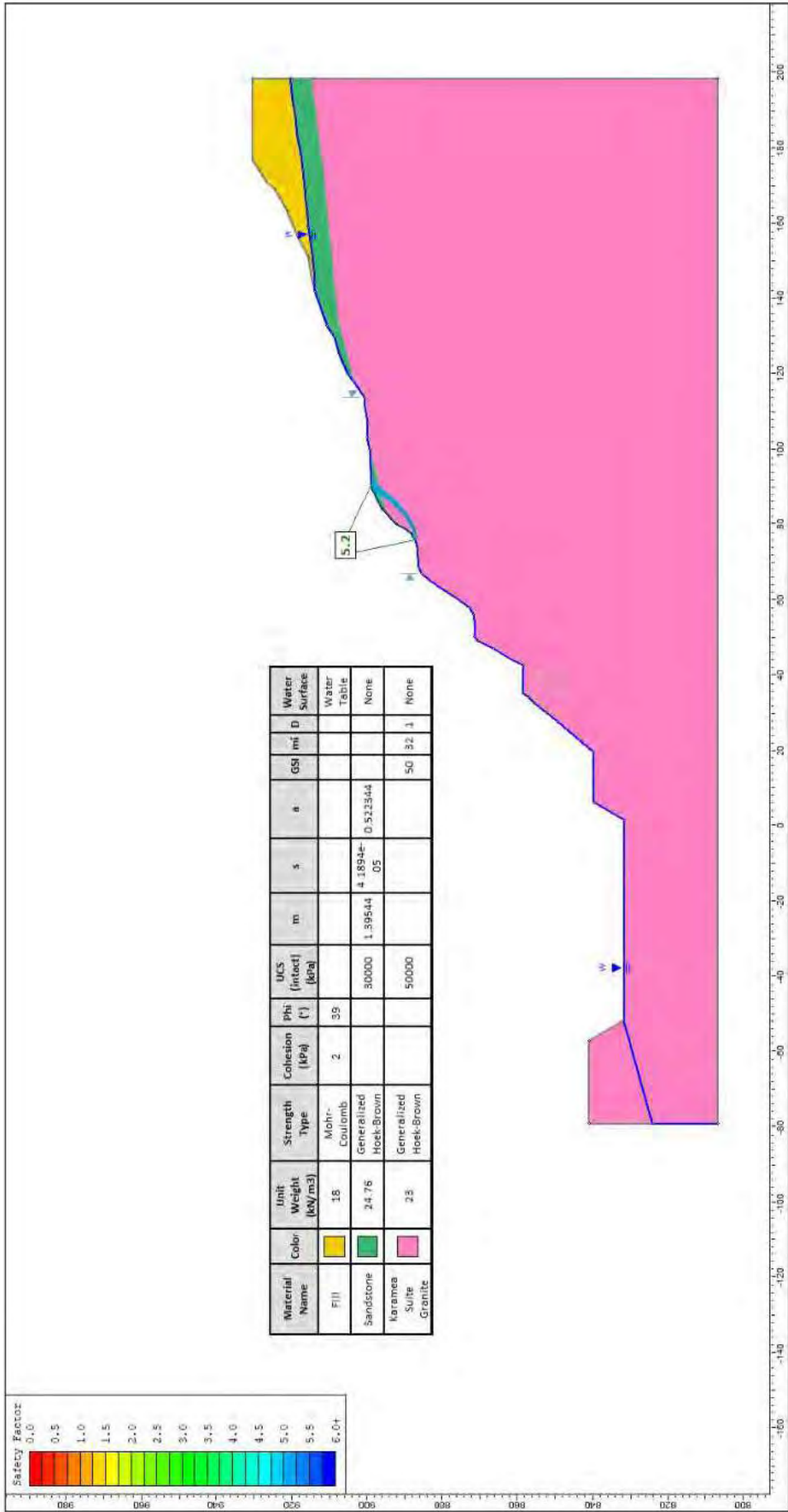


Figure D47: South Wall - Bench 1 - Static.

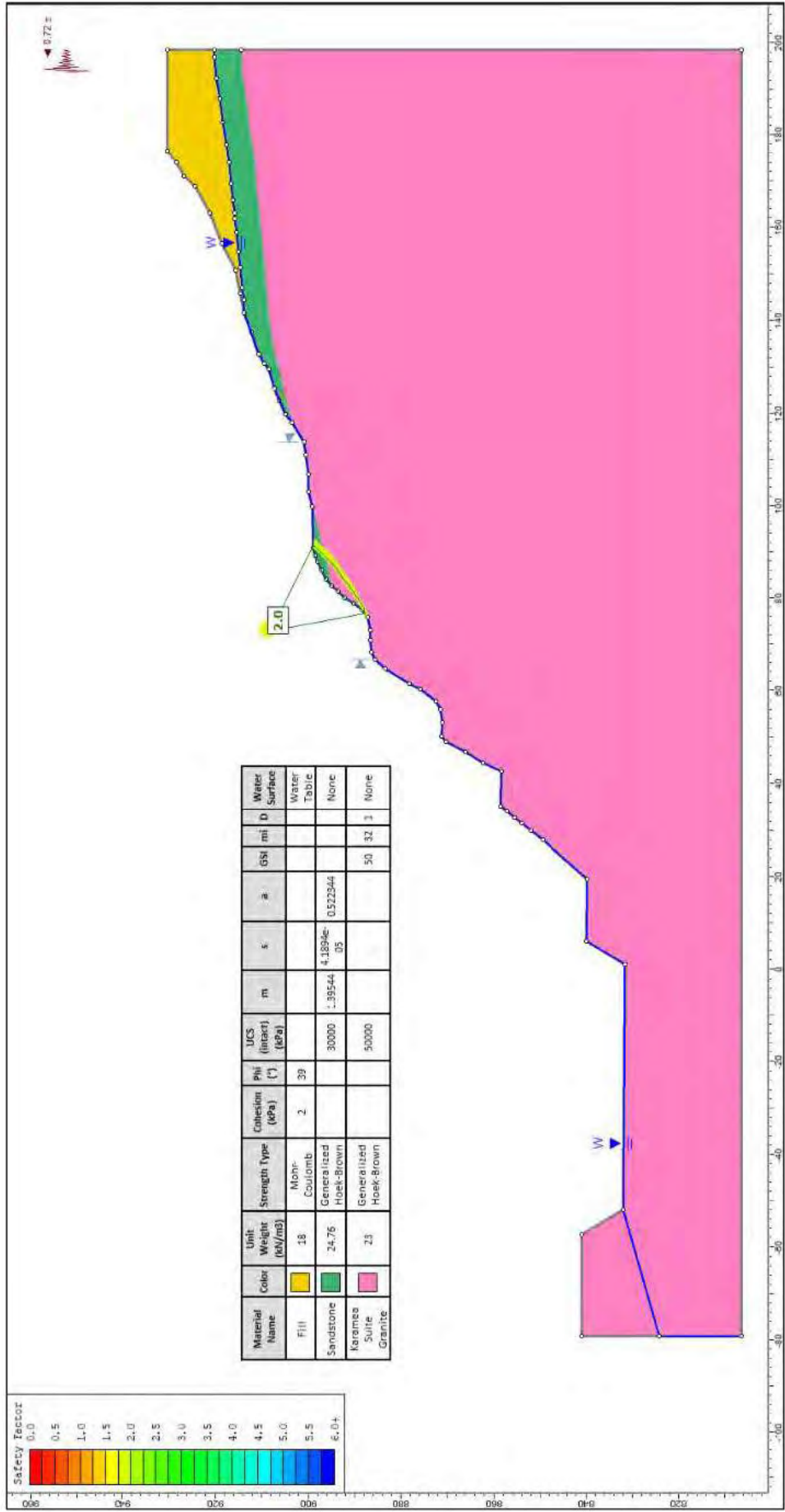


Figure D48: South Wall - Bench 1 - Seismic - PGA - 0.72g.

South Wall Existing - Bench 2

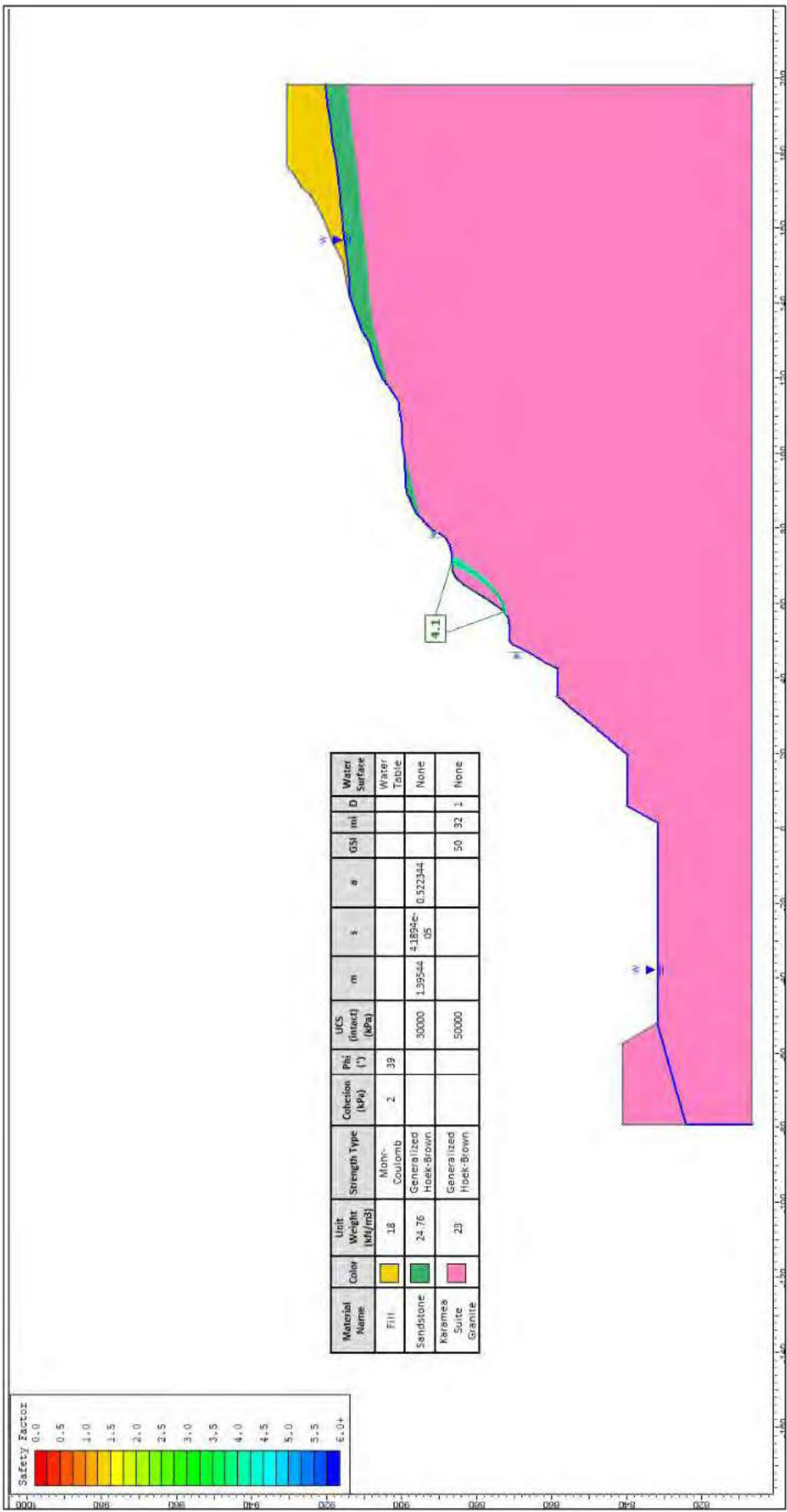
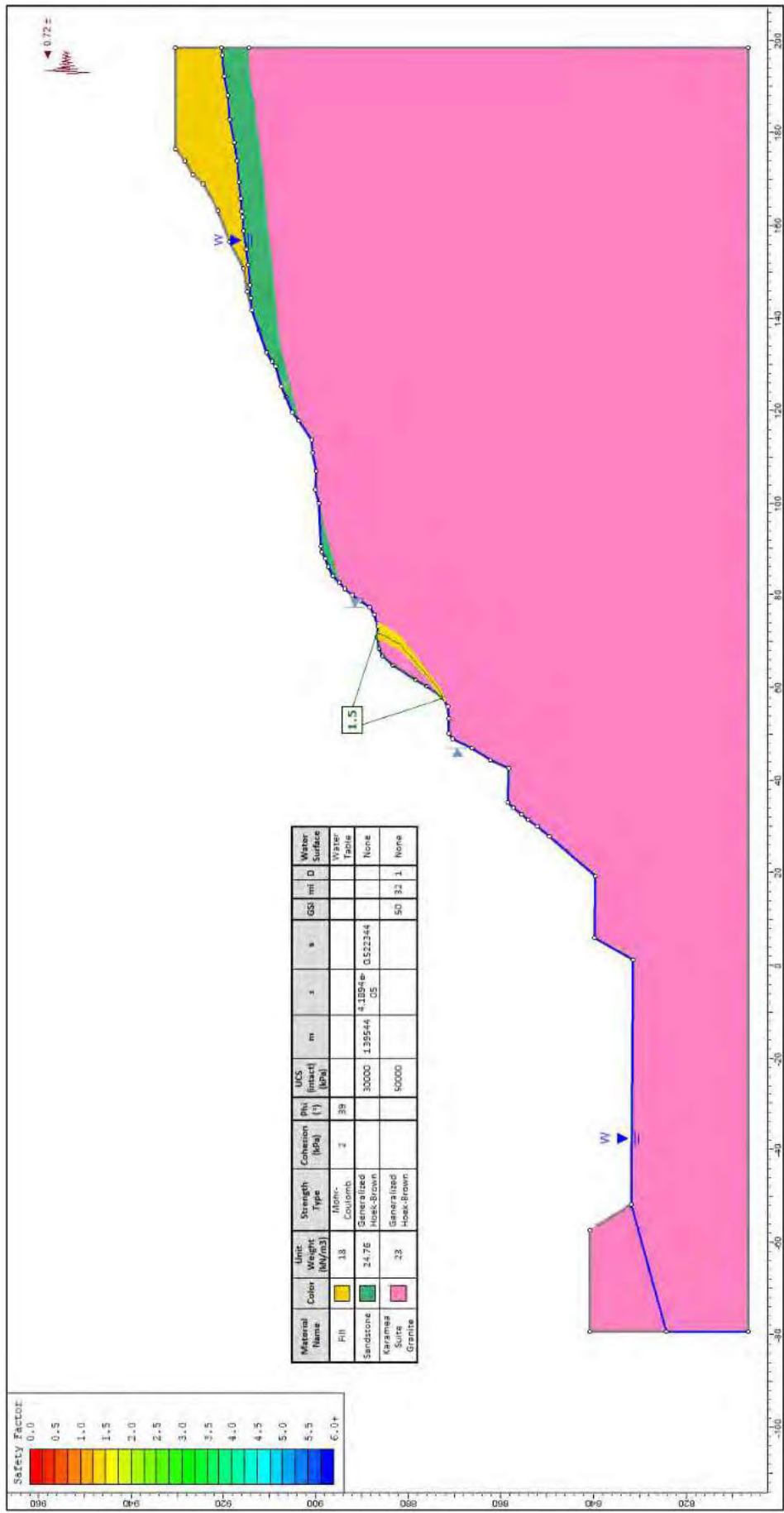


Figure D49: South Wall - Bench 2 - Static.



A-Drive South Wall Existing - Bench 3

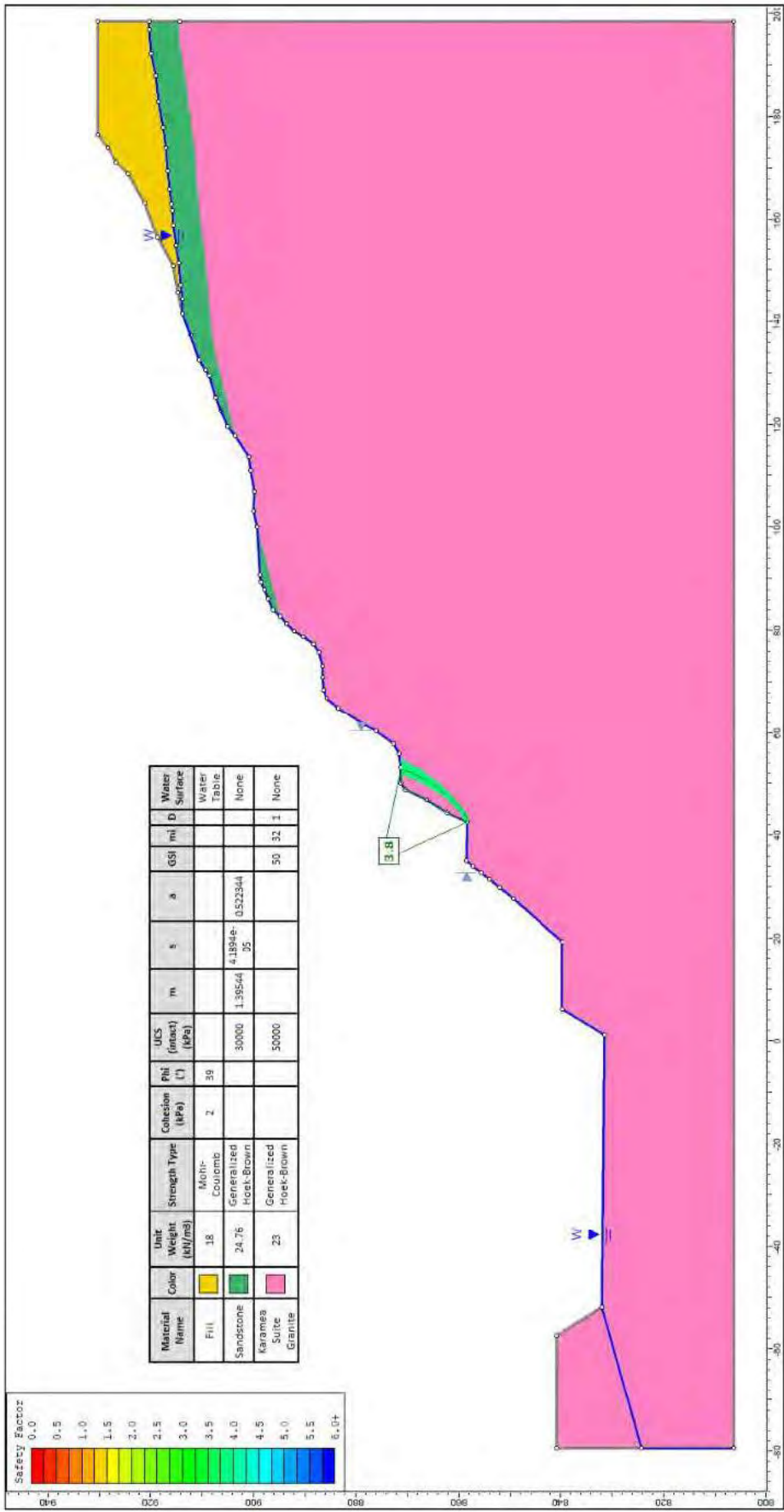


Figure D51: South Wall - Bench 3 - Static.

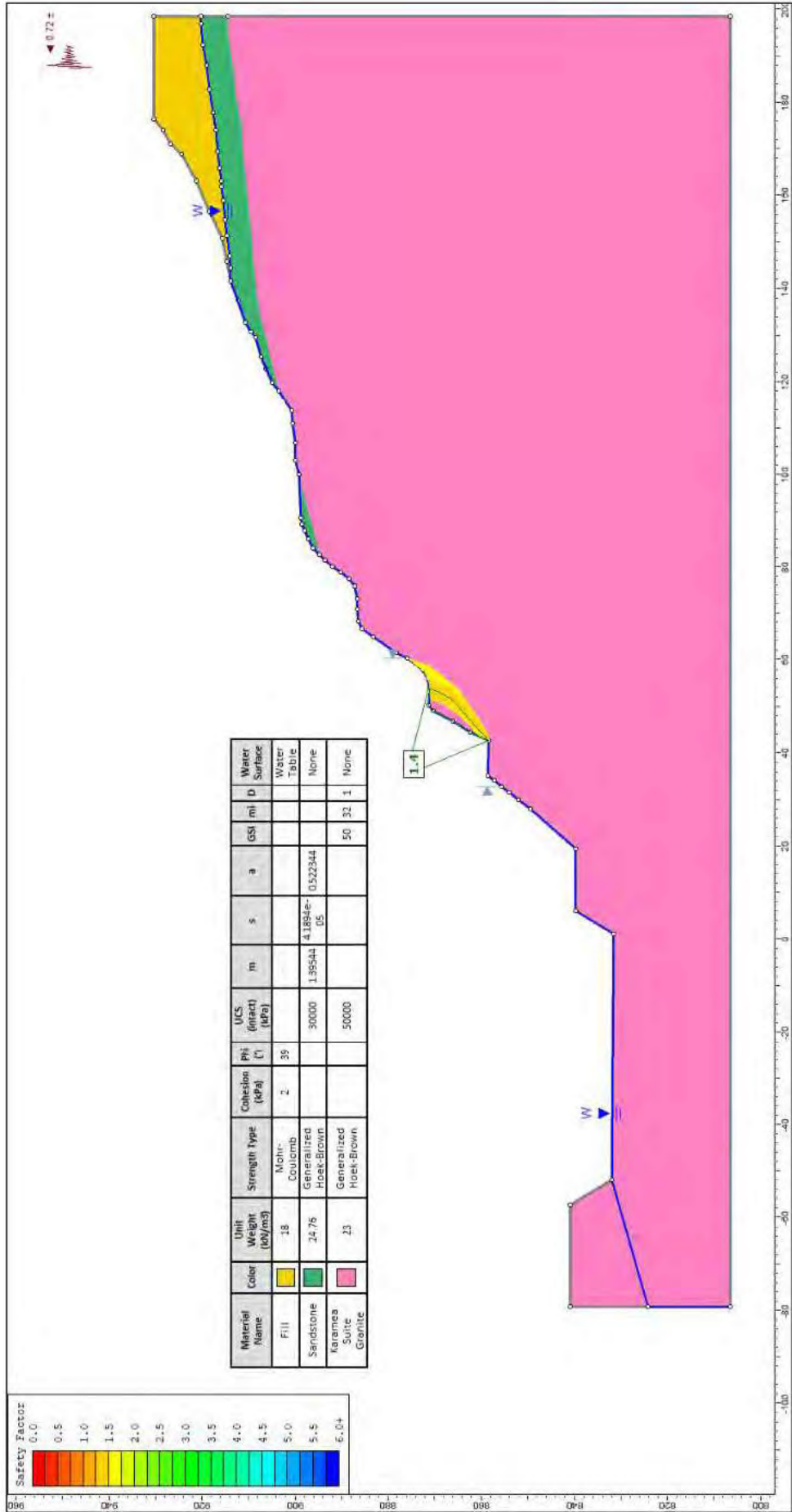
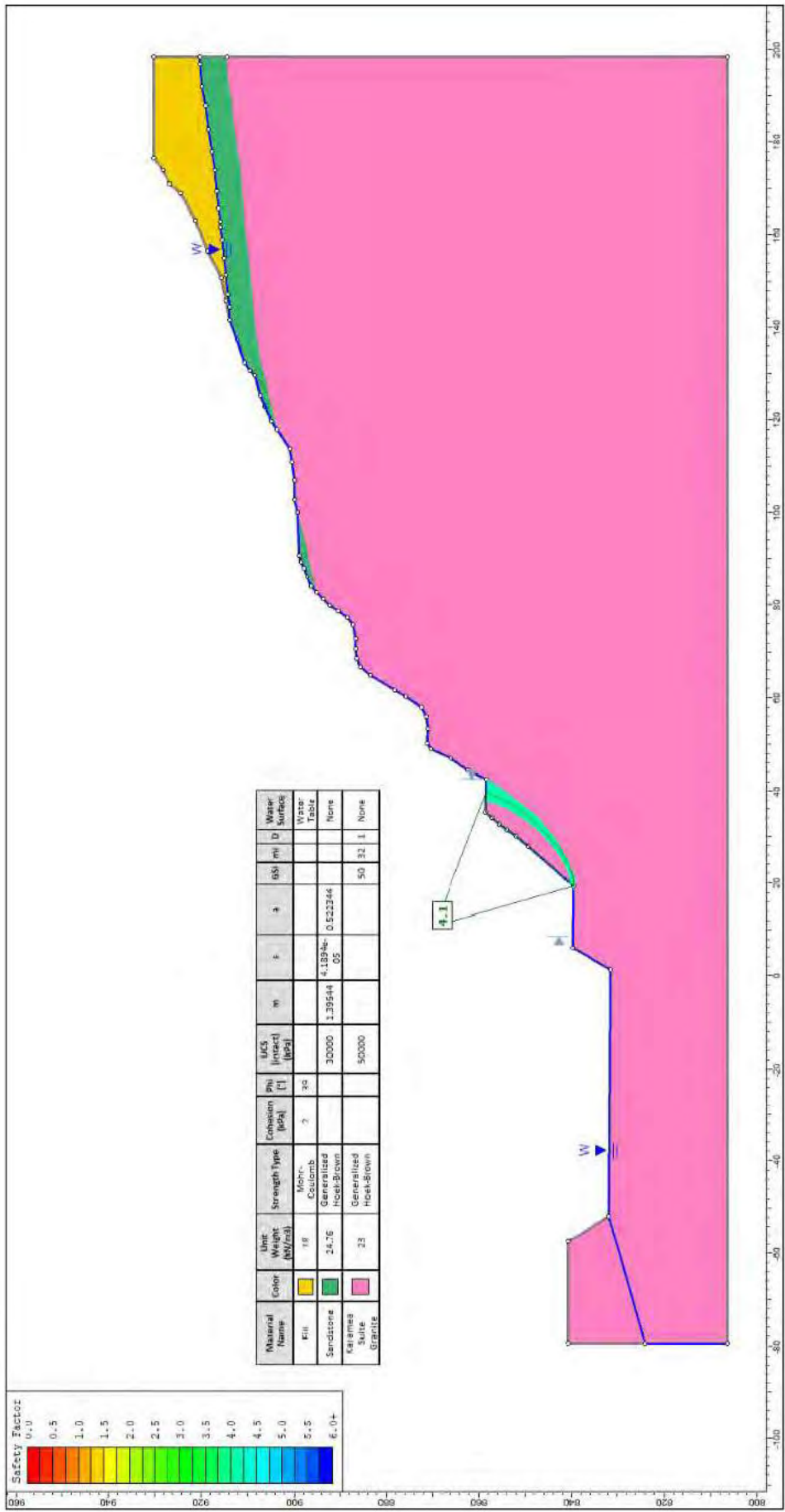
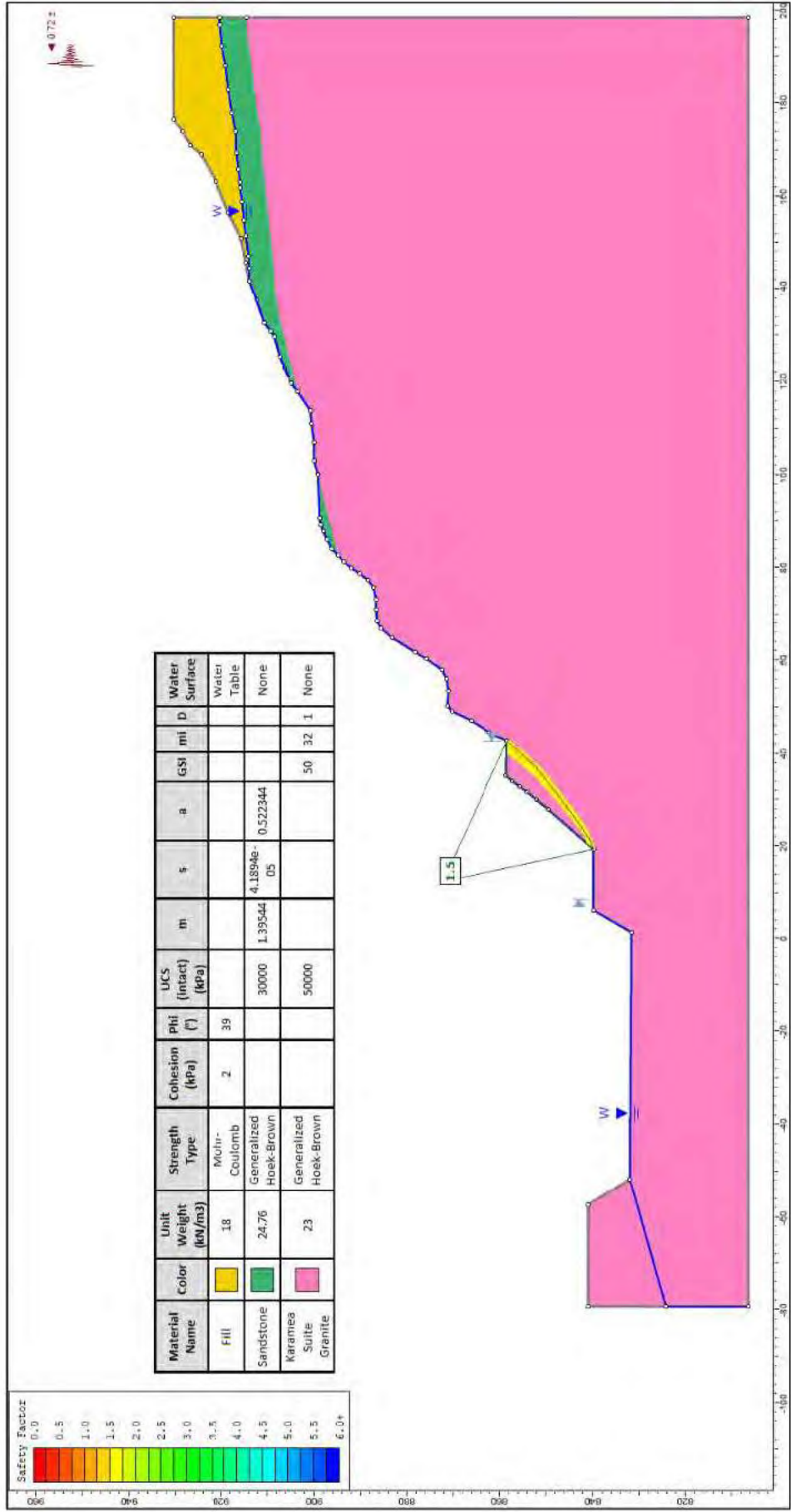


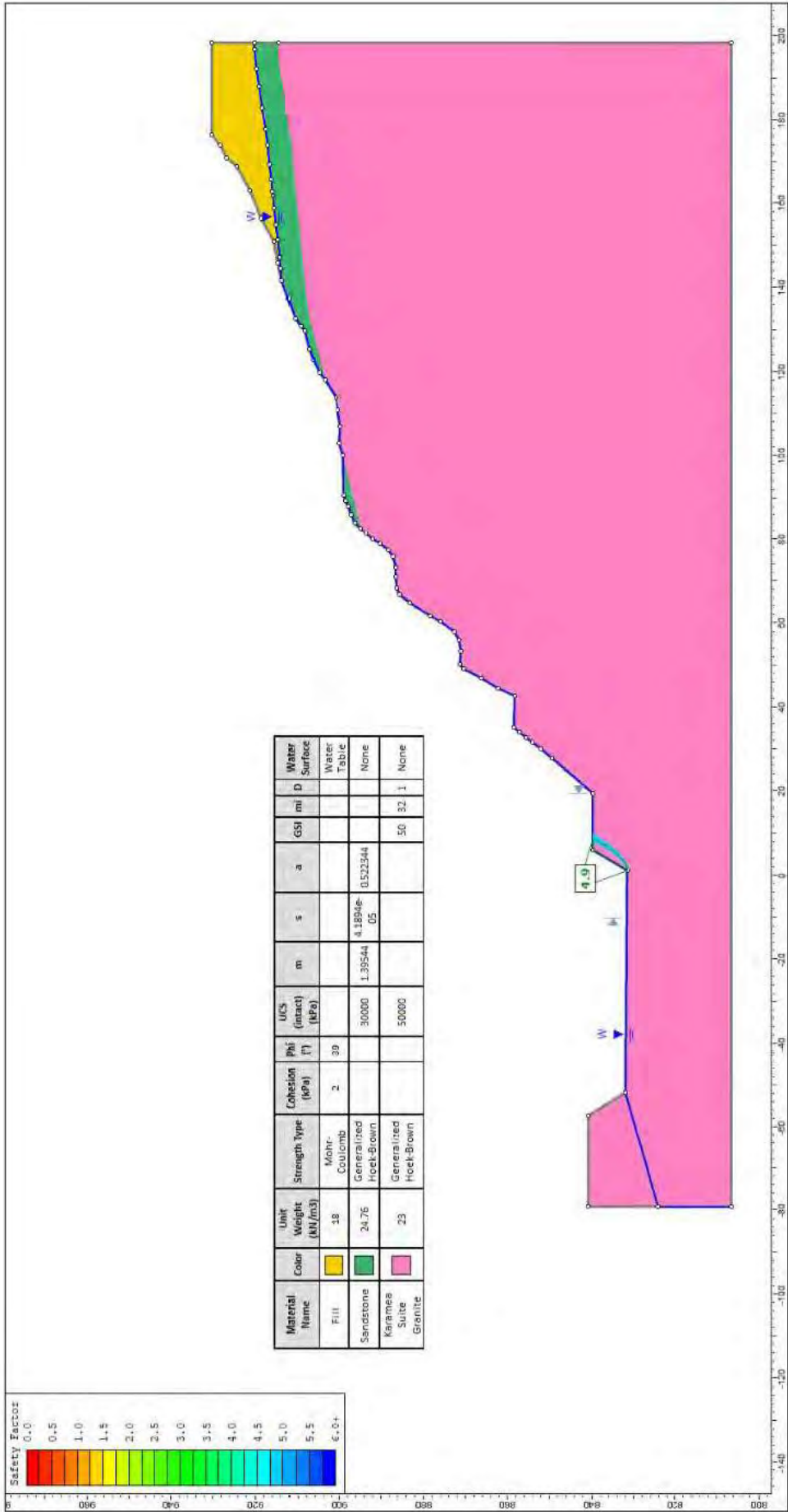
Figure D52: South Wall – Bench 3 – Seismic – PGA – 0.72g.

A-Drive South Wall - Bench 4





A-Drive South Wall - Bench 5



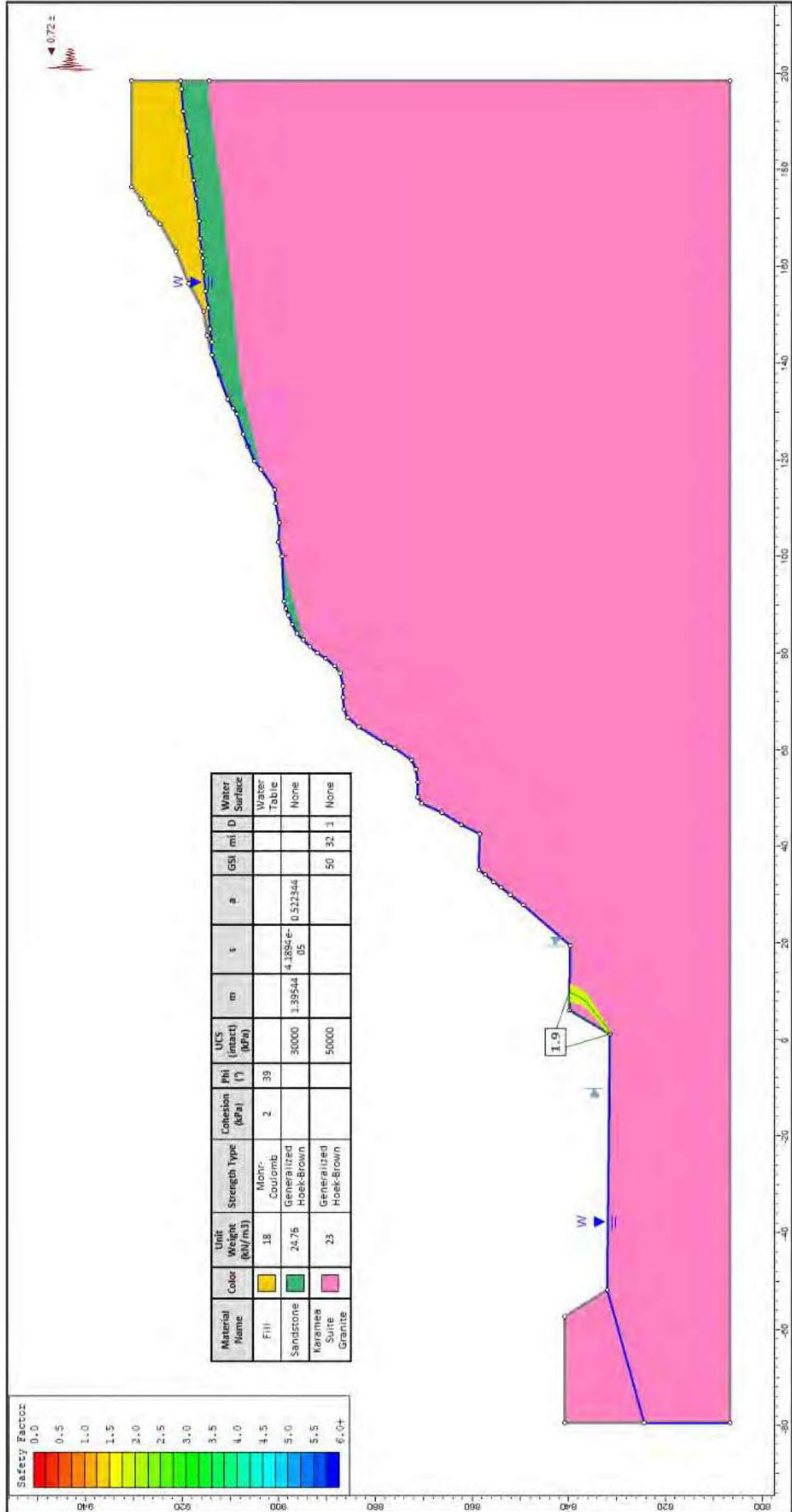


Figure D56: South Wall – Bench 5 – Seismic – PGA – 0.72g.

A-Drive South Wall - Multiple Bench (1-2)

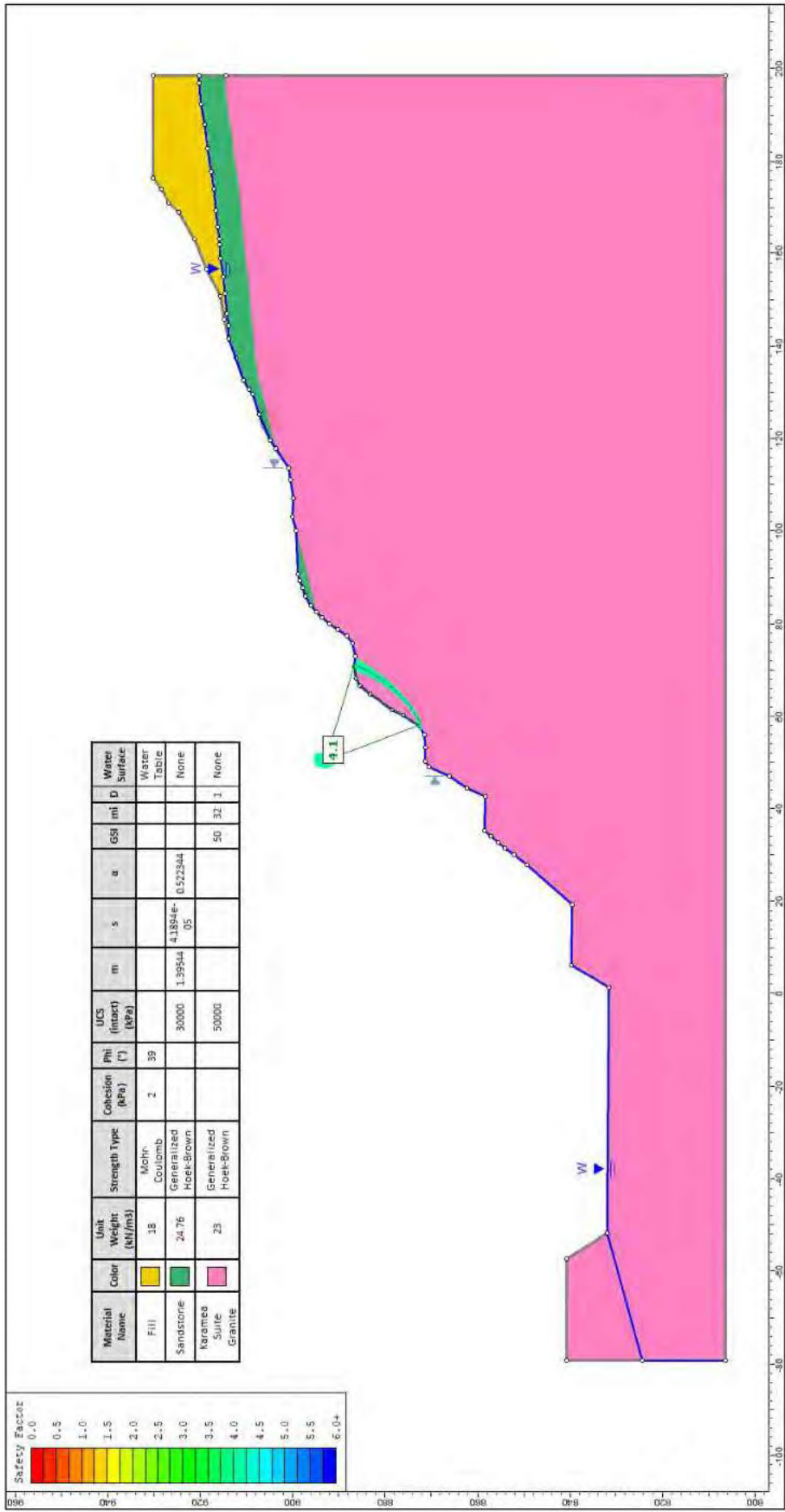


Figure D57: South Wall - Multiple Bench (1-2) - Static.

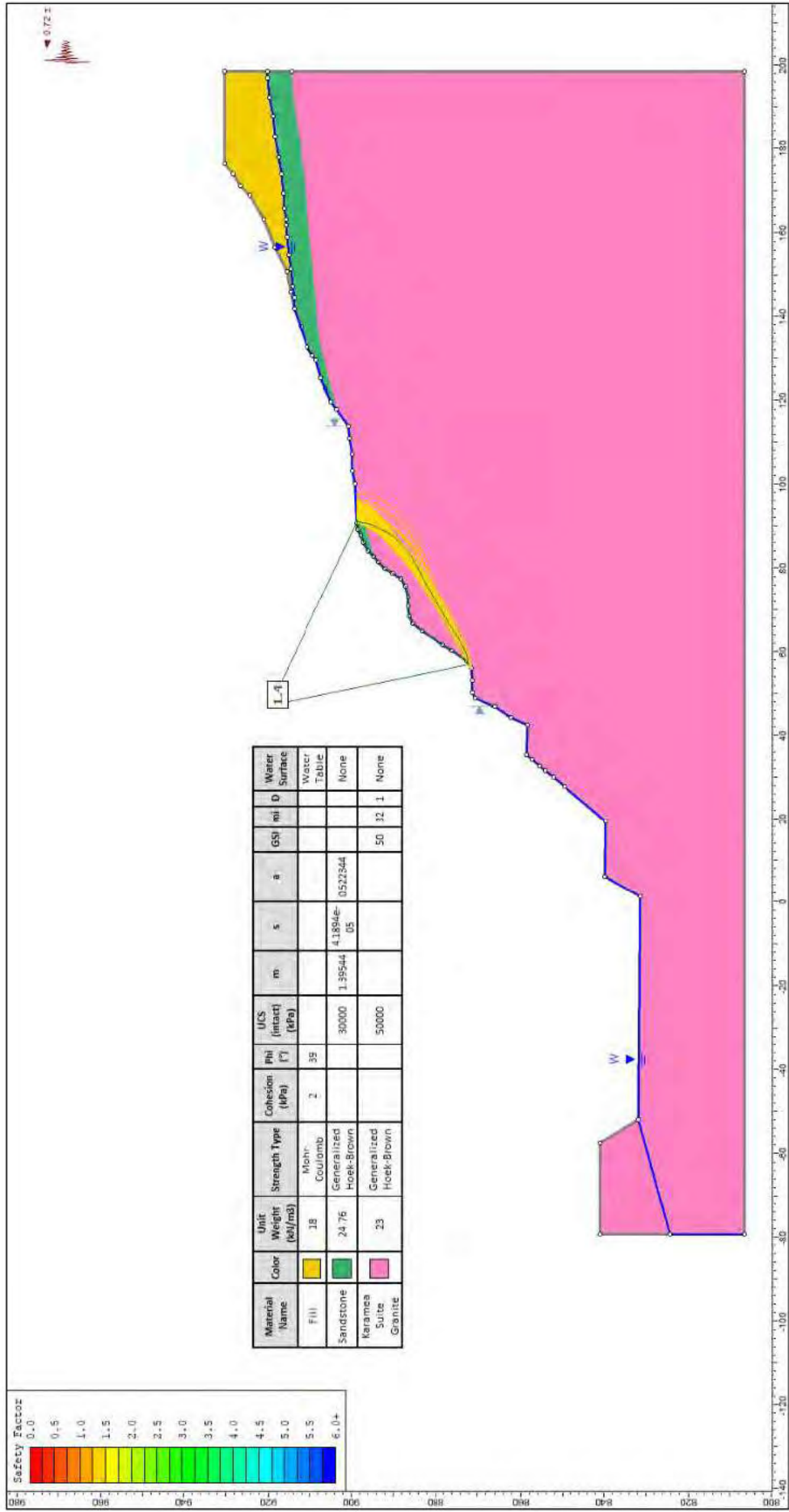


Figure D58: South Wall - Multiple Bench (1-2) - Seismic - PGA - 0.72g.

A-Drive South Wall Existing - Multiple Bench (3-4)

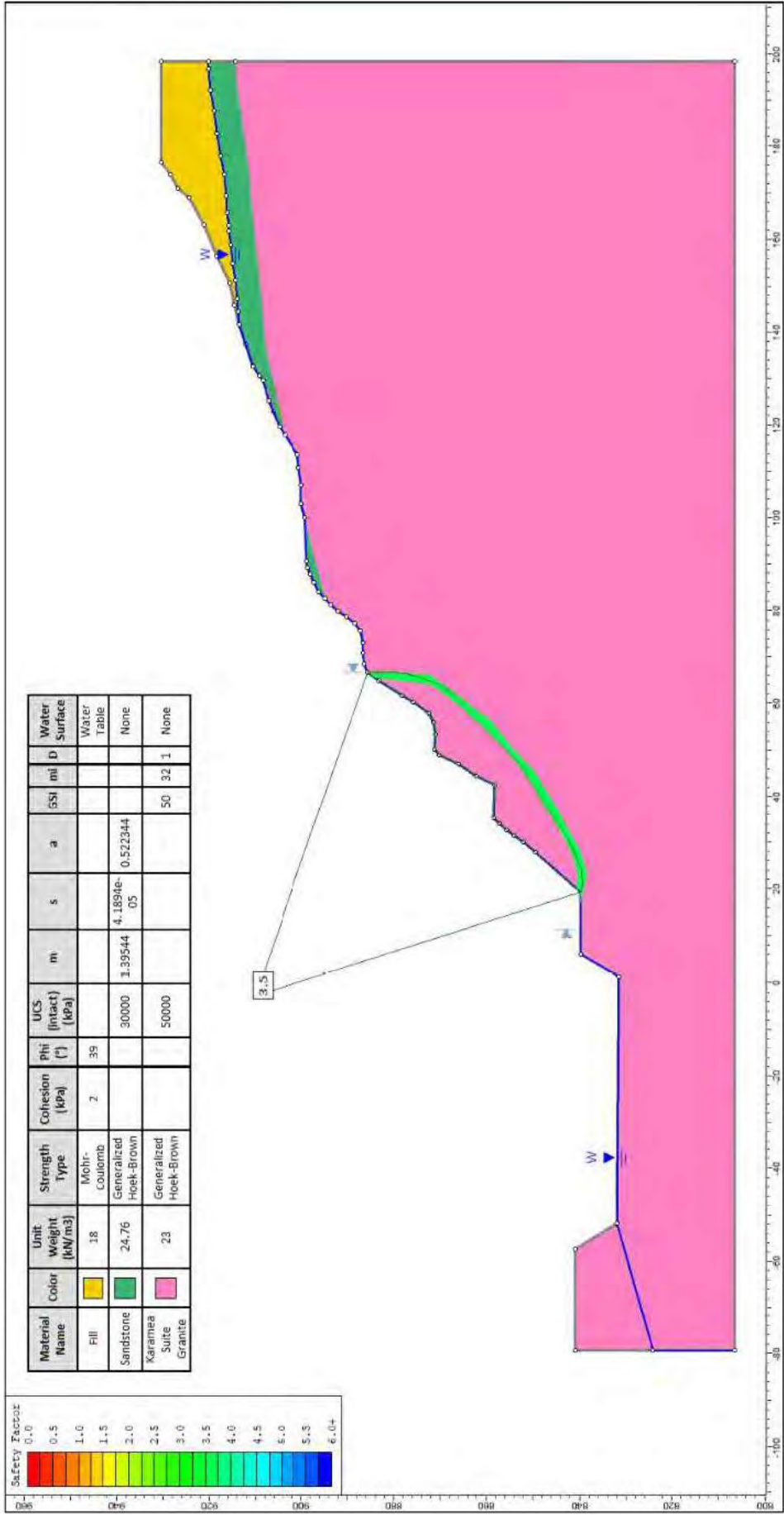


Figure D59: South Wall - Multiple Bench (3-4) - Static.

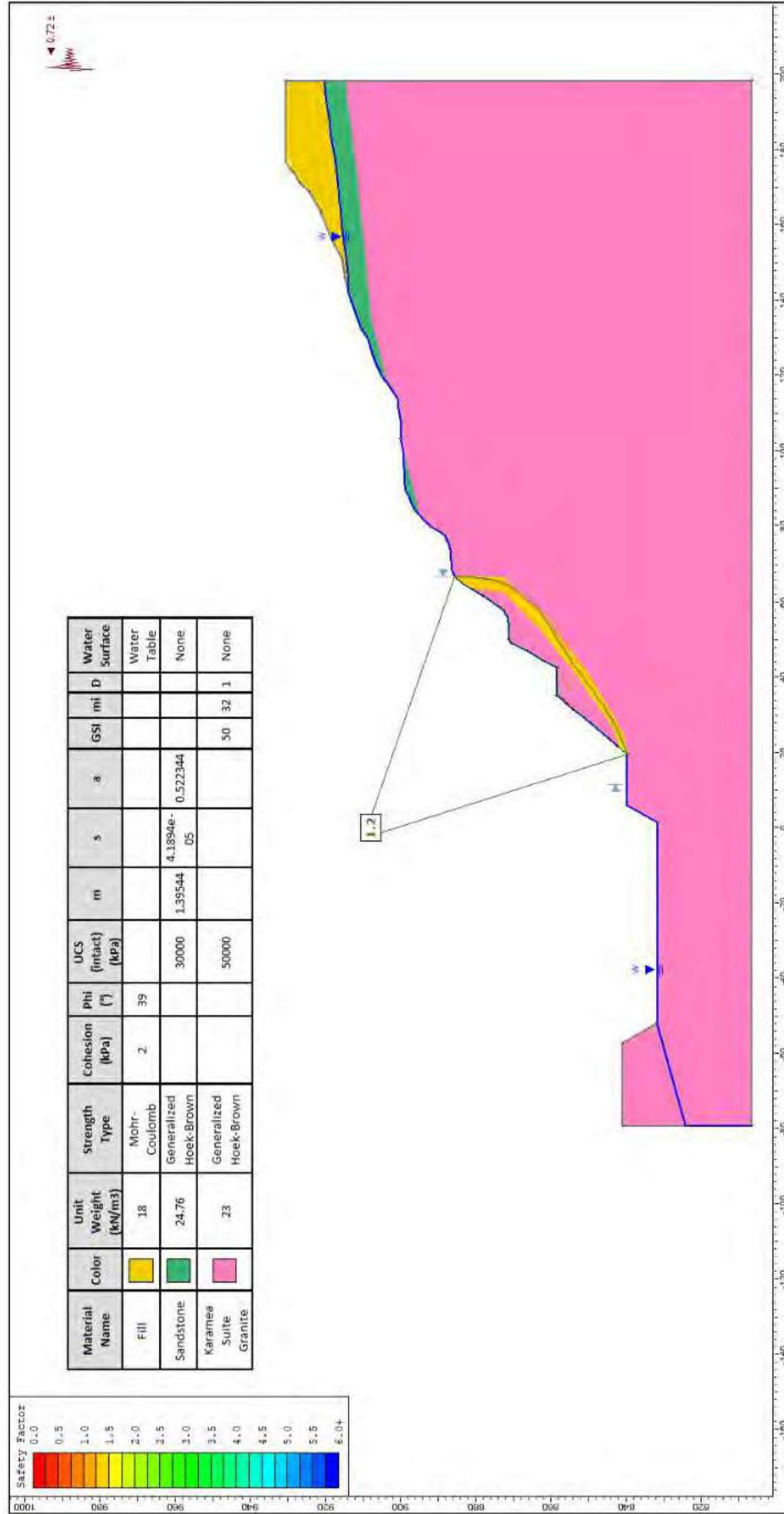


Figure D60: South Wall - Multiple Bench (3-4) - Seismic - PGA - 0.72g.

CFS Stage 2 – Overall Slope

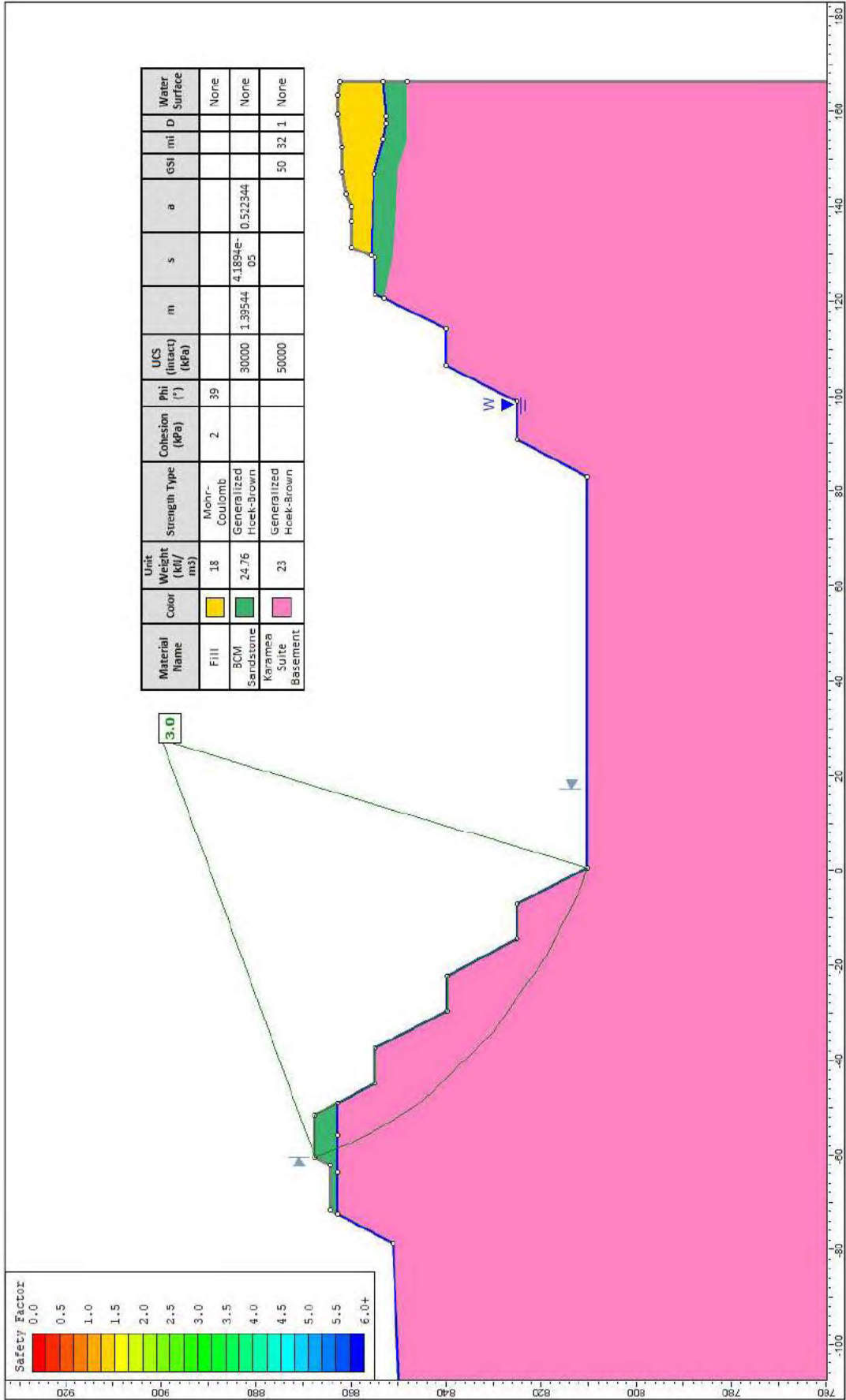


Figure D61: CFS Stage 2 – Overall Slope – Static.

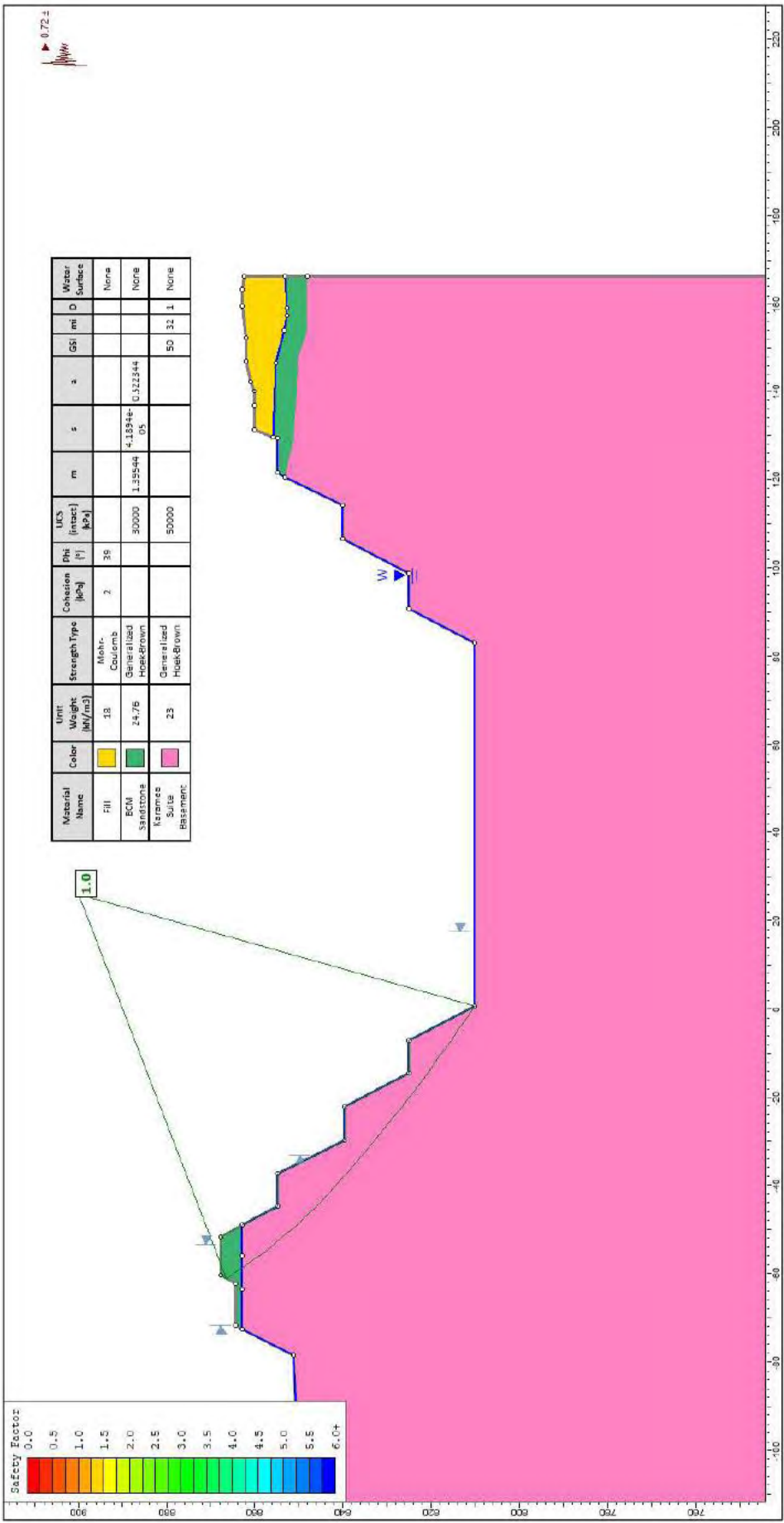


Figure D62: CFS Stage 2 – Overall Slope – Seismic – PGA 0.72g.

CFS Stage 2 – Single Bench (granite and sandstone)

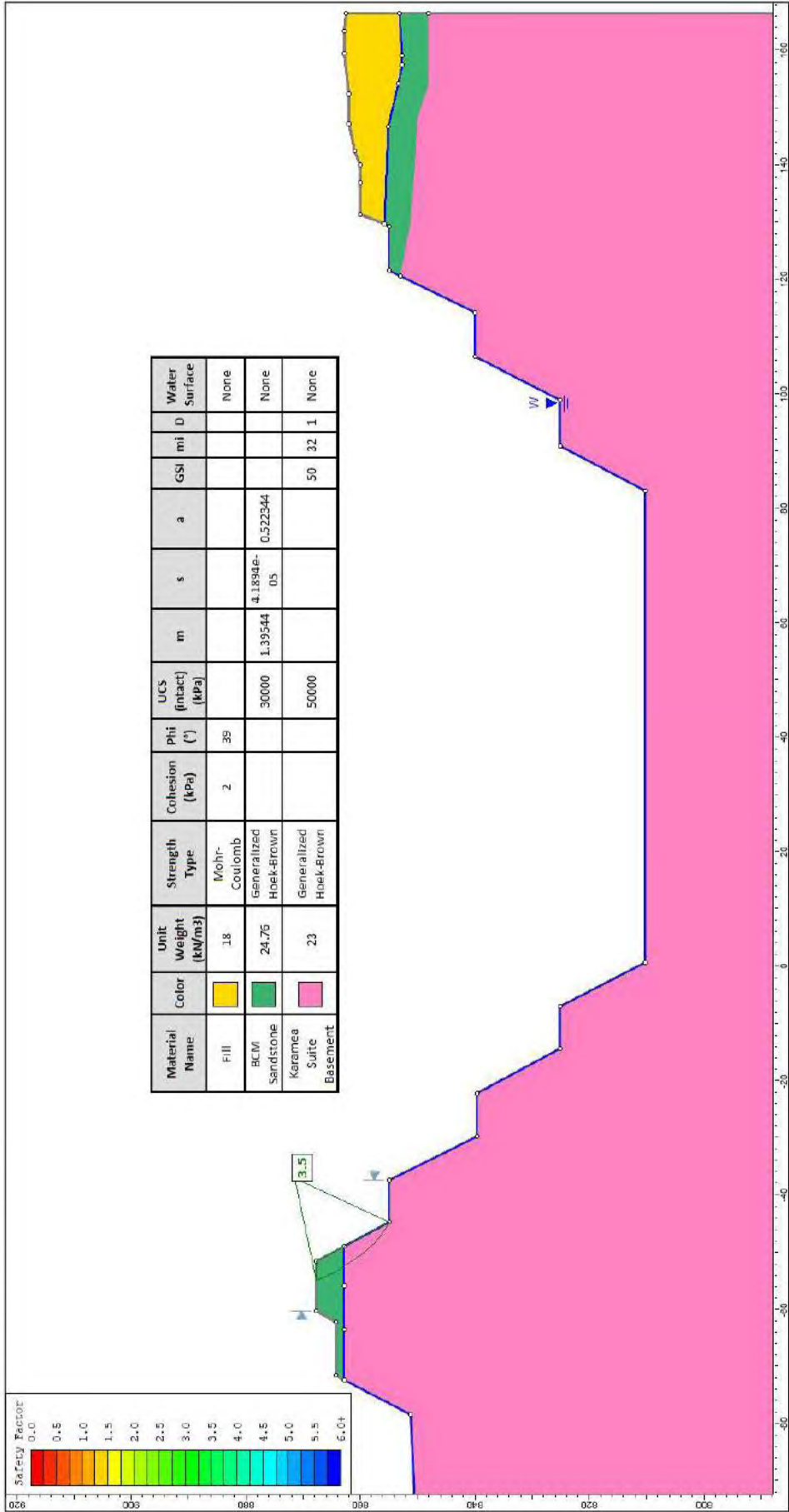


Figure D63: CFS Stage 2 – Single Bench (granite and sandstone) – Static.

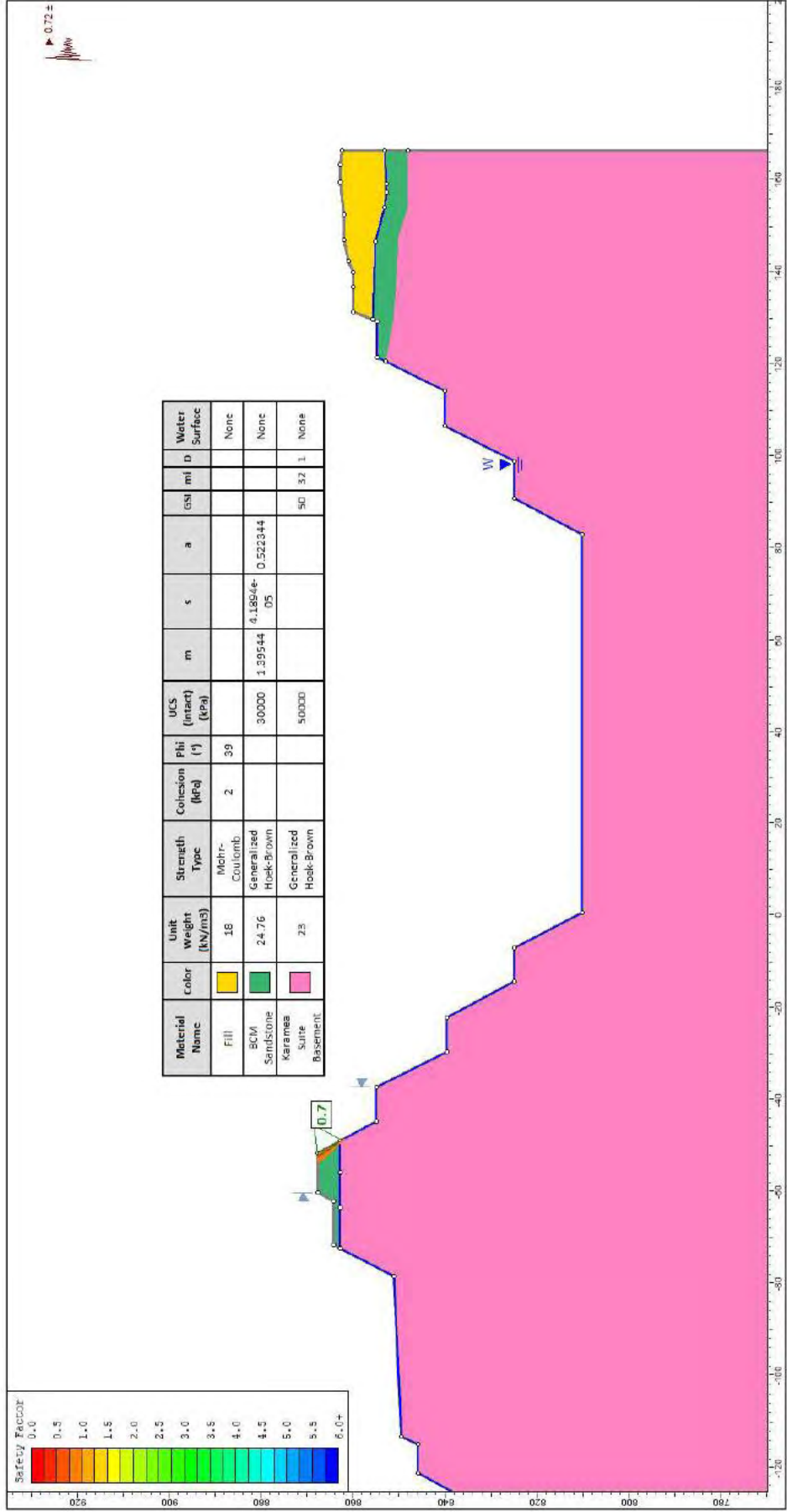


Figure D64: CFS Stage 2 – Single Bench (granite and sandstone) – Seismic – PGA 0.72g.

CFS Stage 2 – Single Bench (granite)

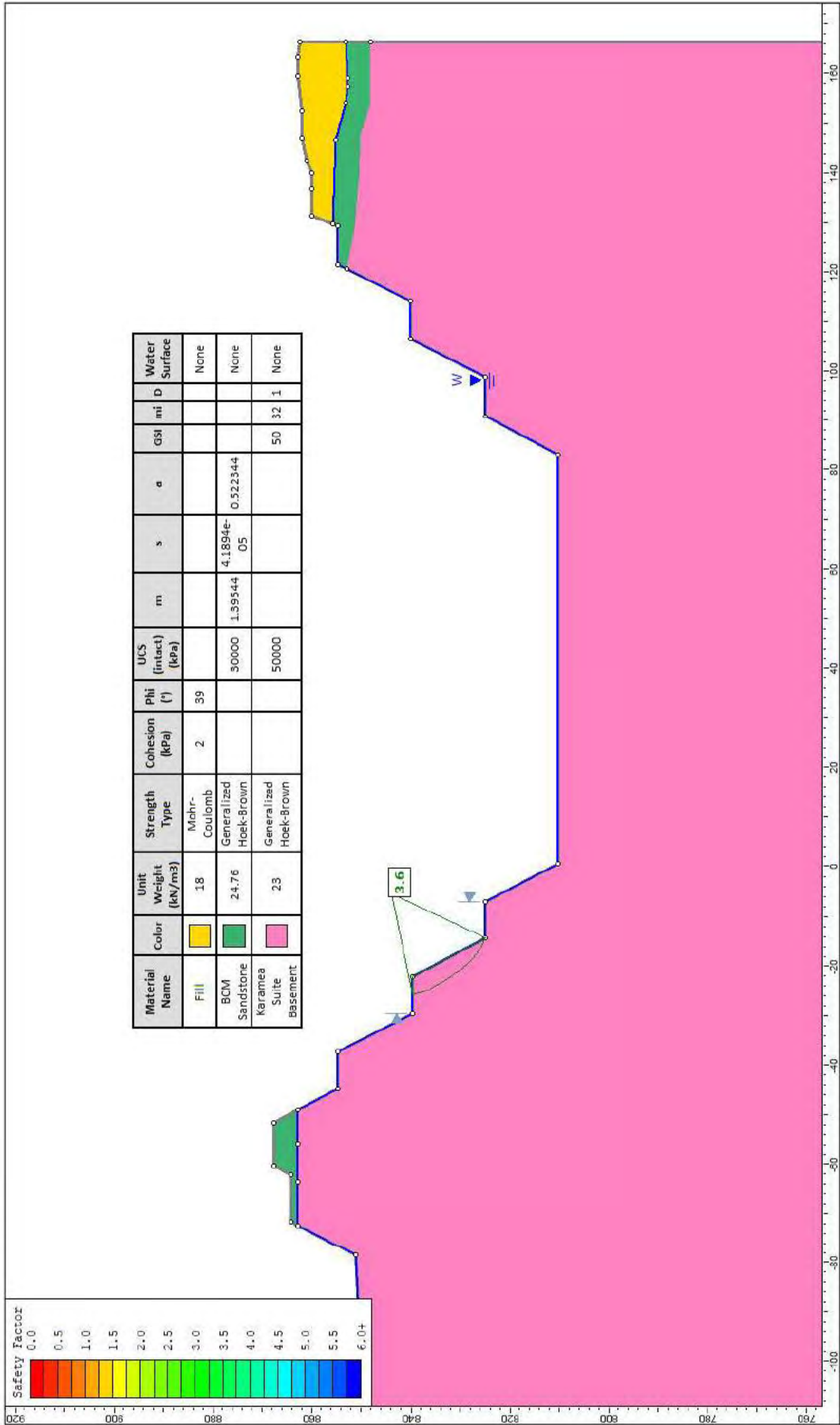
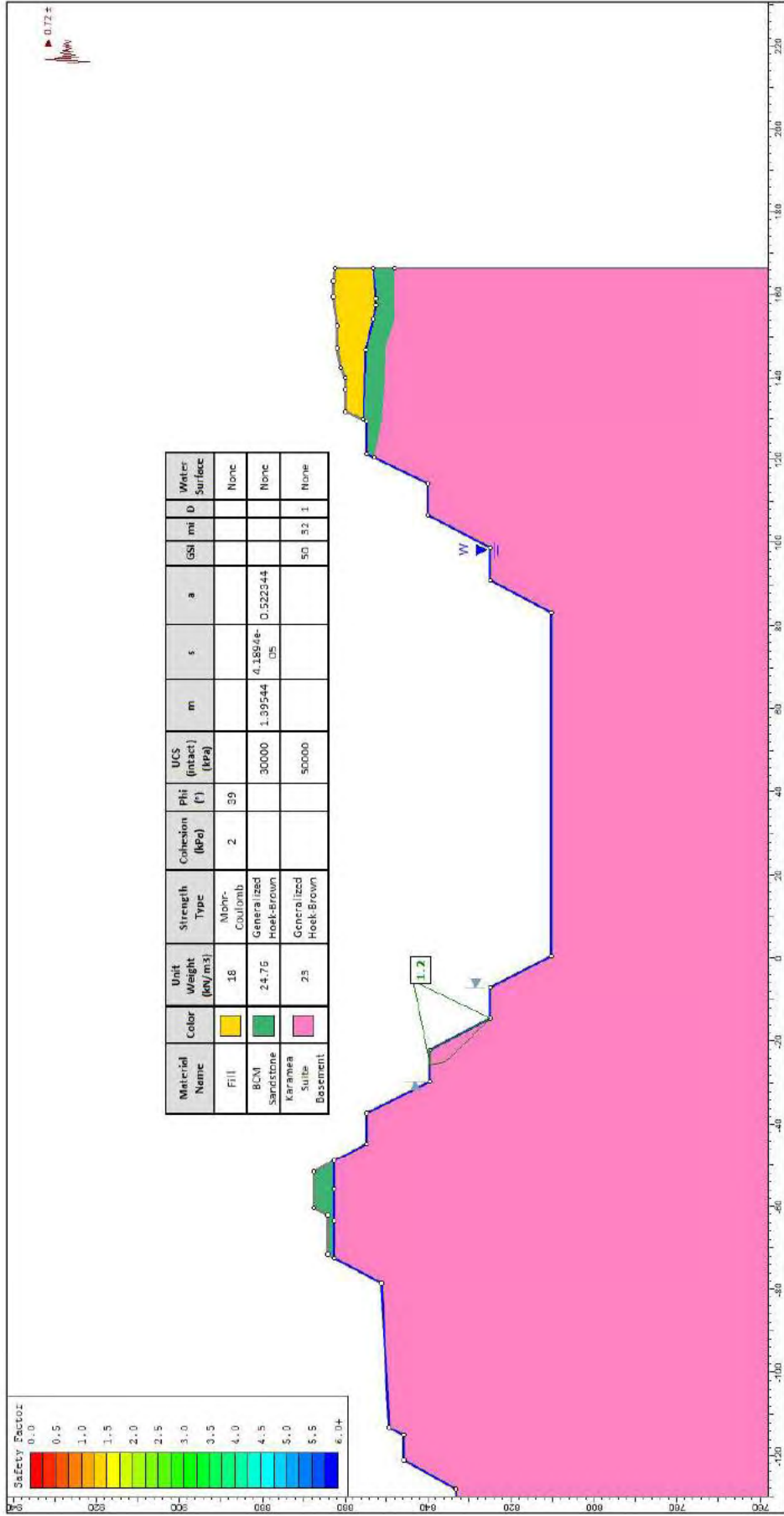


Figure D65: CFS Stage 2 – Single Bench (granite) – Static.



CFS Stage 2 – Multiple Bench (granite and sandstone)

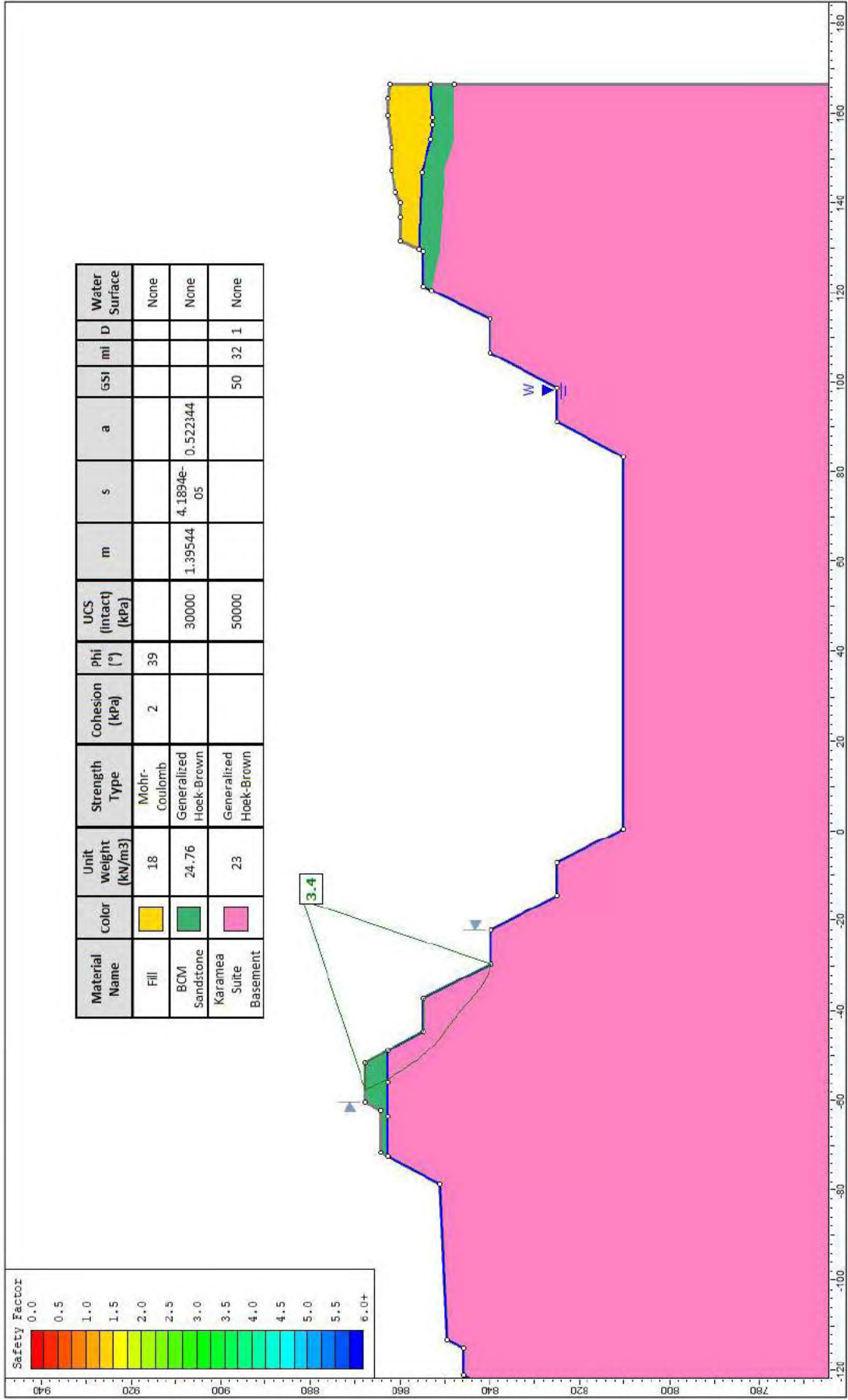


Figure D67: CFS Stage 2 – Multiple Bench (granite and sandstone) – Static.

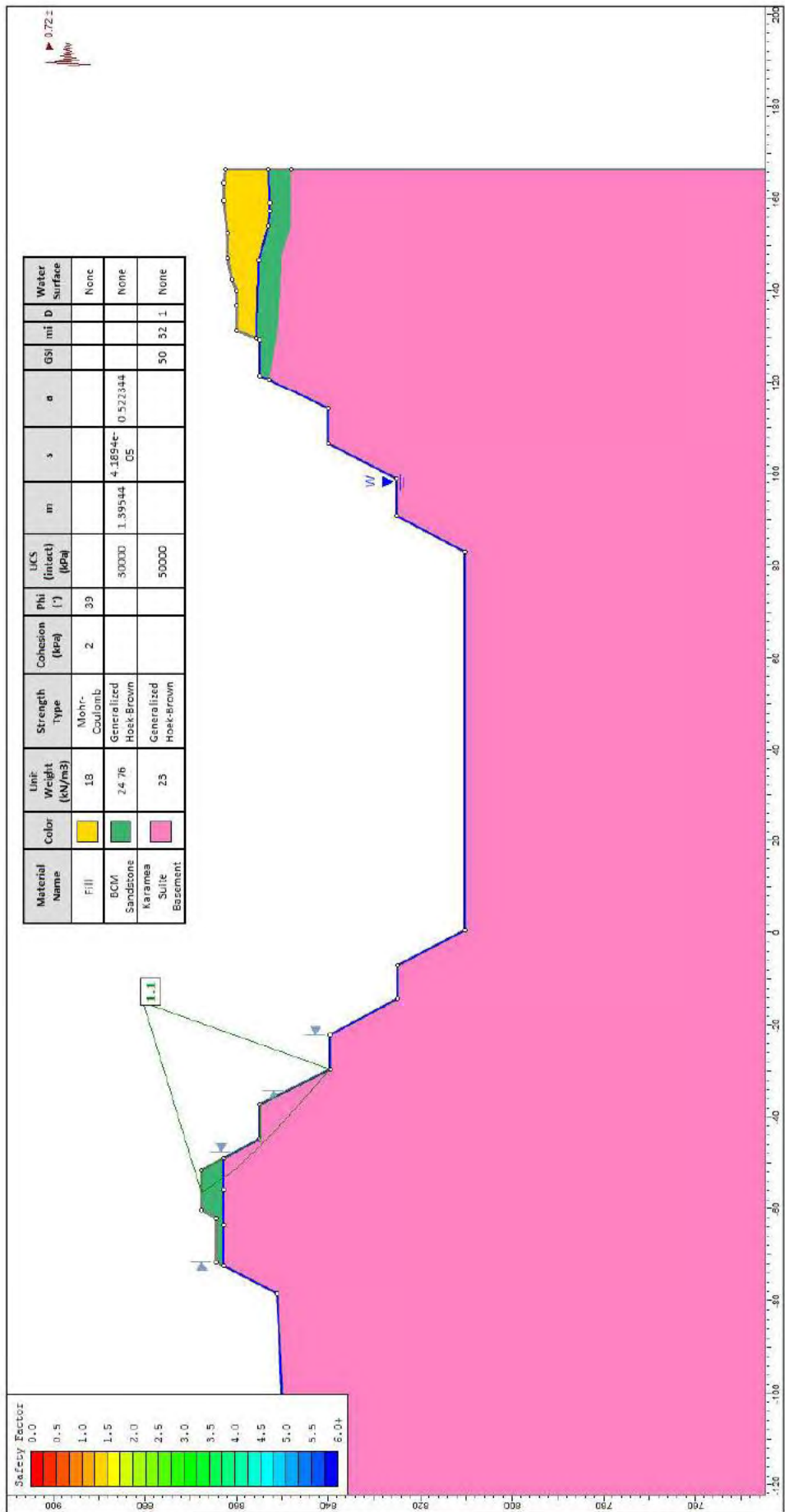


Figure D68: CFS Stage 2 – Multiple Bench (granite and sandstone) – Seismic PGA 0.72g.

CFS Stage 2 – Multiple Bench (granite)

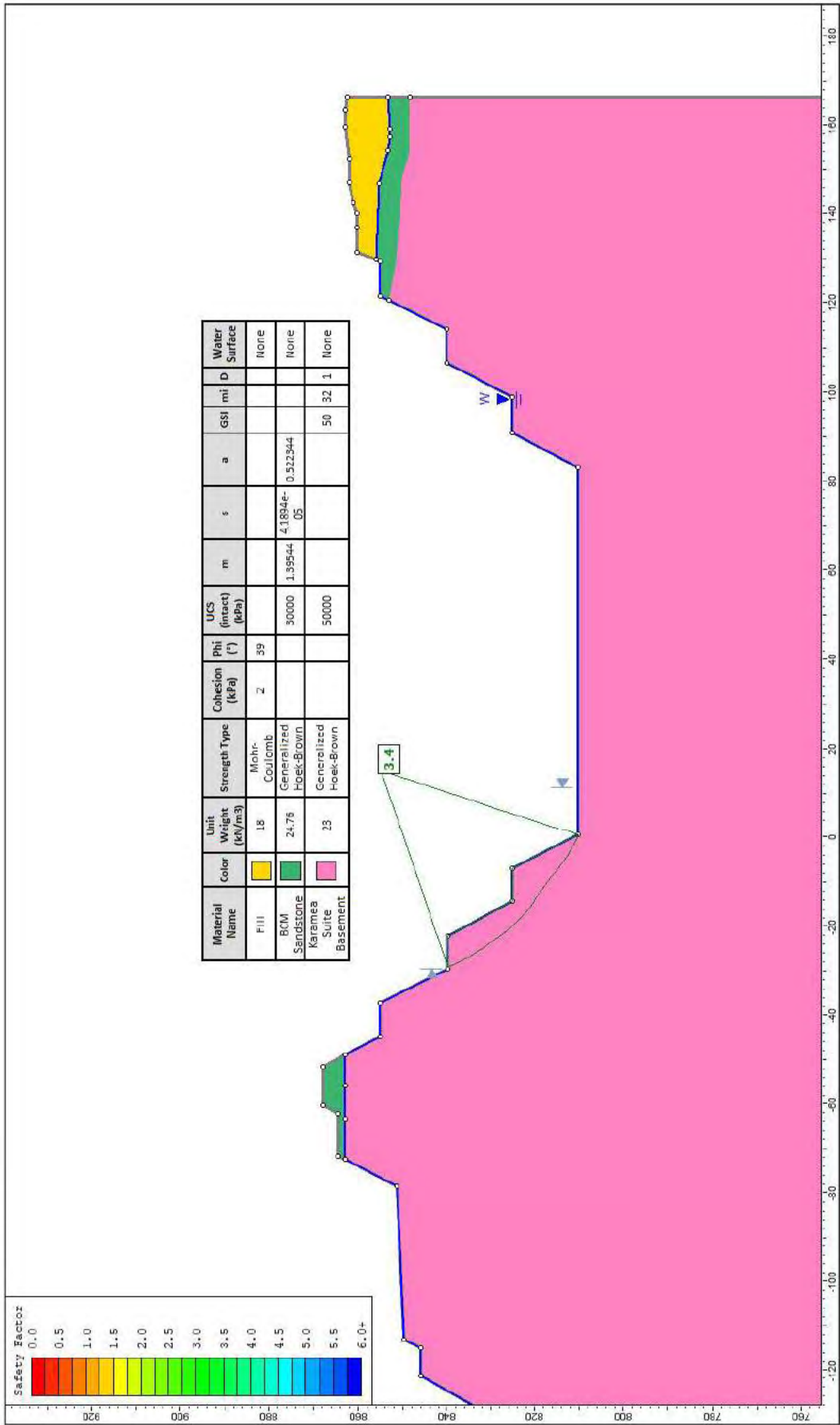


Figure D69: CFS Stage 2 – Multiple Bench (granite) – Static



Figure D70: CFS Stage 2 — Multiple Bench (granite) — Seismic PGA 0.72g.

CFS Stage 3 – Overall Slope

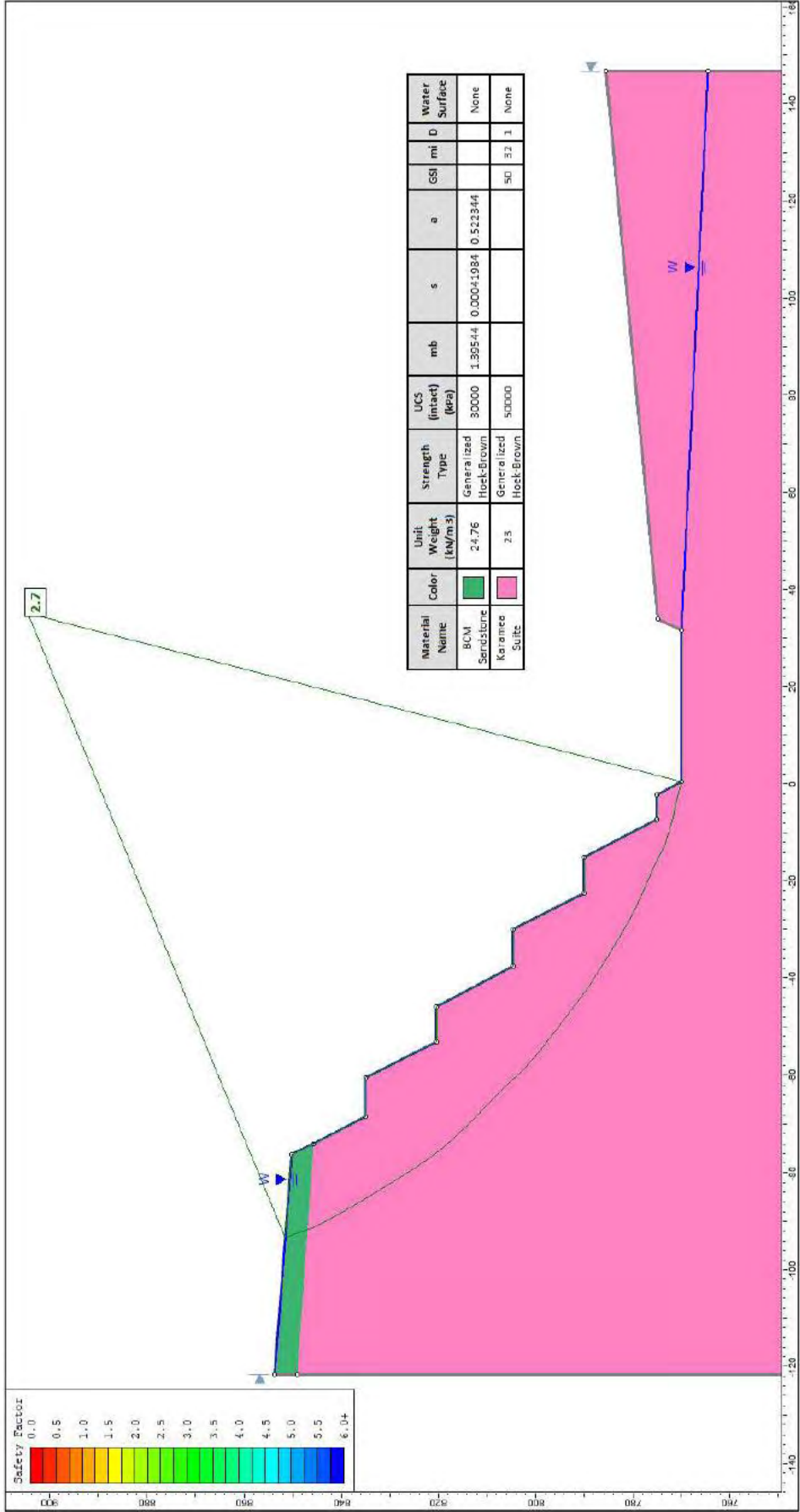


Figure D71: CFS Stage 3 – Overall Slope – Static

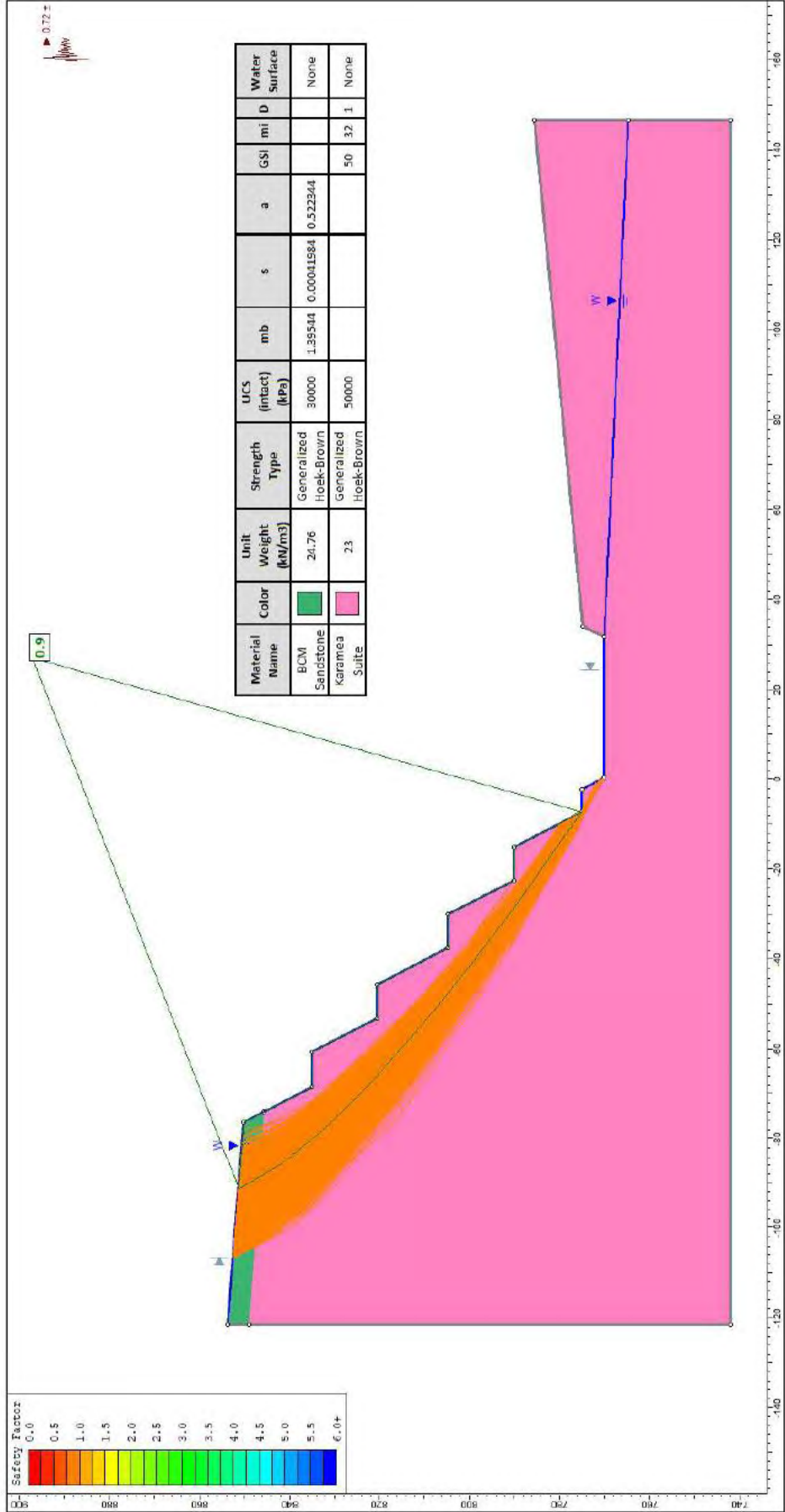


Figure D72: CFS Stage 3 – Overall Slope – Seismic PGA 0.72g

CFS Stage 3 – Single Bench (granite and sandstone)

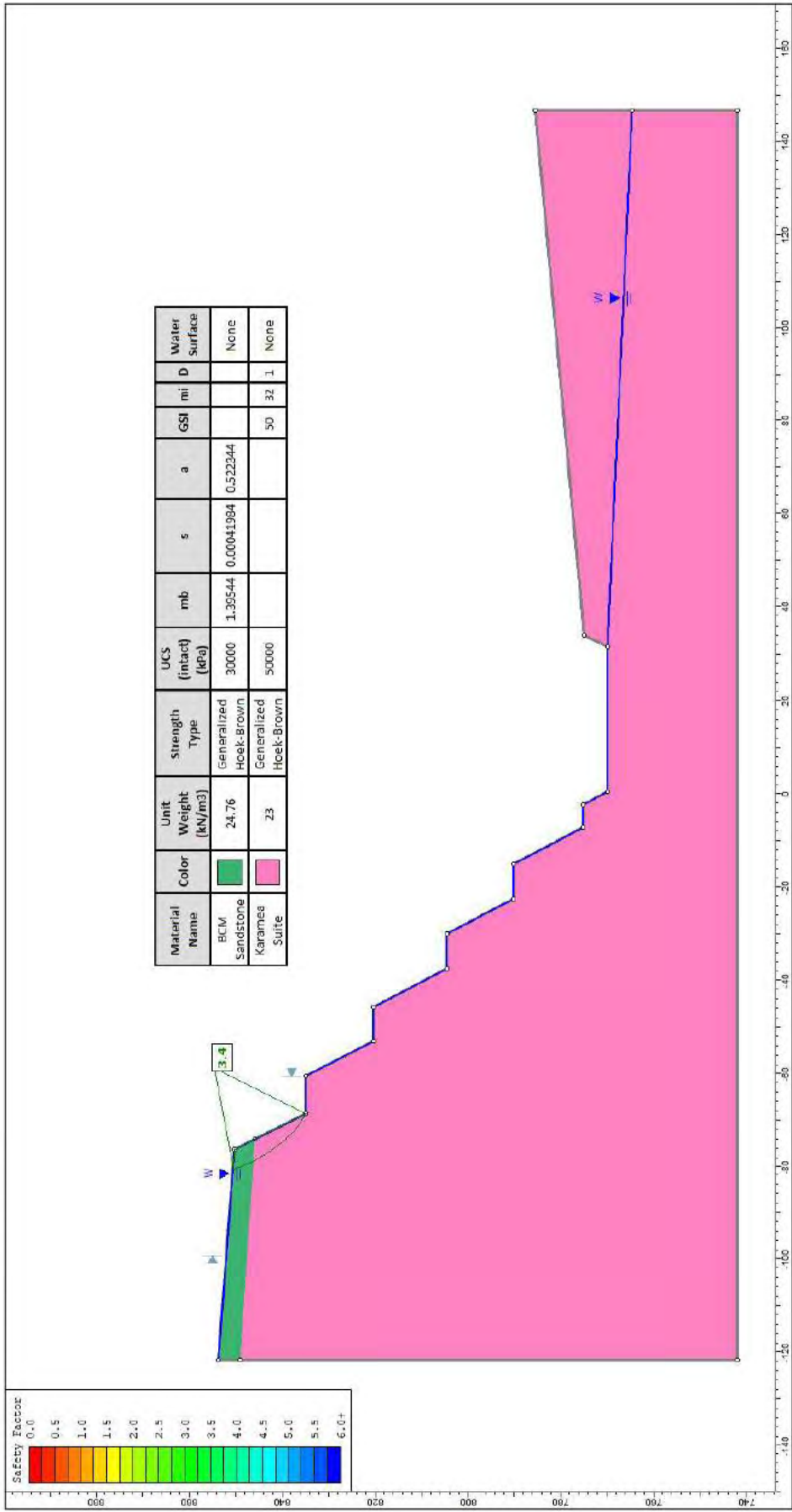


Figure D73: CFS Stage 3 – Single Bench (granite and sandstone) – Static.

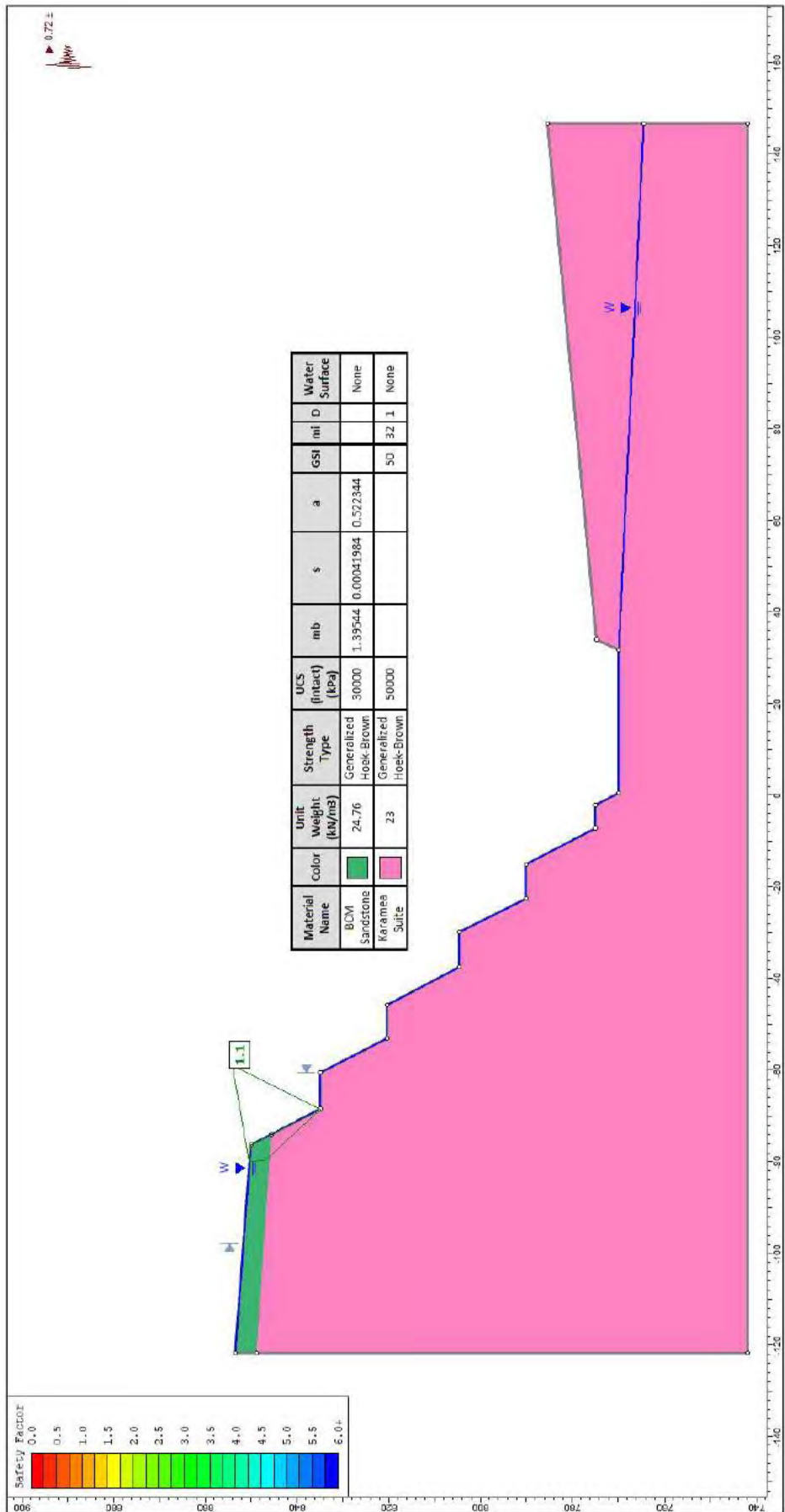


Figure D74: CFS Stage 3 – Single Bench (granite and sandstone) – Seismic PGA 0.72g.

CFS Stage 3 – Single Bench (granite)

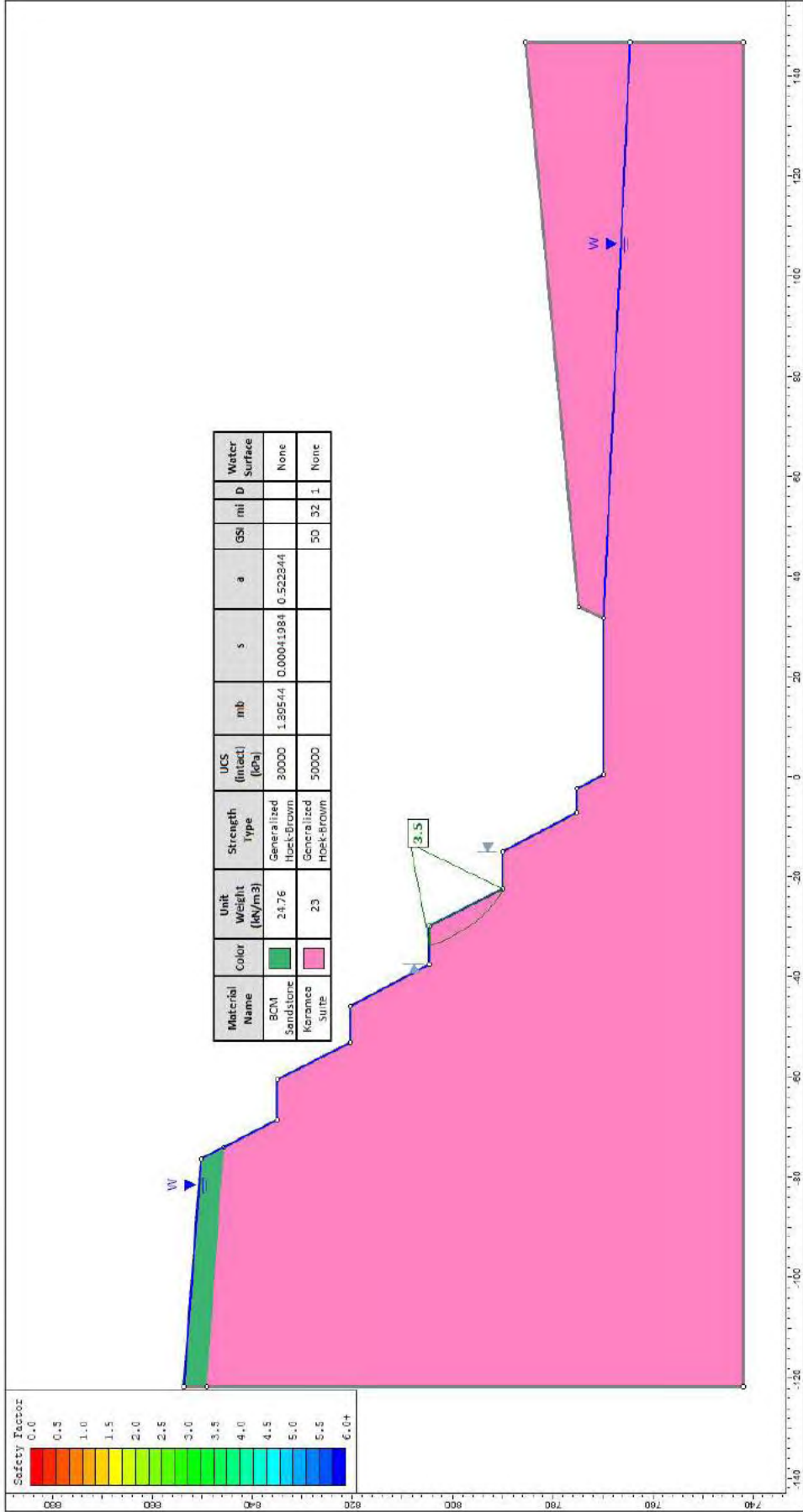


Figure D75: CFS Stage 3 – Single Bench (granite) – Static.

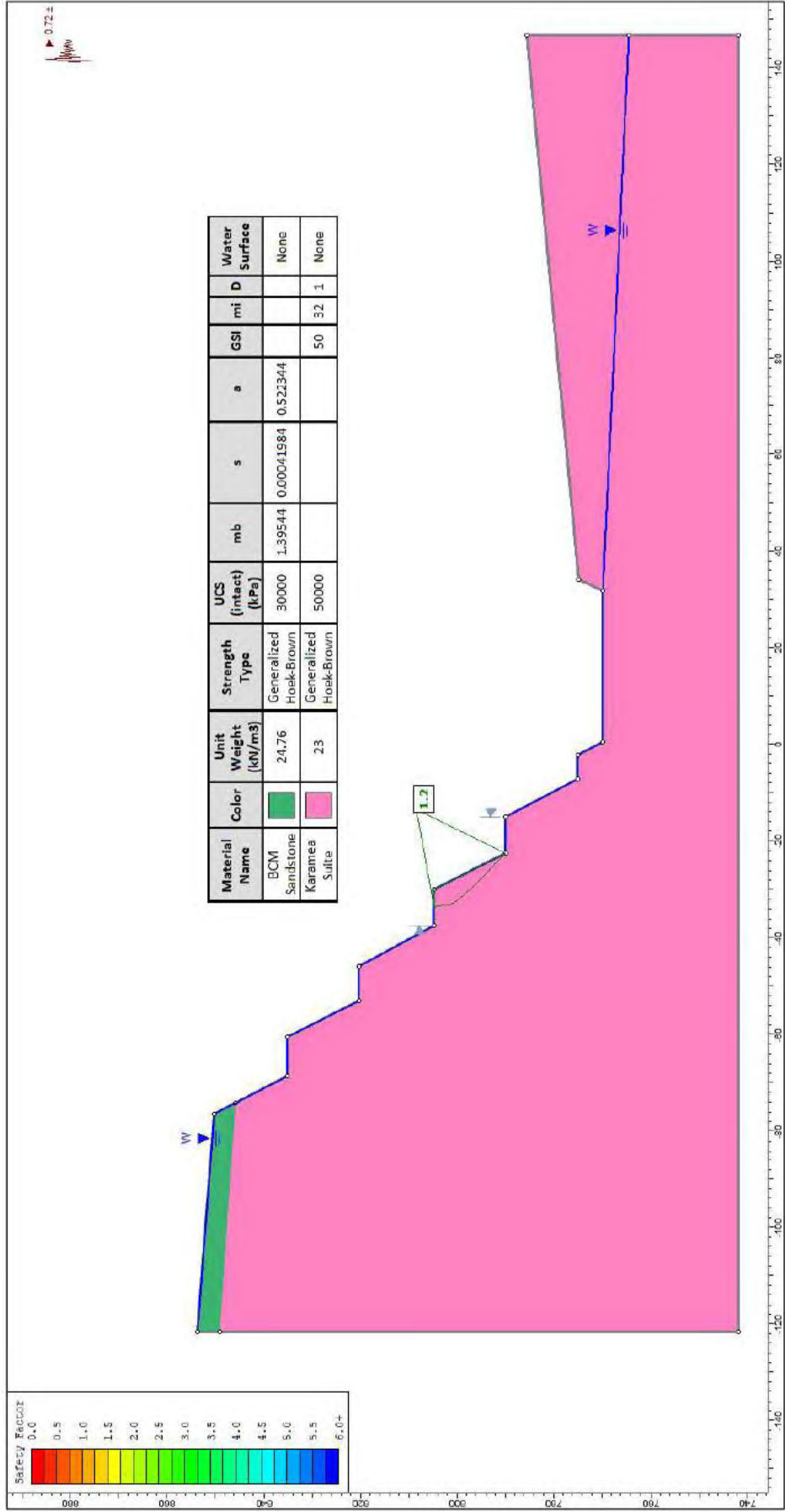
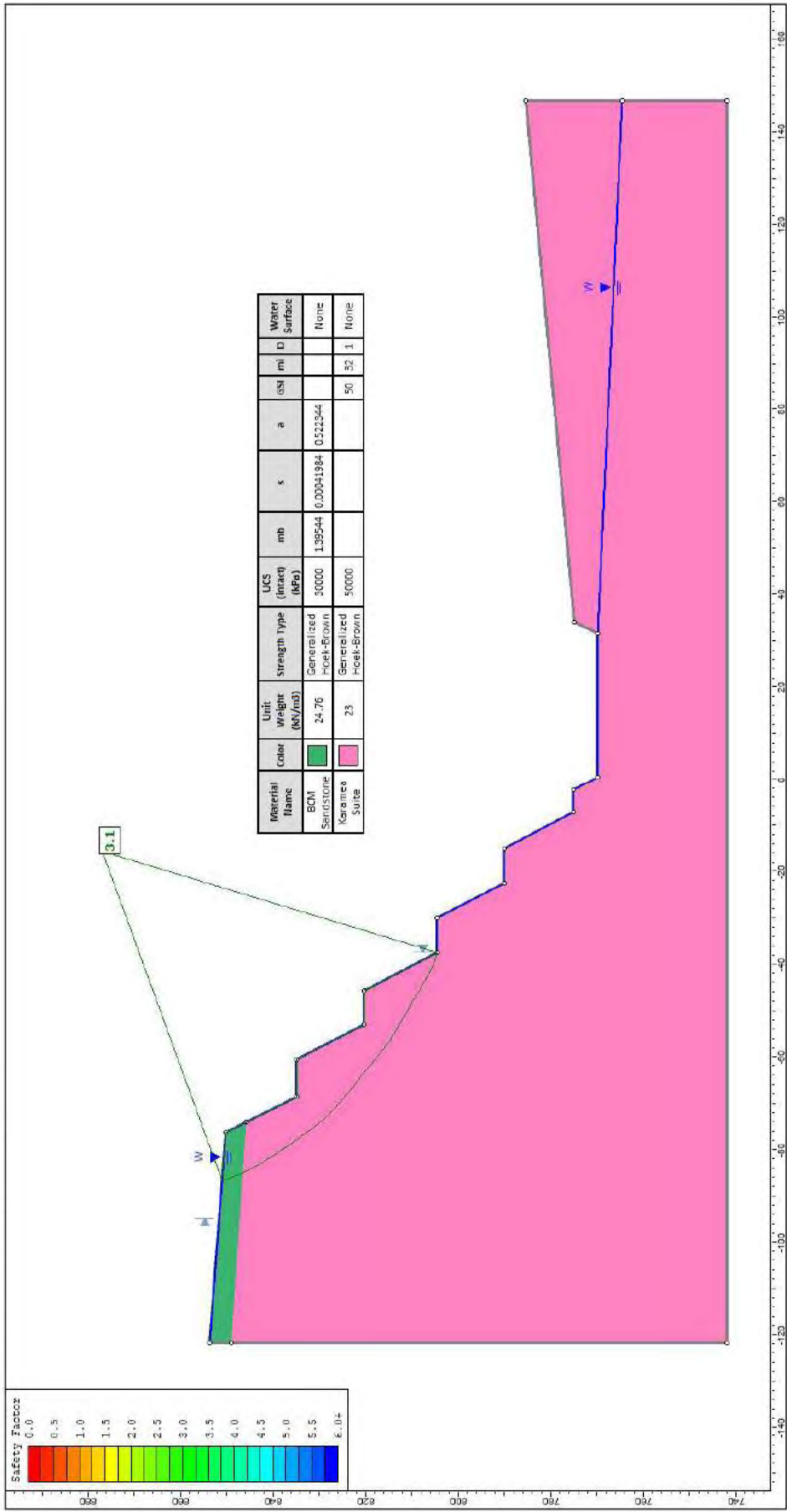


Figure D76: CFS Stage 3 – Single Bench (granite) – Seismic PGA 0.72g.

CFS Stage 3 - Multiple Bench (granite and sandstone)



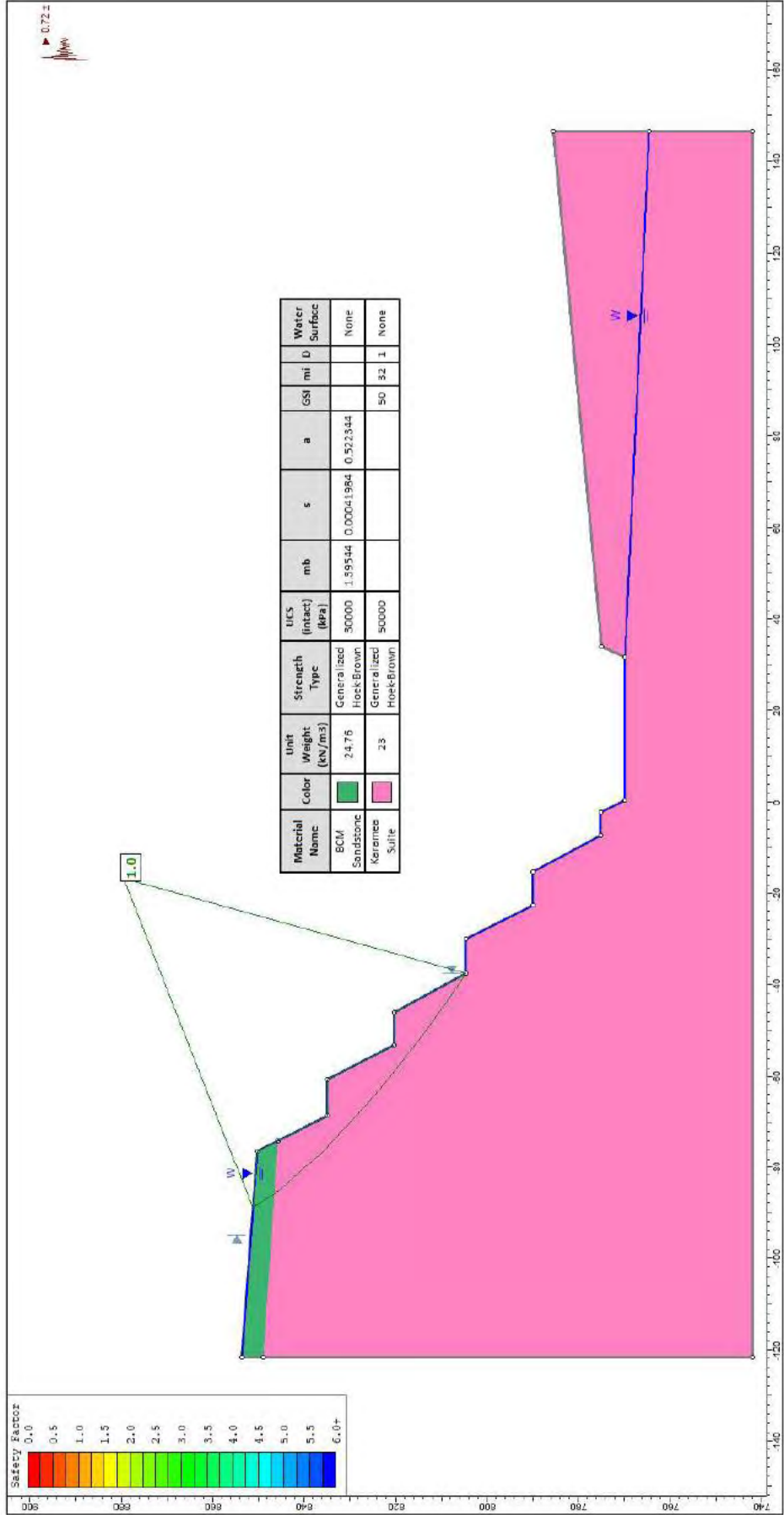


Figure D78: CFS Stage 3 – Multiple Bench (granite and sandstone) – Seismic PGA 0.72.

CFS Stage 3 – Multiple Bench (granite)

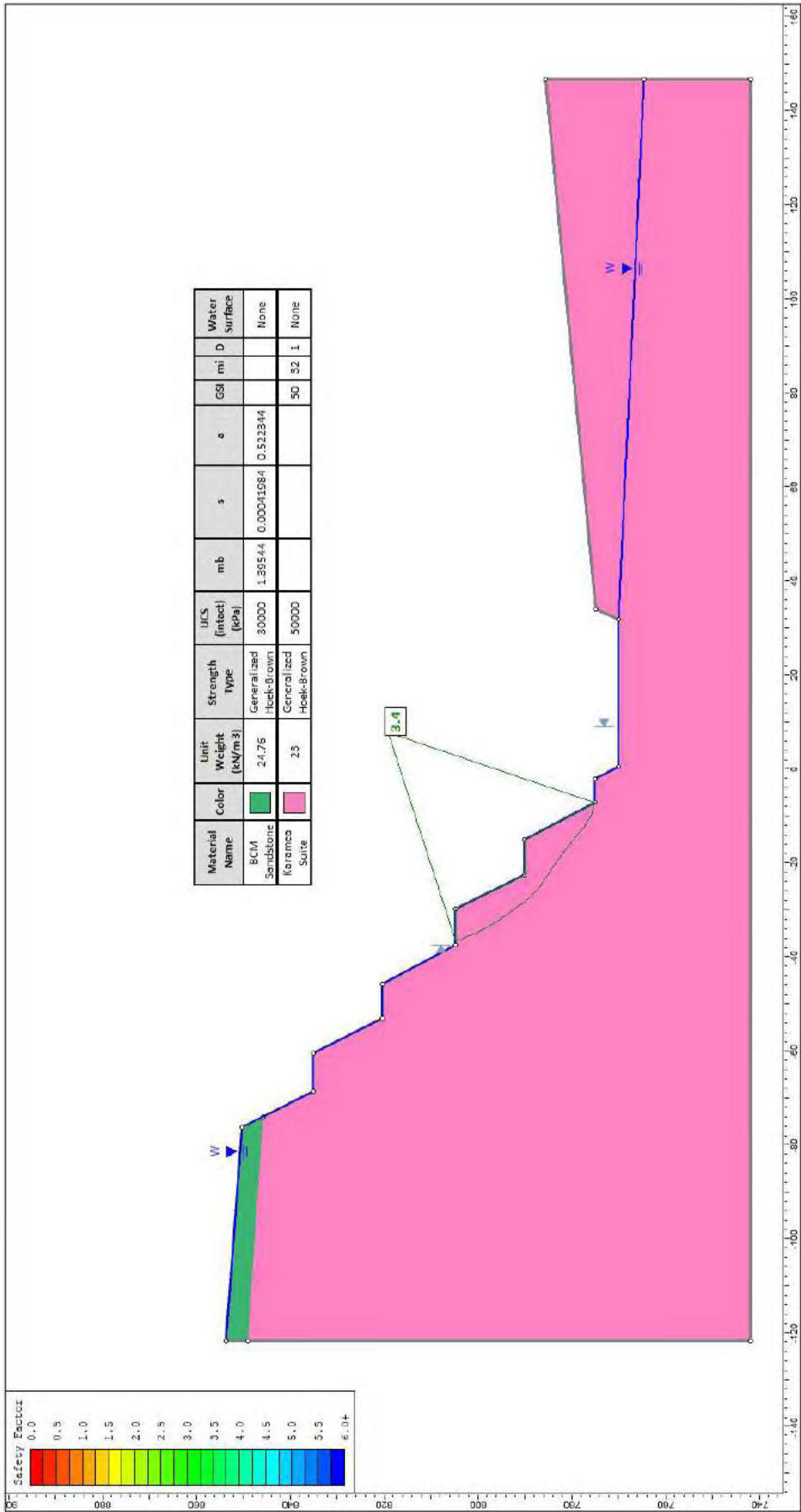


Figure D79: CFS Stage 3 – Multiple Bench (granite) – Static.

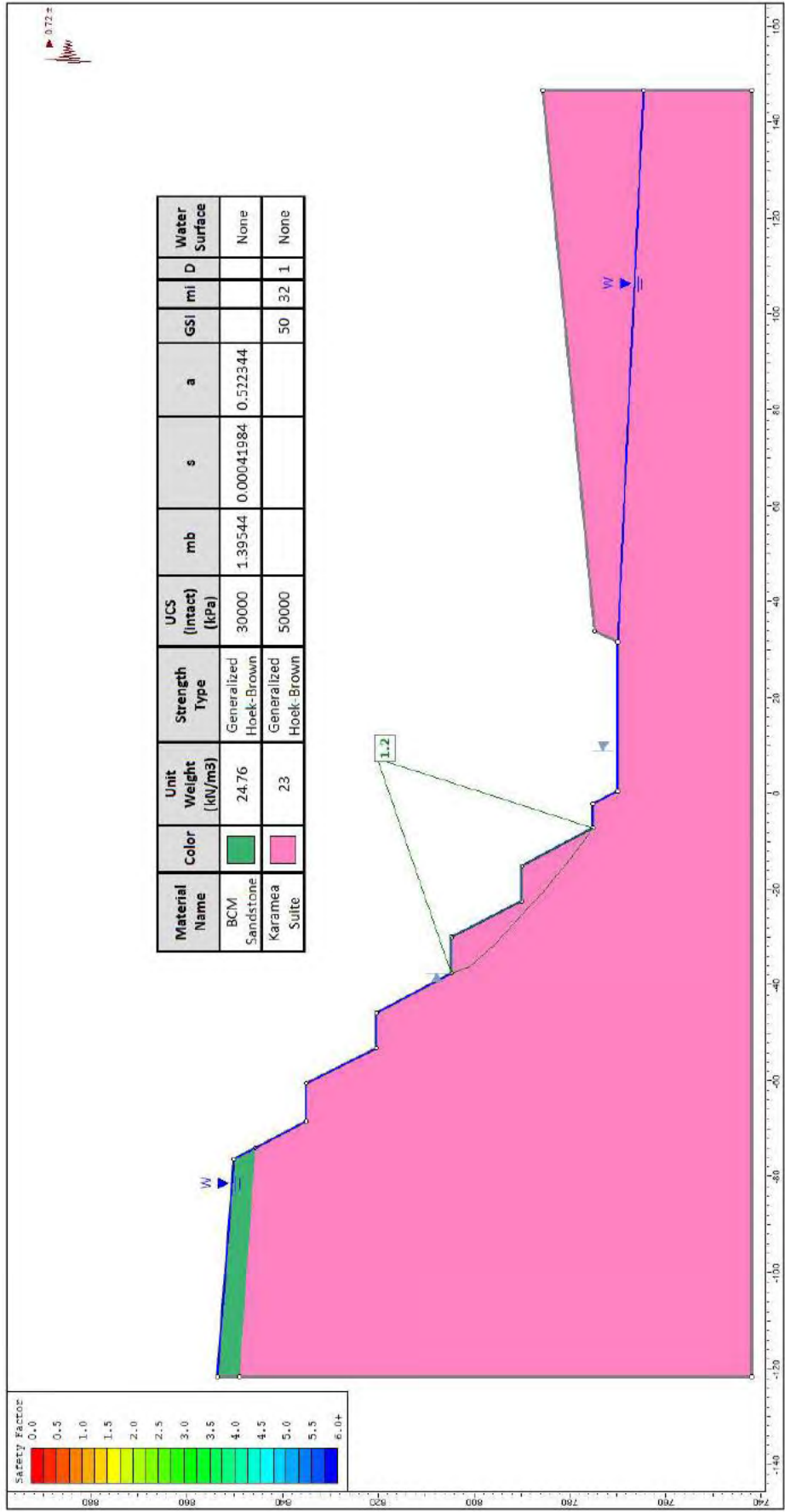
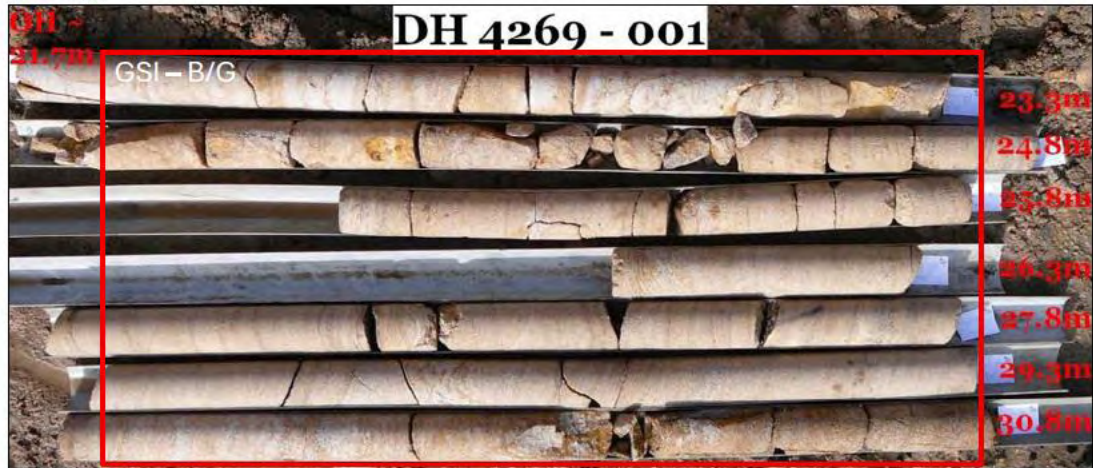


Figure D80: CFS Stage 3 – Multiple Bench (granite) – Seismic PGA 0.72g.

Appendix E: GSI Assessment From Core Photos

APPENDIX E - GEOLOGICAL STRENGTH INDEX (GSI) APPRAISAL - CORE PHOTOS

BH 4269



BH4269 - Photo 1; 29.7-30.8m



BH4269 - Photo 2; 30.8-37.1m



BH4269 - Photo 3; 37.1-43.1m



BH4269 - Photo 4; 43.1-50.3m



BH4269 - Photo 5; 50.3-57.8m



BH4269 - Photo 6; 57.8-65.3m



BH4269 - Photo 7; 65.3-69.8m EOH

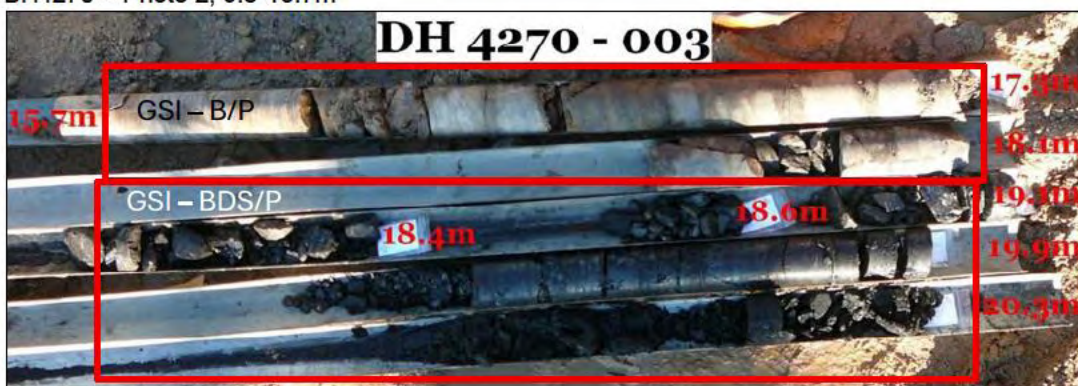
BH4270 -



BH4270 - Photo 1; 4.4-9.8m



BH4270 - Photo 2; 9.8-15.7m



BH4270 - Photo 3; 15.7-20.3m



BH4270 - Photo 4; 20.3-27.3m



BH4270 - Photo 5; 27.3-35.3m



BH4270 - Photo 6; 35.3-42.1m



BH4270 - Photo 7; 42.1-50.0m



BH4270 - Photo 8; 50.0-56.3m



BH4270 - Photo 9; 56.3-62.8m EOH

BH4271



BH4271 - Photo 1; 10.0-17.1m



BH4271 - Photo 2; 17.1-24.6m



BH4271 - Photo 3; 24.6-32.1m



BH4271 - Photo 4; 32.1-39.6m



BH4271 - Photo 5; 39.6-47.1m



BH4271 - Photo 6; 47.1-56.1m EOH

BH4277



BH4277 - Photo 1; 12.1-18.8m



BH4277 - Photo 2; 18.8-22.3m



BH4277 - Photo 3; 22.3-26.2m



BH4277 - Photo 4; 26.2-30.4m



BH4277 - Photo 5; 30.4-35.3m



BH4277 - Photo 6; 35.3-42.8m



BH4277 - Photo 7; 42.8-50.3m



BH4277 - Photo 8; 50.3-56.3m



BH4277 - Photo 9; 56.3-59.3m EOH

BH4285



BH4285 - Photo 1; 20.0-27.2m



BH4285 - Photo 2; 27.2-33.5m



BH4285 - Photo 3; 33.5-39.3m



BH4285 - Photo 4; 39.3-47.2 m



BH4285 - Photo 5; 47.2-54.5m



BH4285 - Photo 6; 54.5-62.1m EOH

BH4286



BH4286 - Photo 1; 12.0-18.1m



BH4286 - Photo 2; 18.1-24.4m



BH4286 - Photo 3; 24.4-30.7m



BH4286 - Photo 4; 30.7-37.7m



BH4286 - Photo 5; 37.7-44.2m



BH4286 - Photo 6; 44.2-51.7m



BH4286 - Photo 7; 51.7-59.2m EOH

BH4307



BH4307 - Photo 1; 10.0-17.5m



BH4307 - Photo 2; 17.5-22.8m



BH4307 - Photo 3; 22.8-29.5m



BH4307 - Photo 4; 29.5-37.0m



BH4307 - Photo 5; 37.0-44.4m

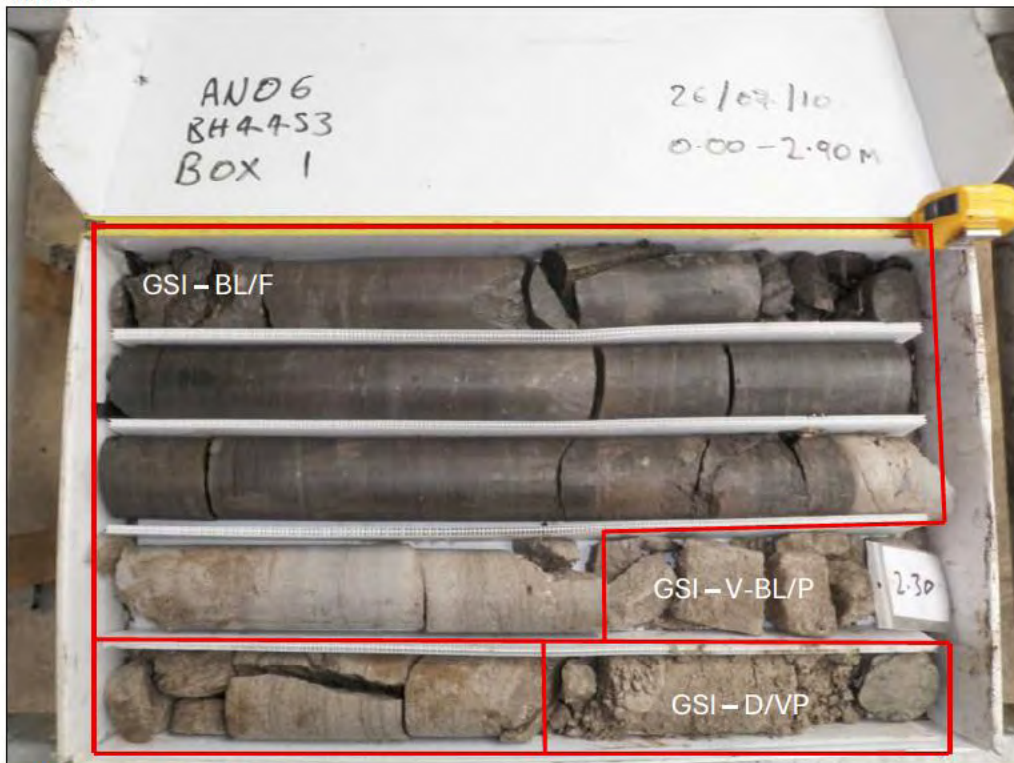


BH4307 - Photo 6; 44.4-52.0m



BH4307 - Photo 7; 52.0-61.0m EOH

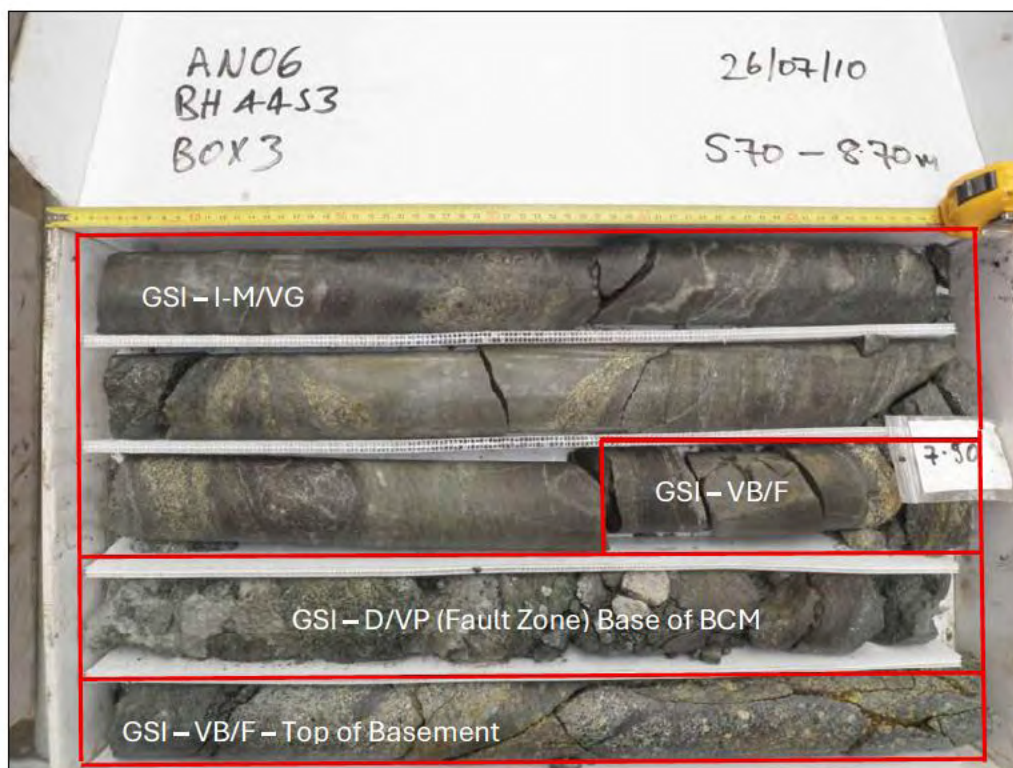
BH4453



BH4453 - Box 1; 0.0-2.90m



BH4453 - Box 2; 2.90-5.70m



BH4453 - Box 3; 5.70-8.70m



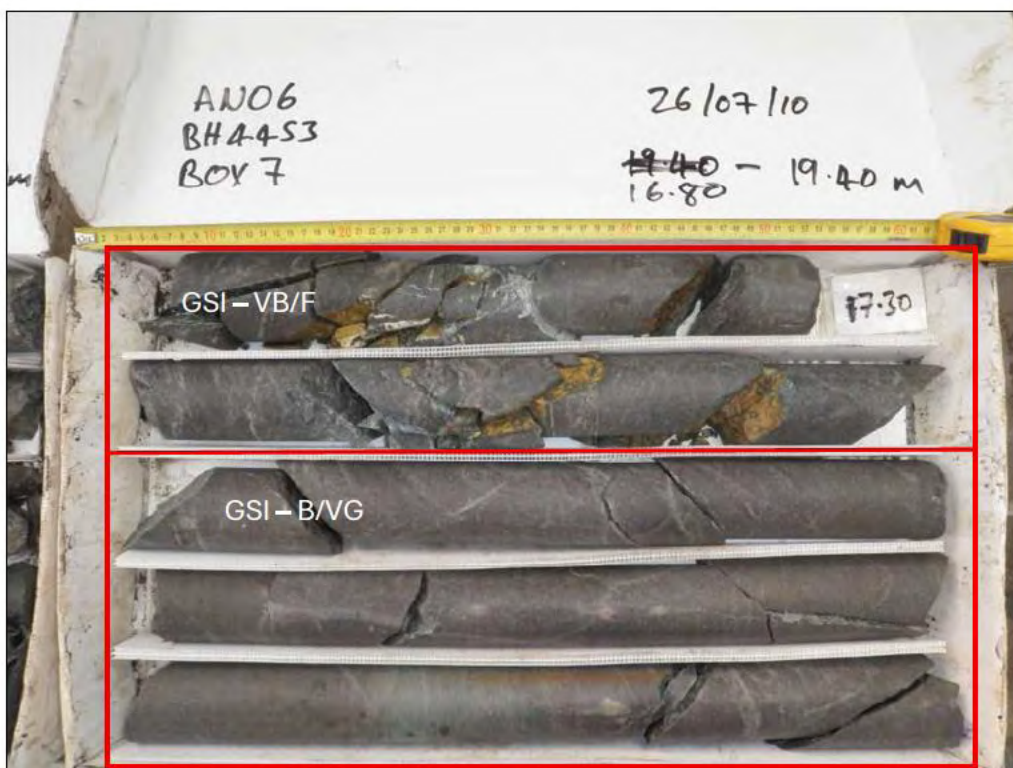
BH4453 - Box 4; 8.70-11.10m



BH4453 - Box 5; 11.10-13.90m



BH4453 - Box 6; 13.90-16.80m



BH4453 - Box 7; 16.80-19.40m



BH4453 - Box 8; 19.40-22.90m



BH4453 - Box 9; 22.90-24.60m



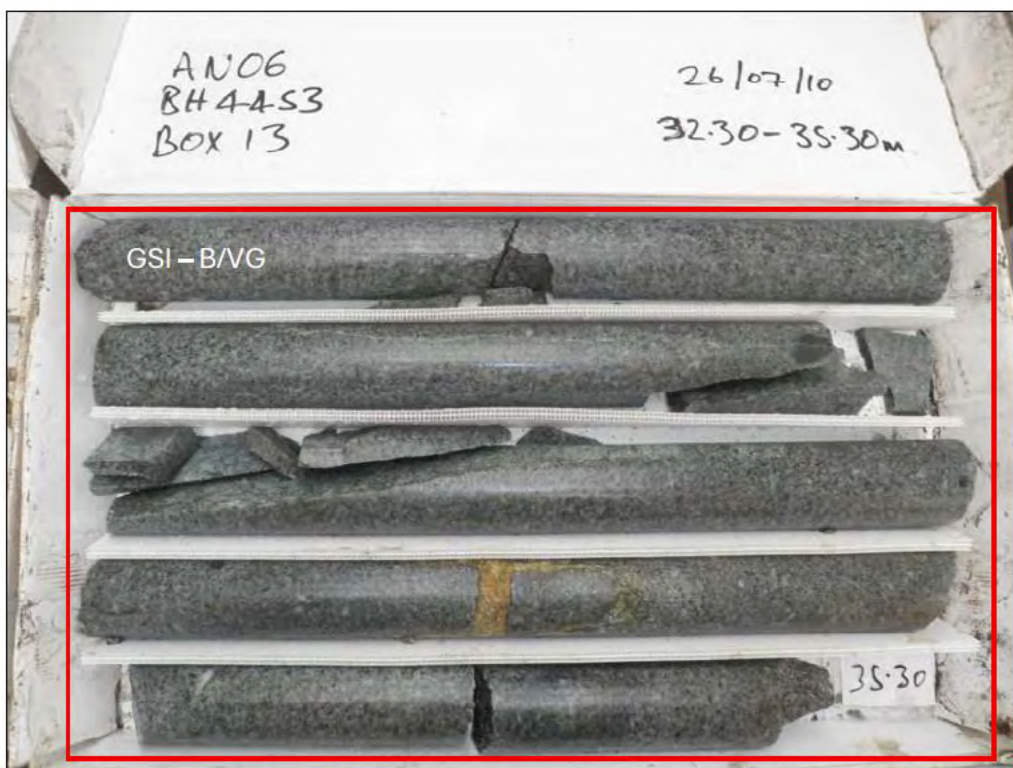
BH4453 - Box 10; 24.60-27.30m



BH4453 - Box 11; 27.30-29.90m



BH4453 - Box 12; 29.90-32.60m



BH4453 - Box 13; 32.30-35.30m



BH4453 - Box 14; 35.30-38.10m



BH4453 - Box 15; 38.10-41.10m



BH4453 - Box 16; 41.10-43.90m



BH4453 - Box 17; 43.90-46.80m

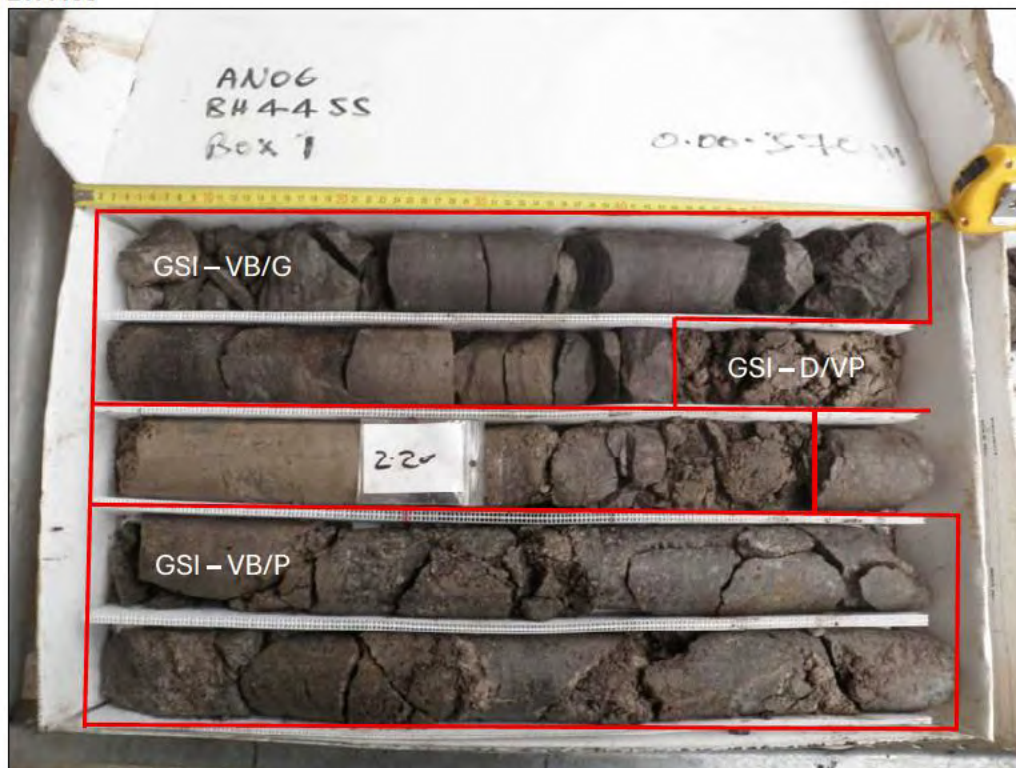


BH4453 - Box 18; 46.80-49.50m

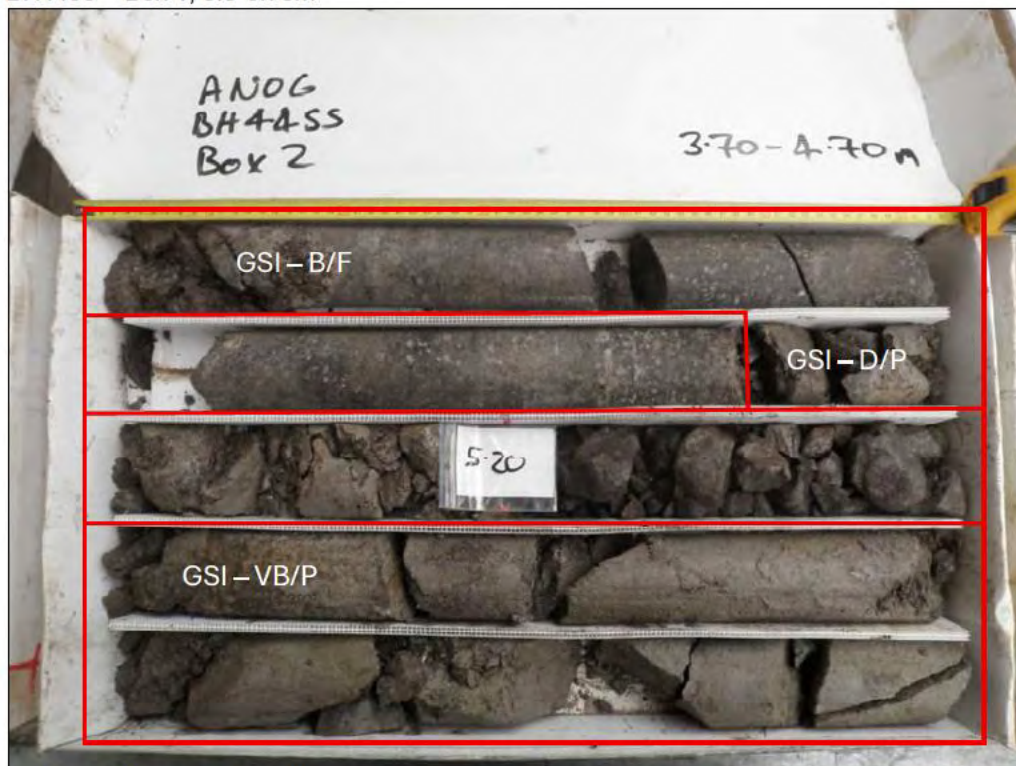


BH4453 - Box 19; 49.50-50.30m EOH

BH4455



BH4455 - Box 1; 0.0-3.70m



BH4455 - Box 2; 3.70-4.70m



BH4455 - Box 3; 4.70-9.05m



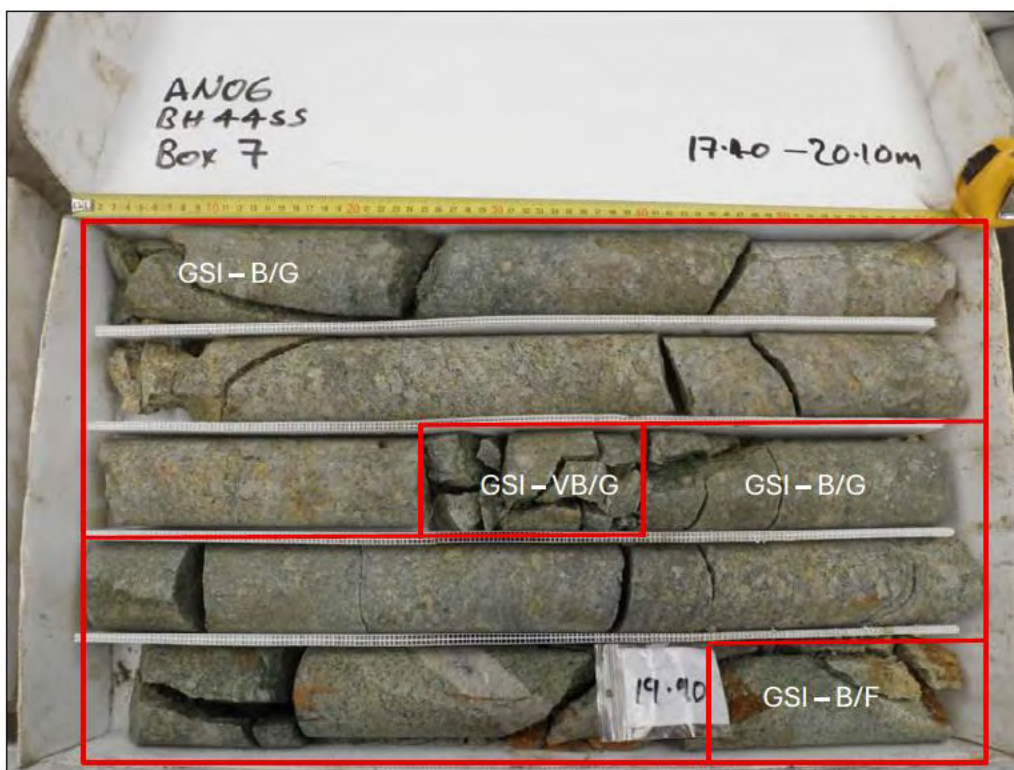
BH4455 - Box 4; 9.05-11.90m



BH4455 - Box 5; 11.90-14.70m



BH4455 - Box 6; 14.70-17.40m



BH4455 - Box 7; 17.40-20.10m



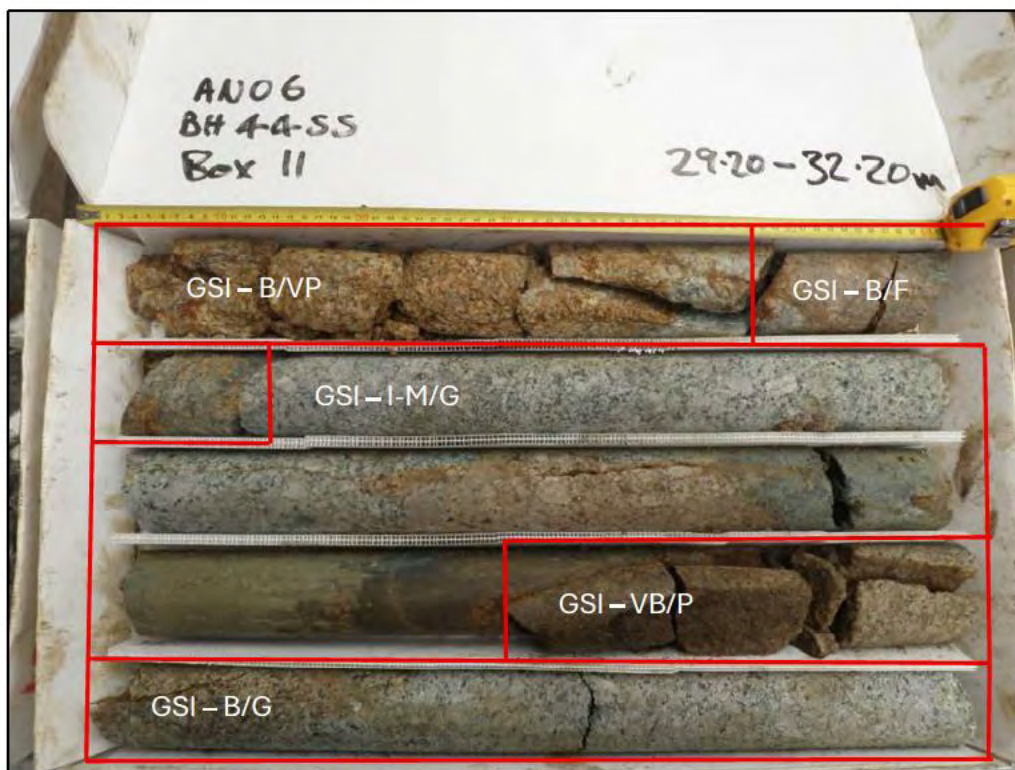
BH4455 - Box 8; 20.10-23.00m



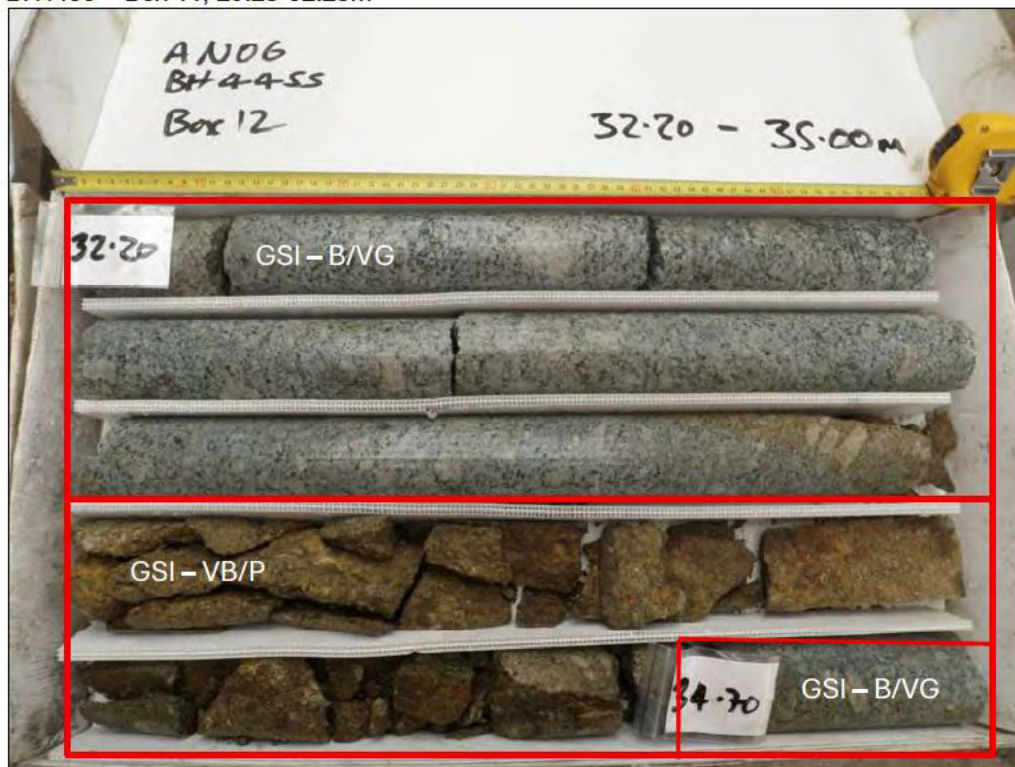
BH4455 - Box 9; 23.00-26.10m



BH4455 - Box 10; 26.10-29.20m



BH4455 - Box 11; 29.20-32.20m



BH4455 - Box 12; 32.20-35.00m



BH4455 - Box 13; 35.00-37.70m



BH4455 - Box 14; 37.70-40.50m



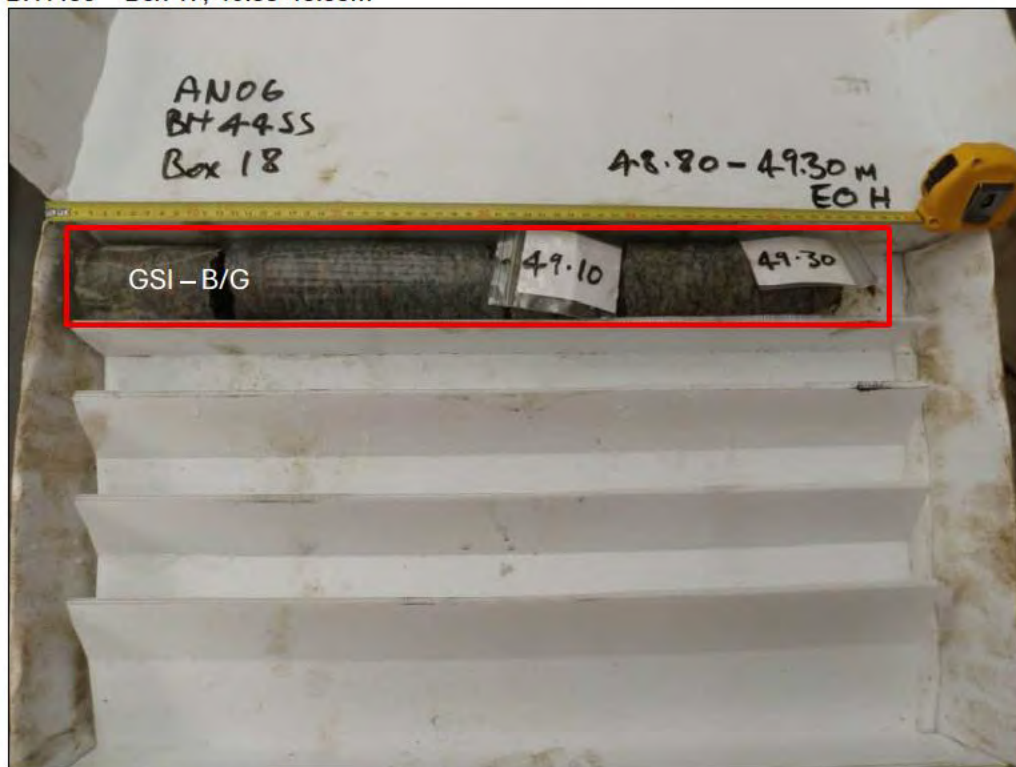
BH4455 - Box 15; 40.50-43.30m



BH4455 - Box 16; 43.30-46.00m

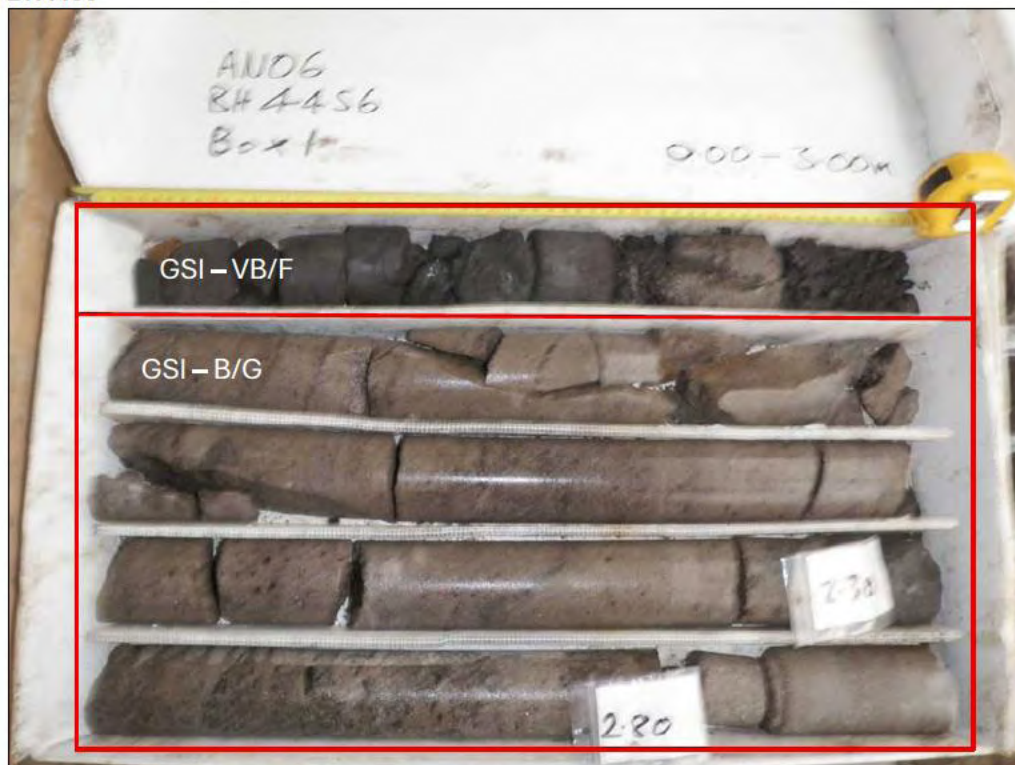


BH4455 - Box 17; 46.00-48.80m

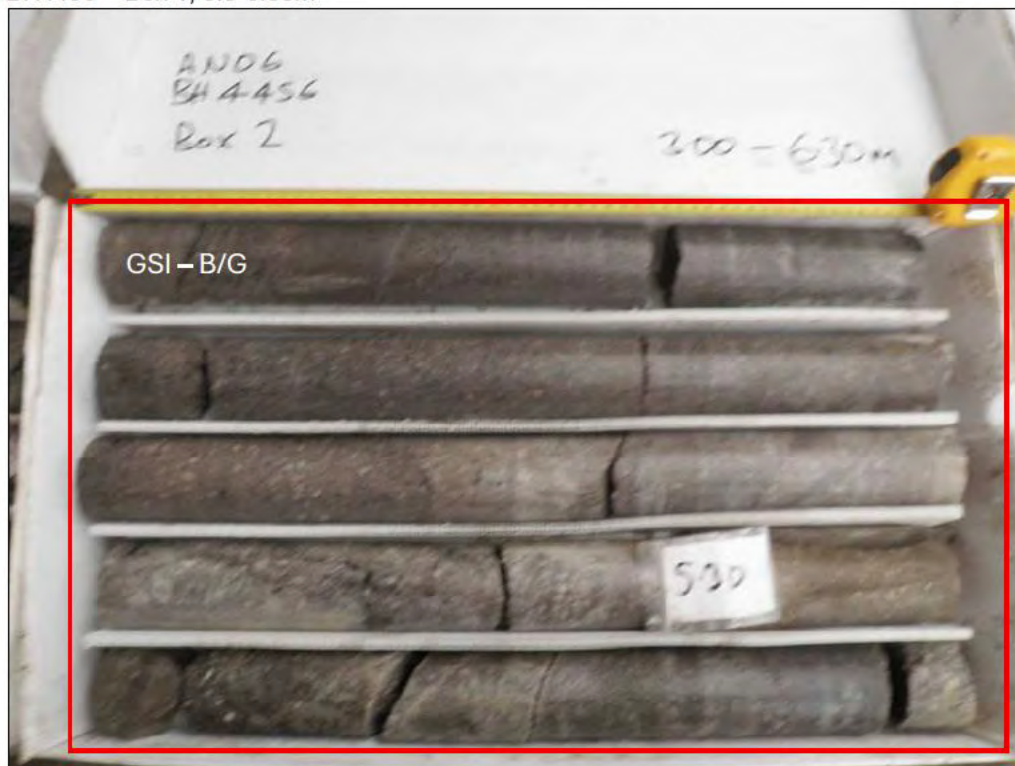


BH4455 - Box 18; 48.80-49.30m EOH

BH4456



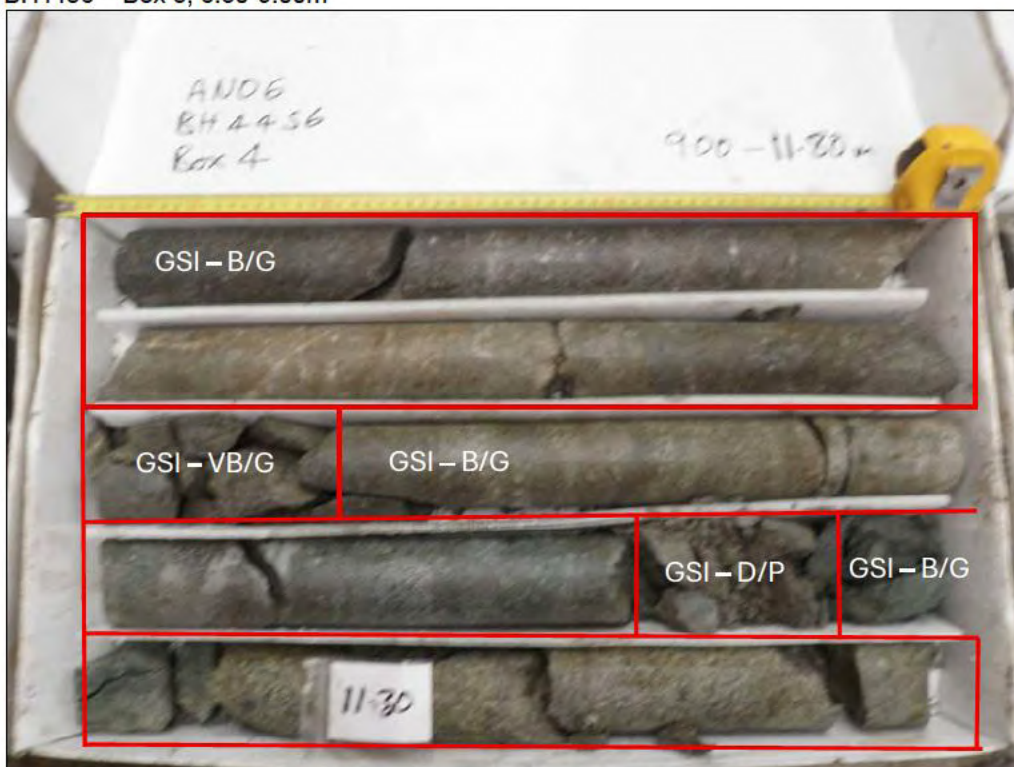
BH4456 - Box 1; 0.0-3.00m



BH4456 - Box 2; 3.00-6.30m



BH4456 - Box 3; 6.30-9.00m



BH4456 - Box 4; 9.00-11.80m



BH4456 - Box 5; 11.80-14.60m



BH4456 - Box 6; 14.60-17.10m



BH4456 - Box 7; 17.10-19.70m



BH4456 - Box 8; 19.70-22.40m



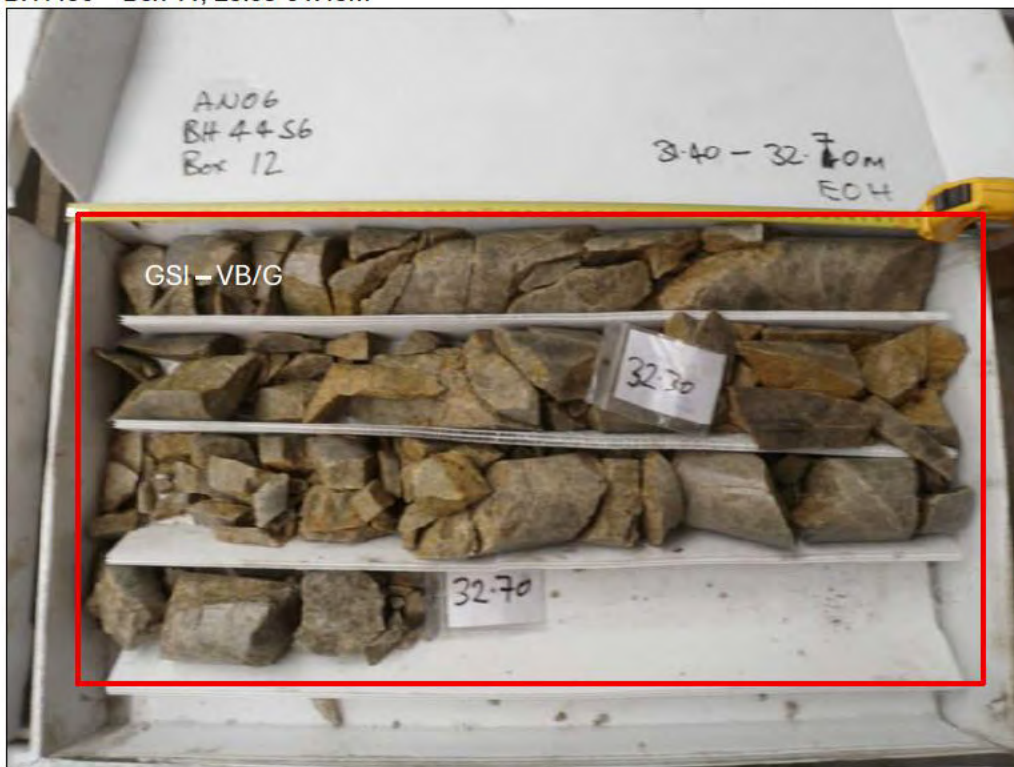
BH4456 - Box 9; 22.40-24.30m



BH4456 - Box 10; 24.30-28.30m



BH4456 - Box 11; 28.30-31.40m



BH4456 - Box 12; 31.40-32.70m EOH

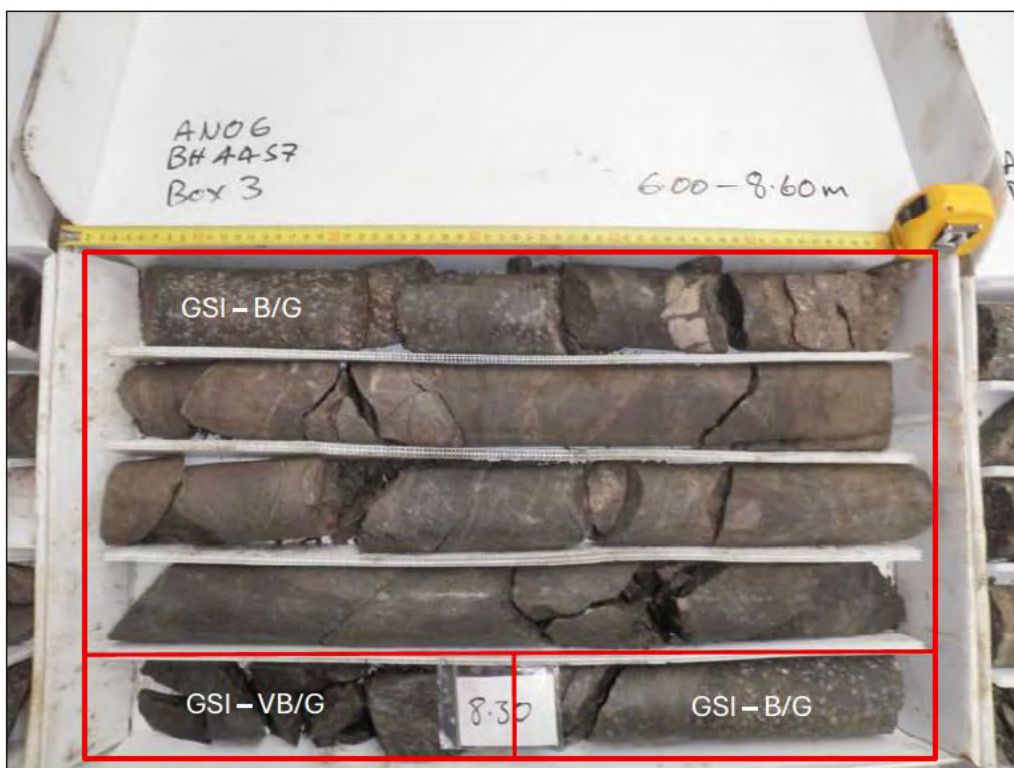
BH4457



BH4457 - Box 1; 0.0-3.20m



BH4457 - Box 2; 3.20-6.00m



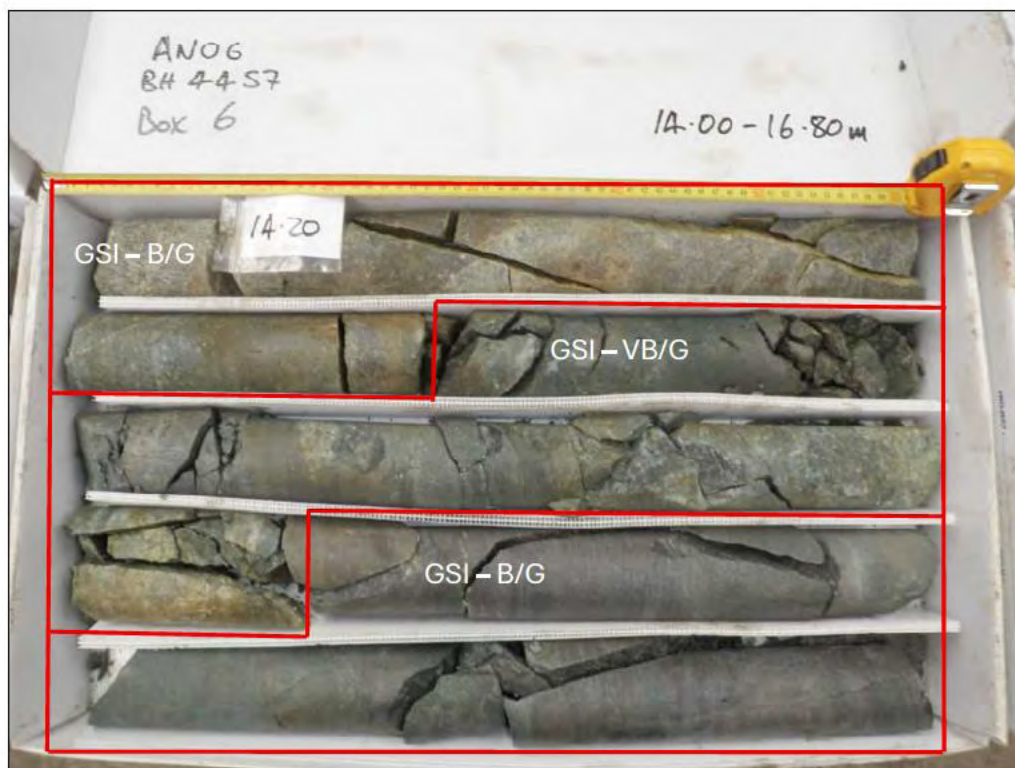
BH4457 - Box 3; 6.00-8.60m



BH4457 - Box 4; 8.60-11.20m



BH4457 - Box 5; 11.20-14.00m



BH4457 - Box 6; 14.00-16.80m



BH4457 - Box 7; 16.80-19.40m



BH4457 - Box 8; 19.40-22.60m



BH4457 - Box 9; 22.60-25.50m



BH4457 - Box 10; 25.50-27.90m



BH4457 - Box 11; 27.90-31.00m



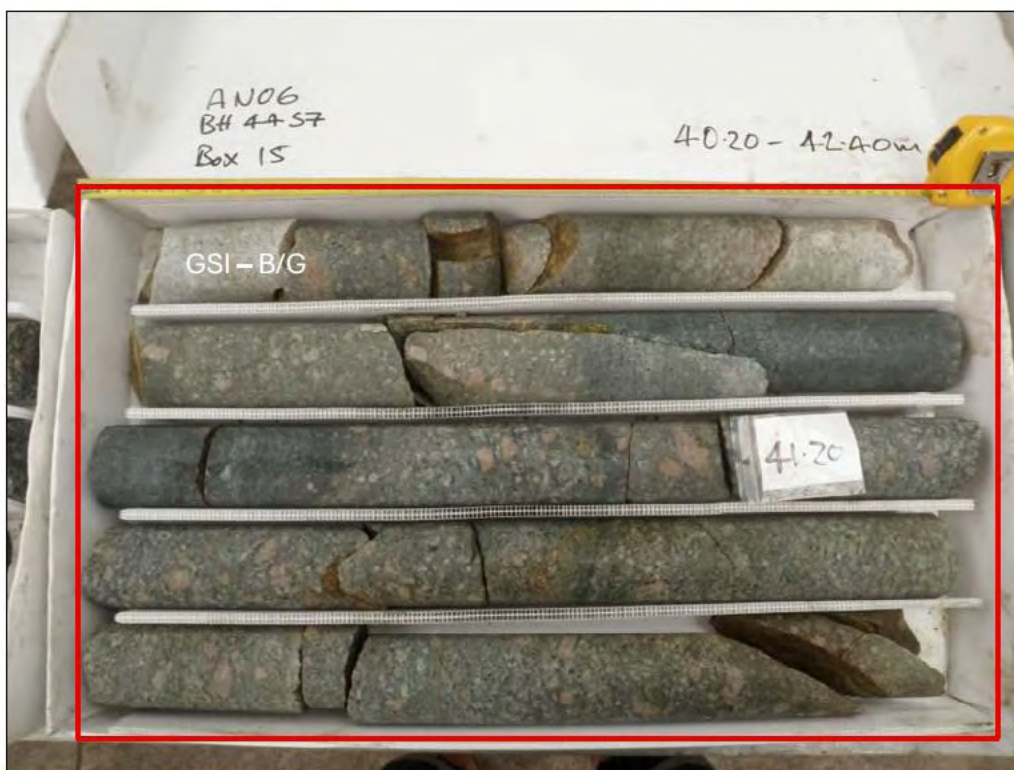
BH4457 - Box 12; 31.00-34.90m



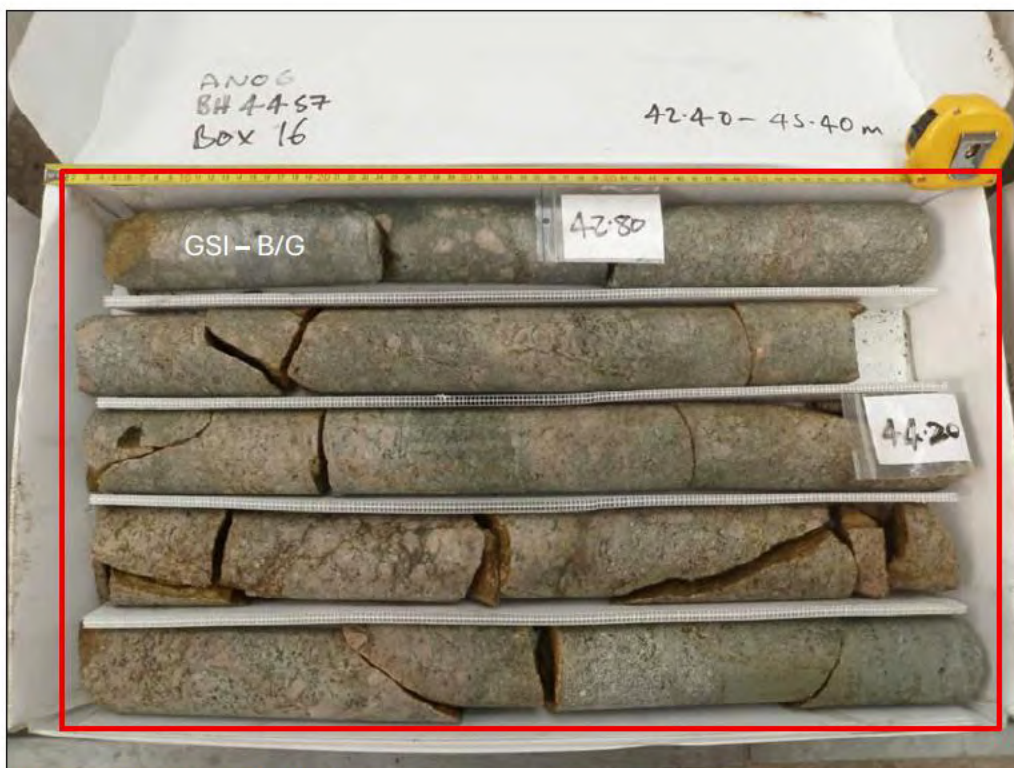
BH4457 - Box 13; 34.90-37.10m



BH4457 - Box 14; 37.10-40.20m



BH4457 - Box 15; 40.20-42.40m



BH4457 - Box 16; 42.40-45.40m

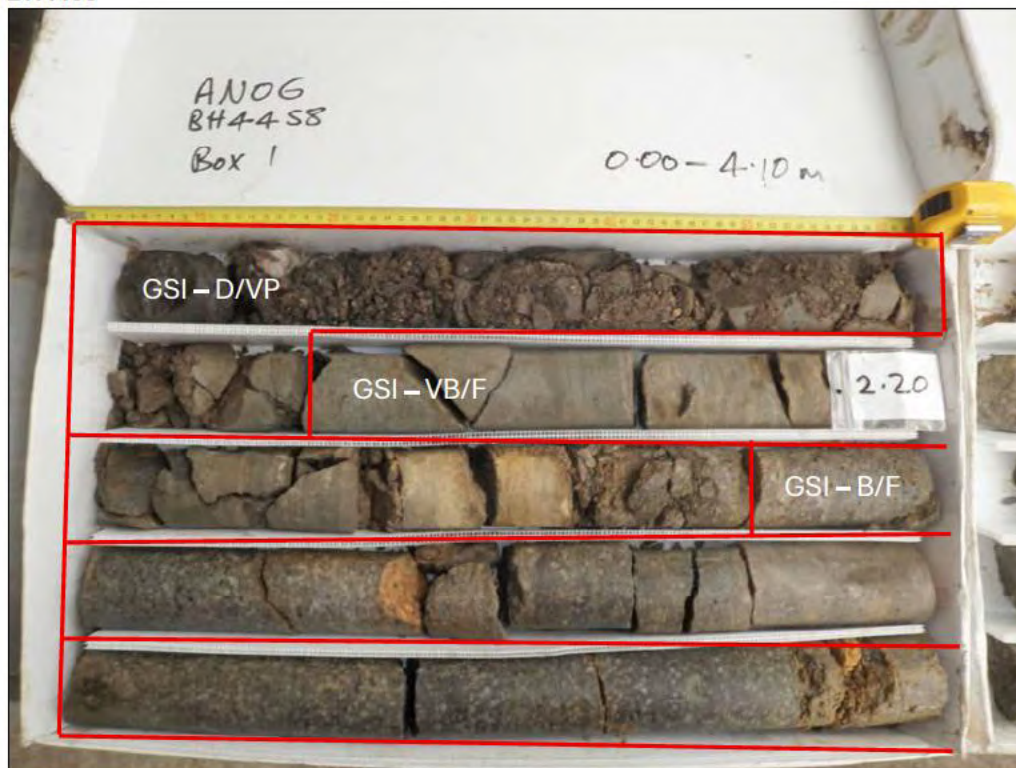


BH4457 - Box 17; 45.40-48.80m



BH4457 - Box 18; 48.80-50.20m EOH

BH4458



BH4458 - Box 1; 0.0-4.10m



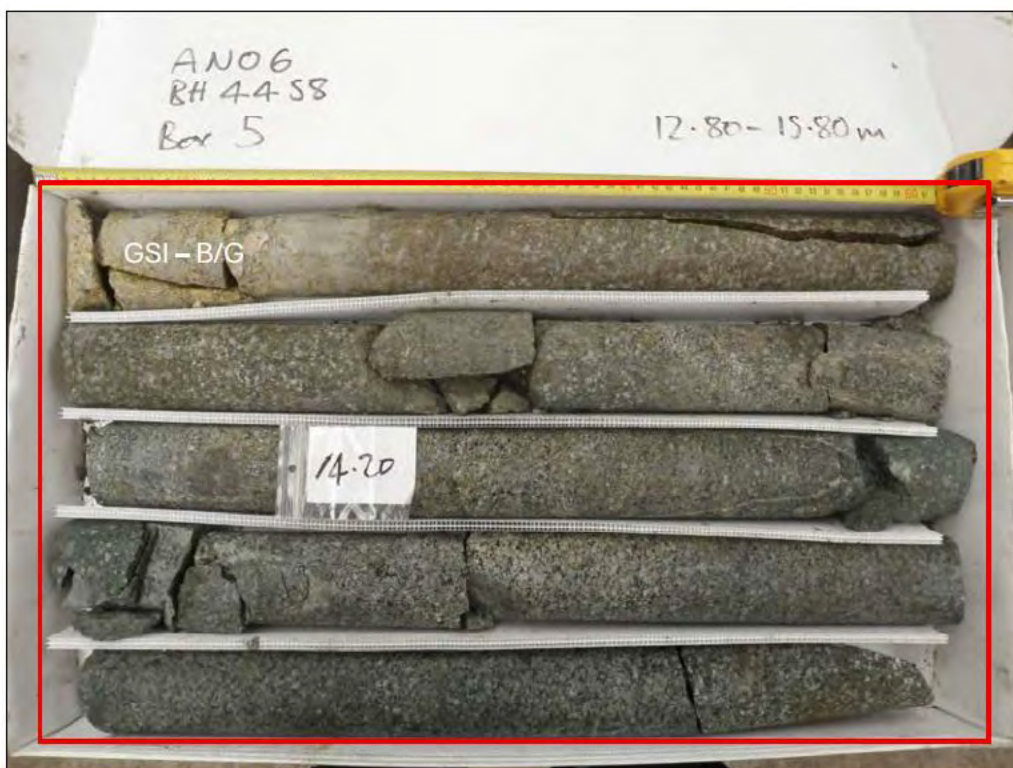
BH4458 - Box 2; 4.10-6.90m



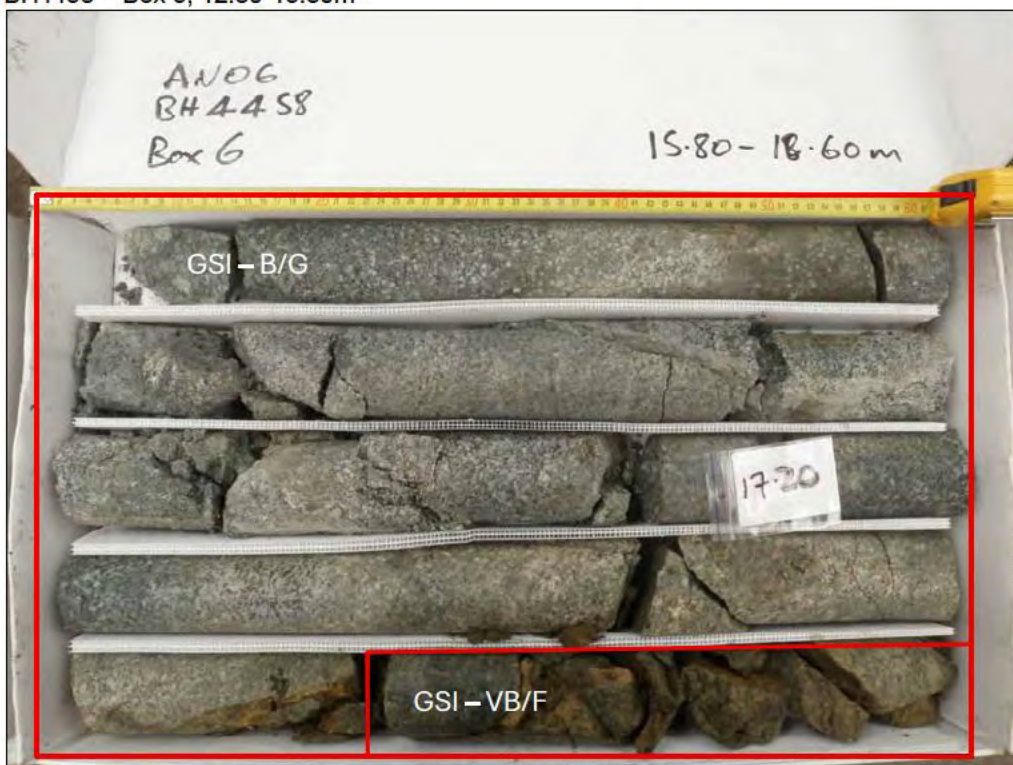
BH4458 - Box 3; 6.90-9.90m



BH4458 - Box 4; 9.90-12.80m



BH4458 - Box 5; 12.80-15.80m



BH4458 - Box 6; 15.80-18.60m



BH4458 - Box 7; 18.60-21.90m



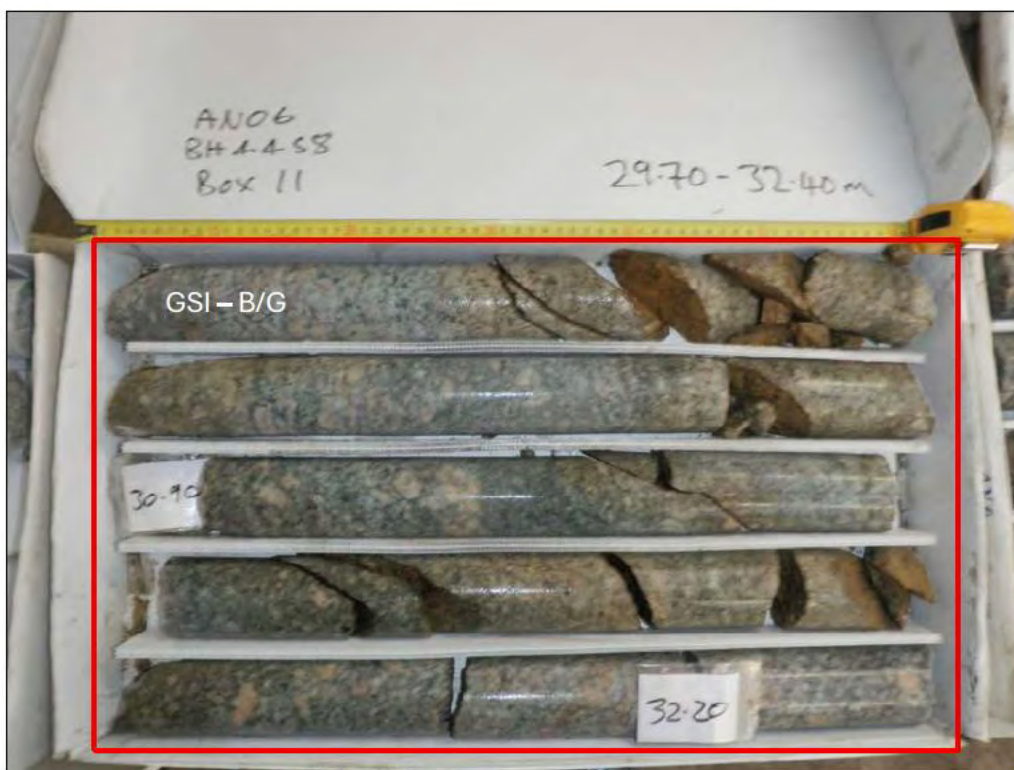
BH4458 - Box 8; 21.90-24.00m



BH4458 - Box 9; 24.00-26.90m



BH4458 - Box 10; 26.90-29.70m



BH4458 - Box 11; 29.70-32.40m



BH4458 - Box 12; 32.40-35.30m



BH4458 - Box 13; 35.30-38.20m



BH4458 - Box 14; 38.20-41.00m



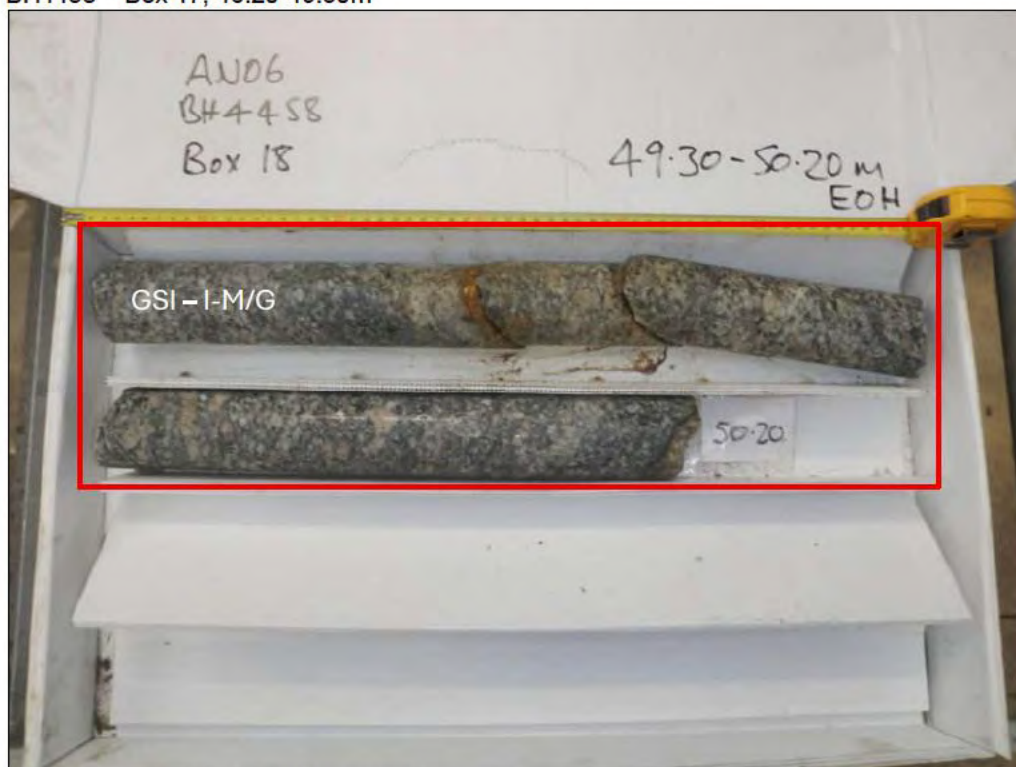
BH4458 - Box 15; 41.00-43.00m



BH4458 - Box 16; 43.00-46.20m



BH4458 - Box 17; 46.20-49.30m



BH4458 - Box 18; 49.30-50.20m EOH

Appendix F: A-Drive Borrow Pit – UCS Test Results

Table F1: Uniaxial Compressive Strength (Intact Rock) Test Results (UCS)

Drillhole ID	Depth (m bgl)	Lithology	Weathering	Logged Strength	UCS (MPa)
BH4453	10.6	Granite -Coarse Grained	SW	MS	26.2
BH4453	12.75	Granite – Fine Grained	UW	ST	49.6
BH4453	20.62	Hornfels	UW	ST	66.6
BH4453	31.31	Granite -Coarse Grained	UW	ST	97.4
BH4453	43.0	Granite -Coarse Grained	UW	VS	151.3
BH4455	13.86	Granite -Coarse Grained	SW	MWK	7.72
BH4455	22.69	Granite -Coarse Grained	SW	MWK	4.99
BH4455	25.64	Granite -Coarse Grained	SW	MS	24.0
BH4455	31.94	Granite -Coarse Grained	SW	MS	43.5
BH4455	40.17	Hornfels	UW	VS	152.2
BH4455	48.78	Granite -Coarse Grained	SW	ST	64.5
BH4456	10.47	Granite -Coarse Grained	HW	MWK	13.8
BH4456	15.7	Granite -Coarse Grained	UW	VS	72.5
BH4456	21.13	Granite -Coarse Grained	UW	VS	116
BH4457	9.67	Granite -Coarse Grained	SW	MWK	10.9
BH4457	13.04	Hornfels	SW	MS	24.3
BH4457	18.51	Granite -Coarse Grained	UW	VS	133.5
BH4457	25.12	Granite -Coarse Grained	UW	VS	109.5
BH4457	34.9	Granite -Coarse Grained	UW	ST	68.4

Table F1: Uniaxial Compressive Strength (Intact Rock) Test Results (UCS)

Drillhole ID	Depth (m bgl)	Lithology	Weathering	Logged Strength	UCS (MPa)
BH4458	5.87	Granite -Coarse Grained	HW	MS	34.4
BH4458	9.55	Granite -Coarse Grained	HW	WK	2.41
BH4458	14.93	Granite -Coarse Grained	SW	MS	41.8
BH4458	26.98	Granite -Coarse Grained	UW	VS	116.8
BH4458	40.07	Granite -Coarse Grained	UW	VS	162.3
BH4458	45.76	Granite -Coarse Grained	UW	VS	183.9
BH7090	8	Granodiorite	MW	MS	31
BH7090	9	Granodiorite	MW	MS	28
BH7090	11	Granodiorite	MW	ST	42
BH7090	13	Granodiorite	MW	ST	41
BH7090	20	Granodiorite	UW	VS	102
BH7090	27	Granodiorite	UW	VS	104
BH7090	40	Granite – Fine Grained	SW	ST	68
BH7091	7	Granite – Fine Grained	HW	WK	12
BH7091	11	Granite – Fine Grained	MW	WK	16
BH7091	13	Granite – Fine Grained	MW	MS	39
BH7091	17	Granite – Fine Grained	UW	VS	105
BH7091	21	Granite – Fine Grained	UW	VS	90
BH7091	28	Granite – Fine Grained	UW	VS	88
BH7091	36	Granite – Fine Grained	MW	MS	26

Table F1: Uniaxial Compressive Strength (Intact Rock) Test Results (UCS)

Drillhole ID	Depth (m bgl)	Lithology	Weathering	Logged Strength	UCS (MPa)
BH7091	43	Granite – Fine Grained	MW	MS	55
BH7092	2	Granite – Fine Grained	SW	VS	82
BH7092	4	Granite – Fine Grained	UW	VS	118
BH7092	13	Granite – Fine Grained	UW	VS	107
BH7092	19	Granite – Fine Grained	UW	VS	120
BH7092	26	Granite – Coarse grained	UW	VS	118
BH7092	31	Granite – Fine Grained	UW	VS	118
BH7092	35	Granite – Fine Grained	UW	VS	121
BH7092	39	Granite – Fine Grained	UW	VS	100
BH7092	44	Granite – Fine Grained	UW	VS	109

Notes:

1. Source: Phillips, Alex (2010). Geotechnical Factual Report - Results of A-Drive Quarry investigation, Stockton Mine. Stockton Alliance, December 2010