Mount Fred South – Geotechnical Assessment Report Pre-Feasibility Study

Prepared for

BT Mining Limited

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Limitations:

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Executive Summary

Limited Buller Plateaux Fast Track Approval (FTA) application. The PFS focuses on the stability of proposed highwall, sump cut and engineered landforms (ELFs) fill profiles within the Mount Fred South (MFS) area.

The PFS design profiles were developed using the BTM supplied Basis of Design (BOD) which is informed by operational experience in comparable ground conditions across Stockton Mine. This approach ensured that the design assumptions were grounded in practical knowledge and site-specific relevance.

The MFS area is located approximately 1.5 km south of the southern extent of the existing Stockton Mine, within the Upper Waimangaroa Mining Permit (MP 41515) and New Zealand Petroleum and Minerals Exploration Permits (EPs 40628 and 61157.01). MFS lies adjacent to the Mt. Fred mine block, which is situated within the Stockton Coal Mining Licence (CML 37150). The MFS project is being considered for development as an operational opencast pit to support the continued expansion of the Stockton opencast mine.

A mine design provided by BT Mining (BTM) focuses on extracting the M2 coal resource from the Brunner Coal Measures (BCM) and includes two open cut pits including the East Pit and the West Pit), associated stormwater sump excavations, ex-pit overburden engineered landforms (ELF's) and in-pit ELF's which are sequenced to partially backfill the open pits during mining. Access to the mine site will be provided by one of two haul road options from either the north or south, the final alignment of the haul roads are yet to be confirmed.

A geotechnical model for the MFS area was developed using existing drillhole data, incorporating lithological and geotechnical logs along with interpolated and triangulated lithological contact surfaces. A structural geology model was also developed based on mapped and inferred fault intersections across the MFS area.

Downhole geophysical data including Sonic, ATV, and density logs were analysed to characterise oriented drill core defects using WellCAD software. A kinematic assessment was undertaken using the defect data to evaluate potential planar sliding, toppling, and wedge sliding mechanisms based on generalised highwall cut orientations. All failure types were found to be feasible, with wedge sliding identified as the most critical case across all highwall configurations. To complement the kinematic analysis, a qualitative risk assessment was conducted to evaluate how highwall orientations influence the likelihood and impact of failure mechanisms. The assessment identified Highwall 3 as having the most favourable orientation, with minimal risk of structurally controlled failure. In contrast, Highwalls 1 and 2 present the least favourable orientations, associated with the highest risk of kinematic instability across all assessed failure modes.

Due to the limited availability of geotechnical laboratory testing within the MFS area, a rock mass model was developed using representative strength parameters sourced from lithologies with similar geological characteristics in the Stockton Mine dataset. These parameters were selected to reflect the expected mechanical behaviour of the rock mass in the absence of site-specific test data. The developed rock mass model was used to support a 2D limit equilibrium stability analysis of the proposed MFS mine features. This analysis, conducted using Rocscience Slide2 software, assessed slope performance under static loading, seismic conditions, and elevated groundwater levels to inform geotechnical design and risk mitigation strategies.

Failure criteria (Factors of Safety) and peak ground acceleration (PGA) values for the slope stability assessment were defined in accordance with NZS1170 and the New Zealand Geotechnical Society's Module 1 guidance, ensuring alignment with industry standards. Modelling results indicate that the proposed highwall and engineered landform configurations are generally stable under static and elevated groundwater conditions. Minor slope failures with displacements of up to 30 cm were observed along bedding plane shears under seismic loading (SLS and ULS), particularly where coal seams are locally uplifted by faulting.

Given that the proposed mine sequencing involves buttressing most highwalls through the construction of in-pit engineered landforms (ELFs), the short-term stability during operations becomes the critical focus of geotechnical analysis. The primary short-term stability risk—common to coal measure deposits—is sliding along pre-existing bedding plane shears (BPS), typically associated with the roof and floor of coal seams. Although BCM coal deposits are generally near-horizontal, local faulting can cause draping and overstepping of these BPS surfaces. The current drillhole dataset is limited, which means these operational risks remain present.

The historical underground workings within the pit and sump footprints are not expected to pose significant geotechnical issues, as they will be excavated during mine construction. However, W_ELF1 is proposed over an area of shallow historical workings, which may collapse under construction activities due to their proximity to the surface.

This pre-feasibility study for the proposed Mount Fred South Mine has assessed key geotechnical factors, including slope geometry, material properties, geological structure, and historical underground workings. Kinematic analysis was used to evaluate potential failure mechanisms, with emphasis on faulting and structural discontinuities. Identified risks such as slope instability, subsidence, and rock mass variability are considered manageable through targeted operational controls, including Ground Control Management Plans (GCMPs), Trigger Action Response Plans (TARPs), and ongoing geotechnical monitoring. No major geotechnical risks have been identified that would preclude development. Further refinement of the geotechnical model and incorporation of probabilistic analysis during the feasibility stage will enhance risk mitigation and support informed design decisions.

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1.0 Introduction

BT Mining Limited (BTM) has engaged Pattle Delamore Partners Limited (PDP) to prepare a pre-feasibility level geotechnical assessment report for the proposed Mount Fred South coal study area (formally known as Deep Creek).

The Mount Fred South (MFS) prospect area is a historically underground mined brownfield site which is proposed to be an operational opencast pit that supports the extension of the currently operating Stockton opencast mine.

MFS is located within the Upper Waimangaroa Mining Permit 41515 (MP) New Zealand Petroleum and Minerals Exploration Permits 40628 (EP) and 61157.01 (EP)). It is adjacent to the current Mount Fred mine block located on the Stockton Coal Mining License (CML 37150). Refer to Figure 1.

The MFS is located approximately 35 km east-northeast of the town of Westport and, approximately 1 km southwest of the Stockton Mount Fred mine block. The Upper Waimangaroa MP 41515 covers an area of approximately 2933 hectares (ha), of which the proposed MFS footprint covers approximately 205 ha. The approximate footprint of the proposed MFS is indicated on Figure 1 (Appendix A).

There have been several previous concept studies undertaken at MFS. The current mine plan proposes constructing an access road from Mount Fred to MFS and a small Engineered Landform (ELF) would be required within the area of EP 61157.01 and EP 40628.

The purpose of this assessment is to provide geotechnical support for the completion of the pre-feasibility study for the MFS mine design, engineered landform design, stormwater sump layout, and access design. It also includes an evaluation of potential geotechnical effects and outlines how these will be managed to support the Buller Plateaux Fast Track Approval (FTA) application.

1.1 Previous Work & Supplied Data

The following reports and drilling programmes have historically been completed for BTM across the MFS area have been provided by BTM:

- Deep Creek – Mining Concept Study by Golder Associates (NZ) Limited, dated December 2019.
- Exploration drilling undertaken in 2005, 2006, 2012 and 2023 with drill holes generally identified with the prefix DC for Deep Creek these are detailed in Table 1.
- BTM mapping and notes from collapsed coal workings.

No previous site-specific geotechnical investigations have been completed across the MFS area.

BTM provided the following data to assist in this pre-feasibility study which has been reviewed as part of this geotechnical assessment:

- Vulcan ISIS drillhole database, Acquire database core logs, core photographs and rock strength data from Mount Fred pit and wider Stockton area.
- Stockton BOD for pit highwalls, ELFs and haul roads.
- Pre-feasibility mine design triangulations for proposed pits, ELF's (staged and final), and stormwater sumps, dated October 2024.
- Northern Haul Road alignment option from the existing Mount Fred pit to MFS.
- Southern Haul Road alignment option from MFS south to Ceder Creek Road upstream of the Waimangaroa Gorge.
- Fault trace locations used in the BTM geological model.
- Drillhole logs and core photographs from DC01 to DC46.
- Down-hole geophysical raw data including Density, Natural Gamma, Full Waveform Sonic and Acoustic Televiewer from DC16, DC19, DC20, DC22 to DC24, DC26, DC27, DC30 to DC32, DC34, DC37, DC39, DC41, DC43, DC 44 and DC45.

Table 1 below is a summary of the mine features and ELF locations, with relevant drillholes, faults and if underground workings are expected.

Table 1: Summary of Proposed Mine Design Layout

Feature	ELF Location	Approx. Height / Depth (m)	Relevant Drill Holes[1]	Fault Intercept	U/G Workings[2]
Western Pit[1]	n/a	80	DC03, ***DC05***, DC06, DC11, DC12, ***DC13***, DC16, DC18, ***DC19***, ***DC20***, ***DC21***, DC23, ***DC26***, DC27, DC32	***DC12, DC20, DC26***	No
Eastern Pit[1]	n/a	90	DC04, DC10, ***DC28***, DC29, DC31, DC34	***DC28***	**Yes**
E_ELF1[1]	Ex Pit	75	None in footprint	No	No
E_ELF2[1]	Ex Pit	40	None in footprint	**Yes**	No
E_ELF3[1]	Ex Pit	50	None in footprint	No	No

Table 1: Summary of Proposed Mine Design Layout

Feature	ELF Location	Approx. Height / Depth (m)	Relevant Drill Holes[1]	Fault Intercept	U/G Workings[2]
E_ILF1	In-Pit	65	DC03 / DC21	No	No
E_ILF2	In-Pit	105	DC12	**Yes**	No
E_ILF3	In-Pit	95	DC11/DC18	No	No
E_ILF4	In-Pit	70	DC23	**Yes**	No
E_ILF5A	In-Pit	120	DC27	No	No
E_ILF6	In-Pit	155	DC06, DC16	**Yes**	No
W_ELF1	Ex Pit	75	None in footprint	No	**Yes**
W_ILF1	In-Pit	75	DC31, DC34	No	No
W_ILF2	In-Pit	75	1421, DC29, DC30,	No	**Yes**
W_ILF3	In-Pit	35	DC04, DC10	No	**Yes**
West Sump	n/a	55	None in footprint	**Yes**	**Yes**
East Sump	n/a	50	1427, DC32	No	No

Notes:

1. *The drillholes selected are the closest ones to the ELF / pit highwalls and the ones considered throughout this assessment. Bold italic – drill hole is in long-term highwall location depending on in-pit backfill timing.*
2. *Where mapped underground workings intercept the proposed highwalls and ELF's.*

2.0 Mine Design & Staging

2.1 Proposed Mine Design

The proposed pre-feasibility mine design pits shells, in-pit and ex-pit ELF, and stormwater sumps proposed (as of January 2025) are shown on Figure 2 and summarised against relevant drill holes, underground workings and faults intercepts are set out in Table 1.

The proposed mine is split into east and west pits, separated by the deeply incised Billo Stream with Deep Creek forming the eastern boundary of the mining area. Each section of the mine has initial ex-pit ELFs that will transition to in-pit ELFs (pit infill) as the mine excavation / extraction progresses. The following outlines the proposed order of construction of the MFS Mine:

- The west & east stormwater sumps will be constructed first before either pit is excavated.
- The East Pit is proposed to mine anticlockwise. Three ex-pit ELFs (E_ELF1 to E_ELF 3) will be constructed using the excavated overburden material. Six in-pit ELFs are proposed as pit backfill.
- The West Pit mining direction is east to west; one ex pit ELF (W_ELF1) is to be constructed using the overburden material. Three in-pit ELFs are proposed as pit backfill.

Access to the mine site will be provided by one of two haul road options from either the north or south, the final alignment of the haul roads are yet to be confirmed. The proposed northern haul road option will involve substantial earthworks, including significant cut sections and minor filling. In contrast, the southern haul road requires smaller cuts and minor filling but includes one bridge crossing and approximately three large culverts to span various streams. The haul options are discussed in detail in Sections 7.1 and 7.2 and 16.0 and 17.0.

Approximate locations of the mine features are indicated on the site plans (Figures 1 to 6) presented in Appendix A.

2.2 Proposed Mine Staging

It is understood that the MFS project will be undertaken in several stages based on the October 2024 BTM staging plans. Table 2 summarises the proposed areas of earthworks to be undertaken at each stage with screen shot images of the staging are presented in Appendix F.

Table 2: Summary of Proposed MFS Staging

Stage #	Excavation Area (Source)	Dump Construction (Sink)
Stage 1 – July 2028	Excavation of East Sump and haul road.	E_ELF1 (East Pit).
Stage 2 – July 2029	Western part of East Pit above fault and haul road.	E_ELF1, E_ELF2 and E_ELF3 (East Pit).
Stage 3- July 2030	Western section of East Pit below fault.	E_ELF1, E_ELF2 and I_ELF1 (East Pit).
Stage 4- July 2031	Continuation of eastern section of the East Pit. Excavation of west sump.	I_EFL2 (East Pit). W_ELF1 (West Pit).
Stage 5- July 2032	Continuation of eastern part of the East Pit. Excavation of the eastern section of West Pit.	I_ELF2 (East Pit). W_ELF (West Pit).

Table 2: Summary of Proposed MFS Staging

Stage #	Excavation Area (Source)	Dump Construction (Sink)
Stage 6- July 2033	Continuation of eastern section of the East Pit. Continuation of west pit to the northwest.	I_ELF2 (East Pit). W_ELF1 (West Pit).
Stage 7- July 2034	Continuation of eastern section of the East Pit. Continuation of West Pit to the northwest.	I_ELF2 (East Pit). W_ELF1 (West Pit).
Stage 8- July 2035	Commencement of excavation in the eastern section of the East Pit. Continuation of excavation of West Pit.	I_ELF2, I_ELF3, I_ELF4 and I_ELF5 (East Pit). W_ELF1 (West Pit).
Stage 9- July 2036	Continuation of excavation in the eastern section of the East Pit. Continuation of excavation of West Pit.	I_ELF3, I_ELF4 and I_ELF5 (East Pit). W_ELF1 (West Pit).
Stage 10- July 2037	Continuation of excavation in the eastern part of the East pit. Continuation of excavation of West Pit.	I_ELF3 and I_ELF4 (East Pit). W_ILF1, W_ELF2 (West Pit).
Stage 11- July 2038	Continuation of excavation in the eastern part of the East Pit. Continuation of excavation of West Pit.	I_ELF3, I_ELF4 and E_IFL6 (East Pit). W_ILF1, W_ELF2 (West Pit).
Stage 12- July 2039	Continuation of excavation in the eastern part of the East Pit. Continuation of excavation of West Pit.	I_ELF3, I_ELF4 and E_IFL6 (East Pit). W_ILF1, W_ELF2 (West Pit).
Stage 13- July 2040	Continuation of excavation of West Pit.	W_ILF3 (West Pit).

Notes:

1. *Mine staging is based on the staging schedule providing by BTM (October 2024).*

3.0 Basis of Design (BOD)

The geotechnical BOD at pre-feasibility level for pit highwall slope profiles, ELFs, sump cuts and haul road earthworks are based on experience from existing pits across the Stockton Mine site cut into Brunner Coal Measures (BCM) and basement rocks. The cut geometries given in Table 3 have been adopted into the design as provided by BTM.

Table 3: Basis of Design Cut / Fill Slope Geometries

Mine Element	Overall Slope (°)	Bench Height (m)	Bench Width (m)	Batter Angle (°)
Engineered Landforms (ELF)	18	15	25.5	36
Pit Highwalls & Stormwater Sumps	38	15	8.1 – 8.7	55
Haul Road[1]				
Cuts	41	15	8.5	55
Fills	32	15	5	37

Notes:
Cut geometries given have been adopted from the BTM Basis of design.

In addition, other BOD items included in this assessment include:

- A swell factor of 25% for overburden placement in the ELF.
- For the haul roads a maximum 10% permanent gradient allowing for a total ramp width (two way) of 30 m.
- A preliminary horizontal offset between sump crest and ELF toe of 50 m.

4.0 Geotechnical Database

The following sections discuss available geological and geotechnical data across the MFS area northern and southern haul road alignments. Data was obtained from the previous drilling campaigns, a site walkover, and a review of reports provided by BTM and geological maps of the Buller Coal Field.

4.1 Regional Geology Setting

The MFS stratigraphy generally consists of Quaternary cover, Eocene Rapahoe Group (Kaiata Formation) and Brunner Coal Measures (BCM) unconformably overlying basements rocks comprising Lower Palaeozoic greywacke and argillite of the Greenland Group, the stratigraphy is discussed below.

The mapped regional geology expected to be encountered in the vicinity of MFS is indicated on Figure 7 (Appendix A).

4.1.1 Quaternary Cover Deposits

Quaternary deposits are limited to colluvial soils on the vegetated slopes generally along the northern and southern haul roads where sandstone caprock (pavement) is not exposed. There is no information available on the composition or thickness of colluvium across the haul road alignments, although it is expected to be relatively thin on the slopes and thicker towards the base of the slope comprising residual soils of the underlying rock. Colluvium is expected to be marginally stable and likely to slip when cut slopes are overly steep.

4.1.2 Rapahoe Group (Kaiata Formation)

The Kaiata Formation typically occurs as a massive, highly weathered, very weak to extremely weak mudstone, which slakes on exposure to atmosphere and readily breaks down to soil sized particles. The Kaiata Formation is only expected to be encountered near the terminus of the southern haul road. No Kaiata Formation mudstone is anticipated to be encountered in any of the proposed MFS highwall or sump cuts.

4.1.3 Brunner Coal Measures (BCM)

The upper part of the BCM is dominated by massive-bedded quartz sandstones, mostly coarse to very coarse grained which is generally exposed at surface across the MFS area as caprock (pavement). Sandstones underlying the caprock are interspersed with minor thin siltstone and mudstone beds and rare, thin coal seams.

The Mangatini coal seams comprise M3 (upper) near surface, M2 (middle) and M1 (lower) seams. All three coal seam groups are present in the MFS area with M1 and M3 being patchy and the main coal bearing horizons being the extensive M2 seam, which has a maximum thickness of 9 m and averages about 4 m. The deeper M1 seam thickens in the northern part of the MFS Block.

M2 Seam split sequences observed in the MFS area are named M21 & M22, M222, M221, M212, M211 noted in DC30. The M3 seam is shallow and comprises most of the historical shallow underground workings where coal has already been extracted.

A review of the interpolated coal floor triangulations for the M2 seam roof indicated dip angles of between 4° and 11° with a dip direction between 165° and 280° based on 8 random measurements across the triangulated surface.

The lithology logs basal units of the BCM generally comprise brecciated sandstone.

4.1.4 Basement – Greenland Group (Late Cambrian – Early Ordovician)

Published geological maps of the Buller Coalfield and limited drillholes completed to date across MFS indicate the Basement (BSM) Greenland Group rock in the south is predominantly unaltered greywacke / argillite. However, closer to the granitic batholith intrusion in the north, alteration of basement rock has occurred to a highly altered biotite hornfels. Altered basement rocks are more common across Stockton and encountered in the drill core from Mount Fred mine block.

Most MFS drillholes intersected basement rock and then terminated. DC22, DC24 and DC28 proved interbedded greywacke / argillite, logged as slightly to moderately weathered weak argillite and unweathered strong greywacke.

4.2 Local Geology Information from BTM

The following summarises salient geological information supplied by BTM during the assessment:

- MFS coal measures are likely to be like Millerton mine at Stockton where there is no Kaiata Mudstone but caprock sandstones to deal with. Caprock sandstone has been measured to have an Unconfined Compressive Strength (UCS) up to 85 MPa but typically closer to 50 MPa and are known to be highly variable which is difficult to delineate just with drillholes.
- BCM mudstones are present at Millerton where Bedding Plane Shears (BPS) along discrete surfaces have formed especially at coal floor. BPS have also been observed at Rockies and Mount Fred Pits with BPS generation considered to be associated with the formation of local faults.
- BTM indicated jointing is very apparent through BCM and with two main joint sets observed parallel to the dominant fault trends, orientation data was provided. Many of the joints are silica-cemented and form small ridges on the sandstone surface.
- The MFS area basement is outside of the granite intrusion and could be just in Greenland Group metasediments (Late Cambrian / early Ordovician). Geotechnical laboratory data available from Stockton are associated with Cypress, Mount Fred, and SN block development all of which are still in the periphery of the granite intrusion so are still altered. No laboratory test data or parameters for greywacke / argillite is available from Stockton.
- No drilling or mapping has been undertaken along the proposed haul roads from Mount Fred pit to MFS in the north, or along the proposed southern alignment.
- Substantial underground coal workings have left very little cover in places, numerous areas show substantial collapse and subsidence where cover is only 0.5 m to 2 m thick.
- At Stockton typically liquefaction is not considered a significant risk in any sandstone ELF's as they are usually free draining and have minimal fines so are clast supported rather than matrix supported.
- The Mangatini sump at Stockton is cut only into BCM with no basement rock exposed.

4.3 Structural Geology

4.3.1 General

The structural geology of the MFS area is complex and poorly understood but is predominantly fault controlled with some drag folding. There are no significant regional structural folds within the MFS or Stockton Mine areas with structural deformation of the coal seams directly related to significant faulting of the coal seams.

4.3.2 Bedding

Topographically the MFS area comprises mainly of a series of dip slopes, which generally dip southwest at 8 to10°.

A review of the available MFS drill core logs identified bedding in the BCM dipping generally between 5° and 20°. No BPS were noted in the drill core logs as would be expected within weak to moderately weak highly carbonaceous mudstones associated with coal roof / floors.

4.3.3 Faulting

A review of published geological information indicates the following about local faults:

- The MFS slopes are deeply dissected and offset by fault scarps trending NNE and NW. The NNE fault set is the more predominant and the fault planes dip steeply (60° to 80°).
- Notable regional scale faults within or near the MFS area include the Kiel Flat and Deep Creek Faults, as well as the Taipo, Cedar, Hutt Creek, Kiwi, Kelly and Mt William Faults located to the south and east of the proposed MFS area. The Kiel Flat, Taipo and Deep Creek Faults have vertical displacements of 10 m to 20 m. There is also evidence of strike slip movement on some of these faults.

Structural fault information supplied by BTM based on their experience of mining at Stockton includes:

- Fault traces are often obscure on the surface making fault interpretation difficult.
- The Mount Fred Fault will still be "present" though it will be transitioning into the Kiwi Fault around this point, either way a large-scale (100 m to 1,000 m of throw) fault runs through or adjacent to this area in the BCM.
- There are several other large to medium scale faults also running through the BCM and can be seen as offsets in the sandstone pavement.

- The "major" roughly north south trending faults are part of the reactivated normal faults for the current compressional phase, and the transverse faults have taken up the differential stress between these faults. Transverse faults have possibly formed through existing weak areas in basement associated with pre-existing faults when New Zealand separated from the Australian continent when the main stress direction was at 90° to current. This means that some of the transverse faults could be very large structures in basement and have in places crosscut the "main" north south trending faults, this has been observed at Stockton between No.2 South Black and Cypress Pits with offsets of between 5 m and 10 m.
- Anecdotal evidence from BTM notes that measurements of the Mount Fred Fault, located to the north of site, generally dips at an angle of 70° to 80° to the west. Faults within the MFS area would likely dip at similarly steep angles, though it was noted that these faults have not been measured.
- Drag folding associated with major faults, in particular Mt William Fault may be present with a limited zone (20 m) of influence adjacent to major reverse faults at the Mount Fred Fault. Some folds may be from draping over basement ridges (although this is unknown in the MFS area).

4.3.4 Fault Traces Supplied by BTM

Fault traces were provided by BTM in a Vulcan layer file as illustrated in Figure 2. These traces have been derived from the 1986 Applied Geology Associates Limited maps and geological map of the Buller Coalfield. Considering the regional fault correlations upon the individual map sources, PDP consider the fault traces and associated dip directions presented on the 1986 maps as potentially present on site and therefore included within this assessment.

Mapping indicates that two predominant fault sets are present within or near the proposed MFS area as shown on Figures 2, 3 and 6 (Appendix A). The general characteristics of these fault sets, labelled F1 and F2 respectively, are described below:

- F1: This fault set is generally orientated in a SW-SSW to NE-NNE direction and includes a portion of the Kiel Flat Fault trace. Fault trace dip directions vary from NW/ NNW to SE/ SSE orientations. The spacing between the fault traces mapped range from approximately 20 m – 350 m between faults.
- F2: This fault set is generally oriented in a SE to NW direction and includes a portion of the Deep Creek Fault trace. Fault trace dip directions are orientated towards the NE. The spacing between the fault traces mapped range from approximately 20 m – 200 m between faults.

Table 4 provides a summary of mapped faults in the general vicinity of the MFS study area.

Table 4: Summary of identified Faults within the MFS study area

Ref.	Fault Name	Scale	Fault Orientation		Relation to MFS and ELF locations
			Strike (°)	Dip (°)	
F1-1	Unknown	Local	020	Unknown	Traverses centre of East Pit, east of the East Sump and the west toe of E-ELF1.
F1-2	Unknown	Local	022	Unknown	Traverses eastern area of East Pit, east of the sump and eastern margin of E_ELF1.
F1-3	Unknown	Local	018	Unknown	Outside mine footprint.
F1-4	Unknown	Local	018	Unknown	Outside mine footprint.
F1-5	Unknown	Local	020	Unknown	Traverses western area of East Pit near DC23.
F1-6	Unknown	Local	040	Unknown	Traverses West Pit near DC04.
F1-7	**Kiel Flat Fault**	Regional	015	Unknown	Traverses along western margin of W_ELF1 and West Pit near DC28.
F1-8	Unnamed	Local	055	Unknown	Traverses eastern part of W_ELF1 and crosses West Sump.
F1-9	Unnamed	Local	040	Unknown	Traverses southwestern corner of West Sump.
F1_10	**Taipo Fault**	Local	010	Unknown	Does not intercept any proposed mine features
F1_11	Unnamed	Local	060/010	Unknown	Does not intercept any proposed mine features
F1_12	**Hutt Creek Fault**	Regional	029	Unknown	Does not intercept any proposed mine features
F1_13	Unnamed	Local	051	Unknown	Does not intercept any proposed mine features

Table 4: Summary of identified Faults within the MFS study area

Ref.	Fault Name	Scale	Fault Orientation		Relation to MFS and ELF locations
			Strike (°)	Dip (°)	
F1_14	**Kiwi Fault**	Regional	033	Unknown	Traverses west and crosses southern haul road.
F1_15	Unnamed	Local	034	Unknown	Crosses eastern part of coal transfer pad.
F1_16	**Kelly Fault**	Regional	065/020	Unknown	Does not intercept any proposed mine features
F1_17	**Mt William Fault**	Regional	044	50-60 E	Does not intercept any proposed mine features
F1_18	Unnamed	Local	030	Unknown	Crosses southern Haul Road north of proposed bridge.
F2-1	Unnamed	Local	310	Unknown	Traverses the western area of East Pit and the southern area of E_ELF2. Dextral offset by 40 m by F1_5.
F2_1-1	Unnamed	Local	330	Unknown	Traverses south-western part of East Pit between F1_5 and F1_1. Dextral offset by 40 m by F1_5.
F2-2	Unnamed	Local	310	Unknown	**Traverses between East and West Pit along Billo Stream northwest of F1_5. Dextral offset of approximately 100 m by F1_5. Inferred continuation of Deep Creek Faults**
F2-3	**Deep Creek Fault**	**Regional**	**325**	**Unknown**	**Traverses between East and West Pit along Billo Stream. Dextral offset of 100 m southeast of F1_5. Crosses river at the location of the proposed southern haul road bridge.**

Table 4: Summary of identified Faults within the MFS study area

Ref.	Fault Name	Scale	Fault Orientation		Relation to MFS and ELF locations
			Strike (°)	Dip (°)	
F2-4	Unnamed	Local	330	Unknown	Located between East and West Pits. Dextral displacement of F1_5 and F1_7 by approximately 30m.
F2-5	Unnamed	Local	320	Unknown	Traverses the toe of E_ELF1 between F1_1 and F1_2 near DC25.
F2-6	Unnamed	Local	330	Unknown	Traverses the southern margin of East Pit near DC20.
F2_7	**Cedar Fault**	Local	340	Unknown	Outside Mine footprint.
F2_8	Unnamed	Local	091	Unknown	Adjacent to horseshoe bend at end of southern haul road.

Notes:

1. *Fault intersections based on August 2024 mine plan.*

4.3.5 Drillhole Fault Zones Identified

In addition to a review of the published information the drillhole database supplied by BTM was checked for any reference to fault zones in the drillhole database, as summarised in Table 5. It should be noted that not all drill cores had geotechnical logging undertaken on them. In summary there is no clear correlation between logged fault zones and mapped fault traces.

Table 5: Fault Zones identified in BTM drillhole logs

Drillhole	Depth (m bgl)		Closest Mapped Fault	Thickness of Fault Zone (m)	Lithology
	From	To			
DC22	22.3	22.5	F1_1	0.2	BCM Carbonaceous Mudstone
DC24	17.3	17.8	F2_4	0.5	BCM Carbonaceous Mudstone
DC26	11.5	11.8	F1_2	0.3	BCM Coal Measures
DC32	27.8	28.5	F2_6	0.7	BCM Basal Sandstone
DC32	28.5	28.8	F2_6	0.3	BSM Greenland Group
DC39	13.9	14.0	F1_7	0.1	BCM Coal Measures

4.3.6 Drillhole Crush Zones

To assess for any other areas of structural damage, crush zones identified in available drillholes logs have been summarised in Table 6.

In summary three drillholes DC30, DC31 and DC34 in both the core logs and WellCAD televiewer data had crush zones and are located within the footprint of the proposed West Pit. The crush zones are up to 0.8 m thick (DC30, DC31) and 1.3 m (DC34).

Table 6: Crush Zones identified in BTM drillhole logs

Drillhole	Depth (m bgl)		Closest Mapped Fault	Thickness of Crush Zone (m)	Lithology
	From	To			
DC30	19.0	19.8	F1_7	0.8	BCM Coal Measures
DC31	2.6	2.9	F2_3	0.3	BCM Coal Measures
DC31	3.2	4.0		0.8	BCM Sandstone
DC31	14.3	14.4		0.1	BCM Coal Measures
DC31	25.8	26.6		0.8	BCM/BSM Contact

Table 6: Crush Zones identified in BTM drillhole logs

Drillhole	Depth (m bgl)		Closest Mapped Fault	Thickness of Crush Zone (m)	Lithology
	From	To			
DC34	11.5	11.5	No close faults	<0.1	BCM Coal Measures
DC34	17.7	19.0		1.3	BCM Carbonaceous Mudstone
DC34	23.0	23.0		<0.1	BCM Carbonaceous Mudstone
DC34	25.0	25.3		0.3	BSM Green Group Greywacke

4.4 Regional Seismicity Review

The MFS study area is located within a seismically active area of New Zealand, and as such it is likely the MFS area will experience some ground shaking during its operational life and post closure.

Prior to 2010 two probabilistic seismic hazard models have been developed for the Cyprus area at the Stockton Mine. These include the McVerry Attenuation Model, URS (2007) and a Probabilistic Seismic Hazard Model (PSHA), Stafford (2003).

The results of both the studies estimated similar Peak Ground Accelerations (PGA) and are derived based on dam safety guidance by ANCOLD. For the Operating Basis Earthquake (OBE) with 150-year return period the PGA estimates were around 0.24 g.

Estimation of the seismic hazard at the Cyprus Mine (Mould, 2004) gave PGA’s 0.32 g for the OBE and for the Maximum Credible Earthquake (MCE) a PGA of 0.97g. These were factored down by a third to 0.21 g and 0.65 respectively.

In summary the assessment completed to date for Stockton Mine are considered out of date due to the recent changes in the seismic design guidance in New Zealand with the release of the National Seismic Hazard Model (NSHM). The PGA’s used in this analysis are based on current guidance issued by MBIE and the New Zealand Geotechnical Society (NZGS).

International experience with mine highwall stability in seismically active regions has shown that, even under strong earthquake shaking, failures are typically limited to small, shallow landslides and rockfalls. These events have not occurred at a scale significant enough to disrupt mining operations. (Read and Stacey, 2009).

Natural slopes which have failed during strong shaking (in large magnitude earthquakes i.e., Mw > 7) may experience localised topographical induced amplification of peak motions. However, this does not occur to the same extent in highwalls formed in rock.

5.0 Drill Programme

As mentioned previously several exploration drill programmes have been completed across the MFS area, however the majority of these were wash drilled with no geotechnical data collected from cored holes.

The 2021 to 2023 drill programme did include a full suite of down hole geophysical testing, geotechnical core logging, core photographs and piezometer installation in most drill holes which are discussed in the following sections.

5.1 Groundwater

Hydro Geochem Group (HGG) supplied PDP with the groundwater level data collected to date. Data had been collected in nine piezometers installed into drillholes, summarised in Table 7. Three piezometers at DC28, DC33 and DC41 have 'continuous' monitoring from level loggers, while the other groundwater measurements were limited to manual measurements taken over a single round of monitoring.

Based on a review of the barometric corrected level logger data, it is inferred that during the short monitoring period of 22 April 2024 to 23 May 2024 there was negligible fluctuation in ground level with some minor response to precipitation. As data collection is on-going the groundwater levels need to be reviewed to assess potential seasonal fluctuations and included in future assessments.

In summary, groundwater levels are shallow in the north and deep in the south but appear to be highly variable. Groundwater might be intersected by the West Sump which is proposed to be up to 60 m deep with groundwater in DC43, DC45 and DC46 measured between approximately 43 m and 49 m below ground level.

Table 7: Summary of Groundwater Measurements (supplied by HGG)

Drill Hole	Mine Location	Well Depth (m bgl)	Ground Elevation (NZVD2016) (m RL)	Approx GW Depth (m btoc)	Groundwater Elevation (m RL)
DC25	Location of Ex-Pit ELF	14.6	793.825	7.235	787.400
DC27	East Pit	24.5	791.567	15.440	776.927
DC28	***West Pit***	***9.0***	***813.110***	8.090	***805.900***
DC33	***Billo Stream between East & West Pits***	***4.3***	***742.526***	2.485	***740.981***
DC37	Possible Coal Workings / Ex-Pit	23.7	776.388	DRY	n/a
DC41	***East of W_ELF1 adjacent to U/G workings***	***12.2***	***740.661***	4.450	***736.606***
DC43	Western Sump	49.3	725.987	48.670	678.117
DC45		53.0	701.188	43.000	658.988
DC46		48.7	681.661	45.000	637.461

Notes:

1. *Data source HGG (2024) groundwater table.*
2. *m btoc = metres below top of casing.*
3. *Bold italics denotes piezometers with level logger data.*

5.2 Additional Laboratory Testing 2024

No additional drilling was completed as part of the pre-feasibility assessment. Selected rock core from the BTM core storage facility from the existing MFS drill core were sampled and sent to an IANZ accredited laboratory (Geotechnics Limited). Laboratory tests included Unconfined Compressive Strength (UCS) and Brazilian Tensile strength testing of the BCM sandstone units only. Any mudstone core still in the core boxes was too dry and friable to sample and not suitable for laboratory testing. Therefore, no data on BPS could be obtained. Laboratory test certificates are given in Appendix D and summarised in Table 8.

The purpose of collecting these samples was to collect rock strength data on the MFS overburden to compare against the larger geotechnical dataset held by BTM for the Mount Fred pit to the north as given in Table 8. The full Mount Fred pit data set is summarised in Table D1 in Appendix D.

Drill core was only available from the proposed East Pit area with non-available for the proposed West Pit, haul road or sump locations.

Table 8: Comparison of MFS BCM Sandstone to Stockton Dataset

Property	Location	Fine SS	Medium SS	Coarse SS	All
Average UCS (MPa)	Stockton	37.2	14.5	30.5	**27.4**
	MFS	*25.6*	*25.8*	*30.6*	***30.2***
	Difference	*11.6*	*11.3*	*0.1*	***2.8***
Average Tensile Strength (MPa)	Stockton	4.4	1.3	1.6	**2.4**
	MFS	*2.1*	*3.5*	*2.3*	***2.4***
	Difference	*2.3*	*2.2*	*0.7*	***0***
Water Content (%)	Stockton	2.8	3.3	2.4	**2.8**
	MFS	*0.6*	*1.2*	*3.0*	***1.3***
	Difference	*2.2*	*2.1*	*0.6*	***1.5***
Bulk Density (t/m^3)	Stockton	2.57	2.40	2.40	**2.46**
	MFS	*2.56*	*2.38*	*2.29*	***2.41***
	Difference	*0.01*	*0.02*	*0.11*	***0.05***

Note:
1. *SS = sandstone*

In summary, the data in Table 8 shows a reasonable correlation in the sandstone strength data between Stockton database and MFS core for sandstone units only and has been utilised further in this assessment. It should be noted that core is aged and had not been wrapped in clingfilm. As a result, the core samples may not fully represent the in-ground conditions which may explain some of the differences observed in the data.

5.3 Laboratory Data Basement Rock – Altered Greenland Group

Table 9 below summarises a review of the laboratory testing completed on drill core from the Mount Fred pit to the north of the MFS and within the altered BSM Greenland Group rock samples (originally logged as hornfels). This data is relevant to any slope stability analysis and stabilisation work needed to support cut ground along the northern haul road. The data shows the altered basement rocks can be variable in strength as would be expected depending on the extent of alteration.

Table 9: Summary of Laboratory Testing Basement - Altered Greenland Group

Laboratory Test	No. Samples	Min	Max	Mean
Bulk Density (t/m^3)	5	2.57	2.66	2.62
Unconfined Compressive Strength (MPa)	5	8.2	35.5	17.6

5.4 Laboratory Test Data Gaps

The following data gaps were identified during the review of the laboratory test data.

- No existing geotechnical laboratory data for West Pit, sumps, haul roads and particularly for any Bedding Plane Shears (BPS).
- No existing geotechnical laboratory data for unaltered basement greywacke / argillite.
- 2024 laboratory testing was undertaken on aged, unwrapped core samples and possibly not fully representative of the in-ground conditions.

6.0 Historical Slope Performance

The following outlines any signs of observed slope instability in aerial photographs, recent LIDAR data or during the site walkover of the MFS study area.

6.1 North Haul Road Alignment

A document provided by BTM regarding the construction of the Mount Frederick life of mine drain to the north of MFS area indicated some geotechnical risks are likely for cut slopes in colluvium, which are often saturated at the colluvium bedrock interface. It is probable that earthworks cut will destabilise these colluvium materials and the vegetation held on it. Shallow slips in colluvium may pose a minor risk to haul road operations and may block stormwater drains. Removal of the colluvium will mitigate these risks. However, this could be extensive and impractical to fully remove considering the local topography and length of the haul road.

The northern haul road is expected to traverse lengths of Greenland Group greywacke basement (BSM) with varying levels of alteration and weathering.

A review of the latest topography LiDAR indicates potential slope instability on the western elevation of the ridge, in the form of reasonably well-defined head scarps. These instability features are possibly due to weaker rock mass / colluvium due to localised alteration / weakening near the granite intrusion.

Due to the size of the proposed cuts, blasting along the northern haul road is probable and could cause damage to the shallow rock mass. There is also a potential risk of larger slips where the slope is cut into weak or altered rock mass or were bisected by local / regional faults. The approximate location of the proposed northern haul road is indicated on Figure 6 (Appendix A).

6.2 Southern Haul Road Stability

The southern haul road route as shown on Figure 6 (Appendix A) is proposed to run south from the eastern end of the west pit down to its terminus at Ceder Creek Road. The southern haul road will require one bridge and approximately three culvert crossings to span the various streams and creeks which crosscut the route. Only minor cut and fill earthworks are anticipated to be required. The northern part of the haul road is proposed to follow the crestline adjacent to the deeply incised Deep Creek, some potential areas of historical slope instability have been identified along the cliff edge. The proposed haul road route is bisected by faults at various locations along haul road route.

6.3 Slope Performance Data Gaps

The following data gaps were identified during the review of slope performance:

- No data on the thickness or composition of colluvium along the haul road alignment.
- No geotechnical drilling information along the northern and southern haul road alignments to assess deep seated failure risks at this stage. Further assessment will be required at the feasibility stage once the final alignment has been selected.

7.0 Historical Underground Workings

7.1 Background

BTM provided the following data showing historical underground workings as indicated on Figure 5 (Appendix A) in relation to the proposed development:

- Digitised georeferenced plans of mapped underground workings.
- Aerial photography of collapsed shallow workings.

Handheld GPS picks of collapsed workings.

The information provided indicates:

- The underground workings are extensive and predominately localised to the shallower M3 and M2 coal seams.
- The lateral extents of the workings can be indicators of coal seam pinching or fault displacements.
- The full extent and depth of the historical mine workings is poorly defined.

7.2 Existing Collapsed Underground Workings

A review of the Mount Fred South Prospect – Historic Voids Plan from Bathurst Resources Limited, dated 20 October 2023, indicates the following:

- Voids V8 & V9 located in the West Sump area, photographs supplied by BTM show cover of approximately 0.5 m.
- Voids V3, V4, V5 & V6 located under the proposed Western ELF, photographs show collapse of the sandstone pavement.
- Voids V1, V2 show collapse of cover material.

Photographs (2023) of the mapped voids provided by BT mining given in Appendix H.

7.3 Possible Geotechnical Risks

Underground workings pose the following geotechnical risks to the proposed MFS mine development:

- Collapse of the roof of the workings from the additional static loading from ELF’s placed directly over them. Collapse might be triggered due to insufficient thickness of cover or insufficient rock strength, which has been observed historically. Collapse could lead to loss of ELF integrity above with possible long term stability issues and formation of stormwater pathways into the underground workings.
- Rock mass deterioration from underground workings collapse / deformation and the effect this may have on toppling / stability when cut through to form pit of sump highwalls.
- The workings may form preferential pathways for groundwater or stormwater migration; this would especially be the case where the sump excavations extend through the workings. Localised saturation of InPit ELFs may occur when placed directly against exposed underground workings.
- Coal gas pocket short term inundation risks which should be considered in any future Principal Hazard Management Plan (PHMP).

7.4 Underground Workings Roof Collapse

The following discusses the thickness of sandstone cover between the ground surface and possible underground workings and the strength of the cover where available.

7.4.1 Western ELF (W_ELF1)

A review of drillhole lithology logs relevant to the western ELF location indicates:

- At location DC41 the shallow M3 seam is very thin (~0.05 m thick) and therefore underground works are probably associated with the M2 seam which is shown at this location to be ~2.8 m thick with 8.9 m of cover from ground surface. In general, the cover material has been logged as moderately weak to moderately strong, moderately to highly weathered interbedded fine sandstone to granular breccia.
- DC37 is located within the same fault block as the underground workings, drillhole records identified:
  - M3 seam ~0.4 m thick with ~1 m cover of weak to moderately strong laminated sandstones.
  - M2 (M22 & M21) seam (3.8 m thick) with 9 m of cover from surface logged as very weak to strong, moderately to highly weathered interbedded fine sandstone to granular breccia.

It is noted that the overburden thickness increases towards the east in the underground workings under the western ELF i.e., the risk of collapse is greater in the west.

7.4.2 West Pit

No drill hole data is available over the underground working areas in the West Pit to assess the cover thickness or strength.

7.4.3 West Sump

The provided M2 map indicates intersections with the proposed SW sump in the east; there is no drill hole information in this area but due to the 50 m depth of the sump it probable that the M3 / M2 seams and possibly the M1 seams will be intersected.

Risks associated with the presence of underground workings, particularly where overburden thickness is limited, have been identified as a key consideration. These conditions may increase the potential for ground movement or surface instability (i.e., roof collapse).

8.0 Rock Mass Model

8.1 Background

The rock mass model characterises engineering properties of various in-situ rock types (e.g., Basement, Brunner Coal Measures, etc.) for use in the slope stability analyses. This includes properties of the intact rock mass and discontinuities derived from drillhole, laboratory and downhole geophysical data.

The rock mass model is used within the geotechnical model to determine likely modes of failure within the highwall including:

1. Structurally controlled failure along existing discontinuities.
2. Rock mass-controlled failure; or
3. A combination of structurally controlled or rock mass failure.

The following section discusses the rock mass data from available laboratory data in relation to probable failure mechanisms for likely rock types across MFS.

8.2 Strength Criterion

A shear strength criterion for the rock mass and associated discontinuities (for BPS) has been assessed previously at Stockton Mine from the results of field investigations, laboratory testing and back analysis of previous slope failures. Due to the amount of data available for the Stockton Mine site, and ground truthing from historical mining activities, the strength parameters derived by BTM for the rock mass and discontinuities are considered suitable for a pre-feasibility level geotechnical assessment for MFS.

For the rock mass the Generalised Hoek-Brown failure criterion strength model is considered suitable for slope stability analysis applied to highwall design and stormwater sump excavations within the rock mass.

Mohr-Coulomb drained strength parameters are considered more suitable to represent BPS discontinuities and ELF fill materials.

Selected strength criterion and related strength parameters for each geotechnical unit is given in Table 10.

Table 10: Geotechnical Strength Parameters

Formation	Material Description	Model	Generalised Hoek- Brown Parameters[1]				Mohr-Coulomb Parameters[2]		
			UCS (MPa)	mb	s	a	Bulk Density (kN/m³)	Cohesion, c' (kPa)	Friction Angle, ϕ' (°)
Brunner Coal Measures	Caprock	Generalised Hoek Brown	50	0.47797	0.0002404	0.50573	20	-	-
	Normal Sandstone	Generalised Hoek Brown	30	1.39544	0.00041894	0.522344	24.76	-	-
	Brunner Mudstone	Generalised Hoek Brown	7	0.2464	0.0002404	0.53127	20	-	-
	Coal	Mohr-Coulomb	-	-	-	-	11.5	80	33
	Coal Floor – Mudstone (Bedding Plane Shear (BPS))	Mohr-Coulomb	-	-	-	-	24.43	10	20
Greenland Group (BSM)	Altered – Hornfels[3]	Generalised Hoek Brown	18	2	0.01	0.5	26.2	-	-
	Unaltered - Greywacke	Generalised Hoek Brown	Altered BSM values used in analysis at pre-feasibility stage.						

Notes:

1. *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.*
2. *Mohr-Coulomb strength parameters supplied by BTM.*
3. *Greenland Group – Basement parameters inferred using UCS data supplied by BTM for samples collected / tested at Mount Fred. 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.*
4. *Effective stress parameters (drained) for cohesion and friction angle.*

The stability analysis of the ELF waste dumps was undertaken using the parameters providing by BTM previously used at Stockton Mine in similar overburden materials. These parameters are presented in Table 11.

Table 11: Mohr Columb Drained Parameters for ELF Stability Analysis

Fill Material	Unit Weight (kN/m3)	Cohesion, c' (kPa)	Friction Angle, ϕ' (°)
Sandstone Fill with minor Brunner Mudstone and Coal[1]	18	2	39
Greywacke / Argillite[2]	-	-	-

Notes:

1. *Based on data supplied from Stockton Mine.*
2. *No information available on the Greywacke / Argillite at this stage.*

8.3 Rock Mass Data Gaps

The following rock mass data gaps were identified.

- Basement laboratory data and rock mass strength parameters provided by BTM is for altered Greenland Group basement and not unaltered greywacke / argillite basement rock and therefore not representative of MFS area. This is primarily an issue for highwall cut configurations for the stormwater sumps and for ELFs filled with greywacke / argillite basement from the sump excavation. It is unclear how MFS greywacke / argillite will disaggregate on excavation, transport and compaction into the ELF and therefore if the derived strength parameters are suitable to be used in stability analysis. There is no ELF at Stockton constructed from greywacke / argillite to use as an operational example of potential slope performance.
- Insufficient representative samples to undertake statistical analysis on laboratory tests for the BCM and BSM Greenland Group across MFS.
- No geotechnical laboratory or in-situ test data of the expected colluvium deposits along the haul road alignments.

9.0 Rock Discontinuity Data

The following section discusses the rock discontinuities based on data from down-hole geophysical surveys and available drill core logging.

9.1 Field Mapping

Due to the lack of rock exposures, no field mapping has been completed to provide information on the persistence of discontinuities in the field.

9.2 Discontinuity Data

The drillhole database was reviewed to assess discontinuities and the rock types they occurred in. This review is limited to assessing the discontinuity type, spacing, aperture, roughness and if any infilling was present. No clear trends in joint roughness, spacing, aperture, or infilling could be discerned from the existing database.

The drill core is not orientated, the dip / dip direction of these discontinuities has been determined from down-hole geophysical data including optical (OTV) and acoustic televiewer (ATV) data as discussed in the following sections.

9.3 Downhole Geophysics

A suite of downhole geophysical tests was undertaken by Weatherford's Limited at selected drill holes across MFS as detailed in Table 12. Raw geophysical data was supplied to PDP by BTM in both PDF and LAS (Log ASCII Standard) file format. The raw data was processed using industry-standard software WellCAD® Version 5.8 supplied from Advance Logic Technology (ALT).

The Acoustic Televiewer (ATV) data was corrected for a magnetic declination of 23° for the Stockton Mine area.

Table 12: Summary of downhole geophysical data available for MFS

Drill Hole	Geophysical Tool				
	Sonic	Density (Open Hole & In Rods)	Optical Tele-Viewer (OTV)	Acoustic Tele-Viewer (ATV)	Calliper Diameter (DD3)
DC16	13.6m – 43.6m	3.0m – 43.4m	4.1m – 14.0m	12.88m - 39.42m	2.15m - 42.6m
DC18	Data not processed	3.0m – 40.7m	–	–	–
DC19	17.4m – 34.7m	3.0m – 34.4m	3.31m – 21.19m	16.3m – 34.0m	2.2m – 33.5m

Table 12: Summary of downhole geophysical data available for MFS

Drill Hole	Geophysical Tool				
	Sonic	Density (Open Hole & In Rods)	Optical Tele-Viewer (OTV)	Acoustic Tele-Viewer (ATV)	Calliper Diameter (DD3)
DC20	5.0m – 13.8m	3.0m – 13.7m	–	4.7m – 13.4m	2.3m - 12.75m
DC22	13.0m – 31.9m	3.0m – 31.8m	5.2m- 13.0m	13.1m – 31.4m	3.3m – 31.0m
DC23	9.3m – 40.8m	3.0m – 40.8m	–	8.8m – 40.4m	3.3m - 40.1m
DC24	5.8m – 22.3m	3.0m – 23.1m	–	7.2m - 22.9m	2.3m - 22.35m
DC25	Data not processed	3.0m – 37.5m-	–	–	–
DC26	8.2m – 22.8m	3.0m – 22.80m	–	5.2m – 37.2m	2.3m – 22.0m
DC27	12.8m – 25.6m	3.0m – 25.8m	–	4.0m – 25.3m	2.3m – 25.0m
DC29	3.0m – 28.8m	3.0m – 28.7m	–	3.48m – 28.12m	–
DC30	Data not processed	3.0m – 40.8m	5.3m – 27.8m	27.6m – 40.4m	2.3m – 39.85m
DC31	1.0m – 27.8m	1.0m – 28.0m	–	3.1m – 27.6m	0.3m – 27.2m
DC32	9.0m – 31.9m	3.0m – 31.8m	–	7.6m – 31.3m	2.3m – 30.85m
DC34	10.6m – 25.5m	3.0m – 25.5m	–	10.3m – 25.2m	2.25m – 24.75m
DC37	3.0m – 27.0m	3.0m – 29.2m	–	3.2m – 26.8m	2.25m – 28.45m
DC39	22.0m – 49.2m	3.0m – 49.6m	3.61m – 23.3m	23.8m – 49.2m	2.15m – 48.6m

Table 12: Summary of downhole geophysical data available for MFS

Drill Hole	Geophysical Tool				
	Sonic	Density (Open Hole & In Rods)	Optical Tele-Viewer (OTV)	Acoustic Tele-Viewer (ATV)	Calliper Diameter (DD3)
DC41	9.0m – 43.8m	3.0m – 43.7m	–	7.6m – 43.6m	2.3m – 42.95m
DC43	Data not processed	3.0m – 49.7m	–	4.0m – 49.5m	2.3m – 48.9m
DC44	4.0m – 49.5m	2.0m – 49.7m	6.57m – 49.6m	6.6m – 49.6m	2.3m – 48.85m
DC45	3.0m – 62.0m	3.0m – 61.9m	4.97m – 61.5m	5.1m – 61.6m	2.3m – 61.1m

Notes:
1. *All data from the Weatherford Downhole geophysical surveys, provided by BT Mining Limited, completed during the 2023/2024 Deep Creek drilling program.*

9.3.1 Sonic Velocity

Sonic velocity data can be used to infer an estimate of rock strength based on the measured compression wave velocity. The sonic velocity data of each borehole listed in Table 12 has been assessed to gauge the presence of any weak or fractured zones.

It is assumed that weak or fractured zones are less dense than surrounding strata. Where apparent sonic velocity wave speed increases have been observed these occurrences are listed below with an explanation if apparent:

- DC16 – 39.1 m to 40 m
- DC19 - 27.5 m to 29.6 m
- DC23 – 17.9 m to 19 m
- DC31 – 0.9 m to 6 m
- DC34 – 10.5 m to 12 m
- DC41 – 34.9 m to 35.8 m
- DC44 – 9 m to 10 m
- DC45 – 57.9 m to 59.5 m

In summary, a review of the sonic velocity there was no data trends which matched with logs to indicate any major structural.

9.3.2 Density

Density data has been assessed to gauge the presence of any weak or fractured zones. It is inferred that weak or fractured zones would be noticeably less dense than surrounding units of intact rock mass. Where noticeable zones of density decreases have been observed throughout the data these occurrences have been listed below with an explanation if apparent:

- DC16 – 39.2 m to 39.5 m – logged as a possible fault with clay infill.
- DC19 – 9.45 m to 9.65 m & 28.7 m to 30.2 m
- DC23 – 6.65 m to 6.85 m & 34.8 m to 35.05 m
- DC26 – 3.0 m to 3.45 m
- DC31 – 4.05 m to 4.5 m
- DC32 – 4.9 m to 5.3 m
- DC34 – 10.75 m to 11.25 m
- DC37 – 25.7 m to 26.15 m

9.3.3 Calliper Data

Each borehole listed in Table 12 has been measured using a calliper log tool. The resulting calliper logs show borehole diameter variations throughout the length of the borehole, which can infer changes in the borehole or lithological conditions.

The calliper logs have been assessed in conjunction with other geophysical data to identify the potential presence of any weak or fractured zones. Where borehole diameter variations have exceeded 40 mm diameter over a minimum 0.3 m zone, these occurrences are listed below:

DC16 – 40 mm; 39.2 m to 39.6 m

DC23 – 50 mm; 6.5 m to 6.8 m & 40mm; 34.7 m to 35 m

DC30 – 100mm; 28.8 m to 32.8 m, intercept with M211 seam with some lost core in log.

DC31 – 40 mm; 4.2 m to 4.5 m

DC32 – 60 mm; 4.65 m to 4.95 m

DC34 – 40 mm; 10.4 m to 11.05 m

DC37 – 100 mm; 24.95 m to 26.9 m, intercept with M1 seam and lost core in log.

In summary, a review of the calliper data indicates some borehole blowouts associated mainly with coal seams, but there is no correlation with large scale rock discontinuities or voids.

9.3.4 Optical (OTV) and Acoustic (ATV) Tele-viewer

ATV data was collected from 18 drill holes shown in Table 12 and was imported into WellCAD® and used to create structural plots and tadpole symbols of discontinuities picked up throughout the geophysical run. In WellCAD® the acoustic image of the drill hole is unravelled to create a flat 360° representation. This unravelled image presents discontinuities as a "waveform" shape with amplitude representing dip angle perpendicular to core axis and wave position representing dip direction in relation to apparent dip. Since the drill holes were vertical (i.e., oriented at 90 degrees to horizontal), all measured discontinuity orientations represent the true dip of the features.

Structural picks from the ATV data were categorised into the following:

- Bedding planes in:
  - Brunner Coal Measures (coal, sandstone, carbonaceous mudstone),
  - Basement rock where encountered.
- Joints, segregated into either open / closed,
  - With dip angle 0 – 30°, >30° and <60° and >60°.
- Fractures segregated into either open / closed,
  - With dip angle 0 – 30°, >30° and <60° and >60°.

The picks were then exported into a spreadsheet to be used in stereographic projection and kinematic analysis. WellCAD® picks are presented in Tables E1 & E2 given in Appendix E.

Significant discontinuities such as fault zones and crush zones have been identified from the drill holes logs.

10.0 Discontinuity Orientations

10.1 Background

The following sections discuss the assessment of possible discontinuity orientation at MFS including bedding planes, joints, fractures, crush zones and faults.

This assessment has been completed using stereographic projection software Rocscience DIPS (Version 8.028).

10.2 Bedding

The following section summarises the bedding planes picked from the from the downhole geophysical data. Based on the data the bedding planes of the Brunner Coal Measures (BCM) generally dip shallowly towards the south and the Greenland Group Basement (BSM) dip shallowly to the northeast. The results are presented in Figures 8 and 9 and summarised in Table 13.

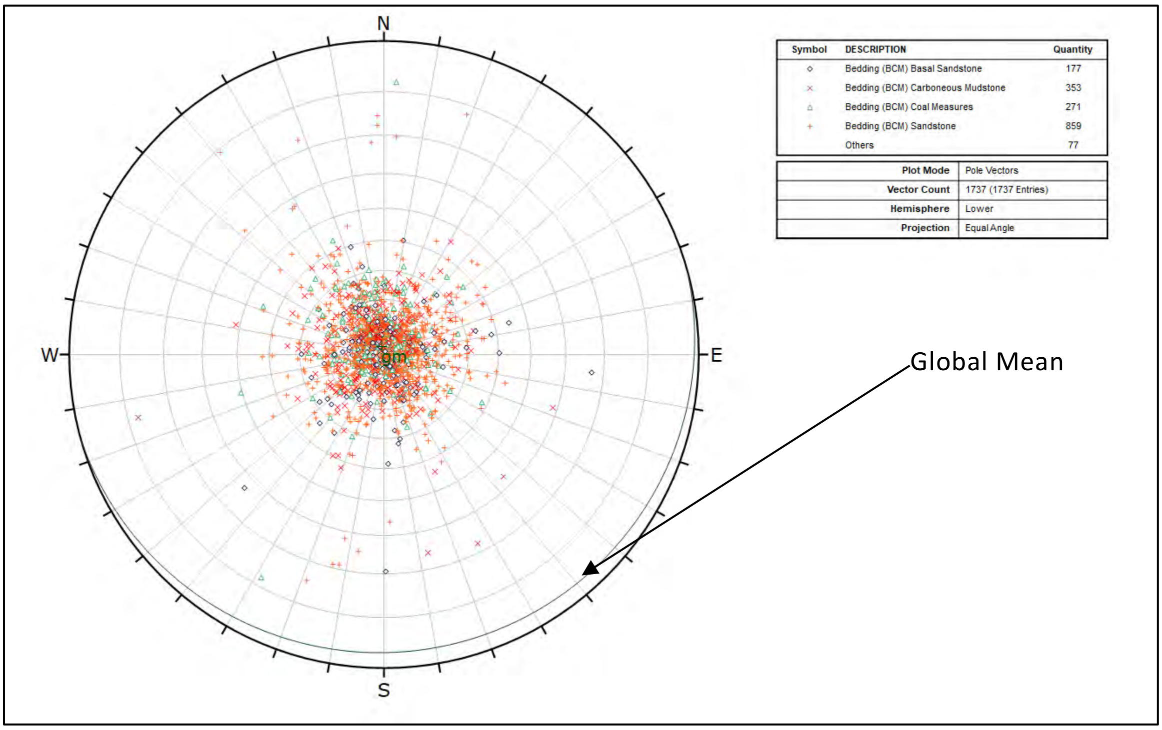

Figure 8: Equal area lower hemisphere projection of poles to plane of all BCM units. The global mean (gm) bedding plane orientation represented by (green) plane.

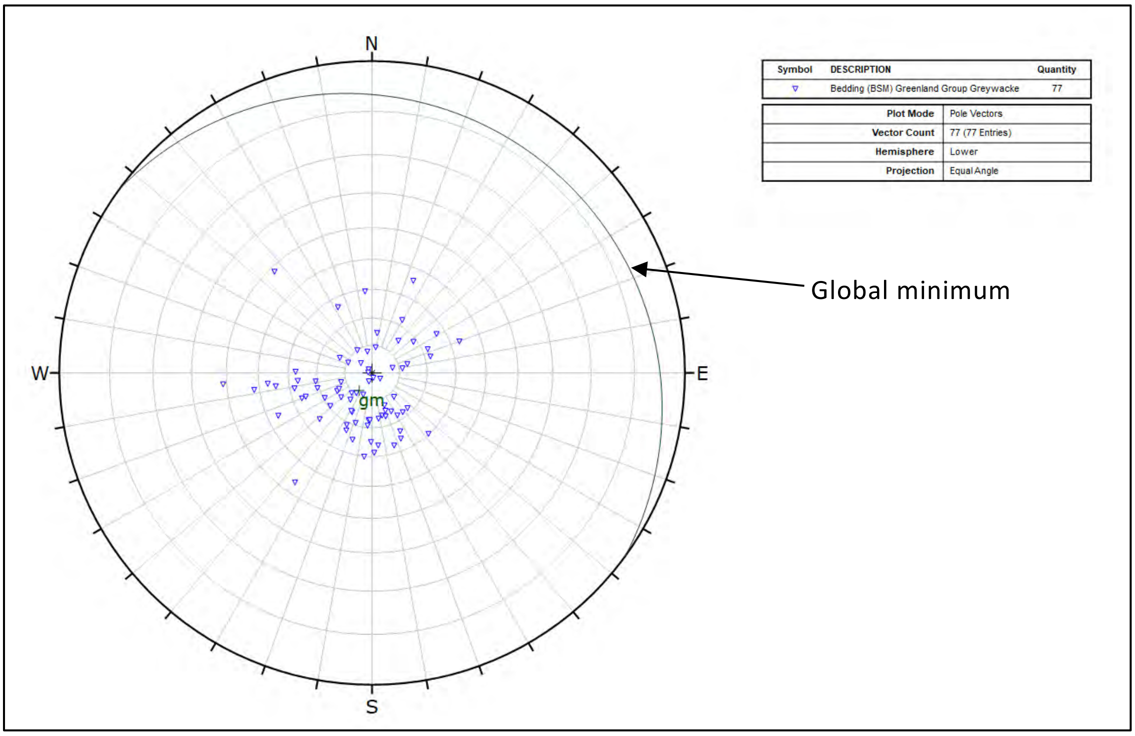

Figure 9: Equal area lower hemisphere projection of poles to plane of all Greenland Group Basement (BSM). The global mean (gm) bedding plane orientation represented by (green) plane.

Table 13: Summary of Bedding Picks Identified in OTV / ATV Data

Group	Lithology	Discontinuity Type	No. Poles	Dip Angle (°)[1]	Dip Direction (°)[1]
BCM	Sandstone (Caprock)	Bedding	859	3	187
BCM	Coal	Bedding	271	6	157
BCM	Carbonaceous Mudstone	Bedding	353	4	154
BCM	Basal Sandstone	Bedding	177	1	065
BCM	All BCM Units	Bedding	1737	3	164
BSM	Greenland Group Basement	Bedding	77	8	036

Notes:

1. *Dip and dip direction are inferred from the generalised global mean generated in the DIPS software.*

10.3 Joints and Fractures

The joint and fracture sets identified in the downhole geophysical data have been grouped into three general groups (<30°, 30-60° and >60°) and separated by lithology. The results are presented Table C2 and C3 (Appendix C).

Joints and fractures generally occur in conjugate pairs (occur along the same strike but dip in opposing directions)

A kinematic analysis has been undertaken to assess the effect of joint and fracture discontinuities relative the proposed mine structure (highwalls, sumps etc). The kinematic analysis is discussed in Section 12.0

10.4 Faults

As discussed in Section 5.3.4 two general faults groups, which have been designated F1 and F2 with locations given in Appendix A, Figures 2 and 3. F1 generally comprises faults which strike northeast-southwest, while the F2 group generally strike northwest-southeast. Additionally, four faults (FT) were identified in drillholes DC24, DC31 and DC32. The fault data is summarised Figure 10 and Table 14.

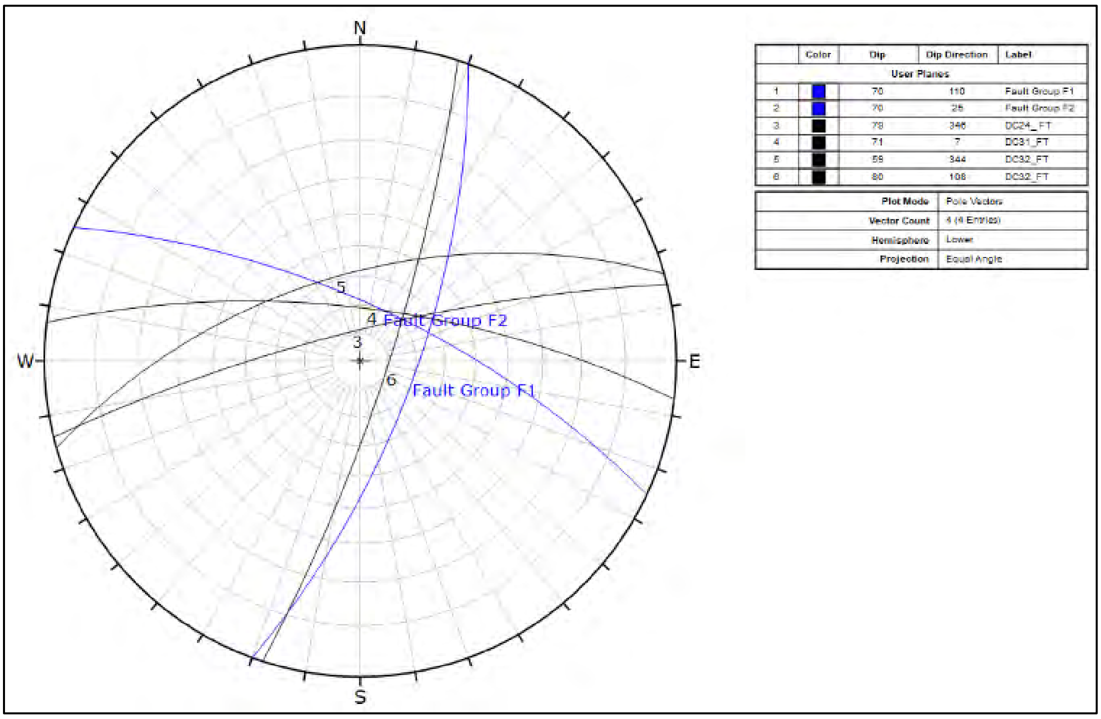

Figure 10: Generalised fault groups F1 and F2 (blue) Plotted against faults / faults zones identified in the drillhole logs DC24, DC31 and DC32.

Table 14: Summary of Fault Dip / Directions identified

Fault ID	Dip (°)	Dip Direction	Depth of Fault intercept (m RL)	Data Origin
F1	70[1]	110[2]	-	As discussed in Section 5.3.4
F2	70[1]	025[2]	-	As discussed in Section 5.3.4
3	79	346	827.7	ATV televiewer – DC24
4	71	007	801.8	ATV televiewer – DC31
5	59	344	743.3	ATV televiewer – DC32
6	80	108	741.3	ATV televiewer – DC32

Notes:

1. *Dip angle estimation only.*
2. *Generalised average fault group strike.*

10.5 Crush Zones

Crush zones were identified in the drillhole logs in Table 6 above. The crush zones identified in both the drillhole, and the downhole geophysical data are summarised in Table 15. Based on the data, crush zones are generally dipping at low angle at approximately 11° towards the southwest in the Brunner Coal Measures (BCM) and 39° towards the north in the Greenland Group greywacke (BSM) and are inferred to be generally controlled by the orientation of the bedding.

Table 15: Crush Zones Identified in Logs and Downhole Geophysical Data

<table>
<tr><th>Drillhole</th><th>Unit</th><th>Lithology</th><th>Depth (m bgl)</th><th>No. Poles</th><th>Dip Angle (°)[1]</th><th>Dip Direction (°)[1]</th></tr>
<tr><td rowspan="3">DC31</td><td rowspan="8">BCM</td><td>CM</td><td>1.41 – 2.69</td><td rowspan="8">17</td><td rowspan="8">11</td><td rowspan="8">229</td></tr>
<tr><td>ZSS</td><td>2.76 – 3.31</td></tr>
<tr><td>CM</td><td>3.89 – 5.36</td></tr>
<tr><td>DC34</td><td>LFSS</td><td>11.49</td></tr>
<tr><td>DC31</td><td>CM</td><td>13.2 – 13.36</td></tr>
<tr><td rowspan="2">DC34</td><td>CZN</td><td>17.72</td></tr>
<tr><td>CM</td><td>18.95</td></tr>
<tr><td>DC30</td><td>CM</td><td>19.06 -19.24</td></tr>
</table>

Table 15: Crush Zones Identified in Logs and Downhole Geophysical Data

Drillhole	Unit	Lithology	Depth (m bgl)	No. Poles	Dip Angle (°)[1]	Dip Direction (°)[1]
		CM	19.65 – 19.76			
DC31		CM	19.87 – 20.78			
		PBBX	25.85 - 25.88			
DC34	BSM	ARG	26.58 - 28.77	5	39	349

Notes:

1. *Crush Zone orientation is based on a generalised global mean automatically generated in the DIPS software.*
2. *BCM = Brunner Coal Measures.*
3. *BSM – Greenland Group Basement.*

11.0 Kinematic Analysis

11.1 Background

To assess discontinuity controlled, kinematically feasible failure mechanisms (i.e., planar, wedge, and topple) relative to the proposed MFS mine and sump highwall configurations, kinematic analysis has been completed in stereographic projection software Rocscience DIPS (Version 8.028), using:

- Structural picks discussed in Section 11.0:
- Proposed cut geometries for the highwalls and sump excavations and fault orientations to assess interactions which may cause kinematics failures as given on Table 16: and
- Friction Angle of 20° as given in Table 10, which is considered to represent worst case scenario for bedding plane shears.

Table 16: Highwall, Fault & Sump Cut Orientations for Kinematic Analysis

Mine Feature	Strike (°)	Dip Direction (°)	Dip Angle (°) (bench / overall)
Highwall 1	123	213	55 / 38
Highwall 2	024	114	55 / 38
Highwall 3	204	294	55 / 38

Table 16: Highwall, Fault & Sump Cut Orientations for Kinematic Analysis

Mine Feature	Strike (°)	Dip Direction (°)	Dip Angle (°) (bench / overall)
Highwall 4	303	033	55 / 38
Fault Grouping (F1 [4]	020	110	70 -80
Fault Grouping (F2)[4]	135	025	70 -80
West Sump (North Wall)	095	185	55 / 45
West Sump (West wall)	020	110	55 / 45
West Sump (East wall)	200	290	55 / 45
West Sump (South wall)	275	005	55 / 45

Notes:

1. *The orientations given are generalised but represent the majority of the highwall / fault orientations based on August 2024 VULCAN model*
2. *Highwalls dip angles are as per overall slope design.*
3. *Dip angles for the highwalls and sumps have been adopted from the Basis of Design provided by BTM. Dip angles of the faults are inferred from faults mapped in the region.*
4. *Fault Group orientations from Section 11.5.*

11.2 Potential Kinematic Failure Mechanisms

Kinematic slope failures—such as planar sliding, wedge failure, and toppling—occur when rock or soil moves along natural weaknesses (i.e., joints or bedding planes). These failures may be triggered by rainfall, earthquakes, or excavation activities. The associated risks can be managed through stabilisation methods such as rock bolts, mesh installation, or slope reshaping to reduce the likelihood of movement. The following sections outline the potential failure mechanisms expected to be encountered in the MFS highwalls.

11.2.1 Planar Sliding

Planar sliding failures in the Brunner Coal Measures are uncommon but could occur where bedding dips toward an exposed face at > 15° which is close to the residual shear strength in pre-sheared mudstone materials (BPS $\phi' = 20°$). The potential for planar sliding failures on bedding dips < 15° should be anticipated where there is elevated pore pressure (i.e. high groundwater level).

11.2.2 Wedge Sliding

Wedge sliding form where discontinuities intersect and the line of intersection daylights in the slope face or highwall. Generally, wedge failures elsewhere in the Brunner Coal Measures do occur regularly but are regarded as a minor stability problem in a mining context being confined to small scale volumes. As they can fail rapidly, they do present a safety risk to personnel working nearby.

11.2.3 Toppling

Toppling can occur in the Brunner Coal Measures where natural cliffs or mine highwalls/batter slopes occur on the down-dip side of bedding sloping downhill at greater than about 10° although this will vary according to block slenderness ratio (i.e. height to width ratio). The vertical release surfaces for the toppling mechanism are joints orthogonal to bedding. These can involve a significant area of exposed slope face such as in an operational or temporary highwalls.

11.3 Kinematic Analysis Results

The results of the kinematic assessment indicate that all three potential failure mechanisms (planar sliding, wedge sliding and toppling) are all kinematically feasible on at least one or more of the proposed highwall orientations detailed feasibility analysis present in Table C1 (Appendix C).

Generally, the orientation of the low angle fracture and joint discontinuity structures align well with bedding surfaces of both the Brunner Coal Measures (BCM) and the Greenland Group Basement. As the dip angle of the discontinuities increase wedge and topple failures along joint, fracture and faults surfaces become more likely to occur.

Although the data indicates with most highwall orientations wedge and topple failures along fractures, joints and faults are kinematically feasible, the occurrence of such failures is also dependent on the persistence and spacing of such features within the rock mass. Due to the spacing of the drillholes and lack of detailed structural mapping in the MFS area it is difficult to gauge the likelihood of wedge and toppling failures.

The following qualitative risk assessment has been undertaken to further qualify the risk associated with the overall and bench configurations of each high wall (See Table 16). The qualitative risk assessment has been undertaken using the definitions described in Table 17. The results of the analysis are presented in Tables 18 and 19.

Table 17: Key Indicators of Highwall Failure

Failure Potential	Key Indicators		
	Planar Sliding	**Wedge Sliding**	**Toppling**
Low	No or rare defects plot in the instability "window" on the stereograph.	No intersections between defect clusters plot in the instability "window" on the stereograph.	No or rare defects are present in the toppling instability "window", and no or rare shallow dipping defects are present in the planar slide instability window to allow sliding along a basal plane.
High	Many defects from a cluster plot in the instability "window" on the stereograph.	The line of intersection between two defects sets plot around the margins of the instability "window" such that any reduction in friction angle, or steepening of IRA, increases the potential for failure.	Many defects from a cluster plot in the planar slide instability "window" and many defects from a second cluster plot in the toppling instability window on the stereograph.
Very High	The centre of a defect cluster plots in the instability "window" on the stereograph.	The line of intersection between two defect sets plots within the instability "window".	The centre of a cluster plots in the planar slide instability "window" and the centre of a second cluster plot in the toppling instability window on the stereograph.

Table 18: Risk of Highwall Failure Based on Borehole Data

Domain[1]	Slope Aspect (°)	Failure Geometry					
		Planar Sliding		Wedge Sliding		Toppling	
		Overall	Bench	Overall	Bench	Overall	Bench
Highwall 1	213	Low to Moderate	Moderate	Moderate	Moderate to High	Moderate	Low to Moderate
Highwall 2	114	Moderate	Moderate to High	Moderate to High	High	Low to Moderate	Moderate
Highwall 3	294	Low	Moderate	Low to Moderate	Moderate to High	Low	Low
Highwall 4	033	Low to Moderate	Moderate to High	Moderate to High	High	Low	Low

Notes:
[1] Highwall details are summarised in Table 16; Risk levels assessed are based on the criterion in Table 17.

Table 18 summarises the kinematic failure risks associated with the overall and bench slope configurations of the MFS highwalls. The results of the qualitative kinematic risk assessment are summarised below:

11.3.1 Overall Slope

- Highwall 1 shows a low to moderate risk of planar sliding, and a moderate risk of both wedge sliding and toppling failure.
- Highwall 2 presents a moderate risk of planar sliding, a moderate to high risk of wedge sliding, and a low to moderate risk of toppling failure.
- Highwall 3 indicates a low risk of planar sliding and toppling, with a low to moderate risk of wedge failure.
- Highwall 4 has a low to moderate risk of planar sliding, a moderate to high risk of wedge sliding, and a low risk of toppling failure.

11.3.2 Bench Slope

For the bench-scale slope configurations:

- Highwall 1 has a moderate risk of planar sliding, a moderate to high risk of wedge sliding, and a low to moderate risk of toppling failure.
- Highwall 2 presents a moderate to high risk of planar sliding, a high risk of wedge failure, and a moderate risk of toppling failure.

- Highwall 3 shows a moderate risk of planar sliding, a moderate to high risk of wedge sliding, and a low risk of toppling failure.
- Highwall 4 has a moderate to high risk of planar sliding, a high risk of wedge sliding, and a low risk of toppling failure.

The results of the risk assessment presented in Table 18, wedge sliding is identified as the most significant kinematic failure mechanism and is considered likely across all highwall configurations. Among the four highwalls, Highwall 3 is assessed as having the most favourable orientation, with the lowest risk of structurally controlled failure. In contrast, Highwalls 1 and 2 present the least favourable orientations, associated with the highest risk of kinematic instability when all three failure mechanisms are considered.

Table 19: Risk of West Sump Highwall Failure Based on Borehole Data

Domain	Slope Aspect (°)	Failure Geometry					
		Planar Sliding		Wedge Sliding		Toppling	
		Overall	Bench	Overall	Bench	Overall	Bench
West Sump (North Wall)	185	Low	Low	Moderate to High	Moderate to High	High	High
West Sump (West Wall)	110	Moderate	Moderate	High	High	Low	Low
West Sump (East Wall)	290	Low	Low	Moderate	Moderate	Low	Low
Highwall 4 (South Wall)	005	Low to Moderate	Low to Moderate	Moderate to High	Moderate to High	Low	Low

Notes:
[1] Highwall details are summarised in Table 16; Risk levels assessed are based on the criterion in Table 17.

The same qualitative analysis was applied to the highwall orientations of the west sump. As shown in Table 19, wedge sliding emerges as the predominant failure mechanism affecting these walls. Of the assessed orientations, the east wall is considered the most kinematically favourable, whereas the north and west walls exhibit the least favourable conditions for kinematic stability.

Optimising highwall geometry by increasing bench widths and reducing batter angles can significantly enhance slope stability and operational safety. Wider benches provide greater catchment for potential rockfall and allow for improved access and monitoring, while flatter batter angles reduce driving forces and improve overall slope resilience. To support informed decision-making and risk management, it is recommended that a detailed probabilistic analysis of the highwalls be undertaken during the feasibility study stage. This approach enables a more comprehensive understanding of failure likelihoods under variable geological and geotechnical conditions, supporting the development of robust and cost-effective slope designs.

12.0 Slope Stability Analysis

12.1 Introduction

Slope stability assessments are a critical component of mine planning and design, aimed at identifying potential failure mechanisms and ensuring safe excavation practices. By evaluating geological, structural, and geotechnical conditions, these assessments help mitigate operational and safety risks associated with slope instability. Early identification of hazards enables the implementation of targeted design modifications and control measures, supporting both regulatory compliance and long-term pit viability.

The following sections discuss the slope stability analyses completed across the MFS area of the proposed pit highwalls, sump cut configurations and ELF fill slope profiles. Evaluating the slope stability was completed using 2D Limit Equilibrium software Rocscience SLIDE2 (Version 9.037) using the Morgenstern/Price and Bishop Simplified methods to provide comparison.

12.2 Cross Sections

Representative cross-section orientations have been chosen to analyse the most critical case(s) from a stability point of view, using the mine design pit shells, ELF and sumps and BTM's latest VULCAN geological model (dated August 2024). Major contacts for M1, M2, M3 coal roof and floor, basal BCM, and basement Greywacke were included in the geological model.

The orientation and dip of faults added into the cross section have been inferred based on the mapped surface expression of the fault and offset lithology visible on cross sections. Many of the faults exhibit lower dip angles than that commonly found in the area. The low angle fault orientations are inferred to be the result of the cross section being oblique to the fault and results in an apparent dip angle.

Each of the cross sections contain multiple mine features (i.e., highwall configuration). A summary of the mine features included in each cross section are presented in Table 20.

12.3 Design Assumptions and Acceptance Criteria

Table 20: Summary of Mount Fred South Cross Sections

Cross Section	Strike	Mine Features
Sct 100	130 (040)	Engineer Landform (E_ELF2). East pit highwall configurations (x4)
Sct 101	183 (093)	E_ELF3 -East pit Highwall – West Pit Highwall – W_ELF1 – West Sump
Sct 102	232 (142)	East Pit Highwall – West Pit Highwall
Sct 103	187 (097)	East Pit Highwall -E_ELF1 – East Sump
Sct 104	226 (136)	East pit Highwall
Sct 105	225 (135)	E_ELF1 -East Pit Highwall – West Pit Highwall
Sct 106	093 (003)	E_ELF2 – East pit Highwall
Sct 107	110 (020)	Haul road cut and road surface.
Sct 108	220 (130)	Haul road cut and road surface.
Sct 109	200 (110)	Haul road cut and road surface.
Sct 110	161 (071)	Haul road cut and road surface.

Notes:
1. Strike bearings indicated direction from left to right.
2. () = bearing into page

The design assumptions and design acceptance criteria (DAC) for the slope stability analysis are given in Table 21. The FoS (Factory of Safety) for varying scales of the highwall slopes and ELF are based on a review of criteria commonly applied in modern mining practice (Read and Stacey, 2009).

Peak ground accelerations (PGA) have been adopted from MBIE Earthquake Geotechnical Engineering Practice, Module 1 Overview of the Guidelines, Table A1, ID 81 (Westport).

Table 21: Design Acceptance Criteria for Slope Stability Analysis

Design Life (Final Landform)		100 years
Importance Level (AS/NZS 1170.0.2002 Table 3.1)		IL1
Static Loading DAC	Overall Slope	FoS ≥ 1.3
	Single Bench	FoS ≥ 1.1
	Multiple Benches	FoS ≥ 1.2
Elevated Groundwater		FoS ≥ 1.1
Seismic Loading[1] DAC	Serviceability Limit State (SLS)	1:100-year (PGA 0.28g) FoS ≥ 1.0
	Ultimate Limit State (ULS)	1:250-year (PGA 0.41g) FoS ≥ 1.0

Notes:
1. *Earthquake Magnitude (Mw) taken as 6.0*

To assess the implications of the degree of displacement (D) that may occur during seismic induced slope failure the conditions described in Table 22 from Bray and Travasarou (2007), outlines the size of seismically induced displacement.

Table 22: Seismic Displacement Thresholds (Bray and Travasarou (2007)

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

12.4 Failure Surfaces

Both circular and non-circular failure surfaces have been examined to determine those with the lowest level of stability. These have been identified using the search routines embodied within the SLIDE2 stability analysis programme.

13.0 Highwall Stability Assessment

The following sections discuss the slope stability assessment undertaken for the proposed pit highwalls.

13.1 Probable Highwall Failure Mechanisms

The kinematic analysis discussed in Section 12 indicates the proposed highwall orientations are theoretically susceptible to planar sliding failures dominated by bedding controlled failure and discontinuity-controlled wedge and topple failures. Localised faulting in the area can also cause a draping effect dragging the overlying stratigraphy up along the fault inclining the bedding. This can cause a localised steepening of the bedding increasing the risk of planar sliding along the bedding plane shear inferred at the base of the coal measures.

The slope stability assessment highwalls have been undertaken using the strength parameters presented in Table 10 to assess non-circular failure using the Morgenstern - Price method.

13.2 Slope Stability Results

Slope stability analysis has been undertaken using a generalised highwall profile with a low angle bedding as indicated by the bedding data acquired from the geophysical downhole data. Additional to the generic highwall assessment seven cross sections cut through areas interpreted to be representative largest highwalls have also been assessed.

The scenarios considered in the analysis and FoS are presented in Table 19.

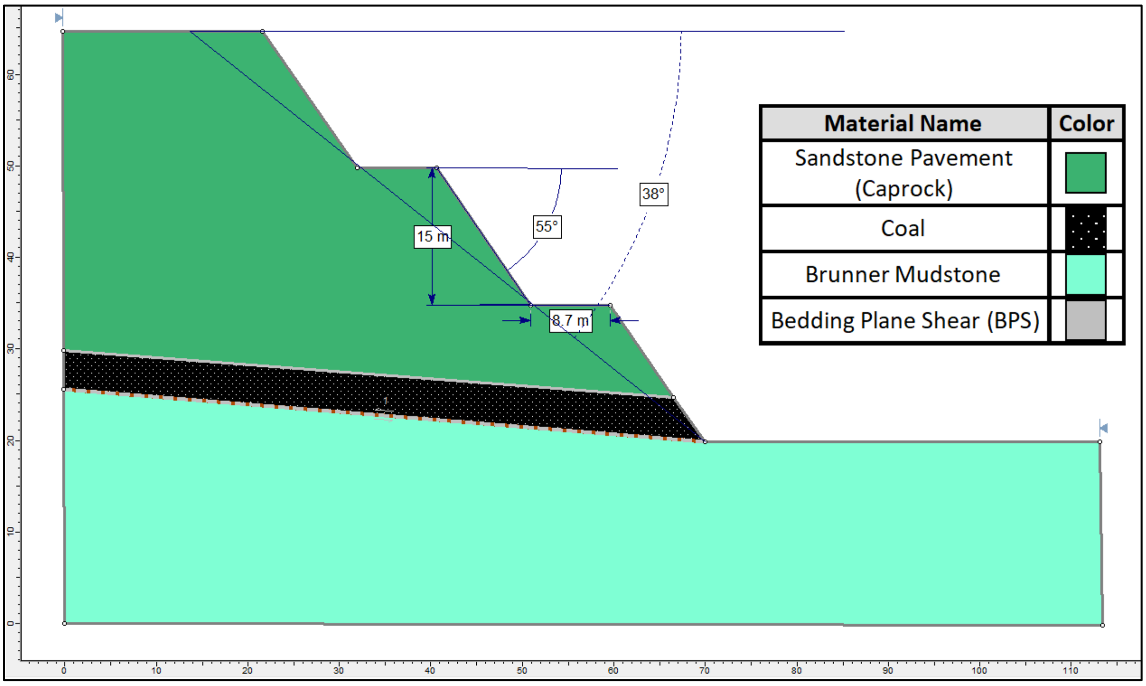

Figure 11: Generic MFS Highwall configuration low angle bedding.

Table 23 summarises the slope stability results for the generic highwall configuration indicated in Figure 11 the results of the proposed mine pit configurations are summarised in Table 24.

Table 23: Summary of Slope Stability Results – Generic Highwall

Scenario	Factor of Safety (FoS) – Global Minimum			
	Static	Elevated Groundwater	SLS (0.28g)	ULS (0.41g)
Overall Slope	1.8	1.8	1.0	**0.7**
Single Bench (Caprock)	4.1	-	2.9	2.5
Single Bench (Caprock / Coal including BPS))	2.3	-	1.2	**0.9**
Multiple Bench	1.8	-	1.0	**0.8**

Notes:
1. *Factor of Safety (FoS) are expressed as global minimum values.*

Table 24: Summary of Slope Stability Results - Highwalls / Sumps (Non-Circular Failure)

Section	Mine Feature	Factor of Safety (FoS) – Global Minimum			
		Static	Elevated Groundwater	SLS (0.28g)	ULS (0.41g)
100	East Pit Northwestern Highwall	2.1	1.3	**0.9**	**0.8**
	East Pit Southeastern Highwall	1.9	**0.5**	**0.9**	**0.7**
	East Pit – Northwestern highwall (East of Sump)	3.2	2.1	1.7	1.2
	East Pit – Southwestern Highwall	3.8	3.3	2.9	1.6
101	West Sump (North wall)	3.3	3.4	2.2	1.8
	West sump- (South wall)	3.4	4.3	2.4	1.9
	East Pit (North) Highwall	1.6	1.3	**0.9**	**0.7**
	East Pit (South) Highwall	3.2	3.0	1.8	1.3
	West Pit (North) Highwall – Downslope Failure	4.8	3.7	1.8	1.2

<table>
<tr><th colspan="6">Table 24: Summary of Slope Stability Results - Highwalls / Sumps (Non-Circular Failure)</th></tr>
<tr><th rowspan="2">Section</th><th rowspan="2">Mine Feature</th><th colspan="4">Factor of Safety (FoS) – Global Minimum</th></tr>
<tr><th>Static</th><th>Elevated Groundwater</th><th>SLS (0.28g)</th><th>ULS (0.41g)</th></tr>
<tr><td rowspan="2"></td><td>West Pit (North) Highwall – Into Pit Failure</td><td>4.4</td><td>3.5</td><td>2.1</td><td>1.9</td></tr>
<tr><td>West Pit (South) Highwall</td><td>4.6</td><td>3.0</td><td>2.3</td><td>1.7</td></tr>
<tr><td rowspan="5">102</td><td>East Pit, Northern Highwall</td><td>1.8</td><td>0.7</td><td>0.9</td><td>0.7</td></tr>
<tr><td>East Pit, South-western Highwall</td><td>3.5</td><td>2.8</td><td>2.7</td><td>2.2</td></tr>
<tr><td>West Pit, Northern Highwall</td><td>3.3</td><td>2.5</td><td>1.4</td><td>1.0</td></tr>
<tr><td>West Pit, Southern Highwall (Failure into Pit)</td><td>5.2</td><td>5.2</td><td>3.5</td><td>2.1</td></tr>
<tr><td>West Pit, Southern Highwall Block (Down slope Failure)</td><td>3.5</td><td>2.7</td><td>1.3</td><td>1.0</td></tr>
<tr><td rowspan="3">103</td><td>East Sump/Highwall – East Pit</td><td>2.3</td><td>2.2</td><td>1.9</td><td>1.4</td></tr>
<tr><td>East Pit – Highwall (South)</td><td>2.5</td><td>2.2</td><td>1.4</td><td>1.0</td></tr>
<tr><td>East Pit -Highwall (North)</td><td>3.2</td><td>1.3</td><td>1.4</td><td>1.0</td></tr>
<tr><td rowspan="3">104</td><td>East Pit - Highwall (North)</td><td>1.6</td><td>1.4</td><td>0.8</td><td>0.6</td></tr>
<tr><td>East Pit Highwall (South Western) into pit.</td><td>4.5</td><td>3.2</td><td>2.6</td><td>1.8</td></tr>
<tr><td>East Pit Highwall (South Western) Downslope failure option.</td><td>4.1</td><td>2.7</td><td>1.7</td><td>1.2</td></tr>
<tr><td rowspan="4">105</td><td>East Pit Highwall (Northeast)</td><td>1.8</td><td>1.3</td><td>1.1</td><td>0.8</td></tr>
<tr><td>East Pit Highwall (Southwest)</td><td>2.7</td><td>2.5</td><td>1.8</td><td>1.0</td></tr>
<tr><td>West Pit Highwall (Northeast)</td><td>6.9</td><td>6.1</td><td>4.6</td><td>3.5</td></tr>
<tr><td>West Pit Highwall (Southwest)</td><td>14.3</td><td>12.5</td><td>6.0</td><td>4.1</td></tr>
<tr><td>106</td><td>East Pit Highwall – Western Side of Pit</td><td>2.0</td><td>1.4</td><td>0.9</td><td>0.7</td></tr>
</table>

Table 24: Summary of Slope Stability Results - Highwalls / Sumps (Non-Circular Failure)					
Section	**Mine Feature**	**Factor of Safety (FoS) – Global Minimum**			
		Static	**Elevated Groundwater**	**SLS (0.28g)**	**ULS (0.41g)**
	East Pit Highwall – Eastern Side of Pit	2.0	1.6	**0.8**	**0.6**
	East Pit - Fault Scarp (F1_1) Circular	6.5	4.1	2.6	1.8
	East Pit - Fault Scarp (F1_1) non-circular	6.4	4.0	2.7	2.0

Notes:
1. *Factor of Safety (FoS) are expressed as global minimum values.*

13.3 Sensitivity analysis

For scenarios which failed under SLS conditions a sensitivity analysis was undertaken to estimate the yield acceleration to trigger a slope failure. The outputs of the sensitivity analysis are presented in Appendix G. Failure angles were interpreted from global minimum failure surface calculated during slope stability analysis. The sensitivity analysis outputs for slope failures are summarised in Table 25.

Table 25: Yield Acceleration Estimation for ULS Failures		
Mine Feature	**Angle of Failure Surface (°)**	**Estimated Yield-Acceleration (g)**
Section 100 - East Pit Northwestern Highwall[1]	13°	0.25
Section 100 - East Pit Southeastern Highwall	11°	0.22
Section 101 - East Pit Highwall	16°	0.19
Section 102 - East Pit, Northern Highwall	6°	0.23
Section 104 - East Pit - Highwall (North)	14°	0.16
Section 105 - East Pit Highwall (Northeast) [1]	5°	0.31
Section 106 - East Pit Highwall – Western Side of Pit	12°	0.15
Section 106 - East Pit Highwall – Eastern Side of Pit	10°	0.24

Notes:
1. *Ultimate limit state (ULS) failure only*

The results of the sensitivity analysis indicate the estimated yield acceleration for highwall failure decreases with increasing slope failure angle.

13.4 Displacement of Failure (Highwalls)

To assess the degree of displacement (D) that may occur due to seismic induced slope failures, an analysis based on sliding block theory, using the method outlined by Jibson (2007) was completed. Table 21 presented in Section 13.3, based on Bray and Travasarou (2007), outlines the size of seismically induced displacement, and the implications associated with such a failure. The results of the analysis are summarised below in Table 26.

Table 26: Estimated Displacements Results (Jibson 2007) Non-Circular Failure

Mine Feature	Displacement (cm) SLS (ULS)	Performance Level SLS (ULS)
Section 100 - East Pit Northwestern Highwall[1]	0.0 (0.4)	Stable (Stable)
Section 100 - East Pit Southwestern Highwall	0.1 (0.8)	Stable (Stable)
Section 101 - East Pit Highwall	0.2 (1.4)	Stable (Stable)
Section 102 - East Pit, Northern Highwall	0.0 (0.7)	Stable (Stable)
Section 104 - East Pit - Highwall (North)	0.8 (3.0)	Stable (Minor)
Section 105 - East Pit Highwall (Northeast)	0.8 (3.0)	Stable (Minor)
Section 106 - East Pit Highwall – Western Side of Pit[1]	0.0 (0.1)	Stable (Stable)
Section 106 - East Pit Highwall – Eastern Side of Pit	0.8 (3.0)	Stable (Minor)

Notes:
1. *Failure under ultimate limit state (ULS) only.*

Based on the results and the displacement thresholds presented in Tables 21 and 26 slope deformation under SLS and ULS seismic conditions are likely to result in minor cracking / displacements with the maximum deformation being 3 cm under ULS seismic load. These levels of displacement are easily managed with site specific Ground Control Management Plans and Trigger Action Response Planes.

14.0 Engineered Landform (ELF) Stability

It is understood that the ELF’s will comprise a mixture of overburden materials, primarily sandstone caprock and mudstone. Because of the nature of the overburden, it is anticipated that the fill will comprise material of varying block sizes, strengths and weathering and these will influence compaction and consolidation.

The following sections summarise the slope stability analysis undertaken for the ELFs and provide commentary regarding the stability of proposed ELF's.

14.1 Probable ELF Failure Mechanism

The slope stability of proposed ELF has been modelled using a generic slope profile (Figure 12) using SLIDE2 software using the Morgenstern - Price method.

The following ELF slope profile has been assumed as:

- Overall slope 18°.
- Maximum bench angle 36°.
- Bench width 5.5 m.
- This slope configuration has been chosen as the worst-case example.

The following assumptions have been made in the ELF stability analysis:

- The material used to construct the ELF is a homogeneous and uniform (this is unlikely to be the case).
- ELFs are free draining and therefore never fully saturated.
- All ex-pit ELFs have had any persistent organics on the surface have been removed.

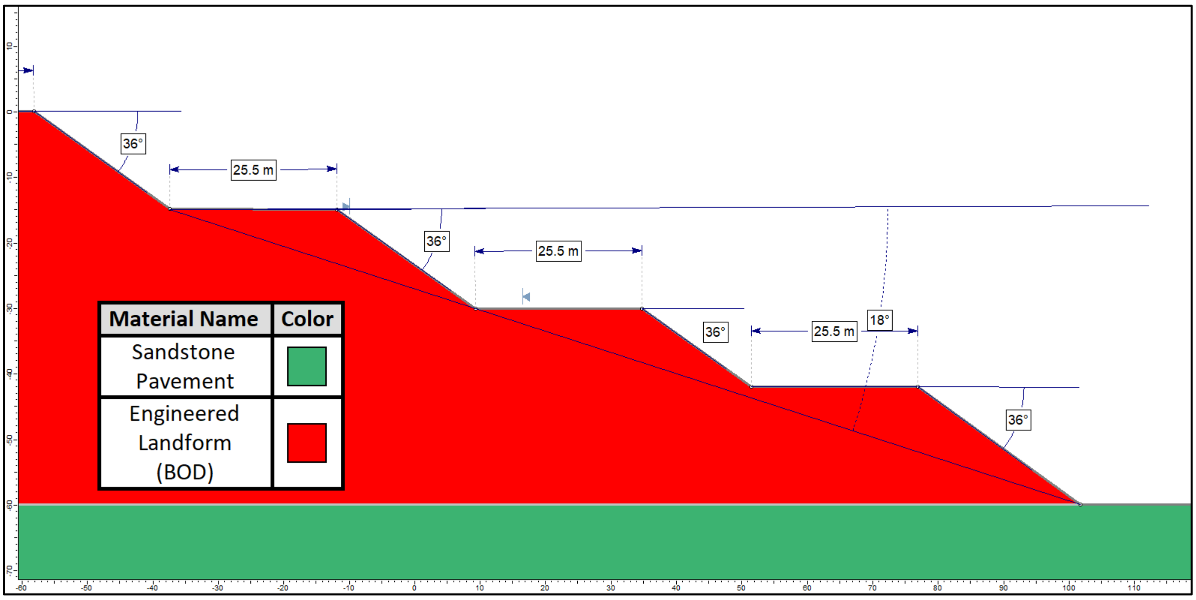

Figure 12: Generic slope profile of engineered landform (ELF).

14.2 Engineered Landform Slope Stability Results

To assess the stability of the proposed ELF a generic profile derived from the BOD was initially modelled using the slope configuration discussed in Section 15.1. It should be noted that this represents the least favourable orientation for slope stability with many of the proposed ELFs having lower overall slope angles. Additionally, multiple cross sections of the ELF profiles have also been assessed to represent the conditions expected at the site. The results of the slope stability assessments of the generic profile are presented in Tables 27 and more detailed assessment in Table 28.

Table 27: Summary of Slope Stability Results – Generic ELF

Scenario	Factor of Safety (FoS) – Global Minimum			
	Static	Elevated Groundwater	SLS (0.28g)	ULS (0.41g)
Overall Slope	1.3	1.3	0.8	0.6
Single Bench	1.3	-	0.8	0.6
Multiple Bench	1.3	-	0.8	0.6

Notes:
1. *Factor of Safety (FoS) are expressed as global minimum values.*

Table 28: Summary of Slope Stability Results (ELF) – Circular Failure

Section	Mine Feature	Factor of Safety (FoS) – Global Minimum			
		Static	Elevated Groundwater	SLS (0.28g)	ULS (0.41g)
100	E_ELF2	1.3	1.3	0.8	0.6
101	W_ELF1	2.6	2.4	1.3	1.0
	E_ELF3	1.3	1.3	0.8	0.6
103	E_ELF1	1.4	1.4	0.8	0.7
105	E_ELF1	1.2	0.9	0.7	0.6
106	E_ELF2	1.4	1.2	0.8	0.6

Notes:
1. *Factor of Safety (FoS) are expressed as global minimum values.*

The results of the stability analysis indicate the ELF slope profile is stable under static seismic and elevated groundwater conditions. However, it is likely to fail under both SLS and ULS seismic conditions. The degree of slope failure displacement is discussed in Section 15.3.

14.3 Newmark Displacement Analysis (ELF)

In Section 14.5 the Jibson sliding block analysis was used to assess the degree of displacement occurring along non-circular failure surface expected to occur in highwall failure. Slope stability analysis of the ELF's indicate that slope failures will be dominated by circular failures, which the Jibson sliding block model is not suitable to assess. Therefore, a Newmark displacement analysis was undertaken to estimate the degree of displacement expected to occur during serviceability (SLS) and ultimate (ULS) limit state conditions. To assess this real-world seismograph profiles with similar PGA's and earthquake magnitudes (Mw) as presented in Table 19 to the were applied to the generic ELF slope profile. For this analysis the magnitude Mw5.9 Santa Barbara 1978 (SBA-222) with a PGA of 0.20g was used to simulate SLS condition and the magnitude Mw6.1 Mammoth Lakes-1 1980 (CVK-090) with a PGA of 0.41g to simulate ULS conditions.

The results of the Newmark analysis indicate global minimum failure depths approximately up to 5.0 m with displacements of up to 0.19 cm under SLS conditions with displacement localised to individual bench faces and displacements of up to 5.0 cm under ULS conditions with most displacement failure occurring on bench slopes. Based on the displacement thresholds presented in Table 19 these levels of displacement are considered stable and minor under SLS and ULS conditions respectively.

15.0 W_ELF1 Underground Workings Foundation Assessment

As discussed in Section 7.0, the MFS area is underlain by extensive historical underground bord and pillar workings associated with the M2 and M3 coal seams. Construction over these features presents elevated geotechnical risks, including potential subsidence and ground instability, particularly where overburden thickness is limited, many of the workings already show signs of collapse. Based on the proposed MFS design layout, several of these underground workings are situated within the footprints of the west pit, west sump, and W_ELF1. The approximate locations of the affected workings and associated mine features are shown in Figure 5 (Appendix A).

The historical workings contained within the pit and sump footprints are not considered to pose any significant geotechnical issues as these underground workings are expected to be excavated during the mine construction. However, the current mine design indicates that W_ELF1 is proposed to be constructed over an area of shallow historical workings. Due to the shallow depth of the historical workings below the existing ground surface it is likely that construction activities (vehicle movements / construction of ELF) in this area may induce collapse in the shallow historical workings.

To manage the geotechnical risks associated with historical underground mine workings, a range of targeted mitigation measures should be implemented. These include detailed geotechnical investigations to delineate voids, selective excavation of unstable or collapsed workings, and backfilling or grouting to stabilise subsurface conditions. Reinforcement of overlying strata may be required to support critical mine infrastructure, and design adjustments should be made to avoid or span high-risk zones. Continuous ground monitoring and adaptive management practices should be implemented to ensure operational safety and long-term stability throughout the life of the mine.

16.0 ELF Preliminary Liquefaction Assessment

The following summarises and preliminary liquefaction assessment of the ELFs.

- The InPit ELFs will mainly be constructed from BCM overburden comprising predominantly free draining sandstones as observed at Stockton Mine. In the eventuality that the In-Pit ELF forms a piezometric surface from shallow groundwater infiltration, the coarse nature of the fill will unlikely liquefy under seismic loading as excess pore pressure will be able to dissipate.
- The Ex-Pit ELFs will be formed mainly from basement rocks (greywacke / argillite) where finer grained deposits may be present. At this stage it is unknown how these basement rocks will disaggregate on excavation and placement. As mentioned previously fine-grained materials have a higher propensity to liquefy. However, in this instance the Ex-Pit ELF’s will be formed above measured groundwater as such it is unlikely a piezometric surface could form from rainfall infiltration alone, especially if the ELFs are constructed to promote surface water shedding. Without water the ELF backfill will not liquify.

17.0 Northern Haul Road

As indicated on Figure 2 and 6 (Appendix A) it is proposed to construct a haul road between the MFS and the southern part of the existing Stockton mine. The haul road concept indicates that cuts of up to approximately 50 m may be required to construct this road. Four cross section profiles through the areas of the greatest cut height for the Northern Haul Road alignment were developed and the overall stability of the cut slope assessed for each. The slope stability analysis has been undertaken using the parameters presented in Table 10. The results of these assessments are presented in Table 29.

Table 29: Summary of Slope Stability Results – Access Road Areas of Maximum Cut

Section	Mine Feature	Factor of Safety (FoS) – Global Minimum			
		Static	Elevated Groundwater	SLS (0.28g)	ULS (0.41g)
107	Haul Road Cut	3.4	3.2	2.2	1.9
108	Haul Road Cut	3.9	3.2	2.5	2.1
109	Haul Road Cut	4.8	3.8	2.9	2.4
110	Haul Road Cut	5.0	4.6	3.2	2.7

Notes:

1. *Factor of Safety (FoS) are expressed as global minimum values.*

18.0 Southern Haul Road

The southern haul road route provided to us by BTM indicated on Figures 2 and 6 (Appendix A) is proposed to run approximately 3.5 km south from the eastern end of the west pit down to its terminus near Ceder Creek Road.

Based on the data provided by BTM the haul road will generally be constructed in BCM sandstone pavement, with only two small segments of the haul road cross's the east side of the West ELF in the north and transitions from BCM Sandstone to Kaiata Formation mudstone in the south at the location of the coal transfer pad and proposed water treatment plant. It is anticipated that the haul road will require only minor cut and fill earthworks. As discussed in Section 5.1 Kaiata Formation mudstone is generally very weak, however, based on the proposed southern haul road design there are no cuts proposed in the mudstone and therefore is not considered to pose any significant risk to the stability of the haul road.

One bridge crossing across Deep Creek and two culvert crossings across the Waimangaroa River and its tributaries are proposed along the southern haul road route. No design for these crossings has been undertaken at this level, but geotechnical assessments for these crossings will be required at future development phases.

19.0 Final Landform

At the time of reporting no design surfaces for the final landforms had been provided to PDP, although it is our understanding that the east and west sumps will remain operational for stormwater water control for a time post the completion/closure of the mine. Additional long-term stability analysis of the final landform is required once determined.

20.0 Geotechnical Summary Residual Risks

During the pre-feasibility geotechnical assessment, no fatal flaws were identified that would preclude development of the MFS area. However, several geotechnical data gaps were noted, which are summarised in Table 30 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 30: Geotechnical Data Gaps and Risk

Identified Data Gap	Geotechnical Risk	Mitigation Measure
	Highwalls / SW Sumps / ELFs	
Limited geotechnical data across the MFS area.	Fault / discontinuity locations, spatial distribution and rock strength parameters likely to vary across MFS.	Site mapping and drilling program to infill areas with limited data. Update database/geotechnical models.
Extent of underground workings needs better resolution.	Collapse of shallow workings from ELF loading. Potential for weakened ground where underground workings intersect pit highwalls.	Detailed mapping of underground workings within the MFS area. Redesign highwall profile to accommodate removal of underground workings. Consider collapse blasting to mitigate risk.
Poor definition of faults, crush zones, intersections of faults in highwalls and localised steepening of BPS due to folding.	Increased risk of slope failure due to unknown structures prior to highwall development.	Geotechnical hazard management, use GCMP and TARP. Ongoing geotechnical mapping of highwall benches and update database/geotechnical models during operations.
Insufficient laboratory data to create a statistical data set on the rock mass strength parameters of BCM across MFS.	Rock strength parameters from Stockton used in this analysis not representative of condition encountered at MFS.	Undertake laboratory testing for rock strength parameters within the MFS area from additional coring.
No strength parameters for BSM greywacke / argillite for assessment for proposed sump excavation.	Estimated parameters used in slope stability analysis may not be representative of actual rock strength.	Undertake laboratory testing for rock strength parameters within the MFS area.

Table 30: Geotechnical Data Gaps and Risk

Identified Data Gap	Geotechnical Risk	Mitigation Measure
Underground workings cover thickness, strength and classification (UCS / RMR / GSI).	Risk of foundation failure or subsidence from ELF static loading. Collapse of underground workings from operational (traffic) loading.	Detailed mapping and probe drilling of cover materials over underground workings. Excavate or deliberately collapse the underground workings situated within operational areas of the mine.
Possible shallow groundwater levels in the north of the MFS area, shallow underground workings forming potential connectivity with open pits.	Localised groundwater seepages from highwalls possible causing saturation at coal floor and within in-pit ELFs.	Work with HGG to assess risks of shallow groundwater interactions and possible groundwater flows.
No strength parameters for greywacke / argillite incorporated in ELFs.	Behaviour of the excavated / blasted greywacke / argillite material when redeposited into ELFs unknown.	Visual assessment of rock quality from drill core. Laboratory testing to determine strength parameters associated with the replacement of excavated / blasted grey wacke rock.
Possible leakage from sumps into existing underground workings, faults / crush zones.	Destabilisation of slopes and underground workings as a result of increased groundwater levels.	Detailed mapping and modelling of any areas likely to have leakage into underground workings, faults / crush zones. Review updates in groundwater model from HGG.
Seismic hazard model requires updating to align with recent updates.	Seismic parameters used in design may not be sufficient relative to the latest seismic model.	Update site seismic hazard model.

Table 30: Geotechnical Data Gaps and Risk

Identified Data Gap	Geotechnical Risk	Mitigation Measure
Preparation of coal floor prior to backfill / construction of ELFs.	Sliding of ELF on coal floor.	Rip / scarify coal floor prior to ELF construction to increase basal friction. Bench on steeper dipping areas. Assess shear key requirements.
Final Landform configuration	Long term instability of sump highwalls and ELF slopes	Additional infill investigation and associated long-term stability modelling once the final landform configuration has been determined.
	Haul Road Alignments	
There are no drillholes along the proposed northern haul road accessway.	Unknown ground condition along proposed haul road route.	Undertake targeted mapping and drillhole investigation / laboratory testing to inform haul road design.
Change in geology from north to south along haul road route from altered greywacke (hornfels) to unaltered greywacke.	Rock strength parameters may vary along haul road route. Potential for areas of highly to completely weathered rock to be present. Increase slope instability risks.	Site mapping and drilling program to infill areas with limited data. Update database/geotechnical models.

Table 30: Geotechnical Data Gaps and Risk

Identified Data Gap	Geotechnical Risk	Mitigation Measure
Poor geological definition along haul road to assess cover / colluvium stability and existing groundwater conditions. No data on the thickness or extent of any shallow colluvium.	Failure of slope along haul road route.	Site mapping and drilling program to infill areas with limited data. Update database/geotechnical models.
No rock mass data to assess slope stabilisation requirements, bolting / mesh etc.	Slope failure due to insufficient ground support design.	Site mapping and drilling program to infill areas with limited data. Update database/geotechnical models. Laboratory testing of rock within road cuts to inform haul road design.
Earthworks specifications for haul road filling, foundation cut requirements, fill design to accommodate truck loads.	Insufficient bearing capacity, slope instability.	Produce earthworks specification specific to the MFS project.
Specific geotechnical drillholes for bridge crossings / culverts.	Bridge abutment bearing failure or settlement.	Undertake geotechnical investigations for any areas requiring bridge crossings.
Specific geotechnical investigation for of the Coal transfer pad and water treatment plant.	Foundation bearing failure or settlement.	Undertake geotechnical investigations for any areas requiring foundations and large fill loadings.

21.0 Conclusion

This pre-feasibility study for the proposed Mount Fred South Mine has assessed key geotechnical factors, including slope geometry, material properties, geological structure, and historical underground workings. Kinematic analysis was used to evaluate potential failure mechanisms, with emphasis on faulting and structural discontinuities. Identified risks such as slope instability, subsidence, and rock mass variability are considered manageable through targeted operational controls, including Ground Control Management Plans (GCMPs), Trigger Action Response Plans (TARPs), and ongoing geotechnical monitoring. No major geotechnical risks have been identified that would preclude development. Further refinement of the geotechnical model and incorporation of probabilistic analysis during the feasibility stage will enhance risk mitigation and support informed design decisions.

22.0 References

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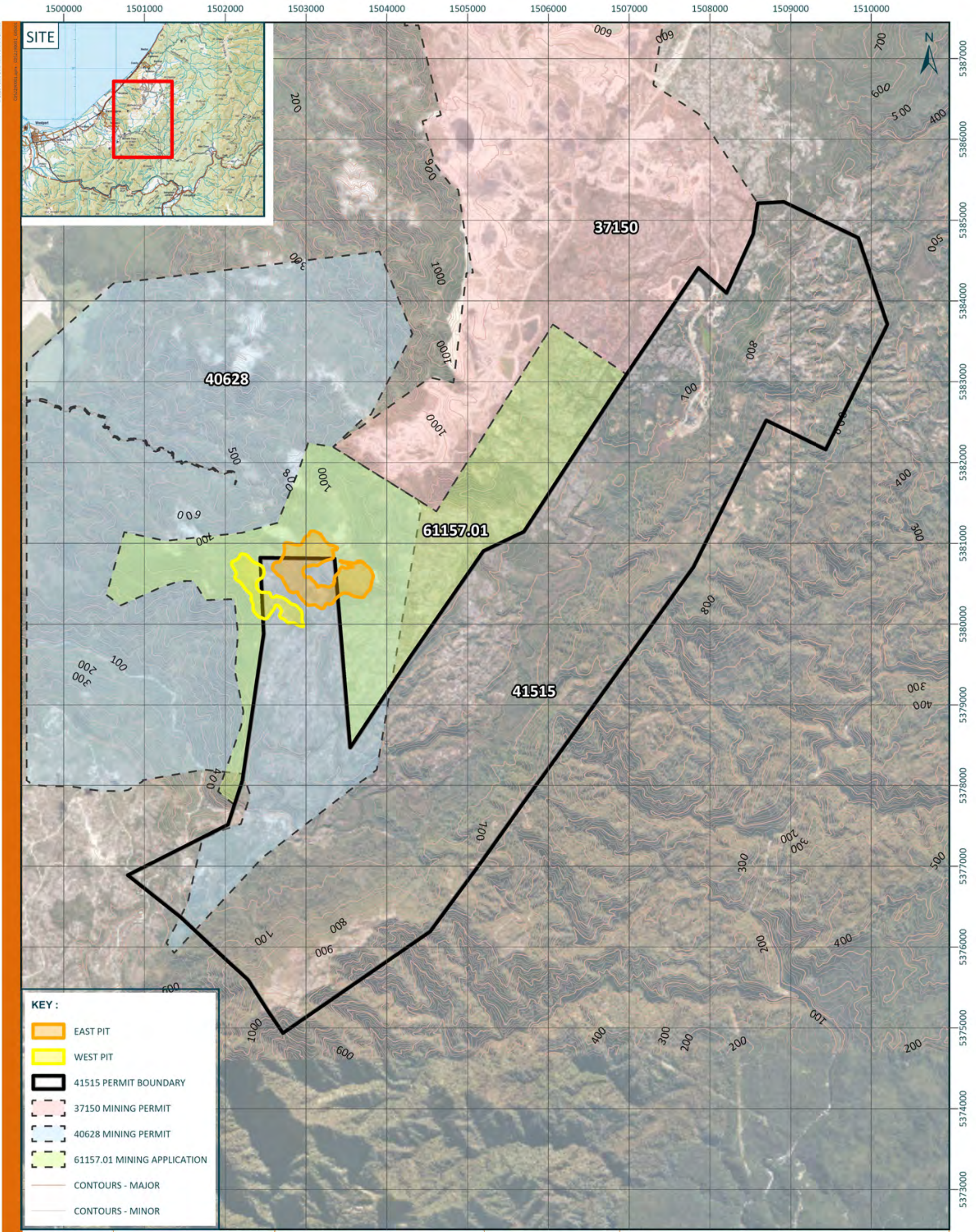
Appendix A: Site Plans



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2. CADASTRAL/TOPOGRAPHICAL INFORMATION AND INSET DERIVED FROM LINZ DATA.

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FIGURE
FIGURE 1: SITE OVERVIEW

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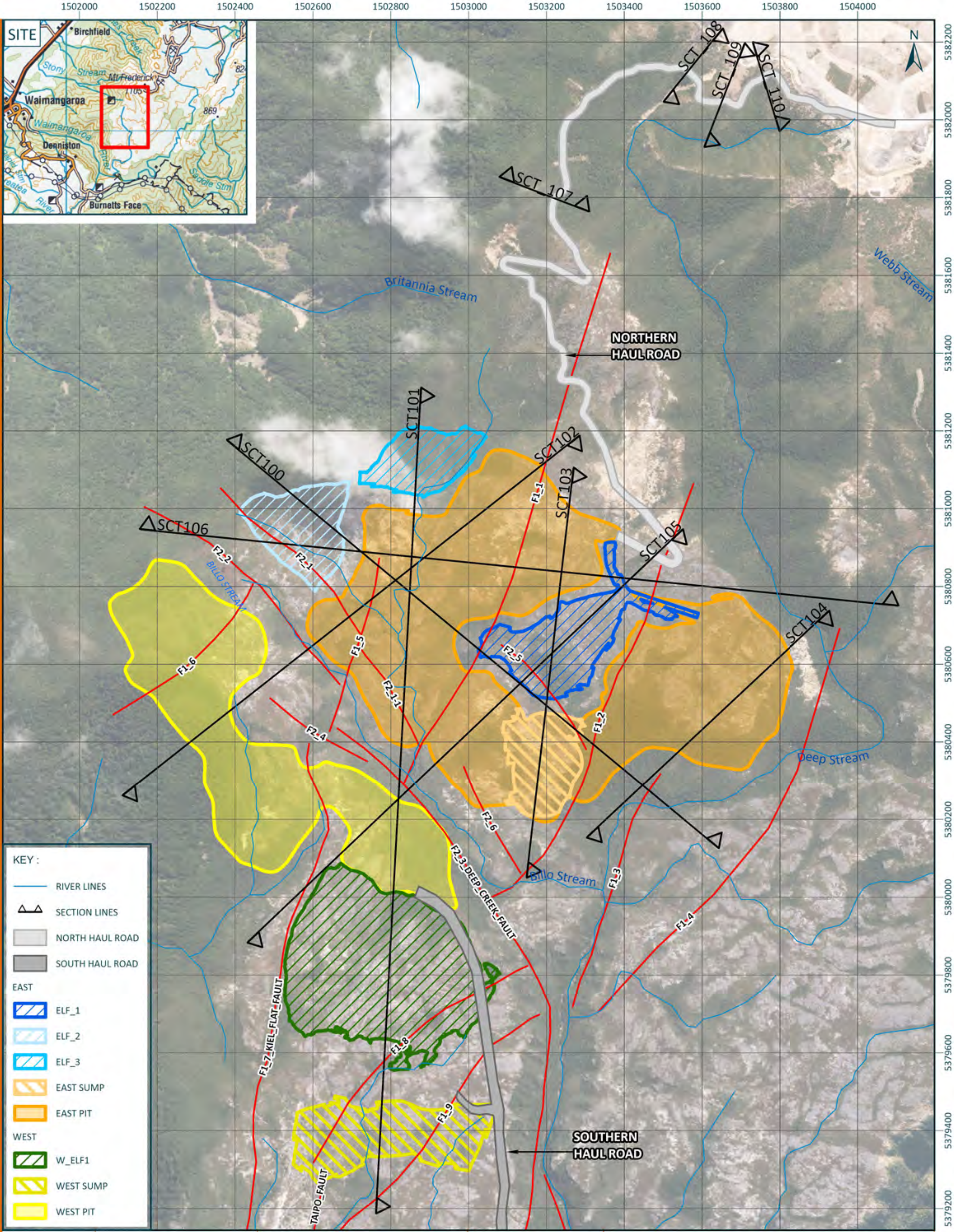

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2. CADASTRAL/TOPOGRAPHICAL INFORMATION AND INSET DERIVED FROM LINZ DATA.

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FIGURE

FIGURE 2: PITS, FAULTS, SECTION LINES AND HAUL ROAD

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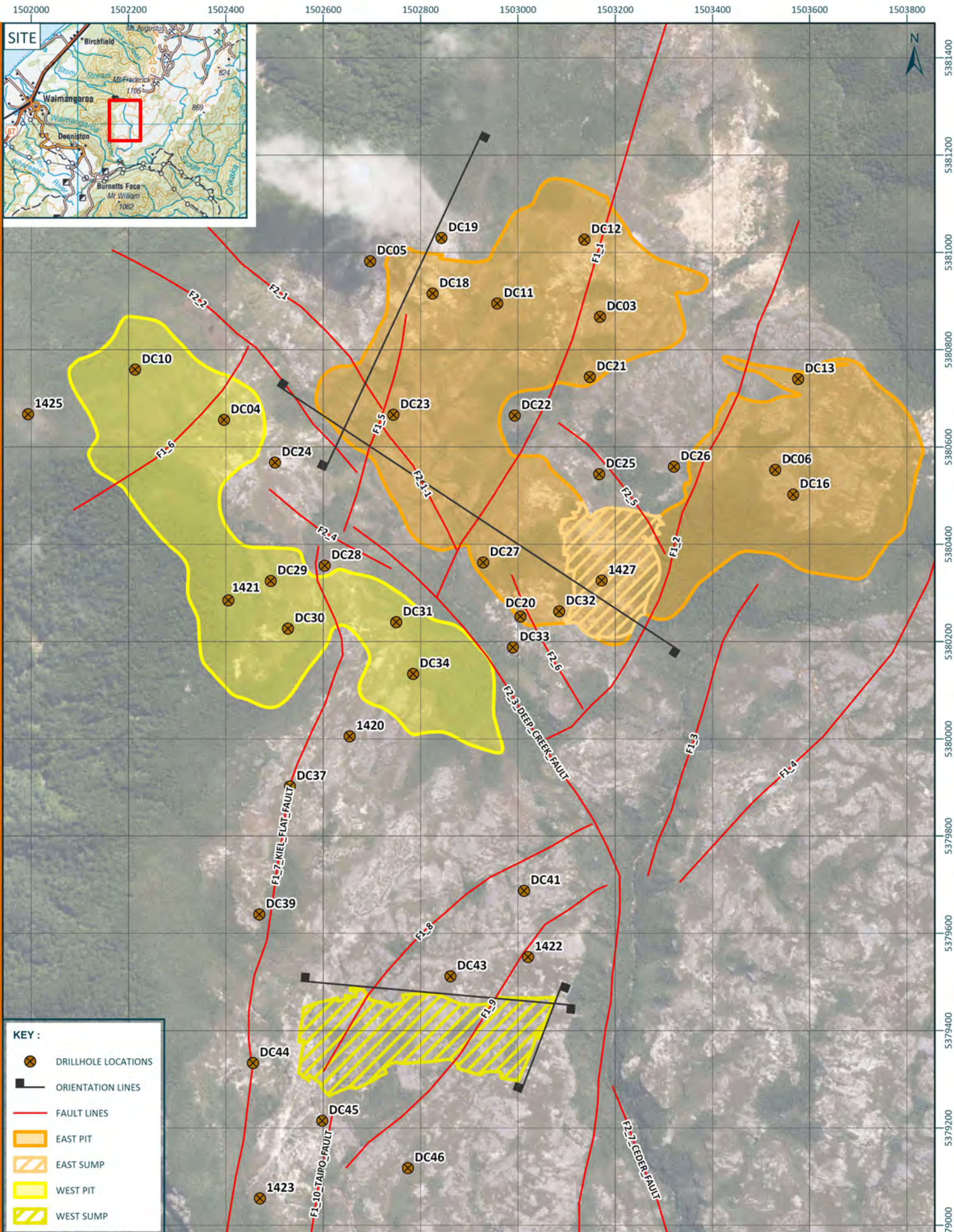

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FIGURE
FIGURE 3: PITSHELLS, FAULTS, SUMP DRILLHOLES

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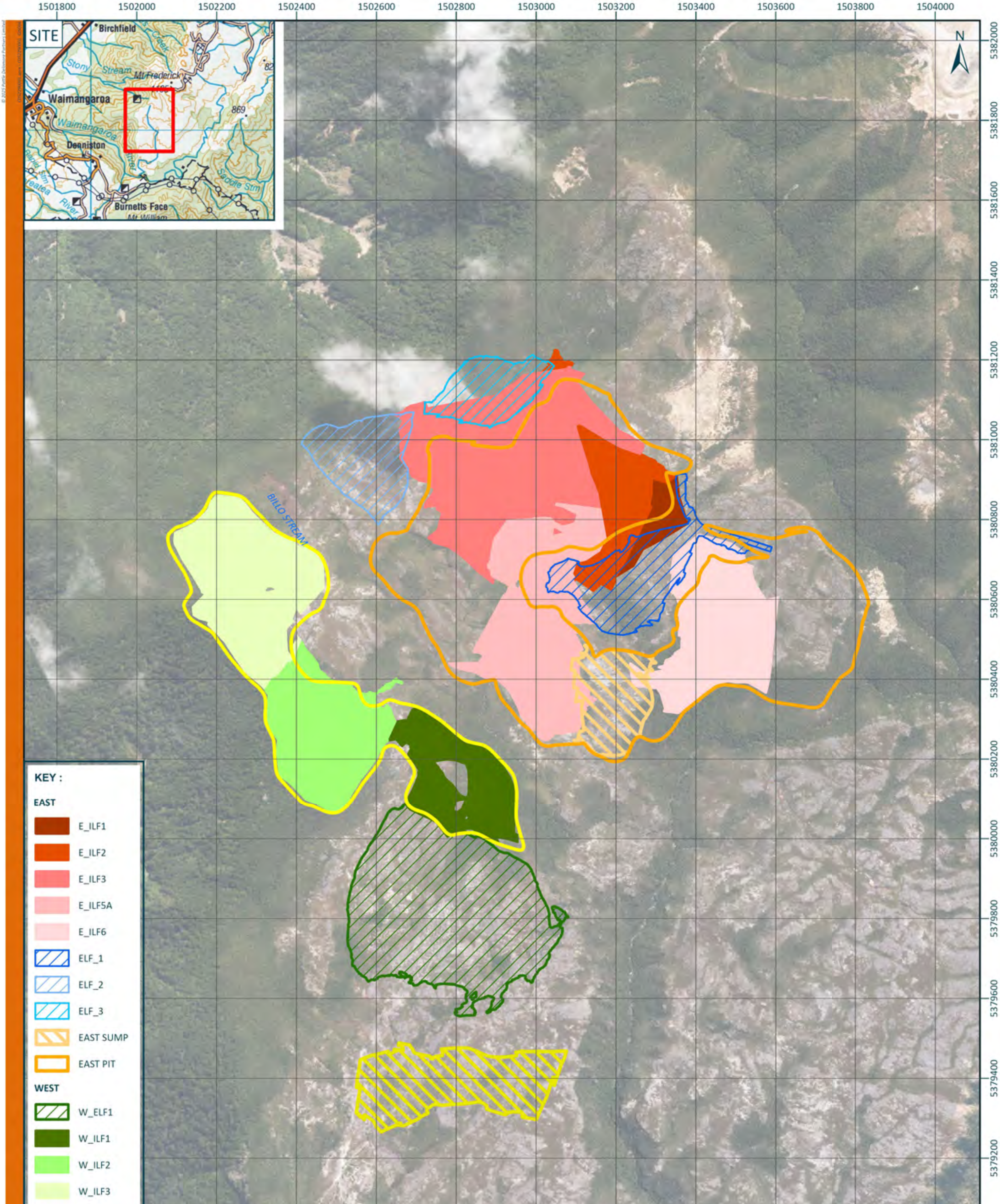

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FIGURE
FIGURE 4: FINAL LANDFORMS

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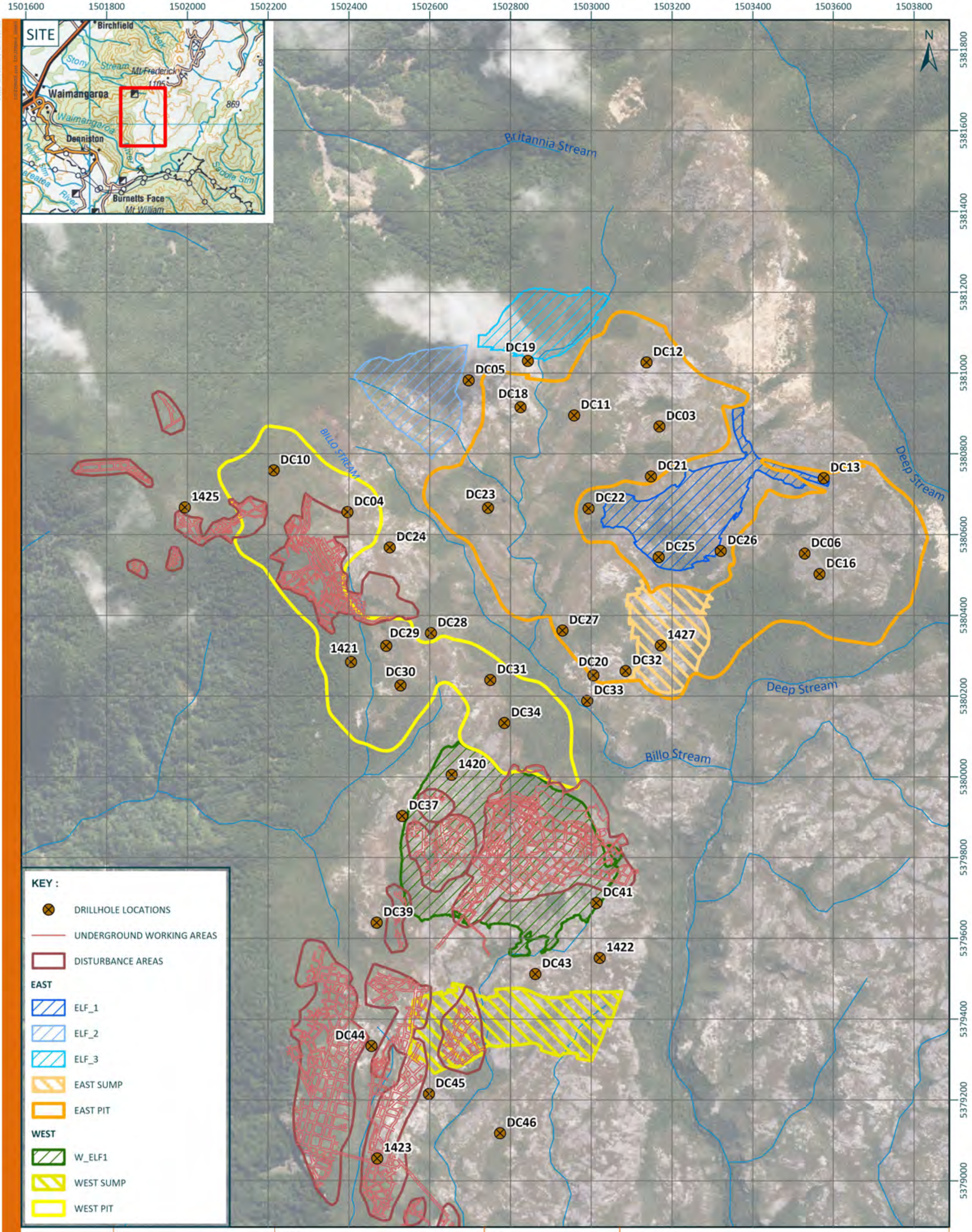

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FIGURE
FIGURE 5: UNDERGROUND WORKINGS

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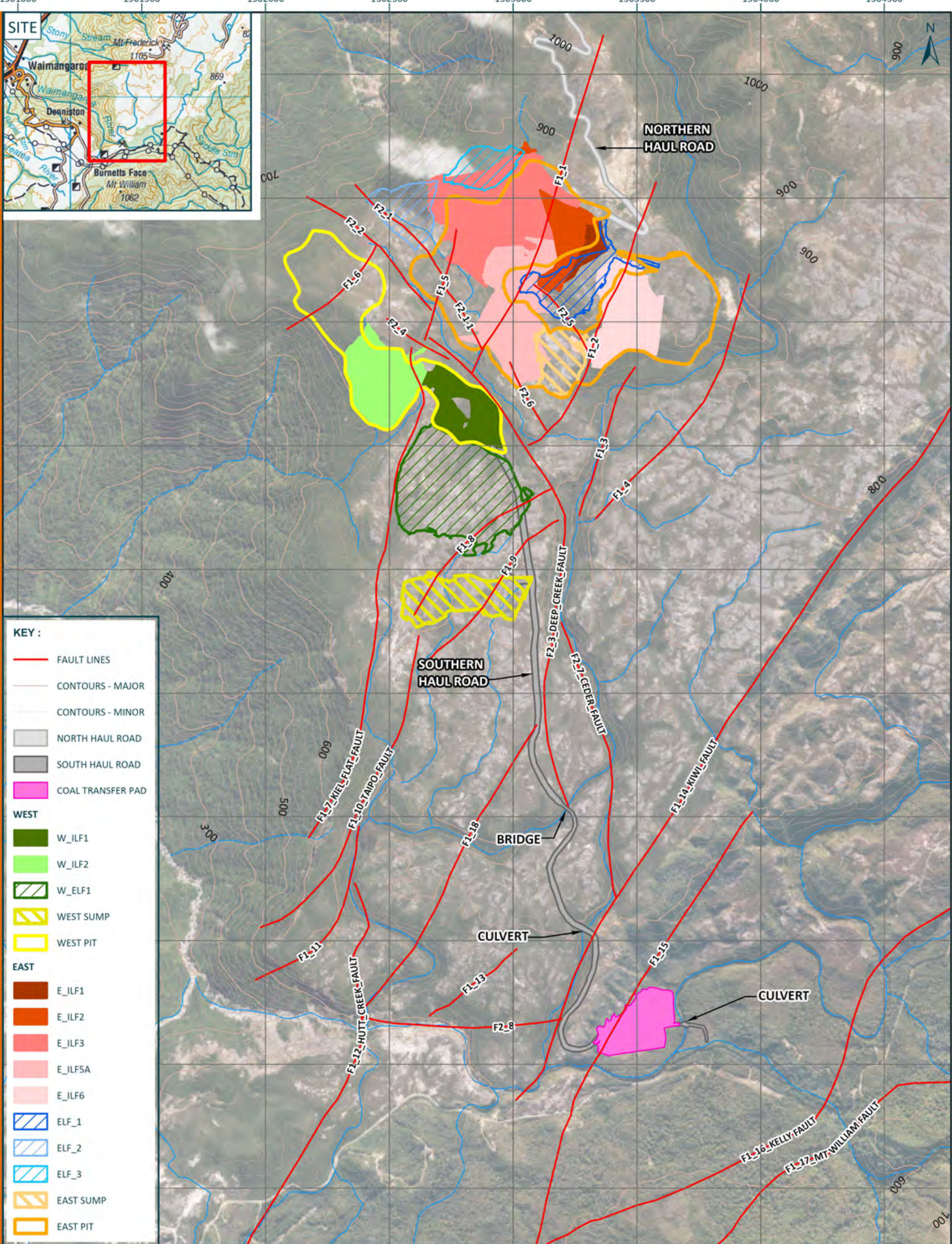

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FIGURE
FIGURE 6: SOUTHERN HAUL ROAD

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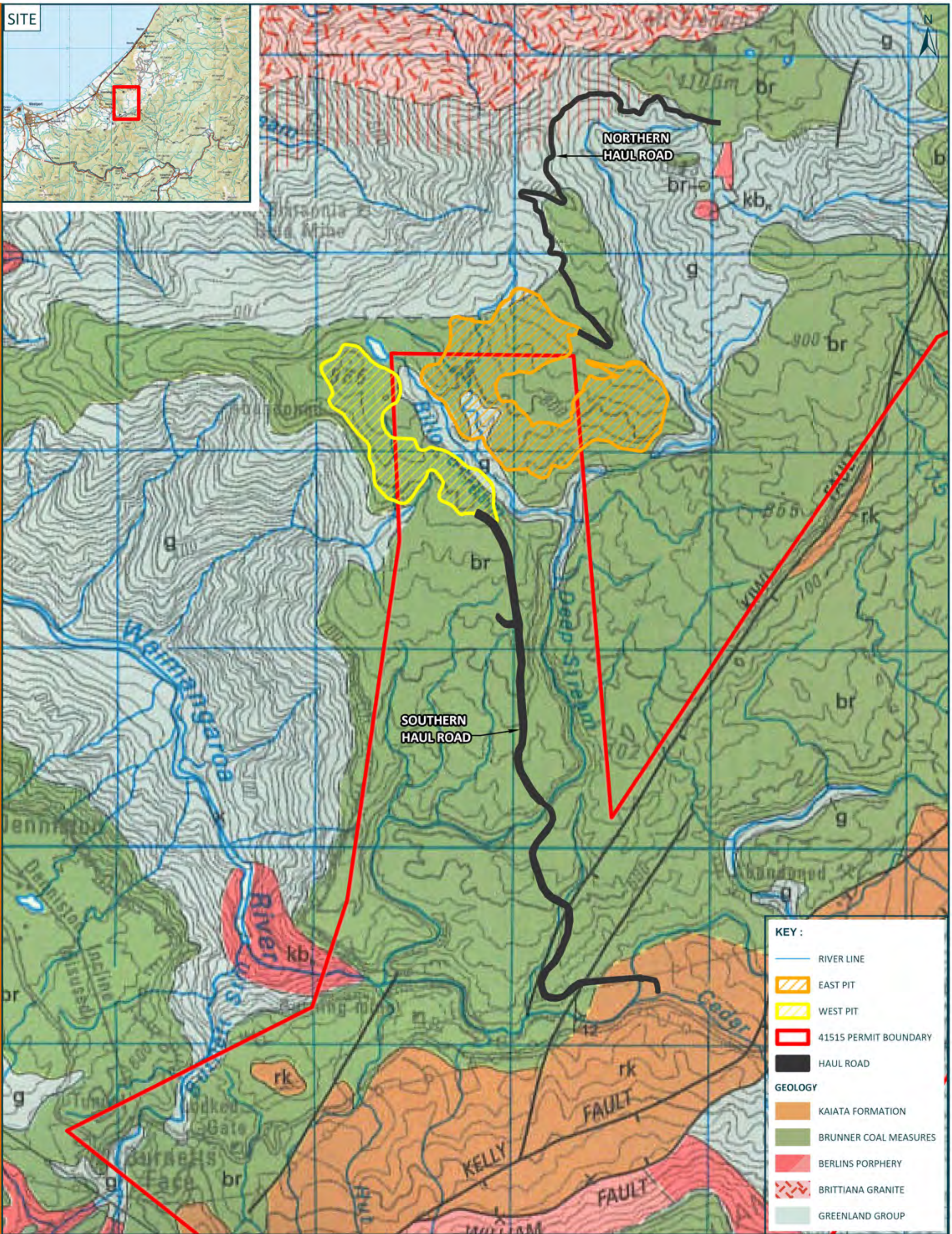

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FIGURE
FIGURE 7: GEOLOGY MAP

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Appendix B: Slope Stability Analysis

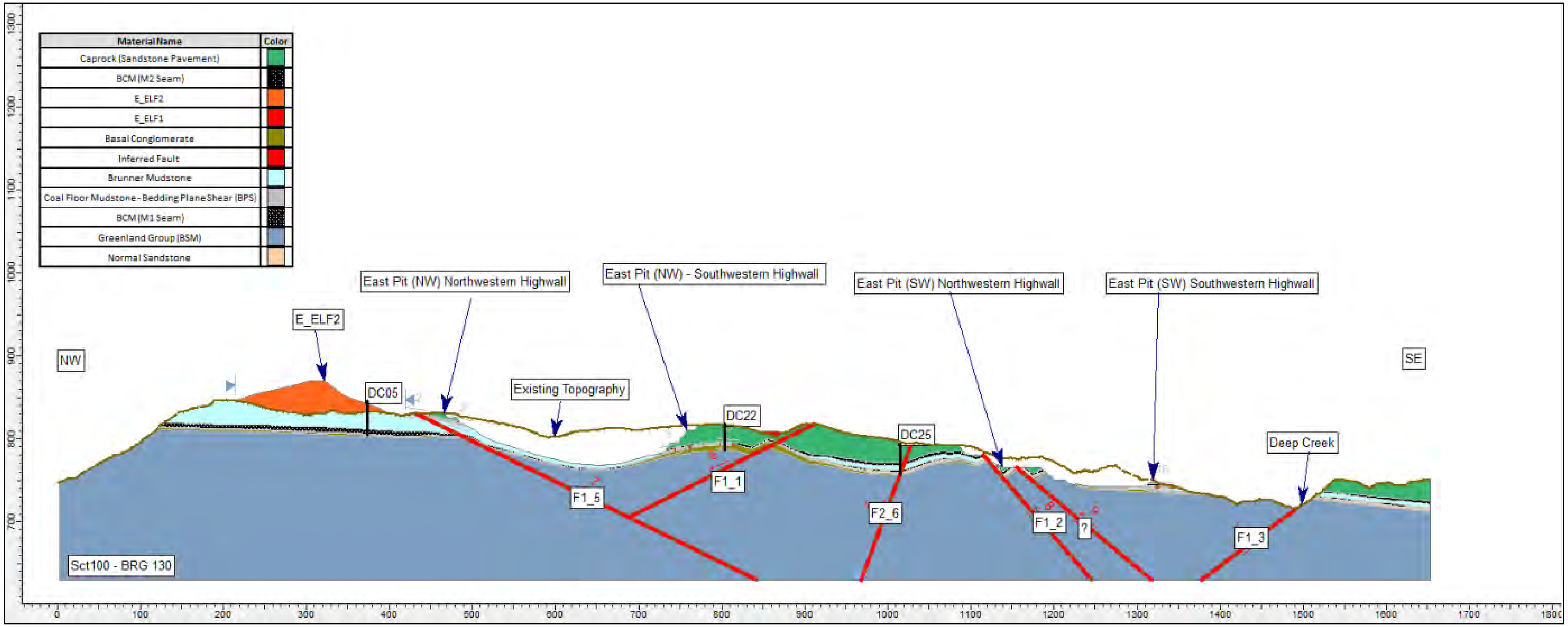

Figure B1 - Section 100 – Overview of Mine Features.

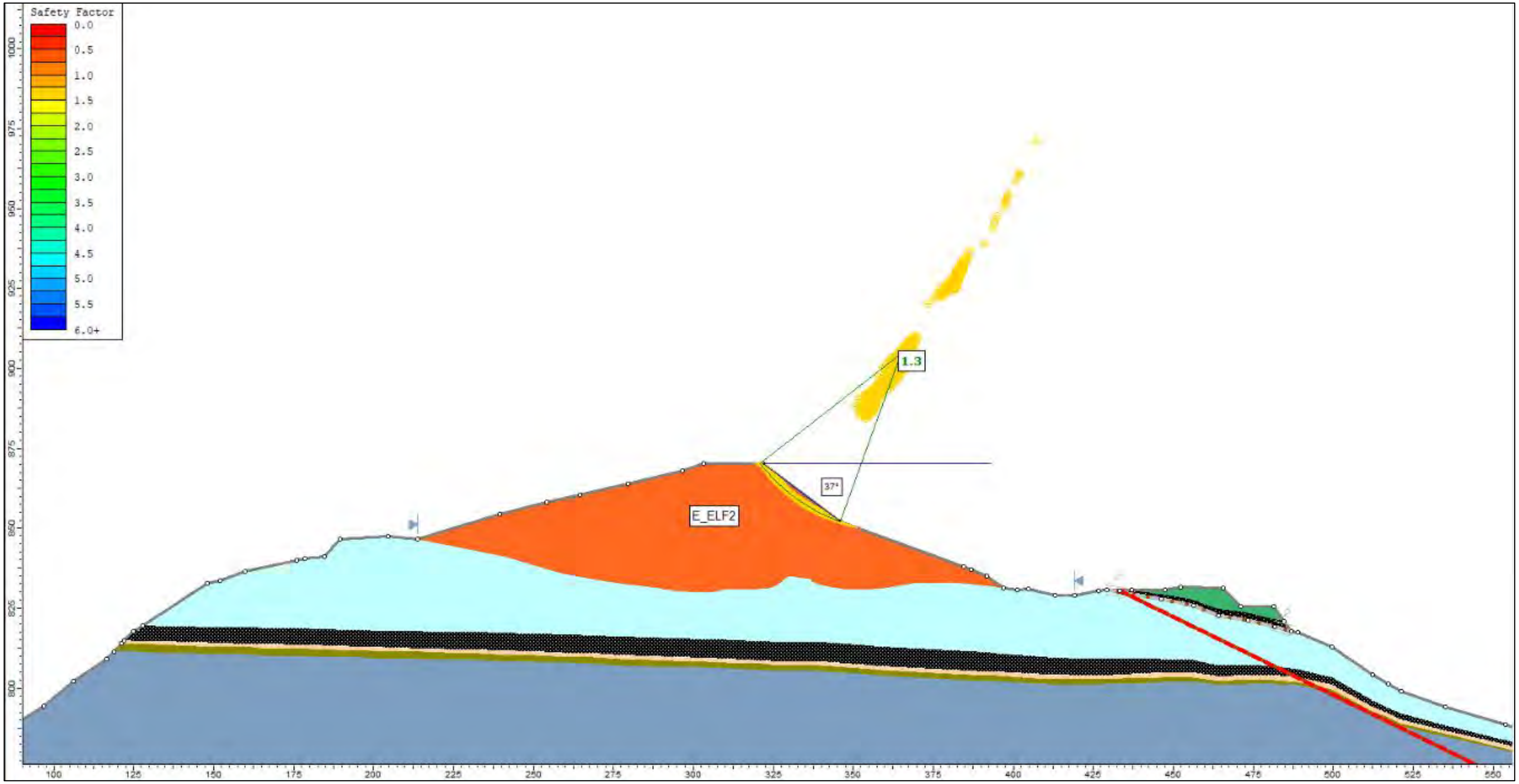

Figure: B2 – Slope Stability Analysis of E_ELF2 – Section 100 – Static Conditions.

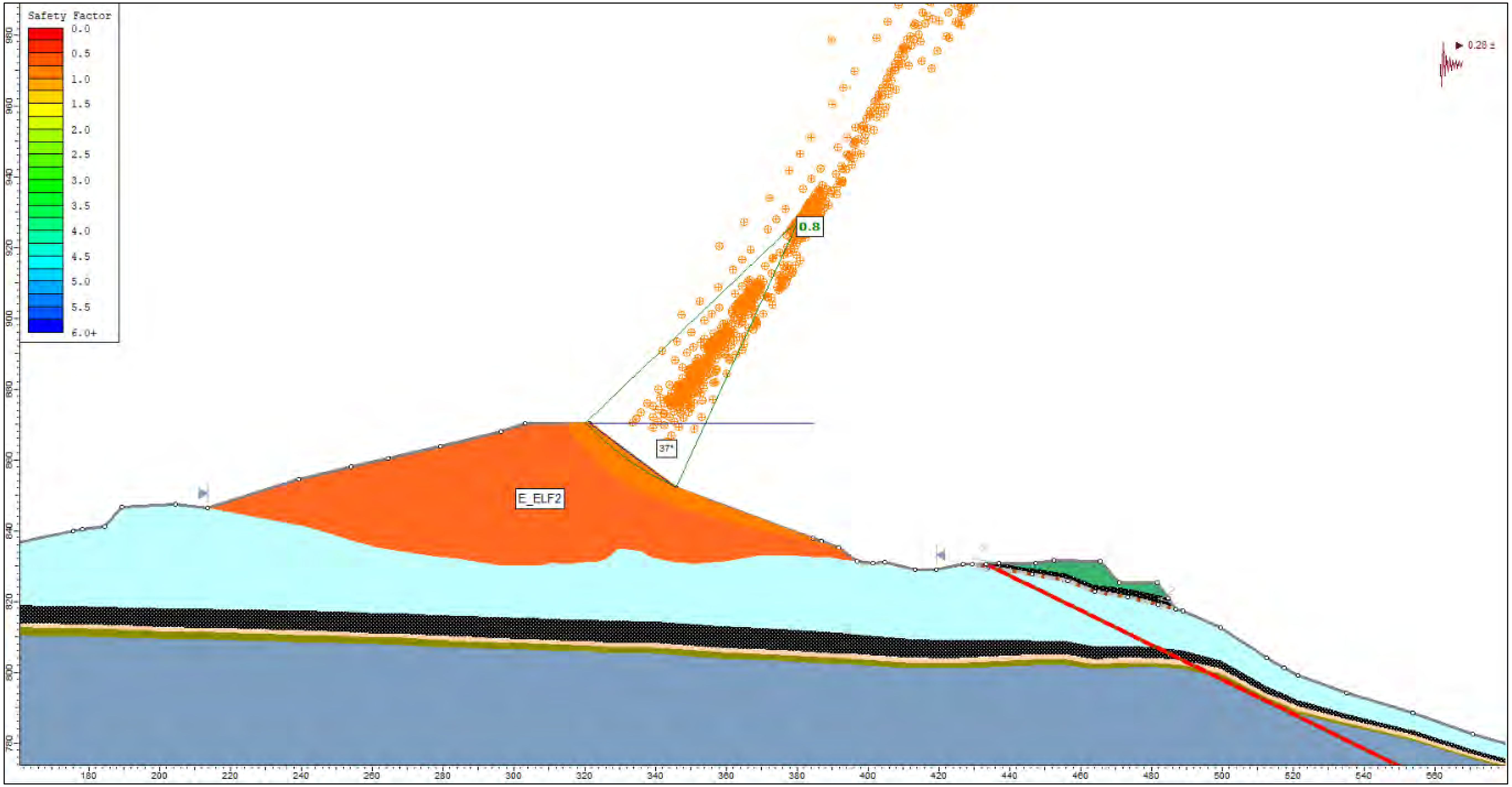

Figure: B3 – Slope Stability Analysis of E_ELF2 – Section 100 – SLS Seismic Conditions.

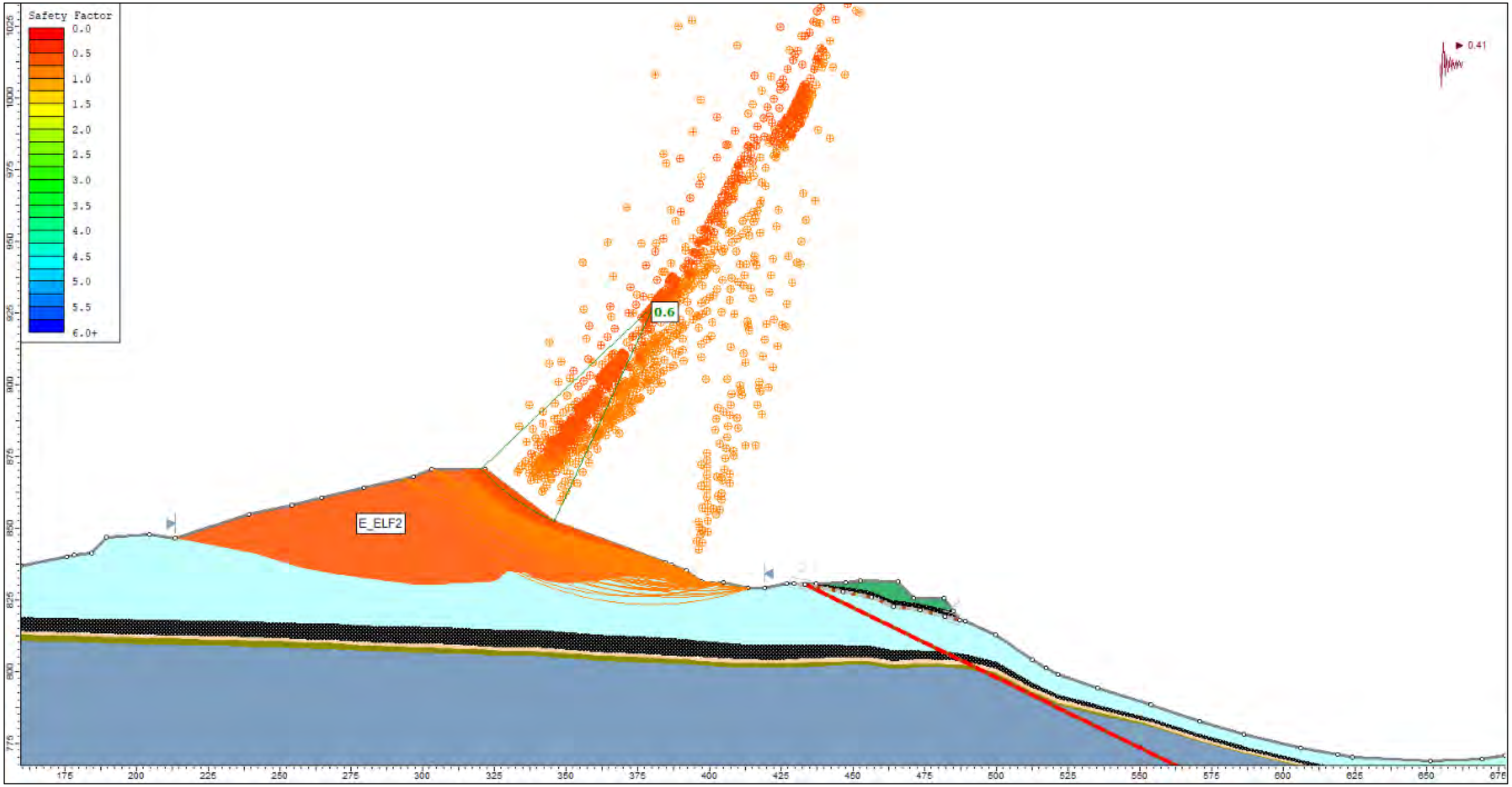

Figure: B4 – Slope Stability Analysis of E_ELF2 – Section 100 –ULS Seismic Conditions.

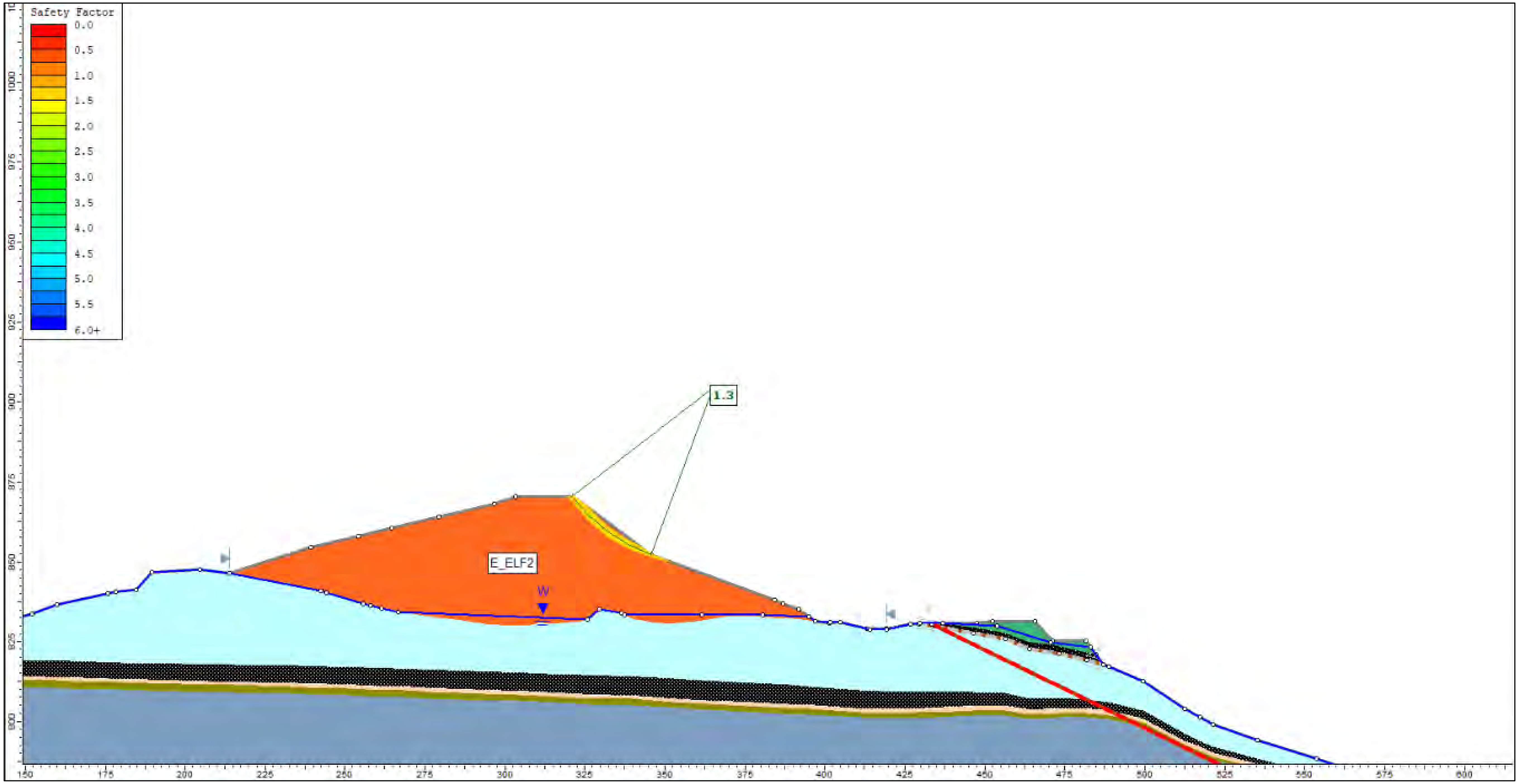

Figure: B5 – Slope Stability Analysis of E_ELF2 – Section 100 – Elevated Groundwater Conditions.

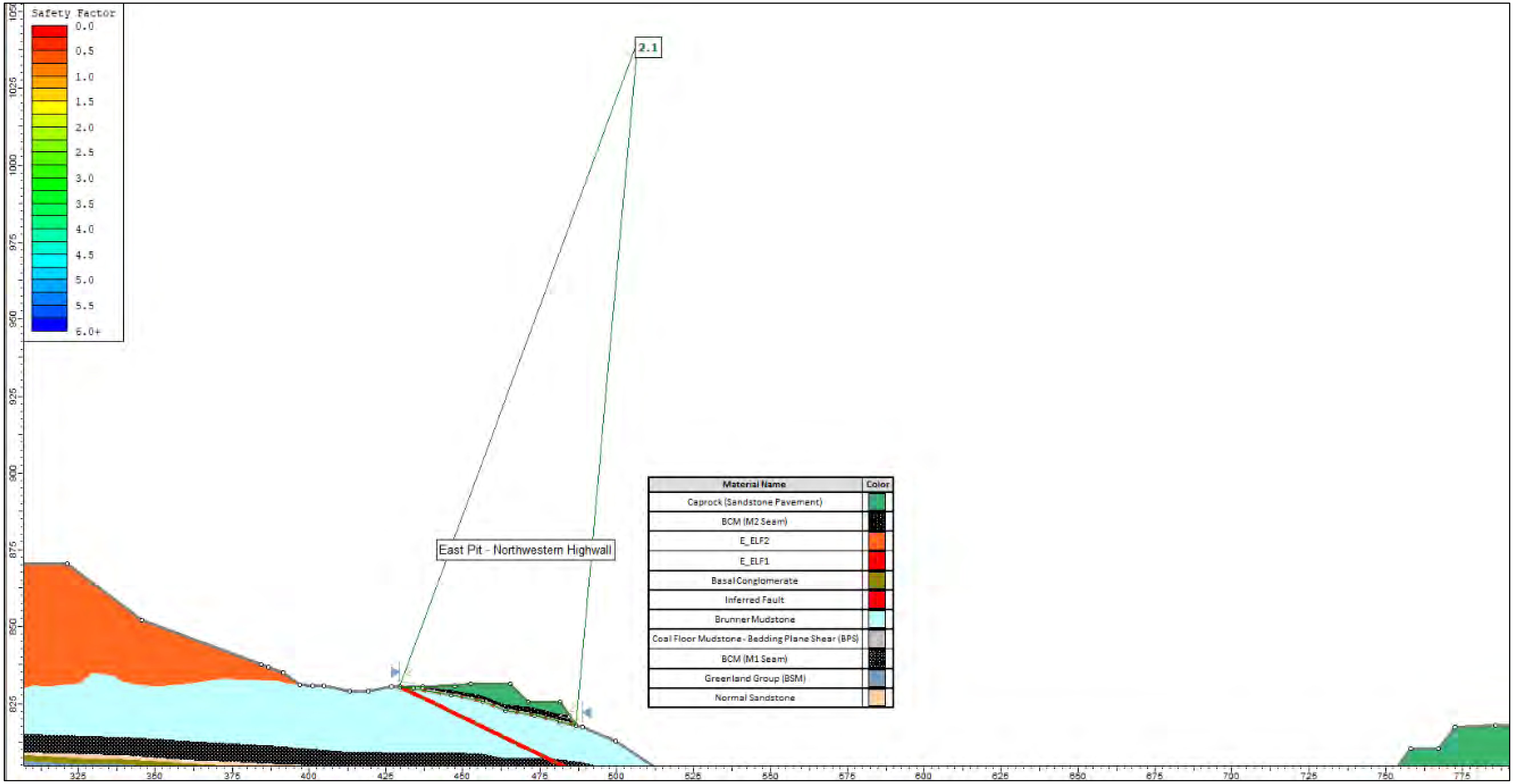

Figure: B6 – Slope Stability Analysis of East Pit (NW) Northwestern Highwall – Section 100 – Static Conditions.

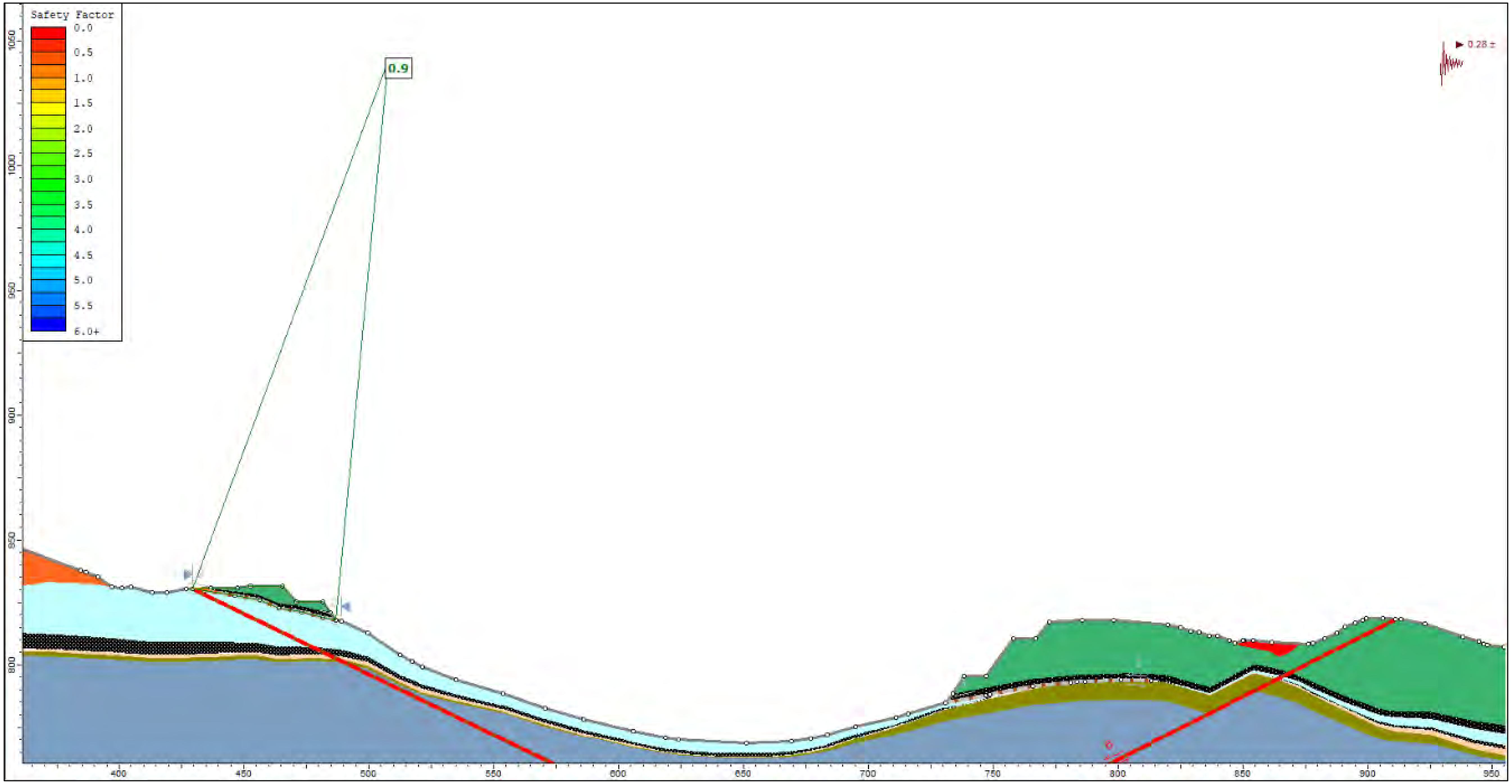

Figure: B7 – Slope Stability Analysis of East Pit (NW) Northwestern Highwall – Section 100 – SLS Conditions.

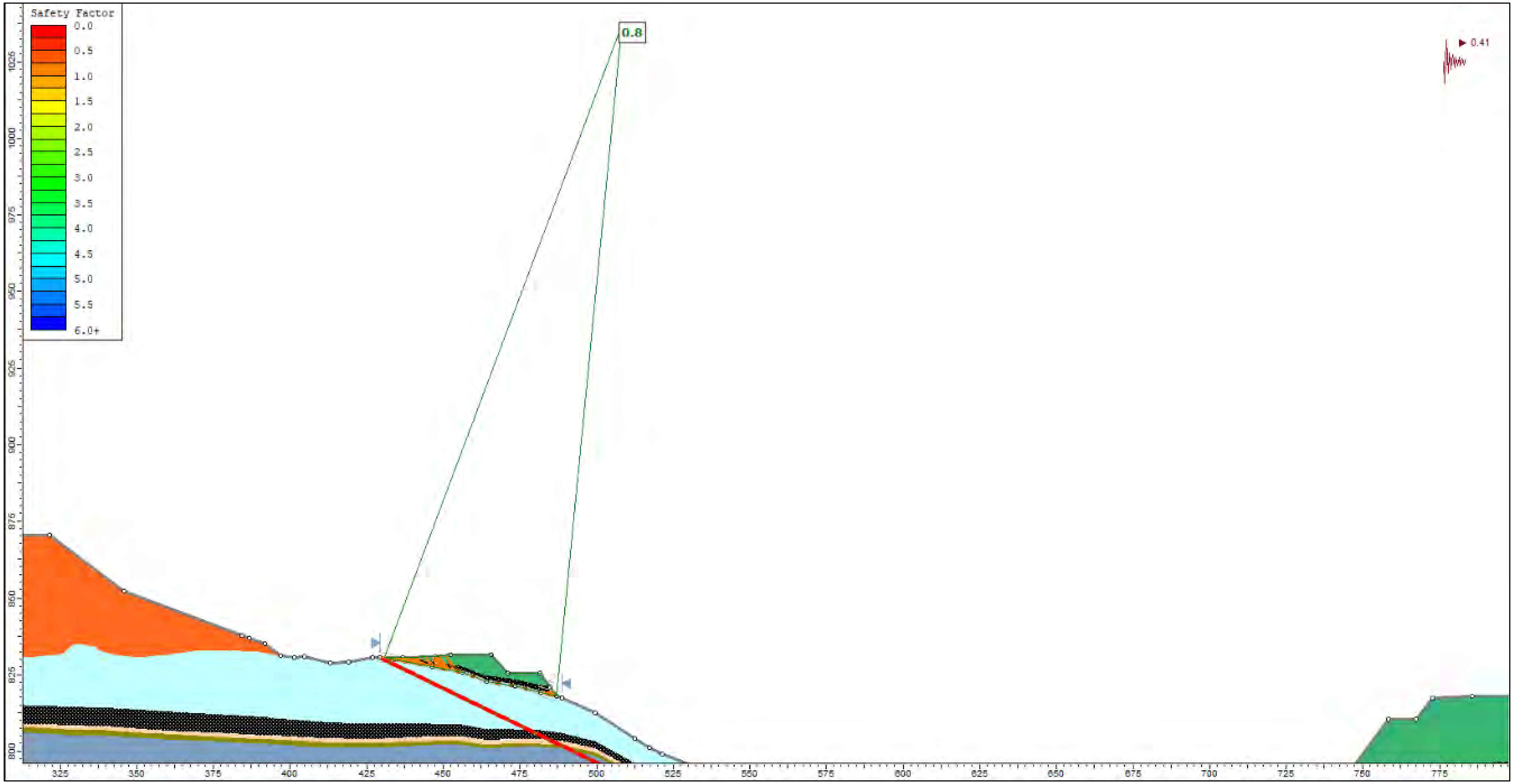

Figure: B8 – Slope Stability Analysis of East Pit (NW) Northwestern Highwall – Section 100 – ULS Conditions.

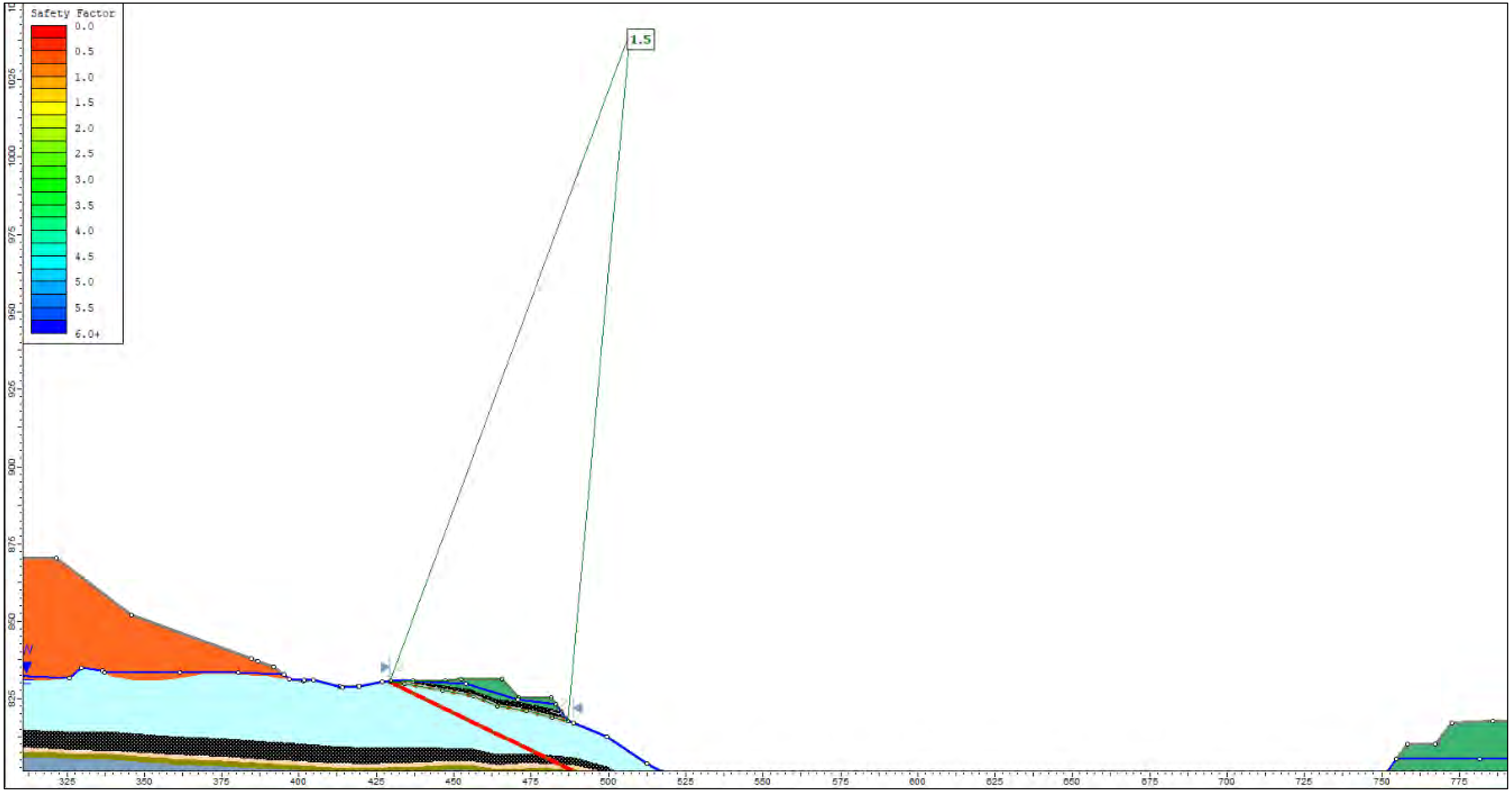

Figure: B9 – Slope Stability Analysis of East Pit (NW) Northwestern Highwall – Section 100 – Elevated Groundwater Conditions.

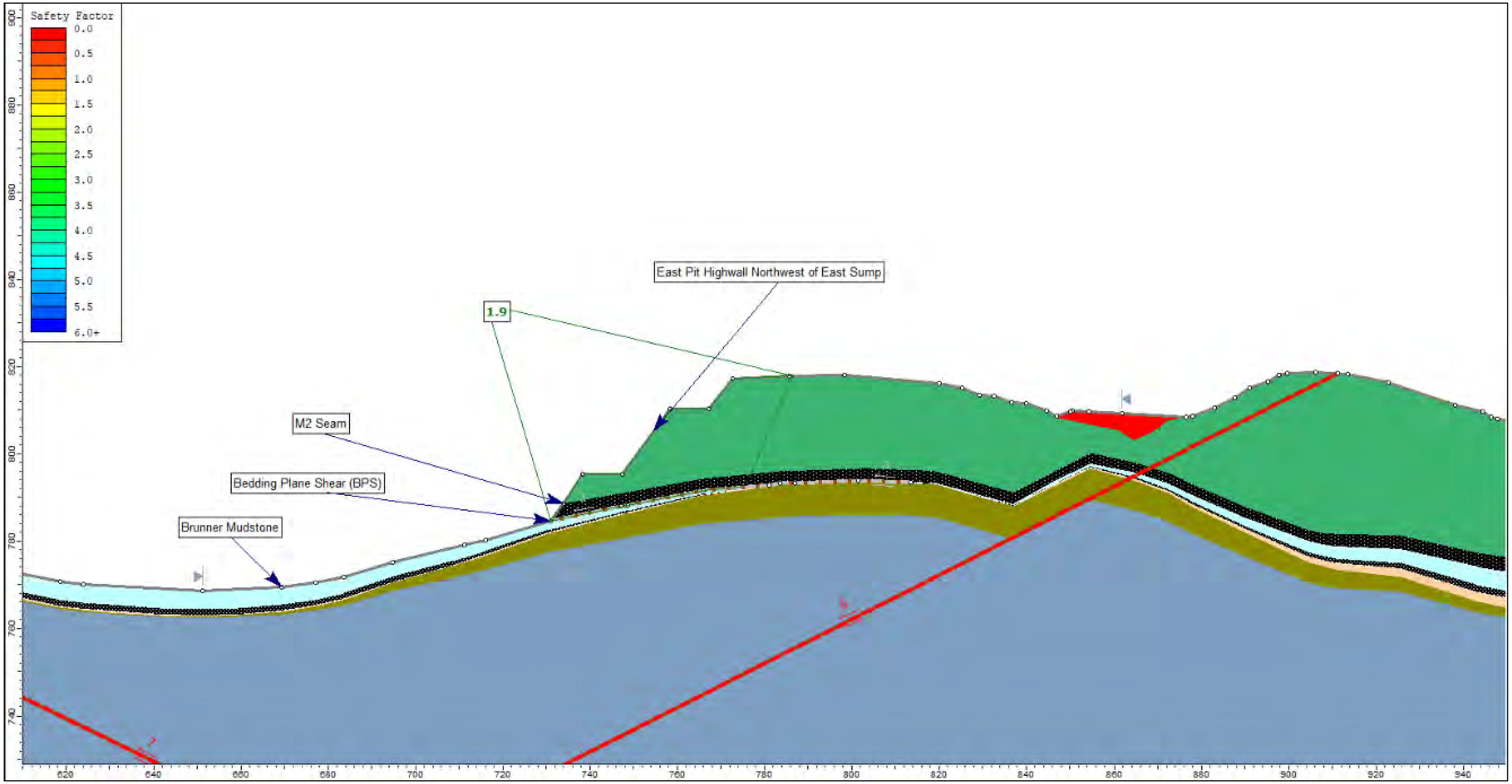

Figure: B10 – Slope Stability Analysis of East Pit (NW) Southwestern Highwall – Section 100 – Static Conditions.

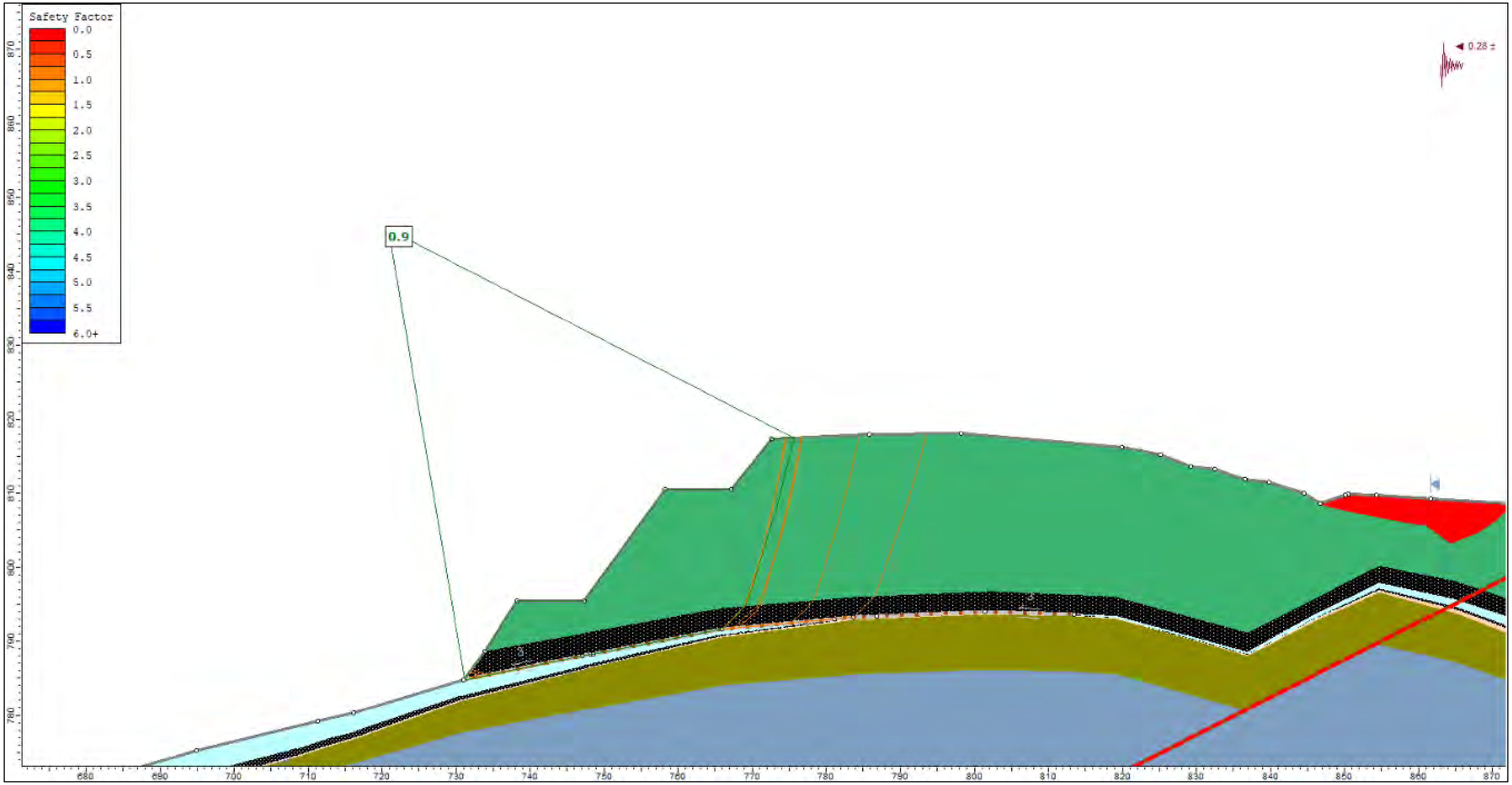

Figure: B11 – Slope Stability Analysis of East Pit (NW) Southwestern Highwall – Section 100 – SLS Conditions.

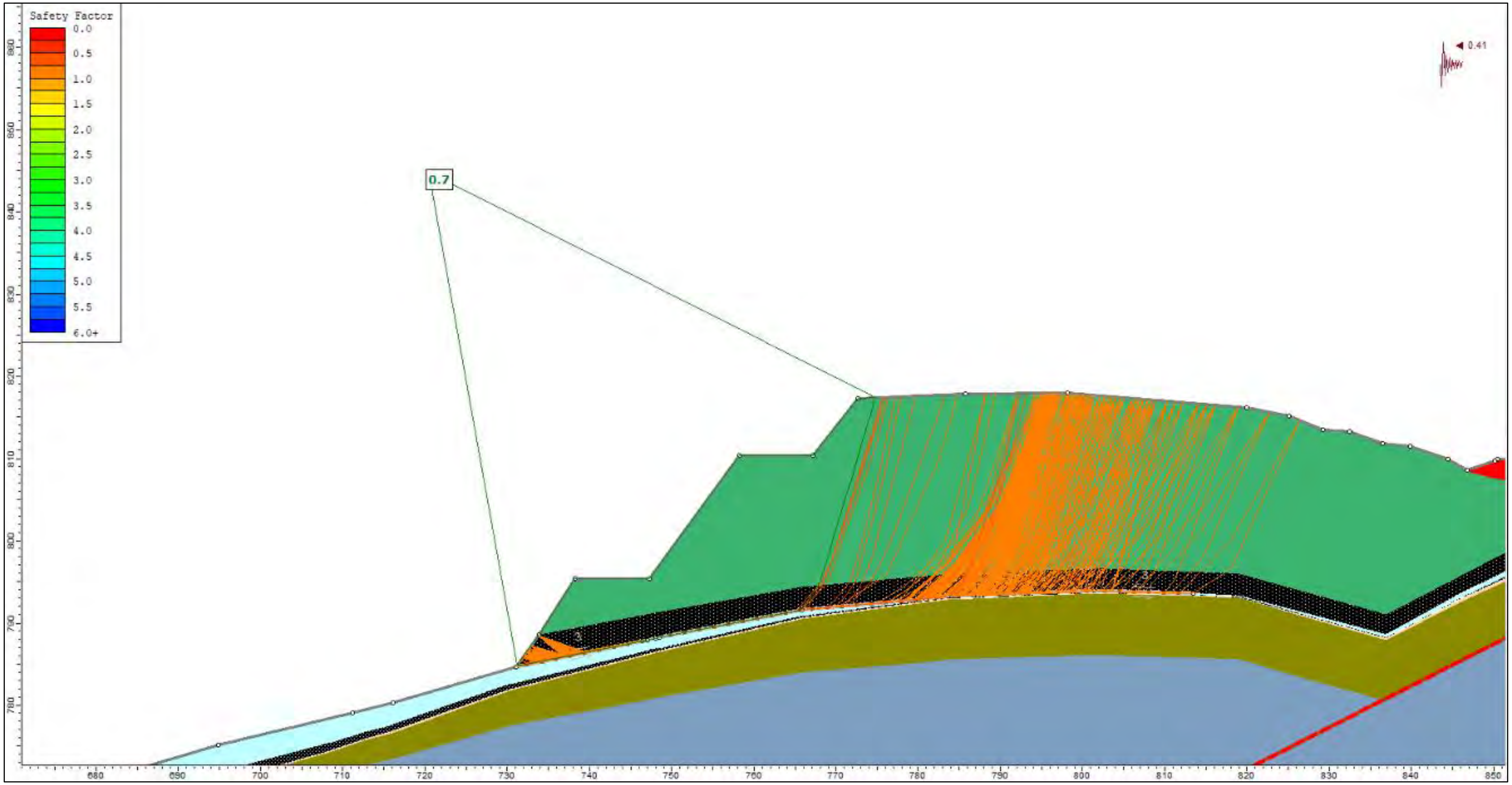

Figure: B12 – Slope Stability Analysis of East Pit (NW) Southwestern Highwall – Section 100 – ULS Conditions.

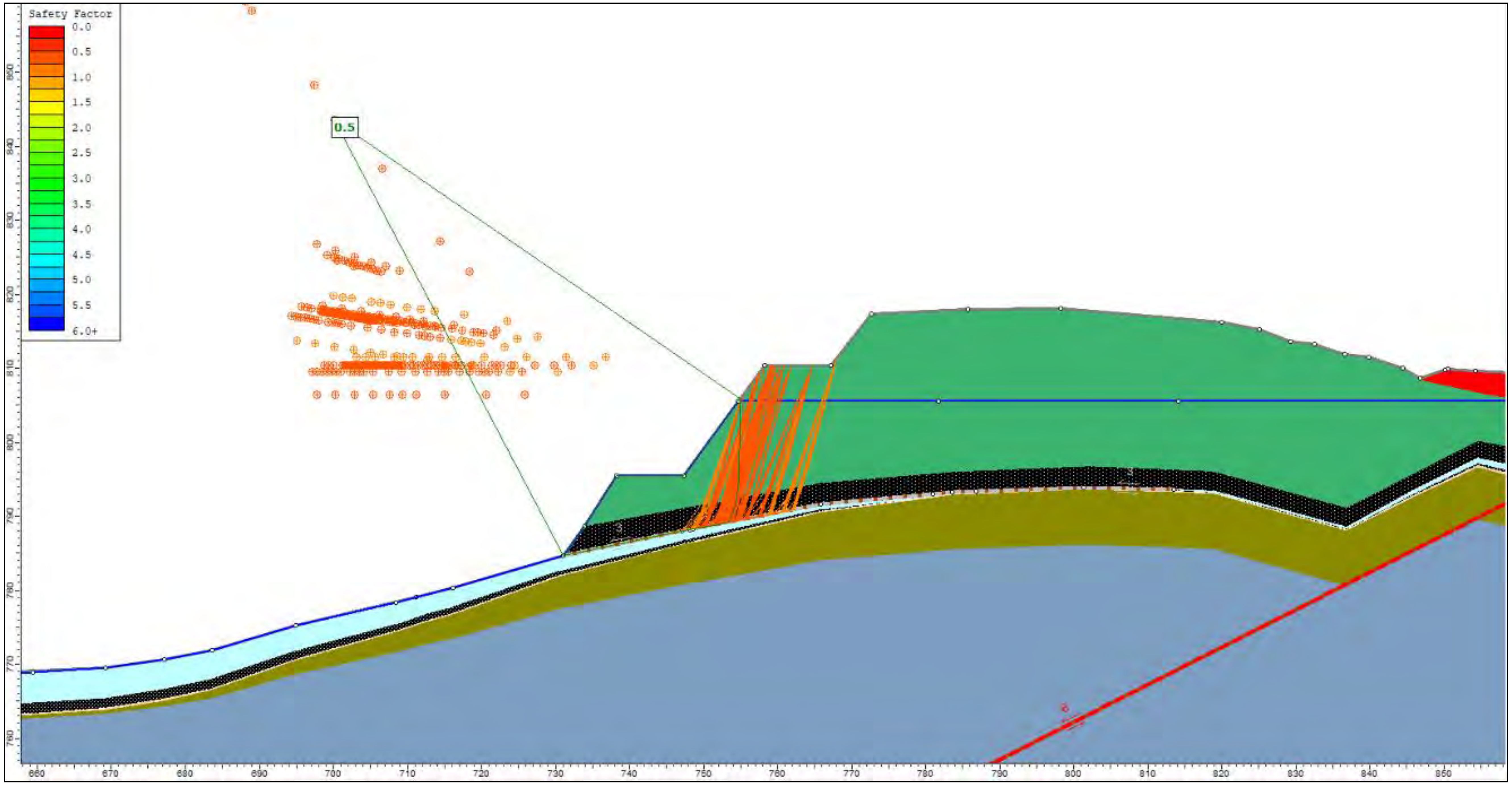

Figure: B13– Slope Stability Analysis of East Pit (NW) Southwestern Highwall – Section 100 – Elevated Groundwater Conditions.

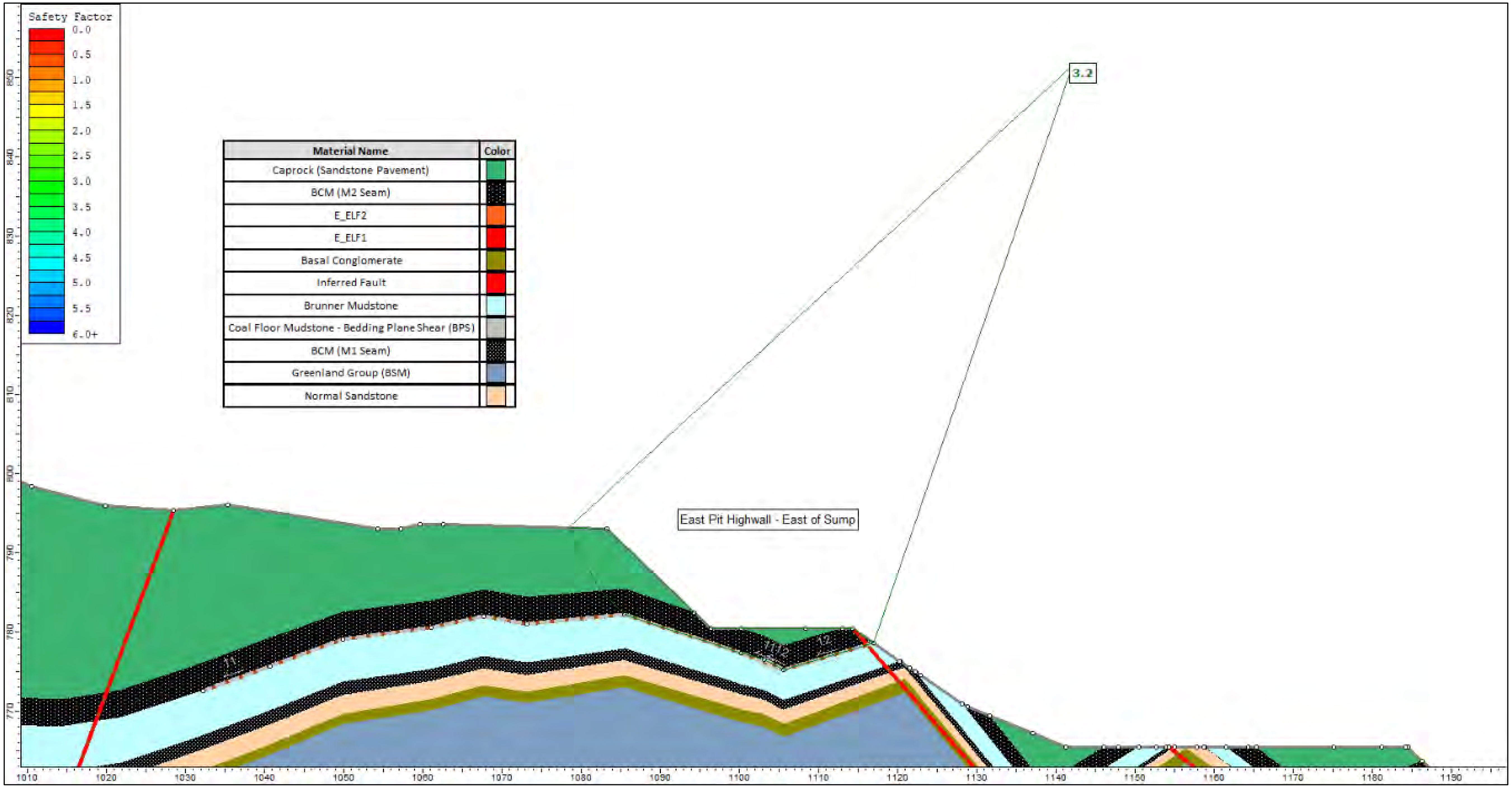

Figure: B14 – Slope Stability Analysis of East Pit (SW) Northwestern Highwall – Section 100 – Static Conditions.

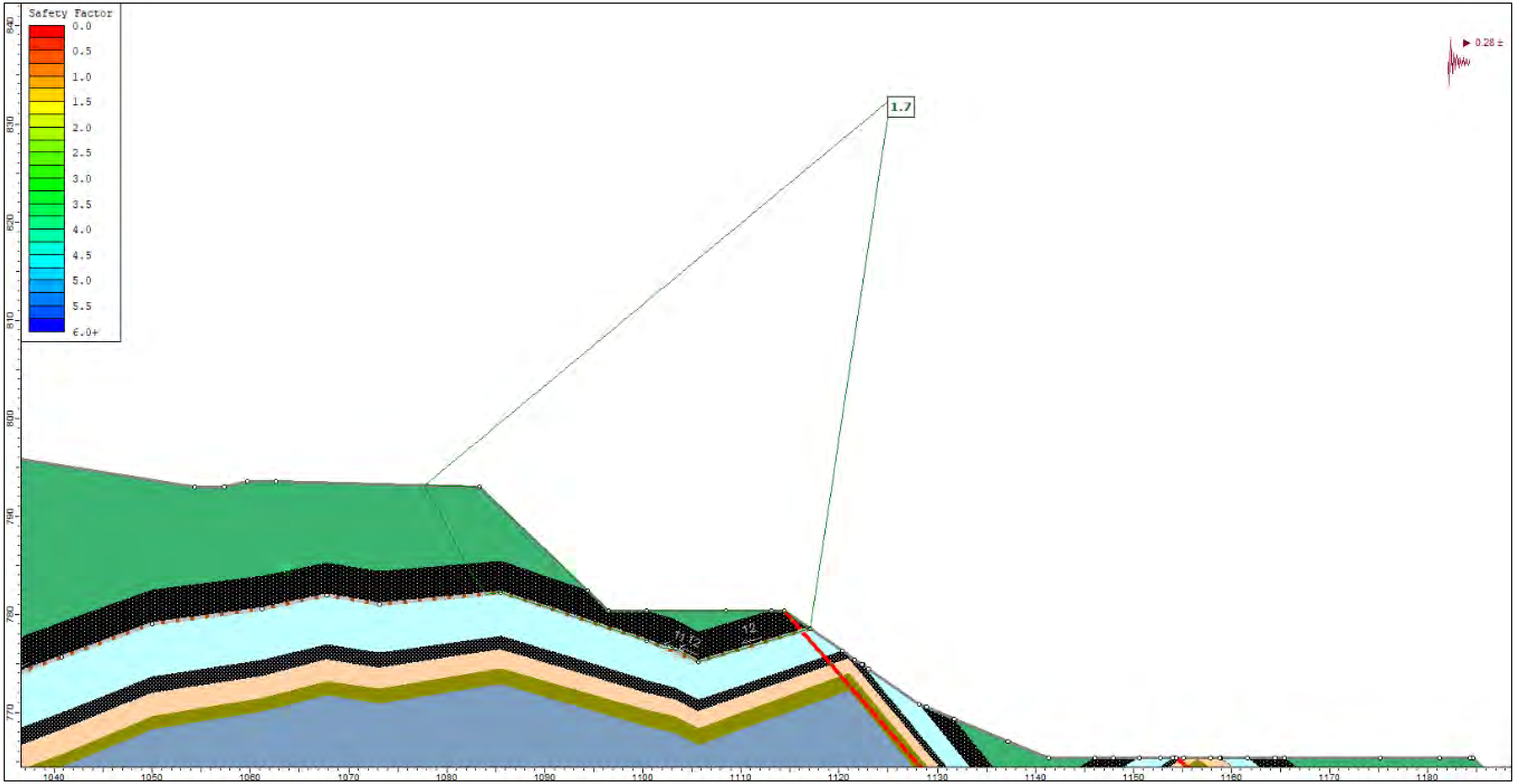

Figure: B15 – Slope Stability Analysis of East Pit (SW) Northwestern Highwall – Section 100 – SLS Conditions.

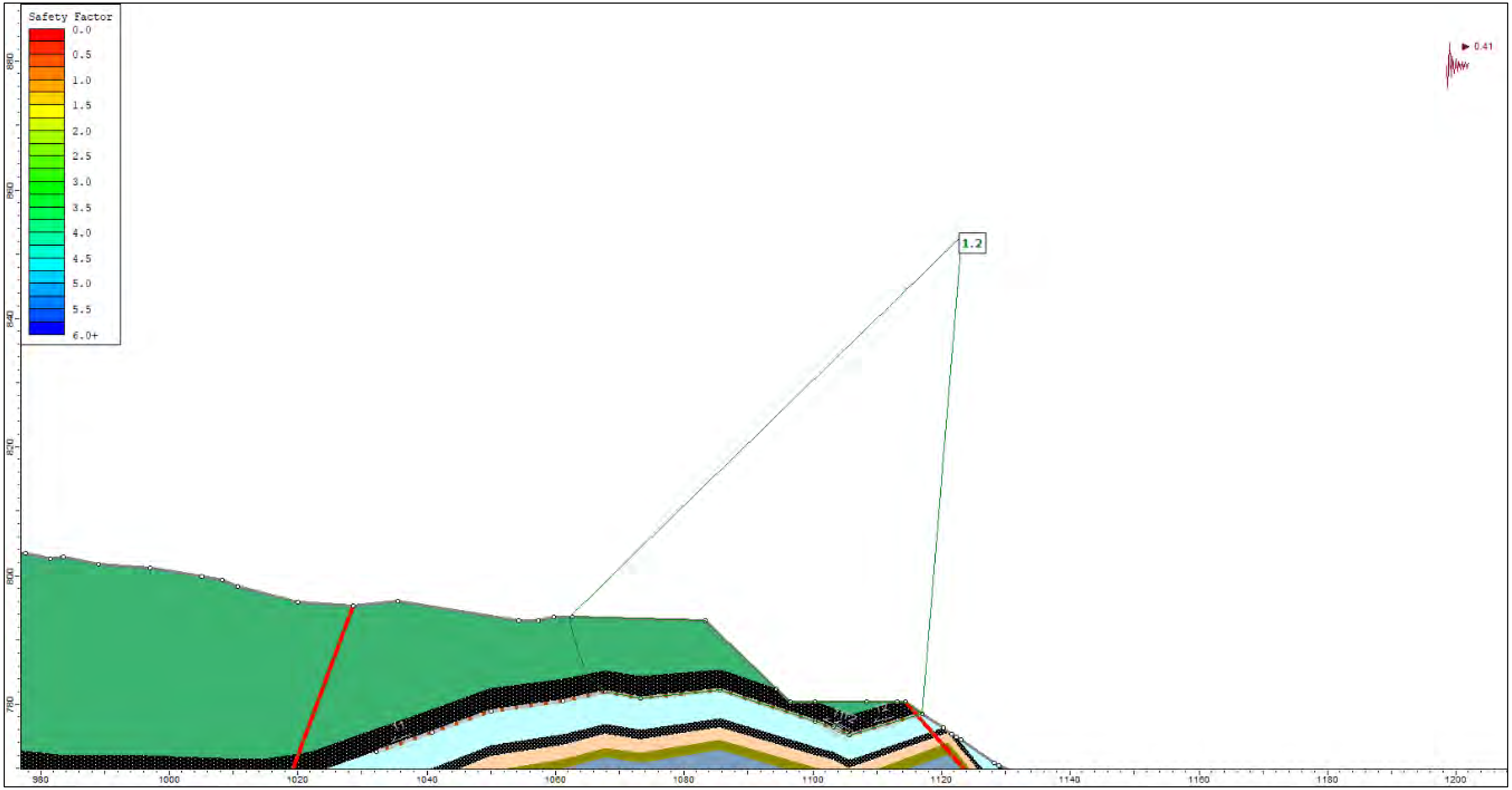

Figure: B16 – Slope Stability Analysis of East Pit (SW) Northwestern Highwall – Section 100 – ULS Conditions.

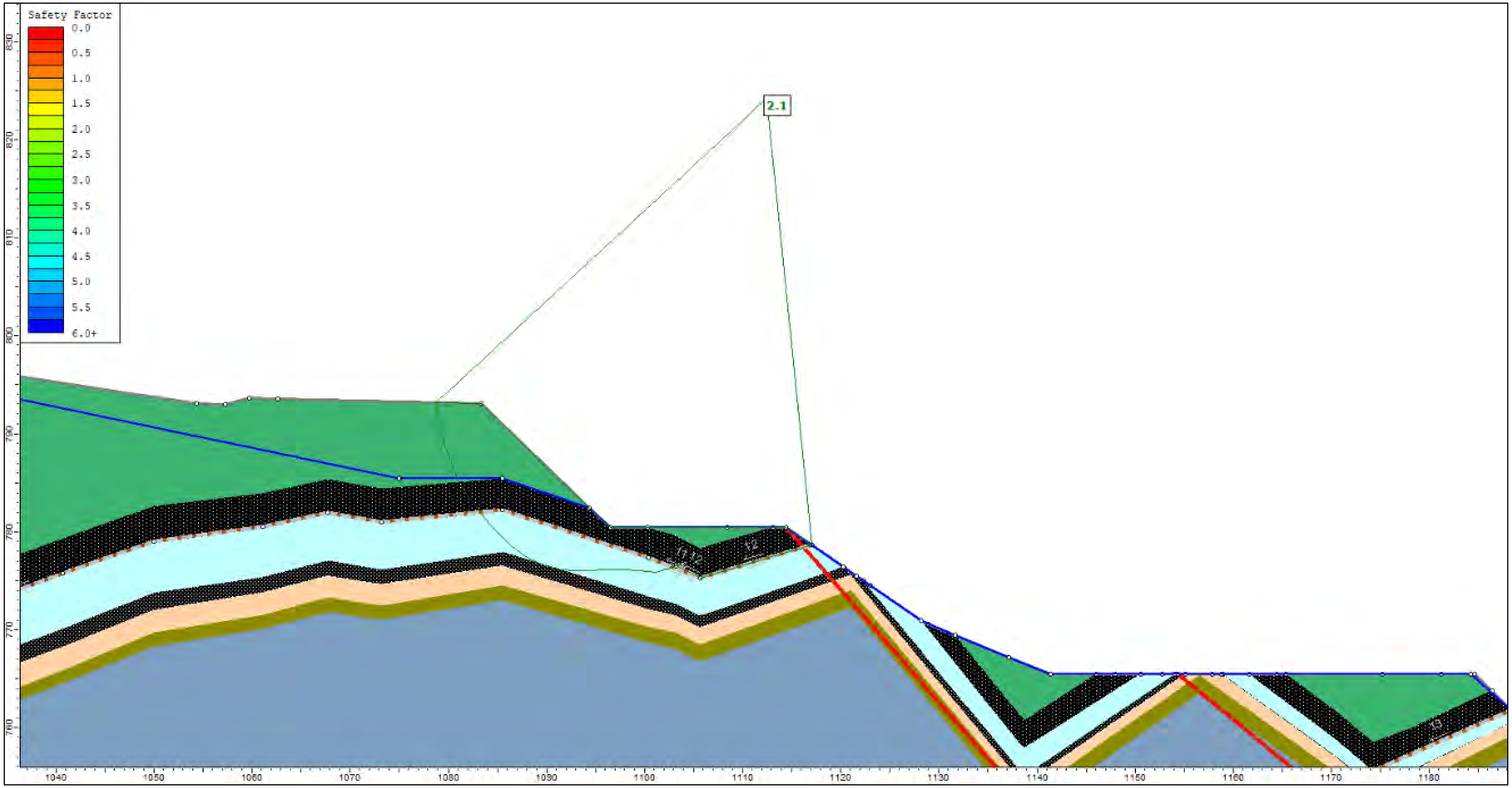

Figure: B17 – Slope Stability Analysis of East Pit (SW) Northwestern Highwall – Section 100 – Elevated Groundwater Conditions.

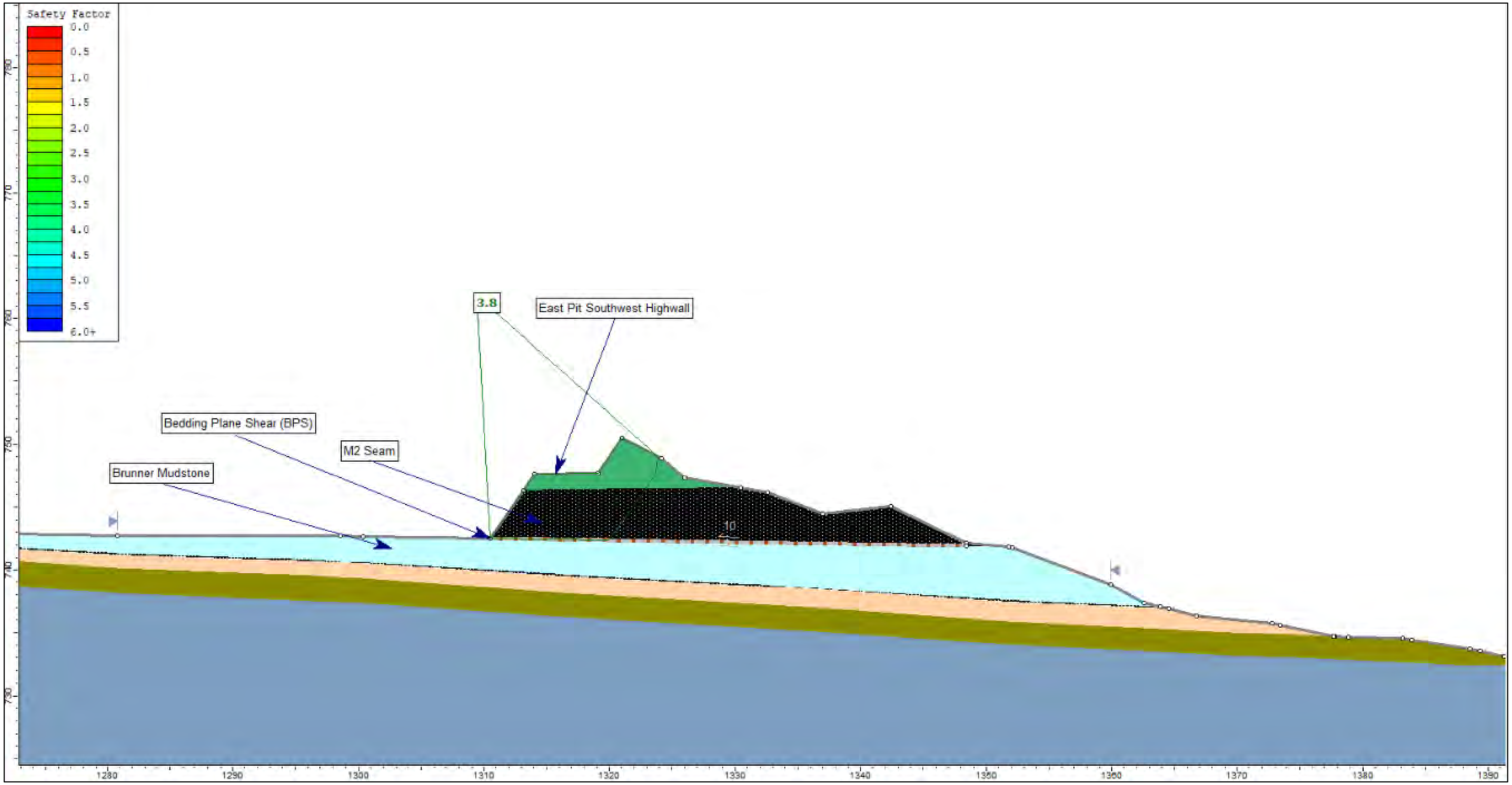

Figure: B18 – Slope Stability Analysis of East Pit (SW) Southwestern Highwall – Section 100 – Static Conditions.

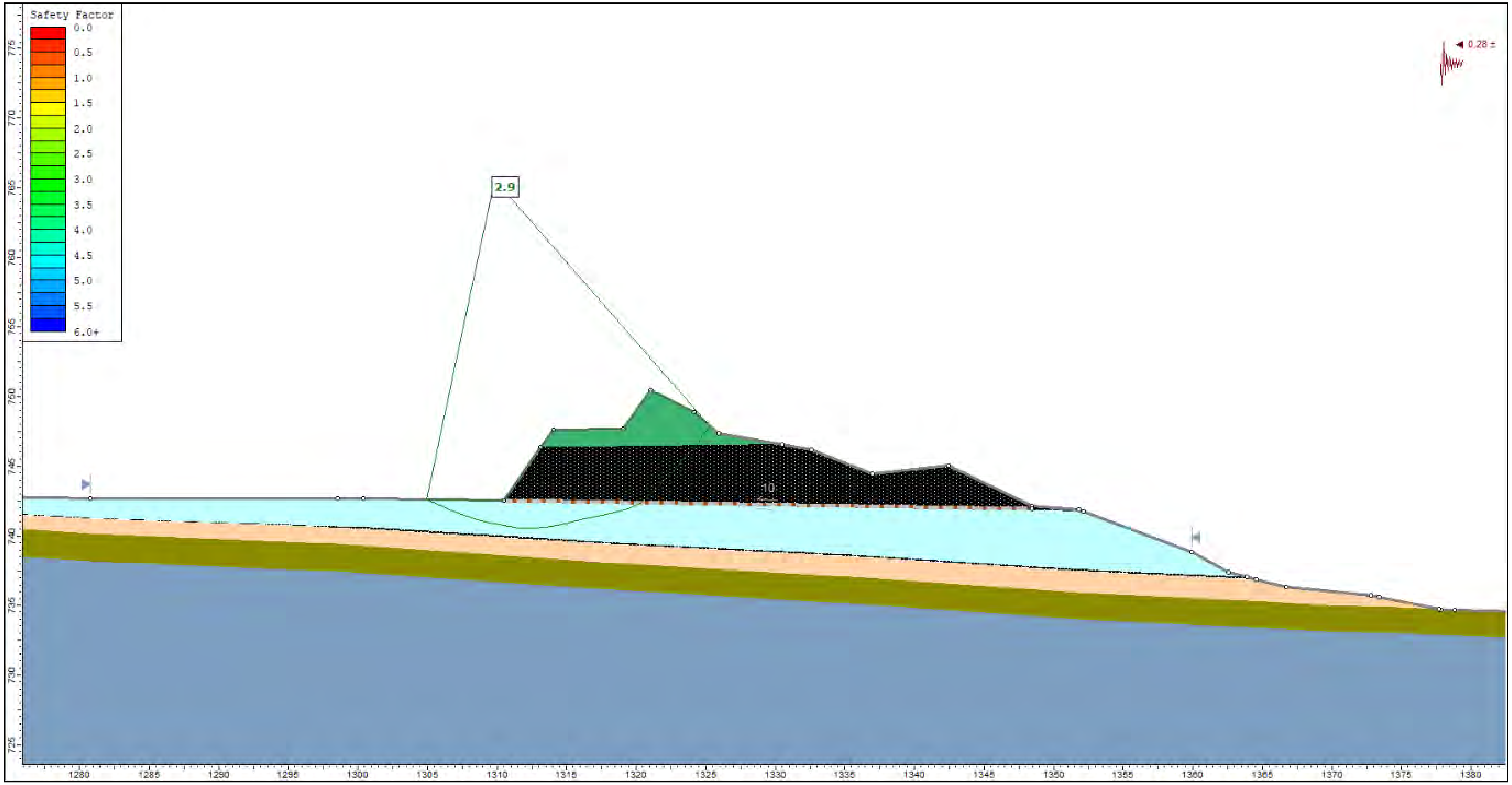

Figure: B19 – Slope Stability Analysis of East Pit (SW) Southwestern Highwall – Section 100 – SLS Conditions.

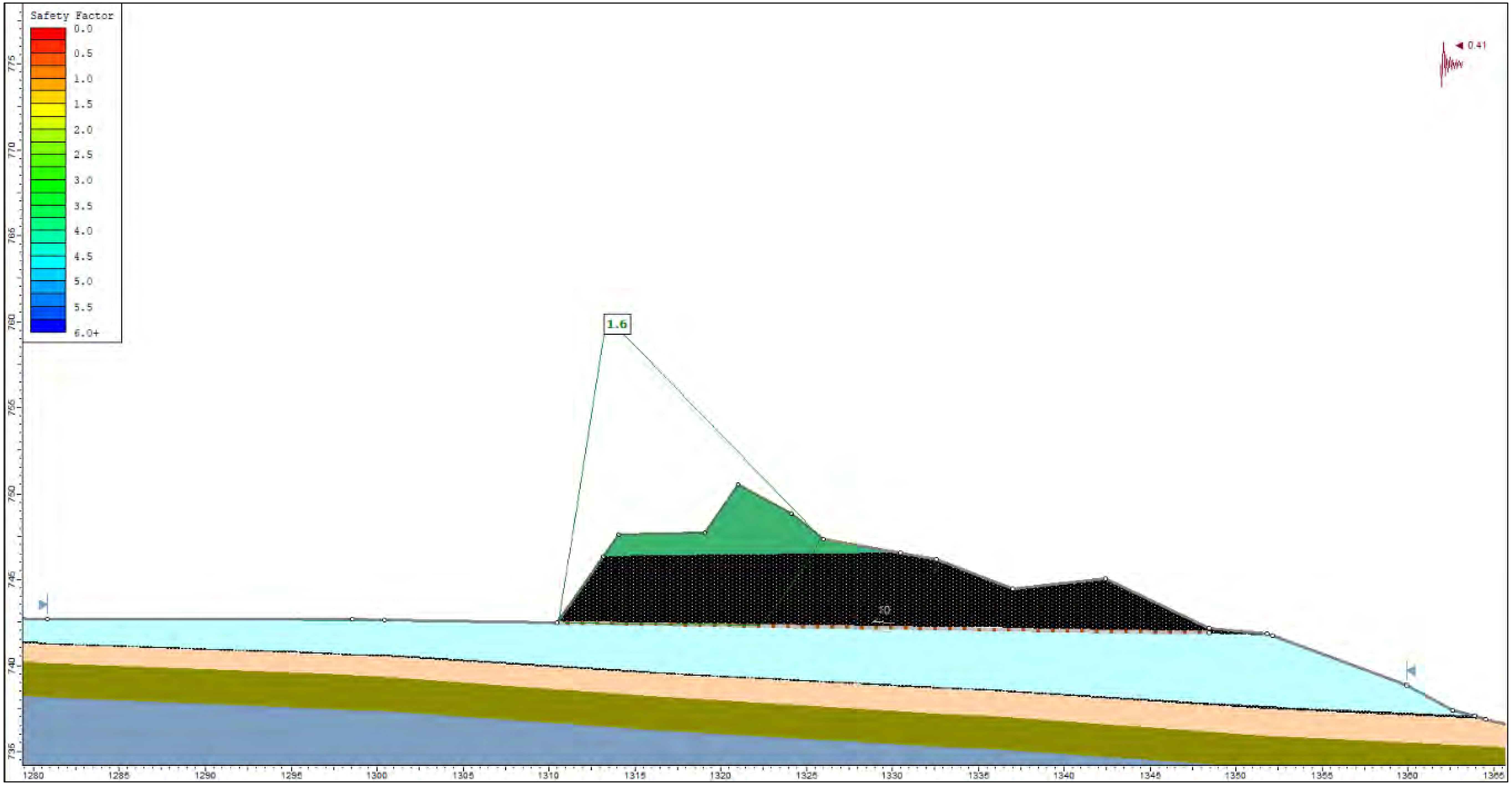

Figure: B20 – Slope Stability Analysis of East Pit (SW) Southwestern Highwall – Section 100 – ULS Conditions.

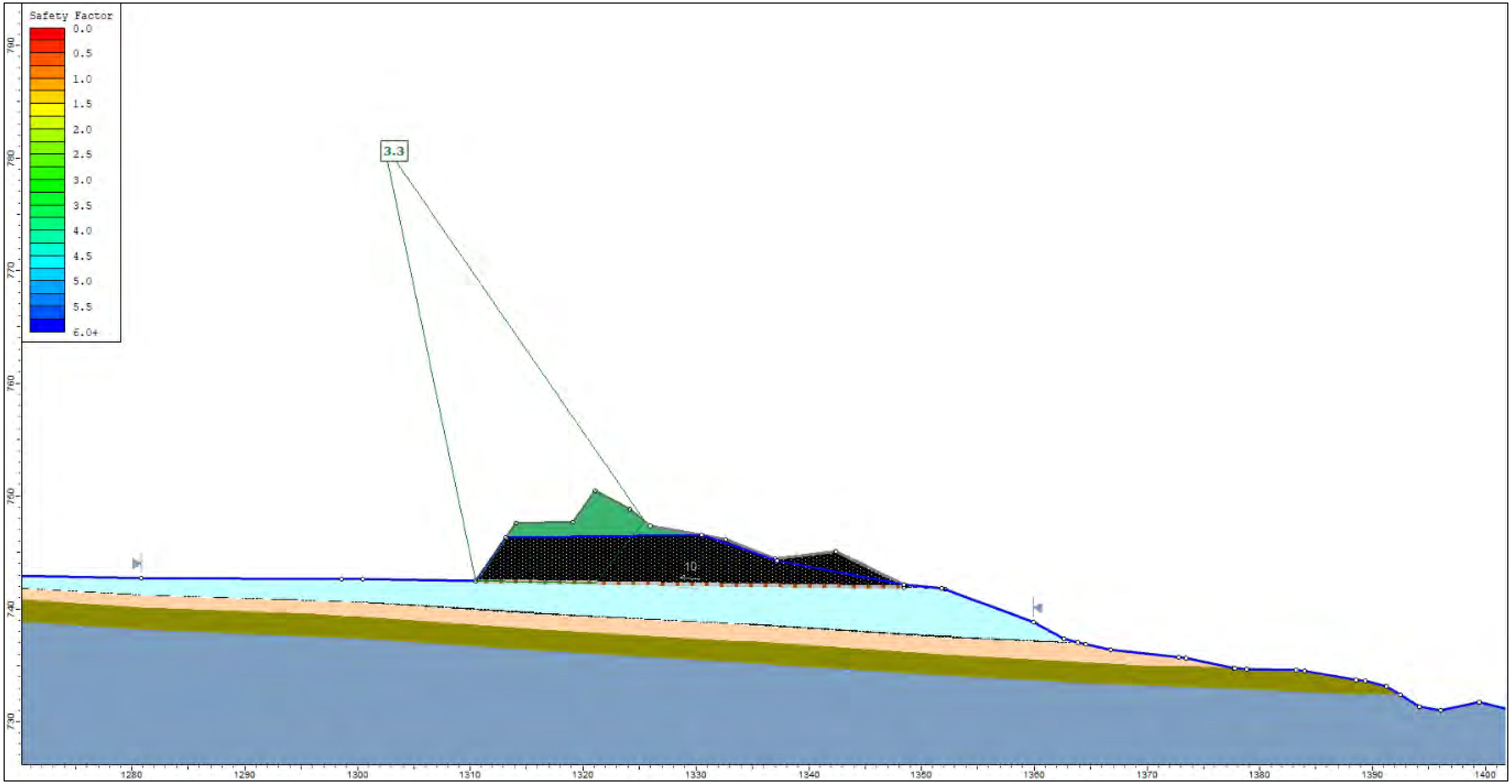

Figure: B21 – Slope Stability Analysis of East Pit (SW) Southwestern Highwall – Section 100 – Elevated Groundwater Conditions.

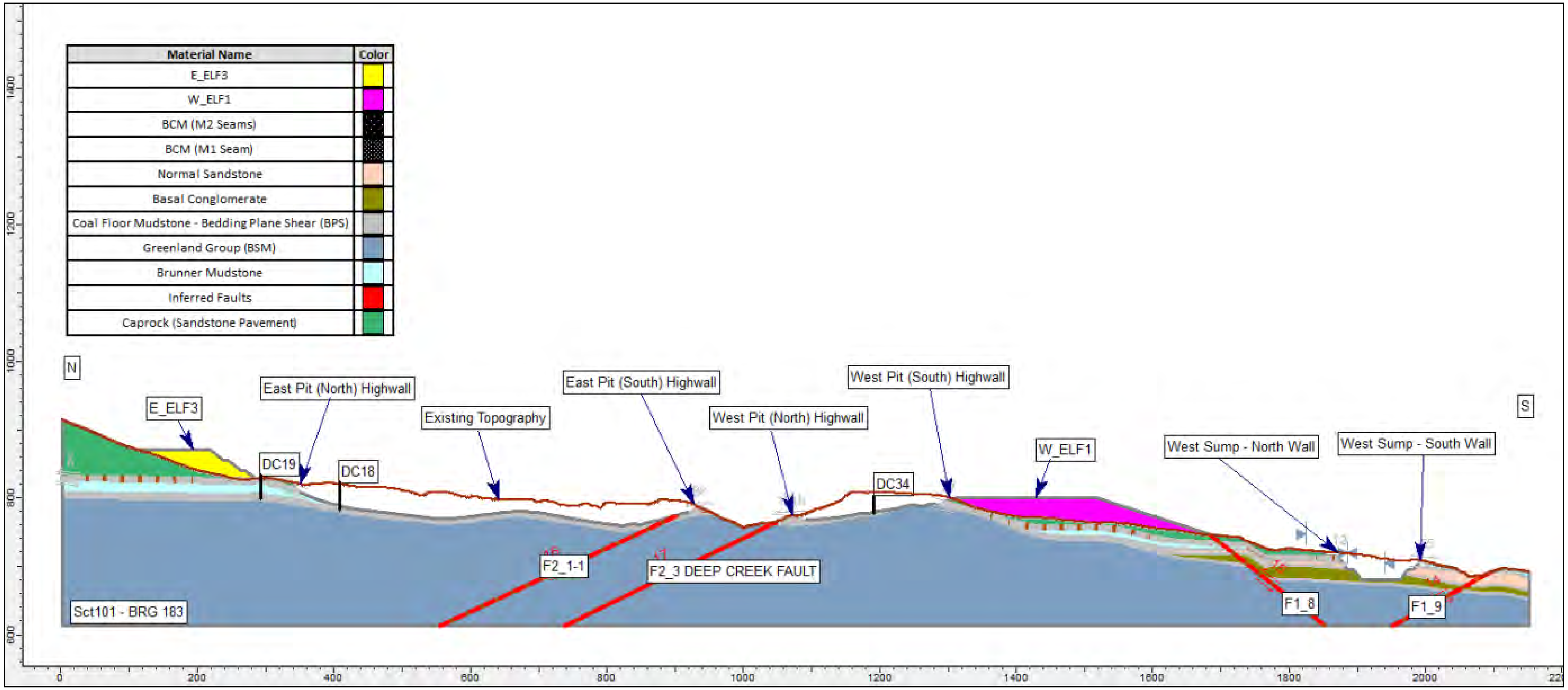

Figure B22 - Section 101 – Mine Feature Overview

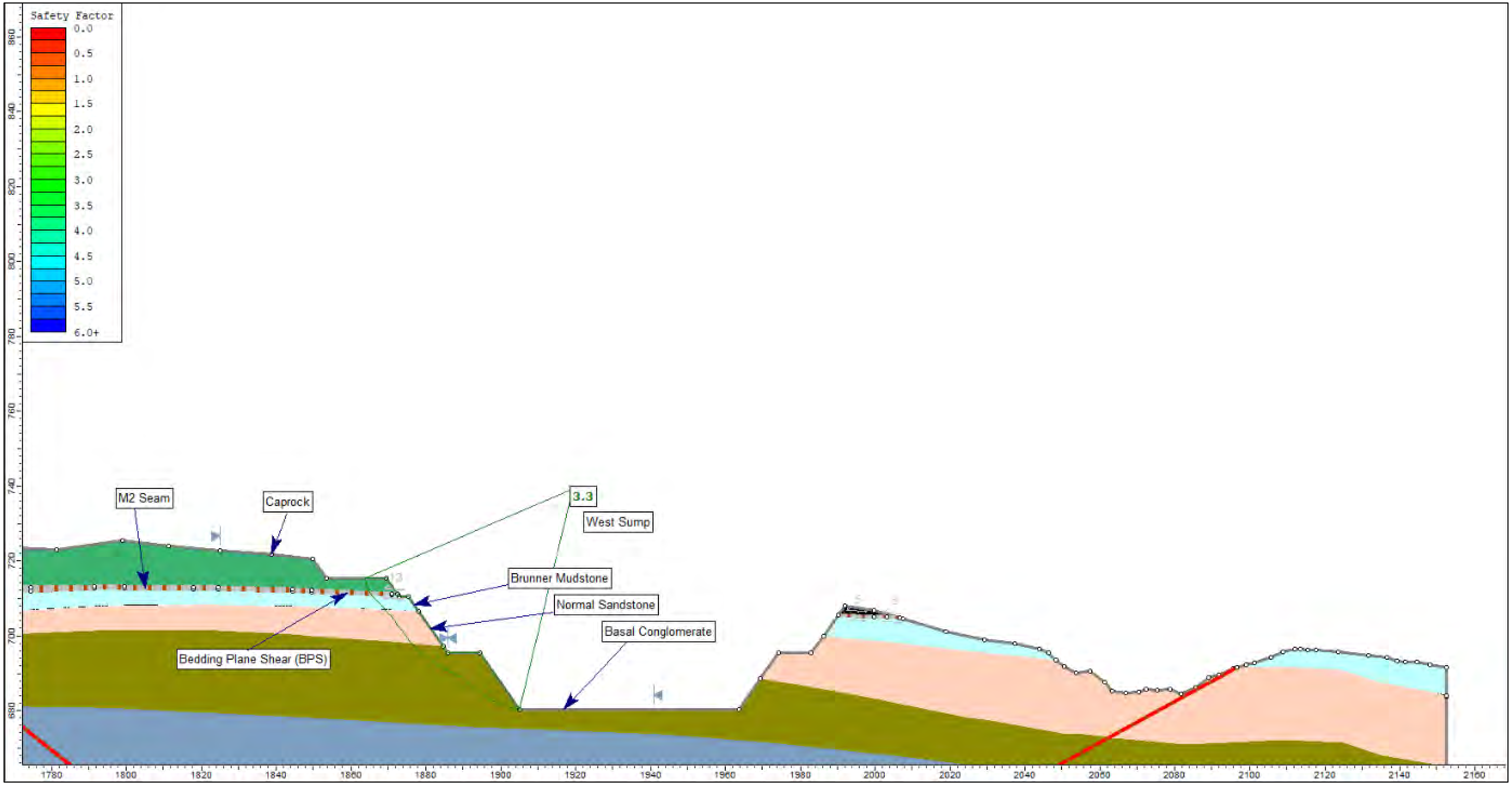

Figure: B23 – Slope Stability Analysis of West Sump Highwall (North wall) – Section 101 – Static Conditions.

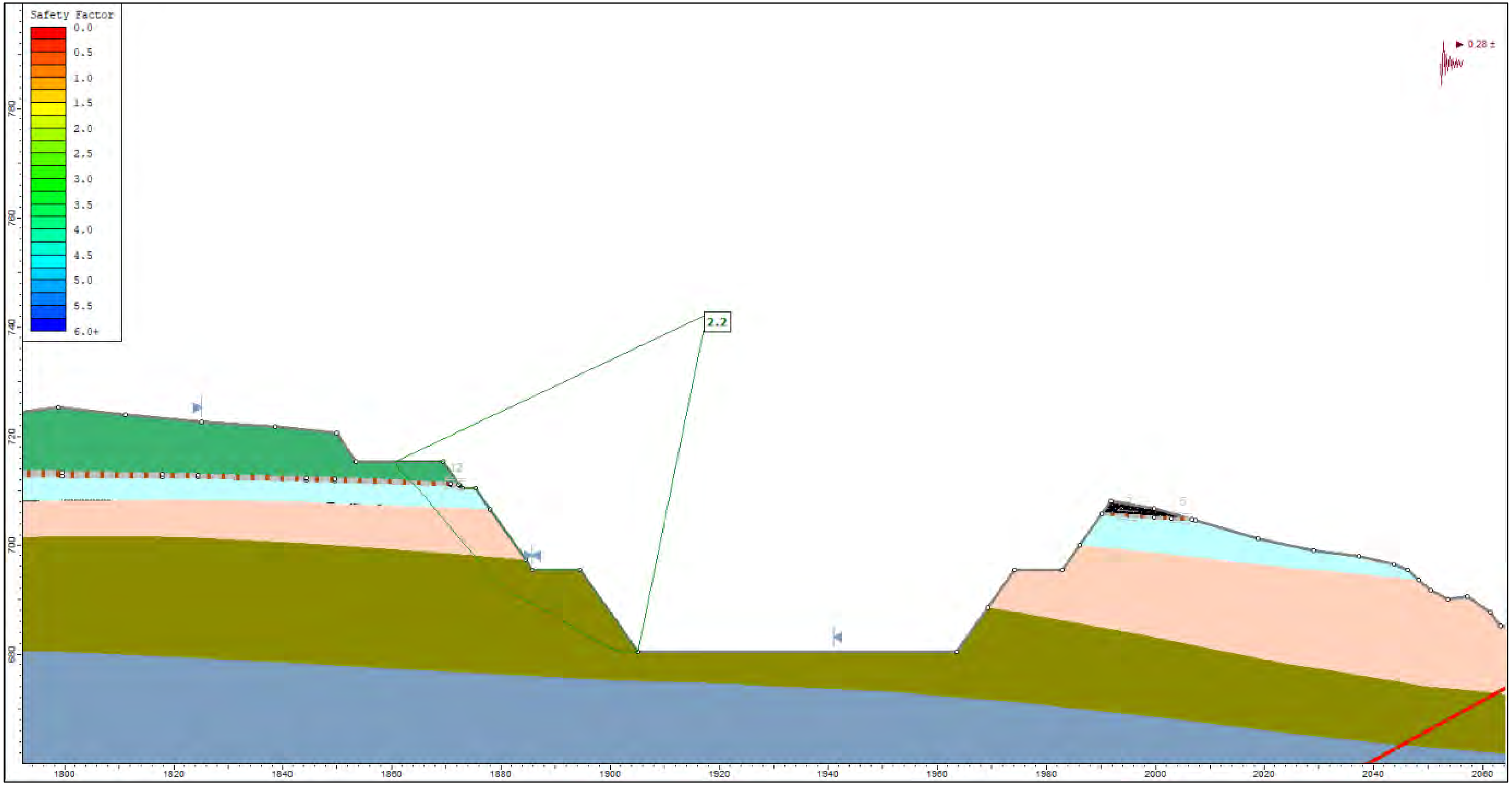

Figure: B24 – Slope Stability Analysis of West Sump Highwall (North wall) – Section 101 – SLS Conditions.

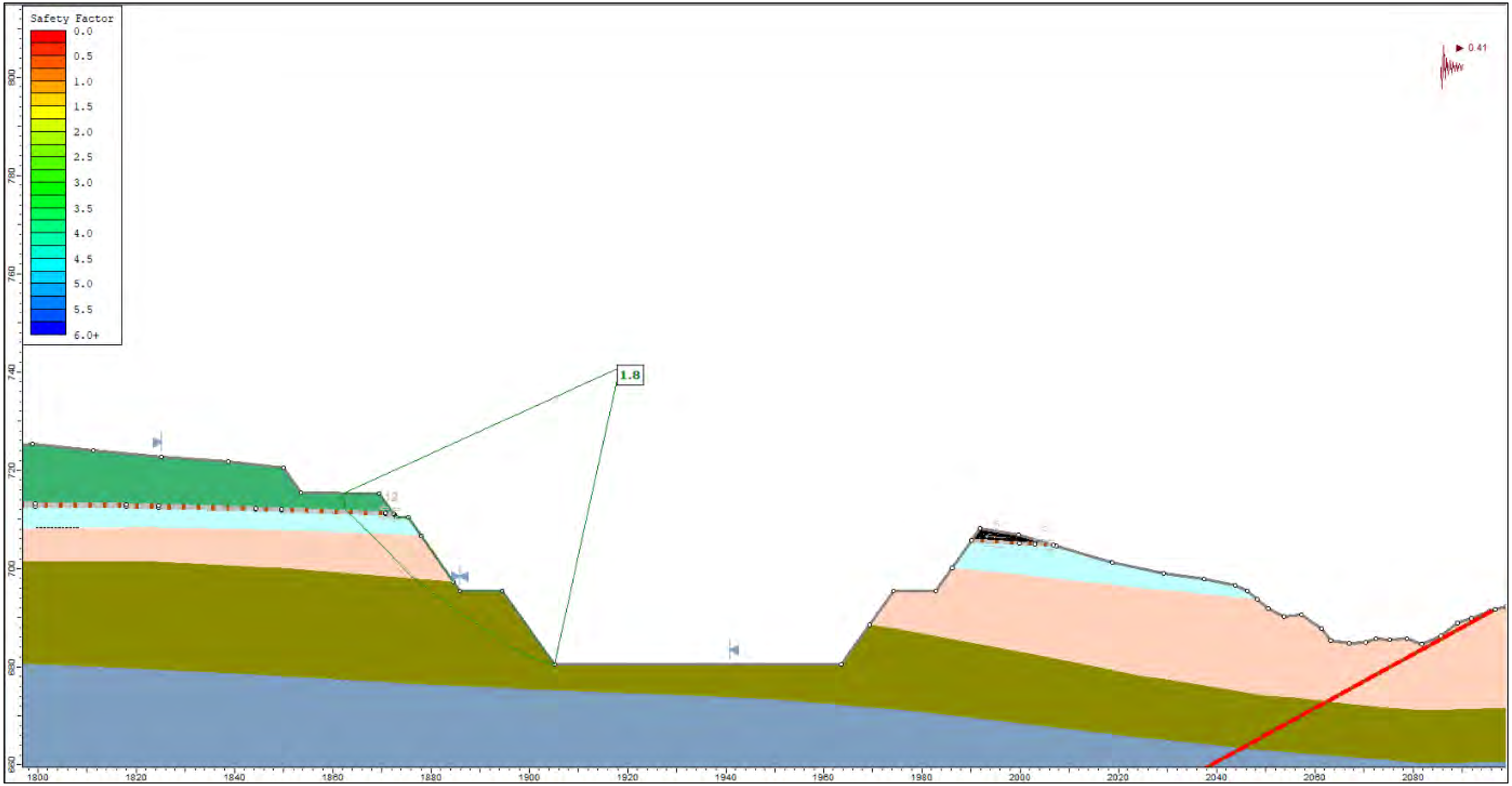

Figure: B25 – Slope Stability Analysis of West Sump Highwall (North wall) – Section 101 – ULS Conditions.

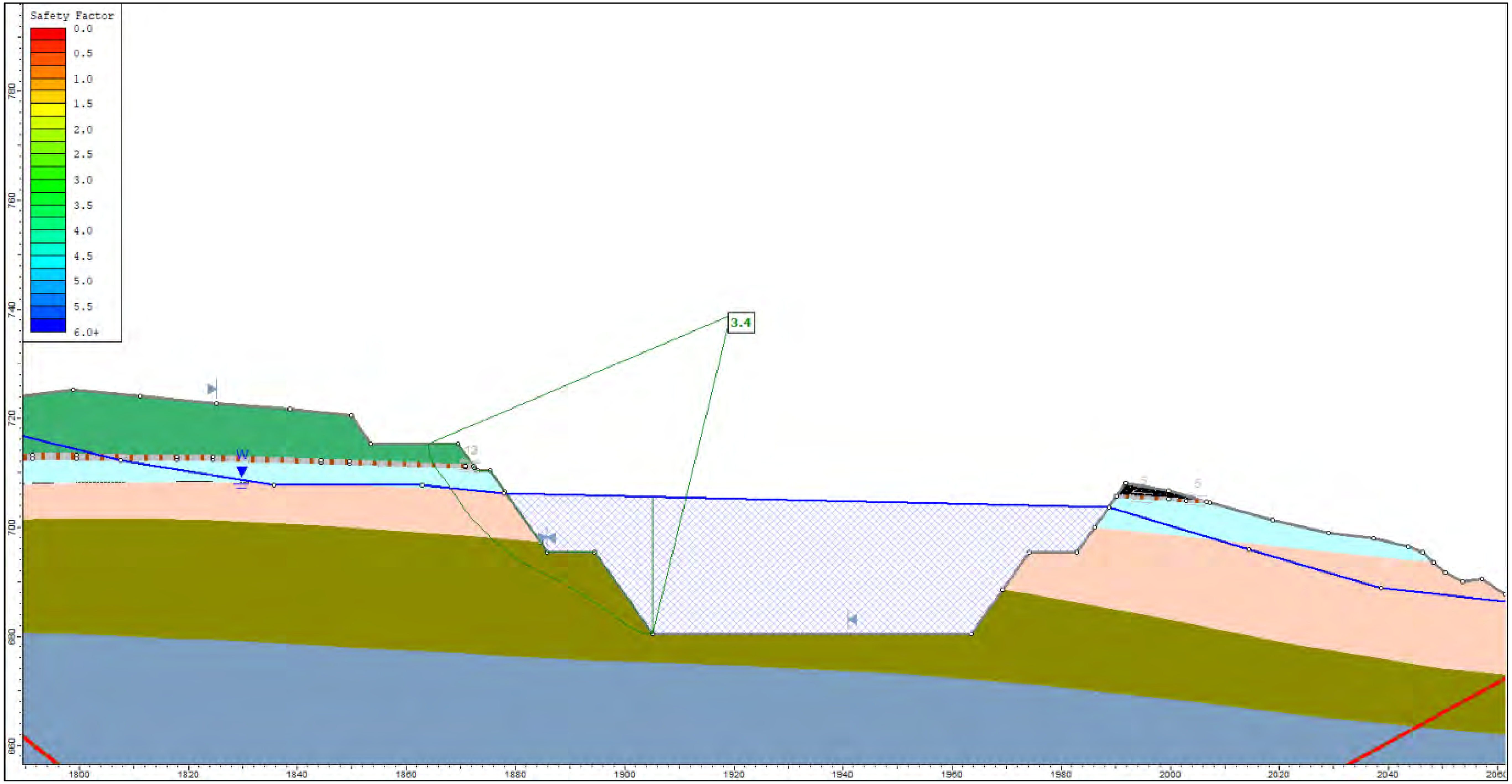

Figure: B26– Slope Stability Analysis of West Sump Highwall (North wall) – Section 101 – Elevated Groundwater Conditions.

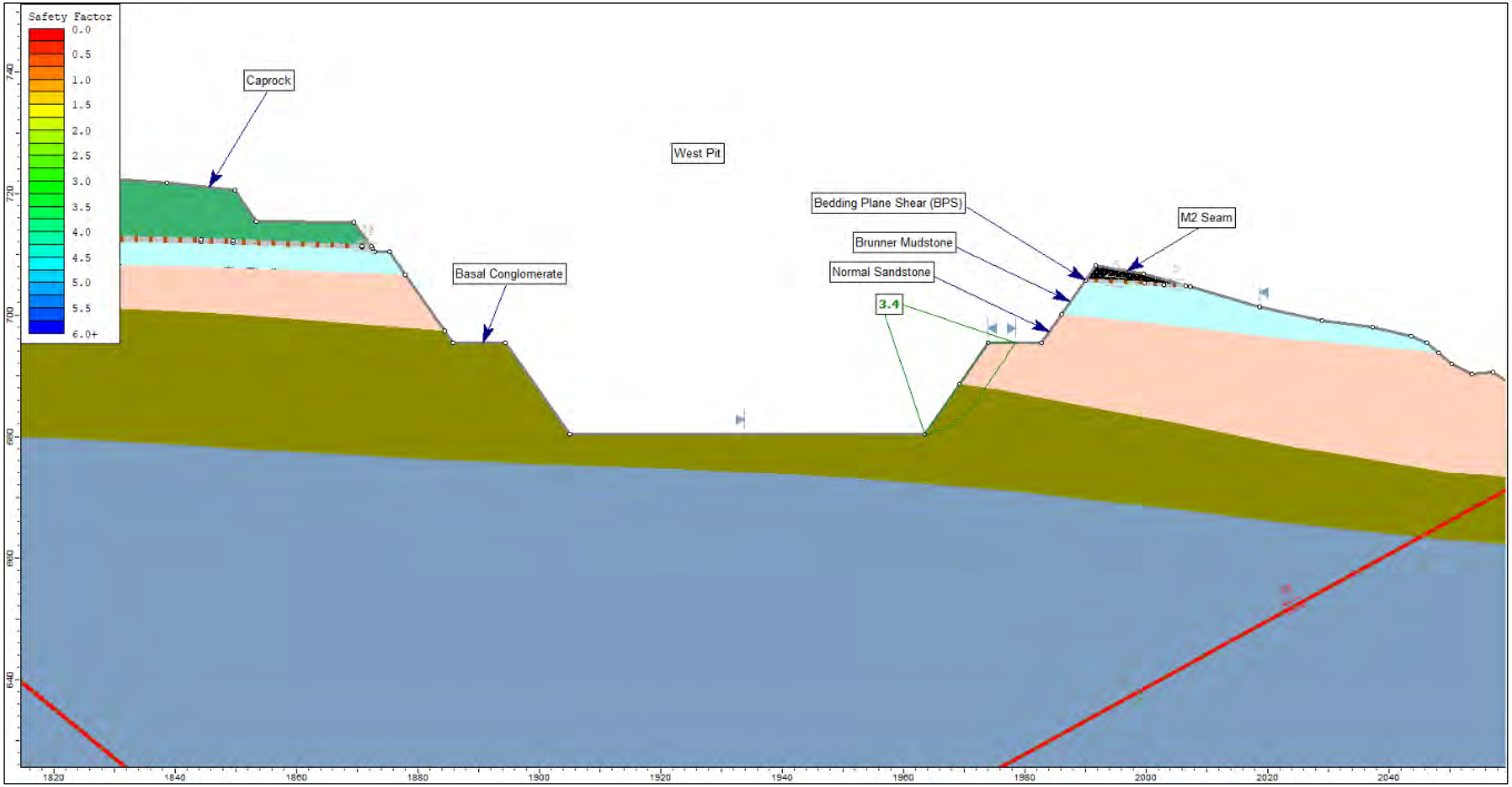

Figure: B27 – Slope Stability Analysis of West Sump Highwall (South wall) – Section 101 – Static Conditions.

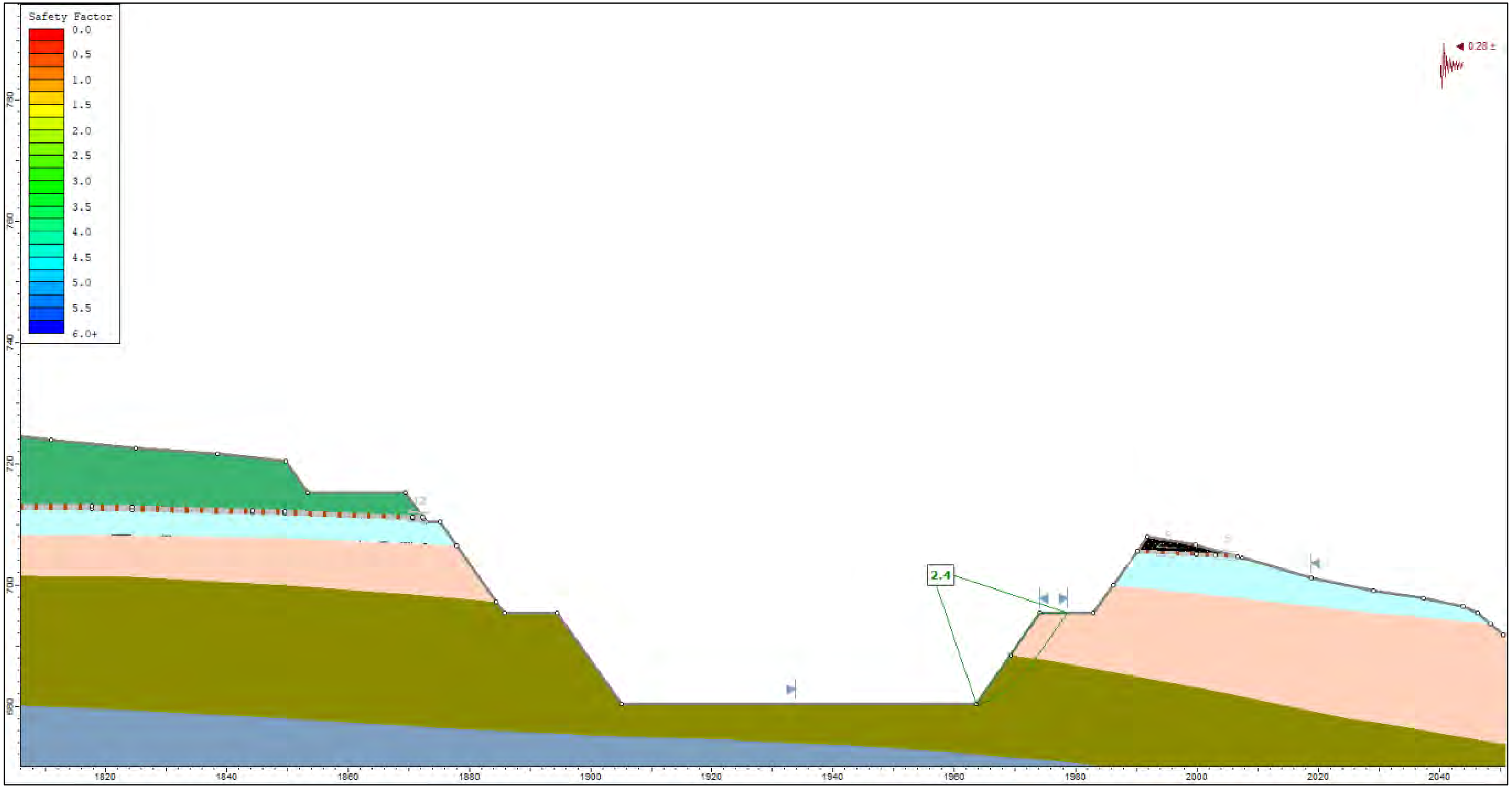

Figure: B28 – Slope Stability Analysis of West Sump Highwall (South wall) – Section 101 – SLS Conditions.

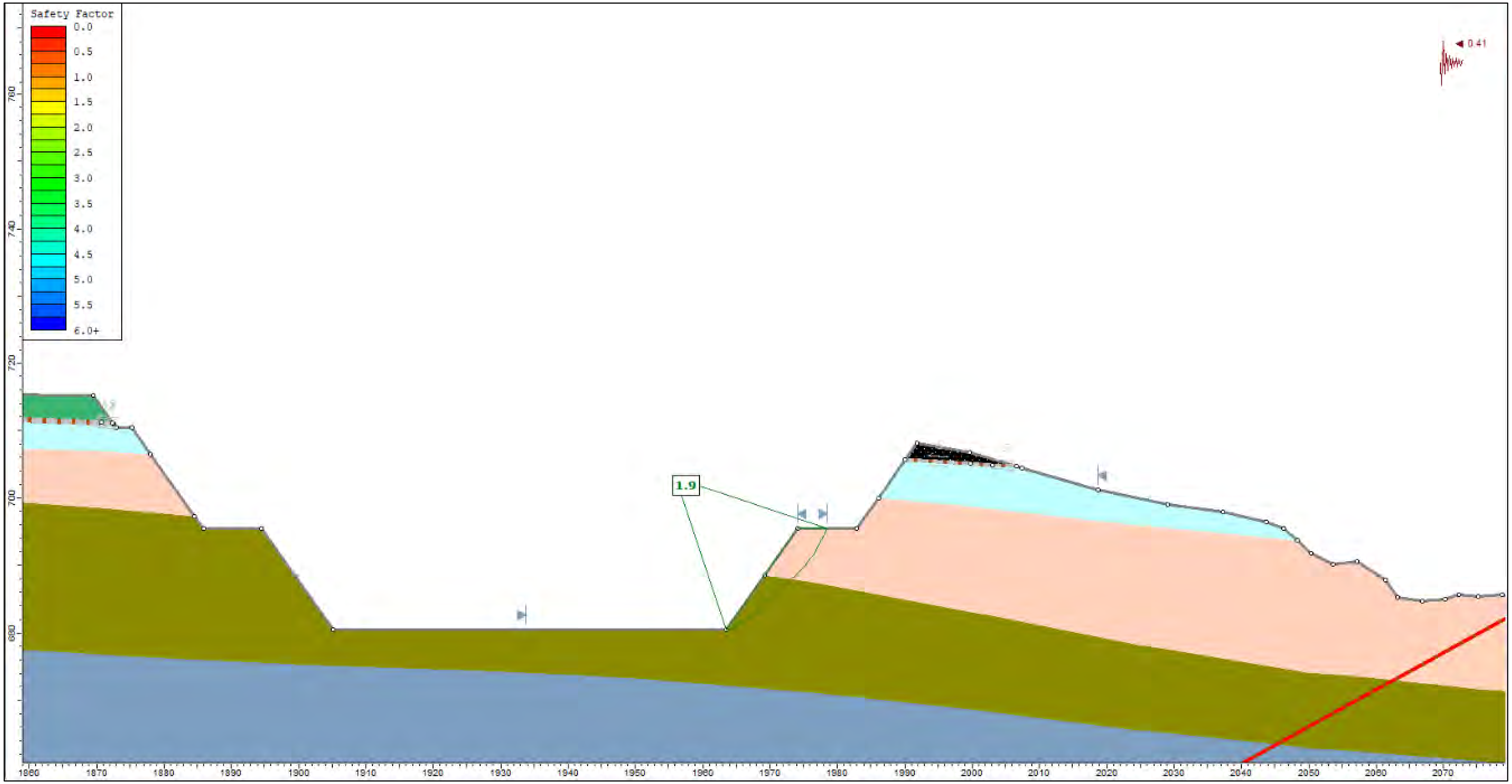

Figure: B29 – Slope Stability Analysis of West Sump Highwall (South wall) – Section 101 – ULS Conditions.

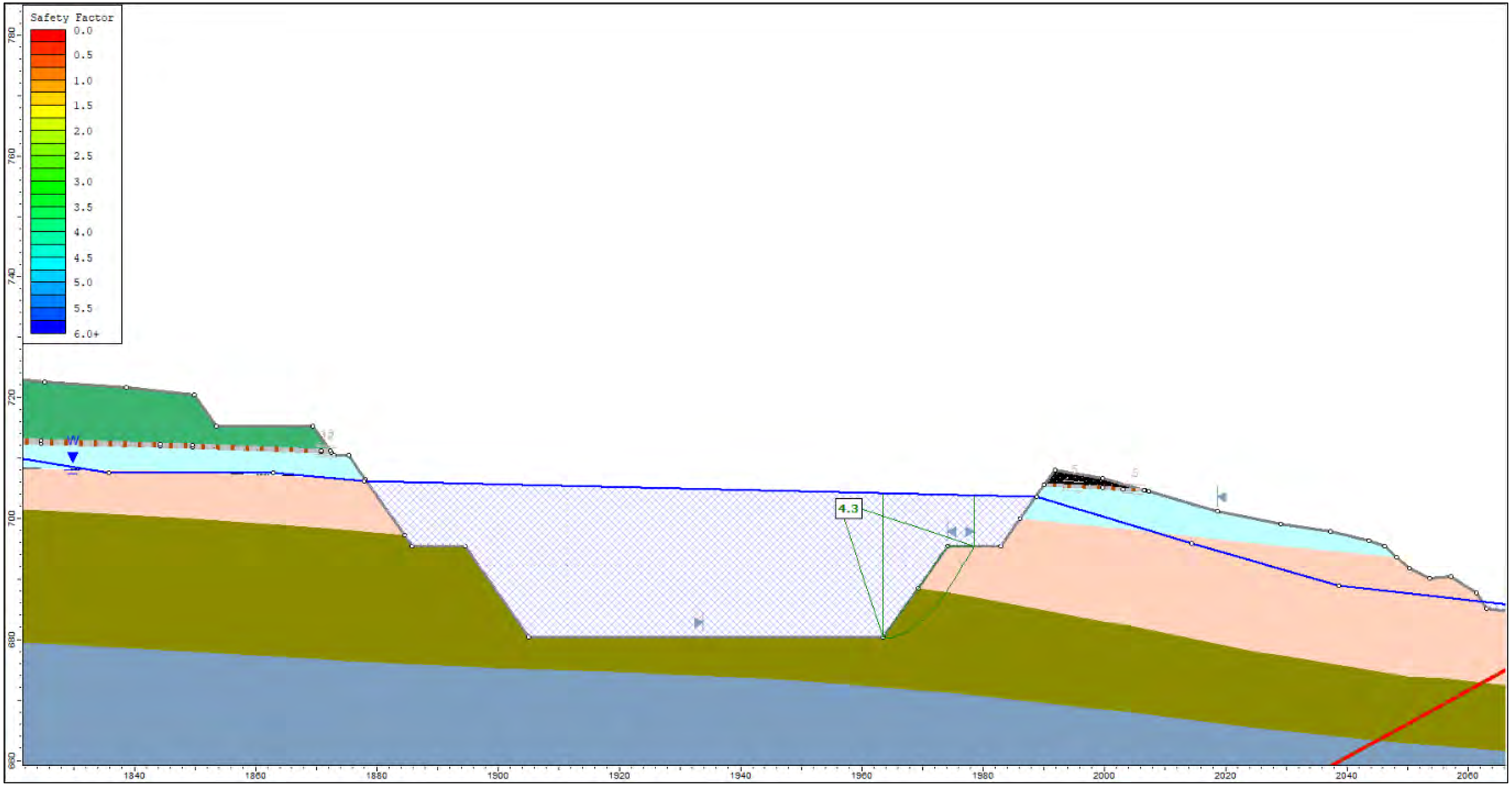

Figure: B30 – Slope Stability Analysis of West Sump Highwall (South wall) – Section 101 – Elevated Groundwater Conditions.

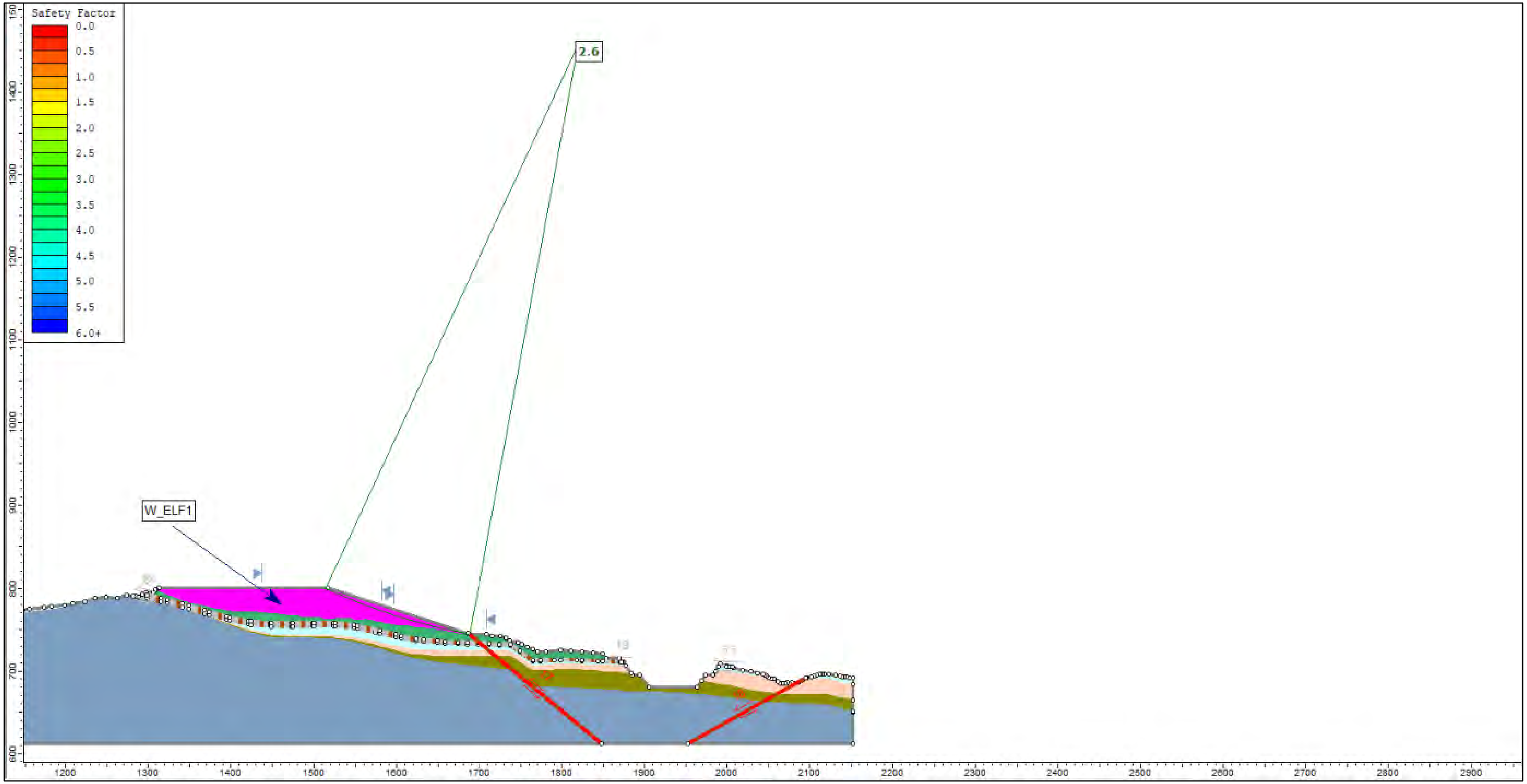

Figure: B31 – Slope Stability Analysis of W_ELF – Section 101 – Static Conditions.

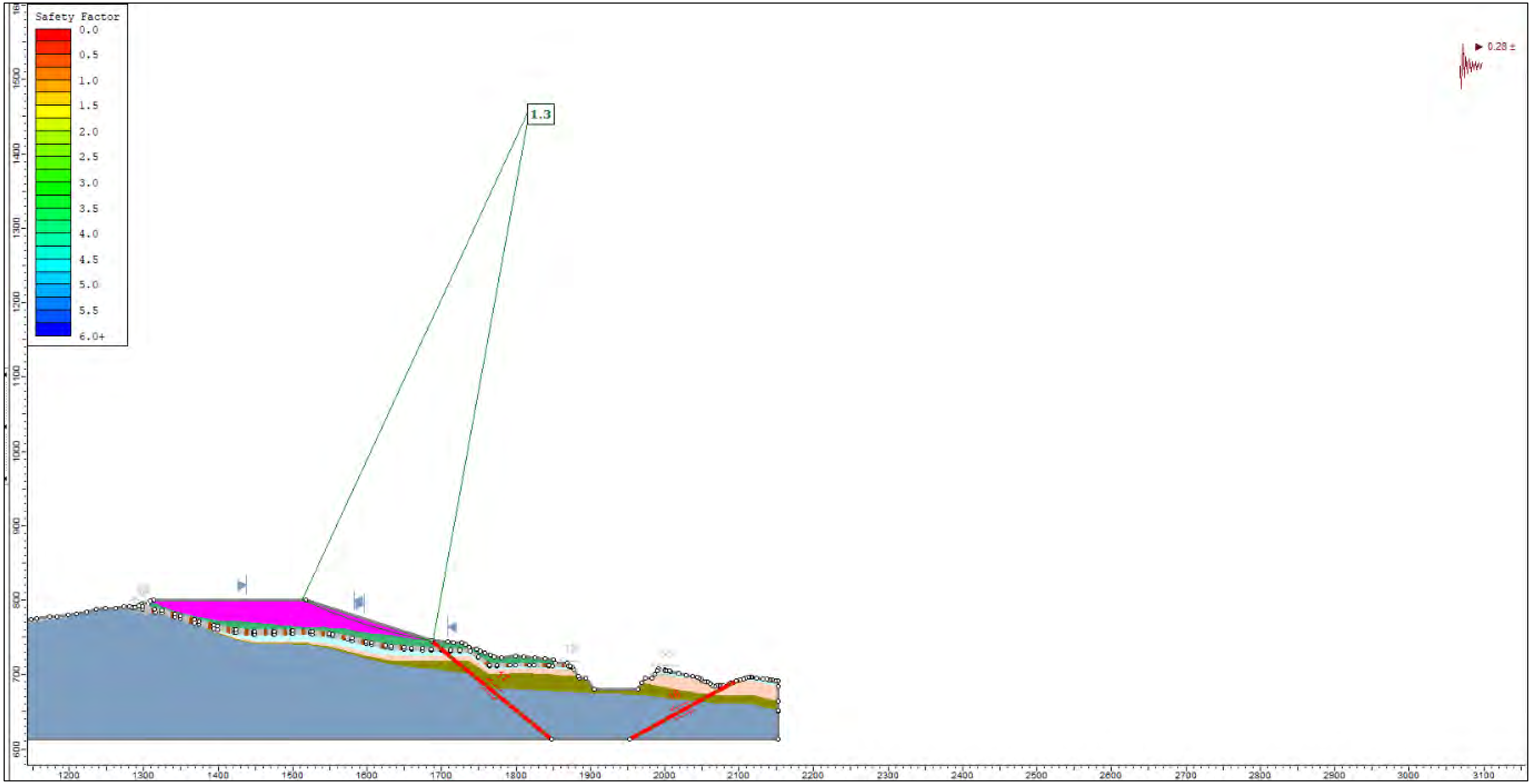

Figure: B32 – Slope Stability Analysis of W_ELF – Section 101 – SLS Conditions.

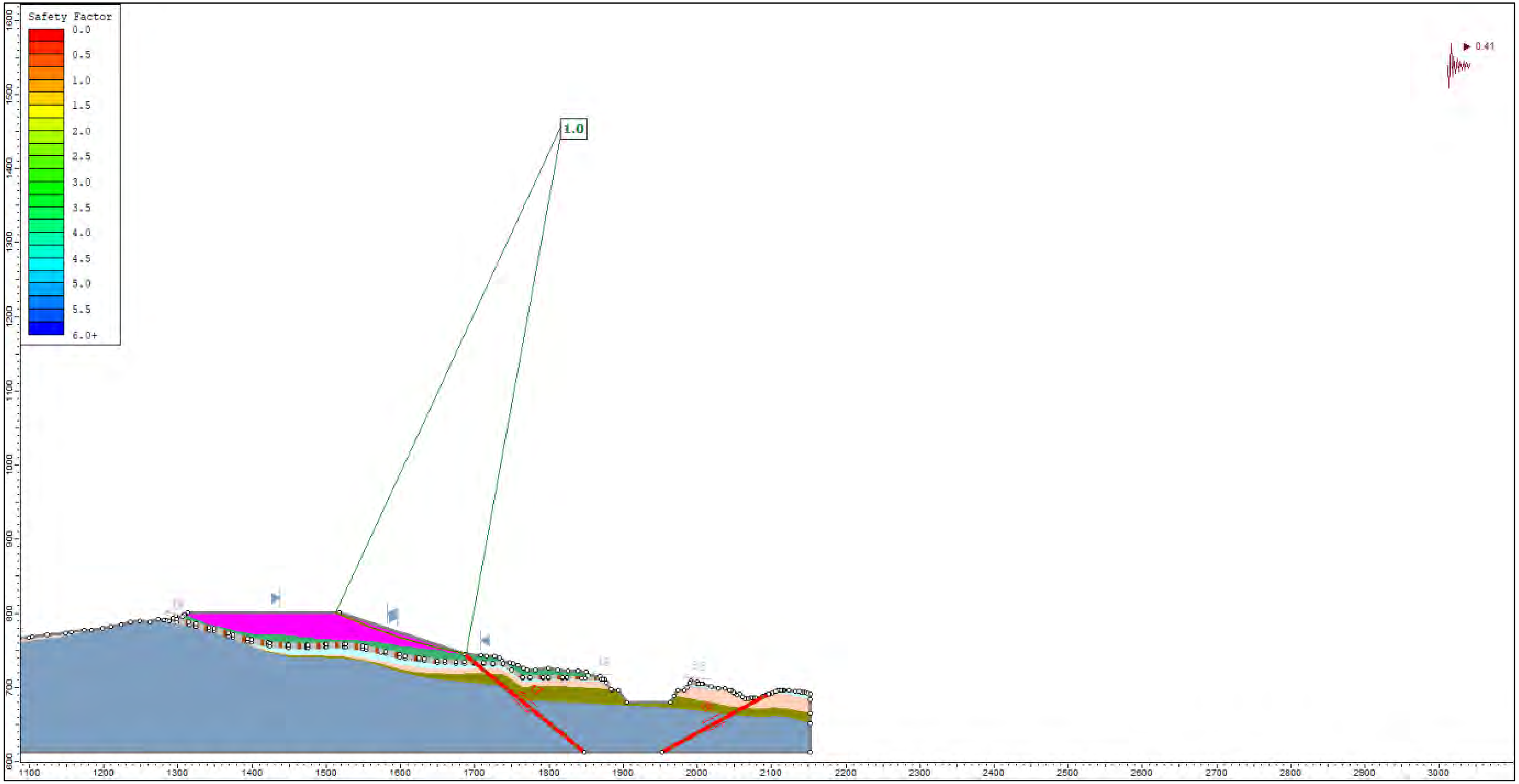

Figure: B33 – Slope Stability Analysis of W_ELF – Section 101 – ULS Conditions.

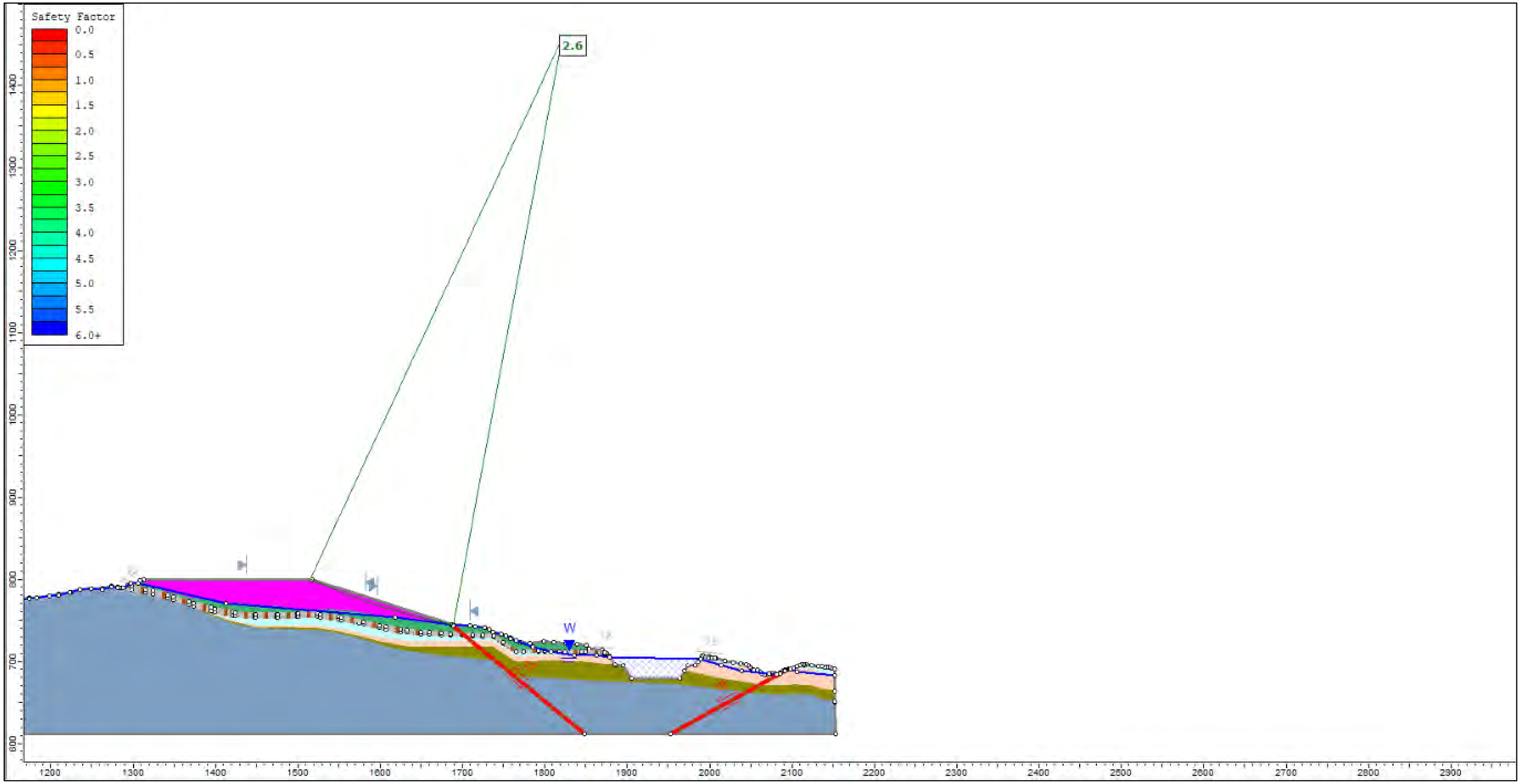

Figure: B34 – Slope Stability Analysis of W_ELF – Section 101 – Elevated Groundwater Conditions.

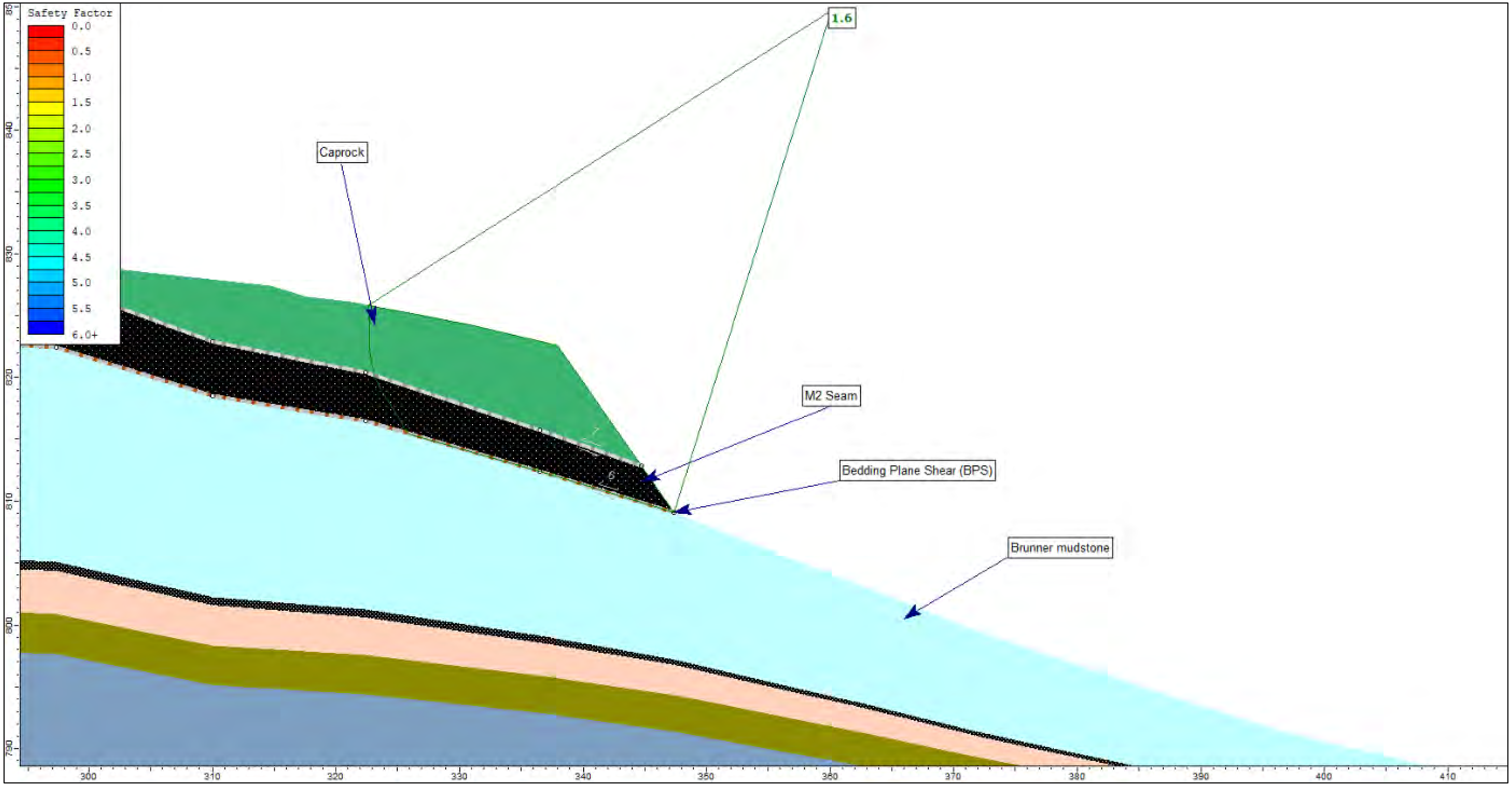

Figure: B35 – Slope Stability Analysis of East Pit (North) Highwall – Section 101 – Static Conditions.

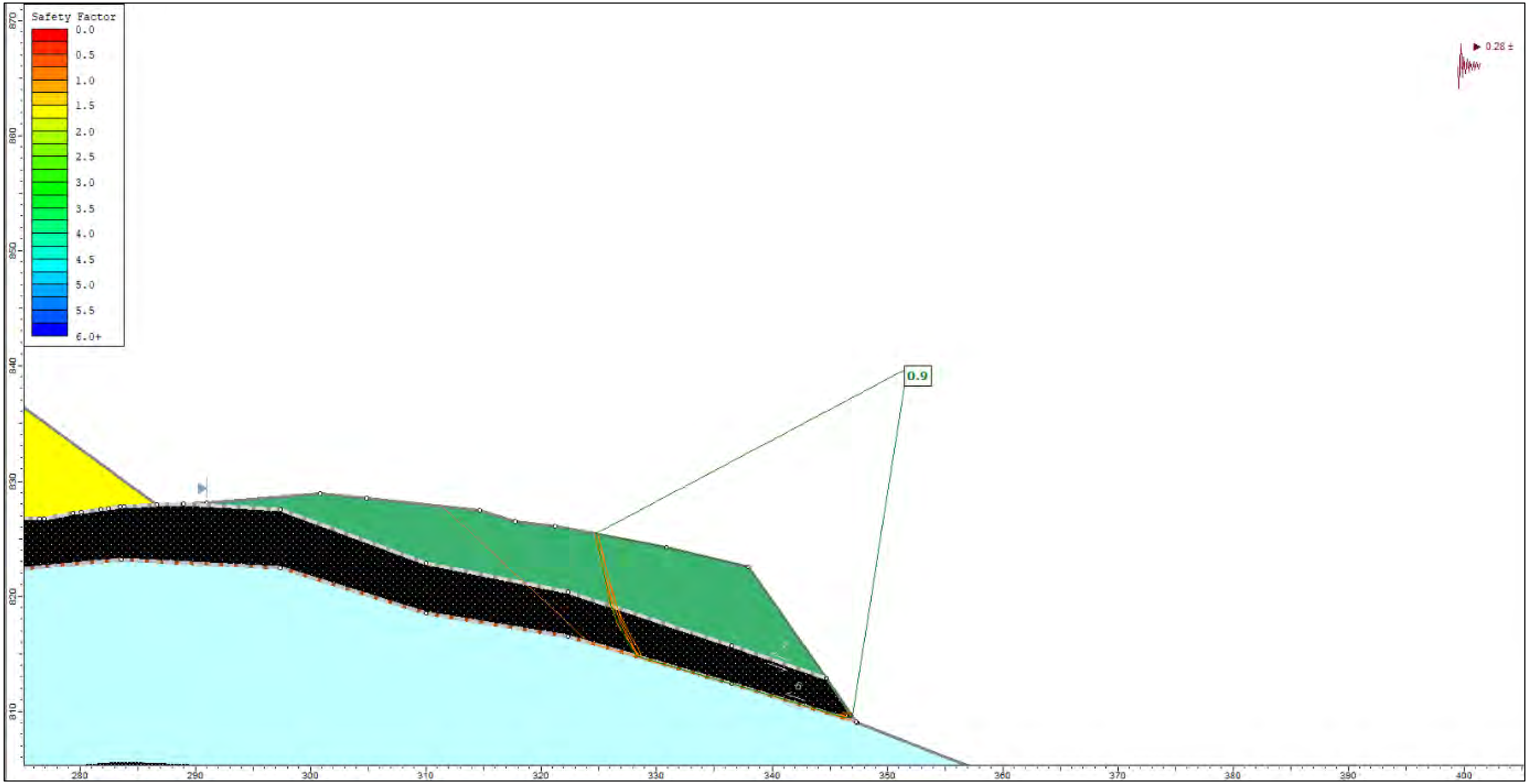

Figure: B36 – Slope Stability Analysis of East Pit (North) Highwall – Section 101 – SLS Conditions.

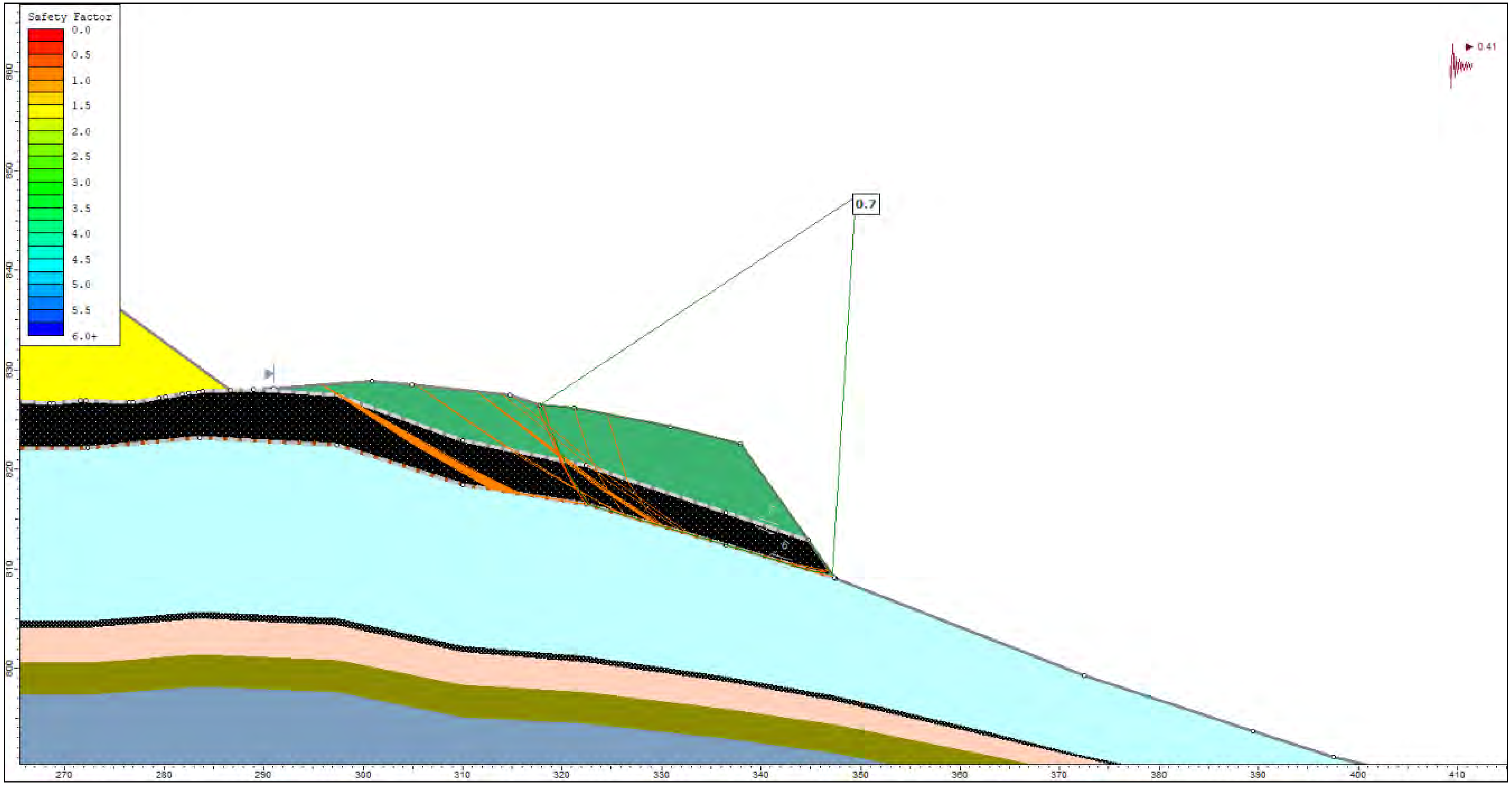

Figure: B37 – Slope Stability Analysis of East Pit (North) Highwall – Section 101 –ULS Conditions.

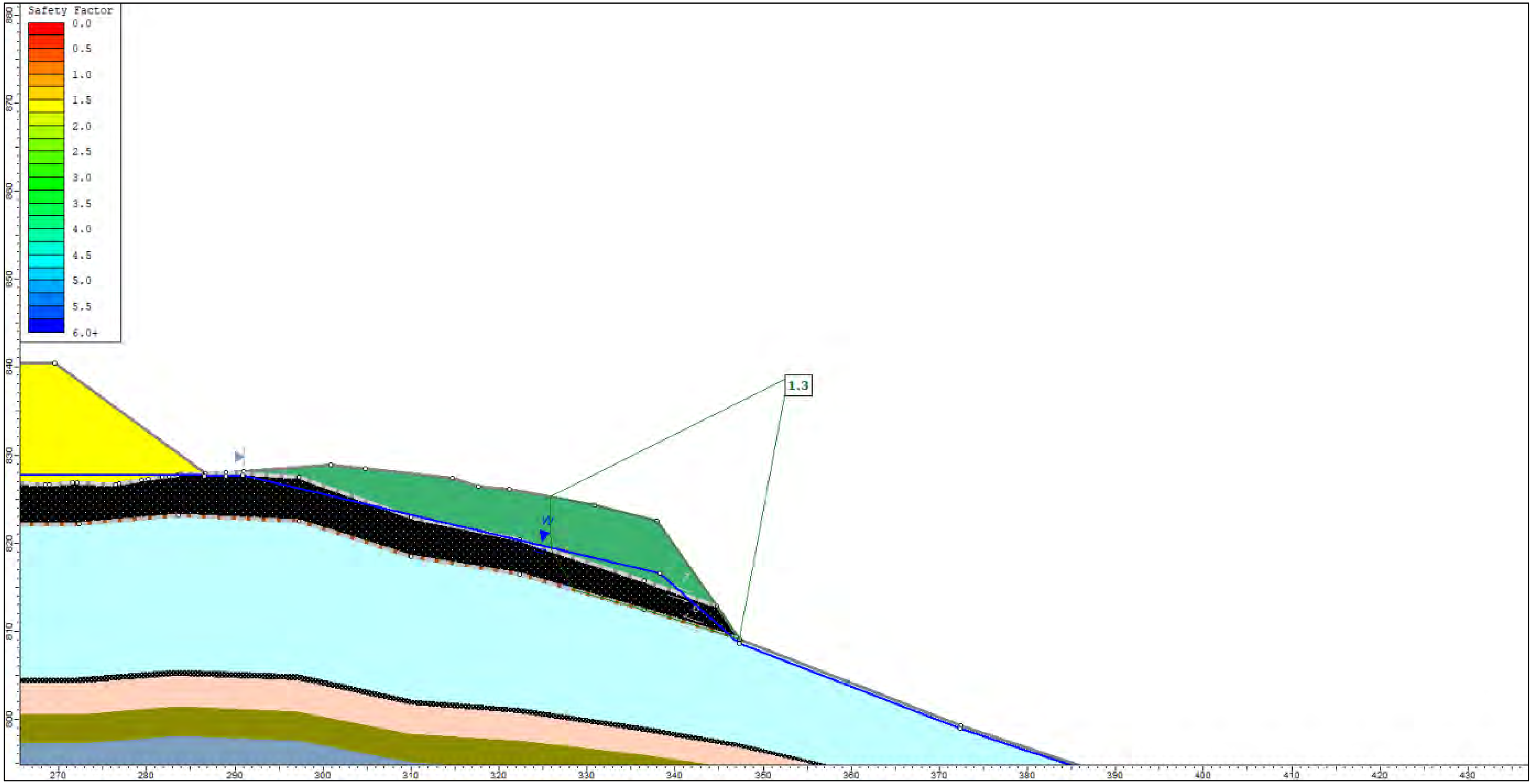

Figure: B38 – Slope Stability Analysis of East Pit (North) Highwall – Section 101 – Elevated Groundwater Conditions.

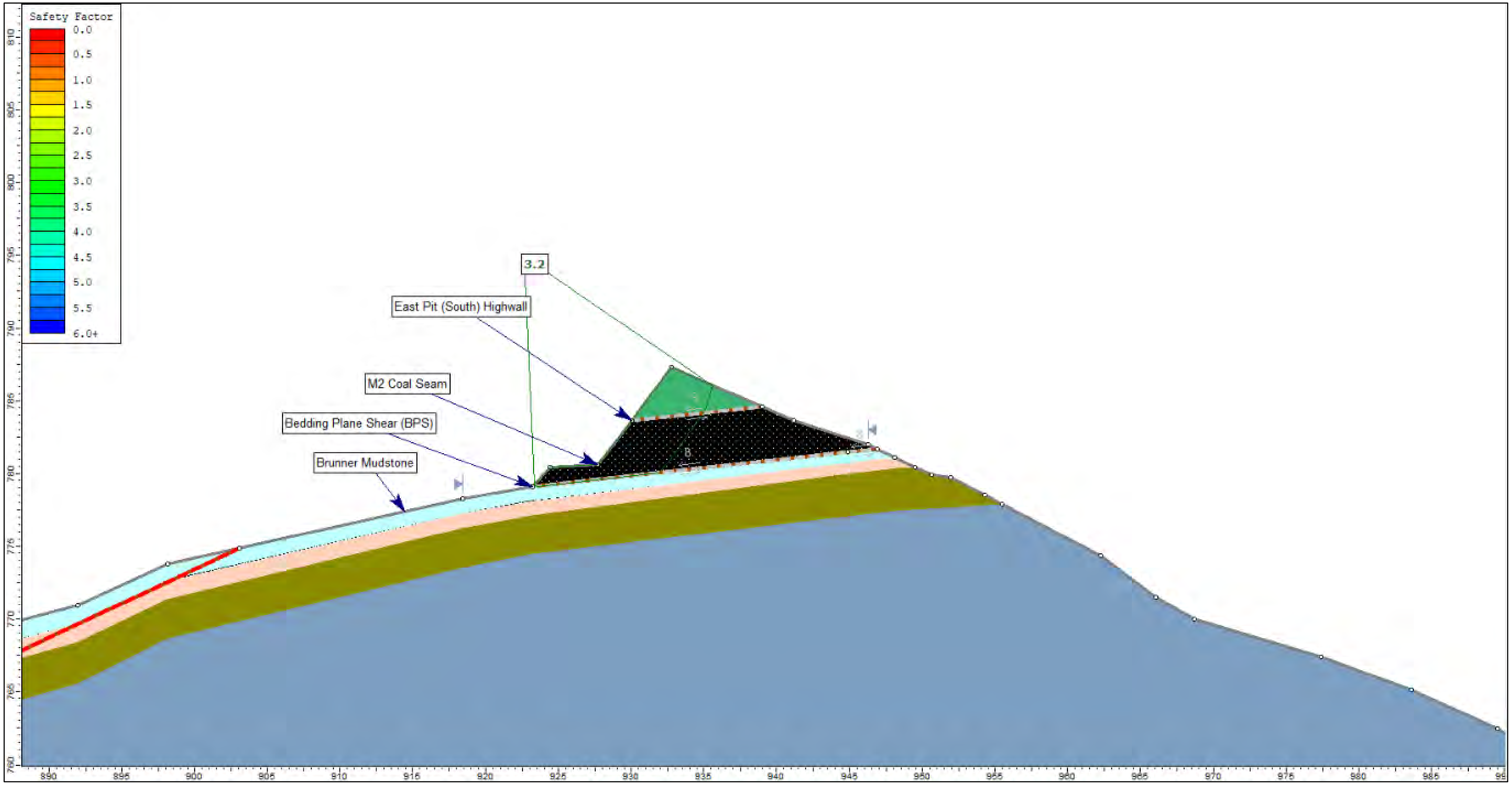

Figure: B39 – Slope Stability Analysis of East Pit (South) Highwall – Section 101 – Static Conditions.

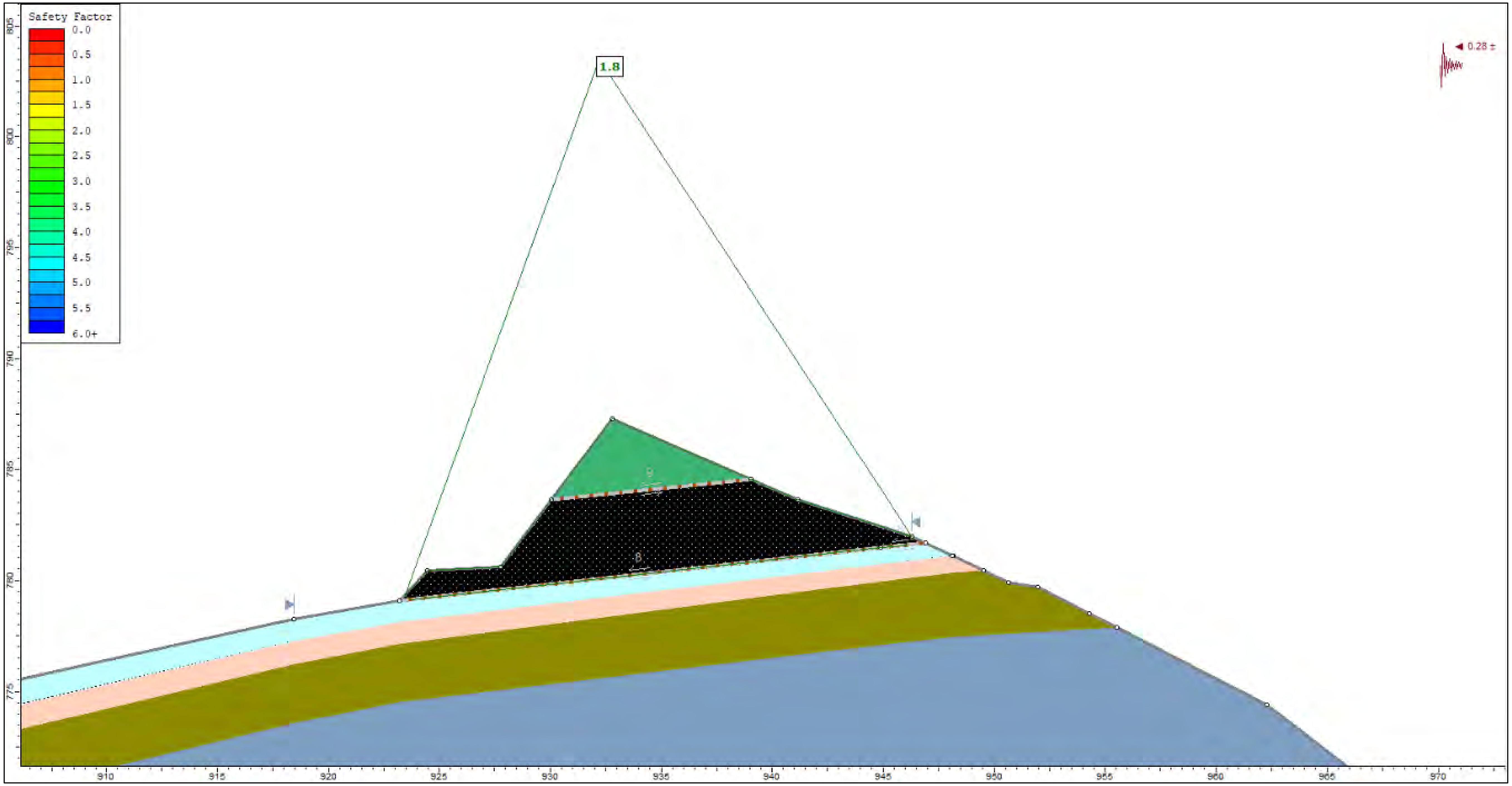

Figure: B40 – Slope Stability Analysis of East Pit (South) Highwall – Section 101 – SLS Conditions.

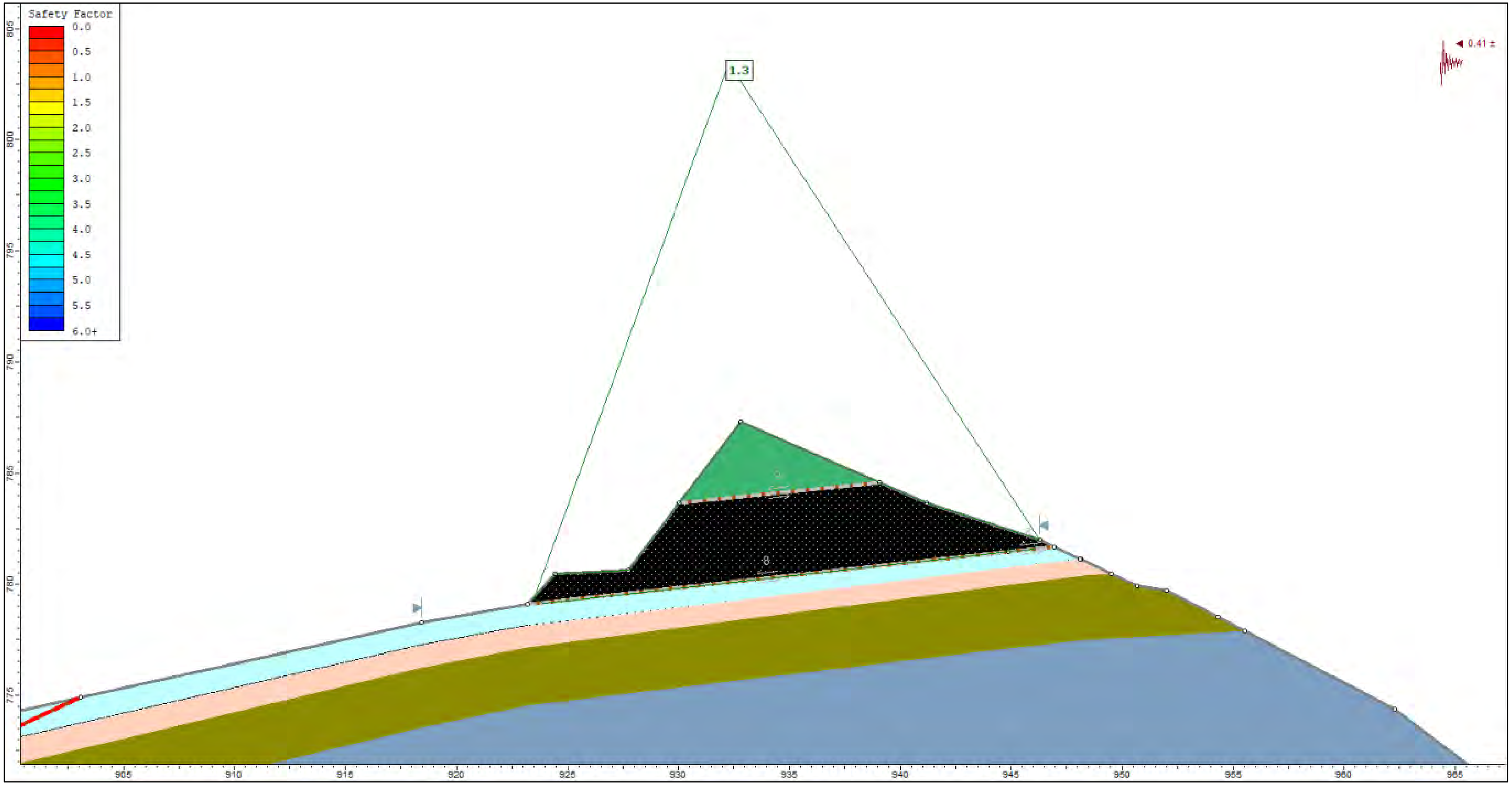

Figure: B41 – Slope Stability Analysis of East Pit (South) Highwall – Section 101 – ULS Conditions.

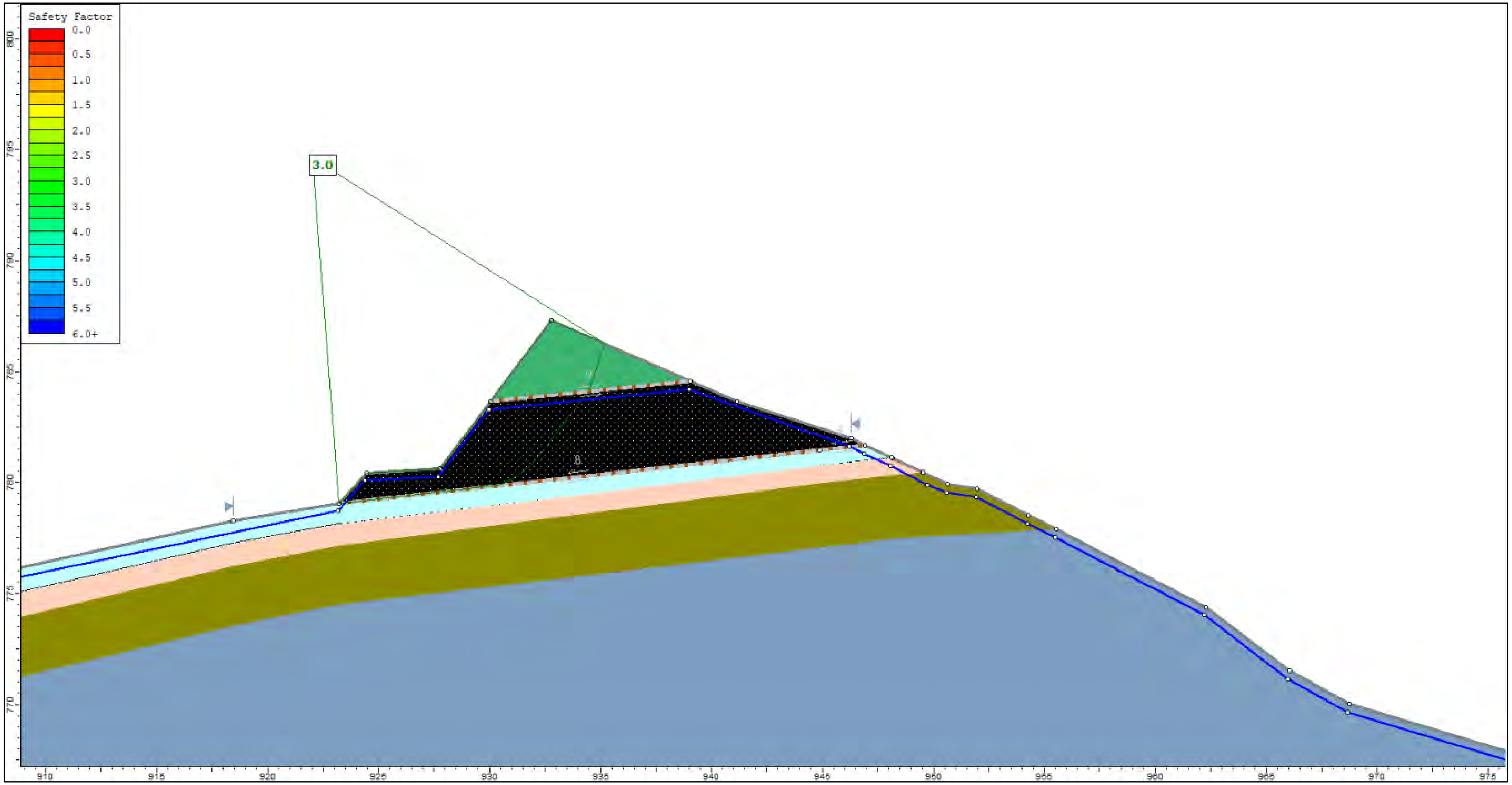

Figure: B42 – Slope Stability Analysis of East Pit (South) Highwall – Section 101 – Elevated Groundwater Conditions.

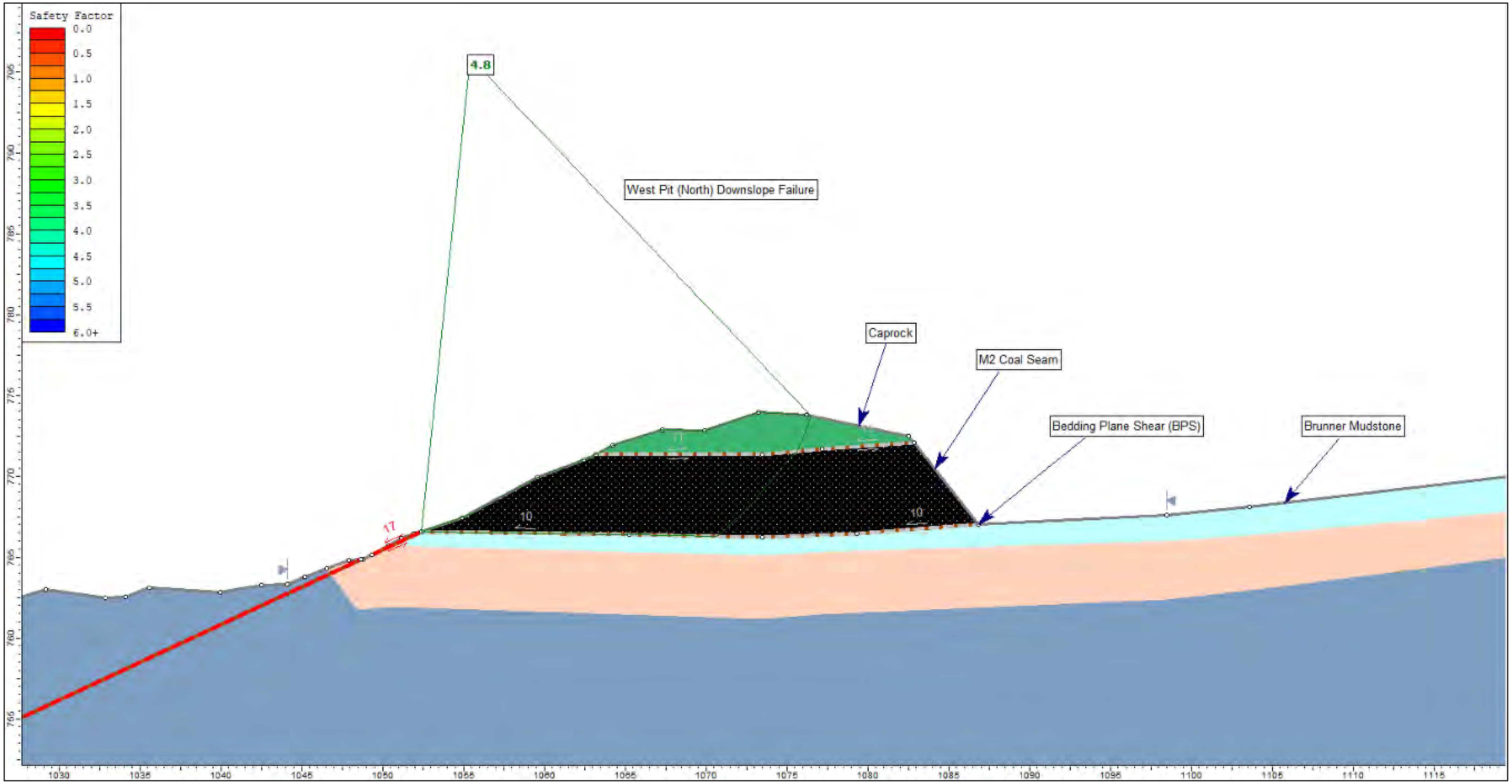

Figure: B43 – Slope Stability Analysis of West Pit (North) Highwall – Downslope Failure – Section 101 – Static Conditions.

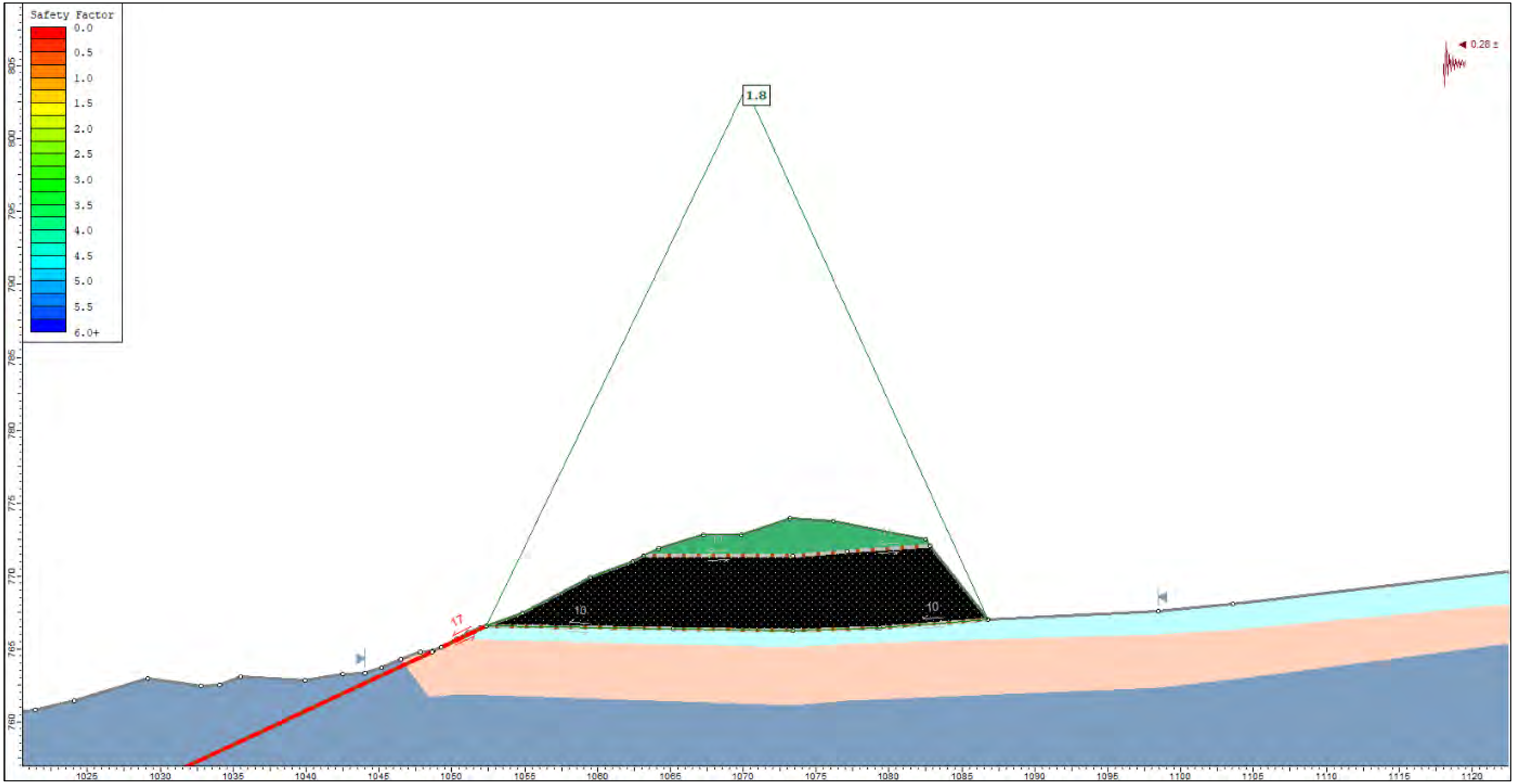

Figure: B44 – Slope Stability Analysis of West Pit (North) Highwall – Downslope Failure – Section 101 – SLS Conditions.

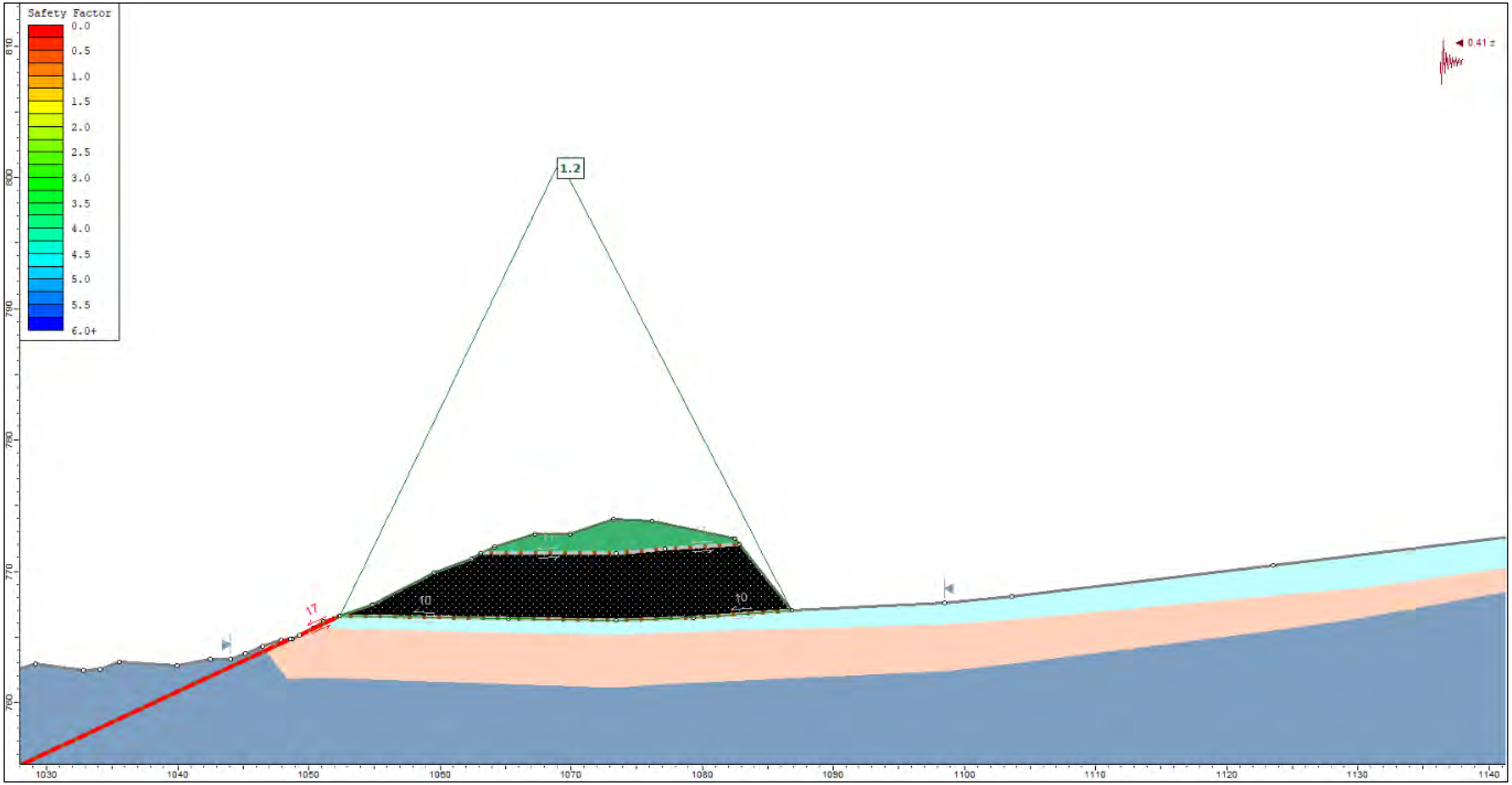

Figure: B45 – Slope Stability Analysis of West Pit (North) Highwall – Down Slope Failure – Section 101 – ULS Conditions.

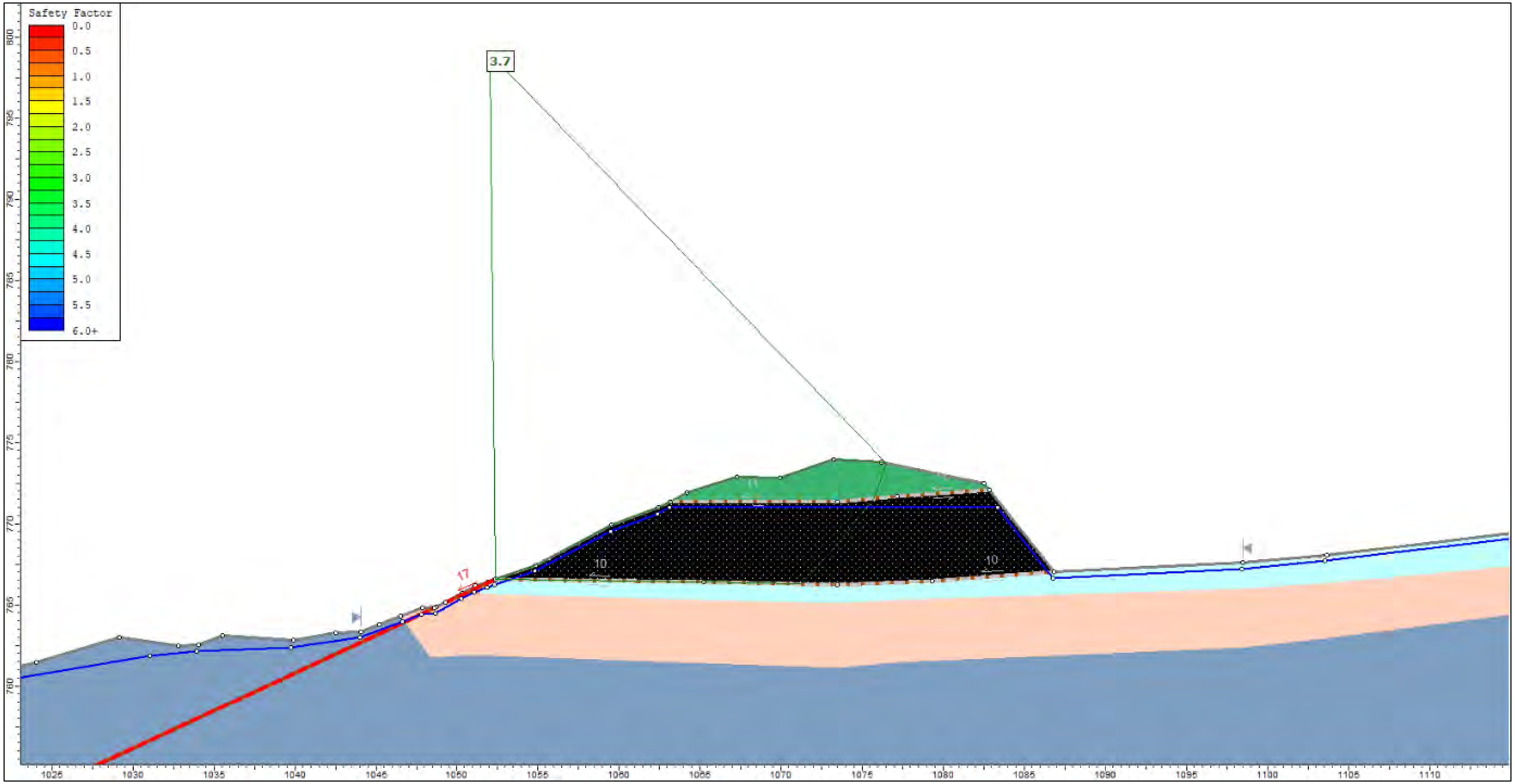

Figure: B46 – Slope Stability Analysis of West Pit (North) Highwall – Downslope Failure – Section 101 – Elevated Groundwater Conditions.

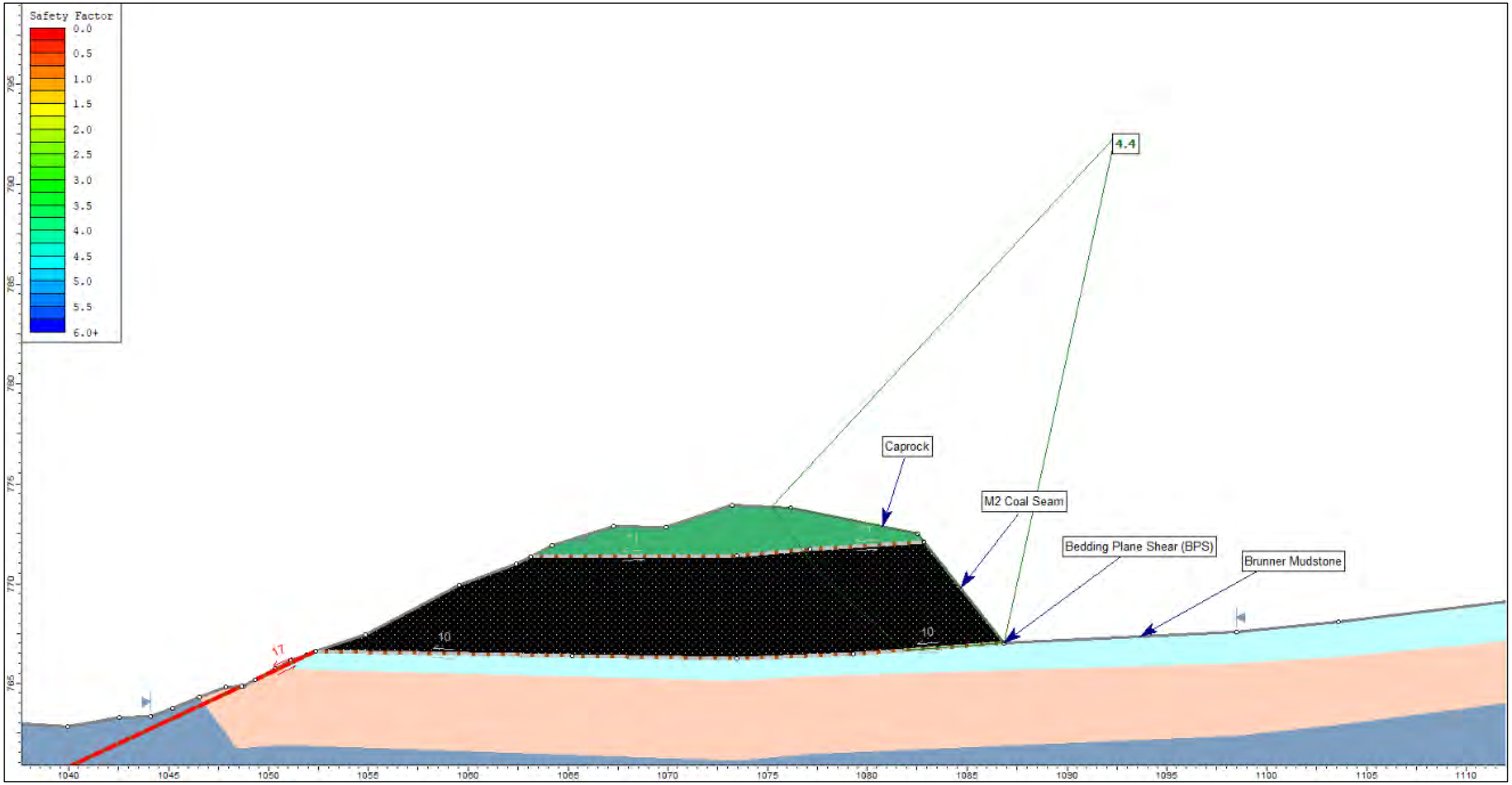

Figure: B47 – Slope Stability Analysis of West Pit (North) Highwall – Into Pit Failure – Section 101 – Static Conditions.

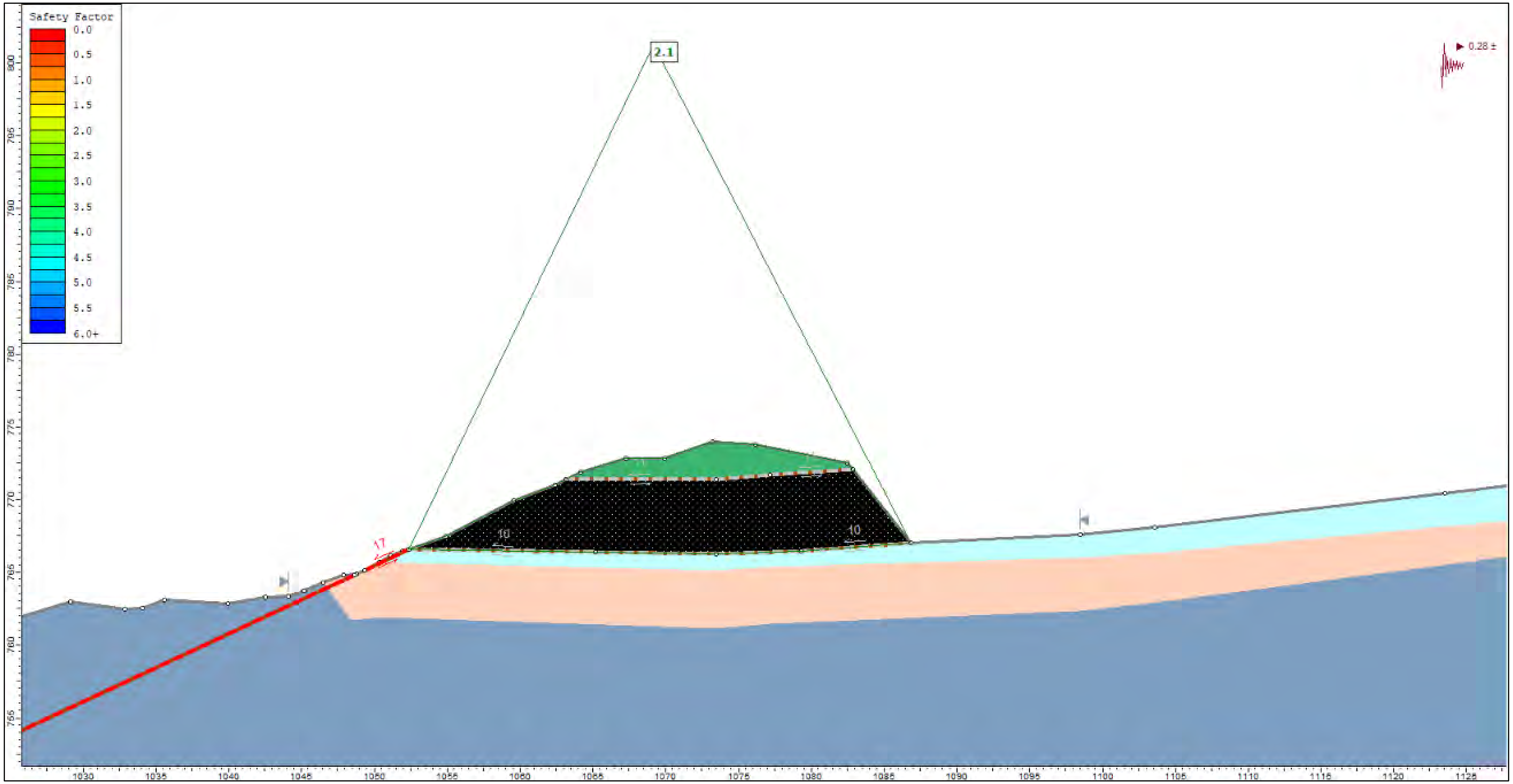

Figure: B48 – Slope Stability Analysis of West Pit (North) Highwall – Into Pit Failure – Section 101 – SLS Conditions.

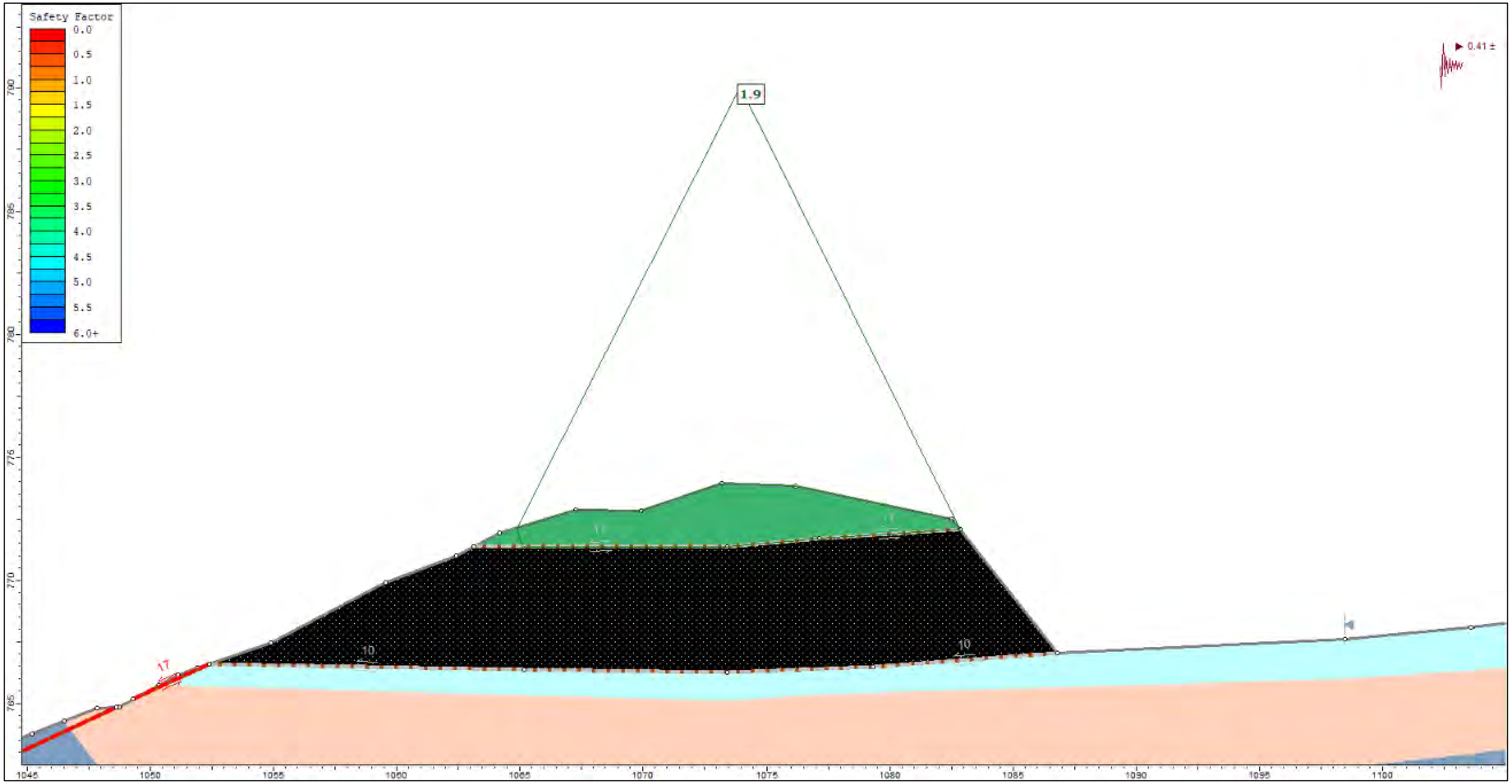

Figure: B49 – Slope Stability Analysis of West Pit (North) Highwall – Into Pit Failure – Section 101 – ULS Conditions.

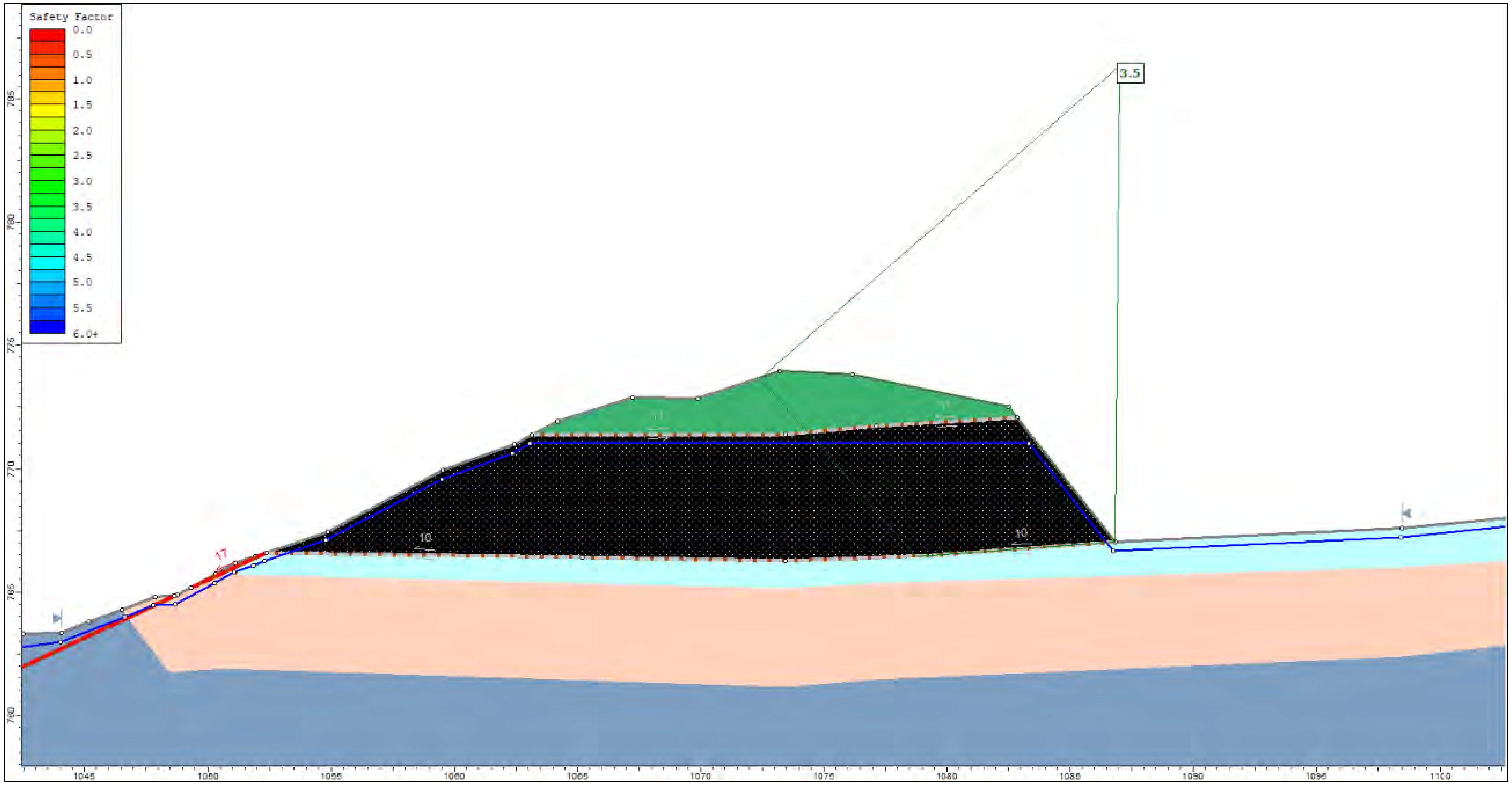

Figure: B50 – Slope Stability Analysis of West Pit (North) Highwall – Into Pit Failure – Section 101 – Elevated Groundwater Conditions.

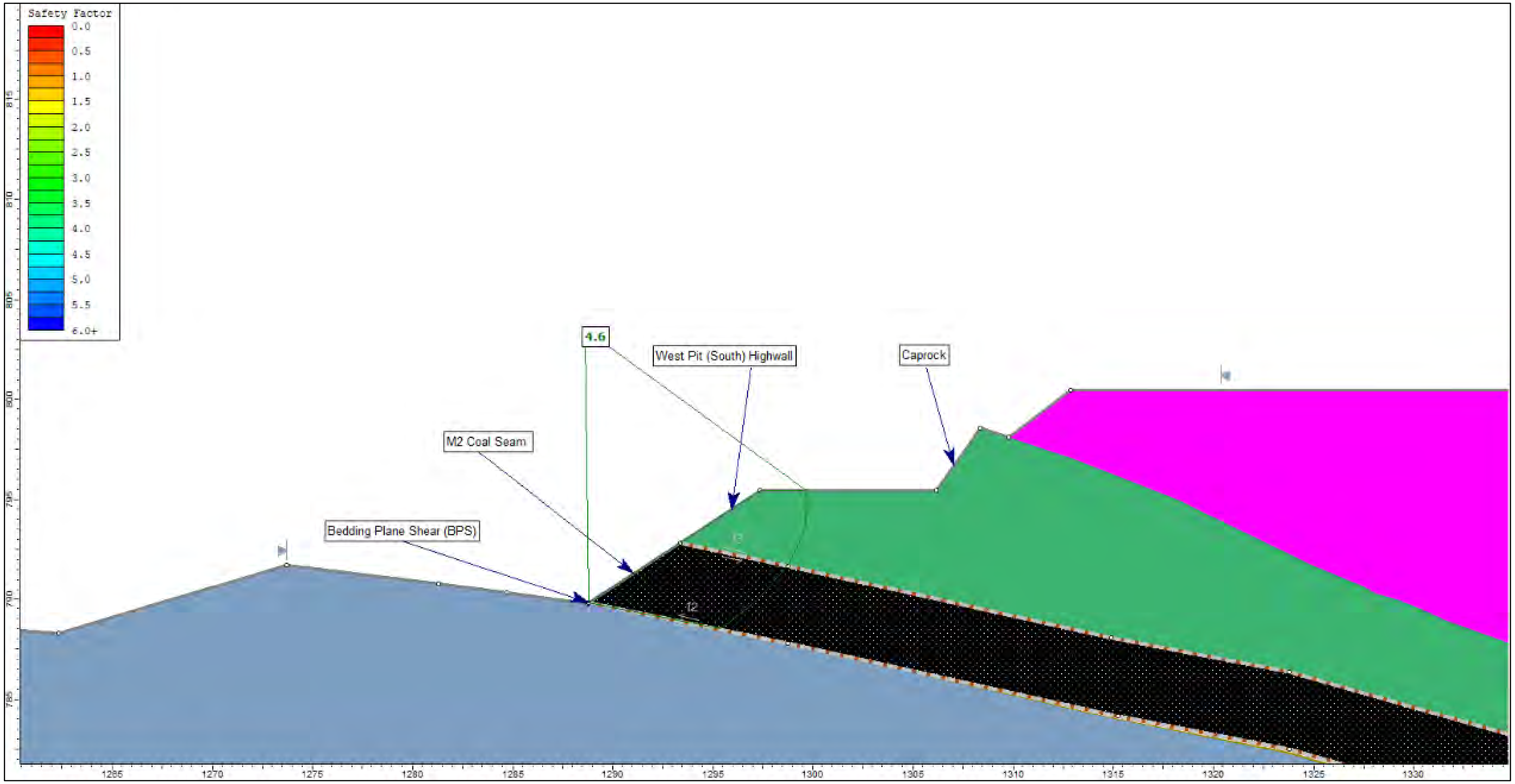

Figure: B51– Slope Stability Analysis of West Pit (South) Highwall - Section 101 – Static Conditions.

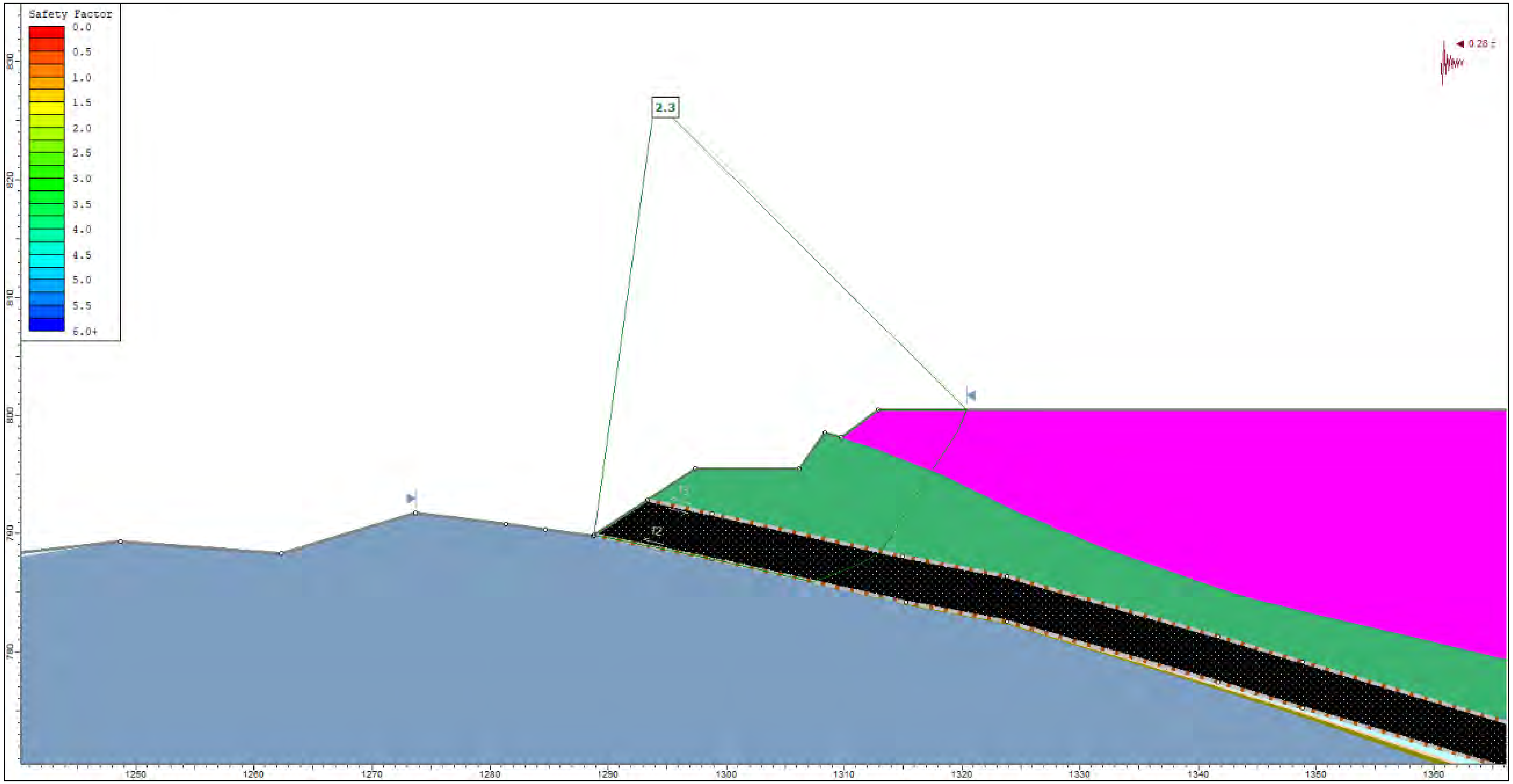

Figure: B52– Slope Stability Analysis of West Pit (South) Highwall - Section 101 – SLS Conditions.

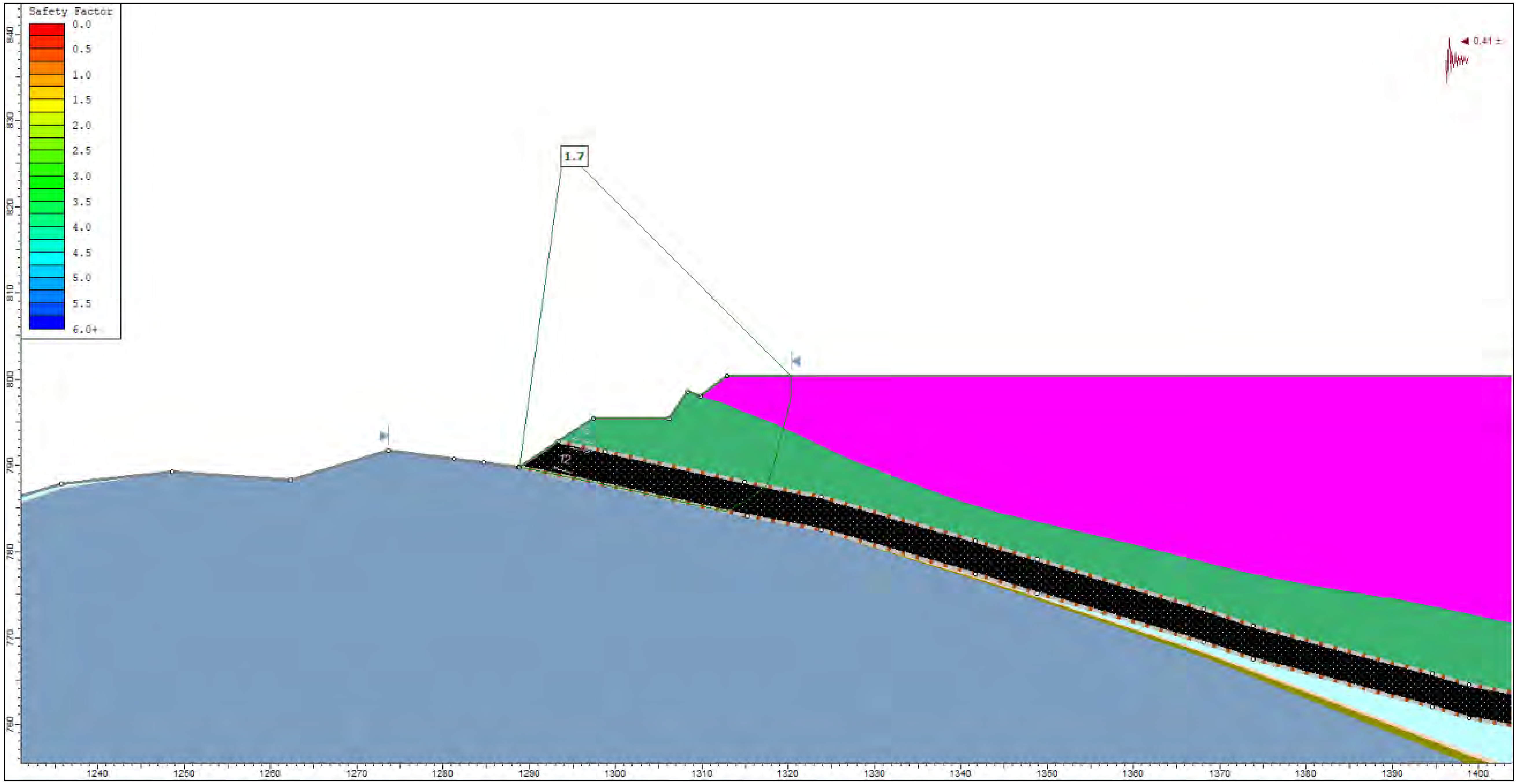

Figure: B53– Slope Stability Analysis of West Pit (South) Highwall - Section 101 –ULS Conditions.

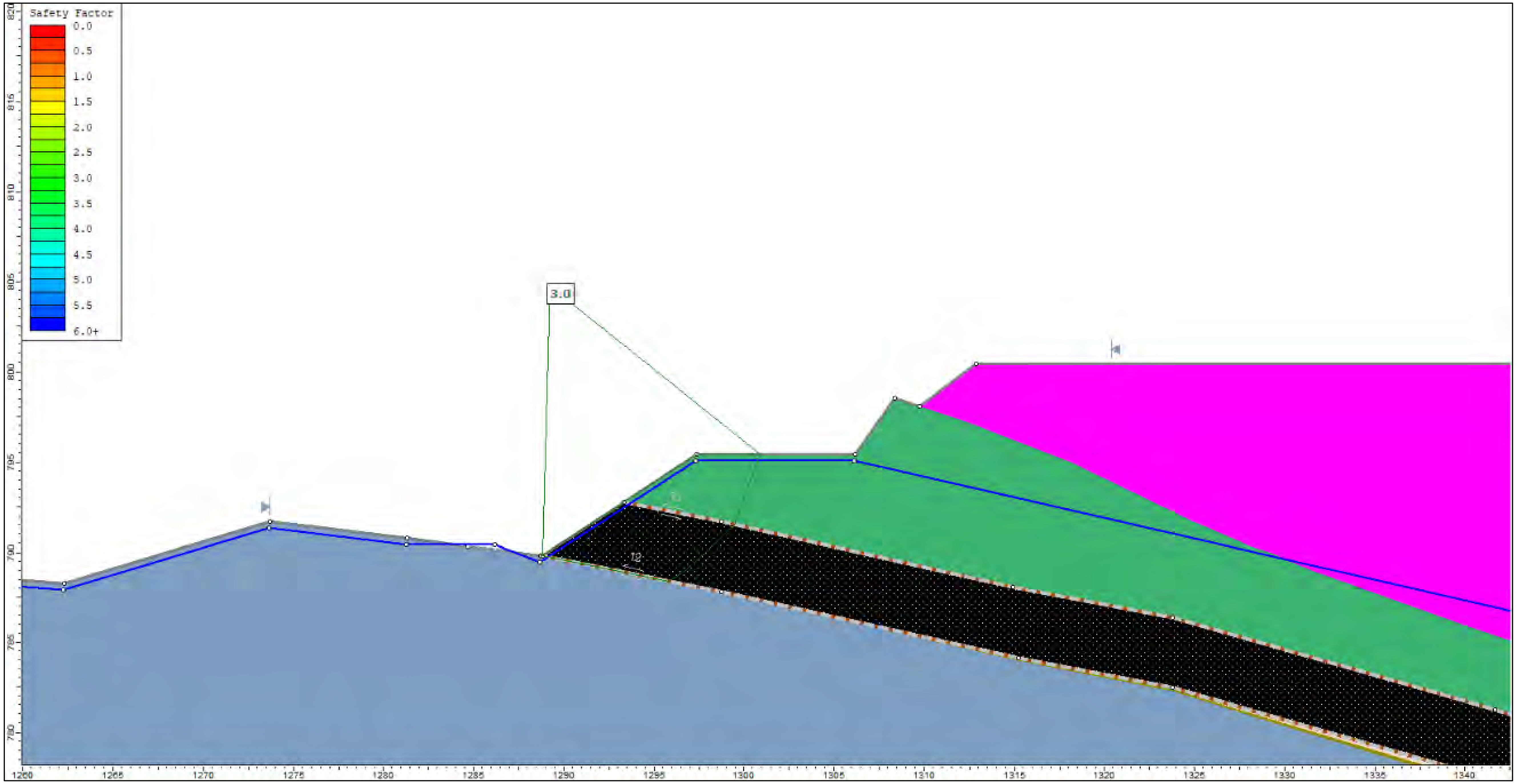

Figure: B54– Slope Stability Analysis of West Pit (South) Highwall - Section 101 –Elevated Groundwater Conditions.

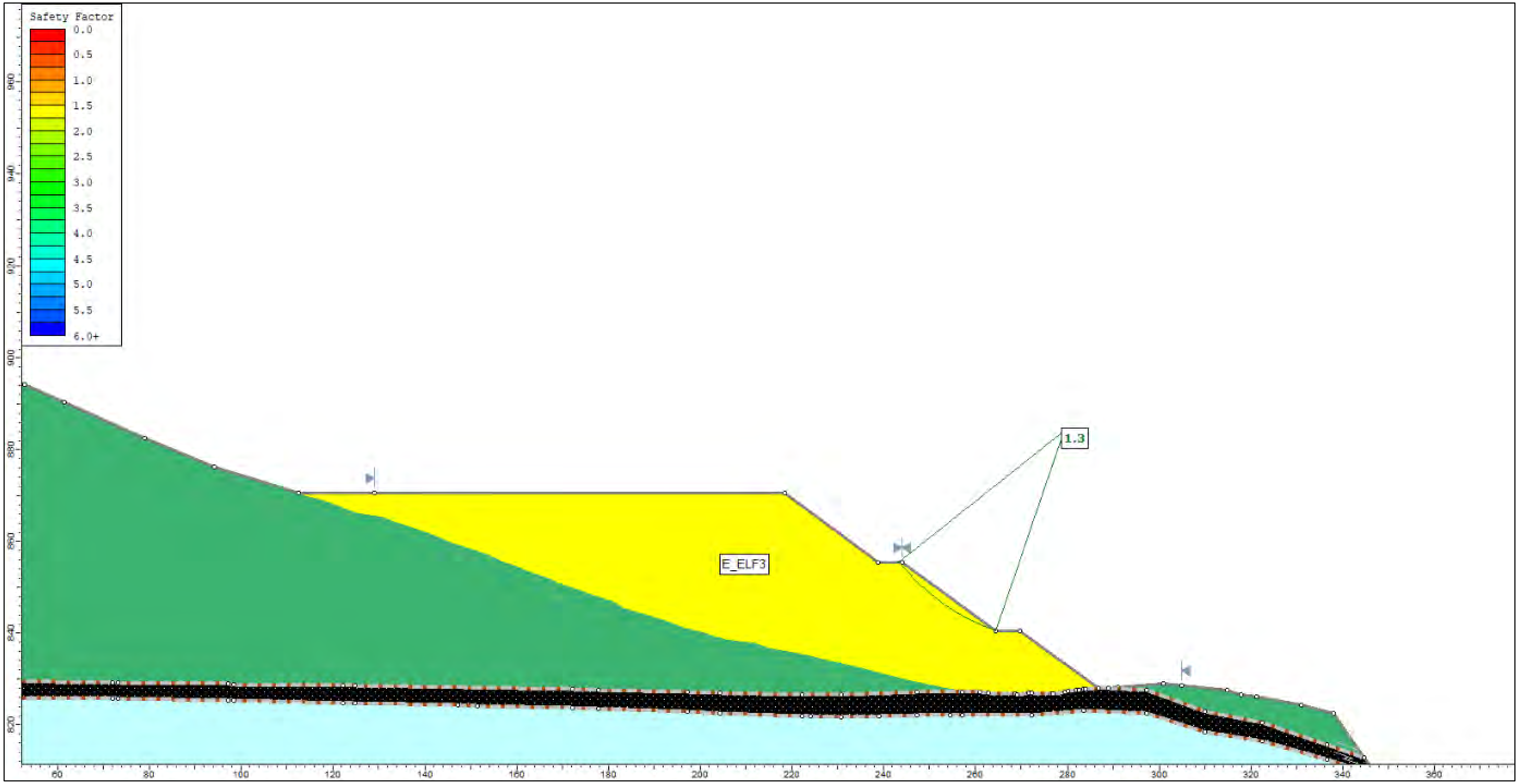

Figure: B55 – Slope Stability Analysis of E_ELF3 – Section 101 – Static Conditions.

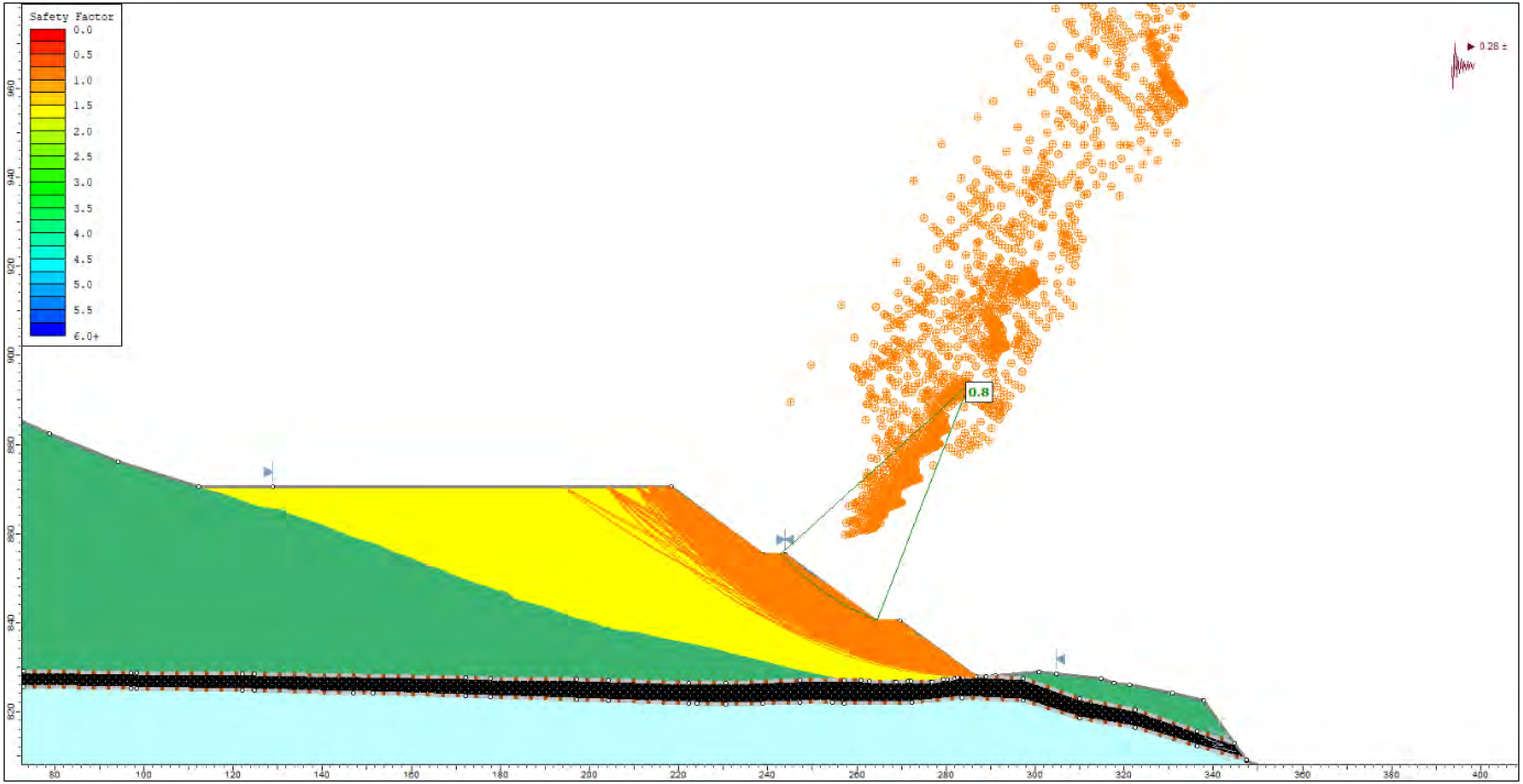

Figure: B56 – Slope Stability Analysis of E_ELF3 – Section 101 – SLS Conditions.

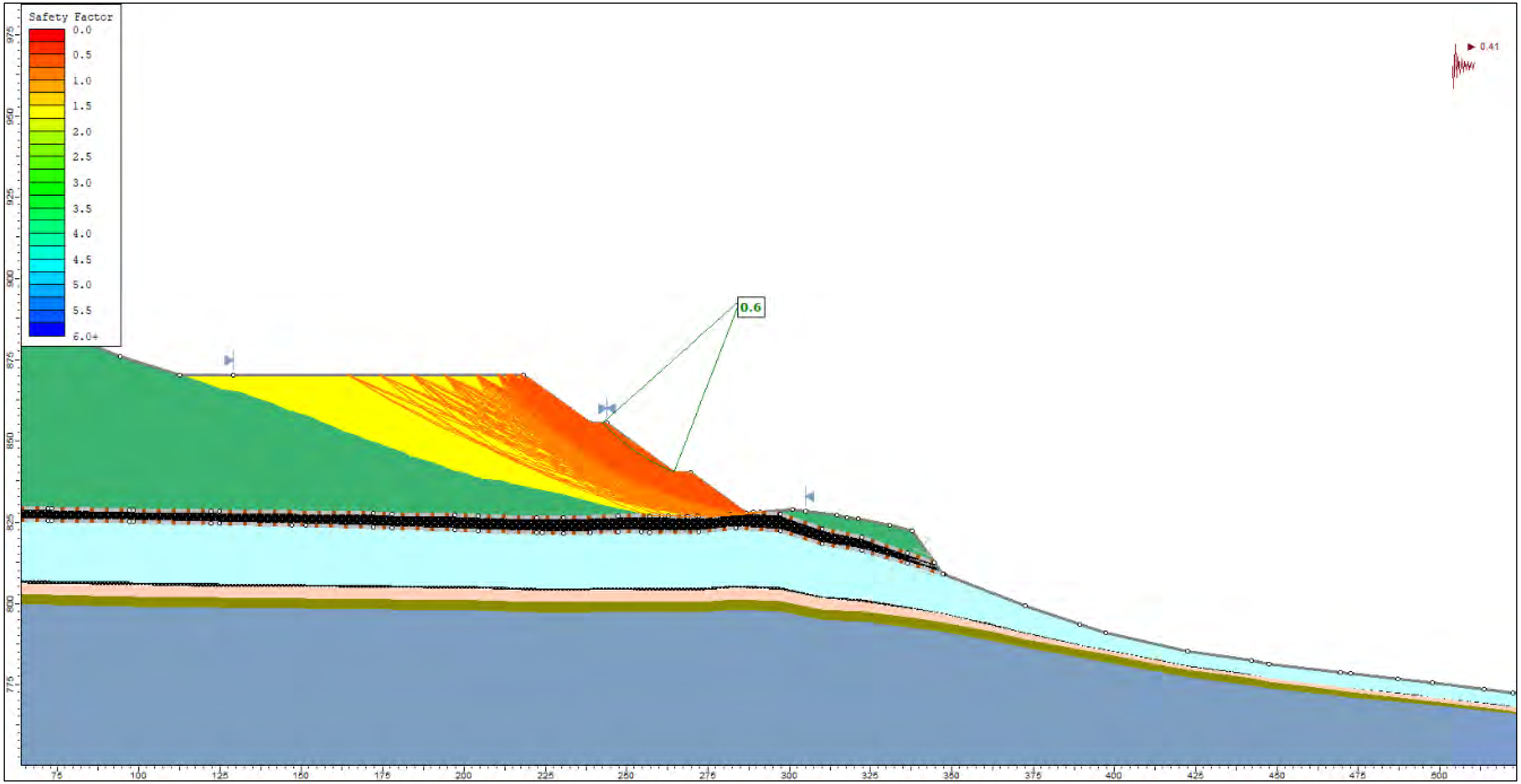

Figure: B57 – Slope Stability Analysis of E_ELF3 – Section 101 – ULS Conditions.

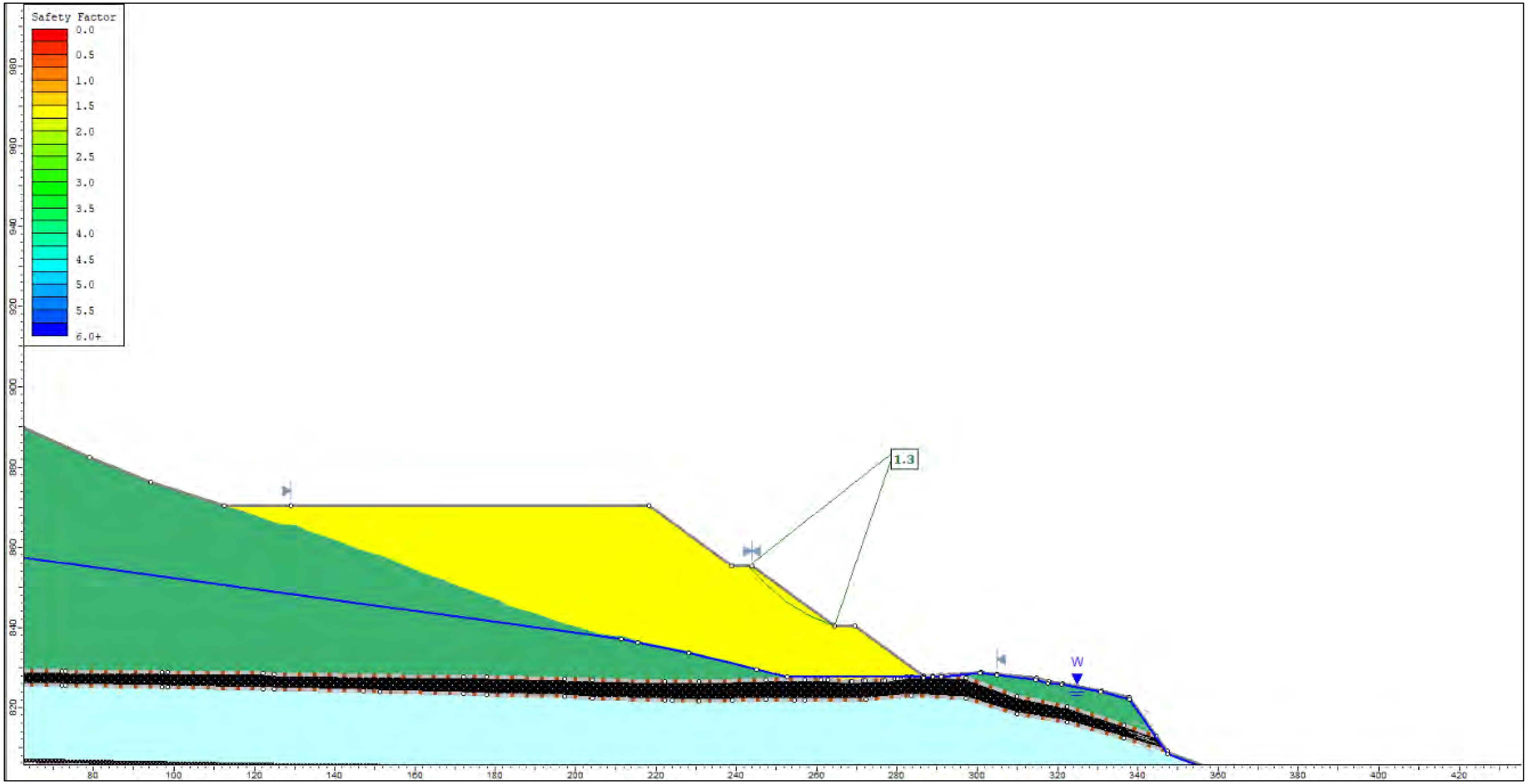

Figure: B58– Slope Stability Analysis of E_ELF3 – Section 101 – Elevated Groundwater Conditions.

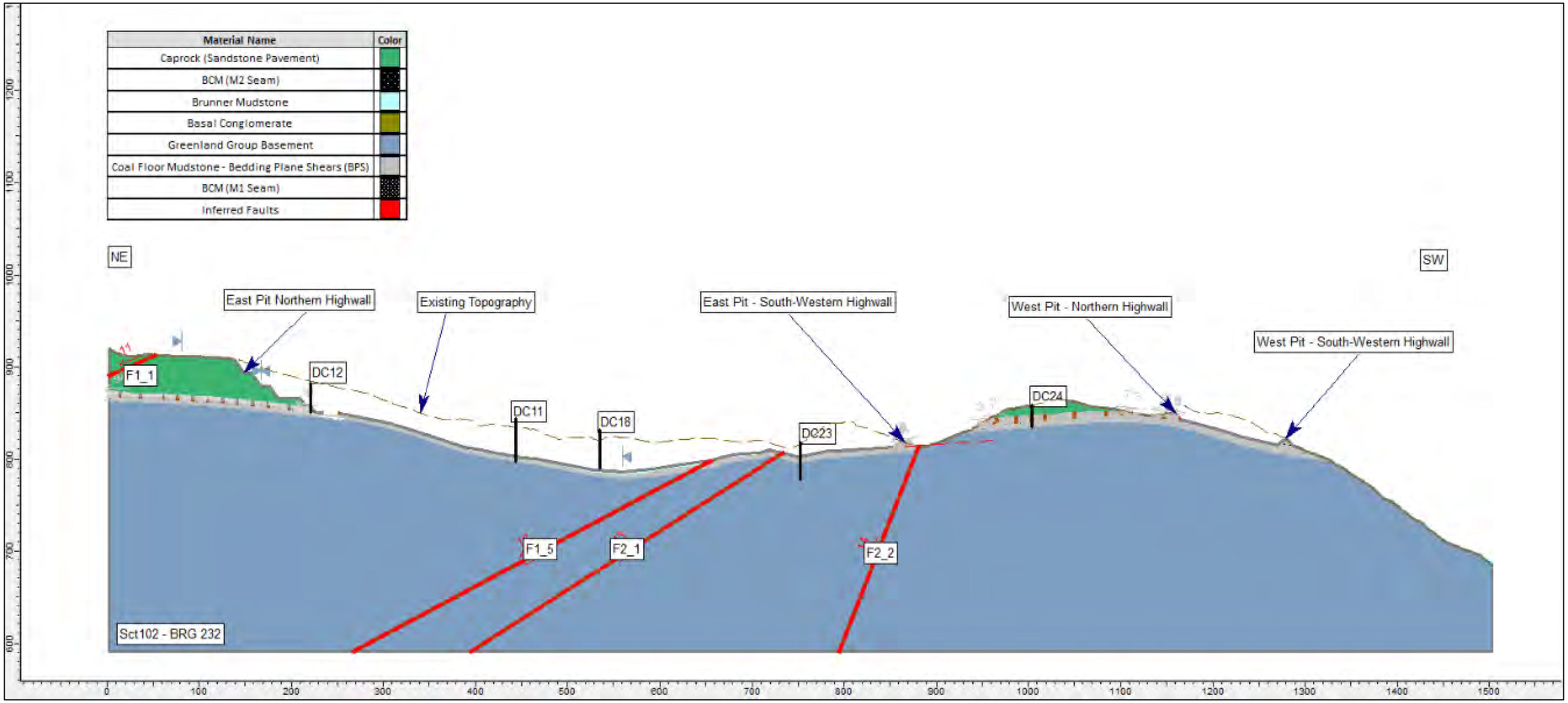

Figure B59 - Section 102 – Mine Feature Overview

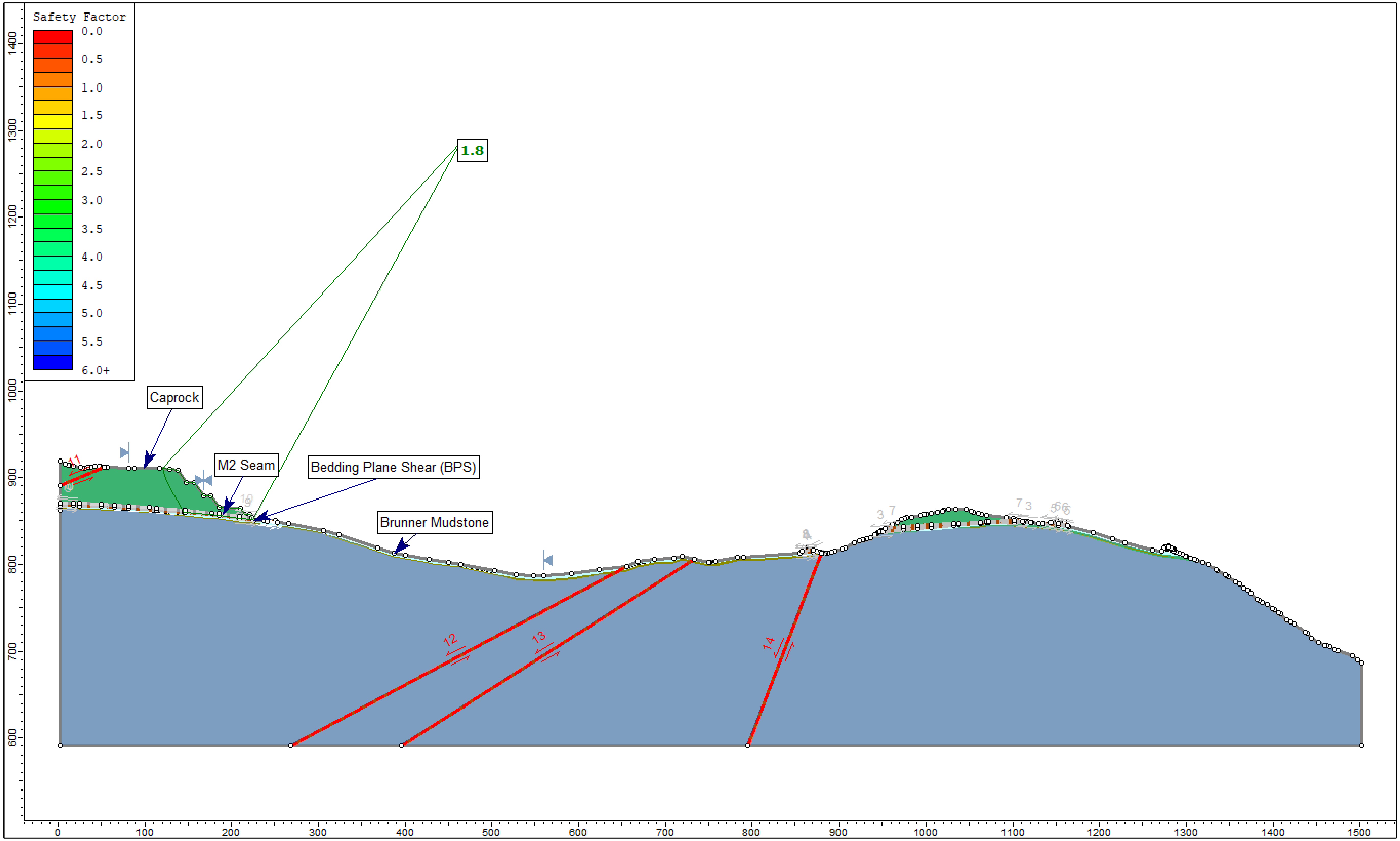

Figure: B60– Slope Stability Analysis of East Pit Northern Highwall– Section 102 – Static Conditions.

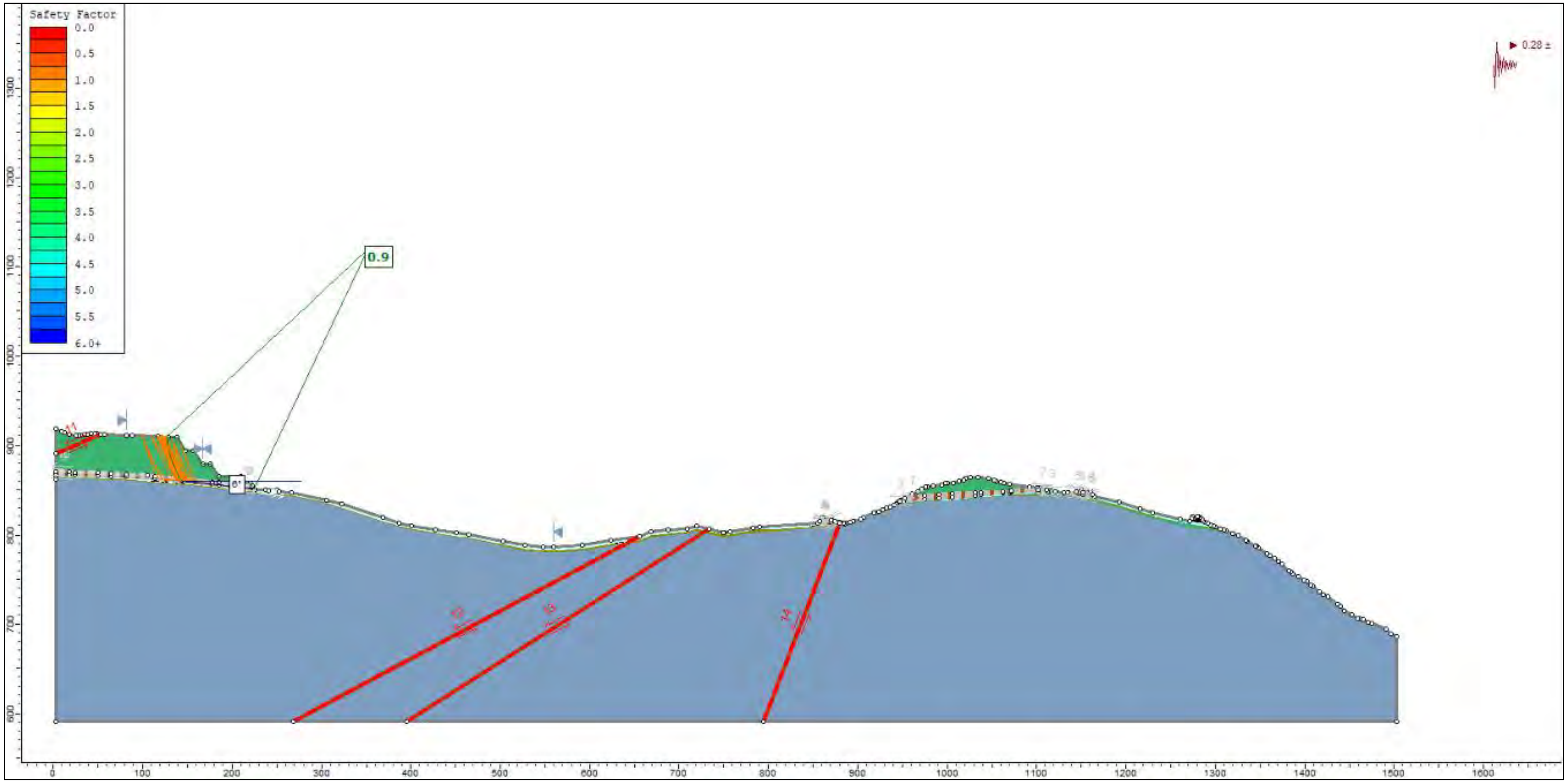

Figure: B61– Slope Stability Analysis of East Pit Northern Highwall– Section 102 – SLS Conditions.

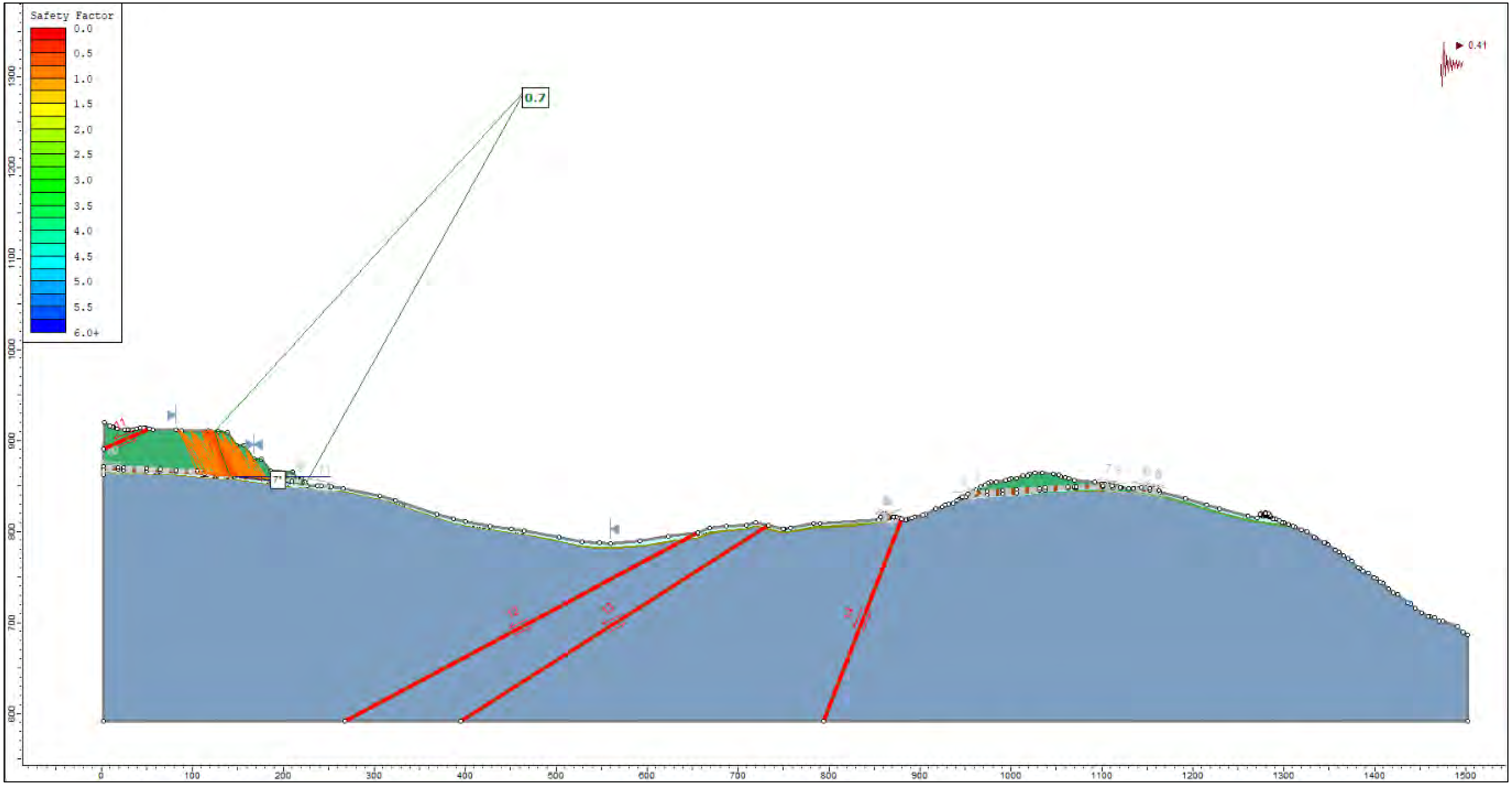

Figure: B62– Slope Stability Analysis of East Pit Northern Highwall– Section 102 – ULS Conditions.

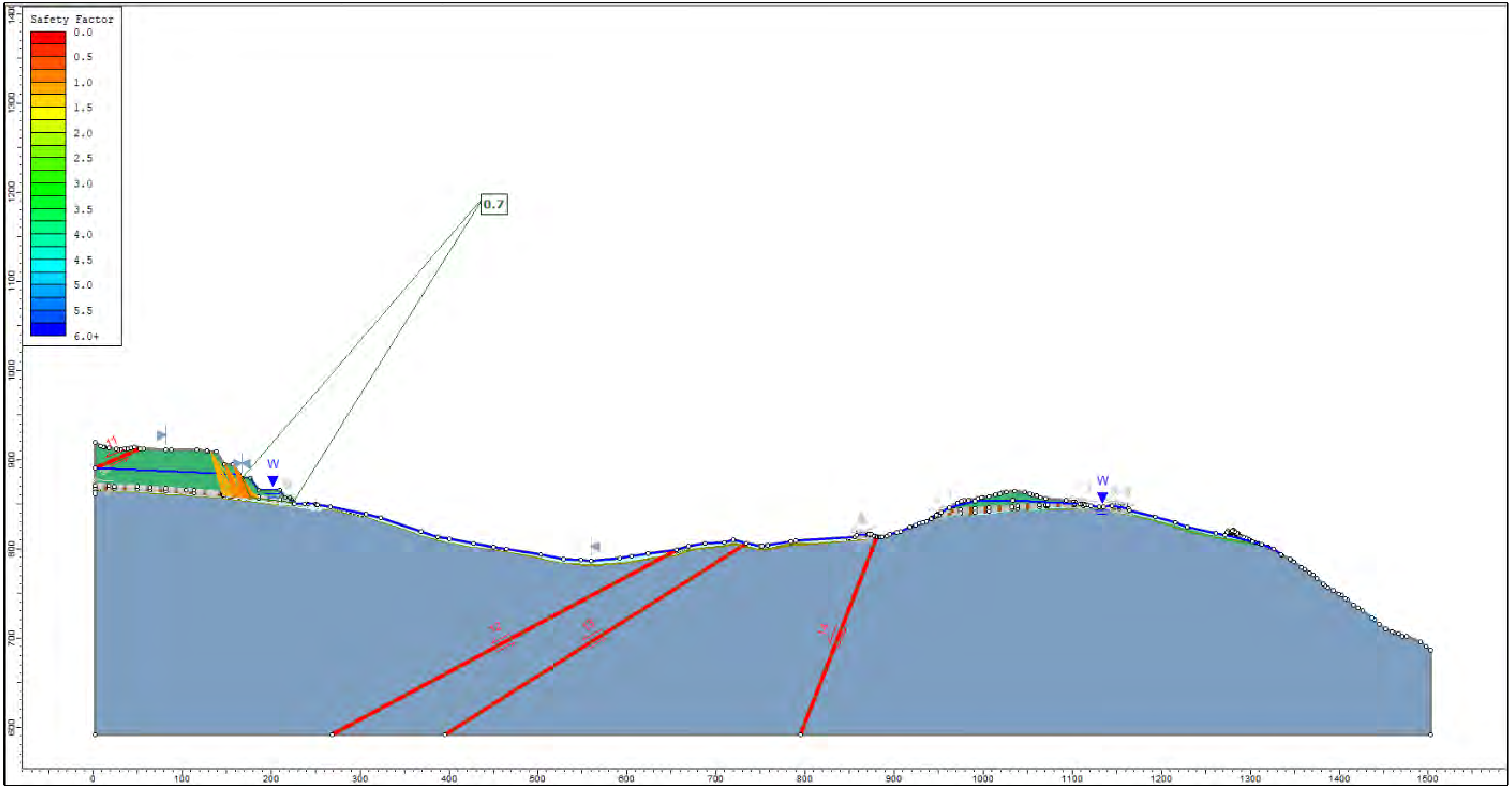

Figure: B63– Slope Stability Analysis of East Pit Northern Highwall– Section 102 – Elevated Groundwater Conditions.

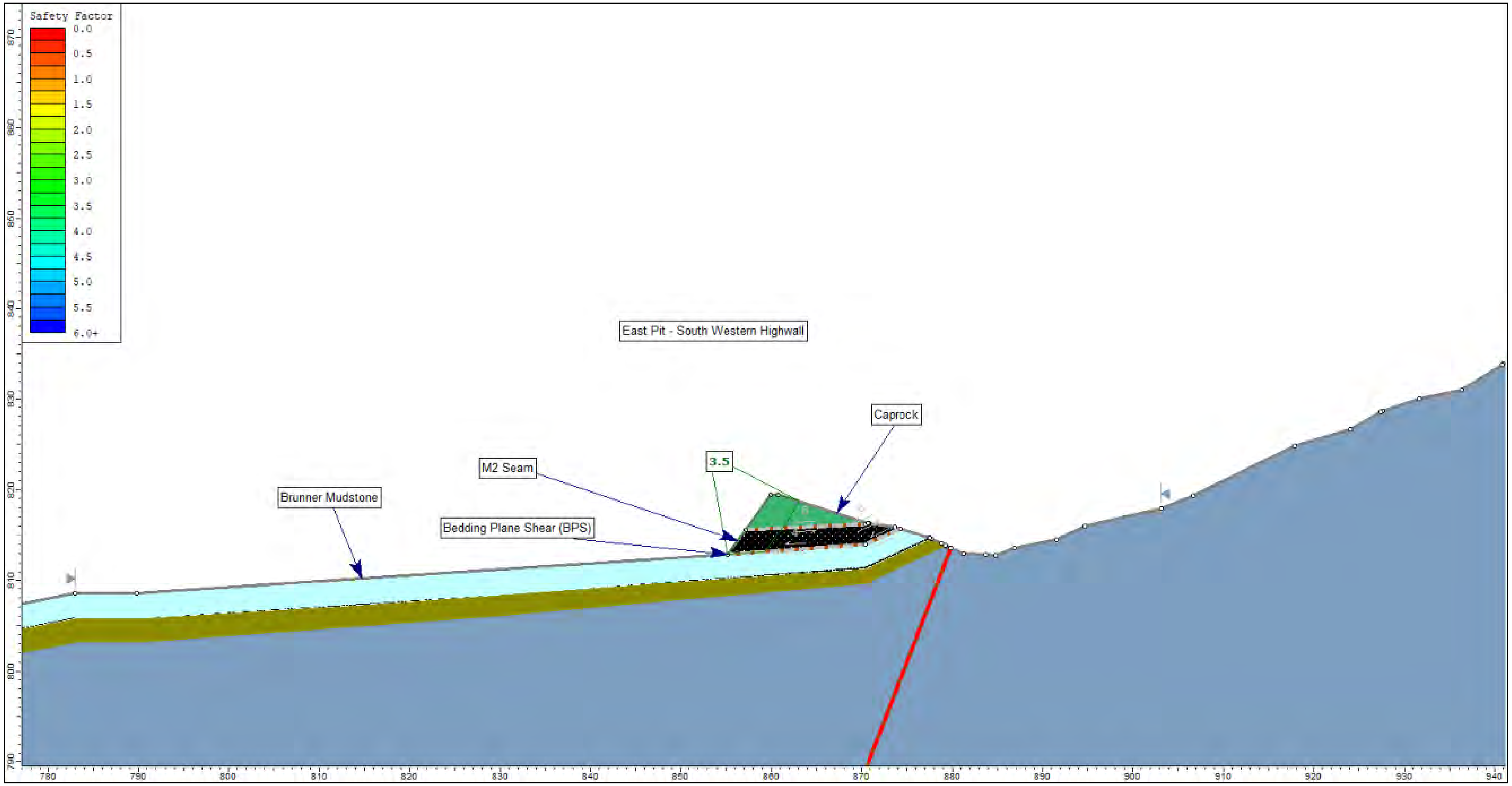

Figure: B64– Slope Stability Analysis of East Pit South-Western Highwall– Section 102 – Static Conditions.

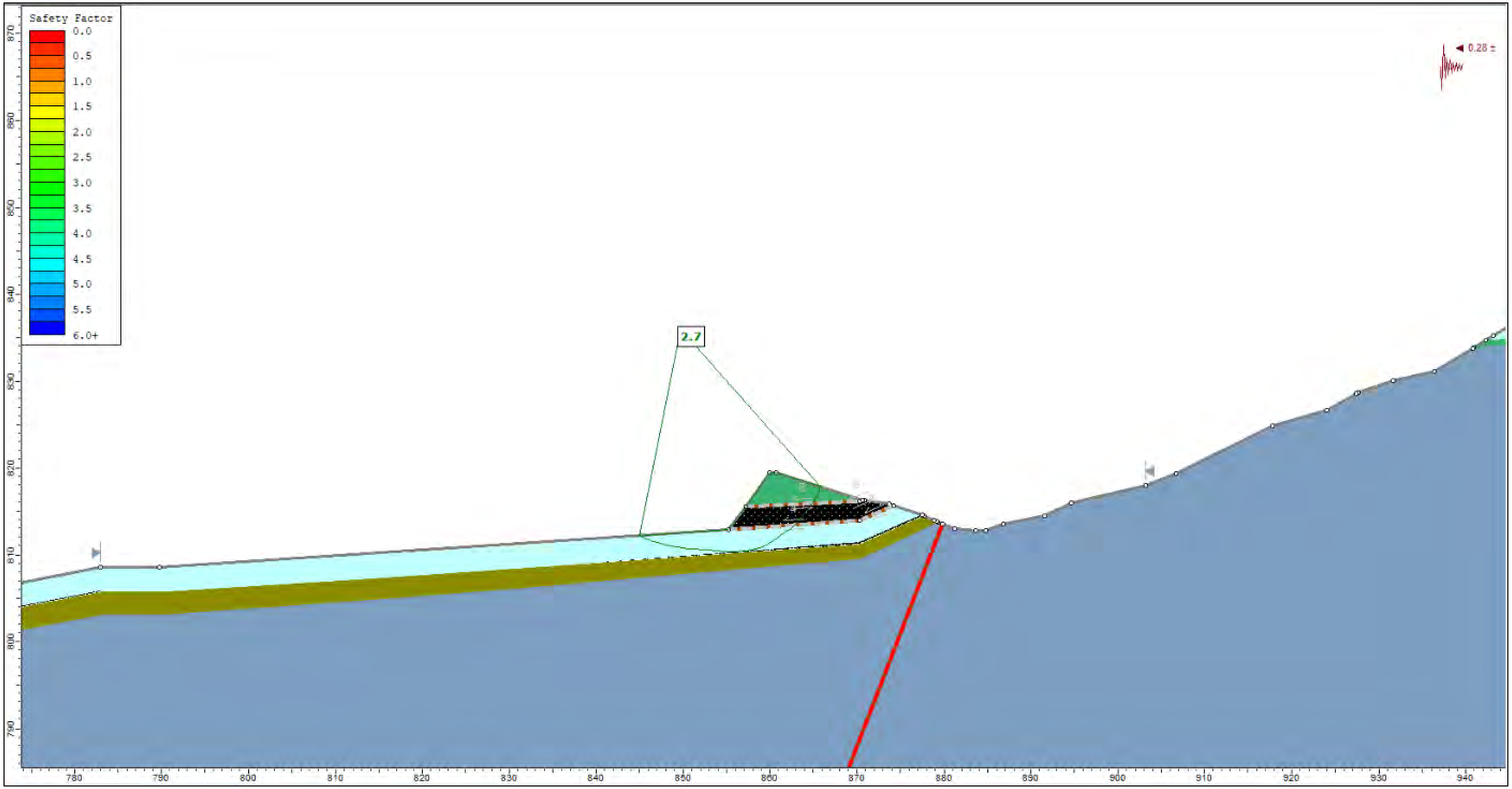

Figure: B65– Slope Stability Analysis of East Pit South-Western Highwall– Section 102 – SLS Conditions.

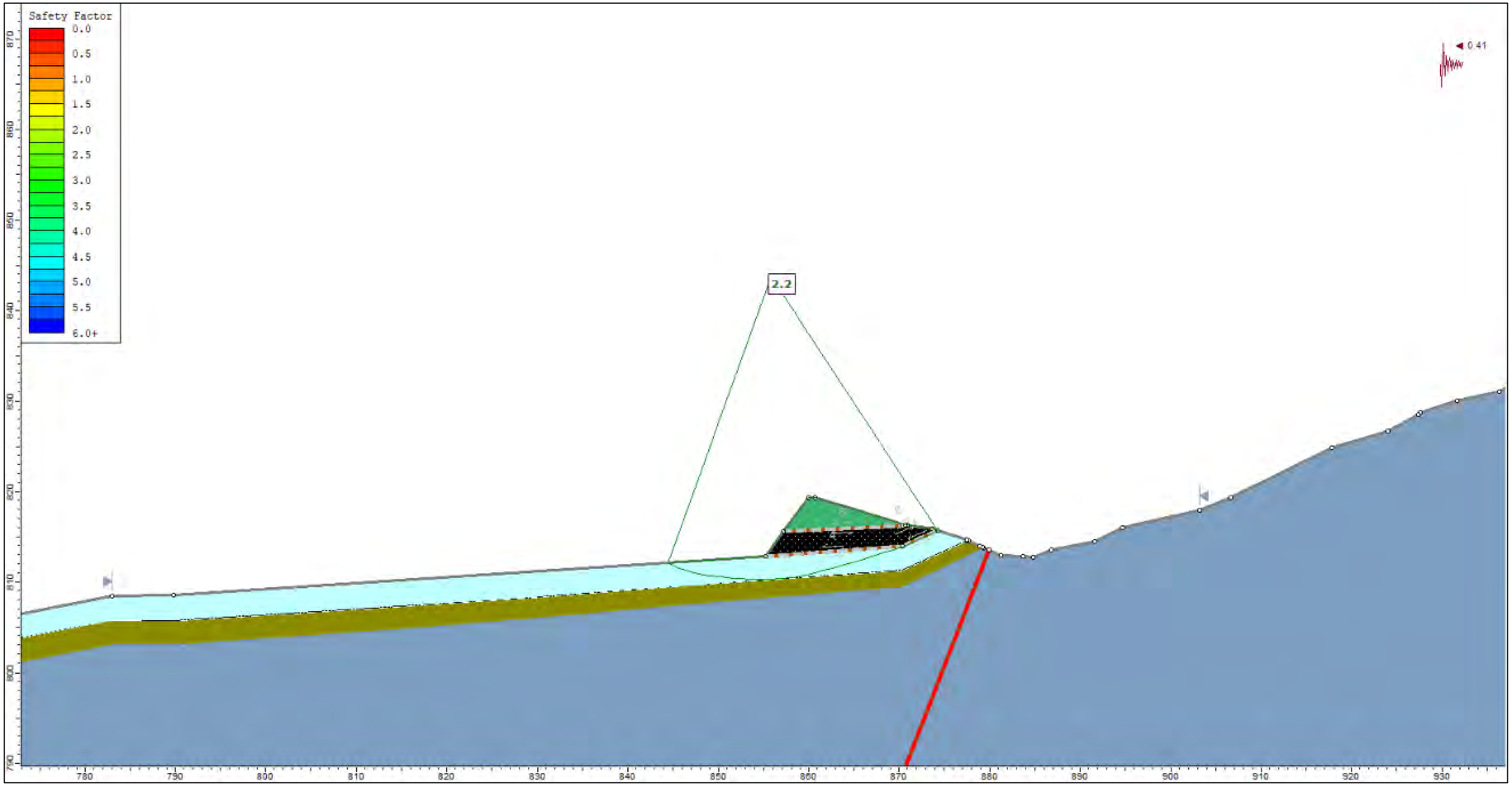

Figure: B66– Slope Stability Analysis of East Pit South-Western Highwall– Section 102 – ULS Conditions.

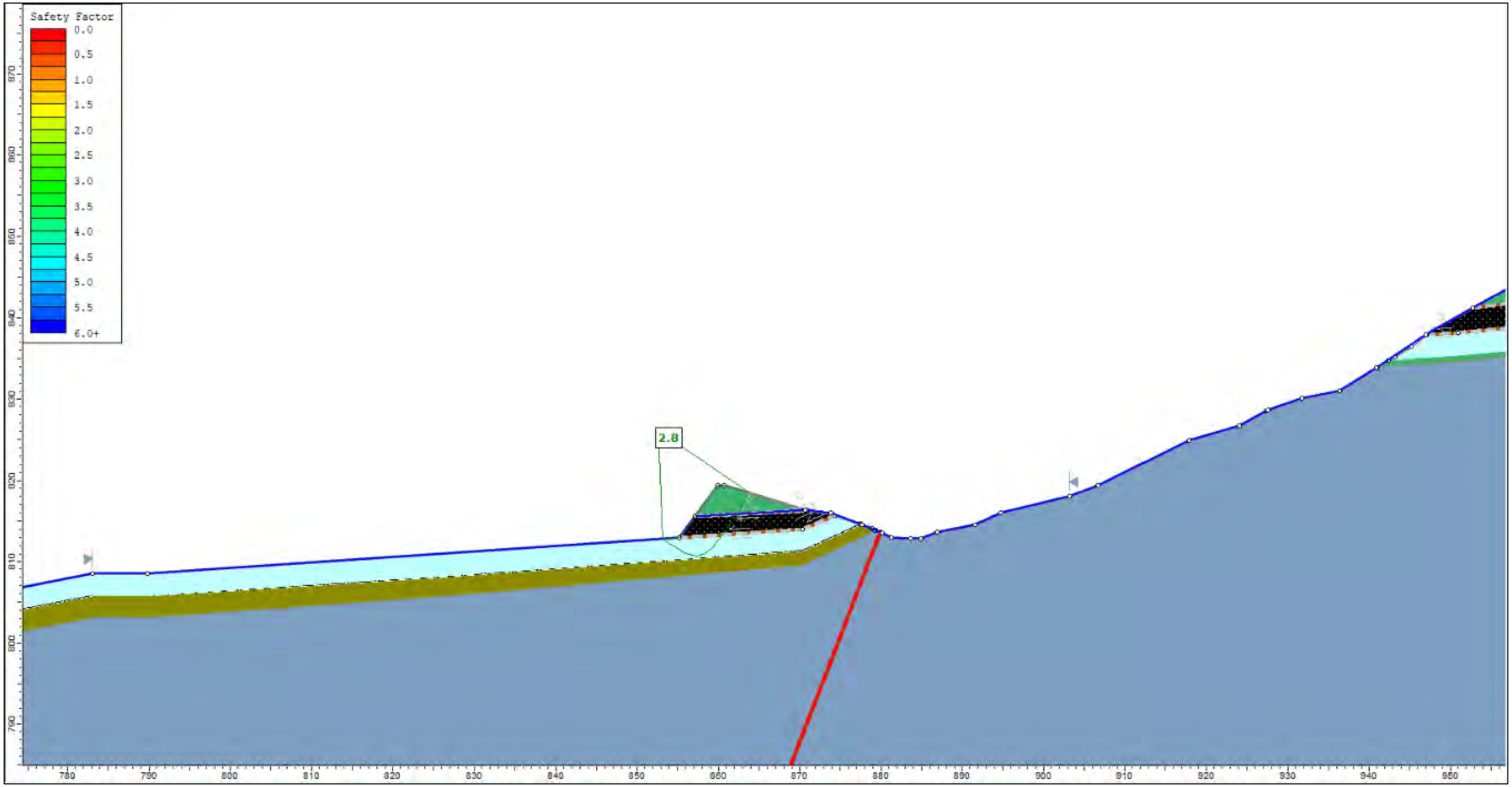

Figure: B67– Slope Stability Analysis of East Pit South-Western Highwall– Section 102 – Elevated Groundwater Conditions.

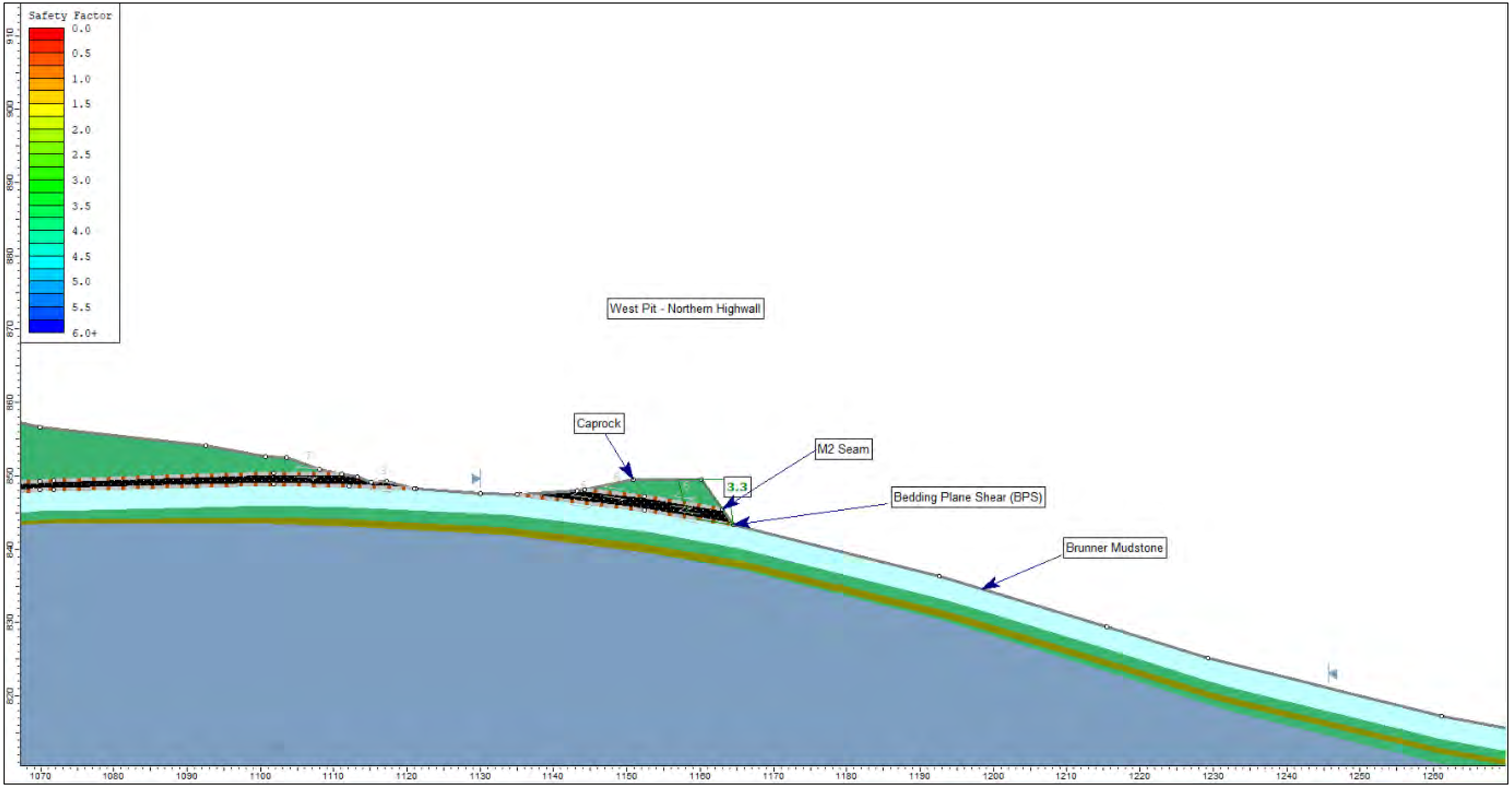

Figure: B68– Slope Stability Analysis of West Pit – Northern Highwall– Section 102 – Static Conditions.

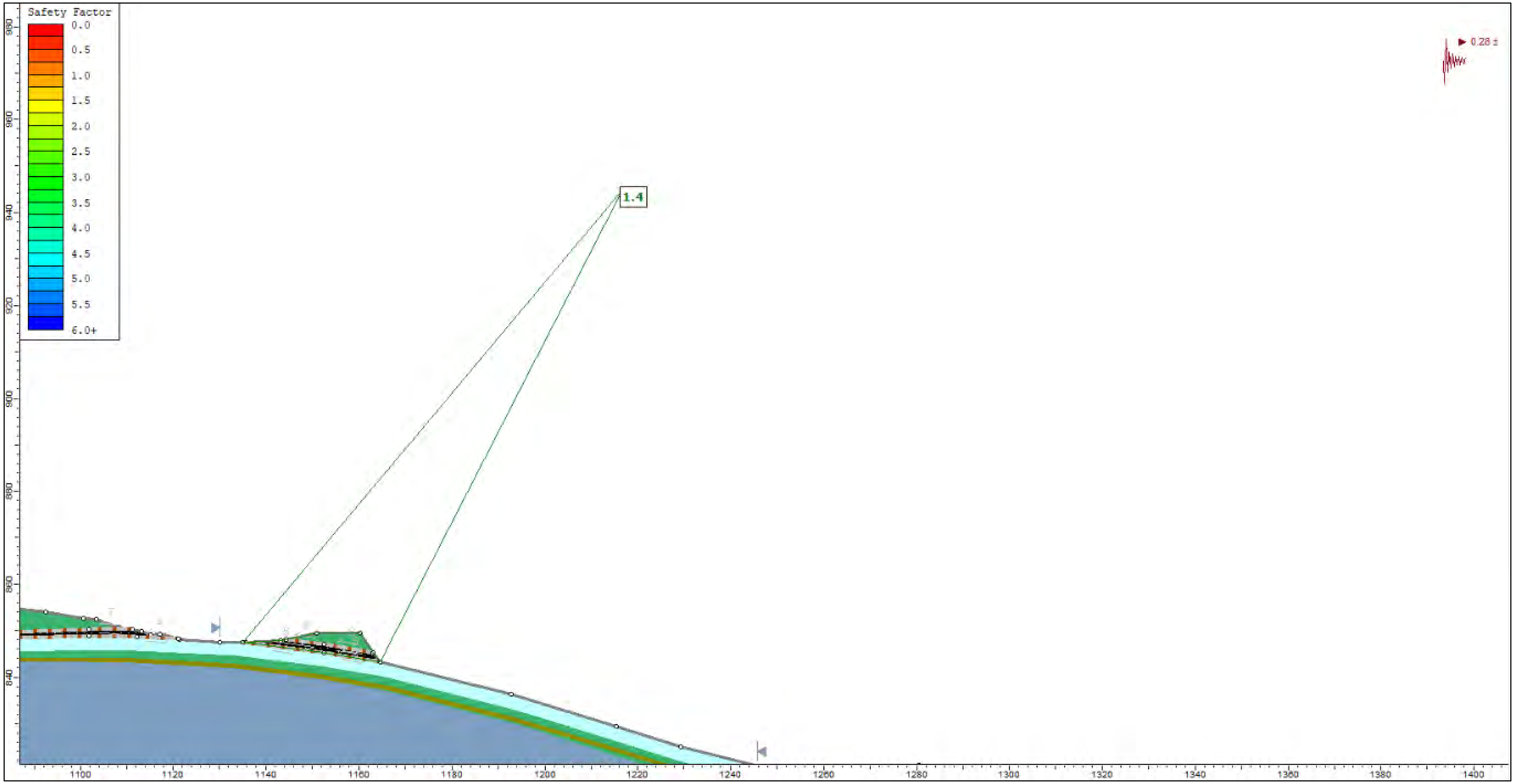

Figure: B69– Slope Stability Analysis of West Pit – Northern Highwall– Section 102 – SLS Conditions.

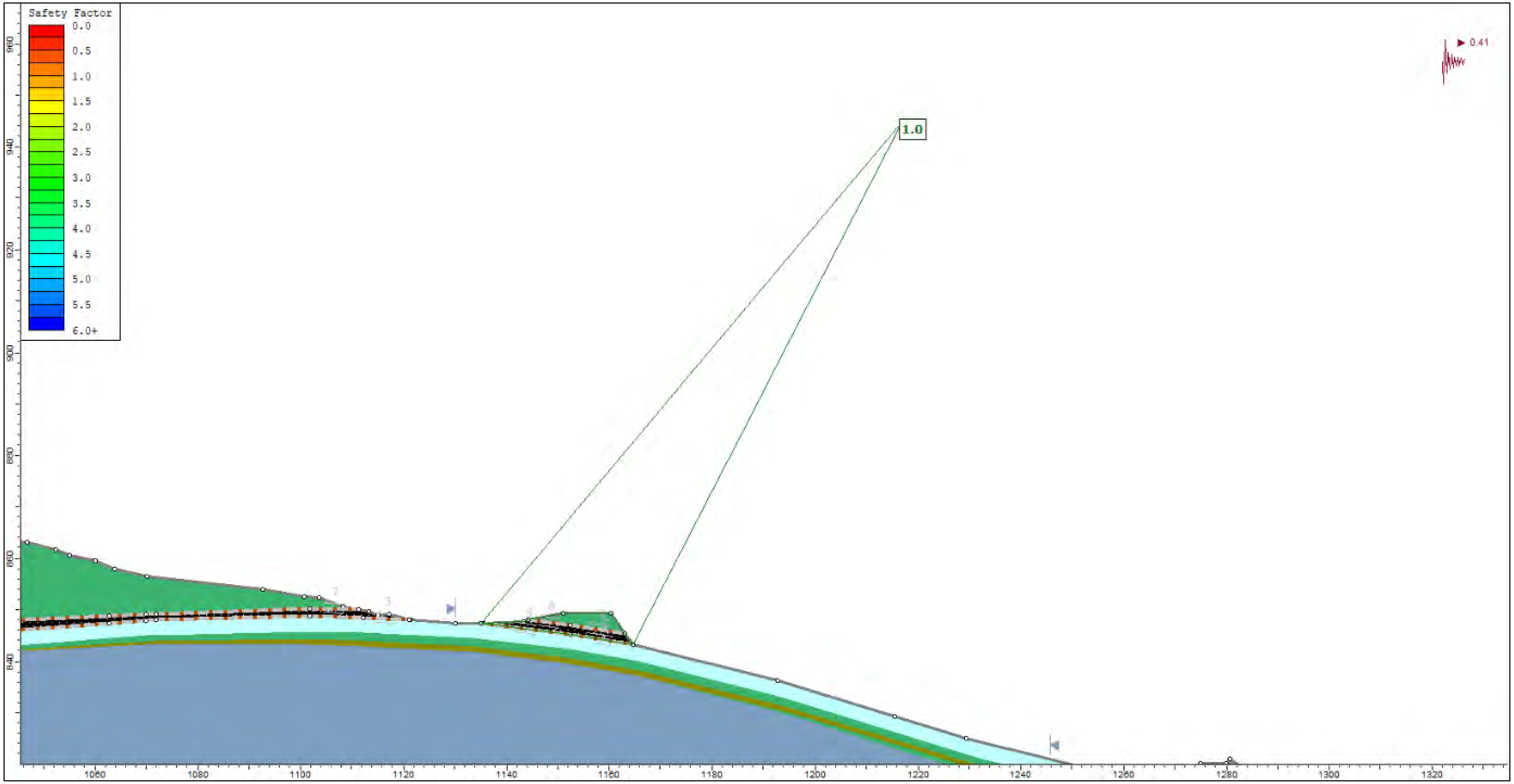

Figure: B70– Slope Stability Analysis of West Pit – Northern Highwall– Section 102 – ULS Conditions.

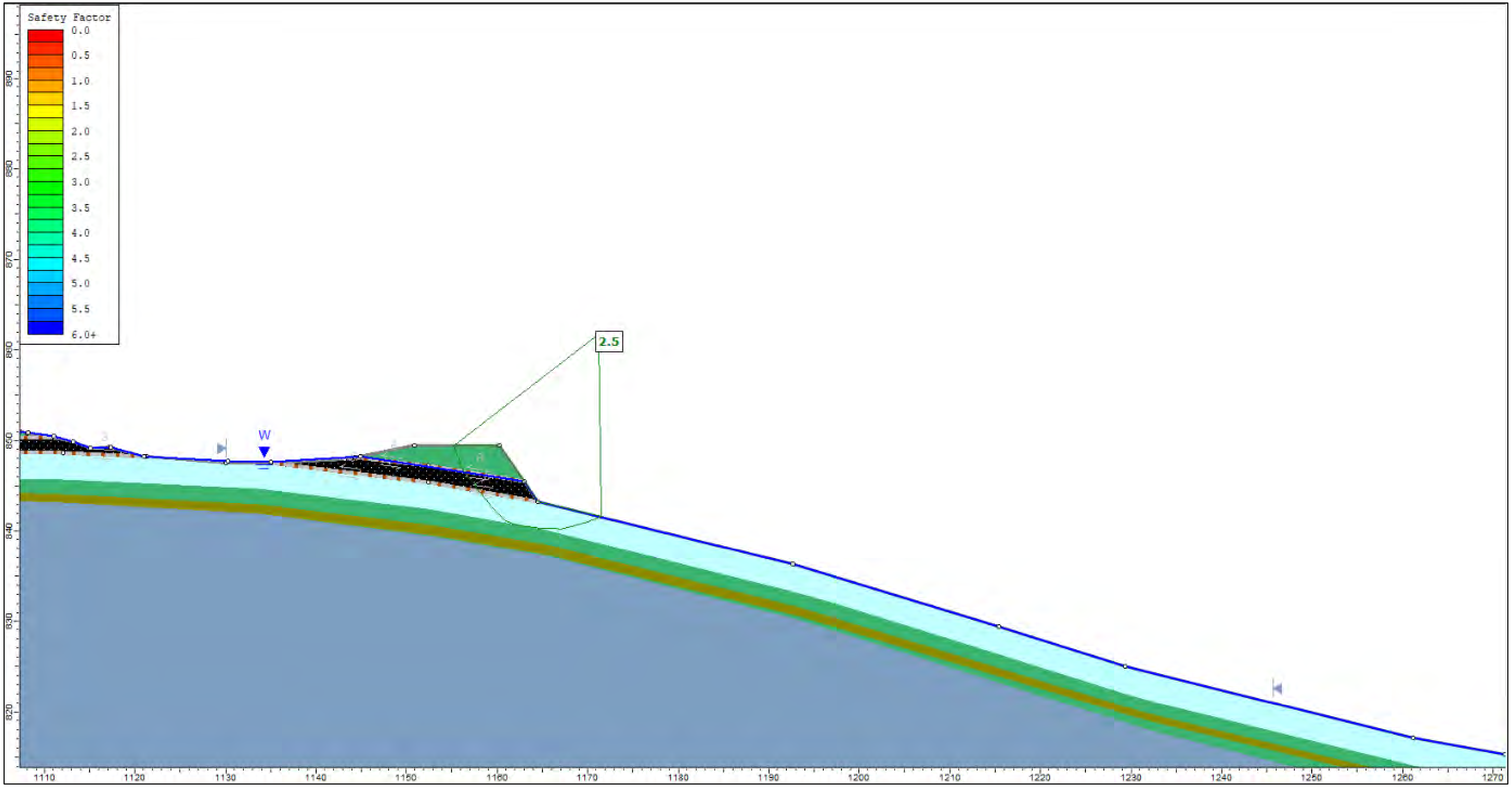

Figure: B71– Slope Stability Analysis of West Pit – Northern Highwall– Section 102 – Elevated Groundwater Conditions.

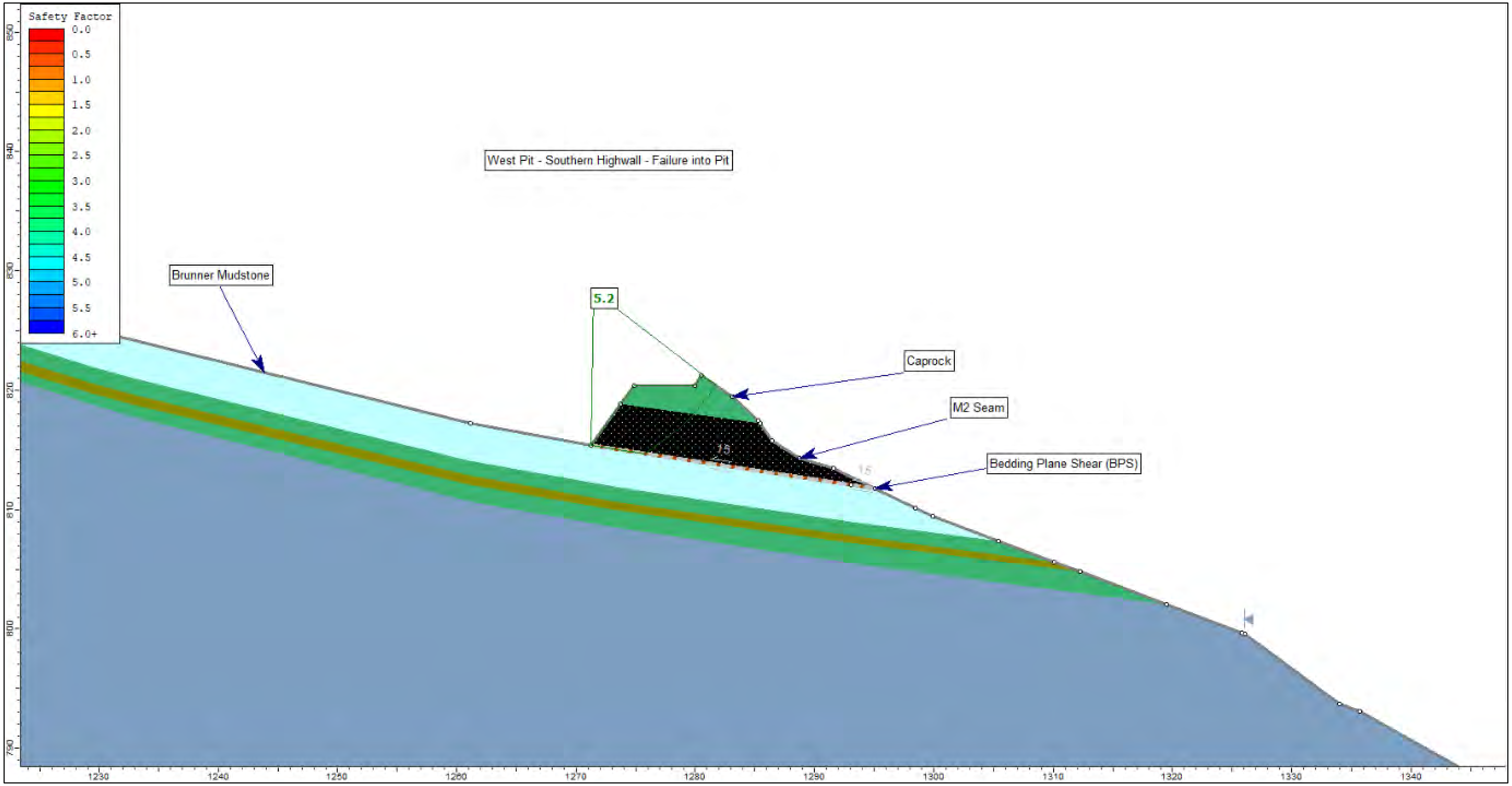

Figure: B72– Slope Stability Analysis of West Pit – Southern Highwall – Failure into Pit– Section 102 – Static Conditions.

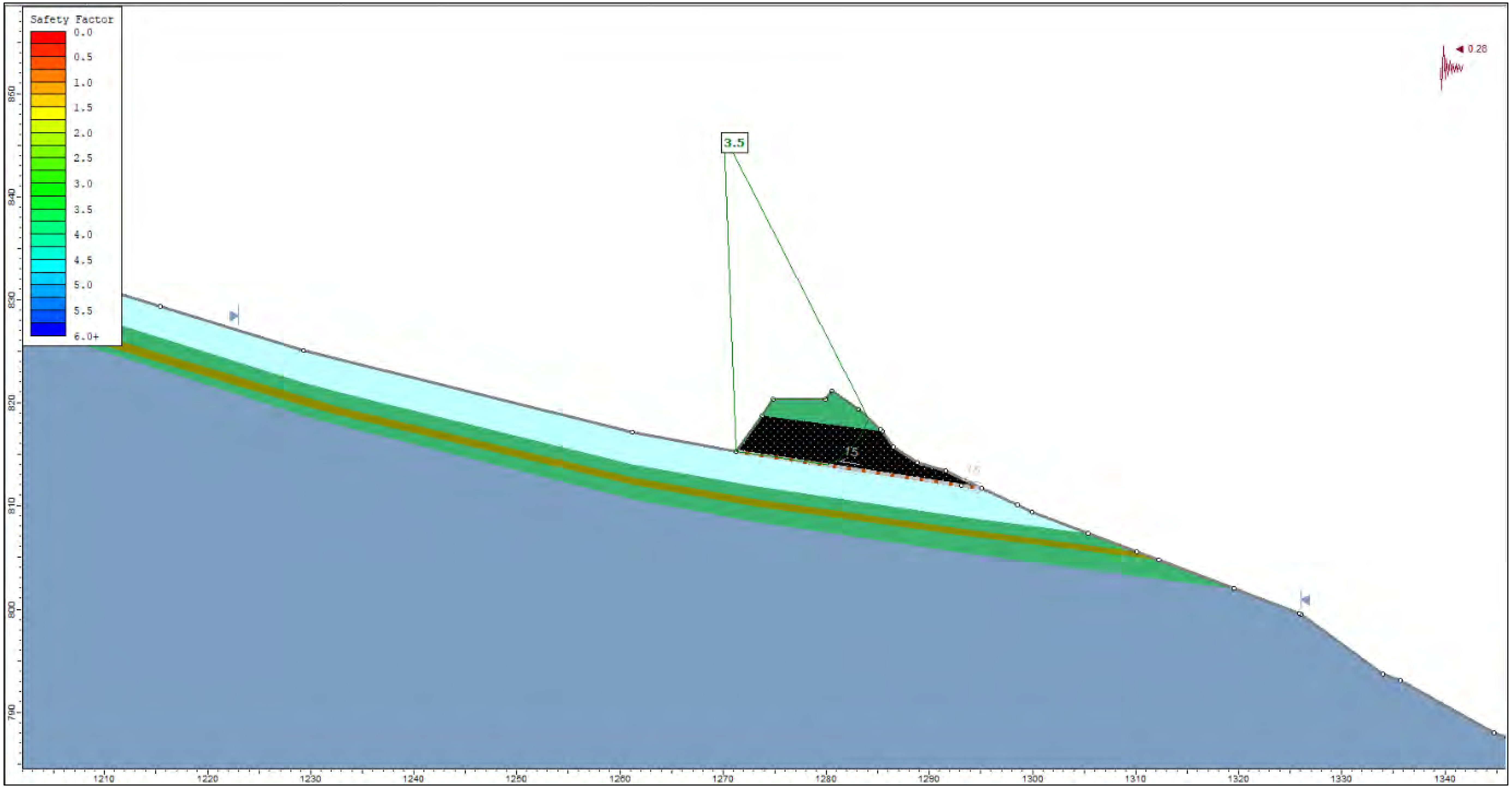

Figure: B73– Slope Stability Analysis of West Pit – Southern Highwall – Failure into Pit– Section 102 – SLS Conditions.

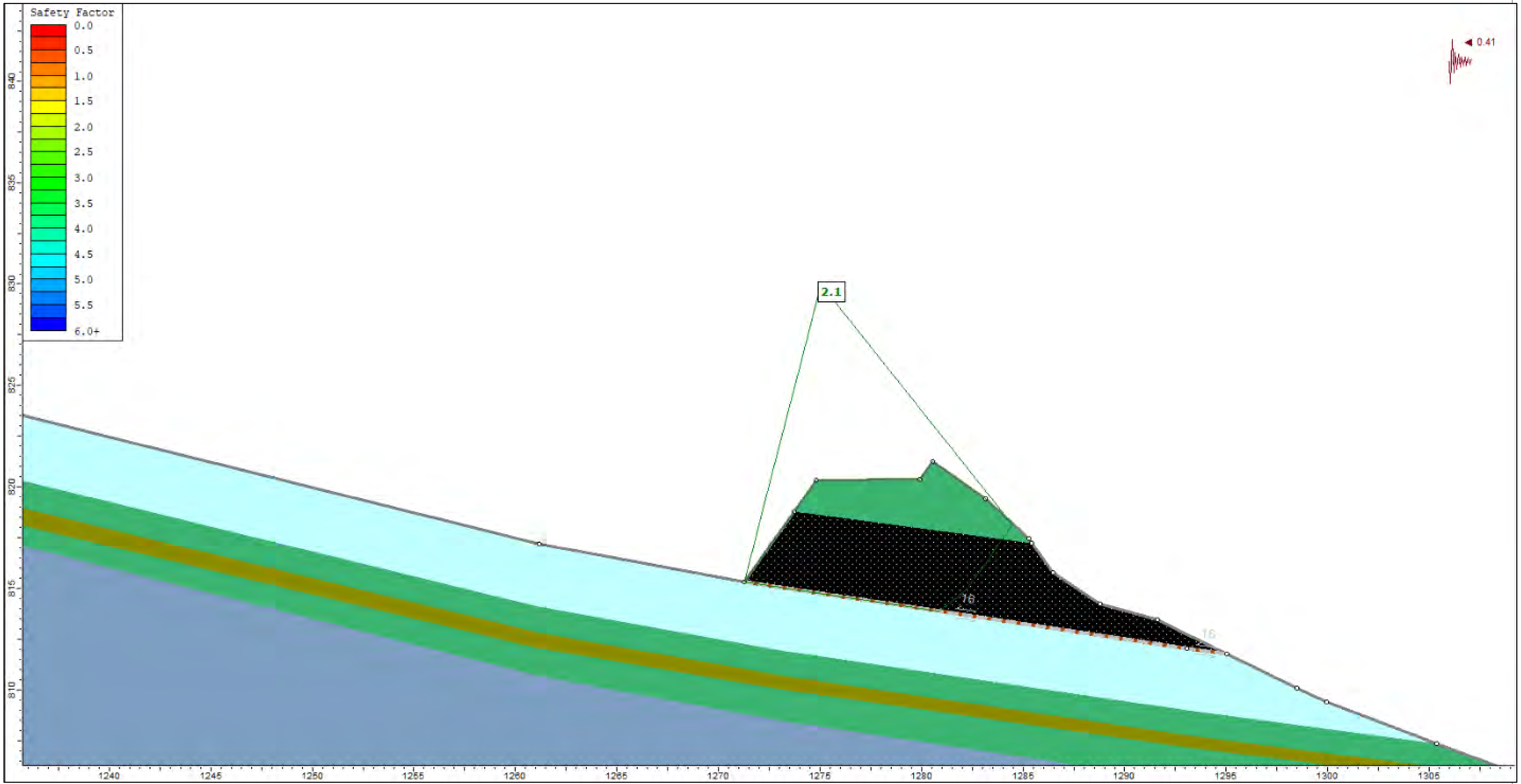

Figure: B74– Slope Stability Analysis of West Pit – Southern Highwall – Failure into Pit– Section 102 – ULS Conditions.

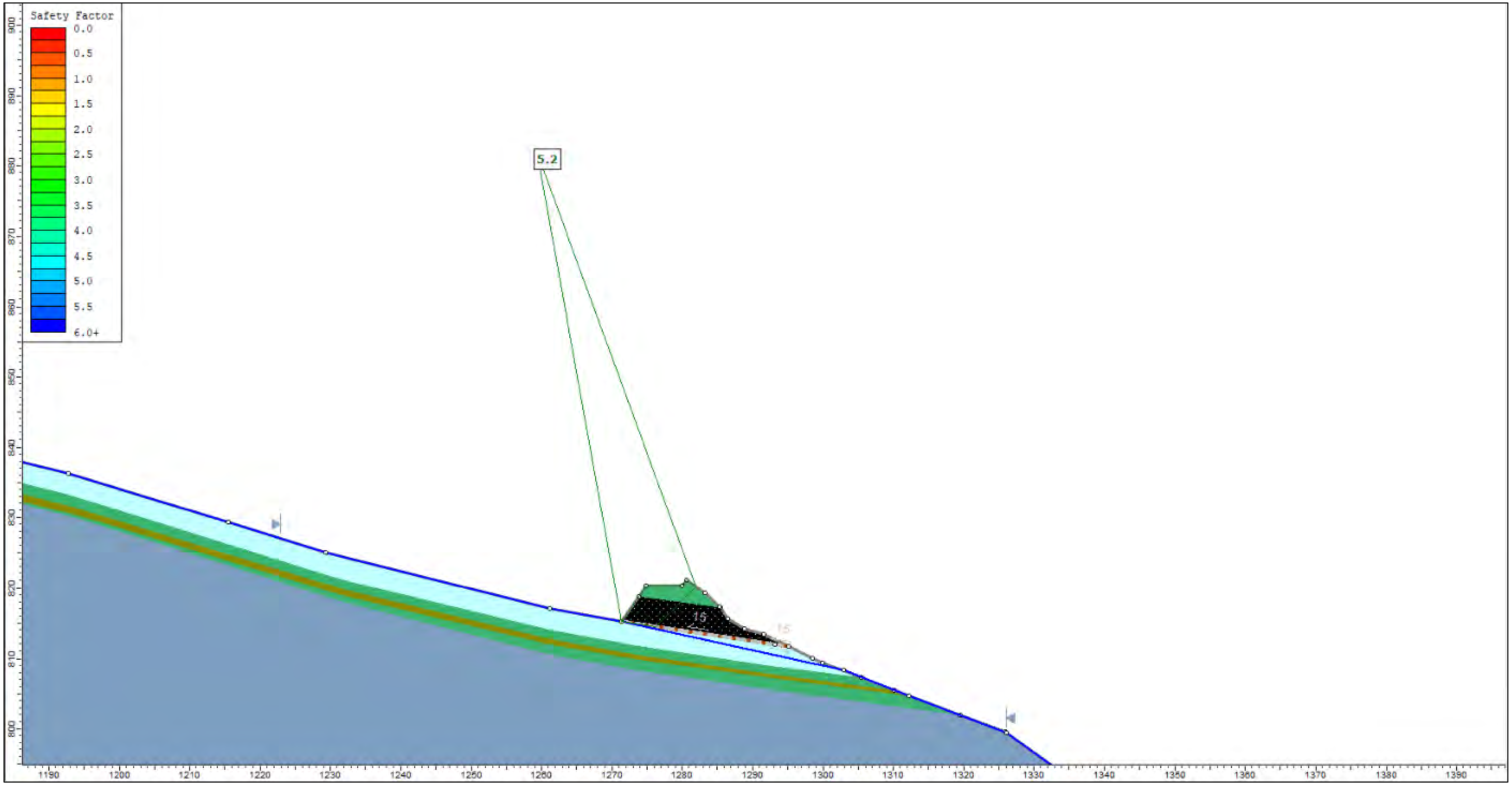

Figure: B75– Slope Stability Analysis of West Pit – Southern Highwall – Failure into Pit– Section 102 – Elevated Groundwater Conditions.

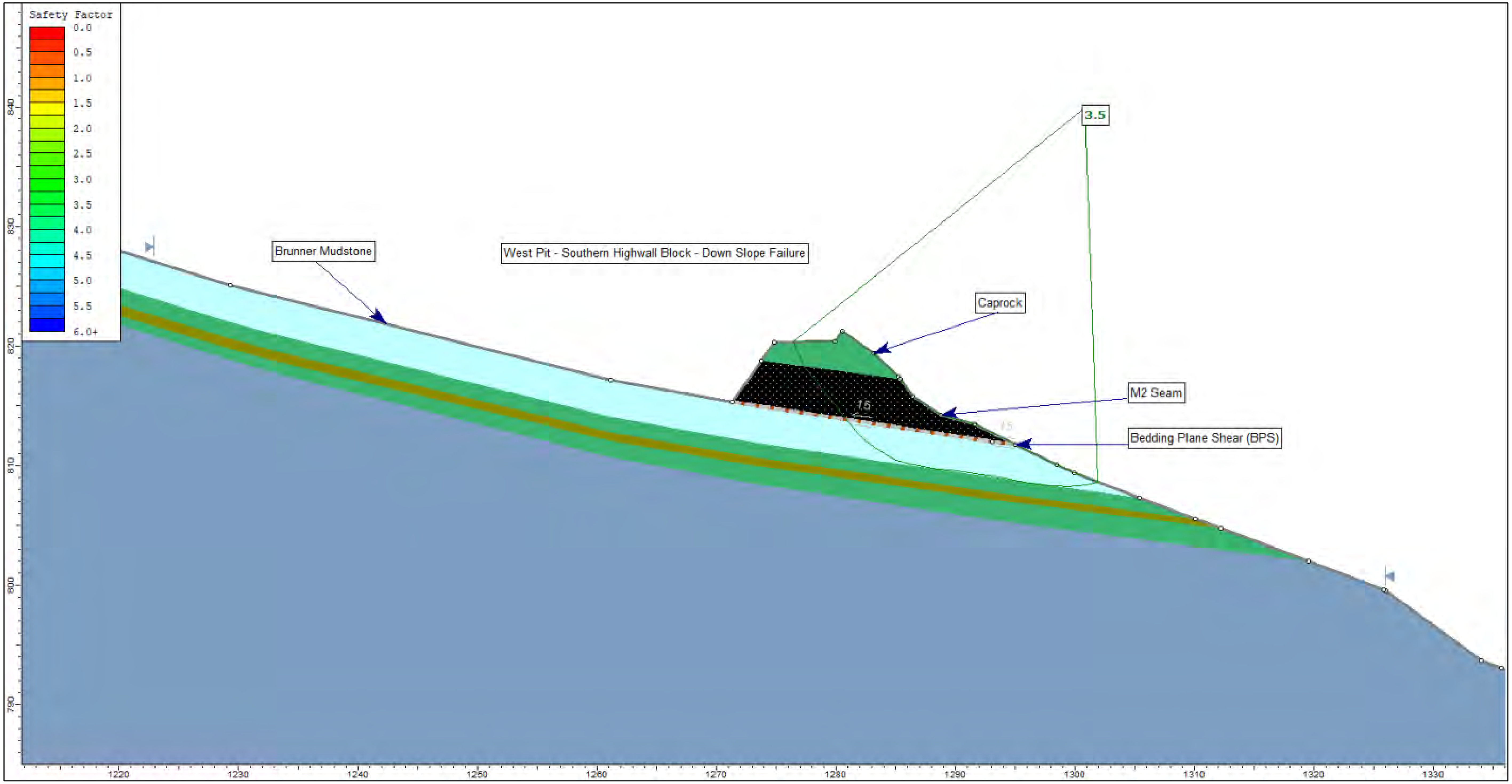

Figure: B76– Slope Stability Analysis of West Pit – Southern Highwall – Downslope Failure– Section 102 – Static Conditions.

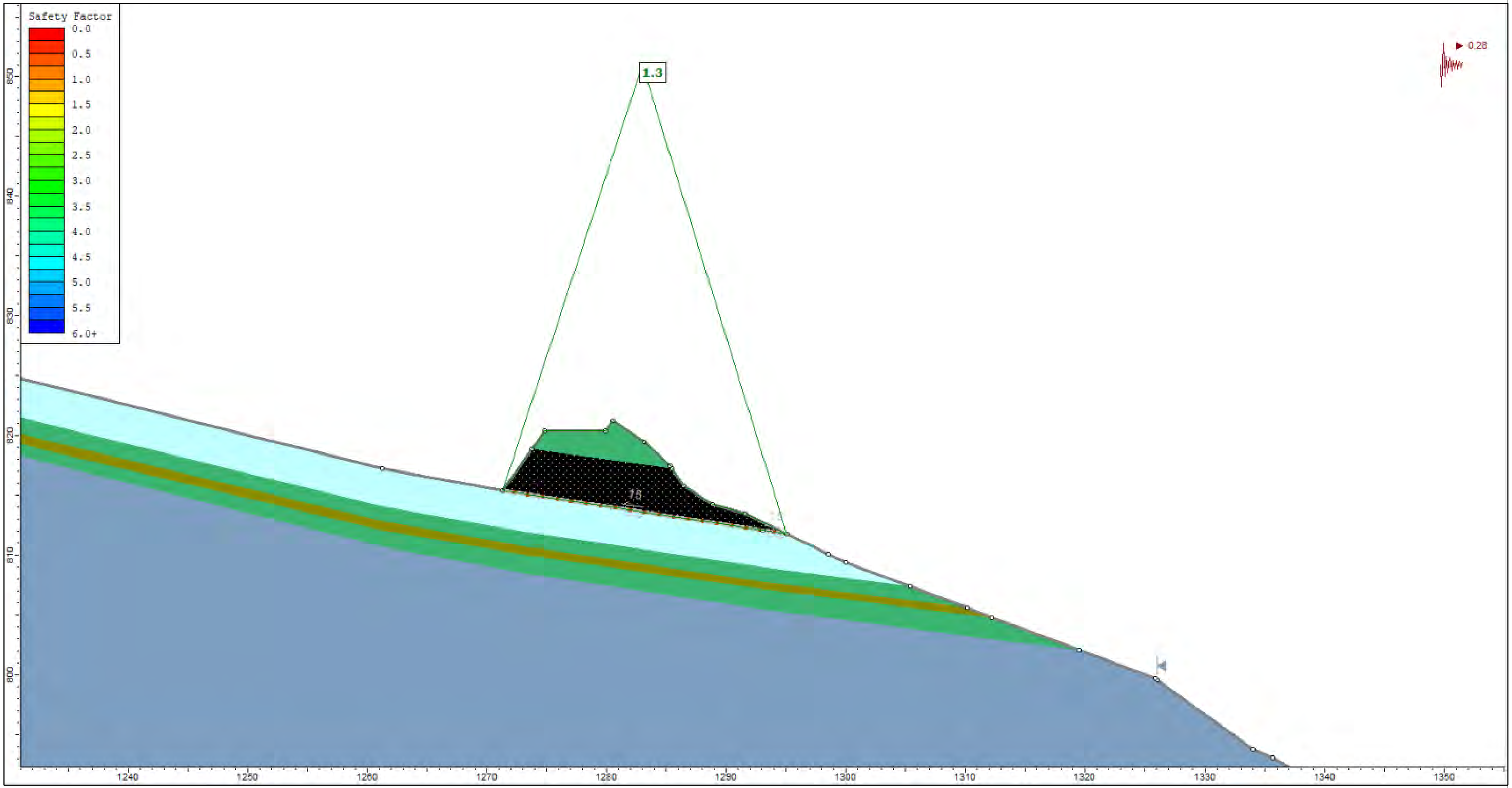

Figure: B77– Slope Stability Analysis of West Pit – Southern Highwall – Failure into Pit– Section 102 – SLS Conditions.

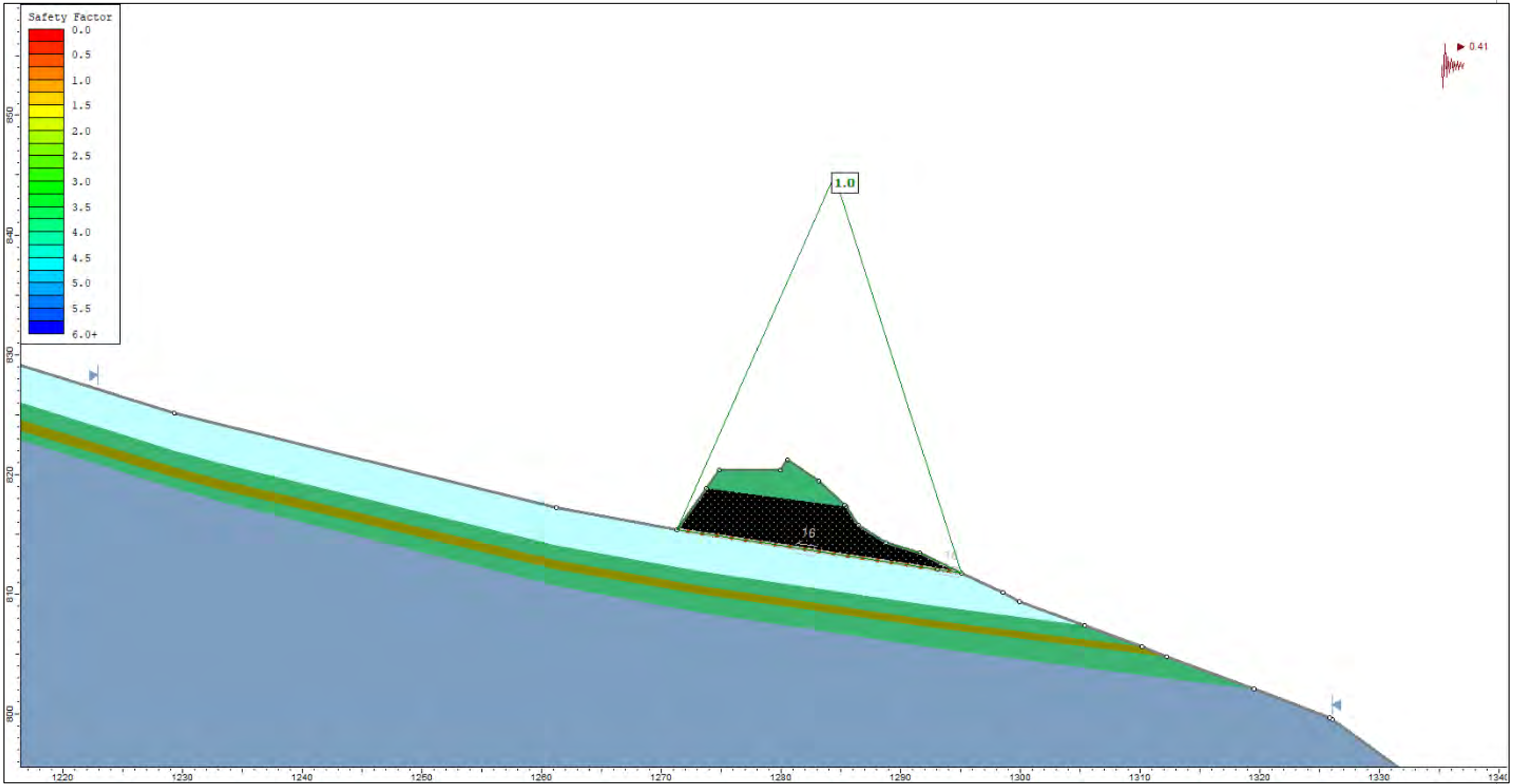

Figure: B78– Slope Stability Analysis of West Pit – Southern Highwall – Failure into Pit– Section 102 –ULS Conditions.

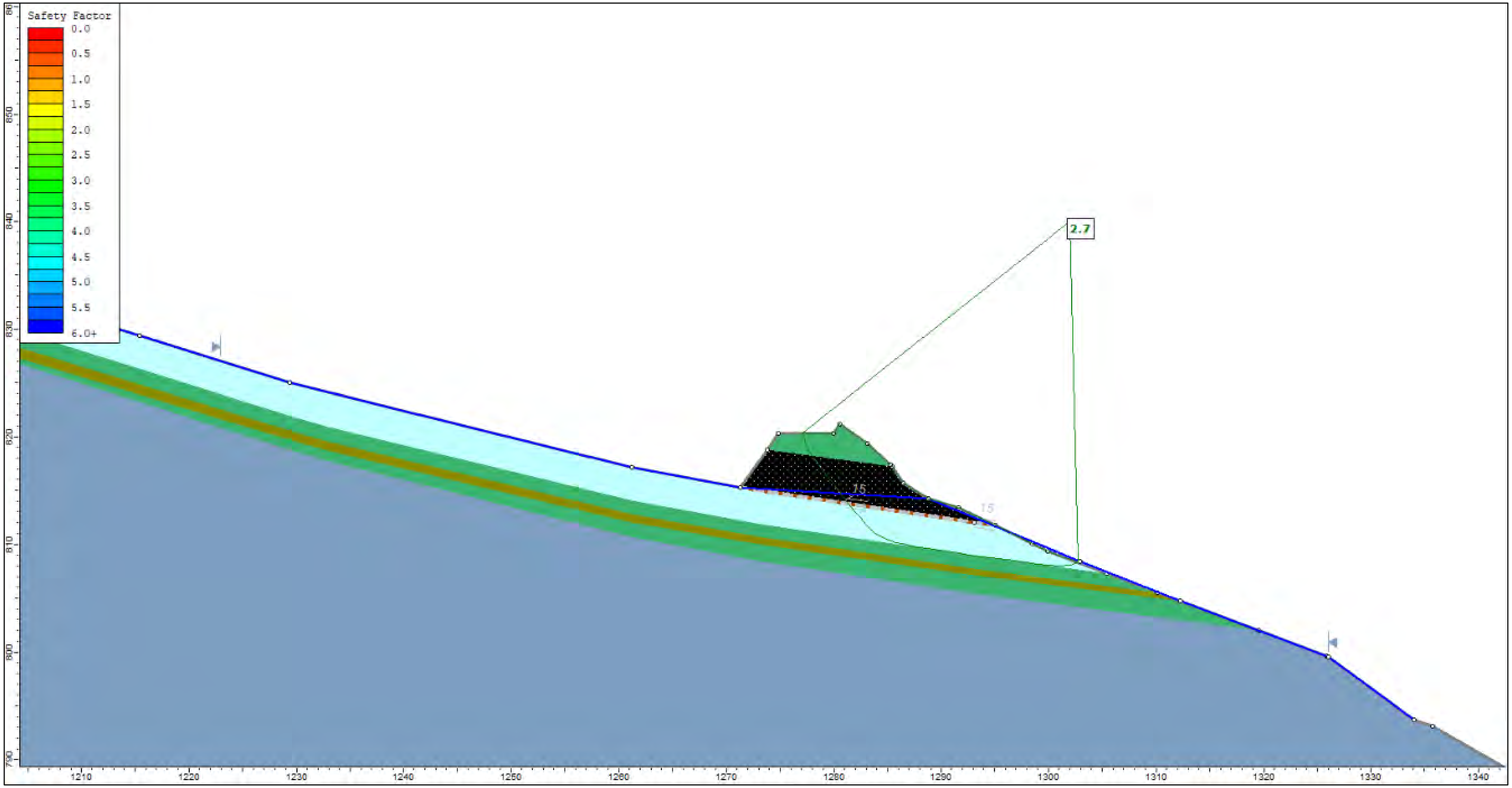

Figure: B79– Slope Stability Analysis of West Pit – Southern Highwall – Failure into Pit– Section 102 – Elevated Groundwater Condition

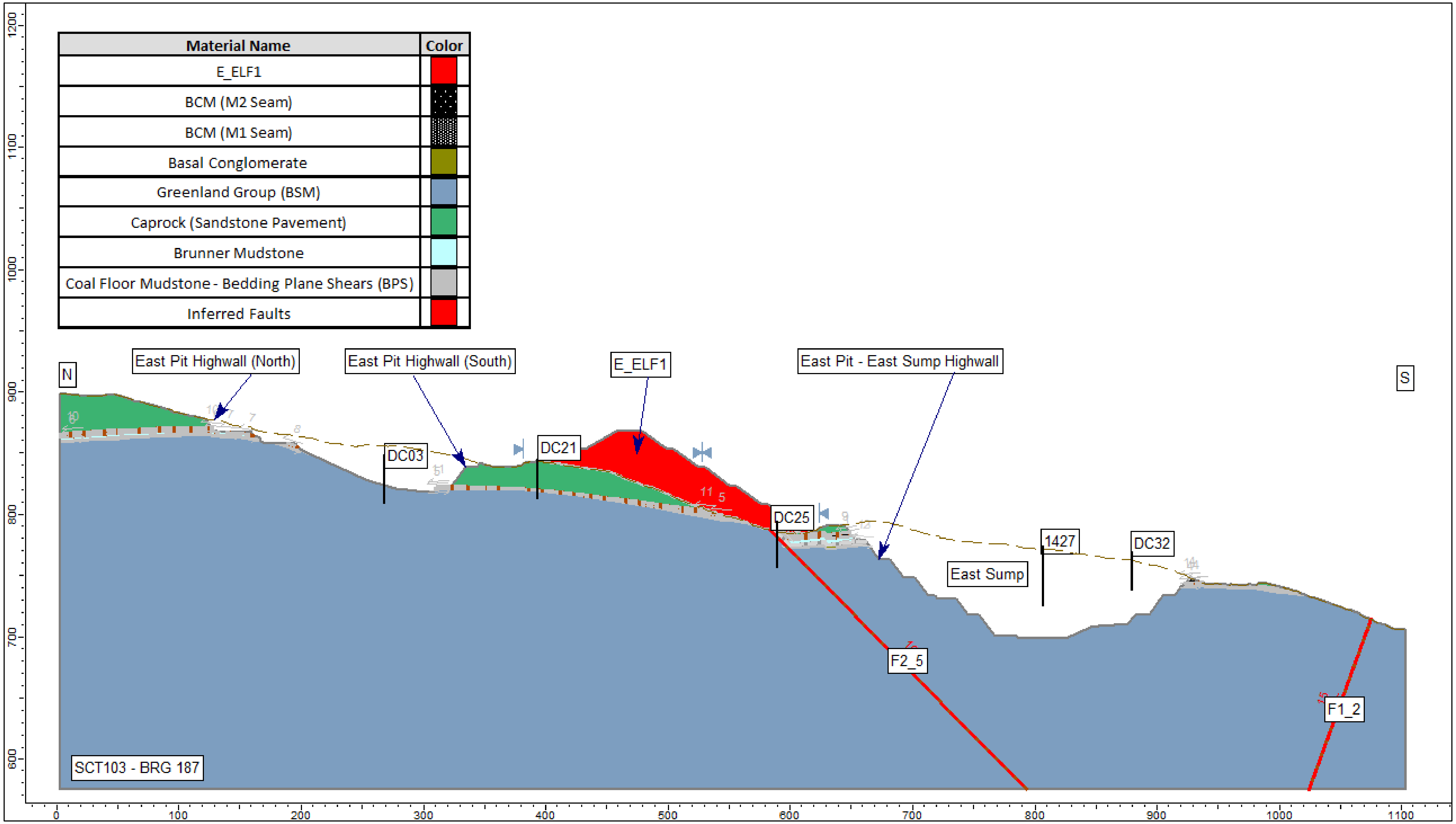

Figure B80– Section 103 – Mine Feature Overview.

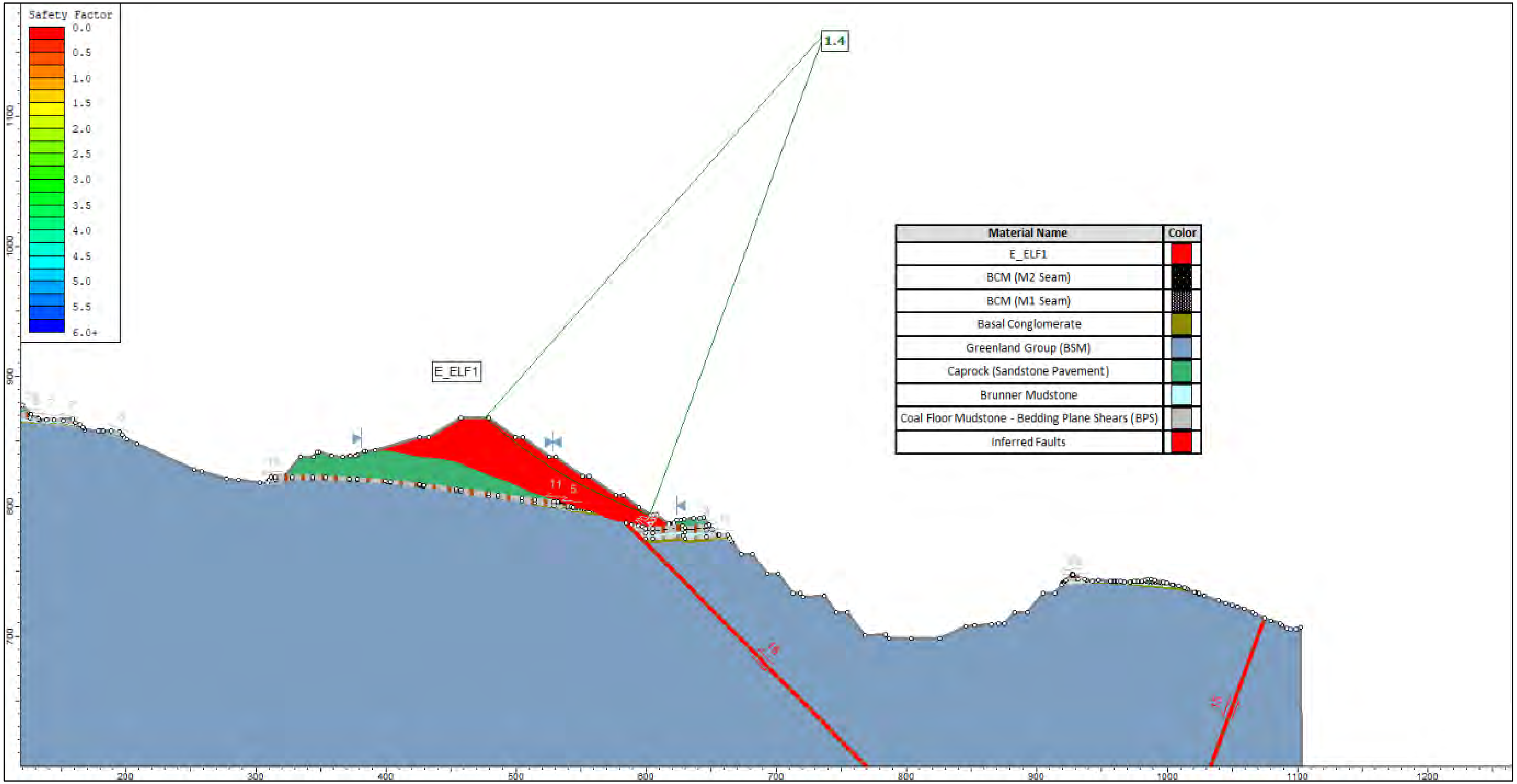

Figure: B81 Slope Stability Analysis- E_ELF1 - Section 103 — Static Conditions.

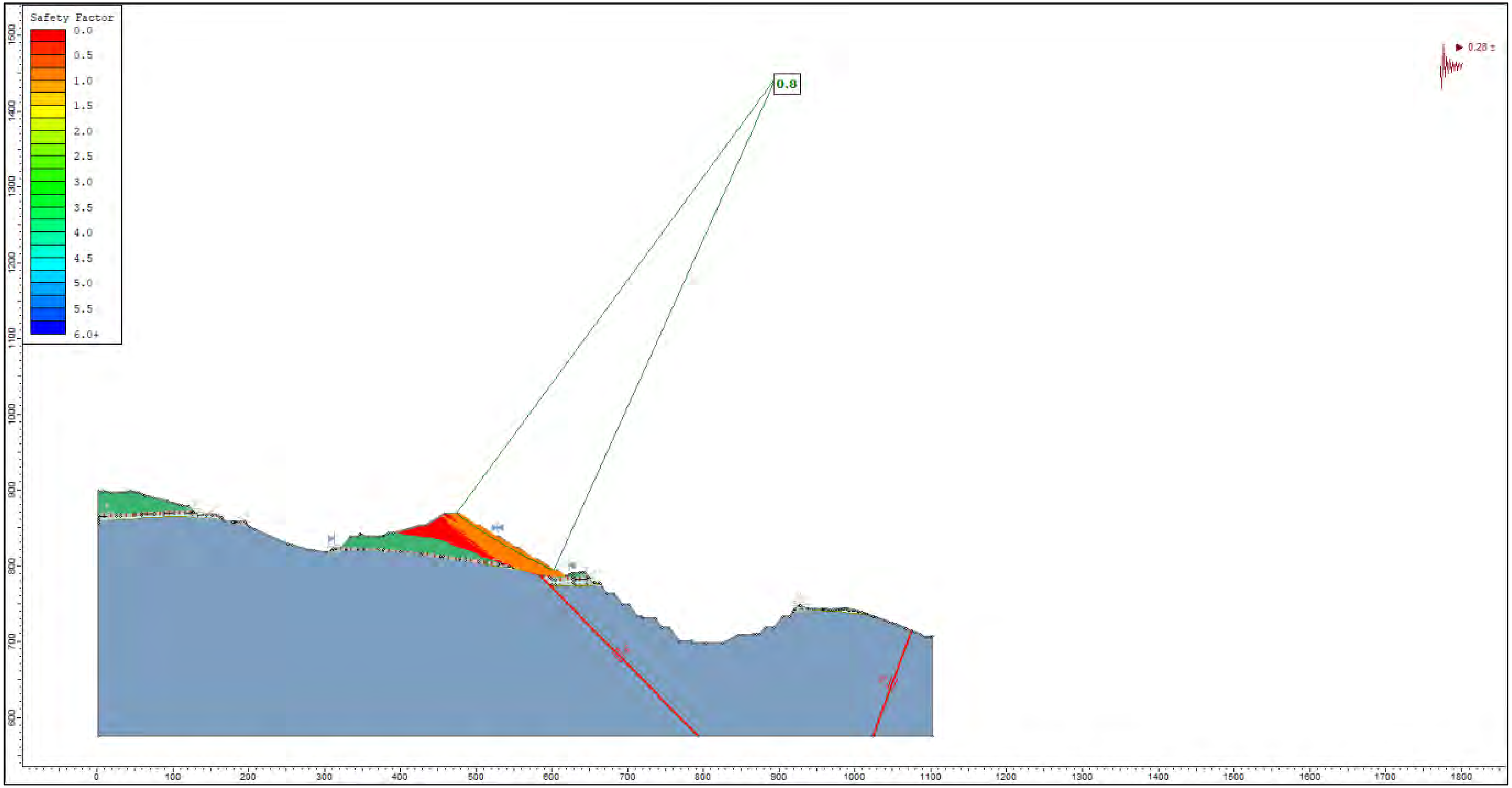

Figure: B82 Slope Stability Analysis- E_ELF1 - Section 103 — SLS Conditions.

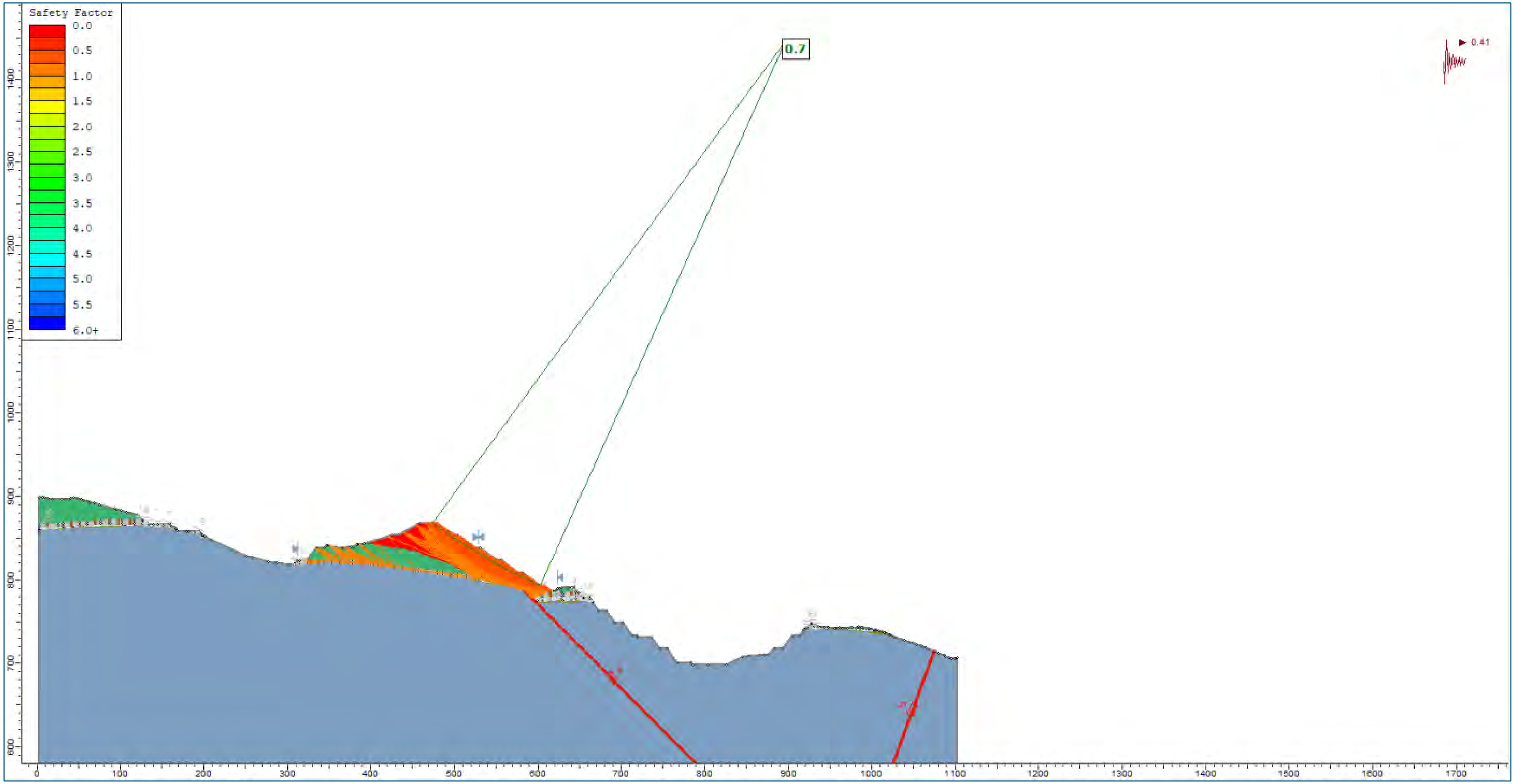

Figure: B83 - Slope Stability Analysis- E_ELF1 - Section 103 — ULS Conditions.

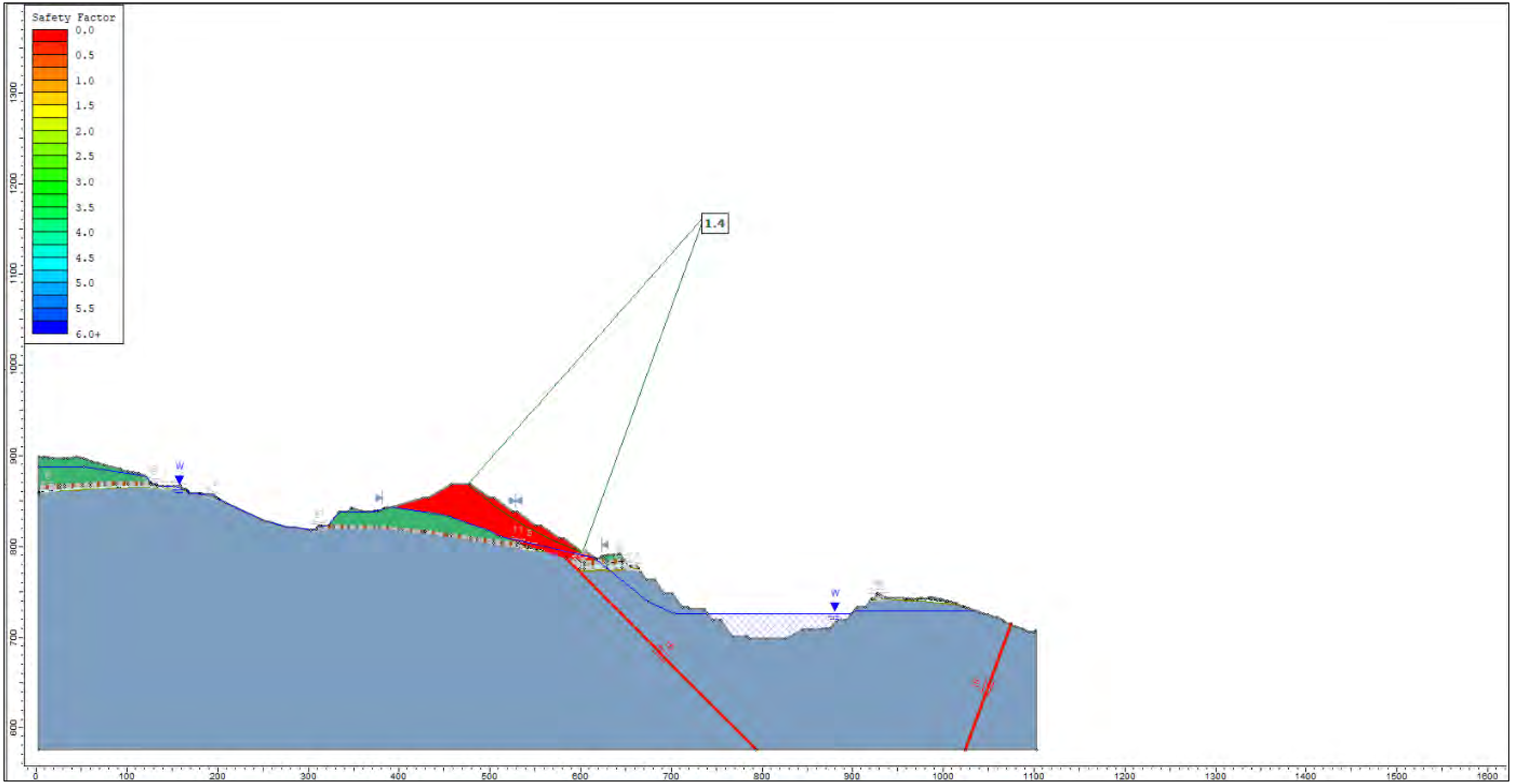

Figure: B84 - Slope Stability Analysis- E_ELF1 - Section 103 — Elevated Groundwater Conditions.

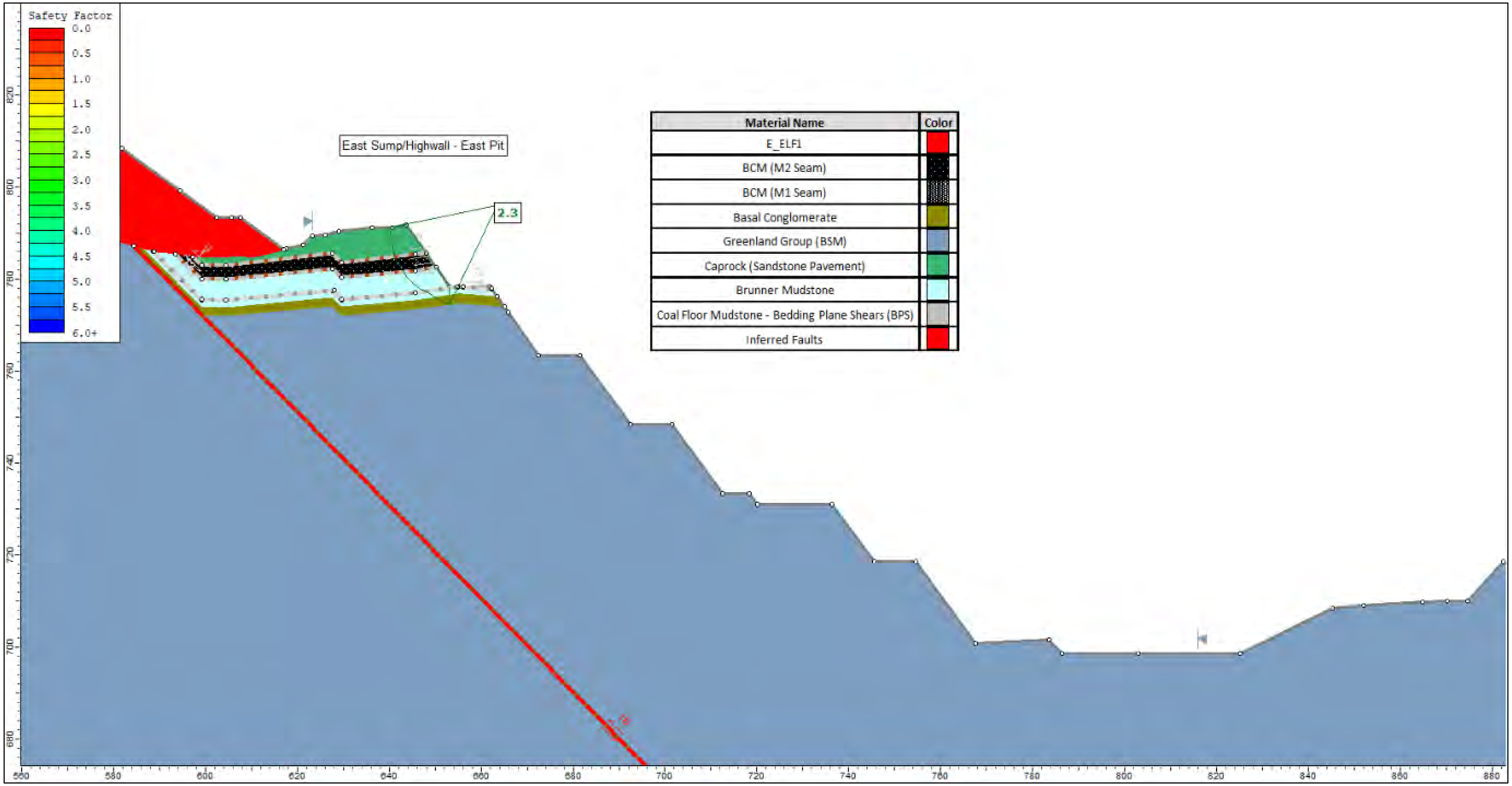

Figure: B85– East Sump/Highwall - East Pit Section 103 –Static Conditions.

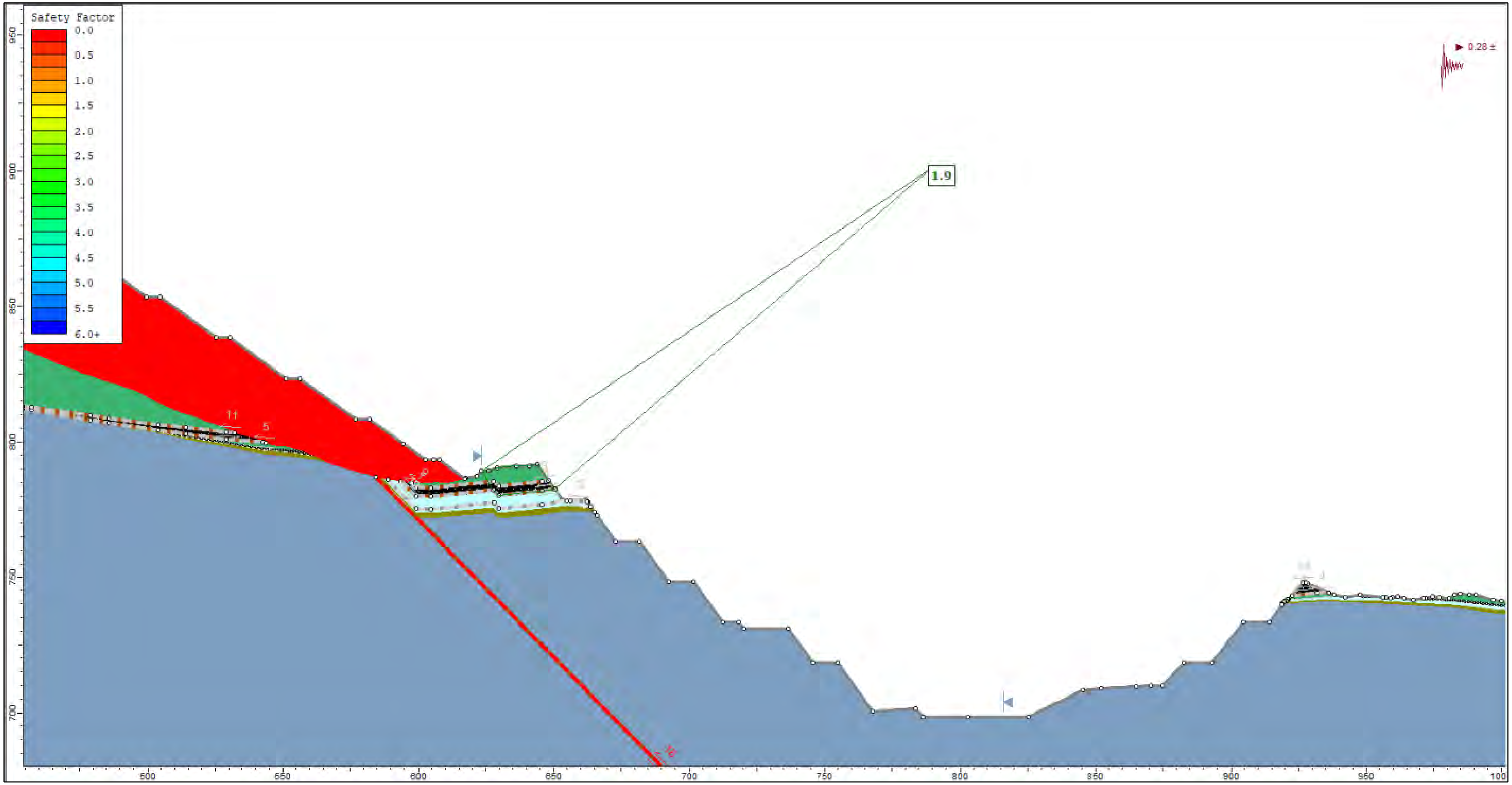

Figure: B86– East Sump/Highwall - East Pit Section 103 –SLS Conditions.

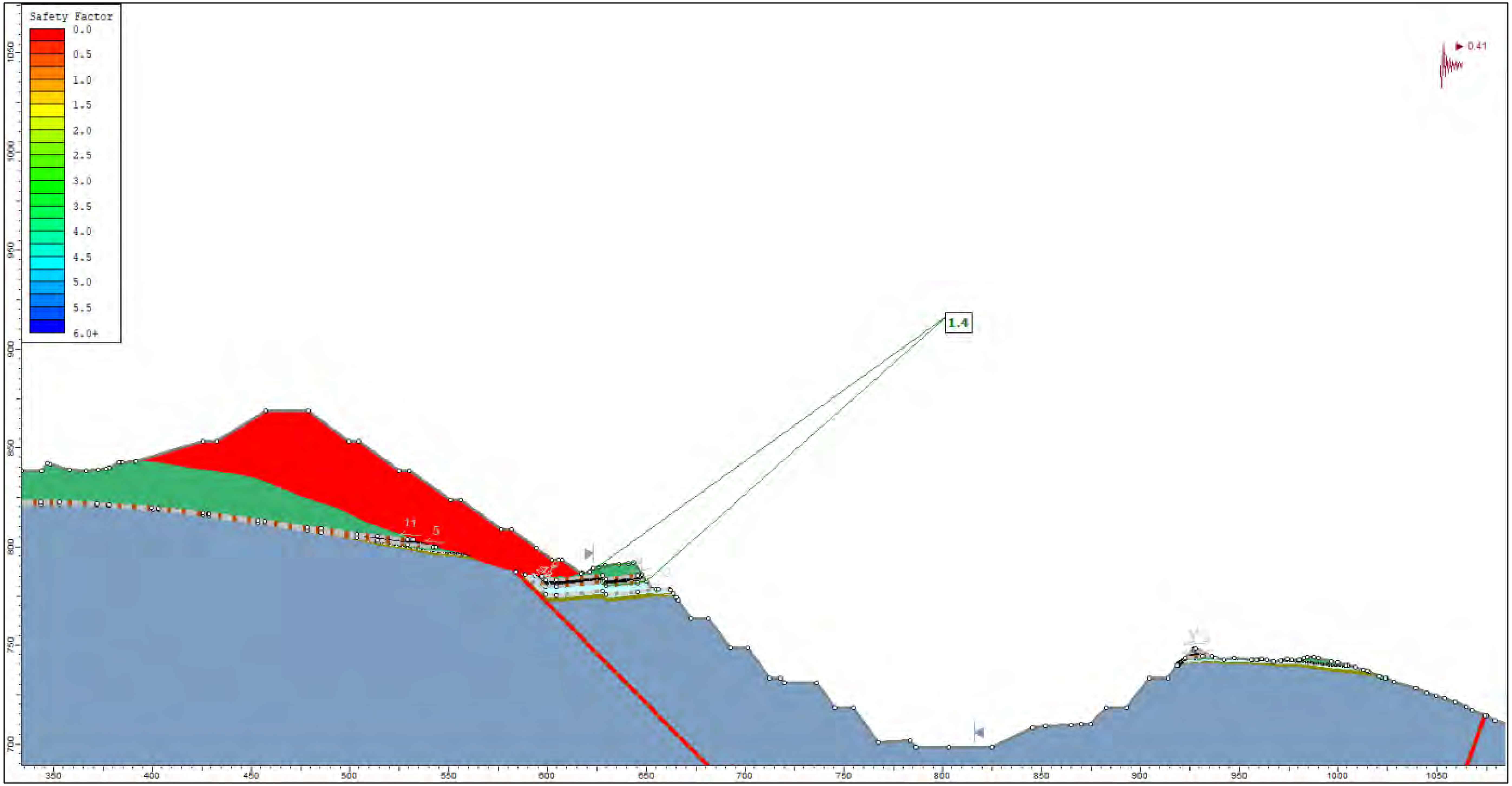

Figure: B87– East Sump/Highwall - East Pit Section 103 –ULS Conditions.

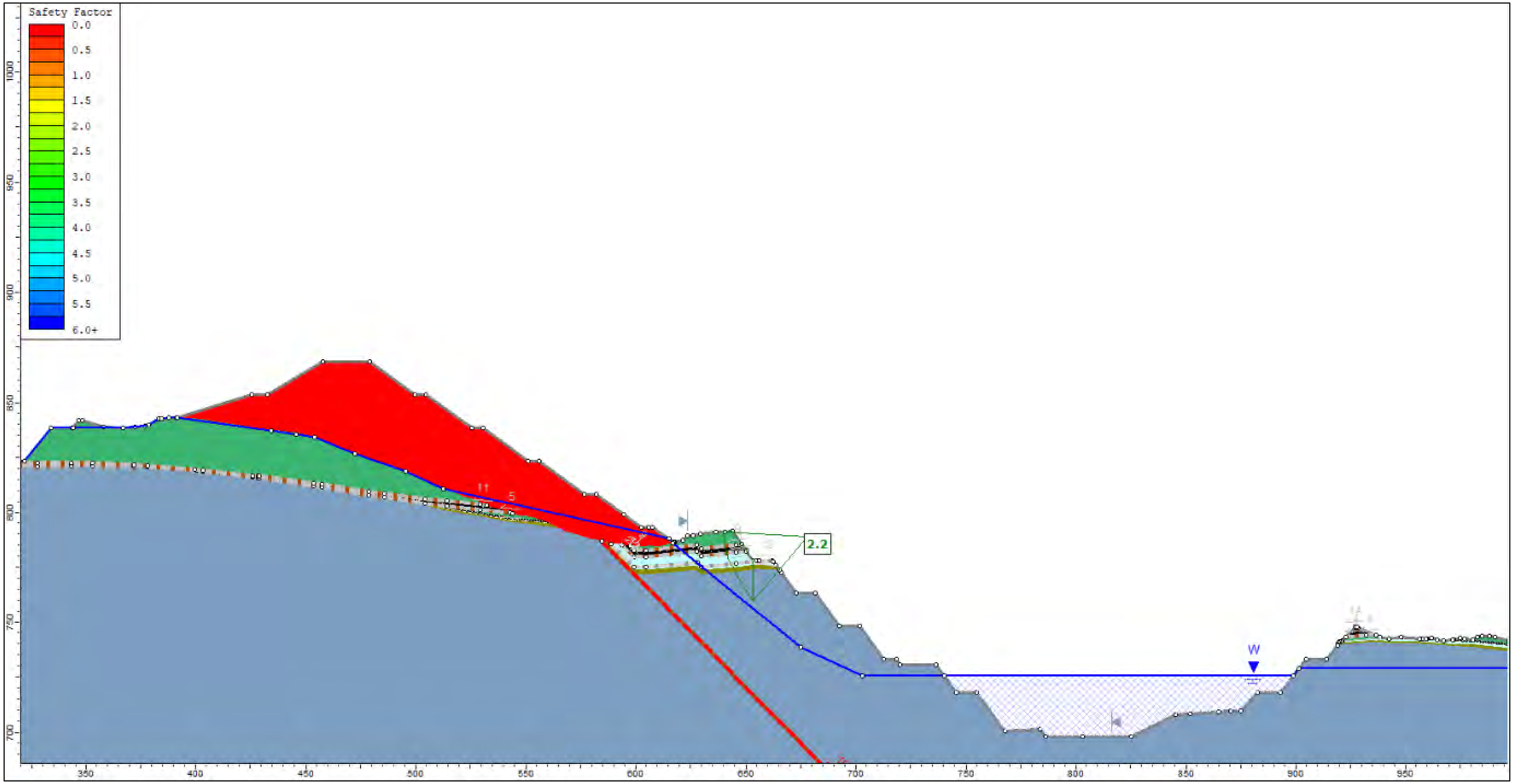

Figure: B88– East Sump/Highwall - East Pit Section 103 –Elevated Groundwater Conditions.

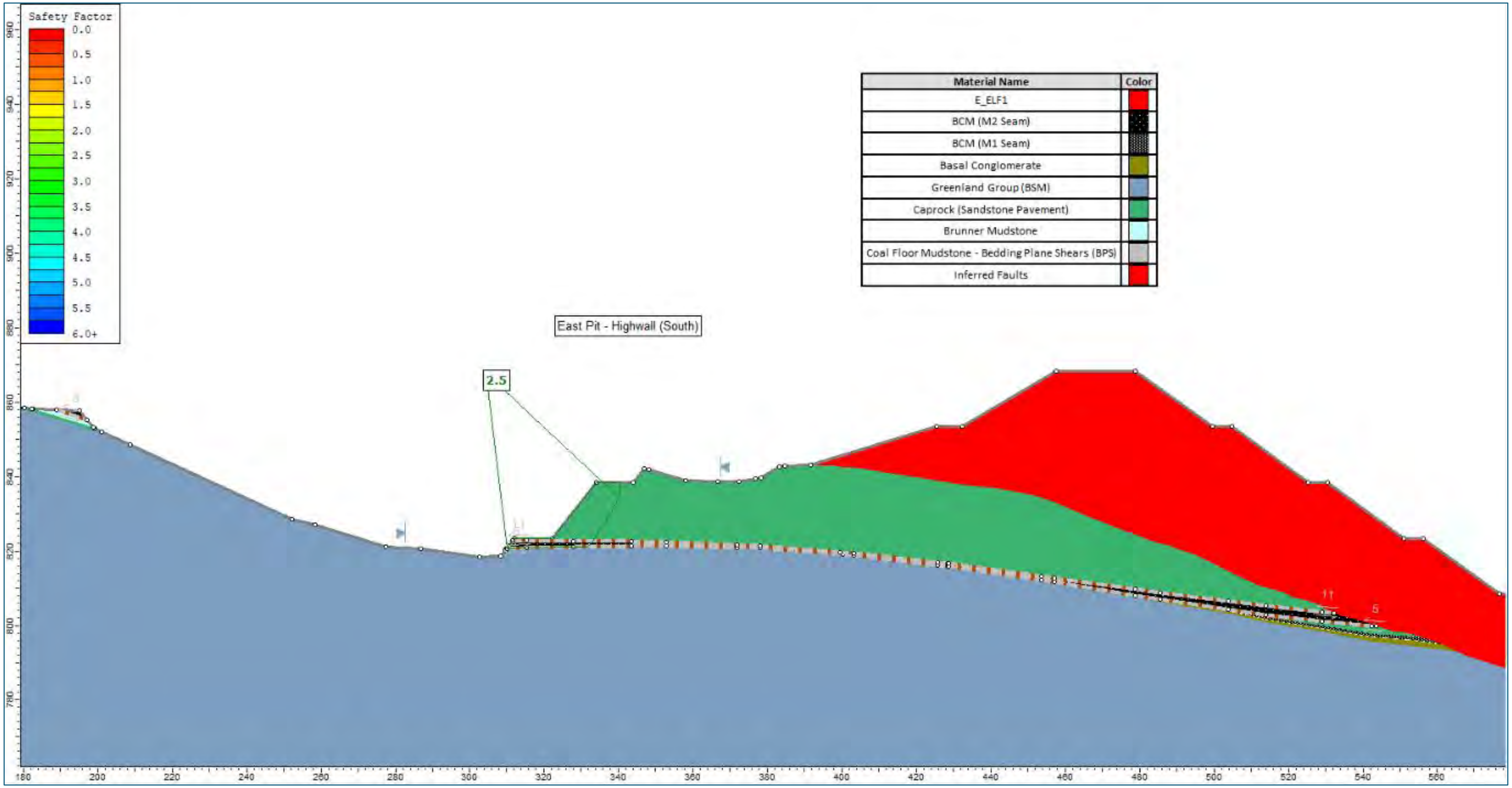

Figure B89 - East Pit Highwall (South) – Section 103 – Static Conditions.

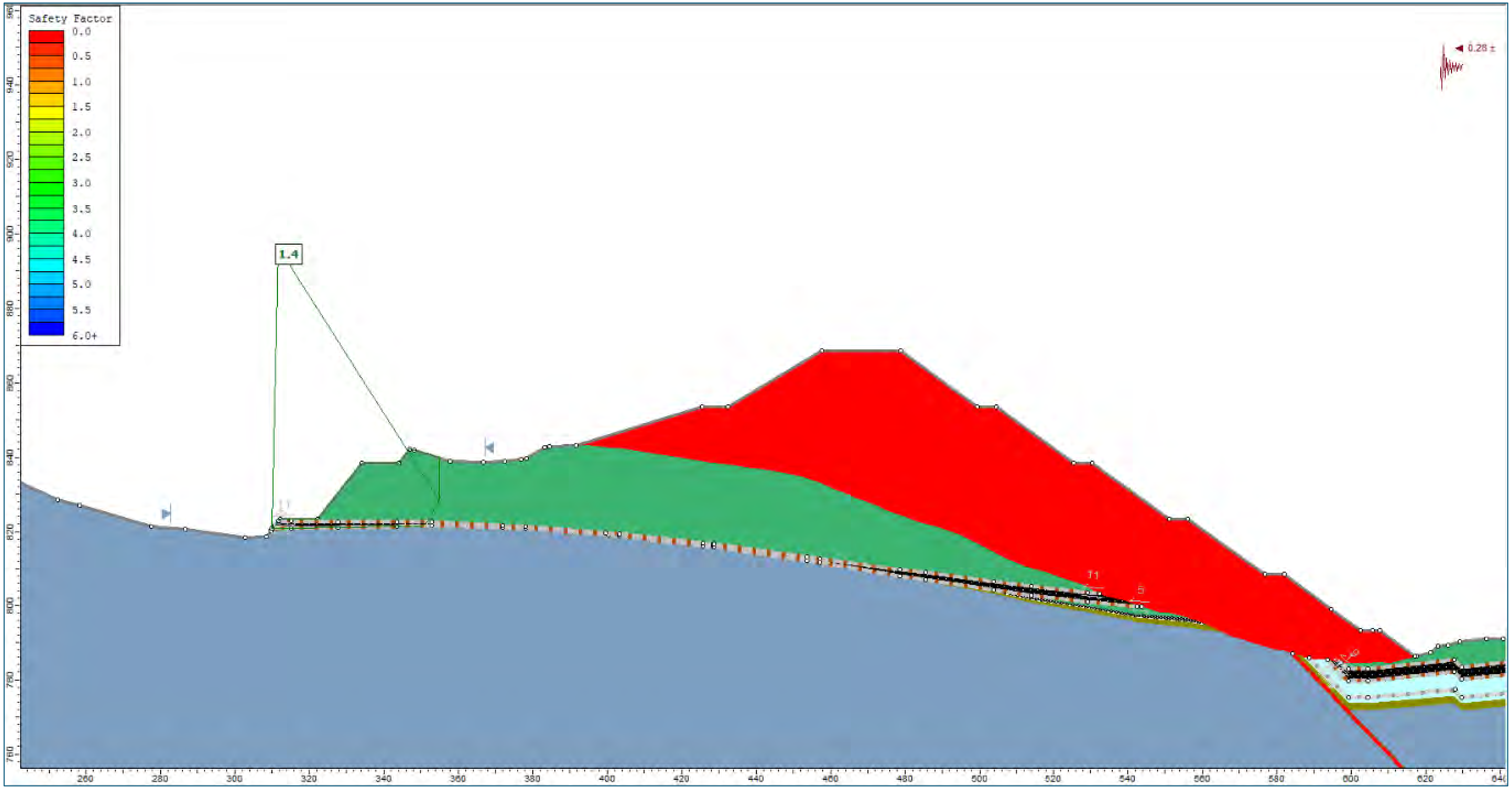

Figure B90 - East Pit Highwall (South) – Section 103 – SLS Conditions.

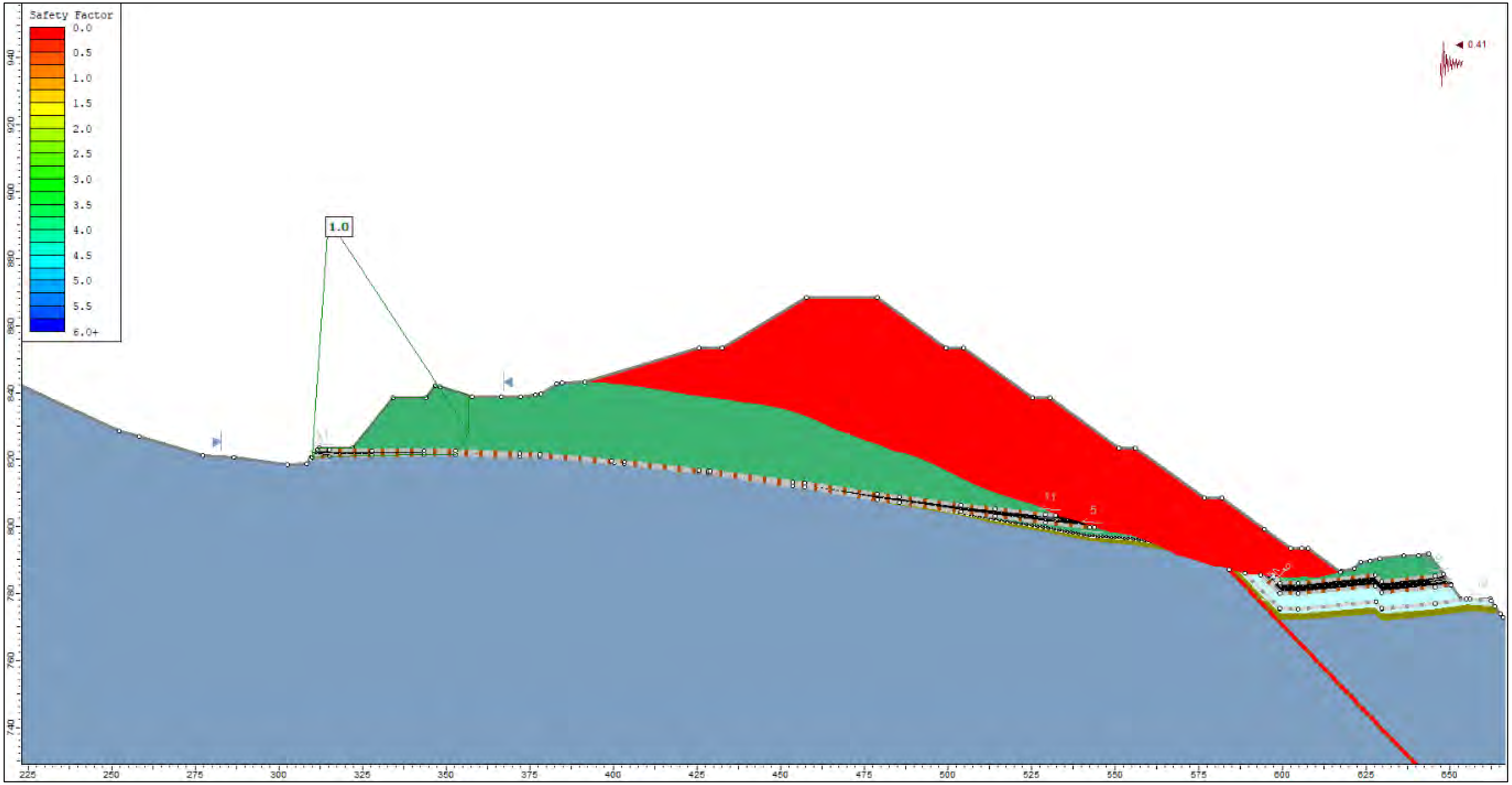

Figure B91 - East Pit Highwall (South) – Section 103 – ULS Conditions.

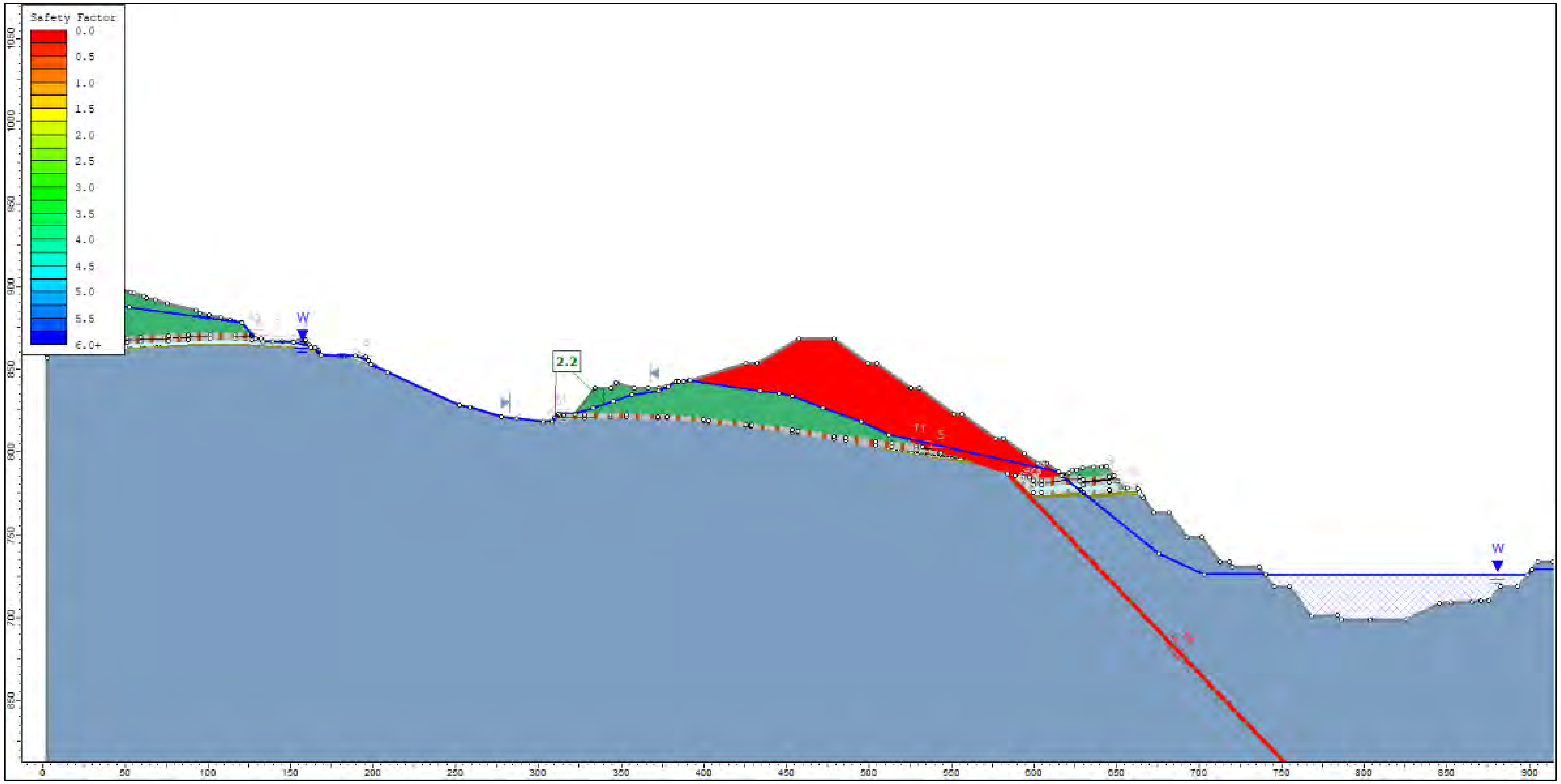

Figure B92 - East Pit Highwall (South) – Section 103 – Static Conditions.

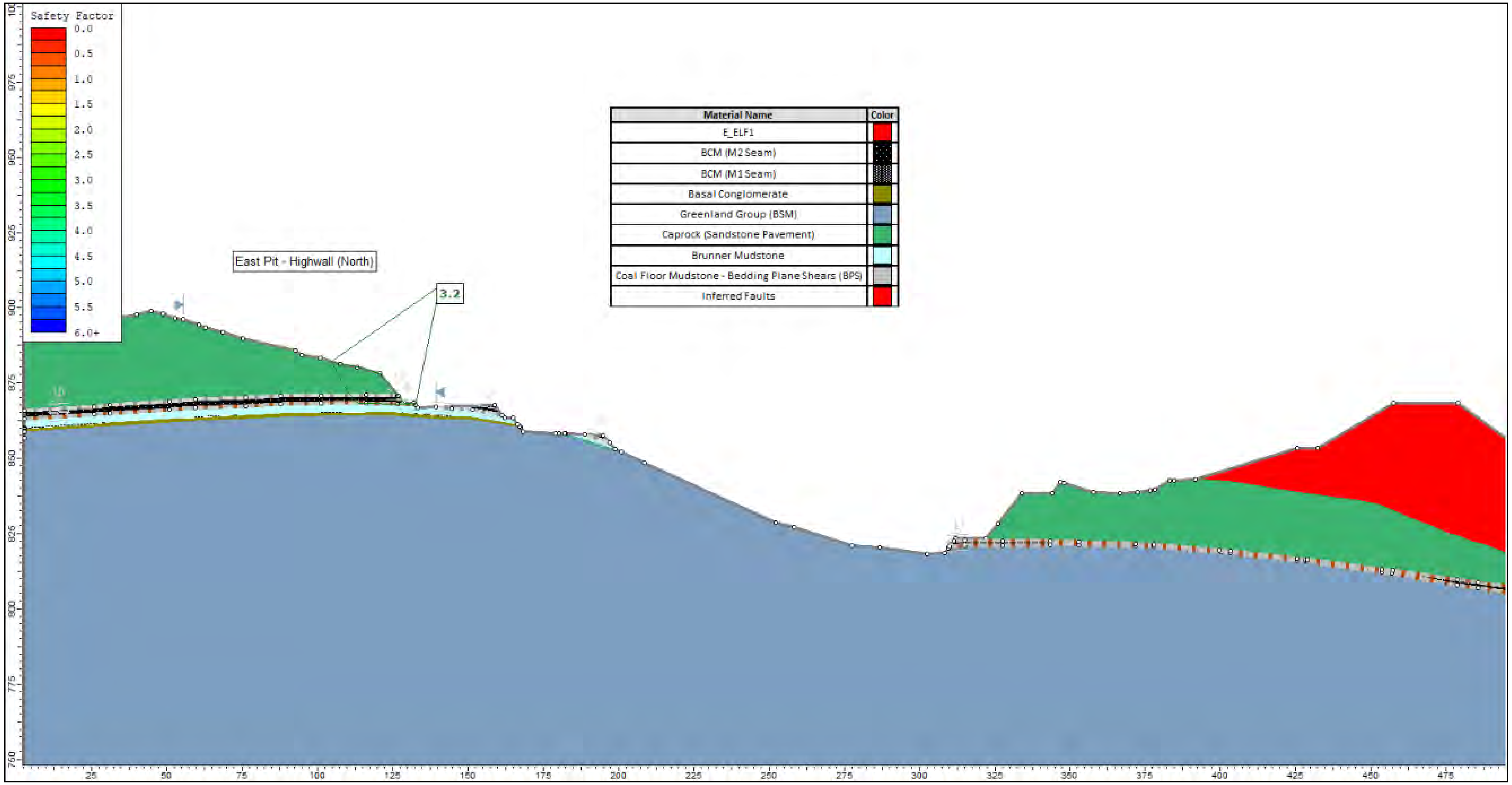

Figure B93- East Pit Highwall (North) – Section 103 – Static Conditions.

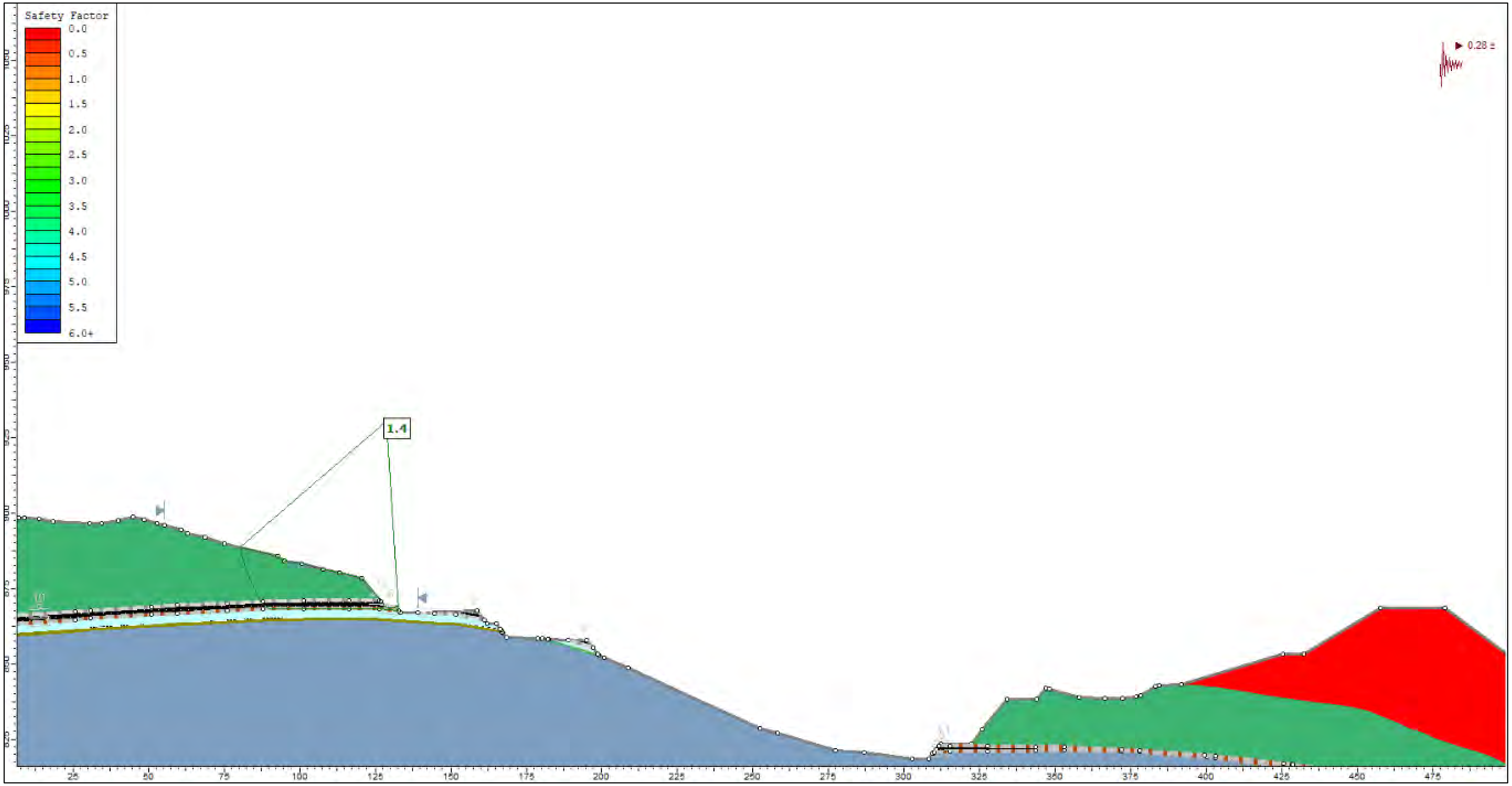

Figure B94- East Pit Highwall (North) – Section 103 – SLS Conditions.

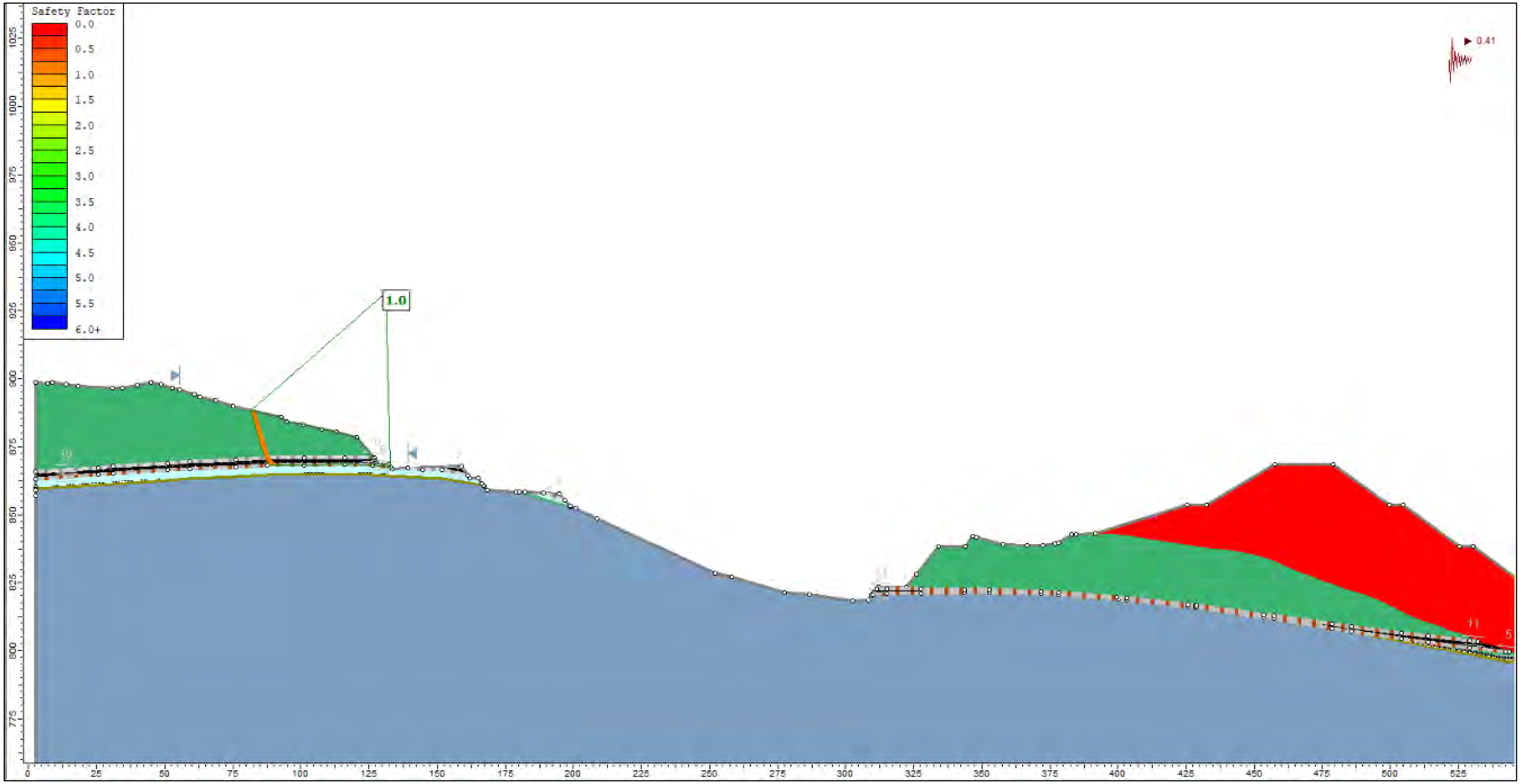

Figure B95- East Pit Highwall (North) – Section 103 – ULS Conditions.

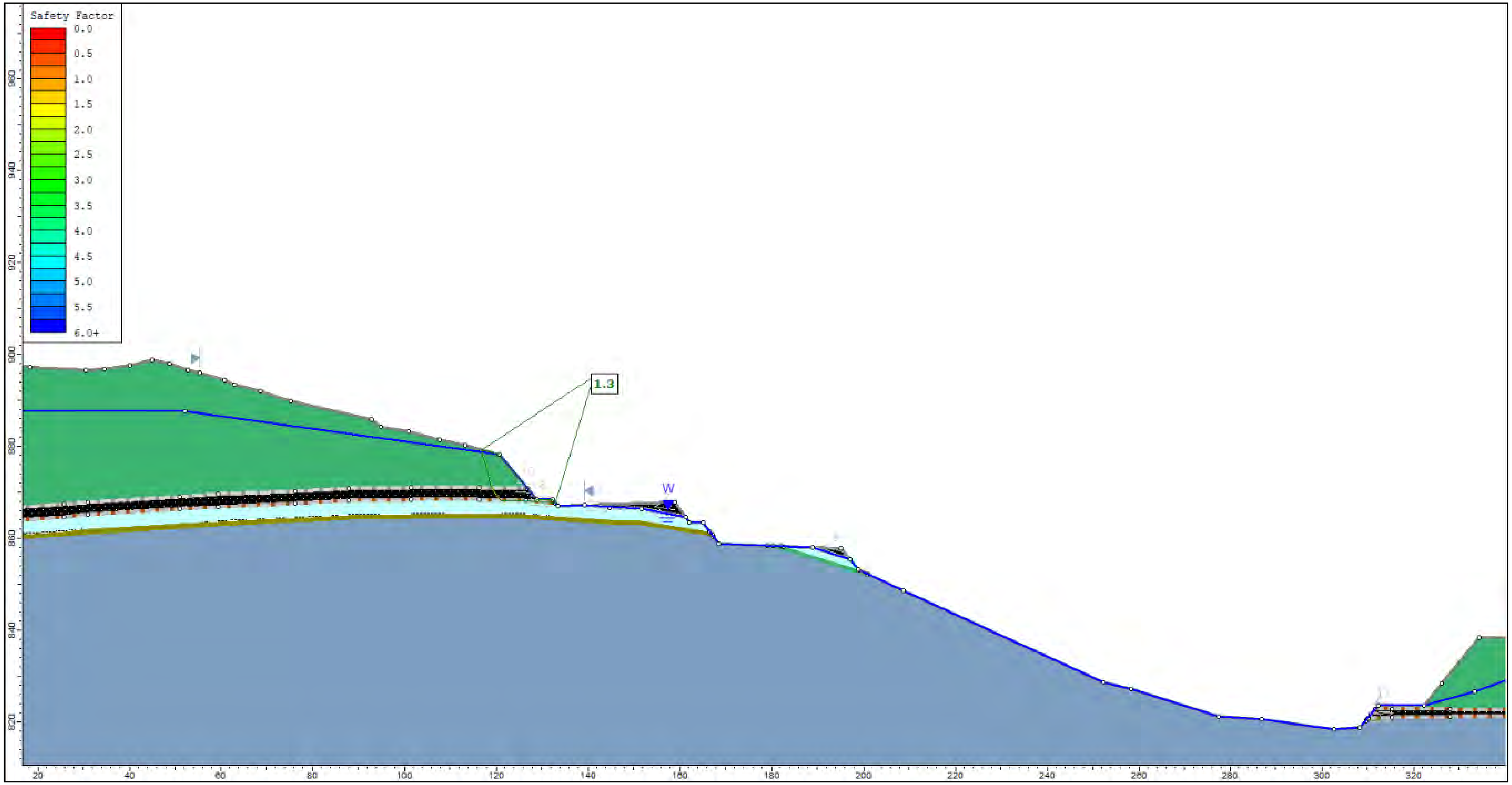

Figure B96- East Pit Highwall (North) – Section 103 – Elevated Groundwater Conditions.

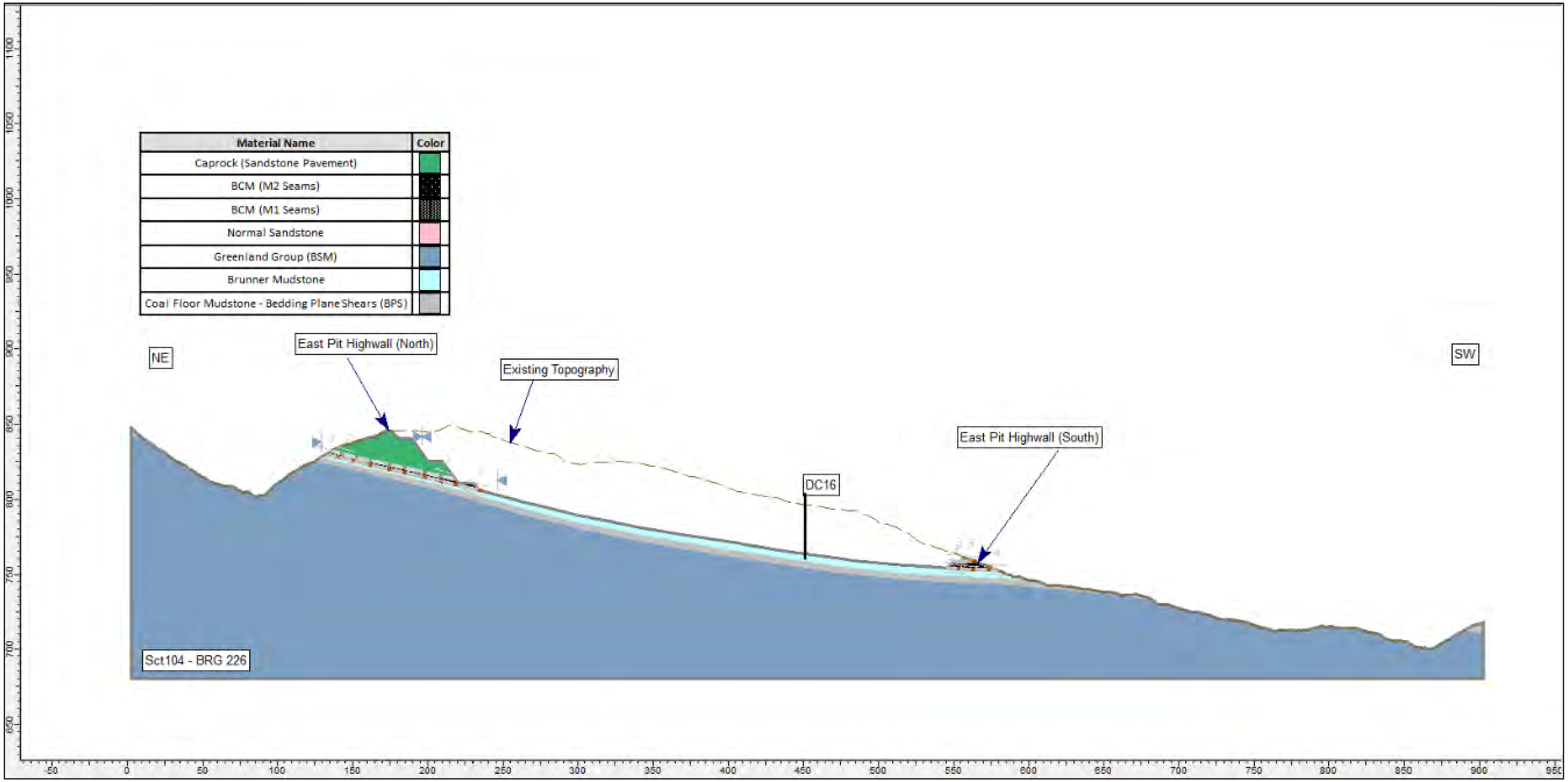

Figure B97 - Section 104 – Mine Feature Overview

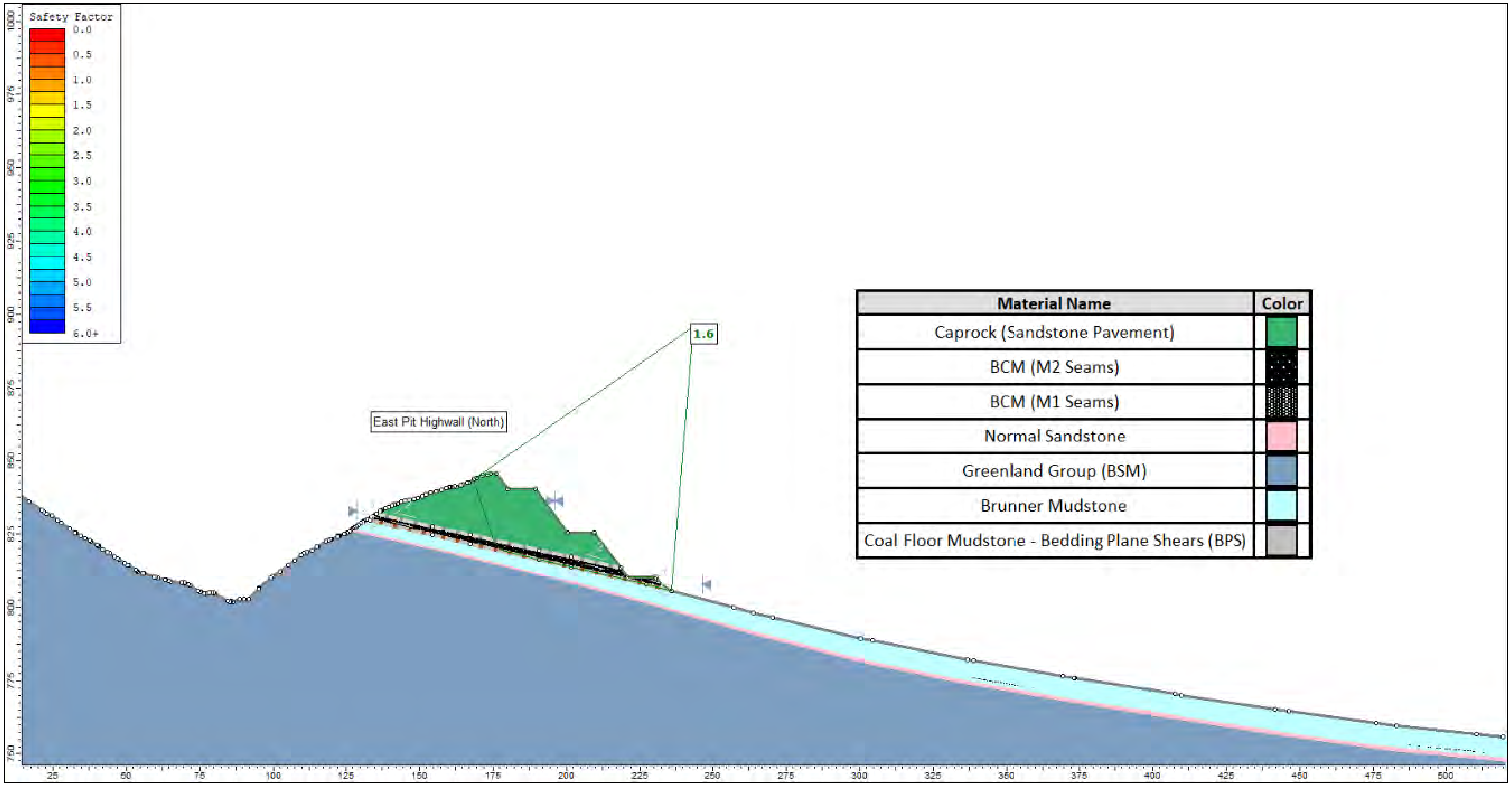

Figure B98 –East Pit Highwall (North) - Section 104 – Static Conditions.

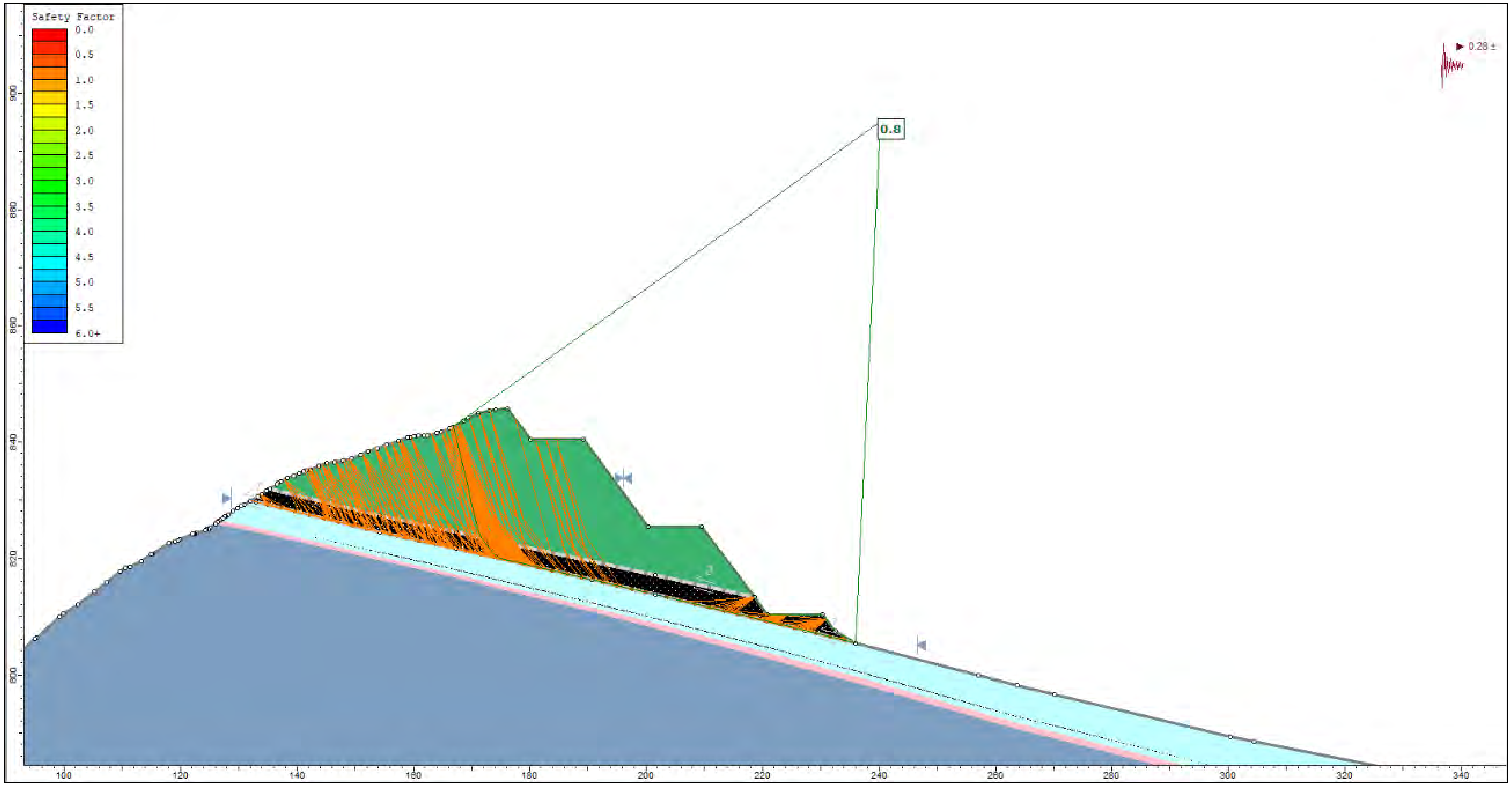

Figure B99 –East Pit Highwall (North) - Section 104 – SLS Conditions.

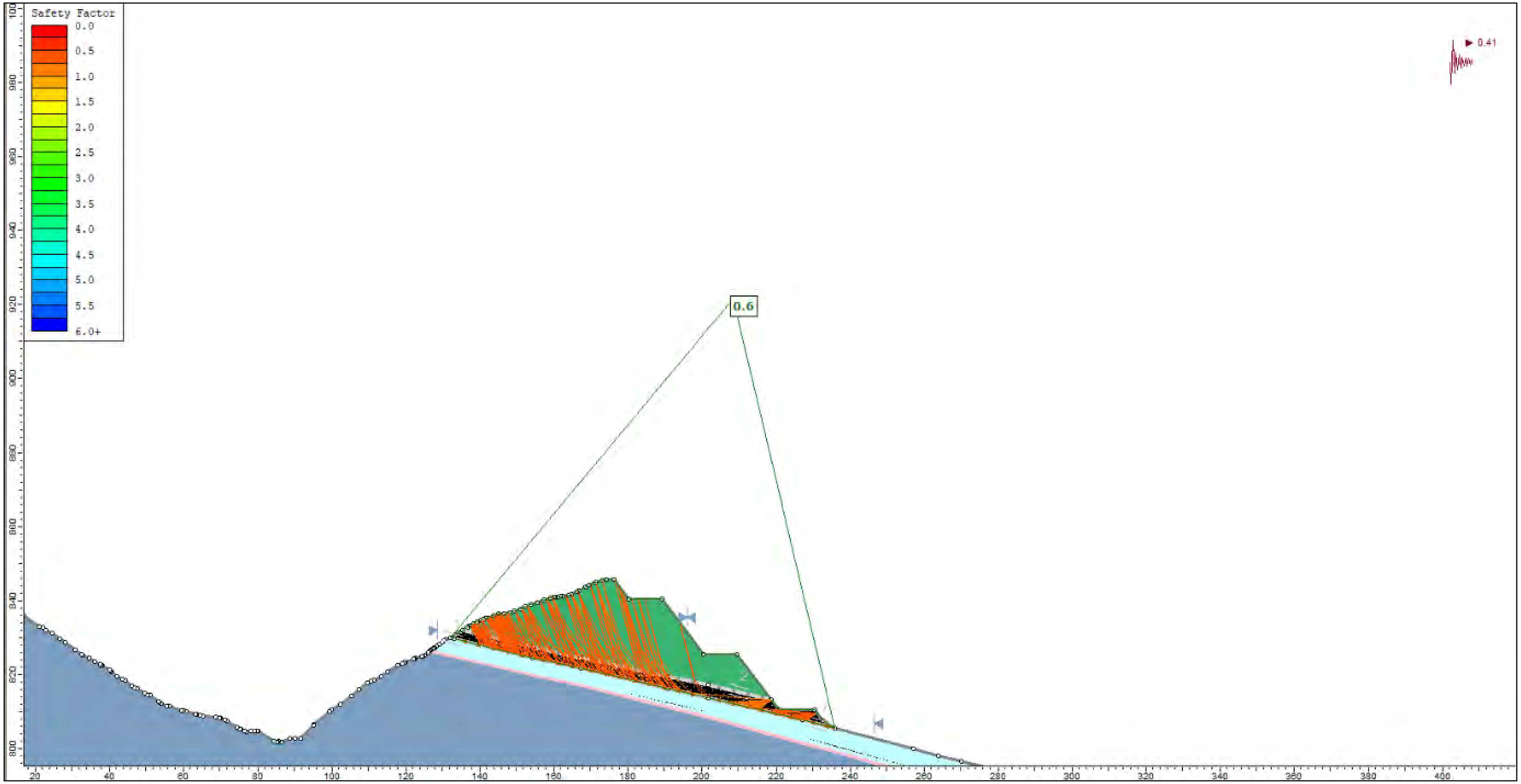

Figure B100 –East Pit Highwall (North) -Section 104 – ULS Conditions.

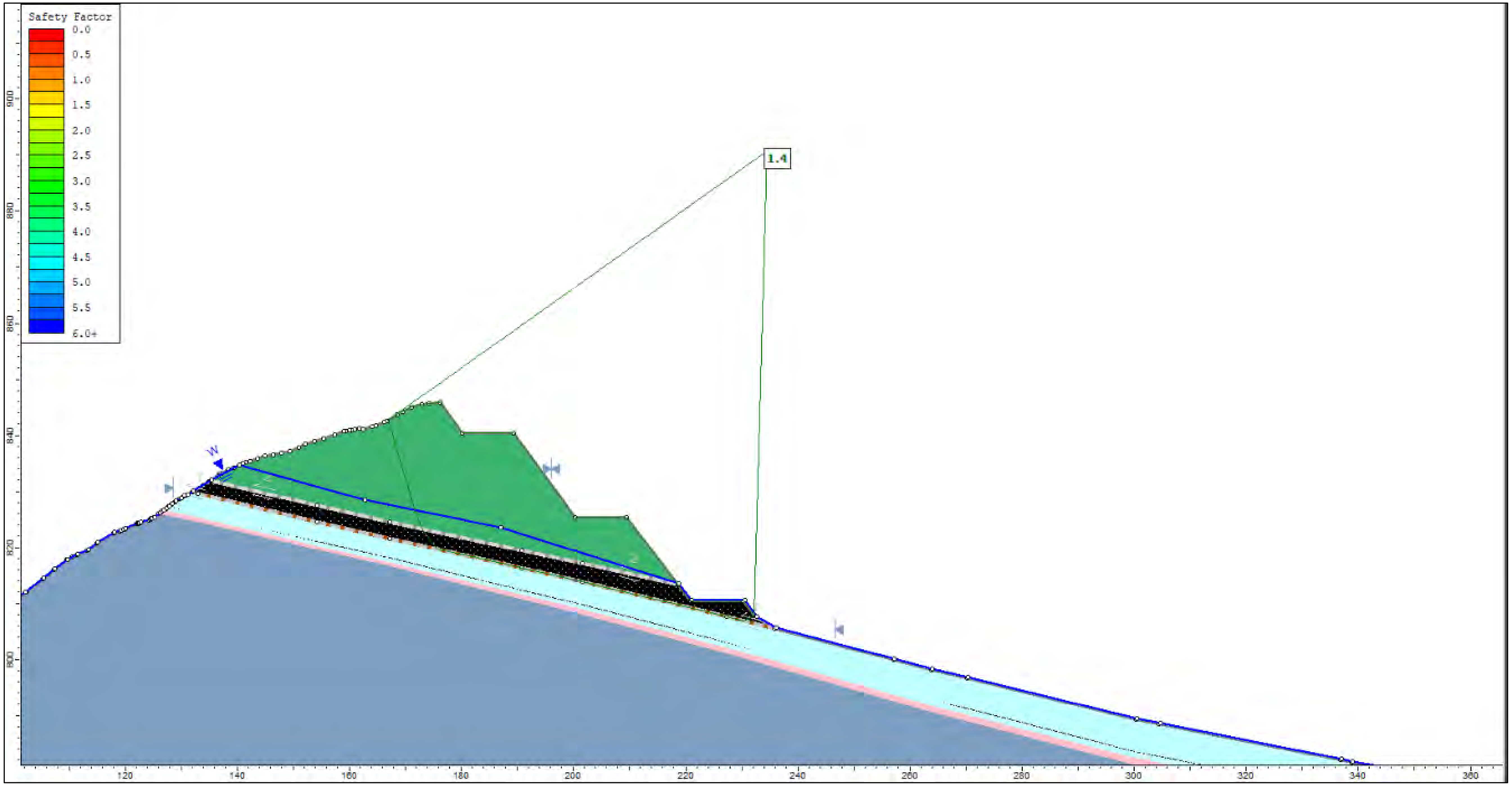

Figure B101 –East Pit Highwall (North) -Section 104 – Elevated Groundwater Conditions.

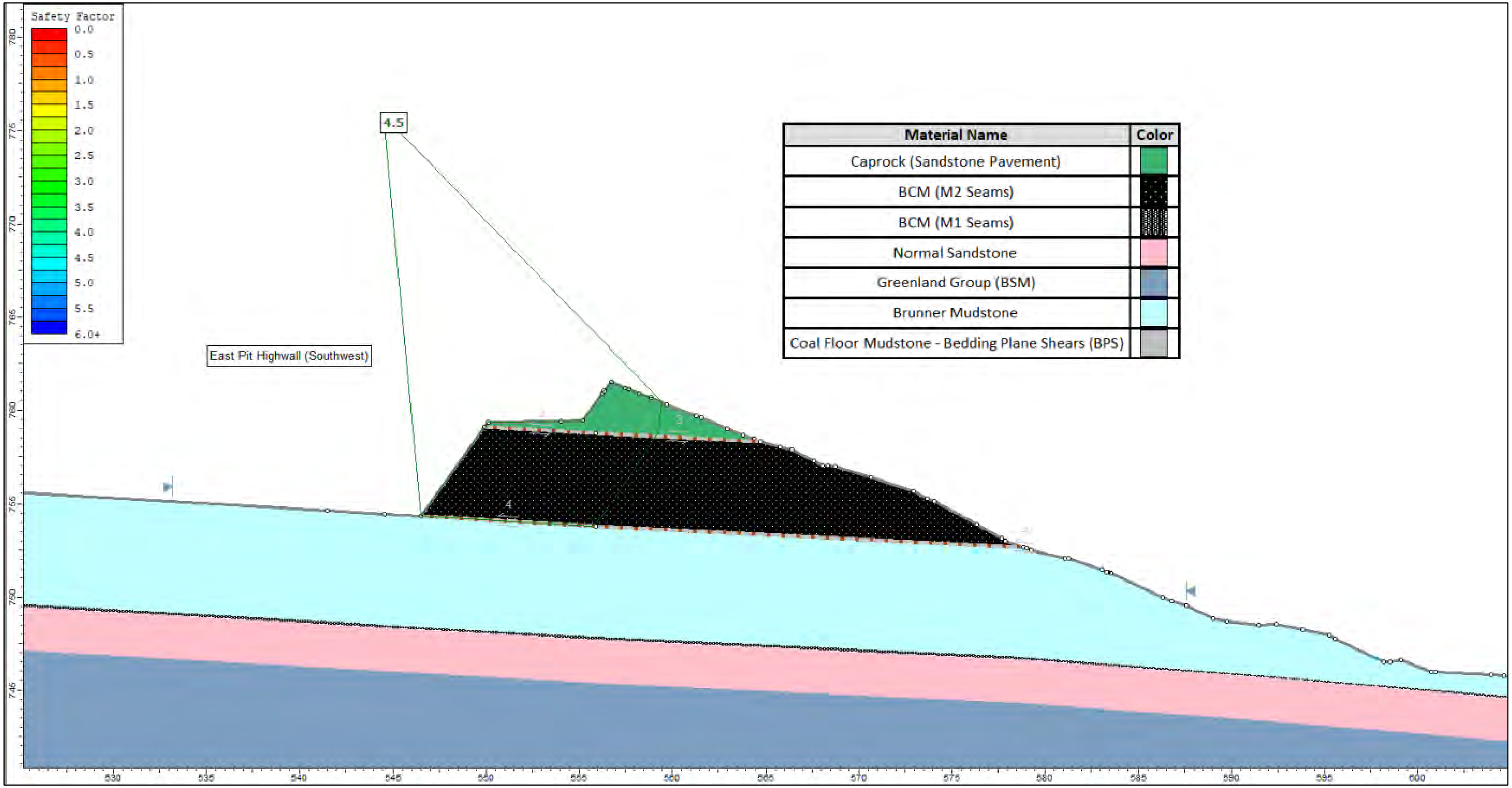

Figure B102 –East Pit Highwall (South-Western) into Pit Failure - Section 104 – Static Conditions.

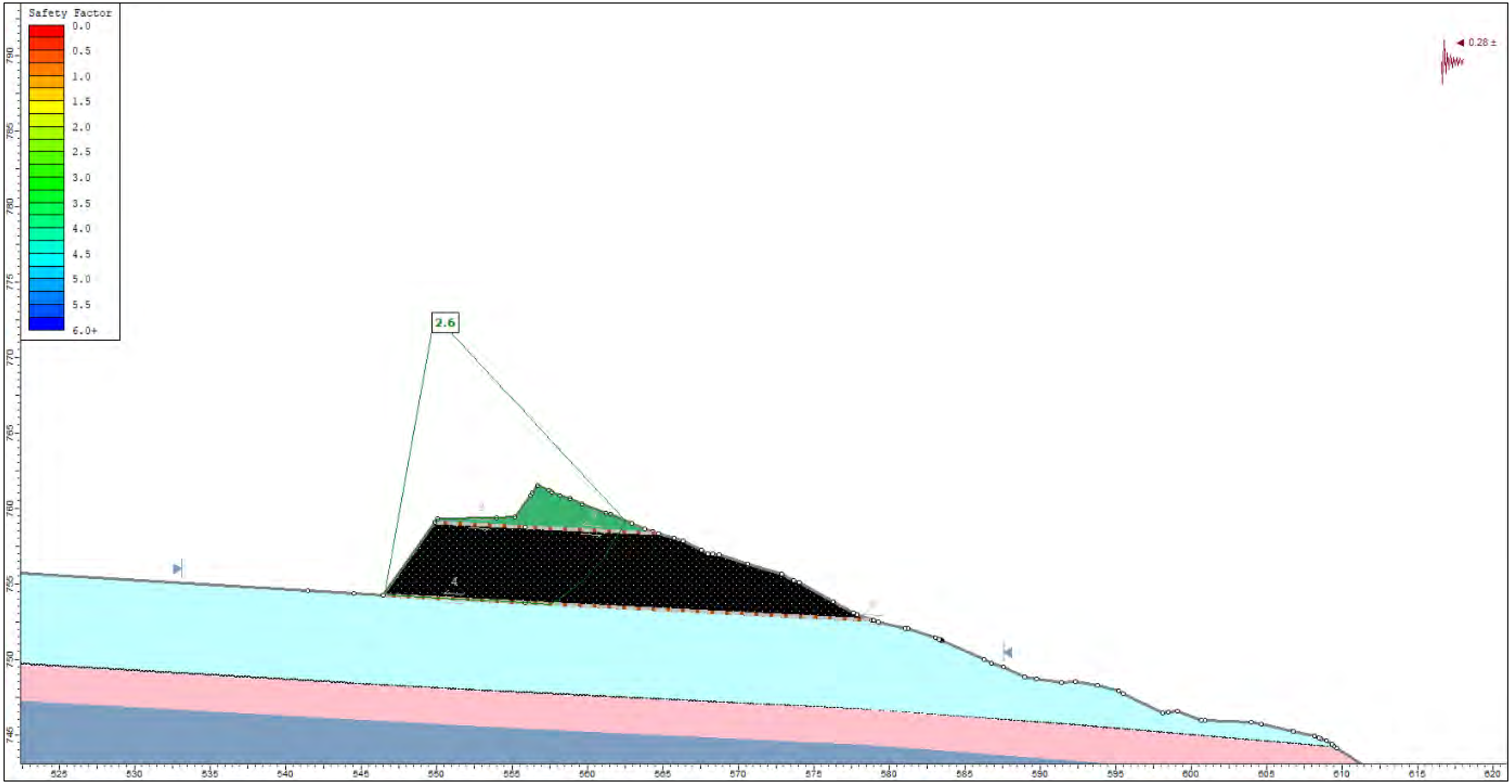

Figure B103 –East Pit Highwall (South-Western) into Pit Failure - Section 104 – SLS Conditions.

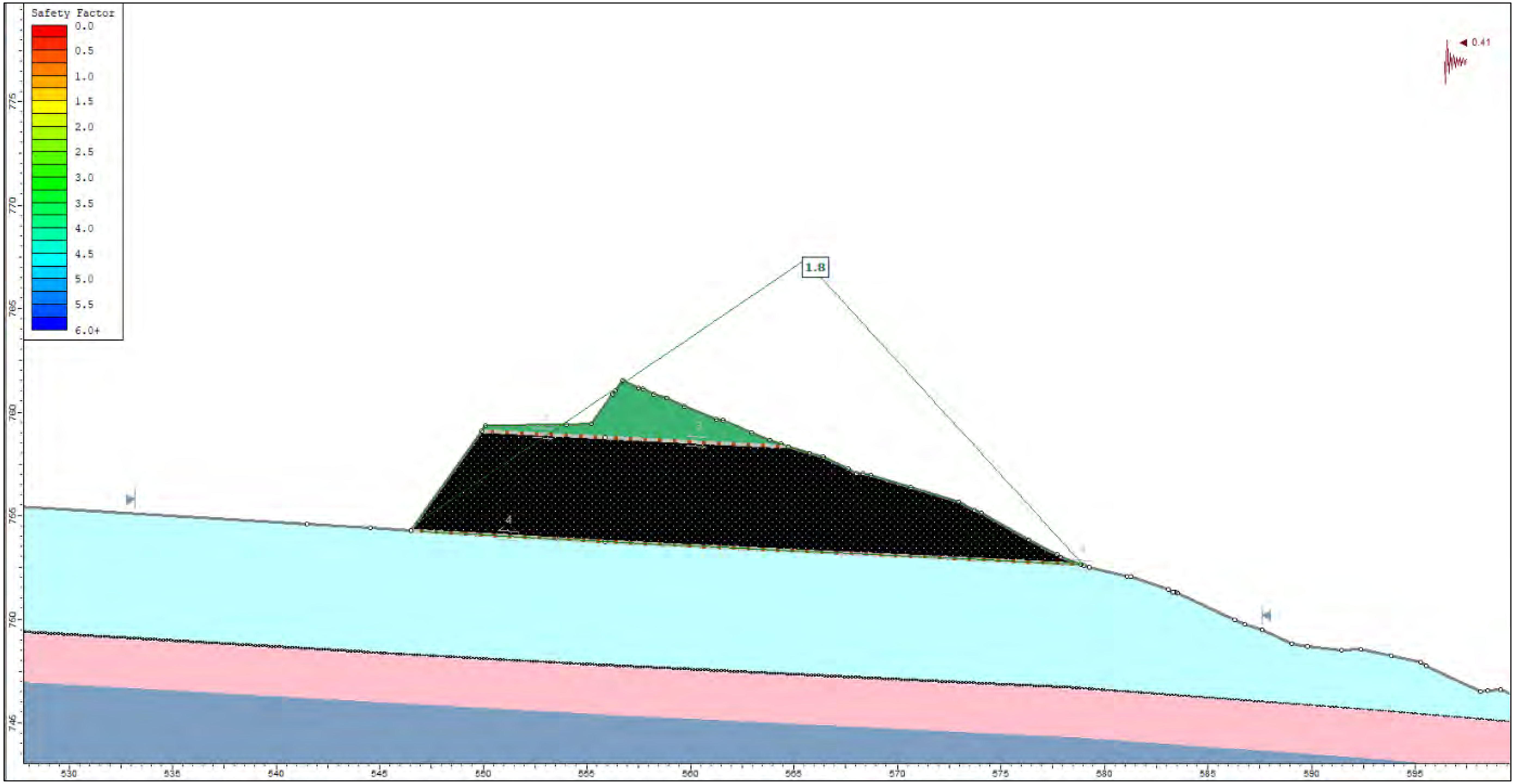

Figure B104 –East Pit Highwall (South-Western) into Pit Failure - Section 104 – ULS Conditions.

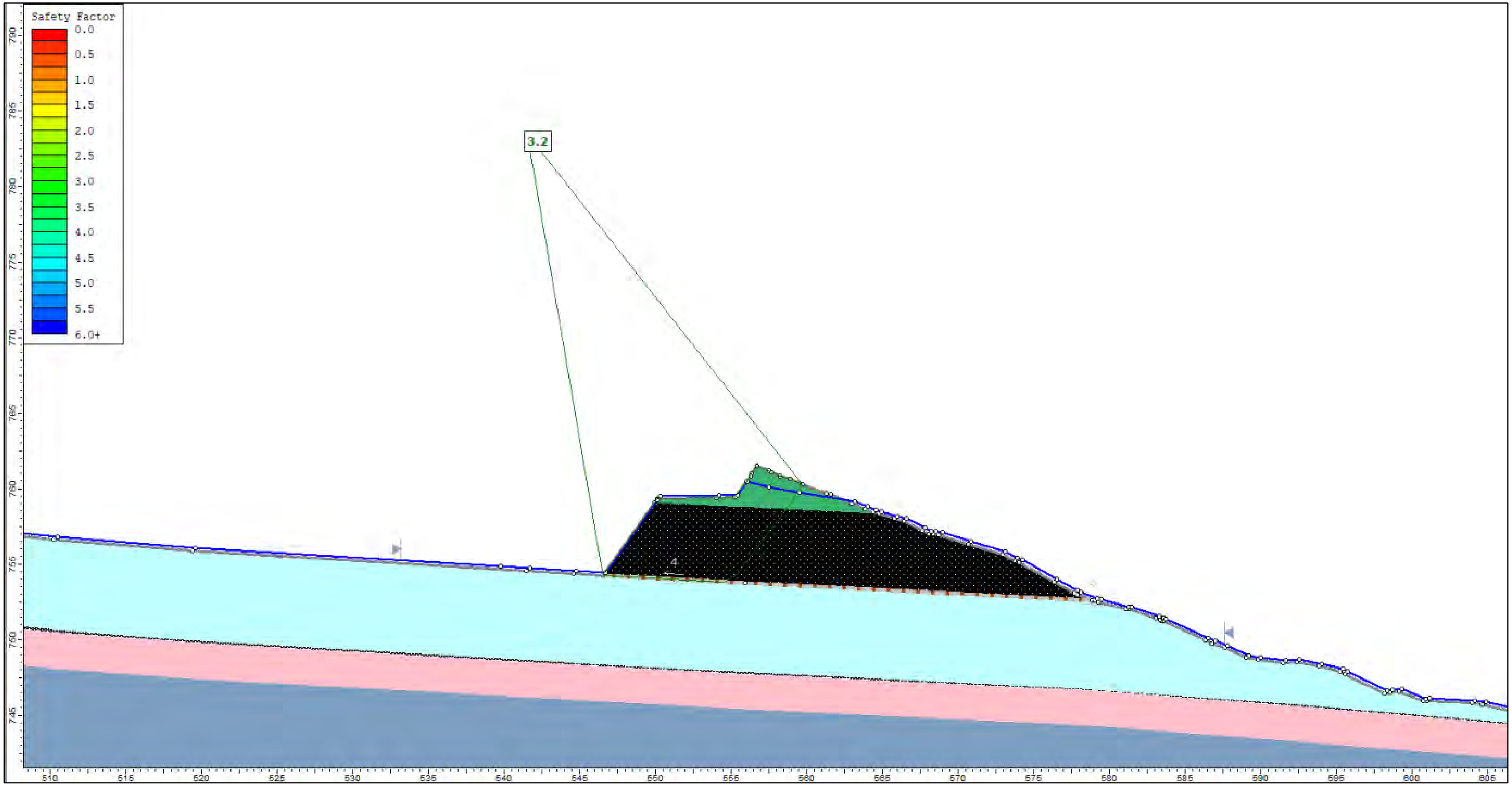

Figure B105 –East Pit Highwall (South-Western) into Pit Failure - Section 104 – Elevated Groundwater Conditions.

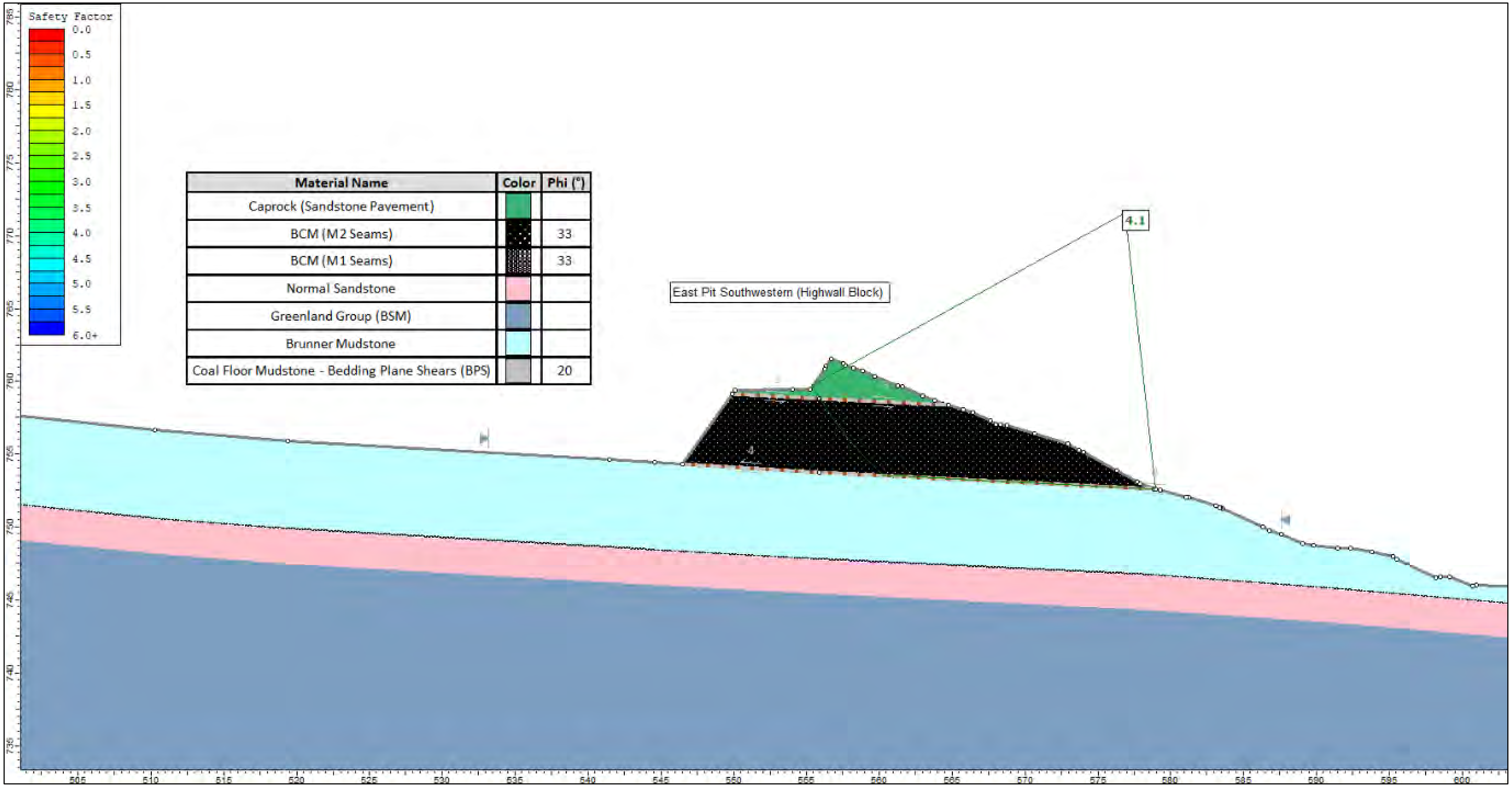

Figure B106 –East Pit Highwall (South-Western) – Downslope Failure - Section 104 – Static Conditions.

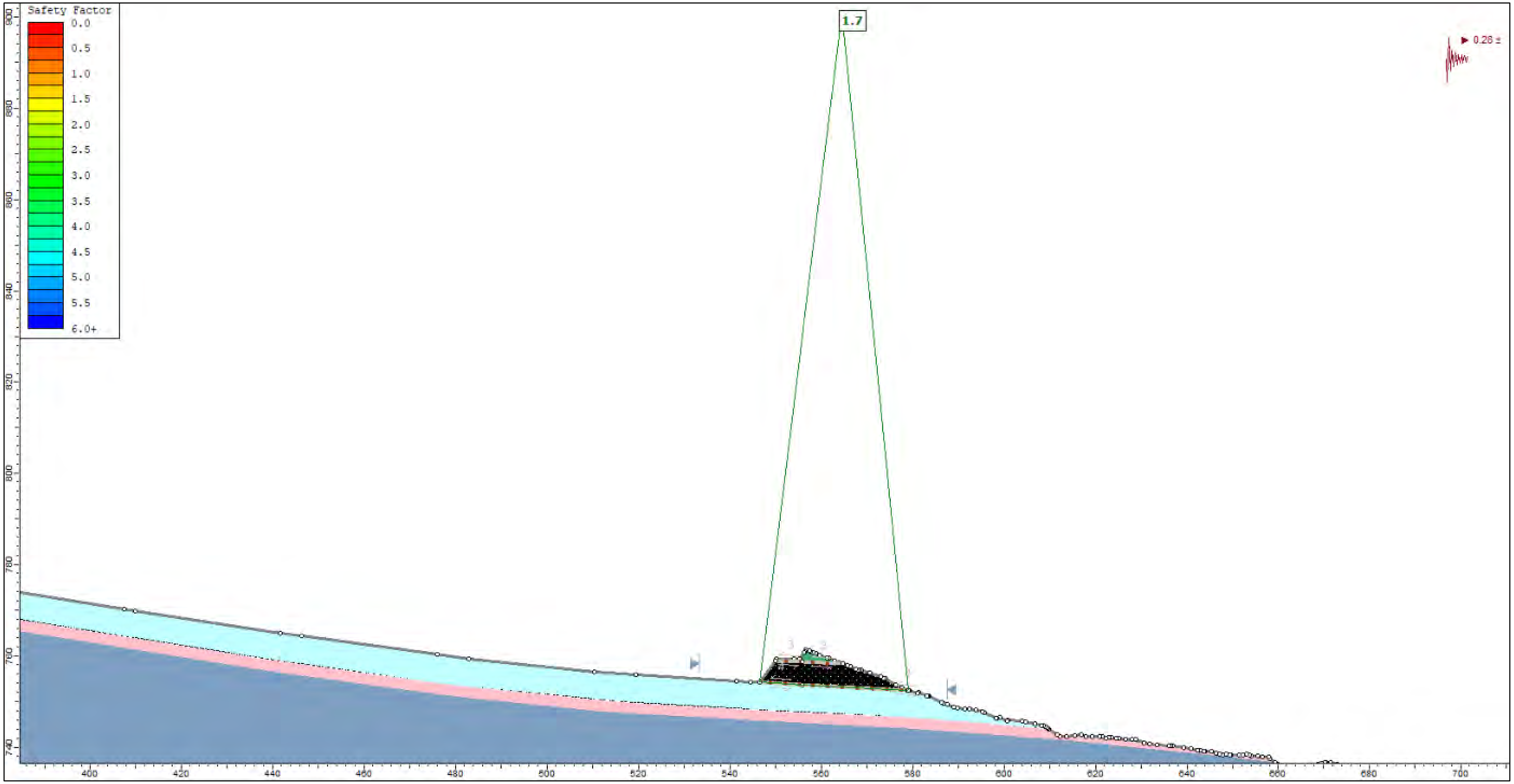

Figure B107 –East Pit Highwall (South-Western) – Downslope Failure - Section 104 – SLS Conditions.

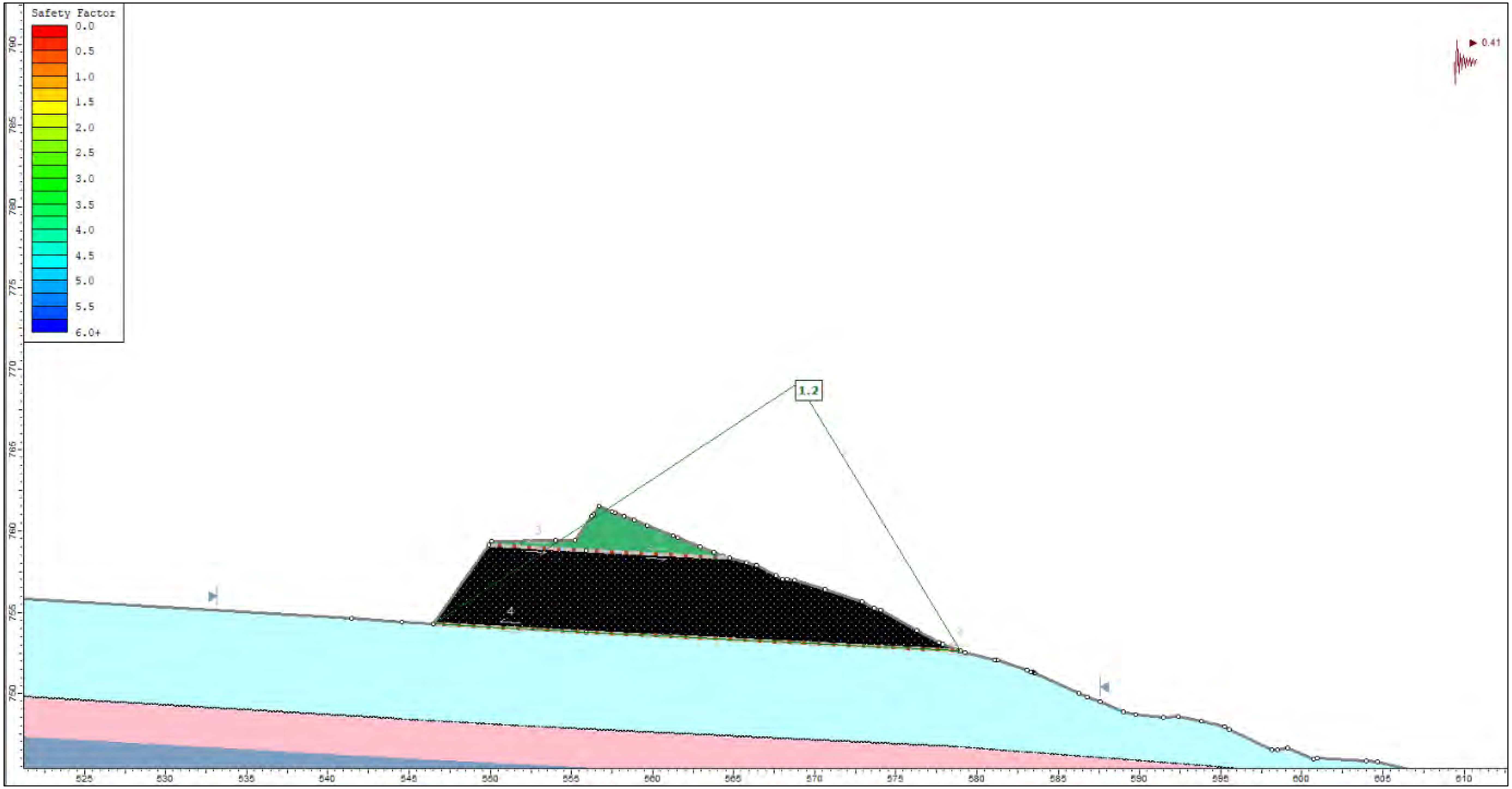

Figure B108 –East Pit Highwall (South-Western) – Downslope Failure - Section 104 – ULS Conditions.

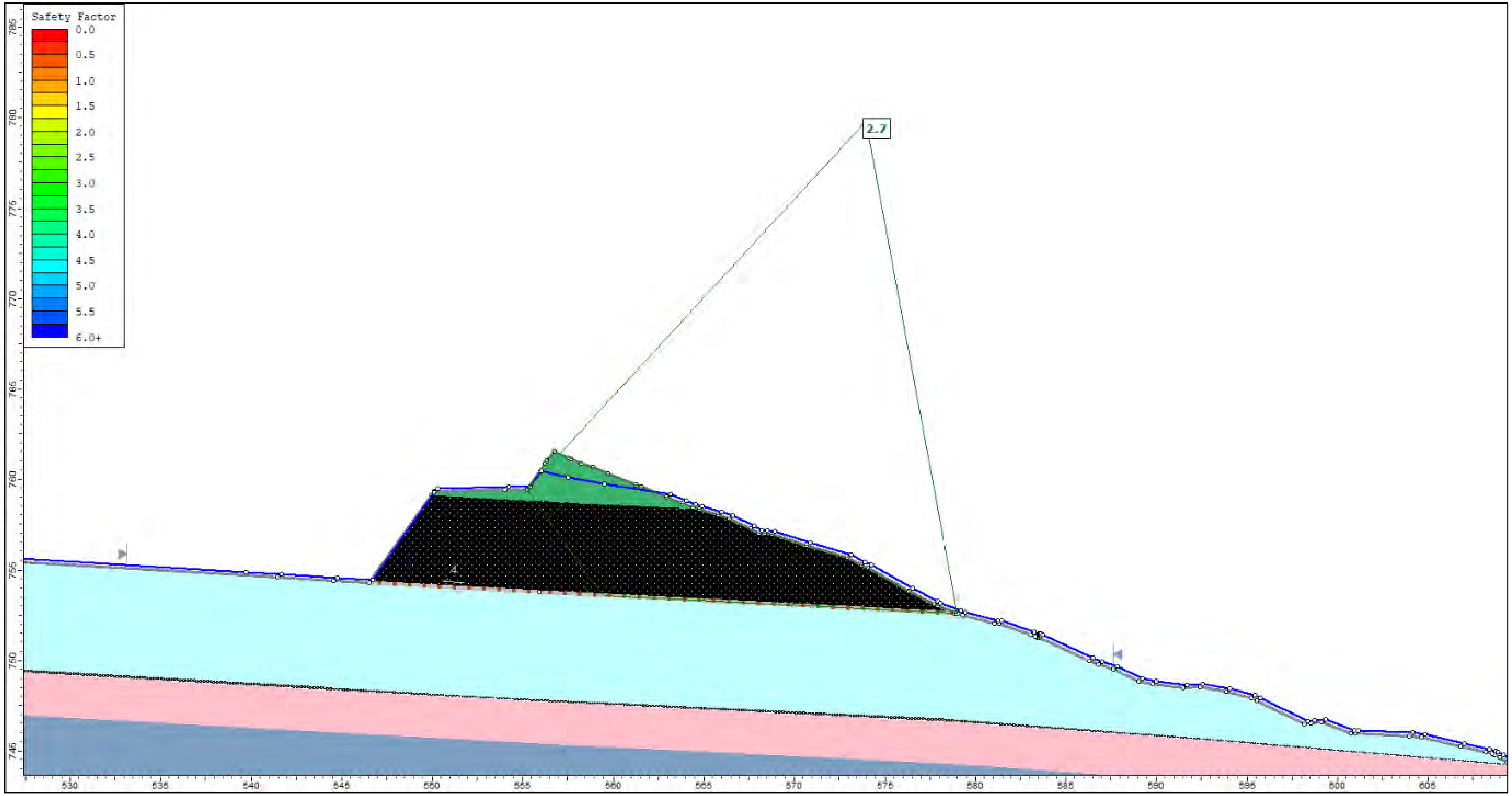

Figure B109 –East Pit Highwall (South-Western) – Downslope Failure - Section 104 – Elevated Groundwater Conditions.

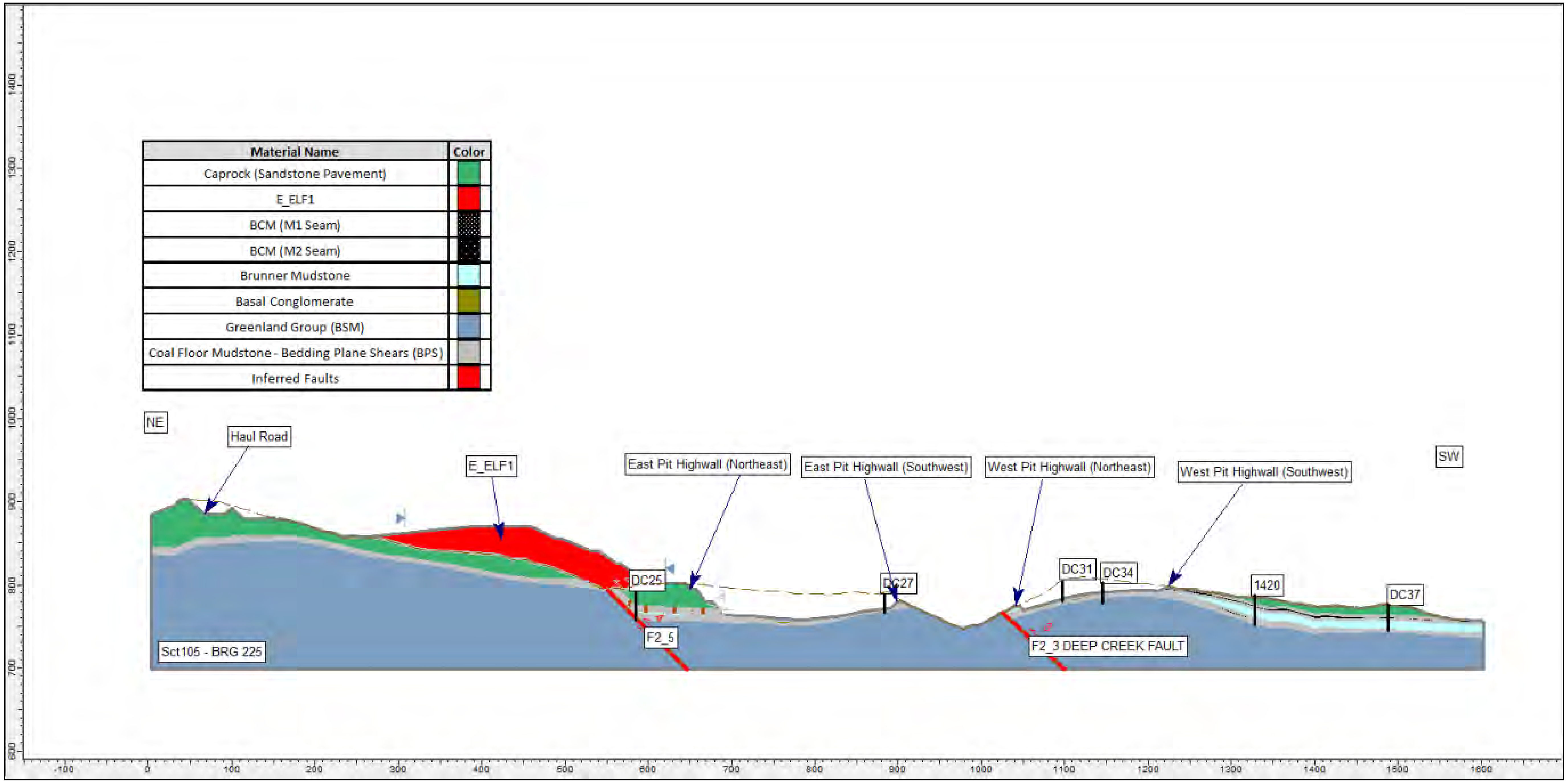

Figure: B110 - Section 105 – Mine Feature Overview

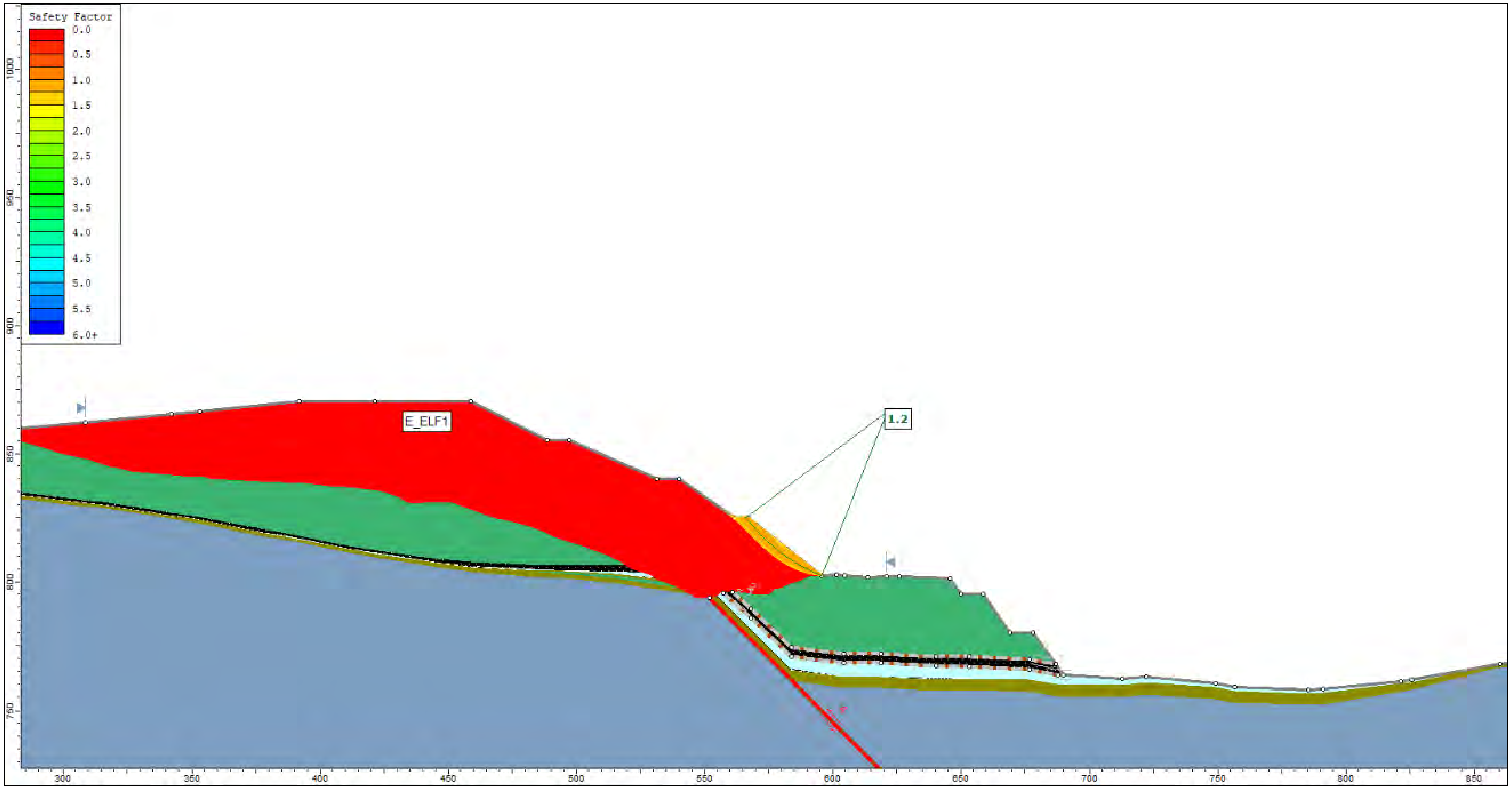

Figure: B111 –E_ELF1 - Section 105 – Static Conditions.

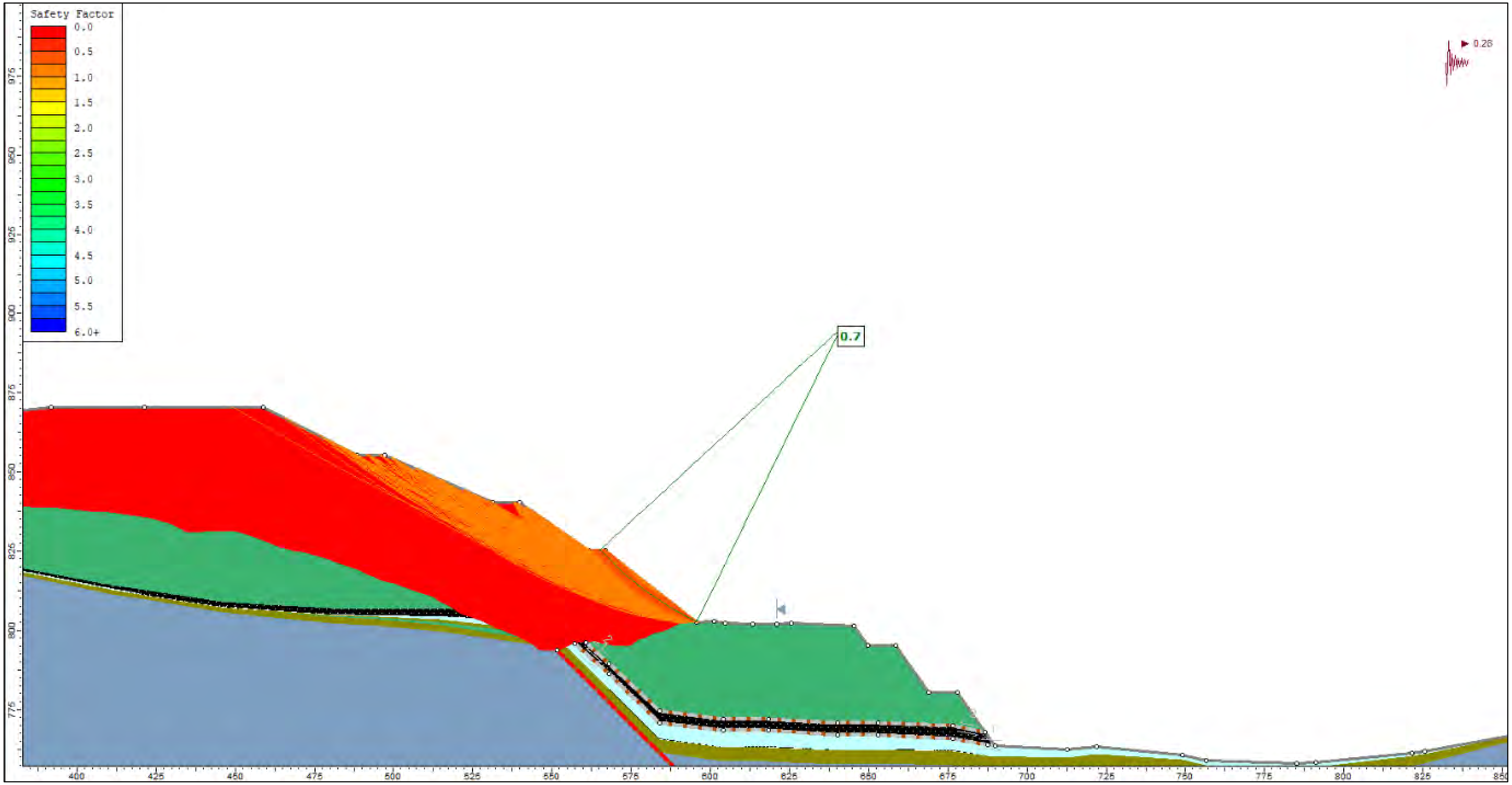

Figure: B112 –E_ELF1 - Section 105 – SLS Conditions.

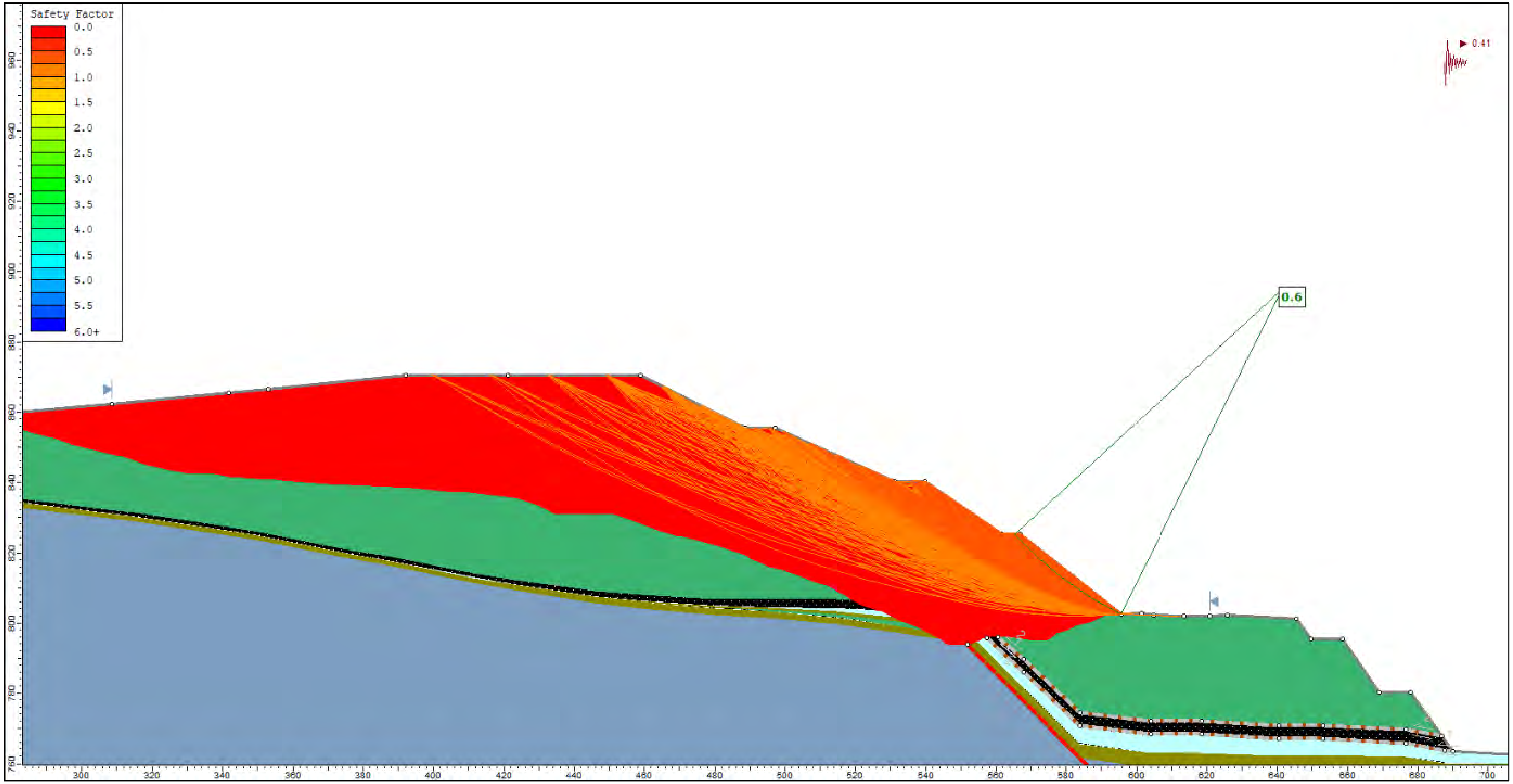

Figure: B113 –E_ELF1 - Section 105 – ULS Conditions.

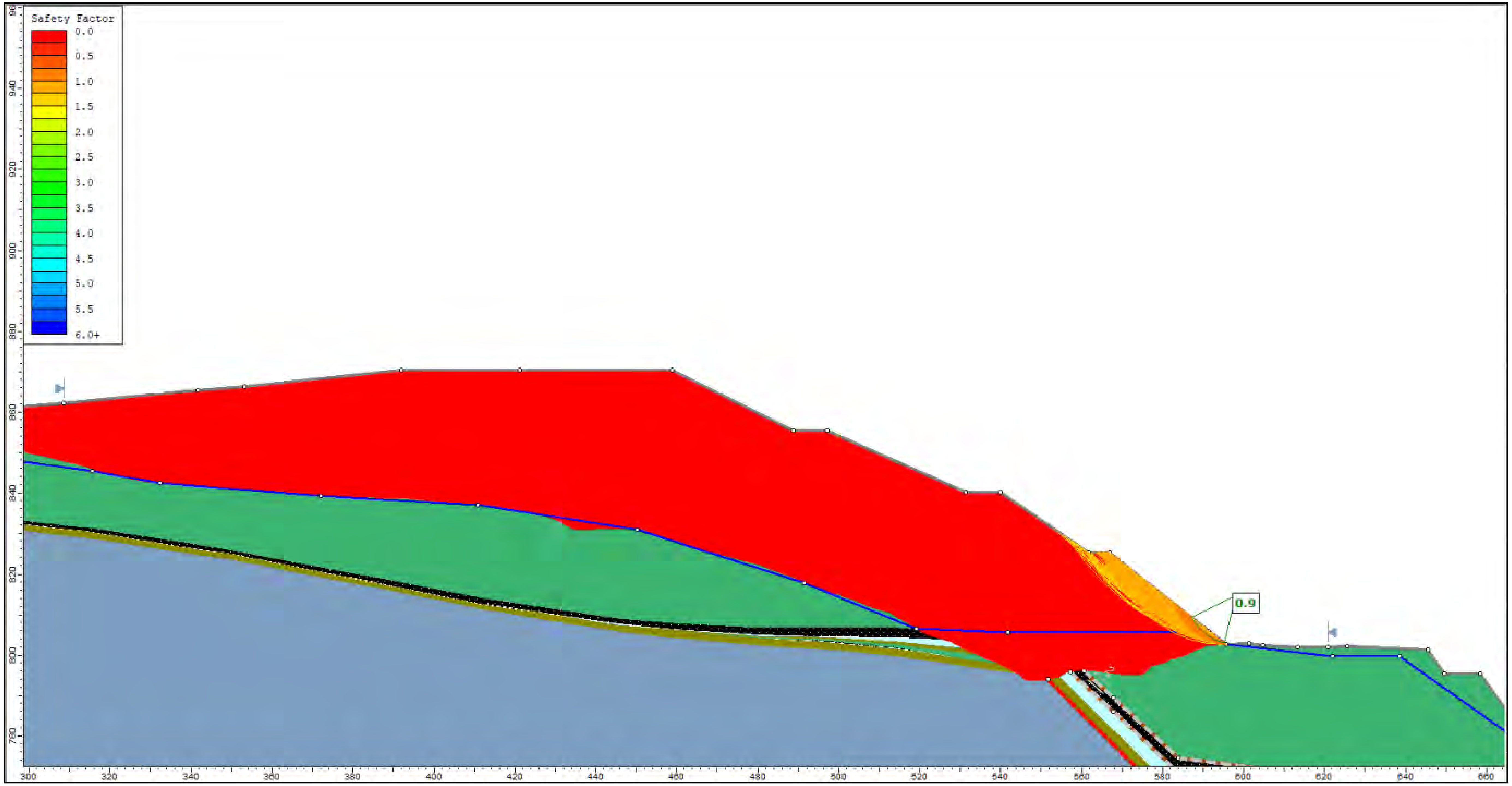

Figure: B114 – E_ELF1 - Section 105 – Elevated Groundwater Conditions.

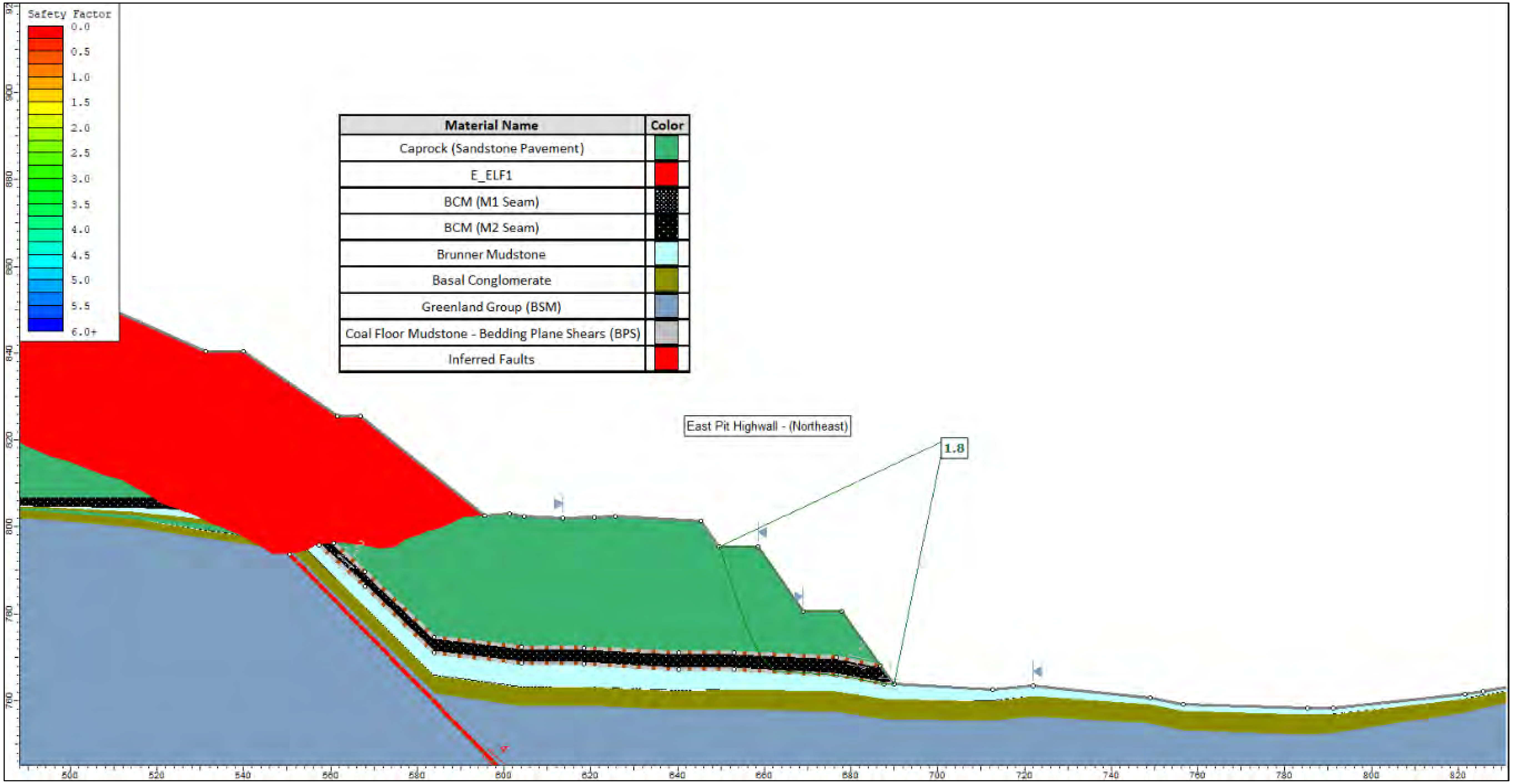

Figure: B115 – East Pit Highwall (Northeast) - Section 105 – Static Conditions.

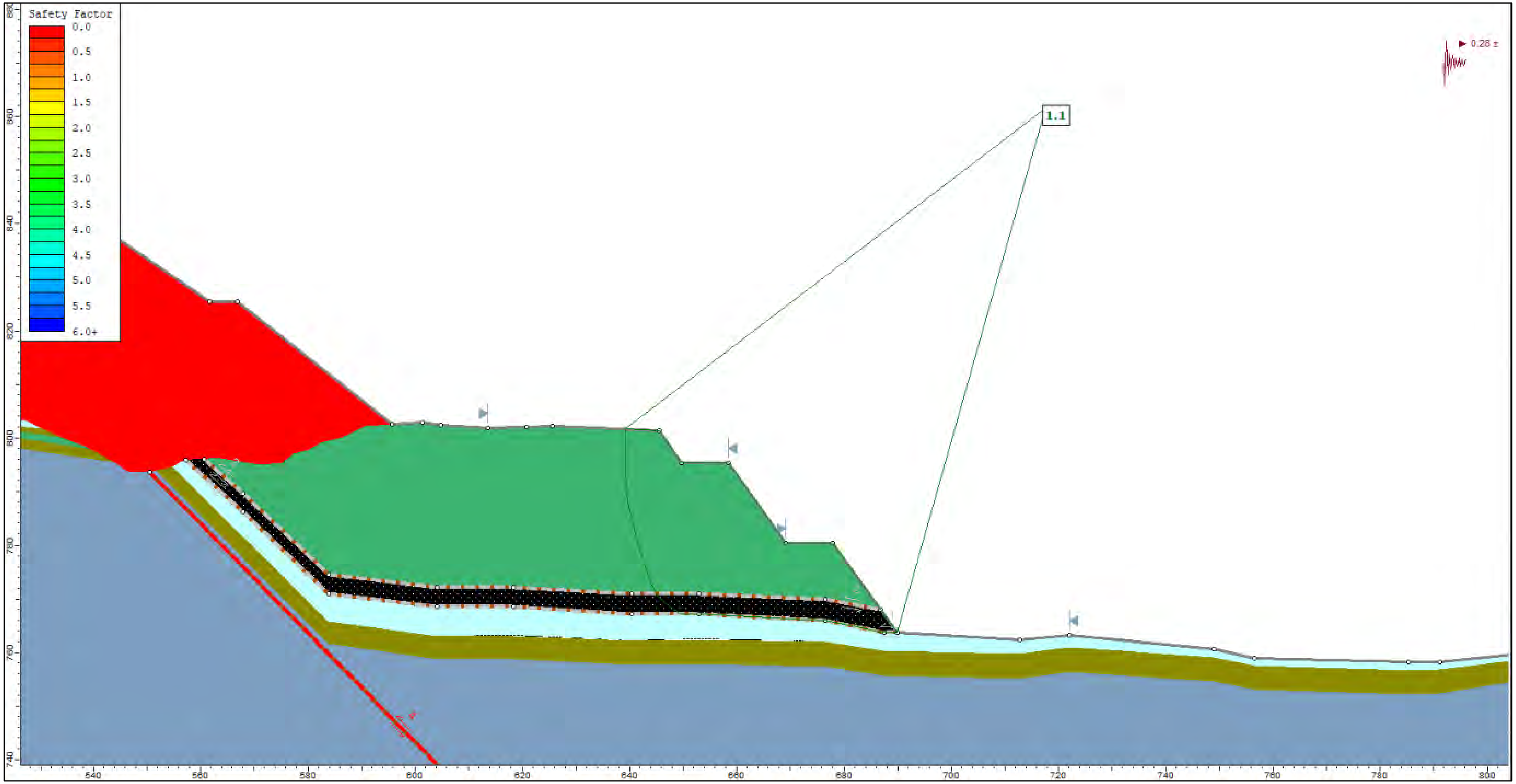

Figure: B116 – East Pit Highwall (Northeast) - Section 105 – SLS Conditions.

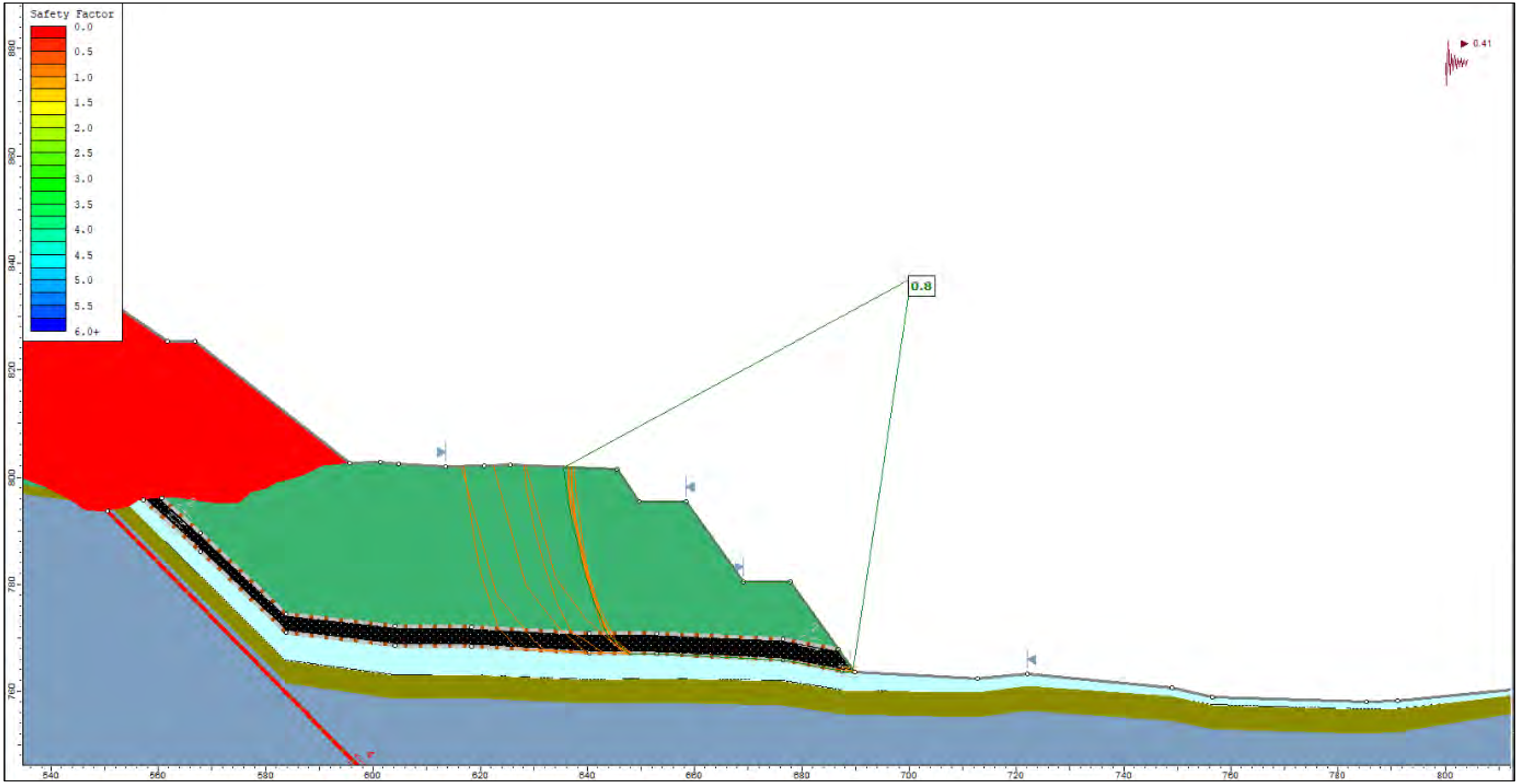

Figure: B117 – East Pit Highwall (Northeast) - Section 105 – ULS Conditions.

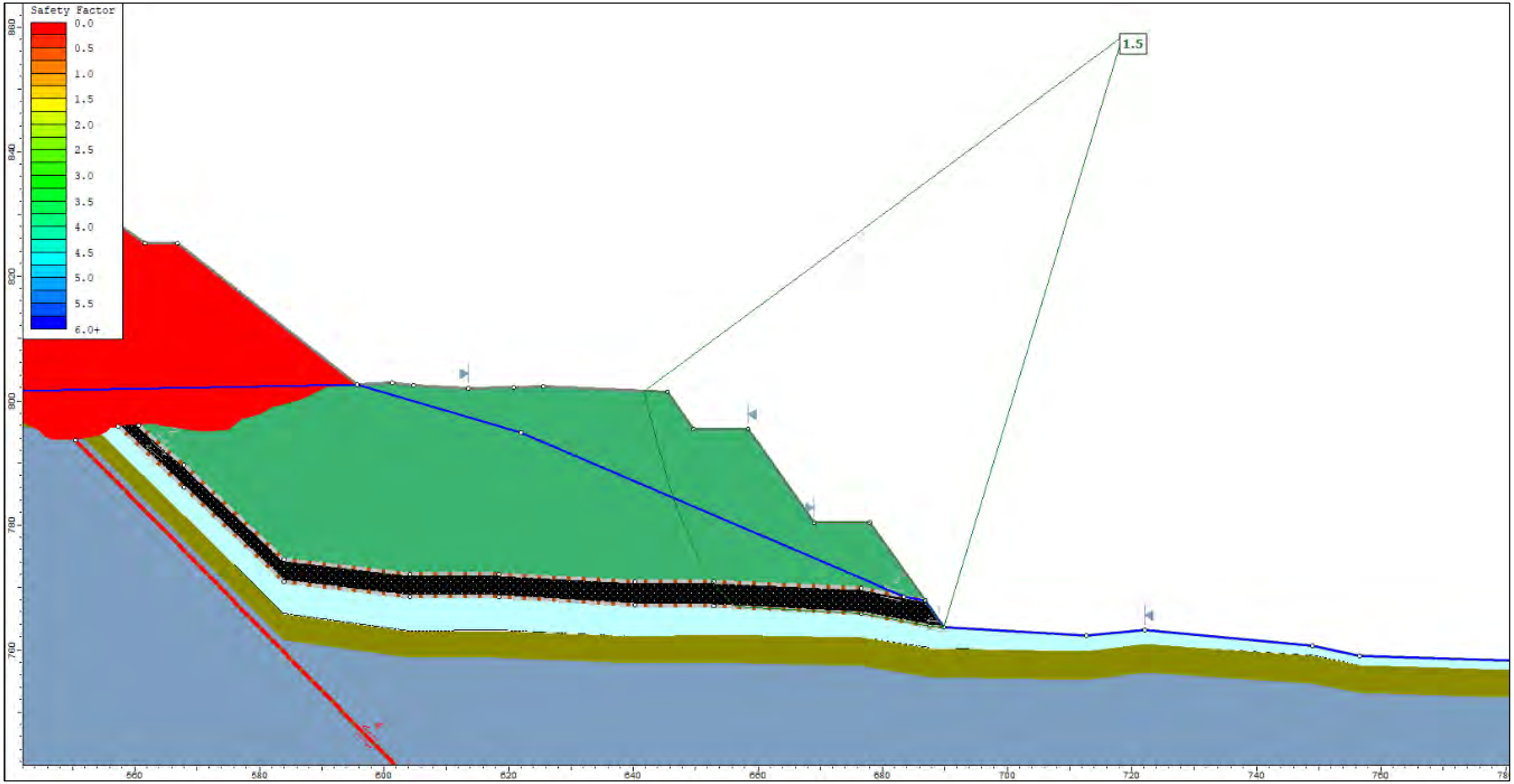

Figure: B118 – East Pit Highwall (Northeast) - Section 105 – Elevated Groundwater Conditions.

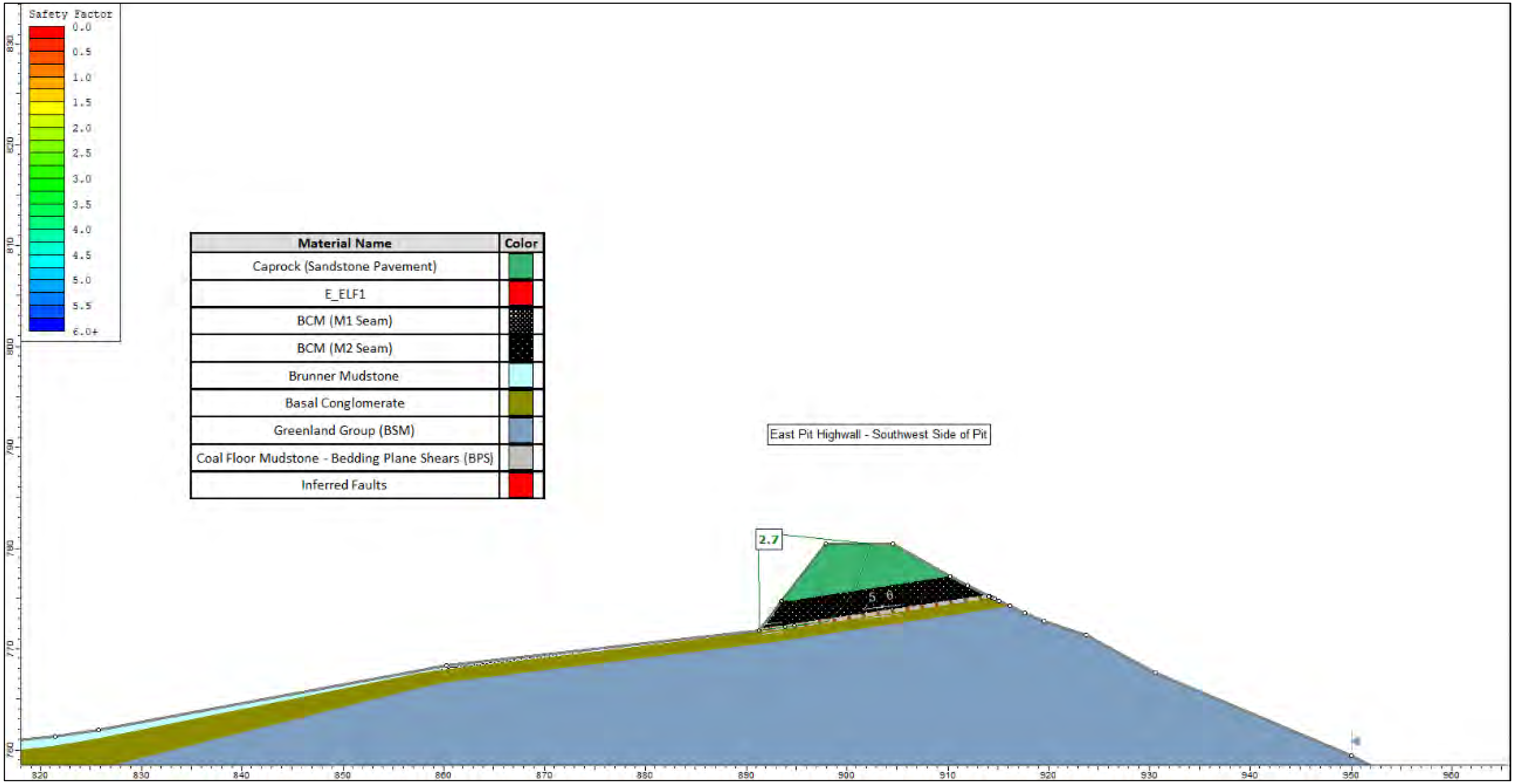

Figure: B119 – East Pit Highwall (Southwest) - Section 105 –Static Conditions.

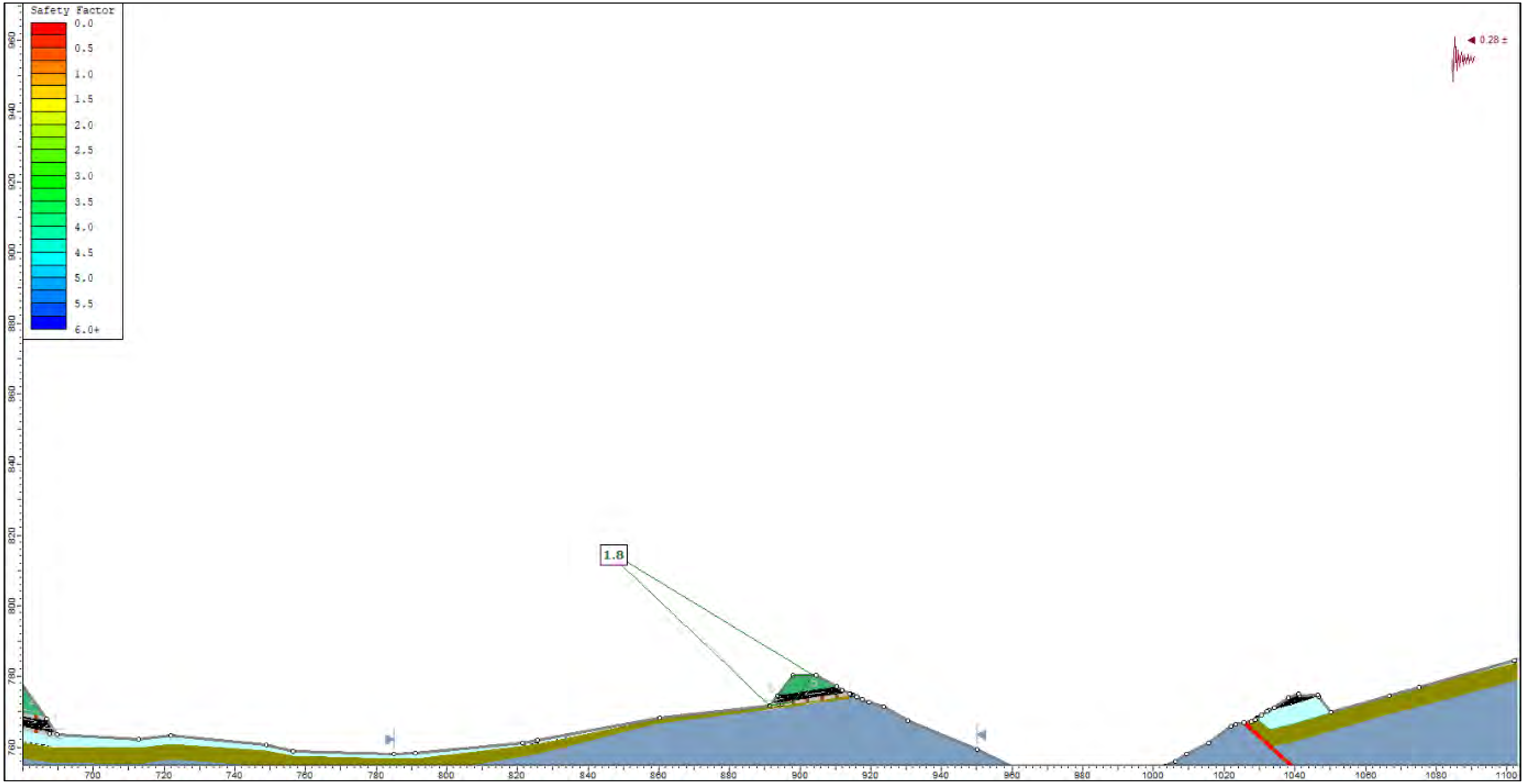

Figure: B120 – East Pit Highwall (Southwest) - Section 105 –SLS Conditions.

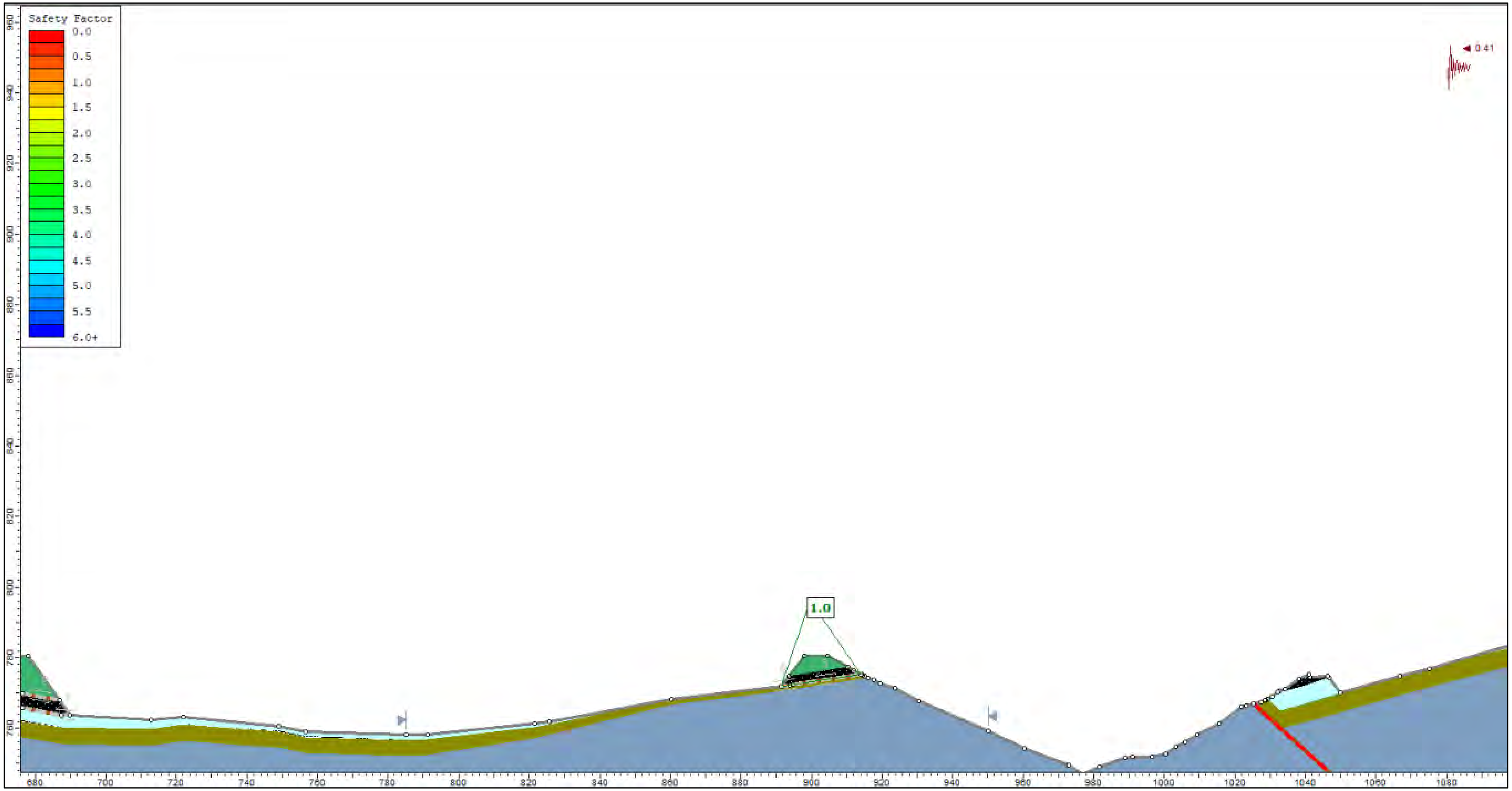

Figure: B121– East Pit Highwall (Southwest) - Section 105 –ULS Conditions.

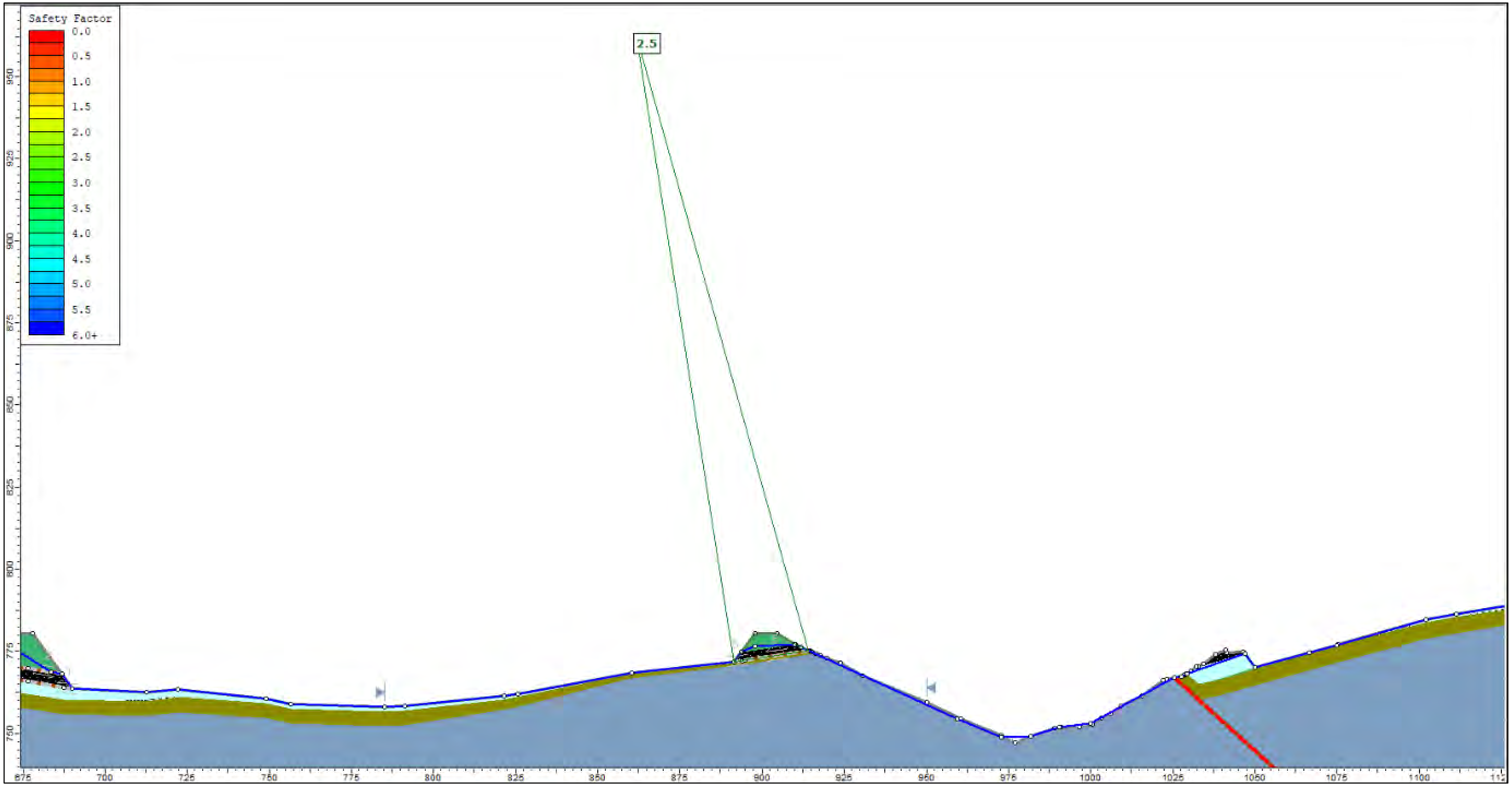

Figure: B122 – East Pit Highwall (Southwest) - Section 105 –Elevated Groundwater Conditions.

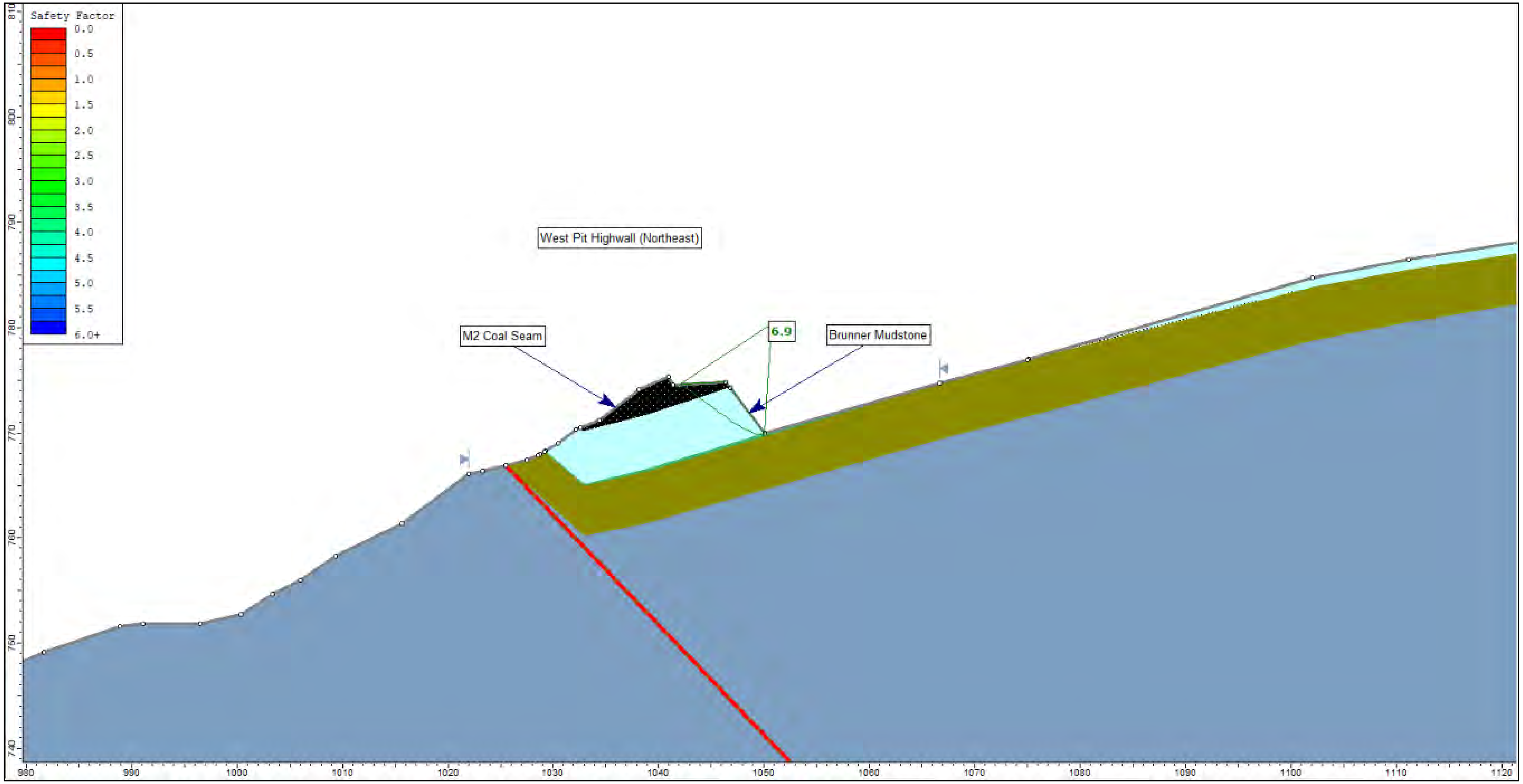

Figure: B123 – West Pit Highwall (Northeast) – Section 105 – Static Conditions.

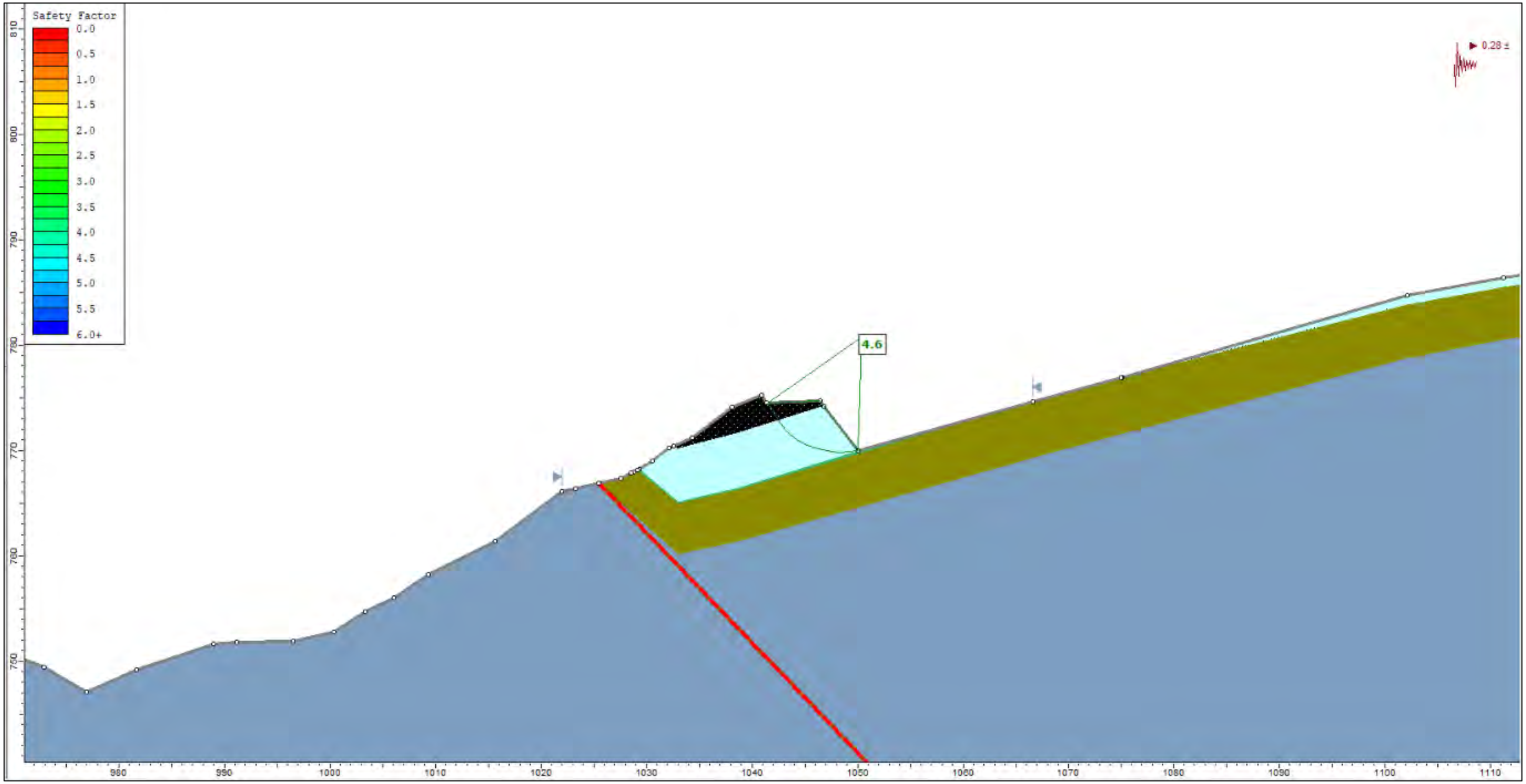

Figure: B124 – West Pit Highwall (Northeast) – Section 105 – SLS Conditions

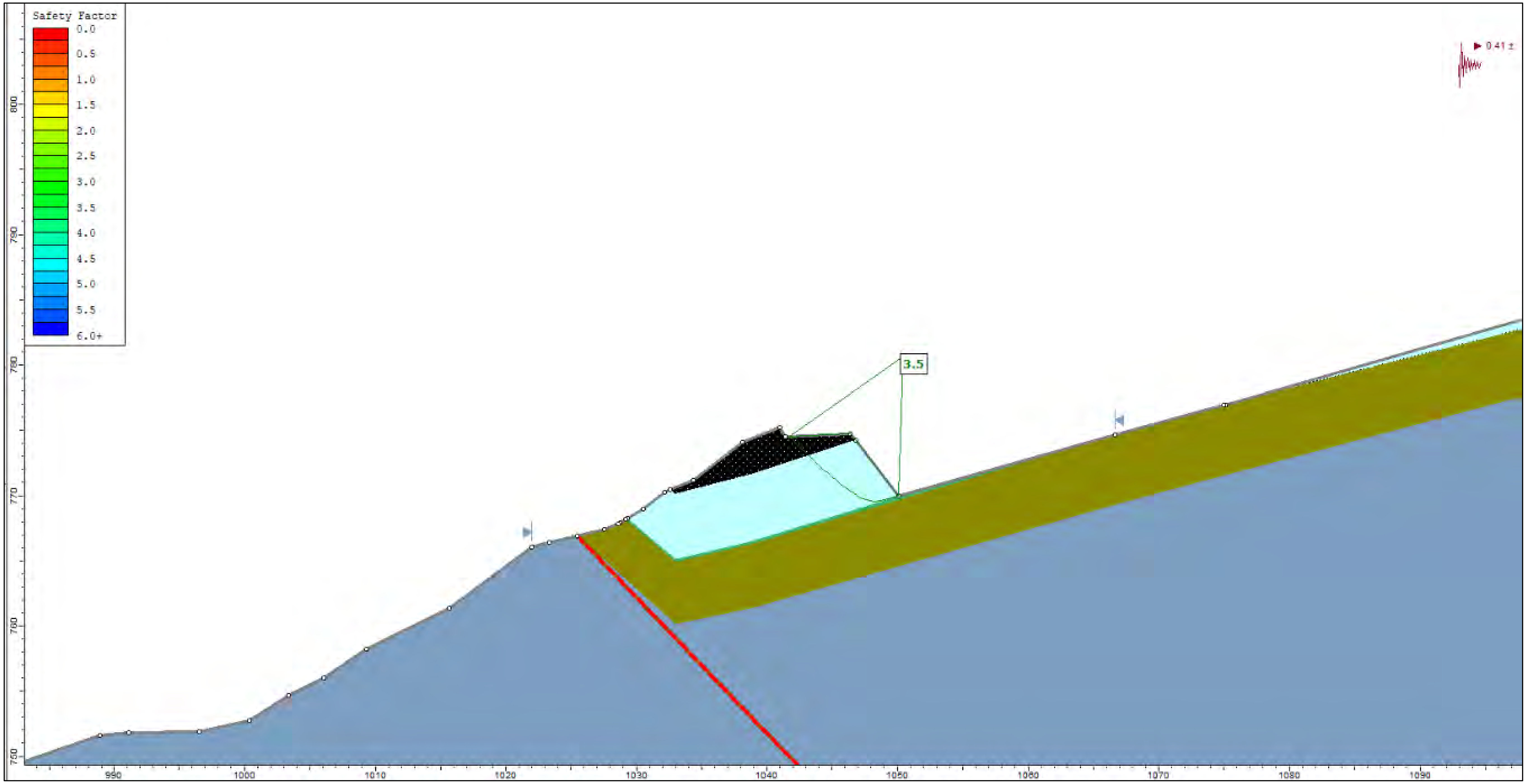

Figure: B125 – West Pit Highwall (Northeast) – Section 105 – ULS Conditions.

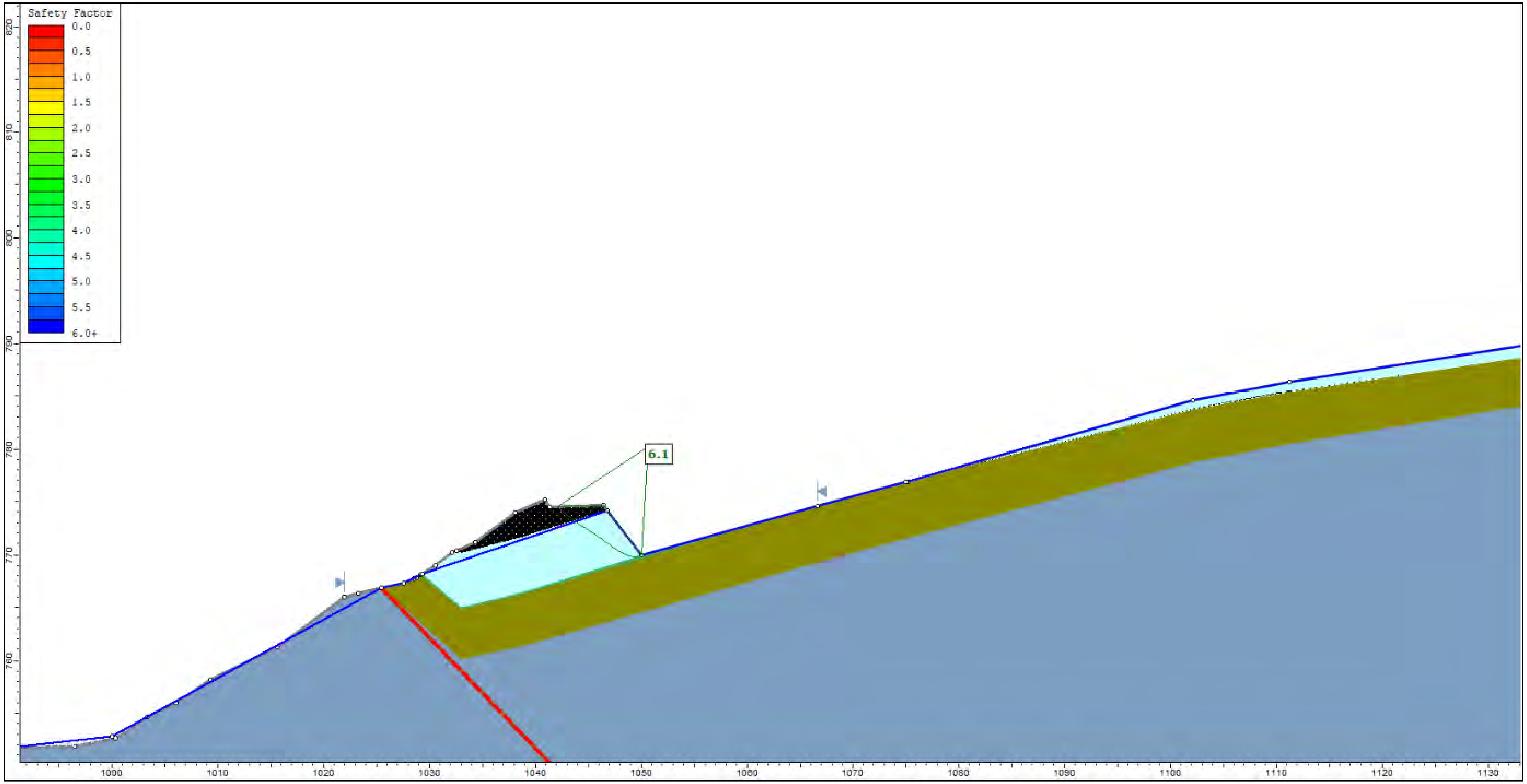

Figure: B126 – West Pit Highwall (Northeast) – Section 105 – Elevated Groundwater Conditions.

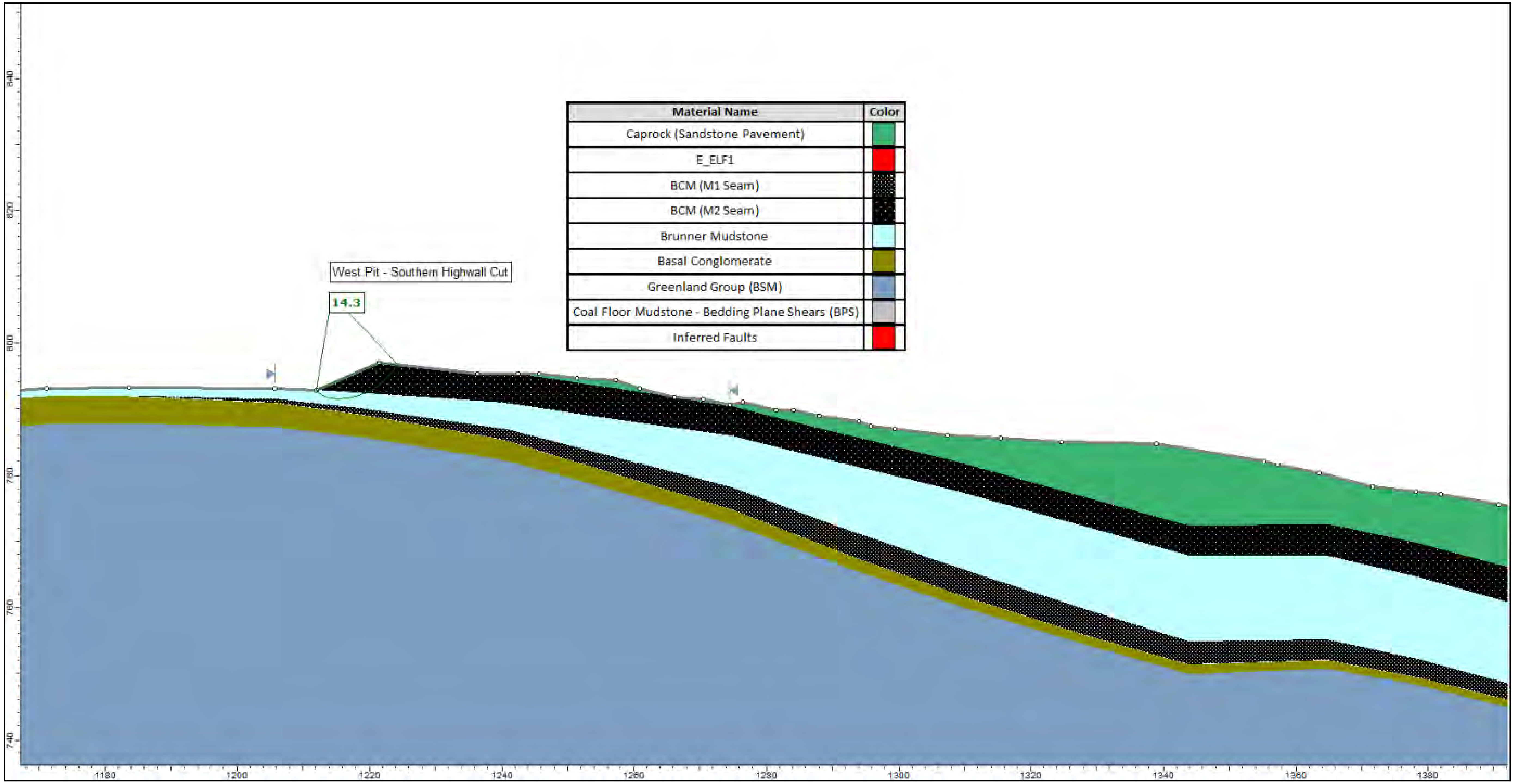

Figure: B127 - West Pit Highwall Cut - Section 105 –Static Conditions.

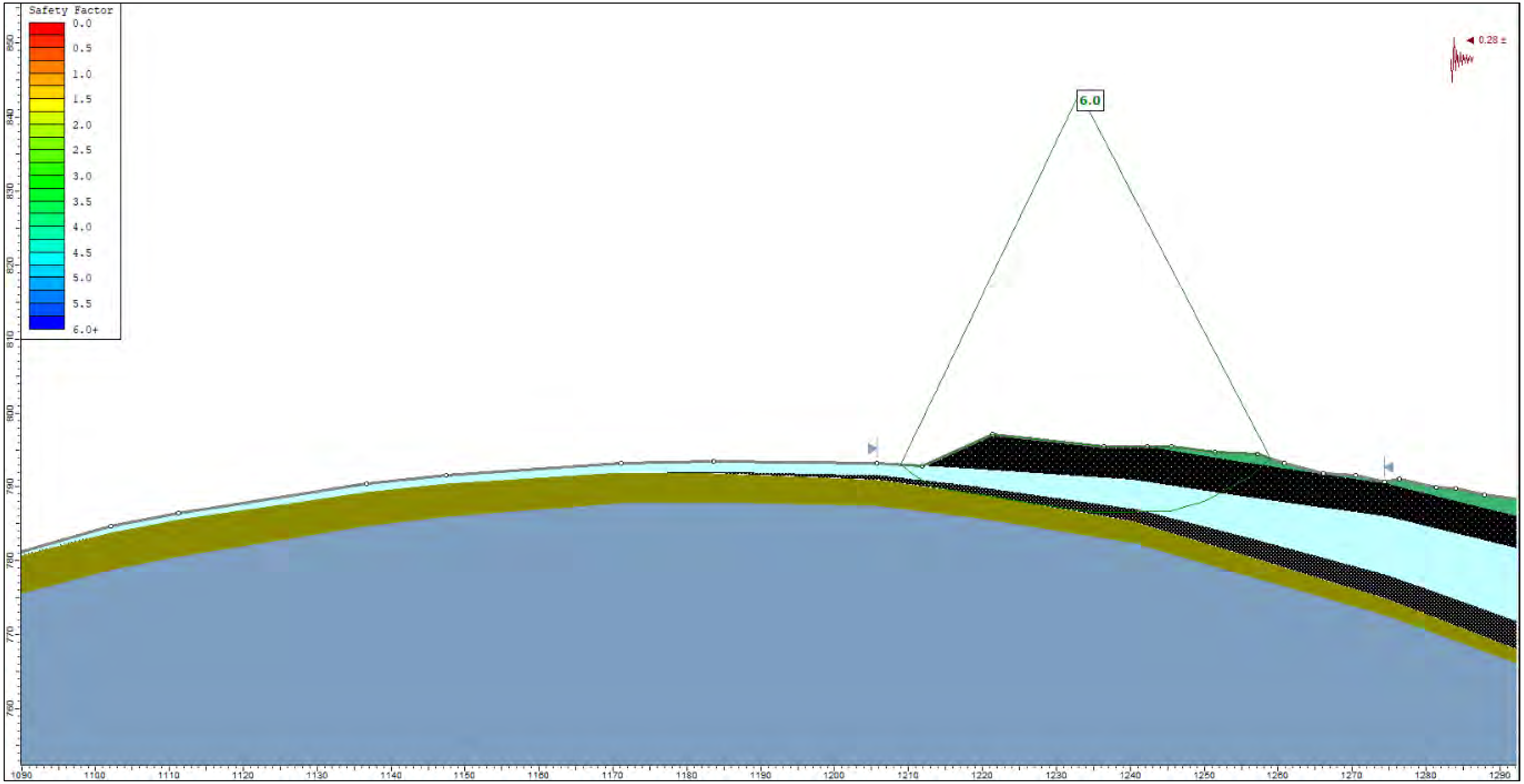

Figure: B128 - West Pit Highwall Cut - Section 105 –SLS Conditions.

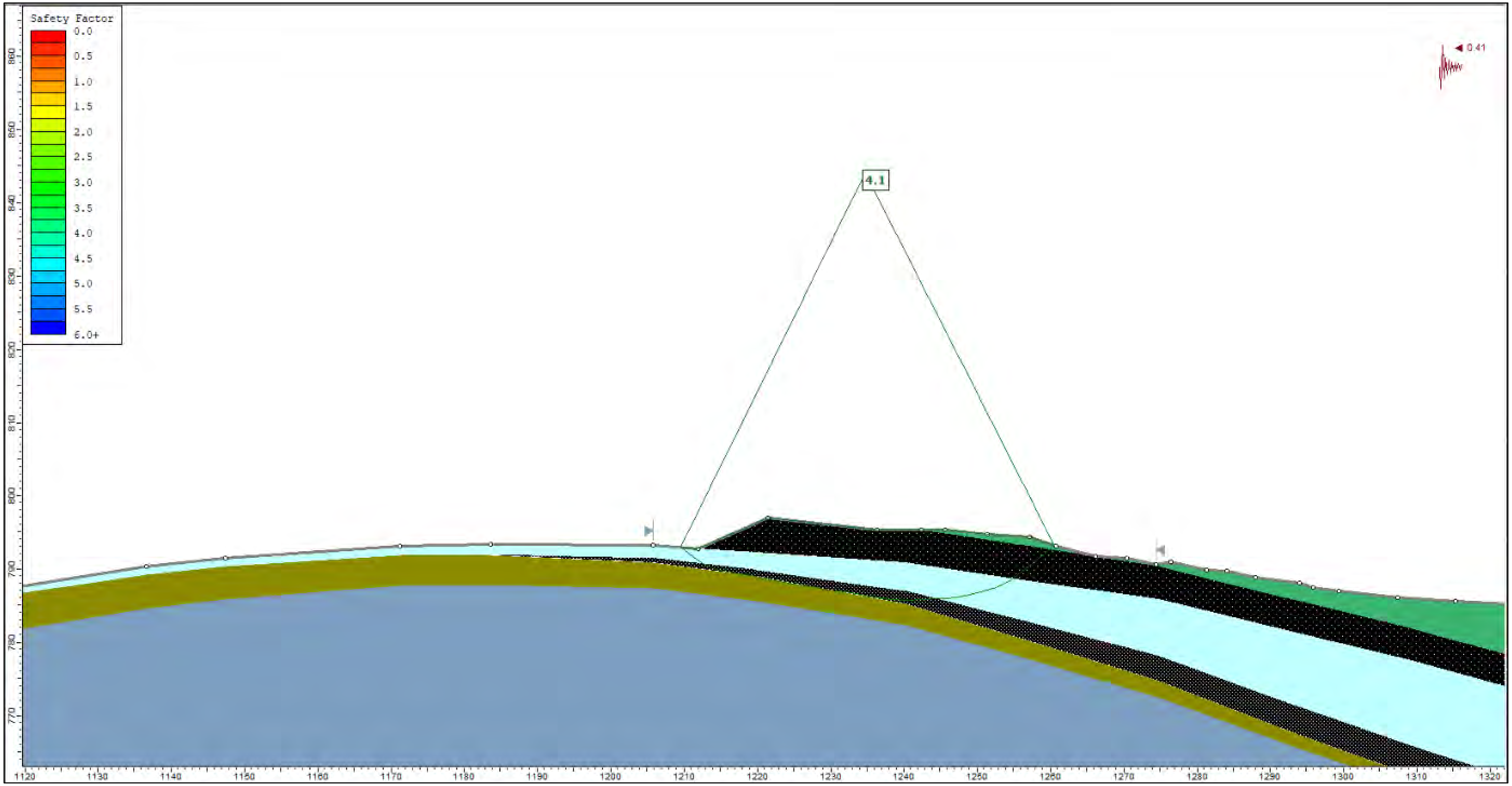

Figure: B129 - West Pit Highwall Cut - Section 105 –ULS Conditions.

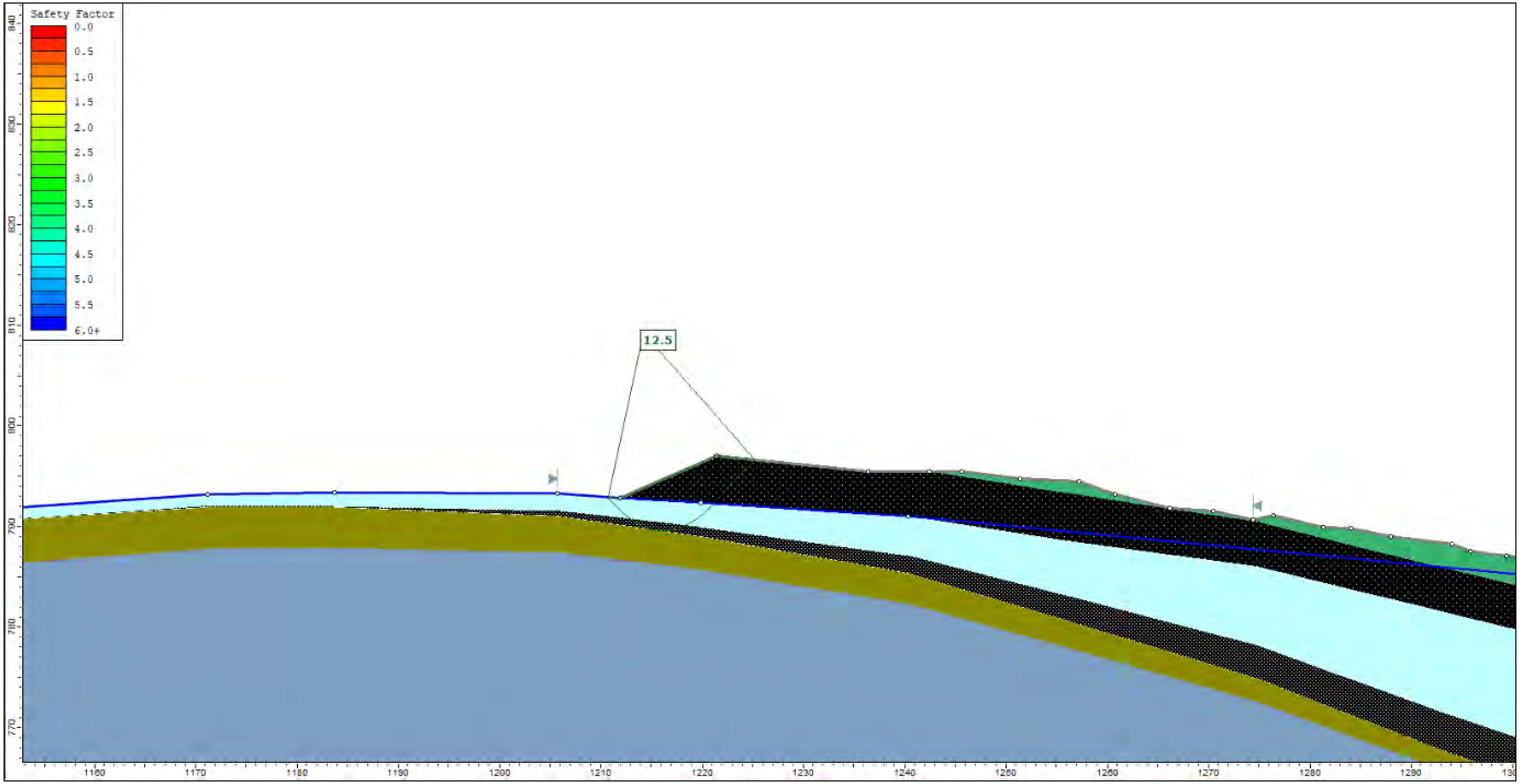

Figure: B130 - West Pit Highwall Cut - Section 105 – Elevated Groundwater Conditions.

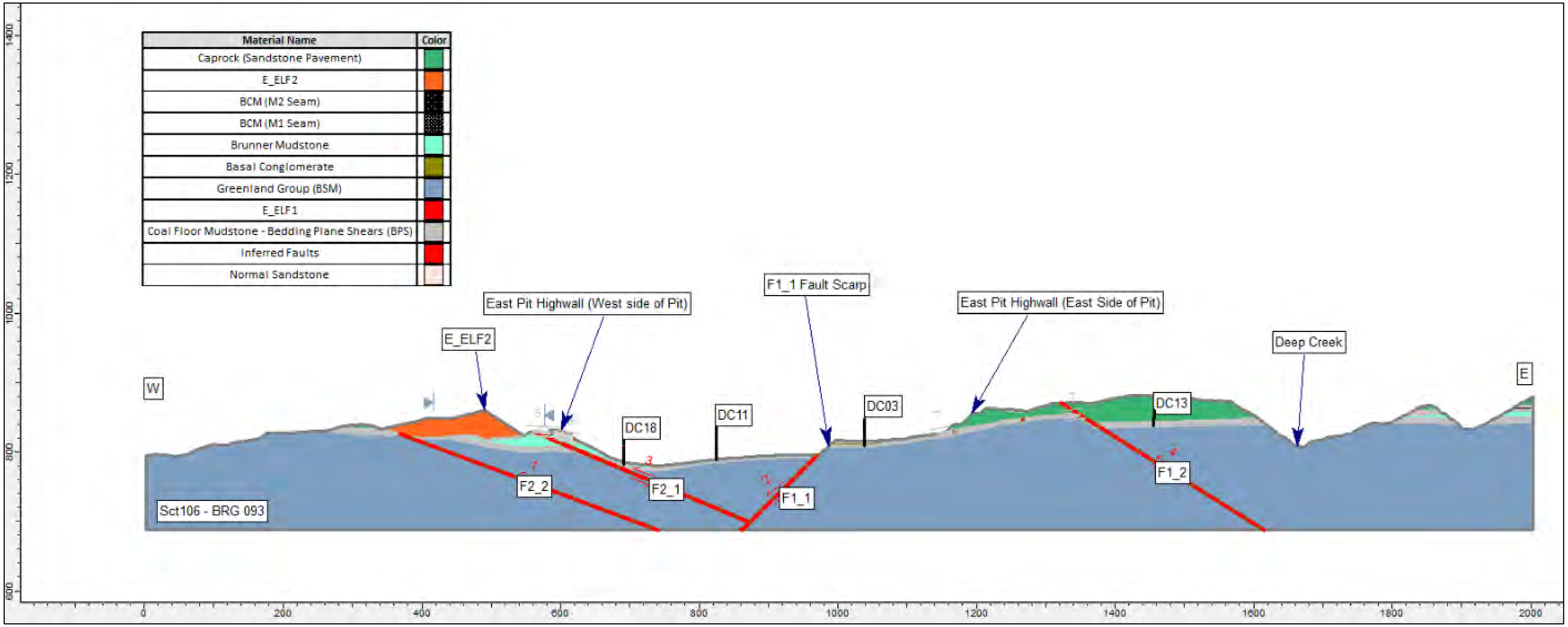

Figure B131 - Section 106 – Mine Feature Overview.

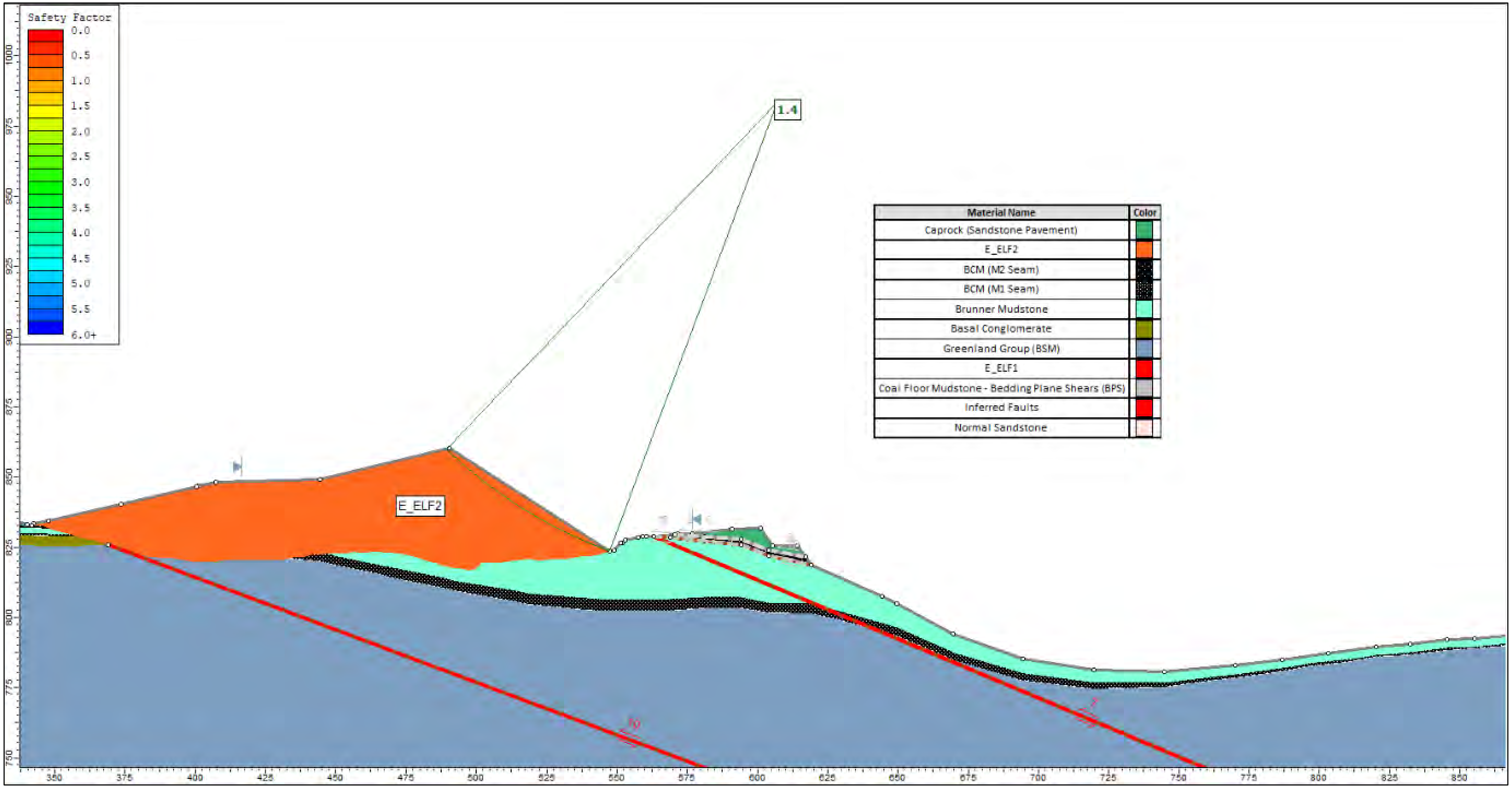

Figure: B132 – E_ELF2 - Section 106 – Static Conditions.

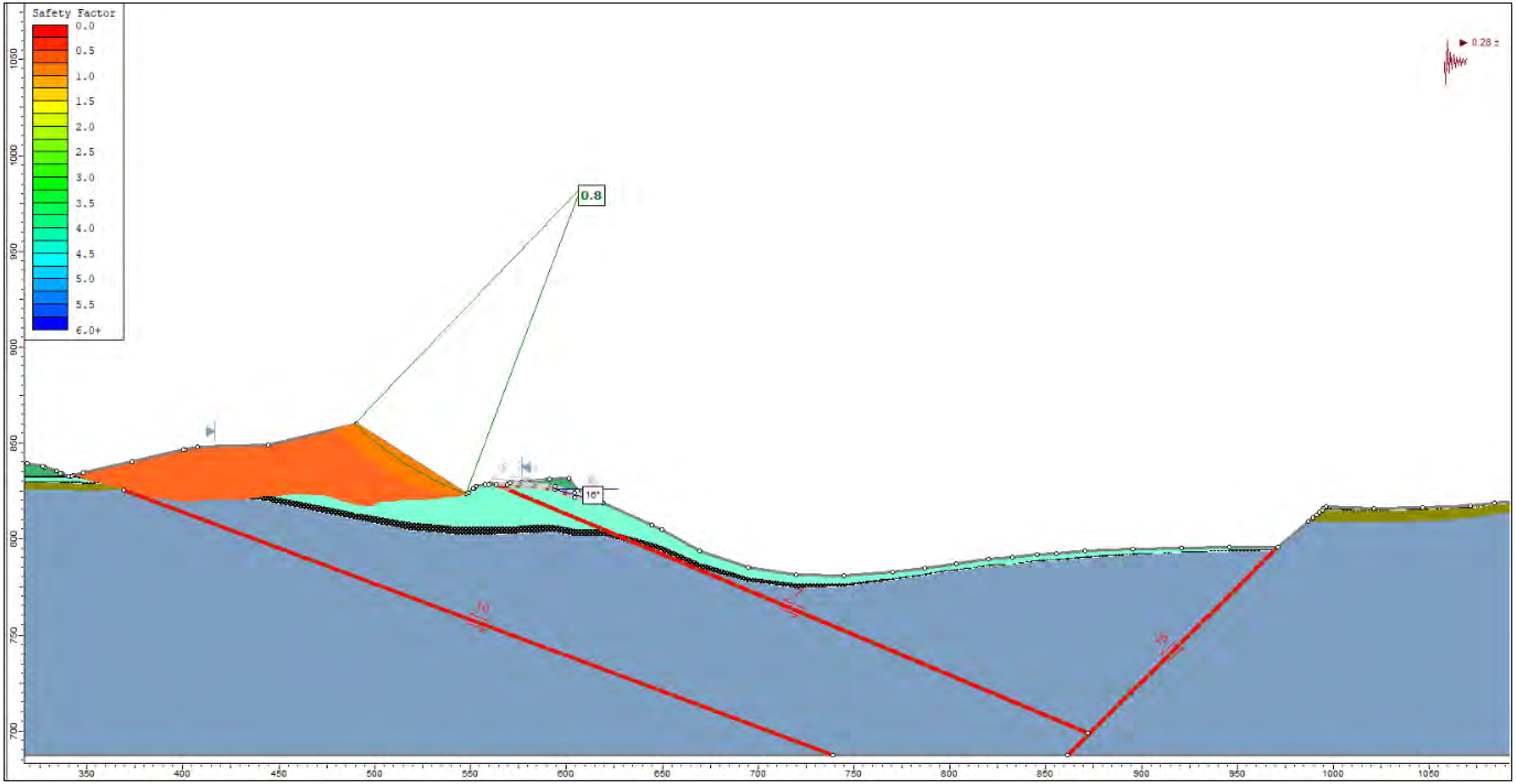

Figure: B133 – E_ELF2 - Section 106 – SLS Conditions.

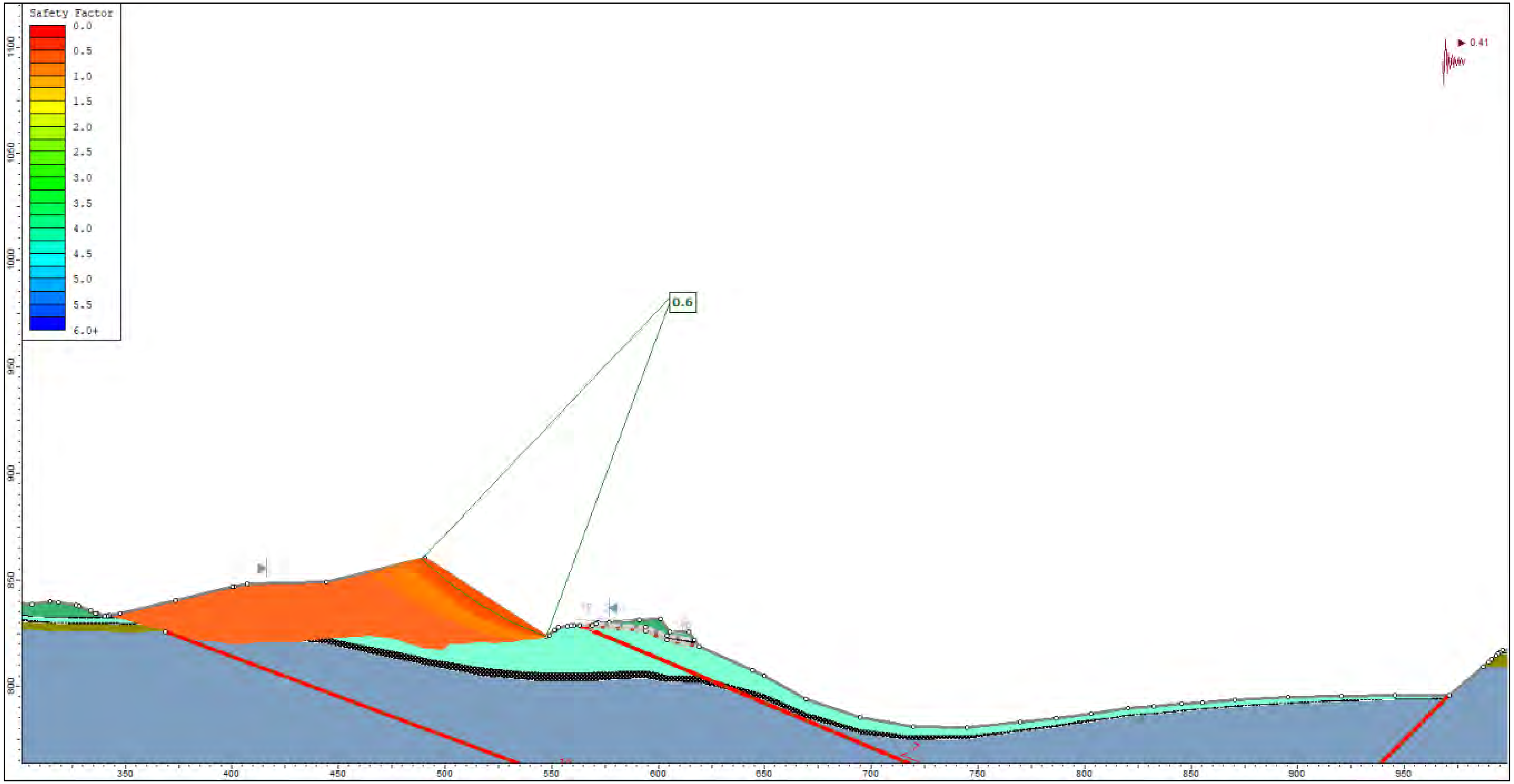

Figure: B134 – E_ELF2 - Section 106 – ULS Conditions.

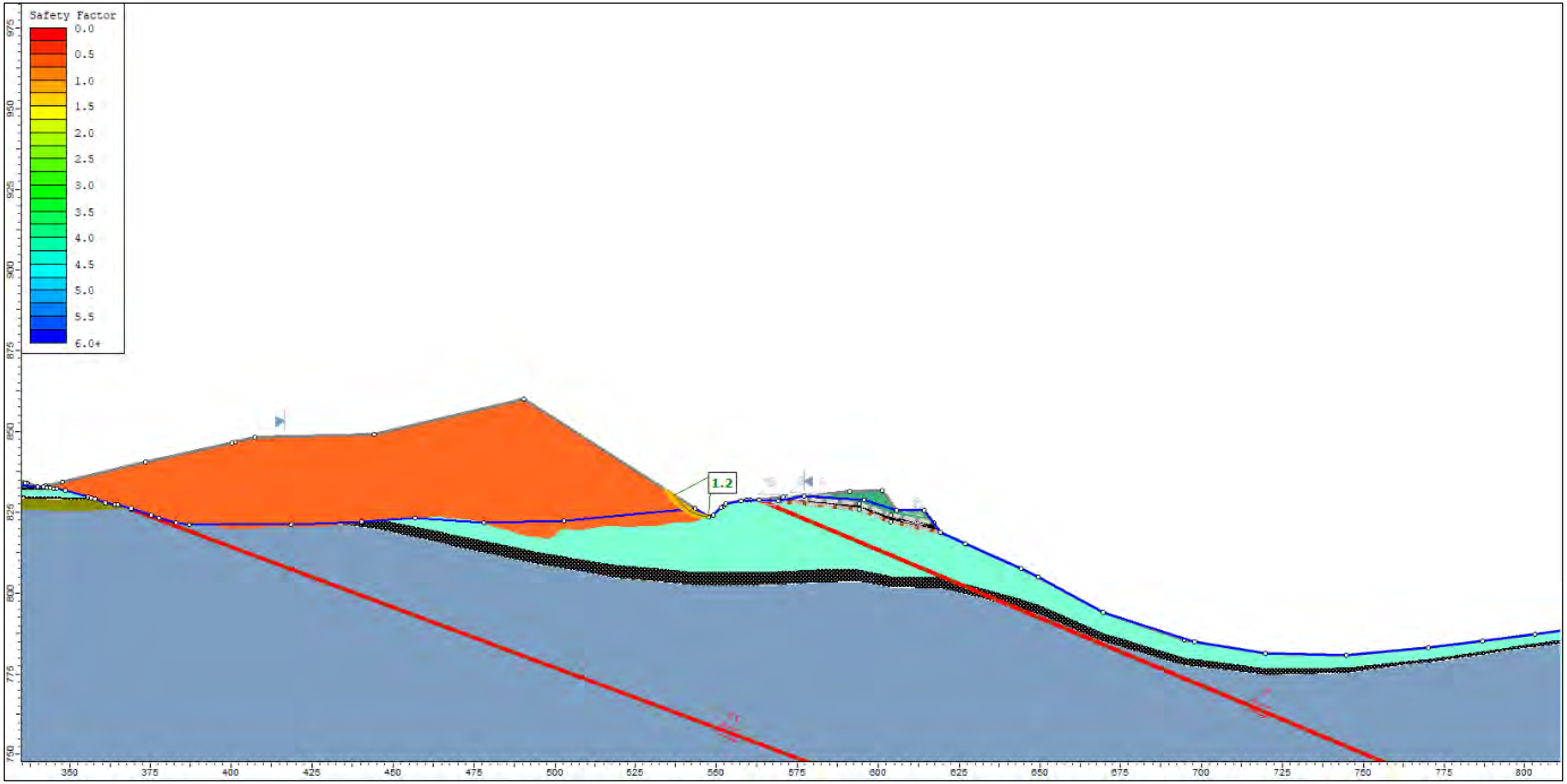

Figure: B135 – E_ELF2 - Section 106 – Elevated Groundwater Conditions.

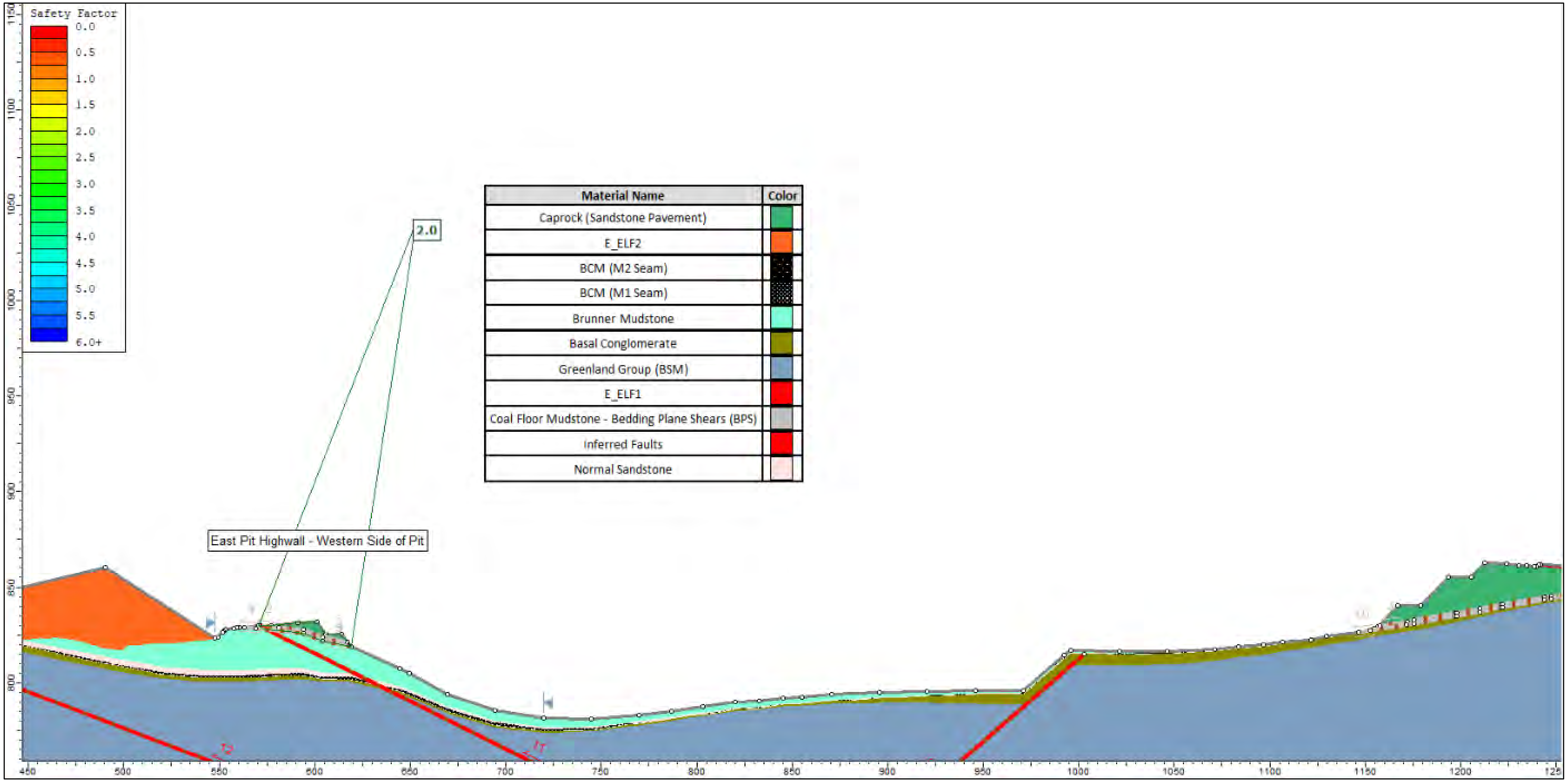

Figure: B136 – East Pit Highwall – Western Side of Pit - Section 106 – Static Conditions.

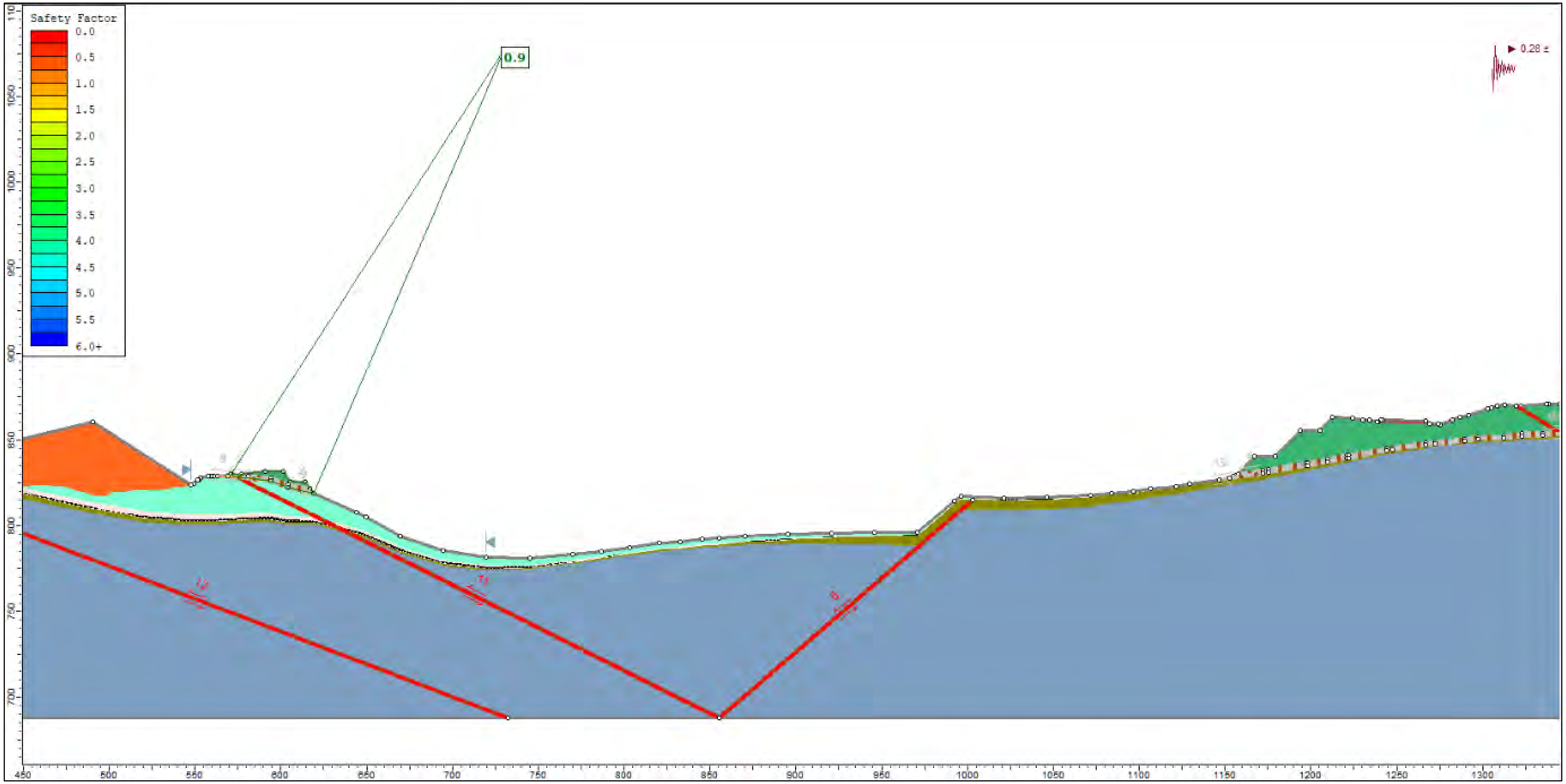

Figure: B137 – East Pit Highwall – Western Side of Pit - Section 106 – SLS Conditions.

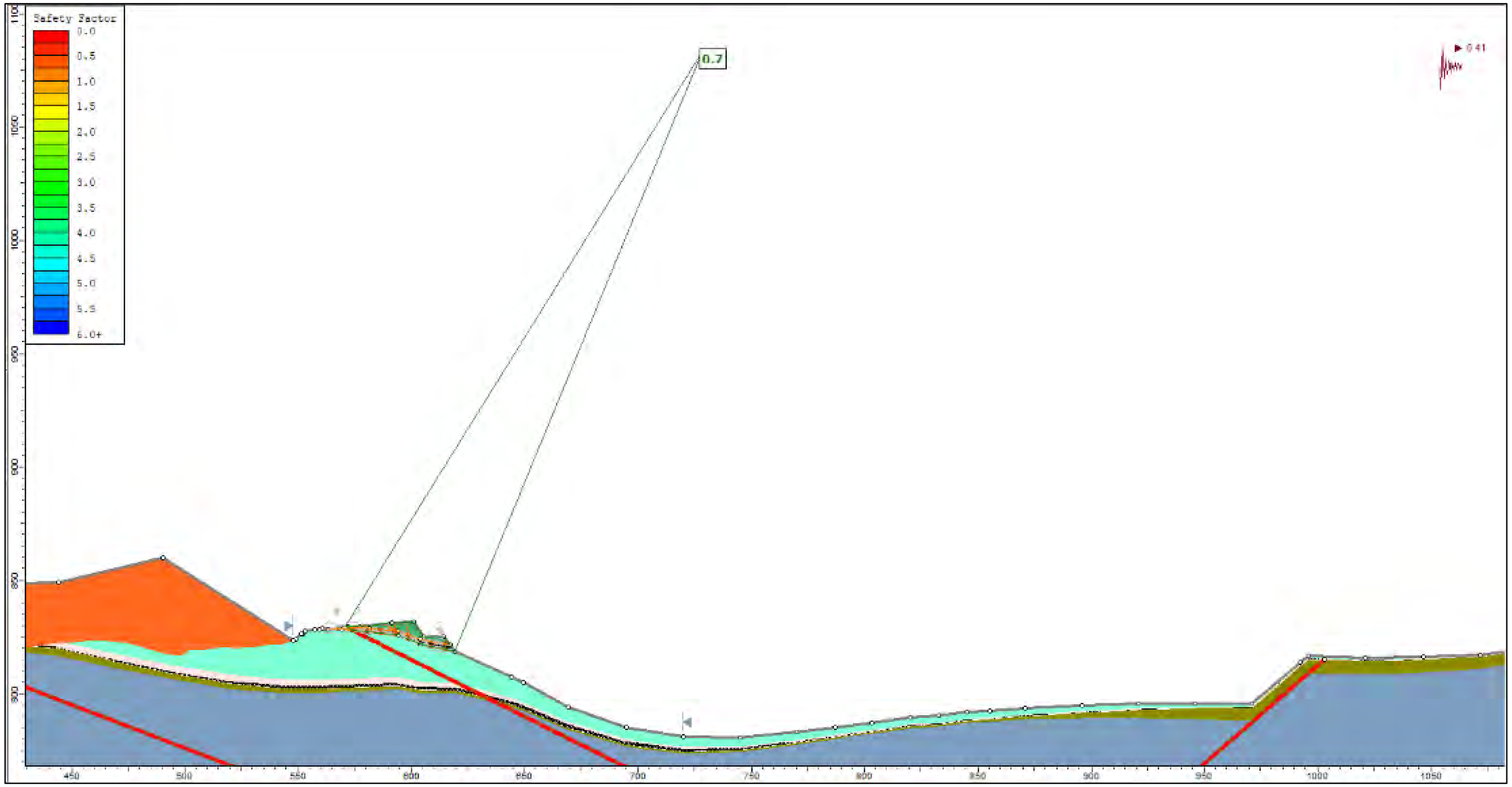

Figure: B138 – East Pit Highwall – Western Side of Pit - Section 106 – ULS Conditions.

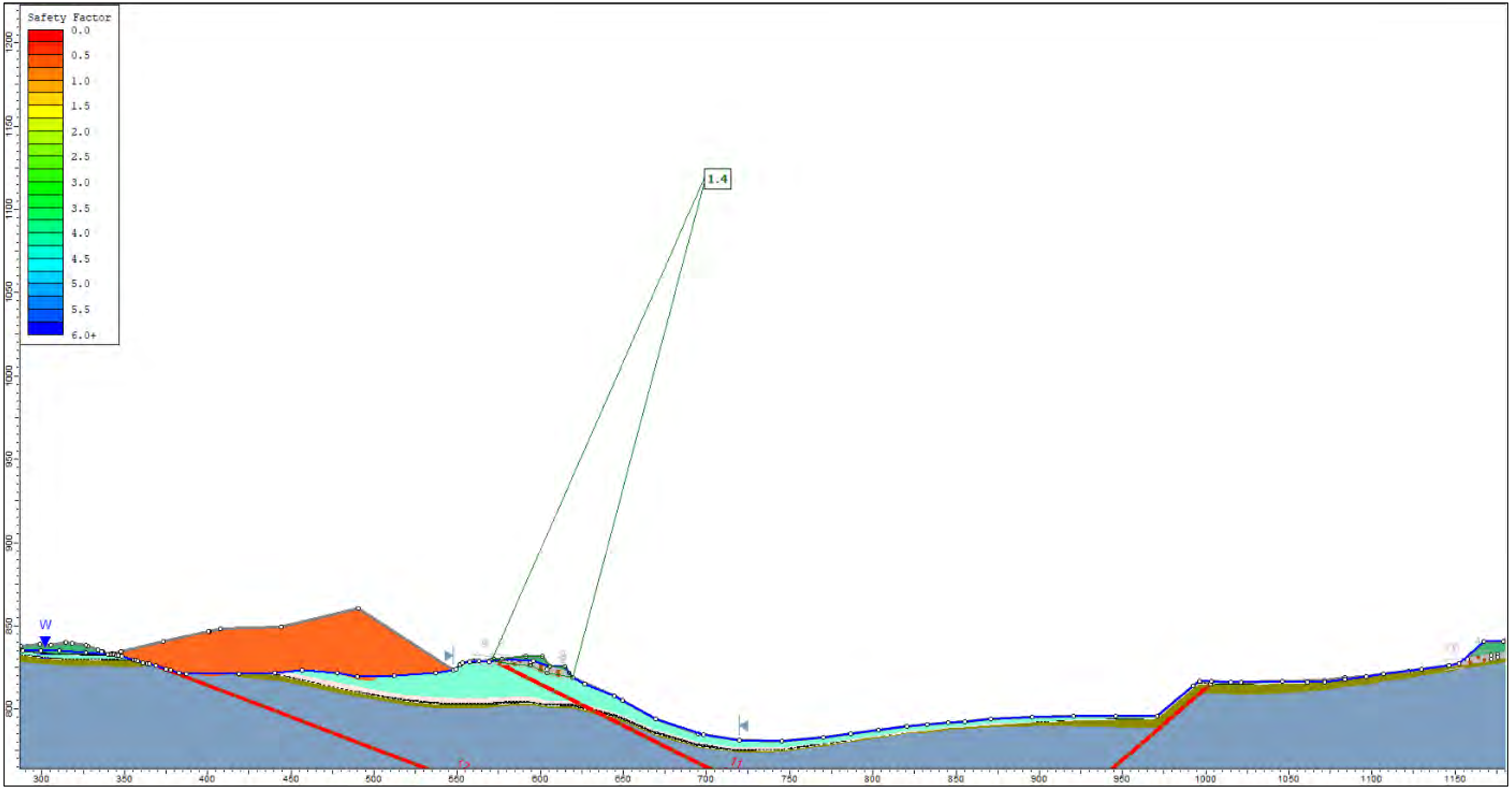

Figure: B139 – East Pit Highwall – Western Side of Pit - Section 106 – Elevated Groundwater Conditions.

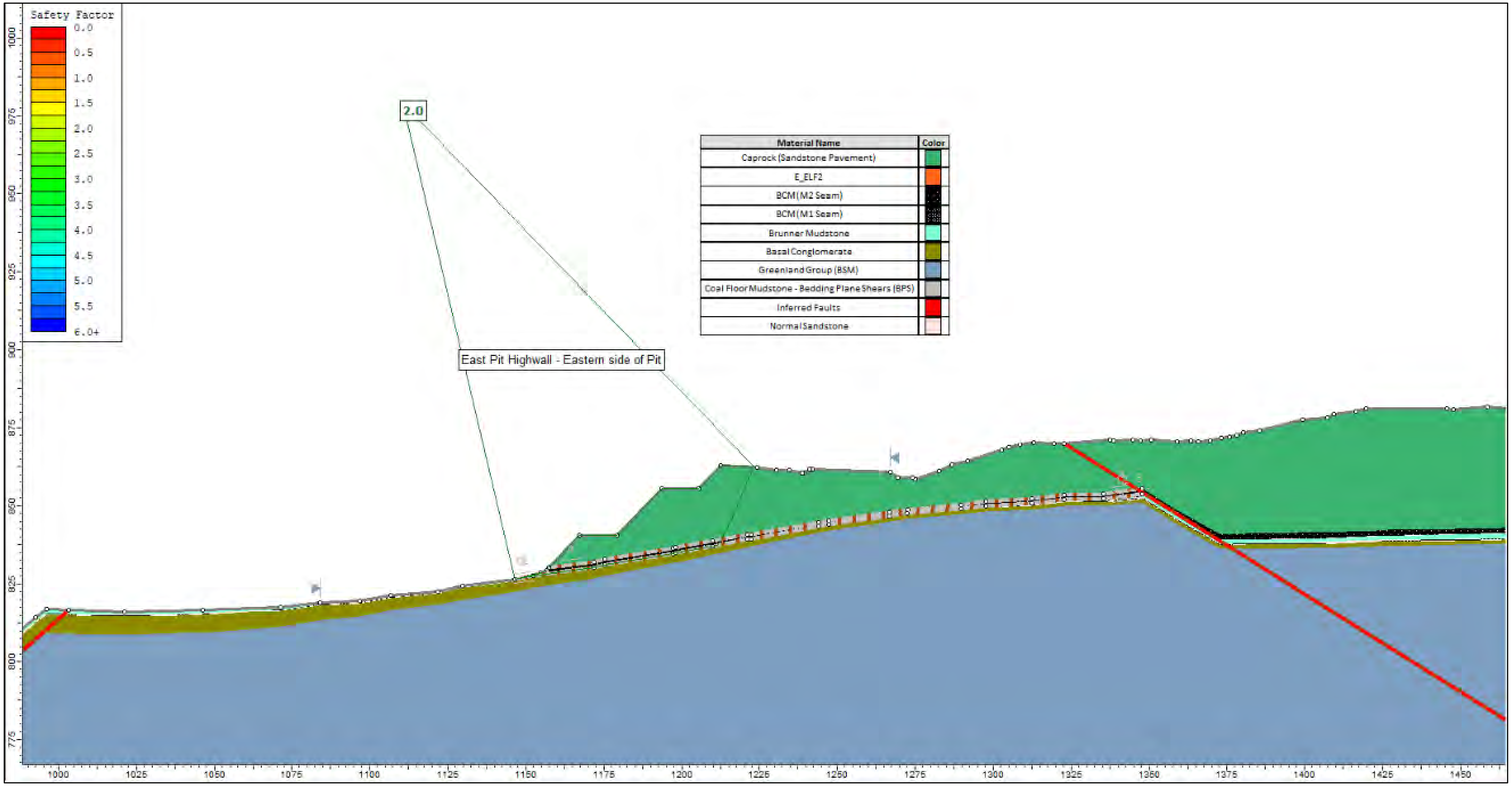

Figure: B140 – East Pit Highwall – Eastern Side of Pit - Section 106 – Static Groundwater Conditions.

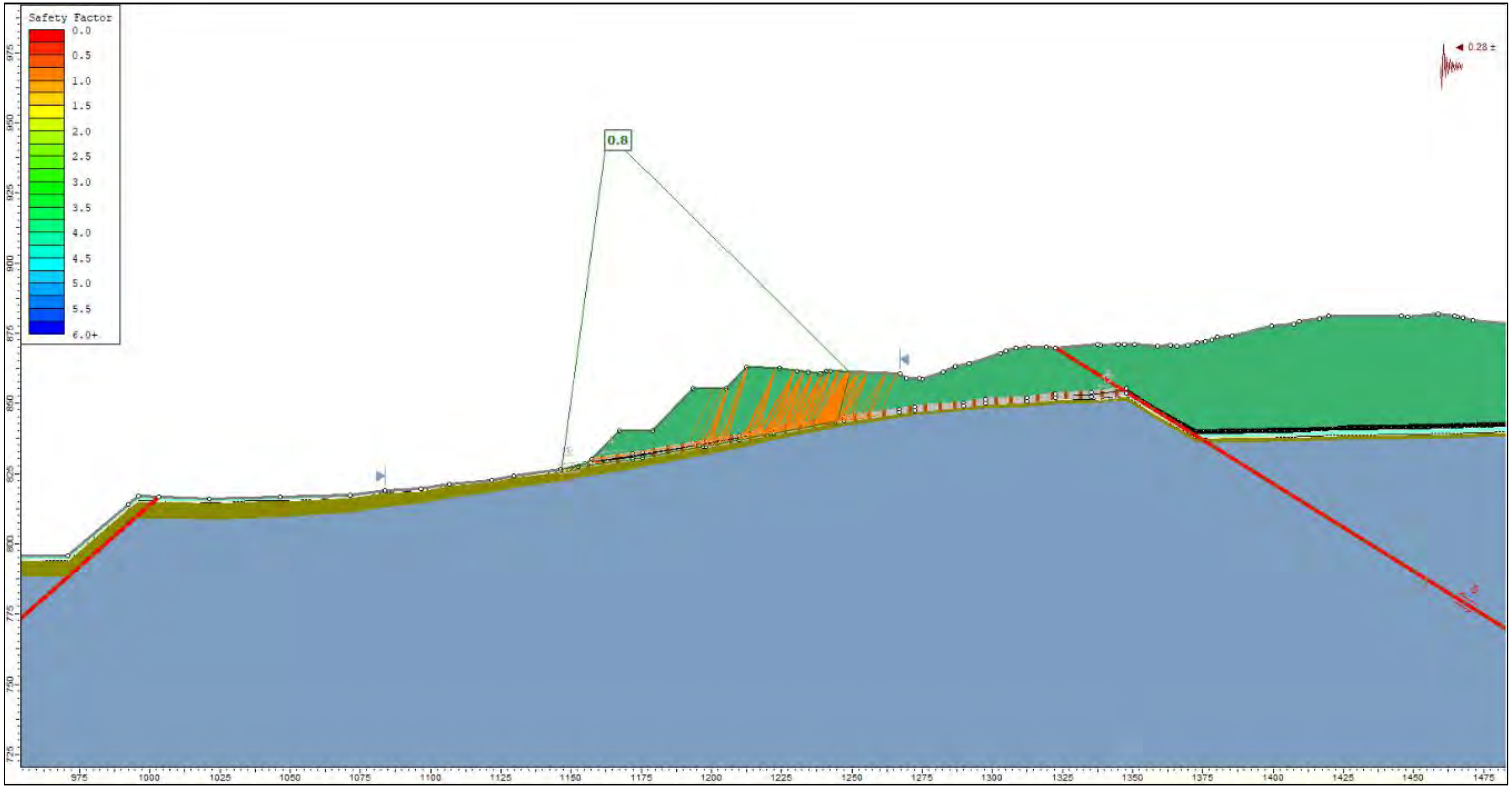

Figure: B141 – East Pit Highwall – Eastern Side of Pit - Section 106 – SLS Conditions.

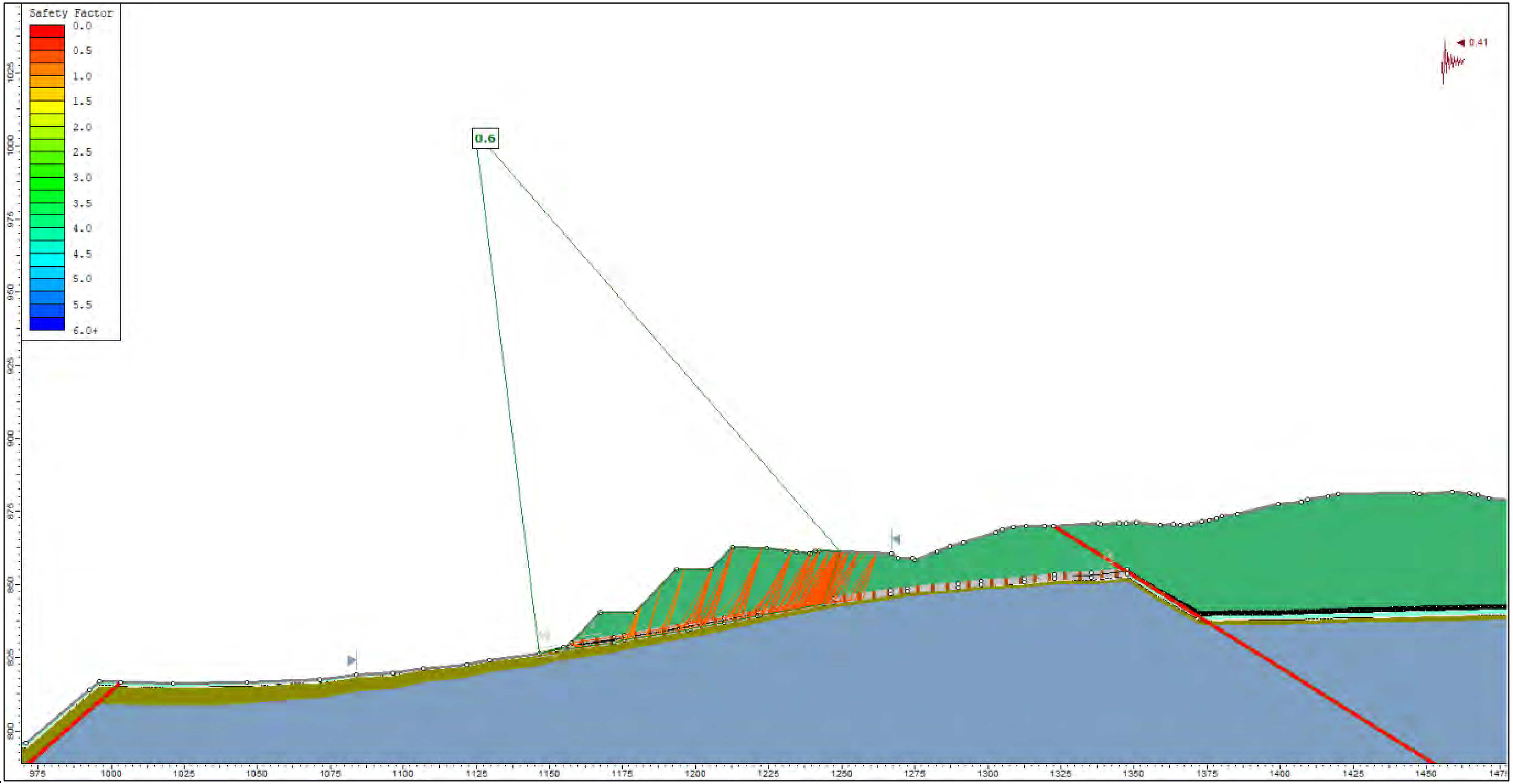

Figure: B142 – East Pit Highwall – Eastern Side of Pit - Section 106 – ULS Conditions.

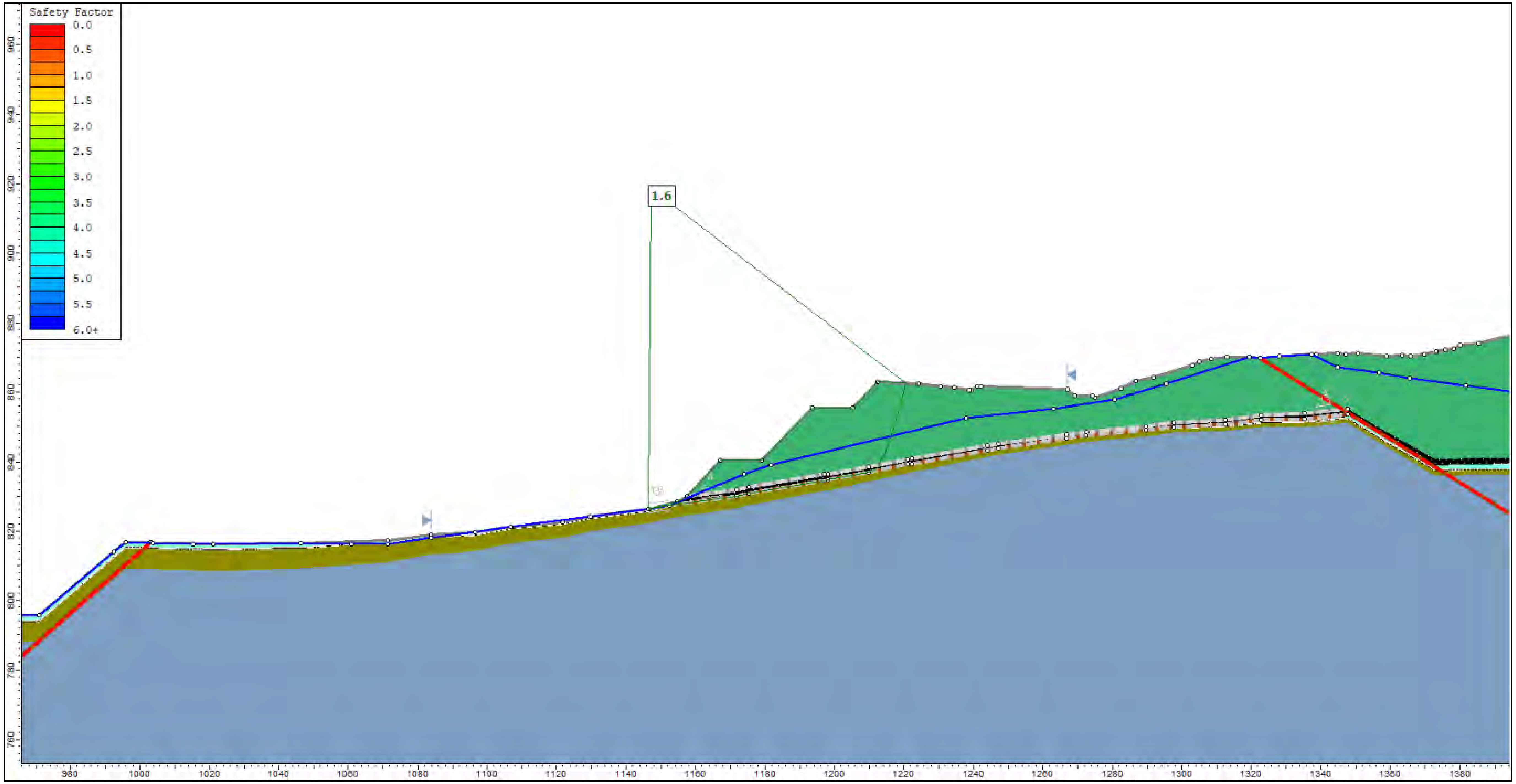

Figure: B143 – East Pit Highwall – Eastern Side of Pit - Section 106 – Elevated Groundwater Conditions.

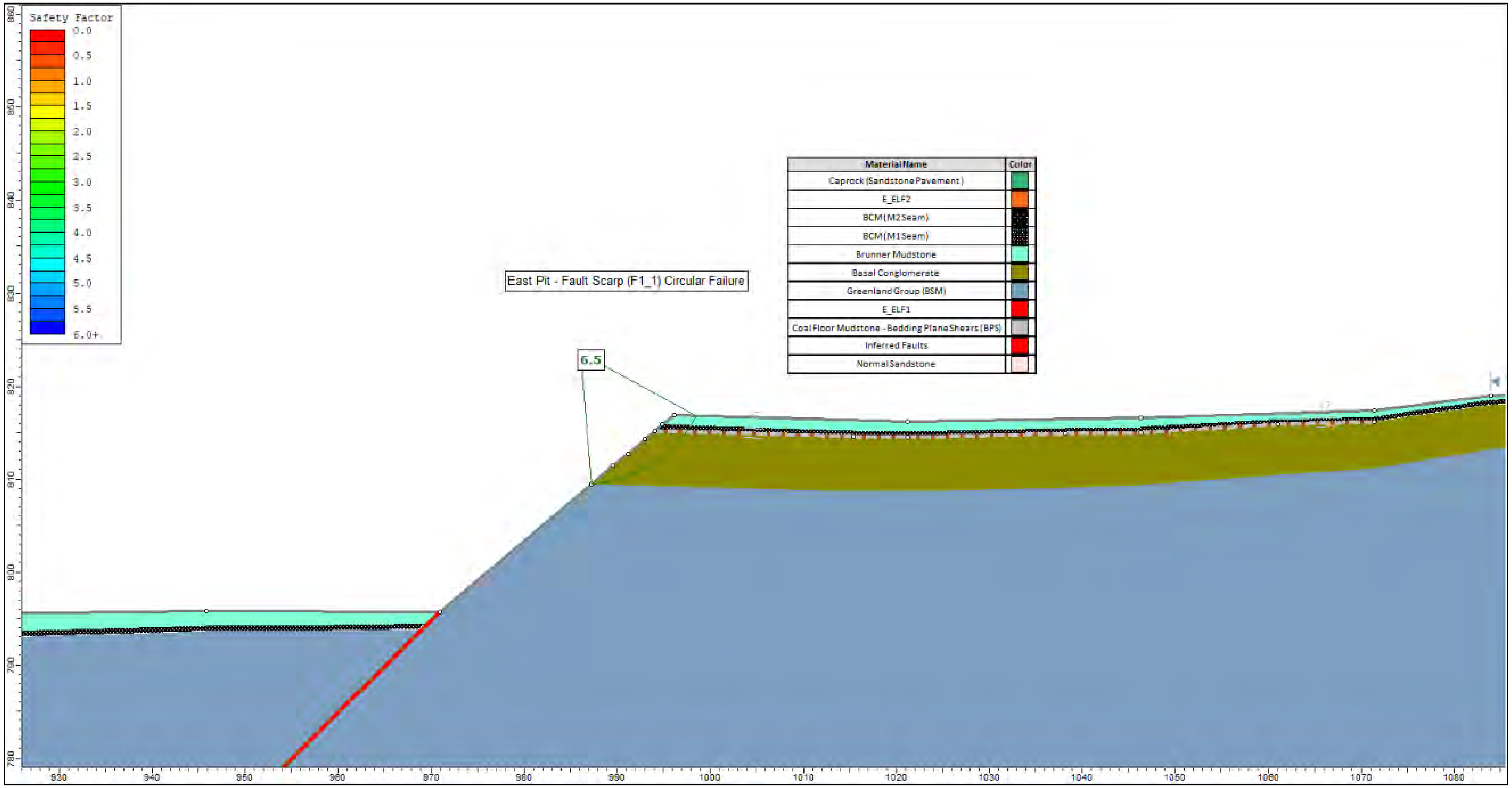

Figure: B144 – East Pit Fault Scarp (F1_1) Circular Failure –- Section 106 – Static Conditions.

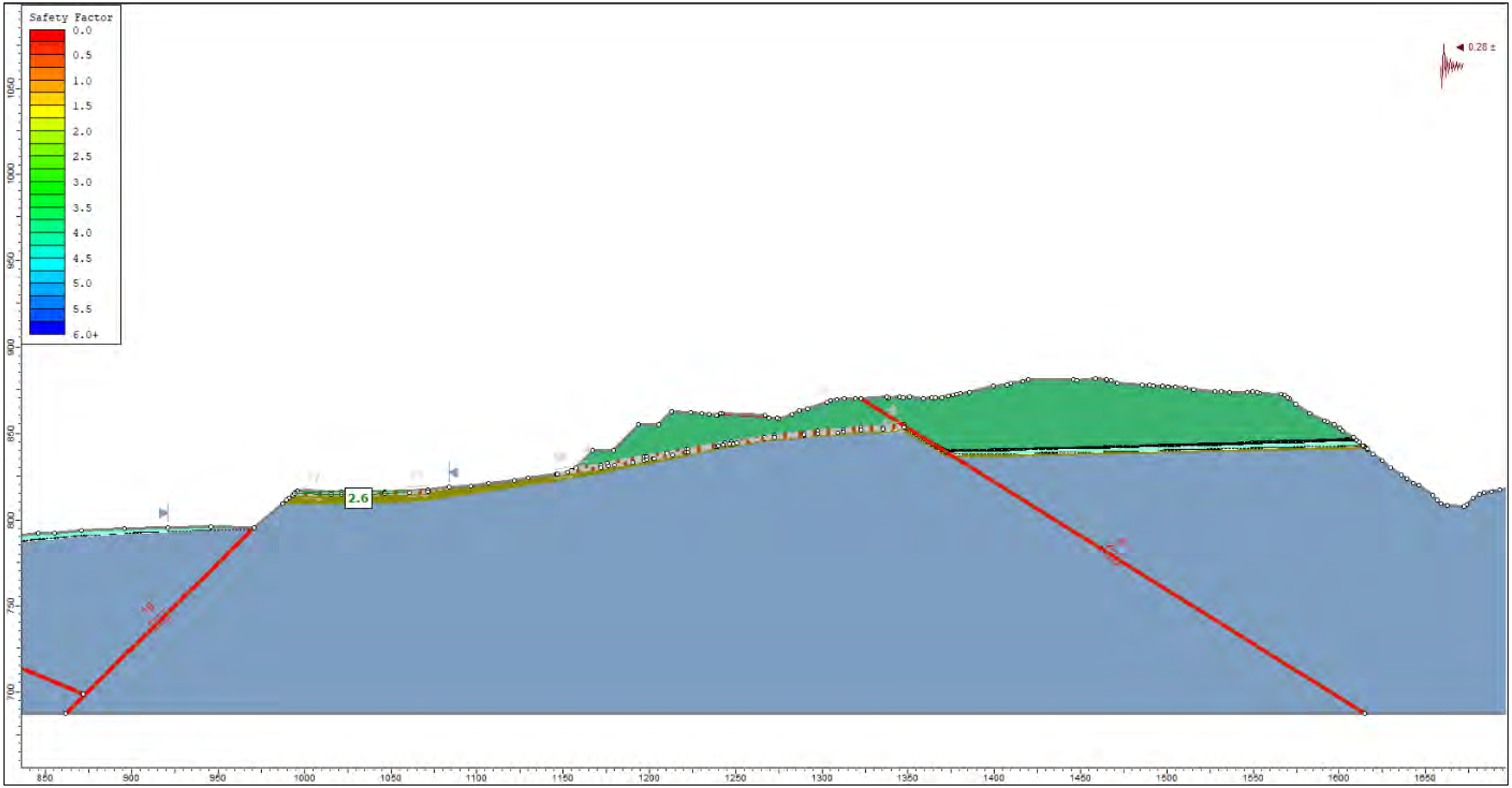

Figure: B145 – East Pit Fault Scarp (F1_1) Circular Failure –- Section 106 – SLS Conditions.

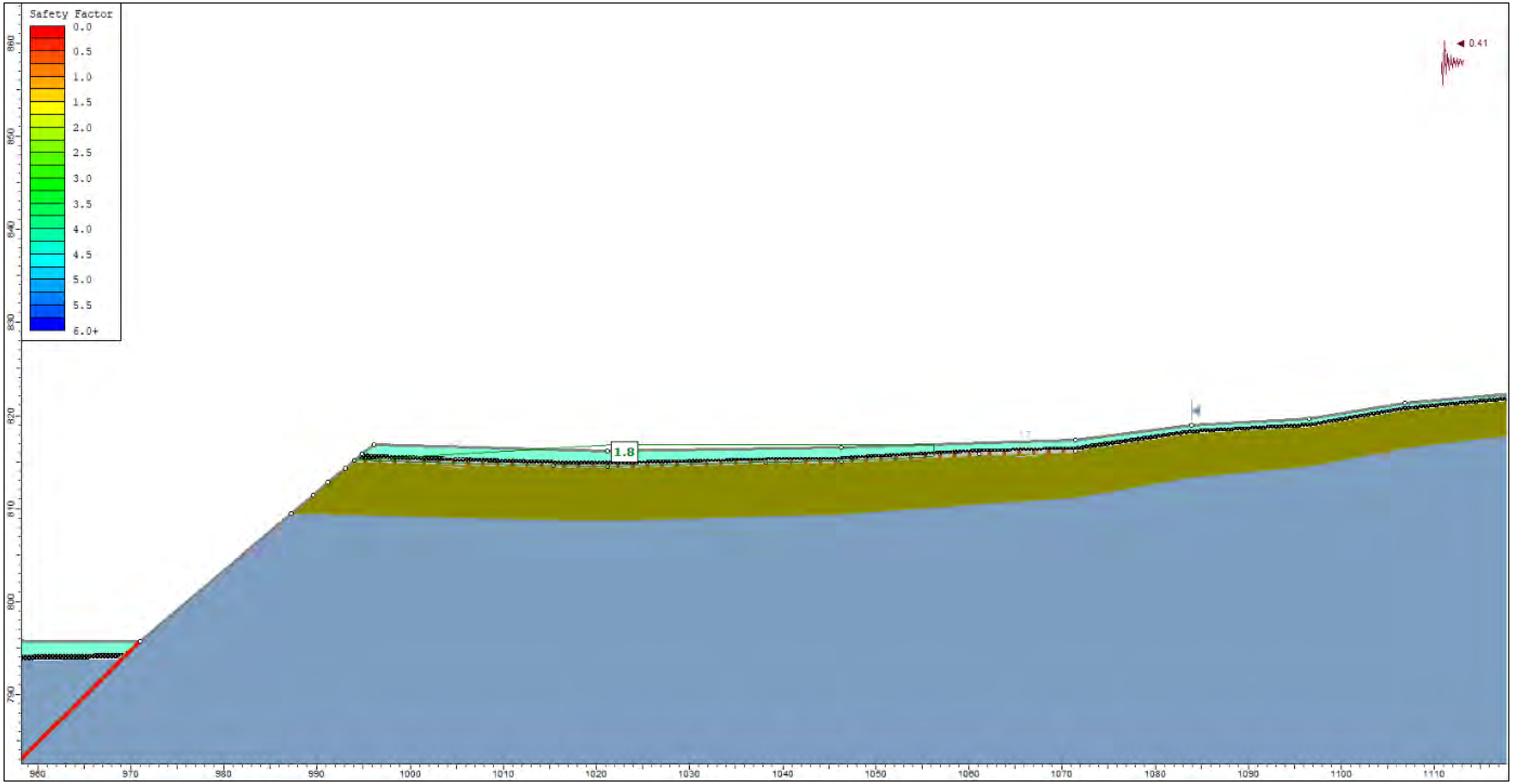

Figure: B146 – East Pit Fault Scarp (F1_1) Circular Failure –- Section 106 – ULS Conditions.

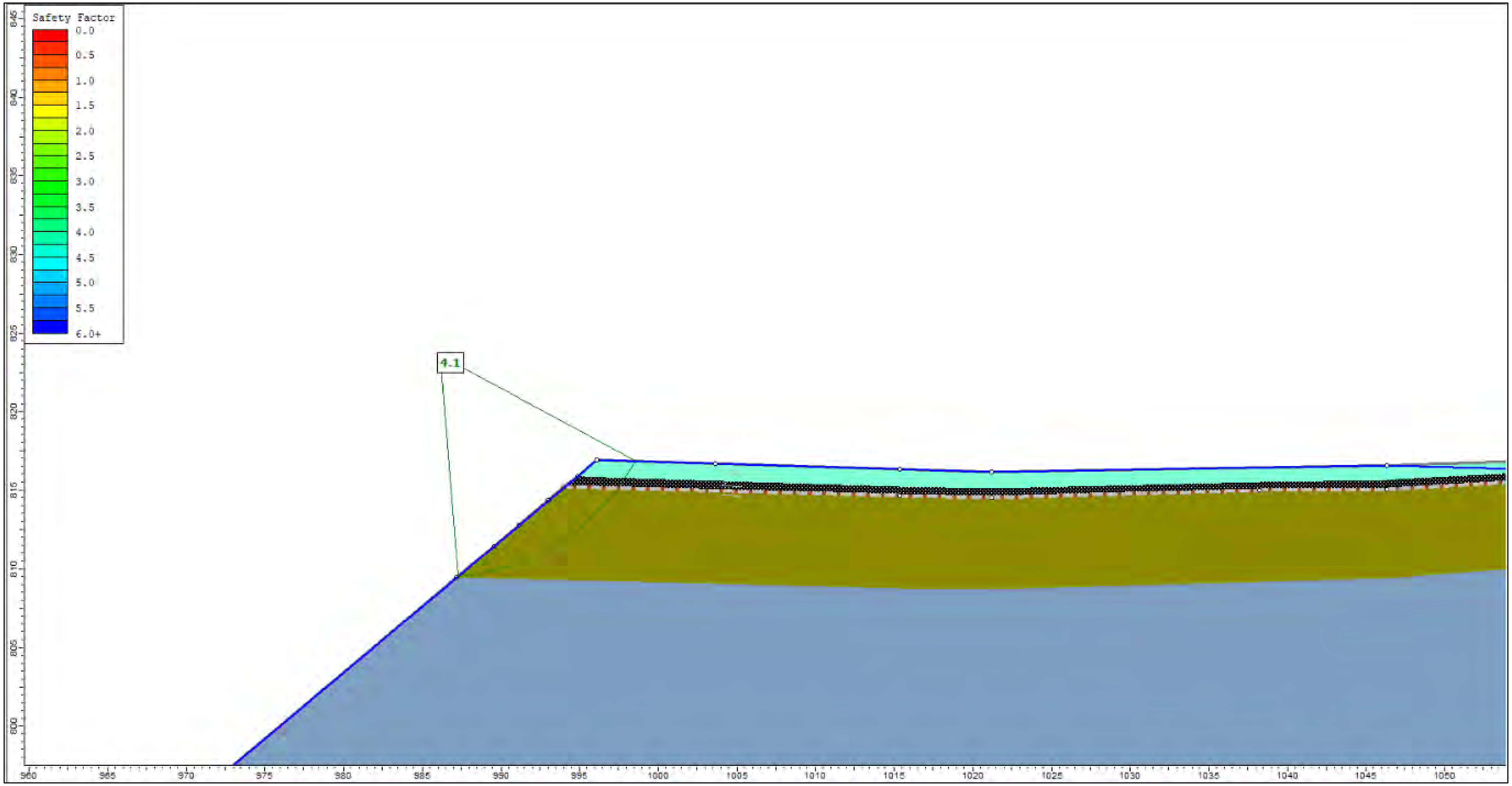

Figure: B147 – East Pit Fault Scarp (F1_1) Circular Failure –- Section 106 –Elevated Groundwater Conditions.

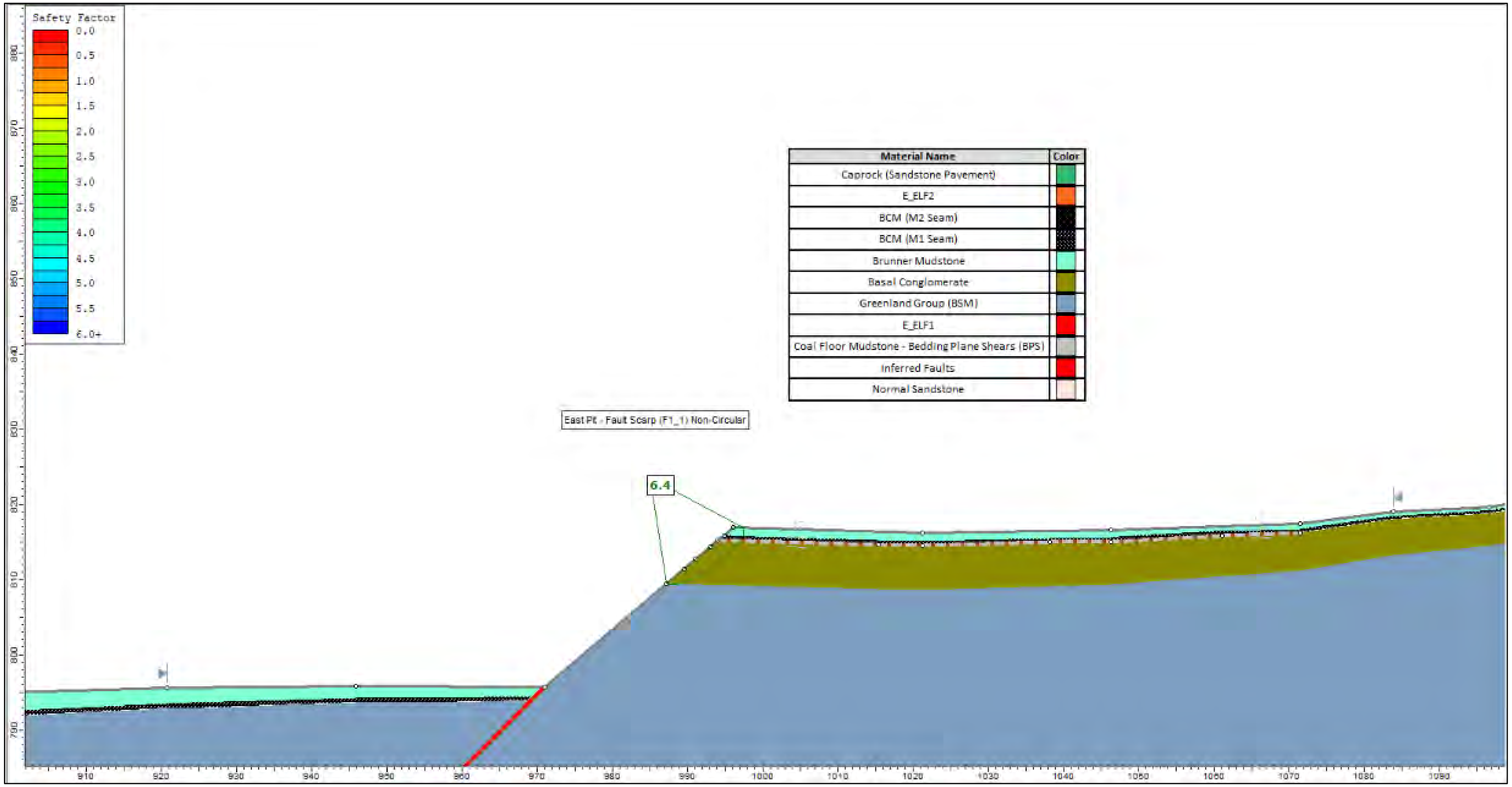

Figure: B148 – East Pit Fault Scarp (F1_1) Non-Circular Failure — Section 106 –Static Conditions.

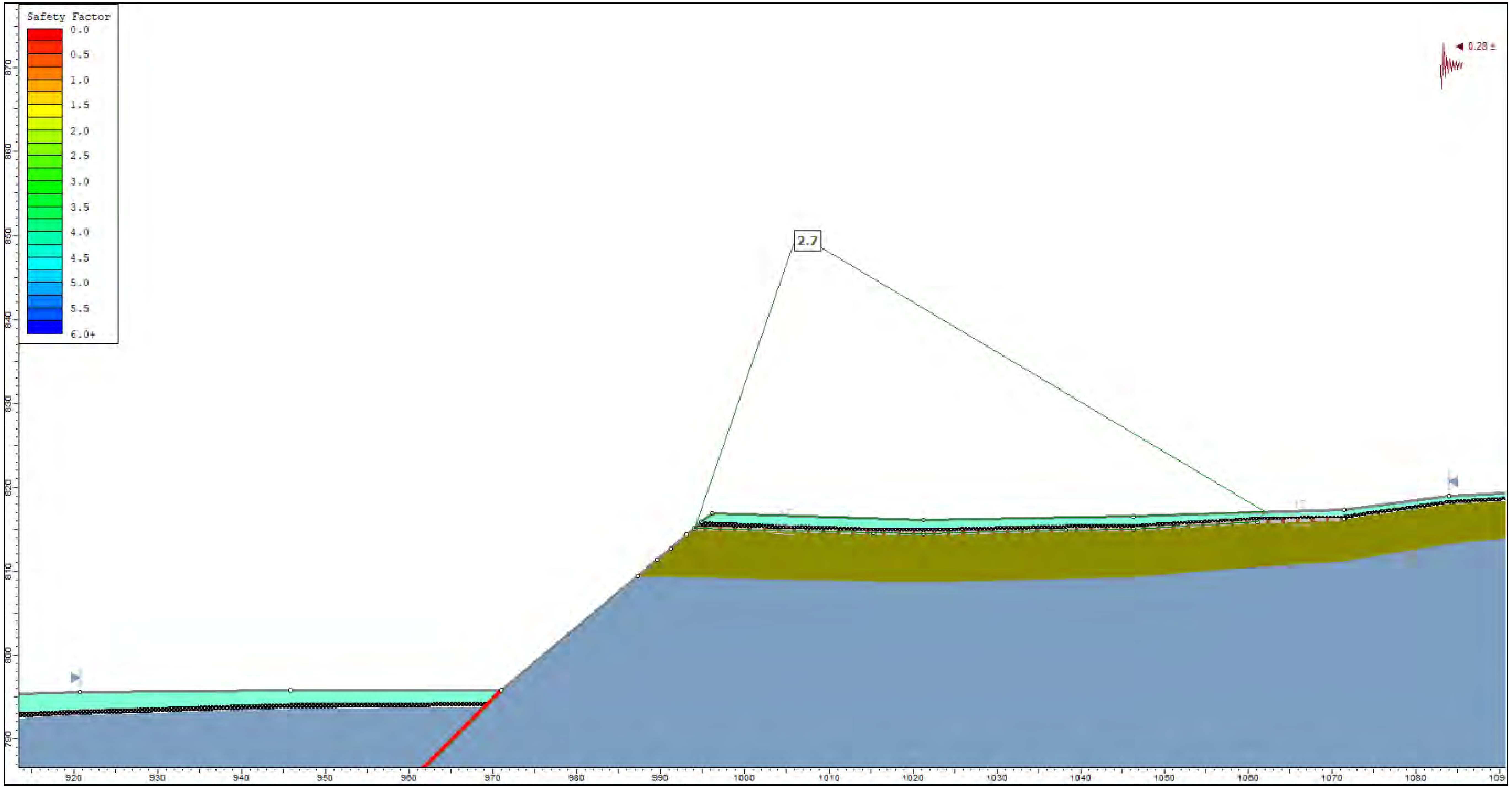

Figure: B149 – East Pit Fault Scarp (F1_1) Non-Circular Failure — Section 106 –SLS Conditions.

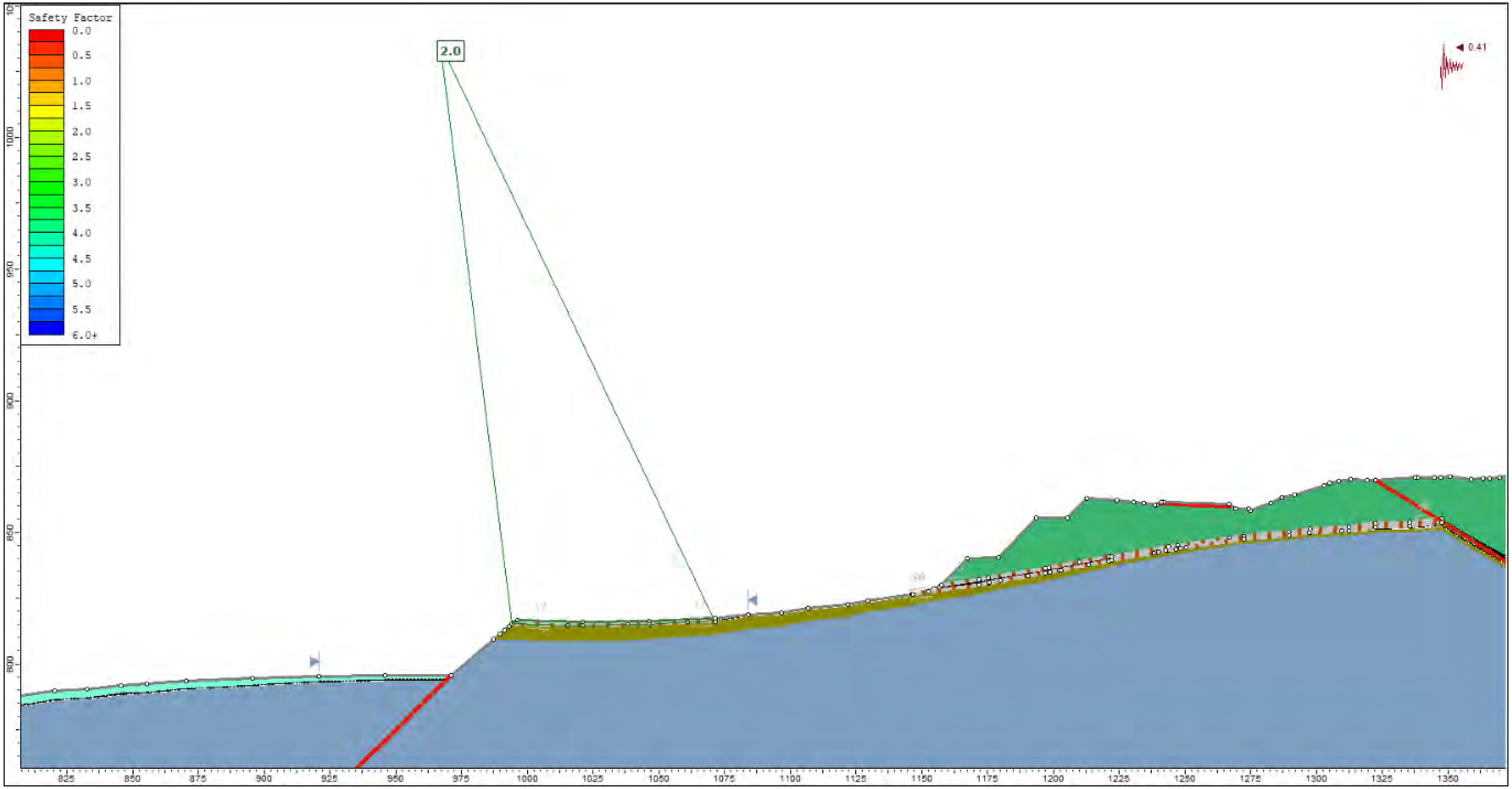

Figure: B150 – East Pit Fault Scarp (F1_1) Non-Circular Failure — Section 106 –ULS Conditions.

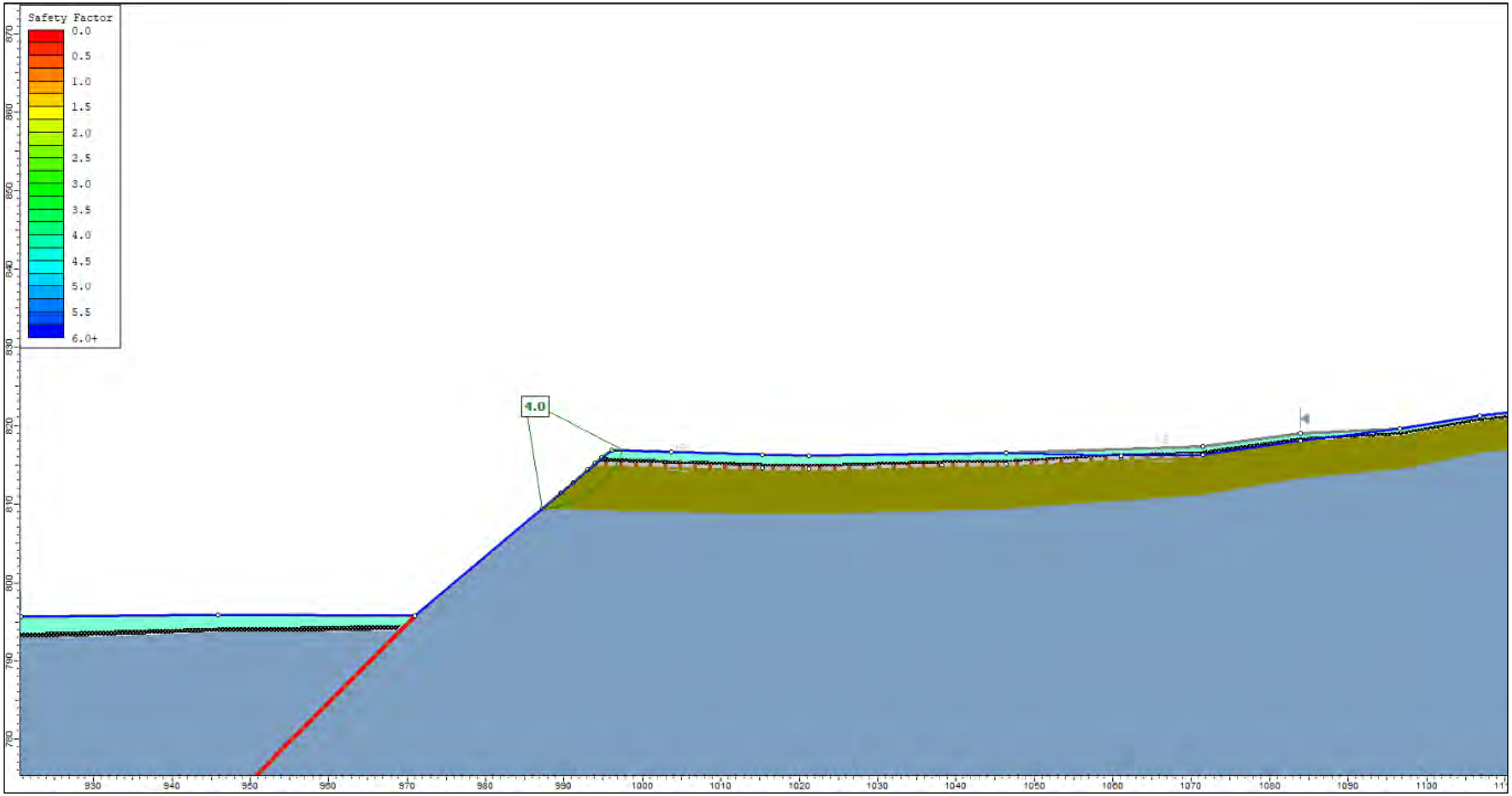

Figure: B151 – East Pit Fault Scarp (F1_1) - Non-Circular Failure –- Section 106 –Elevated Groundwater Conditions.

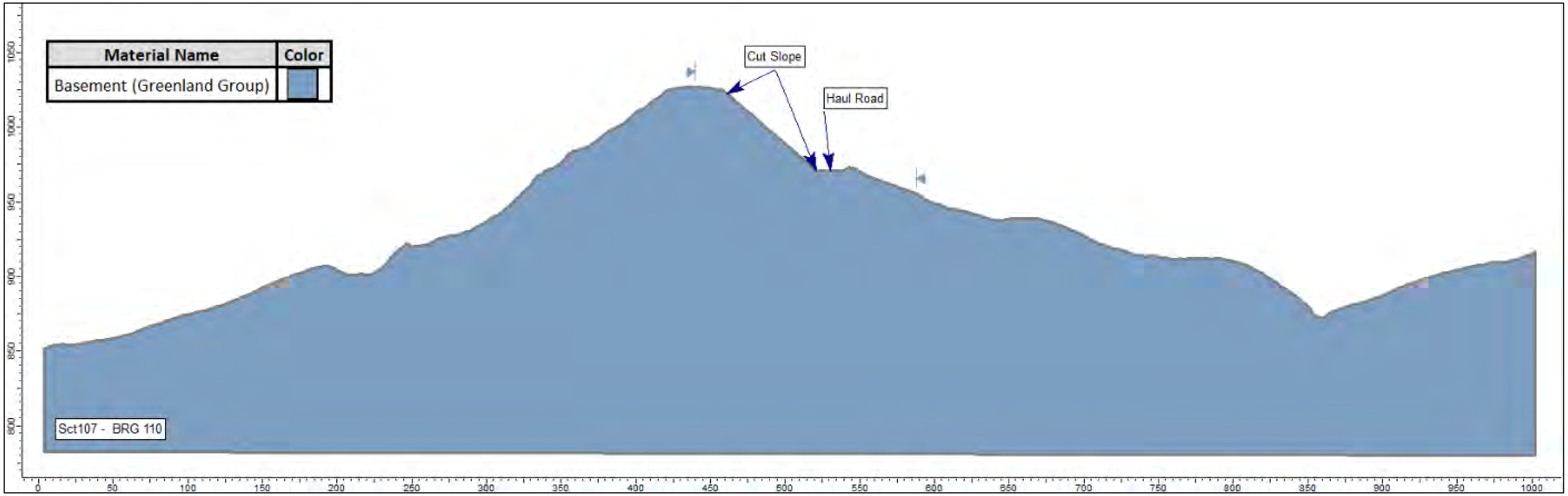

Figure: B152 - Haul Road Cut - Section 107 – Overview

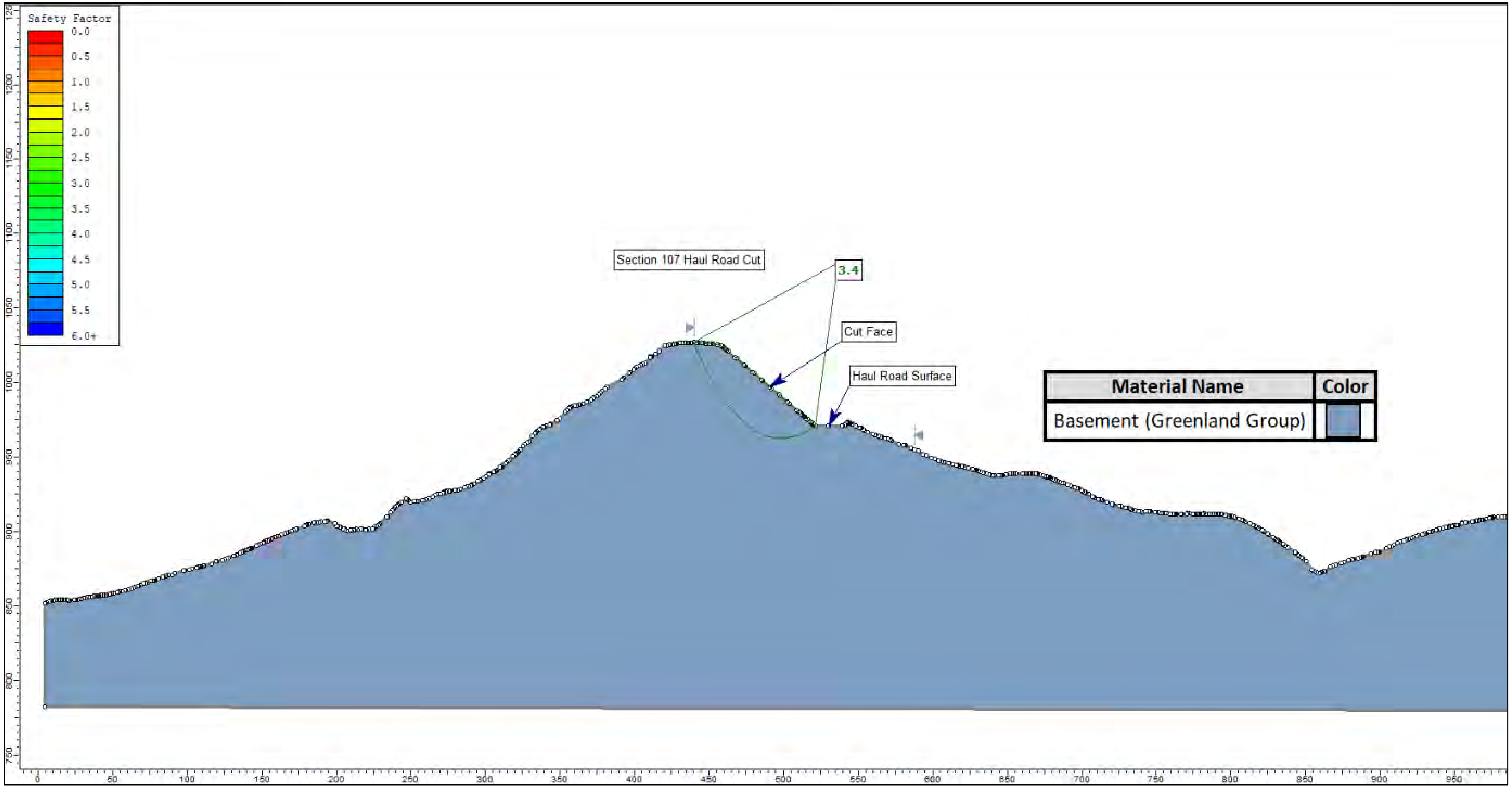

Figure: B153 - Haul Road Cut - Section 107 – Static Conditions.

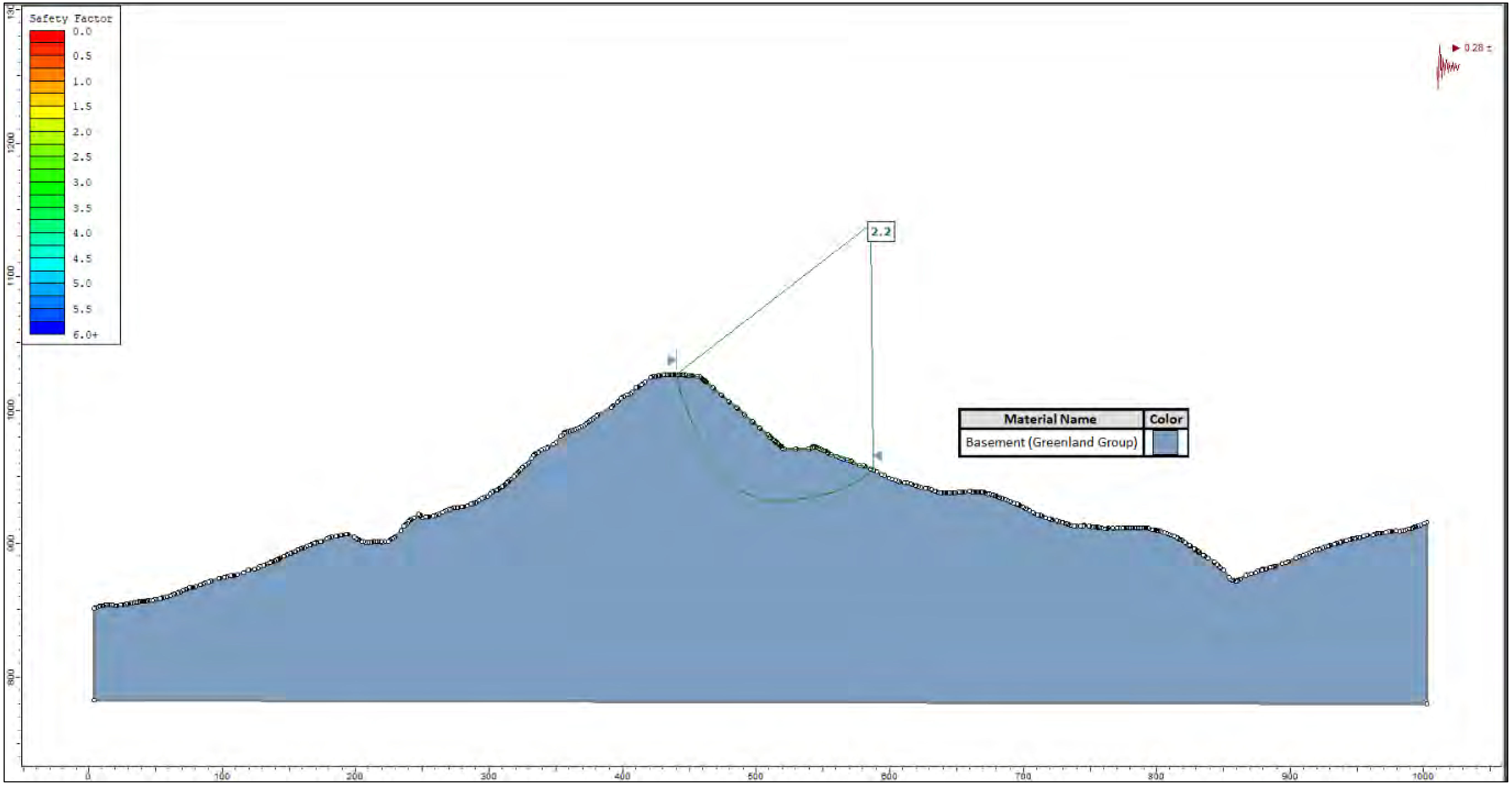

Figure: B154 - Haul Road Cut - Section 107 – SLS Conditions.

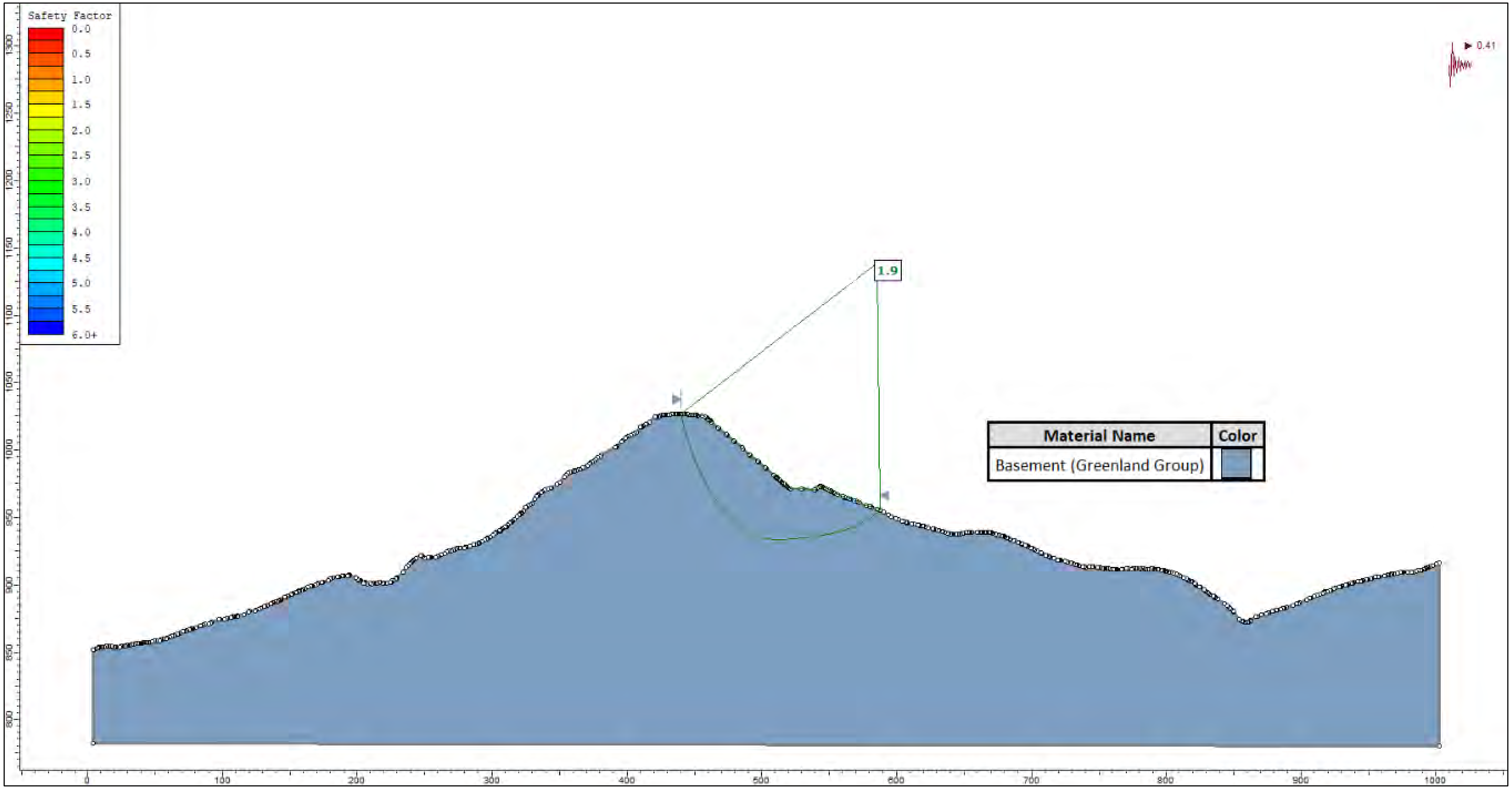

Figure: B155 - Haul Road Cut - Section 107 – ULS Conditions.

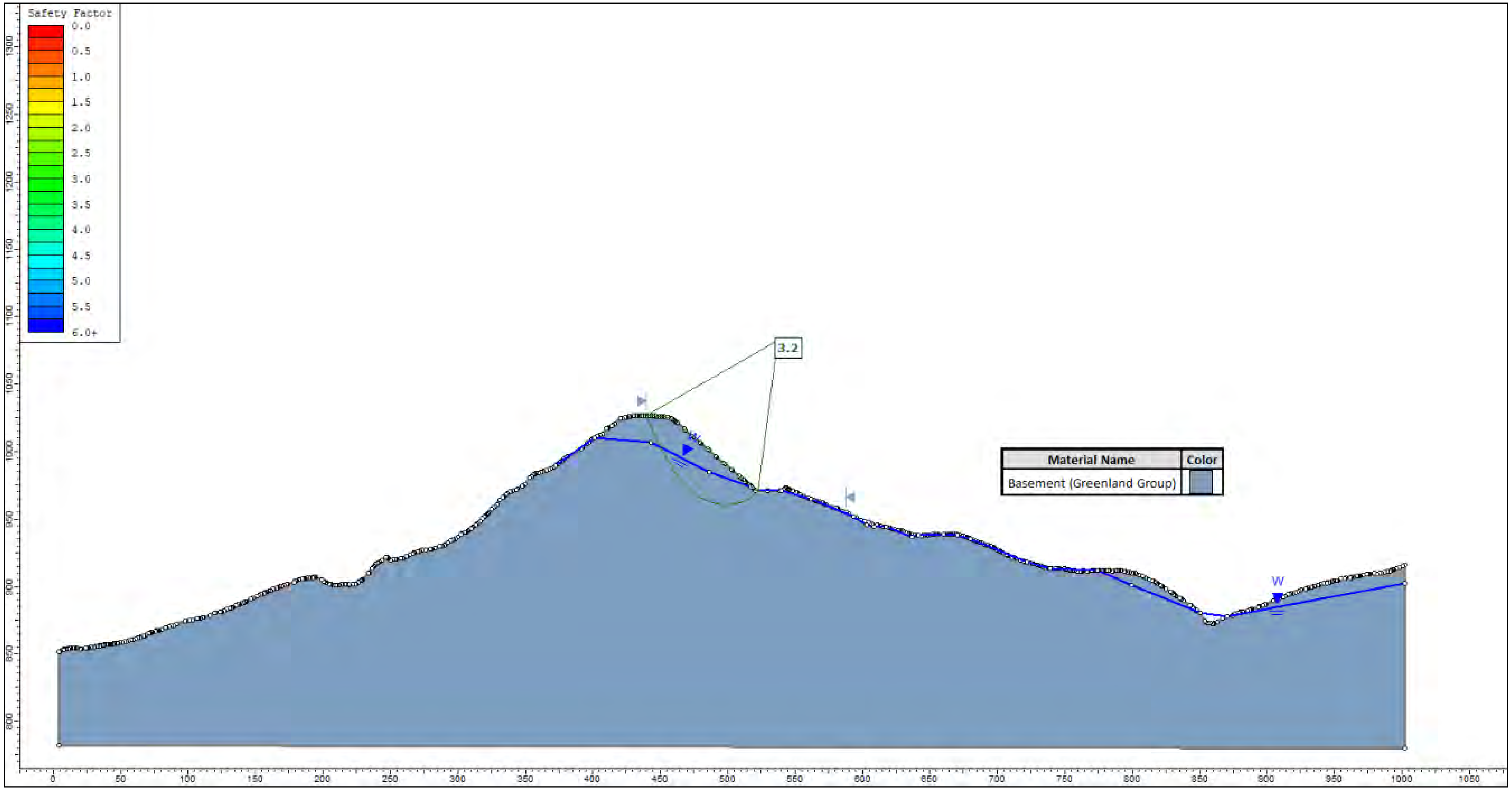

Figure: B156 - Haul Road Cut - Section 107 – Elevated Groundwater Conditions.

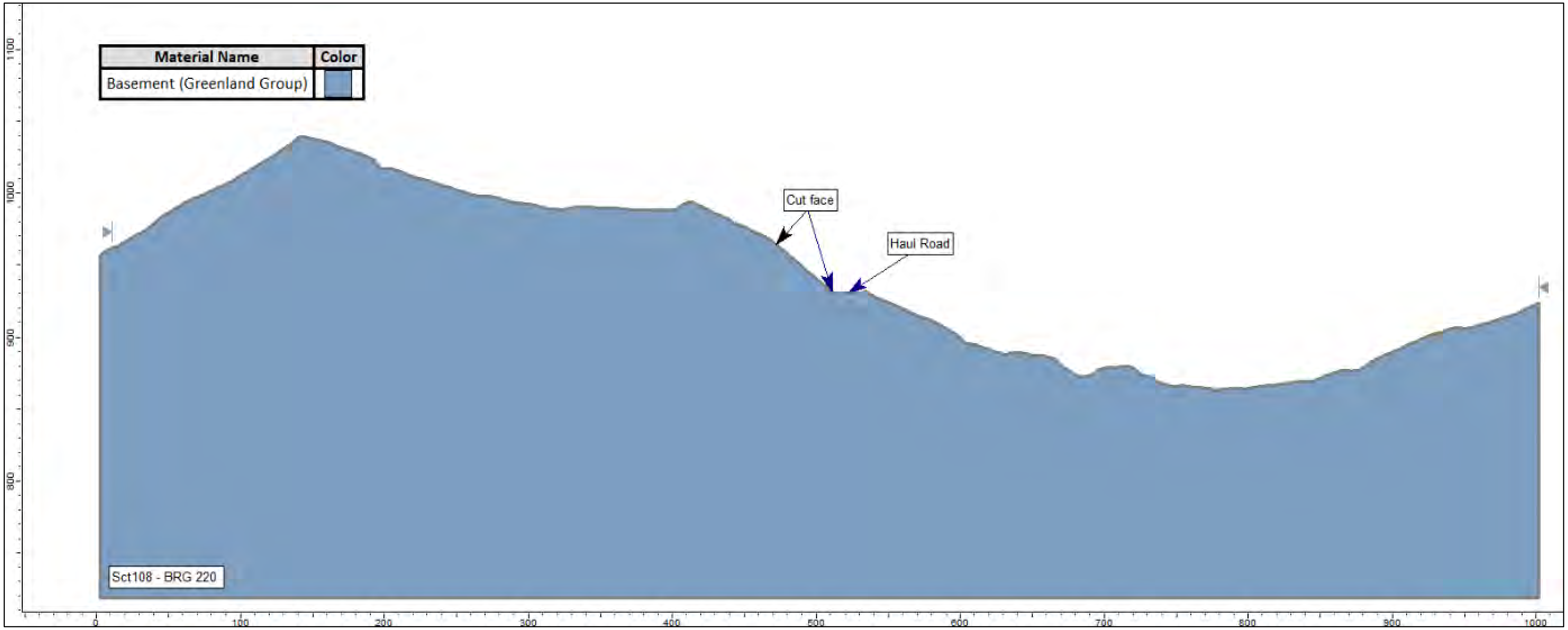

Figure: B157 - Section 108 – Haul Road Cut – Overview.

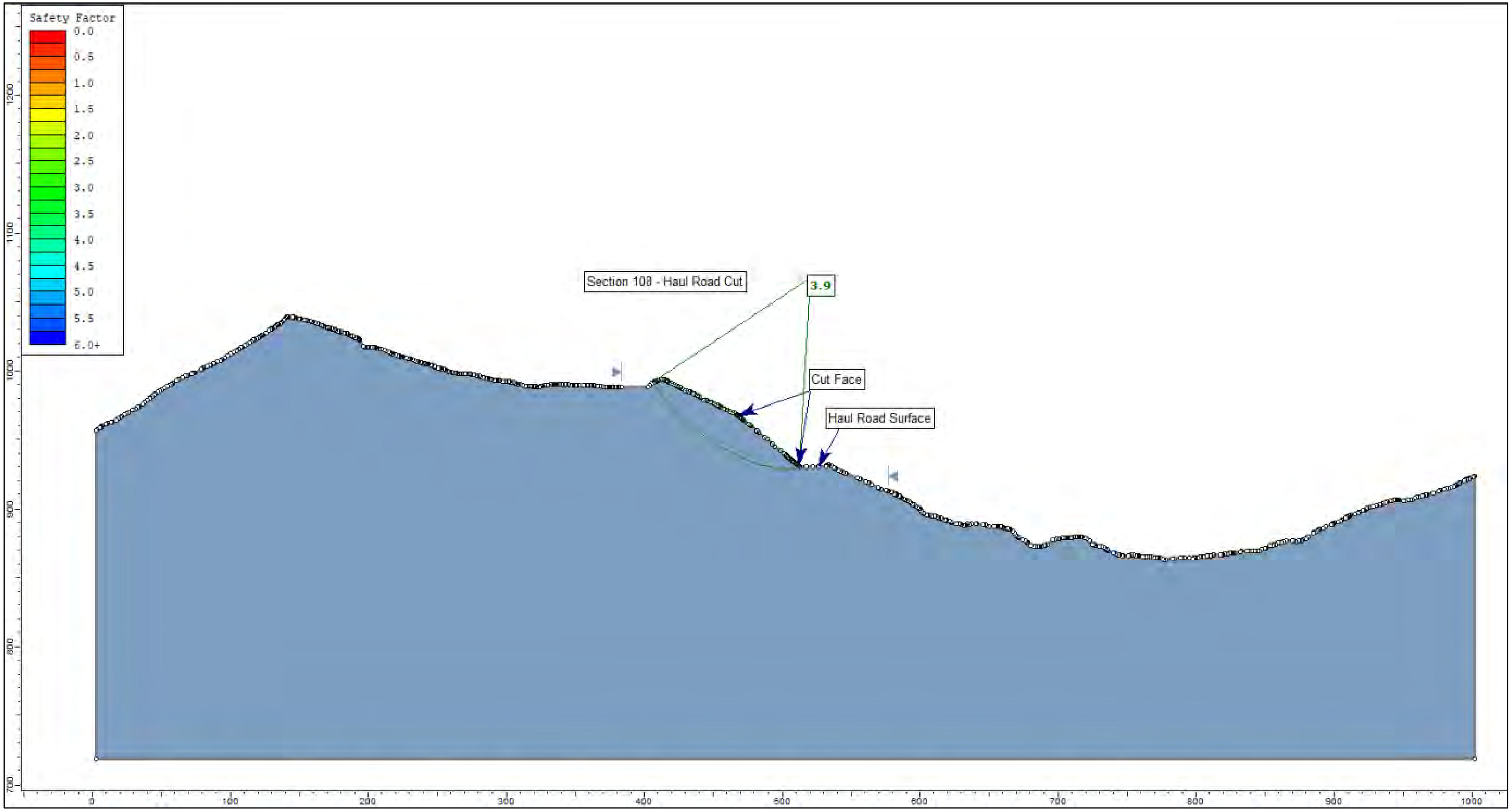

Figure: B158 - Haul Road Cut - Section 108 – Static Conditions.

Figure: B159 - Haul Road Cut - Section 108 – SLS Conditions.

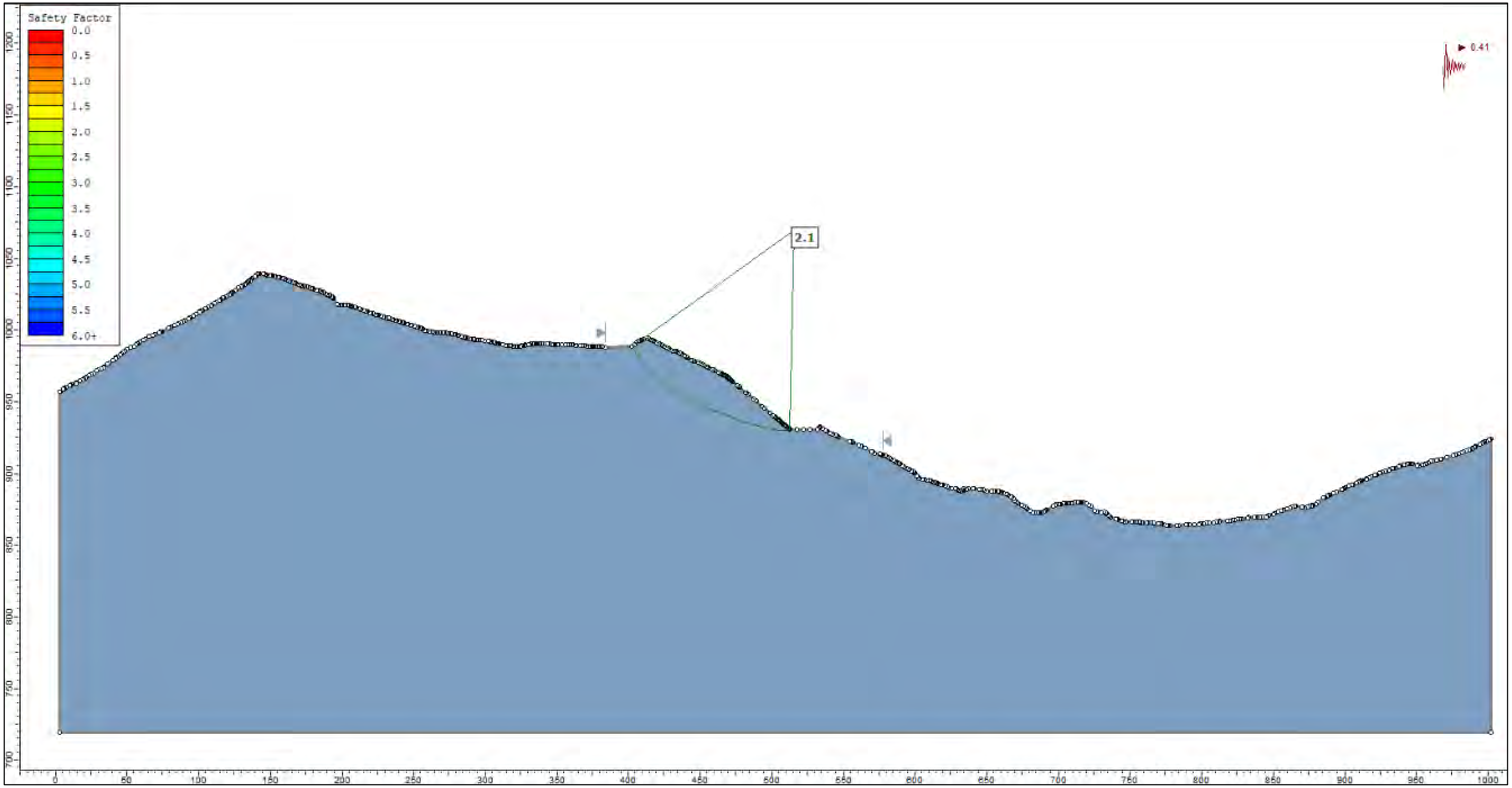

Figure: B160 - Haul Road Cut - Section 108 – ULS Conditions.

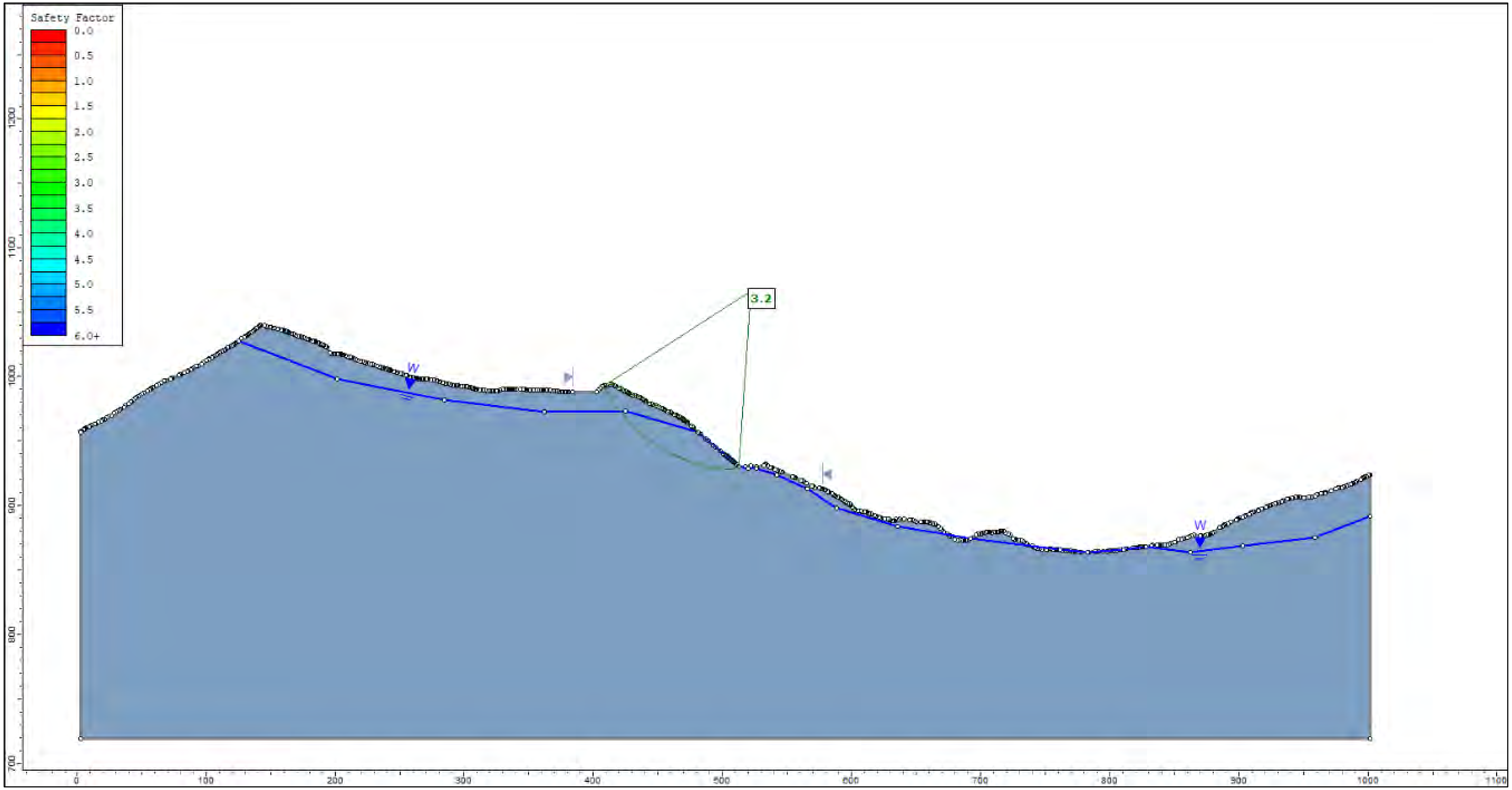

Figure: B161 - Haul Road Cut - Section 108 – Elevated Groundwater Conditions.

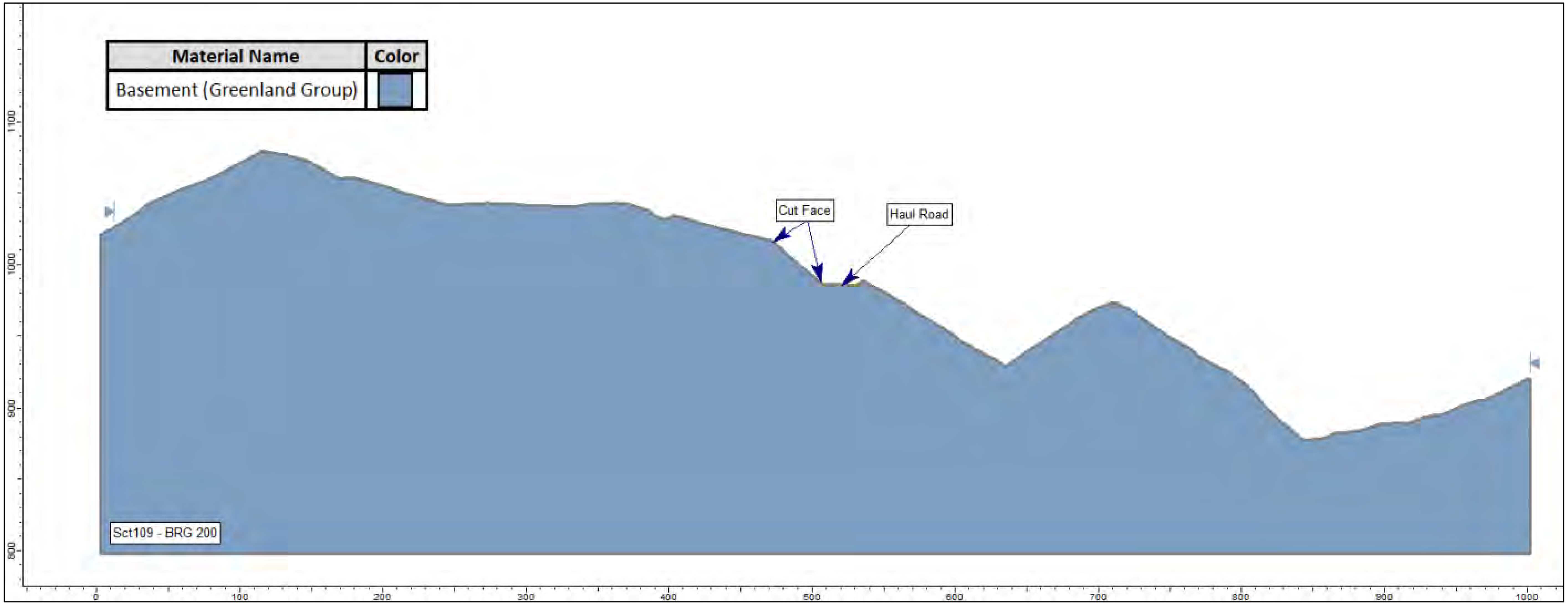

Figure: B162 - Section 109 – Haul Road Cut – Overview.

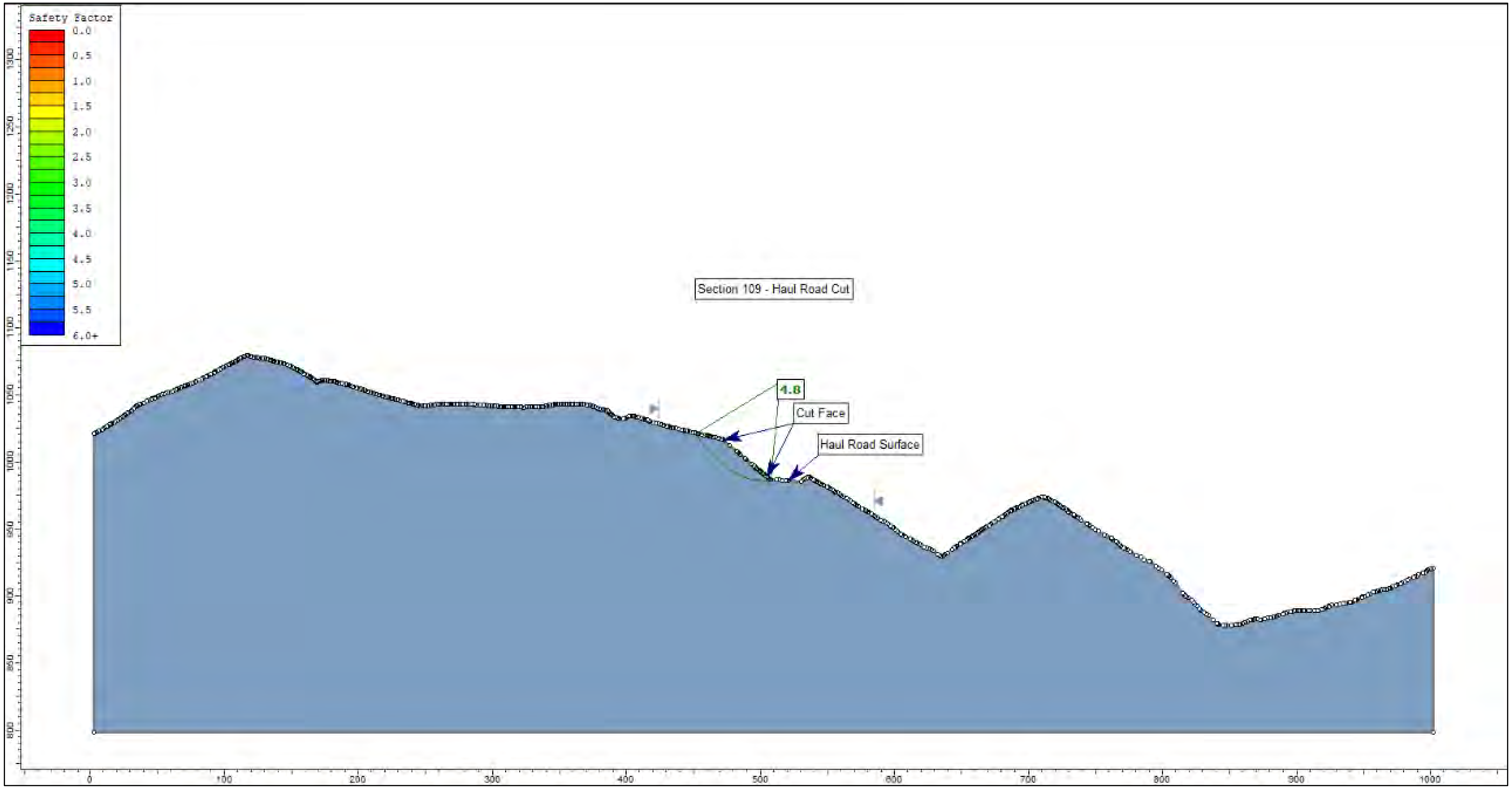

Figure: B163 - Haul Road Cut - Section 109 – SLS Conditions.

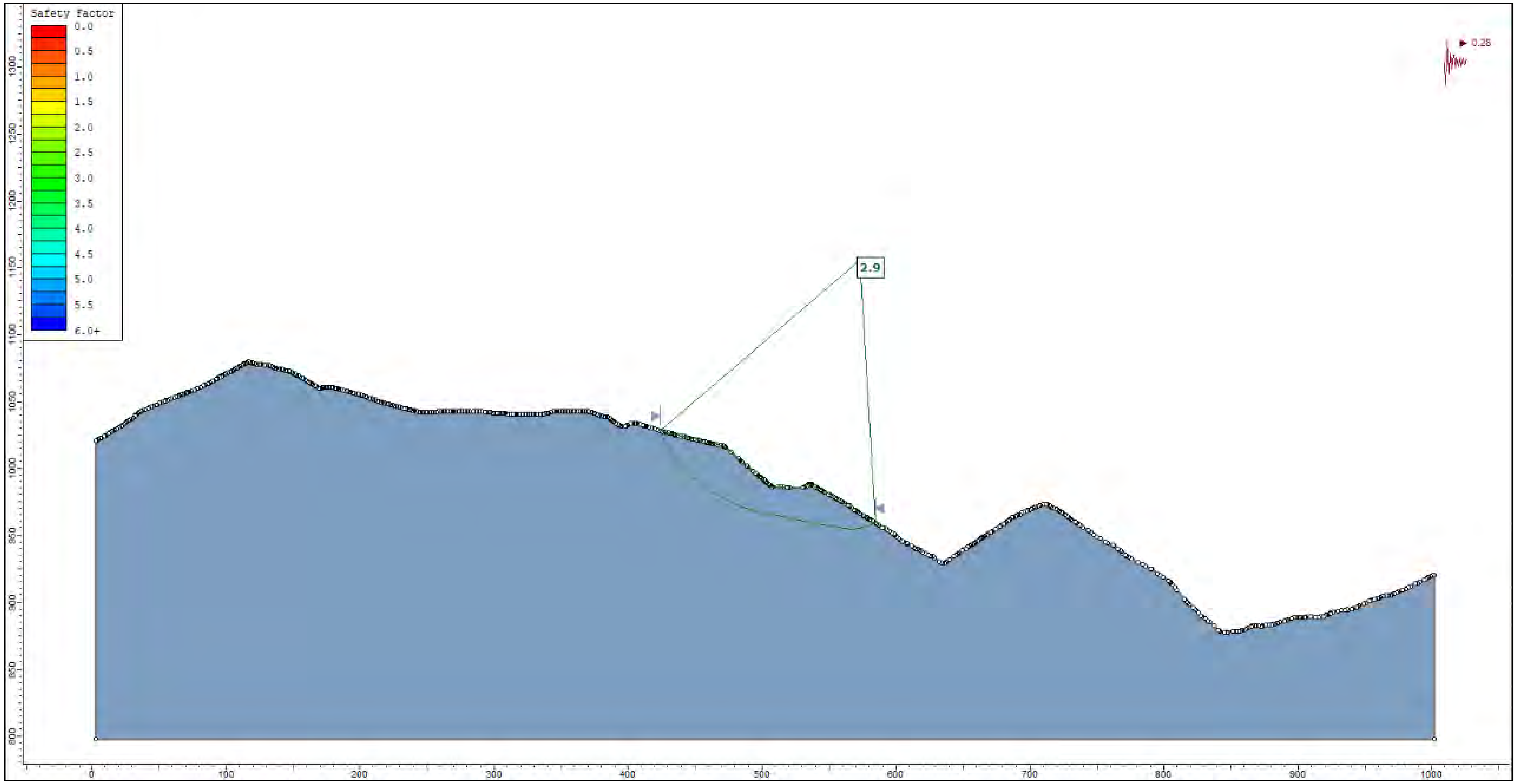

Figure: B164 - Haul Road Cut - Section 109 – SLS Conditions

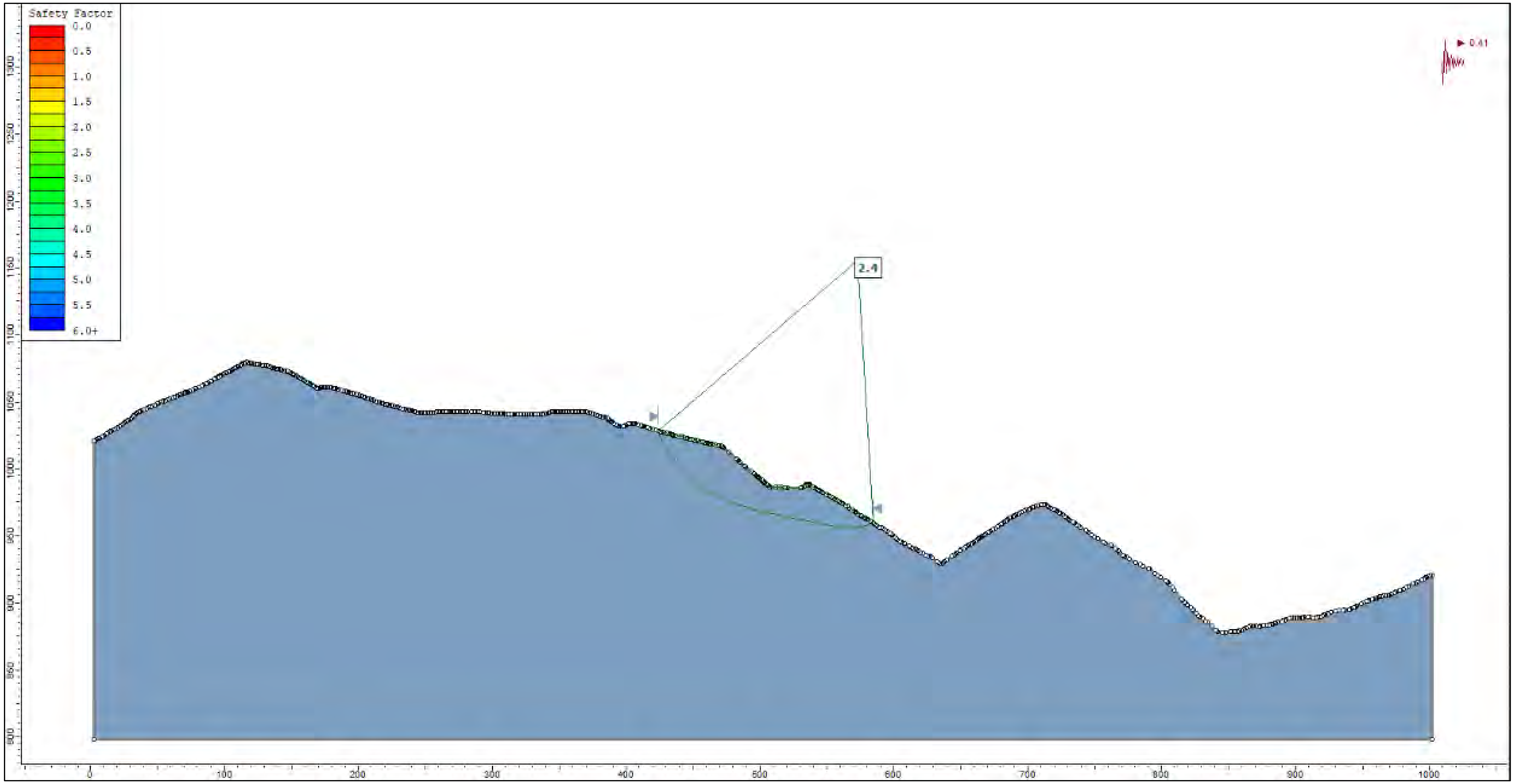

Figure: B165 - Haul Road Cut - Section 109 – ULS Conditions.

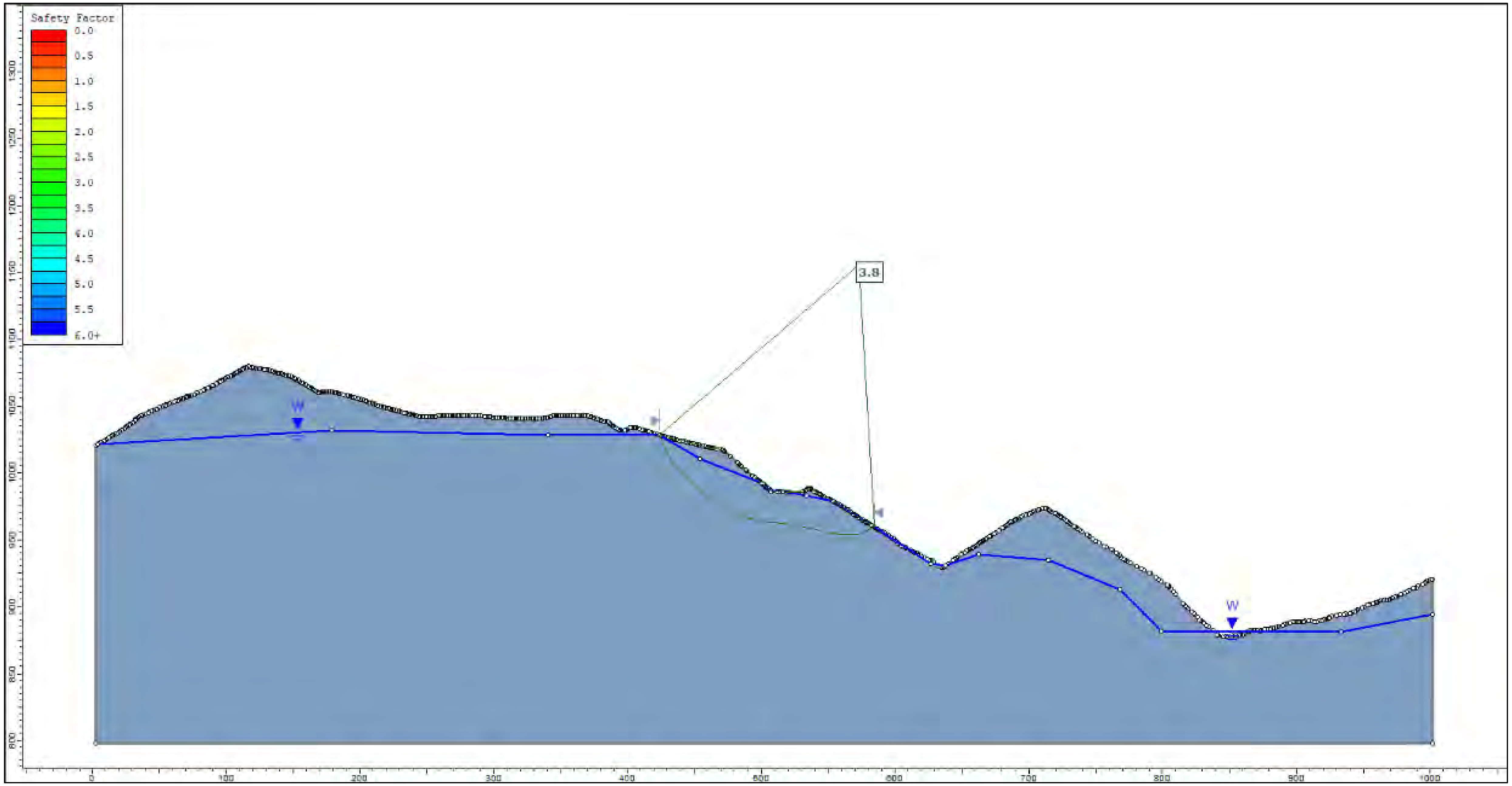

Figure: B166 - Haul Road Cut - Section 109 – Elevated Groundwater Conditions.

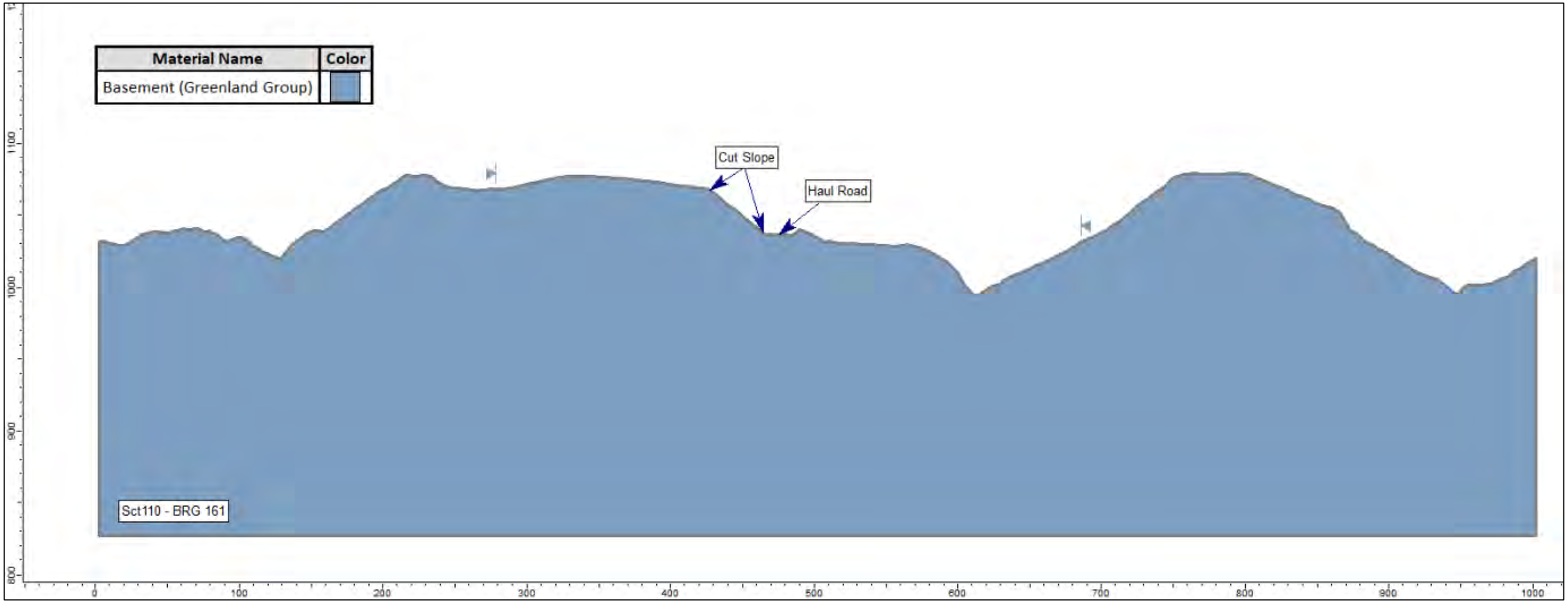

Figure: B167 - Section 110 – Haul Road Cut - Overview

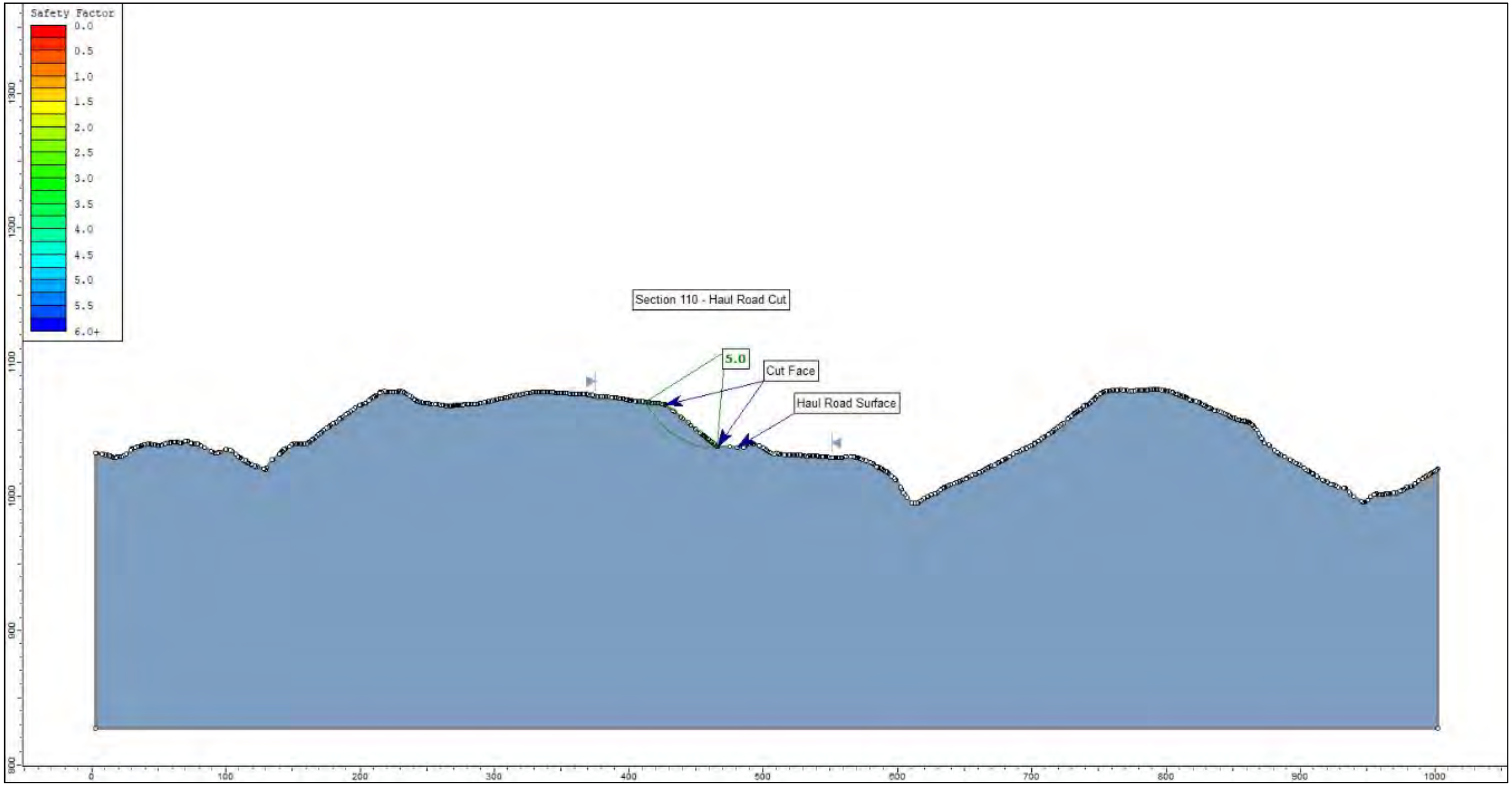

Figure: B168 - Haul Road Cut - Section 110 – Static Conditions.

Figure: B169 - Haul Road Cut - Section 110 – SLS Conditions.

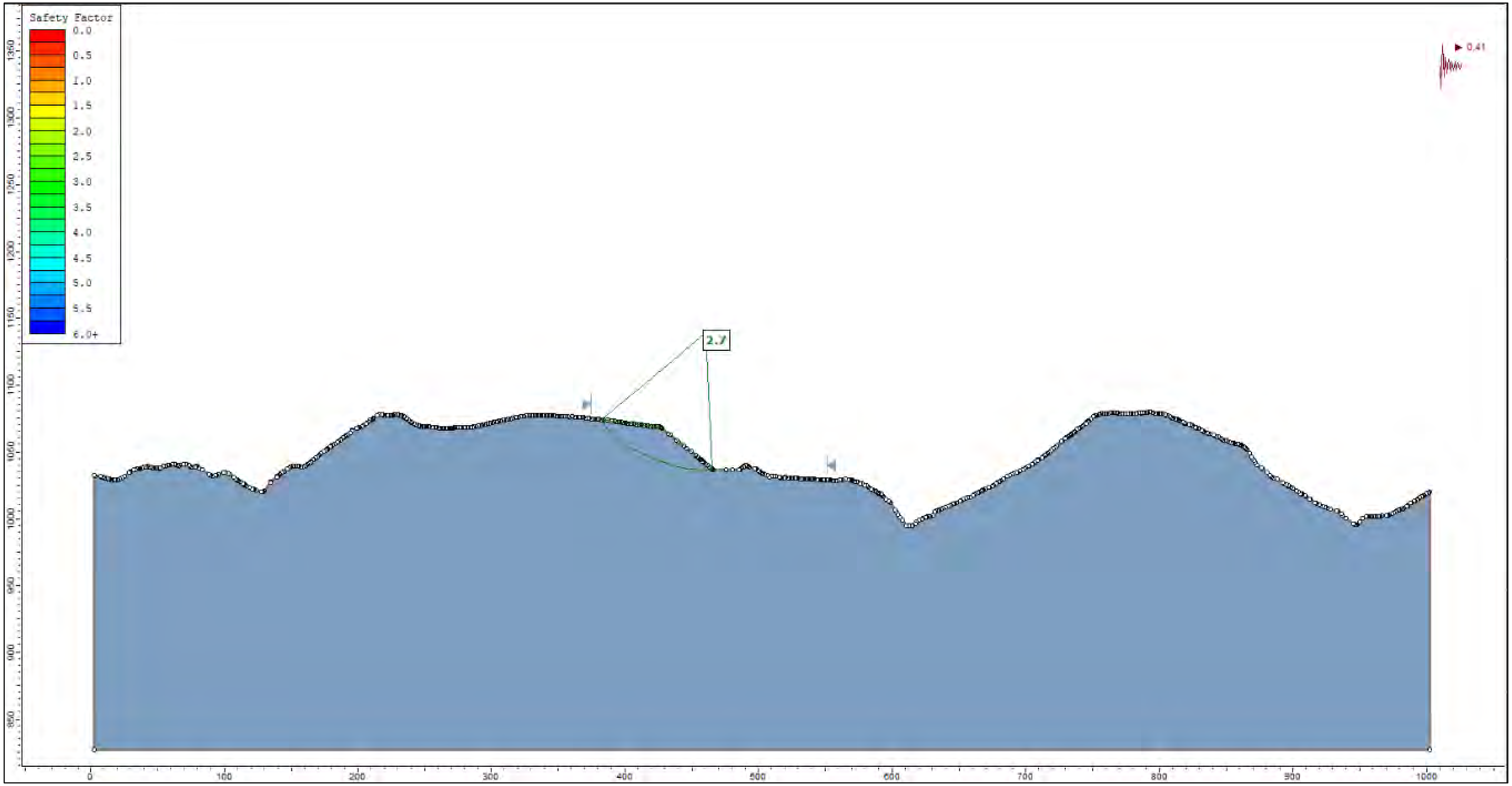

Figure: B170 - Haul Road Cut - Section 110 – ULS Conditions.

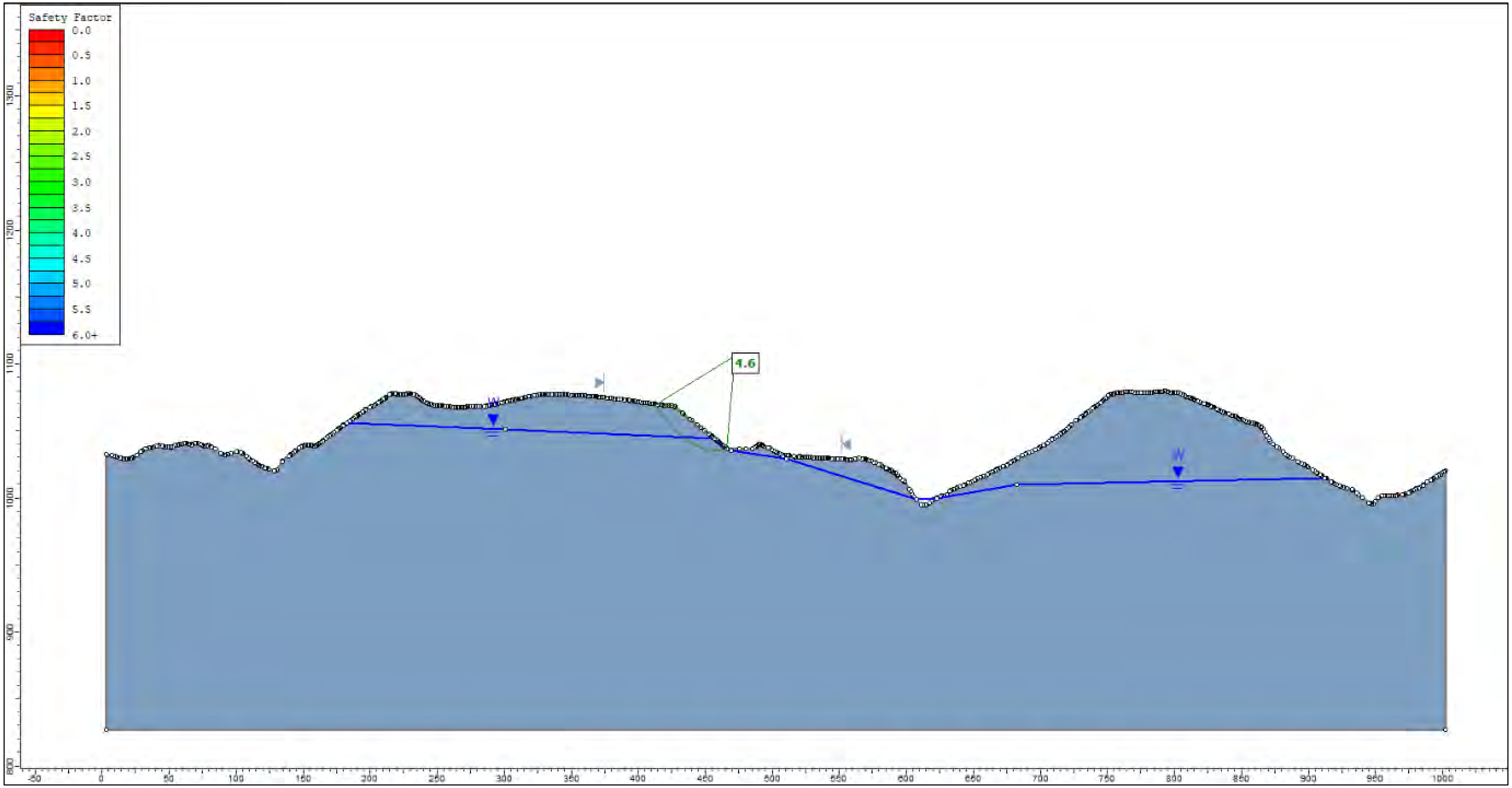

Figure: B171 - Haul Road Cut - Section 110 – Elevated Groundwater Conditions.