

APPENDIX E
QUARRY FACE STABILITY

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E1. INTRODUCTION

This Appendix details the assessment of the excavated quarry walls.

The assessment comprises:

1. Slope stability of quarry faces including the northern slope above the existing concrete drainage ditch,
2. Assessment of potential land use and remedial works,
3. Indicative costs (and implications) of remedial works.

E2. GEOLOGY AND PIT DEVELOPMENT

Much of the information relied upon in the study of the quarry faces has been reported in Appendix A to this report, and as such is not repeated here.

However, some points are worth noting, namely:

1. The southern quarry face appears to have experienced at least one major slope failure. This failure was located in the south eastern part of the quarry based on inspection of aerial photographs, the geometry of the quarry and features noted during our inspections.

While we do not have the benefit of records kept by the quarry operators, it appears a moderately large section of the wall failed by sliding of a mass of slightly weathered (SW) and fresh (FR) breccia rock. Sliding occurred on, or in, a steeply dipping unit of weaker, muddy breccia rock, possibly with a small toe failure through the SW/FR breccia rock. At this time, we do not know what role water pressures played in the failure.

Plate 5 in the main report shows a view of the south face with the sliding plane visible at the left side.

2. A number of failures of bench to inter-bench scale can be seen in the northern quarry face today, and in past aerial photographs.

Plates 1 and 2 in the main report clearly show these failures.

These failures occurred as block sliding on the following features:

North Eastern Part of the Quarry

A combination of steeply dipping bedding planes and contacts/joints in the breccia rock.

Central Part of Northern Batter

On either shallow to moderately dipping seams/contacts in the breccia rock, or a combination of these seams/contacts and steeply dipping contact surfaces in units of weaker, muddy breccia rock.

North Western Part of the Quarry

Either on moderately dipping seams in the breccia rock, or on a combination of the seams in the breccia and steeply dipping contacts within units of muddy breccia rock.

3. No evidence of failures in the eastern quarry face has been observed in the field or from aerial photographs.
4. The south western part of the quarry presents as a highly blocky face. While there are no signs of moderate to large scale inter-bench scale features, gently to moderately dipping features in the breccia and muddy breccia units are likely to influence bench scale stability.

E3. INVESTIGATIONS

Investigation of the rock forming the quarry has been discussed in detail in Appendix A.

E4. INTERPRETED GEOTECHNICAL MODEL

The interpreted geotechnical model of the quarry is discussed in Appendix A. The interpreted models are also shown in a series of cross sections located as shown on Drawing PSM1059-4.

The section locations are also shown below in Figure E1. Figure E1 also includes the structural domains identified at the quarry. These domains define areas of stability assessments presented in Section E6.

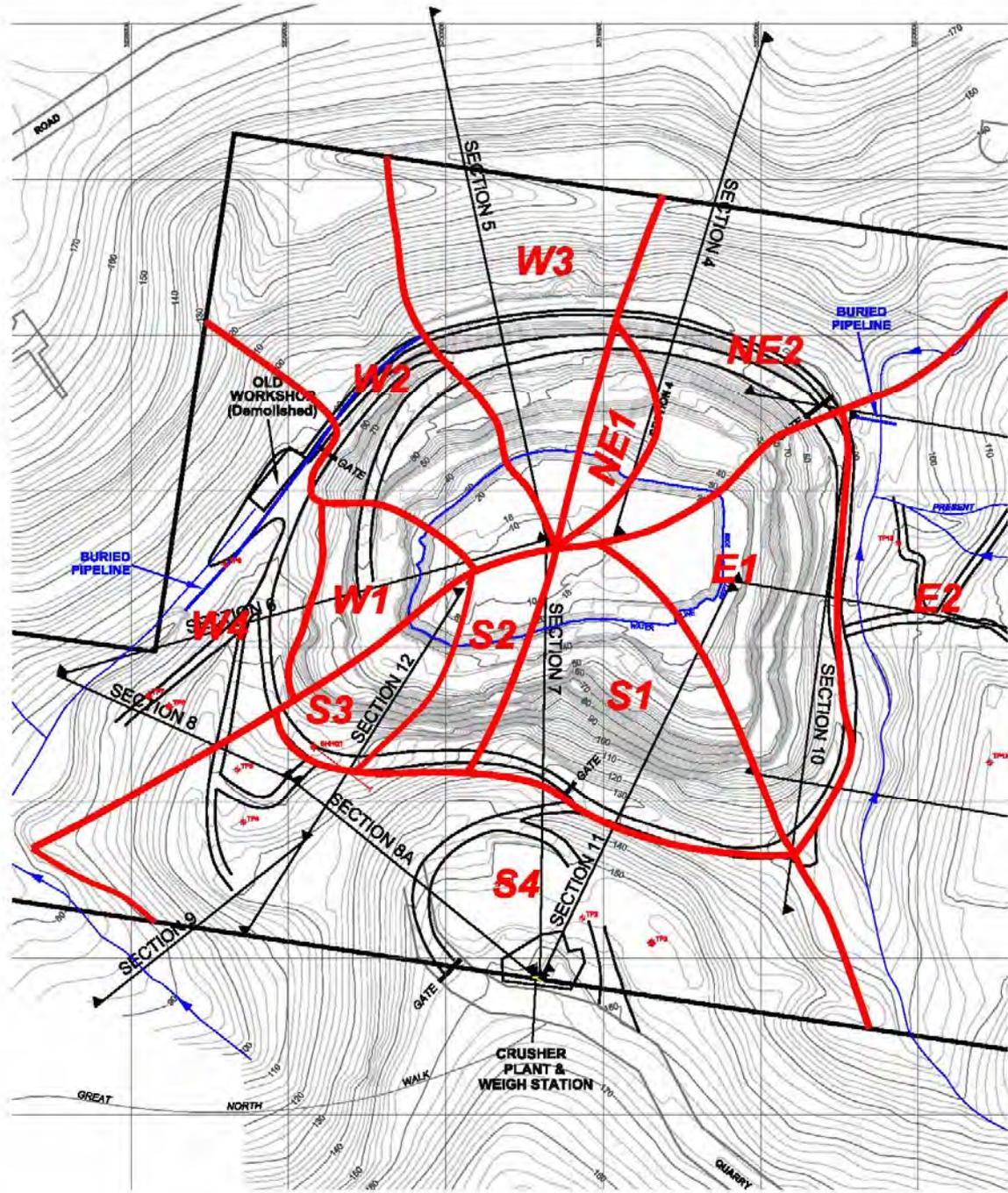


Figure E1: Structural Domains and Cross Section Locations

The cross sections are presented on the following Drawings:

Section 1	PSM1059-6
Section 2	PSM1059-7
Section 3	PSM1059-8
Section 4	PSM1059-9
Section 5	PSM1059-10
Section 6	PSM1059-11
Section 7	PSM1059-12

Section 8 & 8A	PSM1059-13
Section 9	PSM1059-14
Section 10	PSM1059-15
Section 11	PSM1059-16
Section 12	PSM1059-17

E5. GROUNDWATER AND LAKE LEVEL

E5.1. Field Observations

Field observations by PSM in the period September to November 2006 noted a number of minor flow seepages in the excavated quarry faces. These were similar to those noted in the Parsons Brinckerhoff report of 2004.

Seepages have been noted in Figure A5 in Appendix A. The following points summarise these observations.

- i. South Face - No seepages observed.
- ii. East Face - Minor seepages on the benches and on some of the lower batters. It is likely these are from a groundwater table in the fills and residual rocks above and to the east of the face.
- iii. Northern Face (below RL 90m) - Some very minor seepages/zones or damp rock.

Minor seepage of about 0.5L/sec at about RL 50m toward the north western part of the face. It is likely this seepage was associated with seepages noted on the benches above and on the main access track around the quarry at about RL 90m. This seepage is discussed further below.
- iv. Northern Face (above RL 90m) - No groundwater seepage was observed on the ground surface, or from a number of horizontal drains (50mm PVC pipes) installed in the rock faces above the existing concrete drain. The horizontal drains extend to at least 7.5m behind the rock face.

- Parts of the slope were covered in either lush grasses, low scrub or pampas grass. It appears the vegetation is located in areas where the EW/residual breccia rock meets the underlying HW/MW breccia rock, or where more weathered muddy breccia is present. This is likely to indicate groundwater seepage.

- v. Northern Face
(Main Access Track and Concrete lined table drain)
 - No flows were observed entering the drain from the slopes above. Following rain water was found to have ponded in some sections of the drain.

Ponded water was observed on the main access road around the rim of the pit in the north western part of the site. The water was observed on each visit to the area and appears to be continually supplied. The source of the water was not able to be located. However, it appears likely to be resulting from seepages in either zones of filling above the drain or from the more weathered breccia rock above the drain. The ponded water was then found to run down a quarry access ramp below the main roadway and may have contributed to the 0.5L/sec seepage noted in point iii. above.
- vi. Western Face
 - No seepages were observed. However, areas of damp rock in batter faces were noted. These were probably a result of surface runoff a seepage flows for the fills above the face.
- vii. The current lake level is at about RL 28.5m.

E5.2. Piezometers

There are two sources of piezometer data available that indicate ground water levels applicable to the quarry.

The first source is a group of 37 standpipes installed by Coffey Partners in 1989 to study the eastern part of the site – Part 2. These piezometers were monitored for three months between mid December 1989 to mid March 1990, the results for which were presented in the report S8463/3-AG dated July 1990. There is no record of these standpipes being monitored after March 1990 until PSM were able to relocate one of Coffey’s deep piezometers (BH18 – refer to Figure E2). The standpipe was remeasured and the water level found to have been essentially at the same level as measured by Coffey in January 1990 (about 55m below the surface at approximately RL 75m).

It is important to note that during the three months that Coffey monitored the standpipes there was a period of very heavy rainfall. In February 1990 some 430mm of rain was recorded at Wahroonga. This was the month when there was widespread landslip damage in Warringah and Pittwater Shires and the rainfall was of the order of a 1 in 100 year event. Of the 37 standpipes, 13 showed groundwater level rises, followed by similar falls, of between 0.5m and 3m. The rises did not represent a consistent pattern across the eastern area. While most responses were in standpipes located in the weathered breccia horizon below Part 2 of the site (depths between about 8m and 16m), there were similar standpipes that showed no reaction to the heavy rainfall. Similarly three of the standpipes in fresh breccia showed rises of between 0.5m and 2.5m, but others in fresh breccia showed no reaction. This apparent lack of ‘pattern’ does in fact

tell us much about the groundwater regime and associated design criteria, as discussed in Section E5.3, below.



Figure E2: Piezometer Locations

The second source of piezometer data was obtained from a single, vibrating wire piezometer installed in the borehole drilled by PSM in November 2006. The hole was drilled through the breccia rock at the south western part of the site into the surrounding Hawkesbury sandstone. The piezometer was installed at a vertical depth of about 80m at RL 27.45m. Measurements to date indicate a water table at about RL 79.3m. Figure E3 presents a plot of water pressures (as reduced levels) for the piezometer installed in borehole BH HQ1.

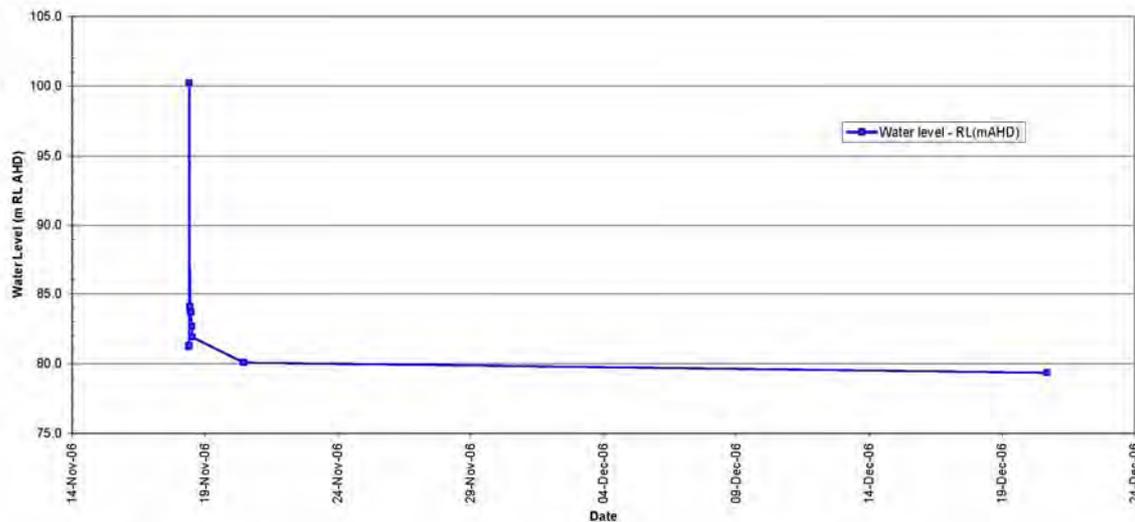


Figure E3: Piezometer Level Measured in BH HQ1

E5.3. Interpreted Groundwater Regime

The interpreted groundwater model for all the quarry faces comprises two systems, namely:

1. System 1 A shallow, perched water system within the fills and underlying weathered breccia rock.
2. System 2 A deep system within the fresh breccia, and surrounding Hawkesbury sandstone.

The perched water of System 1 is controlled by surface infiltration and leakage through the lower permeability weathered breccia to the underlying joined fresh breccia.

Input to System 2 is the downward leakage from System 1, and groundwater movement from the joints and bedding planes in the Hawkesbury Sandstone surrounding the diatreme. Output from System 2 is seepage into the quarry pit.

The groundwater in System 1 (perched) is stored both in the soil and fill pores and in fractures in the weathered breccia. The groundwater in System 2 is stored in the joints and bedding defects in the fresh breccia.

As demonstrated by the Coffey piezometer monitoring in 1989/1999, the fact that most of the groundwater in Systems 1 and 2 is within joints and bedding defects, and that downward seepage from System 1 is not uniform, means that there is a non-uniform response in groundwater pressures to heavy rainfall events.

If a particular standpipe intersects a particular joint that is connected to a location of ready recharge (surface to System 1, or leakage from System 1 to System 2), then that standpipe will show response to rainfall events. Conversely, if a nearby standpipe taps into joints that are shielded from ready recharge, little or no response to rainfall is measured. The importance of this understanding is that readings from individual piezometers, or even sets of piezometer, cannot be used to determine groundwater pressures for engineering design. It is necessary for a conservative view to be taken on the basis of a broad understanding of the groundwater model summarised above.

Following from the discussion above, the current ground water conditions are considered to comprise:

1. System 1 at the base of any fill materials and presenting as seepage at the quarry face at about the contact between the EW/Residual breccia and the HW/MW breccia or within the EW/Residual breccia near the rim of the quarry.
2. System 2 at about RL 75m (or slightly higher) at, or adjacent to the rim of the quarry.

The current ground water regime must also be modelled in stability assessments for a higher design level to consider extreme periods of rainfall or storm events. In these instances, it is System 1 that will be influenced. The design water surface modelled was located at, or just below, the ground surface in the fills or EW/Residual breccia where this is the uppermost unit.

E6. STABILITY ANALYSIS

E6.1. Introduction

Drawing PSM1059-2 presents the Structural Domains identified in the quarry. The drawing also summarises the kinematically possible failure mechanisms in each of the Domains. These mechanisms comprise:

- a. Circular failures in fills, residual soils and weaker rocks.
- b. Planar sliding of blocks of rock on single or complex features dipping out of the quarry walls.
- c. Wedge failures in rock faces on intersecting defect sets in the quarry walls.

A fourth mechanism was considered for the southern quarry face (S1 Domain). This mechanism comprises sliding on steep seams in the breccia rock with possible rock mass failures through the rock mass at/near the toe of the failure.

Finally, at the western rim of the quarry (Domain W1) that may provide a suitable viewing platform location, kinematic assessment of the defects (Appendix A) indicates that wedge type failures are not feasible.

E6.2. Scale of Failure

Inspection of the excavated quarry faces reveals that:

1. There are a large number of individual blocks of rock at the crests, or in the faces of most of the batters, mostly as a result of blasting.
2. These blocks are often loose and partly detached or in zones of numerous blocks.
3. Block sizes vary between 0.1m and 2m in size.

4. Most Domains contain defects dipping out of the quarry faces with some (northern, western and south western faces) containing zones/units of weaker muddy breccia rock.
5. There is no system of water collection or control on the quarry benches.
6. There are a number of areas of debris from past bench scale failures present as well as many individual blocks of rock across bench surfaces that have detached from points higher up in the quarry.



Photo E1: Batter face of NE corner of quarry. Shows block of rock at batter crest of various sizes, a collection of detached blocks along the toe of the rock face and a person for scale.

All of the points above indicate that individual blocks may be expected to continue to fall from the bench faces.

Given the number of blocks present this may be between 10 and 500 blocks of between 0.1m and 2m each and every year.

Further, highly fractured and/or zones of weak rock covering part or full bench heights may also be expected to occur. While we do not have any data to measure the frequency of such events, it does not seem unreasonable given evidence of past failures to assume a rate of 1 bench scale fall every 10 years.

PSM consider the points made above necessitate a HIGH risk of rock fall be associated with regular/permanent public access to the quarry.

This assessment alone requires the quarry area (Part 1) to be quarantined from permanent public access without significant remedial works taking place.

E6.3. Method – Moderate to Large Scale Slope Failure

Following from the discussions in the preceding sections, the computer aided stability assessments undertaken were aimed at assessing the impact of moderate to large scale failures on the quarry, and on the surrounding lands. Specific attention was paid to the northern slopes and the quarry faces that may impact on Parts 2, 3 and 4 of the site.

In considering how to analyse the stability of large scale failures thought was given to the following:

- a. Orientation and extent of defects and units of weaker rock in the quarry faces.
- b. The relatively small size of the quarry void.
- c. The impact of the quarry lake water.
- d. The impact of groundwater in the rock.

In reviewing the above, PSM consider that moderate to large scale failures will be 'confined' by the geometry of the quarry and rock defects.

The analyses were undertaken using the two dimensional computer program Slide.

The obvious limitation with this is that moderate to large scale failure will be influenced by the 'confined' nature of the quarry. To take account of this the methodology outlined below was adopted.

- (a) Consider failure planes defined by measurement of defects and the kinematic assessment.
- (b) Adopt a 'target' FOS of 1.1 for slopes where planar and large scale wedge failure may kinematically be feasible. This value of FOS is considered appropriately conservative given the past performance. That is, there are no large scale failures observed in aerial photographs, no recorded events in available information and no evidence of current, large scale activity in the quarry walls or surrounding areas.
- (c) Initially use the material strength parameters given in Appendix B in the stability analysis.
- (d) Run the analysis, using the current groundwater regime, and vary the design strength parameters to obtain the target FOS.
- (e) Complete the analysis for varying lake and groundwater levels.
- (f) Consider the impact of backfilling the quarry.

The stability analyses present in the following sections are concerned with moderate to large scale failures that:

- a. Impact on the use of the quarry.
- b. Impact on the lands around the quarry such as the main access road or extent to which access of Parts 3 and 4 of the site could be developed.

The stability analysis also considers the impact of rising lake levels.

E6.4. Results

The results of stability analyses have been summarised in Tables E1 to E6. Each table represents stability assessment of the following geotechnical domains.

TABLE	FACE/QUARRY AREA	DOMAIN
E1	East	E1
E2	North East	NE1 and NE2
E3	North	W3
E4	North West	W2
E5	South West	S2 and S3
E6	South	S1

Outputs from each of the stability analyses are included in Attachment E1.

**TABLE E1
RESULTS OF STABILITY ANALYSES – DOMAIN E1**

MODEL AT SECTION 2	LAKE/FILL LEVEL	FOS	OUTPUT FIGURE
2-2	RL 28.5m	1.9	1
2-3	RL 50m	1.8	2
2-4	RL 70m	1.8	3
2-5	RL 88m	1.7	4

**TABLE E2
RESULTS OF STABILTY ANALYSES – DOMAINS NE1 AND NE2**

MODEL AT SECTION 4	LAKE/FILL LEVEL	FOS	OUTPUT FIGURE
NE2-1	RL 28.5m	1.1 Minimum	5
NE2-2	RL 50m	1.0 Moderate to deep failure	6
NE2-3	RL 70m	1.0	7
NE2-4	RL 88m	1.0	8
NE2-5	Fill to RL 30m	1.2	9
NE2-6	Fill to RL 50m	1.4	10
NE2-7	Fill to RL 70m	1.3	11
NE2-8	Fill to RL 88m	1.6	12

**TABLE E3
RESULTS OF STABILTY ANALYSES – DOMAIN W3**

MODEL AT SECTION 5	LAKE/FILL LEVEL	FOS	OUTPUT FIGURE
W3-1	RL 28.5m	1.1 to 1.2 Overall slope – note shallow failures of bench to interbench scale have FOS <1.0.	13
W3-2	RL 50m	1.4	14
W3-3	RL 70m	1.3 to 1.4	15
W3-4	RL 88m	1.3 to 1.5	16
	RL 88m	3.2 assumed. Surface plane back to near houses along Manor Road.	17
W3-5	Fill to RL 30m	1.5 for deep failures surface. Shallow features FOS <1.0.	18
W3-6	Fill to RL 50m	1.7 Overall.	19
W3-7	Fill to RL 70m	1.9 Overall.	20
W3-8	Fill to RL 88m	>2.0 Overall. Surfaces in upper part need to be modelled as rock map failures – see Model 3-9 and 3-10.	21

MODEL AT SECTION 5	LAKE/FILL LEVEL	FOS	OUTPUT FIGURE
W3-9	Fill to RL 88m	<1.0 – Residual/EW breccia saturated.	22
W3-10	Fill to RL 88m	1.3 – Current water table in Residual/EW breccia.	23
Section 5 in Northern Batter	Current Ground Water Conditions	1.2	24
W3-11	Fill Buttress on Upper Face to RL 120m (2H:IV)	1.8 in buttress 1.2 in residual batter above buttress.	25
W3-1	Fill buttress on upper face to RL 130m (2.5H:IV)	1.6 Minimum	26

**TABLE E4
RESULTS OF STABILTY ANALYSES – DOMAIN W2**

MODEL AT SECTION 5	LAKE/FILL LEVEL	FOS	OUTPUT FIGURE
W2-1	RL 28.5m	1.1, shallow bench to interbench scale failures 0.8 to 1.0. Note current groundwater system in place.	27
W2-2	RL 28.5m	1.0 to 1.1	28
W2-3	RL 50m	1.0	29
W2-4	RL 70m	1.0	30
W2-5	RL 88m	0.9 to 1.0	31
W2-6	Fill to RL 30m	1.1	32
W2-7	Fill to RL 50m	1.2	33
W2-8	Fill to RL 70m	1.3	34
W2-9	Fill to RL 88m	1.5	35

**TABLE E5
RESULTS OF STABILTY ANALYSES – DOMAINS S2 & S3**

MODEL AT SECTION 12	LAKE / FILL LEVEL	FOS	OUTPUT FIGURE
S3-1	RL 28.5m	0.7 to 1.0 – If bedding extends 20m to 30m behind quarry face. 2.4 for surface extending into sandstone area	36
S3-2	RL 50m	0.7 to 1.1	37
S3-3	RL 70m	0.7	38
S3-4	RL 88m	0.6 to 0.7	39

**TABLE E6
RESULTS OF STABILTY ANALYSES – DOMAIN S1**

MODEL AT SECTION 11	LAKE / FILL LEVEL	FOS	OUTPUT FIGURE
S-3	RL 28.5m	1.3 – Rock mass failure at toe of slide in breccia : c=300kPa, $\phi = 45\text{deg}$	40
S-3A	RL 28.5m	1.2 – Rock mass failure at toe of slide in breccia : c=250kPa, $\phi = 40\text{deg}$	41
S-3A	RL 28.5m	1.1 – Rock mass failure at toe of slide in breccia : c=200kPa, $\phi = 40\text{deg}$	42
S-3A	RL 28.5m	1.0 – Rock mass failure at toe of slide in breccia : c=150kPa, $\phi = 40\text{deg}$	43
S-3A	RL 28.5m	1.0 – Rock mass failure at toe of slide in breccia : C=150kPa, $\phi = 35\text{deg}$	44
S-3A	RL 28.5m	0.9 – Rock mass failure at toe of slide in breccia : c=100kPa, $\phi = 35\text{deg}$	45
S-4	RL 50m	1.3	46

MODEL AT SECTION 11	LAKE / FILL LEVEL	FOS	OUTPUT FIGURE
S-4A	RL 50m	1.1 ¹	47
S-5	RL 70m	1.3	48
S-5A	RL 70m	1.2	49
S-6	RL 88m	1.4	50
S-6A	RL 88m	1.3	51
S-7	Fill to RL 50m	1.3	52
S-7A	Fill to RL 50m	1.1	53
S-8	Fill to RL 70m	1.4	54
S-8A	Fill to RL 70m	1.2	55
S-9	Fill to RL 88m	1.9	56
S-9A	Fill to RL 88m	1.6	57

E6.5. Assessment

E6.5.1. Eastern Quarry Face

The interpreted geotechnical model, and observations of defects at the quarry face indicate failures would follow a circular path. The stability analyses indicate FOS of greater than 1.7 for all feasible groundwater and lake levels.

E6.5.2. North East Face

Larger scale failures in this location may be able to occur shallow to flat lying contact/joints in the breccia and steeply dipping bedding.

For a target FOS of 1.1, the shear strength parameters were reduced from those in Appendix B to

$$c = 0\text{kPa}, \quad \phi = 20^\circ \text{ (reduced from } 35^\circ\text{)}$$

The results (Table E2) indicate marginally stable batter slopes as lake water levels rise from the current level of RL 28.5m to the levels of the natural outlet from the quarry at about RL 88m. The results also indicate a decreasing level of stability as the lake level rises.

Assessment of the level to which backfill would be required to be dumped to stabilise the NE batter indicates nearly complete filling to the level of the existing concrete drain would be required to achieve a FOS greater than 1.5.

¹ Rock mass failure at toe of slide in SW/FR breccia with $c=200\text{kPa}$ and $\phi=40\text{degrees}$.

E6.5.3. North Face and Batter Above Drain

Stability analyses indicate that moderate to large scale failures² have a FOS of between 1.1 and 1.5, depending on lake level. The computed FOS are typically less than would be accepted in normal civil engineering projects, i.e. a FOS of greater or equal to 1.5.

The results indicate that as the lake level rises stability is increased. That is, the impact of increased water pressure on the rock face is marginally greater than the destabilising impact of water levels within the defects.

The results also indicate that there is no meaningful risk of slope instability on the houses to the north of the quarry.

Backfilling of the quarry provides a FOS of greater than 1.5 for moderate to large scale failures as fill levels rise above RL 30m.

Analysis of the upper, more weathered breccia slopes indicate that stability under current conditions is marginal from a civil engineering viewpoint (FOS = 1.3). The stability reduces to less than acceptable as the water table in the weathered breccia batter increases to near the surface.

Finally, remedial works in the form of a fill buttress indicates a batter is required to a level at least RL 130m with a face formed at about 2.5(H):1(V) to achieve a FOS of greater than 1.5.

E6.5.4. North Western Slopes

Failure planes may form on moderately steep contacts in the muddy breccia and shallow to moderately steep seams in the breccia rock.

The stability analyses indicate marginal levels of FOS for moderately large scale 2D on the defects noted in this area when the starting assumption that the quarry faces are only marginally stable (FOS=1.1) is applied. Indeed, the analyses show that rising lake levels have a minor detrimental impact on the stability of the rock face with values of FOS reducing to about 1.

The actual stability in this area is likely to be better than indicated by the 2D assessment as the defects modeled extend into the quarry wall and hence 3D effects/confinement will assist in stabilizing the slope. Notwithstanding this, this part of the site should be considered to represent a moderate to high risk of failure.

Remedial filling to the quarry only provides a FOS greater than 1.5 once filling has reached the level of the quarry rim, i.e. about RL 88m.

E6.5.5. South Western Faces

Although representing a small section of the quarry, stability analyses indicate stability of the immediate quarry face is marginal. The failure surfaces may potentially extend up to about 40m beyond the crest (rim) of the quarry.

Rising lake levels further destabilise the quarry faces in the area.

² For failure surfaces affecting the majority of the northern batter.

E6.5.6. South Face

Failure of the southern face is expected to be along very steeply dipping seams in the breccia rock. The stability is controlled by breakout through the breccia rock mass.

Back analysis of the current slope geometry indicates that, for a target FOS of 1.1, a reduction in the strength of the rock at the breakout is required. This is considered to represent a lower bound of the slope stability.

The results indicate stability of the rock face, extending up to about 20m to 25m behind the face is less than would be acceptable in a normal civil engineering project.

The stability assessment also indicates a marginal increase in stability as the lake level rises.

If no further seams occur in the breccia rock behind those noted in the quarry faces, then the computed factors of safety for failure surfaces extending back to the contact with the Hawkesbury Sandstone are greater than about 1.8.

Backfilling of the quarry with dumped fill indicates a FOS of greater than 1.5 occurs as filling nears RL 88m.

E6.6. Findings

The stability assessments for the quarry have been summarised into a plan showing our assessed risk of instability. This assessment is presented in Drawing PSM1059-18 which is repeated below as Figure E4.

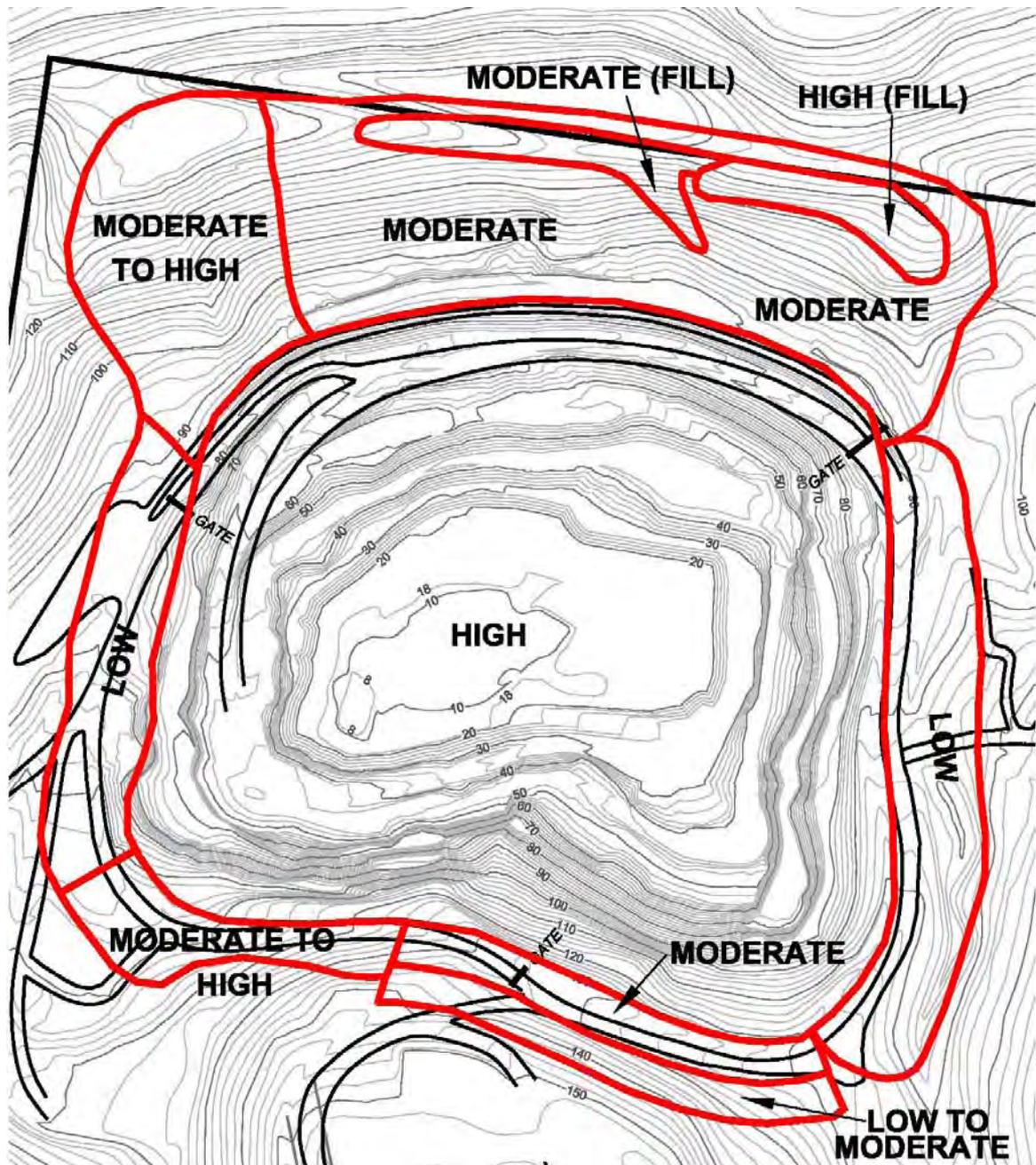


Figure E4: Risk of Quarry Face Instability

Particular points to be noted in terms of the stability of the quarry are discussed below.

1. There is a high risk of small scale instability, at bench scale, around all of the quarry faces. This instability makes it dangerous for the public to access the quarry.

2. There is no meaningful risk of deep seated sliding in the slopes to the north of the quarry site, i.e. below the residential areas beyond the northern batter.
3. There is a moderate risk of failure in the south faces of the quarry, extending up to 25m behind the crest. This impacts directly on the existing access road north of Part 3 of the site.
4. There is a low risk (computed) of failure in the southern face extending back to the contact zone between the breccia rock and the sandstone.

While PSM has confidence the interpreted geotechnical model at this location is appropriate, it is recommended that for permanent public access on land use, the northern limit of Part 3 should be located at the boundary between the sandstone and breccia rock.

5. There is a moderate to high risk of slope instability of moderate to deep failures in the south western faces. The possible failure surfaces extend about 40m beyond the rim of the pit to the Hawkesbury Sandstone. This contact defines the limit at the northern side of Part 4 of the site. It also impacts on the access road at this section of the site.
6. There is only a low risk of deep seated instability in both the western and eastern faces of the quarry. This is primarily due to advantageous orientation of bedding in the breccia at the eastern side and orientation of defects in the western side.
7. There is a moderate and high risk of instability associated with fill materials on the northern batter.

These fills comprise:

- (a) fill in the sound barrier along the crest of the batter,
- (b) fill in a former haul road above the eastern end of the concrete drain,
- (c) possible fill within an old creek alignment on the northern batter extending down the face from the sound barrier. It is possible that this interpretation of the site conditions is inaccurate as it was primarily based on interpretation of aerial photographs by PSM and AAM Hatch.

The assessment of risk instability is based on an assumed high (design) groundwater system saturating the fill.

8. There is a moderate risk of shallow instability in the residual soils and upper, more weathered breccia rock in the northern batter above RL 90m. This risk increases to high as water levels rise in these materials.

E7. QUARRY REMEDIAL OPTIONS

There are three basic options for treatment of the quarry, namely:

1. Do nothing.
2. Backfill the quarry – fill materials sourced externally to site or from cutting back (flattening) the existing quarry faces.
3. Mechanically stabilise the quarry faces.

E7.1. Do Nothing

The do nothing approach is likely to be the least costly option. The following points set out the limitations and advantages of this option.

Limitations

- a. No permanent public access to the quarry itself, i.e. quarantine the quarry.
- b. No permanent public access on the main access roads at the southern side of the quarry. However, infrequent use of these roads is considered to be acceptable. Such users may include the local historical society and Council for maintenance works. Consideration may be given to implementation of ideas set out in Section E7.4 such as monitoring to minimise the risks to casual users of these roads.
- c. PSM recommend the lake level be maintained at about its current level, i.e. about RL 30m.

The prime purpose of this is to limit the risk of moderate sized failures of the quarry walls occurring. While such failures would not affect the use of areas set out in Parts 2, 3 and 4, Council should consider the impact any such failure would have on public perceptions of the quarry stability.

Requirements and recommendations on pumps, associated costs and operating levels are set out in Section E7.5.

- d. Infrequent access to the quarry floor will require some remedial works such as:
 1. Scaling of faces to remove unstable blocks,
 2. Some rock dowels (bolts) and shotcreting of potentially unstable zones and defects.
 3. Mesh draped over areas of loose, small blocks.
 4. Surface and some horizontal drains.
 5. Limitations placed on areas able to be accessed.

Such works combined with monitoring and inspections of the rock faces (refer to Section E7.4) should limit the risk to acceptable levels.

Estimated costs to allow infrequent access to the quarry floor are estimated to be:

- Scaling	\$10,000
- Rock dowels (5% faces)	\$55,000
- Shotcrete (5% faces)	\$45,000
- Mesh	\$10,000
- Drainage works	\$45,000
Total	\$165,000
	say <u>\$180,000</u>

The estimate above should be increased if heavy vehicles such as fill laden trucks and dozers are to access the quarry. The revised cost estimate is:

- Scaling	\$10,000
- Rock dowels (10% faces)	\$110,000
- Shotcrete (10% faces)	\$90,000
- Mesh	\$10,000
- Drainage works	\$100,000
Total	<u>\$320,000</u>

Advantages

- e. Least expensive.
- f. Water pumped from the quarry lake may be used for uses such as watering gardens or possibly some industrial uses. Water level variation is recommended to be kept between RL30m +4m and RL30m -10m.

If this option were adopted, we recommend the existing concrete drain be maintained.

E7.2. Backfill the Quarry

E7.2.1. Conceptual Design

The conceptual backfill design shown in plan in Drawing PSM1059-19 and in Section in Figure E5 below. This concept is based on using natural materials sourced from outside the quarry area.

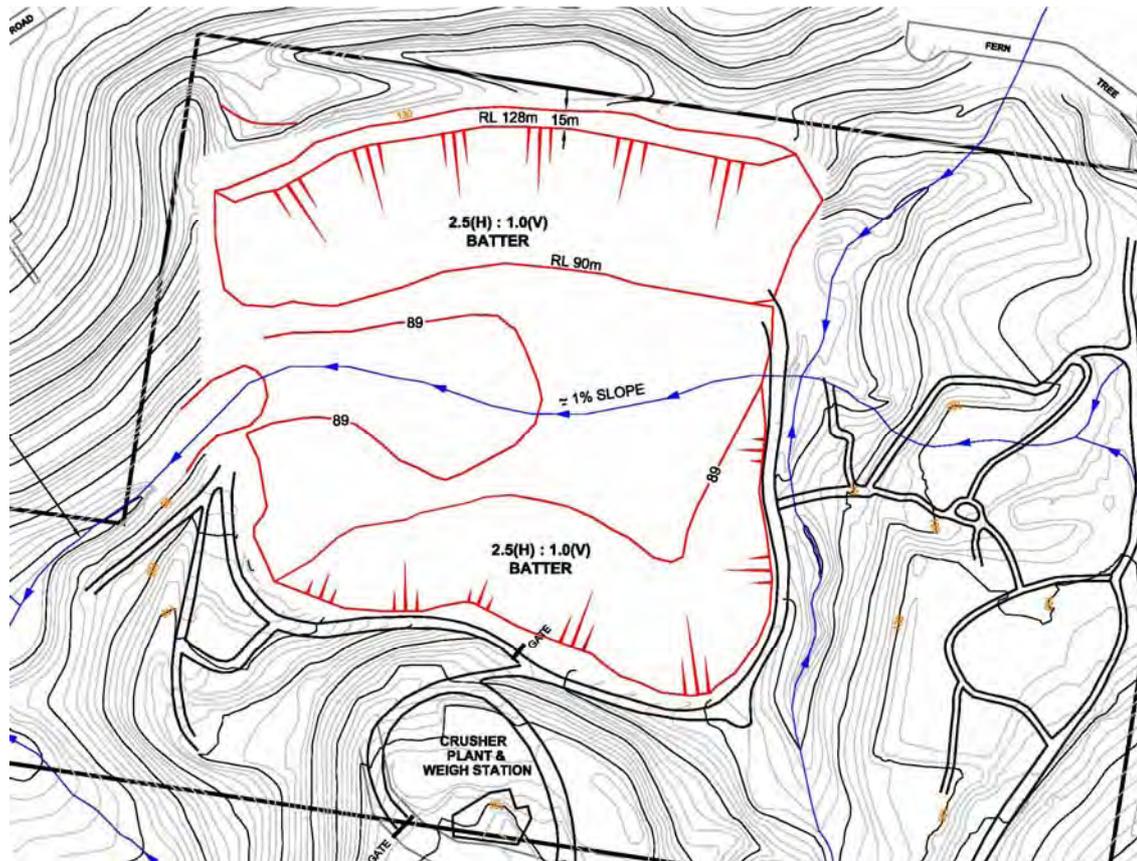


Figure E5: Quarry Backfill – concept design

The concept design is intended to remove all unacceptable instability risks and comprises the following features:

1. Dumped fill (VENM – virgin excavated natural material) placed in quarry void up to a level of between RL88m at the west and RL 90m at the east. Total fill volume is about 3.3 million cubic metres.
2. Compacted fill buttresses (VENM) formed along the northern and southern excavated rock faces of the quarry. These buttresses are to remove the high risk associated with rock falls from the rock faces (refer to Section E6.2) and the risks associated with fill, residual soils and weathered rock at the northern batter.
3. Runoff and crack flows from the eastern areas of the site are to be directed into a new creek. The new creek would be aligned through the central part of the quarry on the dumped fill to deliver waters into Old Man’s Creek to the west/south west.
4. The existing concrete drain is removed.
5. The fill buttresses are to be formed at:
 - a batter angle of 2.5(H):1(V)
 - to RL 128m at the northern batter with a 15m wide crest
 - to the pit rim along the southern, south western and eastern sides.

6. Fill volumes associated with the buttresses are about 0.3 million cubic metres for the southern side and 0.6 million cubic meters for the northern side.

E7.2.2.Reduced Works

Stability assessment results indicate the following sectors of the quarry require backfill to a nominal RL 88m to render them 'safe', i.e. have a FOS of greater than 1.5 against moderate to large scale failure.

1. North eastern faces.
2. North western faces.
3. South western faces.
4. Southern faces.

In light of the above, and assuming that the pit will remain quarantined from public access the following reduced filling may be considered to be those outlined in the concept layout in Section E7.2.1 without the southern buttress. Hence, reducing the volume of fill by only a relatively minor amount (0.3 million cubic metres).

Lastly, information is also provided on stabilising only the northern batters with fill. This work would require the batter at the northern side to simply be extended to the pit floor. We would strongly recommend the quarry lake level be maintained at about RL 30m. This option would be considered to represent a minimum fill option.

The volume of fill required for these works is about 2.8 million cubic metres.

This option requires the quarry to be quarantined. Council should also be aware of the perception issues that may arise due to instability of the southern and south western quarry faces.

E7.2.3.Slope Cut Back

Backfilling of the quarry may also be achieved by sourcing natural materials from the site itself. This could be achieved through cutting back of the existing quarry faces. The advantages and limitations of this approach are listed below.

Advantages

- a. Fill materials are sourced on site thereby limiting, or removing all need for truck movements on the streets surrounding the quarry.
- b. Improvement in the stability of the southern, eastern and western quarry walls to allow public access above, and below these faces. These improvements are expected to be achievable even with a lake formed in the quarry up to the natural overflow at about RL88m.
- c. Improvement in the stability of the northern quarry slopes such that public access would be acceptable and the potential for slope failures extending beyond the boundary are reduced to an acceptable level, ie. have a FOS greater than 1.5.

- d. Some improvement in the stability of the quarry walls in the south western part of the quarry. Calculations indicate that unless cut back works extend into Part 4 then stability of the faces will remain marginal.

Limitations

- e. Excavation of the slightly weathered and fresh breccia may require the use of drill and blast techniques posing noise and dust issues for Council and nearby residents.
- f. If blasting is not allowable, or limited in number/size of blasts the time required, and hence costs to flatten the quarry walls may be significant.
- g. Development of a lake in the quarry void would reduce the impact of cut back works, particularly for the south western and northern slopes of the quarry. This implies pumping of water collecting in the quarry would be necessary.
- h. Drainage works in the form of horizontal drains would be required along at least the northern slopes of the quarry in order to achieve the target factor of safety of 1.5.

Obviously there are numerous combinations of cut back and filling that may be considered by Council. The concept presented below is based on achieving a balance of cut and fill. The concept is also shown in Drawing PSM1059-22.

- Place dumped fill in the quarry to about RL56m – all fill materials sourced from flattening of the existing quarry walls – requires about 1.1 million cubic meters of filling.
- Cut back the northern slope above RL56m to an overall batter form at about 1.85 to 1.9(H):1(V).
- Cut back the southern wall to an overall batter formed at about 40 degrees to the horizontal. This batter was chosen for stability and to ensure no impact on the northern boundary of Parts 3 and 4. This work should extend around into the south western part of the quarry.
- Flattening of batters at the eastern side are not considered necessary. Major stabilisation works to the eastern and western faces may be able to be limited to minor cut backs to simply provide an improved level of stability.
- Provision of slope drainage measures in the form of surface runoff control and horizontal drains in the rock faces, particularly along the northern slopes.
- Do not allow a lake to develop of more than 10m depth to maintain improvements in stability of the northern slopes.
- Excavation and placement costs (dumped fill) estimated to be \$6.6million.

E7.2.4. Land Use

If the quarry is backfilled, the least expensive method is to execute these works as a mining style operation. That is, dump the material at the rim of the quarry and push the material into the void using dozers. Such works could readily be undertaken from the eastern side where access is relatively simple (from an engineering viewpoint).

The major limitation of this style of operation is that significant settlements of the fill would occur over a long period of time. Settlements would be in the order of say;

1. Up to 0.5m to 1m in the first 5 to 10 years after filling is completed.
2. Up to a further 0.5m to 1m over the next 90 years.

In order to limit settlements to values less than the settlements outlined above, an alternative approach is required. This means access the bottom of the quarry with trucks, dozers and/or compactors to allow the materials to be dumped and spread in layers of say 1m to 2m thickness across the fill area of the quarry flow. This style of work would limit settlements to at least half, if not one third of the settlements given above.

Obviously, this limits the land use significantly and in fact places particular requirements on the design and on long term maintenance of the proposed creek.

Possible land uses may be:

1. playing fields/open space,
2. nature reserve,
3. walking tracks.

In order to be able to access the bottom of the quarry for the backfilling discussed above, methods such as those outlined in Section E7.4 would need to be implemented and a certain amount of remedial works to quarry faces would be required.

The estimated costs of backfilling works are discussed in Section E7.2.5.

Finally, Council should be aware of projects such as the Penrith Lakes and backfilling of the Enfield brick pit. Both of these were intended to provide areas for residential development with fill carefully placed, compacted and tested. Yet despite all efforts, these projects are understood to have had either no success or significant problems – mainly due to insurance issues.

E7.2.5. Other Impacts

Backfilling of the quarry would have two major implications to Council land the people who live and work in the areas surrounding the quarry.

1. Provision of the large volume of fill required would require a sources of fill such as a major tunnel or road project. The Epping to Chatswood Railway project realised about 2 million cubic metres of fill, and so indicates at least two projects of this scale would be required to fill the pit.

2. A large number of truck movement would be necessary to deliver the fill to site. Assuming the typical truck is a 6 wheel double bogey that can provide about 10m³ of material per visit, then a total of about 425,000 truck visits, or 850,000 truck movements would be required. Assuming a frequency of one truck every 5 minutes on weekdays for 8 hours a day (about 96 trucks per day), this equates to almost 900 weeks, or almost 20 years.
3. At this time we understand it is necessary to buy VENM (Virgin Excavated Natural Material). If this is the case, and we assume say \$125 to \$150 per truck load of fill then the costs are very high indeed. So, it is likely that the backfilling option may not be feasible until Council can negotiate with a project that requires somewhere to deposit the excess fill.

E7.2.6. Backfilling Cost Estimate

The following sets out our estimate of costs for these works/

- | | | |
|----|---------------------------------|---|
| 1. | Full Quarry Backfill | <ul style="list-style-type: none"> - 4.2 million cubic metres VENM. - Supply costs greater than \$55 million. - Dump and push over edge of quarry requires a dozer (D6/D7 say) for 20 years. <p>Assuming 40 hours per week for 50 weeks per year over 20 years, and a starting rate of \$100/hr (negotiated) with a 3% increase per year the costs would be about \$5 million.</p> <p>An alternative would be to buy a second hand machine and Council operate it.</p> |
| | TOTAL COST | <u>>\$60 million.</u> |
| 2. | Minimum Recommended Quarry Fill | <ul style="list-style-type: none"> - Only difference is a saving of 0.3 million cubic metre of fill. - Reduce costs to 1. above by between \$5 million and \$6 million |
| | TOTAL COST | <u>> \$55 million.</u> |

3. Northern Batter Fill Only
- Supply 2.8 million cubic metres of fill. \$35 million to \$40 million
 - 11 years to supply fill.
 - Dozer costs of about \$2.5 million.
 - Ongoing pumping costs, say \$2,000/year.
 - Pump purchase costs (Davey SS45/07) \$20,000
 - Power Supply \$200,000

TOTAL COST (next 50 years) \$38-43 million

4. Move fill from Part 2, 3 and 4 into quarry void
- Such works would cost about \$5/m³.
 - Assuming Parts 2 and 4 were placed in the quarry as these may be relatively readily accessed, the cost would be about
- | | |
|--------|---|
| Part 2 | \$1.85 million (0.37 x 10 ⁶ m ³) |
| Part 4 | \$1.5 million (0.3 x 10 ⁶ m ³) |

Given the relatively small volume and the costs associate in moving the existing filling in Parts 2 and Part 4, we consider it would not be an effective use of these materials for them to be placed as backfilling in the quarry.

5. Cut Back Batters and Fill Quarry to RL56m
- Move 200,000m³ of soils and EW rock with excavators @ \$5/m³.
 - Excavate via drill and blast remaining 900,000m³ of HW and less weathered breccia rock. Will require pre-split of final batters. Allow \$15/ln of pre split works and \$5 per cubic meter of rock moved.
 - Establish access to quarry floor \$320,000
 - Establish drainage works \$0.5 million
 - Ongoing pumping costs, say \$2,000/year.
 - Pump purchase costs - \$20,000
 - Power Supply \$200,000

TOTAL COST (next 50 years) say \$6.6 million

E7.3. Mechanical Stabilisation

E7.3.1. Work Scope

Mechanical stabilisation of the quarry rock faces could be undertaken to remove the high risk of instability associated with:

- 1 rockfalls from the quarry faces, and
- 2 bench to inter bench scale failures.

The types of works envisaged here comprise:

1. Rock bolts to support key blocks and defects.
2. Shotcrete to support zones of weakness and prevent erosion of the batters.
3. Mesh to support zones of weakness or loose blocks of rock.
4. Drainage holes to control the build up of groundwater pressure.
5. Scaling of the rock faces to remove detached blocks, areas of significant blast damage.

The following provides some detail on the main components listed above.

Rockbolts

1. passive steel dowels,
2. fully cement grouted,
3. nominally 3m to 5m long,
4. corrosion protection measures provided – cement grout, possibly sheathed bolts or stainless steel bolts. Also, bolt components – face plates, nuts, washers must also be corrosion resistant and/or protected against corrosion.

Shotcrete

5. nominal thickness of 100mm assume (average),
6. may be steel fibre or mesh reinforced as required,
7. may be pinned to rock face by rockbolts (see above),
8. have drainage holes or other measures provided to ensure no hydrostatic pressure can develop.

Mesh

9. either corrosion resistant or inspected and replaced as necessary,
10. may be draped over rock faces if used as a temporary protection measure during backfilling or other activities such as pump maintenance (refer to Section E7.4).

Drainage Holes

11. have a nominal length of 10m to 15m,
12. nominal 50mm to 75mm diameter,
13. be able to be inspected and cleaned during service to ensure they are operating correctly.

E7.3.2. Cost Estimate

Table E7 provides a breakdown of estimated costs for the various quarry faces.

**TABLE E7
STABILISATION COST ESTIMATE**

FACE	COMPONENT COSTS \$,000'S			TOTAL
	Scaling	Shotcrete/ Mesh	Bolts	
EAST	\$70	\$700	\$550	\$1,320
NORTH EAST	\$50	\$400	\$500	\$950
NORTH	\$90	\$700	\$850	\$1,640
WEST	\$75	\$280	\$350	\$705
SOUTH AND SOUTH WEST	\$85	\$800	\$1,000	\$1,885
TOTALS	\$285	\$2,080	\$2,250	\$6,500

Other costs that need to be factored into these works are:

- | | | |
|---|-------|--------------------|
| 1. Design, Inspection and Certification | allow | \$300,000 |
| 2. Horizontal and surface drainage | allow | <u>\$750,000</u> |
| | | <u>\$1,050,000</u> |

Finally, estimates of this type of work are very difficult to make. We strongly suggest Council allow for a variation of anything between 20% and 100%. Notwithstanding this, our best estimate is to allow for a contingency of 30%, which gives a total minimum cost of \$10 million dollars.

E7.3.3. Lake Level

PSM recommend the lake level be maintained at About RL30m as discussed in Section E7.1. The requirements, and costs of pumping are discussed in Section E7.5.

E7.4. Monitoring, Inspections and Work Methods

E7.4.1. Introduction

This section provides discussion on the measures that may be implemented by Council to enable a reasonable level of safety to be provided for infrequent access to the quarry (pump maintenance) and to assist with the management and awareness of the quarry performance and safety of areas around the quarry. These measures can also readily be used to augment other stabilisation works.

Further, the measures and ideas can, and are, often applied in mining and construction works where access and work is required in potentially dangerous situations.

1. monitoring of slopes,
2. inspections of slopes and rock batters,
3. work methods and access limits.

E7.4.2. Monitoring

There are many ways that quarry slopes can effectively be monitored and may include:

1. Physical Surveying of points on quarry faces and around the quarry rim. Surveying may be done by physical survey in the field on a regular basis or remotely via GPS (or other suitable methods).
2. Inclinerometers installed to measure failure planes or zones, particularly in the case of moderate to deep seated slope instability.
3. Tension Crack Monitors involving a tensioned steel wire attached to an alarm or other alternative system may be used to monitor specific defects, tension cracks or areas of the quarry rim.

This would only be set up in the case of observed failure, or possible areas of critical importance or public exposure such as below an viewing platform.

The level, extent and frequency of monitoring are very dependent on the use/access being made of/to the quarry and adjoining areas as well as the condition of the quarry.

It is worth noting that failures in rock slopes do not happen instantly. Failures occur over periods ranging from weeks, months and even years. In the period leading up to an actual failure event, many precursors would be evident such as cracking behind the crest

of the slope, additional rockfalls of single or several blocks, deformation of the rock face, rising groundwater levels and opening of joints.

All of these precursors can be looked for, and observed where present, by a suitable monitoring programme to allow the rock face to be suitably managed. In light of this, we suggest that Council consider establishing a monitoring programme such as the one described below to provide some understanding of the quarry performance over time.

The first part would be to establish a series of survey points. This may, in the first case involve say a total of 14 survey stations/markers established around the quarry. Indicative locations are shown in Figure E6.

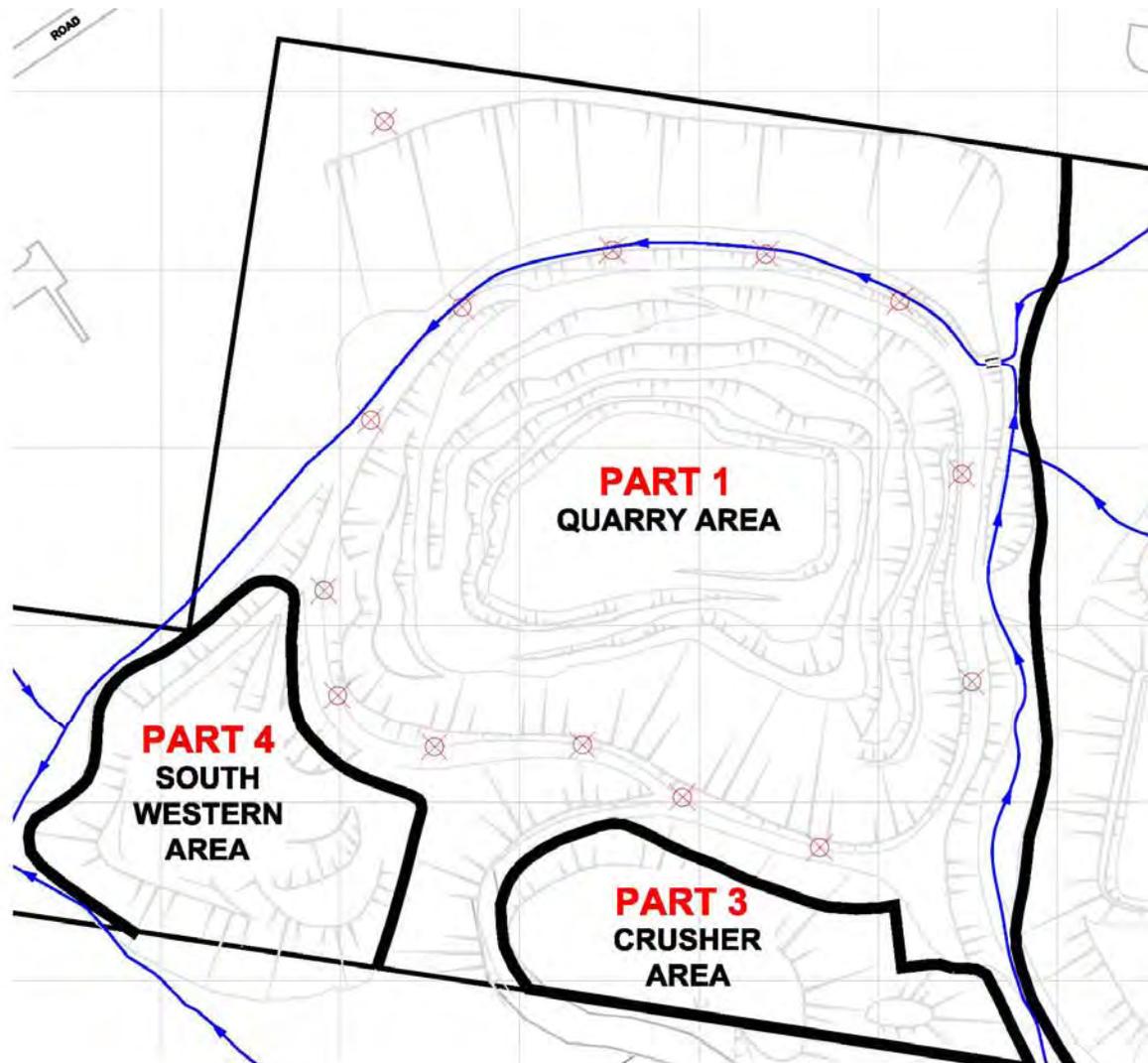


Figure E6: Initial survey monitoring point locations

In order to establish a data set against which future readings can be measured, we suggest the points be surveyed once a week for a month then monthly for a year after which the frequency regime should be reviewed. A review should consider not only the results but also:

- any decisions on land use,
- observations of quarry faces (see inspections discussion below),
- rainfall records,
- lake level.

Suggested frequencies of the initial measurements and inspections are set out in Table E8.

**TABLE E8
INITIAL MONITORING REGIME (12 MONTHS)**

DATA	FREQUENCY
Survey	Weekly for one month, then monthly
Rainfall	Daily – rainfall gauge at Council offices may be appropriate.
Lake Level	Weekly to fortnightly. Council may be able to set up a remote system to limit the number of visits into the pit – such as a painted stake/pole able to be observed from outside the quarry void.
Face Inspections	Quarterly, and also following any significant rainfall events to observe and record any signs of instability or failure.
Piezometer BH HQ1	Monitor level quarterly.

Indicative costs for establishment and ongoing monitoring are presented in Table E9.

**TABLE E9
INITIAL MONITORING – INDICATIVE COSTS**

DATA	ESTABLISHMENT	ONGOING (12 months)
Survey	\$10,000	\$10,000
Rainfall	\$5,000	\$1,000
Lake Level	\$2,000	\$1,000
Face Inspections	Nil	\$10,000
Piezometer BH HQ1	Nil	\$2,000

Finally, the following points are made in relation to inclinometers and their application at the quarry.

- (i) Inclinometers are an excellent means of monitoring deep seated slope instability. They are typically used for monitoring known or suspected landslides.

- (ii) These instruments are typically monitored on a regular basis. We would suggest the normal practice would be to measure an inclinometer on a daily basis for a known landslide. This may be reduced to somewhere between weekly and monthly for a suspected landslide but would be likely to continue for a long period, if not indefinitely.
- (iii) Measurement is done by manually by lowering a probe down the inclinometer hole and hence requires frequent site visits and access.
- (iv) Inclinometers do not typically have a long lifespan and often become blocked after several years or less. This may necessitate replacement.

As discussed above, inclinometers are normally installed in boreholes drilled to target particular geological features involved in slope failure, or particular failure mechanisms – we have no such targets at this time.

If Council's decision as to redevelop the quarry, it may at some time in the future, and at some particular locations, be appropriate to install inclinometers or other slope monitoring equipment. This will depend on particular developments proposed, and in our view it is too soon to make this call.

E7.4.3. Work Methods and Site Preparation

Work methods may be established that take into account the risks associated with access into and working within the quarry.

Infrequent Access

Access for activities such as pump maintenance or inspections of rock faces may consider points such as:

- no access to quarry within one week of heavy rainfall,
- establishment of defined and marked areas/paths within the quarry – such areas may have adjacent batters scaled to remove loose blocks and surface drainage established to limit the potential for water pressures to cause erosion and instability,
- limited time in the quarry and limited number of visits,
- identify areas where people may spend extended amounts of time, such as at the pump location and undertake suitable remedial measures to the slopes above to remove/limit the risk of localised instability. This may include additional survey points.

Backfilling Works Requiring Access to Pit Floor

The points discussed above are applicable. However, other considerations should also be made such as the points below.

- Only allow workers to access quarry within the cabs of trucks.
- Do not allow workers to walk around the benches, access ramps or quarry floor unnecessarily.
- Workers at the quarry floor should remain in vehicles with added protection provided to cabins.
- Contractor to establish a suitable regime of monitoring – this may be weekly, daily or continuous depending on the risks and nature of potential instability being monitored.

E7.5. Lake Level, Stratification and Pumping

E7.5.1. Lake Level

Previous discussions have recommended the quarry lake level be maintained at about RL30m where they quarry is not completely backfilled.

In order to maintain this level, a pump must be installed to remove about 2.5L/s. Further it is recommended that the maximum rise in lake level be kept of 4m, or less. This equates to a 1 in 100 year storm event and requires a pump able to discharge up to 8 L/s, or 28.8m³/hour.

E7.5.2. Stratification

Without intervention we expect the lake would stratify with an annual overturn. To balance inflows and outflows, mean pumping rates in the range of 2 to 3 L/s are required (lake level at about RL30m). Further, as discussed above, a capacity of up to 8 L/s is also required to limit lake rises during significant wet periods.

A detailed analysis is beyond the scale of this study, but experience elsewhere suggests that some excess pump capacity between the peak and typical flows above could easily be used to maintain an unstratified lake.

E7.5.3. Pump Size and Costs

Following on from the preceding sections, and considering safety, visual amenity and net positive suction head requirements a submersible pump is recommended.

For an 8L/s capacity and a total head of 80m, a 10 to 12kW unit is indicated. With a mean (24/7) pumping rate of 2 to 3 L/s for lake water balance, the duty cycle is in the range of 25% to 30%. With a destratification requirement as well, then the duty cycle is still likely to remain below 50%.

Submersibles with these capabilities are manufactured by several companies such as Davey, Grundfos and Peerless. Pumps could be slung off a pontoon located on the lake. We recommend Council consider installing a shroud to the pump and use a stainless steel motor.

For example, the DaveySS45/07 pump (6" borehole pump) would be suitable. A copy of the pump brochure is included at the end of this Appendix.

Indicative costs are:

- Pump purchase \$20,000 (includes control panel)
- 3 Phase Electrical supply \$200,000
- Running Costs \$2,000 per annum

COST SUMMARY

1. Do Nothing to Quarry

a.	Maintain lake at RL30m – purchase & install pump and establish 3 phase electricity supply	\$220,000
b.	Annual pumping costs	\$2000 p.a.
c.	Access to quarry floor For infrequent access/use	\$200,000
TOTAL (50 years)		≈\$0.5 million

2. Backfill Works

New Fill Purchased for use as a dump quarry backfill

a.	Stabilise entire quarry - remove all instability issues	>\$60 million
b.	Stabilise quarry and north batter	>\$55 million
c.	Stabilise north batter only	\$38 to 43 million
d.	Move fill from Parts 2 and 4 into quarry	\$1.85 and 1.5 million respectively
e.	Access to quarry floor For backfilling operations	\$320,000

3. Mechanical Stabilisation

a.	All quarry faces	\$6.5 million
b.	Design, Inspection and Certification	\$0.3 million
c.	Drainage works	\$0.75 million
d.	Contingency (set at 30%)	\$2.3 million Approximately

TOTAL \$10 million

4. Cut Back Existing Batters and Backfill Quarry

New dumped fill sourced from battering back of existing batters. Fill placed in void to about RL56m in a balanced cut/fill operation.

a.	Fills, residual and highly weathered rock	\$1 million
b.	SW/FR breccia – pre spilt with drill and blast	\$4.5 million
c.	Access to quarry floor For backfilling operations	\$320,000
d.	Drainage works	\$0.5 million
e.	Maintain lake at about 10m depth – purchase & install pump and establish 3 phase electricity supply	\$220,000
f.	Annual pumping costs	\$2000 p.a.
	TOTAL	say \$6.6 million

5. Monitoring

We would recommend these costs be included with any of the options above from Do nothing to supporting the quarry. Total ongoing costs will depend on the option selected and results found.

Establishment

a.	Establish Survey	\$10,000
b.	Establish Rainfall	\$5,000
c.	Establish Lake Level Pole	\$2,000
	Total	<u>\$17,000</u>

Ongoing Measurements (initial 12 month period)

d.	Ongoing Survey	\$10,000
e.	Ongoing Rainfall	\$1,000
f.	Ongoing Lake Level	\$1,000
g.	Ongoing Face Inspections	\$10,000
h.	Ongoing Piezometer	\$2,000
	Total	<u>\$24,000</u>
	OVERALL TOTAL COSTS	<u>\$41,000</u>

ATTACHMENT E1

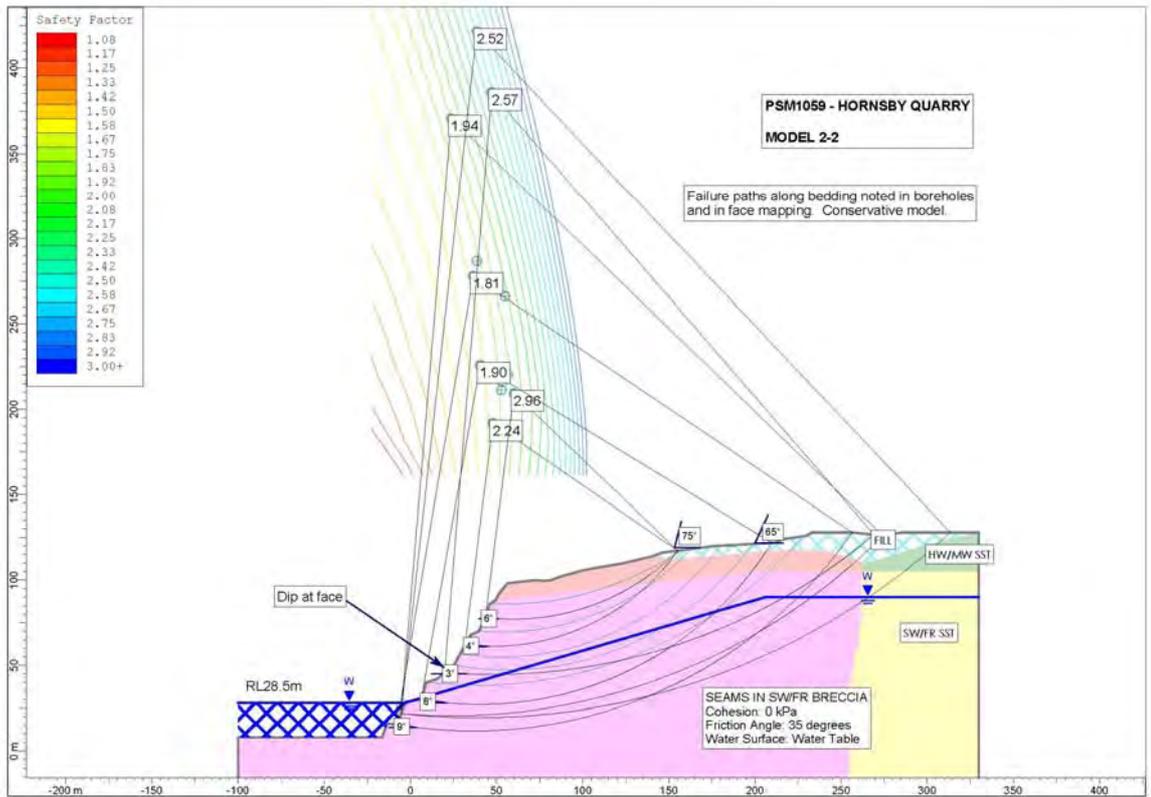


Figure 1 – Model 2-2 Domain E1

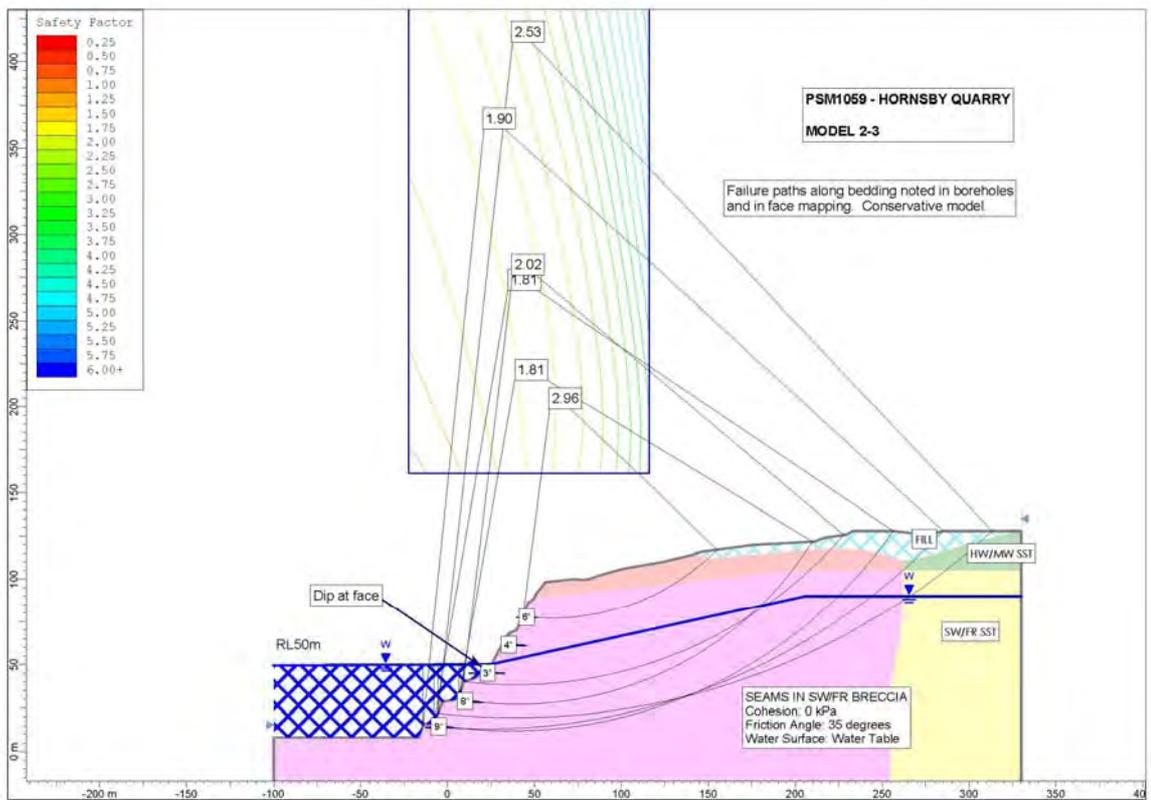


Figure 2 – Model 2-3 Domain E1

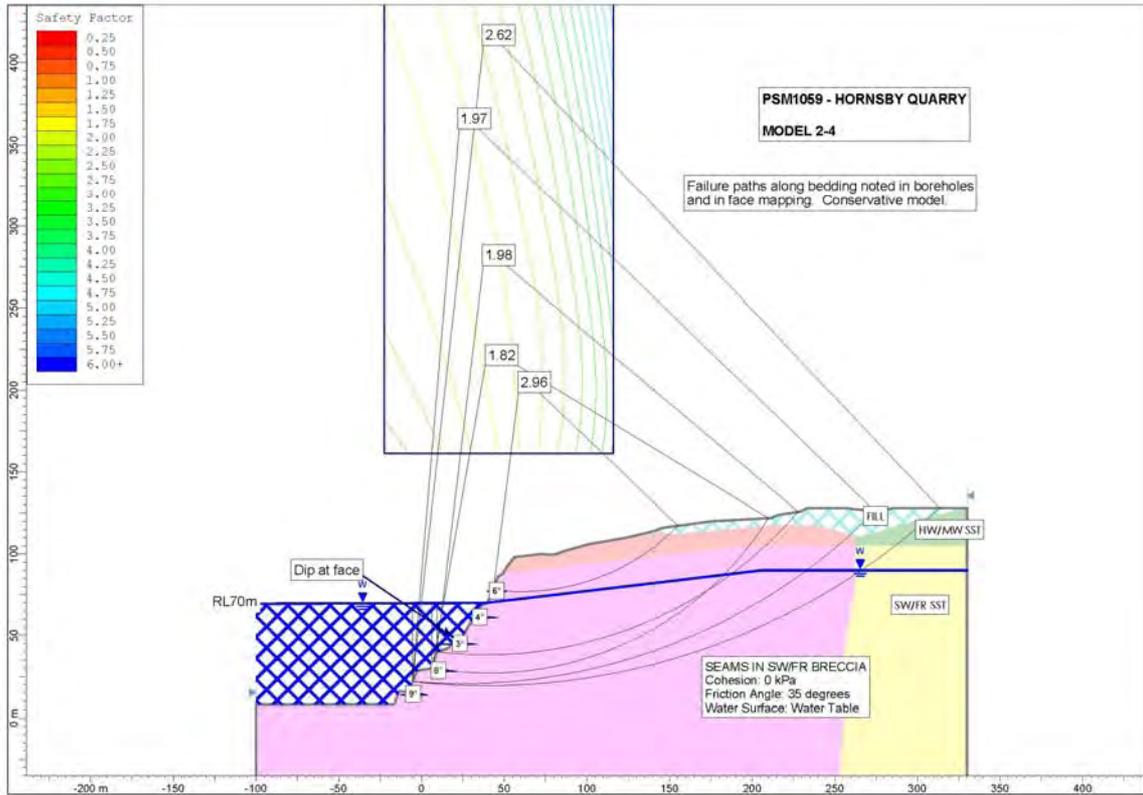


Figure 3 – Model 2-4 Domain E1

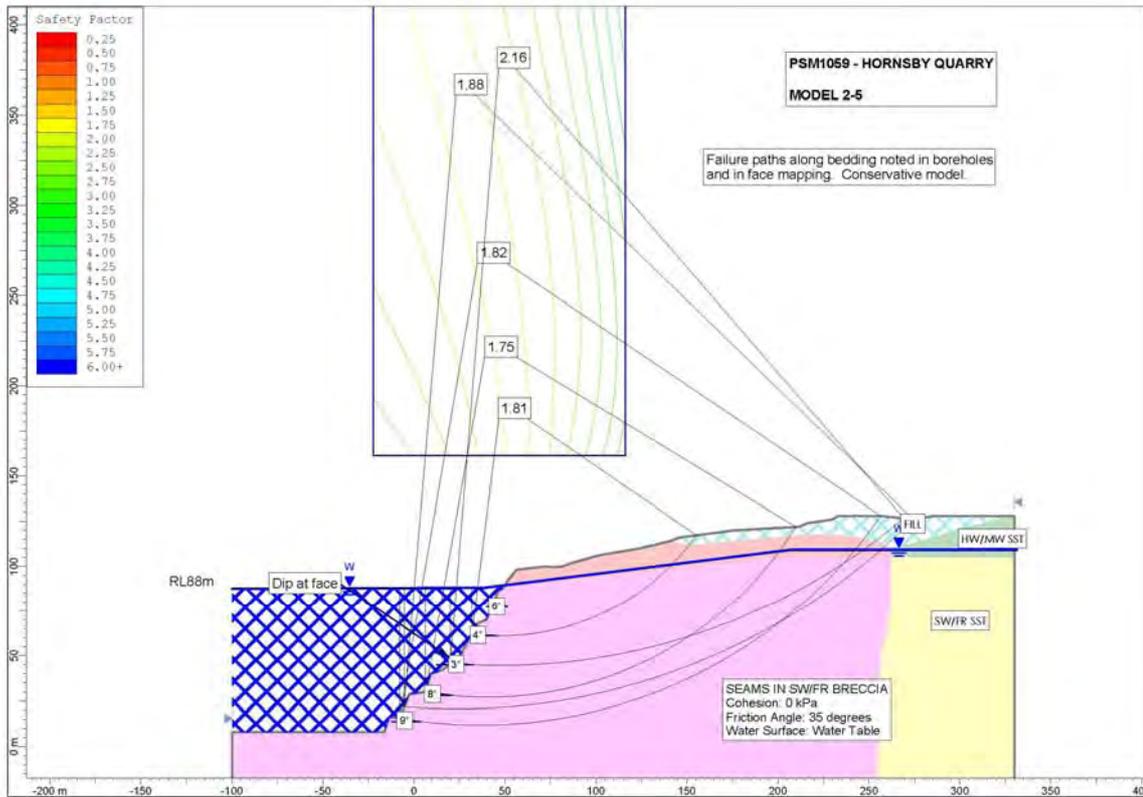


Figure 4 – Model 2-5 Domain E1

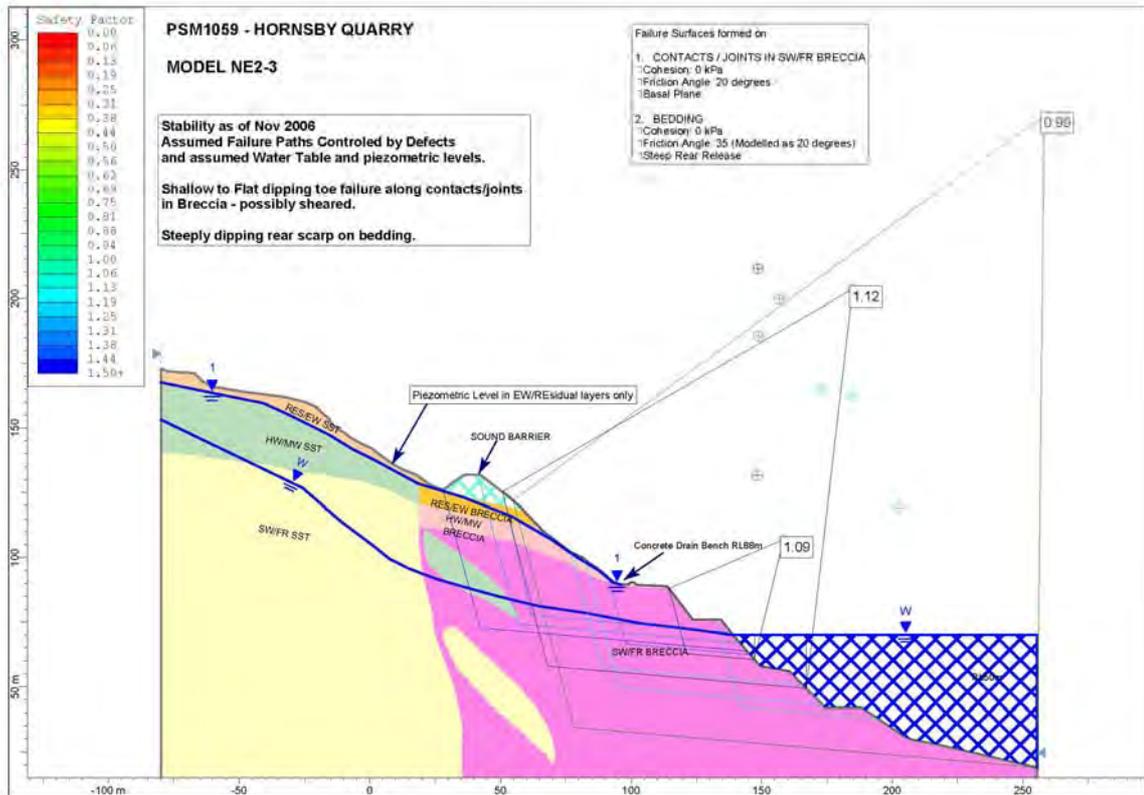


Figure 7 – Model NE2-3 Domains NE1 and NE2

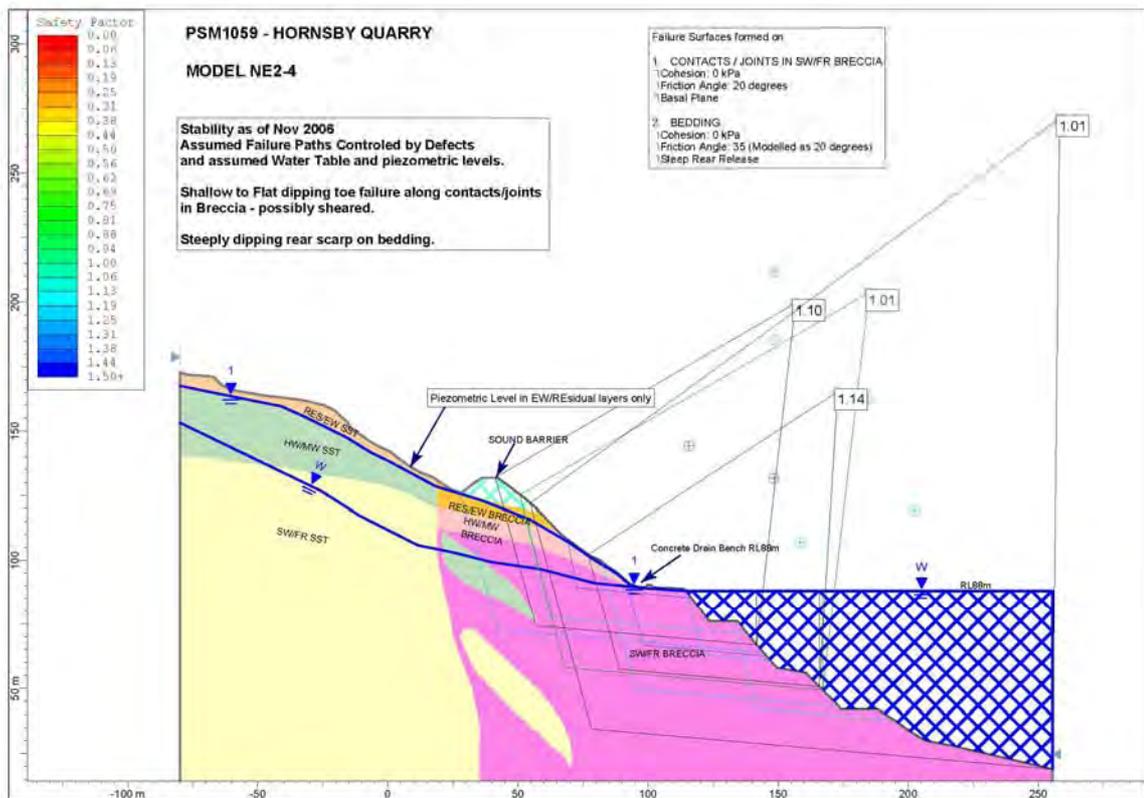


Figure 8 – Model NE2-4 Domains NE1 and NE2

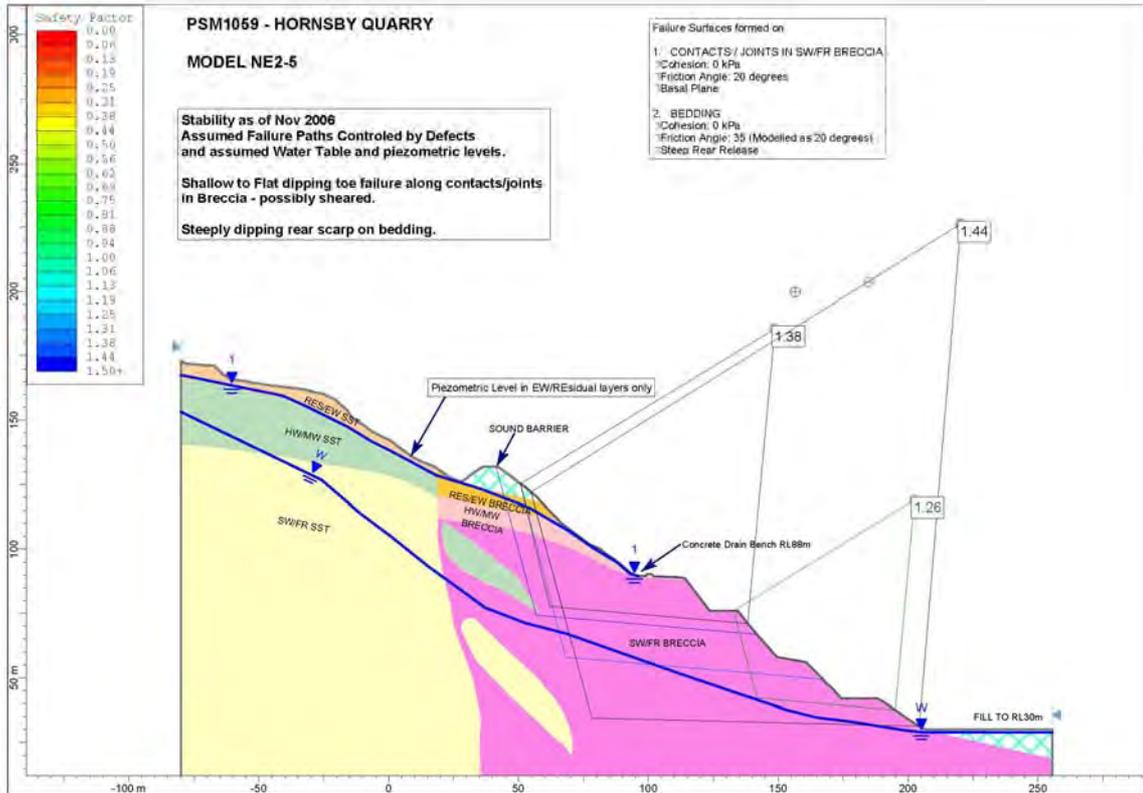


Figure 9 – Model NE2-5 Domains NE1 and NE2

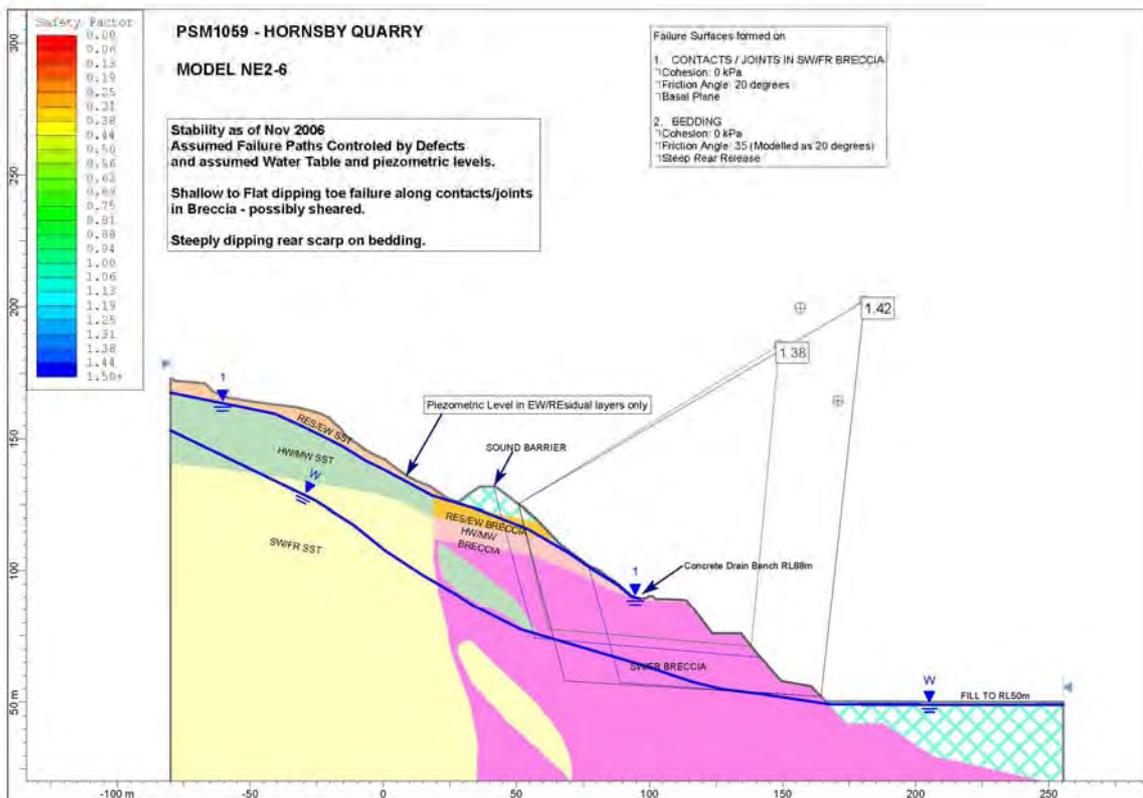


Figure 10 – Model NE2-6 Domains NE1 and NE2

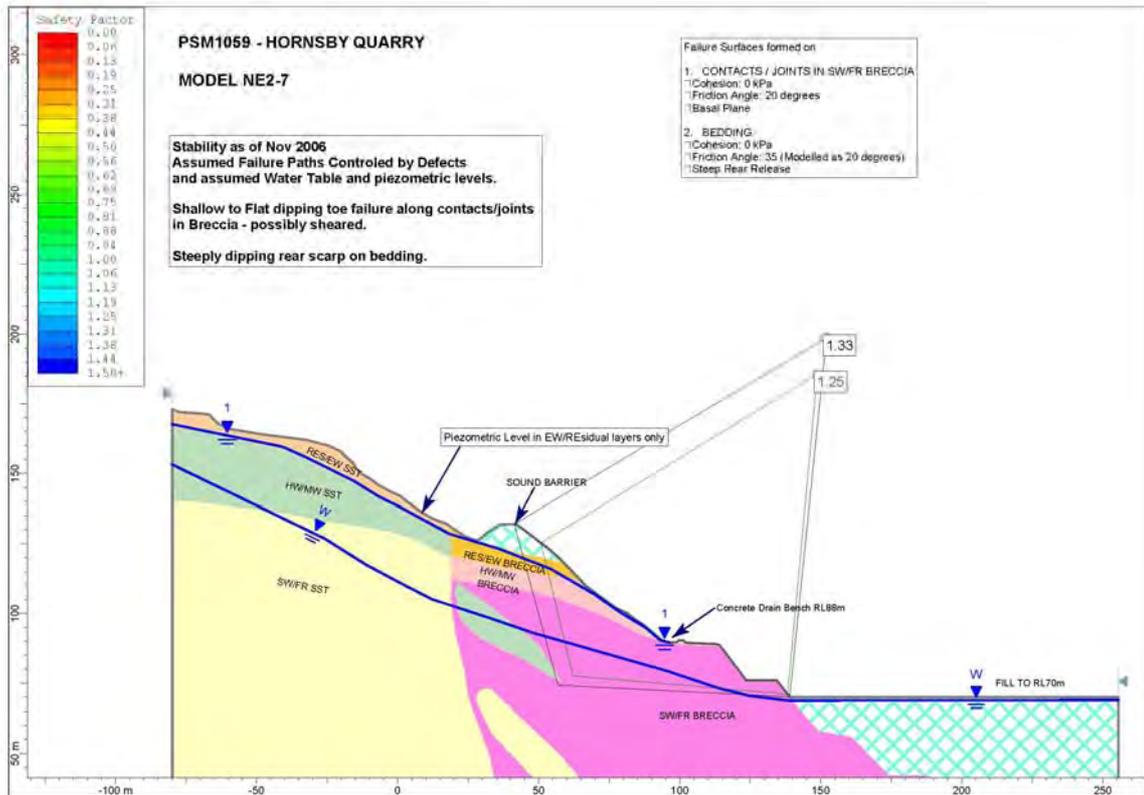


Figure 11 – Model NE2-7 Domains NE1 and NE2

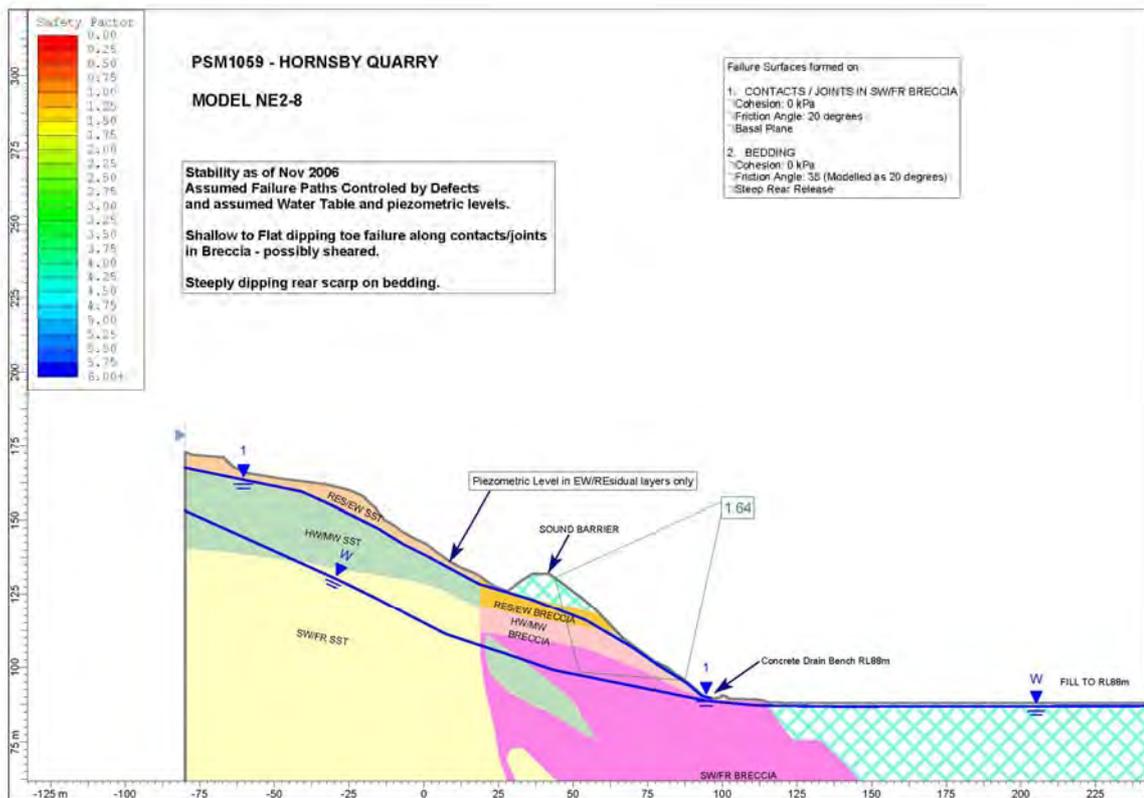


Figure 12 – Model NE2-8 Domains NE1 and NE2

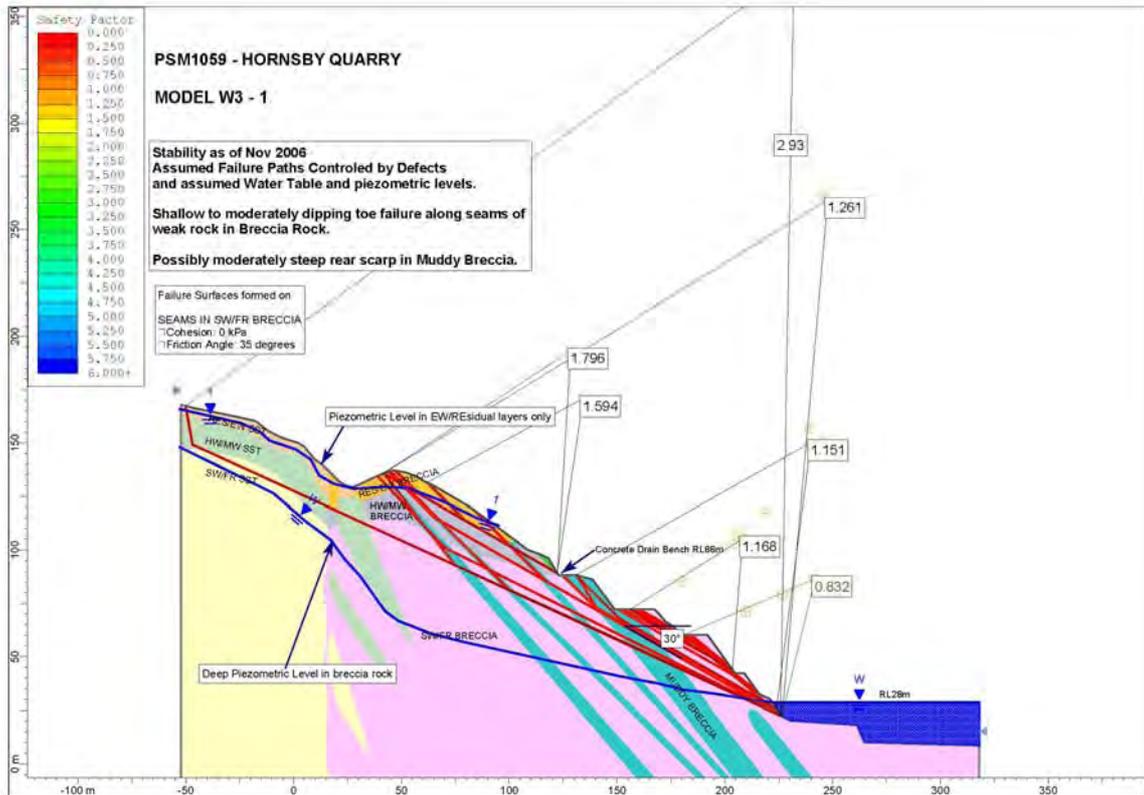


Figure 13 – Model W3-1 Domain W3

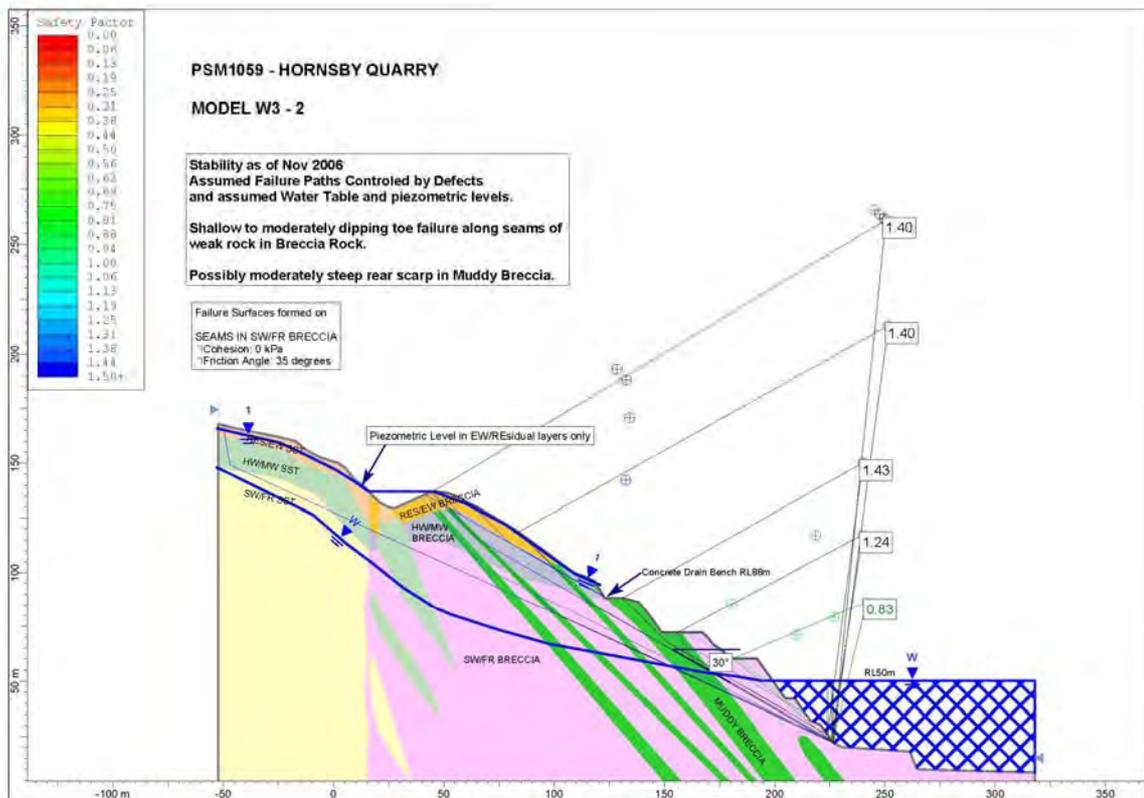


Figure 14 – Model W3-2 Domain W3

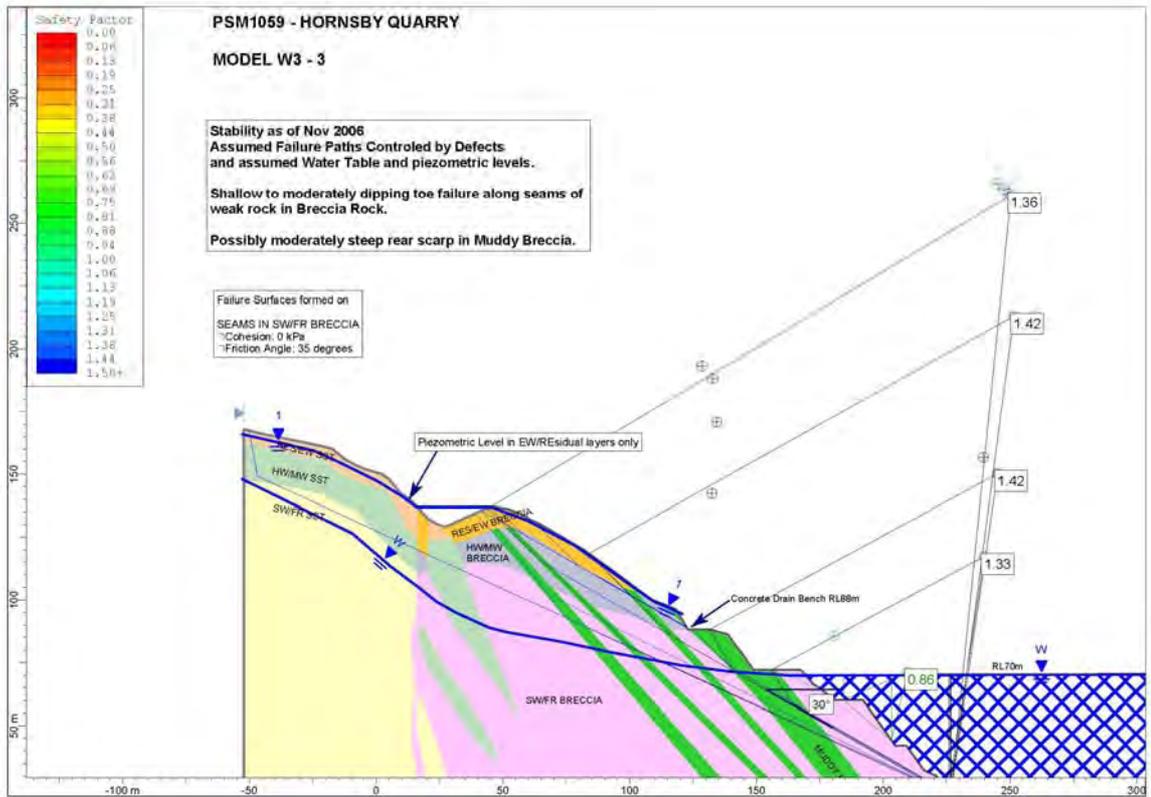


Figure 15 – Model W3-3 Domain W3

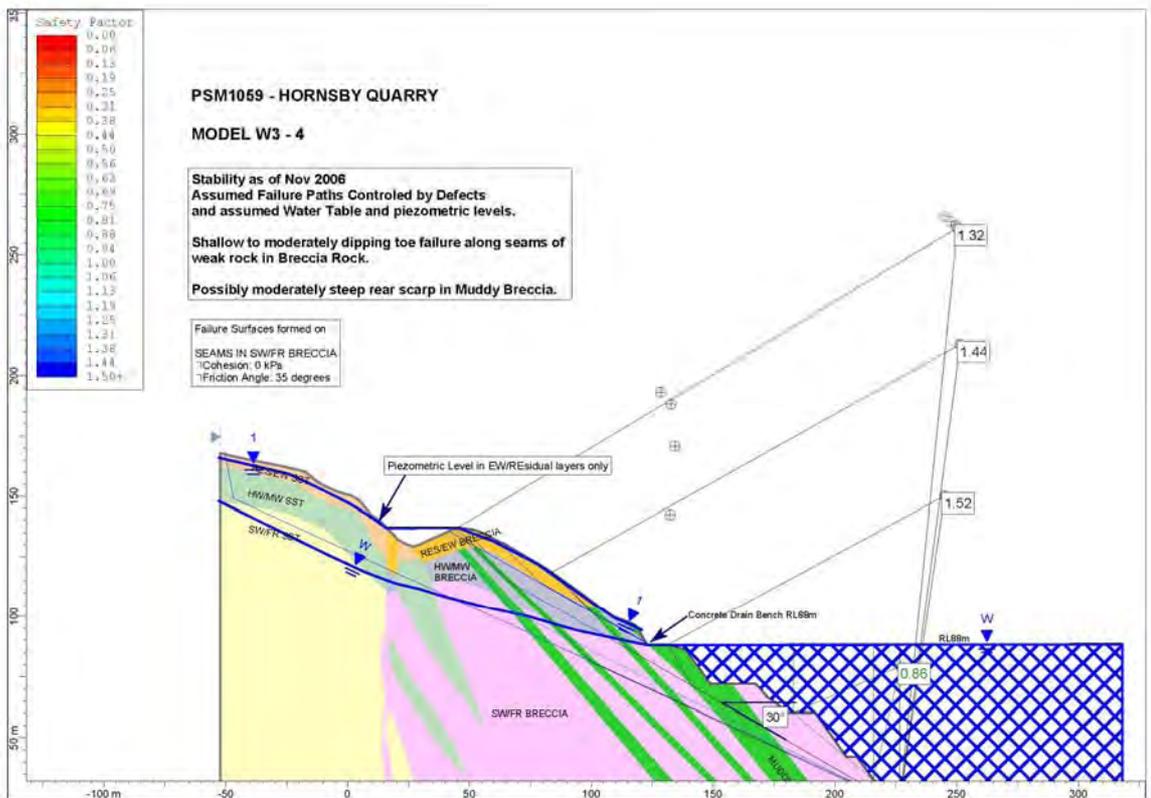


Figure 16 – Model W3-4 Domain W3

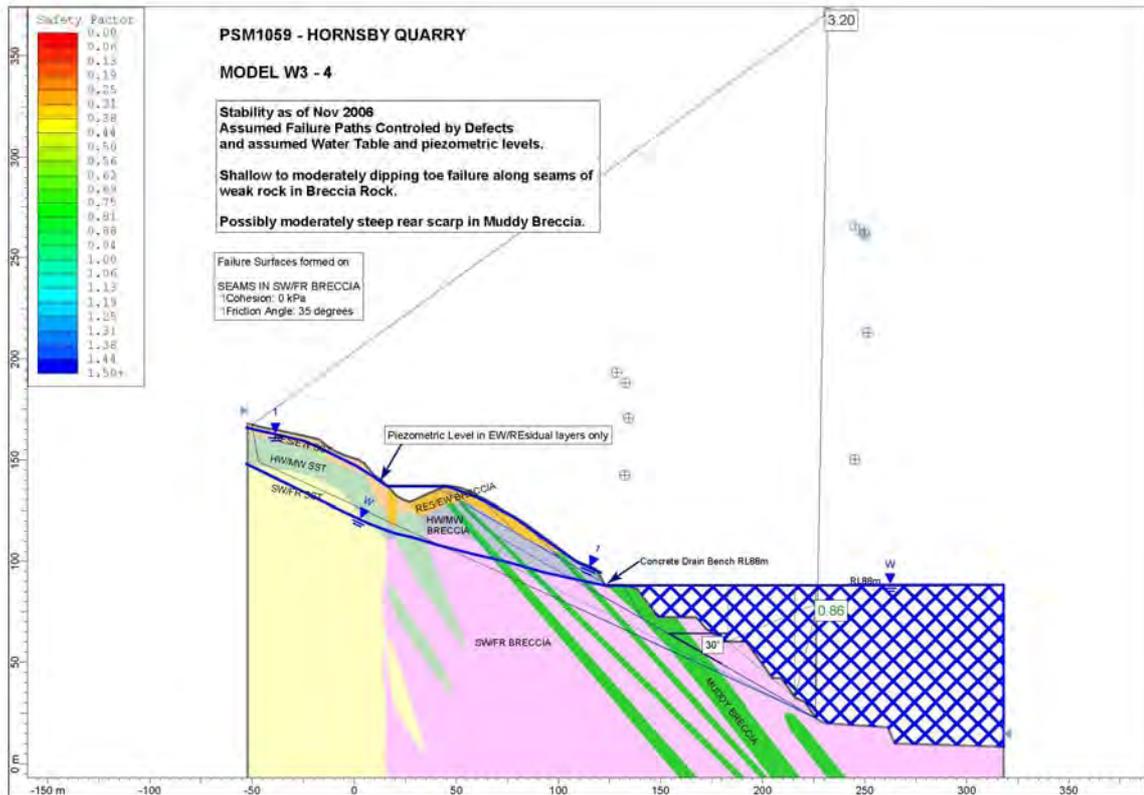


Figure 17 – Model W3-4 – To Houses, Domain W3

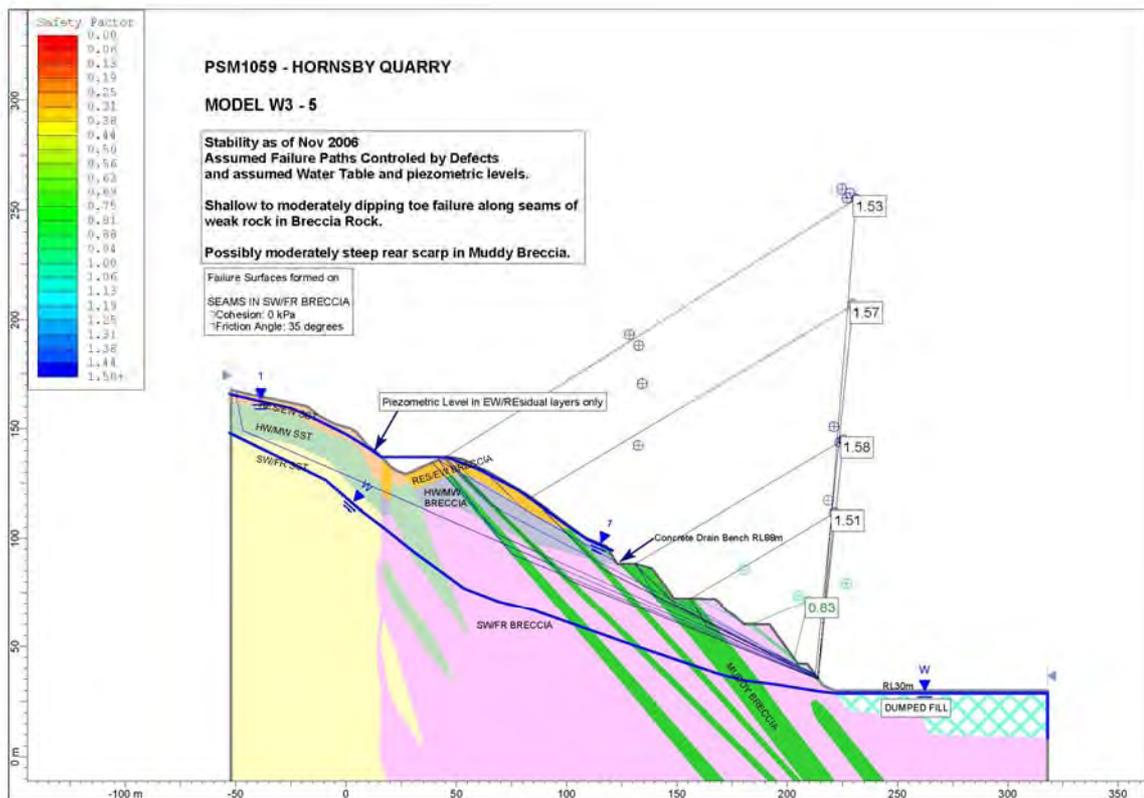


Figure 18 – Model W3-5 Domain W3

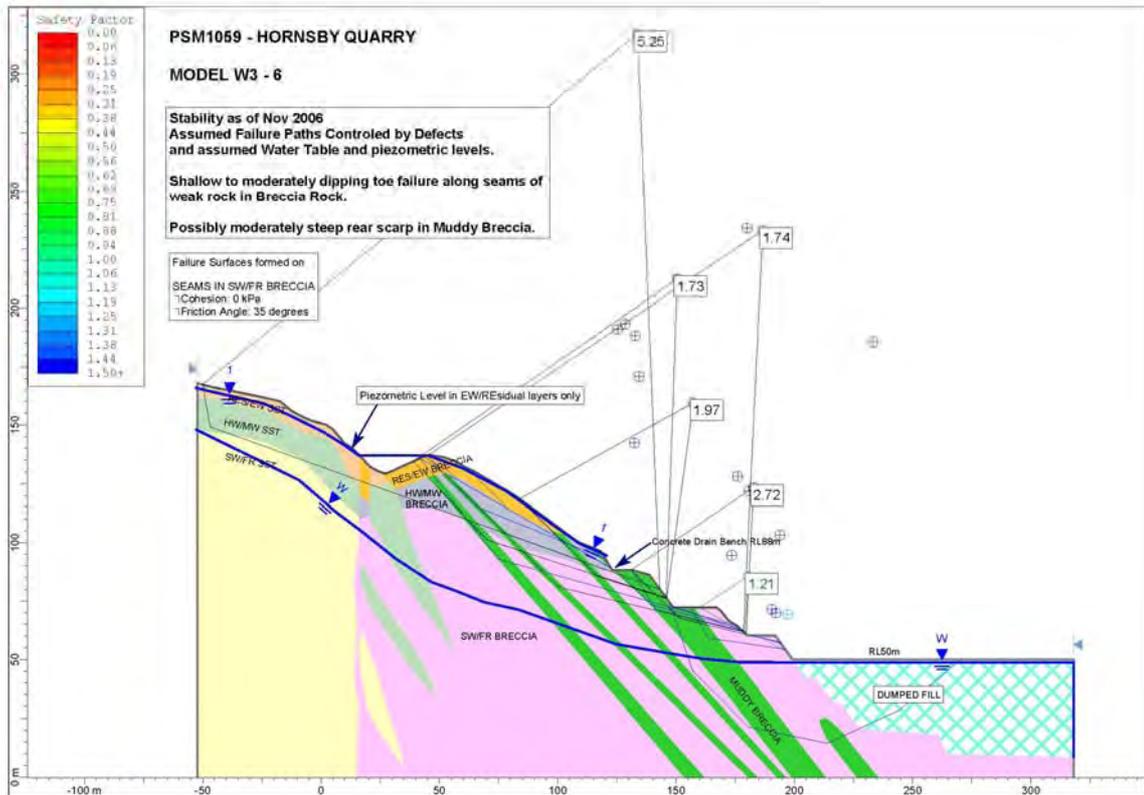


Figure 19 – Model W3-6 Domain W3

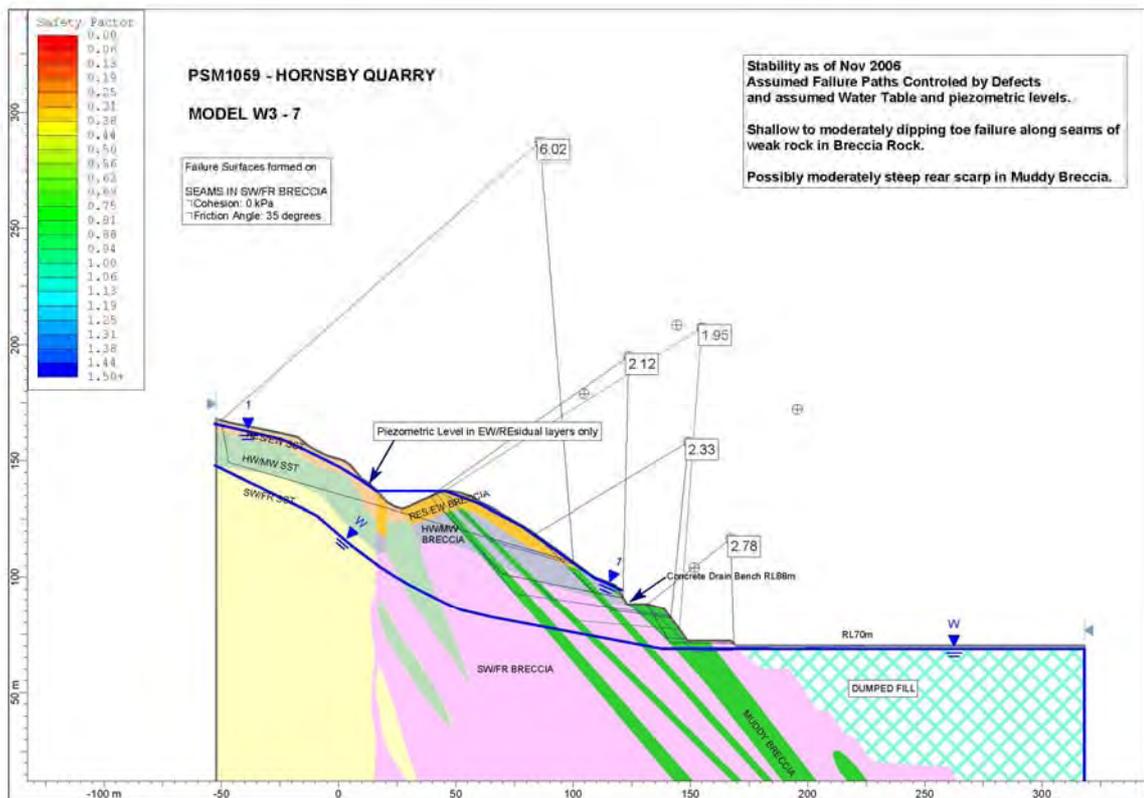


Figure 20 – Model W3-7 Domain W3

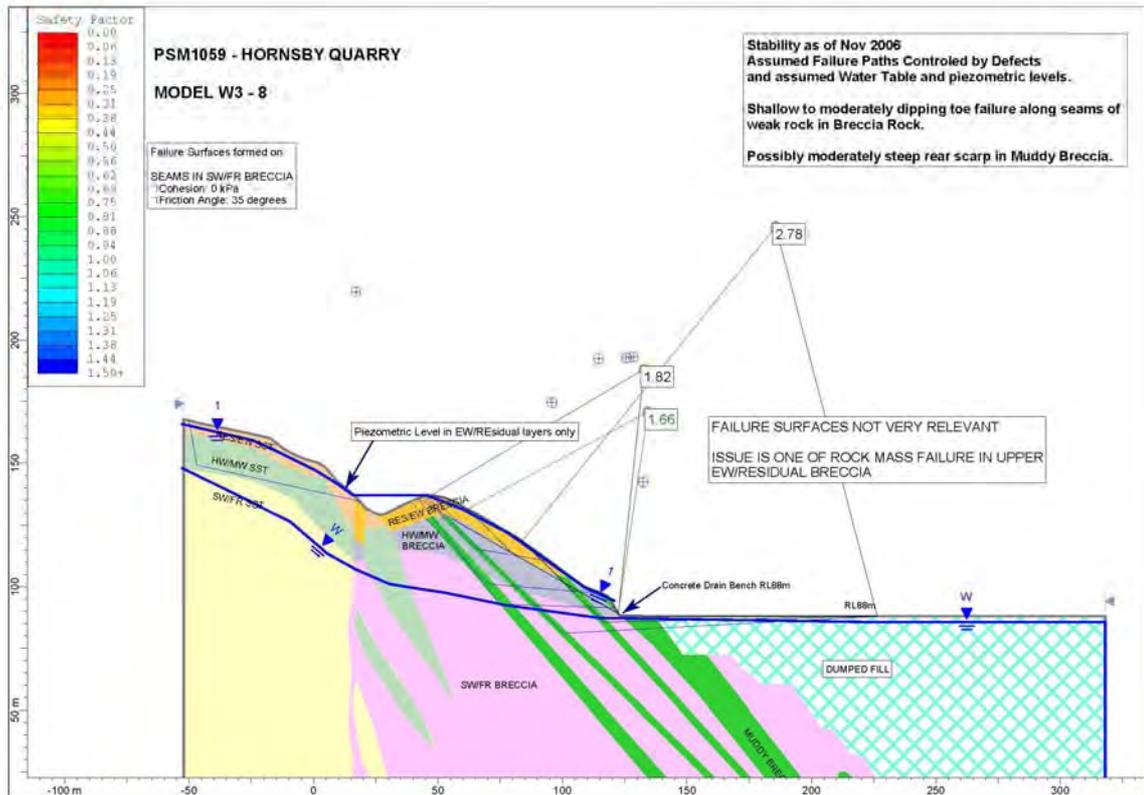


Figure 21 – Model W3-8 Domain W3

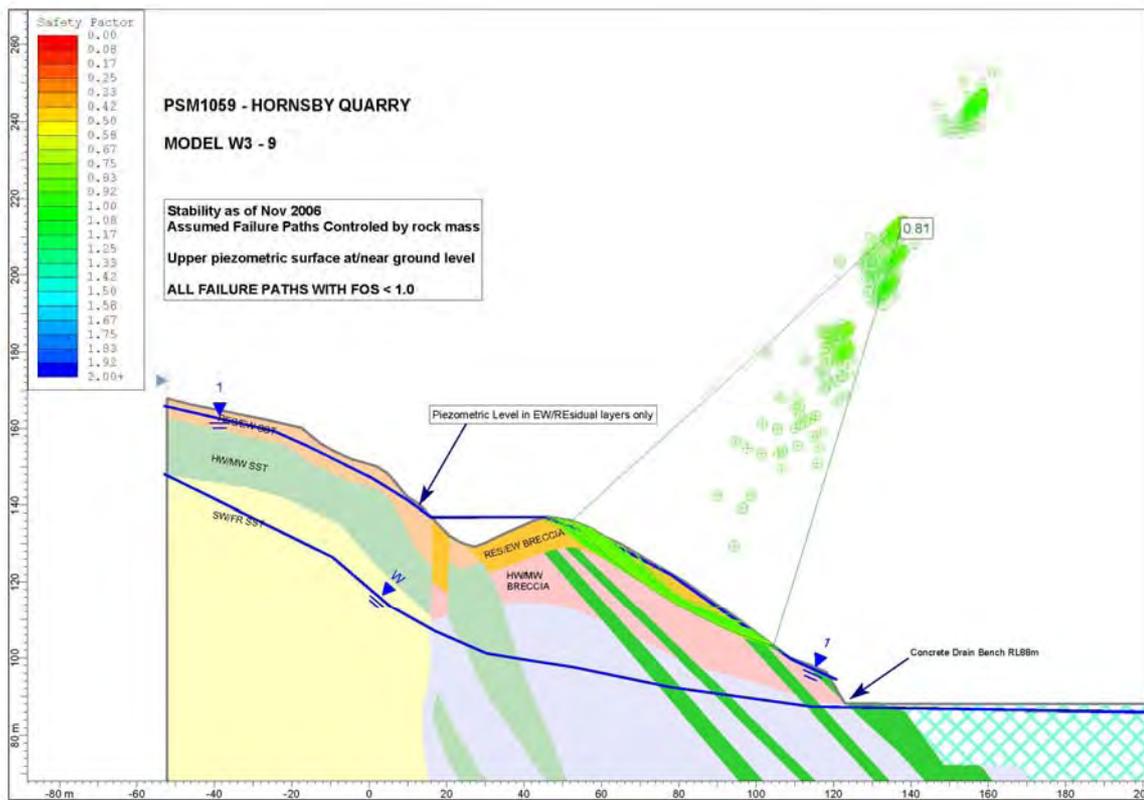


Figure 22 – Model W3-9 Domain W3

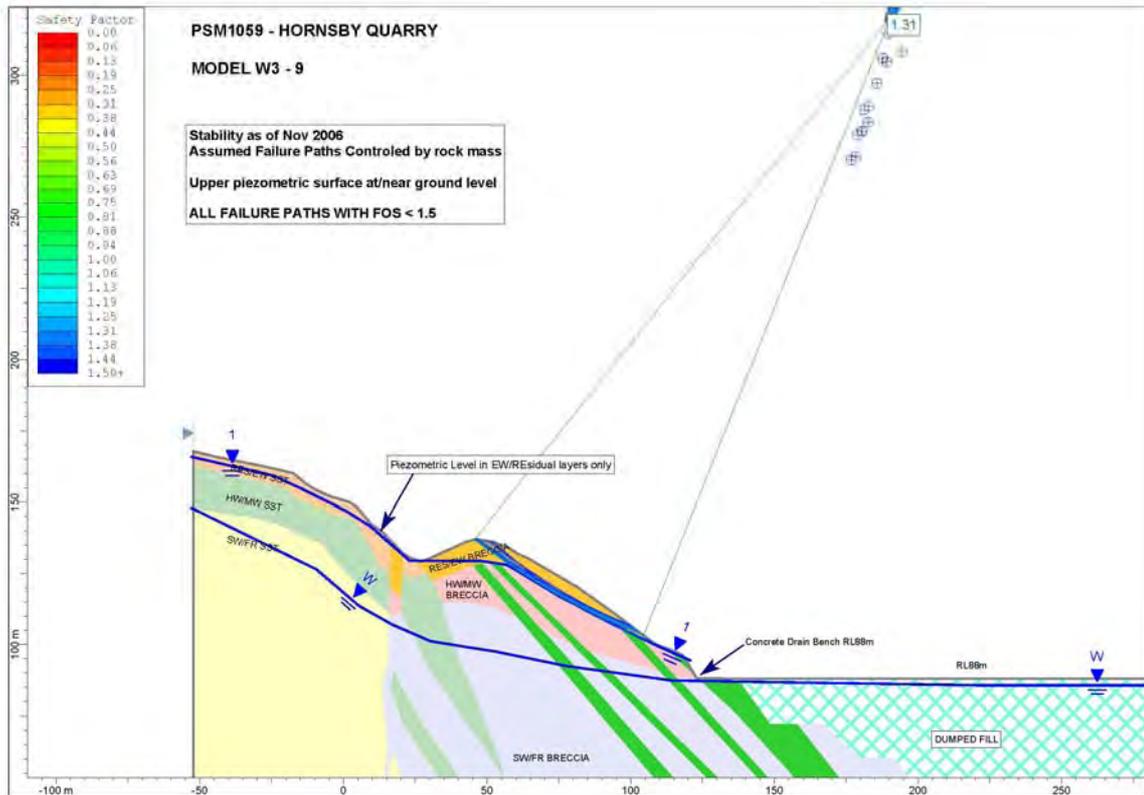


Figure 23 – Model W3-10 Domain W3

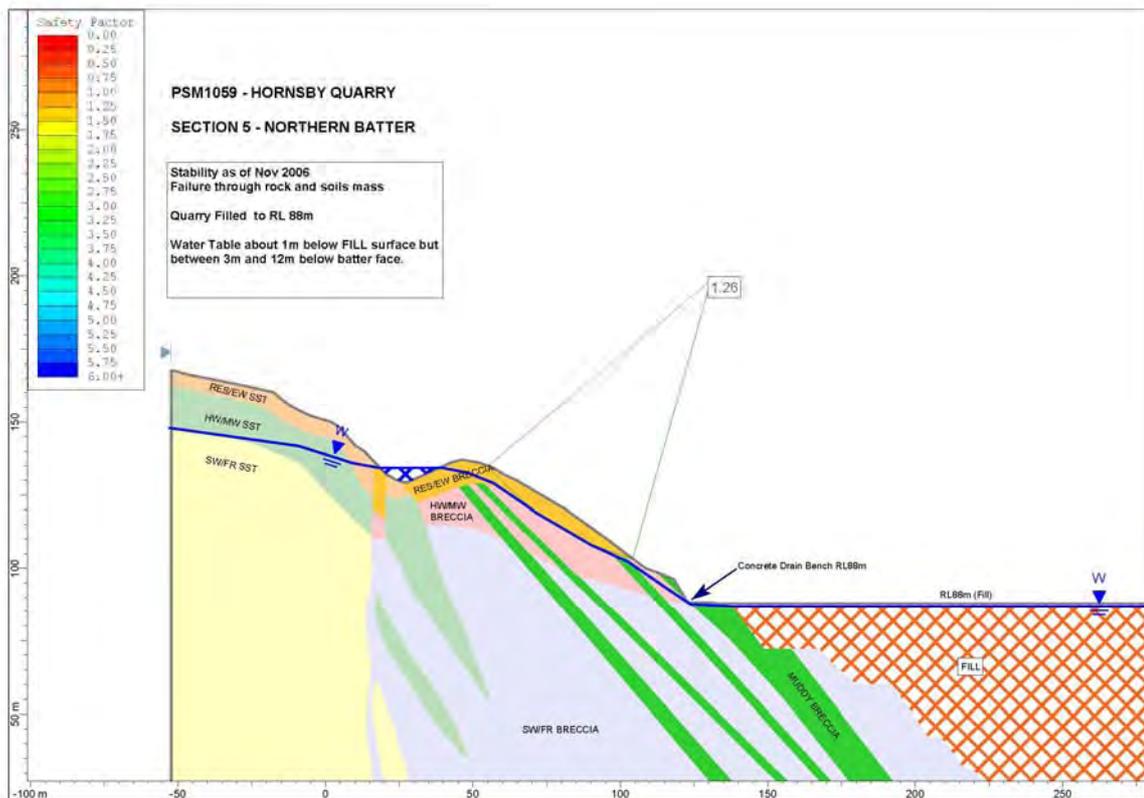


Figure 24 – Section 5 Northern Batter – Mass Failure in Upper Slope Domain W3

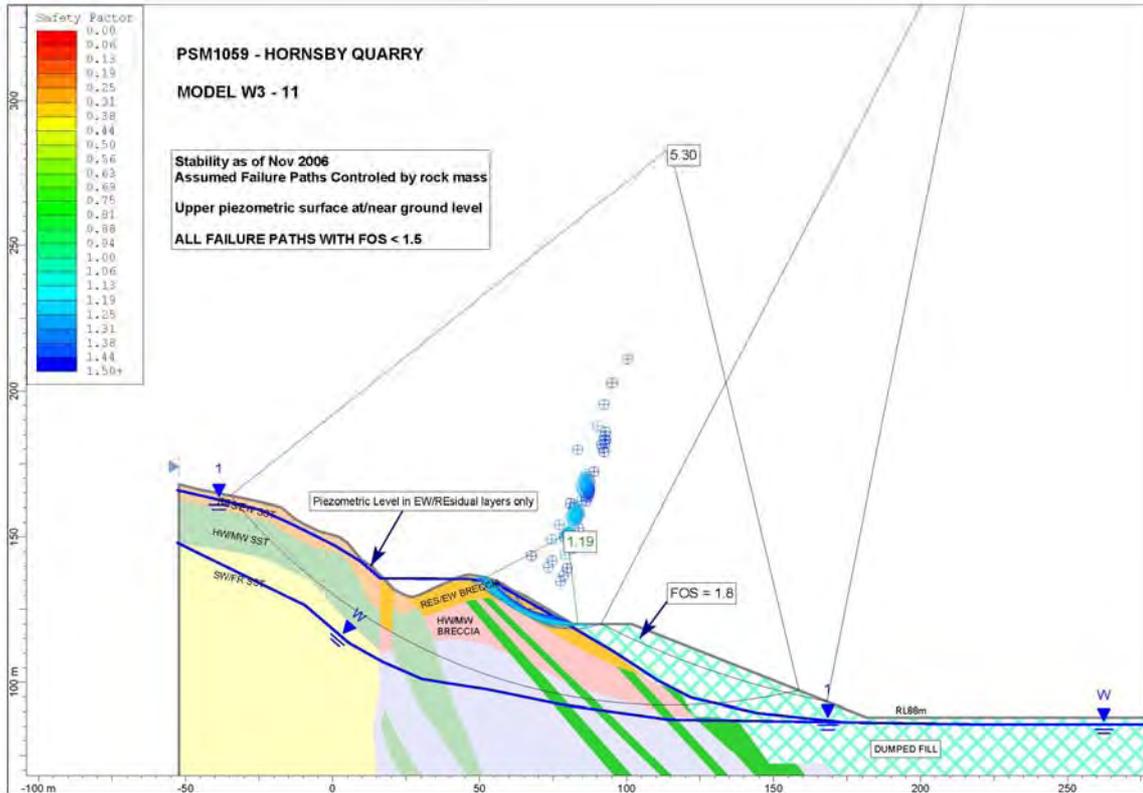


Figure 25 – Model W3-11 Domain W3

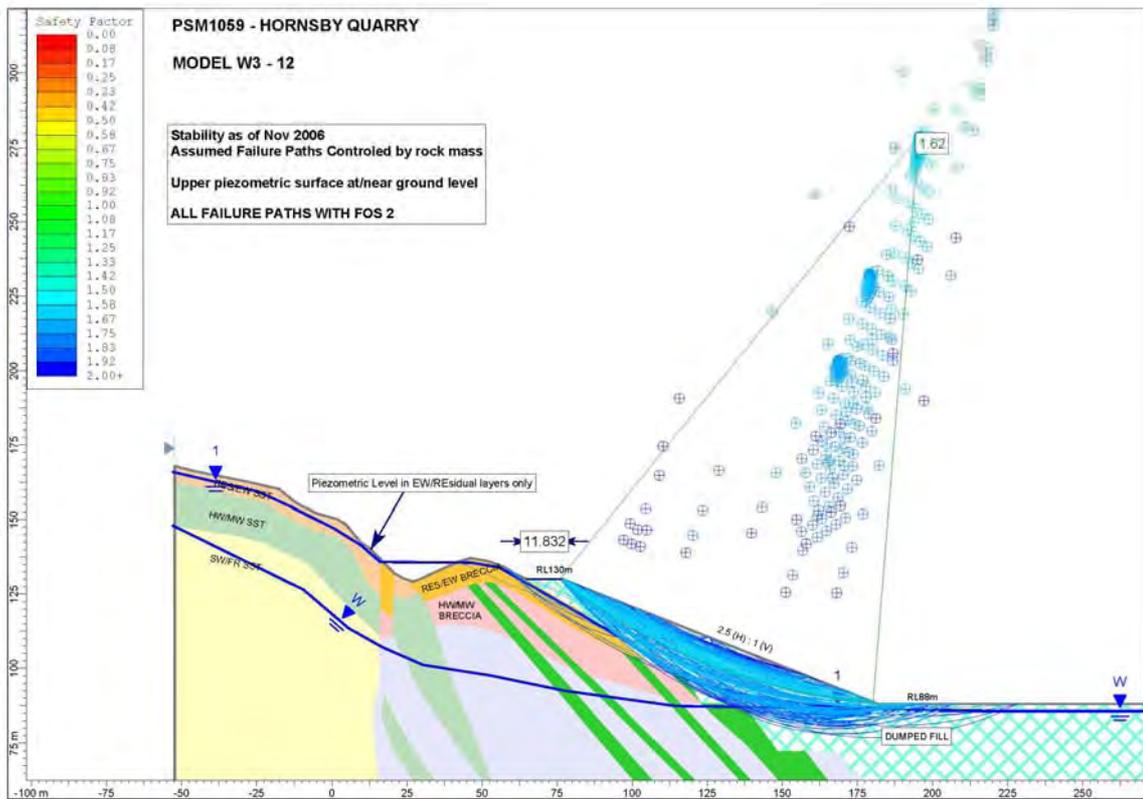


Figure 26 – Model W3-12 Domain W3

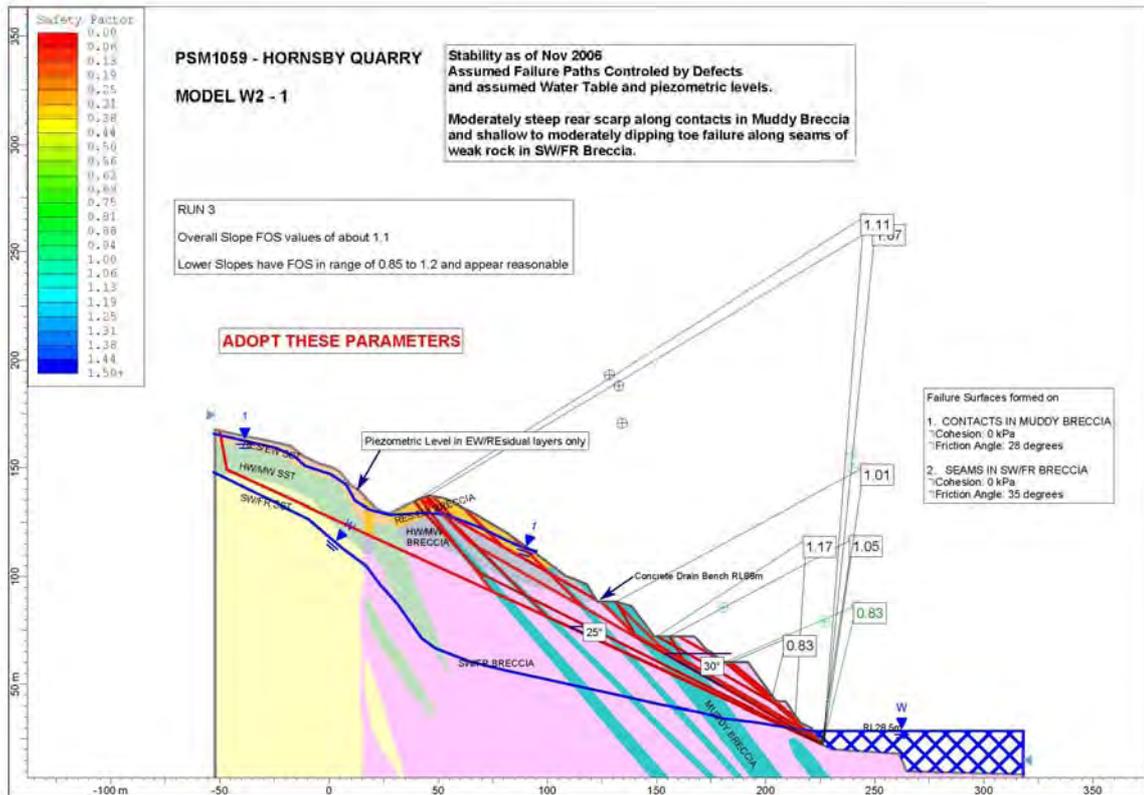


Figure 27 – Model 2-1 Domain W2

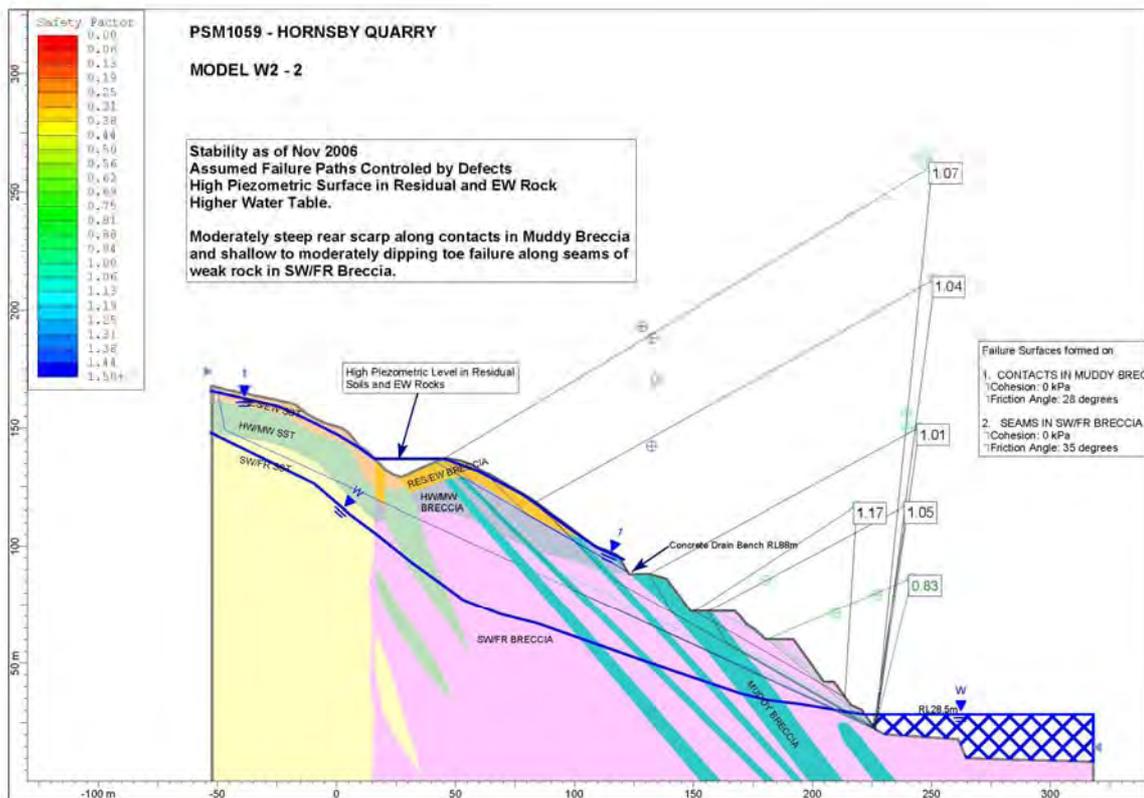


Figure 28 – Model 2-2 Domain W2

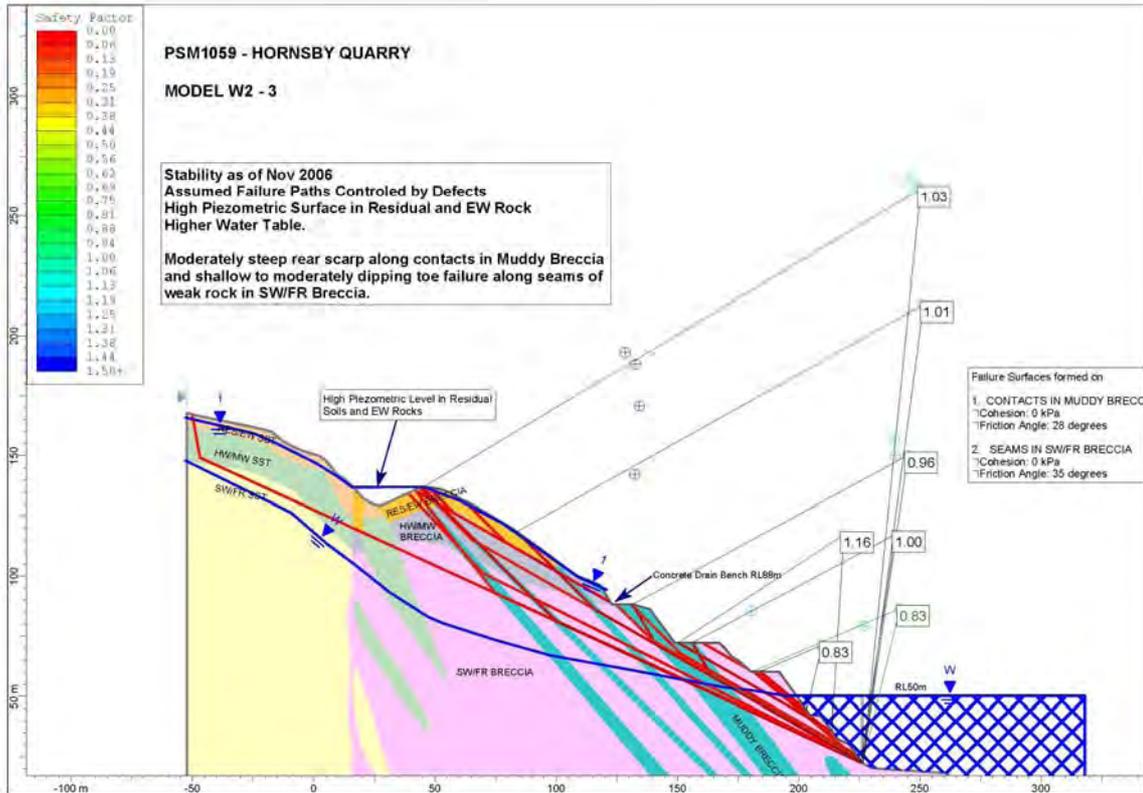


Figure 29 – Model 2-3 Domain W2

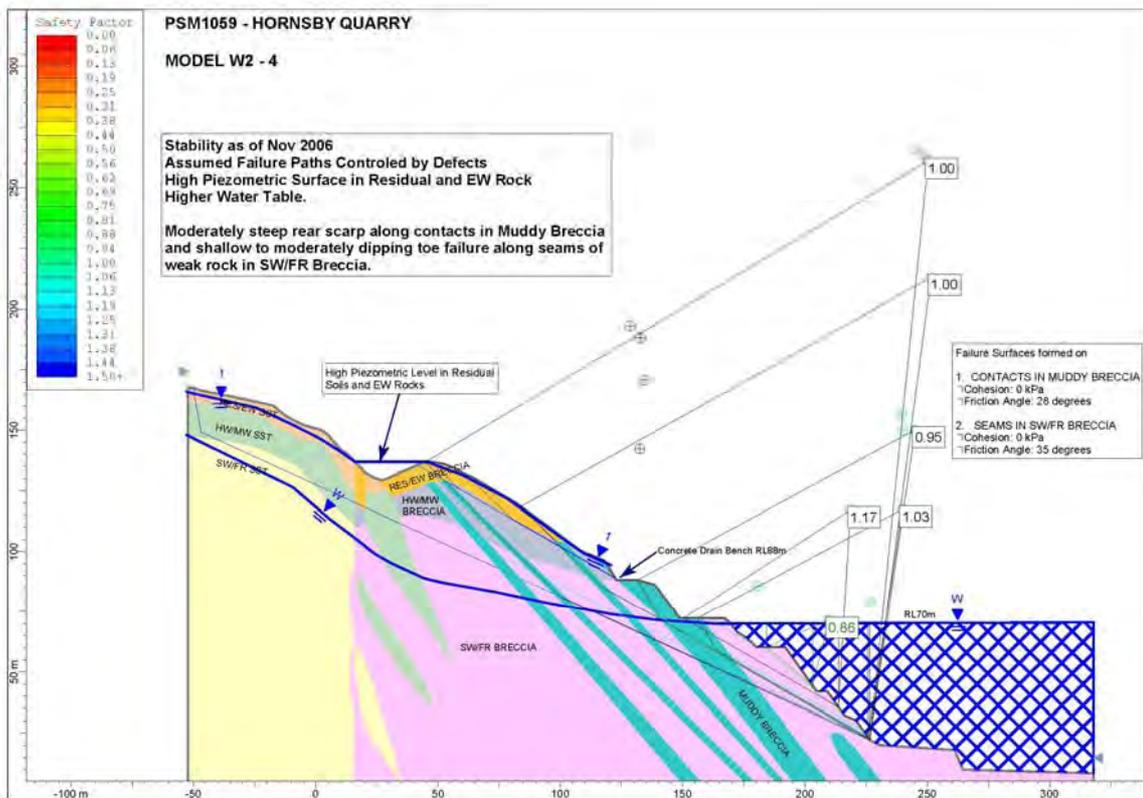


Figure 30 – Model 2-4 Domain W2

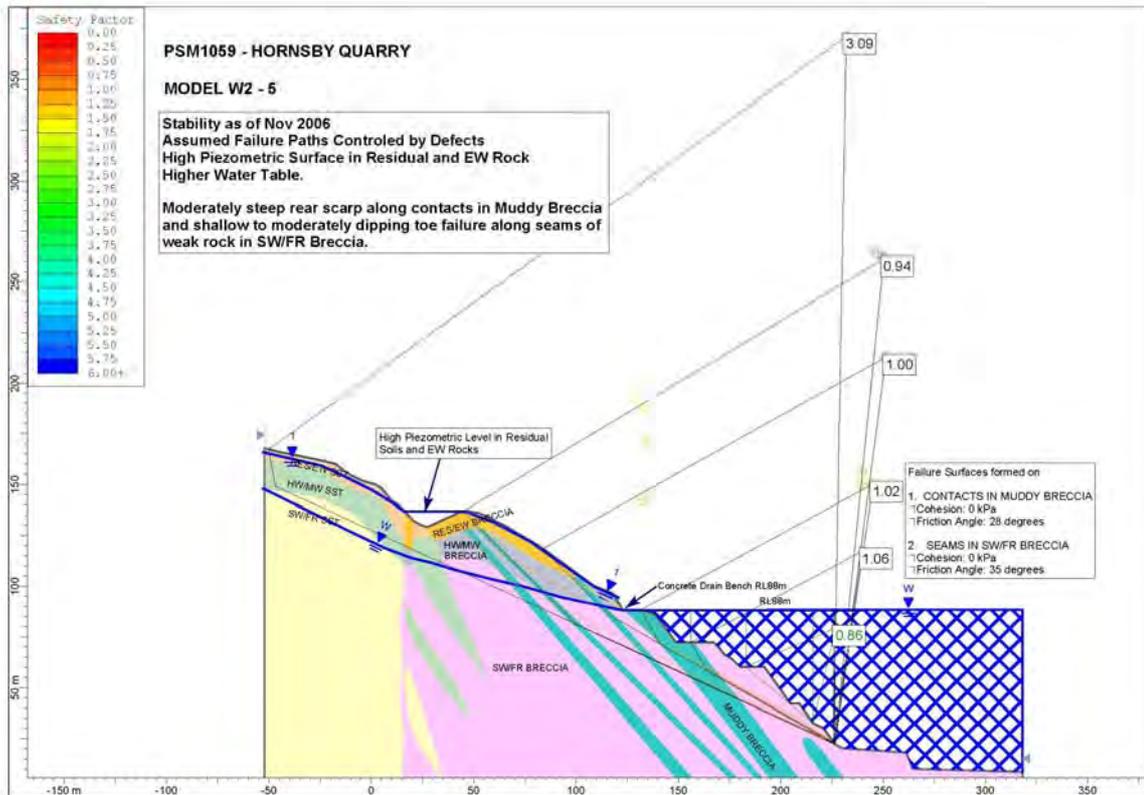


Figure 31 – Model 2-5 Domain W2

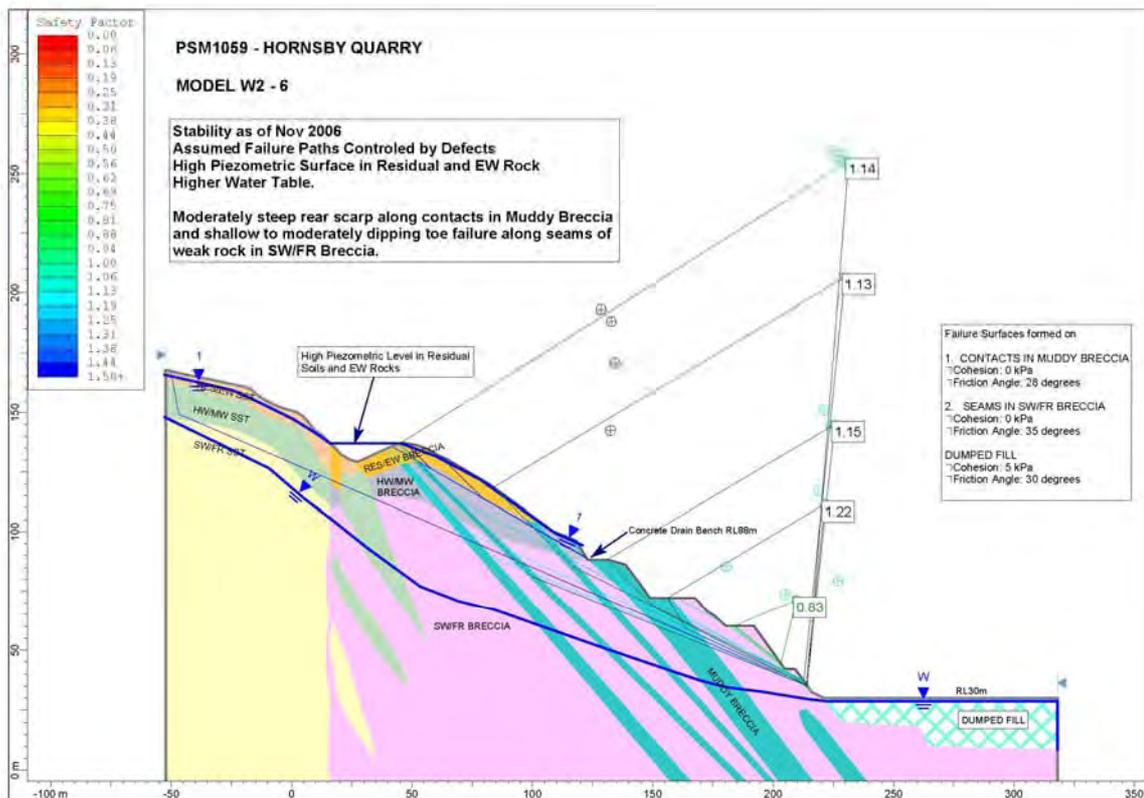


Figure 32 – Model 2-6 Domain W2

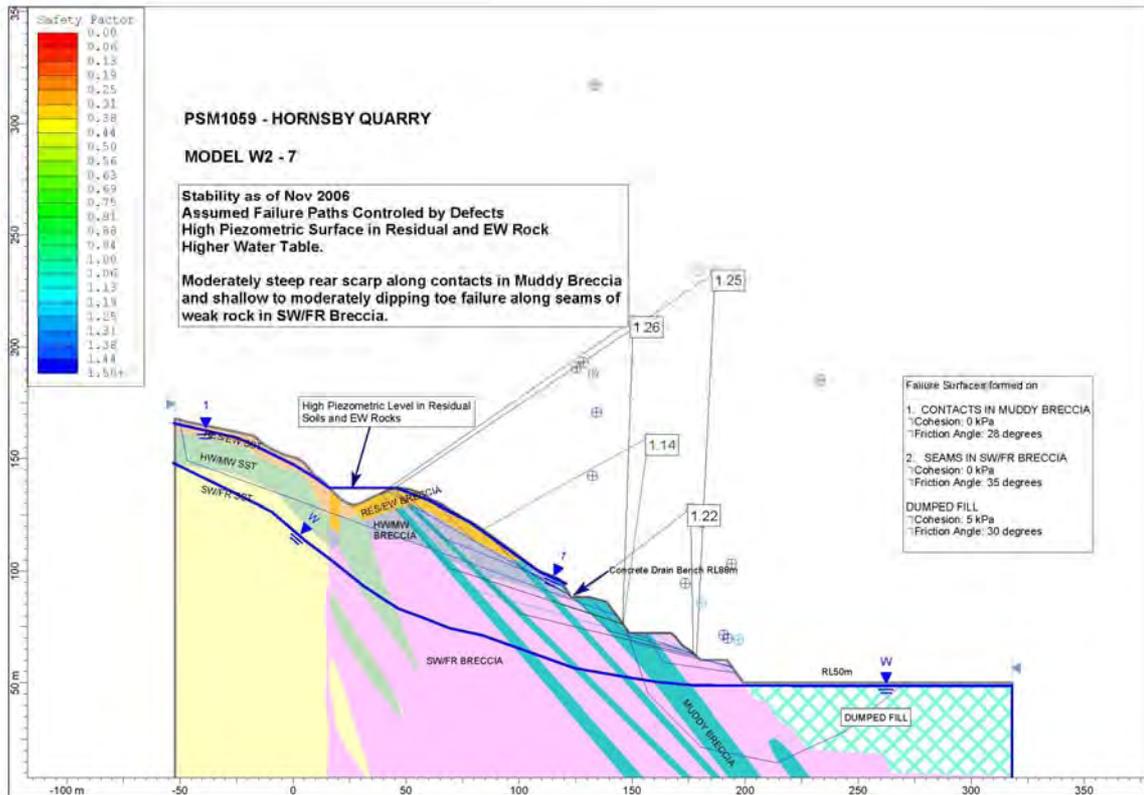


Figure 33 – Model 2-7 Domain W2

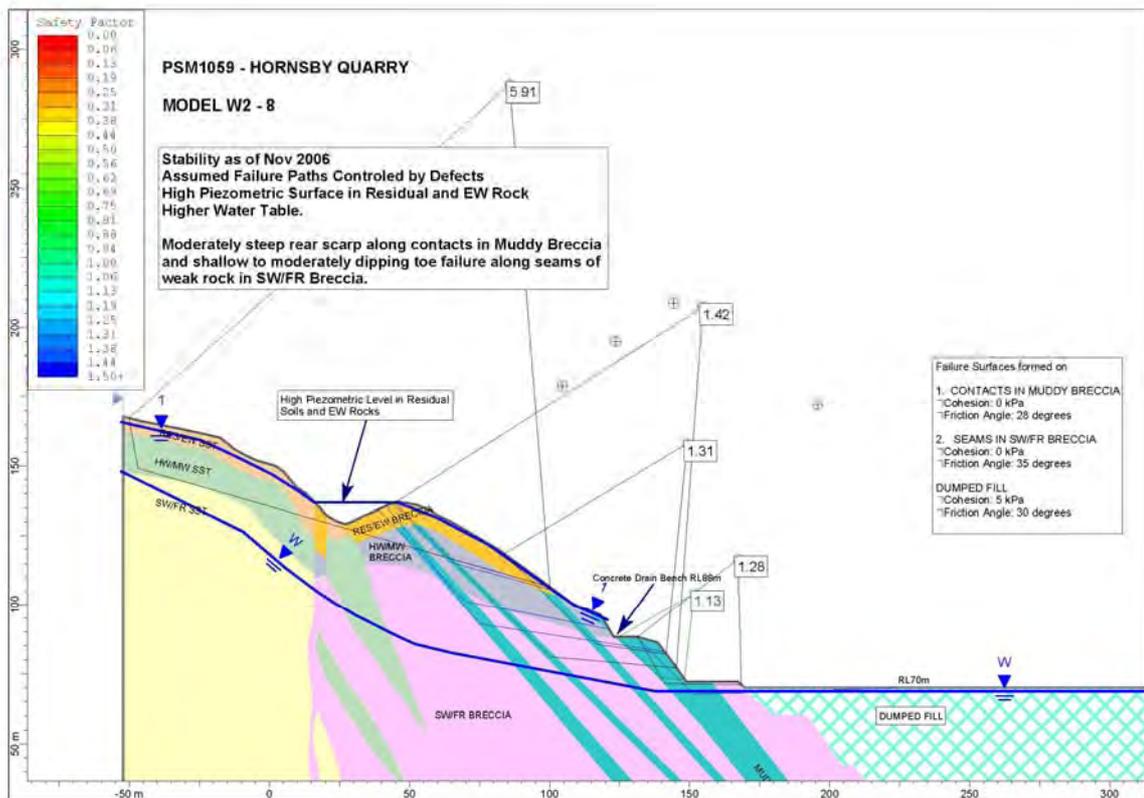


Figure 34 – Model 2-8 Domain W2

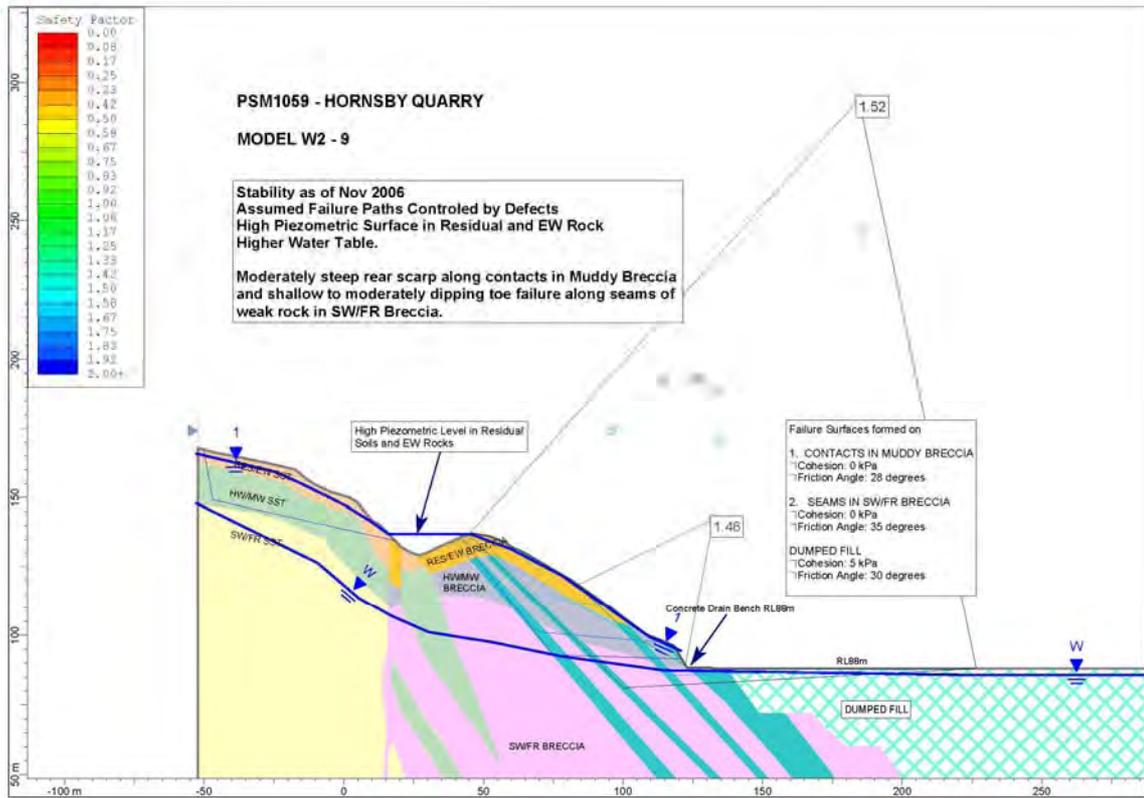


Figure 35 – Model 2-9 Domain W2

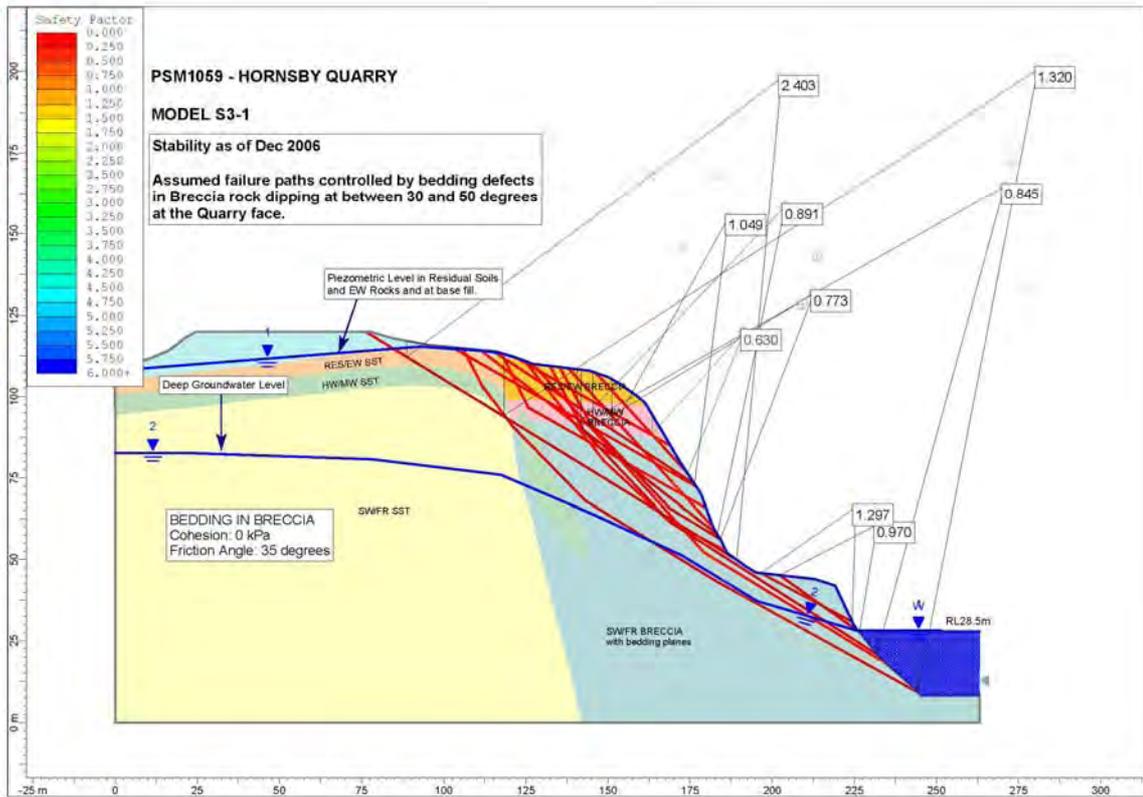


Figure 36 – Model S3-1 Domain S3

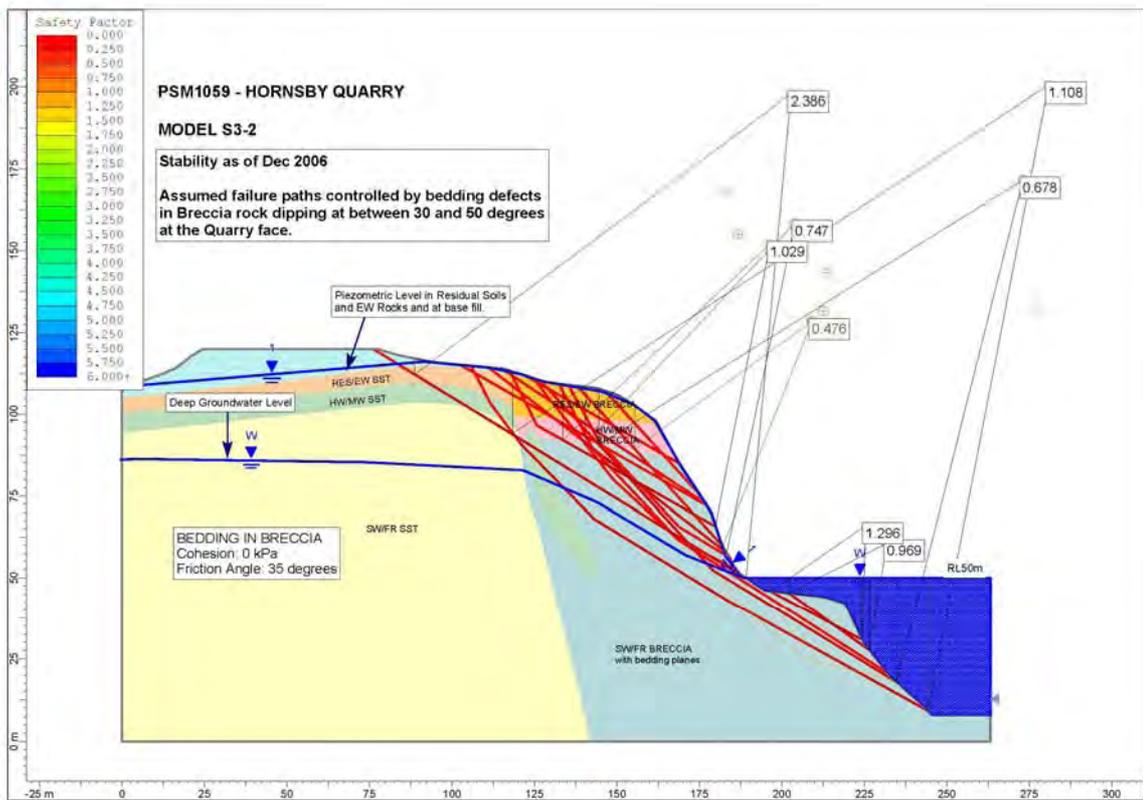


Figure 37 – Model S3-2 Domain S3

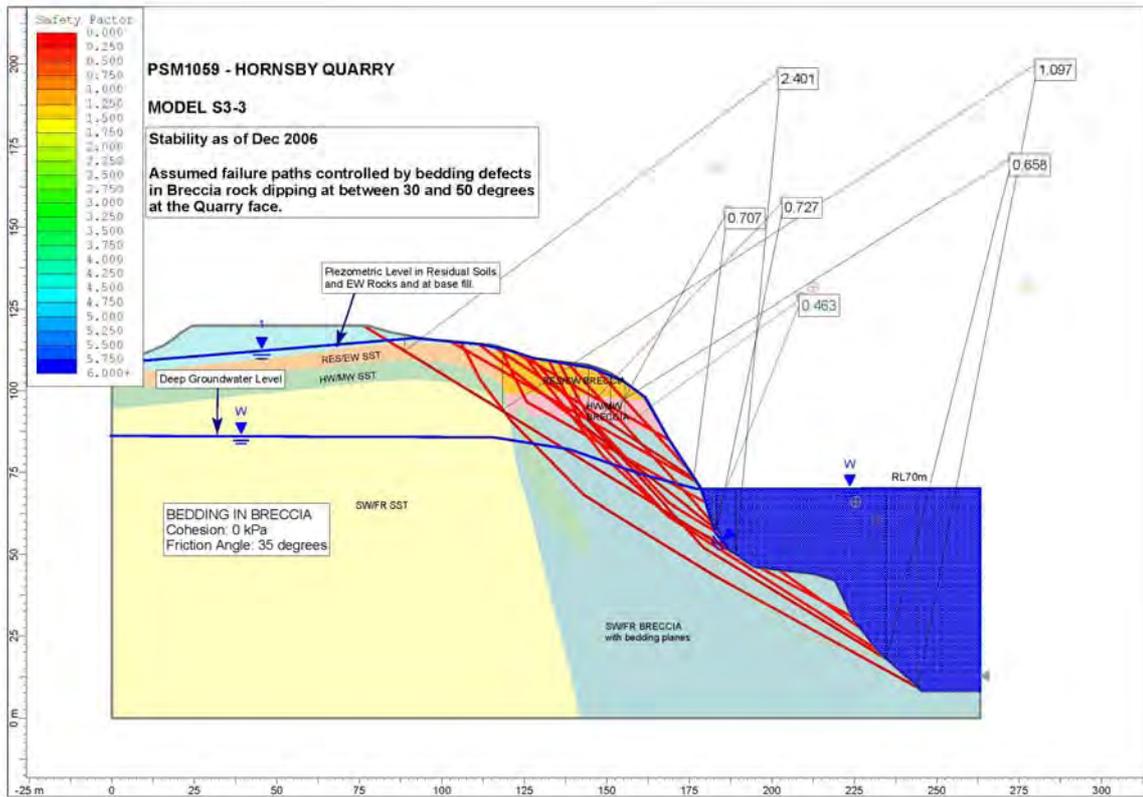


Figure 38 – Model S3-3 Domain S3

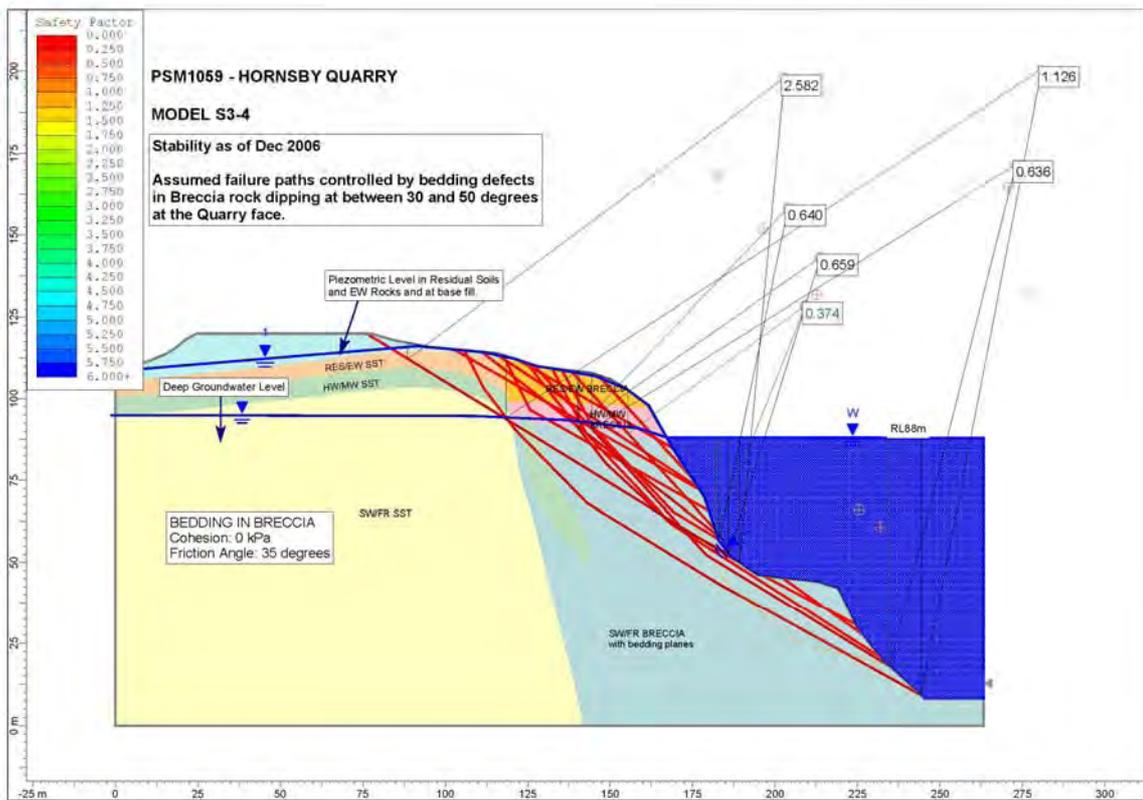


Figure 39 – Model S3-4 Domain S3

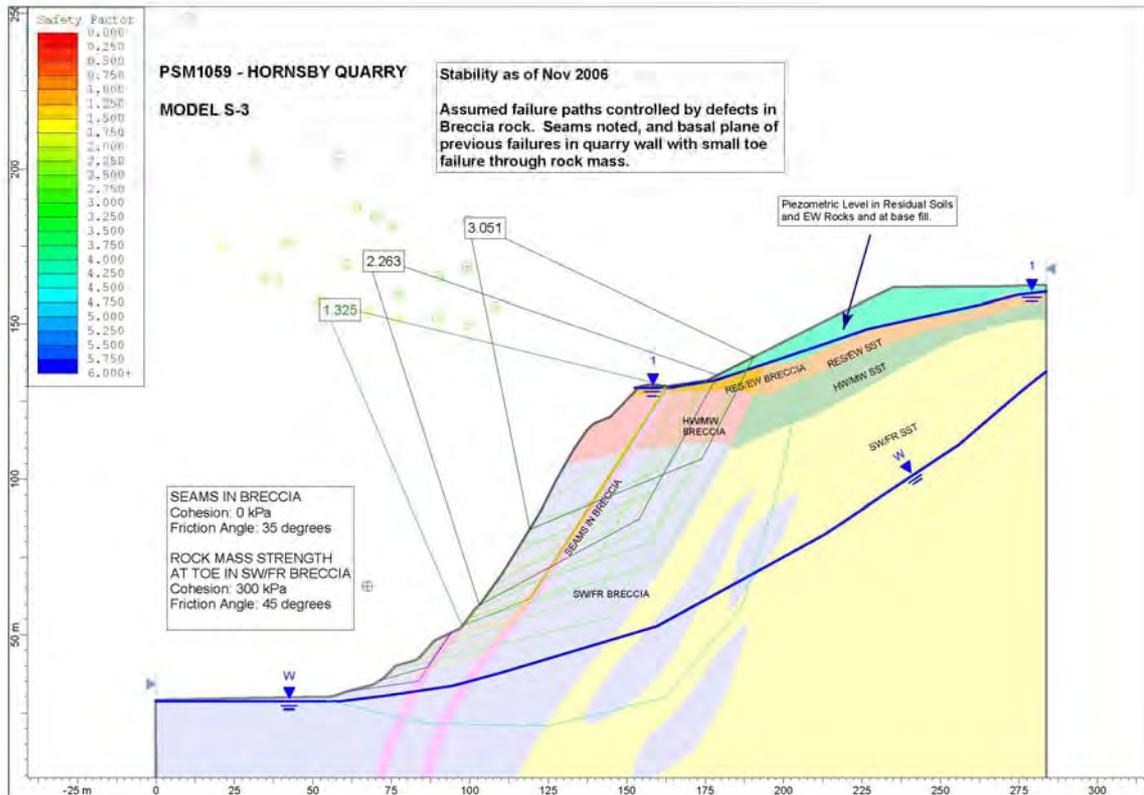


Figure 40 – Model S-3 Domain S1

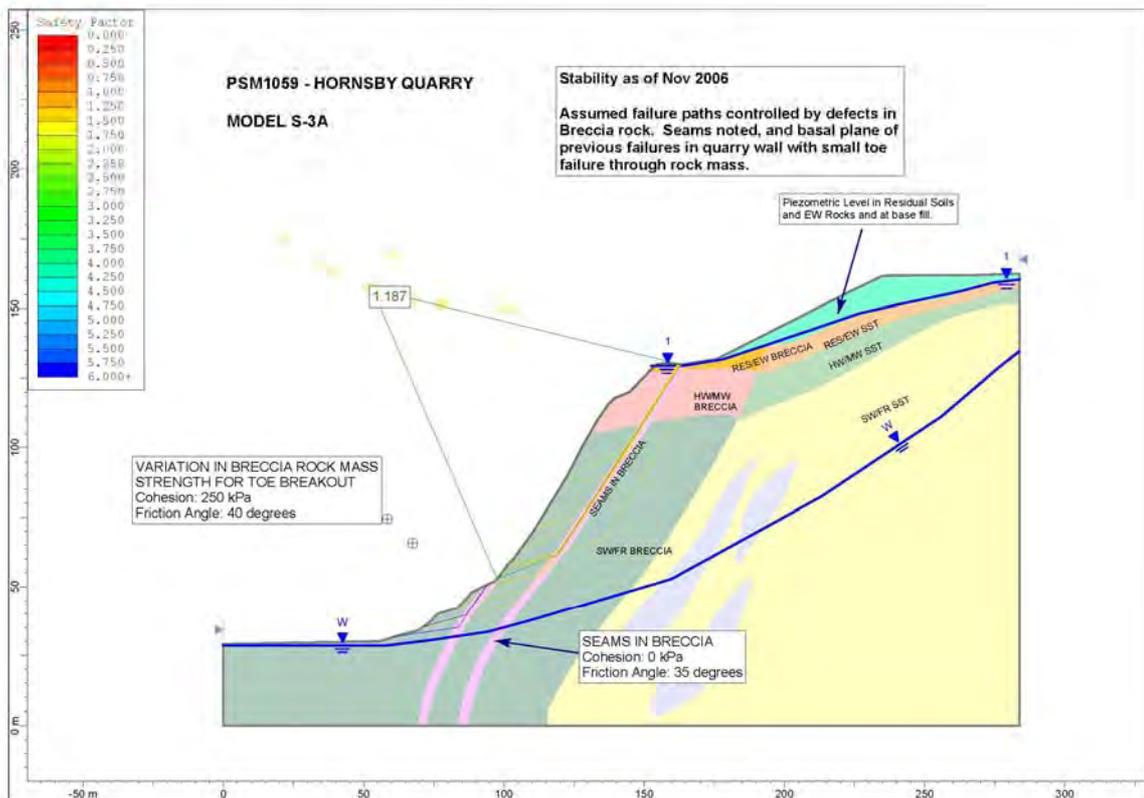


Figure 41 – Model S-3A Domain S1 – Assess variation of SW/FR shear strength

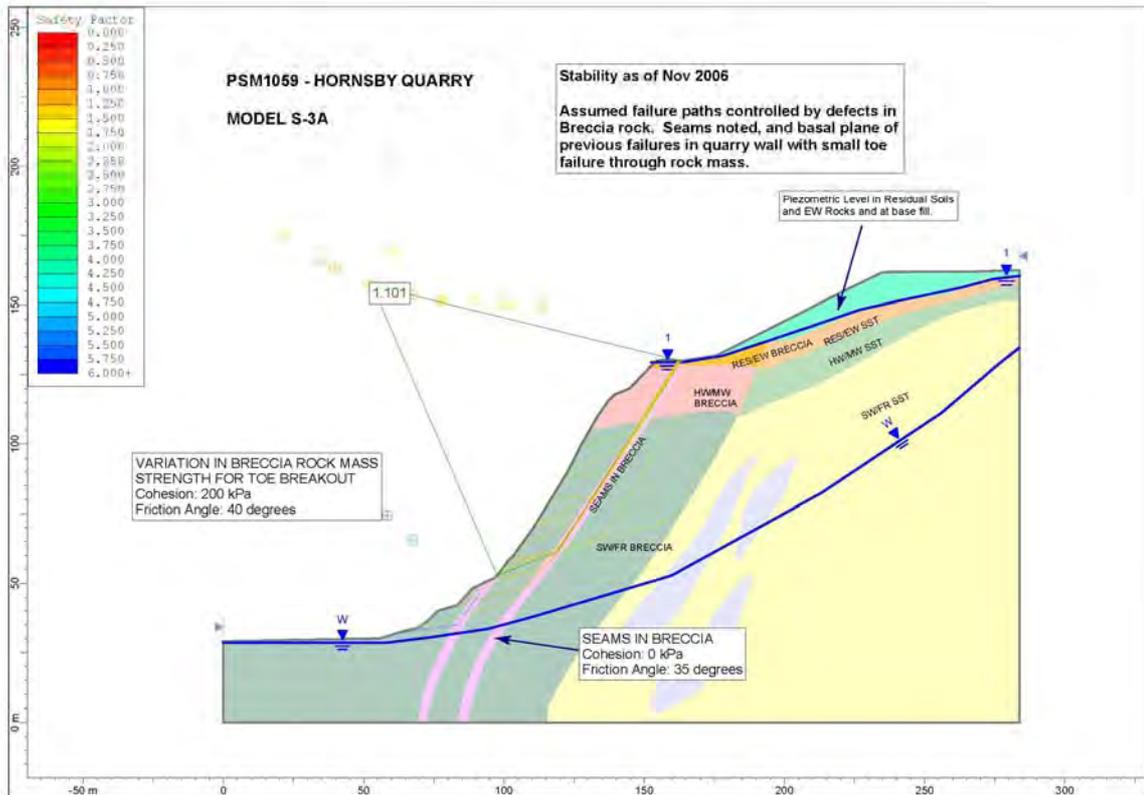


Figure 42 – Model S-3A Domain S1 – Assess variation of SW/FR shear strength

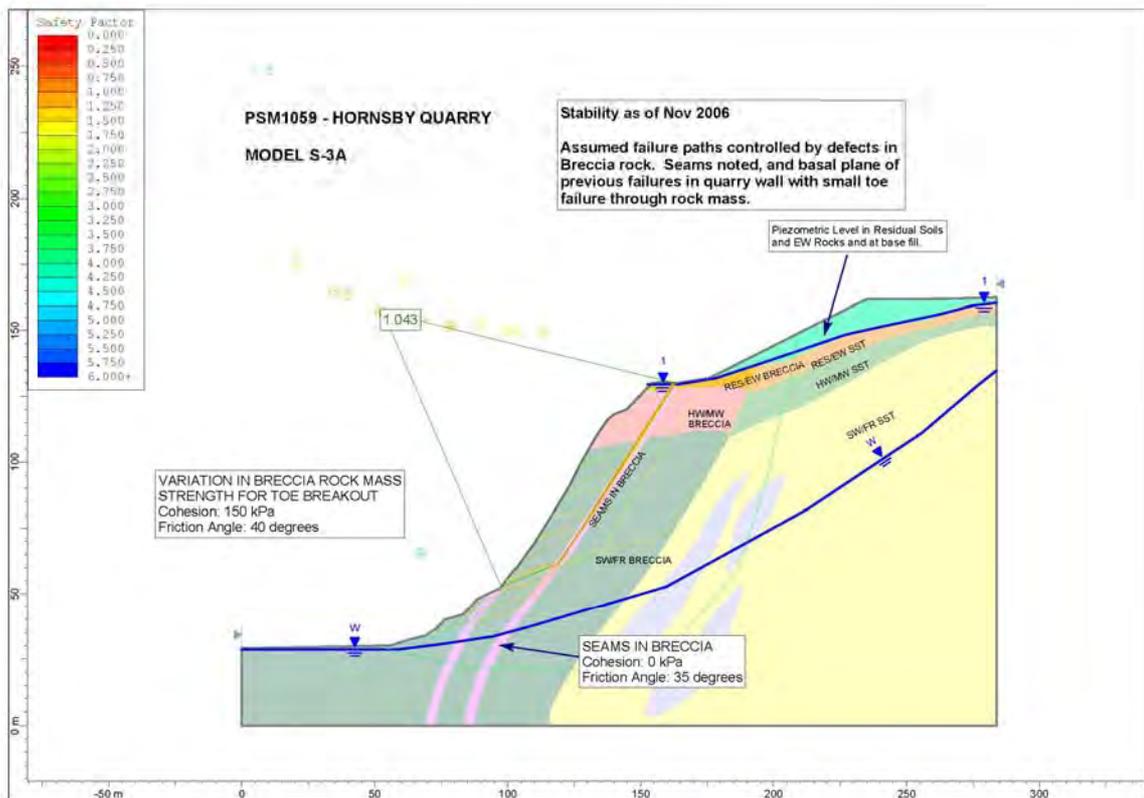


Figure 43 – Model S-3A Domain S1 – Assess variation of SW/FR shear strength

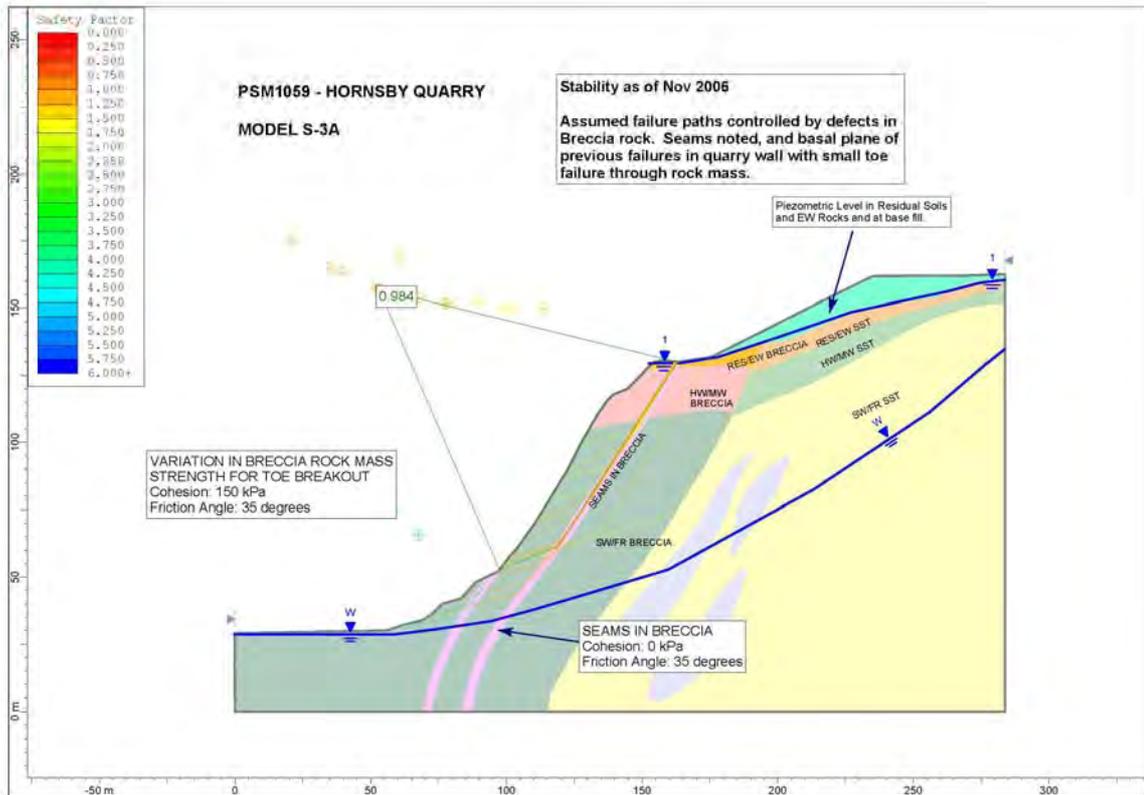


Figure 44 – Model S-3A Domain S1 – Assess variation of SW/FR shear strength

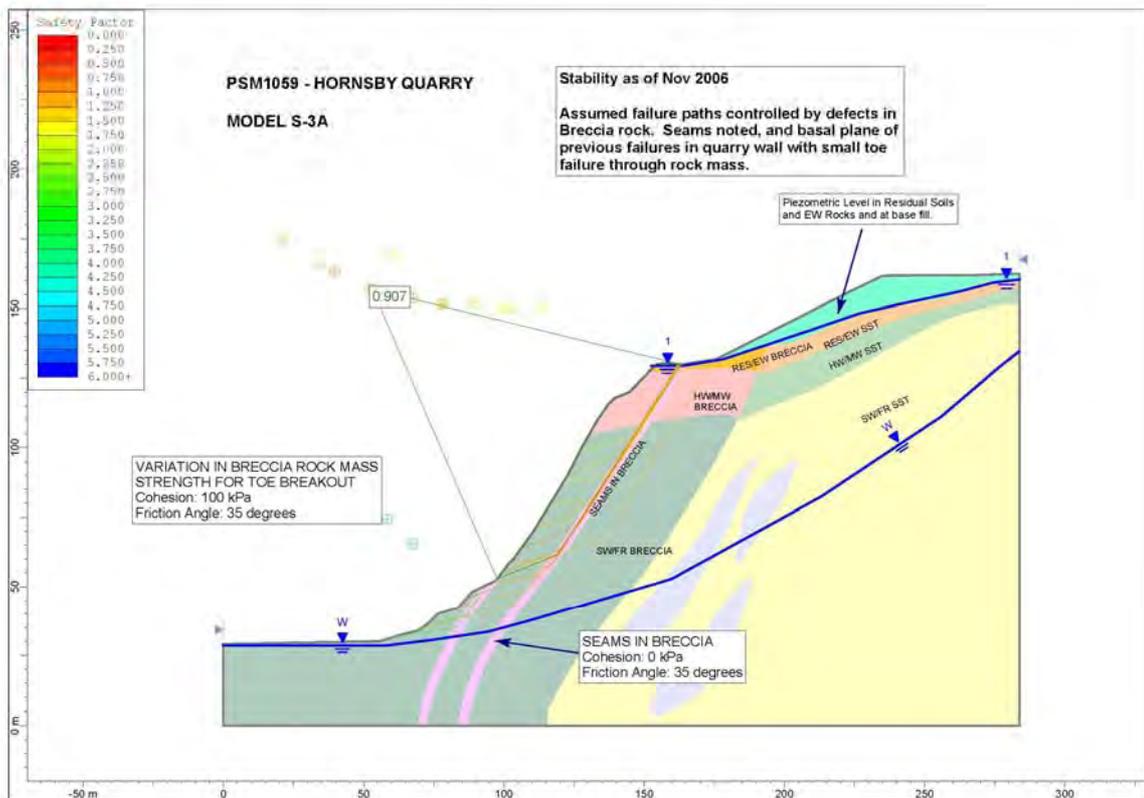


Figure 45 – Model S-3A Domain S1 – Assess variation of SW/FR shear strength

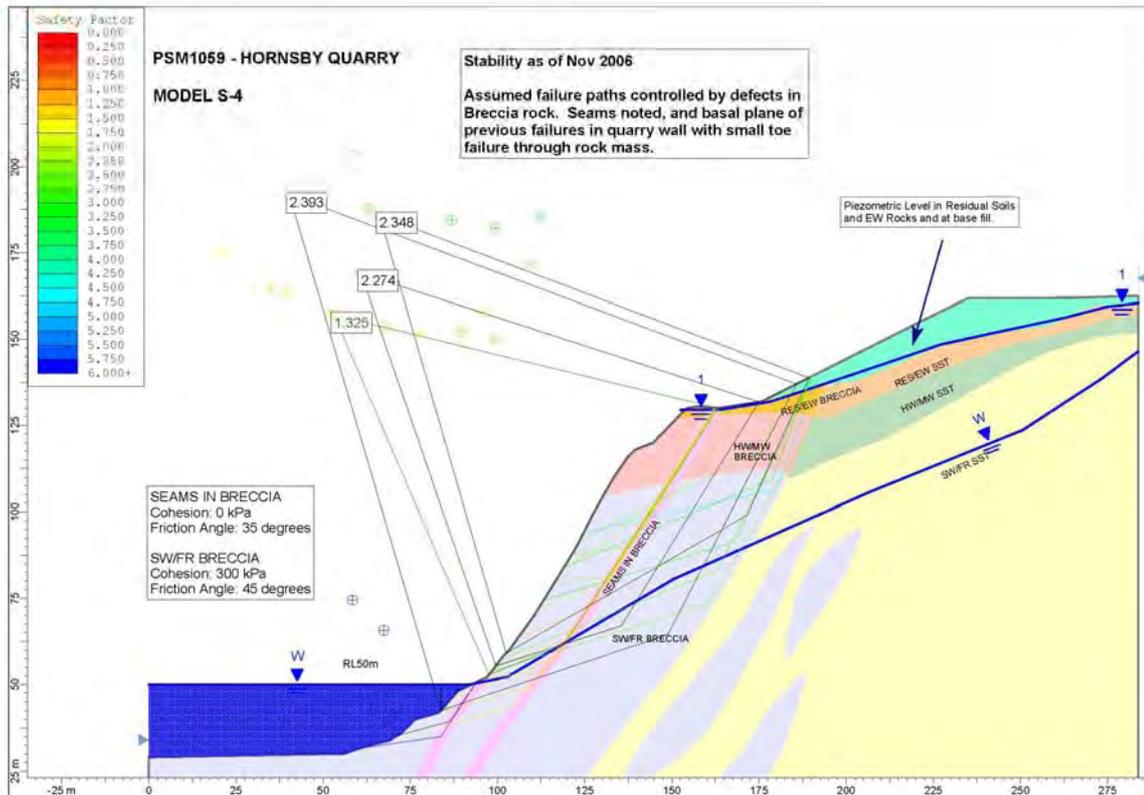


Figure 46 – Model S-4 Domain S1

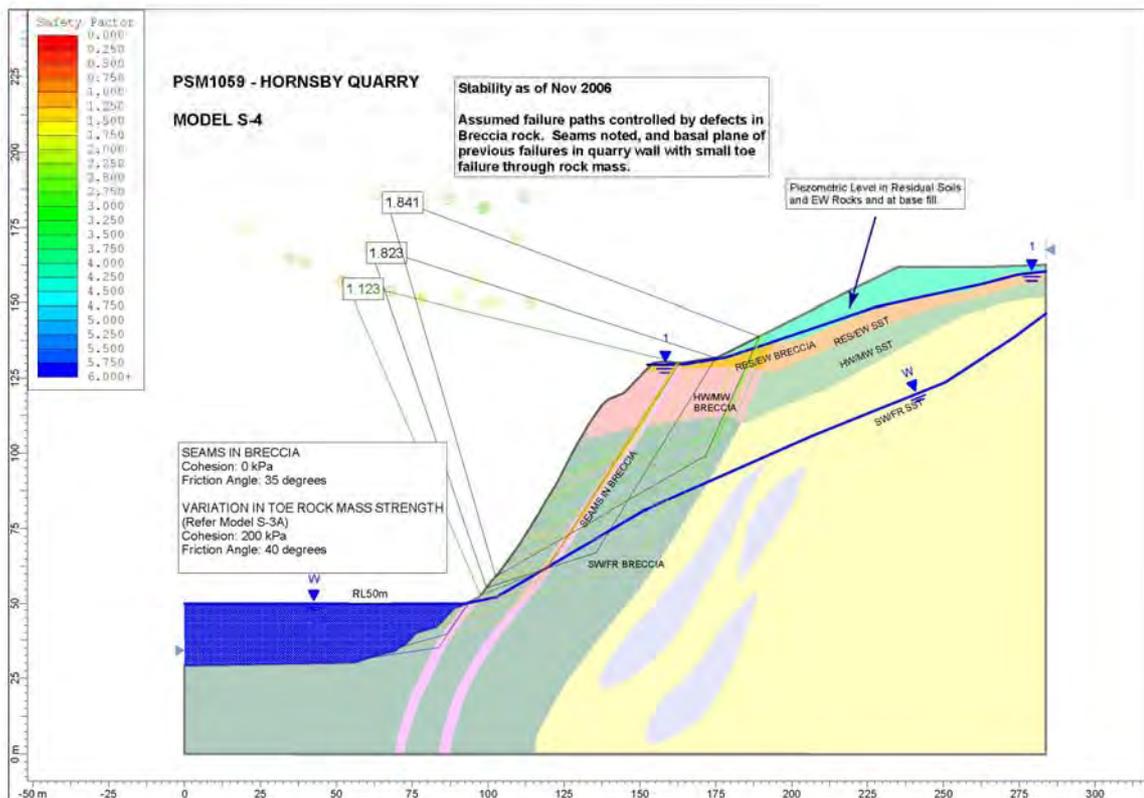


Figure 47 – Model S-4A Domain S1

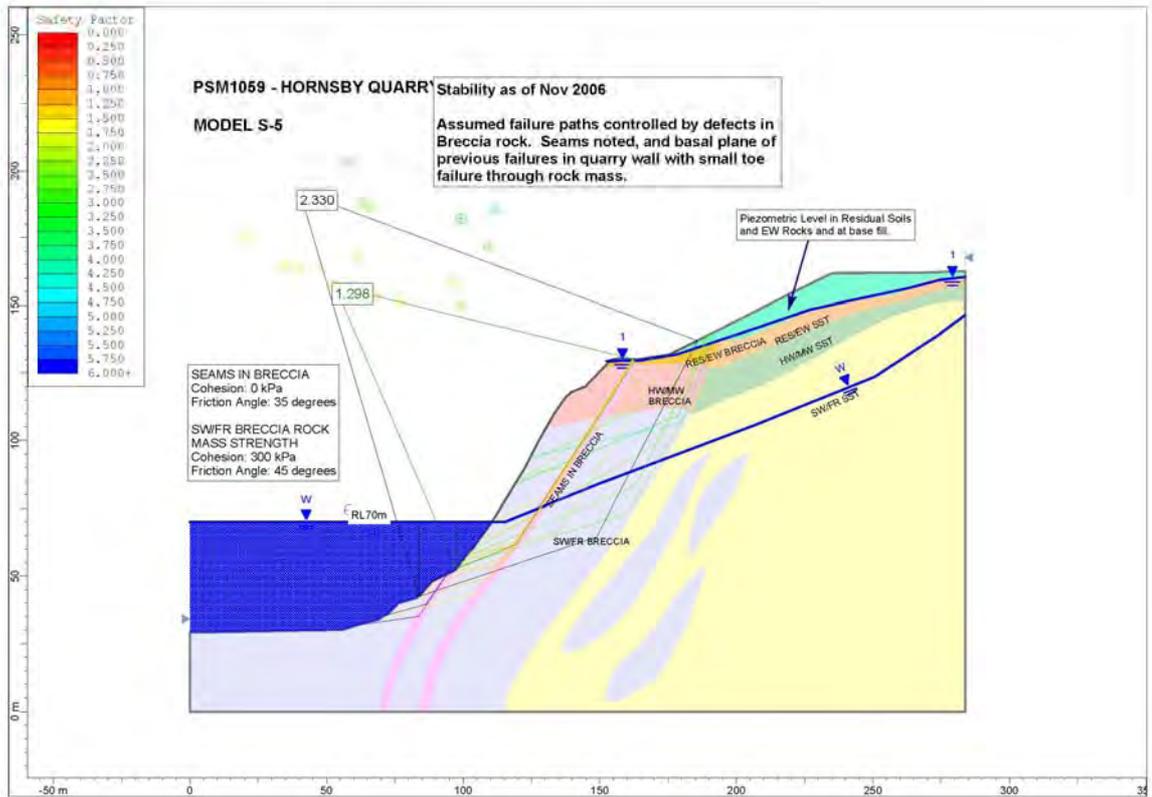


Figure 48 – Model S-5 Domain S1

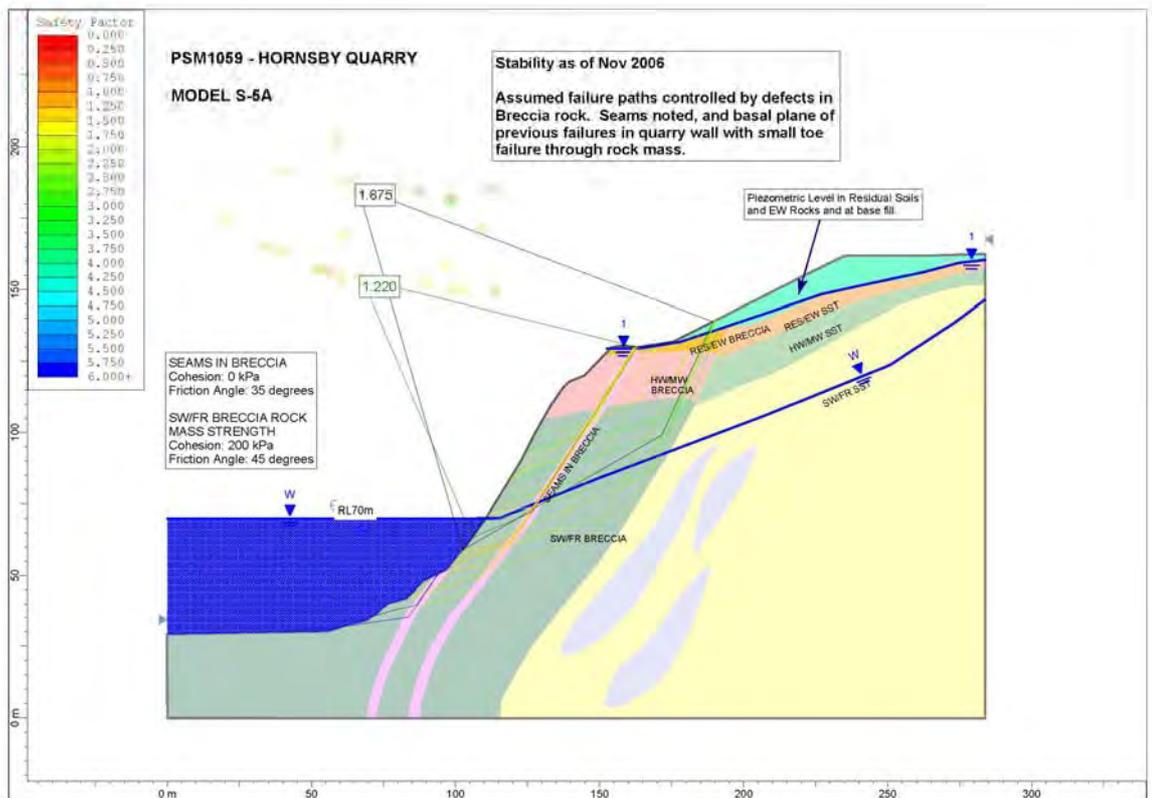


Figure 49 – Model S-5A Domain S1

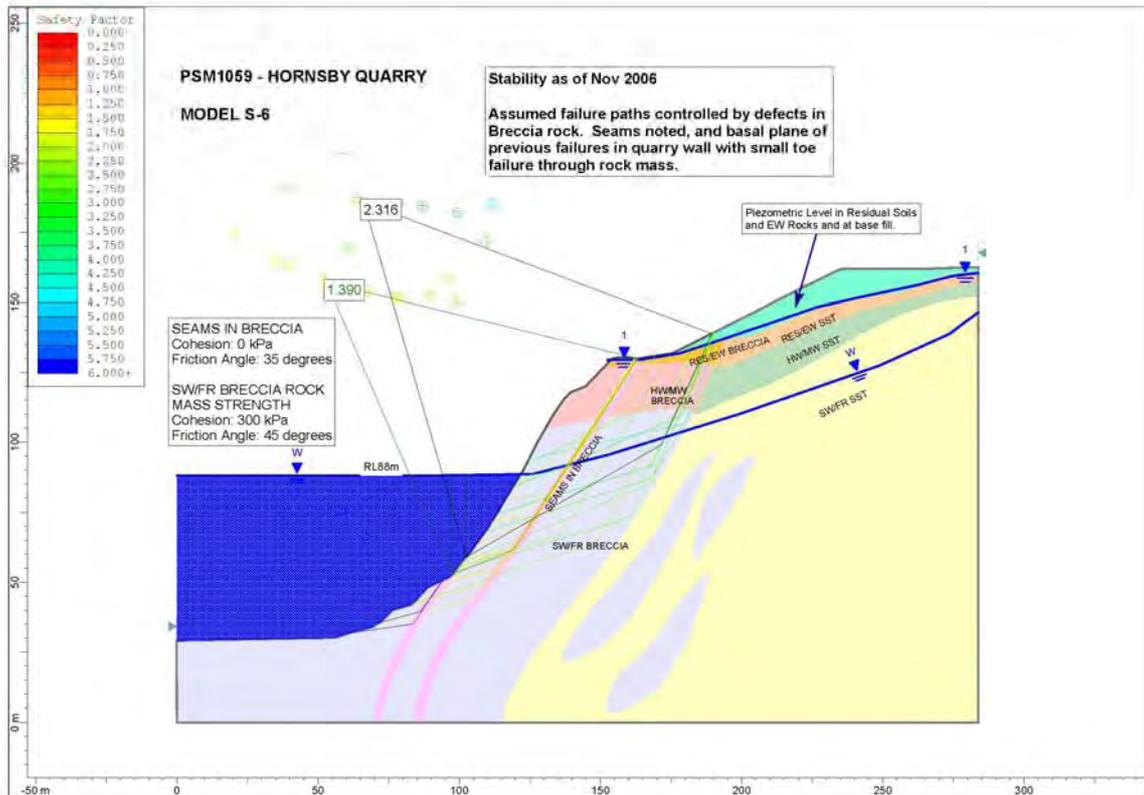


Figure 50 – Model S-6 Domain S1

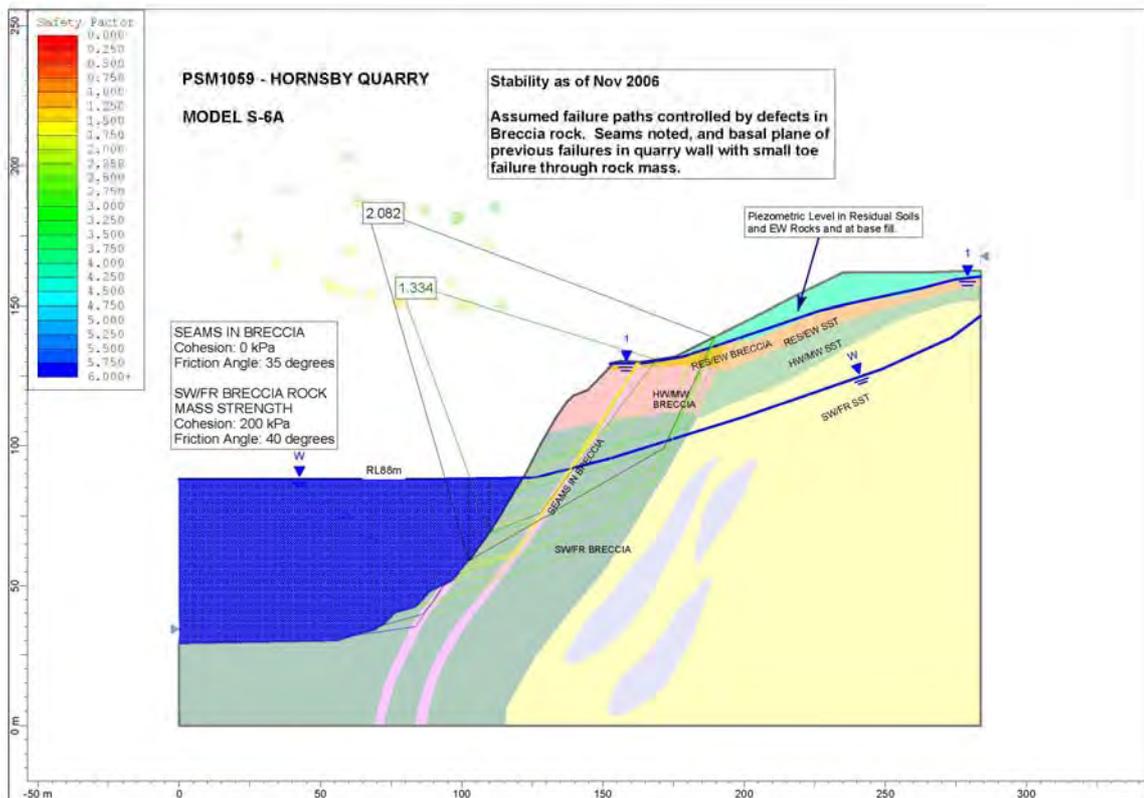


Figure 51 – Model S-6A Domain S1

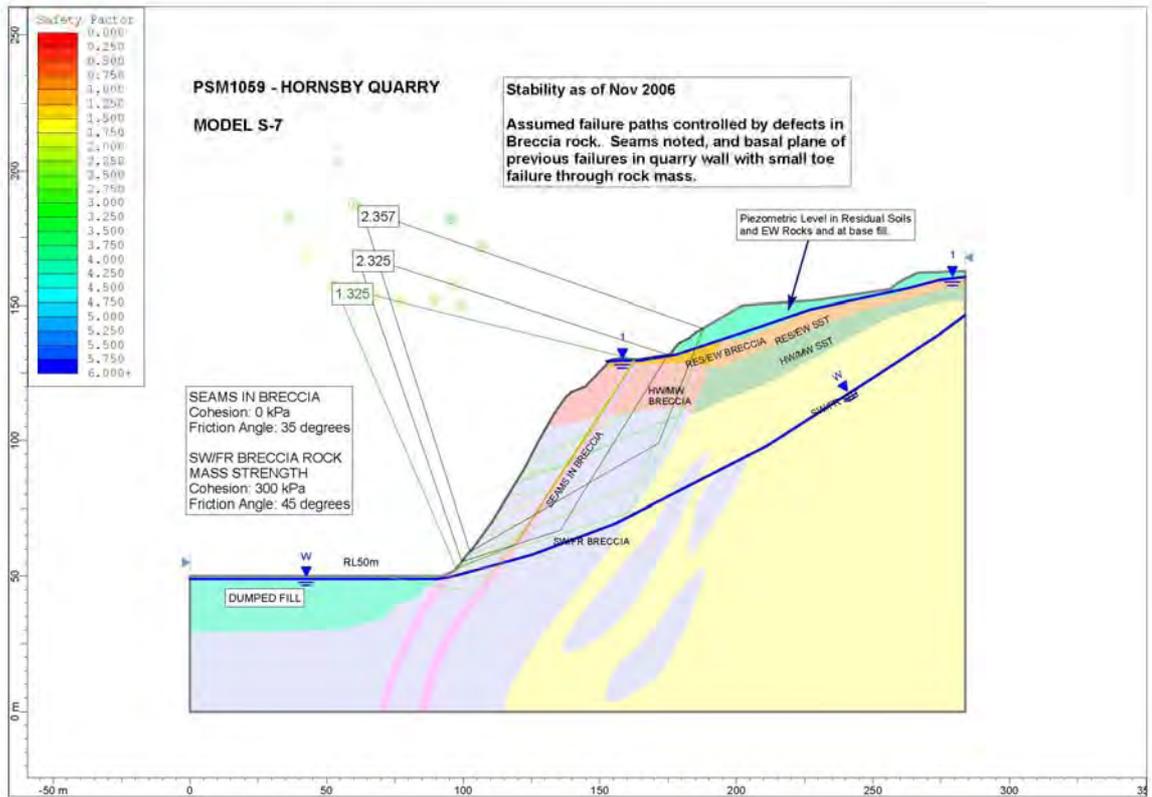


Figure 52 – Model S-7 Domain S1

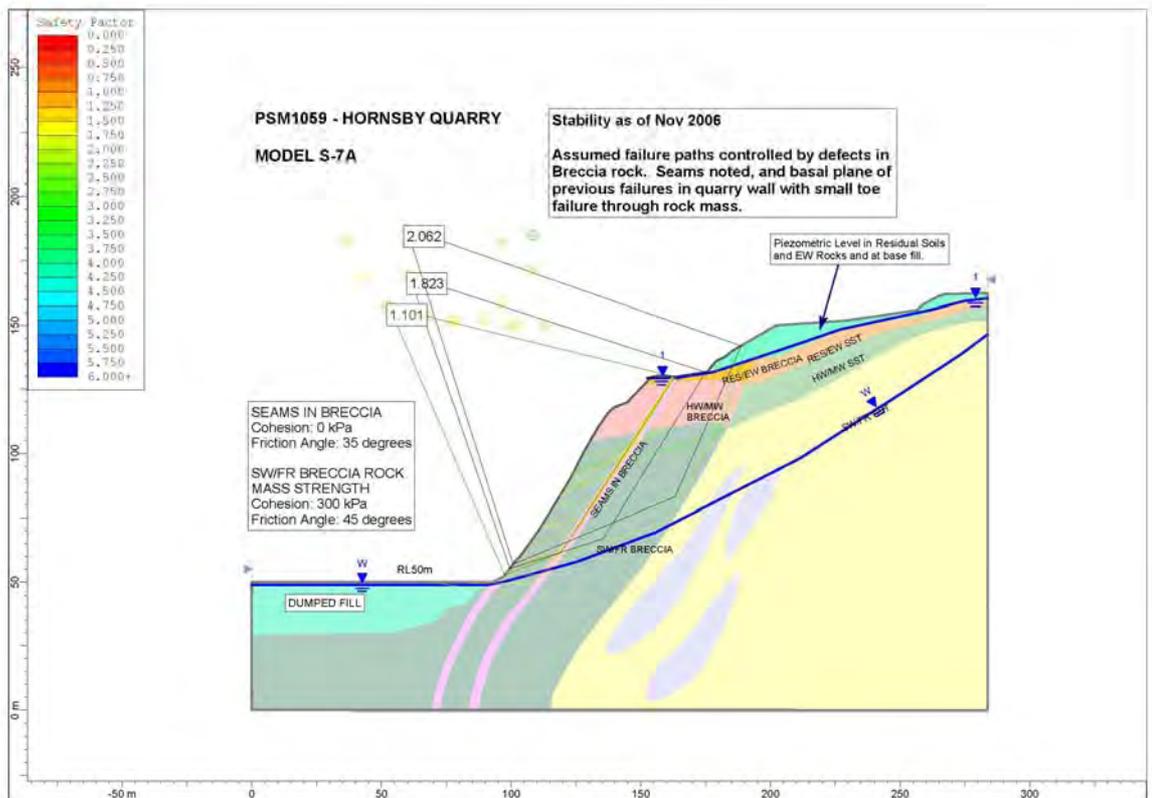


Figure 53 – Model S-7A Domain S1

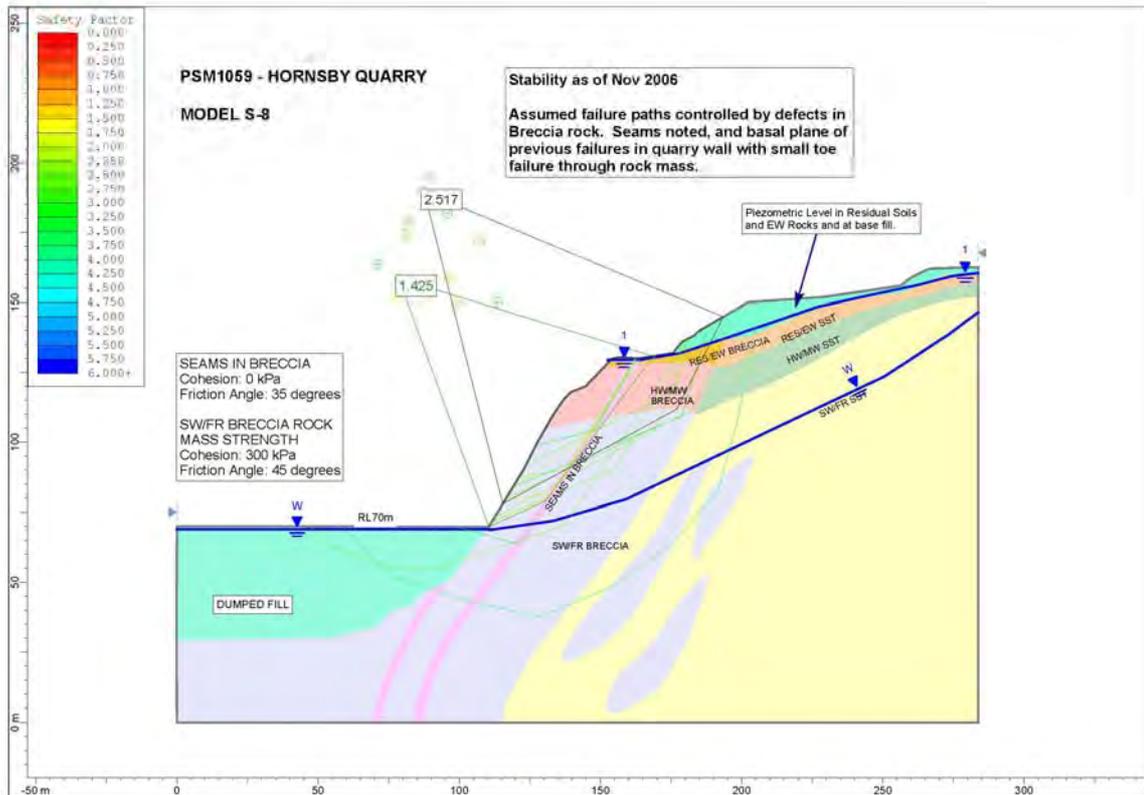


Figure 54 – Model S-8 Domain S1

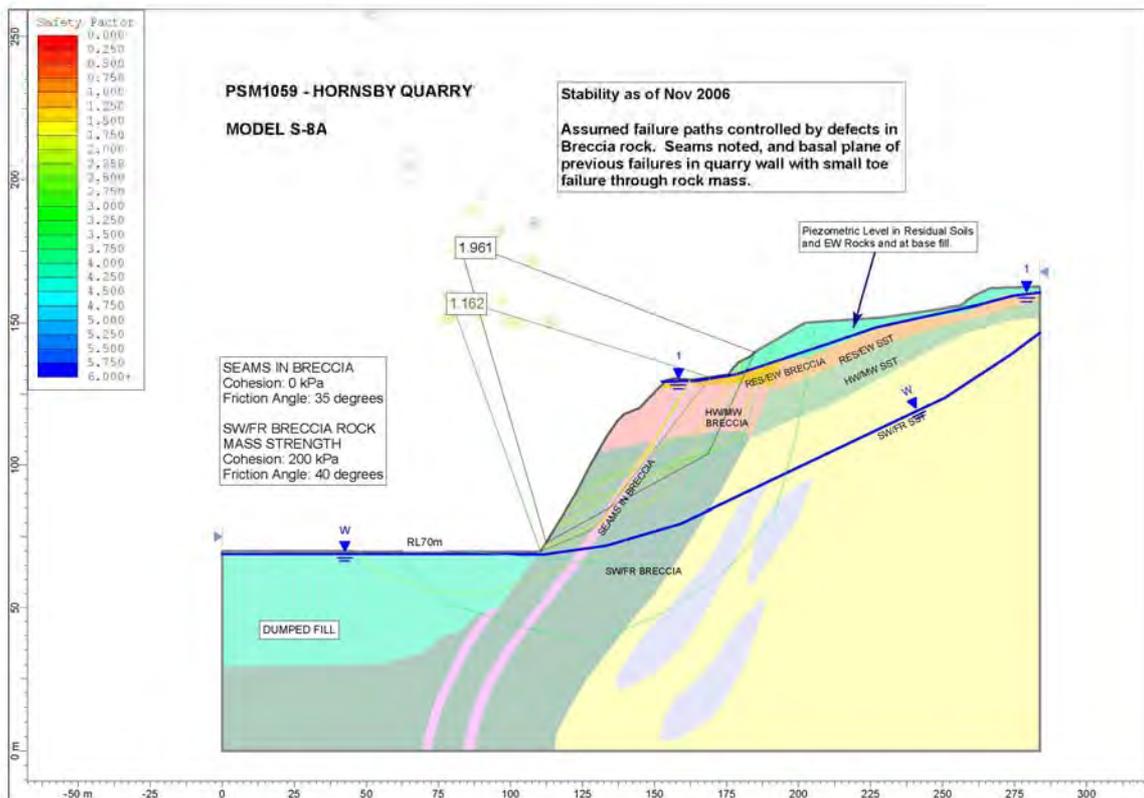


Figure 55 – Model S-8A Domain S1

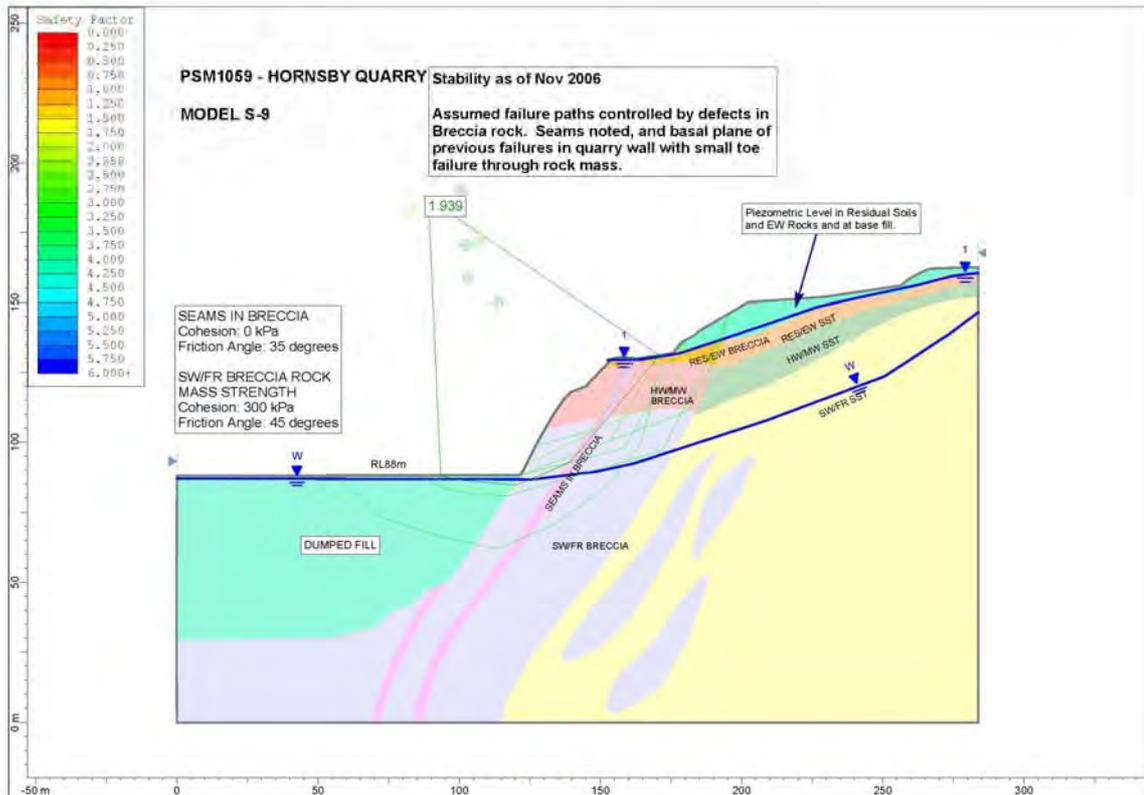


Figure 56 – Model S-9 Domain S1

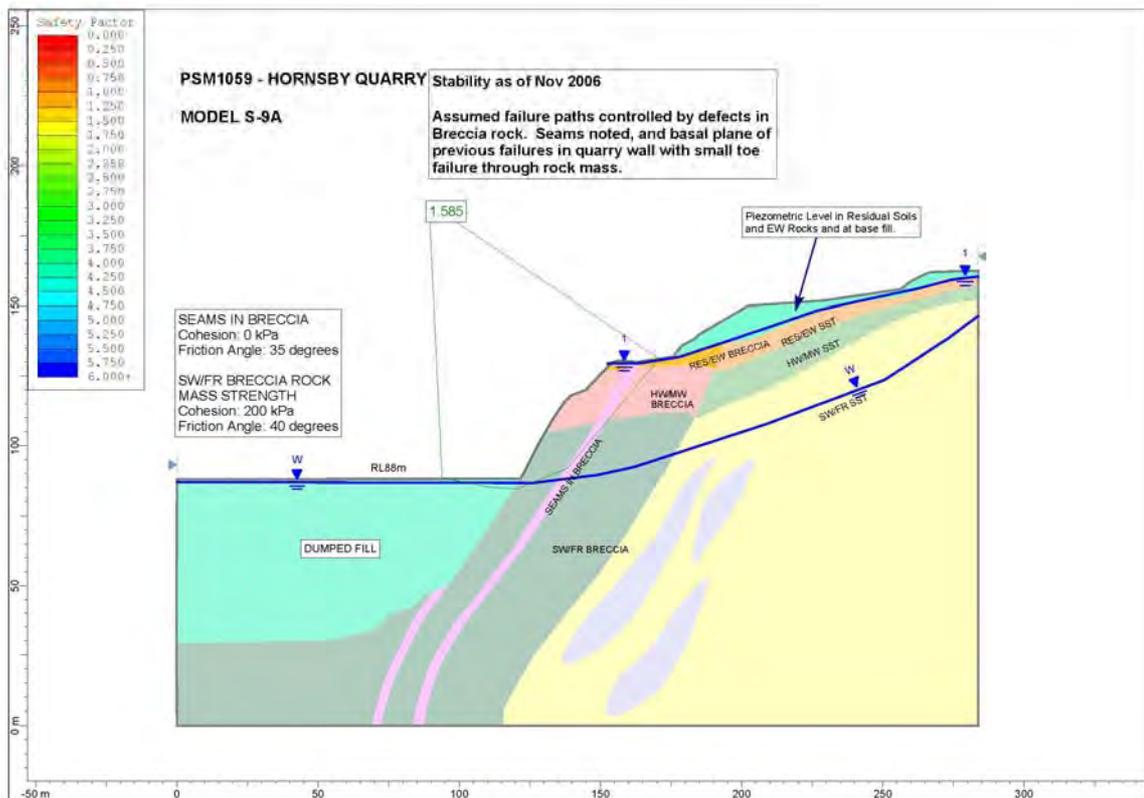


Figure 57 – Model S-9A Domain S1