Canterbury Coal Mine

Final Engineered Landform Geotechnical Review

BATHURST
RESOURCES LIMITED

REVISION HISTORY:			
Date	**Version No.**	**Lead Author**	**Contributing Authors**
16 March 2021	1	M Begbie MSc(Hons), MEngSc, PhD, MAusIMM CP (Geotech) – Principal Geotechnical Engineer, Bathurst Resources Ltd	

Version No.	**Detail / Changes**

TITLE:	**PLANNED REVIEW:**
Canterbury Coal Mine – Final Engineered Landform Geotechnical Review	

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1.0 Introduction

Canterbury Coal Mine (CCM) is an opencast coal mine in the western foothills (Malvern Hills) of the Canterbury region, approximately 70km west of Christchurch (Figure 1). The mine is a truck and excavator opencast operation that currently produces thermal coal for the domestic market and is owned and operated by Bathurst Coal Limited (BCL). BCL is currently in the planning phase for closure of its CCM mining operation.

The closure concept for the site includes filling of various parts of the mining void, development of a pond and outlet channel and re-profiling of some of the existing cut batter slopes. The post-mining landscape will be made up of a combination of farming and forestry land uses on a rehabilitated land surface contoured in sympathy with the surrounding hilly terrain.

The purpose of this review is to assess the long-term stability of the proposed final landforms. The geotechnical modelling reported below provides the data inputs, and modelling results for the proposed final landform.

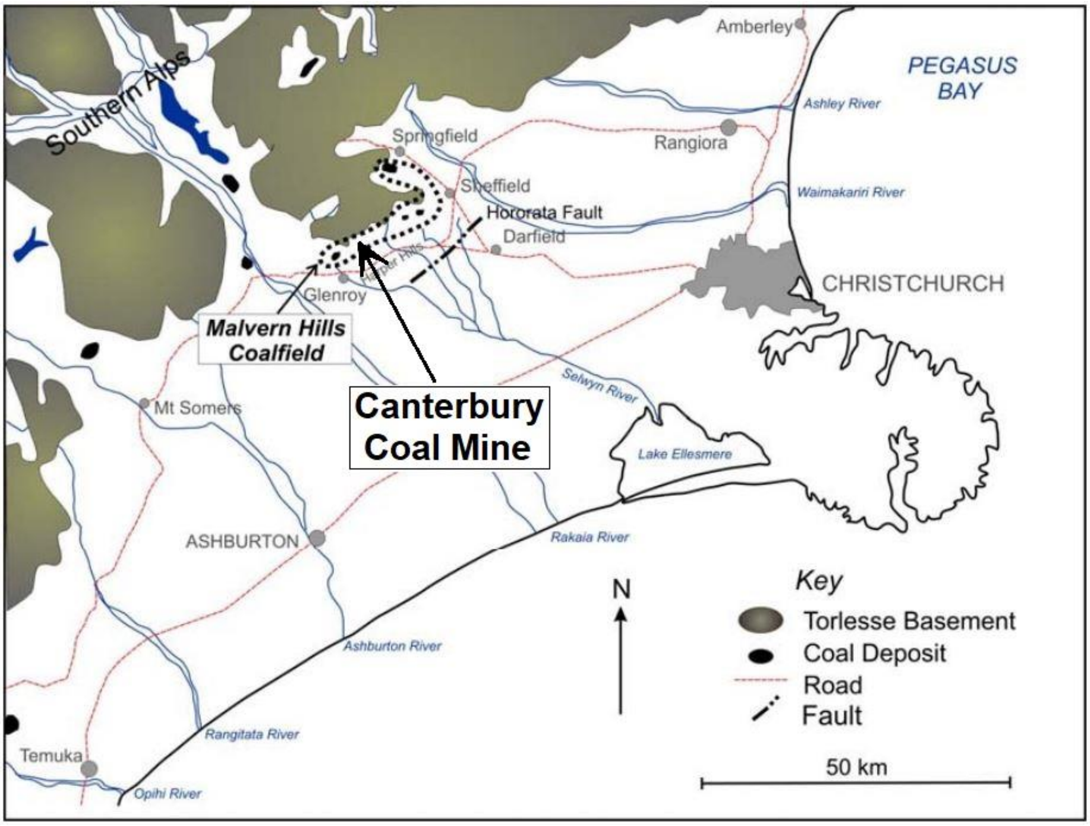

Figure 1. Map showing location of the Canterbury Coal Mine within the Malvern Hills Coalfield (after Duff & Barry, 1989; Seale 2006).

2.0 Previous Work

Prior to this review a number of geotechnical assessments have been completed to investigate the geology and structure, material strength parameters, cut wall designs and engineered landforms (ELF) at CCM and are directly relevant to this review. These include:

- Bathurst Resources Limited (September 2016). *Canterbury Coal Mine – North ELF Geotechnical Assessment Report.*
- Bathurst Resources Limited (May 2018). *Canterbury Coal Mine – North ELF – As-built Geotechnical Assessment Report.*
- Bathurst Resources Limited (July 2018). *Canterbury Coal Mine – Open Cut Geotechnical Report – Update Revision 1.*
- Geosolutions Tasman Ltd (May 2016): *Canterbury Coal Mine – Geotechnical Review of the Proposed Green Dump Design.*
- Geosolutions Tasman Ltd (November 2016): *Canterbury Coal Mine – Geotechnical Review of the Proposed Upper Dump and Tara Dump Designs.*

3.0 Site Description

The CCM opencast mining area currently consists of cut pit slopes that expose a moderate to steeply dipping sedimentary sequence along with ELFs that have progressively backfilled the pit void as mining has extended to the northeast (Figure 2A, B & C). Parts of the mine have been developed over areas of historical underground workings. Workings in coal seams that have been encountered during mining have been mined out then backfilled. The mine is excavated into a hill and ridge site flanked to the northwest and southeast by moderate to steep sided gullies (Figure 2C). The gullies extend down to low areas hosting the Bush Gully Stream to the north and Tara Stream to the south.

The Digital Elevation Model for the site shows the natural slope angles vary across the site and neighbouring land and are generally between 15° and 30°. There are areas steeper than 35° to 40° correlating to the steep gully sides and exposed dip slopes of the sedimentary rock sequence. Areas with slopes shallower than 15° correspond to roads, hill tops/ridge lines and gully/valley bottoms. There is approximately 100m of relief between the valley floors and the ridge tops hosting the mine.

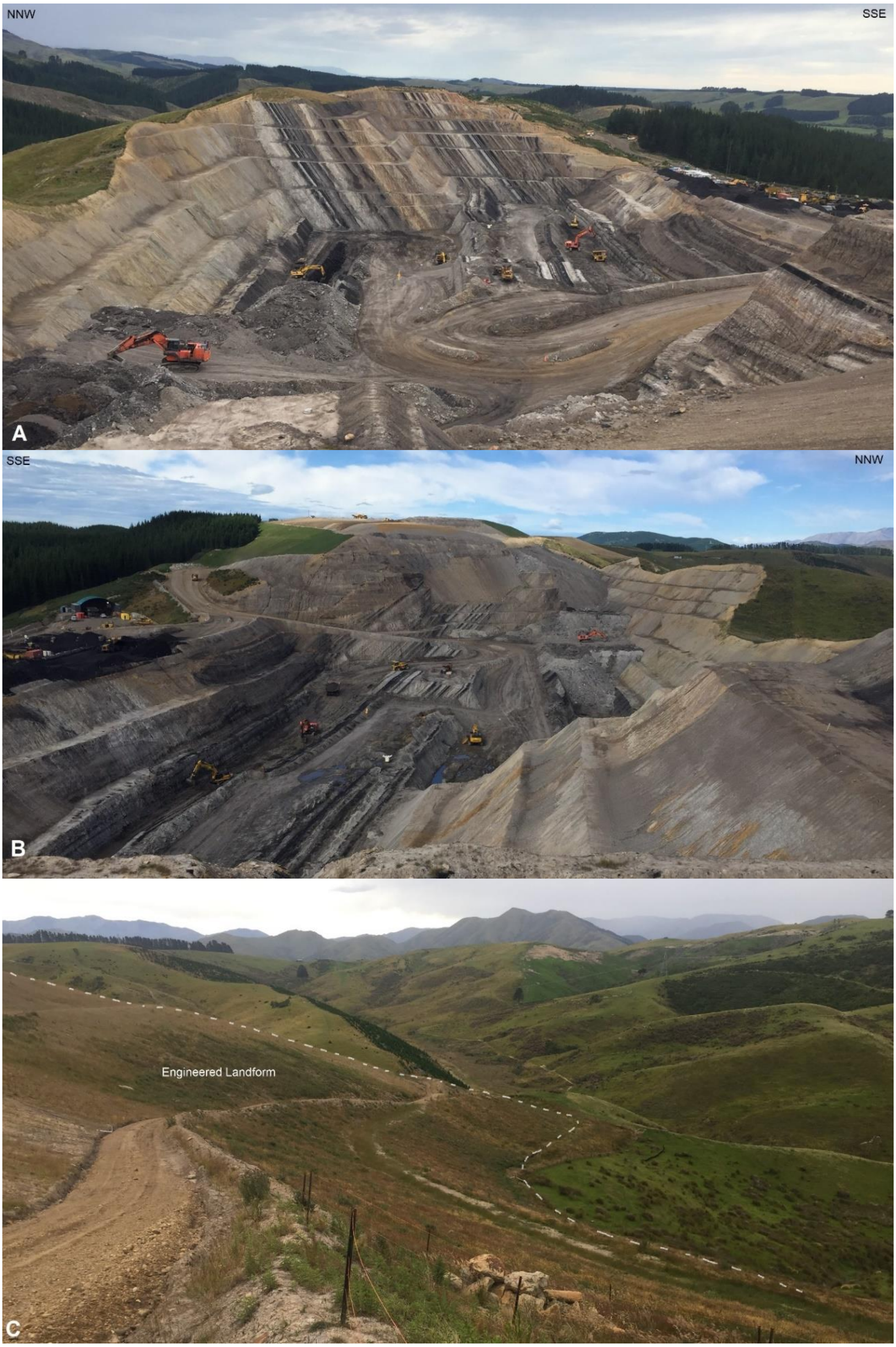

Figure 2. A) Overview of the CCM geology looking NNE at the moderate to steeply SE dipping coal measures strata of the Broken River Formation. B) View looking SW along the strike of the coal measures strata. Advancing pit backfill progressively infilling void can be seen in background. C) View looking WNW across one of the mines Engineered Landforms (North ELF) on the left-hand side and natural terrain of farmland on the right hand side. Lady Barker Range in the background.

4.0 Geotechnical Database

BCL has an extensive database of drill hole information for the CCM area with drill holes spaced at approximately 75m centres across the pit. Drill hole information combined with geological mapping of pit wall exposures has provided a sound level of geological and geotechnical understanding resulting in good control around lithological contacts and structural interpretations.

The geotechnical model for the CCM pit was developed from geological cross-sections through a 3-dimensional Geology Model of the site. Development of the geotechnical model is described in detail in the BRL 2018 Open Cut Geotechnical Report. The model combines geological and engineering geology interpretations (e.g., BRL 2018).

Piezometric monitoring data and previous assumptions on groundwater behaviour from cut pit slopes have been used in the groundwater interpretation in the context of geotechnical stability. These assumptions are consistent with field observations regarding the presence or absence of seepages on the pit walls. Material parameters have been adopted based on historical data (mapping, laboratory tests, back analyses and slope performance monitoring) for the local area and are the same as those used in the BRL (2018) CCM Open Cut Geotechnical Report.

5.0 Ground Conditions

5.1 Geology

The CCM forms part of the larger Malvern Hills Coalfield (Figure 1). The geology of the CCM mining area is dominated by Late Cretaceous to Paleocene age strata that unconformably overlie Jurassic to Triassic greywacke basement rocks (Torlesse Supergroup) and early Cretaceous volcanics (Figure 3; Holm & Bell, 2013). The Monro Conglomerate Formation forms the basal unit exposed at the mine, comprising a fining up succession of conglomerates, grits, sandstones and claystones interbedded with occasional thin coal seams. Overlying the conglomerate is the main coal producing horizon, the Broken River Formation also referred to as Broken River Coal Measures (BRCM). This formation comprises typically thin coal seams interbedded with carbonaceous shale and very fine to fine grained quartz sandstones. The BRCM grades into an upper unit known as the Conway Formation consisting of very fine to fine grained micaceous quartz sands that are locally cemented (Holm & Bell 2013 and references therein). Pleistocene age loess and gravel deposits unconformably overlie the lower rock units.

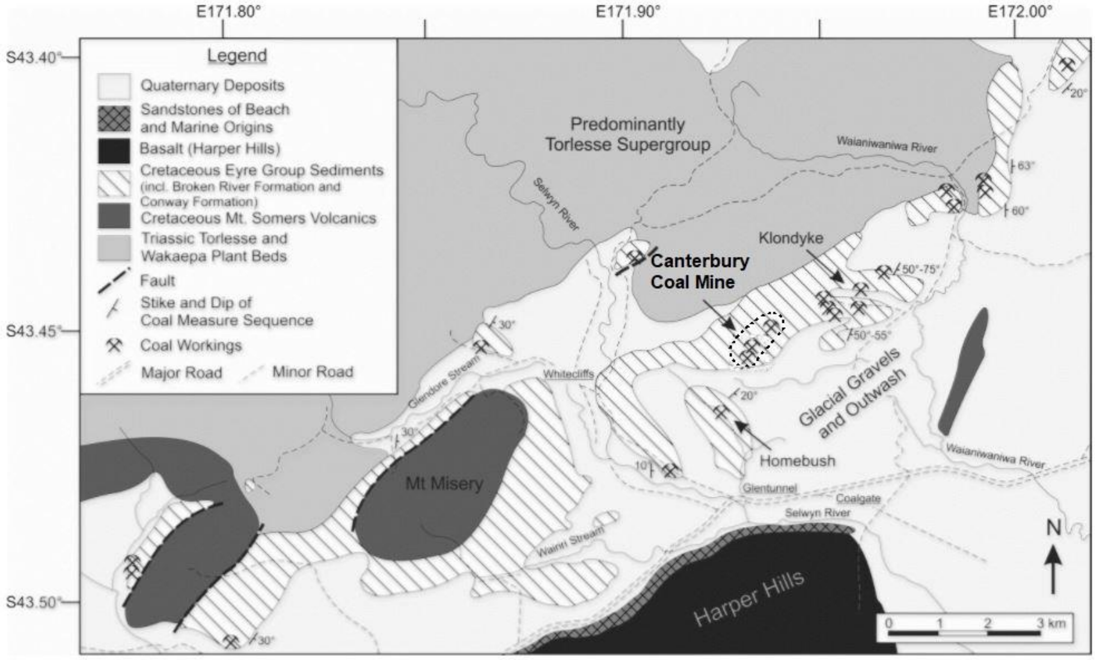

Figure 3. Regional geological map of the Canterbury Coal Mine area (from Holm & Bell, 2013).

5.2 Structure

Bedding within the sedimentary sequence exposed in the current pit has an overall moderate to steep (40° to 50°) dip to the southeast (Figure 2A). The orientation of bedding is very consistent along strike. Bedding is the most persistent rock mass discontinuity identified within the formations. Low strength bedding shear surfaces within the BRCM have been identified as critical features influencing slope stability. Bedding parallel shears are known to exist within highly sheared carbonaceous mudstone units and along the top and bottom of the coal seams. Bedding shear surfaces have very low shear strength and are typically laterally continuous unless offset by faulting.

The orientation of bedding, and hence these low strength bedding parallel shears, relative to the pit walls have been the main control on the slope design angle and orientation at CCM. In most cases, when the bedding surface dips unfavourably out of the slope, the wall has been designed parallel to bedding with restrictions on batter height to reduce the overall slope angle and minimise slab/buckle type failures associated with thinly bedded sequences (e.g., Seale, 2006). This design has ensured adequate slope stability performance.

Minor low displacement strike-slip faults that often form as conjugates cut bedding and coal seams obliquely. Faults exposed within the pit have horizontal offsets on the order of several metres. These faults do not have significant rock mass disturbance but restricted to localised clay-gouge deformation zones. Faults, depending on their location and orientation relative to the pit walls, may have the potential to influence stability in the form of release surfaces for minor wedge or planar / slab failures.

5.3 Groundwater

Available piezometer data indicates groundwater depths are variable within the hilly terrain of the site (BRL

2018). In general, a continuous water table is expected to be present at depth below the ridge rising with topography away from the valley floors, although a more localised complex distribution of water pressures can be expected from the influence of stratigraphic layering of the sedimentary units, faulting/jointing and historic underground workings. Groundwater movement is expected to follow individual beds with higher hydraulic conductivity, following the dip of the beds to the southeast exiting the slopes along the gullies as evidenced by the wetland features in these areas. Groundwater levels are relatively shallow within the valley and gully bottoms, responding seasonally to variation in precipitation and run-off rates.

A single piezometric surface has been defined for stability analyses with a gradient that extends upwards from the toe of the design slope. For the design groundwater condition, a gradient of 1V:6H has been adopted. In the elevated groundwater scenario, a gradient of 1V:3.5H from the toe of the slope has been adopted. For the purposes of the design, the engineered fill is assumed to be partially saturated with a pore water pressure coefficient, Ru of 0.1 (ratio of pore pressure to overburden stress) adopted as a typical case to represent seepage pressures within the fill. This groundwater assumption is supported by an investigation of a CCM ELF where Standard Penetration Test samples were dry, and no groundwater surface was encountered within the compacted fill during testing or in piezometers (BRL, May 2018).

Uncertainty in the groundwater profile due to seasonal fluctuations has been accounted for by adopting a standard deviation of 5m in the sensitivity analyses.

6.0 Existing Slope Performance

The CCM cut slopes and engineered fill slopes have performed well during the mining operation with minimal stability issues. The only exceptions are localised batter scale failures of cut slopes from unfavourable geology, typically related to the intersection of walls with bedding plane shears and/or small faults. This favourable historic stability performance suggests the cut and fill slope angles and wall orientations constructed to form the pit have been appropriate for the material types and height of the slopes.

7.0 Slope Design

7.1 Final Landform

The natural slopes surrounding the mine area comprise moderate to steep hills and localised steeper gully areas (Figure 2C). The final pit is planned to be partially backfilled and cut slopes re-profiled to provide a stable long-term landform (Figure 4). The current highwalls will be supported with buttress fills at the toe of the slopes. The buttress slopes will be constructed with overburden waste with foundations in competent in situ rock. The final topography will generally resemble the surrounding landform in terms of slope angles. The maximum slope angles of the final landform will not pose a geotechnical risk in terms of slope instability.

7.2 Slope Design Parameters

Based on the current understanding of the geological structure and material parameters of the various rock formations and overlying loess/gravels, the following slope design parameters have been developed for final slope angles in the in situ and engineered landform materials as presented in Table 1 below. These recommended slope design parameters are confirmed by geotechnical analysis (see Appendix A).

Table 1. Slope Design Parameters.

Material Type	Geomechanical Unit	Maximum Overall Slope	Maximum Batter Slope	Maximum Batter Height
In Situ	Loess / Pleistocene Gravels	26.5° (1V:2H)	30.5° (1V:1.7H)	20m
	Conway Formation / Broken River Coal Measures	26.5° (1V:2H)	30.5° (1V:1.7H)	20m
	Munro Conglomerate	26.5° (1V:2H)	45° (1V:1H)	15m
Engineered Landform	Compacted mixed waste rock fill (free draining)	~21° (1V:2.5H)	26.5° (1V:2H)	20m

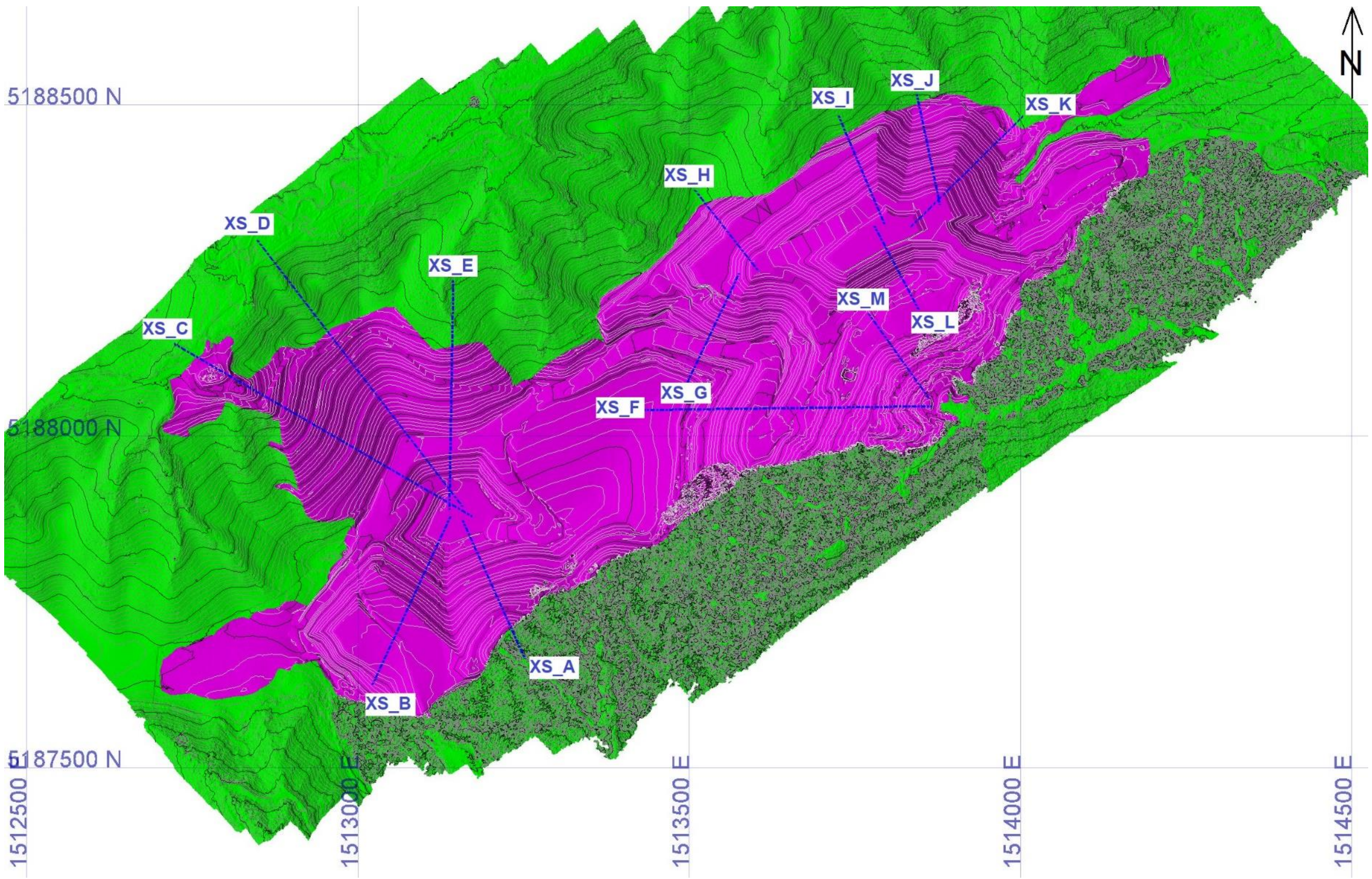

Figure 4. Topographic map of the proposed final landform design (purple colour). Location of slope stability cross-sections through the final landform design shown by blue dashed lines and labelled XS_A to M. Background topography (coloured green) as of February 2020. Major contour lines (black) at 10m interval and minor contour lines (grey) at 2m intervals.

7.3 Material Parameters

A summary of the material parameters adopted for this slope stability review are given in Table 2 below and on slope stability outputs presented in Appendix A. Material parameters have been adopted based on previous investigation reports (BRL 2018 and references therein).

The effective stress shear strength parameters and rock mass strength model inputs used in the stability analyses have been derived from a combination of laboratory tests, back analysis and Geological Strength Index (GSI) methods (Hoek-Brown Technique).

Table 2: Summary of Geotechnical Design Parameters.

Geological Unit	Material Model	Unit Weight (kN/m^3)	UCS (MPa)	GSI	m_i	c' (Cohesion kPa)	ϕ' (Phi degrees)
Fill (compacted overburden / inter-burden waste)	Mohr-Coulomb	18.3	-	-	-	0	36
Quaternary Alluvium (clay/silt, loess and gravels)	Mohr-Coulomb	17	-	-	-	5	22
Conway Formation / Upper BRCM (sandstone and siltstone)	Mohr-Coulomb	21.9	-	-	-	60	30
Weathered BRCM	Generalised Hoek-Brown	20.8	0.7	32	4	-	-
BRCM (high carb mudstone / siltstone and laminated sandstone)	Generalised Hoek-Brown[1]	21.1	3	37	4	-	-
Weathered Monro Conglomerate	Generalised Hoek-Brown	20.8	0.5	55	17	-	-
Monro Conglomerate (interbedded conglomerate and medium-coarse Sandstone)	Generalised Hoek-Brown[1]	21	3	65	17	-	-

[1]Modelled with an anisotropic strength function to account for bedding / bedding plane shears.

8.0 Slope Stability

8.1 General

The stability analyses were undertaken using Geostudios SlopeW limit equilibrium software to assess the long-term stability of the final landform. Slopes were analysed with the Morgenstern Price method with optimised failure surfaces reported. The analysis compares driving and resisting forces within a slope and determines a ratio (or Factor of Safety) where values greater than 1 are increasingly more stable (failure is assumed to occur when the factor of safety is less than 1). The lowest factor of safety (FoS) was assessed by either searching through potential failure surfaces (grid and radius method), and/or analysing a fully specified failure surface. The results of each analysis case are presented in Section 8.4 below. The outputs from the SlopeW analysis are attached in Appendix A.

8.2 Design Criteria

8.2.1 General

Final landform slopes have been designed to achieve suitable levels of stability in terms of Factor of Safety (FoS) along with managing uncertainties in the designs for which sensitivity analyses are undertaken (as outlined below). A sensitivity analysis allows determination of the 'sensitivity' of the safety factor to variation in the input data variables (material strength parameters and groundwater pressure) which may be critical to the assessment of slope stability.

8.2.2 Static

The design criteria for final landform slope stability have been assessed on the basis of the current design slopes, site conditions and previous slope performance. Assuming the rehabilitated land will be returned to a mixture of forestry and farming use a design FoS of 1.3 for static and 1.1 for elevated groundwater cases has been adopted. Table 3 below summarises the adopted design criteria. These criteria for low risk areas such as farm land are in general accordance with suggested limit equilibrium criteria accepted in other regions[1].

Table 3. Final landform slope stability design criteria.

Design Groundwater Conditions	Elevated Groundwater Conditions	Seismic Stability	
		Pseudo-Static	Displacement Analysis
Groundwater conditions expected within design life.	Groundwater conditions associated with heavy and prolonged rainfall event.	Design Ground Acceleration[1] based on a 1:250 Annual Exceedance Probability.	Displacement method of Jibson (2007) using critical acceleration ratio (Ky/Kmax) and magnitude. Method of Bray and Travasarou (2007) with site period used for comparison.
FoS > 1.3	FoS > 1.1	FoS > 1.0[2]	Maximum allowable displacement = 0.5m
[1] PGA = 0.31g (design acceleration) [2]Any movement will be negligible (overall stability maintained)			

8.2.3 Seismic (Pseudo-static)

The mine site is located in an area of high seismicity compared to other regions of New Zealand. A number of active faults that are potential earthquake sources have been identified in close proximity to the mine (e.g., Porters Pass, Springfield, Hororata, Rockwood and Greendale Faults: GNS Active Faults Database). The

[1] Auckland Council – Code of Practice for Land Development and Subdivision – Section 2 Earthworks and Geotechnical Requirements. Version 1.6, 24 September 2013. Table 2.C.1 Factors of Safety.

Greendale Fault approximately 16km southeast of the CCM site produced the M7.1 2010 Darfield Earthquake. Despite the intense ground shaking at the CCM site, minimal damage was reported (BRL 2018).

An earthquake event for the final landform was analysed using a pseudo-static approach, in which a horizontal load (Peak Ground Acceleration (PGA)) is applied to the model to simulate the seismic loading.

The Peak Ground Acceleration (PGA) for the site has been derived using the MBIE/New Zealand Geotechnical Society – Earthquake Geotechnical Engineering Practice Module 1 method assuming a Site Soil Class of C and an effective earthquake magnitude of 6.3 suitable for the Darfield area. This method is consistent with the NZTA Bridge Manual (BM3).

Final landforms are not specifically referenced in AS/NZS1170.0:2002, however the landform is assumed to have an importance level of 1 (*'Low consequence for loss of human life, or small or moderate economic, social or environmental consequences'*) for this assessment to give guidance to possible design lifetimes and annual expected return periods. A level 1 structure with a 100 year or more design life is expected to resist earthquake loadings with return periods of 1:250 years. This return period equates to a PGA of 0.31g. A site specific hazard assessment was completed by Davis Ogilvie (2016) for the CCM site recommending a PGA of 0.23g should be used for slope stability analysis. The lower PGA reflects a shorter design life of 50 years.

For seismic stability either the factor of safety must be ≥ 1.0 or if FoS < 1.0 then permanent displacements must be less than 0.5m for a 250 year event design earthquake (e.g., Bray 2017). Estimates of permanent displacements from seismic loadings were assessed using simplified empirical model (Newmark sliding-block analysis) from Jibson (2007). In this type of approach, the yield acceleration (K_y: threshold ground acceleration necessary to overcome basal sliding resistance and initiate permanent slope movement) of a sliding mass is estimated by finding the average horizontal acceleration that results in a factor of safety of 1.0. The ratio of the average yield acceleration to the maximum seismic acceleration (K_y/K_{max}) is then used to estimate the displacement. Sliding block analysis was conducted using the United States Geological Survey (USGS) SLAMMER program.

8.3 Failure Mechanisms

The stability models assess the potential for instability through different failure mechanisms applicable to the final landform to determine those with the lowest level of stability. These include:

- Circular failure of engineered backfill;
- Circular failure through backfill and basal sliding along *in situ* foundation;
- Circular / slumping failure of upper Pleistocene soils and gravels;
- Planar failure along low strength bedding shear surfaces and associated structures within the *in situ* sedimentary sequence;
- Complex non-circular failure through *in situ* rock mass and backfill.

8.4 Slope Stability Results

The locations of thirteen cross sections that have been used to assess the final landform slope stability are illustrated on Figure 4. Stability analysis outputs for Cross Sections A to M are presented in Appendix A and summarised in Table 4.

8.4.1 Static case

Overall, slope analysis results indicate satisfactory levels of stability for the final landform slopes under design groundwater conditions. The minimum calculated FoS was > 1.4. The slopes can meet the adopted design criteria.

8.4.2 Elevated Groundwater Case

The effects of elevated groundwater pressure in the slopes of the final landform were undertaken by steepening the gradient of the piezometric line affecting both the backfill and in situ formations. In the elevated groundwater scenario, where there is potential for higher piezometric pressures the slopes are expected to perform satisfactorily and meet the stability design criteria. The minimum calculated FoS was > 1.2.

8.4.3 Seismic Case

The effects of a seismic load have been assessed for the long-term stability of the proposed final landform. The analysis results indicate that displacements could occur during the design earthquake (FoS < 1.0) and so an assessment of the seismic yield acceleration (Ky, the acceleration at which a FoS of 1.0 is calculated) has been undertaken for use in a displacement assessment. This assessment indicates displacement up to 0.1m could be expected for the 250 year event design earthquake. This level of displacement meets the adopted design criteria set out in Table 3 and the expected performance is therefore considered to be suitable for the proposed end land use of farming and forestry.

8.4.4 Sensitivity Analyses

The most critical geotechnical model parameters from a landform stability perspective for CCM are:

- Strength of backfill material; and,
- Elevated piezometric groundwater pressures.

Results of sensitivity analyses demonstrate for the possible range of input parameters the calculated FoS values can vary by ± 0.2.

Table 4. Factors of Safety for Canterbury Coal Mine Final Engineered Landform.

Cross - Section	Factor of Safety				
	Design Scenario (Static)		Seismic 1:250 year AEP (0.31g M 6.3)		Failure Surface
	Expected Groundwater	Elevated Groundwater	FoS[A]	Displacement[B]	
A	1.52	1.34	0.80 (K_y = 0.19g)	< 1 cm (5.3 cm)	~55m high fill slope. Circular failure of fill.
B	1.59	1.35	0.78 (K_y = 0.19g)	< 1 cm (5.3 cm)	~45m high fill slope. Circular failure of fill.
C	1.74	1.26	0.86 (K_y = 0.23g)	< 1 cm (3.3 cm)	~15m thick veneer of fill placed on sloping foundation. Basal sliding along interface.
D	1.74	1.36	0.83 (K_y = 0.21g)	< 1 cm (4.2 cm)	Fill placed onto original topo and mined surface (Fill buttressed into gully). Circular failure of fill.
E	2.23	1.63	1.0	0 cm	Fill placed on original topo. Complex failure through fill and basal sliding along in situ.
F	1.65	1.40	0.72 (K_y = 0.16g)	< 1 cm (8.0 cm)	Fill slope placed over original topo extending ~110m vertically. Circular failure of fill and in situ.
G	1.68	1.49	0.81 (K_y = 0.21g)	< 1 cm (4.2 cm)	~50m high fill slope. Circular failure of fill.
H	1.96	1.47	0.88 (K_y = 0.24g)	< 1 cm (3.0 cm)	~45m high slope of in situ with toe buttress of fill. Circular failure of fill.
I	1.58	1.32	0.78 (K_y = 0.18g)	< 1 cm (6.1 cm)	~40m high slope of in situ with toe buttress of fill. Circular failure of fill.
J	1.44	1.30	0.80 (K_y = 0.17g)	< 1 cm (6.9 cm)	~55m high slope of re-profiled in situ with toe buttress of fill. Complex failure.
K	1.59	1.45	0.84 (K_y = 0.21g)	< 1 cm (4.2 cm)	~55m high slope of re-profiled in situ with toe buttress of fill. Section parallel to strike. Circular failure of fill.
L	1.57	1.50	0.85 (K_y = 0.22g)	< 1 cm (3.7 cm)	~30m high slope of reprofiled in situ with fill at crest. Circular failure.
M	1.45	1.36	0.81 (K_y = 0.19g)	< 1 cm (5.3 cm)	~50m high fill slope. Circular failure of fill.

[A] K_y (g) = Critical (yield) acceleration calculated from horizontal acceleration that results in a factor of safety of 1.0

[B] Displacement method of Jibson (2007) using critical acceleration ratio and magnitude. Value in parentheses is method of Bray and Travasarou (2007) Ts=4H/Vs (200m/150 m/s = 1.33m/s).

9.0 Final Landform Settlement

As with any area where fill has been placed the CCM site will also be subject to some settlement or consolidation over time. Fill to be placed for the proposed engineered landform will be constructed from predominantly coal measures siltstone and sandstone. To achieve suitable fill strength and density, the fill will be placed in lifts of approximately 2m in thickness and truck rolled to achieve adequate compaction as per the CCM fill placement specification.

Limited literature is available for calculating self settlement of bulk rock fill. Zipper and Winter (1997) suggest that up to 0.4% self settlement can occur for compacted sandstone and up to 2% for loose siltstone. By assuming 0.5% settlement for the truck rolled fill, the settlement of the fill at the site is expected to range between 50 and 250 mm. Most of the settlement is expected to take place during construction and shortly after completion (~<12 months). Any settlement post construction, albeit very small, will pose no hazard to the safe use of the land.

10.0 Monitoring

The final landform will require monitoring as the landform is constructed and for a period post construction. While the proposed landform has been designed to appropriate standards to ensure long-term stability, monitoring is required to ensure that the slopes perform as designed. The landform should be inspected monthly during construction and then 3 monthly for a period of 12 months following completion. These inspections should check for foundation preparation prior to fill placement (install underdrainage if and when required), fill placement methodology, cracking, settlement, subsidence or slope failures and areas of water ponding. During construction, the landform should be surveyed with a drone on a monthly basis to similarly assess for signs of instability and conformance with slope design parameters. Any issues or deviations should be referred to the design engineer for risk assessment.

11.0 Conclusion

BCL have designed a final ELF that has slopes similar to the surrounding landform and are expected to provide a high level of stability over the long term. The ELF is to be built with a similar methodology to existing ELFs within the mine that have performed acceptably. In summary, the proposed ELF is expected to provide a stable landform for the expected farming and forestry land uses. We consider there is a low risk of future instability assuming the adopted design geometries and stability criteria are implemented. To assist in stability risk management a monitoring programme has been recommended including inspections, routine surveying and the application of TARPs to manage any deviation from expected behavoiur.

12.0 References

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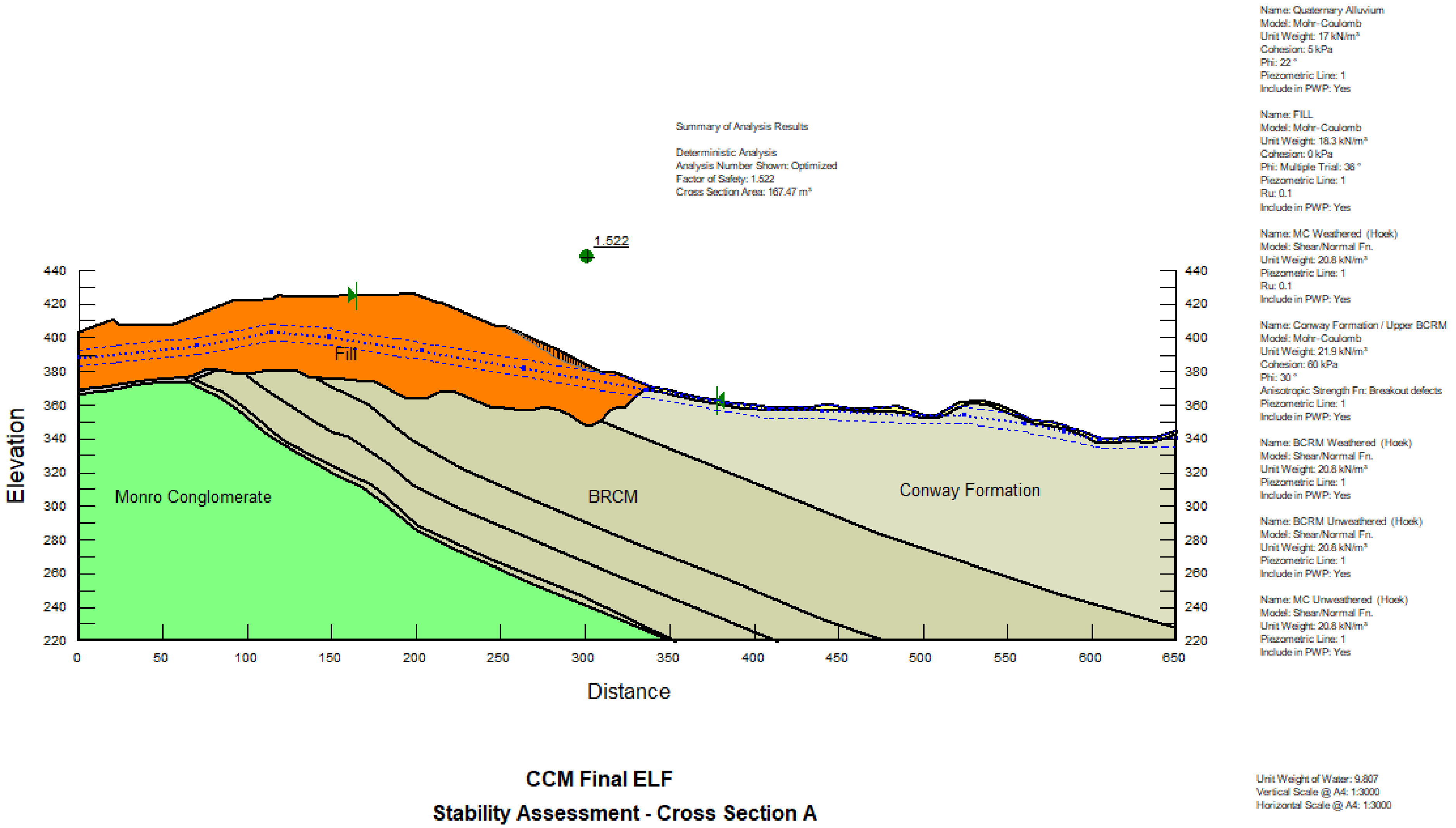
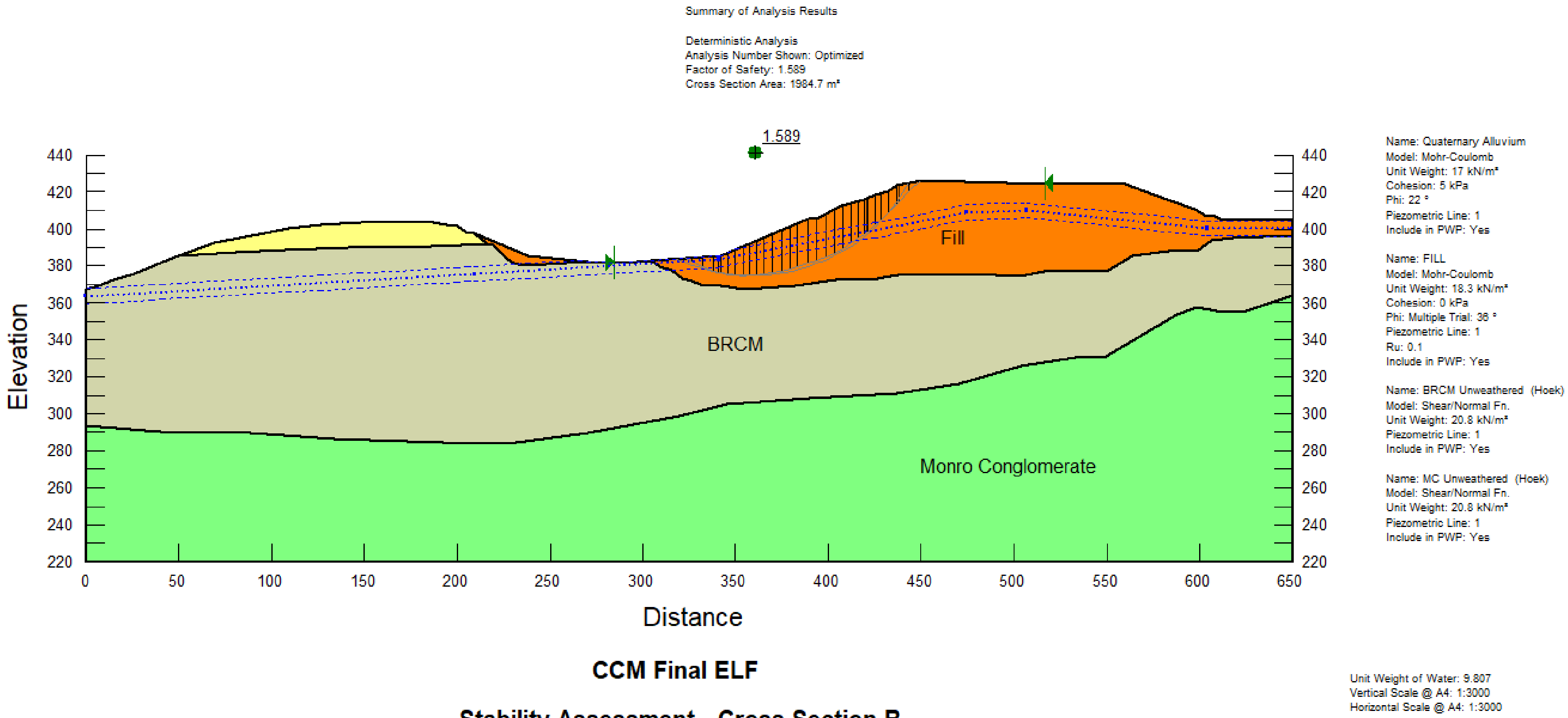
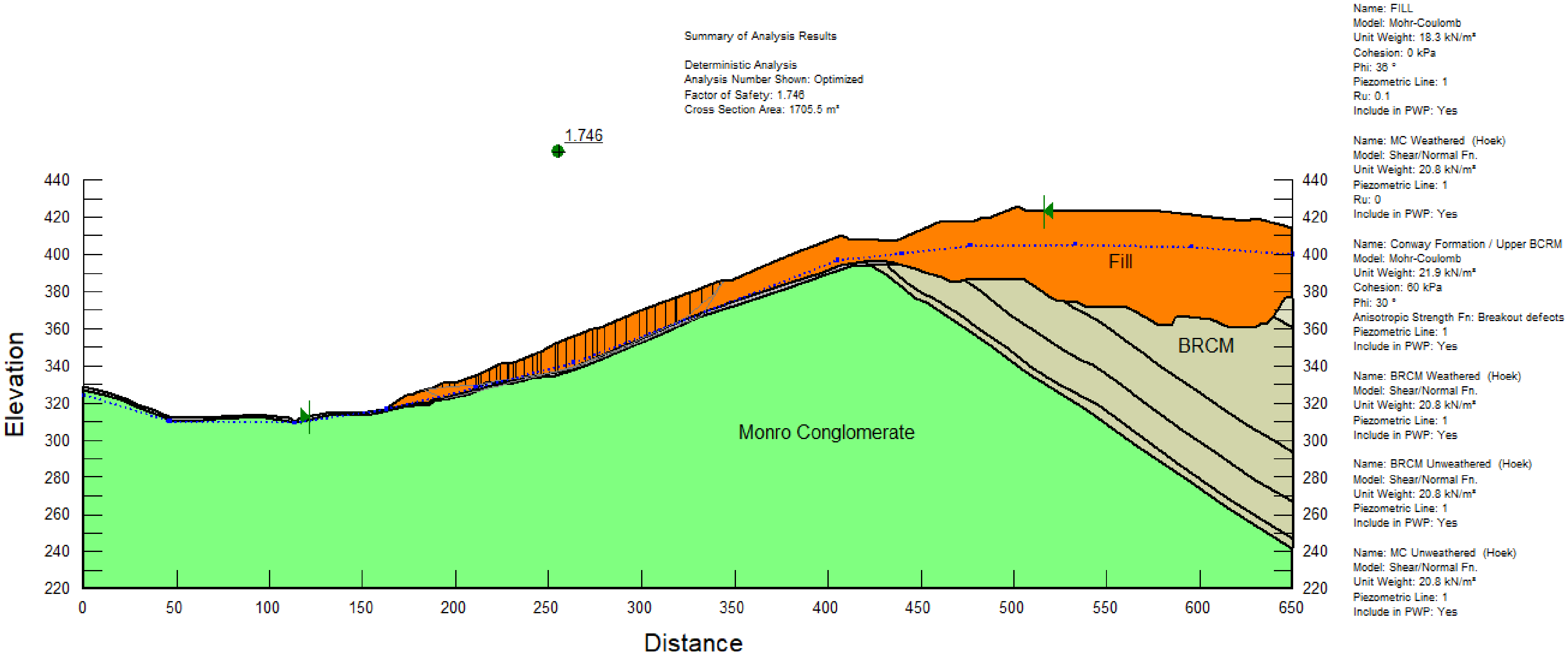
13.0 Appendix A – Slope stability analyses





CCM Final ELF

Stability Assessment - Cross Section B



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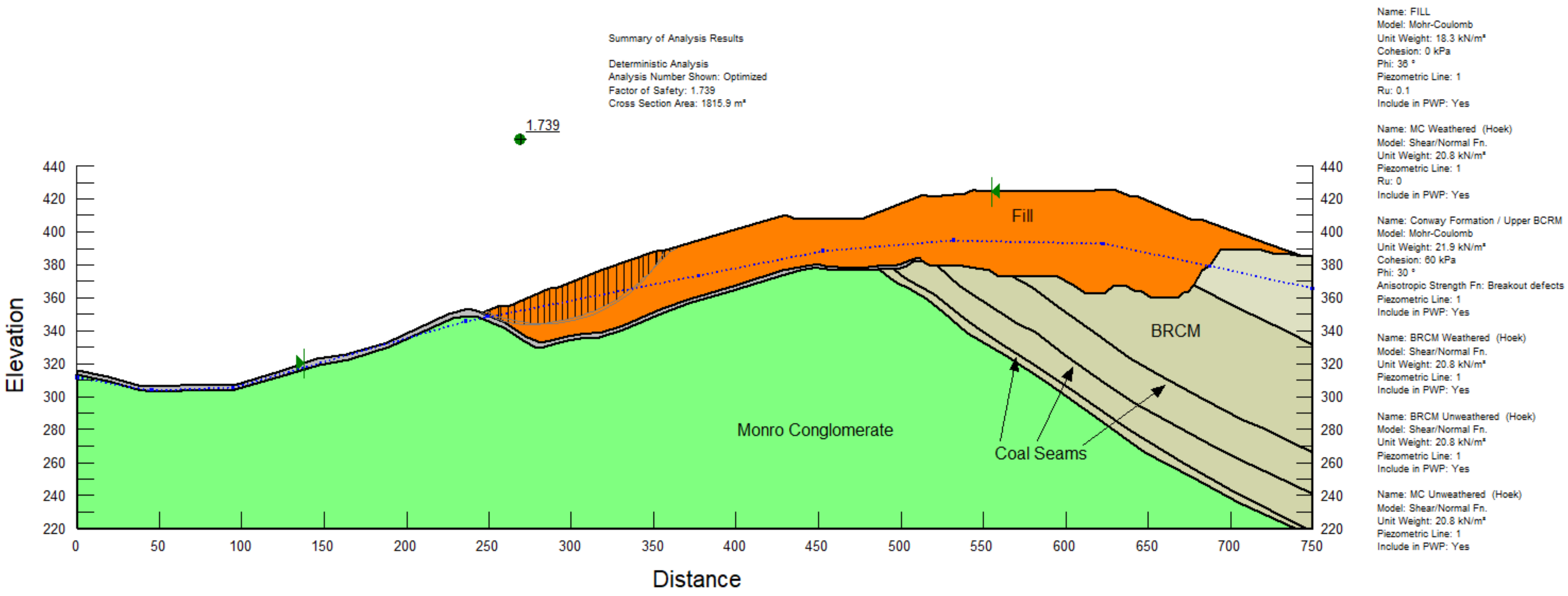
Horizontal Scale @ A4: 1:3000

CCM Final ELF
Stability Assessment - Cross Section C



CCM Final ELF

Stability Assessment - Cross Section D

Unit Weight of Water: 9.807
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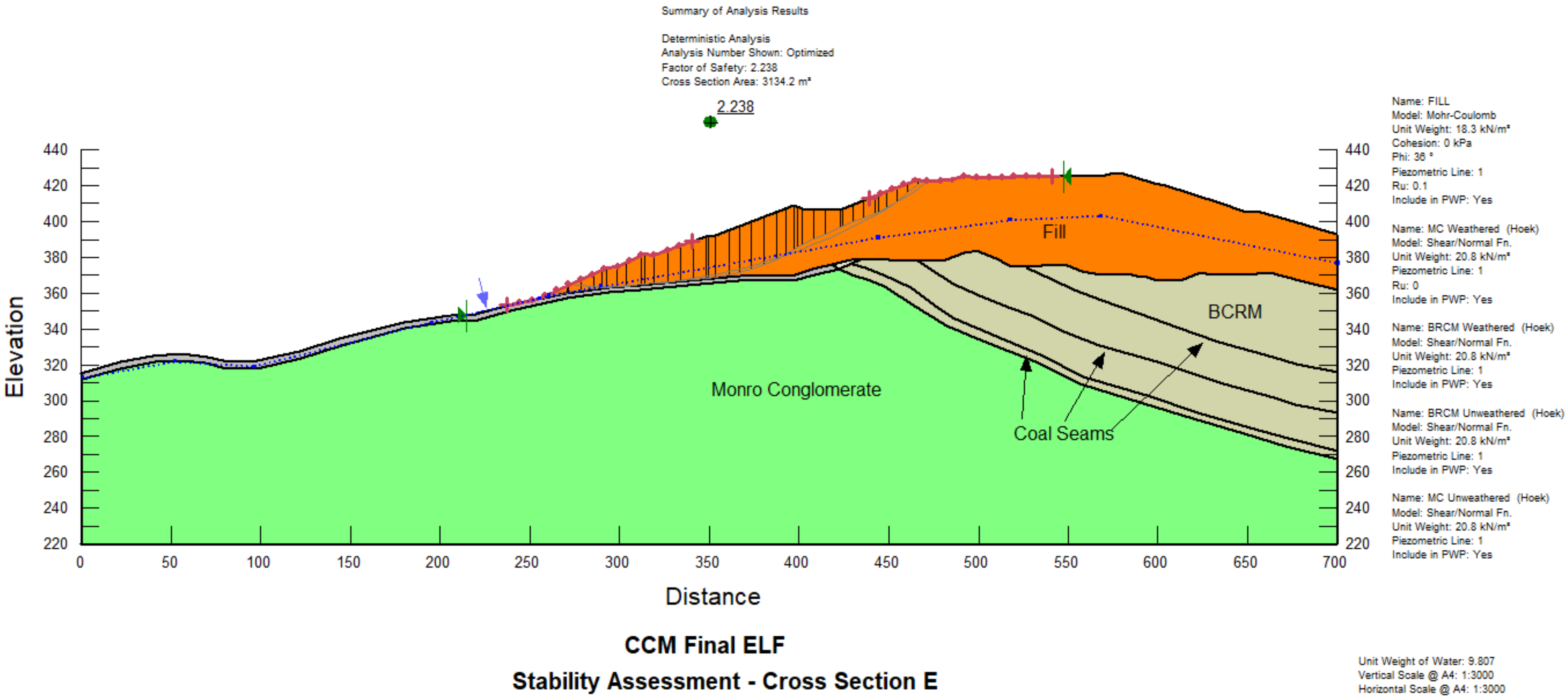
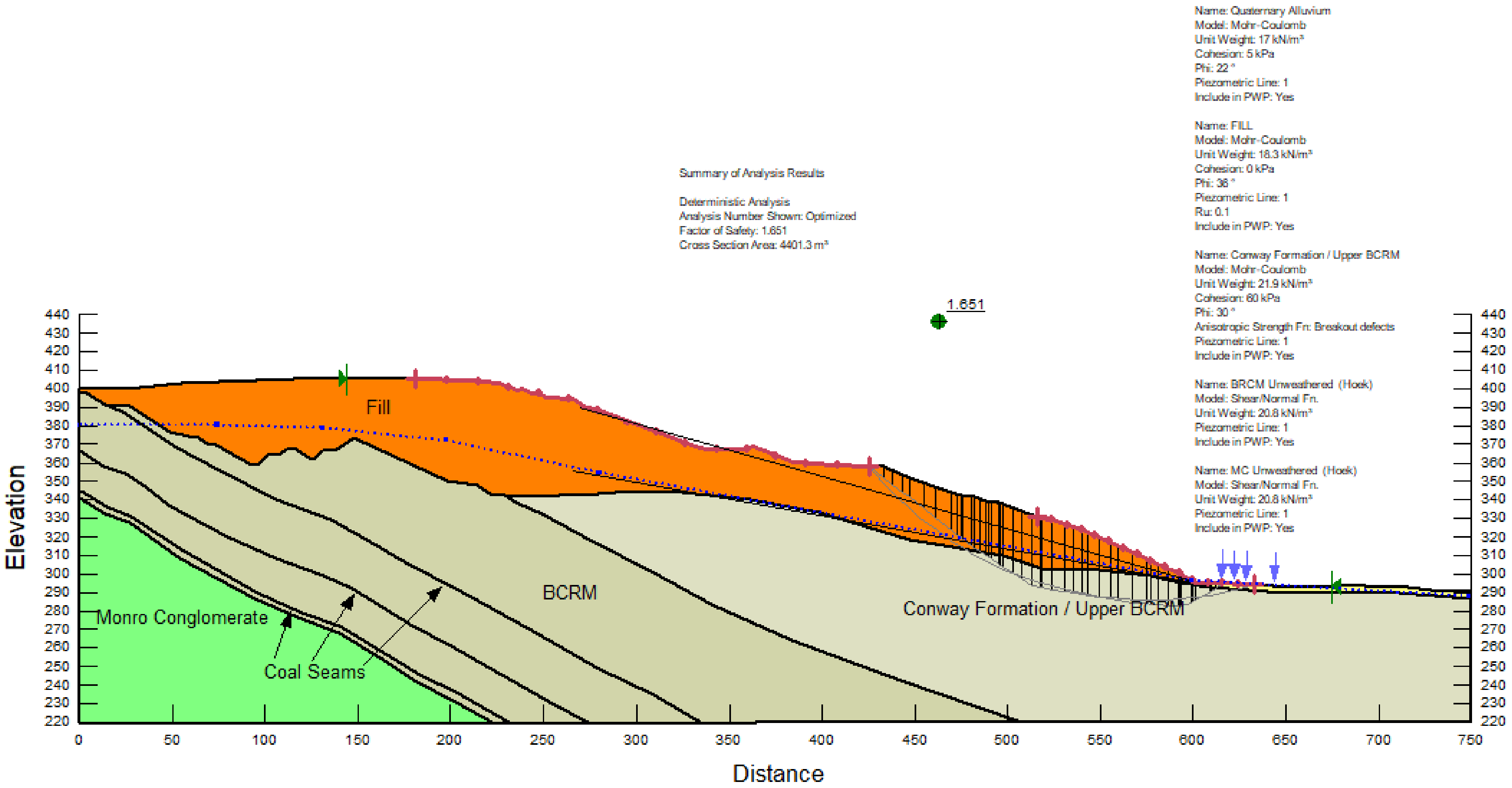
Horizontal Scale @ A4: 1:3000



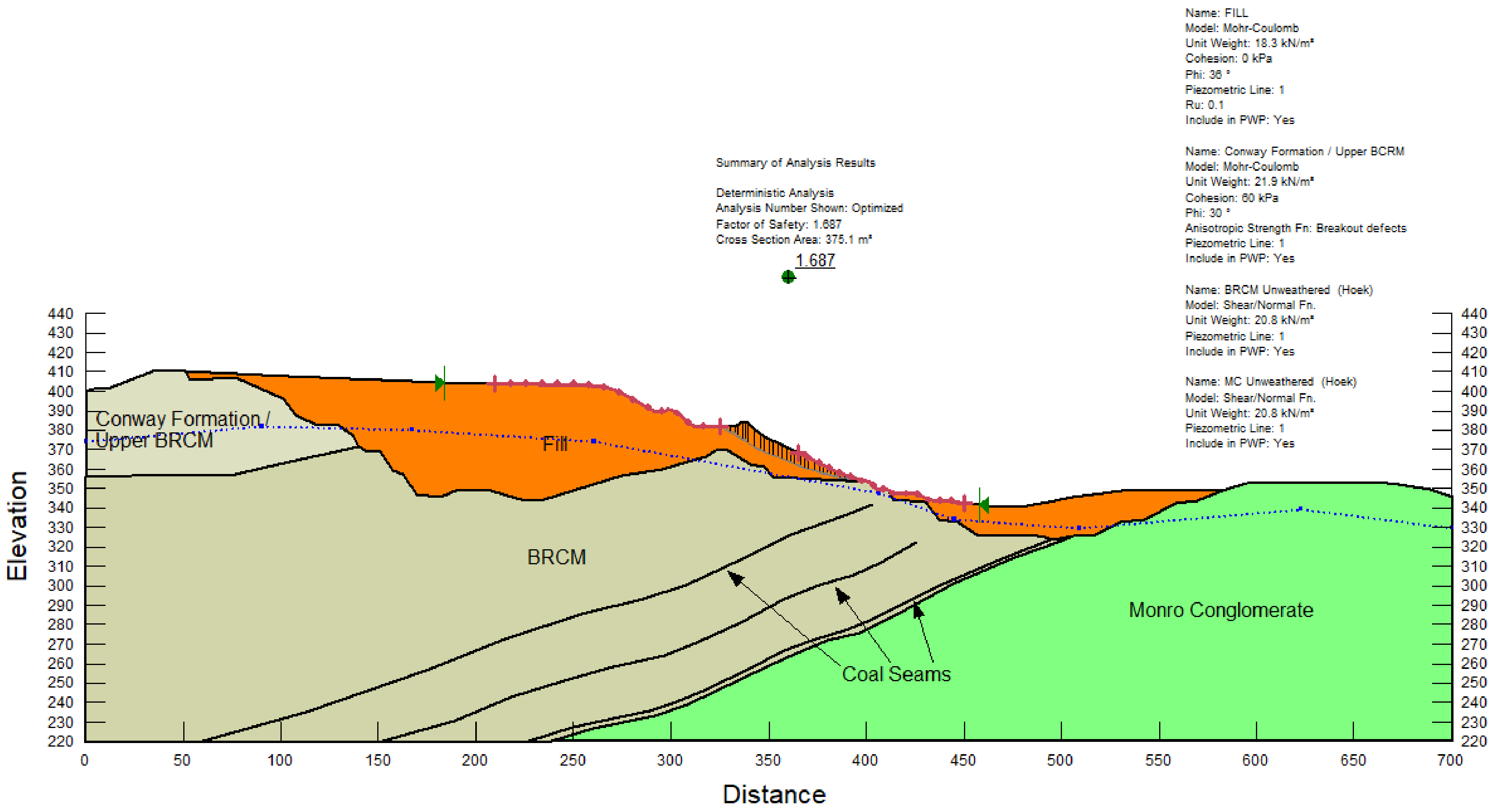
CCM Final ELF

Stability Assessment - Cross Section E



CCM Final ELF

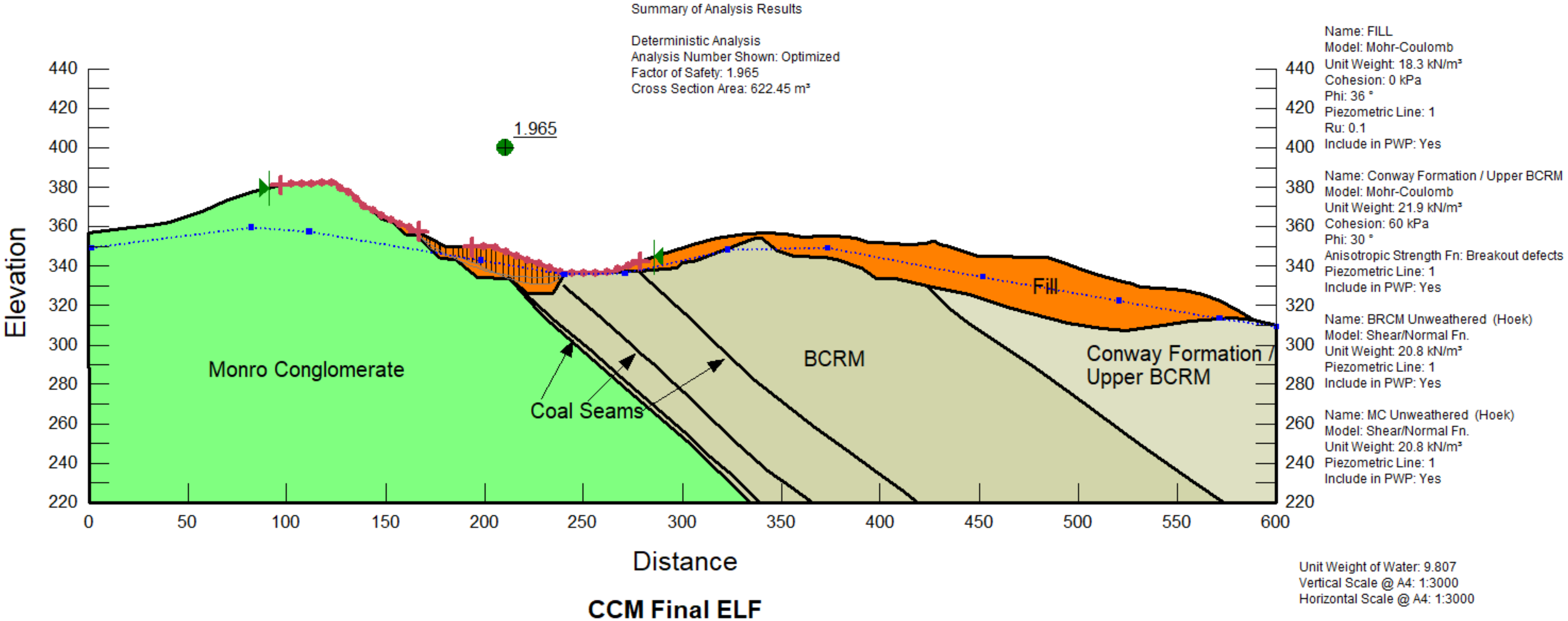
Stability Assessment - Cross Section F



Unit Weight of Water: 9.807
Vertical Scale @ A4: 1:3000
Horizontal Scale @ A4: 1:3000

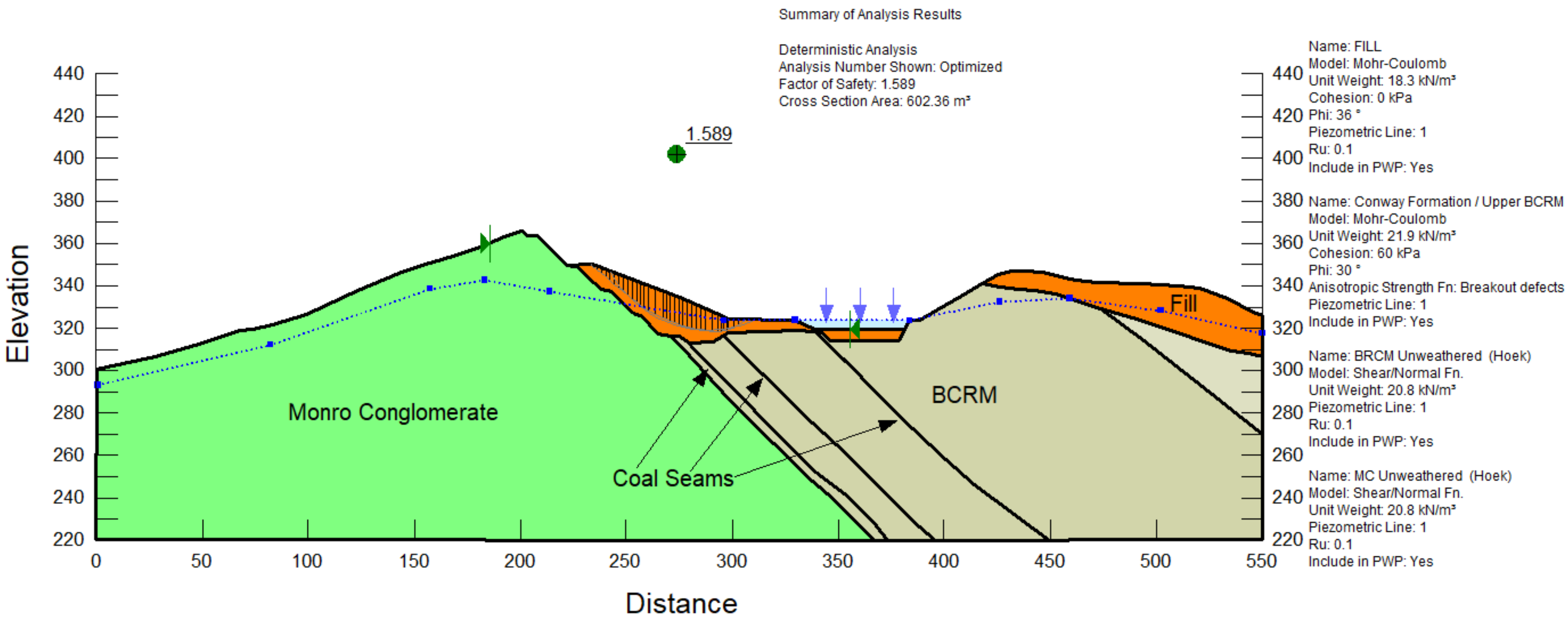
CCM Final ELF

Stability Assessment - Cross Section G



CCM Final ELF

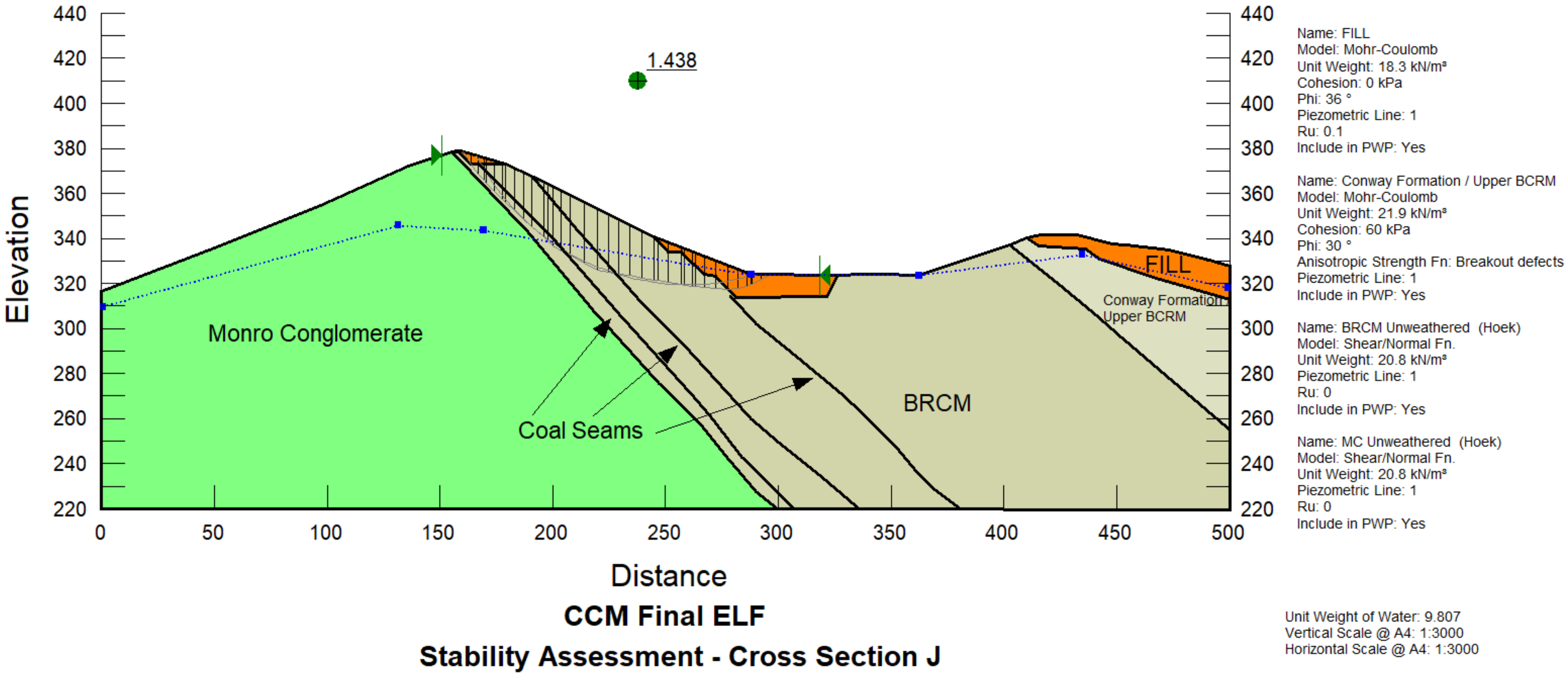
Stability Assessment - Cross Section H



CCM Final ELF

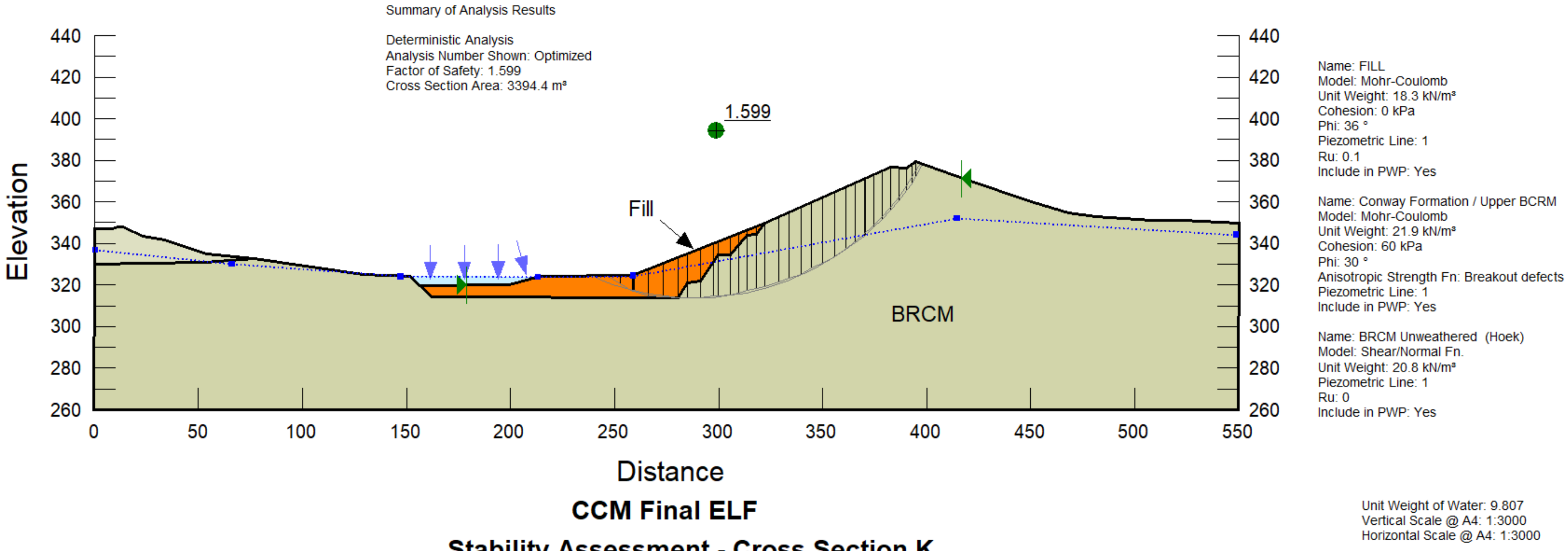
Stability Assessment - Cross Section I

Unit Weight of Water: 9.807
Vertical Scale @ A4: 1:3000
Horizontal Scale @ A4: 1:3000



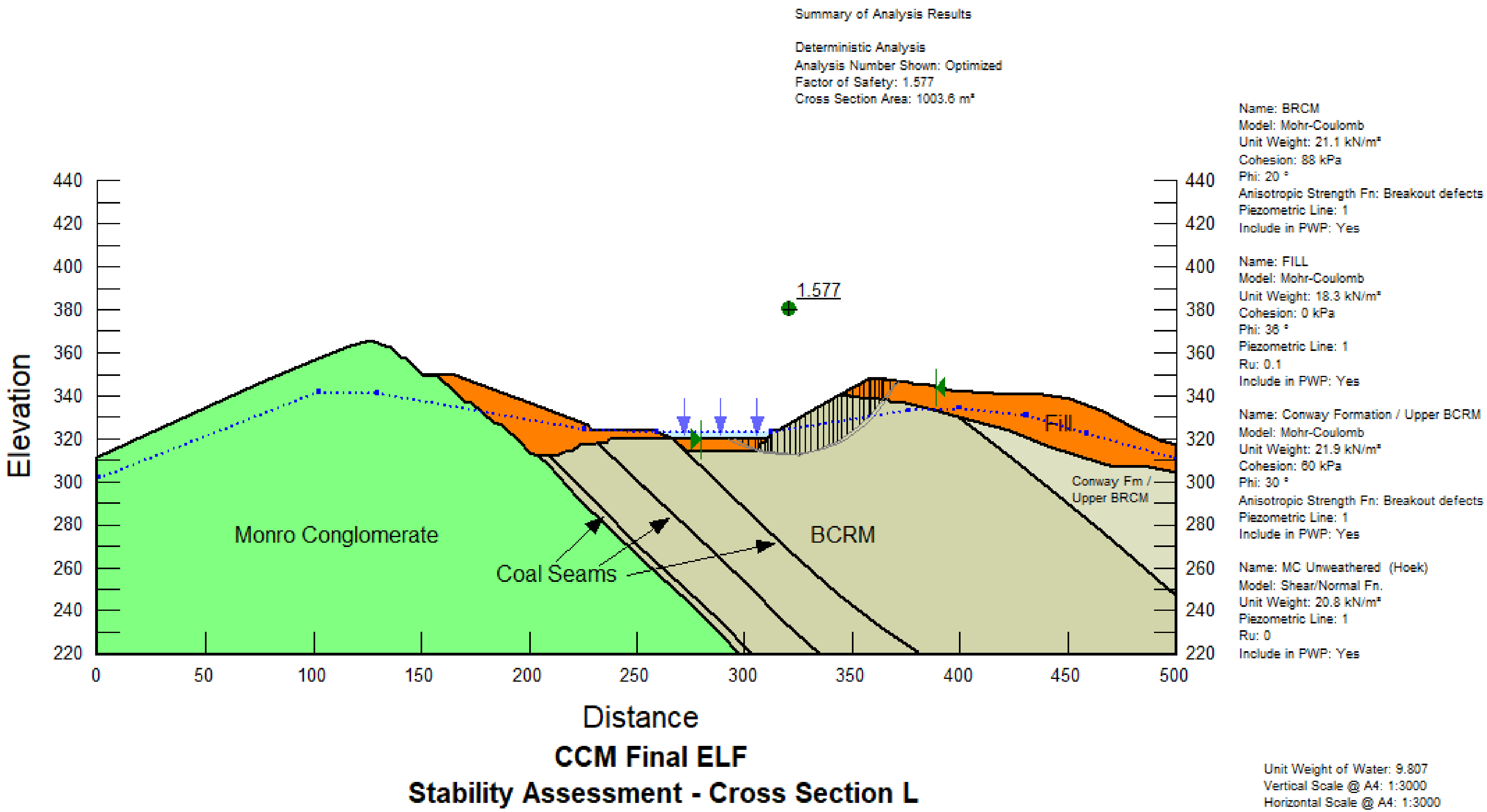
CCM Final ELF

Stability Assessment - Cross Section J



CCM Final ELF

Stability Assessment - Cross Section K

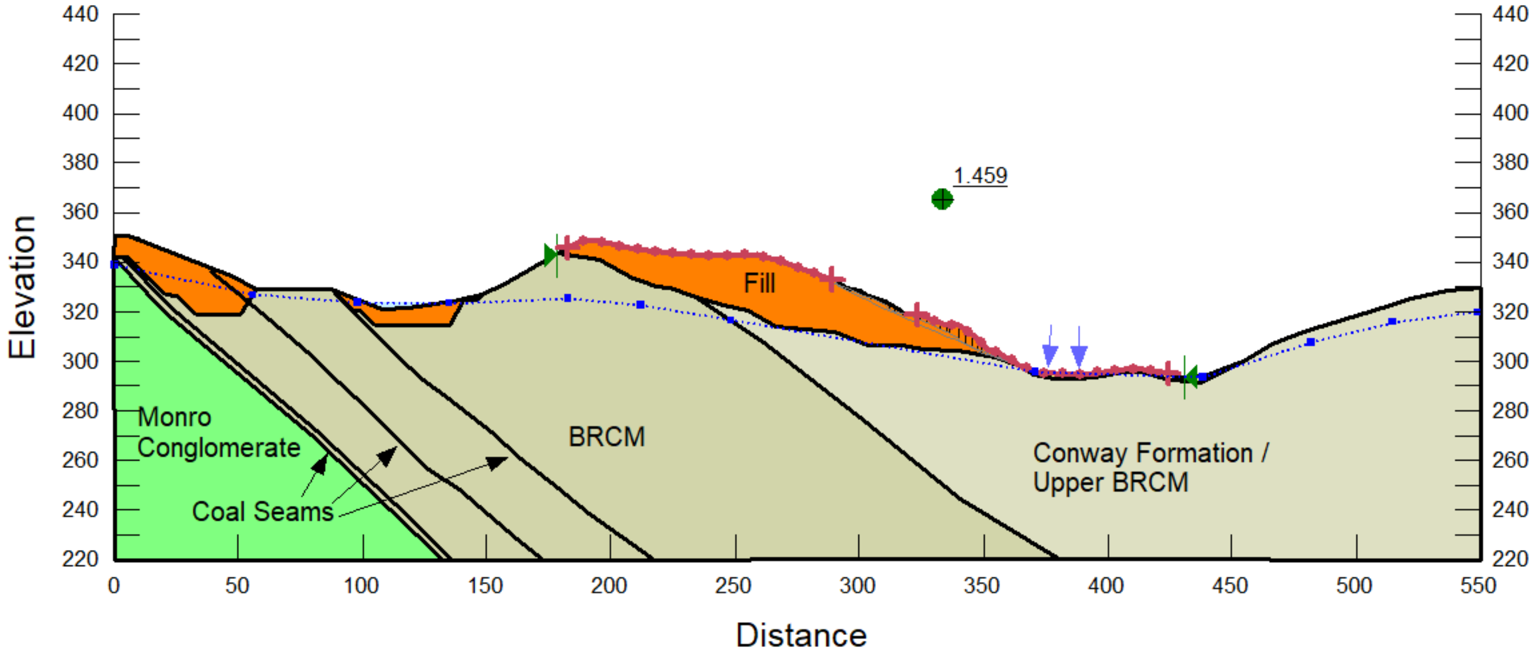


CCM Final ELF

Stability Assessment - Cross Section L

Summary of Analysis Results

Deterministic Analysis
Analysis Number Shown: Optimized
Factor of Safety: 1.459
Cross Section Area: 146.75 m³



Name: Quaternary Alluvium
Model: Mohr-Coulomb
Unit Weight: 17 kN/m³
Cohesion: 5 kPa
Phi: 22 °
Piezometric Line: 1
Include in PWP: Yes

Name: FILL
Model: Mohr-Coulomb
Unit Weight: 18.3 kN/m³
Cohesion: 0 kPa
Phi: 36 °
Piezometric Line: 1
Ru: 0.1
Include in PWP: Yes

Name: Conway Formation / Upper BRCM
Model: Mohr-Coulomb
Unit Weight: 21.9 kN/m³
Cohesion: 60 kPa
Phi: 30 °
Anisotropic Strength Fn: Breakout defects
Piezometric Line: 1
Ru: 0
Include in PWP: Yes

Name: BRCM Unweathered (Hoek)
Model: Shear/Normal Fn.
Unit Weight: 20.8 kN/m³
Piezometric Line: 1
Include in PWP: Yes

Name: MC Unweathered (Hoek)
Model: Shear/Normal Fn.
Unit Weight: 20.8 kN/m³
Piezometric Line: 1
Include in PWP: Yes

CCM Final ELF

Stability Assessment - Cross Section M

Unit Weight of Water: 9.807
Vertical Scale @ A4: 1:3000
Horizontal Scale @ A4: 1:3000