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Appendix E – Updated Geotechnical Report

MRM

McArthur River Mine

Updated Preliminary Geotechnical Assessment

NOEF

December 2017

Pando

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1 Introduction

A preliminary geotechnical assessment was completed to support an Environmental Impact Statement (EIS) as part of an approval process due to a change in waste classification. After the submission, a revision of the NOEF design has been completed in response to feedback.

Notable changes to the design are associated with the cover system: The Compacted Clay Layer (CCL) used to restrict infiltration into the NOEF has been replaced with a Geosynthetic Liner (GSL) to further decrease Net Percolation (NP); and the upper aspect of the trilinear slope has been relaxed by several degrees.

The implications on expected geotechnical performance have been assessed in this update.

1.1 Scope and objectives

This assessment aims to address the following criteria for the NOEF:

- Assessment of overall long-term stability;
- Assessment of proposed construction methods, their impact on stability, and identification of areas where opportunity may exist for improvement;
- Identification of limiting areas requiring further assessment; and
- Recommendation of monitoring for validating the performance of the facility against the model.

2 NOEF Assessment

2.1 Material types

The materials used in construction of the NOEF are described below using a classification system based on the governing design criteria.

2.1.1 Alluvials (foundation)

The foundation of the NOEF has been sampled for a range of purposes, including investigation and construction of the CW (NOEF) stage, exploration for clayey alluvium for use in CCL manufacture, and characterisation for groundwater assessment. The foundation alluvials are typically comprised of red-brown clay of moderate to high plasticity. Minor sand and gravel lenses have been noted in some of the sampling programs. The alluviuals range in thickness from 1m to 16m. The thicker zones are encountered in the eastern extent.

2.1.2 Overburden rock

The overburden rock used to form the facility is categorised using a geochemical classification system (MRM/KCB 2014) based on the reactivity of the material when exposed to air and water. As the design of the facility is fundamentally defined by geochemical considerations, the internal structures and layout are broadly described with geochemical designations. The current classification subsets the overburden rock into five categories:

- Low Salinity Non-Acid Forming rock (High Capacity) [LS-NAF(HC)];
- Metalliferous Saline Non-Acid Forming rock (High Capacity) [MS-NAF(HC)];
- Metalliferous Saline Non-Acid Forming rock (Low Capacity) [MS-NAF(LC)];
- Potentially Acid Forming rock (High Capacity) [PAF(HC)]; and
- Potentially Acid Forming rock (Reactive) [PAF(RE)].

All classes except the LS-NAF(HC) are regarded as environmentally non-benign, with only the LS-NAF(HC) considered benign. Only benign materials may be left exposed to the external environment in the long-term without being capped. Note that alluvial material is classified as LS-NAF(HC). The zones in the NOEF (Figure 6) where these materials may be placed is summarised in Table 4.

The geotechnical classification of the rock overburden consists of two categories: breccia; and shale, and are further described below:

2.1.3 Shale

"Shale" is a term used at MRM to describe well bedded siltstones and mudstones that preferentially shatter along bedding and jointing planes. They are typically fine grained, well bedded, jointed and fissile sedimentary rocks, comprised primarily of dolomite, with bituminous material that has variable amounts of micro-crystalline pyrite laminae. The life-of-mine (LOM) material balance for overburden rock is dominated by Shale.

The lithostratigraphic units that belong to the Shale group include: Upper Dolomitic Shale (UdH), Upper Pyritic Shale (UpH), Black Bituminous Shale (BbH), Lower Pyritic Shale (LpH), Lower Dolomitic Shale (LdH) and W-Fold Shale (WFS). Rock from these units form most of the overburden materials designated for storage within the encapsulated sections of the NOEF – though WFS is typically LS-NAF(HC).

Figure 1 is a photograph of MS-NAF waste rock as tipped and dozed (but not graded).

Figure 1 An example of MS-NAF "Shale" rock

Geotechnical characterisation of the shale overburden by field or laboratory testing is difficult due to the size and range of clast sizes and variable composition of clast material types. Historically, material strength criteria has been based on industry experience with similar materials (URS 2008), and described with Mohr-Coulomb strength criteria as shown in Table 1.

Table 1 Shale overburden historic strength parameters (URS 2008)

Material	Unit Weight (kN/m^3)	Friction Angle deg.	Cohesion kPa
All Waste Rock	20	38	0

To improve understanding, large scale direct shear tests were completed at two university testing laboratories; the University of Queensland (UQ) and the University of Newcastle (UNC). A range of materials to be stored in the OEF's were tested with dry (UQ and UNC) and inundated (UNC only) testing methods. There is a clear indication that friction angles in wet samples are slightly lower compared with those obtained for dry samples (UNC tests only), though the difference between the dry and wet test results were not significant in most cases. Further discussion of the testing and results is presented in Section 3.2.6.

2.1.4 Breccia

The term "Breccia" is used to describe clastic sedimentary rocks composed of angular to sub-angular, randomly oriented clasts of predominantly dolomites and shales cemented with a carbonate matrix. The Breccias constitute the hardest and most competent rocks in the MRM sequence. The results from the large scale direct shear testing (UNC) did not show a discernible difference in shear strength between the shale rock and breccia rock. However, testing is expected to underestimate the material strength of the Breccia due to sample preparation where larger clasts are scalped or resized to fit into the apparatus.

With a high proportion of large clasts, the test results reported from breccia samples are potentially an underestimate and therefore conservative. Breccia shear strengths are therefore likely representative of upper bound rock strength estimates. For stability modelling purposes by comparison, strength parameters for shale have conservatively adopted the lower bound strength envelope for shale waste and mid bound for breccia. The testing results and adopted strength criteria are discussed in Section 3.2.6.

Figure 2 provides an example of weathered upper breccia rock sourced from within the mine immediately beneath alluvial cover.

Figure 2 An example of MRM "breccia" rock.

2.1.5 Compacted clay liners

A compacted clay liner (CCL) is an engineered layer constructed from selectively sourced materials onsite that conform to the criteria outlined in Table 2 (MRM 2015). The Draft and Supplementary EIS Project Description proposes the use of a CCL as a low permeability foundation layer in some areas of the NOEF base (i.e. where natural materials do not meet the requirements), as well as the barrier layer in the WOEF cover system.

Table 2 Geotechnical specification for the basal CCL

Property	Requirement
Maximum Hydraulic Conductivity (Ksat) (AS 1289 6.7.3.5.1.1)	1×10^{-9} m/s
Size of largest particle	*Not greater than 75mm*
Minimum % by weight passing 37.5 mm AS 1152 sieve	*90%*
Material Classification (AS 1726:2017)	*ML, CL, CI, MH, CH*
Minimum Plasticity Index (AS 1289.3.3.1)	*15%*

Note: The objective for this specification is to achieve a Ksat lower than 10^{-9}m/s. PSD and plasticity specifications are intended to guide material selection as a practical field acceptance criterion only.

The materials used to develop CCLs must not be significantly dispersive under the effects of anticipated leachate waters. To manage the risk of dispersive clays being used in inappropriate zones, ongoing testing of clay sources is required to be conducted using waters that are representative of the likely solute concentrations encountered within the final landform. Typically, pinhole dispersion test results are classified as ND1 or ND2.

Uniformity of the CCL is important to its performance. To manage conformity, rigorous testing of both the source and liner material will be performed to the sample frequencies outlined in Table 3**.** The sampling frequencies will be revised (increased or decreased) upon review of CCL construction performance.

Table 3 Indicative geotechnical testing frequency for CCL construction

Test Type	Clay Liner Borrow Area Frequency	Placed Clay Liner Frequency
Particle Size Distribution **Atterberg Limit, including Linear Shrinkage**	1 test per 5,000m^3 or Soil Material Change	1 test per 20,000m^3
Emerson Class **Chemical Analysis (Exchangeable Sodium Percent and Sodium Absorption Ratio)** **Pinhole Dispersion**	1 test per 10,000m^3 or Soil Material Change	1 test per 20,000m^3
Moisture Content and Dry Density Ratio		1 test per 500m^3
Permeability/Hydraulic Conductivity		1 test per 10,000m^3

In conjunction with the selection and preparation of CCL materials for use as a barrier layer in selected cover systems, a construction methodology will be adopted that forms a stable and uniform subgrade prior to placement of the CCL; i.e. a uniform, well-graded, dense and compacted surface with limited depressions, voids and protrusions. This will thereby provide a stable base upon which to accurately construct the CCL layer.

MRM have developed a proven methodology (MRM 2015), located several onsite clay sources, and are maintaining a QA/QC register for CCL manufacture and installation as part of the current NOEF construction.

Internal advection barriers, formed from fine-grained alluvials, will be placed during construction. These barriers serve to impede the flow of oxygen into the overburden.

2.1.6 Geosynthetic liners (GSLs)

Following further work including improved modelling and the resolution of some engineering uncertainties, an important change in the cover system design to that described in the Draft EIS submission is the replacement of the cover system CCL with a GSL to provide an even lower permeability barrier to air and water infiltration. For the purposes of this assessment the GSL is a heavy duty bituminous geomembrane (BGM) liner. A GSL is favoured for its extremely low hydraulic permeability, mechanical strength and resistance to penetration from plant root systems. Figure 3 and Figure 4 are examples of a BGM GSL.

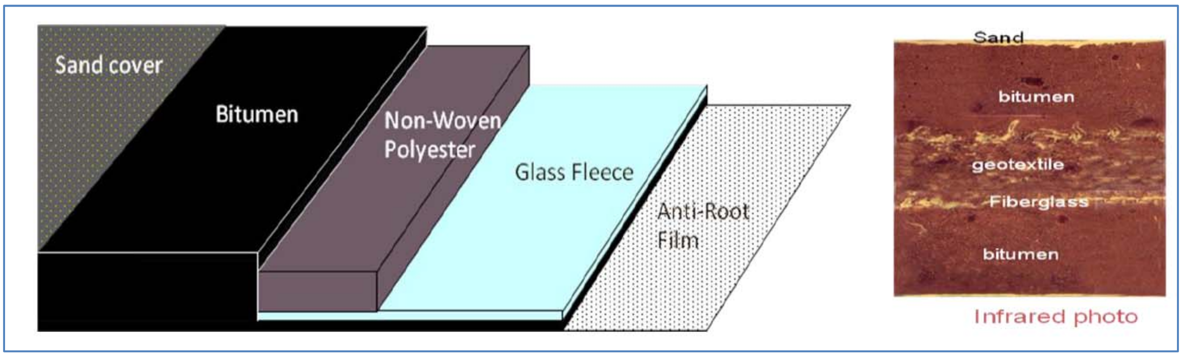

Figure 3 BGM section showing component layers (e.g. ES 3 BGM Coletanche[R])

Figure 4 Coletanche[R] ES4 BGM

2.2 NOEF design and construction methodology

The design and construction sequence for the NOEF was devised after numerous geochemical, hydrological, geotechnical and geomorphological studies, technical workshops, assessments and consideration of various design options.

2.2.1 Stage sequencing

The NOEF is staged with 7 phases progressively developing in a clockwise direction from the current Central West (CW) facility. Advection barrier layers, consisting of fine alluvial material covered by a layer of MS-NAF, are placed at the internal interface of each stage, and additional horizontal layers may be installed to reduce oxidation over wet seasons or if stages of the dump will be dormant for extended periods of time.

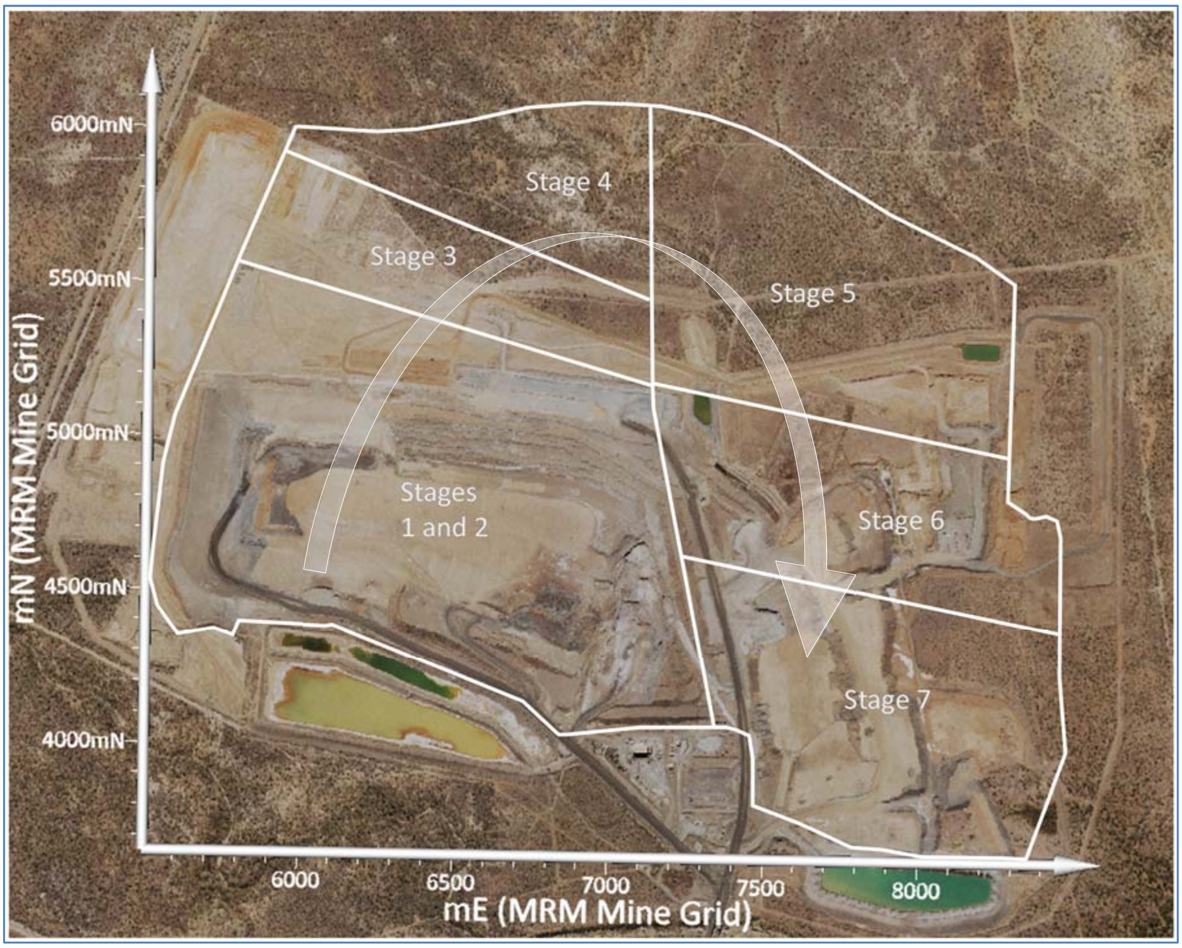

Figure 5 NOEF design with indicative stage boundaries (2016 aerial photo background)

The staging allows progressive rehabilitation, so the performance of each stage can be assessed and monitored as a final landform, enabling any improvements to be remedied and incorporated into subsequent stages.

2.2.2 Design

The key design aspects relevant to overall geotechnical design pertain to the overall shape, internal layout of materials and structures designed to manage hydraulic flow.

The layout (shown schematically in Figure 7) consists of:

- A maximum height of 140m;

- A “trilinear slope” profile, where the gradient of the slope progressively steepens with height in three stages, to simulate the mature profile of a naturally evolving landform based on the following configuration:
    - A revised flatter 3.0H:1V “Upper” slope → slope height (H) = 40 m, horizontal length (L) = 120 m;
    - 3.5H:1V “Mid” slope (unchanged) → H = 50 m, L = 175 m; and
    - 4.5H:1V “Lower” slope (unchanged) → H = 50 m, L = 225 m.

It has a cover system characterised by:

- A GSL as a barrier layer. For the purposes of this study a heavy duty bituminous geomembrane (BGM) approximately 5mm thick has been considered.
- A “cushioning layer” between 200 and 300mm thick that may be placed directly over the GSL to protect it during construction. The nature of the material used for this purpose is subject to field trails. On the slopes, the cushioning layer is required to prevent puncture from the breccia, however it is not expected to persist, with the breccia clasts working into the layer during the construction process and over time.
- For the batters, the GSL and cushion layer will be overlain by 1.1m to 1.2m of breccia rock and 0.1m of topsoil, with these overlying layers designed to be free draining. The total thickness of benign materials above the GSL is a minimum of 1.5m.
- The inwardly draining plateau will consist of a 0.2 - 0.5m thick drainage layer, then 0.6 - 0.9m of growth media and 0.1m topsoil above. The total thickness of benign materials above the GSL is a minimum of 1.5m.
- A drainage network that would be built into the cover system to convey peak flows from the dump surface down to the toe of the facility, limiting flow over the slopes to their immediate catchment.
- The cover system barrier layer is tied into the low permeability foundation at the toe of the slope to restrict ingress by flood waters.

Figure 6 is a sketch describing the evolution of the cover system on the slopes.

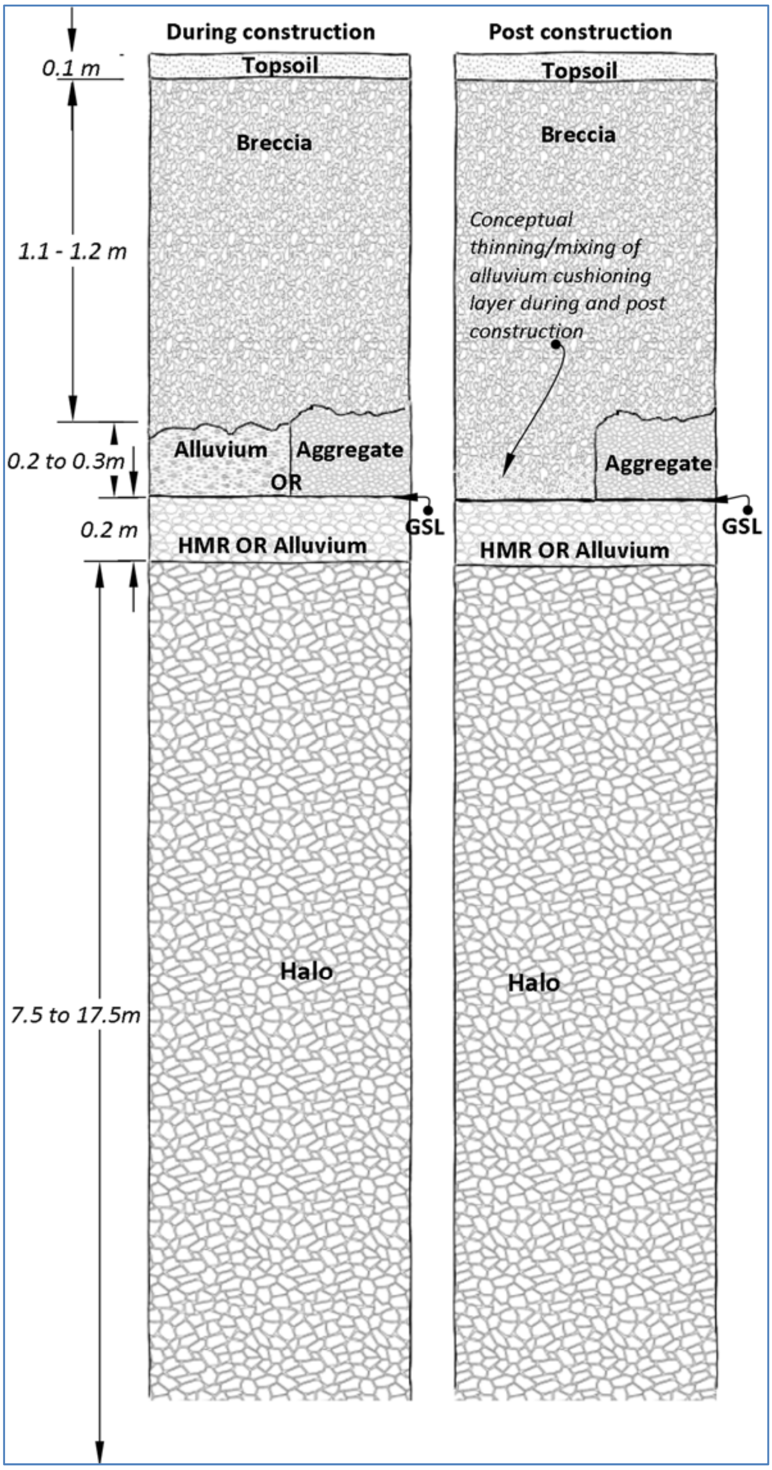

Figure 6 Cover system design options for slopes, with the sketch on the left representing the construction layout and the sketch on the right representing a final or long**-term construction**.

The facility's internal structure, encapsulated by the cover system (as indicated in Figure 7), is comprised of:

- A nominal, 5-20m thick "Halo zone" of MS-NAF materials placed around the Core.

- A Core zone that will store PAF(HC) and MS-NAF(LC) or better materials with advection limiting construction, such as paddock dumping or in lifts of no more than 7.5m high with fine-grained advection barriers.
- PAF(RE) cells tipped in 2m lifts with lower permeability advection barriers on each lift, located towards the centre of the NOEF to limit oxidation and gas movement, and assist in restricting fluid movement.
- A Base zone, constructed as an approximate 5m thick layer of MS-NAFs to provide a stable base of lower geochemical reactivity below the Core.

Details of the foundation are listed here:

- The foundation has a role to enhance migration of infiltrating seepage waters to report as toe seepage rather than basal seepage, and remove uncontrolled preferential pathways that do not assist the underdrain network.
- If insufficient in-situ low permeability natural foundations are encountered, a 0.25 m thick basal CCL will be constructed to inhibit seepage through the base.
- The foundation will feature a network of lined drains, graded to direct drainage paths to seepage collection points.
- To be geotechnically stable, in particular near the toe of the facility, low strength materials will be identified and replaced as required. Figure 7 provides a three-dimensional schematic section of the NOEF used to form the geotechnical stability assessment models. It is considered a “worst case” scenario from a stability point of view, as it includes:
- An alluvium advection layer between the Halo and Core zones. It is expected that this layer will only be placed throughout a small portion of the NOEF. It will be installed if construction of the cover system falls behind the development of the core, to limit oxygen ingress into the core until the cover system installation catches up.

Note that in the stability sections (Figure 14 to Figure 18), the basal CCL and MS-NAF base is not shown as a discrete layer for simplicity, as the foundation alluvium has been assigned CCL-like properties, and the Base assigned with Core properties.

The material types used in each zone are summarised in Table 4.

Table 4 Material types permitted in NOEF zones

Zone	Waste Classification	
	Preferred	**Also allowed**
BASE	MS-NAF(HC) MS-NAF(LC)	Alluvium LS-NAF(HC)
CORE	PAF(HC) MS-NAF(LC)	Alluvium MS-NAF(HC)
PAF(RE) Cell	PAF(RE)	PAF(HC) MS-NAF(HC) MS-NAF(LC) Alluvium
HALO	MS-NAF(HC)	Alluvium MS-NAF(LC)
BATTER COVER	LS-NAF(HC)	Alluvium (only for cushioning layer above GSL)
PLATEAU COVER	Alluvium	LS-NAF(HC)

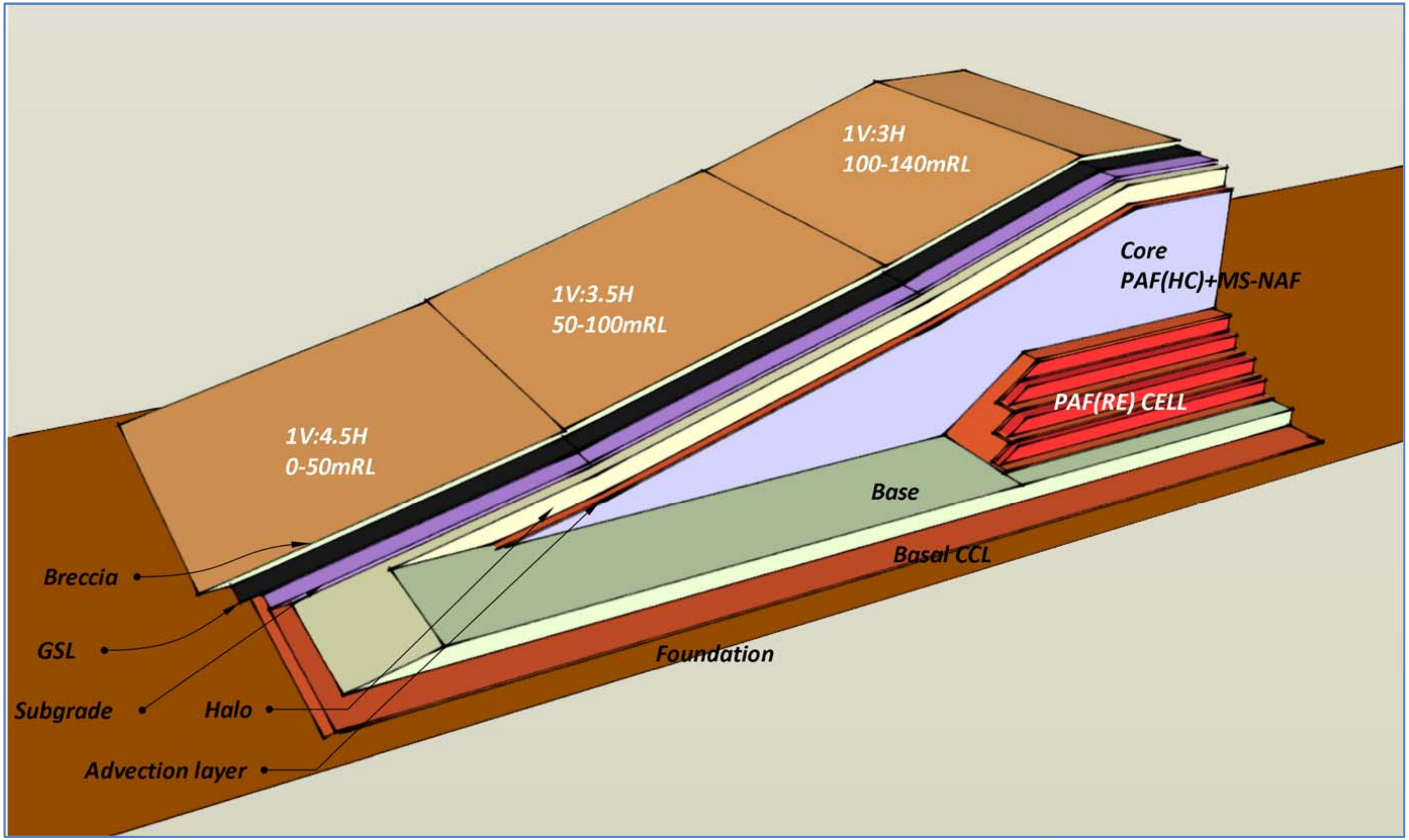

Figure 7 Generalised stability section geometry with key features

2.2.3 Construction methodology

There are four fundamental construction methods proposed that may affect the strength and hydraulic properties of the material:

1. Direct tip (over a tip head): Material is dumped from height over a tip head. This technique has the effect of vertically size grading material, resulting in an increase in material strength and permeability with depth as coarser and more mechanically competent material finds its way to the base of the pile. The degree of particle segregation is proportional to the tip height; therefore, low tip heights will have little to no particle segregation.
2. Direct tip and doze (including paddock dump): Material is tipped in a single lift (approximately 2m high) and dozed level. This forms a relatively even particle grading of material with truck rolled compacted surfaces that form a horizontal layering that is finer, thereby creating a media with lower hydraulic permeability that potentially impedes fluid flow vertically.
3. Tip, doze and grade: A similar process to direct tip and doze with grading and more accurate survey control to prepare a subgrade for liner placement or final landform. This method may also utilise the addition of selected material, such as alluvials, to improve the performance of the subgrade. Drains, high traffic routes and CCL subgrade will typically use this method.
4. Prepare and place: This process requires subgrade preparation to facilitate placing of an engineered layer i.e. GSL.

Specific construction specifications include:

- For the NOEF base, low tip heads (5m) and/or direct tip and doze will be used for the construction;
- For the NOEF core, two potential construction methods will be used, as they afford equivalent geochemical controls over oxygen ingress:
    - low tip heads (5.5m) over a direct tip layer (ca 2m) to form a 7.5m lift, in conjunction with regular advection layers at every lift.
    - Direct tip and doze in low lifts with no advection barriers.
- For PAF(RE) cells, all PAF(RE) will be placed by direct tip and doze in low (ca 2m) lifts with advection barriers on every lift.
- The Halo will be constructed in lifts as required by the construction of the cover system, to a maximum of 7.5m lifts.

The batter cover system requires careful construction as follows:

- Shaping each slope aspect to the required grade prior to placement and rolling a subgrade such as alluvium or heavy media rejects (HMR). HMR is a crushed rock of 20 to 40mm, produced as a by-product of the crushing and heavy-media separation process. The subgrade media is placed and rolled to form a consistent surface without excessive protrusion prior to unfurling the GSL.

- After placing and joining the GSL, a cushioning overliner layer may be required to prevent subsequent puncturing during Breccia placement. The irregular clast size and angular nature of the breccia has the potential to penetrate the GSL during placement with the additional load of machinery.
- The cushioning layer may consist of alluvium, an aggregate from screened LS-NAF(HC) or a geotextile. It is expected that the breccia will settle into the cushioning layer. The requirement for and nature of the cushioning layer will be finalised during optimisation of the construction process as part of the field trials schedule for 2018.

The plateau cover system is constructed with:

- A rolled approximate 200mm thick subgrade placed directly over the Halo rock, and profiled away from the crest, to slope towards the drainage network.
- A continuous GSL layer placed directly over the subgrade and connected to the slope GSL layer.
- Nominal 300mm cushioning layer (e.g. alluvium or aggregate) to protect the GSL from the coarser drainage layer material.
- Each subsequent layer of the cover system (i.e. drainage layer, growth media and topsoil) is placed and graded to ensure consistent thickness and grade is maintained across the plateau.

2.2.4 Surface drainage and the Cover System

The NOEF surface drainage system design is an important requirement to ensure the long-term function of the landform. The drainage system needs to have enough capacity to prevent water "backing up", or mounding in response to the design rain event, in areas where generation of excess pore pressure could destabilise sensitive structures within the NOEF. For slope stability the drainage considerations are:

- Adequate crest drainage to prevent water ponding at, or near, the crest of the slope for extended periods. This would reduce the risk of elevated pore pressures developing near the crest, which have the potential to destabilise the cover system of the slope.
- The cover system media is of sufficient capacity to manage the store and release of water along the slope, to limit excessive runoff that could scour the slope.

The drainage system of the NOEF has been devised to limit the crest catchment, by directing runoff to engineered drainage lines. Inter-slope drainage is also limited, as the main trunk drains traverse obliquely down the slope along the old haulage ramps, limiting the overall area of NOEF slopes that are exposed to runoff from the entire length of the slope.

The drains are carefully engineered to ensure long-term performance. They will be rip rap armoured, and lined with a very low permeability geosynthetic liner system to enhance long-term integrity and limit infiltration into the underlying strata via the drain.

The cover system and growth media used for the plateau and batters is also designed to limit erosion and enhance long-term stability. Erosion modelling completed by O'Kane Consultants (O'Kane 2016[1]) assesses two materials suitable as growth media, namely alluvium and breccia.

Of the two growth media materials, alluvium is susceptible to excessive erosion and suitable only for flatter areas such as the plateau. Breccia is well suited for all slopes with significantly lower potential for erosion, but with a lower likelihood for vegetation establishment and evapotranspiration.

Erosion has the potential to compromise the integrity of the NOEF and expose the GSL to potential damage or degradation, thereby allowing excess water to infiltrate the facility. Given this sensitivity, and considering that fire could temporarily remove vegetation on the NOEF, a conservative approach is adopted where only breccia is used on slopes and alluvium is only to be used on the plateau. O'Kane Consultants (O'Kane 2017) have designed batter cover systems of thicknesses ranging from 1.5m (supplementary EIS) to 2.1m (Draft EIS) above the barrier layer in consideration of 1:1000 year rainfall scenarios.

3 NOEF Stability analyses

3.1 Standards and guidelines

In the absence of recognised standards in Australia for mine overburden storage structures such as the NOEF, previous studies (URS 2008, UQ 2016) have utilised ANCOLD's tailings dam embankment guidelines for minimum design criteria. These standards have been adopted for this analysis.

Table 5 Design Criteria

Stability Assessment Case	Assessments Stability Assessment Case Minimum Acceptable Factor of Safety
Long-term static (drained)	Min FOS ≥ 1.5
Short-term (extreme wet season)	Min FOS ≥ 1.3

3.2 Shear strength parameters

3.2.1 Cover system materials

The majority of the material for use in the cover system is of the Breccia type. Breccia is regarded as geochemically stable, free draining with limited fines and highly resistant to erosion. However, given the slope cover is a fundamental component of the system protecting the GSL from erosion, a mid-bound overburden rock shear strength envelope has been adopted (Figure 8) for the analysis. As the Halo zone is constructed from similar materials to the Core, it is assigned a conservative lower bound shear strength envelope (Figure 8).

3.2.2 Geosynthetic liner (GSL)

A GSL is planned to act as the barrier layer between the cover system and NOEF waste rock. The GSL acts as a hydraulically impermeable layer, preventing rainfall infiltration into the rock mass of the NOEF. There are a range of GSL products available to suit the application. For the mining environment a heavy-duty membrane with a high mechanical resistance is recommended.

For the purposes of analysis, a heavy-duty bitumen based geomembrane (BGM) such as the Coletache^R ES 3 product has been selected. BGM has viscoelastic properties (afforded by the bitumen) and will conform to the materials it is placed within, with mechanical properties provided by a non-woven polyester geotextile and glass fleece reinforcement.

With a propensity to conform to the underlying and overlying materials, the BGM does not readily offer a discrete independent plane of weakness; rather, its behaviour is a function of the interface between the underlying and overlying media with the BGM.

Direct shear interface testing with the various materials proposed to be in contact with the BGM, namely the cushioning layer (screened aggregate and/or alluvium), Breccia and HMR, and the BGM have not been conducted on specific MRM samples. However, published testing from projects with similar subgrade and overburden materials describes interface shear strengths that are typically higher than the adjacent material, i.e. the overlying or underlying material shear strength will determine overall cover stability (Lew et al 2013). For this assessment, shear strength parameter values of c'=5kPa and friction angle of 30^o have been applied for the interface between the BGM and adjacent materials. Further works will be completed in 2018 to confirm these parameters in conjunction with the constructability trial pad works for the various GSL liner configurations.

3.2.3 Natural alluvium

Based on the previously described construction methods, triaxial test results from the 2013 Golder investigation (Golder 2013) have been adopted as representative of the strength characteristics of the foundation.

3.2.4 Advection and basal compacted clay layers

MRM have a proven site capability in constructing and installing CCLs at a large scale, as evident in the current CW NOEF operations. Basal CCL strength parameters have been adopted from the current iteration of the MRM CW NOEF Operations Manual (MRM 2015).

Advection layers will be created from site sourced clayey alluvium materials, but may have a higher sand and silt content than CCL material. They will be up to approximately 1.2m thick, traffic and/or dozer compacted, then covered by approximately 1.5m of MS-NAF material to limit erosion and diffusion of the Core. They are not intended to be impermeable but are designed to restrict air flow. For the stability analysis, advection layers have been assigned the same values for shear strength parameters as the basal CCL.

3.2.5 Bedrock

The underlying bedrock consists of weathered to weakly weathered rock identified as the Cooley and Reward Dolomites and Barney Creek Formation dolomitic shales. Bedrock strengths have been adopted for consistency with an estimate of overall bedrock strength derived from 2008 site investigations.

3.2.6 Overburden rock

Studies conducted prior to 2016 have used widely recognised generic values for competent mine spoil rock using a Mohr-Coulomb approach to characterise shear strength with values of 38 degrees for the effective internal angle of friction and zero for effective cohesion, with a unit weight of $20kN/m^3$.

To improve the understanding of the shear strength parameters of MRM overburden rocks, direct shear testing was undertaken in 2015, using large scale direct shear box apparatus at the University of Newcastle (UNC) and the University of Queensland (UQ). MRM commissioned Prof. D Williams of UQ to report on the testing and recommend suitable parameters for use in stability assessments of the expanded NOEF (UQ/Williams 2016).

Of note, is the observation by Williams that the results of the shear box testing show no discernible difference by material type. However, Williams recommended application of a range of material strengths based on setting and sensitivity. Table 6 and Figure 8 summarise the UQ recommended Mohr-Coulomb shear strength parameters from the results of both UNC and UQ testing programs.

Table 6 UQ recommended strength parameters

Application	Apparent cohesion	Friction angle
Near the surface	50±25kPa	40±3 degrees
Within the overburden facility	100±50kPa	35±3 degrees
Rock/CCL interfaces	20±10kPa	33±3 degrees

For the purposes of NOEF stability modelling, lower bound parameter estimates have been adopted to identify areas of potential instability and conservatively allow for parameter uncertainty (see Table 7 for adopted parameters).

For applications where the construction life is limited or where the rock mass is actively monitored and managed, such as working interfaces or in-pit facilities, a mid-range curve is adopted.

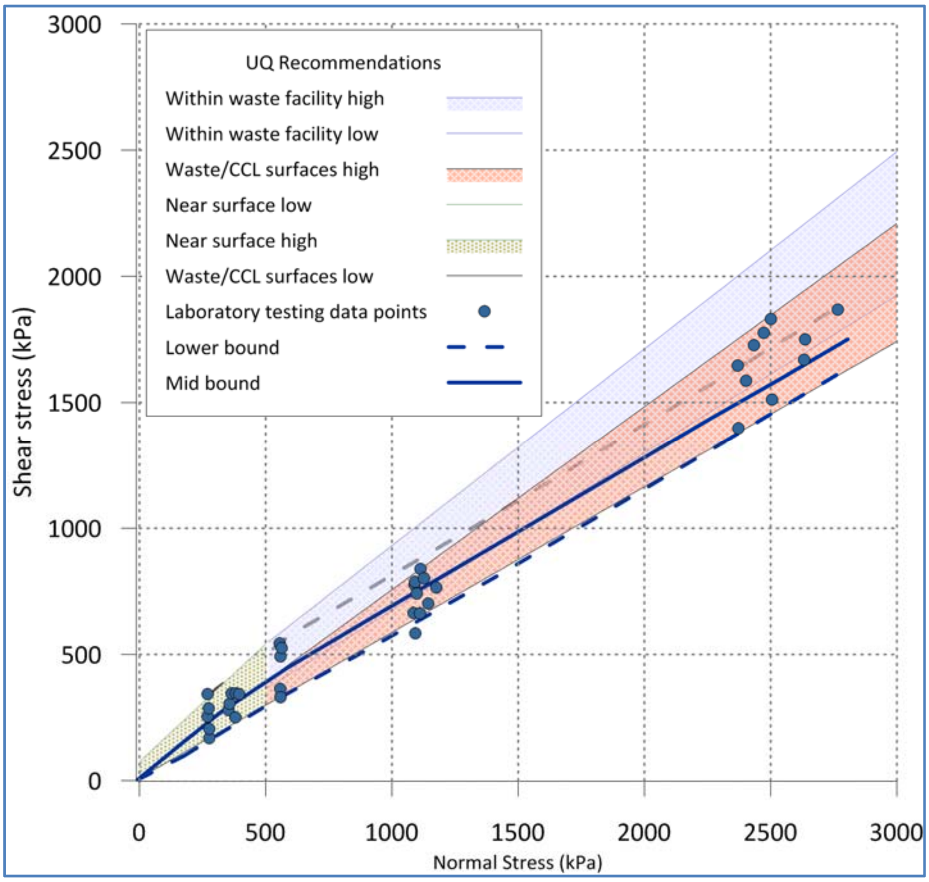

Figure 8 UNC and UQ data points with recommended shear strength envelopes

3.2.7 Summary of adopted parameters

The adopted parameters are presented in Table 7 with scenario specific values presented on modelled results in Appendix A.

Table 7 Summary of adopted parameters for stability analyses

Material Type (designation)	Unit Weight (KN/m^3)	Cohesion (kPa)	Friction Angle (degrees)	Source
Breccia – cover	20	UQ mid-bound shear strength function		UQ 2016
Breccia – cover (sensitivity cases)	*20*	*0*	*40*	
GSL Interface (BGM)	12	5	30	Estimate (Lew et al 2013)
Alluvium under liner	18	5	30	UQ 2017
Basal CCL drained	18	10	22	Golder 2013/GHD 2015
Advection layer	18	10	22	Adopted as conservative after GHD 2015
Natural ground	21	10	22	Golder 2013
Weathered bedrock dolomite	20	100	35	URS 2008
Overburden (NOEF)*	20	UQ lower-bound shear strength function		UQ 2016

** The overburden material type includes all broken rock material sourced from the mine, including HMR. Termed in models with descriptors PAF Cell, Core, Central West, Halo, PAF (RE)).*

The overburden rock, once emplaced, may undergo weathering effects over time. Degradation of minerals could weaken the strength, but precipitation products may cement and strengthen the mass (UQ/Williams 2015). However, the facility is specifically designed to limit infiltration of water and gas. Should weathering occur, it is expected to be spatially variable. Selection of a lower bound strength envelope captures the potential degradation weathering may cause, but ignores any potential improvement due to cementation.

Shear strengths for alluvial based materials (CCL, advection layers, and foundation alluvials) have been adopted from previous studies. Interface shear strengths for GSL have been estimated from published laboratory testing with similar materials to those proposed for the cover system (Lew et al 2013).

3.3 Pore pressure characterisation

An understanding of the transient behaviour of pore pressure is required to assess the NOEF slope stability, as changes in pore pressure can affect material shear strength and therefore slope stability.

3.3.1 Conceptual pore pressure model

The GSL and associated cover system is designed to maintain a growth media and limit water infiltration into the overburden core. For the underlying groundwater system, restriction of infiltration by the cover systems GSL significantly reduces the gradual rise of groundwater levels (i.e. mounding) beneath the facility, as a significant volume of water that would normally percolate through is shed from the facility via down-gradient foundation drainage lines. However, with the infiltration reduction, the cover system is required to manage large volumes of water in intense rain events. This introduces an elevated risk of pore pressure within the batter cover system.

Potential development of pore pressures in the waste dump as a result of rainfall percolation or flooding was investigated by O'Kane Consultants (O'Kane 2017[3]) in association with the cover system design (O'Kane 2016[2]). The O'Kane conceptual model and parameter estimates have been adopted and incorporated into this revised stability assessment.

A schematic of the conceptual model for the NOEF batter is provided in Figure 9 to simply outline the conceptual behaviour of rainfall during a wet season event. It is shown that rainfall saturates the cover. Once saturated, any additional water reports to surface runoff, flowing down slope to the ground or intercepted by the lined haul ramp drains. Within the NOEF beneath the GSL, suction pressures are expected to progressively develop.

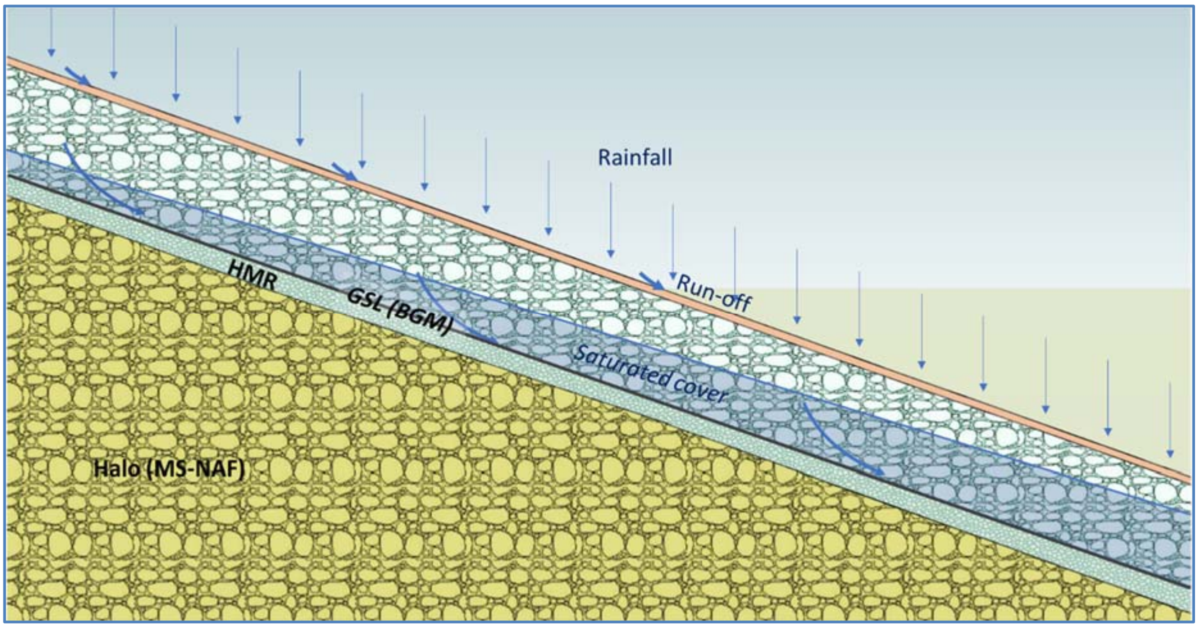

Figure 9 Conceptual model of GSL (BGM) and cover system of the slope during the wet season

At its foundation, the majority of the NOEF is underlain by thin (<4m) layers of alluvium, except for the eastern extents where significantly thicker zones of clay (up to 15m) have been encountered (Golder 2015). The alluvium, comprised predominately of silty clays, may act as a hydraulic barrier for any water percolating through the NOEF. If unable to drain via the underlying groundwater system, elevated pore pressures may develop at the base, which if allowed to build significantly at the toe, could pose a risk for localised instability at the toe. The foundation drainage system is designed to mitigate this potential mechanism.

The groundwater system beneath the facility is not expected to mound in the long-term, as recharge via vertical infiltration into the facility will be very limited under the expected cover system performance. The construction of the sub-soil drainage system beneath the NOEF will reduce the potential for a build-up of water in the base. There may however be short-term mounding during construction until the cover system is in place, and therefore it is important that the foundation drainage system remains functional during the construction and dump drain-down periods, but less important once the GSL is in place and ongoing water infiltration is limited. The following figure, based on the KCB NOEF Unsaturated Flow modelling works completed for the Supplementary EIS, shows that drain-down is expected to take approximately 20 years following completion of the cover system.

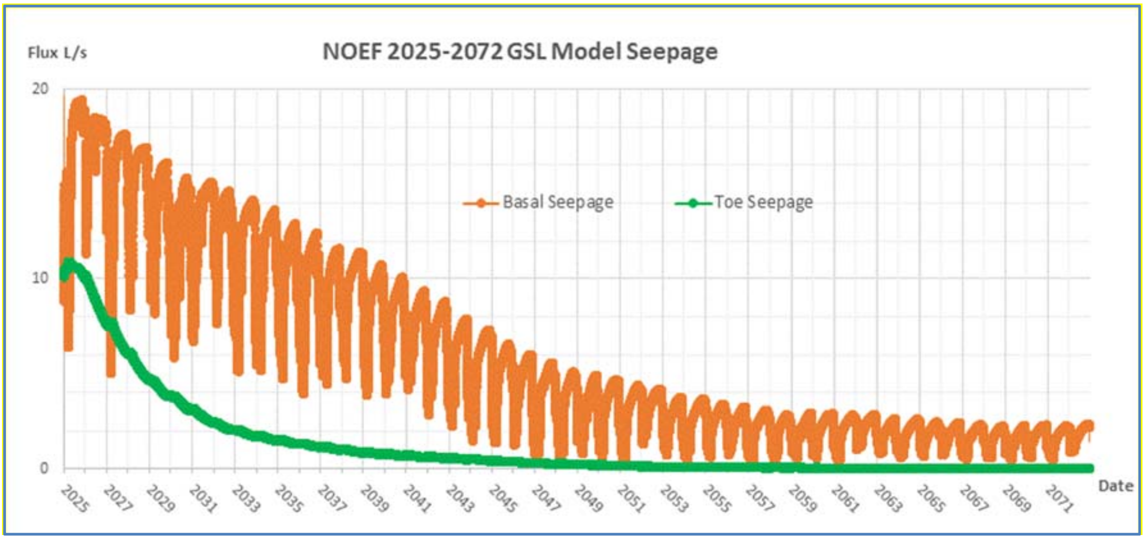

Figure 10 Modelled foundation seepage (KCB 2017)

To assist in determining the risk of toe instability in the event that the under-drainage ceases to function, KCB (KCB 2017) modelled a scenario where the foundation drains clog immediately after completion of the facility (2040). Figure 11 shows the predicted mounding in 2042, 2 years after the drains have clogged.

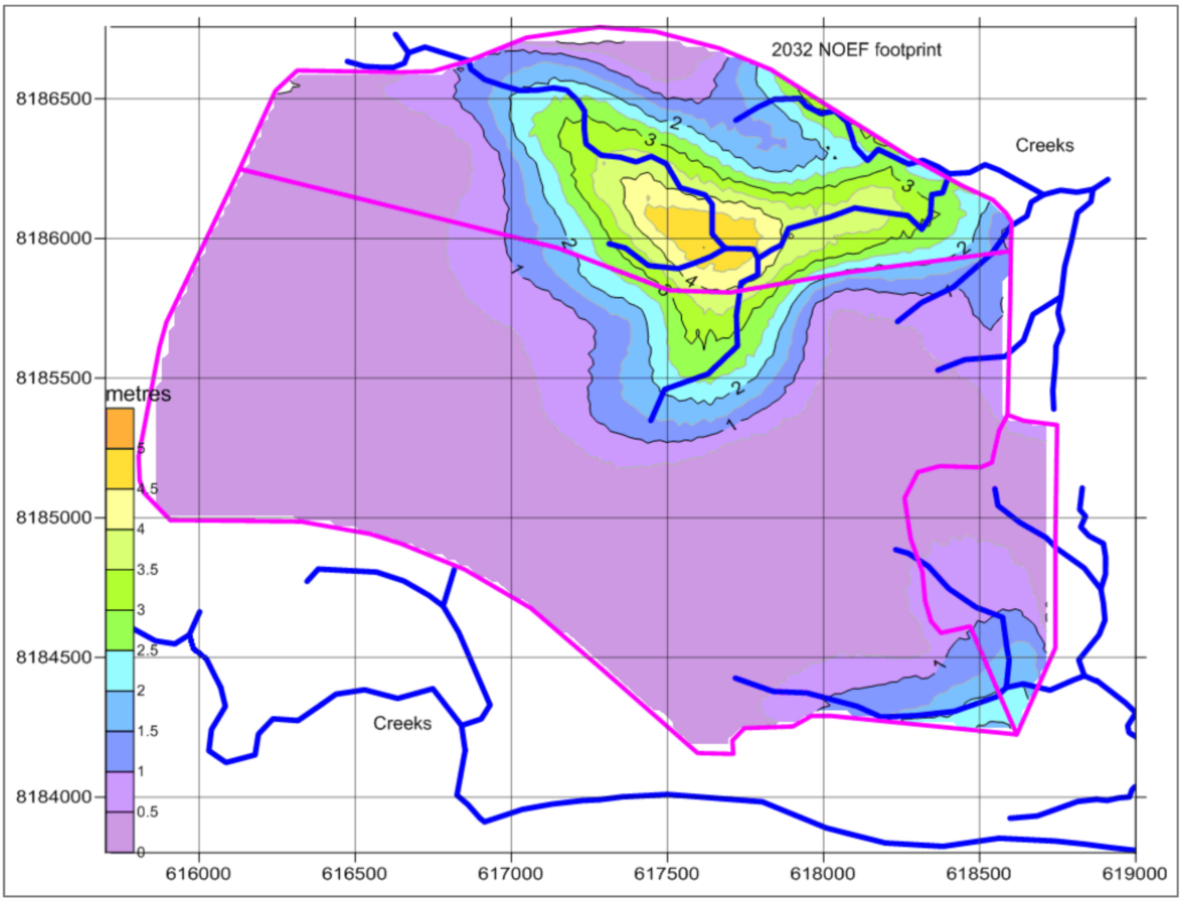

Figure 11 Contours of the phreatic surface in metres relative to the base of the facility (i.e. m's saturated thickness above the base of the facility) at 2042 (KCB 2017).

With very low infiltration rates and a recession of the seepage originating from the pre-cover system construction phase (Figure 9), there is insufficient water available to further elevate groundwater levels within the facility. The levels shown in Figure 11 are regarded as indicative of peak levels, with subsequent years showing a gradual recession. Figure 12 shows groundwater levels as predicted for 2062. A gradual recession and migration of the mound to extents of the facility is predicted.

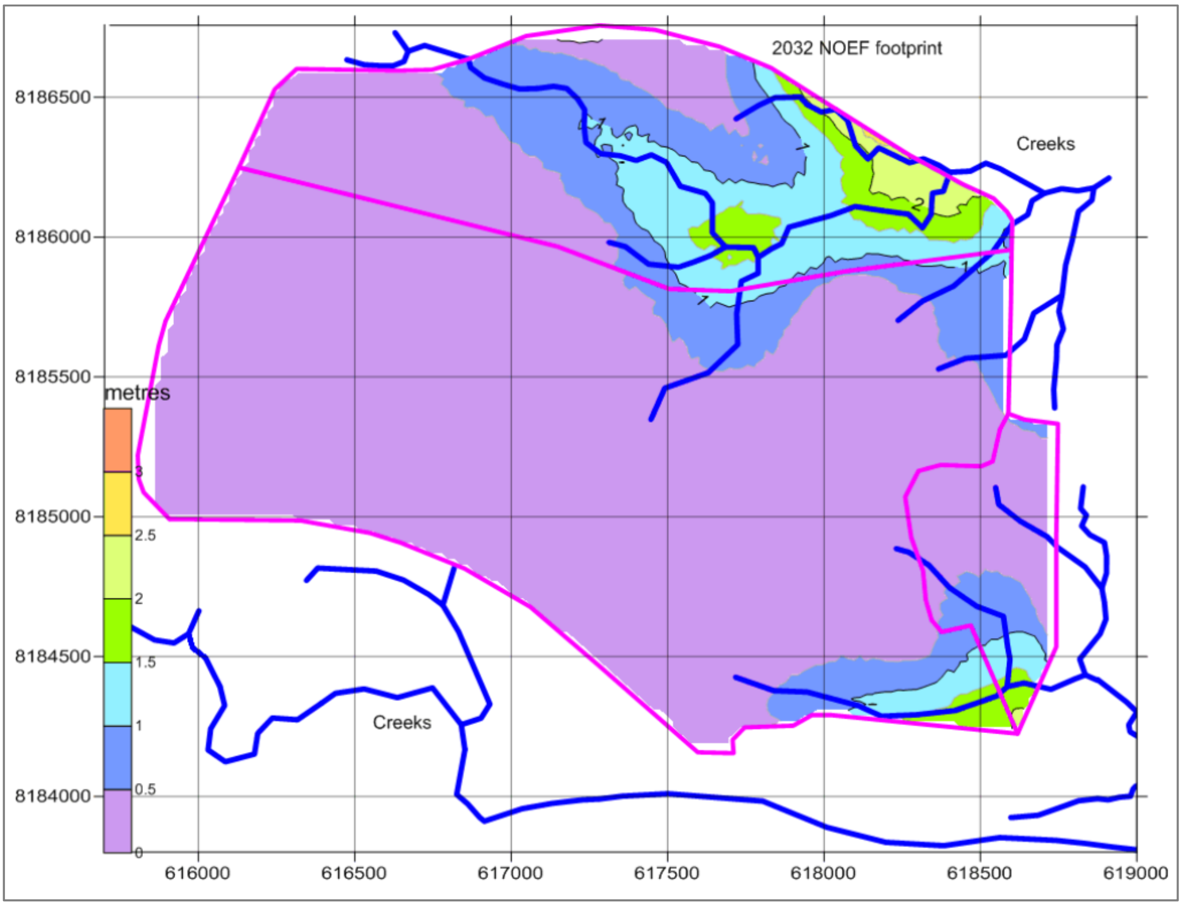

Figure 12 Contours of the phreatic surface in metres relative to the base of the facility (i.e. m's saturated thickness above the base of the facility) at 2062 (KCB 2017).

To assess the potential for instability due to elevated groundwater levels at the toe of the facility, elevated groundwater levels have been applied to Section C-C North at the highest level predicted to be experienced at the toe, nominally 4m above the foundation across the base of the facility. It is important to note that with an effective GSL, levels at this height would be detected within the monitoring and maintenance phase during closure, and remedial measures taken. If the drains were to clog after the maintenance period, groundwater levels are predicted to be less than those described in Figure 11 and Figure 12.

Results from the stability analysis are included in Section 4.

3.4 Design Sections

A total of five sections were selected for analysis. Sections A-A, B-B, C-C South and D-D were selected by the MRM multi-disciplinary technical team as representing the highest risk for instability. Section C-C North has been added to provide a representative section through the northern slope and complete a full transect through the facility when combined with C-C South. The sections are also on the same alignment as previous studies completed by O'Kane Consultants (O'Kane 2016[3]).

Figure 13 presents a plan view location of the five sections against a June 2016 air photograph of the site, and Figure 14 to Figure 18 show sectional outputs from Slide 7™ of the five stability models with the adopted material strength parameters for the base case simulations.

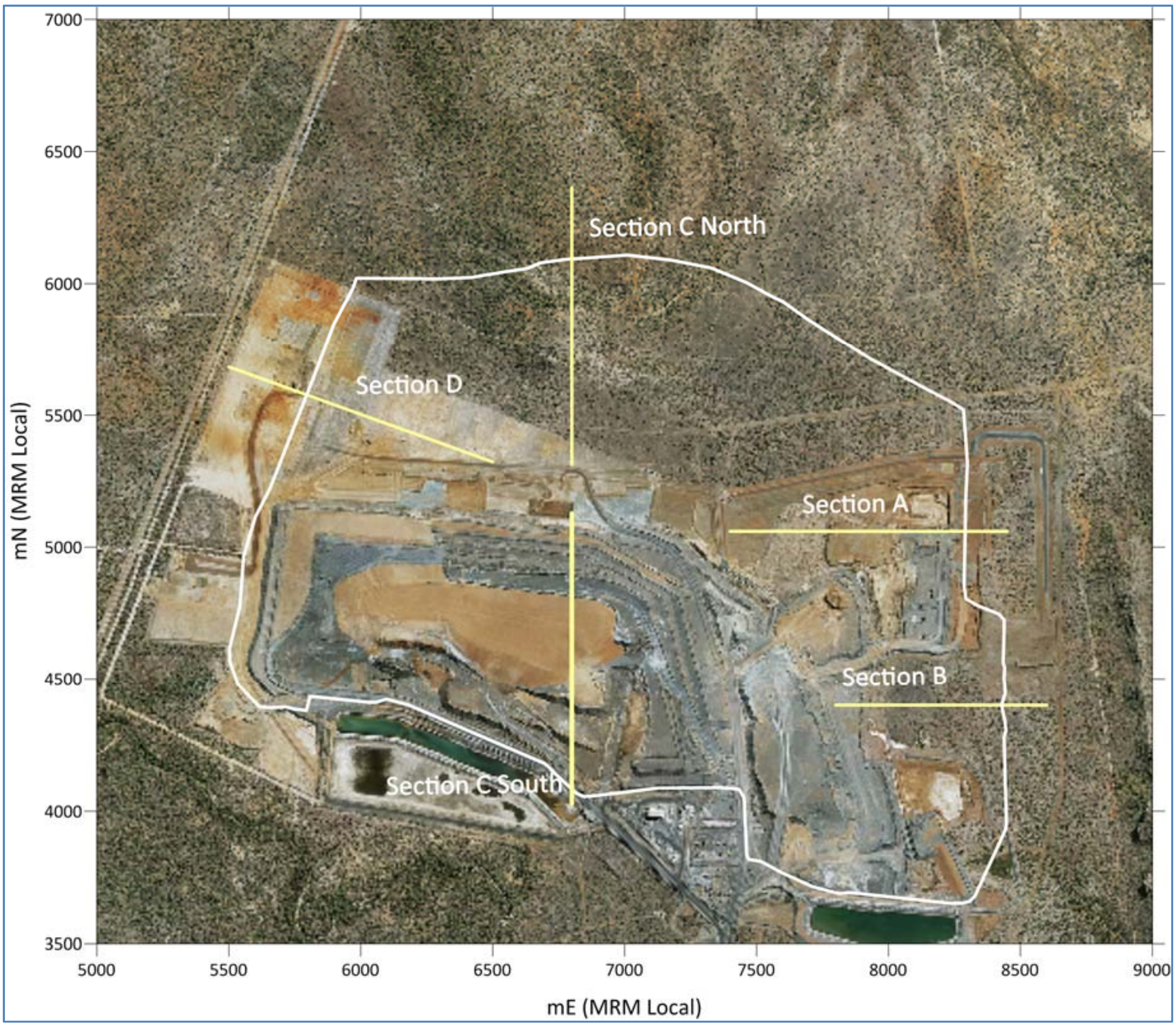

Figure 13 Location of 2D stability analysis sections

Assessment of interim stage designs has not been completed, as overall inter-stage slopes typically have slopes of 1V:4.5H or shallower. With their limited life at that geometry, they are not regarded as a significant slope stability risk. However, their stability should be assessed as part of the detailed design phase, which should also consider the final construction schedule and detailed drainage design.

3.4.1 Model framework

Figure 14 to Figure 18 show the general layout and boundary conditions for each of the sectional models. Note dimensions and model details have been omitted for simplicity but are presented in Appendix A.

Internal complexity associated with stage boundaries, the Basal CCL, 5m MS-NAF Base and individual lifts within the core are not specifically shown in the model sections as their properties are captured within the model generalisation. For example, the Basal CCL and foundation alluvium have been assigned the same properties. A similar approach is adopted for the MS-NAF Base with the Core, PAF(RE) cells and Halo all assigned the same properties. The advection layer between the Core and Halo is an exception to this approach. It is modelled as a complete layer to simulate a "worst case" scenario where a continuous advection layer is required due to delays in forming the outer Halo and cover.

Hydraulic models have adopted setups from O'Kane cover system modelling (O'Kane[2] and O'Kane[3]):

- A cover pore pressure distribution to match the simulated extreme wet season response and 1:1000 year rainfall year as estimated by O'Kane[3]2017.
- Groundwater levels are modelled to within 1m of historic groundwater highs, with the highest level applied as a total head boundary condition at the base of the model (KCB 2016).
- A seepage function was applied to all upper boundaries, preventing pressures above the natural surface developing. This assumes that the cover system surface will be free-draining.

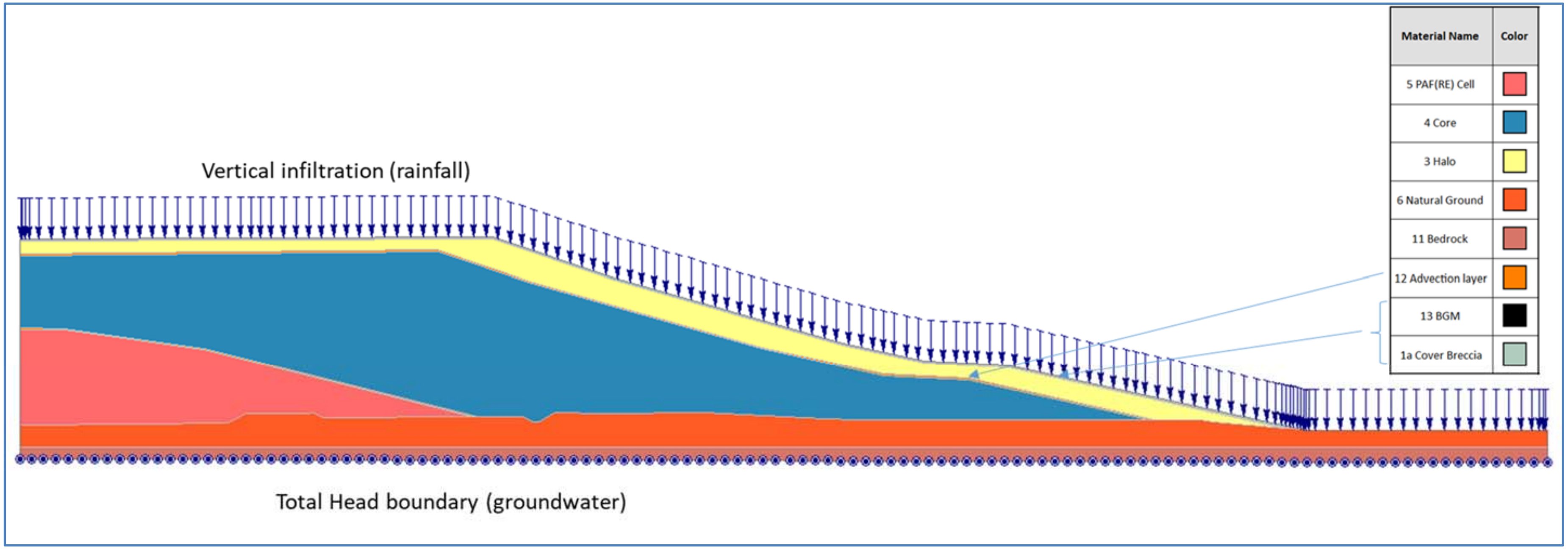

Figure 14 Section A-A (5600N)

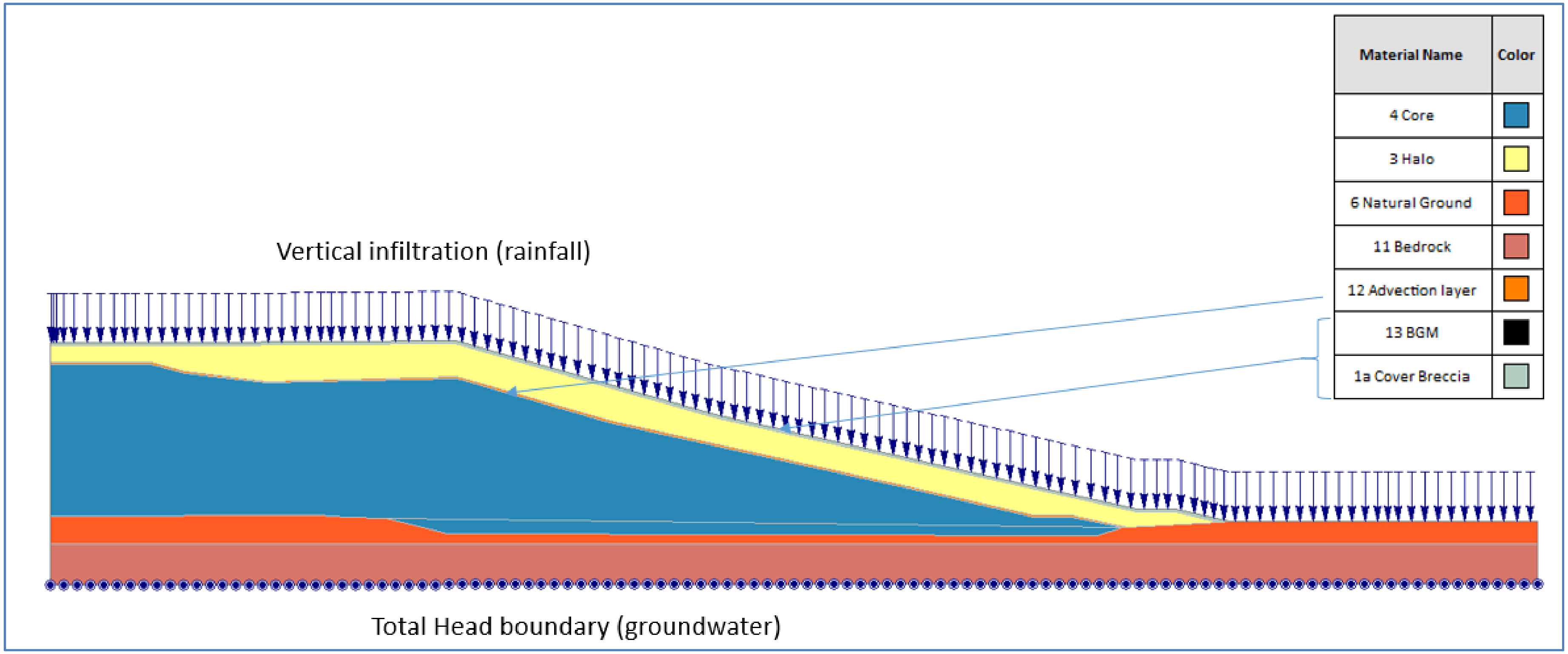

Figure 15 Section B-B (4400N)

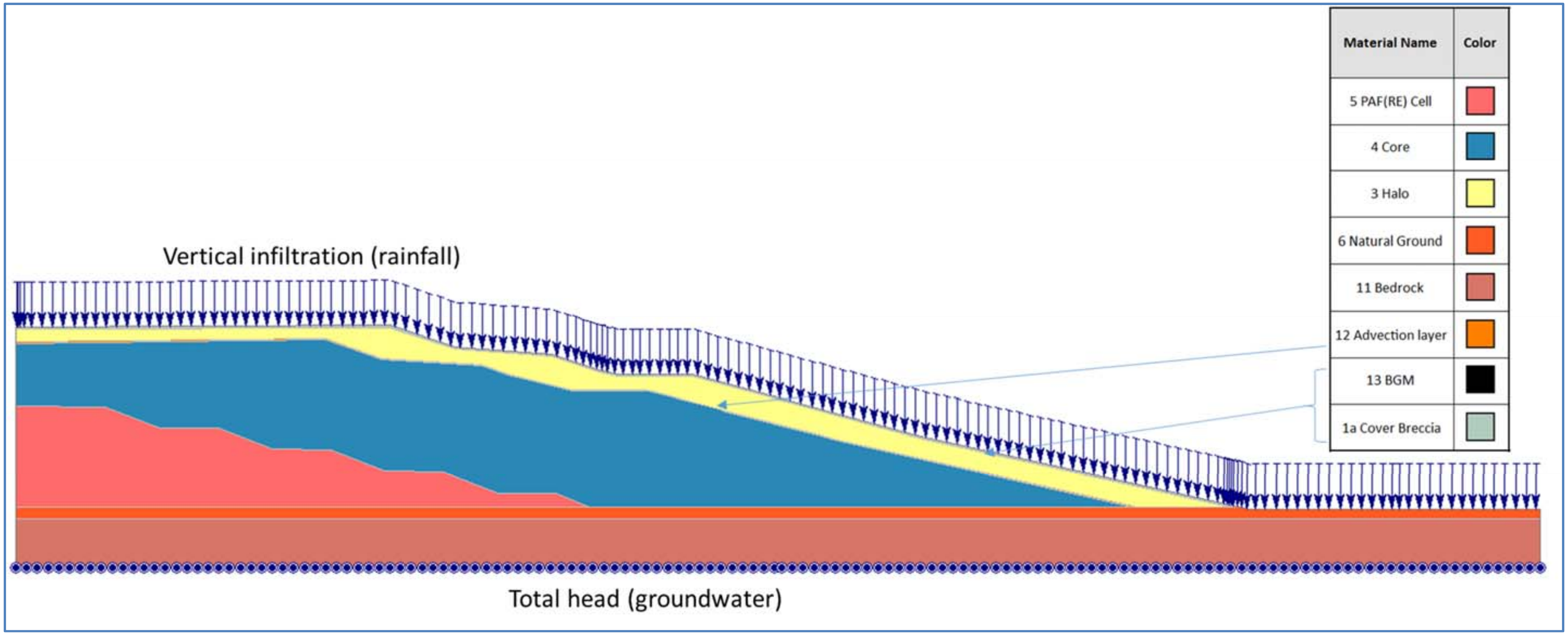

Figure 16 Section C-C North (6800E)

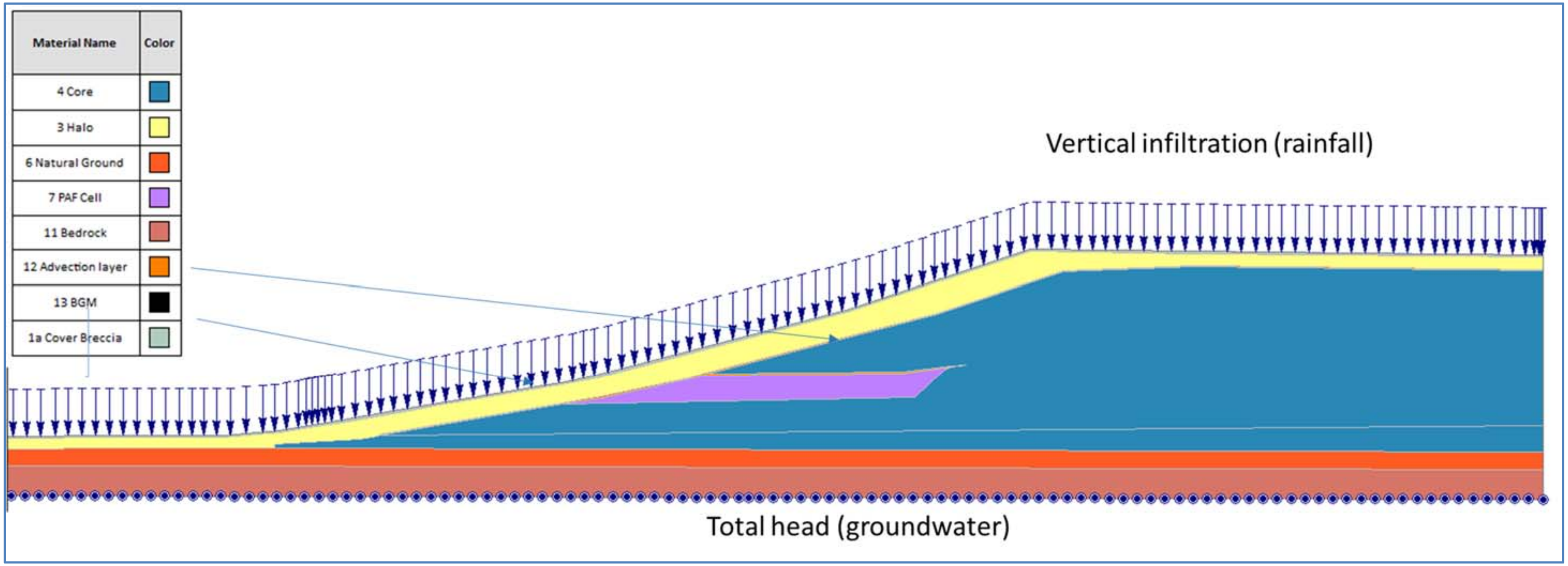

Figure 17 Section C-C South (6800E)

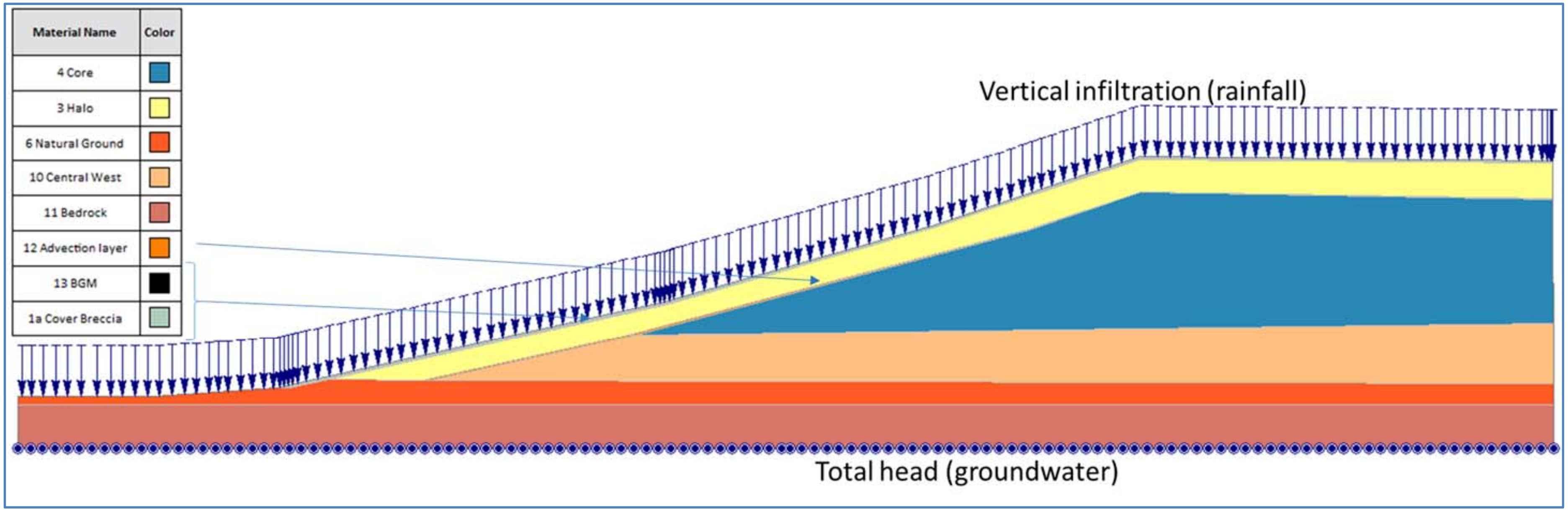

Figure 18 Section D-D (oblique NW sector)

3.4.2 Precursor conditions

The starting conditions for each of the models are:

- Steady state simulations using the annual average rainfall and historically high groundwater levels, typically within 1m of the natural surface at the toe of the facility.
- Transient wet season simulations using the pore pressure response of the steady state simulation as a starting condition.

3.4.3 Adopted hydraulic parameters

Pore water pressures were simulated with steady state and transient 2D finite element modelling using Slide 7.0™. Saturated and unsaturated behaviour was modelled with the aid of water content and permeability curves developed by O'Kane Consultants. Categorised as:

- Breccia (OVERBURDEN);
- Natural surface (NATURAL GROUND); and,
- NOEF rock (OVERBURDEN *–PAF, NAF, CORE, HALO).*

Figure 19 and Figure 20 show water content retention against matric suction, and permeability against matric suction respectively. The O'Kane interpretation includes data by UQ (UQ 2016[2]) from field and laboratory testing of the existing CWOEF.

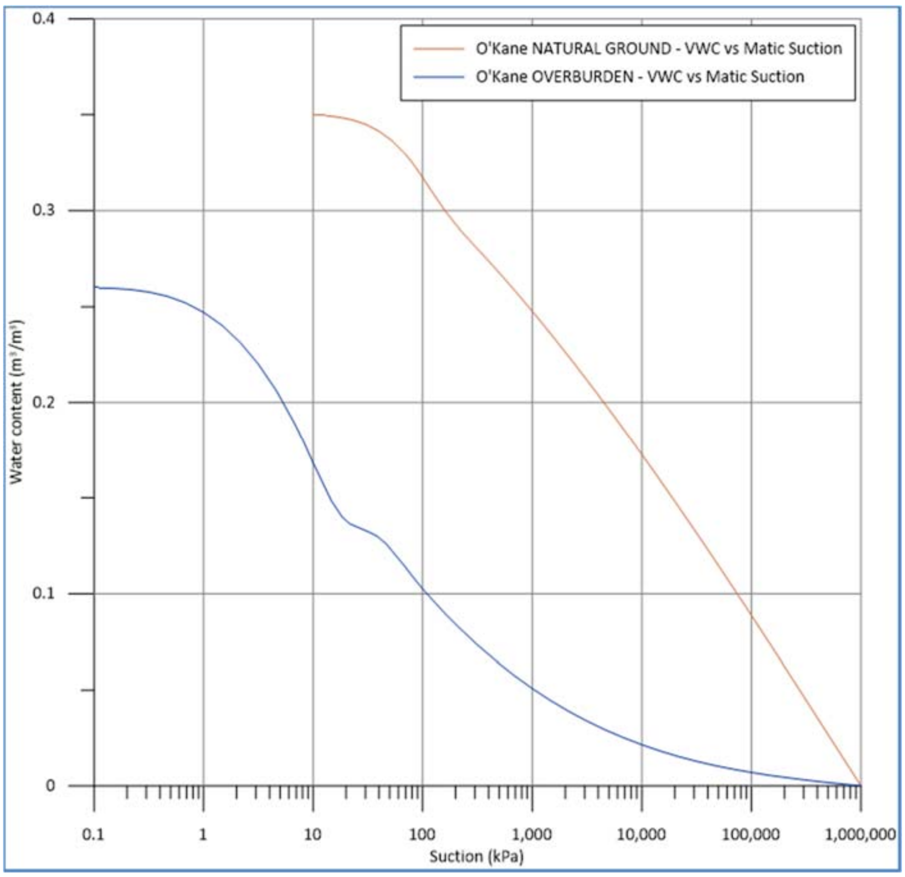

Figure 19 Water content retention curves (after O'Kane 2016[3], UQ 2016[2])

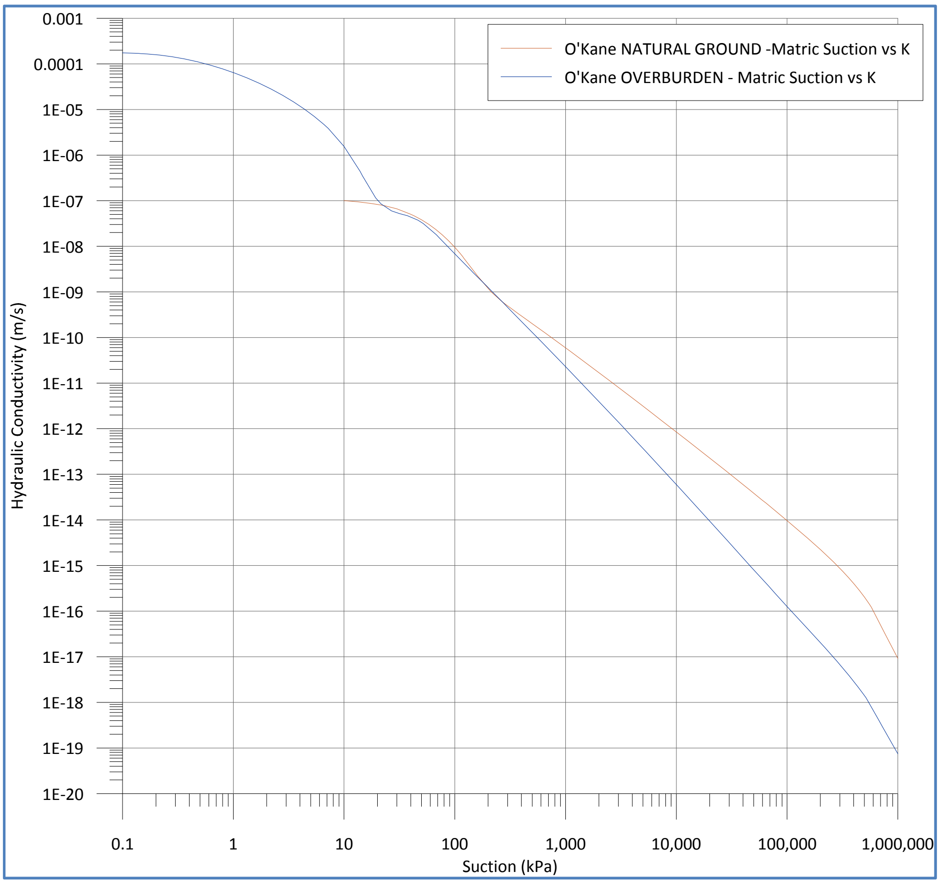

Figure 20 Permeability functions (after O'Kane 2016[3])

In addition, two additional classifications were included to incorporate basement and advection units. Basement and advection layers were assigned the following parameters:

- Basement: Ksat = $1x10^{-7}$m/s, Kv/Kh = 0.5;
- Advection layers: Ksat=$5x10^{-8}$m/s (assumes compaction by overlying waste layers), Kv/Kh = 1. Unsaturated conditions using a Van Genuchten "silty clay" type curve.

The basement typically consists of variably weathered dolomite, overlain by alluvial sediments that primarily consist of silty to sandy clays with minor sands and gravels. A lower bound estimate of weathered basement (KCB 2016) hydraulic conductivity was selected as a conservative estimate.

3.4.4 Pore pressure model setup

The pore pressure models were constructed to be consistent with parameters and scenarios developed by O'Kane for the cover system design (O'Kane 2016[1&3]). Table 8 outlines the model boundary conditions. The groundwater height is an exception and it has been set to the highest groundwater level intersected on section as interpreted by historic groundwater monitoring of foundation strata (KCB 2016), summarised in Table 9.

Table 8 Pore pressure model boundary setup approach

Scenario	Upper boundary condition	Lower boundary condition*	Lateral boundary conditions
Steady State	Annualised interflow** with seepage face condition at 179mm/year (O'Kane[3] 2017).	Fixed total head	No flow
Transient *(Steady state starting conditions)*	12-month 1:1000 wet season response (as modelled by O'Kane[3]2017)	Fixed total head Identical to steady state	No flow

* Lower boundary fixed head conditions are conservatively placed to simulate a background water table just below the natural surface at the toe of the facility.
** Interflow describes proportion of rainfall that enters the breccia cover.

Table 9 Pore pressure modelling, lower boundary condition.

Scenario	Lower boundary condition level (mRL)
A-A	10024
B-B	10024
C-C North	10032
C-C South	10032
D-D	10036

The foundation and slope drainage networks have not been incorporated into the base case models. The foundation drains are designed to reduce the potential for groundwater mounding within the facility, particularly near the toe. Omission of the drains from the base case simulates a potential long-term scenario where drains clog or lose effectiveness.

The mid-slope surface drains have not been simulated in the 2D infiltration models.

3.4.5 Discussion of pore pressure response

Pore pressure modelling indicates that:

- The cover system GSL (BGM) acts to effectively limit water from rainfall infiltrating into the NOEF.
- During high rainfall events the slope's cover system could be progressively saturated from the base. Extreme event modelling by O'Kane[3] with 1:1000 year rainfall scenarios predict that most of the cover system (from natural surface level) could be saturated.
- During the construction period and prior to the placement of the GSL Cover System, the advection layers above the core and within the PAF(RE) cells have the potential to impede flow and could develop areas where saturated conditions occur.
- Groundwater levels within the base of the facility show a modest response (rise) closer to the toe of the facility. More detailed modelling by KCB (KCB 2017) provides a more

accurate characterisation of the anticipated response (Figure 10). Groundwater is not expected to adversely impact on the overall stability of the facility.

The pore pressure response and the potential implications for stability are discussed in Section 4.

3.5 Seismic loading

The site is located on the northern part of the Indo-Australian tectonic plate, regarded as a low to moderate seismic region and at large distance from the plate margin. Geoscience Australia has been collecting seismic data for the region since 1934. No events have been recorded with epicentres within 150km of the site and events no greater than magnitude 4.0 within 300km of the site.

The large-scale lineaments (faults) have been identified within the geology of the project area. Although these structures are regarded as aseismic, there is the possibility that if favourably oriented with respect to the prevailing stress regime, reactivation could occur. Recent studies regard the maximum credible earthquake within Australia's stable continental regions as between Mw 7.0 and 7.5±0.2 (Allen et al, 2011).

The landform response to an earthquake has been assessed in previous studies (URS 2008) with application of a directional peak horizontal ground acceleration to simulate the seismic load of 0.05g (based on AS1170.4, Minimum Design Loads on Structures, Part 4: Earthquake Loads adopted), with an acceptance criteria of factor of safety (FOS)>1.1 or seismic displacement <0.5m (USACE, 1984).

Subsequent to these studies a probabilistic seismic hazard analysis (PHSA) was completed to assess Tailings Storage Facility (TSF) stability (GHD 2015). The outcome of the study provided recommended values for peak ground acceleration (PGA) and event magnitude for return periods of 1,000 years to assess an operating basis event (OBE)), 10,000 years to represent a maximum design event (MDE)) and maximum credible event (MCE).

These values have been utilised in application of a simplified method of estimating earthquake induced deformation devised by Bray and Travasarou (2007). Application of this method is described in Section 3.6.

3.6 Analysis methods

Consistent with previous studies, a 2-dimensional limit equilibrium slope stability tool (Slide 7.0™ by Rocscience) was utilised to assess representative sectional models. Bishop simplified and GLE/Morgenstern-Price methods of slices were adopted to calculate the Factors of Safety (FOS) against both circular and path specified block (non-circular) failures. The finite element groundwater package available in Slide 7.0™ was utilised to generate pore pressure grids for both steady state and transient simulations.

The potential response to seismic loading has been assessed utilising the seismic function in Slide to determine the seismic yield coefficient (*Ky*) for limiting failure surfaces at a FOS=1. The coefficient is subsequently used to estimate a deviatoric slope displacement using a simplified method offered by Bray and Travasarou (2007):

$$\ln(D) = -0.22 - 2.83\ln(k_y) - 0.333(ln(k_y))^2\ 0.566ln(k_y)\ln(PGA) + 3.04(\ln PGA)^1 - 0.244(\ln(\mathrm{PGA}))^2 + 0.278(M-7) \mp \varepsilon$$

Where

P(D=0) = probability (as a decimal number) of occurrence of zero displacement
D = seismic displacement in cm
Φ = standard normal cumulative distribution function
ky = yield coefficient in units of g
PGA=peak ground acceleration
ε = normally distributed random variable with zero mean and standard deviation σ=0.67

3.7 Stability model scenarios

The base scenario for all sections is considered representative of a conservative long-term stability case. It is defined by:

- Drained soil shear strength parameters, to simulate long-term, post construction conditions.
- A foundation water table that is set as a total head boundary condition at the base with levels to match historically high levels.
- The foundation drainage network has been omitted in the base case.
- Circular and block slip surface search methods with block search paths set to follow CCL, advection and alluvial foundation layers.
- Annual interflow rates (179mm/year) to simulate rainfall entering the cover system.
- An extreme 1:1000 year rainfall scenario response (provided by O'Kane[3]) was used to assess worst case stability response to elevated pore pressure within the cover system.

3.7.1 Sensitivity scenarios

Ranged sensitivity scenarios were considered to examine the potential implications for failure of key controls, undetected conditions and variation in parameter estimation:

- The substitution of breccia with an **alluvial based** cover material. Where the stability of the cover system utilised a directly placed, or poorly compacted alluvium on the slopes there is a risk of instability during high rainfall events, where elevated transient pore pressures lower the effective shear strength.
- Substitution of the GSL underliner aggregate (HMR), with a track rolled alluvium. The alluvium would be placed with natural moisture and track rolled. Shear strength parameters from UQ testing were adopted (Table 7). With the GSL limiting the ingress of water and high permeability Halo rock beneath, suction pressures are expected to develop and persist within the alluvium underliner.
- **Shear strength parameter reduction** of breccia with an apparent cohesion of c'=0 and worst case (1:1000 year) wet season pore pressure estimates to assess cover system

sensitivity to extreme transient pore pressure events without the influence of vegetative growth or interstitial cementation.

- Elevated groundwater levels that could occur if the foundation drainage system prematurely blocks. The foundation water levels have been raised by 5m above the base of the foundation for Section C-Cn to the match maximum groundwater levels predicted by KCB 2017.

Lower bound strengths have been adopted as a base case for all internal (i.e. below cover) rock overburden materials in all scenarios.

Block and circular and Cuckoo search functions were used for both base and breccia sensitivity cases. Paths for the block searches were set to direct search paths at the breccia cover contact with GSL, advection and foundation alluvial layers.

4 NOEF Stability Analysis results

4.1 General

The analyses are specifically intended to identify areas of sensitivity in the design to inform detailed investigations, monitoring and management plans. Parameters selected for the base cases are lower bound estimates from existing datasets and studies, and as such are considered conservative.

4.2 Results

The results of the cases are summarised in Table 10, with graphical results presented in Appendix A.

4.2.1 Overall slope stability

Shown in Table 10, the base case modelling predicts that the overall slope is very stable with both circular and directed block search and Cuckoo search analyses reporting FOS>1.5 for all sections, indicating long-term stability.

4.2.2 Cover system stability

The cover system stability is governed by the Breccia shear strength. Breccia was conservatively modelled with the UQ mid-bound shear strength envelope for rock overburden. As indicated in Table 10, all sections report FOS greater than 1.5 for the base case.

Sensitivity analyses performed with alluvium instead of breccia for the batter cover system with low shear strengths (i.e. apparent cohesion ~0kPa) did not achieve the minimum FoS, when the cover was inundated (i.e. heavy rainfall events), indicating that these materials are not recommended for use on the cover system on the slopes without further investigation into shear strength behaviour.

Analyses performed with the Breccia shear strength parameter for apparent cohesion set to 0kPa show that FOS approach 1.3 for the upper slope aspect (1V:3H) only for "worst case" pore pressure scenarios (1:1000 rainfall scenario O'Kane[3] 2017).

Stability modelling with Section A-A found that the alluvium underliner did not present as a limiting component.

Table 10 Base case summary of results (FOS)

Section	**Search Method**	**Base Dry Season**	**Base Average Wet Season**	**Sensitivities**	***Base 1:1000 Wet Season***	***Breccia Sensitivity 1:1000 Wet Season***
A-A	Circular	1.9	1.9	1.9[1]	*1.8*	*1.3*
A-A	Block	1.7	1.7	1.7[1]	*1.7*	*1.4*
A-A	Cuckoo	1.8	1.8	1.8[1]	*1.8*	*1.3*
B-B	Circular	2.1	2.1		*2.1*	*1.5*
B-B	Block	1.9	1.9		*1.9*	*1.5*
B-B	Cuckoo	1.9	2.1		*2.1*	*1.5*
C-Cs	Circular	1.8	1.8		*1.8*	*1.3*
C-Cs	Block	1.7	1.7		*1.7*	*1.4*
C-Cs	Cuckoo	1.8	1.8		*1.8*	*1.3*
C-Cn	Circular	1.9	2.2	2.0[2]	*2.1*	*1.3*
C-Cn	Block	1.9	1.9	1.8[2]	*1.9*	*1.6*
C-Cn	Cuckoo	2.2	1.9	1.9[2]	*1.9*	*1.6*
D-D	Circular	1.8	1.8		*1.8*	*1.3*
D-D	Block	1.6	1.6		*1.6*	*1.4*
D-D	Cuckoo	1.8	1.8		*1.6*	*1.3*

Design acceptance criteria: FOS≥1.5
Design acceptance criteria 1:1000 FOS≥1.3
[1]Section A-A alluvium under liner scenario.
[2]Section C-Cn blocked drain, elevated groundwater level scenario.

4.2.3 Pore pressure

General observations of the pore pressure distributions indicate that:

- The majority of the facility's core develops suction pore pressures;
- The cover system is typically simulated as saturated in the lower half of the slope for the annualised wet season case and most of the slope's cover system is simulated as potentially saturated during 1:1000 year wet season case;
- Advection layers have the potential to impede vertical flow, however the modelling predicts that negative pressures are maintained in any case.

Controlling groundwater levels at the toe is important as high groundwater levels at the toe have the potential to destabilise the toe of the facility. However, stability modelling that incorporates groundwater level results from simulations with drainage system clogging meet acceptable stability criteria.

4.2.4 Seismic loading

The Draft EIS presented a seismic assessment with potential deviatoric displacements as calculated with the Bray and Travasarou, (2007) method. With yield accelerations (Ky) calculated from base case stability runs the estimated displacements are very low, at less than 2cm. This demonstrates a low risk of large deformations in the event of a sizable earthquake and within tolerance for acceptance criteria adopted for previous studies (URS 2008). The change in design to replace the upper CCL with a GSL decreases the risk of damage, with the GSL able to accommodate greater strains without compromising performance.

4.2.5 Residual shear strength

Residual shear strengths develop as a result of the soil particles undergoing displacement (strain) such that soils with a relatively high content of plate-like particles (e.g. clays) experience a realignment of those particles to be parallel with the direction of shear. Circumstances applicable to the NOEF that could contribute to realisation of this mechanism are:

- Seismic activity where earthquakes of sufficient magnitude and ground acceleration cause temporary instability to an extent that sufficient displacement occurs within the clay based layers to develop residual strengths. Modelled displacements in the order of 10 to 50cm are typically considered worthy of further consideration.

Of the units capable of developing residual strengths, notably advection layers and foundation alluvial units, neither are likely to develop residual shear strengths as displacements likely to be realised are relatively minor (<10cm).

4.3 Settlement

The NOEF at its highest consists of up to 140 metres of overburden rock. The facility is developed in stages (Figure 5) and therefore constructed with berms and benches with vertical heights of less than 10m. These stage geometries could give rise to differential settlement within the facility which, if excessive, could compromise the integrity of the cover system. Other factors that could give rise to differential settlement are associated with variation in the material type, level of compaction and consolidation rate (influenced by water drainage).

With a continuous cover system encapsulating the entire facility, differential settlement could induce an intolerable strain on the hydraulic barrier layer, potentially compromising its integrity.

To gain insight into the magnitude of the potential strains, GHD (GHD 2017) completed a preliminary study, comprised of field investigation and 2D finite element modelling to assess the immediate settlement and also an assessment of the extent of creep that the facility may experience over time. The results and findings are summarised and the memorandum provided as Appendix B.

The field study scope included 20 plate load tests on a range of OEF materials at selected locations within the current OEF. The tests were performed with a 50T jack and 600x600mm plate. The tests were in accordance with ASTM D1196. Two of the locations were "soaked" with 40ML of water to assess the consolidative response. The results of the tests derived estimates for Youngs Modulus that served to inform a simplified 2D finite element model developed in Plaxis™.

Samples were also assessed for Particle Size Distribution (PSD). The results classified the samples as gravel with cobbles, although particles greater than 200mm were not included in the sample.

The Plaxis model was designed to simulate the basic construction of the NOEF, with simulation of successive 10m lifts to construct the core, and subsequent covering with NAF materials in the halo. Also included was wetting up of the bottom 10m of key stages.

The model provided estimates for the immediate (elastic) total settlement for the completed 140m high NOEF, which were between 2.4 and 3.8m. GHD found this settlement mechanism would not impact on the cover system barrier layer as most settlement would have occurred before the liner was placed.

However, long-term creep, through gradual deformation of the rock particles, could result in a total settlement for the 140m structure of between 2.1 and 6.3m over a 1000 year period.

Current estimates of differential strain over 1000 years due to creep are in the order of 6% (GHD 2017). With shallow slope angles (1V:4.5V) also formed within the stage designs, the strain realised from differential settlement is likely distributed across a significant length. The tolerance for elongation of a GSL utilising a bitumen base can be up to 60% (e.g. Coletache® ES 3), which is well within the estimated maximum differential settlement strain.

5 NOEF findings and considerations

5.1 Key findings

The overall long-term stability of the facility as modelled with 2D limit equilibrium methods is shown to be conceptually stable with proper foundation preparation, careful manufacture and emplacement of materials, attentive monitoring and frequent review and reconciliation of design assumptions against actual parameters and performance.

The cover system is sensitive to the strength and hydraulic properties of the growth media (breccia), particularly in the upper third of the slope, where a strong and free draining growth media is important to manage transient hydraulic loading and erosion. The reduction in slope of this upper batter to 3H:1V has contributed to enhanced slope stability compared to the Draft EIS.

The hydraulic and geotechnical properties of the foundation and advection layers are important for overall stability of the facility, particularly at the toe. Continued assessment of the foundation permeability and hydraulics along with direct measurement of pore pressures within the advection and basal CCL layers is recommended to further assess transient pore pressure behaviour and calibrate the modelled understanding.

Estimates of maximum differential strain for the NOEF are in the order of 6% (GHD 2017). With shallow slope angles (1V:4.5V) also formed within the stage designs, in conjunction with low lift heights relative to the footprint, the strain realised from any differential settlement is likely distributed across a significant length. The tolerance for elongation of a GSL utilising, for example, a bitumen base is comfortably accommodated with the estimated differential settlement strain.

5.2 Required work program in preparation for implementation

This section describes the sampling, testing, monitoring and design recommendations to be considered for the NOEF management plan. It is not intended as a detailed work program or design for implementation and will require further development as part of a detailed design process.

5.2.1 Sampling and testing

a Foundation

Consistent with the current construction practices, foundation preparation will be required to ensure the foundation is stable and of low risk for differential settlement. Continued maintenance of the foundation geological model with testing and sampling of the foundation materials will inform an understanding of the hydraulic and consolidative behaviour of the foundation.

Ongoing evolution of the geological model is required to expand and improve the model, specifically:

- A detailed representation of basement geology and hydrogeology that captures fault zones and areas of preferential weathering.
- A detailed representation of alluvial sediments prior and post foundation preparation, to enable an understanding of the pre- and post-geomorphological system overlying the basement.

Additional drilling, test pitting and geophysical surveys will be required to inform the model.

Geotechnical sampling of representative foundation areas is recommended, initially with sampling frequencies established for the current CW Stage construction (MRM 2015). The sampling frequencies should be routinely reviewed as part of an overall site review process.

b GSL installation and post-installation testing

Unlike CCLs, the GSL is manufactured in an offsite quality controlled environment, however its installation requires a high level of quality control and attention to detail. The cushioning layer used to protect the GSL during breccia placement requires further field assessment in the detailed design phase to assess the range of suitable materials and complement laboratory interface testing of the cover system with GSL to validate modelled shear strength parameters.

c Cover system performance testing

Given the shear strength of breccia material easily exceeds requirements for stability in the cover system geometry, the long-term stability of the cover system therefore relies on its resistance to erosive mechanisms. Trial test sites are recommended to test and demonstrate the cover system erosional performance.

O'Kane Consultants (O'Kane 2016[4]) describe the requirements for erosion monitoring with a purpose-built field testing system consisting of an integrated weather station, sediment dam and automated sensor system to measure the rainfall, runoff, interflow, moisture, pore pressure and sediment loading. The data gathered from the system will provide a means to test and calibrate the design parameters.

5.2.2 Design and analysis

To transition from the current CW NOEF design and construction process, a detailed design and construction process is also required for the expanded facility, incorporating the requirements of:

- The cover system design and monitoring plan;
- Groundwater management and monitoring;
- Interim stage and final design stability modelling; and
- Detailed monitoring and management plan.

5.3 Monitoring network

5.3.1 Pore pressure monitoring

Pore pressure behaviour associated with the installed fluid control layers - cover (GSL), advection, basal CCL and foundation - are an important component of the NOEF construction.

O'Kane Consultants describe a comprehensive automated monitoring network for the cover system and underlying Halo (O'Kane 2016[4]). This system is suitable to monitor pore pressures that may develop within the cover system.

In conjunction with the foundation permeability testing and modelling described earlier, the planned vibrating wire piezometer array should be installed in an area where representative low permeability alluvial sediments are within the foundation of the current CW NOEF. The array, monitored while the facility is constructed, will provide insight into how the underlying foundation drains and inform mine planning of sustainable rates of vertical advance, foundation consolidation rates and changes in permeability. The results of the monitoring will also inform of the requirement and location of additional foundation monitoring systems.

The monitoring array design consists of a section through the facility where a high rate of vertical advance will occur, over relatively thick low permeability alluvials. An indicative array is comprised of:

- Three sites on a section from the centre (highest load) to the outer slope (toe);

- Each site consists of four grouted vibrating wire piezometer gauges (e.g. Geokon 4500) installed vertically within a borehole drilled below the base of alluvials and into the weathered bedrock;
- Gauges are sited within the borehole to measure pore pressures within representative zones (e.g. weathered basement, lower alluvials and upper alluvials);
- Gauge cabling is trenched and buried below the surface to prevent damage during construction and terminated in a monitoring station;
- The monitoring station can be automated with a similar system to the cover monitoring network. Such a system constructed with Campbell Scientific components could consist of a CR800, AVW200 and AM16/32 and be telemetered via a 3G network or local short haul radio network.

Elevated pore pressures could also accumulate at the toe of the facility where the cover system GSL ties into the natural alluvial sediments and Basal CCL of the foundation, potentially pressurising the toe of the facility if there is a fault within the GSL of the cover system. Fully grouted, vibrating wire piezometer arrays consisting of at least three gauges should be installed within the facility at approximately 50 to 100 metres from the toe of the facility with gauges sited in the alluvial foundation, Halo zone and subgrade below the cover system. At least two sites per slope (at least 8 sites) should be installed, targeting areas where during closure interflow could be focussed (e.g. along stage advection layers or relict paleo-drainage) before equilibrating with the underlying groundwater system.

If toe pressures prove excessive, additional relief drains and associated collection systems may need to be installed to alleviate pressures.

5.3.2 Settlement

Given the large area and difficulty predicting where differential settlement may occur, a monitoring system with the capability to monitor millimetre displacements over a long period of time with reliable accuracy may be beneficial.

A network of survey surface monuments in conjunction with satellite Interferometric Synthetic Aperture Radar (InSAR) monitoring would enable detection of relative millimetric X,Y,Z movement during and post construction without the need for site maintenance.

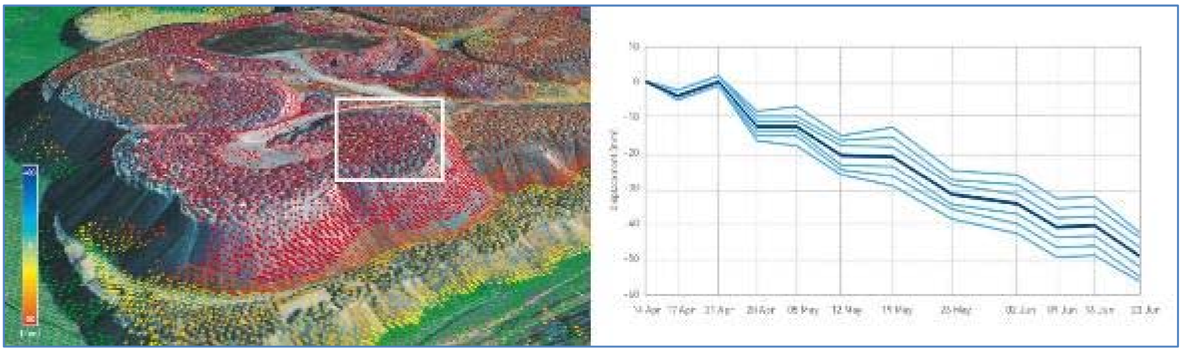

Figure 21 Satellite monitoring with SqueeSAR™ monitoring system *(http://tre-altamira.com/mining/#pit-monitoring)*

During construction, installation and maintenance of surface reflectors, in conjunction with site managed survey monuments, provides data with a higher level of displacement accuracy that is suitable for calibration of settlement models and effective demonstration of facility stability prior to closure.

Monitoring frequencies are dependent on the satellite visit time, typically between 8 and 35 days and with displacement accuracies at <1mm/year and spatial accuracies of <10m for X-Y and <2m for Z. Whilst it is not comparable with the spatial accuracy possible with site based survey techniques or dedicated monitoring, it does allow for large areas to be assessed for relative displacement at millimetre accuracy and provides a system for monitoring during closure that does not require site maintenance.

An initial program of settlement monitoring in conjunction with cover GSL emplacement planning is required on an early development stage to confirm that GSL placement has occurred after the majority of settlement has taken place. Attention is required when progressing the facility over former stage boundaries or where significant differentials in subsurface geology occur, for example, where thicker alluvials are encountered in historic ephemeral drainage lines.

5.3.3 Infiltration rates

An understanding of infiltration rates is required for pore pressure modelling, an important parameter in the slope stability assessment, but critical to long-term geochemical performance of the NOEF. O'Kane Consultants (O'Kane 2016[4]) describe installation of a large scale lysimeter to assess net percolation rates.

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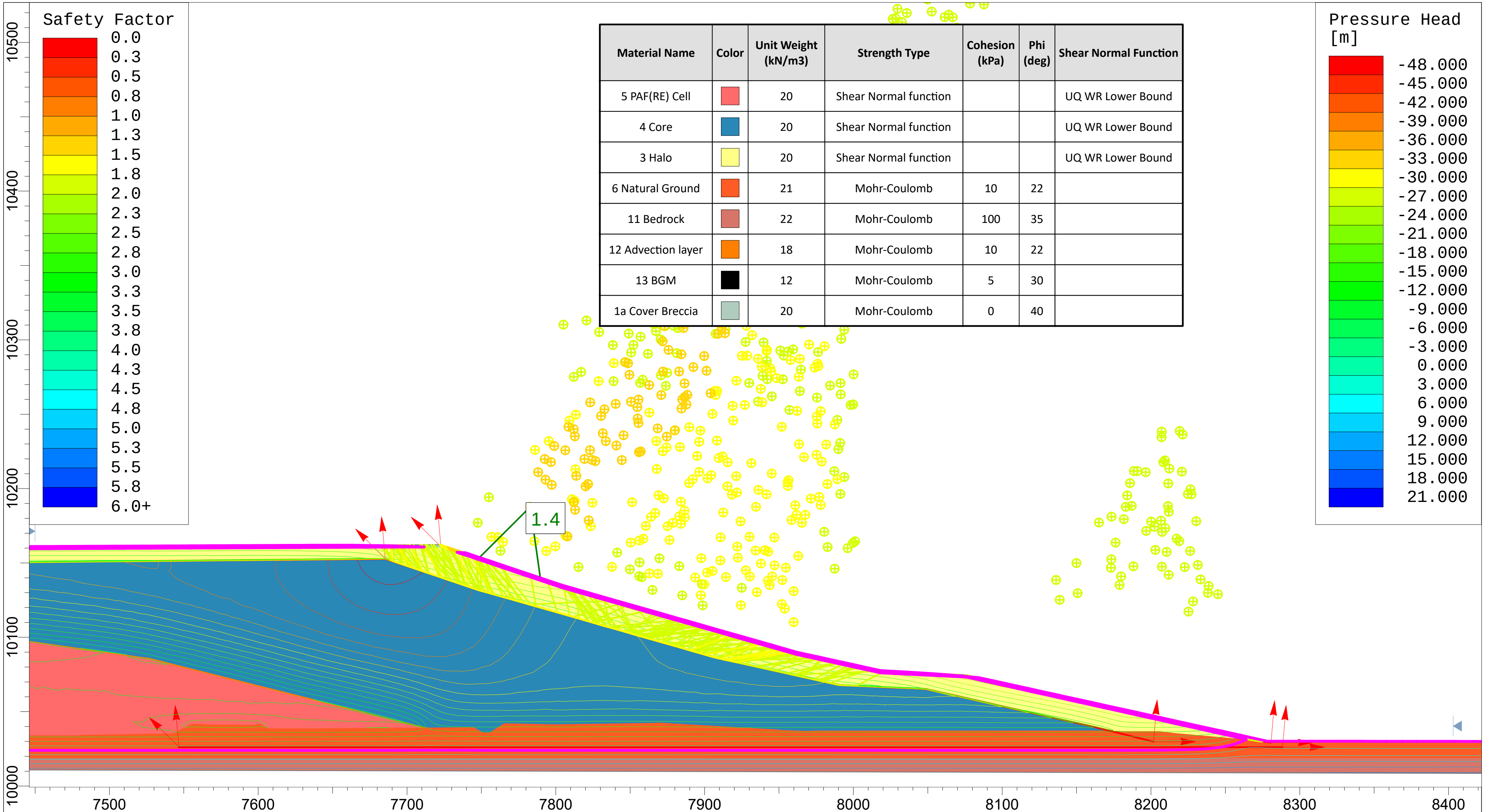
Appendix A – Stability model outputs



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4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

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SLIDEINTERPRET 7.030

Project MRM NOEF EIS

Description:
Section A-A Breccia Sensitivity 1:1000 Event

Analysis Method: Block

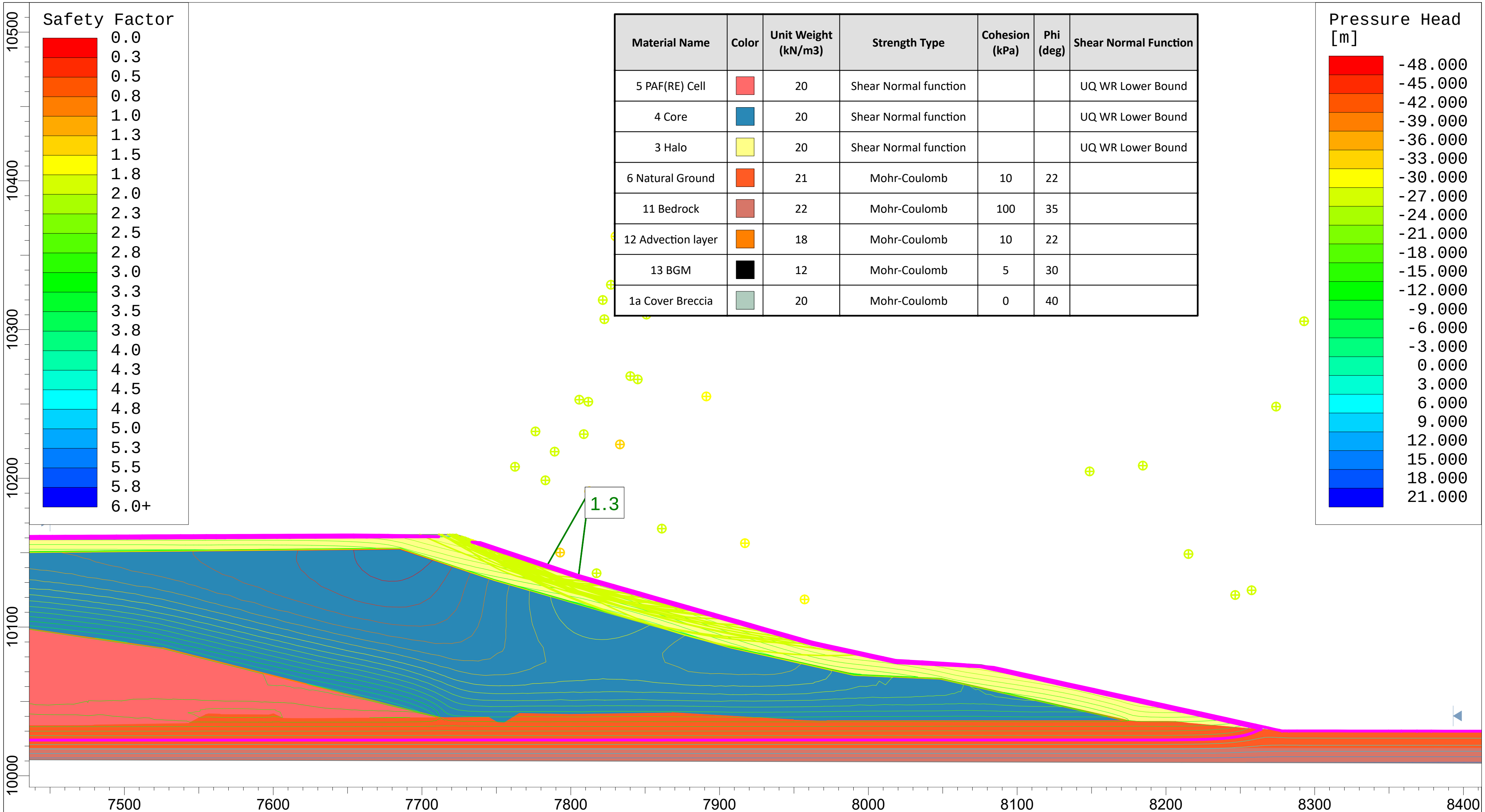
Drawn By SB

Scale 1:3586

Company MRM/Pando

Date November 2017

File Name: section a-a update_bgm_breccia_1-1000.slmd



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4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

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Project MRM NOEF EIS

Description:
Section A-A Breccia Sensitivity 1:1000 Event

Analysis Method: Circular

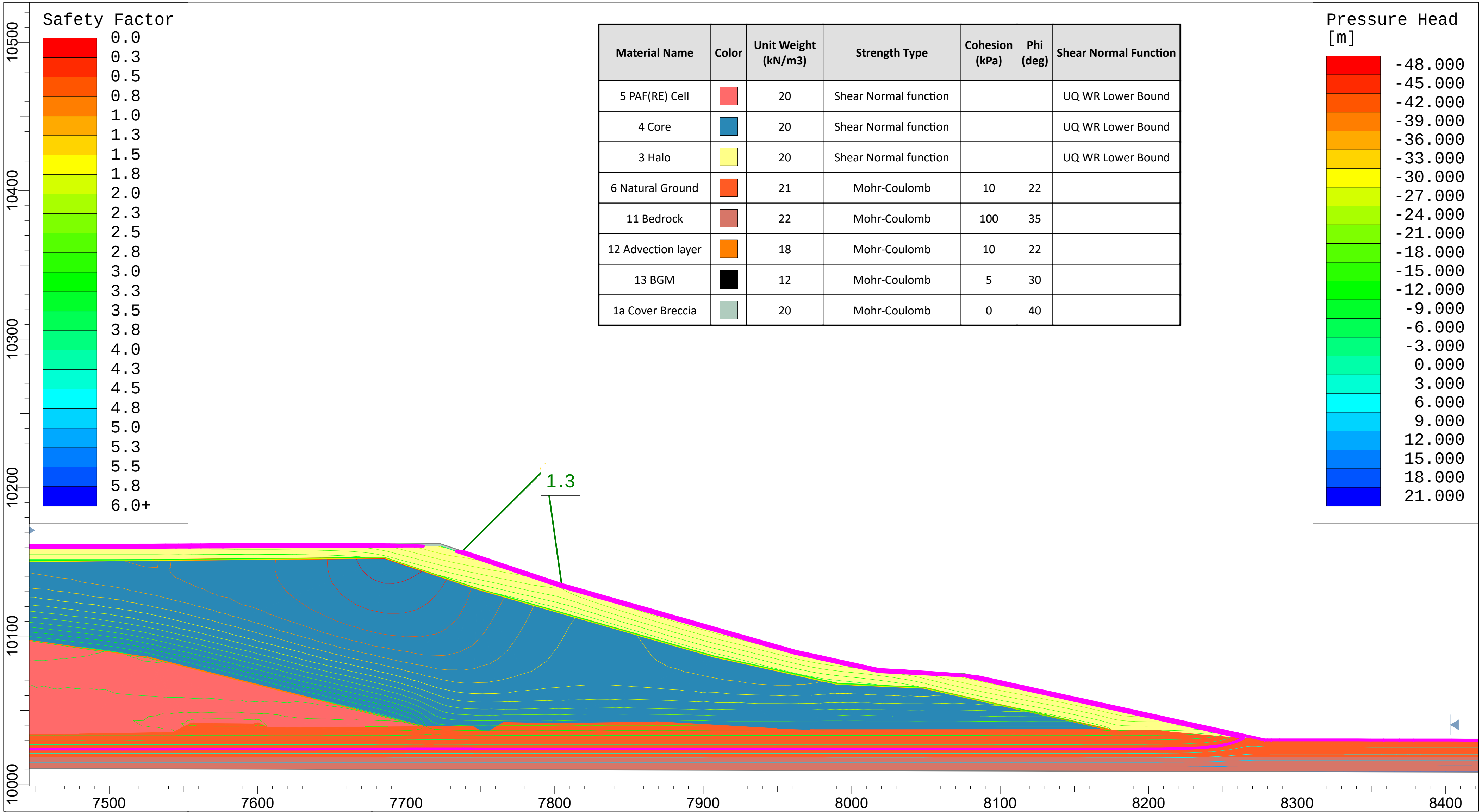
Drawn By SB

Scale 1:3586

Company MRM/Pando

Date November 2017

File Name: section a-a update_bgm_breccia_1-1000.slmd



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3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
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Project	MRM NOEF EIS
Description: Section A-A Breccia Sensitivity 1:1000 Event	Analysis Method: Cuckoo
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Company	MRM/Pando
Date	November 2017
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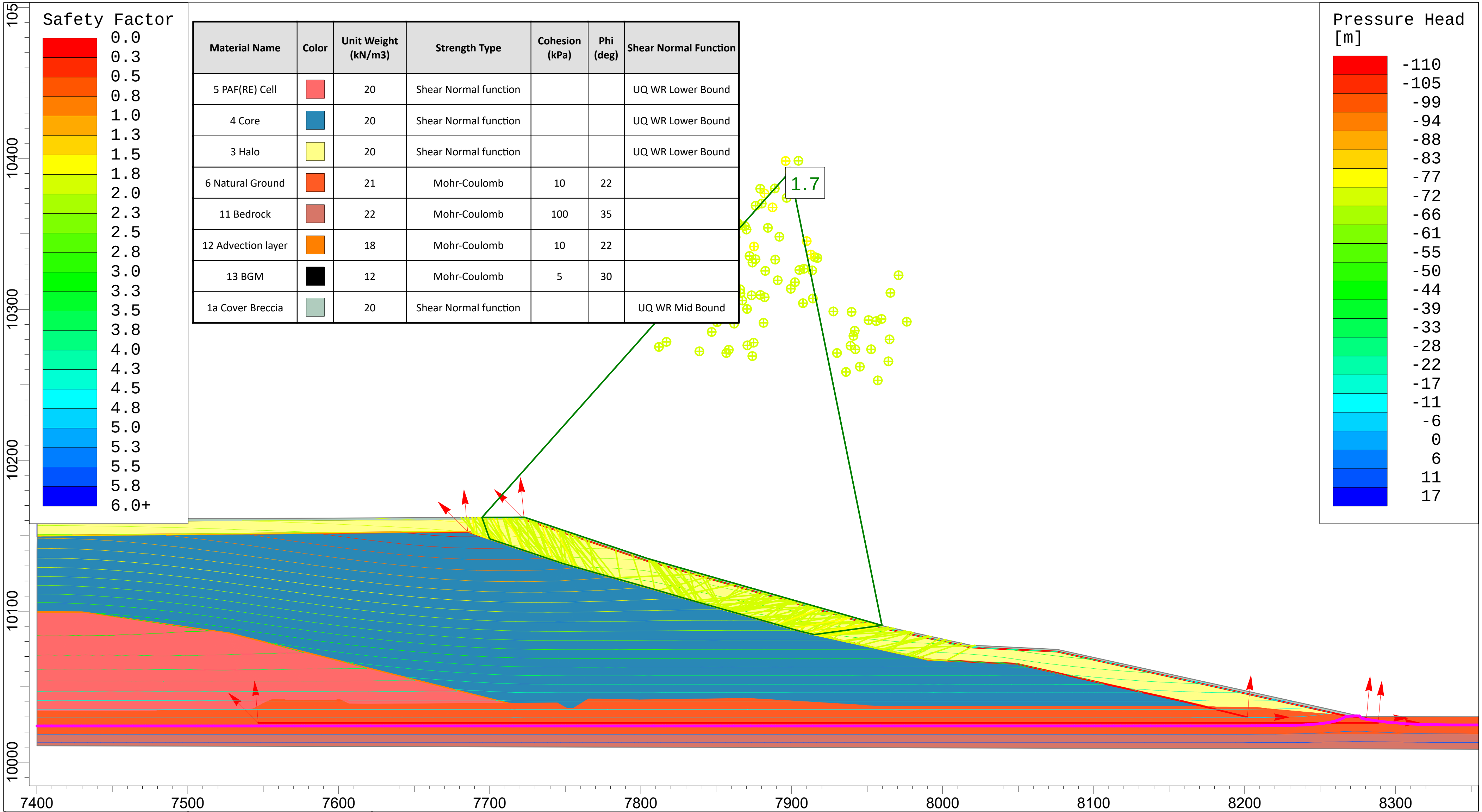

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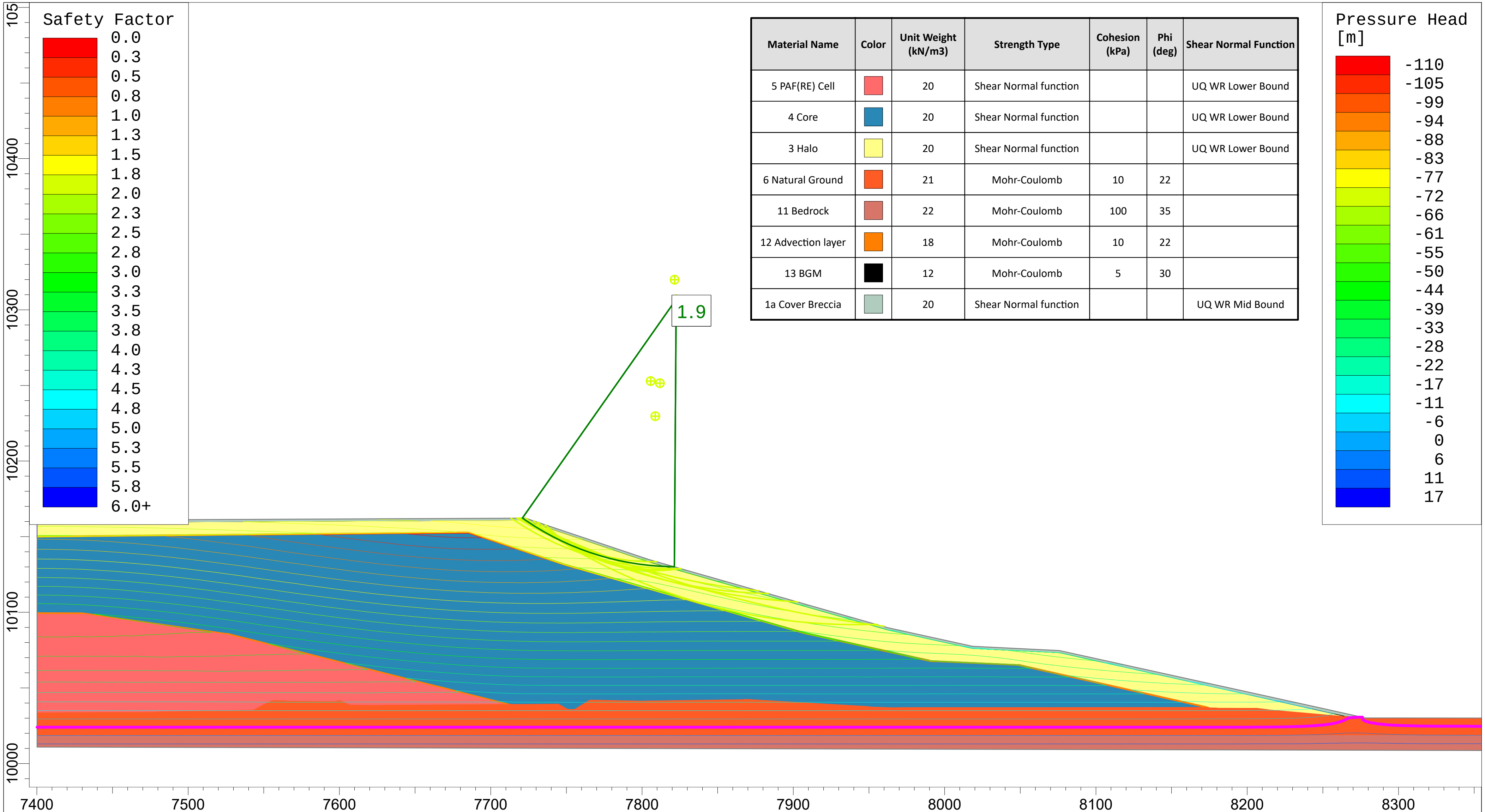

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3 Halo		20	Shear Normal function			UQ WR Lower Bound
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11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
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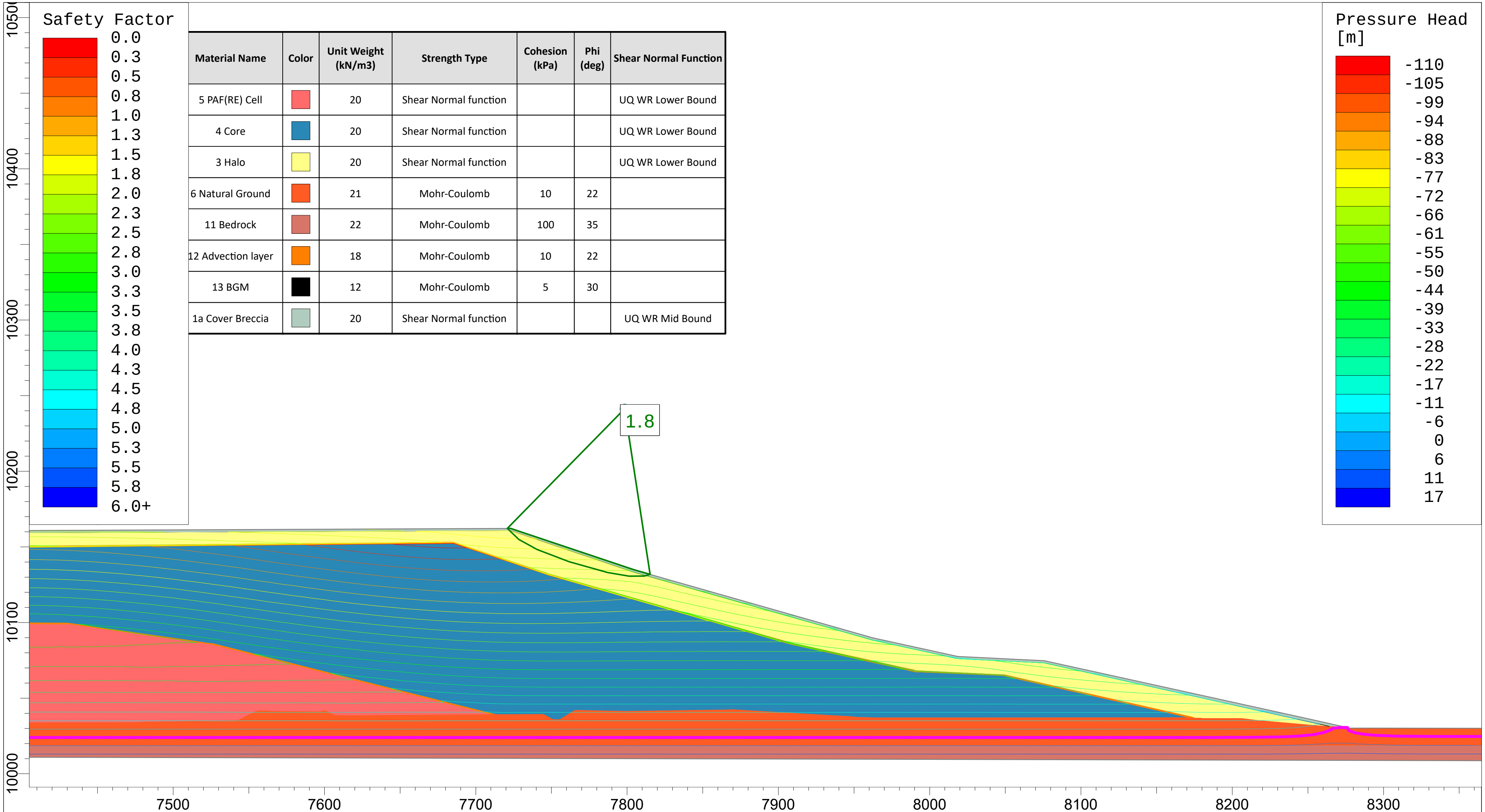

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Project	MRM NOEF EIS		
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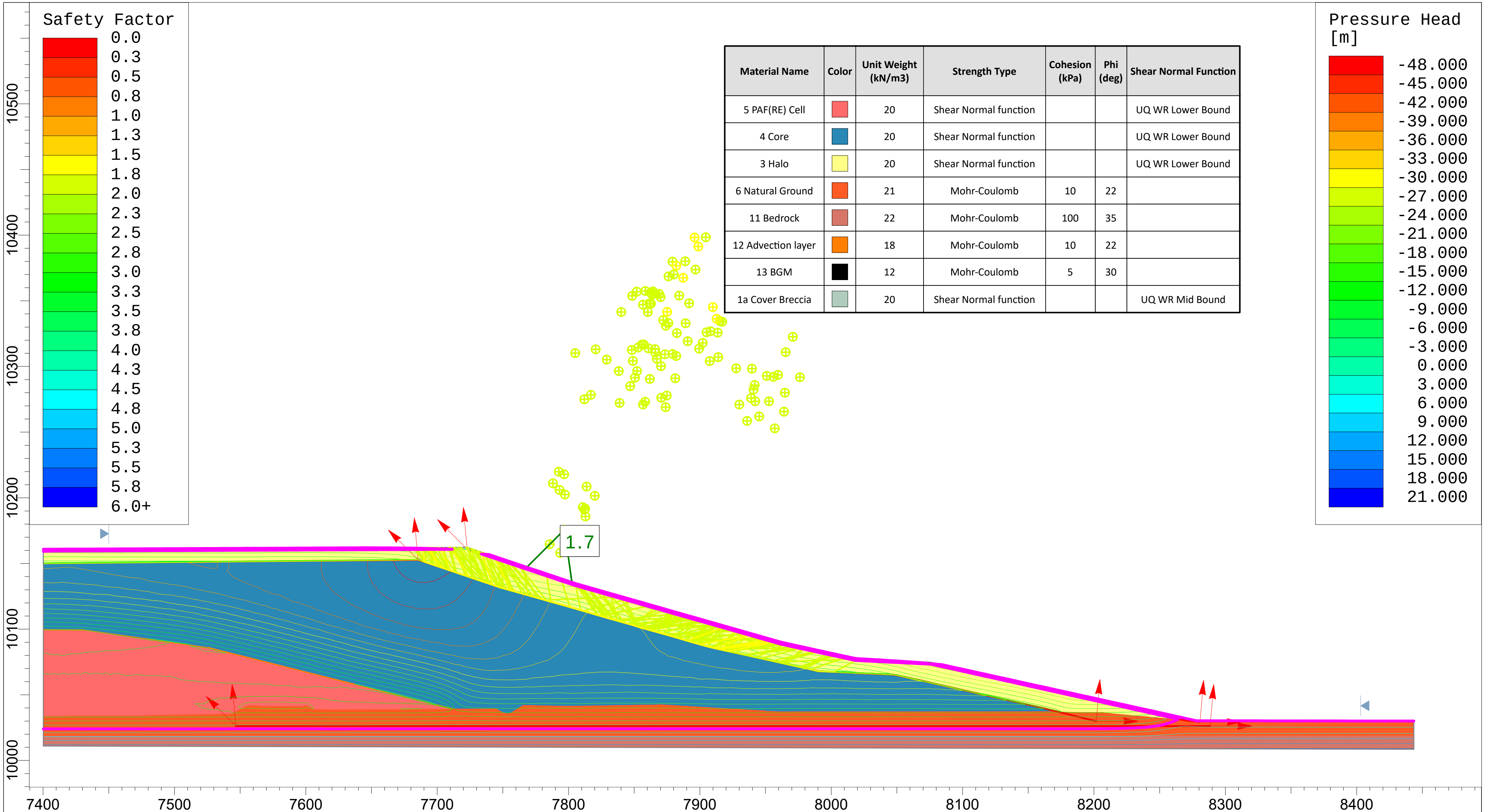

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11 Bedrock		22	Mohr-Coulomb	100	35	
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Project MRM NOEF EIS

Description:
Section A-A Sensitivity 1:1000 Event

Analysis Method: Block

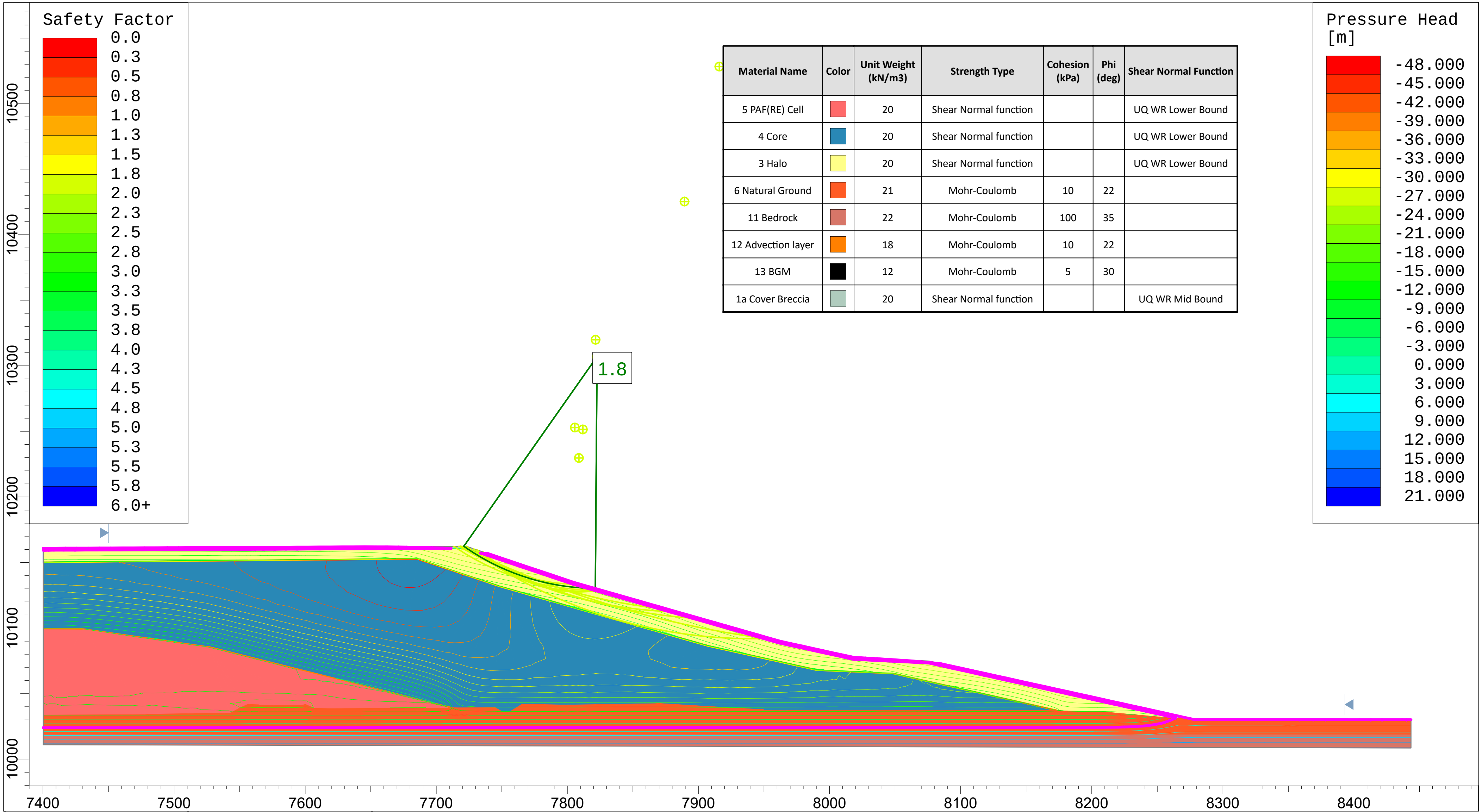
Drawn By SB

Scale 1:4062

Company MRM/Pando

Date November 2017

File Name: section a-a update_bgm_1-1000.slmd



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Project	MRM NOEF EIS
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Company	MRM/Pando
Date	November 2017
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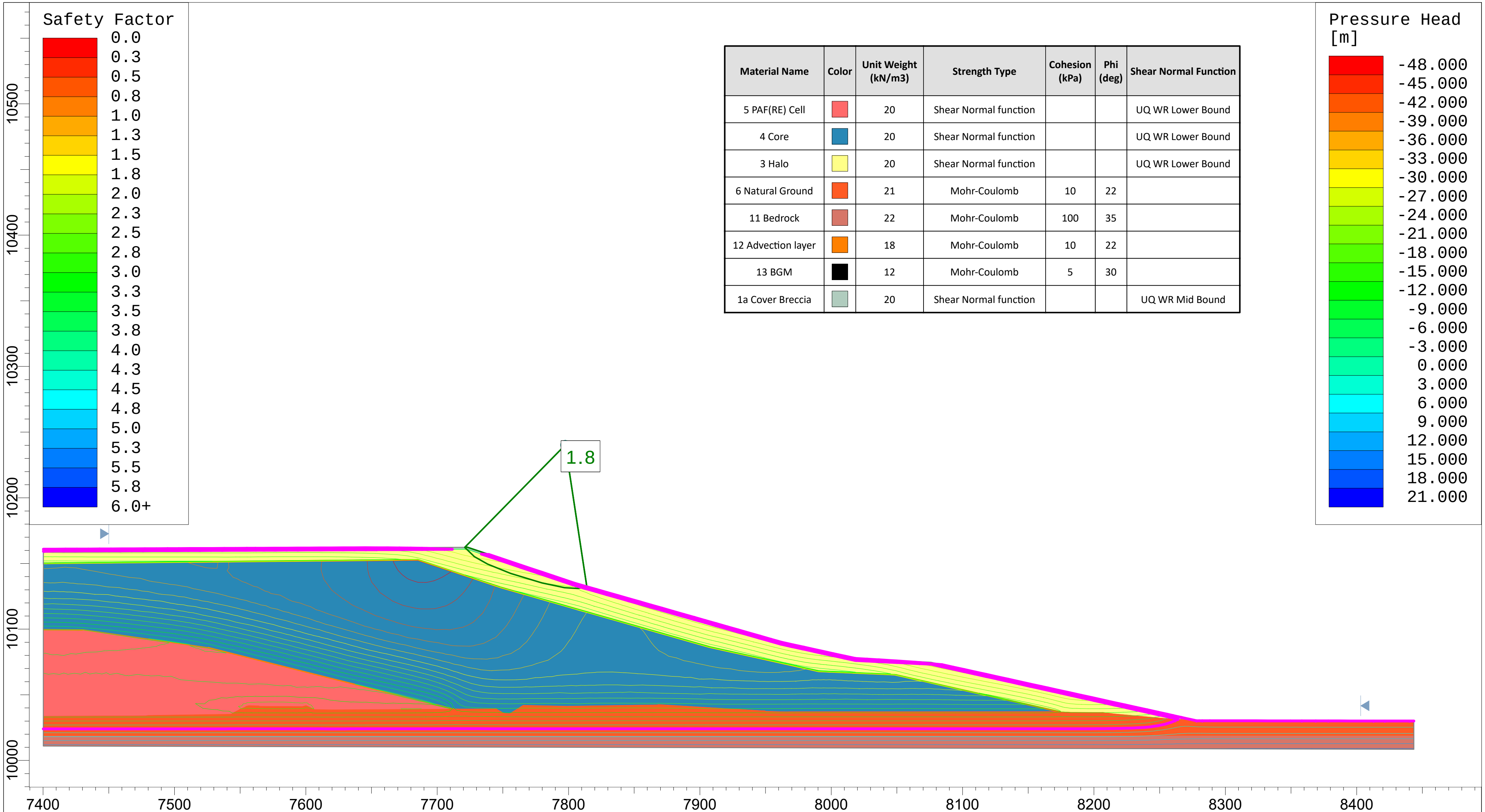

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Project	MRM NOEF EIS		
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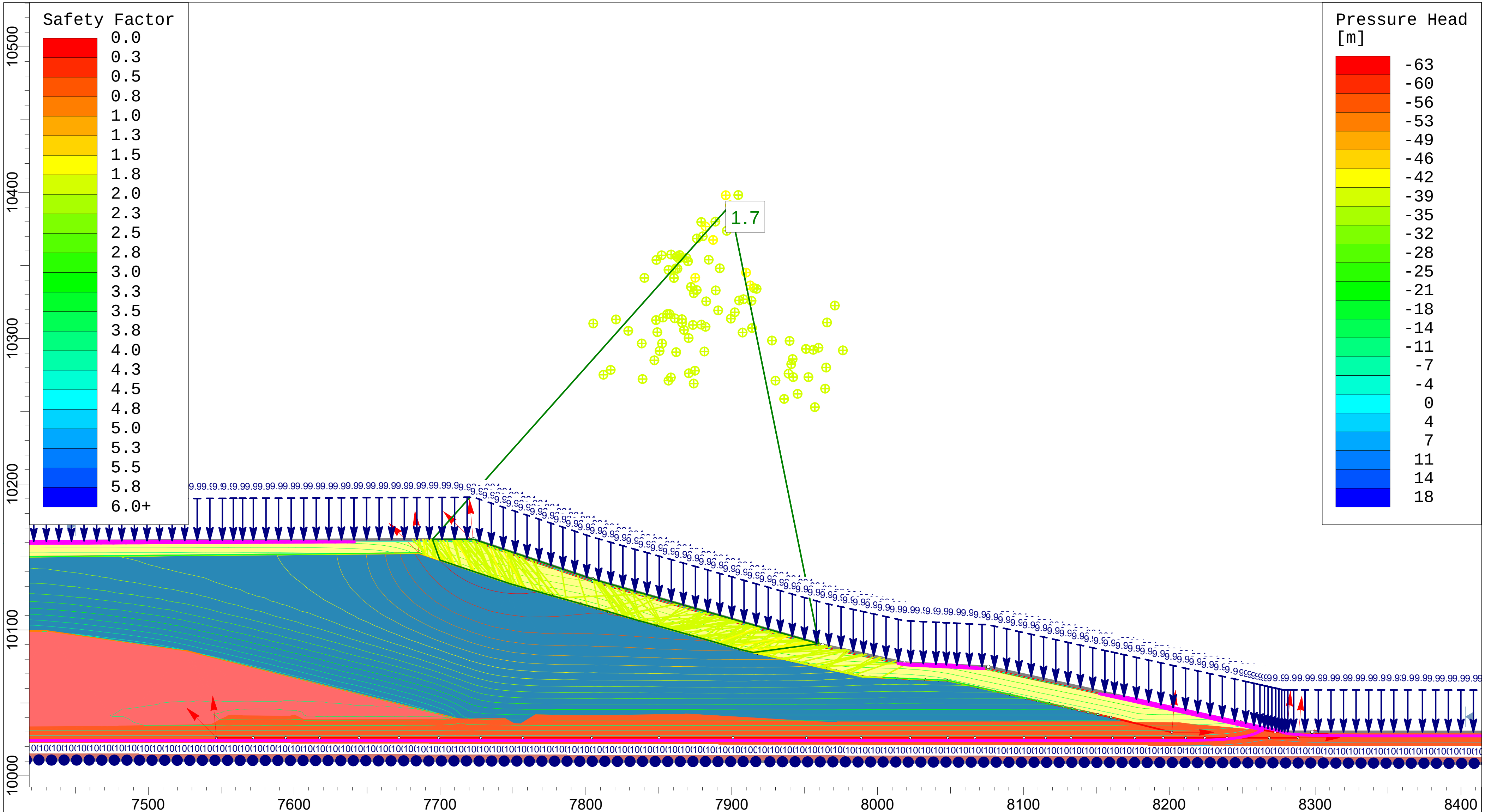

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Date: November 2017		File Name: section a-a update_bgm_wet.slmd

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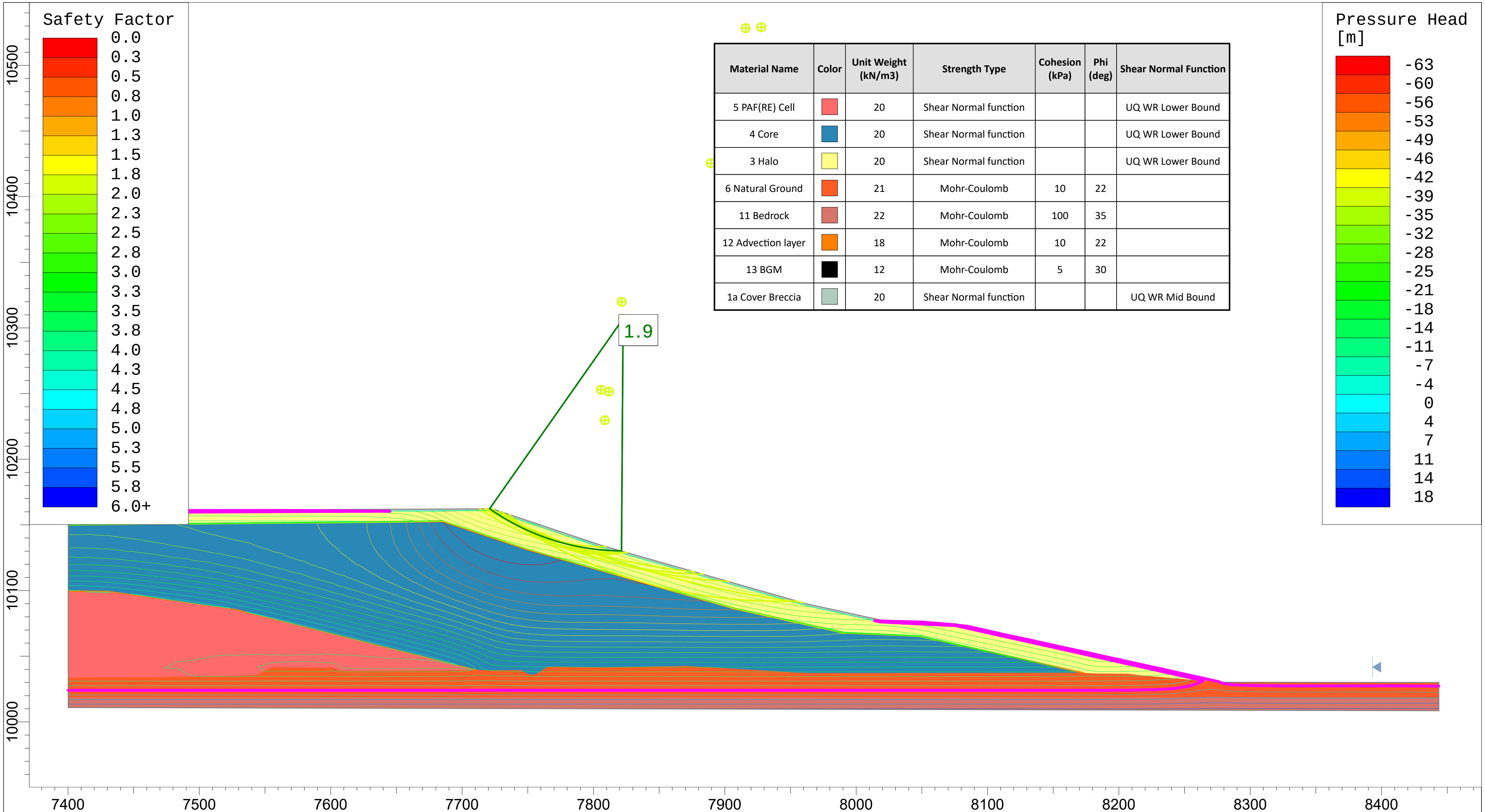

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Project	MRM NOEF EIS		
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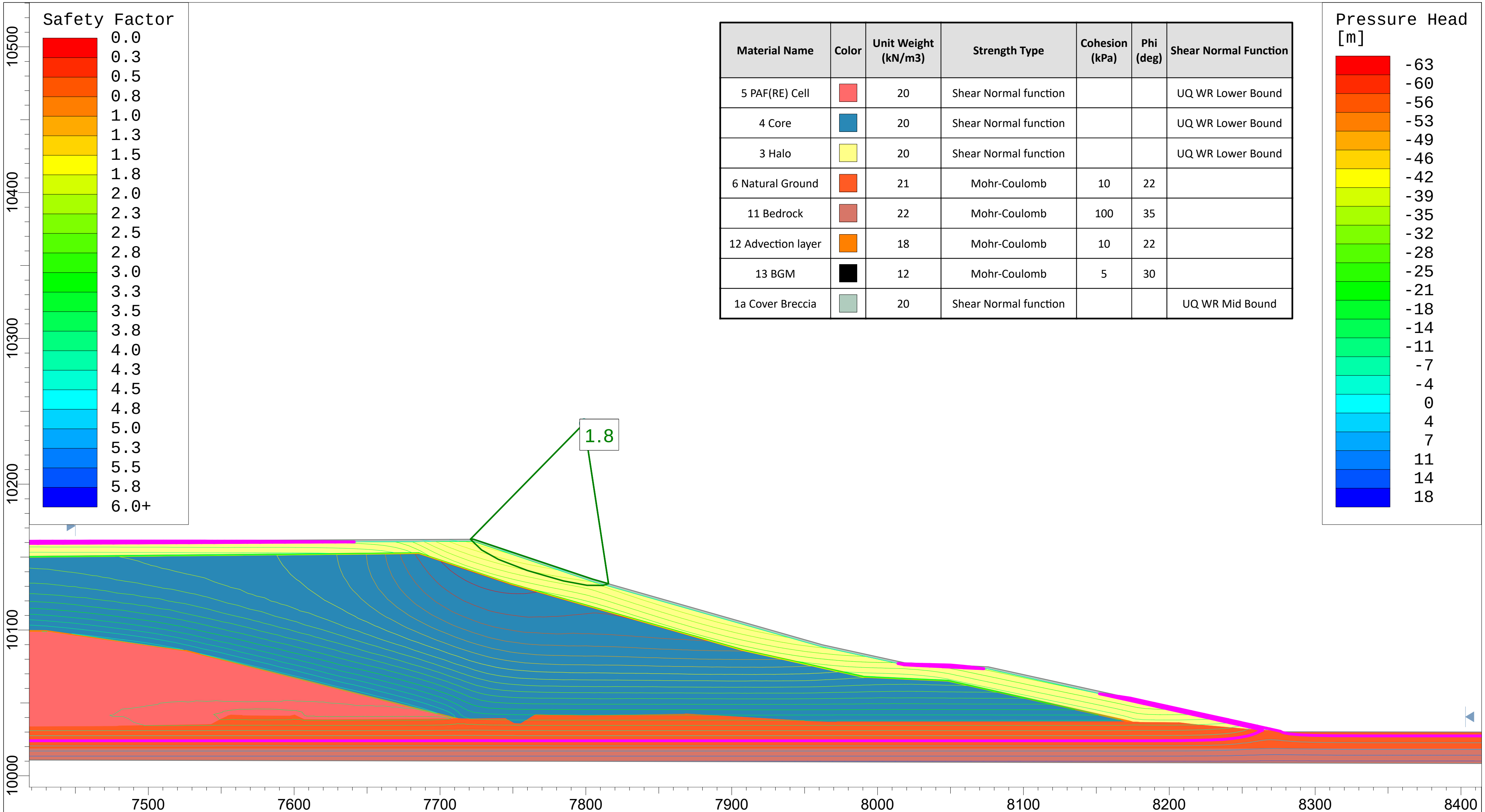
Date	November 2017	*File Name:*	section a-a update_bgm_wet.slmd



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13 BGM		12	Mohr-Coulomb	5	30	
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Project	MRM NOEF EIS		
Description: Section A-A Wet Season		Analysis Method: Cuckoo	
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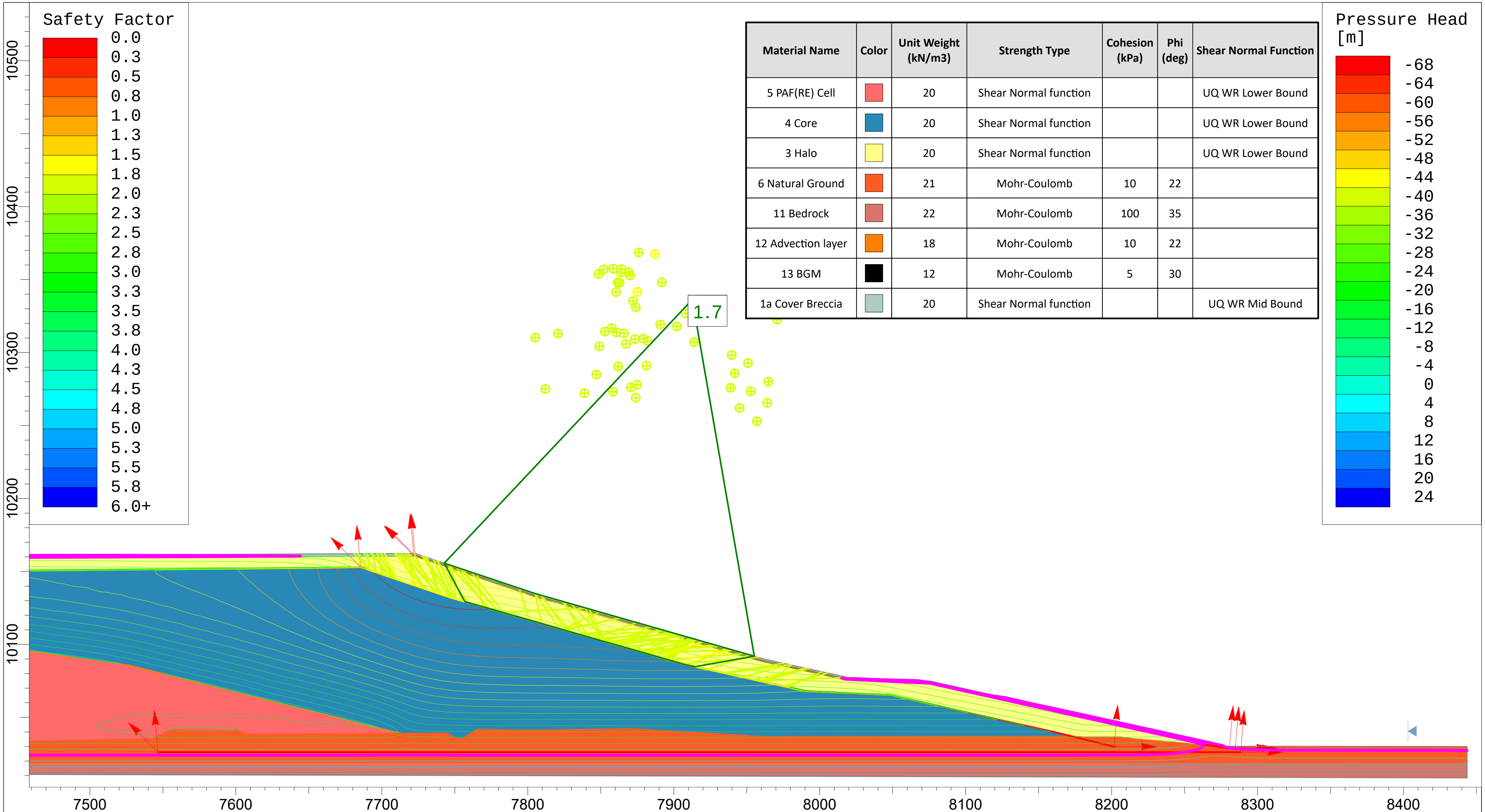

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6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

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SLIDEINTERPRET 7.030

Project	MRM NOEF EIS
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Company	MRM/Pando
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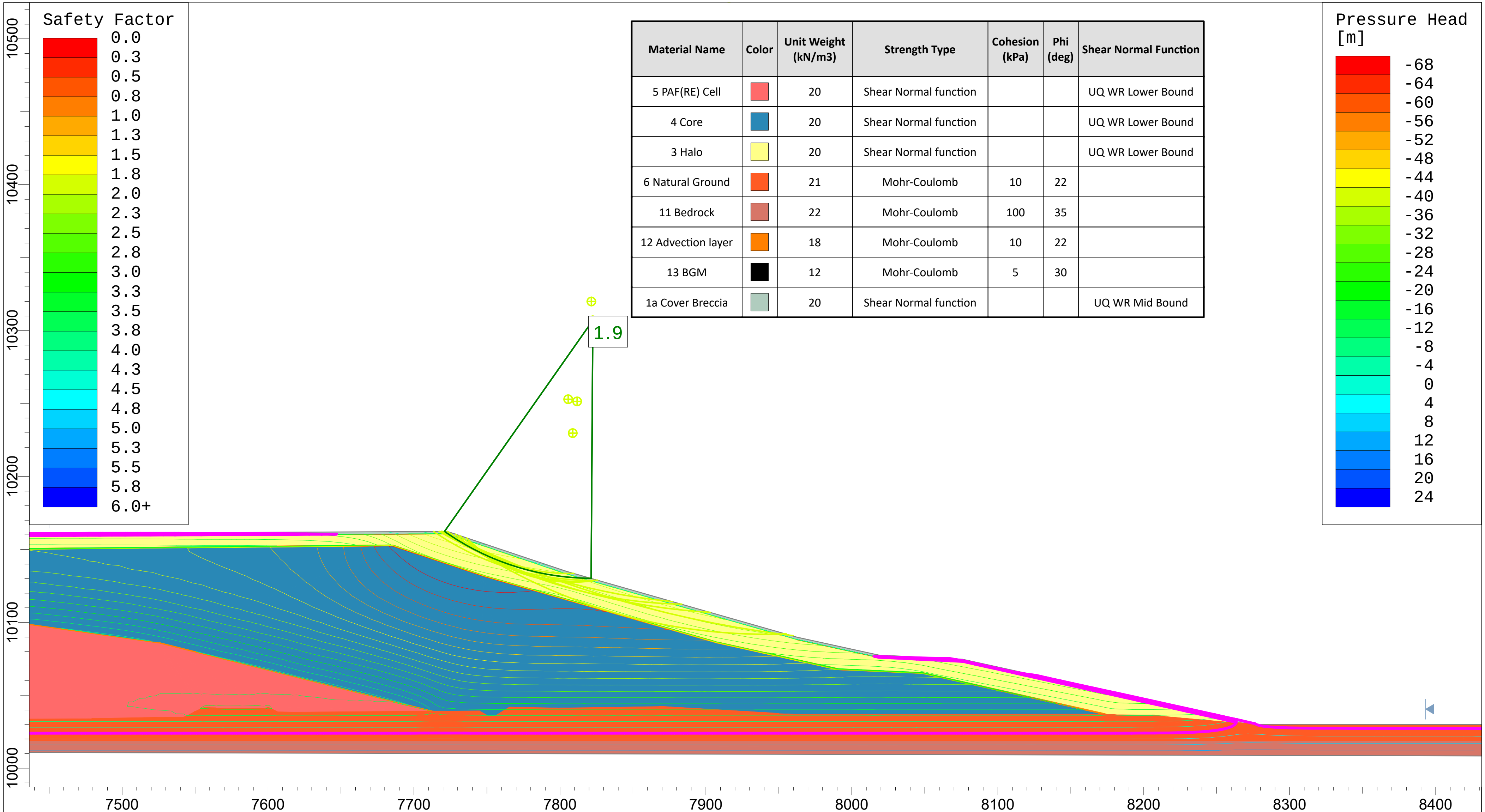


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project MRM NOEF EIS

Description:
Section A-A Wet Season Alluvium Under Liner

Analysis Method: Circular

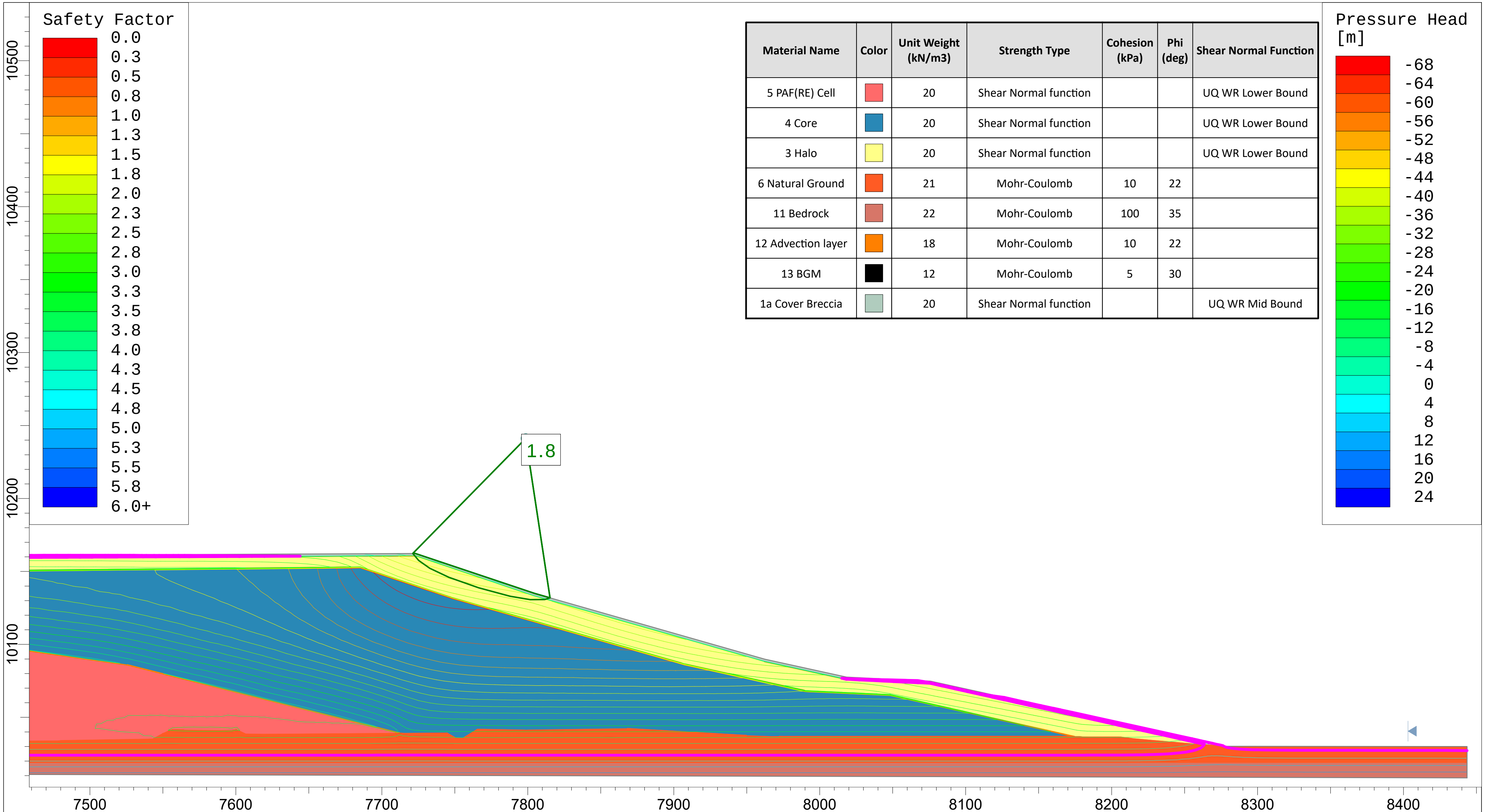
Drawn By SB

Scale 1:3656

Company MRM/Pando

Date November 2017

File Name: section a-a update_bgm_wet_uliner.slmd



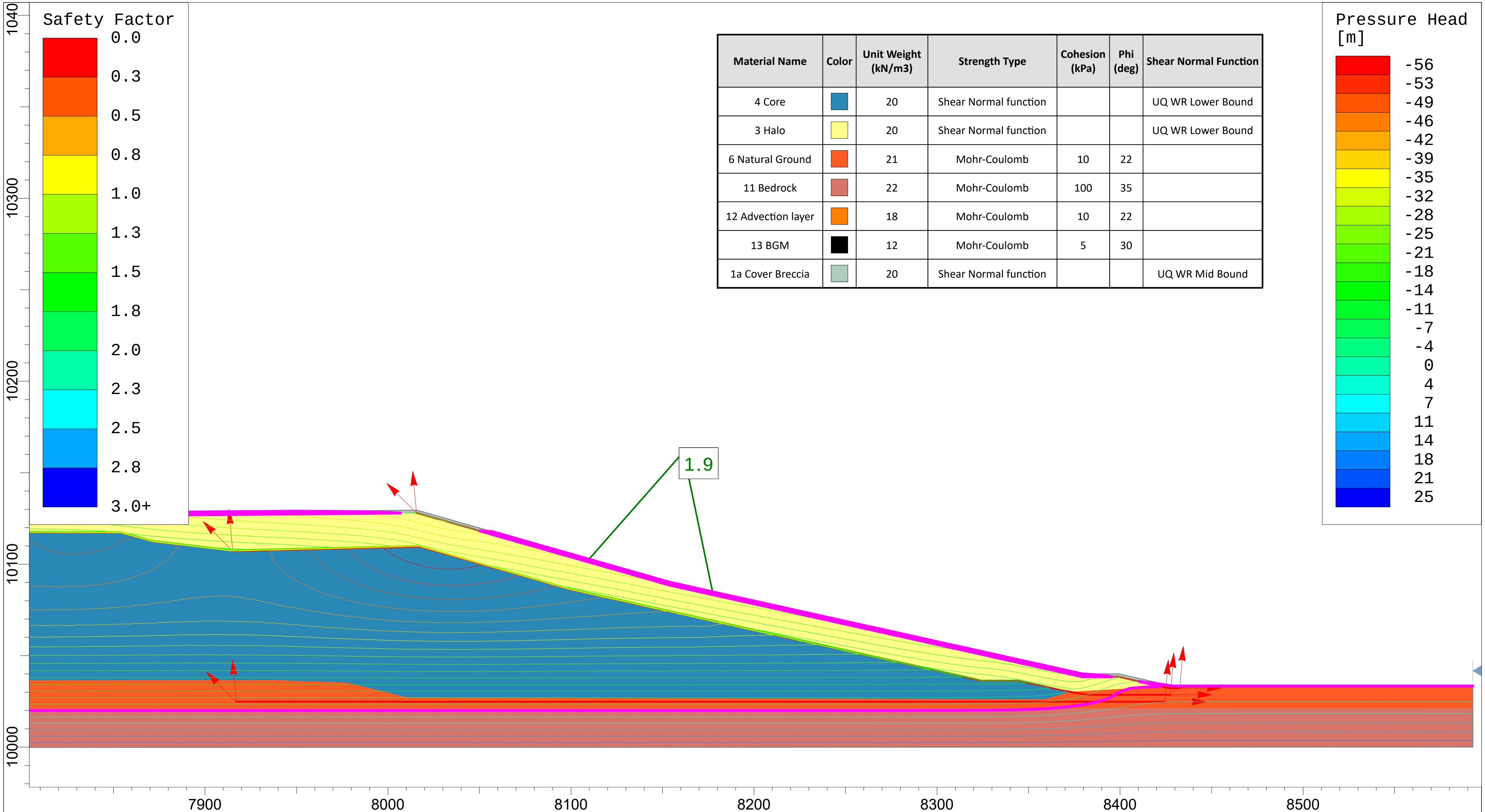
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section A-A Wet Season Alluvium Under Liner		Analysis Method: Cuckoo	
Drawn By SB	*Scale* 1:3656	*Company*	MRM/Pando
Date	November 2017	*File Name:*	section a-a update_bgm_wet_uliner.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

SLIDEINTERPRET 7.030

Project MRM NOEF EIS

Description:
Section B-B 1:1000

Analysis Method: Block

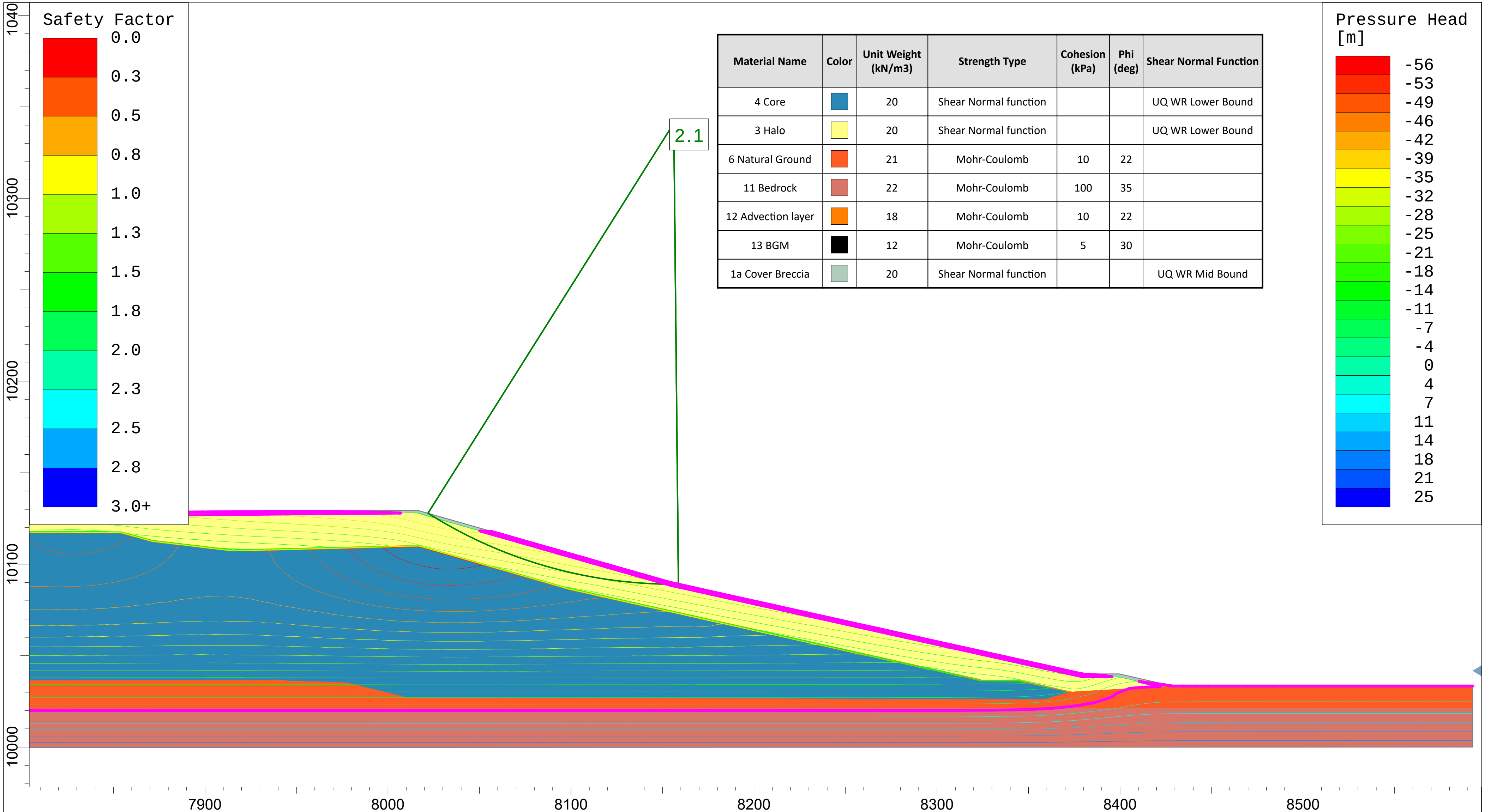
Drawn By SB

Scale 1:2915

Company MRM/Pando

Date November 2017

File Name: section b-b update_bgm_1-1000.slmd



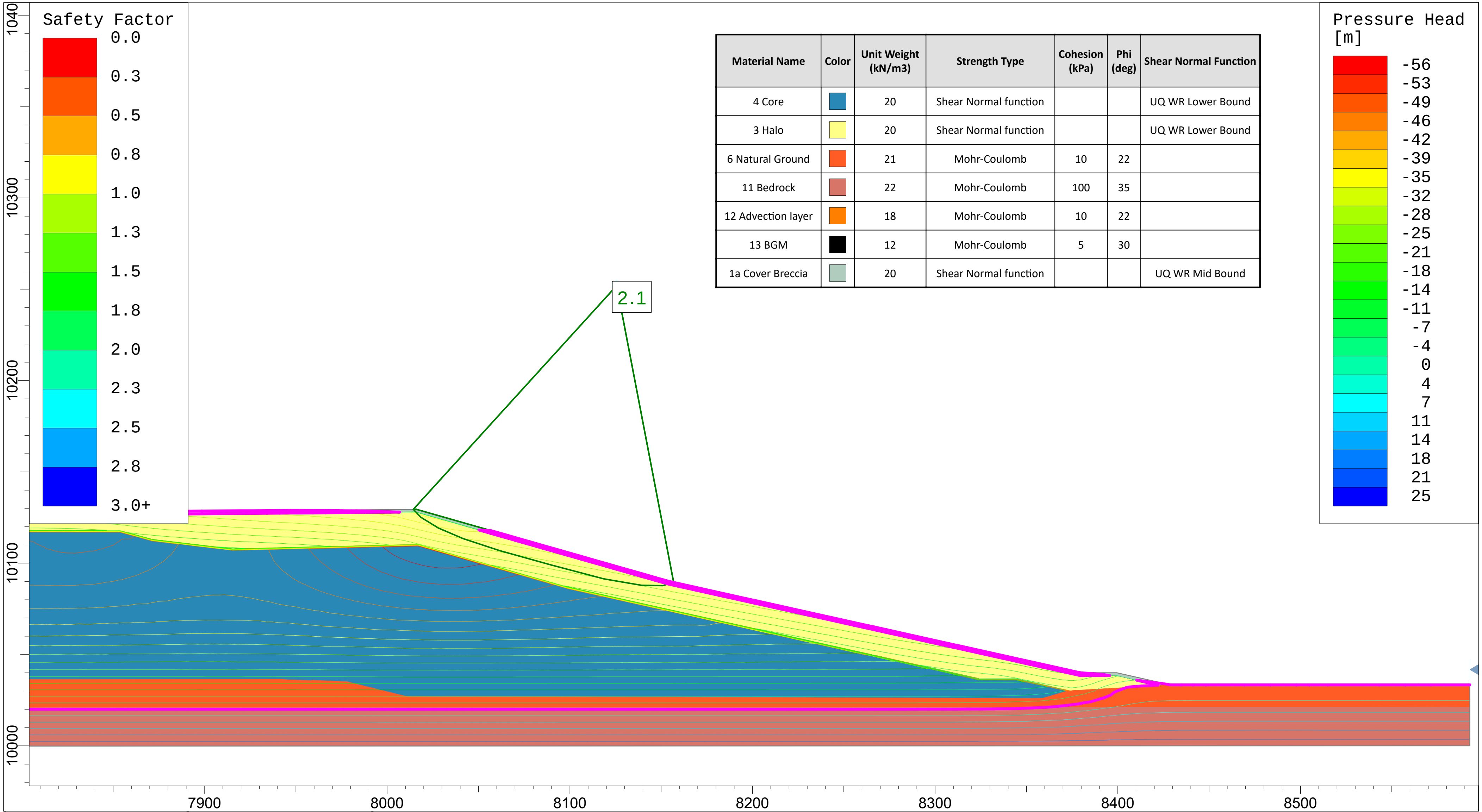
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section B-B 1:1000		Analysis Method: Circular	
Drawn By SB	*Scale* 1:2915	*Company*	MRM/Pando
Date	November 2017	*File Name:*	section b-b update_bgm_1-1000.slmd



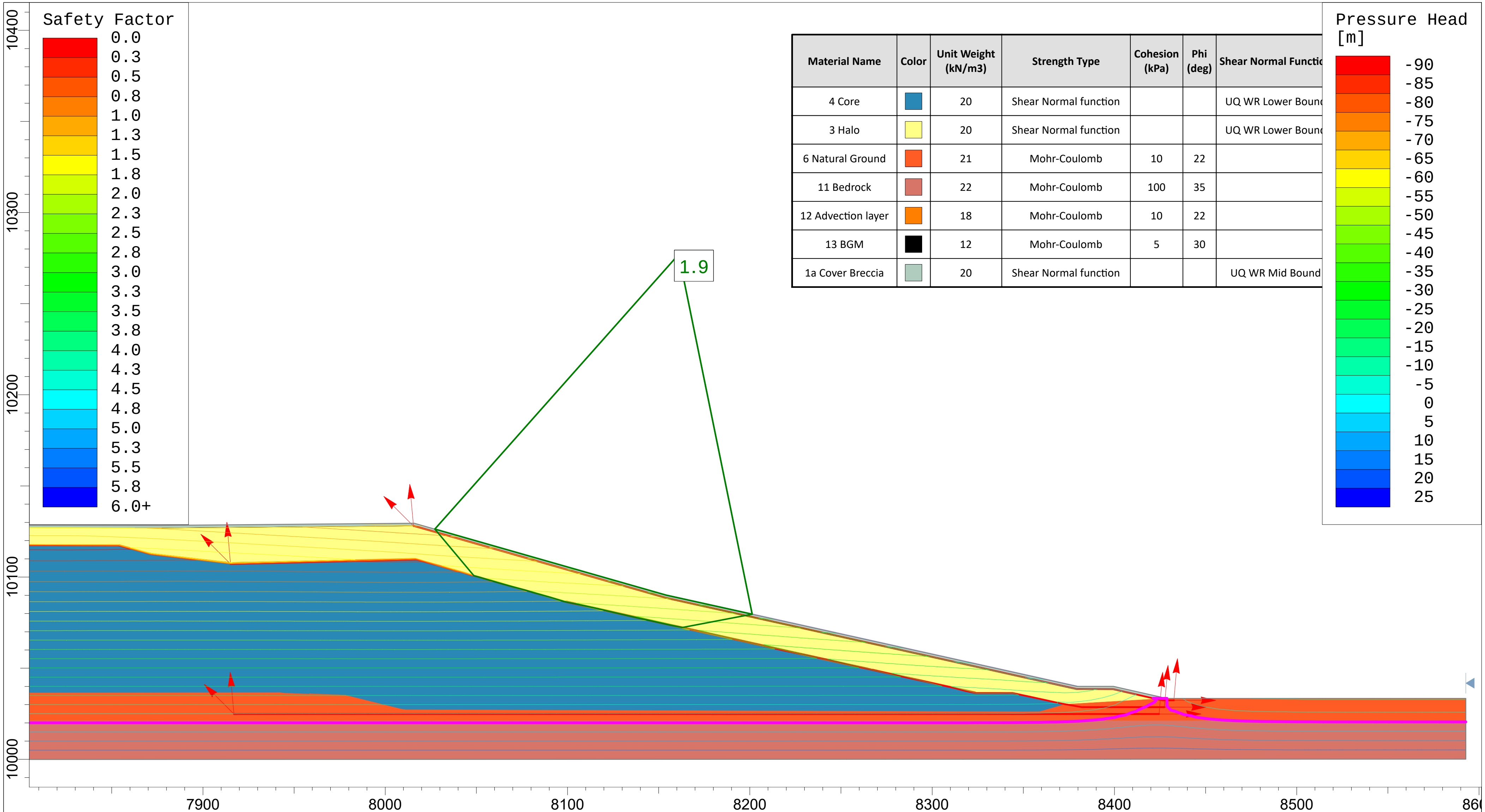
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

SLIDEINTERPRET 7.030

McArthur River Mining

Project	MRM NOEF EIS		
Description: Section B-B 1:1000		Analysis Method: Cuckoo	
Drawn By SB	*Scale* 1:2915	*Company*	MRM/Pando
Date	November 2017	*File Name:*	section b-b update_bgm_1-1000.slmd



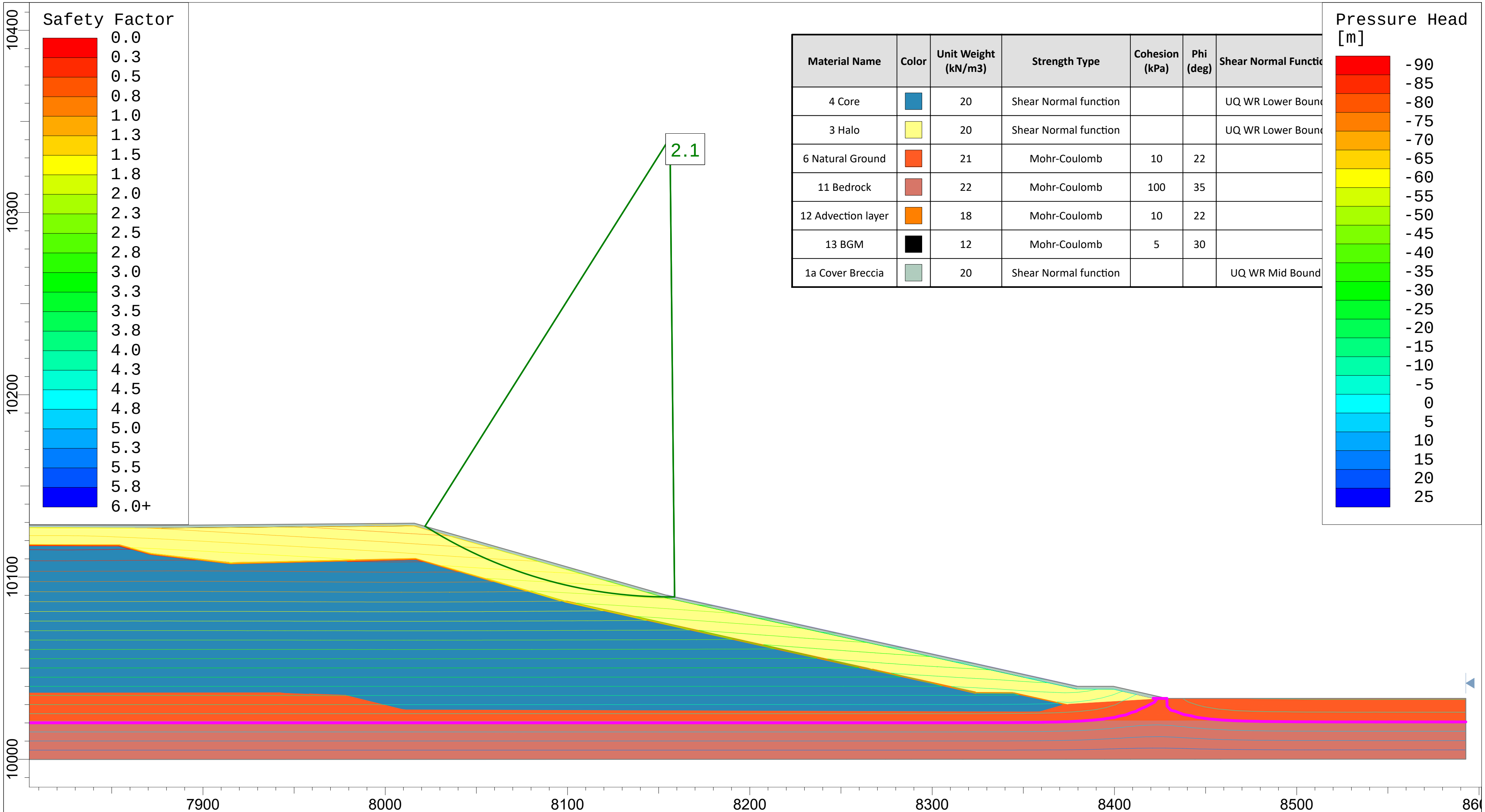
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Functio
4 Core		20	Shear Normal function			UQ WR Lower Boun
3 Halo		20	Shear Normal function			UQ WR Lower Boun
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section B-B Dry Season		Analysis Method: Block	
Drawn By SB	*Scale* 1:2927	*Company*	MRM/Pando
Date	November 2017	*File Name:*	section b-b update_bgm_dry.slmd



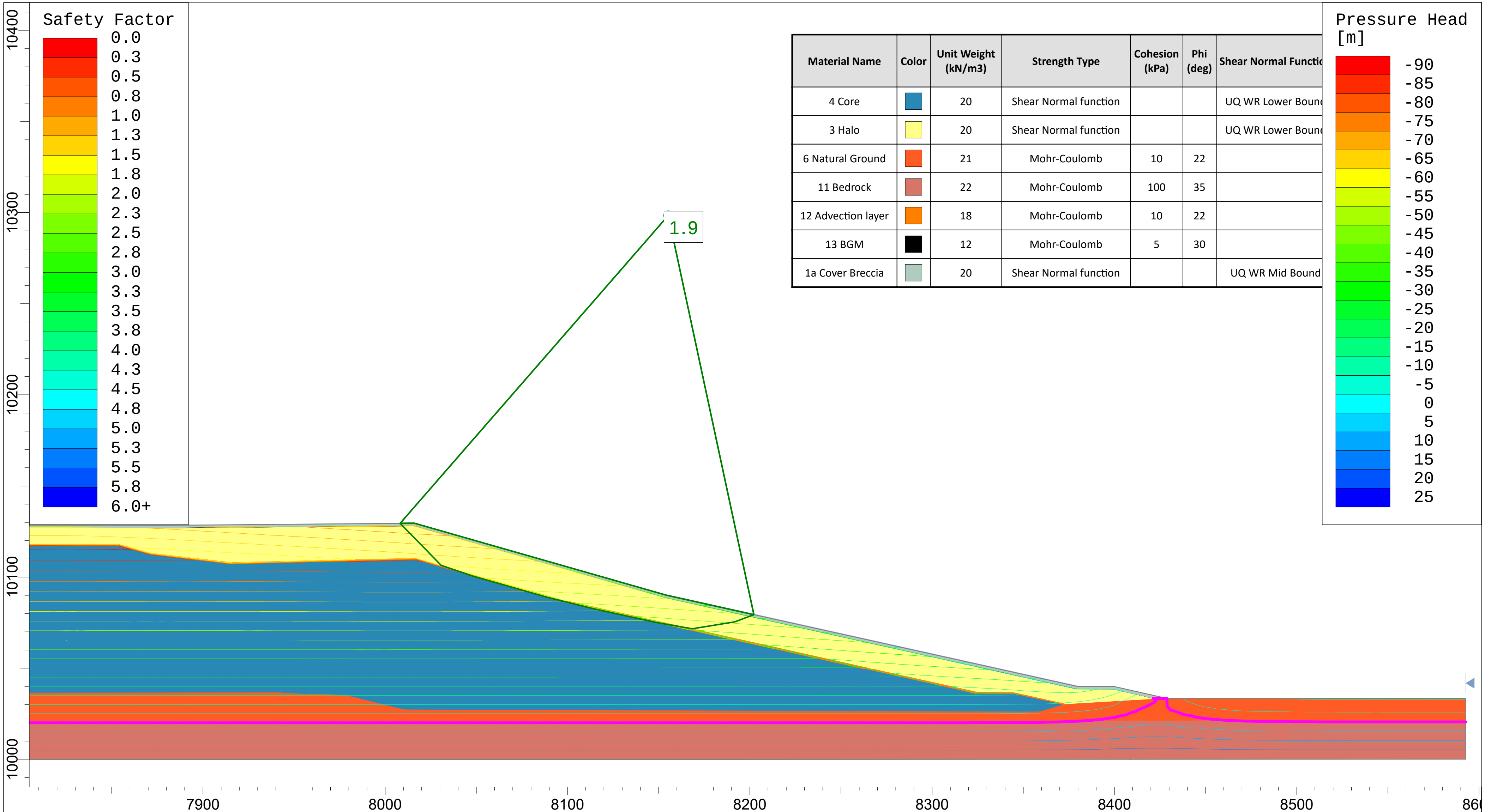
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Functic
4 Core		20	Shear Normal function			UQ WR Lower Bounc
3 Halo		20	Shear Normal function			UQ WR Lower Bounc
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

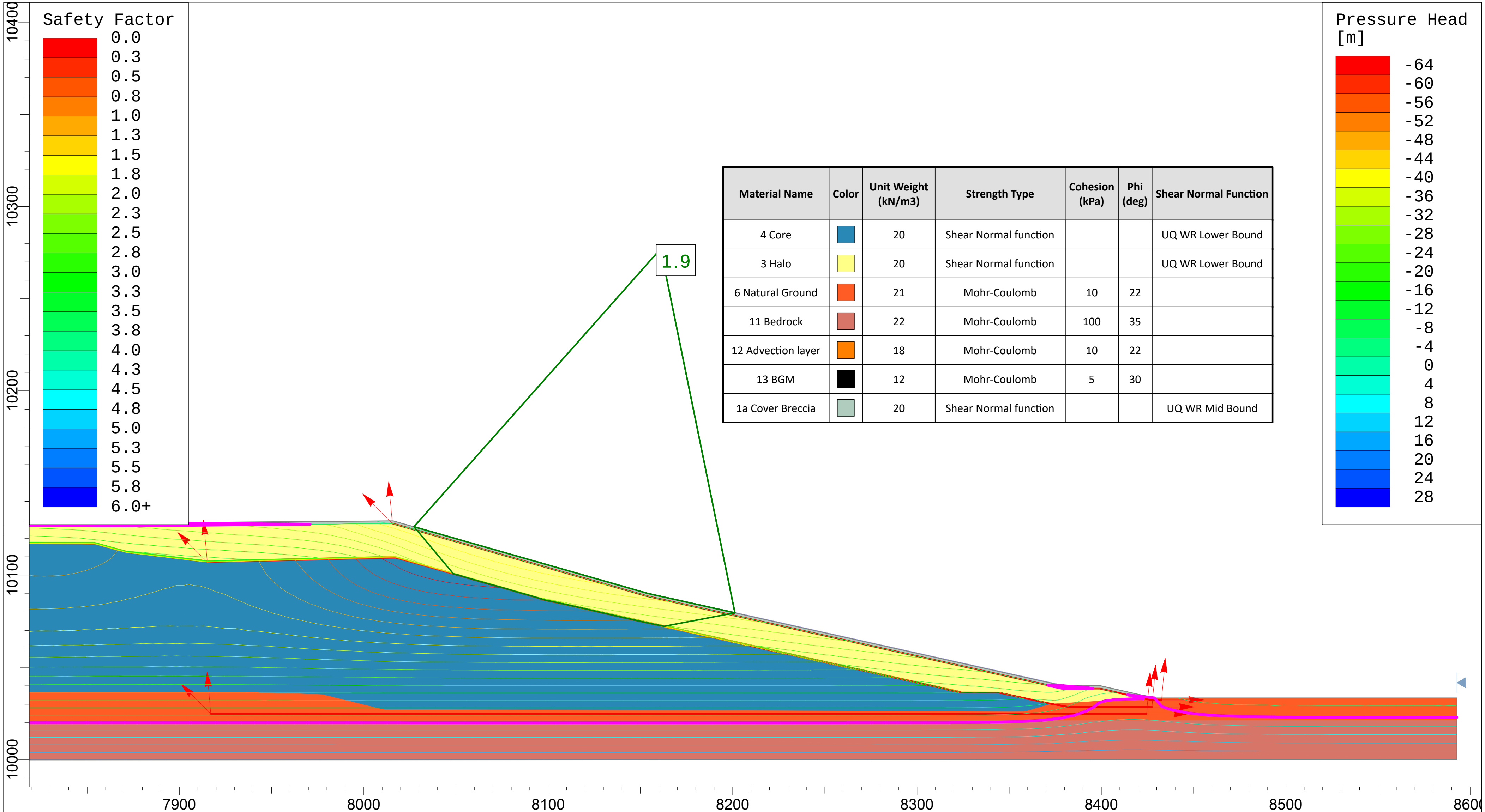
McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section B-B Dry Season		Analysis Method: Circular	
Drawn By SB	*Scale* 1:2927	*Company*	MRM/Pando
Date November 2017		*File Name:*	section b-b update_bgm_dry.slmd



Pando	McArthur River Mining	*Project*	MRM NOEF EIS
		Description: Section B-B Dry Season	Analysis Method: Cuckoo
		Drawn By SB	*Scale* 1:2927
		Company	MRM/Pando
SLIDEINTERPRET 7.030		*Date* November 2017	*File Name:* section b-b update_bgm_dry.slmd



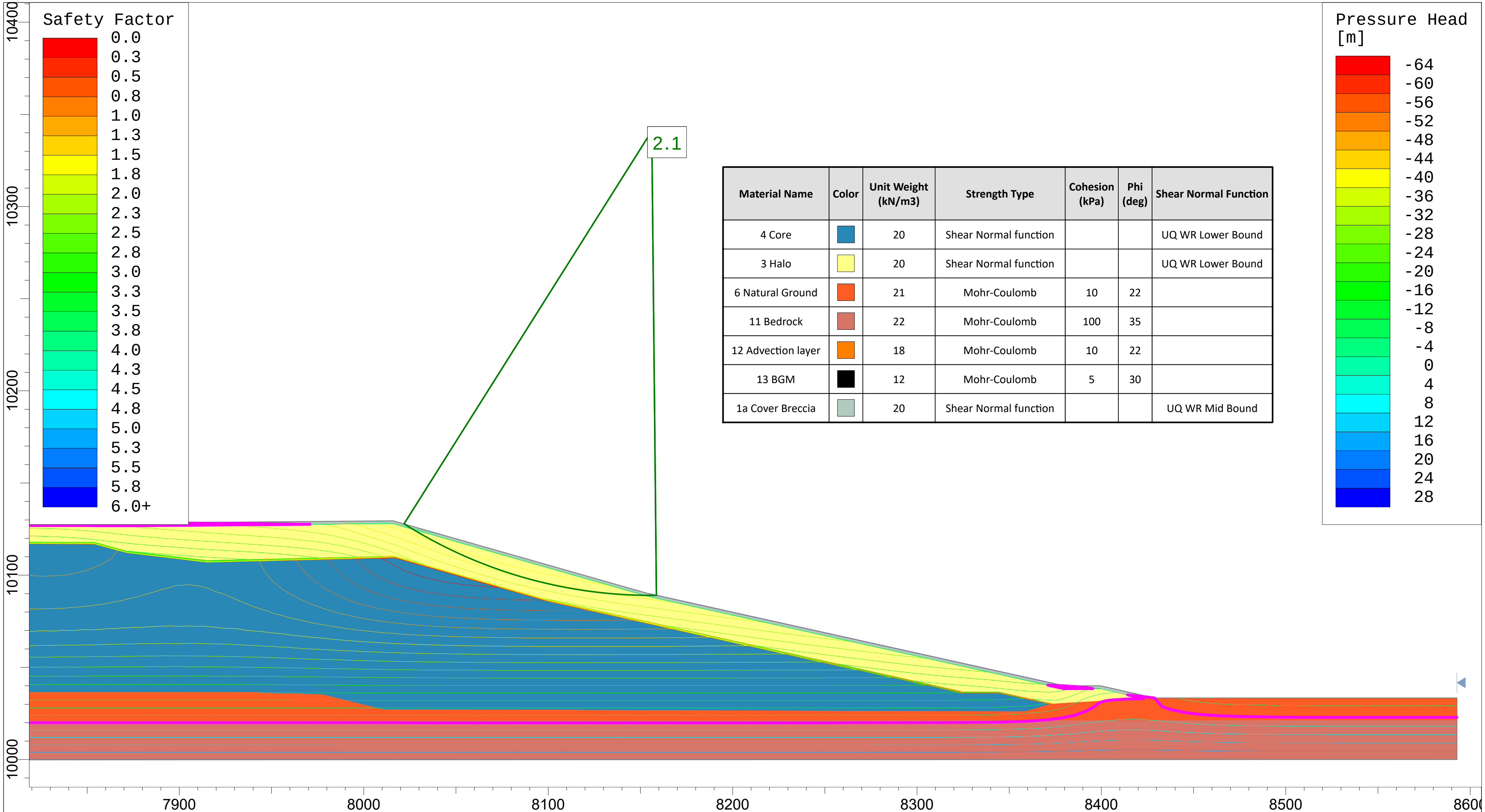
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section B-B Wet Season		Analysis Method: Block	
Drawn By: SB	Scale: 1:2893	Company: MRM/Pando	
Date: November 2017		File Name: section b-b update_bgm_wet.slmd	



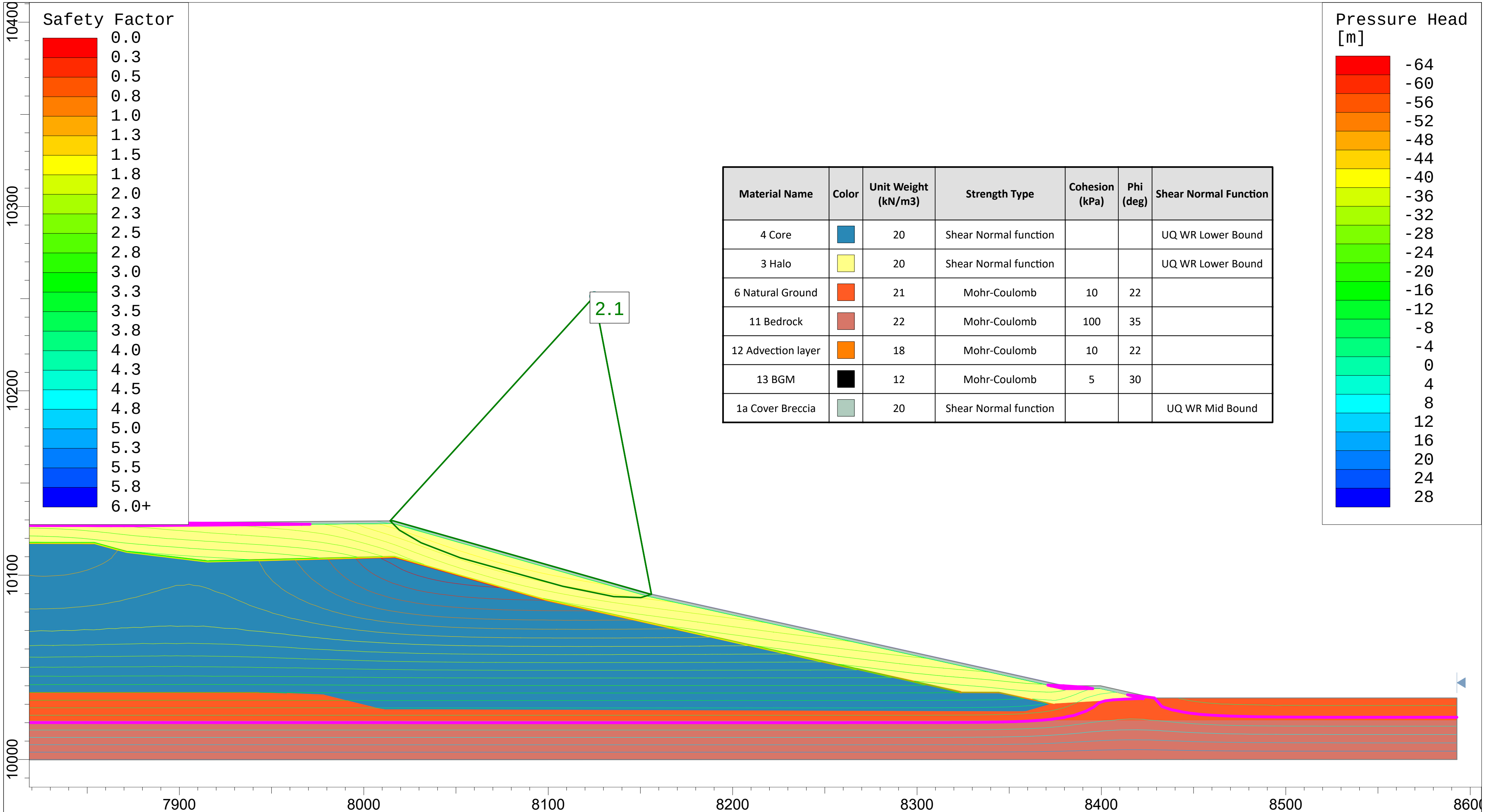
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

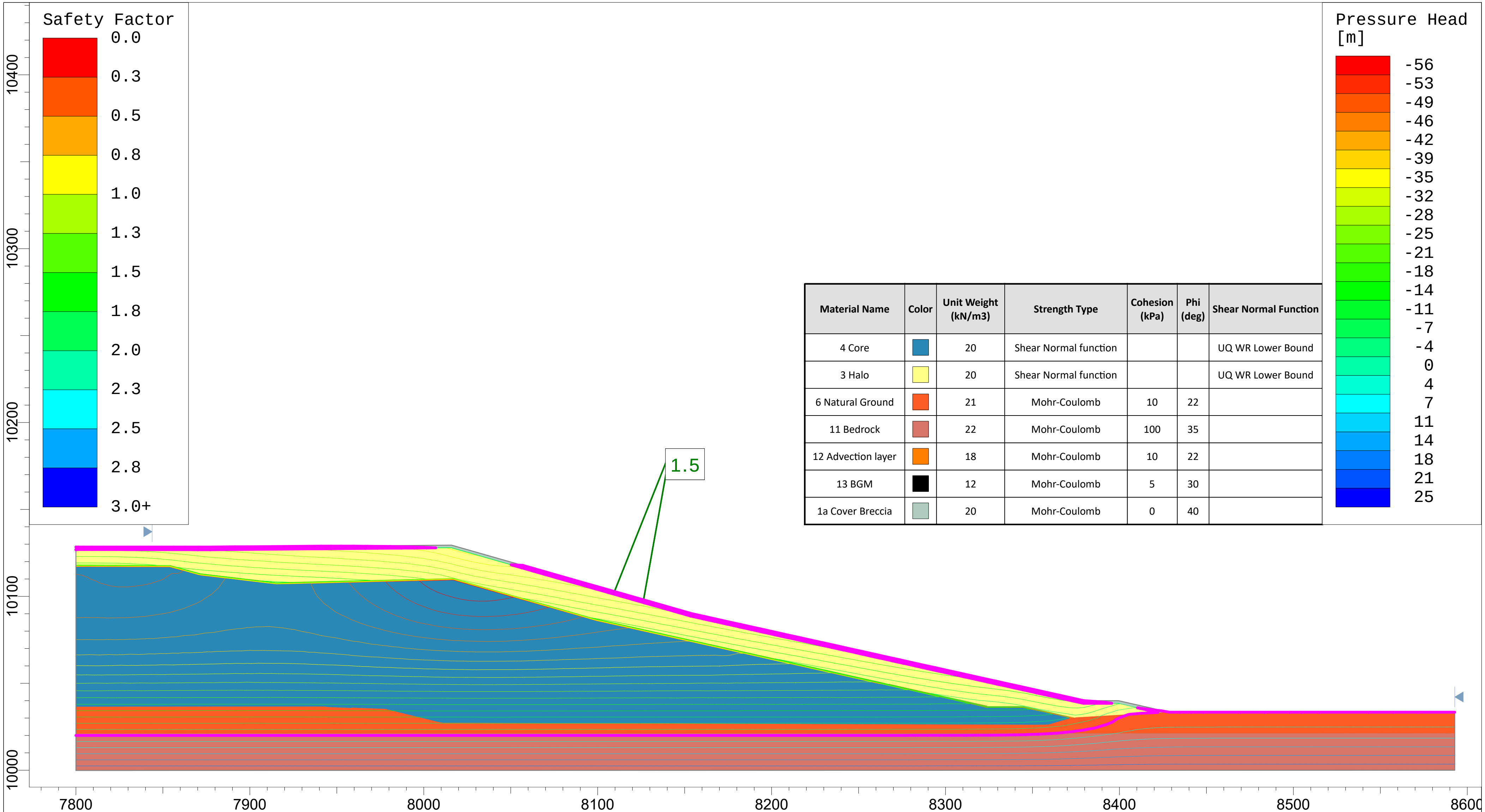
Project	MRM NOEF EIS		
Description: Section B-B Wet Season		Analysis Method: Circular	
Drawn By SB	*Scale* 1:2893	*Company*	MRM/Pando
Date	November 2017	*File Name:*	section b-b update_bgm_wet.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Project	MRM NOEF EIS		
Description: Section B-B Wet Season		Analysis Method: Cuckoo	
Drawn By	SB	Scale	1:2893
Company	MRM/Pando		
Date	November 2017	File Name:	section b-b update_bgm_wet.slmd

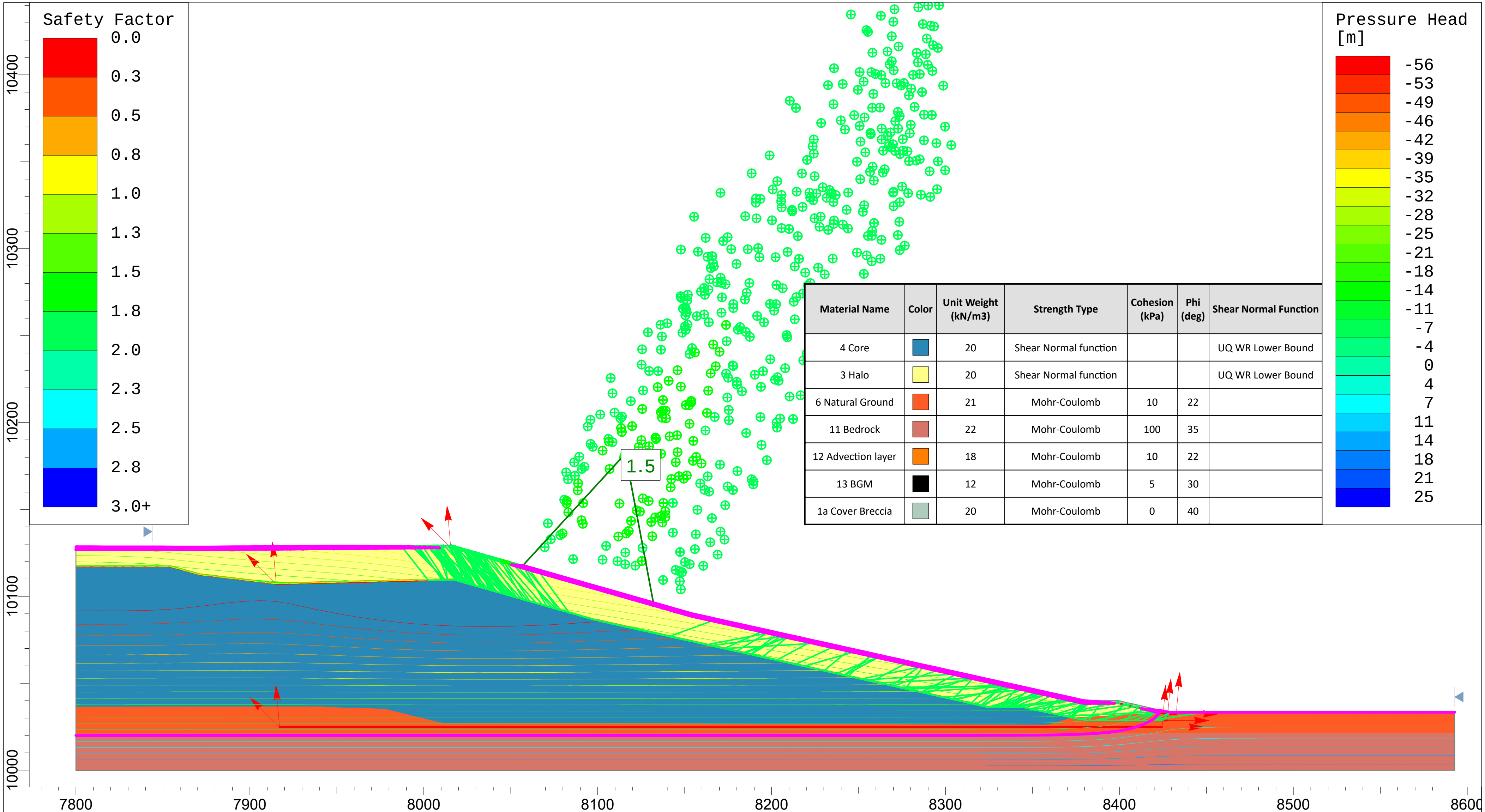
SLIDEINTERPRET 7.030



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

Pando	McArthur River Mining	*Project* MRM NOEF EIS
	Description: Section B-B beccia sensitivity 1:1000	Analysis Method: Circular
	Drawn By SB	*Scale* 1:3068
	Company MRM/Pando	
	Date November 2017	*File Name:* section b-b update_bgm_breccia_1-1000.slmd

SLIDEINTERPRET 7.030



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

SLIDEINTERPRET 7.030

Project: MRM NOEF EIS

Description:
Section B-B beccia sensitivity 1:1000

Analysis Method: Block

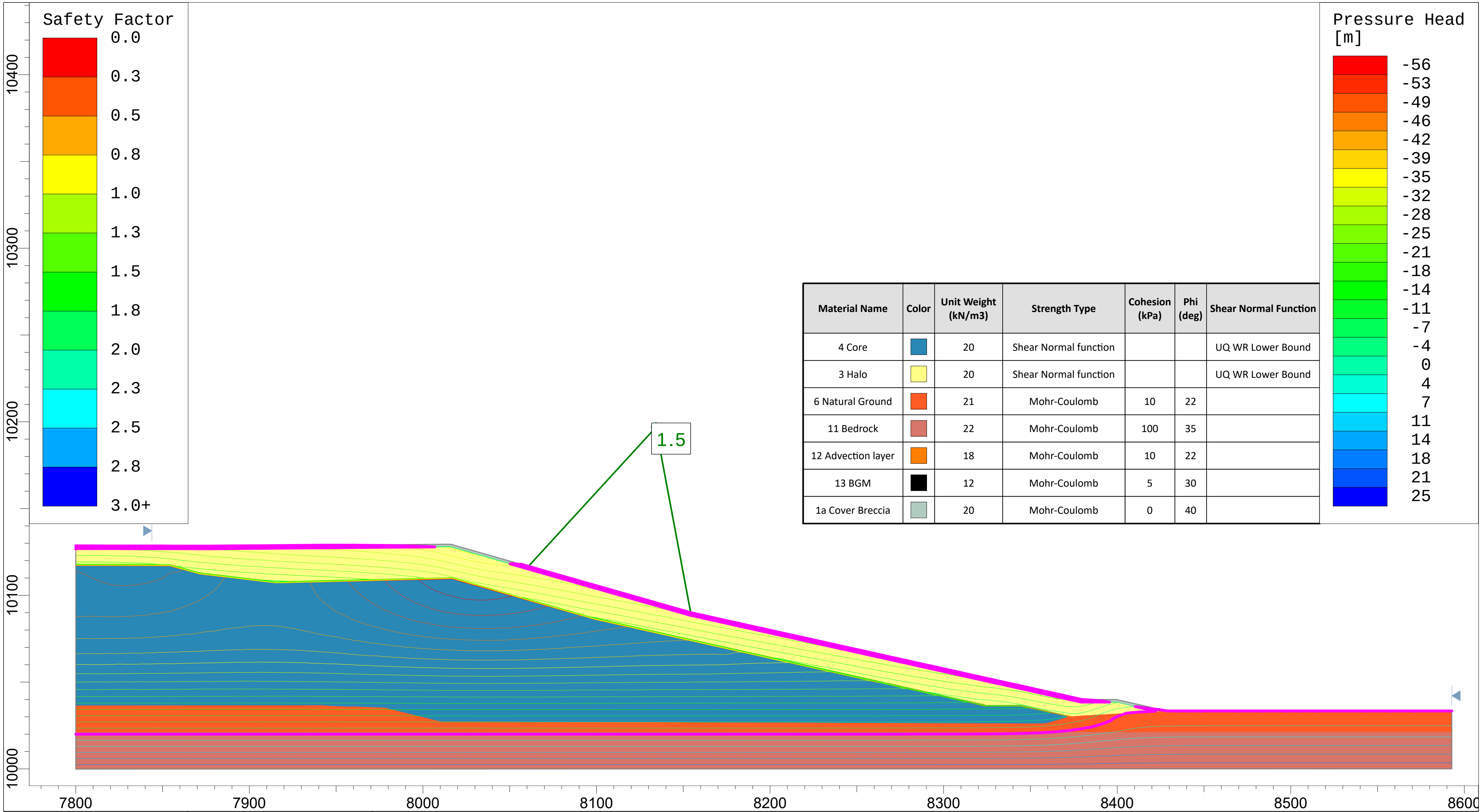
Drawn By: SB

Scale: 1:3068

Company: MRM/Pando

Date: November 2017

File Name: section b-b update_bgm_breccia_1-1000.slmd



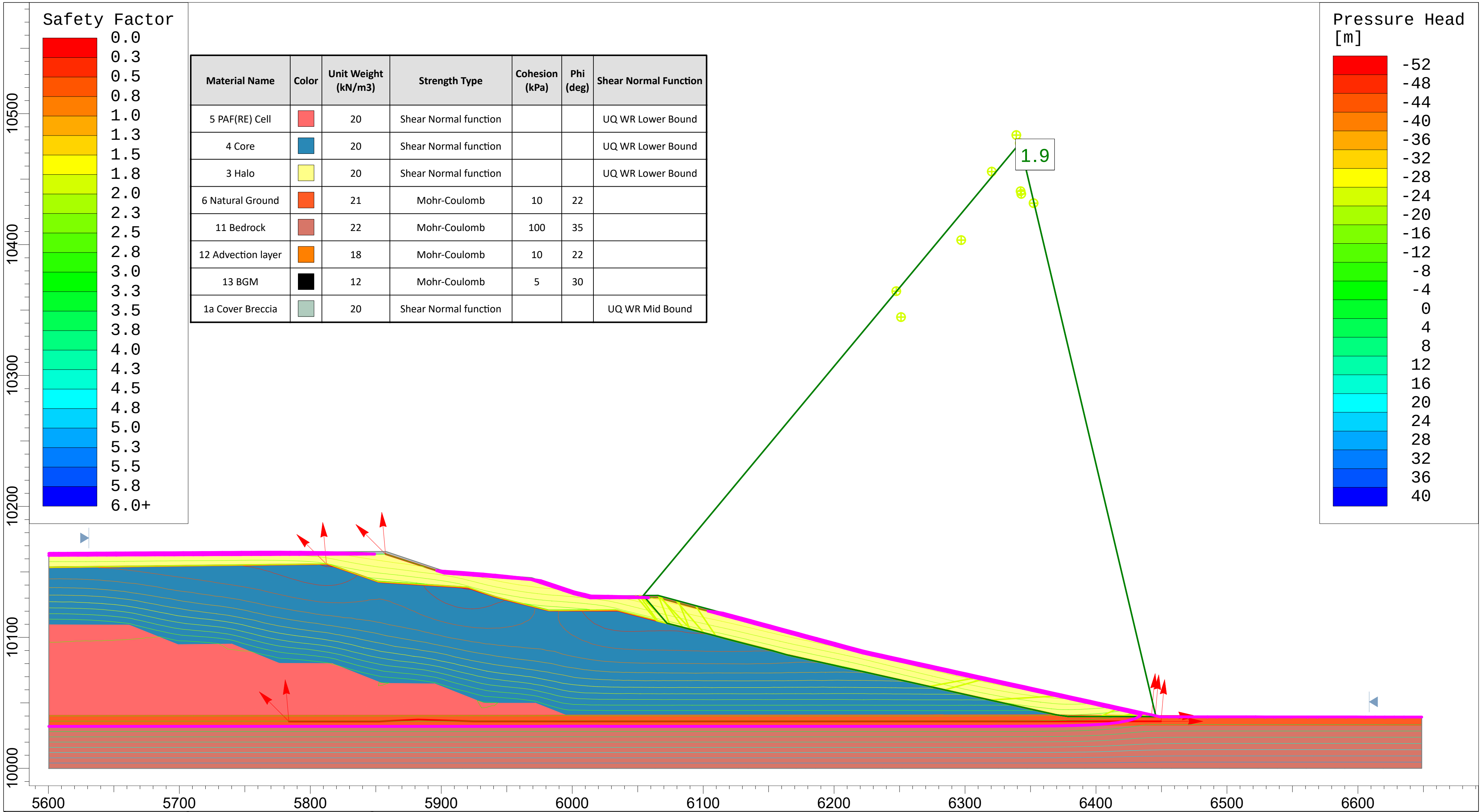
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

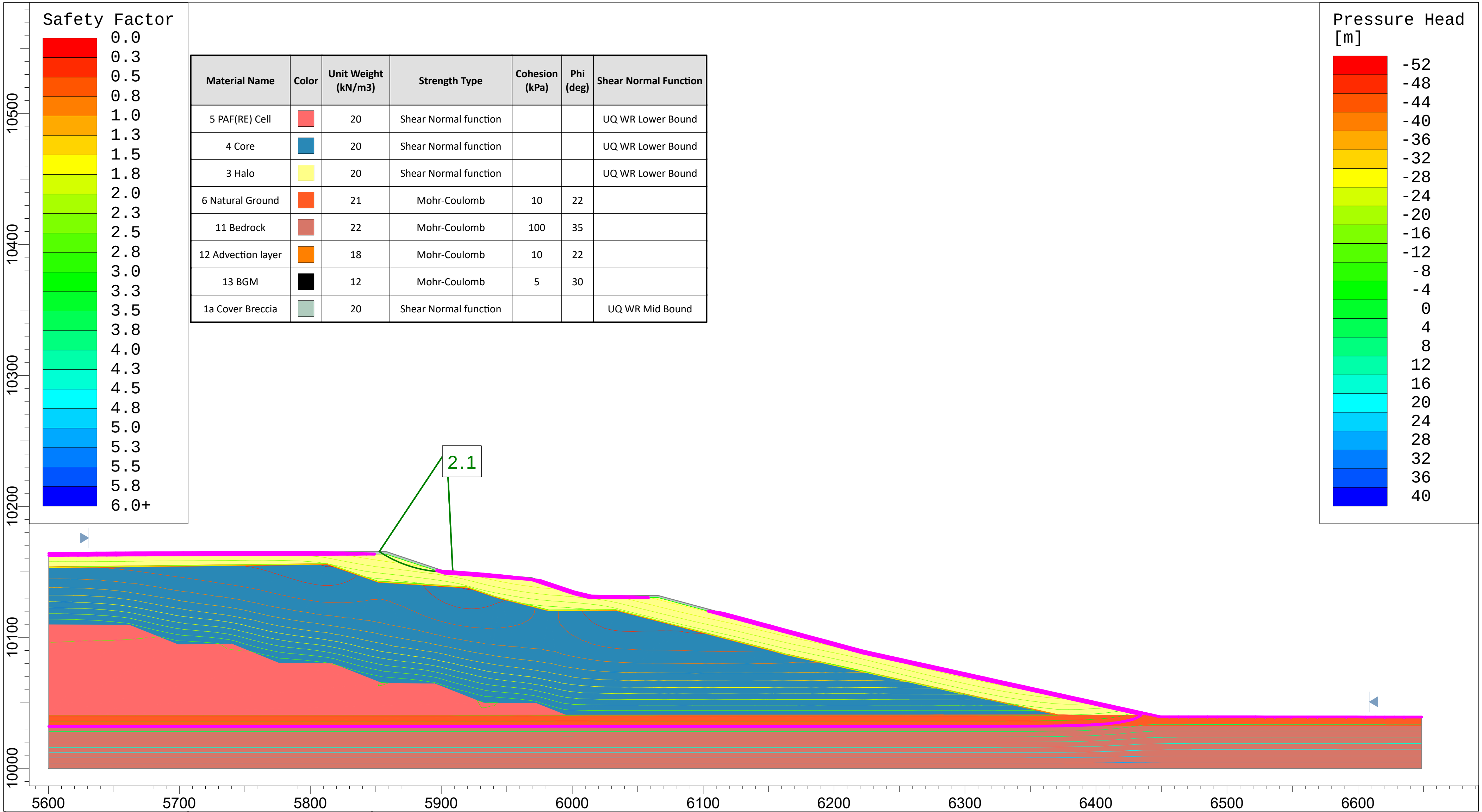
Project	MRM NOEF EIS		
Description: Section B-B beccia sensitivity 1:1000		Analysis Method: Cuckoo	
Drawn By SB	*Scale* 1:3068	*Company*	MRM/Pando
Date	November 2017	*File Name:*	section b-b update_bgm_breccia_1-1000.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

SLIDEINTERPRET 7.030

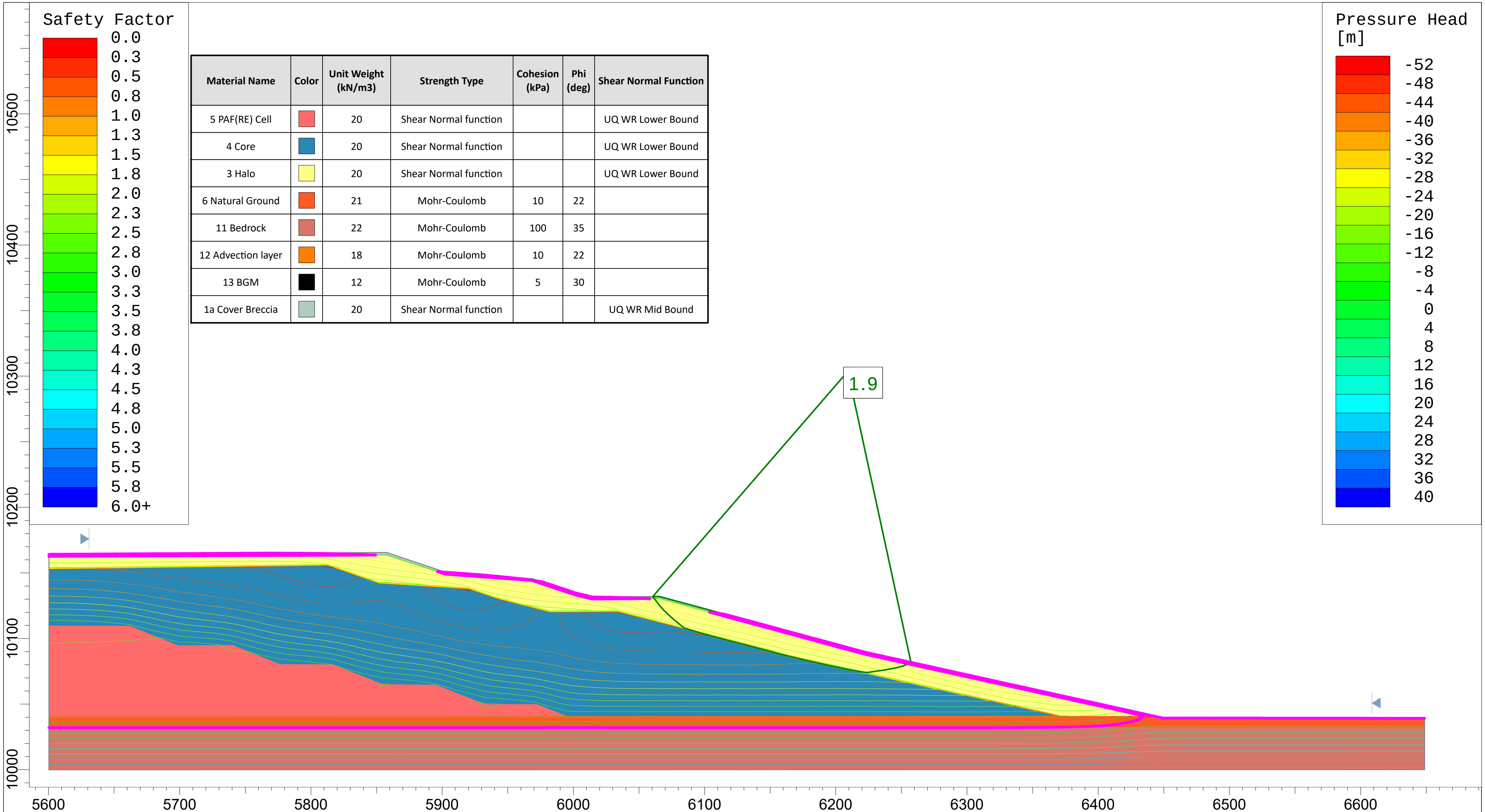
Project	MRM NOEF EIS		
Description: Section C-C North_1-1000		Analysis Method: Block	
Drawn By SB	*Scale* 1:4066	*Company*	MRM/Pando
Date	June 2017	*File Name:*	section c-c north update_bgm_1-1000.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Project	MRM NOEF EIS	
Description: Section C-C North_1-1000	Analysis Method: Circular	
Drawn By: SB	Scale: 1:4066	Company: MRM/Pando
Date: June 2017	File Name: section c-c north update_bgm_1-1000.slmd	

SLIDEINTERPRET 7.030



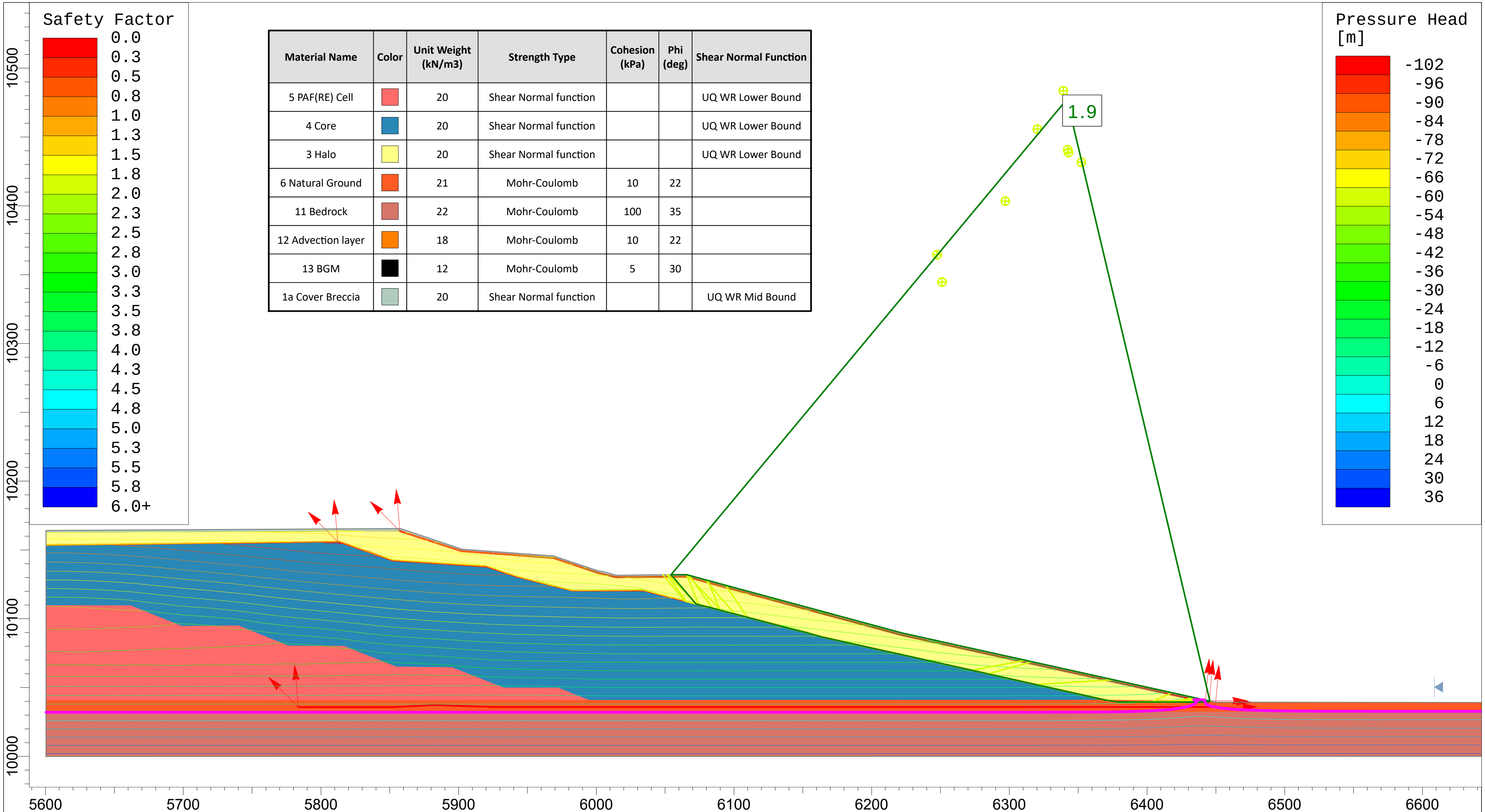
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section C-C North_1-1000		Analysis Method: Cuckoo	
Drawn By SB	*Scale* 1:4066	*Company*	MRM/Pando
Date	June 2017	*File Name:*	section c-c north update_bgm_1-1000.slmd



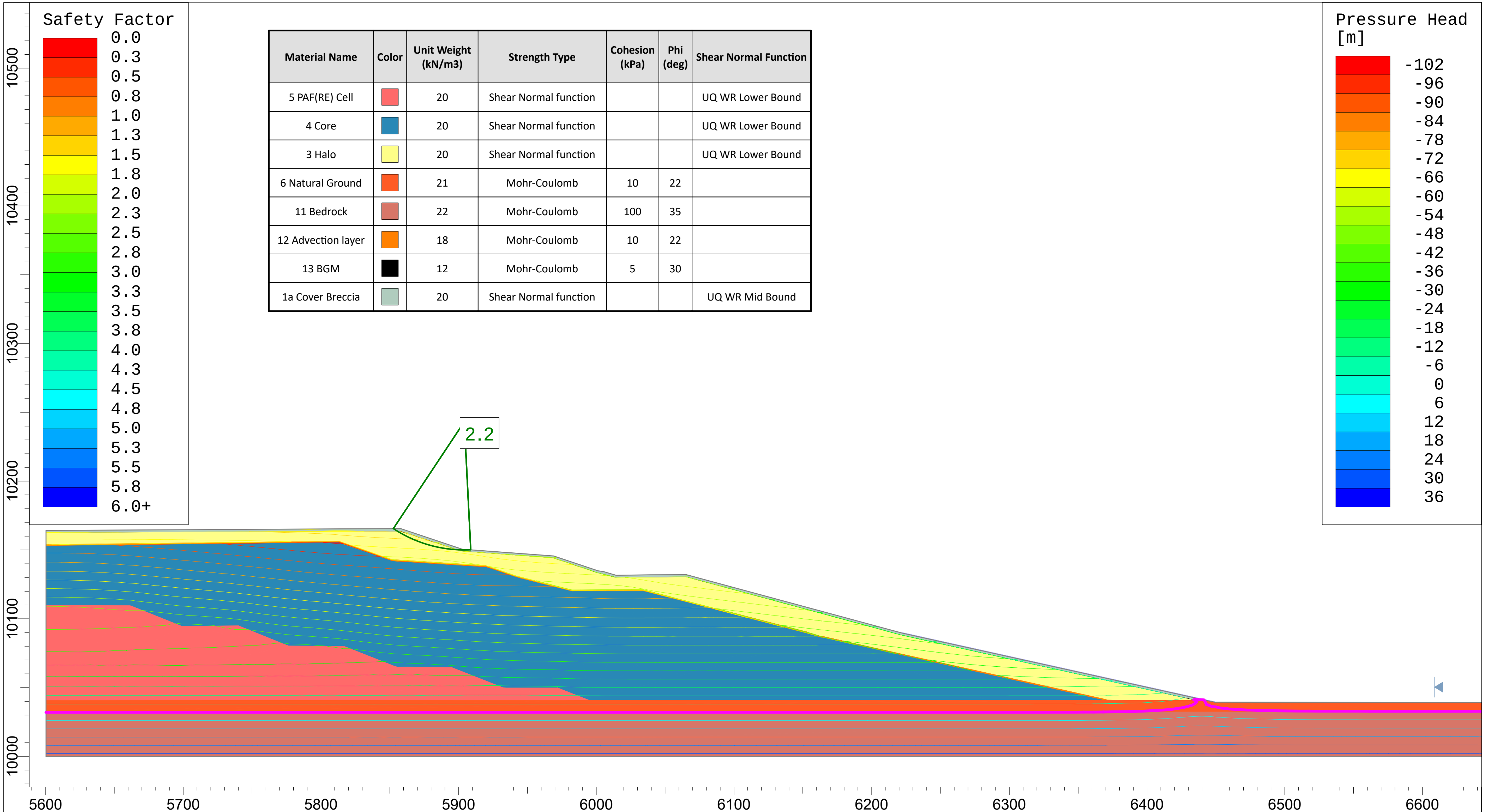
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS
Description: Section C-C North-Dry Season	Analysis Method: Block
Drawn By: SB	Scale: 1:3875
Company	MRM/Pando
Date	June 2017
File Name:	section c-c north update_bgm_dry.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

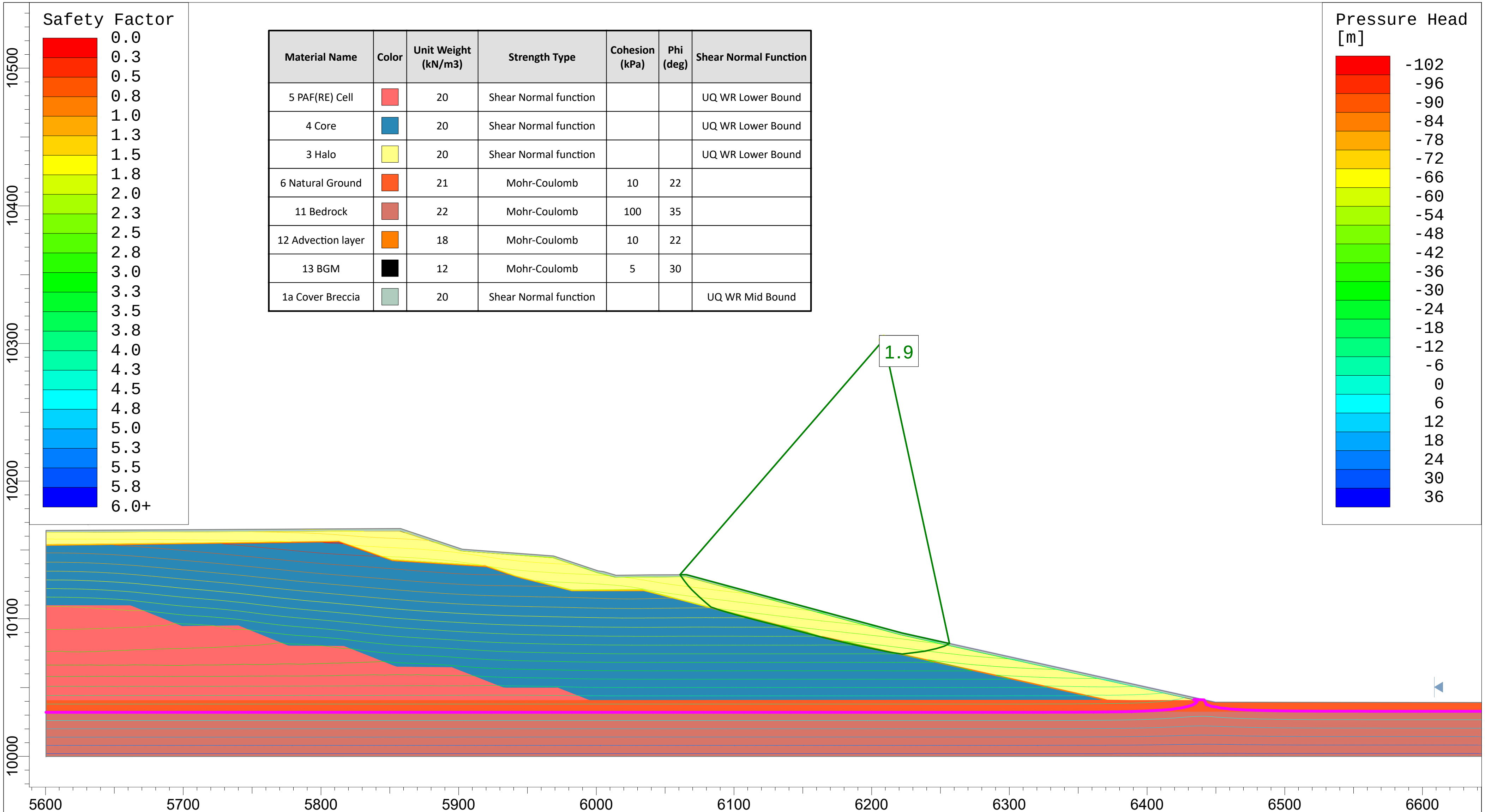
Pando

McArthur River Mining

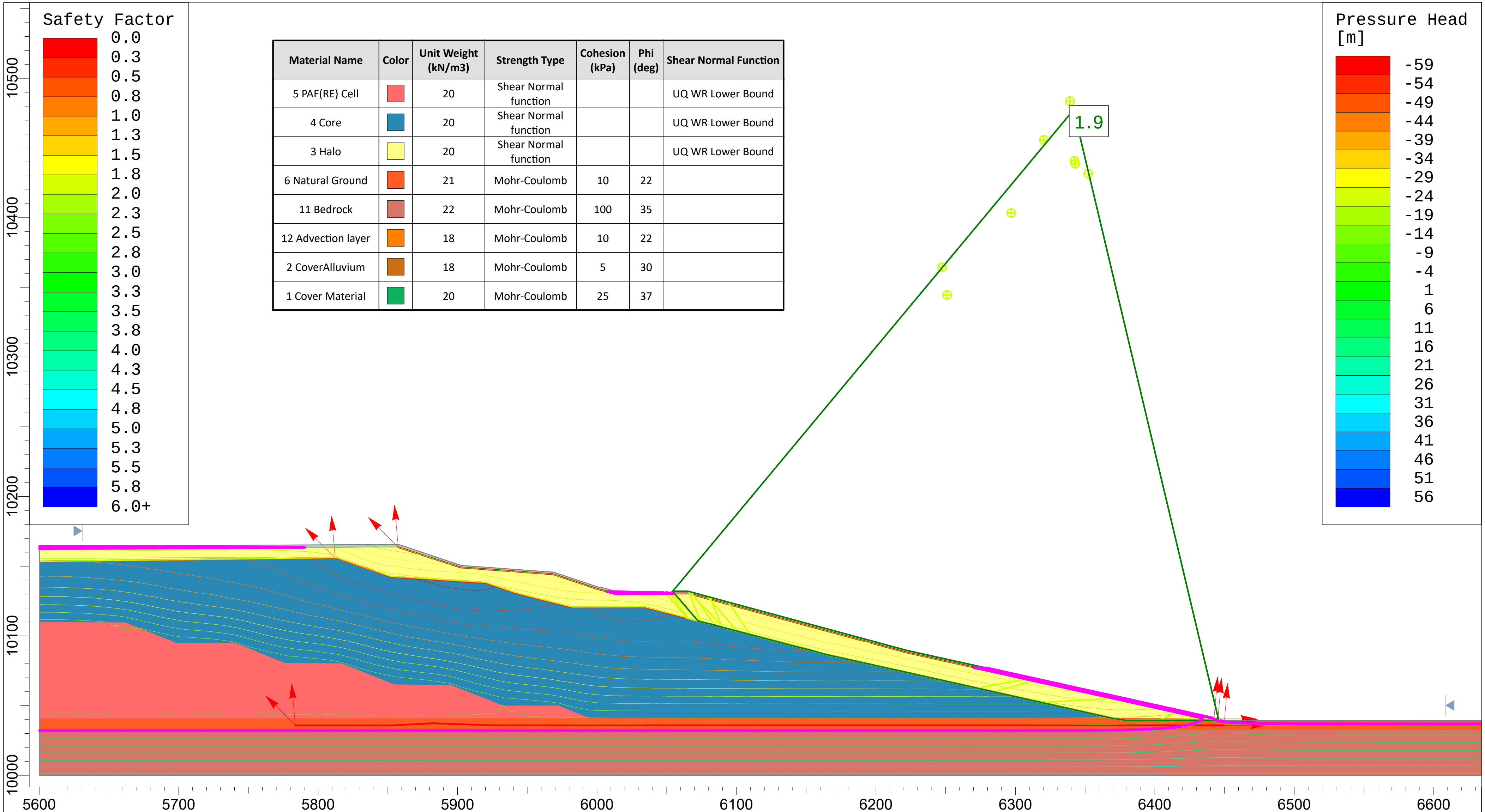
SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section C-C North-Dry Season		Analysis Method: Circular	
Drawn By SB	*Scale* 1:3875	*Company* MRM/Pando	
Date June 2017		*File Name:* section c-c north update_bgm_dry.slmd	

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound



Pando	McArthur River Mining	Project: MRM NOEF EIS
		Description: Section C-C North-Dry Season
		Analysis Method: Cuckoo
		Drawn By: SB
		Scale: 1:3875
		Company: MRM/Pando
		Date: June 2017
		File Name: section c-c north update_bgm_dry.slmd
SLIDEINTERPRET 7.030		



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
2 CoverAlluvium		18	Mohr-Coulomb	5	30	
1 Cover Material		20	Mohr-Coulomb	25	37	

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project MRM NOEF EIS

Description:
Section C-C North-Wet Season

Analysis Method: Block

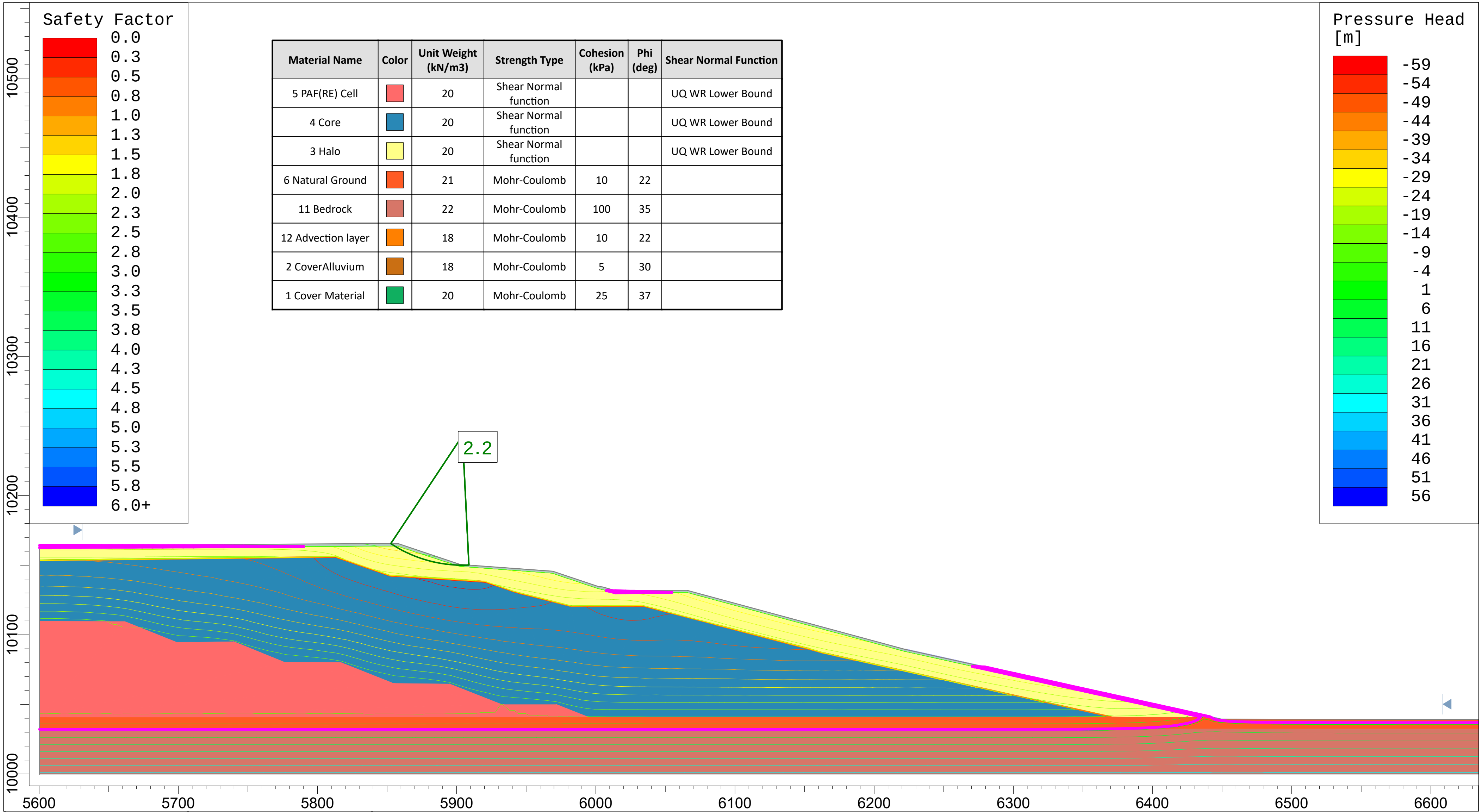
Drawn By SB

Scale 1:3826

Company MRM/Pando

Date June 2017

File Name: section c-c north update_bgm_wet.slmd



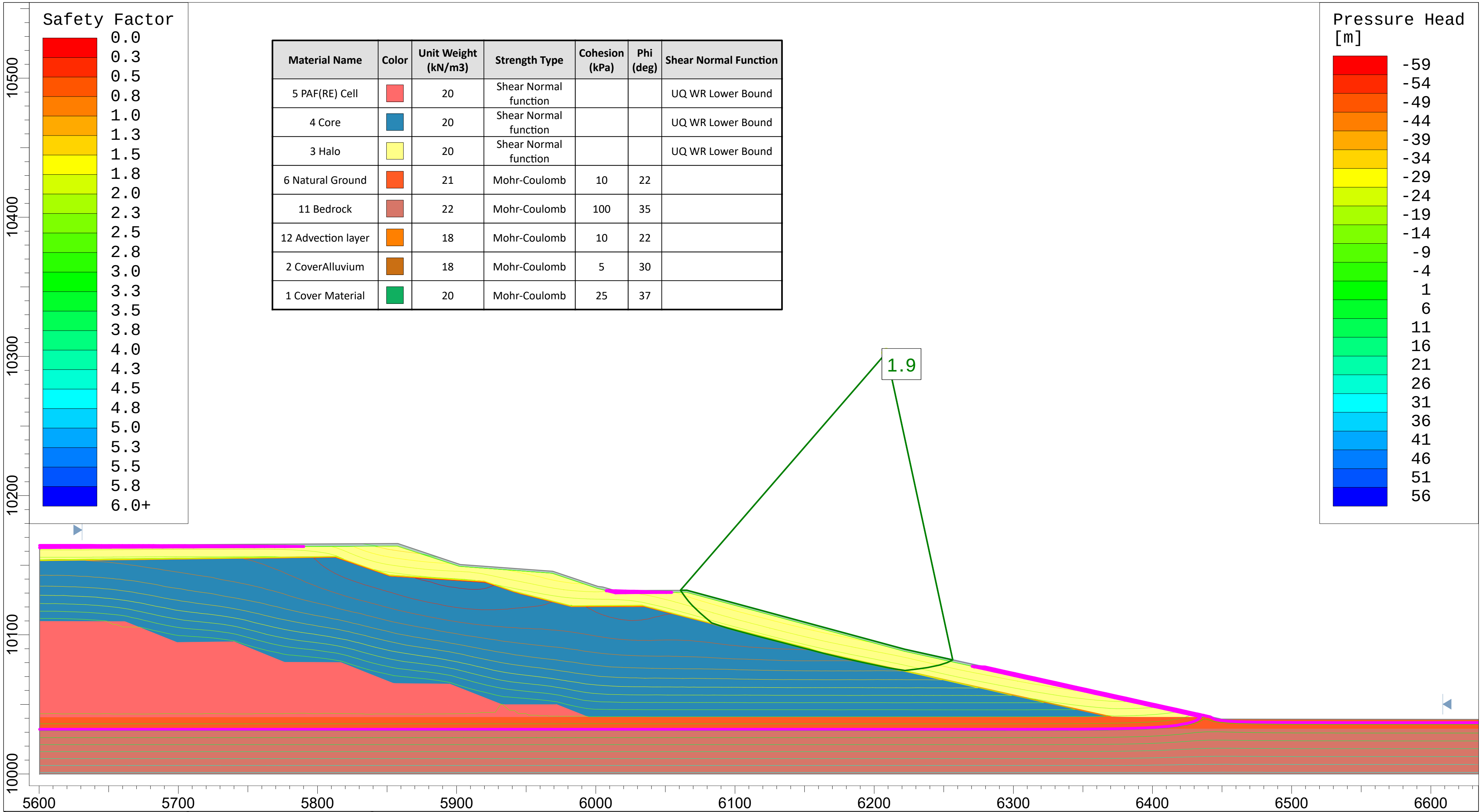
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
2 CoverAlluvium		18	Mohr-Coulomb	5	30	
1 Cover Material		20	Mohr-Coulomb	25	37	

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section C-C North-Wet Season		Analysis Method: Circular	
Drawn By SB	*Scale* 1:3826	*Company*	MRM/Pando
Date	June 2017	*File Name:*	section c-c north update_bgm_wet.slmd



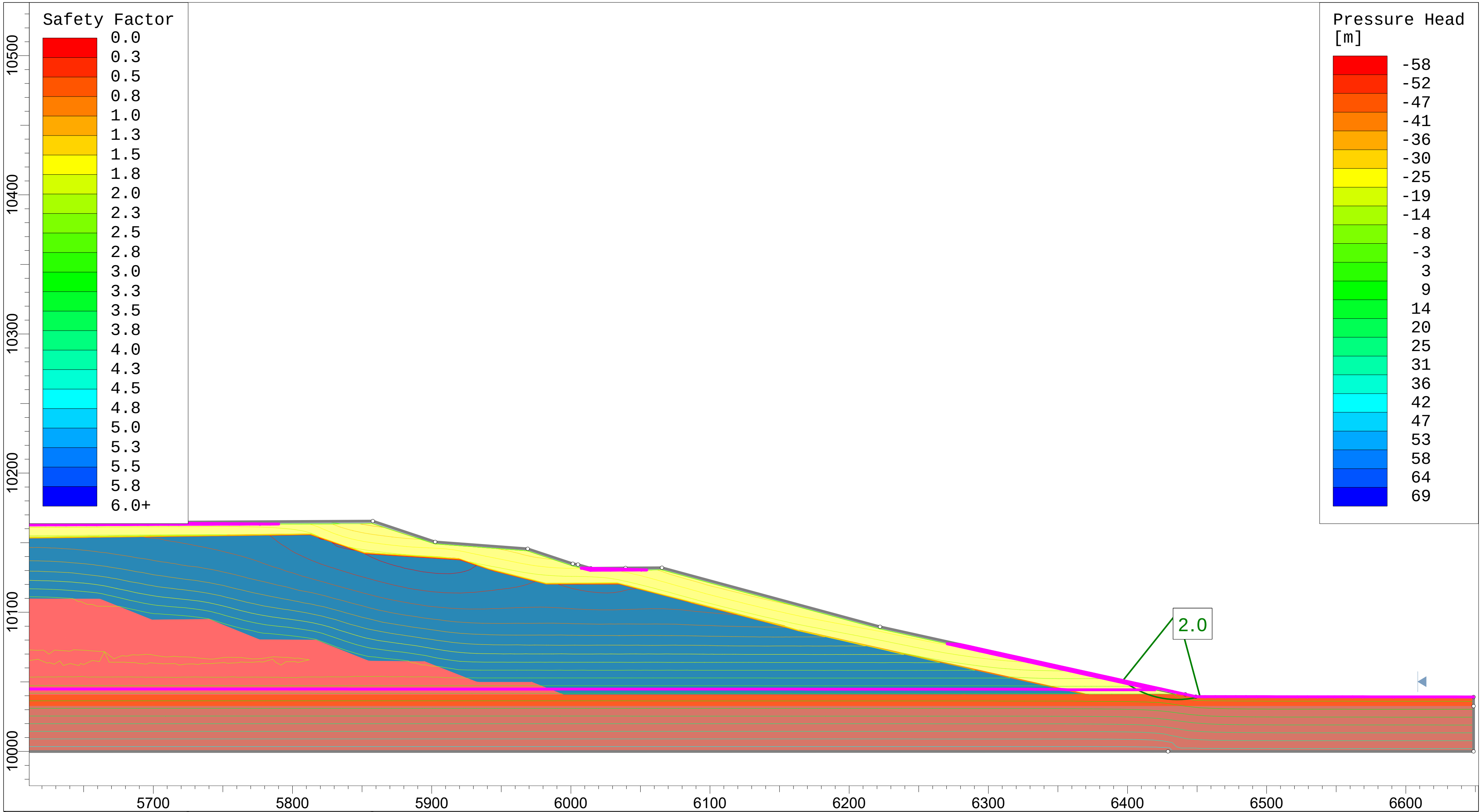
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
2 CoverAlluvium		18	Mohr-Coulomb	5	30	
1 Cover Material		20	Mohr-Coulomb	25	37	

Pando

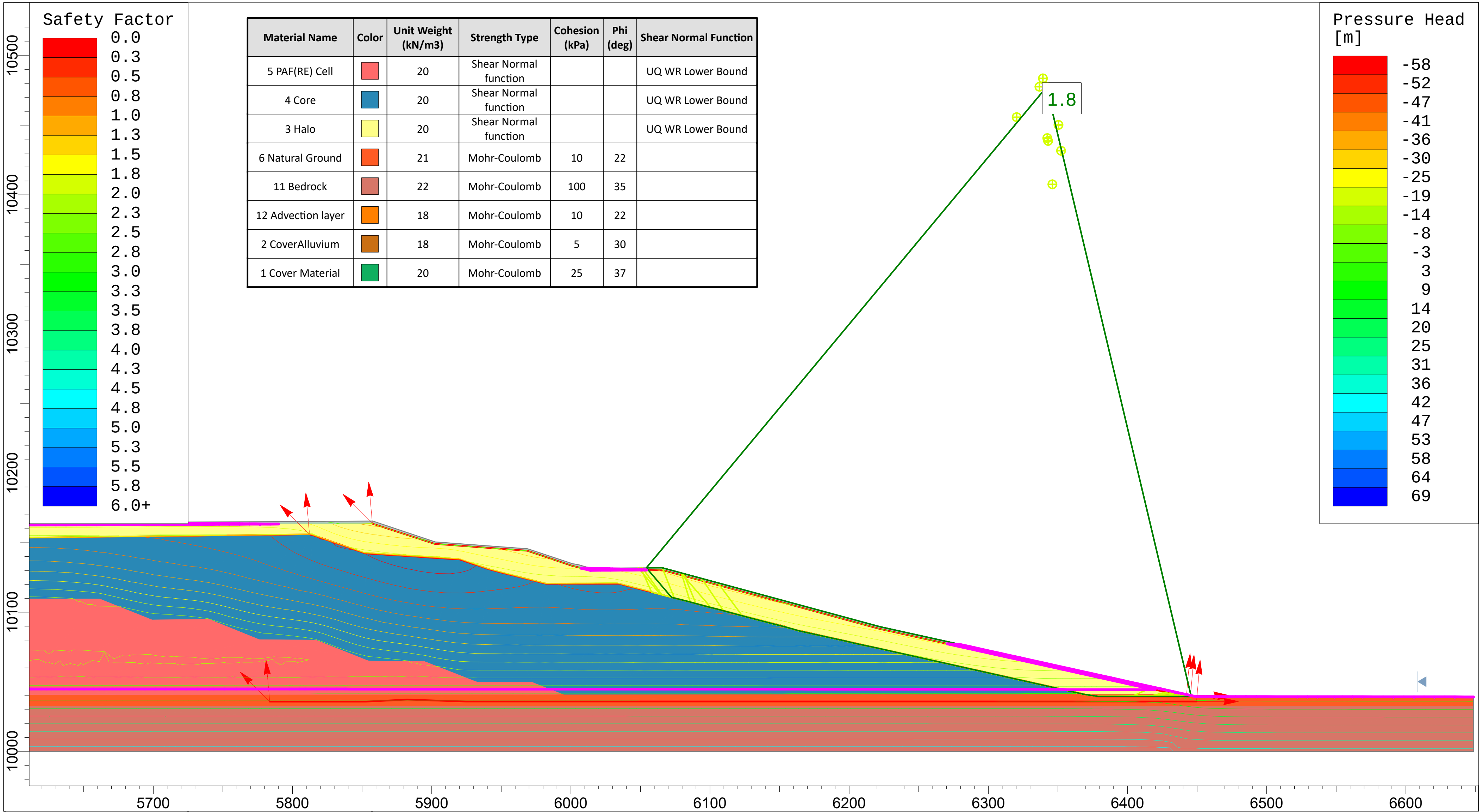
McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section C-C North-Wet Season		Analysis Method: Cuckoo	
Drawn By	SB	*Scale*	1:3826
Company	MRM/Pando		
Date	June 2017	*File Name:*	section c-c north update_bgm_wet.slmd



Pando	McArthur River Mining	Project	MRM NOEF EIS	
		Description: Section C-C North-Blocked Drains		Analysis Method: Circular
		Drawn By: SB	Scale: 1:3826	Company: MRM/Pando
SLIDEINTERPRET 7.030		Date: June 2017		File Name: section c-c north update_bgm_drain.slmd



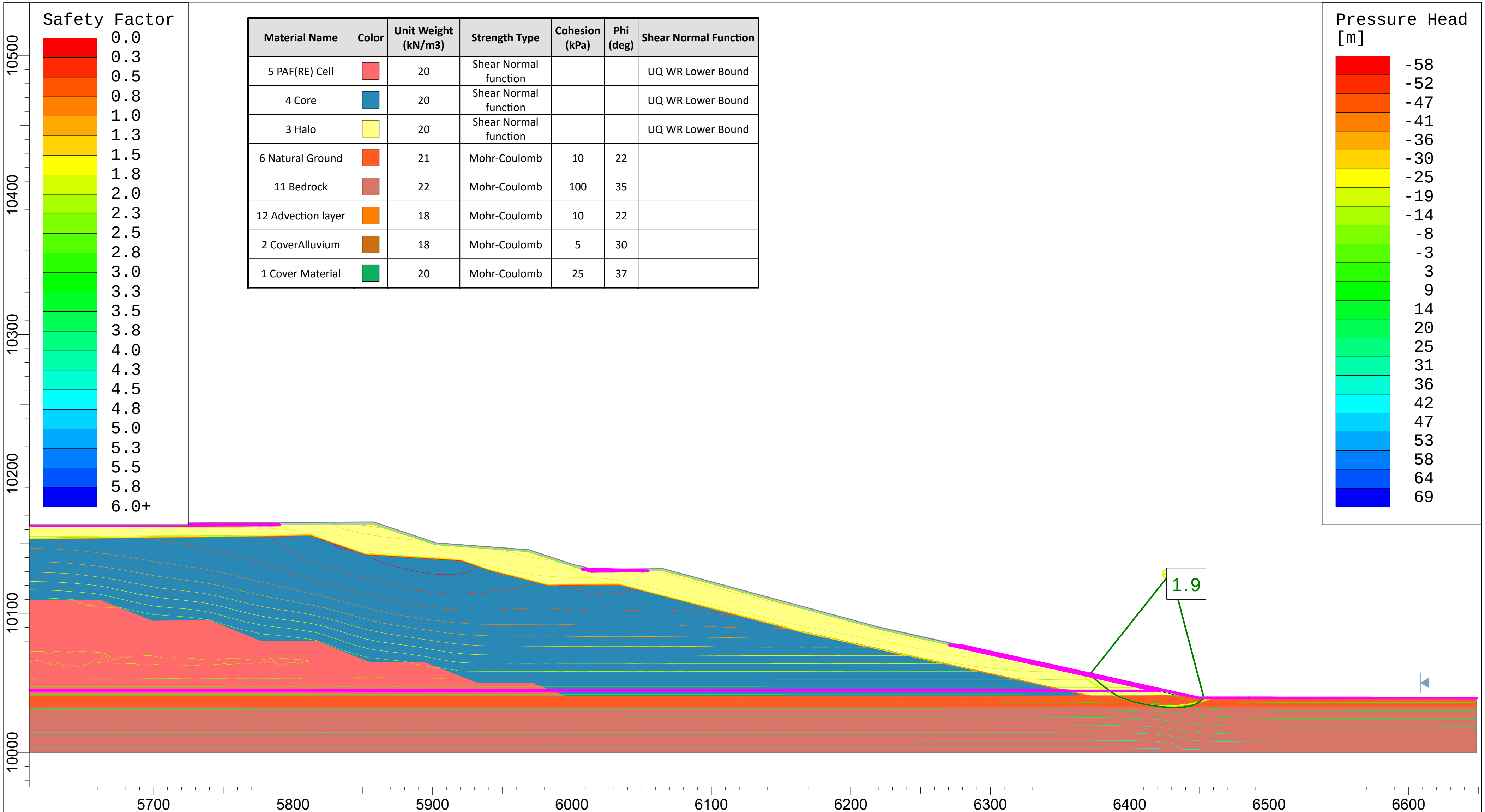
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
2 CoverAlluvium		18	Mohr-Coulomb	5	30	
1 Cover Material		20	Mohr-Coulomb	25	37	

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section C-C North-Blocked Drains		Analysis Method: Block	
Drawn By SB	*Scale* 1:3826	*Company*	MRM/Pando
Date June 2017		*File Name:*	section c-c north update_bgm_drain.slmd



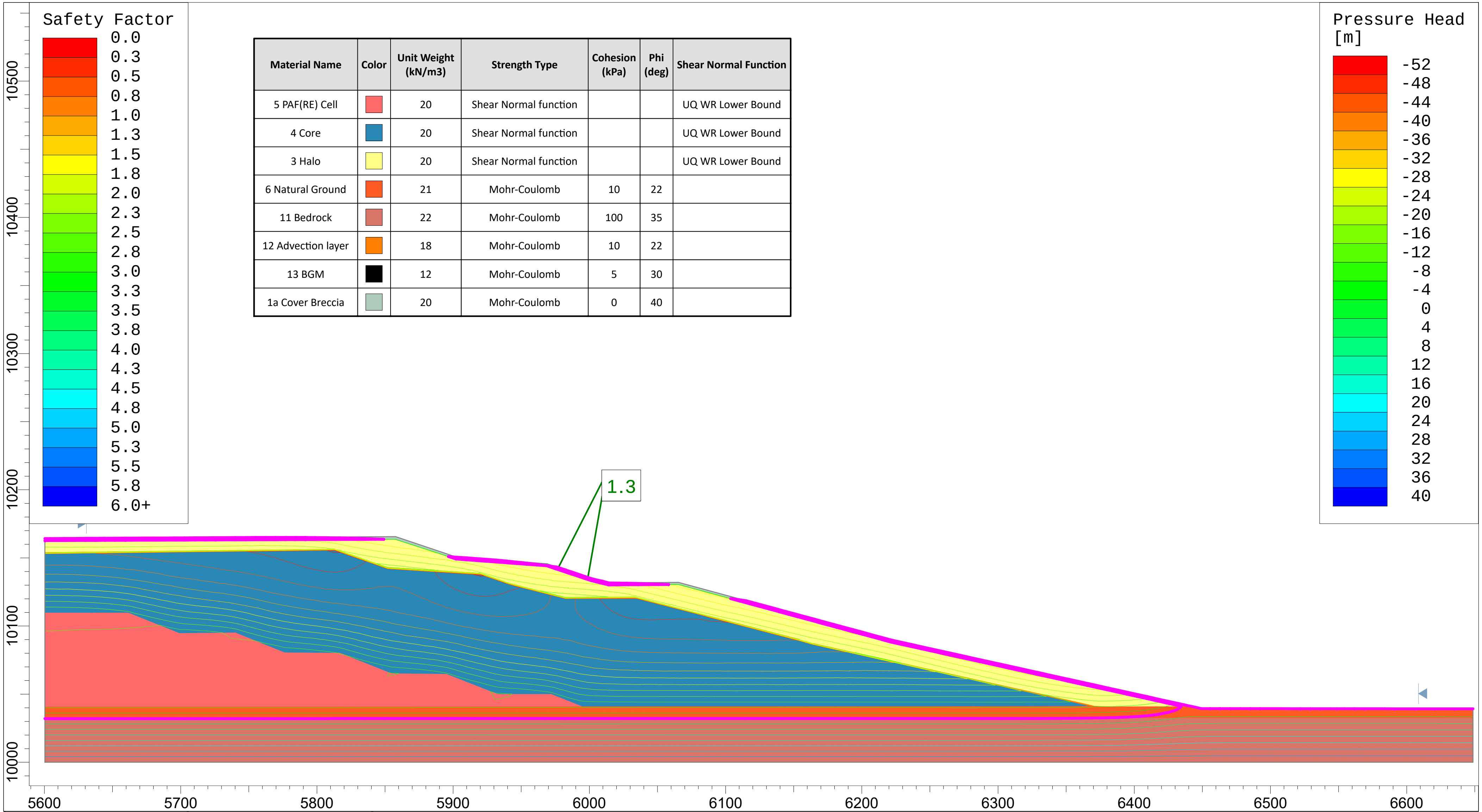
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
2 CoverAlluvium		18	Mohr-Coulomb	5	30	
1 Cover Material		20	Mohr-Coulomb	25	37	

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS
Description: Section C-C North-Blocked Drains	Analysis Method: Cuckoo
Drawn By: SB	Scale: 1:3826
Company	MRM/Pando
Date	June 2017
File Name:	section c-c north update_bgm_drain.slmd



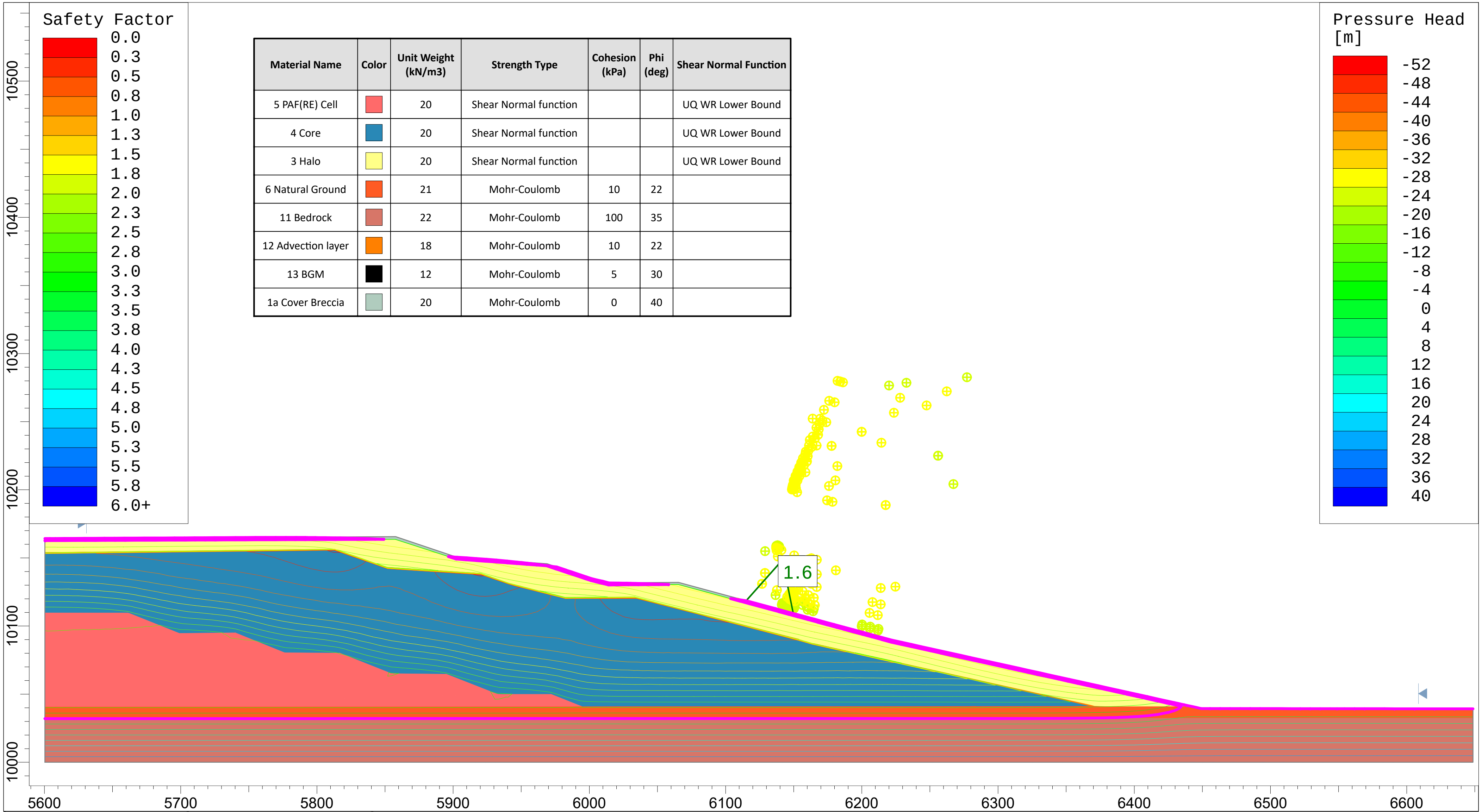
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section C-C North_breccia_sensitivity_1-1000		Analysis Method: Circular	
Drawn By	SB	Scale	1:3909
Company	MRM/Pando		
Date	June 2017	File Name:	section c-c north update_bgm_breccia_1-1000.slmd



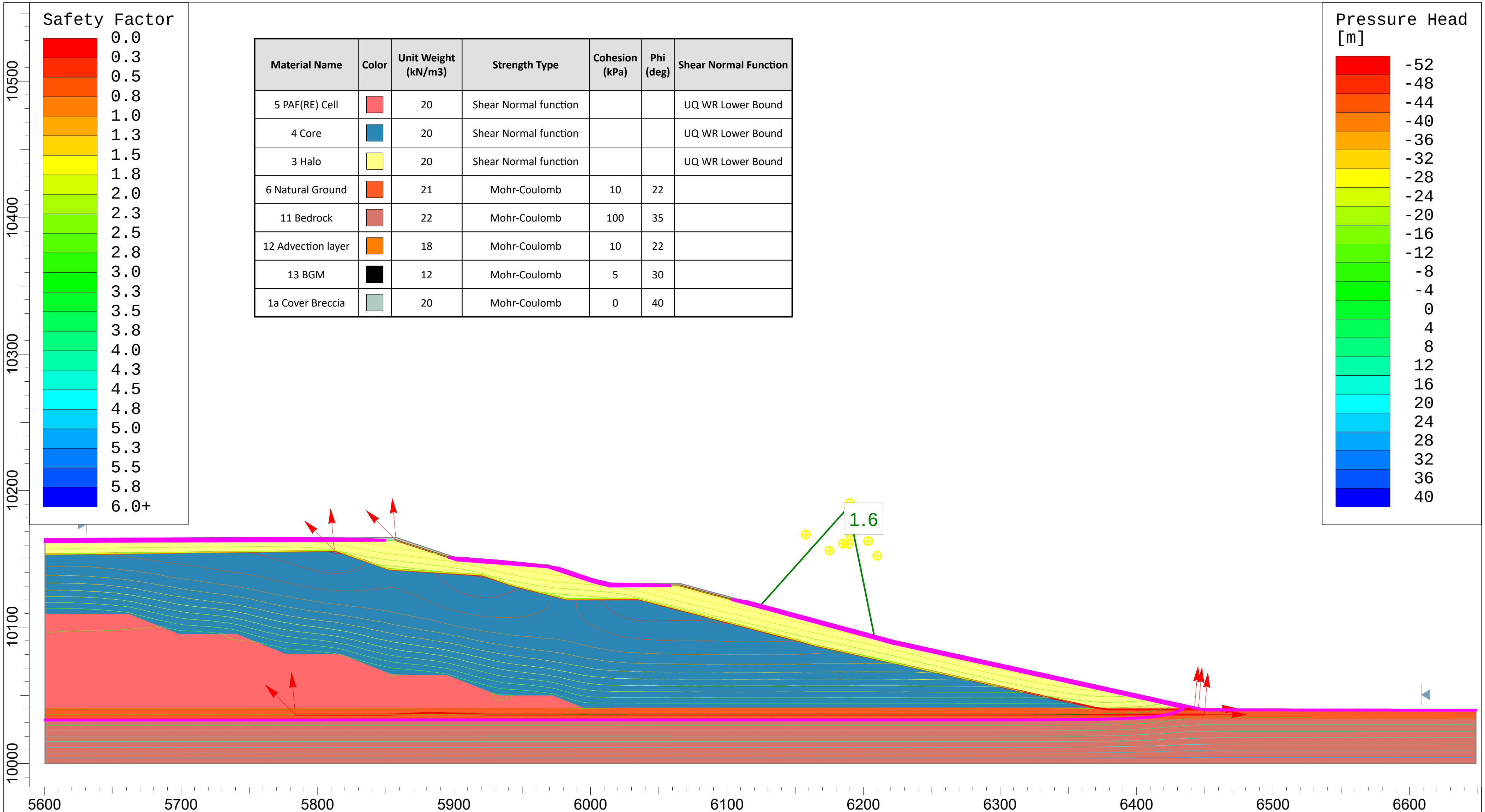
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section C-C North_breccia_sensitivity_1-1000		Analysis Method: Cuckoo	
Drawn By SB	*Scale* 1:3909	*Company*	MRM/Pando
Date	June 2017	*File Name:*	section c-c north update_bgm_breccia_1-1000.slmd



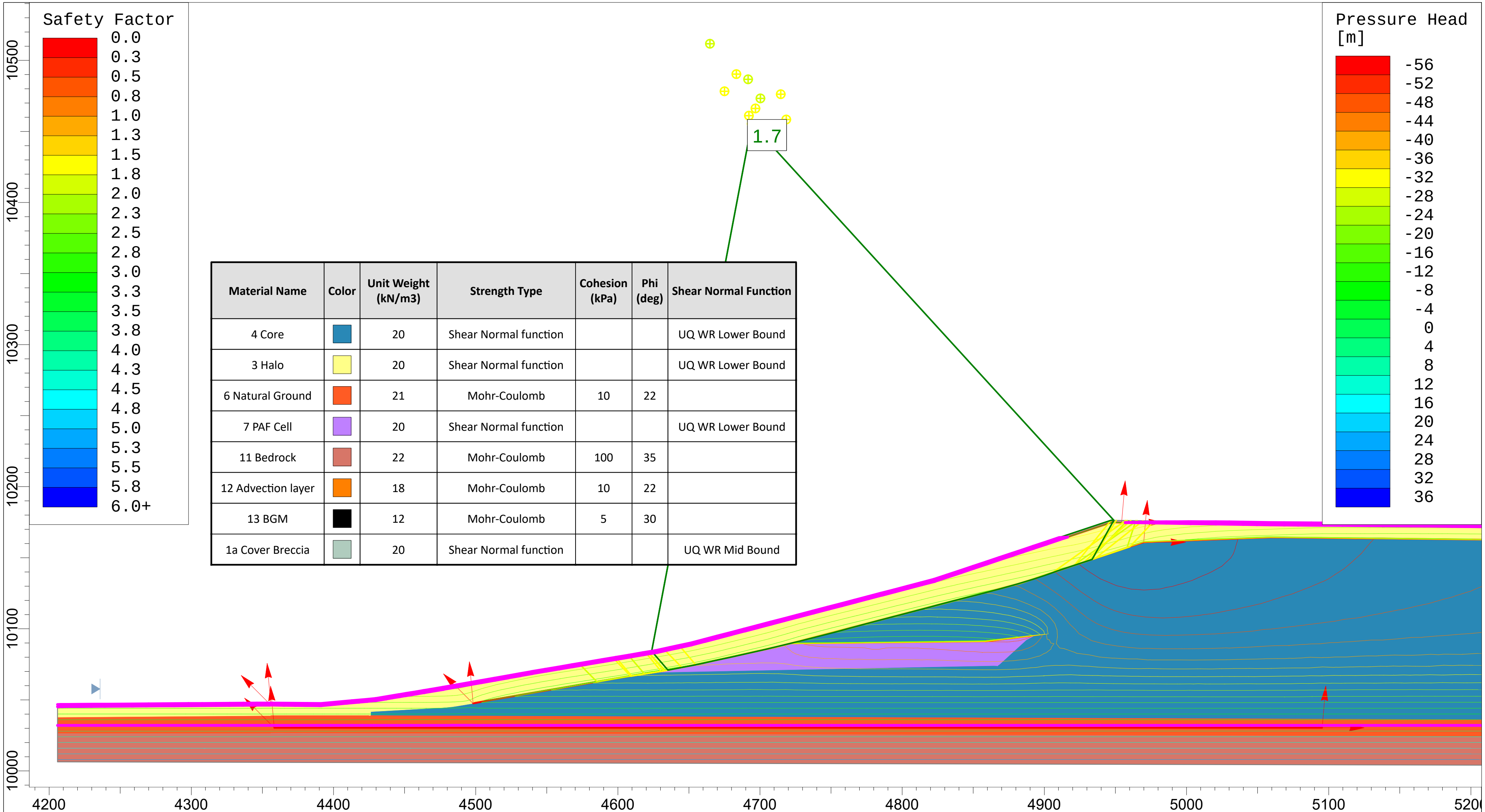
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
5 PAF(RE) Cell		20	Shear Normal function			UQ WR Lower Bound
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

Pando

McArthur River Mining

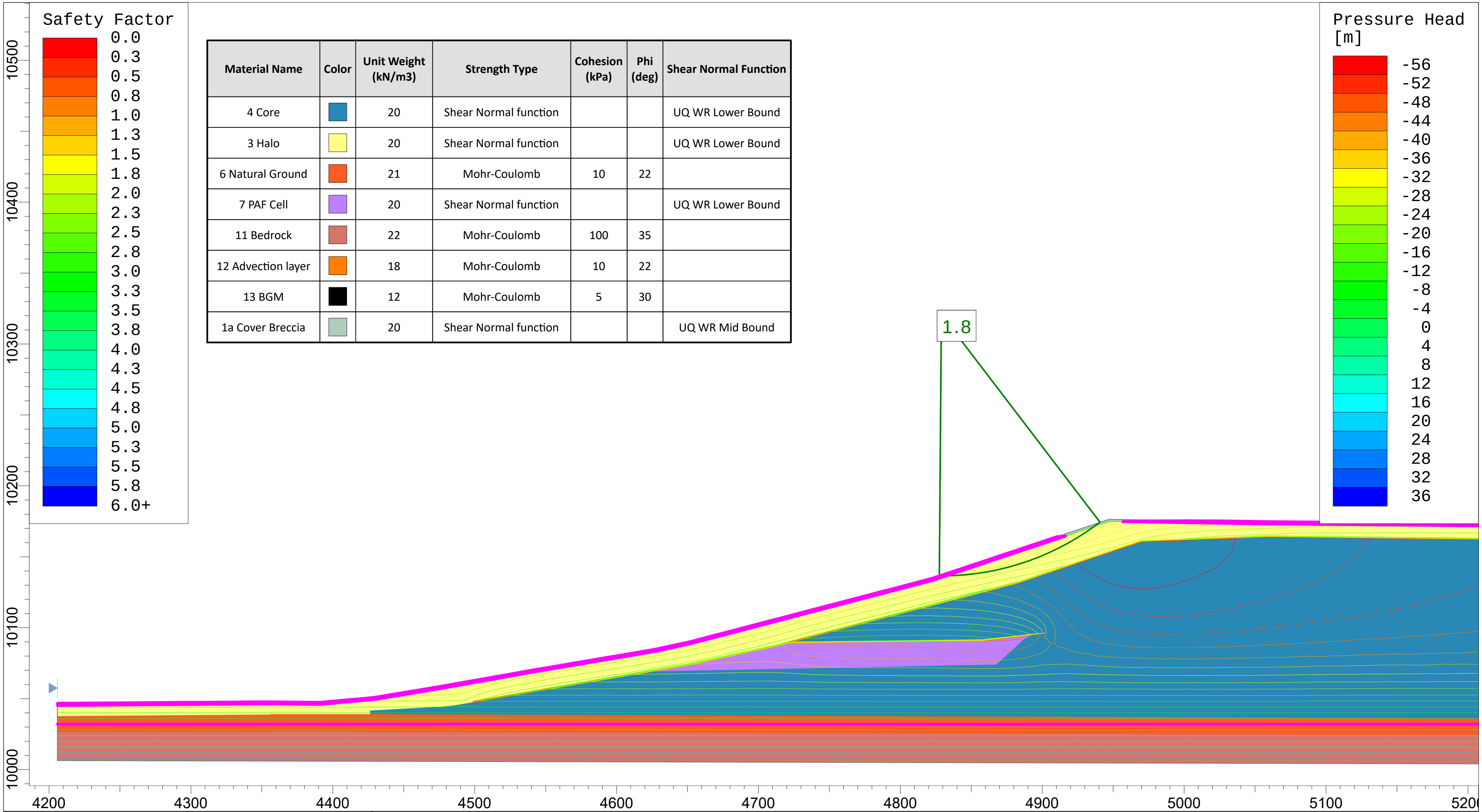
SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section C-C North_breccia_sensitivity_1-1000		Analysis Method: Block	
Drawn By SB	*Scale* 1:3909	*Company*	MRM/Pando
Date June 2017		*File Name:*	section c-c north update_bgm_breccia_1-1000.slmd



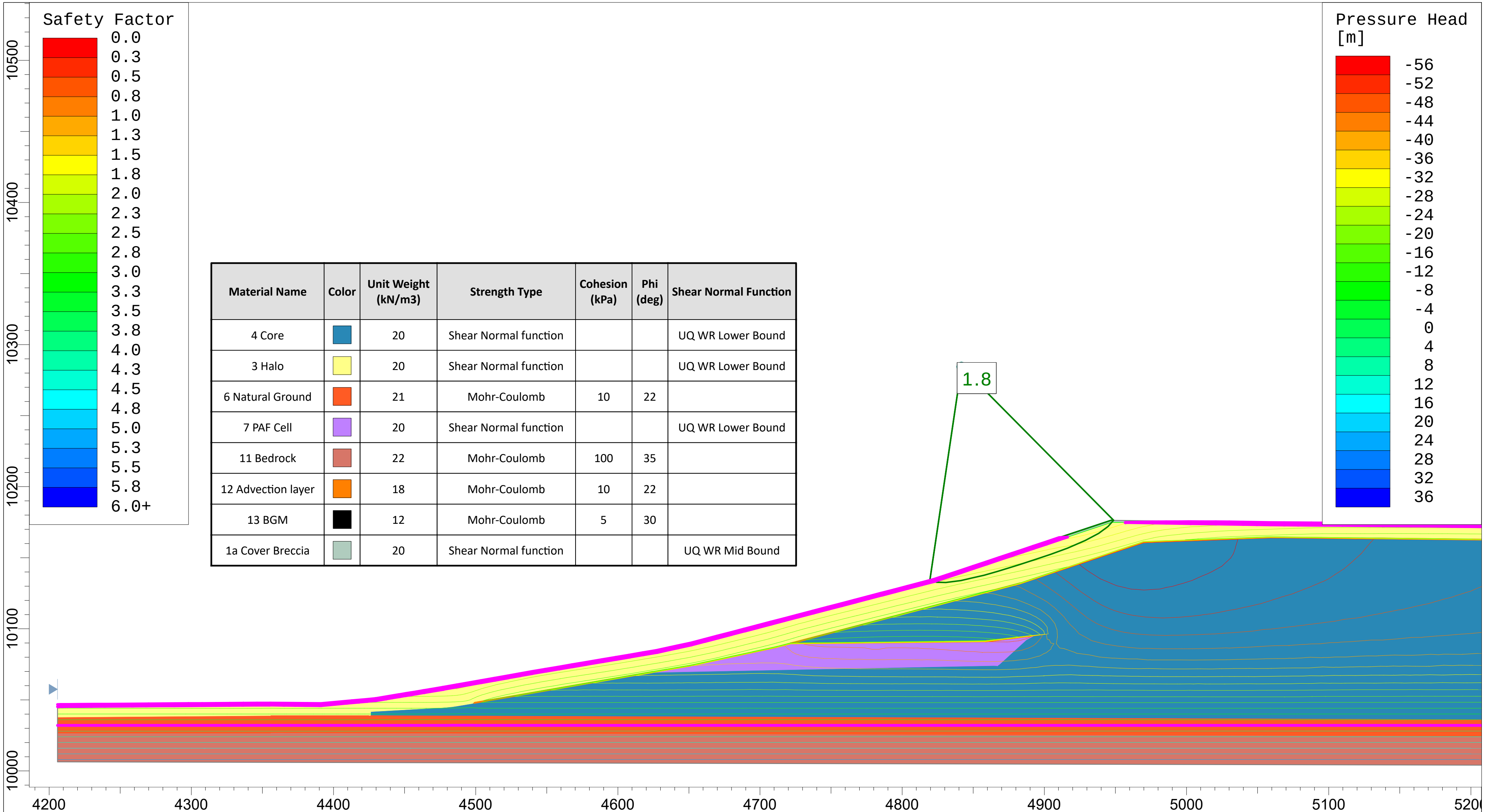
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando	McArthur River Mining	*Project*	MRM NOEF EIS
		Description: Section C-C South_1-1000	Analysis Method: Block
		Drawn By SB	*Scale* 1:3753
		Company	MRM/Pando
SLIDEINTERPRET 7.030		*Date* June 2017	*File Name:* section c-c south update_bgm_1-1000.slmd



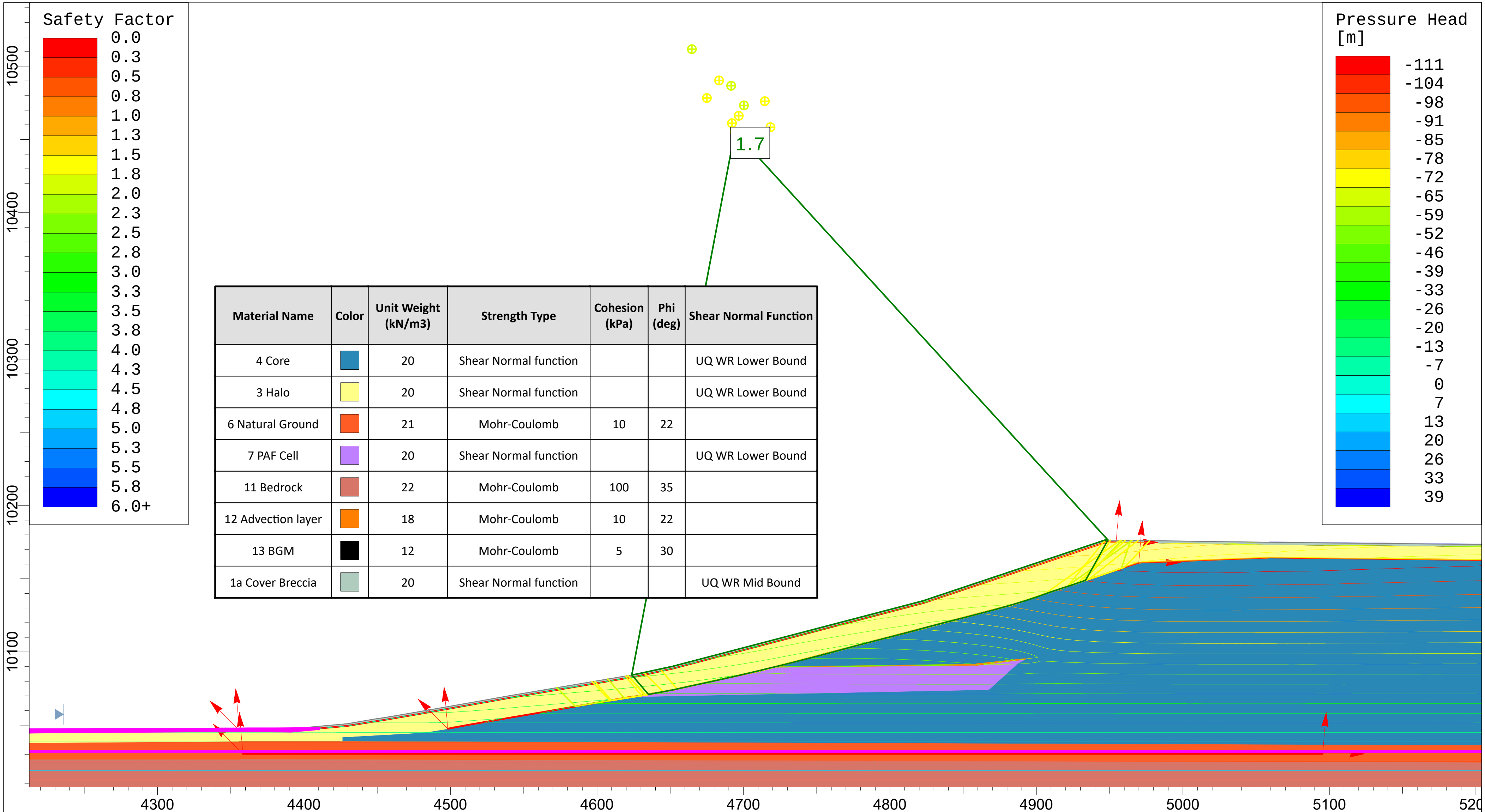
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando	McArthur River Mining	Project: MRM NOEF EIS
		Description: Section C-C South_1-1000
		Analysis Method: Circular
		Drawn By: SB
		Scale: 1:3753
		Company: MRM/Pando
		Date: June 2017
		File Name: section c-c south update_bgm_1-1000.slmd
SLIDEINTERPRET 7.030		



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando	McArthur River Mining	Project: MRM NOEF EIS
	Description: Section C-C South_1-1000	Analysis Method: Cuckoo
	Drawn By: SB	Scale: 1:3753
	Company	MRM/Pando
SLIDEINTERPRET 7.030	Date: June 2017	File Name: section c-c south update_bgm_1-1000.slmd



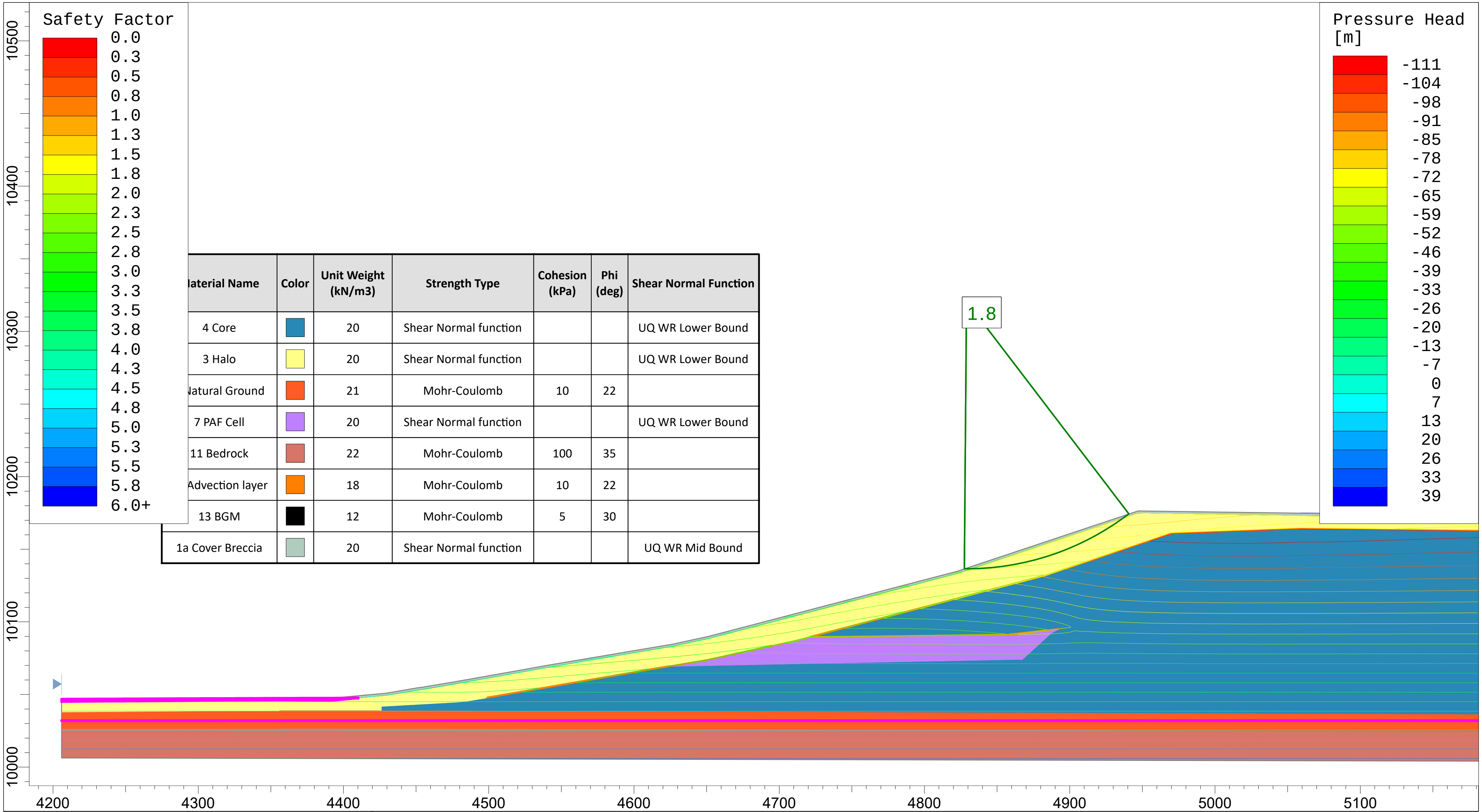
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

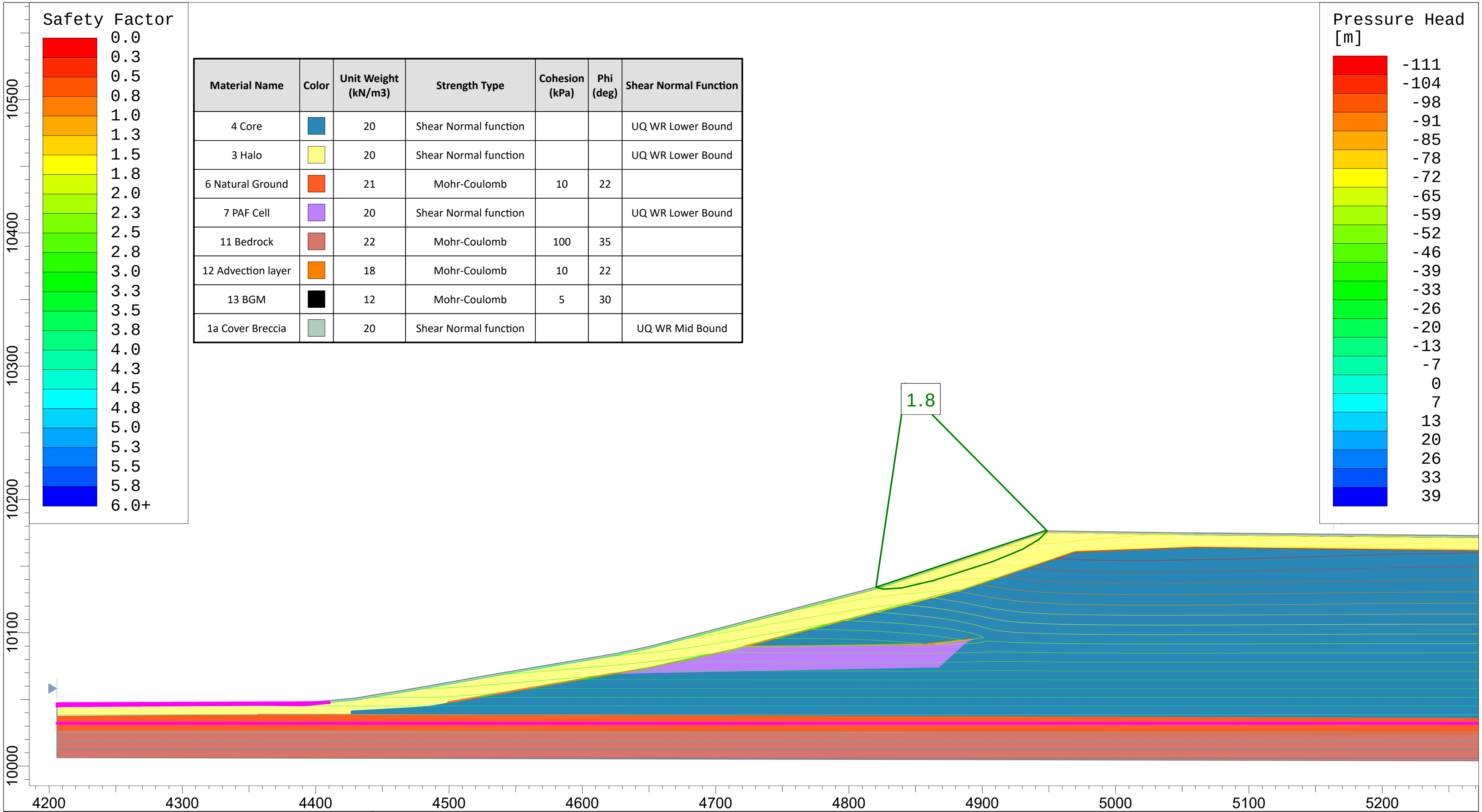
SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section C-C South-Dry Season		Analysis Method: Block	
Drawn By SB	*Scale* 1:3643	*Company*	MRM/Pando
Date June 2017		*File Name:*	section c-c south update_bgm_dry.slmd



laterial Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
latural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando	McArthur River Mining	Project: MRM NOEF EIS
		Description: Section C-C South-Dry Season / Analysis Method: Circular
		Drawn By: SB / Scale: 1:3666 / Company: MRM/Pando
SLIDEINTERPRET 7.030		Date: June 2017 / File Name: section c-c south update_bgm_dry.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project MRM NOEF EIS

Description:
Section C-C South-Dry Season

Analysis Method: Cuckoo

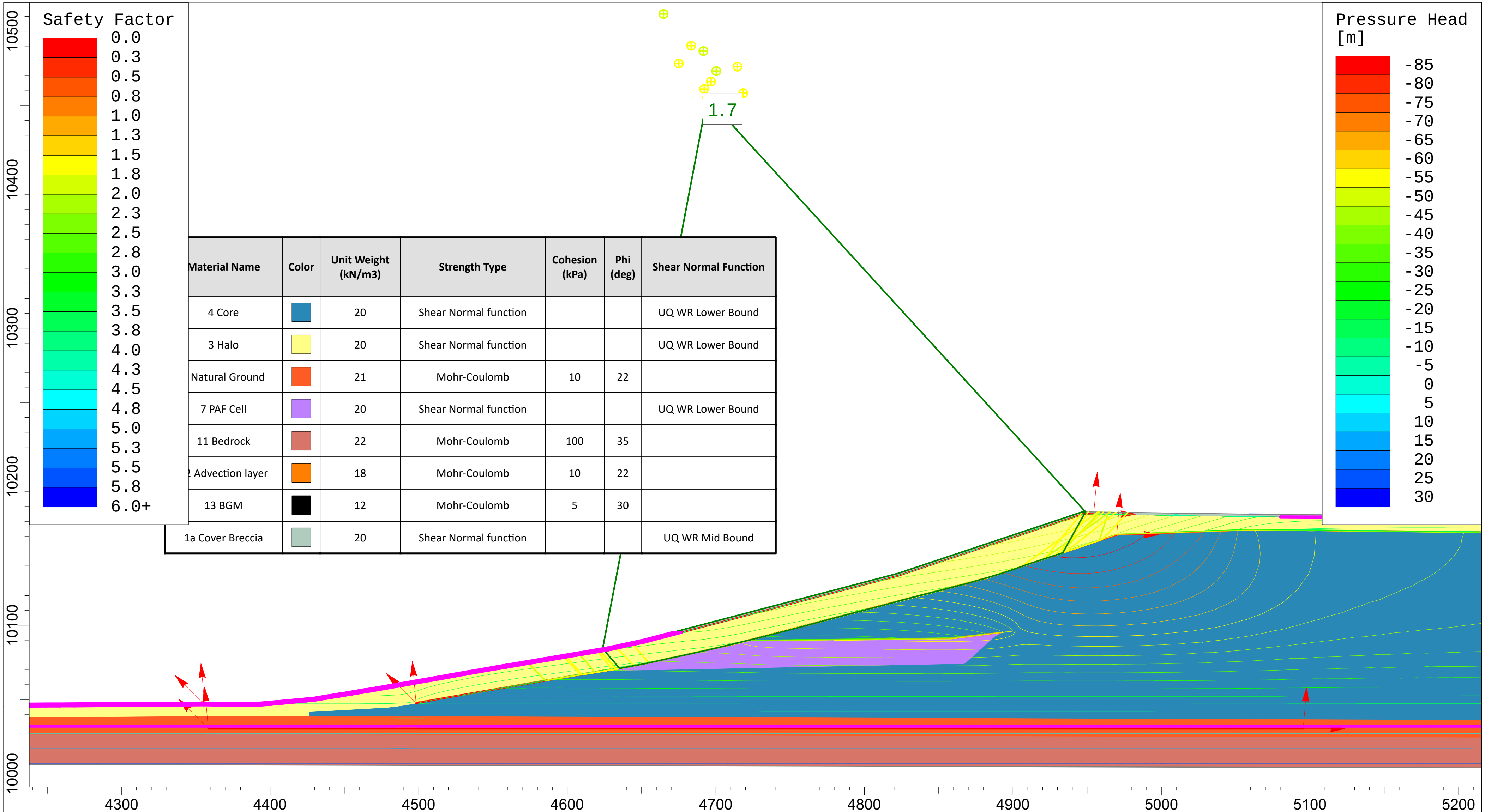
Drawn By SB

Scale 1:3994

Company MRM/Pando

Date June 2017

File Name: section c-c south update_bgm_dry.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
Natural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

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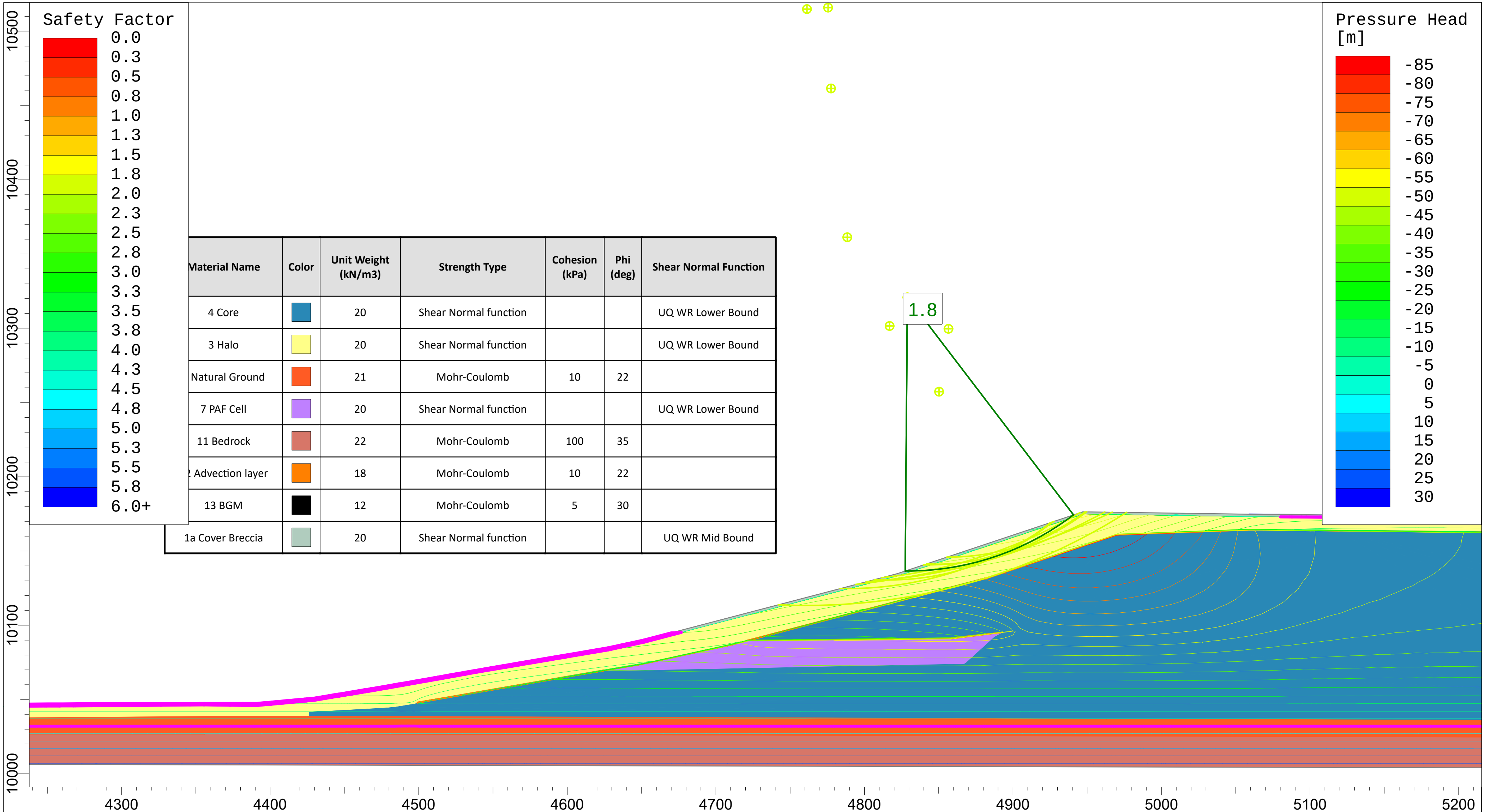
SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section C-C South-Wet Season		Analysis Method: Block	
Drawn By	SB	Scale	1:3592
Company	MRM/Pando		
Date	June 2017	File Name:	section c-c south update_bgm_wet.slmd



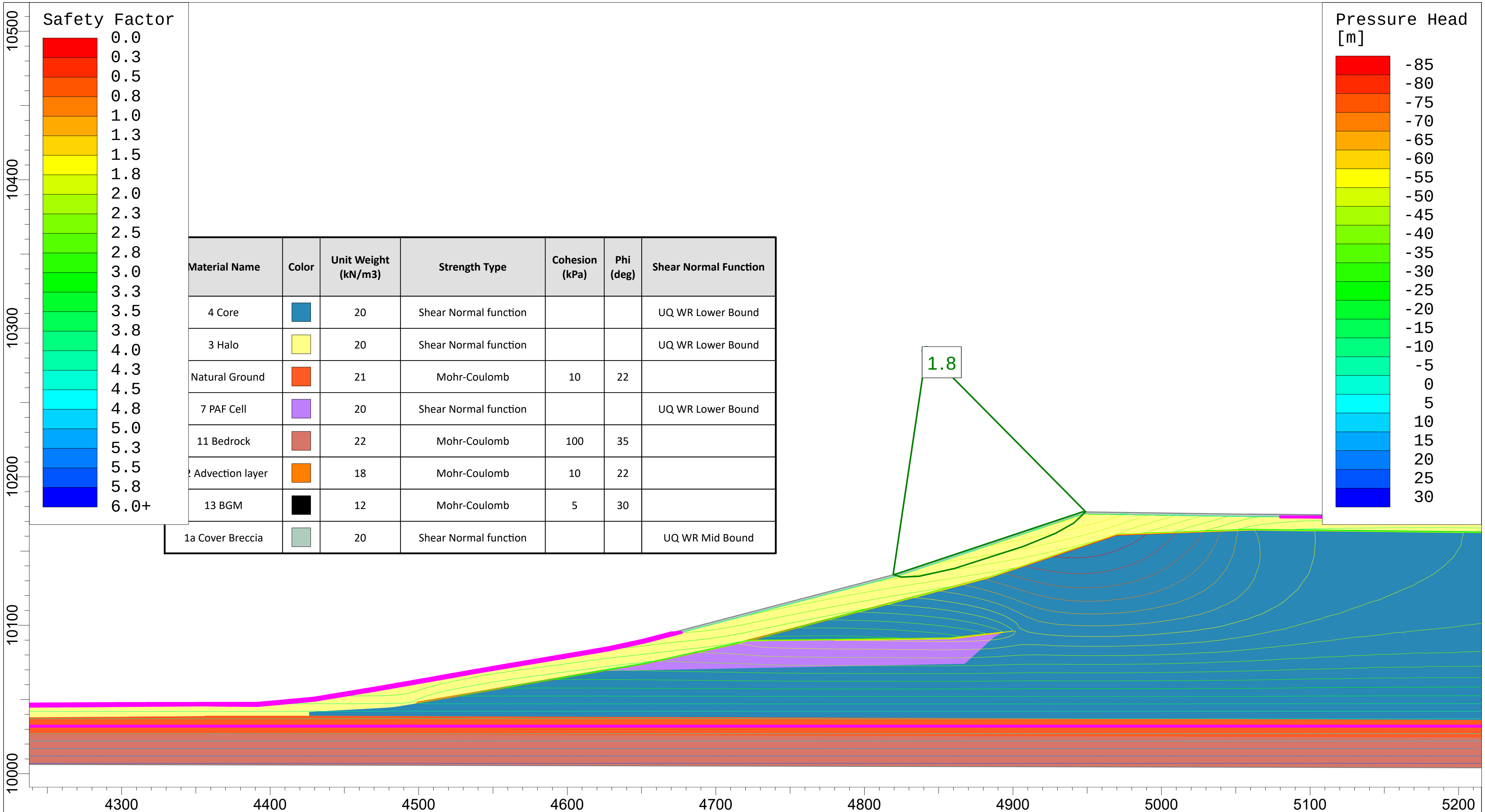
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
Natural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
2 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS
Description: Section C-C South-Wet Season	Analysis Method: Circular
Drawn By: SB	Scale: 1:3592
Company	MRM/Pando
Date	June 2017
File Name:	section c-c south update_bgm_wet.slmd



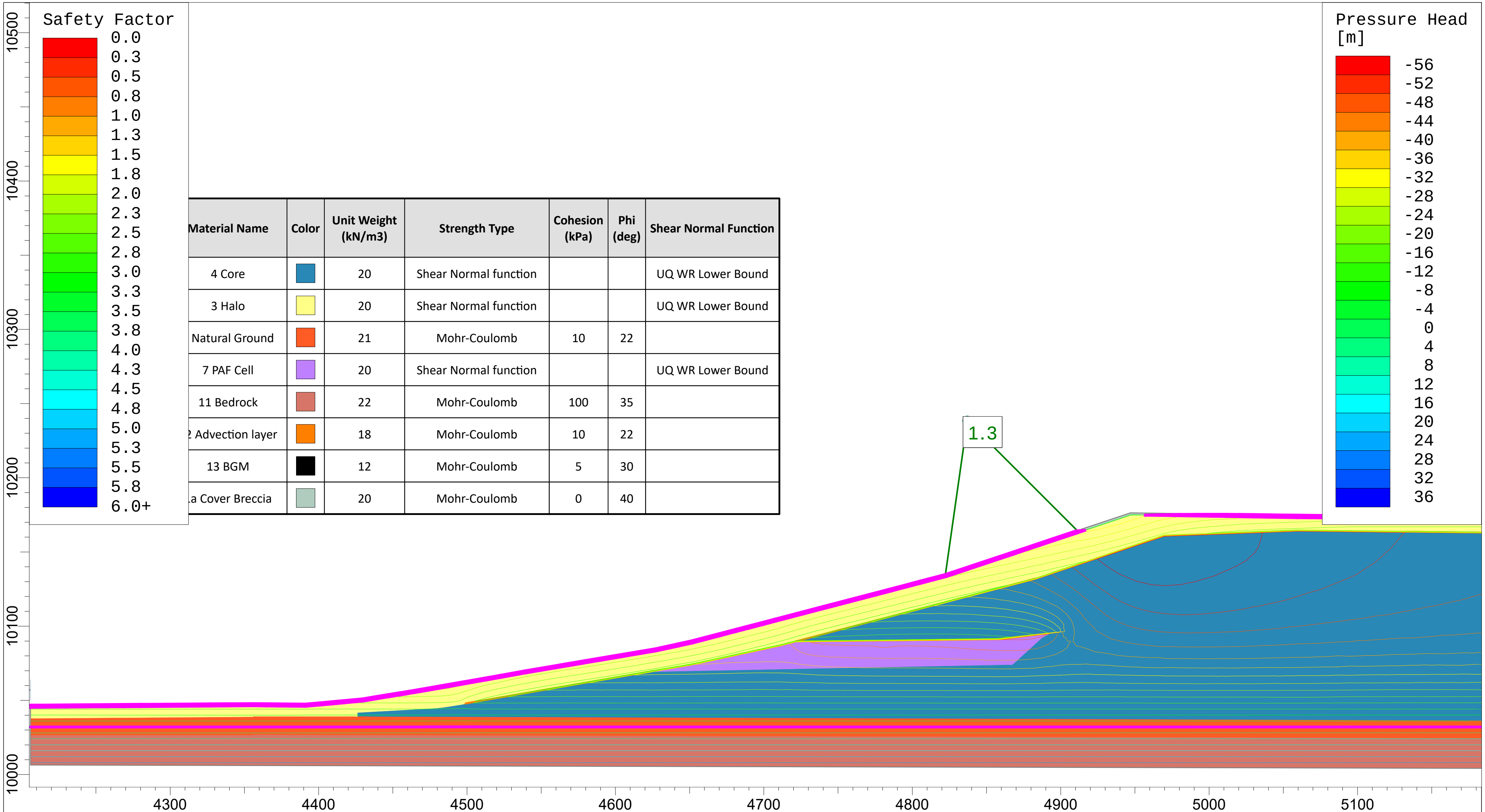
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
Natural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section C-C South-Wet Season		Analysis Method: Cuckoo	
Drawn By SB	*Scale* 1:3592	*Company*	MRM/Pando
Date	June 2017	*File Name:*	section c-c south update_bgm_wet.slmd



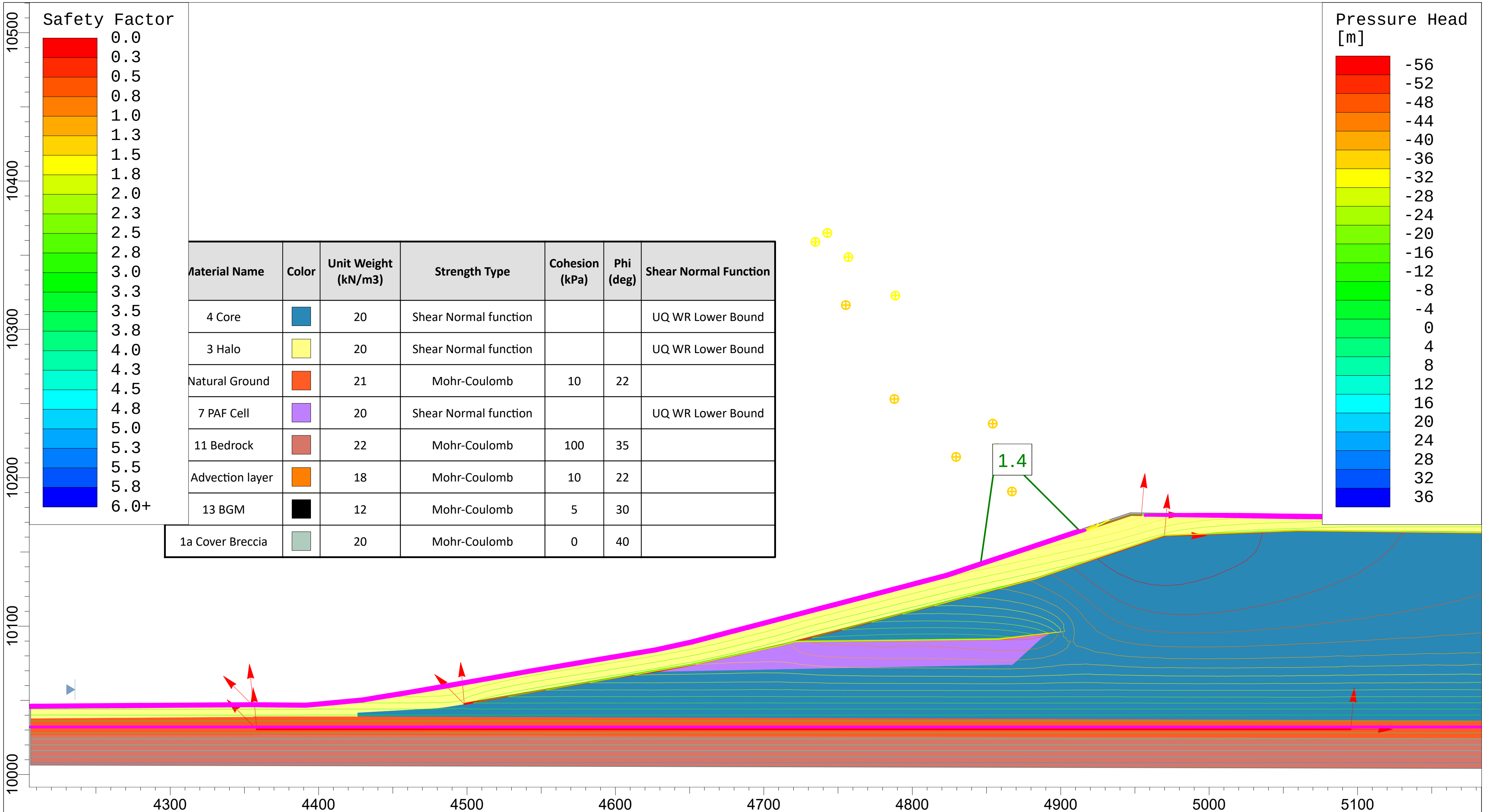
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
Natural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
2 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
a Cover Breccia		20	Mohr-Coulomb	0	40	

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS
Description: Section C-C South_breccia_sensitivity_1-1000	Analysis Method: Cuckoo
Drawn By: SB	Scale: 1:3595
Company	MRM/Pando
Date	June 2017
File Name:	section c-c south update_bgm_breccia_1-1000.slmd



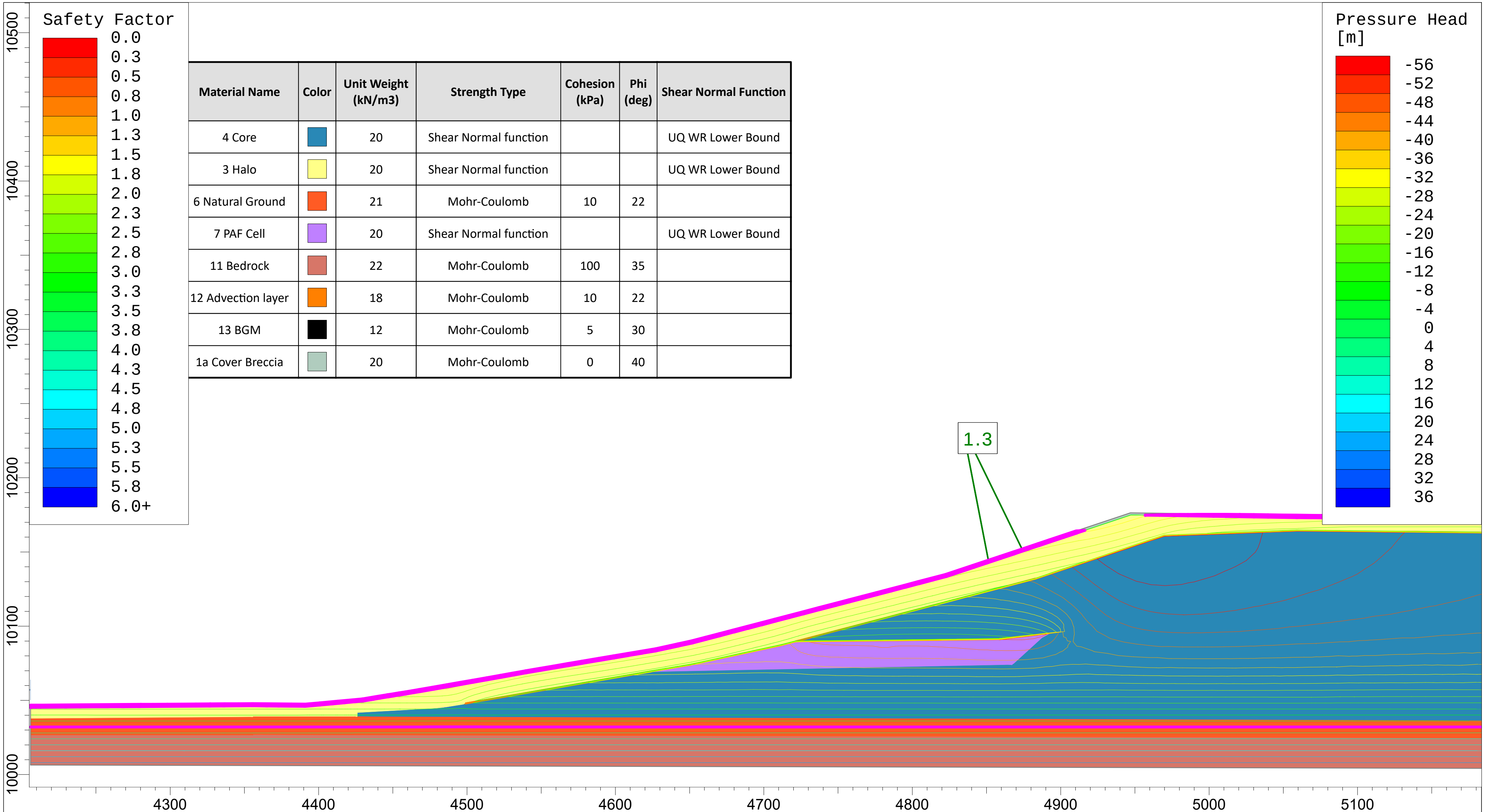
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
Natural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

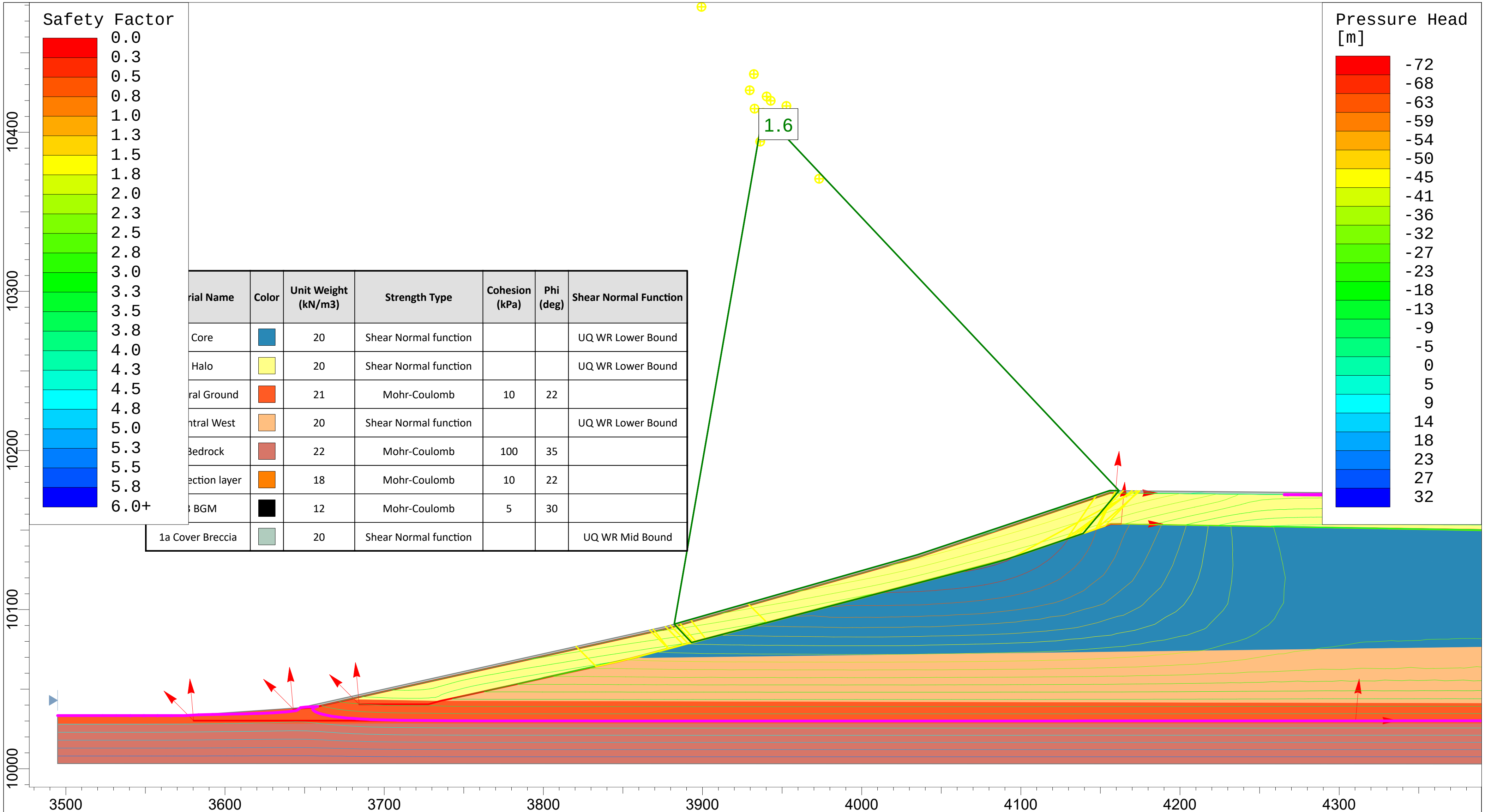
Project	MRM NOEF EIS
Description: Section C-C South_breccia_sensitivity_1-1000	Analysis Method: Block
Drawn By: SB; Scale: 1:3595	Company: MRM/Pando
Date: June 2017	File Name: section c-c south update_bgm_breccia_1-1000.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
7 PAF Cell		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

Project	MRM NOEF EIS
Description: Section C-C South_breccia_sensitivity_1-1000	Analysis Method: Circular
Drawn By: SB / Scale: 1:3595	Company: MRM/Pando
Date: June 2017	File Name: section c-c south update_bgm_breccia_1-1000.slmd

SLIDEINTERPRET 7.030



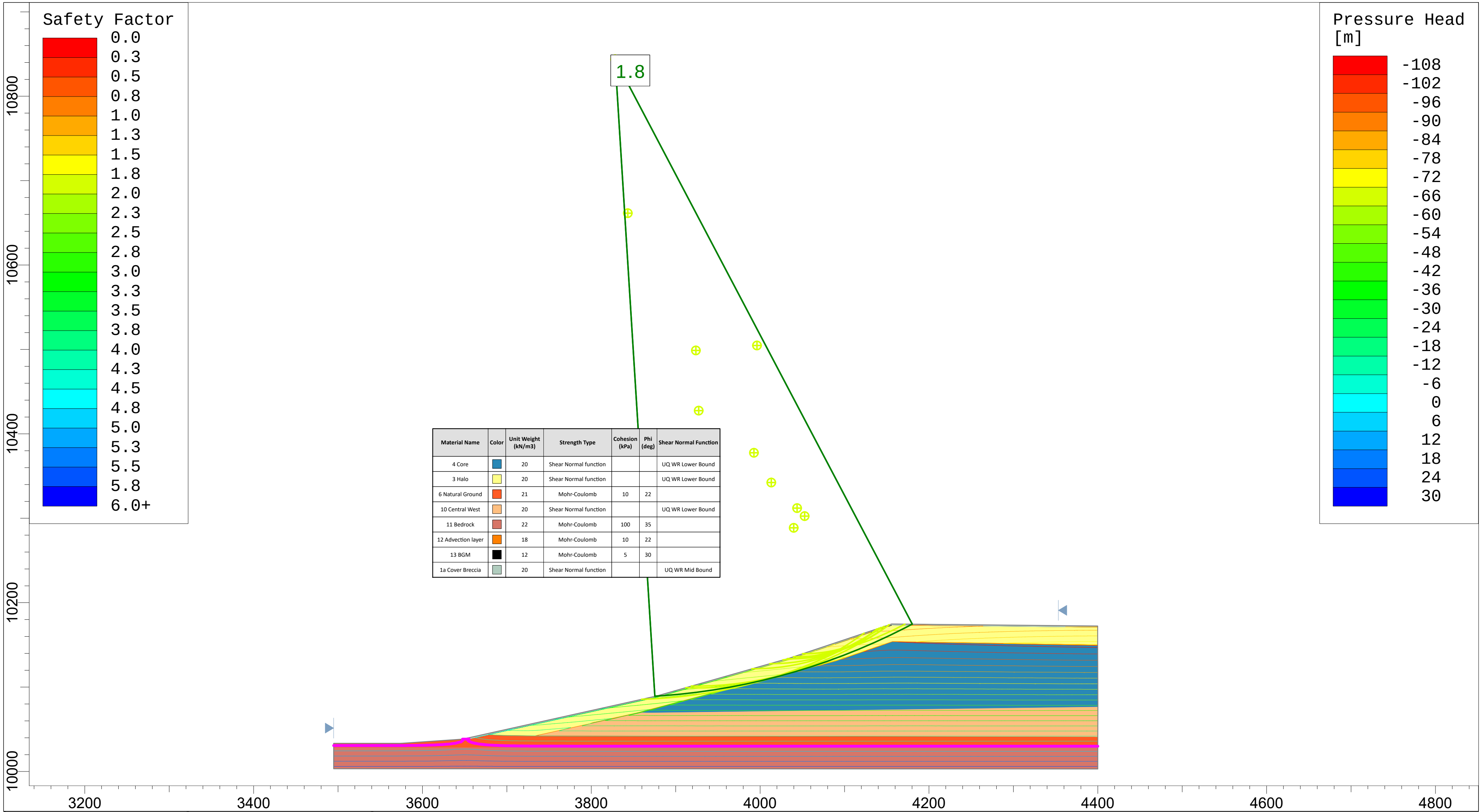
rial Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
Core		20	Shear Normal function			UQ WR Lower Bound
Halo		20	Shear Normal function			UQ WR Lower Bound
ral Ground		21	Mohr-Coulomb	10	22	
ntral West		20	Shear Normal function			UQ WR Lower Bound
Bedrock		22	Mohr-Coulomb	100	35	
ection layer		18	Mohr-Coulomb	10	22	
BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section D-D Dry Season		Analysis Method: Block	
Drawn By SB	*Scale* 1:3352	*Company*	MRM/Pando
Date	June 2017	*File Name:*	section d-d update_bgm_dry.slmd



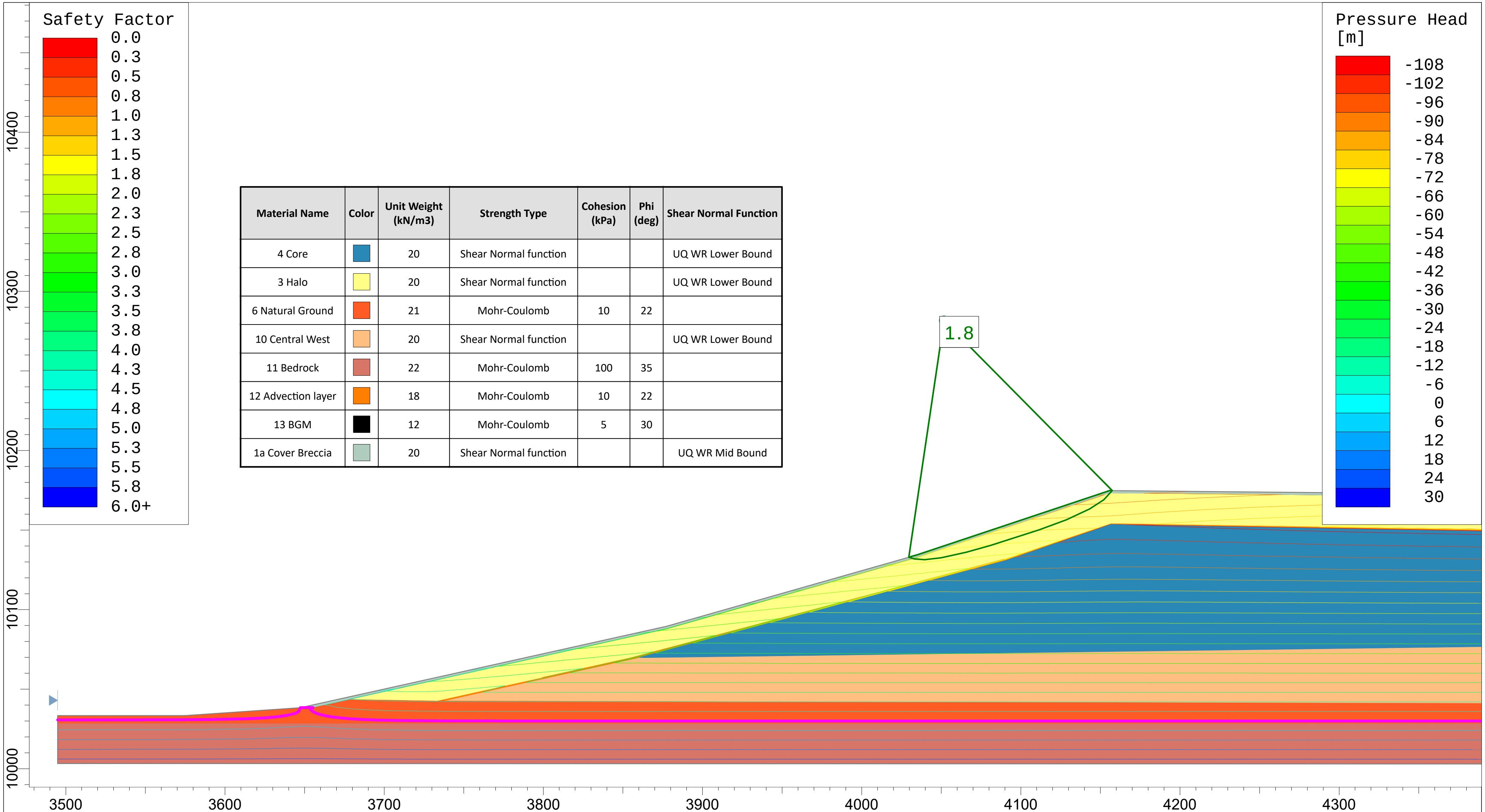
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
10 Central West		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS
Description: Section D-D Dry Season	Analysis Method: Circular
Drawn By SB	*Scale* 1:6308
Company	MRM/Pando
Date	June 2017
File Name:	section d-d update_bgm_dry.slmd



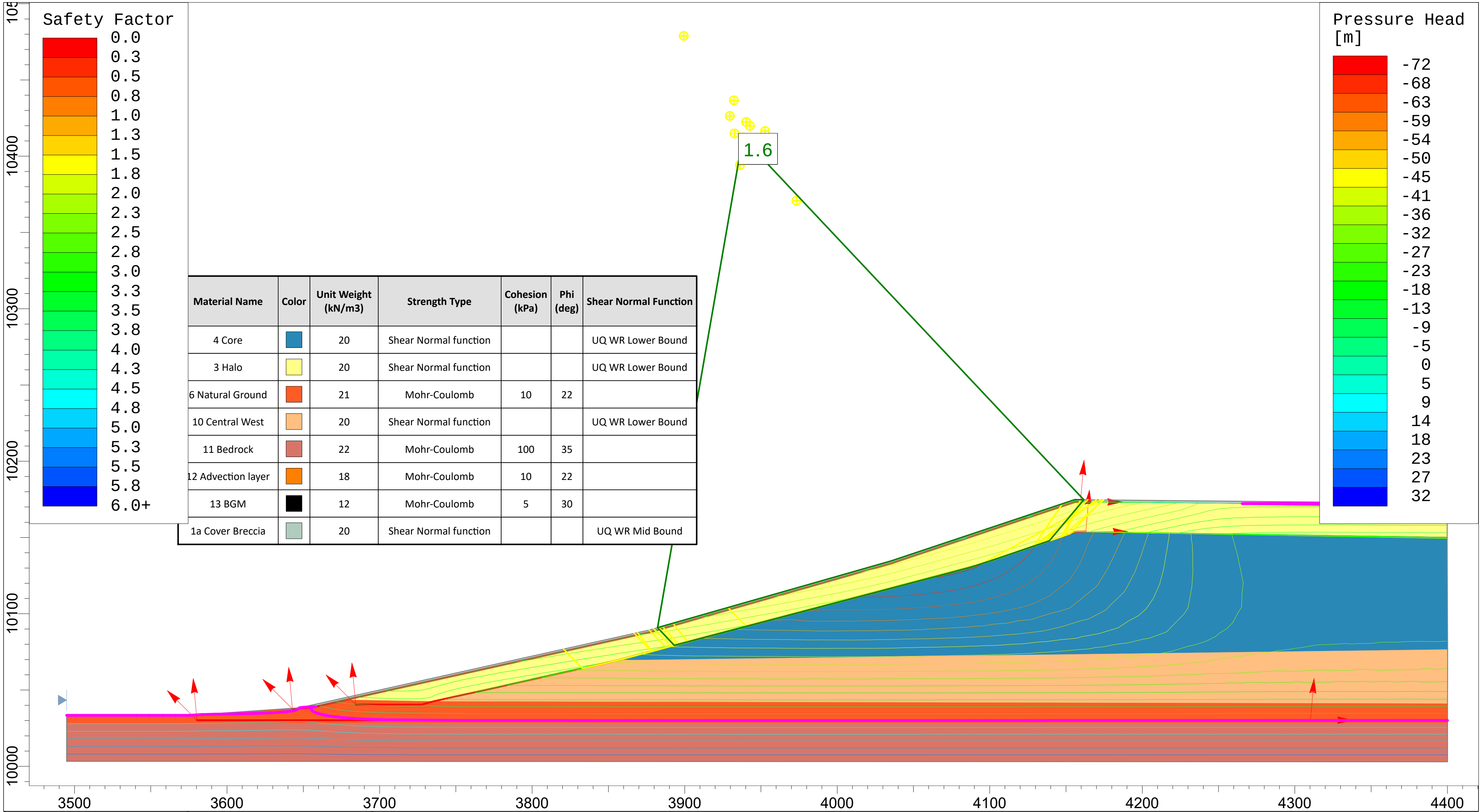
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
10 Central West		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section D-D Dry Season		Analysis Method: Cuckoo	
Drawn By SB	*Scale* 1:3352	*Company*	MRM/Pando
Date	June 2017	*File Name:*	section d-d update_bgm_dry.slmd



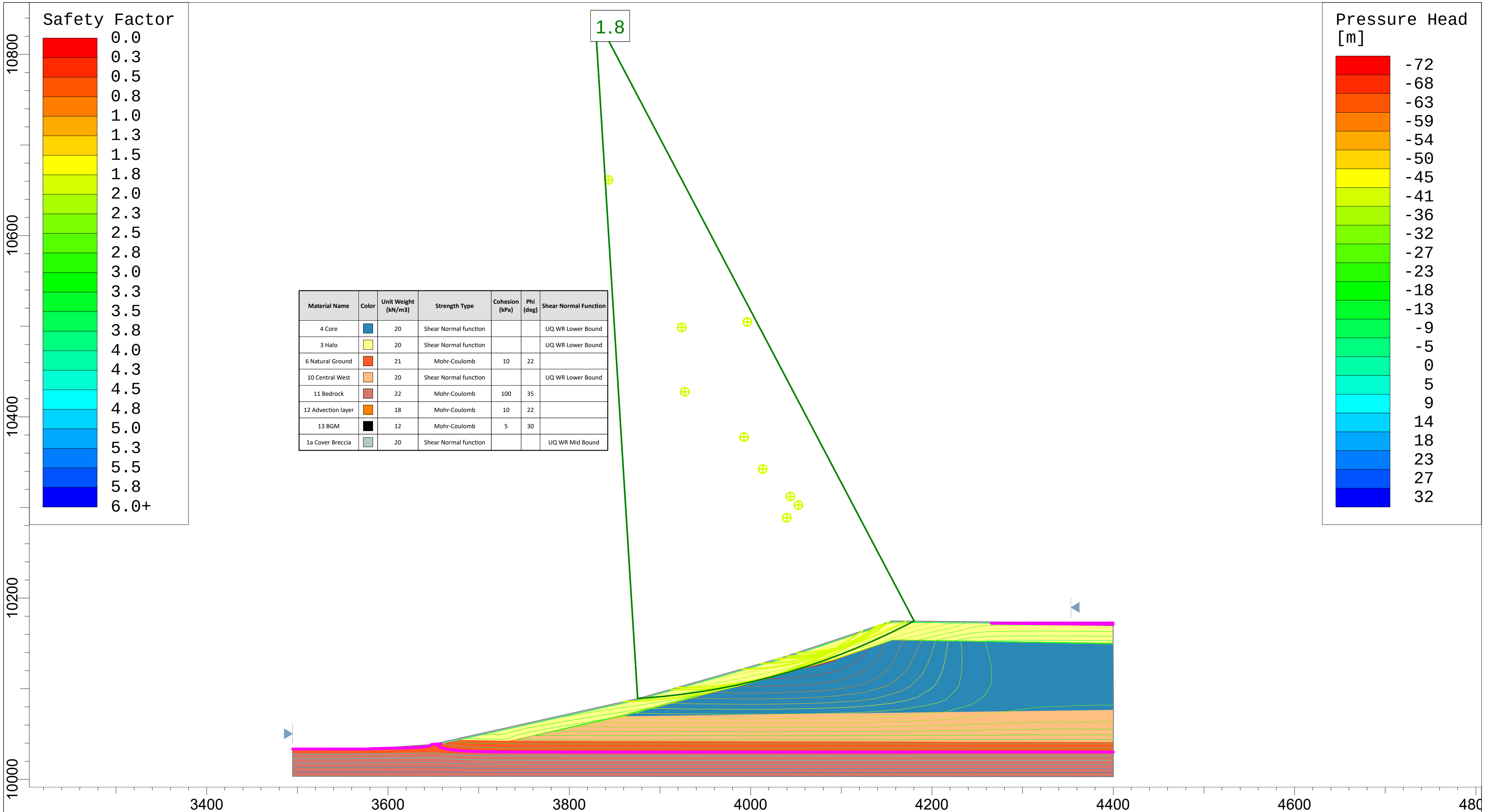
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
10 Central West		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

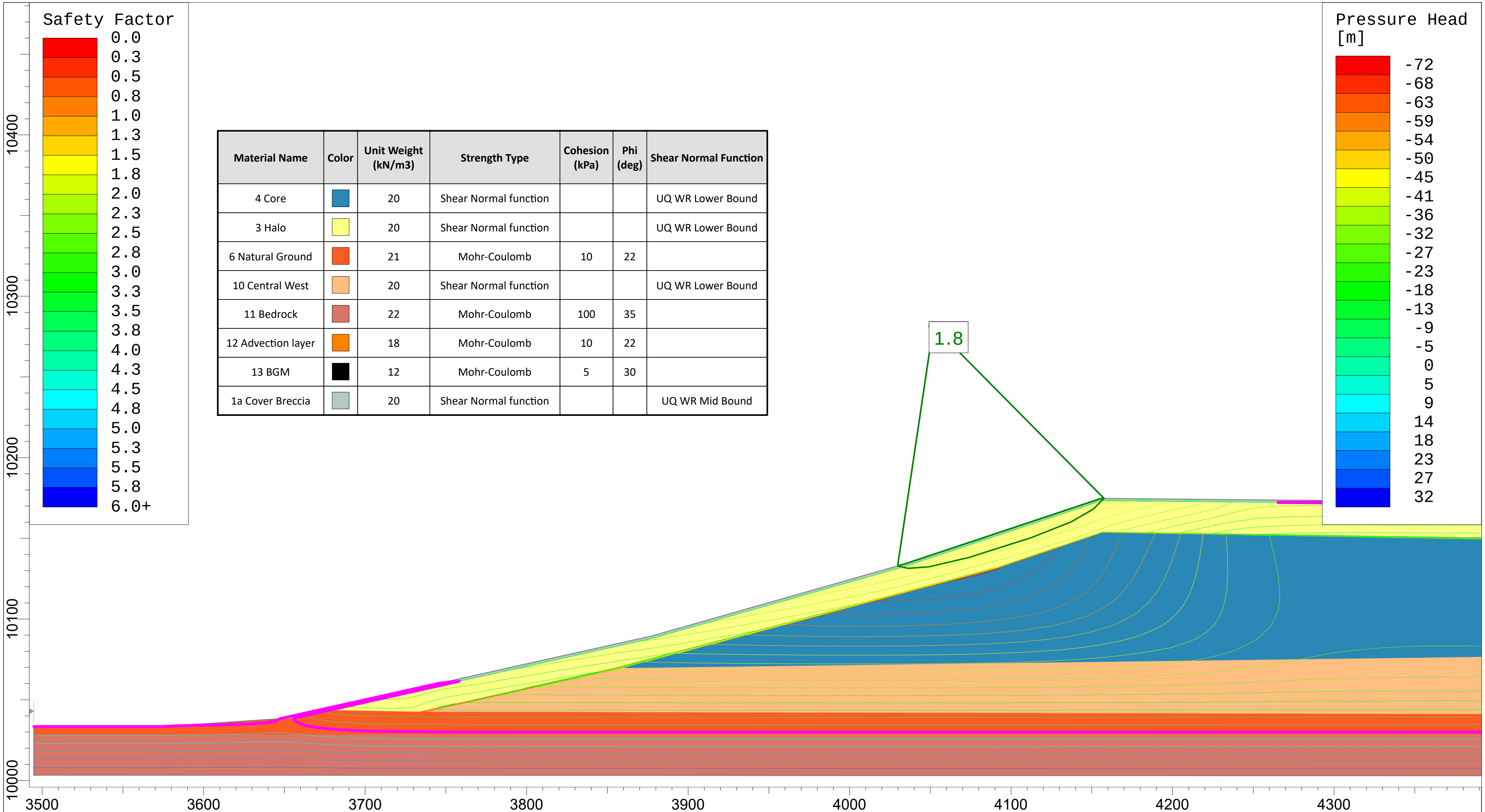
Project	MRM NOEF EIS		
Description: Section D-D Wet Season		Analysis Method: Block	
Drawn By SB	*Scale* 1:3491	*Company*	MRM/Pando
Date June 2017		*File Name:*	section d-d update_bgm_wet.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
10 Central West		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section D-D Wet Season		Analysis Method: Circular	
Drawn By SB	*Scale* 1:5885	*Company*	MRM/Pando
Date	June 2017	*File Name:*	section d-d update_bgm_wet.slmd



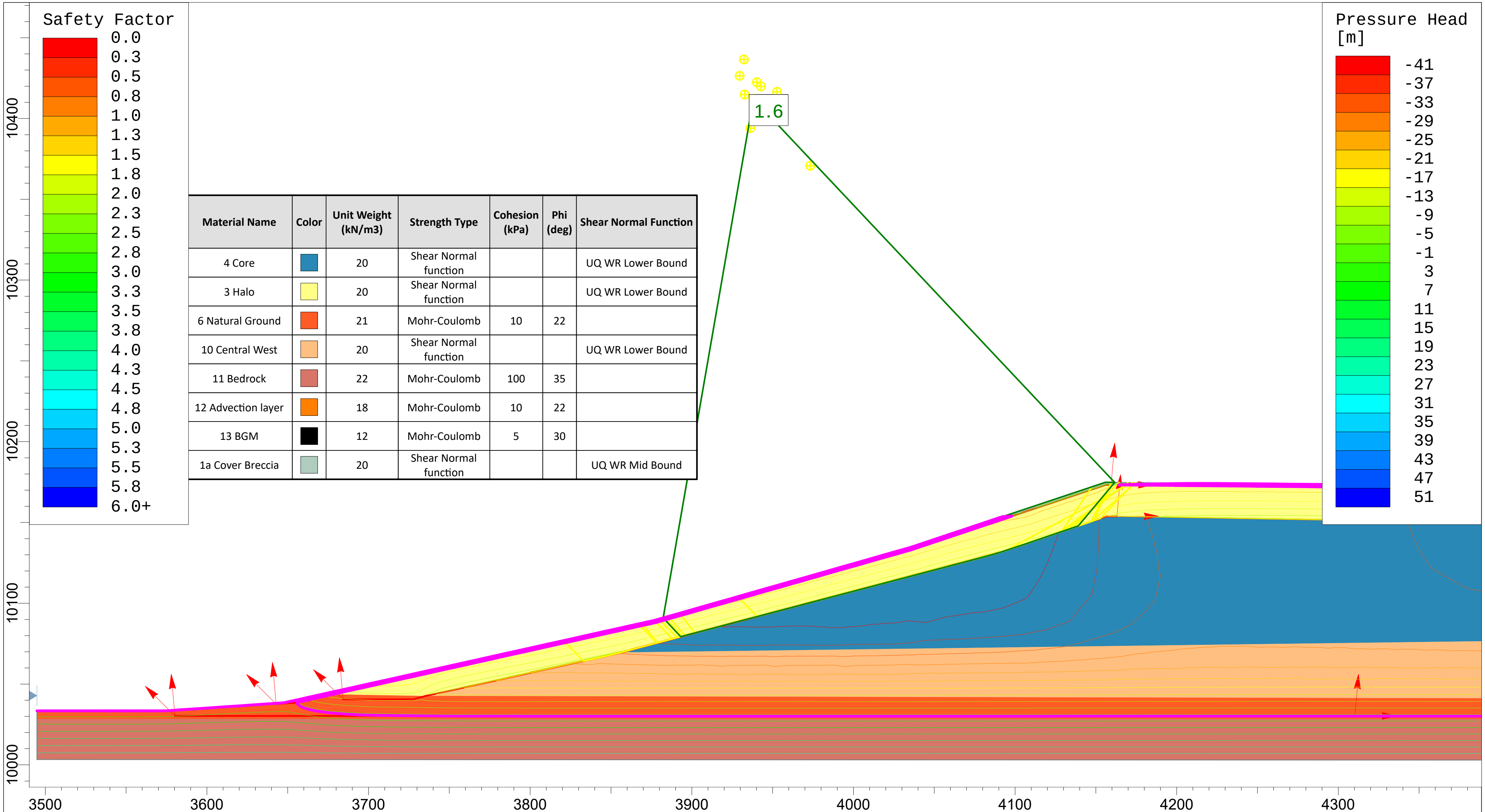
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
10 Central West		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section D-D Wet Season		Analysis Method: Cuckoo	
Drawn By SB	*Scale* 1:3304	*Company* MRM/Pando	
Date June 2017		*File Name:* section d-d update_bgm_wet.slmd	



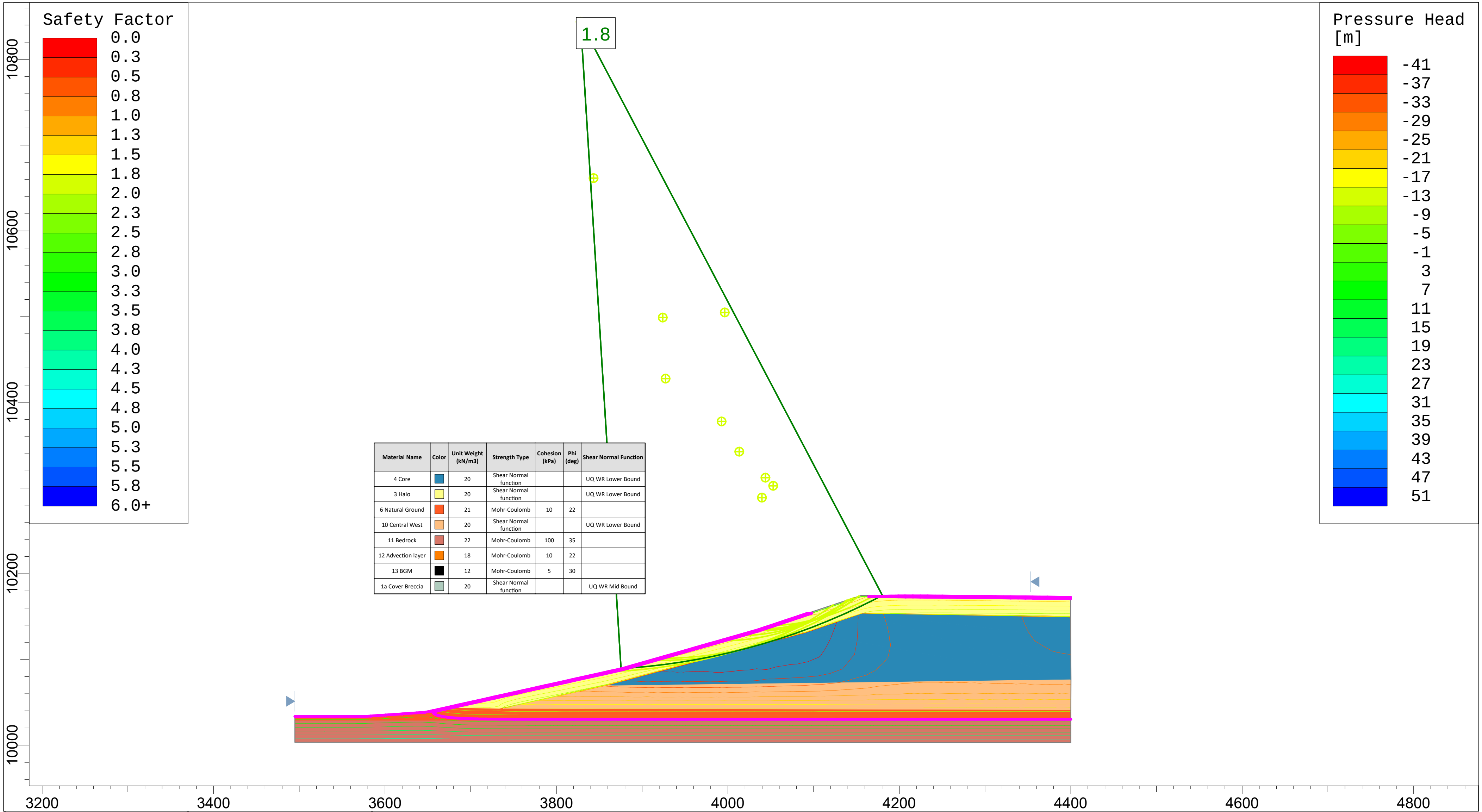
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
10 Central West		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

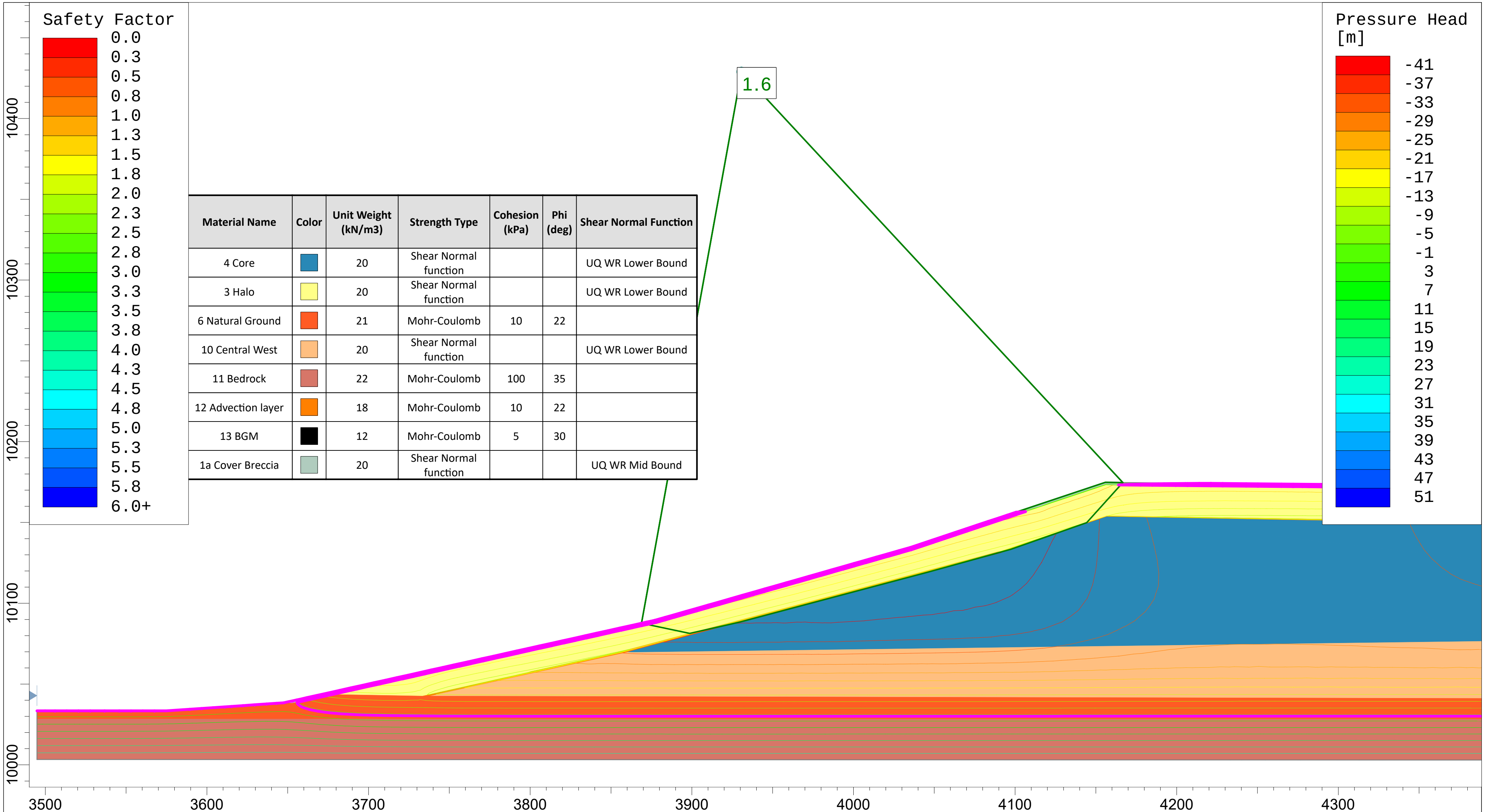
Project	MRM NOEF EIS		
Description: Section D-D_1-1000		Analysis Method: Block	
Drawn By	SB	Scale	1:3301
Company	MRM/Pando		
Date	June 2017	File Name:	section d-d update_bgm_1-1000.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
10 Central West		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Project	MRM NOEF EIS	
Description: Section D-D_1-1000		Analysis Method: Circular
Drawn By: SB	Scale: 1:6212	Company: MRM/Pando
Date: June 2017		File Name: section d-d update_bgm_1-1000.slmd

SLIDEINTERPRET 7.030



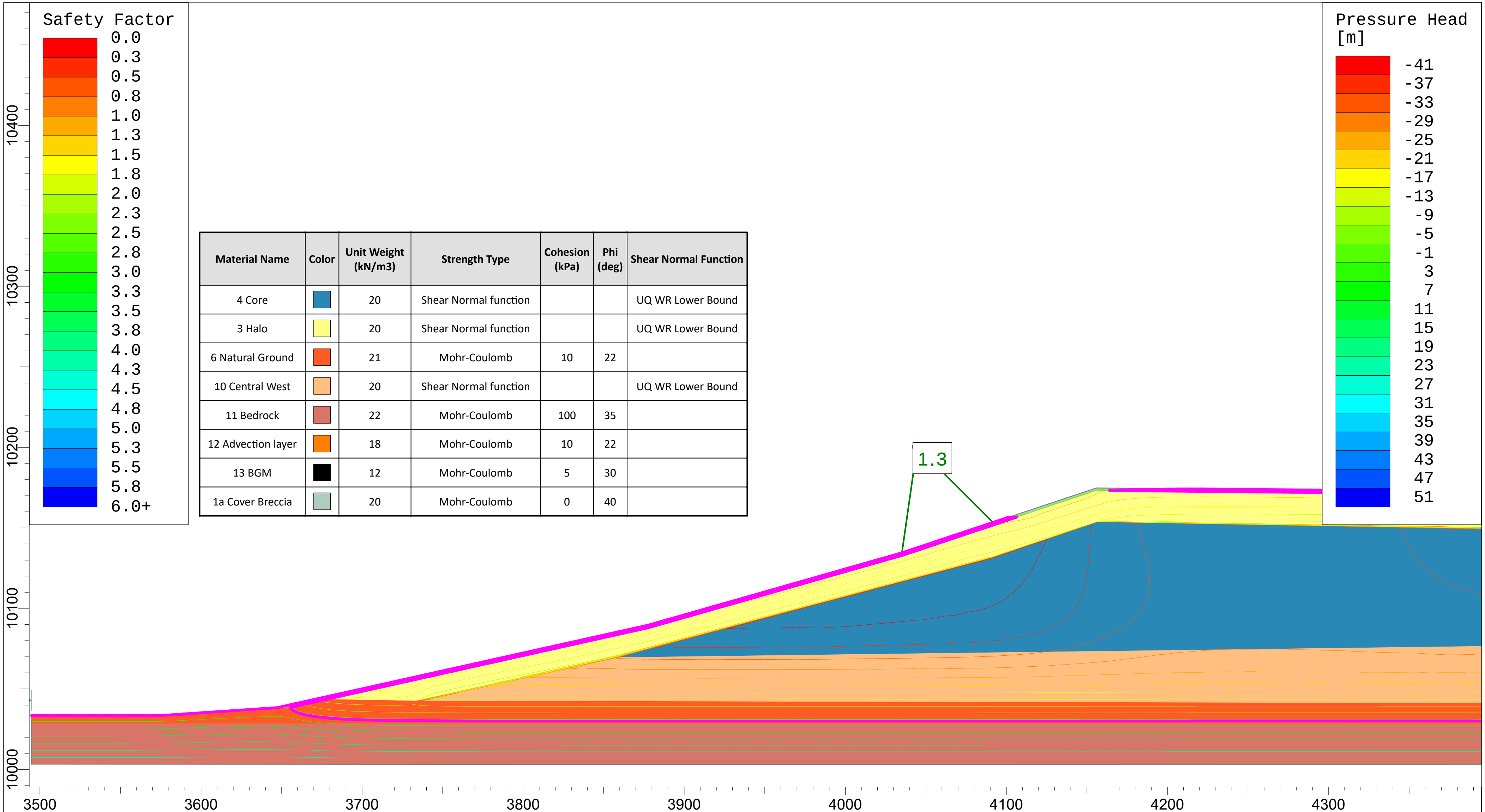
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
10 Central West		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Shear Normal function			UQ WR Mid Bound

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS
Description: Section D-D_1-1000	Analysis Method: Cuckoo
Drawn By SB	*Scale* 1:3301
Company	MRM/Pando
Date	June 2017
File Name:	section d-d update_bgm_1-1000.slmd



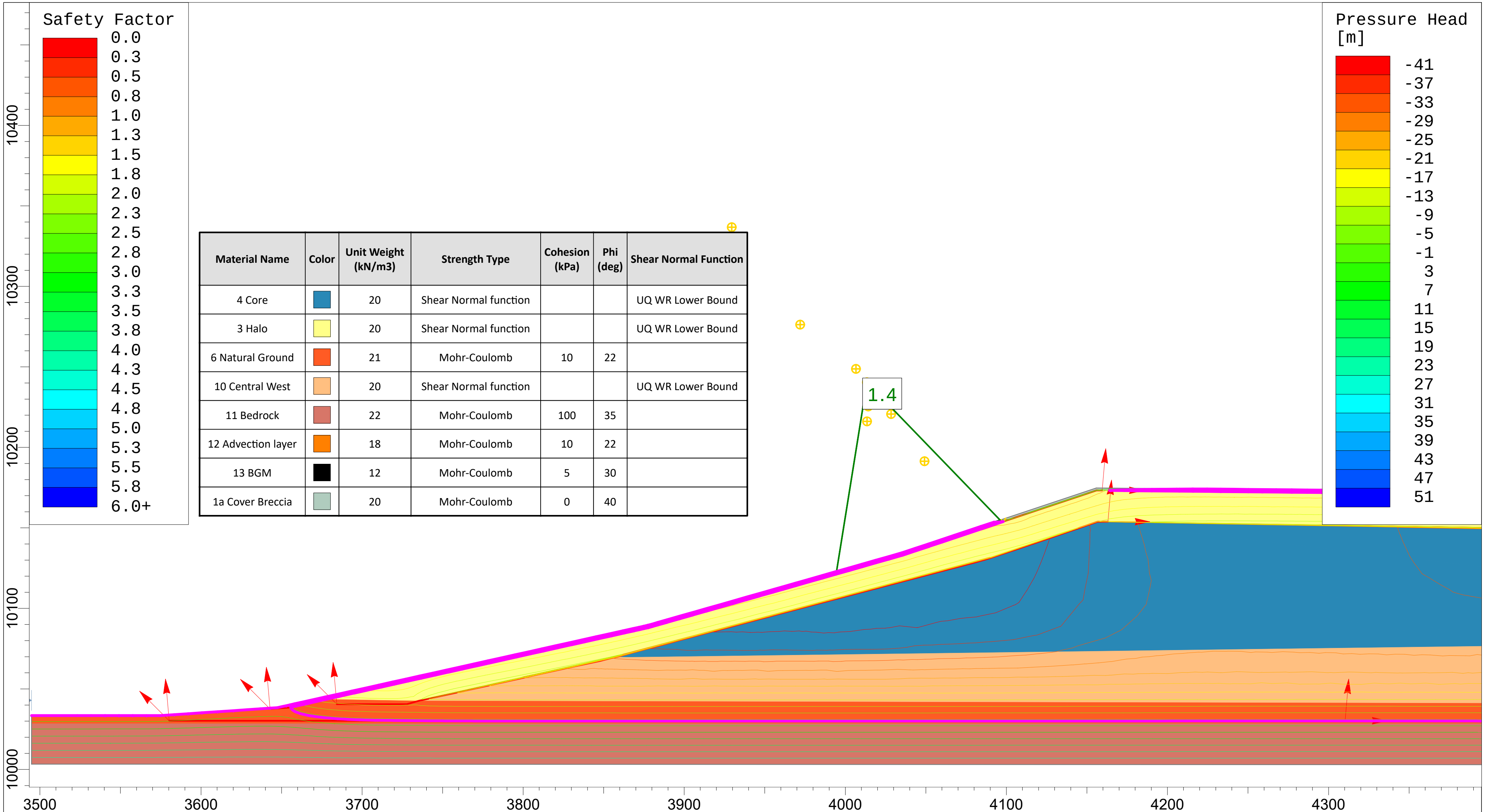
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
10 Central West		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section D-D_breccia_sensitivity_1-1000		Analysis Method: Cuckoo	
Drawn By SB	*Scale* 1:3312	*Company*	MRM/Pando
Date	June 2017	*File Name:*	section d-d update_bgm_breccia_1-1000.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
10 Central West		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

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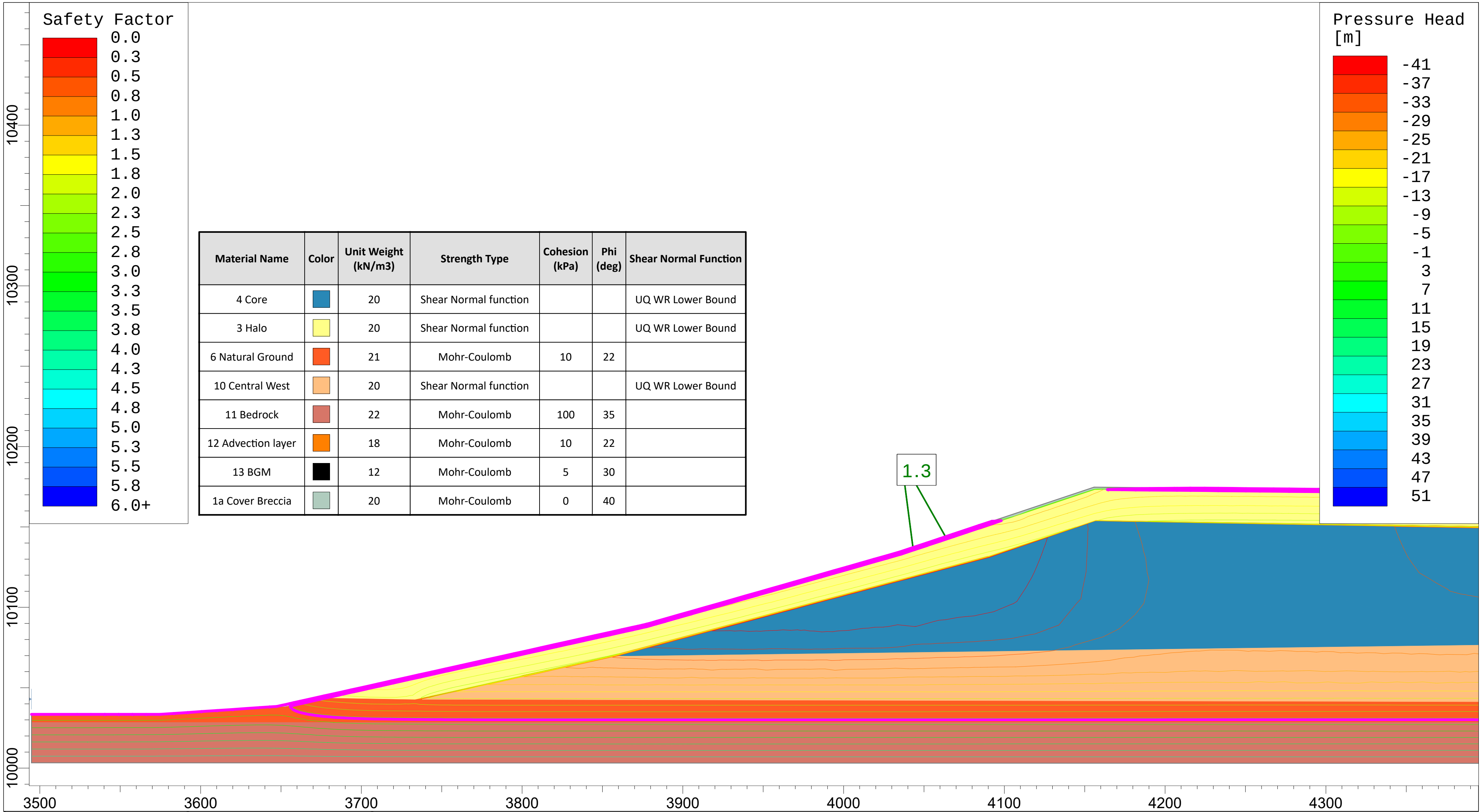
McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section D-D_breccia_sensitivity_1-1000		Analysis Method: Block	
Drawn By SB	*Scale* 1:3312	*Company*	MRM/Pando
Date June 2017		*File Name:*	section d-d update_bgm_breccia_1-1000.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Shear Normal Function
4 Core		20	Shear Normal function			UQ WR Lower Bound
3 Halo		20	Shear Normal function			UQ WR Lower Bound
6 Natural Ground		21	Mohr-Coulomb	10	22	
10 Central West		20	Shear Normal function			UQ WR Lower Bound
11 Bedrock		22	Mohr-Coulomb	100	35	
12 Advection layer		18	Mohr-Coulomb	10	22	
13 BGM		12	Mohr-Coulomb	5	30	
1a Cover Breccia		20	Mohr-Coulomb	0	40	

Pando

McArthur River Mining

SLIDEINTERPRET 7.030

Project	MRM NOEF EIS		
Description: Section D-D_breccia_sensitivity_1-1000		Analysis Method: Circular	
Drawn By SB	*Scale* 1:3312	*Company*	MRM/Pando
Date	June 2017	*File Name:*	section d-d update_bgm_breccia_1-1000.slmd

Appendix B GHD settlement assessment

Memorandum

22 December 2017

To	Jamie Hacker/Stan Blanks (MRM)		
Copy to	James Thorp		
From	Javier de la Rosa	Tel	61 8 6222 8262
Subject	MRM NOEF - Preliminary Settlement Estimation	Job no.	32/17428

1 Introduction

The McArthur River Mine (MRM) proposal for the closure of the Northern Overburden Emplacement Facility (NOEF) includes a NOEF cover system.

MRM engaged GHD Pty Ltd (GHD) to carry out a preliminary assessment of the NOEF settlements and liner strains after closure.

2 Purpose of memorandum

The purpose of this memorandum is to present the data obtained during recent geotechnical fieldwork and to summarise the assumptions, interpretation and results of the analyses carried out.

3 Scope of work

The scope of works includes the following:

- Carry out a desktop study review of published literature that addresses creep settlement of rock fills.
- A fieldwork programme comprising in situ Plate Load Tests (PLTs) on the materials contained within the NOEF.
- A laboratory testing programme carried out on selected samples recovered from the PLT locations.
- Finite Element (FE) analyses on a simplified NOEF section.
- Estimate settlements and liner strains during the NOEF development and progressive capping activities and after NOEF closure is complete
- Provide a memorandum to summarise the fieldwork and the results of the FE analyses carried out.
- Provide recommendations for further work, if required.

4 Background information

The following information has been referred to:

- GHD Memorandum, McArthur River Mine, CW Stage – Deformation Analyses Update and Preliminary Seismic Deformation Analyses, March 2015, Reference 32/17428/148635.
- NOEF historical information provided by MRM, email dated 9th October 2017

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- Five sections provided by MRM, email dated 24th October 2017.

5 Fieldwork

5.1 General

The fieldwork was carried out in October 2017 and comprised the following:

- A total of 20 PLTs; on NOEF materials, at various locations to include different fill age or lift thickness.
- Two ‘soaked’ PLTs carried out on NOEF materials, carried out adjacent to an existing PLT, after soaking the area with about 70 kL of water.

The fieldwork was carried out under the assistance and coordination of a GHD Engineer and in general accordance with Australian Standard AS 1726-1993–Geotechnical Site Investigations and standard GHD procedures.

The locations were surveyed by MRM with a differential GPS. The accuracy was not provided at the time of writing.

5.2 Material description

The fill material at the PLTs locations was described as dense, silty sandy GRAVEL with cobbles, flat / equidimensional, angular / sub-angular, well graded, shale / breccia.

From a geochemical perspective, test locations covered areas of benign and non-benign rock types however this has no significant influence on the geotechnical characteristics being investigated.

5.3 Plate Load Tests (PLTs)

PLTs were carried out by Construction Sciences using a 600 mm plate. A 50-ton jack, provided and operated by Construction Sciences, was used during the tests.

The PLTs were carried out generally in accordance with ASTM D1196. Each test had two loading and two unloading cycles.

The location of the PLTs are included in Table 1, and shown graphically in Figure 1, in Appendix A.

Table 1 Plate Load Tests Location Summary

Test ID	Coordinates (Mine Grid)		Elevation (mAHD)
	Easting (m)	Northing (m)	
PLT01	6964.1	4697.3	10100.2
PLT02	6938.5	4559.4	10092.8
PLT03	6813.0	4712.9	10099.1
PLT04	6654.4	4639.8	10097.2

Test ID	Coordinates (Mine Grid)		Elevation (mAHD)
	Easting (m)	Northing (m)	
PLT05	6130.8	4910.1	10099.7
PLT06(*)	7186.0	4409.1	10079.8
PLT07	7217.2	4698.8	10082.7
PLT08	6258.2	4745.7	10098.0
PLT09	5883.9	4717.7	10092.1
PLT10	6834.4	4563.5	10091.5
PLT11	6652.0	5175.5	10049.8
PLT12	6432.2	5300.2	10048.6
PLT13	7579.8	4717.5	10054.3
PLT14	7828.6	4683.6	10052.6
PLT15(*)	7855.3	4294.2	10051.0
PLT16	7993.1	4064.9	10039.9
PLT17	7996.6	3946.7	10041.1
PLT18	7716.8	3776.4	10043.6
PLT19	7558.1	4286.1	10067.9
PLT20 (+)	7615.3	4065.3	10063.7

Note: (*): soaked PLT carried out at this location.

(+) test result not received

The results of the PLTs are summarised in Figure 2 to Figure 6.

The results in first loading indicates an average Young's Modulus (E) of 60 MPa and an average of 100 MPa for the second loading, as shown in Figure 7. The soaked PLT indicate an E of 30 MPa, suggesting a significant reduction on soaking.

The unloading cycles indicate an average unloading/reloading Young's Modulus (Eur) of 300 MPa for both cycles (refer to Figure 8). This value does not appear to be affected by the 'soaking' process.

When reviewing the test results, the material geochemical classification has no clear influence on the resulting Young's modulus. Similarly, the lift height or the age of the fill at the test location does not significantly influence the results.

6 Laboratory testing

Four Particle Size Distribution (PSDs) tests were performed on surface samples collected at selected PLT locations. The tests were performed by Construction Science Pty Ltd at their MRM site laboratory.

The results are summarised in Table 2 and shown graphically in Figure 9.

It should be noted that particles over 200 mm size were not tested in the PSDs. This is a limitation of the test (particles greater than 200 mm are present within the NOEF); however, this is not considered a significant issue, as the tests were undertaken mainly to confirm field logging and to ensure that the PLTs were performed on similar materials.

The PSDs confirmed the field description and the relative homogeneity in grading for the material underlying the different test locations.

Table 2 Particle Size Distribution Summary

Sample	Particle Size Distribution			
	% Fines (<0.75mm)	%Sand (<2.36mm)	% Gravel (<63mm)	% Cobbles (<200mm)
PLT5	8	23	68	1
PLT7	1	3	92	4
PLT13	4	12	72	12
PLT20	3	13	81	3

7 Settlement estimation

7.1 General

Settlements can be interpreted as the sum of three separate components, as follows:

- Immediate settlements; the result of constant volume distortion of the soil mass, due to an applied load. The immediate settlements occur during the construction of the lifts.
- Consolidation settlement; the result of time dependant flow of water from the mass, due to load application.
- Secondary settlement; or creep, also time dependent but occurring essentially at constant effective stress. Creep will continue many years after the end of the construction, but will reduce with time.

The NOEF materials are granular; as such, consolidation settlements are not expected. Any excess pore water pressure will dissipate relatively quickly, generally within the construction period.

Immediate and creep settlements are discussed in the following sections.

7.2 Section analysed

Sections of the proposed NOEF were provided by MRM (refer to Section 4). It should be noted that the sections have been simplified, as shown in Figure 10. This is suitable for the preliminary settlement estimation carried out herein.

7.3 Immediate settlements

7.3.1 General

The analyses of the immediate settlements were carried out via Finite Element (FE) modelling, using the commercially available software, Plaxis 2D.

The software allows for the simulation of non-linear behaviour of the elasticity modulus. Additionally, the software allows for the modelling of construction stages, which provides more realistic deformation estimates.

7.3.2 Soil models

The soil behaviour can be modelled with various degrees of accuracy, depending on the soil model used. The selection depends on a number of aspects, such as load-unloading history and groundwater regime. The soil models used in the analyses are summarised in Table 3.

Table 3 Soil Models Summary

Material	Soil Model	Soil Model Description
NOEF Waste Rock	Hardening Soil (HS)	An advanced soil model. The limiting states of stress are described by ϕ', c' and ψ. The soil stiffness is modelled by using three different input stiffnesses; E_{50}, E_{ur} and E_{oed}. This model accounts for stress-dependency of the stiffness moduli.
Clay Fill	Mohr-Coulomb (MC)	A linear elastic – perfectly plastic model with a fixed yield surface. It has five input parameters, E and ν for soil elasticity, and limiting states of stress are defined by ϕ', c' and ψ. It is a good, reliable, first-order model.
Bedrock	Linear Elastic (LE)	It is the simplest available stress-strain relationship. Based on Hooke's law, it only requires two input parameters, E and ν.

Note: ϕ': friction angle; c': cohesion; ψ: dilatancy angle; E_{50}: triaxial stiffness; E_{ur}: unloaded/reload stiffness; E_{oed}: oedometer stiffness; E: Young's modulus; ν: Poisson's ratio.

7.3.3 Geotechnical parameters

The parameters used for the settlement estimation (refer to Table 4) are based on both the in-situ testing carried out and published correlations with similar materials.

For use in the HS model, the Young's Modulus obtained from the PLTs was normalised to a confined horizontal stress of 100 kPa, using the following relationship; $E_{50}^{ref} = \frac{E_{50}}{(\sigma'_h/p_{ref})^m}$.

Where:

E_{50}: interpreted modulus from PLTs.
σ'_h: effective horizontal stress.
p_{ref}: reference pressure.
m: stress dependency parameter, assumed to be 0.50 based on published literature recommendations for granular materials.

Table 4 Geotechnical Parameters Summary

Material	Soil Model	Geotechnical Parameter										
		γ (kN/m³)	c' (kPa)	ϕ' (˚)	ψ (˚)	E (MPa)	ν	E_{50}^{ref} (MPa)	E_{oed}^{ref} (MPa)	E_{ur}^{ref} (MPa)	m	p_{ref}
Waste Rock	HS	18	5	40	10	-	0.2	125$^{(*)}$	125$^{(*)}$	600	0.5	100
Clay Fill	MC	19	5	28	0	25	0.3	-	-	-	-	-
Bedrock	LE	22	-	-	-	500	0.3	-	-	-	-	-

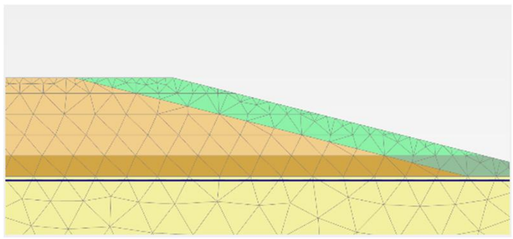
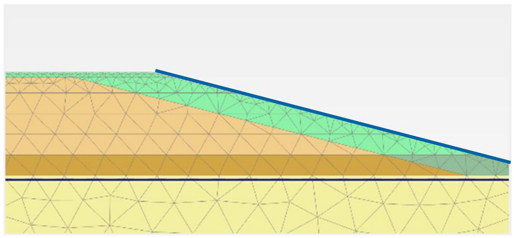
Note: γ: unit weight; (*): reduced to 60 MPa when soaked.

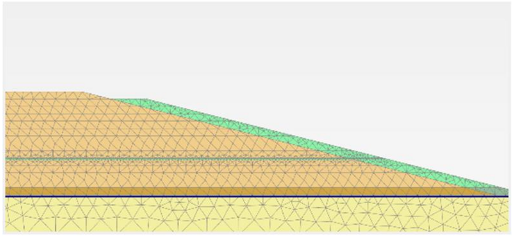
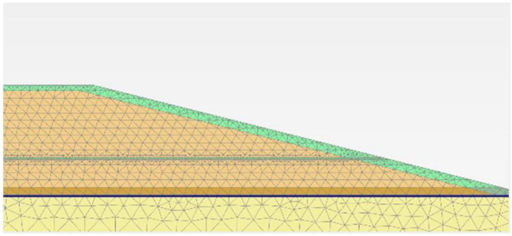
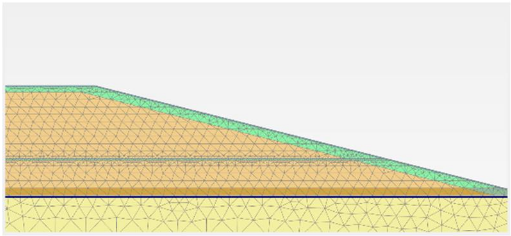
7.3.4 *Modelling stages*

The stages used in the analyses are summarised in Table 5. All stages were computed in drained conditions, i.e. no excess pore pressure, as the soils on site are granular and excess pore pressure will dissipate in a relatively short period. The following assumptions were made during the modelling:

- 10 m thick NOEF embankment lifts. This assumption allows the stresses to develop in the model consistently with the backfilling process. It is different from the actual construction lifts (between 3 and 15m) for the purpose of this simplified preliminary model. It is deemed a reasonable assumption to evaluate the liner settlements.
- As a conservative assumption which might simulate wetting up at the base of the dump above the basal clay layer, the bottom 10 m of the NOEF has been assumed to be 'soaked' (with reference to the plate load test results); therefore, reduced elastic moduli were used (refer to Table 4). It is noted that this condition is less likely below the main core of the dump and more likely around the periphery of the dump.
- The NOEF core will be covered by a waste rock "halo" forming the base for the cover system.
- The cover installation occurs concurrently with the liner installation (not visible in sketches below due to scale). (refer to Figure 10 attached).
- The cover includes the proposed liner covered by a 1.5 m thick layer to provide surface protection (drainage layer, growth layer and topsoil).

Table 5 Model Stages Summary

Stage	Description	Analyses Type	Graphical Summary (halo in green, core in brown)
1 to 6	Raise core and halo to 50m high in 10m lifts Bottom 10 m are assumed 'soaked'.	Plastic	
7	Place cover on the slope to 50 m height	Plastic	

Stage	Description	Analyses Type	Graphical Summary (halo in green, core in brown)
8 to 16	Raise NOEF to 140m height in 10m lifts. Bottom 10 m are assumed 'soaked'.	Plastic	
17	Place top halo at 140 m height	Plastic	
18	Place remaining cover	Plastic	

The results of the immediate settlement estimation are summarised in Table 6, and shown graphically in Figure 11 and Figure 12.

Table 6 Immediate Settlements Summary of NOEF Simplified Embankment

Height (m)	Total Immediate Settlements (mm)			
	Embankment Plateau		Batter Slope	
	From	To	From	To
50	590	500	500	40
140	2,100	1,940	1,940	40

7.4 Secondary Settlements

Secondary settlement, or creep, is time dependant and occurs at constant effective stress. Creep is affected by numerous factors, including time, effective stress ratio, and temperature. For this reason the estimation of creep settlement is generally based on empirical procedures.

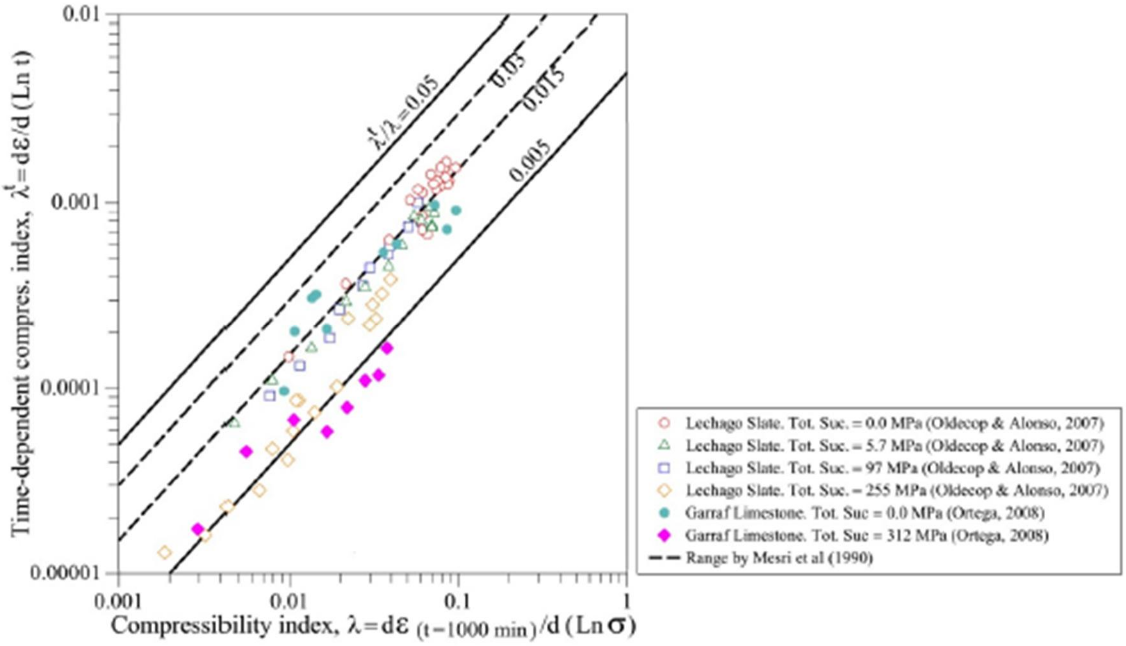
It is expected that both future and existing NOEF materials will be subject to creep settlements.

A number of large rock fill structures, such as dams or waste storage facilities, have been monitored over a number of years. Creep settlements have been reported by Kermani (2016); Hunter (2002) and Lade (2010) to occur mainly due to:

- Deformation of the rock particles.
- Deterioration of the contact points.
- Progressive breakage.
- Rearrangement of the particles to a more stable position.

For soils, studies demonstrated a relationship between short-term compressibility and creep. This was also supported by Alonso (2013) for rock fills, as shown on the Plate 1 (reproduced from Kermani, 2016).

Plate 1 Short Term Compressibility / Creep Relationship (Kermani, 2016)



Creep settlements could also be estimated using the following equation, relying on a number of case studies (Sower, 1965; Hunter, 2002).

$\delta_s = \alpha H log(t_2/t_1)$.

Where:

δ_s: secondary settlement (creep).
α: creep factor (noted λ in Plate 1)
H: thickness of material subject to secondary settlement.
t_2: final creep time, i.e. design life of 1,000 years.
t_1: initial creep time, i.e. assumed 1 year.

Referred literature suggest that α values for dumped rock fills range from 0.3 % to 1.5%. The creep factors assumed for the preliminary analyses are as summarised in Table 7.

Based on the Kermani (2016) approach would lead to lower creep settlements (α < 0.1%) than values discussed by Hunter (2002). The Kermani (2016) approach is however reliant on more advanced testing that is discussed in Section 9, and will not be considered at this stage.

It should be noted that the lift height influences directly the compaction level of the material, hence influences the elastic modulus of each layer. This in turn will influence the elastic settlements when the next lift is placed. One can notice that the plate load tests did not show remarkable differences in the measured moduli, Using the Kermani (2016) approach would allow the direct consideration of the lift height. The selection of the upper bound creep factor from the literature review is deemed for the consideration of the various lift heights and resulting compaction levels.

Table 7 Assumed Creep Factors Summary

Material	Creep Factor, α (%)		
	Lower Bound	Upper Bound	Best Estimate
NOEF Waste Rock	0.5	1.5	1.0

The total (elastic+creep) settlements were estimated for typical fill heights from the sections provided, shown graphically in Figure 13 to Figure 15.

The creep settlement starts at the end of the construction of the last lift. The construction time is not included in the simplified models. The settlement values at different dates can be obtained from the graphs and will be developed further during the detailed design.

The creep settlements at the end of the design life (i.e. 1,000 years) are summarised in Table 8.

Table 8 Estimates of Creep Settlements

Height (m)	Creep Settlements at End of Design Life (1,000 years) (mm)		
	Lower Bound	Upper Bound	Best Estimate
60	900	2,700	1,800
130	1,950	5,850	3,900
140	2,100	6,300	4,200

8 Cover system liner

The proposed NOEF cover system includes a Geosynthetic Liner (GSL). For the purposes of this assessment, the GSL is assumed to be a heavy duty bituminous geomembrane (BGM) liner such as the Coletanche © ES3 or ES4 products. This is favoured for its extremely low hydraulic permeability, mechanical strength and resistance to penetration from plant root systems. An example of the BGM GSL is included as Plate 2.

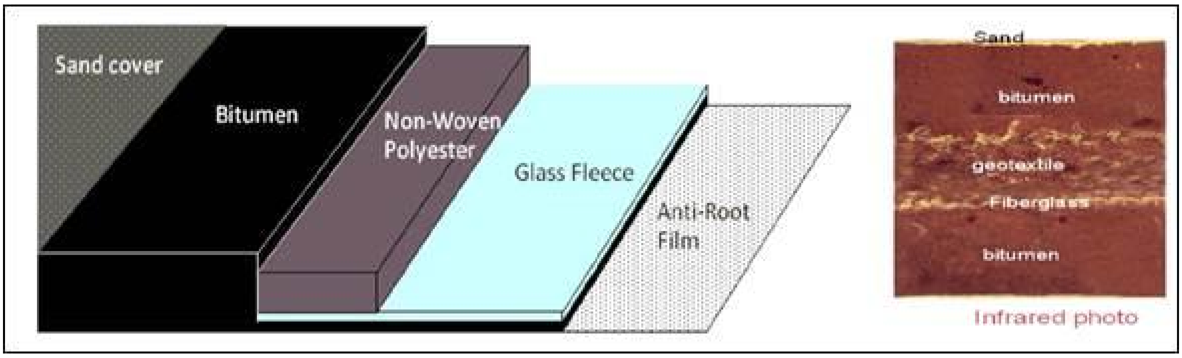


Plate 2 BGM Section Showing Component Layers

The technico-commercial documentation from Coletanche (design and application handbook) was reviewed, and provides a number of reference applications for the containment, dams, and tailings areas.

The technical data-sheets shows elongation at break over 60%, which is significantly higher than the values obtained for the clay liners.

9 Liner Strains

9.1 Strains due to elastic settlements

The strains due to elastic settlements were estimated, along both the top of the embankment and the batter slope, based on the results of the FE analyses carried out. In this case, the strains are due to the changes in fill height along the batter. The preliminary estimates indicate that the maximum strains are lower than 5%.

Variations in the characteristics of the fill, as well as variations in the compaction and lift height may result in further differential settlement.

For a fill of 60m, 130m and 140m height, the effects of the variation of modulus of elasticity is illustrated below:

Table 9 Estimates of Elastic Settlements

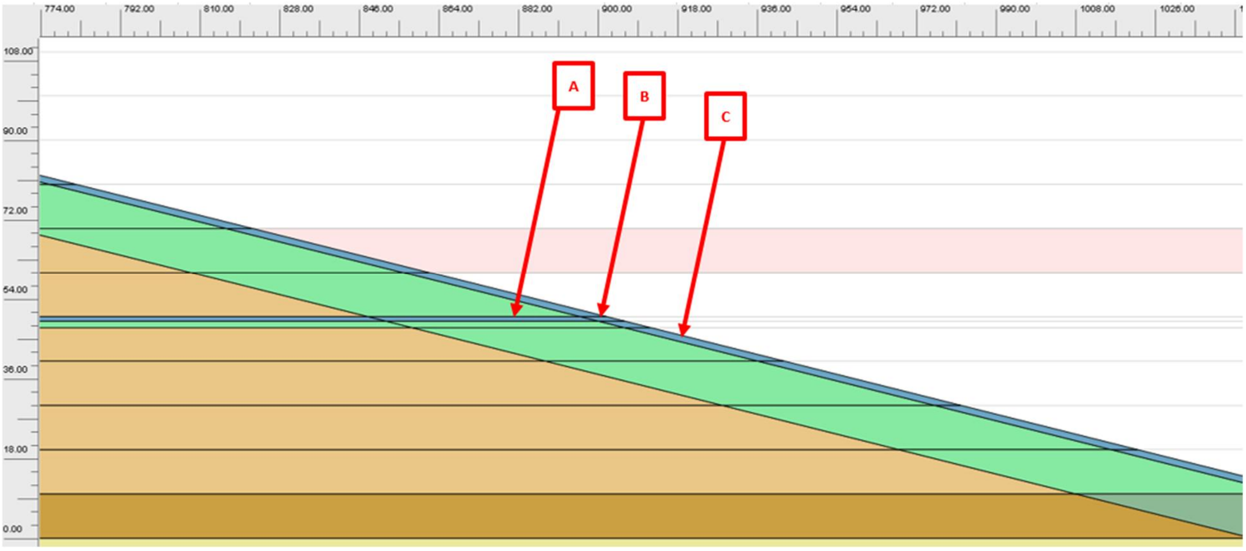
Embankment Height (m)	Elastic Settlements (immediate) (mm)		
	Lower Bound E=100 MPa	Upper Bound E=50 MPa	Best Estimate E=80 MPa
60	290	576	360
130	1580	3160	1975
140	1890	3770	2360

As the liner is installed over the halo zone, the elastic settlements will occur before the liner is installed.

However, strains may affect the liner at singular points, where the liner is installed in a completed portion of the slope, and the fill continues as the stockpile raises.

This is illustrated by considering 3 points situated at current stockpile level (approximately 50 m height of NOEF), where liner will be installed, and stockpile will continue to be raised, as illustrated in the next plate. Point A and C are set at 10m distance from point B.

Plate 3 Points for strain evaluation



As the fill is raised to 140m height, the elastic settlement induced at point A is 140mm, point B 120mm and point C 110mm. For a liner installed at this location, the induced strain would very small (less than 1%).

From the above analysis, the strain induced by elastic settlements is found to be negligible, with values lower than 1%.

9.2 Strain due to creep

When considering the creep settlement that will happen over time, the differential settlements will be primarily due to differences in stress history (the time between building the halo zone and the liner installation), but will also be due to the differences in lift thickness and fill properties. However, it is not realistic to consider that the different behaviours will occur over a short distance and consistently over the full fill height.

For illustration purposes, the following differential settlements are evaluated with “extreme”, unforeseen circumstances where the full height of the embankment will have one specific creep factor, and, in close proximity, there is a change in creep factor. Changes in fill height are assumed to follow the simplified design profile with slope batters of 4H:1V.

Table 10 Estimates of Differential Creep Settlements

Differential	Creep Settlements at End of Design Life (1,000 years)			
	140m Upper Bound – 130m Upper bound	**140m Best Estimate – 130m Best Estimate**	**140m Upper Bound – 130m Best Estimate**	**140m Upper Bound – 130m Lower Bound (*)**
Creep in mm	450	300	2,400	4,350

(*) only for sensitivity, unlikely scenario for illustration purposes

Even considering the extreme (and highly unlikely) scenario in the above preliminary analysis, it can be seen that the strains are less than 1%, well below the BGM strain limit of 60%. Therefore, the liner is not expected to be damaged from strain settlement.

10 Recommendations for further work

For the next stages of design, the following is recommended:

- Monitoring of settlements, for at least 12 months, of existing parts of the NOEF. This will assist with the selection of more accurate creep factors. This could be achieved by placing settlement markers (or settlement plates) on existing fill, at locations where the final elevation is already reached, preferably where the fill has recently been placed. Other monitoring points should be placed on "active" parts of the dump, to monitor the elastic settlements linked to a specific lift. The monitoring regime would include survey once a month over a minimum period of 12 months.
- Determine the stiffness moduli for the foundation materials via in-situ and laboratory testing.
- Carry out further FE analyses, on less simplified cross sections, with the updated stiffness moduli and creep factors.

11 Closure

We trust that the above meets with your requirements. Please do not hesitate to contact us should you have any queries.

Yours sincerely;

Reviewed by

On behalf of

Javier de la Rosa
Senior Geotechnical Engineer

Sylvie Bretelle
Principal Geotechnical Engineer

Appendix A

Figures

Content

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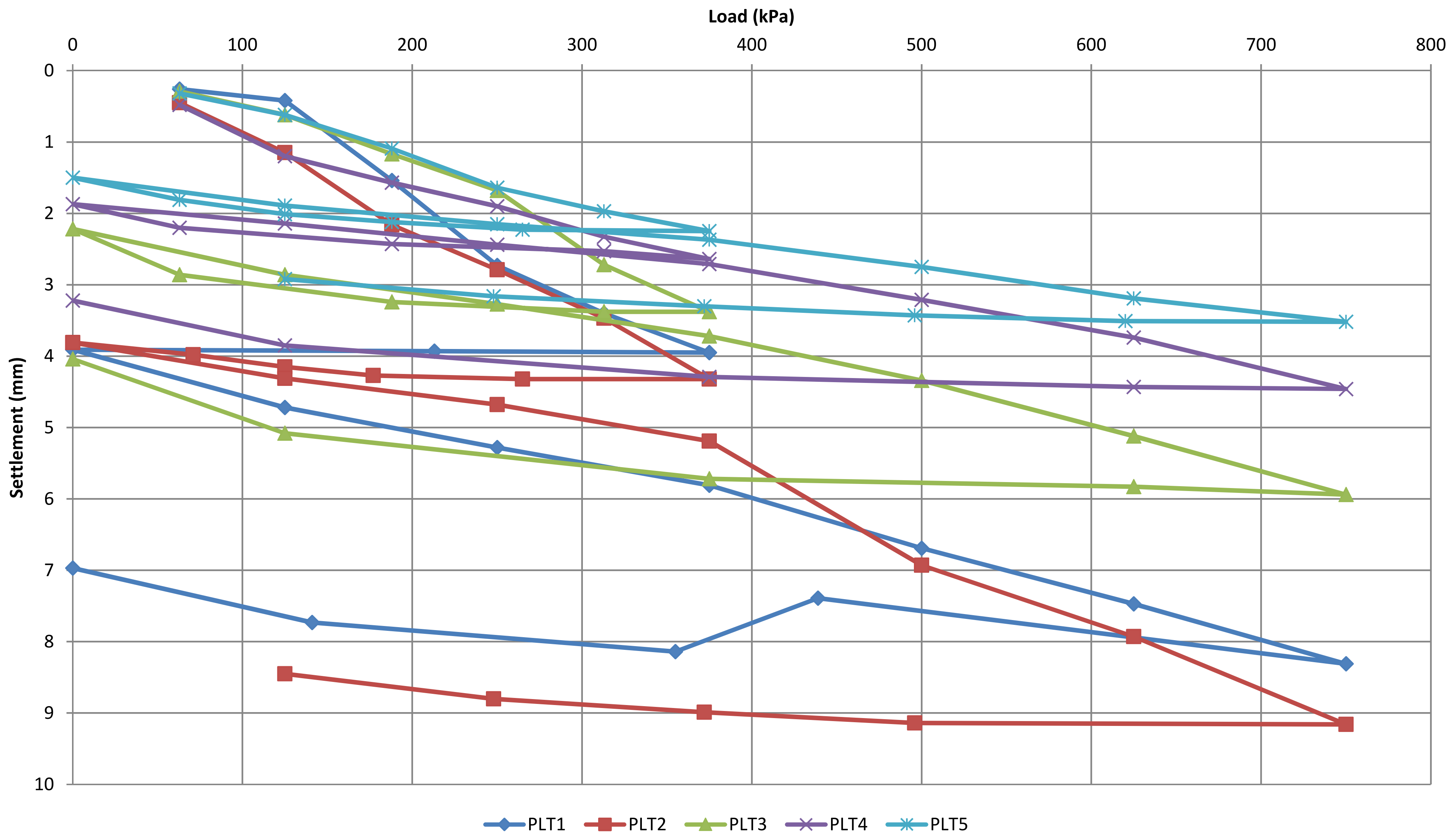
Drawn	Date	Job Number		Title	Client
JDLR	22/12/2017	61/32/17428	A4	Plate Load Test Location Plan	McArthur River Mine
Checked	Date				Project
SB	22/12/2017				MRM NOEF - Settlement Estimation
Revision	Date	Cad Reference			
A	22/12/2017				
This drawing should be read in conjunction with report number 61/32/17428/162370					Figure 1



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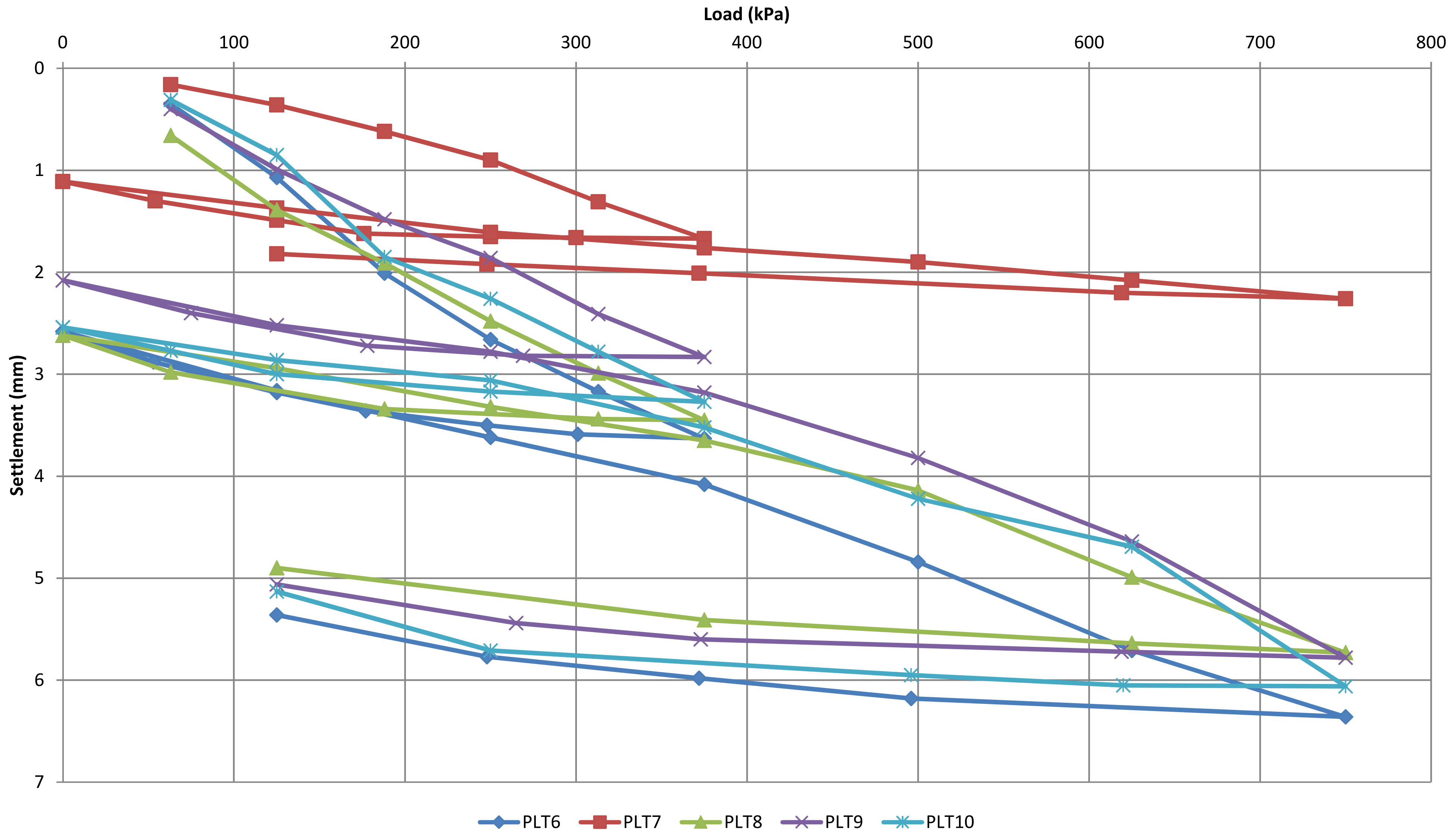
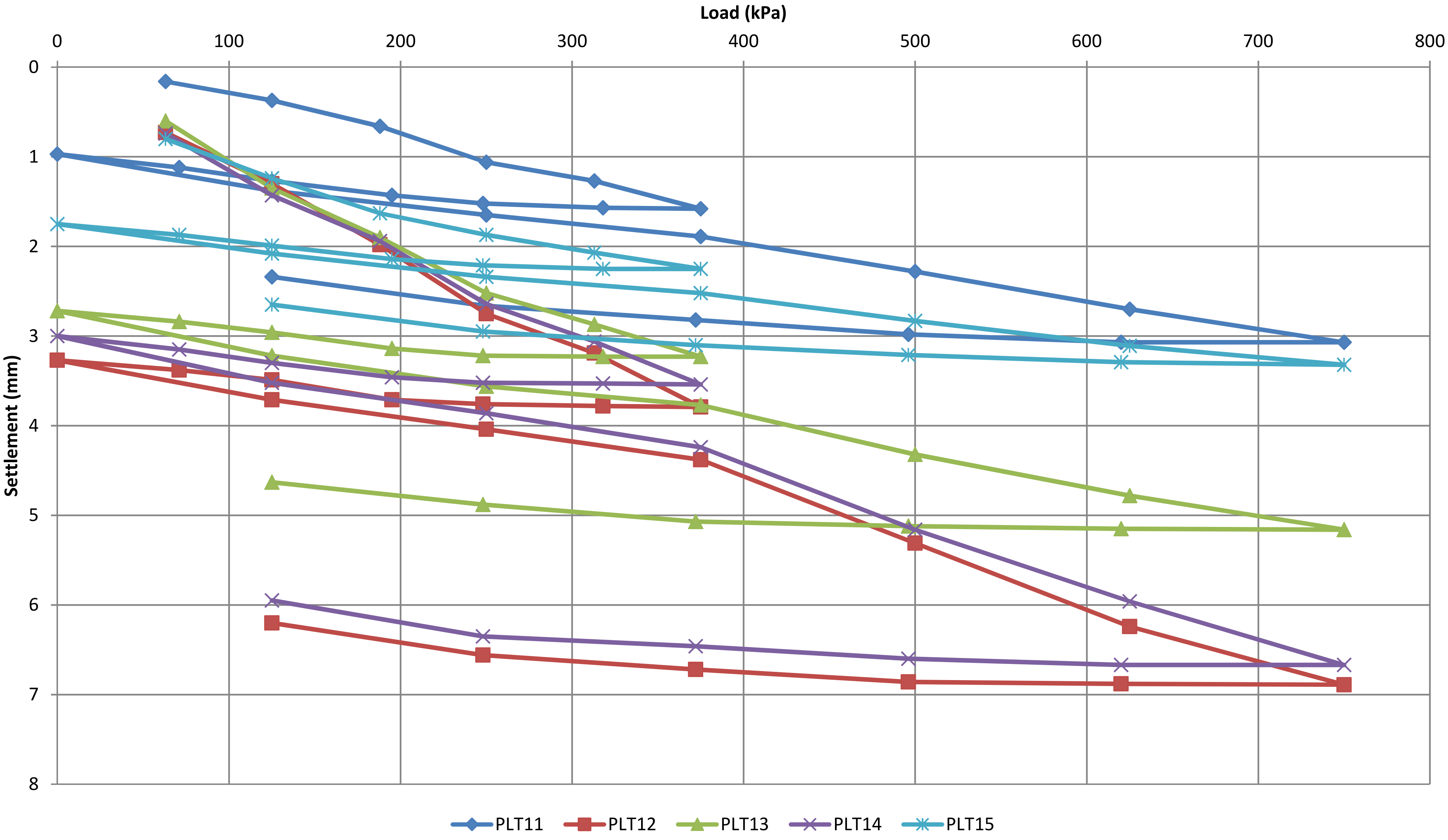
Drawn: JDLR	Date: 22/12/2017	Job Number: 61/32/17428	A4	Title: Plate Load Test Summary 1 of 5	Client: McArthur River Mine
Checked: SB	Date: 22/12/2017				Project: MRM NOEF - Settlement Estimation
Revision: A	Date: 22/12/2017	Cad Reference			Figure 2

This drawing should be read in conjunction with report number 61/32/17428/162370



GHD Pty Ltd GHD Pty Ltd 999 Hay Street Perth WA 6000 PO Box Y3106 Perth WA 6832 T 61 8 6222 8222 F61 8 6222 8555 E permail@ghd.com.au www.ghd.com.au	Drawn JDLR Date 22/12/2017 Checked SB Date 22/12/2017 Revision A Date 22/12/2017	Job Number 61/32/17428 Cad Reference	A4	Title Plate Load Test Summary 2 of 5	Client McArthur River Mine Project MRM NOEF - Settlement Estimation Figure 3

This drawing should be read in conjunction with report number 61/32/17428/162370



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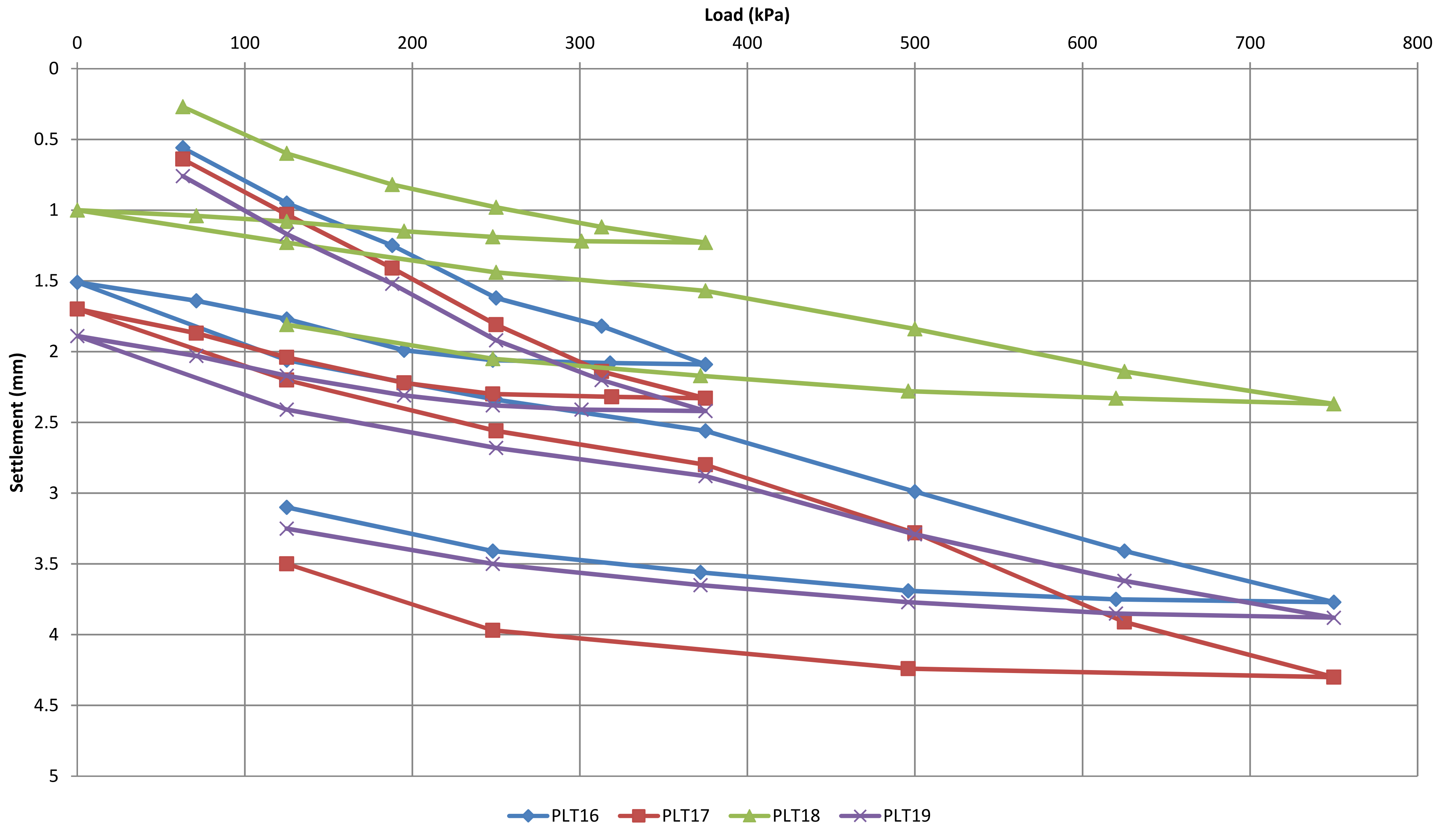
Drawn	JDLR	Date	22/12/2017	Job Number	61/32/17428	A4	Title	Plate Load Test Summary 3 of 5	Client	McArthur River Mine
Checked	SB	Date	22/12/2017						Project	MRM NOEF - Settlement Estimation
Revision	A	Date	22/12/2017	Cad Reference						Figure 4

This drawing should be read in conjunction with report number 61/32/17428/162370



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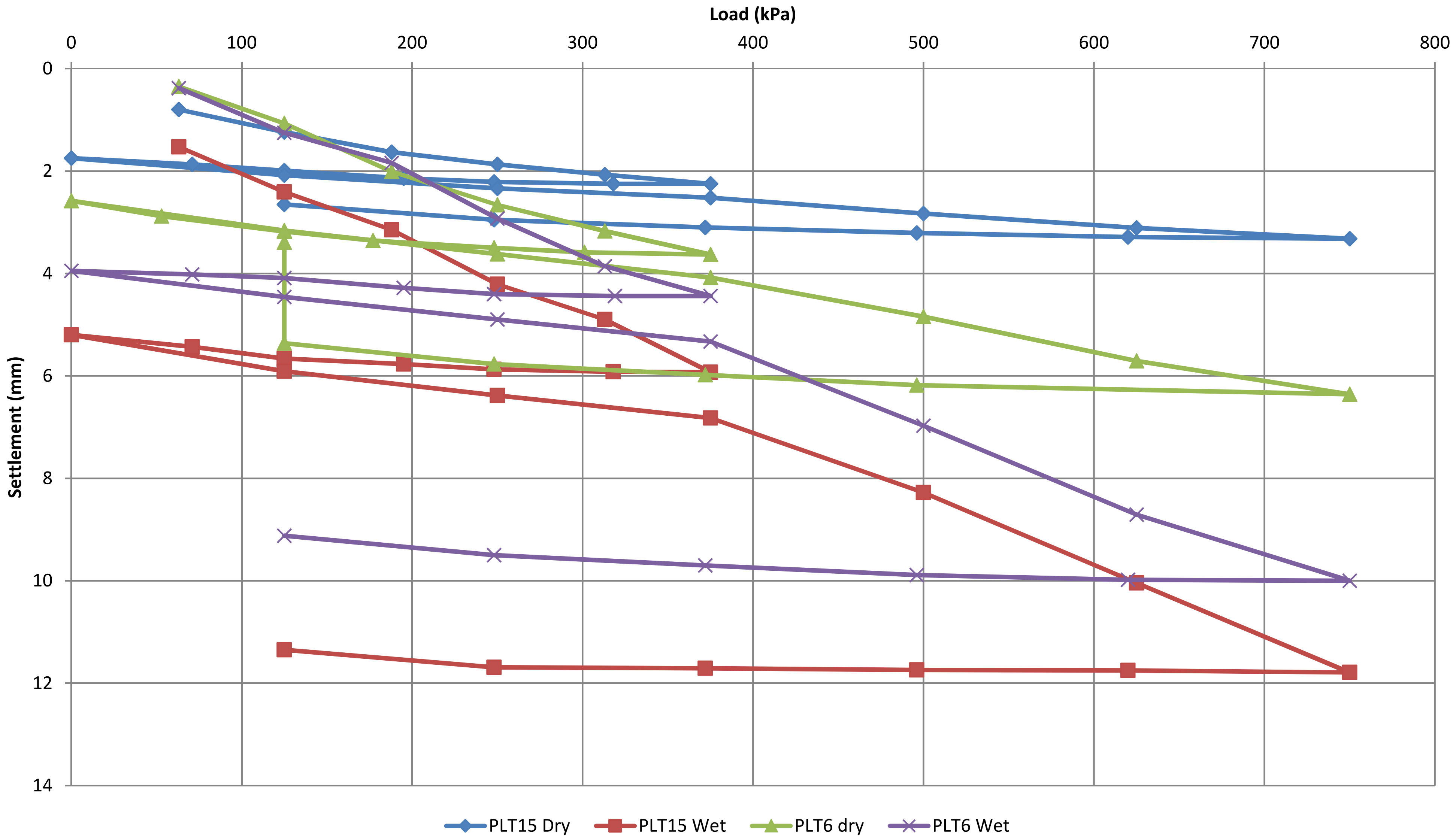
This drawing should be read in conjunction with report number 61/32/17428/162370



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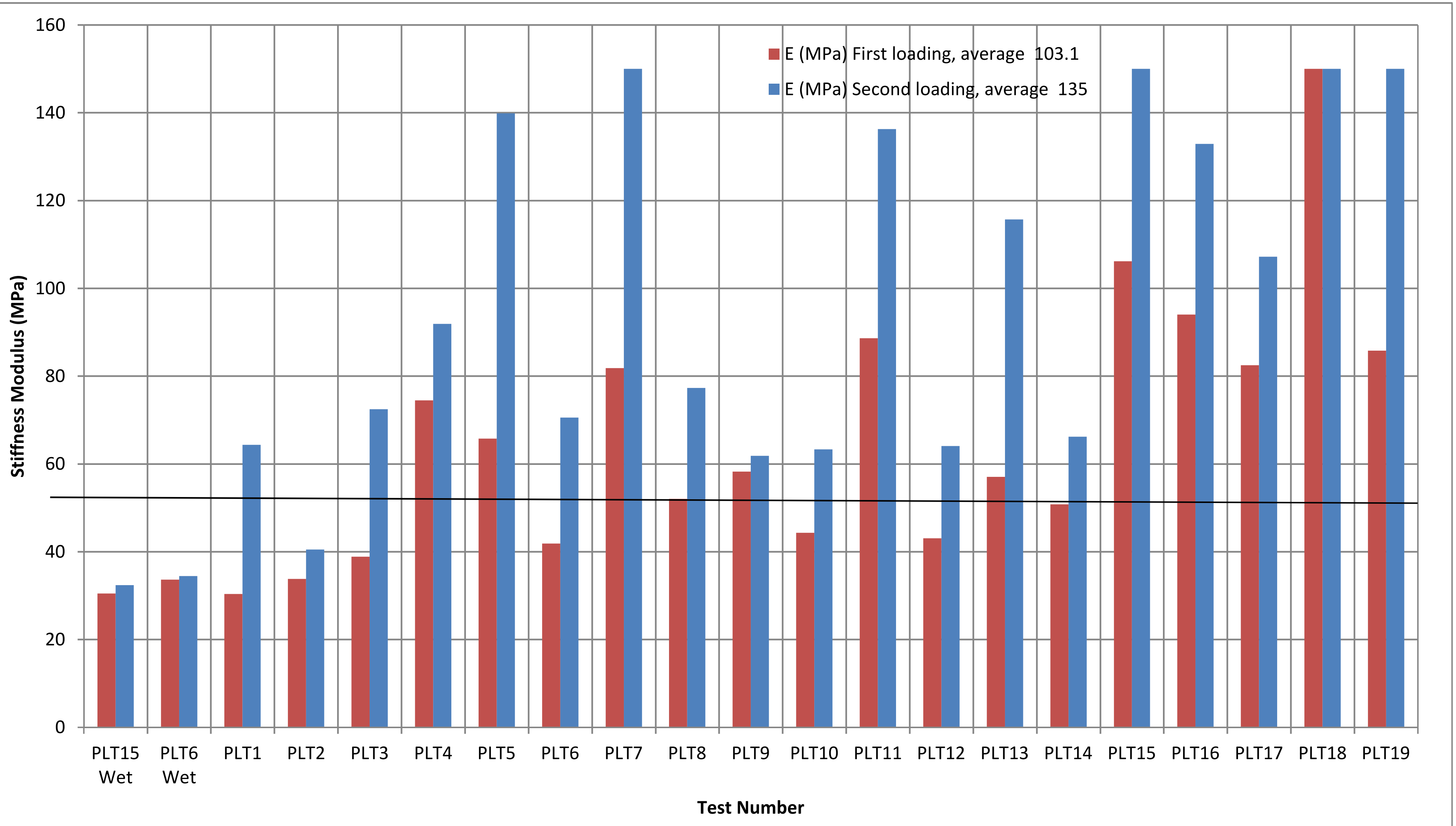
	Revision A	Date 22/12/2017	Cad Reference			
	This drawing should be read in conjunction with report number 61/32/17428/162370					Figure 6



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Checked: SB	Date: 22/12/2017		
Revision: A	Date: 22/12/2017	Cad Reference:	

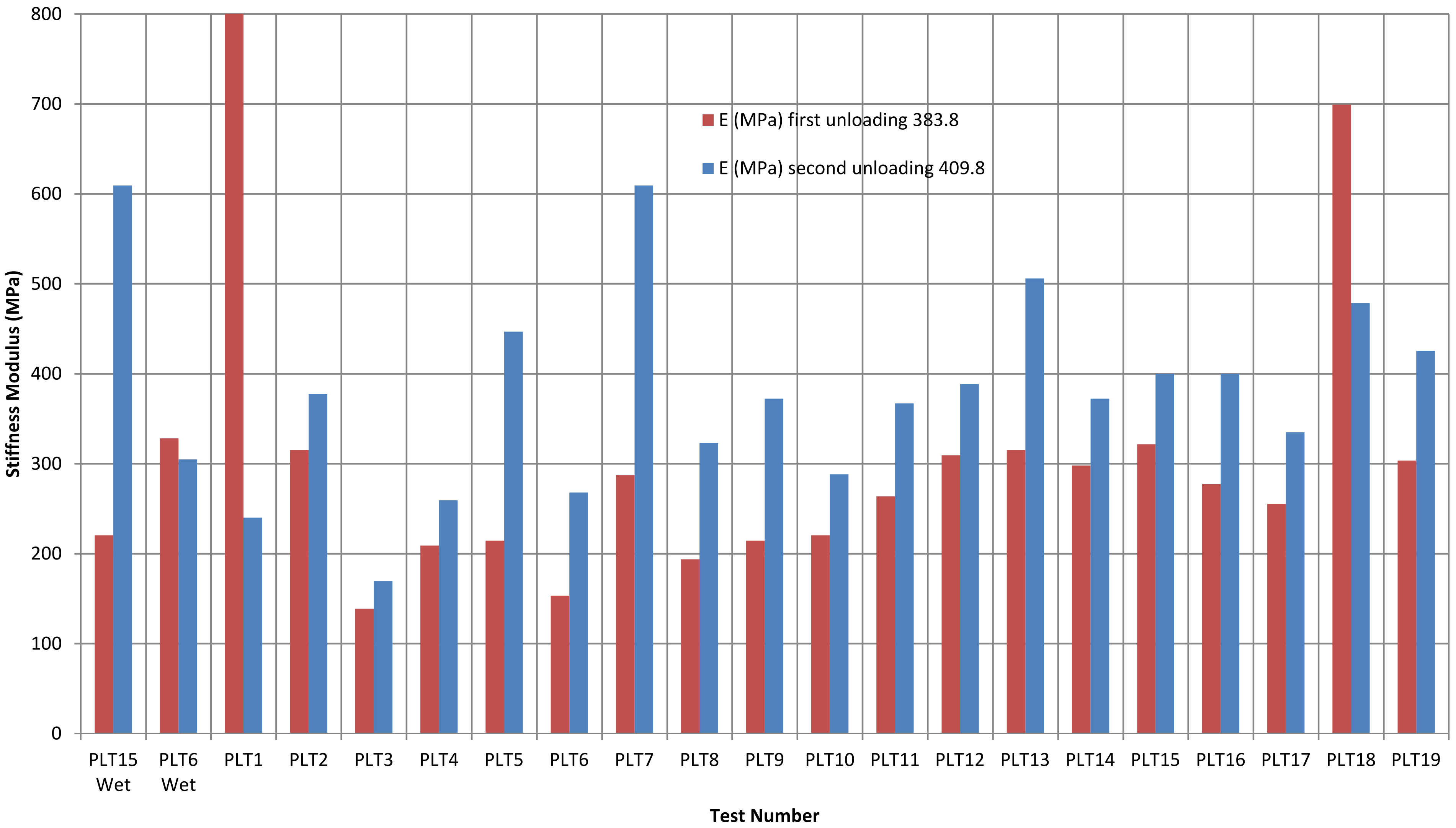
This drawing should be read in conjunction with report number 61/32/17428/162370

Title: Plate Load Tests Loading Cycles Summary

Client: McArthur River Mine

Project: MRM NOEF - Settlement Estimation

Figure 7



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Revision A	Date 22/12/2017	Cad Reference		Project MRM NOEF - Settlement Estimation

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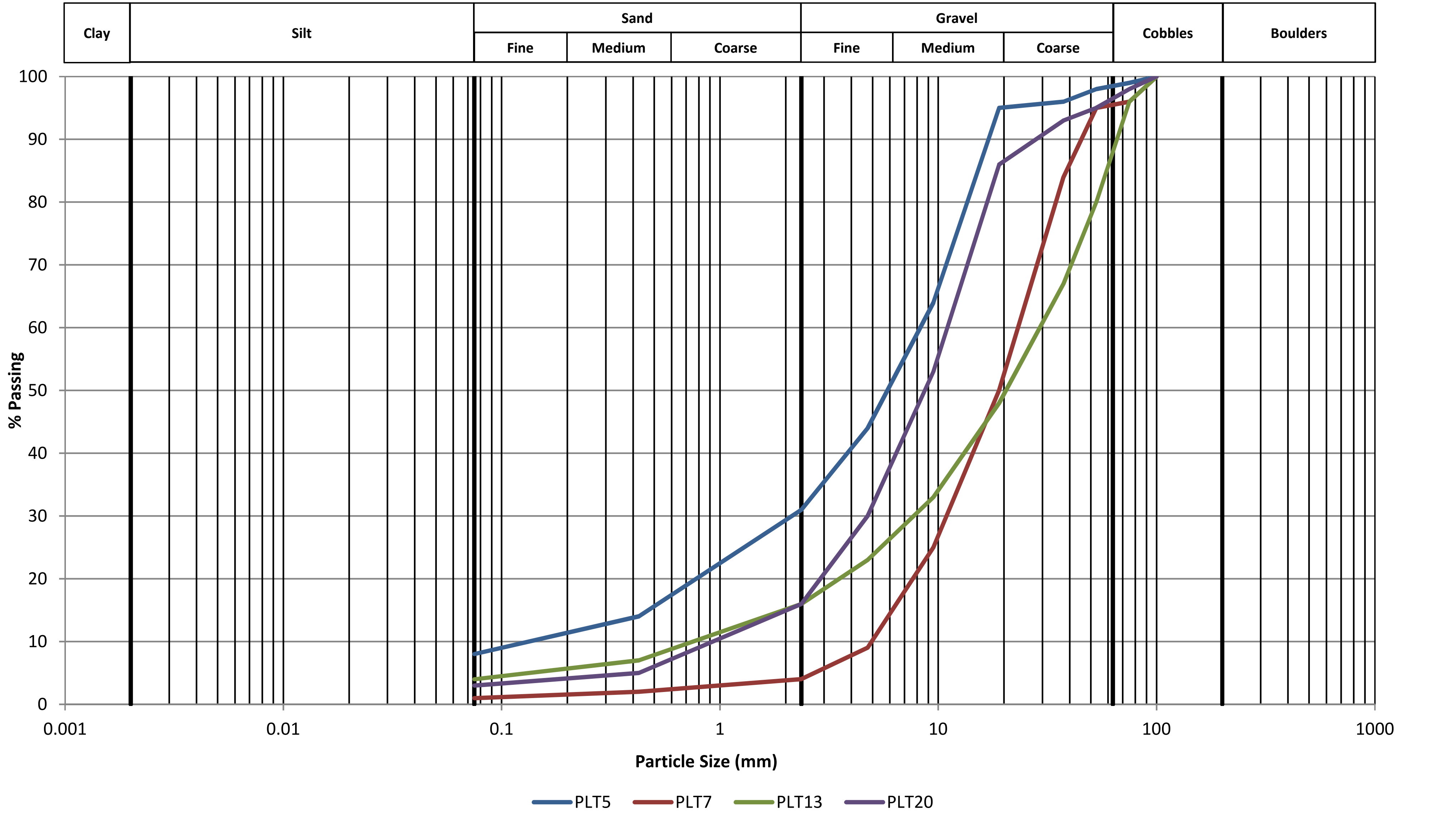
Figure 8



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	Checked SB	Date 22/12/2017				Project MRM NOEF - Settlement Estimation
	Revision A	Date 22/12/2017	Cad Reference			Figure 9
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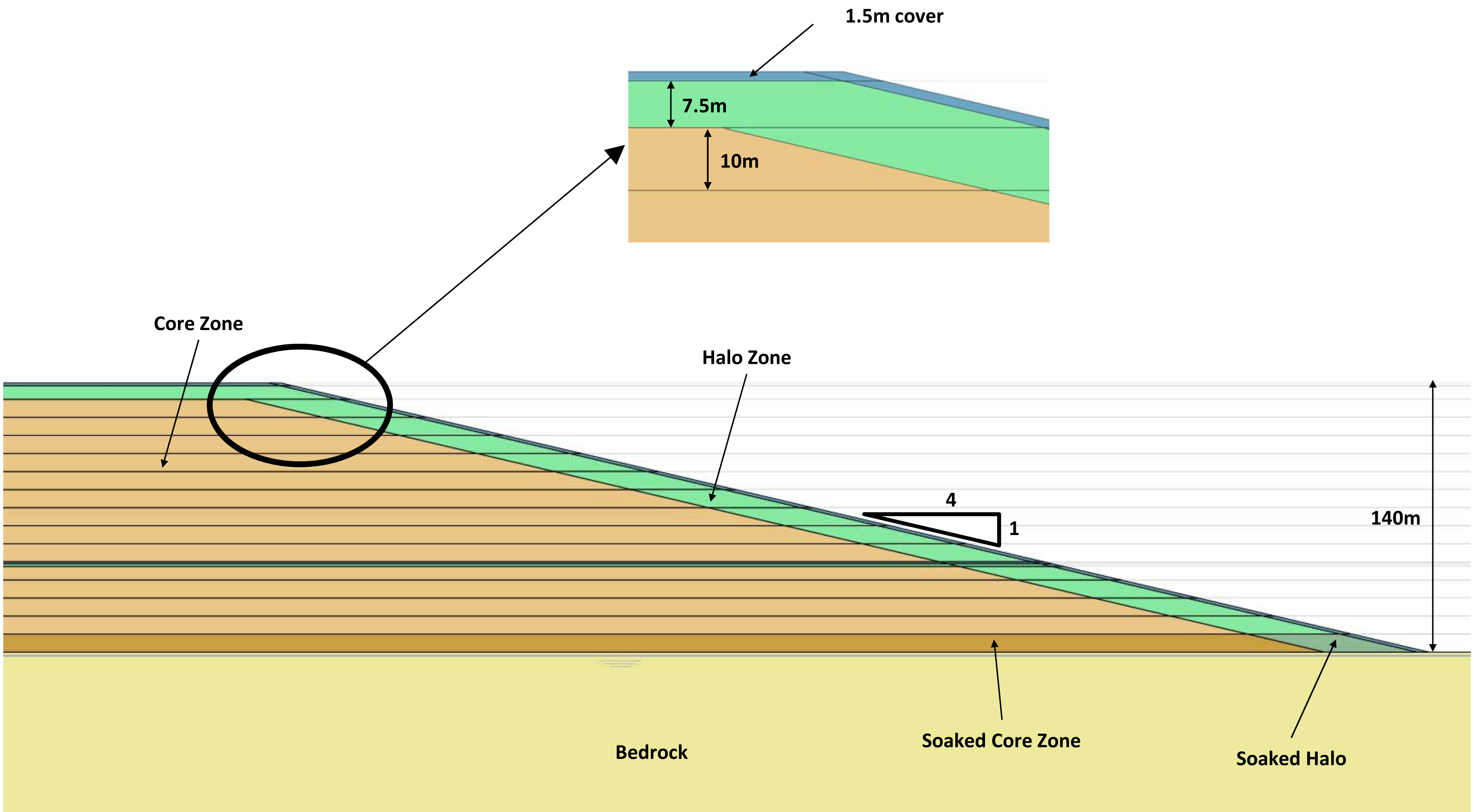

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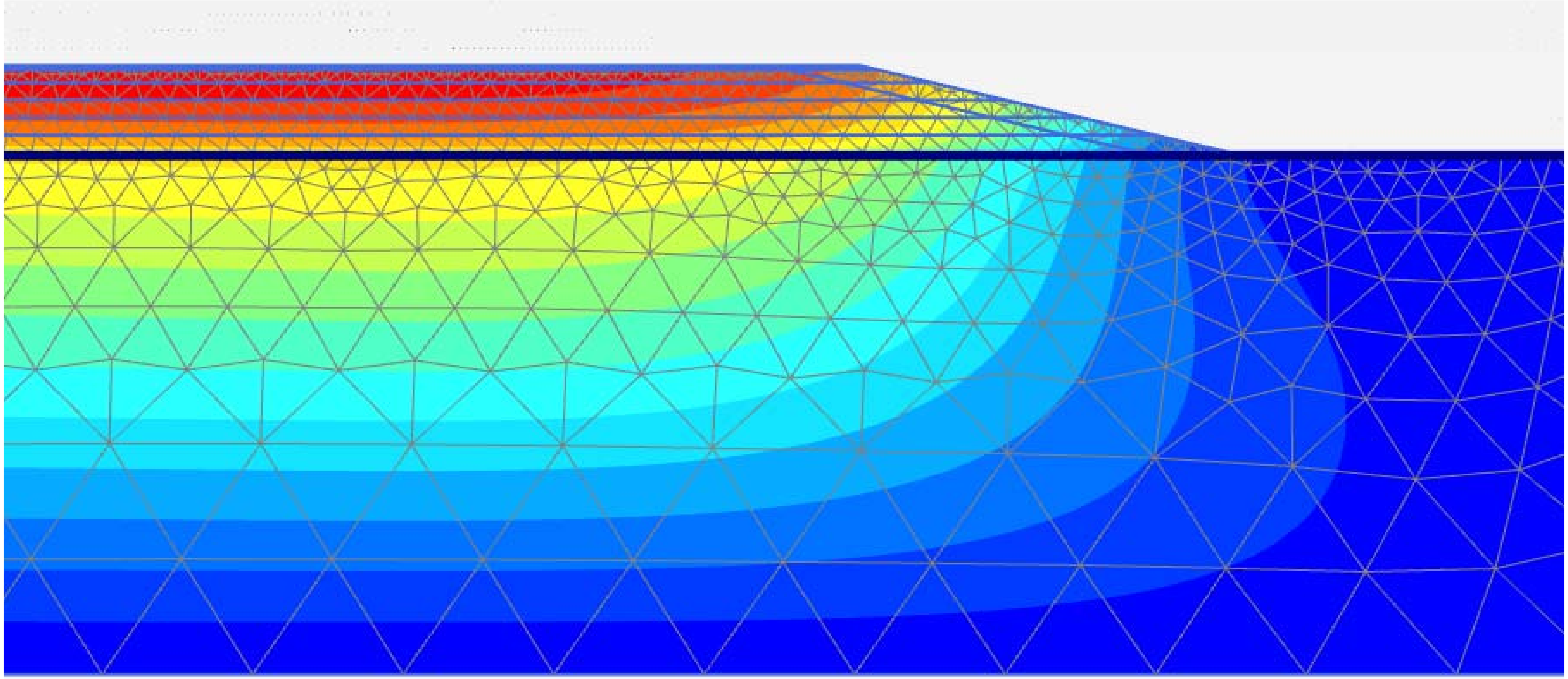
Drawn JDLR	Date 22/12/2017	Job Number 61/32/17428	A4	Title: Simplified Cross Section
Checked SB	Date 22/12/2017			Client: McArthur River Mine
Revision A	Date 22/12/2017	Cad Reference		Project: MRM NOEF - Settlement Estimation

This drawing should be read in conjunction with report number 61/32/17428/162370

Figure 10



maximum in red (0.6m)
minimum in blue (0)

Total displacements |u|

Maximum value = 0.5892 m (Element 5728 at Node 27980)

GHD Pty Ltd	Drawn: JDLR	Date: 22/12/2017	Job Number: 61/32/17428	A4	Title: Immediate Settlement Estimation Embankment at 50 m	Client: McArthur River Mine
	Checked: SB	Date: 22/12/2017				Project: MRM NOEF - Settlement Estimation
	Revision: A	Date: 22/12/2017	Cad Reference:			Figure 11

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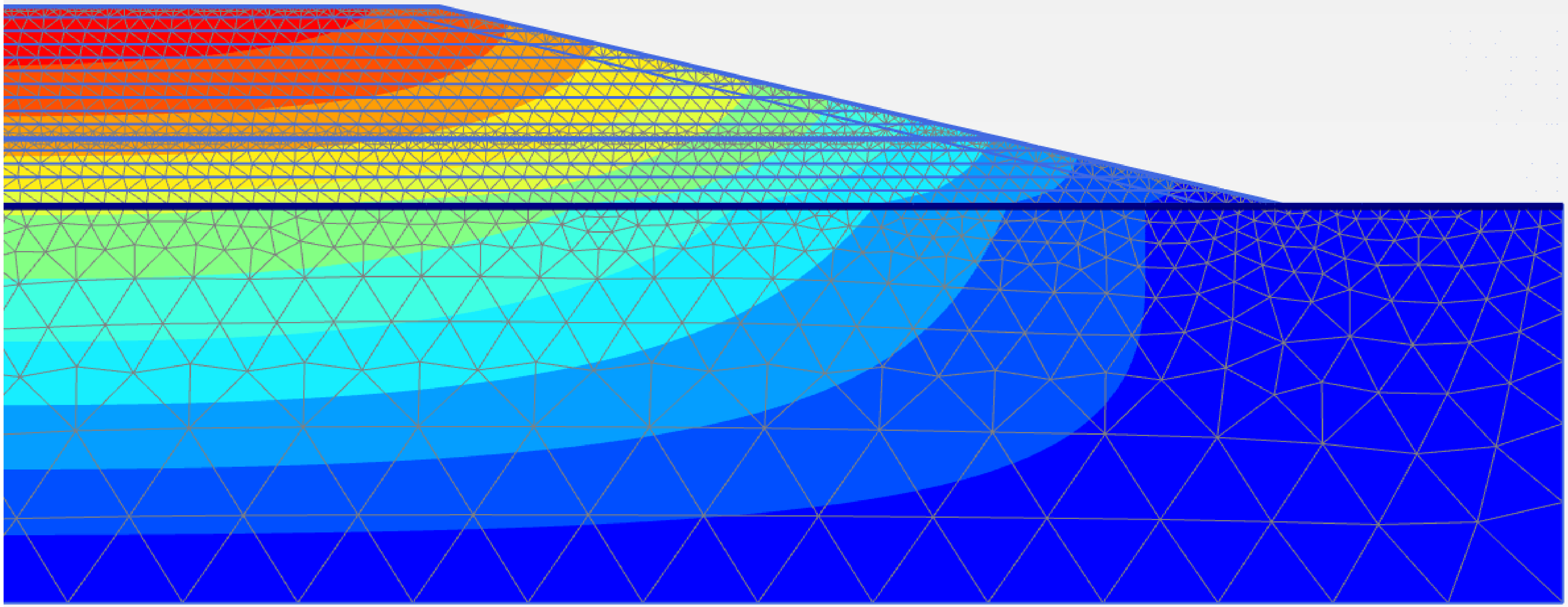

maximum in red (2.1m)
minimum in blue (0)

Total displacements |u|

Maximum value = 2.129 m (Element 464 at Node 125)

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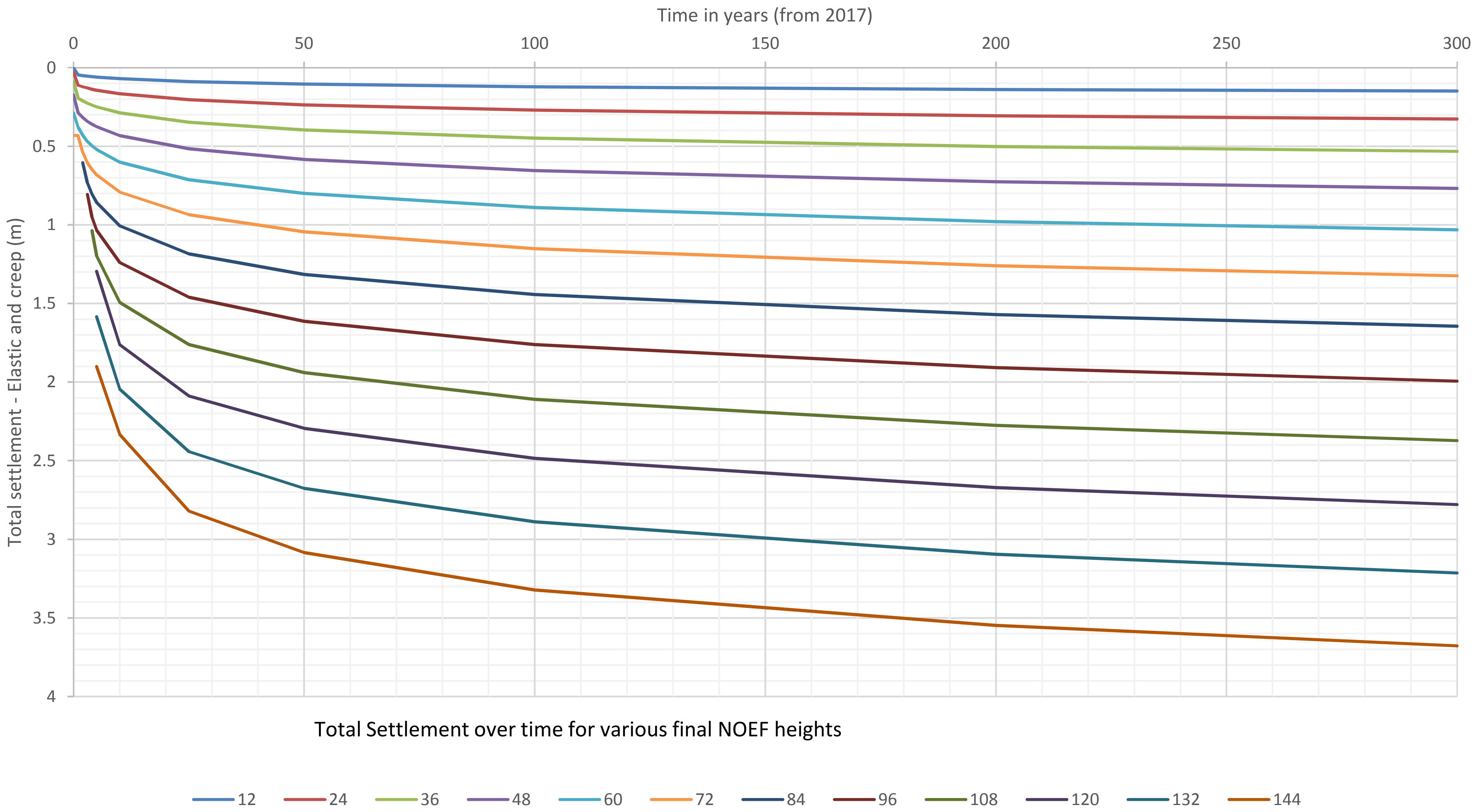
JDLR	22/12/2017	61/32/17428	A4	Immediate Settlement Estimation Embankment at 140 m	McArthur River Mine
Checked	Date				Project
SB	22/12/2017				MRM NOEF - Settlement Estimation
Revision	Date	Cad Reference			
A	22/12/2017				Figure 12

This drawing should be read in conjunction with report number 61/32/17428/162370



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Drawn: JDLR	Date: 22/12/2017	Job Number: 61/32/17428	A4	Title: Creep Settlements – Lower Bound
Checked: SB	Date: 22/12/2017			
Revision: A	Date: 22/12/2017	Cad Reference		

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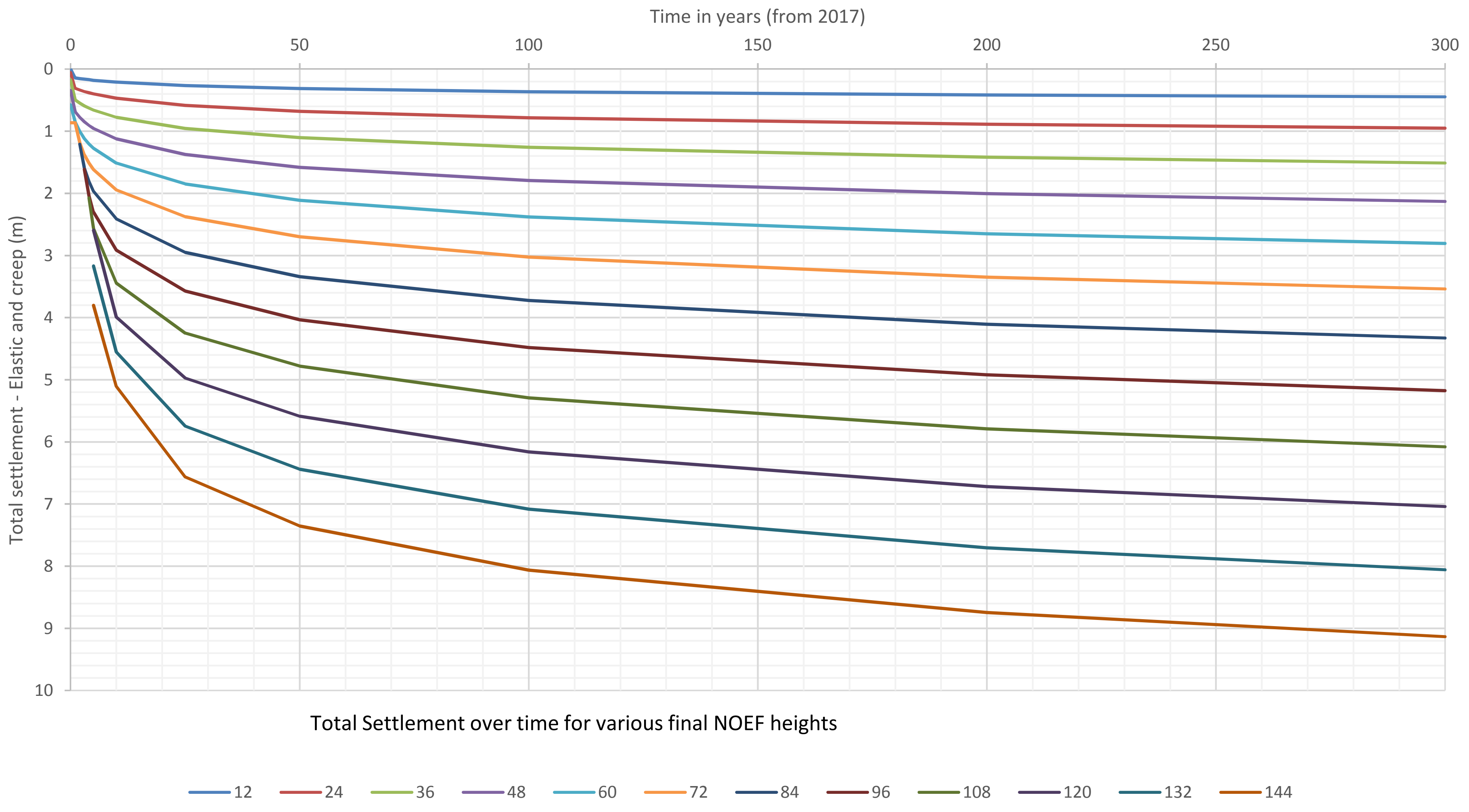
Client: McArthur River Mine

Project: MRM NOEF - Settlement Estimation

Figure 13



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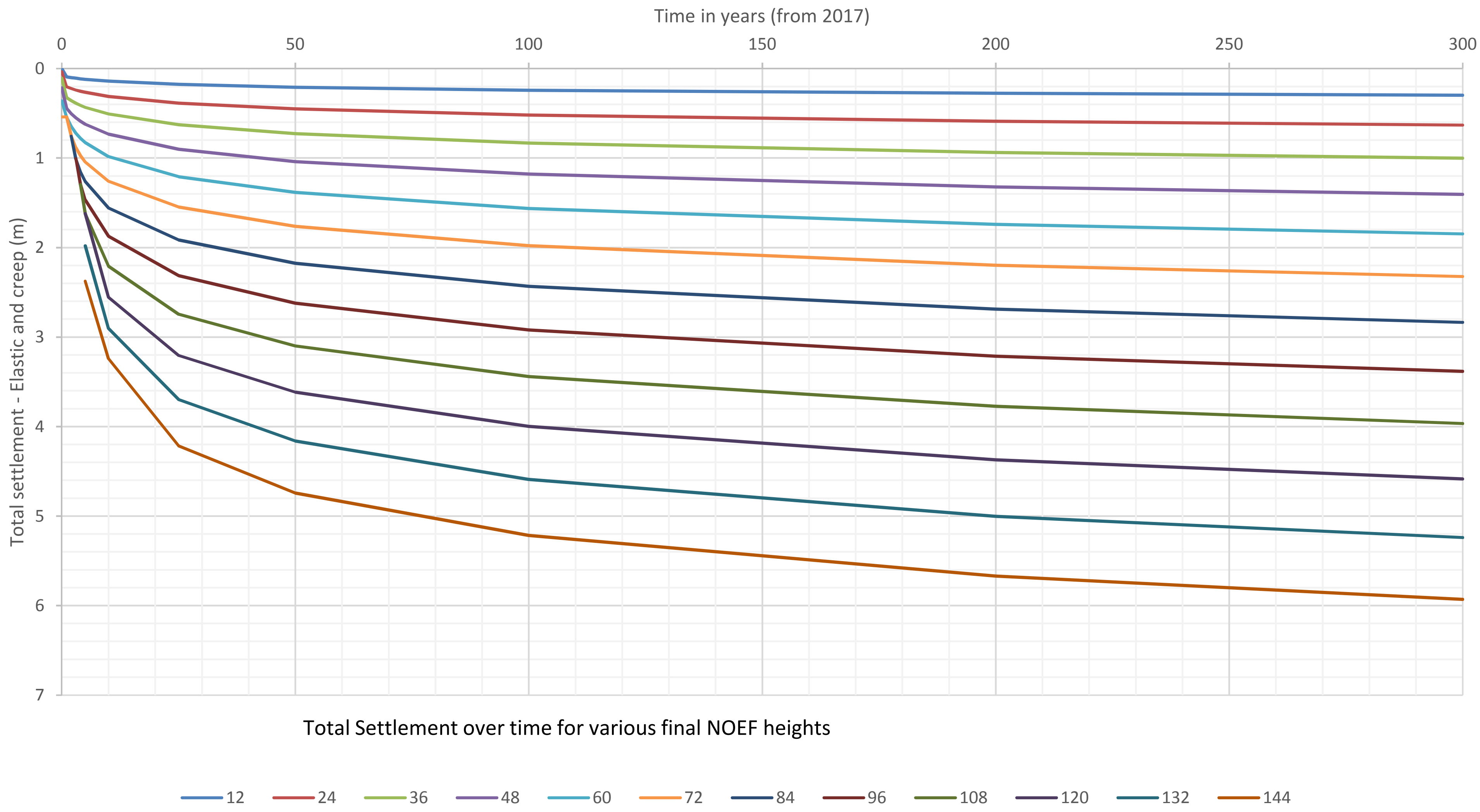
Drawn	Date	Job Number		Title
JDLR	22/12/2017	61/32/17428	A4	Creep Settlements – Upper Bound
Checked SB	Date 22/12/2017			
Revision A	Date 22/12/2017	Cad Reference		

This drawing should be read in conjunction with report number 61/32/17428/162370

Client: McArthur River Mine

Project: MRM NOEF - Settlement Estimation

Figure 14



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	Checked: SB	Date: 22/12/2017				Project: MRM NOEF - Settlement Estimation
	Revision: A	Date: 22/12/2017	Cad Reference			Figure 15
	This drawing should be read in conjunction with report number 61/32/17428/162370					

Appendix C – UQ Geotechnical reports

McARTHUR RIVER MINE
SHEAR STRENGTH OF COMPACTED CLAY

Report prepared by

Professor David John Williams, CPEng, RPEQ, FIEAust, MAusIMM

School of Civil Engineering
The University of Queensland, Brisbane QLD 4072

June 2017

1 INTRODUCTION

Professor David John Williams was commissioned by Mr Jamie Hacker of McArthur River Mine (MRM) to report on the results and interpretation of direct shear strength testing of MRM compacted clay carried out at The University of Queensland (UQ). This report presents the testing methodology, selected test results and the interpretation of the shear strength parameters for MRM compacted clay. It also compares the shear strength results for MRM compacted clay with previous shear strength data from large direct shear tests carried out by UQ and the University of Newcastle (UN), and with the results of triaxial tests carried out by Trilab on MRM compacted clay. From the current test results and comparisons, shear strength parameters for the MRM compacted clay are recommended for use in design.

2 TESTING METHODOLOGY

The shear strength of compacted clays may be determined in a laboratory direct shear box (see schematic in Figure 1), with the rigid box providing containment for the specimen. Compacted clay is “stiff” (and over-consolidated by virtue of the compaction process), and would be expected to exhibit a “peak” shear strength at low shear strain. The shear strength would reduce to an “ultimate” shear strength at moderate shear strain, and possibly reduce further to a “residual” shear strength at very large shear strains if a residual (often slickensided) shear plane forms. Not all clayey soils develop a residual shear strength.

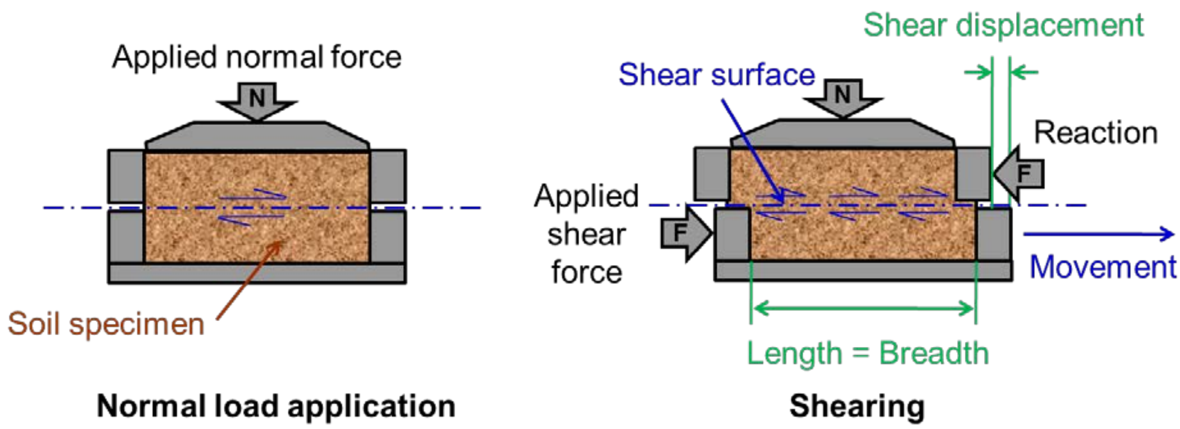

Figure 1 Schematic of a direct shear strength test

The peak and ultimate shear strengths may readily be determined in a direct shear box test at shear strains of up 10%. The development of a residual shear plane may be tested by reversal and re-shearing in a direct shear box test. In the first shearing, the peak and ultimate shear strengths are determined. The normal load is then removed and the box wound back to its starting position. The normal load is then re-applied and the sample re-sheared. Several reversals are applied until there is no further reduction of shear strength.

Separate direct shear box tests are generally carried out on specimens of the same material under normal stresses that increase in a geometric series; that is, doubling the normal stress at each successive stage (see Figure 2). Alternatively, multi-stage testing could be carried out on the same specimen. However, multi-stage testing limits the shear strain that can be applied in each of the three stages to about 3%, which may not be sufficient to achieve shear failure, and earlier stages may affect the shear strength achieved in later stages. Single-stage testing was adopted.

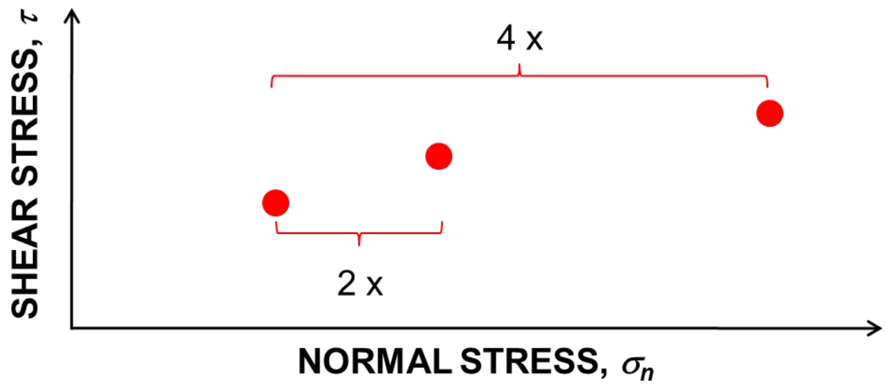

Figure 2 Results of three-stage laboratory direct shear strength testing

2.1 Sample State and Preparation

A representative clay sample was delivered by MRM from site. The sample had an average gravimetric moisture content of about 13.6%, although the sample was received in clods and the moisture was not evenly distributed. It was found on wash sieving to have 88% passing the 0.075 mm sieve (fines), indicating a Sandy Silty CLAY. The Liquid and Plastic Limits were found to be 36.2% and 14.9%, respectively, giving a Plasticity Index of 21.3% and placing the gravimetric moisture content at about the Plastic Limit of the clay.

2.2 Direct Shear Testing

The direct shear testing was carried out in a 100 mm by 100 mm shear box. The clay was prepared to a target gravimetric moisture content of 13.6% and compacted in the direct shear box to a dry density of 1.85 t/m^3 (representing the field compaction specification adopted by MRM), to a specimen height of about 30 mm.

Single-stage testing was carried out at nominal initial normal stresses of 50 kPa, 100 kPa and 200 kPa, representing shallow depths of burial in the North Overburden Emplacement Facility (NOEF) of about 2.8 m, 5.6 m and 11.1 m, respectively (assuming a waste rock wet unit weight of 18 kN/m^3). The testing was carried out at the compacted gravimetric moisture content, not in a water bath, since the compacted clay within the NOEF will not be inundated sufficiently long to cause it to wet-up. The following series of direct shear box test was carried out:

- **Rapid tests (intended to simulate “undrained” behaviour):**
  - Each normal stress was applied and the specimen immediately sheared at 1 mm/min (10 min of shearing).
  - The normal stress was removed and the shear box was wound to its starting position, before the normal stress was re-applied and the specimen immediately re-sheared.
  - Repeat reversals were applied until the shear strength stabilised.

- **Intermediate rate tests (likely "drained" behaviour):**
    - Each normal stress was applied and the specimen allowed to compress overnight before shearing at 0.1 mm/min (100 min of shearing).
    - The normal stress was removed and the shear box was wound to its starting position, before the normal stress was re-applied and the specimen immediately re-sheared.
    - Repeat reversals were applied until the shear strength stabilised.
- **Slow tests (to simulate "drained" behaviour):**
    - Each normal stress was applied and the specimen allowed to compress overnight before shearing at 0.01 mm/min (1,000 min or 16.6 hours of shearing).
    - The normal stress was removed and the shear box was wound to its starting position, before the normal stress was re-applied and the specimen immediately re-sheared.
    - Repeat reversals were applied until the shear strength stabilised.

The initial dry density to which all test specimens were compacted was 1.85 t/m^3. The compression under the applied normal stresses, the final specimen dry densities, and the initial and gravimetric moisture contents of the test specimens are summarised in Table 1.

Table 1 Summary of direct shear strength tests carried out at UQ on MRM compacted clay

SHEARNG RATE	NOMINAL NORMAL STRESS (kPa)	COMPRESSION (mm)	FINAL DRY DENSITY (t/m^3)	FINAL GRAVIMETRIC MOISTURE CONTENT (%)
Rapid	50	0.23	1.86	14.3
	100	0.82	1.90	14.4
	200	1.09	1.92	13.7
Intermediate	50	0.27	1.87	12.8
	100	0.58	1.89	13.1
	200	0.76	1.90	12.8
Slow	50	0.29	1.87	13.2
	100	0.58	1.89	12.6
	200	0.60	1.89	12.9
AVERAGES	**117**	**0.58**	**1.89**	**13.3**

3 TEST RESULTS

Selected test results are presented in the following sections, highlighting the effect of shear reversals, the effect of shearing rate, the shear strength/normal stress ratios, and providing the shear strength envelopes obtained. Both the applied normal stresses and the measured shear stresses were corrected for reducing shear area during the course of the tests.

3.1 Effect of Shear Reversals

The effect of shear reversals for the series of rapid shearing tests are shown in Figures 3 and 4. Similar behaviour was observed for the intermediate and slow shearing rate tests. As shown in Figure 3, the initial rapid shearing produced the expected peak shear strength at small shear displacement, followed by a reduction to an ultimate shear strength at moderate shear displacement. The peak shear strength was most pronounced under the smallest normal stress of 50 kPa, and became somewhat less pronounced with increasing applied normal stress as the effective preconsolidation stress induced by compaction was progressively overcome. The two shear reversals produced an ultimate shear strength similar to that obtained on initial shearing, and at about the same shear displacement. There was no apparent drop-off towards a residual shear strength, and no residual plane was observed on removing the specimens from the shear box. Hence, no further shear reversals were applied.

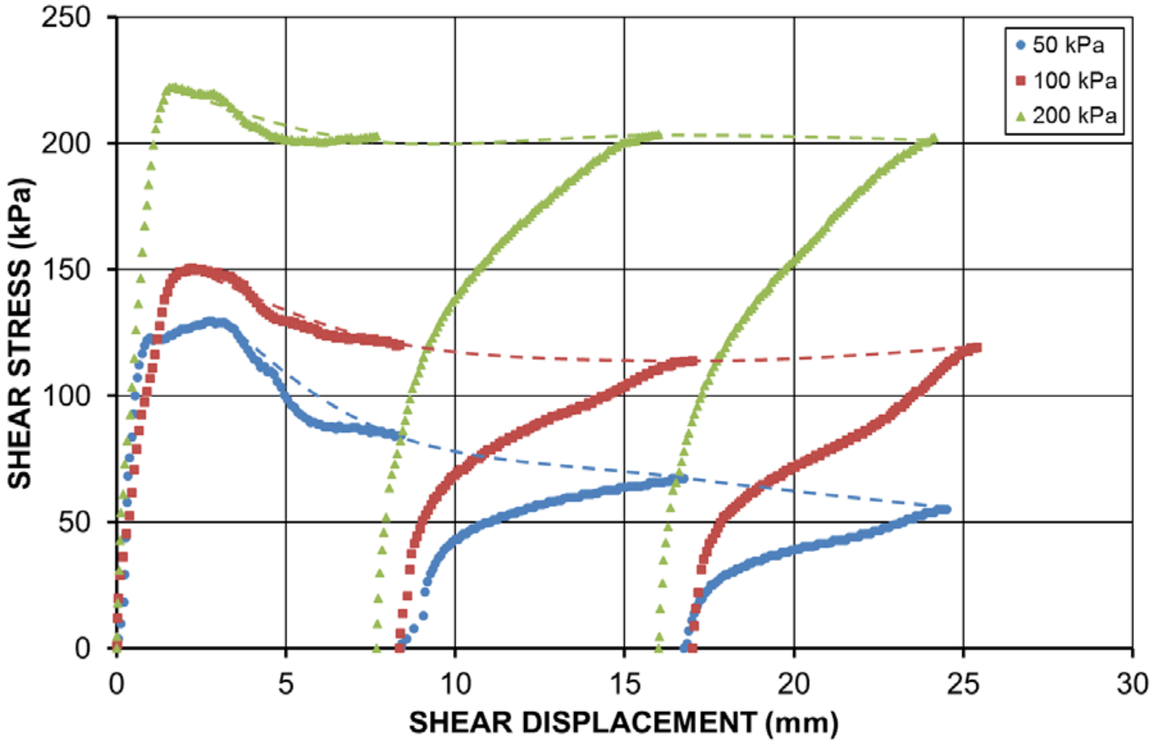

Figure 3 Shear stress versus shear displacement on rapid shearing reversals

As shown in Figure 4, the initial rapid shearing resulted in contraction of the specimens, with decreasing contraction with increasing applied normal stress. On second and third rapid shearing, minor contraction was observed under 50 kPa applied normal stress, while dilation was observed under 100 kPa and 200 kPa applied normal stress.

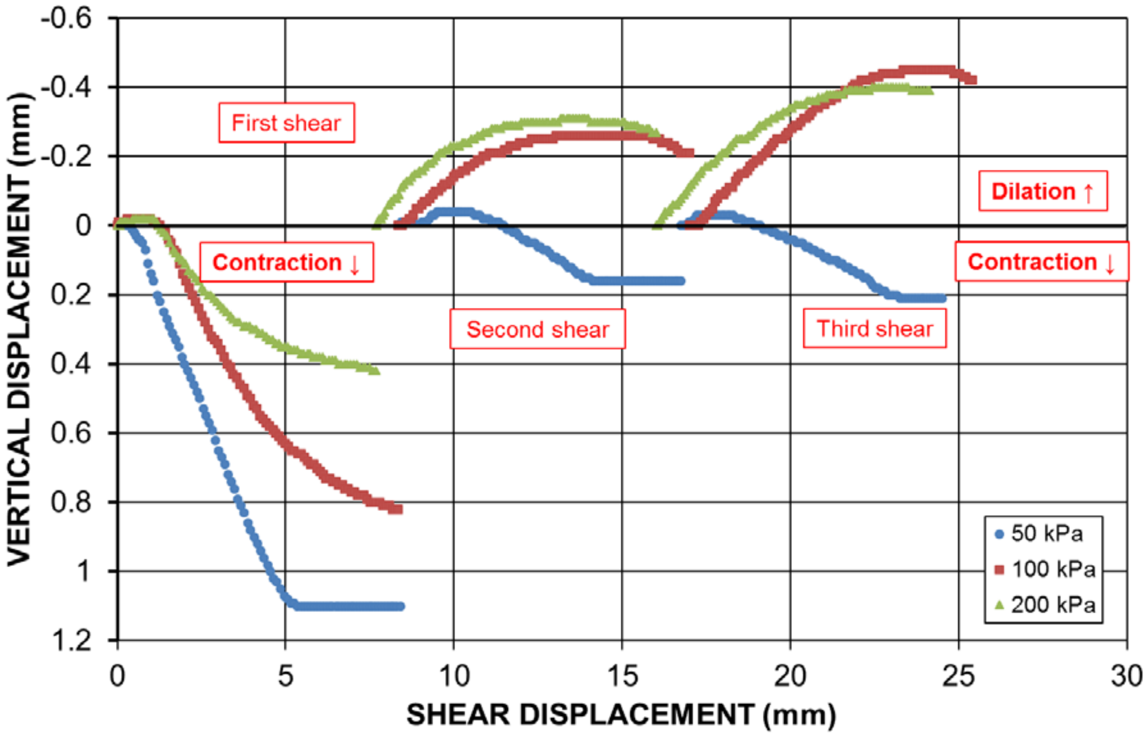

Figure 4 Shear stress versus shear displacement on rapid shearing reversals

3.2 Effect of Shearing Rate

The effect of shearing rate (rapid shearing at 1 mm/min, intermediate shearing at 0.1 mm/min and slow shearing at 0.01 mm/min) is shown in Figures 5 and 6 to be small, both for the peak and ultimate shear strengths. Overall, the straight line-of-best-fit peak shear strength (first shearing) envelopes are very similar over the two orders of magnitude variation in shearing rate. The peak friction angles increase slightly with increasing shearing rate, while the apparent cohesion intercepts decrease slightly. The straight line-of-best-fit ultimate shear strength envelopes obtained for the first, second and third shearing tests, are very similar over the two orders of magnitude variation in shearing rate. The ultimate friction angles are very similar, while the apparent cohesion intercepts vary slightly. The peak shear strength envelopes indicate about three times the apparent cohesion intercepts and slightly lower friction angles compared with the ultimate shear strength envelopes. The similarity and largely frictional behaviour of all shear strength envelopes, irrespective of the applied shear rate, imply drained behaviour at all shearing rates. This can be explained by the relatively small size of the test specimens, and the very small drainage path lengths associated with the induced shear plane.

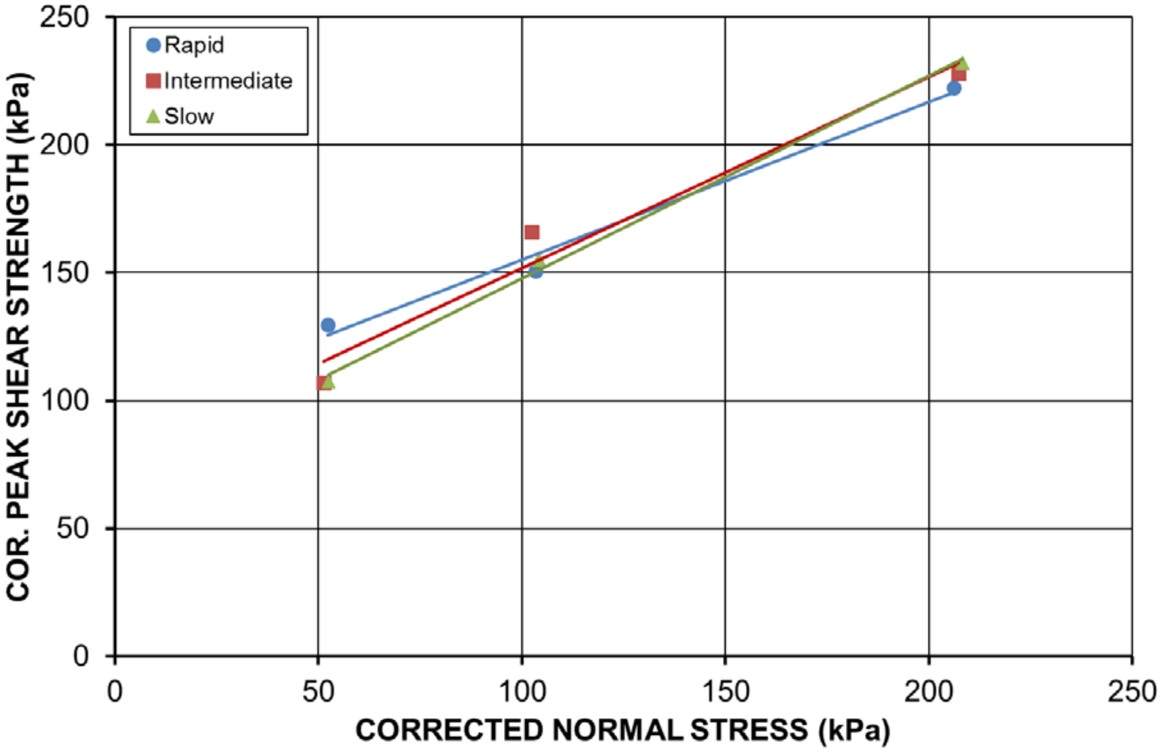

Figure 5 Peak shear strength envelopes

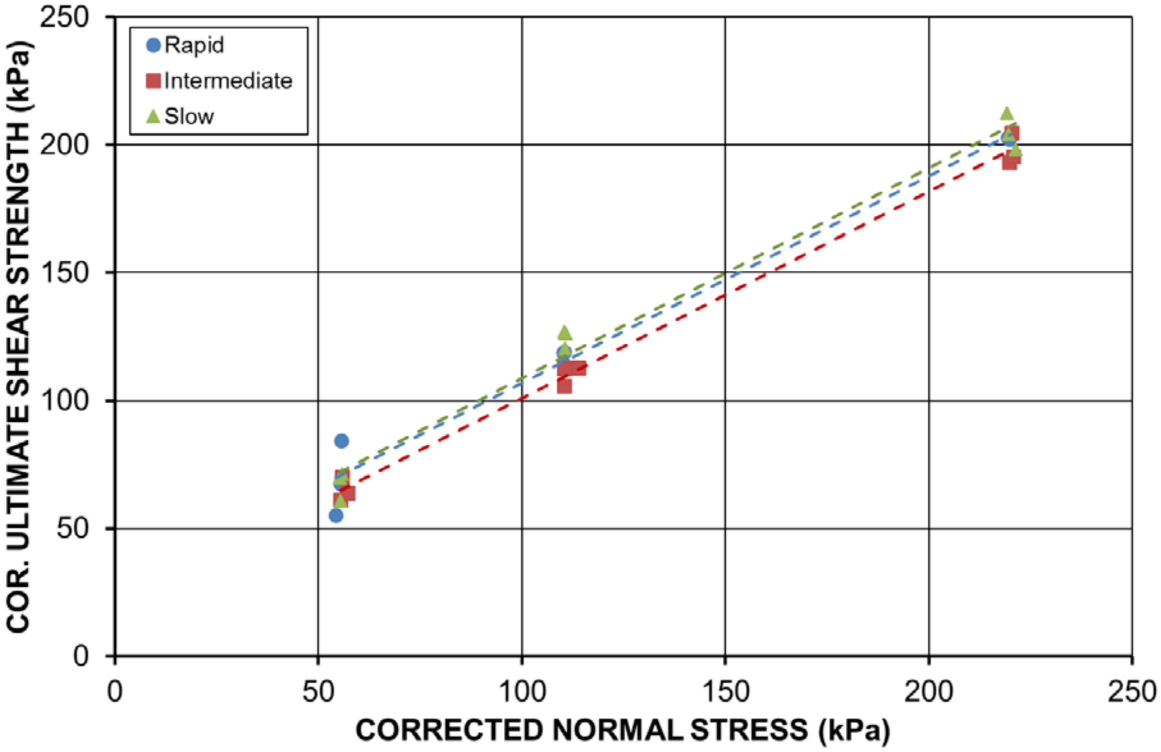

Figure 6 Ultimate shear strength envelopes

3.3 Shear Strength/Normal Stress Ratios

The shear strength/normal stress ratios versus applied normal stress are shown in Figure 7. The ratios decrease with increasing applied shear stress, most notably for the peak shear strength/normal stress ratio. All plots asymptote towards a shear strength/normal stress ratio of about 0.9, representing an equivalent secant friction angle of about 42°.

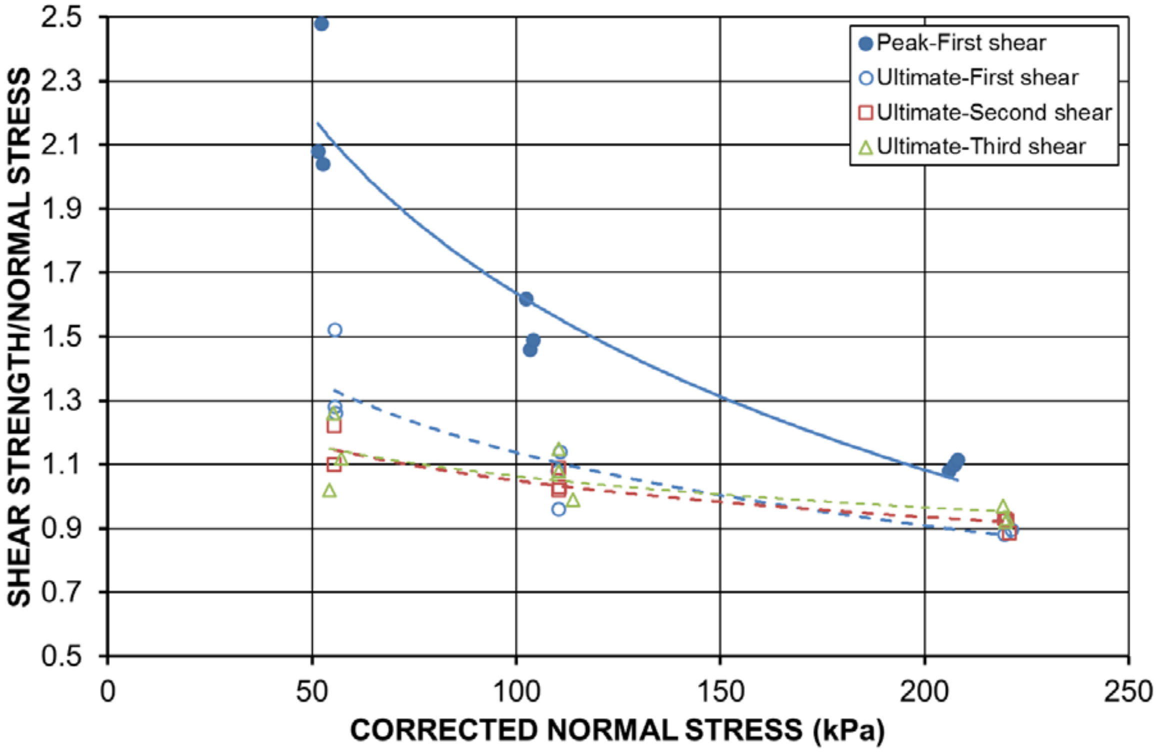

Figure 7 Ultimate shear strength envelopes

3.4 Shear Strength Envelopes

The straight line-of-best-fit peak and ultimate shear strength envelopes are summarised in Table 2 and Figure 8.

Table 2 Summary of direct shear strength parameters for MRM compacted clay

TEST CONDITIONS	APPARENT COHESION (kPa)	FRICTION ANGLE (°)
First shear-Peak	**79.4**	**35.8**
First shear-Ultimate	34.2	36.7
Second shear-Ultimate	20.0	39.6
Third shear-Ultimate	19.5	40.3
Average Ultimate	**24.6**	**38.9**

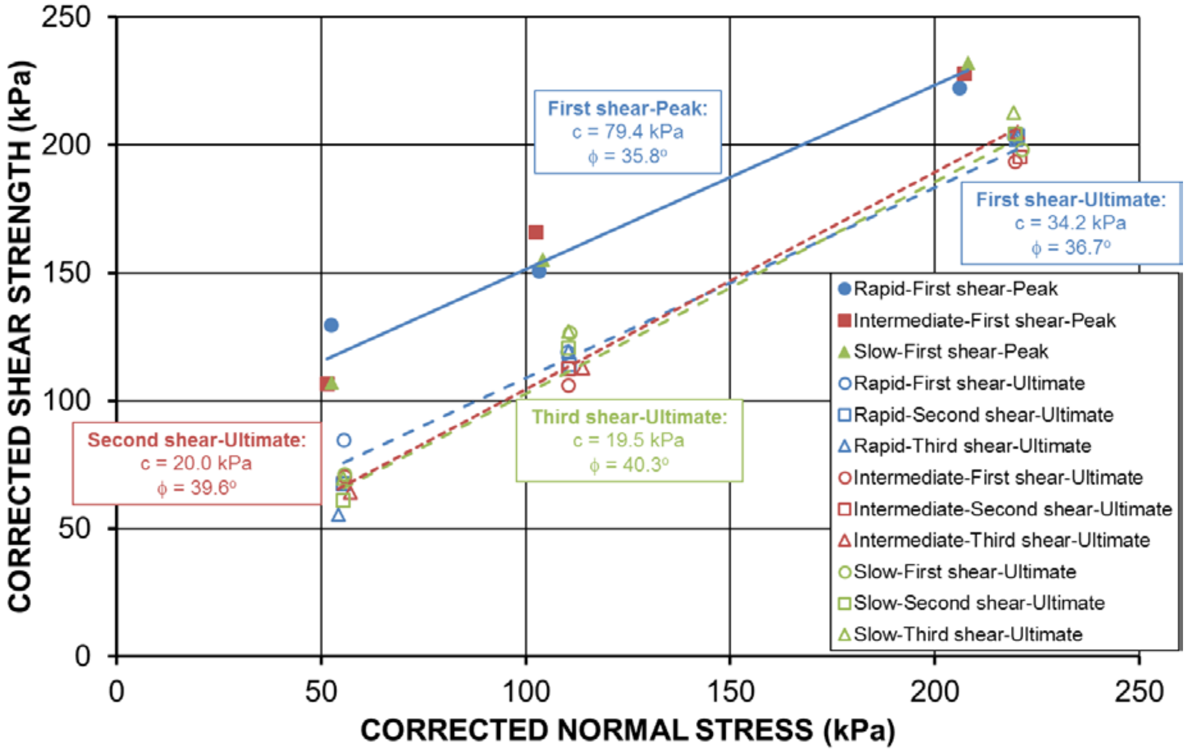

Figure 8 Summary of peak and ultimate shear strength envelopes

4 COMPARISON WITH OTHER MRM DATA

The apparent cohesion versus friction angle values obtained from a range of testing of MRM compacted clay are summarised in Table 3 and Figure 9. The direct shear test data obtained by UQ involved the testing of MRM clay compacted at an average field gravimetric moisture content of about 13.6% to a dry density of 1.85 t/m^3. MRM experience is that it has been difficult to wet-up the *in situ* clay borrow prior to its compaction, resulting in the adoption of Modified laboratory compaction testing, with an associated lower Optimum Moisture Content closer to the *in situ* moisture content of the clay borrow. The triaxial test data obtained by Trilab in 2015 involved the testing of MRM clay compacted at an average gravimetric moisture content (Standard laboratory Optimum Moisture Content) of 20.0% to a dry density (Standard laboratory Maximum Dry Density) of 1.63 t/m^3.

The UQ tests were single-stage to a (shear) strain of 10%, while the Trilab tests were multi-stage, with each stage taken to between 2% and 7% (axial) strain. Figure 3 suggests that the single-stage UQ direct shear tests were taken to sufficient strain to determine the ultimate shear strengths. Figure 10 reproduced from a Trilab multi-stage triaxial test suggests that the peak or ultimate shear strength was not reached.

Hence, the substantially lower shear strength parameters obtained by Trilab for MRM clay compacted to Standard laboratory compaction can be explained by: (i) a wetter and hence lower dry density, and (ii) the multi-stage triaxial tests not being taken to a strain sufficient to mobilise the full shear strength.

The UQ testing of loose weathered shale over compacted clay in 300 mm direct shear, single-stage, dry and wet tests gave somewhat lower shear strength parameters than the compacted clay tested alone.

Table 3 Summary of direct shear strength parameters for MRM compacted clay

TEST	INITIAL GRAVIMETRIC MOISTURE CONTENT (%)	INITIAL DRY DENSITY (t/m^3)	APPARENT COHESION (kPa)	FRICTION ANGLE (º)
UQ 2017-Peak for compacted Clay (100 mm direct shear, single-stage, tested dry)	13.6	1.85	79.4	35.8
UQ 2017-Ultimate for compacted Clay (100 mm direct shear, single-stage, tested dry)	13.6	1.85	24.6	38.9
UQ 2016-Loose Weathered Shale/ compacted Clay (300 mm direct shear, single-stage, tested dry)	1.7/13.6	1.78/1.85	15.0	32.7
UQ 2016-Loose Weathered Shale/ compacted Clay (300 mm direct shear, single-stage, tested wet)	Near-saturated	1.78/1.85	1.5	30.7
Trilab 2015-Compacted Clay (85.4 mm diameter triaxial, multi-stage, CU with pore water pressure measurement, tested near-saturated)	20.0 (saturated)	1.63	15.8	22.6
AGLAB 2007-Peak for compacted Clay (60 mm direct shear, single-stage, tested wet)	19.8	1.63	25.7	27.4
AGLAB 2007-Ultimate for compacted Clay (60 mm direct shear, single-stage, tested wet)	19.8	1.64	0.0	29.1
AGLAB 2004-Compacted Clay (48 mm diameter triaxial, multi-stage, CU with pore water pressure measurement, tested near-saturated)	16.1	1.73	12.1	28.2
Overall Average	**14.6**	**1.75**	**21.8**	**30.7**
Range	**13.6 – 20.0**	**1.63 – 1.85**	**0.0 – 79.4**	**22.6 – 38.9**

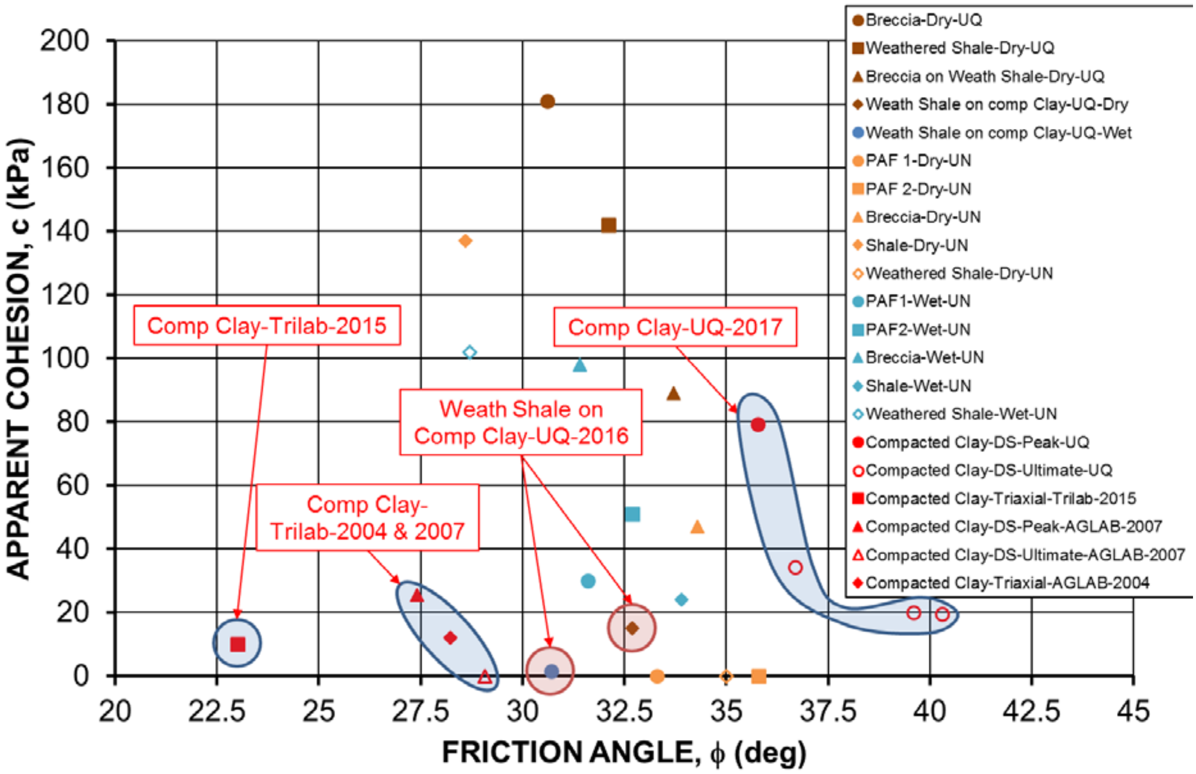

Figure 9 Comparison of shear strength parameters

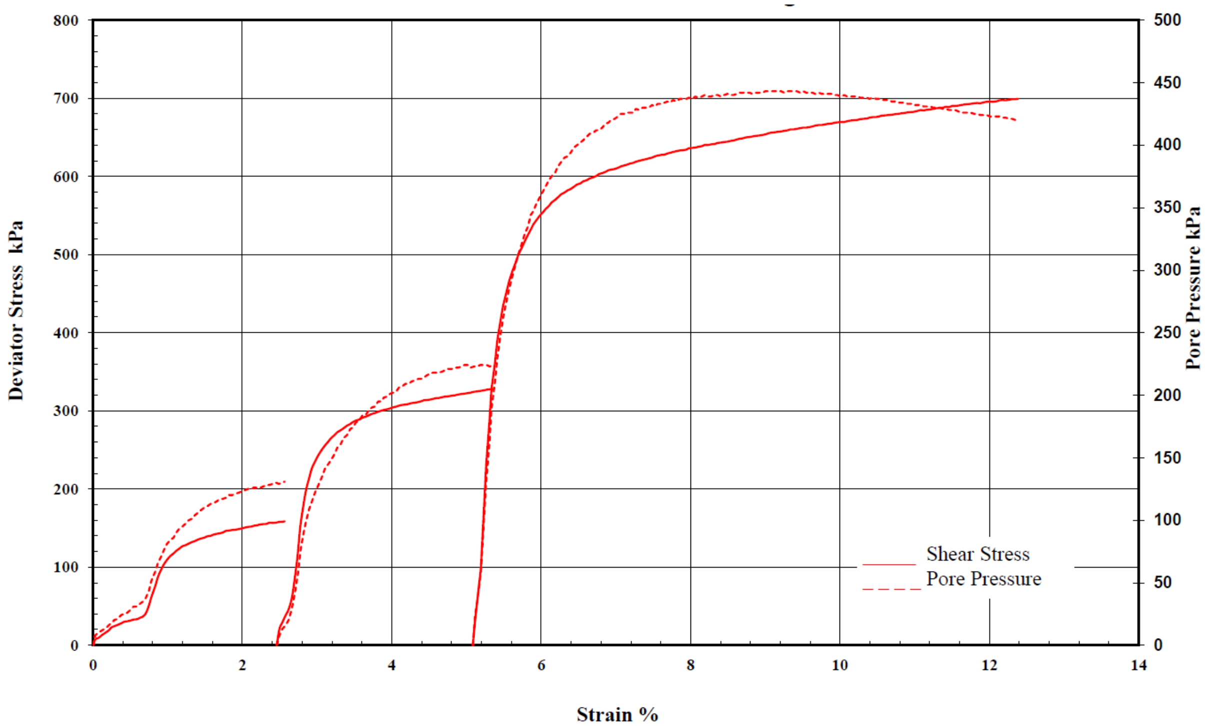

Figure 10 Typical Trilab 2015 triaxial, multi-stage, CU test with pore water pressure measurement, tested saturated, shear stress versus axial strain plots

It is clear from Table 2 and Figure 9 that the shear strength of compacted MRM clay is strongly dependent on the initial moisture content at which it is compacted and hence the dry density achieved on compaction. To better visualise this effect, shear strengths at a nominal normal stress of 100 kPa are compared against initial gravimetric moisture content and initial dry density in Figures 11 and 12, respectively. Figure 11 shows about a 50% reduction in shear strength for about a 50% increase in initial gravimetric moisture content, while Figure 12 shows about a 50% reduction in shear strength for only about a 12% decrease in initial dry density.

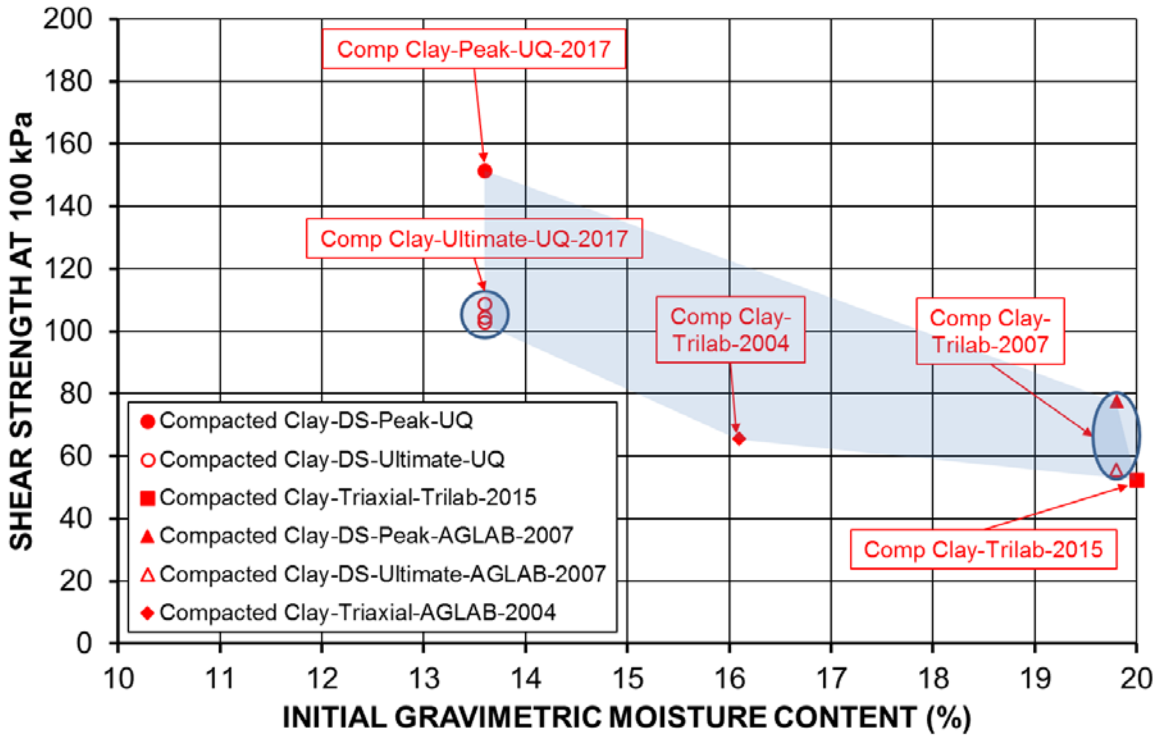

Figure 11 Shear strength at a nominal normal stress of 100 kPa versus initial gravimetric moisture content for compacted MRM clay

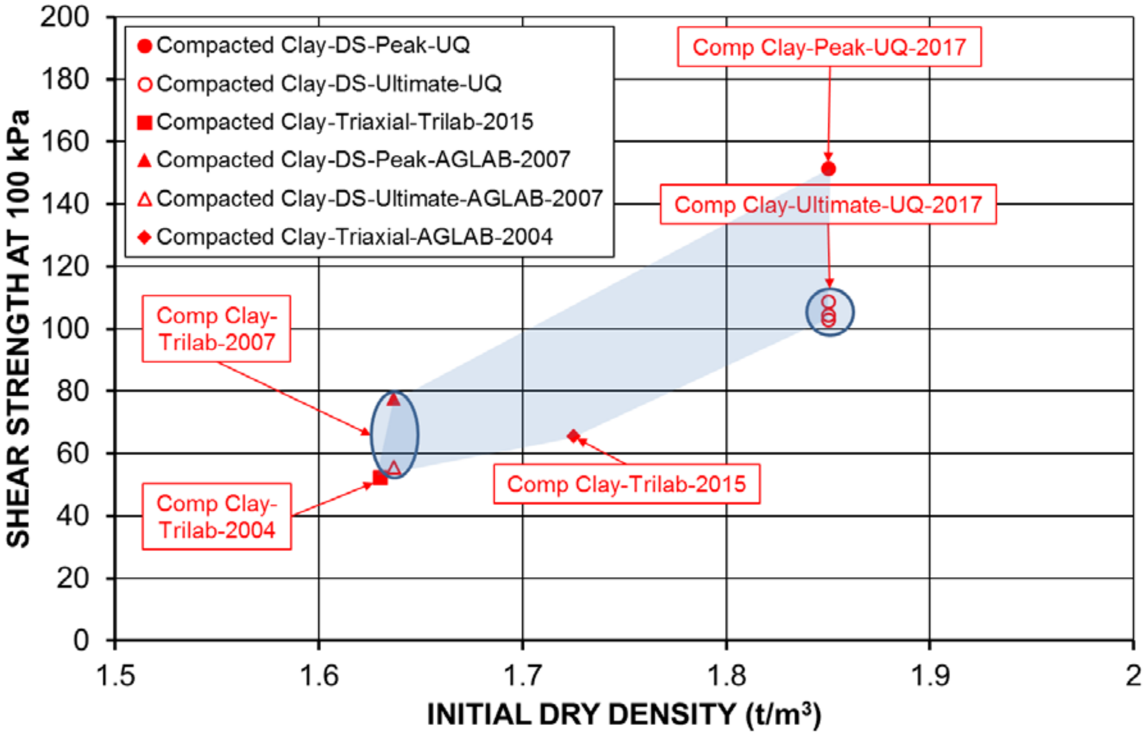

Figure 12 Shear strength at a nominal normal stress of 100 kPa versus initial dry density for compacted MRM clay

5 RECOMMENDED SHEAR STRENGTH PARAMETERS

The recommended shear strength parameters are:

- Near the surface in waste rock:
  - Apparent cohesion = 50 ± 25 kPa.
  - Friction angle = 40 ± 3º.
- Within the waste rock:
  - Apparent cohesion = 100 ± 50 kPa.
  - Friction angle = 35 ± 3º.
- On waste rock/compacted clay interfaces:
  - Apparent cohesion = 20 ± 10 kPa.
  - Friction angle = 33 ± 3º.
- Within a compacted clay layer (assuming ultimate conditions):
  - Apparent cohesion = 20 ± 10 kPa.
  - Friction angle = 37 ± 3º.

It is recommended that these average and ranges of shear strength parameters be applied in sensitivity analyses of geotechnical slope stability of the MRM waste rock dump.

APPENDIX A – Curriculum Vitae

Professor David J Williams

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QUALIFICATIONS

1979	PhD, Soil Mechanics	University of Cambridge, England
1975	BE (Hons I), Civil Engineering	Monash University, Australia

AWARDS/DISTINCTIONS/FELLOWSHIPS

1996	Japan Society for the Promotion of Science Fellow
1995	The University of Queensland Collaborative Research Travel Grant
1995	Australian Minerals and Energy Environment Foundation (AMEEF) Travelling Scholarship
1993	Australian Research Fellow (Industry)
1992	AMEEF Environmental Excellence Award (Individual)
1990	Masuda Fellow for Collaborative Research in Japan, Jan-Feb
1989	The University of Queensland Collaborative Research Travel Grant

MEMBERSHIPS

From 1980	Member, Institution of Engineers, Australia
From 1980	Member, Australian Geomechanics Society
From 1984	Member, Queensland Committee, Australian Geomechanics Society, Chair in 1986
1986-1987	Member, National Committee, Australian Geomechanics Society
2007-2008	

EMPLOYMENT HISTORY

2007 – Present	Director Geotechnical Engineering Centre School of Civil Engineering The University of Queensland

1994 – 2007	Associate Professor of Geomechanics Department of Civil Engineering The University of Queensland
1990 – 1994	Senior Lecturer in Geomechanics Department of Civil Engineering The University of Queensland
1983 – 1989	Lecturer in Geomechanics Department of Civil Engineering The University of Queensland
1980 – 1983	Geotechnical Engineer Melbourne and Brisbane Golder Associates Pty Ltd
1979 – 1980	Engineer Country Roads Board (CRB) of Victoria
1976 – 1979	Research Student University of Cambridge, England
1972 – 1976	Engineer, Cadet Engineer, CRB, Victoria

SUMMARY OF CONSULTING COMMISSIONS

Board Memberships

- Member of Northern Territory EPA Board, from 2012 to 2014

Peer Reviews of Major Projects

- Sole Independent Expert Geotechnical Reviewer for Unity Mining Limited from 2016
- Sole Independent Expert Geotechnical Reviewer for Bluestone Mines Tasmania JV Pty Ltd from 2015
- Sole Independent Expert Geotechnical Reviewer for Rio Tinto Alcan Gove Residue Disposal Area from 2015
- Sole Independent Expert Geotechnical Reviewer and Annual Dam Inspections for QAL Residue Disposal Area and Ash Dams from 2013
- Sole Independent Expert Geotechnical Reviewer for Rio Tinto Alcan Yarwun Residue Management Area from 2013
- Led International Peer Review for the South Deposit TSF at Savage River Mine in Tasmania in 2012/13
- Sole Independent Expert Geotechnical Reviewer for Rio Tinto Alcan Weipa Tailings Storage Facilities in 2012 and 2014
- Peer Review of Harvey Creek Non-Erodable Waste Rock Dump Design for Ok Tedi Mining Limited in 2010/11
- Member of Expert Peer Review Team for Rio Tinto Alcan Weipa Tailings Storage Facilities from 2009
- Member of the International Technical Advisory Group reporting to the South Australian Government on Rehabilitation of Brukunga Pyrite Mine from 2007

- Led International Peer Reviews for the Savage River Rehabilitation Project in Tasmania in 2002, 2005, 2009 and 2013
- Led International Peer Review on handling acid generating waste rock dumping and dump closure strategies at Cadia Hill Gold Mine in New South Wales in 2002/3
- Member of the Peer Review Team for Stage 2 of the Stuart Oil Shale Project at Gladstone in Queensland in 2004
- Peer Reviewer of the rehabilitation of the San Manuel Copper Mine tailings facility in Arizona, USA in 2004
- Member of the 2005 Peer Review Team that reviewed future red mud disposal, containment and rehabilitation at QAL at Gladstone in Queensland in 2005
- Geotechnical Reviewer of the breach of the co-disposal dam at Burton Coal in Queensland in 2005
- Peer Reviewer of the conceptual closure plan for Worsley Alumina red mud storage in Western Australia in 2005
- Peer Reviewer for waste rock dump covers for Century Mine in North Queensland from 2007
- During 2006, David was an Expert Advisor to the EIS team for the Olympic Dam Expansion Project in South Australia, providing expert input on disposal, hydrology and closure issues for both waste rock and tailings

Expert Witness

- Expert witness through Corrs Chambers Westgarth Lawyers, in relation to coal washery rejects used as filling for residential sub-division purposes
- Expert witness through McCullough Robertson Lawyers, in relation to the failure of a concrete arch reclaim tunnel beneath a coal stockpile
- Expert witness in relation to professional misconduct cases brought by the Queensland Professional Engineers Registration Board
- Numerous expert witness commissions related to residential and commercial building footing failures and slope instability

Consultancies

Professor David John Williams is widely sought for his expert input, in particular to mine waste disposal and mine site rehabilitation and remediation at operating mines throughout Australia and overseas. In Australia, he has consulted on numerous coal mines throughout Queensland and New South Wales; on Red Dome Gold Mine closure, Kidston closure, Osborne waste disposal, Ivanhoe Cloncurry mine closure, Phosphate Hill gypsum disposal, QERL processed waste storage facility closure, and Century Zinc Mine waste rock dumping in Queensland; Cadia Hill Gold Mine waste rock dumping and dump closure in New South Wales; Mt Morgans Gold Mine co-disposal, WMC Resources" nickel operations tailings closure and Minara heap leaching in Western Australia; waste disposal issues at the Ballarat East and Heathcote gold mines in Victoria; and a review of ARD treatments at Savage River Mine in Tasmania. Overseas he has consulted on tailings depositional design and water balance for the Kori Kollo Mine in Bolivia, a review of co-disposal of tailings and waste rock at Porgera Gold Mine and the closure of Misima Gold Mine in PNG, waste disposal design for the Goro Nickel project in New Caledonia, and advice on co-disposal for the Martabe Project in Indonesia.

David has been involved in material characterisation testing and the design of numerous mine waste covers throughout Australia, and the design, installation and monitoring of lysimeters and mine waste covers at Kidston Gold Mines, WMC Resources" Mt Keith Nickel Operations, QERL"s Stuart Oil Shale Project, a large-scale trial waste rock dump at Cadia Hill Gold Mine, and a large-scale trial tailings cell at Jubilee Nickel Mine.

David has been invited to visit numerous mining regions and individual mines throughout Australia, and in Canada, the USA, Brazil, South Africa, UK, China, Chile, PNG, New Caledonia, Spain and Mozambique.

MAJOR RESEARCH ACHIEVEMENTS

From 1989, Professor Williams carried out research under NERDDC and ACARP Projects on the characterisation of the deposit formed on the pumped co-disposal of combined washery wastes, which has since been adopted at numerous coal mines in Australia and Indonesia.

From 1996, David developed the store/release cover system suited to seasonally dry climates, for application to covering acid generating rock dumps at Kidston Gold Mine in north Queensland, and has had a long-term involvement in researching and monitoring this cover system, as evidenced by his numerous papers on his research on this topic. The store/release cover system on the tops of the Kidston rock dumps has been shown to limit percolation to less than 1% of rainfall, and to support a sustainable vegetation cover comparable to that occurring along water courses in the area. He was also involved in the development of a rehabilitation strategy for the side slopes of the rock dumps at Kidston designed to maximise geotechnical and erosional stability while promoting vegetation, and analysed the wetting up by rainfall infiltration and subsequent drain-down of and seepage from the rock dumps. Store/release covers have now been adopted at numerous mine sites in dry climates worldwide.

From 1999 to 2001, David led ACARP Project C8039 to develop a risk assessment and cost-effectiveness analysis for the rehabilitation of Bowen Basin coal mine spoil. The results of the project were reported in a Literature Review and Commentary and Project Final Report, plus a spreadsheet-based risk assessment and cost-effectiveness analysis, available at: www.uq.edu.au/civil/. In 2006, David undertook a closure study for Xstrata"s new Rolleston Coal Project in the Bowen Basin Coalfields.

David has since 2000 been involved in the closure design for the waste rock dump at Cadia Hill Gold Mine in New South Wales, including studies on the use of mixtures of benign trafficked rock and tailings as an alternative cover material, to overcome the shortage of suitable natural materials. In 2002/3, he led an international peer review of the rock dumping operation and closure plan. In 2004, David was successful in an ARC Linkage grant application with Cadia totalling over $ 700,000 over 3 years, which has led to the construction of a 15 m high, world-class, demonstration, instrumented rock dump covering 7,000 m^2. The instrumentation includes a full weather station, 24 lysimeters at the base of the dump to monitor seepage, lysimeters on the top surface to monitor rainfall infiltration and three store/release trial covers constructed using natural and mine waste materials. To date it has shown that about 70% of the rainfall incident on the traffic-compacted top of the dump infiltrates, with the majority going into storage within the dump during the first year, and only small amounts percolating to the base of the dump. The behaviour of the cover trials has to date been dominated by the moisture state at which they were constructed. Monitoring of the instrumented rock dump is expected to continue for at least 10 years.

From 2000 to 2003, David was a principal researcher into the physical and geochemical nature of acid generating waste rock dumps in Southern Carolina, USA (Rio Tinto"s Ridgeway Mine) and Sudbury, Canada (Inco"s Whistle Dump), sampled as they were being excavated and moved to a pit.

From 2001 to 2005, David led an ARC Spirt research project with industry partner WMC Resources focussed on an assessment of the long-term seepage and runoff from mine tailings storage facilities, to facilitate lease surrender. This included the monitoring of trial covers on tailings over the duration of the project and large-scale laboratory column testing and numerical analyses. Natural salt pan and rocky slope analogues under the same climatic and similar geochemical conditions were also studied to point to sustainable approaches for rehabilitating the tailings storage facilities.

From 2010, David has led two ACARP Projects, C19022 and C20047, investigating the settlement and stability of high coal mine spoil, and the behaviour of problematic clay-rich coal mine tailings.

David has been sponsored by mining companies and consultants to visit numerous mining regions and mine sites worldwide, both to impart and extend his knowledge. Since 2000, he has developed a relationship with the International Network for Acid Prevention (INAP), and has contributed to INAP-sponsored research and development projects and workshops involving mine sites in the USA, Canada, Australia and PNG.

Research funding has totalled over $7 million, including funding from ARC, ARC-SPIRT, ARC Linkage, NERDDC, ACARP-AMIRA, ACARP, MIM CRA-ATD, Kidston Gold Mines, BHP Coal and WMC Resources, Cadia Holdings, Jubilee Mines NL. Professor Williams has over 250 refereed publications, with about two-thirds of them in the mine waste field.

SELECTED PUBLICATIONS

Book Chapters

1. **Williams, D.J.** (2005). Chapter 17: Placing covers on soft tailings. In: *Ground Improvement-Case Histories*, 491-512. Eds B. Indraratna and Chu Jian. Elsevier.
2. **Williams, D.J.** (2001). Chapter 30: Assessment of Embankment Parameters. In: *Slope Stability in Surface Mining*, 275-284. Eds W.A. Hustrulid, M.J. McCarter and D.J.A Van Zyl. Society for Mining, Metallurgy, and Exploration, Inc., Littleton, Colorado, USA.
3. **Williams, D.J.** (1996). Chapter 7: Minimisation and Management of Solid Wastes. In: *Environmental Management in the Australian Minerals and Energy Industry*, 157-188. Ed D.R. Mulligan. Sydney, UNSW Press in association with Australian Minerals and Energy Environment Foundation, 1996.

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1. Serati, M., Alehossein, H. and **Williams, D.J.** (2015). Estimating the tensile strength of super hard brittle materials using truncated spheroidal specimens. *Journal of the Mechanics and Physics of Solids*, **78**, 123-140.
2. Zbik, M.S., **Williams, D.J.**, Song, Y-F and Wang, C.C. (2015). How the hydro-gel flocculation microstructure changes. *Colloids and Surfaces A: Physicochemical and Engineering Aspects*, **469**, 11-19.
3. Zbik, M.S., **Williams, D.J.**, Song, Y.-F. and Wang, C.-C. (2015). Smectite clay microstructural behaviour on the Atterberg limits transition. *Colloids and Surfaces A: Physicochemical and Engineering Aspects*, **467**, 89-96.
4. Li, Y., Topal, E. and **Williams, D.J.** (2014). Optimisation of waste rock placement using mixed integer programming. *Transactions of the Institutions of Mining and Metallurgy, Section A: Mining Technology*, **123**(4), 220-229.
5. Poulsen, B.A., Shen, B., **Williams, D.J.**, Huddlestone-Holmes, C., Erarslan, N. and Qin, J. (2014). Strength reduction on saturation of coal and coal measures rocks with implications for coal pillar strength. *International Journal of Rock Mechanics and Mining Sciences*, **71**, 41-52.
6. Erarslan, N., Alehossein, H. and **Williams, D.J.** (2013). Tensile fracture strength of Brisbane tuff by static and cyclic loading tests. *Rock Mechanics and Rock Engineering, Online First*, 1-17.
7. Galindo-Torres, S.A., Pedroso, D.M., **Williams, D.J.** and Muhlhaus, H.B. (2013). Strength of non-spherical particles with anisotropic geometries under triaxial and shearing loading configurations. *Granular Matter*, **15**(5), 531-542.
8. Serati, M., Alehossein, H. and **Williams, D.J.** (2013). 3D elastic solutions for laterally loaded discs: generalised Brazilian and Point Load tests. *Rock Mechanics and Rock Engineering, Online First*, 1-15.
9. Dight, P.M., Douglas, B., Henley, K., Lumley, G., McAree, P.R., Miller, D., Saydam, S., Topal, E., Wesseloo, J. and **Williams, D.J.** (2013). Developments in open cut mining. *Australasian Mining and Metallurgical Operating Practices, 3rd ed.,* 47-80. Australasian Institute of Mining and Metallurgy.
10. Topal, E. and **Williams, D.J.** (2013). Mine waste rock management. *Australasian Mining and Metallurgical Operating Practices, 3rd ed.*, 76-77. Australasian Institute of Mining and Metallurgy.
11. Eraslan, N. and **Williams, D.J.** (2013). Mixed-mode fracturing of rocks under static and cyclic loading. *Rock Mechanics and Rock Engineering*, **46**(5), 1035-1052.
12. **Williams, D.J.** and Kho, A.K. (2013). Laboratory geotechnical characterisation of scalped coal mine spoil. *Australian Geomechanics Journal*, **48**(1), 101-110.
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Selected Refereed Conference Papers

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McARTHUR RIVER MINE
NOEF SONIC DRILLING SAMPLES
LABORATORY TESTING AND INTERPRETATION

Report prepared by

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1 INTRODUCTION

Professor David John Williams was commissioned by Mr Gary Taylor of McArthur River Mine (MRM) to advise on the sampling, and laboratory testing at The University of Queensland (UQ), of North Overburden Emplacement Facility (NOEF) sonic drilling samples obtained during an extensive drilling campaign from late 2015 to early 2016. This report presents the advice given on sonic drilling and the sampling methodology, the testing methodology, the test results, and their interpretation.

2. SONIC DRILLING AND SAMPLING METHODOLOGY

In August and September 2015, Professor Williams provided the following advice regarding sonic drilling and sampling procedures to investigate the NOEF:

- Sonic drilling should be carried out to ensure that recovered samples are not contaminated by re-drilled materials (e.g. from the advancement of casing), and drilling rates should be recorded.
- Sonic drilling at each location on the NOEF should be extended to three levels:
    - Through the PAF waste rock to just above the liner.
    - Through the PAF waste rock, liner and underlying NAF waste rock to just above the natural surface.
    - To the underlying weathered rock aquifer (deepest hole, which should be sampled; the shallower holes could be advanced without sampling to speed up the sonic drilling program, while still be available for gas sampling and piezometer installation).
- The choices for sonic drilling are without or with water:
    - Drilling without water would better maintain the *in situ* moisture state, and would avoid adding water to a hot dump, but would slow progress (up to 2-fold) and lead to more particle crushing.
    - Drilling with water would better maintain the particle size distribution (PSD) of the waste rock and, hence, possibly its density, but would risk adding water to the dump that might form steam.
    - On balance, estimating the moisture state is more important (and possibly more reliable) than estimating the PSD and density (both of which will be affected by thee drilling), and drilling without water is preferred to enable an estimate to be made of the moisture profile to the aquifer at each location. Oxygen, SO_2, CO_2 (and any other relevant gases), and temperature profiles should also be measured down these holes.
    - Drilling without water is particularly important above and through PAF waste rock, to avoid steam generation from hot rock.

- The following should be carried out on cores recovered from at least the deepest holes at each location:
    - Geotechnically log and photograph the recovered “core”, noting any losses.
    - For each 1 m of core (or a lesser frequency if the samples are reasonably consistent with depth), and for the liner:
        - Bag (use two plastic bags to maintain moisture) a 200 g (about 200 mm of core) representative sub-sample for determinations of:
            - Average core diameter and length (on site, to be reported separately).
            - Wet mass of the recovered core (on site, to be reported separately; from which the wet density can be estimated).
            - Gravimetric moisture content (= Mass of water/Mass of solids, expressed as a %), using a 50°C oven to avoid spontaneous combustion (on site or at UQ; from which the dry density can be estimated).
            - Total suction, requiring a sample to fit into a dish 30 mm in diameter by 10 mm high (at UQ; from the measured moisture contents and inferred matric suctions a field SWCC may be plotted).
            - Specific gravity of the solids (at UQ, using a helium pycnometer).
            - Electrical conductivity and pH of any pore water (at UQ).
            - % finer than 0.075 mm (at UQ; on selected, obviously fine-grained, samples).
        - In the cold holes, a dry state was expected towards the surface due to dry season evaporation, becoming more moist at depth in material that was there during previous wet seasons and is storing moisture, then perhaps drier at greater depth, and finally the most wet towards the base where there may even be a perched water table.
        - In the hot holes, depleted moisture was expected where oxidation is taking place, due to moisture consumption in the oxidation reaction and evaporation due to high temperatures. Either side of these zones there may be increased moisture due to “sweating” and the drawing of moisture towards the hot layers.
        - Retain representative samples for possible geochemical testing.
    - If a relatively undisturbed sample can be recovered from the liner, laboratory permeability testing should be attempted and representative samples retained for possible geochemical testing.

- To avoid steam formation, in hole permeability testing is not recommended, particularly in PAF waste rock. Further, it would be near impossible to achieve a constant head, the flow would be three-dimensional, making interpretation difficult, and it would be difficult to seal the hole to enable *in situ* permeability testing.
- All completed holes should have vibrating wire piezometers installed for monitoring perched or permanent water tables, and for possible water sampling.

3. TESTING METHODOLOGY

The scope and purpose of the laboratory testing carried out at UQ are summarised Table 1. The testing was carried out in accordance with AS 1289 Testing Soils for Engineering Purposes.

Table 1 Summary of scope and purpose of laboratory testing

TEST	TEST FREQUENCY	PURPOSE OF TEST
Gravimetric Moisture Content = Mass of water/Mass of Solids, expressed as a %	Every 1 m	Assess saturation due to rainfall infiltration, and extent of drying due to spontaneous combustion
Total Suction in kPa using a WP4C Dewpoint Potentiometer		
Specific Gravity (SG) of solids, obtained using a helium pcynometer		Index on mineralogy (pyrite content, given its SG of 4.5)
Electrical Conductivity in µS/cm	Every 1 m on 5:1 (water:solids, by mass) paste	Salinity (a measure of oxidation of PAF waste rock), and used to estimate osmotic suction
pH		Extent of acidity generation due to oxidation of pyrite in PAF waste rock
% Finer than 0.075 mm, essentially silt and clay-size	Selected, obviously fine-grained, samples	Measure of weathering and extent of breakdown due to oxidation of PAF waste rock

3. TEST RESULTS

3.1 Locations of NOEF Sonic Drill Holes

The locations of the sonic drill holes in the NOEF are shown in plan in Figure 1, with selected early drill holes shown in a cross-section in Figure 2.

3.2 NOEF Sonic Drill Hole Logs

Sonic drill hole logs are shown in Figures 3 to 7, for GWNOEF 5D to 24.0 m depth, for GWNOEF 7S to 47.5 m depth, for GWNOEF 8S to 79.5 m depth, for GWNOEF 9S to 40.0 m depth, and for GWNOEF 10S to 28.5 m depth, respectively, with the main material types overlain. The main material types include compacted clay, PAF waste rock, NAF waste rock, and natural alluvium, underlain by weathered and then fresh bedrock. For ease of comparison, the logs shown in Figures 3 to 7 are drawn to the same vertical scale (depth below the surface of the NOEF). Samples collected from these sonic drill holes were subjected to laboratory testing at UQ.

Figure 8 shows the combined simplified sonic drill hole logs for GWNOEF 5D, 7S, 8S, 9S and 10S plotted against depth below the surface of the NOEF and with the main material types shown. Figure 8 highlights the predominance of PAF waste rock beneath a compacted clay seal GWNOEF 8S, 9S and 10S.

Figure 1 Locations of sonic drill holes in the NOEF in plan

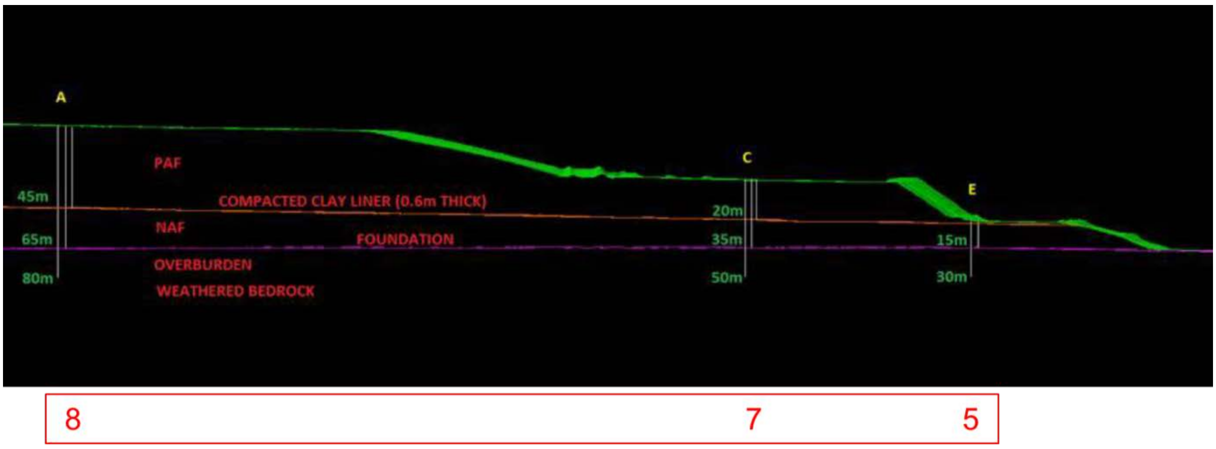

Figure 2 Locations of selected sonic drill holes in NOEF in cross-section

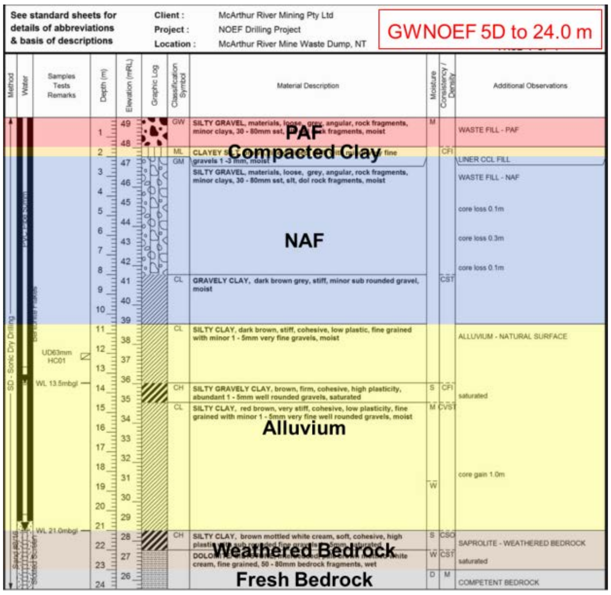

Figure 3 Log of sonic drill hole GWNOEF 5D to 24.0 m

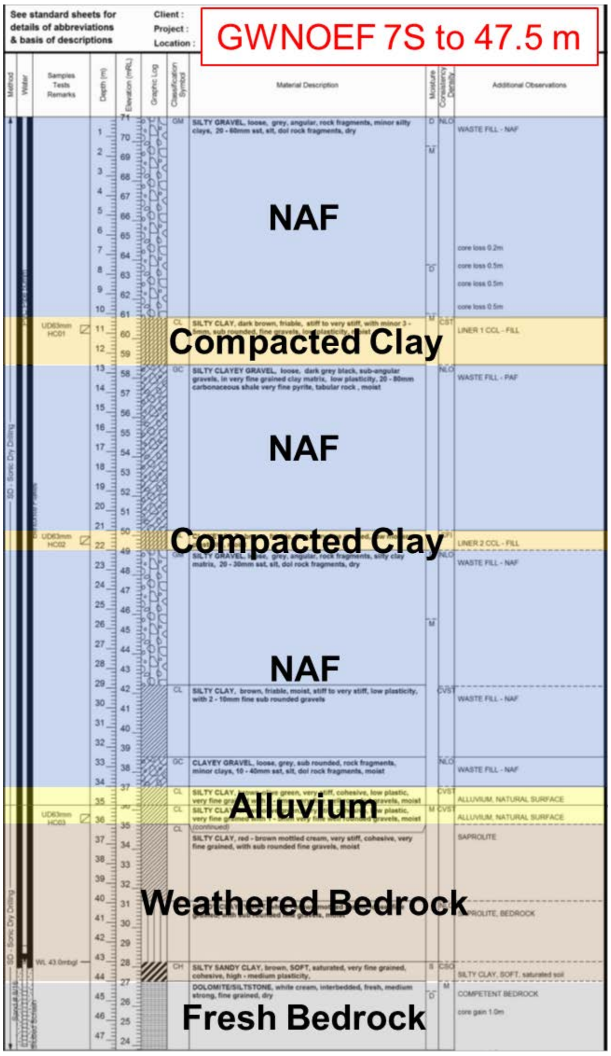

Figure 4 Log of sonic drill hole GWNOEF 7S to 47.5 m

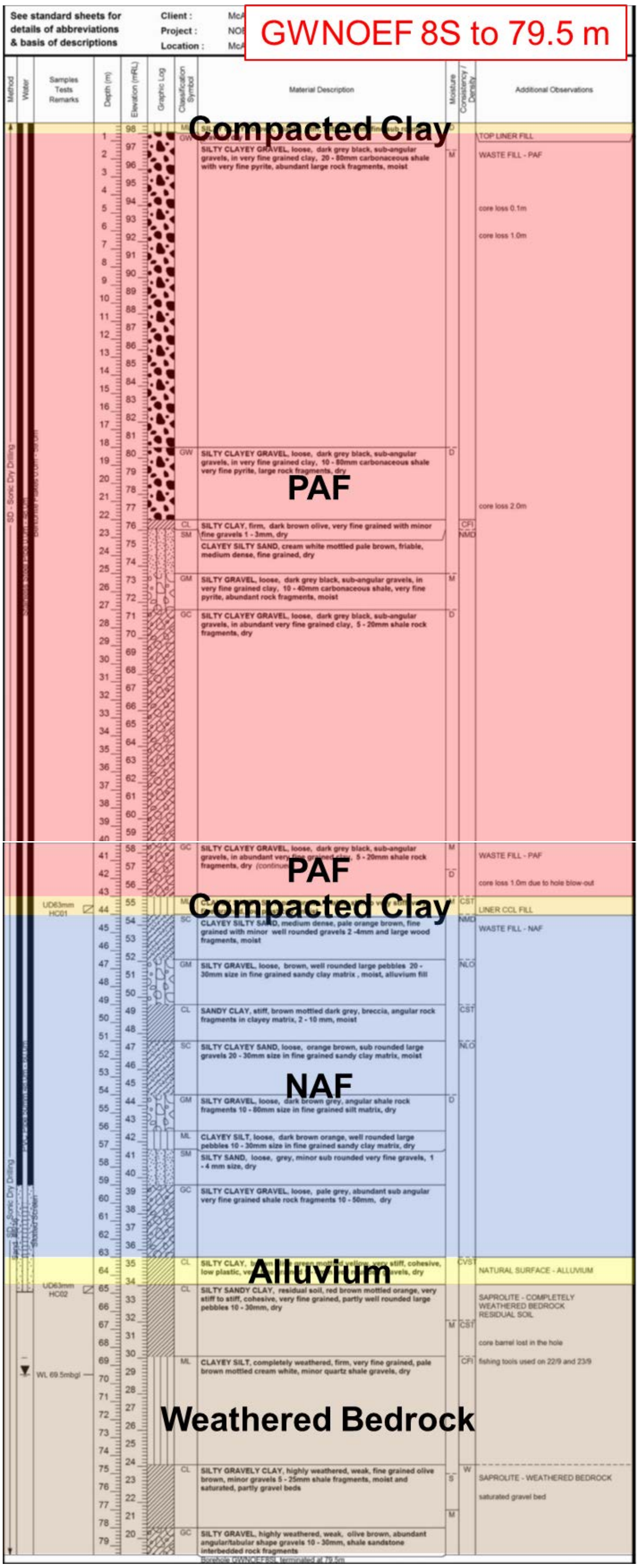

Figure 5 Log of sonic drill hole GWNOEF 8S to 79.5 m

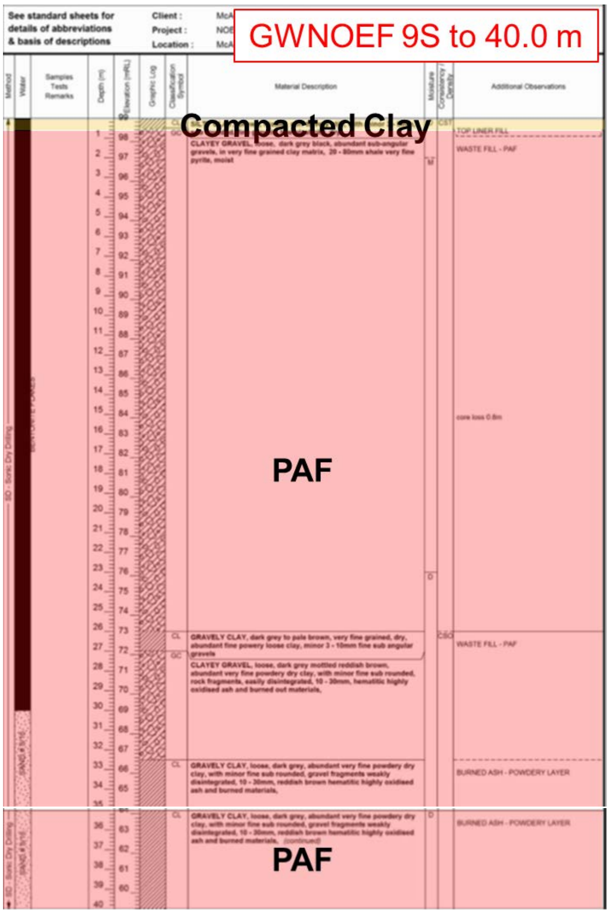

Figure 6 Log of sonic drill hole GWNOEF 9S to 40.0 m

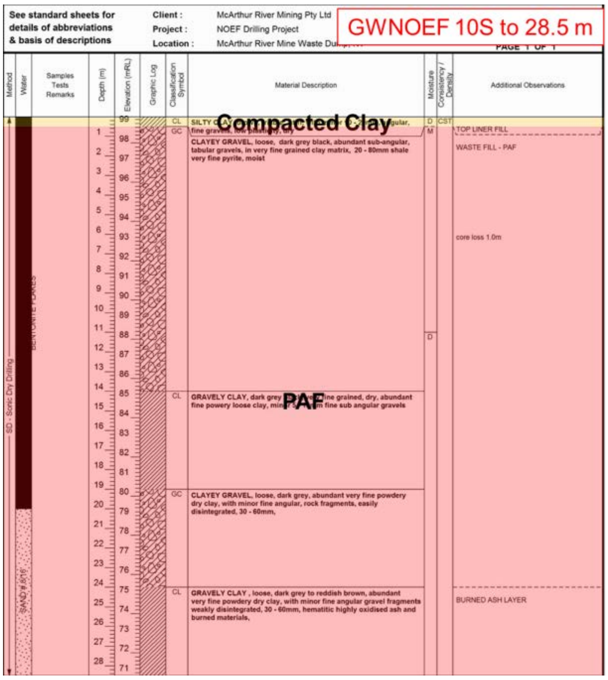

Figure 7 Log of sonic drill hole GWNOEF 10S to 28.5 m

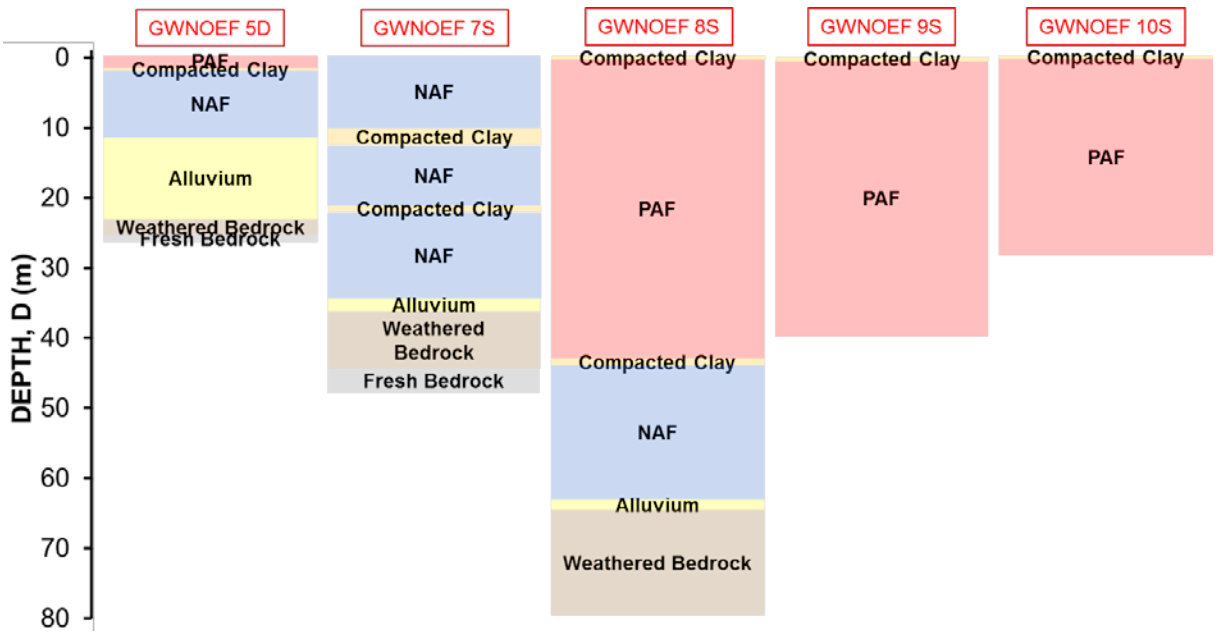

Figure 8 Simplified sonic drill hole logs showing main material types, plotted against depth below surface of NOEF

3.3 Laboratory Test Results and Their Interpretation

The results of the laboratory tests carried out at UQ on 1 m frequency samples from sonic drill holes GWNOEF 5D, 7S, 8S, 9S and 10S are presented in the following sections.

3.3.1 Moisture state and specific gravity

Figures 9 and 10 show, respectively, the as-sampled gravimetric moisture content and specific gravity profiles with depth. From these profiles, the as-sampled degree of saturation of the samples may be estimated using:

$$\boldsymbol{\rho_{dry} = G_s / (1 + e)} \tag{1}$$

where $\boldsymbol{\rho_{dry} = G_s / (1 + e)}$ is the dry density of the NOEF, which is taken to be 1.8 t/m^3, $\boldsymbol{G_s}$ is the specific gravity, which may be taken as constant for each drill hole, and $\boldsymbol{e}$ is the void ratio.

and

$$\boldsymbol{w.G_s = S.e} \tag{2}$$

where $\boldsymbol{w}$ is the gravimetric moisture content expressed as a decimal, and $\boldsymbol{S}$ is the degree of saturation expressed as a decimal. Combining Equations (1) and (2), the degree of saturation may be estimated from:

$$\boldsymbol{S = w.G_s / [(G_s / \rho_{dry}) - 1] \sim w.G_s / (0.556\ G_s - 1)} \tag{3}$$

Figure 11 shows the estimated degree of saturation profiles with depth. The average values of gravimetric moisture content, specific gravity and estimated degree of saturation are given in Table 1.

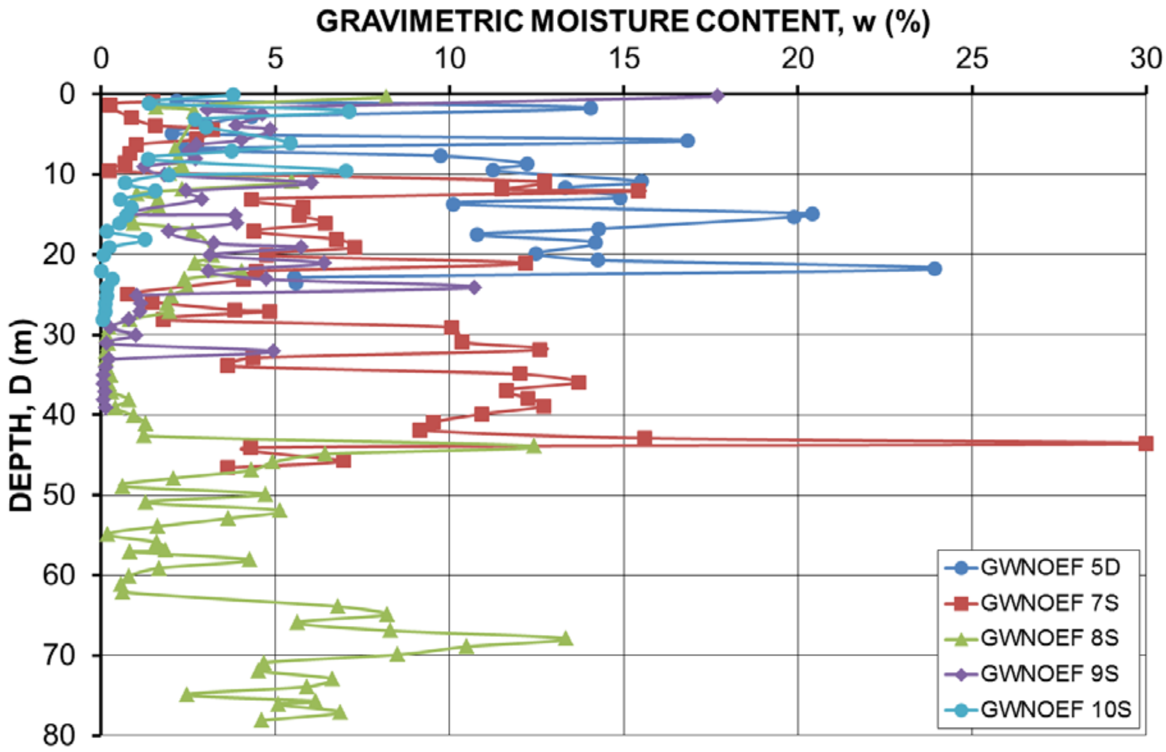

Figure 9 Profiles of gravimetric moisture content with depth

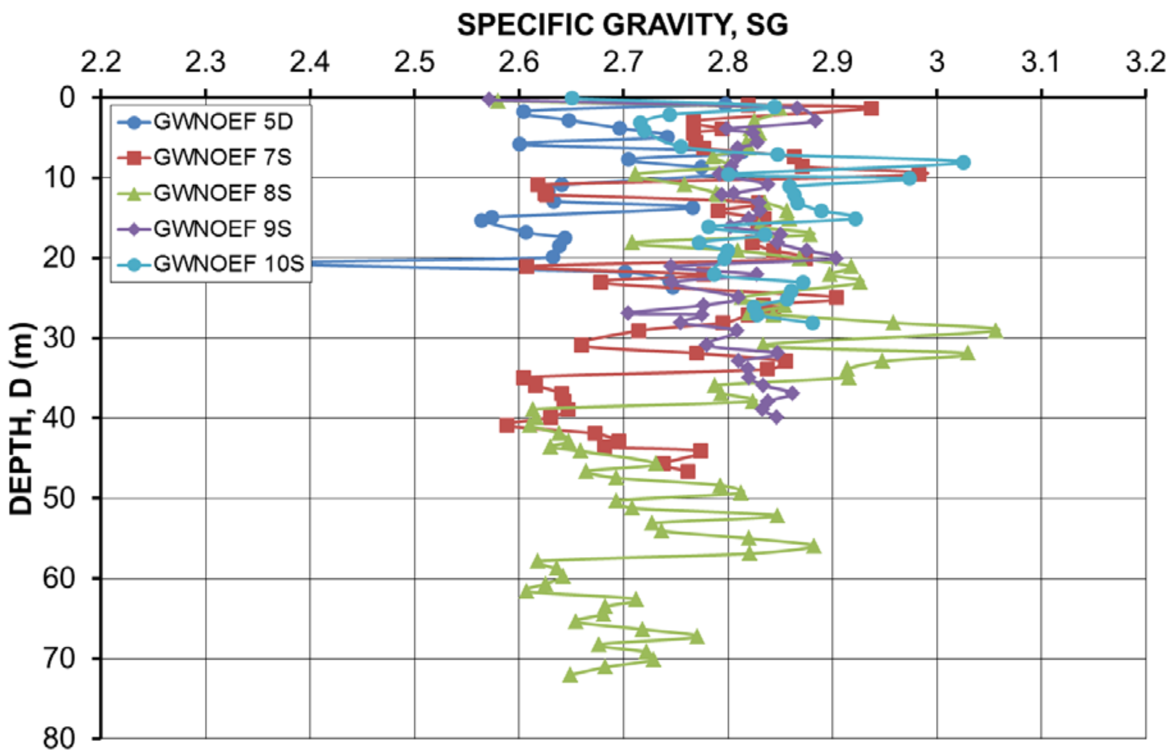

Figure 10 Profiles of specific gravity with depth

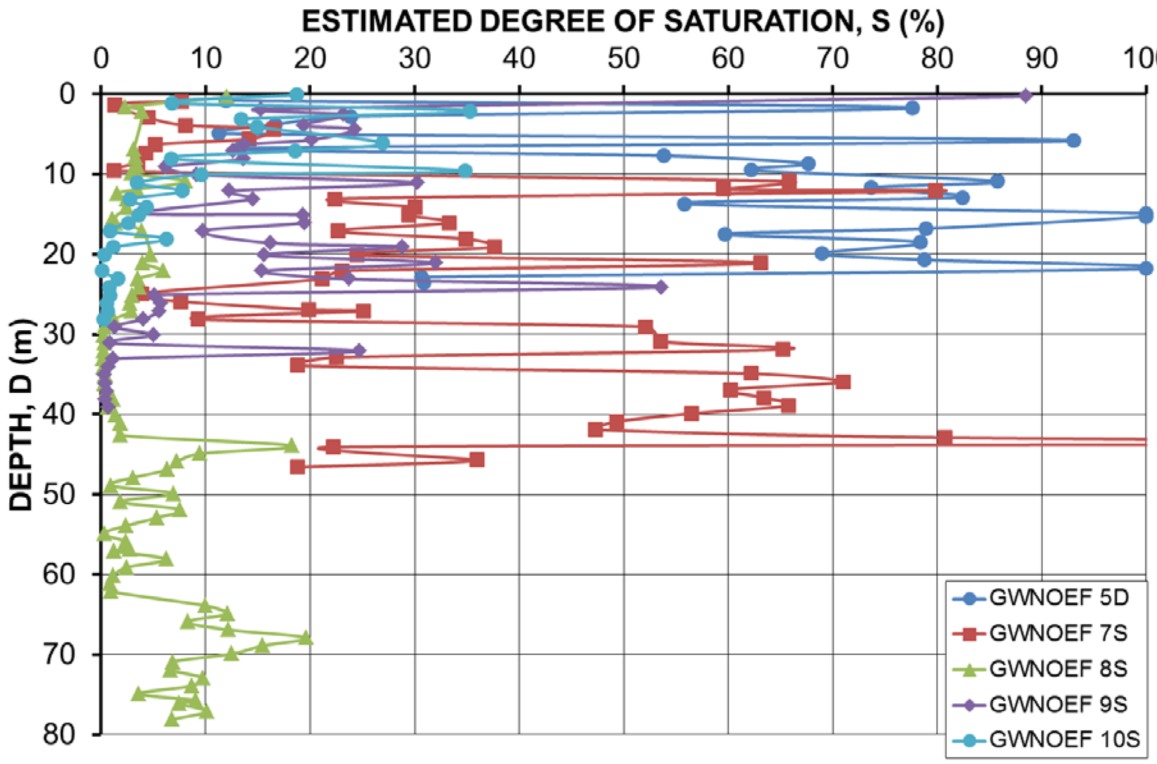

Figure 11 Profiles of estimated degree of saturation with depth

Table 1 Average values of gravimetric moisture content, specific gravity and estimated degree of saturation

SONIC DRILL HOLE	AVERAGE AS-SAMPLED MOISTURE CONTENT (%)	AVERAGE SPECIFIC GRAVITY	ESTIMATED DEGREE OF SATURATION (%)
GWNOEF 5D	11.4	2.670	62.9
GWNOEF 7S	6.9	2.759	35.8
GWNOEF 8S	3.2	2.772	16.2
GWNOEF 9S	3.0	2.811	14.9
GWNOEF 10S	1.7	2.828	8.3

Comparing Figures 9 and 11 with Figure 8, the NAF waste rock in the profiles (GWNOEF 5D, 7S and the lower part of 8S) is seen to be the wettest and most saturated, while the PAF waste rock is the driest and least saturated. This is attributed to the loss of moisture due to the oxidation of the PAF waste rock, which is accompanied by elevated temperature resulting from the exothermic oxidation reactions. The higher specific gravity values in Figure 10 generally coincide with PAF waste rock, reflecting the higher specific gravity of the pyrite content. The average values in Table 1 bear out these interpretations.

3.3.2 Total suction profiles

The total suction, which can readily be measured using a potentiometer, is the sum of matric (or capillary) and osmotic (or solute) suctions. Figures 12 and 13 show the total suction profiles with depth to natural and semi-$\log_{10}$ scales, respectively. Total suction is seen to bear an inverse relationship to gravimetric moisture content and degree of saturation, as expected (the wetter and more saturated a material, the lower its total suction).

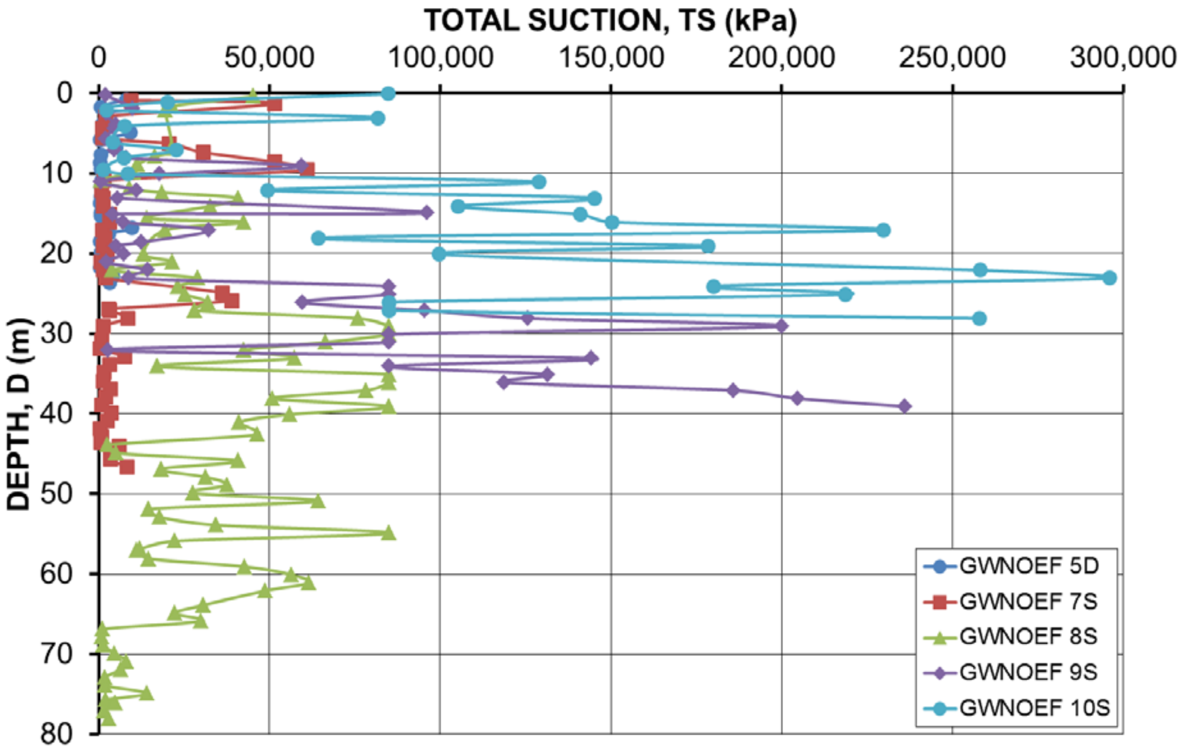

Figure 12 Profiles of total suction (to a natural scale) with depth

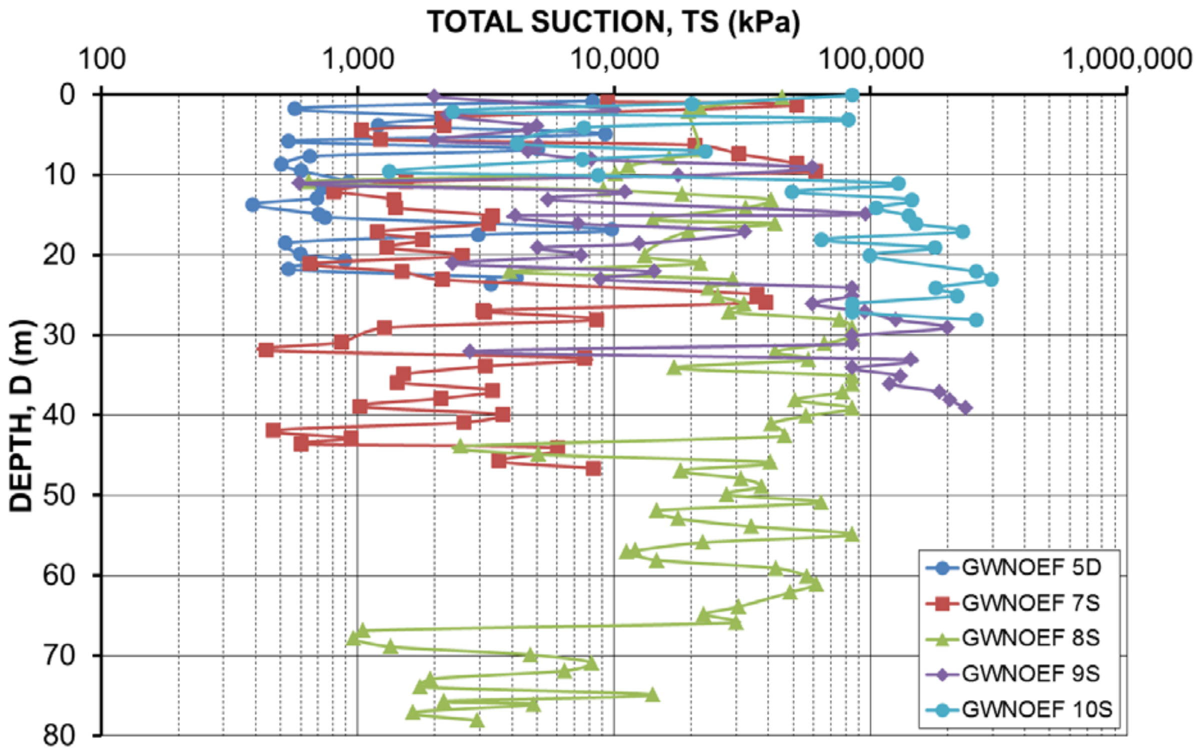

Figure 13 Profiles of total suction (to $\log_{10}$) with depth

3.3.3 Electrical conductivity and osmotic suction profiles

The osmotic suction is difficult to measure directly, but can be estimated from the electrical conductivity, which is generally measured in a paste. The 5 (deionised water):1 (solids) (mass basis) paste electrical conductivity profiles with depth to natural and semi-log$_{10}$ scales, respectively, are shown in Figures 14 and 15. The 5:1 paste electrical conductivity is seen to be generally lowest for NAF waste rock and highest for PAF waste rock, reflecting the greater salinity of the PAF waste rock due to the oxidation products formed.

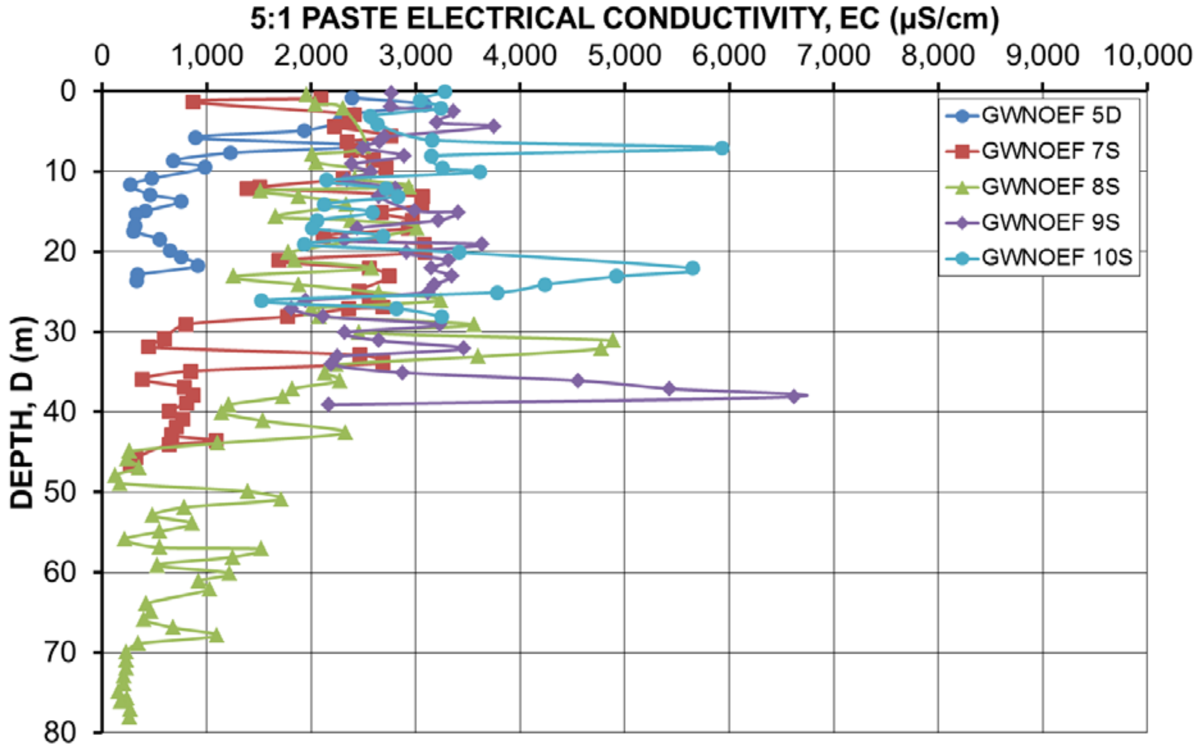

Figure 14 Profiles of 5:1 paste electrical conductivity (to a natural scale) with depth

The paste electrical conductivity should be converted to an actual electrical conductivity of the pore fluid within the sample. This requires that the pore fluid be extracted to enable the electrical conductivity probe to be used. A relationship may be determined between the water:solids mass ratio or the gravimetric moisture content and the ratio of the actual or corrected electrical conductivity to the measured electrical conductivity $EC_{corrected}/EC_{measured}$. This was done for PAF 1 and PAF 2 samples, giving the relationships shown in Figures 16 and 17. Using these relationships, the corrected electrical conductivity profiles with depth to natural and semi-log$_{10}$ scales, respectively, are shown in Figures 18 and 19. The corrected electrical conductivities are about twice the 5:1 paste values.

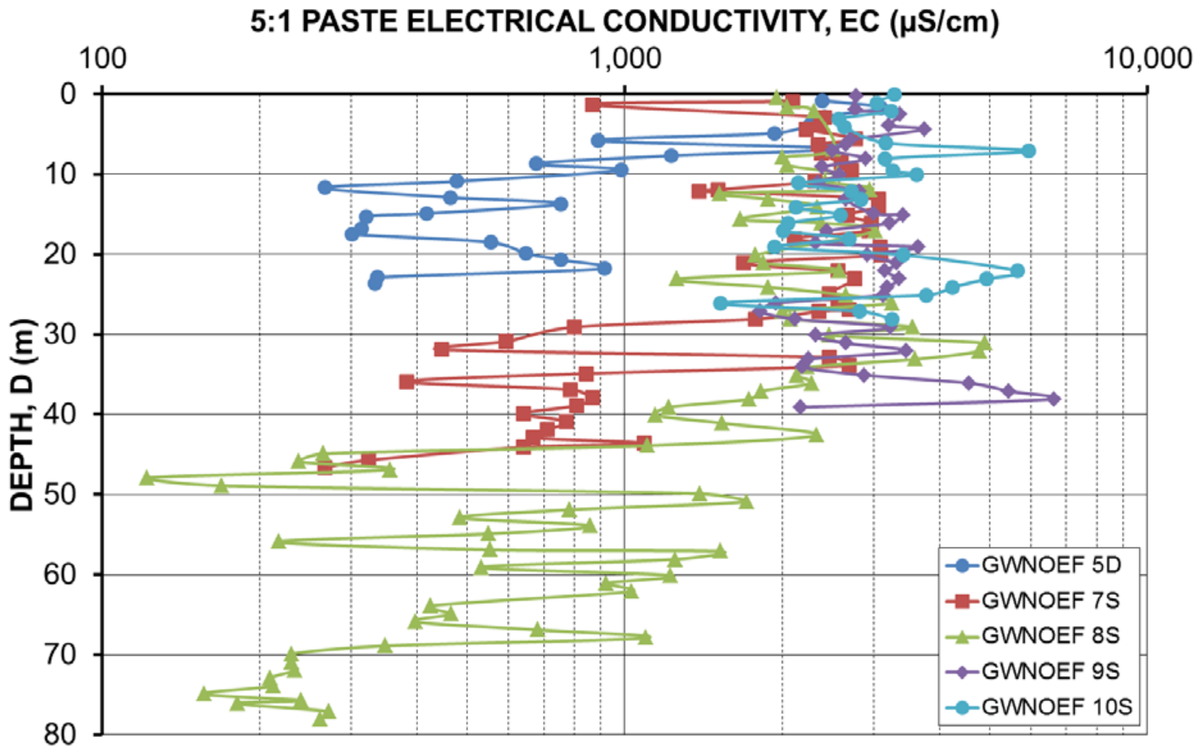

Figure 15 Profiles of 5:1 paste electrical conductivity (to $\log_{10}$) with depth

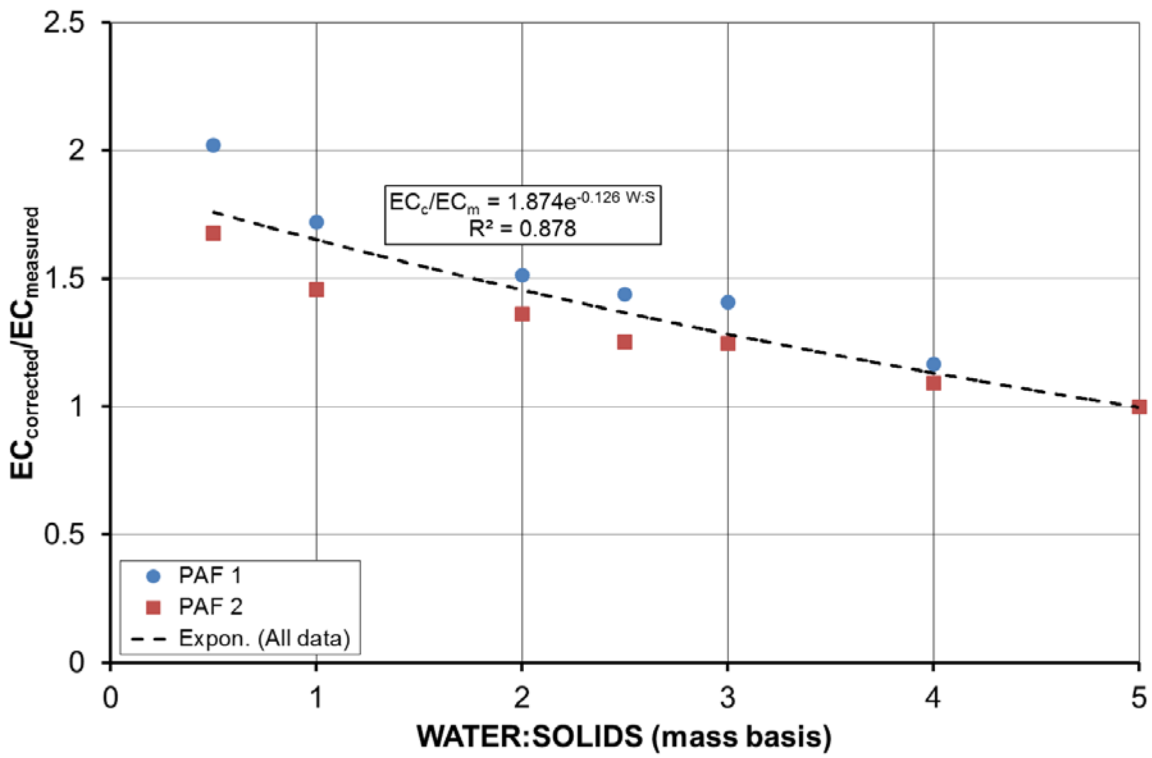

Figure 16 $EC_{corrected}/EC_{measured}$ versus water:solids (mass basis) for PAF 1 and PAF 2 samples

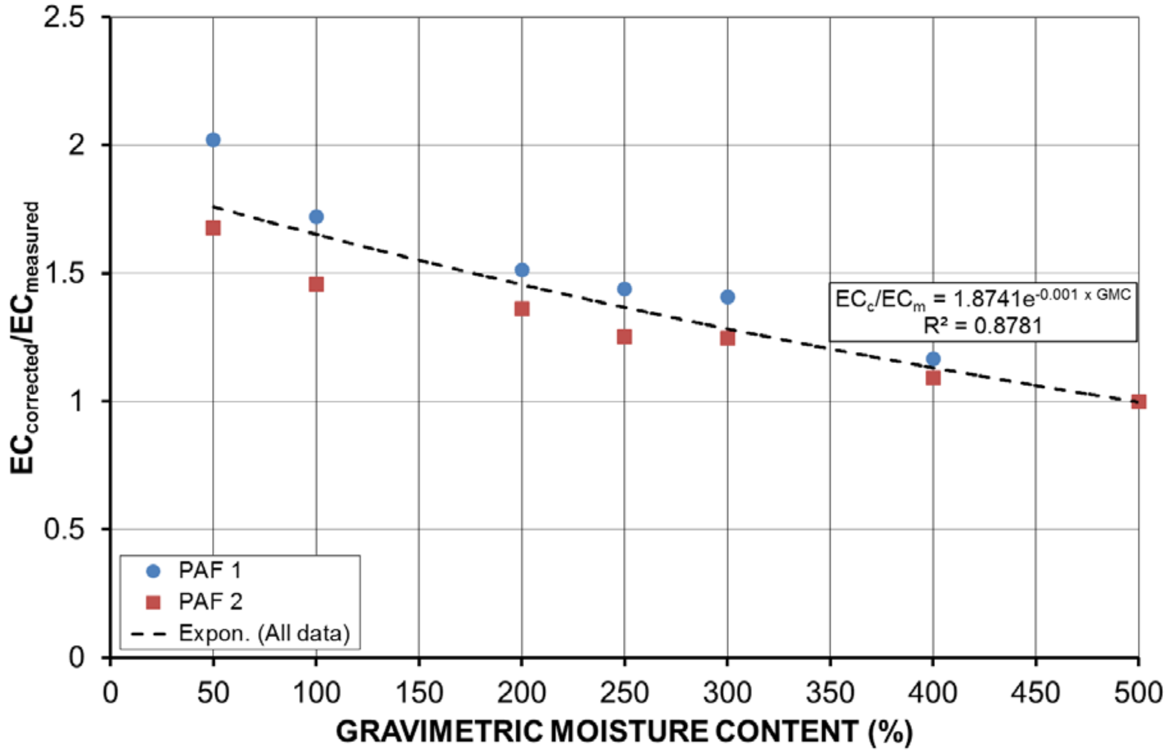

Figure 17 $EC_{corrected}/EC_{measured}$ versus gravimetyric moisture content for PAF 1 and PAF 2 samples

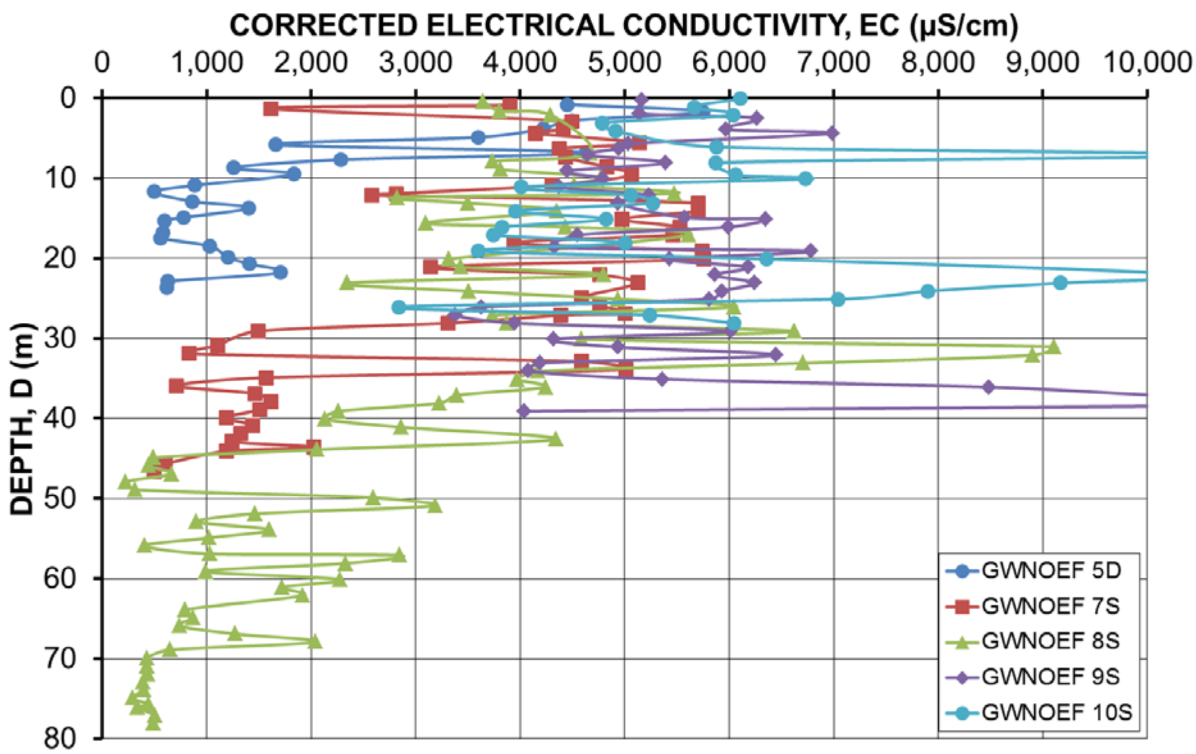

Figure 18 Profiles of corrected electrical conductivity (to a natural scale) with depth

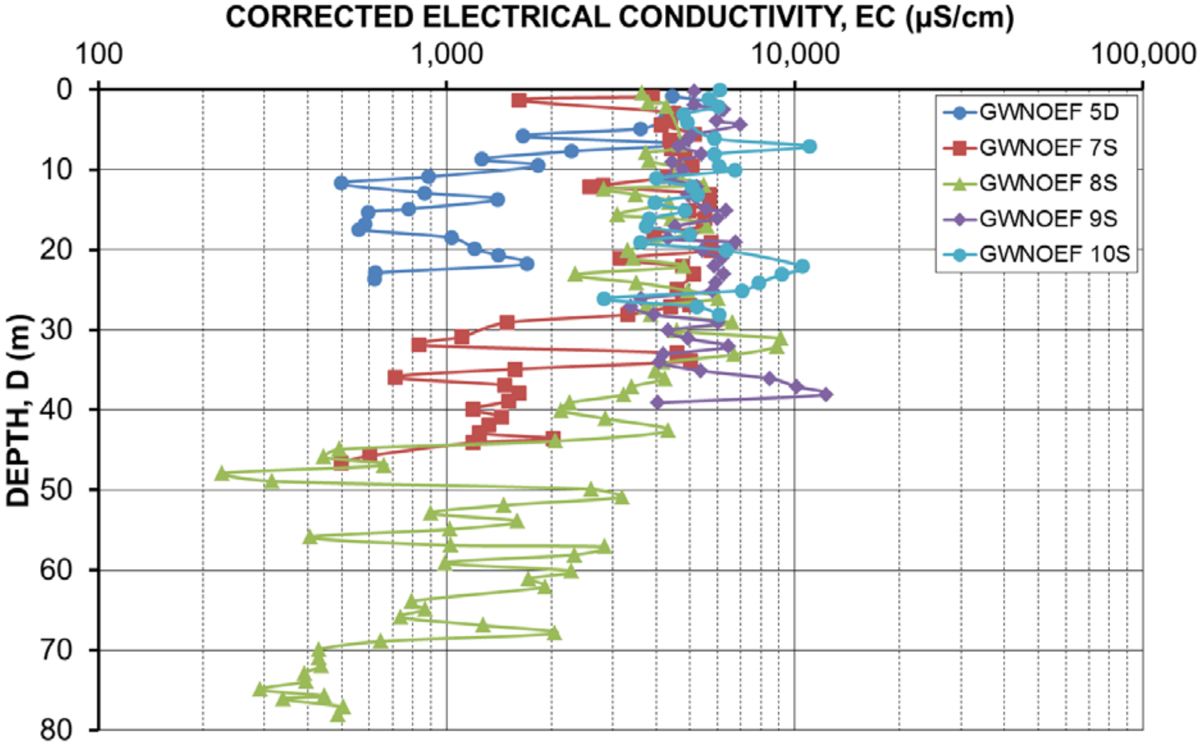

Figure 19 Profiles of corrected electrical conductivity (to $\log_{10}$) with depth

Using these corrected electrical conductivities, the osmotic suction may be estimated using the relationship derived by the US Department of Agriculture (1956), which is reproduced in Figure 20.

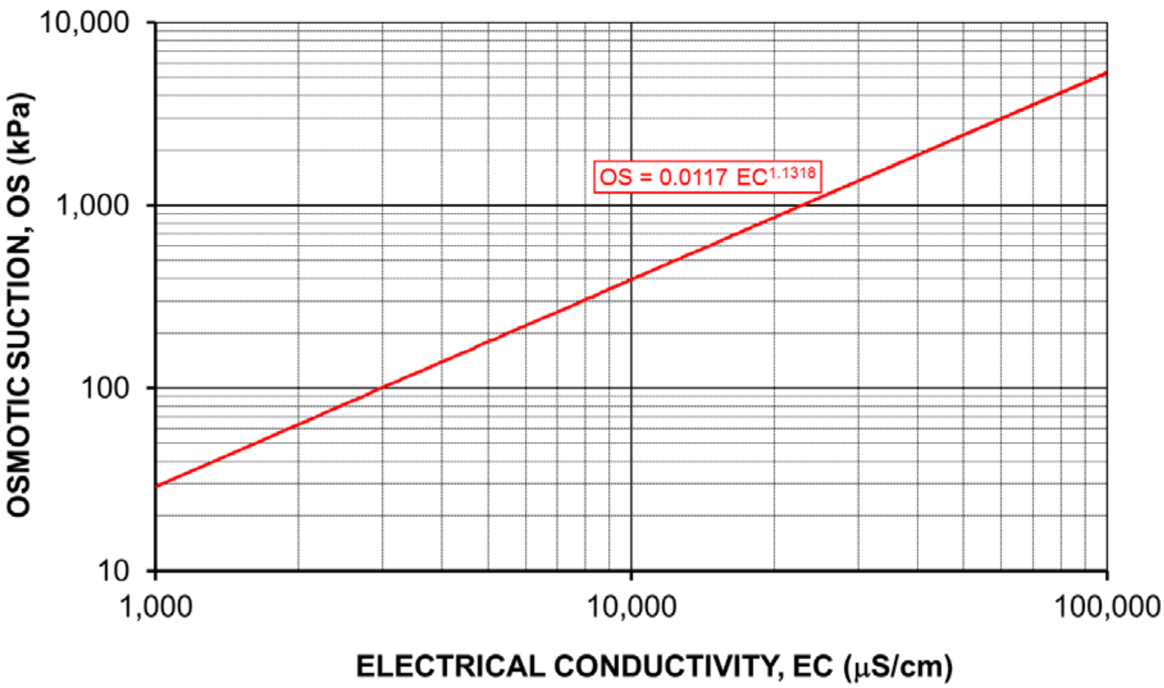

Figure 20 Relationship between osmotic suction and electrical conductivity (after USDA, 1954)

The measured total suction (TS) and estimated osmotic suction (OS) profiles with depth to semi-$\log_{10}$ scales, are shown in Figure 21. The matric suction is then determined by subtracting the estimated osmotic suction from the measured total suction. Profiles with depth of osmotic suction/total suction, expressed as a percentage, are shown in Figure 22. On average, the osmotic suction is only 4% of the total suction, which is considered to be generally within experimental error. Hence, the total suction is mainly matric suction.

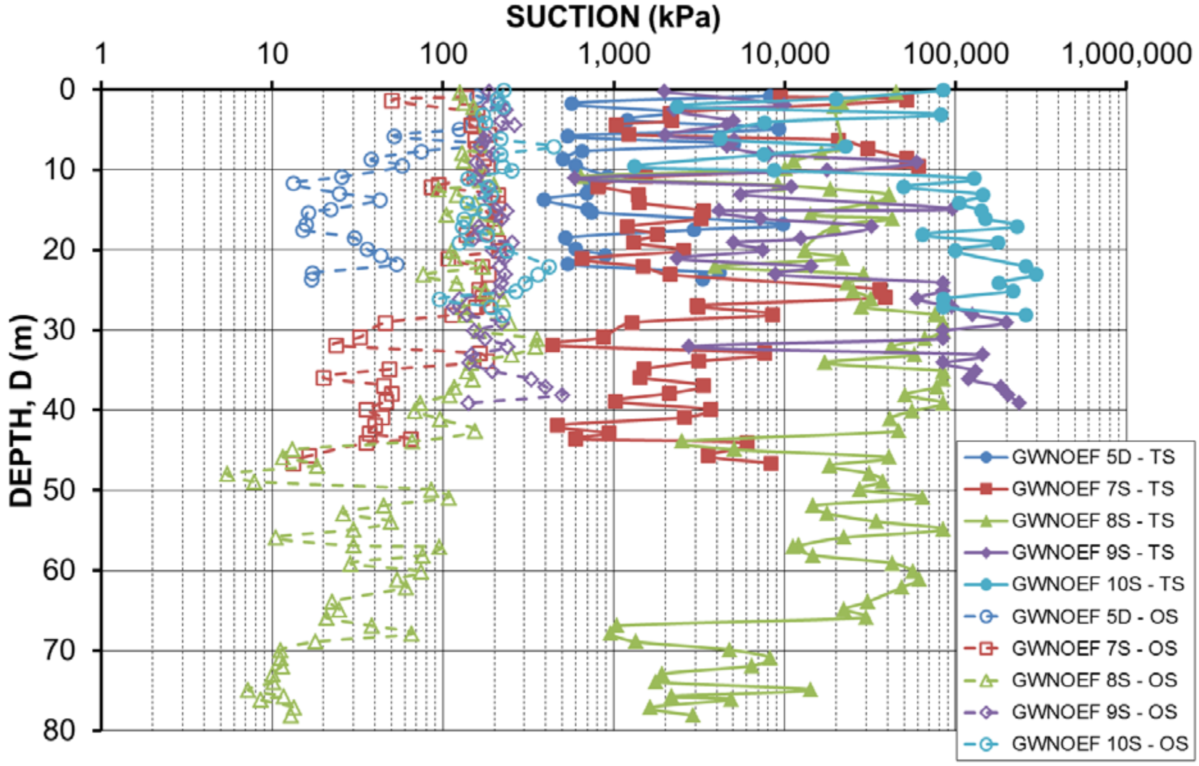

Figure 21 Profiles of measured total suction and estimated osmotic suction (to $\log_{10}$) with depth

3.3.4 Estimated Soil Water Characteristic Curves

Combining the gravimetric moisture content data from Figure 9 and the total suction data from Figure 13, SWCCs may be estimated for each of the sonic drill holes, together with an overall SWCC, as shown in Figure 23 (data points only) and 24 (data points and best-fit SWCCs). Figure 24 suggests that full saturation corresponds to a gravimetric moisture content of about 25%, which is supported by the measured data in Figures 9 and 11. The air-entry value is perhaps as high as a total suction of about 300 kPa.

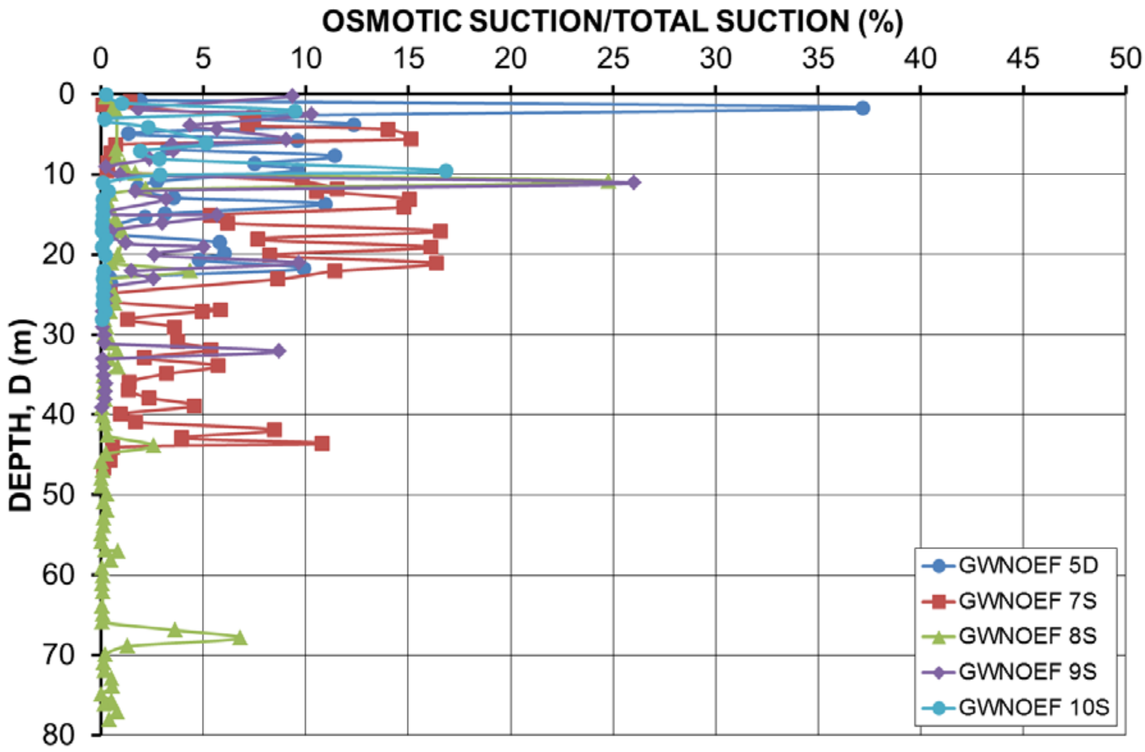

Figure 22 Profiles of osmotic suction/total suction, expressed as a percentage, with depth

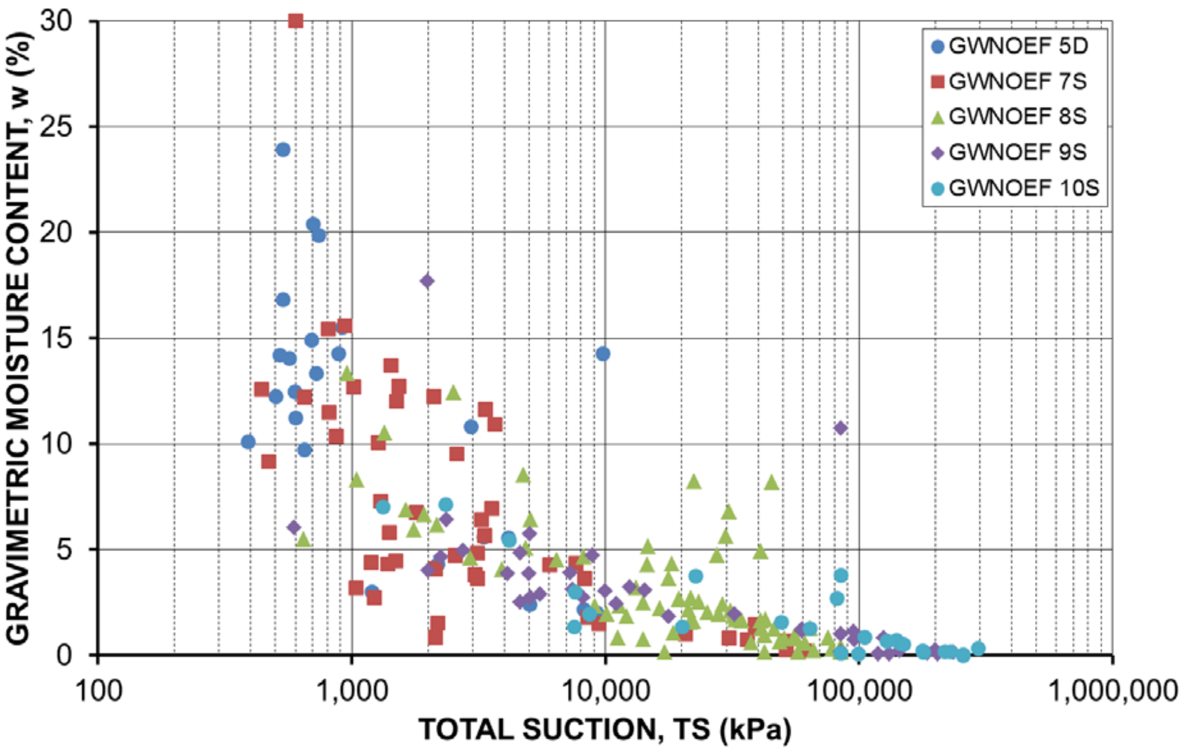

Figure 23 SWCC data

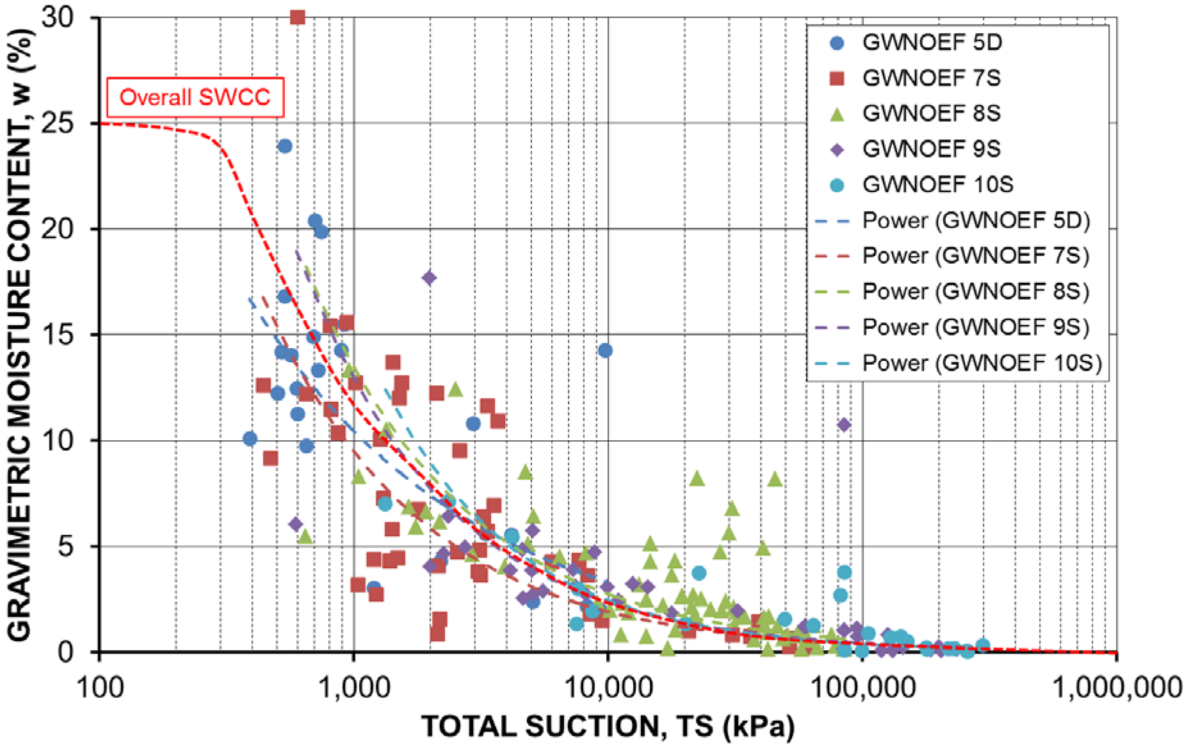

Figure 24 SWCC data and best-fit curves

3.3.5 Measured pH

The measured pH profiles with depth are shown in Figure 25 to be generally within the acceptable range from 6 to 9.

3.3.6 Measured % fines

None of the samples was fine-grained and, as a consequence, only limited sieving through the 0.075 mm sieve was undertaken, giving the data shown in Figure 26. There is no consistent relationship between gravimetric moisture content and % fines. The NAF waste rock might be expected to be finer-grained than the fresher PAF waste rock.

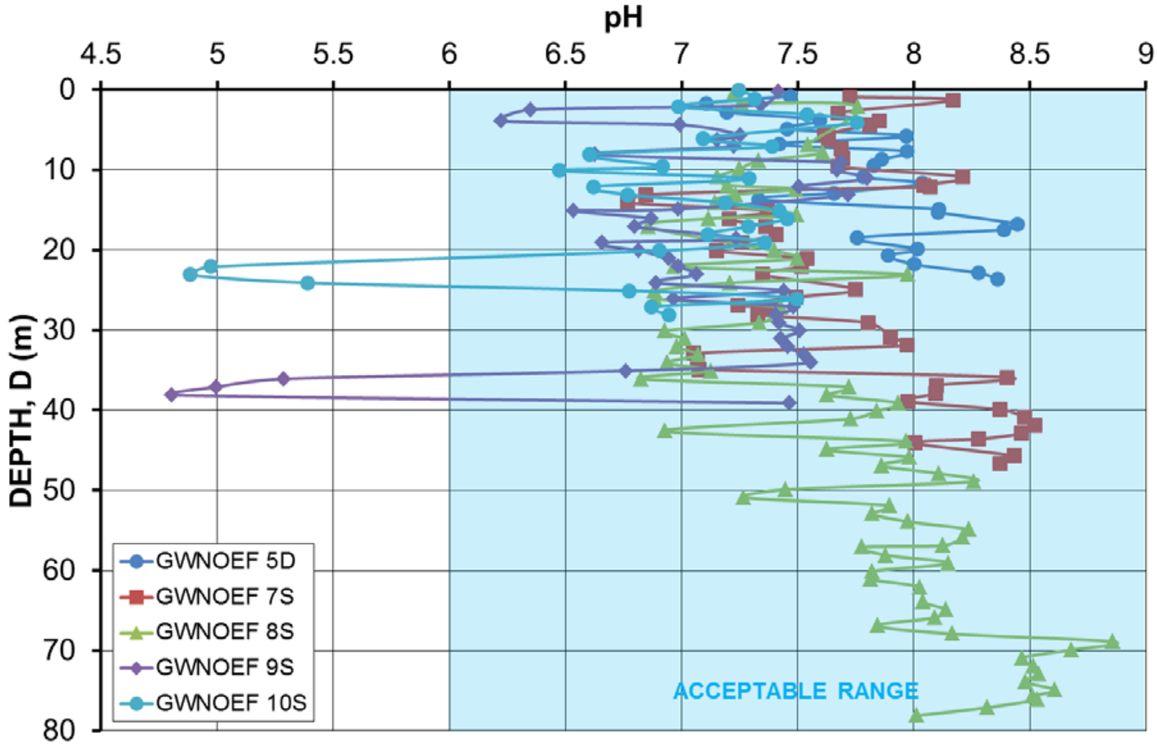

Figure 25 Profiles of pH with depth

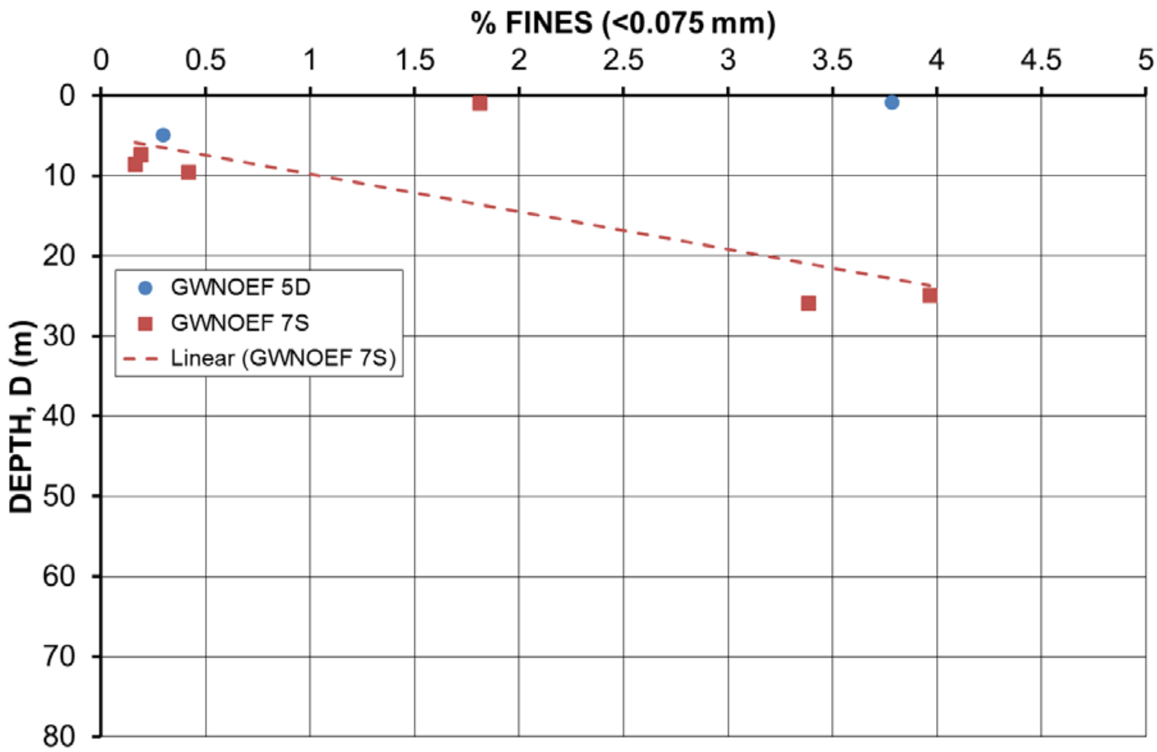

Figure 26 % fines versus depth

APPENDIX B – Curriculum Vitae

Professor David J Williams

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QUALIFICATIONS

1979	PhD, Soil Mechanics	University of Cambridge, England
1975	BE (Hons I), Civil Engineering	Monash University, Australia

AWARDS/DISTINCTIONS/FELLOWSHIPS

1996	Japan Society for the Promotion of Science Fellow
1995	The University of Queensland Collaborative Research Travel Grant
1995	Australian Minerals and Energy Environment Foundation (AMEEF) Travelling Scholarship
1993	Australian Research Fellow (Industry)
1992	AMEEF Environmental Excellence Award (Individual)
1990	Masuda Fellow for Collaborative Research in Japan, Jan-Feb
1989	The University of Queensland Collaborative Research Travel Grant

MEMBERSHIPS

From 1980	Member, Institution of Engineers, Australia
From 1980	Member, Australian Geomechanics Society
From 1984	Member, Queensland Committee, Australian Geomechanics Society, Chair in 1986
1986-1987	Member, National Committee, Australian Geomechanics Society
2007-2008	

EMPLOYMENT HISTORY

2007 – Present
Golder Professor of Geomechanics
Director Geotechnical Engineering Centre
School of Civil Engineering
The University of Queensland

1994 – 2007	Associate Professor of Geomechanics Department of Civil Engineering The University of Queensland
1990 – 1994	Senior Lecturer in Geomechanics Department of Civil Engineering The University of Queensland
1983 – 1989	Lecturer in Geomechanics Department of Civil Engineering The University of Queensland
1980 – 1983	Geotechnical Engineer Melbourne and Brisbane Golder Associates Pty Ltd
1979 – 1980	Engineer Country Roads Board (CRB) of Victoria
1976 – 1979	Research Student University of Cambridge, England
1972 – 1976	Engineer, Cadet Engineer, CRB, Victoria

SUMMARY OF CONSULTING COMMISSIONS

Board Memberships

- Member of Northern Territory EPA Board, from 2012 to 2014

Peer Reviews of Major Projects

- Sole Independent Expert Geotechnical Reviewer for Rio Tinto Alcan Gove Residue Disposal Area from 2015
- Sole Independent Expert Geotechnical Reviewer and Annual Dam Inspections for QAL Residue Disposal Area and Ash Dams from 2013
- Sole Independent Expert Geotechnical Reviewer for Rio Tinto Alcan Yarwun Residue Management Area from 2013
- Led International Peer Review for the South Deposit TSF at Savage River Mine in Tasmania in 2012/13
- Sole Independent Expert Geotechnical Reviewer for Rio Tinto Alcan Weipa Tailings Storage Facilities in 2012 and 2014
- Peer Review of Harvey Creek Non-Erodable Waste Rock Dump Design for Ok Tedi Mining Limited in 2010/11
- Member of Expert Peer Review Team for Rio Tinto Alcan Weipa Tailings Storage Facilities from 2009
- Member of the International Technical Advisory Group reporting to the South Australian Government on Rehabilitation of Brukunga Pyrite Mine from 2007
- Led International Peer Reviews for the Savage River Rehabilitation Project in Tasmania in 2002, 2005, 2009 and 2013

- Led International Peer Review on handling acid generating waste rock dumping and dump closure strategies at Cadia Hill Gold Mine in New South Wales in 2002/3
- Member of the Peer Review Team for Stage 2 of the Stuart Oil Shale Project at Gladstone in Queensland in 2004
- Peer Reviewer of the rehabilitation of the San Manuel Copper Mine tailings facility in Arizona, USA in 2004
- Member of the 2005 Peer Review Team that reviewed future red mud disposal, containment and rehabilitation at QAL at Gladstone in Queensland in 2005
- Geotechnical Reviewer of the breach of the co-disposal dam at Burton Coal in Queensland in 2005
- Peer Reviewer of the conceptual closure plan for Worsley Alumina red mud storage in Western Australia in 2005
- Peer Reviewer for waste rock dump covers for Century Mine in North Queensland from 2007
- During 2006, David was an Expert Advisor to the EIS team for the Olympic Dam Expansion Project in South Australia, providing expert input on disposal, hydrology and closure issues for both waste rock and tailings

Expert Witness

- Expert witness through Corrs Chambers Westgarth Lawyers, in relation to coal washery rejects used as filling for residential sub-division purposes
- Expert witness through McCullough Robertson Lawyers, in relation to the failure of a concrete arch reclaim tunnel beneath a coal stockpile
- Expert witness in relation to professional misconduct cases brought by the Queensland Professional Engineers Registration Board
- Numerous expert witness commissions related to residential and commercial building footing failures and slope instability

Consultancies

Professor David John Williams is widely sought for his expert input, in particular to mine waste disposal and mine site rehabilitation and remediation at operating mines throughout Australia and overseas. In Australia, he has consulted on numerous coal mines throughout Queensland and New South Wales; on Red Dome Gold Mine closure, Kidston closure, Osborne waste disposal, Ivanhoe Cloncurry mine closure, Phosphate Hill gypsum disposal, QERL processed waste storage facility closure, and Century Zinc Mine waste rock dumping in Queensland; Cadia Hill Gold Mine waste rock dumping and dump closure in New South Wales; Mt Morgans Gold Mine co-disposal, WMC Resources" nickel operations tailings closure and Minara heap leaching in Western Australia; waste disposal issues at the Ballarat East and Heathcote gold mines in Victoria; and a review of ARD treatments at Savage River Mine in Tasmania. Overseas he has consulted on tailings depositional design and water balance for the Kori Kollo Mine in Bolivia, a review of co-disposal of tailings and waste rock at Porgera Gold Mine and the closure of Misima Gold Mine in PNG, waste disposal design for the Goro Nickel project in New Caledonia, and advice on co-disposal for the Martabe Project in Indonesia.

David has been involved in material characterisation testing and the design of numerous mine waste covers throughout Australia, and the design, installation and monitoring of lysimeters and mine waste covers at Kidston Gold Mines, WMC Resources" Mt Keith Nickel Operations, QERL"s Stuart Oil Shale Project, a large-scale trial waste rock dump at Cadia Hill Gold Mine, and a large-scale trial tailings cell at Jubilee Nickel Mine.

David has been invited to visit numerous mining regions and individual mines throughout Australia, and in Canada, the USA, Brazil, South Africa, UK, China, Chile, PNG, New Caledonia, Spain and Mozambique.

MAJOR RESEARCH ACHIEVEMENTS

From 1989, Professor Williams carried out research under NERDDC and ACARP Projects on the characterisation of the deposit formed on the pumped co-disposal of combined washery wastes, which has since been adopted at numerous coal mines in Australia and Indonesia.

From 1996, David developed the store/release cover system suited to seasonally dry climates, for application to covering acid generating rock dumps at Kidston Gold Mine in north Queensland, and has had a long-term involvement in researching and monitoring this cover system, as evidenced by his numerous papers on his research on this topic. The store/release cover system on the tops of the Kidston rock dumps has been shown to limit percolation to less than 1% of rainfall, and to support a sustainable vegetation cover comparable to that occurring along water courses in the area. He was also involved in the development of a rehabilitation strategy for the side slopes of the rock dumps at Kidston designed to maximise geotechnical and erosional stability while promoting vegetation, and analysed the wetting up by rainfall infiltration and subsequent drain-down of and seepage from the rock dumps. Store/release covers have now been adopted at numerous mine sites in dry climates worldwide.

From 1999 to 2001, David led ACARP Project C8039 to develop a risk assessment and cost-effectiveness analysis for the rehabilitation of Bowen Basin coal mine spoil. The results of the project were reported in a Literature Review and Commentary and Project Final Report, plus a spreadsheet-based risk assessment and cost-effectiveness analysis, available at: www.uq.edu.au/civil/. In 2006, David undertook a closure study for Xstrata"s new Rolleston Coal Project in the Bowen Basin Coalfields.

David has since 2000 been involved in the closure design for the waste rock dump at Cadia Hill Gold Mine in New South Wales, including studies on the use of mixtures of benign trafficked rock and tailings as an alternative cover material, to overcome the shortage of suitable natural materials. In 2002/3, he led an international peer review of the rock dumping operation and closure plan. In 2004, David was successful in an ARC Linkage grant application with Cadia totalling over $ 700,000 over 3 years, which has led to the construction of a 15 m high, world-class, demonstration, instrumented rock dump covering 7,000 m^2. The instrumentation includes a full weather station, 24 lysimeters at the base of the dump to monitor seepage, lysimeters on the top surface to monitor rainfall infiltration and three store/release trial covers constructed using natural and mine waste materials. To date it has shown that about 70% of the rainfall incident on the traffic-compacted top of the dump infiltrates, with the majority going into storage within the dump during the first year, and only small amounts percolating to the base of the dump. The behaviour of the cover trials has to date been dominated by the moisture state at which they were constructed. Monitoring of the instrumented rock dump is expected to continue for at least 10 years.

From 2000 to 2003, David was a principal researcher into the physical and geochemical nature of acid generating waste rock dumps in Southern Carolina, USA (Rio Tinto"s Ridgeway Mine) and Sudbury, Canada (Inco"s Whistle Dump), sampled as they were being excavated and moved to a pit.

From 2001 to 2005, David led an ARC Spirt research project with industry partner WMC Resources focussed on an assessment of the long-term seepage and runoff from mine tailings storage facilities, to facilitate lease surrender. This included the monitoring of trial covers on tailings over the duration of the project and large-scale laboratory column testing and numerical analyses. Natural salt pan and rocky slope analogues under the same climatic and similar geochemical conditions were also studied to point to sustainable approaches for rehabilitating the tailings storage facilities.

From 2010, David has led two ACARP Projects, C19022 and C20047, investigating the settlement and stability of high coal mine spoil, and the behaviour of problematic clay-rich coal mine tailings.

David has been sponsored by mining companies and consultants to visit numerous mining regions and mine sites worldwide, both to impart and extend his knowledge. Since 2000, he has developed a relationship with the International Network for Acid Prevention (INAP), and has contributed to INAP-sponsored research and development projects and workshops involving mine sites in the USA, Canada, Australia and PNG.

Research funding has totalled over $7 million, including funding from ARC, ARC-SPIRT, ARC Linkage, NERDDC, ACARP-AMIRA, ACARP, MIM CRA-ATD, Kidston Gold Mines, BHP Coal and WMC Resources, Cadia Holdings, Jubilee Mines NL. Professor Williams has over 250 refereed publications, with about two-thirds of them in the mine waste field.

SELECTED PUBLICATIONS

Book Chapters

1. **Williams, D.J.** (2005). Chapter 17: Placing covers on soft tailings. In: *Ground Improvement-Case Histories*, 491-512. Eds B. Indraratna and Chu Jian. Elsevier.
2. **Williams, D.J.** (2001). Chapter 30: Assessment of Embankment Parameters. In: *Slope Stability in Surface Mining*, 275-284. Eds W.A. Hustrulid, M.J. McCarter and D.J.A Van Zyl. Society for Mining, Metallurgy, and Exploration, Inc., Littleton, Colorado, USA.
3. **Williams, D.J.** (1996). Chapter 7: Minimisation and Management of Solid Wastes. In: *Environmental Management in the Australian Minerals and Energy Industry*, 157-188. Ed D.R. Mulligan. Sydney, UNSW Press in association with Australian Minerals and Energy Environment Foundation, 1996.

Selected Refereed Journal Articles

1. Serati, M., Alehossein, H. and **Williams, D.J.** (2015). Estimating the tensile strength of super hard brittle materials using truncated spheroidal specimens. *Journal of the Mechanics and Physics of Solids*, **78**, 123-140.
2. Zbik, M.S., **Williams, D.J.**, Song, Y-F and Wang, C.C. (2015). How the hydro-gel flocculation microstructure changes. *Colloids and Surfaces A: Physicochemical and Engineering Aspects*, **469**, 11-19.
3. Zbik, M.S., **Williams, D.J.**, Song, Y.-F. and Wang, C.-C. (2015). Smectite clay microstructural behaviour on the Atterberg limits transition. *Colloids and Surfaces A: Physicochemical and Engineering Aspects*, **467**, 89-96.
4. Li, Y., Topal, E. and **Williams, D.J.** (2014). Optimisation of waste rock placement using mixed integer programming. *Transactions of the Institutions of Mining and Metallurgy, Section A: Mining Technology*, **123**(4), 220-229.
5. Poulsen, B.A., Shen, B., **Williams, D.J.**, Huddlestone-Holmes, C., Erarslan, N. and Qin, J. (2014). Strength reduction on saturation of coal and coal measures rocks with implications for coal pillar strength. *International Journal of Rock Mechanics and Mining Sciences*, **71**, 41-52.
6. Erarslan, N., Alehossein, H. and **Williams, D.J.** (2013). Tensile fracture strength of Brisbane tuff by static and cyclic loading tests. *Rock Mechanics and Rock Engineering, Online First*, 1-17.
7. Galindo-Torres, S.A., Pedroso, D.M., **Williams, D.J.** and Muhlhaus, H.B. (2013). Strength of non-spherical particles with anisotropic geometries under triaxial and shearing loading configurations. *Granular Matter*, **15**(5), 531-542.
8. Serati, M., Alehossein, H. and **Williams, D.J.** (2013). 3D elastic solutions for laterally loaded discs: generalised Brazilian and Point Load tests. *Rock Mechanics and Rock Engineering, Online First*, 1-15.
9. Dight, P.M., Douglas, B., Henley, K., Lumley, G., McAree, P.R., Miller, D., Saydam, S., Topal, E., Wesseloo, J. and **Williams, D.J.** (2013). Developments in open cut mining. *Australasian Mining and Metallurgical Operating Practices, 3rd ed.,* 47-80. Australasian Institute of Mining and Metallurgy.
10. Topal, E. and **Williams, D.J.** (2013). Mine waste rock management. *Australasian Mining and Metallurgical Operating Practices, 3rd ed.*, 76-77. Australasian Institute of Mining and Metallurgy.
11. Eraslan, N. and **Williams, D.J.** (2013). Mixed-mode fracturing of rocks under static and cyclic loading. *Rock Mechanics and Rock Engineering*, **46**(5), 1035-1052.
12. **Williams, D.J.** and Kho, A.K. (2013). Laboratory geotechnical characterisation of scalped coal mine spoil. *Australian Geomechanics Journal*, **48**(1), 101-110.
13. **Williams, D.J.** (2012). Some mining applications of unsaturated soil mechanics. *Geotechnical Engineering*, **43**(1), 83-98.
14. Erarslan, N. and **Williams, D.J**. (2012). Mechanism of rock fatigue damage in terms of fracturing modes. *International Journal of Fatigue*, **43**, 76-89.
15. Erarslan, N. and **Williams, D.J.** (2012). The damage mechanism of rock fatigue and its relationship to the fracture toughness of rocks. *International Journal of Rock Mechanics and Mining Sciences*, **56**, 15-26.
16. Galindo-Torres, S.A., Pedroso, D.M., **Williams, D.J.** and Li, L. (2012). Breaking processes in three-dimensional bonded granular materials with general shapes. *Computer Physics Communications*, **183**(2), 266-277.
17. Liang, Z.Z., Xing, H., Wang, S.Y., **Williams, D.J.** and Tang, C.A. (2012). A three-dimensional numerical investigation of the fracture of rock specimens containing a pre-existing surface flaw. *Computers and Geotechnics*, **45**, 19-33.
18. Erarslan, N., Liang, Z.Z. and **Williams, D.J.** (2012). Experimental and numerical studies on determination of indirect tensile strength of rocks. *Rock Mechanics and Rock Engineering*, **45**(5), 739-751.
19. Pedroso, D.M., **Williams, D.J.** (2011). Automatic calibration of soil-water characteristic curves using genetic algorithms. *Computers and Geotechnics*, **38**(3), 330-340.
20. Pedroso, D. and **Williams, D.J.** (2010). A novel approach for modelling soil–water characteristic curves with hysteresis. *Computers and Geotechnics*, **37**(3), 374-380.
21. Liu, H.Y., Small, J.C., Carter, J.P. and **Williams, D.J.** (2009). Effects of tunnelling on existing support systems of perpendicular crossing tunnels. *Computers and Geotechnics*, **36**:5, 880-894.
22. **Williams, D.J.** (2002). Engineering closure of an open pit gold operation in a semi-arid climate. *International Journal of Surface Mining and Reclamation, Special Edition on Mining and the Environment*, 35-50.

23. **Williams, D.J.** (2001). Prediction of erosion from steep mine slopes. *International Journal of Environmental Management and Health*, **12:1**, 35-50.
24. Morris, P.H. and **William**s, D.J. (2000). A revision of Blight"s model of field vane testing. *Canadian Geotechnical Journal,* **37**, 1089-1098.
25. Morris, P.H. and **William**s, D.J. (1999). Segregation of co-disposed coal mine washery wastes. *Canadian Institute of Mining Bulletin*, **92**, 72-76.
26. Rassam, D.W. and **Williams, D.J.** (1999). A numerical study of steady state evaporative conditions applied to mine tailings. *Canadian Geotechnical Journal*, **36**, 640-650.
27. Rassam, D.W. and **Williams, D.J.** (1999). Bearing capacity of desiccated tailings. *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, **125:7**, 600-610.
28. Rassam, D.W. and **Williams, D.J.** (1999). Three-dimensional effects on slope stability of high waste rock dumps. *International Journal of Surface Mining, Reclamation and Environment*, **13**, 19-24.
29. Rassam, D.W. and **Williams, D.J.** (1999). Unsaturated hydraulic conductivity of mine tailings under wetting and drying conditions. *ASTM Geotechnical Testing Journal*, **2:2**, 138-146.
30. Mahalinga-Iyer, U. and **Williams, D.J.** (1997). Properties and performance of lateritic soil in road pavements. *Engineering Geology*, **46:2**, 71-80.
31. Morris, P.H. and **Williams, D.J.** (1997). Co-disposal of washery wastes at Jeebropilly Colliery, Queensland, Australia. *Transactions IMM, A: Mining Industry*, **106**, A25-A29.
32. Morris, P.H. and **Williams, D.J.** (1997). Hydraulic sorting of co-disposed coarse and fine coal wastes. *Transactions IMM, C: Mineral Processing*, **106**, C21-C26.
33. Morris, P.H. and **Williams, D.J.** (1997). Results of field trials of co-disposal of coarse and fine coal wastes. *Transactions IMM, A: Mining Industry*, **106**, A38-A41.
34. Naderian, A.R. and **Williams, D.J.** (1997). Bearing capacity of open-cut coal-mine backfill materials. *Transactions IMM, A: Mining Industry*, **106**, A30-A33.
35. Morris, P.H. and **Williams, D.J.** (1996). Prediction of mine tailings delta profiles. *Transactions IMM, A: Mining Industry*, **105**, A63-A68.
36. Naderian, A.R. and **Williams, D.J.** (1996). Simulation of groundwater rise and its effects on settlements of open-cut coal mine back-fills. *International Journal of Surface Mining, Reclamation, and Environment*, **10**, 83-89.
37. Naderian, A.R., **Williams, D.J.** and Clark, I.H. (1996). Numerical modelling of settlements in back-filled open-cut mines. *International Journal of Surface Mining, Reclamation, and Environment*, **10**, 25-29.
38. Mahalinga-Iyer, U. and **Williams, D.J.** (1995). Unsaturated strength behaviour of compacted lateritic soils. *Geotechnique*, **45:2**, 317-320.
39. Zou, J.Z., **Williams, D.J.** and Xiong, W.L. (1995). Search for critical slip surfaces based on finite element method. *Canadian Geotechnical Journal*, **32:2**, 233-246.
40. Morris, P.H., Graham, J. and **Williams, D.J.** (1994). Depths of cracks in drying soils using elastic fracture mechanics. *ASCE Geotechnical Special Publication No. 43, Fracture Mechanics Applied to Geotechnical Engineering*, 40-53.
41. Morris, P.H. and **Williams, D.J.** (1994). Effective stress vane shear strength correction factor correlations. *Canadian Geotechnical Journal*, **31**, 335-342.
42. Mahalinga-Iyer, M. and **Williams, D.J.** (1993). Consolidation and shear strength properties of a lateritic soil. *Engineering Geology*, **38**, 53-63.
43. Mahalinga-Iyer, M. and **Williams, D.J.** (1993). Road construction using lateritic soil. *Engineering Geology*, **37**, 199-209.
44. Morris, P.H. and **Williams, D.J.** (1993). A new model of vane shear strength testing in soils. *Geotechnique*, **43:3**, 489-500.
45. Wong, K.Y. and **Williams, D.J.** (1993). Methods of interpreting structural incompatibility in bored pier uplift test. *ASCE, Journal Geotechnical Engineering Division*, **GT119:12**, 1892-1909.
46. Morris, P.H., Graham, J. and **Williams, D.J.** (1992). Cracking in clays undergoing drying. *Canadian Geotechnical Journal*, **29**, 263-277.
47. **Williams, D.J.** and Kuganathan, V. (1992). Co-disposal of fine and coarse grained coal mine washery wastes by combined pumping. *International Journal of Environmental Issues in Minerals and Energy Industry*, 53-58.
48. **Williams, D.J.** and Sibley, J.W. (1992). Behaviour at the shrinkage limit of clay undergoing drying. *ASTM, Geotechnical Testing Journal*, **15:3**, 217-222.
49. Morris, P.H. and **Williams, D.J.** (1990). Generalised calibration for the nuclear moisture/density gauge. *ASTM, Geotechnical Testing Journal*, **13:1**, 24-35.

50. Morris, P.H. and **Williams, D.J.** (1990). Sample size selection for laboratory calibration of subsurface neutron moisture gauges. *ASTM, Geotechnical Testing Journal*, **14:1**, 71-77.
51. Sibley, J.W. and **Williams, D.J.** (1990). A new filter material for measuring soil suction. *ASTM, Geotechnical Testing Journal*, **13:4**, 381-383.
52. **Williams, D.J.** (1989). Geotechnical input to a major bridge project. *ASCE, Journal Geotechnical Engineering Division*, **GT115:3**, 322-339.
53. Sibley, J.W. and **Williams, D.J.** (1989). A procedure for determining volumetric shrinkage of an unsaturated soil. *ASTM, Geotechnical Testing Journal*, **12:3**, 181-187.
54. **Williams, D.J.** and Morris, P.H. (1989). Comparison of two models for the sub-aerial deposition of mine tailings. *Transactions IMM, A: Mining Industry*, **98**, A73-A77.
55. **Williams, D.J.** (1988). Potential engineering risks in the earthquake hazard to the east coast of Queensland. *IEAust, Civil Engineering Transactions*, **CE30/5**, 307-317.
56. **Williams, D.J.** and Parry, R.H.G. (1985). Experimentally determined distribution of stress around a horizontally loaded model pile in dense sand. *IEAust, Civil Engineering Transactions*, **CE27/3**, 263-268.
57. **Williams, D.J.** and Walker, L.K. (1985). Laboratory and field strength of mine waste rock. *IEAust, Civil Engineering Transactions*, **CE27/3**, 299-305.

Selected Refereed Conference Papers

1. **Williams, D.J.** (2014). Theme Lecture: Applying soil mechanics principles to tailings dewatering, densification and strengthening. *Proceedings of Tailings and Mine Waste 2014, Keystone, CO, USA, 5-8 October 2014*, 10 p. **(Invited)**.
2. Shokouhi, A., **Williams, D.J.** and Kho, A.K. (2014). Settlement and collapse behaviour of coal mine spoil and washery wastes. *Proceedings of Tailings and Mine Waste 2014, Keystone, CO, USA, 5-8 October 2014*, 10 p.
3. Galindo Torres, S., Pedroso, D., **Williams, D.J.** and Muhlhaus, H. (2014). An analysis of the strength of anisotropic granular assemblies via discrete methods. *Proceedings of First Australasian Conference on Computational Mechanics (ACCM2013), Sydney, Australia, 3-4 October 2013*, 525-530.
4. **Williams, D.J.** (2014). Keynote Lecture: Mine site rehabilitation – laying geotechnical foundations. *Proceedings of Mine Closure 2014, Johannesburg, South Africa, 1-3 October 2014*, 14 p. **(Invited)**.
5. Tun, Y.W., Pedroso, D.M., Scheuermann, A. and **Williams, D.J.** (2014). Multiple-slope stability assessment by limit equilibrium and genetic algorithms. *Proceedings of Fourteenth International Conference of International Association for Computer Methods and Recent Advances in Geomechanics, IACMAG 2014, Kyoto, Japan, 22-25 September 2014*, 1427-1432.
6. **Williams, D.J.** (2014). An alternative whole-of-life approach to tailings management. *Proceedings of Life-of-Mine 2014, Brisbane, Australia, 16-18 July 2014*, 14 p.
7. **Williams, D.J.**, Kirsch, P.A., Gasparon, M., Baumgartl, T., Edraki, M., Rowe, D. and Harris, J. (2014). Application of RISKGATE to coal mine tailings dams. *Proceedings of Life-of-Mine 2014, Brisbane, Australia, 16-18 July 2014*, 219-230.
8. Chapman, P.J. and **Williams, D.J.** (2014). Comparison of field and laboratory data in relation to cover design on TSF closure. *Proceedings of Sixth International Conference on Unsaturated Soils, Sydney, Australia, 2-4 July 2014*, 1475-1480.
9. Indrawan, I.G.B., **Williams, D.J.** and Scheuermann, A. (2014). Comparison of tensiometer, thermal conductivity and capacitance sensor measurements of pore water pressure in compacted clay columns. *Proceedings of Sixth International Conference on Unsaturated Soils, Sydney, Australia, 2-4 July 2014*, 1587-1595.
10. Scheuermann, A., Galindo-Torres, S.A., Pedroso, D.M., **Williams, D.J.** and Li, L. (2014). Dynamics of water movements with reversals in unsaturated soils. *Proceedings of Sixth International Conference on Unsaturated Soils, Sydney, Australia, 2-4 July 2014*, 1053-1059.
11. **Williams, D.J.** and Kho, A.K. (2014). Laboratory compression of scalped coal mine spoil materials tested under dry and wet conditions. *Proceedings of Sixth International Conference on Unsaturated Soils, Sydney, Australia, 2-4 July 2014*, 1551-1557.
12. Ghamgosar, M., Erarslan, N. and **Williams, D.J.** (2014). Evolution of damage on tensile fracturing of rock by means of elastic ultrasonic wave velocity. *Proceedings of EUROCK 2014, Vigo, Italy, 26-28 May 2014*, 297-302.

13. **Williams, D.J.** (2014). Improved waste rock management – How to achieve optimal water recovery and tailings density, and facilitate closure. *Proceedings of Fifth International Mining and Industrial Waste Management Conference, Rustenburg, South Africa, 10-14 March 2014*. 18 p. **(Invited)**.
14. **Williams, D.J.** (2014). Improved tailings management – How to minimise potential AMD, and facilitate closure. *Proceedings of Fifth International Mining and Industrial Waste Management Conference, Rustenburg, South Africa, 10-14 March 2014*. 20 p. **(Invited)**.
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McARTHUR RIVER MINE
WASTE ROCK SHEAR STRENGTH

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1 INTRODUCTION

Professor David John Williams was commissioned by Mr Gary Taylor of McArthur River Mine (MRM) to report on the results and interpretation of direct shear strength testing of MRM waste rock. This report presents the testing methodology, selected raw results and the interpretation of the shear strength parameters, together with recommended design vales.

2. TESTING METHODOLOGY

The shear strength of particulate materials such as waste rock is traditionally determined in a laboratory direct shear box (see schematic in Figure 1), with the rigid box providing containment for the specimen, which is generally placed loose. Depending upon the scale of the direct shear box, waste rock would normally need to be scalped (removing particles larger than a nominal maximum of five times the height of the box) prior to testing, to ensure that there are sufficient particles over the height of the specimen to generate shear along the interface between the two halves of the direct shear box. Scalping would be expected to reduce the friction angle of the material by several degrees (Williams, 2015; included as Appendix A).

The scalped specimen is generally placed loose to fill the box, and a normal force is applied to simulate the appropriate overburden stress. Traditionally, the direct shear box is immersed in a water bath and the specimen tested wet, which is generally a worst case. However, there is no reason why the specimen cannot be tested at the as-sampled moisture state, and this would often be more representative of field conditions. The specimen is allowed to settle under the applied normal stress, and the settlement should be recorded with time until it effectively ceases. The specimen is then sheared to failure, or to a shear strain of 10%, whichever comes first. The rate of shearing will dictate whether or not the specimen remains drained, although the coarse-grained nature of scalped waste rock would generally allow drainage at a faster rate than the rate of shearing. Being loosely-placed, the shear stress of scalped waste rock would be expected to increase monotonically at a reducing rate with increasing shear strain to a (maximum) ultimate shear strength.

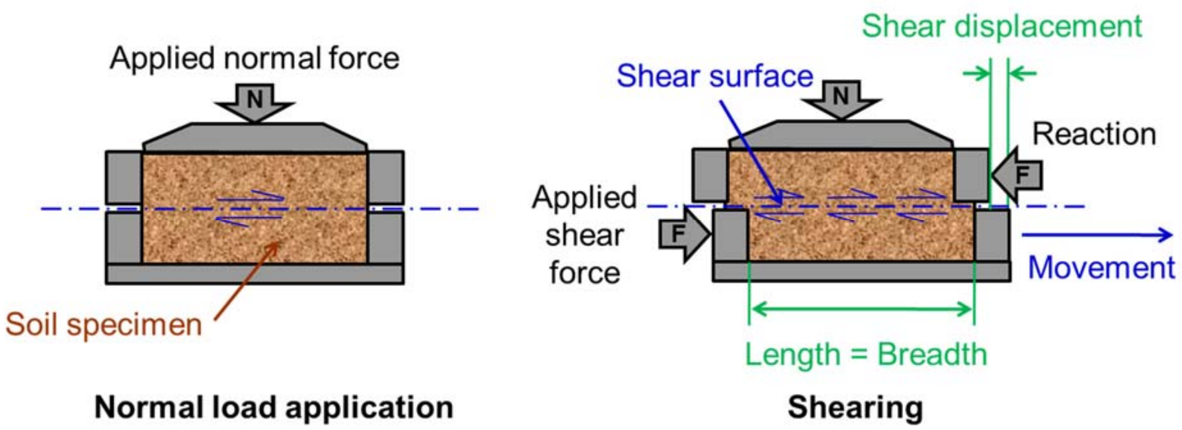


Figure 1 Schematic of a direct shear strength test

Separate direct shear box tests are generally carried out on specimens of the same material under normal stresses that increase in a geometric series; that is, doubling the normal stress at each successive stage (see Figure 2). Alternatively, multi-stage testing could be carried out on the same specimen. However, multi-stage testing limits the shear strain that can be applied in each of the three stages to about 3%, which may not be sufficient to achieve shear failure, and earlier stages may affect the shear strength achieved in later stages.

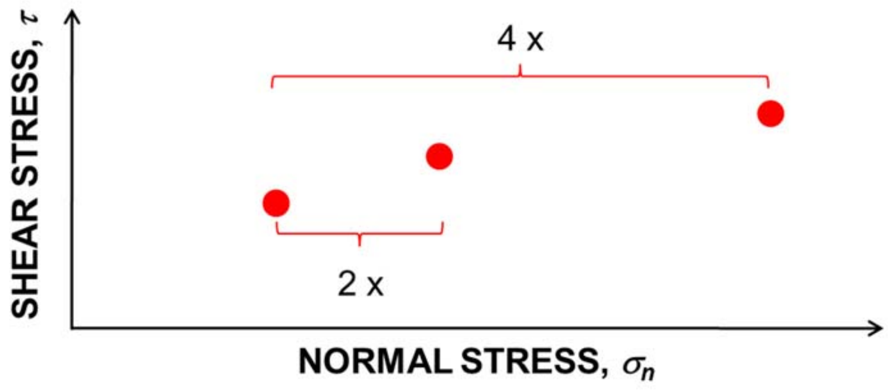


Figure 2 Results of three-stage laboratory direct shear strength testing

2.1 Medium-Sized Direct Shear Testing at UQ

The following sections describe sample preparation and the medium-sized direct shear strength testing methodology carried out at The University of Queensland (UQ).

2.1.1 Sample preparation

The samples came from two sources. An initial batch of Breccia, Weathered Shale and Clay was collected from Trilab in Brisbane. The Breccia and Weathered Shale samples had been scalped to pass 26.5 mm. Separately, MRM delivered approximately 40 kg each of Breccia and Weathered Shale in buckets.

The bucket samples of Breccia and Weathered Shale were near dry and had a pre-scalped maximum particle size of about 75 mm. To assess the potential for breakdown of the Breccia and Weathered Shale, particle size distribution curves were obtained by dry and partial wet sieving, as shown in Figure 3. There was negligible difference between the dry and wet sieving results for each sample, indicating that there is little potential for breakdown on wetting. This was confirmed by the results of slake durability testing, which showed negligible breakdown of either the Breccia or Weathered Shale waste rock. Further, the Weathered Shale is only slightly finer-grained than the Breccia sample.

The Clay sample had dried to a gravimetric moisture content of about 5% and was wet up prior to compaction to a target gravimetric moisture content of 13.6%, representative of the Optimum Moisture Content for Standard Compaction applied on site.

No scalping or crushing of the coarsest waste rock particles was applied prior to direct shear testing. To achieve sufficient Breccia waste rock sample for direct shear testing, the samples sourced from Trilab and MRM were thoroughly mixed, ensuring that the dry sieving particle size distribution for Breccia shown in Figure 3 was maintained. There was sufficient Weathered Shale waste rock sample supplied by MRM for all direct shear testing carried out. The Clay sourced from Trilab was used after wetting-up to a nominal 13.6% gravimetric moisture content.

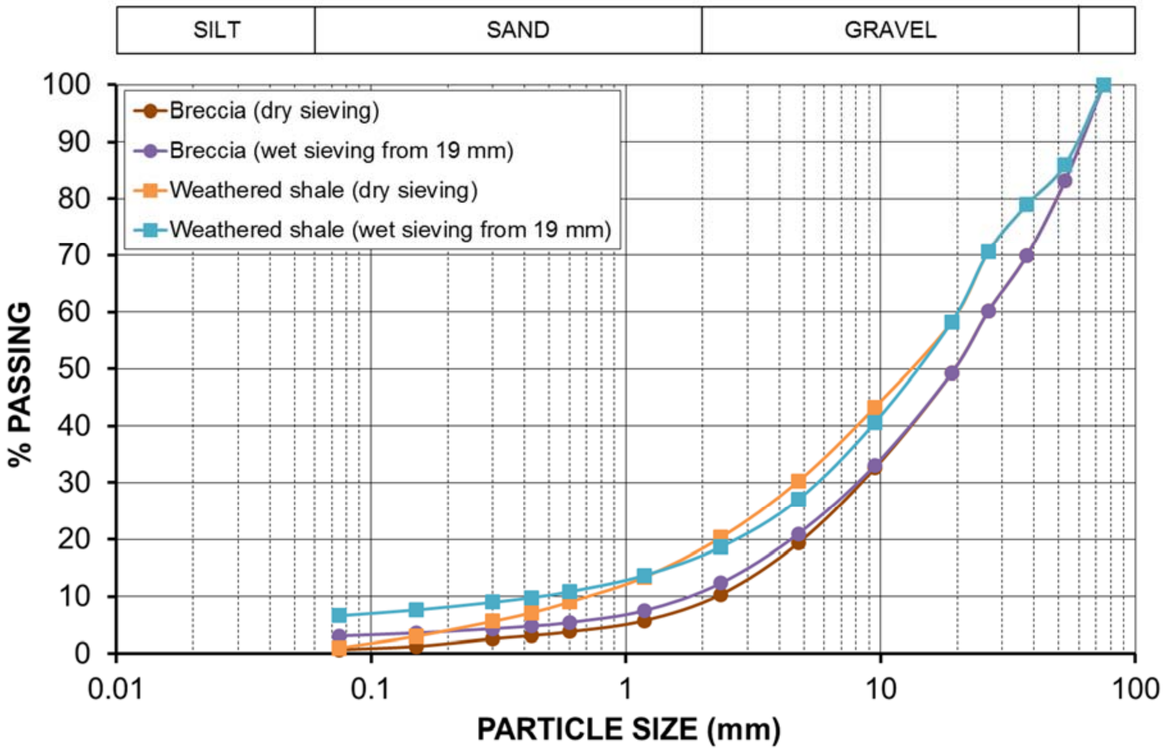


Figure 3 Particle size distribution curves for Breccia and Weathered Shale subjected to dry and partial wet sieving

2.1.2 Testing methodology

The medium-sized direct shear testing machine available at UQ (see Figure 4), manufactured by Wille-Geotechnik of Germany, measures 300 mm by 300 mm in plan and accommodates a specimen up to about 200 mm deep. It is capable of applying a normal stress of up to 1,000 kPa. The Wille machine is moderately stiff, to accommodate the forces applied, and the sides of the direct shear box are about 20 mm thick.

Single-stage testing was carried out at nominal initial normal stresses of 250 kPa, 500 kPa and 1,000 kPa, representing waste rock dump heights of about 14 m, 28 m and 56 m, respectively (assuming a wet unit weight of 18 kN/m^3). The testing was carried out either at the as-sampled gravimetric moisture content ("dry"), or in a water bath ("wet"). The wet test specimen was allowed to soak overnight prior to the normal stress being applied.

Following the application of the normal stress and the virtual cessation of compression under that stress, shearing was carried out at a shearing rate of 0.1 mm/min to a nominal 10% shear strain (which is considered the norm, to avoid excessive distortion of the top cap and possible erroneous results). During a test, the Wille machine maintains a separation of the two halves of the direct shear box so that there is no metal on metal contact on which friction can develop. Further, the Wille machine monitors vertical displacement at all four corners of the top loading cap, and automatically stops the test when the tilt exceeds 10% or any one of the four settlement sensors (LVDTs) exceeds 50 mm travel, avoiding erroneous results due to tilting.

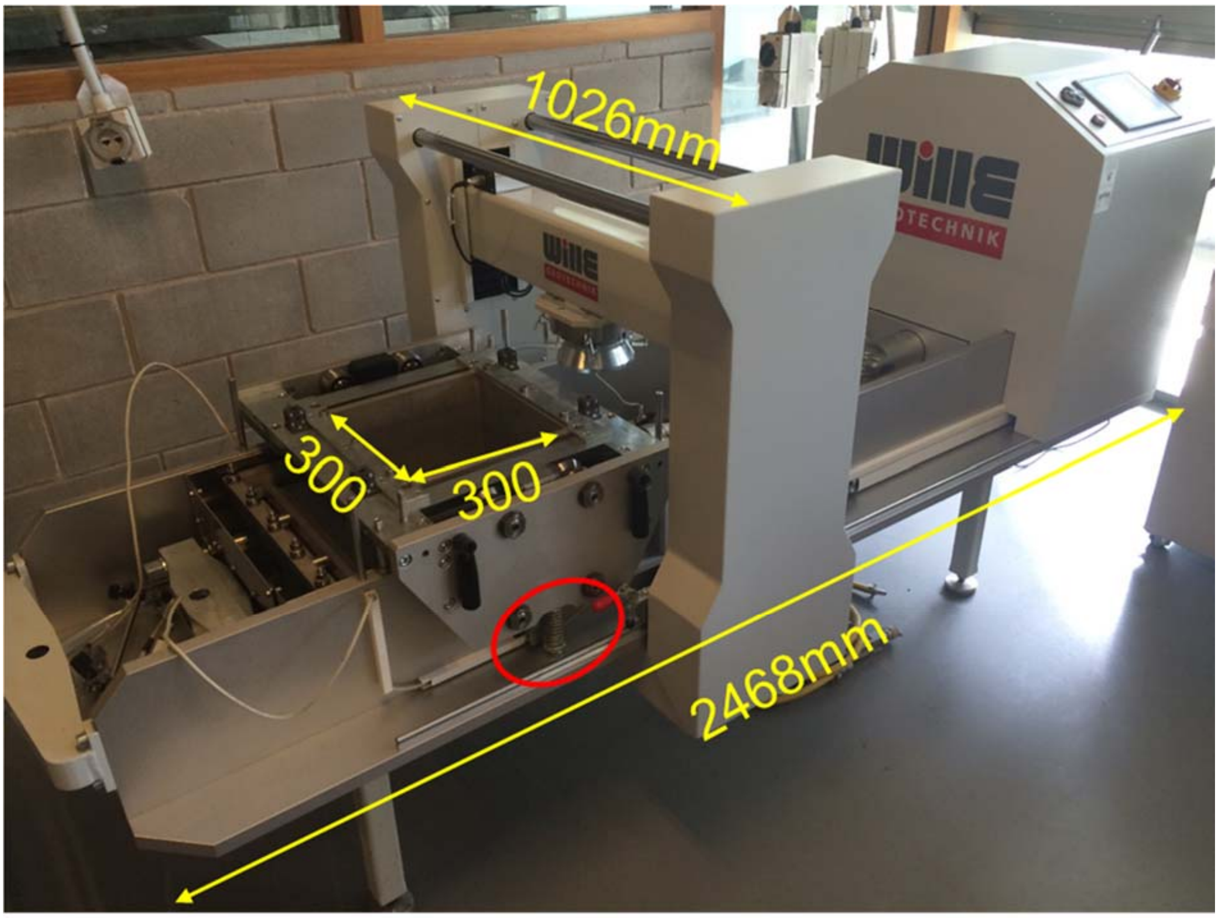
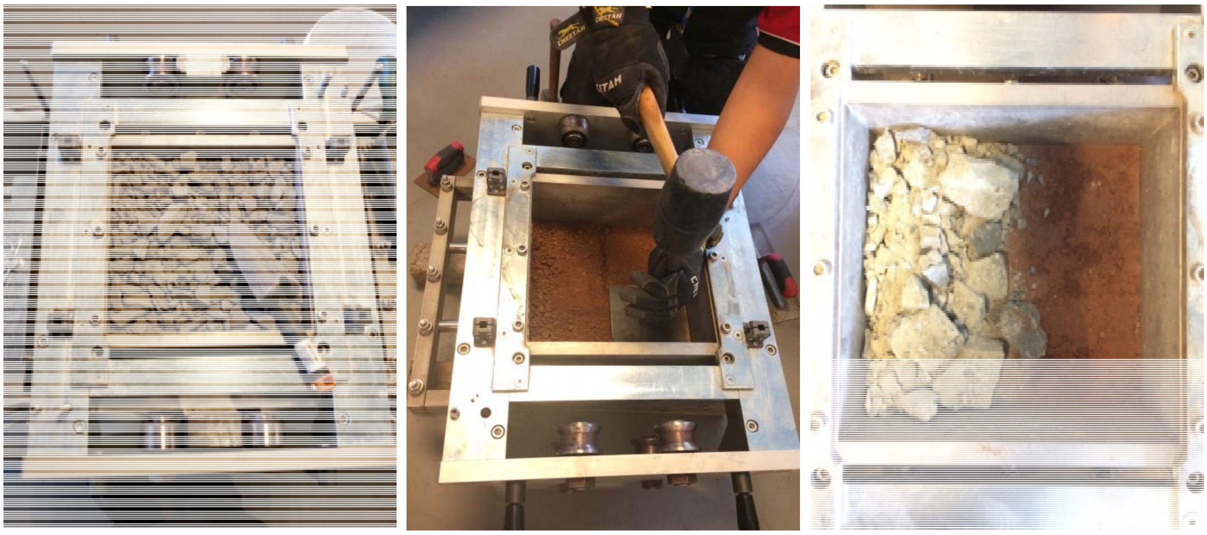


Figure 4 UQ's medium-sized direct shear testing machine manufactured by Wille-Geotechnik of Germany

The tests carried out in the medium-sized direct shear machine at UQ are summarised in Table 1. The testing of Breccia, Weathered Shale and Breccia on Weathered Shale under dry conditions represented the bulk of the waste rock dump volume, while the testing of Weathered Shale on compacted Clay under dry and wet (worst case) conditions represented the interfaces with the compacted clay liners (CCLs). Figure 5 shows photographs of sample preparation for some of the direct shear tests carried out. Relatively little particle breakdown was observed following testing.

Table 1 Summary of medium-sized direct shear strength tests carried out at UQ

MATERIAL TESTED	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (t/m^3)
Breccia	0.4 (Dry)	1.769
Weathered Shale	1.1 (Dry)	1.624
Breccia on Weathered Shale	1.1 (Dry)	1.632
Weathered Shale on compacted Clay	1.7/13.6 (Dry)	1.783/1.850
	Near-saturated (Wet)	1.783/1.850



(a) (b) (c)

Figure 5 Sample preparation of: (i) loose Breccia, (ii) compacted Clay, and (iii) loose Weathered Shale on compacted Clay, in the UQ direct shear box

2.2 Large-Sized Direct Shear Testing at UN

The following sections describe sample preparation and the large-sized direct shear strength testing methodology carried out at The University of Newcastle (UN).

2.2.1 Sample preparation

The samples, as-received from MRM in cubic metre pods, were coarse-grained, segregated and texturally variable. Three pods of each of five waste rock types were delivered:

- PAF1
- PAF2
- Breccia
- Shale
- Weathered Shale

In most cases, the particle size distribution and texture of each waste rock type visible at the tops of the pods were noticeably different between the three pods. In nearly every case, the pods contained particles up to 300 mm in size. The amount of sample exceeding 100 mm varied between 10% and 30%.

Sampling for testing involved tipping a pod to spill around 500 kg of material onto a concrete floor. Over-sized material was removed, and broken with a sledge hammer until it was smaller than 100 to 120 mm in size and returned to the sample. No attempt was made to homogenise the material in the three pods of each waste rock type, resulting in variation in the particle size distribution and texture of the samples loaded in the direct shear box, and hence variation in the density achieved on loading in the direct shear box.

2.2.2 Testing methodology

The large-sized direct shear testing machine available at UN (see Figure 6), which was manufactured in-house, measures 720 mm by 720 mm in plan and accommodates a specimen up to about 500 mm deep. It is capable of applying a normal stress of up to 3,500 kPa. The UN machine is very stiff, to accommodate the large forces applied, and the sides of the direct shear box are about 100 mm thick. The combination of the machine stiffness and higher normal stresses applied results in particle crushing and very high compaction of the specimens, with very high horizontal stresses locked-in against the rigid walls of the box. The combination of particle crushing and compaction makes the specimens resemble a "brick", as evidenced by the need to jack-hammer the specimens out of the box after testing. This results in the shear strength results tending to converge, and the crushing of particles would be expected to reduce their shear strength.

Figure 6 UN's large-sized direct shear testing machine

Single-stage testing was carried out at nominal initial normal stresses of 300 kPa, 900 kPa and 2,000 (wet testing) to 2,100 kPa (dry testing), representing waste rock dump heights of about 17 m, 50 m and 117 m, respectively. The testing was carried out either at the as-sampled gravimetric moisture content ("dry") or in a water bath ("wet"). Wet test specimens were allowed to soak for at least 2 hours prior to the normal stress being applied.

Following the application of the normal stress and the virtual cessation of compression, shearing was carried out at a shearing rate of 2 mm/min (20 times faster than the shearing rate applied in the UQ tests) to a nominal 20% shear strain (twice the maximum strain applied in the UQ tests). The sliding surfaces of the UN direct shear box are coated with Teflon to minimise friction between contacts.

The tests carried out in the large-sized direct shear machine at UN are summarised in Table 2. The testing of the various waste rock types under dry conditions represented the bulk of the waste rock dump volume, while the testing under wet conditions represented the worst case of a saturated interface, which is not expected to occur since the waste rock will drain.

Table 2 Summary of large-sized direct shear strength tests carried out at UN

MATERIAL TESTED	INITIAL MOISTURE STATE
PAF1	Dry Wet
PAF2	Dry Wet
Breccia	Dry Wet
Shale	Dry Wet
Weathered Shale	Dry Wet

3. INTERPRETATION OF DIRECT SHEAR TEST DATA

Alternative interpretations of laboratory shear strength test results, strictly for saturated soil specimens, are shown in Figure 7. Coulomb (1773) proposed a straight line shear strength envelope, while Mohr in about 1882 proposed a more general (curved) fit to Mohr circles, which can be approximated as a straight line. At some time, the two strength criteria were combined and named the Mohr-Coulomb straight line shear strength criterion (or shear strength envelope), which for strictly saturated conditions is given by:

$$\tau = c + \sigma_n \tan \phi \qquad (1)$$

where τ is the shear strength, c is the cohesion (intercept at zero normal stress, referred to by some as the "apparent" cohesion), σ_n is the normal stress, and ϕ is the friction angle. The Mohr-Coulomb shear strength criterion may be a reasonable representation of the shear strength of a soil over a limited range of applied normal stresses.

For purely cohesive soils tested under undrained conditions, equation (1) reverts to:

$$\tau = c_u \qquad (2)$$

where c_u is the undrained cohesion.

For purely cohesive soils tested under drained conditions, equation (1) reverts to:

$$\tau = c' + \sigma_n' \tan \phi' \qquad (3)$$

where c' is the drained cohesion, σ_n' is the effective normal stress, and ϕ' is the drained friction angle.

For purely frictional soils, tested dry, equation (1) reverts to:

$$\tau = \sigma_n' \tan \phi' \qquad (4)$$

where ϕ' is the drained friction angle.

For partially saturated conditions, equation (1) expands to:

$$\tau = c + (\sigma_n - u_w) \tan \phi \qquad (5)$$

where u_w is the pore water matric suction. The unsaturated shear strength may simplistically be handled by adding an apparent cohesion term to account for the strengthening effect of matric suction.

It is worth noting, as shown in Figure 7, that the higher the normal stress applied to a loosely-placed coarse-grained specimen, the higher the density it will be compressed to. This would be expected to increase its shear strength, although high applied normal stresses may cause particle crushing, which may reduce the shear strength.

The shearing of the box results in a loss in contact area for the specimen, which is allowed for by applying an area correction to both the applied normal stress and the measured shear stress. Applying an area correction to the stresses felt by purely frictional materials has no impact on the resulting Mohr-Coulomb friction angle, since the failure point simply moves up the strength envelope. However, an area correction will increase the cohesion intercept and may change the friction angle, where the cohesion is non-zero. Area corrections are applied to both the shear and normal stresses in all of the direct shear tests reported herein.

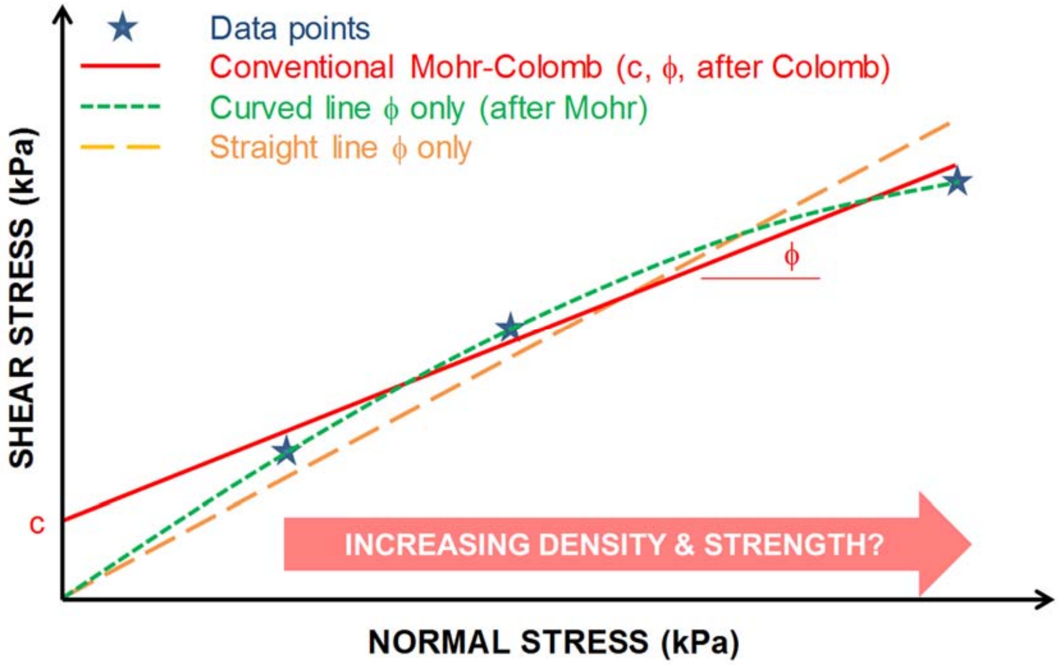


Figure 7 Alternative interpretations of laboratory direct shear strength test results

All of the above equations are simplifications aimed at facilitating the selection of shear strength parameters that can be used in analyses such as geotechnical slope stability analyses. They presume that the material being sheared "feels" either cohesive or frictional, or some combination; and feels either undrained or drained, or partially drained. Equations (1) to (4) are also strictly for saturated conditions.

For testing under partially saturated conditions, matric suction will increase the shear strength. However, as a specimen is compressed under the applied normal stress and, to a lesser degree on shearing, its degree of saturation will increase and hence its matric suction (and shear strength) will decrease. The shear strength of a soil may be quoted simply in terms of the shear strength τ, or as a (Mohr-Coulomb) combination of c and ϕ, or as a series of secant ϕ values with increasing σ_n, as shown in Figure 8.

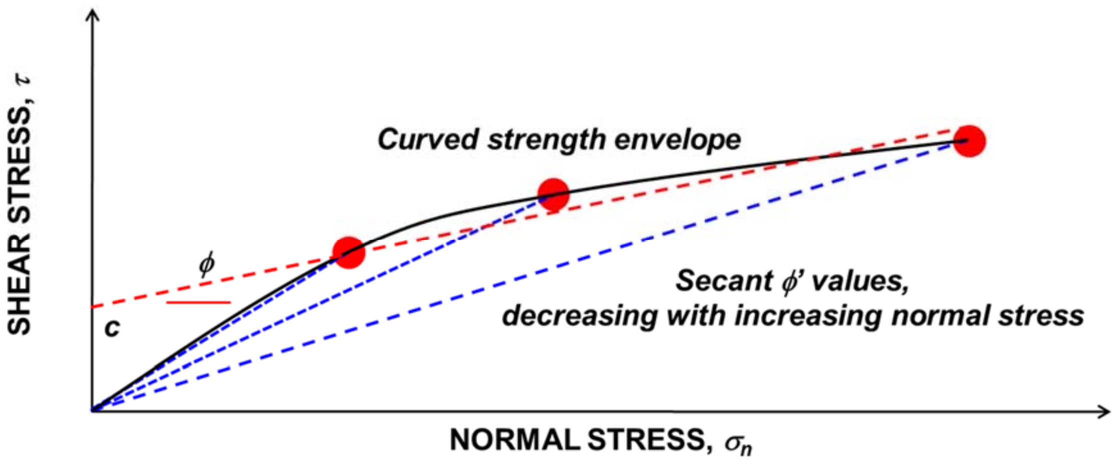


Figure 8 Mohr-Coulomb versus secant friction angle interpretations

4. DIRECT SHEAR TEST RESULTS

The results of the direct shear testing carried out at UQ and UN are described in the following sections.

4.1 Results of Medium-Sized Direct Shear Testing at UQ

Typical raw results (not area-corrected) for the testing of Breccia in the medium-sized direct shear machine at UQ, at its as-sampled gravimetric moisture content (dry) are shown in Figures 9 and 10. Figure 9 shows shear stress versus shear displacement plots expected of initially loose, coarse-grained specimens, in that they approach an “ultimate” shear strength, with no apparent peak. Figure 10 shows that the specimens are generally “contractive” (settling on shearing, settlement being shown as positive), again expected of initially loose, coarse-grained specimens.

The results for all of the tests carried out in the medium-sized direct shear machine at UQ are summarised in Table 3, and Figure 11 shows the shear stresses at failure (or at 10% shear strain, whichever occurs first) at the respective applied normal stresses at failure for all of these tests.

The higher the normal stress applied to a loosely-placed coarse-grained specimen, the higher the density it will be compressed to, which would be expected to increase its shear strength. Figure 12 shows the trend of increasing shear stress at failure due mainly to increasing dry density with increasing applied normal stress. This compensates for an expected decrease in shear stress at failure with increasing applied normal stress, resulting in an approximately linear shear strength failure envelope.

Typically, the specimens underwent a settlement of about 15 mm (7.5% of the loose height) under the applied normal stress, and a further settlement of about 5 mm (2.5% of the loose height) on shearing. Both these settlements are substantially less than the maximum particle size of 75 mm. In particular, the shear plane is not displaced substantially from the centre of the direct shear box during shearing.

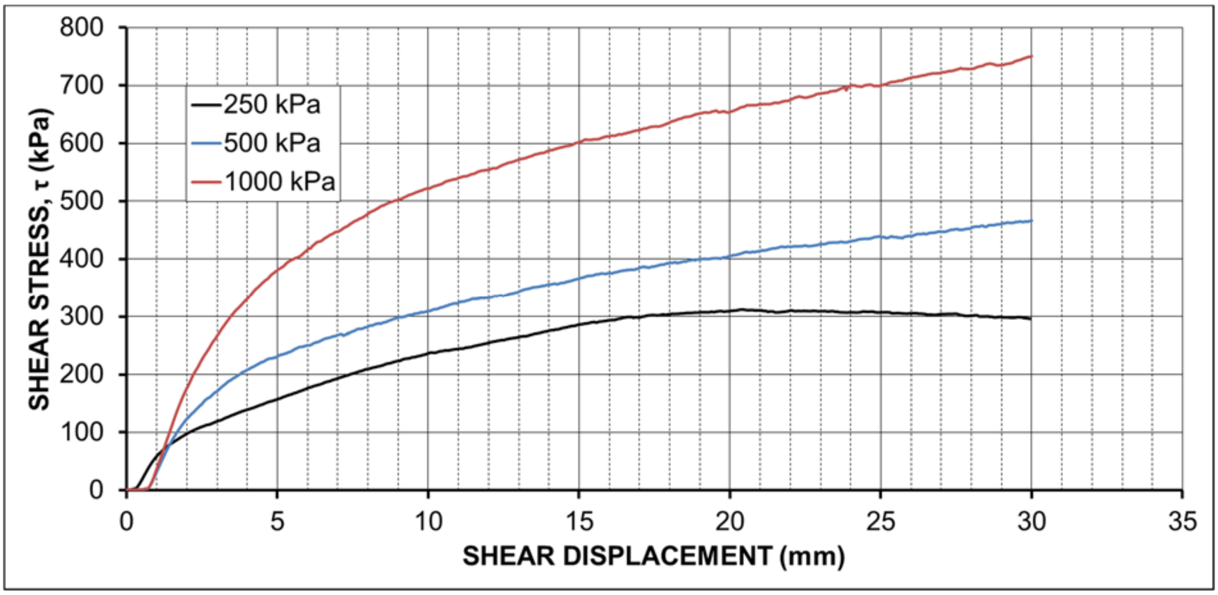


Figure 9 Shear stress versus shear displacement plots obtained for Breccia tested dry at UQ

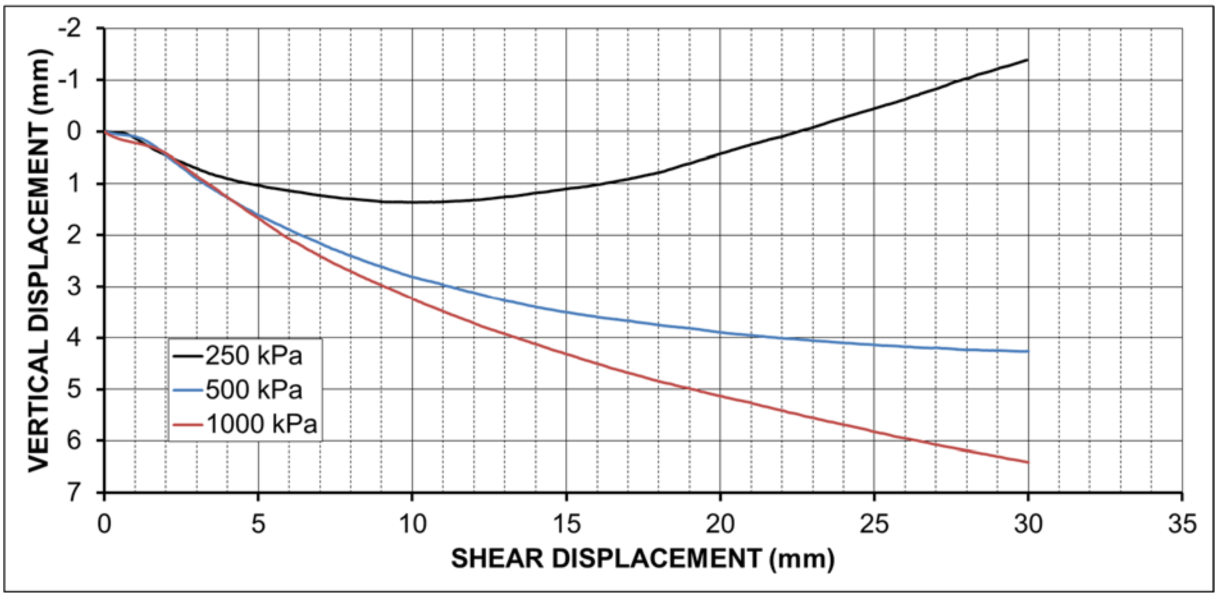


Figure 10 Vertical displacement versus shear displacement plots obtained for Breccia tested dry at UQ

Table 3 Summary of results of medium-sized direct shear strength tests carried out at UQ

MATERIAL TESTED	MOISTURE STATE	INITIAL DRY DENSITY (t/m³)	FINAL DRY DENSITY (t/m³)	NORMAL STRESS AT FAILURE (kPa)	SHEAR STRESS AT FAILURE (kPa)
Breccia	Dry	1.769	1.834 1.837 1.884	272 556 1,111	336 518 834
Weathered Shale	Dry	1.624	1.743 1.776 1.796	278 556 1,111	281 542 821
Breccia on Weathered Shale	Dry	1.632	1.704 1.727 1.759	272 555 1,108	253 486 819
Weathered Shale on compacted Clay	Dry	1.850/1.783	1.837 1.888 1.955	277 556 1,110	199 362 730
	Wet	1.850/1.783	1.996 2.044 2.132	278 556 1,111	167 330 662

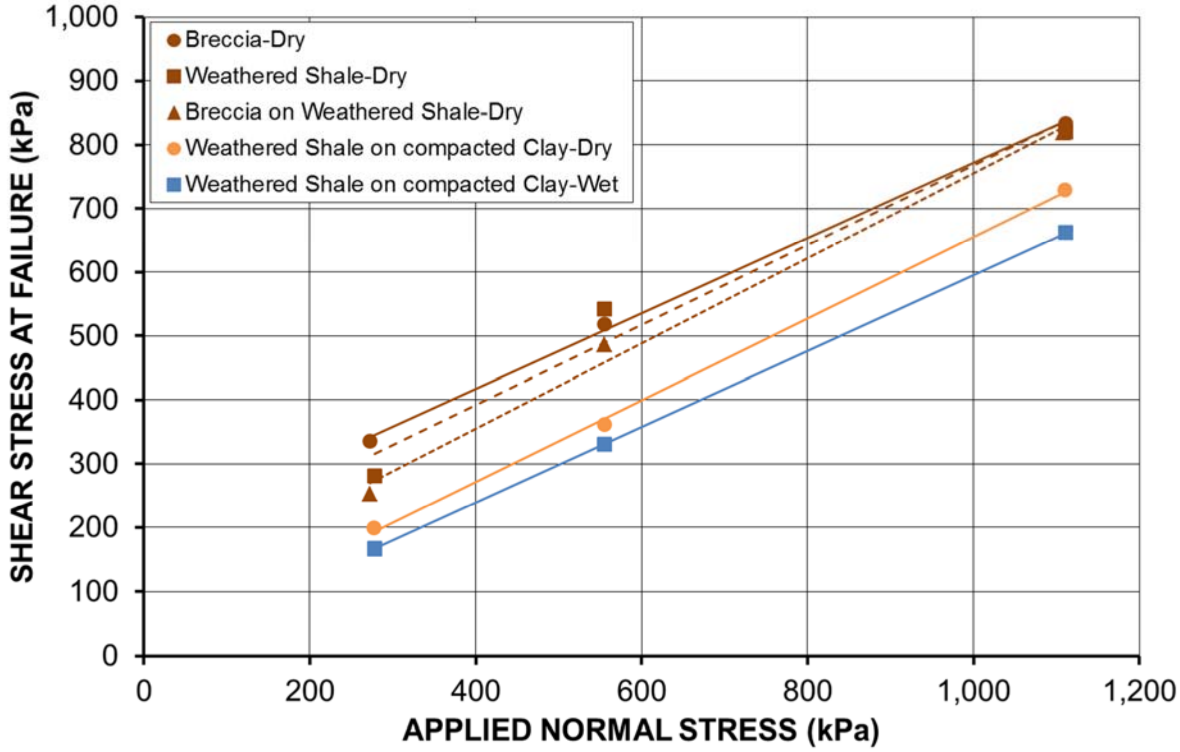


Figure 11 Shear stress at failure versus applied normal stress from UQ tests

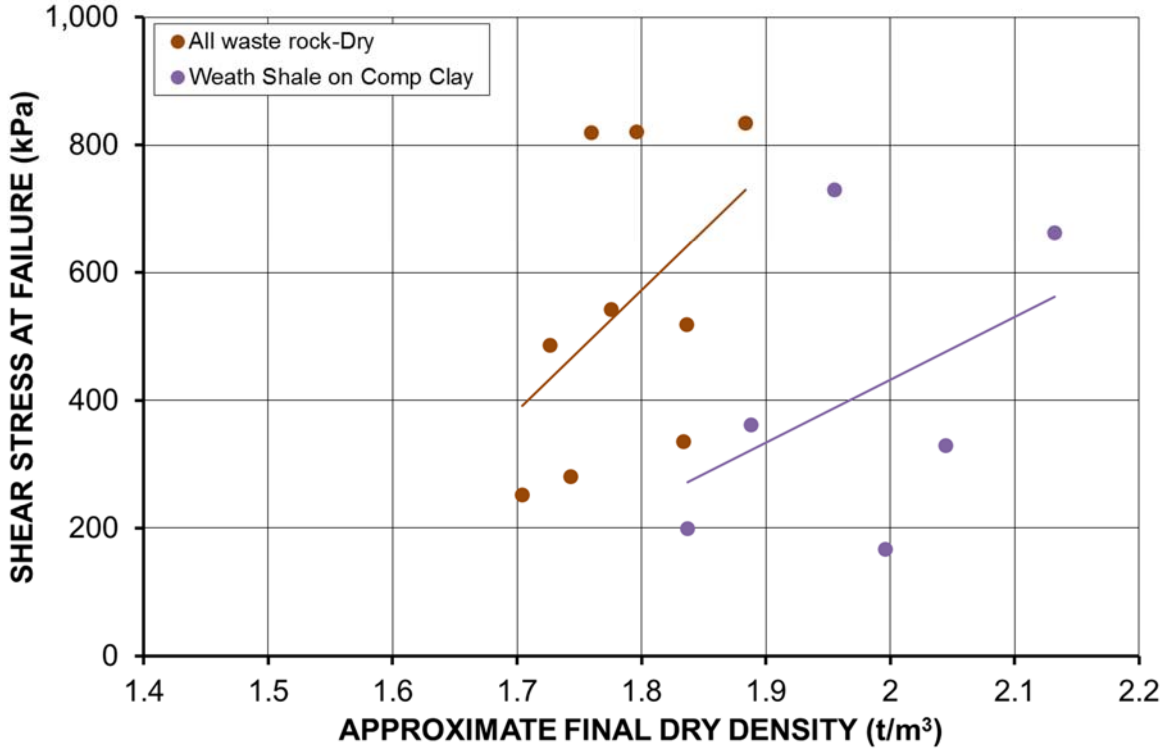


Figure 12 Increasing trend of shear stress at failure with increasing final dry density due mainly to increasing applied normal stress in UQ tests

3.2 Results of Large-Sized Direct Shear Testing at UN

Typical raw results (not area-corrected) for the testing of Breccia under dry conditions in the large-sized direct shear machine at UN are shown in Figures 13 to 15. Figure 13 shows shear stress versus shear displacement plots approaching an ultimate shear strength, as expected of initially loose, coarse-grained specimens. Figure 14 shows that the applied normal stresses tending to drop off, particularly under higher normal stresses, as the specimen was sheared, due to the loading arrangement not being able to maintain the normal load as the specimens underwent settlement. This drop off in applied normal stress was allowed for in the interpretation of the results. Figure 15 shows that the specimens are "contractive", again expected of initially loose, coarse-grained specimens.

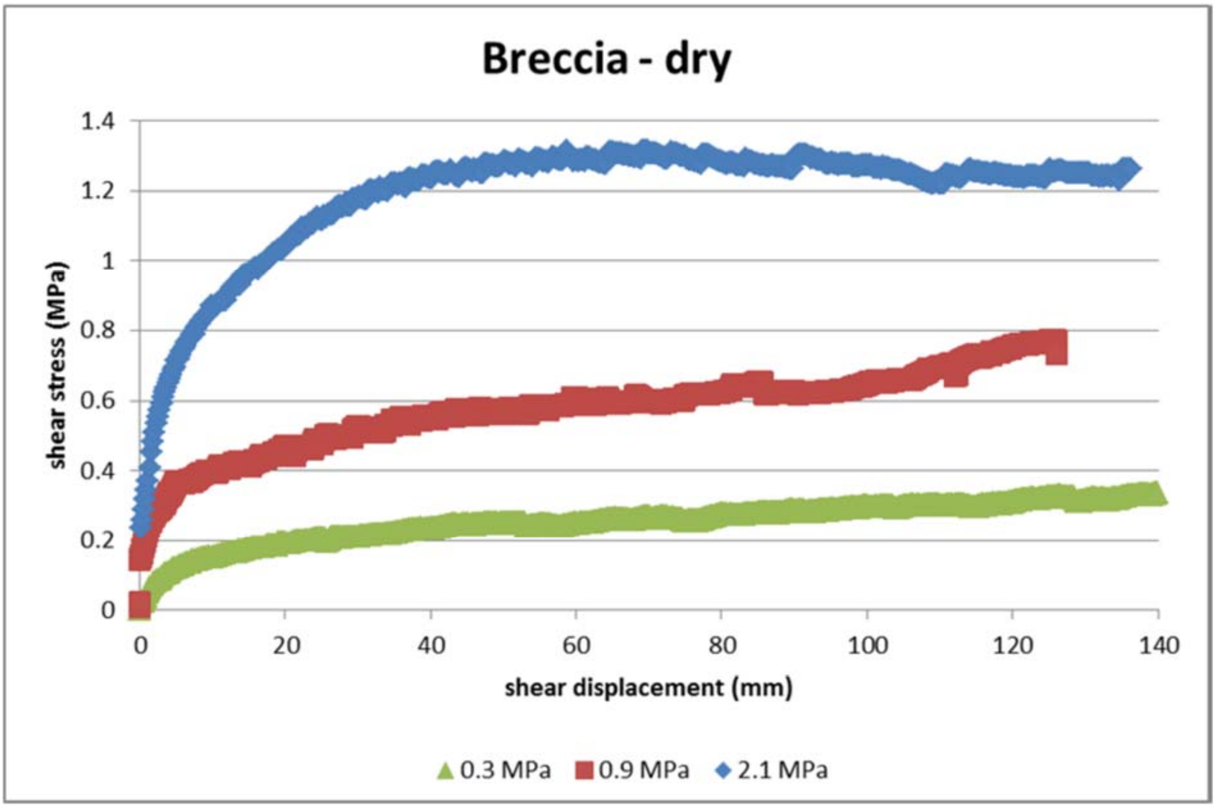


Figure 13 Shear stress versus shear displacement plots obtained for Breccia tested dry at UN

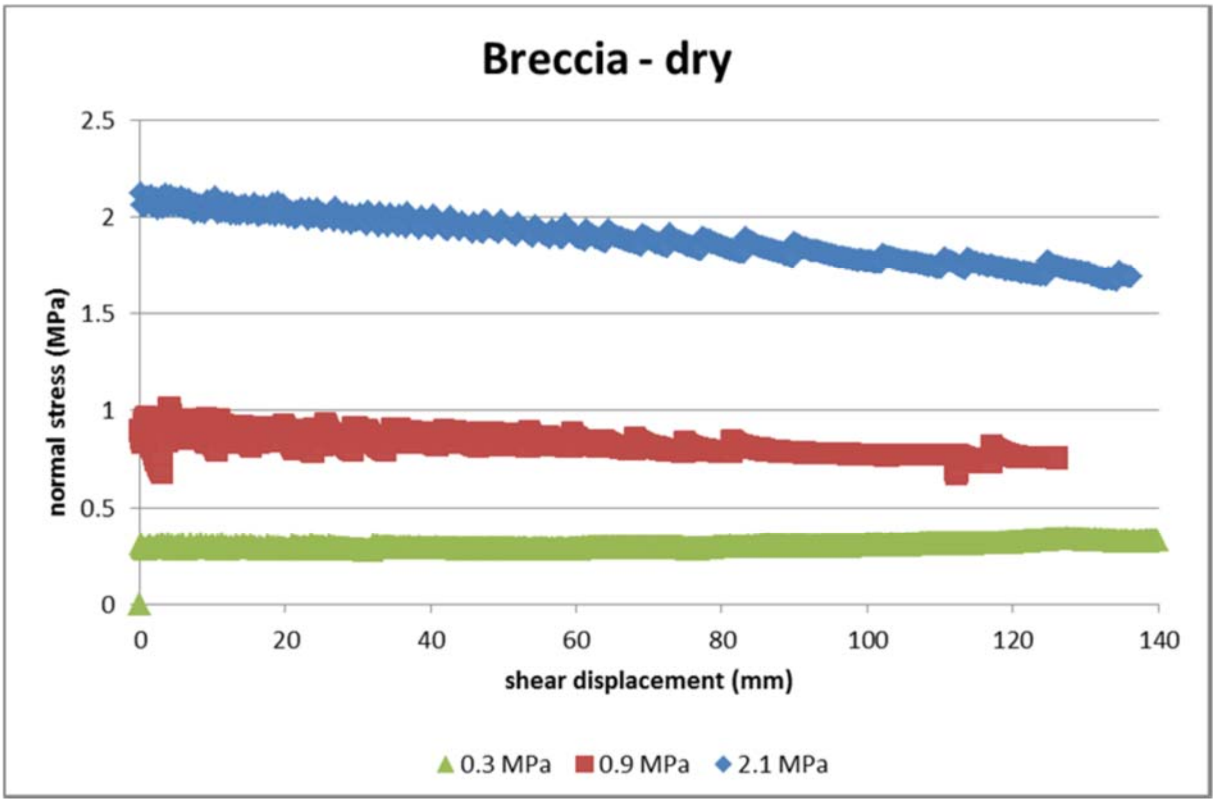


Figure 14 Normal stress versus shear displacement plots obtained for Breccia tested dry at UN

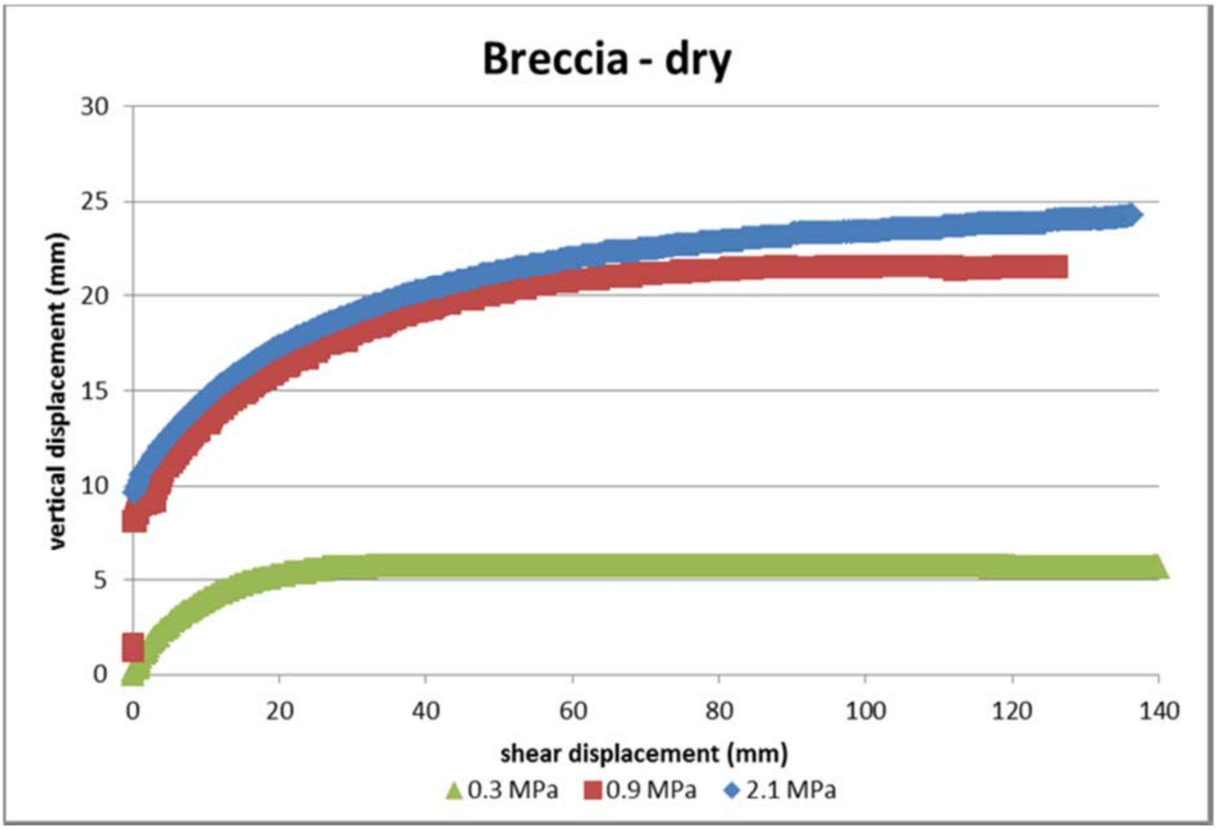


Figure 15 Vertical displacement versus shear displacement plots obtained for Breccia tested dry at UN

Typical raw results (not area-corrected) for the testing of Breccia under wet (inundated) conditions in the large-sized direct shear machine at UN are shown in Figures 16 to 18. Figure 16 shows shear stress versus shear displacement plots approaching an ultimate shear strength, as expected of initially loose, coarse-grained specimens. Figure 17 shows that the applied normal stresses tended to drop off as the specimen was sheared, particularly at higher normal stresses, due to the loading arrangement not being able to maintain the normal load as the specimens underwent settlement. This drop off in applied normal stress was allowed for in the interpretation of the results. Figure 18 shows that the specimens are “contractive”, again expected of initially loose, coarse-grained specimens.

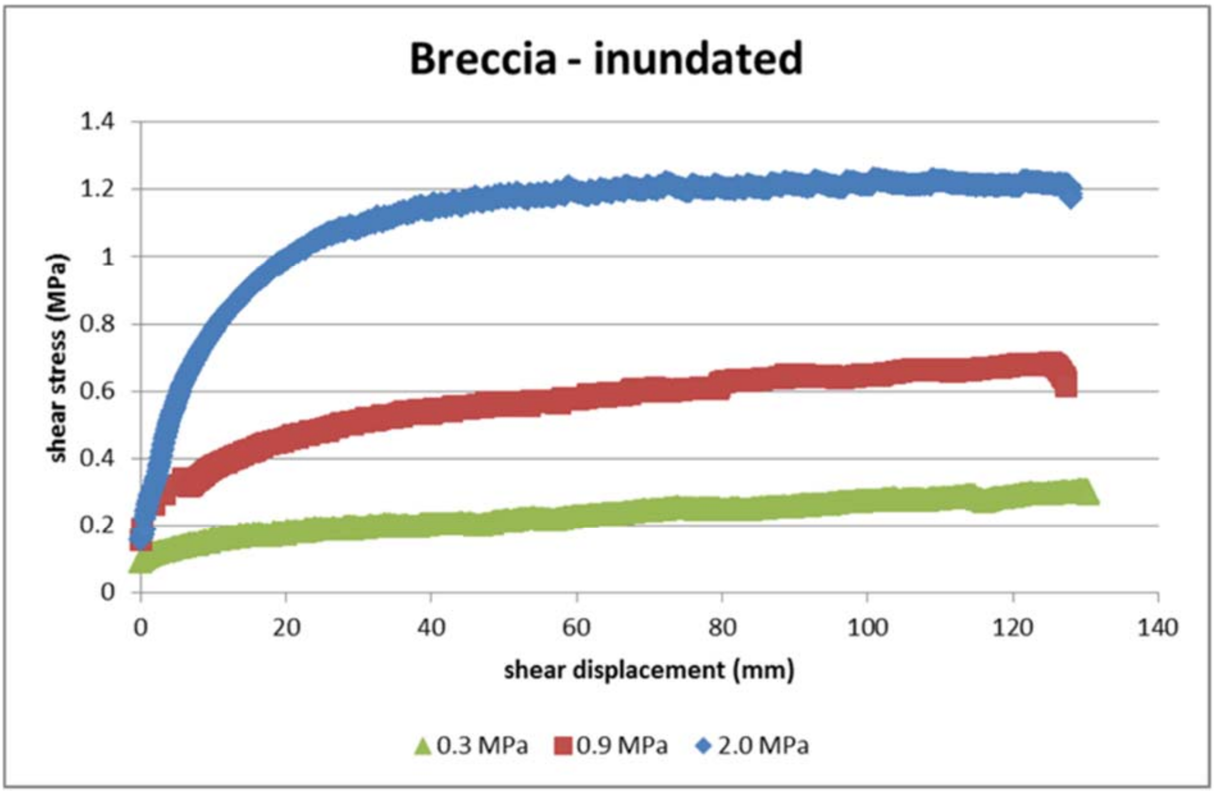


Figure 16 Shear stress versus shear displacement plots obtained for Breccia tested wet at UN

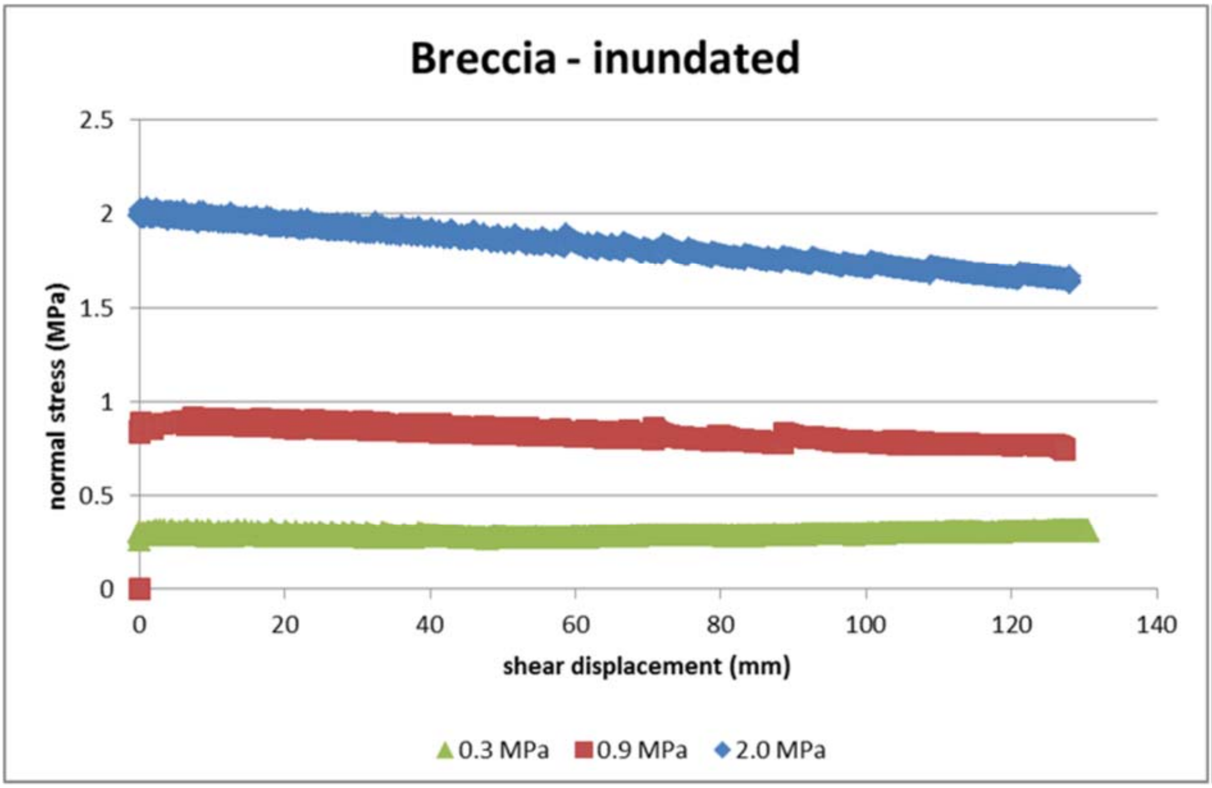


Figure 17 Normal stress versus shear displacement plots obtained for Breccia tested wet at UN

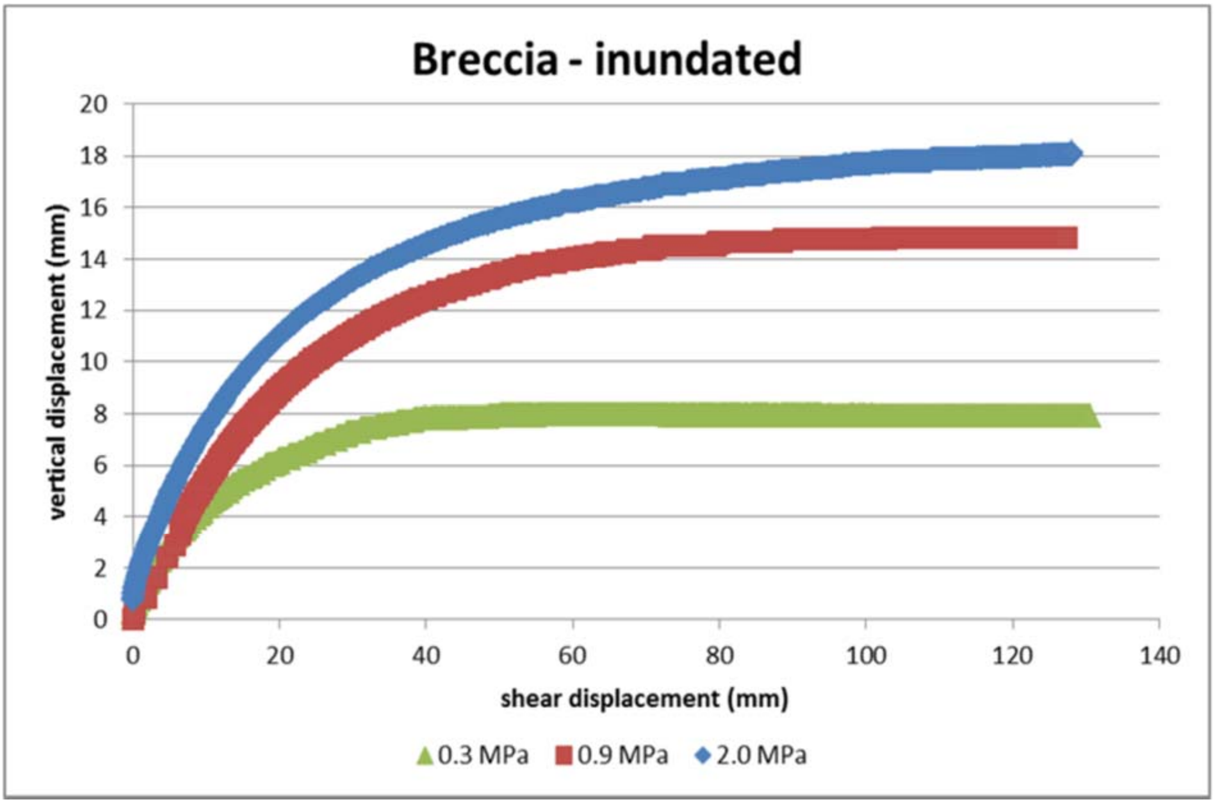


Figure 18 Vertical displacement versus shear displacement plots obtained for Breccia tested wet at UN

The results for all of the tests carried out in the large-sized direct shear machine at UN are summarised in Table 4, and Figure 19 shows the shear stresses at 10% shear strain at the respective applied normal stresses for all of these tests. The "failure" shear stress is taken as the shear stress at 10% shear strain since the stresses continue to increase slightly with increasing shear strain, and distortion of the top cap and erroneous results are considered likely beyond 10% shear strain.

Table 3 Summary of results of large-sized direct shear strength tests carried out at UN

MATERIAL TESTED	MOISTURE STATE	INITIAL DRY DENSITY (t/m^3)	FINAL DRY DENSITY (t/m^3)	NORMAL STRESS AT 10% SHEAR STRAIN (kPa)	SHEAR STRESS AT 10% SHEAR STRAIN (kPa)
PAF1	Dry	1.490 1.626 1.845	1.565 1.930 2.092	382 1,092 2,764	217 596 1,768
	Wet	1.490 1.626 1.845	1.538 1.697 1.957	362 1,086 2,632	289 659 1,671
PAF2	Dry	1.699 1.820 1.936	1.772 1.869 2.041	382 1,092 2,500	204 586 1,520
	Wet	1.699 1.820 1.936	1.775 1.918 2.027	362 1,086 2,632	289 763 1,750
Breccia	Dry	1.490 1.569 1.904	1.507 1.639 2.007	395 1,118 2,434	265 646 1,433
	Wet	1.490 1.569 1.904	1.514 1.615 1.970	382 1,105 2.401	342 803 1,579
Shale	Dry	1.572 1.671 1.850	1.588 1.778 1.923	368 1,118 2,500	342 803 1,513
	Wet	1.572 1.671 1.850	1.606 1.737 1.913	355 1,184 2,368	303 763 1,645
Weathered Shale	Dry	1.577 1.912 1.999	1.609 1.964 2.048	368 1,145 2,474	289 697 1,776
	Wet	1.577 1.912 1.999	1.609 1.963 2.048	355 1,086 2,368	276 776 1,395

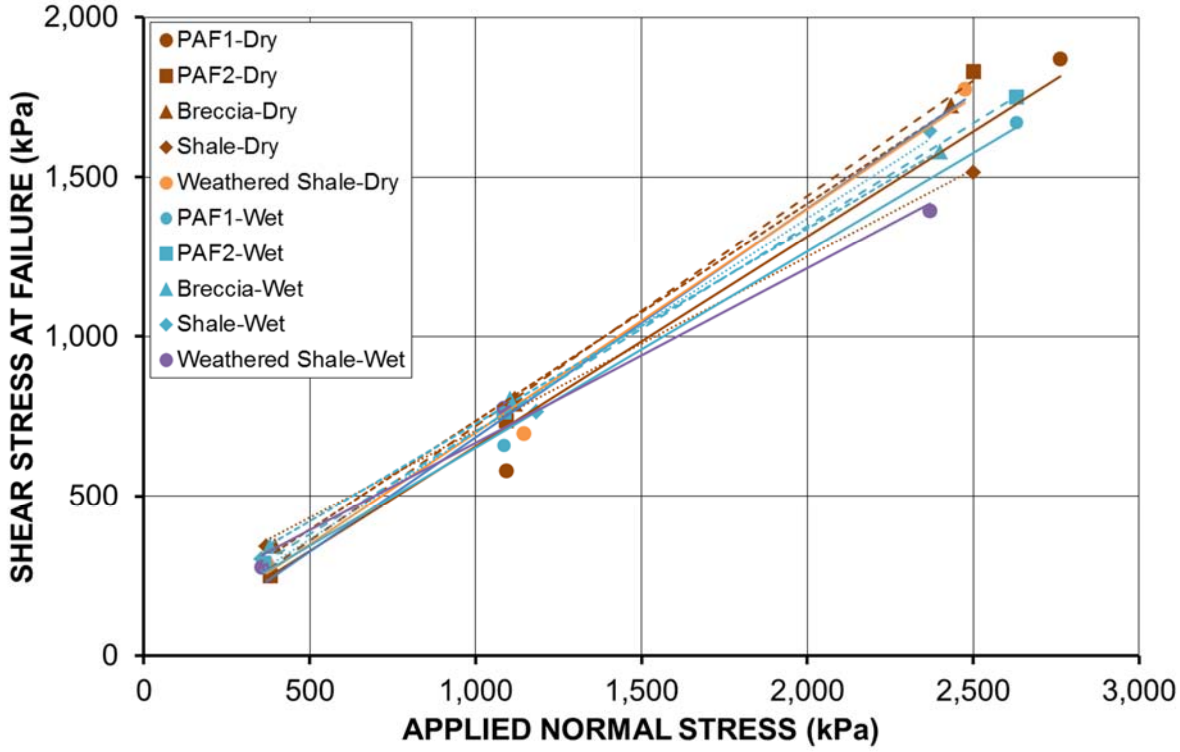


Figure 19 Shear stress at 10% shear strain versus applied normal stress from UN tests

Figure 20 compares the UQ and UN shear stress versus applied normal stress data for Breccia and Weathered Shale tested dry. There is somewhat more difference between the results obtained from the different direct shear machines than there is between the different waste rock types. The UQ data imply a higher cohesion intercept and lower friction angle than the UN data. Likely explanations for this are the higher normal stress range and correspondingly substantially higher dry densities achieved in the UN tests. This would result in reduced suction-induced apparent cohesion, since the degree of saturation increases with increasing dry density (for a given moisture content).

Figure 21 shows the increasing shear stress at 10% shear strain due mainly to increasing dry density with increasing applied normal stress, compensating for an expected decrease in shear stress with increasing applied normal stress and resulting in an approximately linear shear strength failure envelope. Figure 22 compares the effect on shear stress at failure of increasing dry density for both the UQ and UQ tests. Overall, the trends are similar, although the effect of dry density on shear strength is more pronounced in the UN direct shear tests.

Typically, the dry UN specimens underwent a total settlement (under the combined effects of the applied normal stress and shearing) of about 24 mm (5% of the loose height). The wet UN specimens underwent a reduced total settlement (under the combined effects of the applied normal stress and shearing) of about 17 mm (3.5% of the loose height), due to the settlement induced by wetting-up prior to the application of the normal stress. These settlements are substantially less than the maximum particle size of 100 to 120 mm.

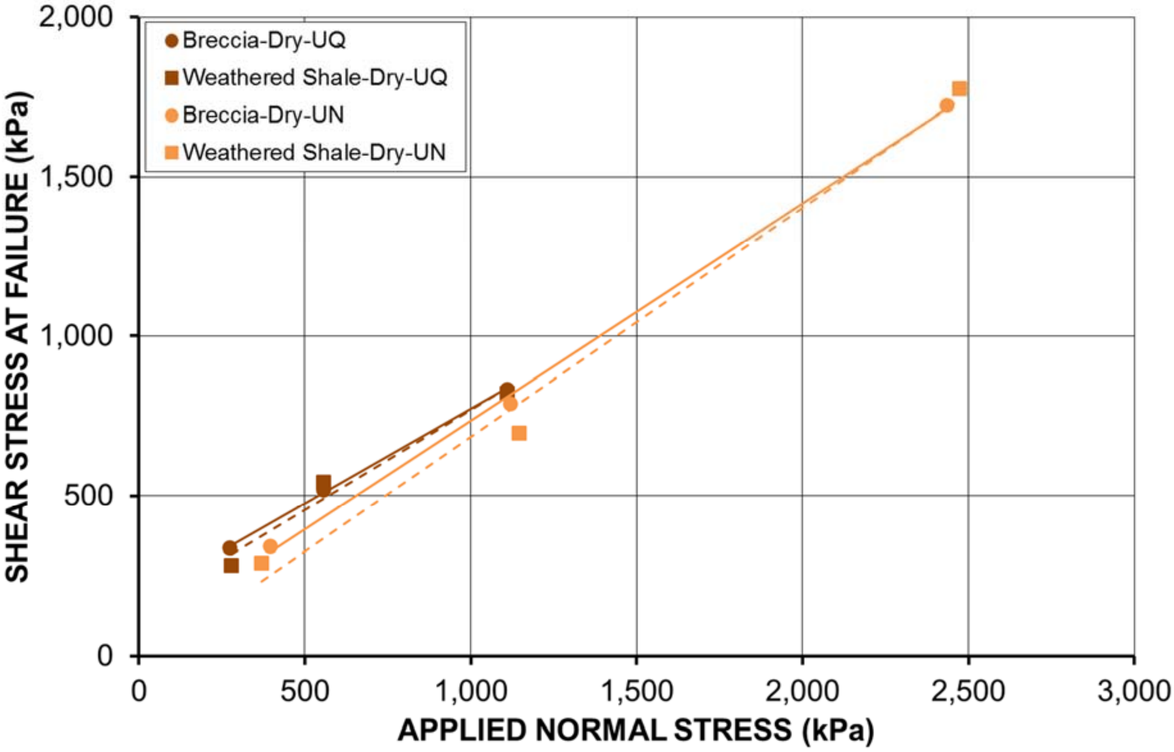


Figure 20 Comparison between UQ and UN shear stress versus applied normal stress data for Breccia and Weathered Shale tested dry

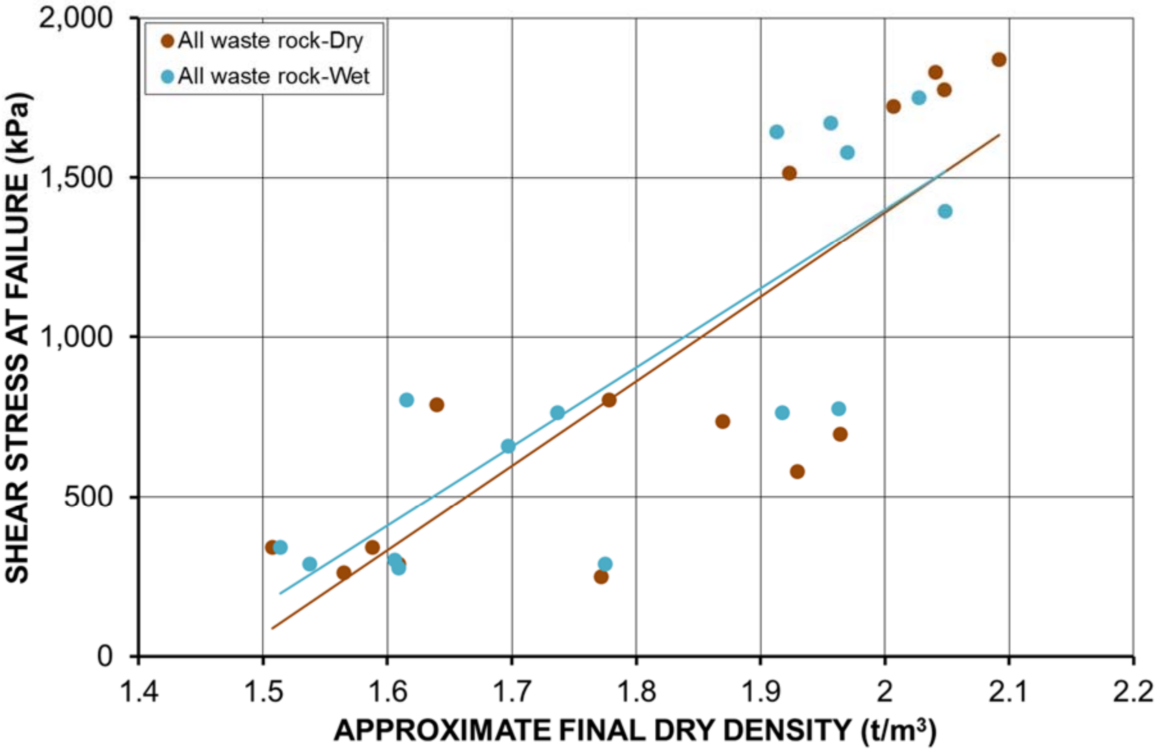


Figure 21 Increasing shear stress at 10% shear strain with increasing final dry density due mainly to increasing applied normal stress in UN tests

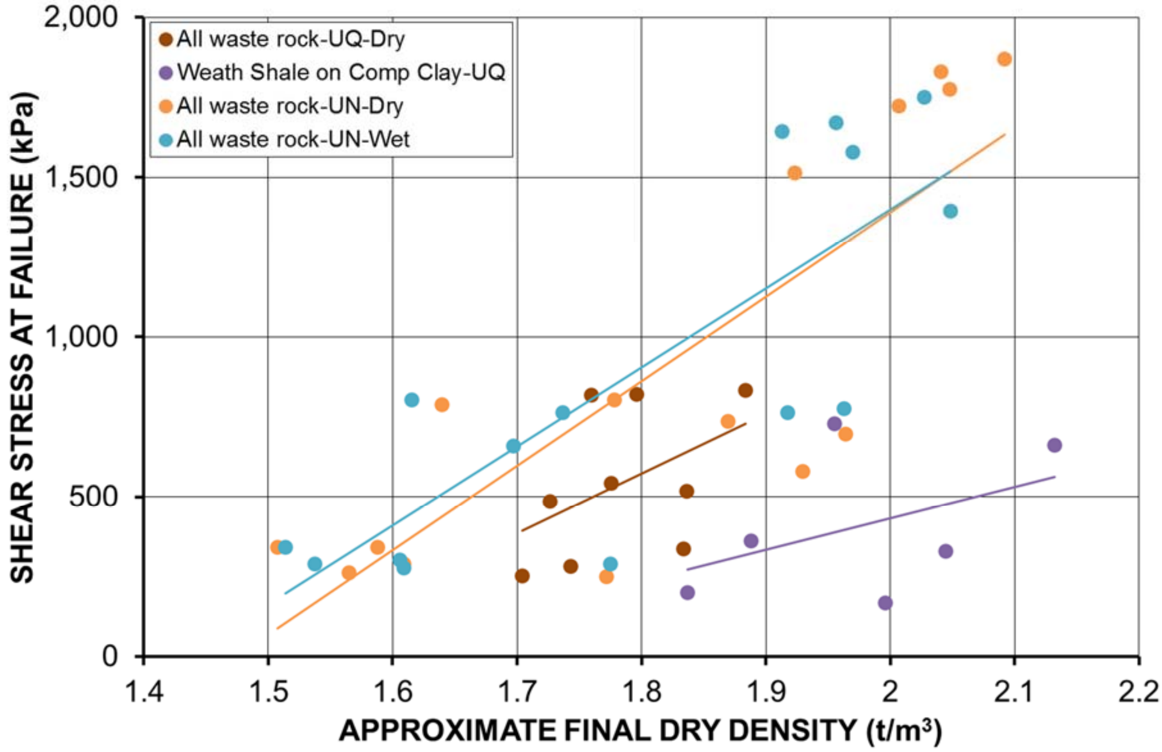


Figure 22 Comparison between UQ and UN shear stress increases with increasing final dry density due mainly to increasing applied normal stress

5. INTERPRETED SHEAR STRENGTH PARAMETERS

The shear strength parameters interpreted from the results of the direct shear testing carried out at UQ and UN are described in the following sections.

5.1 Mohr-Coulomb Interpretation

Figure 23 presents the UQ direct shear test results grouped under: (i) dry testing of waste rock, (ii) dry testing of Weathered Shale over compacted Clay, and (iii) wet testing of Weathered Shale over compacted Clay. Included in Figure 23 are the best-fit Mohr-Coulomb shear strength parameters for each group of test results.

Figure 24 presents the UN direct shear test results grouped under: (i) dry testing of the five waste rock types, and (ii) wet testing of the same waste rock types. Figure 19, and Figure 24 in particular, show little spread between the results for the different waste rock types, and only a slight flattening of the best-fit lines from dry to wet testing of a given waste rock type. Included in Figure 24 are the best-fit Mohr-Coulomb shear strength parameters for each group of test results, which show a higher friction angle and lower apparent cohesion for the dry tests compared with the wet tests. Adopting the Mohr-Coulomb straight line shear strength criterion, the apparent cohesion values and friction angles for all UQ and UN direct shear test data are plotted in Figure 25. The spread of apparent cohesion and friction angle values obtained makes it difficult to select appropriate shear strength parameters for use in geotechnical slope stability analyses. Figure 26 groups the data into UQ and UN, and dry and wet testing, which also groups the apparent cohesion and friction angle values. However, it is still it difficult to select appropriate shear strength parameters for use in geotechnical slope stability analyses.

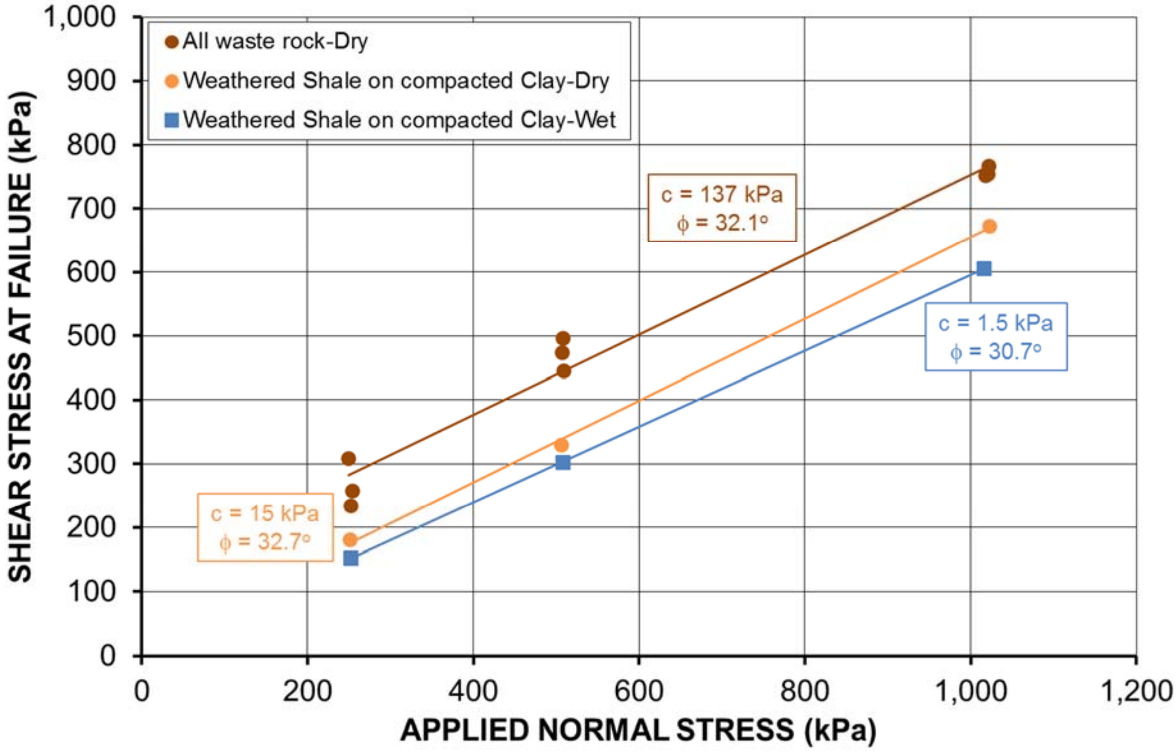


Figure 23 Grouped shear stress at failure versus applied normal stress data from UQ tests

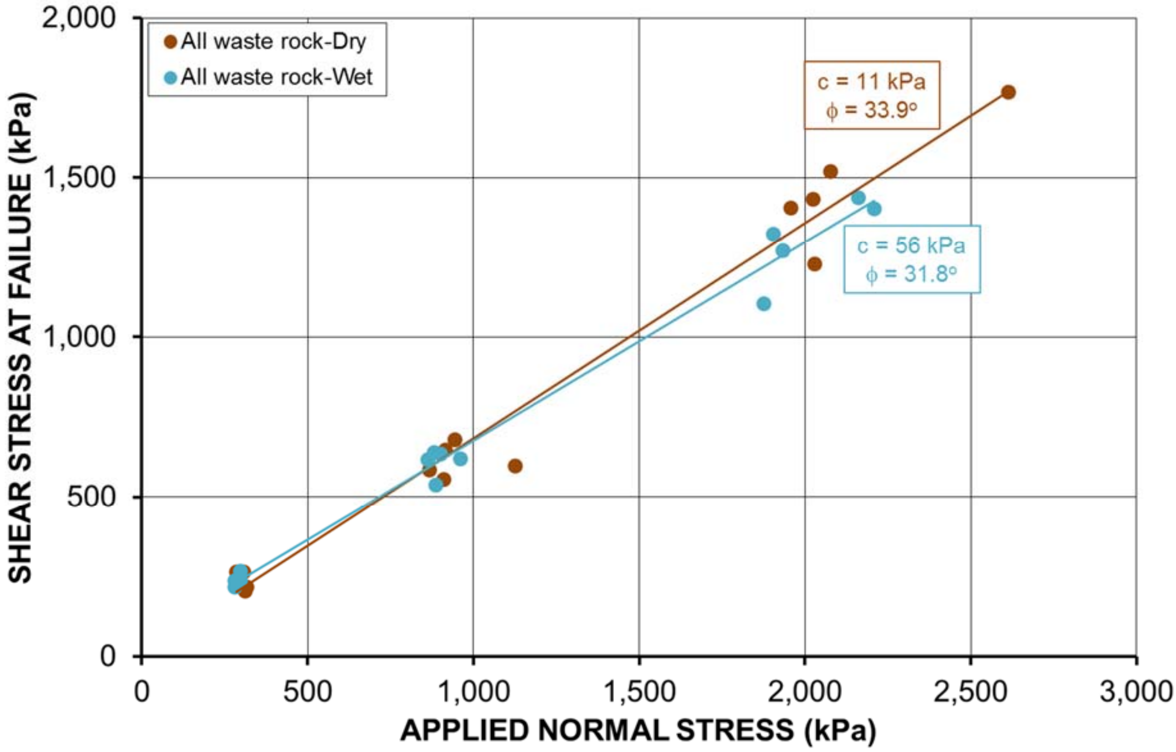


Figure 24 Grouped shear stress at 10% shear strain versus applied normal stress data from UN tests

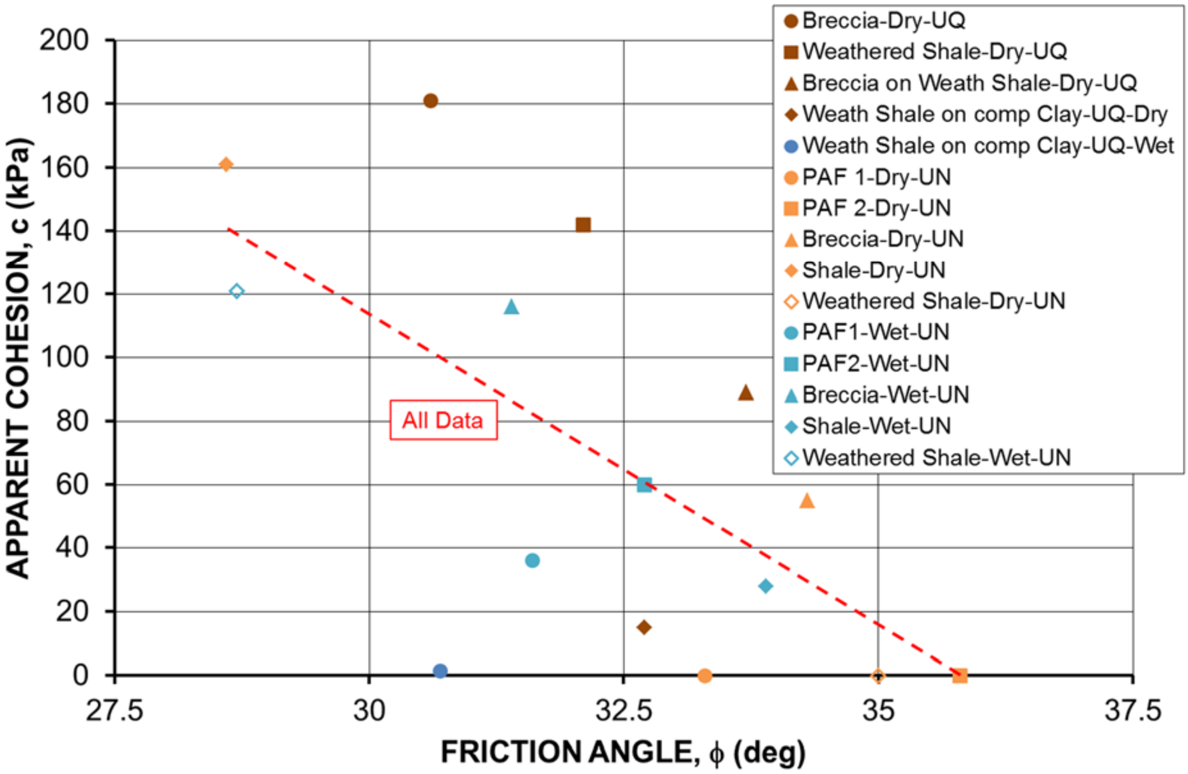


Figure 25 Mohr-Coulomb apparent cohesion versus friction angle for all UQ and UN direct shear test data

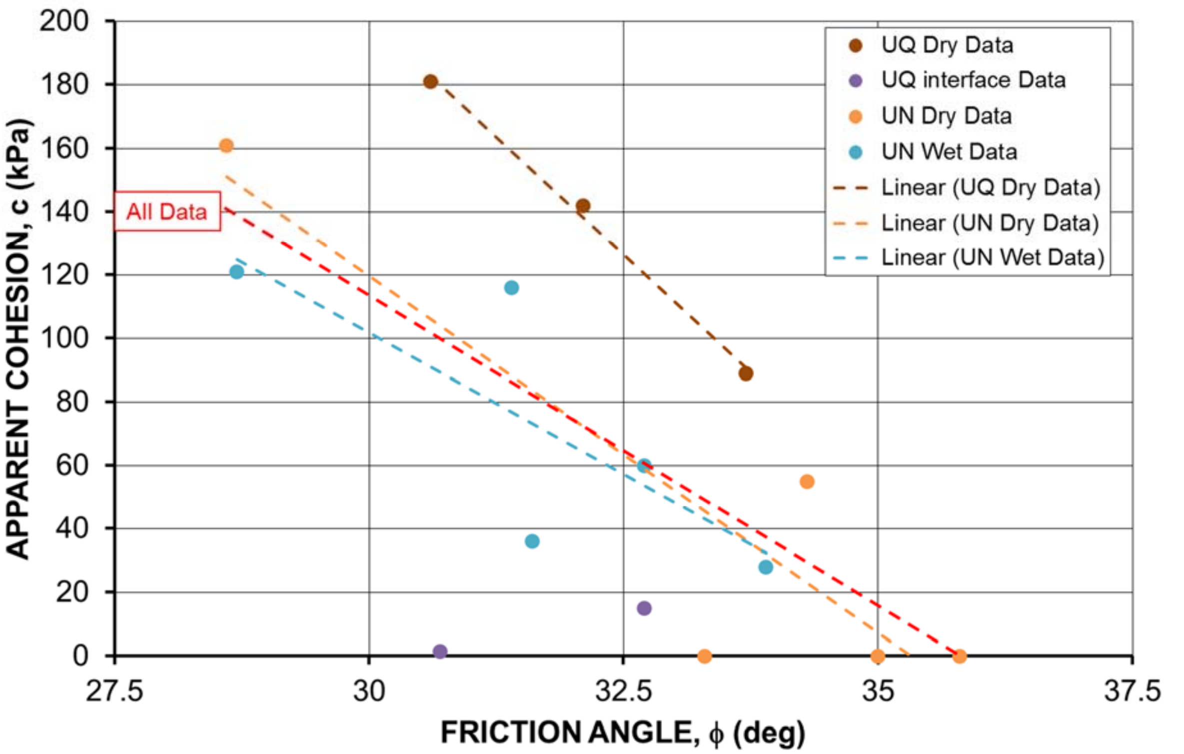


Figure 26 Mohr-Coulomb apparent cohesion versus friction angle for grouped UQ and UN direct shear test data

5.2 Alternative Interpretations

An alternative interpretation is to consider the shear strength simply in terms of secant friction angles at each applied normal stress, for each material tested. The secant friction angles calculated for all materials subjected to direct shear testing at UQ and UN are plotted in Figure 27, with the data grouped in Figure 28 according to direct shear box scale, material type and moisture state. Also shown in Figures 27 and 28 are the range of data from poor to good quality rock fill obtained from 200 mm diameter triaxial testing by Leps (1970), a typical angle of repose for loose-dumped waste rock at MRM of 37º, and an average applied stress of 900 kPa corresponding to about 50 m depth of waste rock. It can be seen from Figures 27 and 28 that the better quality MRM waste rock tested dry has secant friction angles within the range expected for rock fill and well above the angle of repose. The poorer quality MRM waste rock, and the interface between waste rock and compacted Clay, particularly when tested wet, have inferior secant friction angles, as do the values obtained in the large-scale, stiff UN direct shear machine.

Care must be exercised when comparing friction angles with the angle of repose of a material. Waste rock would be expected to have a friction angle of typically 4 to 6º higher than the angle of repose of the material on loose-dumping, due to the effects of overburden stress (Williams, 2015). Potential geotechnical instability of a loose-dumped, angle of repose, frictional waste rock slope would necessarily be shallow, implying a low applied normal stress and a high friction angle.

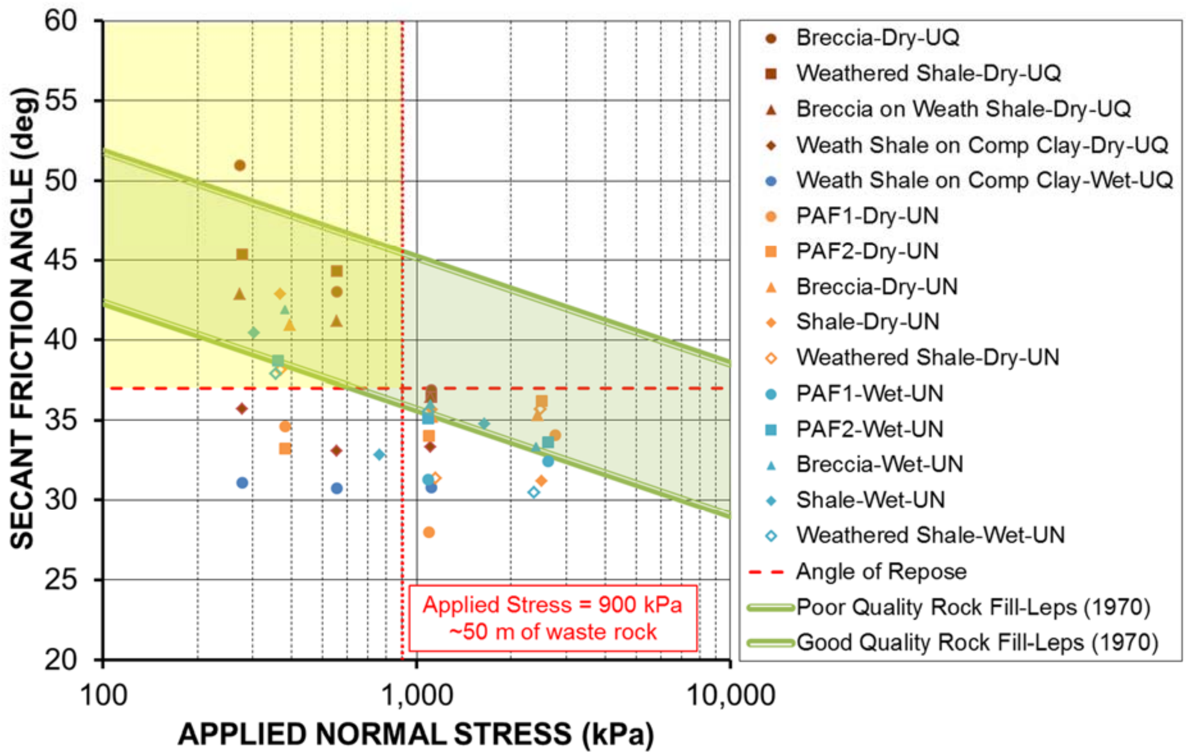


Figure 27 Secant friction angle versus applied normal stress for all UQ and UN direct shear test data, compared with data from Leps (1970)

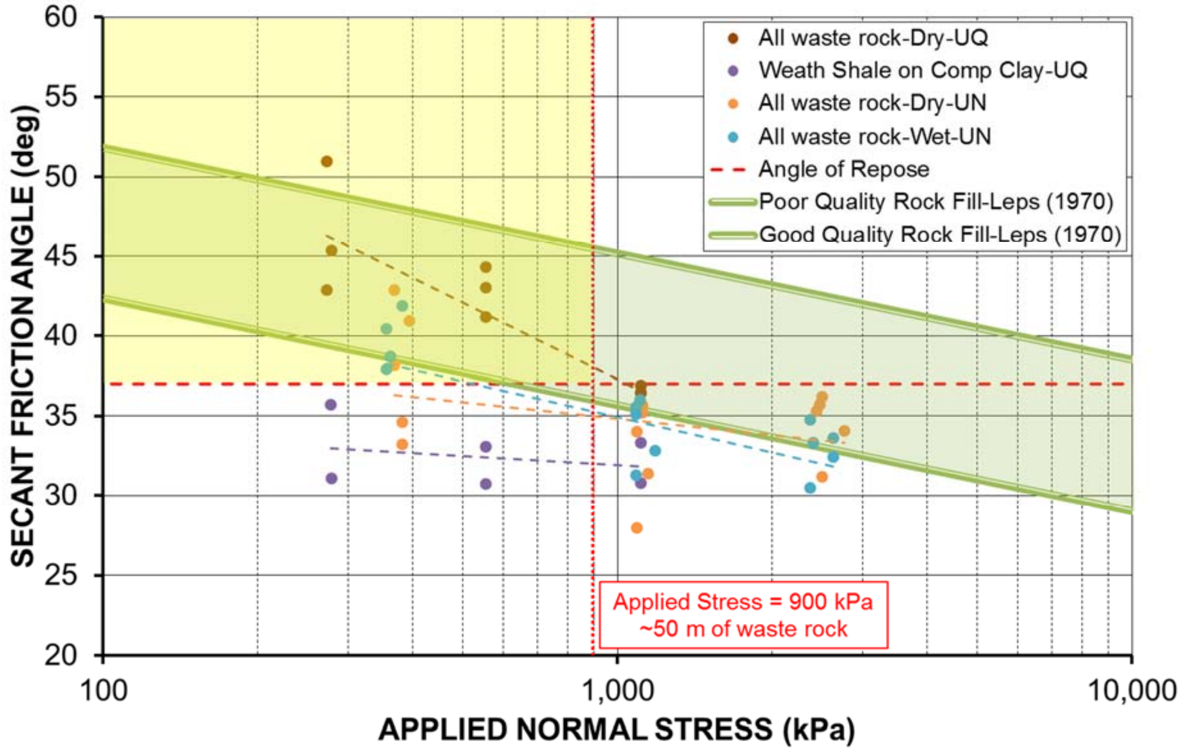


Figure 28 Secant friction angle versus applied normal stress for grouped UQ and UN direct shear test data, compared with data from Leps (1970)

5.3 Effect of Size (and Stiffness) of Direct Shear Box

The effect of shear box size, and hence its stiffness, is illustrated in Figures 29 and 30. In Figure 29, shear strengths are calculated using the appropriate calculated Mohr-Coulomb shear strength parameters for shallow (5 m) and deep (50 m) locations within a waste rock dump. The calculated shear strengths based upon the UQ-derived shear strength parameters are seen to be higher than those based upon the UN-derived shear strength parameters.

In Figure 30, the shear strengths are expressed in terms of secant friction angles. Again, the UQ-derived secant friction angles are seen to be higher than the UN-derived secant friction angles, with the former plotting above the angle of repose of the waste rock, and the latter straddling the angle of repose. The necessarily very high stiffness of the large UN direct shear machine is farthest from *in situ* conditions, in which there is no stiff boundary laterally encapsulating the material. Hence, the shear strength results obtained using the UN machine are more adversely affected by machine stiffness than those obtained using the smaller and less stiff UQ machine, despite the larger machine being able to test particles up to twice as large. The larger particles tested in the UN machine are more prone to crushing on testing, and all particles will undergo more crushing in this machine (and hence loss of shear strength) due to the higher stiffness and higher applied normal stresses.

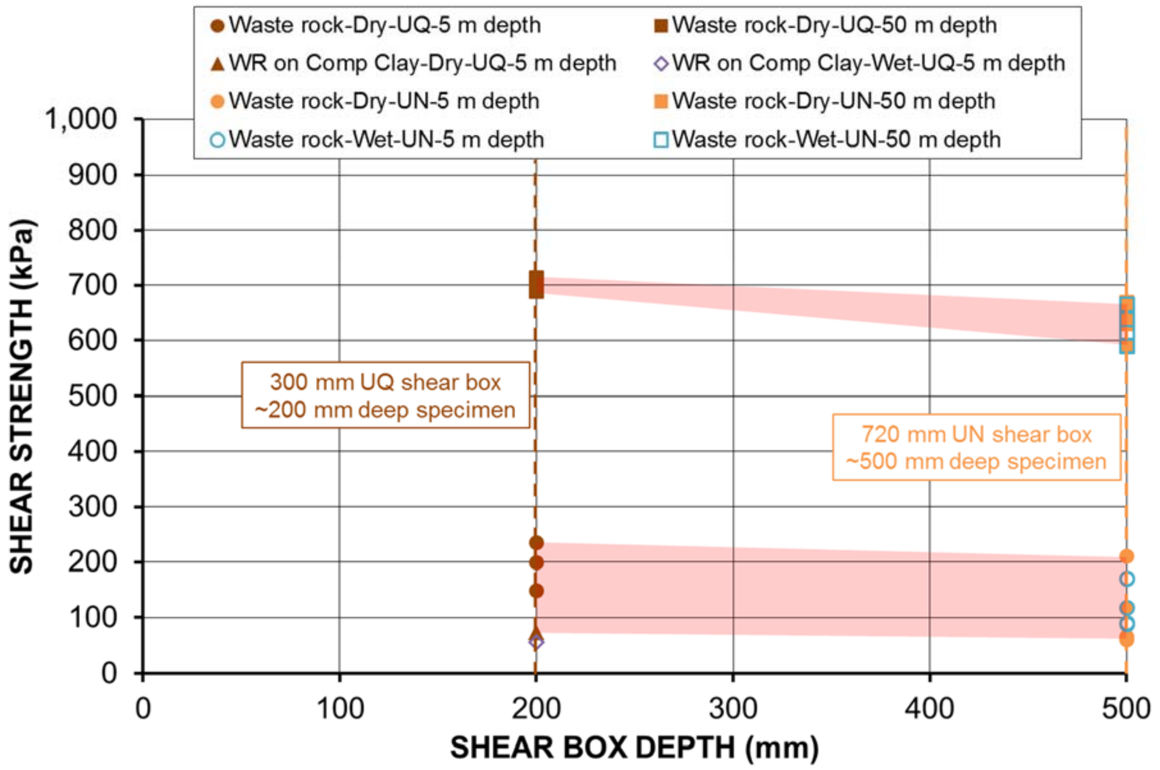


Figure 29 Shear strength at 5 m and 50 m depth versus shear box depth for grouped UQ and UN direct shear test data

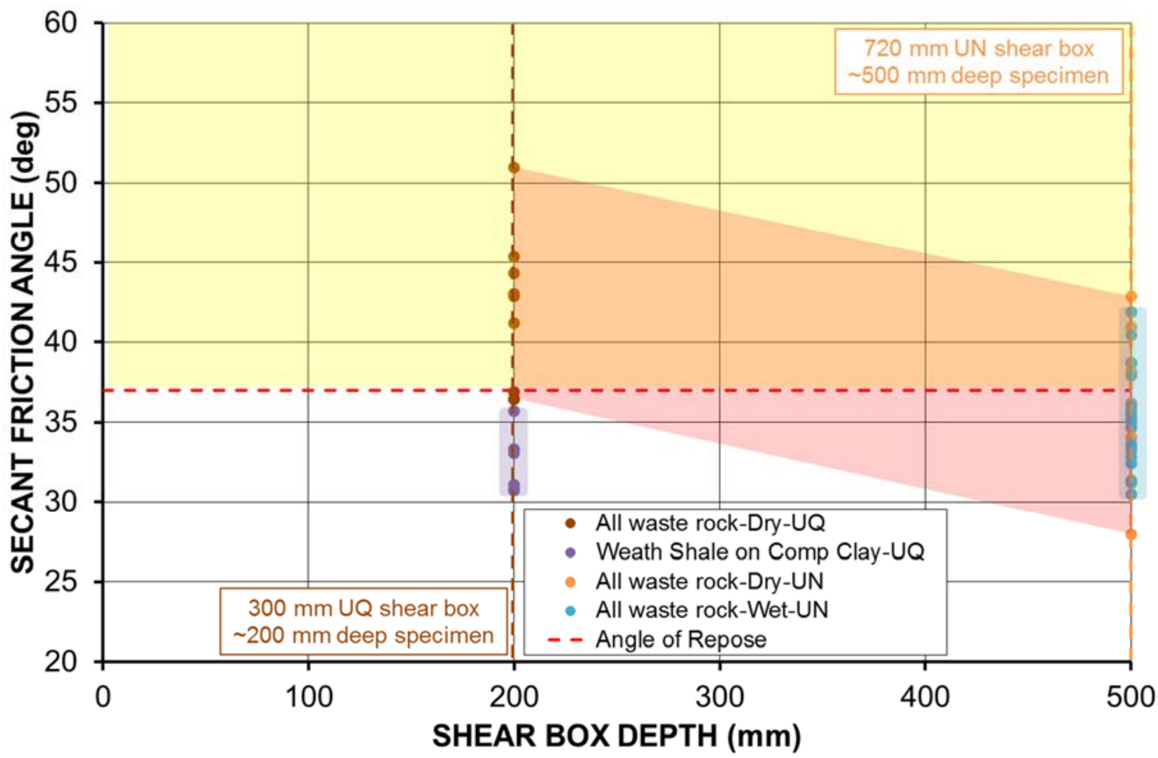


Figure 30 Secant friction angle versus shear box depth for grouped UQ and UN direct shear test data

6. RECOMMENDED SHEAR STRENGTH PARAMETERS

In recommending shear strength parameters for the MRM waste rock types and waste rock/compacted clay interfaces, account must be taken of whether the materials are best described as frictional, the depth of interest, their expected moisture state *in situ*, and whether or not the waste rock is likely to degrade.

Waste rock and waste rock/compacted clay interfaces are largely frictional, but with a significant suction-induced apparent cohesion. Since the waste rock will be relatively free-draining, it is never likely to saturate, and suction-induced apparent cohesion can be relied upon. Being largely frictional, the depth of interest with respect to potential geotechnical slope instability is shallow (see Williams, 2015, included as Appendix A). Based upon dry versus wet sieving of the MRM waste rock, and slake durability testing, the MRM waste rock does not degrade significantly.

Since it is likely that scalping will reduce the friction angle of coarse-grained waste rock, the laboratory-derived friction angles are likely to be conservative by up to several degrees. It is worth noting that angle of repose slopes formed by loose-dumping of waste rock, including at MRM, are generally geotechnically stable. They are more susceptible to erosion on over-topping by rainfall runoff.

The larger particles tested in the UN machine are more prone to crushing on testing, and all particles will undergo more crushing in this machine (and hence loss of shear strength) due to the higher stiffness and higher applied normal stresses.

The recommended shear strength parameters are:

- Near the surface:
  - Apparent cohesion = 50 $\pm$ 25 kPa
  - Friction angle = 40 $\pm$ 3^{o}
- Within the waste rock:
  - Apparent cohesion = 100 $\pm$ 50 kPa.
  - Friction angle = 35 $\pm$ 3^{o}.
- On waste rock/compacted clay interfaces:
  - Apparent cohesion = 20 $\pm$ 10 kPa.
  - Friction angle = 33 $\pm$ 3^{o}.

It is recommended that these average and ranges of shear strength parameters be applied in sensitivity analyses of geotechnical slope stability of the MRM waste rock dump.

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Williams, D.J. (2015). How stable are angle of repose coarse-grained mine waste slopes? *Proceedings of International Symposium on Slope Stability in Open Pit Mining and Civil Engineering, Cape Town, South Africa, 12-14 October 2015*, 13 pp.

APPENDIX A – Paper by Williams (2015)

How stable are angle of repose coarse-grained mine waste slopes?

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Abstract

There is ongoing confusion about the relationship between the angles of repose of conventionally end-dumped, coarse-grained mine wastes, and their internal friction angles. Coarse-grained mine wastes include mine waste rock generated on the open pit mining of hard rock ore bodies, spoil generated on the open pit mining of coal, the coarse-grained wastes (predominantly coarser than 0.06 mm) generated on the mineral processing and smelting of metalliferous ores (known as scats), and coarse reject generated on the washing of run-of-mine coal. These wastes are often initially relatively dry and are conventionally end-dumped off a tip-head in dumps or piles, undergoing ravelling at the angle of repose of the material. At their angle of repose, the wastes are generally geotechnically stable, at least in the short-term. However, geotechnical slope stability may be impaired as the wastes wet-up and/or degrade, as they saturate and lose matric suction, and if a phreatic surface develops within the dump or pile. The paper describes the relationships between the angles of repose of coarse-grained mine wastes and their shear strength parameters, and the impacts on short-term and long-term geotechnical stability.

Introduction

In order to carry out a geotechnical slope stability analysis for a coarse-grained mine waste dump or pile, the slope angle and height of slope must be input, together with the unit weight and shear strength parameters of the waste. The angle of repose of coarse-grained mine wastes is often assumed to be 37°, although it can range from perhaps 35° for weak materials to 40° or higher for strong, durable materials (Williams 2001). The angle of repose is also affected by the height of the tip-head, any three-dimensional concavity of the slope, the particle size distribution and shape, and the moisture content of the wastes. The slope height may also be assumed, generally as some nominal value for the particular mining operation. The unit weight of the material is difficult to measure and is also often assumed, typically at a value of approximately 18 kN/m^3. The applicable unit weight to use in stability analyses is the total unit weight, which will increase as the material wets-up and saturates, and this is not always allowed for. At the same time, wetting-up will generally cause a reduction in the shear strength of the material.

The shear strength parameters of coarse-grained mine wastes are difficult to measure due to their often large particle sizes, and values are also often assumed. Coarse-grained mine wastes are often assumed to be purely frictional, and their internal friction angle is often, wrongly, taken to be the angle of repose, while it is typically approximately 6° greater than the angle of repose due to the densifying and confining effects of the overburden stress. The shear strength parameters are also affected by matric suction (increasing the shear strength), saturation (reducing the shear strength), material degradation on dumping and subsequent wetting-up in the dump (also reducing the shear strength), and the formation of secondary minerals on oxidation (which may increase or reduce the shear strength).

Angle of Repose

On end-dumping from a truck, waste rock or spoil ravels (falls) downslope at the angle of repose of the material. The angle of repose of granular material is affected by (Rowe 1962):

- particle size, shape and surface roughness (increasing with increases in size, angularity and roughness);
- specific gravity of the particles (increasing with increasing specific gravity);
- height of fall (decreasing with increasing height of fall);

- amount of pore water present (increasing with the addition of limited pore water, but decreasing as water-saturation is approached);
- curvature of the slope in plan (concave slopes being more stable than planar and convex slopes due to arching effects);
- base conditions; and
- whether the slope is natural or artificial.

Simons and Albertson (1960) presented angle of repose data that show an increase in the angle of repose of granular materials with increasing mean particle size. The more angular the particles, the higher the angle of repose will be. The data show, for instance, that the effect of scalping (the removal of particles larger than a certain size) to allow laboratory testing may reduce the angle of repose of the scalped material by 6^o compared with the field value for the full-scale material. The field value would typically be in the range from 37 to 40^o for durable, angular mine waste rock.

Rapid dumping will lead to an initial over-steepening of the upper part of the slope. On continued rapid dumping, the over-steep section of the slope will lengthen, possibly leading to a slip. Over-steepening results from the apparent cohesion of fines hanging-up near the crest of the slope, while the coarse particles ravel to the base of the slope.

The angle of repose will also be affected by the degradation of particles over time. The production of fines on degradation will normally be accompanied by an increase in the density of the material and a slight flattening of the slope, making it more stable with time. Provided that the slope does not fully saturate, it will remain at this angle in the long-term; although some erosion would likely occur. Full saturation by inundation could gradually reduce the slope angle, to as low as half the initial angle of repose for materials that are prone to water-softening (Stratham 1974).

At large strains (typically greater than 10%) and in a loose state, such as occur on the ravelling of coarse-grained mine wastes down an angle of repose slope, the friction angle of the material reverts to its ultimate value. The associated void ratio is termed the critical state void ratio (Roscoe, Schofield and Wroth 1958) and the friction angle is designated the critical state friction angle ϕ_{cv}, numerically equivalent to the angle of repose at which the material ravels.

Internal friction angle

The strength of dry, fresh, durable waste rock or spoil is characterised by a substantial friction angle and zero cohesion, with the value of the friction angle a function of:

- particle size distribution (reducing with decreasing particle size);
- particle shape and surface roughness (increasing with increasing angularity and surface roughness);
- strength and specific gravity of individual particles;
- state of packing (increasing with increasing density); and
- applied stress level (decreasing with increasing stress, resulting in a curved strength envelope passing through the origin).

It has long been recognised (Holtz and Gibbs 1956; Holtz 1960) that an increase in the proportion of coarse material in an otherwise fine-grained granular soil can result in an increase in its friction angle. Alternatively, when the voids within a coarse granular material are filled with fines, its friction angle is increased by as much as 10^o (Stratham 1974). The amount of fines required to have a significant effect on the friction angle is relatively small (<10% by mass).

The limitation on the maximum particle size of a coarse-grained waste rock or spoil that can readily be strength-tested in the laboratory will result in a low estimate of the friction angle of the coarser whole material. Over time, the friction angle will increase further due to infilling by the fine-grained products generated on degradation.

Leps (1970) presented friction angle data based on triaxial strength testing of coarse-grained (up to 200 mm) rock fill particles. These data suggest that the friction angle of durable spoil could be as high as 55^o at low applied stress, and is likely to be at least 50^o at moderate stress levels. Durable waste rock or spoil would be expected to have a friction angle in the range from 40^o to 50^o, the lower end of the range corresponding to weathered or crushed, fine-grained material, and the upper end of the range corresponding to fresh, coarse-grained material. Spoil generated from open cut coal mining would be expected to have a substantially lower friction angle, due to its generally poor cementation, but it would have some apparent cohesion.

For medium dense sandy gravel materials, the value of the lower bound critical state friction angle would be expected to be 4º to 6º less than the peak friction angle (Lambe and Whitman 1979). Together with Leps (1970) data, this suggests that durable waste rock or spoil would be expected to have a peak friction angle of typically 45º, substantially higher than the angle of repose of 37 to 40º typically achieved for durable, angular mine waste rock or spoil.

The following paragraphs discuss the effects of segregation, slope profile, suction, a high water table and rainfall, tree roots, material degradation, progressive failure, and post-failure stability.

Effect of segregation

Waste rock and spoil tend to segregate according to particle size (and also according to specific gravity if this varies) during end-dumping from a truck, due to a combination of Bagnold's grain dispersive pressure and particle kinematics (Middleton 1970). Since the friction angle of granular material tends to increase with particle size, segregation tends to produce alternating weaker and stronger angle of repose layers. As the permeability of the material decreases with mean particle size, alternating layers of lower and higher permeability result. However, the continuity of the individual layers may be insufficient to produce clearly defined weak layers or preferred seepage flow paths.

Effect of slope profile

The stability of a granular slope for a given type of failure depends not only on the slope angle but also on the plan geometry of the slope (Azzouz, Baligh and Ladd 1983). Longitudinally concave slopes (steepest towards the crest and flattest towards the toe) tend to be intrinsically more stable than conventional straight, planar slopes (Schor and Gray 1995). Slopes that are concave in plan are stable at an angle of approximately 3º steeper than convex-shaped slopes, with planar slopes in between (Rassam and Williams 1999).

Effect of suction

Waste rock and spoil emerge from an open pit at a low gravimetric moisture content (mass of water/mass of solids, expressed as a percentage) of typically 2 to 5%, which equates to a volumetric water content (volume of water/total volume, expressed as a decimal) of 0.035 to 0.09 (Williams 2006). The precise moisture content depends on the climate, and the degree of cementation and weathering of the waste rock or spoil, and hence their particle size distribution and moisture retention characteristics. As a result of the initially low moisture content, the pore water within the voids between the waste rock or spoil particles will be under negative pore water pressure or suction. This suction will equate to an effective stress, which mobilises a measure of shear strength. This is often simplistically expressed as an apparent cohesion (Fredlund and Rahardjo 1993).

Even in a dry climate, a waste rock dump or spoil pile will allow ready infiltration of rainfall and will gradually wet-up. A proportion of the rainfall infiltration will go into storage within the voids in the dump, with any excess infiltrating further into the dump as partially-saturated "fingers", ultimately emerging as seepage at the toe and into the foundation. Due to its very low unsaturated hydraulic conductivity, the initially dry waste rock or spoil will store infiltration from light rainfall events, and a high waste rock dump or spoil pile of relatively dry material may be capable of storing considerable infiltration without significant breakthrough. The wetting front will progress through the dump or pile as the ability of the pores in the waste rock or spoil to store water is exceeded, this occurring well below the fully-saturated state, since the waste rock or spoil will have achieved a sufficiently high hydraulic conductivity to pass further rainfall infiltration.

For fresh, durable waste rock or spoil, which is open-pored, the pore space needs to reach only approximately 25% saturation to become sufficiently permeable to pass all rainfall infiltration. For more well-graded weathered waste rock, the pore space needs to reach approximately 60% saturation to become sufficiently permeable to pass all rainfall infiltration. Initial percolation into the foundation is also limited by the very low hydraulic conductivity of the unsaturated zone within the foundation. As the waste rock dump or spoil pile wets-up and undergoes settlement involving the compression of air voids, the waste rock or spoil will become more saturated and the suction will decrease. As a result the apparent cohesion will decrease, although the friction angle may increase due to the greater density.

Effect of a high water table and rainfall

High water tables or phreatic surfaces reduce the factor of safety for deep-seated slope failures, possibly by a factor of two. Perched water tables may give rise to surficial failures, and may arise due to degradation of near-surface weak waste rock or spoil, which produces fines, preventing the infiltration of rainfall. The effect of water near the surface of a waste rock or spoil slope depends on the direction of any flow. The greatest reduction in the factor of safety (by a factor of up to two) occurs for flows parallel to the surface of the slope (Gray 1995), which is promoted by the angle of repose layering of waste rock or spoil end-dumped by truck.

Slope stability is influenced significantly by intense or prolonged rainfall (Chowdhury and Nguyen 1987). If waste rock or spoil remains stable during placement, subsequent failures are generally rainfall-related. However, direct correlations between rainfall intensity and the frequency and magnitude of waste rock or spoil slope instability have not been established. In dispersive waste rock or spoil (Emerson and Seedsman 1982) sinkholes can develop as a consequence of rainfall infiltration.

Effect of tree roots

The factor of safety for surficial failures may be increased significantly by the roots of any trees that may become established on waste rock or spoil slopes (Gray 1995). According to Hubble and Hull (1996), the increase in strength due to the presence of the roots is more than sufficient to offset the extra loading due to wind acting on the trees, although contrary views have been expressed (Brown and Sheu 1975; Barker 1995). Surficial sliding or planar failures limited to a depth of 250 mm can be stabilised by normal engineering methods, plus vegetation (Lawrance 1995). The removal of inappropriate shrub and tree vegetation may result in significant benefits.

Integrated hydrology and slope stability models are required to adequately account for the effects of vegetation on slope stability (Collinson, Anderson and Lloyd 1995; Anderson and Lloyd 1991). The factor of safety of bare slopes with a high permeability (>10^{-5} m/s) may be increased by 20% due to the effects of tree cover. Although the factor of safety of a slope with a medium permeability (10^{-6} m/s) may be increased by 20% due to trees in the long-term, it may be reduced by 20% in the short-term as the tree cover is becoming established. The effects of the tree cover on the slope hydrology must also be considered (Collinson and Anderson 1996).

Effect of material degradation

The generation of clay-sized fines (finer than 0.001 mm) on the physical degradation of waste rock or spoil may reduce the friction angle by 2º or 3º (Seedsman and Emerson 1985). This reduction does not occur gradually, as the clay fraction increases, but relatively suddenly, at a clay content of approximately 10%. At this clay content, the larger particles in the waste rock or spoil are no longer in direct contact which each other, but tend to be supported in a matrix of clay-sized particles. Degradation may occur to a relatively shallow depth beneath the surface of a dry waste rock or spoil slope, or deep within a waste rock dump or spoil pile due to a fluctuating water table. The physical breakdown of coal mine spoil is a relatively short-term process (Taylor and Spears 1970). Chemical, possibly including biological, degradation can reduce the friction angle by 6 to 12º (Taylor 1984). The physical and chemical degradation of waste rock and spoil is exacerbated by dump construction involving loose end-dumping, compared with placement and compaction in thin layers, since end-dumping facilitates oxygen ingress through the base rubble zone and rainfall infiltration. Susceptibility to erosion must also be considered.

Progressive failure

In waste rock and spoil slopes, the shear strength may decrease significantly as a result of moisture softening. Moreover, waste rock and spoil can exhibit brittleness so that over-loading (by additional waste rock or spoil) can also lead to a marked decrease in shear strength. Progressive phenomena are important in such circumstances (Chowdhury, Nguyen and Nemcik 1986).

Since moisture softening is important in promoting progressive failure, the extent of rainfall infiltration, the wetting sequence, and the development of pore water pressures in a submerged waste rock dump or spoil pile, are of considerable importance. Other important factors include the development of tensile failure in the material and the potential for slip at the interface between old and recently-placed material.

Delayed waste rock and spoil slope failures can occur, usually associated with rainfall infiltration, and it may be necessary to monitor waste rock and spoil slopes over a number of years and use the observational data to update stability analyses.

Post-failure stability

Following the failure of a waste rock or spoil slope, a steep back-scarp is often observed, which can be as steep as 60^{o} to the horizontal, and can remain stable at that angle for many years. This begs the question about what shear strength parameters must be operating. In fact, the steep slope is primarily the result of over-consolidation and high suctions.

Geotechnical slope stability of angle of repose coarse-grained mine waste slopes

Taking into account the previous discussion of the angle of repose and friction angle of waste rock and spoil, the geotechnical slope stability of angle of repose coarse-grained mine waste slopes is now considered.

Some simple geotechnical slope stability calculations

The aim of the geotechnical slope stability calculations of angle of repose coarse-grained mine waste dumps or piles is to answer the question posed in the title of the paper: "How stable are angle of repose coarse-grained mine waste slopes?" For simplicity, consideration is restricted to circular slip within homogeneous slopes, which facilitates hand calculations. This approach enables readers to readily check the calculations, and removes the vagaries of the many computer codes available for circular slip analysis.

Interpretation of shear strength parameters

Conventionally, a straight line shear strength envelope is applied to experimental strength data, resulting in general in an apparent cohesion intercept **c** and a friction angle ϕ, which defines the slope of the envelope. This is described by the well-known Mohr-Coulomb equation:

$$\tau = c + \sigma_v'.\tan \phi \qquad [1]$$

where τ is the shear strength, and σ_v' is the vertical effective stress. A further interpretation is made about the drainage state. Rapid loading of fine-grained soils is assumed to be so rapid as to render the test undrained, while in the long-term soils are assumed to be drained. In laboratory shear strength testing in a direct shear box, drainage cannot be controlled, while in laboratory triaxial shear strength testing it can be, at least external to the specimen. However, the shear strength parameters **derived** from laboratory testing often cast doubt about whether the specimens were in fact undrained or drained. The fact is that **c** and ϕ, and often the drainage state, are artifacts of the interpretation imposed on the shear strength behaviour or soils, and there is no way of checking whether a given soil or waste rock or spoil "feels" cohesive or frictional, or undrained or drained.

Effect of shear strength parameters on geotechnical slope stability

Before detailing the slope stability methodology and calculations, it is worth highlighting the effect of the balance between apparent cohesion **c** and friction angle ϕ on the location of the critical slip circle. This is illustrated in Figure 1. Slopes in soils, waste rock or spoil that is predominantly cohesive will generate deep-seated failures, while slopes in predominantly frictional materials typical of durable rock will generate shallow failures. Intermediate **c,** ϕ materials will generate intermediate-depth failures. These are important considerations when interpreting and assigning shear strength parameters.

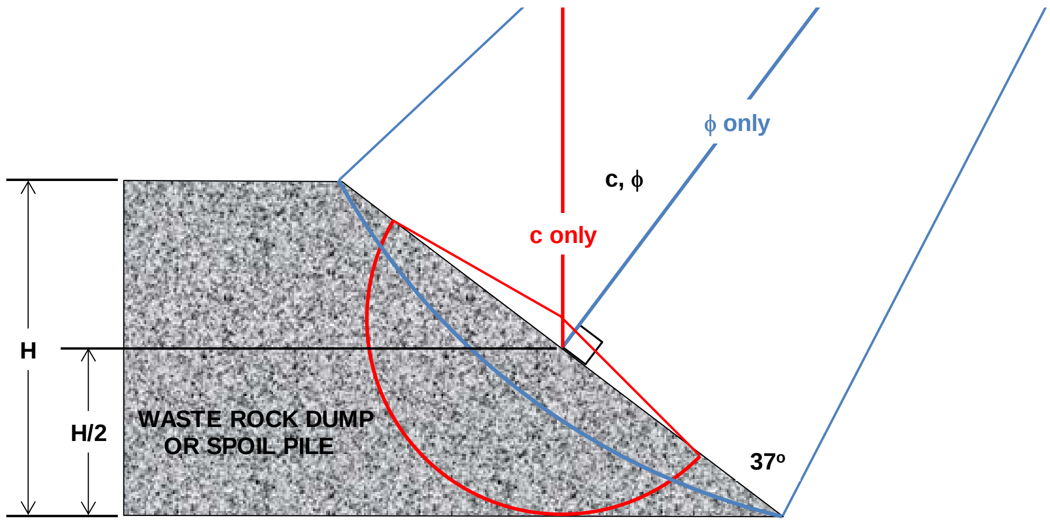

Figure 1 Effect of balance between apparent cohesion and friction angle on location of critical slip circle

Methodology

The hand calculation analysis selected is the Bishop and Morgenstern (1960) method. This simple analysis restricted consideration to single waste rock or spoil slopes at the angle of repose of the material.

The basic equation involved in the Bishop and Morgenstern (1960) method is:

$$\mathbf{F = m - n.r_u} \qquad [2]$$

where **F** is the factor of safety; **m** and **n** are stability coefficients, which are a function of the depth to a hard stratum, the dimensionless cohesion factor **c' / γ.H** (where **c'** is the effective cohesion, γ is the total unit weight of the slope material, and **H** is the overall height of the slope), and the effective friction angle ϕ'; and $\mathbf{r_u}$ is the average pore pressure ratio given by:

$$\mathbf{r_u = u / \gamma h} \qquad [3]$$

where **u** is the average pore water pressure within the slope, and **h** is the average depth to the slip surface.

Assumptions

For the purposes of analysis, the following assumptions have been made:

- Consideration is restricted to a single lift, surface waste rock dump or spoil pile formed by end-dumping off a tip-head, resulting in loosely-dumped waste rock or spoil, with an assumed angle of repose of 37° (or 1.3 horizontal to 1 vertical).
- Dump/pile heights of 15 m, 30 m, 60 m and 120 m are considered, which are reasonably representative of the range of waste rock dump and spoil pile heights.
- The foundation beneath the waste rock dump or spoil pile is assumed to be horizontal and represents a hard stratum through which the slip surface will not pass.
- The waste rock or spoil is assumed to be homogeneous, with uniform shear strength parameters and a constant γ assumed to be 18 kN/m^3. This is an over-simplification, but is required to enable the use of a simple hand calculation, and is sufficiently rigorous to demonstrate the salient features of the geotechnical stability of a waste rock or spoil slope. A conventional end-dumped waste rock dump or spoil pile will in fact have a base rubble zone due to the ravelling of the coarsest particles to the base; discontinuous alternating coarse-grained and fine-grained angle of repose layers, and a traffic-compacted top surface layer.
- The average pore water pressure is assumed to be zero, since for a surface waste rock dump or spoil pile this is not unreasonable for a dry climate such as exists over much of Australia.

Shear strength parameters considered

The shear strength parameters considered are summarised in Table I, with basic references provided by way of justification of the values selected, which also relate to the experience of the author.

Table I Shear strength parameters considered

MATERIAL	STATE	c' (kPa)	ϕ' (deg)	REFERENCE
Waste rock	Dry, fresh, durable	0	45	Williams (2001)
	Moist, weathered	40 *	42	
	Wet, weathered	20	40	
Spoil	Dry, cemented	25	40	Seedsman *et al.* (1988); Simmons (1995)
	Dry, weathered	75	37	
	Moist, weathered	150 *	35	
	Wet, weathered	100	20	

* Due to a combination of effective stress and suction-induced apparent cohesion.

In order to estimate the corresponding factors of safety using the Bishop and Morgenstern (1960) method, which was developed for soil slopes with lower shear strengths than those of waste rock or spoil, and for flatter slopes, it was necessary to extrapolate the charts to cater for higher shear strengths and steeper slopes, and to interpolate the charts for intermediate values of the dimensionless cohesion factor **C = c' / γ.H**. The Bishop and Morgenstern (1960) method was used to develop equations for the factor of safety **F** as a function of friction angle ϕ, and for **F** as a function of the dimensionless cohesion factor **C**. These are shown in Figures 2 and 3, respectively.

Estimated factors of safety

The factors of safety estimated for the shear strength parameters considered are summarised in Table II and in Figure 4. It is clear that for the shear strengths selected, the factor of safety against the geotechnical slope instability of dry, angle of repose, surface, coarse-grained mine waste dumps or piles remains above unity. Further, it is strongly influenced by slope height, due to the relatively high apparent cohesion values for all materials except the purely-frictional dry, fresh, durable waste rock (for which it is constant with slope height at a value of 1.3).

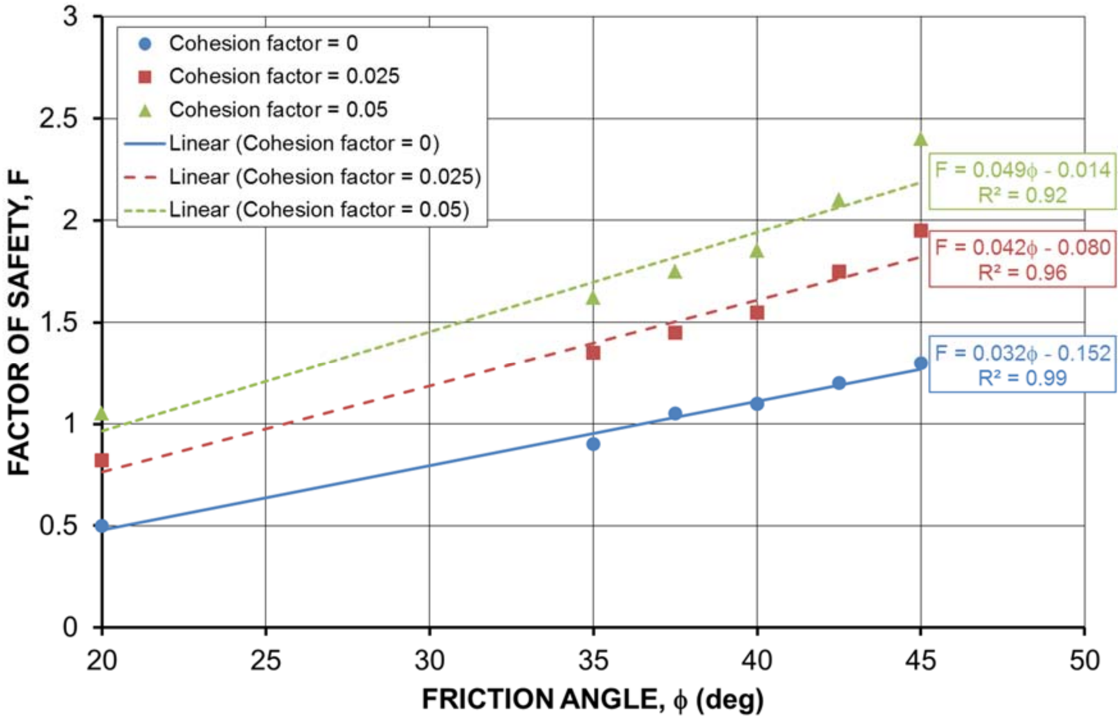

Figure 2 Estimated factor of safety versus friction angle

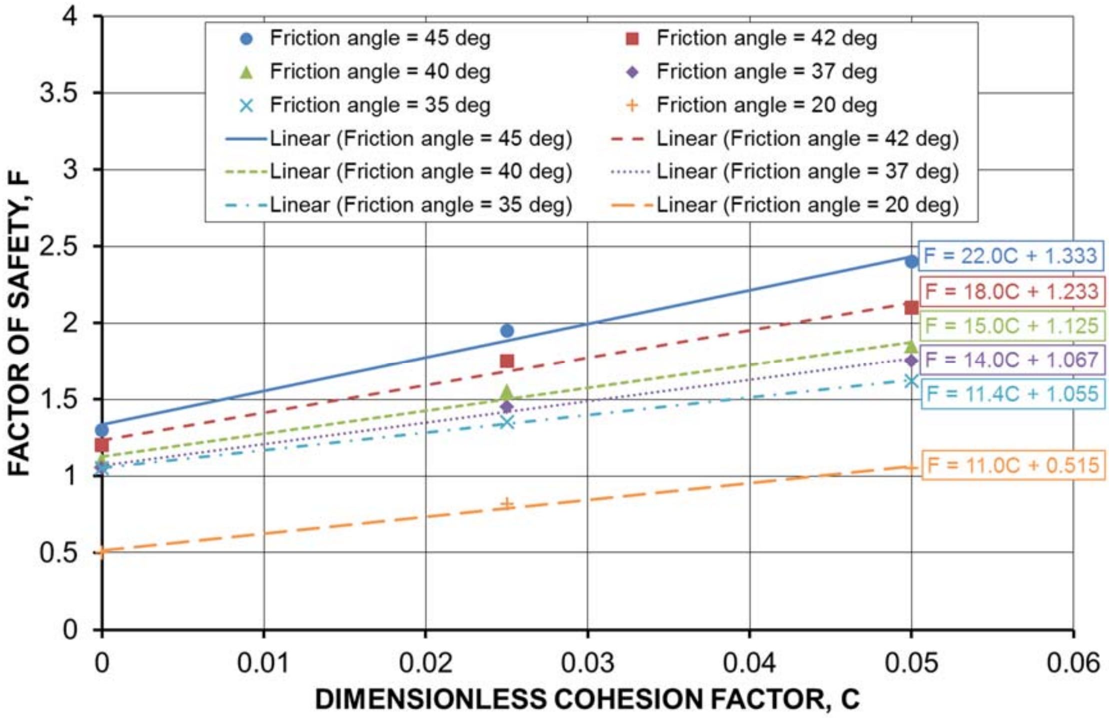

Figure 3 Estimated factor of safety versus dimensionless cohesion factor

Table II Estimated factors of safety for 37º angle of repose slopes of various heights

MATERIAL	STATE	H = 15 m	H = 30 m	H = 60 m	H = 120 m
Waste rock	Dry, fresh, durable	1.33	1.33	1.33	1.33
	Moist, weathered	3.90	2.57	1.90	1.57
	Wet weathered	2.24	1.68	1.40	1.26
Spoil	Dry, cemented	2.51	1.82	1.47	1.30
	Dry, weathered	4.96	3.01	2.04	1.55
	Moist, weathered	7.39	4.22	2.64	1.85
	Wet, weathered	4.59	2.55	1.53	1.02

Other factors affecting geotechnical slope stability of mine waste slopes

There are many other factors affecting the geotechnical slope stability of mine waste dumps or piles. These include the selection of an appropriate factor of safety, the safe operation of plant on mine waste slopes (usually for rehabilitation purposes), the settlement of waste rock and spoil, the bearing capacity of waste rock and spoil, and the erosion of waste rock and spoil slopes.

Appropriate factor of safety

The appropriate factor of safety or probability of failure for the stability of a waste rock or spoil slope depends on the coefficients of variation of the shear strength parameters (cohesion showing considerably more variation than friction angle) and the waste rock or spoil density (which shows relatively little variation), and on the acceptable failure rate. An acceptable annual failure rate may be 1/10,000th of the area of the waste rock or spoil slopes, corresponding to “dams” defined by Whitman (1984). However, the acceptable failure rate and the appropriate factor of safety also depend on the consequences of failure, including potential loss of life, damage to infrastructure, and loss of function, both on and off lease. The consequences of the possible failure of waste rock or spoil slopes are very difficult to assess, and will be very site-specific.

Generally, waste rock and spoil slopes designed with a factor of safety of 1.10 to 1.15 have only a minor risk of failure (Khandelwal and Mozumdar 1992; Miller *et al.* 1979). Waste rock and spoil slopes designed for a factor of safety of less than 1.10 are subject to a greater risk, even if the input data used are accurate, due to variability in the height of the slope, or of the shear strength of the waste rock or spoil, or of the foundation. These conditions may result in local fluctuations in the factor of safety of approximately 10%, leaving little safety margin.

The magnitude of the displacement of the waste rock or spoil during a slope failure may be a significant safety consideration. For example, large displacements may threaten roads, railways, or other structures. If the debris of a waste rock or spoil slope failure is removed, by a stream at the toe of the slope for example, the slope will retreat at the critical angle (the angle of repose for the material, in the absence of ground water). Otherwise, the slope angle will gradually reduce due to the loss of material from the crest, and the accumulation of debris and erosion products at the toe.

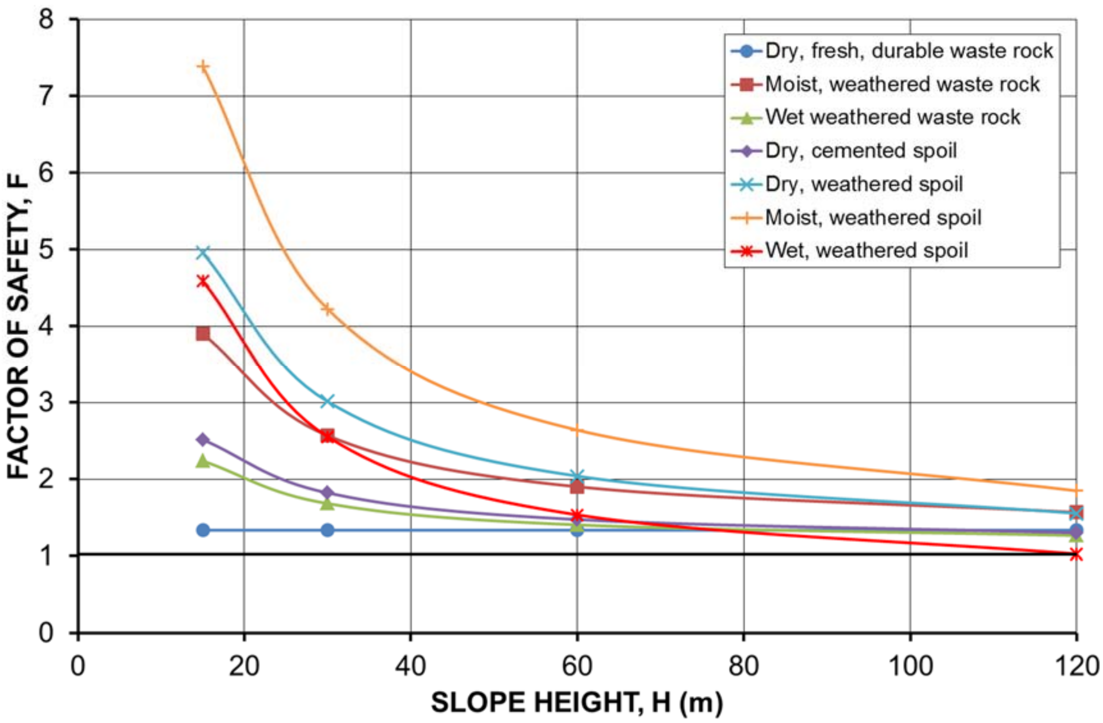

Figure 4 Estimated factor of safety versus slope height at 37º angle of repose

Safe operation of plant on mine waste slopes

Waste rock and spoil slope angles of 18º, (3 horizontal to 1 vertical) or flatter, allow machinery to work along the contours, constructing contour banks, seeding, and fertilising. A dozer can work up and down slope angles of up to 26º (2 horizontal to 1 vertical; Walker 1987), but operators are more comfortable working on slope angles of less than 22º. Conventional dozers suffer lubrication problems on steep slopes (Williams 1997) and, for this reason, most manufacturers will not warrant their equipment if used continuously on steep slopes.

Settlement of waste rock and spoil

Since waste rock and spoil is conventionally loose-dumped, it can suffer large settlements, which may be confused as slope instability. The mechanisms of waste rock settlement are not well understood, but could include the following:

- particle reorientation;
- water-induced weakening of inter-particle bonding;
- degradation (swell/slake) of high clay content materials; and
- dispersion and transport of fine particles through the coarse voids.

According to Williams (2012), the three components of coal mine spoil settlement are:

1. Self-weight settlement of initially dry spoil, including particle breakage under high stress, approximately 80% of which occurs during placement and hence is not "seen". It occurs at a rate that decreases exponentially with time.
2. "Collapse" settlement of the spoil on wetting-up, which perhaps requires only a 3 to 4% increase in gravimetric moisture content, sufficient to cause "corrosion cracking" at highly-stressed particle contacts, with possibly negligible further collapse as the spoil wets up further, since water merely fills the voids. Collapse settlement is often episodic, occurring during and after flooding rainfall events, and may be a one-off occurrence. Depending on the duration of placement, and the extent to which the spoil is wet-up, much of the collapse settlement may occur during placement and not be "seen".
3. Degradation-induced spoil settlement, which occurs over a variable timeframe depending on climate and spoil durability, within months or a year for weakly-cemented or uncemented rocky spoil, to decades or longer for cemented sandstones. Degradation-induced settlement is likely to be episodic, occurring following flooding rainfall events and subsequent dry periods (wetting and drying cycles). Depending on the duration of placement, and the extent to which the spoil is wet-up and its propensity for degradation on exposure to air and water, much of the degradation-induced settlement may occur during placement and not be "seen". Degradation-induced settlement is likely to approach a limit over time.

The magnitudes of post-construction settlement components of waste rock and spoil are self-weight-induced settlement of the order of 1.5% of the height for loose-dumped material, 1% collapse settlement and 1% degradation-induced settlement (Knipe 1979). The settlement of waste rock dumps and spoil piles has the effect of densifying the material, increasing its shear strength. Dump settlement also lowers the height of the dump slightly, and kicks out the toe of the slope slightly, and hence reduces the slope angle by approximately 2 or 3^o. These effects all serve to increase slightly the geotechnical stability of the dump. However, for coarse-grained wastes that degrade significantly on exposure to oxygen and water, any settlement-induced increase in geotechnical stability may be offset by a reduction in the shear strength of the degraded material.

Bearing capacity of waste rock and spoil

Plate bearing tests conducted by Naderian and Williams (1997) showed that the bearing capacity of mine waste rock or spoil is mainly a function of the compaction it has undergone, not its age. The compaction of backfill afforded by mine vehicle traffic appears to be sufficient to provide a reasonable bearing capacity and the ability to support ordinary light weight structures. Further, the densification that the material undergoes on loading results in an increase in its bearing capacity. Limited testing has shown that waste rock and spoil is capable of supporting stresses of up to 200 kPa, while sustaining settlements of the order of 5 mm and 20 mm for traffic-compacted and freshly-placed waste rock or spoil, respectively.

Erosion of waste rock and spoil slopes

Steep waste rock and spoil slopes can suffer extensive erosion, which again may be confused as geotechnical slope instability. Erosion rates are difficult to measure and even more difficult to predict with any accuracy. The accuracy of estimated erosion rates is often little better than $\pm$100% (So, 1999).

The Queensland Department of Mines and Energy (QDME) target erosion rate for rehabilitated mine sites is 12 to 40 t/ha/year (0.81 to 2.7 mm/year; QDME 1993). This range is much higher (by typically 30-fold) than natural erosion rates and river sediment yields, higher than erosion rates from agricultural land by typically four-fold, but lower than erosion rates from construction sites by approximately 10-fold.

The erosion rate is strongly influenced by the reconstructed slope profile (Meyer and Kramer 1969 in Lal 1990). Concave slopes deliver water to the toe of the slope with less erosion than uniform slopes of the same average slope (Loch 1999). Convex slopes display an exponentially increasing erosion rate with increasing slope length. The erosion rate on uniform slopes tends to be more proportional to slope length, and the erosion rate on concave slopes peaks at intermediate slope lengths (of the order of 50 m) before decreasing for longer slopes.

Haan *et al.* (1982) in Lal (1990) suggested that for slope angles up to approximately 8^o (14%), erosion loss is "transport limited"; the supply of erodable materials is plentiful and erosion is limited by the carrying capacity of the runoff. Under this regime, which applies to agricultural land use on relatively flat land, the erosion loss is directly proportional to the slope steepness. For slope angles steeper than approximately 8°, typical of mine slopes, erosion loss is limited by the ability of the runoff to detach particles from the slope. Under this regime, the erosion loss is not strongly dependent on the slope steepness.

The percentage surface cover (which could include grasses, trees, litter and rock) on soil erosional loss is dramatic, particularly where the surface cover comprises large elements (So *et al.* 1998). For vegetative cover alone, 50% or higher cover is required to dramatically lower erosion loss.

Erosion changes the geometry of an angle of repose, surface, coarse-grained waste dump or pile slightly due to a rounding of the crest and the deposition of erosion sediment at the toe, while the intermediate slope angle may remain at the settled angle of repose of the material. This serves to increase slightly the geotechnical stability of the dump. However, for coarse-grained wastes that degrade significantly on exposure to oxygen and water, any erosion-induced reduction in the effective height of the angle of repose section of a dump may be offset by a reduction in the shear strength of the degraded material.

CONCLUSIONS

This paper provided an extensive discussion on the factors influencing the geotechnical slope stability of angle of repose coarse-grained mine waste dumps or piles. The angle of repose and internal friction angle of waste rock and spoil were discussed, along with the effects of segregation, slope profile, suction, a high water table and rainfall, tree roots, material degradation, progressive failure, and post-failure stability.

Some simple geotechnical slope stability calculations were presented of dry, angle of repose, surface coarse-grained mine waste dumps or piles to answer the question posed in the title of the paper: "How stable are angle of repose coarse-grained mine waste slopes?" These were limited to circular slip and hand calculation using the Bishop and Morgenstern (1960) method. The cases considered were limited to single lift surface dumps 15 m, 30 m, 60 m and 120 m high, with no phreatic surface, and with the outer slope at a presumed angle of repose of 37°. An interpretation of shear strength parameters and their effect on geotechnical slope stability was also presented. The hand calculations showed that for the shear strengths selected, the factor of safety against the geotechnical slope instability of angle of repose coarse-grained mine waste dumps or piles remained above unity. Further, the calculated factor of safety was strongly influenced by slope height, due to the relatively high apparent cohesion values for all materials except the purely-frictional dry, fresh, durable waste rock (for which it is constant with slope height at a value of 1.3).

Among the other factors affecting geotechnical slope stability of mine waste slopes, the selection of an appropriate design factor of safety, the safe operation of plant on mine waste slopes (usually for rehabilitation purposes), the settlement of waste rock and spoil, the bearing capacity of waste rock and spoil, and the erosion of waste rock and spoil slopes were also briefly discussed.

The overall conclusion is that the outer angle of repose slopes of surface dumps or piles comprising coarse-grained mine waste are intrinsically geotechnically stable. However, there is a proviso that there is no significant mounding of the phreatic surface within the dump or pile, and the material in the dump or pile does not significantly soften due to the effects rainfall infiltration and atmospheric degradation of the waste on exposure to oxygen and water. Durable coarse-grained mine wastes will be relatively unsusceptible to degradation and softening, while weathered, clay mineral-rich and uncemented spoil materials may be highly susceptible to degradation and softening. Settlement and erosion of the dump or pile slope over time will increase slightly its geotechnical stability, provided that the material in the dump does not significantly soften due to atmospheric degradation.

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Williams, D. J. 2012. *ACARP C19022 Stage 1 Report Bulking and Subsequent Self-Weight and Saturation Settlements, and Geotechnical Stability, of High Coal Mine Spoil Piles*.

APPENDIX B – Curriculum Vitae

Professor David J Williams

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QUALIFICATIONS

1979	PhD, Soil Mechanics	University of Cambridge, England
1975	BE (Hons I), Civil Engineering	Monash University, Australia

AWARDS/DISTINCTIONS/FELLOWSHIPS

1996	Japan Society for the Promotion of Science Fellow
1995	The University of Queensland Collaborative Research Travel Grant
1995	Australian Minerals and Energy Environment Foundation (AMEEF) Travelling Scholarship
1993	Australian Research Fellow (Industry)
1992	AMEEF Environmental Excellence Award (Individual)
1990	Masuda Fellow for Collaborative Research in Japan, Jan-Feb
1989	The University of Queensland Collaborative Research Travel Grant

MEMBERSHIPS

From 1980	Member, Institution of Engineers, Australia
From 1980	Member, Australian Geomechanics Society
From 1984	Member, Queensland Committee, Australian Geomechanics Society, Chair in 1986
1986-1987	Member, National Committee, Australian Geomechanics Society
2007-2008	

EMPLOYMENT HISTORY

2007 – Present	Golder Professor of Geomechanics Director Geotechnical Engineering Centre School of Civil Engineering The University of Queensland

1994 – 2007	Associate Professor of Geomechanics Department of Civil Engineering The University of Queensland
1990 – 1994	Senior Lecturer in Geomechanics Department of Civil Engineering The University of Queensland
1983 – 1989	Lecturer in Geomechanics Department of Civil Engineering The University of Queensland
1980 – 1983	Geotechnical Engineer Melbourne and Brisbane Golder Associates Pty Ltd
1979 – 1980	Engineer Country Roads Board (CRB) of Victoria
1976 – 1979	Research Student University of Cambridge, England
1972 – 1976	Engineer, Cadet Engineer, CRB, Victoria

SUMMARY OF CONSULTING COMMISSIONS

Board Memberships

- Member of Northern Territory EPA Board, from 2012 to 2014

Peer Reviews of Major Projects

- Sole Independent Expert Geotechnical Reviewer for Rio Tinto Alcan Gove Residue Disposal Area from 2015
- Sole Independent Expert Geotechnical Reviewer and Annual Dam Inspections for QAL Residue Disposal Area and Ash Dams from 2013
- Sole Independent Expert Geotechnical Reviewer for Rio Tinto Alcan Yarwun Residue Management Area from 2013
- Led International Peer Review for the South Deposit TSF at Savage River Mine in Tasmania in 2012/13
- Sole Independent Expert Geotechnical Reviewer for Rio Tinto Alcan Weipa Tailings Storage Facilities in 2012 and 2014
- Peer Review of Harvey Creek Non-Erodable Waste Rock Dump Design for Ok Tedi Mining Limited in 2010/11
- Member of Expert Peer Review Team for Rio Tinto Alcan Weipa Tailings Storage Facilities from 2009
- Member of the International Technical Advisory Group reporting to the South Australian Government on Rehabilitation of Brukunga Pyrite Mine from 2007
- Led International Peer Reviews for the Savage River Rehabilitation Project in Tasmania in 2002, 2005, 2009 and 2013

- Led International Peer Review on handling acid generating waste rock dumping and dump closure strategies at Cadia Hill Gold Mine in New South Wales in 2002/3
- Member of the Peer Review Team for Stage 2 of the Stuart Oil Shale Project at Gladstone in Queensland in 2004
- Peer Reviewer of the rehabilitation of the San Manuel Copper Mine tailings facility in Arizona, USA in 2004
- Member of the 2005 Peer Review Team that reviewed future red mud disposal, containment and rehabilitation at QAL at Gladstone in Queensland in 2005
- Geotechnical Reviewer of the breach of the co-disposal dam at Burton Coal in Queensland in 2005
- Peer Reviewer of the conceptual closure plan for Worsley Alumina red mud storage in Western Australia in 2005
- Peer Reviewer for waste rock dump covers for Century Mine in North Queensland from 2007
- During 2006, David was an Expert Advisor to the EIS team for the Olympic Dam Expansion Project in South Australia, providing expert input on disposal, hydrology and closure issues for both waste rock and tailings

Expert Witness

- Expert witness through Corrs Chambers Westgarth Lawyers, in relation to coal washery rejects used as filling for residential sub-division purposes
- Expert witness through McCullough Robertson Lawyers, in relation to the failure of a concrete arch reclaim tunnel beneath a coal stockpile
- Expert witness in relation to professional misconduct cases brought by the Queensland Professional Engineers Registration Board
- Numerous expert witness commissions related to residential and commercial building footing failures and slope instability

Consultancies

Professor David John Williams is widely sought for his expert input, in particular to mine waste disposal and mine site rehabilitation and remediation at operating mines throughout Australia and overseas. In Australia, he has consulted on numerous coal mines throughout Queensland and New South Wales; on Red Dome Gold Mine closure, Kidston closure, Osborne waste disposal, Ivanhoe Cloncurry mine closure, Phosphate Hill gypsum disposal, QERL processed waste storage facility closure, and Century Zinc Mine waste rock dumping in Queensland; Cadia Hill Gold Mine waste rock dumping and dump closure in New South Wales; Mt Morgans Gold Mine co-disposal, WMC Resources" nickel operations tailings closure and Minara heap leaching in Western Australia; waste disposal issues at the Ballarat East and Heathcote gold mines in Victoria; and a review of ARD treatments at Savage River Mine in Tasmania. Overseas he has consulted on tailings depositional design and water balance for the Kori Kollo Mine in Bolivia, a review of co-disposal of tailings and waste rock at Porgera Gold Mine and the closure of Misima Gold Mine in PNG, waste disposal design for the Goro Nickel project in New Caledonia, and advice on co-disposal for the Martabe Project in Indonesia.

David has been involved in material characterisation testing and the design of numerous mine waste covers throughout Australia, and the design, installation and monitoring of lysimeters and mine waste covers at Kidston Gold Mines, WMC Resources" Mt Keith Nickel Operations, QERL"s Stuart Oil Shale Project, a large-scale trial waste rock dump at Cadia Hill Gold Mine, and a large-scale trial tailings cell at Jubilee Nickel Mine.

David has been invited to visit numerous mining regions and individual mines throughout Australia, and in Canada, the USA, Brazil, South Africa, UK, China, Chile, PNG, New Caledonia, Spain and Mozambique.

MAJOR RESEARCH ACHIEVEMENTS

From 1989, Professor Williams carried out research under NERDDC and ACARP Projects on the characterisation of the deposit formed on the pumped co-disposal of combined washery wastes, which has since been adopted at numerous coal mines in Australia and Indonesia.

From 1996, David developed the store/release cover system suited to seasonally dry climates, for application to covering acid generating rock dumps at Kidston Gold Mine in north Queensland, and has had a long-term involvement in researching and monitoring this cover system, as evidenced by his numerous papers on his research on this topic. The store/release cover system on the tops of the Kidston rock dumps has been shown to limit percolation to less than 1% of rainfall, and to support a sustainable vegetation cover comparable to that occurring along water courses in the area. He was also involved in the development of a rehabilitation strategy for the side slopes of the rock dumps at Kidston designed to maximise geotechnical and erosional stability while promoting vegetation, and analysed the wetting up by rainfall infiltration and subsequent drain-down of and seepage from the rock dumps. Store/release covers have now been adopted at numerous mine sites in dry climates worldwide.

From 1999 to 2001, David led ACARP Project C8039 to develop a risk assessment and cost-effectiveness analysis for the rehabilitation of Bowen Basin coal mine spoil. The results of the project were reported in a Literature Review and Commentary and Project Final Report, plus a spreadsheet-based risk assessment and cost-effectiveness analysis, available at: www.uq.edu.au/civil/. In 2006, David undertook a closure study for Xstrata"s new Rolleston Coal Project in the Bowen Basin Coalfields.

David has since 2000 been involved in the closure design for the waste rock dump at Cadia Hill Gold Mine in New South Wales, including studies on the use of mixtures of benign trafficked rock and tailings as an alternative cover material, to overcome the shortage of suitable natural materials. In 2002/3, he led an international peer review of the rock dumping operation and closure plan. In 2004, David was successful in an ARC Linkage grant application with Cadia totalling over $ 700,000 over 3 years, which has led to the construction of a 15 m high, world-class, demonstration, instrumented rock dump covering 7,000 m^2. The instrumentation includes a full weather station, 24 lysimeters at the base of the dump to monitor seepage, lysimeters on the top surface to monitor rainfall infiltration and three store/release trial covers constructed using natural and mine waste materials. To date it has shown that about 70% of the rainfall incident on the traffic-compacted top of the dump infiltrates, with the majority going into storage within the dump during the first year, and only small amounts percolating to the base of the dump. The behaviour of the cover trials has to date been dominated by the moisture state at which they were constructed. Monitoring of the instrumented rock dump is expected to continue for at least 10 years.

From 2000 to 2003, David was a principal researcher into the physical and geochemical nature of acid generating waste rock dumps in Southern Carolina, USA (Rio Tinto"s Ridgeway Mine) and Sudbury, Canada (Inco"s Whistle Dump), sampled as they were being excavated and moved to a pit.

From 2001 to 2005, David led an ARC Spirt research project with industry partner WMC Resources focussed on an assessment of the long-term seepage and runoff from mine tailings storage facilities, to facilitate lease surrender. This included the monitoring of trial covers on tailings over the duration of the project and large-scale laboratory column testing and numerical analyses. Natural salt pan and rocky slope analogues under the same climatic and similar geochemical conditions were also studied to point to sustainable approaches for rehabilitating the tailings storage facilities.

From 2010, David has led two ACARP Projects, C19022 and C20047, investigating the settlement and stability of high coal mine spoil, and the behaviour of problematic clay-rich coal mine tailings.

David has been sponsored by mining companies and consultants to visit numerous mining regions and mine sites worldwide, both to impart and extend his knowledge. Since 2000, he has developed a relationship with the International Network for Acid Prevention (INAP), and has contributed to INAP-sponsored research and development projects and workshops involving mine sites in the USA, Canada, Australia and PNG.

Research funding has totalled over $7 million, including funding from ARC, ARC-SPIRT, ARC Linkage, NERDDC, ACARP-AMIRA, ACARP, MIM CRA-ATD, Kidston Gold Mines, BHP Coal and WMC Resources, Cadia Holdings, Jubilee Mines NL. Professor Williams has over 250 refereed publications, with about two-thirds of them in the mine waste field.

SELECTED PUBLICATIONS

Book Chapters

1. **Williams, D.J.** (2005). Chapter 17: Placing covers on soft tailings. In: *Ground Improvement-Case Histories*, 491-512. Eds B. Indraratna and Chu Jian. Elsevier.
2. **Williams, D.J.** (2001). Chapter 30: Assessment of Embankment Parameters. In: *Slope Stability in Surface Mining*, 275-284. Eds W.A. Hustrulid, M.J. McCarter and D.J.A Van Zyl. Society for Mining, Metallurgy, and Exploration, Inc., Littleton, Colorado, USA.
3. **Williams, D.J.** (1996). Chapter 7: Minimisation and Management of Solid Wastes. In: *Environmental Management in the Australian Minerals and Energy Industry*, 157-188. Ed D.R. Mulligan. Sydney, UNSW Press in association with Australian Minerals and Energy Environment Foundation, 1996.

Selected Refereed Journal Articles

1. Serati, M., Alehossein, H. and **Williams, D.J.** (2015). Estimating the tensile strength of super hard brittle materials using truncated spheroidal specimens. *Journal of the Mechanics and Physics of Solids*, **78**, 123-140.
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