ENSHAM RESOURCES PTY LTD

STAGE 3 GEOTECHNICAL ASSESSMENT

FEBRUARY 2019

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Stage 3 Geotechnical Assessment

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1 EXECUTIVE SUMMARY

This report provides a geotechnical assessment of the final landform options for Ensham Mine at Emerald, QLD. The mine commenced operation in 1993, with open cut mining operations currently comprised of Pits A, B, C, D, E, and F, referred to as the 'Ensham Project', and the 'Yongala' or Y Pit.

Three preferred rehabilitation options have been identified for assessment as part of the rehabilitation strategy for the Ensham Mine. As the mine is partially located on the Nogoa River flood plain, impacts from the river are to be addressed in the rehabilitation. The options are as follows:

- Option 1: Landform Levee - This option aims to convert these maintained structures effectively into permanent landforms - augmenting the existing earthworks by backfilling between the levees and the pits they protect where practical. Spoil piles and highwalls would also need to be left as long-term stable structures.
- Option 2: Flood Mitigation and Beneficial Use – Engineered intake structures will be provided in the existing levee embankments to permit river off-takes to feed water from the river to remaining open cut voids within the floodplain. Spoil piles and highwalls would also need to be left as long-term stable structures.
- Option 3: Backfill to PMF – Pits located within the pre-mining floodplain would be backfilled to original (pre-mining) ground levels. Parts of those pits straddling the boundary of the floodplain, would also be backfilled, whilst parts lying outside of that boundary would remain unfilled. Remaining spoil piles and highwalls would also need to be left as long-term stable structures.

The Ensham EA (G4) requires all surface areas significantly disturbed by mining activities to be rehabilitated to a safe, stable and non-polluting landform, with self-sustaining vegetation cover in accordance with the approved Completion Criteria. Condition G15 requires that Residual Voids must not cause any serious environmental harm to land, surface waters or any recognised groundwater aquifer, other than the harm constituted by the existence of the residual void itself.

This report provides an assessment of the long-term stability of project earthwork features, and makes recommendations regarding maximum landform slopes that should be adopted for all the options discussed above including the influence of groundwater. For Option 3, voids within the PMF footprint will be backfilled to approximate pre-mining levels, and hence no stabilisation will be necessary for these pits to render then safe and stable.

Limit equilibrium slope stability analyses have been carried out to assess the overall slope factor of safety (FOS) of representative sections through the highwalls and spoil piles at Ensham Mine. The stability of remaining highwalls applies to all remediation options. The analysis was carried out using SLOPE/W to assess the circular slip-surface geometry for a range of factors of safety. The results indicate:

- Tertiary clay soils overlying Permian materials at any of the highwall areas would not be considered long-term stable. Regrading of these materials to 1V:3H or flatter is expected to improve the stability of the materials to an acceptable standard.
- Weathered Permian and fresh Permian materials cut at the batter angles typically encountered in A to F Pits would be considered overall to be long-term stable in their current configuration. Instability could develop around exposed faults or structures with unfavourable orientations, or where surface water intersects the cuts or where low durability materials are present. Regrading of these materials will help incorporate these aspects. It is recommended that the fresh Permian be regraded to 1V:1H and weathered Permian to 1V:2H. Alternatively buttressing by partial filling of the pits using lowwall spoil could be adopted for stabilising these materials.
- Due to access constraints as well as a lack of available space at the top of the highwalls, regrading of the fresh Permian materials exposed in the highwalls and endwalls to an overall slope of 1V:1H may not be feasible. Consideration should be given to the following:
  - surface water should be managed
  - placement of spoil material battered against the base of the highwall

  - restricting public access to base of pit highwalls
  - depth of water expected in the pits over the long-term may submerge these materials, therefore removing the risk of public access to the base of the pits.
- Weathered Permian materials cut at steeper angles as seen in Y-Pit would not be considered overall long-term stable. Regrading of these materials, where required, to at least 1V:2H is expected to improve the stability of the materials to an acceptable standard. Where regrading of the weathered Permian materials is not feasible, the following should be considered:
  - restricting public access to top of pit highwalls where steep slopes could be encountered
  - surface water should be managed as mentioned above.

The spoil piles at Ensham Mine consist mainly of dragline dumped spoil at the angle of repose on the pit low walls. The spoil material generally consists of:

- Tertiary Clays – consisting mainly of yellow/brown silt and clay.
- Weathered/Fresh Permian Rock - typically grey in colour and contains less clay than the Tertiary materials.

SLOPE/W modelling of the spoil piles using limit equilibrium analyses in their current configuration indicated that most the spoil piles would not be considered long-term stable. This is supported by evidence of slips observed during the January 2018 site inspection.

- Rehabilitated areas observed during the site inspection typically had slopes of 1V:10H (10%) for Tertiary materials and 1V:7H (15%) for Permian materials, in accordance with the Rehabilitation Management Plan for the site developed pursuant to condition G12 of permit EPML00732813. These areas appeared generally stable and supported vegetation growth. Stability analyses indicate that a slope of 1V:4H (25%) or flatter should be geotechnically stable. It is recommended that spoil areas do not exceed this slope for long-term stability. Flatter slopes in line with current rehabilitation would yield a very stable final landform.

Two levees have been constructed to protect several of the open cut pits from flooding from the Nogoa River and the Anabranch River. The northern levee protects C and D Pits and the southern levee protects B Pit.

- To accommodate planned extension of the open cut pits and following breaches in the levees following flooding in 2008, the levees were upgraded for a 1:1000 (0.1%) Annual Exceedance Probability (AEP). The levees were designed to be between 1.2m to 1.4m above the 0.1% AEP flood level.
- Design drawings or as-built drawings were available at the time of compiling this report, and have been utilised as the basis for assessment of these levees.
- The existing levees are part of final landform for both Options 1 & 2. We have assessed the long-term stability of the levees and they appear to be stable based on information provided. Due to the proximity of the levees to the final pit wall locations it would be prudent to confirm the local geotechnical conditions at a few identified critical locations, and undertaking a general levee condition site assessment, including the following activities:
  - detailed inspection and mapping of the levee banks
  - further site investigation to assess construction materials, construction methods and condition of the levees and seepage if needed
  - depending on the outcomes further investigation, stabilisation of the B-Pit levee for Option 2 may be required.

Regular inspection and ongoing maintenance of the levees would be required for Option 2 to rectify any issues which could lead to instability in the long-term.

An assessment of the potential impacts of each of the three preferred options on relevant environmental values (EVs) has been undertaken based on the outcomes of the studies presented in this report. The table below provides a summary of the assessment using the adopted ranking criteria ranging from -3 (significant negative impact) to +3 (significant benefit) when compared with the current mine affected landform.

Environment Aspect	Environmental Value	Criteria	Scoring Notes	Option 1 Landform levee[1]	Option 2 FMBU[1]	Option 3 Backfill to PMF[1]
Land	Landform stability	Slope failure	Risk of failure generated over the life of the option	3	3	3
		Erosion	Erosion of landform generated over the life of the option	3	2	2

The EV of geotechnical aspects of the project, was assessed by considering landform stability in terms of slope stability and erosion. In terms of slope stability, all the options result in a significant improvement over current mine affected landforms, with reduced batter slopes on highwall, endwall and spoil slopes and hence higher factors of safety against failure in terms of industry guidelines for long term conditions. All options have hence been ranked +3. In terms of erosion, all of the options result in improved conditions over the current mine affected landform due to flatter batter slopes and revegetation measures, with Option 1 offering significant benefits as opposed to the medium benefits of Options 2 and 3 for the reasons described in the assessment.

2 INTRODUCTION

2.1 PROJECT CONTEXT

Ensham Mine, an open cut and underground bord and pillar coal mine located approximately 35km east of Emerald, is operated by Ensham Resources Pty Ltd (Ensham), a wholly owned subsidiary of Idemitsu Australia Resources Pty Ltd (Idemitsu), on behalf of the Ensham Mine joint venture (JV) partners. The JV partners, and holders of the Environmental Authority, are Bligh Coal Limited, Idemitsu and Bowen Investment (Australia) Pty Ltd. EA EPML00732813 (the EA), dated 9 August 2018, is the relevant environmental authority under which Ensham operates the mine.

Condition G16 of the EA states that a Residual Void Project (RVP) must be completed and submitted to the administering authority for review and comment by 31 March 2019. The minimum content of the RVP is specified within Condition G16 of the EA as:

— Terms of Reference;

— Residual Void Study;

— Progress Reports; and

— Rehabilitation success criteria for voids.

In compliance with Condition G19 of the EA, “the Residual Void Project must be carried out in accordance with the approved Terms of Reference”. Terms of Reference (ToR) (Ensham Resources, 2017a) were approved by Queensland’s Department of Environment and Science (DES, formerly the Department of Environment and Heritage Protection, DEHP) on 21 July 2017.

Condition G20 of the EA identifies the minimum content of the RVP identified in Condition G16.

In accordance with the ToR, the project has been divided into five stages:

— Stage 1 - Project definition and options identification

— Stage 2 - Preferred options technical studies

— Stage 3 - Preferred options detail design

— Stage 4 - Most preferred option identification

— Stage 5 - Regulatory documentation.

Stage 1 - Project definition and options identification for the RVP have been completed. The Stage 1 Options Assessment report has been finalised and issued to the Department of Environment and Science (DES), the Department of Natural Resources Mines and Energy (DNRME) and the Community Reference Group (CRG). The report was independently peer reviewed and revised to address peer review comments. The final report has been delivered to DES, DNRME and the CRG.

The Options Analysis workshop held in Stage 1 of the RVP identified two options:

— Option 1: Landform Levee

— Option 2: Flood Mitigation and Beneficial Use.

DES required a third option, Backfill to Probable Maximum Flood level, be included in the study.

All three options have been advanced through Stage 2 and into Stage 3 of the RVP and are referred to as the ‘preferred options’. The preferred options are discussed in Section 3.1 to Section 3.3 of this report.

Stage 2 identified the Environmental Values (EVs) in the immediate and surrounding area of Ensham Coal Mine and determined which EVs are likely to be affected by each preferred option. Similar to Stage 1, the Stage 2 EV report and technical studies have been Independently Peer Reviewed and issued in final to DES, DNRME and the CRG.

Stage 3 builds on the technical studies completed in Stage 2 to develop feasibility level designs required to prevent or minimise the potential impacts to EVs for each preferred option. Detailed designs for each of the preferred options will provide information for a risk assessment of each option and includes as a minimum:

- the long-term stability of the final landform;
- safety of access to the site; and
- the short, medium and long-term risks associated with each preferred option.

The output of Stage 3, in addition to the associated technical reports, will be an Environmental Assessment report for each preferred option which identifies the design and management practices which will be implemented to reduce impacts on the identified EVs.

On completion, each preferred option report will be peer reviewed by an independent suitably qualified third party before submission to the administering authority for review and comment.

2.2 PURPOSE

The purpose of this document is to present the findings of the geotechnical assessment of landform elements study to evaluate landform stability associated with each of the 3 preferred options identified in Stage 1 of the RVP. The report will identify benefits/impacts where applicable for each option and will provide information for the overall Environmental Impact Assessment, the Economic Impact Assessment and the Social Impact Assessment reports.

2.3 SCOPE

This report provides a geotechnical assessment of the final landform options for open cut portion of the Ensham Mine at Emerald in Central Queensland. The impact of the underground operation has not been assessed in this report.

Three rehabilitation options have been identified for assessment as part of the rehabilitation strategy for the Ensham Mine. The options are summarised as follows:

- Option 1: Landform Levee

  The existing northern and southern levees are currently engineered structures and require routine inspection and maintenance. This option aims to convert these maintained structures effectively into permanent landforms with associated geotechnical factors of safety.

- Option 2: Flood Mitigation and Beneficial Use

  The levees would include engineered intake and outlet structures to allow interaction between some of the residual voids (A/B Pits and C/D Pits) to permit river off-takes to specified annual recurrence interval (AEP) flood events. A minimum water level would be maintained to offset groundwater head and reduce saline groundwater inflow.

- Option 3: Backfill to PMF

  Those pits occupying the pre-mining floodplain of the Nogoa River would be backfilled to original (pre-mining) ground levels. Parts of those pits straddling the boundary of the floodplain, as defined by the probable maximum flood (PMF) event extent, would also be backfilled, whilst parts lying outside of that boundary will be rehabilitated in accordance with the EA and approved Rehabilitation Management Plan.

This report provides an assessment the long-term stability of landforms, including the influence of groundwater, adopted for the options discussed above. This has comprised:

— stability of highwalls, endwalls and spoil dumps
— levee design characteristics
— pit backfilling characteristics
— geotechnical design advice for engineered structures.

2.4 FORMAT OF REPORT

An overview of the layout of this report is presented below:

— **Section 1** Executive Summary.
— **Section 2** Introduction.
— **Section 3** Preferred Rehabilitation Options Description.
— **Section 4** Methodology/Discussion on Findings.
— **Section 5** Conclusions & Recommendations for Next Stage.
— **Section 6** Limitations.
— **Section 7** References and Abbreviations Used in the Main Body of the Report.

3 PREFERRED REHABILITATION OPTIONS DESCRIPTION

3.1 PREFERRED OPTION 1: LANDFORM LEVEE

Having conceptually evolved since Stage 1, proposed Option 1 will develop permanent landforms along the existing levee alignment to provide flood immunity for the 0.1% (1 in 1,000) Annual Exceedance Probability (AEP) flood event having had consideration of the risk of a PMF level event (as proposed in the Stage 2 assessment). Figure 3-1 illustrates the current placement of the landform.

When compared to the landform levee designed at a PMF level (as considered in Stage 2) the proposed 0.1% AEP landform along the existing levee alignment:

— eliminates afflux impacts for upstream landholders in a greater than 0.1% AEP event

— eliminates any potential increased impacts on downstream landholders associated with widening the river floodplain

— eliminates the need to realign the Nogoa anabranch.

It is proposed to incorporate the existing levees into the landform design with overburden emplacement areas behind the levee being reshaped in a manner that achieves the minimum stable landform slope requirements.

Figure 3-1: Option 1 - Landform levee

In addition to any impacts associated with the existing farm levees and mining pit levees, flood levels in the vicinity of Ensham Mine are significantly affected by the confluence of flood flows from the Comet River and Nogoa River, which occurs immediately downstream of the mine. Pits would be subject to rehabilitation in accordance with the approved Ensham site Rehabilitation Management Plan and the landform design criteria.

A treed corridor will be developed along the western (highwall) side of the rehabilitated A and B pits to provide connectivity between Corkscrew Creek and the Nogoa River flood plain as seen in Figure 3-1.

3.2 PREFERRED OPTION 2: FLOOD MITIGATION AND BENEFICIAL USE

Option 2 proposes to utilise the post-mining voids to form water storages to capture a proportion of high flow flood water and store this water for potential beneficial use as shown in Figure 3-2. Flood water harvesting is able to quickly fill the post-mining voids with minimal downstream impact, achieving improved water quality to support a range of reuse options, and/or environmental, and social values.

This option is founded on the concept of capturing a small fraction of larger magnitude flood event flows in the Nogoa River, storing this water in residual voids and releasing it back to irrigation and industrial users via a series of pipes to the Weemah Channel and Yamala Inland Port. No discharge to the Nogoa River by this option is currently envisaged, however functionality to do so is incorporated in water management infrastructure, should this be considered at some stage in the future.

The design of rehabilitation should comply with the current site Rehabilitation Management Plan and landform design criteria to optimise water capacity. Overburden emplacement areas located adjacent to the water storage voids are to be reshaped in a manner that achieves stable landform slopes without resulting in significant void backfilling. Low wall areas are to be reshaped in-pit to achieve minimum stable slope requirements to ensure safe access and stability of exposed slope surfaces.

Option 2 would utilise storage afforded by residual voids remaining in A Pit and B Pit south of the Nogoa River, and C Pit and D Pit north of the river. The quantity of water likely to be required to operate the system – or put another way, the headroom storage in the pits – is likely to be negligible when compared to overall discharges during flood events from the Nogoa River catchment into the Mackenzie River located downstream of the Ensham Coal Mine. However, in the context of irrigation usage, the headroom storage represents a significant volume and a potential economic asset.

Future assessment and optimisation of Option 2 will consider the potential for interactive operation of the voids with Fairbairn Dam to improve water use efficiency across the water supply system.

Currently Fairbairn Dam's southern irrigation channel, known as the Weemah Channel, extends eastward to within approximately 10 km of Ensham Coal Mine. Water captured from the upper Nogoa River catchment and retained in Queensland's second largest but relatively shallow Fairbairn Dam, is subject to significant evaporative losses. Furthermore, allocated water releases from the dam into the Weemah Channel (and the corresponding northern Channel, the Selma Channel) experience significant seepage and seasonal evaporative losses before reaching their intended customers, particularly where these customers are close to the end of the Weemah Channel. This option includes linking the residual voids located to the south of the Nogoa River to the existing Weemah Channel with large diameter pipes and pumps to transfer water to and from the voids.

Water captured in Fairbairn Dam would be released into the Weemah channel when hydrologic conditions are likely to result in minimal evaporative and seepage losses (i.e. at times when the catchment is receiving rainfall, the ground is saturated and evaporation is minimal). Whilst the water may not be required by customers at these times, the water would be transferred to the residual voids via the proposed Weemah channel(s) (refer red line on Figure 3-3) and stored in the more evaporatively-efficient residual voids at Ensham. In times of irrigation water demand at the lower reaches of the Weemah Channel (i.e. where the evaporative and seepage distribution losses are likely to be greatest), water would be returned to the Weemah Channel from the residual voids via the Weemah channel.

Because the Weemah Channel and proposed channel(s) lie on the southern side of the Nogoa River floodplain, it would be necessary to maintain a hydraulic connection between the residual voids on the northern flanks of the floodplain and those on the southern flanks. It is proposed that an upgrade of the existing water distribution main, that runs parallel with the main haulage route between B Pit and C Pit, be undertaken early in the project to provide the required hydraulic connection (refer blue line on Figure 3-3).

Option 2 proposes that pontoon-based pumping stations would be sited at each pit to transfer water as required. The Weemah channel coming into the mining lease would be configured to deliver water initially to A pit. Similarly, pumping from the mine to the Weemah Channel would be done from A Pit.

An offtake from the pipe to Weemah channel would be used to meet water demand for the prospective Yamala Inland Port located to the south west of Ensham Mine.

The intakes from the Nogoa River to B and C Pits would allow temporary storage of peak flood flows during flood events. As the river rises during a flood event, it would reach the overflow level of the inlet structures constructed in the levee (the intakes) and flow into the residual voids. The water would rise in the voids to reflect the height of the flood. As flood levels recede, water would ebb back into the river floodplain through the intakes to the base level of the intakes leaving the voids at full level. The intake level for B and C Pits has been considered as part of Stage 3.

A further key aspect of Option 2 is the depth of the residual voids. Shallow expansive voids experience greater evaporative water losses and hence potential salt concentration. Hence improved water quality outcomes are likely to be delivered with deeper inundated pits.

There remain several opportunities to manage power demands of the scheme including solar-power to generate an income to cover some or all of the overall annual operating cost of this option.

Residual voids that are not within the floodplain, for example E, F and Y pits, would be rehabilitated to achieve minimum stable slope requirements and comply with currently approved site Rehabilitation Management Plan and landform design criteria.

A treed corridor will be developed along the western (highwall) side of the rehabilitated A and B pits to provide connectivity between Corkscrew Creek and the Nogoa River flood plain as seen in Figure 3-2.

Figure 3-2: Option 2 – Flood Mitigation and Beneficial Use

Figure 3-3: Conceptual drawing of the pipeline from Ensham to the Weemah irrigation channel and Yamala Inland Port (red), and the mine internal pump system (blue)

3.3 PREFERRED OPTION 3: BACKFILL TO PMF

Preferred Option 3 comprises backfilling residual mining voids located within the pre-mining floodplain up to the elevation of the original floodplain within the lateral extent of the pre-mining Probable Maximum Flood (PMF) level.

Conceptually, the residual voids lying within this PMF extent would be backfilled up to the approximate original (pre-mining) topography with an additional surcharging to accommodate settlement of the backfill. In practice, it may be necessary to extend the backfilling beyond the modelled extent of the PMF to ensure stability of the backfilled areas within the PMF extent and protect against collapse into the adjacent residual voids. Excess mining spoil that is currently present in the floodplain and that is not required for backfilling of residual mining voids, would be retained as seen in Figure 3-4.

The existing levees constructed to protect the voids from flooding would be removed, with the material re-used for backfilling voids. Material required to backfill residual voids would be drawn from the nearest cost-effective source eg. low wall spoil. Any negative material balance will need to be met from adjacent low wall and high wall spoils.

Virgin rock typically exhibits an increase in volume when excavated - this is referred to as 'bulking'. The degree of bulking will vary with the geo-mechanical properties and size distribution of the excavated rocks and the methods used in excavation and transport. Furthermore, it is likely to vary both along the linear extent of the open cut mine and within different parts of spoil tips created through the extraction of rock dominated by lithologies characterising the local stratigraphy. Re-excavation of spoil and re-emplacement within voids within the modelled PMF will again exhibit bulking. Whether subjected to dynamic compaction or allowed to settle with subsequent loading by overlying backfill, the spoil within the voids will inevitably exhibit uncontrolled settlement. This will lead to the development of low areas within the PMF extent which, though shallow, lie below the original level of the floodplain. These low areas will not necessarily be connected and are likely to collect surface water runoff but be subject to intense evaporation and surface accumulation of evaporative salts which would be flushed clean by fluvial flood events.

Static surcharging of the replaced spoil material may reduce the risk of long-term settlement below original floodplain. However, this will require material to be placed above the original floodplain elevation in direct contradiction of the intent of this option.

Beyond the modelled extent of the PMF, residual voids would be rehabilitated in accordance with a combination of Option 1 and option 2 landform criteria requirements.

Replaced spoil, however comprehensively compacted, is unlikely to provide durability equal to the original virgin rock and hence during times of fluvial flood, of magnitudes such that the current floodplain pinch point between B Pit and C Pit begins to develop afflux, it is likely that the Nogoa River would scour spoil within the adjacent backfilled pits. This has the potential over time to result in sink holes and ultimately a repeat of the 2008/2010 inundation events with the Nogoa River cutting a channel into one or more backfilled pits and flooding the remaining un-backfilled parts of each pit. Additionally, impacts on turbidity downstream of the backfilled areas would need to be considered.

As part of the rehabilitation process, the establishment of a treed corridor along the western (highwall) side of the rehabilitated A and B pits is proposed to link Corkscrew Creek and the Nogoa River flood plain as seen in Figure 3-4.

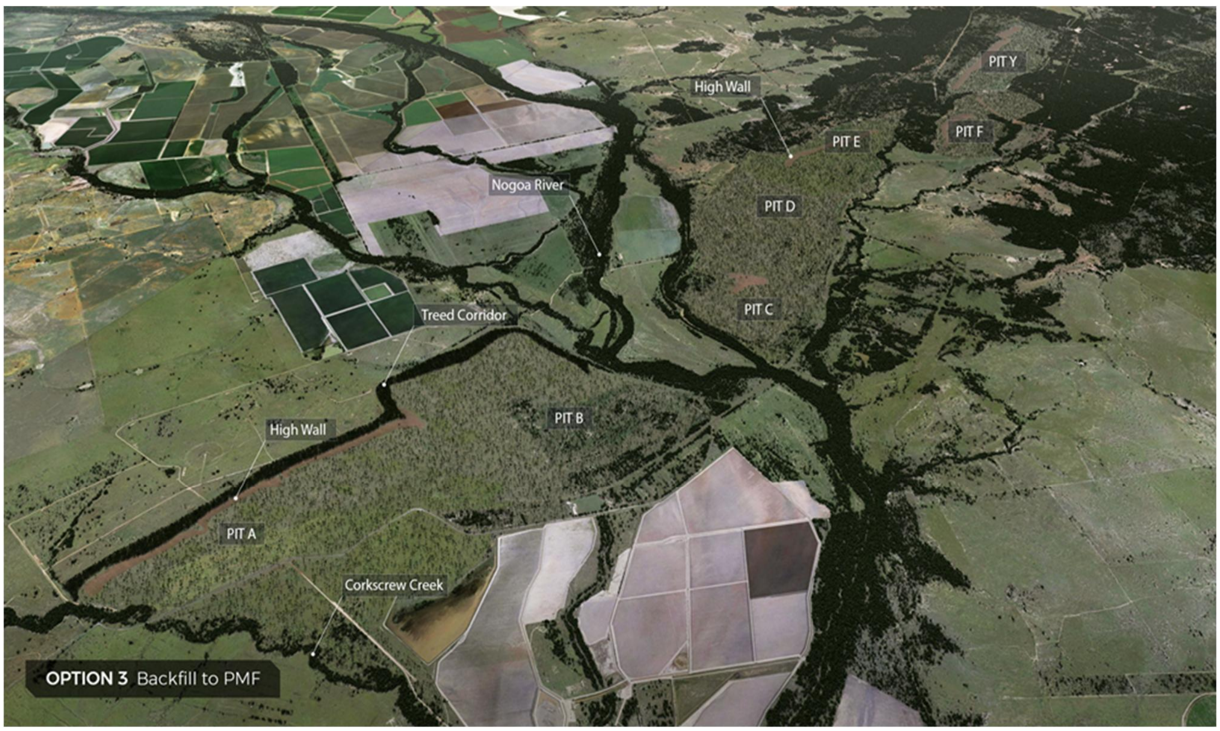


Figure 3-4: Backfill to PMF

3.4 EXISTING PIT EXTENTS

The extents of pits are illustrated on the plan provided in Figure 3-5.

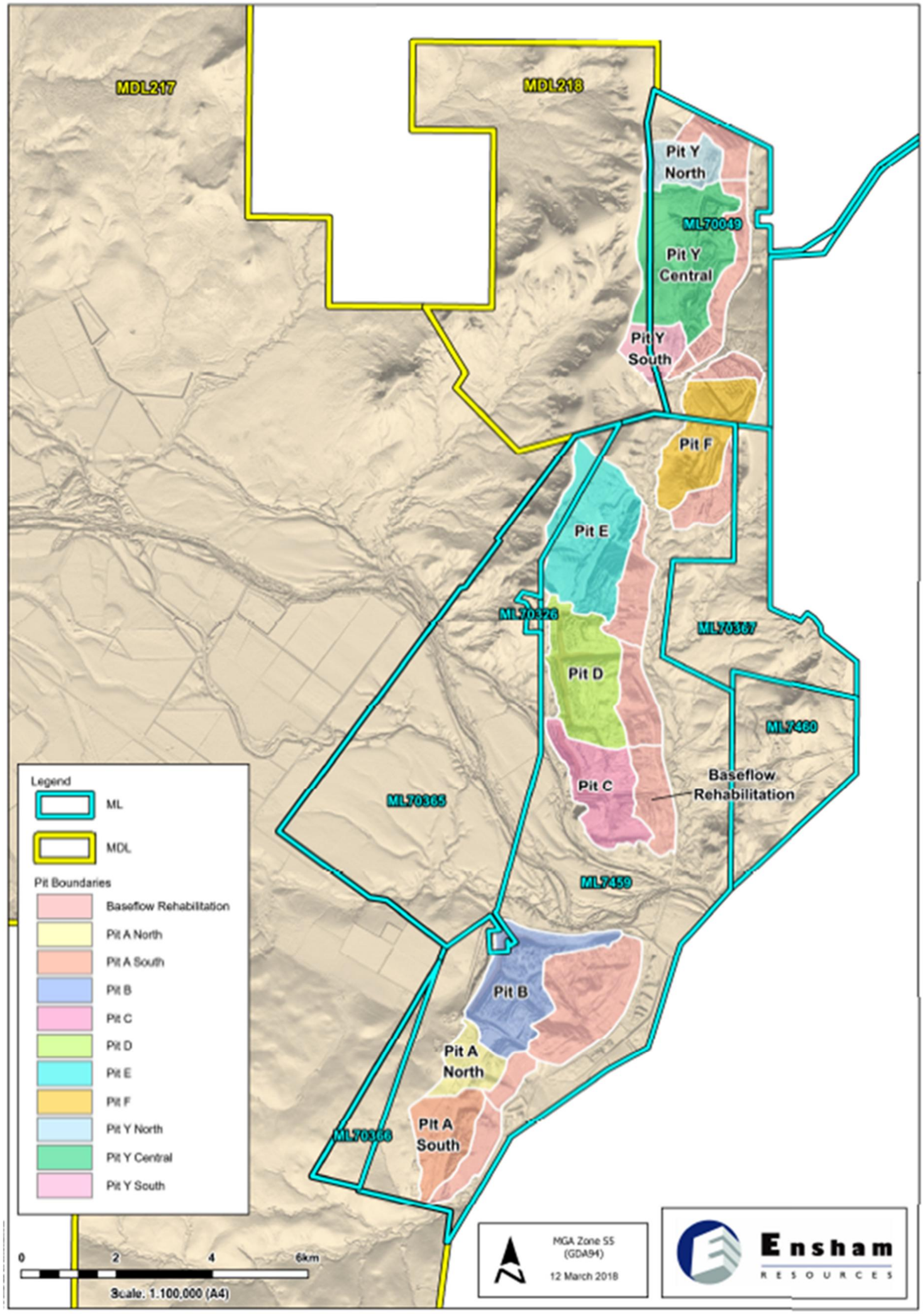


Figure 3-5: Location of open-cut pits

4 METHODOLOGY / DISCUSSION OF RESULTS

4.1 METHODOLOGY

The methodology adopted for the Stage 3 Geotechnical Study is described below. This included identification and description of the geological features to be considered in the study, a desktop review of geotechnical information made available by Ensham, definition of the regional geology, undertaking a site to observe geological conditions on site, and selection of stability modelling parameters. SLOPE/W and SEEP/W software has been utilised for stability analysis.

4.1.1 GEOLOGICAL FEATURES

The open cut pits at Ensham are partially located on the Nogoa River flood plain, with A and B-pits located on the south side of the river and C, D, E, F and Y-pits are on the northern side. A figure indicating the extent of the Pit Shells is included in Figure 3-5 above. Mining operations included a combination of dragline and truck and shovel excavation of the overburden. Typically, dragline highwalls have been pre-split blasted at 70^{o}, with overburden and weathered Permian pre-stripped by truck and shovel to a flatter slope. A summary of the pit details as observed during the site visit undertaken during the study are as follows:

— A-pit extends north of Corkscrew Creek and consists of 3 mine voids which contained water. The voids were between approximately 900 m and 1400 m long and down to approximately 70m deep. Highwall angles were typically between 65^{o} and 70^{o}.

— B-pit is immediately north of A-pit and south of the Nogoa River and consists of one mine void which contained water. The void was approximately 1800 m long and 60 m deep. Highwall angles were typically between 75^{o} and 70^{o}.

— C-pit is north of the Nogoa River and consists of one mine void approximately 1400 m long and down to a depth of 50 m. Highwall angles ranged between 34^{o} and 70^{o}. C-pit did not contain water when observed during the site inspection.

— D-pit is immediately north of C-pit and consisted of two mine voids approximately 1000 m and 1400 m long. The pits contained some water when observed during the site inspection and were up to 60m deep. Highwall angles ranged between 43^{o} and 70^{o}.

— E-pit is immediately north of D-pit and consisted of two mine voids approximately 1600 m and 700 m long. The pits contained some water when observed during the site inspection and were up to 60m deep. Highwall angles were 70^{o}. E-pit has been partially backfilled.

— F-pit consists of two mine voids. The northern most void was active, while the southern void contained water when observed during the site inspection. Mining in the southern void ceased due to stability problems in the lowwall associated with steeper floor dips. The orientation of mining in the northern pit was altered to reduce the risk of lowwall instability. Highwall angles were 70^{o}.

— Y-pit (Yongala) is the northern most pit at the mine and consisted of four voids. The pit depths ranged between 14 m and 94 m deep and highwall angles ranged from 35^{o} to 70^{o}. The northern most void contained water when observed during the site inspection.

Levees to the north and south of the Nogoa River were constructed as part of the original mine development to protect the open voids from flooding from a 1 in 100-year AEP flood event. The northern levee encompasses C-pit and D-pit and is 4 m to 8 m in height. The southern levee encompasses B-pit and is 5 m to 6 m in height.

The levees have been repositioned and raised over time. Construction of the B-pit levee (southern levee) was approved in 1998 and the interim B-pit levee was constructed in 2006. The levee was overtopped during flooding in 2008, and was raised following the flooding to provide protection for 1 in 1000-year annual recurrence interval (AEP) event. The C-pit levee (northern levee) was constructed in 1995, and breached in the 2008 flood. The levee was also raised following the flooding to the 1 in a 1000-year AEP flood event, and extended northward in front of D-pit after 2008.

4.1.2 DESKTOP LITERATURE REVIEW

A desktop assessment was undertaken as part of the Stage 2 works. The assessment involved a review of all geotechnical reports supplied by Ensham. The results of the assessment are provided in WSP report PS107225-RES-REP-001 Rev2 Geotechnical Design. The reports which have been used as part of this assessment are summarised below:

Table 4-1: Review Data Summary - Levees

FLOOD LEVEE REPORTS	CONSULTANT	DATE
Ensham Mine Flood Levee Upgrade – 47793	Douglas Partners	2009
Proposed C Pit Levee and Upgrade of B Pit Levee Design Report BEE603.009-C-REP-004 Rev A	Kellogg Brown and Root Pty Ltd	2009
Summary Report on Geotechnical Assessment Reports for Levees	Douglas Partners	2013
As Built drawings and reports	Douglas Partners and KBR	Varies

Table 4-2: Review Data Summary – Pit Areas

MINE PIT REPORTS	CONSULTANT	DATE
Engineering Geology of Ensham Central Area	Seedsman Geotechnics	2003
Engineering Geology of Ensham Central Area - Subsidence	Seedsman Geotechnics	2003
Site Visit	PSM	2004
Dragline Mining - Yongala	PSM	2004
Ensham Central Project – EIS - Rehabilitation Instability in F Pit and Highwall Design	Hansen PSM	2006 2007
Highwall Risk Assessment	PSM	2007
Highwall Mining in the Nogoa Floodplain	PSM	2007
Highwall Risk Assessment - October	PSM	2007
Assessment of Stability of Final Void and Landform Shaping Options, Ensham Coal Mine	Landloch	2015
Geotechnical Review of the Final Landform	PSM	2015
Groundwater Modelling of Residual Voids	AGE	2015

Table 4-3: Review Data Summary – Spoil Piles

SPOIL PILE REPORTS	CONSULTANT	DATE
Geochemical Characterisation and Assessment of Overburden and Potential Coal Reject Material at the Ensham Central Project	URS	2007

4.1.3 REGIONAL GEOLOGY AND SOILS

Ensham Mine is located in the Comet Ridge portion of the Bowen Basin (Figure 4-1) and is underlain by the Rangal Coal Measures which are the stratigraphically highest of the coal measures in the Bowen Basin (Figure 4-2). They form the top of the Blackwater group and are Late Permian in age. The Rangal Coal Measures are made up of interbedded fine sandstone, siltstone and claystone which have been deposited in an alluvial environment. There are typically no massive units (Seedsman, 2003). The Rangal Coal Measure typically dip at 2° to 3° to the west, dips of up to 8° have been recorded at Ensham Mine (Seedsman, 2003).

The Rangal Coal Measures are overlain by Tertiary sediments which consist of silcrete, laterite, clay, sand and gravel. They occur in topographically higher areas. (Seedsman, 2003). Quaternary alluvium associated with the Nogoa River flood plain overlies the Tertiary sediments and Permian rocks. The alluvial soils are very thin or not present in topographically higher areas and increase in thickness toward the river to a maximum thickness of approximately 24 m. The upper units generally consist of silt and clay, grading to sand and gravel with depth (Seedsman, 2003).

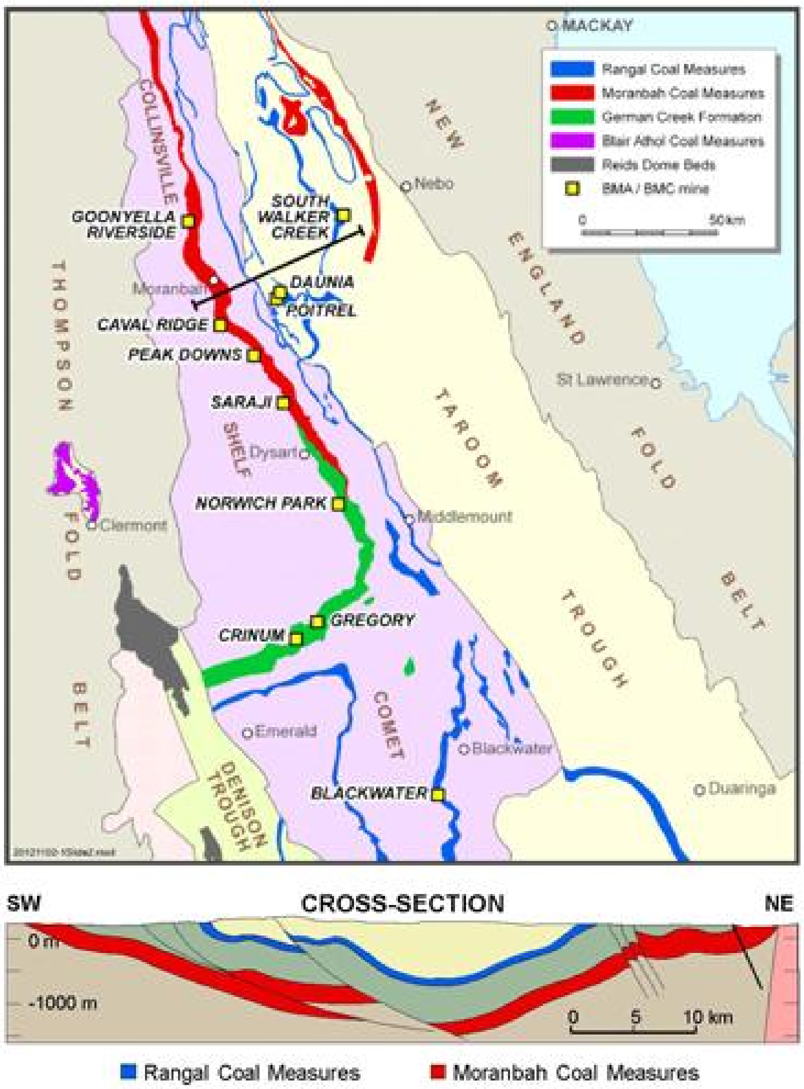


Figure 4-1: Bowen Basin Structure (BHP Billiton, 2013)

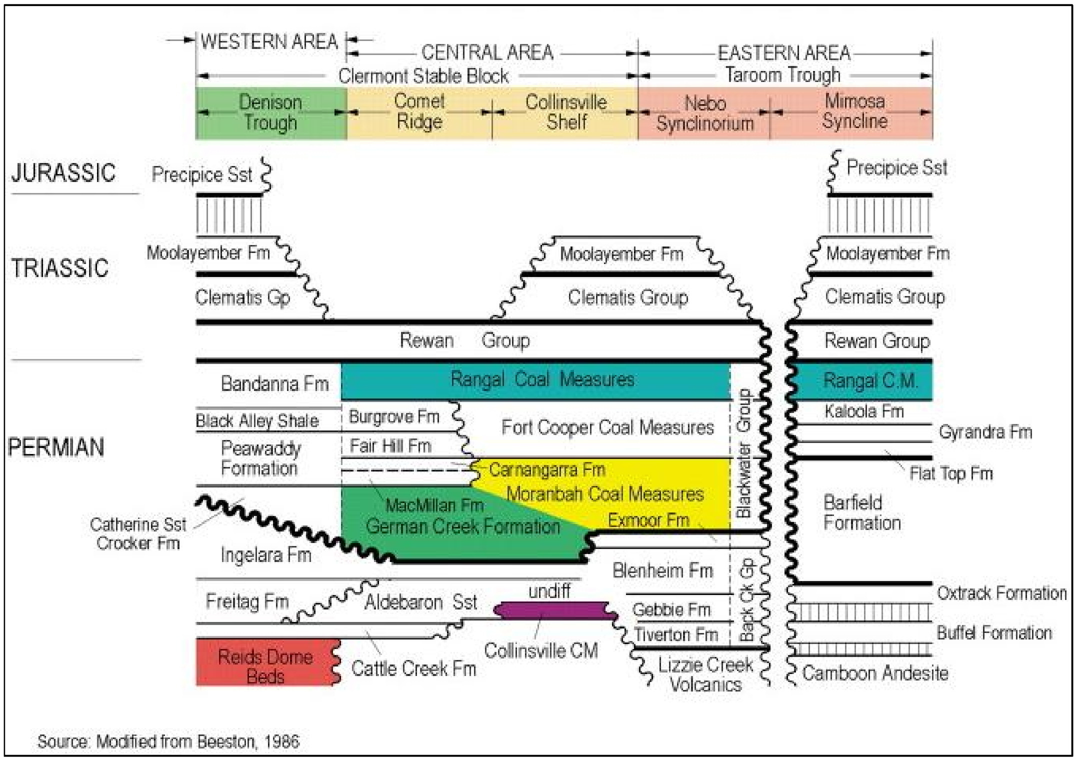


Figure 4-2: Bowen Basin Stratigraphy

4.1.4 SITE INSPECTION

A site inspection was undertaken by WSP in the company of Ensham Mine staff in January 2018. The inspection involved a drive by and walk over inspection of the open cut pits, soil piles and levees. Photos and observations made during the inspection are included below:

- Existing rehabilitated areas have performed well, and were well vegetated with grasses and had little indication of erosion. Spoil composed of Tertiary aged material (clayey soil) appeared to be performing well at a 1V:10H slope (10%) and Permian aged spoil (excavated overburden rock) can be graded up to 1V:7H (15%).
- Spoil placed on the pit low walls was generally at the angle of repose, which is typically around 37^{o}, (Williams *et al*, 1991) and showed signs of deep gully and rill erosion.
- In-situ Tertiary material and Tertiary spoil showed signs of severe erosion and appeared to be dispersive.
- In-situ weathered Permian material and Permian spoil showed signs of erosion and slaking, and appeared to be dispersive.
- In-situ fresh Permian material consisted of interbedded siltstone, sandstone and claystone showed little erosion.
- Tertiary material exposed in high walls consisted of red brown clayey soils that showed signs of instability in the form of shallow slumping.
- Weathered to fresh Permian material exposed in the highwalls appeared to be stable. Some instability was noted around structural features such as faults. Based on information provided in Pells, 2015 and survey provided by Ensham, the highwalls slopes in the Permian materials are generally between 60^{o} to 70^{o}. Differential weathering and low durability materials were also observed.
- Some minor block falls were noted in Permian rock exposed in the end walls, this was associated with the orientation of exposed bedding and joints.

— All the pits to the south of the Nogoa River contain water, while those to the north do not, apart from the inactive area of F-pit and some areas of Y-pit.

Photo 4-1: Low-wall spoil, mostly Permian materials showing signs of erosion

Photo 4-2: B-pit High-wall, fresh Permian, overlain by weathered Permian and weathered Tertiary material

Photo 4-3: B-pit End Wall, exposed Permian material, note breakdown of material and undercutting

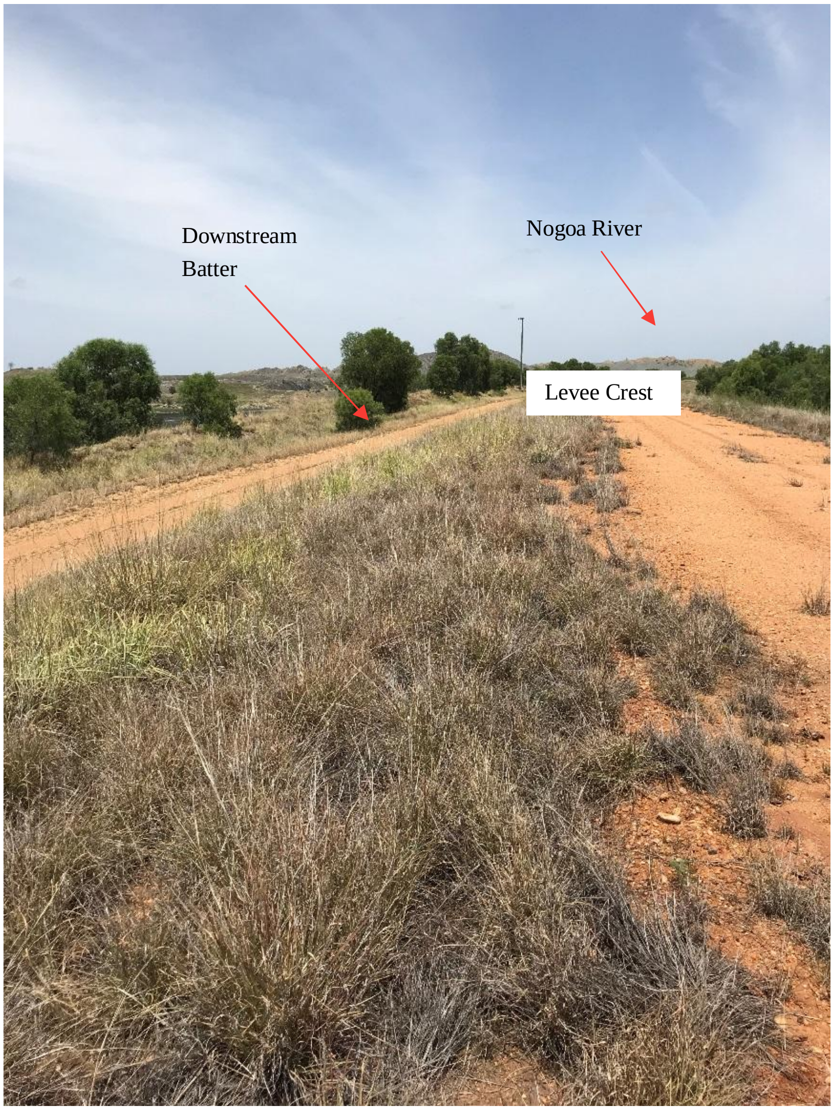


Photo 4-4: Southern Levee (looking west)

Photo 4-5: Yongala (Y-pit), weathered Permian overlain by weathered Tertiary material, with typical Tertiary failure

Photo 4-6: Existing rehabilitated slope east of A-pit (approximately 10% slope)

Photo 4-7: Spoil area undergoing rehabilitation

Photo 4-8: Exposed Tertiary soils observed in A-pit

4.1.5 SELECTION OF MODEL PARAMETERS

4.1.5.1 SOIL AND ROCK MATERIALS

Soil and rock material parameters used in the SLOPE/W model have been based on those used by Pells Sullivan Meynik (PSM) in previous stability assessments in 2015 (report reference: PSM687-222L Geotechnical Review of Final Landform), and Simmons and McManus (2004). Laboratory data used by PSM, 2015 was made available for review and these parameters have been assessed against typical values for soil and rock in published literature (Look, 2007 and

Simmons and Simpson, 2006), and WSP's previous experience with similar materials within the Hunter Valley and Bowen Basin. The parameters provided by PSM, 2015 were considered appropriate for the materials observed during the site inspection and described in the provided consultant's reports. The soil parameters for the analyses were based on a Mohr-Coulomb failure criteria and the rock parameters on a Hoek-Brown strength criterion. The adopted parameters are presented in Table 4-4.

Table 4-4: Adopted Material Parameters

UNIT	ɣ kN/m³	MOHR COLOUMB		HOEK BROWN			
		C' kPa	Ø' DEG	GSI	UCS MPa	MI	D
Alluvial Sand	18	0	35	-			
Tertiary Clay	18	10	23	-			
Spoil - Unsaturated	18	30	28	-			
Spoil – Saturated	20	15	23				
Weathered Permian	20	80	40	-			
Fresh Permian	24	-		65	20	10	0
Coal	15	-		36	12	25	1
Floor Rock	24	-		65	20	10	0

Long-term modelling has also been based on the above parameters. Final landforms must achieve a FOS of 1.5 to be considered stable in the long-term. (PSM, 2005; ACARP, 2006; Ensham Rehabilitation Management Plan, 2017). A sensitivity analysis has been undertaken for recommended maximum regrade slopes (highwall and lowwall spoil regrade), varying selected material parameters to assess the sensitivity of the FOS to potential variations that may occur due to deterioration of material over time. The parameters for the sensitivity analyses are shown in Table 4-5.

Table 4-5: Sensitivity analysis parameters

UNIT	ɣ kN/m³	C' kPa	Ø' DEG
Spoil – Unsaturated – Low[1]	17	15	25
Spoil – Unsaturated – high[1]	19	45	31
Spoil – Saturated – Low[1]	17	7.5	21.5
Spoil – Saturated – high[1]	21	22.5	25.5
Weathered Permian low[2]	19	75	36

1 – based on one standard deviation above and below as shown in Table 1 in Simmons and McManus (2004)

2 – based on Table 4-15 in KBR report 21 Feb 2014

It must be noted that no exploratory borehole or test results were included in data made available by Ensham, particularly for the floor rock of the pits, which could impact on stability analysis results. Ensham site geotechnical staff were contacted regarding knowledge of any soft floor conditions, and it was noted that weak pit floor layers have been identified in current mining operations in F Pit. These issues are addressed as encountered on site, but no record of other known or recorded floor weaknesses could be provided. This issue should be pursued in the next stage of the project to confirm the parameters adopted in the Table 4-4 above.

4.1.6 GROUNDWATER

The groundwater levels for the current state of the pit walls has been assumed to be at the base of the Tertiary clay or weathered Permian materials and fall to the highwall toe at a gradient of approximately 1V:5H (PSM, 2015 and PSM, 2005). The current pit water levels have been obtained from a survey provided by Ensham for the assessment. It should be noted that these levels fluctuate on site depending on water management activities at any time.

Groundwater levels for the long-term stability of the highwalls has been based on information provided by Ensham in July 2018, with equilibrium groundwater levels as shown in Table 4-6. These water levels were provisional when provided, and the final design would need to be reviewed once these levels have been finalised. Updated long-term residual pit water levels were provided in November 2018, which have been included in the table for comparison. The updated water levels are relatively similar for Pits A to E, with an increase of approximately 9 m in Pit F, and up to 28 m in Pit Y, which are significant changes. No geotechnical analysis has been undertaken based on these November levels, and it is recommended that calculations be reassessed in the detailed design phase of the project based on the most recent groundwater model results at that time. It was not considered necessary to undertake revised modelling as the rest water levels in Pits A to E are very similar to the levels used in the stability models, and for Pits F & Y the elevated water levels, although significantly higher than used in stability models, are still within the fresh Permian profile, while critical failure planes were identified in the upper tertiary materials. For lowwall spoil slopes it was found that elevated water levels were slightly beneficial to spoil stability due to the additional weight this adds to the slope toe, hence no additional modelling has been undertaken.

Table 4-6: Equilibrium conditions – pit water levels

SECTION	WATER LEVEL AHD m (07/2018)	WATER LEVEL AHD m (11/2018)
Section A-E (Pits A-E)	133	132 (average all options)
Section F (Pit F)	141	148 (average all options)
Section Y (Pit Y)	143	168 (average all options)

4.2 HIGHWALL STABILITY

4.2.1 INTRODUCTION

Slope stability analyses have been carried out to assess the overall slope factor of safety (FOS) of representative sections through the exposed highwalls at Ensham. The analysis was carried out using SLOPE/W considering ACARP Rehabilitation of Highwalls, November 2006, Pells *et al*, 2015; Pells *et al*, 2007 and Pells *et al*, 2005. SLOPE/W has been used to assess the circular slip-surface geometry for a range of FOS. Planar, wedge and toppling failures have not been assessed as part of this work. To make sensible predictions for these types of failures it is best to have detailed information regarding structure orientation and location, defect shear strengths and water pressure conditions within the rock walls in their current state (Simmons and Simpson, 2006), which were not available at the time of this assessment. It is noted that slopes flatter than the existing slopes are proposed for final design and the flatter slopes should help address this issue, which should be reviewed in final design.

Low durability rock and wave action in the flooded voids can also impact stability. Further information would be required to adequately assess these potential impacts. The analyses have also assumed that the rock in the underlying pit floor is competent.

Three typical highwall sections have been assessed for the pit areas for short-term and long-term stability. The typical sections have been developed by reviewing the highwall geometries and the soil and rock types typical of each pit. Pits A, B, C, D and E (Section A-E) have similar highwall geometries and similar types of materials are exposed and have been represented on one typical section (Section A-E). The highwalls from F-pit (Section F) and Y-pit (Section Y) have been assessed separately, as the highwall geometries for these pits vary from the other pits, as does the types and strengths of materials exposed. A summary of the details for each section is provided in Table 4-7.

Table 4-7: Typical Section Summary

SECTION	MATERIAL	SLOPE HEIGHT (m)	APPROX ANGLE (O)
A-E	Fresh Permian	56	65
	Weathered Permian	24	30
F	Fresh Permian	26	35
	Weathered Permian	15	35
	Tertiary Clay	12	40
Y	Weathered Permian	40	50
	Tertiary Clay	27	21

Generally, a FOS of 1.3 is adopted for mine slopes which are not required to remain stable for long periods of time (Hoek and Bray, 1981). For slopes that are required to be long-term stable a FOS of 1.5 is generally adopted (Hoek and Bray, 1981; Simmons, 1995; PSM, 2005). In the Ensham Rehabilitation Management Plan Section 9 – Values, Indicators and Completion Criteria (Table 9-1), a completion criteria FOS for land stability against failure of greater than or equal to 1.5 is stipulated. A FOS of 1.5 has hence been adopted as the minimum required for long-term stability for final landforms.

4.2.2 CURRENT CONFIGURATION

The SLOPE/W results for the FOS for the representative highwall sections are shown below. It is noted that the actual FOS could be lower due to the impact of discontinuities/faulting, reduction in rock strength due to increased moisture content, and low durability in the rocks which could not be modelled during this assessment.

4.2.2.1 SECTION A-E (A TO E-PITS)

Two sections have been modelled to assess the current stability of the pits which do not contain water and the pits which do hold water. The model results indicate that the weathered and fresh Permian materials exposed in the pit walls cut at grades typically encountered in A to E-pits have an overall FOS of >1.5 and are expected to be long-term stable in their current condition based on limit equilibrium analyses, with the exceptions noted above.

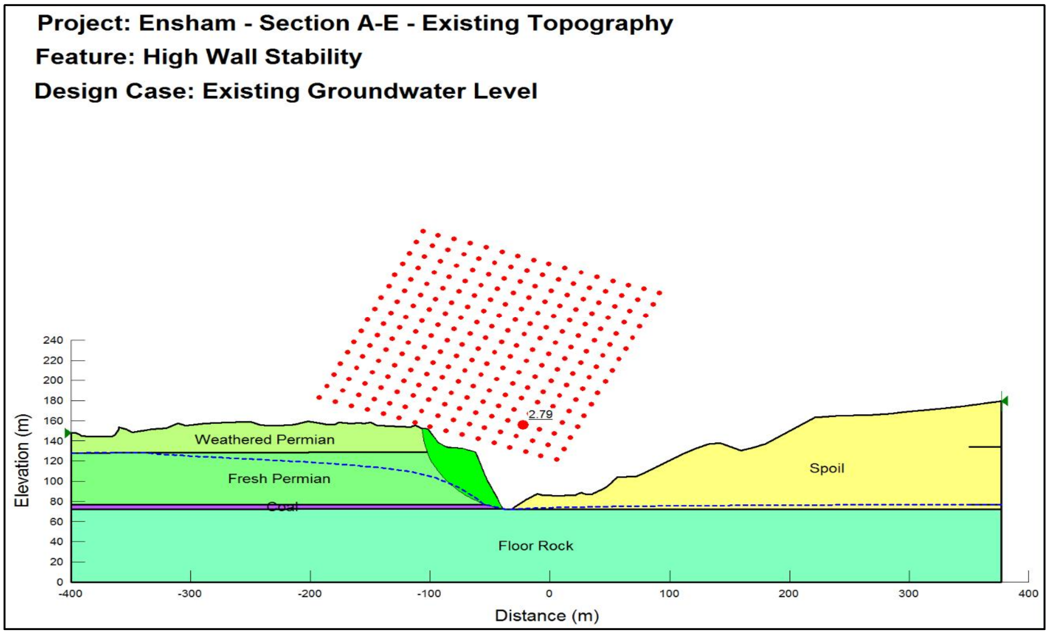


Figure 4-3: SLOPE/W Results for Typical Section A-E, Dry Pit – Current Configuration FOS = 2.79

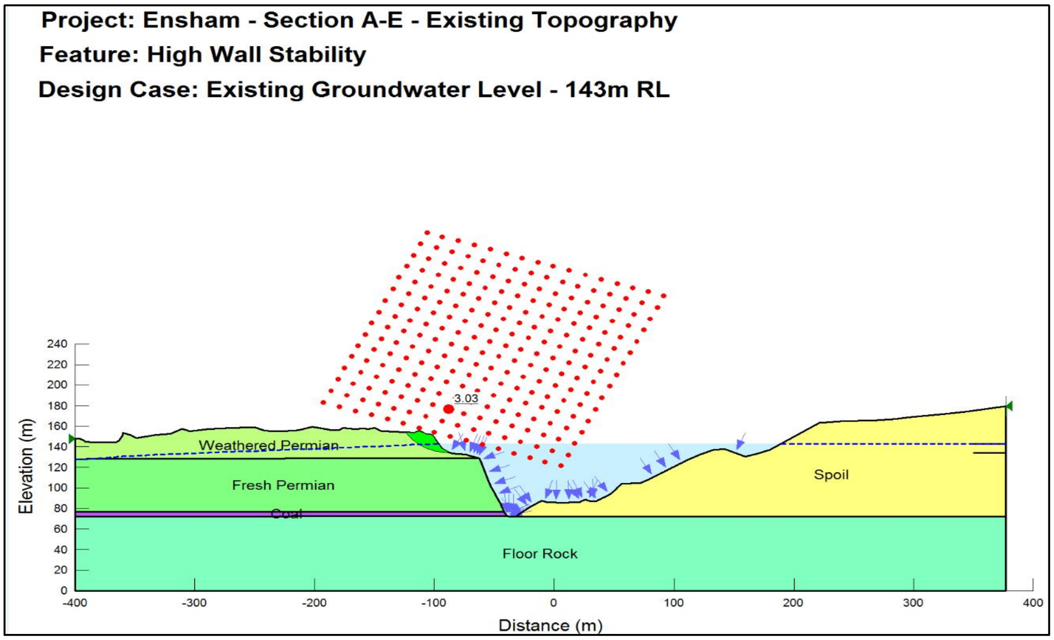


Figure 4-4: SLOPE/W Results for Typical Section A-E, Pit Contains Water – Current Configuration FOS = 3.03

4.2.2.2 SECTION F (F-PITS)

The model results indicate that failures are not expected through the weathered and fresh Permian materials exposed in the pit walls with grades typically encountered in F-pit. The Tertiary clay soils in their current configuration have a FOS of 1 to <1 and are would not be considered stable. The model shows that regrading of these materials would improve their short-term (FOS – 1.3) and long-term stability (FOS – 1.5).

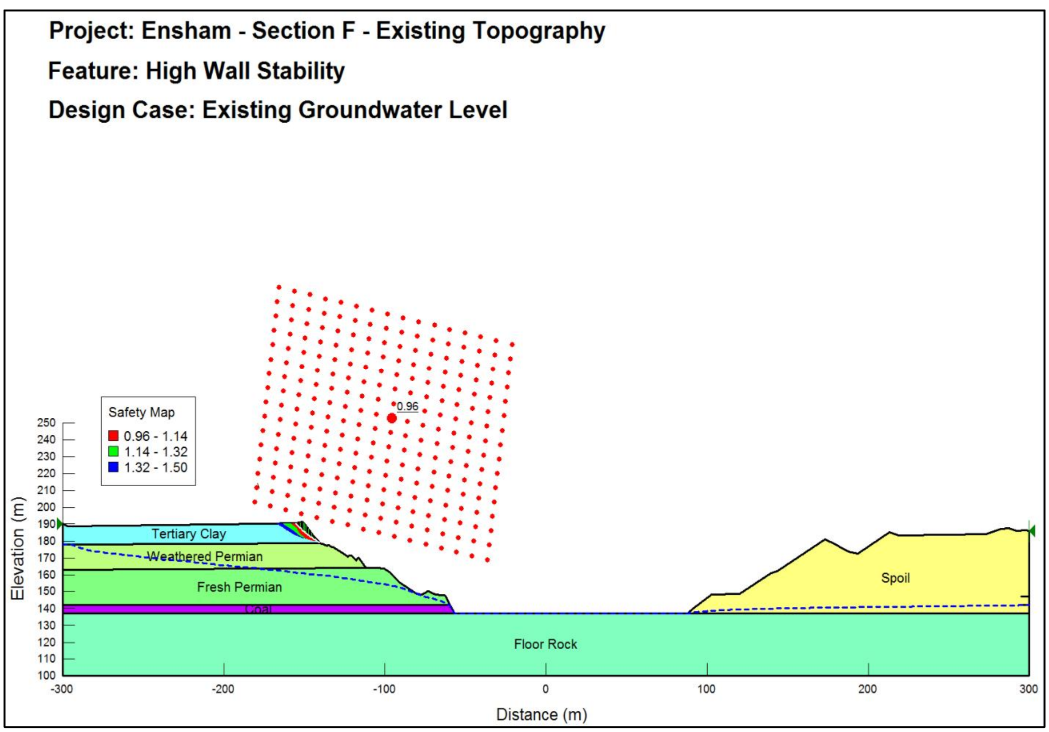


Figure 4-5: SLOPE/W Results for Typical Section F – Current Configuration FOS=0.96

4.2.2.3 SECTION Y (Y-PITS)

The model results indicate that the weathered Permian materials exposed in the pit walls with grades typically encountered in Y-pit are marginally short-term stable in their current condition with FOS of 1.28. The Tertiary clay soils in their current configuration have a FOS of 1 to <1. The model shows that regrading of Permian and Tertiary materials would improve their short-term and long-term stability.

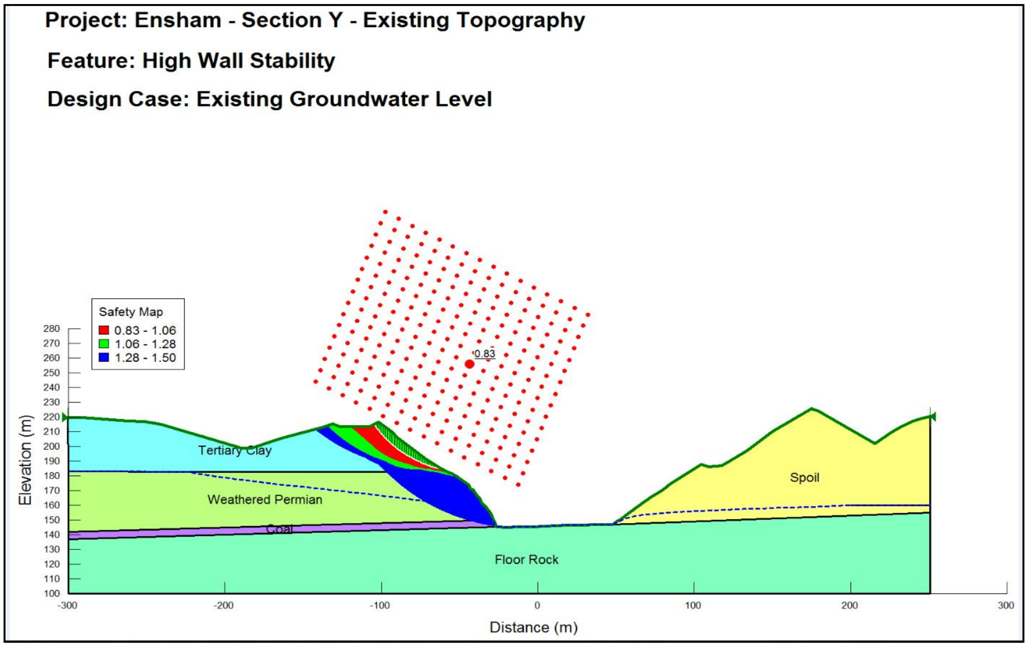


Figure 4-6: SLOPE/W Results Typical Section Y – Current Configuration FOS=0.83

4.2.2.4 SUMMARY

Based on previous highwall risk assessments undertaken by PSM, 2007 in A, B, C and D-pits and observations made during the site inspection, the highwalls in their current state appear to be reasonably stable as no large-scale failures have been observed, which is supported by the SLOPE/W model FOS results. Instability in the form of small areas of block failures and deterioration of the highwalls and endwalls have typically been observed around faulting and unfavourable structure orientations during inspections by PSM, 2007. Photo 4.9 below shows an area of the highwall in B-pit which shows deterioration and undercutting of the highwall around a fault.

Highwall conditions in the Yongala Pit (Y-pit) are substantially different as noted by PSM, 2007 and during the site inspection, as the pit walls consisted of softer Permian materials and/or deeply weathered Tertiary materials. Instability in these materials has been observed during the site inspection in January 2018, and is reflected in the SLOPE/W modelling results. Flatter highwall angles have been utilised in other areas of Y-pit during mining to try and mitigate the risk of instability.

Photo 4-9: Fault in B-pit Highwall

4.2.3 LONG-TERM STATE

4.2.3.1 SECTION A-E (PITS A, B, C, D AND E)

The SLOPE/W results for the long-term overall FOS for the representative rock wall sections are shown in Figure 4-7. It is noted that the actual FOS could be lower due to the impact discontinuities/faulting in the rocks, low durability rock, possible reduction in rock strength due to increased moisture content, as well as the impact of wave action from water in the pits which could not be modelled during this assessment. The models have included the long-term groundwater conditions. A long-term FOS of 1.5 has been adopted for overall stability analyses.

The model results indicate that the weathered and fresh Permian materials exposed in the rock walls cut at grades typically encountered in A to E-pits have FOS of >1.5 and are expected to be long-term stable under the groundwater conditions modelled except indicated in the previous section.

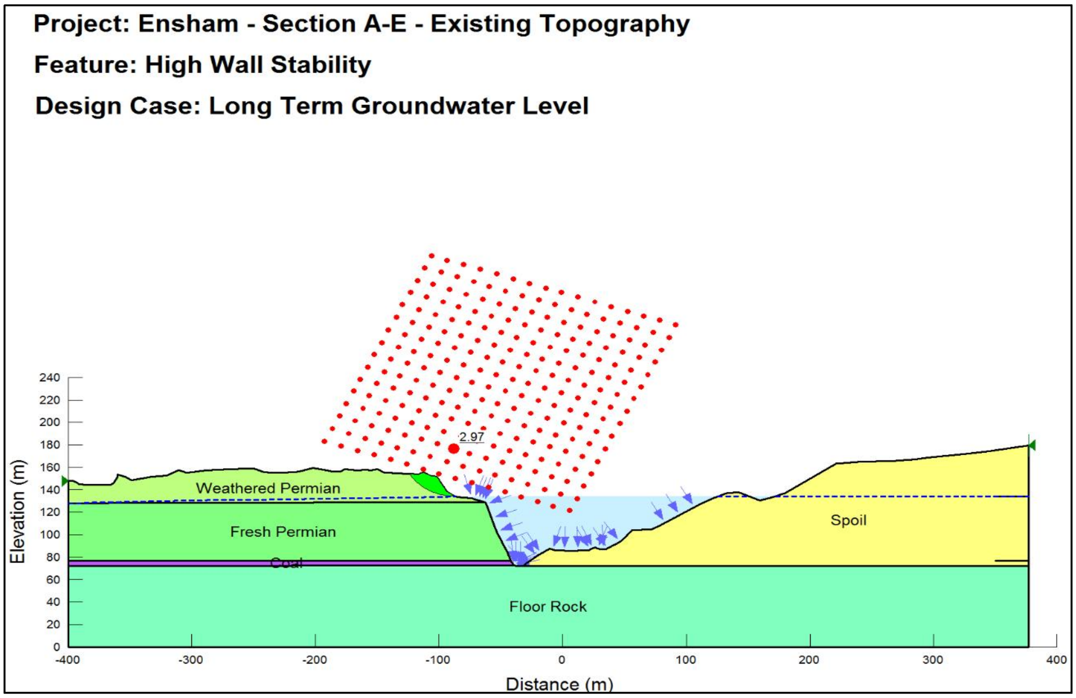


Figure 4-7: SLOPE/W Results for Typical Section A-E – Long-term FOS = 2.97

4.2.3.2 SECTION F (PIT F)

The model results indicate that failures in the overall rock mass are not expected through the weathered and fresh Permian materials exposed in the rock walls with grades typically encountered in F-pit. The Tertiary clay soils have a FOS of 1 to <1 and would not be considered stable as shown on Figure 4-8. The model shows that regrading of these materials would improve their long-term stability (FOS – 1.5).

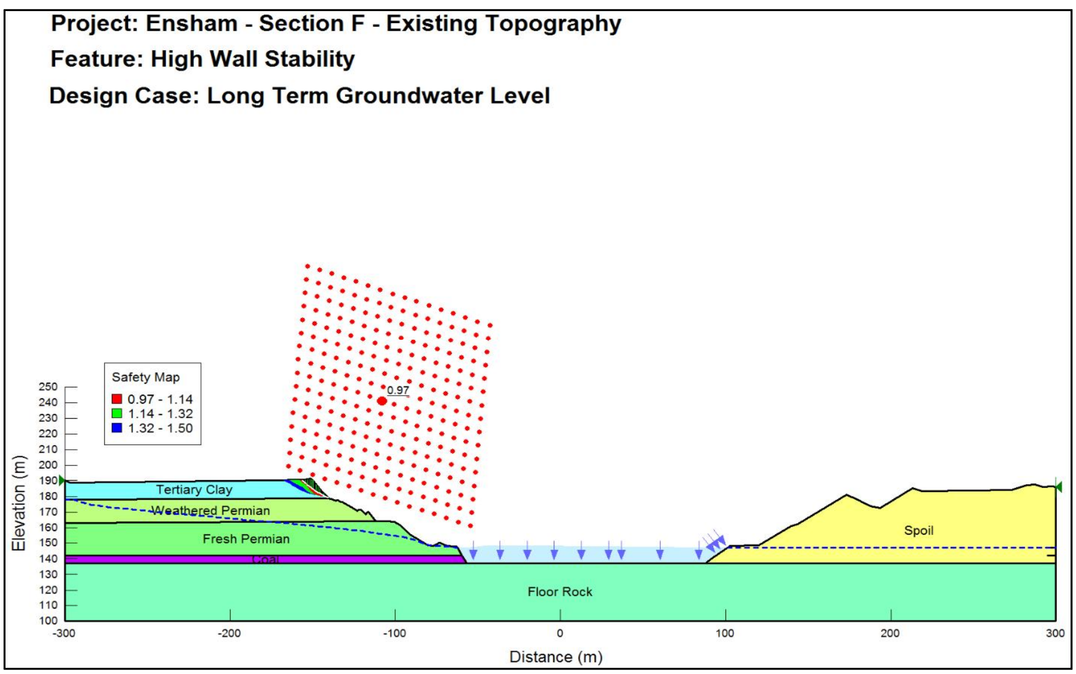


Figure 4-8: SLOPE/W Results for Typical Section F – Long-term FOS=0.97

4.2.3.3 SECTION Y (Y PIT)

The model results in Figure 4-9 indicate that the overall stability of the weathered Permian materials exposed in the pit walls with grades typically encountered in Y-pit are marginally short-term stable in their current condition with FOS of 1.27. The Tertiary clay soils in their current state have a FOS of 1 to <1. The model shows that regrading of Permian and Tertiary materials would improve their long-term stability.

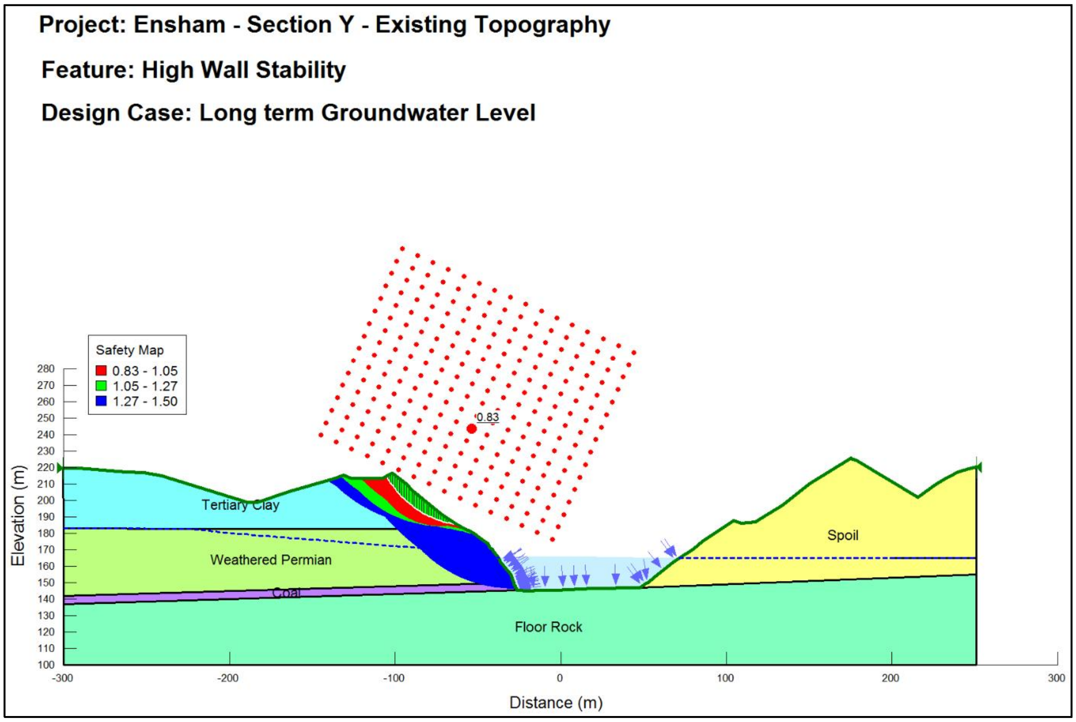


Figure 4-9: SLOPE/W Results for Typical Section Y – Long-term FOS=0.83

4.2.3.4 SUMMARY

The Slope/W results indicate that the weathered Permian and fresh Permian materials exposed in the highwalls and endwalls of the open cut voids at Ensham have an overall FOS of >1.5 to 3 and would therefore be considered long-term stable in their current state and under conditions expected in the longer-term. Faulting and discontinuities, low durability materials, possible reduction in rock strength due to increased moisture content, and wave action in the rock faces can lead to deterioration and failures, also surface water flow over the exposed rock wall faces could promote erosion and localised instability.

Pits which have exposures of weathered Tertiary clay or steeply cut weathered Permian have FOS of <1.3 to <1 and would not be considered stable in the short-term or long-term without regrading.

4.2.4 REGRADED CONFIGURATION (STABILISATION MEASURES)

The typical sections were re-run using SLOPE/W for the long-term condition with regraded slope profiles to assess the change in FOS. Tertiary clay soils were regraded to 1V:3H, weathered Permian materials were modelled at 1V:2H and fresh Permian materials were graded to 1V:1H. The assessment considered the highwall stability for flooded and dry pit water levels, with results as indicated in Figure 4-10 to Figure 4-16, including a sensitivity check on the weathered Permian in Figure 4-13, and a case where the current water level is maintained and all regrading is above that level in Figure 4-17. The results indicate that the FOS for the scenarios modelled increase as expected, with all slope FOS >1.5.

However, durability, possible reduction in rock strength due to saturation, and wave action may reduce the long-term stability in some cases.

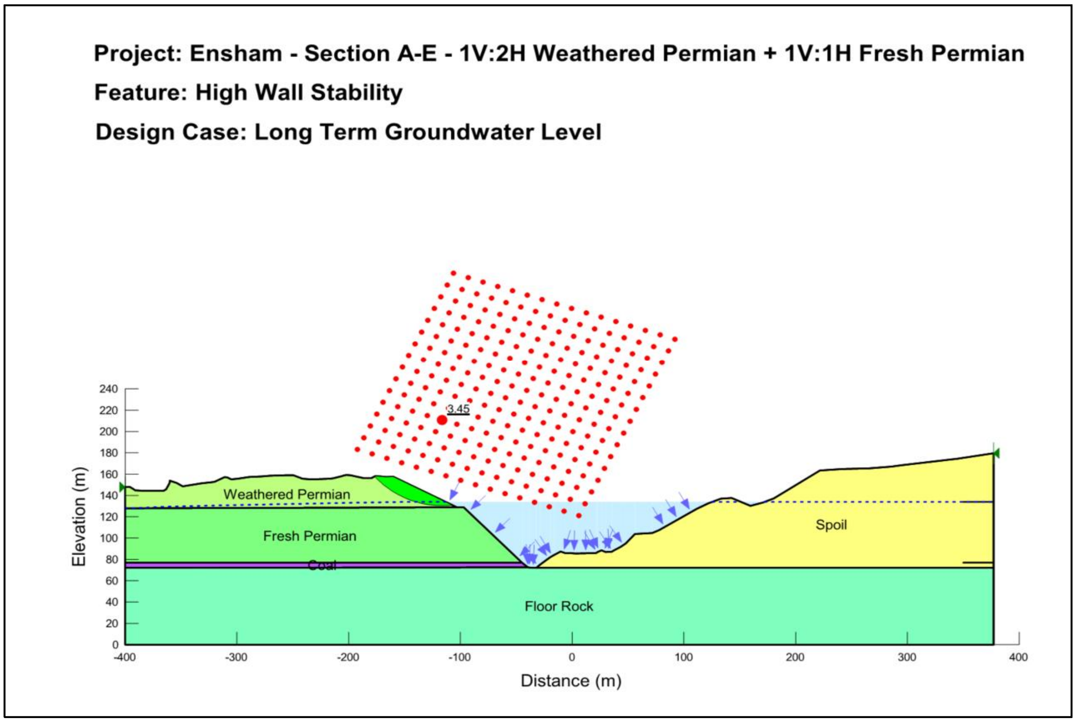


Figure 4-10: SLOPE/W Results for Typical Section A-E – Long-term with flooded pit, Regraded FOS=3.45

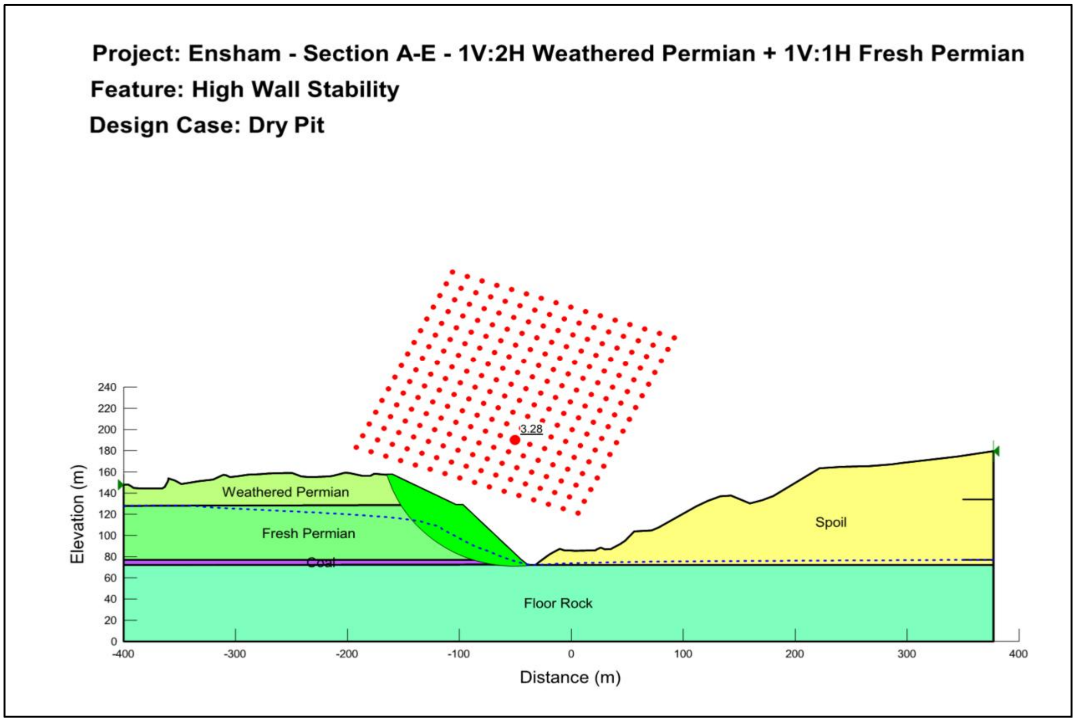


Figure 4-11: SLOPE/W Results for Typical Section A-E – Long-term with dry pit, Regraded FOS=3.28

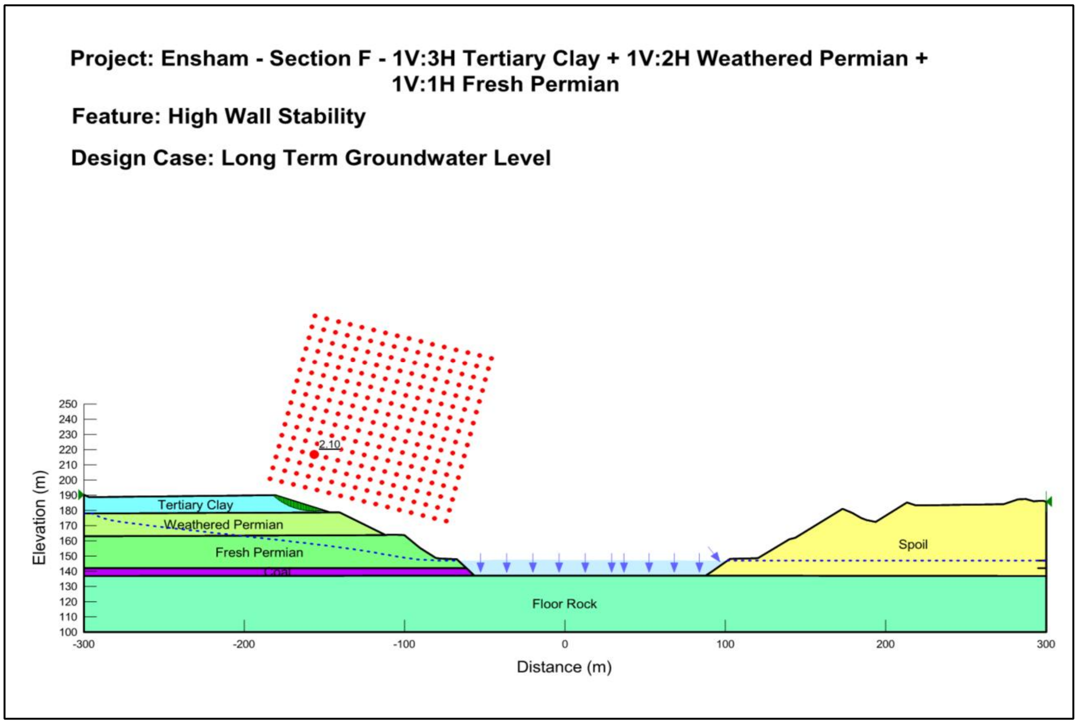


Figure 4-12: SLOPE/W Results for Typical Section F – Long-term with flooded pit, Regraded FOS=2.1

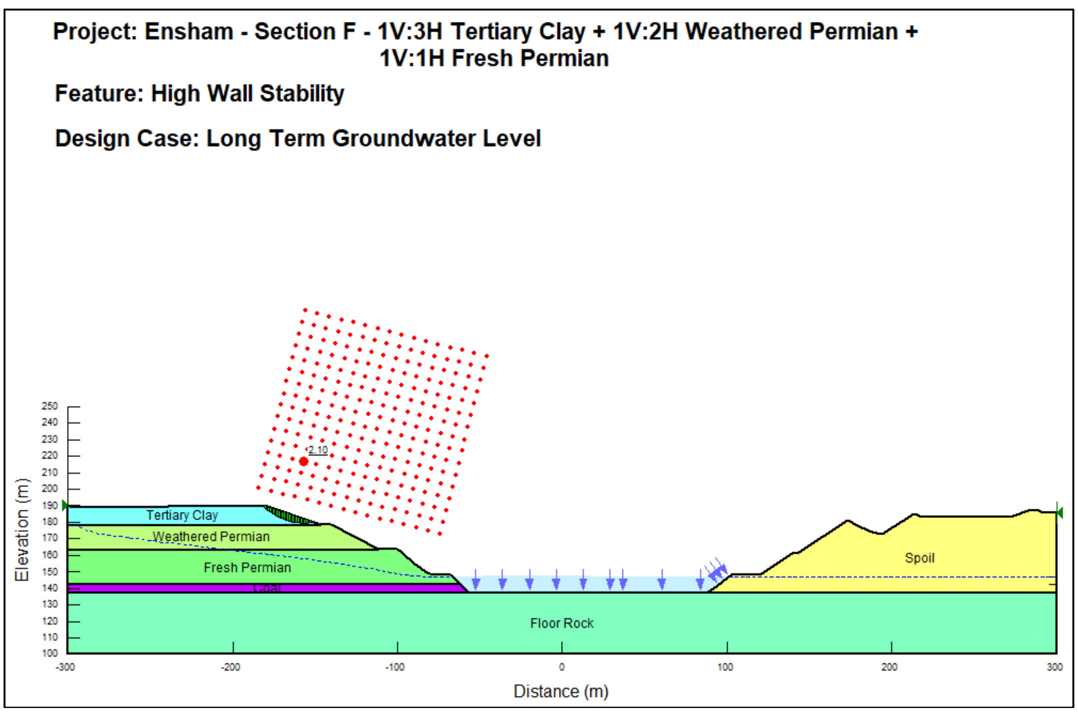


Figure 4-13: SLOPE/W Results for sensitivity analyse with lower weathered Permian strength Typical Section F – Long-term with flooded pit, Regraded FOS=2.10

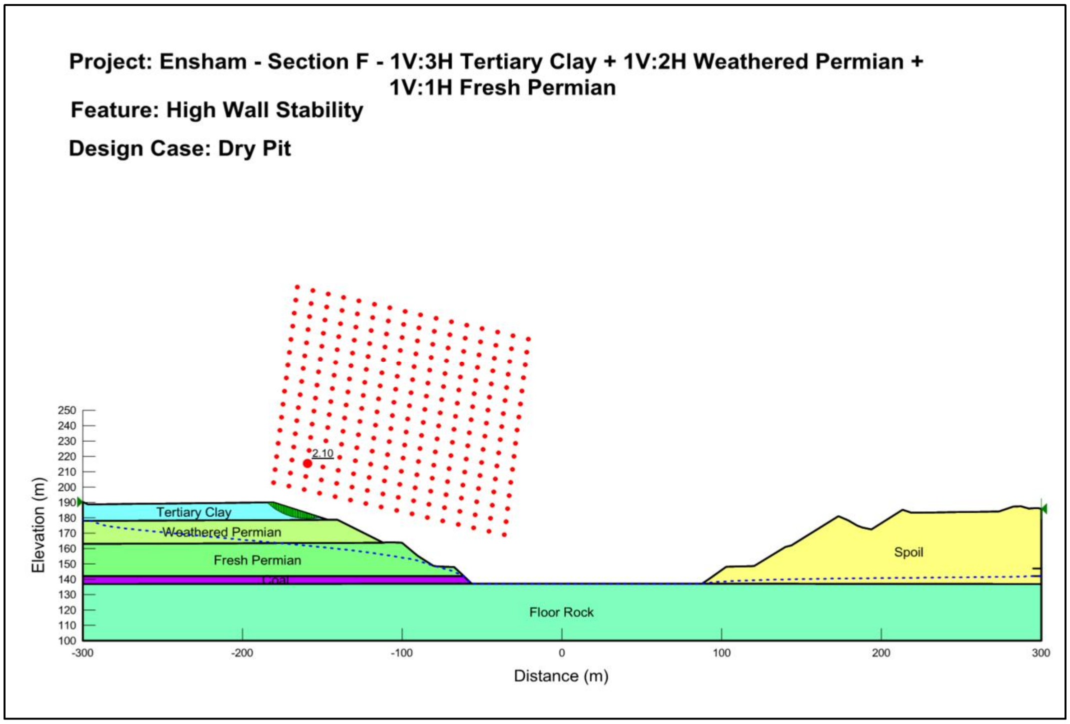


Figure 4-14: SLOPE/W Results for Typical Section F – Long-term with dry pit, Regraded FOS=2.1

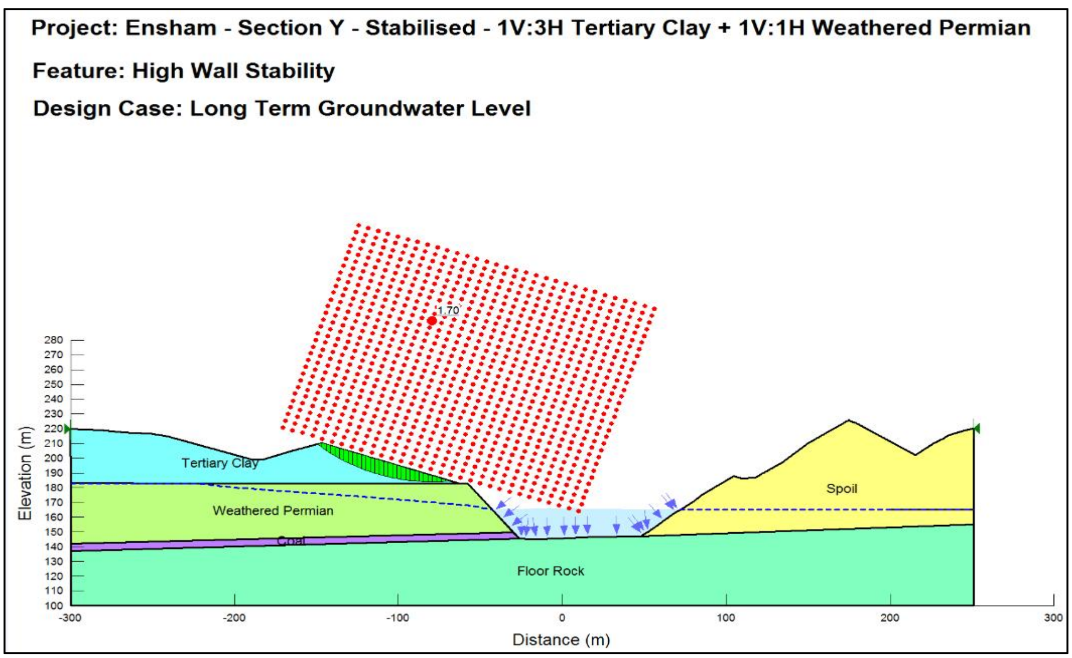


Figure 4-15: SLOPE/W Results for Typical Section Y – Long-term with flooded pit, Regraded FOS=1.7

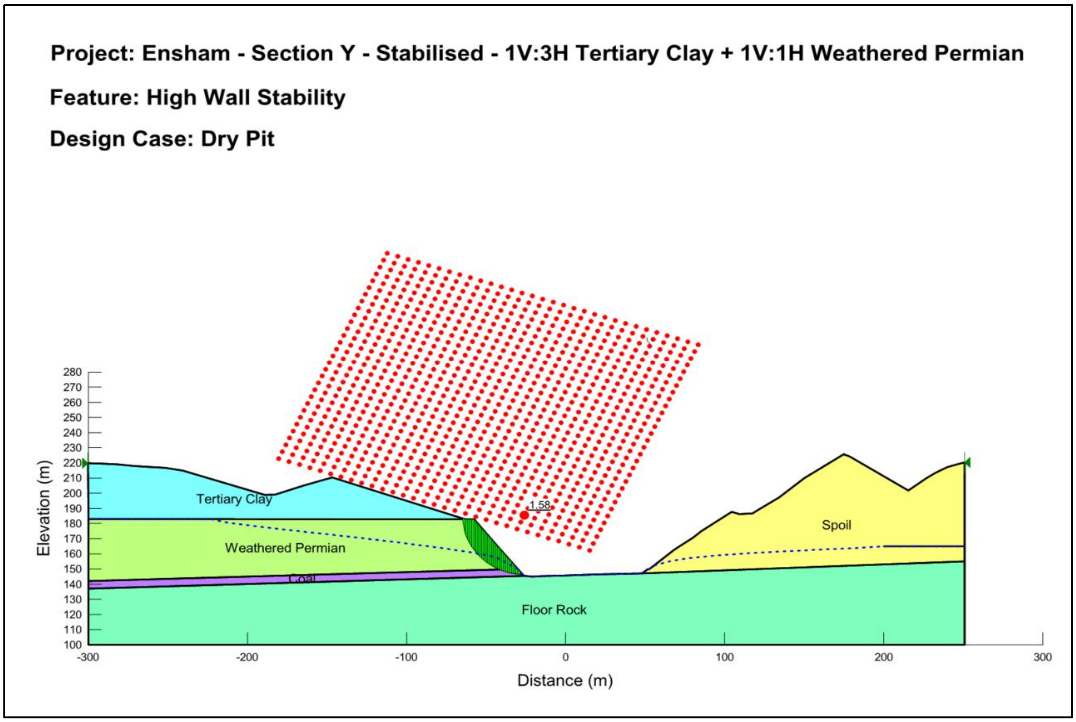


Figure 4-16: SLOPE/W Results for Typical Section Y – Long-term with dry pit, Regraded FOS=1.6

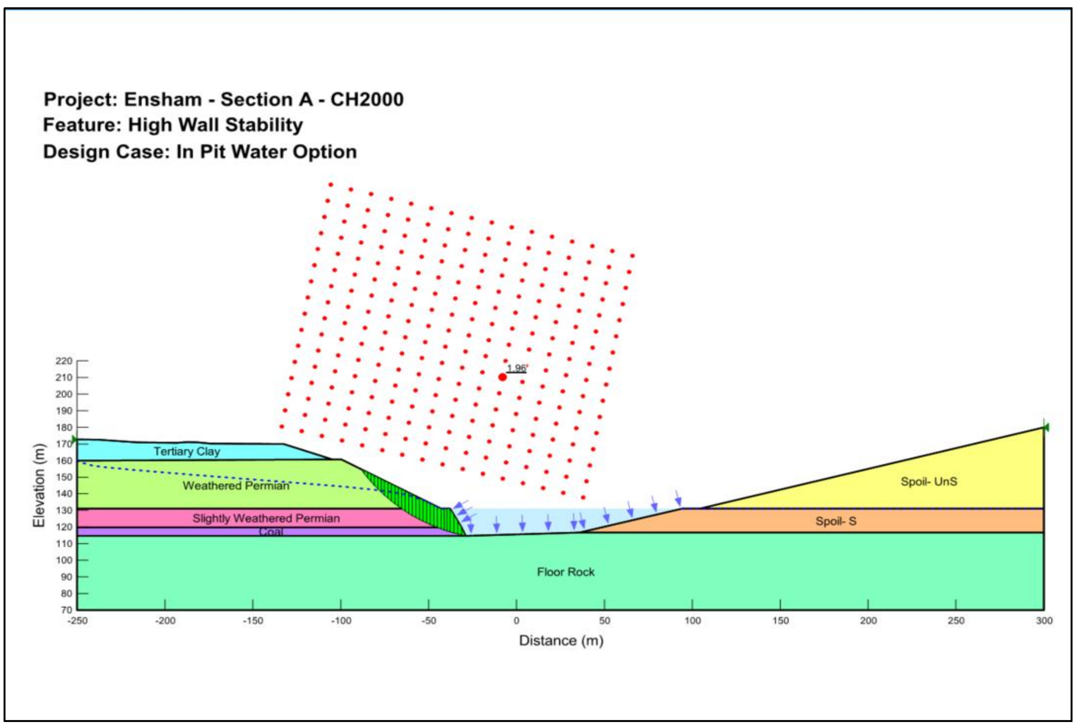


Figure 4-17: SLOPE/W Results for Pit A, CH2000 – Long-term with in pit water and regrading above water level FOS=1.96

4.3 SPOIL PILES

4.3.1 INTRODUCTION

The spoil piles at Ensham Mine are typical of dragline operations and consist mainly of dragline dumped spoil at the angle of repose on the pit low walls. The spoil material generally consists of:

- Tertiary Clays – this material is found mainly to the north in Y-pit and F-pit. The material typically dominates the pre-strip horizon and consists mainly of yellow/brown silt and clay. These materials typically have a marked loss of strength when wet and tend to be prone to instability and severe erosion. (ACARP, 2004). Photos below taken during the January inspection show examples of the Tertiary spoil materials at Ensham mine.
- Weathered/Fresh Permian Rock - This material is found mainly to the south in A-pit, B-pit, C-pit, D-pit and E-pit. The material is typically grey in colour and contains less clay than the Tertiary materials. PSM, 2015 noted that the Permian materials tend to slake and soften over time. (ACARP, 2004) indicates that the materials typically behave more like a soil than weathered rock over time, and can be susceptible to erosion. Samples of Permian materials observed in the spoil piles during the January inspection appeared to be micaceous. The exposed rocks appeared to break down along micaceous and carbonaceous laminations. Photos 4-10 to 4-14 taken during the January inspection below show examples of the Permian materials at Ensham mine.

Photo 4-10: Tertiary Clay and Permian Rock exposed at C-pit

Photo 4-11: Tertiary Spoil Y-pit

Photo 4-12: Instability in Tertiary Spoil Overlying Weathered Permian Material in Y-pit

Photo 4-13: Permian Spoil Showing Gullying and Rill Erosion near B-pit

Photo 4-14: Permian Spoil (note degradation)

4.3.2 CURRENT CONFIGURATION SPOIL PILE STABILITY

A typical section through the soil piles on the low walls of the pit voids in their current state was modelled using SLOPE/W as shown on Figure 4-18. The typical section was based on the spoil pile geometry on the Y-pit low wall. Soil material parameters used in the model have been based on those used by PSM in previous stability assessments in 2015 (report reference: PSM687-222L Geotechnical Review of Final Landform). The parameters provided by PSM, 2015 were considered appropriate for the materials observed during the site inspection and fit with those used in the Bowen Basin (Simmons and McManus, 2004), and described in the provided consultant’s reports. The adopted parameters are presented in Table 4-4.

A FOS of 1.5 has been adopted for long-term stability. This value is typically adopted for earth embankments including dams, Fell 2005. The model produced FOS values varying from 1.05 to 1.26 depending on the degree of saturation of the spoil. These FOS values indicate that most spoil piles in their current state (at the angle of repose, with a low groundwater level) would not be considered long-term stable and are marginally stable in the short-term. This is supported by evidence of slips observed during the January site inspection. The modelled output is in Figure 4-18 for unsaturated conditions and Figure 4-19 for a 5m thick saturated zone in the spoil at the bottom of the pile. Regrading of the spoil piles will be required to achieve a landform that would be considered long-term stable.

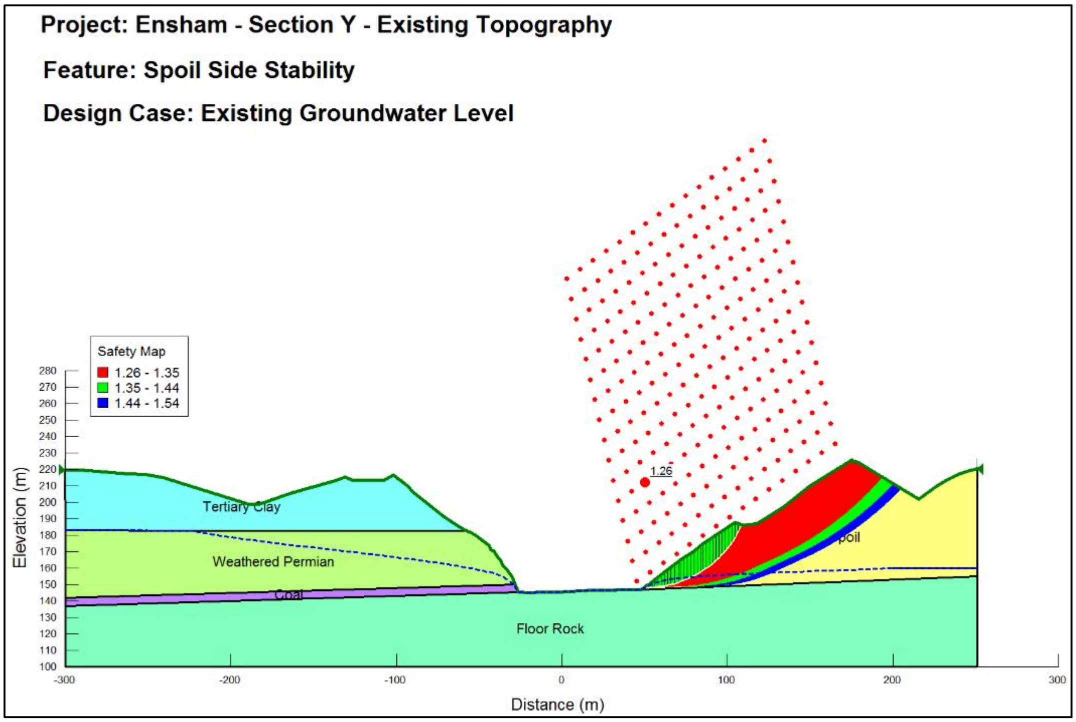


Figure 4-18: SLOPE/W Results for Typical Unsaturated Spoil Section, FOS=1.26

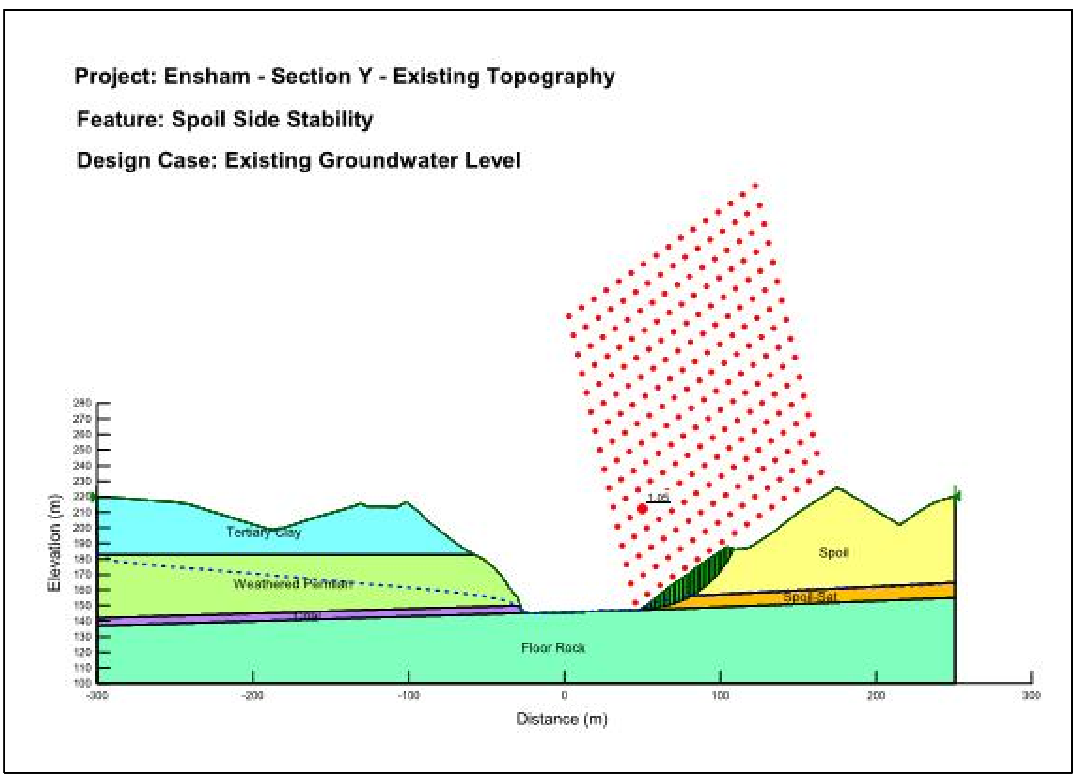


Figure 4-19: SLOPE/W Results for Typical Saturated Spoil Section, FOS=1.05

4.3.3 REGRADED CONFIGURATION (STABILISATION MEASURES)

Rehabilitated areas observed during the site inspection typically had slopes of 1V:10H (10%) for Tertiary materials and 1V:7H (15%) for Permian materials. These areas appeared generally stable and supported vegetation growth. To achieve a landform that would be considered long-term stable will require significant regrading of the spoil piles. An example of a spoil pile regraded to 1V:4H is provided in Figure 4-20. Movement of some of the spoil material into the pit voids could assist with improving the stability of high wall areas and reduce the quantity of earthworks required to regrade the highwall areas. Regrading of the spoil piles should consider the potentially erosive nature of the spoil materials and provide adequate drainage for surface water. It is noted that regraded spoil may settle due to changed loadings or degradation of materials. Settlement could impact surface water drainage.

A further model was run for the scenario where pits have been flooded historically resulting in saturation of the spoil. According to Simmons and McManus (2004), a saturated strength mode in spoil is not reversible, even after free water has been drained from the void space due to slaking, softening, and weakening that occurs during saturation. This model utilises saturated spoil parameters for the full profile of the spoil material. The results for this simulation are included in Figure 4-21.

Sensitivity analyses using lower and higher spoil parameters as in indicated in Table 4-5 were performed and the results shown on Figure 4-22 and Figure 4-23. The FOS values were greater than 1.5 and therefore, acceptable.

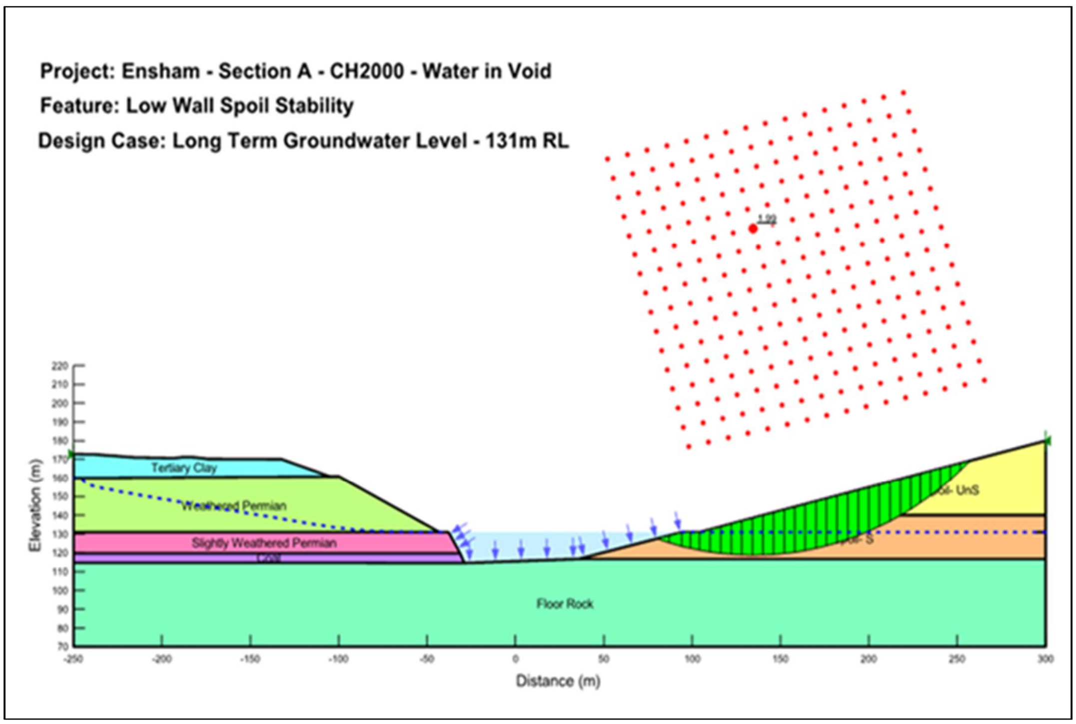


Figure 4-20: SLOPE/W Results for Regraded Typical Saturated Spoil Section, FOS=1.99

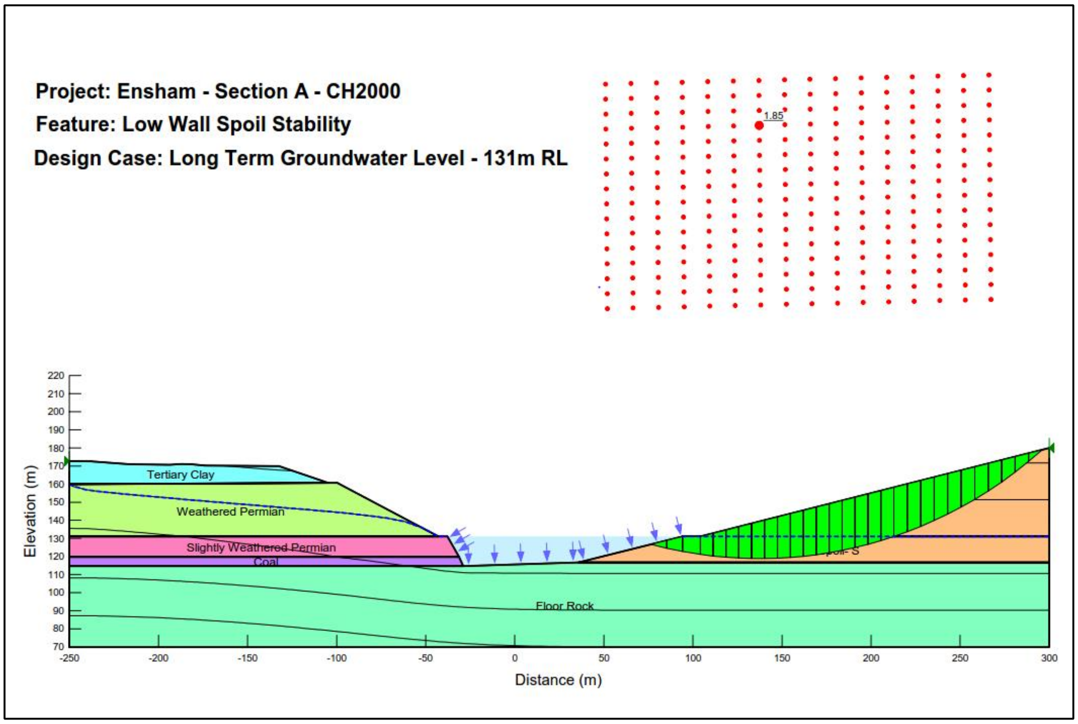


Figure 4-21: SLOPE/W Results for Regraded Spoil Section Previously Flooded, FOS=1.85

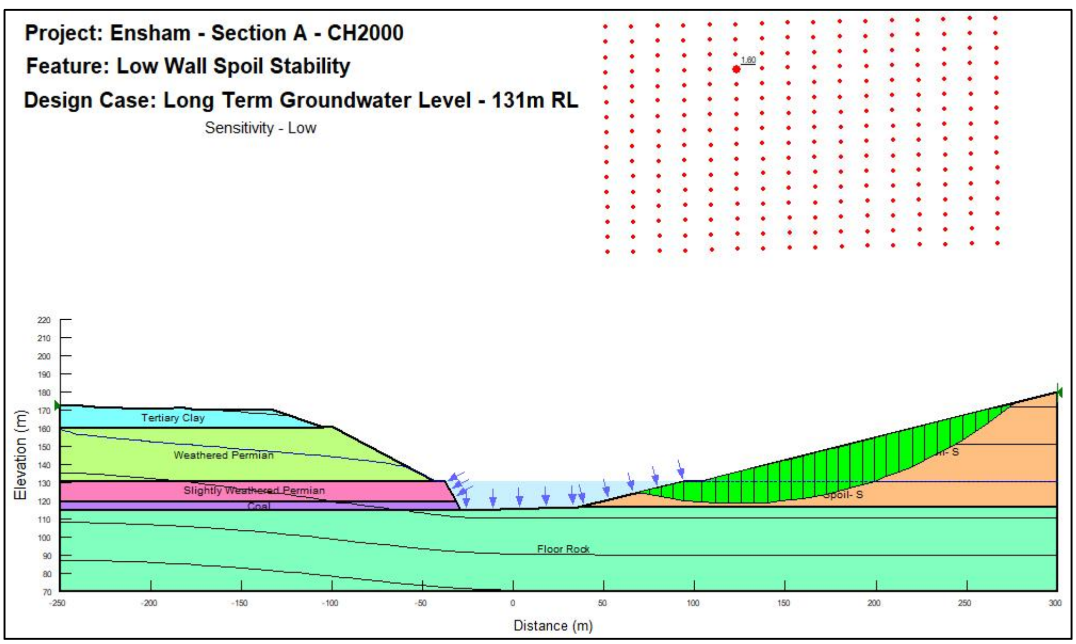


Figure 4-22: SLOPE/W Results for sensitivity analyses with low parameters for Regraded Spoil Section Previously Flooded, FOS=1.60

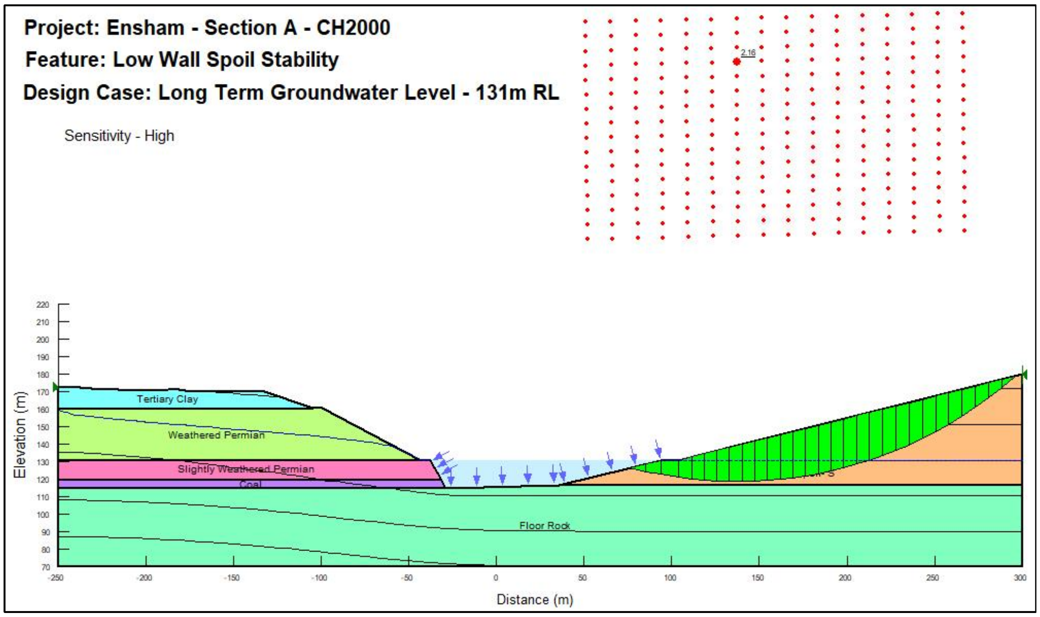


Figure 4-23: SLOPE/W Results for sensitivity analyses with high parameters for Regraded Spoil Section Previously Flooded, FOS=2.16

4.4 EXISTING LEVEES

4.4.1 CURRENT CONFIGURATION

For Options 1 and 2 the existing levees on the site will form some part of the final landform. For Option 2 these levees will be retained in their existing form, and will continue to perform as flood control structures. For Option 1, permanent landforms will be constructed, that will isolate residual voids in the flood plain preventing inundation for all events up to a 0.1% (1:1000) AEP event, requiring a slight raising of the existing levees. These larger landform levees may encapsulate the existing levees, requiring them to form part of a new, larger levee structure. These existing levees will be removed for Option 3, hence all discussion relating to levees relates to only Option 1 and 2.

Two levees have been constructed to protect several of the open cut pits from flooding from the Nogoa River and the Anabranch River. The northern levee protects C and D-pits and the southern levee protects B-pit. The levees were constructed in stages between 2006 and 2010 and are regulated structures under Ensham Environmental Authority (No. MIN102573611).

The C-pit (northern) and B-pit levees (southern) were initially designed for a 1% (1:100) AEP flood event. These levees were upgraded for a 0.1% (1:1000) AEP event by Kellogg Brown and Root (KBR) and Douglas Partners (DP) in 2008/2009, who also completed design of the D Pit levee. The levees were designed to be between 1.2m to 1.4m above the 0.1% AEP event. The heights of the levee banks are shown in Table 4-8, and a summary of each levee and designs by KBR and DP follows. Preferential overtopping points were provided in both levees, with the B Pit levee overtopping point being 0.2 m below that of C Pit to protect the C Pit underground access portal in the event of overtopping. (KBR 2014 report BEE603-012-C-REP-001 Rev 2). Levee design details were based on information provided by Ensham, and are as follows:

— batter slopes 1V:3H

— 6m wide crest for vehicle access

— homogenous embankment for new embankments or general fill lined with a 3m wide low permeability clay zone on the upstream face to provide a seepage barrier for upgraded embankments

- constructed from borrow material sourced within the mine site with following properties:
  - % passing 150 mm – 100
  - % passing 0.075mm – 30-90
  - PI > 12%
  - Emerson Crumb > 2
  - Liquid Limit > 40%

Levee construction was to include the following as recommended in DP's 2009 report (47793.doc):

- stripping of gravel wearing course and outer levee face of existing embankments, forming horizontal benches to accommodate a compactor for placement of new fill
- excavation of a key trench; as-constructed drawings indicate the extent of the constructed key trench; however, the information provided did not confirm that these had been constructed into an impermeable sub-stratum
- moisture conditioning of exposed soils to achieve compaction requirements.

Table 4-8: Height of Levee

LEVEE	LOCATION	APPROX MAX HEIGHT (m)
Northern	West section	8 (10)*
	Central & East section	4
Southern	West section	6 (10)*
	Central & East section	5

4.4.2 C-PIT LEVEE BREACH

The C Pit levee, was originally designed to provide 1% (1:100) AEP flood event protection. The western end of the levee was overtopped and breached during flooding in 2008. The breach occurred at a former billabong and an area of scour in the Anabranch River course. The breach was originally backfilled with end dumped mine spoil, however this material was found to be permeable and provided preferential seepage paths.

The levee design was upgraded in 2009 to provide 0.1% (1:1000) AEP flood protection to the underground mine entrance portal and replace the mine spoil in the breach area with engineered fill. DP's 2013 report (47793.09. doc) indicates that the breach reconstruction included a well compacted support embankment on the river side of the breach zone, placement of an impermeable HDPE liner keyed into the bedrock and ends of the former breach zone, then backfilling of the key trench and breach zone with compacted levee fill.

4.4.3 B-PIT LEVEE BREACH

The B-pit levee, was originally designed to provide 1% AEP flood immunity. The levee was extended and upgraded to provide 0.1% AEP flood protection in 2009. During the site inspection undertaken in January 2018, Ensham Mine Representatives indicated that there was a breach of the levee during the 2008 flood event. The breach occurred where the levee crosses the former bed of Old Winton Creek. The breach occurred through the B Pit Interim Levee on an alignment that has subsequently been mined through and replaced with the B Pit Levee Extension, further to the west of the interim levee alignment, and is hence not considered in this report.

4.4.4 LONG-TERM STABILITY OF THE LEVEES

The design of the levees described above was undertaken by KBR with geotechnical input provided by Douglas Partners, and is described in the following reports:

- Ensham Mine Flood Levees – Proposed Levees Design Report - BEE603-W-REP-005 Rev 2, 21 February 2014, KBR
- Ensham Mine Flood Levees – Design Report for the Proposed C Pit Levee and Upgrade of Existing B Pit Levee - BEE603-009-C-REP-004 Rev 3, 21 February 2014, KBR
- Ensham Mine Flood Levees – Design Report for Preferential Overtopping Points - BEE603-012-C-REP-001 Rev 2, 21 February 2014, KBR
- Hydrogeological Assessment of Sub-Surface Conditions Beneath Ensham Mine B Pit Levee – Project 47793.01 September 2009, Douglas Partners

Overarching upgrade design criteria adopted for these levees are described in Report BEE603-009-C-REP-004 Rev 3 as being:

- a minimum factor of safety of 1.5 against instability
- sufficient distance between the levee and highwall/endwall to achieve an acceptable risk of seepage piping failure.

In Report BEE603-009-C-REP-004 Rev 3, a critical area was identified for the B Pit levee adjacent to the Ramp 26 pit endwall, where long term FOS of 1.5 could not be achieved. To achieve a FOS of more than 1.5 it was found that the following measures would be required:

- partial infilling of Ramp 26 to a level of the top of the Permian material
- provision of a 20 m wide undisturbed bench at the toe of the levee
- provision of a compacted, free draining sand fill zone, battering down from the levee toe bench to the partial fill level in the pit at 1V:4H, with top soil and grass placement on the sand embankment. The sand fill should provide a stable batter through which seepage will occur.

It should be noted that at the time of this assessment the B Pit highwall had not advanced as far west as its current location, and the ramp associated with this critical section has subsequently been partially backfilled with spoil. However, the comments in this KBR report regarding criticality of this levee between the Nogoa River floodplain and B Pit end wall are still relevant to current conditions on site.

A similar 100 m long critical section was evaluated for the D Pit levee extension, where mining had resulted in a progression of the highwall pit excavation to within 30 m of the toe of the batter. Due to the proximity of the pit wall to the levee, the design was based on the construction of a 3.5 m wide cut-off trench extending a minimum of 0.3 m into the underlying clay layer for the length of this critical section to reduce seepage below the levee which could cause piping instability in the highwall profile. A seepage and stability analysis was undertaken for a typical cross section of the levee / highwall in this area, with subsurface strata material properties inferred from CPT test results undertaken along the levee alignment.

The results of the assessment indicated long term factors of safety for the levee and highwall for this critical location of 2.23 for 0.1% AEP flood conditions, and 2.11 for long term seismic conditions, which are more than the required 1.5 FOS (Table 4.14 BEE603-W-REP-005 Rev 2).

Based on a review of the above-mentioned reports, the existing levee structures, if constructed in accordance with design guidelines and the materials encountered were as per the assumptions made for the design, will comply with the design criteria adopted.

For this project the current levees will be incorporated in proposed options in the following manner:

Option 1 – Landform levee – the current levee structures will be transformed into landform with criteria adopted from those identified from landform design, and will be raised to provide 0.1% AEP protection, based on current hydrological design methods. No preferential overtopping points will be provided. Where there are highwall spoil dumps between the levee and edge of the highwall, these may be reshaped to form a landform with free draining surface drainage. The intention of this option is that these structures will be long term stable without the requirement for a higher level of maintenance than surrounding natural landform.

Option 2 – Flood mitigation and beneficial use – the current levees will be incorporated in their existing configuration wherever possible, retaining alignment, height, and section geometry. The option requires the ongoing presence of system operator, who would do periodic inspections and maintenance of the levee structures to maintain their integrity.

Option 3 – Backfill to PMF – the levees will be removed completely.

For Options 1 and 2, the proximity of the highwall crest to the levee toe have shifted significantly since the design and construction of the southern and northern levee extensions were undertaken. In the levee design report (Table 4.6 and 4.7 BEE603-W-REP-005 Rev 2), it was recommended that 150 m be maintained between the toe of the levee and the pit edge for there to be an unlikely or extremely unlikely risk of piping failure during a design flood event (non-compliances with this recommendation were identified for both B and D pit levees, and specific analysis was undertaken to determine stability as described earlier). For the most part, the existing alignment of the levees still complies with this recommendation (see Appendix A); however, the "pinch points", where the levee toe and highwall extent are closer than recommended distances, identified in the original design are still present.

4.5 PROPOSED LEVEE CONFIGURATION

4.5.1 GEOTECHNICAL CHARACTERISTICS OF NOGOA RIVER FLOOD PLAIN

The alignment of the levees for Option 1 is as per existing levee alignments, and are shown on the layout Plan in Appendix A. The levee alignments pass close to the final highwall regrade extent at critical sections adjacent to both B and C Pits. These sections were identified and investigated for seepage and stability compliance during original design work of these levee extensions (KBR Report BB603-W-REP-005 Rev 2). This work included a geotechnical investigation comprising drilled boreholes at locations along the B Pit levee extension alignment, and CPT testing along both B and D Pit levee profiles. CPT testing was used to infer subsurface profiles in the absence of borehole data. In the KBR Report the section of levee adjacent to C Pit from Red Hill to Ch 3600 (Chainage locations as per Appendix A) was referred to as D Pit Levee Extension (South), while the section from Ch 3800 northwards was referred to as D Pit Levee Extension (North). In this KBR report, the subsurface profile for the critical sections of levee were described as follows:

D Pit Levee Extension (South) – (which equates to C Pit levee Red Hill to Ch 3600 - Appendix A of this report):

- silty sand/sandy silt with clay: fine to medium grain sand with lenses of silty clay and clayey silt, layer thickness from 2 to 3 m
- clay (alluvium): high plasticity very stiff clay, layer thickness from 3 to 6 m
- rock: extremely to moderately weathered Permian siltstone rock at 1 to 9 m depth from surface

D Pit Levee Extension (North) – (C Pit levee Ch 3800 north - Appendix A of this report):

- clay and silt (alluvium): high plasticity, stiff to very stiff clay, layer thickness from 2 to 10.5 m
- sand and silty sand (alluvium): fine to medium grain sand with lenses of silty clay and clayey silt, layer thickness from 1 to 4.5 m
- rock: extremely to moderately weathered Permian siltstone rock at 1 to 14 m depth from surface

B Pit Levee Extension – (B Pit levee Ch 2000 – 2900 - Appendix A of this report):

- clay and silt (alluvium): high plasticity, stiff to very stiff clay, layer thickness from 2 to 7 m
- sand and silty sand (alluvium): fine to medium grain sand with lenses of silty clay and clayey silt, layer thickness from 1 to 9 m
- rock: extremely to moderately weathered Permian siltstone rock.

In the Groundwater Study undertaken by Australian Groundwater and Environmental (AGE) in 2006 (included in Appendix F of the KBR Report BEE603-W-REP-005 Rev 2), 20 boreholes were drilled in the flood plain adjacent to the Nogoa River in the area of the Ensham Mine, to identify the degree of hydraulic connection between the Nogoa River and underlying alluvial sand and gravel aquifer. The study found that "the base and sides of the Nogoa River consist of clay and clay-bound sand and gravel that form an effective seal, and therefore there is effectively no connection between the Nogoa River waters and coarse sands and gravel aquifer of the associated flood plain alluvium". The study found that the clay layer sealing the alluvial substrata varies, but is approximately 10 m deep. This is supported by borehole and CPT testing undertaken for levee extension designs, although variability in subsurface materials was also evident.

For the purposes of seepage analysis and stability assessment, the following parameters have been adopted for Option 1 and 2 levee configurations.

Table 4-9: Adopted material parameters for levee slope analysis

MATERIAL	DENSITY (γ - kN/m^3)	C' (kPa)	Ø' (deg)
Levee embankment	18	14	32
Alluvium clay	18	10	23
Alluvium sand	18	0	35
Weathered Permian	20	80	40

Table 4-10: Adopted permeability parameters for levee seepage analysis

MATERIAL	PERMEABILITY (K - m/s)	
	High	Low
Levee embankment	$1x10^{-7}$	$1x10^{-7}$
Alluvium clay	$1x10^{-6}$	$1x10^{-7}$
Alluvium sand	$5x10^{-3}$	$5x10^{-4}$
Weathered Permian	$5x10^{-8}$	$5x10^{-9}$
Fresh Permian	$5x10^{-12}$	$5x10^{-12}$

Permeability parameters are as per those adopted for levee modelling described in KBR Report BEE603-W-REP-005 Rev 2. High permeability values have been used for seepage analysis.

4.5.2 OPTION 1

4.5.2.1 GENERAL LEVEE CONFIGURATION

Option 1 incorporates raising of the current levee structures, and flattening upstream and downstream batters to slopes of 1V:6.7H (15%), in line with final landform criteria. An alternative profile with a 1V:4H downstream batter is proposed for sections where spatial constraints occur on the void side of the levee. It has been assumed all levee modifications works will be undertaken with suitable low permeability, non-dispersive, clay-rich materials.

Slope stability analysis has been undertaken for both of these general levee configuration (where levees are located 150m away from highwalls), which assumed a 10 m high levee, a water surface at the top of the levee, and fully saturated embankment conditions (no seepage analysis was undertaken as fully saturated conditions represents worst case scenario in terms of phreatic surface within the levee embankment). This analysis indicated a factor of safety of 2.17 for a 1V:6.7H downstream embankment, and 1.53 for a 1V:4H downstream embankment, as shown in Figure 4-24 and Figure 4-25 below, which are above the required FOS of 1.5 with conservative assumptions regarding phreatic surface in the downstream levee embankments.

Additional analysis has been undertaken at identified locations where the levee alignments are sufficiently close to highwalls to warrant an assessment of the levee / highwall as a single geotechnical unit (critical sections). A description of the analysis is included under sections 4.5.2.2 and 4.5.2.3 below.

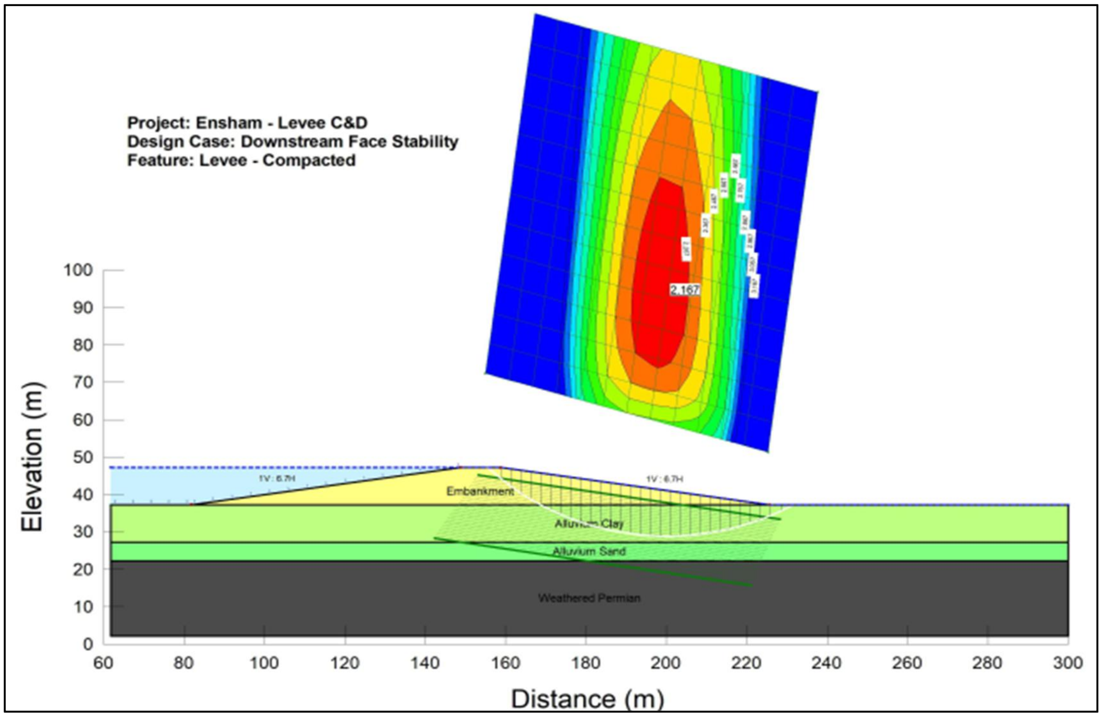


Figure 4-24: SLOPE/W results - Option 1 - general levee profile - 1V:6.7H embankments – FOS 2.17

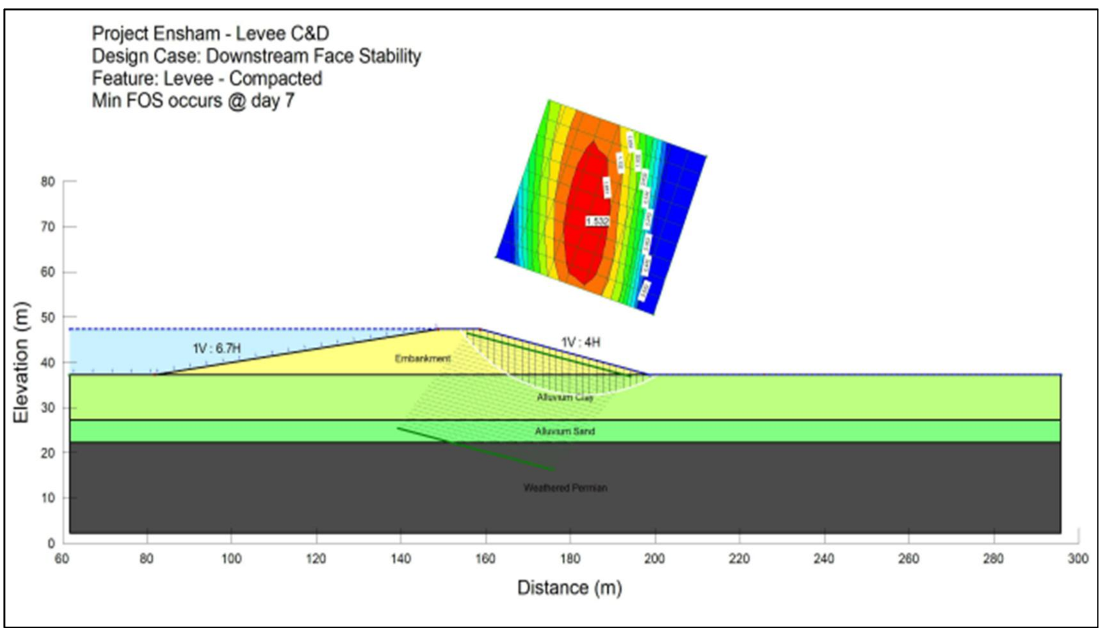


Figure 4-25: SLOPE/W results - Option 1 - general levee profile – 1V:4H downstream embankment – FOS 1.53

4.5.2.2 B PIT CRITICAL SECTION (ADJACENT TO NORTHERN END WALL)

In order to achieve a materials balance for the Option 1 landform, the B Pit will be substantially backfilled in the vicinity of the previously identified critical section of levee adjacent to the B Pit endwall. A section has been taken through this area reflecting the final landform profile, and is included in Figure 4-26 below. This section indicates spoil infill for almost the full depth of pit, with 10% (1V:10H) gradient slopes away from the levee. The spoil fill will substantially stabilise the levee / pit interface and will eliminate seepage considerations, hence no seepage or stability analysis has been undertaken for this area.

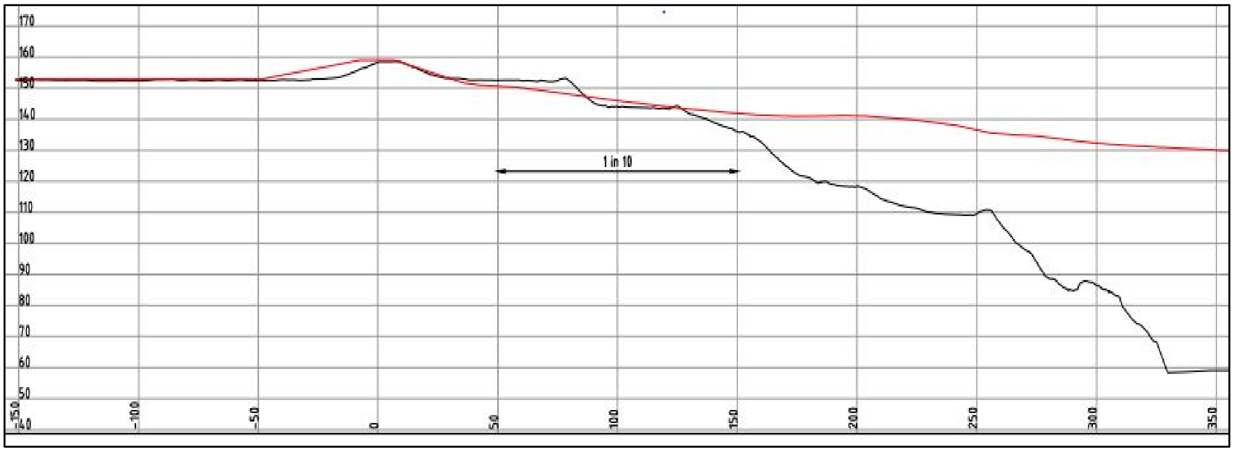


Figure 4-26: Option 1 levee Ch 2500 - B Pit critical section infill profile (red line represents final landform)

4.5.2.3 C PIT CRITICAL SECTION (CH 3400 AREA)

The C Pit levee is within 150 m of the highwall edge for the most of its alignment from Ch 3400 to approximately Ch 4400 (chainages as per Appendix A levee plan). The distance from the downstream toe of the levee to the top of the void regrade embankment crest varies along this area, but final landform design incorporates a shifting of the levee where necessary to ensure that this distance is a minimum of 10 m to allow for access and separation of the levee structure from void regrade.

A single cross section at Ch 3400 has been taken along the C Pit levee alignment for analysis, where the levee is closest to the residual void edge based on the final Option 1 landform. This area of the levee was referred to as D Pit Extension (South) in KBR Report BEE603-W-REP-005 Rev 2, and was investigated for seepage and stability compliance due to its proximity to the pit edge at that time (30 m bench from levee embankment toe to pit edge). The cross section using the Option 1 landform indicates a bench of 10 m from the levee toe to the edge of regrade embankment into the void, and hence represents a worst-case scenario in terms of proximity of levee to the final highwall profile. The soil subsurface profile shown on this section is in line with the description of materials in this area in the KBR report and as described earlier, as well as the profile used for analysis in the KBR report. Steady state and transient seepage models were undertaken for this section using SEEP/W. The steady state assumes a constant water level at the base of the Nogoa Anabranch (conservative assumption for steady state), while the transient model utilised the 0.1% AEP flood hydrograph in the vicinity of the levee to determine subsurface layer exposure to flood waters. The seepage results were imported into the SLOPE/W stability analysis, the results from which are shown in Figure 4-27 and Figure 4-28 below. High range permeability parameters were used for the seepage analysis.

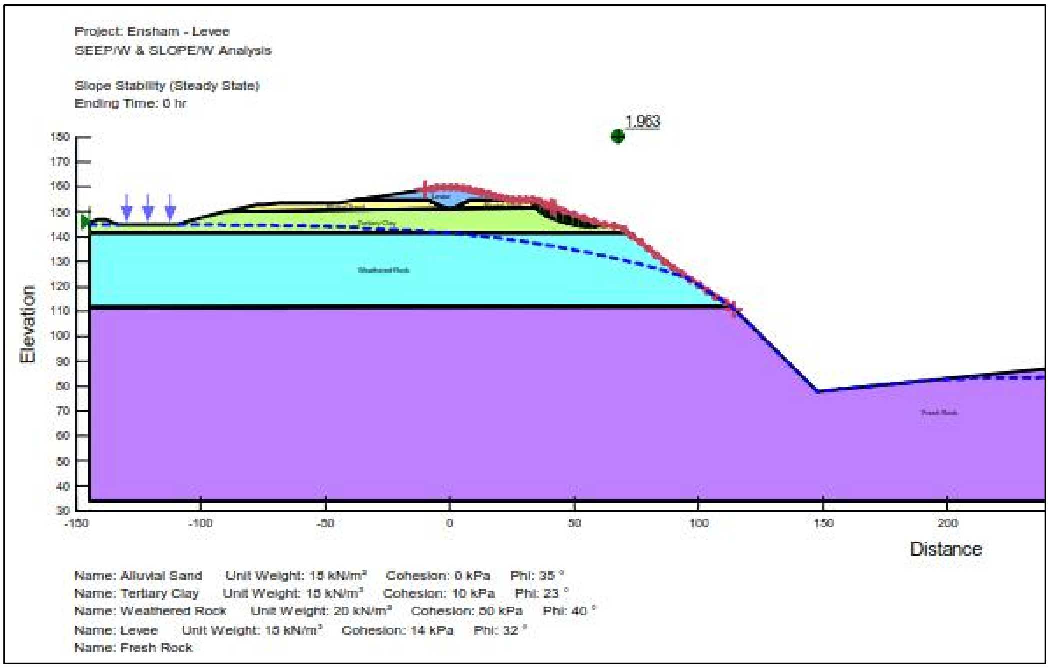


Figure 4-27: SLOPE/W results - Option 1 - C Pit levee Ch 3400 – Steady state - FOS 1.96

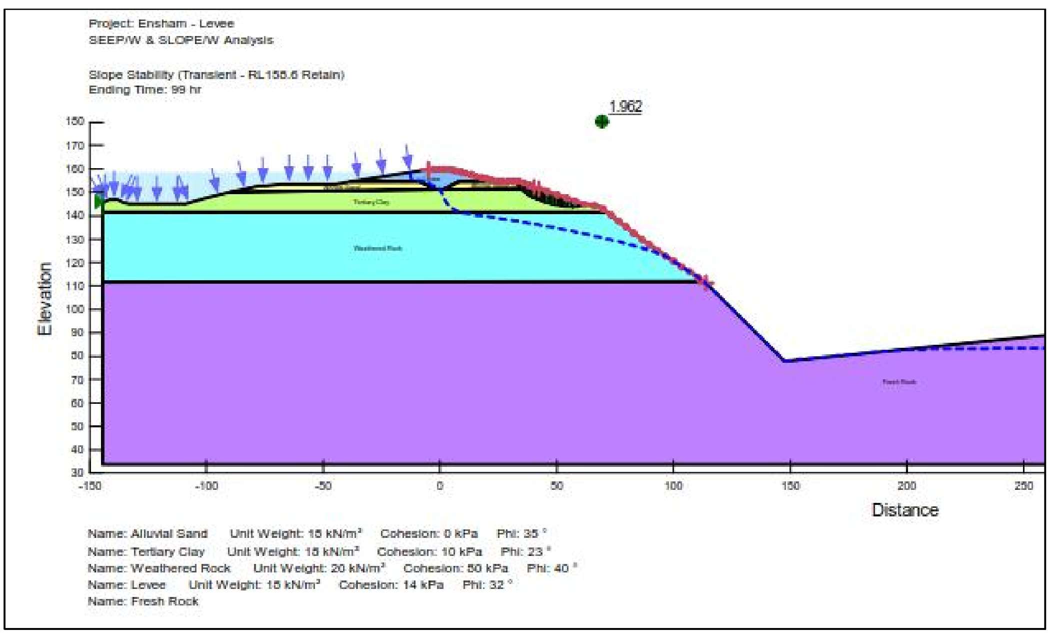


Figure 4-28: SLOPE/W Results - C Pit levee Ch 3400 – 0.1% AEP transient state – FOS 1.96

These results indicate a FOS of 1.96 for both steady and transient states, and are in line with the KBR seepage and stability analysis undertaken previously, which found an overall stability FOS of 2.33 for a profile that included a wider 30 m bench. These FOS are greater than the required 1.5. The analyses assume that the key is effective, and subsurface conditions should be verified to confirm profiles adopted.

4.5.3 OPTION 2

4.5.3.1 GENERAL LEVEE CONFIGURATION

Option 2 incorporates the current levee structures in their existing configuration (height, width, and geometry), the stability of which was addressed in the KBR Report BEE603-W-REP-005 Rev 2, where it was found that levees would have factors of safety in excess of 1.5. A review of the design parameters adopted for the analysis has been undertaken, and provided construction met design intent and expectation in terms of construction materials and subsurface conditions, then these existing levees should still be stable, with factors of safety in excess of the required 1.5.

4.5.3.2 B PIT CRITICAL SECTION (ADJACENT TO NORTHERN END WALL – CH 2800)

Option 2 landform incorporates regrading of highwall and lowwall slopes to achieve safe and stable conditions, however earthworks have been optimised to maximise storage volumes in A, B, C and D-pits, resulting in a different final levee / pit wall geometry in the critical section of B-pit adjacent to the northern end-wall to that of Option 1 described earlier. To support design of the B-pit levee extension detailed in KBR Report BEE603-W-REP-004 Rev 2, an assessment was undertaken at a critical section where the pit excavation had encroached on the levee toe at the B-pit endwall. This section was analysed in its state at that time (section in Figure 4-29 below with no compacted sand fill or minespoil backfill), which showed that with no stabilisation, the profile had a factor of safety of less than 1, with failure occurring in the sand lenses some depth below the levee due to seepage. Mitigation measures proposed to improve overall stability of the section, which are shown in Figure 4-29 below extracted from the KBR report, provided a factor of safety of 1.51. This stabilisation measure, or section profile, was proposed for all similar excavation profiles in this area, and the report indicates that this requirement would be written into Ensham's Standard Operations Procedures.

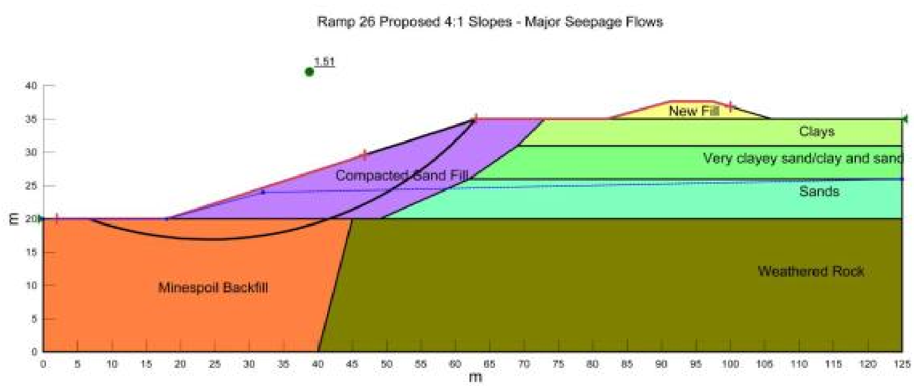


Figure 7-5 Stability Ground Model for B Pit – Ramp 26

Table 7-3 Adopted Soil Parameters

Strata	Typical Depth Range (m)	Cohesion, c (kPa)	Internal Friction Angle, φ (°)	Density (kN/m)
Upper fine grained soils	Surface to 4	5	28	17
Very clayey sand	4 to 8-9	3	30	17
Silty to clayey sand	8-9 to 11-14	0	34	17
Weathered siltstone and sandstone	Below 11-14	100	38	20
Compacted sand fill	-	0	33	20
Mine overburden spoil	Fill on top Permian	0	31	17

Figure 4-29: Stability analysis - B-Pit levee in vicinity of endwall (KBR Report BEE603-W-REP-004 Rev 2)

In summary, the proposed stable levee / pit section in the KBR report included a 20 m wide bench adjacent to the toe of the proposed levee, partial infill of the ramp/endwall to the top of Permian, and provision of a compacted, free draining sand fill zone battering down at 1V:4H to the top of minespoil fill. Sand batters would be topsoiled and grassed for erosion protection.

Sections taken through the current levee / pit wall along the B-pit endwall, show minimum off-set distances between the pit and levee toe in the section from Ch 2600 and 3000 (Appendix A), of between 60 and 20m respectively, which is near borehole location DP4 shown in Figure 4-30 below. In-situ slopes into the pits generally exceed the 1V:4H incorporated in proposed KBR stable section. These sections representing the current endwall status are similar to the excavation profile investigated in the KBR report prior to stabilisation mitigation. Previous geotechnical investigation work in this area included drilled boreholes at the locations shown below (DP references), field permeability testing at borehole locations, particle size distribution testing and pinhole dispersion classification from materials extracted from boreholes, cone penetrometer testing, and electromagnetic transects. In general, the section modelled by KBR is representative of conditions identified in borehole DP4, and hence the subsurface conditions in the vicinity of Ch 2600 to 3000, although test results did indicate subsurface variability along the levee length.

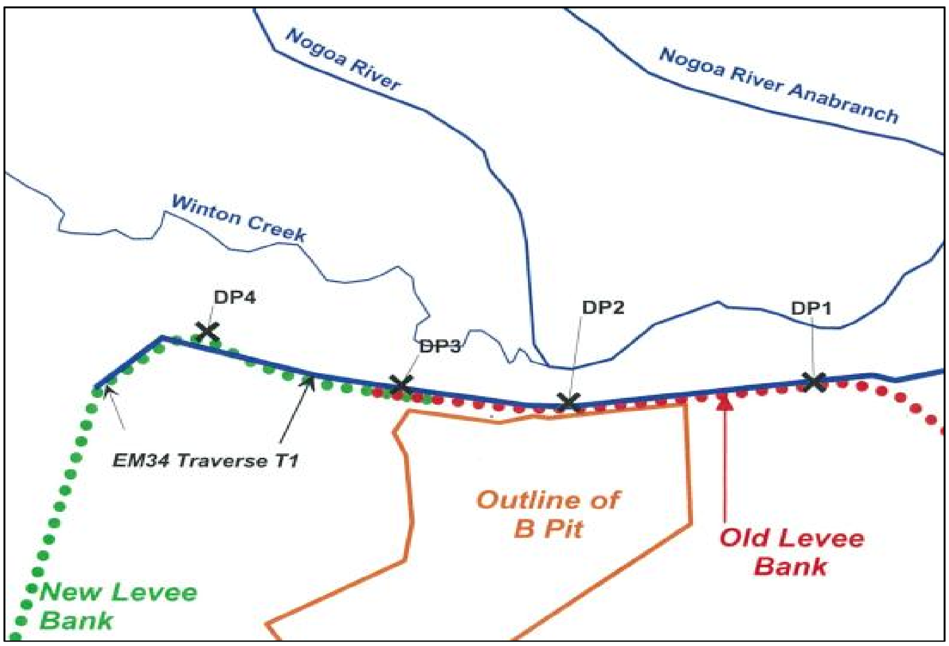


Figure 4-30: Location of investigative boreholes (Douglas Partners - Project 47793.01- Sept 2009)

An alternative subsurface section was modelled to that extracted from the KBR report, with a single upper sand layer representing the layers of silty and clayey material found in the DP4 profile, with a permeability of $8.5x10^{-5}$ m/sec. This permeability is lower than the sandy material in the KBR section ($2x10^{-3}$m/sec to $5x10^{-4}$m/ sec), but higher than clay. This composite layer hence represents potential variability of materials in this upper layer, recognising that the sandy materials in the upper subsurface layers contain a high proportion of fines, and are hence less permeable than sand but more permeable than clay. The section adopted for this analysis is shown in Figure 4-31 below.

The analysis indicates a FOS of 0.97, with failure occurring in the upper sandy layer with fines. The section assumes a 1V:4H regrade of the endwall into the pit, which has been adopted for Option 2. This slope would be dressed with rock cover to a specified depth, which has not been included in the model, and which would provide some additional resistance to failure. This result demonstrates a sensitivity in stability based on the how the upper clay, sandy, and silty layers are interpreted and modelled, and how effectively the clay upper surface layer seals off the flood plain from interaction with sandier lower layers. An alternative was analysed where a slurry wall cut-off trench is incorporated in the design using the same subsurface layer interpretation. This trench was taken through the upper sand layers into less permeable strata below, and resulted in a FOS of 2.36 (Figure 4-32).

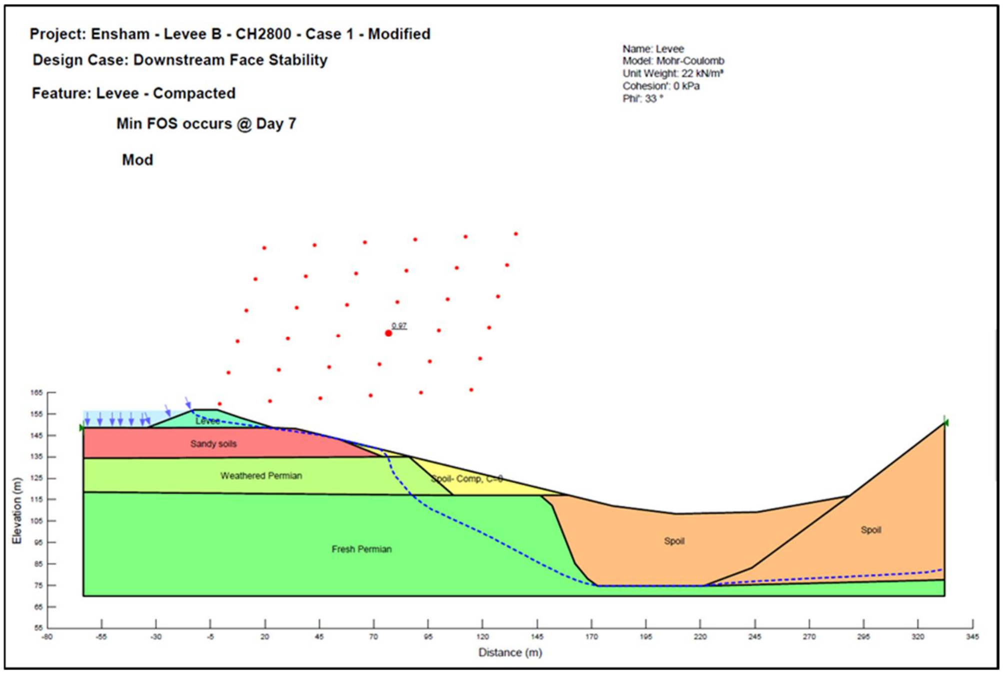


Figure 4-31: SLOPE/W results – Option 2 - B-Pit levee with 25% (1H:4V) regrade into void (alternate subsurface profile sensitivity analysis

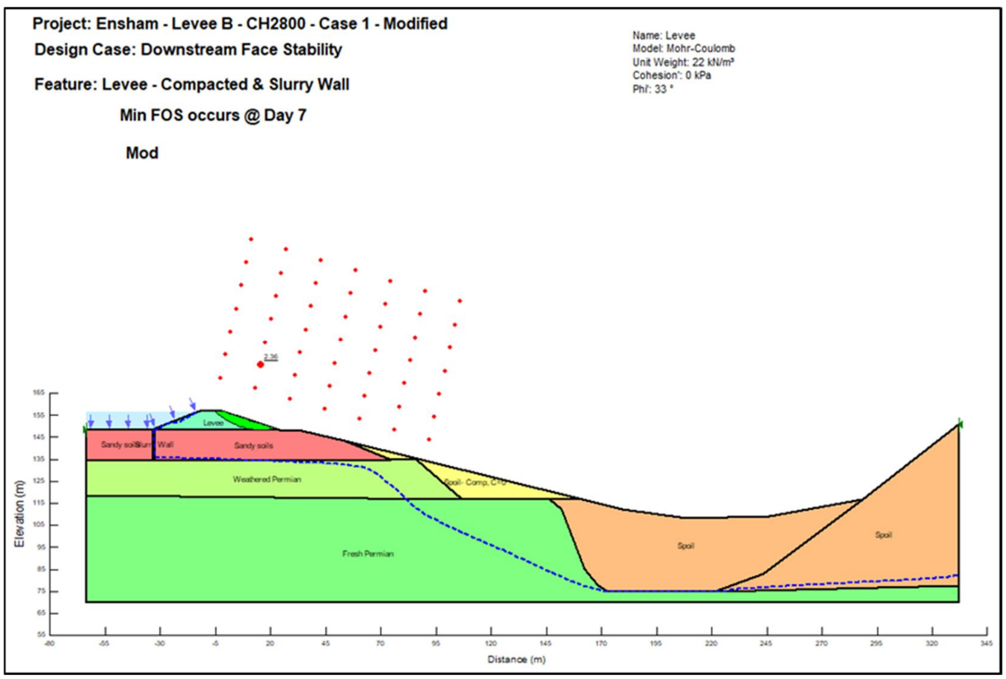


Figure 4-32: SLOPE/W results - Option 2 - B-pit levee with slurry trench - FOS 2.36

Based on these findings, the following conclusions can be drawn:

— if subsurface conditions in the vicinity of Ch 2600 to 3000 are as represented in the KBR modelling, then the Option 2 landform including the levee and pit endwall will be stable in this area and meet a FOS of 1.5

— due to variability in geotechnical subsurface conditions in this area from previous investigations, it is recommended that if Option 2 is considered favourably, a targeted geotechnical investigation should be undertaken to confirm conditions modelled in the KBR report

- if further investigation identifies conditions that are somewhat different from those modelled in the KBR report, with sandier layers identified higher in the subsurface profile, a viable solution would be to provide a limited length of slurry cut-off trench to intercept the sandy layers and extend to a suitable low permeability materials resulting in satisfactory factors of safety, or flatten the embankment slopes into the void to achieve a more stable profile
- due to these geotechnical uncertainties and marginally acceptable factors of safety even with favourable conditions (1.51 as opposed to the required 1.5), it is considered prudent at this feasibility stage of the project to assume that a slurry cut-off trench wall will be required for a length of approximately 300 m to ensure a stable final Option 2 landform for this area, which will mitigate seepage from passing through the sandy layers.

4.5.3.3 C PIT CRITICAL SECTION (CH 3400 AREA)

The Option 1 levee alignment is concurrent with the Option 2 alignment for this critical section where the alignment is closest to the void edge. The Option 2 landform is consistent with Option 1 in this area, with both having the same high wall regrade treatment for final landform stability. This results in a levee / final highwall profile that is in all respects identical other than the outward facing slope of the levee. This difference will be negligible in terms of seepage and slope stability assessment, hence the results reported in Section 4.5.2.3 will be equally applicable to Option 2, and no further assessment has been undertaken.

4.5.4 SCOUR PROTECTION OF LEVEES

Velocity profiles along the levee faces for a 0.1% AEP event indicate that they are likely to be exposed to velocities of up to 1.5 m/s (Figure 4-33), which would not normally be considered high enough to warrant the use of rip rap protection. For Option 1, where no inspection or maintenance of the levees will be undertaken, it would be prudent to line the water facing embankment at sharp changes in alignment where turbulence could occur with hard, durable rock rip rap. These areas would be at Ch 6000 and 2200 for the B Pit levee and Ch 700 for the C Pit levee (see Appendix A for chainage locations).

The information provided below is provisional, and the modelling grid is 30 m, which may not be sufficiently accurate along the levee faces to properly predict velocity magnitudes, and it is recommended that the requirement for levee facing be revisited during the detailed design phase of the project.

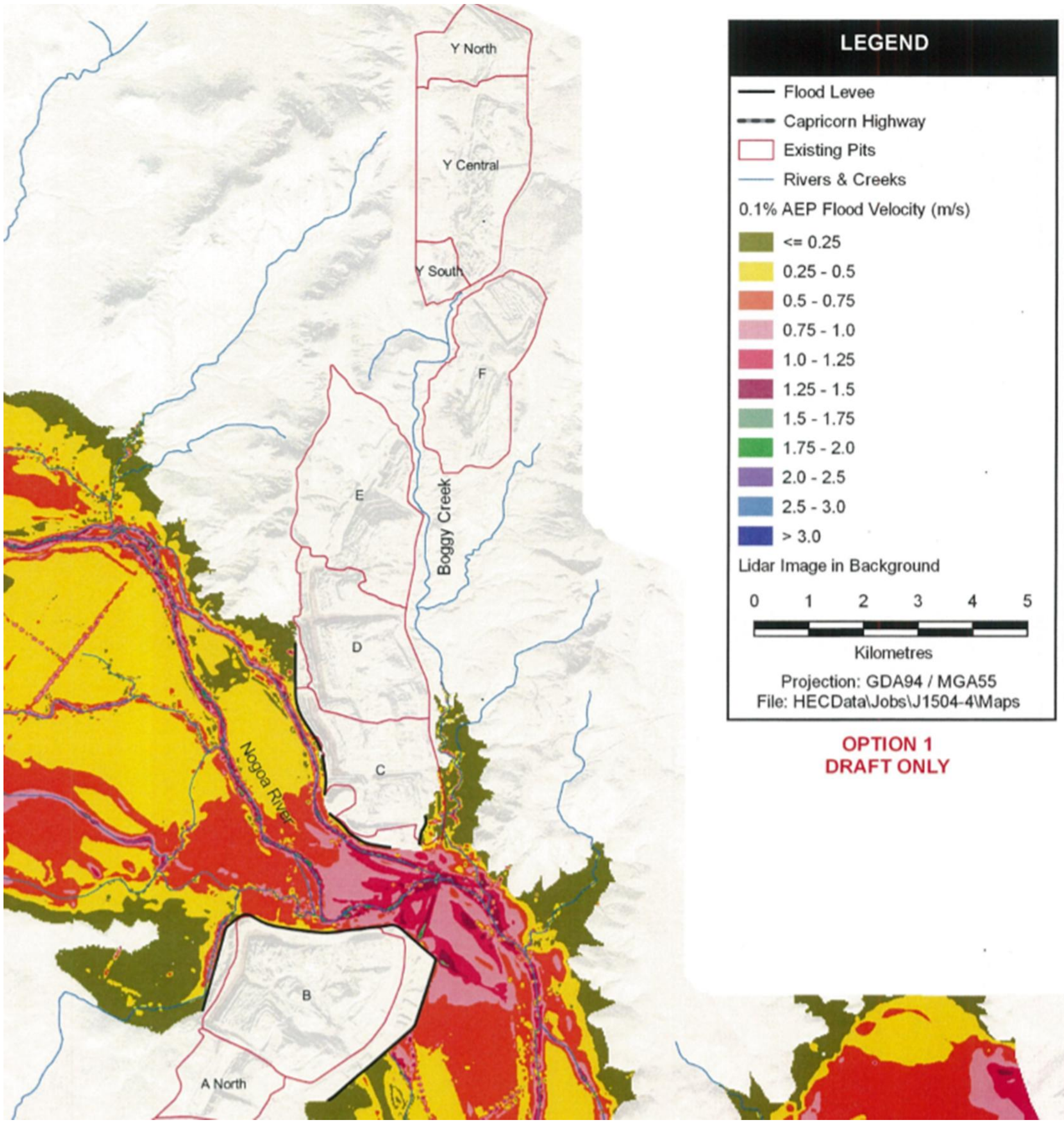


Figure 4-33: Option 1 - 0.1% AEP velocity profile

4.6 PIT BACKFILLING CHARACTERISTICS

Option 3 requires the backfilling of voids that lie within the extent of the PMF event to approximate pre-mining ground level. This requires the backfilling or partial backfilling of the A, B, C and D pits with spoil from adjacent spoil dumps. Once placed the re-created landform will be subject to ongoing settlement for many years after placement has been completed, which may result in surface irregularity regarding desired final landform, impeded drainage and surface soil waterlogging issues associated with depressions. Soil piping may occur as the backfill settles. However, its occurrence should reduce over time due to:

— backfill densification due to material breakdown, piped and / or broken down material filling voids between boulders, and compression due to the overlying spoil load

— backfill groundwater level reaching equilibrium, thus reducing the potential flow path.

In addition, the backfill is contained within the surrounding materials in the sides of the pit, negating movement of soil particles away from the pit. Wetting of the backfill after achieving grade during construction may help to reduce long-term settlement further by speeding up the breakdown process.

A further issue associated with backfilling within a flood plain is the exposure of the backfilled areas to flood waters, with associated potential for erosion and submergence by flood waters.

4.6.1 SETTLEMENT CHARACTERISTICS

A preliminary assessment of settlement has been undertaken based on studies undertaken by Charles and Burford (1987) and Hills and Denby (2002), regarding settlement in open cut mine backfills in the UK, which were placed without systematic compaction. The studies identified the mechanisms of collapse due to re-establishment of groundwater levels and long-term creep settlement occurring in these fills, and identified several treatments, including tolerating settlement without treatment. The reported settlement measurements indicated that most of collapse settlement would be completed within 3 or 4 years following the commencement of inundation of the fill, while creep settlement generally followed a log-time relationship. In the case of a landform in remote, agricultural settings, large settlements can be accommodated; however, it is recommended that deep fills be overfilled to help compensate for settlement over time and by so doing, reduce the potential for large depressions forming.

The above references are believed to be relevant to this project, as coal measures and overburden materials both in the UK and on this project area consist of similar materials including mudstone, sandstone and coal. However, it is noted that the settlements also depend on other factors such as mining methods, material type and size, durability, compaction conditions, and groundwater conditions.

The creep settlement (S_{cl}) at any time (t_2) is predicted using equation:

$$S_{cl} = H.\ \alpha(\log t_2 - \log t_1)/100, \text{ and}$$

$\alpha=0.2$, when the spoil is systematically compacted
$\alpha=0.4$, when the spoil is partly/poorly compacted
$\alpha=0.8$, when the spoil is uncompacted

Where H is the total original spoil depth, α is creep settlement rate, t_1 is the time when creep starts.

The collapse settlement (S_{co}) is estimated as:

$$Sco = \beta H', \text{ and}$$

$\beta=0.4\%$, when the spoil is systematically compacted
$\beta=1.0\%$, when the spoil is partly/poorly compacted
$\beta=2.5\%$, when the spoil is uncompacted

Where β is the ratio of settlement to original spoil thickness, and H' is the inundated depth of the spoil/fill.

An assessment has been made of the likely backfill settlement that could occur in the footprint of the backfilled pits, and results are included in the Table 4-11.

Table 4-11: Estimated backfill settlement

PIT	FINAL GROUNDWATER LEVEL (m AHD)*	ROCK FLOOR LEVEL (m)	TOP OF FILL LEVEL (m AHD)	FILL HEIGHT H (m)	INUNDATION DEPTH -H' (m)	LONG-TERM SETTLEMENT (m)
A (south)	147.2	110	160	50	37	0.85
B	146.6	60	158	98	87	1.81
C	147.8	60	154	94	88	1.78
D	141.4	60	158	98	81	1.75

* Groundwater recovery levels provided by Ensham on 12 November 2018 (Option 3 levels).

The following assumptions are made for the estimation of the total settlement:

— the approximate depth of void, fill height, and inundation height are as tabulated above, extracted from LIDAR survey and a survey provided by Ensham of seam levels in the pits
— backfill will be by way of truck and shovel resulting in partly / poorly compacted fill hence the factors α=0.4; and β=1.0% have been utilised
— the long-term settlement due to creep is the settlement at 500 years after void filling, and it has been assumed that this settlement commences immediately after completion of void filling, which has been assumed to take 2 years to complete.

Based on this assessment, a maximum settlement of approximately 2 m is anticipated for the deepest sections of the voids, approximately half of which is collapse settlement which should occur once groundwater levels in the pits have re-established. Due to the nature of spoil settlement mechanisms, an accurate prediction is not possible, and it is likely to vary significantly spatially. In addition, significant volumes of fill will be required to re-create the landforms, placement of which is likely to take place over the period of a year, meaning some of the early settlement will be taken up by progressive placement of overlying fill material. To mitigate the potential issues associated with differential settlement, the following remedies may be considered:

— compact the backfill systematically, which would not eliminate settlement but would reduce the predicted magnitude to approximately half of that shown in Table 4-11
— overfill the voids to create a surcharged landform that should settle back to the desired levels over time – this could be a partial or full overfilling, contoured to reflect the depth of fill below the surcharge. Note the surcharge may need to be regraded over time
— design the landform to incorporate slopes that reduce the formation of localised depressions, allowing the surface water to drain should they occur
— retention of a stockpile of soil material to fill in depressions as they occur over time.

Due to impracticalities in systematic compaction of fills of this magnitude, and difficulties in predicting the location and extent of differential settlement in backfill areas, a combination of the third and fourth remedial measures described above is likely to provide a practical, cost-effective solution.

4.6.2 INUNDATION OF SPOIL AND EROSION POTENTIAL

The re-created landform within the PMF extent will be exposed to flood waters during flood events, resulting in flow across, and inundation of the backfilled areas. The implications of this exposure in qualitative terms may be described as follows:

— spoil backfill will be relatively porous initially, but should become less so over time as the material compacts due to settlement and breaks down
— saturation of the spoil will contribute to breakdown or collapse settlement in the backfill, although only to a limited extent once the water-levels in this area become re-established, and for smaller magnitude flood events it is unlikely that significant penetration of water will occur into the spoil due to the depth of flow and duration of exposure
— saturated spoil backfill material will lose trafficability when saturated, but should dry out after the flood event and regain trafficability
— flow over unvegetated areas of backfilled spoil will result in erosion and scouring, and these surfaces will require replacement of topsoil, grass, and fertilisation, so that a good cover of vegetation becomes established to bind the topsoil in the root zone and provide appropriate shear resistance to water flow

- to avoid exposure of unvegetated backfill to flood waters, it is recommended that the existing levees be retained during backfilling of voids and rehabilitation operations, and that they only be removed when a sufficient cover of vegetation has established to protect the backfill materials
- removal of levees and haul roads allowing runoff to flow across the backfilled spoil, which may result in erosional instability during flood events.

5 CONCLUSIONS AND RECOMMENDATIONS FOR NEXT STAGE

5.1 PIT HIGHWALLS AND ENDWALLS

Based on the methodology described above for stability analysis of the pit highwalls and endwalls, the following conclusions have been reached:

— Tertiary clay soils overlying Permian materials at any of the highwall areas are expected to have an overall FOS of <1.5 and would not be considered long-term stable. Regrading of these materials to at least 1V:3H is expected to improve the stability of the materials to an acceptable standard. Surface water should be managed as mentioned above.

— Weathered Permian and fresh Permian materials cut at the batter angles typically encountered in A to F-pits have an overall FOS of >1.5 and would be considered long-term stable in their current configuration based on limit equilibrium analyses. Instability could develop around exposed faults or structures with unfavourable orientations, or where surface water intersects the cuts, reduction in rock strength due to increased moisture content, or where low durability materials are present. These aspects were not considered due to lack of detailed information. Regrading of these materials will help incorporate these aspects. It is suggested that the fresh Permian be regraded to 1V:1H and weathered Permian to 1V:2H where possible.

— Due to access constraints as well as a lack of available space at the top of the highwalls, regrading of the fresh Permian materials exposed in the highwalls and endwalls to an overall slope of 1V:1H may not be feasible. Consideration should be given to the following:

  — Surface water should be managed so the flows are directed away from the pits walls, or to engineered spillway structures constructed at low points on the pit walls to control the flow of water into the pits and reduce the risk of erosion and destabilisation of the exposed materials.

  — Placement of spoil material battered against the base of the highwall would reduce the height of the exposed highwall and improve the overall stability.

  — Restricting public access to base of pit highwalls where block falls and scree build-up would be expected to develop with time.

  — Depth of water expected in the pits over the long-term may submerge these materials, therefore removing the risk of public access to the base of the pits.

— Weathered Permian materials cut at steeper angles as seen in Y-pit have an overall FOS of <1.5 and would not be considered long-term stable. Regrading of these materials where required to at least 1V:2H is expected to improve the stability of the materials to an acceptable standard. Where regrading of the weathered Permian materials is not feasible, as discussed above the following should be considered:

  — Restricting public access to top of pit highwalls where steep slopes could be encountered.

  — Surface water should be managed as mentioned above.

It is recommended that the following issues be addressed in the next stage of the project:

- Review of the final water levels in the pits once pit hydrology studies have been completed, to assess the impact of changed water levels on final stability of proposed rehabilitation profiles.

5.2 SPOIL PILES

Based on the methodology described above for stability analysis of the spoil piles, the following conclusions have been reached:

- The spoil piles at Ensham Mine consist mainly of dragline dumped spoil at the angle of repose on the pit low walls. The spoil material generally consists of:
  - Tertiary Clays – The material typically dominates the pre-strip horizon and consists mainly of yellow/brown silt and clay.
  - Weathered/Fresh Permian Rock - The material is typically grey in colour and contains less clay than the Tertiary materials.
- SLOPE/W modelling of the spoil piles using limit equilibrium analyses in their current configuration produced a FOS that varied from 1.05 to 1.26 depending on the degree of saturation of the spoil at the bottom of the pile. These FOS values indicate that the majority spoil piles would not be considered long-term stable. This is supported by evidence of slips observed during the January 2018 site inspection.
- Rehabilitated areas observed during the site inspection typically had slopes of 1V:10H (10%) for Tertiary materials and 1V:7H (15%) for Permian materials. These areas appeared generally stable and supported vegetation growth. Stability analyses indicate that a slope of 1V:4H or flatter should be stable. It is recommended that spoil areas are regraded to match these slopes to achieve a landform that would be considered long-term stable.
- Movement of some of this material into the pit voids could assist with improving the safety and stability of high wall areas and reduce the quantity of earthworks required to regrade the highwall areas and improve spoil pile stability.
- Regrading of the spoil piles should consider the potentially erosive nature of the spoil materials and provide adequate drainage to control water flow across the face of the regraded slopes. A concave slope may help reduce erosion.
- The regraded spoil may settle due to changed loading and/or degradation of the material, possibly leading to ponding and erosion. This issue should be addressed in final design.
- The potential for spontaneous combustion and acidic runoff needs to be assessed and special treatment may be required for some materials.

It is recommended that the following issue be addressed in the next stage of the project:

- Review of the geological characteristics of the floor rock of the pits to confirm parameters used in the modelling to ensure that weak floor layers, if present, are incorporated in the stability assessment.
- Review of the final water levels in the pits once pit hydrology studies have been completed, to assess the impact of changed water levels on final stability of proposed rehabilitation profiles.

5.3 EXISTING LEVEES

Based on an evaluation of available information, the following conclusions have been reached regarding incorporation of these levees in final landforms for Options 1 and 2:

- Two levees have been constructed to protect several of the open cut pits from flooding from the Nogoa River and the Anabranch River. The northern levee protects C and D-pits and the southern levee protects B-pit.

- To accommodate planned extension of the open cut pits and following breaches in the levees following flooding in 2008, the levees were upgraded for a 0.1% AEP event. The levees were designed to have between 1.2m to 1.4m freeboard above this event.
- Design drawings or as-built drawings were available and have been utilised to assess the condition of the current levees for incorporation into Options 1 and 2 as permanent features.
- For the levees to be considered long-term stable for inclusion as landforms in Options 1 and 2 a minimum FOS of 1.5 should be maintained. The requirements to sustain the long-term stability of the levees was also discussed by DP, 2009.
- To assess if the levees are long-term stable structures for Options 1 and 2, or if further work is required to upgrade the levees so they can remain as permanent structures, the following should be undertaken:
  - detailed inspection and mapping of the levee banks (to check for cracking, erosion, vegetation cover etc.)
  - further site investigation to assess construction materials, construction methods and condition of the levees and seepage if needed.
- Regular inspection and ongoing maintenance of the levees for Option 2 would be required to rectify and issues which could lead to instability in the long-term such as:
  - erosion, cracking, settlement
  - seepage or piping
  - scour from high water flow periods.

It is recommended that the following issue be addressed in the next stage of the project:

Perform a geotechnical investigation in critical areas assess the as-constructed engineering properties of the structures, identify more accurately, the characteristics of the soils underlying the levees, and perform analyses if needed. These works should be scheduled prior to construction of the preferred option commences, to support detailed design of infrastructure.

5.4 PROPOSED LEVEE CONFIGURATION

In general, the majority of the alignment length of the Option 1 and Option 2 levees are in excess of the 150 m set-off distance from pit edges as recommended in previous studies for them to be considered long term stable, with highly unlikely or unlikely risk of failure of adjacent highwalls through piping failure during a 0.1% AEP flood event. Critical locations were assessed, considering final proposed landforms, where the levee alignments are close to final void highwall regrade extents, with the following findings:

- due to substantial infilling of the B Pit void at the northern endwall and adjacent ramp into the void for Option 1, the stability and seepage characteristics of the endwall / levee structure will be significantly improved
- for Option 2 infilling of the void adjacent to the northern endwall is less substantial than for Option 1, resulting in 1V:4H slopes into the void, which may, depending on geotechnical subsurface conditions, result in unstable long term conditions, which would require stabilisation works to achieve desired factors of safety
- a critical section along the C Pit levee was assessed by way of seepage and stability analysis to confirm that the overall highwall regrade / adjacent levee structure meets required stability requirements, and it was found that stability and seepage requirements would be met by the final landform and levee for both Options 1 and 2
- all levee structures for Options 1 and 2 assessed meet the seepage and stability requirements for long term stability, subject to the recommendations described below.

Seepage and stability analysis undertaken for the levees has been based on previous geotechnical investigations undertaken to support the design of B Pit and D Pit Levee Extensions performed by KBR. For the C / Pit area the subsurface ground conditions were inferred from CPT testing, it is therefore recommended that:

— further geotechnical investigation in critical areas be undertaken to support a final review of the existing and proposed levees for both Options 1 and 2, in order to accurately identify the depth and extent of sandy alluvial underlying materials that may provide a seepage path to the voids during flood events
— further geotechnical assessment based on the findings of the targeted geotechnical investigations
— provision of a slurry cut-off trench for the critical section of the B-pit levee for Option 2 if assessments indicate factors of safety of less than 1.5, or unacceptably high levels of seepage into the adjacent void
— targeted investigation of existing levee embankment construction materials and extent of cut off trenching to confirm permeability parameters assigned to them
— review of the 0.1% AEP hydrograph in the vicinity of critical levee / highwall locations to confirm the transient parameters utilised in the current seepage modelling.

These works should be scheduled prior to construction of the preferred option commences, to support detailed design of infrastructure.

5.5 POTENTIAL IMPACTS ON ENVIRONMENTAL VALUES

An assessment of the potential impacts of each of the three preferred options on relevant environmental values (EVs) has been undertaken based on the outcomes of this study. The EVs are described in the Ensham Residual Void Project Stage 2 Environmental Values Workshop Report (March, 2018) and the EVs relevant to each of the Stage 3 studies have been determined by Ensham Resources. The assessment of impacts on EVs has been undertaken by assigning a ranking for each of the preferred options to each EV. The adopted scoring criteria, definitions of impacts and summary of the EV assessment are tabulated below.

Table 5-1: EV scoring criteria

SCORING CRITERIA	RANKING
Significant negative impact for this criterion	-3
Medium negative impact for this criterion	-2
Minor negative impact for this criterion	-1
No impact for this criterion	0
Minor benefit for this criterion	1
Medium benefit for this criterion	2
Significant benefit for this criterion	3

Table 5-2: EV definitions

IMPACT	DEFINITION
Significant impact/benefit	Results in a change which is important, notable or of consequence to the EV having regard to its intensity/frequency. For an impact the change will result in not being able to meet published standards (if there are any). For a benefit the change should meet best practice standards (if there are any published).
Medium impact/benefit	Results in a change which is potentially important, notable or of consequence to the EV having regard to its intensity/frequency. For an impact, the change will result in occasions where the criterion will not meet published standards (if there are any). For a benefit the change should meet good practice standards (if there are any published).
Minor impact/benefit	Results in a change which is identifiable but is not important, notable or of consequence to the EV having regard to its intensity.
Note: The definition of significant impact has been based on the Federal Government's definition of significant impact contained within its "Matters of National Environmental Significance, Significant Impact Guidelines 1.1, Environment Protection and Biodiversity Conservation Act 1999"	

The EV of geotechnical aspects of the project, was assessed by considering landform stability in terms of slope stability and erosion (included in Appendix C). In terms of slope stability, all options will result in factors of safety in excess of best practice standards, which represents a significantly improvement over current mine affected conditions, and hence all options have been ranked +3 for this criterion.

There are no published best practice standards for erosion, and practices vary from site to site. All options considered have final landforms with considerably improved batter slopes for highwall, endwall and spoil in terms of erosion potential, compared with existing mine affected landform. Options 1 and 3 follow tested site best practice in terms of rehabilitation strategy for inward facing spoil regrade at 10 - 15 %, while Option 2 adopts a maximum of 25 % for inward facing spoil with rock mulching to limit erosion. This is not a site best practice and would need to be tested with trial areas to prove acceptable performance over time. It is therefore likely that Option 2 provides a lesser benefit in terms of erosion potential than for Options 1 and 3 in terms of inward facing spoil regrade. It should be noted that for Option 2, the A, B, C and D-pits will ultimately be inundated, eliminating the potential for erosion of all void surfaces below the inundation level, other than in the beach zone. Option 3, although resulting in less residual voids than the other options (and hence potential for void erosion) and having lower inward facing spoil slopes, exposes uncompacted backfill in PMF areas to flood waters, which may cause loss of topsoil and erosion, resulting in beneficial outcomes slightly lower Option 1. For these reasons Option 1 has been ranked at +3 in terms of the erosion criterion, and Options 2 and 3 have been ranked +2.

6 LIMITATIONS

This report should be read in conjunction with the Limitations for Geotechnical Investigations attached in Appendix B.

7 REFERENCES AND ABBREVIATIONS

7.1 REFERENCES

The following references have been used in compilation of this report:

- Williams, D.J; Zou, J.Z and Graham, J. 1991. Reliability Index Versus Safety Factor for Coal Mine Spoil Pile Stability., Slope Stability Engineering: Developments and Applications, *Proceedings of the International Conference on Slope Stability Organised by the Institution of Civil Engineers and held on the Isle of Wight, 1991*.
- Simmons, J.V, 1995. Slope Stability for Open Cut Pit Coal Mining. *Australian Geomechanics*.
- Look, B. 2007. Handbook of Geotechnical Investigation and Design Tables.
- Kellogg Brown and Root, 2009 report BEE603.009-C-REP-004 Rev A.
- Kellogg Brown and Root. (21 February 2014). Ensham Mine Flood Levees – Proposed Levees Design Report - BEE603-W-REP-005 Rev 2
- Kellogg Brown and Root. (21 February 2014). Ensham Mine Flood Levees – Design Report for the Proposed C Pit Levee and Upgrade of Existing B Pit Levee - BEE603-009-C-REP-004 Rev 3
- Kellogg Brown and Root. (21 February 2014). Ensham Mine Flood Levees – Design Report for Preferential Overtopping Points - BEE603-012-C-REP-001 Rev 2
- Department of Mines and Energy Guideline "Open Pit Rehabilitation – 1995".
- ACARP, 2006. Rehabilitation of Highwalls.
- ACARP, 2004. Rehabilitation of Dispersive Tertiary Spoil in the Bowen Basin.
- PSM, 2015. PSM687-222L Geotechnical Review of Final Landform.
- PSM, 2007. PSM687-L22 Highwall Risk Assessment.
- PSM, 2005. PSM687-R3 Geotechnical Review Central Project.
- Douglas Partners, 2009 report (47793.doc).
- Hoek, E and Bray, J.W, 1981. Rock Slope Engineering, 3rd Edition
- Australian Groundwater and Environmental Consultants, 2015 (AGE report: G1787_EnshamVoid)
- Seedsman Geotechnics, 2003. Engineering Geology of Ensham Central Area.
- WSP, 2018. PS107225-RES-REP-001 Rev2 Geo Design.
- WSP,2018. PS107225-RES-REP-002 Rev2 – Stage 2 Landform Design Report.
- WSP,2018. PS107225-RES-REP-006 Rev D - Stage 3 Landform Design Report
- Simmons, J.V. and Simpson, P.J, 2006. Composite Failure Mechanisms in Coal Measures Rock Masses – Myths and Reality. The South African Institute of Mining and Metallurgy.
- Simmons, J.V. and McManus, D.A, 2004. Shear Strength Framework for Design of Dumped Spoil Slopes for Open Pit Coal Mines.
- La Tierra Pty Limited for Ensham Resources, 20 June 2017. Rehabilitation Management Plan V2.0.
- Landloch Pty Ltd. (September 2016). Assessment of Stability of Final Void and Landform Shaping Options, Ensham Coal Mine.

— Hills, C.W.W. and Denby, B. (1996) The prediction of opencast backfill settlement. Proceedings of the Institution of Civil Engineers, Geotechnical Engineering, 119, pp. 167–176.

— Charles, J.A. and Burford, D. (1986) Settlement and groundwater in opencast mining backfills. Proceedings of 9th European Conference on soil mechanics and foundation engineering, Durham.

— Douglas Partners. (September 2009). Hydrogeological Assessment of Sub-Surface Conditions Beneath Ensham Mine B Pit Levee – Project 47793.01

7.2 ABBREVIATIONS

The following abbreviations have been used in this report:

AEP - Annual Exceedance Probability

AHD – Australian Height Datum

C – Cohesion

Ch – Chainage (reference distance along linear infrastructure)

CRG – Community Reference Group

DEHP – Department of Environment and Heritage Protection

DES – Department of Environment and Science

DNRM – Department of Natural Resources Mines and Energy

EA – Environmental Authority

EV – Environmental Value

FOS – Factor of Safety

JV – Joint Venture

LIDAR – Light Detection and Ranging (survey method)

PMF – Probable Maximum Flood

RVP – Residual Void Project

ToR – Terms of Reference

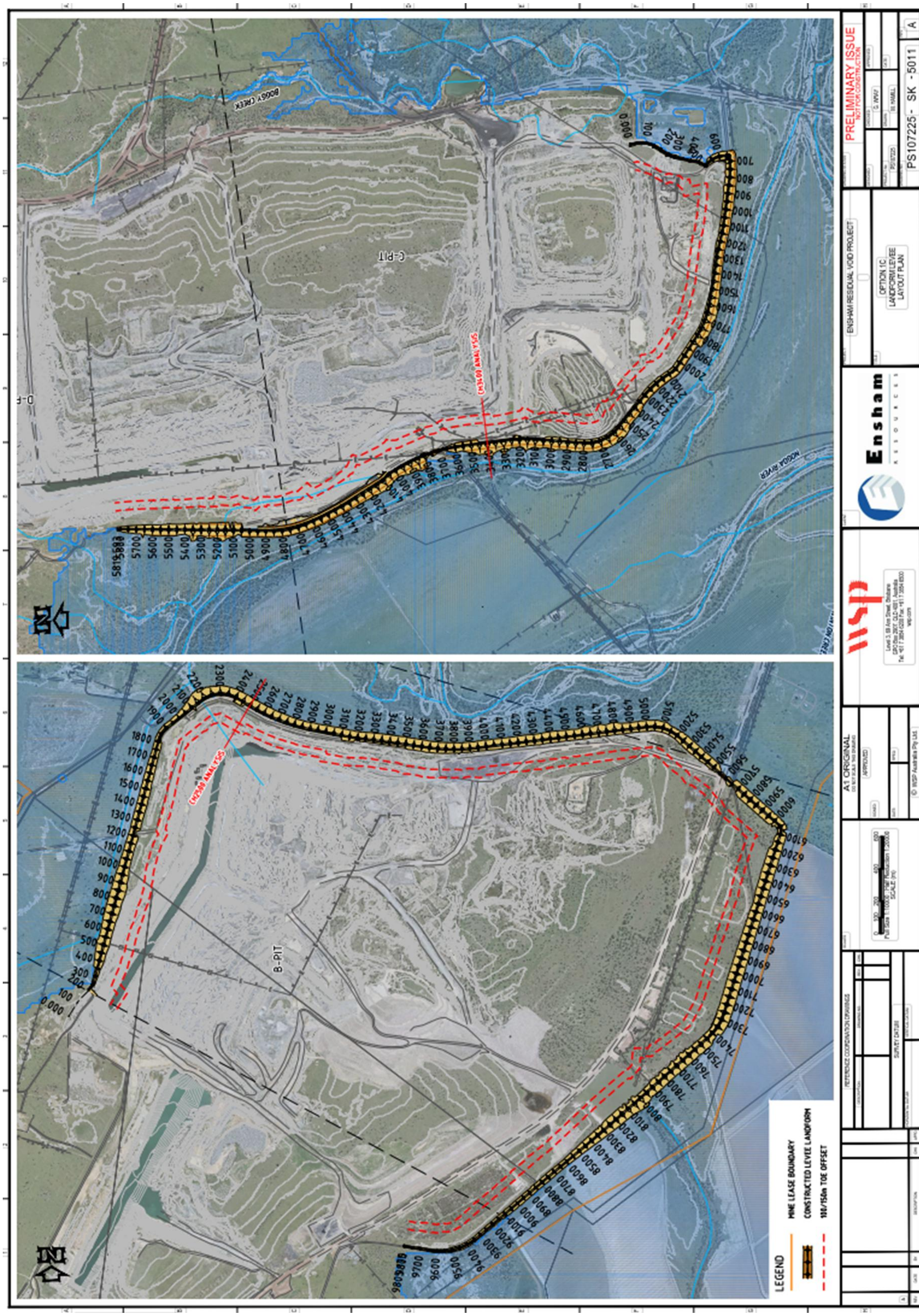
UCS – Unconfined Compressive Strength (of rock)

Ø – Angle of internal friction (of soil)

γ – Density of soil

APPENDIX A

LEVEE ALIGNMENT


PRELIMINARY ISSUE
ENSHAM RESIDUAL VOID PROJECT
OPTION 1C
LANDFORM LEVEE
LAYOUT PLAN
PS107225 - SK - 5011
A
A1 ORIGINAL
LEGEND
MINE LEASE BOUNDARY
CONSTRUCTED LEVEE LANDFORM
100/150m TOE OFFSET
B-PIT
C-PIT
NOGOA RIVER
BOGGY CREEK

APPENDIX B

LIMITATIONS

Limitations of Geotechnical Site Investigation

The limitations of site investigation

In making an assessment of a site from a limited number of boreholes or test pits there is the possibility that variations may occur between test locations. Site exploration identifies specific subsurface conditions only at those points from which samples have been taken. The risk that variations will not be detected can be reduced by increasing the frequency of test locations; however this often does not result in any overall cost savings for the project. The investigation program undertaken is a professional estimate of the scope of investigation required to provide a general profile of the subsurface conditions. The data derived from the site investigation program and subsequent laboratory testing are extrapolated across the site to form an inferred geological model and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite investigation the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies.

The borehole logs are the subjective interpretation of subsurface conditions at a particular location, made by trained personnel. The interpretation may be limited by the method of investigation, and can not always be definitive. For example, inspection of an excavation or test pit allows a greater area of the subsurface profile to be inspected than borehole investigation, however, such methods are limited by depth and site disturbance restrictions. In borehole investigation, the actual interface between materials may be more gradual or abrupt than a report indicates.

Subsurface conditions are time dependent

Subsurface conditions may be modified by changing natural forces or man-made influences. A geotechnical engineering report is based on conditions which existed at the time of subsurface exploration.

Construction operations at or adjacent to the site, and natural events such as floods, or groundwater fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept appraised of any such events, and should be consulted to determine if additional tests are necessary.

Avoid misinterpretation

A geotechnical engineer should be retained to work with other appropriate design professionals explaining relevant geotechnical findings and in reviewing the adequacy of their plans and specifications relative to geotechnical issues.

Bore/profile logs should not be separated from the engineering report

Final bore/profile logs are developed by geotechnical engineers based upon their interpretation of field logs and laboratory evaluation of field samples. Customarily, only the final bore/profile logs are included in geotechnical engineering reports. These logs should not under any circumstances be redrawn for inclusion in architectural or other design drawings. To minimise the likelihood of bore/profile log misinterpretation, contractors should be given access to the complete geotechnical engineering report prepared or authorised for their use. Providing the best available information to contractors helps prevent costly construction problems. For further information on this matter reference should be made to 'Guidelines for the Provision of Geotechnical Information in Construction Contracts' published by the Institution of Engineers Australia, National Headquarters, Canberra 1987.

Geotechnical involvement during construction

During construction, excavation is frequently undertaken which exposes the actual subsurface conditions. For this reason geotechnical consultants should be retained through the construction stage, to identify variations if they are exposed and to conduct additional tests which may be required and to deal quickly with geotechnical problems if they arise.

Legal

Limitations of Geotechnical Site Investigation

The limitations of site investigation

In making an assessment of a site from a limited number of boreholes or test pits there is the possibility that variations may occur between test locations. Site exploration identifies specific subsurface conditions only at those points from which samples have been taken. The risk that variations will not be detected can be reduced by increasing the frequency of test locations; however this often does not result in any overall cost savings for the project. The investigation program undertaken is a professional estimate of the scope of investigation required to provide a general profile of the subsurface conditions. The data derived from the site investigation program and subsequent laboratory testing are extrapolated across the site to form an inferred geological model and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite investigation the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies.

The borehole logs are the subjective interpretation of subsurface conditions at a particular location, made by trained personnel. The interpretation may be limited by the method of investigation, and can not always be definitive. For example, inspection of an excavation or test pit allows a greater area of the subsurface profile to be inspected than borehole investigation, however, such methods are limited by depth and site disturbance restrictions. In borehole investigation, the actual interface between materials may be more gradual or abrupt than a report indicates.

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APPENDIX C

EV IMPACT ASSESSMENT

Environment Aspect	Environmental Value	Criteria	Scoring Notes	Option 1 Landform Levee[1]	Option 1 Evidence (short description/reference to section in technical report)	Option 2 FMBU[1]	Option 2 Evidence (short description/reference to section in technical report)	Option 3 Backfill to PMF[1]	Option 3 Evidence (short description/reference to section in technical report)
Land	Landform stability	Slope failure	Risk of failure generated over the life of the option	3	All slopes have been regraded to achieve a FOS of >1.5. Spoil slopes are no greater 15%.	3	All slopes have been regraded to achieve a FOS of >1.5. Spoil slopes are no greater 25%.	3	All slopes have been regraded to achieve a FOS of >1.5. Spoil slopes are no greater 15% with all areas in the PMF area backfilled to pre- mining levels.
		Erosion	Erosion of landform generated over the life of the option	3	All spoil slopes < 33% will be topsoiled and or ameliorated to promote and sustain vegetation growth. Spoil slopes are no greater than 15%. Slope lengths are restricted with benches at intervals. Highwall and endwall batters flattened to retard erosion.	2	All spoil slopes < 25% will be topsoiled and or ameliorated to promote and sustain vegetation growth. All sopes that equal 25% will be rock mulched with tree seed to reduce erosion potential. Spoil slopes are no greater than 25%. Slope lengths are restricted with benches at intervals. Highwall and endwall batters flattened to retard erosion.	2	PMF spoil backfill area will be exposed to flood water, increasing potential for erosion - refer section 4.6.2 of report. Less residual voids in final landform than other options, less potential for erosion of inward facing voids embankments. Potential settlement of backfill in PMF areas over time with associated low points in landform, creating ponds that may limit or destroy revegetation in affected areas. All spoil slopes < 33% will be topsoiled and or ameliorated to promote and sustain vegetation growth. Spoil slopes are no greater than 15%. Highwall and endwall batters flattened to retard erosion.