Tarrawonga Coal Project

Environmental Assessment

APPENDIX R

CONCEPT DESIGN FOR LOW PERMEABILITY BARRIER AND PERMANENT GOONBRI CREEK ALIGNMENT

Resource Strategies

Tarrawonga Coal Pty Ltd

TARRAWONGA COAL PTY LTD

TARRAWONGA COAL PROJECT

CONCEPT DESIGN FOR LOW PERMEABILITY BARRIER AND PERMANENT GOONBRI CREEK ALIGNMENT

Client:
Tarrawonga Coal Pty Ltd

Project:
Concept Design for
Low Permeability Barrier and
Permanent Goonbri Creek
Alignment

Date:
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AWA Project No:
0353-rst-003 (r001-g)

ALLAN WATSON ASSOCIATES
Waste and Water Management Consultants

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SECTION 1.0 - INTRODUCTION

1.1 BACKGROUND AND SCOPE

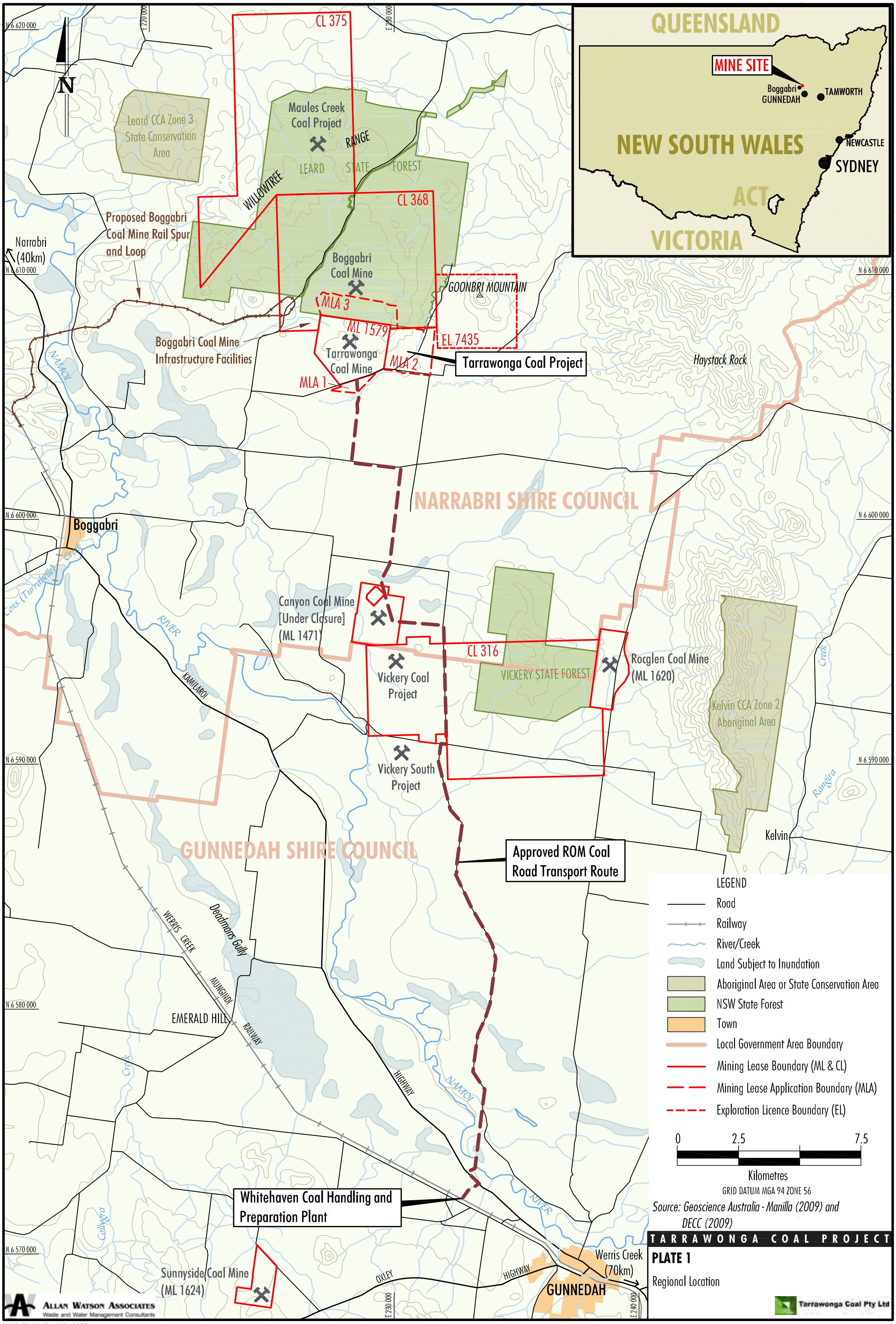
The Tarrawonga Coal Mine is located approximately 15 kilometres (km) north-east of Boggabri and 42 km north-northwest of Gunnedah in New South Wales (NSW). Site locality details are provided in **Plate 1**. The Tarrawonga Coal Mine commenced operations in 2006 and currently produces up to approximately 2 million tonnes per annum (Mtpa) run-of-mine (ROM) coal.

Tarrawonga Coal Pty Ltd (TCPL) (a joint venture between Whitehaven Coal Mining Pty Ltd [Whitehaven] [70% interest] and Boggabri Coal Pty Limited [BCPL] [a wholly owned subsidiary of Idemitsu Australia Resources Pty Ltd] [30% interest]) is proposing to expand the existing pit as part of the Tarrawonga Coal Project (the Project). The proposed development layout shows that the mine development will extend generally in an easterly direction, and will intersect Goonbri Creek and encroach onto alluvial materials and an associated alluvial groundwater system that occurs on the flatter plains to the south-east of the Project. The general layout of the current Tarrawonga Coal Mine, showing the Mining Lease (ML)1579 boundary and the proposed footprint of proposed mining to the east, is shown on **Drawing 001**.

Based on the proposed mine development layout, the Project requires that a low permeability barrier around the eastern edge of the proposed pit be installed, with a permanent easterly diversion of Goonbri Creek to be constructed. The key performance objectives of these works are as follows:

- to reduce the potential for local drainage of groundwater from the Alluvial groundwater system into the open cut mine pit during operational and post-closure periods;
- to reduce the potential for local instability of pit batters as a result of groundwater infiltration;
- to avoid and significantly reduce the risk of Goonbri Creek inflows reporting to the open cut mine pit, whilst ensuring that the hydrological character of the Goonbri Creek system is maintained in a permanent Goonbri Creek alignment and that the potential for loss of baseflow from Goonbri Creek to the mine workings (both operationally and post-closure) is reduced; and
- to reduce the potential for impacts on quality value of the regional groundwater resource.

This report provides a concept design, seepage analysis and preliminary cost estimate for the low permeability barrier and permanent Goonbri Creek alignment included as a component of the proposed Project.



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1.2 REPORT STRUCTURE

This report has been prepared to present the results of the concept design for the low permeability barrier and permanent Goonbri Creek alignment for the Project. The structure of the report to address the scope as outlined in **Section 1.1** is as follows:

Section 2.0 : Describes site conditions relevant to the preliminary engineering works carried out with respect to the low permeability barrier and proposed Goonbri Creek alignment, including a description of the Project. These conditions have been assessed based on available data sources.

Section 3.0 : Outlines preliminary seepage analysis carried out in relation to the final mining pit configuration, as a means of confirming the need for a low permeability barrier.

Section 4.0 : Presents a concept engineering assessment for the low permeability barrier, with three barrier system options considered. Preliminary capital cost estimates are also provided.

Section 5.0 : Identifies a preferred low permeability barrier option, with an overview of the application of this option provided by relevant literature sources. The preliminary engineering of the system is described, as well as relevant construction aspects. A 2 dimensional seepage model analysis including the low permeability barrier and sensitivity analysis is also presented.

Section 6.0 : Describes the relevant aspects of the permanent Goonbri Creek alignment based on hydrological assessment in relation to the concept design completed by Gilbert and Associates (2011) and preliminary engineering works as part of this study to estimate civil works quantities for the purpose of capital cost estimation.

Appendix A reproduces data provided by other sources used in this assessment.

SECTION 2.0 - RELEVANT SITE CONDITIONS

2.1 DATA SOURCES

Data utilised for the purpose of preliminary engineering for the low permeability barrier and permanent Goonbri Creek alignment is as follows:

GENERAL SITE LAYOUT DATA

- Project Application and Preliminary Environmental Assessment for the Tarrawonga Coal Project, prepared by TCPL (2011);
- Topographical data for the Project site and surrounds; and
- Pit extent at various stages of development, with year 2029 pit configuration being most relevant to this study.

GEOLOGICAL/HYDROGEOLOGICAL DATA

- Major regional geological boundaries from NSW Department of Primary Industries (geological mapping), identifying extent of alluvium subcrop.
- Excerpts from NSW Department of Natural Resources, now NSW Office of Water [NOW], Upper Namoi Groundwater Flow Model (2010).
- Summary logs for exploration boreholes drilled in a transect running west to east through the eastern pit margin, completed in May 2011.
- Technical background provided in discussions with and email advice from Andrew Fulton (hydrogeologist commissioned as part of the Environmental Assessment works and involved in exploration drilling works completed in May 2011 for RPS Aquaterra).
- Transient electromagnetic (TEM) groundwater investigation findings (Groundwater Imaging, 2011)
- Hydrogeological assessment and groundwater modelling results (Heritage Computing, 2011)

A summary of relevant information from these data sources is provided in the following sections.

2.2 PROJECT DESCRIPTION

The proposed life of the Project is 17 years, commencing 1 January 2013 and would extend the life of the current open cut mining operations at the Tarrawonga Coal Mine.

The approximate extent of the existing and approved surface development (including open cut, mine waste rock emplacement, soil stockpiles and infrastructure areas) at the Tarrawonga Coal Mine are shown on **Plate 2**. The approximate extent of the Project surface development (incorporating the existing and approved development) lies within Mining Lease Applications (MLAs) 1, 2 and 3 as well as within existing ML 1579. These mining tenure boundaries are shown on **Plate 2**.

The proposed Project involves extension of the open cut mining operations towards the east, beyond the limits of ML1579, into MLA 2. **Plate 2** also shows the current alignment of Goonbri Creek beyond the eastern edge of ML1579, and the proposed permanent Goonbri Creek alignment. The open cut will be expanded to the north into an existing mine tenement Coal Lease (CL) 368 (MLA 3).

Existing conditions within ML1579, and within the proposed Project area to the east (within MLA2) are shown in more detail on **Drawing 001**, with the development conditions proposed on completion of mining (projected to Year 2029) shown on **Drawing 002**.

A description of the Project is provided in *Section 2* in the Main Report of the EA.

2.3 TOPOGRAPHY AND DRAINAGE

The Tarrawonga Coal Mine is located at the foothills of the Willowtree Range approximately 12 km east of the Namoi River (**Plate 1; Section 1.1**). Goonbri Mountain lies approximately 4 km north-east of ML 1579, and in conjunction with the Willowtree Range, form the main topographic features to the north and east. The main local drainage systems in the vicinity of the Project area are Nagero Creek, Goonbri Creek and Bollol Creek. These creeks discharge onto the expansive alluvial flats to the south and south-east, and transition into relatively poorly defined drainage paths which become expansive ponded overland flow areas during and following heavy rainfall. The overland flow moves slowly down-gradient (west and south-west) toward the Namoi River.

Surface elevations in the region vary from approximately 260 metres (m) Australian Height Datum (AHD) on the floodplains of Bollol Creek up to 540 m AHD at the peak of Goonbri Mountain.

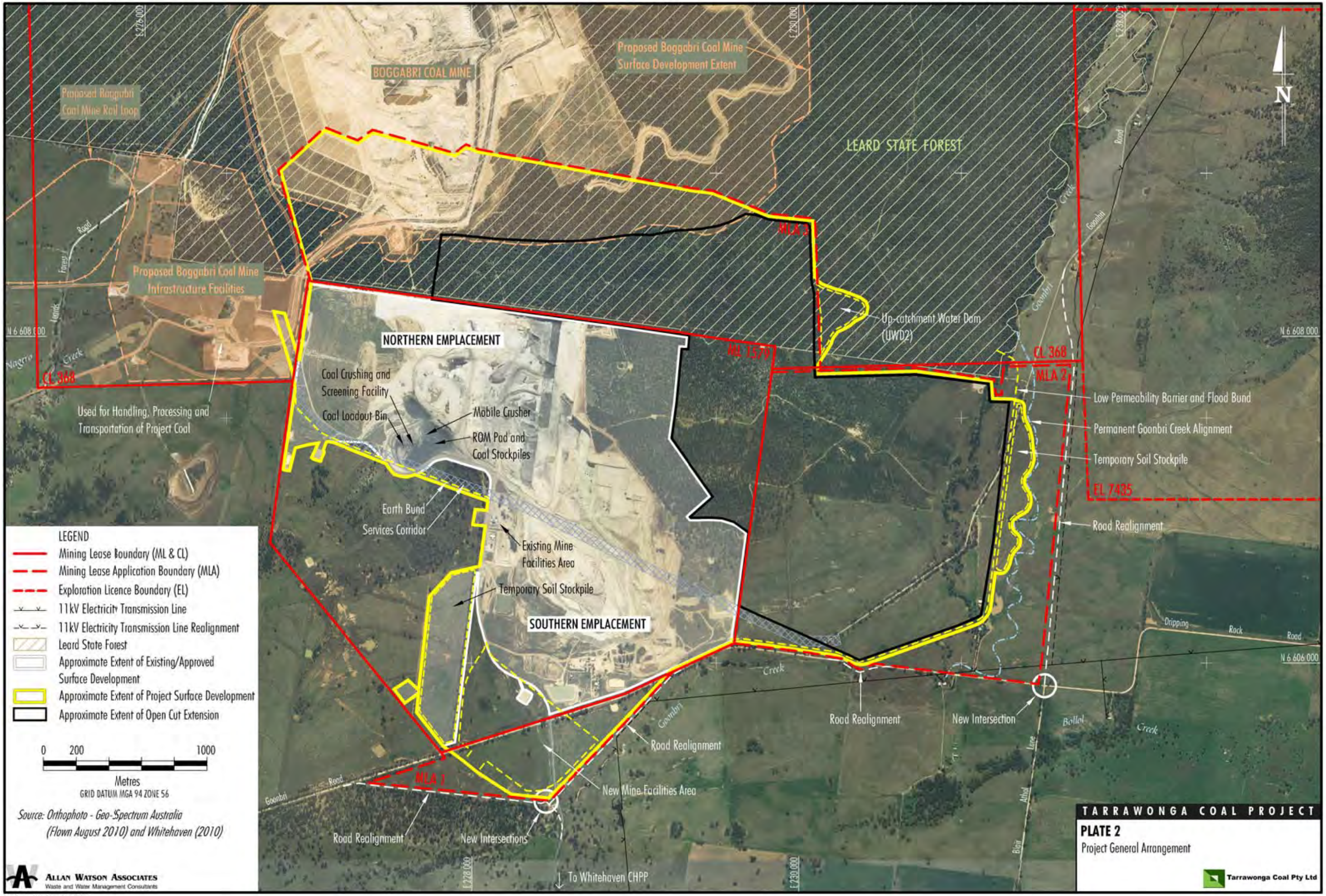

PLATE 2

Project General Arrangement

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2.4 GEOLOGICAL CONDITIONS

REGIONAL GEOLOGY

The Tarrawonga Coal Mine is located in the Gunnedah Basin, which contains sedimentary rocks, including coal measures, of Permian-Triassic age. The Gunnedah Basin forms the central part of the Permo-Triassic Sydney- Gunnedah-Bowen Basin system. A north-south-trending ridge of Early Permian volcanic rocks, known as the Boggabri Ridge, divides the Gunnedah Basin into the Maules Creek sub-basin to the east, and the Mullaley sub-basin on the western side of the Boggabri Ridge.

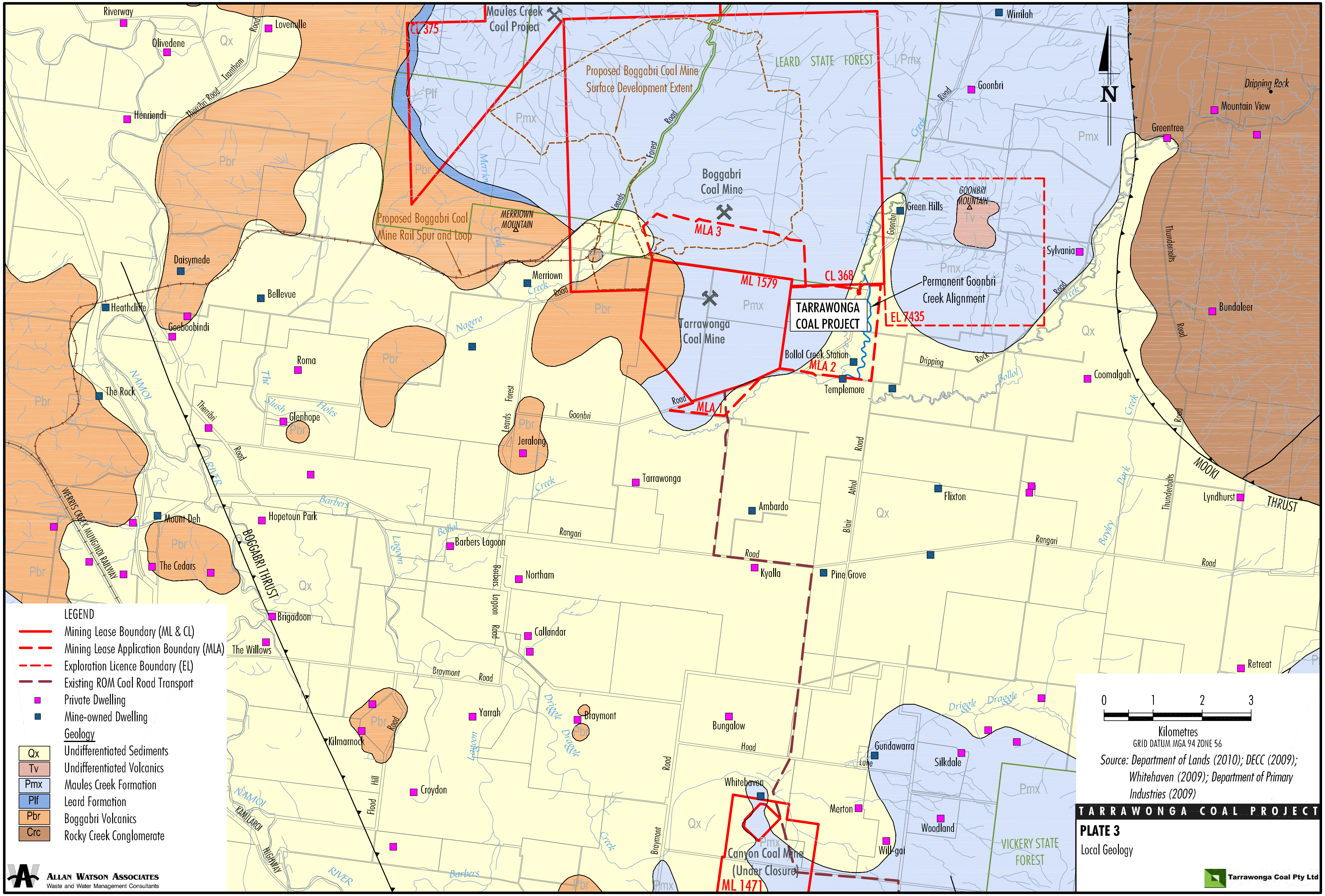
PROJECT COAL GEOLOGY

The Project site is located on the western side of the Maules Creek sub-basin, which contains the Maules Creek Formation, hosting the economic coal seams to be accessed by the project. The thickness to this formation increases from approximately 200m to in excess of 400m from west to east. The Maules Creek Formation subcrops on low hills to the west and dip towards the east. Eight coal seams are to be mined as part of the Project. Individual coal seams range up to approximately 4.5 m thick, and average 1.5 m. The coal reserve for the Project is 50.5 million tonnes of ROM coal.

ALLUVIAL GEOLOGY

The Project area is bordered by alluvial sediments which are associated locally with the Bollol Creek and Goonbri Creek drainages, and more regionally to the west with the Upper Namoi River. The Bollol Creek and Goonbri Creek embayments possess alluvial thicknesses in the order of 30 m maximum. Quaternary sediments of the Upper Namoi Valley (referred to as the Upper Namoi Alluvium) comprise the (upper) Narrabri Formation and the (lower) Gunnedah Formation. To the south of Bollol Creek, these sediments are generally 40 to 70 m thick. More broadly, the Upper Namoi Alluvium can reach maximum thicknesses of 170 m associated with the Namoi River. Separately, the Narrabri Formation has a maximum thickness of 70 m and the Gunnedah Formation peaks at 115 m (Heritage Computing, 2011). For the purpose of this assessment, the local creekline alluvium is considered to form part of the Upper Namoi Alluvium.

A regional scale geological map, covering the Project area, is provided on **Plate 3**, showing that mining tenements for the Project are located across Maules Creek Formation geology, and are overlain by undifferentiated sediments (comprising alluvial deposits from creek embayments as well as the wider Namoi Valley alluvial floodplain) in the south-eastern corner of MLA2.



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2.5 HYDROGEOLOGICAL CONDITIONS

2.5.1 GENERAL CHARACTERISATION

BASEMENT GEOLOGY

Within the Project area, basement geology (i.e. associated with the porous rock - Maules Creek Formation) generally comprises low transmissivity and low potential groundwater yield. It is considered generally that the basement would not exist as a groundwater aquifer. Notwithstanding, individual coal seams within the basement possess sufficient permeability to be regarded as aquifers but the groundwater within the seams is of low yield/sustainability and poorer quality.

QUATERNARY SEDIMENTS (UPPER NAMOI ALLUVIUM)

As outlined in **Section 2.4**, alluvial sediments of the Upper Namoi incorporate creekline alluvial deposits to the east and south of the Project site. The Upper Namoi Alluvium forms part of the Upper Namoi Alluvial Groundwater Source (specifically the Upper Namoi Zone 4 water source in the vicinity of the Project).

The Upper Namoi Alluvium comprises two formations. The uppermost Narrabri Formation consists predominantly of clays with minor sand and gravel beds. Underlying the Narrabri Formation is the Gunnedah Formation which consists predominantly of gravel and sand with minor clay beds. The lower formation is a productive groundwater aquifer accessed for water supply (predominantly irrigation) purposes.

Within the Project area, the characteristics of alluvium have been assessed by a range of data sources, with general details summarised below:

(I) NOW GROUNDWATER MODEL

The NOW groundwater flow model for the Upper Namoi (2010) indicates that the alluvium occurring to the south east of the proposed pit comprises the northern margin of the Namoi River floodplain, with these sequences forming part of the Upper Namoi Alluvium. The edge of the alluvial area coincides broadly with the interface between steeper topography extending into the Project area to the north-west and flatter topography to the south-east. Contours showing depth to the base of the alluvium have been reproduced on **Drawing 003**, which shows that the alluvium depth adjacent to the final void is of up to 40m. These contours also indicate increasing alluvium thickness to the south, away from the alluvium edge.

(II) EXPLORATION DRILLING

A transect of boreholes has been drilled by TCPL along the cross section line A-A shown on **Drawing 003**. The general area of these bores is shown on **Drawing 003**. The general area of these bores is shown with a more detailed layout plan and drilling logs for these bores, labelled TAWB17, 18, 20, 21 and 22, reproduced in **Appendix A1**.

These logs indicate that the thickness of alluvium mapped in boreholes increases from 3 m (TAWB22, western-most borehole) to 41 m (TAWB20, eastern-most borehole). A summary of logs from these boreholes is provided in **Table 1.**

Table 1 – Subsurface Conditions Encountered in Transect Boreholes

Hole	TAWB17	TAWB18	TAWB20	TAWB21	TAWB22
Unit	Depth encountered in boreholes (m bgl)				
Surface Soils & clay	0 - 3	0 - 2.5	0 - 4	0 - 2	0 - 0.5
Clayey gravels			4 - 9	2 - 17	
Clayey gravels and sands	3 - 14	2.5 - 21	9 - 31	17 - 27	0.5 - 3
Clayey gravels	14 - 21	21 - 26	31 - 41	27 - 30 (clay)	
Basement Rock	> 21	>26	>41	>30	>3

An interpretation of geological conditions along the transect, reflecting borehole logs and the summary presented in **Table 1**, is shown in **Appendix A1**, which indicates the dominance of clayey gravel and clayey sand material (conceptually representing the alluvial profile) across the defined Namoi River floodplain area. It is also noted that on the western margin of the section, where the topography rises, basement sequences (conglomerate and sandstone/siltstone) are intersected nearer to the ground surface, with the alluvial sequences reducing in thickness.

(III) TEM SURVEY

Groundwater Imaging (2011) conducted a transient electromagnetic (TEM) groundwater investigation, carried out around Goonbri Creek within the proposed Project area. The objectives of this survey were to identify groundwater flow restrictions and creek connectivity, and assist in delineating the alluvium depths/extent within this area, as support for low permeability barrier and permanent Goonbri Creek alignment design works. Output from the TEM survey is presented as electrical conductivity (or resistivity), with higher electrical conductivity representing clayey gravel aquifers (white-red) and higher resistivity representing fresh basement (green-blue) – refer **Appendix A2**.

Summary results of the TEM survey are reproduced in **Appendix A2**. These results are presented in terms of (inverted) true resistivity (ohm.metres) for eight depths ranging from 1 metre to 58 metres. Interpretation of these results, as reported in Heritage Computing (2011) indicates that for depths to 12 m, clayey near-surface conditions vertically and horizontally occur. The central-western part of the survey area is underlain by weathered rock, although these sequences have similar electrical character to alluvium, therefore no clear delineation of the weathered rock-alluvium interface exists. The Upper Namoi Zone 4 boundary in the vicinity of Goonbri Creek extends beyond the eastern limit of the TEM survey area, except for the north-eastern tip.

Vertically, there is a clear change in resistivity character between 28 and 45 metres depth. Higher resistivities to the west and at depth are likely to be indicative of less weathered conglomerate with little if any alluvial vestiges.

As the depth to the water table is typically 5m in the south to 10m in the north of the TEM survey area (refer **Section 2.5.2**) the resistivity patterns are more likely to indicate spatial variations in clay content rather than moisture content or salinity.

In summary, the available data sources indicate that the thickness of alluvium within the vicinity of the site area, and particularly around Goonbri Creek, varies from less than 10m (from exploration bores on the north western margin of the alluvial area) to approximately 30m to the south east (from TEM survey). For the purpose of this assessment, it is concluded that where the Project overlaps the alluvium footprint, alluvium thickness could potentially extend to depths up to 40 m, typically on the south-eastern margin, particularly towards Goonbri Creek. It is possible that alluvium thickness within the proposed open cut development area will not exceed 30 m, although a maximum depth of 40 m has been adopted for this assessment for conservatism.

2.5.2 GROUNDWATER OCCURRENCE (UPPER NAMOI ALLUVIUM)

In conjunction with general hydrogeological characterisation undertaken in **Section 2.5.1**, an indication of groundwater occurrence and groundwater level within the Upper Namoi alluvium has been obtained. These details are summarised below:

(I) NOW GROUNDWATER MODEL

From NOW (2010), model output interpretation indicates groundwater flow within the alluvium horizon occurs in a westerly direction, generally perpendicular to the alluvium subcrop alignment (i.e. extent of alluvium as shown on **Drawing 003**). Predicted groundwater contours from this model are reproduced on **Drawing 003**. A section through the pit in the final year of mining (i.e. 2029), coincident with exploration boreholes as described in **Section 2.5.1** and showing final void geometry, thickness of alluvium and regional groundwater levels, is provided on **Drawing 004**. This section infers that alluvial groundwater falls below the surface of the basement horizon, implying that at this location, groundwater is not present in the alluvial sequences.

Further interpretation of the NOW (2010) model is that permeabilities mapped on the margins of the alluvial sequences are of the order of 0.5 to 1 metres per day (m/day) (equivalent to $5x10^{-6}$ to 10^{-6} metres per second [m/s]).

(II) EXPLORATION DRILLING

The borehole drilling program undertaken as part of the groundwater investigation program by RPS Aquaterra as described in **Section 2.5.1** included a groundwater monitoring bore (labelled TAWB16), located to the north of the borehole transect. The location of this bore is provided in **Appendix A1**. The groundwater level measured in TAWB16 was some 5 m below existing ground level.

A "rising head" test carried out within the bore (TAWB16) also indicated permeability for the alluvial sequences of the order of 10^{-3}m/day (equivalent to 10^{-8}m/s). This measured permeability correlates with the observed clayey sequences. Further details of the groundwater investigation program are presented in the Groundwater Assessment prepared by Heritage Computing (2011) (Appendix A of the EA).

(III) HERITAGE COMPUTING (2011) HYDROGEOLOGICAL ASSESSMENT

Compilation of regional and site-based groundwater monitoring site data has led to an understanding of groundwater levels within the Upper Namoi Alluvium, and associated groundwater flow directions. The data comprised 15 regional alluvial bores and 159 mine bores. Based on data interpretation presented in Heritage Computing (2011) (Appendix A of the EA), regional groundwater flow direction in the alluvial sequences is towards the west and south-west. The hydraulic gradient flattens appreciably to the south-west between the Tarrawonga Coal Mine and the Namoi River. Representative groundwater level in the alluvium bordering the Tarrawonga Coal Mine site is typically is 5 to 10 m below ground.

Further to this assessment, modelling of the regional groundwater system within and close to the existing mine and proposed Project was undertaken. Details of this model are presented in Heritage Computing (2011) (Appendix A of the EA), which comprised upper alluvium layers (accommodating both Narrabri and Gunnedah Formations) overlying 10 other layers (accommodating the Maules Creek Formation and Boggabri Volcanics). The regional groundwater model was separately utilised to assess the highest predictable groundwater level within the alluvial layer, with recharge conditions based on the 2010 wet season, with maximum recorded rainfall of 159 millimetres (mm) in February 2011. Output from this separate modelling exercise undertaken by Heritage Computing (2011) is presented in **Plate 4**.

In summary, these regional model outputs indicate that groundwater flow within the alluvial groundwater system adjacent to the Tarrawonga Coal Mine site occurs in a west south-westerly direction. It is indicated in Heritage Computing (2011) that permeabilities within the alluvium is predominant in the horizontal, with lower vertical permeabilities. Alluvium permeability values adopted for groundwater modelling purposes are as follows:

Alluvium Layer	Horizontal Permeability	Vertical Permeability
Narrabri Formation	5×10^{-5} m/s (5 m/day)	1×10^{-8}m/s (0.01 m/day)
Gunnedah Formation	9×10^{-5}m/s (8 m/day)	1×10^{-8}m/s (0.009m/day)

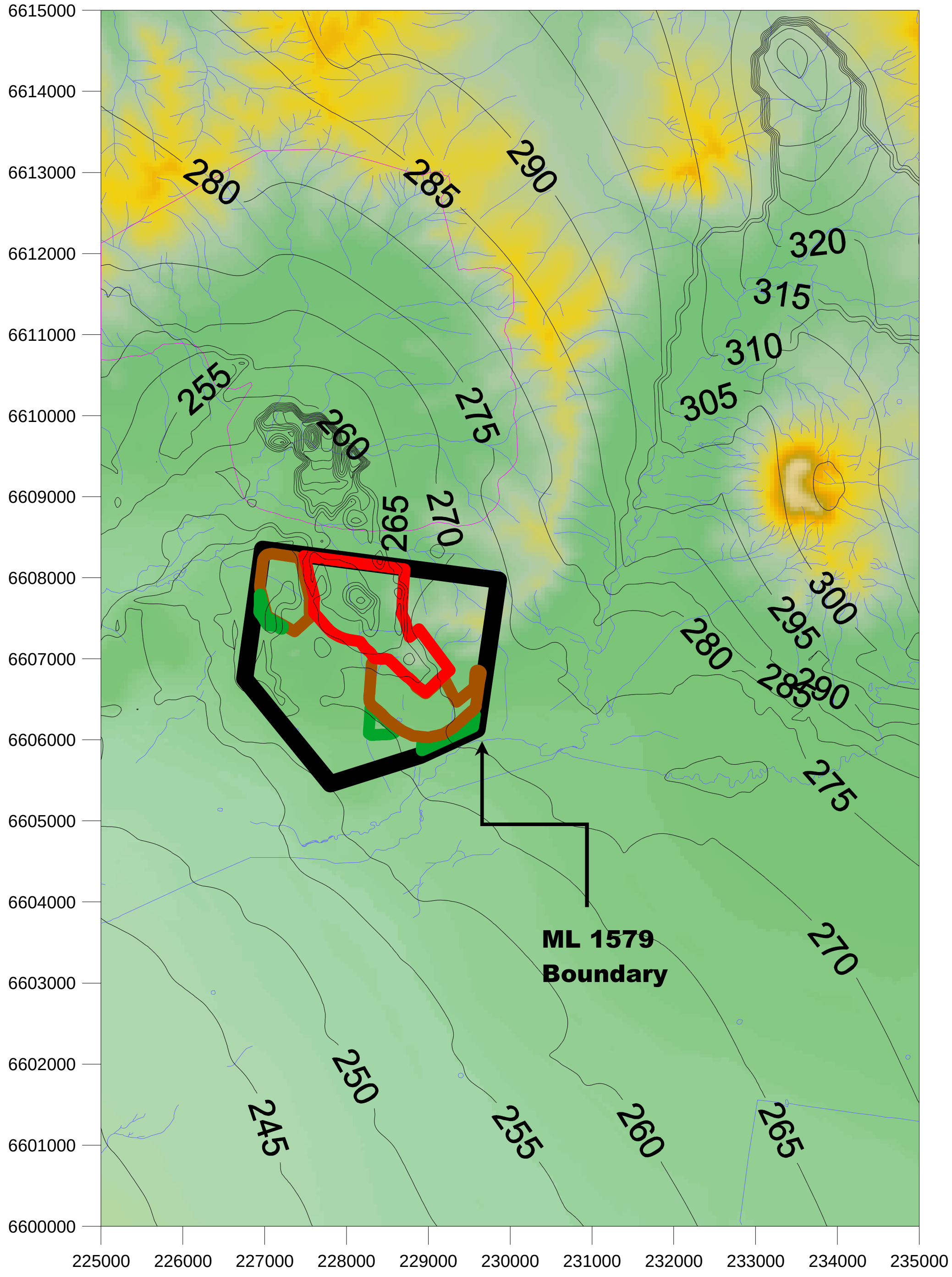
For assessment purposes, the following layer thicknesses for these units have been adopted:

Narrabri Formation 0 to 15 m

Gunnedah Formation 0 to 25 m

Uppermost groundwater levels occurring within the alluvium have been recorded locally to reach within 5m below existing ground surface, although based on groundwater model output, a highest predicted regional groundwater level of 270 m AHD (refer **Plate 4**) has been adopted. It is also assumed that the alluvial layers are hydraulically connected, and that the principal recharge mechanism is surface infiltration from rainfall infiltration or stormwater runoff from the elevated topography to the north-west.

Plate 4 – Groundwater Model Output – Highest Predicted Groundwater Level (in Alluvial Layer)



SECTION 3.0 - PRELIMINARY SEEPAGE MODELLING

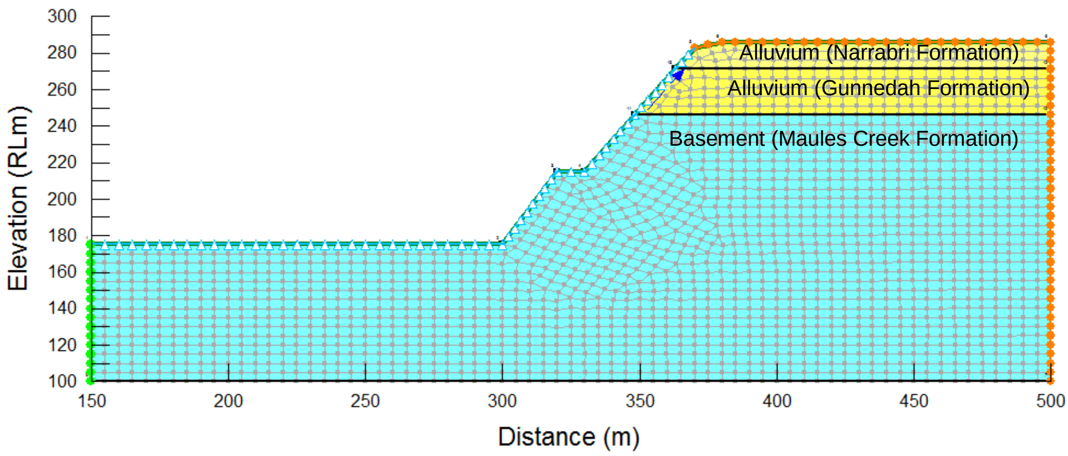
3.1 SCOPE AND MODEL DEVELOPMENT

The need for a low permeability barrier, to satisfy requirements as outlined in **Section 1.1**, will depend on the seepage flow rate into the open cut from that portion of the alluvium that is intersected by mining. Where a “severe” groundwater level drawdown within the alluvium, or where significant reduction in Goonbri Creek baseflow results, a low permeability barrier formed as a groundwater flow cut-off can be justified. Further justification could be made where a substantial pit dewatering requirement develops or pit batter instability is anticipated.

Preliminary seepage modelling has been carried out as a means of estimating the potential groundwater inflow quantities into the proposed open cut from the intersected alluvial sequences (as part of the Upper Namoi Alluvium – refer **Section 2.5**). These inflow quantities provide a basis for assessment of the need for a seepage cut-off system, and to provide design criteria for use in preliminary engineering of the system. Modelling has been carried out for typical section through the final year (2029) pit configuration, as shown on **Drawing 003** and **Drawing 004**, using the computer-based numerical (finite element) seepage modelling package, SEEP/W. SEEP/W is formulated on the basis of Darcy’s Law for both saturated and unsaturated flow. The model iteratively solves mass balance differential equations for a grid of finite elements, based on appropriate boundary conditions.

The thickness of alluvium adopted for this section has been taken as 40 m, as outlined in **Section 2.5.2**. The mesh developed for SEEP/W modelling in relation to this section is shown on **Plate 5**.

Plate 5 – Finite Element Mesh



The model comprises three layers, with adopted thicknesses as follows:

Alluvium (Narrabri Formation)	15 m
Alluvium (Gunnedah Formation)	25 m
Basement (Maules Creek Formation)	To base of open cut (up to 120 m)

Model boundary conditions comprised the following:

- Constant head boundary (RL 270 m) on vertical boundary at x=500 m (refer **Section 2.5.2**)
- Review boundaries across natural surface and internal mine batter
- No flow boundary along base of model (at y=100 m)

The analysis was run under long-term (steady state) conditions. No recharge to groundwater through any surface was therefore considered in the model.

Model inputs comprising permeability values for each layer are summarised in **Table 2**. Permeabilities for alluvial sequences are based on **Section 2.5.2**, with basement permeability taken as an average of basement permeabilities used in the groundwater model presented in Heritage Computing (2011).

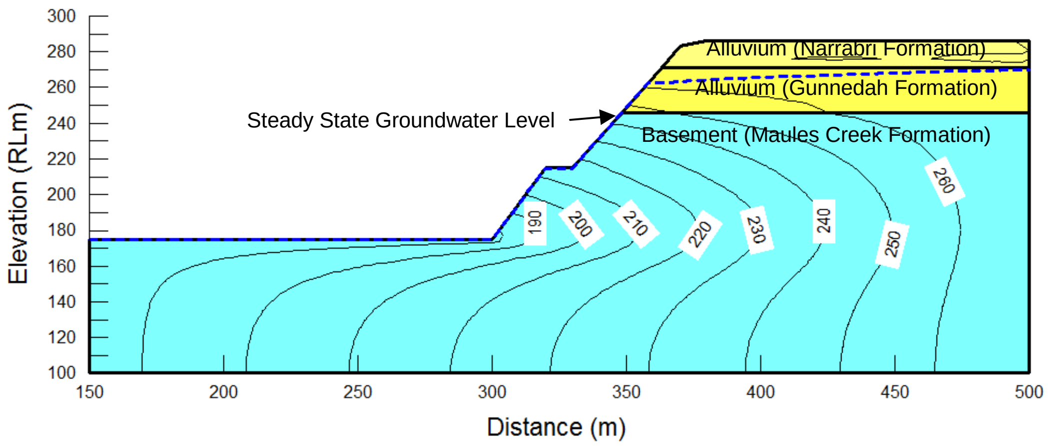
Table 2 – Summary of Permeability Values Used in Seepage Modelling

Zone Description	Adopted Permeability (m/s)	
	Horizontal	Vertical
Alluvium (Narrabri Formation)	5×10^{-5}	1×10^{-8}
Alluvium (Gunnedah Formation)	9×10^{-5}	1×10^{-8}
Basement (Maules Creek Formation)	8×10^{-7}	3×10^{-9}

3.2 SEEPAGE MODEL OUTPUT (BASE CASE)

Seepage model output for the model cross section (adopted as a base case condition), is presented in **Plate 6**.

Plate 6 – Seepage Model Output – Steady State Condition



This output shows that a relatively constant groundwater level within the upper alluvial horizon is maintained across the section under steady state conditions, which controls the rate of inflow into the open cut. The estimated rate of inflow under these theoretical conditions is some 1.9×10^{-5} cubic metres per second (m^3/s) for a 1 m wide section.

Extrapolating over a theoretical 2,000 m long section of open cut, assuming constant boundary conditions and alluvium thicknesses, indicates a potential (i.e. worse case) total groundwater inflow rate of some 3.3 megalitres per day (ML/day) (i.e. without a low permeability barrier).

From a practical perspective, it would be expected that any discharge from the alluvial sequences into the pit would occur as expression along the alluvium basement interface, with discharge over the pit walls formed in basement sequences.

SECTION 4.0 - CONCEPT OPTIONS ASSESSMENT FOR ENGINEERING FOR LOW PERMEABILITY BARRIER

4.1 CONCEPT OPTIONS

Seepage modelling as described in **Section 3.0** predicts potential (worse case) groundwater inflow rates of up to 3.3ML/day from the alluvial aquifer into the open cut development in the absence of any mitigation works (i.e. low permeability barrier). For the purposes of this concept design it was considered that any flow rate of greater than 1.0 ML/day would be significant in terms of loss of groundwater resource, and associated need for water supply allocation from the Upper Namoi Zone 4 water source. To this end, it is considered that justification exists for development of a system to control the rate of groundwater loss into the void, with the use of a low permeability barrier to be targeted.

Concept engineering for the low permeability barrier has comprised consideration of a number of options, with a preferred option to be selected on the basis of effectiveness, constructability and capital cost. Several types of cut-off walls are available to provide a low permeability barrier to seepage both during and post operation. Common barrier types that have been utilised in mining applications, and are considered appropriate for the proposed application include:

- Soil-bentonite wall type.
- Clay core constructed in open excavation.

These systems would be constructed outside the limits of the mine pit, with installation occurring prior to the pit reaching its final configuration. In broad terms, the construction approach would comprise excavation through the alluvium to intersect the underlying basement, with backfilling of the excavation using a low permeability medium. With the bentonite wall approach, several alternative forms of low permeability medium are available, including cement-bentonite and HDPE-bentonite. The soil-bentonite approach is however considered to provide the most cost effective option with easier and quicker construction, therefore reducing overall costs. The open excavation approach comprises a clay zone compacted in the centre of an excavation, with the remainder of the excavation filled with general fill material (either excavated spoil or mine overburden). These walls are simple and quick to construct using conventional plant. The principal disadvantage however is the large quantities of excavation usually required, and the need to dewater the excavation during construction.

A further barrier option was also prompted by TCPL, comprising a clay liner constructed against the final void face. This system would require a cut back of the mine pit within the limits of the alluvium to facilitate construction. A clay wall would be constructed using a zone of conventional compacted clay placed against the batter of the cut-back excavation. The advantage of this option is that the excavation can be integrated with the mining process, and therefore is less costly. This option also has inherent disadvantages, principally the need to control the seepage during pit excavation (i.e. during mining) and before the final liner can be installed.

Dewatering approaches within the pit have not been considered in this study due to the inferred permeability conditions of the alluvium, with dewatering efficiencies expected to be low. High operating costs and the requirement to store or dispose of pumped water is also of consideration.

Three options have therefore been considered as part of this study, with these options including:

Option 1 - Soil-Bentonite Barrier

Option 2 - Trench Excavation and Engineered Backfill

Option 3 - In-Pit Batter Lining

Basic concepts for these options are shown on **Plate 7**. The alignment assumed for each of these options, around the eastern end of the final pit configuration (Year 2029), is shown on **Drawing 003.** It is noted that for each option, a flood levee has been incorporated, to be constructed across the surface of the barrier. This levee would provide flood mitigation for the mining operations subject to a flood event across the Upper Namoi River floodplain, or other backwater effects through local flooding through Goonbri Creek or Bollol Creek.

4.2 CAPITAL COST COMPARISON

As a principle basis for comparison of low permeability barrier options as described in **Section 4.1**, a preliminary capital cost estimate for each option has been undertaken. Costing has been undertaken based on benchmarked pricing available from past experience, with support from available costing handbooks and preliminary costing analysis. This costing approach can provide an accuracy of not greater than ±35% applying to construction rates. Accuracy on construction quantities cannot be defined, give the current uncertainty on the site conditions assumed, as described in **Section 2.0**. Regardless of this uncertainty, the costing approach adopted is considered suitable for cost comparison between options.

As a basis for costing, the typical cross sections for each option as shown on **Plate 7** have been adopted**,** with the following dimensions used for volume calculations:

- Maximum depth of barrier to be 40 m near the centre, rising to 25 m coincident with the section taken through the pit, 5 m at the return near the ends of the alignment and 2 m at each end. The alignment and cut off depths assumed are shown on **Drawing 003**.

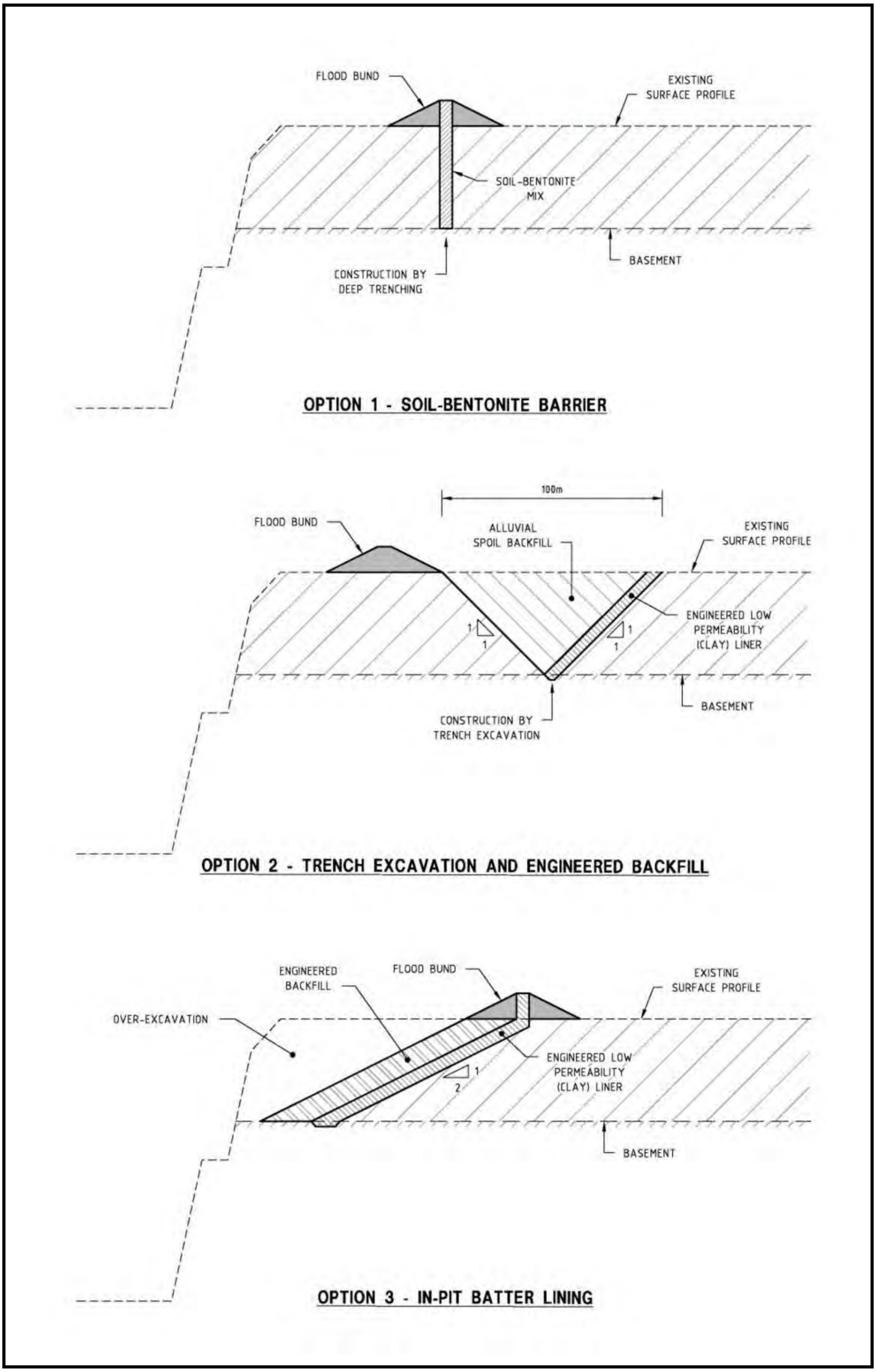
- Total length of barrier of 2.9 km.

An overview of capital costing for each option based on these layouts is provided as follows:

4.2.1 OPTION 1 - SOIL-BENTONITE BARRIER

This option would comprise construction by deep trenching with bentonite stabilisation. Excavation would be undertaken either by long reach excavator, or by clamshell digger or hydromill trench cutter. Long reach excavation up to 35 to 40 m is considered to be feasible under appropriate subsurface conditions. A batched soil-bentonite mix, prepared on site preferably utilizing soil material sourced from the trench excavation or mine spoil (whichever is appropriate), would be pushed into the trench excavation by dozer to displace the bentonite slurry. Depending on the type of excavation, it is possible that the wall would be formed in discrete panels, connected with construction joints.

Plate 7 – Concepts for Low Permeability Barrier Options



The estimated sectional surface area of the barrier is some 50,200 square metres (m^2). The unit rate for soil-bentonite barrier construction, by benchmarking against similar construction projects in the Hunter Valley, is \$200 to \$270/m^2. Resultant costs are therefore in the range of \$10.0 to 13.6 million.

4.2.2 OPTION 2 - TRENCH EXCAVATION AND ENGINEERED BACKFILL

This option would comprise excavation of a trench wide enough to remain stable, and to allow plant entry for excavation and backfilling purposes. Nominal excavation bench heights to 5m at 1(H) to 1(V) slopes with 3m wide berms have been allowed. Total excavation to achieve this trench, to the depths and extents as described, is estimated to be 5.1 million cubic metres (Mm^3) (maximum width of 150 m), with the trench sectional area increasing as the barrier depth increases to intersect the alluvial horizon. The backfill proposal is to construct an engineered clay fill liner on the outer face of the trench. A barrier width of 4m (machine width) has been allowed, with suitable clay fill material assumed to be sourced from mine spoil. This layer would be constructed (by height) in advance of with trench backfilling using mine spoil

Costing rates adopted for construction works as described above are:

- Excavation \$3.50/m^3 (using mix of mining and civil plant)
- Engineered backfill \$7.50/m^3 (using civil plant predominantly)
- Spoil backfill \$3.00m^3 (using essentially mining fleet)

Based on these rates, the estimated capital cost for Option 2 is \$35.5 million.

4.2.3 OPTION 3 - IN PIT BATTER LINING

This option would comprise lining the completed internal face of the pit to the extent of alluvium intersection. To facilitate this construction, a mining cut-back would be undertaken to provide a bench wide enough to accommodate the liner as well as a spoil buttress, placed to protect the liner and to stabilize the cut-back batter. The cut-back bench width would vary from around 70m at the deepest section of alluvium to around 20m at each end. The batter liner would be constructed using engineered clay fill liner. A liner width of 4m (machine width) has been allowed, with suitable clay fill material assumed to be sourced from mine spoil. This layer would be constructed (by height) in advance of buttress placement using mine spoil.

Based on these conditions, estimated construction quantities under Option 3 are as follows:

• Cut-Back Excavation			1.8 Mm^3
• Backfill	-	Clay Fill	250,000 m^3
	-	Mine Spoil	900,000 m^3

Costing rates adopted for capital cost comparison purposes are:

- Cut-Back Excavation $3.50/m^3$
- Clay Fill $7.50/m^3$
- Mine Spoil Backfill $3.00/m^3$

A total capital cost of $10.8 million has been estimated.

4.3 OPTIONS ASSESSMENT

The criteria adopted for comparison of low permeability barrier options, and for selection of a preferred approach are effectiveness, constructability and capital cost. An assessment of options based on these criteria is presented in **Table 3**.

Table 3 – Low Permeability Barrier Options Assessment

Criteria	Option 1 Soil-Bentonite Barrier	Option 2 Trench Excavation and Engineered Backfill	Option 3 In-Pit Backfill Lining
Effectiveness	Based on objectives (**Section 1.1**), Option 1 can effectively achieve seepage cut off to open cut both operationally and post closure. Performance relies on appropriate construction.	Similar to Option 1, Option 2 can effectively achieve seepage cut off to open cut both operationally and post closure. Due to construction approach, quality control is easier to achieve.	Option 3 provides an effective post-closure option. The cut off cannot be constructed until near the end of mining therefore is of limited effectiveness during mining.
Constructability	Constructability varies with cut-off depth, with greater difficulties with deeper trenches and for greater strength/ stronger materials. Other issues to be considered include trench stability, barrier width and suitability of soils for use in the backfill mix.	Option 2 is easily constructed, with the key constraints being strength of material to be excavated, management of seepage water during excavation and volume of excavation. Quality control issues relate to quality of liner material and material compaction, although are readily controllable given the access.	Similar constructability issues as Option 2. Key issues therefore relate to strength of material to be excavated and management of seepage water during excavation. Quantity of excavation is less of an issue, being possible using mine plant.
Capital Cost	Based on costing basis presented in **Section 4.2.1**, estimated capital costs for Option 1 are $10.0 to $13.6 million.	Based on costing basis presented in **Section 4.2.2**, estimated capital costs for Option 2 are $35.5 million.	Based on costing basis presented in **Section 4.2.3**, estimated capital costs for Option 3 are $10.8 million.

On balance, Option 1 achieves most effectively the key objectives as described in **Section 1.1**, and provides an economical outcome compared to the other options. The only potential shortcoming is constructability, although with issues that can be identified and managed by appropriate engineering and construction planning.

SECTION 5.0 - PRELIMINARY ENGINEERING FOR PREFERRED LOW PERMEABILITY BARRIER

5.1 DETAILS OF PREFERRED OPTION

Based on the comparison of options as described in **Section 4.3**, the preferred low permeability barrier option for the Project is a soil–bentonite barrier (Option 1). This option has been preferred based on the overall effectiveness of the system, with application for both operational and post-closure periods, and relative cost. By the nature of the system, constructability issues may arise, although these potential issues can be identified and managed by appropriate engineering and construction planning. Furthermore, it is considered that the subject site is ideally suited to a soil-bentonite wall as it is relatively level, has ample space for the construction process, and the alluvial layers are likely to be well suited for slurry trenching.

As a basis for preliminary engineering of this preferred system, an overview of the typical method of construction of the soil – bentonite wall is provided below:

- The construction of a soil-bentonite wall is a continuous process and utilises the excavated spoil from the trench to form the final impermeable wall.

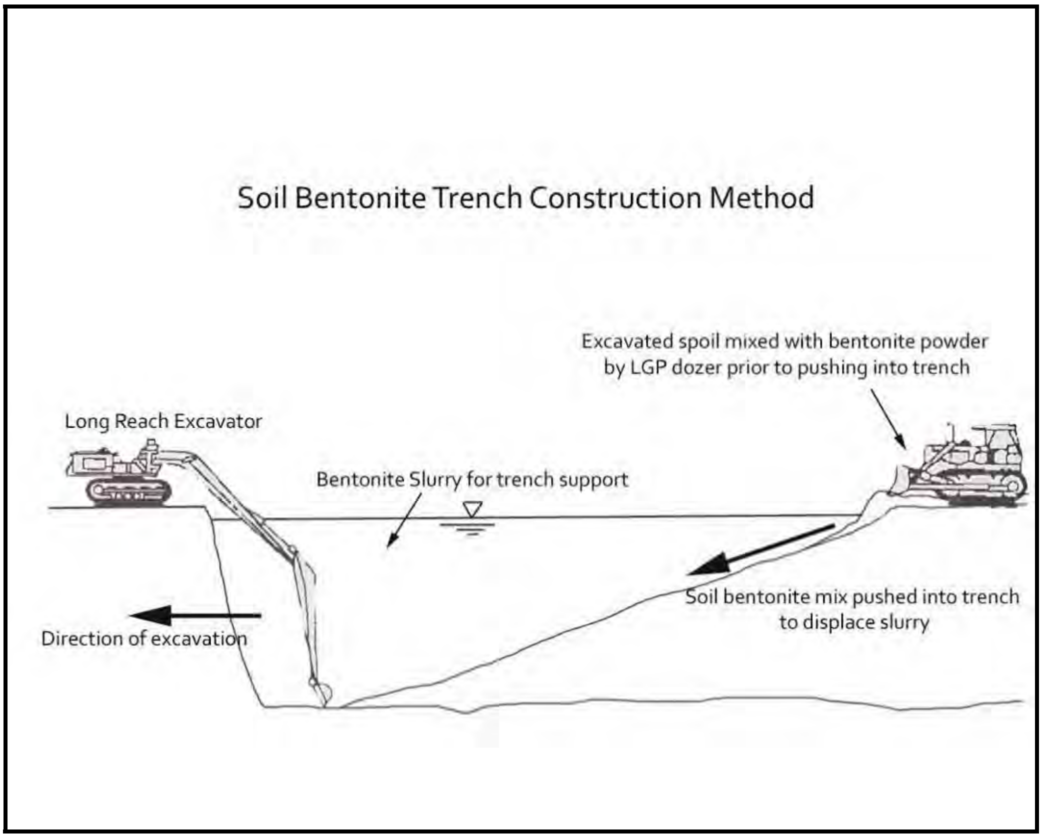
- Construction proceeds by excavation of a few hundred metres of trench to full depth under bentonite slurry, using a specialised long reach excavator and/or clamshell bucket (dependent of trench depth and subsurface conditions). A typical trench width is 0.9m to 1.1m

- Select spoil from the excavation would be spread adjacent to the trench, with this spoil likely to be blended with bentonite slurry from the trench. Any spoil considered marginal or unsuitable would be discarded, with the backfill material quantity supplemented using mine overburden material (as required). The mixing area would require bunding to contain the wet spoil. Typically this mixing area would be located within 30m of the trench.

- Bulk powdered bentonite would be spread over the excavated spoil within the prepared mixing area. A low ground pressure (swamp) bulldozer or similar amphibious plant would then be used to mix the spoil and powdered bentonite by tracking through the mix continuously. A typical mix might consist of 5% dry bentonite and possibly 10% to 20% of imported clay combined with the excavated spoil. The amount of imported clay required will depend on the clay content of the excavated spoil, and as needed to achieve the target permeability of the wall. Care is required through this mixing process, particularly with the addition of clay, to ensure a homogeneous mix is achieved, with no clay clods remaining.

- On mixing of the soil-bentonite, the 'mix' would then be pushed using a dozer into the end of the open trench where it flows forward to form a beach slope in the order of 10% which displaces the bentonite slurry in the trench. A bulkhead for pushing is maintained at the end of the trench. The operation is planned so that the amount of soil-bentonite pushed into the trench roughly equals the amount of forward excavation at the front face, and therefore the bentonite slurry in the trench can be kept at a reasonably constant volume. This slurry is recycled regularly to remove trapped fines and soil. The typical operation is shown in the sketch provided as **Plate 8**.

Plate 8 – Typical Operation for Construction of Soil-Bentonite Barrier



- Normally the cut-off would be keyed into the basement rock by about 1m. This is usually achieved by a ripper bucket fitted to the excavator. Experience on previous projects indicates that cleaning of the base of the trench by excavator bucket is adequate (i.e. no greater effort to remove spoil being necessary), as the advancing soil-bentonite mix tends to push any remaining spoil ahead of the backfill.

- On completion of the trench, the bentonite slurry is disposed of and the soil bentonite backfill allowed a period to consolidate before construction of the flood levee. Soil-bentonite settles considerably as it consolidates, and specific design detailing is required of any connection with the overlying levee. (Note that this aspect is covered in **Section 5.5**.)

5.2 ASSESSMENT OF SOIL-BENTONITE BARRIER PERFORMANCE

Further to the preliminary seepage modelling as presented in **Section 3.0**, an assessment of improved performance subject to installation of a soil-bentonite barrier around the perimeter of the proposed open cut development has been undertaken. The proposed alignment of the barrier is as shown on **Drawing 003**, with base case seepage model conditions as described in **Section 3.1**. The parameters adopted for the soil-bentonite barrier are as follows:

Parameter	Adopted Model Value
Width of Soil-Bentonite Barrier	1.0 to 2.0
Setback of Barrier behind Edge of Open Cut	50m
Depth of Basement Embedment	0 to 5m
Permeability of placed Soil-Bentonite Mix	10^{-8} to 10^{-9}m/s

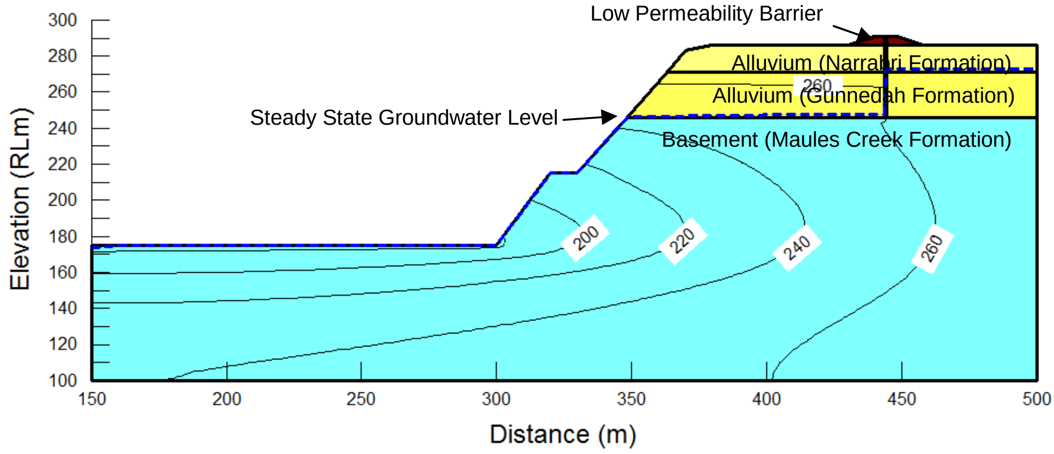
A base case analysis has been undertaken based on the following parameters:

- Barrier width 1.0 m
- Setback from open cut edge 50 m
- Depth of Basement Embedment 0 m
- Barrier Permeability 10^{-8} m/s

This configuration was incorporated into the model as a separate layer.

Model output from this base case analysis under steady state conditions is presented as **Plate 9**. Predicted seepage flux into the open cut for this base case is 5.99×10^{-7} m^3/s for a 1m wide section. For an equivalent section width of 2,000m, the equivalent seepage rate is some 0.1 ML/day. This rate compares to 3.3 ML/day for the base case condition excluding the barrier described in **Section 3.2**.

Plate 9 – Seepage Model Output – Inclusion of Soil-Barrier Wall (Steady State Condition)



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Based on the range of values adopted for parameters listed above, a sensitivity analysis using the seepage model has been undertaken, with summary output from this analysis provided in **Table 4**.

Table 4 – Low Permeability Barrier Options Assessment – Sensitivity Analysis

Case No	Description (Variation from Base Case – refer Plate 9)	Sensitive Parameter	Predicted Seepage Flux*	Drawdown Depth (in Phreatic Surface)
1	Varying Barrier Width	Width 2m	0.06 ML/day	27.3 m
2 (a)	Depth of Barrier Embedment	Depth 1m	0.10 ML/day	26.0 m
2 (b)	Depth of Barrier Embedment	Depth 5m	0.10 ML/day	25.9 m
3	Barrier Permeability	10^{-9}m/s	0.02 ML/day	28.3 m

* Based on a barrier length of 2,000 m

The results of this analysis indicate that the range of conditions analysed achieves groundwater inflow rates from the alluvial horizon to the open cut development generally less than 0.1 ML/day, which has been highlighted in **Section 4.1** as a target maximum. It is noted that key parameters, based on predicted groundwater inflow rate, are barrier width and barrier permeability. Notwithstanding, it is considered that all sensitive parameters considered are important design factors.

In summary, seepage modelling indicates that under realistic construction conditions achievable for a soil-bentonite barrier, an appropriate outcome is considered to be achievable. The conditions assessed as part of this analysis therefore provide a range of conditions of barrier width, depth and permeability.

5.3 OVERVIEW OF PREVIOUS SOIL-BENTONITE APPLICATIONS

Three case studies have been reviewed to examine previous experience with the use of soil-bentonite barrier for seepage control. A summary of the key design and construction aspects on these cases is provided below.

The Investigation and Design of a Soil-Bentonite Cut-Off Wall in Alluvial Gravels for the Hunter Valley Mine (Thorley et al., 1994)

Aspect	Findings/Description
Objective	Isolation of open cut mine pit from Hunter River floodplain
Design	4km long barrier wall, 1.1m thick, 15 to 28m deep Incorporates 4 to 9m high flood levee over trench alignment
Investigation Works	Engineering investigations included seismic refraction surveys, small diameter borehole drilling and test pitting. In addition, large diameter (1,250mm diameter) cased boreholes were drilled to characterise the alluvium.
Setting/Conditions	Investigation indicated following soil units (in order of intersection): • Sandy gravel and gravelly sand • Silty sand and relict topsoil • Sand and gravelly sand (with occasional cobbles) • Gravel, sandy gravel and cobbles/boulders (deposited Fairford Claystone) • Basement comprising mudstone, sandstone, claystone and coal
Mix Characteristics	Excavated spoil mixed with 5% dry bentonite and 15 to 25% natural clay, to achieve a wall permeability of 10^{-8}m/s
Design Approach	Soil-bentonite selected based on ease of construction, cost and flexibility under high hydraulic gradients. Design approach comprised assessment of: • seepage behaviour (including embedment) • overall stability of levee and wall system subject to adjacent mining • soil-bentonite mix suitability subject to high hydraulic gradient • consolidation behaviour of mix immediately following placement • filter requirements Design analyses indicated sensitivity in seepage rate with wall thickness as well as embedment, and also potential erodibility of weathered and fractured coal under high hydraulic gradients. Predicted that up to 1m of settlement could occur post-construction. A clay core trench in the levee was incorporated to envelop the upper portion of the backfill to provide an impermeable barrier, and included a post-backfill capping of wet plastic clay to provide additional sealing.
Construction Trials	Trial clam shell excavation and soil-bentonite mix was carried out to confirm suitability of the proposed method of construction
Post-Construction Monitoring	Instrumentation provided to measure soil-bentonite consolidation (using settlement plates, extensometers, piezometers and settlement profile gauges) and to provide data on potential leakage (piezometers)

A general layout and typical detail for the wall, as an excerpt from Thorley *et al.* (1994) is provided as **Plate 10**.

Plate 10 – Layout and Detail for Soil-Bentonite Wall (after Thorley *et al.*, 1994)

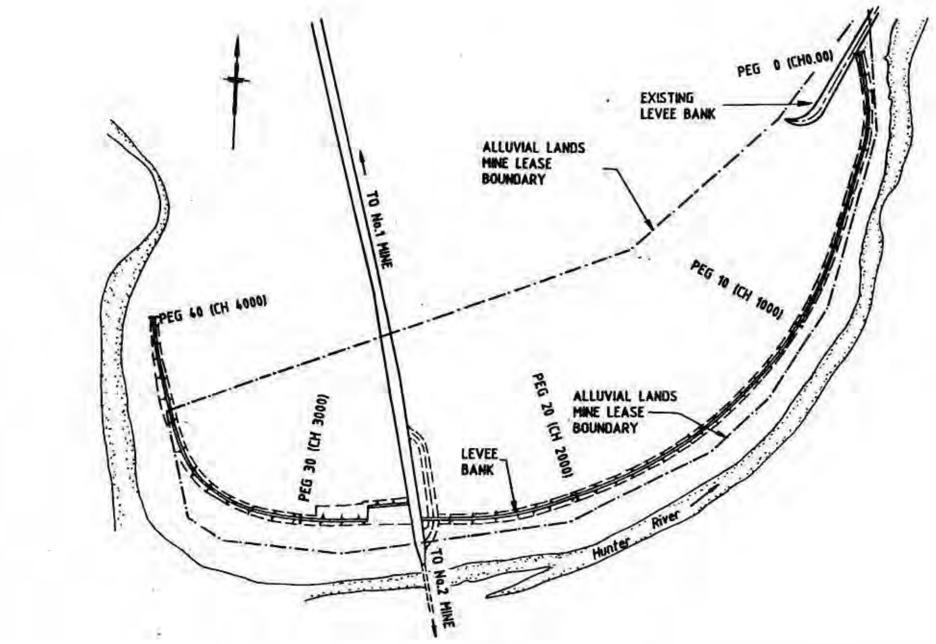


Figure 1: Site Plan

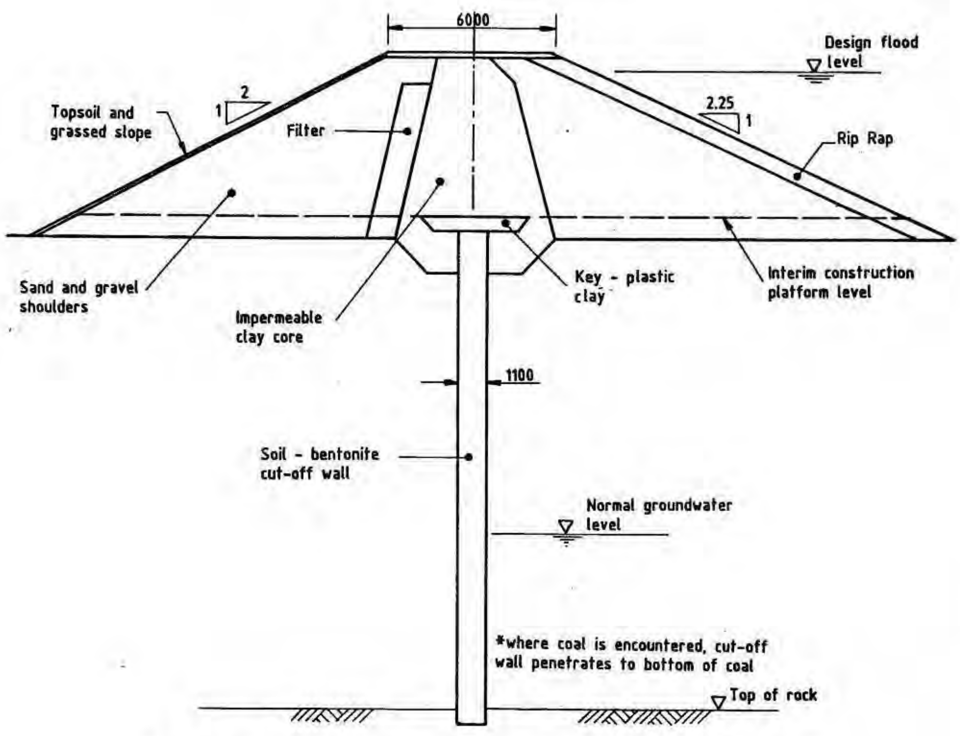


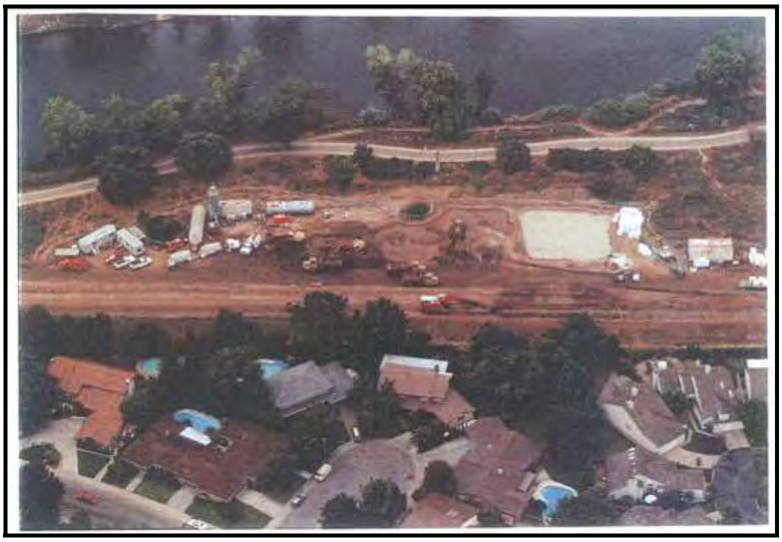
Figure 2: Typical Cross-Section of Levee and Cut-Off

Construction of a Soil-Cement-Bentonite Slurry Wall for a Levee Strengthening Program (Owaidat et al., 1998)

Aspect	Findings/Description
Objective	Reduction of seepage through and beneath an existing flood levee to protect major commercial and residential areas, and therefore to prevent potential destabilisation of levee
Design	Use of soil-cement-bentonite, rather than soil-bentonite only, to provide shear strength to enhance levee stability (as part of the strengthening program)
Investigation Works	Laboratory tests undertaken to confirm mix design based on performance, with the use of site materials to be excavated. Key parameter was strength, with laboratory testing based on design hydraulic gradient.
Setting/Conditions	Subsurface conditions comprise: • Clayey sand • Silty sand • Clayey sand • Silty sand • Poorly graded sand with silt • Gravel and cobbles • Gravel and cobbles • Silty sand • Sandy clay • Clayey sand
Mix Characteristics	Target permeability of 5 x 10^{-9}m/s, with unconfined compressive strength of 100 kilopascals (kPa). Mixed comprised around 10% bentonite and 5% cement.
Design Approach	Key design aspect was to select a cut off wall depth required to obtain a suitable factor of safety for the levee (targeting an appropriate factor for safety against soil boils on the land-side. Analyses indicated a need to cut off the gravel and cobble layers.
Construction	Long reach excavators, with maximum excavation depth of 25m used to excavate a trench some 750 mm width, while pumping bentonite slurry into the trench and maintaining its level at or near the top of the trench during excavation. The trench extended into the aquiclude underlying the cobble/gravel layer, by a depth of around 1m. Slurry was mixed with excavated spoil adjacent to the trench using an excavator or dozer to achieve a smooth consistency, and is then pushed into the trench so that backfill slope displaces the bentonite slurry forward. Excavating and backfilling was carried out in phases make the operation continuous with relatively small quantities of new slurry required to key the trench fill and to mix backfill. Following construction, a cap consisting of compacted impervious fill materials was placed between the top of the slurry wall and the base of the levee.

Photographs of the construction process, as an excerpt from Owaidat *et al*. (1998) is provided as **Plate 11**.

Plate 11 – Photographs of Soil-Bentonite Wall Construction (after Owaidat *et al*., 1998)

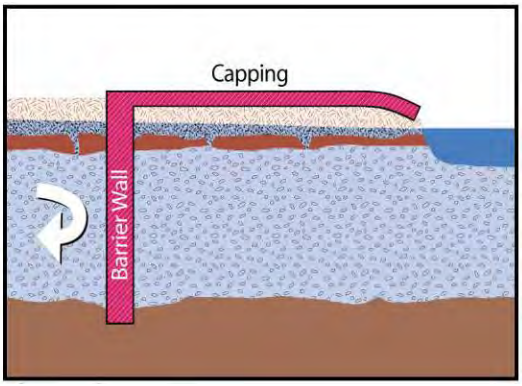


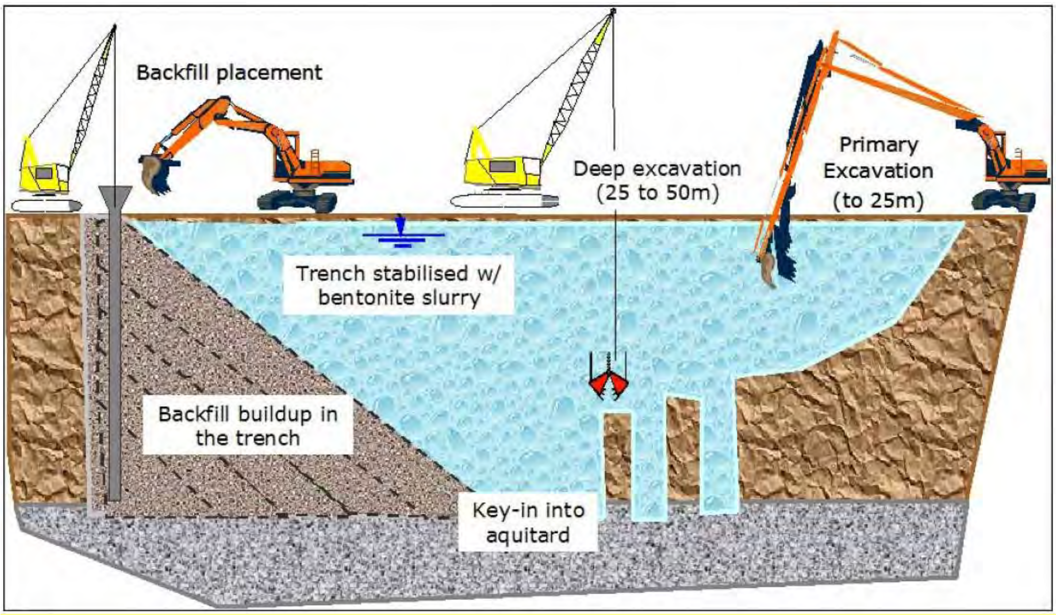
Design and Construction of a Deep Soil-Bentonite Groundwater Barrier Wall at Newcastle, Australia (Jones et al., 2008)

Aspect	Findings/Description
Objective	Diversion of groundwater around a contaminated site, as part of a former steelworks site in Newcastle. The works comprised part of a $155 million remediation project for the steelworks site.
Design	1.5 km long soil-bentonite barrier wall, 0.8m thick, up to 50m deep (depth range 25 m to 49 m. (Note that the system is currently the deepest soil-bentonite wall constructed anywhere in the world) Incorporates capping (sealing) layer constructed over contaminated site area, to prevent surface water infiltration and to provide barrier to secondary contact.
Investigation Works	Geotechnical investigations included 16 test bores, 27 cone penetration tests (CPTs) and 20 test pits. The key investigation components were boreholes and CPTs with a spacing of sites if around 35 m along the barrier length.
Setting/Conditions	The site housed copper smelters, steelwork and ancillary operations, with slag used to fill much of the site area. The subject area was occupied by the most heavily contaminated portion of the site, comprising coke ovens, gas holders and other steelmaking processes. Investigations indicated the following soil units (in order of intersection): • Fill (slag, chitters and rubble), between 3 and 5 m thick) • Silty Sand (surface soil) and Sand, between 20 and 30 m thick • Residual Clay and Basement, between 25 and 40 m, deep
Mix Characteristics	Objectives of the soil-bentonite mix design was to: • Obtain lower and upper limits of fines content and fraction of dry bentonite necessary to achieve the maximum permeability with in-situ soil samples • Carry out permeability testing on in-situ samples using the leachate from contaminated areas as the permeant, to assess any potential impacts on long term hydraulic performance, and to assess and adverse reactions on the bentonite slurry. The key design output was to adopt 20% fines to achieve the target/design permeability.
Design Approach	Geotechnical design comprised a balance between slurry pressure and lateral earth pressure. The key design criterion was for the completed wall to withstand a differential head of 5m across the wall, and avoid the onset of hydraulic fracturing. The design found that this condition would be unlikely for a wall thickness exceeding 0.5 m (with an actual wall thickness of 0.8 m selected).
Construction Approach	Trench excavation under bentonite slurry. Given the depth of excavation, two plant items required, comprising a long reach excavator (with excavation reach up to 25 m), and mechanical clamshell excavating to deeper depth. As excavation proceeds, the trench is backfilled with the low permeability soil-bentonite mixture, consisting of excavated soil, imported natural clay soils and bentonite slurry. Backfill material was placed by tremie.
Quality Control Testwork	Permeability testing was carried out as the principle quality control approach. A total of 108 tests was undertaken, indicating permeabilities between $9x10^{-11}$ and $3x10^{-9}$m/s.

A general layout and typical detail for the wall, as an excerpt from Jones *et al.* (2008) is provided as **Plate 12**.

Plate 12 – Layout and Detail for Soil-Bentonite Wall (after Jones *et al.*, 2008)





In addition to the case studies outlined above, a list of recent soil-bentonite barrier projects occurring has been compiled (based on available literature) and is provided in **Table 5**. This list is confined to North American projects, having been compiled by a group of specialist contractors based in the United States.

Table 5 – Dam Embankment Slurry Wall Cutoff Precedents

Dam	Location	Dam Type	Cutoff Type	Construction Method	Function	Hydraulic Head (m)	Depth (m)	Width (mm)	Length (m)	Area (m^3)	Year	Unit Price ($\$US/m^2$)	Owner	Contractor	Comments	Reference
Manasquan	New Jersey	Homogeneous Sand Emb.	Soil Bentonite	2 Stage, Long Stick Backhoe	Primary Seepage Barrier	5.5	21.3	915 1525	-	-	1989	-	Manasquan Water Authority	Inquip	Fully Instrumented	Khoury *et al.*, 1991
Chamber Lake	Colorado	Earthfill	Soil Bentonite	Backhoe Chisel	Seepage Repari	13.7	15.2	915	-	-	1991	64.5	Irrigation & Supply	Inquip	Boulders	R. Davidson, 1993 (personal communication)
Huxtable Pumping Plant	Arkansas	Concrete	Soil Bentonite	Clamshell	Construction Perm. Water control	7.9	24.0	1525	-	-	1979	-	Corps of Engineers	ICOS	-	Corps of Engineers, 1978
Tenn. Tom. Project	Tennessee	Earthfill	Soil Bentonite	Backhoe	Cofferdam	-	9.1 11.0	1525 1525	-	-	-	-	Corps of Engineers	ICOS	-	Winter, 1976
Saylorville	DesMoines	-	Soil Bentonite	-	Upstream Cutoff	47.9	19.8	2196	-	17395.3	-	-	Corps of Engineers	ICOS	-	ICOS, 1976
Calamas	Nebraska	Earthfill	Soil Bentonite	Bucket Auger	Embankment & Foundation	24.0	34.1	1525	-	-	1985	-	Bureau of Reclamation	ICOS	Hydraulic Fracturing	Engemoen, *et al.*, 1986
Locks & Dams 9 & 13	Arkansas	Hydropower Concrete	Soil Bentonite	Backhoe	Construction Water Control	-	9.1 11.9	915 915	-	-	1988 1991	43	AECC	Inquip	-	W. Smith, 1993 (personal communication)
Cholla	Arizona	80 ft Earth Rockfill	Soil Bentonite	Backhoe	Fdn. Cutoff	24.4	33.5	915	305,000	-	1978	-	APS	ECI	-	J. Ehasz, 1993 (personal communication)
LG2 Duncan Dike	Quebec	Dike with Clay Core	Soil Bentonite Concrete	Dragline Clamshell	Dike Fdn. Fdn. Cutoff	-	21.9 70.1	1525 610	1,446,920 274,195	22028.5 9962.9	1977	-	Hydro Quebec	Soletanche	-	Soletanche, 1985
LG3	Quebec	Clay Core Rockfill	Concrete Soil Bentonite	Clamshell	Fdn. Cutoff Dike Fdn.	97.6	46.9 9.1	610 1525	-	6007.7 13016.8	1983	-	Hydro Quebec	Soletanche	-	Solentache, 1985
Addicks Barker	Houston	30 ft. Clay Dike	Soil Bentonite	Kelly Clamshell	Sealing Dike and Fdn.	-	20.1 17.0	915 915	6,405,000	115148.8 45058.2	1978 1979	-	Corps of Engineers	Soletanche	-	Soletanche, 1985

Further understanding of soil-bentonite barrier wall construction is provided by photographs as included in **Plate 13**.

Plate 13 – Typical Soil-Bentonite Barrier Wall Construction

5.4 BARRIER DESIGN CONSIDERATIONS

The aim of a soil-bentonite cut-off is to form a stable impermeable barrier against groundwater flow and which can be constructed in a cost effective manner. The detailed design therefore needs to address both the constructability and the final performance of the barrier.

Soil bentonite walls are constructed by backfilling an excavated trench with the designed backfill material. The temporary works design therefore involves assessing the stability of the trench excavation, and selecting a backfilling method so that the final wall performs as required.

5.4.1 SITING OF BARRIER

Initial design considerations, in the context of the Project, relate to siting of the low permeability barrier, and resulting alignment. The principal controls to siting of the barrier are:

(i) to provide sufficient land area (in terms of corridor width) to facilitate construction, including establishment of spoil stockpiles, plant access and soil-bentonite mixing areas; and

(ii) to maintain long term (mine post-closure) stability of the barrier, being located adjacent to the open cut.

Optimally, a corridor width of 200 m would be preferred for construction of the barrier, although this land area is rarely available for such projects. Regardless, a nominal buffer distance of 50 to 100 m between the proposed final edge of the open cut and the barrier alignment should be provided for pre-design considerations. This distance is based on the potential for slope instability within the alluvial sequences, with a maximum thickness in the area of the barrier of some 40m. A buffer distance of 50m would facilitate a slope failure plane of up to 1(V) to 1.75(H) between the base of the alluvium at its thickest, with a 100m buffer providing of up to 1(V) to 3(H). Based on typical properties for the predominantly gravelly and dense nature of the alluvium as described in available borehole logs (refer **Section 2.5.1**), it is unlikely that a slope failure plane would exceed 1.5(H) to 1(V), therefore a buffer distance within this range would be suitable. As such, a buffer distance of 100m as been shown on **Drawing 003**.

5.4.2 BARRIER DESIGN

Ensuring trench stability during excavation and prior to backfilling is achieved by filling the trench with bentonite slurry. This method is widely adopted throughout the industry for excavating cut-off walls and deep foundations such as barettes and basement diaphragm walls. The bentonite slurry, which is a mix of bentonite powder and water, acts by forming a thin filter cake on the wall of the trench as it is excavated. Even in coarse sands and gravels, this filter cake will form on the trench walls and thus prevents the slurry from seeping into the ground next to the trench. The slurry then supports the trench walls by its hydrostatic pressure against the filter cake. The design requires the slurry level to be selected so its pressure at any level in the trench exceeds the effective earth pressures, with a factor of safety of 1.2 typically adopted.

A high groundwater level requires higher slurry levels to achieve the desired factor of safety, and in severe cases guide walls are used to raise the slurry level above the ground surface level. Given the groundwater levels at the Tarrawonga Coal Mine, it is not anticipated that guide walls will be required. Because the bentonite slurry is a liquid, the ground levels may also need to be adjusted by benching to ensure the slurry can be contained within the trench at all times.

Backfilling of the trench occurs by pushing the mixed backfill into the end of the trench so it forms a beach slope, usually at a flat angle of about 1V:10H if the backfill moisture content is sufficiently high. As the new backfill is pushed onto the top of the backfill slope already in the trench, it loads the top of the beach slope causing the slope to effectively fail and squeeze forward. Using this approach, the backfill is not required to be pumped, tremied or 'dropped' into the trench and the risk of segregation is reduced. Tremieing can be adopted to place the backfill in the base of the trench, but this requires additional plant and is a slower process, usually only being adopted where space for mixing is limited.

A concept for the soil-bentonite barrier wall, based on the above is shown in **Plate 14**.

Plate 14 – Concept for Soil-Bentonite Barrier Wall

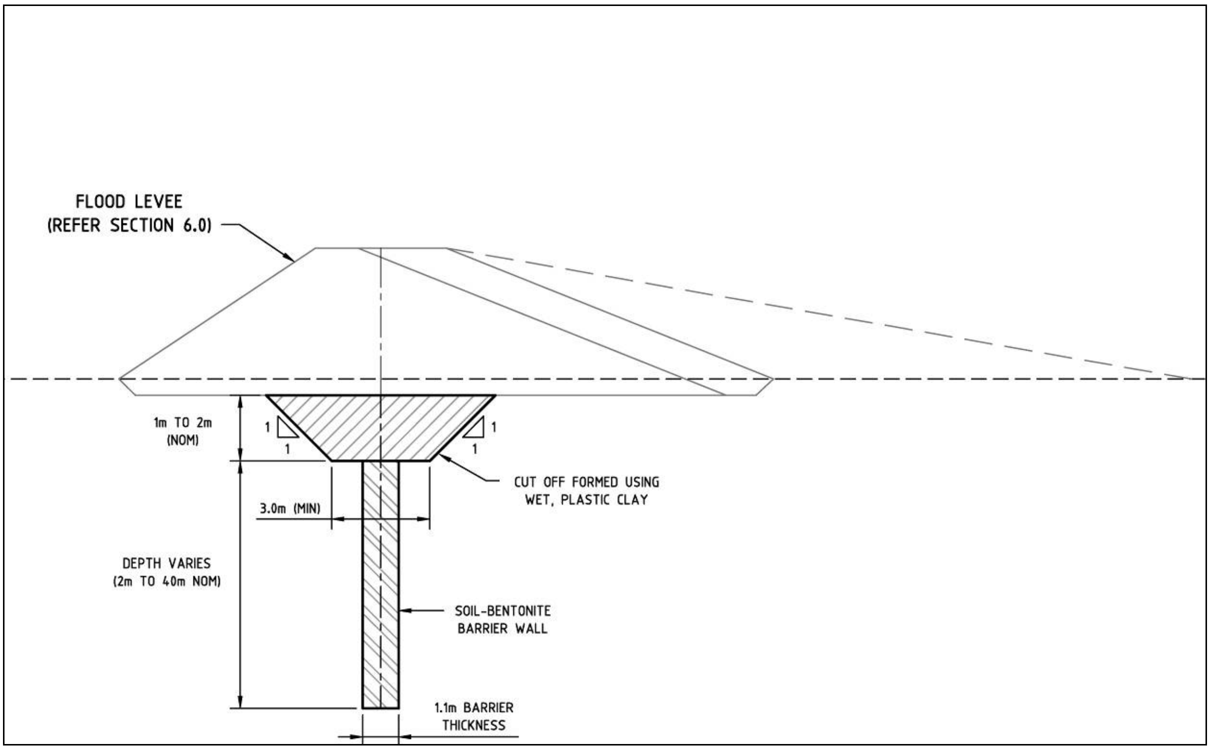


Plate 14 shows a flood levee, forming part of the permanent Goonbri Creek alignment, which is described in more detail in **Section 6.0**.

The cut-off design is based on the following considerations:

- the target permeability required from the detailed seepage analyses is achieved;
- the grading is such that the backfill material will not erode or pipe into the coarser (adjacent) alluvial materials;
- the trench width is sufficient so that hydraulic fracturing of the backfill will not occur under the highest anticipated hydraulic gradients across the wall;
- the key into the basement (i.e. Maules Creek Formation) is sufficient to restrict seepage under the cut-off;
- the detail between the top of the cut-off and the levee is robust enough to withstand potential flood events;
- the backfill grading required to ensure the target permeability is achievable with economic bentonite quantities and imported clay fines if required; and,
- the consolidation behaviour of the backfill is such that 90% consolidation is expected within a reasonable time frame.

In designing the backfill grading, it is necessary to first define the particle size distribution for alluvial materials occurring naturally along the trench alignment, and at all depths. This will allow assessment of the suitability of these materials for use in the soil-bentonite mix.

Typically trial mixes are then prepared, with varying proportions of bentonite and additional clay material (as required), which can be tested for permeability. Lateral variations in the alluvium can also be assessed to determine if a changing backfill grading is required as the trench progresses.

From the backfill mix selected, gradings, permeabilities and consolidation testing is then required to assess the stability and performance of the final wall.

The design investigations to allow the detailed design to proceed would involve geotechnical boreholes at intervals along the cut-off alignment, with sampling of the alluvium at regular intervals in each hole. These holes would also allow selection of the trench target depth along the alignment. The alluvial samples would then be categorised, and grading and permeability analyses undertaken on selected samples. Design of the backfill could then proceed with backfill mixes selected. A supplementary program, comprising grading permeability testing, would be undertaken at this post-design stage to confirm the preferred mix.

5.5 KEY CONSTRUCTION ASPECTS

Construction of a soil-bentonite wall involves the following key operations:

- Initial preparation of a slurry mixing and cleaning area and installation of the slurry tanks and pumps required for the trenching operation. The liquid slurry can be centrally stored in either 'silos' or in ground dams, with pumps used to deliver the slurry to the trench location;

- Opening and development of a borrow area for winning clay fines, if required for the backfill;

- Clearing and formation of mixing areas and the trench alignment, including construction of bunds to isolate the mixing area and contain any spillages. Benching of the trench alignment may be necessary to achieve a level trench surface.

- Initial excavation at one end of the trench, forming a length sufficient to allow a 1 in 10 beach slope to be commenced, plus a buffer zone between the base of the beach slope and the excavation face. This initial excavation is completed under a full head of bentonite slurry.

- Concurrent with the initial excavation, any clay fines required are spread adjacent to the trench and the excavated material placed over.

- Once the trench is sufficiently advanced, backfill mixing commences by spreading dry bentonite on the fines and the excavated material. This is then mixed thoroughly by tracking with a low ground pressure (LGP) dozer or similar. Once the mix grading is achieved and the material thoroughly mixed, it is pushed into the top of the trench from the end, where it will form a beach slope to the base of the trench.

- The excavating, mixing and backfill process then continues for the full length of trench. These concurrent operations need to be adjusted continuously to maintain a constant slurry volume and to ensure sufficient backfill is placed to keep up with the excavation. The bentonite slurry is also recycled frequently to clean it of trapped soil particles from the excavation process.

- On completion of backfilling, the trench backfill is given a period to consolidate under its own weight. This could be up to 1 to 2 months, with settlements of 500 mm to 800 mm possible.

- On effective completion of consolidation, the upper 1m or so of the backfill is usually excavated and the key detail and levee construction can commence over the backfill.

- Final clean up will require disposal of the bentonite slurry, and clean up of the mixing areas of remaining backfill.

Mobile plant involved in the construction operation would include the long reach excavator, crane and clamshell and LGP dozer, all of which remain within the trench alignment area during construction. In addition, scrapers or trucks plus a loader or dozer would be required to operate the clay borrow area, as well as transporting clay materials to the trench alignment. These items would also be used in levee construction and the initial site preparation. Road registered semi-trailers would need to access the site on a frequent basis to deliver bulk bentonite product, which is usually sourced from specialist bentonite mines. Unloading of the trucks could be accomplished using either the long reach excavator or the crane.

Non-mobile plant used in the construction will be mainly the bentonite tanks and pumps used for the slurry manufacture and delivery.

No major environmental concerns are associated with the construction, the only imported material being natural bentonite clay powder. The main concerns are containment of the bentonite slurry and the wet backfill mix. Adequate bunding on both sides of the alignment would be sufficient to avoid any spillages into the adjacent alluvial areas. Similarly all bentonite slurry pipelines would be laid within the bunded area.

Construction of the section of the low permeability barrier that crosses the existing alignment of Goonbri Creek would be timed to avoid periods when surface water flow in the creek is occurring. A temporary means of channelling flows around the in-creek construction activities (e.g. cutting and/or dam and pumping system) would be installed as a contingency measure if a rainfall event was to occur. Until such time that the permanent flood bund is completed and the permanent Goonbri Creek alignment is commissioned, this portion of the low permeability barrier would be protected from erosion scouring at the surface by rock armouring or equivalent.

5.6 PRELIMINARY CAPITAL COST ESTIMATION (SOIL-BENTONITE BARRIER)

A preliminary estimation of capital costs associated with construction of the soil-bentonite barrier, as described in **Section 5.4** has been undertaken based on unit cost rates for a range of works activities and resources, as listed below:

(i) Project establishment

(ii) Site preparation works

(iii) Trench excavation

(iv) Soil-bentonite slurry mixing and trench backfilling

(v) Clean up and dis-establishment

(vi) Engineering, Procurement and Project Management

These rates have been compiled based on past experience with similar construction projects, or review of current civil works costing handbooks and first principles costing analysis. Quantities for construction works have been estimated based on concept design works as presented on **Drawing 003** and **Drawing 004**, and on soil-bentonite barrier wall details described in **Section 5.4**.

A list of relevant aspects of capital cost estimation is provided below, with summary costs included in **Table 6**.

- Establishment of temporary buildings, fixed plant/amenities and connection of services, as required, and removal on completion including mobilisation of plant to site. On site supervision (including vehicle hire), set out and survey of works, payment of insurances and other fees, and implementation of appropriate management systems. Includes maintenance of temporary works, and rental and maintenance of temporary buildings and amenities etc, as required.

- Site preparation works including clearing and stripping along alignment of trench. Installation of stormwater management works, to divert clean water around construction area. Construction of haul roads and bentonite stockpile hardstands. Assumes availability of clay fill borrow areas within mine site.

- Earthworks to bench alignment for access purposes and to form slurry mixing basins, including embankment construction (completed as a staged operation). Embankment construction materials to be sourced from on-site borrow excavations.

- Trenching, assuming upper 15m depth to be excavated using long reach excavator, and lower depth by mechanical clamshell.

- Slurry mixing operation within prepared basins based on 20% (by weight) of imported clay, 5% (by weight) of bentonite and the balance by site soils (excavated by trench).

- Trench backfilling using dozer, and with provision for tremie placement (in sequence with trench excavation)

- Clean up of site on completion including removal of all temporary works and provision of as-constructed drawings, and returning site to pre-existing condition (in preparation for flood levee construction, as described in **Section 6.0**). Dis-establishment, including removal of all plant.

Table 6 – Preliminary Capital Cost Estimate for Soil-Bentonite Barrier

Item No	Description	Costing Basis	Capital Cost
1.0	Establishment	Assume establishment, supervision and overheads for 40 week construction program.	$460,000
		Assume mobilisation and demobilisation of suitable plant for construction purposes, required for trench excavation and general site works, including mixing basin construction	$130,000
		Set up for bentonite slurry mixing	$200,000
2.0	Site Preparation	Clearing and stripping (10 days duration)	$90,000
		Site benching and mixing basin preparation (total 75 days duration over construction period)	$450,000
		General site maintenance (50 days duration over construction period)	$280,000
3.0	Trench Excavation	Long reach excavator (<15m depth) x 40,000m^3 bank Total excavation period of 150 days	$1,200,000
		Clamshell excavation (>15m depth) x 22,000m^3 bank Total excavation period of 200 days	$2,100,000
		Bentonite supply for slurry (assume 5% mix)	$1,370,000
4.0	Trench Backfilling	Slurry mixing and backfilling	$1,940,000
5.0	Clean up		$230,000
6.0	EPCM (10%)		$850,000
7.0	Contingency (20%)		$1,800,000
	TOTAL		**$11,100,000**

It is noted that the construction of the flood levee, with layout and form as described in **Section 6.0**, will be completed as a separate activity following construction of the barrier. A lag period between construction programs is necessary, to enable settlement of the soil-bentonite slurry forming barrier wall to occur. This consolidation period is expected to be of the order of 1 to 2 months, although with a lag period of 6 to 12 months to be allowed to enable any residual settlement to take place.

SECTION 6.0 - PERMANENT GOONBRI CREEK ALIGNMENT

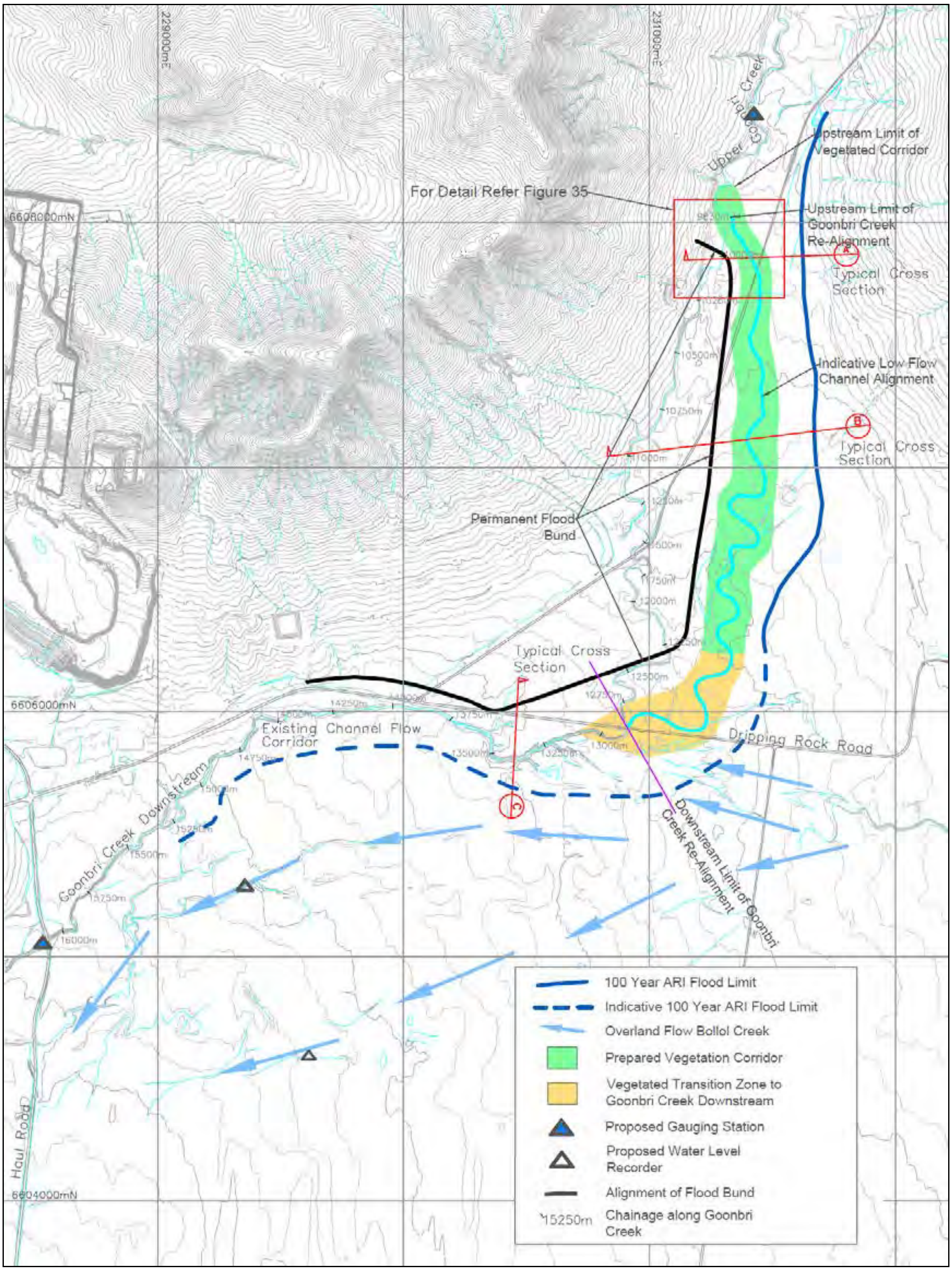
6.1 CONCEPT DESIGN DESCRIPTION

The eastern extension of the open cut associated with the Project will ultimately intersect the existing alignment of Goonbri Creek, which is located on the eastern edge of MLA 2 (refer **Drawing 001**). To facilitate the open cut extension, it is proposed that Goonbri Creek will be re-aligned further to the east in advance of mine development. This re-aligned section is herein referred to as the permanent Goonbri Creek alignment, with a nominal alignment shown on **Drawing 002**. Concept design works for the permanent Goonbri Creek alignment are presented in Gilbert and Associates (2011), with key design aspects outlined as follows:

Aspect	Description
General Characteristics	Goonbri Creek is formed on the eastern slopes of the Willowtree Range. The creekline passes the Project area to the east, then flows generally westward and south-westward, crossing the ROM coal transport route, and ultimately disperses as overland flow on the adjacent alluvial flats and the Namoi River floodplain.
Catchment Area	The Goonbri Creek catchment above the Dripping Rock Road (refer **Drawing 001**) is some 34 km^2. The catchment above the upstream end of the proposed Permanent Goonbri Creek Diversion is some 27 km^2. The length of the existing channel to be re-aligned is some 3.0 km.
Geology and Soils	The upper catchment of Goonbri Creek comprises Permian-aged Maules Creek Formation and Boggabri Volcanics. (These sequences are described in **Section 2.4**). The broad valley and outflow plain areas further downstream comprise predominantly undifferentiated alluvial sediments. These plains are Cainozoic aged deposits, which contain Holocene alluvial channels and overbank deposits of sand, silt and clay. Soils in the vicinity of the diversion comprise predominantly Sodisols, with the depth to gravel and sand generally being between 1 and 2 m. Soil dispersion is moderate, as is compaction severity.
Streamflow Characteristics	Goonbri Creek is subject essentially to ephemeral flow. Hydrological analysis indicates that the average number of flow days per year is some 55%. The 80^{th} percentile daily flow rate is estimated to be some 1.42 ML/day.
Design Objectives and Concept Outcomes	The key design objectives for the permanent Goonbri Creek alignment is as follows: • to provide a permanent alternative alignment for Goonbri Creek around the eastern edge of the proposed open cut extent; • to minimise disturbance upstream of the proposed open cut extent; • to revegetate the proposed creek alignment and thereby to extend the vegetated, higher value habitat conditions of the upper reaches of Goonbri Creek to the lower floodplain areas; and • to provide stability with increased vegetation along the corridor within and downstream of the re-aligned section. Design criteria would be to provide similar or lower flow velocities and flow energy levels within the diversion sections over a wide range of flows, with peak flow resulting from a 1 in 100 year ARI rainfall event to be contained within the vegetated corridor. The criteria for prevention of flood inundation of the mining area would be based on a probable maximum flood. The concept is to develop a wide revegetated floodway corridor, being separated from the proposed mining area by a flood levee. The corridor would incorporate a low capacity shallow channel, which has a proportionately similar form as the natural low flow channel. Under flood conditions, the flow would disperse onto the wide vegetated floodway where it would move slowly downstream through the vegetated overbank area limiting the flow energy on the central channel. The development approach would be to establish the vegetated floodway corridor in advance of the re-alignment. A key aspect of this floodway development would be construction of the flood levee. The diversion would then comprise excavation and stabilisation of the low flow channel.

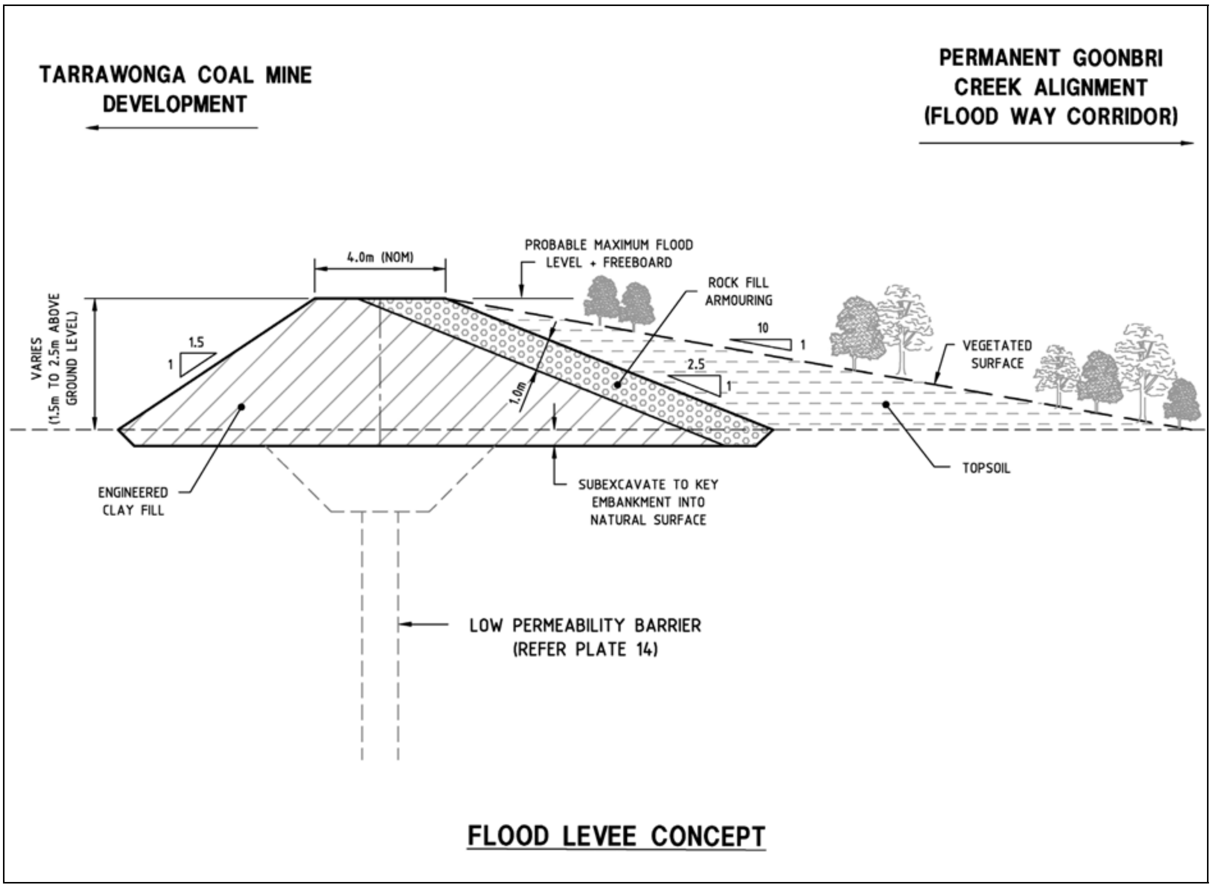
Plate 15 provides a concept for development of the floodway corridor (as an excerpt from Gilbert and Associates, 2011), showing the general proposed alignment and extent of the floodway. The alignment shown on **Plate 15** is reproduced on **Drawing 002**.

Plate 15 – Concept for Floodway Development (excerpt from Gilbert and Associates, 2011)



The alignment of the flood levee, constructed as part of the floodway, would generally coincide with the alignment of the low permeability barrier as described in **Section 5.0**. A section providing a concept for the barrier and flood levee is provided in **Plate 16.**

Plate 16 – Concept for Flood Levee and Low Permeability Barrier Construction



6.2 PRELIMINARY CAPITAL COST ESTIMATE (FLOOD LEVEE)

A preliminary estimation of capital costs associated with construction of the flood levee, as described in **Section 6.1** has been undertaken based on unit cost rates for a range of works activities and resources, as listed below:

(i) Project establishment

(ii) Site preparation

(iii) Flood Levee construction

(i) Clean up and dis-establishment

(vii) Engineering, Procurement and Project Management

These rates have been compiled based on past experience with similar construction projects, or review of current civil works costing handbooks and first principles costing analysis. Quantities for construction works have been estimated based on concept design works as presented on **Drawings 003** and **004**.

A list of relevant aspects of capital cost estimation is provided below, with summary costs included in **Table 7**.

- Establishment of temporary buildings, fixed plant/amenities and connection of services, as required, and removal on completion including mobilisation of plant to site. On site supervision (including vehicle hire), set out and survey of works, payment of insurances and other fees, and implementation of appropriate management systems. Includes maintenance of temporary works, and rental and maintenance of temporary buildings and amenities etc, as required.

- Site preparation including clearing and stripping within footprint of flood levee and adjacent topsoil placement.

- Earthworks associated with cut off key excavation and backfilling, and construction of clay fill portion of flood levee. Assumes that clay fill can be sourced from the mine (within a distance of 1.5km).

- Placement of rock fill armouring against outer batter of flood levee embankment, assuming that suitable material will be sourced from the mine (within a distance of 1.5km).

- Placement of topsoil against flood levee embankment

- Clean up and dis-establishment, including removal of all site works.

Table 7 – Preliminary Capital Cost Estimate for Flood Levee

Item No	Description	Costing Basis	Capital Cost
1.0	Establishment	Assume establishment, supervision and overheads for 12 week construction program.	$100,000
		Assume mobilisation and demobilisation of suitable plant for construction purposes, required for levee construction	$70,000
2.0	Site Preparation	Clearing and stripping (15 days duration)	$340,000
		Subexcavation beneath flood levee embankment (assume 0.5m depth across embankment footprint)	$136,000
3.0	Flood Levee Construction	Clay fill construction to form flood levee embankment, including cut off excavation and backfill	$1,490,000
		Rock fill armouring placement on external batter	$230,000
		Topsoil placement against levee batter	$590,000
4.0	Clean up		$50,000
5.0	EPCM (10%)		$300,000
6.0	Contingency (10%)		$330,000
	TOTAL		**$3,630,000**

This costing assumes that sufficient topsoil material will be available from site preparation works and available stockpiles from mine pre-strip works to satisfy requirements. Note also that this costing excludes revegetation works.

6.3 PRELIMINARY CAPITAL COST ESTIMATE (LOW FLOW CHANNEL)

A preliminary estimation of capital costs associated with construction of the low flow channel, as described in **Section 6.1** has been undertaken based on unit cost rates for a range of works activities and resources, as listed below:

(i) Project establishment

(ii) Site preparation

(iii) Low flow channel construction (including rockfill and finishing works i.e. revegetation)

(ii) Clean up and dis-establishment

(vii) Engineering, Procurement and Project Management

For the purposes of this costing, it is assumed that the works proposed would be completed by a civil earthworks contractor. The basis for costing has been unit cost rates derived from first principles analysis using rates for plant, material and labour obtained from current civil works costing handbooks and recent project experience. Note that engineering design works are required to provide the basis for definitive capital cost estimation for these works.

A list of relevant aspects of capital cost estimation is provided below, with summary costs included in **Table 8**.

This costing assumes that sufficient topsoil material will be available from site preparation works and available stockpiles from mine pre-strip works to satisfy requirements.

Table 8 – Preliminary Capital Cost Estimate for Low Flow Channel

Item No	Description	Costing Basis	Capital Cost
1.0	Establishment	Assume establishment, supervision and overheads for 75 day construction program	$40,000
		Assume mobilisation and demobilisation of suitable plant for construction purposes, required for low flow channel construction and allowing for accommodation and messing.	$110,000
2.0	Site Preparation	Clearing and stripping (assume 20 hectares)	$90,000
		Sub-excavation for drainage cut-off (assume 6,000 m^3)	$81,000
3.0	Low Flow Channel Construction	Excavation to form channel within upper portion with spoil used to form swales in lower portions (assumes haul distances ranging up to 500 m)	$546,000
		Excavation to form channel within lower portions (assumes haul distances ranging up to 1000 m)	$119,000
		Rock fill armouring placement on drainage cut-off embankment	$99,000
		Rock fill placement for check dams within low flow channel alignment (assumes 2,200 m^3)	$7,000
		Revegetation of upper and lower channel sections (assumes hydromulching plus tubestock planting)	$451,000
		Revegetation of drainage cut-off embankment (assumes hydromulching)	$129,000
4.0	Clean up		$10,000
5.0	EPCM (15%)		$252,000
6.0	Contingency (10%)		$193,000
	TOTAL		**$2,127,000**

SECTION 7.0 - REFERENCES

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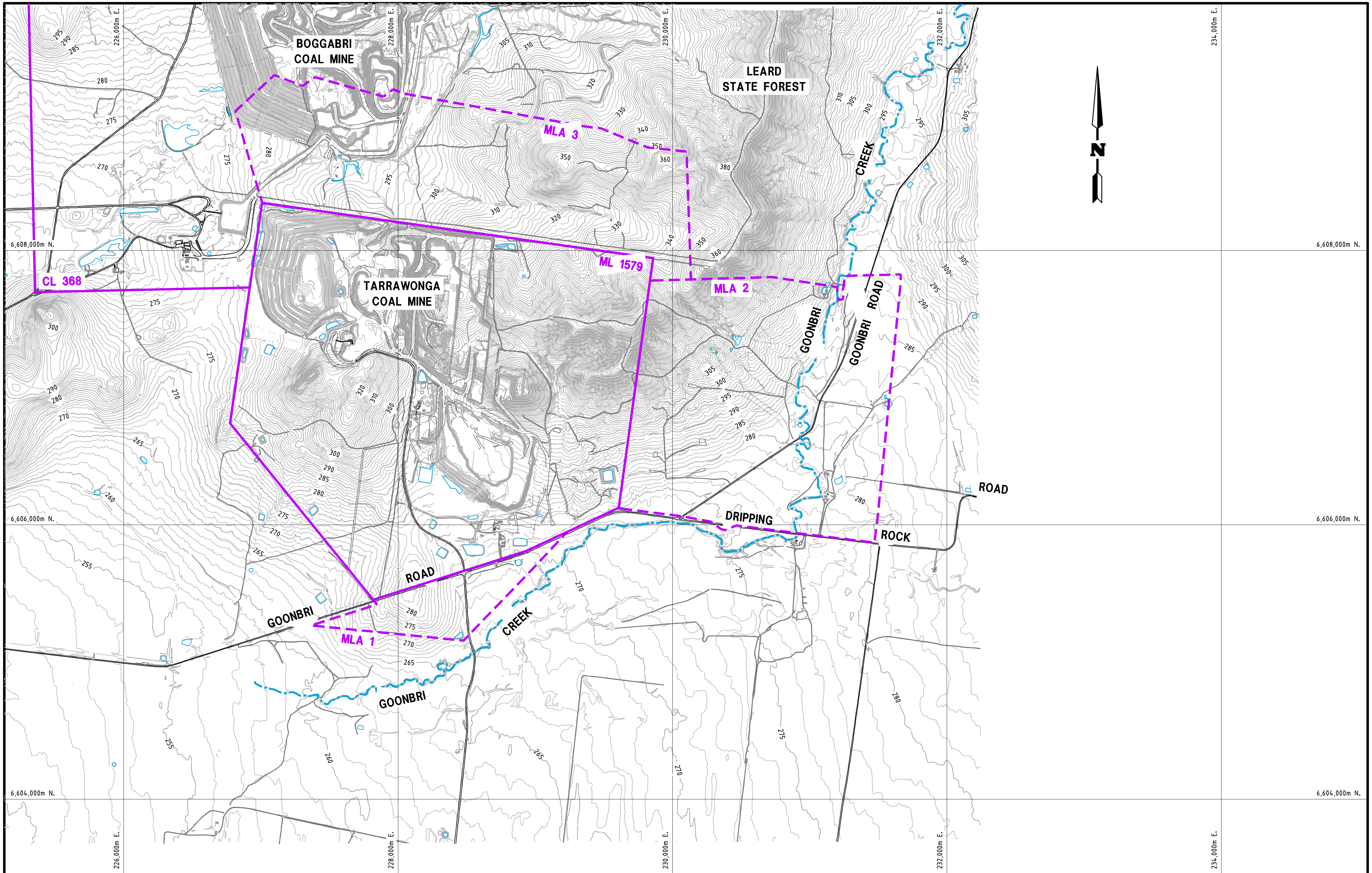
DRAWINGS



REV	DESCRIPTION	DATE	APPROVED
A	FOR REVIEW	16-09-11	ASW

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DESIGNED		SCALE	1: 25,000
DRAWN	RJS	LEVEL DATUM	
CHECKED	ASW	SHEET SIZE	A3
APPROVED		CAD FILE	0353-RST-003-003A
DATE	SEPT. 2011	ISSUE	REVIEW

CLIENT: TARRAWONGA COAL PTY LTD

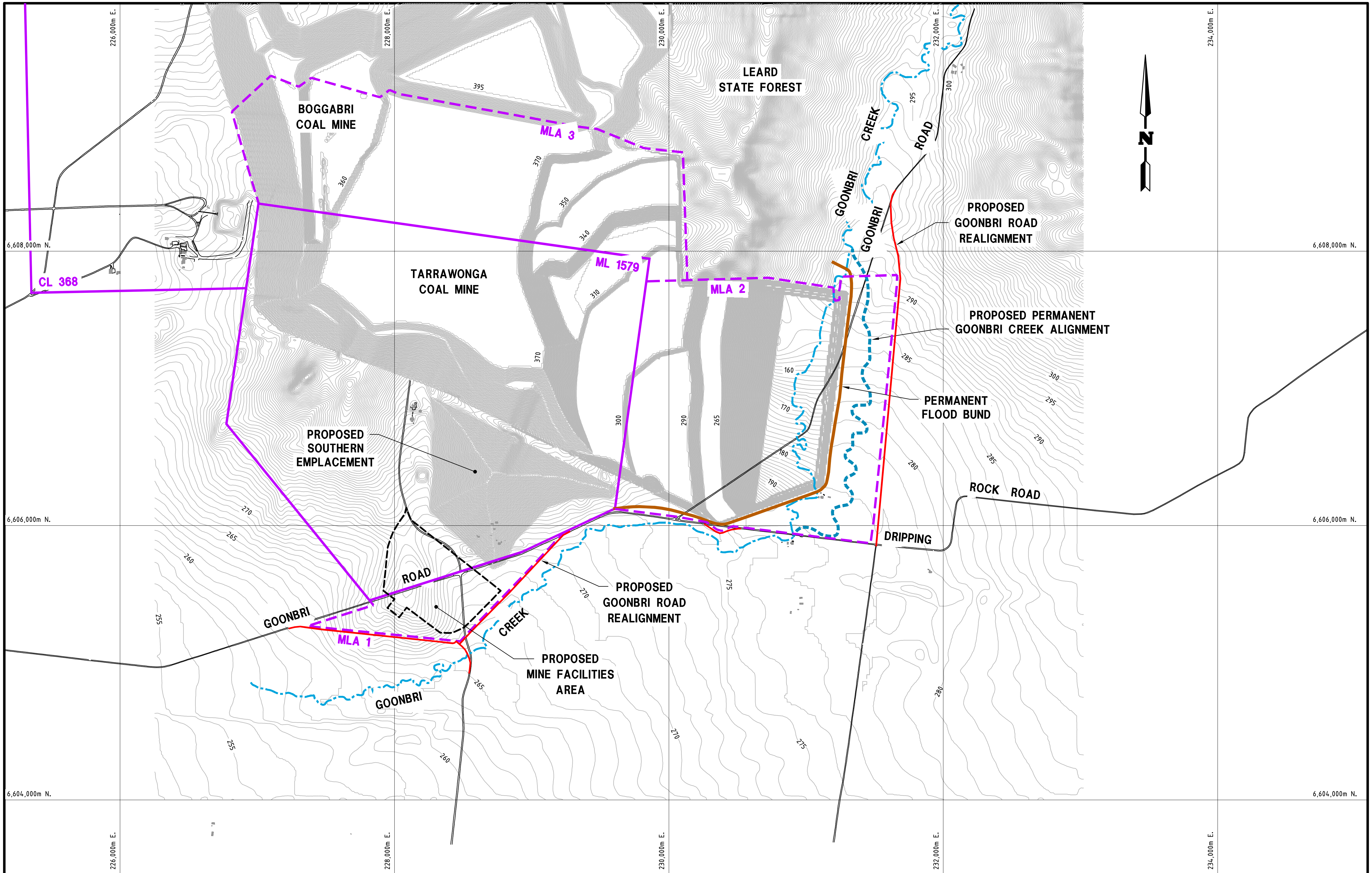
PROJECT: TARRAWONGA COAL PROJECT
CONCEPT DESIGN FOR LOW PERMEABILITY BARRIER

TITLE: SITE LAYOUT PLAN SHOWING
EXISTING CONDITIONS

JOB NUMBER: 0353-RST-003

DRAWING: 001

REV: A



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DESIGNED	SCALE 1:25,000	CLIENT	TARRAWONGA COAL PTY LTD
DRAWN RJS	LEVEL DATUM	PROJECT	TARRAWONGA COAL PROJECT CONCEPT DESIGN FOR LOW PERMEABILITY BARRIER
CHECKED ASW	SHEET SIZE A3		
APPROVED	CAD FILE 0353–RST–003–004A	TITLE	PROPOSED END-OF-MINE LIFE (YEAR 2029) DEVELOPMENT CONDITIONS
DATE SEPT. 2011	ISSUE REVIEW		

JOB NUMBER 0353–RST–003

DRAWING 002

REV A