February 2010

RASP MINE, BROKEN HILL, NSW

TAILINGS STORAGE FACILITY FEASIBILITY DESIGN

REPORT

Submitted to:
Broken Hill Operations Pty Ltd
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Report Number: 087611001 012 R Rev3

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1.0 INTRODUCTION

Broken Hill Operations Pty Ltd (BHOP) commissioned Golder Associates Pty Ltd (Golder) to prepare feasibility level designs for tailings storage facilities (TSFs) at the Rasp Mine in Broken Hill, New South Wales. Should the designs proceed to construction, a "For Construction" design of the TSFs would be prepared.

The mine is located on Consolidated Mine Lease 7 (CML7) which has been the subject of mining since 1887. BHOP has indicated the life of mine for the Rasp Project is 10 years.

2.0 BACKGROUND

Golder prepared a scoping study and preliminary design in June 2007 to assess tailings storage options for the Rasp Project (Golder ref: 077611001/006). The report was revised in November 2007 (ref: 077611001/020a) to include input from BHOP and Abesque Engineering and Construction Ltd (Abesque) and the results of a preliminary geotechnical investigation at the existing tailings storage facility (TSF). Reports presented in December 2007 (ref: 077611001/030 and 077611001/031) provided revisions to the preliminary design, based on feedback from BHOP.

An additional geotechnical investigation at the existing tailings dam was carried out in January 2008 and results of both stages of investigation were presented in a report dated March 2008 (Golder ref: 087611001 001 R Rev0).

An updated scoping study report (ref: 087611001 008 Rev0) was prepared in March 2008 to reflect the results of the additional geotechnical investigation and a revised production schedule. The Feasibility Design report was prepared in May 2008 (ref: 087611001 012 Rev0) and is now updated (ref: 087611001 012 Rev3) to include the following design revisions:

- Revised embankment geometry to facilitate installation of a geomembrane on the upstream slope of the TSF-1 embankment raise.
- Inclusion of an interceptor filter zone in the TSF-1 embankment raise.
- Revised dust management plan to include installation of a spray system, comprising sprinklers around the perimeter of the TSF-1 cells and a polymer and water mixture to form a crust over the tailings during periods of inactive tailings deposition.

3.0 SUPPLIED INFORMATION

The following information was supplied by BHOP as the basis of the tailings storage design.

- General history of mine site.
- Proposed method of mining and indicative timeframe.
- Forecast monthly production schedule (dated May 2009). Total tailings production is approximately 5.12 Mt of which 2.47 Mt would be used for underground hydraulic backfill and 2.65 Mt would be stored in surface facilities.
- Aerial survey dated January 2000.
- Aerial photograph of the site, indicating residential properties in close proximity to the mine.
- Layout plans supplied during a project meeting in April 2007.

In addition to the above information, observations were made by representatives of Golder during site visits.

4.0 MINING HISTORY

Mining was carried out at CML 7, in Broken Hill, since 1887 by a number of companies including BHP, BH South and Minerals, Mining and Metallurgy Ltd (MMM) and Normandy Mining Inc.

Ore was recovered from open pit and underground operations, and most recently from the MMM Kintore Pit. Operations ceased in 1991 when Normandy Mining Inc. (NMI) purchased the lease. CBH Resources Ltd purchased the lease from NMI in 2000 and has undertaken surface drilling and underground drive development for the project. Broken Hill Operations is a wholly owned subsidiary of CBH Resources Ltd.

The Rasp Project is based on a large zinc, lead and silver resource of intermediate grade.

5.0 SITE CHARACTERISTICS

5.1 Topography

The project site has been the subject of extensive mining operations since the late 1800's. The site hosted the original "Broken Hill" orebody that was first mined by Broken Hill Proprietary Company Limited. The hill has been mined and has been replaced by numerous mine pits, waste rock storages and tailings storage sites. The existing conditions layout (as at January 2000) is presented in Figure 1, with some detail provided on the locations of previous mining activity. From discussions with mine personnel, it is understood that the survey detail in the area of the proposed tailings storage facilities is considered to be unchanged since the aerial survey of January 2000.

5.2 Geology

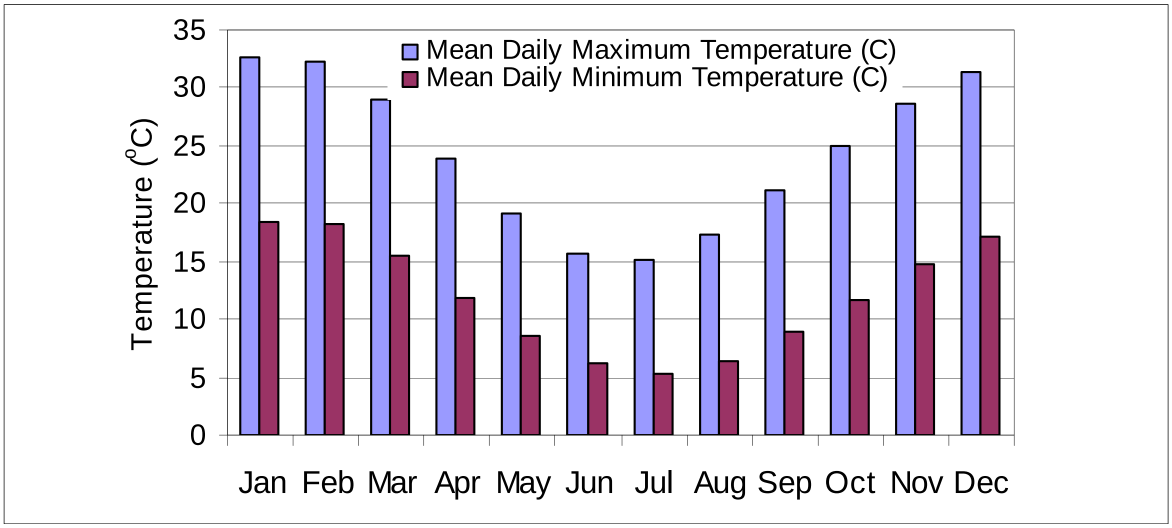
The Broken Hill orebody lies within the Hores Gneiss stratigraphic package which is part of the Broken Hill Group. Orebodies at Broken Hill occur in a 6 km to 7 km thick Willyama Supergroup which has been described as "*a highly deformed sequence of layered rocks which have been subjected to high to low grade regional metamorphism. Within the Willyama Supergroup the Broken Hill Group comprises a 500 m thick sequence of metasediments with minor felsic and basic gneiss, and a suit of volumetrically insignificant exhalative rocks, which include the sulphide orebodies"* **(Ref.1).**

5.3 Climate

Climatic data for the site was sourced from the Commonwealth Bureau of Meteorology website **(Ref.2)**. The closest weather station is at Patton Street (station ID 047007), and located within a few hundred metres of the mine site. The station is situated at elevation 315 m AHD, consistent with the average surface elevation of the mine site. It is understood from the Bureau website that rainfall observations from the Broken Hill Airport are included in the data set. No evaporation data is available for the Patton Street or Broken Hill Airport weather stations. Evaporation data for this study was sourced from the Stephens Creek Reservoir weather station (ID 047007), located approximately 16 km from the mine site.

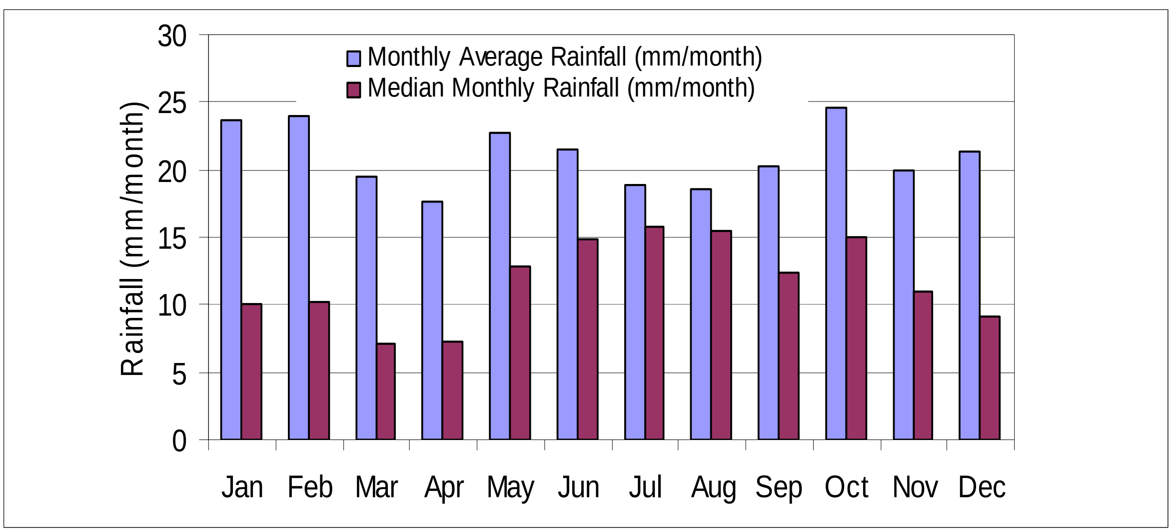
The site experiences hot summers and cold winters, with mean daily maximum temperature exceeding 32ºC in January and falling to about 15ºC in July. Mean daily minimum temperatures may vary from about 18 ºC in January to 5 ºC in July. The variation in seasonal temperatures is illustrated in Chart 1.

Chart 1: Mean Daily Minimum and Maximum Temperatures for Broken Hill (1891 to 2007)



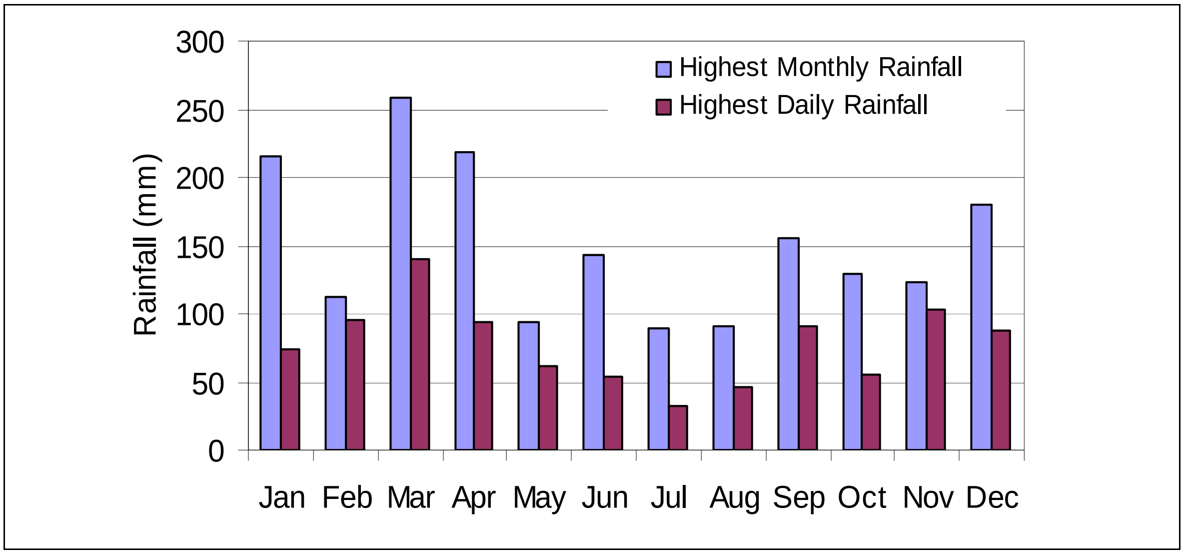
Rainfall is spread throughout the year and there is no notable temporal distribution of average rainfall for Broken Hill, although rainfall is more likely during the cooler months of the year. During the hotter summer months, rainfall is associated with storm activity, whilst during the winter months rainfall is influenced by low pressure systems in the Southern Ocean. The average annual rainfall for Broken Hill is 253.3 mm and average monthly and median rainfall are illustrated in Chart 2. The monthly average and median rainfall distribution reflects the impact of summer storms on the rainfall experienced in Broken Hill.

Chart 2: Monthly Average and Median Rainfall for Broken Hill (1889 to 2008)



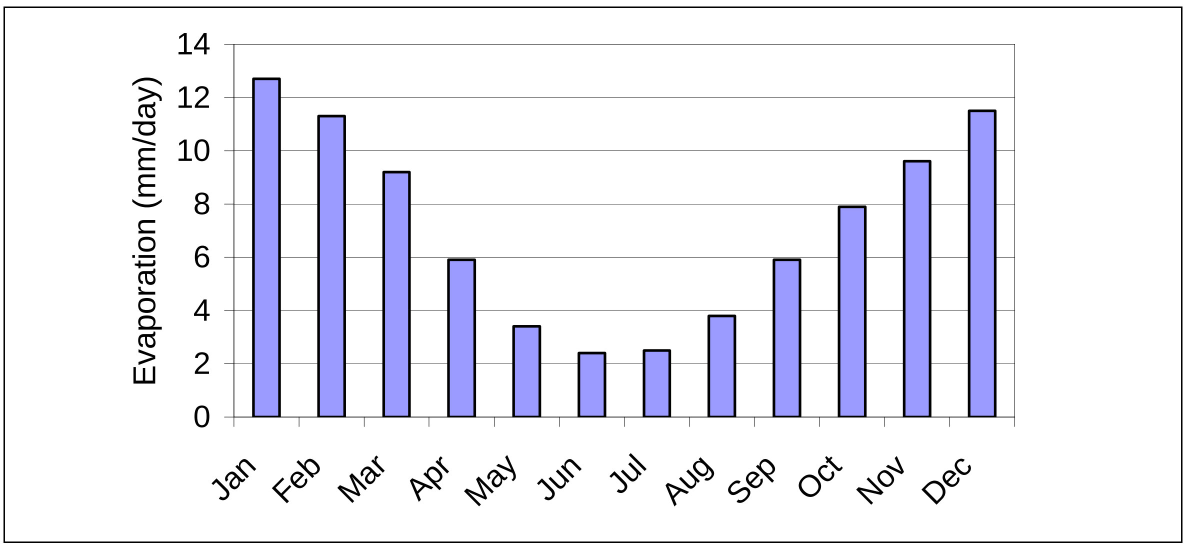
The highest monthly and highest daily rainfall are presented in Chart 3.

Chart 3: Highest Monthly and Highest Daily Rainfall for Broken Hill (1889 to 2008)



Mean annual evaporation data for the Stephens Creek Reservoir weather station is 2,614.2 mm. The annual mean daily evaporation is 7.2 mm, varying from 12.7 mm in January to about 2.4 mm in June. The seasonal variation is presented in Chart 4.

Chart 4: Seasonal Variation in Evaporation for Broken Hill (Stephens Creek Reservoir, 1975 to 2005)



The statistics show that mean annual evaporation exceeds precipitation by a factor of approximately 10, although this factor varies from approximately 17 in December and January to about 3 in June.

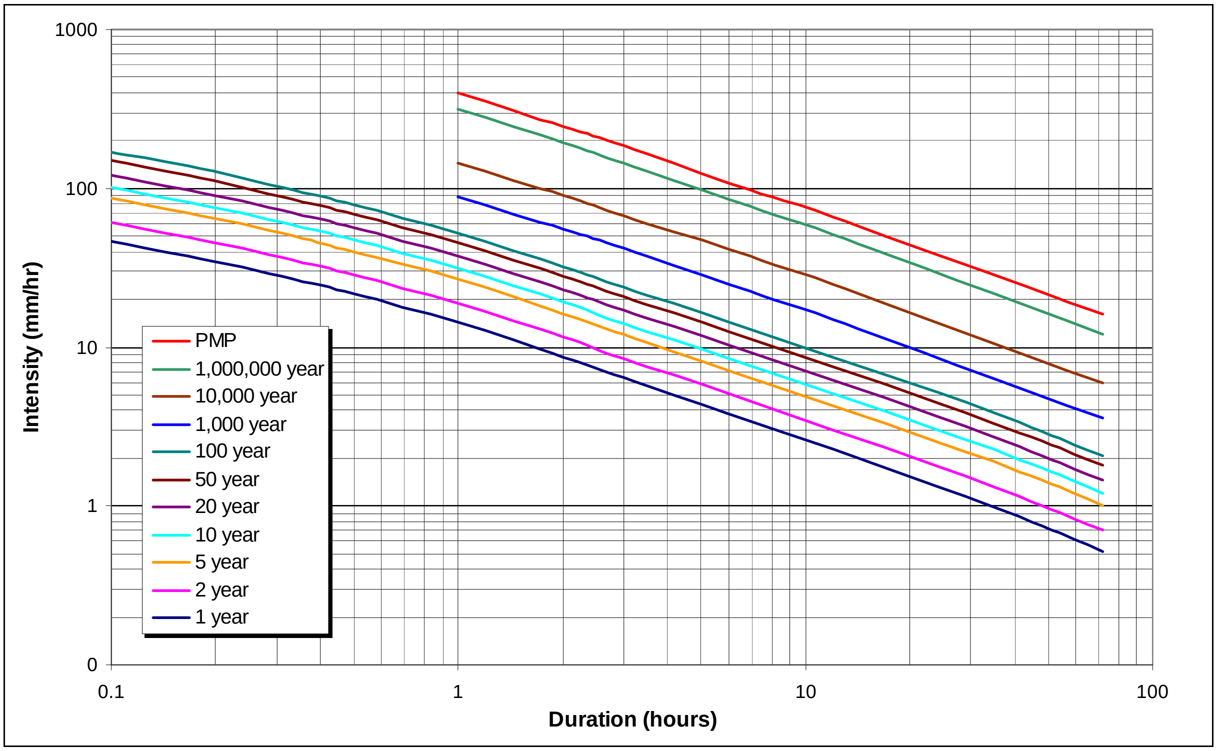
5.4 Intensity-Frequency-Duration Data and Design Storm Events

Intensity-Frequency-Duration (IFD) curves were developed for the site using AusIFD software, in accordance with Australian Rainfall and Runoff (AR&R) procedures **(Ref.3)**.

The Probable Maximum Precipitation (PMP) plot has been developed using the procedures of Australian Bureau of Meteorology publication "Maximum Precipitation in Australia, Generalised Short Duration Method (GSDM)" **(Ref.4)**. Based on the GSDM, the critical storm duration for the PMP is limited to 3 hours.

The IFD curves, including estimates of the 1:1,000 ARI, 1:10,000 ARI, 1:1,000,000 ARI and PMP are plotted in Chart 5.

Chart 5: Rainfall Intensity-Frequency-Duration Curves



5.5 Seismicity

Based on a qualitative risk assessment carried out for the proposed tailings storages, it is considered that the TSF-1 storage classifies as a category High B facility, in terms of the consequences of a breach of the facility. Because of its location and configuration (i.e. tailings will be contained within the pit and there is no requirement for embankment construction during the proposed life of mine) it is considered that TSF-2 classifies as a "Low" hazard facility.

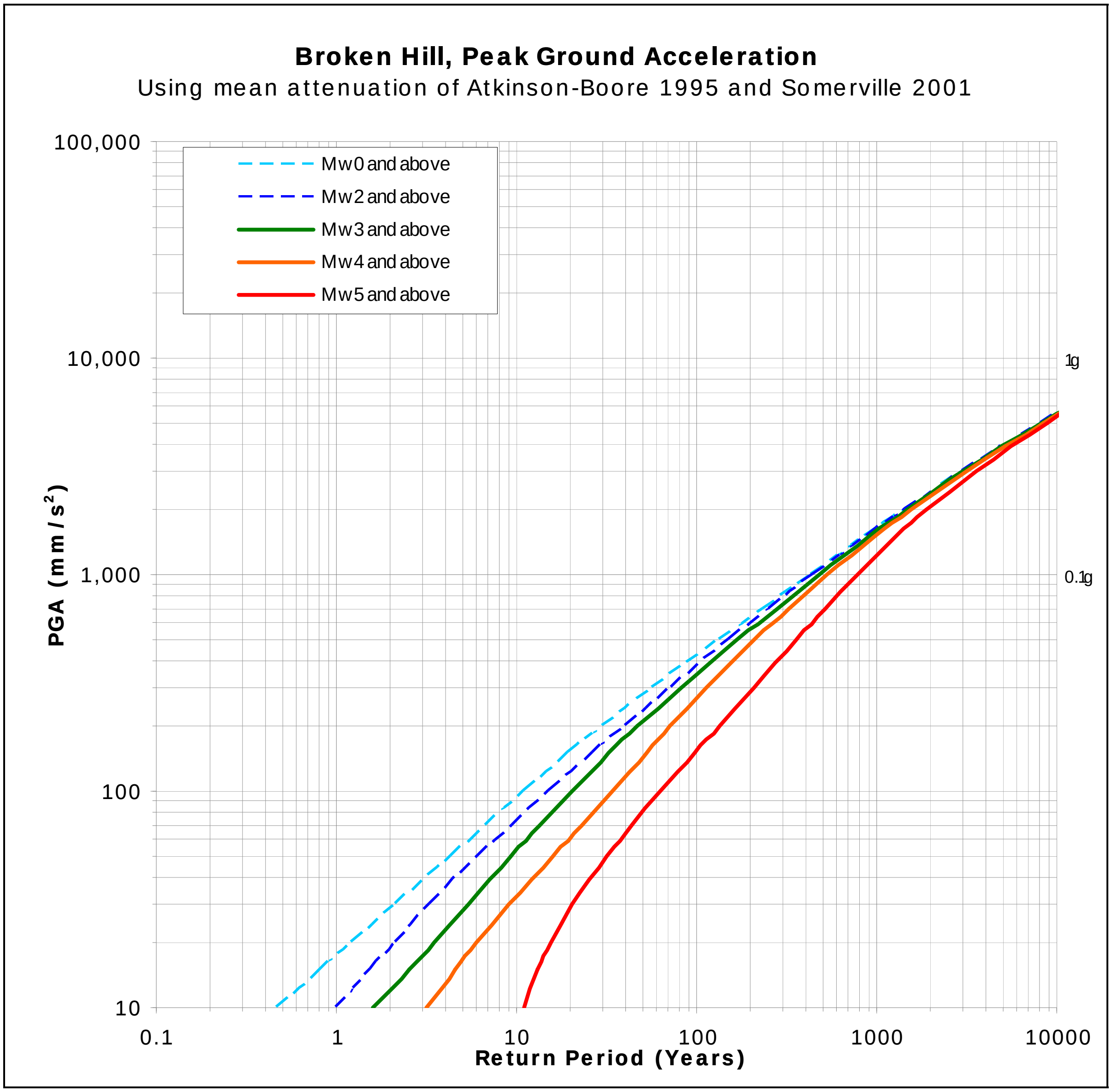
For a High B category facility, the 1 in 475 year event is typically adopted for the operating basis earthquake (OBE) and the 1 in 10,000 year event (equivalent to a 0.5% chance of exceedence in 50 years) is adopted for the maximum design earthquake (MDE). **(Ref.5)**

For the initial Scoping Study reports, the Australian Standard AS 1170.4, "Minimum Design Loads on Structures – Part 4: Earthquake Loads (AS 1170.4, 1993) was used for assessment of the seismic risk for Broken Hill. The Earthquake Hazard Map for New South Wales indicates that for a 10% chance of exceedence in 50 years (i.e. 1 in 475 years) the Peak Ground Acceleration (PGA) coefficient for Broken Hill is 0.045g.

For the feasibility design, a seismic hazard assessment was undertaken to provide a site specific indication of the earthquake risk for TSF-1. This assessment was undertaken by the Seismology Research Centre (SRC) which is a division of Environmental Systems & Services. The report is presented in Appendix C.

The mean attenuation of the Atkinson-Boore 1995 and Somerville 2001 methods were adopted by SRC to determine PGA values over a range of return periods. The curves presented in Chart 6 are for earthquakes of differing moment magnitude (Mw) for a 50 year design life. Peak Ground Accelerations for "Mw5 and above" were adopted for the preliminary design. This has been recommended by SRC, as earthquakes smaller than Magnitude 5 rarely cause any damage.

Chart 6: Peak Ground Acceleration Recurrence



The design life of TSF-1 is considerably less than 50 years, and it therefore considered reasonable to consider seismic loadings for return periods relevant to the probability of exceedence over the design life. Operation of TSF-1 will be over a period of approximately 4.25 years and it is anticipated that closure, including significant consolidation of tailings adjacent to the embankment, would occur within less than 4 years of cessation of tailings placement. For stability analysis purposes, a design life of 8 years has been adopted. The curves presented in Chart 7 indicate the relationship between PGA and Probably of Exceedence for a 50 year design life and also an 8 year design life. The PGA selected for the analysis are summarised in Table 1.

Chart 7: Design Life Reduction Chart for TSF-1

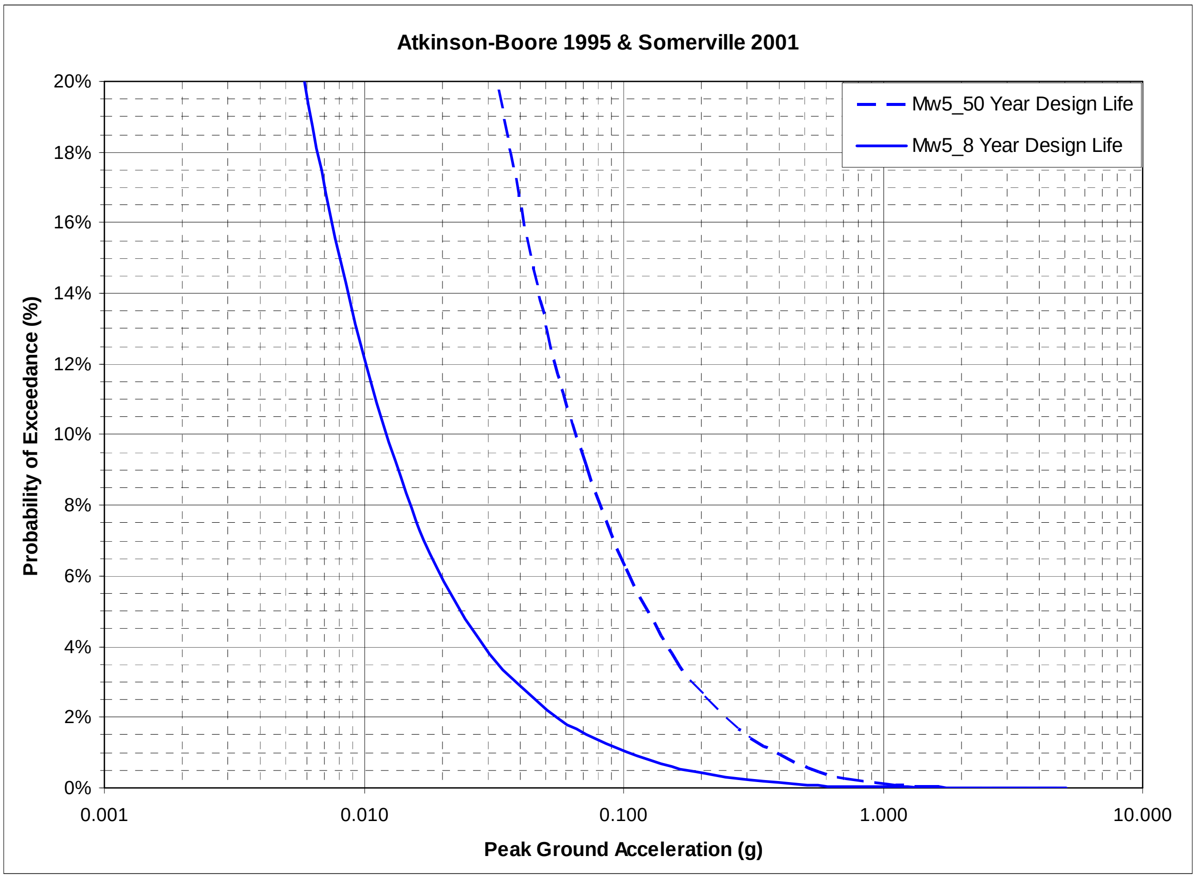


Table 1: Adopted Peak Ground Acceleration Coefficients for TSF-1

Return Period (years)	**Peak Ground Acceleration (m/sec^2)**
10% in 8 years (OBE)	0.013
0.5% in 8 years (MDE)	0.2

6.0 ORE TREATMENT PROCESS

BHOP and Abesque have indicated that the run of mine lead-zinc ore, from the underground operation would be crushed in a three stage crushing plant to minus 15 mm and stored in a 2,250 tonne fine ore bin. Crushed ore would be recovered from the fine ore bin at nominal rate of 95 tonnes per hour (tph) and fed to a three stage milling circuit. The milling circuit would grind the ore to less than 200 microns particle size.

The ground ore would be fed to the lead flotation circuit to produce a lead concentrate. The lead concentrate would be filtered and loaded into rail wagons for delivery to the smelter. The lead flotation tail would then be processed in the zinc flotation circuit to produce a zinc concentrate which is filtered and loaded into rail wagons for delivery to the export port.

The tailings produced from the zinc flotation circuit would be processed in the hydraulic fill plant to separate out coarse particles for use as fill material underground. The remaining predominantly fine particles are thickened and stored in either TSF-1 or TSF-2.

7.0 LIFE OF MINE PRODUCTION SCHEDULE

The current mine life is based on a resource of 6.2 million tonnes (Mt) of ore, producing about 1.1 Mt of concentrate and 5.1 Mt of tailings.

After the initial start-up period, it is proposed to split the tailings stream between underground backfill and surface disposal. At this stage the proposed split is 50% underground and 50% to surface disposal.

BHOP has developed a forecast production schedule, which is summarised below:

- For the initial 4 months, all tailings are to be stored in a surface facility. Forecast mill feed rate during this period commences at 30,000 tonnes per month (tpm) and increases to 35,000 tpm.
- For the next 4 months, 75% of tailings are to be stored on the surface with the remainder going to underground backfill. Forecast mill feed increases to 40,000 tpm, equivalent to 480,000 tonnes per annum (tpa).
- From month 9 to the end of mine life, tailings are to be split equally (50/50) between surface storage and underground. Forecast mill feed gradually increases to 62,500 tpm (equivalent to 750,000 tpa) over the next approximate 5 years, with the mill feed at 62,500 tpm for the final four years.

The forecast production schedule prepared by BHOP is presented in Appendix A. A summary of annual surface tailings production is presented in Table 2.

Table 2: Summary of Annual Surface Tailings Production

Year	Surface Tailings Production (tonnes)	Cumulative Surface Tailings Production (tonnes)
1	273,281	273,281
2	195,938	469,219
3	195,938	665,156
4	216,563	881,719
5	247,500	1,129,219
6	278,438	1,407,656
7	309,375	1,717,031
8	309,375	2,026,406
9	309,375	2,335,781
10	309,375	2,645,156

8.0 TAILINGS STORAGE SITE SELECTION AND OPERATING STRATEGY

8.1 Storage Locations

The topography of CML 7 has been extensively altered since mining commenced and little undisturbed ground remains on the site. The Scoping Study identified two suitable sites on the mining lease for life of mine tailings storage. The storage strategies are:

- Raise existing Tailings Dam to form a new Tailings Storage Facility (TSF-1), and
- Fill existing Blackwood Pit to form TSF-2.

An indicative filling schedule for TSF-1 and TSF-2 is summarised in Table 3.

Table 3: Indicative Surface Tailings Storage Schedule

Facility	Tailings (tonnes)	Estimated Storage Volume at 1.3 t/m^3 (m^3)	Estimated Storage Elevation (RL m)
TSF-1	970,000	750,000	331.0
TSF-2	1,680,000	1,290,000	295.5
	2,925,000	2,250,000	307.0

A brief description of these sites, including the proposed method of storage is provided below. The site locations are shown on Figure 2.

8.1.1 Existing Tailings Dam and Blackwood Pit

8.1.1.1 Tailings Dam Background

The existing tailings dam comprises two cells of approximately equal surface area and a maximum height of approximately 20 m. The tailings dam was constructed in the early 1980's and was closed in 1991.

Based on discussions with mine personnel, we understand that the tailings dam is located over a historic tailings dam where tailings were re-mined during one of the previous mining campaigns. The existing tailings dam was constructed with a starter embankment of about 2 m to 3 m height and using remnant tailings. The starter embankment was formed using an excavator and was compacted by tamping with the excavator bucket and track rolling only.

The facility was progressively raised by constructing embankments using old tailings materials and using the upstream construction method. The raise embankments were generally 2 m to 3 m in height, depending on depositional requirements and had a crest width of 3 m to 4 m. Each raise was generally stepped in 1 m to 3 m from the outside edge of the underlying embankment crest. The current crest elevation of the tailings dam is approximately RL 322.0 m.

The existing upper surface of the tailings has been covered with a nominal 0.5 m thick layer of slag, sourced from stockpiles at the old roaster areas and the tailings dam side slopes have been covered with mine waste rock. We understand the mine waste rock was tipped from the crest of the facility, resulting in a mine waste rock thickness of 1 m to 2 m at the crest and upwards of 3 m to 4 m at the toe.

Excess tailings process water and stormwater was discharged from the surface of the tailings in each cell via centrally located gravity decants. The decants comprised 300 mm diameter by either 150 mm or 300 mm (approximately) long sections of steel pipe joined by bolting together through flanges. Access to the decants was via a timber causeway which has since been removed. The decants discharged via gravity to the Horwood Dam which is located to the north of the tailings dam. The decant outlets could not be located during a site visit by a Golder representative during the second stage geotechnical investigation in January

2008. It is possible that the outlets were covered by the waste rock placed on the downstream slope of the tailings dam. The decant inlets would need to be grouted prior to recommissioning of the tailings dam.

8.1.2 Blackwood Pit

The Blackwood Pit is located to the north west of the existing tailings dam and will be used as TSF-2 as part of the tailings storage strategy. The depth of the pit varies from about 40 m at the southern end to about 70 m at the northern end. Portions of the northern part of the pit have been backfilled with mine waste rock.

8.2 Tailings Storage Strategy

8.2.1 Recommissioning of the Tailings Dam (TSF-1)

The crest area of the existing tailings dam is approximately 10 hectares. A waste rock storage is located to the west of the tailings dam and an old waste rock covered tailings dam is located to the south. The eastern embankment crest of TSF 1 is located adjacent to the mine lease boundary and to the north is the Horwood Dam. A valley known as “Mt Hebbard Gully” is located between to the south west corner of the facility and west of the old waste rock covered tailings dam. This is the location of the existing tailings dam spillway. This gully has not been considered for tailings storage as it will form part of the operational emergency spillway for Stage 1 of TSF-1.

Future storage of tailings on the tailings dam involves construction of an embankment on the old tailings, with a minimum offset of 10 m from the inside crest edge of the existing eastern embankment. A 6 m high starter embankment would be constructed followed by a subsequent 4 m high raise. The embankment would be constructed using selected waste rock to a maximum elevation of RL 332.0 m. The starter embankment would be constructed to the full footprint of the proposed final embankment, therefore avoiding the need to construct any earthworks on freshly deposited tailings. Additional design information, including a proposed drainage system is presented in Section 10.3.

The eastern and northern slopes of TSF-1 would be buttressed by constructing a waste rock buttress against the toe of the slopes. The buttress would be about 7 m high and have a 10 m crest width.

Other than for the initial 8 months of production, it is assumed that 50% of tailings produced at the Rasp Mine will be stored on surface and 50% will be used as underground fill. BHOP has indicated a preference to initially store tailings in TSF-1 and then to store the remaining tailings production in TSF-2. It is understood that this staging is based on remnant ore reserves that will be mined from Blackwood Pit (TSF-2) in the early stages of the project.

Therefore, the proposed approach to surface tailings storage on the CML 7 lease, is to raise the existing tailings dam embankment from RL 322.0 m to RL 332.0 m, to form TSF-1. A 6 m high waste rock starter embankment would be constructed to RL 328.0 m, followed by a single 4 m high raise. Subject to additional stability analyses, the raise may be constructed in a single 10 m lift.

Deposition of thickened tailings would alternate between the two cells and tailings would be deposited into one cell for between one to two weeks before switching to the second cell to allow the recently placed tailings in the first cell to drain and consolidate. Tailings would be deposited from spigots located on the embankments surrounding the cell and the tailings would form a beach sloping down from the spigots towards the west of the cell.

Supernatant water from the tailings and stormwater runoff would collect in a pond and be pumped from the surface of the cell to the decant dam. It is proposed to locate the decant dam on the waste rock storage to the west of TSF-1 as shown on Figure 2. Based on an overall increase in wall height of 10 m and the assumed tailings properties presented in Section 9.0, the TSF-1 could accept tailings from the first 4.25 years of production.

A Stage Capacity Curve has been prepared for TSF-1 and is presented as Figure 10. The curve indicates an average final storage (tailings) level of approximately RL 331.0 m after about 4.25 years of operation. The average final storage level assumes a flat tailings surface whereas the tailings beach will have a slope towards the supernatant water pond. The tailings against the perimeter embankment slope would be at approximately RL 331.5 m. This would provide sufficient capacity to store the design rainfall storm event and provide an average operational freeboard of 0.5 m. Assuming an average final tailings elevation of RL 331.0 m, the facility will have capacity to store the flood water resulting from a 3 hour PMP storm event.

8.2.2 Storage in Blackwood Pit (TSF-2)

Once TSF-1 has been filled to its freeboard capacity, tailings deposition would be switched to TSF-2. A capacity assessment of TSF-2 (Blackwood Pit) based on the January 2000 survey, indicates sufficient capacity to store the forecast production of mine surface tailings. A Stage Capacity Curve for TSF-2 is presented as Figure 11 and indicates a final storage level of approximately RL 295.5 m after tailings placement in TSF-2 for 5.75 years. Typical Sections through TSF-2 are presented on Figure 8.

Should there be an increase in the surface tailings production schedule, there is scope to fill TSF-2 above RL 295.5 m, as indicted in Table 3. If storage is required above RL 308.5 m, an engineered bund wall would be required near the northern end of the pit, on the western side. The stage capacity curve for TSF-2 was prepared assuming an average dry density of the tailings to be 1.3 t/m^3. Given the depth of the pit the tailings may consolidate to a higher density which would also increase the TSF-2 storage capacity.

We understand BHOP intend to mine remnant ore from the Blackwood Pit and mining would be carried out before the pit is used for tailings storage. BHOP would also assess and implement underground barricading and stope stability works under the pit if required.

9.0 TAILINGS CHARACTERISTICS

9.1 Existing Tailings

Geotechnical investigations were carried out in two stages by Golder to determine the characteristics of the existing tailings. The results of the investigations are presented in a report dated March 2008 (Golder reference: 087611001 001 R Rev0).

The first stage of investigation comprised the drilling of five boreholes from the surface of the tailings, performance of Standard Penetration Tests (SPT) at intervals in the tailings, recovery of disturbed samples of tailings, laboratory testing of the samples and installation of standpipe piezometers in four boreholes.

The second stage of investigation comprised eight Piezocone Penetration Tests (CPTu) to provide a continuous record of undrained shear strength and pore water pressure in the tailings. Samples were also retrieved for laboratory testing from probe holes adjacent to two of the CPTu test locations.

The investigations encountered sandy silt, silty sand and sand tailings to a thickness up to 21.6 m. The SPT results suggested the upper 6 m of tailings to be of soft/loose consistency and the underlying tailings to be stiff/dense. The CPTu results provided confirmation of this profile, provided strength parameters for the design of the embankment raises, and also indicated the presence of drained conditions near the perimeter of the embankment. Towards the centre of the facility, where finer particle sizes were encountered (due to segregation during deposition) the tailings were typically of lower strength and undrained.

Results of the first stage laboratory testing indicated in-situ moisture contents ranging from 19.6% to 33.4% in the upper 6 m of tailings and 11.3% to 30.5% in the underlying tailings. Second stage laboratory testing indicated in-situ moisture contents ranging from 4.8% to 20.4% (with the exception of 41.8% near the base) near the perimeter of the facility. Towards the centre of the facility, the moisture contents ranged from 30.1% to 44.0%.

9.2 Proposed Rasp Tailings

A sample of indicative total tailings was prepared by G & T Metallurgical in Canada from a sample of ore from the Rasp Mine. This sample was then tested by Golder Paste Technology Ltd (PasteTec) to determine the suitability of the cyclone underflow for use as underground backfill (cemented hydraulic fill). Testwork was also carried out on the overflow, which would typically be discharged to surface storage. The PasteTech laboratory report is presented as Appendix B.

9.2.1 Particle Size Gradation

The proposed total tailings grind size is P_{80} = 200 µm but this may be reduced to P_{80} = 150 µm. Total tailings would be deposited into the North and South cells for the first six months of operation after which the tailings would be cycloned and the coarse underflow used as backfill for underground workings. The cyclone overflow tailings would be deposited into the proposed surface storage facilities. Particle size distribution of the overflow tailings indicates P_{80} = 40 µm.

9.2.2 Solids Specific Gravity

Results of laboratory testing by PasteTec has shown a specific gravity of between 2.88 (overflow tailings) and 3.05 (underflow tailings).

9.2.3 Settled Density

We have assumed tailings will be thickened and discharged at a solids content of 50% (solids by mass relative to total mass). Based on this assumption, and experience with similar tailings materials, we anticipate the dry density of the tailings would be about 1.3 t/m^3 for surface disposal of tailings into TSF-1. Given the depth of the Blackwood Pit and assuming removal of supernatant water from the tailings surface, it is likely that the average dry density of the tailings in TSF-2 would be higher and possibly up to 1.5 t/m^3 by the end of mine life. For the design we have adopted a conservative dry density of 1.3 t/m^3 for the life of mine tailings storages.

9.2.4 Tailings Beach Slopes

Based on site observations and aerial survey, the existing tailings dam has beach slopes ranging from 1.5% to 2.5%, with the steeper beach slopes towards the centrally located decants. These beach slopes are probably steeper than would be achieved for the Rasp tailings because the proposed overflow tailings will probably be finer than previous tailings and the current beach slopes have undergone significant consolidation since deposition ceased. For design we have conservatively adopted a flat tailings beach slope for capacity assessments and 1.5% for freeboard assessment.

10.0 TSF DESIGN

10.1 Consequence Category Guidelines

The NSW Dam Safety Committee (DSC) guidelines on “Consequence Categories for Dams” (DSC13) were used to assess the consequence category for TSF-1 and TSF-2. DSC13 takes into account the ANCOLD “Guidelines on Assessment of the Consequences of Dam Failure” **(Ref.6).**

10.2 TSF-1 Consequence Category

The existing tailings dam is about 20 m high and is adjacent to the lease boundary. The nearest dwellings are about 100 m to the east of the facility, separated by a road and powerlines. There is also an active quarry approximately 500 m to the north east of the facility.

Based on criteria presented by DSC and potential occupation of nearby houses and cars on Eyre Street, we consider that the ‘population at risk’ (PAR) would be in the range of 11 to 100 people at any time. The severity of damage or loss in the event of a breach of the embankments is considered to be “Major”.

On this basis, the consequence category of the proposed TSF-1 is considered to be a “High B”.

10.3 TSF-1 Layout

TSF-1 will cover the existing tailings dam and part of the waste rock storages to the west and south as shown on Figure 2. The constraints on the layout are:

- The “Horwood Dam” located to the north of the facility is required for site stormwater management.
- The lease boundary to the east limits the potential for expansion in that direction.
- The “Mt Hebbard” waste rock covered tailings dam to the south provides a physical boundary.
- The “Mt Hebbard Gully” to the south west of the facility is required for site stormwater management.
- The “Old BHP Pit” and waste rock storage to the west of the facility limit expansion in that direction.

The embankment raise for TSF-1 will involve construction over the existing slag, adjacent to the perimeter embankment along the northern and eastern crest of the facility. The embankment raises would be keyed into Mt Hebbard. This is to limit the potential for development of preferential flow paths for seepage water from the TSF at the south east corner.

The recommissioned TSF will comprise two approximately equal cells separated by a dividing wall. The cells are shown on Figure 2 and designated as “South Cell” and “North Cell”. Tailings deposition will be cycled between the two cells to facilitate consolidation to achieve appropriate storage in the tailings cells. Design layouts and sections through the proposed embankment including proposed staged construction, are presented on Figures 3 to 7.

As indicated on the embankment design section in Figure 6, the geomembrane will be installed on the upstream face of the embankment and also extend 20 m over the floor. The geomembrane will be installed along the north and eastern sides of the facility (i.e. not the dividing wall). Where the dividing wall meets the eastern embankment, the geomembrane will extend through the dividing wall with a cushion geotextile and select fill over the geomembrane for damage protection against the rockfill. A cushion geotextile will also be installed over the geomembrane for the Stage 2 raise, as a 3 m wide horizontal width of fill will be placed over the geomembrane. This configuration facilitates staged geomembrane installation.

Three parallel Geocomposite strip drains will be installed on top of the geomembrane along the upstream toe of the embankment along the eastern and northern side, as indicated on Figure 7. The strip drains will facilitate rapid consolidation of the tailings adjacent to the embankment. Drainage from the strip drains will be collected in sumps located in the north east corner of each cell.

Emergency spillways will be constructed in each cell to allow for discharge of water in a controlled manner in the event of a PMP design storm event. The spillways would only become operational if a design storm occurred just prior to raising of the cell embankments and near the end of the life of mine. During Stage 1 of operation (as indicated on Figure 12), the South Cell spillway would be located at the same position as the existing spillway and would discharge into the Mt Hebbard Gully. The North Cell spillway would discharge into the South Cell. During the final stage (as indicated on Figure 13), a spillway would be constructed to discharge from the South Cell to the North Cell and from the North Cell, a spillway would discharge towards the Horwood Dam.

The surface of the tailings dam is currently covered by a layer of slag and it is not proposed to remove the slag. Rockfill will form the shell of the embankment and an intercepting filter comprising sand will be constructed on the upstream face, in accordance with DSC dam design guidelines (DSC18). The filter is incorporated into the design to control potential development of a hydraulic gradient through the embankment and also to act as cushioning for the geomembrane liner.

A seepage collection drain will be installed in the intercepting filter along the upstream toe of the embankment raise. This drain will collect water that is intercepted by the filter layer between the rockfill and the geomembrane and will direct flow to Horwood Dam.

10.4 TSF-1 Staged Construction

At the commencement of construction the centrally located existing decants will be grouted to reduce potential seepage paths through the TSF.

It is proposed to construct the new TSF-1 embankments in stages using the centreline raise method. This approach is recommended, as upstream raises over freshly deposited tailings are likely to present construction and stability risks.

The northern starter embankment of the North Cell and eastern starter embankments of both the South and North cells would be constructed on the existing tailings. The second and final raise would not exceed the footprint of the starter embankments. The dividing wall between the two cells would be raised using the centreline method and may be constructed in a number of low height lifts. Along the south western side, a staged embankment would be constructed to close off the storage facility from the Mt Hebbard Gully.

The embankment raises would be constructed with an outer batter slope of 1V:2.0H. Stability analyses indicate the upstream slope can be constructed at 1V:1.0H, as tailings would provide buttress support. However to facilitate construction of the interceptor filter and installation of the geomembrane, the upstream slope has been reduced to the 1.5V:1H along the northern and eastern sides.

The starter embankment on the TSF is to be constructed to take advantage of the existing consolidated tailings in the TSF and would be 6 m high (to approximately RL 328.0 m). Allowing for 1.0 m freeboard, this initial raise will provide for about 2 years storage of surface tailings production.

It is envisaged embankments would be constructed using non acid forming waste rock. A geochemical assessment (Golder ref: 087611001 002 R Rev0, dated April 2008) of waste rock stockpiles adjacent to the tailings dam indicate sufficient quantities of non acid forming and low acid forming waste rock suitable for embankment construction.

10.5 TSF-1 Seepage Analysis

10.5.1 Model Description

Seepage modelling was conducted along a representative cross section to analyse seepage from the new tailings into the new embankment and through the existing underlying materials. The section analysed is represented by Alignment D on Figure 3.

From a hydrogeological perspective, the existing tailings dam is underlain by relatively low permeability bedrock, identified as weathered gneiss in the geotechnical investigation (ref: 087611001 001 R Rev0). The existing tailings were characterised by laboratory testing as silty sand, sandy silt and clayey silt, with the estimated permeability varying from the perimeter to the centre of the facility due to variations in the old tailings properties. The existing perimeter embankment was formed by “compacted” tailings (i.e. of lower permeability than the stored tailings) with an outer sheeting of waste rock.

The analysis simulates saturated tailings over most of the old tailings surface, with the phreatic surface drawn down over the area of the proposed strip drains. The analysis considers steady state conditions, which is a conservative approach for the relatively term tailings deposition operation over old drained tailings.

The SEEP/W modelling software **(Ref.7)** was used to simulate seepage from the TSF. SEEP/W is a two-dimensional finite element model and is an industry standard for embankment seepage analyses.

10.5.2 Material Zones

The tailings near the centre of the existing tailings dam are considered to be finer than those near the outer edges due to perimeter deposition and segregation of tailings. This assumption is supported by the CPTu results which indicated that near the centre of the cells, there was no significant increase in cone or sleeve resistance with depth and relatively slow dissipation of pore pressure compared to the perimeter test locations. Similar conditions are expected to apply to the new tailings deposited into TSF-1.

The gradation of particle sizes from the perimeter of the cells (silty sand) to the centre (sandy silt) has an influence on the hydraulic conductivity of the tailings. A ratio of vertical to horizontal hydraulic conductivity (anisotropy) of 0.5 was adopted for the tailings based on layered slurry deposition. Assumed hydraulic conductivities for all materials modelled are summarised in Table 4.

Table 4: Assumed Hydraulic Conductivities for Seepage Analysis

Zone	Model Label	Material Description	Horizontal Hydraulic Conductivity (m/s)	Anisotropy Kv : Kh
1	Fine Tailings	Clayey silty sand	1×10^{-8}	0.5
2	Medium Tailings		1×10^{-7}	0.5
3	Coarse Tailings	Silty sand	1×10^{-6}	0.5
4	Rock Facing	Old rock facing	2×10^{-5}	1.0
5	Compacted Rock Fill	Newly compacted rock fill	1×10^{-6}	1.0
6	Weathered Rock	Weathered rock	5×10^{-9}	1.0
7	Geomembrane Liner	Geomembrane	1×10^{-10}	1.0

The geomembrane liner has been modelled conservatively assuming a number of defects in the liner.

The geotechnical investigation provided estimates of the hydraulic conductivity of the tailings, based on particle size distribution and Hazen’s formula. Estimates of permeability from particle size distribution results indicate tailings hydraulic conductivity in the range of 1×10^{-8} m/s to 1×10^{-6} m/s.

The hydraulic conductivity of the existing rock facing and the proposed embankment rockfill were estimated based on site observations and engineering judgement.

The hydraulic conductivity of the weathered rock at the site has been estimated to be approximately 5×10^{-9} m/s.

10.5.3 Boundary Conditions

The following boundary conditions were applied:

i) The wet tailings surface (potentially flooded) was modelled by applying a constant zero water pressure over the tailings surface.

ii) Seepage was permitted to both the lower and upper embankment toe drains with the maximum groundwater head fixed at the drain level.

iii) The underlying weathered rock included in the model is 30 m thick. At the base of this layer a unit head gradient boundary condition was applied. This implies the water table is well below this level (RL 270 m) and water can seep freely downwards from the base of the model.

10.5.4 Seepage Results

The steady stage model is presented on Figure 30 and the output presenting water pressure contours is presented on Figure 31. Seepage to the upstream toe drain of the embankment raise was estimated to be 0.003 m^3/d/m and seepage to the lower toe drain was estimated to be 0.16 m^3/d/m.

Figure 31 presents the model results of the steady state seepage conditions with no limit on the time needed to reach steady state. Most of the existing tailings on site are relatively dry so it is expected it will take a long time from when the new wet tailings are placed on the existing surface until a seepage front is established through the existing tailings and into the underlying rock formations. This significant time has been ignored in the modelling, as ignoring it results in a conservative outcome, and the site specific data required to carry out such modelling is not readily available. The slow development of a seepage front is as a result of the old tailings having to become saturated before a seepage front is developed. Considering the proposed short operating life of TSF-1 and the proposed management of the supernatant water pond size, it is unlikely that the hydraulic gradient will develop to the extent presented on Figure 31, before tailings deposition ceases on the facility.

Figure 31 presents a near vertical contour at the toe of the rock facing (at the arrow of “lower toe drain”). This line presents the modelled long term phreatic surface, with the area to the left being saturated and the area to the right unsaturated. The unsaturated material will develop suction in the pore spaces which is represented by the negative values on the contours to the right of the phreatic surface. The modelling indicates that a saturated mound is expected to form below the TSF area only.

The houses on the southern side of Eyre Street are approximately 90 m from the toe of the slope, with is 30 m from the right edge of Figure 31 (ends 60 m from the slope toe). So the houses are another 30 m to the right of the edge of Figure 31 and well away from the modelled wetting front. At a suction of 300 kPa, as indicated on the furthest to the right contour of Figure 31, the permeability of the unsaturated material is estimated at approximately 0.02 mm / year, so liquid from the TSF is highly unlikely to reach the houses.

10.6 TSF-1 Stability Analysis

At the commencement of operations a low phreatic surface, close to the interface of the tailings and underlying ground, has been assumed. This assumption is based on the results of the geotechnical investigations which indicate that the tailings are predominantly drained.

Embankment stability was evaluated using the SLOPE/W modelling software **(Ref.8)**, which adopts a conventional limit equilibrium approach to stability analysis. The stability of the Stage 2 embankment raise and the underlying existing slope on the northern or eastern side of the facility was modelled under both static and dynamic (earthquake load) conditions. The cross section alignment adopted is represented by Alignment D on Figure 3 and the cross-sectional geometry for staged embankment construction is shown on Figure 6.

The ANCOLD guidelines for the design of dams for earthquake loads **(Ref.5)** recommends two levels of earthquake motion be considered. Assuming a "High B" hazard rating for TSF-1 we have considered the following earthquake motion criteria:

- Operating Basis Earthquake (OBE), for serviceability conditions. For this level of earthquake motion, the TSF, associated structures and equipment should be functional and the damage should be easily repairable. The OBE is generally considered to be the earthquake that has a 10% probability of exceedence in a 50 year period (equivalent to a recurrence period of 475 years). The OBE is used to assess the stability of tailings storages for the operating life of the structure.
- Maximum Design Earthquake (MDE). For this level of earthquake motion, dam structures and equipment may be damaged but the impounding capacity of the embankments must be maintained. The return period of the MDE is typically about 1 in 10,000 years. The MDE is typically used for the design of closure measures for tailings storages.

The following recommended minimum Factors of Safety (FoS) for tailings storages are based on recommendations from several sources and ANCOLD guidelines **(Ref.9)**:

- Steady state load conditions (i.e. static load): FoS = 1.5
- Operating Base Earthquake (OBE): FoS = 1.2
- Maximum Design earthquake (MDE): FoS = 1.0

We consider these minimum FoS's would satisfy the requirements of the DSC.

Shear strength parameters for the stability analyses were selected using the results of the geotechnical investigations on the tailings dam and our experience with similar materials.

The starter embankment would be completed within a few months of commencement of construction. Based on the results of the geotechnical investigation, the upstream stability of the embankment constructed on the existing tailings was assessed considering partially drained conditions and effective stress parameters.

The overall downstream stability of the new tailings storage was also considered for the maximum height of the storage, which is the worst case loading condition. This condition would occur about 4.25 years after commencement of tailings deposition and the overall stability was assessed based on a conservative estimate of the phreatic surface and effective stress parameters. The adopted phreatic surface provides consideration of the proposed geomembrane and drainage systems on the upstream side of the embankment.

Material parameters adopted for the stability analyses are presented in Table 5. The properties for both the existing and proposed embankments are based on anecdotal evidence from mine staff and engineering judgement.

Table 5: Material Parameters Adopted for Stability Analyses

Material	Unit Weight (γ_m) (kN/m^3)	Effective Stress Parameters	
		Friction Angle (ϕ') (degrees)	Cohesion (c') (kPa)
Foundation Soils (including weathered Gneiss)	25	30	200
Stiff Tailings (in existing embankment)	17	30	15
Existing Embankment Rockfill	20	40	0
Proposed Embankment Rockfill	20	40	0
Existing Tailings 0 to 6 m (Upper) 6 m to 21 m (Lower)	 17 17	 30 35	 0 0
Future Tailings	17	20	4

Stability analyses were carried out assuming the new tailings storage is constructed on the surface of the existing tailings dam and a waste rock buttress is constructed at the downstream toe of the existing embankment.

Models and outputs of the stability analyses showing the failure surfaces with the minimum factors of safety (FoS) are presented in Figures 26 to 29. A summary of the predicted minimum FoS for static and seismic loading conditions for the sections analysed is presented in Table 6. The results indicate factors of safety which are greater than the recommended minimum values.

Table 6: Predicted Minimum Factor of Safety

Case Modelled	Minimum Factor of Safety		
	Static Loading	OBE Loading (0.013g)	MDE Loading (0.2g)
Starter embankment, no tailings, upstream failure	1.9	1.8	1.3
Final Height, global downstream failure with a conservative phreatic surface	2.0	1.9	1.2

10.7 TSF-2 Consequence Category

Operation of TSF-2 comprises the discharge of tailings into Blackwood Pit. The pit is considered inherently stable as there is no outer embankment to erode. As the storage does not require embankment construction, the risk of dam break is not considered to be an issue. The severity of damage resulting from a potential release through the emergency spillway is considered to be "minor" to "medium", with low-level contamination of downstream flora and soils.

We have assigned a "Low" consequence category to the proposed TSF-2.

10.8 TSF-2 Layout

The proposed TSF-2 is located to the north of the existing tailings dam, as shown on Figure 2.

There are existing perimeter bunds around the Blackwood Pit. Additional perimeter bunds of nominal 1 m height would be constructed where required around the pit to divert stormwater runoff, including at the edge of waste rock storages adjacent to the pit, as indicated on Figure 15.

Tailings deposition would initially commence at the northern end of the pit and after the void at the northern end has been filled, deposition would be switched to the southern end to form a downward sloping beach to the north, facilitating efficient water extraction during operation.

Flood containment and spillway design considerations are provided in Section 12.0. The preliminary design includes an emergency spillway located at the northern end of the pit, as indicated on Figure 15. A spillway release is considered unlikely during operation of the facility, as there is approximately 13 m of freeboard above the final predicted tailings beach for flood containment.

Ignoring any potential storage created by the perimeter safety bund, the lowest perimeter elevation of the facility is RL 308.5 m (in the area of the proposed spillway). If additional tailings storage (and/or flood capacity) is required, an engineered embankment would be constructed at the northern end.

Supernatant water is expected to cover the entire tailings surface at certain times during the initial operation of the facility, however it is envisaged that once tailings deposition from the south is established, the water pond will mainly be located at the northern end.

11.0 OPERATIONAL CONSIDERATIONS

11.1 Operations Manual Development

During the construction phase of the embankment raise, an operations manual would be developed to address the proposed depositional strategy for TSF-1 and TSF-2. The manual would outline the procedures required to operate the facility in accordance with the design, and in particular with respect to the management of water and dust.

The roles and responsibilities of the mine personnel assigned to the facility would be outlined, including the key criteria that should be monitored during routine inspections to assess performance of the facility. The manual would provide a framework to ensure the facility meets regulatory requirements and a response plan in the unlikely event of an emergency. The response plan would provide procedures to minimise risks to the health and safety of mine personnel, the integrity of the surrounding environmental and the continual operation of the mine.

11.2 Deposition Strategy

Tailings would be discharged through a ring main pipe with spigots located at approximately 25 m centres around the two cells. The spacing of the spigots would be adjusted based on the actual tailings size distribution achieved by the mill. Spigots will be positioned so that tailings beaches toward the western side of the cells. This will direct the position of the supernatant water pond close to the western side of the TSF. Discharge from each spigot would be rotated to ensure deposition is evenly distributed around the facility and deposition would be switched between the two cells to maximise drainage and consolidation of the tailings. The indicative timeframe for deposition in each cell is less than two weeks before switching to the alternate cell. This allows for tailings to be deposited in thin layers and release supernatant water, resulting in increased tailings strength.

Guidelines will be provided in the operations manual for managing the supernatant ponds on the surface of the cells in TSF-1 by pumping supernatant water and stormwater to the external decant dam. Routine pumping of water from the TSF will help increase the tailings density in the storage, reduce seepage from the tailings and improve the safety of the storage.

At TSF-2, tailings would initially be deposited in the northern end. During the filling of the deeper northern end of the pit, deposition would occur from different deposition spigots to facilitate decant water off-take and evaporative drying.

If possible, efficient filling of the northern end would be aided by deposition of total tailings. As the total tailings contain more sandy material, the consolidation process would be faster due to the higher permeability of the coarser tailings. Once the northern end of the pit has been filled, deposition would then switch to the southern end to form a downwards sloping beach to the north.

Pond water from TSF-2 would also be pumped to the decant dam used for TSF-1. The decant dam would also serve as a secondary settlement pond, allowing for clarification of water prior to return to the processing plant, and may be the primary source of water for the dust management system.

11.3 Dust Management Plan

A preliminary dust management plan for the tailings storages has been developed to suppress dust during construction, operation and closure of the facilities. Dust management will be critical for both TSF-1 and TSF-2, however TSF-1 requires additional attention due to the location and operational aspects of the facility.

The plan comprises the installation of a spray system around the perimeter of each TSF-1 cell. The system comprises application of a dust suppressant after a cycle of tailings deposition ceases. The dust suppressant is a polymer and water mixture which forms a crust over the tailings surface.

11.3.1 TSF Construction Dust

During construction of the TSF-1 embankment raise, the potential exists for dust to be generated by placement of soil materials. We note that the risk of dust generation will be minimal as the proposed embankment materials predominantly comprise rockfill. To further mitigate generation of dust from the fines that may exist in the material, the following measures are proposed.

- Regular moisture conditioning during excavation of rockfill from stockpiles via dedicated water cart spray and/or hosing.
- Routine water spray along proposed haulage routes from the waste rock stockpile to the embankment construction site using a water cart and dribble bar. A spray system will be installed along permanent haul roads.
- Additional water spray during placement of rockfill layers at embankment via water cart after spreading and during compaction.

A construction dust management plan will be developed with the construction contractor to implement the above measures. The construction schedule will also include limitations on works permitted during windy days. No excavation of the existing tailings is proposed.

11.3.2 TSF Operation Dust

Tailings deposition will be cycled between two cells. During a period of active deposition, discharge of tailings will be rotated from nominated spigots. The tailings surface is likely to initially be a slurry, changing slowly over a few days to wet to moist tailings. Given the size of the TSF cells, the length of wet beach is expected to extend from the active spigot to the opposite side of the cell.

During the period in which a cell or part of a cell is inactive, there is a potential risk of dust generation from the tailings surface if the tailings dries sufficiently to release dust. A spray system comprising the following components is proposed to manage this risk:

- Sprinklers and Reticulation Pipe
- Water Supply, Pump and Control System
- Dust Suppressant (Crusting) agent

The spray system will apply a coating of polymers over the surface of the wet to moist tailings, using water as a medium to place the polymer. The polymer coating forms a crust with the tailings, resulting in a surface that does not release fine particles as dust. The crusting agent is resistant to wind and water erosion and the durability of the crusting agent is related to the severity of surface disturbance.

The spray system is to be installed as part of the initial works for TSF-1. Hence the piping and sprays, with the associated control and mixing system could be activated at any time during the operation of the TSF.

The crusting agent would be applied by one sprinkler at a time, and would take a few minutes per sprinkler to apply the recommended rate of crusting agent over the designated sprinkler area. Each cell could therefore be covered by a coating of crusting agent in less than one hour.

The recommended dosing rate for the mixing of the crusting agent, and hence the application rate on the tailings surface, may be varied over the weather seasons at the site, with a higher concentration or more frequent application of crusting agent prior to a period of high wind or following intense rainfall events, or if disturbance of the tailings surface has occurred.

The sprinkler system also applies a significant amount of water onto the surface of the tailings, as part of the crusting agent application process. In addition to the application of the crusting agent water could be sprayed on a specific part of the tailings surface if a localised issue developed with potential for the tailings to generate dust.

The sprinklers for the system will be installed around the perimeter of each cell. This provides flexibility to apply more water and crusting agent from one side of the facility if windy conditions occur.

Generally only one application of the crusting agent is expected to generally be required after tailings deposition is switched to the adjacent cell; i.e. so generally once per two weeks. Additional applications of either crusting agent or water may be activated if the surface of the tailings appears to be at risk of generating dust.

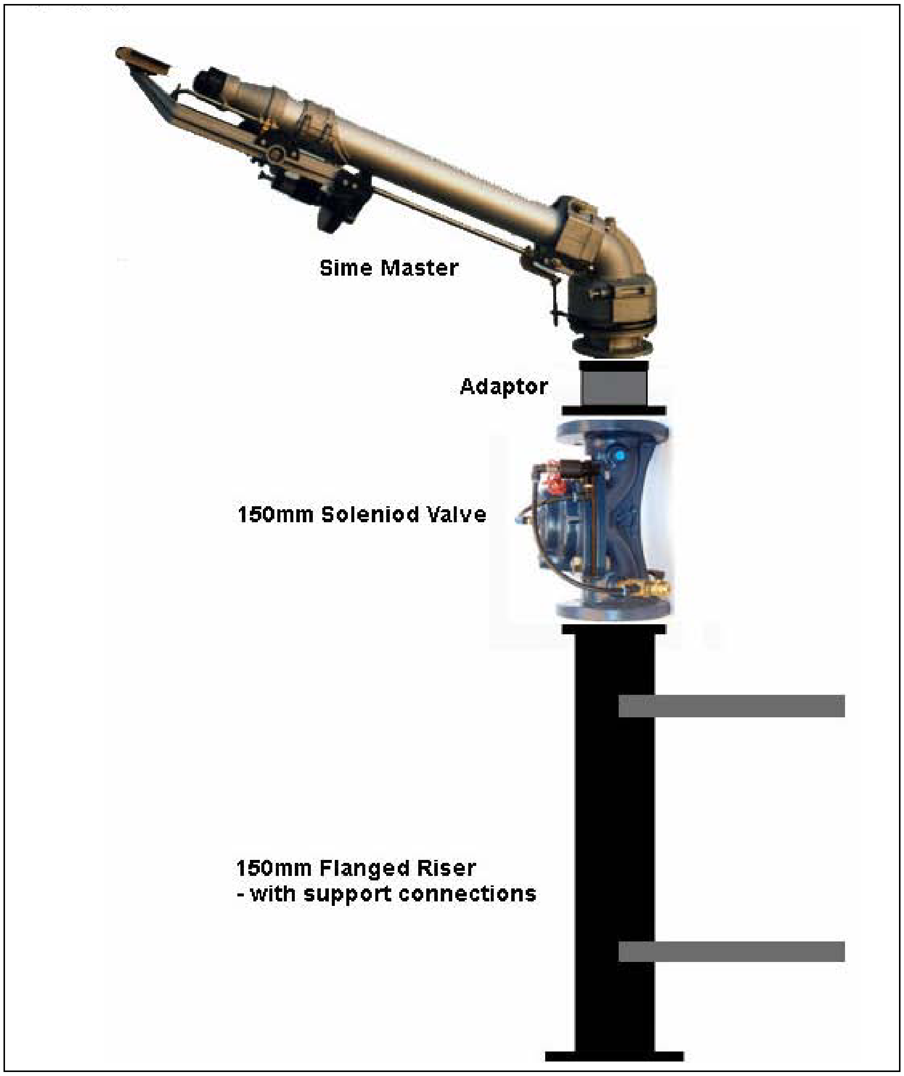
11.3.2.1 Sprinklers and Reticulation Pipe

Six sprinklers are proposed around the perimeter of each cell (i.e. 12 total), as indicated on Figure 32. A typical illustration of a spray unit is presented in the diagram below. Each sprinkler has a maximum throw distance of 95 m and a maximum spacing of 75 m is proposed between sprinkler units. The operating pressure of each unit is 8 bar.

The sprinklers will be reticulated by 250 mm plastic pipe and each sprinkler unit comprises:

- 150 mm diameter, 0.5 m high riser with support footings
- Solenoid Valve for reticulation control
- 48 mm diameter spray nozzle with capacity to spray 4500 litres/minute

Diagram of Proposed Spray System (provided by Wet Earth Irrigation)



11.3.2.2 Crusting Agent

A crusting agent would be added to water at an indicative rate of approximately 3% by volume. Supplier information and wind tunnel testing on similar tailings materials has indicated that a mixture of water and crusting agent at a 3% solution applied at a rate of approximately 2 L/m^2 would provide dust control for a number of months. The actual concentration of crusting agent to be adopted for the site will be subject to field trials prior to commencement of operation. Trial criteria will include resistance to wind speeds of up to 50 km/hour.

The estimated volume of agent required to cover the entire surface of TSF-1 is approximately 6000 litres, based on an area of 100,000 m^2. At this preliminary design stage it is expected each cell will be operated for a period of two weeks, resulting in one spray cycle per cell every 4 weeks. Supplier information indicates that the product is supplied in 200 litre drums or 1000 litre bulk containers.

The agent dosage and water requirements for the Rasp site will be ascertained during proposed trials prior to commencement of operation. For the estimated dosage of 3% approximately 100,000 litres of water is required per spray cycle per cell (i.e. total 200 m^3 per month or 2400 m^3 per year).

This volume of water is significantly less than the expected rate of return of supernatant water from the deposited tailings in the TSF, so a significant source of water is available at the TSF to supply water for the application of the crusting agent.

11.3.2.3 Water Supply, Pump and Control System

The decant dam is the nominated source of water for the proposed spray system. Water would be pumped from the dam and the pump will be sized to supply at least 4500 litres/minute at a pressure of 8.5 bar. The pump would have a power requirement of 132 KW, sufficient to support one sprinkler at a time during activation of the spray system. The water balance estimates presented in Section 12.9 indicate that the decant dam has sufficient capacity to meet the demands of the sprinklers.

The control system would include an agent flow rate meter to ensure sufficient agent is delivered. If the agent source is depleted, an alarm would be triggered and the system paused. A mainline flow meter would also be included to monitor the overall flow through the system.

11.3.3 TSF Closure Dust

After completion of tailings deposition in each cell of TSF-1 the spray system will be activated for a final cycle. After application of the crusting agent, vehicles, pedestrians and animals would be restricted from gaining access to the surface of the treated tailings. Temporary roads would be constructed over the treated tailings for vehicle access to the interior of the cells.

When the tailings has consolidated sufficiently the surface will be covered with a layer of selected waste rock as a long term protection against dust formation. The dust management spray system will also be activated throughout the placement of the waste rock cover.

12.0 STORMWATER AND PROCESS WATER MANAGEMENT

12.1 Stormwater Design Criteria

12.1.1 Tailings Storage Facilities

The tailings storage facilities have been designed to provide flood containment for design storm events and also spillways to manage stormwater discharge in the event of extreme design events. Given the limited external catchments for both TSF-1 and TSF-2 no external stormwater diversions are proposed.

The adopted design criteria for flood containment and flood discharge are based on the consequence category assigned for each facility (refer Section 10.0). The following flood management criteria were adopted based on DSC guidelines for tailings dams (DSC19) and ANCOLD flood guidelines **(Ref.10)**.

Table 7: Flood Criteria Adopted for TSF-1 and TSF-2

Facility	Consequence Category	Environmental Containment Freeboard (1)	Operational Freeboard (m) – min (2)	Total Freeboard (3)	Flood Discharge
TSF-1	High B	1 in 1000 year, 72 hour	0.5	1 in 1,000,000 year, critical duration	1 in 10,000 year to PMP
TSF-2	Low	1 in 1 year, 72 hour	0.3	1 in 1,000 year, critical duration	1 in 1000 year

Notes:

1. The environmental containment freeboard represents the required flood storage capacity between the tailings beach and the spillway elevation (i.e. storage prior to discharge from the facility).
2. The operational freeboard represents the vertical distance between the elevation of the tailings beach and the adjacent embankment crest elevation.
3. The total freeboard represents the required storage capacity above the operating pond volume and the crest of the embankment.

12.1.2 Stormwater Management Facilities

For external stormwater management, the NSW Department of Planning (DOP) require that stormwater ponds on site have capacity to contain the runoff resulting from the critical duration of the 1 in 100 year design storm event.

12.2 TSF-1 Flood Containment

The proposed TSF-1 consists of two approximately square cells. The stormwater catchment of the two cells is limited to the area of the cells and the slopes of the adjacent waste rock storages to the south and to the west. The external catchment area at TSF-1 is approximately 1.6 ha and the total catchment area is approximately 11.2 ha.

The preliminary design allows for an average 0.5 m above the predicted final tailings beach to spillway elevation and a further 0.5 m to embankment crest elevation, which satisfied the Operational Freeboard. Including consideration of an operating pond volume of 5,000 m^3 and conservative runoff of 100%, there is sufficient flood containment capacity in TSF-1 to satisfy the Environmental Freeboard and the Total Freeboard. For the Total Freeboard assessment, a conservative storm duration of 72 hours was adopted.

To efficiently manage process water and stormwater and to promote consolidation of the tailings, the operational pond should be located at the western side of each cell. Surface water from rainfall and supernatant water released from tailings as they settle, will collect in the pond. Water will be pumped from the supernatant ponds to a lined decant dam located on the waste rock storage to the west of TSF-1. The decant dam will provide a water source for the proposed dust management spray system. Excess water may also be returned to the process plant.

Section 12.4 describes the locations of spillways for the TSF-1 cells. It is likely the spillways may only be activated if a design storm or greater were to occur just prior to, or during construction of an embankment raise, when the tailings surface is near the design capacity stage of the facility.

12.3 TSF-2 Flood Containment

The proposed TSF-2 includes construction of a perimeter bund around the perimeter of the pit to prevent surface water runoff from the surrounding areas from entering the pit. Indicative alignments of bund walls are shown on Figure 15 and a typical bund section is provided on Figure 9. After perimeter bund construction, the external catchment for TSF-2 is reduced to approximately 0.3 ha. The total catchment area for TSF-2 is approximately 9 ha.

The predicted final tailings beach elevation in TSF-2 is RL 295.5 m and the minimum top elevation of the pit is approximately RL 308.5 m. The Operational Freeboard requirement is satisfied, as there is approximately 13 m of freeboard. The flood containment capacity satisfies both the Environmental Freeboard and the Total Freeboard.

During operation, supernatant water and stormwater runoff will collect in a pond at the northern end of the pit. The pond water on TSF-2 will be pumped to the decant dam by submersible pumps mounted on a floating pontoon.

12.4 TSF-1 Spillway Design

The layout for the TSF-1 Stage 2 main spillway is presented on Figure 17 and typical sections are shown on Figure 18 and Figure 19. The spillway is 0.5 m deep and 40 m wide and has been designed to manage the peak flow resulting from a PMP storm event, satisfying the flood discharge criteria for a “High B” consequence category facility. The spillway channel would direct stormwater into the Horwood Dam. The Horwood Dam is not designed to contain water discharged from TSF-1 however it would provide flow attenuation in the unlikely event that the TSF-1 spillway is activated.

The Stage 1 main spillway will have similar dimensions to the Stage 2 spillway. The layout of this spillway is shown on Figure 12.

The Stage 1 and Stage 2 spillways through the dividing embankment of the two cells will be 0.5 m deep and 20 m wide to discharge the peak flow from a PMP storm event. During Stage 1, runoff from the design storm event will flow from the north cell to the south cell and discharge into the Mt Hebbard Gully. During Stage 2, runoff from the design storm event will flow from the south cell to the north cell and then discharge via the main spillway into Horwood Dam.

12.5 TSF-2 Spillway Design

The proposed spillway location for TSF-2 is shown on Figure 15 and will be designed for the 1 in 1000 year storm event to satisfy flood discharge criteria for a “Low” consequence category facility. The spillway will be formed by building up side bunds, with minimal excavation so that the storage capacity in the pit is not reduced.

12.6 TSF-1 External Stormwater Management

Toe drains will be constructed along the downstream toe of the tailings dam, adjacent to Eyre St. The “v” shaped drain at the toe of the proposed toe buttress will collect seepage from TSF-1 and also direct runoff from the embankment batter to the Horwood Dam, as indicated on Figures 2 and 20.

The proposed modifications to the Horwood dam include removal of the existing bund located along the eastern side and construction of an engineered bund to a crest elevation of RL 299.5 m. A spillway will be constructed for the Horwood Dam with crest elevation of RL 299.0 m. Typical sections of the engineered bund and spillway are presented on Figures 21 and 22.

The modified Horwood Dam will provide storage capacity of approximately 30,000 m^3. The estimated runoff volume from the contributing catchments to the Horwood Dam for the 1 in 100 year ARI, 24 hour (critical duration) design storm event is approximately 20,000 m^3. The spillway through the Horwood Dam will provide for stormwater discharge for events greater than the 1 in 100 year ARI.

The additional capacity in the Horwood Dam allows for potential operational storage prior to the design storm event. However during normal operation the Horwood Dam is expected to be empty, as water will either evaporate or be pumped to the Decant Dam for return to the process plant. In the event of an extreme storm (i.e. exceeding the 1 in 100 year event) or activation of the TSF-1 emergency spillway, some damage may occur to the Horwood Dam and spillway. Accepting this risk is considered reasonable given the height and elevation of the embankment, and the limited storage capacity of the facility.

12.7 TSF-2 External Stormwater Management

The perimeter safety bunds constructed around TSF-2 will also serve as external catchment diversions. These bunds will prevent runoff from adjacent waste dumps into the pit and drainage outside the bunds will be directed away from the pit.

12.8 Decant Dam

A decant dam is included in the design to manage excess water from TSF-1 and TSF-2 during respective operation. The Decant Dam is located over waste rock storages to the west of TSF-1, in an area of proposed borrow for embankment rockfill. A general layout of the proposed Decant Dam is shown on Figure 23 and typical sections are presented on Figures 24 and 25.

The preliminary Decant Dam design requires a 3 m deep excavation from an elevation of RL 332.0 m and has been sized to contain approximately 4 weeks of TSF-1 pond off-take under normal winter operating conditions. Waste rock excavated from the area may also be used for the purpose of TSF-1 embankment construction.

The Decant Dam will be lined to provide a reliable water containment facility. As indicated on Figure 23, a sump would be formed to facilitate pumping of water from the dam for the dust management spray system and also potentially to the process plant.

In the event the Decant Dam becomes full and requires emergency discharge, a spillway will direct water back into TSF-1, as indicated on the layout.

During operation of TSF-2, the Decant Dam may also serve as a settlement pond for potentially turbid water.

12.9 Water Balance

Monthly water balance models have been prepared for TSF-1 and TSF-2, assuming steady state production over a 12 month period. The models are based on expected final storage conditions and have been developed to provide an indication of the likely storage of water on the storage facilities, and the required seasonal pump rates to control pond size and provide water for plant return.

The key input data has been sourced from information provided by Abesque, the Bureau of Meteorology and engineering judgement. The following parameters have been selected for the water balance models:

- A tailings particle density (SG) of 3.0 t/m^3, a slurry density of 50% solids by mass and a deposition rate of 320,800 tpa of solids.
- TSF-1 beach area of 9.6 ha and an external catchment of 1.6 ha.
- TSF-2 beach area of 8.7 ha and an external catchment of 0.3 ha.
- Average annual rainfall (from the Patton Street monitoring station) of 253 mm and annual average evaporation of 2,614 mm.

- Pan evaporation factor of 0.7 for the ponds and an average evaporation factor of 0.3 for tailings beaches to cater for wet and drying beaches.
- A seepage rate of 83 m^3/day (0.96 L/sec) for TSF-1 from the deposited tailings into the underlying existing tailings. We have assumed the hydraulic conductivity of the existing tailings governs the seepage rates.
- A seepage rate of 7.5 m^3/day (0.09 L/sec) from TSF-2 based on the predicted hydraulic conductivity of the bedrock.
- The interstitial moisture content of the beached tailings was assumed to be 35% by mass of the dry solids deposited in a 12-month period. This correlates to a degree of saturation of about 80% for tailings at an average dry density of 1.3 t/m^3.
- A water requirement rate of 200 m^3/month (2400 m^3/year) for the dust management spray system.

The results of the water balance modelling for TSF-1 and TSF-2 are summarised in Table 8. Indicative pump rates for pond water off-take to the decant dam are presented in Table 9.

Table 8: Summary of Annual Water Balance Results

Facility	Inflows		Outflows			Net Annual Water Reporting to Decant Pond (m^3)
	Rainfall Runoff from Tailings (m^3)	Bleed Water from Tailings (m^3)	Evaporation from Pond and Tailings (m^3)	Seepage from Tailings (m^3)	Water for Dust Management Spray (m^3)	
TSF-1	13,300	208,500	72,700	30,300	2400	116,400
TSF-2	16,100	208,500	55,600	13,700	0	155,200

Note: It is assumed for the models that water is regularly pumped off the facility to maintain a constant pond volume.

Table 9: Summary of Estimated Excess Water Extraction (after dust spray requirements)

Month	Average Monthly Excess Water (L/sec)	
	TSF-1	TSF-2
January	1.6	2.9
February	2.8	4.3
March	2.8	4.2
April	4.3	5.5
May	5.0	6.0
June	5.5	6.4
July	5.2	6.1
August	4.7	5.8
September	4.3	5.6
October	3.3	4.7
November	2.9	4.3
December	2.0	3.3

Note: All rates are based on 24 hours, 7 days per week pumping.

The water balance modelling indicates that approximately 204,000 m^3/year of make up water would be required for operation of TSF-1. For TSF-2 approximately 166,000 m^3/year of make up water would be required.

13.0 CLOSURE PLAN

At cessation of tailings discharge in both TSF-1 and TSF-2, it is proposed that cover soil and rock be placed over the final tailings surface to provide a rehabilitated landform. Preliminary closure plans for TSF-1 and TSF-2 are presented on Figure 14 and 16 respectively. The concept for stormwater management is to store and release runoff via engineered spillway channels.

Stormwater discharge from TSF-1 would be directed down the ramp towards TSF-2 (Blackwood Pit). The thickness and composition of the cover design and spillway sizing would be designed post operation of each facility when a final survey would be completed.

14.0 REFERENCES

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Report Signature Page

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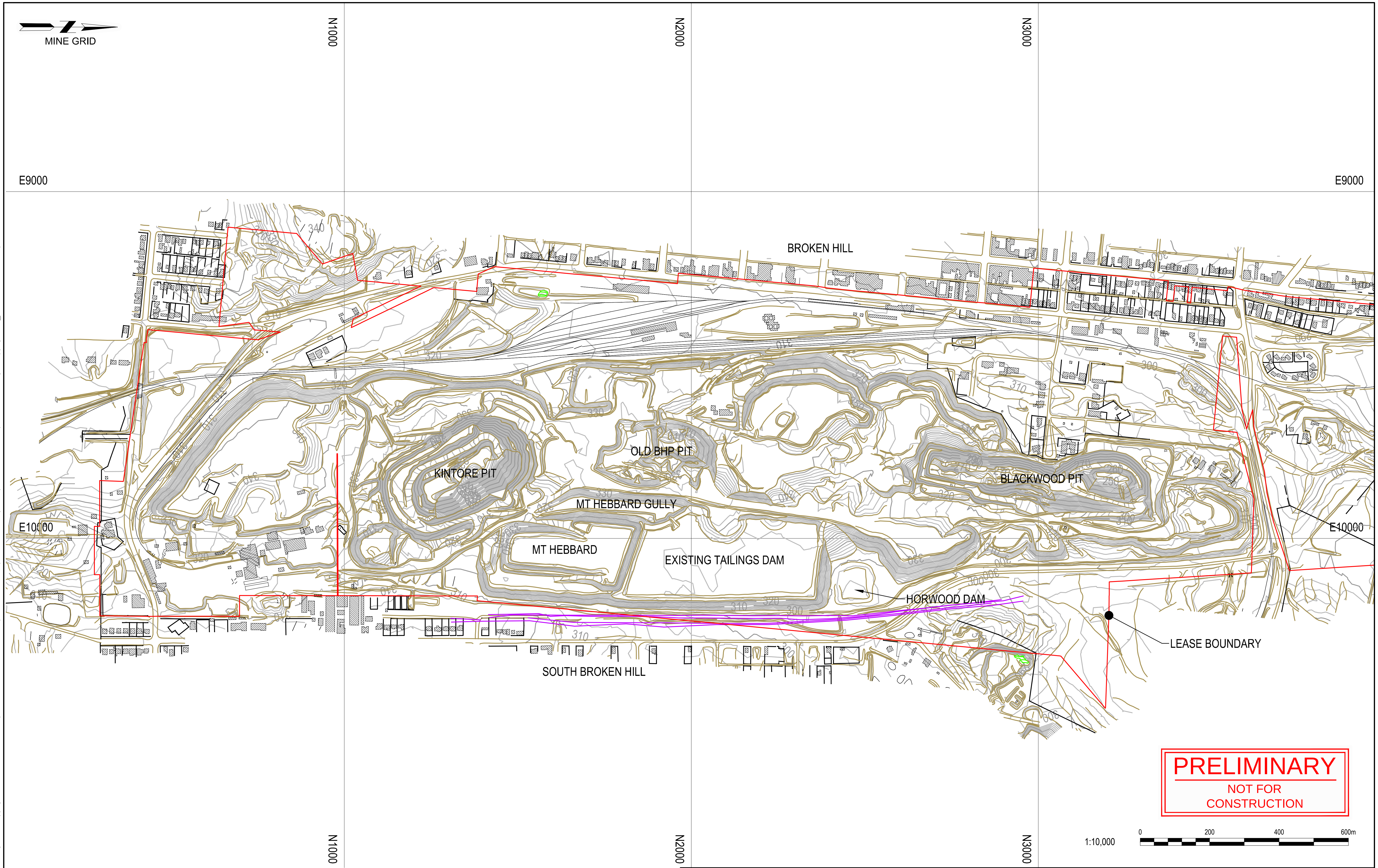

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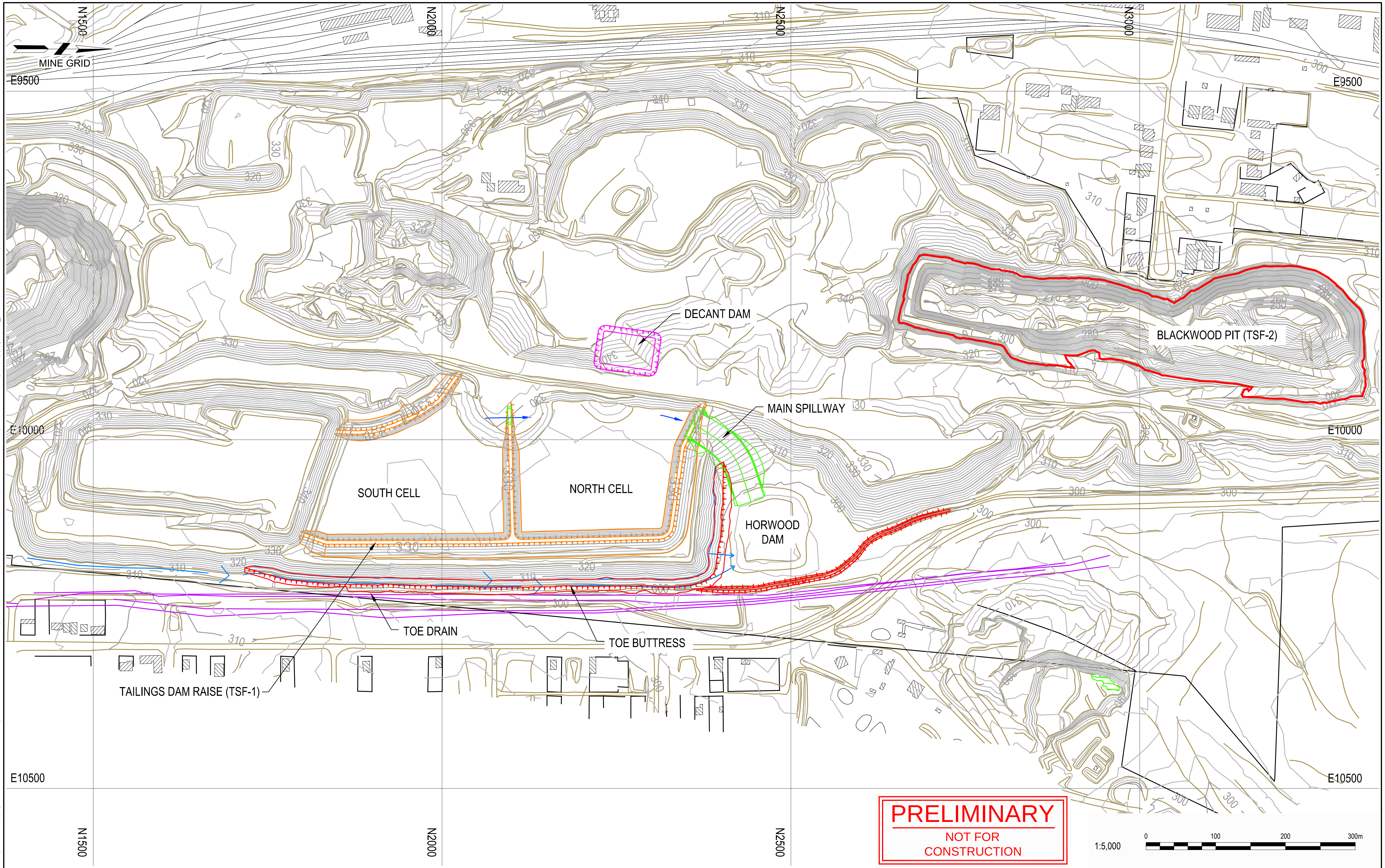

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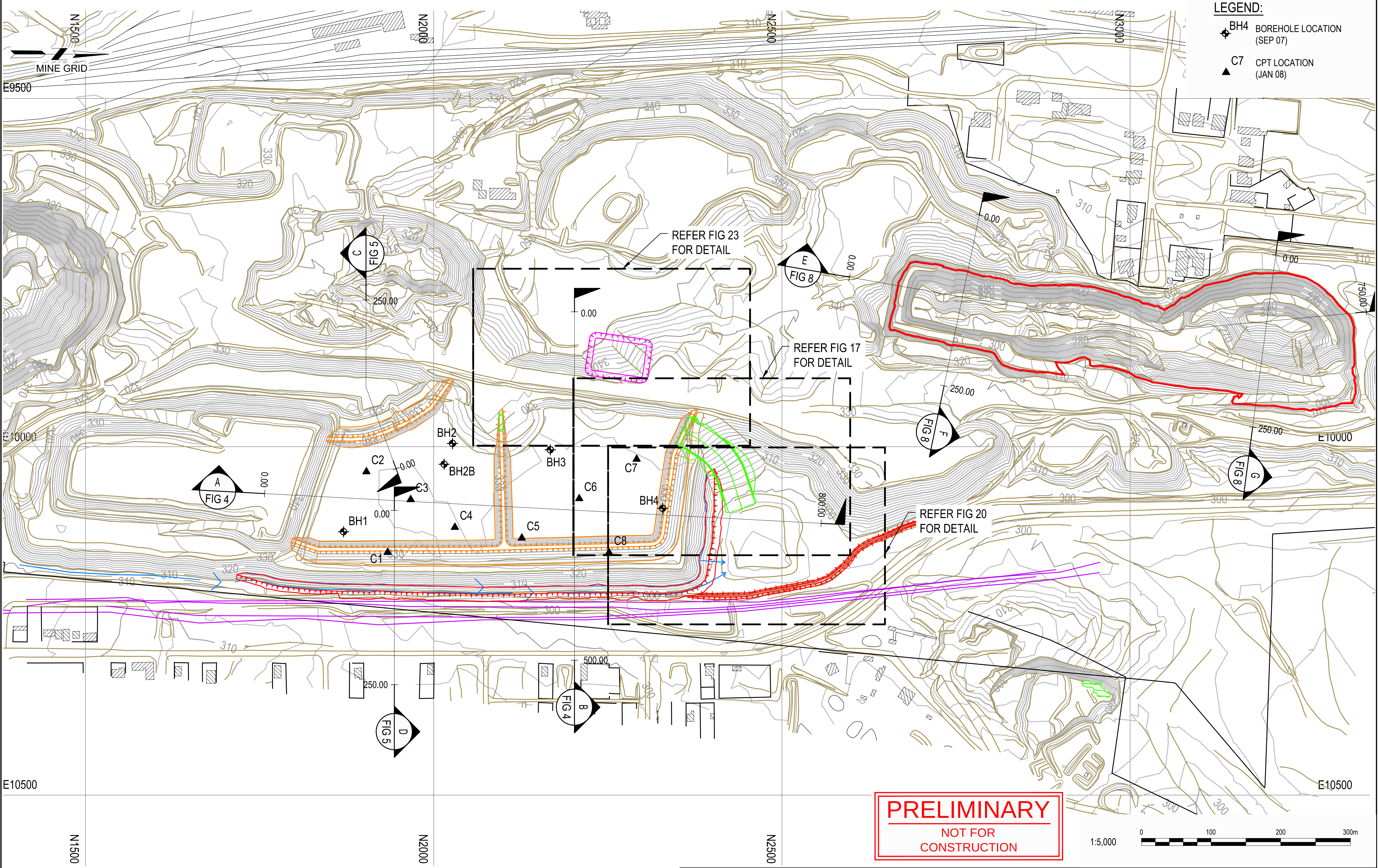

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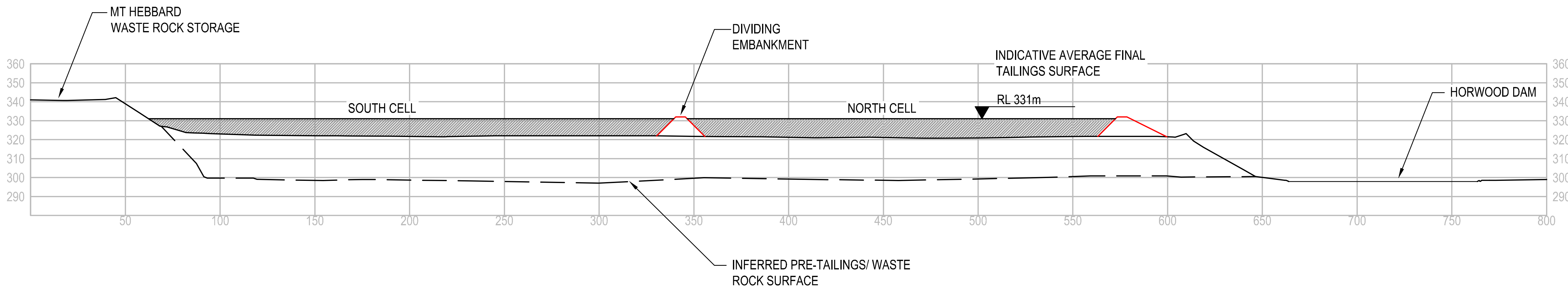

SECTION A FIG 3
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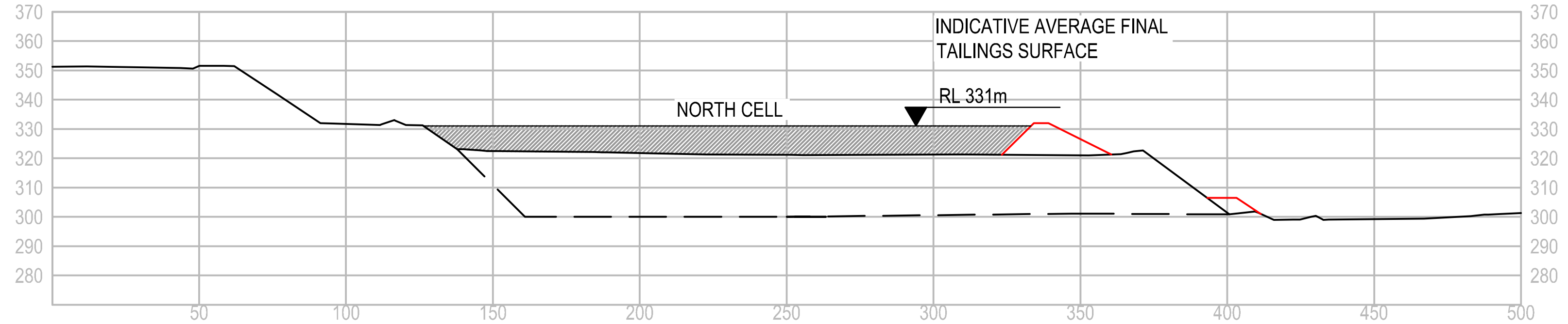

SECTION B FIG 3
SCALE 1:2,500

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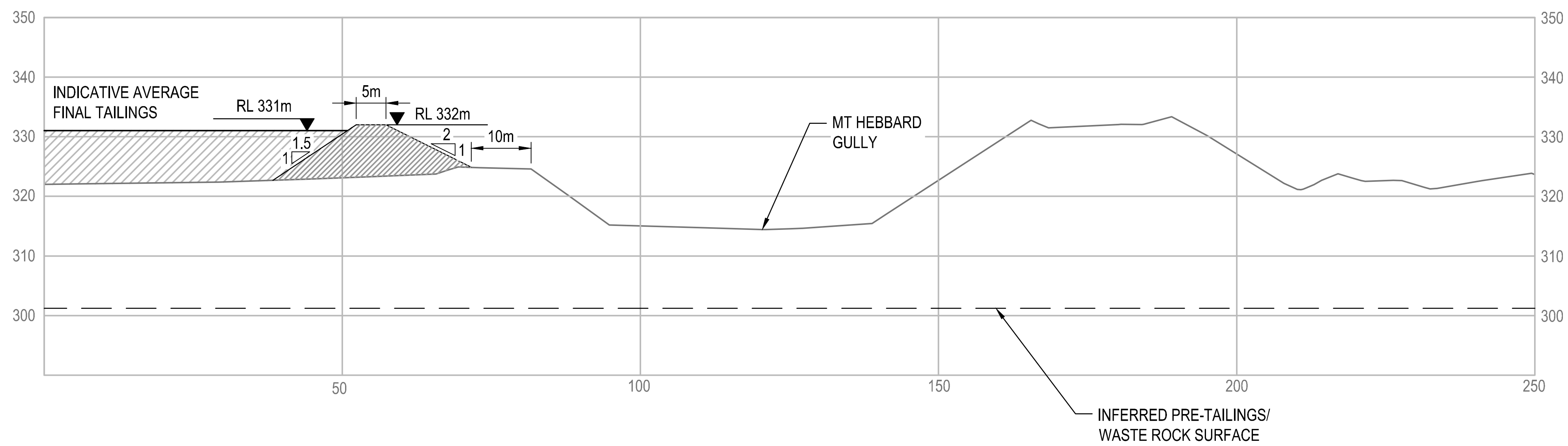

SECTION C
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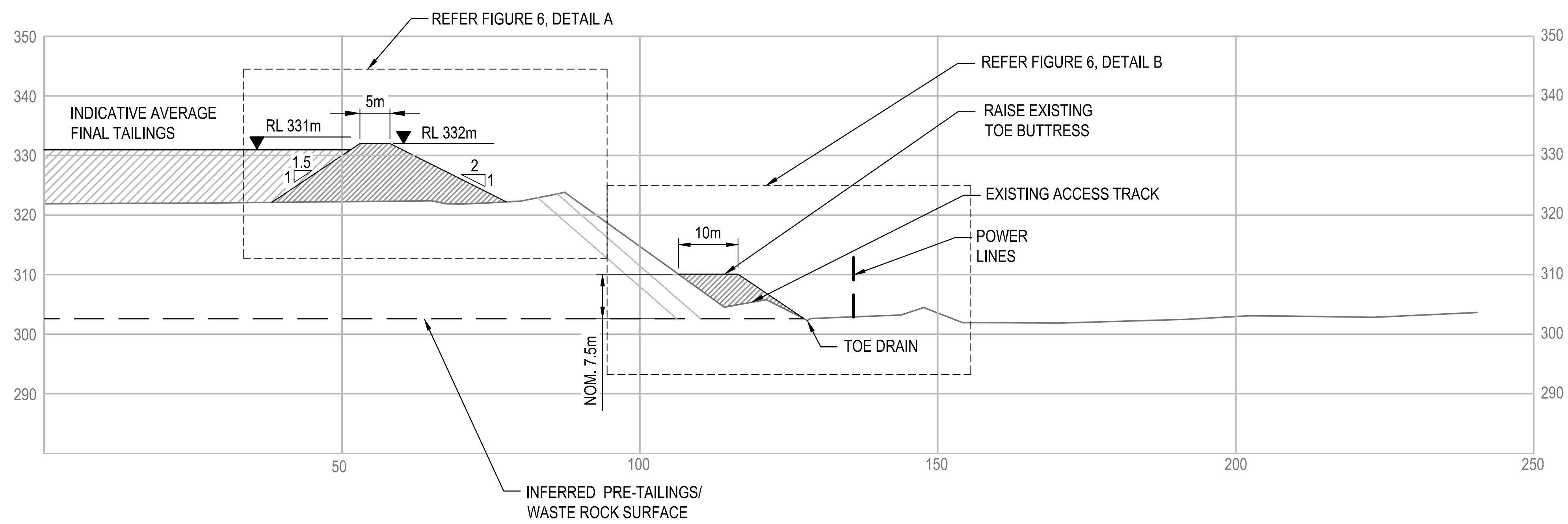

SECTION D
1:1,000 FIG 3

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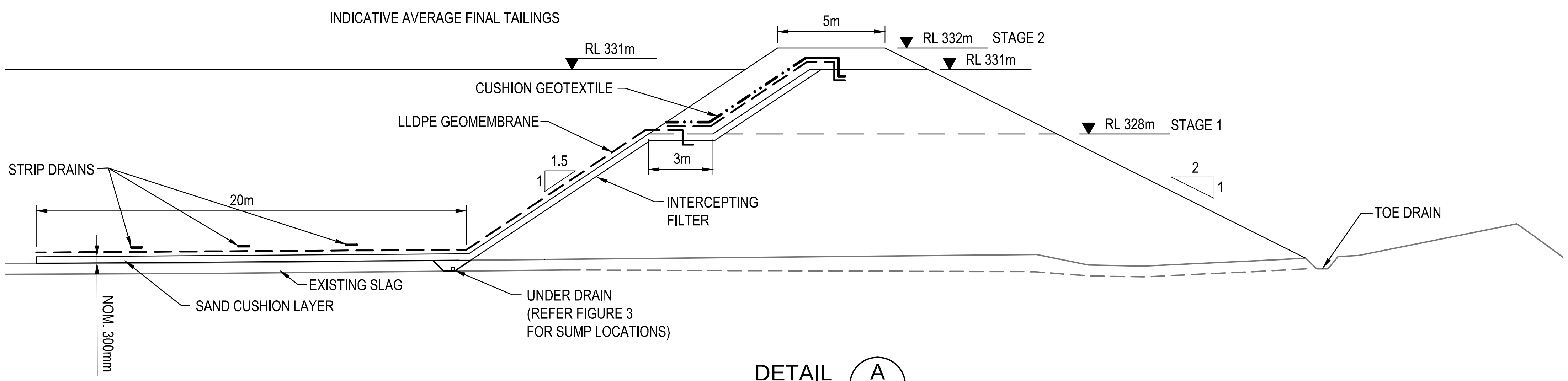
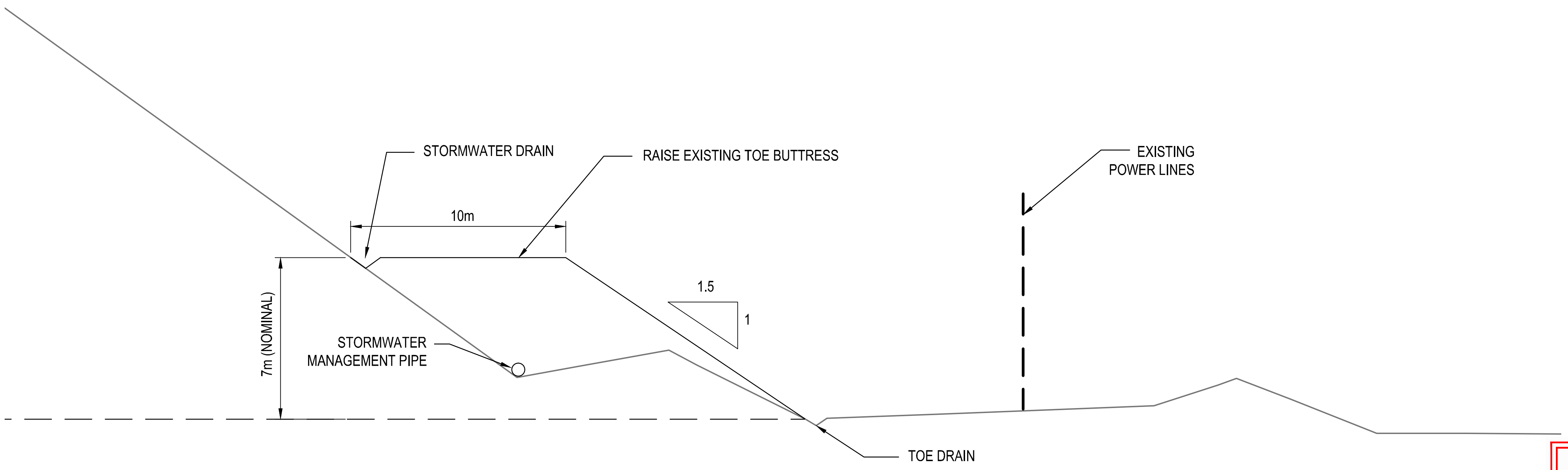


DETAIL A FIG 5
SCALE 1:250



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DETAIL B FIG 5
SCALE 1:250

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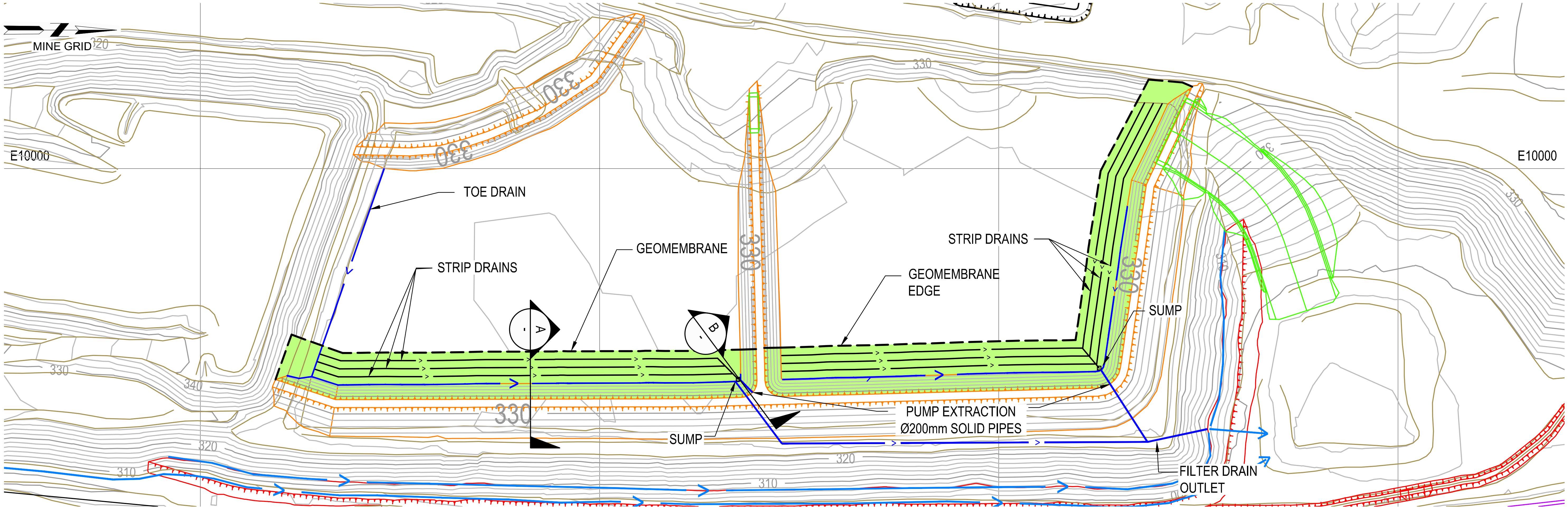

PLAN VIEW
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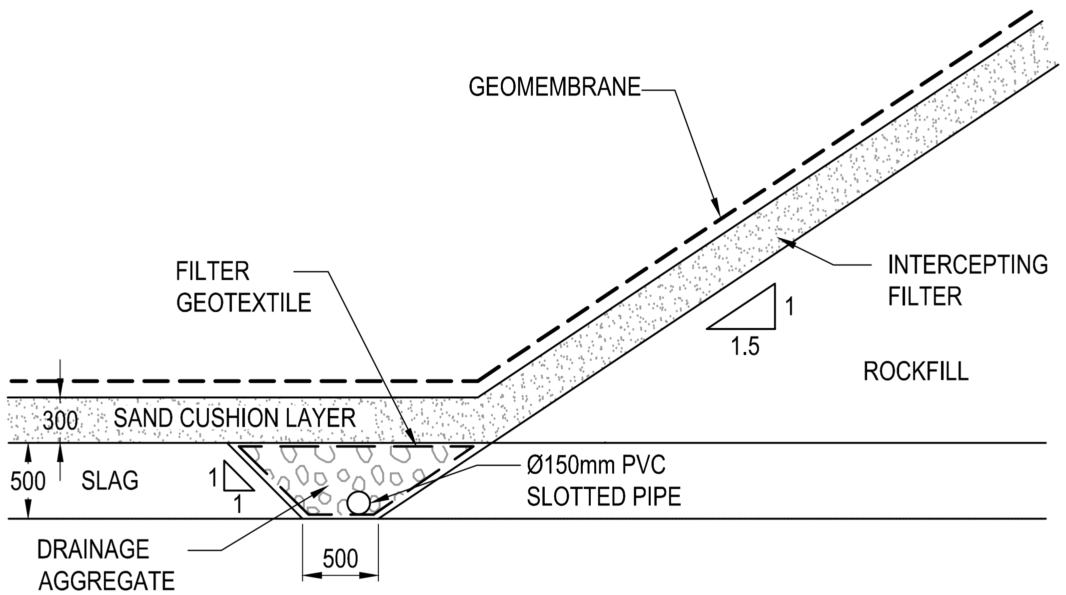

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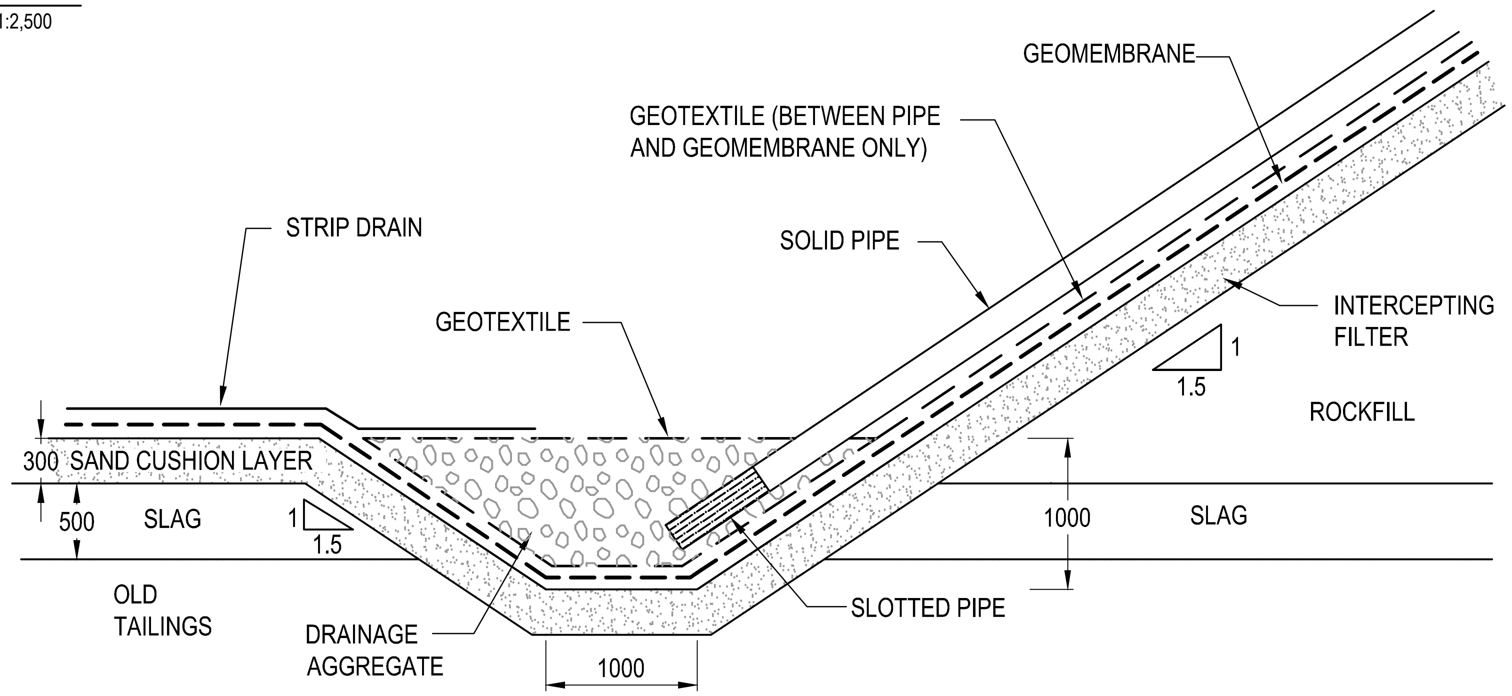

SECTION (B) SUMP DETAIL
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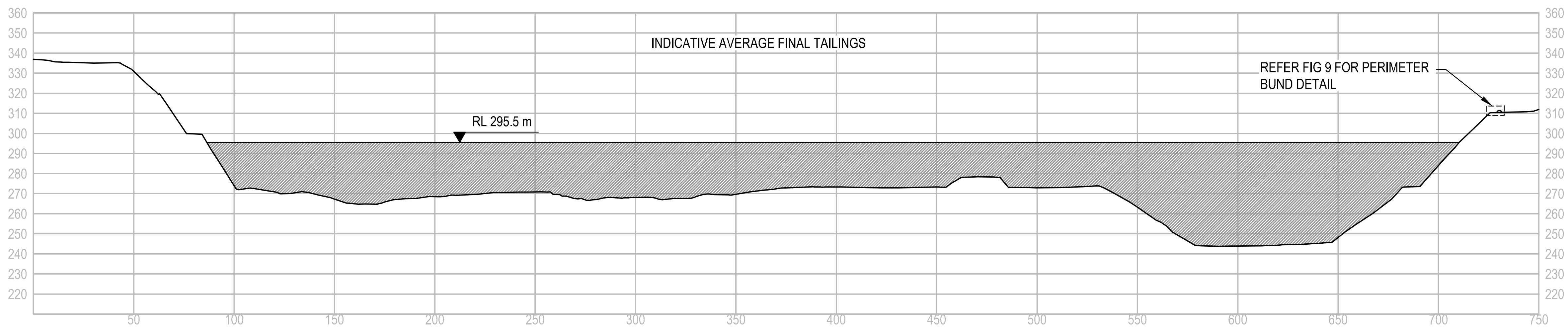
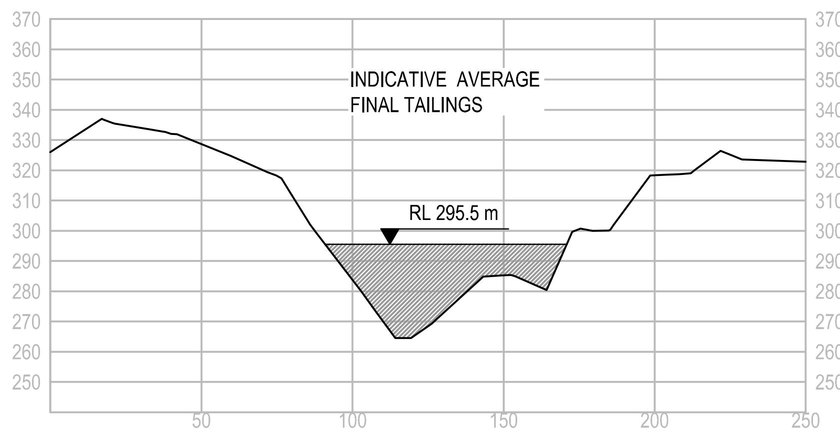
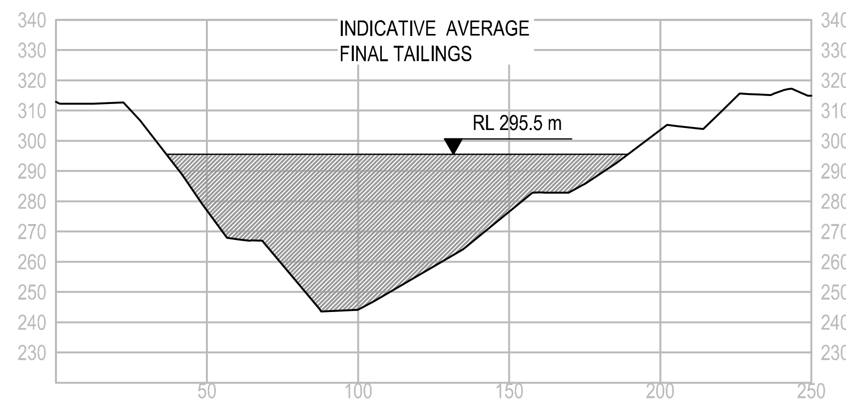


SECTION E
FIG 3
SCALE 1:2,500



SECTION F
FIG 3
SCALE 1:2,500



SECTION G
FIG 3
SCALE 1:2,500

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XREF:- NONE

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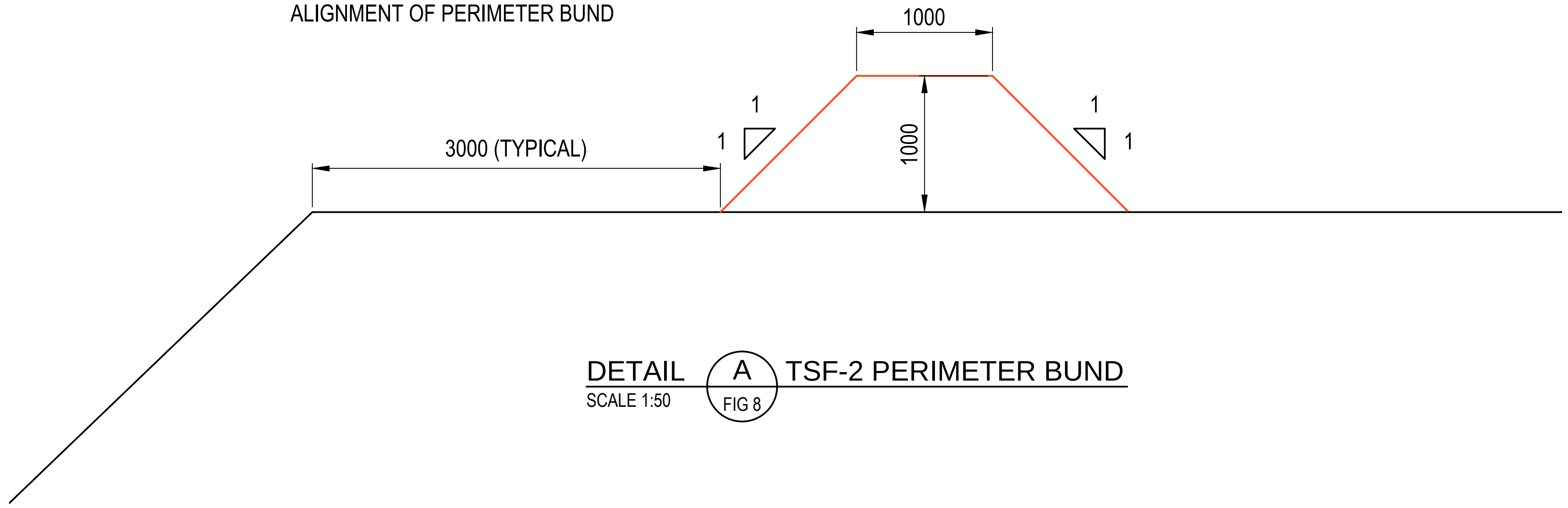
NOTE: REFER FIG 15 FOR INDICATIVE
ALIGNMENT OF PERIMETER BUND



DETAIL A TSF-2 PERIMETER BUND
SCALE 1:50 FIG 8

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			REV No 2	FIGURE 9		

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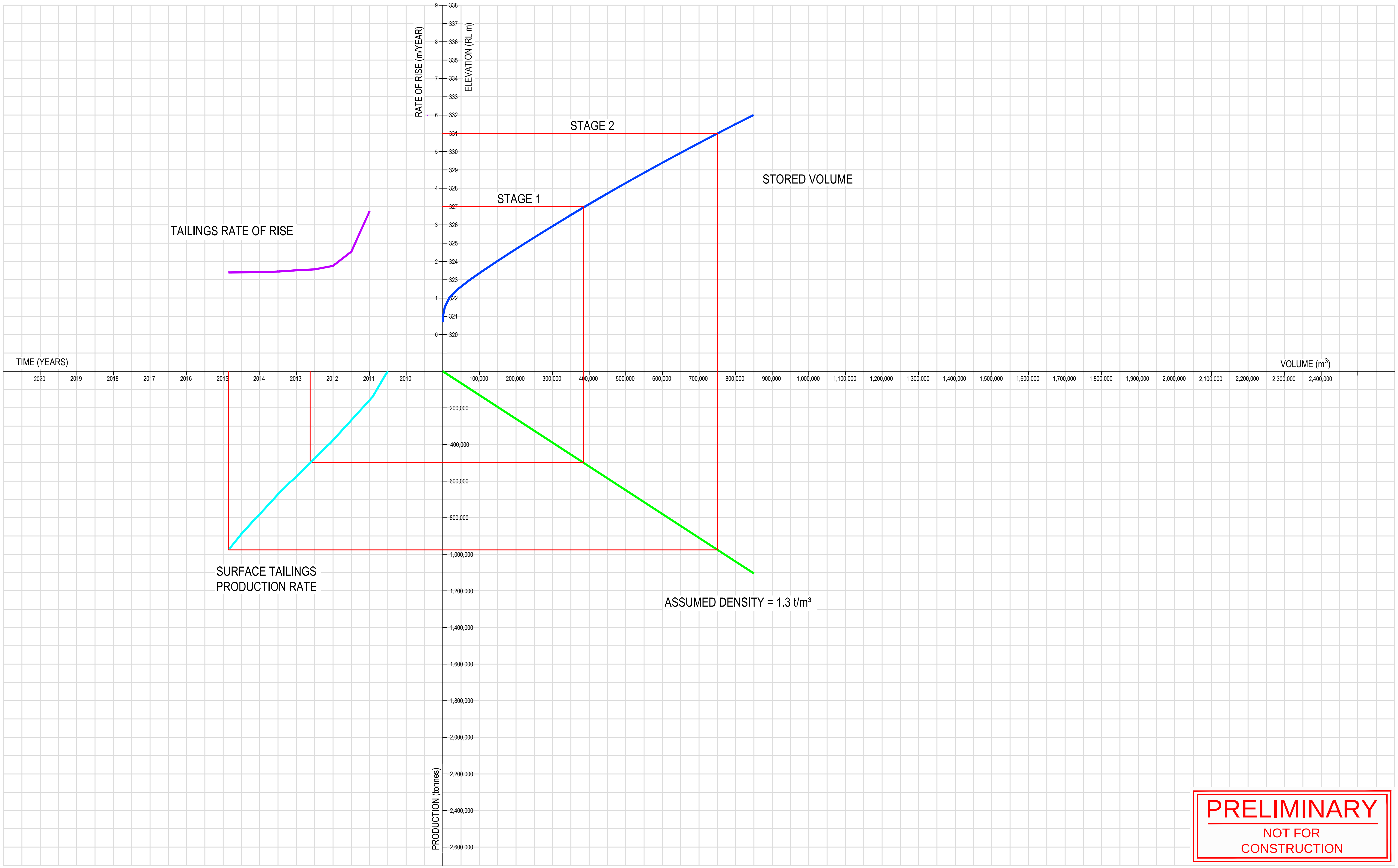

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	CHECKED DAA	DATE 14.12.09	
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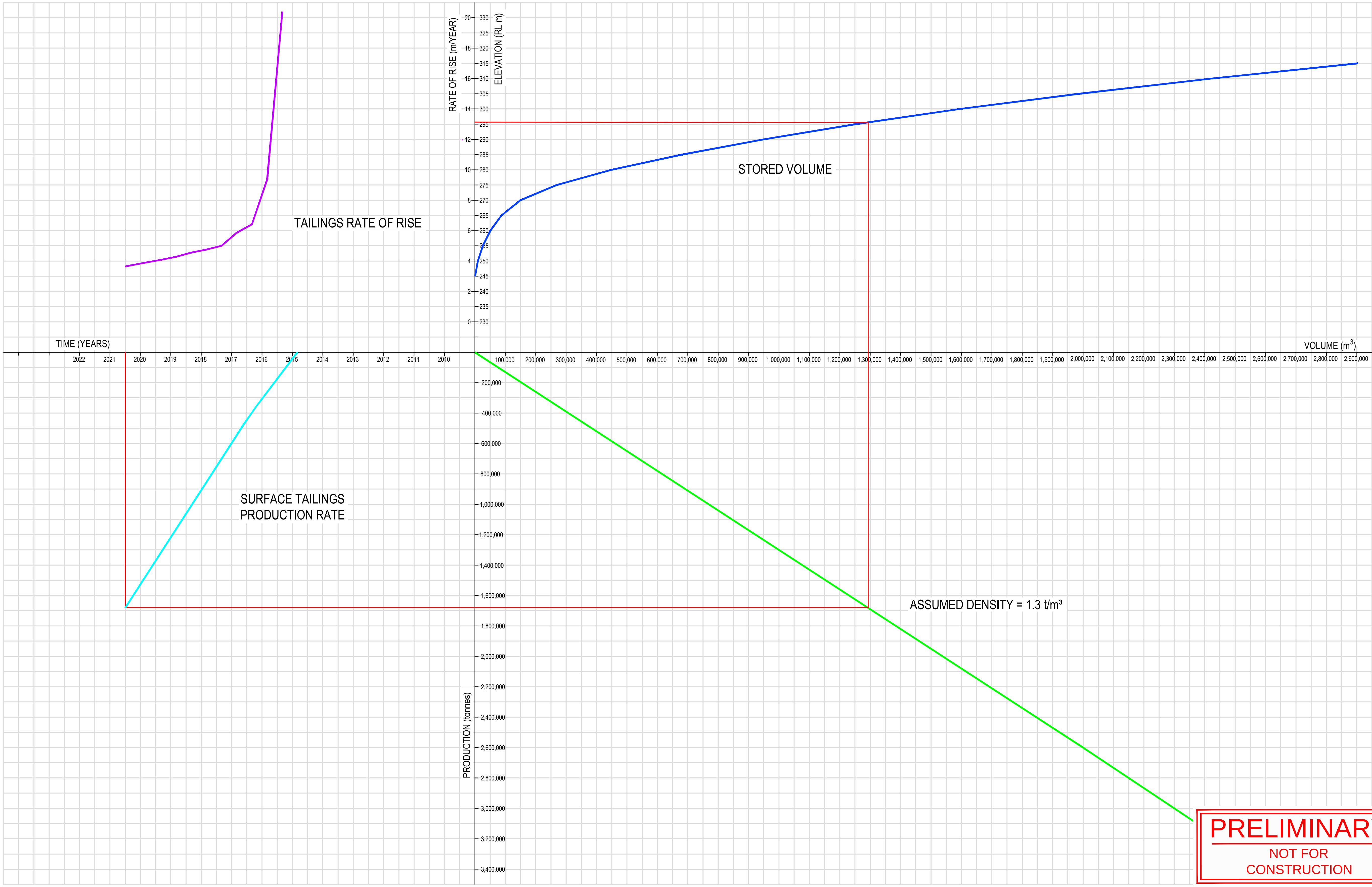

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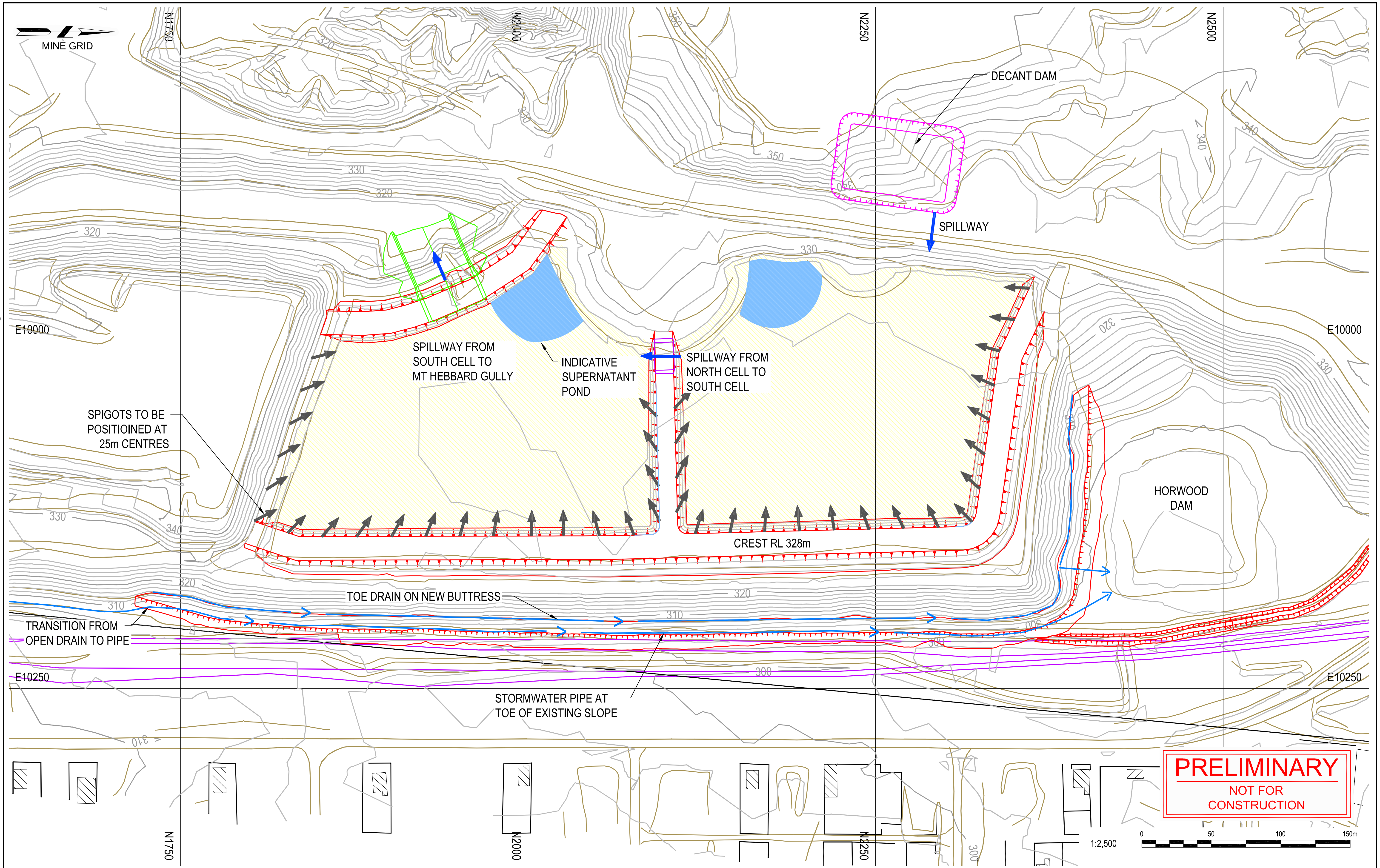

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CHECKED: DAA	DATE: 01.02.10		
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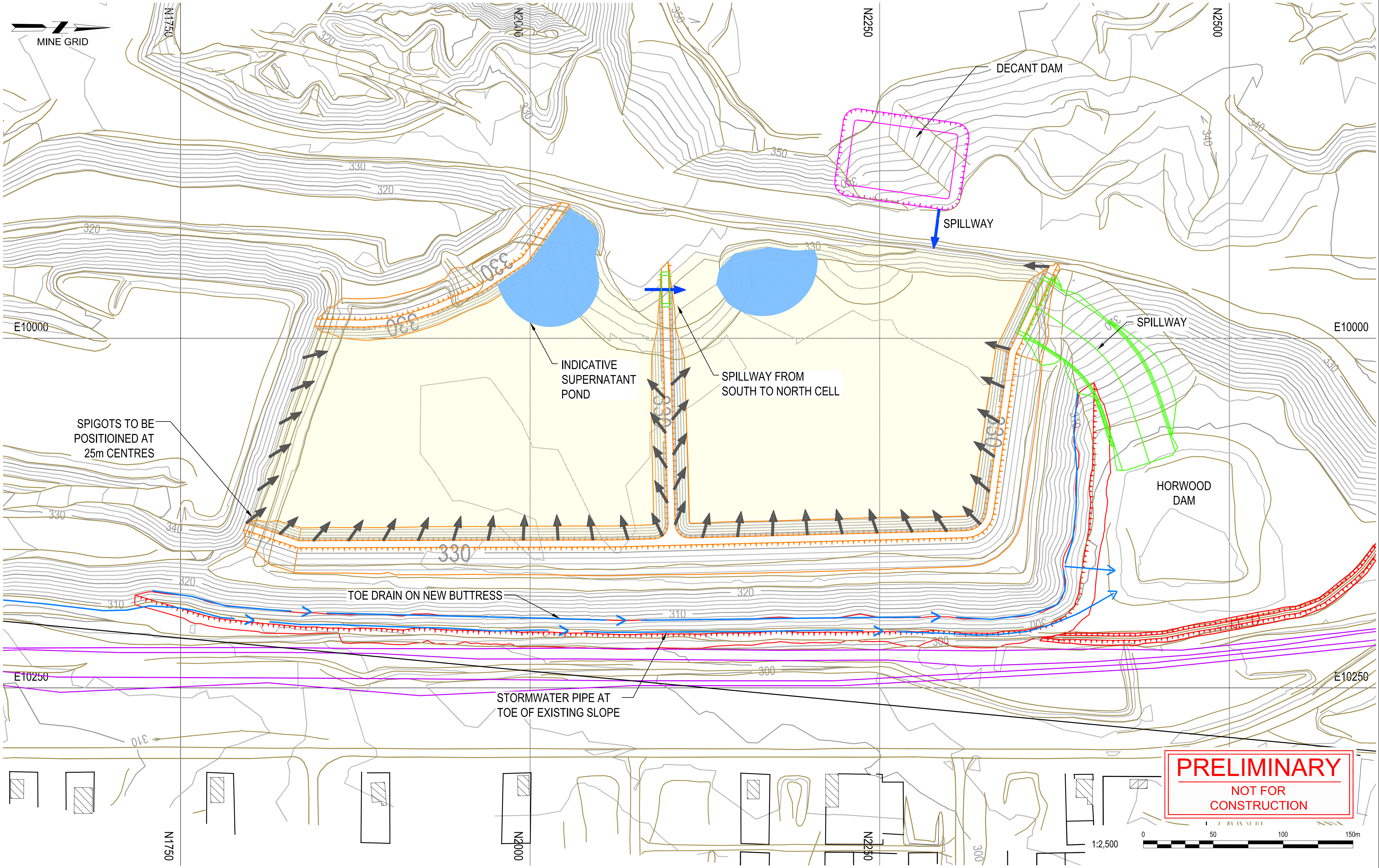

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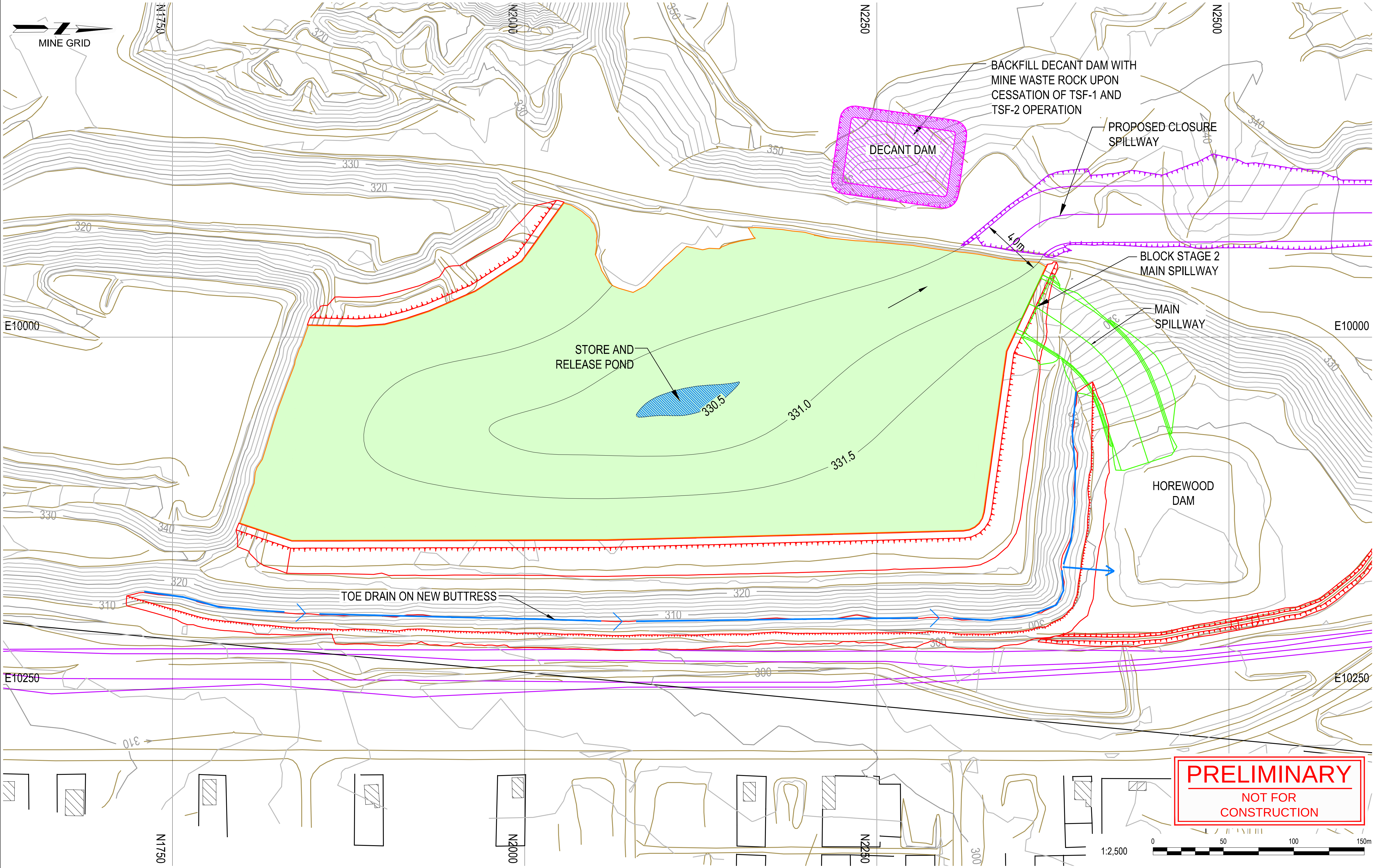
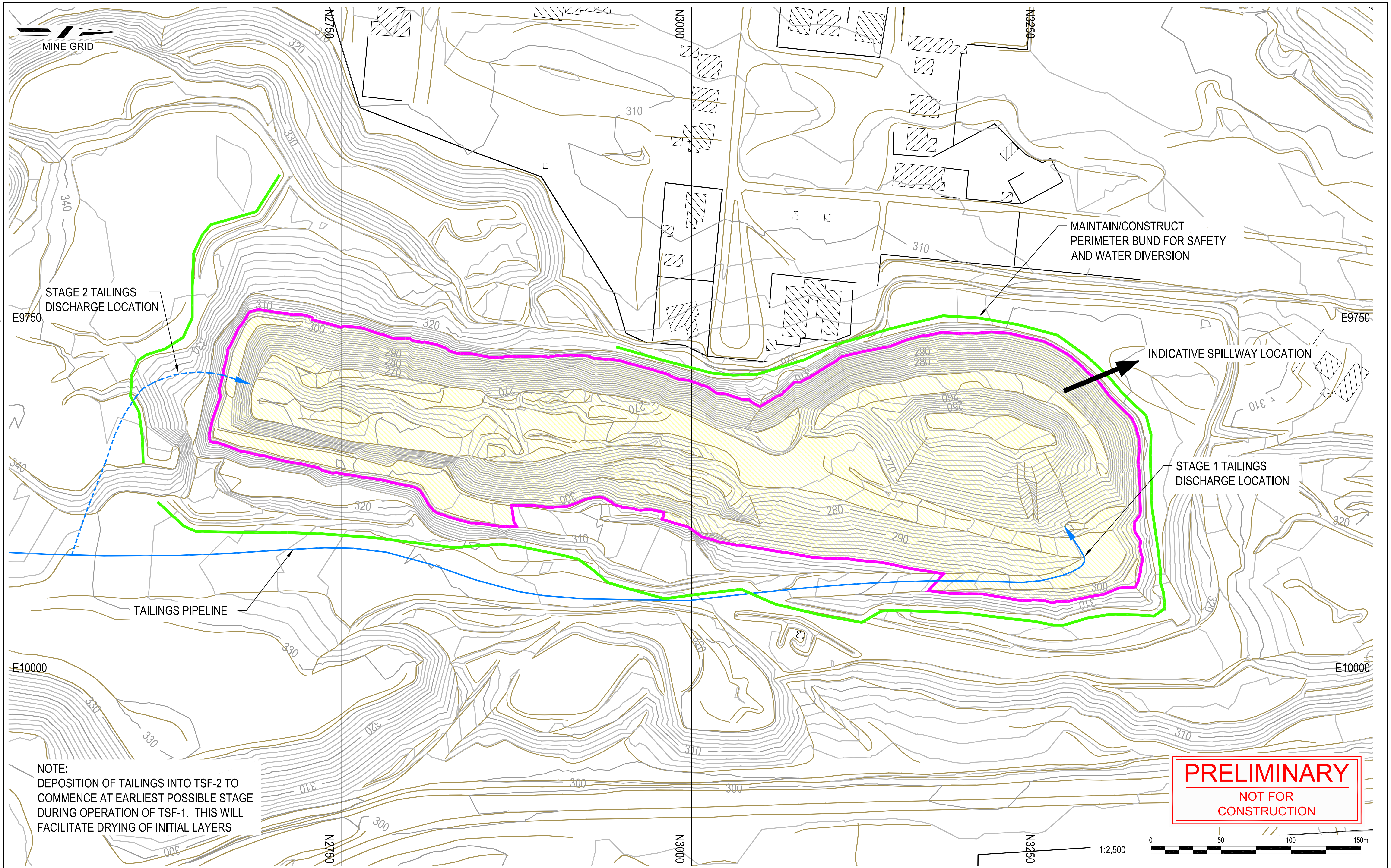


DRAWING TAKEN FROM bhill_mine.dxf DATED JANUARY 2008 (AERIAL SURVEY OF JANUARY 2000). SURVEY RE-CONTOURED IN TERRAMODEL (087611001_TM_004.PRO)

XREF:- J:\Mining\2008\087611001\Technical Doc\CADD\XREFS\087611001_XREF_Rasp Mine Base Survey.dwg, 087611001_XREF_Rasp Mine Contours 2500 (mine).dwg ,087611001_XREF_decant dam etc.dwg

Golder Associates www.golder.com GOLDER ASSOCIATES PTY LTD	CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE		
	DRAWN: DAA/MLL/MJM	DATE: 11.12.09	TITLE: TAILINGS STORAGE FEASIBILITY DESIGN TSF-1 CLOSURE PLAN		
	CHECKED: DAA	DATE: 14.12.09			
	SCALE: 1:2,500	A3	PROJECT No: 087611001	DOC No: 012	DOC TYPE: R
			FIGURE No: F0014	REV No: 2	FIGURE 14

J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES\087611001_012_R_F0014_Rev2.dwg Dec 15, 2009 - 10:43am

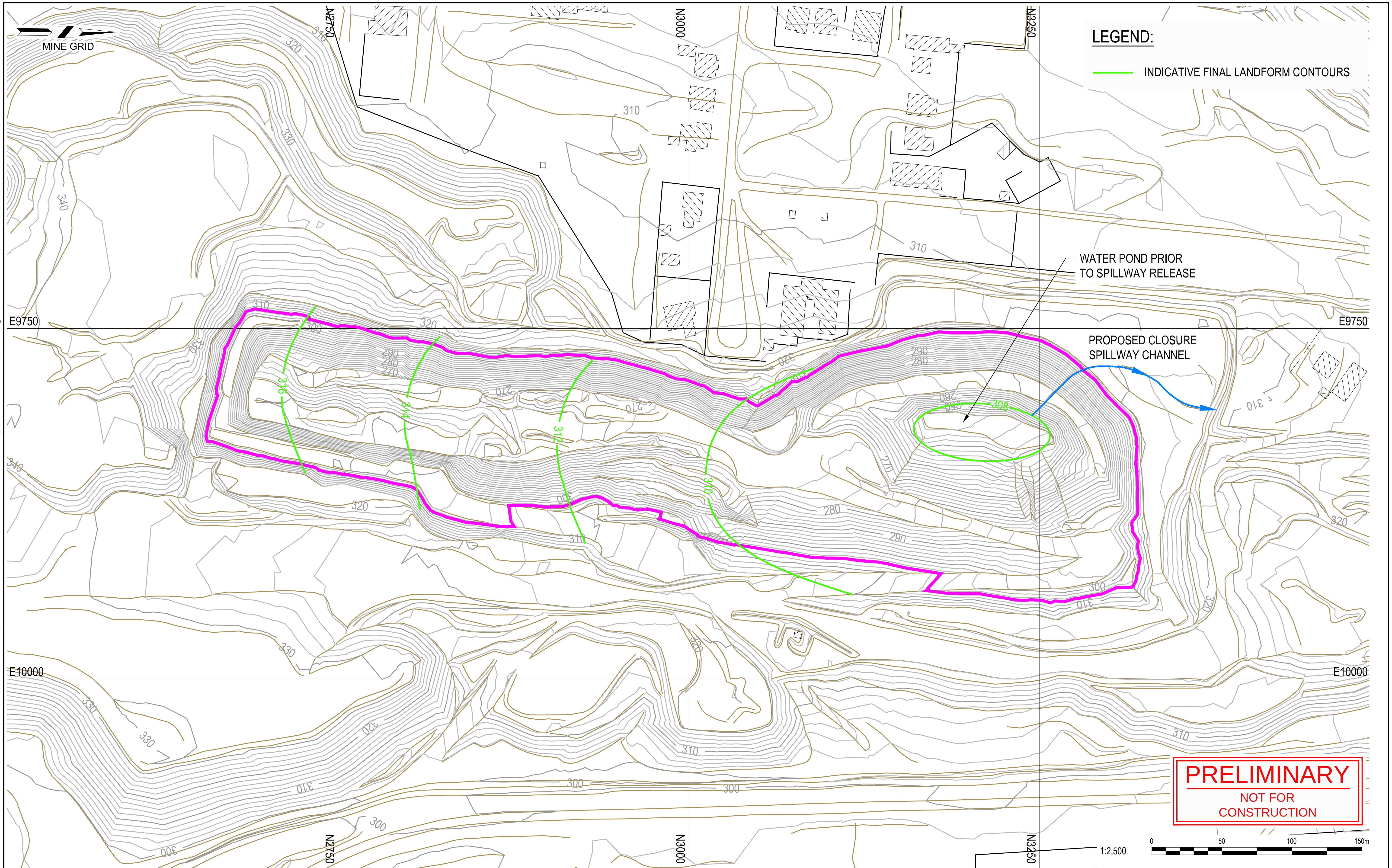


DRAWING TAKEN FROM bhill_mine.dxf DATED JANUARY 2008 (AERIAL SURVEY OF JANUARY 2000). SURVEY RE-CONTOURED IN TERRAMODEL (087611001_TM_004.PRO)

Golder Associates www.golder.com GOLDER ASSOCIATES PTY LTD	CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE	
	DRAWN: DAA/MLL/MJM	DATE: 01.02.10	TITLE: TAILINGS STORAGE FEASIBILITY DESIGN TSF-2 OPERATIONAL LAYOUT	
	CHECKED: DAA	DATE: 01.02.10		
	SCALE: 1:2,500	A3	PROJECT No: 087611001 · DOC No: 012 · DOC TYPE: R · FIGURE No: F0015 · REV No: 3	FIGURE 15

XREF:- J:\Mining\2008\087611001\Technical Doc\CADD\XREFS\087611001_XREF_Rasp Mine Base Survey.dwg, 087611001_XREF_Rasp Mine Contours 2500 (mine).dwg

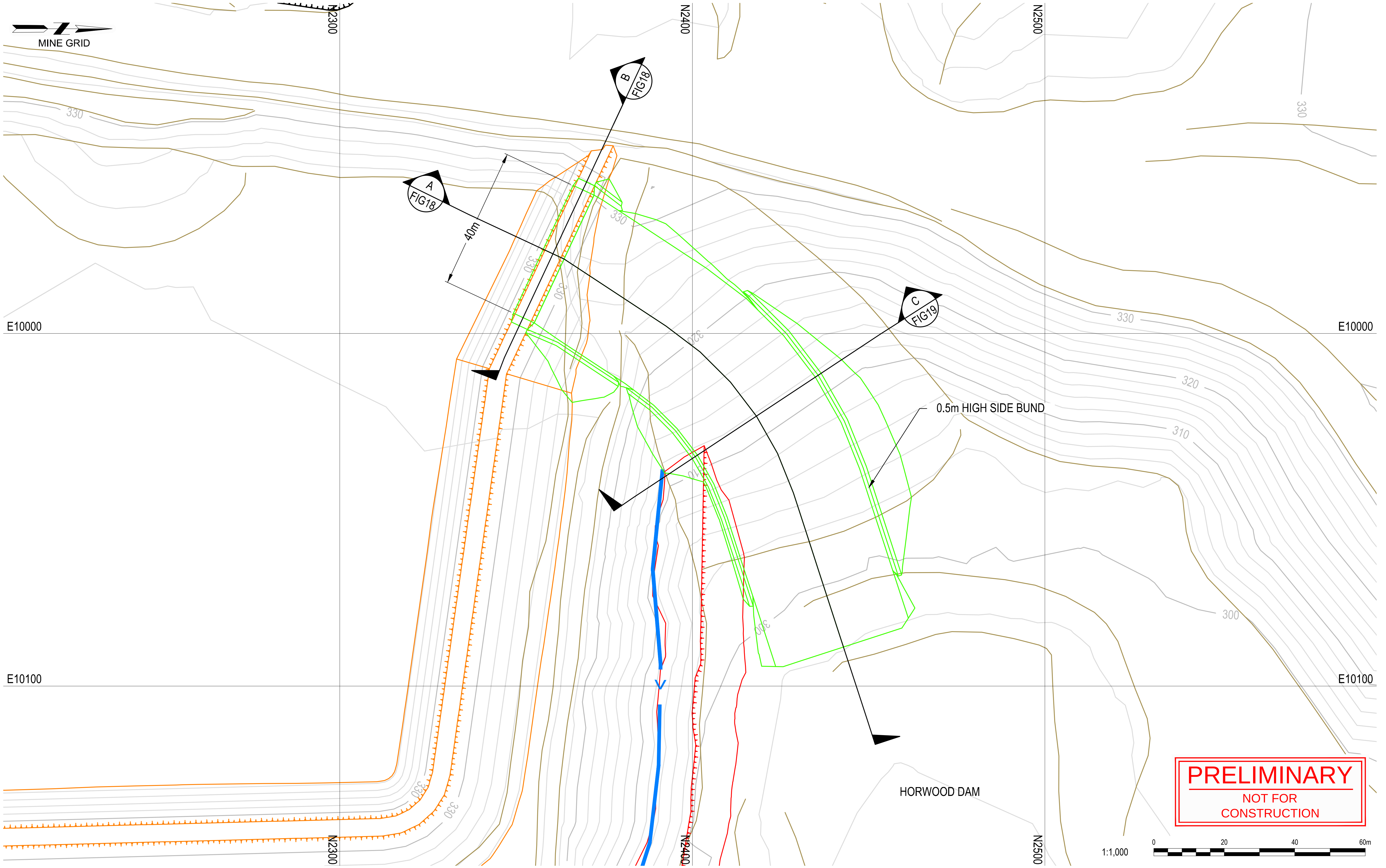
J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES\087611001_012_R_F0015_Rev3.dwg Feb 01, 2010 - 2:50pm



DRAWING TAKEN FROM bhill_mine.dxf DATED JANUARY 2008 (AERIAL SURVEY OF JANUARY 2000). SURVEY RE-CONTOURED IN TERRAMODEL (087611001_TM_004.PRO)

Golder Associates	CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE
	DRAWN: DAA/MLL/MJM	DATE: 11.12.09	TITLE: TAILINGS STORAGE FEASIBILITY DESIGN TSF-2 CLOSURE PLAN
	CHECKED: DAA	DATE: 14.12.09	
www.golder.com GOLDER ASSOCIATES PTY LTD	SCALE: 1:2,500	A3	PROJECT No 087611001 · DOC No 012 · DOC TYPE R · FIGURE No. F0016 · REV No 2 · FIGURE 16

XREF-: J:\Mining\2008\087611001\Technical Doc\CADD\XREFS\087611001_XREF_Rasp Mine Base Survey.dwg, 087611001_XREF_Rasp Mine Contours 2500 (mine).dwg



DRAWING TAKEN FROM bhill_mine.dxf DATED JANUARY 2008 (AERIAL SURVEY OF JANUARY 2000, IN MINE GRID). SURVEY RE-CONTOURED IN TERRAMODEL (087611001_TM_004.PRO)
EMBANKMENT DESIGN MODELLED IN TERRAMODEL (087611001_TM_006.PRO)

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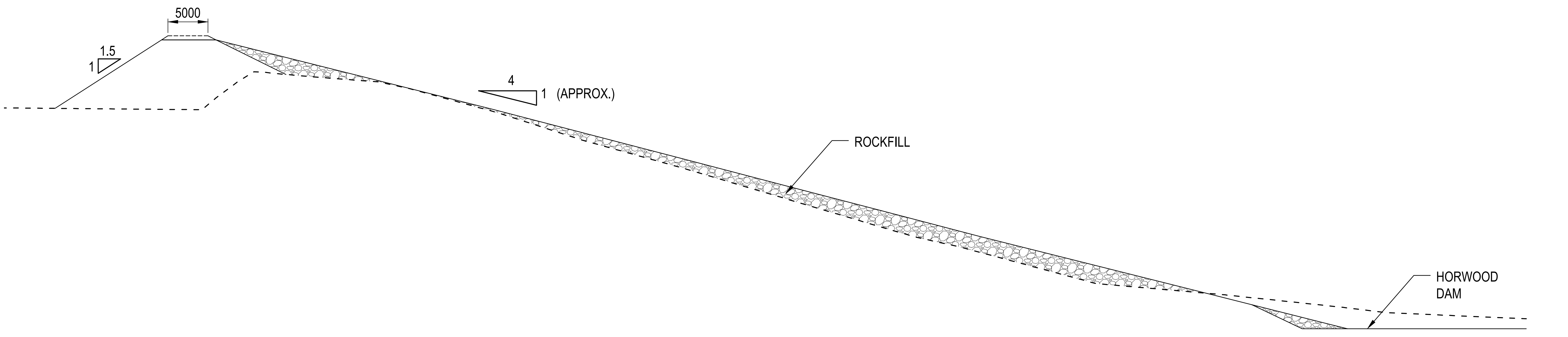
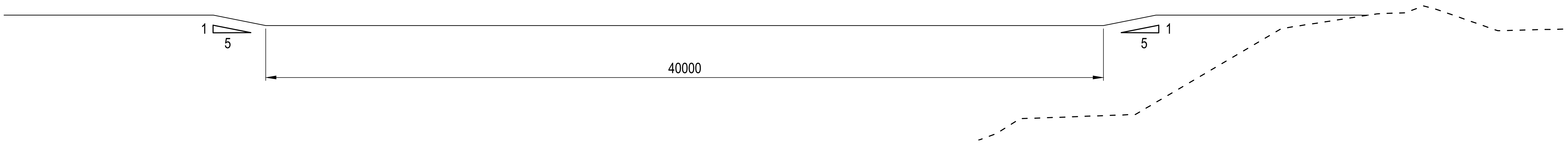
CLIENT BROKEN HILL OPERATIONS		PROJECT RASP MINE	
DRAWN DAA/MLL/MJM	DATE 11.12.09	TITLE TAILINGS STORAGE FEASIBILITY DESIGN	
CHECKED DAA	DATE 14.12.09	TSF-1 OPERATIONAL SPILLWAY - GENERAL LAYOUT	
SCALE 1:1,000	A3	PROJECT No 087611001 · DOC No 012 · DOC TYPE R · FIGURE No. F0017 · REV No 2	FIGURE 17

J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES\087611001_012_R_F0017_Rev2.dwg Dec 15, 2009 - 10:53am



SECTION (A / FIG17) SPILLWAY LONG SECTION
SCALE 1:500



SECTION (B / FIG17) SPILLWAY CREST
SCALE 1:200

PRELIMINARY
NOT FOR CONSTRUCTION

1:200 — 0, 4, 8, 12m
1:500 — 0, 10, 20, 30m

CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE
DRAWN: DAA/MLL/MJM	DATE: 11.12.09	TITLE: TAILINGS STORAGE FEASIBILITY DESIGN
CHECKED: DAA	DATE: 14.12.09	TSF-1 OPERATIONAL SPILLWAY - CROSS SECTIONS - SHEET 1 OF 2
SCALE: AS SHOWN	A3	PROJECT No 087611001 · DOC No 012 · DOC TYPE R · FIGURE No F0018 · REV No 2 · FIGURE 18

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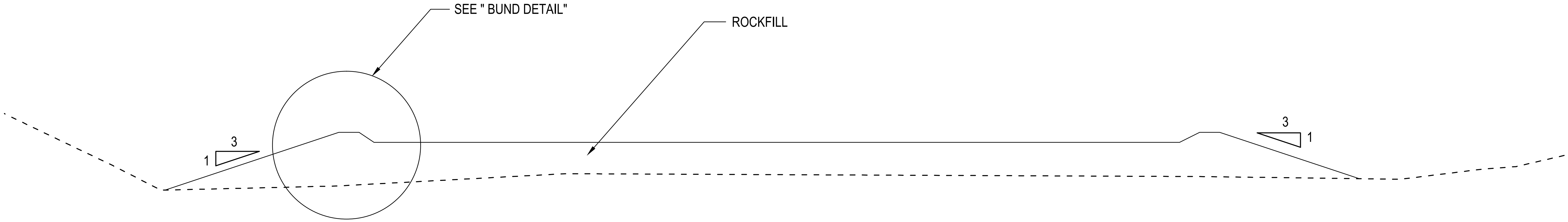
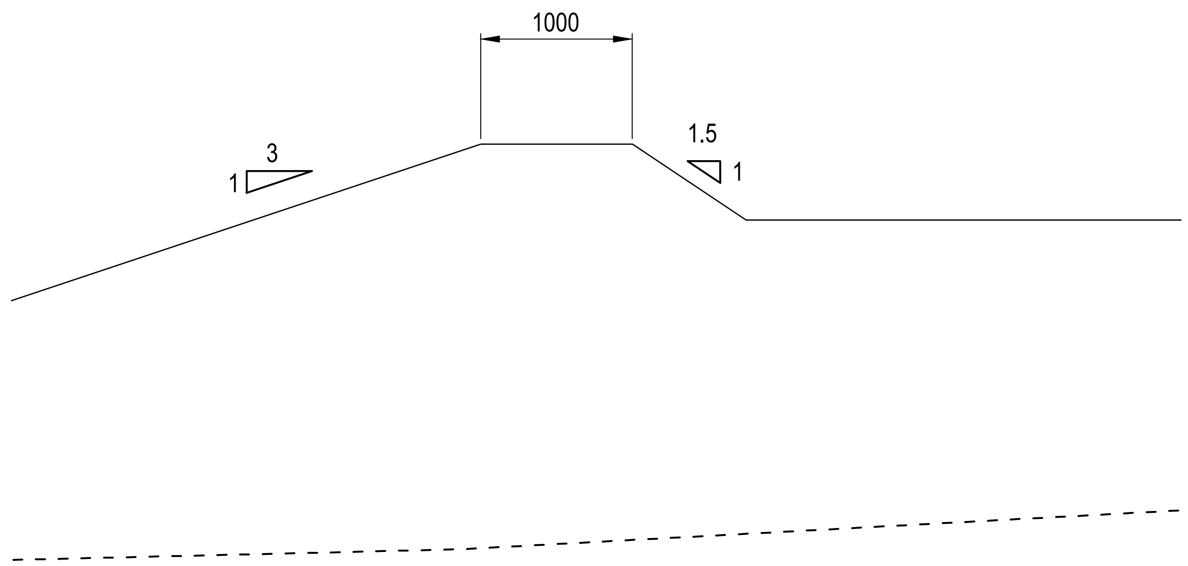
J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES\087611001_012_R_F0018_Rev2.dwg Dec 15, 2009 - 10:54am



SECTION C SPILLWAY CHUTE
FIG17
1:200

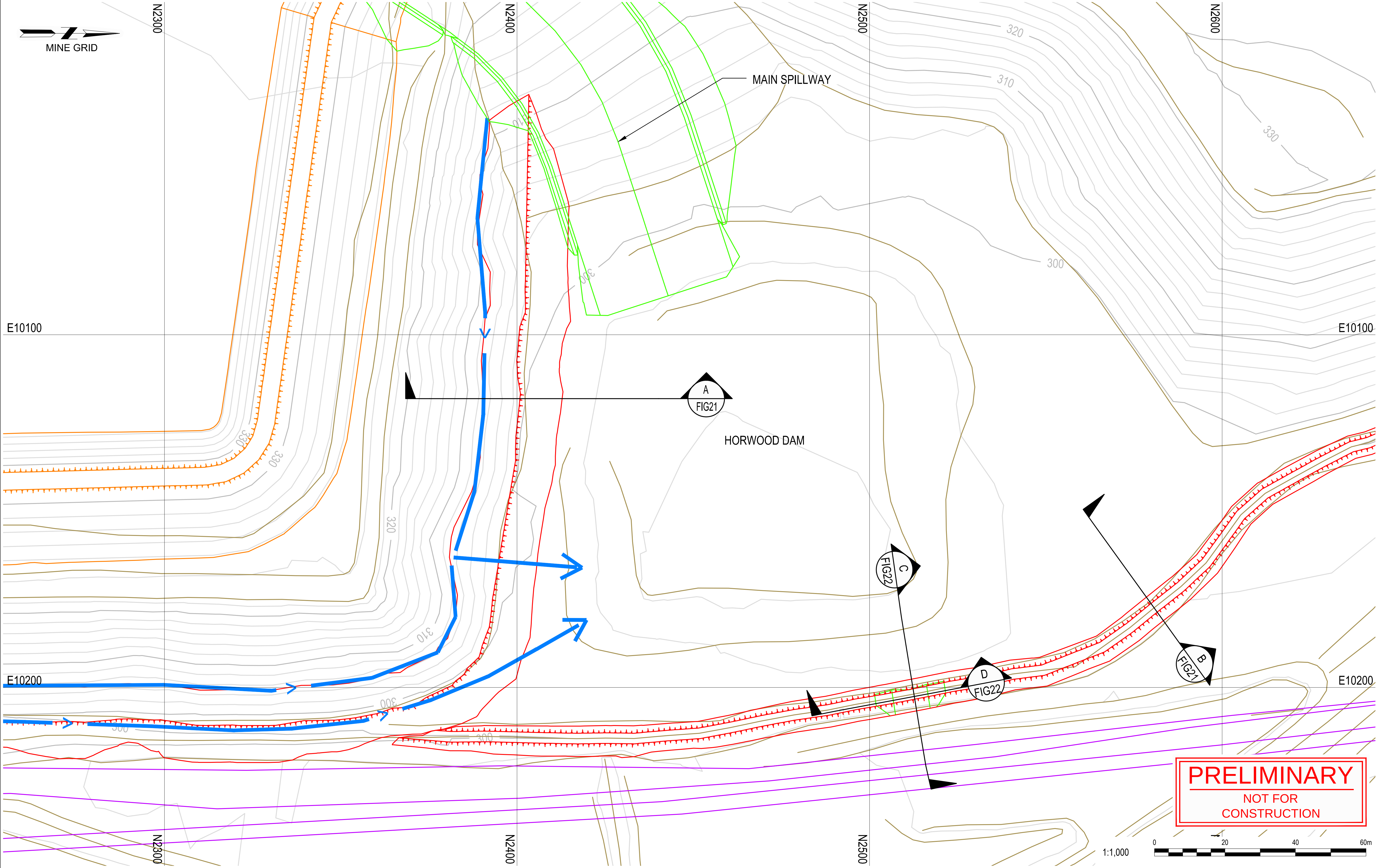


BUND DETAIL
SCALE 1:50

PRELIMINARY
NOT FOR CONSTRUCTION

1:50 — 0, 1,000, 2,000, 3,000mm
1:200 — 0, 4, 8, 12m

Golder Associates www.golder.com GOLDER ASSOCIATES PTY LTD	CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE
	DRAWN: DAA/MLL/MJM	DATE: 11.12.09	TITLE: TAILINGS STORAGE FEASIBILITY DESIGN
	CHECKED: DAA	DATE: 14.12.09	TSF-1 OPERATIONAL SPILLWAY - CROSS SECTIONS - SHEET 2 OF 2
	SCALE: AS SHOWN	A3	PROJECT No: 087611001; DOC No: 012; DOC TYPE: R; FIGURE No: F0019; REV No: 2; FIGURE 19



DRAWING TAKEN FROM bhill_mine.dxf DATED JANUARY 2008 (AERIAL SURVEY OF JANUARY 2000, IN MINE GRID). SURVEY RE-CONTOURED IN TERRAMODEL (087611001_TM_004.PRO)
EMBANKMENT DESIGN MODELLED IN TERRAMODEL (087611001_TM_006.PRO)

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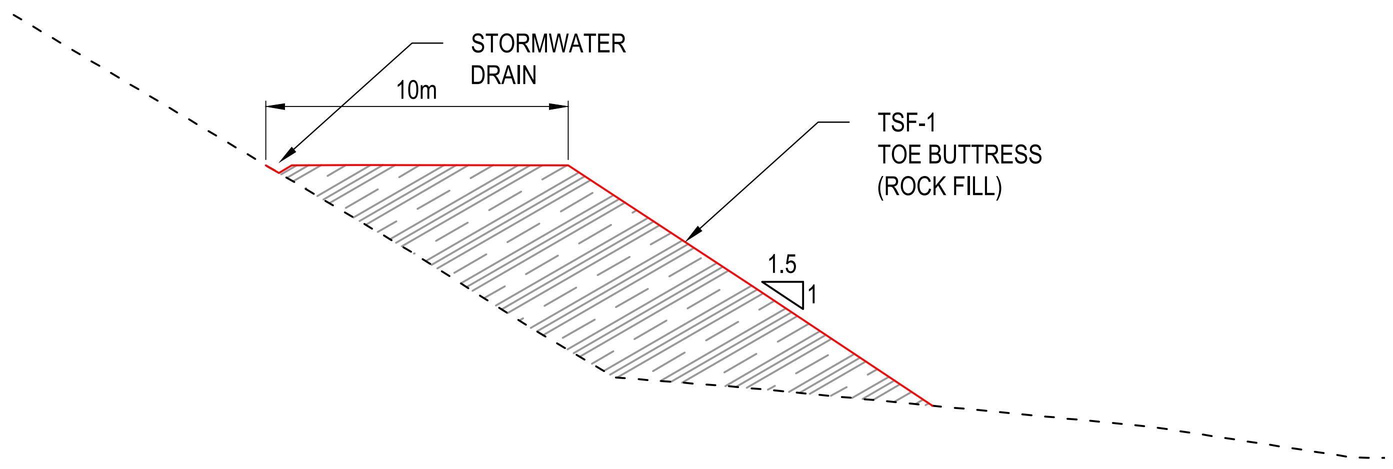
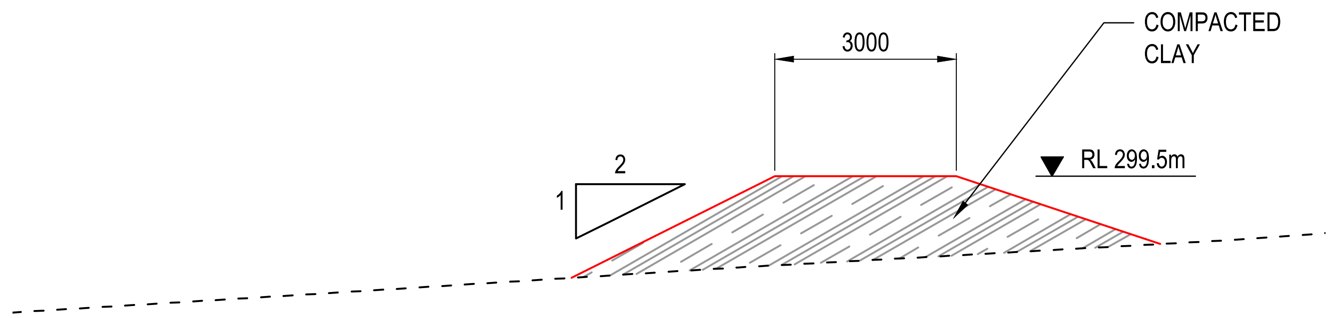
CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE		
DRAWN: DAA/MLL	DATE: 19.05.08	TITLE: TAILINGS STORAGE FEASIBILITY DESIGN HORWOOD DAM - GENERAL LAYOUT		
CHECKED: DAA	DATE: 14.12.09			
SCALE: 1:1,000	A3	PROJECT No: 087611001	DOC No: 012	DOC TYPE: R
FIGURE No: F0020	REV No: 1	FIGURE 20		

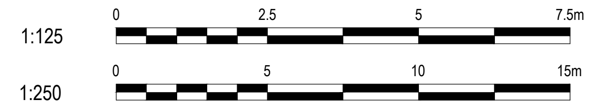


SECTION (A / FIG20) TOE BUTTRESS
SCALE 1:250



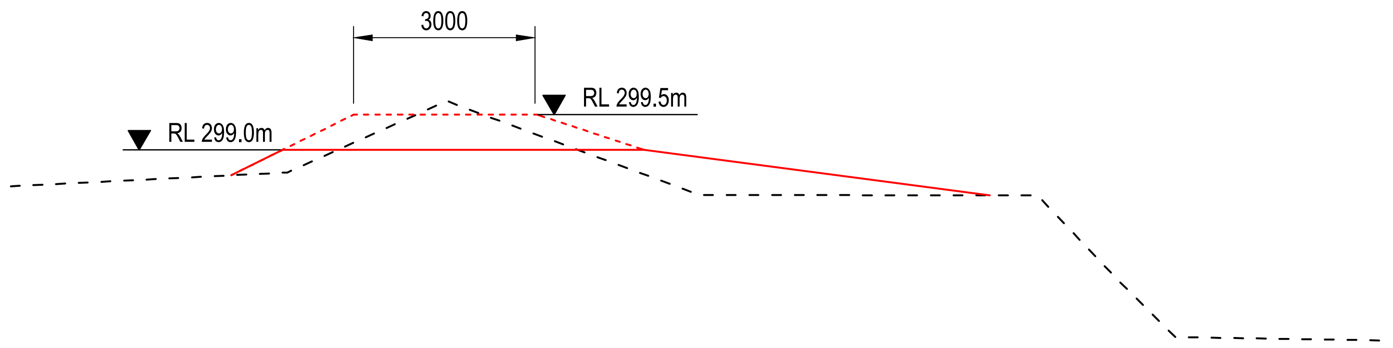
SECTION (B / FIG20) HORWOOD DAM EMBANKMENT
SCALE 1:125

PRELIMINARY
NOT FOR
CONSTRUCTION

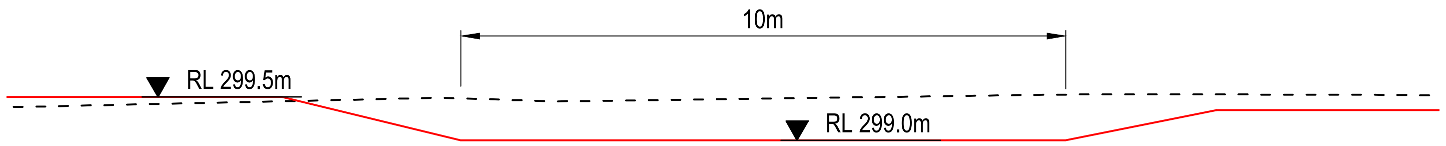


Golder Associates www.golder.com GOLDER ASSOCIATES PTY LTD	CLIENT BROKEN HILL OPERATIONS		PROJECT RASP MINE		
	DRAWN DAA/MLL/MJM	DATE 11.12.09	TITLE TAILINGS STORAGE FEASIBILITY DESIGN HORWOOD DAM - CROSS SECTIONS - SHEET 1 OF 2		
	CHECKED DAA	DATE 14.12.09			
	SCALE AS SHOWN	A3	PROJECT No 087611001	DOC No 012	DOC TYPE R
			FIGURE No F0021	REV No 2	FIGURE 21

J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES\087611001_012_R_F0021_Rev2.dwg Dec 15, 2009 - 10:59am



SECTION C FIG20 SPILLWAY LONG SECTION
SCALE 1:125

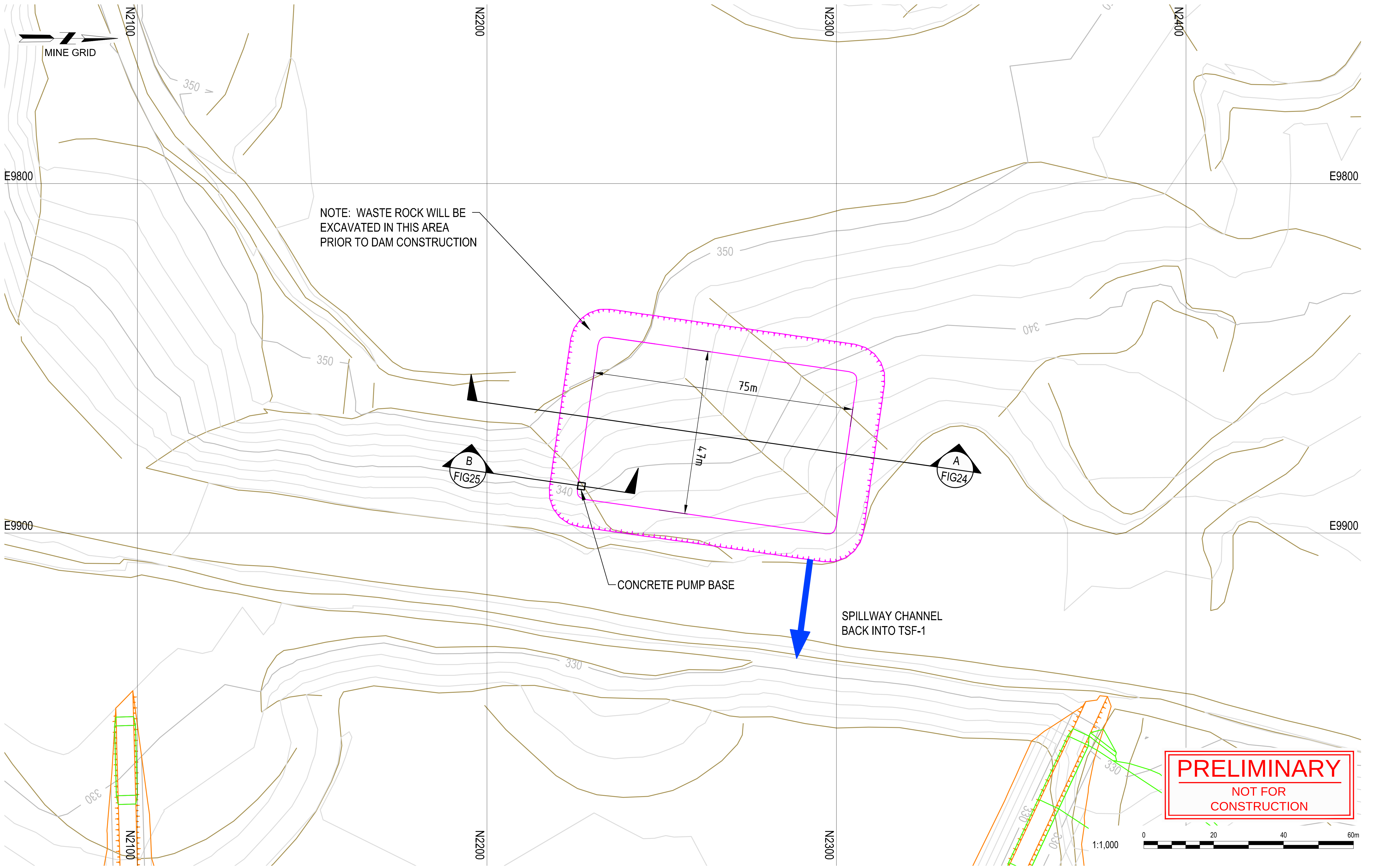


SECTION D FIG20 SPILLWAY CREST
SCALE 1:125

PRELIMINARY
NOT FOR CONSTRUCTION

1:125 0 2.5 5 7.5m

Golder Associates	CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE			
	DRAWN: DAA/MLL/MJM	DATE: 11.12.09	TITLE: TAILINGS STORAGE FEASIBILITY DESIGN			
	CHECKED: DAA	DATE: 14.12.09	HORWOOD DAM - CROSS SECTIONS - SHEET 2 OF 2			
www.golder.com GOLDER ASSOCIATES PTY LTD	SCALE: 1:125	A3	PROJECT No: 087611001	DOC No: 012	DOC TYPE: R	FIGURE No: F0022 REV No: 2 FIGURE 22



DRAWING TAKEN FROM bhill_mine.dxf DATED JANUARY 2008 (AERIAL SURVEY OF JANUARY 2000, IN MINE GRID). SURVEY RE-CONTOURED IN TERRAMODEL (087611001_TM_004.PRO)
EMBANKMENT DESIGN MODELLED IN TERRAMODEL (087611001_TM_006.PRO)

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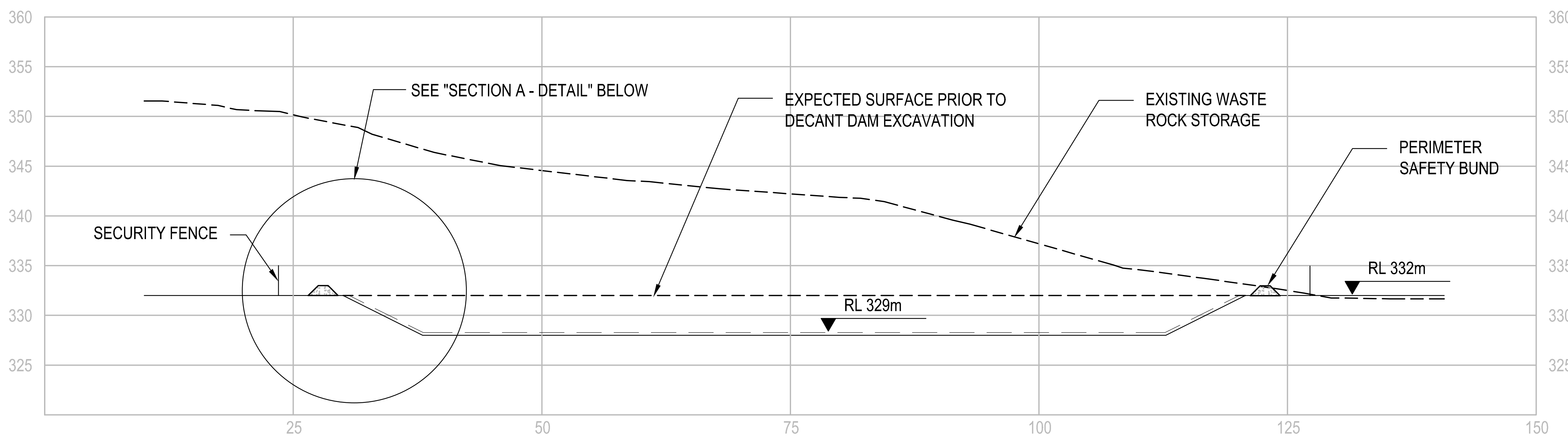
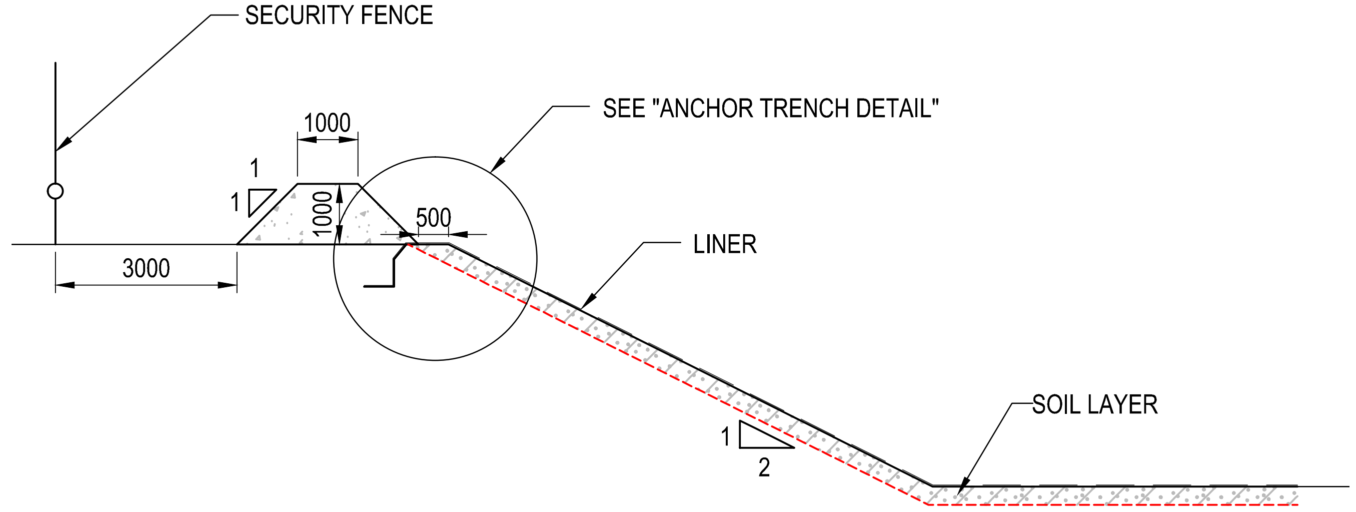
CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE	
DRAWN: DAA/MLL/MJM	DATE: 11.12.09	TITLE: TAILINGS STORAGE FEASIBILITY DESIGN DECANT DAM - GENERAL LAYOUT	
CHECKED: DAA	DATE: 14.12.09		
SCALE: 1:1,000	A3	PROJECT No 087611001, DOC No 012, DOC TYPE R, FIGURE No F0023, REV No 2	FIGURE 23

J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES\087611001_012_R_F0023_Rev2.dwg Dec 15, 2009 - 11:03am



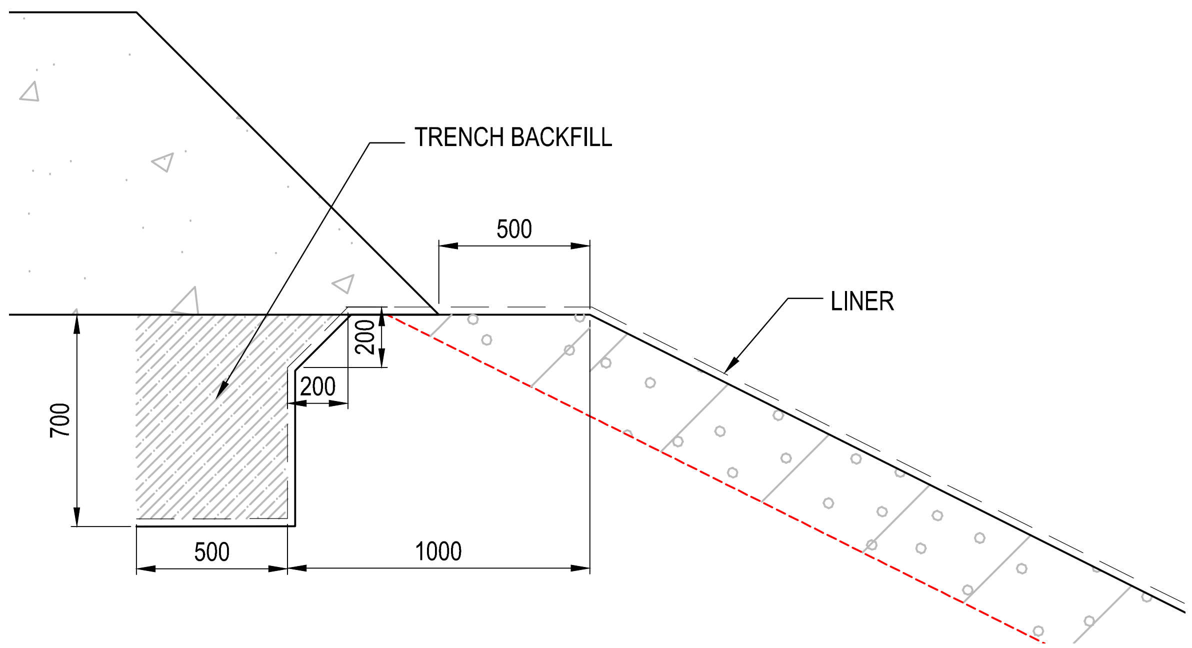
SECTION (A / FIG23) DECANT DAM

SCALE 1:500



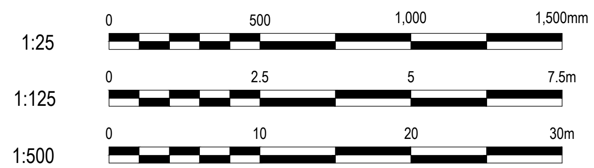
SECTION A - DETAIL

SCALE 1:125



ANCHOR TRENCH DETAIL

SCALE 1:25

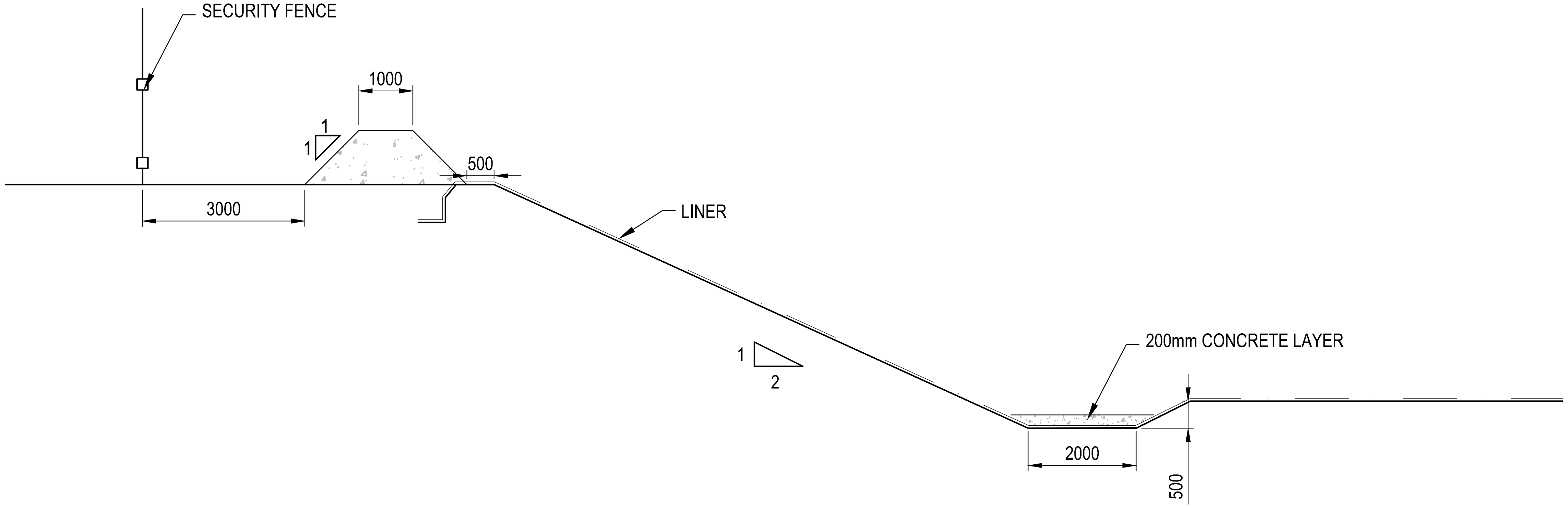


PRELIMINARY

NOT FOR CONSTRUCTION

Golder Associates	CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE	
	DRAWN: DAA/MLL/MJM	DATE: 11.12.09	TITLE: TAILINGS STORAGE FEASIBILITY DESIGN DECANT DAM - CROSS SECTIONS 1 OF 2	
	CHECKED: DAA	DATE: 14.12.09		
www.golder.com GOLDER ASSOCIATES PTY LTD	SCALE: AS SHOWN	A3	PROJECT No: 087611001 · DOC No: 012 · DOC TYPE: R · FIGURE No: F0024 · REV No: 2	FIGURE 24

J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES\087611001_012_R_F0024_Rev2.dwg Dec 15, 2009 - 11:04am



SECTION (B / FIG23) CONCRETE PUMP BASE

SCALE 1:100

PRELIMINARY
NOT FOR CONSTRUCTION

1:100 0 2 4 6m

CLIENT BROKEN HILL OPERATIONS			PROJECT RASP MINE	
DRAWN DAA/MLL/MJM	DATE 11.12.09		TITLE TAILINGS STORAGE FEASIBILITY DESIGN DECANT DAM - CROSS SECTIONS 2 OF 2	
CHECKED DAA	DATE 14.12.09			
SCALE 1:100	A3		PROJECT No 087611001 / DOC No 012 / DOC TYPE R / FIGURE No F0025 / REV No 2	FIGURE 25

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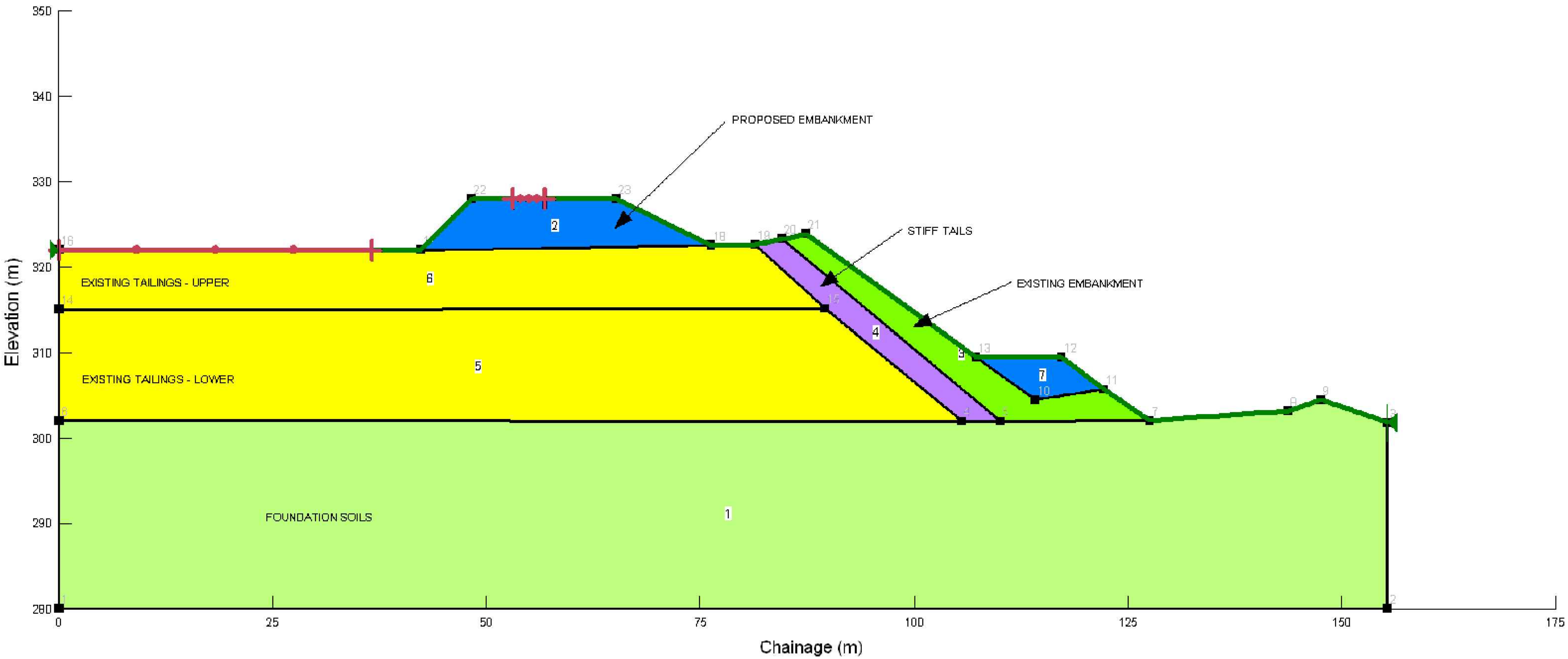
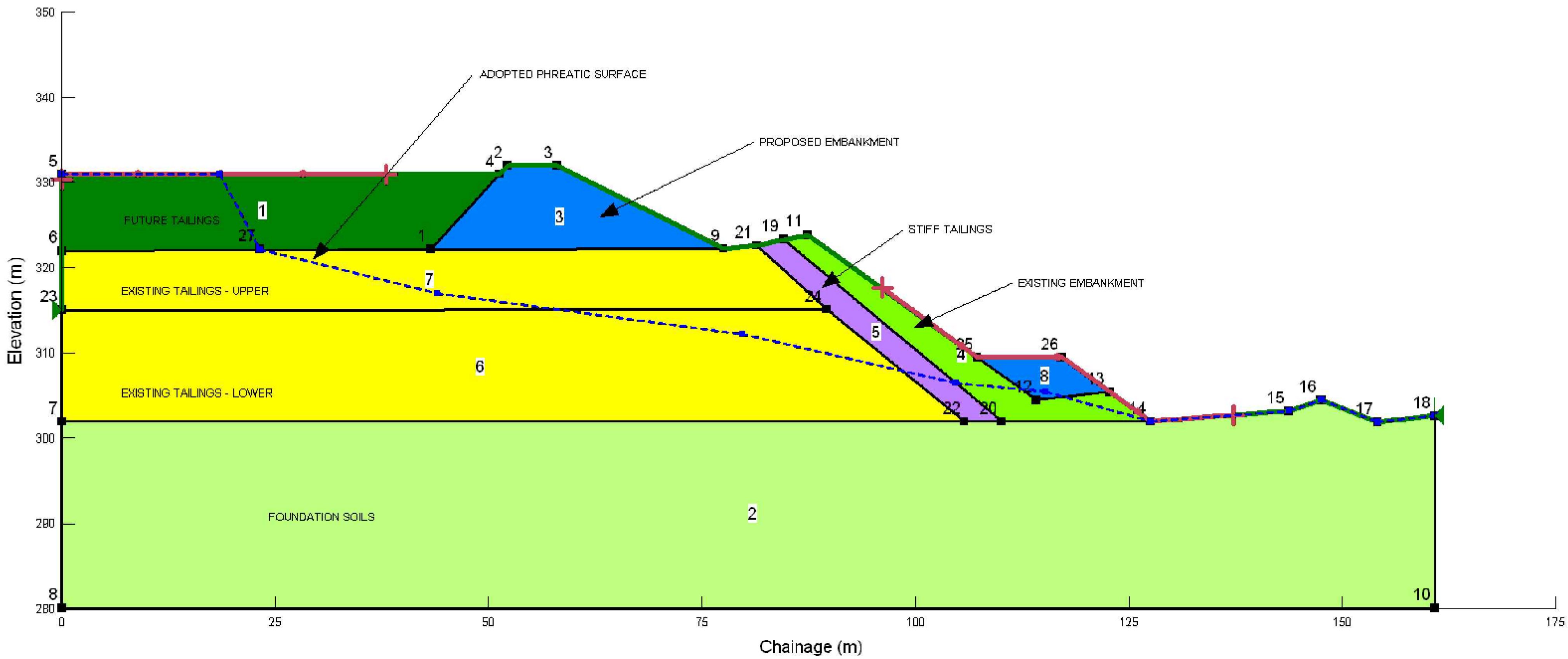
J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES\087611001_012_R_F0025_Rev2.dwg Dec 15, 2009 - 11:06am



PRELIMINARY

Golder Associates	CLIENT: BROKEN HILL OPERATIONS				PROJECT: RASP MINE			
	DRAWN BY: DH	DATE: 3.12.09			DRAWING TITLE: TAILINGS STORAGE FEASIBILITY DESIGN STABILITY ANALYSIS SECTION - START EMBANKMENT			
	CHECKED BY: DAA	DATE: 14.12.09						
www.golder.com GOLDER ASSOCIATES PTY LTD	SCALE: NOT TO SCALE		SHEET SIZE: A4	PROJECT No: 087611001	DOC No: 012	DOC TYPE: R	FIGURE No: F0026	REVISION: 2 — FIGURE 26

Plot Date: 15 December 2009 Time:11:07:30 AM By: Hrustanpasic, Dado Path: J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES - File Name:087611001_012_R_F0026_Rev2.dwg
Xref: ############



PRELIMINARY

Golder Associates	CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE
	DRAWN BY: DH	DATE: 3.12.09	DRAWING TITLE: TAILINGS STORAGE FEASIBILITY DESIGN STABILITY ANALYSIS SECTION - FINAL CONFIGURATION
	CHECKED BY: DAA	DATE: 14.12.09	
www.golder.com GOLDER ASSOCIATES PTY LTD	SCALE: NOT TO SCALE	SHEET SIZE: A4	PROJECT No: 087611001 · DOC No: 012 · DOC TYPE: R · FIGURE No: F0027 · REVISION: 2 · FIGURE 27

Plot Date: 15 December 2009 Time:11:08:33 AM By: Hrustanpasic, Dado Path: J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES - File Name:087611001_012_R_F0027_Rev2.dwg
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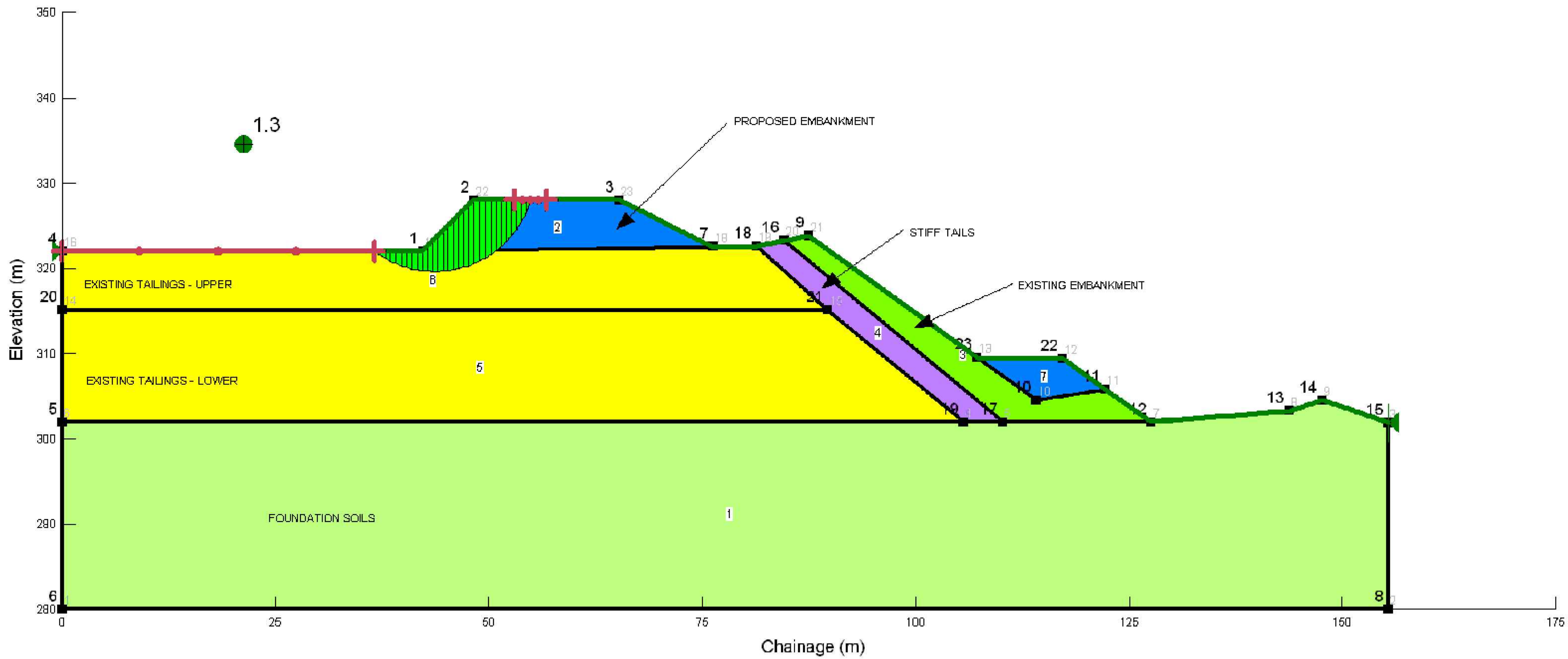

PRELIMINARY

CLIENT	BROKEN HILL OPERATIONS	PROJECT	RASP MINE
DRAWN BY	DH	DATE	3.12.09
CHECKED BY	DAA	DATE	14.12.09
SCALE	NOT TO SCALE	SHEET SIZE	A4

DRAWING TITLE
TAILINGS STORAGE FEASIBILITY DESIGN
STABILITY ANALYSIS OUTPUT - START EMBANKMENT

PROJECT No	DOC No	DOC TYPE	FIGURE No	REVISION	
087611001	012	R	F0028	2	FIGURE 28

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Plot Date: 15 December 2009 Time:11:09:24 AM By: Hrustanpasic, Dado Path: J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES - File Name:087611001_012_R_F0028_Rev2.dwg
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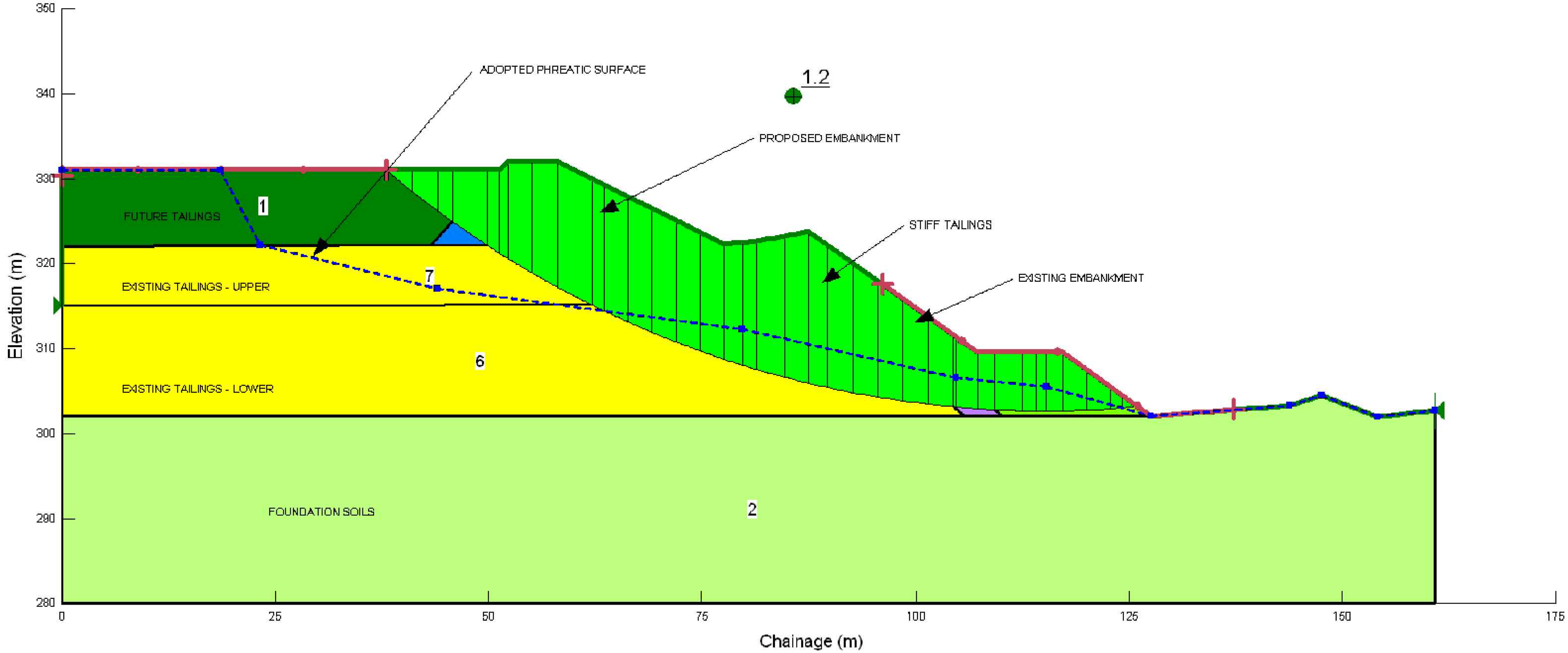
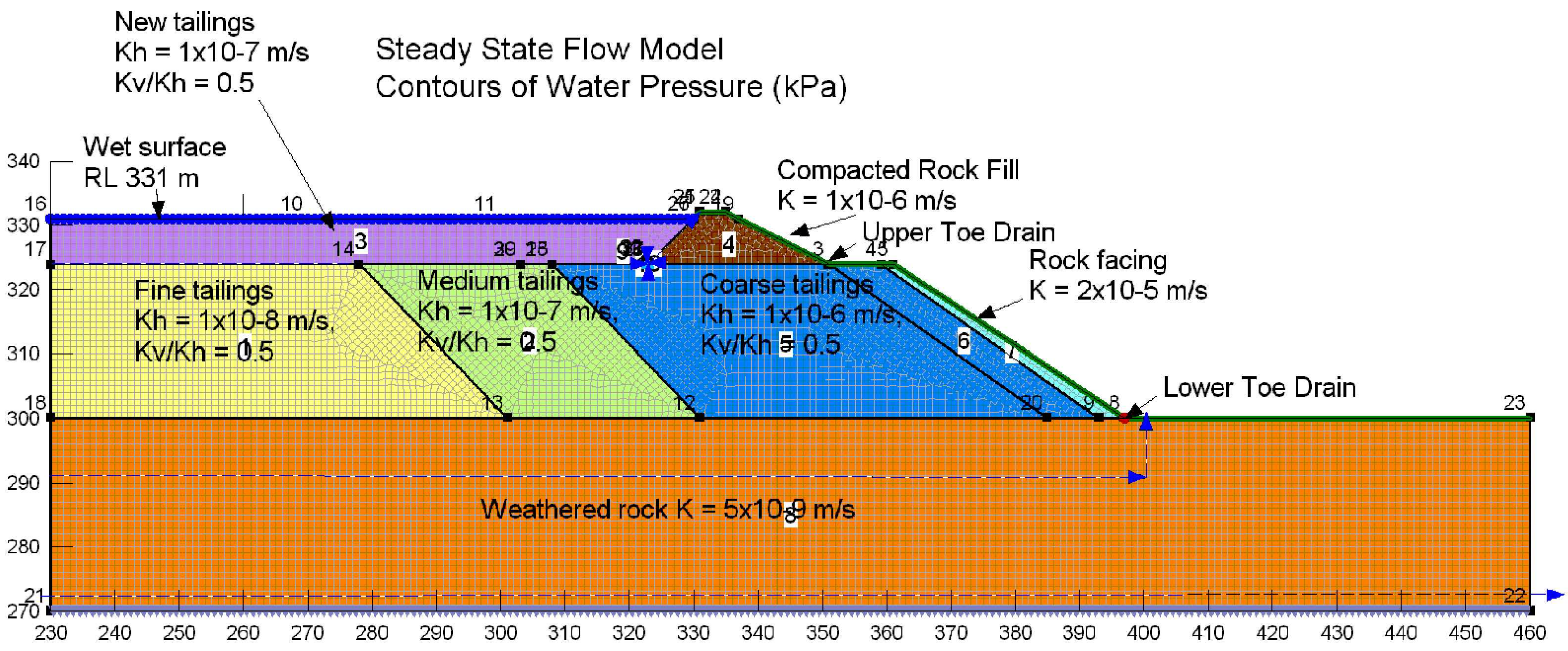


PRELIMINARY

Golder Associates	CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE				
	DRAWN BY: DH	DATE: 3.12.09	DRAWING TITLE: TAILINGS STORAGE FEASIBILITY DESIGN STABILITY ANALYS OUTPUT - FINAL CONFIGURATION				
	CHECKED BY: DAA	DATE: 14.12.09					
www.golder.com GOLDER ASSOCIATES PTY LTD	SCALE: NOT TO SCALE	SHEET SIZE: A4	PROJECT No: 087611001	DOC No: 012	DOC TYPE: R	FIGURE No: F0029	REVISION: 2 — FIGURE 29

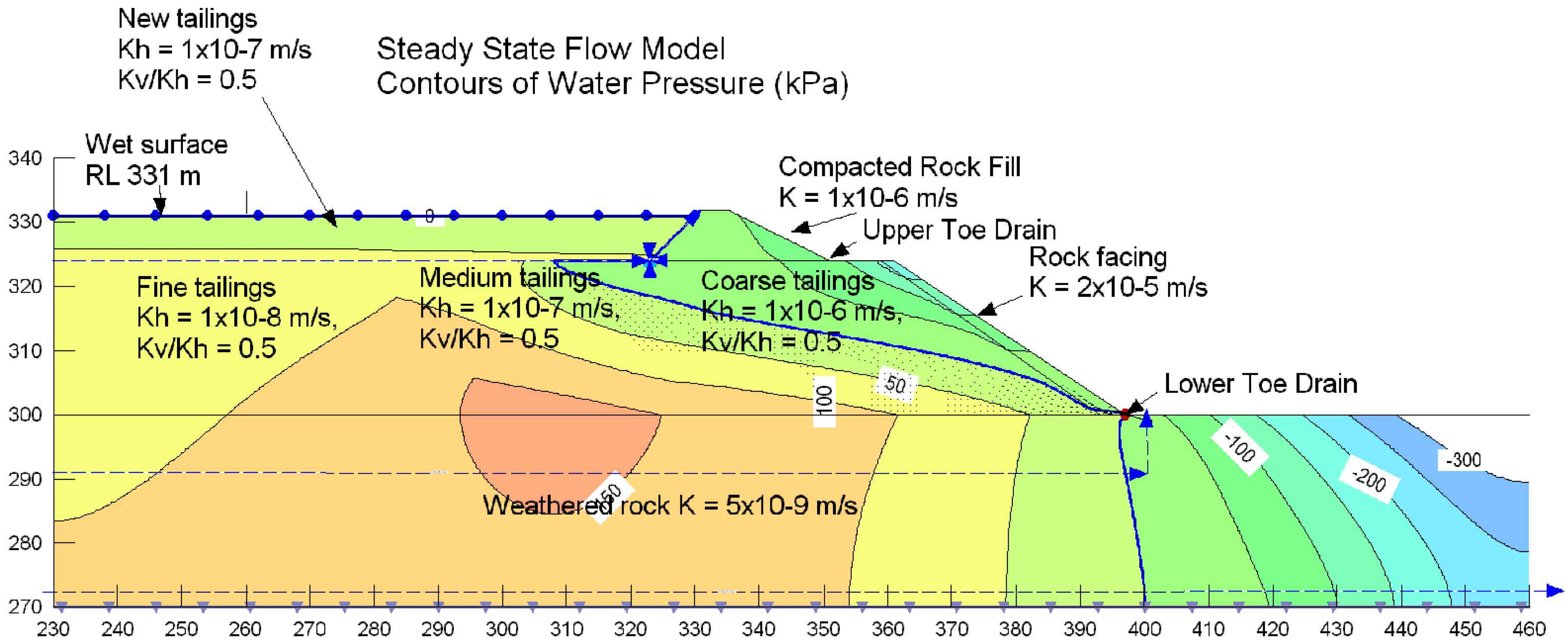
Plot Date: 15 December 2009 Time:11:10:19 AM By: Hrustanpasic, Dado Path: J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES - File Name:087611001_012_R_F0029_Rev2.dwg
Xref: ###########



PRELIMINARY

Golder Associates www.golder.com GOLDER ASSOCIATES PTY LTD	CLIENT: BROKEN HILL OPERATIONS	PROJECT: RASP MINE
	DRAWN BY: DH — DATE: 3.12.09	DRAWING TITLE: TAILINGS STORAGE FEASIBILITY DESIGN STEADY STATE SEEPAGE MODEL
	CHECKED BY: DAA — DATE: 14.12.09	
	SCALE: NOT TO SCALE — SHEET SIZE: A4	PROJECT No: 087611001 — DOC No: 012 — DOC TYPE: R — FIGURE No: F0036 — REVISION: 2 — FIGURE 30

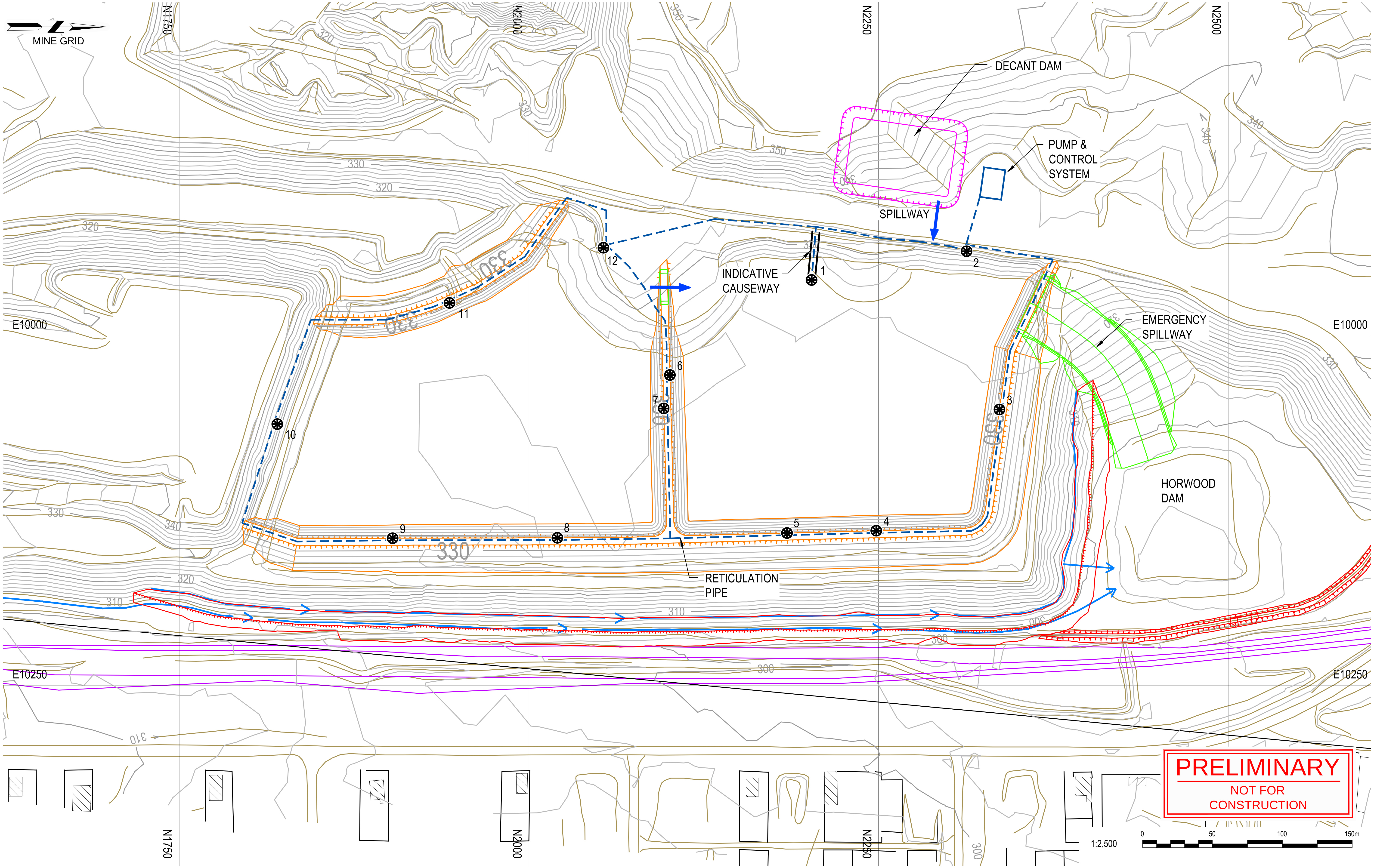
Plot Date: 15 December 2009 Time:11:54:07 AM By: Hrustanpasic, Dado Path: J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES - File Name:087611001_012_R_F0036_Rev2.dwg
Xref: ############



PRELIMINARY

Golder Associates	CLIENT: BROKEN HILL OPERATIONS		PROJECT: RASP MINE	
	DRAWN BY: DH	DATE: 3.12.09	DRAWING TITLE: TAILINGS STORAGE FEASIBILITY DESIGN SEEPAGE ANALYSIS OUTPUT	
	CHECKED BY: DAA	DATE: 14.12.09		
www.golder.com GOLDER ASSOCIATES PTY LTD	SCALE: NOT TO SCALE	SHEET SIZE: A4	PROJECT No: 087611001; DOC No: 012; DOC TYPE: R; FIGURE No: F0038; REVISION: 2	FIGURE 31

Plot Date: 15 December 2009 Time:11:56:42 AM By: Hrustanpasic, Dado Path: J:\Mining\2008\087611001\Technical Doc\CADD\FIGURES - File Name:087611001_012_R_F0038_Rev2.dwg
Xref: ############



DRAWING TAKEN FROM bhill_mine.dxf DATED JANUARY 2008 (AERIAL SURVEY OF JANUARY 2000). SURVEY RE-CONTOURED IN TERRAMODEL (087611001_TM_004.PRO)

CLIENT	BROKEN HILL OPERATIONS
PROJECT	RASP MINE
DRAWN	DH/MJM
DATE	11.12.09
CHECKED	DAA
DATE	14.12.09
TITLE	TAILINGS STORAGE FEASIBILITY DESIGN TSF-1 DUST MANAGEMENT SYSTEM
SCALE	1:2,500 A3
PROJECT No	087611001
DOC No	012
DOC TYPE	R
FIGURE No	F0035
REV No	2
	FIGURE 32

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APPENDIX A

Forecast Monthly Production Data

Date	Mill Feed	Cumulative Feed	Tailings	Cumulative Tailings	Hydraulic Fill	Surface Tailings	Surface Tailings	Cumulative Surface Tailings	Volume @ 1.3 t/m3	Cumulative Volume
	(tonnes)	(tonnes)	(tonnes)	(tonnes)	(tonnes)	%	(tonnes)	(tonnes)	(m3)	(m3)
1-Jul-10	30,000	30,000	24,750	24,750	-	100	24,750	24,750	19,038	19,038
1-Aug-10	35,000	65,000	28,875	53,625	-	100	28,875	53,625	22,212	41,250
1-Sep-10	35,000	100,000	28,875	82,500	-	100	28,875	82,500	22,212	63,462
1-Oct-10	35,000	135,000	28,875	111,375	-	100	28,875	111,375	22,212	85,673
1-Nov-10	35,000	170,000	28,875	140,250	7,219	75	21,656	133,031	16,659	102,332
1-Dec-10	40,000	210,000	33,000	173,250	8,250	75	24,750	157,781	19,038	121,370
1-Jan-11	40,000	250,000	33,000	206,250	8,250	75	24,750	182,531	19,038	140,409
1-Feb-11	40,000	290,000	33,000	239,250	8,250	75	24,750	207,281	19,038	159,447
1-Mar-11	40,000	330,000	33,000	272,250	16,500	50	16,500	223,781	12,692	172,139
1-Apr-11	40,000	370,000	33,000	305,250	16,500	50	16,500	240,281	12,692	184,832
1-May-11	40,000	410,000	33,000	338,250	16,500	50	16,500	256,781	12,692	197,524
1-Jun-11	40,000	450,000	33,000	371,250	16,500	50	16,500	273,281	12,692	210,216
1-Jul-11	40,000	490,000	33,000	404,250	16,500	50	16,500	289,781	12,692	222,909
1-Aug-11	40,000	530,000	33,000	437,250	16,500	50	16,500	306,281	12,692	235,601
1-Sep-11	40,000	570,000	33,000	470,250	16,500	50	16,500	322,781	12,692	248,293
1-Oct-11	40,000	610,000	33,000	503,250	16,500	50	16,500	339,281	12,692	260,986
1-Nov-11	40,000	650,000	33,000	536,250	16,500	50	16,500	355,781	12,692	273,678
1-Dec-11	40,000	690,000	33,000	569,250	16,500	50	16,500	372,281	12,692	286,370
1-Jan-12	40,000	730,000	33,000	602,250	16,500	50	16,500	388,781	12,692	299,063
1-Feb-12	35,000	765,000	28,875	631,125	14,438	50	14,438	403,219	11,106	310,168
1-Mar-12	40,000	805,000	33,000	664,125	16,500	50	16,500	419,719	12,692	322,861
1-Apr-12	40,000	845,000	33,000	697,125	16,500	50	16,500	436,219	12,692	335,553
1-May-12	40,000	885,000	33,000	730,125	16,500	50	16,500	452,719	12,692	348,245
1-Jun-12	40,000	925,000	33,000	763,125	16,500	50	16,500	469,219	12,692	360,938
1-Jul-12	40,000	965,000	33,000	796,125	16,500	50	16,500	485,719	12,692	373,630
1-Aug-12	40,000	1,005,000	33,000	829,125	16,500	50	16,500	502,219	12,692	386,322
1-Sep-12	40,000	1,045,000	33,000	862,125	16,500	50	16,500	518,719	12,692	399,014
1-Oct-12	40,000	1,085,000	33,000	895,125	16,500	50	16,500	535,219	12,692	411,707
1-Nov-12	40,000	1,125,000	33,000	928,125	16,500	50	16,500	551,719	12,692	424,399
1-Dec-12	40,000	1,165,000	33,000	961,125	16,500	50	16,500	568,219	12,692	437,091
1-Jan-13	40,000	1,205,000	33,000	994,125	16,500	50	16,500	584,719	12,692	449,784
1-Feb-13	35,000	1,240,000	28,875	1,023,000	14,438	50	14,438	599,156	11,106	460,889
1-Mar-13	40,000	1,280,000	33,000	1,056,000	16,500	50	16,500	615,656	12,692	473,582
1-Apr-13	40,000	1,320,000	33,000	1,089,000	16,500	50	16,500	632,156	12,692	486,274
1-May-13	40,000	1,360,000	33,000	1,122,000	16,500	50	16,500	648,656	12,692	498,966
1-Jun-13	40,000	1,400,000	33,000	1,155,000	16,500	50	16,500	665,156	12,692	511,659
1-Jul-13	44,000	1,444,000	36,300	1,191,300	18,150	50	18,150	683,306	13,962	525,620
1-Aug-13	44,000	1,488,000	36,300	1,227,600	18,150	50	18,150	701,456	13,962	539,582
1-Sep-13	44,000	1,532,000	36,300	1,263,900	18,150	50	18,150	719,606	13,962	553,543
1-Oct-13	44,000	1,576,000	36,300	1,300,200	18,150	50	18,150	737,756	13,962	567,505
1-Nov-13	44,000	1,620,000	36,300	1,336,500	18,150	50	18,150	755,906	13,962	581,466
1-Dec-13	44,000	1,664,000	36,300	1,372,800	18,150	50	18,150	774,056	13,962	595,428
1-Jan-14	44,000	1,708,000	36,300	1,409,100	18,150	50	18,150	792,206	13,962	609,389
1-Feb-14	41,000	1,749,000	33,825	1,442,925	16,913	50	16,913	809,119	13,010	622,399
1-Mar-14	44,000	1,793,000	36,300	1,479,225	18,150	50	18,150	827,269	13,962	636,361
1-Apr-14	44,000	1,837,000	36,300	1,515,525	18,150	50	18,150	845,419	13,962	650,322
1-May-14	44,000	1,881,000	36,300	1,551,825	18,150	50	18,150	863,569	13,962	664,284
1-Jun-14	44,000	1,925,000	36,300	1,588,125	18,150	50	18,150	881,719	13,962	678,245
1-Jul-14	50,000	1,975,000	41,250	1,629,375	20,625	50	20,625	902,344	15,865	694,111
1-Aug-14	50,000	2,025,000	41,250	1,670,625	20,625	50	20,625	922,969	15,865	709,976
1-Sep-14	50,000	2,075,000	41,250	1,711,875	20,625	50	20,625	943,594	15,865	725,841
1-Oct-14	50,000	2,125,000	41,250	1,753,125	20,625	50	20,625	964,219	15,865	741,707
1-Nov-14	50,000	2,175,000	41,250	1,794,375	20,625	50	20,625	984,844	15,865	757,572
1-Dec-14	50,000	2,225,000	41,250	1,835,625	20,625	50	20,625	1,005,469	15,865	773,438
1-Jan-15	50,000	2,275,000	41,250	1,876,875	20,625	50	20,625	1,026,094	15,865	789,303
1-Feb-15	50,000	2,325,000	41,250	1,918,125	20,625	50	20,625	1,046,719	15,865	805,168
1-Mar-15	50,000	2,375,000	41,250	1,959,375	20,625	50	20,625	1,067,344	15,865	821,034
1-Apr-15	50,000	2,425,000	41,250	2,000,625	20,625	50	20,625	1,087,969	15,865	836,899
1-May-15	50,000	2,475,000	41,250	2,041,875	20,625	50	20,625	1,108,594	15,865	852,764
1-Jun-15	50,000	2,525,000	41,250	2,083,125	20,625	50	20,625	1,129,219	15,865	868,630
1-Jul-15	56,500	2,581,500	46,613	2,129,738	23,306	50	23,306	1,152,525	17,928	886,558
1-Aug-15	56,500	2,638,000	46,613	2,176,350	23,306	50	23,306	1,175,831	17,928	904,486
1-Sep-15	56,500	2,694,500	46,613	2,222,963	23,306	50	23,306	1,199,138	17,928	922,413
1-Oct-15	56,500	2,751,000	46,613	2,269,575	23,306	50	23,306	1,222,444	17,928	940,341
1-Nov-15	56,500	2,807,500	46,613	2,316,188	23,306	50	23,306	1,245,750	17,928	958,269
1-Dec-15	56,500	2,864,000	46,613	2,362,800	23,306	50	23,306	1,269,056	17,928	976,197
1-Jan-16	56,500	2,920,500	46,613	2,409,413	23,306	50	23,306	1,292,363	17,928	994,125
1-Feb-16	53,500	2,974,000	44,138	2,453,550	22,069	50	22,069	1,314,431	16,976	1,011,101
1-Mar-16	56,500	3,030,500	46,613	2,500,163	23,306	50	23,306	1,337,738	17,928	1,029,029
1-Apr-16	56,500	3,087,000	46,613	2,546,775	23,306	50	23,306	1,361,044	17,928	1,046,957

1-May-16	56,500	3,143,500	46,613	2,593,388	23,306	50	23,306	1,384,350	17,928	1,064,885
1-Jun-16	56,500	3,200,000	46,613	2,640,000	23,306	50	23,306	1,407,656	17,928	1,082,813
1-Jul-16	62,500	3,262,500	51,563	2,691,563	25,781	50	25,781	1,433,438	19,832	1,102,644
1-Aug-16	62,500	3,325,000	51,563	2,743,125	25,781	50	25,781	1,459,219	19,832	1,122,476
1-Sep-16	62,500	3,387,500	51,563	2,794,688	25,781	50	25,781	1,485,000	19,832	1,142,308
1-Oct-16	62,500	3,450,000	51,563	2,846,250	25,781	50	25,781	1,510,781	19,832	1,162,139
1-Nov-16	62,500	3,512,500	51,563	2,897,813	25,781	50	25,781	1,536,563	19,832	1,181,971
1-Dec-16	62,500	3,575,000	51,563	2,949,375	25,781	50	25,781	1,562,344	19,832	1,201,803
1-Jan-17	62,500	3,637,500	51,563	3,000,938	25,781	50	25,781	1,588,125	19,832	1,221,635
1-Feb-17	62,500	3,700,000	51,563	3,052,500	25,781	50	25,781	1,613,906	19,832	1,241,466
1-Mar-17	62,500	3,762,500	51,563	3,104,063	25,781	50	25,781	1,639,688	19,832	1,261,298
1-Apr-17	62,500	3,825,000	51,563	3,155,625	25,781	50	25,781	1,665,469	19,832	1,281,130
1-May-17	62,500	3,887,500	51,563	3,207,188	25,781	50	25,781	1,691,250	19,832	1,300,962
1-Jun-17	62,500	3,950,000	51,563	3,258,750	25,781	50	25,781	1,717,031	19,832	1,320,793
1-Jul-17	62,500	4,012,500	51,563	3,310,313	25,781	50	25,781	1,742,813	19,832	1,340,625
1-Aug-17	62,500	4,075,000	51,563	3,361,875	25,781	50	25,781	1,768,594	19,832	1,360,457
1-Sep-17	62,500	4,137,500	51,563	3,413,438	25,781	50	25,781	1,794,375	19,832	1,380,288
1-Oct-17	62,500	4,200,000	51,563	3,465,000	25,781	50	25,781	1,820,156	19,832	1,400,120
1-Nov-17	62,500	4,262,500	51,563	3,516,563	25,781	50	25,781	1,845,938	19,832	1,419,952
1-Dec-17	62,500	4,325,000	51,563	3,568,125	25,781	50	25,781	1,871,719	19,832	1,439,784
1-Jan-18	62,500	4,387,500	51,563	3,619,688	25,781	50	25,781	1,897,500	19,832	1,459,615
1-Feb-18	62,500	4,450,000	51,563	3,671,250	25,781	50	25,781	1,923,281	19,832	1,479,447
1-Mar-18	62,500	4,512,500	51,563	3,722,813	25,781	50	25,781	1,949,063	19,832	1,499,279
1-Apr-18	62,500	4,575,000	51,563	3,774,375	25,781	50	25,781	1,974,844	19,832	1,519,111
1-May-18	62,500	4,637,500	51,563	3,825,938	25,781	50	25,781	2,000,625	19,832	1,538,942
1-Jun-18	62,500	4,700,000	51,563	3,877,500	25,781	50	25,781	2,026,406	19,832	1,558,774
1-Jul-18	62,500	4,762,500	51,563	3,929,063	25,781	50	25,781	2,052,188	19,832	1,578,606
1-Aug-18	62,500	4,825,000	51,563	3,980,625	25,781	50	25,781	2,077,969	19,832	1,598,438
1-Sep-18	62,500	4,887,500	51,563	4,032,188	25,781	50	25,781	2,103,750	19,832	1,618,269
1-Oct-18	62,500	4,950,000	51,563	4,083,750	25,781	50	25,781	2,129,531	19,832	1,638,101
1-Nov-18	62,500	5,012,500	51,563	4,135,313	25,781	50	25,781	2,155,313	19,832	1,657,933
1-Dec-18	62,500	5,075,000	51,563	4,186,875	25,781	50	25,781	2,181,094	19,832	1,677,764
1-Jan-19	62,500	5,137,500	51,563	4,238,438	25,781	50	25,781	2,206,875	19,832	1,697,596
1-Feb-19	62,500	5,200,000	51,563	4,290,000	25,781	50	25,781	2,232,656	19,832	1,717,428
1-Mar-19	62,500	5,262,500	51,563	4,341,563	25,781	50	25,781	2,258,438	19,832	1,737,260
1-Apr-19	62,500	5,325,000	51,563	4,393,125	25,781	50	25,781	2,284,219	19,832	1,757,091
1-May-19	62,500	5,387,500	51,563	4,444,688	25,781	50	25,781	2,310,000	19,832	1,776,923
1-Jun-19	62,500	5,450,000	51,563	4,496,250	25,781	50	25,781	2,335,781	19,832	1,796,755
1-Jul-19	62,500	5,512,500	51,563	4,547,813	25,781	50	25,781	2,361,563	19,832	1,816,587
1-Aug-19	62,500	5,575,000	51,563	4,599,375	25,781	50	25,781	2,387,344	19,832	1,836,418
1-Sep-19	62,500	5,637,500	51,563	4,650,938	25,781	50	25,781	2,413,125	19,832	1,856,250
1-Oct-19	62,500	5,700,000	51,563	4,702,500	25,781	50	25,781	2,438,906	19,832	1,876,082
1-Nov-19	62,500	5,762,500	51,563	4,754,063	25,781	50	25,781	2,464,688	19,832	1,895,913
1-Dec-19	62,500	5,825,000	51,563	4,805,625	25,781	50	25,781	2,490,469	19,832	1,915,745
1-Jan-20	62,500	5,887,500	51,563	4,857,188	25,781	50	25,781	2,516,250	19,832	1,935,577
1-Feb-20	62,500	5,950,000	51,563	4,908,750	25,781	50	25,781	2,542,031	19,832	1,955,409
1-Mar-20	62,500	6,012,500	51,563	4,960,313	25,781	50	25,781	2,567,813	19,832	1,975,240
1-Apr-20	62,500	6,075,000	51,563	5,011,875	25,781	50	25,781	2,593,594	19,832	1,995,072
1-May-20	62,500	6,137,500	51,563	5,063,438	25,781	50	25,781	2,619,375	19,832	2,014,904
1-Jun-20	62,500	6,200,000	51,563	5,115,000	25,781	50	25,781	2,645,156	19,832	2,034,736
Totals	**6,200,000**		**5,115,000**		**2,469,844**		**2,645,156**		**2,034,736**	

APPENDIX B

Laboratory Evaluation of Rasp Mine Tailings for Use as Hydraulic Backfill (Golder Paste Technology Ltd.)

Golder Paste Technology Ltd.

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REPORT ON

LABORATORY EVALUATION OF RASP MINE TAILINGS FOR USE AS HYDRAULIC BACKFILL

Submitted to:

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February 14, 2008 07-1900-0029

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1.0 EXECUTIVE SUMMARY

The RASP sample ‘as received’ contained close to the target fines content for hydraulic slurry, about 8% passing 20 µm. As a result percolation tests were carried out on the sample before modifying the particle size distribution.

Although the particle size distribution of the sample ‘as received’ satisfied the general ‘rule of thumb’ 8% passing 20 µm, the RASP sample turned out to have a low permeability of 0.7 cm/hour. A sample of hydraulic slurry (control sample) from a local backfill operation was obtained and tested to determine a reasonable percolation target, which was 6.5 cm/hour using standard constant head permeability equipment. Acceptable percolation rates vary anywhere from 4 - 6 cm/hour for classified tailings and up to 10 cm/hour for very coarse fill such as alluvial sands.

Simulated cyclone tests were then carried out on the RASP sample to further reduce the fines concentration to satisfy the percolation requirements. The final percolation rate was 4.5 cm/hour after three cyclone tests. The fines content was reduced to just over 4% passing 20 µm by ‘desliming’ the tailings. The actual laboratory split was about 10 wt% (dry) solids reporting to the Cyclone O/F, and 90 wt% (dry) solids reporting to the Cyclone U/F.

A data sheet containing the operating parameters and target PSD was sent to Krebs to verify cyclone performance. The split was confirmed using their modelling software to be around 14 wt% (dry) solids reporting to the overflow and 84 wt% (dry) solids reporting to the underflow. The resulting PSD’s are presented in the body of the report.

Testing was carried out on both streams, and a blended stream. The Cyclone U/F tailings were tested for material characterization, percolation rate, total sulphur content and strength properties. The Cyclone O/F tailings were tested for material characterization, dewatering, and rheological properties. An additional sample, beyond the original scope of work, labeled ‘Blend’ was prepared to represent the average composition of the material in the tailings tanks which then reports to surface. This blend was reported to be 4 parts cyclone underflow mixed with 1 part cyclone overflow (80/20). The testing requested for the blended product was particle size distribution, settling characteristics and percent sulphur. Rheological testing of the blend was not requested.

The sulphur content was lower than the expected levels communicated throughout the project, at <50 to 79 ppm.

Overall, the tests indicated that it is possible to produce a good strength hydraulic slurry backfill. The Cyclone O/F and Blend dewatered favourably with clear overflow, using low polymer additions. Rheological testing of the Cyclone O/F showed positive handling and transport properties.

2.0 INTRODUCTION

CBH Resources Limited has retained Golder Associates Pty Ltd. and Golder Paste Technology Ltd. (PasteTec) to carry out laboratory testing on RASP Mine tailings for the purpose of determining its suitability for use as a cemented hydraulic fill (CHF). This testing was carried out to substantiate previous assumptions presented in the preliminary pre-feasibility report, dated April 2007.

The classification split for hydraulic fill is based on the particle size distribution (PSD) and percolation rate of the tailings. Typically, the material should have a fines content of around 8% passing 20 µm and a percolation rate of between 4 - 6 cm/hour.

Once the split ratio was confirmed by the cyclone vendor (see Appendix A), the material was prepared batch-wise until the final product was achieved. The fines and coarse portion were then subjected to a suite of tests including material characterization, rheological index testing, dewatering performance, percolation rate and strength gain characteristics. The purpose of testing the fines portion was to determine its dewatering and transport characteristics for surface disposal.

Total sulphur content was also determined for the fine and coarse portions as well as a blended product representative of the material in the tailings storage tanks. The sulphur content was lower than the expected levels communicated throughout the project; at <50 to 79 ppm.

3.0 TAILINGS MATERIAL PROPERTY CHARACTERIZATION

RASP Classified Mine samples were received by PasteTec's Sudbury laboratory September 13, 2007. Three 20 litre pails were received in good condition with all seals intact. The samples were shipped from G&T Metallurgical Services in Kamloops, British Columbia where other metallurgical tests were performed. A total of 45 kg of oven dried tailings were received in three separate pails. Upon confirmation, the three samples were blended together and labelled 'Final Composite'. The pH of the Final Composite was determined to be 9.3 by mixing a small amount of the dry solids in distilled water.

All samples received by PasteTec are subjected to material property characterization tests to establish properties and allow comparison should future testing be required. The following is a brief description and relevant procedures of each test.

3.1 Particle Size Distribution

Particle size distribution (PSD) is determined using mechanical sieving and a Fritsch laser particle size analyzer according to ASTM D4464. The laser operating principle is based on light diffraction caused by particles suspended in a medium (usually water) which pass through the laser beam. The resulting diffraction, which is measured by a detector grid opposite the laser, is transformed by software using the appropriate algorithms into a particle size distribution. As in other techniques, the PSD curve is based on 'equivalent spherical diameters' so particle shape can have an influence on the results.

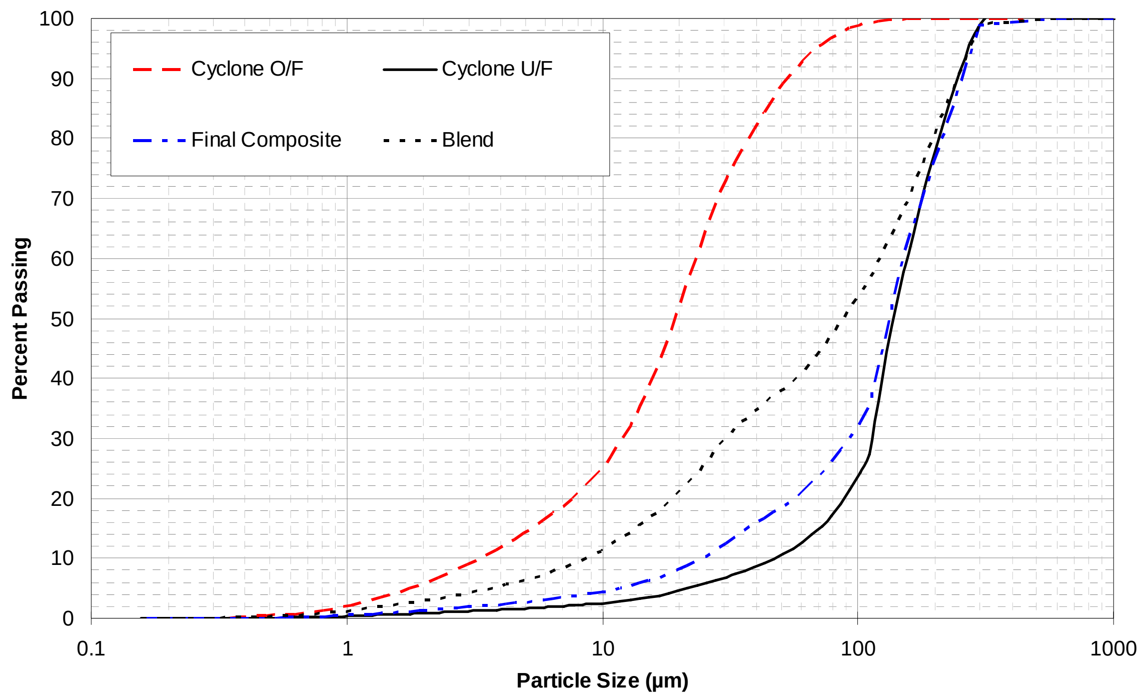
Specific D-values are presented below in Table 1.

TABLE 1
PARTICLE SIZE DISTRIBUTION

Sample	D10	D30	D50	D60	D85
Final Composite	25	93	134	148	245
Cyclone O/F	3	12	19	23	44
Cyclone U/F	47	114	140	158	249
Blend	9	30	88	127	232

The results are presented on Figure 1.

FIGURE 1
PARTICLE SIZE DISTRIBUTION



The PSD for the hydraulic slurry was reduced from 8% passing 20µm to just over 4% passing 20 µm, increasing the percolation rate from 0.7 cm/hour to 4.5 cm/hour. This final PSD (Cyclone U/F) satisfied the percolation requirements of hydraulic fill.

3.2 Specific Gravity

The specific gravity of each sample was measured according to ASTM D854 and the results are presented below:

TABLE 2
SPECIFIC GRAVITY RESULTS

SAMPLE	SPECIFIC GRAVITY		
	Trial 1	Trial 2	Average
Final Composite	3.00	3.03	3.01
Cyclone O/F	2.88	2.89	2.88
Cyclone U/F	3.04	3.06	3.05

3.3 Metals Analysis

Metals analysis is performed to identify health and safety hazards which might be present in the sample. The method used for metals analysis is inductively coupled plasma with a mass spectrometer detector (ICP-MS). The Final Composite sample was sent for analysis and the results are presented in Appendix B.

The concentrations of heavy metals were found to be within typical ranges of material handled in the PasteTec laboratory. No additional safety precautions over and above the standard Health and Safety and material handling practices were introduced.

3.4 Mineralogy and Chemistry

Chemical and mineralogical analyses are performed using XRF and XRD techniques, respectively. Computer analysis of the XRD patterns provides a more detailed semi-quantitative mineralogical composition. The results are presented in Tables 3 and 4. Each minerals composition is presented in Table 5 for reference.

TABLE 3
CHEMICAL COMPOSITION

Sample	SiO_2	Al_2O_3	Fe_2O_3	MgO	CaO	Na_2O	K_2O	TiO_2	P_2O_5	MnO	Cr_2O_3	V_2O_5	LOI	Sum
	%	%	%	%	%	%	%	%	%	%	%	%	%	%
Final composite	79.6	6.75	7.55	1.17	0.71	0.43	0.89	0.19	0.23	0.79	0.03	0.01	0.99	99.3

TABLE 4
SEMI QUANTITATIVE MINERALOGICAL COMPOSITION

Mineral	% Composition
SQ-XRD	Final Composite
Quartz	69.3
Almandine	6.6
Biotite	14.9
Chloritoid	4.7
Chamosite	4.5
Total	100

TABLE 5
MINERAL COMPOSITION

Mineral	Composition
Chlorite	$(Fe,(Mg,Mn)_5,Al)(Si_3Al)O_{10}(OH)_8$
Chloritoid	$(Fe,Mg)Al_4O_2(SiO_4)_2(OH)_4$
Garnet	$(Ca,Mg,Mn^{2+})_3(V,Al, Fe^{3+})_2(SiO_4)_3$
Mica	$K(Mg,Fe)Al_2Si_3AlO_{10}(OH)_2$
Quartz	SiO_2

3.5 Sulphur Analysis

All samples were submitted for sulphur analysis by the optical emission spectroscopy (OES) technique. The minimum detection limit for testing is 50 ppm. The results are presented below in Table 6. Detailed results are presented in Appendix B.

TABLE 6
SULPHUR CONTENT

Sample	Sulphur Content
Final Composite	79 ppm
Cyclone O/F	<50 ppm
Cyclone U/F	76 ppm
Blend	71 ppm

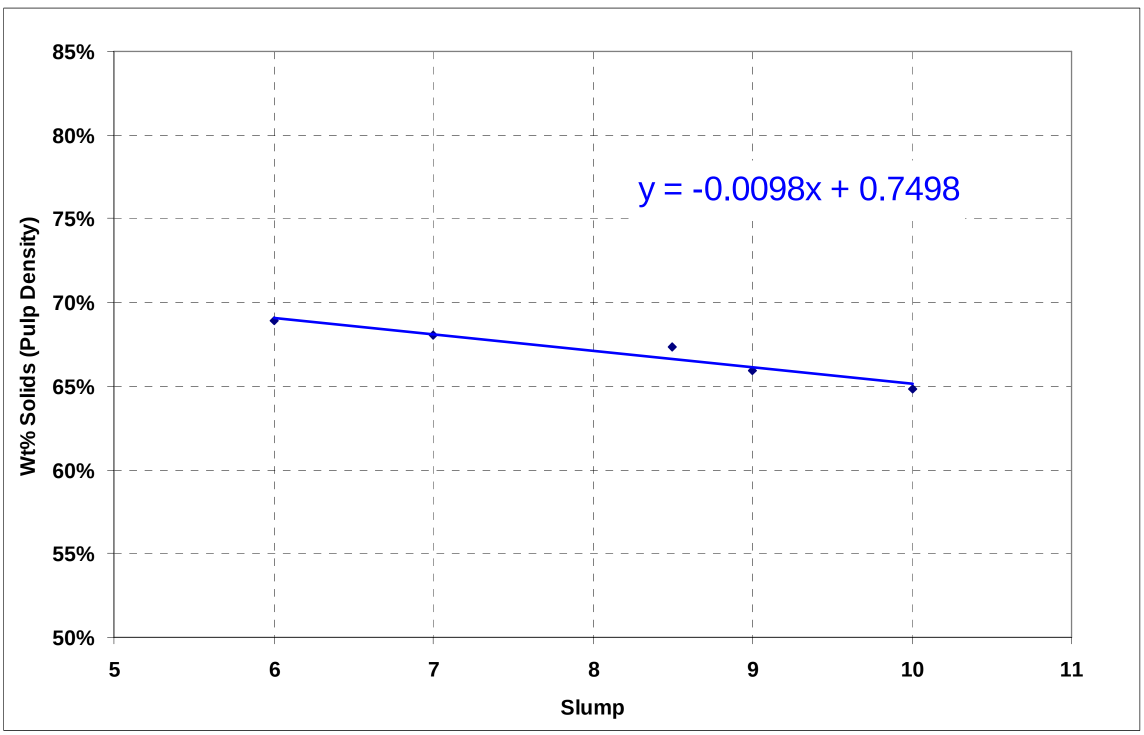
4.0 RHEOLOGICAL CHARACTERIZATION

Rheological testing is carried out to evaluate flow and handling properties. These indicator tests provide a feel for how the material will behave during mixing, slump adjustment, pumping, flowing and while sitting idle.

4.1 Slump vs. Solids Content

To gauge sensitivity to water additions, small increments of water are added to the bulk sample. After each addition, slump and solids content is determined. This generates a relationship between slump and solids content which is used to determine the degree of process control required to maintain a consistent final product. The results are presented below on Figure 2.

FIGURE 2
SLUMP VS. SOLIDS CONTENT - CYCLONE O/F



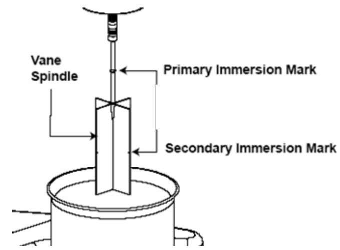
The results show that approximately 1% change in solids content will result in a 1” (25mm) change in slump. This means that the material is only mildly sensitive to water addition and should only require moderate levels of control to maintain consistency.

4.2 Static Yield Stress Testing

Yield stress is defined as the minimum force required to initiate flow. It is also indicative of material consistency once the relationship is established. There are different test methods to determine yield stress, one termed 'static' and the other 'dynamic'. This section of the report discusses static testing.

Static yield stress is determined using the yield vane technique. This method uses a sensitive rheometer to slowly turn a vane immersed in the material, while constantly monitoring the torque on the spindle. Once the maximum torque is registered, the peak or 'yield' stress is calculated and reported in Pascals.

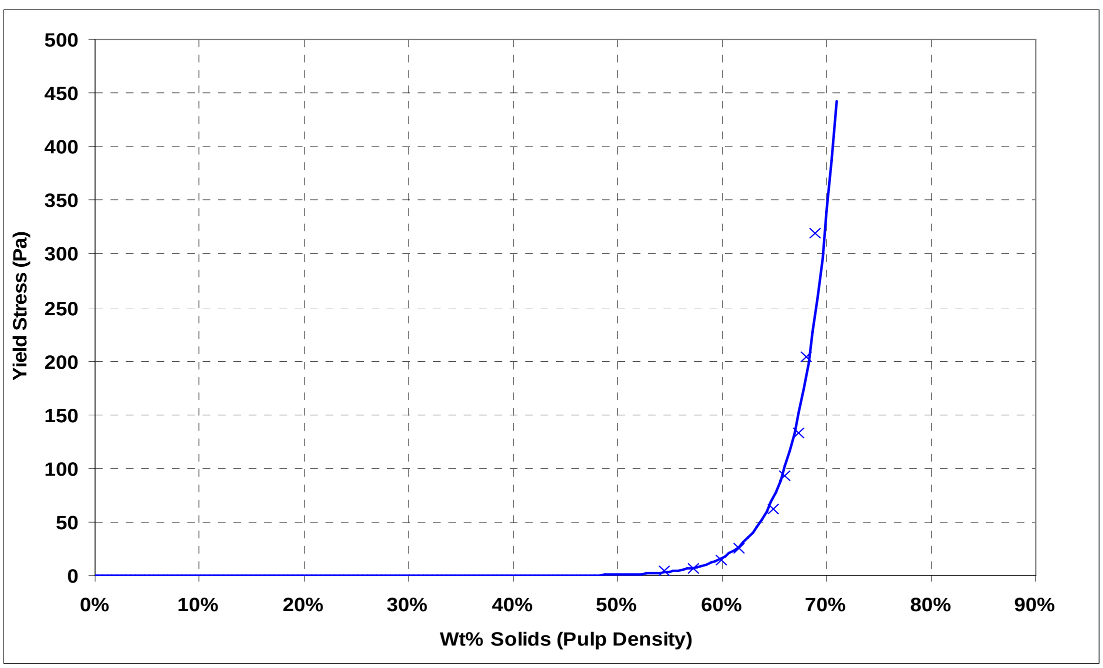
Instrument – Brookfield DVII+ Viscometer
Spring Torque – 5xHB, 0.0287 N·m
Speed – 0.2 RPM (constant shear rate)
Spindle – V71, V72, V73 Yield Vanes



Yield Vane Diagram

The same method is carried out on several pulp densities to generate a relationship between yield stress and solids content. The relationship between solids content and yield stress is usually exponential for stable mineral pastes. The curve is presented below on Figure 3.

FIGURE 3
YIELD STRESS VS. SOLIDS CONTENT - CYCLONE O/F

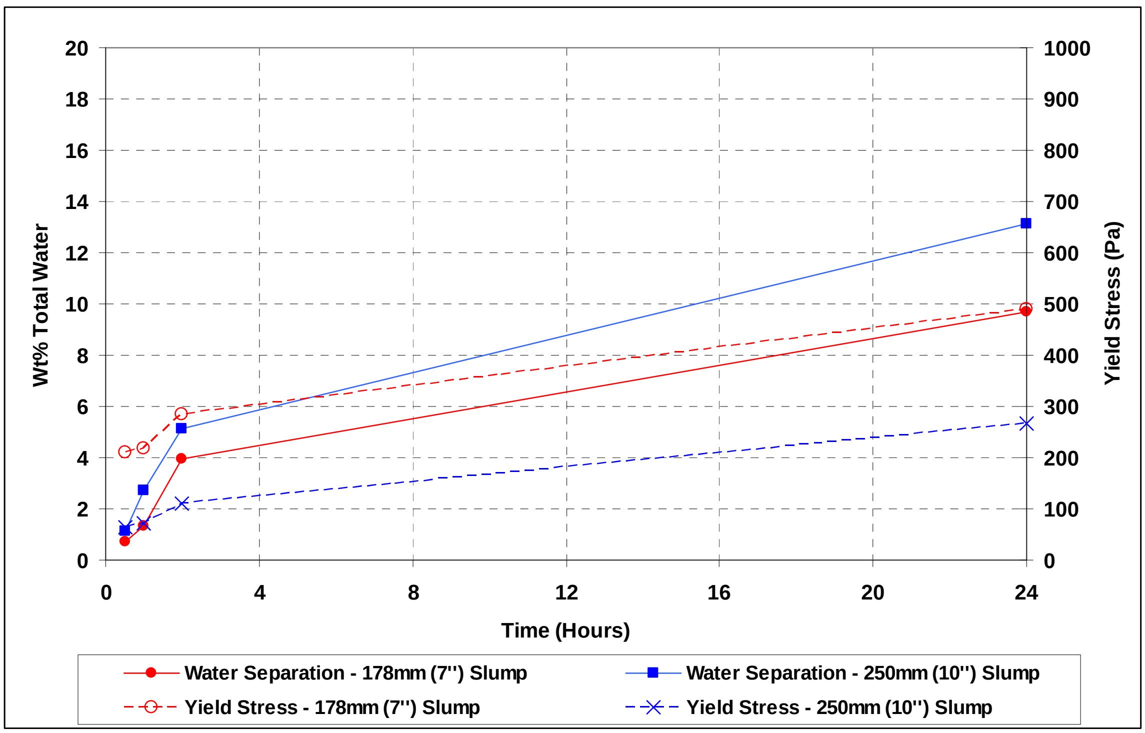


4.3 Water Bleed and Yield Stress vs. Time

Moisture retention testing is carried out to assess the water bleed properties of the paste while sitting idle in test beakers. Two slump consistencies are tested at four time intervals. At each time interval the water bleed, material segregation and yield stress is measured.

Water bleed/yield stress testing was only performed on the blend sample. No substantial water bleed or yield stress development was observed over the 24 hour period. The results are presented below on Figure 4.

FIGURE 4
WATER BLEED AND YIELD STRESS VS. TIME - CYCLONE O/F

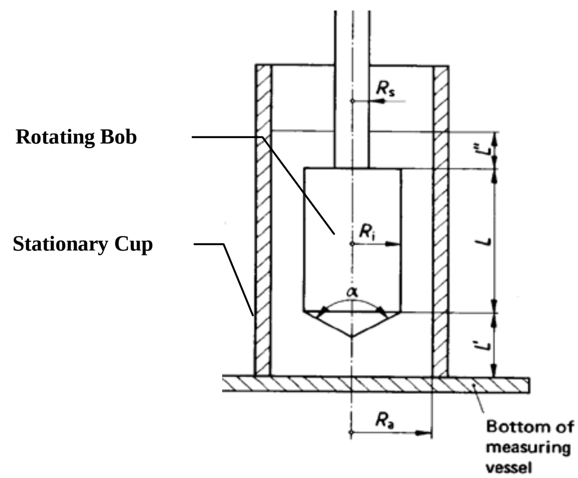


4.4 Viscosity Determination

Viscosity testing provides bench scale flow properties and fluid characterization, as well as essential data for mixer, pump and pipeline design. In order to compare or duplicate viscosity results of non-Newtonian fluids, it is important to test according to the same conditions. Proper test conditions are critical to producing usable data from bench scale viscometers. The following outlines the testing conditions found to be most applicable for material similar to the Cyclone O/F tailings;

Instrument – Brookfield R/S Rheometer
Spindle – CC25 Bob and Cup
R_i – 12.50 mm
R_a – 13.56 mm
R_s – 3.5 mm
α – 120º
L' – 15.5 mm
L'' – 12.5 mm
L – 37.5 mm
R_a / R_i – 1.0848
Surface – Sand Blasted
Shear Rate – 0 - 400 sec^{-1}

Bob and Cup Diagram



Each ramp cycle (increase then decrease in shear rate) is repeated at least three times, with fresh sample introduced each time. The resulting flow curves are analyzed automatically by software and the best fit model is applied to the raw data. It has been found through numerous laboratory and production scale flow loops that most mineral pastes are Bingham fluids, which indicates a linear relationship between shear stress and shear rate with a defined yield stress. The yield stress determined through this testing is referred to as dynamic yield stress since it is extrapolated from dynamic test data. The rheograms are presented in Appendix C. Summarized test results are presented below on Figure 5 and Figure 6.

FIGURE 5
BINGHAM YIELD STRESS – CYCLONE O/F

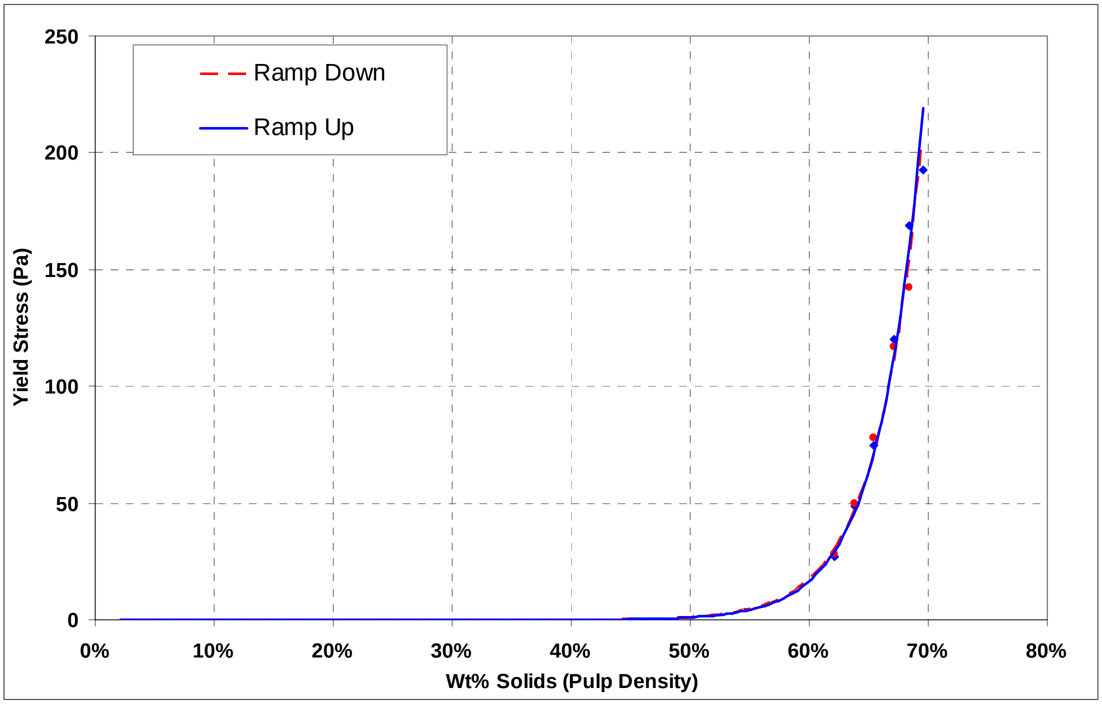
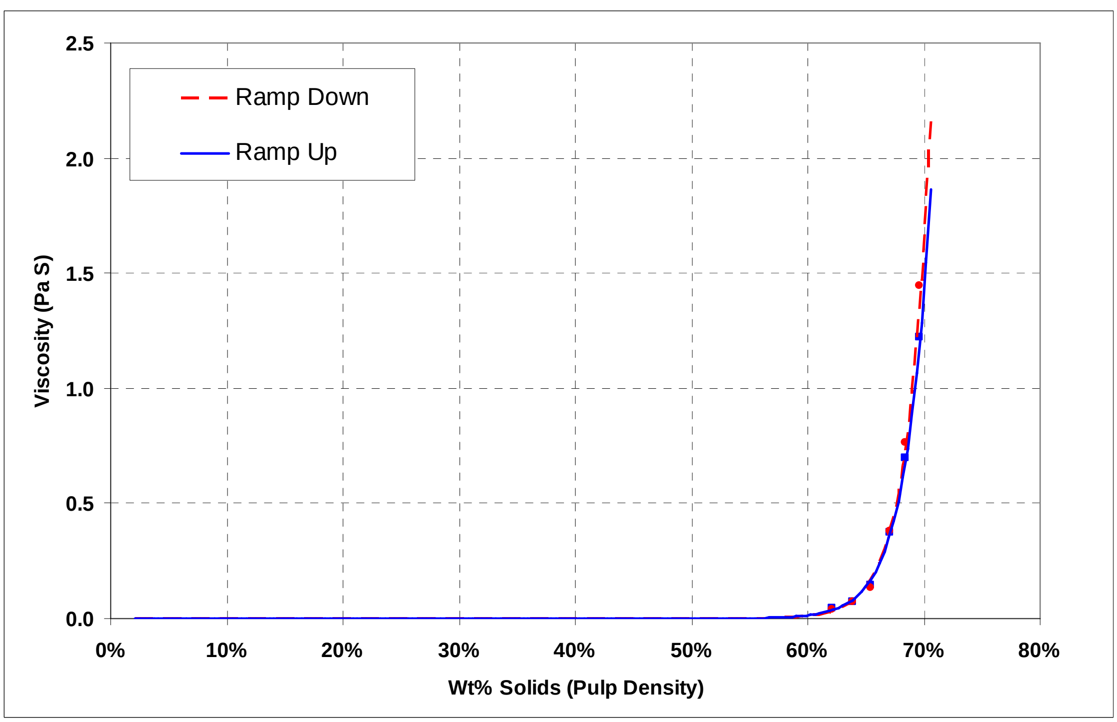


FIGURE 6
BINGHAM VISCOSITY – CYCLONE O/F



5.0 DEWATERING TESTING

5.1 Settling Tests

The first stage of settling tests is assessing the potential for thickening through use of synthetic polymers. Several flocculants are screened to select the most effective, considering several factors such as initial settling velocity, overflow clarity, flocculant structure and underflow density. These tests are performed on a bench scale in small flasks to conserve sample.

The typical ranges of flocculants considered are the anionic and non ionic polymers. Within each group, there are other variables such as molecular weight and charge density. Once the flocculant type is chosen, dosage and feed solids content must be optimized. The screening results are presented below in Table 7.

TABLE 7
FLOCCULANT SCREENING RESULTS – CYCLONE O/F

Flocculant	Overflow Clarity	Initial Settling Velocity	Bed Density After 2 min.	Floc Size / Structure
AN 926 VHM	Good	Fast	32 ml	Medium / Good
AN 945 SH	Cloudy	Slow	38 ml	Small / Good
AN 905 VHM	Cloudy	Slow	44 ml	Small / Weak
AN 920 VHM	Cloudy	Medium	34 ml	Small / Good

Three samples were tested for dewatering potential, the Final Composite, Cyclone O/F and the Blended tailings.

TABLE 8
FLOCCULATION PARAMETERS

Sample	Flocculant	Type	Anionicity (mole %)	Molecular Weight	Dosage (g/tonne)	Feed Solids Density
Final Composite	AN 926 VHM	Anionic	25	High	20	10
Cyclone O/F	AN 926 VHM	Anionic	25	High	30	5
Blend	AN 926 VHM	Anionic	25	High	20	10

Flocculation parameters are presented above in Table 8. The same flocculant worked effectively for all samples, however feed solids density and flocculant dosage was varied to promote optimum flocculation.

Upon completion of flocculant screening, tests were carried out in larger 4 liter vessels to increase the mud bed depth. Figure 7 presents the settling curve for each sample.

FIGURE 7
SETTLING CURVES – ALL SAMPLES

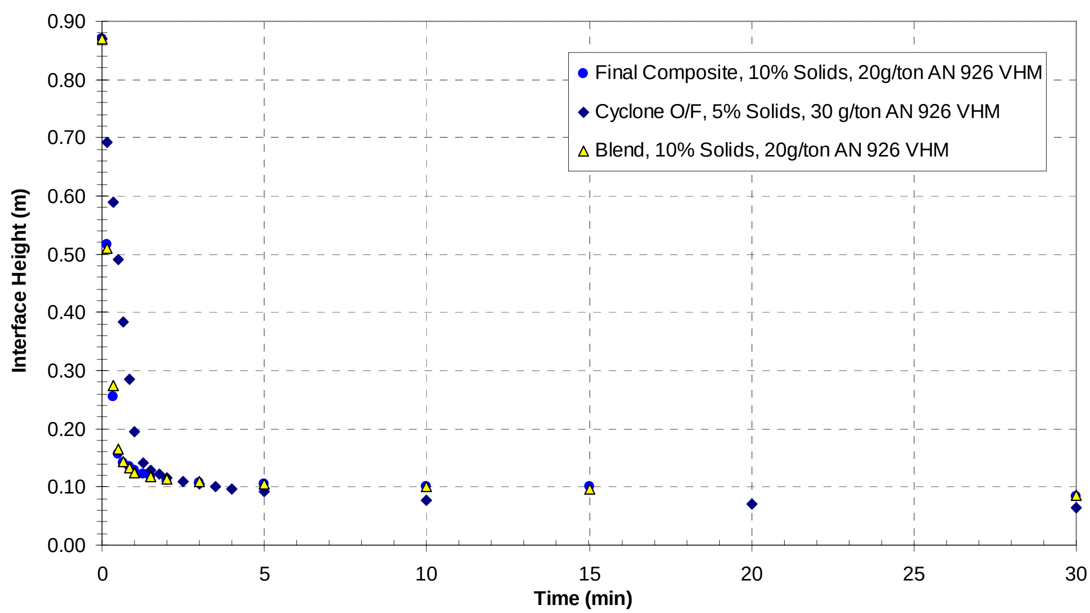


TABLE 9
SETTLING TEST SUMMARY

Sample	Initial Settling Velocity (m/hour)	Final Underflow Density (wt% solids)	Unit Area Sizing Basis (m^2/tonne/day)	Centrifuge Underflow Density (wt% solids)
Final Composite	86	69	0.03	75
Cyclone O/F	35	52	0.07	68
Blend	85	65	0.02	73

5.2 Filtration Testing

Tests were conducted to evaluate vacuum filtration as a possible dewatering treatment option for paste production. Concentrated Cyclone O/F slurry was used for vacuum filtration leaf tests to simulate disc filtration performance. The feed solids concentration is determined first by flocculation / settling techniques. The final density achieved after 24 hours of thickening, as well as the final density plus 5 wt% solids is used as a conservative filter feed density estimate. This provides a range of data under different testing conditions.

The filter leaf is equipped with a small section of industrial grade polypropylene felt filter cloth. The leaf test is dipped into the slurry and simulates production vacuum filters where the sectors dip into the slurry in an agitated filter tank as the disc rotates. Proper technique and cycle times simulating continuous filters provide an estimate of cake loading, moisture and discharge characteristics.

Tests are conducted at varying cycle times from 60 to 150 seconds. The filter cake is removed, thickness and cake weight is recorded and a sub-sample is analyzed for moisture content. After each test, the filter leaf is blown with compressed air to simulate the discharge cycle.

The following parameters were used for testing:

- Vacuum Level - 17’’ Hg
- Temperature - 20˚C
- Filter Cloth - Industrial Grade Polypropylene Felt 133-03
- Apparatus - 100mm diameter dip style filter head

Test results are presented below in Table 10 and Table 11.

TABLE 10
FILTRATION RESULTS – CYCLONE O/F 54WT% SOLIDS FEED

Cycle Time (sec)	Cake Thickness (mm)	Cake Loading ($kg/m^2/hour$)	Moisture (wt%)
60	5	553	26
75	7	542	26
90	9	471	26
120	11	377	26
150	13	330	26

TABLE 11
FILTRATION RESULTS – CYCLONE O/F 59WT% SOLIDS FEED

Cycle Time (sec)	Cake Thickness (mm)	Cake Loading ($kg/m^2/hour$)	Moisture (wt%)
60	6	797	26
75	9	637	27
90	12	623	26
120	14	509	26
150	17	449	26

The Cyclone O/F tailings filtered quite well given the relatively fine particle size distribution. As expected, the feed density affected the loading rate showing significant cake loading improvement at higher concentrations.

6.0 UNCONFINED COMPRESSIVE STRENGTH

Unconfined compressive strength (UCS) testing is carried out using a Humboldt HM3000 digital load frame. The load is measured using s-type load cells; depending on strength either a 2000lb or 10 000lb load cell is utilized.

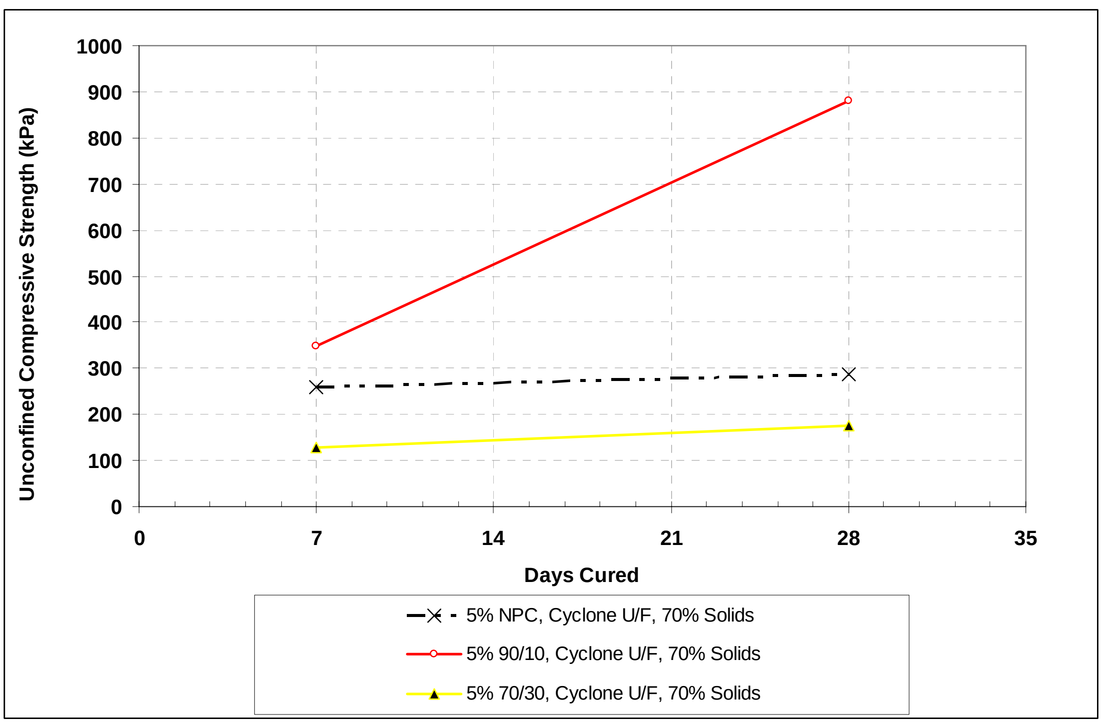
The specimen is placed between two platens and the bottom platen advances at a rate of 2 mm per minute. The load is continuously monitored and the peak load is automatically recorded by the instrument.

6.1 Binder Screening

Binder screening was carried out on the Cyclone U/F using 50 x 100 mm (2” x 4”) specimens to conserve sample for the final UCS program. Three binders were tested, straight Type 10 Normal Portland cement (NPC), a blend of 70% NPC and 30% Type CI Fly Ash, and a blend of 90% Blast Furnace Slag (BFS) and 10% NPC. For the initial screening tests, 5wt% binder is used.

The binder screening UCS results are presented below in Figure 8.

FIGURE 8
UCS SCREENING RESULTS



The results show that the blast furnace slag produced much higher strength, short and longer term. The blast furnace slag was carried forward to the next phase of strength testing.

6.2 Final UCS Program

The final UCS program was carried out to assess the backfill strength gain using larger 50 x 75mm (3 x 6”) cylinders and a range of binder contents. The mix designs consisted of 3 and 5% (90/10) binder. The solids content (70 wt%) was dictated by the cyclone underflow concentration.

The final UCS results are presented below in Figure 9.

FIGURE 9
FINAL UCS RESULTS

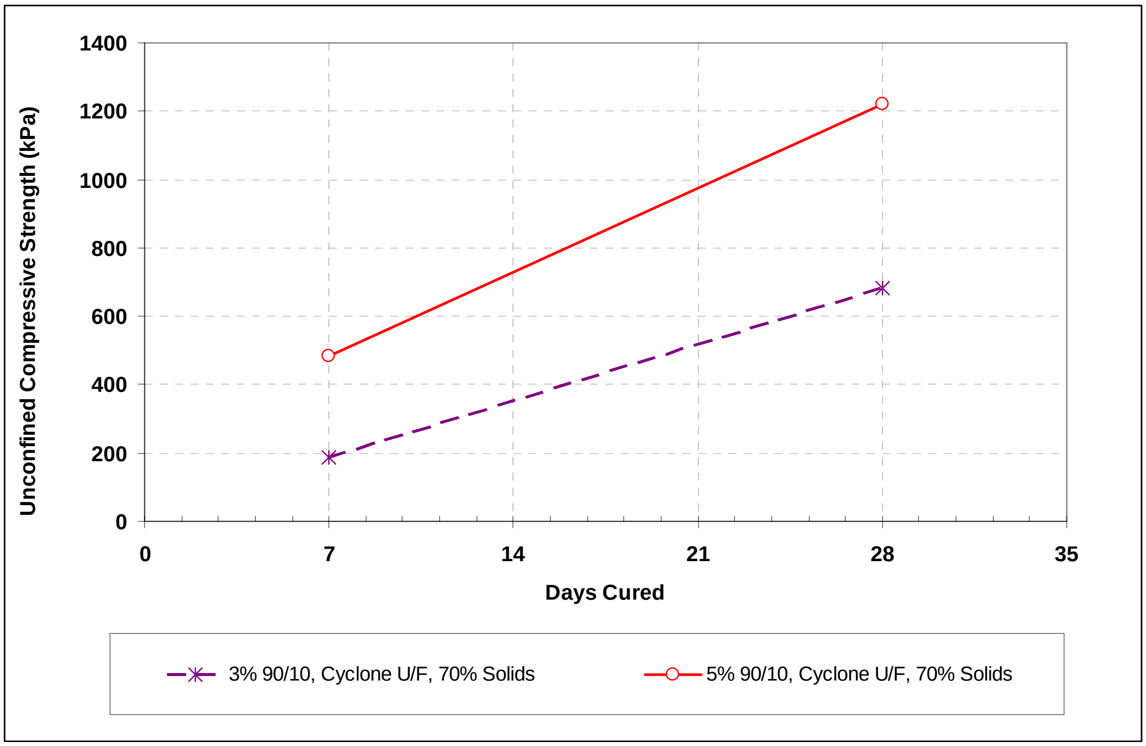
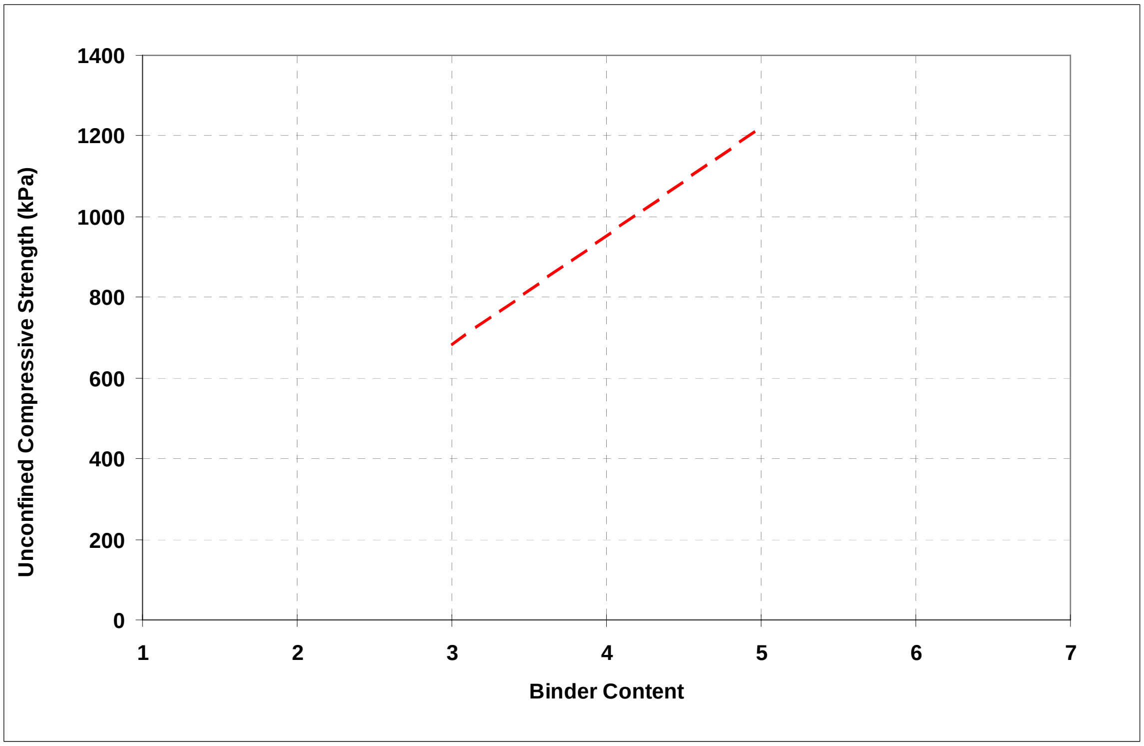


Figure 10 illustrates that good strengths of around 1 MPa could be achieved using just over 4% slag / cement binder. Actual target strengths will depend on mining method and other factors.

There was no indication of strength degradation over the time period considered. If significantly higher levels of sulphur are expected throughout the ore body, long term strength testing should be carried out.

FIGURE 10
UCS VS. BINDER CONTENT



7.0 RECOMMENDATIONS

RASP tailings produced a suitable hydraulic slurry, once cycloned to remove some of the fines. Flow loop testing of both the Blend and Cyclone U/F is recommended in order to determine expected friction loss data, and critical flow velocity for the Cyclone U/F. Small scale (50 & 75mm piping) lab loop tests benefit from a high level of control and instrumentation accuracy/precision, however a scale up is required to determine expected losses in production scale piping. Field flow loops are more expensive, but benefit from the ability to measure the pressure losses in production scale piping. Further testing should be performed prior to purchasing pumps and / or pipelines.

Sulphur is known to deteriorate cement bonds and weaken backfill over time. Some types of binder are resistant to sulphate attack. Longer term unconfined compressive strength testing is recommended if higher levels of sulphides are expected throughout the ore body. Testing of additional sample should also be carried out to provide strength variability data.

8.0 CLOSURE

If there any questions on the enclosed please do not hesitate to contact the undersigned.

GOLDER PASTE TECHNOLOGY LTD.

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Project/Lab Manager

Frank Palkovits
VP Project Development

RF/FP/mmp

APPENDIX C

Seismic Hazard Assessment (Seismology Research Centre)

Rasp Mine, Broken Hill

Seismic Hazard Assessment

February 2008

seismology research centre
a division of
environmental systems and services

es&s
environmental systems & services

Approach

This seismic hazard assessment of the Broken Hill Rasp Mine site (longitude 141.466° East, latitude 31.966° South), south of Broken Hill, New South Wales, has been produced for Golder Associates (reference David Accadia), by the Seismology Research Centre (SRC), a division of Environmental Systems & Services Pty Ltd (ES&S).

This is a probabilistic hazard assessment which employs a seismotectonic model that considers the seismicity and geology of the area in order to estimate seismic activity rates. The seismotectonic model allows for calculations of expected ground motion recurrence at the site, including peak ground acceleration (PGA) and response spectra, the results of which are included herein. Seismotectonic source contributions are presented to indicate the relative significance of each source with respect to PGA. Also included are peak ground velocity (PGV) and intensity (MMI) recurrence estimates. A deaggregation plot provides a representation of the contribution to ground motion that can be expected from different magnitude earthquakes at different distances.

All calculations have been performed assuming the site is situated on bedrock geology and all results are for horizontal motion. Note that the attenuation functions used for the different calculations are not necessarily consistent - spectral and PGA calculations are produced using the mean of Atkinson-Boore (1995) and Somerville (2001) attenuation functions, while PGV and MMI calculations have been performed using attenuation functions published by Gaull, Michael-Leiba and Rynn (1990).

Results

The PGA for the Rasp Mine site has been calculated as being approximately 0.094 g for a return period of 500 years (10% chance of exceedence in 50 years) when considering earthquakes of Richter magnitude ML 4 and above. This value is above average by Australian standards. In comparison Newcastle in New South Wales has a PGA for the same return period of approximately 0.1 g, while tectonically active areas such as Wellington in New Zealand and Los Angeles, California have a PGA of 0.5 g.

The Rasp Mine site is expected to experience earthquake intensities of MMI 6 – when standard housing experiences damage – approximately once every 500 years.

This seismic assessment has taken into consideration the structural geology in the region surrounding the Rasp Mine site, however a detailed field investigation would be required to determine if any movement has occurred along faults in the region in recent geological time and, if necessary, to estimate activity rates for inclusion in hazard calculations.

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Tables & Figures

1.1 Outline

The earthquake catalogue available to the Seismology Research Centre shows the area surrounding the Rasp Mine site to be above average in recorded seismicity by Australian standards.

Earthquake hazard calculations presented in this report take into consideration the known seismicity of the surrounding area. The structural geology of the surrounding area has also been reviewed, however a detailed field investigation would help ascertain what (if detectible) neotectonic movement has recently occurred along faults within the area.

This report concentrates on the earthquake ground shaking hazard for a bedrock site. The recurrence estimates presented herein do not incorporate frequency-dependent site effects due to surface sediments or topography. Other earthquake-related hazards, such as fault surface rupture, liquefaction and landslides, also depend on the local geology. Quantification of site dependent hazards which require detailed local information has not been presented in this study.

1.2 Background

Due to Australia's position within a major tectonic plate, its relative tectonic activity is low when compared with countries situated near to plate boundaries, where high activity is attributed to relative plate movement. Nevertheless, Australia does experience intraplate stress - typically horizontal compression - due to the collision and thrusting of the Indo-Australian plate under the Eurasian plate. The rate of accumulation of this stress is low and, consequently, earthquakes allowing the release of these stresses are less frequent. However, a low level of activity does not imply that large damaging earthquakes do not occur, but it does mean long recurrence intervals between such events.

1 Introduction

Historical earthquake records show that the Australia-wide average recurrence interval for an earthquake larger than Mw 6.5 is about 20 years. Long recurrence intervals and the relatively short duration of historical and instrumental records make it difficult to determine any additional patterns in the seismic data for Australia.

Earthquakes tend to cluster in both time and place, with previously inactive areas becoming active for relatively short periods of time. A good example of this is the area surrounding Tennant Creek in the Northern Territory. Prior to the late 1980's this area was void of any known activity and as a consequence was assigned an activity level well below the Australian average. However, in 1988 a series of three earthquakes with magnitudes of Ms 6.3, 6.5 and 6.8 struck southwest of Tennant Creek during a 12 hour period. These earthquakes produced surface faulting and were followed by thousands of aftershocks and adjustment events which are still occurring. This area is likely to be relatively active for the next hundred or so years, and will then probably revert to a low level of activity for hundreds of thousands of years.

Due to low neotectonic activity rates relative to the ongoing process of weathering, erosion and deposition of sediment in stable continental regions such as Australia, it is often difficult to identify palaeoseismic evidence, such as relative fault movements and recurrence intervals, in the geomorphology. Often geological mapping is carried out using remote sensing and other geophysical techniques, so although analysis of maps and other published material provides sufficient information for a review of the geological history of an area, specialised field work needs to be conducted for an adequate account of neotectonics.

1.3 Approach

Calculations of ground motion for this study are carried out using a modified version of the AUS5 seismotectonic model (Brown and Gibson, 2000; 2004). The model divides Australia into zones based primarily on seismicity and major geological boundaries, and activity rates are assigned to each zone based on the Gutenberg-Richter seismicity recurrence equation (see Section 2). Where the regional geology of an area is well understood, decisions can be made as to whether nearby faults have recently been active and slip rates can be determined. Where a zone is found to contain an active fault, a proportion of the zone's activity is attributed to that fault.

Attenuation functions are important in ground motion calculations. Attenuation is a term for the reduction in ground motion amplitude with distance, by geometric spreading, scattering and absorption of energy within the rock; higher frequency vibrations being attenuated quicker with distance than lower frequencies.

Once activity rates have been assigned and an appropriate attenuation function has been selected, a probabilistic seismic hazard analysis (PSHA) is completed, taking into account the ground motion from the full range of earthquake magnitudes that can occur on each fault or source zone that can affect the site (Cornell, 1968).

Outputs that are calculated from a PSHA include the following ground motion quantities:

- Peak ground acceleration (PGA) and peak ground velocity (PGV) recurrence indicate the likely level of motion that can be expected on average at a locality for a given return period from all earthquakes above a specified magnitude – the minimum considered magnitude is chosen depending on the structure being analysed.

- Uniform probability response spectral recurrence indicates the frequency dependent response spectral acceleration or velocity resulting from all earthquakes – of all magnitudes at all locations - expected in a given return period above a certain magnitude.

- Modified Mercalli intensity (MMI) calculations are produced using the same PSHA method, but using a different attenuation function. The Modified Mercalli scale provides a ‘physical’ description of the effects of shaking that can be expected at a specific locality and indicates the severity of an earthquake.

- Source contributions are plotted with respect to ground motion and return period, and show which source zones and faults contribute most to the hazard as well as providing some information on the character of this contribution. This is done for motion at a specified frequency, as distant sources dominate low frequency motion, while nearby sources dominate high frequency motion.

- A deaggregation plot indicates the contribution to the total hazard from each magnitude and distance combination. The plot is calculated for a specified ground motion frequency and return period. This also enables appropriate earthquakes to be chosen for time history calculations.

2.1 Outline

Cornell's (1968) probabilistic seismic hazard assessment (PSHA) takes into account the ground motion from a minimum considered earthquake magnitude up to the maximum considered earthquake that can occur in each source zone or along each fault that can affect a site. It is calculated from the rate of occurrence of these earthquakes, their distance from the site, and the attenuation of ground motion between the earthquake and the site. The PSHA numerically integrates ground motion probabilities to produce the annual frequency of exceedence of each different ground motion variable of interest. This relationship between ground motion level and annual frequency is called a ground motion hazard curve. The probabilities of occurrence for each of the motion are integrated, giving a uniform probability response spectrum. Uncertainties in each of the input parameters, including the location, geometry, style of faulting, the maximum magnitude of earthquake sources, fault slip rates, earthquake recurrence relationships and ground motion attenuation relationships, can have an effect on the results.

2.2 Earthquake Recurrence

2.2.1 Seismicity Equation

The estimation of ground motion requires the following seismicity information about the surrounding area:

- The rate of occurrence of earthquakes
- Relative proportion of small to large events (b-value)
- Maximum earthquake size expected (maximum credible magnitude)
- The spatial distribution of earthquakes, including delineation of faults

Seismicity can be defined by a modified Gutenberg-Richter relation:

$$\log_{10}(P) = -\log_{10}[10^{-bM} - 10^{-bMmax}] - \log_{10}(A_0)$$

- P is the return period in years for an earthquake of magnitude M or greater
- 'A_0' is the rate of occurrence of earthquakes, given as the number of earthquakes of magnitude zero or greater per year per unit volume or per unit area. An area of 100 x 100 kilometres is commonly used. This may be converted to 'per square kilometre' or 'per cubic kilometre' for ground motion calculations to allow comparison of activity in different source zones.
- 'b' is the Richter b-value (corresponding to the gradient of the plot), which gives the relative number of small earthquakes to large. It is the logarithm to the base 10 of the ratio of the number of events exceeding magnitude M over the number exceeding M+1. A value of 1.0 would correspond to ten times as many earthquakes exceeding magnitude M as would exceed magnitude M+1.
- M_{max} is the magnitude of the maximum credible earthquake for the area. It is the magnitude of an earthquake with infinite return period. Because of the low probability of very large earthquakes, M_{max} does not critically affect ground motion recurrence estimates for return periods up to hundreds of years, especially when the b-value is high. The maximum credible magnitude causes the magnitude recurrence plot to flatten out and asymptote.

The seismicity parameters A_0 and b are determined using available earthquake data. Where there are insufficient earthquake data to determine M_{max} from historical records, values can be estimated by considering the tectonic situation and local fault dimensions.

The b-value varies with location over the earth, and it can vary with time at a particular location. Reliable values are normally within the range 0.7 to 1.4, although lower and higher values have been reported. The b-value tends to be higher in very active earthquake zones, such as Papua New Guinea or Indonesia where it is usually above 1.0. It is lower in less active areas such as Australia where it is often about 0.80 to 0.95.

By measuring the rate of occurrence of small earthquakes over a relatively short period, it is possible to estimate the return periods for large earthquakes. The b-value estimate is usually critical in this extrapolation, especially if the seismicity data has only been accumulated over a short period. Because the linear relation is between the logarithm of the return period and magnitude (which is proportional to the logarithm of ground motion), a small error in the b-value may give a highly misleading hazard estimate - if the b-value has been estimated too low then the extrapolated return period for a large earthquake will be low, and the hazard will be over-estimated. This can easily happen if the available catalogue has not properly incorporated small earthquakes over the whole region.

See Figure 7 for an example of the application of the Gutenberg-Richter recurrence relation.

2.2.2 Recurrence of Large Earthquakes

In most places, the duration of seismograph coverage and the duration of the historical earthquake records are both much less than the return period for large earthquakes.

Two methods are used to estimate the recurrence rates for large earthquakes:

- It is assumed that activity in a particular tectonic regime is reasonably uniform on a large scale, and area is traded for time. For example the activity in an area of 1000 x 1000 km = 1,000,000 km^2 accumulated for 100 years gives an idea of what may be expected in an area of 100 x 100 km = 10,000 km^2 over a period of 10,000 years.

- It is assumed that the relative number of small to large earthquakes (the b-value) remains constant. At the normal b-value of 1.0, an area will experience 10 times as many magnitude 4 earthquakes as magnitude 5, and ten times more magnitude 3 events than magnitude 4. It is possible to extrapolate to get the rate of occurrence of rare larger earthquakes by assuming that this value remains constant for earthquakes to magnitude 6 or larger. This assumption would be invalid if there is a characteristic earthquake magnitude for a particular fault segment.

2.2.3 Large Australian Earthquakes

The earthquake catalogue only covers a short period compared with the return period needed for design purposes, therefore a large area must be considered when estimating M_{max}, perhaps even as large as the whole of Australia.

The largest earthquake known to have occurred in eastern Australia struck near Beachport in south-east South Australia during the evening of 10 May 1897 (at 0526 UTC). It produced modified Mercalli intensities of 8 in the epicentral area, and was felt from the Eyre Peninsula to Melbourne, at distances exceeding 500 kilometres. Extensive liquefaction was observed at Robe, Beachport and Kingston. The magnitude exceeded Mw 6.5, and was probably almost Mw 7.0. The epicentre was most likely offshore.

An earthquake of magnitude approximately ML 6.0 occurred off Bundaberg in Queensland on 7 June, 1918 at 4:15 am local time (6 June at 1815 UTC). There was some damage in Rockhampton, with fallen chimneys, cracked walls and broken windows. It was felt widely in Brisbane, particularly in the bayside suburbs.

The largest known onshore Australian earthquake was probably that which occurred near Meeberrie in Western Australia on 29 April 1941 (at 0135 UTC). It was felt at distances exceeding 900 kilometres, so its magnitude may have exceeded Mw 7.0. Damage was minimal because of the low population density in the epicentral area.

The Meckering earthquake in Western Australia was the most damaging to have occurred in Australia when it destroyed the town of Meckering and caused other damage over a wide area east of Perth. It occurred at 1058 am WST (0258 UTC) on 14 October 1968. The faulting produced a surface rupture in a 32 kilometre arc with over 2 metres of total movement. By good fortune there were no fatalities and few injuries. The magnitude determined by the Mundaring Geophysical Observatory was Ms 6.9.

A series of three large earthquakes, with magnitudes of Ms 6.3, 6.5 and 6.8, struck southwest of Tennant Creek in the Northern Territory during a twelve-hour period on 22 January 1988. The earthquakes produced surface faulting comparable with that seen at Meckering. The earthquakes were followed by many thousands of aftershocks and adjustment events, which are still occurring. The epicentre was in a sparsely inhabited area and there were no injuries. The total damage was about A$1 million, the most significant being to a natural gas pipeline which passed through the fault rupture.

These events indicate the magnitude of the largest earthquakes that can be expected in Australia. Although it is not likely that we will experience a great earthquake exceeding Mw 8.0, it is possible that events up to Mw 7.5 will occur. These will be very rare, and most earthquake damage will be from smaller earthquakes of magnitudes 6 to 7.

2.3 Seismograph Network

The quality of any earthquake hazard analysis depends on the quality of the data used. The quality of instrumental seismic data depends on the number, type and distribution of recording instruments, and on the noise level at each site.

The minimum magnitude for which an earthquake catalogue will be complete varies with time and location. Prior to the installation of a seismograph network, it depends on the distribution of the population and the presence of a system for reporting and storing earthquake information. Where a seismograph network has been installed, it depends on the distribution and sensitivity of the seismographs.

If an earthquake is recorded on only one seismograph, its distance may be estimated but its precise location cannot be determined. Unless an epicentre is surrounded by several seismographs, there will be an uncertainty in its location estimate of up to tens of kilometres.

The seismograph coverage around Rasp Mine is represented in Figure 1. The closest seismograph to the site is approximately 510 kilometres to the southeast, in north central Victoria, near Echuca, there is an additional seismograph recorder within 20 kilometres of the site monitored by an external government agency, Geoscience Australia and within 280 kilometres of the site monitored by Primary Industries & Resources, South Australia.

2.4 Earthquake Catalogues & Seismicity Maps

Prior to the installation of seismographs, earthquakes large enough and close to populated areas, especially capital cities, were much more likely to be reported than smaller earthquakes or those in remote areas. Therefore, plots of historical earthquake epicentres can give a misleading bias towards moderate to large earthquakes occurring in populated areas. During the 1960's, and especially from 1965, significant improvements in the Australian seismograph network were made allowing smaller events and those in more remote areas to also be detected. Figure 2 shows those earthquakes reported prior to 1960, while Figure 3 illustrates the capture of more earthquakes, including smaller-sized events, since the network improvements. The relative number of earthquakes shown in these figures is not necessarily an indication of earthquake hazard. Due to the sparse network around the Broken Hill area, earthquakes recorded nearby and especially to the north, continue to be dominated by those that are large enough to be felt or recorded by distant seismographs. A denser seismograph network is required in order to record smaller earthquakes, thereby providing a more realistic indication of the seismicity of the area.

Table 1 lists all recorded earthquakes that occurred within 80 km of Rasp Mine. Table 2 lists earthquakes which produced an intensity of MMI 3 and above at the site.

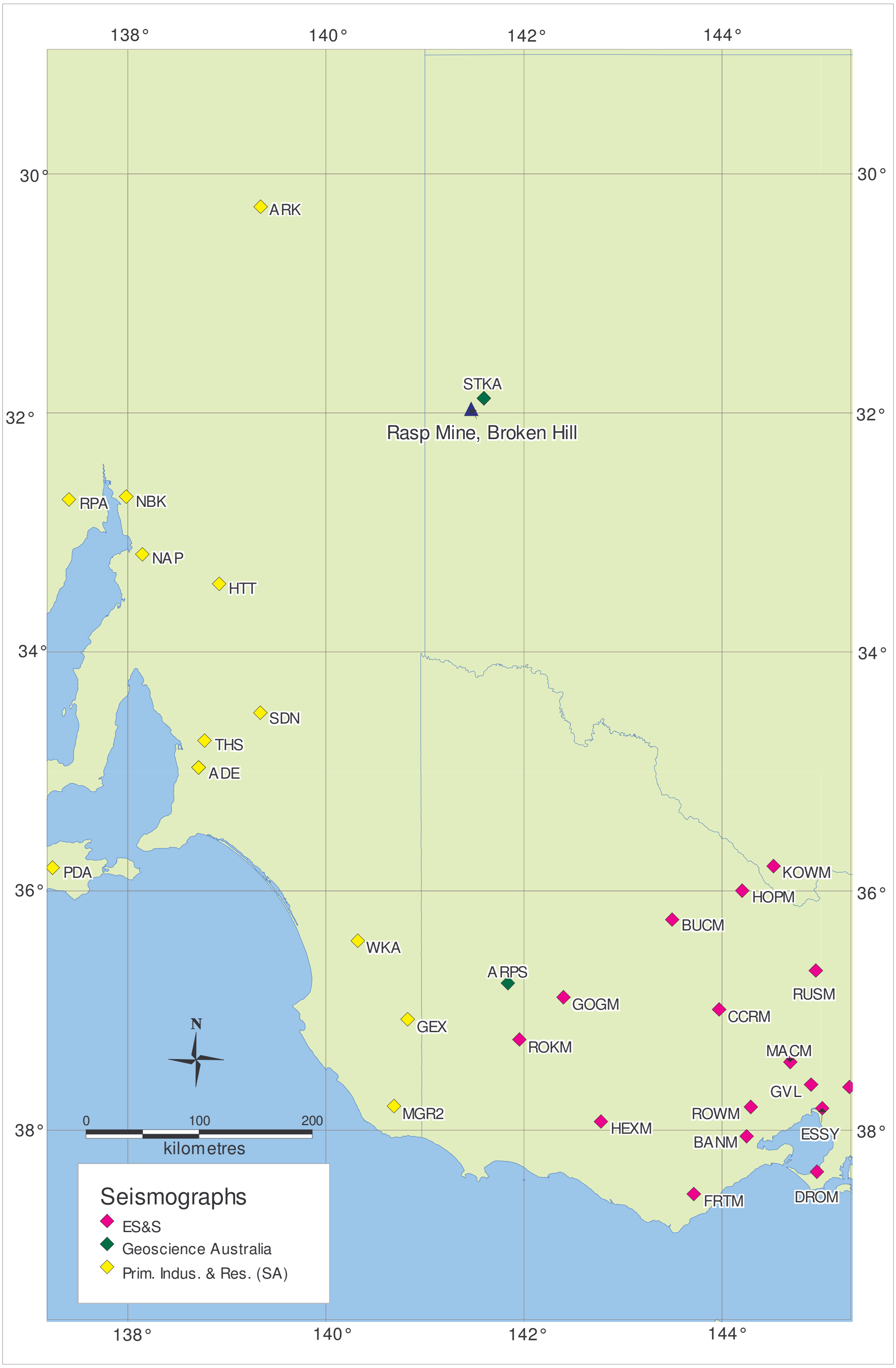

Figure 1: Seismograph sites surrounding Rasp Mine, Broken Hill

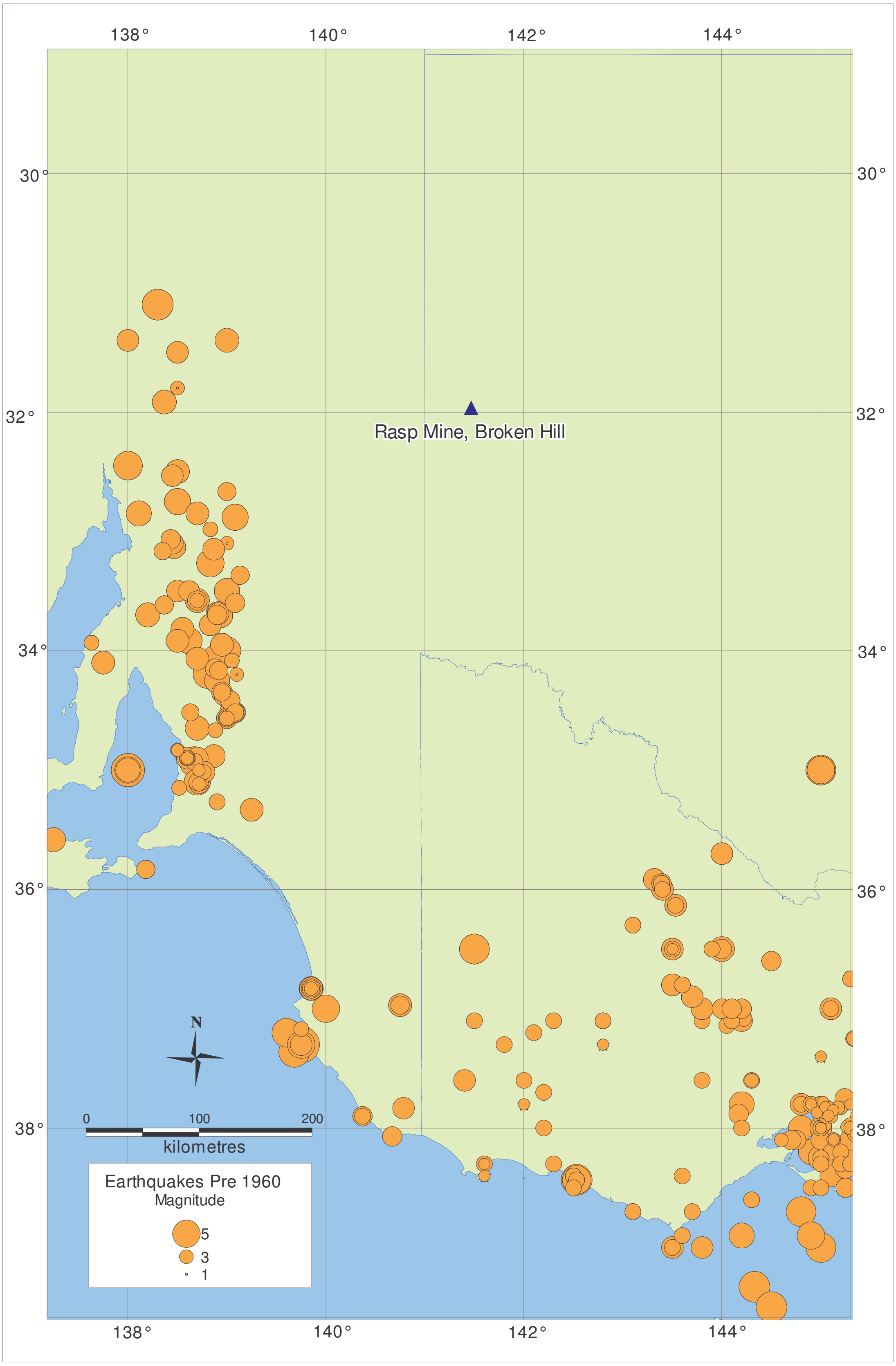

Figure 2: Earthquakes reported before network installation, prior to 1960

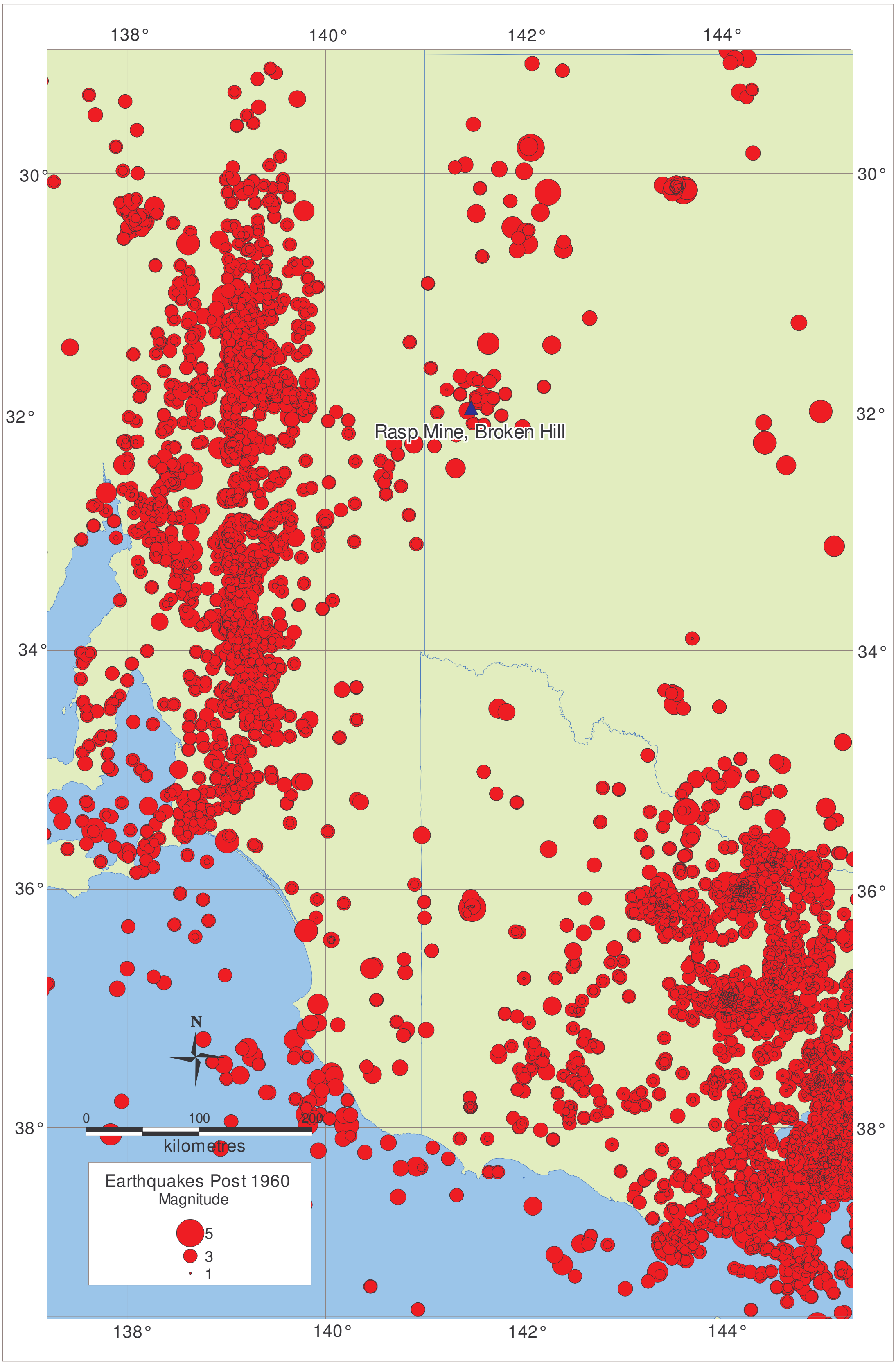

Figure 3: Earthquakes recorded after network installations, from 1960

UTC Date/Time	Place	Longitude ° E	Latitude ° S	Distance (km)	Depth (km)	Magnitude
1899-04-12 0025	STAWELL	142.80	-37.10	52	10	MP 3.0
1907-04-04 2225	STAWELL	142.80	-37.10	52	10	MP 3.0
1907-05-04 0000	WARTOOK	142.30	-37.10	37	10	MP 3.0
1907-05-05 0245	ARARAT	142.80	-37.30	69	10	MP 2.0
1908-04-13 0200	HAMILTON	142.10	-37.20	54	10	MP 3.0
1911-08-26 0915	BALMORAL	141.80	-37.30	78	10	MP 3.0
1976-01-08 1806	ROCKLANDS RESERVOIR	142.06	-37.29	65	4	MD 2.4
1976-01-18 0447	ROCKLANDS RESERVOIR	142.11	-37.35	69	4	ML 2.6
1977-08-16 1128	MINYIP	142.50	-36.52	28	17	ML 3.2
1977-08-27 1606	MIRRANATWA	142.55	-37.45	76	7	MD 1.8
1978-07-01 1613	BELLFIELD	142.50	-37.17	45	17	ML 1.2
1979-07-23 1950	WARRACKNABEAL	142.43	-36.31	51	15	MD 2.5
1980-08-15 2108	ST ARNAUD	142.97	-36.74	52	10	MD 1.6
1980-10-05 2011	BALMORAL	141.93	-37.07	52	12	MD 2.6
1981-10-19 1628	ST ARNAUD	142.96	-36.73	51	9	ML 1.8
1982-05-16 1634	WYCHEPROOF	143.11	-36.39	77	4	MD 2.1
1982-10-30 0718	ROCKLANDS RESERVOIR	142.19	-37.45	78	15	ML 2.1
1984-01-24 1113	ST ARNAUD	143.00	-36.61	57	8	ML 1.5
1985-04-18 0209	BELLFIELD	142.49	-37.01	29	10	ML 1.3
1985-05-24 0312	ROCKLANDS RESERVOIR	142.10	-37.30	64	8	ML 1.9
1985-06-19 0750	NATIMUK	142.00	-36.75	34	2	M? 0.0
1985-06-20 0845	NATIMUK	142.00	-36.75	34	2	M? 0.0
1985-06-20 0915	NATIMUK	142.00	-36.75	34	2	M? 0.0
1985-06-20 0930	NATIMUK	142.00	-36.75	34	2	M? 0.0
1985-06-25 2155	WONWONDAH	142.28	-36.98	25	14	ML 3.4
1986-03-15 1304	TOOLONDO	141.81	-37.05	60	12	ML 2.2
1988-09-30 1040	ARARAT	142.75	-37.33	70	10	ML 2.2
1988-09-30 1048	ARARAT	142.70	-37.31	66	12	ML 1.3
1988-11-06 1136	COPE COPE	142.91	-36.50	55	4	ML 2.9
1988-11-07 2355	MURTOA	142.70	-36.86	30	26	ML 1.5
1990-02-16 0316	MT WILLIAM	142.67	-37.32	66	16	ML 2.6
1990-03-05 2330	MT WILLIAM	142.60	-37.28	60	16	ML 1.9
1991-10-12 1708	HORSHAM	142.32	-36.75	66	3	ML 1.7
1993-01-15 0524	MURTOA	142.51	-36.63	19	5	ML 1.6
1993-01-15 1050	GLENORCHY	142.59	-36.94	26	9	ML 1.2
1993-05-28 0522	HALLS GAP	142.55	-37.02	31	10	ML 0.9
1993-07-15 0443	LAKE LONSDALE	142.53	-37.02	31	7	ML 1.1
1993-08-13 0149	LAKE LONSDALE	142.55	-37.01	31	4	ML 0.9
1996-06-11 0209	MURTOA	142.49	-36.66	15	4	ML 2.1
1998-06-06 1219	BALMORAL	141.87	-37.32	76	12	ML 1.4
1998-07-25 2233	POMONAL	142.62	-37.23	55	10	ML 2.2
1999-06-16 1345	GRAMPIANS	142.52	-37.41	73	24	ML 2.4
2000-09-27 1628	WARRACKNABEAL	142.62	-36.08	78	10	ML 2.6
2003-05-04 1153	HALLS GAP	142.05	-37.12	49	10	ML 0.7
2003-08-11 1712	DIMBOOLA	141.95	-36.36	59	10	ML 1.5
2003-10-30 2257	CAVENDISH	142.08	-37.42	77	10	ML 1.0
2003-10-31 0715	CAVENDISH	142.19	-37.39	71	10	ML 0.6
2004-01-31 0123	STAWELL	142.80	-36.77	36	10	ML 1.5
2004-05-10 0731	CAVENDISH	142.10	-37.37	71	10	ML-0.1
2004-09-21 2002	MARNOO	142.88	-36.62	46	10	ML 1.5
2006-10-09 1930	NAVARRE	143.06	-36.90	62	10	ML 2.0

Table 1: Earthquakes recorded within 80 kilometres of Rasp Mine

UTC Date/Time	Place	Longitude ° E	Latitude ° S	Distance (km)	Depth (km)	Magnitude	Intensity (MM)
1897-05-10 0526	KINGSTON	139.75	-37.30	241	14	MP 6.6	4
1903-07-14 1028	WARRNAMBOOL	142.53	-38.43	185	10	MP 5.6	3
1905-08-21 1600	WEST VICTORIA	141.50	-36.50	84	15	MP 5.5	5
1907-05-04 0000	WARTOOK	142.30	-37.10	37	10	MP 3.0	3
1977-08-16 1128	MINYIP	142.50	-36.52	28	17	ML 3.2	3
1985-06-25 2155	WONWONDAH	142.28	-36.98	25	14	ML 3.4	4
1987-12-22 1506	YANAC	141.48	-36.16	105	16	ML 4.9	3
1996-06-11 0209	MURTOA	142.49	-36.66	1	4	ML 2.1	3

Table 2: Earthquakes resulting in MMI 3 and above at Rasp Mine

3.1 Ground Motion Attenuation Functions

The ground motion resulting from a fault rupture reduces in amplitude with distance from the earthquake for three reasons - geometric spreading, scattering and absorption of energy within the rock. The combined effect of these is the attenuation of seismic wave amplitudes with distance.

Earthquake ground motion can be measured as a ground displacement, ground velocity, or ground acceleration. The recorded motion at any point varies with time as the different wave types, reflections, refractions and converted waves arrive at the point. The motion can be converted between displacement, velocity and acceleration by differentiation and integration. Displacement is highest for low frequency motion, while acceleration is higher for high-frequency motion.

A crude measure of the amplitude of the motion is the peak ground displacement, peak ground velocity (PGV) or peak ground acceleration (PGA). These values give no indication of the vibration frequency of the ground motion, or the duration of the ground motion.

3.2 Strong Motion Data

To the current day, few accelerograms have been recorded in Australia, other than some from small nearby events or from large distant events. It is not yet possible to estimate local attenuation using just these.

Until local attenuation functions can be derived, functions from areas with a similar geology must be used. Attenuation in shield areas such as Western Australia is low, while in volcanically active areas, such as Papua New Guinea, it is high. It is estimated that attenuation in eastern Australia, where basement rocks are Palaeozoic or younger, is a little higher than average and similar to that experienced in California where a number of attenuation studies have been conducted.

3.2.1 Peak Ground Motion & Intensity

By restricting the magnitude contributions used in the estimation of the peak ground acceleration, the high frequency, high acceleration, low displacement, and short duration motion from small earthquakes is eliminated. The vibrations from small earthquakes are unlikely to have any adverse effect on large structures, but will increase the peak ground acceleration recurrence estimates for all return periods.

Attenuation functions used in ground motion estimates usually have limitations in earthquake magnitude and distance. Most functions have been determined using data from earthquakes in the magnitude range 5 to 7, and many are unreliable if extrapolated outside this range. For example Esteva and Rosenblueth (1964) assume that the earthquakes are at distances between 10 and 300 kilometres, and outside those limits the estimated motion becomes unreliable. In intraplate locations, earthquakes with distances or depths greater than about 300 kilometres can usually be ignored in ground motion calculations, as the effect of these earthquakes on ground motion estimates is negligible. Large earthquakes at greater distances may be felt, though never at levels that will produce damage.

3.2.2 Spectral Attenuation

Earthquake ground motion studies almost invariably require spectral analysis - examining the variation of ground motion with frequency (or its inverse, period). Two types of spectra are commonly used, the Fourier spectrum and the response spectrum.

The Fourier spectrum gives another way of looking at a time series (or seismogram), and allows easy and rapid conversion between a time representation of the motion (the wiggly line) and a frequency content (or spectral) representation. Some tasks are best done in the time domain and others in the frequency domain. It is easy to Fourier transform in either direction, time series to spectra, or vice versa.

The response spectrum gives the peak response, or deformation, of a damped harmonic oscillator to the ground motion as a function of frequency. The response spectrum of a particular earthquake vibration may be computed for a particular level of damping from the time series using the Duhammel integral, a method analogous to the Fourier spectrum. However, it is not possible to invert back to a time series from a response spectrum.

The Uniform Probability Response Spectra included in our report are not the response spectra from particular earthquakes, but are the cumulative result from probabilities of all possible earthquakes, small and large, near and far, for the particular return period.
A Pseudo Relative Velocity spectrum is computed from the acceleration response spectrum by dividing by $2\pi f$, and gives a better indication of low frequency motion, especially when plotted on a log-log tripartite plot.

A typical spectral response spectrum is of the form:

$$\ln[PSV(f)] = C_1 + C_2M + C_3\,(8.5\text{-}M)^{2.5} + C_4\ln(R_{rup} + \exp(C_5 + C_6M)) + C_7\ln(R_{rup} + 2)$$

- PSV is pseudo relative velocity
- f is frequency
- M is local or moment magnitude
- R_{rup} is the minimum distance to the rupture surface
- C_1 to C_7 are coefficients that vary with frequency and magnitude

Because of many source, path and local site effects, the scatter from mean values for earthquake peak motion or spectra is very high. Values ranging from half to double the mean values are very common. The distribution is approximately log-normal, so motion stronger than the mean has a greater impact on ground motion recurrence estimates than motion weaker than the mean. The greater the distribution the higher the ground motion recurrence, and therefore higher the hazard. For the study of a particular site this variation is usually considered within the calculation of ground motion recurrence. Alternatively the mean plus standard deviation values may be used, but may be too conservative.

One of the first combined source and attenuation functions to provide the spectral content of ground motion was given by Trifunac (1976). This yields values for the Fourier spectrum of ground acceleration as a function of earthquake magnitude, distance and depth, also distinguishing between horizontal and vertical motion.

Sadigh *et. al.* (1997) have given an attenuation function derived mainly from Californian data, including the 1989 magnitude Mw 7.0 Loma Prieta, the 1992 Magnitude Mw 7.3 Landers, and the 1994 magnitude Mw 6.7 Northridge earthquakes. This gives acceleration response spectra as a function of magnitude (from about Mw 4 to Mw 8 and above), distance (0 to 100 kilometres) and for a bedrock site.

3.2.3 Variation in Attenuation Functions

Figure 4 is a plot of PGA for an M 6.0 earthquake at a range of distances as estimated using several attenuation functions.

These plots demonstrate the wide variation in PGA values that are possible depending on which attenuation function is used. The functions give values varying by a factor of 5 for events at distances of 10 to 100 kilometres; for earthquakes outside this range a larger variation is possible.

In addition to the variation between attenuation functions it must be remembered that each of these is an average of widely dispersed data with considerable variance. The standard deviation is typically about a factor of 2.0 although modern attenuation functions, which consider more parameters, may reduce this to a factor of 1.6.

Any difference in attenuation between western North America and eastern Australia should not significantly affect the computed strong motion near the epicentre of larger events, where the distance travelled by seismic waves is short. If the Australian attenuation is much higher than is assumed, then the ground motion results will be conservative for events at moderate to large distances. At large distances the motion will normally be too low to provide a significant hazard.

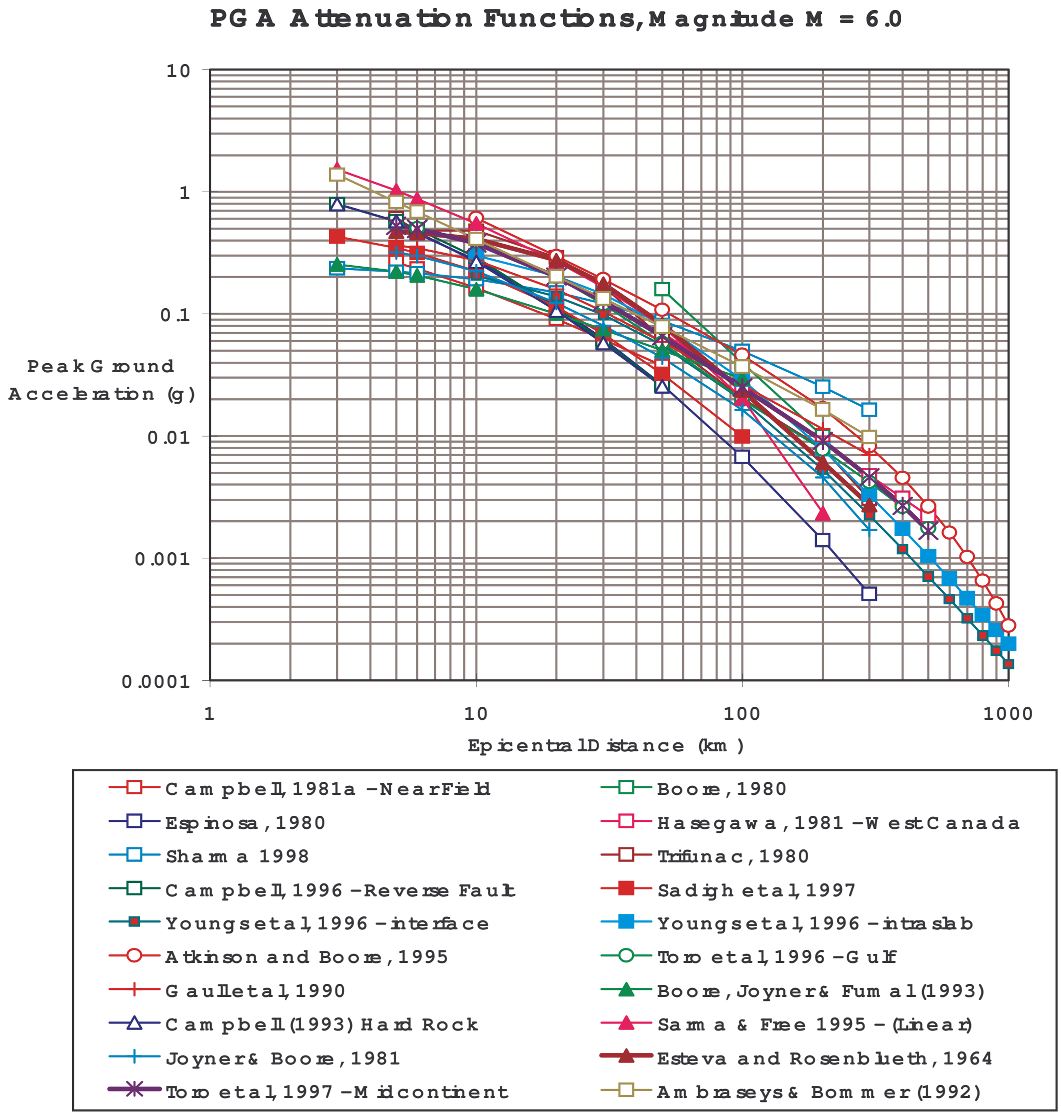

Figure 4: Variation in PGA attenuation functions

4.1 Geology

The Rasp Mine site is located on the southern boundary of Broken Hill, in western New South Wales.

The site is situated on the eastern margin of the Adelaide Fold Belt, where inliers of the Curnamona Craton, a component of the Australian Craton, are exposed. The Curnamona Craton is represented by rocks of the Willyama Supergroup, namely marine to continental sediments and igneous rocks of Palaeo-Mesoproterozoic age which are believed to have been deposited in a failed rift and later exposed to intense deformation resulting in faulting and thrusting.

The Neoproterozoic to early Cambrian sediments and volcanics of the Adelaide Fold Belt were deposited over the older basement rocks and experienced deformation and metamorphism during the Cambrian Delamerian orogeny. The area experienced further continental sediment accumulation and possible intraplate igneous activity during the Cainozoic (Scheibner & Basden, 1996).

The geology surrounding the Rasp Mine consists of gneiss, schist, phylite, quartzite, sandstone and slate of the Willyama Supergroup which is often obscured by a cover of Quaternary sand and gravel. Further to the west and east, Quaternary silt and sand dominates the geology (Rose, 1967; 1968).

A number of faults have been identified in the area surrounding the Rasp Mine, such as the east-west oriented Thakaringa-Pinnacle Fault and, with a similar orientation, the East-West Fault; both located a few kilometres south of the site. A few kilometres east of the site is the northeast trending Globe-Vauxhall Fault, while to the northwest is the Mundi Mundi Fault (Rose, 1967; 1968). Although movement along these faults was significant in the deformation periods of the Palaeozoic, most are no longer considered active. However, the prominent scarp delineating the Mundi Mundi Fault suggests that it has been active in recent geological time with an estimated 5m/Myr rate of activity assigned.

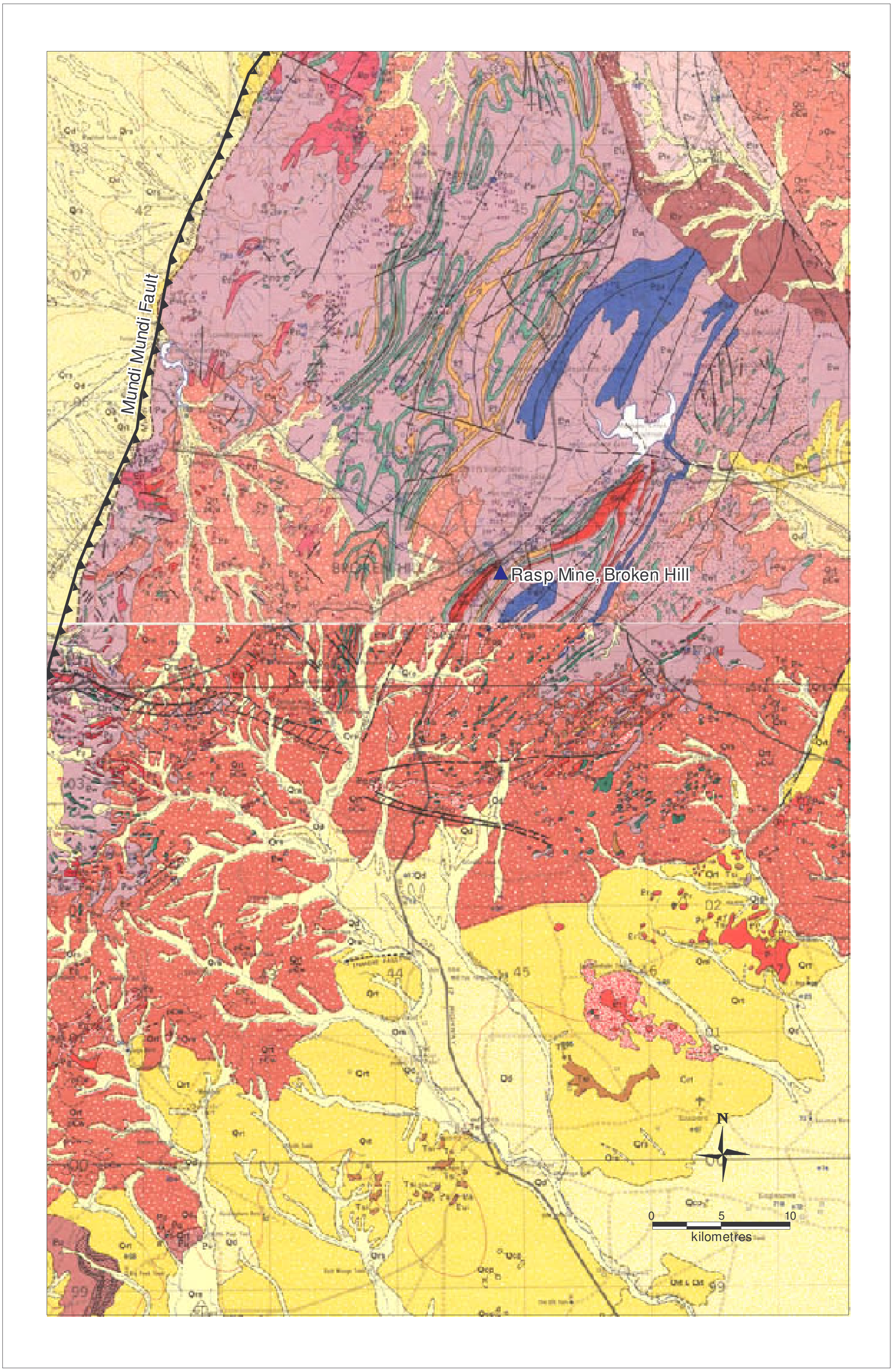

Figure 5: Regional geology of Rasp Mine site (Rose, 1967; 1968)

4.2 The AUS5 Model

Australia's seismicity data has been summarised in a seismotectonic model called AUS5 (after Brown and Gibson, 2000; 2004), which has been applied to this study. The AUS5 model divides Australia into seismotectonic area source zones based on seismicity and the spatial distribution of earthquakes, as well as geology – particularly neotectonics relating to Quaternary and Tertiary deformation – and assigns a rate of activity to each zone. Within each zone, earthquakes are assumed to be distributed uniformly with depth from 2 to 20 kilometres and the long-term level of earthquake activity is assumed to be uniform.

If an area source zone contains an active fault, then its treatment in the hazard computation depends on its proximity to the site. For distant sites, all earthquakes are distributed uniformly across the source zone, and the fault activity is well understood, the earthquakes associated with the fault are included on a separate fault source zone and subtracted from the area source zone, giving a background source zone with a lower level of activity.

Although many of the faults nearby to the Rasp Mine site are believed to have long ceased being active, the scarp delineating the Mundi Mundi Fault suggests movement along this fault in recent geological time. The Mundi Mundi Fault has therefore been considered active in the calculation performed for this study. Figure 6 shows the area source zones nearest to the Rasp Mine site.

4.3 Earthquake Magnitude Recurrence

Figure 7 is an example of the quantification of earthquake magnitude recurrence for the Broken Hill seismotectonic zone, within which the Rasp Mine site is located. A lighter line is plotted when the return period has been computed using fewer than ten events. The plots become increasingly uncertain towards large magnitudes.

It is assumed that the Gutenberg-Richter seismicity distribution applies and the data will fit the estimated activity level and b-value plot down to the 'catalogue completeness magnitude' for the period. Earthquakes smaller than the 'catalogue completeness magnitude', may not be recorded on sufficient recorders to be located.

The density of a seismograph network limits what magnitude size will be missing from the catalogue. The lower parts of the plots lie above the selected line, because many smaller events are missing from the catalogue. Because the catalogue includes all of the larger earthquakes that have occurred during the period, the plot asymptotes the selected line for larger magnitudes.

The plot for the period from 1966 is similar to that for the period since 1974 because of the few events recorded prior to the network improvements in the mod 1970's.

For zones with a low level of activity, the gradient of the Gutenberg-Richter line (the b value) is usually determined from a regional earthquake magnitude recurrence study by combining a number of neighbouring zones with a similar geological setting.

For small magnitudes, the plots for the periods from 1900 are well above that for the period since 1960 because of the very poor seismograph network coverage prior to this time, so have not been consider in the determination of A_0.

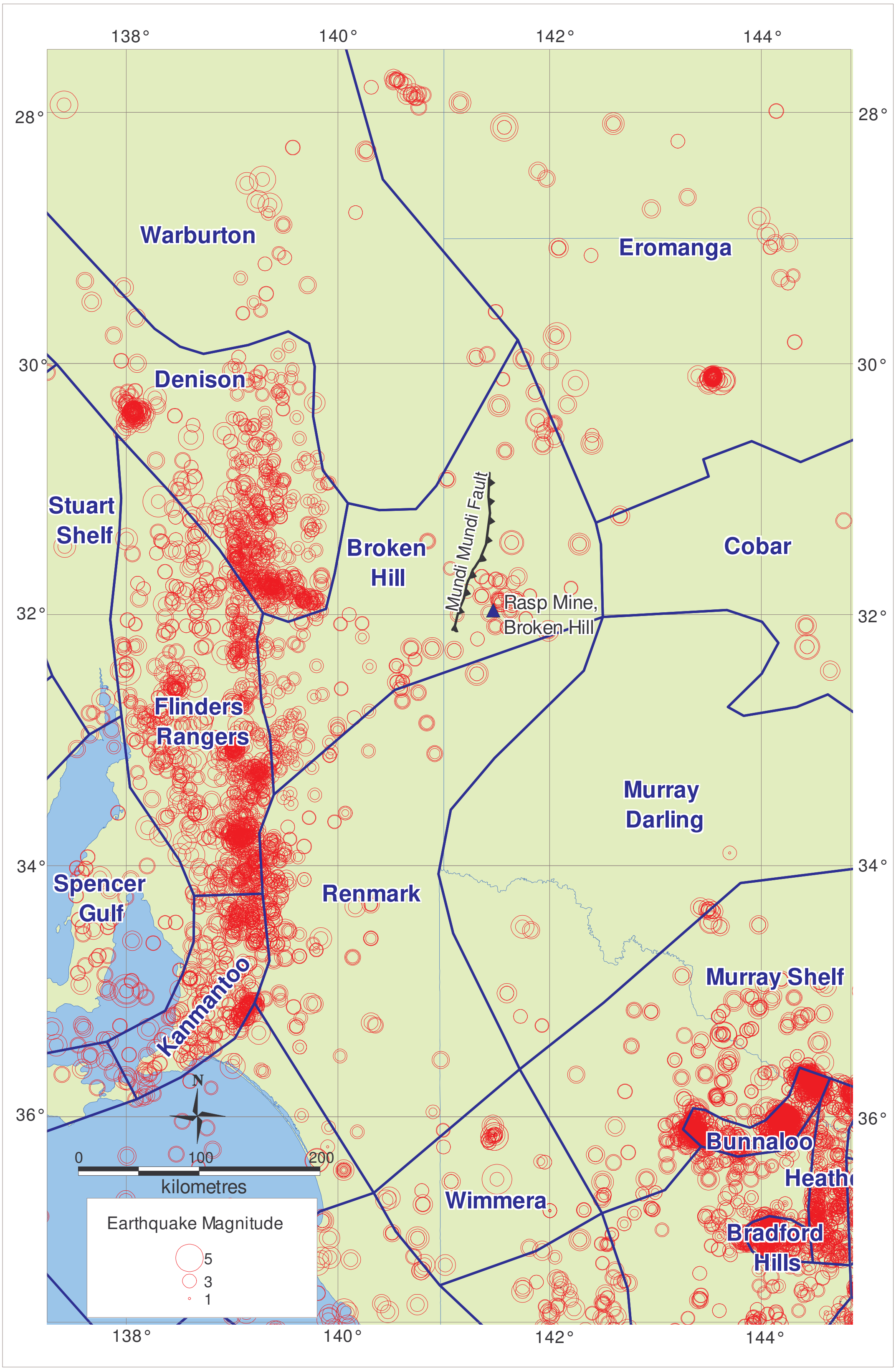

Figure 6: Seismotectonic source zones surrounding Rasp Mine

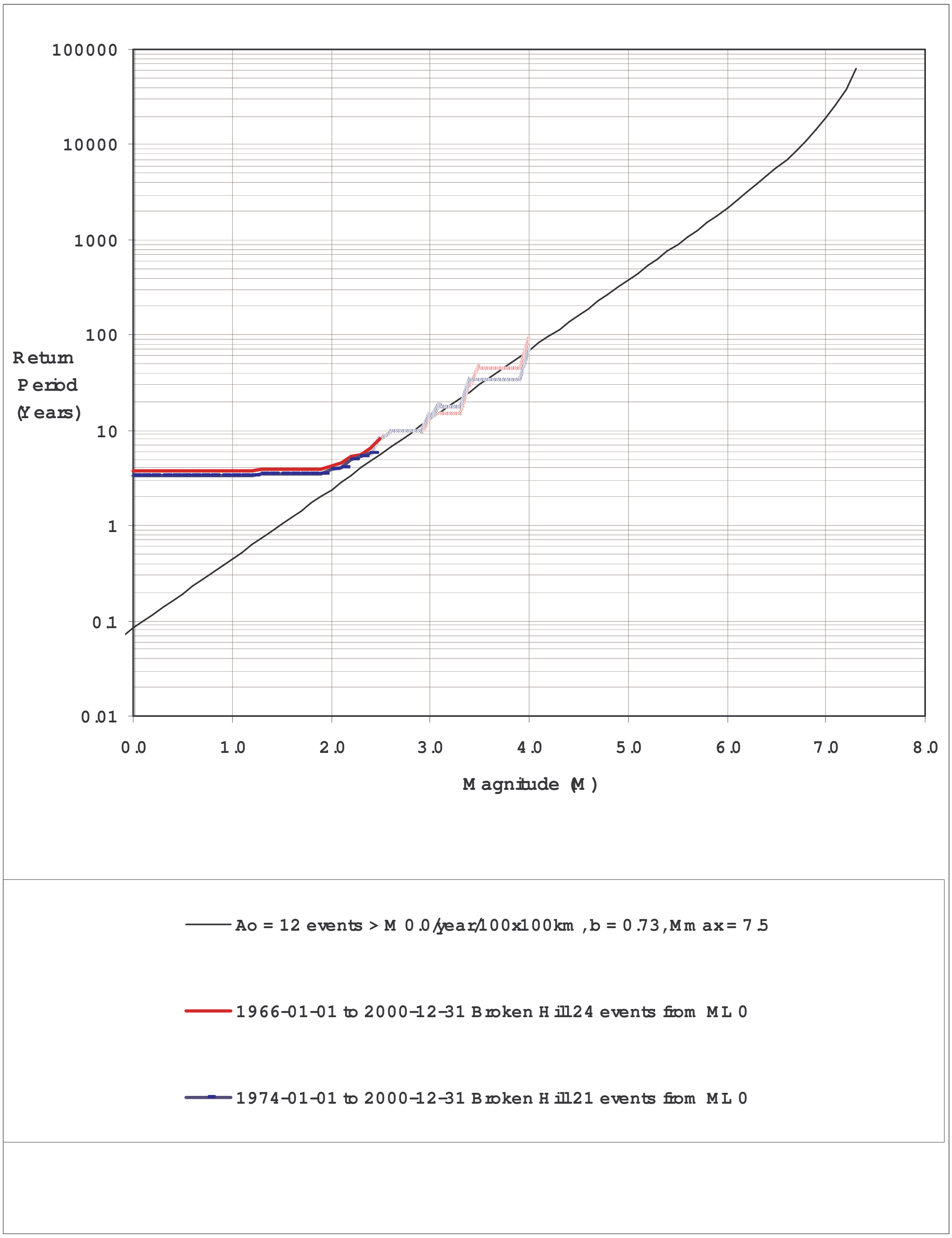

Figure 7: Earthquake magnitude recurrence plot – Broken Hill source zone

It is assumed that extrapolation from smaller magnitudes gives a better long-term estimate for earthquake activity than consideration of a very small number of larger earthquakes. This assumption would not apply if special circumstances suggested that a major earthquake could be predicted, such as a fault that seems to follow a cycle of activity, or if a comparatively rare larger event has occurred during the catalogue period.

The maximum credible earthquake magnitude is not constrained by the seismicity data, but has been estimated after consideration of the tectonic setting and magnitudes of other large Australian earthquakes. There are many known faults in Australia that are large enough to produce an earthquake of magnitude Mw 7.5, and it is possible that other such faults exist and are not yet discovered or are hidden by the surface geology. In areas of low seismicity, such as Australia, the choice of maximum credible magnitude does not significantly affect ground motion recurrence estimates for return periods up to tens of thousands of years.

4.4 Attenuation & Peak Ground Acceleration

Determining a mean attenuation function of Atkinson-Boore (1995) and Somerville (2001) is an appropriate choice for a hazard study on cratonic interiors in Australia, yielding relatively high frequency motion consistent with earthquakes in a low-seismicity continental region. The spectral shapes are also more consistent with the high stress-drop earthquakes that are experienced in central Australia. The functions are out of range for magnitudes less than about Mw 4.0, so the ground motion for smaller events may not be realistic.

A summary of various attenuation functions is presented in table 3, thus illustrating the differences between them. For example, the Australian Standard for Broken Hill is between 0.04 to 0.05 g for M 4.0 for 500 years. The function for Sadigh (1997) is also presented here although it is not relevant, as it is based on California and western USA, hence most comparable to eastern Australia. The mean of Atkinson-Boore (1995) and Somerville (2001) using area source zones only, is closer to the Australian Standard with 0.07 g, however there is a further complication of the Mundi Mundi fault which is taken into account for this site.

A conservative slip rate was used for the calculation of this fault (5 m/Myr), as other authors claim a slip rate of up to 48 m/Myr (Quigley et al, 2006), an excessively high value by Australian comparative rates. This rate would assume that the site is situated in an area that is more tectonically active than the Gippsland region in eastern Victoria or even the Flinders Ranges in South Australia. There is no neotectonic evidence to suggest that this has continued into the Holocene, also supported by Gibson (1997). Therefore the best estimates of hazard are demonstrated by using the attenuation function of Atkinson-Boore (1995) and Somerville (2001) including the Mundi Mundi fault.

Attenuation Function	500 years	10,000 years
Australian Standard (AS 1170.4)	0.04 --> 0.05 g	N/A
Sadigh (1997)	0.084 g	0.330 g
Atkinson-Boore (1995) & Somerville (2001), area sources only	0.068 g	0.410 g
Atkinson-Boore (1995) & Somerville (2001), area sources and faults	0.094 g	0.520 g

Table 3: Comparison of peak ground acceleration using various attenuation functions M4.0

4.5 Site Conditions

Results given in this study assume that bedrock outcrops at the site. Soft sediments amplify the low frequency seismic waves of large earthquakes, but will reduce the ground motion of smaller local events by the absorption of high frequency seismic waves. If surface sediments are located at Rasp Mine, the spectra given must be multiplied by a frequency-dependent transfer function fitting to the sediment depth.

4.6 Minimum Considered Magnitude

It is observed that earthquakes smaller than about magnitude 5 rarely cause any damage, even if earthquakes are shallow. If earthquakes are deep, then events smaller than magnitude 6 will not cause damage. The vibrations from small earthquakes are unlikely to have any adverse effect on large structures, but will increase the peak ground acceleration estimates for shorter return periods.

When computing the ground motion recurrence, it is possible to consider only earthquakes larger than some magnitude up to the maximum credible magnitude for each source zone. The minimum magnitude may be chosen depending on the type of structure being considered, with larger structures being less affected by small earthquakes than small structures. Minimum magnitude values may vary in the range from 4 up to 6 depending on the engineering structure being considered and local conditions.

If a minimum magnitude is chosen, then the results of the earthquake hazard computations should not be applied generally, but consideration should be given to the suitability of the chosen minimum magnitude for other purposes.

A minimum magnitude of 4.0 was used for the remaining calculations in this report. This is appropriate for large structures and is within the range of events used to derive the attenuation function.

5.1 Previous Studies

The Australian Standard on earthquake loadings (AS1170.4) gives a map showing contours of earthquake hazard described as an acceleration coefficient with a return period of 475 years (10% probability of exceedance in 50 years) (Figure 8). The coefficient was defined to approximate the numerical PGA value in g for the same return period.

Figure 8: AS1170.4 earthquake hazard map for southeast Australia

The Australian Standard has an acceleration coefficient of between 0.04 g and 0.05 g for the Rasp Mine site. The AUS5 PGA value calculated in this study (Figure 9, Table 4 - Appendix A) gives a value of approximately 0.094 g for a return period of 500 years when considering earthquakes of magnitude 4.0 or greater (approximately 0.066 g for magnitude 5.0 and above).

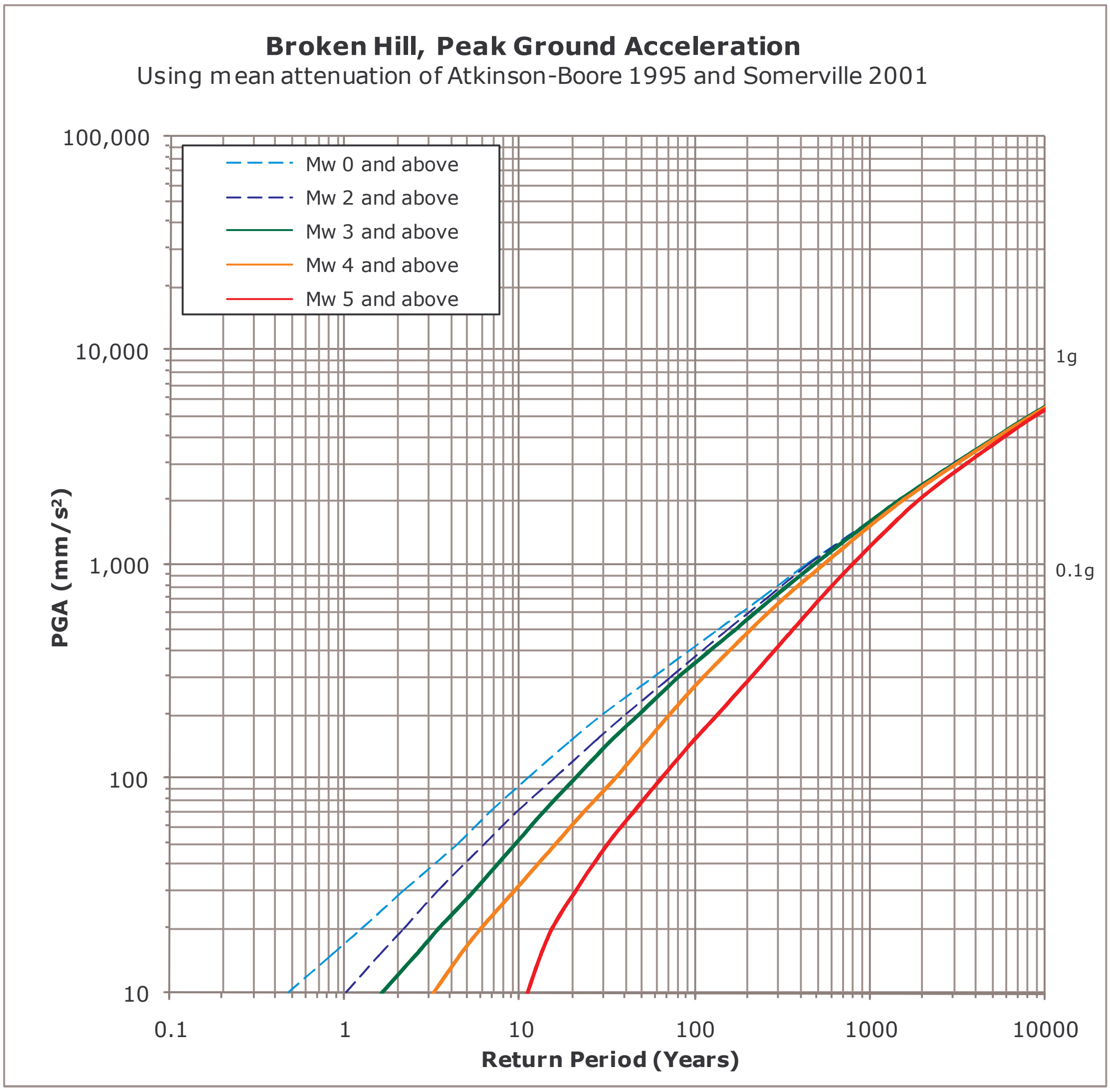

Figure 9: Peak ground acceleration recurrence

5.2 Source Zone Contributions

Figure 10 shows that the PGA source contributions for the Rasp Mine site is mainly dominated by the Broken Hill seismotectonic zone within which the site is located, as well as the Mundi Mundi Fault.

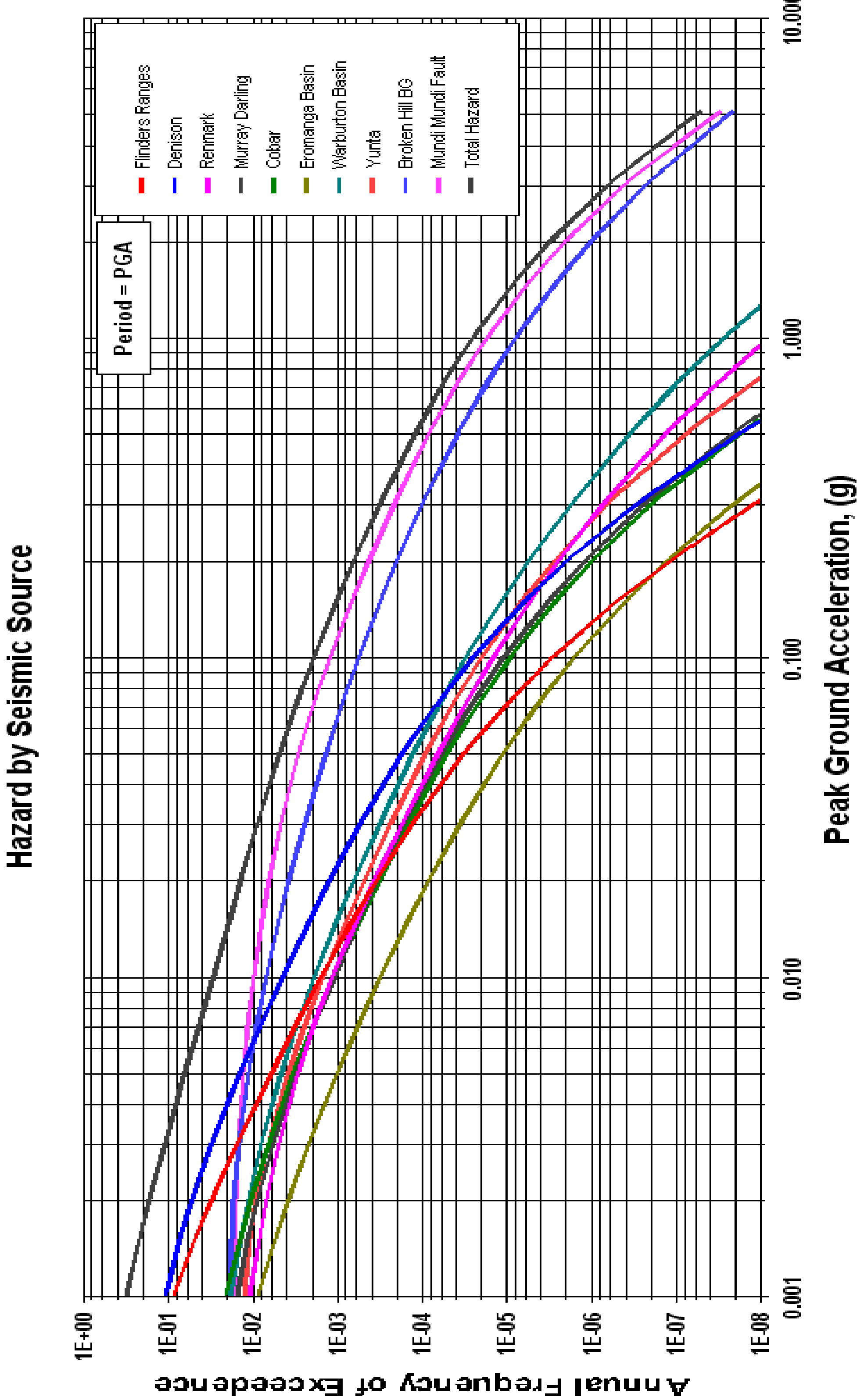

Figure 10: Source contributions for the Rasp Mine site

5.3 Peak Ground Velocities

The peak ground velocity plot (Figure 11) is computed using the Gaull, Michael-Leiba and Rynn, (1990) attenuation, which was derived from western Australian intensity data modified according to an empirical relationship with PGV.

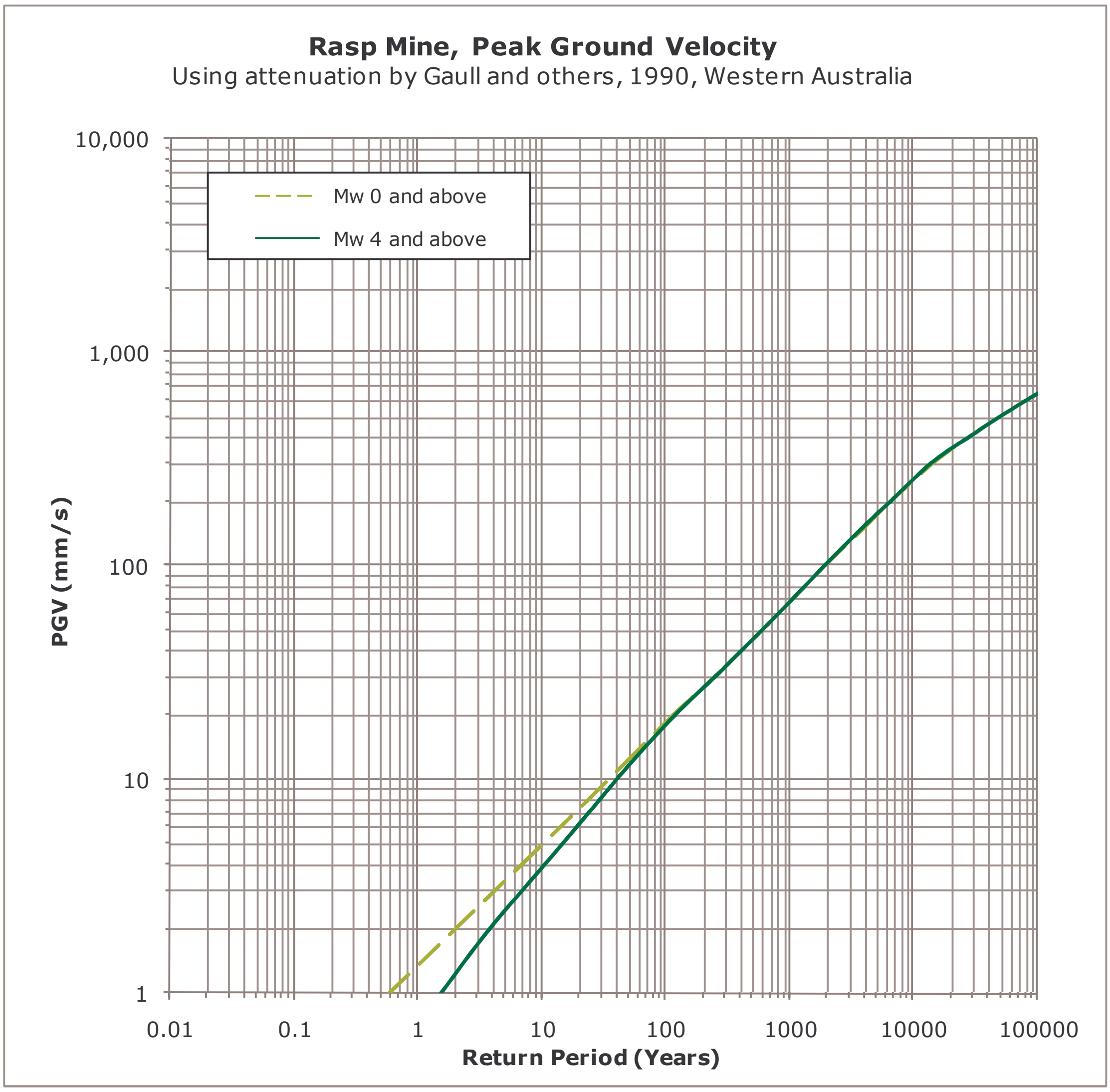

Figure 11: Peak ground velocity recurrence

5.4 Modified Mercalli Intensities

The estimated modified Mercalli intensity plot (Figure 12) has been computed using the Gaull, Michael-Leiba and Rynn, 1990 attenuation function, which was derived from intensity maps of western Australian earthquakes.

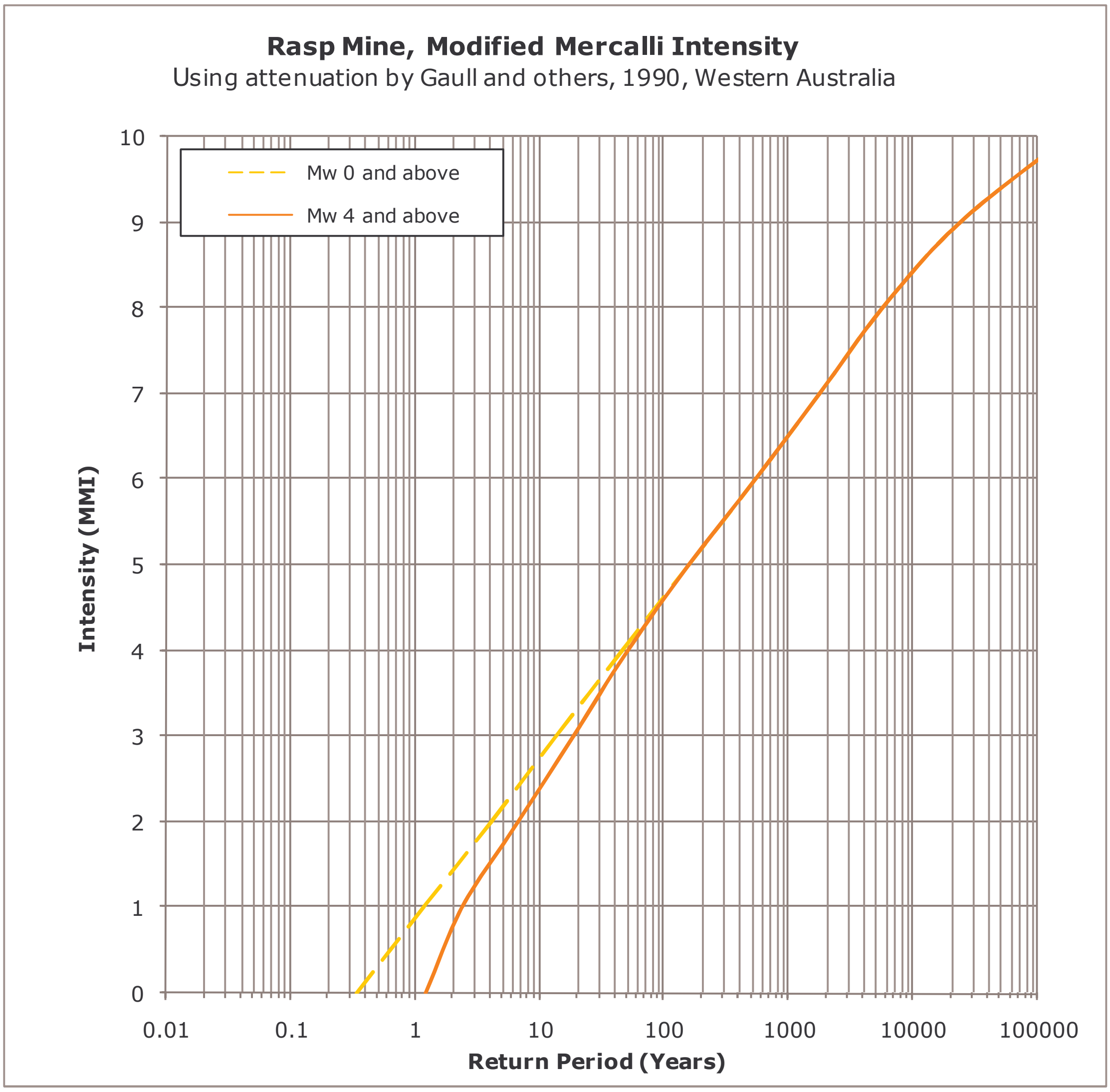

Figure 12: Modified Mercalli intensity recurrence

5.5 Uniform Probability Response Spectra

The uniform probability response spectra have been computed using the mean of Atkinson-Boore (1995) and Somerville (2001) relationships. For magnitudes of 4 and above, return periods of 500, 1000, 5000 and 10000 years have been produced, for 5% damping. These results are presented in Figures 13 to 15. Figures 13 and 14 both plot period (in seconds) versus response acceleration (in mm/s^2), on a linear-linear and a log-linear plot respectively. In Figure 15 the results have been converted to pseudo-velocity (in mm/s) and plotted versus frequency (in Hz) on a tripartite plot, with acceleration (in gravity 'g') and pseudo-displacement (mm) lines also plotted.

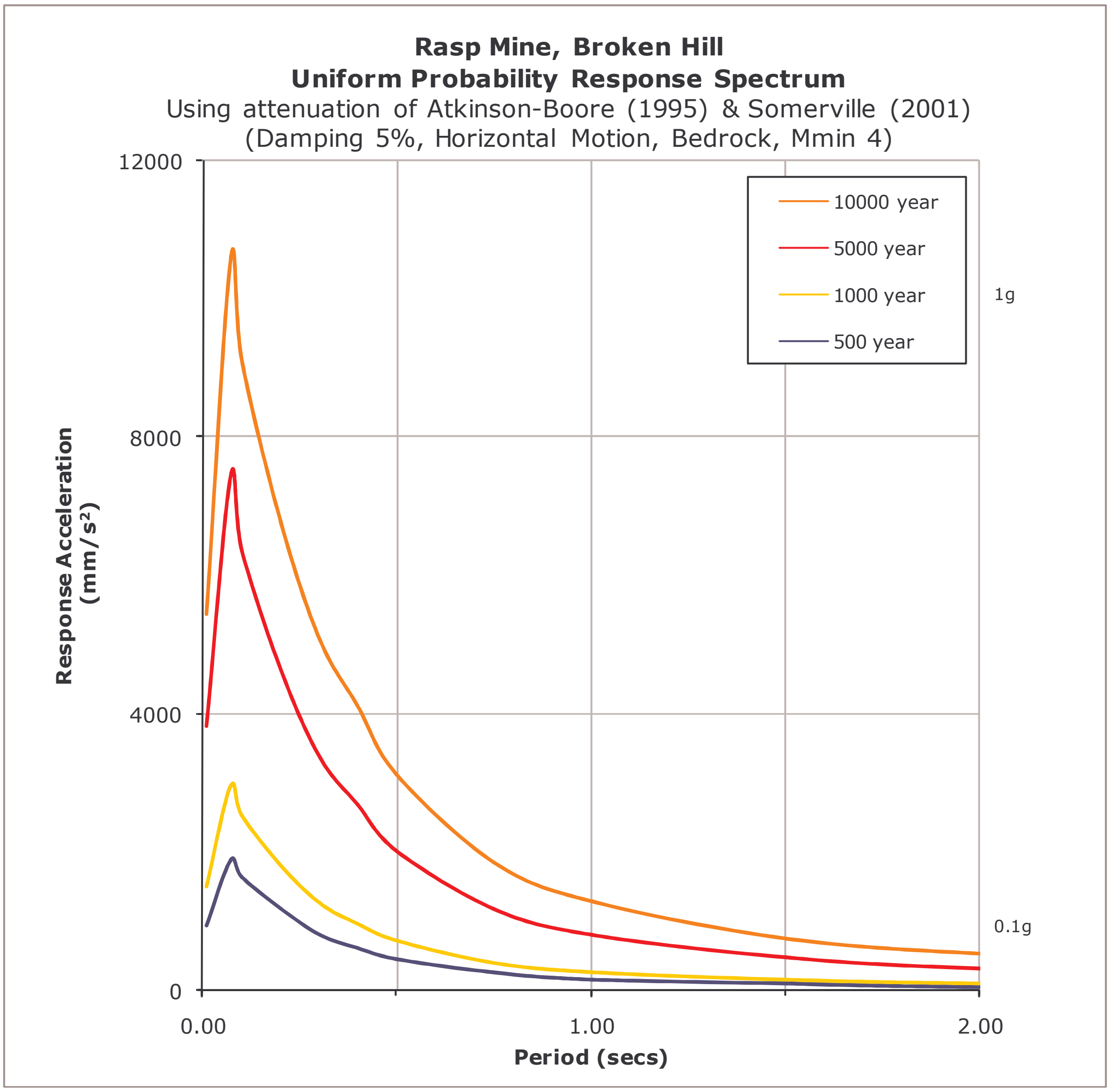

Figure 13: Response spectra acceleration, 5% damping (linear-linear)

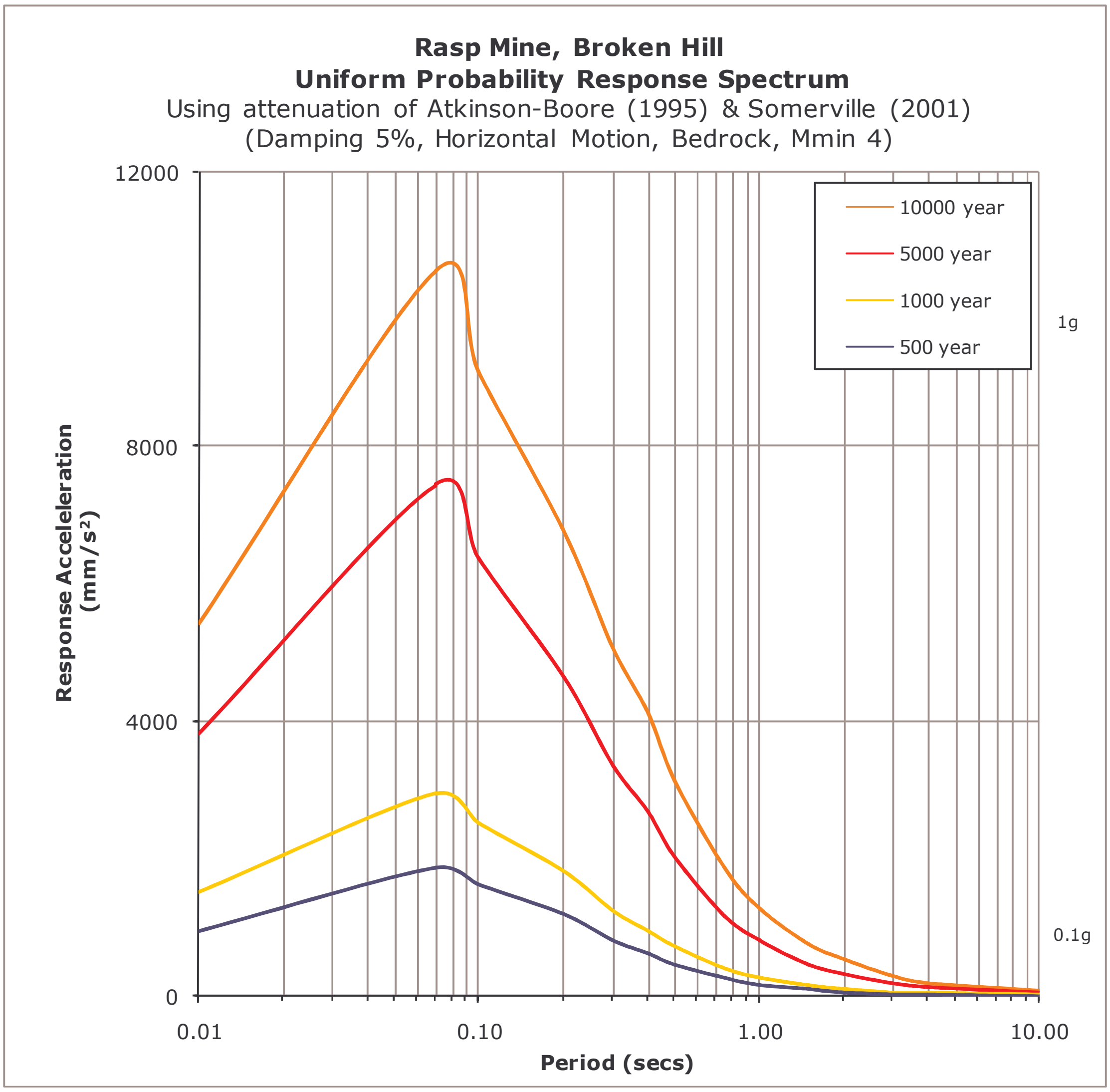

Figure14: Response spectra acceleration, 5% damping (log-linear)

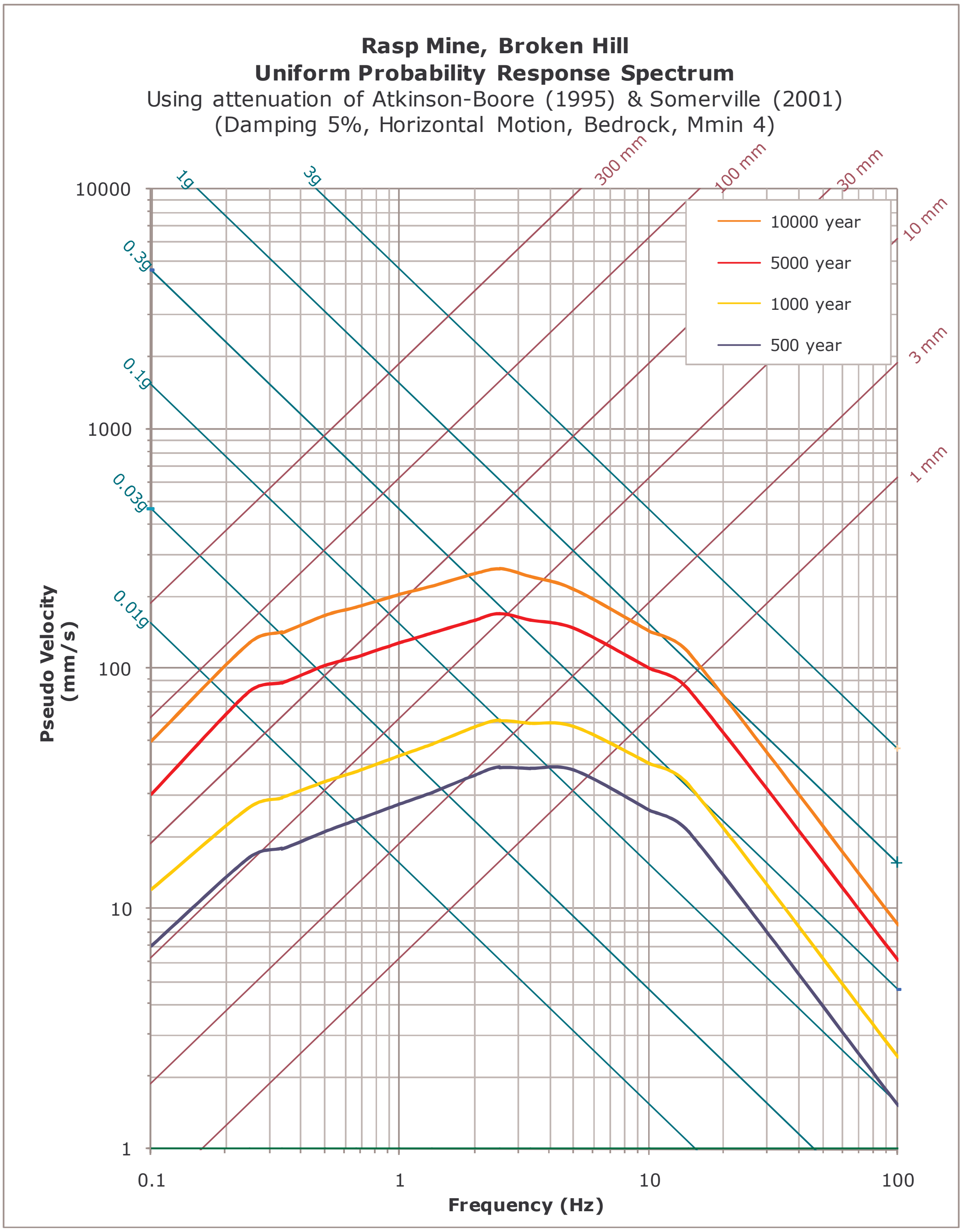

Figure15: Response spectra pseudo-velocity recurrence, 5% damping

5.6 Deaggregation

Deaggregation plots are the result of a probabilistic assessment of ground motion estimates with respect to an infinite set of earthquakes of different magnitudes at all possible locations. The magnitude-distance deaggregation plot shows how the various combinations of magnitude and distance contribute to ground motion at specified return periods and ground motion frequencies.

Figure 19 gives the deaggregation for the Rasp Mine site with a period estimated as being 0.169491 seconds (a natural frequency of approximately 6 Hz) for a return period of 1,000 years (a probability of 0.001 per year).

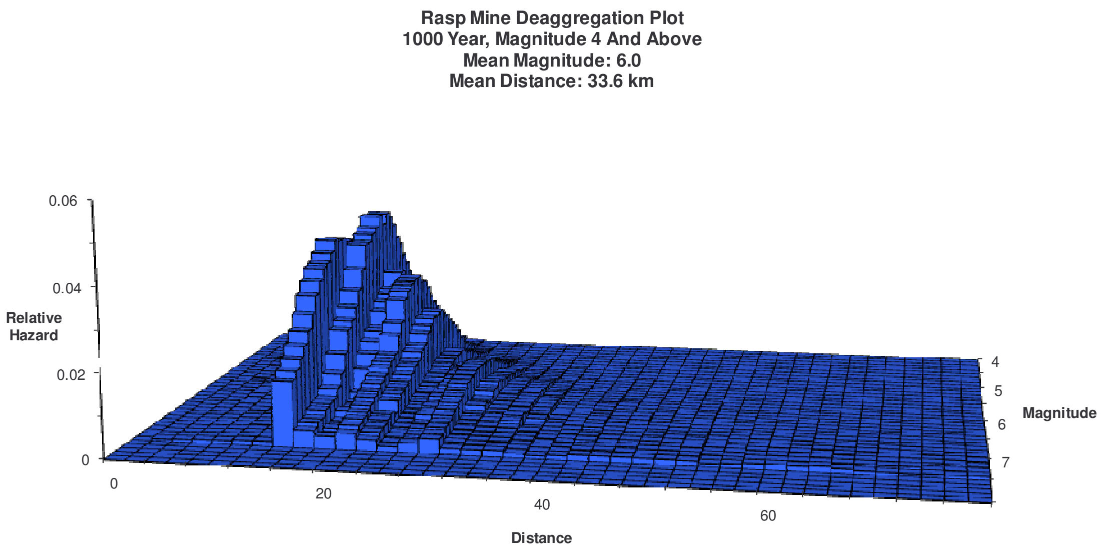

Figure 16: Rasp Mine Deaggregation Plot, 1,000 year motion, 6 Hz

The deaggregation plot shows the typical increase in ground motion due to larger and more nearby earthquakes. The peak in the plot is likely to be contributed to by the active Mundi Mundi fault.

The earthquake hazard at the Rasp Mine site, with a PGA calculated as approximately 0.09 g for a return period of approximately 500 years (a 10% chance of exceedence in 50 years) (approximately 0.06 g for magnitude 5.0 and above), is above average by Australian standards.

The source zone contribution and deaggregation plots show that most of the estimated ground motion at the Rasp Mine site will be due to activity within the Broken Hill seismotectonic zone within which the site is located. The Mundi Mundi Fault source was considered in the seismotectonic model used in this study. To date, no active faults have been identified in the region surrounding the Rasp Mine site, other than the Mundi Mundi Fault. Further investigations including a detailed geological field survey would be required to determine if the faults in the region show any evidence of movement in recent geological time.

Site response has not been considered within this report and the calculations have been made assuming the area is situated on bedrock.

A minimum considered magnitude of 4.0 was used for this study. Increasing the magnitude cut-off will eliminate some high frequency, high acceleration, low displacement and short duration motion from the record. It was assumed that no motion from an earthquake below magnitude 4.0 will have any effect on the structures under consideration. A higher minimum cutoff magnitude would reduce ground motion recurrence, especially high frequency motion and peak ground acceleration.

The maximum magnitude for our ground motion calculations was set at Mw 7.5. This is not constrained by the seismicity data, but has been estimated after consideration of the tectonic setting and magnitudes of other large Australian earthquakes. There are several known faults in Australia that are large enough to produce an earthquake of magnitude Mw 7.5, and it is possible that other such faults exist but are hidden or not yet discovered. In areas of low seismicity, the choice of maximum credible magnitude does not significantly affect ground motion recurrence estimates for return periods up to hundreds of years.

6 Summary

The methodology used for estimating the PGA recurrence in the Australian Standard (AS1170.4) differs from that used in AUS5. Three of the main differences are:

- AUS5 has smaller source zones based on geological boundaries, whereas the original AS1170.4 map was produced by interpolation between few isolated points yielding significant smoothing of the hazard.
- More seismicity data is now available leading to refinement of both A_0 and b-values, and the computational methods have greatly improved over the past thirteen years.
- More appropriate spectral attenuation functions are now available.

We believe that the results presented in this report are in accordance with the best practices of this time.

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Appendix A – Ground Motion Recurrence Tables

PGA (mm/s²)	Mw 0 and above Return Period (Years)	Mw 2 and above Return Period (Years)	Mw 3 and above Return Period (Years)	Mw 4 and above Return Period (Years)	Mw 5 and above Return Period (Years)
10	0.467	0.995	1.59	3.15	10.9
15	0.815	1.55	2.51	4.52	13
20	1.21	2.12	3.46	5.99	15.3
30	2.14	3.34	5.41	9.14	20.3
50	4.3	6.15	9.38	15.8	31.3
70	6.75	9.46	13.5	22.9	43.4
100	10.9	15.2	20.1	33.8	62.6
150	19.4	26.8	32.7	52.3	97
200	30	40.3	47.1	71.4	133
300	57.3	71.9	81.3	112	207
500	134	152	168	207	361
700	236	253	276	322	521
1000	434	447	476	531	783
1500	871	880	912	982	1290
2000	1440	1440	1470	1560	1910
3000	3020	3020	3040	3130	3540
5000	8240	8240	8250	8330	8810
7000	17100	17100	17100	17100	17700
10000	41000	41000	41000	41000	41400
15000	130000	130000	130000	130000	130000
20000	340000	340000	340000	339000	337000
30000	1650000	1650000	1650000	1650000	1630000
50000	18600000	18600000	18600000	18500000	18300000

Table 4: Rasp Mine site PGA Recurrence results

Using mean attenuation by Atkinson-Boore (1995) & Somerville (2001)

	Response Spectra (mm/s²), 5% Damping			
Period (secs)	500 Year	1000 Year	5000 Year	10000 Year
0.01	956.87	1516.06	3827.88	5442.92
0.07	1884.54	2953.72	7448.98	10574.20
0.10	1631.70	2522.52	6367.06	9106.16
0.20	1187.76	1817.90	4633.44	6769.84
0.30	806.83	1254.40	3371.20	5073.46
0.40	611.52	961.97	2683.24	4114.04
0.50	457.37	720.59	2028.60	3136.98
0.75	258.03	409.35	1178.94	1860.04
1.00	171.89	274.20	805.56	1283.80
1.50	98.10	157.78	473.54	763.22
2.00	65.92	106.72	326.14	530.38
3.00	37.48	60.88	184.93	298.21
4.00	25.74	41.90	126.91	202.96

Table 5: Rasp Mine site Response Spectra results, 5% Damping

(M4 and above, bedrock)
Using mean attenuation by Atkinson-Boore (1995) & Somerville (2001)

PGV (mm/s)	Mw 0 and above Return Period (Years)	Mw 4 and above Return Period (Years)
1	0.575	1.520
2	1.980	3.700
4	6.73	10.50
8	22.60	28.60
12	46.2	51.7
20	114.0	118.0
32	267.0	269.0
60	816	814
100	1930	1930
150	3830	3820
200	6330	6310
300	13800	13800
400	26500	26500
600	81900	81600
900	292000	291000
1200	713000	711000
1500	1540000	1530000
2000	5520000	5500000
3000	76600000	76400000
6000	133000000000	132000000000

Table 6: Rasp Mine site PGV Recurrence results

Using attenuation by Gaull, Michael-Leiba and Rynn (1990)

Intensity (MMI)	Mw 0 and above Return Period (Years)	Mw 4 and above Return Period (Years)
0	0.336	1.180
1	1.160	2.400
2	3.98	6.74
3	13.40	18.70
4	45.3	50.6
5	156	158
6	540	540
7	1760	1760
8	5700	5680
9	23100	23000
10	175000	174000
11	1620000	1610000
12	120000000	120000000

Table 7: Rasp Mine site MMI Recurrence results

Using attenuation by Gaull, Michael-Leiba and Rynn (1990)

Appendix B – Modified Mercalli Intensity

The effects are those of large earthquakes. The higher frequencies of seismic waves from smaller nearby events yield different effects, and in particular are more likely to be heard rather than felt at low intensity.

1	Not felt, except under especially favourable circumstances.
2	Felt by persons at rest, on upper floors, or favourable places.
3	Felt indoors. Hanging objects swing. Vibrations like a passing light truck. Duration estimated. May not be recognised as an earthquake.
4	Vibration like a passing heavy truck. Sensation like an object striking walls. Windows, dishes and doors rattle, crockery clashes. Standing cars rock. In upper ranges, wood walls and frames creak.
5	Felt outdoors, direction estimated. Sleepers wakened. Small unstable objects displaced or upset. Doors swing closed or open. Pictures move. Liquids disturbed, some spilled. Some cracked plaster.
6	Felt by all. Many frightened and run outdoors. People walk unsteadily. Windows, dishes, glassware broken. Small items fall from shelves. Pictures off walls, furniture moved or overturned. Weak plaster and masonry D cracked. Trees shaken visibly.
7	Difficult to stand. Noticed by car drivers. Furniture broken. Damage to masonry D, some cracks in masonry C. Waves on water. Small slides and caving in along sand and gravel banks.
8	Partial collapse of masonry C, damage to masonry B, none to masonry A. Car steering affected. Twisting or fall of chimneys, monuments, towers and tanks. Frame houses moved if not bolted down. Tree branches broken. Cracks in wet ground and on slopes.
9	General panic. Masonry D destroyed, masonry C heavily damaged, masonry B seriously damaged. General damage to foundations. Frames cracked. Underground pipes broken.
10	Most masonry and frame structures destroyed with their foundations. Serious damage to dams. Large landslides. Rails bent slightly.
11	Rails bent greatly. All underground pipes destroyed.
12	Near total damage. Objects thrown into the air.

Masonry A	Good workmanship, mortar and design; reinforced or bound; Designed to resist lateral forces.
Masonry B	Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.
Masonry C	Ordinary workmanship and mortar; no extreme weaknesses, but neither reinforcement or design against lateral force.
Masonry D	Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally.

Appendix C – Glossary

Accelerograph
A seismic recording instrument that records the acceleration of ground motion (compare with a seismograph).

Aftershock
Smaller earthquakes located in the same volume within the earth as a major or moderate earthquake, and occurring in the following days or weeks (sometimes longer). Shallow mainshocks often have many aftershocks, while deep earthquakes have few. Aftershocks in regions that are normally inactive can last for months or years.

Attenuation
The reduction of seismic wave amplitude with distance from an earthquake. This is due to geometric spreading, absorption of energy within the rock, and scattering of seismic waves.

Attenuation Function
A function describing the attenuation of seismic waves, usually giving ground motion as a function of earthquake magnitude, distance and depth, and sometimes including other earthquake source, travel path or surface site parameters. Spectral attenuation functions also give the variation of ground motion with frequency.

Depth (of an earthquake)
The distance from the earthquake hypocentre to its epicentre, usually measured in kilometres. Note that the hypocentre of a large earthquake that produces a surface rupture is usually a number of kilometres beneath the surface.

Epicentral Distance
The distance from the earthquake epicentre around the surface of the earth to the point concerned.

Epicentre
The point on the earth's surface vertically above the hypocentre.

Frequency (of seismic waves)
The number of cycles per second (hertz) of a seismic waveform.

Hypocentre
The point within the earth at which the earthquake rupture initiated.

Intensity
A measure used to indicate the effect of ground motion at a point. A number of intensity scales such as the modified Mercalli and Rossi-Forel have been defined (see Appendix A).

Liquefaction
Temporary loss of shear strength due to an increase in pore pressure during strong ground motion, particularly hazardous with unconsolidated saturated silty sands.

Magnitude
A number indicating the "size" of an earthquake. It is closely related to the amount of energy released during the rupture, or to the rate at which energy is released. There are a number of magnitude scales in use, each measured in a different way. If the word magnitude is used without qualification, in the past it usually referred to the Richter magnitude ML, but is now usually the moment magnitude Mw. The ML, Ms and Mw scales give similar numerical values.

mb
Body wave magnitude used for large distant earthquakes, especially deep earthquakes. In Australia, magnitude mb often does not correlate well with the other magnitude scales.

ML
Richter local magnitude is measure of the size of the earthquakes used for smaller nearby earthquakes, usually within 600 km. defined by Richter in 1935 as "the logarithm of the calculated trace amplitude, expressed in microns with which the standard short period torsion seismometer … would register the shock at an epicentral distance of 100 km". Usually can only be used for events up to about magnitude ML 5.5 to 6.0.

MP
Perceptibility magnitude, based on the radius over which an earthquake is felt. Recent earthquakes are used to calibrate the scale, usually to ML, so MP is numerically equivalent to ML. Useful for estimating magnitudes of historical earthquakes. Sometimes denoted by ML(I).

Ms
Surface wave magnitude, used for large distant shallow earthquakes. Usually can only be used for events of magnitude Ms 6.0 or larger.

Mw
Moment magnitude, used for larger earthquakes recorded by digital seismographs. This is probably the most reliable magnitude code, and it is hoped that it will be applied to smaller earthquakes in the near future.

Modified Mercalli Intensity
An intensity scale defined by Mercalli and later modified (see Appendix A).

Response Spectrum
A spectrum indicating the peak response of a damped simple harmonic oscillator to a specified ground motion. The ordinate plotted may be peak acceleration, velocity or displacement response.

Richter (local) Magnitude, ML
See ML above.

Seismogram
The record produced by a seismograph, either an analogue wiggly line on paper or film, or a digital computer record. An accelerogram is a seismogram where the measure of motion is acceleration.

Seismograph
A seismic recorder. This includes both the detector and the recording system. Precision timing on modern seismographs is often provided by GPS satellites. Historically, a seismograph was used by seismologists and recorded either the displacement or velocity of ground motion and an accelerograph was used by engineers and recorded the acceleration of ground motion. This distinction is becoming obsolete and the term seismograph is used for all types of seismic recorders.

SRC

The Seismology Research Centre, originally established at Preston Institute of Technology in Melbourne during 1976. PIT was renamed Phillip Institute of Technology in 1982, and was amalgamated into RMIT university in 1992. In 1998 the centre became a commercial group of Mindata Australia, and in 2002 became a division of Environmental Systems and Services Pty Ltd.

Tsunami

Seismic ocean wave produced by a vertical offset in the ocean floor during a large earthquake, by an undersea landslide perhaps triggered by an earthquake, or rarely by a comet or asteroid impact. Crosses oceans at high speed (up to hundreds of kilometres per hour, depending on water depth) and low amplitude, slowing and building in amplitude when approaching a coast. May cause severe damage along coast lines.

Universal Time

To avoid confusion associated with different time zones in different states, and with daylight saving time, all seismologists use Universal Time. This is almost identical with Greenwich Mean Time. In eastern Australia, UT is ten hours behind Eastern Standard Time, or eleven hours behind Eastern Daylight Saving Time.

Appendix D – Geological Time Scale

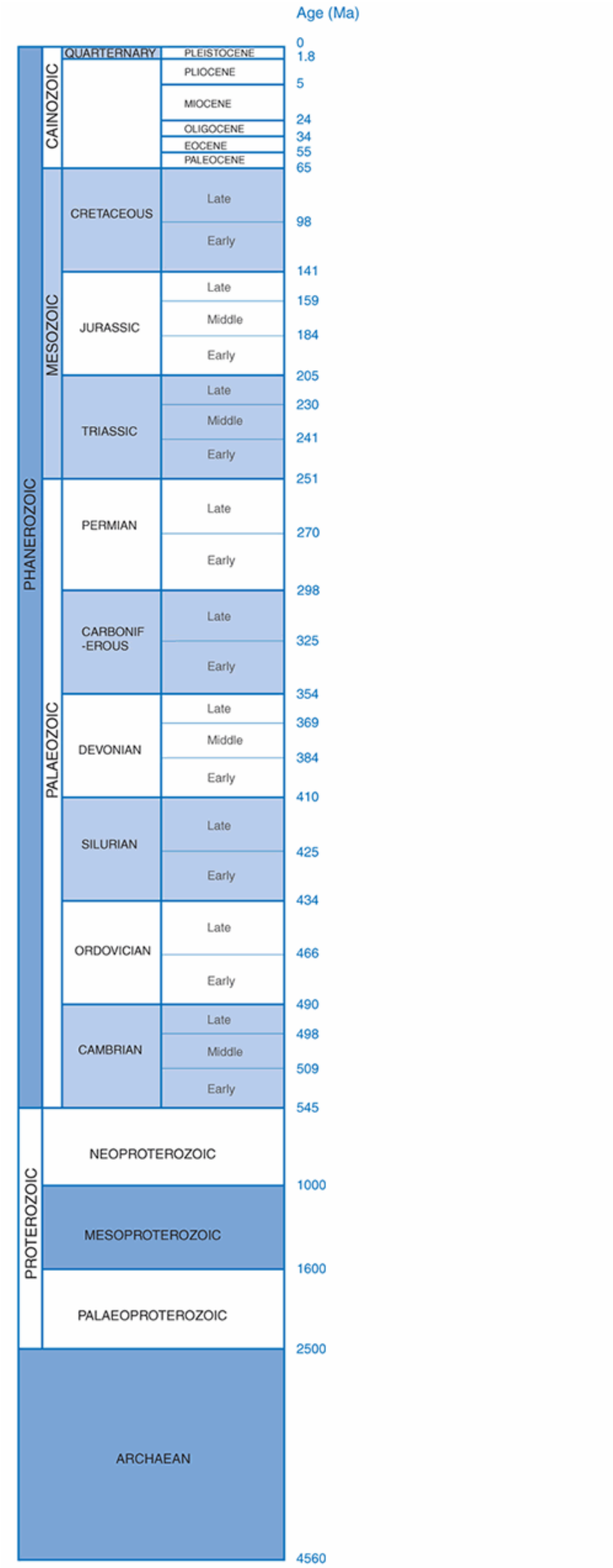

Figure 17: Geological Time Scale (courtesy of Australian Museum)

APPENDIX D
Limitations

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