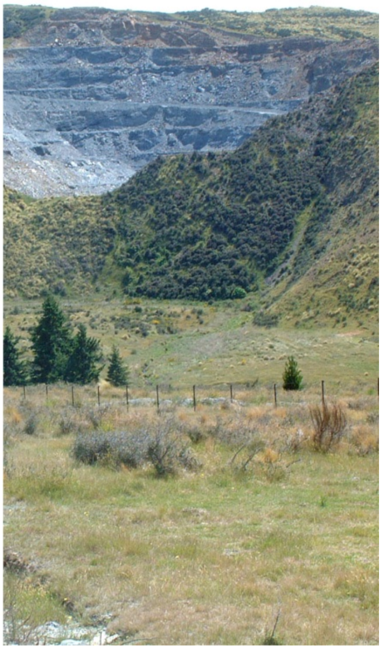
April 2011

MACRAES PHASE III PROJECT

Top Tipperary Tailings Storage Facility Hydrogeological Assessment

Submitted to:
Oceana Gold (New Zealand) Limited



REPORT

Report Number. 0978110-562 R004 vC

Distribution:

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Executive Summary

Oceana Gold (New Zealand) Limited (OceanaGold) operates the Macraes Gold Project (MGP) located in central Otago, approximately 25 km west of Palmerston. The MGP consists of a series of opencast pits and an underground mine supported by ore processing facilities, waste storage areas and water management systems.

OceanaGold operates an ongoing program of exploration, ore reserves review and mine design optimisation. Consequently, operational pit designs are regularly updated. The performance of existing tailings storage facilities and the requirement for additional tailings storage capacity is also regularly reviewed. As a result of recent reviews, OceanaGold have determined that additional tailings storage capacity is necessary to support the Macraes Phase III proposed expansion of mining operations at the site.

OceanaGold needs capacity to store an additional 43.5 Mt of tailings to take the MGP from mid 2012 through until early 2020 at current processing rates. OceanaGold is therefore seeking to obtain resource consents for the construction and operation of a new tailings storage facility at the MGP. Golder Associates (NZ) Limited (Golder) has been contracted by OceanaGold to undertake an evaluation of potential tailings water discharge from the proposed Top Tipperary Tailings Storage Facility (TTTSF). The outcomes of this evaluation are to be used in support of a resource consent application for the construction and operation of the TTTSF.

The proposed TTTSF is located in the headwaters of Tipperary Creek outside of the catchments of the North Branch of the Waikouaiti River (NBWR) and Deepdell Creek. The final TTTSF footprint is to be approximately 184 ha, with a tailings storage capacity of 36.7 Mm^3. The design embankment crest height is 560 mRL and the operating height would be 70 m at the highest embankment point.

It is proposed to pump tailings to the TTTSF via a pipeline from the processing plant and deposit the tailings from the TTTSF embankment. An initial embankment is to be constructed to hold two to three years capacity of tailings. Subsequent lifts (in a downstream manner) would be added to the main embankment, eventually incorporating “wing” embankments along the northern and southern flanks. The footprint of the TTTSF would therefore gradually expand, reaching its maximum extent by about 2017.

It is planned to decommission the existing SP11 TSF in 2012. Tailings excavated from SP11 are to be mechanically re-handled once dry enough and placed as a reclaimed tailings stack on the existing Mixed Tailings Impoundment with potentially some also being placed in the TTTSF.

The rehabilitation plan at closure calls for the facility to be fully capped with brown rock and topsoil and for pasture to be re-established.

Two proposed waste rock stacks (WRS’s) are planned to be constructed adjacent to the upper boundary of the Tipperary Creek catchment.

- The existing Back Road WRS is to be expanded (additional capacity of 228 Mt) and will be wholly contained within the present Deepdell catchment. Construction is to start early in 2013 and would continue until 2019.
- The Frasers East WRS, which is currently under construction, is to be expanded to include a northern addition to the current consented WRS (additional capacity of 26 Mt). Construction of the Frasers East WRS including the northern extension is expected to be completed in early 2013.

The construction of the two WRS’s described above has the potential to affect the location of the existing groundwater divides separating the Tipperary catchment from the Deepdell Creek and North Branch Waikouiti River catchments. Accumulation of groundwater within the WSR’s could result in the Tipperary Creek groundwater catchment expanding slightly at the expense of the above two catchments. In addition, leachate from the Frasers East WSR is expected to be transported through the groundwater system to Tipperary Creek. Consequently, these WSR's have been taken into account in assessing groundwater flows and contaminant transport within the Tipperary Creek catchment.

A 3D groundwater flow and contaminant transport model has been developed to simulate seepage flows within the catchment and the potential transport of contaminants lost from the TTTSF. The model is based on similar models of the MGP site groundwater system developed by Kingett Mitchell Ltd (Kingett Mitchell) to support past resource consent applications. The model simulates an eight year TSF operational period followed by a 150 year post-closure period.

The effects of contaminant loss from the TTTSF have been evaluated with respect to:

- The drainage systems built into the TTTSF, including underdrains installed beneath the tailings and chimney drains constructed in the embankment.
- Tipperary Creek and tributary gullies, including both reaches managed through the use of sediment settling ponds and unmanaged sections.
- Cranky Jims Creek and tributary gullies, including both reaches managed through the use of sediment settling ponds and unmanaged sections.

Transport of each of the simulated contaminants with the exception of arsenic has been undertaken on the basis of conservative transport within the groundwater system. Arsenic transport has been modelled based on arsenic (III) being the main form of this element in the tailings seepage water. The adsorption parameters for arsenic (III) have been derived from testing of rock and soil samples from the site. The results of the tests have been applied to the contaminant transport model in the form of:

- A distribution coefficient (K_d) of 20 L/kg, applicable to the model zones simulating weathered schist and loess and the underlying moderately to slightly weathered schist.
- Maximum arsenic adsorption of 230 mg per kilogram of rock and soil for the upper 20 m of the weathered schist and loess.
- Maximum arsenic adsorption of 46 mg per kilogram of rock for the moderately to slightly weathered schist between approximately 20 m and 60 m below the ground surface.
- Conservative transport of arsenic at depths below 60 m.

The groundwater modelling of seepage and drain discharge flows potentially carrying leachate water from the TTTSF to natural receiving water channels is summarised below. The modelled decrease in seepage flows over time is a result of the cessation of tailings slurry deposition in the TTTSF and the consequent dewatering of the tailings mass.

Flow component	**Short term flow (m^3/day)**	**Long term flow (m^3/day)**
TTTSF drainage systems	1,800	260
Tipperary Creek main channel	46	30
Tipperary Creek western tributary	16	7
Cranky Jims Creek	85	55

The main contaminants of interest derived from the stored tailings are considered to be arsenic, cyanide and sulphate. The mass loads of these contaminants transported in TSF drainage systems and groundwater seepage to surrounding receiving waters are summarised below.

The operational and long term contaminant mass loads transported by the discharge water from the TTTSF drainage systems are calculated to be:

Contaminant	Operational period mass load (kg/day)	Long term mass load (kg/day)
Sulphate	5,000	2,260
Arsenic	10	0.3
$\text{Cyanide}_{\text{WAD}}$	0.6	0.09

The operational and long term contaminant mass loads transported by groundwater seepage to the main channel of Tipperary Creek are calculated to be:

Contaminant	Operational period mass load (kg/day)	Long term mass load (kg/day)
Sulphate	19	16
Arsenic	<0.001	<0.001
$\text{Cyanide}_{\text{WAD}}$	<0.001	<0.001

The operational and long term contaminant mass loads transported by groundwater seepage to the western tributary of Tipperary Creek are calculated to be:

Contaminant	Operational period mass load (kg/day)	Long term mass load (kg/day)
Sulphate	8.6	3.5
Arsenic	<0.001	<0.001
$\text{Cyanide}_{\text{WAD}}$	<0.001	<0.001

The operational and long term sulphate mass load transported by groundwater seepage to Cranky Jims Creek is calculated to be approximately 32 kg/day. The contaminant concentrations in groundwater discharging to Cranky Jims Creek are calculated to be <0.001 g/m^3 for arsenic and 0.05 g/m^3 for $\text{cyanide}_{\text{WAD}}$.

A surface water compliance point is proposed to be established at TC01, downstream from all simulated contaminant discharge points to Tipperary Creek. The outcomes of the groundwater flow and contaminant transport modelling indicate that the contaminant concentrations in groundwater discharges to Tipperary Creek, excluding the TSF drain discharges, are below the proposed surface water quality limits applicable at TC01. Dilution of the groundwater discharges through surface run-off would reduce the peak simulated concentrations further.

The contaminant concentrations in TTTSF drain discharge water are calculated to exceed the proposed TC01 compliance limits for arsenic, sulphate, $\text{cyanide}_{\text{WAD}}$, copper, iron and lead. These discharges are to be managed through the application of mitigation measures as discussed in other reports.

A surface water compliance point is proposed to be established at CJ01, downstream from all simulated contaminant discharge points to Cranky Jims Creek. The outcomes of the groundwater flow and contaminant transport modelling indicate that the contaminant concentrations in groundwater discharges to Cranky Jims Creek are below the proposed surface water quality limits applicable at CJ01.

ABBREVIATIONS

CTI	Concentrate Tailings Impoundment
EGL	Engineering Geology Limited
FTI	Flotation Tailings Impoundment
K_d	Distribution coefficient
MGP	Macraes Gold Project
MTG	Maori Tommy Gully
MTI	Mixed Tailings Impoundment
SPI	Southern Pit Tailings Impoundment
SP10	Southern Pit Tailings Impoundment SP10 currently incorporated in SP11
SP11	Southern Pit Tailings Impoundment SP11
TSF	Tailings storage facility
TTTSF	Top Tipperary Tailings Storage Facility
WRS	Waste rock stack

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1.0 INTRODUCTION

1.1 Background

Oceana Gold (New Zealand) Limited (OceanaGold) operates the Macraes Gold Project (MGP) located in east Otago, approximately 25 km west of Palmerston (Figure 1). The MGP consists of a series of opencast pits and an underground mine supported by ore processing facilities, waste storage areas and water management systems (Figure 1).

OceanaGold operates an ongoing program of exploration, ore reserves review and mine design optimisation. Consequently, operational pit designs are regularly updated. The performance of existing tailings storage facilities and the requirement for additional tailings storage capacity is also regularly reviewed.

As a result of recent reviews, OceanaGold have determined that additional tailings storage capacity is necessary to support the Macraes Phase III proposed expansion of mining operations at the site. OceanaGold is planning to decommission both of the current tailings storage facilities (TSF's) by mid 2012 and commence using a new TSF. At this point, it is likely that both existing TSF's will have remaining resource consent life, however in review of OceanaGold's new mining schedule it is a more economic alternative to switch to the new facility.

A major component of Macraes Phase III involves the requirement for a new TSF. A number of options have been investigated with the selected option being located in the very upper reaches of the Top Tipperary catchment (Figure 2) hence, it is called the TTTSF.

OceanaGold is seeking to obtain resource consents for the construction and operation of the TTTSF. Golder Associates (NZ) Limited (Golder) has been retained by OceanaGold to undertake an evaluation of tailings water seepage and contaminant losses from the TTTSF. This report[1] presents the results of studies undertaken to assess groundwater flow and contaminant mass transport budgets for the TTTSF. The outcomes of this evaluation are to be used in support of an Assessment of Environmental Effects.

1.2 Project Description

OceanaGold needs capacity to store an additional 43.5 Mt of tailings to take the MGP from mid 2012 through until early 2020 at current processing rates. The final TTTSF footprint is 184 ha and the tailings storage capacity will be 36.7 Mm^3. The embankment crest height will be 560 mRL and the operating height will be 70 m at the highest embankment point.

The TTTSF will be located in the headwaters of Tipperary Creek outside of the catchments of the North Branch of the Waikouaiti River (NBWR) and Deepdell Creek. The main embankment will straddle the Macraes-Dunback Road and as such a road diversion will be required prior to commencement of construction of the TTTSF.

It is proposed to pump tailings to the TTTSF via a pipeline from the processing plant and deposit the tailings sub-aerially from the TTTSF embankment. An initial embankment will be constructed to hold two to three years capacity of tailings. Subsequent lifts (in a downstream manner) will be made to the main embankment, eventually incorporating "wing" embankments along the northern and southern flanks. The footprint of the TTTSF will therefore gradually expand, reaching its maximum extent by about 2017.

Tailings material from the SP11 TSF, which is to be de-commissioned in 2012, will be mechanically re-handled once dry enough placed as a reclaimed tailings stack (RTS) on the Mixed Tailings Impoundment (MTI) with some being placed as a RTS in the TTTSF. These re-handled tailings will be stacked at a 1 m vertical to 8 m horizontal (1:8) slope if being deposited on upstream embankment construction or 1:5 if being deposited on downstream embankment construction.

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[1] This report is provided subject to the conditions and limitations presented in Appendix A.

1. Aerial: Oceana Gold, Copyright Reserved.
2. Based on concept provided by Oceana Gold.
3. Schematic only, not to be interpreted as an engineering design or construction drawing.

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TITLE SITE LAYOUT

MARCH 2011

PROJECT 0978110562

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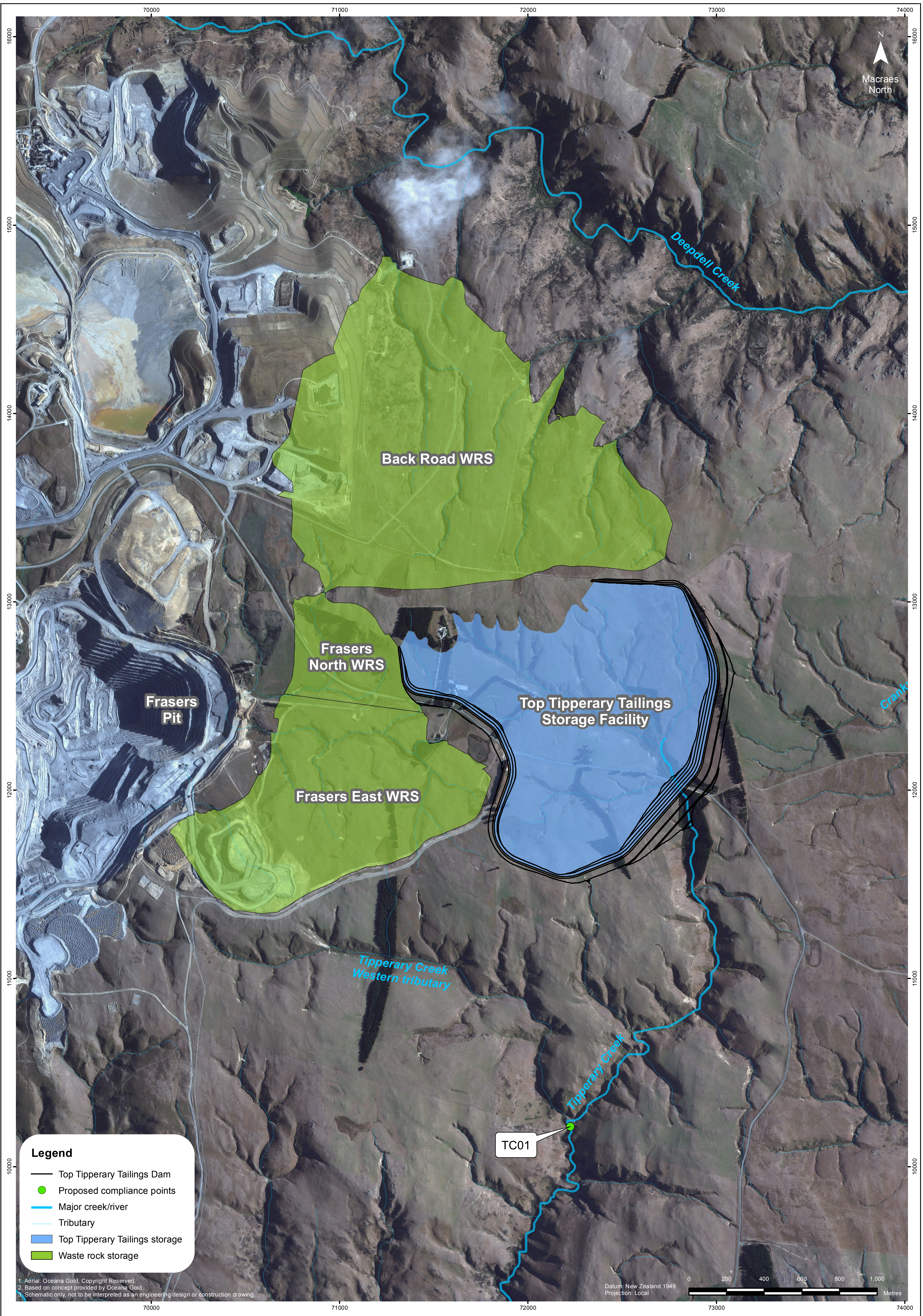

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TITLE TOP TIPPERARY TAILINGS STORAGE FACILITY LAYOUT

APRIL 2011

PROJECT 0978110562

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The required steps and sequence of construction will be as follows:

1) Macraes-Dunback Road diversion earthworks starting from April 2011 with final alignment and public usage by December 2011.
2) Top-soil stripping and embankment footprint preparation (Sept 2011 to Nov 2011).
3) Construction of the main embankment (starter wall to hold 3 years tailings), late 2011 until April 2012. Tailings deposition is to begin during the second quarter of 2012.
4) Continued downstream lifts of the main embankment to reach a final maximum height of approximately 70 m at the highest point.
5) Closure of the TTTSF in 2020.
6) Post closure capping and instigation of water management measures suitable to ensure long-term compliance with consent conditions.

The rehabilitation plan at closure calls for the facility to be fully capped with brown rock and topsoil and for pasture to be re-established.

Two proposed waste rock stacks (WRS's) are planned to be constructed adjacent to the upper boundary of the Tipperary Creek catchment.

- The existing Back Road WRS is to be expanded (additional capacity of 228 Mt) and will be wholly contained within the present Deepdell catchment. Construction is to start early in 2013 and would continue until 2019.
- The Frasers East WRS, which is currently under construction, is to be expanded to include a northern addition to the current consented WRS (additional capacity of 26 Mt). Construction of the Frasers East WRS including the northern extension is expected to be completed in early 2013.

The construction of the two WRS's described above has the potential to affect the location of the existing groundwater divides separating the Tipperary catchment from the Deepdell Creek and North Branch Waikouiti River catchments. Accumulation of groundwater within the WSR's could result in the Tipperary Creek groundwater catchment expanding slightly at the expense of the above two catchments. In addition, leachate from the Frasers East WSR is expected to be transported through the groundwater system to Tipperary Creek. Consequently, these WSR's have been taken into account in assessing groundwater flows and contaminant transport within the Tipperary Creek catchment.

1.3 Scope of Work

The scope of work for this study is to:

1) Assess the potential rate of tailings leachate loss from the TTTSF, both through drainage systems installed in the TSF and through the underlying soils and rock.
2) Evaluate potential contaminant mass loads discharging from the TTTSF and the adjacent WRS's to Tipperary Creek and surrounding surface water bodies.

The above objectives have been fulfilled through the development of a 3D groundwater flow and contaminant transport model to simulate seepage flows across the site and the potential transport of contaminants lost from the TTTSF. The model is based on similar models of the MGP site groundwater system developed by Kingett Mitchell Ltd (Kingett Mitchell) to support past resource consent applications. The model simulates an eight year TSF operational period followed by a 150 year post-closure period.

1.4 Previous Studies

A range of groundwater evaluations at the MGP site have been undertaken in support of mining feasibility studies and subsequently in support of applications for resource consents to authorise various operations at the site. These past studies have been summarised in a report by Kingett Mitchell (2005a).

Prior to 2002, assessments of groundwater mass transport budgets at the mine site concentrated on specific questions and the requirements of individual consents for the mine site. Increasing mine site complexity, expanding tailings storage requirements and concerns about the cumulative contaminant loading to regional drainage systems led to a shift from analytical groundwater flow path calculations to digital modelling of groundwater movement across the entire site. As the MGP has increased in area, digital groundwater simulations have become progressively larger and more complex (Kingett Mitchell 2002, 2005a).

The outcomes from groundwater and mass transport models developed by Kingett Mitchell for the MGP site have been used in support of several applications by OceanaGold for resource consents since 2002. The model outcomes have been accepted by the Otago Regional Council. Although it was possible to extend the most recent site-wide groundwater model (Kingett Mitchell 2005a) to include the TTTSF facility, it was considered more efficient to simulate details of the TTTSF embankment in a separate model. A new model focused specifically on the Tipperary Creek catchment was therefore developed.

Previous assessments have also been undertaken of the groundwater system at Macraes as they related to the construction of the Frasers East WRS (Kingett Mitchell 2005b) and the Frasers Underground Mine (Kingett Mitchell 2006). Both of these assessments covered areas that intersect or are adjacent to the TTTSF. The underground mine has not been taken into account in the groundwater modelling for this project as it is not expected that the mine would have a significant effect on leachate losses from the TTTSF. The assessment work completed in 2005 for the Frasers East WRS is superseded with respect to outcomes for the Tipperary catchment by the current study.

The modelling undertaken during previous studies has been taken into account in the current study. In particular, the structure of the groundwater model and the values applied to input parameters have been derived from these previous studies. Where additional calibration of hydrogeological parameters has been undertaken, the process is discussed in the appropriate sections.

1.5 Report Structure

In addition to the introductory Section 1, this report contains the following sections:

- Section 2 summarises the hydrogeology of the site.
- Section 3 summarises the quality of groundwater, natural surface water, tailings decant and pore water and waste rock seepage water at the site. In addition, the attenuation of contaminants during transport within the groundwater system is discussed in this section.
- Section 4 summarises the conceptual groundwater model of the site.
- Section 5 summarises the translation of the conceptual model into a numerical model. In addition this section outlines important aspects of the numerical model and the calibration and validation process.
- Section 6 presents the contaminant discharge projections derived from the groundwater model.
- Section 7 summarises this report and presents the conclusions of the investigation.
- A list of referenced reports and documents referred to in this document is provided in Section 8.

2.0 HYDROGEOLOGY

2.1 Topography and Drainage Pattern

The TTTSF is located in the very upper reaches of the Top Tipperary catchment, adjacent to the divides between Tipperary Creek catchment and the catchments of the North Branch Waikouaiti River, Deepdell Creek and Cranky Jim's Creek (Figure 1). The topography ranges in elevation from about 580 mRL at the top of the Tipperary catchment to about 490 mRL in the creek bed at the toe of the TTTSF. Downstream from the TTTSF the elevation of the stream bed drops off to about 410 mRL over a 2.6 km stretch to the confluence with the next substantial tributary of Tipperary Creek. This tributary joins Tipperary Creek from the west and drains a catchment that abuts the southern edge of the Frasers East WRS (Figure 1).

The upper tributaries of Tipperary Creek around Glendale Station are eroded into surficial loess deposits that blanket this area of the site. Deeply incised gullies, which are characteristic of the Deepdell Creek catchment, only appear downstream from where the TTTSF embankment is to intersect Tipperary Creek. Valley slopes are generally gentle (<5°) except adjacent to incised streams where slopes can locally approach 30° (Golder 2011a).

The eastern abutment of the TTTSF is to be constructed on top of the catchment divide between Tipperary Creek and Cranky Jims Creek. From the toe of the abutment in the Cranks Jims catchment the ground slopes down to the creek at a gradient of between 5° and 10°.

It is expected that both Tipperary Creek and Cranky Jims Creek in the vicinity of the TTTSF are ephemeral (Golder 2011g).

2.2 Geology

2.2.1 Introduction

The eastern area of Otago is underlain principally by Mesozoic age schist of the Torlesse Terrane (Forsyth, 2001). Weathering and erosion over a long period formed the distinctive low relief of the Otago peneplain. Deposition of alluvium, rich in quartz gravel occurred in east Otago during the Eocene (Hogburn Formation) and Miocene (Manuherikia Group). Miocene age volcanics were also widespread. Post-Miocene tectonic deformation and erosion has removed most of the Tertiary age deposits, along with an unknown thickness of schist. The resulting landscape in the Macraes area comprises widespread outcrops of schist and thin cover soils (Figure 3).

2.2.2 Schist

The schist, being a crystalline metamorphic rock, has effectively no primary or intergranular porosity or permeability, except where weathered. Secondary porosity and permeability in the form of fractures and faults provide the major groundwater seepage routes below the surficial, strongly weathered zone.

It is considered that hydraulic conductivity of the schist increases upward through the schist rock mass due to the increasing intensity of weathering and reducing overburden pressures. Similar trends or decreasing rock mass permeability with depth have been recorded with respect to fractured crystalline rocks in other areas of the world (e.g., Masset & Loew 2010). This trend has been incorporated in several groundwater models of the MGP site (Kingett Mitchell 2002, 2005a) and is based primarily on an assessment of hydraulic conductivity variation with depth for the Maori Tommy Gully area (GCNZ 1988).

1. SOURCE: Institute of Geological and Nuclear Sciences Geological Map 7. Copyright Reserved

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TITLE REGIONAL GEOLOGICAL MAP

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The results of packer tests performed at the TTTSF site indicate the permeability of the rock mass at the site does not differ substantially from that of the rock mass elsewhere at the MGP site (Appendix B). The hydraulic conductivity applied to previous MGP site groundwater models and packer test results over the proposed TTTSF site are summarised in Table 1.

The hydraulic conductivity applied to previous MODFLOW models of the MGP site has been anisotropic, with a higher value applied in the north-south direction than in the east-west direction (Table 1). This anisotropy has been applied to simulate the presence of minor faults and near vertical fractures aligned approximately north-south across the site as well as to place an emphasis on the low dip of the schist foliations toward the east. The eastward dip of the foliation in the TTTSF area is similar to that across the remainder of the TTTSF site (Golder 2011a). The horizontal anisotropy in hydraulic conductivity could not be confirmed by packer tests performed at the TTTSF site as the tests were not designed for this purpose. Minor north-south trending faults and fractures have been mapped in the TTTSF area (refer Section 2.2.4) and it is expected that these features will prove to be ubiquitous across the site.

Table 1: Comparison of hydraulic conductivity results.

Depth interval (m)	**Hydraulic conductivity applied to previous MGP models (m/s)**		**TTTSF site hydraulic conductivity (1) (m/s)**
	K_x (2)	**K_y (3)**	
0 – 20	3.7 x 10^{-7}	1 x 10^{-6}	5 x 10^{-7}
20 – 60	1.0 x 10^{-7}	2.5 x 10^{-7}	2 x 10^{-7}

Notes:
1) Logarithmic mean of results from packer tests (Appendix B).
2) K_x = hydraulic conductivity in east-west direction.
3) K_y = hydraulic conductivity in north-south direction.

2.2.3 Alluvium and colluvium

Subsurface investigations identified a surficial cover of loess, colluviums and topsoil. The investigations typically exposed 0.2 m to 0.4 m of soil materials, with a maximum identified thickness of 1.8 m (Golder 2011a).

Geotechnical drilling indicates that a thin layer of loess covers much of the proposed TTTSF embankment footprint and, through landform comparison, most of the upper Tipperary Creek catchment. The loess soils comprise a very stiff, light yellow grey silt, sandy silt or silty fine sand. Close to the catchment divide between Deepdell Creek and Tipperary Creek, occasional small bogs have developed on top of the loess. These bogs are generally located in slight depressions where run-off is concentrated and shallow drainage may be impeded by accumulated dust and decomposing vegetation.

Colluvium has accumulated at the base of steep slopes around the MGP site. Colluvium mainly comprises fine angular schist gravel in a sandy or silty matrix, with the matrix mainly derived from reworked loess.

The alluvial fill in the Tipperary Creek valley is not considered to have a significant effect on the regional groundwater flow regime. The fill is neither voluminous enough nor covers a sufficient area to act as an aquifer or aquitard at the scale represented in the groundwater model.

The small thicknesses of loess and colluvium have not been specifically simulated in the groundwater flow model as this material is not expected to have a major effect on groundwater flow routes and rates at the mine site. Groundwater that seeps along the schist/colluvium interface appears either to report rapidly to discharge areas as ephemeral springs on the surface or to be lost to evaporation.

The representation of colluvium and loess in the contaminant transport model is considered important to gain an understanding of the large scale movement of conservatively transported contaminants. These materials do however have a significant effect on the transport of arsenic by groundwater due to the adsorption of arsenic onto the soils and oxidised schist. In this latter case, the arsenic adsorption parameters applicable to these materials have been incorporated in the upper layer of the weathered schist (refer Section 5.3).

2.2.4 Geological structures

In a similar manner to the bedrock across most of the MGP area, the schist bedrock at the TTTSF site is characterised by eastward dipping foliation and foliation parallel fractures. These foliations typically dip at about 30° towards the east or south east. Foliation orientation measurements appear to define a fold associated with the Macraes Fault. Discontinuities observed in the schist comprise mainly foliation partings.

In addition to the foliation parallel discontinuities, several structures have been reported to intersect the TTTSF site (Figure 4). The major tectonic structure is the Macraes Fault, which is inferred to affect an east trending zone approximately 700 m wide north of Tipperary Creek. An assessment of the Macraes Fault did not find evidence of deformation associated with movement of the Macraes Fault during the last 12,000 to 15,000 years (Golder 2011b). The schist observed in investigation trenches close to the Macraes Fault is generally highly fractured and contains a significant proportion of gouge material.

North to northwest striking high angle faulting has been identified through interpretation of drillhole data, evaluation of aerial photograph lineaments and direct mapping of outcrops during investigation of potential tailings embankment locations (Golder 2011a).

Joints in the TTTSF area are typically steeply dipping, have a rough surface and are planar to undulating. The most common strike is approximately southeast.

The northwest trending Hyde-Macraes Shear Zone, which is the gold bearing structure mined by OceanaGold, dips at about 15° towards the east. It is located at a depth of about 1,000 m below surface in the area of the TTTSF (Golder 2011a). Due to its depth the HMSZ is not considered to be significant in terms of contaminant transport from the TTTSF and it is not incorporated in the groundwater model simulating the TTTSF.

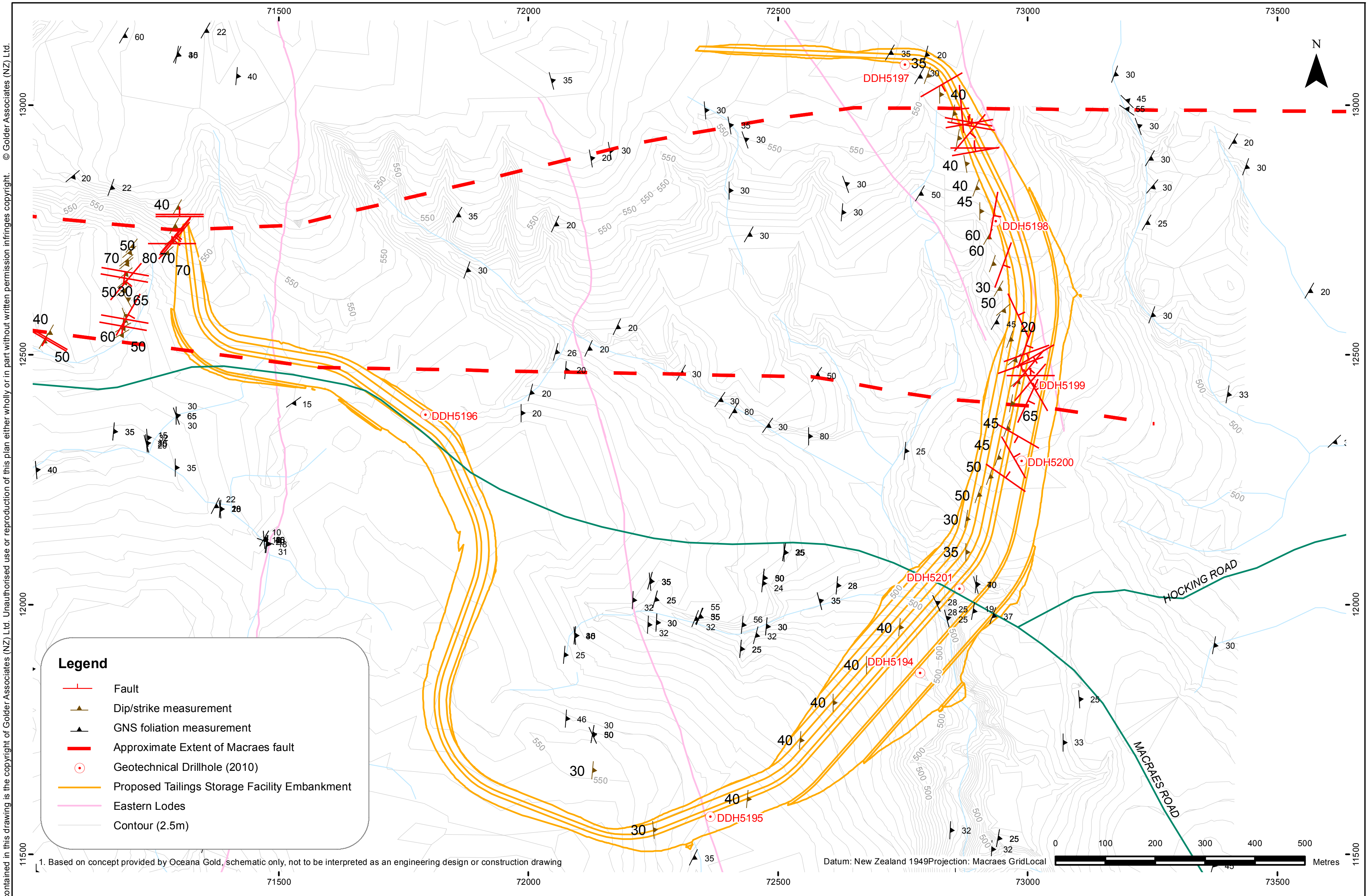
2.3 Weathering

The intensity of rock mass weathering decreases with increasing depth. Minerals in the shallow loess, colluvium and schist to a depth of about 10 m have been subjected to strong oxidation. At greater depths this oxidation decreases in intensity to thin oxidation zones along fracture surfaces and then to fresh rock with no indication of oxidation.

This geochemical interpretation of weathering differs slightly from the geotechnical interpretation of weathering intensity (Golder 2011a). Geotechnical investigation of the TTTSF site has indicated that moderate weathering of the schist as indicated from drillhole cores generally does not extend past a depth of about 5 m, while slight weathering has been identified in drill cores to a depth of about 35 m.

From the point of view of groundwater modelling for the site, the concept of weathering zones has been used loosely to indicate a combination of:

- Decreasing geochemical weathering of the rock mass with depth.
- Increasing overburden pressure that leads to decreasing rock mass porosity and permeability, especially as the foliation partings become less frequent and the apertures decrease.



1. Based on concept provided by Oceana Gold, schematic only, not to be interpreted as an engineering design or construction drawing

Datum: New Zealand 1949 Projection: Macraes GridLocal

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TITLE **SUMMARY OF GEOLOGICAL INFORMATION**

MARCH 2011

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2.4 Groundwater Recharge

An evaluation of the Deepdell catchment water balance (Kingett Mitchell 2005a, Appendix 3) indicated that the regional groundwater recharge rate is approximately 32 mm/year. This recharge value was used in previous groundwater flow and mass transport modelling of the MGP (Kingett Mitchell 2005a) and has been retained for this purpose in the current study.

3.0 WATER QUALITY

3.1 Introduction

Process water from the ore processing plant is to be mixed with mine tailings and deposited sub-aerially as a slurry in the TTTSF. If saturated tailings need to be excavated from the SP11 TSF these tailings would also be pumped to the TTTSF and deposited as a slurry, in contrast to the unsaturated tailings which would be deposited to form a RTS.

During operations there is a permanent standing pool of water covering part of the tailings surface, which is recycled to the plant or lost through evaporation or seepage into the tailings. This standing pool of water is referred to as the decant pond. Samples of decant water and water discharging from the TSF embankment drainage systems are collected and analysed on a regular basis. The MGP water management system includes recycling of as much decant water as possible to the ore processing plant.

During the operational period of the TTTSF it is expected that the tailings will effectively remain in a fully saturated condition as tailings deposition is planned to be continuous. This contrasts with the management of tailings deposition between the MTI and the SPI, where tailings deposition is alternated between the two TSF’s and some of the shallow tailings within the MTI become unsaturated during non-operational periods (Kingett Mitchell 2005c). Monitoring of pore water pressures within tailings stored in the existing TSF’s indicates there is generally a downward hydraulic gradient through the tailings mass (Kingett Mitchell 2005a).

As the decant water seeps through the stored tailings, this water interacts with tailings material. The pore water quality is altered due to dissolution and precipitation reactions that occur within the tailings mass. In addition, oxygen in the pore water reacts with the tailings minerals. Consequently, the geochemical conditions in the tailings mass become more reducing with depth. These interactions and changes mean that the seepage water quality at the base of the tailings mass differs from the decant water quality.

Groundwater monitoring wells have been installed around each of the existing tailings storage areas. Specifically, the water quality in the groundwater system down-gradient from the MTI is intensively monitored. Some of the ions present in tailings seepage water are considered to be conservatively transported. Other ions may be delayed by being precipitated, adsorbed or broken down during their transport through the groundwater system at the MGP. The observed breakthrough curves at monitoring wells have been evaluated for a range of parameters. This analysis can support an assessment of the degree to which contaminant loads are reduced within the groundwater system prior to discharge to receiving surface waters at greater distance from the tailings storage than the monitor bores.

Environmental water quality data from the OceanaGold environmental monitoring database is documented in a separate report by Golder (2011c). The water quality information from that report that has been applied to groundwater modelling of the Tipperary catchment is summarised below.

3.2 Tailings Decant Water

Assessment of the geochemistry of tailings from MGP and RGP ore indicates the differences in decant water quality generated by processing the two streams of ore are not likely to be substantial (Golder 2011d). Tailings decant water quality is more strongly controlled by the quality of the water used in the process plant than by the nature of the ore being processed at any particular time. The concentrations of dissolved contaminants in the process water tend to increase over time due to the ongoing recycling of water through the TSF's to the process plant. The treatment processes and reagents used in the processing plant add further contaminants during each cycle of water through the plant. Consequently the ore being processed at any particular time is not the most important factor in controlling tailings water quality. This conclusion applies to the operational period following the implementation of the pressure oxidation stage in the process plant. Introduction of a pressure oxidation stage to the process plant resulted in a substantial decrease in decant water quality compared to the previously observed water quality.

Tailings decant water quality data from the past 10 years of operation, since the implementation of pressure oxidation at the MGP process plant, is considered to be the most appropriate dataset to use to derive the projected decant water quality for TTTSF groundwater modelling purposes. For modelling purposes the 90th percentile of the water quality analysis results from the combined MTI and SPI drainage systems for the period from 2000 to 2010 have been applied (Table 2).

Table 2: Tailings decant water summary statistics – 2000 to 2010.

Parameter (1)	**MTI**		**SP10 and SP11**		**Combined TSF's**
	Median	**90th Percentile**	**Median**	**90th Percentile**	**Mean of 90th percentiles**
Sodium	430	580	470	590	585
Potassium	92	130	90	120	125
Calcium	570	680	560	680	680
Magnesium	310	430	330	420	420
Chloride	29	62	29	46	54
Sulphate	3,800	5,500	4,200	5,800	5,650
Arsenic	0.7	2.4	0.8	4.5	3.4
Copper	0.52	0.99	0.26	0.29	0.64
Iron	39	520	100	660	590
Lead	0.02	0.014	0.004	0.006	0.01
Zinc	0.028	0.062	-	<0.001	0.035
Cyanide_{WAD}	0.7	0.85	0.10	0.23	0.47 (2)

Notes: 1) All units in g/m^3. Values rounded to two significant figures.
2) Cyanide_{WAD} concentrations in the decant water based on the 90th percentile of decant water quality data from the past two years due to decreased concentrations resulting from optimisation of the process plant.

The mine water management modelling for the site is expected to generate decant water quality projections that differ in detail from the groundwater model input values presented here. The values applied to the water quality parameters modelled for groundwater transport are considered to be conservative for generating contaminant transport projections.

3.3 Tailings Seepage Water

Tailings pore water projections for the TTTSF have been based on the analysis of MTI drainage discharges from 2000 through to 2010. The 90th percentile value from this data set has been applied as a conservative indicator of seepage water quality from the TTTSF during the operational period of the facility.

Following closure of the TSF it is expected that a gradual change in seepage water quality will occur over time. This change would reflect a shift in recharge water quality to the top of the tailings mass from process water to rainwater. During the operational period of the TSF the seepage water quality is expected to be controlled to a large extent by the quality of the water discharging from the slurry pipeline to the impoundment. Following closure the groundwater level in the tailings mass would decline due to reduced recharge, reducing the volume of water left over from the operational period. At the same time infiltrating rainwater would start to replace the existing pore water. The infiltrating rainwater would leach contaminants from the tailings mass, but the eventual concentrations are expected to be considerably lower than those indicated from analysis of the drainage records from most of the MTI and SPI drain discharges.

The concentrations applied in modelling the tailings pore water quality have been based on the 90th percentiles of water quality data from MTI and SPI drain water quality datasets for the period from 2000 through to 2010 (Table 3). The exception is the water quality value for cyanide, which has been reduced due to the lower concentrations observed in TSF decant water over the past two years.

Table 3: Projected TSF pore water quality.

Parameter (1)	**Tailings pore water quality**		**Waste rock leachate quality (2)**
	Operational (3)	**Post-closure (4)**	
Sodium	498	416	62
Potassium	46	17	13
Calcium	411	410	470
Magnesium	245	200	390
Chloride	107	111	11
Sulphate	2,769	2,260	2,500
Arsenic	5.38	1	0.007
Copper	0.02	0.02	0.0027
Iron	31	21	1
Lead	0.01	0.013	0.00021
Zinc	0.02	0.009	0.035
$\text{Cyanide}_{\text{WAD}}$	0.47 (5)	0.35	0

Notes:
1) All units in g/m^3.
2) Based on water quality from Northern Gully WRS underdrains.
3) Based on the 90th percentile of the average of MTI drainage water quality since 2000.
4) Based on the 90th percentile of the water quality from the MTI underdrains since 2000.
5) $\text{Cyanide}_{\text{WAD}}$ concentrations based on the 90th percentile of decant water data from the past two years due to decreased concentrations resulting from optimisation of the process plant.

3.4 Waste Rock Seepage

The leaching of stored waste rock occurs under more oxidising conditions than those present in stored tailings. As such, the leachate water quality from WRS's differs from that expected to apply to long term tailings leaching. Discharge water from underdrains installed beneath the Northern Gully WRS has occasionally been collected and analysed. Although this data set is small, the 90th percentile of these results is considered to be reasonably indicative of the water quality that may be expected in seepage water from WRS's at the MGP into the future (Table 3). These concentrations have been applied in the groundwater model to represent seepage water quality form the waste rock stacks and the exposed TSF embankment area.

3.5 Contaminant Attenuation in Groundwater

3.5.1 Introduction

Conceptually, dilution of contaminants within the groundwater system affects all contaminants equally, provided the source concentrations exceed the background water concentrations. Conservative transport of contaminants implies the relative decrease in concentration with distance from the source is the same for all parameters. If the decrease is greater, other factors may be acting to remove the contaminant from the groundwater. If the decrease is less, the background concentrations in the groundwater may be similar to those at the contaminant source. Changes in groundwater quality may also result in a specific contaminant being taken into solution at an increased rate from the surrounding rock mass.

The water quality monitoring data from the MTI seepage drains and the MTI detection wells in MTG indicates that there is a considerable decrease in the non-conservative parameters occurring. In order to assess the relative importance of dilution and active removal processes, the decrease in concentration between the seepage water and the detection wells has been calculated for the MTI (Table 4).

Table 4: Concentration reduction between seepage and groundwater wells.

Parameters (1)	**MTI seepage concentration** (2) **(g/m^3)**	**Detection well concentration** (3) **(g/m^3)**	**Concentration reduction factor** (4)
Conservative Parameters			
Sodium	530	110	4.8
Potassium	37	5.1	7.3
Chloride	102	25	4.1
Non-conservative parameters			
Arsenic	3.9	0.019	205
Iron	8.7	7.8	1.1 (5)
$\text{Cyanide}_{\text{WAD}}$	0.85	0.040	21

Notes:
1) Source: Golder (2009).
2) Mean concentration in Sump B_CDE and Sump B_CDW from 2007 to 2009.
3) Mean concentration in detection wells GW46 to GW51 from 2007 to 2009.
4) Concentration reduction factor = MTI seepage concentration/Groundwater well concentration.
5) Implies iron being brought into solution from in-situ rock mass.

Sodium, potassium and chloride, which are considered to be conservatively transported contaminants, are reduced in concentration by a factor of between 4.1 and 7.3 during transport through the area up-gradient from the detection wells. This indicates that dilution between the seepage from the impoundment and the groundwater wells is responsible for approximately an approximate 4 to 7 fold decrease in concentration.

3.5.2 Arsenic

The mean arsenic concentration in the groundwater wells is approximately 200 times lower than in the seepage water. As dilution is estimated to be responsible for a 4 to 7 fold reduction in concentration, it is evident that a considerable mass of arsenic is being removed by active processes in the aquifer. The most likely active removal processes for arsenic are mineral precipitation and adsorption. It is currently unclear which of these processes is primarily responsible for maintaining low arsenic concentrations in the groundwater.

If mineral solubility is the dominant factor then the concentration of arsenic should remain stable in the aquifer provided the hydrochemical conditions (redox and pH) remain stable in the aquifer. If adsorption of arsenic onto the rock mass is the dominant process, there will be a finite number of adsorption sites in the aquifer and there is potential for the arsenic concentrations to increase in the groundwater over time.

Arsenic adsorption by loess and weathered schist has been investigated through laboratory testing of samples of these materials obtained from the MGP site. The adsorption test procedure, the results of the testing and the derivation of adsorption isotherms applied to the mass transport model are documented in Appendix C.

As discussed in Section 3.1, the seepage water from the TTTSF is expected to be generated under reducing conditions. The K_d value and the maximum adsorption capacity of the rock mass has therefore been applied with respect to the arsenic (III) reduced form (Appendix C).

Adsorption of arsenic onto three different modelled materials has been taken into account in the mass transport model through the application of Langmuir isotherms as indicated in Table 5. Specifically, the isotherm applied in the modelling requires values for K_d and the concentration of available adsorption sites (SP2) within the rock mass. In assigning the SP2 values used for the different model layers, factors considered included the degree of rock weathering and the fracture density (Appendix C).

The adsorption characteristics of the embankment rock fill are considered to be similar to those of weathered schist. This estimate is based on the use of compacted weathered rock to construct a low permeability zone within the embankment to reduce seepage losses. In addition, the broken and crushed rock used to construct the main body of the embankment is considered to have a considerably higher concentration of adsorption sites than would apply to the in-situ schist that has been classed as moderately weathered for modelling purposes.

Table 5: Langmuir adsorption input parameters for arsenic (III).

Model layer	**Distribution coefficient (K_d) L/mg**	**Concentration of adsorption sites (SP2) kg/kg**
Highly weathered schist	2×10^{-5}	0.82
Moderately weathered schist	2×10^{-5}	0.1645
Slightly weathered schist	0	0
Embankment body	2×10^{-5}	0.82
Unweathered schist	0	0
Tailings	0	0

The values listed in Table 5 have been applied to the contaminant transport model in the form of:

- A K_d of 2×10^{-5} L/mg, applicable to the model zones simulating weathered schist and loess and the underlying moderately to slightly weathered schist.

- Maximum arsenic adsorption of 230 mg/kg for the upper 20 m of the weathered schist and loess.
- Maximum arsenic adsorption of 46 mg/kg for the moderately to slightly weathered schist between approximately 20 m and 60 m below the ground surface.
- Conservative transport of arsenic at depths below 60 m.

3.5.3 Iron

Iron concentrations in the MTI detection wells increased rapidly during the first three years of monitoring. Subsequently, the concentrations have remained relatively stable. The reduction factor of 1 (refer Section 3.5.1) suggests iron is taken into solution from the surrounding rock mass to a greater extent than may be occurring with the conservatively transported parameters.

The concentration of iron in the aquifer is likely to be dependent on the solubility limits of iron oxyhydroxide minerals such as ferrihydrite. The solubility of these iron minerals is highly dependent on pH and redox conditions in the groundwater. The initial increase in iron concentrations detected down-gradient from the MTI may have been due to a change in these conditions following the construction of the TSF.

Assuming the redox or pH conditions in the groundwater down-gradient from the TTTSF will be similar to those down-gradient from the MTI, it is likely that the concentrations of iron in groundwater down-gradient from the TTTSF are likely to be similar to those detected in groundwater samples from the MTI detection wells.

3.5.4 Cyanide$_{WAD}$

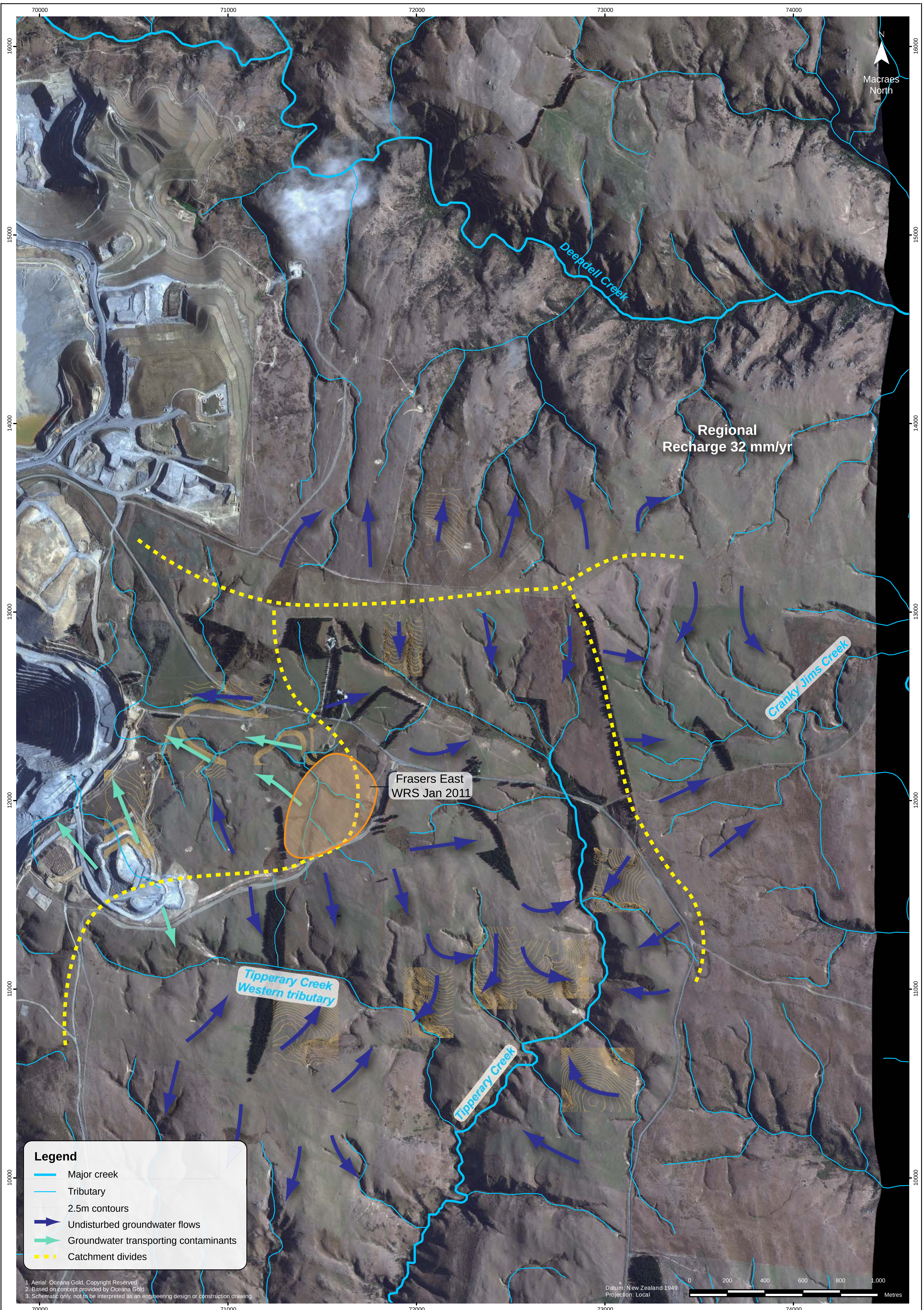
The mean cyanide$_{WAD}$ concentration in the MTI detection wells is approximately 21 times lower than that in the MTI seepage water. Given that dilution is estimated to be responsible for a 4 to 7 fold reduction in concentration, it is evident that a considerable mass of cyanide$_{WAD}$ is being removed by active processes in the groundwater. These processes could include complexation and precipitation, adsorption and biodegradation. The primary mechanism of cyanide$_{WAD}$ removal is unclear.

The reduction in cyanide$_{WAD}$ concentration within the groundwater system down-gradient from the MTI is approximately 3 to 5 times greater than that for conservatively transported contaminants. As such, modelling of cyanide$_{WAD}$ transport within the groundwater system assuming conservative transport is likely to generate mass load results 3 to 5 times greater than may be expected at site. This factor may increase with increasing flow path length. There is no indication that cyanide$_{WAD}$ from the MTI has reached the down-gradient MTI compliance wells. As such, there is insufficient information available to estimate cyanide$_{WAD}$ attenuation rates over greater transport distances (Golder 2011c).

4.0 CONCEPTUAL FLOW AND MASS TRANSPORT MODEL

The conceptual groundwater model for the TTTSF site, as presented in Figure 5 and Figure 6 is based on the following site characteristics:

- The schist basement rock has been differentiated into weathering zones on the same basis as applied in previous groundwater modelling work for the MGP site (Kingett Mitchell 2005a). In this modelling approach, each weathering zone has been defined with an approximate thickness as measured from the original topographic surface.



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Datum: New Zealand 1949
Projection: Local

Golder Associates

TITLE

CONCEPTUAL GROUNDWATER AND MASS TRANSPORT MODEL - EXISTING CATCHMENT LAYOUT

MARCH 2011

PROJECT 0978110562

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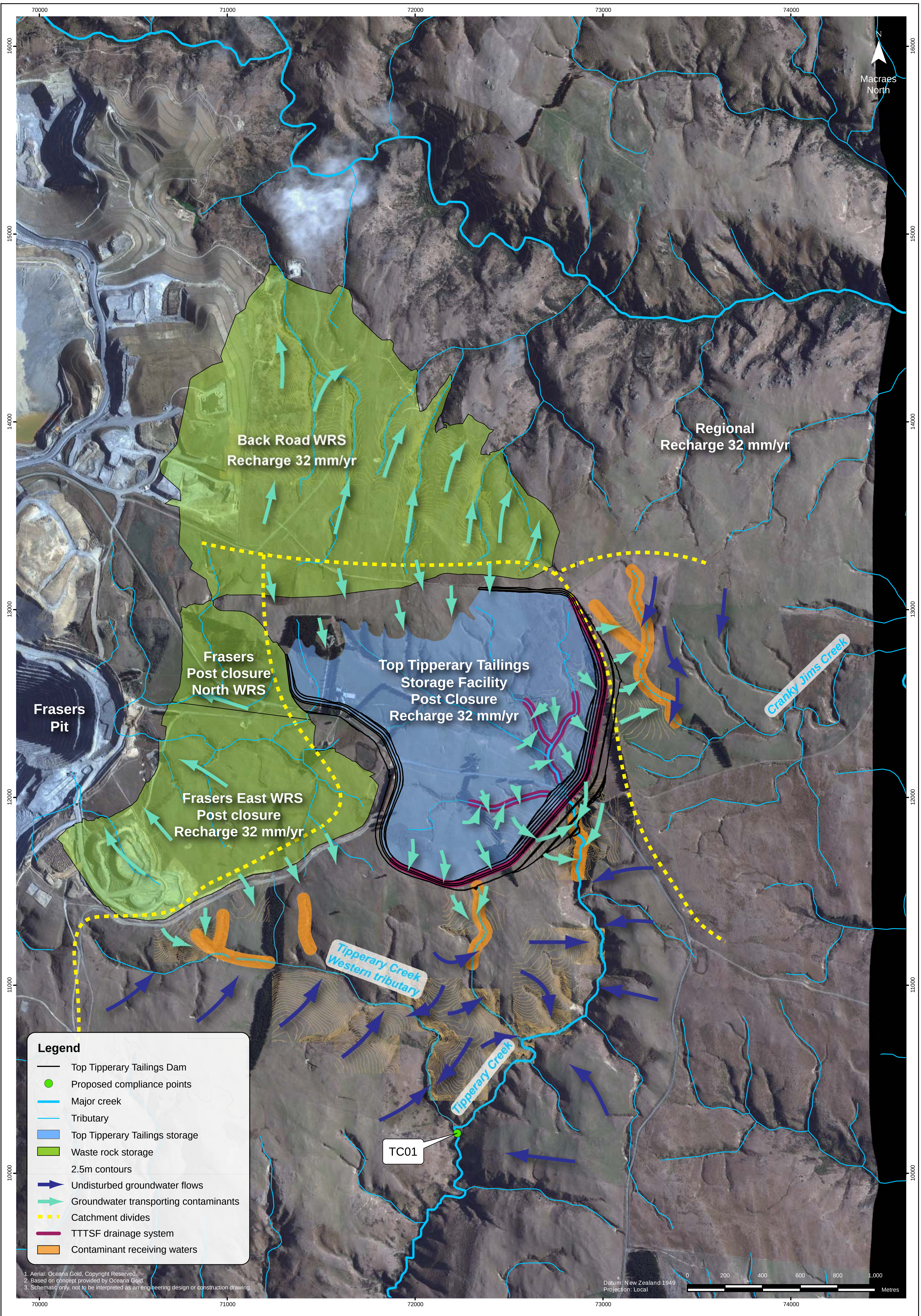

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TITLE

CONCEPTUAL GROUNDWATER AND MASS TRANSPORT MODEL - TTTSF CATCHMENT LAYOUT

MARCH 2011

PROJECT 0978110562

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- Hydraulic conductivity and storativity applied to the schist decrease with depth until values consistent with unweathered, unrelaxed schist are reached at a depth of approximately 100 m below the ground surface.
- Based on previous work (Kingett Mitchell 2005a) the rate of recharge to the regional groundwater system has been defined as 32 mm/year.
- Recharge applied to the tailings impoundment during the operational phase greatly exceeds the regional recharge. The exact value is to be sufficient to ensure the groundwater table within the simulated tailings mass is equal to the top of the tailings mass at the close of the 8-year operational period of the TTTSF.
- Following closure of the TTTSF and its rehabilitation, recharge to (and through) the tailings is expected to decrease to the regional background recharge rate. This reduction in recharge reflects the proposed installation of drainage systems at the rear of the impoundment to prevent ponding on top of the tailings. OceanaGold plans to pump down the tailings decant pond prior to closure of the TTTSF. The reduction in recharge also reflects the planned capping of the tailings with a soil and weathered rock layer suitable for rehabilitation with vegetation similar to the surrounding area.
- Natural drainage channels consist of:
  - Tipperary Creek and its upper tributaries.
  - Deepdell Creek and a few tributaries not covered by the Back Road WRS. Although Deepdell Creek is not expected to receive seepage water from the TTTSF, it is incorporated as a fixed hydraulic boundary condition for the model.
  - Cranky Jims Creek and its upper tributaries.
- Frasers Pit and Golden Point Pit are simulated at their maximum extent in order to ensure the groundwater gradients across the model toward the west from Tipperary catchment are reasonable.
- The TTTSF consists of:
  - An embankment body with three permeability zones based on the design layout from EGL (2011). The hydraulic conductivity of the embankment body is consistent with that of waste rock. The upstream face of the embankment is defined with a low-permeability Zone A (EGL 2011).
  - A tailings surface at the design elevation of 559 mRL. Although the tailings surface is expected to have a gradient of approximately 1 in 200, sloping away from the embankment crest, this gradient is not significant in terms of the hydraulic behaviour of the tailings mass and contaminant losses from the tailings.
  - The permeability of the tailings mass and the waste rock used for construction of the TTTSF embankment is the same as that previously applied to the calibrated groundwater models of the Mixed Tailings and Southern Pit storage facilities.
  - The total porosity, effective porosity and specific yield values applied to the tailings mass are within the range of 0.35 to 0.4, reflecting published values for the tailings material.
  - Drains built into the embankment to simulate the planned chimney drains (EGL 2011). The simulated drainage capacity for each drain is sufficient to maintain the embankment in an unsaturated state. These drainage capacities are based on calibrated values applied to the MTI in previous modelling of the MGP area (Kingett Mitchell 2005a).
  - Underdrains installed in each gully beneath the tailings storage area. The simulated drainage capacity of each underdrain is based on the demonstrated capacity of similar underdrains constructed in the MTI (Kingett Mitchell 2005a).
  - Following closure, the drainage systems built into the TTTSF remain active as the tailings mass becomes progressively unsaturated.

- Contaminants are introduced to the model in three areas: Back Road WRS, Frasers East WRS's and the TTTSF including the embankment. The regional groundwater system has not been assigned with background concentrations.
- The water quality in the model has been defined to simulate the following factors:
  - During the operational period of the TSF, tailings pore water quality beneath the expected decant pond are defined to be representative of decant water. As the decant pond moves away from the embankment toward the top of the catchment area, the contaminant concentrations in the tailings mass are changed to reflect this movement.
  - During the operational period of the TSF, tailings pore water in areas not overlain by the decant pond is defined to be representative of the observed drainage water quality from the existing TSF's.
  - On closure of the TTTSF, the decant pond is removed and the quality of the tailings pore water across the entire TSF gradually reverts to be representative of the expected long term leachate water quality.
  - Contaminants are introduced to the model in areas corresponding to exposed surfaces of the TTTSF embankment through the use of constant concentrations in the recharge water. The concentrations applied are the same as those applied to WRS areas.
  - The Back Road WRS and the Frasers East WRS are simulated as bounding the Tipperary Creek catchment to the north and west of the TTTSF, respectively. For the purpose of this modelling project it is assumed that both of these WRS's are in place at the start of the simulation period although construction of the Back Road WRS is not planned to begin until about 2015. The difference in the planned construction schedule from that assumed for the model is not significant in terms of contaminant mass introduced to the groundwater system over the term of the model. Contaminants are introduced to these areas as constant concentrations representative of waste rock leachate. The concentrations applied do not vary for the period of the model.
- The groundwater model incorporates an 8 year TTTSF operational period followed by a post-closure period of approximately 150 years to enable seepage flows to reach the local receiving waters and peak mass loads to these receiving waters to be assessed.

5.0 NUMERICAL FLOW AND MASS TRANSPORT MODEL

5.1 Software

Industry standard groundwater flow and mass transport modelling packages were used for the numerical modelling. The Visual MODFLOW Pro software package was used to construct the groundwater model.

The groundwater flow field in the model and physical flow calibration procedures were calculated using MODFLOW 2000 public domain code from the United States Geological Survey. The mass transport simulation was calculated utilising the MT3D99 code attached to Visual MODFLOW package.

The digital model developed for the simulation of the groundwater flow system and mass transport simulation is documented in Appendix D attached to this report.

5.2 Flow Model

Details of the model grid and layout are presented in Appendix D. The numerical groundwater model developed for this study is based on and aligned to the MGP site grid. The numerical groundwater flow model incorporates the following main features:

- Model limits are:
  - 70,000 to 74,500 m (MGP grid) East – West.
  - 10,000 to 15,500 m (MGP grid) North – South.
  - Total modelled area is 24.75 km^2.
  - A model base set at 0 mRL, approximately 270 m below the Deepdell Creek invert at its lowest point in the model area.
- A total of 7 layers have been defined to the model.
- The main natural drainage channels in the model domain, Tipperary Creek, Deepdell Creek and Cranky Jims Creek, as well as the main tributaries to these creeks were simulated using drainage cells.
- Frasers Pit and Golden Point Pit are simulated at the western boundary of the model using drainage cells.
- Regional groundwater recharge is applied to the uppermost active cells at a rate of 32 mm/year.
- Hydraulic conductivity parameters have been applied as indicated in Table 6. These values are the same those applied to the Kingett Mitchell (2005a) model used to simulate the wider MGP area.
- The storage parameters applied to the schist rock mass on a regional basis (Table 7) are consistent with those applied in existing calibrated models of the MGP. The storage and porosity values applied to the tailings mass have been based on survey and mass balance data for the MTI as well as from documented values for similar tailings impoundments worldwide.

The TTTSF has been simulated in the numerical model through:

- Increasing the thickness of the uppermost model layer to match the final proposed form of the TTTSF.
- Matching the hydraulic conductivity and storage parameters within the simulated TTTSF to correspond to those defined above for the embankment and tailings materials.
- Defining drainage cells to simulate the planned construction of underdrains in gullies that intersect the TTTSF footprint.
- Defining drainage cells to simulate chimney drains and collector drains to be constructed within the upstream face of the embankment. The conductance values applied to the drainage cells have been defined to ensure the overlying embankment remains in an unsaturated state. The embankments for the MTI and SP11 have remained in an unsaturated state due to the installation of a low permeability upstream liner and substantial drainage systems. It has been assumed that similar drainage efficiencies would be achieved for the TTTSF embankment.

Table 6: Hydraulic conductivity values applied to groundwater model.

Geological feature $^{(1)}$	**K_X**	**K_Y**	**K_Z**
Highly weathered schist	3.5×10^{-7}	1.0×10^{-6}	2.5×10^{-7}
Moderately weathered schist	1.0×10^{-7}	2.5×10^{-7}	6×10^{-8}
Slightly weathered schist	5.0×10^{-9}	9.0×10^{-9}	1.0×10^{-9}
Unweathered schist	1.0×10^{-9}	5.0×10^{-9}	5.0×10^{-10}
Embankment Zone A	1.0×10^{-7}	1.0×10^{-7}	1.0×10^{-7}
Embankment body – Zone B	5.0×10^{-6}	5.0×10^{-6}	5.0×10^{-6}
Embankment body – Zone C and WRS	1.0×10^{-6}	1.0×10^{-6}	1.0×10^{-6}
Fine tailings	2.0×10^{-7}	2.0×10^{-7}	2.0×10^{-7}
Coarse tailings	5.0×10^{-6}	5.0×10^{-6}	5.0×10^{-6}

Note: 1) All units in m/s.

The piezometric head within the tailings body during the operational period of the TTTSF has been maintained at an elevation equivalent to the final tailings surface elevation through the use of constant head cells. At closure of the TTTSF, the constant head cells are turned off and the water level within the tailings mass is allowed to gradually recover to an elevation appropriate for a recharge rate of 32 mm/year. This recharge rate is applied to the top of the tailings mass for the remainder of the simulation period.

Table 7: Storage property values applied in the groundwater model.

Geological feature	Specific storage	Specific yield	Effective porosity	Total porosity
	(m^{-1})	(m^{-1})	(m^3/m^3)	(m^3/m^3)
Heavily and moderately weathered schist	1.0×10^{-5}	0.02	0.02	0.03
Slightly weathered and unweathered schist	1.0×10^{-5}	0.005	0.005	0.006
Tailings	1.0×10^{-5}	0.38	0.38	0.4
Waste rock	1.0×10^{-5}	0.2	0.2	0.25

5.3 Mass Transport Simulation

As discussed in Section 4, there are three potential sources of contaminant discharges to the Tipperary Creek catchment. These sources are decant water ponded on top of the tailings mass, tailings pore-water and leached contaminants from the TTTSF embankment and the WRS's located at the top of the catchment.

Contaminants are introduced to the groundwater model through two different mechanisms:

- In the area of the exposed tailings embankment and across the WRS areas contaminants are introduced to the model as constant concentrations in the recharge water being applied to the uppermost active model cells (Table 8).
- Contaminant concentrations in tailings beneath areas expected to be covered by decant water are applied as constant concentrations defined for the uppermost model cells. The concentrations applied reflect the decant pond water quality over the past 4 years (Table 9). As the decant pond moves away from the embankment toward the top of the catchment area, the contaminant concentrations in the tailings mass are changed to reflect this movement. At closure of the TTTSF the constant concentrations reflecting decant pond water quality cease to be applied.
- Contaminant concentrations in tailings not covered by ponded decant water are also applied as constant concentrations defined for the uppermost model cells. The concentrations applied reflect drainage discharge water quality from the MTI and SP11 over the past 4 years (Table 9). The tailings volume characterised by operational tailings pore water quality increases over the operational period of the TTTSF to reflect the increasing mass of stored tailings. At closure of the TTTSF the constant concentrations reflecting operational tailings pore water quality cease to be applied.
- On closure of the TTTSF, contaminants start to be introduced to the tailings mass through the application of constant concentrations to the tailings recharge water. The concentrations applied are representative of the expected long-term tailings pore water quality (Table 9). Simulating this change in water quality through applying contaminated recharge water to the tailings enables a gradual shift in tailings water quality to occur. This addition of contaminant mass continues for the entire post-closure simulation period of the model.

Table 8: Waste rock recharge water quality.

Parameter [1]	Water quality (g/m^3)
Sodium	62
Potassium	13
Calcium	470
Magnesium	390
Chloride	11
Sulphate	2,500
Arsenic	0.007
Copper	0.0027
Iron	1.0
Lead	0.00021
Zinc	0.035
Cyanide$_{WAD}$	0

Note: 1) All units in g/m^3.

Following closure of the MGP it is expected that the seepage water quality from the TSF's and the WSR's would gradually improve over time. It is however difficult to assess the actual rate of post-closure pore water quality improvement. In order to be conservative in long term model predictions:

- The quality of the recharge water applied to the WRS's and to the TTTSF embankment is not changed during the simulation period, and
- The quality of the recharge water applied to the TTTSF following closure is not changed during the remaining 150 years of the simulation.

Table 9: Tailings water quality.

Parameter (1)	Operational period		Post-closure period
	Decant water (g/m^3)	Pore water (g/m^3)	Long term leachate (g/m^3)
Sodium	585	498	416
Potassium	125	46	17
Calcium	683	411	410
Magnesium	424	245	200
Chloride	54	107	111
Sulphate	5,654	2,769	2,260
Arsenic	3.43	5.38	1.0
Copper	0.64	0.02	0.02
Iron	590	31	21
Lead	0.01	0.01	0.013
Zinc	0.33	0.02	0.009
Cyanide$_{WAD}$	0.47	0.35	0.35

Notes: 1) All units in g/m^3.
2) 90th percentile of decant water data since Jan 2006.
3) 90th percentile of drain water data since July 2008.

All of the contaminants introduced to the groundwater model, with the exception of arsenic, are simulated as being conservatively transported. As discussed in Section 3.5.2, arsenic is not transported conservatively in the natural groundwater system at the MGP site. The distribution coefficients and concentration of adsorption sites presented in Table 5 have been applied to simulate a reversible adsorption of arsenic onto the surrounding rock mass.

Contaminant transport for each of the simulated contaminants with the exception of arsenic has been undertaken on the basis of conservative transport within the groundwater system. Arsenic transport has been modelled based on arsenic (III) being the main form of this element in the tailings seepage water. The adsorption parameters for arsenic (III) have been derived from testing of rock and soil samples from the site. The results of the tests have been applied to the contaminant transport model in the form of:

- A K_d of 20 L/kg, applicable to the model zones simulating weathered schist and loess and the underlying moderately to slightly weathered schist.
- Maximum arsenic adsorption of 230 mg/kg for the upper 20 m of the weathered schist and loess.
- Maximum arsenic adsorption of 46 mg/kg for the moderately to slightly weathered schist between approximately 20 m and 60 m below the ground surface.
- Conservative transport of arsenic at depths below 60 m.

5.3.1 Mass load calculation

The seepage flows discharging to surface water channels in the Tipperary Creek and Cranky Jims Creek catchments have been monitored using a zone budget facility available with Visual MODFLOW. This facility enables the groundwater flows and contaminant mass in pre-defined zones within the model to be monitored and mass loads through these zones to be calculated. Contaminant mass loads discharging to Deepdell Creek, the North Branch of the Waikouiti River, Murphy’s Creek and the opencast pits have been simulated using a separate groundwater model (Golder 2011e) and are not discussed in this report.

Zones have been specified in the groundwater model covering all of the potential natural receiving water bodies as well as cells simulating the drainage system within the TTTSF. Contaminant discharge mass loads have been derived from recording water flows to drains within the specified zones of the model. Water quality in the cells is monitored and the mass load discharging to these zones is calculated by multiplying the flow by the contaminant concentration.

6.0 CONTAMINANT MASS LOAD PROJECTIONS

6.1 Introduction

Leachate from the proposed TTTSF is expected to discharge to:

- The drainage systems built into the TTTSF, including underdrains installed beneath the tailings and chimney drains constructed in the embankment.
- Tipperary Creek and tributary gullies, including both reaches managed through the use of sediment settling ponds and unmanaged sections.
- Cranky Jims Creek and tributary gullies, including both reaches managed through the use of sediment settling ponds and unmanaged sections.

Leachate from the WRS's at the upper boundaries of the Tipperary catchment is expected to discharge to Tipperary Creek and its tributary gullies.

The groundwater model of the Tipperary Creek catchment indicates contaminants from the TTTSF are unlikely to be transported toward Deepdell Creek, Frasers Pit or Murphy's Creek.

The reported mass loads for the natural drainage channels to not necessarily translate directly into mass loading in streams, especially during summer. Evaporative losses from gullies during summer often results in no surface water flow at the lower end of the gully and consequently no surface water mass load contribution to the main catchment drainage channel. Simulated mass loads calculated for tributaries to Tipperary Creek and Cranky Jims Creek, should be considered worst case values which would normally only apply during periods of medium to high surface water flows.

Similar modelling previously undertaken for sulphate and other conservatively transported contaminants at the MGP has produced conservative estimates of mass transport across the site (Kingett Mitchell 2005a; Golder 2011e). The validation process for modelling of contaminant transport from existing MGP operations cannot be replicated for the TTTSF, as this facility has not yet been constructed.

As cyanide$_{\mathrm{WAD}}$ is not transported conservatively in the groundwater at the site (refer Section 3.5), the projections for cyanide$_{\mathrm{WAD}}$ mass loads over the short term are very conservative. Over the long term, the simulation results for cyanide$_{\mathrm{WAD}}$ will probably continue to be conservative.

6.2 Tailings Seepage Processes

Pore water from the tailings mass is expected to discharge to the underlying rock mass and to the TTTSF drainage system at progressively decreasing rates over a period of several decades. The rate at which seepage losses decline is governed by:

- The permeability of the tailings mass, including variations in permeability with distance from the embankment and anisotropic behaviour of the tailings mass.
- The extent of the TSF drainage system.
- Tailings compaction and compaction of the underlying soils.
- Climate factors such as major rainfall events.

Evaluation of drainage records from the MTI and SP11 (Golder 2011f) indicates that discharge flows to the embankment drainage systems of the TSF could be expected to decrease by between 50% and 90% during the first two years following closure of the TSF. This decline in flow primarily represents the dewatering of the coarse tailings close to the embankment.

Subsequent dewatering of the finer tailings further from the embankment is a slower process which is likely to require a considerably longer time. The progressive decrease in the groundwater level within the tailings mass causes a corresponding reduction in hydraulic pressure at the base of the tailings mass. This pressure reduction implies a progressive decrease in downward seepage rates into the underlying rock mass and toward the underdrains.

It is expected that much of the stored tailings mass would become unsaturated during a 30 year period following closure of the TTTSF. There is however, considerable uncertainty with respect to the length of time required for this dewatering process to occur. The uncertainty is due to the inherent variability of the factors governing the seepage rates as listed above.

When most of the tailings mass has become unsaturated the contaminant loads discharging from the tailings would be associated with the residual moisture content and ongoing recharge from precipitation. Further transport of contaminants from the tailings would mainly occur in response to significant rainfall events. These events would lead to pulses of seepage water travelling downward through the unsaturated tailings to the groundwater table. These pulses, averaged on a long term annual basis, are expected to be equivalent to the natural 32 mm/year groundwater recharge rate for the region.

Tailings seepage discharges from the TTTSF are limited to:

- Upward seepage to the tailings surface driven by settlement of the tailings mass.
- Discharges to the drainage systems installed in the TSF embankment and underdrains installed in gullies prior to deposition of the tailings.
- Seepage to nearby surface water receiving waters including the main channel of Tipperary Creek, the western tributary to Tipperary Creek and Cranky Jims Creek.

6.3 Drainage System Discharges

Seepage losses from the tailings mass to the TTTSF drainage systems are calculated to reach a maximum of approximately 1,800 m^3/day, including discharges from the planned underdrains beneath the tailings mass (Figure 7) and the embankment drainage system (Figure 8). Over the long term, the combined drainage flows from the TTTSF are expected to decline to approximately 260 m^3/day.

The modelled rate at which the drainage flows initially decrease, especially discharges from the embankment drainage system, is slower than observed decreases in discharges from the MTI (Golder 2011f). On that basis, it is expected that the discharge flows would decrease more rapidly than indicated in Figure 7 and Figure 8. As decrease in flow is directly reflected in the calculated decrease in contaminant mass load, it is expected that the contaminant loads will also decrease more rapidly than the model results indicate.

Modelled drainage flows from the drainage systems are dependent on the final extent and design of these systems. The flows modelled for the underdrain system are particularly sensitive to changes in the length of drains installed in the gullies prior to the deposition of tailings. At this stage a sensitivity analysis has not been undertaken, however it is clear that a more extensive underdrain system would lead to greater initial and long term discharge flows from these drains. These increased discharges would however be balanced by corresponding reductions in long term contaminant losses to the underlying rock mass and eventually to nearby receiving water bodies. In effect, the seepage flows through the tailings mass would remain the same however a greater proportion would discharge via the expanded drainage system compared to seepage through the underlying rock mass.

6.4 Discharges to Tipperary Creek

Groundwater modelling indicates groundwater discharges to Tipperary Creek upstream from the proposed silt pond are currently in the order of 140 m^3/day. Although surface water flow monitoring has been instigated on Tipperary Creek (Golder 2011g), the record is not yet long enough to provide supporting base flow data.

Construction of the TTTSF would result in these groundwater discharges being substantially reduced as much of the upper catchment is infilled and the groundwater flows are diverted to the TTTSF drainage systems. Calculated groundwater flows to the main channel of Tipperary Creek upstream from the proposed silt dam peak at approximately 46 m^3/day (Figure 9). These flows are small as this stretch of Tipperary Creek is only about 300 m long and most of the seepage flows from the TTTSF are captured by the embankment drainage system. Over the long term the flows are expected to decline to about 30 m^3/day.

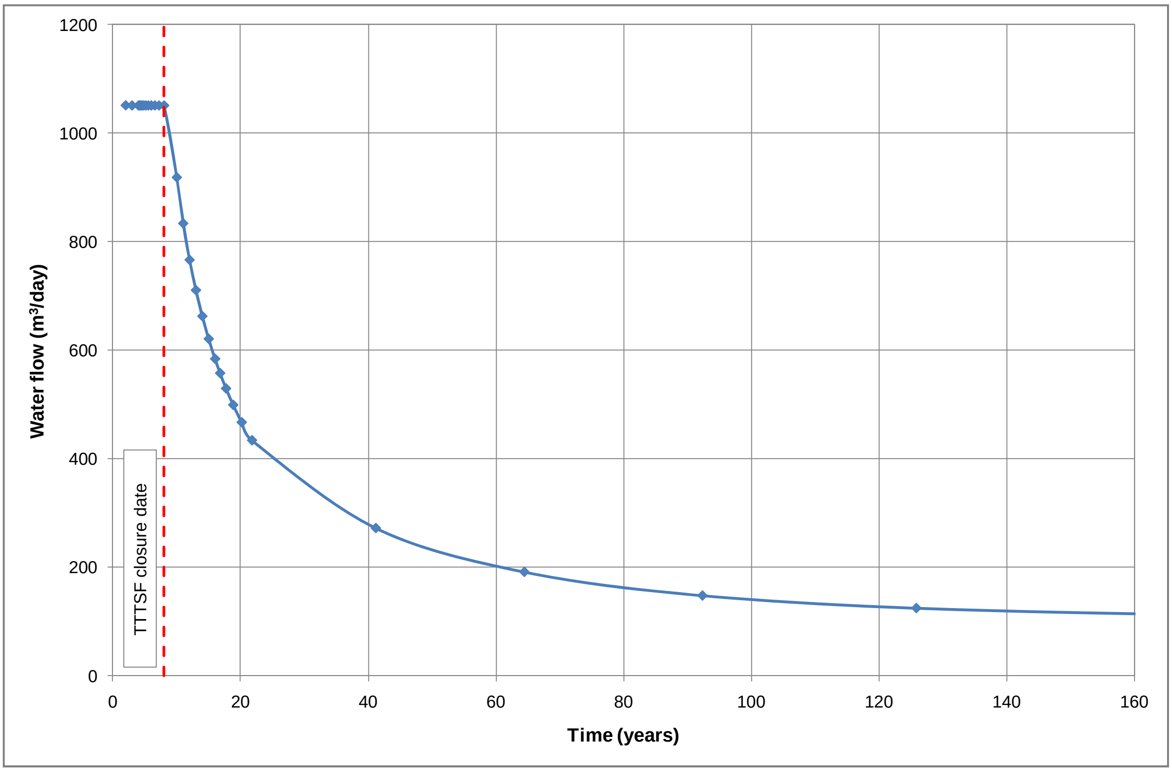

Figure 7: TTTSF - projected seepage flows to underdrain system.

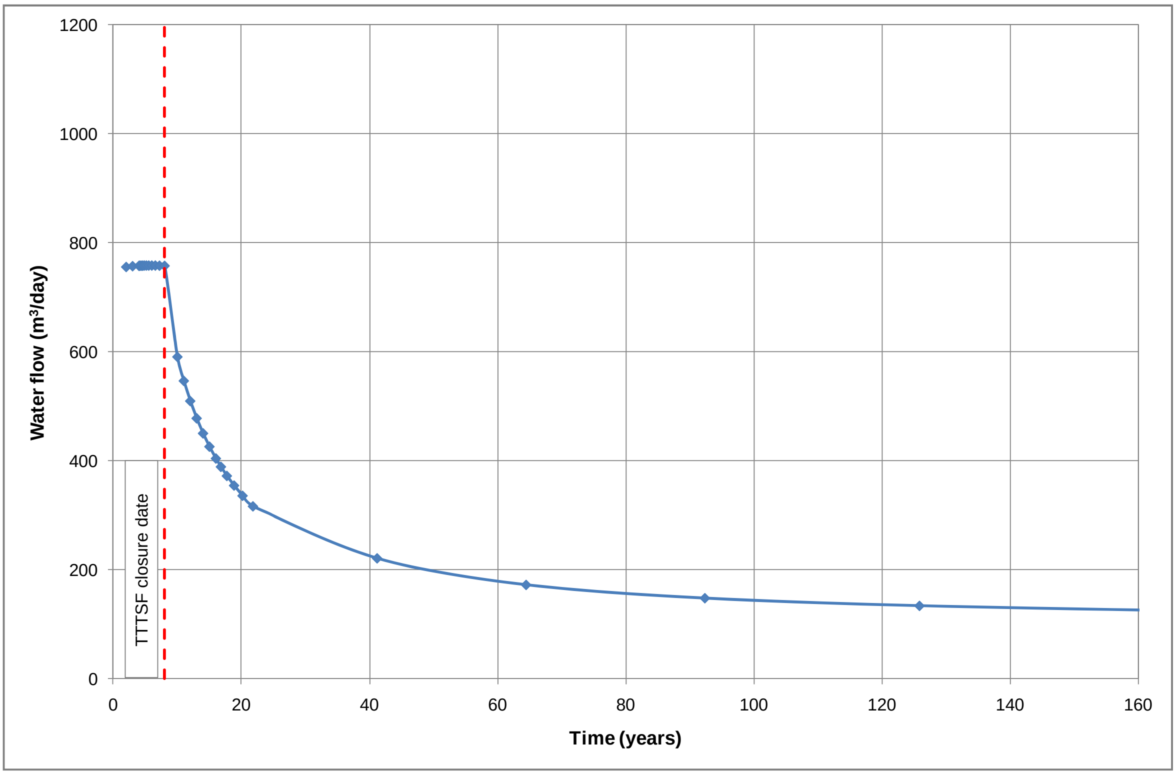

Figure 8: TTTSF – projected seepage flows to embankment drainage systems.

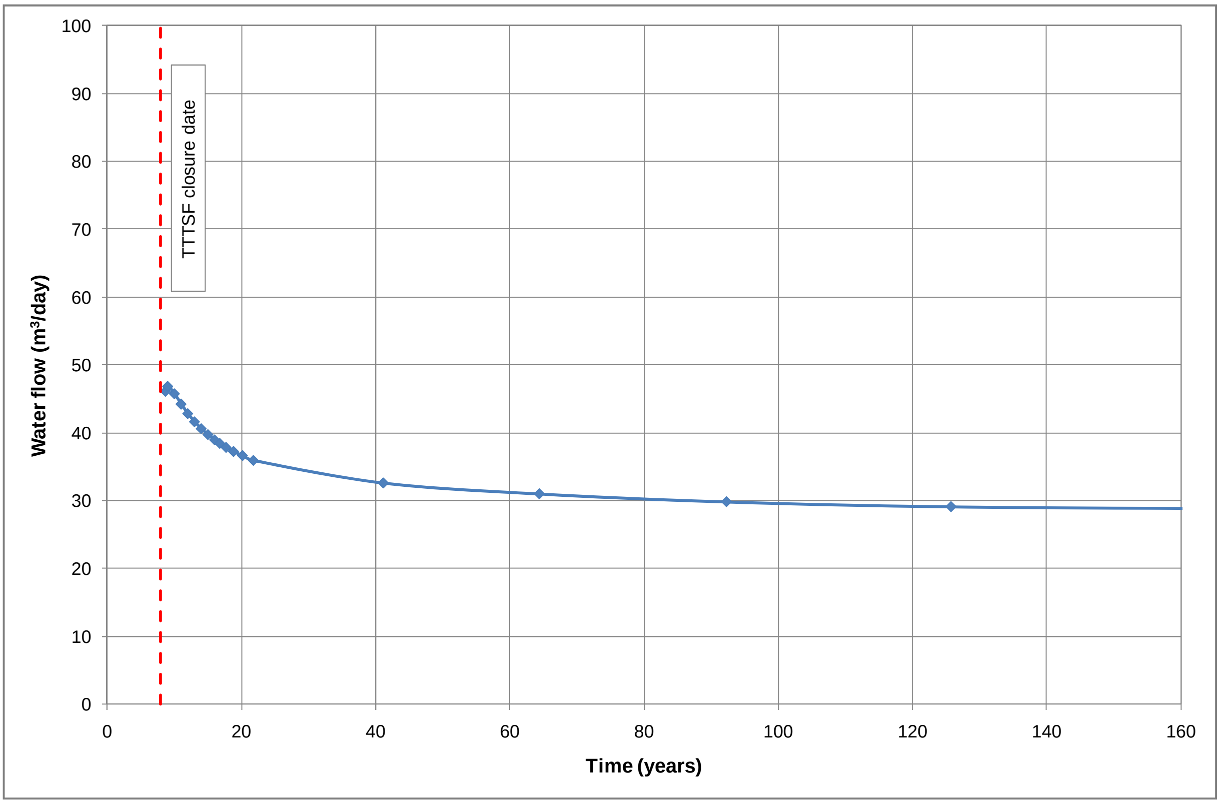

Figure 9: Tipperary Creek main channel – projected groundwater seepage flows.

A tributary of Tipperary Creek approaches the TTTSF from the south. This tributary receives most of the tailings seepage losses transported by groundwater in this direction. Simulated groundwater flows to this tributary peak at approximately 16 m^3/day during the operational period of the TSF and decrease rapidly following closure of the facility (Figure 10). Groundwater flows to this tributary creek are expected to decline to a long term rate of approximately 7 m^3/day.

Simulated groundwater discharges to other stretches of this western tributary to Tipperary Creek total approximately 300 m^3/day. These flows are not affected by the establishment of the TTTSF, however, the upper reaches receive seepage water from Frasers East WRS. These flows are not included in Figure 10.

Calculated seepage flows to Cranky Jims Creek are expected to peak during the operational period of the TTTSF at approximately 85 m^3/day (Figure 11) before declining to a long term seepage rate of approximately 55 m^3/day. These flows include discharges to each section of Cranky Jims Creek that may potentially receive seepage water from the TTTSF.

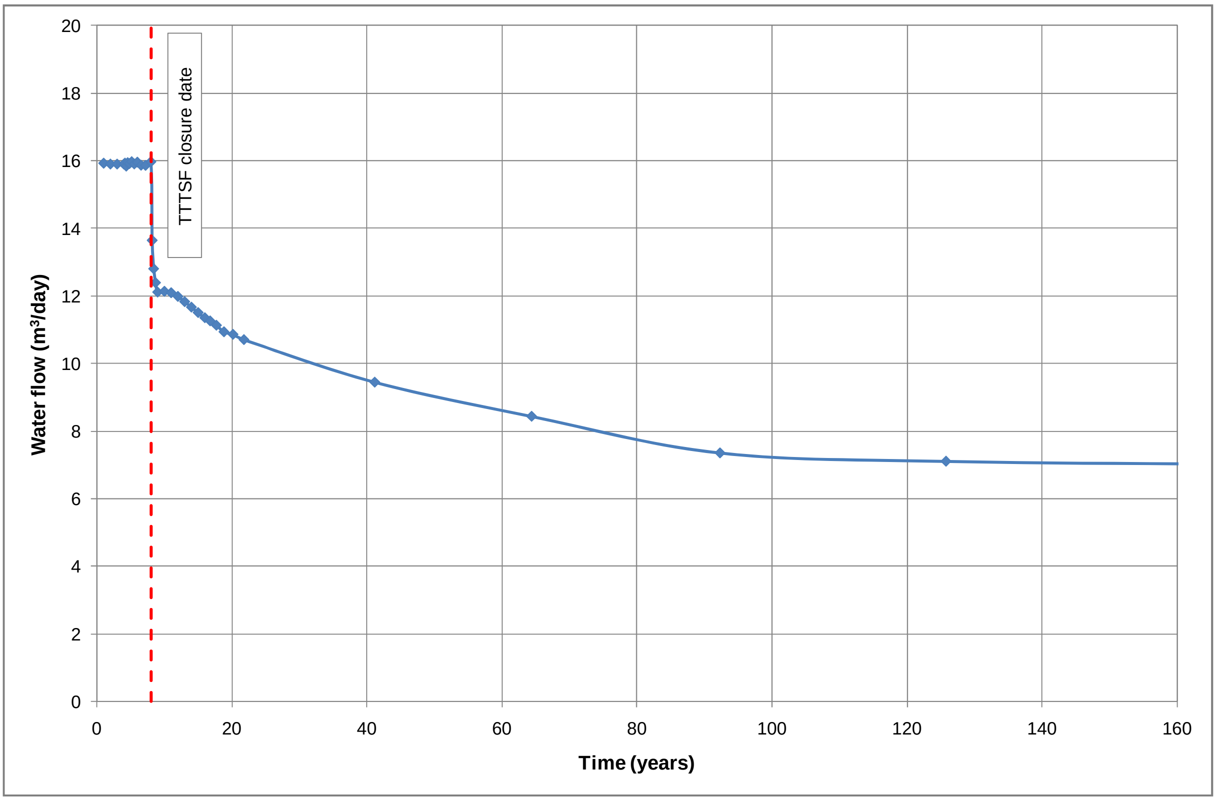

Figure 10: Tipperary Creek western tributary – projected groundwater seepage flows.

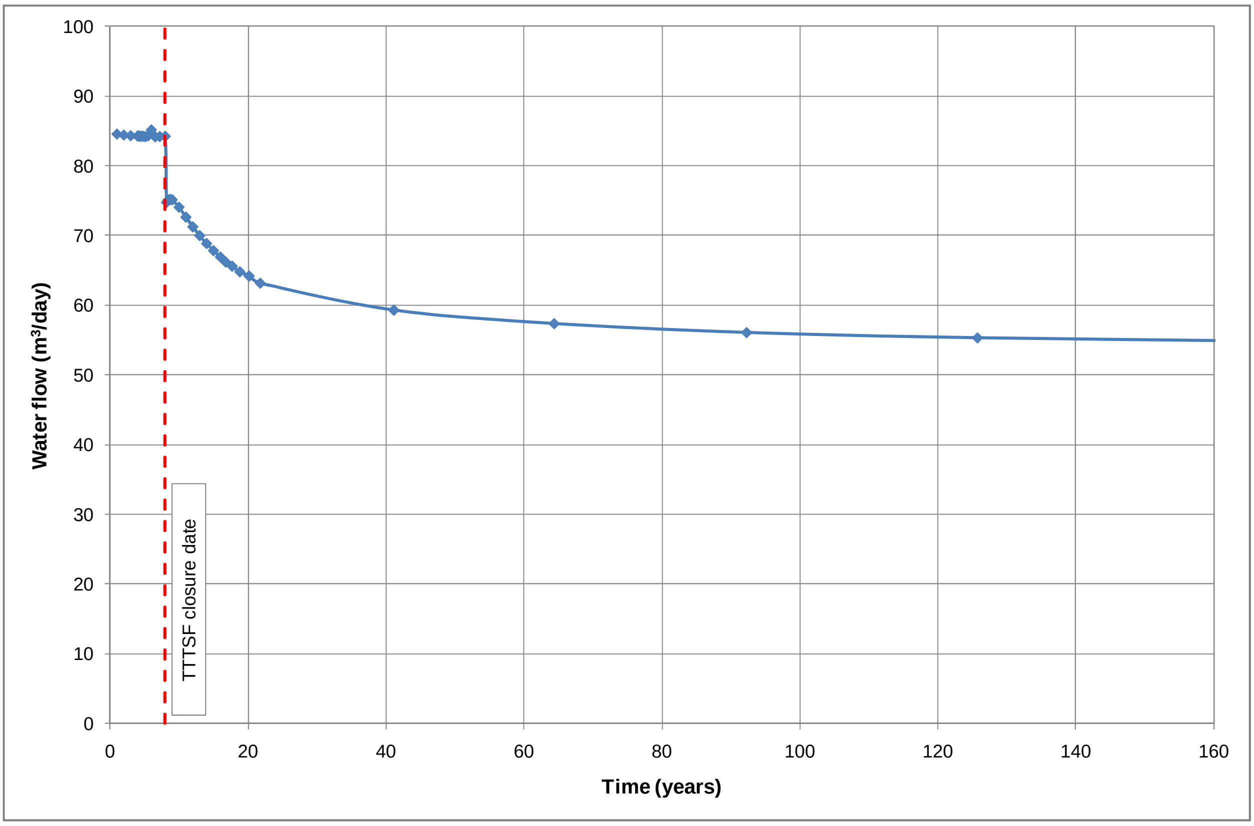

Figure 11: Cranky Jims Creek and tributaries – projected groundwater seepage flows.

6.5 Contaminant Mass Loads and Average Concentrations

6.5.1 Embankment drainage systems

For the purposes of this project, contaminant loads in water discharging from the TTTSF drainage systems have been assessed for the operational period and for the long term post-closure period rather than on a continuous basis (Table 10). Over the first 20 years following closure of the TSF, it is expected that contaminant mass loads would vary primarily in response to declines in the drain discharge flows rather than in response to water quality changes.

Table 10: Projected contaminant loads carried by TTTSF drain discharges.

Parameter	Operational period – peak loads		Post-closure period – long term loads	
	Concentration (1)	Mass load (2,3)	Concentration (1,6)	Mass load (2,4)
Sodium	498	900	416	100
Potassium	46	80	17	4
Calcium	411	740	410	110
Magnesium	245	440	200	50
Chloride	107	190	111	30
Sulphate (5)	2,769	5,000	**2,260** (560)	590 (150)
Arsenic (5)	5.4	10	**1** (0.15)	0.3 (0.04)
Copper	0.02	0.04	**0.02**	0.005
Iron	31	60	**21**	5
Lead	0.01	0.02	**0.013**	0.003
Zinc	0.02	0.04	0.009	0.002
Cyanide$_{WAD}$	0.35	0.6	**0.35**	0.09

Notes:
1) Concentrations presented in units of g/m^3. Refer Table 3 for source.
2) Mass loads presented in units of kg/day.
3) Calculated using discharge flow of 1,800 m^3/day.
4) Calculated using discharge flow of 260 m^3/day.
5) Values in brackets indicate concentrations indicated by laboratory analyses and the equivalent calculated mass loads (Golder 2011d).
6) Values in bold font exceed surface water compliance limits applied elsewhere at the MGP (refer Table 14).

6.5.2 Tipperary Creek

The simulated sulphate mass loads discharging to natural drainage channels of the Tipperary Creek catchment have been calculated separately for the main Tipperary Creek channel and for the western tributary. As previously discussed, the western tributary receives seepage flows from both the TTTSF and from Frasers East WRS.

The sulphate mass load transported in groundwater to the main channel of Tipperary Creek is expected to peak at approximately 19 kg/day before declining slowly (Figure 12). The average sulphate concentration in this groundwater discharge is calculated to be approximately 540 g/m^3 (Figure 13). As the long term sulphate concentration in seepage water from the stored tailings is expected to be similar to that during the operational period, the calculated long term concentrations do not decline significantly.

The main discharge zone for contaminants from the TTTSF is within approximately 300 m downstream from the toe of the embankment. Should these seepage discharges require management to mitigate for potential downstream water quality issues, a sump or other system could be installed to capture the groundwater discharges to this reach of Tipperary Creek. For the purposes of this report, a proposed Tipperary Sump is located approximately 300 m downstream from the toe of the TTTSF embankment. This location differs from that of a sump proposed by EGL for the management of TTTSF drain discharges. The eventual location of any sump installed for effects mitigation in Tipperary Creek can be finalised in conjunction with design work for a system to capture groundwater seepage in preference to stormwater run-off.

The shape of the discharge curves for mass loads and average concentrations for most of the other contaminants in the TTTSF seepage water are expected to be similar to that of sulphate. The main exception is arsenic, where the adsorption incorporated in the model results in practically no arsenic reaching the creek for much of the simulated period (Table 11).

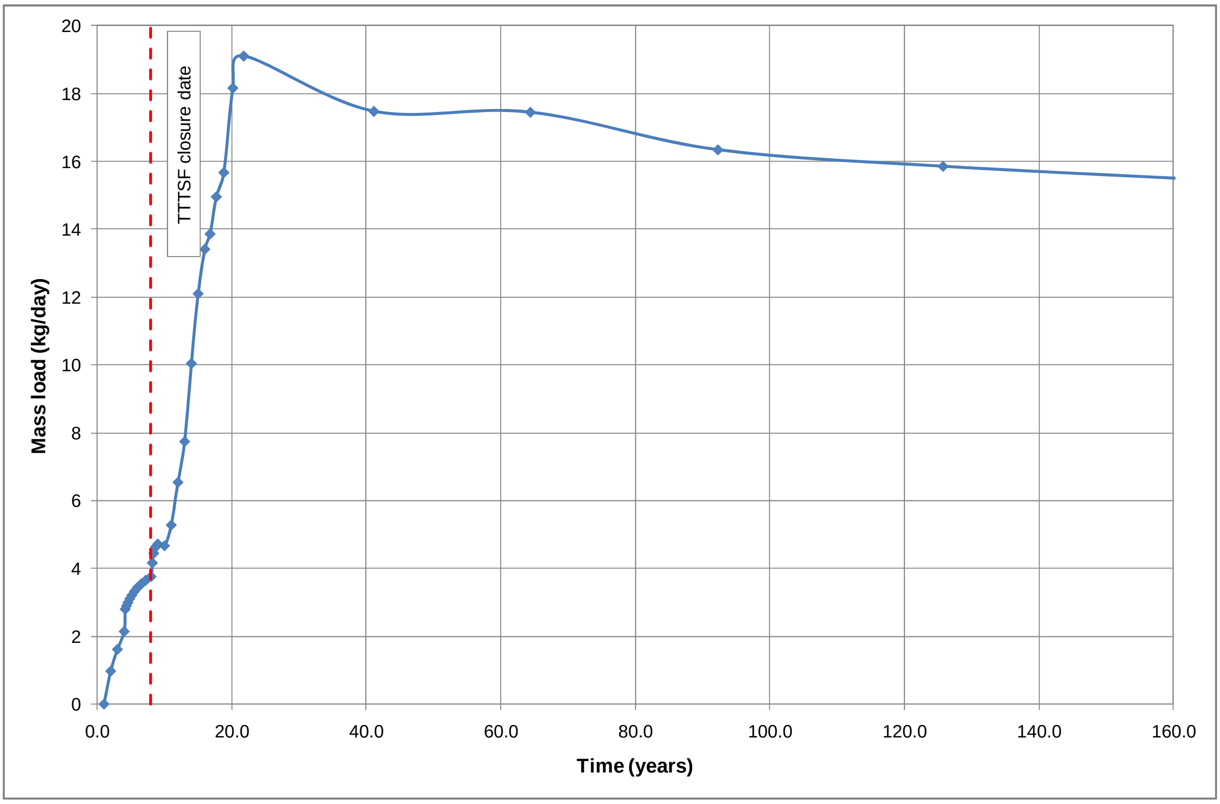

Figure 12: Tipperary Creek main channel – projected groundwater sulphate mass load.

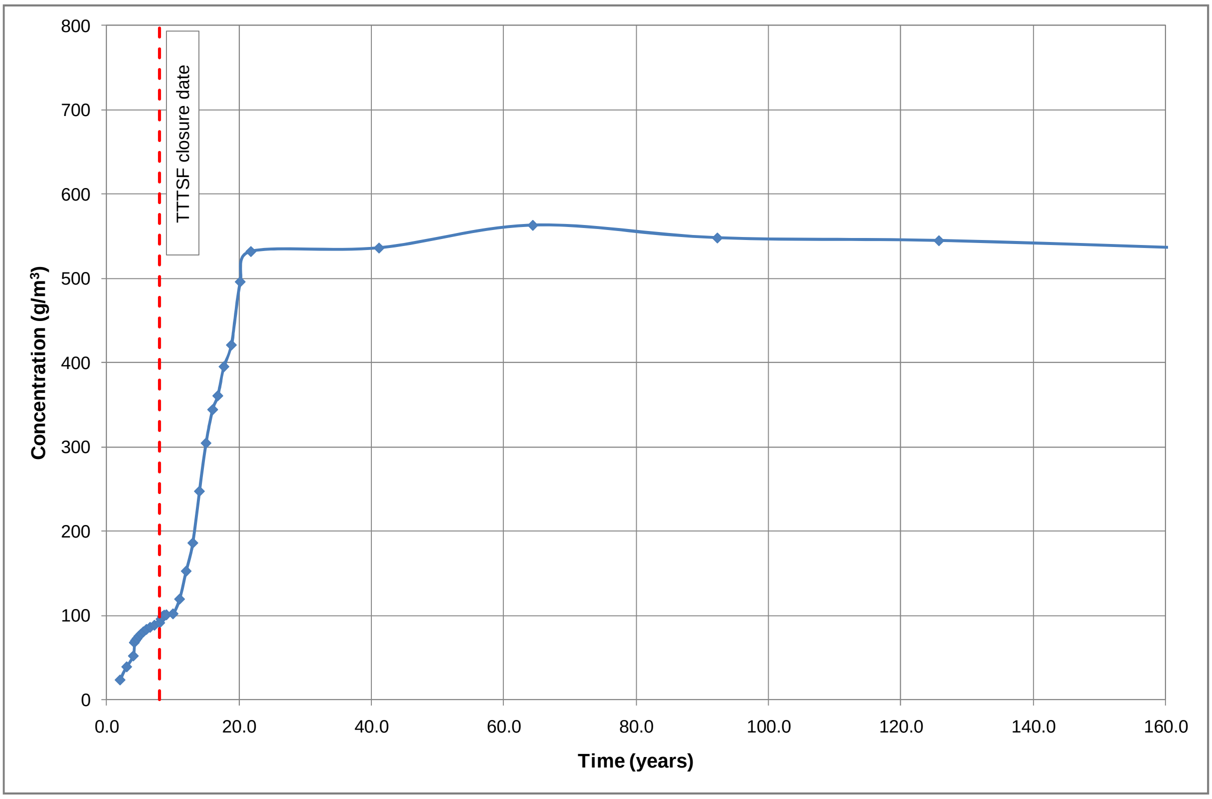

Figure 13: Tipperary Creek main channel – projected average groundwater sulphate concentration.

Table 11: Groundwater mass loads and average concentrations – Tipperary main channel.

Parameter [(1)]	**Contaminant mass load (kg/day)**		**Average concentration (g/m^3)**	
	Peak	**Long term**	**Peak**	**Long term**
Sodium	2.1	1.4	60	50
Potassium	0.12	0.09	3.5	3.2
Calcium	3.5	2.8	103	98
Magnesium	2.3	1.9	75	70
Chloride	0.3	0.2	9.5	8.3
Sulphate	19	16	560	540
Arsenic	<0.001	<0.001	<0.001	<0.001
Copper	<0.001	<0.001	0.002	0.001
Iron	0.12	0.08	3.5	2.9
Lead	<0.001	<0.001	<0.001	<0.001
Zinc	<0.001	<0.001	0.01	0.01
Cyanide$_{WAD}$	<0.001	<0.001	0.01	0.01

Note: 1) All parameters were modelled as being conservatively transported except arsenic. The results are likely to be conservatively large for cyanide$_{WAD}$, copper and zinc.

The sulphate mass load transported in groundwater to the western tributary of Tipperary Creek is expected to peak at about 8.6 kg/day (Figure 14). The mass load to this tributary is strongly affected by the short period during which the decant water level in the TTTSF exceeds the level of the ridge separating the TTTSF from the western tributary. In effect, a short pulse of tailings water is expected to reach this tributary, before seepage flows decrease again. Long term sulphate mass loads in groundwater discharging to this creek are expected to be approximately 3.5 kg/day (Figure 14). As with contaminants discharging to the main Tipperary Creek channel, the shape of the discharge curves for mass loads and average concentrations for most of the other contaminants are expected to be similar to that of sulphate (Table 12).

The Frasers East WRS bounds the western edge of the Tipperary Creek catchment and can be expected to eventually contribute slightly to the contaminant loads in the catchment. The extent to which the additional contribution from the WRS would increase the loads indicated for the western tributary (Table 12) is minor.

6.5.3 Cranky Jims Creek

Seepage water discharges from the tailings to natural drainage channels of the Cranky Jims Creek catchment are expected to peak within a few years of tailings closure. The simulated sulphate mass load discharging to these gullies however peaks at approximately 32 kg/day (Figure 15) and does not decrease substantially following the peak.

The extended period of the sulphate mass load curve to Cranky Jims Creek contradicts the concept that tailings seepage water should only be lost from the TTTSF toward this catchment during the period shortly before and shortly after the impoundment is full. Investigation of the model indicates seepage toward Cranky Jims catchment is dependent on the placement of the embankment drainage system along the catchment divide. The long term projections indicate average sulphate concentrations in the groundwater discharging to Cranky Jims Creek would peak at under 600 g/m^3 (Figure 16).

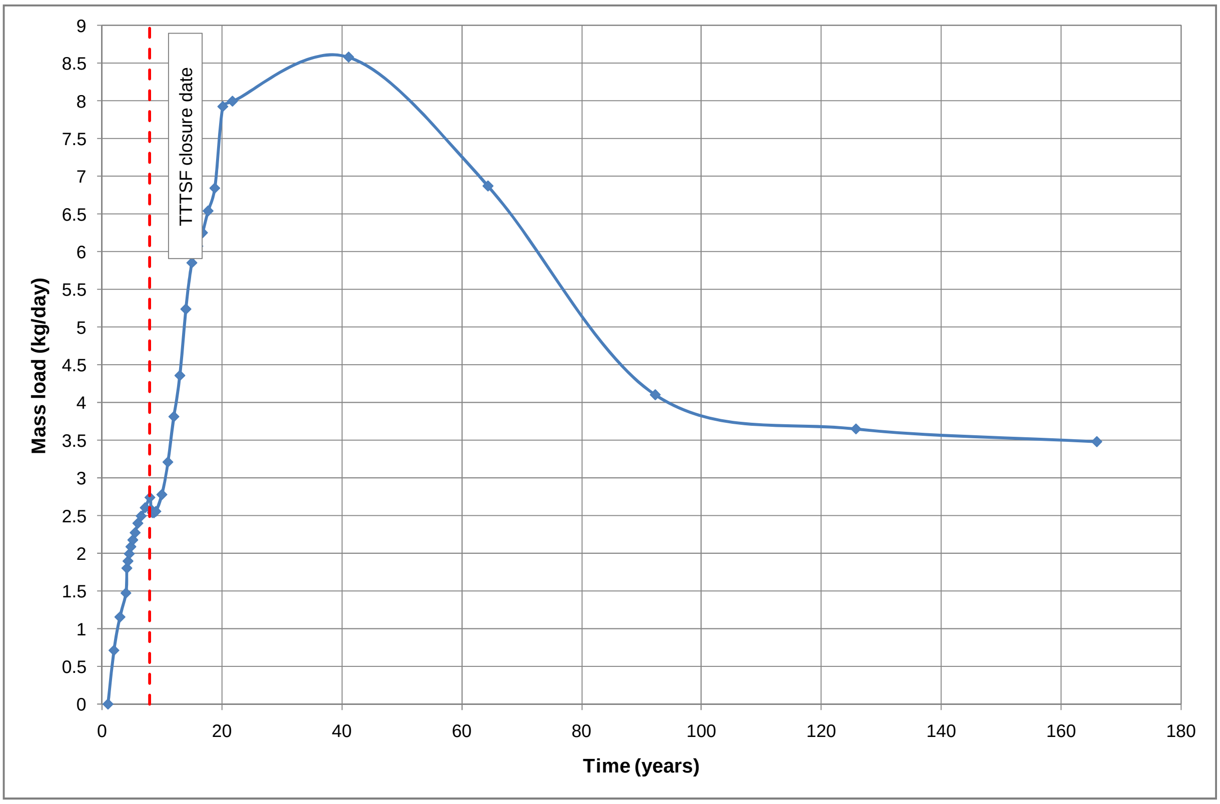

Figure 14: Tipperary Creek western tributary - projected groundwater sulphate mass load.

Table 12: Groundwater mass loads and average concentrations – Tipperary western tributary.

Parameter [1]	Contaminant mass load (kg/day)		Average concentration (g/m^3)	
	Peak	**Long term**	**Peak**	**Long term**
Sodium	0.9	0.1	86	18
Potassium	0.06	0.02	5.9	2.7
Calcium	1.6	0.65	170	93
Magnesium	1.2	0.5	125	70
Chloride	0.15	0.02	14	2.8
Sulphate	8.6	3.5	910	500
Arsenic	<0.001	<0.001	<0.001	<0.001
Copper	<0.001	<0.001	0.002	<0.001
Iron	0.06	0.005	5.2	0.7
Lead	<0.001	<0.001	0.001	<0.001
Zinc	<0.001	<0.001	0.02	0.01
Cyanide$_{WAD}$	<0.001	<0.001	0.03	<0.001

Note: 1) All parameters were modelled as being conservatively transported except arsenic. The results are likely to be conservatively large for cyanide$_{WAD}$, copper and zinc.

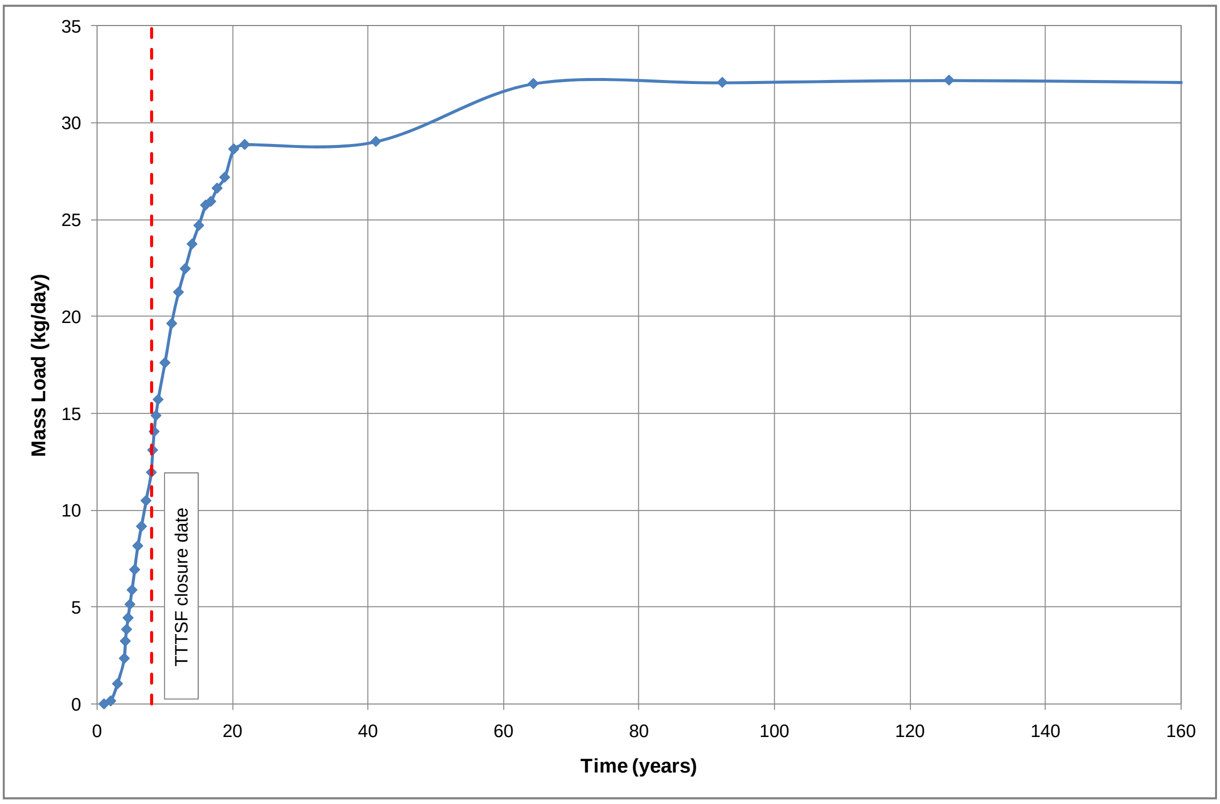

Figure 15: Cranky Jims Creek and tributaries - projected groundwater sulphate mass load.

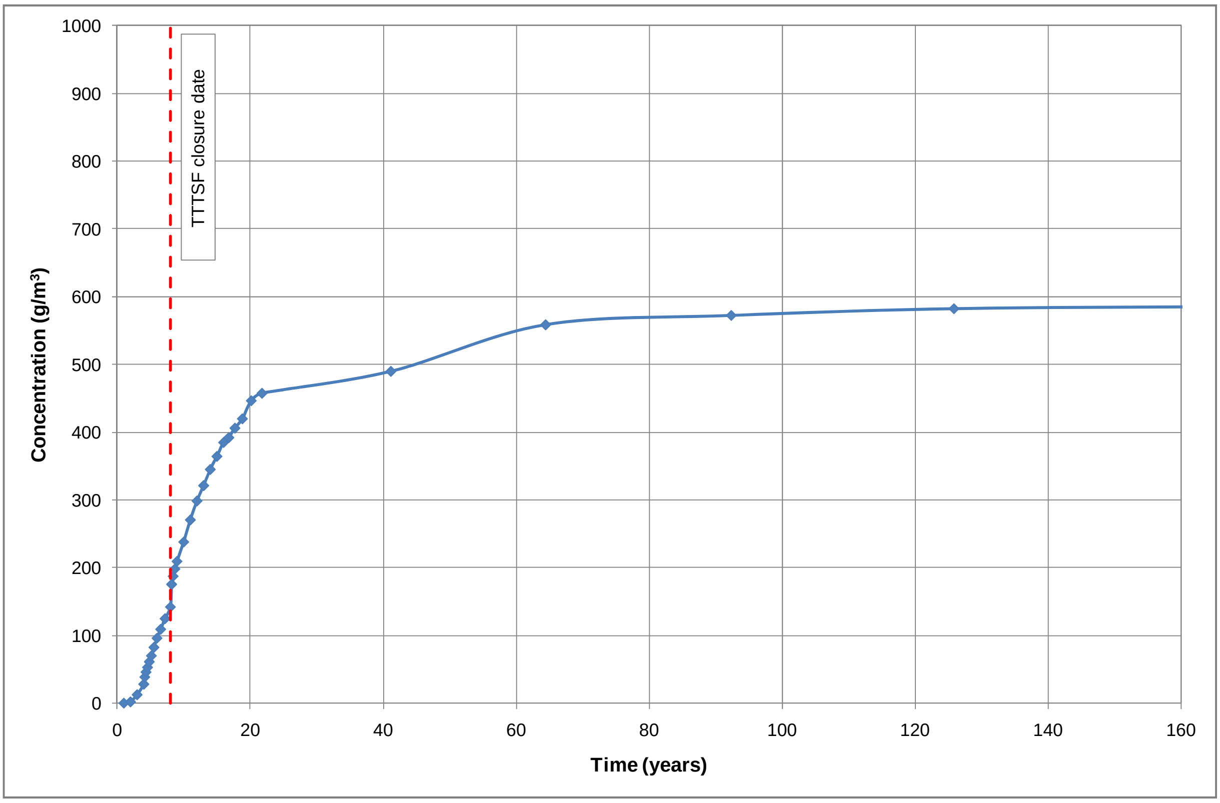

Figure 16: Cranky Jims Creek and tributaries - projected average groundwater sulphate concentration.

The model documented in this report has the embankment chimney drain system positioned practically on top of the catchment divide, allowing seepage to pass beneath it out toward the east. An earlier iteration of the model for a slightly different design of embankment had the drainage system positioned deeper within the Tipperary catchment. The seepage losses toward Cranky Jims Creek indicated by the earlier model were similar up to the 20 year point, following which they declined.

Based on the above information, modification of the design drainage system in the TTTSF embankment in the area between Tipperary and Cranky Jims catchments can potentially reduce seepage and contaminant losses toward Cranky Jims Creek. For the purposes of this project, variations of drainage systems to potentially reduce these seepage flows have not been investigated further.

Simulated contaminant concentrations were monitored at a range of points along the Cranky Jims Creek tributaries. The peak projected contaminant concentrations from any monitored point (Table 13) indicate sulphate and iron are the only simulated parameters that may exceed the water quality consent limits that currently apply to Deepdell Creek. The limits for copper, lead and zinc are however pH dependent and are expected to vary over time (refer Section 7.3). The maximum point concentrations listed in Table 13 are considerably higher than the peak groundwater concentrations averaged through the Cranky Jims Creek tributaries that receive leachate from the TTTSF. For example, model outputs indicate the peak average sulphate concentration is less than 600 g/m^3 (Figure 16) whereas the maximum point concentration is approximately 1,720 g/m^3 (Table 13).

Table 13: Cranky Jims Creek – maximum projected single point groundwater concentration.

Parameter (1)	Maximum projected concentration (g/m^3)	Deepdell Creek consent limit
Sodium	140	
Potassium	12	
Calcium	320	
Magnesium	260	
Chloride	24	
Sulphate	1,740	1,000
Arsenic	<0.001	0.15
Copper	0.004	0.009 (2)
Iron	7.7	1.0
Lead	0.002	0.0025 (2)
Zinc	0.03	0.12 (2)
Cyanide$_{WAD}$	0.05	0.1

Note: 1) All parameters in units of g/m^3.
2) Metal limits hardness adjusted as per Section 7.3.

7.0 DISCUSSION

7.1 Seepage Flows to Deepdell Creek and Frasers Pit

The groundwater model indicates very limited or no seepage flows from the TTTSF are likely to discharge to either Frasers Pit or Deepdell Creek. The tailings level in the TTTSF does not exceed the elevation of the catchment divides separating the Tipperary catchment from those of the NBWR and Deepdell Creek.

In addition, the construction of the WRS's along the northern and western boundaries of the TTTSF is expected to result in the groundwater level along the catchment divides to rise slightly. These increases in the groundwater levels should serve to reinforce the hydraulic separation of the three catchments in this area.

7.2 Decline in Discharge Flows

As discussed in Section 6.3, the simulated rates at which the discharge flows to drainage systems constructed in the TTTSF are expected to be substantially slower than would actually occur at closure of this facility. The same is not necessarily correct for the calculated rates at which groundwater discharges to natural channels decline. The long term discharge flows to the natural channels are driven by relatively low hydraulic gradients and the considerable volume of water stored in the tailings mass.

Experience from the MGP indicates the mass of relatively coarse tailings close to the embankments would become mostly dewatered within perhaps a couple of years following closure (Golder 2011e). The long term discharges to the surrounding receiving water bodies are, however, primarily a consequence of the dewatering time required for finer tailings closer to the centre of the impoundment.

7.3 Existing MGP Water Quality Compliance Limits

The outcomes of the groundwater flow and contaminant transport modelling have been compared to proposed surface water quality limits applicable at the proposed compliance monitoring point TC01 (refer Golder 2011h). The model outcomes indicate that the average contaminant concentrations for groundwater discharges to the natural receiving waters around the TTTSF are below the limits proposed for TC01 (Table 14). Dilution of the groundwater discharges through surface run-off would reduce the peak simulated concentrations further as discussed in the site wide surface water modelling report (Golder 2011h).

Table 14: MGP water quality compliance limits.

Parameter (1)	**Groundwater compliance wells** (2)	**Proposed compliance monitoring sites** (3)	
		TC01	**Shag River at McCormicks**
pH (unitless)	6.0 – 9.5	6.0 – 9.5	7.0 – 8.5
Sulphate	-	1,000	250
Arsenic	0.15	0.15	0.01
$\text{Cyanide}_{\text{WAD}}$	0.10	0.10	0.10
Copper (4)	0.009	0.009	0.009
Iron	1.0	1.0	0.2
Lead (4)	0.0025	0.0025	0.0025
Zinc (4)	0.12	0.12	0.12

Notes:
1) Units g/m^3 unless otherwise stated.
2) Values proposed are based on limits applicable in groundwater at compliance wells in Maori Tommy Gully.
3) Refer to Golder (2011h) for sources of proposed consent limit concentrations.
4) Metal limits hardness adjusted as per equations 1 to 3 below.

The water quality limits for copper, lead and zinc are hardness dependent based on the following equations:

1) Copper (g/m^3) = $(0.96\exp^{0.8545[\ln(\text{hardness})] - 1.702}) / 1000$.

2) Lead (g/m^3) = $(1.46203 - [\ln(\text{hardness})(0.145712)]\exp^{1.273[\ln(\text{hardness})] - 4.705}) / 1000$.

3) Zinc (g/m^3) = $(0.986\exp^{0.8473[\ln(\text{hardness})] + 0.884}) / 1000$.

Projected contaminant concentrations in the TSF drain discharges would exceed the existing surface water compliance limits for several parameters including sulphate, arsenic and $\text{cyanide}_{\text{WAD}}$ (Table 10).

7.4 TSF Monitoring Wells

It is expected that monitoring wells will be required to be installed around the TTTSF for groundwater sampling purposes. As is the practice elsewhere at the MGP, detection wells may be installed to enable monitoring of expected contaminant plumes sourced from the TTTSF, evaluation of contaminant attenuation rates and confirm expectations of contaminant mass loads to receiving waters. Compliance wells may be installed close to the receiving water bodies to confirm that contaminant losses to creeks are within acceptable limits.

In terms of the primary contaminant discharge routes it is expected that detection wells would be installed in Tipperary Creek upstream from the proposed Tipperary Sump. The sump is expected to be the collection point for groundwater discharges to the main Tipperary Creek channel and possibly the discharge flows from the TTTSF drainage systems. Compliance wells are proposed to be installed downstream from the Tipperary Sump to verify that contaminant losses from the TTTSF are mainly discharging to the valley upstream from the sump and can thereby be collected.

Detection and compliance wells may also be installed down-gradient from the TTTSF embankment where it overlooks the western tributary of Tipperary Creek and Cranky Jims Creek. These wells would be used to verify the nature of the contaminant plumes in each of these directions. In the case of Cranky Jims Creek, detection wells should also be installed in the highly fractured zones of the Macraes Fault Zone to enable monitoring of any seepage losses along this structural feature.

Monitoring of groundwater pressures along the catchment divide between the Tipperary and Deepdell Creeks may be undertaken to monitor the position of the hydraulic catchment divide during and immediately following the operational period of the TTTSF. This monitoring would provide data to verify that contaminants from the TTTSF are not transported in the direction of Deepdell Creek.

8.0 SUMMARY

Modelling of contaminant mass transport within the Tipperary Creek catchment groundwater system has been undertaken to cover the period from the start of tailings storage in the TTTSF until 150 years following closure of the facility. Beyond that period, potential changes in the hydrogeological behaviour of the tailings material and climactic conditions are considered to limit the usefulness of predictive modelling.

Groundwater flow modelling for the Tipperary Creek catchment and the TTTSF has been based on existing calibrated groundwater models covering the MGP operations to date. Water quality parameters applied to the contaminant transport model have been based on environmental monitoring data from the MGP, laboratory analysis of tailings leachate water quality and laboratory analysis of the capacity of soils and rocks at the site to adsorb arsenic.

Contaminant transport for each of the simulated contaminants with the exception of arsenic has been undertaken on the basis of conservative transport within the groundwater system. Arsenic transport has been modelled based on arsenic (III) being the main form of this element in the tailings seepage water. The adsorption parameters for arsenic (III) have been derived from testing of rock and soil samples from the site. The results of the tests have been applied to the contaminant transport model in the form of:

- A K_d of 2×10^{-5} L/mg, applicable to the model zones simulating weathered schist and loess and the underlying moderately to slightly weathered schist.
- Maximum arsenic adsorption of 230 mg/kg for the upper 20 m of the weathered schist and loess.
- Maximum arsenic adsorption of 46 mg/kg for the moderately to slightly weathered schist between approximately 20 m and 60 m below the ground surface.
- Conservative transport of arsenic at depths below 60 m.

Simulated groundwater discharge flows to simulated sections of the natural receiving water channels are summarised in Table 15.

Table 15: Groundwater discharge flows to receiving waters.

Receiving water body	Short term flows (m^3/day)	Long term flows (m^3/day)
TTTSF drainage systems	1,800	260
Tipperary Creek main channel	46 (1)	30
Tipperary Creek western tributary	16 (2) (300) (3)	7 (2) (300) (3)
Cranky Jims Creek	85 (1)	55

Notes:
1) Operational TTTSF period and up to two years following closure.
2) Flows to single gully directly influenced by seepage water from TTTSF.
3) Figure in brackets for wider area of western tributary to Tipperary Creek which is unaffected by TSF seepage flows.

The contaminant transport simulation indicates that the TTTSF drainage system operational flows may peak at approximately 1,800 m^3/day and subsequently decrease to a long term flow of approximately 260 m^3/day. The operational and long term contaminant mass loads transported by the discharge water are calculated to be:

- Sulphate: operation load 5,000 kg/day long term load 2,260 kg/day.
- Arsenic: operation load 10 kg/day long term load 0.3 kg/day.
- $\text{Cyanide}_{\text{WAD}}$: operation load 0.6 kg/day long term load 0.09 kg/day.

The peak loads and concentrations for other simulated parameters are provided in Table 10.

The contaminant transport simulation indicates that groundwater flows to the main channel of Tipperary Creek may peak at approximately 46 m^3/day and subsequently decrease to a long term flow of approximately 30 m^3/day. The operational and long term contaminant mass loads transported by the groundwater flows are calculated to be:

- Sulphate: peak load 19 kg/day long term load 16 kg/day.
- Arsenic: peak load <0.001 kg/day long term load <0.001 kg/day.
- $\text{Cyanide}_{\text{WAD}}$: peak load <0.001 kg/day long term load <0.001 kg/day.

The peak loads and concentrations for other simulated parameters are summarised in Table 11.

The contaminant transport simulation indicates that groundwater flows to the western tributary of Tipperary Creek may peak at approximately 316 m^3/day and subsequently decrease to a long term flow of approximately 307 m^3/day. The operational and long term contaminant mass loads transported by the groundwater flows are calculated to be:

- Sulphate: peak load 8.6 kg/day long term load 3.5 kg/day.
- Arsenic: peak load <0.001 kg/day long term load <0.001 kg/day.
- $\text{Cyanide}_{\text{WAD}}$: peak load <0.001 kg/day long term load <0.001 kg/day.

The peak loads and concentrations for other simulated parameters are summarised in Table 12.

The contaminant transport simulation indicates that groundwater flows to Cranky Jims Creek may peak at approximately 85 m^3/day and subsequently decrease to a long term flow of approximately 55 m^3/day. Due to the complexity of the receiving water channels the groundwater discharges have generally been reported as maximum point concentrations to any of the simulated tributaries. The operational and long term sulphate mass load transported by the groundwater flows to the Cranky Jims Creek catchment is calculated to be approximately 32 kg/day. The maximum single point concentrations simulated for discharges to Cranky Jims Creek are:

- Arsenic: <0.001 g/m^3.
- Cyanide$_{WAD}$: 0.05 g/m^3.

The maximum single point concentrations for other simulated parameters are summarised in Table 13.

9.0 CONCLUSIONS

The effects of contaminant loss from the TTTSF have been evaluated with respect to:

- The drainage systems built into the TTTSF, including underdrains installed beneath the tailings and chimney drains constructed in the embankment.
- Tipperary Creek and tributary gullies, including those reaches managed through the use of sediment settling ponds and unmanaged sections.
- Cranky Jims Creek and tributary gullies, including those reaches managed through the use of sediment settling ponds and unmanaged sections.

Transport of each of the simulated contaminants with the exception of arsenic has been undertaken on the basis of conservative transport within the groundwater system. Arsenic transport has been modelled based on arsenic (III) being the main form of this element in the tailings seepage water. The adsorption parameters for arsenic (III) have been derived from testing of rock and soil samples from the site and have been incorporated in the groundwater model.

A surface water compliance point is proposed to be established at TC01, downstream from all simulated contaminant discharge points to Tipperary Creek (Golder 2011h). The outcomes of the groundwater flow and contaminant transport modelling indicate that the contaminant concentrations in groundwater discharges to Tipperary Creek, excluding the TSF drain discharges, are below the proposed surface water quality limits applicable at TC01. Dilution of the groundwater discharges through surface run-off would reduce the peak simulated concentrations further.

The contaminant concentrations in TTTSF drain discharge water are calculated to exceed the proposed TC01 compliance limits for arsenic, sulphate, cyanide$_{WAD}$, copper, iron and lead. These discharges are to be managed through the application of mitigation measures as discussed in other reports (Golder 2011h, i).

A surface water compliance point is proposed to be established at CJ01, downstream from all simulated contaminant discharge points to Cranky Jims Creek (Golder 2011h). The outcomes of the groundwater flow and contaminant transport modelling indicate that the contaminant concentrations in groundwater discharges to Cranky Jims Creek are below the proposed surface water quality limits applicable at CJ01.

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APPENDIX A

Report limitations

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APPENDIX B

Hydraulic test documentation

1.0 INTRODUCTION

A program of packer delineated hydraulic tests in investigation drillholes was undertaken in the area of the proposed Top Tipperary tailings storage facility (TTTSF) area between 3 June 2010 and 25 June 2010. The objective of the test program was to assess whether the schist in the TTTSF area has similar permeability to schist in other areas of the Macraes Gold Project.

2.0 PACKER TESTS AT TTTSF

The packer test data analysis sheets are attached to this appendix. Data derived from the hydraulic test program was analysed using standard methods. A summary of the results is presented in Table 1. Graphic representations of the results plotted against the test interval depth below the ground surface are presented in Figure 1 and Figure 2. In Figure 3 and Figure 4 the same charts are plotted together with the hydraulic conductivity applied to the uppermost layers of the MODFLOW model used to simulate groundwater flow across the TTTSF site.

The minimum measurable flow was approximately 0.1 L/min. This flow rate, which is considered to be the sensitivity limit for the test procedure using the available equipment, equates to a hydraulic conductivity of 3×10^{-9} m/s. In effect, if the injection rate during a test is below the detection limit of the equipment, this implies the rock mass surrounding the test interval has a hydraulic conductivity of less than 3×10^{-9} m/s.

Of the 16 packer tests undertaken:

- 10 tests produced data that could be analysed to produce hydraulic conductivity results.
- 2 tests did not generate injection flows that were measurable with the equipment available on the drill rig.
- 2 tests resulted in seepage flows bypassing the packer and discharging to the surface. These tests did not generate useful data for analysis.
- 2 tests resulted in water losses from the test interval that exceeded the capacity of the pump on the rig. In general this implies seepage bypassing the packer or some other preferred pathway is present for seepage to the ground surface. These tests did not generate useful data for analysis.

3.0 HYDRAULIC TEST DATA FROM SURROUNDING AREAS

3.1 Tipperary TSF Investigations

Early in the operational period of the Macraes Gold Project investigations were undertaken into the feasibility of the construction and operation of a tailings storage facility in the Tipperary Creek catchment to the south of the planned TTTSF. A series of packer delineated injection tests were performed on investigation drillholes within the footprint of the planned TSF. The results of the tests (Woodward Clyde 1996) are summarised in Table 2. The results are presented in comparison to the depth of the test interval below the ground surface in Figure 5.

Table 1: Top Tipperary TSF packer test results summary.

Drillhole	Test interval			Hydraulic conductivity		Comments
	Top	Base	Length	(Golder 1997)	(Sharp 1975)	
	(mbgl)	(mbgl)	(m)	(m/s)	(m/s)	
TT001 (DDH5201)	14.23	20.23	6.00	4×10^{-6}	7×10^{-6}	
	44.23	50.13	5.90	4×10^{-7}	7×10^{-7}	
TT002 (DDH5194)	11.10	17.10	6.00			Flows exceed pump capacity at 50 kPa.
	32.10	38.10	6.00			Flows bypass packer and discharge to surface.
TT003 (DDH5200)	5.31	11.31	6.00	5×10^{-6}	8×10^{-6}	
	25.36	31.36	6.00	1×10^{-6}	2×10^{-6}	
TT004 (DDH5195)	5.00	11.00	6.00	1×10^{-6}	2×10^{-6}	
	23.00	30.50	7.50	3×10^{-7}	5×10^{-7}	
TT005 (DDH5199)	20.11	25.83	5.72	3×10^{-7}	5×10^{-7}	
	44.13	50.13	6.00			Flows exceed pump capacity at 20 kPa.
TT006 (DDH5198)	11.00	17.00	6.00	3×10^{-9}	5×10^{-9}	Hydraulic conductivity value is considered a maximum as no flows recorded.
	23.00	30.82	7.82	3×10^{-9}	5×10^{-9}	Hydraulic conductivity value is considered a maximum as no flows recorded.
TT007 (DDH5197)	11.22	17.22	6.00			Flows bypass packer and discharge to surface.
	23.22	30.12	6.90	2×10^{-7}	4×10^{-7}	
TT008 (DDH5196)	11.00	17.00	6.00	4×10^{-7}	6×10^{-7}	
	23.12	30.62	7.50	4×10^{-7}	5×10^{-7}	

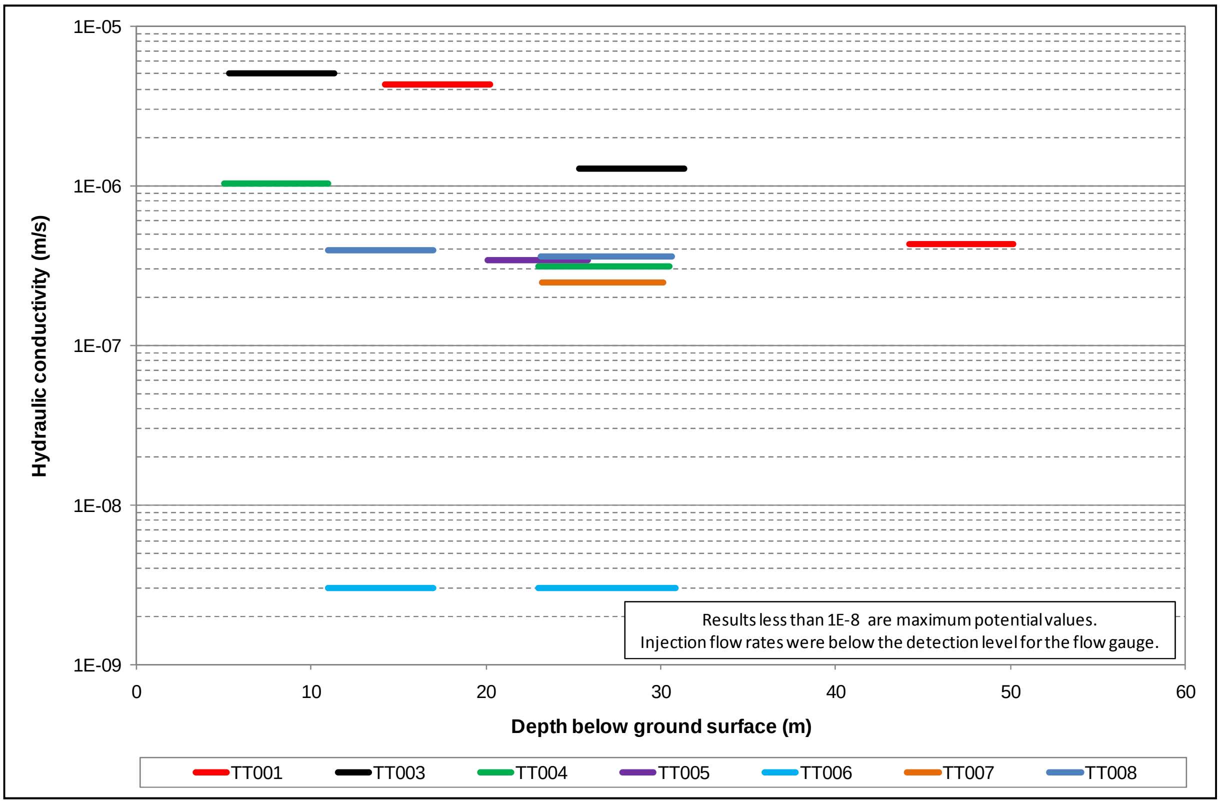


Figure 1: Packer test results plotted against depth of test interval below ground surface, including detection limit results.

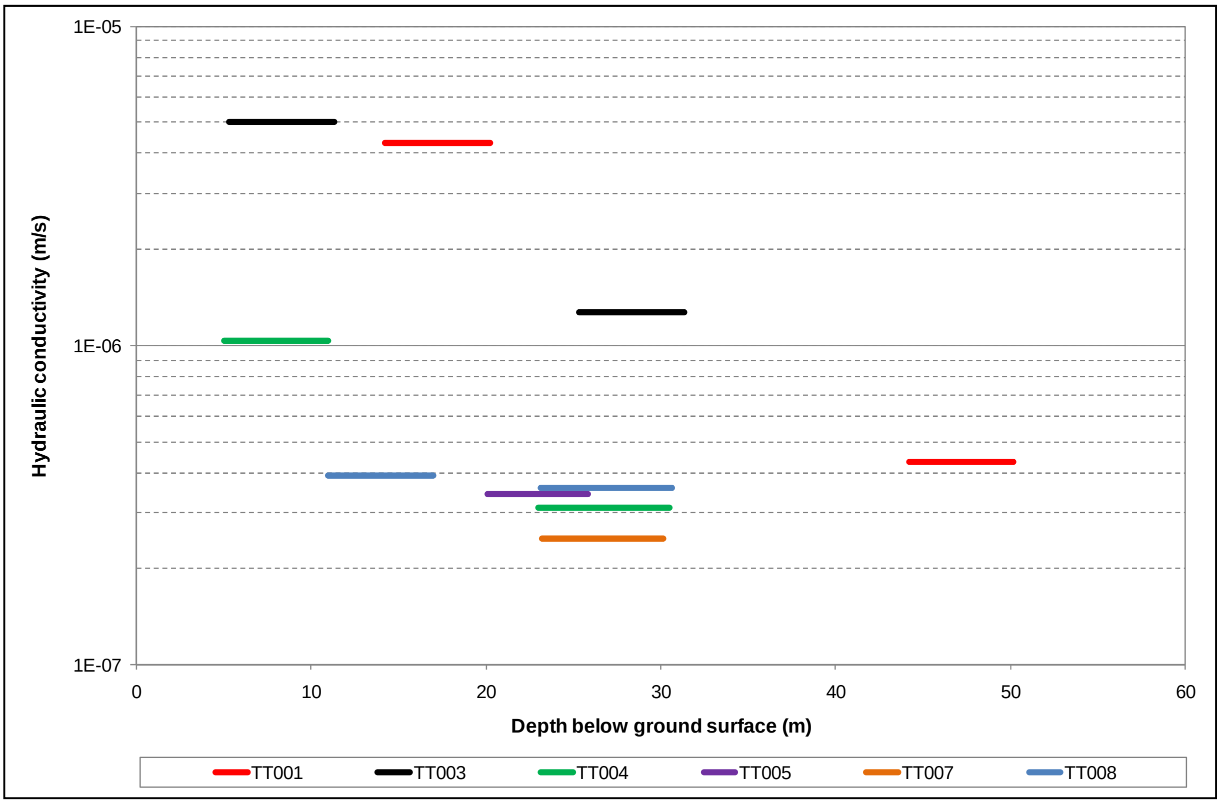


Figure 2: Packer test results plotted against depth of test interval below ground surface, excluding detection limit results.

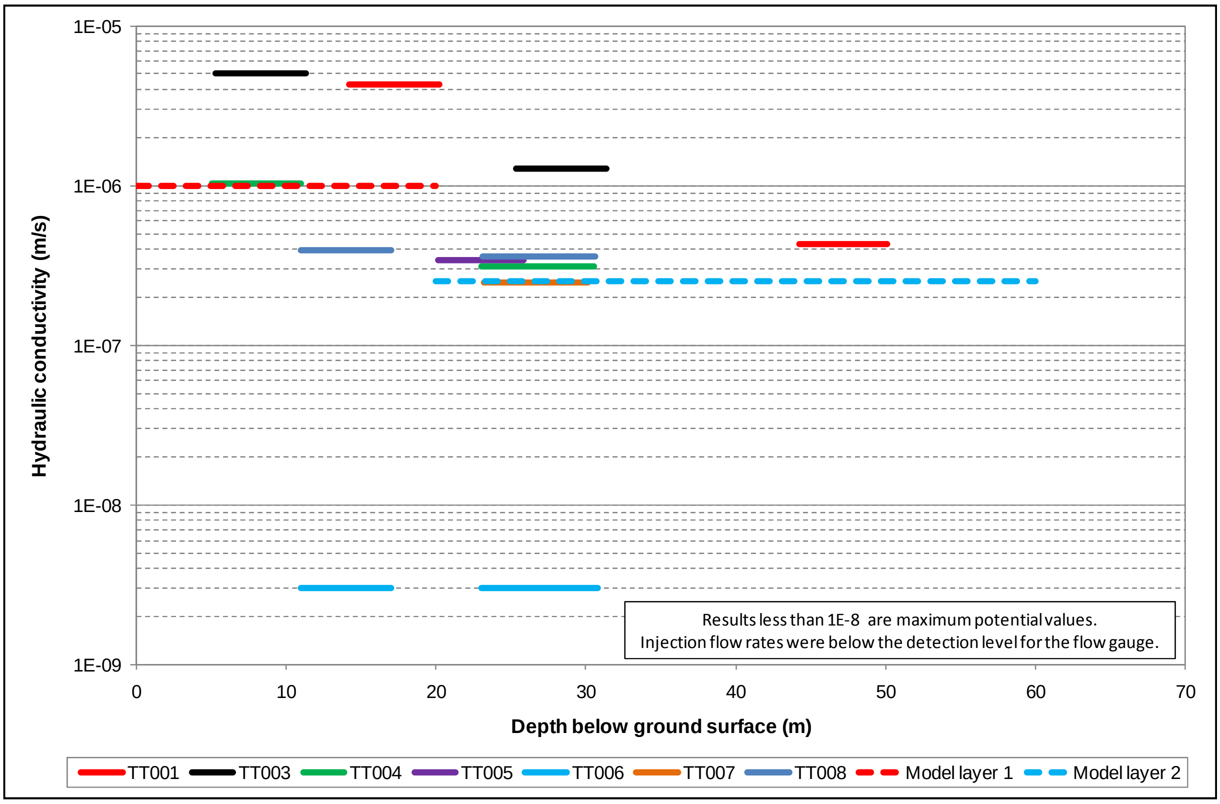


Figure 3: Packer test results and MODFLOW model permeability parameters including detection limit results.

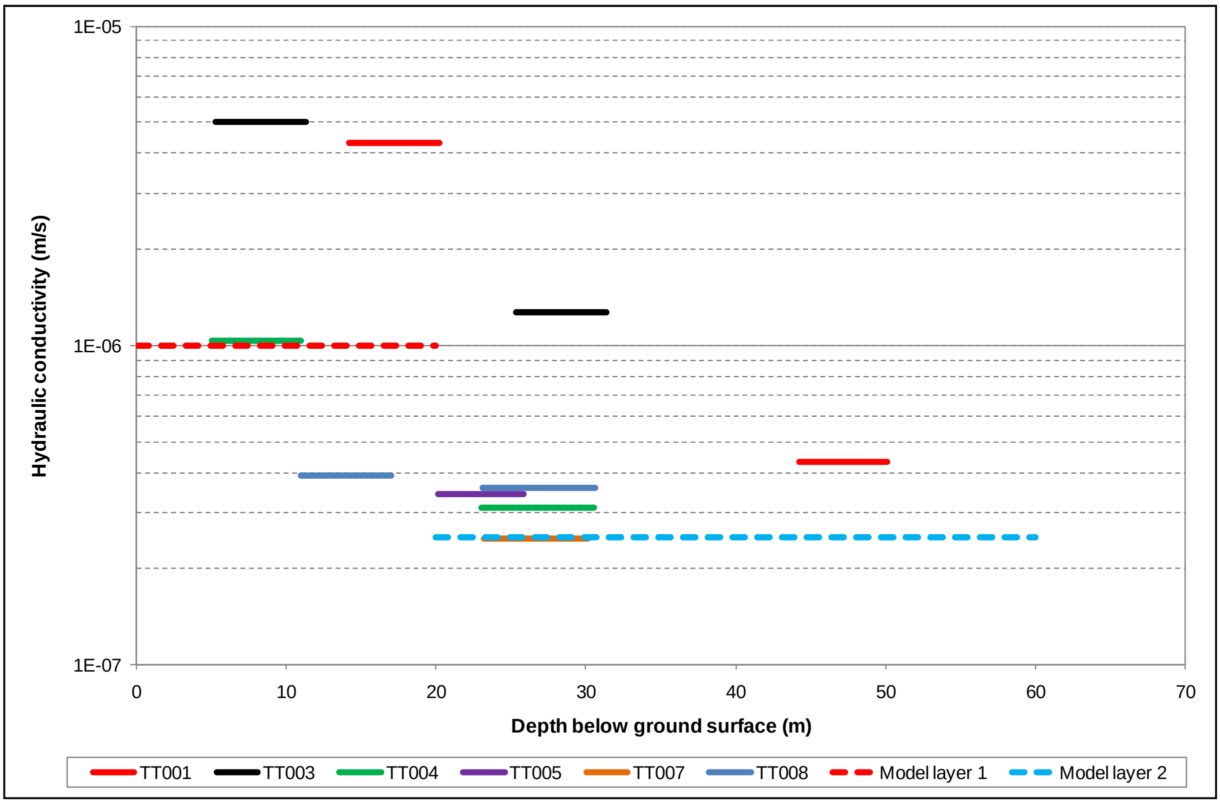


Figure 4: Packer test results and MODFLOW model permeability parameters excluding detection limit results.

Table 2: Tipperary TSF investigation hydraulic test summary

Drillhole number	Easting (1)	Northing (1)	Test top (mbgl)	Test base (mbgl)	Length (m)	Hydraulic conductivity (m/s)
TC01	71800	10700	8.3	13.1	4.8	<1 x 10^{-7} (2)
			25.6	31.5	5.9	1.3 x 10^{-7}
			56.3	61.5	5.2	<1 x 10^{-7} (2)
			83.1	88.5	5.4	3.9 x 10^{-7}
			65.1	88.5	23.4	<1.3 x 10^{-7} (2)
TC02	71920	10790	7.5	12.9	5.4	<1 x 10^{-7} (2)
			25.25	30.65	5.4	1.3 x 10^{-6}
			53.2	59.6	6.4	1.3 x 10^{-6}
			74.2	79.6	5.4	<1 x 10^{-7} (2)
TC06	70480	11310	5.5	11.12	5.62	1.3 x 10^{-6}
			20.6	26.3	5.7	2 x 10^{-6}
			35.41	40.03	4.62	7 x 10^{-7}
			44.72	50.12	5.4	<1.3 x 10^{-7} (2)
TC07	70450	10800	6.6	12	5.4	1.3 x 10^{-6}
			14	18	4	1.3 x 10^{-5}
			23.55	30.1	6.55	5.2 x 10^{-5}
TC08	72110	10970	7.63	13.03	5.4	<1 x 10^{-7} (2)
			25.9	31.3	5.4	<1 x 10^{-7} (2)
			56.72	62.12	5.4	<1 x 10^{-7} (2)
			74.65	99.61	24.96	<1.3 x 10^{-7} (2)
TC09	72350	10700	11.74	17.14	5.4	<1.3 x 10^{-7} (2)
			24.66	30.06	5.4	<1.3 x 10^{-7} (2)
			55.19	60.59	5.4	3.9 x 10^{-7}
			96.75	102.15	5.4	7 x 10^{-7}

Note: 1) Locations approximate only.
2) Detection limit of test.

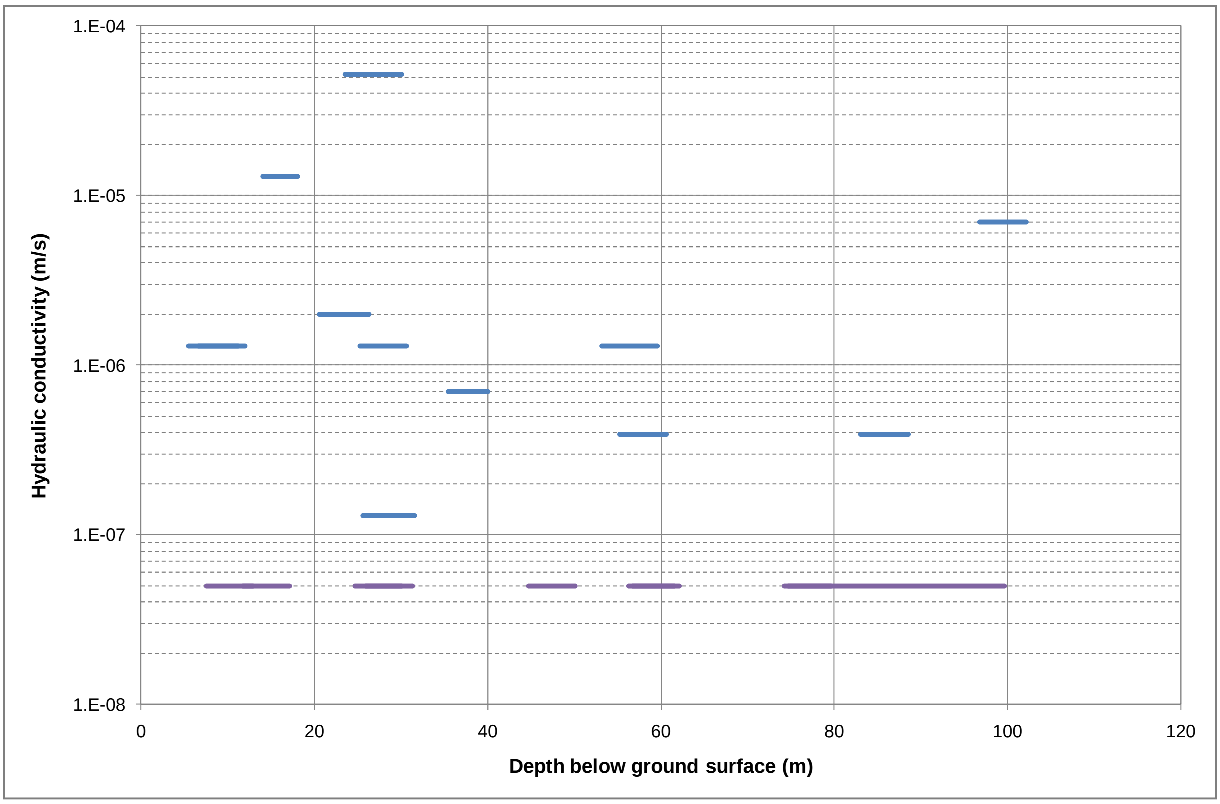


Figure 5: Packer test results – Tipperary TSF investigation – 1996.

3.2 Back Road investigations

Geotechnical investigations in the area of the planned Back Road WRS have been undertaken by Engineering Geology Ltd (EGL). These investigations included the drilling and logging of investigation holes at the site. Drilling was undertaken by McNeill Drilling Co. Ltd.

Packer delineated hydraulic tests were performed by McNeill Drilling on three of the investigation drillholes in the Back Road WRS area. The locations of the tested drillholes are summarised in Table 3.

The tests were undertaken during the period 5 June to 18 June 2009 and the data analysed by Golder (Golder 2009). The analysis sheets for these tests are attached to this appendix. The hydraulic conductivity analysis results are summarised in Table 4.

Table 3: Back Road investigation drillhole locations.

Drillhole ID	Easting	Northing	RL
DH03	71681.555	14343.856	498.817
DH06	72274.442	13900.691	509.253
DH09	71520.017	13159.639	572.348

Table 4: Back Road packer test analysis results.

Drillhole ID	Test Number	Depth from	Depth to	Hydraulic conductivity (1)
		(mbgl)	(mbgl)	(m/s)
BH3	Test 1	11.76	20.31	1.7×10^{-7}
BH3	Test 2	25.26	35.31	7.0×10^{-7}
BH3	Test 3	40.21	50.31	4.2×10^{-8}
BH6	Test 1	12.76	20.31	2.7×10^{-7}
BH6	Test 3	27.95	35.31	1.1×10^{-6}
BH6	Test 4	40.30	50.31	1.3×10^{-6}
BH9	Test 1	10.26	20.31	1.7×10^{-6}
BH9	Test 3	29.76	35.31	2.2×10^{-6}

Note: 1) Hydraulic conductivity value presented is calculated after Sharp (1975). Equation presented in analysis sheets.

4.0 MODFLOW MODEL INPUT PARAMETERS

The hydraulic conductivity applied to the MODFLOW model simulating groundwater flow across the TTTSF area is anisotropic, with a higher value applied in the north-south direction than to the east-west direction. This anisotropy is applied to reflect the presence of faults that are oriented parallel to the Macraes grid north-south axis. In addition, the hydraulic conductivity in both horizontal directions is greater than that in the vertical direction, reflecting the shallow dip of the schist foliation and foliation parallel joints toward the east.

The hydraulic conductivity values applied to the upper 20 m of schist simulated in past models of the MGP site were 3.7×10^{-7} m/s and 1×10^{-6} m/s. This compares well to the logarithmic mean of the results from test intervals at depths less than 20 m below the ground surface, which is 5×10^{-7} m/s.

The values applied to schist simulated for depths between 20 m and 60 m below the ground surface in past existing models were 5×10^{-8} m/s and 2.5×10^{-7} m/s. This also compares well to the logarithmic mean of the results for test intervals at depths greater than 20 m, which is 2×10^{-7} m/s.

In general, where two successful tests were performed in a single drillhole hole, the hydraulic conductivity decreases with increasing depth. This pattern of decreasing hydraulic conductivity with increasing depth is however not reflected in the data from the previous test program performed in the Tipperary catchment (Table 2) or that from the Back Road WRS area (Table 4). The variability of the test results reflects the limited volume of rock being put under hydraulic stress during these packer tests. The degree of variability is not considered to be excessive and is comparable to that generated from previous packer test programs at the MGP site.

Investigation drillhole DDH5198 is located in an area of sheared and faulted schist associated with the Macraes Fault Zone. The results of two hydraulic tests performed in this drillhole were both below the detection limit of the equipment. These results indicate disruption of the normal foliation in the schist reduces the permeability of the rock mass.

5.0 CONCLUSIONS

The test results indicate the permeability of the schist at the TTTSF site does not differ substantially from that of schist elsewhere within the MGP area. The Macraes Fault appears to be a zone of reduced permeability in comparison to the schist rock mass outside the fault zone. As such, the Macraes Fault is not considered to constitute a preferred pathway by which tailings pore water may escape the TTTSF.

Golder Associates

WATER PRESSURE TEST (5 Stage)

Revision Nº : 18

Job Nº : 0978110-562
Client : OceanaGold NZ Ltd
Project : Macraes Flat Expansion
Location : Macraes Gold Project
Tested By : McNeill Drilling
Date : 25/06/2010

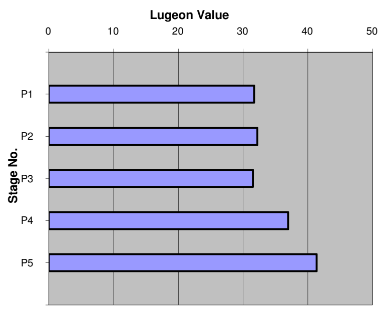
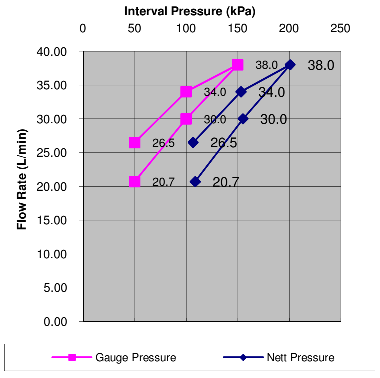
Hole Nº : TT001
Dip (Deg) : -90
DH Interval Top (m) : 14.23
DH Interval Base (m) : 20.23
Computed By : P. Mangeya
Date : 19/07/2010

Drilling Method : HQ
Hole Diameter (m) : 0.096
DH Tested Length (m) : 6.00
Packer Type: Pneumatic - Wireline - Single
Rock tested : Otago Schist
Water Meter Reading in Litres
Checked By : B. Sinclair

Vertical depth to Groundwater
Immediately prior to test (m bgl) : 5.53
Used in analysis (m bgl) : 5.53
Pressure Gauge Height (m agl) : 0.80
Presumed Water Temperature : 5
Casing Inner Diameter (mm) : 77.80
Date : 19/07/2010

Pressure Stage	Gauge Pressure kPa	No	Actual Time (h:m:s)	Time Intervals (min)	Water Meter Readings Reading (Litres)	Volume (L)	Discharge (L/min)	Meter discharge - Leakage (L/min)	Discharge/m (L/min/m)	Remarks	
										c :	1
		0	0:00:00	0	111.0	0.00	0.00	0.00	0.00	Start Date/Time:	
		1	0:05:00	05:00	218.0	107.00	21.40	21.40	3.57		
		2	0:10:00	05:00	318.0	100.00	20.00	20.00	3.33		
		3									
		4									
P1	50	5									
		6									
		7									
		8									
		9									
		10									
						Total :	41.40	41.40	6.90		
						Average:	20.700	20.700	3.450	Leak Flow Rate =	0
		0	0:15:00	0	370.0	0.00	0.00	0.00	0.00	c :	1
		1	0:20:00	05:00	520.0	150.00	30.00	30.00	5.00		
		2	0:25:00	05:00	670.0	150.00	30.00	30.00	5.00		
		3									
		4									
P2	100	5									
		6									
		7									
		8									
		9									
		10									
						Total :	60.00	60.00	10.00		
						Average:	30.000	30.000	5.000	Leak Flow Rate =	0
		0	0:30:00	0	720.0	0.00	0.00	0.00	0.00	c :	1
		1	0:35:00	05:00	920.0	200.00	40.00	40.00	6.67		
		2	0:40:00	05:00	1100.0	180.00	36.00	36.00	6.00		
		3									
		4									
P3	150	5									
		6									
		7									
		8									
		9									
		10									
						Total :	76.00	76.00	12.67		
						Average:	38.000	38.000	6.333	Leak Flow Rate =	0
		0	0:45:00	0	220.0	0.00	0.00	0.00	0.00	c :	1
		1	0:50:00	05:00	390.0	170.00	34.00	34.00	5.67		
		2	0:55:00	05:00	560.0	170.00	34.00	34.00	5.67		
		3									
		4									
P4	100	5									
		6									
		7									
		8									
		9									
		10									
						Total :	68.00	68.00	11.33		
						Average:	34.000	34.000	5.667	Leak Flow Rate =	0
		0	0:55:00	0	600.0	0.00	0.00	0.00	0.00	c :	1
		1	1:00:00	05:00	733.0	133.00	26.60	26.60	4.43	Finish Date/Time:	
		2	1:05:00	05:00	865.0	132.00	26.40	26.40	4.40		
		3									
		4									
P5	50	5									
		6									
		7									
		8									
		9									
		10									
						Total :	53.00	53.00	8.83		
						Average:	26.500	26.500	4.417	Leak Flow Rate =	0

TEST RESULTS

Stage No.	Lugeon (1933) Value	Lugeon Value Curve	Nett Pressures	Pressure Vs Flow
P1	31.7		108.8	
P2	32.2		155.2	
P3	31.5		201.0	
P4	37.0		153.2	
P5	41.4		106.7	

Lugeon (1933) method includes nett pressure in calculation that is Hp (gauge pressure) + Hg (height gauges above watertable) - Hf (head losses).

Interpreted Result & Hydraulic Conductivity

H_{LOSS} 7.12 kPa
Stage No. Average
Gauge Pressure 90 kPa
Q 30 L/min
H 15 m
Interpreted Result 35 uL

Reported k at Stage Average

Analytical Method 1: (ref = Golder geotechnical field notes draft 1997) k =
$Q/H \times 6.10889 \times 10^{-6} x((\log(2L/D))/L)$

k = 4.3E-06 m/s

Analytical Method 2: (ref = Sharp, J.C 1975 Pit Slope Manual, CANMET report). k = 1/(2Lx3.14) x (Q/H) ln(R/r) m/s (convert L/min to m/s). Assume R = radius of influence of 100m & r = radius of borehole.

k = 6.8E-06 m/s

Flow Type: LAMINAR FLOW

COMMENTS:

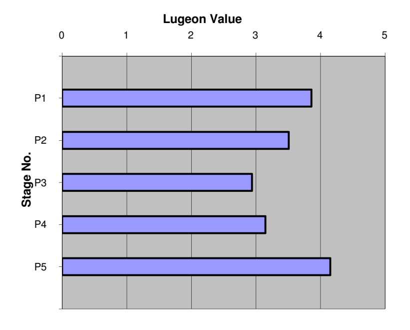
WATER PRESSURE TEST (5 Stage)

						Revision Nº :	18
Job Nº :	0978110-562	Hole Nº :	TT001	Drilling Method :	HQ	Vertical depth to Groundwater Immediately prior to test (m bgl) :	9.17
Client :	OceanaGold NZ Ltd	Dip (Deg) :	-90	Hole Diameter (m) :	0.096	Used in analysis (m bgl) :	9.17
Project :	Macraes Flat Expansion	DH Interval Top (m) :	44.23	DH Tested Length (m) :	5.90	Pressure Gauge Height (m agl) :	0.80
Location :	Macraes Gold Project	DH Interval Base (m) :	50.13	Packer Type:	Pneumatic - Wireline - Single	Presumed Water Temperature :	5
Tested By :	McNeill Drilling	Computed By :	P. Mangeya	Rock tested :	Otago Schist	Casing Inner Diameter (mm) :	77.80
Date :	25/06/2010	Date :	19/07/2010	Water Meter Reading in	Litres	Checked By : B. Sinclair	Date : 19/07/2010

Pressure Stage	Gauge Pressure kPa	No	Actual Time (h:m:s)	Time Intervals (min)	Water Meter Readings Reading (Litres)	Volume (L)	Discharge (L/min)	Meter discharge - Leakage (L/min)	Discharge/m (L/min/m)	Remarks	
										c :	1
		0	0:00:00	0	935.5	0.00	0.00	0.00	0.00	Start Date/Time:	
		1	0:05:00	05:00	958.2	22.70	4.54	4.54	0.77		
		2	0:10:00	05:00	980.5	22.30	4.46	4.46	0.76		
		3									
		4									
P1	100	5									
		6									
		7									
		8									
		9									
		10									
						Total :	9.00	9.00	1.53		
						Average:	4.500	4.500	0.763	Leak Flow Rate =	0
		0	0:15:00	0	986.0	0.00	0.00	0.00	0.00	c :	1
		1	0:20:00	05:00	1012.5	26.50	5.30	5.30	0.90		
		2	0:25:00	05:00	1047.6	35.10	7.02	7.02	1.19		
		3									
		4									
P2	200	5									
		6									
		7									
		8									
		9									
		10									
						Total :	12.32	12.32	2.09		
						Average:	6.160	6.160	1.044	Leak Flow Rate =	0
		0	0:30:00	0	54.0	0.00	0.00	0.00	0.00	c :	1
		1	0:35:00	05:00	88.4	34.40	6.88	6.88	1.17		
		2	0:40:00	05:00	123.0	34.60	6.92	6.92	1.17		
		3									
		4									
P3	300	5									
		6									
		7									
		8									
		9									
		10									
						Total :	13.80	13.80	2.34		
						Average:	6.900	6.900	1.169	Leak Flow Rate =	0
		0	0:45:00	0	133.2	0.00	0.00	0.00	0.00	c :	1
		1	0:50:00	05:00	160.0	26.80	5.36	5.36	0.91		
		2	0:55:00	05:00	188.5	28.50	5.70	5.70	0.97		
		3									
		4									
P4	200	5									
		6									
		7									
		8									
		9									
		10									
						Total :	11.06	11.06	1.87		
						Average:	5.530	5.530	0.937	Leak Flow Rate =	0
		0	0:55:00	0	194.5	0.00	0.00	0.00	0.00	c :	1
		1	1:00:00	05:00	218.6	24.10	4.82	4.82	0.82	Finish Date/Time:	
		2	1:05:00	05:00	242.9	24.30	4.86	4.86	0.82		
		3									
		4									
P5	100	5									
		6									
		7									
		8									
		9									
		10									
						Total :	9.68	9.68	1.64		
						Average:	4.840	4.840	0.820	Leak Flow Rate =	0

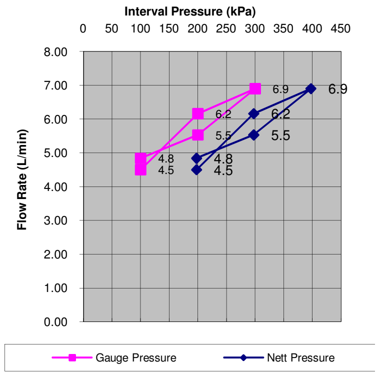
TEST RESULTS

Stage No.	Lugeon (1933) Value	Lugeon Value Curve	Nett Pressures
P1	3.9		197.6
P2	3.5		297.5
P3	2.9		397.4
P4	3.2		297.5
P5	4.2		197.6



Lugeon (1933) method includes nett pressure in calculation that is Hp (gauge pressure) + Hg (height gauges above watertable) - Hf (head losses).

Pressure Vs Flow



Interpreted Result & Hydraulic Conductivity

H $_{LOSS}$	0.25	kPa
Stage No.	Average	
Gauge Pressure	180	kPa
Q	6	L/min
H	28	m
Interpreted Result	4	uL

Reported k at Stage Average

Analytical Method 1: (ref = Golder geotechnical field notes draft 1997) k =
Q/H x 6.10889 x 10^{-6}x((log(2L/D))/L)

k = 4.3E-07 m/s

Analytical Method 2: (ref = Sharp, J.C 1975 Pit Slope Manual, CANMET report). k = 1/(2Lx3.14) x (Q/H) ln(R/r) m/s (convert L/min to m/s). Assume R = radius of influence of 100m & r = radius of borehole.

k = 6.8E-07 m/s

Flow Type: TURBULENT FLOW

COMMENTS:

WATER PRESSURE TEST (5 Stage)

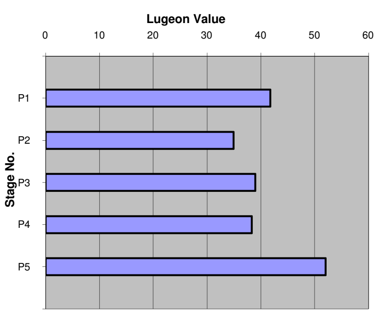
						Revision Nº :	18
Job Nº :	0978110-562	Hole Nº :	TT003	Drilling Method :	HQ	Vertical depth to Groundwater Immediately prior to test (m bgl) :	0.77
Client :	OceanaGold NZ Ltd	Dip (Deg) :	-90	Hole Diameter (m) :	0.096	Used in analysis (m bgl) :	0.77
Project :	Macraes Flat Expansion	DH Interval Top (m) :	5.31	DH Tested Length (m) :	6.00	Pressure Gauge Height (m agl) :	0.80
Location :	Macraes Gold Project	DH Interval Base (m) :	11.31	Packer Type:	Pneumatic - Wireline - Single	Presumed Water Temperature :	5
Tested By :	McNeill Drilling	Computed By :	P. Mangeya	Rock tested :	Otago Schist	Casing Inner Diameter (mm) :	77.80
Date :	23/06/2010	Date :	20/07/2010	Water Meter Reading in	Litres	Checked By : B. Sinclair	Date : 20/07/2010

Pressure Stage	Gauge Pressure kPa	No	Actual Time (h:m:s)	Time Intervals (min)	Water Meter Readings Reading (Litres)	Volume (L)	Discharge (L/min)	Meter discharge - Leakage (L/min)	Discharge/m (L/min/m)	Remarks	
P1	50	0	0:00:00	0	224.0	0.00	0.00	0.00	0.00	c :	1
		1	0:05:00	05:00	307.0	83.00	16.60	16.60	2.77	Start Date/Time:	
		2	0:10:00	05:00	383.0	76.00	15.20	15.20	2.53		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	31.80	31.80	5.30		
						Average:	15.900	15.900	2.650	Leak Flow Rate =	0
P2	100	0	0:15:00	0	417.0	0.00	0.00	0.00	0.00	c :	1
		1	0:20:00	05:00	530.0	113.00	22.60	22.60	3.77		
		2	0:25:00	05:00	650.0	120.00	24.00	24.00	4.00		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	46.60	46.60	7.77		
						Average:	23.300	23.300	3.883	Leak Flow Rate =	0
P3	150	0	0:30:00	0	698.0	0.00	0.00	0.00	0.00	c :	1
		1	0:35:00	05:00	880.0	182.00	36.40	36.40	6.07		
		2	0:40:00	05:00	1061.0	181.00	36.20	36.20	6.03		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	72.60	72.60	12.10		
						Average:	36.300	36.300	6.050	Leak Flow Rate =	0
P4	100	0	0:45:00	0	124.0	0.00	0.00	0.00	0.00	c :	1
		1	0:50:00	05:00	251.0	127.00	25.40	25.40	4.23		
		2	0:55:00	05:00	378.0	127.00	25.40	25.40	4.23		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	50.80	50.80	8.47		
						Average:	25.400	25.400	4.233	Leak Flow Rate =	0
P5	50	0	0:55:00	0	404.0	0.00	0.00	0.00	0.00	c :	1
		1	1:00:00	05:00	502.0	98.00	19.60	19.60	3.27	Finish Date/Time:	
		2	1:05:00	05:00	599.0	97.00	19.40	19.40	3.23		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	39.00	39.00	6.50		
						Average:	19.500	19.500	3.250	Leak Flow Rate =	0

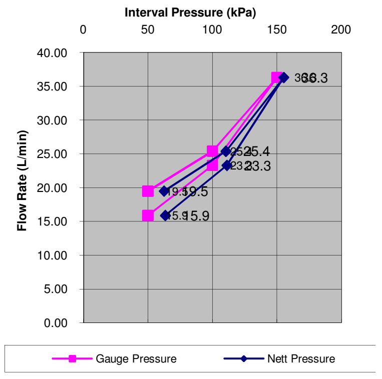
TEST RESULTS

Stage No.	Lugeon (1933) Value	Nett Pressures
P1	41.8	63.5
P2	34.9	111.2
P3	39.0	155.3
P4	38.3	110.5
P5	52.0	62.5

Lugeon Value Curve



Pressure Vs Flow



Lugeon (1933) method includes nett pressure in calculation that is Hp (gauge pressure) + Hg (height gauges above watertable) - Hf (head losses).

Interpreted Result & Hydraulic Conductivity

H LOSS	4.81	kPa
Stage No.	Average	
Gauge Pressure	90	kPa
Q	24	L/min
H	10	m
Interpreted Result	41	uL

Reported k at Stage Average

Analytical Method 1: (ref = Golder geotechnical field notes draft 1997) k = Q/H x 6.10889 x 10^{-6}x((log(2L/D))/L)

k = 5.0E-06 m/s

Analytical Method 2: (ref = Sharp, J.C 1975 Pit Slope Manual, CANMET report). k = 1/(2Lx3.14) x (Q/H) ln(R/r) m/s (convert L/min to m/s). Assume R = radius of influence of 100m & r = radius of borehole.

k = 7.9E-06 m/s

Flow Type: LAMINAR FLOW

COMMENTS:

WATER PRESSURE TEST (5 Stage)

Revision Nº : 18

Job Nº : 0978110-562
Client : OceanaGold NZ Ltd
Project : Macraes Flat Expansion
Location : Macraes Gold Project
Tested By : McNeill Drilling
Date : 23/06/2010

Hole Nº : TT003
Dip (Deg) : -90
DH Interval Top (m) : 25.36
DH Interval Base (m) : 31.36
Computed By : P. Mangeya
Date : 20/07/2010

Drilling Method : HQ
Hole Diameter (m) : 0.096
DH Tested Length (m) : 6.00
Packer Type: Pneumatic - Wireline - Single
Rock tested : Otago Schist
Water Meter Reading in Litres
Checked By : B. Sinclair

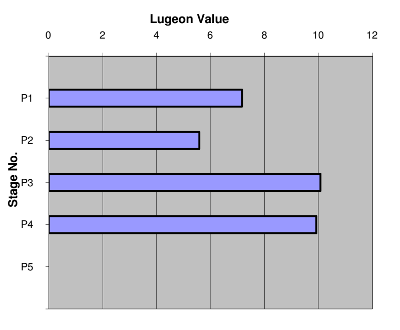
Vertical depth to Groundwater
Immediately prior to test (m bgl) : 0.77
Used in analysis (m bgl) : 0.77
Pressure Gauge Height (m agl) : 0.80
Presumed Water Temperature : 5
Casing Inner Diameter (mm) : 77.80
Date : 20/07/2010

Pressure Stage	Gauge Pressure kPa	No	Actual Time (h:m:s)	Time Intervals (min)	Water Meter Readings Reading (Litres)	Volume (L)	Discharge (L/min)	Meter discharge - Leakage (L/min)	Discharge/m (L/min/m)	Remarks	
										c :	1
P1	100	0	0:00:00	0	637.0	0.00	0.00	0.00	0.00	Start Date/Time:	
		1	0:05:00	05:00	665.0	28.00	5.60	5.60	0.93		
		2	0:10:00	05:00	686.5	21.50	4.30	4.30	0.72		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	9.90	9.90	1.65		
						Average:	4.950	4.950	0.825	Leak Flow Rate =	0
P2	200	0	0:15:00	0	694.0	0.00	0.00	0.00	0.00	c :	1
		1	0:20:00	05:00	731.0	37.00	7.40	7.40	1.23		
		2	0:25:00	05:00	766.0	35.00	7.00	7.00	1.17		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	14.40	14.40	2.40		
						Average:	7.200	7.200	1.200	Leak Flow Rate =	0
P3	300	0	0:30:00	0	780.0	0.00	0.00	0.00	0.00	c :	1
		1	0:35:00	05:00	874.0	94.00	18.80	18.80	3.13		
		2	0:40:00	05:00	969.0	95.00	19.00	19.00	3.17		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	37.80	37.80	6.30		
						Average:	18.900	18.900	3.150	Leak Flow Rate =	0
P4	200	0	0:45:00	0	977.0	0.00	0.00	0.00	0.00	c :	1
		1	0:50:00	05:00	990.0	13.00	2.60	2.60	0.43		
		2	0:55:00	05:00	1104.5	114.50	22.90	22.90	3.82		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	25.50	25.50	4.25		
						Average:	12.750	12.750	2.125	Leak Flow Rate =	0
P5	100	0				0.00	0.00	0.00	0.00	c :	1
		1								Finish Date/Time:	
		2									
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :					
						Average:				Leak Flow Rate =	0

TEST RESULTS

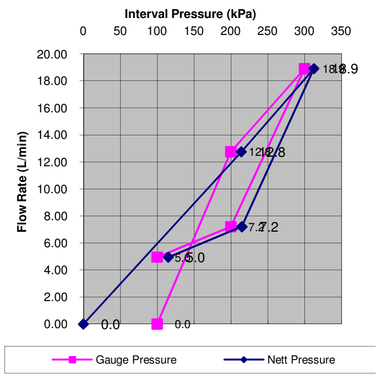
Stage No.	Lugeon (1933) Value	Nett Pressures
P1	7.2	115.2
P2	5.6	215.0
P3	10.1	312.6
P4	9.9	214.1
P5		#VALUE!

Lugeon Value Curve



Lugeon (1933) method includes nett pressure in calculation that is Hp (gauge pressure) + Hg (height gauges above watertable) - Hf (head losses).

Pressure Vs Flow



Interpreted Result & Hydraulic Conductivity

H LOSS 2.76 kPa
Stage No. P3
Gauge Pressure 300 kPa
Q 19 L/min
H 32 m
Interpreted Result 10 uL

Reported k at Stage P3

Analytical Method 1: (ref = Golder geotechnical field notes draft 1997) k =
$Q/H \times 6.10889 \times 10^{-6} x((\log(2L/D))/L)$

k = 1.3E-06 m/s

Analytical Method 2: (ref = Sharp, J.C 1975 Pit Slope Manual, CANMET report). k = 1/(2Lx3.14) x (Q/H) ln(R/r) m/s (convert L/min to m/s). Assume R = radius of influence of 100m & r = radius of borehole.

k = 2.0E-06 m/s

Flow Type: LAMINAR FLOW

COMMENTS:

WATER PRESSURE TEST (5 Stage)

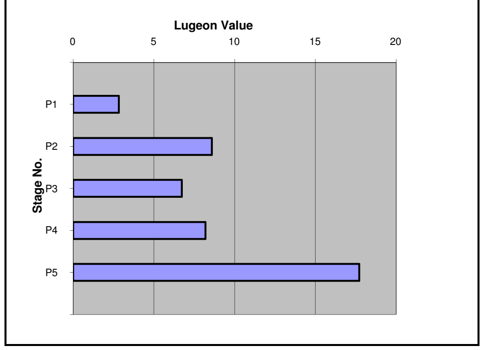
Job Nº :	0978110-562	Hole Nº :	TT004	Drilling Method :	HQ	Revision Nº :	18
Client :	OceanaGold NZ Ltd	Dip (Deg) :	-70	Hole Diameter (m) :	0.096	Vertical depth to Groundwater Immediately prior to test (m bgl) :	0.00
Project :	Macraes Flat Expansion	DH Interval Top (m) :	5.00	DH Tested Length (m) :	6.00	Used in analysis (m bgl) :	0.00
Location :	Macraes Gold Project	DH Interval Base (m) :	11.00	Packer Type:	Pneumatic - Wireline - Single	Pressure Gauge Height (m agl) :	0.80
Tested By :	McNeill Drilling	Computed By :	P. Mangeya	Rock tested :	Otago Schist	Presumed Water Temperature :	5
						Casing Inner Diameter (mm) :	77.80
Date :	3/06/2010	Date :	24/06/2010	Water Meter Reading in	Litres	Checked By : B. Sinclair	Date : 24/06/2010

Pressure Stage	Gauge Pressure kPa	No	Actual Time (h:m:s)	Time Intervals (min)	Water Meter Readings Reading (Litres)	Volume (L)	Discharge (L/min)	Meter discharge - Leakage (L/min)	Discharge/m (L/min/m)	Remarks
P1	50	0	0:00:00	0	796.2	0.00	0.00	0.00	0.00	c : 1
		1	0:05:00	05:00	801.1	4.90	0.98	0.98	0.16	Start Date/Time:
		2	0:10:00	05:00	806.0	4.90	0.98	0.98	0.16	
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
						Total :	1.96	1.96	0.33	
						Average:	0.980	0.980	0.163	Leak Flow Rate = 0
P2	100	0	0:15:00	0	815.0	0.00	0.00	0.00	0.00	c : 1
		1	0:20:00	05:00	844.5	29.50	5.90	5.90	0.98	
		2	0:25:00	05:00	870.5	26.00	5.20	5.20	0.87	
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
						Total :	11.10	11.10	1.85	
						Average:	5.550	5.550	0.925	Leak Flow Rate = 0
P3	150	0	0:30:00	0	882.0	0.00	0.00	0.00	0.00	c : 1
		1	0:35:00	05:00	915.4	33.40	6.68	6.68	1.11	
		2	0:40:00	05:00	945.6	30.20	6.04	6.04	1.01	
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
						Total :	12.72	12.72	2.12	
						Average:	6.360	6.360	1.060	Leak Flow Rate = 0
P4	100	0	0:45:00	0	951.0	0.00	0.00	0.00	0.00	c : 1
		1	0:50:00	05:00	977.1	26.10	5.22	5.22	0.87	
		2	0:55:00	05:00	1004.0	26.90	5.38	5.38	0.90	
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
						Total :	10.60	10.60	1.77	
						Average:	5.300	5.300	0.883	Leak Flow Rate = 0
P5	50	0	0:55:00	0	1012.0	0.00	0.00	0.00	0.00	c : 1
		1	1:00:00	05:00	1041.0	29.00	5.80	5.80	0.97	Finish Date/Time:
		2	1:05:00	05:00	1073.2	32.20	6.44	6.44	1.07	
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
						Total :	12.24	12.24	2.04	
						Average:	6.120	6.120	1.020	Leak Flow Rate = 0

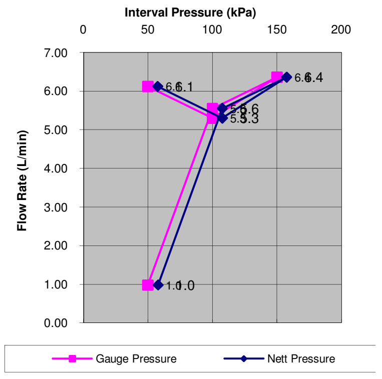
TEST RESULTS

Stage No.	Lugeon (1933) Value	Nett Pressures
P1	2.8	57.8
P2	8.6	107.6
P3	6.7	157.5
P4	8.2	107.6
P5	17.7	57.6

Lugeon Value Curve



Pressure Vs Flow



Lugeon (1933) method includes nett pressure in calculation that is Hp (gauge pressure) + Hg (height gauges above watertable) - Hf (head losses).

Interpreted Result & Hydraulic Conductivity

H_{LOSS}	0.22	kPa
Stage No.	P4	
Gauge Pressure	100	kPa
Q	5	L/min
H	11	m
Interpreted Result	8	uL

Reported k at Stage P4

Analytical Method 1: (ref = Golder geotechnical field notes draft 1997) k = Q/H x 6.10889 x 10^{-6}x((log(2L/D))/L)

k = 1.0E-06 m/s

Analytical Method 2: (ref = Sharp, J.C 1975 Pit Slope Manual, CANMET report). k = 1/(2Lx3.14) x (Q/H) ln(R/r) m/s (convert L/min to m/s). Assume R = radius of influence of 100m & r = radius of borehole.

k = 1.6E-06 m/s

Flow Type: WASH-OUT

COMMENTS: P4 stage taken as being representative for permeability assessment.

WATER PRESSURE TEST (5 Stage)

Revision Nº : 18

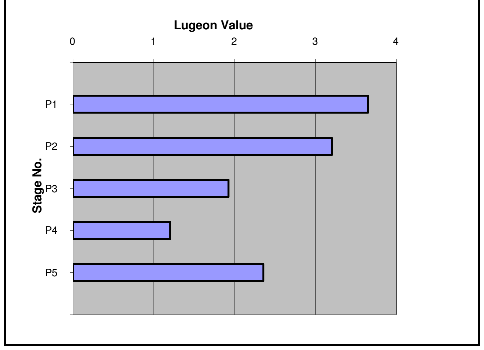
Job Nº :	0978110-562	Hole Nº :	TT004	Drilling Method :	HQ	Vertical depth to Groundwater	Immediately prior to test (m bgl) :	0.25
Client :	OceanaGold NZ Ltd	Dip (Deg) :	-70	Hole Diameter (m) :	0.096		Used in analysis (m bgl) :	0.25
Project :	Macraes Flat Expansion	DH Interval Top (m) :	23.00	DH Tested Length (m) :	7.50		Pressure Gauge Height (m agl) :	0.80
Location :	Macraes Gold Project	DH Interval Base (m) :	30.50	Packer Type:	Pneumatic - Wireline - Single		Presumed Water Temperature :	5
Tested By :	McNeill Drilling	Computed By :	P. Mangeya	Rock tested :	Otago Schist		Casing Inner Diameter (mm) :	77.80
Date :	3/06/2010	Date :	24/06/2010	Water Meter Reading in	Litres	Checked By : B. Sinclair	Date :	24/06/2010

Pressure Stage	Gauge Pressure kPa	No	Actual Time (h:m:s)	Time Intervals (min)	Water Meter Readings Reading (Litres)		Volume (L)	Discharge (L/min)	Meter discharge - Leakage (L/min)	Discharge/m (L/min/m)	Remarks	
P1	100	0	0:00:00	0	5.0		0.00	0.00	0.00	0.00	c :	1
		1	0:05:00	05:00	20.4		15.40	3.08	3.08	0.41	Start Date/Time:	
		2	0:10:00	05:00	35.2		14.80	2.96	2.96	0.39		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	6.04	6.04	0.81		
							Average:	3.020	3.020	0.403	Leak Flow Rate =	0
P2	200	0	0:15:00	0	131.5		0.00	0.00	0.00	0.00	c :	1
		1	0:20:00	05:00	164.0		32.50	6.50	6.50	0.87		
		2	0:25:00	05:00	182.0		18.00	3.60	3.60	0.48		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	10.10	10.10	1.35		
							Average:	5.050	5.050	0.673	Leak Flow Rate =	0
P3	300	0	0:30:00	0	190.2		0.00	0.00	0.00	0.00	c :	1
		1	0:35:00	05:00	212.0		21.80	4.36	4.36	0.58		
		2	0:40:00	05:00	235.0		23.00	4.60	4.60	0.61		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	8.96	8.96	1.19		
							Average:	4.480	4.480	0.597	Leak Flow Rate =	0
P4	200	0	0:45:00	0	240.0		0.00	0.00	0.00	0.00	c :	1
		1	0:50:00	05:00	249.7		9.70	1.94	1.94	0.26		
		2	0:55:00	05:00	259.0		9.30	1.86	1.86	0.25		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	3.80	3.80	0.51		
							Average:	1.900	1.900	0.253	Leak Flow Rate =	0
P5	100	0	0:55:00	0	251.1		0.00	0.00	0.00	0.00	c :	1
		1	1:00:00	05:00	265.7		14.60	2.92	2.92	0.39	Finish Date/Time:	
		2	1:05:00	05:00	270.6		4.90	0.98	0.98	0.13		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	3.90	3.90	0.52		
							Average:	1.950	1.950	0.260	Leak Flow Rate =	0

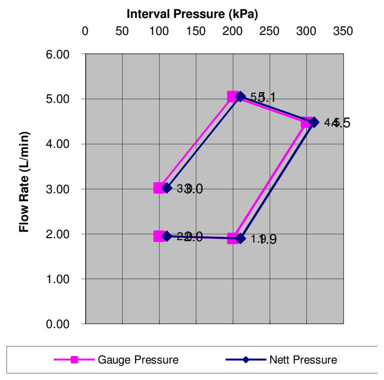
TEST RESULTS

Stage No.	Lugeon (1933) Value	Nett Pressures
P1	3.7	110.2
P2	3.2	210.1
P3	1.9	310.1
P4	1.2	210.3
P5	2.4	110.3

Lugeon Value Curve



Pressure Vs Flow



Lugeon (1933) method includes nett pressure in calculation that is Hp (gauge pressure) + Hg (height gauges above watertable) - Hf (head losses).

Interpreted Result & Hydraulic Conductivity

H $_{LOSS}$	0.03	kPa
Stage No.	P5	
Gauge Pressure	100	kPa
Q	2	L/min
H	11	m
Interpreted Result	2	uL

Reported k at Stage P5

Analytical Method 1: (ref = Golder geotechnical field notes draft 1997) k = Q/H x 6.10889 x 10^{-6}x((log(2L/D))/L)

k = 3.1E-07 m/s

Analytical Method 2: (ref = Sharp, J.C 1975 Pit Slope Manual, CANMET report). k = 1/(2Lx3.14) x (Q/H) ln(R/r) m/s (convert L/min to m/s). Assume R = radius of influence of 100m & r = radius of borehole.

k = 4.7E-07 m/s

Flow Type: VOID FILLING

COMMENTS: Stage P5 flow applied to hydraulic conductivity calculation returns result little different from that resulting from the mean of the results for all five stages.

WATER PRESSURE TEST (5 Stage)

Job Nº :	0978110-562	Hole Nº :	TT005	Drilling Method :	HQ
Client :	OceanaGold NZ Ltd	Dip (Deg) :	-90	Hole Diameter (m) :	0.096
Project :	Macraes Flat Expansion	DH Interval Top (m) :	20.11	DH Tested Length (m) :	5.72
Location :	Macraes Gold Project	DH Interval Base (m) :	25.83	Packer Type:	Pneumatic - Wireline - Single
Tested By :	McNeill Drilling	Computed By :	P. Mangeya	Rock tested :	Otago Schist
Date :	17/06/2010	Date :	24/06/2010	Water Meter Reading in	Litres

Checked By : B. Sinclair

Vertical depth to Groundwater

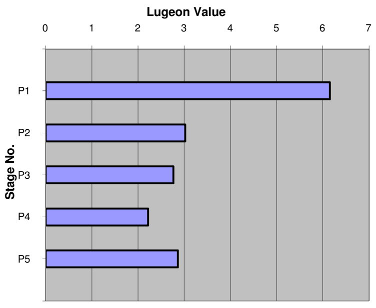
Revision Nº :	18
Immediately prior to test (m bgl) :	1.91
Used in analysis (m bgl) :	1.91
Pressure Gauge Height (m agl) :	0.80
Presumed Water Temperature :	5
Casing Inner Diameter (mm) :	77.80
Date :	24/06/2010

Pressure Stage	Gauge Pressure kPa	No	Actual Time (h:m:s)	Time Intervals (min)	Water Meter Readings Reading (Litres)		Volume (L)	Discharge (L/min)	Meter discharge - Leakage (L/min)	Discharge/m (L/min/m)	Remarks	
											c :	1
P1	100	0	0:00:00	0	590.0		0.00	0.00	0.00	0.00	Start Date/Time:	
		1	0:05:00	05:00	617.0		27.00	5.40	5.40	0.94		
		2	0:10:00	05:00	634.5		17.50	3.50	3.50	0.61		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	8.90	8.90	1.56		
							Average:	4.450	4.450	0.778	Leak Flow Rate =	0
P2	150	0	0:15:00	0	640.0		0.00	0.00	0.00	0.00	c :	1
		1	0:20:00	05:00	656.0		16.00	3.20	3.20	0.56		
		2	0:25:00	05:00	670.5		14.50	2.90	2.90	0.51		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	6.10	6.10	1.07		
							Average:	3.050	3.050	0.533	Leak Flow Rate =	0
P3	250	0	0:30:00	0	680.0		0.00	0.00	0.00	0.00	c :	1
		1	0:35:00	05:00	702.0		22.00	4.40	4.40	0.77		
		2	0:40:00	05:00	723.7		21.70	4.34	4.34	0.76		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	8.74	8.74	1.53		
							Average:	4.370	4.370	0.764	Leak Flow Rate =	0
P4	150	0	0:45:00	0	730.0		0.00	0.00	0.00	0.00	c :	1
		1	0:50:00	05:00	742.1		12.10	2.42	2.42	0.42		
		2	0:55:00	05:00	752.4		10.30	2.06	2.06	0.36		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	4.48	4.48	0.78		
							Average:	2.240	2.240	0.392	Leak Flow Rate =	0
P5	100	0	0:55:00	0	753.1		0.00	0.00	0.00	0.00	c :	1
		1	1:00:00	05:00	763.7		10.60	2.12	2.12	0.37	Finish Date/Time:	
		2	1:05:00	05:00	773.8		10.10	2.02	2.02	0.35		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	4.14	4.14	0.72		
							Average:	2.070	2.070	0.362	Leak Flow Rate =	0

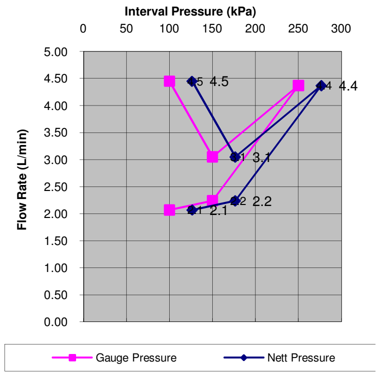
TEST RESULTS

Stage No.	Lugeon (1933) Value	Nett Pressures
P1	6.2	126.4
P2	3.0	176.5
P3	2.8	276.4
P4	2.2	176.5
P5	2.9	126.5

Lugeon Value Curve



Pressure Vs Flow



Lugeon (1933) method includes nett pressure in calculation that is Hp (gauge pressure) + Hg (height gauges above watertable) - Hf (head losses).

Interpreted Result & Hydraulic Conductivity

H_{LOSS}	0.15	kPa
Stage No.	P3	
Gauge Pressure	250	kPa
Q	4	L/min
H	28	m
Interpreted Result	3	uL

Reported k at Stage P3

Analytical Method 1: (ref = Golder geotechnical field notes draft 1997) k = Q/H x 6.10889 x 10^{-6}x((log(2L/D))/L)

k = 3.4E-07 m/s

Analytical Method 2: (ref = Sharp, J.C 1975 Pit Slope Manual, CANMET report). k = 1/(2Lx3.14) x (Q/H) ln(R/r) m/s (convert L/min to m/s). Assume R = radius of influence of 100m & r = radius of borehole.

k = 5.5E-07 m/s

Flow Type: LAMINAR FLOW

COMMENTS: Stage P1 flow discounted as being anomalous. Subsequent stages indicate laminar flow conditions.

WATER PRESSURE TEST (5 Stage)

Revision Nº : 18

Job Nº : 0978110-562
Client : OceanaGold NZ Ltd
Project : Macraes Flat Expansion
Location : Macraes Gold Project
Tested By : McNeill Drilling
Date : 13/06/2010

Hole Nº : TT007
Dip (Deg) : -90
DH Interval Top (m) : 23.22
DH Interval Base (m) : 30.12
Computed By : P. Mangeya
Date : 24/06/2010

Drilling Method : HQ
Hole Diameter (m) : 0.096
DH Tested Length (m) : 6.90
Packer Type: Pneumatic - Wireline - Single
Rock tested : Otago Schist
Water Meter Reading in Litres

Vertical depth to Groundwater
Immediately prior to test (m bgl) : 0.81
Used in analysis (m bgl) : 0.81
Pressure Gauge Height (m agl) : 0.80
Presumed Water Temperature : 5
Casing Inner Diameter (mm) : 77.80

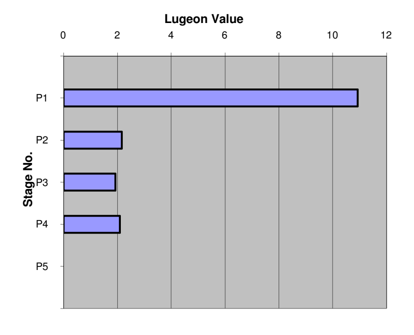
Checked By : B. Sinclair
Date : 20/07/2010

Pressure Stage	Gauge Pressure kPa	No	Actual Time (h:m:s)	Time Intervals (min)	Water Meter Readings Reading (Litres)	Volume (L)	Discharge (L/min)	Meter discharge - Leakage (L/min)	Discharge/m (L/min/m)	Remarks	
P1	50	0	0:00:00	0	964.5	0.00	0.00	0.00	0.00	c :	1
		1	0:05:00	05:00	990.3	25.80	5.16	5.16	0.75	Start Date/Time:	
		2	0:10:00	05:00	1014.0	23.70	4.74	4.74	0.69		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	9.90	9.90	1.43		
						Average:	4.950	4.950	0.717	Leak Flow Rate =	0
P2	200	0	0:15:00	0	1021.8	0.00	0.00	0.00	0.00	c :	1
		1	0:20:00	05:00	1037.7	15.90	3.18	3.18	0.46		
		2	0:25:00	05:00	1054.0	16.30	3.26	3.26	0.47		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	6.44	6.44	0.93		
						Average:	3.220	3.220	0.467	Leak Flow Rate =	0
P3	300	0	0:30:00	0	1060.0	0.00	0.00	0.00	0.00	c :	1
		1	0:35:00	05:00	1082.0	22.00	4.40	4.40	0.64		
		2	0:40:00	05:00	1102.0	20.00	4.00	4.00	0.58		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	8.40	8.40	1.22		
						Average:	4.200	4.200	0.609	Leak Flow Rate =	0
P4	200	0	0:45:00	0	1106.5	0.00	0.00	0.00	0.00	c :	1
		1	0:50:00	05:00	1121.0	14.50	2.90	2.90	0.42		
		2	0:55:00	05:00	1137.6	16.60	3.32	3.32	0.48		
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :	6.22	6.22	0.90		
						Average:	3.110	3.110	0.451	Leak Flow Rate =	0
P5	50	0		0		0.00	0.00	0.00	0.00	c :	1
		1								Finish Date/Time:	
		2									
		3									
		4									
		5									
		6									
		7									
		8									
		9									
		10									
						Total :					
						Average:				Leak Flow Rate =	0

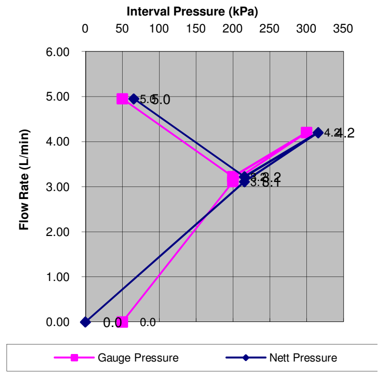
TEST RESULTS

Stage No.	Lugeon (1933) Value	Nett Pressures
P1	10.9	65.6
P2	2.2	215.7
P3	1.9	315.7
P4	2.1	215.7
P5		#VALUE!

Lugeon Value Curve



Pressure Vs Flow



Lugeon (1933) method includes nett pressure in calculation that is Hp (gauge pressure) + Hg (height gauges above watertable) - Hf (head losses).

Interpreted Result & Hydraulic Conductivity

H LOSS 0.14 kPa
Stage No. P3
Gauge Pressure 300 kPa
Q 4 L/min
H 32 m
Interpreted Result 2 uL

Reported k at Stage P3

Analytical Method 1: (ref = Golder geotechnical field notes draft 1997) k = Q/H x 6.10889 x 10^{-6}x((log(2L/D))/L)

k = 2.5E-07 m/s

Analytical Method 2: (ref = Sharp, J.C 1975 Pit Slope Manual, CANMET report). k = 1/(2Lx3.14) x (Q/H) ln(R/r) m/s (convert L/min to m/s). Assume R = radius of influence of 100m & r = radius of borehole.

k = 3.8E-07 m/s

Flow Type: LAMINAR FLOW

COMMENTS Stage P1 data excluded as anomalous. Stage P5 data indicated water flowing in reverse direction through gauge and also excluded. Remaining data indicates laminar flow.

WATER PRESSURE TEST (5 Stage)

Revision Nº : 18

Job Nº : 0978110-562
Client : OceanaGold NZ Ltd
Project : Macraes Flat Expansion
Location : Macraes Gold Project
Tested By : McNeill Drilling
Date : 10/06/2010

Hole Nº : TT008
Dip (Deg) : -90
DH Interval Top (m) : 11.00
DH Interval Base (m) : 17.00
Computed By : P. Mangeya
Date : 24/06/2010

Drilling Method : HQ
Hole Diameter (m) : 0.096
DH Tested Length (m) : 6.00
Packer Type: Pneumatic - Wireline - Single
Rock tested : Otago Schist
Water Meter Reading in Litres
Checked By : B. Sinclair

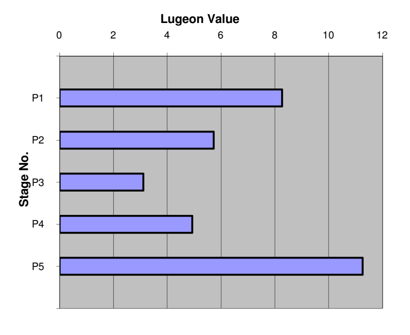
Vertical depth to Groundwater
Immediately prior to test (m bgl) : 0.00
Used in analysis (m bgl) : 0.00
Pressure Gauge Height (m agl) : 0.80
Presumed Water Temperature : 5
Casing Inner Diameter (mm) : 77.80
Date : 24/06/2010

Pressure Stage	Gauge Pressure kPa	No	Actual Time (h:m:s)	Time Intervals (min)	Water Meter Readings Reading (Litres)		Volume (L)	Discharge (L/min)	Meter discharge - Leakage (L/min)	Discharge/m (L/min/m)	Remarks	
P1	50	0	0:00:00	0	350.0		0.00	0.00	0.00	0.00	c :	1
		1	0:05:00	05:00	375.2		25.20	5.04	5.04	0.84	Start Date/Time:	
		2	0:10:00	05:00	380.8		5.60	1.12	1.12	0.19		
		3	0:15:00	05:00	393.0		12.20	2.44	2.44	0.41		
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	8.60	8.60	1.43		
							Average:	2.867	2.867	0.478	Leak Flow Rate =	0
P2	100	0	0:15:00	0	397.5		0.00	0.00	0.00	0.00	c :	1
		1	0:20:00	05:00	415.6		18.10	3.62	3.62	0.60		
		2	0:25:00	05:00	434.5		18.90	3.78	3.78	0.63		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	7.40	7.40	1.23		
							Average:	3.700	3.700	0.617	Leak Flow Rate =	0
P3	200	0	0:30:00	0	439.2		0.00	0.00	0.00	0.00	c :	1
		1	0:35:00	05:00	459.1		19.90	3.98	3.98	0.66		
		2	0:40:00	05:00	478.0		18.90	3.78	3.78	0.63		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	7.76	7.76	1.29		
							Average:	3.880	3.880	0.647	Leak Flow Rate =	0
P4	100	0	0:45:00	0	483.0		0.00	0.00	0.00	0.00	c :	1
		1	0:50:00	05:00	498.8		15.80	3.16	3.16	0.53		
		2	0:55:00	05:00	514.9		16.10	3.22	3.22	0.54		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	6.38	6.38	1.06		
							Average:	3.190	3.190	0.532	Leak Flow Rate =	0
P5	50	0	0:55:00	0	520.0		0.00	0.00	0.00	0.00	c :	1
		1	1:00:00	05:00	539.8		19.80	3.96	3.96	0.66	Finish Date/Time:	
		2	1:05:00	05:00	559.0		19.20	3.84	3.84	0.64		
		3										
		4										
		5										
		6										
		7										
		8										
		9										
		10										
							Total :	7.80	7.80	1.30		
							Average:	3.900	3.900	0.650	Leak Flow Rate =	0

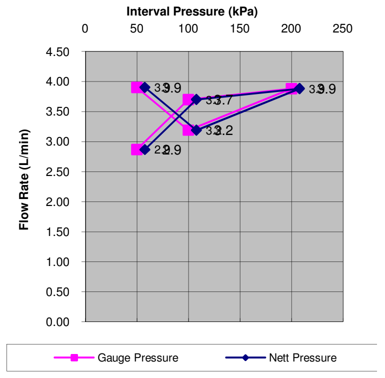
TEST RESULTS

Stage No.	Lugeon (1933) Value	Nett Pressures
P1	8.3	57.8
P2	5.7	107.7
P3	3.1	207.7
P4	4.9	107.8
P5	11.3	57.7

Lugeon Value Curve



Pressure Vs Flow



Lugeon (1933) method includes nett pressure in calculation that is Hp (gauge pressure) + Hg (height gauges above watertable) - Hf (head losses).

Interpreted Result & Hydraulic Conductivity

H_{LOSS} 0.12 kPa
Stage No. P3
Gauge Pressure 200 kPa
Q 4 L/min
H 21 m
Interpreted Result 3 uL

Reported k at Stage P3

Analytical Method 1: (ref = Golder geotechnical field notes draft 1997) k = Q/H x 6.10889 x 10^{-6}x((log(2L/D))/L)

k = 3.9E-07 m/s

Analytical Method 2: (ref = Sharp, J.C 1975 Pit Slope Manual, CANMET report). k = 1/(2Lx3.14) x (Q/H) ln(R/r) m/s (convert L/min to m/s). Assume R = radius of influence of 100m & r = radius of borehole.

k = 6.2E-07 m/s

Flow Type: TURBULENT FLOW

COMMENTS: Hydraulic conductivity may be slightly higher than calculated due to turbulence losses close to drillhole wall. Artesian flows from drillhole prior to start of test.

WATER PRESSURE TEST (5 Stage)

Revision Nº : 18

Job Nº : 0978110-562
Client : OceanaGold NZ Ltd
Project : Macraes Flat Expansion
Location : Macraes Gold Project
Tested By : McNeill Drilling
Date : 10/06/2010

Hole Nº : TT008
Dip (Deg) : -90
DH Interval Top (m) : 23.12
DH Interval Base (m) : 30.62
Computed By : P. Mangeya
Date : 24/06/2010

Drilling Method : HQ
Hole Diameter (m) : 0.096
DH Tested Length (m) : 7.50
Packer Type: Pneumatic - Wireline - Single
Rock tested : Otago Schist
Water Meter Reading in Litres
Checked By : B. Sinclair

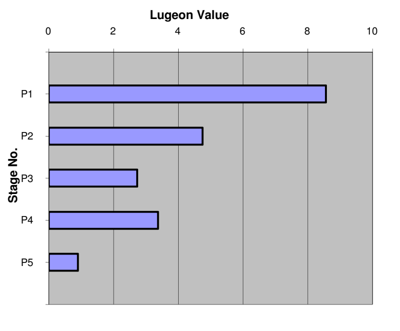
Vertical depth to Groundwater
Immediately prior to test (m bgl) : 0.00
Used in analysis (m bgl) : 0.00
Pressure Gauge Height (m agl) : 0.80
Presumed Water Temperature : 5
Casing Inner Diameter (mm) : 77.80
Date : 24/06/2010

Pressure Stage	Gauge Pressure kPa	No	Actual Time (h:m:s)	Time Intervals (min)	Water Meter Readings Reading (Litres)	Volume (L)	Discharge (L/min)	Meter discharge - Leakage (L/min)	Discharge/m (L/min/m)	Remarks
P1	100	0	0:00:00	0	588.0	0.00	0.00	0.00	0.00	c : 1
		1	0:05:00	05:00	623.0	35.00	7.00	7.00	0.93	Start Date/Time:
		2	0:10:00	05:00	657.0	34.00	6.80	6.80	0.91	
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
						Total :	13.80	13.80	1.84	
						Average:	6.900	6.900	0.920	Leak Flow Rate = 0
P2	200	0	0:15:00	0	665.5	0.00	0.00	0.00	0.00	c : 1
		1	0:20:00	05:00	706.0	40.50	8.10	8.10	1.08	
		2	0:25:00	05:00	739.5	33.50	6.70	6.70	0.89	
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
						Total :	14.80	14.80	1.97	
						Average:	7.400	7.400	0.987	Leak Flow Rate = 0
P3	300	0	0:30:00	0	747.0	0.00	0.00	0.00	0.00	c : 1
		1	0:35:00	05:00	778.7	31.70	6.34	6.34	0.85	
		2	0:40:00	05:00	810.0	31.30	6.26	6.26	0.83	
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
						Total :	12.60	12.60	1.68	
						Average:	6.300	6.300	0.840	Leak Flow Rate = 0
P4	200	0	0:45:00	0	817.0	0.00	0.00	0.00	0.00	c : 1
		1	0:50:00	05:00	843.6	26.60	5.32	5.32	0.71	
		2	0:55:00	05:00	869.6	26.00	5.20	5.20	0.69	
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
						Total :	10.52	10.52	1.40	
						Average:	5.260	5.260	0.701	Leak Flow Rate = 0
P5	100	0	0:55:00	0	872.0	0.00	0.00	0.00	0.00	c : 1
		1	1:00:00	05:00	876.0	4.00	0.80	0.80	0.11	Finish Date/Time:
		2	1:05:00	05:00	879.3	3.30	0.66	0.66	0.09	
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
						Total :	1.46	1.46	0.19	
						Average:	0.730	0.730	0.097	Leak Flow Rate = 0

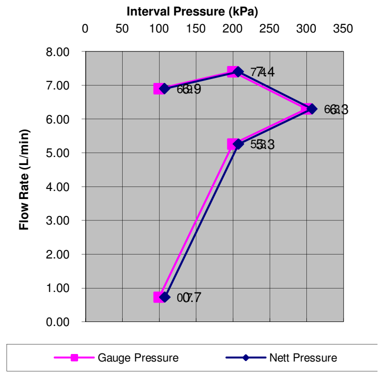
TEST RESULTS

Stage No.	Lugeon (1933) Value	Nett Pressures
P1	8.6	107.5
P2	4.8	207.4
P3	2.7	307.5
P4	3.4	207.6
P5	0.9	107.8

Lugeon Value Curve



Pressure Vs Flow



Lugeon (1933) method includes nett pressure in calculation that is Hp (gauge pressure) + Hg (height gauges above watertable) - Hf (head losses).

Interpreted Result & Hydraulic Conductivity

H LOSS 0.31 kPa
Stage No. P3
Gauge Pressure 300 kPa
Q 6 L/min
H 31 m
Interpreted Result 3 uL

Reported k at Stage P3

Analytical Method 1: (ref = Golder geotechnical field notes draft 1997) k = Q/H x 6.10889 x 10^{-6}x((log(2L/D))/L)

k = 3.6E-07 m/s

Analytical Method 2: (ref = Sharp, J.C 1975 Pit Slope Manual, CANMET report). k = 1/(2Lx3.14) x (Q/H) ln(R/r) m/s (convert L/min to m/s). Assume R = radius of influence of 100m & r = radius of borehole.

k = 5.4E-07 m/s

Flow Type: VOID FILLING

COMMENTS: Artesian flows from drillhole prior to start of test.

APPENDIX C

Arsenic adsorption report

March 2010

SOIL AND ROCK TESTING FOR ARSENIC ADSORPTION

Macraes Gold Project

Submitted to:
OceanaGold (NZ) Limited

REPORT

Report Number. 0978201-048 R003 v5

A world of capabilities delivered locally

Golder Associates

Executive Summary

OceanaGold New Zealand Limited proposes to construct a new tailings storage facility, the Back Road Tailings Storage Facility (BTSF), at the Macraes Gold Project (MGP) site. An initial conservative contaminant transport model of the BTSF indicated that arsenic seepage from the BTSF could potentially have an adverse effect on receiving water quality in Deepdell Creek. The initial model scenarios did not take into account arsenic adsorption occurring in the groundwater system. A comparison between seepage water quality of the Mixed Tailings Impoundment (MTI) and down-gradient groundwater wells indicates a considerable mass of arsenic is being attenuated by the groundwater system.

In order to account for the effects of arsenic adsorption in the contaminant transport modelling, two parameters need to be determined, the maximum adsorption capacity of the material and the partition coefficient (K_d). This report summarises the laboratory testing undertaken to determine these input values for the contaminant load model, the test results and the means used to apply the laboratory results to the groundwater model.

The maximum adsorption capacity and K_d were determined in batch adsorption tests. Samples of loess, un-weathered schist and weathered schist obtained from the MGP site were tested by adding a solution with a known concentration of dissolved arsenic. The adsorption tests were carried out using batches prepared at a 1:1 solid to liquid ratio. Mine water collected from a MTI chimney drain was used as a basis for the arsenic solution in the tests. The concentration of arsenic in the mine water was adjusted through the addition of arsenic (III) or arsenic (V) to provide solutions at a range of concentrations for test purposes. A 24 hour stabilisation period was applied for each sample, following which the solution was filtered prior to analysis for dissolved arsenic.

The maximum adsorption capacity of all of the tested materials was greater for arsenic (V) than arsenic (III). The most weathered and oxidised material (the loess) had the highest adsorption capacity and the least weathered and oxidised material (the un-weathered schist) had the lowest adsorption capacity. The maximum adsorption capacity results are:

■ Loess	As (III) 510 mg/kg solids	As (V) 900 mg/kg solids
■ Weathered schist	As (III) 460 mg/kg solids	As (V) 790 mg/kg solids
■ Unweathered schist	As (III) 270 mg/kg solids	As (V) 240 mg/kg solids

The adsorption tests did not conclusively exceed the maximum adsorption capacity of the materials within the range of arsenic concentrations tested. Due to this limitation it is likely that the maximum adsorption derived from these tests understates the maximum potential adsorption for the samples tested.

The arsenic in tailings seepage water is predominately arsenic (III), which is less readily adsorbed that arsenic (V). The maximum adsorption capacities for arsenic (III) presented were therefore used to derive a maximum adsorption capacity for the rock mass beneath the BTSF.

Groundwater flow paths through the schist are predominantly along fractures. Not all of the rock mass will contact the seepage water. The maximum adsorption capacities of the in-situ schist materials were corrected for differences in the likely fraction of the rock mass in contact with seepage water. This correction has been done on the basis of evaluating drillhole cores on a metre interval basis to derive the percentage of the rock mass that has been exposed to oxidation. As this percentage varies locally as well as with depth the calculated percentage of the rock mass available for arsenic adsorption was averaged across the weathering zones applied in the contaminant transport models. The corrected maximum arsenic adsorption values applied to the contaminant transport models are:

- 230 mg/kg for the upper 20 m of combined weathered schist and loess.
- 46 mg/kg for the moderately to slightly weathered schist between 20 m and 60 m below the ground surface.
- Conservative transport or arsenic at depths below 60 m.

Recent water quality data indicates drainage water from the Southern Pit tailings impoundment has an arsenic concentration of 14 g/m^3. As a conservative approach a single K_d value applicable for this concentration was calculated for use in the contaminant transport. The calculated K_d value was 20 L/kg.

A Langmuir isotherm has been applied to describe arsenic adsorption in each layer of the contaminant transport model. The isotherm has been separately for each of the schist weathering layers in the contaminant transport model, with no adsorption applied at depths greater than 60 m below the ground surface. The isotherm for each layer was derived using the K_d value above and a maximum number of arsenic adsorption sites calculated for each material based on the adjusted maximum arsenic adsorption capacities.

The key assumptions required for the use of the experimental values in the contaminant transport model were:

- The quality of the bulk seepage water sample used for the adsorption tests is representative of long-term tailings seepage water quality;
- The differences between field in-situ redox conditions and laboratory testing conditions do not significantly affect the arsenic adsorption capacities or K_d;
- No precipitation occurred during the adsorption testing; and
- The pH changes observed in the adsorption testing are similar to those that occur *in-situ*.

Validation of the results from this test program may be achieved through a field scale test program. Experience however indicates that such tests would require considerable time to generate measurable results. It is clear that arsenic has been adsorbed by the loess and weathered schist beneath the MTI since tailings storage began approximately 19 years ago. Concentrations of conservatively transported contaminants began increasing at detection wells in Maori Tommy Gully, down-gradient from the MTI, within a year of tailings storage beginning at the site. To date there is no indication of arsenic from the MTI in samples obtained from the detection wells.

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APPENDIX A

1.0 INTRODUCTION

1.1 Background

OceanaGold New Zealand Limited (OGL) proposes to construct a new tailings storage facility at the Macraes Gold Project (MGP) (Figure 1). Golder Associates (NZ) Limited (Golder) has recently undertaken work in support of the consenting process for the proposed Back Road tailings storage facility (BTSF). This work has included a review of water quality data derived from environmental monitoring at the MGP site. The review has highlighted changes in the quality of water discharging from drains installed in the Mixed Tailings Impoundment (MTI) and Southern Pit Impoundment (SPI) (Golder, 2010a) over time. Of specific concern are the increases in arsenic concentrations detected in the drainage water.

An initial contaminant transport model of the BTSF incorporating an assumption of conservative mass transport indicated that arsenic seepage from the BTSF could potentially have an adverse effect on receiving water quality in Deepdell Creek. The initial model scenarios did not take into account arsenic adsorption occurring in the groundwater system.

A comparison between seepage quality data from the MTI chimney drains and underdrains and the MTI detection wells in Maori Tommy Gully indicates there is considerable removal of arsenic from the seepage water between the embankment and the wells (Golder, 2010a). To date there is no indication that arsenic from the MTI is being detected in water from the detection wells. Concentrations of other contaminants at the detection wells began increasing shortly after storage of tailings was initiated at the MTI. It is considered likely that arsenic in seepage water from the MTI is being adsorbed by loess and schist underlying the MTI.

In order to account for the effects of adsorption in the contaminant transport modelling, two parameters need to be determined. These parameters relate to a reversible adsorption relationship between the contaminant of concern (arsenic) and the rock mass through which the seepage water passes. These parameters are:

- The maximum adsorption capacity of the solid material for the contaminant of concern; and
- The arsenic partition coefficient (K_d) between the rock or soil solids and the seepage water.

1.2 Scope of Work

Golder has been commissioned by OGL to undertake an evaluation of the adsorption capacity of samples of loess and schist host rock materials from the MGP site. The objective of the study is to determine maximum adsorption capacity values and K_d values for loess and weathered and unweathered schist. These values are to be incorporated into contaminant transport models of the MGP site.

This report[1] documents the adsorption study undertaken, including:

- A brief review of the factors affecting arsenic adsorption;
- A summary of arsenic adsorption capacity of soils reported in the literature;
- A summary of the methodology used in the adsorption testing; and
- A summary of the adsorption capacities and K_d's determined for the loess and schist samples tested.

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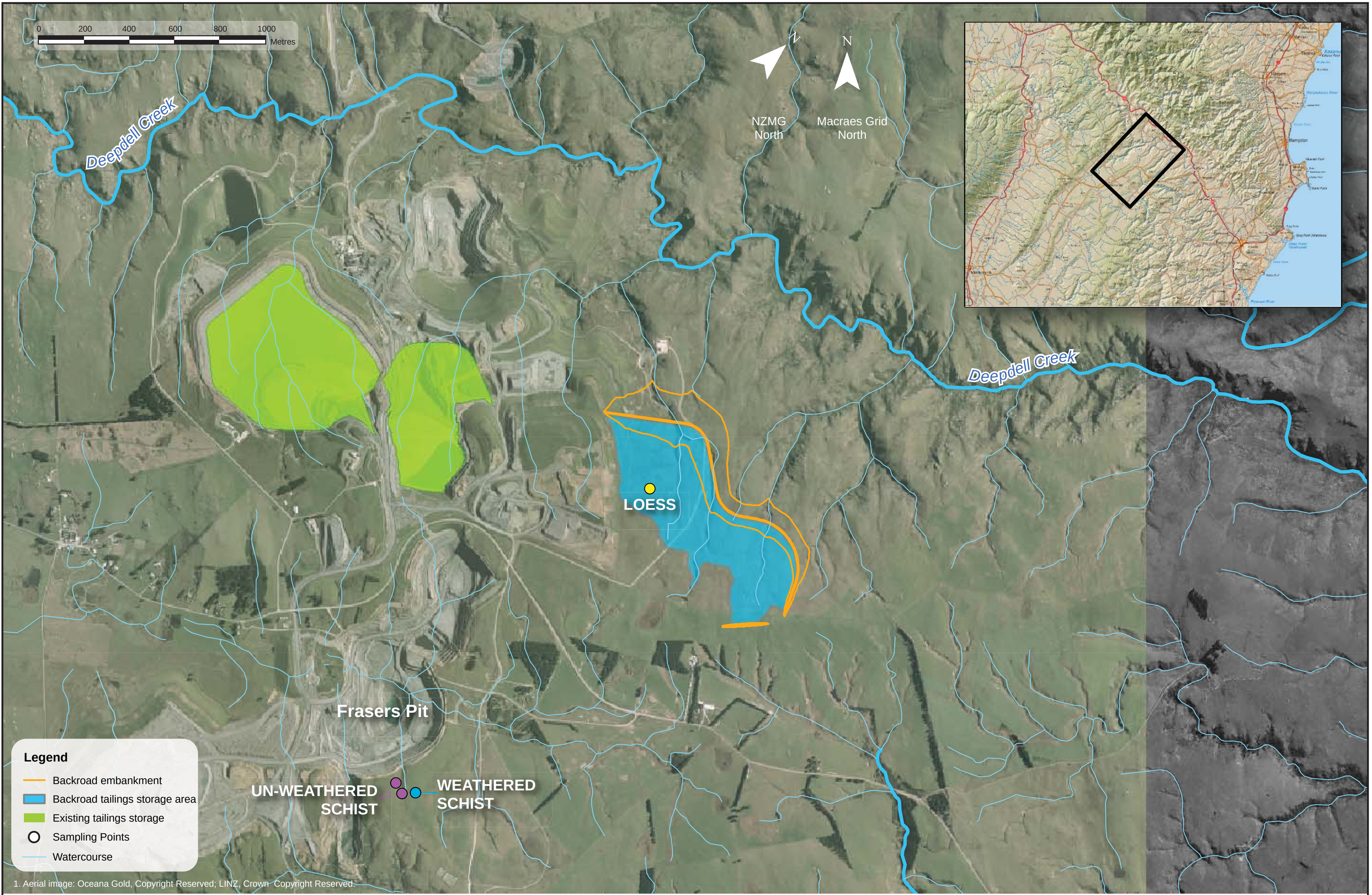
[1] This report is provided subject to the limitations and conditions presented in Appendix A.



1. Aerial image: Oceana Gold, Copyright Reserved; LINZ, Crown Copyright Reserved.

Golder Associates

TITLE SITE LOCATION AND SOIL SAMPLING SITES.

FEBRUARY 2010

PROJECT 0978201048

1

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2.0 METHODOLOGY

2.1 Introduction

With respect to contaminant adsorption from liquids onto solids, both the maximum adsorption capacity and the K_d can be determined from batch adsorption experiments. Batch adsorption methods are commonly used for determining adsorption capacities and K_d values the method developed in this study was based on that described in USEPA (2004).

The maximum adsorption capacity of a solid material can be determined by saturating all the adsorption sites of the solid material tested. In batch adsorption studies this is achieved by either varying the solids to liquid ratio between batches while maintaining a fixed contaminant concentration, or by maintaining a constant solid to liquid ratio and varying the contaminant concentration.

The K_d is used in contaminant transport models to determine partitioning of the contaminant between the liquid and solid phases in the groundwater system. The K_d is often relatively constant over narrow and low concentration ranges (USEPA, 1999) provided the chemical conditions (redox, pH, competing species concentrations) in a system remain constant. However, the K_d may vary considerably over larger concentration ranges or with changes in chemical conditions of the system. The K_d can be determined from the data collected in batch experiments by calculating the mass adsorbed and mass remaining in solution from the various tests.

2.2 Factors Affecting Adsorption

The adsorption capacity of any media is finite unless there are other mechanisms active which are capable of removing the adsorbate and freeing up adsorption sites. In the case of arsenic the adsorption capacity of the soil or rock through which it is being transported will be dependent on a number of factors including the following:

- The volume of material interacting with the contaminated groundwater;
- The number of adsorption sites on the surface of the adsorbing material and variability of that material;
- The potential for precipitation reactions;
- The pH and redox conditions of the seepage water and groundwater system (arsenic is highly sensitive to both);
- The contact time of the water with the media;
- The concentration of competing species in solution; and
- The concentration of the arsenic in solution.

Simulation of the processes occurring in the field would require further investigations into the nature of the seepage water and the groundwater system. However, these factors were considered in the experimental design and simulated where possible.

Geochemical Variation

The geochemistry of soils and rocks can have considerable spatial variation as a result of differing deposition methods, differing geochemical environments within the system and varying exposure to groundwater and the atmosphere. Therefore geochemical assessment programmes typically use a large number of samples to determine the geochemical variability of the different soils and rocks. To date this variability has not been determined for the loess or schist at the MGP site.

Competition for adsorption sites

The seepage water is a highly complex solution containing may different species of major ions, metals and metalloids. The presence of the various compounds and elements can affect the adsorption of arsenic onto the aquifer materials. For example it is known that some anionic species such as phosphate compete for arsenic adsorption sites (Goh & Lim, 2004). To account for these effects seepage water collected from the SumpB_CWD underdrain of the MTI was used as the basis for the test solution. Samples of this water were spiked with analytical grade arsenic compounds for the batch tests.

Precipitation and Co-precipitation

There is potential for arsenic to undergo precipitation reactions as well as adsorption when conditions are altered as they are in the adsorption batch testing. According to the USEPA (USEPA, 2004) there is considerable conjecture regarding the potential for removal of arsenic by precipitation, since limited thermodynamic data exists regarding arsenic minerals. It is possible that arsenic can be removed from solution in a batch test due to a combination of precipitation and adsorption reactions. The relative importance of these reactions cannot be determined directly from the batch tests.

Control samples containing added arsenic but no adsorbing media were tested to determine if arsenic precipitation would occur. However, this test cannot account for the possible introduction of species from the solids during the batch tests that may induce precipitation.

Solid to liquid ratio

The solid to liquid ratio has been determined to be an important factor for adsorption. Experimental observations indicate that the adsorption capacity of a solid material increases with decreasing solid to liquid ratio (Limousin et al. 2007). Ideally adsorption tests should be carried out in a way that best mimic's the situation being investigated. In the ground the saturated loess and schist material will have high solids to liquid ratio. A 1:1 ratio of solid to liquid was used in the testing as a higher solid to liquid ratio introduces other practical test issues.

Arsenic concentrations

The arsenic adsorption capacity of soils documented in the literature is highly variable, ranging from 20 mg arsenic/kg soil to 900 mg arsenic/kg soil (Table 1). Based on a 1:1 solid to liquid ratio and the adsorption capacities in Table 1 a range of arsenic concentrations were tested from raw mine water (approximately 5 g/m^3 to 1,000 g/m^3).

Table 1: Literature values for maximum adsorption capacity of arsenic in soils.

Source	Maximum adsorption (mg arsenic/kg soil)[1]
Williams et al. 2003	250
Carey et al. 1996	100 – 400
Smith et al. 1999	30 – 900
Smith et al. 2002	20 – 300

Note – [1]Values have been estimated from graphical results. The raw data was not presented in the source papers.

pH

Arsenic adsorption is highly dependent on pH and redox state. For the reduced arsenic (III) species higher adsorption rates at neutral pH have been reported in the literature (Banerjee et al. 2008; Goh & Lim, 2004). The oxidised arsenic (V) species has been observed to be adsorbed more effectively under acidic pH conditions.

The pH of the MGP tailings seepage water is consistently in the range of 6.0 to 6.7 with a median of 6.4 (Golder, 2010a). The addition of the arsenic compounds to test solutions has the potential to raise the pH as the compounds are mildly alkaline. Therefore if the pH of a solution following the addition of arsenic was greater than 6.7, the solution was adjusted to approximately the same pH as the initial mine water (around

6.6) by addition of hydrochloric acid. Hydrochloric acid was chosen since the counter ion chloride is conservative.

Redox

In general arsenic (V) is characterised by higher adsorption rates than arsenic (III) (Lim et al. 2004; Bowell, 1994). The predominant arsenic species in the underdrain water is arsenic (III) (Table 2). As there is potential for oxidation and transformation of arsenic (III) to occur in the aquifer, both arsenic (III) and arsenic (V) adsorption was assessed in the testing. However, it is noted that all tests were carried out in an oxygenated environment as the reducing conditions that may occur in the groundwater system could not be replicated in the laboratory.

Contact time

The contact time of the soil and seepage water will influence the extent of adsorption. Maximum adsorption will only be reached if the contact time is sufficient. Groundwater seepage through soil and rock in the MGP area is slow. The seepage water from the tailings dams is therefore expected to have considerable contact time with the surrounding rock mass. Numerous studies have assessed the kinetics of adsorption in batch laboratory experiments (Williams et al, 2003, Goh &Lim, 2004, Banerjee et al. 2008, Smith & Naidu, 2009). The outcomes of these studies demonstrated the rate of adsorption is fast in the initial 24 hours and considerably slower after that. As a conservative approach, a 24 hour reaction time was used in the batch experiments, although it is possible that further adsorption would occur in the field as a result of the longer contact time.

2.3 Method Summary

Sample collection

Batch adsorption testing was undertaken on one composited sample each of loess, weathered schist and unweathered schist. The sampling sites are shown in Figure 1. The loess samples used to generate the composite were obtained from within the footprint of the proposed BTSF. The schist samples used for compositing were obtained from an exposed operations face of Frasers Pit. For both the schist and loess spatial variation was not accounted for in the current test work. The following samples were collected and supplied for testing by OGL:

- Loess at depths of 0.2 m, 0.75 m and 1.5 m depths from within the footprint of the proposed BTSF;
- Weathered schist collected from the pit wall at depths of 1 m and 3 m below the ground surface;
- Unweathered schist collected from the pit wall at depths of approximately 40 m and 80 m below the ground surface; and
- Mine water from the Sump B_CDW discharge at the MTI.

Sample preparation

The adsorption testing was carried out by Hill Laboratories using a modified synthetic leaching procedure (SPLP). The pre-treatment of the samples included:

- Drying of the soil and rock samples at 35°C;
- Loess samples were composited and screened to <2 mm;
- Weathered schist samples were composited and screened to <10 mm;
- Unweathered schist samples were composited and screened to <10 mm and
- Mine water was filtered through a GFC filter followed by a 0.45 μm membrane filter prior to testing.

Adsorption testing

The mine water collected from SumpB_CDW was spiked using arsenic trioxide (As_2O_3) as the arsenic (III) source and sodium arsenate ($NaAsO_3$) as the arsenic (V) source. Between 5 g/m^3 and 1,000 g/m^3 of arsenic (III) or arsenic (V) was added to the raw mine water. Following addition of the arsenic the pH of each solution was adjusted to 6.5 - 6.7 using hydrochloric acid.

A 250 g sub-sample of each of the composited samples was mixed with a 250 mL sample of spiked mine water. The solutions were shaken on an end shaker used for synthetic leaching procedure (SPLP) for 24 hours. After shaking the samples were filtered through a 0.45 µm membrane filter and analysed for arsenic, iron and pH.

3.0 ADSORPTION TESTING RESULTS

3.1 Tailings Seepage Water Quality

The mine water seepage water from site SumpB_CDW and SP10 outlet drain were tested for water quality parameters and arsenic speciation prior to collection of bulk samples for adsorption testing. The SP10 outlet drain water quality is used in the contaminant load modelling as the input for the SPI and MTI seepage inputs, as such the arsenic speciation was verified here to justify the contaminant load model inputs.

The screening samples were filtered and preserved using sulfuric acid at the site and analysed for dissolved As(III) and dissolved As (V) therefore these samples are representative of the seepage water quality entering the groundwater system (Table 2). The predominant form of arsenic in the seepage water is As (III) with 93% and 100% of the arsenic being detected as As(III) for SumpB_CDW and SP10 outlet drain respectively.

The water quality of both the screening and bulk samples was typical of samples from these sites during the last year with near neutral pH, elevated sulfate, arsenic, iron and ammoniacial nitrogen concentrations which has been previously described in Golder (2010a).

Table 2: Mine water quality and arsenic speciation.

	SP10 Outlet Drain	**SumpB _CDW**	**SumpB _CDW**
Date	3 November 2009	3 November 2009	10 November 2009
Sample type	Screening sample	Screening sample	Bulk sample for adsorption testing
Preservation	Filtered onsite for metals/metalloids and preserved for arsenic (III)	Filtered onsite for metals/metalloids and preserved for arsenic (III)	Not preserved
Physico-chemical			
pH	6.8	6.6	6.6
Total suspended solids	30	4.5	-
Total alkalinity (as $CaCO_3$)	350	140	150
Electrical conductivity (mS/m)	530	420	418
Major ions			
Calcium	460	260	270
Magnesium	330	130	130
Potassium	58	41	41
Sodium	440	510	510

	SP10 Outlet Drain	SumpB _CDW	SumpB _CDW
Sulfate	3,600	2,300	2,100
Chloride	35	100	110
Dissolved metals/metalloids			
Aluminium	<0.015	0.017	-
Arsenic (III)	11	4.2	2.7
Arsenic	11	4.6	3.5
Cadmium	<0.00025	<0.00010	<0.00025
Copper	<0.0025	0.0015	<0.0025
Iron	33	9.2	4.7
Lead	<0.00050	0.00081	<0.00050
Manganese	9.4	14	-
Nickel	0.17	0.017	0.018
Zinc	0.044	0.0086	0.0075
Nitrogen and cyanide			
Weak acid dissociable cyanide	0.11	0.56	-
Total cyanide	0.51	1.2	-
Total ammoniacal nitrogen	11	13	-
Total oxidised nitrogen	0.012	0.0085	0.026

Note – All units g/m^3 unless otherwise stated.

3.2 Sieve Analysis of Solids

The dried solid samples were screened determine the appropriate grain size to be used for producing composite samples (Table 3). The loess samples were finer than the schist samples with over 79% of the sample containing grain size of less than 2 mm. The grain size of schist samples ranged between 37.5% and 40.3% of the sample with grain size less than 2 mm and over 80% of the sample with grain size less than 10 mm. To obtain a representative composite sample of the groundwater materials the loess materials were screened to less than 2 mm prior to compositing and the schist samples were screened to less than 10 mm prior to compositing.

Table 3: Initial sieve analysis – prior to composite.

Sample	Fraction less than 2 mm (%)	Fraction less than 10 mm (%)
0.2 m loess	79.3	
0.75 m loess	80.3	
1.5 m loess	79.8	
1 m weathered Schist	40.3	81.3
3 m weathered Schist	38.4	87.6
40 m unweathered	37.5	81.4
80m unweathered	38.3	86.6

The sieve analysis results for the composited samples, after screening to less than 2 mm and less 10 mm for loess and schist respectively, are presented in Table 4. The loess materials consisted of a finer grain size than the schist materials. Over 80% of loess composite sample had grain size of less than 63 µm. Approximately 70% of the weathered schist composite sample was less than 2 mm and approximately 50% was less than 63 µm. The unweathered schist composite sample was the coarsest of the three samples with approximately 50% with grain size less than 2 mm and approximately 25% less than 63 µm.

Table 4: Sieve analysis results of screened composite samples.

Grain size	Fraction greater than or equal to grain size (%)		
	Loess	**Weathered schist**	**Unweathered schist**
2 mm	-	30.7	50.1
1 mm	2.8	33.9	61.8
500 µm	5.3	35.2	66
250 µm	6.8	36.8	69.1
180 µm	7.4	37.6	70
90 µm	12.4	42.8	73.2
75 µm	15.1	45.2	74.3
63 µm	19	47.9	75.5

Note – Composite samples had been previously screened to less than 2 mm for loess and less than 10 mm for schist materials.

3.3 Mine Water Experimental Controls

3.3.1 Bulk sample monitoring

The bulk mine water sample was collected by OceanaGold on 10 November 2009 and stored at room temperature. As the adsorption tests were not carried out immediately following the sampling, there was potential for chemical changes to occur within the sample prior to the start of the adsorption tests, particularly with respect to oxidation and pH. The sample was therefore monitored for changes in pH, dissolved arsenic and dissolved iron (Table 5).

In the period between the collection of the bulk mine water sample and the start of the adsorption tests, the pH increased from 6.6 to approximately 7.5. Approximately 95% of the dissolved iron and 35% of the dissolved arsenic was removed from solution prior to adsorption testing on 26 November. The pH and dissolved arsenic concentration subsequently remained relatively constant from 26 November to 7 December. A further reduction in dissolved iron concentration was observed, with more than 98% of the dissolved iron being removed from solution in the sample by 7 December.

The removal of dissolved iron and arsenic from solution in the bulk sample is most likely due to oxidation of iron (II) to iron (III), resulting in the precipitation of iron oxyhydroxides and adsorption of arsenic to the surface of the precipitate. As such these changes are due to the oxidative environment in the laboratory and may not occur in the ground; however, the measured changes in concentration are relatively minor compared to the high concentration ranges of arsenic used in the testing.

Table 5: Changes in filtered mine water.

Date	pH (pH units)	Arsenic	Iron
13 November 2009	6.6[1]	3.5	4.7
26 November 2009 (date of As (III) testing)	7.5[2]	2.3	0.36
7 December 2009 (date of As (V) testing)	7.6[2]	2.2	<0.1

Note – Units g/m^3 unless otherwise stated, arsenic and iron are the dissolved fraction. [1]Unfiltered result. [2]Filtered through 0.45 µm filter.

3.3.2 Adsorption test controls

Control tests were run for each adsorption test. The control tests were performed to assess if changes in pH and dissolved arsenic concentrations would occur due to the experimental procedure, in the absence of any rock sample. The control tests were expected to confirm that arsenic removal from solution by precipitation or adsorption to container surfaces during the experimental procedure would not be significant.

Each control test was performed by shaking a mine water sample with added arsenic (i.e., spiked) for 24 hours in a test container with an air space of similar volume to the adsorption tests. The analytical uncertainty in the measurement of pH was +/- 0.2 pH units (at 95% confidence), while for dissolved arsenic the analytical uncertainty was typically +/- 6% of the concentration (at 95% confidence).

Taking the analytical uncertainty into account there appears to be a minor increase in pH of the test solutions during the arsenic (V) control tests but no change in pH was detected in the arsenic (III) control tests (Table 6). The control test results indicated that initial and final dissolved arsenic concentrations were the same for all of the test solutions, within the analytical uncertainty of the method, with one exception. The initial and final arsenic concentrations for the sample spiked 100 g/m^3 arsenic (III) was outside the 95% confidence limit, however this difference is still considered to be minor.

The control test results indicate that arsenic precipitation or adsorption reactions unrelated to the solids introduced to the adsorption tests were not having a significant effect on the outcome of the adsorption tests.

Table 6: Mine water control test results.

Sample	Initial pH (pH units)	Final pH (pH units)	Initial arsenic (g/m^3)	Final arsenic (g/m^3)
Arsenic (III)				
Un-spiked mine water	6.6	7.1	2.3	2.1
Spiked mine water 5 g/m^3	6.6	7.2	4.3	4.6
Spiked mine water 10 g/m^3	6.6	7.3	8.9	8.8
Spiked mine water 25 g/m^3	6.6	7.2	22	22
Spiked mine water 100 g/m^3	6.6	7.2	86	98
Spiked mine water 200 g/m^3	6.6	6.9	170	170
Spiked mine water 500 g/m^3	6.6	6.8	430	430
Spiked mine water 1,000 g/m^3	6.6	6.8	840	850
Arsenic (V)				
Un-spiked mine water	6.7	7.0	2.1	2.0
Spiked mine water 50 g/m^3	6.6	6.8	47	48
Spiked mine water 100 g/m^3	6.6	6.7	92	97
Spiked mine water 200 g/m^3	6.5	6.6	180	170
Spiked mine water 500 g/m^3	6.6	6.6	440	440
Spiked mine water 1,000 g/m^3	6.6	6.6	910	870

Note – Arsenic is the dissolved fraction.

3.4 Arsenic (III) Adsorption

3.4.1 pH changes

The adsorption of arsenic is pH dependant. The pH of the test solutions was therefore measured at the start and the end of the adsorption test (Table 7). The final pH of the solutions treated with the loess and weathered schist was lower than the control at between 4.5 and 4.7. The final pH of the solutions treated with unweathered schist was mildly alkaline and slightly higher than the control solutions.

The tests were run at a 1:1 solid to liquid ratio. In the groundwater system the ratio is much higher. It is considered likely that pH changes similar to those observed in the tests also occur within the groundwater system affected by tailings seepage close to the impoundments. This is supported by findings that groundwater pH values down-gradient of the MTI are in the range 5 to 6 (Golder, 2010a).

Table 7: Arsenic (III) adsorption test pH results.

Test	**Initial pH**	**Final pH**			
		Control	**Loess**	**Weathered schist**	**Unweathered schist**
Un-spiked mine water	6.6	7.1	4.8	4.6	7.7
Spiked mine water 5 g/m^3	6.6	7.2	4.8	4.6	7.7
Spiked mine water 10 g/m^3	6.6	7.3	4.7	4.6	7.7
Spiked mine water 25 g/m^3	6.6	7.2	4.7	4.6	7.7
Spiked mine water 100 g/m^3	6.6	7.2	4.7	4.6	7.7
Spiked mine water 200 g/m^3	6.6	6.9	4.6	4.6	7.6
Spiked mine water 500 g/m^3	6.6	6.8	4.6	4.6	7.6
Spiked mine water 1,000 g/m^3	6.6	6.8	4.5	4.5	7.6

Note – All units are pH units. All samples except the control run at 1:1 solids to liquids ratio.

3.4.2 Maximum arsenic adsorption

The adsorption of arsenic was assessed by measuring the dissolved arsenic concentrations at the start and end of the tests (Table 8) and calculating the amount of arsenic adsorbed per mass of solid (Figure 2). The amount of arsenic adsorbed is calculated by subtracting the final dissolved arsenic concentration from the initial dissolved arsenic concentration. As the tests were performed at a solid to liquid ratio of 1:1 and the control tests indicate there are no other significant effects to be taken into account, the result of the subtraction is the mass of arsenic adsorbed per kilogram of solid material. The results are typical of adsorption testing with an initially steep adsorption curve which flattens off as the adsorption sites become more saturated.

It is apparent from the adsorption curves that saturation of possible arsenic adsorption sites on the solids tested may not have been reached under the test conditions. However, the calculated adsorption capacities are within the range of reported literature values (Table 1). The adsorption capacity was greatest for the most oxidised material (the loess) and lowest for the least oxidised material (the unweathered schist). This observation is consistent with the arsenic adsorption capacity being driven by adsorption onto oxidised iron surfaces. The maximum adsorption capacities measured were 510 mg/kg, 460 mg/kg and 270 mg/kg for loess, weathered schist and unweathered schist, respectively.

Table 8: Arsenic (III) adsorption test dissolved arsenic concentrations.

Test	Initial	Final			
		Control	Loess	Weathered schist	Unweathered schist
Un-spiked mine water	2.3	2.1	0.059	0.076	0.061
Spiked mine water 5 g/m^3	4.3	4.6	0.14	0.16	0.12
Spiked mine water 10 g/m^3	8.9	8.8	0.39	0.41	0.36
Spiked mine water 25 g/m^3	22	22	1.3	1.4	1.5
Spiked mine water 100 g/m^3	85	98	13	14	18
Spiked mine water 200 g/m^3	170	170	34	36	54
Spiked mine water 500 g/m^3	430	430	130	150	220
Spiked mine water 1,000 g/m^3	840	850	330	380	570

Note – Units g/m^3. All samples except the control run at 1:1 solids to liquids ratio.

3.4.3 Distribution coefficient

The K_d value for each test result was calculated for the arsenic (III) results from the following equation:

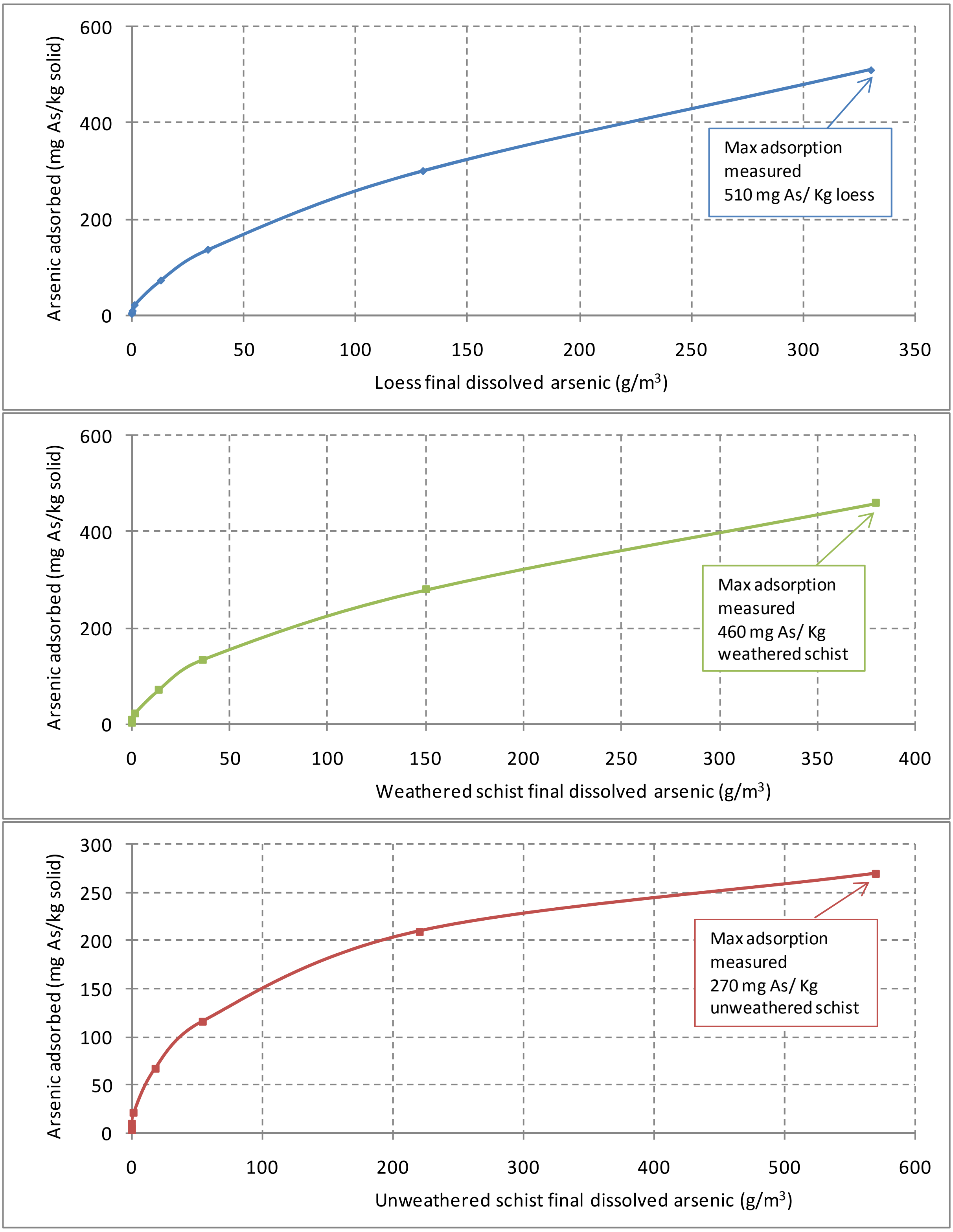
$$Kd = A_i/C_i \quad \text{(L/kg)}$$

Where:

A_i = Mass of arsenic adsorbed per mass of solid at equilibrium (mg/kg).

C_i = Mass of arsenic in solution at equilibrium (mg/L).

The calculated K_d values are dependent on the solids being tested and on the initial concentration of arsenic in the test solution. The calculated K_d decreases as the initial concentration of the solution increases (Figure 3). This trend indicates that the potential for arsenic III to be adsorbed onto the surrounding rock or soil mass decreases as the initial arsenic concentration increases.



Max adsorption measured 510 mg As/ Kg loess

Max adsorption measured 460 mg As/ Kg weathered schist

Max adsorption measured 270 mg As/ Kg unweathered schist

Figure 2: Arsenic (III) adsorption results

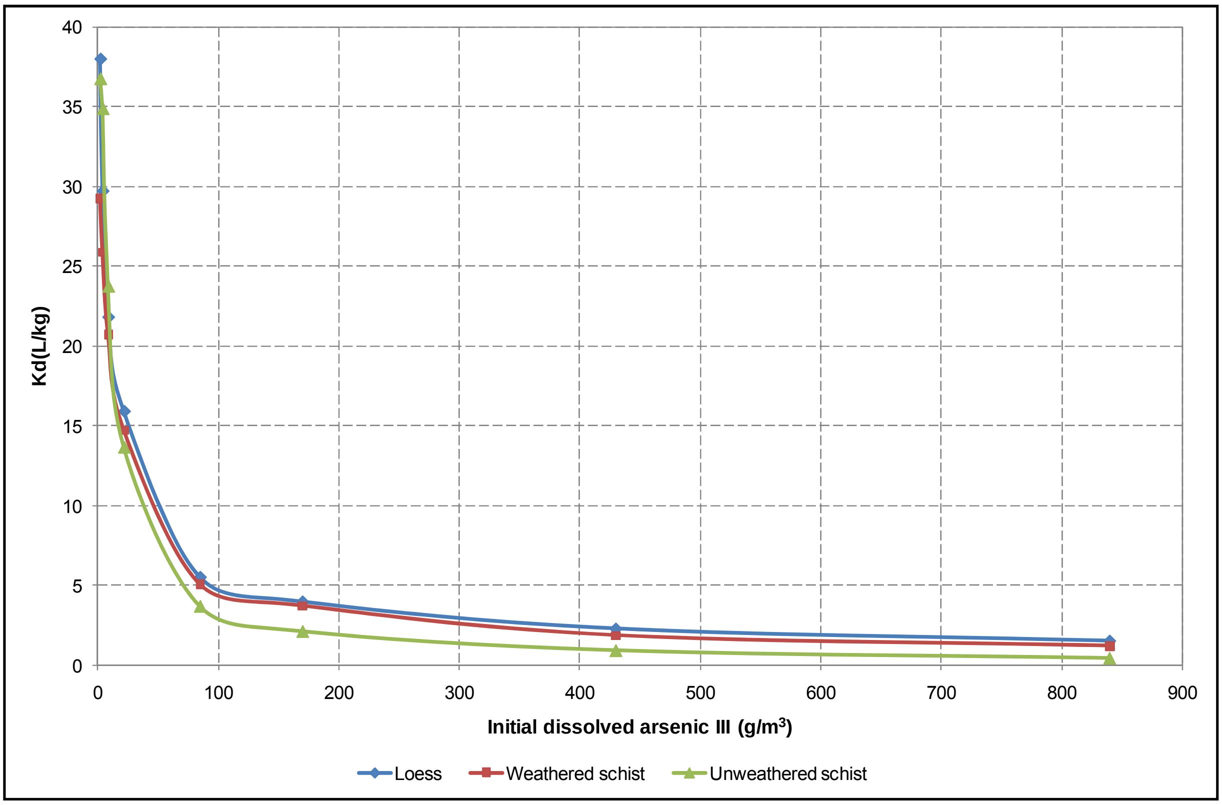


Figure 3: Calculated K_d values for arsenic III.

3.5 Arsenic (V) Adsorption

3.5.1 pH changes

As with the arsenic (III) experiments, the pH of the test solutions was measured at the beginning and end of each test (Table 9). The pH changes observed in the arsenic (V) adsorption tests followed the same trend to those observed in the arsenic (III) tests. Mixing the test solution with the loess or the weathered schist resulted in the final solutions being acidic with a pH of between 4.6 and 4.9, while the unweathered schist resulted in mildly alkaline solution elevated compared to the controls.

Table 9: Arsenic (V) adsorption test pH results.

	Initial	**Final pH**			
		Control	**Loess**	**Weathered schist**	**Unweathered schist**
Un-spiked mine water	6.7	7.0	4.7	4.6	7.6
Spiked mine water 50 g/m^3	6.6	6.8	4.7	4.6	7.7
Spiked mine water 100 g/m^3	6.6	6.7	4.7	4.6	7.8
Spiked mine water 200 g/m^3	6.5	6.6	4.7	4.6	7.8
Spiked mine water 500 g/m^3	6.6	6.6	4.8	4.7	7.6
Spiked mine water 1,000 g/m^3	6.6	6.6	4.9	4.8	7.4

Note – All units pH units. All samples except the control run at 1:1 solids to liquids ratio.

3.5.2 Maximum arsenic adsorption

The adsorption of arsenic (V) followed the same trend as arsenic (III) with the more weathered materials having higher adsorption capacities (Table 10 and Figure 4). As with the arsenic (III) test it is possible that maximum adsorption capacity of the solids tested was not reached during the testing. The adsorption capacity of arsenic (V) was greater than the adsorption capacity of arsenic (III). The maximum measured adsorption capacities were 900 mg/kg, 790 mg/kg and 240 mg/kg for loess, weathered schist and unweathered schist, respectively.

Table 10: Arsenic (V) adsorption results.

Test	**Initial**	**Final**			
		Control	**Loess**	**Weathered schist**	**Unweathered schist**
Un-spiked mine water	2.1	2.0	0.096	0.014	0.018
Spiked mine water 50 g/m^3	47	48	0.037	0.065	2.4
Spiked mine water 100 g/m^3	92	97	0.093	0.24	16
Spiked mine water 200 g/m^3	180	170	0.29	0.79	68
Spiked mine water 500 g/m^3	440	440	1.9	13	300
Spiked mine water 1,000 g/m^3	910	870	10	120	670

Note – Units g/m^3. All samples except the control run at 1:1 solids to liquids ratio.

3.6 Assumptions

The following assumptions were required to generate the maximum adsorption capacities and K_d values presented above:

- The quality of the bulk seepage water sample used for the adsorption tests is representative of long-term tailings seepage water quality.
- The differences between field in-situ redox conditions and laboratory testing conditions do not significantly affect the arsenic adsorption capacities;
- No precipitation occurring during the adsorption testing;
- The pH changes observed in the adsorption testing are similar to those that occur *in-situ*. This implies that dilution by natural groundwater in the system does not affect the K_d value.

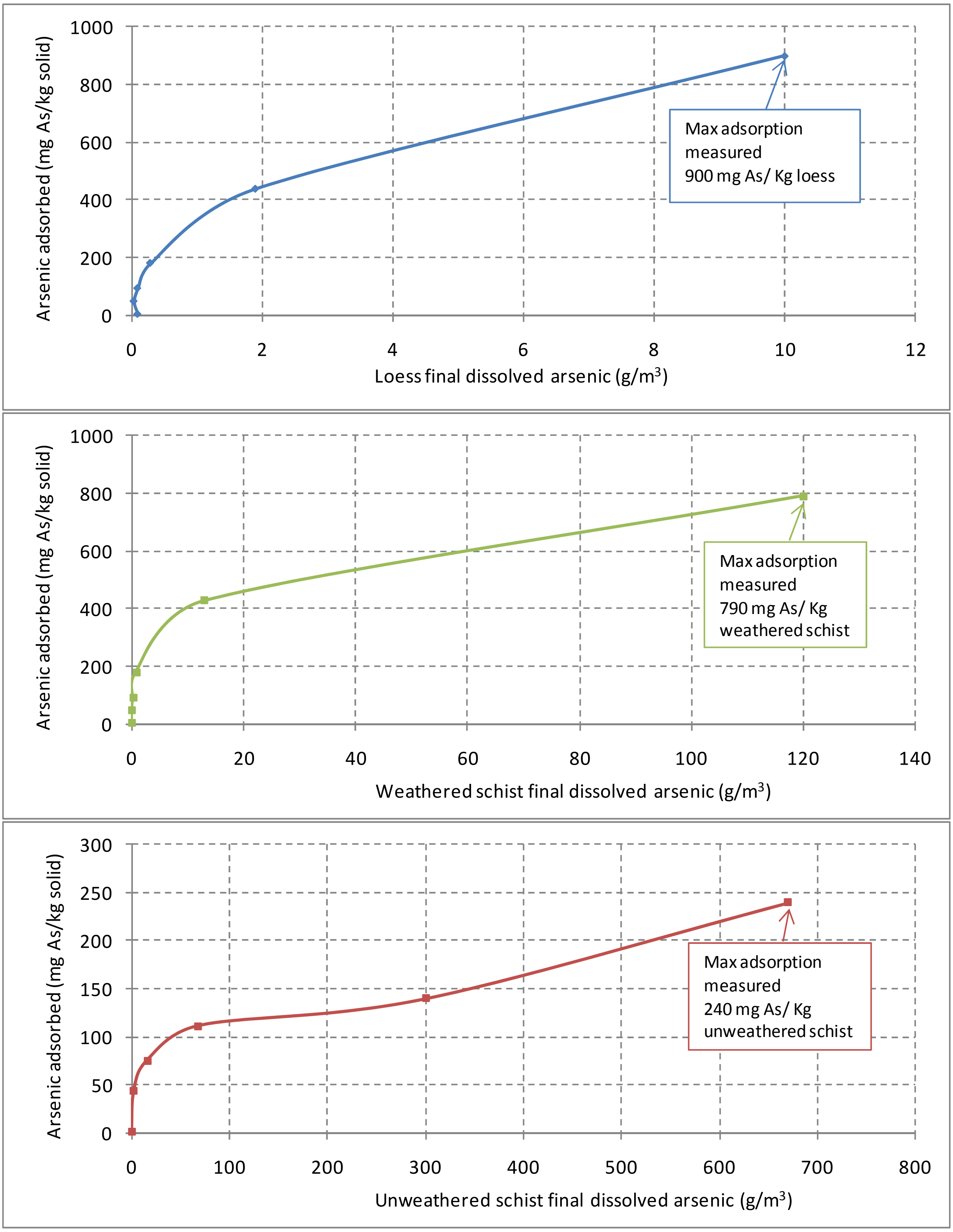


Figure 4: Arsenic (V) adsorption results

4.0 REACTIVE CONTAMINANT TRANSPORT MODEL INPUTS

4.1 Arsenic Oxidation State

The reactive contaminant transport groundwater model to be created for the site requires inputs for the maximum adsorption capacities and K_d for the three aquifer materials. The arsenic contained in tailings seepage water is predominately in the arsenic (III) oxidation state (Table 2). As the tailings seepage water enters the groundwater system there is some potential for the arsenic (III) to be oxidised to the arsenic (V) oxidation state. The degree to which this oxidation occurs depends, amongst other factors, on:

- The local ratios between tailings seepage and natural groundwater, and
- The rate of oxygen diffusion into the groundwater system.

The oxidation state of arsenic in the groundwater system down-gradient from the tailings impoundments has not been clearly defined. It is likely the dissolved arsenic in the groundwater system immediately beneath the existing tailings impoundments is predominantly in a reduced state. It is possible the proportion of oxidised arsenic in the seepage water may increase with increasing distance from the impoundment. As a conservative measure the maximum adsorption and K_d inputs for the reactive contaminant transport model are based on the arsenic (III) test results, which indicated lower maximum adsorption capacities than arsenic (V) test results.

4.2 Maximum Arsenic Adsorption Capacity

The schist samples used in the adsorption tests were obtained from zones characterised by highly fractured rock. This targeted rock sampling was applied in order to avoid the need to crush samples prior to testing and thereby artificially influence the adsorption capacity of the rock through the creation of fresh rock surfaces. As a result of the targeted sampling, the exposed rock surface area per kilogram of sample is greater than what could be considered to be an average for the MGP site. This difference between the samples and the intact schist rock mass increases with increasing depth below the ground surface. Areas of the site with relatively high fault and fracture density are more similar to the test samples than areas of the site that have been subjected to less tectonic disturbance.

An above average ratio of surface area to rock volume also implies an above average number of exposed arsenic adsorption sites. To extrapolate the adsorption test data from the laboratory scale to the field scale, it is necessary to correct for the differences between the test samples and the intact rock at the site. This scaling is not necessary for the loess results as the arsenic adsorption derived for the schist is not dependent on fracture density.

The groundwater flow and mass transport model used to simulate seepage losses from the MGP site incorporates four schist weathering zones, with the uppermost zone also incorporating the overlying loess layer. The uppermost weathered zone is approximately 20 m thick, with the underlying weathered zone being approximately 40 m thick.

The method used to derive a correction for the change from laboratory scale to field scale maximum adsorption values incorporated the following steps:

1) Evaluate available drill core from the BTSF site to determine the percentage of the core per metre interval that is accessible to arsenic transport by advection or diffusion. Each metre of core, as measured from the ground surface down, is thereby allocated an arsenic accessibility between 0% and 100%.
    a. For loess, this fraction was automatically defined as 100% as groundwater seepage through the loess is not controlled by fracture density. All of the loess is therefore available for arsenic adsorption.

   b. For the weathered schist it was judged that rock core showing oxidation of iron would also be accessible to arsenic transported in the groundwater. The non-oxidised rock was therefore considered to be unavailable for arsenic adsorption. The extent of oxidation was evaluated through visual inspection of photographs of drill core for indications of oxidised iron. The drill core was produced from investigation drilling at the BTSF site.

   c. For the unweathered schist it was judged that only fractures and crushed sections of core would be accessible for groundwater flow. To that extent a zone of 1 mm on either side of the fracture was defined as being eventually accessible to arsenic transported in the groundwater. Crushed rock zones identified in the core ware assumed to have similar grain size properties to the samples of unweathered schist provided by OGL. The crushed rock within these zones was therefore considered to have a similar maximum adsorption capacity to the test samples.

2) For every metre of core, multiply the appropriate maximum arsenic (III) adsorption value derived from the adsorption tests by the arsenic accessibility number as defined above. The result is considered to be the maximum arsenic adsorption capacity for that section of core. In effect, this calculation results in the maximum arsenic adsorption capacity of the rock decreasing with decreasing intensity of fracturing and weathering.

3) Calculate the median value for the arsenic adsorption capacity defined in Step 2 for:

   a. The top 20 m of core in the nine drillholes evaluated, which is defined in the contaminant transport model as highly weathered schist.

   b. The immediately underlying 40 m of core in the nine drillholes evaluated, which is defined in the model as moderately weathered schist.

Dissolved arsenic transported through schist at depths greater than 60 m was considered for modelling purposes to be conservatively transported.

The results of the arsenic accessibility calculation are summarised in Figure 5. The results of the correction for the maximum arsenic adsorption capacity of the rock mass for each drillhole and for the mass transport model layers are summarised in Figure 6. The corrected maximum arsenic (III) adsorption capacities applied to the contaminant mass transport model for the MGP are presented in Table 11.

Table 11: Maximum As (III) adsorption capacities used in MODFLOW model.

Model layer [1]	**Maximum adsorption capacity based on arsenic (III) testing (mg As/kg solid)**	**Corrected maximum arsenic (III) adsorption capacity (mg As/kg solid)**
Highly weathered schist	510 (loess), 460 (weathered schist) and locally 270 (unweathered schist).	230
Moderately weathered schist	270 and locally 460.	46
Slightly weathered schist [2]	270	14

Note: 1) Layers defined from permeability zones applied to the groundwater model.
2) Arsenic adsorption results not applied in mass transport model.

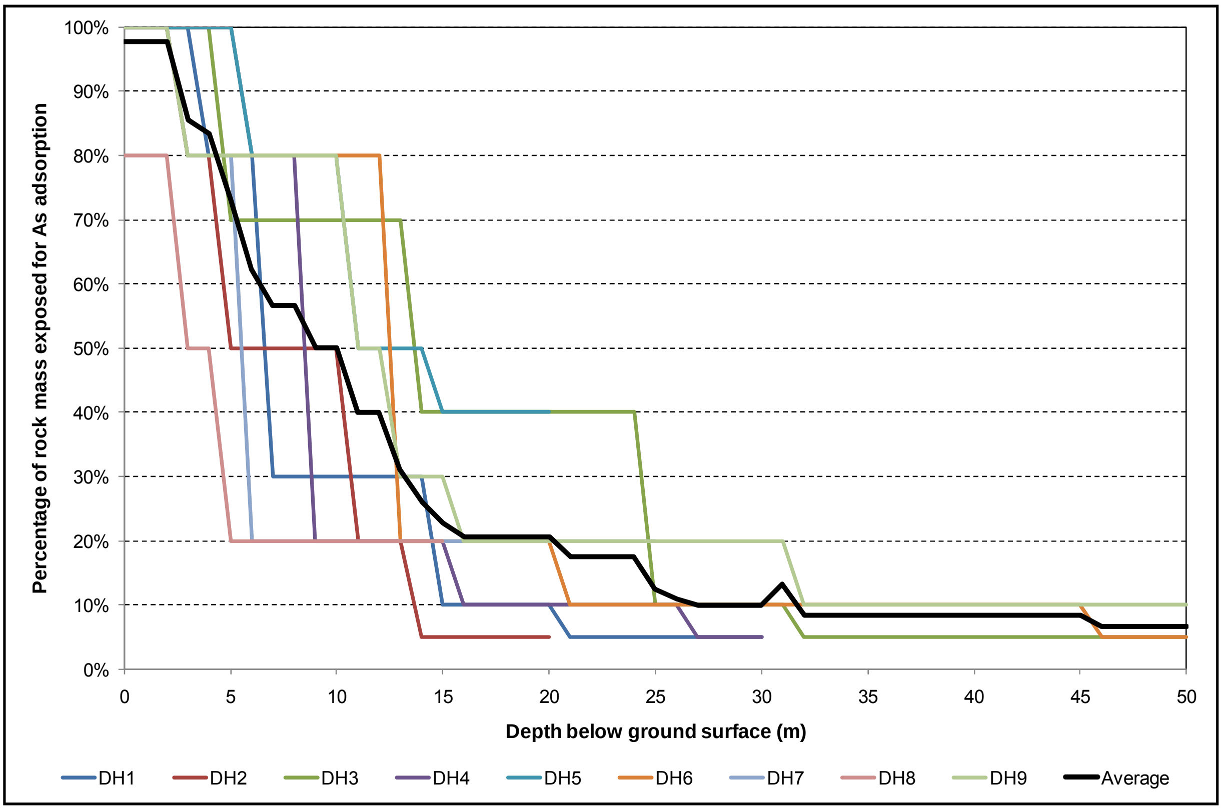


Figure 5: Percentage of rock mass exposed for arsenic adsorption.

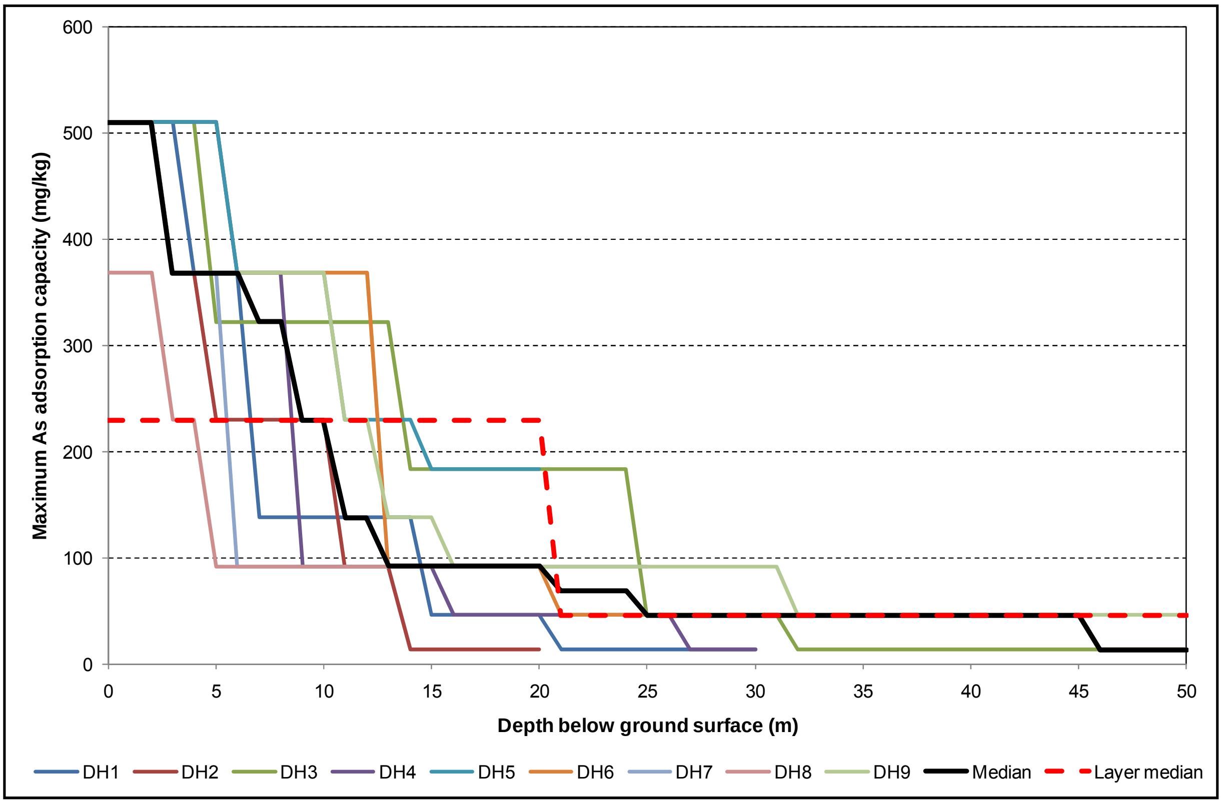


Figure 6: Maximum arsenic adsorption capacity.

4.3 Distribution Coefficient

Measured arsenic concentrations in tailings seepage from the MTI and SPI are approximately 4 g/m^3 and 14 g/m^3, respectively (Golder, 2010a). A dissolved arsenic concentration of 14 g/m^3 has been applied to the tailings seepage for modelling of contaminant transport from both existing tailings impoundments into the long term future (Golder, 2010b). The K_d derived from the arsenic (III) adsorption testing is approximately 20 L/kg for all three solids tested (Figure 7). This K_d is applied to the two uppermost weathering zones of the mass transport model for the MGP site. As previously noted, dissolved arsenic transported through schist at depths greater than 60 m was considered for modelling purposes to be conservatively transported.

A dissolved arsenic concentration of 2.4 g/m^3 has been applied to the tailings seepage for modelling of the proposed BTSF (Golder 2010a). Although the corresponding K_d derived for this concentration is approximately double that derived for the wider MGP model, it was considered that the difference would not result in a significant difference to the model outcome. Consequently, for consistency, the K_d of 20 L/kg is also applied to the seepage model of the BTSF.

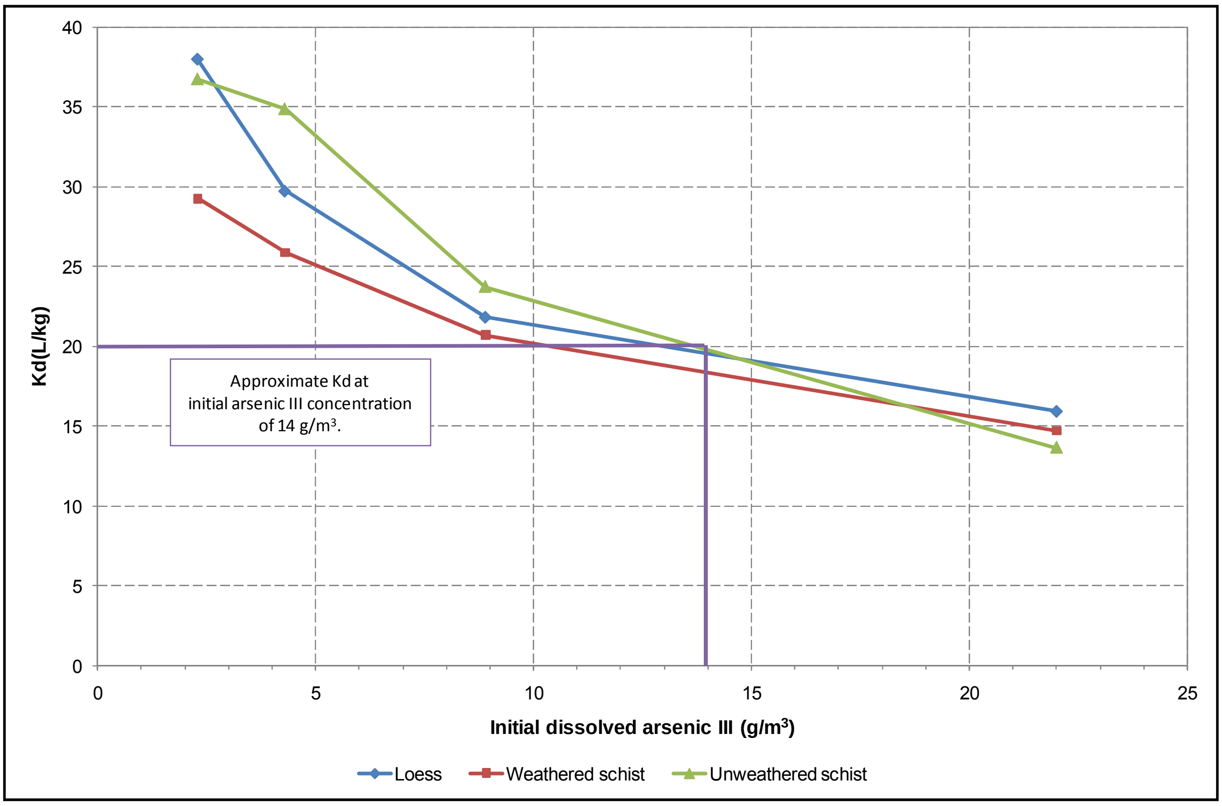


Figure 7: Calculated K_d for arsenic III at initial concentrations characteristic of MGP tailings seepage.

4.4 Adsorption isotherm

For the purpose of modelling arsenic adsorption from tailings seepage at the MGP site the maximum adsorption capacity and the K_d derived from analysis of the adsorption test results need to be fitted to an adsorption isotherm equation. Three options are immediately available for fitting the calculated maximum adsorption capacity and the K_d to an isotherm. These options are:

- Application of a linear isotherm without taking into account the maximum adsorption capacity of the solids;

- Application of the Freundlich isotherm without taking into account the maximum adsorption capacity of the solids; and
- Application of a Langmuir isotherm, which takes into account both K_d and the maximum adsorption capacity.

It was considered inappropriate to use the linear or Freundlich isotherms, due to the implied lack of an upper limit on the adsorption capacity of the rock mass. A Langmuir isotherm was therefore derived to represent the observations from the adsorption tests. The Langmuir isotherm (Zheng & Wang 1998) takes the general form of the equation:

$$C_a = \frac{K_d S_a C}{1 + K_d C}$$

Where:

C_a = Sorbed contaminant concentration (in kg/kg).

C = Solute concentration (mg/L)

S_a = The number or adsorption sites available (kg/kg)

K_d = The distribution coefficient. (L/kg)

For the purpose of modelling mass transport across the site, the maximum sorbed contaminant concentration has been derived in Section 4.2. The K_d values have been derived directly from the adsorption testing for the materials represented in the contaminant transport model. The number of adsorption sites available for the rock materials tested has not been calculated from the adsorption tests. An approximation suitable for use in the transport modelling can be calculated by rearranging the equation presented above.

$$S_a = \frac{C_a(1 + K_d C)}{K_d C}$$

Applying the values appropriate for the model layers generates the following results:

Layer 1:

$$S_a = \frac{0.00023 \times (1 + 2 \times 10^{-5} \times 14)}{2 \times 10^{-5} \times 14}$$

$$S_a \approx 0.82$$

Layer 2:

$$S_a = \frac{0.000046 \times (1 + 2 \times 10^{-5} \times 14)}{2 \times 10^{-5} \times 14}$$

$$S_a \approx 0.1645$$

In terms of modelling outcomes however the results with respect to dissolved and adsorbed contaminant concentrations are equivalent. The parameters applied to the Langmuir isotherm for the highly weathered schist and loess layer and the moderately weathered schist layer in the contaminant transport model are summarised in Table 12.

Table 12: Langmuir isotherm model input parameters for arsenic (III).

Parameter	**Units**	**Highly weathered schist layer (1)**	**Moderately weathered schist layer (1)**
C – solute concentration (maximum)	mg/L	14	14
S_a – sorption sites available	kg/kg	0.82	0.1645
K_d – distribution coefficient	L/kg	2 x 10^{-5}	2 x 10^{-5}
C_a – maximum sorbed concentration	mg/kg	230	46

Note: 1) Layers defined from permeability zones applied to the groundwater model.
2) Applicable to maximum sorbed arsenic concentration of 14 mg/L.

Applying these Langmuir isotherm to the mass transport model produces conservative results compared to applying the linear isotherm. In effect, the Langmuir isotherm reduces the adsorbed to dissolved ratio of a contaminant across the full range of concentrations in a model. The ratio of sorbed to dissolved arsenic that apply to the highly weathered schist and the moderately schist zone represented in the model are presented in Figure 8 and Figure 9. As the highest arsenic concentration represented in the models is approximately 14 mg/L, this limits the maximum sorbed concentrations the models will generate.

In the moderately weathered schist zone of the model the ratio of sorbed arsenic to dissolved arsenic is a factor of five less than what would be indicated by applying the linear isotherm. This difference is considered to be a conservative factor in the model, with the potential adsorption of arsenic onto the rock mass probably being closer to the linear isotherm until the maximum adsorbed concentration is neared.

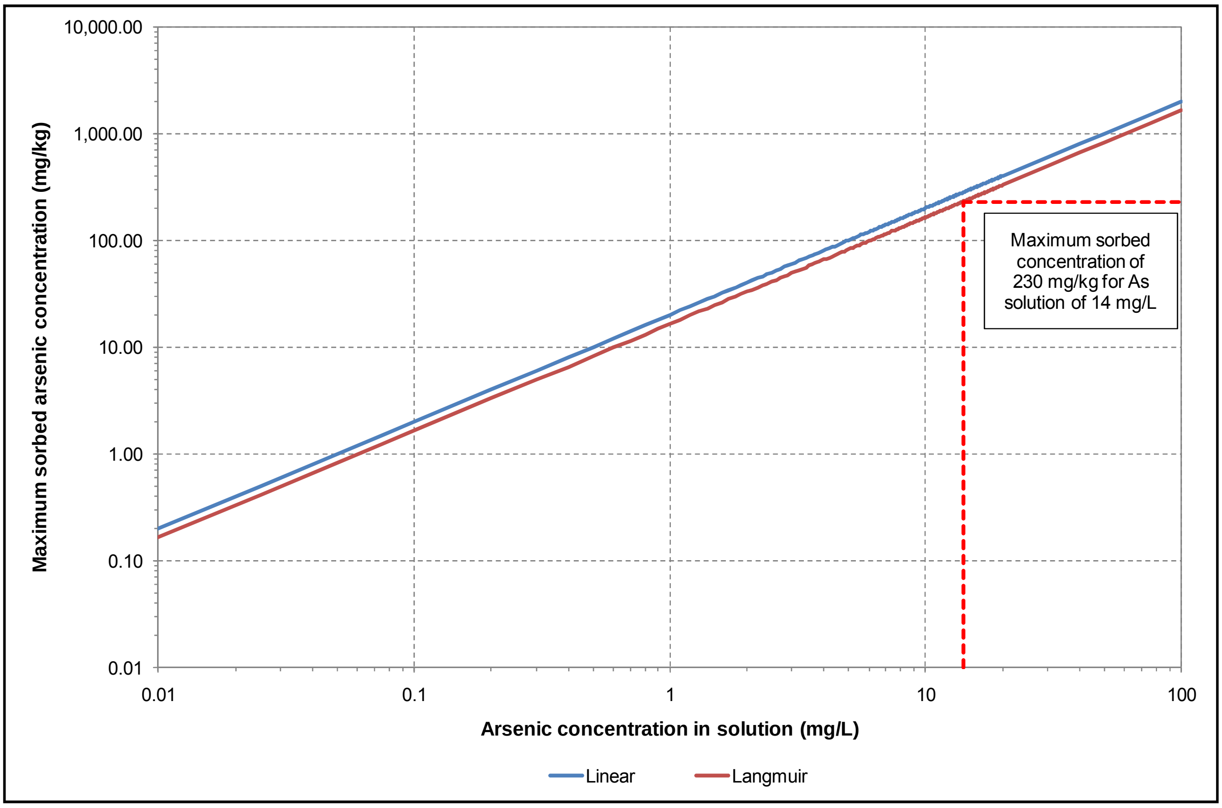


Figure 8: Sorbed versus solution concentrations for highly weathered schist and loess model layer.

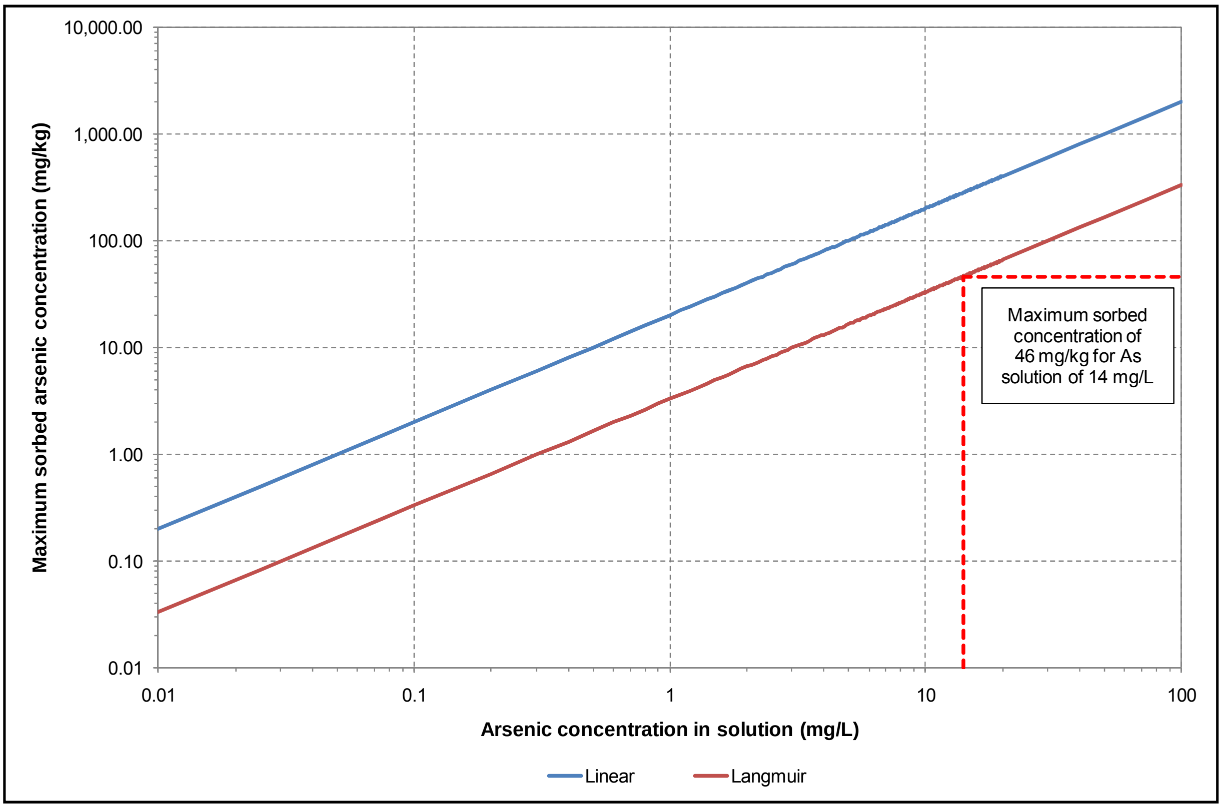


Figure 9: Langmuir curve outcome for moderately weathered schist model layer

4.5 Assumptions

In addition to the assumptions incorporated in evaluating the data from the arsenic adsorption tests, the following assumptions have been made in deriving the maximum adsorption capacities and K_d values to be applied to the contaminant transport models:

- The rock and soil samples collected are characteristic of the groundwater system at the site.
- The maximum arsenic seepage concentration in tailings water is approximately 14 g/m^3.

5.0 SUMMARY AND CONCLUSIONS

Batch adsorption tests have been used to determine the arsenic (III) and (V) adsorption capacities of loess, weathered schist and unweathered schist samples collected from the MGP site. The results of these tests have been used as input parameters for contaminant mass transport models simulating tailings seepage losses from the MGP.

The test program undertaken to evaluate the K_d and maximum arsenic adsorption for the samples obtained from the MGP site has incorporated a number of assumptions as discussed in this report. These assumptions define the limits to the applicability of the test results. The test program was however designed to replicate the seepage water and seepage conditions associated with the areas surrounding the existing and proposed tailings storage facilities as much as possible.

The maximum calculated adsorption capacities for each of the solids tested were greater for arsenic (III) than for arsenic (V). As the reduced form is the main arsenic species present in the MGP tailings seepage water, the maximum arsenic adsorption capacity and the K_d for arsenic (III) have been applied in recent mass transport models simulating the MGP site.

For both arsenic (III) and arsenic (V) the highest maximum adsorption capacity was associated with the loess tested and the lowest was associated with the unweathered schist tested. The maximum adsorption capacity results are:

■ Loess	As (III) 510 mg/kg solids	As (V) 900 mg/kg solids
■ Weathered schist	As (III) 460 mg/kg solids	As (V) 790 mg/kg solids
■ Unweathered schist	As (III) 270 mg/kg solids	As (V) 240 mg/kg solids

The adsorption tests did not conclusively exceed the maximum adsorption capacity of the materials within the range of arsenic concentrations tested. Due to this limitation it is likely that the maximum adsorption derived from these tests understates the maximum potential adsorption for the samples tested.

The maximum adsorption capacity results from the laboratory tests on the schist samples needed to be corrected for differences in exposed rock surfaces between the test samples and in-situ rock. This correction has been done on the basis of the percentage of the rock mass exposed to dissolved arsenic transported in the groundwater system. As this percentage varies locally, the correction was calculated to correspond to the weathering zones applied in models constructed by Golder to simulate contaminant transport in groundwater across the MGP site. The corrected maximum arsenic adsorption values applied to the contaminant transport models are:

- 230 mg/kg for the upper 20 m of the weathered schist and loess.
- 46 mg/kg for the moderately to slightly weathered schist between 20 m and 60 m below the ground surface.
- Conservative transport at depths below 60 m.

The K_d derived from the adsorption tests varies with the initial concentration of arsenic in the test solution. The maximum arsenic concentrations in seepage water from the existing tailings impoundments is approximately 14 g/m^3. The K_d for arsenic (III) corresponding to this value is approximately 20 L/kg. This K_d has been applied to the contaminant transport models for the site.

For contaminant transport modelling purposes, a Langmuir isotherm equation has been derived for each weathering layer in the contaminant transport models to correspond to the maximum adsorption and K_d values presented above. The input parameters and values for the Langmuir equations applied to the contaminant transport models of the MGP site are presented in Table 12.

Validation of the results from this test program may be achieved through a field scale test program. Experience however indicates that such tests would require considerable time to generate measurable results. It is clear that arsenic has been adsorbed by the loess and weathered schist beneath the MTI since tailings storage began approximately 19 years ago. Concentrations of conservatively transported contaminants began increasing at the detection wells in Maori Tommy Gully, down-gradient from the MTI, within a year of tailings storage beginning at the site. To date there is no indication of arsenic from the MTI in samples obtained from the detection wells in Maori Tommy Gully.

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APPENDIX A

Report Limitations

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APPENDIX D

TTTSF groundwater model structure and calibration

1.0 INTRODUCTION

OceanaGold (NZ) Ltd (OceanaGold) has retained Golder Associates (NZ) Limited (Golder) to develop a 3D groundwater model to simulate seepage losses from the proposed Top Tipperary Tailings Storage Facility (TTTSF) and down-gradient contaminant transport. The models developed are summarised in the TTTSF Hydrogeological Assessment Report, to which this Appendix is attached.

In addition to this introductory section, this Appendix contains the following sections:

- Section 2 outlines the objectives of the numerical modelling and the models that were developed to achieve these objectives.
- Section 3 summarises the model extent and structure of the groundwater models developed to simulate the TTTSF.
- Section 4 summarises the calibration procedure for the steady state model of the existing site, on which the TTTSF model is based. The outcomes of the calibration process with respect to simulating the groundwater system at the proposed TTTSF are documented.

The conceptual groundwater model for the site has been documented in the body of the report and is not repeated in this Appendix.

Documentation for all reports referred to in this Appendix is presented in the reference list provided with the main report.

Industry standard groundwater flow and mass transport modelling packages were used for the numerical modelling. The Visual MODFLOW Pro software package was used to construct the groundwater model.

The groundwater flow field in the model and physical flow calibration procedures was calculated using MODFLOW 2000 public domain code from the United States Geological Survey. The mass transport simulation was calculated utilising the MT3D99 code attached to Visual MODFLOW package.

2.0 NUMERICAL MODELS AND OBJECTIVES

The objectives of the three models that were developed to simulate different stages of the TTTSF operational and post-closure period are outlined below:

- Model 1 – steady state model of current site conditions (calibration model). This model is calibrated against measured piezometric data from the proposed TTTSF area. There is no variation in recharge to the site or groundwater levels with time in this steady state model. For this reason, calibration of the model did not extend to assessment of the values applied to the hydrogeological storage parameters.
- Model 2 – transient model designed to simulate conditions during the operational phase of the TTTSF. The piezometric head within the tailings body during the operational period of the TTTSF has been maintained at an elevation equivalent to the final tailings surface elevation through the use of constant head cells. Contaminants were introduced as constant concentrations over the top of the tailings body to simulate tailings deposition during the operational phase of the TTTSF. This model serves to set up the initial groundwater and contaminant concentration conditions for Model 3.
- Model 3 – transient model designed to simulate conditions at and after TTTSF closure. The model was designed to simulate groundwater flow and contaminant transport conditions up to 150 years after closure of the TTTSF. The constant head boundary cells over the tailings body, which served to simulate tailinigs deposition in Model 2, were not applied for this simulation. Instead a recharge rate of 32 mm/yr was applied over the tailings body. Contaminants were introduced to the tailings body through the recharge concentration boundary condition. The recharge concentration boundary condition specifies the concentration of each species accompanying the recharge flux in the flow model. The final hydraulic head conditions in Model 2 were used as the initial head conditions for Model 3.

Dissolved and adsorbed contaminant concentrations at the end of the Model 2 simulation were used as the initial concentrations in Model 3.

3.0 MODEL DESIGN

3.1 Model Extent

The numerical groundwater model developed for this study is based on and aligned to the MGP site grid. The limits of the model (Figure D1) are as follows:

- 70,000 m to 74,500 m (MGP grid) East – West.
- 10,000 m to 15,500 m (MGP grid) North – South.
- Total modelled area is 24.75 km^2.
- The model base is set at 0 mRL, approximately 270 m below the Deepdell Creek invert at its lowest point in the modelled area.

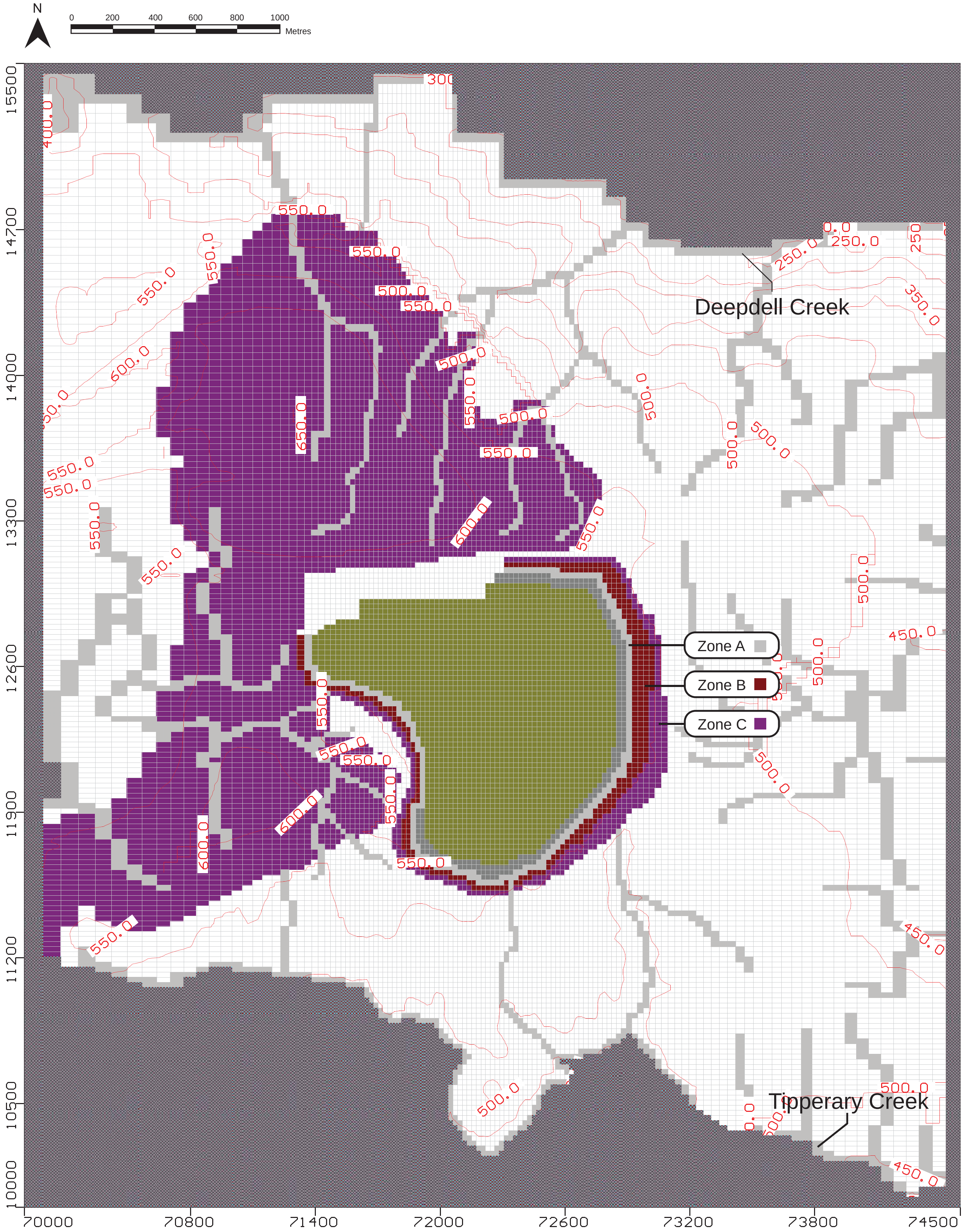
3.2 Model structure

The numerical groundwater flow model incorporates the following main features:

- A total of 7 layers have been defined in the model to represent the different natural hydrogeological units and the structures of the TTTSF (Figure D2).
- The main natural drainage channels in the model domain including Tipperary Creek, Deepdell Creek and Cranky Jims Creek, as well as the main tributaries to these creeks were simulated using drainage cells (Figure D2).
- Frasers Pit and Golden Point Pit are simulated at the western boundary of the model using drainage cells.
- Regional groundwater recharge is applied to the uppermost active cells at a rate of 32 mm/year.
- The hydraulic conductivity parameters applied (Table D1) are the same those applied to the Kingett Mitchell (2005a) model used to simulate the wider MGP area.
- The storage parameters applied to the schist rock mass on a regional basis (Table D2) are consistent with those applied in existing calibrated models of the MGP. The storage and porosity values applied to the tailings mass have been based on survey and mass balance data for the MTI as well as from documented values for similar tailings impoundments worldwide.

In Model 2 and Model 3 the TTTSF has been simulated in the numerical model through:

- Increasing the thickness of the uppermost model layer to match the final proposed form of the TTTSF.
- Matching the hydraulic conductivity and storage parameters within the simulated TTTSF to correspond to those defined above for the embankment and tailings materials.
- Defining drainage cells to simulate the planned construction of underdrains in gullies that intersect the TTTSF footprint.
- Defining drainage cells to simulate chimney drains and collector drains to be constructed within the upstream face of the embankment. The conductance values applied to the drainage cells have been defined to ensure the overlying embankment remains in an unsaturated state. The embankments for the MTI and SP11 have remained in an unsaturated state due to the installation of a low permeability upstream liner and substantial drainage systems. It has been assumed that similar drainage efficiencies would be achieved for the TTTSF embankment.



Legend

🟪	Waste rock stack	⬜	Surface drains
◼	Coarse tailings material	—	Ground surface contours
🟩	Fine tailings material	▩	Inactive cells

Schematic only, not to be interpreted as an engineering design or construction drawing

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TITLE MODEL GRID STRUCTURE

MARCH 2011

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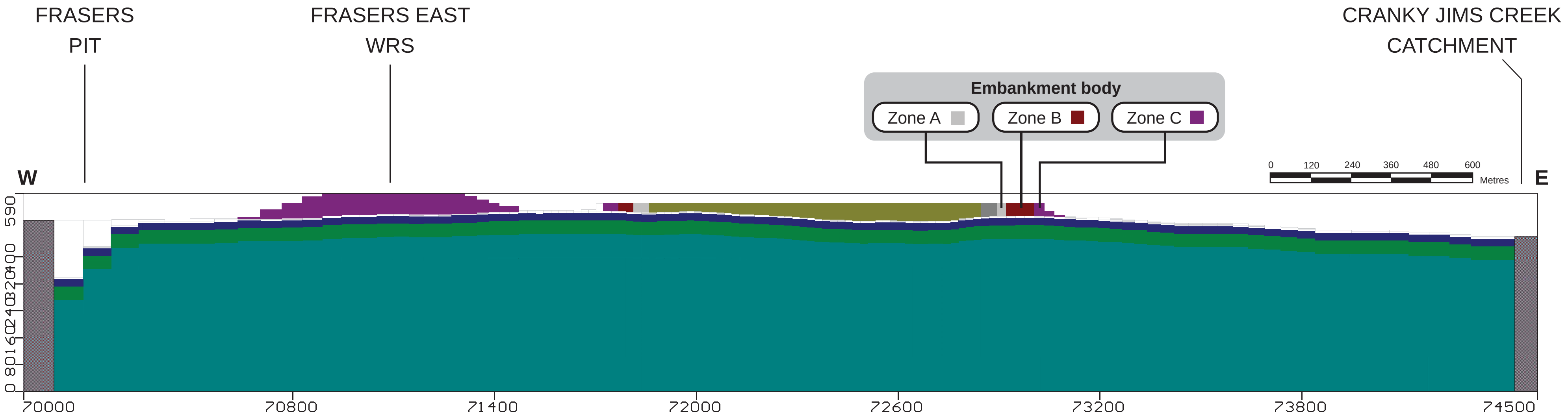
D1



Legend

- ☐ Highly weathered schist
- ■ Moderately weathered schist
- 🟩 Slightly weathered schist
- ■ Unweathered schist
- ■ Fine tailings
- ■ Coarse tailings
- 🟪 Waste rock stack
- ▨ Inactive cells

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TITLE TYPICAL EAST WEST CROSS SECTION THROUGH TTTSF

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D2

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Table D1: Hydraulic conductivity values applied to groundwater model.

Geological feature (1)	K_X	K_Y	K_Z
Highly weathered schist	3.5×10^{-7}	1.0×10^{-6}	2.5×10^{-7}
Moderately weathered schist	1.0×10^{-7}	2.5×10^{-7}	6×10^{-8}
Slightly weathered schist	5.0×10^{-9}	9.0×10^{-9}	1.0×10^{-9}
Unweathered schist	1.0×10^{-9}	5.0×10^{-9}	5.0×10^{-10}
Embankment Zone A	1.0×10^{-7}	1.0×10^{-7}	1.0×10^{-7}
Embankment body – Zone B	5.0×10^{-6}	5.0×10^{-6}	5.0×10^{-6}
Embankment body – Zone C and WRS	1.0×10^{-6}	1.0×10^{-6}	1.0×10^{-6}
Fine tailings	2.0×10^{-7}	2.0×10^{-7}	2.0×10^{-7}
Coarse tailings	5.0×10^{-6}	5.0×10^{-6}	5.0×10^{-6}

Notes: 1) All units in m/s.

Table D2: Storage property values applied in the groundwater model.

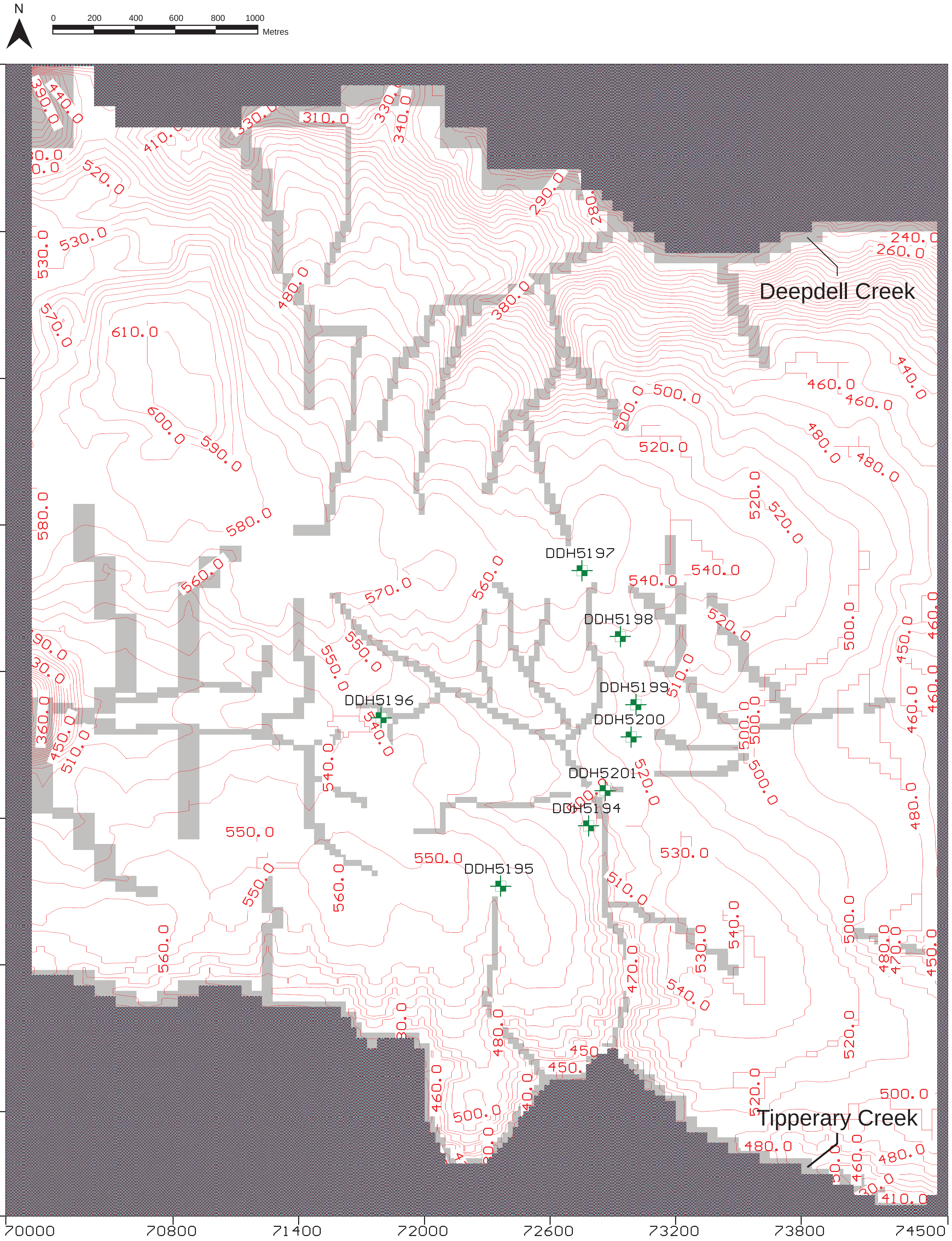
Geological feature	Specific storage	Specific yield	Effective porosity	Total porosity
	(m^{-1})	(m^{-1})	(m^3/m^3)	(m^3/m^3)
Heavily and moderately weathered schist	1.0×10^{-5}	0.02	0.02	0.03
Slightly weathered and unweathered schist	1.0×10^{-5}	0.005	0.005	0.006
Tailings	1.0×10^{-5}	0.38	0.38	0.4
Waste rock	1.0×10^{-5}	0.2	0.2	0.25

4.0 GROUNDWATER FLOW MODEL CALIBRATION

The hydraulic conductivity values applied to the groundwater models were the same as those applied to the calibrated Kingett Mitchell (2005a) model used to simulate the wider MGP area. Simulated groundwater levels from Model 1, representing the current conditions at the TTTSF, were then compared to groundwater level measurements from the TTTSF site. These measurements were obtained from eight monitoring wells installed during the geotechnical test work programme (Figure D3).

At the time of modelling only one complete set of water level measurements was available for calibration purposes. Ongoing monitoring of groundwater levels at the MGP site has shown that seasonal fluctuation in groundwater levels is not substantial (Kingett Mitchell 2005a). The model was simulated under steady state conditions and it has been assumed that measured piezometric levels also represent approximately steady state conditions under the current groundwater conditions.

Applying the hydraulic conductivity values from the existing calibrated model (Kingett Mitchell 2005) to the current model resulted in a good fit between the observed and the simulated groundwater levels in the monitoring wells (Figure D4). On the basis of this fit between simulated and observed groundwater levels the calibration results, the hydraulic conductivity values applied to past models (Kingett Mitchell, 2005a) were accepted as being appropriate for the simulation of groundwater conditions at the TTTSF site.



Legend

🟩 Groundwater monitoring well

Inactive cells

▨ Surface drains

Ground surface contours

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TITLE

LOCATION OF GROUNDWATER MONITORING WELLS

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PROJECT 0978110562

D3

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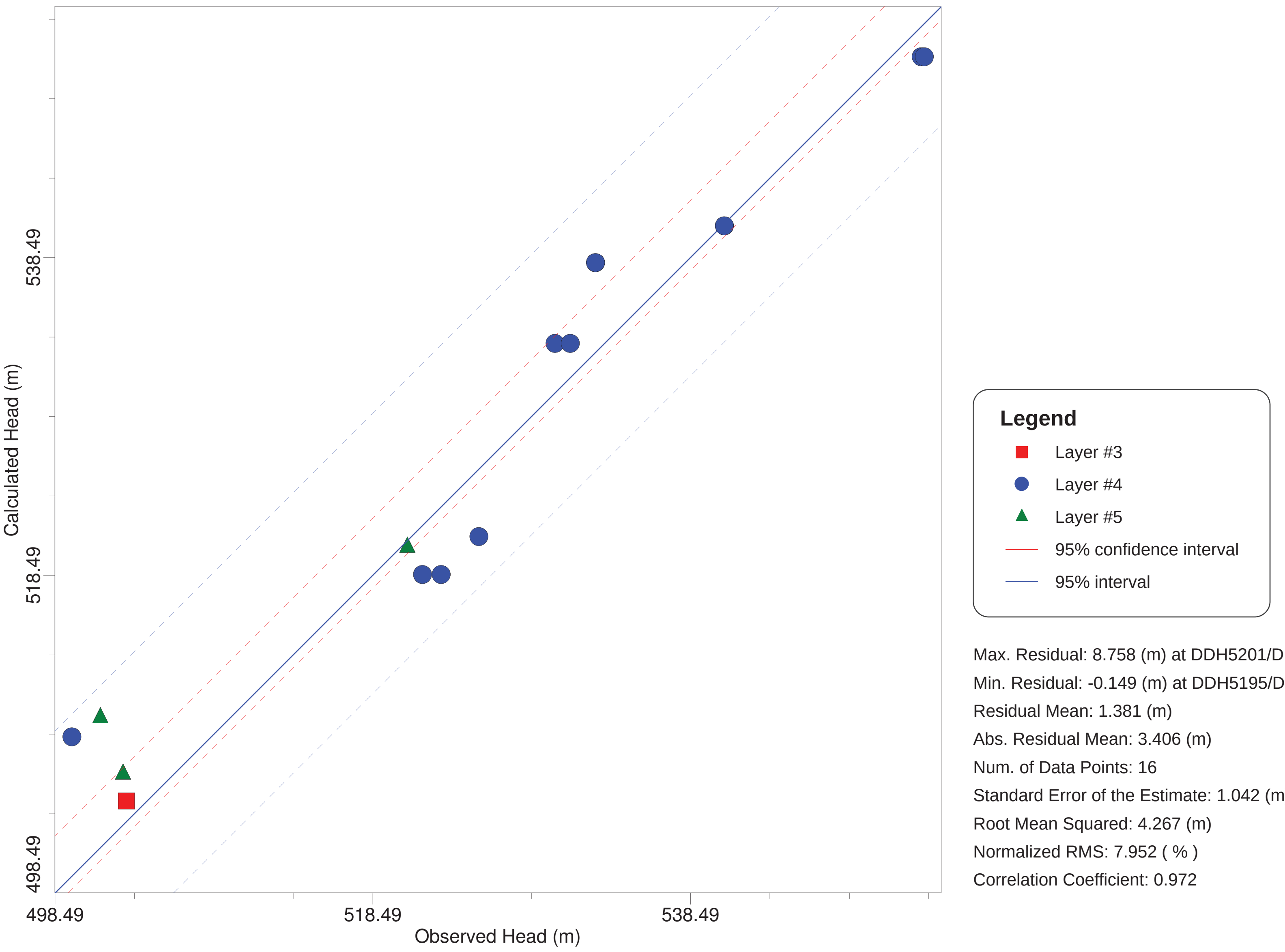
Calculated vs. Observed Head: Steady state



Max. Residual: 8.758 (m) at DDH5201/DDH5201B

Min. Residual: -0.149 (m) at DDH5195/DDH5195B

Residual Mean: 1.381 (m)

Abs. Residual Mean: 3.406 (m)

Num. of Data Points: 16

Standard Error of the Estimate: 1.042 (m)

Root Mean Squared: 4.267 (m)

Normalized RMS: 7.952 (%)

Correlation Coefficient: 0.972

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TITLE MODEL CALIBRATION RESULT

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D4

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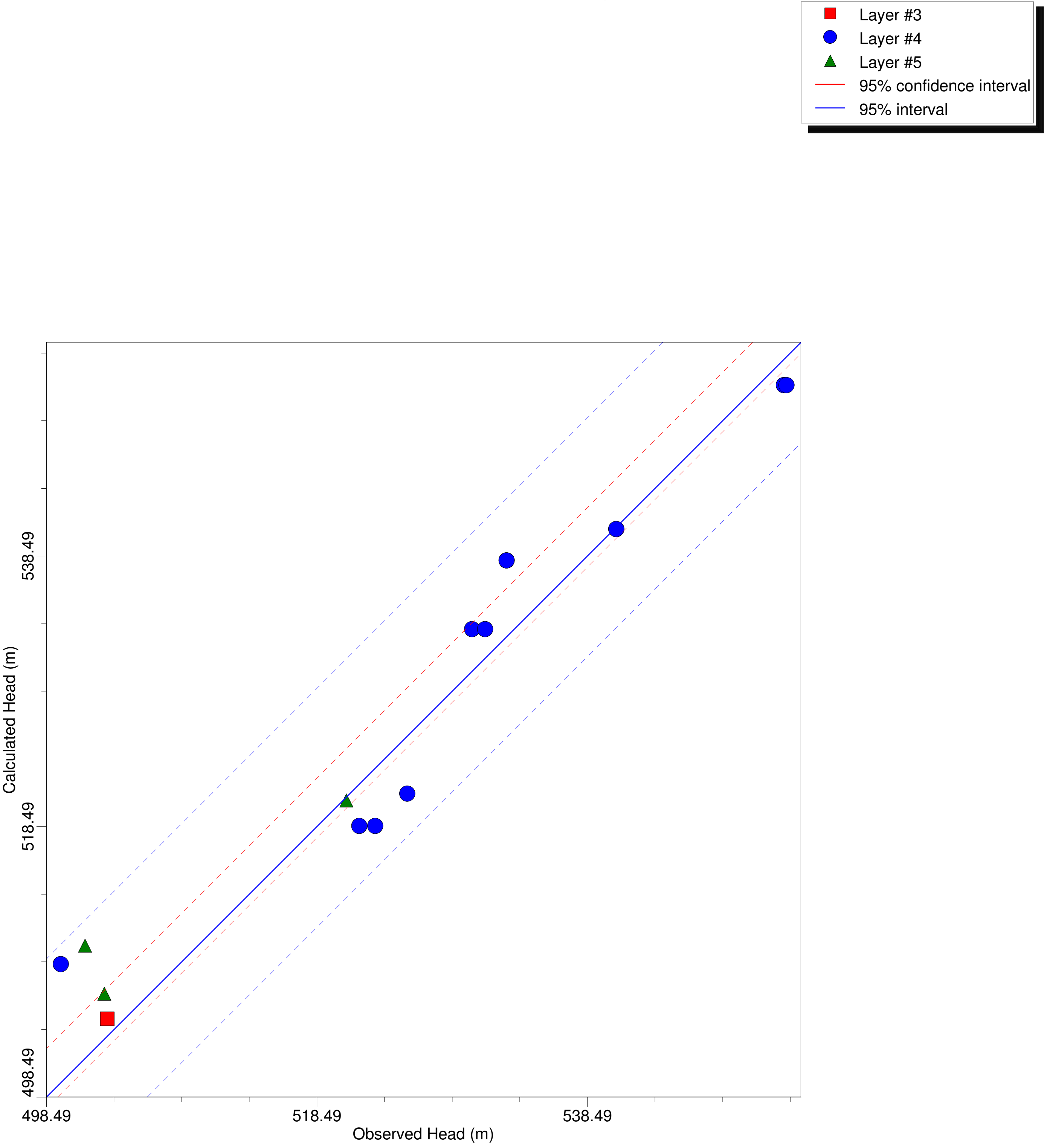
Calculated vs. Observed Head : Steady state



Max. Residual: 8.758 (m) at DDH5201/DDH5201B
Min. Residual: -0.149 (m) at DDH5195/DDH5195B
Residual Mean : 1.381 (m)
Abs. Residual Mean : 3.406 (m)

Num. of Data Points : 16
Standard Error of the Estimate : 1.042 (m)
Root Mean Squared : 4.267 (m)
Normalized RMS : 7.952 (%)
Correlation Coefficient : 0.972

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