Appendix D

Ian Grey Groundwater Consulting (2009),
Proposed Light Horse Landfill Site, Eastern Creek:
Detailed Hydrogeological Investigation and Assessment

Proposed Light Horse Landfill Site, Eastern Creek: Detailed Hydrogeological Investigation and Assessment

Dial A Dump Industries Pty Ltd

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Ian Grey Groundwater Consulting Pty Limited
ACN 109 204 434, ABN 28 109 720 434

PO Box 248
Newtown NSW 2042
Australia
Telephone +61 2 9029 2995
Facsimile +61 2 9519 0905
Email ian@iggc.com.au

Author: .... Ian Grey

Signed: ....

Date: ....15th September 2009

Distribution: .... Ian Malouf, Chris Biggs: DADI

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Background

Dial-A–Dump Industries Pty Ltd (DADI) proposes development of a non-putrescible general solid waste landfill site in a deep, hard-rock quarry site at Eastern Creek. Ian Grey Groundwater Consulting Pty Ltd (IGGC) has been engaged to undertake a detailed hydrogeological investigation and assessment of the quarry site, including suitability of the site, potential impacts from landfilling and mitigation measures by undertaking work including drilling of core hole, packer testing and numerical modelling. Previous investigation and assessment undertaken at the site includes desk-based study, drilling of boreholes and water level/inflow monitoring.

The existing quarry is a deep excavation with a maximum vertical depth of around 140 metres, and plan dimensions of around 600m (east-west) by 400m (north-south), with steep, stepped sides and a fairly flat base which drains to a sump from which groundwater ingress and rainwater run-off is pumped. The remainder of the site comprises an area of VENM spoil heaps and areas of cleared pasture.

The main local surface water feature in the area is Ropes Creek and a gully (minor tributary of Ropes Creek) runs east to west across the southern part of the site. Excess water pumped from the quarry has been discharged to this gully for around 40 years. The dominant regional drainage systems run from south to north and comprise South Creek (c.6km west of the quarry), Ropes Creek (c.1km west of the quarry) and Eastern Creek (c.3km east of the quarry). These creeks drain to the Hawkesbury River downstream of Windsor and originate from a topographical divide trending approximately east-west c.8km south of the quarry.

The area around the site is underlain by strata of the Wianamatta Group. The upper unit is the Bringelly Shale, a formation dominated by claystone and siltstone with thin laminite horizons and minor sandstone and with a thickness of at least 100m. This is underlain by the Minchinbury Sandstone and the Ashfield Shale followed by the Hawkesbury Sandstone, the top of which is expected to occur at below -80mAHD in the area of the site. The Minchinbury Diatreme occurs beneath the site and is exploited by the quarry. This is remnant of an explosive volcanic vent, and forms a steep-sided or vertical inverted conical structure of volcanic breccia with associated ring faulting.

The hydrogeology of the site and surrounding area is largely controlled by the geology. The strata of the Wianamatta Shale group are generally of low permeability, and the majority of groundwater flow occurs via fractures and bedding planes. The formation generally forms a layered aquifer system, with discrete aquifers occurring within horizontal fracture zones and with limited inter-connection between zones. The groundwater pressure surface generally follows topography; groundwater levels generally reflect the level of the nearest discharge zones and a slight downward hydraulic gradient typically exists between horizontal aquifer zones. Groundwater use in the area is limited and the low level of groundwater exploitation reflects the generally low yields and high salinity.

A weathered profile comprising mottled clays generally overlies the shale, and a perched shallow groundwater system can occur within this stratum.

The Minchinbury Diatreme would originally have formed a large, fractured rock mass within the Bringelly Shale.

Results of Field Investigation

Mapping of fractures within the quarry indicated that the majority of defects are orientated approximately parallel to the perimeter of the quarry with a small number orientated approximately perpendicular. Defects within the site orientated parallel to the diatreme margins would not be expected to extend outside of the site and those orientated perpendicular to the diatreme margins would be expected to terminate at the ring fault. Defects present within the country rock outside of the quarry prior to intrusion would be expected to terminate at the ring fault.

The observed seepages rates within the quarry were of low volume. The defect pattern described above would suggest that groundwater contained in the surrounding country rock would flow towards the site along defects and be intercepted by the ring fault, from where seepage into the quarry would only occur along defect planes connected to the ring fault. Any substantial connectivity would be expected to result in discrete areas of concentrated and high volume groundwater inflow, none of which were observed.

Five potential drill sites were selected based on all available information including the results of fracture mapping. These were locations where the greatest degree of fracturing and/or the greatest occurrence of groundwater might be expected to occur. The two preferred locations were selected from these five and agreed with DECCW. Drilling of boreholes was undertaken at these sites and comprised one deep cored hole (to c.150m) and one intermediate hole (to c.100m) drilled by air hammer at each site. The stratigraphy generally consisted of the upper weathered profile to c.30m, an upper fine sandstone interbedded with siltstone, a sequence of interbedded siltstone and sandstone and a gradational transition into more laminated shale. These units are interpreted as the Bringelly Shale and underlying Ashfield Shale units of the Wianamatta Formation. The lower 5 metres of BH10d intersected coarse sandstone which is interpreted as the top of the Hawkesbury Sandstone. All strata exhibited sparse fracturing. Packer tests were carried out every 10 m in the cored holes and the tested strata showed generally very low hydraulic conductivity values. All boreholes were completed as piezometers and developed and water level data collected during and after recovery.

Consideration of all available data was undertaken to confirm the accuracy of the understanding and conceptual model developed previously. This can be summarised as follows:

- *The hydrogeological setting comprises a layered aquifer system including a perched aquifer in the upper weathered profile and a series of aquifers in the underlying bedrock;*
- *The upper weathered profile shows low to moderate hydraulic conductivity. Groundwater levels are around 67 mAHD with limited hydraulic connection between the shallow aquifer and the quarry;*
- *The intermediate bedrock aquifer layers show very low hydraulic conductivities with occasional zones of higher values of up to 0.04 m/d. Stabilised groundwater levels are around 55 mAHD showing the effect of depressurisation caused by pumping;*
- *The deep bedrock aquifer layers show very low hydraulic conductivity values with occasional zones of higher values of up to 0.01 m/d. Stabilised groundwater levels are around 31 mAHD showing the effect of depressurisation;*
- *The Hawkesbury Sandstone occurs beneath the Wianamatta Shale Group strata around six metres below the deepest parts of the quarry. Hydraulic conductivity is low and groundwater levels are similar to those is the overlying deep Wianamatta Group strata;*

- *The quarry exploits volcanic breccia of the Minchinbury Diatreme and these strata form the walls of the quarry beneath the first one or two benches. Observational data of the extent of fracturing and seepage within the quarry indicate that these strata are of very low hydraulic conductivity.*
- *Pumping of groundwater from the quarry has results in a steep inward hydraulic gradient in the bedrock strata. Effects appear limited in the shallow weathered profile indicating limited hydraulic connection between these strata and the quarry. Despite the steep gradients seepage rates into the quarry are low (c.30 m^3/day) reflecting the very low hydraulic conductivity values of the strata;*
- *Under natural conditions a low, downward hydraulic gradient would be expected to occur. This has been increased as a result of depressurisation resulting in relatively high downward gradients;*
- *The regional groundwater system is fed by low levels of rainfall recharge with groundwater flow controlled by discharge to creeks to the east and west of the site and to the Hawkesbury-Nepean system to the north.*

Numerical Modelling

A numerical groundwater model has been developed as a three-dimensional representation of the area around the former quarry. The conceptual model consists of a layered aquifer system to represent the upper residual soil, the weathered shale, the fresh shale (including more transmissive horizons) and the underlying sandstone. The model represents an area of 120 square kilometres and is bounded by distant constant head boundaries to the south and north, up and down the dominant groundwater flow direction, and distant no-flow boundaries to the east and west, across the dominant flow direction. The base of the model is a no-flow boundary set at a depth of minus 150 metres. The basic hydraulic model was developed using the best available estimates for the various parameters using site-specific data where possible. Model calibration results were excellent for intermediate groundwater but variable for deep groundwater probably due to incomplete recovery of measured water levels. Quarry inflow is over-estimated by the model by around 100%. This will provide a conservative assessment as the predicted rate of groundwater recovery will be faster than that likely to occur in reality and may be due to over-estimation of hydraulic conductivity of some strata in the model. Overall, results of calibration are considered to be acceptable particularly for a complex hydrogeological setting such as this. Model predictions are expected to be conservative

The calibrated model was used as the basis for a transient-state model to simulate the effects of cessation of groundwater pumping from the quarry. Results predict that the quarry groundwater/leachate level shows predicted rates of rise of around 5 metres per year in the first two years but up to 23.5 m/yr in years 3 and 4 before declining to less than 5 m/yr by year 9. This indicates that leachate level management will be required during the operational phase, including installation of a leachate collection system and pumping of leachate for appropriate disposal. Groundwater level recovery in the deep wells is predicted to be c.15 m over the first ten years with recovery slower in the intermediate and deep wells. Full recovery is not predicted to occur within the simulation period.

Simulation of groundwater conditions after completion of landfilling and with no leachate/groundwater pumping was undertaken to provide an assessment of the potential for migration of leachate from the site under such conditions. Final leachate levels are predicted to be c.77 mAHD, i.e. with a recharge mound predicted to form with levels above the surrounding groundwater level in all strata and above the local ground surface level in some areas. This results in a potential for migration of leachate contamination from the site into the surrounding groundwater system. Migration of groundwater away from the site is predicted to be very slow, reflecting the low hydraulic conductivity of the surrounding *strata and the relatively low outward hydraulic gradient. The fastest migration rates are predicted to be around 100 years for a conservative solute to travel 400m (i.e. around 4 m/year) and occur in areas of the highest hydraulic gradients (generally to the north and west). Such slow migration is expected to be sufficient to allow attenuation of pollutants and no detectable impact on groundwater quality would be expected. This assessment is based on highly conservative assumptions and is based on migration from strata around the site; it therefore does not take account of the time required for leachate to migrate from within the quarry through the volcanic breccia and into the surrounding strata.*

Conclusions and Recommendations

The quarry represents a very low risk site for landfill site development in terms of potential groundwater impacts because of the very low permeability of the surround strata and limited degree of hydraulic connection; the strong inward hydraulic gradient; and the low groundwater inflow rate. Results of numerical modelling indicate that the potential for impacts on groundwater due to leachate migration from the site is very low, with migration rates predicted to be very slow even for worst-case condition in which no pumping takes place for an extended period.

The site is therefore considered highly suited for landfill development providing that appropriate management and control measures are implemented. Provision of a low permeability barrier or landfill liner is not considered necessary and would offer no environmental or management benefits because of the above factors and because of the nature of the proposed fill material. This includes the upper parts of the quarry where the shallow weathered strata occur. Control of leachate levels using a carefully designed leachate management system in conjunction with monitoring of groundwater levels is the surrounding strata is considered to be a more effective and practicable means of ensuring environmental protection.

A leachate management system to allow interception, collection and removal of water accumulating in the landfill site is required and should involve construction of a series of drainage systems progressively during filling at various levels through the fill profile with only the upper drainage system in use at any time. Leachate levels should be maintained as required operationally, either a few metres below the fill surface, or at a lower level to provide buffering storage. Leachate levels should also be kept below the groundwater levels in the surrounding strata. Ongoing monitoring of groundwater and leachate levels and water quality will be required during the active landfilling period and post-closure. The existing groundwater monitoring network is considered to be sufficient to ensure protection of the local groundwater systems.

No further mitigation measures are considered necessary to protect groundwater.

1. Introduction

Dial-A–Dump Industries Pty Ltd (DADI) proposes development of a non-putrescible solid waste landfill site at Eastern Creek. Current land use comprises a deep hard-rock quarry with other areas of the site comprising an area used for spoil storage/disposal in the form of large heaps of virgin excavated natural material (VENM), and an area of cleared pasture.

The proposed redevelopment involves rehabilitation of the quarry by controlled filling to allow subsequent development.

Ian Grey Groundwater Consulting Pty Ltd (IGGC) has been engaged to undertake a detailed hydrogeological investigation and assessment of the quarry site, including suitability of the site, potential impacts from landfilling and mitigation measures required.

This report presents the results of this investigation and assessment.

2. Scope of Work

The scope of work covered in this report is based on IGGC's proposal of 17th March 2009 (LT_188 RevB). This includes requirements from the NSW Department of Environment, Climate Change and Water (DECCW) as outlined in Attachment A of the Development Approval for the site and is summarised below.

Inception: confirmation of contractual and access arrangements and of timing of site works.

Fracture Mapping: mapping of fracture occurrence and orientation within the quarry to assist in identification of areas of potentially increased fracturing around the quarry. Fracture mapping to be carried out primarily by J&K with input from IGGC with results transferred onto a detailed site plan;

Drill Site Selection: selection of potential drill sites based on existing data, results of fracture mapping, access availability etc. targeting areas where the greatest degree of fracturing is expected to occur. Liaison with DECC to confirm acceptability of the proposed scope of work and bore locations;

Field Program: the field program includes the following:

- Drilling of two core holes to c.150m depth including logging of core and packer testing (c. 1 test every 10m during coring) followed by reaming out to c.150mm diameter;

- Drilling of two air hammer boreholes to c.100m depth;

- Installation of monitoring wells with designs based on the detailed geological profile and results of packer testing. Well will be installed using Class 18 screen and casing with sand packs and bentonite seals carefully placed and bore annuli grouted to surface. Surface completions will comprise lockable monuments cemented into place. Wells will be flushed with water as need to remove cuttings then air lifted to remove introduced water and develop the wells. Final well installations will be surveyed to provide accurate locations and elevations (survey to be provided by LHBC).

Numerical Modelling: a numerical model will be constructed and calibrated to steady state conditions using the results of previous and proposed investigations and representing the existing pit and the surrounding strata to the expected limits of the pit's influence. This model will then be used to predict groundwater behaviour under the following transient conditions:

- Cessation of pumping and groundwater rebound, including the rate of rebound and time required for complete recovery;

- Inflow rates during landfill operation;

- Migration rates of leachate contamination from the site assuming a positive hydraulic gradient (i.e. complete groundwater recovery and high recharge across the landfill site compared to the surrounding area).

Assessment and Reporting: IGGC will provide a detailed draft report for review, containing full details of the above investigation and assessment and suitable for submission in support of the EPL application. Comments will be incorporated prior to the final report being issued. The report will include bore logs and core photographs, results of packer testing and full detailed of the numerical modelling process and outcomes. It will also include a thorough hydrogeological assessment covering all of the above.

The report will not address detailed design, geotechnical issues, contamination or surface water drainage.

3. Summary of Previous Work

The following summarises relevant investigations and assessments carried out at the site prior to the current investigation.

Archbold Road, Eastern Creek: Groundwater and Salinity Assessment for Proposed Quarry Rehabilitation Project and Developable Land. IGGC, March 2006.

This desk-based study included collation of existing data and assessment of the following:

- Geological and groundwater conditions beneath the site including likely effects of long-term pumping from the quarry;
- Viability and potential impacts of rehabilitation of the quarry by controlled filling including requirements for provision of a low-permeability liner and a leachate management system;
- Potential impacts associated with groundwater and salinity due to development of the developable land (i.e. the site excluding the quarry area).

The findings of relevance to the current study were that the quarry represents a very low risk site for rehabilitation in terms of potential environmental impacts, because of the low permeability of the strata; the strong inward hydraulic gradient; and the low groundwater inflow rate. It was therefore considered highly suited to rehabilitation by controlled filling, providing that appropriate management and control measures are implemented, including collection and pumping of groundwater seepage and rainfall infiltration. Provision of a low permeability barrier or landfill liner was not considered necessary. Recommendations included additional investigation groundwater conditions to determine baseline conditions, and ongoing monitoring during rehabilitation. Drilling of at least three multi-level piezometers was recommended around the quarry, followed by monitoring of these and pumped volumes. Numerical modelling of the local groundwater system and repressurisation is also recommended, to allow prediction of final groundwater levels and flow regime.

Light Horse Business Centre, Eastern Creek, Australia. Groundwater Assessment. ERM, August 2008.

ERM carried out field investigation and assessment of the site including suitability for development as a solid waste landfill site, potential risks and mitigation requirements, including the following:

- Drilling of three sets bores around the quarry site and completion as piezometers. Each set comprised one shallow (to c.20 m depth), one intermediate (to c.50m depth) and one deep (to c.130m depth) piezometer;
- Monitoring of groundwater levels, hydraulic testing and sampling from all nine piezometers;

- Assessment of groundwater conditions around the quarry including expected groundwater inflow rates;

- Development of a spreadsheet-based water balance model to allow estimation of potential groundwater and surface water inflow rates into the quarry void and to determine requirements for leachate storage and disposal.

Groundwater Inflow Assessment, Former Hanson Quarry, Eastern Creek. IGGC, February 2009.

IGGC carried out monitoring of the rate of water level rise during a period of cessation of groundwater pumping to allow estimation of actual groundwater inflow rates to the quarry. Full details of this work are provided in *Section 4.5.2*.

Review of Hydrogeological Investigations and Considerations for Development of Disused Quarry and Eastern Creek, NSW. Red Earth Geosciences, March 2009.

This comprised a review of previous hydrogeological investigations, namely IGGC, 2006 and ERM, 2008 and recent inflow assessment (IGGC, 2009). Key findings were as follows:

- Development of a comprehensive surface drainage map is recommended to allow identification of surface water features and groundwater/surface water connectivity;

- Re-examination of potential quarry inflows is recommended included detailed topographic cross-sections showing the relationship between piezometers and the quarry;

- The hydraulic testing undertaken by ERM is less than optimal and should be repeated using more appropriate techniques and analysis. Piezometers should be rehabilitated where blocked (BH1d) or subject to surface water ingress;

- Re-evaluation of hydrochemistry.

Despite the shortfalls identified above the hydrogeological setting is considered to be very constrained and groundwater inflows are likely to make up a small fraction of leachate generation; provision of a low-permeability barrier is therefore not considered necessary except perhaps where leachate may contact the host sedimentary strata at levels above the regional groundwater level. Control and management of leachate within the pit void is considered practicable subject to provision of appropriate management systems.

4. Background and Site Setting

4.1 Proposed Development

LHBC propose to develop the former quarry site and surrounds as a landfill site for the disposal of non-putrescible general solid waste which will comprise VENM, construction and excavation waste, paper and cardboard and non-putrescible household and commercial waste. Excavation from the quarry has ceased, and controlled filling with suitable waste materials will take place to allow rehabilitation of the quarry area, and to allow subsequent redevelopment. This will be preceded by preparation of the quarry site as required, including installation of a conveyor for transfer of waste to the tipping face and installation of a leachate management system. It is anticipated that the proposed landfill site will be operation for a period of up to 20 years.

The location of the site is shown on *Figure 4.1*.

4.2 Site Features and Topography

The site as a whole can be divided into three main areas: the existing quarry; the spoil heap area to the west and north-west; and the cleared farmland to the south-west. The main features are summarised as follows:

Quarry: the quarry is a deep excavation with a maximum vertical depth of around 140 metres, and plan dimensions of around 600m (east-west) by 400m (north-south). The quarry sides are stepped, comprising steep slopes (70 to 80º) 10 to 15m high separated by flat benches around 7m wide (J&K, 2004). The upper part of the quarry is excavated through shale and sandstone, and has variable but generally lower-angled slopes (30º to sub-vertical). The base of the quarry is fairly flat, and drains to a sump from which groundwater ingress and rainwater run-off is pumped. The quarry was previously operated by Hanson (formerly Pioneer) but extraction has now ceased and current activity is limited to pumping of collected water from the quarry sump.

VENM spoil heap area: the area to the west and north-west of the quarry has been used for storage/disposal of quarry overburden and spoil (VENM), and contains large, fairly flat-topped spoil heaps up to 30m high with side slope angles typically around 45º. The spoil heaps occupy the majority of this area of the site, although the northern area (up to 250m from the northern boundary) and a narrow strip along the western boundary appear relatively undisturbed.

Cleared Pasture: the area to the south-west of the quarry comprises undulating, cleared pasture which generally slopes to the south and west at around 5º. A minor drainage line runs through the southern part of this area and joins Ropes Creek west of the site. Vegetation comprises grasses, with a few trees in the south-eastern part of the area.

The triangular area west of Archbold Road comprises a generally flat and low-lying area of cleared pasture, with few trees.

Site features are shown on *Figure 4.2.*

4.3 Surface Water Features

Local Surface Water Features

The main surface water feature in the area is Ropes Creek, located approximately 400 metres west of the site boundary. A gully (minor tributary of Ropes Creek) runs east to west across the cleared farmland that forms the southern part of the site. Excess water pumped from the quarry has been discharged to this gully for around 40 years, and this will probably have changed the character of the gully considerably. Two or more other minor drainage lines cross the cleared land west of Archbold Road (both within and outside of the proposed development site). These would originally have had some expression on the site but are assumed to have been obscured by spoil heaps.

A dam is present in the north-western corner of the site, and would be retained as a conservation feature. A small dam is also present on the minor drainage line crossing the triangular area west of Archbold Road.

The majority of the surface drainage from the site is to Ropes Creek either via tributaries or directly via overland flow. A small area of the site immediately east of the quarry drains to Angus Creek, a tributary of Eastern Creek. Runoff and groundwater seepage from the quarry sub-catchment drains to the basal quarry pond, from where it is pumped to an intermediate pond, and from there to surface dams for re-use on site or for discharge to the tributary of Ropes Creek.

A review of historical aerial photography taken in 1947 prior to site development (CH2M Hill, 2004) indicates two drainage lines, one running east to west across the southern part of the site (existing) and a smaller one running south-east to north-west across the northern part of the site. The latter drainage line has been completely disrupted by placement of spoil, but is still present to the west of the site boundary and Archbold Road.

Filed water quality measurements taken during discharge of pumped water from the quarry to the southern gully showed water quality similar to that measured in the quarry pond, with highly alkaline (pH 9.85) and fresh to brackish (EC 1,241 µS/cm) conditions. Discharge of pumped water over many years is likely to have altered the nature of the gully substantially, both in terms of the flow regime and water quality.

Surface water and drainage features are shown on *Figure 4.2*.

Regional Surface Water Features

The dominant drainage systems for the area of the site run from south to north and comprise South Creek (c.6km west of the quarry), Ropes Creek (c.1km west of the quarry) and Eastern Creek (c.3km east of the quarry). These creeks drain to the

Hawkesbury River downstream of Windsor and originate from a topographical divide trending approximately east-west c.8km south of the quarry.

4.4 Geology and Soil

Reference to the published 1:100,000 Penrith area geology map (Clarke & Jones, 1991a) indicates that the area around the site is underlain by strata of the Wianamatta Group. The upper unit is the Bringelly Shale, a formation dominated by claystone and siltstone with thin laminite horizons and minor sandstone and with a thickness of at least 100m. This is underlain by the Minchinbury Sandstone, a 3m to 6m thick quartz-lithic sandstone; followed by the Ashfield Shale which comprises sandstone-siltstone laminite and sideritic claystone.

The Wianamatta Group is underlain by the Hawkesbury Sandstone, the top of which is expected to occur at below -80mAHD in the area of the site due to the presence of a palaeochannel (Jones and Clarke, 1991b), and is therefore likely to occur below the base level of the quarry.

The Minchinbury Diatreme occurs beneath the site and is exploited by the Hanson quarry. This is considered to be the remnant of an explosive volcanic vent, and forms a steep-sided or vertical inverted conical structure approximately 850m by 300m and pear-shaped in plan. The diatreme comprises volcanic breccia made up of basaltic lapilli (4 to 32mm fragments) and blocks in a fine-grained matrix of tuff and siltstone. Vertically bedded sandstone/siltstone (Bringelly Shale) has been dragged down a ring fault surrounding the diatreme (Jones and Clarke, 1991b).

The edge of the diatreme is generally within the quarry, with the upper benches excavated through weathered or unweathered shale country rock. However, the diatreme appears to extend beyond the south-western limit of the quarry, forming a low hill in the northern part of the cleared farmland. Volcanic strata are exposed in the road cuttings in this area.

Alluvial deposits of Quaternary age occur along Ropes Creek, located to the west of the site. Minor alluvium may occur along the course of a tributary creek which crosses the southern part of the site.

Reference to the 1:100,000 scale soil landscape map of the Penrith area (Bannerman & Hazleton, 1990) indicates the following soil types:

- Moderately reactive highly plastic clay soils up to 1m deep over the outcrop of the Bringelly Shale;
- Moderately reactive deep layered fluvial soils around Ropes Creek;
- Disturbed ground over the site of the quarry.

4.5 Hydrogeology

4.5.1 Hydrogeological Setting

The hydrogeology of the site and surrounding area is largely controlled by the geology. The strata of the Wianamatta Shale group are generally of low permeability, and have a limited potential to transmit groundwater flow. The majority of groundwater flow occurs via fractures and bedding planes, with negligible flow through the rock mass.

The formation generally forms a layered aquifer system, with discrete aquifers occurring within horizontal fracture zones and with limited inter-connection between zones. The groundwater pressure surface generally follows topography, with groundwater flowing from recharge areas on high ground to discharge areas (generally creeks, rivers and wetland areas). Groundwater levels generally reflect the level of the nearest discharge zones and in the area of the site would be expected to be around 50mAHD. A slight downward hydraulic gradient typically exists between horizontal aquifer zones.

Prior to development of the quarry the diatreme formed a low hill and groundwater flow may have radiated from this area towards the surrounding low ground and creeks.

Groundwater quality is generally poor, with high salinity levels from connate salts within the formation and the limited flushing due to low groundwater flow rates.

A weathered profile comprising mottled clays generally overlies the shale, and a perched shallow groundwater system can occur within this stratum.

The Minchinbury Diatreme would originally have formed a large, fractured rock mass within the Bringelly Shale. The permeability of the volcanic breccia relative to the surrounding shales and sandstone is not known, however the intrusion originally formed a low hill and the local high point, and would be expected to represent a groundwater recharge area, with groundwater flowing from high levels around the intrusion towards likely discharge areas associated with Ropes Creek to the west and Eastern Creek to the east. Groundwater quality associated with igneous bodies such as the diatreme can show highly alkaline water, and high levels of inorganic nitrogen can also be present.

Intrusion of the diatreme will have resulted in faulting and increased fracturing of the surrounding strata, and subsequent quarrying activities will have also increased local fracturing as a result of blasting and pressure relief. This is likely to have increased the permeability of the strata immediately surrounding the quarry.

Alluvial deposits occur around Ropes Creek, and limited alluvial material may occur immediately around the tributary. Such strata are highly variable, but are likely to comprise sands, silts and clays. Groundwater is likely to be hydraulically connected to the creek. Localised recharge from creek water is likely to result in relatively fresh groundwater, although discharge of more saline groundwater from the shale can occur through the alluvial material.

A search of the DECCW database provided details of 18 registered bores located within 5 km of the site. The majority of these are test/monitoring bores, although there are also two shallow irrigation wells, an aquaculture waste disposal bore and a shallow domestic bore.

Bore details are summarised in *Table 4.1* and locations are shown on *Figure 4.3*.

Information from the DWE records confirms the hydrogeological setting, with groundwater levels typically 10 to 25 metres below surface. Water quality data are limited, but the reported salinity levels are relatively low for Bringelly Shale.

Groundwater use in the area is limited, with only three registered bores licensed for abstraction of groundwater, all three of which are shallow and exploit perched groundwater in residual clays or minor alluvium. There is also an aquaculture waste disposal bore. All other recorded bores in the area are monitoring or test bores. This low level of groundwater exploitation reflects the generally low yields and high salinity obtained from bores drilled into the shale.

Table 4.1 Summary Details of Registered Bores

Ref	Bore No	Easting (mMGA)	Northing (mMGA)	Depth (m)	Purpose	Standing Water Level (m)	Salinity (mg/L TDS)	Date Drilled	Screen (m)	Geology
1	GW101087	294624	6255732	90.3	Monitoring			1996	70.5 to 88.3	
2	GW101083	294912	6255522	78	Monitoring			1996	58.2 to76	
3	GW102673	295163	6255774	78	Monitoring	9.68	4750	1993	Multiple	Siltstone/sandstone/shale
4	GW102674	295369	6255779	71.9	Monitoring		4400	1993	Multiple	Shale/siltstone/sandstone
5	GW101085	295857	6255789	99.3	Monitoring/test			1996	79.5 to 97.3	
6	GW101082	296112	6255918	40.3	Monitoring/test	12.43		1996	30.4 to 39.3	
7	GW104060	301538	6255572	24.6	Monitoring			2001	8.6 to 23.6	5m clay over shale
8	GW104061	301820	6255566	24.5	Monitoring			2001	8.5 to 23.5	siltstone/shale
9	GW104062	302387	6255420	24.4	Monitoring	17	2800	2001	5.4 to 23.4	4m clay over shale
10	GW104063	302689	6255343	27.4	Monitoring			2001	8.4 to 26.4	5m clay over shale
11	GW075076	294522	6261087	13.5	Monitoring (DWR)	7		1999	10.5 to 13.5	clay
12	GW075077	295109	6260936	12.5	Monitoring (DWR)	12.5		1999	9.5 to 12.5	12.5m clay over shale
13	GW075078	295501	6260807	8	Monitoring (DWR)			1999	1 to 8	7.8m clay over shale
14	GW028415	297090	6260390	7.6	Irrigation			1966		3m clay, 1.8m gravel over shale
15	GW028414	298655	6259660	6.1	Irrigation	3.9		1966		clay over shale
16	GW018361	300615	6259765	217.9	Aquaculture Waste Disposal			1961	OH from 12.1	14m clay over basalt/shale/sandstone
17	GW105479	296998	6262176	14	Monitoring (mobil)	12.9		2003		
18	GW026226	300760	6263530	8.5	Domestic	1		1966		7.9m clay over shale

Notes: MGA is Map Grid of Australia; mg/L is milligrams per litre; TDS is total dissolved solids; OH is open hole.

Some investigation of hydrogeological conditions around the quarry has been undertaken (ERM, 2008) and including the drilling and installation of shallow, intermediate and deep piezometers at three locations. This investigation indicated the following hydrogeological conditions:

- A shallow, weathered profile comprising clay and weathered shale extends to depths of around 32m. This is host to an intermittent shallow, perched groundwater system with hydraulic conductivity values of 0.0015 m/d to 0.25 m/d and limited hydraulic connection to the quarry (ERM, 2008);

- A deeper, regional groundwater system occurs within the Bringelly Shale strata with very low calculated hydraulic conductivity values of 1.75×10^{-6} m/d to 8.7×10^{-6} m/d. Groundwater elevations were above 24 mAHD, i.e. c.82 m above the quarry base. Groundwater levels are generally lower in the deeper water bearing zones within the shale indicating a downward hydraulic gradient and limited inter-connectivity between these zones.

4.5.2 Quarry Hydrogeology

The presence of a deep quarry for over 40 years has resulted in substantial depressurisation of the local groundwater systems. The base of the quarry is presently at an elevation of around -66 mAHD, i.e. around 116m below the estimated natural groundwater level. This head difference represents a very high hydraulic gradient into the quarry from the surrounding aquifers.

Rainfall runoff from the quarry catchment and groundwater seepage from the sides and base of the quarry are currently collected in a sump at the base of the quarry and pumped to Ropes Creek. No formal measurement of pumped volumes was made by the former quarry operator. Anecdotal information indicates that water is pumped from the basal pond at a rate of around 40 L/s, with pumping typically taking place for 2 hours every 2 to 3 days, with pumping occurring more frequently during wet weather and less so during dry periods. Some recirculation of pumped water probably occurs due to leakage from the intermediate and surface level pond. This suggests a typical inflow rate of around 125 kL/day, although this figure is likely to include a large component of rainfall runoff. This is a very low rate of inflow for a quarry of this size and depth, and indicates that the surrounding strata are of low permeability.

Additional investigation and assessment was undertaken by IGGC in early 2009 (IGGC, 2009). This comprised monitoring of the rate of water level rise in the base of the quarry. Dewatering pumping was suspended between the 5th February and the 11th February 2009 to allow monitoring of the rate of water level rise. Two pressure transducers with data loggers (referred to hereafter as "loggers") were placed in a length of well screen for protection and lowered into the sump hole in the quarry floor prior to suspension of pumping. A barometric pressure logger was left in the site office to allow correction of data for barometric variations. The loggers were retrieved and downloaded on the 11th February 2009.

Data collected by the loggers were corrected for barometric variations and graphed to allow analysis. A graph showing the full record from both loggers is attached as *Figure 4.4*.

Inspection of *Figure 4.4* indicates several features as follows:

- Consistent water levels between the two loggers with a small difference of around 0.07m due to their relative positions;
- Declining water levels due to pumping in the early part of the graph;
- Steady or slightly rising water levels after initial pump switch off followed by a further decline when the pump was switched on again for an additional 1 hour and 20 minutes;
- Steady or slowing rising water levels for the last six days of the recording period with evidence of tidal variation of up to 0.012m;
- An apparent sharp water level rise of 0.2m near the end of the record due to disturbance of the loggers during relocation of the pump.

The data from Logger 1 were then used for further analysis. The rise at the end of the record was removed by correcting the subsequent data to provide a consistent record. The rate of groundwater inflow to the quarry pond was then estimated by comparing the observed water level change with that expected based on rainfall and evaporation alone. Rainfall and evaporation data were obtained for Bureau of Meteorology station 067019 located at Prospect Reservoir, approximately 7km east of the quarry. These data are summarised in **Table 4.2**.

Table 4.2: Summary of Climate Data (to 9am on date given)

Date	Day	Rain to 9am	Evaporation to 9am	Net Gain
5/02/2009	Thurs	0	5	-5
6/02/2009	Fri	0	7.6	-7.6
7/02/2009	Sat	0	8.8	-8.8
8/02/2009	Sun	0	9.4	-9.4
9/02/2009	Mon	0	9.4	-9.4
10/02/2009	Tues	3.2	1.6	-.6
11/02/2009	Weds	5.6	1.1	4.5
	TOTAL	8.8	42.9	42.9

Starting with the water level on 5th February 2009, the predicted water level based on rainfall and evaporation alone has been projected. This assumes that both rainfall and evaporation are only applied to the pond surface area: this is realistic for evaporation but will underestimate the effect of rainfall as some runoff from higher levels of the quarry will have occurred. Insufficient information is available to estimate the effective catchment area which in any case will vary depending on the size and duration of rainfall events. This approach will under-estimate the rainfall contribution and lead to some over-estimation of the groundwater inflow rate and will therefore be conservative for the purposes of this assessment. There is some potential for under-estimation of groundwater inflows where minor seepages from the higher levels of the quarry are sufficiently small so as to be lost by evaporation prior to reaching the pond; however by definition these will be small.

The quarry pond was estimated to have a surface area of around 3,600 m^2 during the monitoring period (pers. comm., LHBC). A check calculation was performed using the estimated pump rate (30 L/s) and the observed rate of decline during pumping (0.8m/day). This indicates an effective pond area of 3,240 m^2, and the estimate of 3,600m^2 will therefore give a slightly conservative results. The calculations presented herein assume that the surface area remains constant during the monitoring period, i.e. the pond has vertical sides. Some change in surface area will result from the observed water level rise but this only occurs on one side of the pond (the others having near-vertical faces) and is considered to be negligible compared to the overall area.

Comparison of the projected water level changed based on rainfall and evaporation only with that observed shows an effective rise of 0.049 m over 6 days, equivalent to 0.008 m/d. Based on the estimated pond area of 3,600m^2 this indicates a net volume gain of 29.4 m/d. This is likely to represent an over-estimate of groundwater inflow due to the factors described previously but is consistent with the anecdotal average inflow rate of 125 m^3/d comprising both groundwater inflow and rainfall contributions; and with anecdotal information that water level rises are very small except during rainfall.

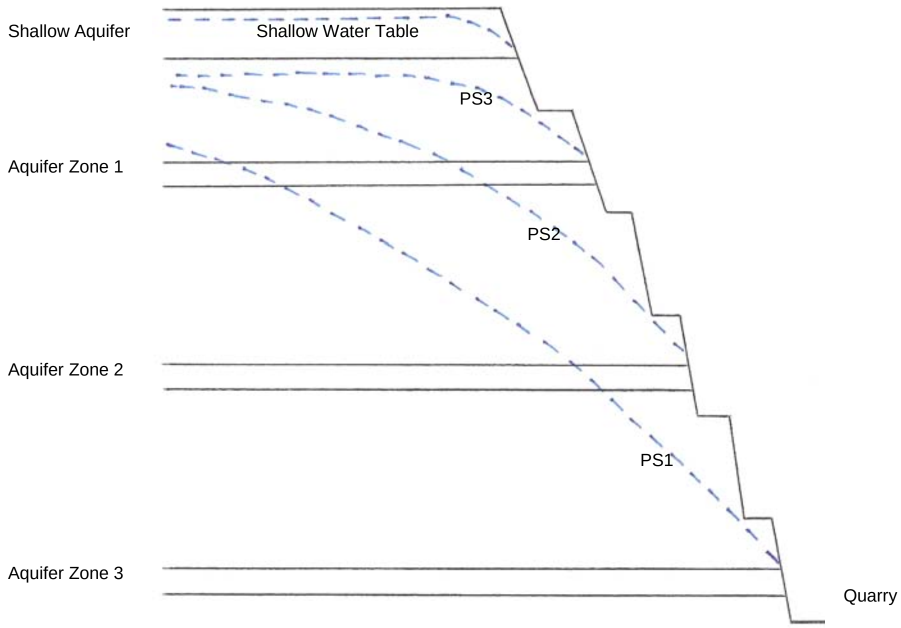
Previous assessment of the hydraulic conductivity of the deep shale strata surrounding the quarry derived from slug tests indicated values of $1.75x10^{-6}$ m/d to $8.7x10^{-6}$ m/d with a calculated inflow of around 2 m^3/day (ERM, 2008). This is around an order of magnitude below the observed inflow probably due to a combination of the conservatism noted previously, potential flaws in these slug test results and localised higher hydraulic conductivity zones associated with fracturing.

In the long term operation of the proposed landfill the leachate level should be allowed to rise as waste is placed, with a final level maintained at an appropriate margin below the regional groundwater level (c.50 mAHD) to ensure an inward hydraulic gradient. This will reduce the hydraulic gradient by at least an order of magnitude and will therefore result in an equivalent reduction in groundwater inflow. The long-term groundwater inflow rate is therefore estimated to be below 3 m^3/day.

4.5.3 Conceptual Groundwater Regime

The low permeability of the strata in and around the quarry means that depressurisation is likely to have resulted in a steep drawdown cone. The extent of depressurisation is likely to be fairly limited in the shallow aquifers within the soils/weathered profile and upper shale, but may extend to a kilometre or more from the quarry in the deep aquifers. The conceptual groundwater regime around the quarry is illustrated in *Figure 4.5*.

Figure 4.5: Conceptual Groundwater Regime (Simplified, not to scale)



Notes: PS is piezometric or pressure surface

Observations made by quarry staff are that seepages generally occur immediately after rain and persist for a few days to a few weeks. There are some areas of permanent seepage, although the inflow rates from these are reportedly low. In general seepage is greatest from the north-eastern quarry face and lowest in the western area. This suggests that the permeability of the remaining igneous body is relatively low.

4.6 Rainfall and Climate

4.6.1 Average Rainfall and Evaporation

Rainfall and evaporation data have been obtained for Bureau of Meteorology (BoM) Station 067019 located at Prospect Reservoir, approximately 7 km east of the quarry. Average monthly rainfall and evaporation data are summarised in *Table 4.3*.

Table 4.3: Summary of Monthly Rainfall and Evaporation (millimetres)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Rainfall	94.4	95.8	95.8	75.4	72.1	75.3	57.4	50.8	47.8	59.4	72.6	75.1	871.6
Evaporation	170	136.5	122.2	89.8	61.7	49.2	55.6	80.8	108.9	140.1	149.7	180.8	1346.6

Rainfall is highest during the summer months peaking in January/February, and lowest in winter and early spring. Evaporation is highest in December and lowest in June and evaporation exceeds rainfall for all months except May, June and July.

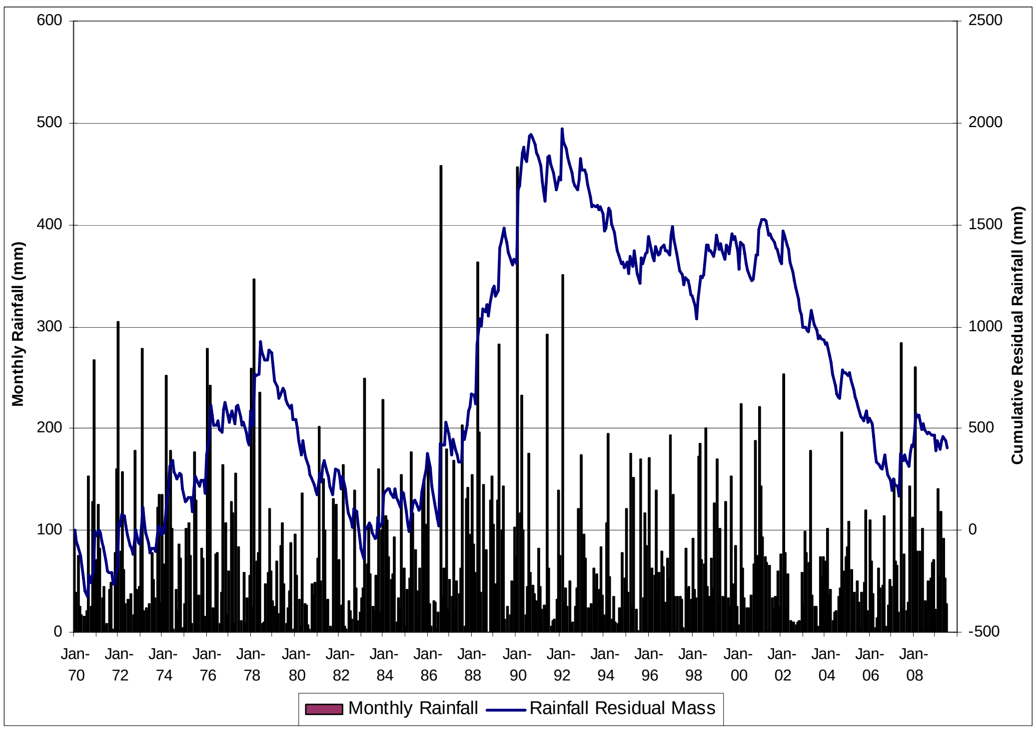
4.6.2 Long-Term Rainfall Data

Long-term monthly rainfall data has been obtained for the Prospect Reservoir BoM station. Data have been subject to residual rainfall analysis to assist in identification of rainfall trends, particularly during recent years for which some observational information is available regarding groundwater levels and quarry inflows.

Cumulative residual rainfall is calculated by subtracting the monthly average rainfall from the actual monthly rainfall for each month and adding each monthly residual value to the previous cumulative total. Time series graphs of cumulative residual rainfall allow long-term rainfall patterns to be assessed, with periods of above average rainfall are indicated by upward trends and periods of below average rainfall by downward trends.

A graph of cumulative residual rainfall from 1970 to date is provided as *Figure 4.6*.

Figure 4.6 Cumulative Residual Rainfall Graph



Examination of this graph indicates the following:

- Periods of generally above average rainfall occurred between 1971 and 1978 and between 1984 and 1990;
- Periods of generally below average rainfall occurred between 1979 and 1983 and between 2000 and 2007;

- The last two years have been characterised by generally above average rainfall in 2007 and average or slightly below average rainfall from May 2007 to date;

- Rainfall has typically been above average for period 1970 to 2008, with an average for this period of 1,184 mm compared to the long-term average of 872 mm.

The analysis above suggests that groundwater levels and therefore pit seepage will have been higher than typical for the period since 1970 but lower than typical for the recent period of 2000 to 2007.

5. Results of Field Investigations

5.1 Fracture Mapping

Mapping of fracture occurrence and orientation within the quarry was undertaken to assist in identification of areas of potentially increased fracturing around the quarry and selection of potential drilling sites (J&K, 2009). The detailed report is provided in *Appendix A* and includes a detailed description of the defects and a plan showing locations and orientations. Results are summarised below.

Mapping was carried out on 24th March 2009 by Paul Roberts of Jeffery and Katauskas Pty Ltd (J&K) accompanied by Ian Grey of IGGC. During mapping, defects were measured, photographed and described and estimates made of associate seepage rates. In addition, defect locations were marked to allow accurate mapping using optical surveying techniques. This latter task was undertaken by Crux Surveying Pty Ltd (Crux) on 2nd April 2009.

Defects were measured using a hand-held inclinometer and tape measure or by estimation where features where not directly accessible.

5.1.1 Pattern of Defects and Implications for Groundwater Flow

The geological setting of the site comprises an igneous diatreme approximately ovoid in plan with the perimeter defined by a ring fault feature. Defects associated with the diatreme would be expected to follow a pattern approximately parallel to the ring fault. Results of mapping indicate that this appears to be the case, with the majority of defects orientated approximately parallel to the perimeter of the quarry and a small number orientated approximately perpendicular. The following conclusions are drawn regarding the defect pattern within the quarry:

- Defects within the site orientated parallel to the diatreme margins would not extend outside of the site;
- Defects within the site orientated perpendicular to the diatreme margins would be expected to terminate at the ring fault;
- Defects present within the country rock outside of the quarry prior to intrusion would be expected to terminate at the ring fault.

The observed seepages rates within the quarry were generally of low volume; typically at or below 0.1 L/s and rarely approaching 1 L/s.

The defect pattern described above would suggest that groundwater contained in the surrounding country rock would flow towards the site along defects and be intercepted by the ring fault, from where seepage into the quarry would only occur along defect planes connected to the ring fault. A steep hydraulic gradient is present in the regional groundwater system around the site (ERM, 2008) and any substantial connectivity would be expected to result in discrete areas of concentrated and high volume groundwater inflow, none of which were observed during this or previous inspections.

5.2 Bore Site Selection

Five potential drill sites were selected based on all available information including the results of fracture mapping. These were locations where the greatest degree of fracturing and/or the greatest occurrence of groundwater might be expected to occur, and the primary criteria for selection were location of sites on the projection of mapped fracture system orientations, and location close to areas of visible seepage within the quarry. The proposed locations and the reasons for their selection were as follows:

1. North-eastern corner along the projection of fracture system #1 and due to the presence of seepage in this part of the quarry;

2. South-south-east area along the projection of fracture features #16 and #20;

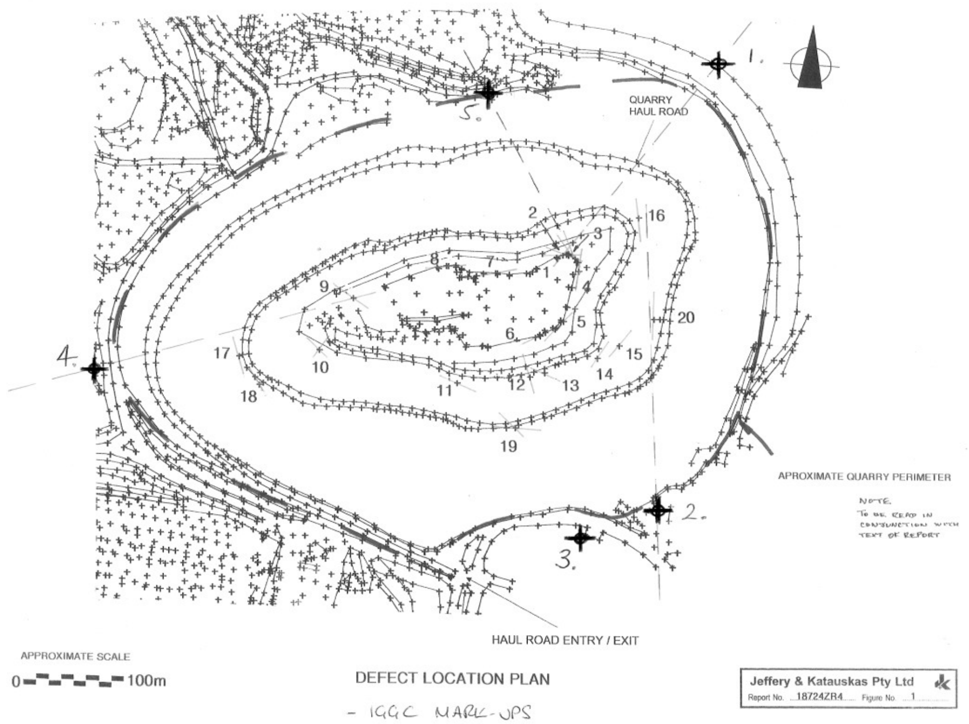
3. Southern area due to the presence of the strongest seepage. It should be noted, however, that it is IGGC's view that this seepage is associated with leakage from the quarry dewatering system over a long period (pipes, transfer pumps and surface channel) rather than reflecting true groundwater discharge;

4. West-south-west area along the eastern extension of the diatreme, on the projection of fracture systems #7, #8, #9 and due to seepage in the western corner of the quarry;

5. Northern area along the projection of fracture system #3 and due to seepage from the northern quarry face.

These locations are shown on a marked-up plan taken from J&K's report presented as *Figure 5.1*.

Figure 5.1 Potential Drill Sites



The proposed work scope required the drilling of a pair of bores (nominally 100m and 150m deep) at each of two locations. Based on review of the available data (summarised in the bullet points above) IGGC recommended selection of Location 1 and Location 4 as being sited in the areas most likely to host enhanced fracturing and greater occurrence of groundwater and therefore providing the greatest possible contribution to understanding of the local hydrogeological regime and the degree of connection between the pit and the surrounding groundwater system. This recommendation was provided to DECCW in a letter report (IGGC, 2009b) and a reply received (e-mail, 7/5/09) indicating acceptability.

5.3 Drilling

Drilling was conducted by Terratest Drilling. The nominal 150 m deep holes were drilled using an Edson 3000 drill rig. Auger drilling was undertaken to auger refusal, followed by roller bit drilling to 31 and 30 metres in Core Hole 1 (BH10d) and Core Hole 2 (BH12d) respectively. HQ coring (outside diameter ~93 mm) was undertaken to the base of each borehole (nominal 150 metres). During core drilling packer testing was conducted over every ten metre interval, as outlined below.

Coring was undertaken in 3 metre core runs, alternating with a 1 and 2 metre core run at the beginning or end of each ten metre interval (i.e. 3, 3, 3 and 1 m runs, followed by 2, 3, 3 and 2 metre runs) to balance the ten metre interval and undertake the packer testing on the bottom 6 metres of each ten metre interval.

Core was collected using HQ triple tube, with the triple tube core splits pumped out of the barrel and the core transferred to core trays. Recovered core sticks were often in excess of 1 metre in length hence it was necessary to break the core to fit it in core boxes. The site geologist evaluated all core, to assess whether core breaks were natural or induced by drilling and breakage, to fit the core trays. Where breaks were observed or assessed to be drilling related they were marked with a black cross over the break. The vast majority of breaks observed in the core are drilling induced. Photographs of core are provided in *Appendix B*.

Natural fractures are generally at 30^o or less to the long core axis (along the length of the core) and typically display polished striations or are associated with calcite veinlets. Natural breaks were typically noted with an S (for shear) written on the core for photographs.

Once core was marked up it was wetted down and photographed in sequence, to provide a photographic record of the drilling and rock quality. Core recovery was recorded by reconciling the intervals drilled (i.e. 1, 2 or 3 metres) against the core recovered. Overall recovery was excellent. Recovery in Core 1 was 99.5% and in Core 2 99.4%. Core recovery provides an important assessment of rock quality and allowed definition of areas with greater intensity of fracturing. Core recovery data are provided in *Appendix B*.

Core was geologically logged when core recovery was complete. The full geological logs of holes are provided in *Appendix B*. Units of siltstone, sandstone and shale were recognised. Overall the stratigraphy consists of an upper fine sandstone interbedded with siltstone, a sequence of interbedded siltstone and sandstone and a gradational transition into more laminated shale. These units are interpreted as the Bringelly Shale and underlying Ashfield Shale units of the Wianamatta Formation. The lower 5 metres of BH10d intersected coarse sandstone which is interpreted as the top of the Hawkesbury Sandstone.

Drilling of the nominal 100 m deep holes was undertaken using a Hydrapower drill rig, using a 6 inch (152 mm) diameter open hole hammer. Surface casing was installed by drilling with a rock roller to c.6 m, before the hole was continued by hammer drilling.

To ensure quality piezometer construction, with sufficient sand pack, bentonite and grout the core holes were reamed from the c.93 mm diameter to 152 mm diameter using the air hammer. This involved setting the Hydrapower drill rig up on the two core holes, before reaming, flushing the drill hole and installing the piezometer.

Drill cuttings from the auger, rock roller and hammer drilling were geologically logged in addition to the core from holes BH10d and BH12d. Geological logs are provided in *Appendix B*.

Final locations of the new and pre-existing piezometers are shown on *Figure 5.2*.

5.4 Packer Testing

Packer testing was carried out during core drilling, to collect information on formation permeability. Testing was conducted over the bottom 6 m of each 10 m core interval (e.g. 44-50 m in interval 40-50 m) from the initiation of coring at 30 m or 31 m (BH12d and BH10d respectively). The rock tested during packer testing was entirely fresh, the base of oxidation being at around 10 m depth, noted by a change from orange-brown to grey in the drill cuttings.

Packer testing is a standard geotechnical engineering test method used to evaluate the permeability of the rock mass surrounding a drill hole. For measurements at the site a single pneumatic packer device was used. Photographs showing the equipment used and the packer installation process are provided in *Appendix C*. The testing technique consisted of:

- Pulling back 7 metres of drill rods;
- Lowering the packer through the annulus of the drill rods with the wire line. The packer extends a metre below the base of the rods, to seal a 6 metre section between the base of the rods and the bottom of hole;
- Once seated in the core barrel the packer was fully inflated using compressed air, to a pressure of 1700-2500 kPa, depending on the depth of the packer;
- Water was pumped into the rods and packer from surface, allowing the packer and water line to fill. The rods were maintained full of water throughout the operation, to detect any leakage of water if the packer failed to seal fully;
- The pressure gauge test pressures were then used to select the appropriate test pressures, monitoring the test pressure and adjusting the water flow valve throughout the test, to keep gauge pressures as close to constant as possible;
- Test pressures were chosen using the depth of each packer test and the expected groundwater level - based on information from previous wells drilled on site, using data from wells closest in depth to the wells installed in this program. The test pressure was cross checked with the recommended design curve for packer testing;
- Packer tests were conducted at three different water pressures, i.e. 0.25, 0.5 and 1 times the calculated maximum test pressure. Testing was undertaken by stepping up to the maximum pressure and then back down through the first two pressure stages, noting changes in water flow on a minute by minute basis, with measurements consolidated into 5 minute intervals;
- In general measurements were attempted to obtain repeatability to within 10%. However, this was not possible for a number of measurements, where despite repeated measurements flow values varied by more than 10%. In part this is likely to reflect the variation associated with measuring small water volumes (<100 ml/minute), close to the limit of the flow measuring equipment. Typically flow rates took between 2 and 10 minutes to stabilise for each test;

- Packer test results were calculated during each test, to evaluate any irregularities. Increases in water loss were noted in some tests when stepping down through the water pressure levels in the later part of the tests. Only in one instance was the packer considered to have sealed incompletely, with unexpectedly high flow, despite a near absence of fractures in the core (BH12d 104-110 m). The packer was deflated, moved by less than half a metre and re-sealed. Upon reinflation of the packer the test was re-run, with stable flow measurements observed and recorded;
- After installation of piezometers in the drill holes repeated measurements of water level were made to evaluate whether water levels had returned to equilibrium.

5.5 Piezometer Installation

5.5.1 Installation

Following drilling of the hammer holes and reaming of the core holes to 152 mm diameter the holes were cleaned out. This involved mixing foam into approximately 500 litres of water and injecting this into the drill hole. Compressor air pumped through the drill rods (sitting above the base of the hole) was used to flush the water and foam mixture back up the hole, lifting cuttings that remained in the hole after drilling. Flushing lasted between half and one hour. Over this time the foam exiting the hole changed from brown to white, with the colour of the foam used as an indicator of when each borehole was sufficiently flushed.

Following cleaning each hole out with foam the hole was filled with potable water. This was undertaken to provide some buoyancy to the drill pipe during installation and to assist piezometer development. When the hole was full of water the piezometer installation was started. Note that during filling of BH10d a strong natural flow of water into the hole was detected at a depth of around 6 metres below surface. This is interpreted to be perched groundwater within the weathered upper sandstone at this site.

All 3 metre pipe and screen sections are screw jointed class 18 PVC, with O rings at each screw joint. An end cap was chemically bonded to the base of the screen section, and the screen lowered into the hole, followed by the solid pipe sections. Triangular plastic spacers were added to the piezometers at the screw joins generally every 9 metres, to centralise the piezometer in the hole.

Once the piezometer pipe was successfully installed in the hole 2 mm washed sand was slowly added around the standpipe at a rate of 3-5 L/minute. The depth to the sand was plumbed periodically to ensure that the appropriate amount of sand was added, bringing the sand a minimum of 5 metres above the top of the screen section. In BH12d there appears to have been a significant drilling cavity, or drilling deviation of the hole. Consequently more sand than calculated was required to reach the level of 5 metres above the top of the screen. Additional sand was added in this hole bringing the sand level to almost 14 metres above the screen level.

The bentonite seal was added in the form of bentonite coated quartz chips. These provide a higher density than bentonite chips or pellets and sink more quickly to ensure that a cohesive seal layer results. Two buckets were added to each hole, equal to two metres thickness when plumbed. A further bucket was added to hole BH12d, as the water level within the standpipe and in the annulus of the well (prior to grouting), suggested that bentonite seal may not have been completely effective prior to this.

A bentonite-cement mixture was used to seal the upper levels of each hole from above the bentonite plug to surface. Bentonite and cement was mixed in a drum, before being pumped into each hole with a trammie pipe. Grout was pumped into each hole until the grout mix reached surface level. As the grout mix shrunk as it set additional cement grout was added to fill the holes to surface level a week after the initial grout mix was added to each hole. Following the grouting to surface a steel monument was established over each of the piezometers and labelled with borehole name and depth. The standpipes were sealed with an orange lockable pressurised cap.

It was noted that borehole BH12d consumed a larger amount of grout mix than calculated, suggesting a number of cavities or deviation of the reamed drill hole from the original core hole, creating an additional hole that was grouted along with the annulus of the piezometer. Grouting was completed and the final completion is considered successful.

There were some difficulties with the installation of the piezometer BH12d, as fill material below the casing at the top of the hole blocked the hole during the addition of sand pack, when the sand level was close to the top of the screen section. It was necessary to set up over the piezometer with the drilling rig and insert 18 metres of PW casing into the hole around the piezometer standpipe. The PW casing was locked in position to prevent any further upper level blockage of the piezometer. HQ rods were subsequently lowered into the hole over the top of the piezometer, to the top of the sand pack to ensure the blockage was cleared, before the remaining sand pack was installed.

5.5.2 Development

Piezometers were developed using air from an industrial compressor rented for this purpose. $1^{1/4}$" MDPE rural irrigation pipe was connected to the compressor and run down the hole to approximately 2/3 the borehole depth, before gradually increasing the air flow to the pipe. The pipe airlifted a stream of water from the piezometer and the pipe was progressively pushed towards the base of the hole. When set less than a metre above the base of the hole the pipe was secured to the piezometer with duct tape and airlifting continued for one to two hours, depending on the water flow noted from the piezometer. Generally the water produced by each hole was relatively clean following initial airlifting.

5.6 Water Level Measurement

Water level measurements were taken from existing piezometers during the drilling and piezometer installation and from the new piezometers once they were completed. Measurements from the new wells were used to evaluate whether the new piezometers were reaching equilibrium water levels, following the addition of large volumes of fresh water to each hole during the drilling and flushing process. Dipping rounds were conducted on the new and pre-existing piezometers on 2nd July and 6th August 2009.

6. Results and Data Evaluation

6.1 Drilling and Piezometer Construction

6.1.1 Fracture observations

The rocks observed in drill core are very weakly fractured overall, with the vast majority of fractures observed in the drill core induced as part of the drilling process and loading of core into boxes. Core typically breaks along bedding planes, which are perpendicular to the drilling. When core breaks during drilling core sticks often grind against each other, contributing to core loss.

Where fractures are considered to be drilling induced they were marked by a black cross (*Appendix B – Photographs*). Fractures that are not considered to be drilling induced are generally 30^o to 60^o LCA and display striations on polished fracture planes, suggesting movement (possibly in the normal orientation).

Overall fracture densities are significantly below 1/m. The most fractured core zones do not generally exceed 5/m.

6.1.2 Piezometer Construction

Details of both existing and new piezometers are provided here for completeness. The numbering used for the existing piezometers is somewhat confusing, and while this has largely been retained for consistency a suffix has been added to each piezometer for clarity: "s" for shallow, "i" for intermediate and "d" for deep piezometers. Location and construction details and recent water levels for all piezometers are provided in *Table 6.1*.

Table 6.1 Piezometer Details

Bore	Easting mMGA	Northing mMGA	Ground Elevation mAHD	Datum (TOC) mAHD	Top of Screen mbgl	Base of Screen mbgl	Top of Screen mAHD	Base of Screen mAHD	Dip mTOC	Groundwater Level mAHD
BH1d	398585	6258169	69.32	69.19	127.8	133.8	-58.5	-64.5	49.19	20.03
BH2i	398585	6258165	69.32	69.22	43.7	49.7	25.6	19.6	31.32	37.87
BH3d	399044	6258501	79.66	80.35	134.8	140.8	-55.1	-61.1	48.99	31.36
BH4i	399062	6258501	79.88	80.49	43.6	49.6	36.3	30.3	39.04	41.45
BH5s	399068	6258499	80.03	80.55	14.7	20.7	65.3	59.3	12.68	67.86
BH6d	399215	6258043	84.43	85.02	141.9	147.9	-57.5	-63.5	74.17	11.02
BH7i	399212	6258041	84.52	85.19	44.8	50.8	39.7	33.7	30.34	54.78
BH8s	399212	6258037	84.62	85.12	14.8	20.8	69.8	63.8	17.78	67.24
BH9s	398585	6258161	69.30	69.23	14	20	55.3	49.3	9.01	60.22
BH10d	398563	6258102	69.96	71.51	135.3	150.3	-65.3	-80.3	40.305	31.20
BH11i	398562	6258101	70.57	70.92	88	100	-17.4	-29.4	31.6	39.32
BH12d	399211	6258432	79.99	80.95	136	151	-56.0	-71.0	92.38	-11.43
BH13i	399215	6258432	80.33	81.31	88	100	-7.7	-19.7	39.78	41.53

Notes. mMGA is metres Map Grid of Australia. mAHD is metres Australian Height Datum. TOC is Top of Casing. mbgl is metres below ground level.

Groundwater levels were measured on 6th August 2009 except that for BH1d which was measured on 21st February 2008. BH1d is blocked by a sampling pump and is not operational.

6.2 Packer Testing

6.2.1 Packer Test Interpretation and Results

Interpretation of packer test results was undertaken using the methodology presented by Burgess (1983) and that of Houlsby (1976). The former method calculates the lugeon value by taking the averaged slope defined by the five test data points and extending it to 1000 kPa, when drawn as a line from the origin of the water loss versus gauge pressure graph. The water loss value in uL is the Burgess Lugeon permeability. In the Burgess interpretation, zero values equated to decreasing flow with increasing pressure and a negative graph slope.

The Houlsby Lugeon permeability is calculated by dividing the (L/min/m) value measured for each step of the test by 1000/the corrected pressure (in KPa). Based on the relationship of the individual measurements a value is chosen as representative for the test.

The majority of packer test results returned Lugeon values of below 0.5 µL: this is the lower limit of reliable values for the technique although values below this are reported. Test results are summarised in *Table 6.2*. Tests where leakage flow rates were noted to be high (>1 L/min) generally returned positive graph slope gradients and Lugeon values up to 5 uL. Full results of packer test analyses are provided in *Appendix D*.

Plotting packer test results (uL) against depth shows some indication of a general decrease in hydraulic conductivity with depth, as is generally observed in geological environments. These are shown graphically in *Figure 6.1*.

In general packer test results indicate generally low or negligible hydraulic conductivities. BH10d shows results ranging from 0 uL to 5 uL (0.043 m/d) with a geometric mean value of 0.15 uL or 0.0013 m/d (note: zero values are treated as half the lowest value of 0.04 uL to allow calculation of geometric mean values). BH12d shows results ranging from 0 uL to 0.5 uL (0.0043 m/d) with a geometric mean value of 0.08 uL or 0.0007 m/d. There is little clear evidence of a correlation between the highest lugeon values and either rock type or occurrence of fracturing, however the rock types are reasonably consistent and the degree of fracturing is invariably low.

6.3 Water Level Measurement

Water level measurements were undertaken on both new and existing piezometers to provide a full dataset for modelling and assessment. Water levels were measured on 2nd/3rd July 2009 and 6th August 2009 and this data collated with earlier data available for the pre-existing piezometers. These data are presented graphically as time series graphs for the deep, intermediate and shallow piezometers as *Figure 6.2a*, *Figure 6.2b* and *Figure 6.2c* respectively and are summarised as follows:

- Deep groundwater levels vary from around -20 mAHD to +30 mAHD. Only BH03d and BH10d show levels that appear to have stabilised with final groundwater levels of around 31 mAHD; other bores show evidence of continued groundwater level rise due to recovery after drilling and development and/or sampling. Early data for the pre-existing bores show the effects of slug testing by addition of water;

- Intermediate groundwater levels vary from around 9 mAHD to 55 mAHD. Only BH07i shows levels that appear to have stabilised with a final groundwater level of 54.78 mAHD; other bores show evidence of continuing recovery;

- Shallow groundwater levels vary from 54 mAHD to 76 mAHD with recovered water levels of around 86 mAHD in BH05s and BH08s and of around 60.2 mAHD in BH09s. The lower groundwater level in the latter bore is likely to reflect topographic effects with the surface elevation at this bore being 10 m to 15 m lower. Shallow groundwater levels appear to have stabilised relatively quickly after drilling and development etc.

Overall, groundwater levels are consistently highest in the shallow aquifer levels and lowest in the deep aquifer levels, consistent with the conceptual model of the local groundwater system. Deep and intermediate piezometers show slow recovery after drilling and development etc., reflecting the very low hydraulic conductivity of the strata. Recovery in some of the pre-existing bores is still not complete over 18 months after sampling. Only three out of the eight operational deep and intermediate bores show stabilised groundwater levels and only BH10d has show rapid stabilisation.

6.4 Assessment of Detailed Hydrogeological Setting

Consideration of all available data including those from previous investigation and from the recent study allow re-assessment of the detailed hydrogeological setting of the site to confirm the accuracy of the understanding and conceptual model developed previously and to modify these as necessary. To assist in this process a detailed hydrogeological cross-section has been constructed through the site from BH10d to BH12d via the deepest part of the quarry. This is presented as *Figure 6.3*.

Consideration of the detailed hydrogeological cross-section and of the other available data indicates that the conceptual model developed previously is broadly correct and that the hydrogeological setting can be summarised as follows:

- The hydrogeological setting comprises a layered aquifer system including a perched aquifer in the upper weathered profile and a series of aquifers in the more transmissive horizons of the underlying bedrock;

- The upper weathered profile shows low to moderate hydraulic conductivity. Groundwater levels are around 67 mAHD and the hydraulic connection between the shallow aquifer and the quarry appears limited;

- The intermediate Wianamatta Group aquifer layers (i.e. the upper to middle zones in the bedrock, c.30 m to 100 m depth) show generally negligible or very low hydraulic conductivities with occasional zones of higher values of up to 0.04 m/d. Stabilised groundwater levels are around 55 mAHD; natural levels would be expected to be slightly below those of the shallow groundwater zone and this shows the effect of depressurisation caused by pumping of groundwater from the quarry;

- The deep Wianamatta Group aquifer layers (c.100 m to 150m depth) show generally negligible or very low hydraulic conductivity values with occasional zones of higher values of up to 0.01 m/d. Stabilised groundwater levels are around 31 mAHD showing the effect of depressurisation although this is less than appeared the case from the results of previous investigation;

- The Hawkesbury Sandstone occurs beneath the Wianamatta Shale Group strata at an elevation of around -72 mAHD, around six metres below the deepest parts of the quarry. Hydraulic conductivity is low (0.003 m/d) and groundwater levels are similar to those is the overlying deep Wianamatta Group strata;

- The quarry exploits volcanic breccia of the Minchinbury Diatreme and these strata form the walls of the quarry beneath the first one or two benches. Observational data of the extent of fracturing and seepage within the quarry indicate that these strata are of very low hydraulic conductivity.

- Pumping of groundwater from the quarry has results in a steep inward hydraulic gradient in the bedrock strata. Effects appear limited in the shallow weathered profile indicating limited hydraulic connection between these strata and the quarry. Despite the steep gradients seepage rates into the quarry are low (c.30 m^3/day) reflecting the very limited occurrence of fracturing and therefore very low hydraulic conductivity values of the bedrock strata;

- Under natural conditions a low, downward hydraulic gradient would be expected to occur. This has been increased as a result of depressurisation resulting in relatively high downward gradients;

- The regional groundwater system is fed by low levels of rainfall recharge with groundwater flow controlled by discharge to creeks to the east and west of the site and to the Hawkesbury-Nepean system to the north.

7. Numerical Modelling

7.1 Conceptual Model

The groundwater model has been developed as a three-dimensional representation of the area around the former quarry.

The conceptual model consists of a layered aquifer system to represent the upper residual soil profile, the weathered shale, the fresh shale (including the more transmissive horizons as indicated from the results of packer testing) and the underlying sandstone. The model represents an area of 120 square kilometres and is bounded by distant constant head boundaries to the south and north, up and down the dominant groundwater flow direction, and distant no-flow boundaries to the east and west, across the dominant flow direction. The base of the model is a no-flow boundary set at an arbitrary depth of minus 150 metres AHD, 77 metres below the top of the sandstone.

Regional groundwater flow is controlled by discharge to creeks to the east and west of the site and to the Hawkesbury-Nepean system to the north. The southern limit of the groundwater flow system is likely to be a groundwater divide coinciding with the topographical divide located around 8 kilometres south of the site. Rainfall recharge will have resulted in the creation of a recharge mound centred on this mound with local recharge mounds present between the creek lines and other discharge zones, including the quarry.

A schematic diagram of the conceptual hydraulic model is provided as *Figure 7.1*.

Migration of potential contaminants from the site after complete re-pressurisation is simulated using MODPATH to provide information on directions and timescales for migration of a conservative solute and therefore potential impacts under such conditions.

7.2 Detailed Hydraulic Model

Modelling was undertaken with the VISUAL MODFLOW V4.4 software package. This uses the USGS MODFLOW code which is an industry-standard finite-difference modelling code for simulation of groundwater flow. Both the code and the software package are widely used in Australia and overseas.

The basic hydraulic model has been developed using the best available estimates for the various parameters using packer test and other site-specific data where possible. Sensitivity analysis has been carried out to determine the relative importance of the various parameters, and the potential effects on model results of variations.

The hydraulic model has been constructed in two stages. Firstly the model was built to represent the multi-layered groundwater flow system under natural, steady-state conditions, i.e. prior to quarry development. This was undertaken to ensure that simulated groundwater levels and flow patterns under such conditions were realistic (albeit with very little data available for calibration) to provide a basis for subsequent model development. The second stage included simulation of the quarry by means of drain cells to represent groundwater inflows and steady-state simulation to allow model calibration against observed groundwater levels and quarry inflow rates.

7.2.1 Model Grid Design

The total model extent is 12 kilometres (east-west) by 10 kilometres (north-south), with a total area of 120 square kilometres. The required extent was based on topographical features including distance to the nearest major creeks and test modelling to determine the likely extent of the cone of depression generated by pumping from the quarry.

Initial grid spacing was set at 100 metres, refined to 50 metres around the quarry and to 25 metres across the quarry and immediate surrounds. The model extent is shown on *Figure 7.2* and the model grid is shown on *Figure 7.3*.

The model has been set up using thirteen layers to represent the various strata as detailed in *Table 7.1*. The surface elevation is not be used in the model, and is therefore set at a default value of 85 mAHD. The base of the aquifer was set at -150 mAHD: this is 78 m below the top of the Hawkesbury Sandstone underlying the Wianamatta Shale group and is considered to be sufficient deep so as not to affect model results.

7.2.2 Aquifer Parameters

The model has been constructed using the interpreted geological profile based on the results of drilling (particularly logging of core), packer testing and data from earlier investigation and assessment. A cross section of the model geological profile is provided as *Figure 7.4*. The various strata types and their properties are given in *Table 7.1* and are represented by different colours in *Figures 7.4*.

Table 7.1 Summary of Aquifer Properties of the Model Layers

Model Layer	Stratum	Interval	Hydraulic Conductivity (m/d)		Specific Storage (per m)	Specific Yield (%)	Effective Porosity (%)	Total Porosity (%)
		mAHD	Kx/y	Kz				
1	Residual Clay	+85 to +52	0.004	0.0004	$1x10^{-5}$	5	5	10
2	Weathered Shale	+52 to +36	0.001	0.0001	$1x10^{-5}$	1	1	5
3	Fresh Shale (high k)	+36 to +30	0.043	0.0043	$1x10^{-5}$	1	1	5
4	Fresh Shale (low k)	+30 to +6	0.001	0.0001	$1x10^{-5}$	1	1	5
5	Fresh Shale (high k)	+6 to 0	0.018	0.0018	$1x10^{-5}$	1	1	5
6	Fresh Shale (low k)	0 to -4	0.001	0.0001	$1x10^{-5}$	1	1	5
7	Fresh Shale (high k)	-4 to -10	0.04	0.004	$1x10^{-5}$	1	1	5
8	Fresh Shale (low k)	-10 to -44	0.001	0.0001	$1x10^{-5}$	1	1	5
9	Fresh Shale (high k)	-44 to -50	0.011	0.0011	$1x10^{-5}$	1	1	5
10	Fresh Shale (low k)	-50 to -66	0.001	0.0001	$1x10^{-5}$	1	1	5
11	Fresh Shale (low k)	-66 to -72	0.001	0.0001	$1x10^{-5}$	1	1	5
12	Sandstone	-72 to -105	0.0035	0.00035	$1x10^{-5}$	1	1	5
13	Sandstone	-105 to -150	0.0035	0.00035	$1x10^{-5}$	1	1	5
3 to 12	Volcanic Breccia	+36 to -155	0.001	0.0001	$1x10^{-5}$	1	1	5
1 to 10	Compacted Fill	0 to -66	0.01	0.001	0.001	0.2	0.2	0.3

The values for the aquifer properties used are based on site specific data, published values (Domenico & Schwartz, 1990; Fetter, 2001); IGGC's experience of Sydney Basin aquifers and descriptive information available from site investigation borehole logs. The available site and region-specific data are discussed in *Section 2.2*, and the derivation of the different values is discussed below:

Residual Clay: these strata are derived from weathering of shale, and comprise plastic mottled clays with varying amounts of relict structure from the source strata. The selected hydraulic conductivity value is based on the results of on-site testing in wells BH8 and BH9 (ERM, 2009) which have response zones in this stratum.

Weathered Shale: the upper section of the shale strata (generally up to 30m depth) varies from slightly weathered to extremely weathered. Hydraulic conductivity is largely governed by the degree of fracturing and while some fracture enhancement can exist from weathered infilling of fractures by weathering-derived clays also occurs. The selected hydraulic conductivity value is based on observations made during drilling and published values.

Wianamatta Group Strata (unweathered Bringelly Shale and Ashfield Shale): this geological unit comprises interbedded claystone, siltstone, laminite and minor sandstone. Hydraulic conductivity is mostly controlled by the degree of fracturing with the rock mass being virtually impermeable. Results of packer testing indicate hydraulic conductivity values of less than >0.5 µL (0.004 m/day) for the unfractured intervals to 4.6 µL (0.04 m/day). A value of 0.001 m/d has been assigned for those intervals showing packer test results below 0.5 µL with hydraulic conductivity for other intervals based on measured values. Porosity values have been selected based on published data (Domenico & Schwartz, 1990) with the low effective porosity assigned reflecting transmission of groundwater via fractures of limited occurrence and size.

Sandstone: the upper part of the Hawkesbury Sandstone/Mittagong Formation has been assigned a horizontal hydraulic conductivity of 0.003 m/day based primarily on the results of packer testing of the lowest section of borehole BH10D. The low effective porosity assigned reflects transmission of groundwater via fractures.

Volcanic Breccia: the remnant igneous strata not removed by quarrying occur immediately around and beneath the quarry. Visual observation of this material indicates a very low hydraulic conductivity controlled by the degree of fracturing which is sparse.

Compacted Fill: the proposed landfill will accept non-putrescible general solid waste which will comprise VENM, construction and excavation waste, paper and cardboard and non-putrescible household and commercial waste. Default characteristics for moderately compacted fill provided in the HELP user manual (Schroeder et al, 1994) indicates vertical hydraulic conductivity values for a range of material types varying from 5.9×10^{-4} m/day to 1.6×10^{-2} m/day. Horizontal values are expected to be up to an order of magnitude above this range. The selected value is around the mid-point of the range. Porosity values have been selected from published data (Schroeder et al, 1994, Domenico & Schwartz, 1990).

The values adopted for the model are considered to be the most likely to prevail for the groundwater systems beneath the site. Adjustments were made during the hydraulic modelling process to confirm that the flow model is relatively robust, and that flow conditions do not change substantially in response to minor changes in selected values.

7.2.3 Hydraulic Boundaries

River Boundaries

The local creek systems are represented as river boundaries with river stage (water) levels estimated from topography. River boundaries allow water to enter the groundwater model when predicted groundwater levels are below river stage levels and to leave it when groundwater levels are above river stage levels. River bed conductance is calculated by MODFLOW based on a vertical hydraulic conductivity value of 1 m/d for the river bed. This results in a low to moderate conductance to represent the clay-rich nature of the river bed material although the hydraulic conductivity of the underlying strata is expected to be the main control of interchange rates (note: the actual conductance value is calculated by MODFLOW on a cell-by-cell basis). River boundaries are shown on *Figure 7.2*.

Constant Head Boundaries

Constant head boundaries have been assigned at the upstream and downstream model limits based on estimated groundwater level in these areas. These are located approximately 8 kilometres south and 8 kilometres north of the quarry respectively. These boundaries will allow groundwater flow into and out of the model to be simulated, but are sufficiently distant from the quarry so as not to influence water level behaviour. The location of the model boundaries is shown on *Figure 7.2*.

No-flow Boundaries

The eastern and western model boundaries are located approximately 5 kilometres ad 7 kilometres from the quarry respectively. These are approximately parallel to the direction of regional groundwater flow and are represented as no-flow boundaries. Given their orientation approximately parallel to the direction of regional groundwater flow, their distance from the quarry and the presence of river boundaries between each no-flow boundary and the quarry these are not expected to influence groundwater behaviour in the area of interest.

The base of the model is represented as a horizontal no-flow boundary at minus 150 mAHD. This is 84 m below the quarry base and 78 m below the top of the Hawkesbury Sandstone underlying the Wianamatta Shale group and is considered to be sufficient deep so as not to affect model results.

7.2.4 Recharge

Rainfall recharge has been applied over the upper model layer. The selected value is 2 mm per year, just above 2% of the average annual rainfall of 872 mm. This value has been selected based on published estimates of rainfall recharge to the shale of 1% to 3% of precipitation (McNally, 2009) and IGGC's experience with such terrain. Model sensitivity to recharge was assessed during the model calibration progress by upward and downward adjustment of the selected value and observation of the effect on predicted groundwater levels.

During simulation of groundwater repressurisation after landfilling and cessation of quarry pumping a value of 40 mm per year was applied, around 5% of average annual rainfall. Research on performance of compacted sandstone capping at sites operated by Waste Service NSW in the Sydney area has indicated that estimated infiltration rates of 15 to 20% of total rainfall greatly overestimated the leachate generation (Pym & Thom, 1996) and the selected value is considered to be realistic or a slight over-estimated for a site with good surface water management and an effective capping layer.

7.2.5 Evapo-transpiration

Evapo-transpiration (EVT) simulates removal of water by plants when the groundwater surface is close to the ground surface beneath landscaped areas. Removal is determined by the maximum evapo-transpiration rate (groundwater at surface) and the extinction depth (groundwater depth at which evapo-transpiration becomes zero), and MODFLOW calculates removal based on a linear relationship with depth.

Insufficient data were available for accurate representation of ground surface elevations across the model area and EVT was therefore not simulated. The focus of this modelling study was groundwater behaviour in the deeper aquifer systems. EVT will only be a significant process with respect to shallow perched groundwater in the soil zone and in areas of low surface elevation i.e. around creeks where water that would be removed from the model by EVT will be removed by the river boundary cells instead.

7.3 Hydraulic Model Testing and Calibration

7.3.1 Simulation of Pre-Quarry Groundwater Conditions

The first stage of model development was steady-state simulation of groundwater conditions prior to excavation of the quarry and associated dewatering. This was undertaken to ensure that groundwater behaviour under such conditions was realistic. While limited data were available for calibration under such conditions, expected groundwater levels can be estimated based on existing levels in the shallow, perched aquifer system which is in limited hydraulic connection with the quarry, and from understanding of the regional groundwater flow pattern.

Shallow groundwater levels in piezometers located around the quarry vary from 49.3 mAHD (BH9s) to 63.8 mAHD (BH8s). Other piezometers on the site installed as part of earlier contamination investigation (ADI, 1995) show shallow groundwater levels up to 68.8 mAHD (MW4).

Simulated groundwater contours in the shallow aquifer are shown on *Figure 7.5*. This shows groundwater levels of between 60 mAHD and 65 mAHD in the area of the quarry. Groundwater flow is generally from south to north with groundwater mounds beneath the ridges and groundwater lows beneath the creeks indicating that groundwater discharge to the surface water system is occurring. Groundwater contours in the deeper aquifer layers show a similar pattern of groundwater flow but with generally slightly lower groundwater heads beneath the quarry area, indicating a relatively low but consistent downward vertical hydraulic gradient in areas located away from discharge zones.

Overall this is considered to be a realistic representation of pre-quarry groundwater conditions.

The water balance for the pre-quarry model is summarised in *Table 7.2*. This shows a very low discrepancy and based on this, the realistic predicted groundwater conditions and good model convergence the model is considered to be robust.

Table 7.2 Pre-Quarry Model Water Balance

Source/Sink	In (m^3/day)	Out (m^3/day)
Recharge	657.49	0
Constant Head	13.36	186.66
River	2.28	486.31
Drain	0	0
Total	673.12	673.0
Discrepancy	**0.16 m^3/day 0.02%**	

7.3.2 Simulation of Existing Groundwater Conditions

The second stage of model development was steady-state simulation of existing groundwater conditions, i.e. with the quarry present and dewatering taking place.

Dewatering from the quarry was represented using drain cells. These were placed in concentric circles in the relevant model layers at the locations and with the elevations estimated from the topographical survey of the quarry. Model cells located inside of the drain cells represent the quarry void space and were made inactive. The final layer of drain cells represents the quarry base and has an elevation of -66 mAHD.

The model was calibrated against observed groundwater levels and against the estimated rate of groundwater inflow to the quarry. Results of calibration are summarised in *Table 7.3*.

Table 7.3 Summary of Model Calibration, Existing Groundwater Conditions

Calibration Point	Observed Value (m)	Model Prediction (m)	Difference (m)
Deep Wells			
BH10d	31.2	37.12	+5.92
BH12d	-11.43	39.30	+50.73
BH3d	31.36	37.04	+5.68
BH6d	11.02	39.39	+28.37
Intermediate			
BH11i	39.32	39.38	+0.06
BH13i	41.53	41.58	+0.05
BH2i	37.87	43.59	+5.72
BH4i	41.45	45.95	+4.50
BH7i	54.78	48.25	-6.53
Shallow Wells			
BH5s	67.86	57.39	-10.47
BH8s	67.24	58.77	-8.47
BH9s	60.22	50.43	-9.79
Quarry Inflow	29.4 m^3/day	67.0 m^3/day	37.6 m^3/day

Results of calibration show the following:

- Deep groundwater shows good calibration for BH10d and BH3d but poor results for BH6d and particularly BH12d. The observed water level for the latter, however, does not represent the recovered water level, with the groundwater level rising by 10 m between 2nd July 2009 and 6th August 2009. This calibration point should therefore be disregarded for the time being. The reason for the discrepancy for BH6d is not clear: the water level in this well does appear to be anomalously low compared to the other deep bores: this may reflect incomplete recover due to a poor connection to the regional groundwater system or a relatively good connection to a seepage point within the quarry. It may also reflect inaccuracy in placement of the drain cells in this area due to limitations of the available data.

- Intermediate groundwater shows excellent calibration results for BH11i and BH13i and good results for the remaining intermediate wells.

- Shallow groundwater shows that the model consistently under-estimates shallow groundwater levels by around 10 m. This is likely to indicate that the actual hydraulic connection between the shallow groundwater system is less strong than that represented in the model, or that surface disturbance and presence of fill material etc. have led to localised increases in rainfall recharge. The behaviour of the shallow groundwater system is not critical to this assessment and this error is therefore considered to be tolerable.

- Quarry inflow is over-estimated by the model by around 100%. This will provide a conservative assessment as the predicted rate of groundwater recovery will be faster than that likely to occur in reality. This discrepancy may be due to over-estimation of hydraulic conductivity of some strata in the model or to under-estimation of the actual rate of groundwater inflow perhaps due to evaporative losses (see *Section 3.5.2*). Improved calibration could not be achieved as decreasing the hydraulic conductivity of the strata generally results in increased groundwater levels which will worsen calibration results.

Overall, results of calibration are considered to be acceptable particularly for a complex hydrogeological setting such as this. Model predictions are expected to be conservative as calibration results suggest that the degree of connectivity between the quarry and the surrounding groundwater system and perhaps the hydraulic conductivity of some strata may be over-represented.

The water balance for the steady-state quarry model is summarised in *Table 7.4*. This shows a very low discrepancy.

Table 7.4 Quarry Model Water Balance

Source/Sink	**In (m^3/day)**	**Out (m^3/day)**
Recharge	657.49	0
Constant Head	19.14	178.52
River	2.50	432.83
Drain	0	67.02
Total	679.18	678.37
Discrepancy	**0.81 m^3/day 0.12%**	

7.3.3 Simulation of Cessation of Quarry Pumping

The third stage of modelling comprised use of the model developed previously as a basis for a transient-state model to simulate the effects of cessation of groundwater pumping from the quarry. This was undertaken as follows:

- Setting a model copied from the quarry simulation to run in transient state for a period of 11,000 days (c.30 years);

- Modifying the quarry drain cells to be active for the first 1,000 days of the model run to ensure stable conditions then switching them off for the remainder of the model period to represent cessation of pumping;

- Making the quarry cells active and setting aquifer parameters to simulate the presence of compacted waste;

- Placement of an imaginary observation well in the centre of the quarry screened through the simulated waste mass to allow assessment of the predicted rate of leachate level rise;

- Increasing the rate of rainfall recharge across the quarry to 40 mm/year from the 1,000 point in the model run.

It was not attempted to simulate development of the quarry or associated pumping of leachate/groundwater inflows during filling as the aim was to simulate the worst-case response: i.e. groundwater recovery and leachate accumulation without any pumping.

Results of this model run can be summarised as follow:

- The groundwater/leachate level in the imaginary quarry well shows a relatively slow rate of increase from 1,000 days to 1,700 days probably reflecting the limited saturate thickness of aquifer via which groundwater inflow can occur until some recovery has taken place. A greater rate is then predicted from 1,700 days to 2,500 days followed by a slowing rate for the remainder of the simulation due to the decreasing hydraulic gradient. Leachate levels are predicted to rise as shown in *Table 7.5* and *Figure 7.6*.

  Table 7.5 Predicted Leachate Level Recovery

Day after Cessation of Pumping	Leachate Level (mAHD)
500	-57.34
1,000	-34.50
2,000	6.17
3,000	20.31
4,000	29.84
5,000	41.08
7,500	46.78
10,000	54.68

  Complete recovery is not predicted to occur within the modelled period. The rate of rise is, however, likely to be greater than the rate of waste placement, with predicted rates of rise of around 5 metres per year in the first two years but up to 23.5 m/yr in years 3 and 4 before declining to less than 5 m/yr by year 9. This indicates that leachate level management will be required during the operational phase, including installation of a leachate collection system and pumping of leachate for appropriate disposal.

- Groundwater level recovery in the deep wells is predicted to be c.15 m over the first ten years but full recovery is not predicted to occur within the simulation period;

- Groundwater level recovery in the intermediate wells is predicted to be slightly slower than that in the deep wells at c.13 m over the first ten years with complete recovery not predicted.

- Groundwater level recovery in the shallow wells is negligible for the first ten years as shallow groundwater conditions are effectively unchanged until recovery has occurred in the deeper groundwater systems. Complete recovery is not predicted within the simulation period.

7.3.4 Steady-State Simulation of Final Conditions

The fourth stage of modelling was simulation of groundwater conditions after completion of landfilling and with no leachate/groundwater pumping. This is intended to provide an assessment of the potential for migration of leachate from the site under such conditions and therefore the risk posed by development of the site to the groundwater environment. The quarry drain cells were removed and the landfill simulated as in the previous model and the model was then run in steady state to predicted final conditions. In addition, imaginary groundwater "particle" were placed in a circle immediately outside of the quarry (in the strata beyond the limits of the diatreme) in each model layer to allow prediction of the rate of migration of a conservative solute from the site, i.e. with no retardation processes occurring. The final predicted hydraulic heads in the upper model layer are shown in *Figure 7.7* and results of particle tracking are shown in *Figure 7.8*. Results are summarised as follows:

- Final leachate levels are predicted to be c.77 mAHD, i.e. above the surrounding groundwater level in all strata and above the local ground surface level in some areas. This is the result of the recharge mound predicted to develop as a result of the higher rate of recharge across the landfill area compared to the surround natural strata and results in a potential for migration of leachate contamination from the site into the surrounding groundwater system.

- Migration of groundwater away from the site is predicted to be very slow, reflecting the low hydraulic conductivity of the surrounding strata and the relatively low outward hydraulic gradient. The fastest migration rates are predicted to be around 100 years for a conservative solute to travel 400m (i.e. around 4 m/year) and occur in areas of the highest hydraulic gradients (generally to the north and west).

The results of simulation of final conditions assuming no pumping of leachate and with levels permitted to rise higher than would be realistic (i.e. above local ground surface levels) indicates that migration in groundwater from the site is predicted to be very slow. Such slow migration is expected to be sufficient to allow attenuation of pollutants and no detectable impact on groundwater quality would be expected. This assessment is based on highly conservative assumptions and is based on migration from strata around the site; it therefore does not take account of the time required for leachate to migrate from within the quarry through the volcanic breccia and into the surrounding strata.

In addition, the above assessment assumes a relatively high rate of rainfall recharge into the waste mass equivalent to that which would be expected for a capped and vegetated surface. In this case, however, it is proposed to redevelop the quarry site for commercial and industrial use after completion of landfilling. This will result in most of the area being covered by hard, impermeable surfaces with effective stormwater drainage and long term rainfall recharge under such conditions is expected to be negligible.

7.3.5 Sensitivity Analysis

Adjustments were made to the recharge and hydraulic conductivity and constant head boundaries to confirm that the flow model is relatively robust, and that flow conditions do not change substantially in response to minor changes in model parameters. The response to the main hydraulic parameters is summarised below:

- Recharge: higher recharge values resulted in elevated groundwater levels, and at high values groundwater levels exceed ground surface elevations. There were no significant changes to the groundwater flow regime other than an increase in hydraulic gradient and therefore groundwater flux.

- Hydraulic Conductivity: increasing the hydraulic conductivity of the bedrock strata resulted in increased groundwater flow including predicted inflows to the quarry. This also resulted in decreased groundwater levels and worsened model calibration significantly. There was no significant change to the flow regime.

8. Assessment of Potential Impacts and Mitigation Requirements

8.1 Groundwater Inflow

Groundwater inflow to the quarry has been estimated at around 30 m^3/day based on measurement of water level recovery in the quarry pond during cessation of pumping. Results of numerical modelling predict a rate of inflow of around 67 m^3/day and this will result in model predictions being conservative. This very low rate of inflow means groundwater inflow will make a very minor contribution to leachate generation with the great majority being generated from rainfall infiltration. This low inflow rate also means that there are unlikely to be any major operational difficulties with groundwater management. In addition, the rate of groundwater inflow will decrease over time should water levels within the quarry be allowed to rise.

Leachate comprising groundwater seepage together with rainfall runoff and infiltration will be collected and pumped from the quarry during filling. This water is expected to show chemistry broadly similar to that from the existing quarry pond which currently comprises groundwater seepage mixed with rainwater runoff, with high pH and elevated nitrogen levels (both natural). This water may require treatment prior to discharge to the local surface water system (if required).

8.2 Predicted Groundwater Level Behaviour and Implications

Groundwater levels in the aquifer systems surrounding the quarry have been subject to substantial depressurisation as a result of groundwater pumping during the 40+ years of quarrying. This has resulted in groundwater heads up to 31 m below natural levels immediately around the quarry, and the quarry forms the centre of a cone of depression or drawdown. The lateral extent of this drawdown cone is not known although results of modelling suggest that the cone is steep due to the low hydraulic conductivity of the surrounding strata and is unlikely to be significant beyond one kilometre or so from the quarry rim. The extent of drawdown is expected to be negligible in the shallow groundwater system, and most extensive in the deep aquifers.

If pumping from the quarry were to cease, groundwater levels would rebound, eventually returning to close to natural levels of around 60 to 65 mAHD or slightly greater (depending of local rainfall recharge conditions). The timescale for complete recovery of groundwater levels under conditions where the site has been developed as a landfill but with no pumping taking place is predicted to be in excess of 30 years. Repressurisation is expected to bring a return to groundwater conditions similar to those that would have occurred naturally prior to quarry and dewatering with development of a recharge mound centred on the quarry due to the higher rate of rainfall infiltration into the waste mass compared to natural recharge to the Wianamatta Shale strata. Should such conditions be allowed to develop there is some potential for impacts from the landfill due to migration of leachate into the surrounding groundwater system; however results of modelling indicate that such migration would be extremely slow due to the low hydraulic conductivity of the strata and the relatively low hydraulic gradient that would result. The potential for adverse impacts on the local groundwater system is therefore considered to be negligible even under such conditions. In reality a leachate management system will be maintained and pumping of leachate will take place such that levels are kept below groundwater levels in the surrounding strata, thereby maintaining an inward hydraulic gradient and removing any potential for outward migration.

8.3 Suitability for Landfill Site Development

The quarry represents a very low risk site for development of a solid waste landfill in terms of potential groundwater and related impacts, because of the following factors:

- the strong inward hydraulic gradient under existing conditions removes the possibility of migration of contaminated groundwater away from the quarry during the operational phase, and during the initial post-closure period while leachate level are being controlled;

- the very low hydraulic conductivity of the surrounding strata, poor natural groundwater quality and low level of groundwater use in the area greatly limit the potential for impacts on groundwater should an outward hydraulic gradient develop in the future;

- the low groundwater inflow rate means that groundwater inflow will not present operational difficulties in terms of water management;

- the nature of the quarry will necessitate active management of stormwater, with collected water pumped to discharge points via settlement ponds etc.;

The quarry is therefore considered highly suited to landfill site development, providing that appropriate management and control measures are implemented.

In addition, the proposed development is for a landfill accepting non-putrescible solid waste. The leachate generated within such a site is unlikely to be highly polluting and this further decreases the risk posed by development.

8.4 Outline Design Requirements

The quarry site is in a very safe hydrogeological setting for rehabilitation from both an operational viewpoint and in terms of potential groundwater impacts. The following outlines recommended design requirements for the site.

8.4.1 Requirement for of a Low Permeability Barrier

Provision of a low permeability barrier or landfill liner is not considered necessary across the base or up the sides of the quarry for the following reasons:

- The very low rate of groundwater inflow and the limited contribution that this will make to leachate generation compared to rainfall infiltration means that there is no requirement for a liner in terms of controlling groundwater inflow;
- The low-risk hydrogeological setting afforded by the low permeability of the surrounding strata, the poor natural water quality and low level of groundwater use in the area, and the strong inward hydraulic gradient. This gradient will be maintained throughout the operational and post-closure periods by management of leachate within the site;
- The limited degree of hydraulic connection between the quarry and the upper weathered strata which are host to a shallow, perched groundwater system; and,
- The low-risk nature of the proposed fill material, i.e. non-putrescible solid waste rather than material with a great pollution potential.

Provision of a barrier system in the quarry would offer no management or environmental benefits, other than perhaps some reduction in the already low rate of groundwater inflow. There is therefore no justification for provision of such a barrier. Construction of landfill liner systems in deep, hard-rock quarries is in any case very difficult and the practicability of construction of a barrier system offering effective, long-term benefits in this case is doubtful.

Provision of a barrier system in the upper part of the quarry is also considered to offer little benefit as these strata are also of very low hydraulic conductivity; the degree of hydraulic connection between the shallow groundwater system and the quarry is very limited and because leachate management will be required such that leachate levels are maintained below the surround groundwater levels. Provision of effective vertical drainage around the perimeter of the landfill in the upper level is considered to be a better and more practicable means of ensuring protection of the shallow groundwater system.

8.4.2 Leachate Management Requirements

Groundwater and rainfall runoff are currently pumped from the quarry, previously to allow quarrying and currently to maintain access. Limited groundwater seepage into the quarry will continue during rehabilitation via fractures in the base and sidewalls of the quarry. Rainwater will also collect in the base of the quarry via infiltration through the placed fill and runoff from the quarry sides.

In a landfill site, water seeping through the waste and collecting in the base of the site is referred to as leachate. This water undergoes chemical changes within the waste mass, both by leaching chemicals from the waste and through chemical and biological processes occurring during decomposition of the limited degradable content. In this case, the fill material will comprise non-putrescible solid waste, and the potential for leaching and chemical changes will be limited.

Provision of an interception and collection system is required to allow control of water accumulation within the quarry during filling, both for operational reasons (to prevent water levels rising too close to the surface of placed fill) and to allow control of the depressurisation process that will take place in the surrounding groundwater system. There are two broad options for design of the collection system:

- A permanent basal drainage system, comprising a basal drainage blanket with a herringbone arrangement of slotted pipes (alternatively a herringbone arrangement of slotted pipes surrounding by rubble drains may be acceptable), a main basal sump fed by the piped drains, and a riser to allow pumping of collected water. The riser should ideally comprise an inclined solid pipe running up the side of the quarry and fixed to the sidewalls to prevent damage or dislocation due to settlement of fill. However a vertical riser progressively constructed through the fill would be acceptable if preferred. A secondary sump and riser is recommended to allow contingency water management in the event of failure of the primary system; or,

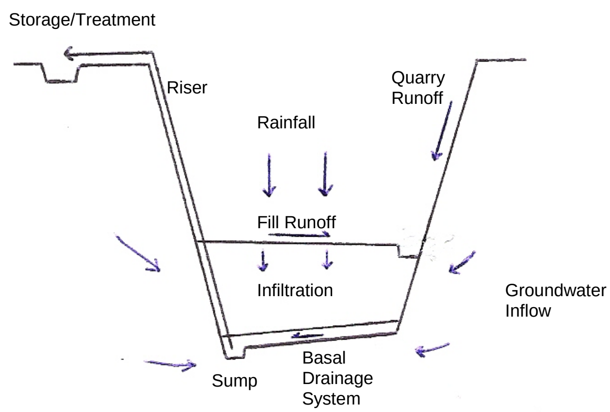
- Progressive construction of drainage systems at various levels during filling, to allow control of leachate levels during each phase of filling. The first drainage layer and sump would therefore be constructed on the quarry base, and filling would proceed with the sump raised progressively until the final height of the first filling phase was reached. The fill surface would then be laid to fall to a new sump and compacted, and a new drainage layer placed (with piped drains as needed and overlain by geotextile). Filling would then proceed again.

The former allows full control of water level within the quarry at all times, although it does rely on efficiency of drains, sumps and risers being maintained throughout and after filling, with a final burial depth of around 180m. The latter approach avoids this problem, although care would be needed with water management during construction of each new drainage system to ensure that sufficient collection and pumping capacity is available at all times. This latter approach is the preferred option and is recommended.

In addition, the fill surface should be laid and compacted at a suitable gradient, and surface runoff directed to a collection dam where possible to minimise the contribution of rainfall run-off to leachate generation. Run-off from haul roads and stockpiling/processing areas should also be collected.

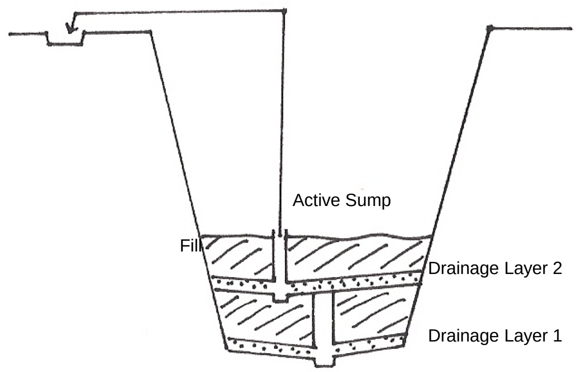
The main features and conceptual design of the water collection system are shown in *Figure 8.1*.

Figure 8.1: Water Collection System – Conceptual Design

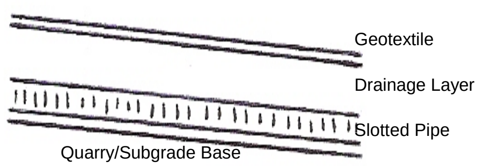


Conceptual Permanent Drainage System and Inflows

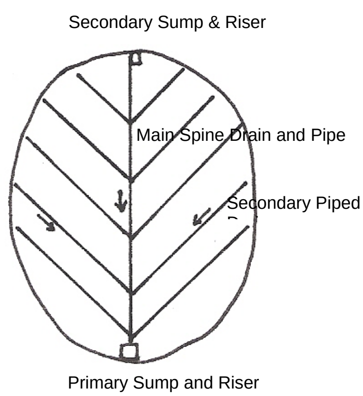
Figure 8.1: Water Collection System – Conceptual Design (continued)



Conceptual Progressive Drainage Systems



Drainage Layer (Section)



Drainage Layer (Plan)

All pipes and risers should have appropriate crush and shear resistance, and should be designed to allow cleaning for maintenance and in the event of blockage.

Water levels in the base of the quarry should be maintained as required operationally. Maintenance of very low water levels is <u>not</u> recommended, as this would result in maximum groundwater inflow. Water levels should be maintained a few metres below the lowest point on the fill surface at any time; or at a lower level if buffering storage within the fill mass is required for runoff generated during storm events, based on requirements to be determined from water balance calculations and on groundwater management requirements; i.e. maintenance of leachate levels below groundwater levels in the surrounding strata.

Collected water should be pumped to holding ponds for testing and treatment (if required), prior to reuse on site for dust suppression etc., or discharge to the stormwater system. Irrigation over the fill mass to promote evaporation could also be considered if volumetric reduction is required. Based on the available data, collected water is expected to be suitable for on-site reuse, but treatment is likely to be required to reduce nutrient levels prior to discharge to the local surface water system.

8.5 Mitigation Measures and Requirements for Further Investigation

Assessment of the existing quarry excavation and surrounding groundwater regime indicates that the site is well-suited to development as a non-putrescible solid waste landfill site, with a low risk to the environment and no difficult management issues identified. The local groundwater regime is well understood including the likely timescale of rebound of groundwater levels on cessation or reduction of pumping.

Control of water levels within the quarry will allow management of groundwater levels in the quarry and surrounding strata, if required. The nature of the waste material to be accepted at the site will be carefully controlled. No further mitigation measures are considered necessary to protect groundwater.

Ongoing monitoring of groundwater and leachate levels and water quality will be required during the active landfilling period and post-closure. The existing groundwater monitoring network is considered to be sufficient to ensure protection of the local groundwater systems. Water level monitoring should be undertaken using pressure transducers and dataloggers to allow transient groundwater level responses to pumping and rainfall recharge to be determined.

Numerical modelling of the local groundwater system and repressurisation due to cessation of groundwater pumping has been undertaken and provides a good degree of confidence regarding groundwater behaviour. The numerical model is suitable for use in assessment of future leachate level control strategies, if required, and comparison of future groundwater monitoring data to model results can be used to provide further confidence and to allow pumping strategies to be refined as necessary.

9. Conclusions

DADI proposes to develop a non-putrescible solid waste landfill site at the former quarry site Eastern Creek. The existing quarry forms a deep excavation with steep, stepped sides, approximately 180m deep and plan dimensions of around 600m by 400m.

Geology and Soil

The site is underlain by strata of the Wianamatta Group, generally comprising claystone, siltstone and minor sandstone. The Minchinbury Diatreme occurs beneath the site and is exploited by the quarry. This is a steep-sided conical structure approximately 850m by 300m, comprising volcanic breccia. The diatreme extends beyond the south-western limit of the quarry. Alluvial deposits of Quaternary age occur along Ropes Creek, and minor alluvium may occur along drainage lines.

Hydrogeology

The strata of the Wianamatta Shale group have limited potential to transmit groundwater flow, with the majority of flow occurring via fractures and bedding planes. The formation generally forms a layered aquifer system, with discrete aquifers occurring within horizontal fracture zones. The groundwater pressure surface generally follows topography. Natural groundwater levels in the area of the site are around 65 mAHD. Groundwater quality is generally poor, with high salinity levels. Groundwater usage in the area is very limited.

A weathered profile comprising mottled clays generally overlies the shale, and a perched shallow groundwater system occurs within this stratum.

The Minchinbury Diatreme would originally have formed a large, poorly fractured rock mass within the Bringelly Shale. Groundwater quality associated with such igneous bodies can show highly alkaline water and elevated levels of inorganic nitrogen.

Pumping from the quarry has resulted in substantial depressurisation of the local groundwater systems, with levels over 30 m below the estimated natural groundwater level. Estimated inflow rates are around 30 m^3/day, indicating the very low permeability of the surrounding strata. Limited water quality data suggests relatively low salinity but high pH and presence of inorganic nitrogen, typical for groundwater associated with an igneous body mixed with rainfall runoff.

Assessment of Potential Impacts –Landfill Site Development

Groundwater inflow to the quarry is very low, with the estimate of 30 m^3/day of groundwater alone and around 125 m^3/day including rainfall runoff and recirculation. Groundwater seepage and rainfall infiltration will be collected and pumped from the quarry during filling: this water is expected to be alkaline with elevated nitrogen levels (both natural), and treatment may be required.

Pumping from the quarry has resulting in substantial depressurisation of the surrounding groundwater systems, with the quarry forming the centre of a drawdown cone. The extent of drawdown is very localised in the shallow groundwater system and most extensive in the deep aquifers with the maximum extent of significant drawdown expected to be limited to a distance of one kilometre from the quarry. If pumping ceases, groundwater levels will rebound, eventually returning to close to natural levels of around 65 mAHD over a timescale of over 30 years. Pumping from the landfill site for leachate management will further reduce the rate of re-pressurisation.

The quarry represents a very low risk site for landfill site development in terms of potential groundwater impacts because of the very low permeability of the surround strata and limited degree of hydraulic connection with the shallow groundwater system; the strong inward hydraulic gradient; and the low groundwater inflow rate.

Results of numerical modelling indicate that the potential for impacts on groundwater due to leachate migration from the site is very low, with migration rates predicted to be very slow even for worst-case condition in which no pumping takes place for an extended period.

The site is therefore considered highly suited for landfill development providing that appropriate management and control measures are implemented. Provision of a low permeability barrier or landfill liner is not considered necessary and would offer no environmental or management benefits because of the above factors and because of the nature of the proposed fill material. This includes the upper parts of the quarry where the shallow weathered strata occur. Control of leachate levels using a carefully designed leachate management system in conjunction with monitoring of groundwater levels is the surrounding strata is considered to be a more effective and practicable means of ensuring environmental protection.

10. Recommendations

General

Assessment of the existing quarry excavation and surrounding groundwater regime indicates that the site is well-suited to landfill development, with a low risk to the environment and no difficult management issues identified. The local groundwater regime is well understood and use of numerical modelling has provided detailed assessment of the likely rebound of groundwater levels on cessation or reduction of pumping and potential impacts on groundwater from the site.

Landfill Development

Provision of a low permeability barrier or landfill liner is not considered necessary and would offer no environmental or management benefits. This includes the upper parts of the quarry where the shallow weathered strata occur. Control of leachate levels using a carefully designed leachate management system in conjunction with monitoring of groundwater levels is the surrounding strata is considered to be a more effective and practicable means of ensuring environmental protection.

A leachate management system to allow interception, collection and removal of water accumulating in the landfill site is required. The recommended approach is to construct series of drainage systems progressively during filling at various levels through the fill profile with only the upper drainage system in use at any time. Leachate levels should be maintained as required operationally, either a few metres below the fill surface, or at a lower level to provide buffering storage. Leachate levels should also be kept below the groundwater levels in the surrounding strata. Pumped water is expected to be suitable for on-site reuse, but treatment is likely to be required prior to discharge to surface waters.

Control of leachate levels will allow management of groundwater levels in the quarry and surrounding strata, if required. The nature of the waste accepted will be carefully controlled. Ongoing monitoring of groundwater and leachate levels and water quality will be required during the active landfilling period and post-closure. The existing groundwater monitoring network is considered to be sufficient to ensure protection of the local groundwater systems.

No further mitigation measures are considered necessary to protect groundwater.

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Figures

FIGURE 4.1: Site Location

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

IGGC

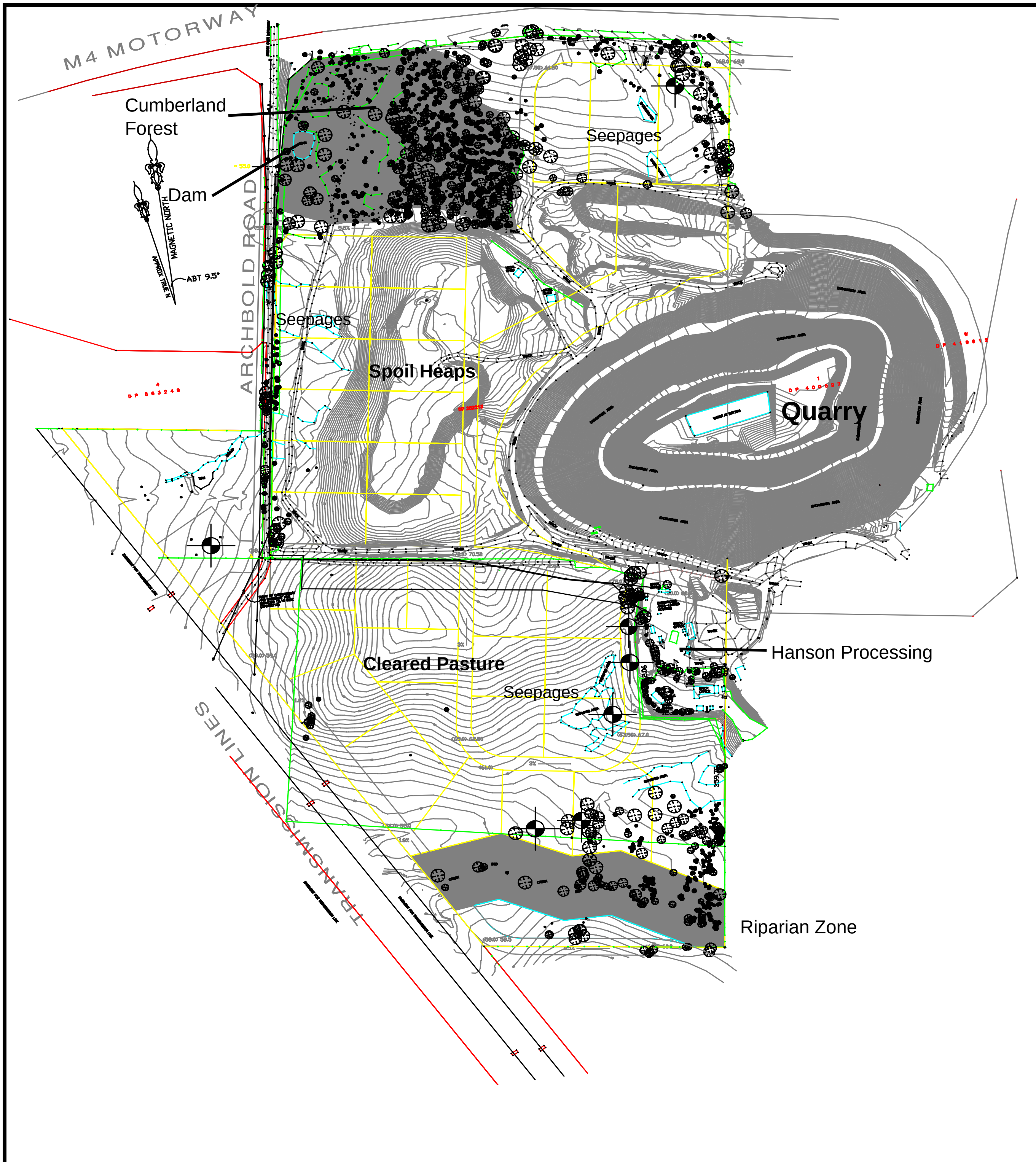

FIGURE 4.2: Site Features

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

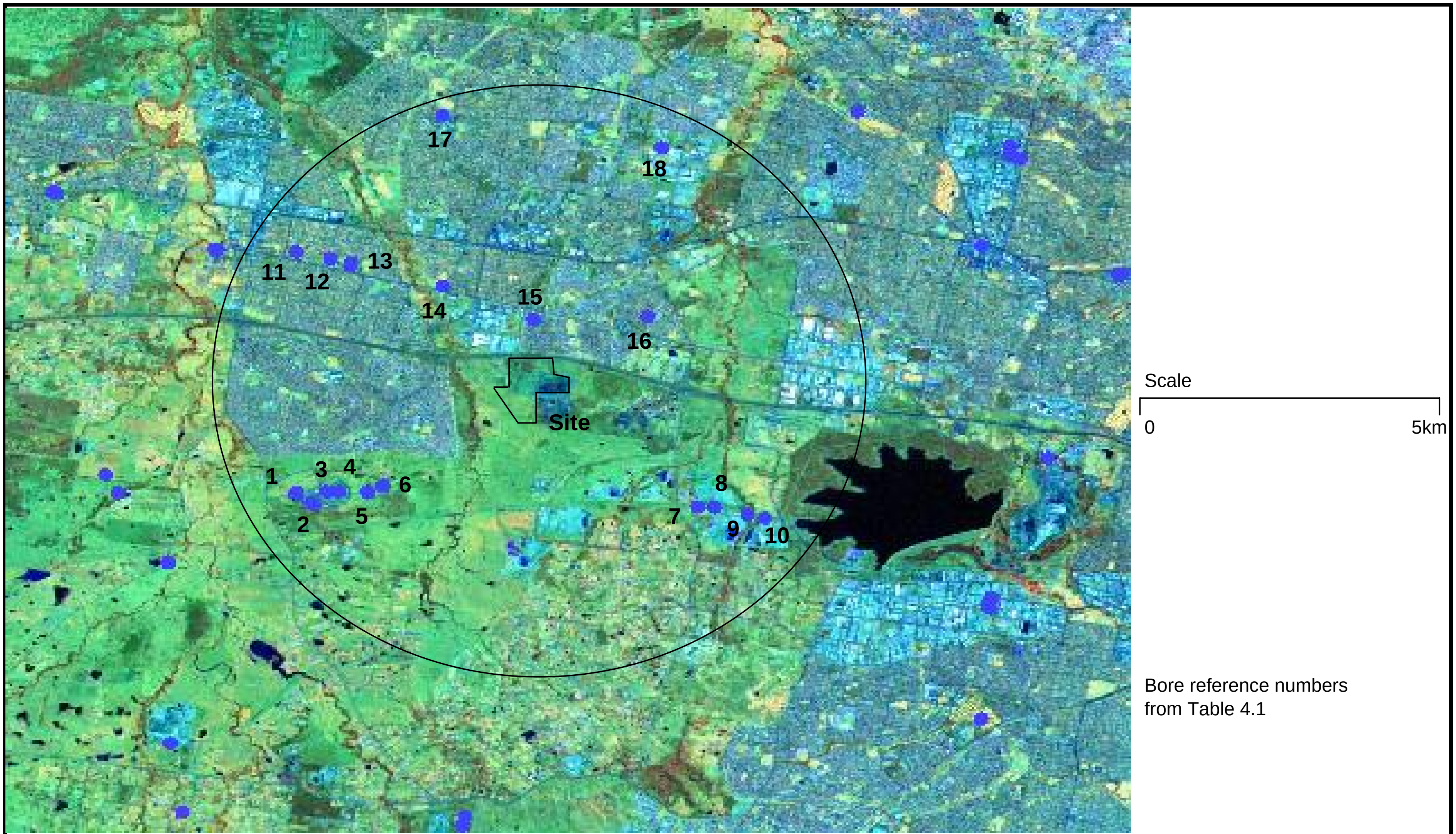

FIGURE 4.3: Locations of Registered Bores

Project: Detailed Hydrogeological Investigation and Assessment

Location: Proposed Light Horse Landfill Site, Eastern Creek

Client: Dial A Dump Industries Pty Ltd

Project No: BJ07

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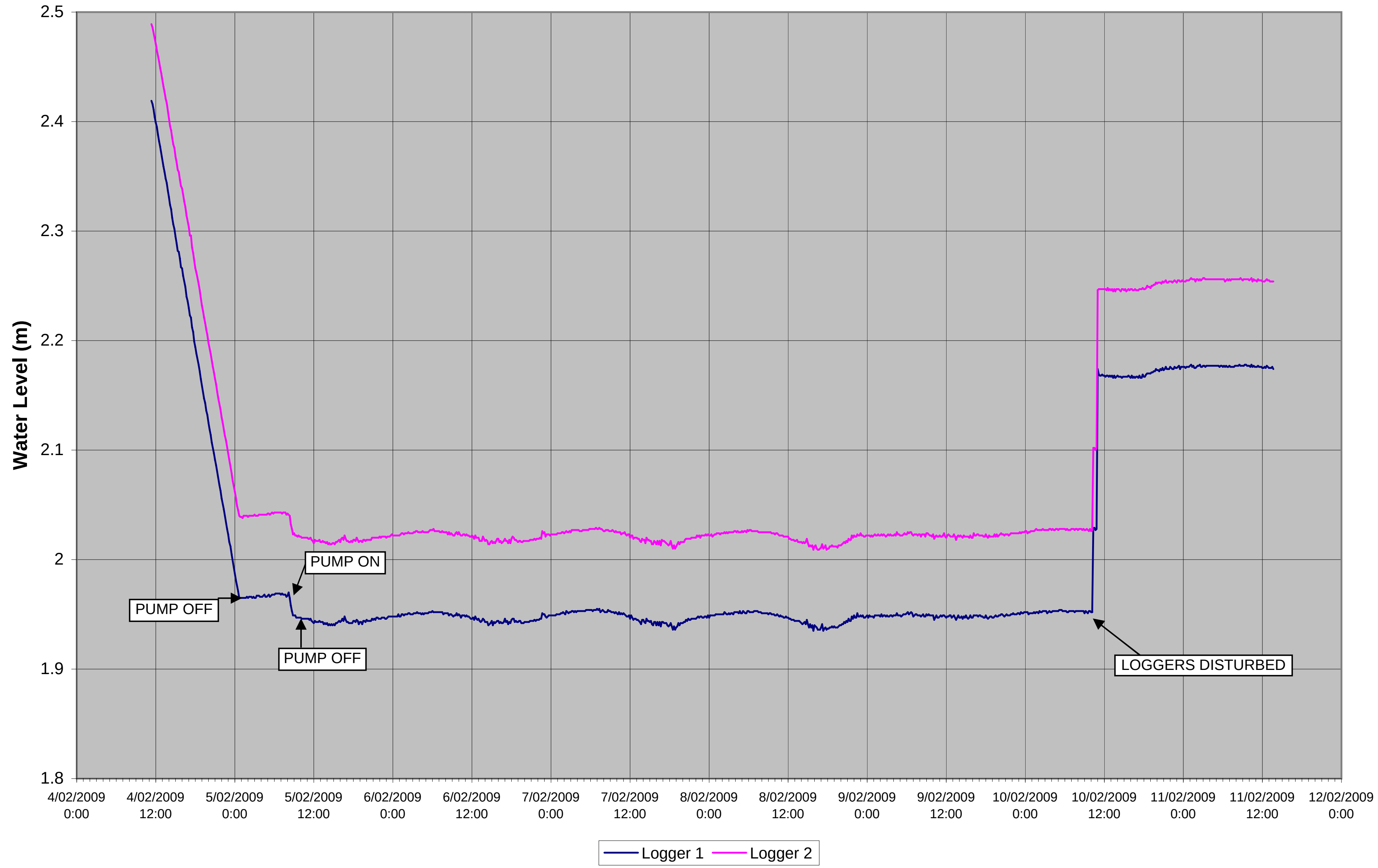

FIGURE 4.4: Quarry Pond Water Level Record

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

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FIGURE 5.2: Piezometer Locations (approximate)

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

Deep Piezometer
Intermediate Piezometer
Shallow Piezometer

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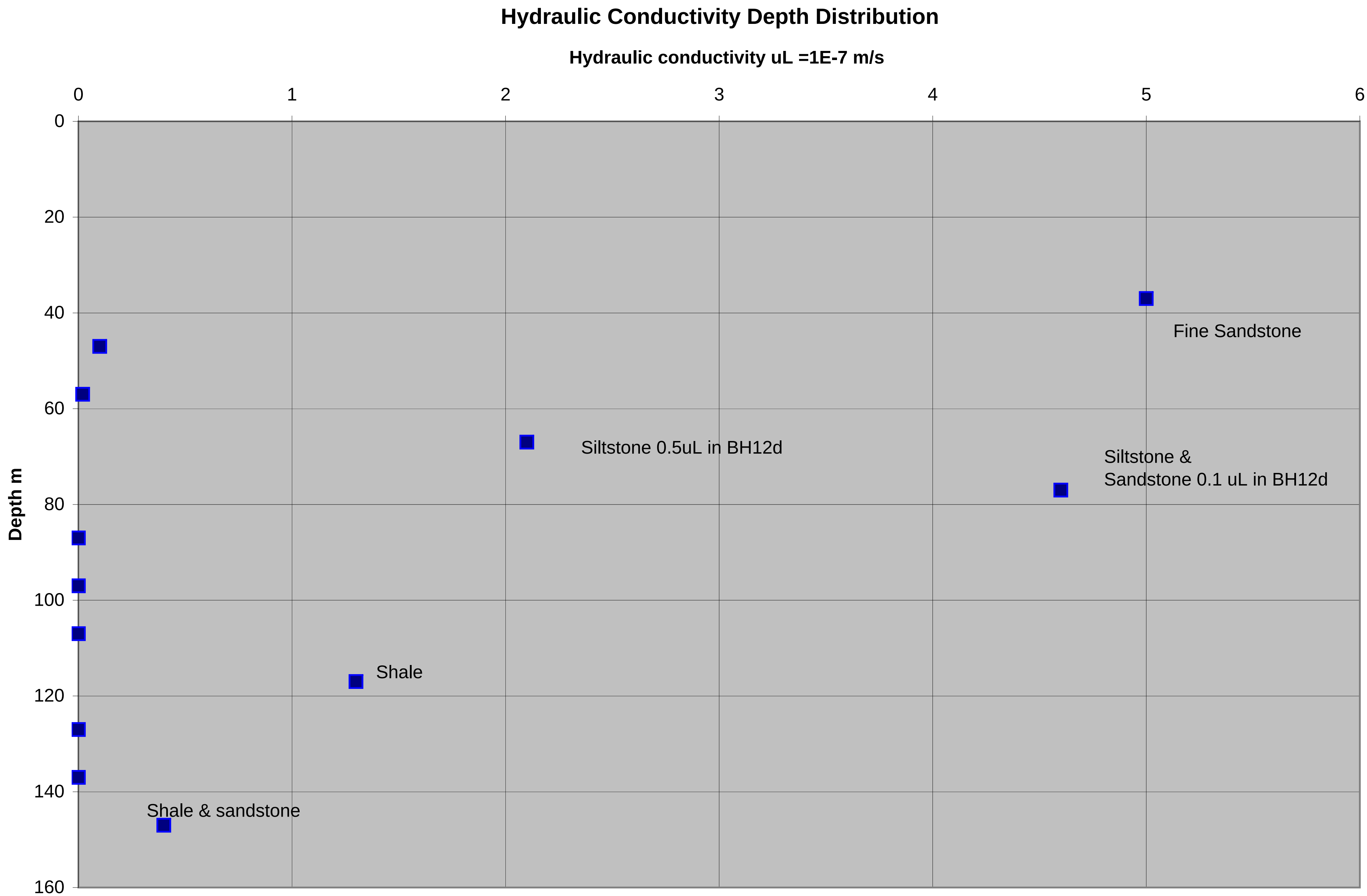

FIGURE 6.1a: Hydraulic Conductivity Depth Distribution from Packer Test Results - BH10d

Project:	Detailed Hydrogeological Investigation and Assessment
Location:	Proposed Light Horse Landfill Site, Eastern Creek
Client:	Light Horse Business Centre Pty Ltd
Project No:	BJ07

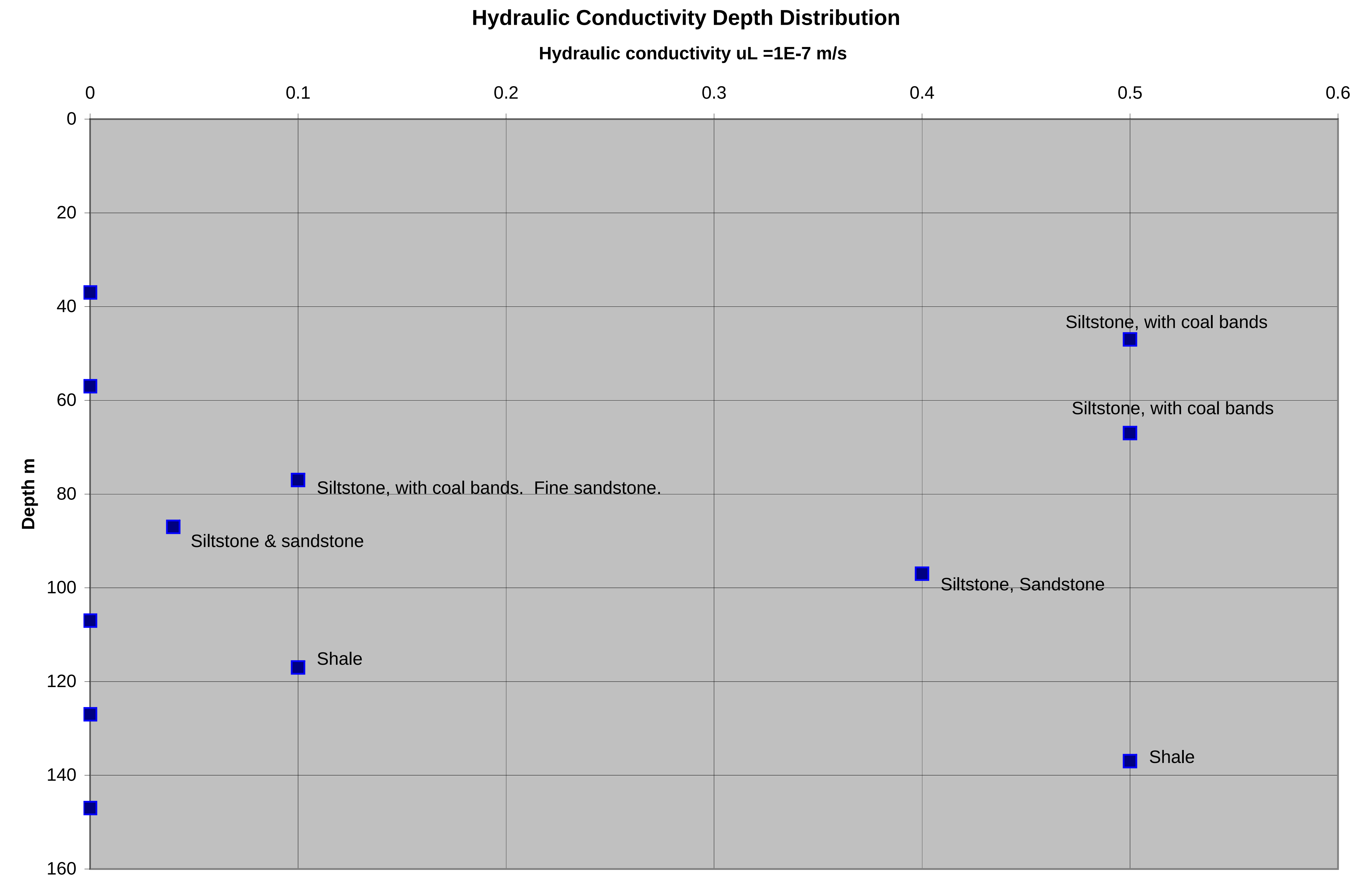

FIGURE 6.1b: Hydraulic Conductivity Depth Distribution from Packer Test Results - BH12d

Project:	Detailed Hydrogeological Investigation and Assessment
Location:	Proposed Light Horse Landfill Site, Eastern Creek
Client:	Light Horse Business Centre Pty Ltd
Project No:	BJ07

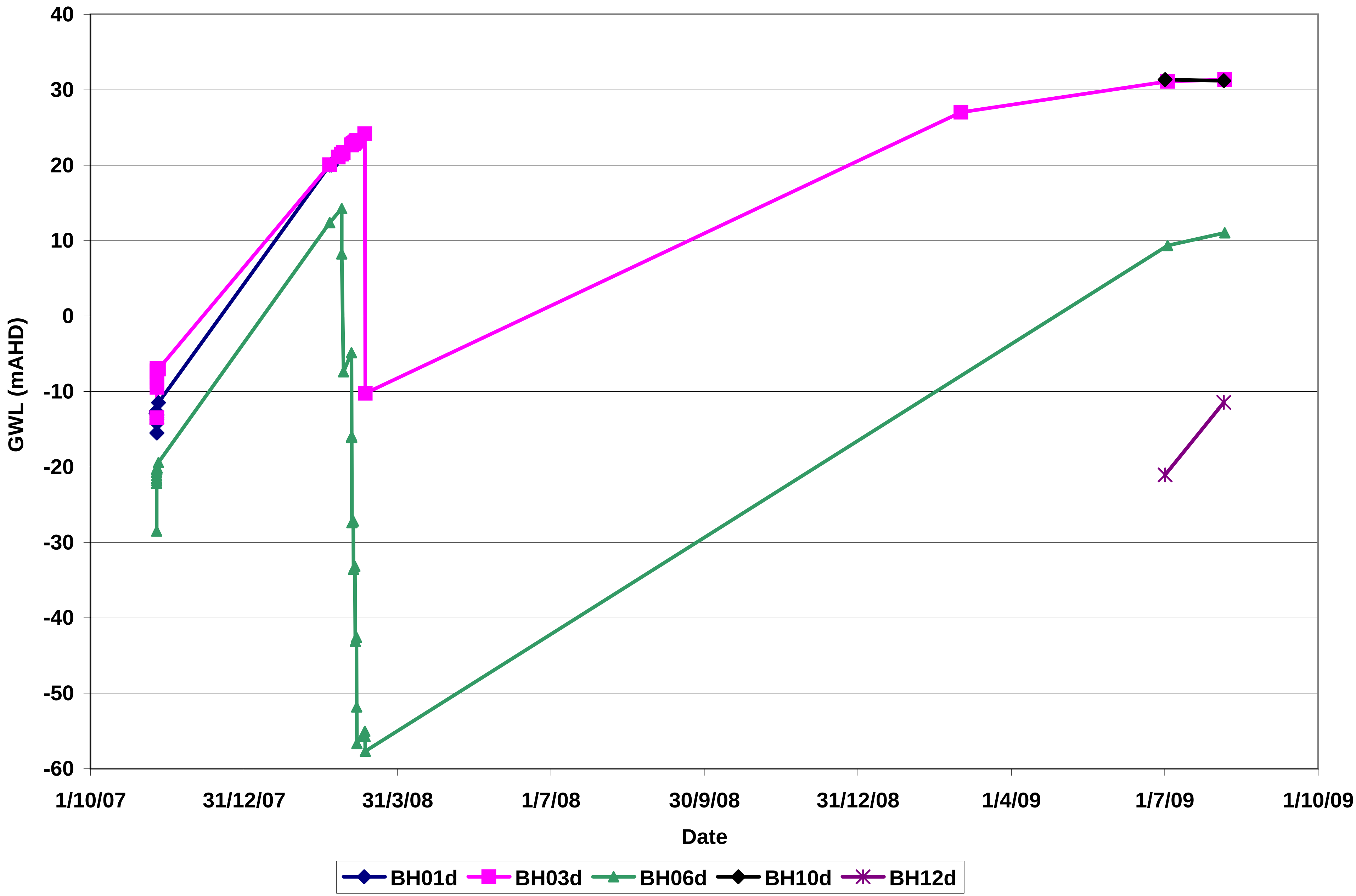

FIGURE 6.2a: Groundwater Levels - Deep Bores

Project: Detailed Hydrogeological Investigation and Assessment

Location: Proposed Light Horse Landfill Site, Eastern Creek

Client: Dial A Dump Industries Pty Ltd

Project No: BJ07

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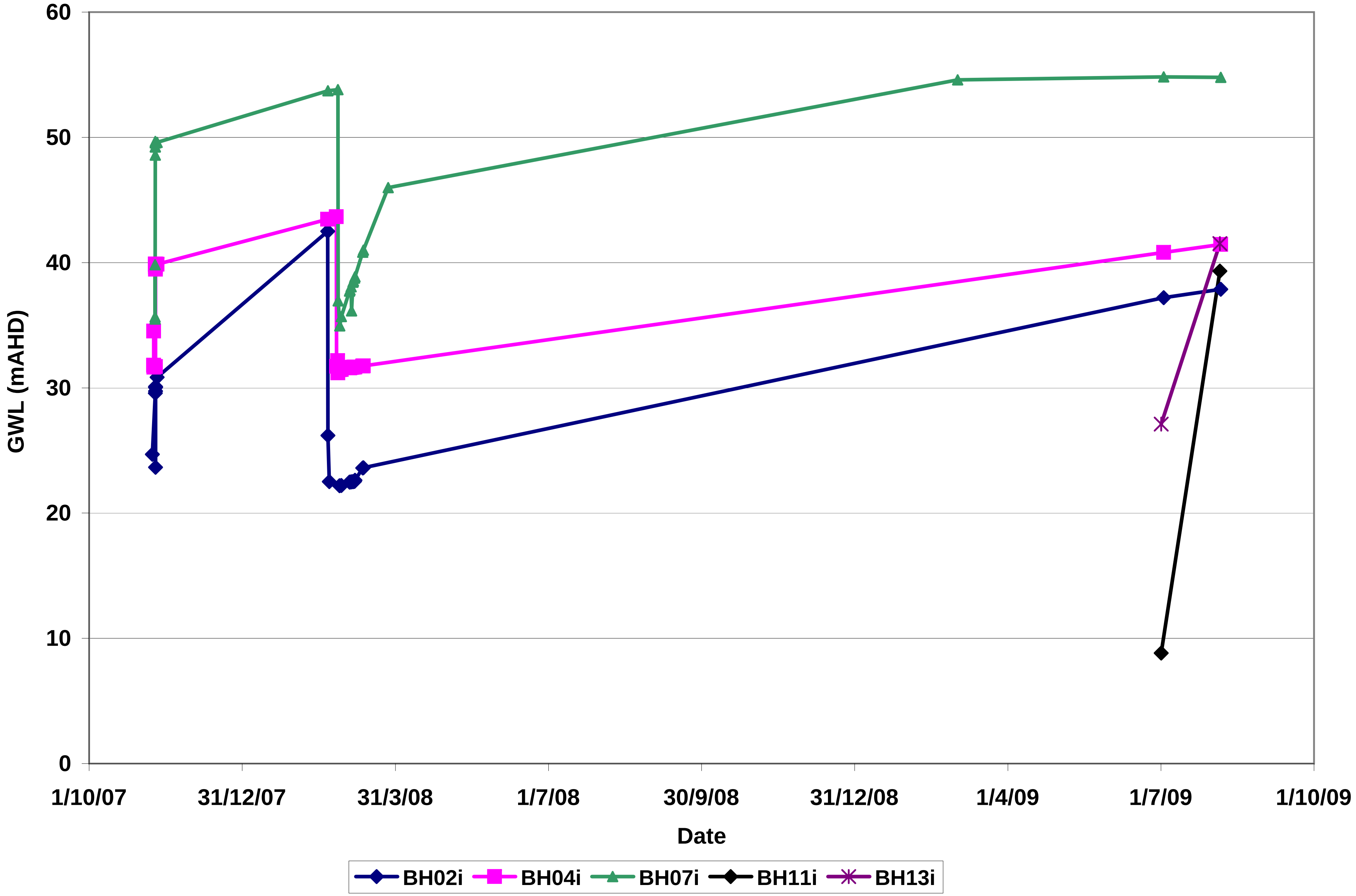

FIGURE 6.2b: Groundwater Levels - Intermediate Bores

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

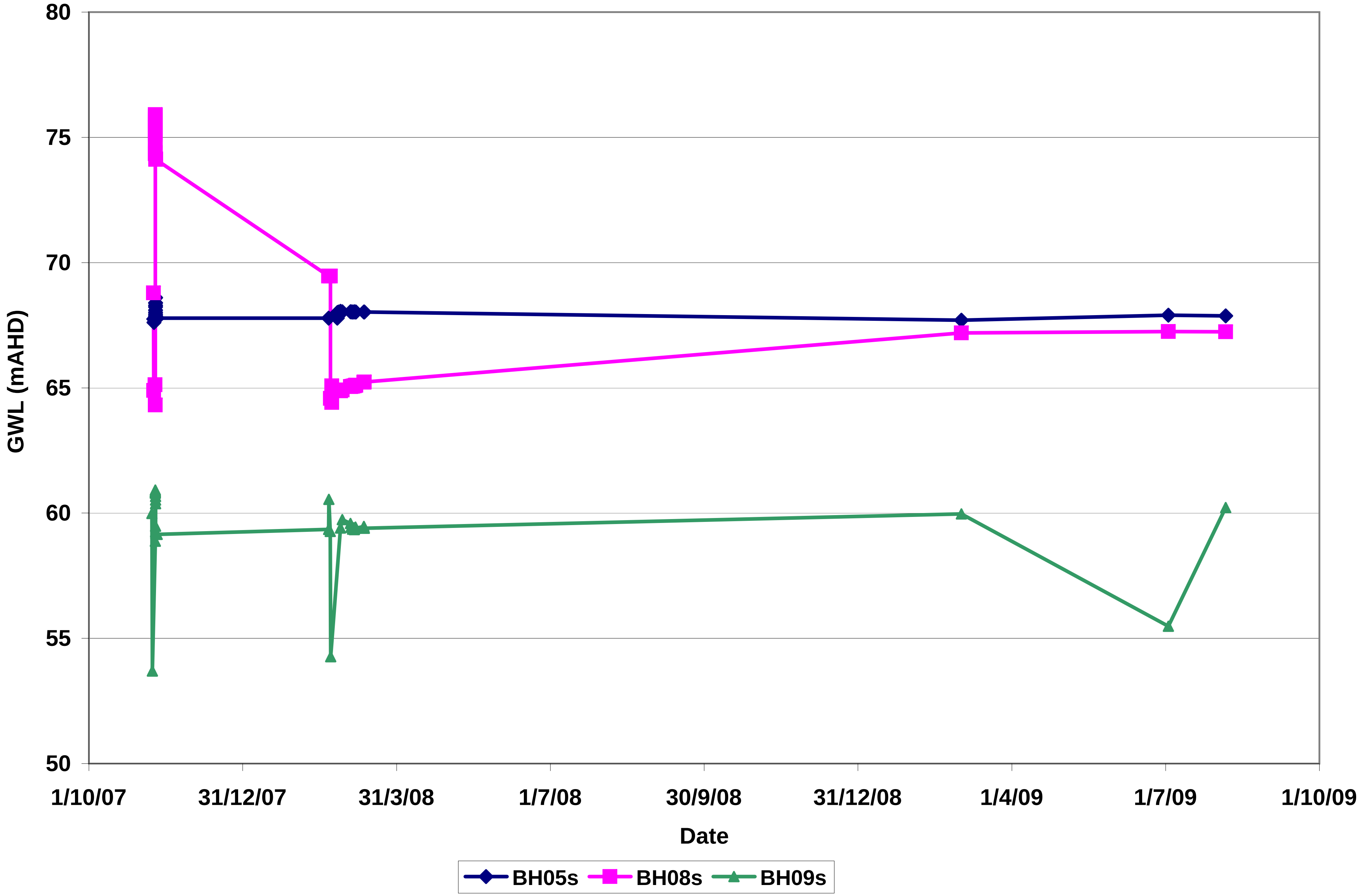

FIGURE 6.2c: Groundwater Levels - Shallow Bores

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

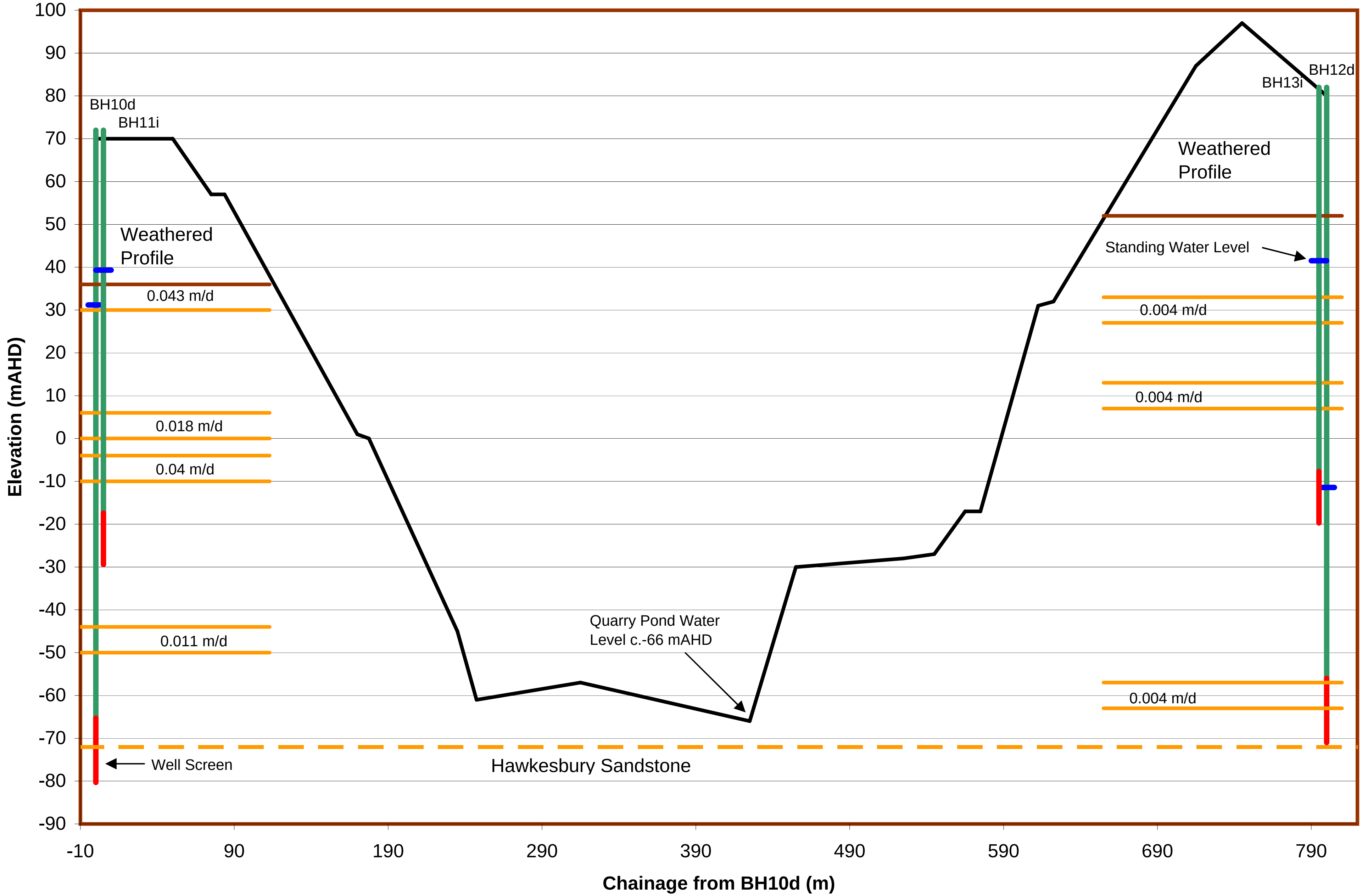

FIGURE 6.3: Detailed Hydrogeological Cross Section

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

Notes:
k values indicate more transmissive zones
Inspection of quarry walls does not provide any evidence of these zones being laterally extensive

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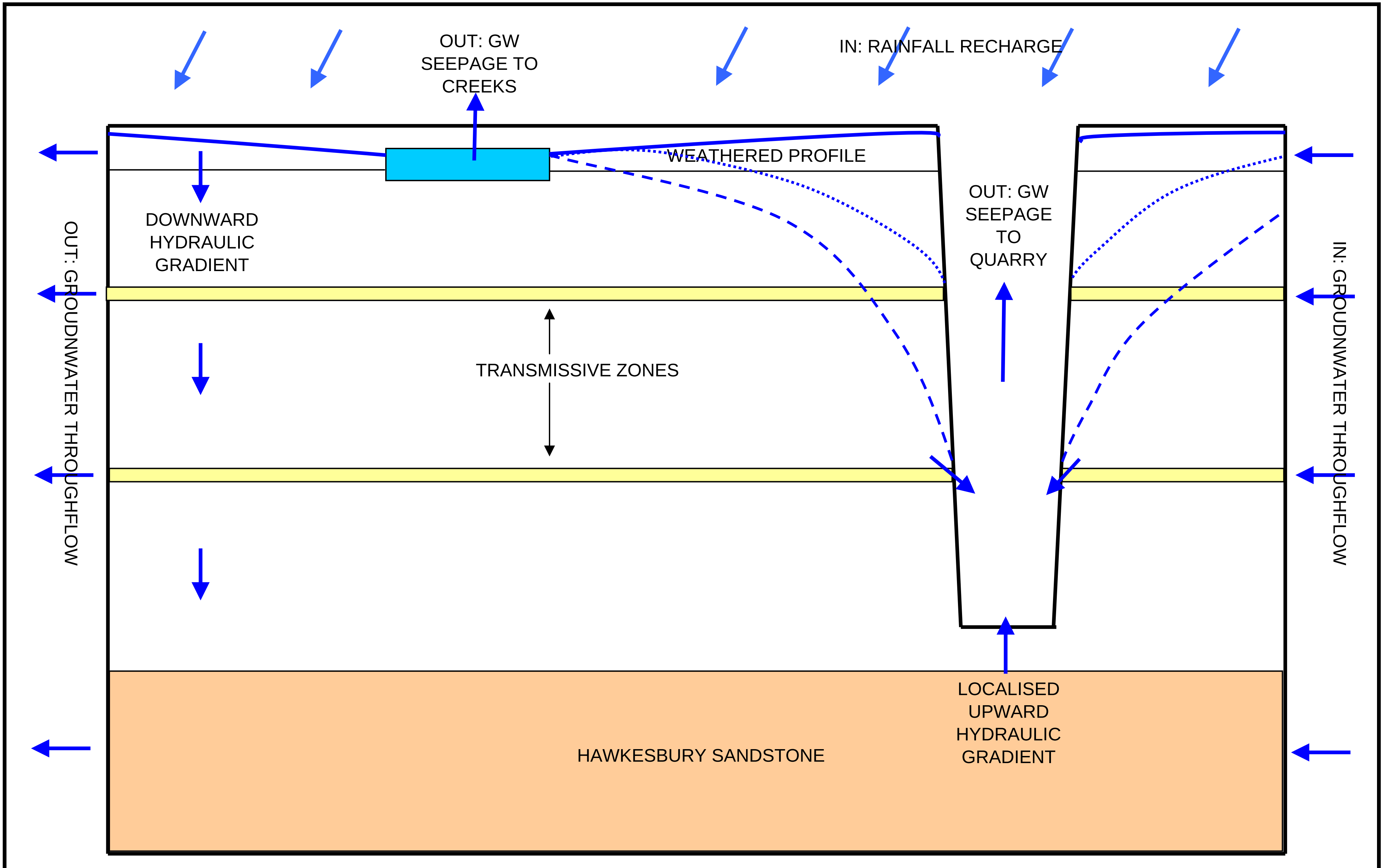

FIGURE 7.1: Conceptual Hydraulic Model

Project:	Detailed Hydrogeological Investigation and Assessment
Location:	Proposed Light Horse Landfill Site, Eastern Creek
Client:	Dial A Dump Industries Pty Ltd
Project No:	BJ07

Groundwater Level: Shallow System
Groundwater Pressure Surfaces
- Deeper Systems

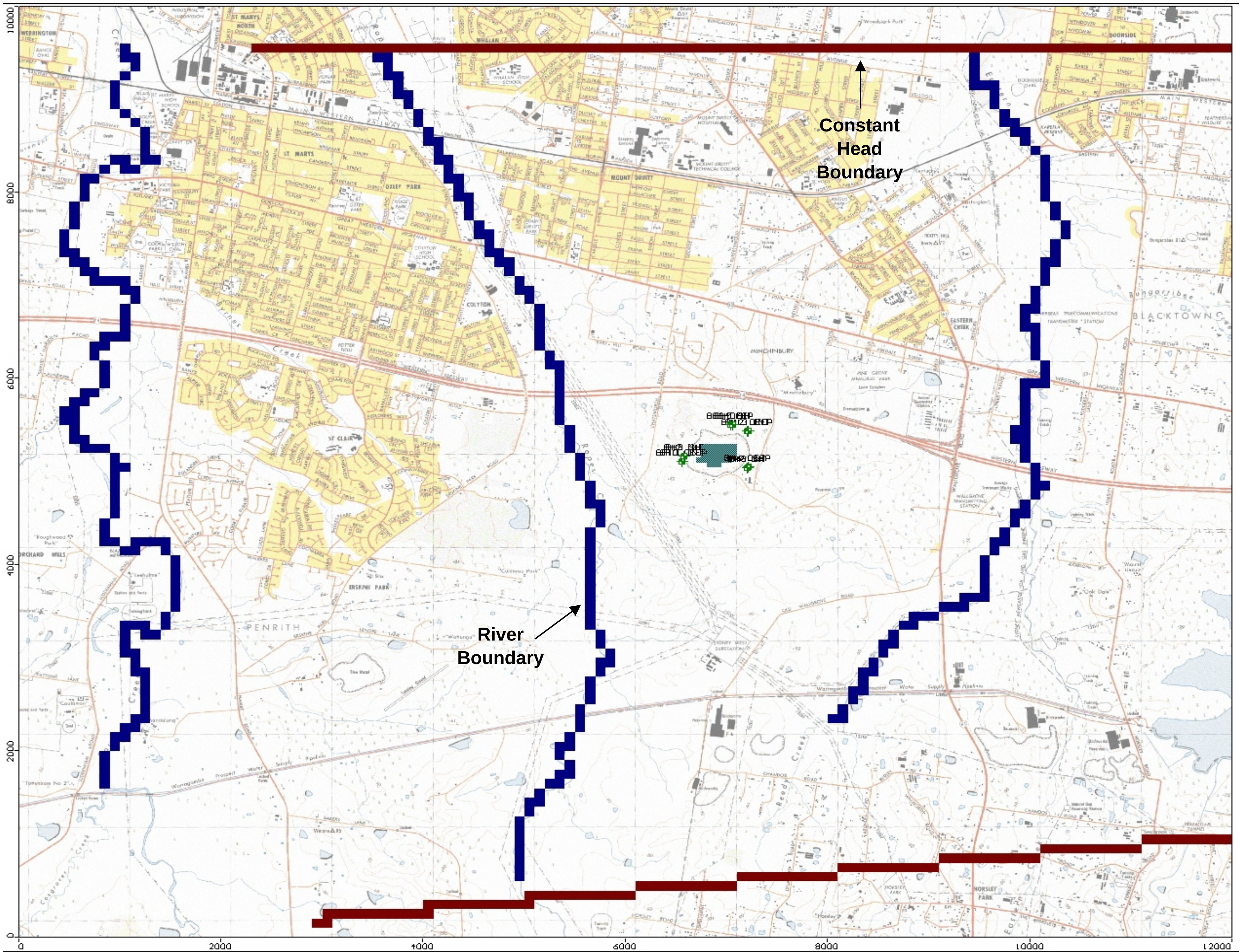

FIGURE 7.2: Model Extent and Boundaries

Project: Detailed Hydrogeological Investigation and Assessment

Location: Proposed Light Horse Landfill Site, Eastern Creek

Client: Dial A Dump Industries Pty Ltd

Project No: BJ07

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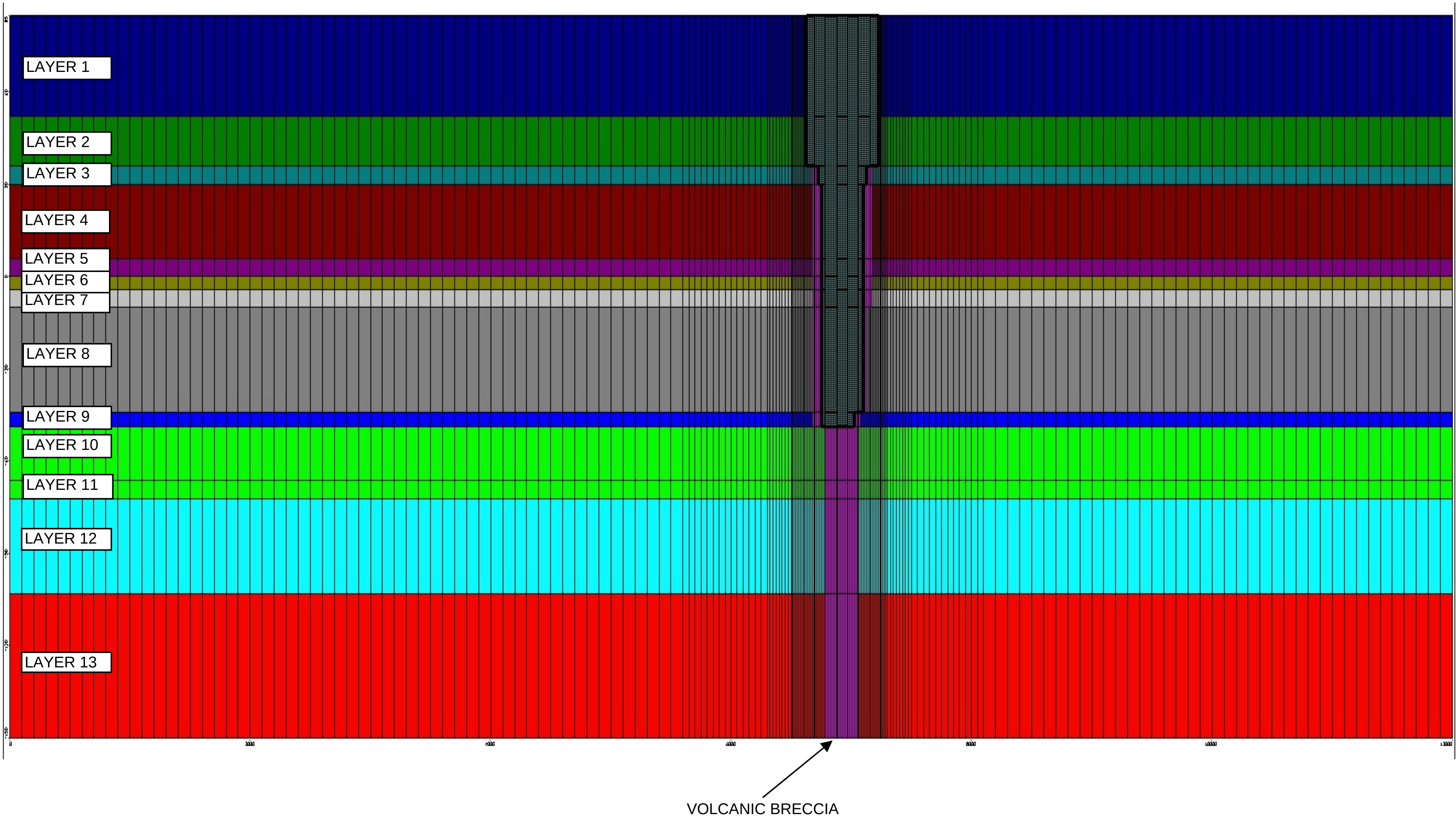

FIGURE 7.3: Model Geological Profile

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

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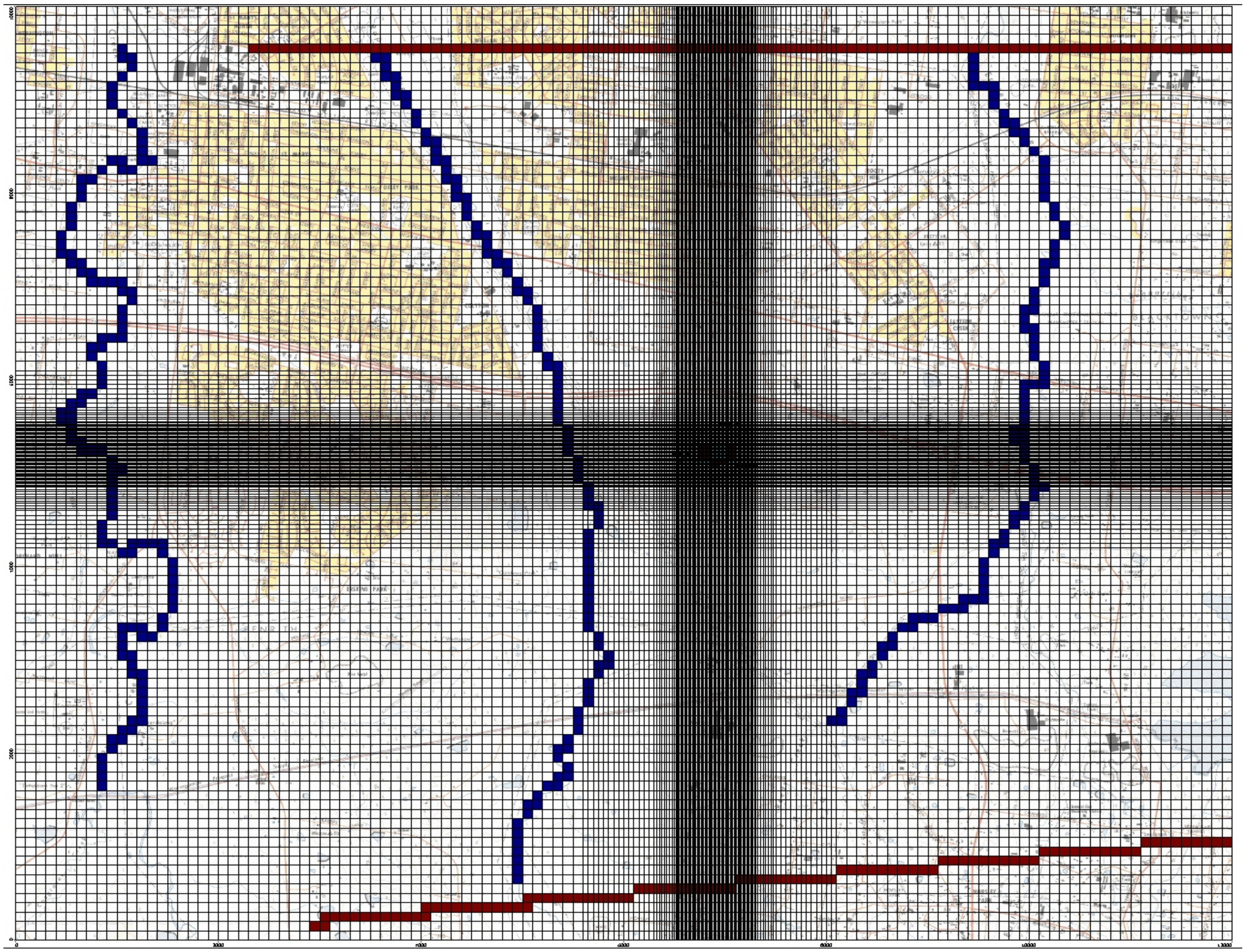


FIGURE 7.4a: Model Grid - Entire Model

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

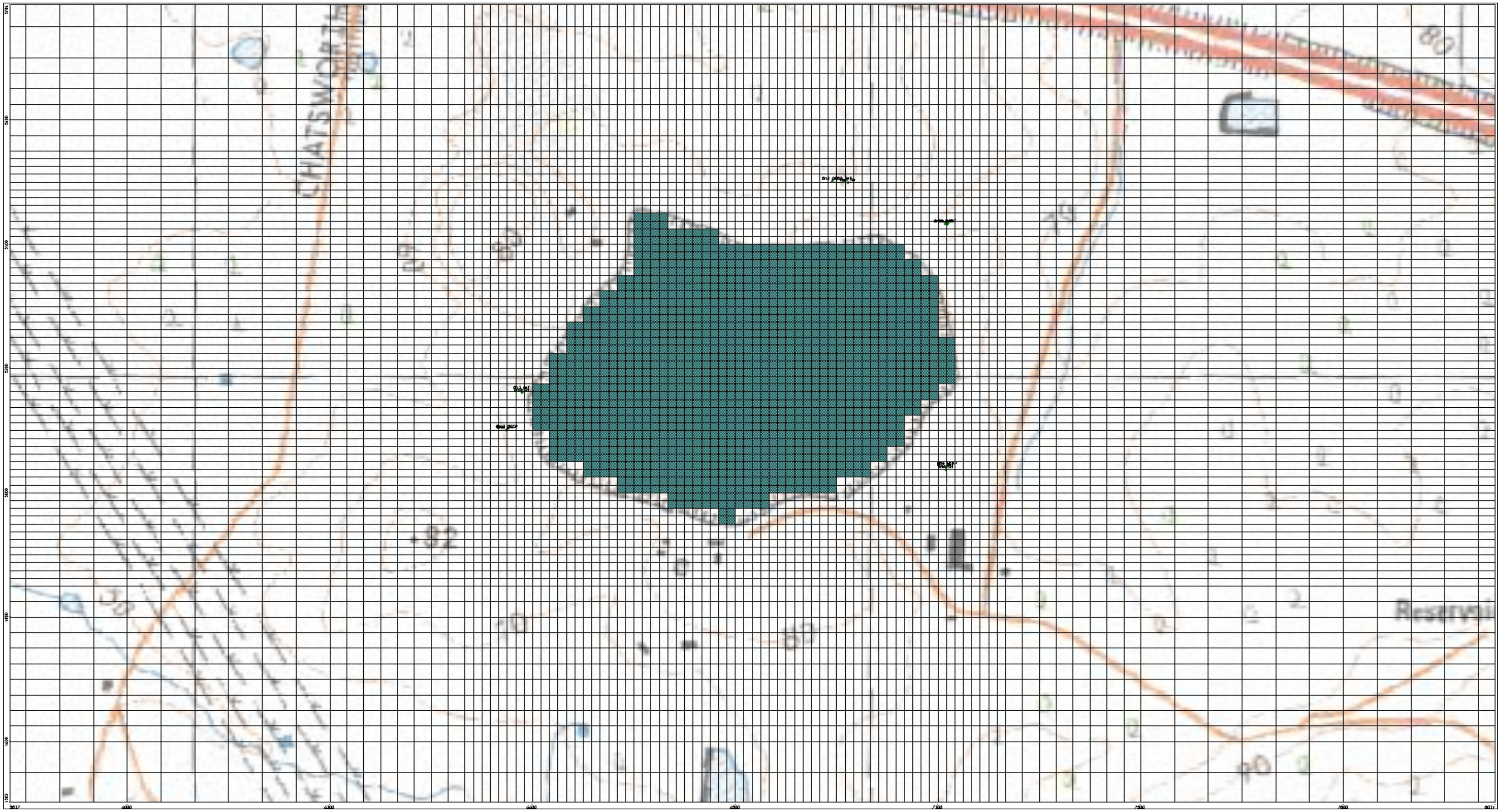


FIGURE 7.4b: Model Grid - Quarry Area

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

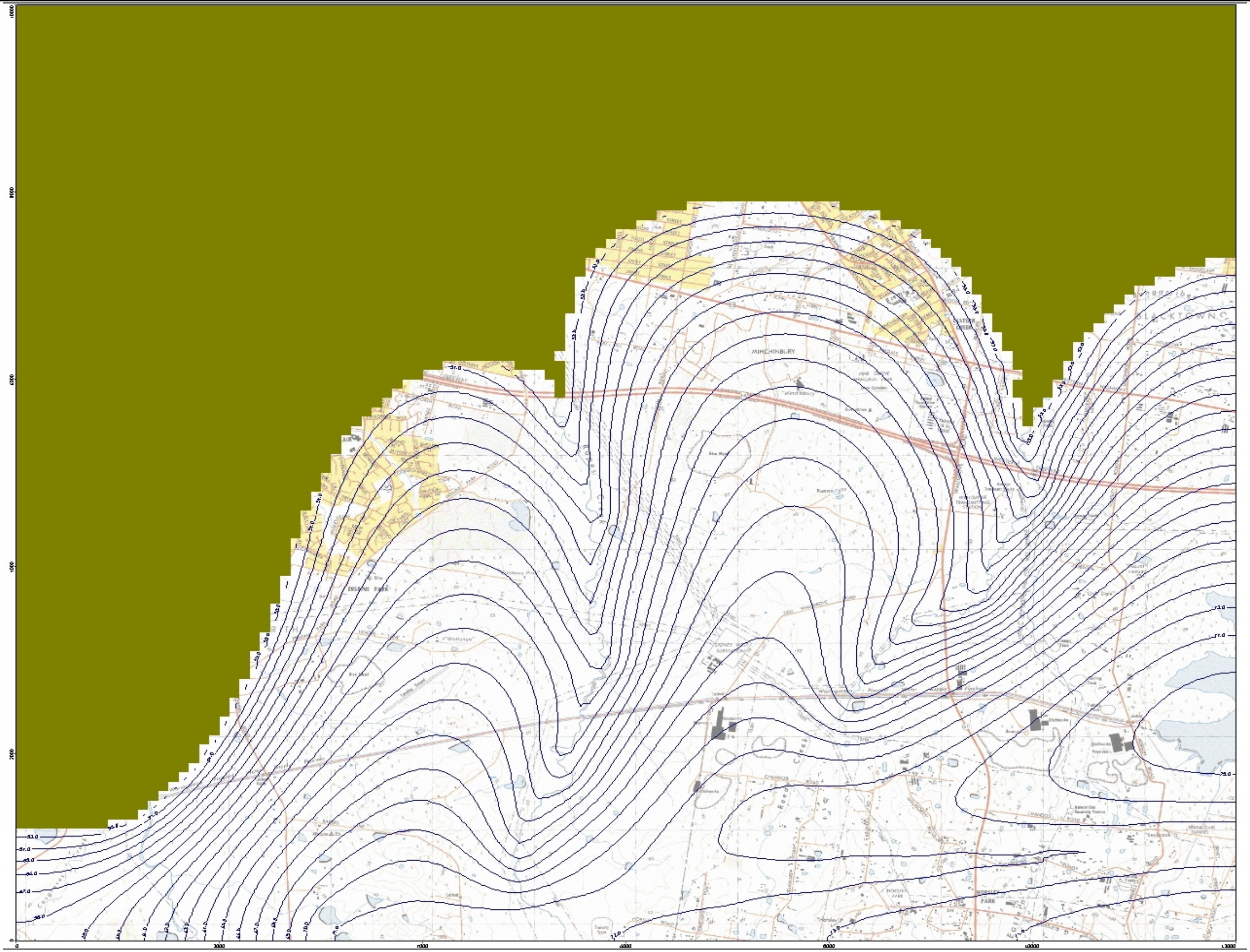


FIGURE 7.5: Simulated Shallow Groundwater Contours, Pre-Quarry Conditions (1 m Intervals)

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

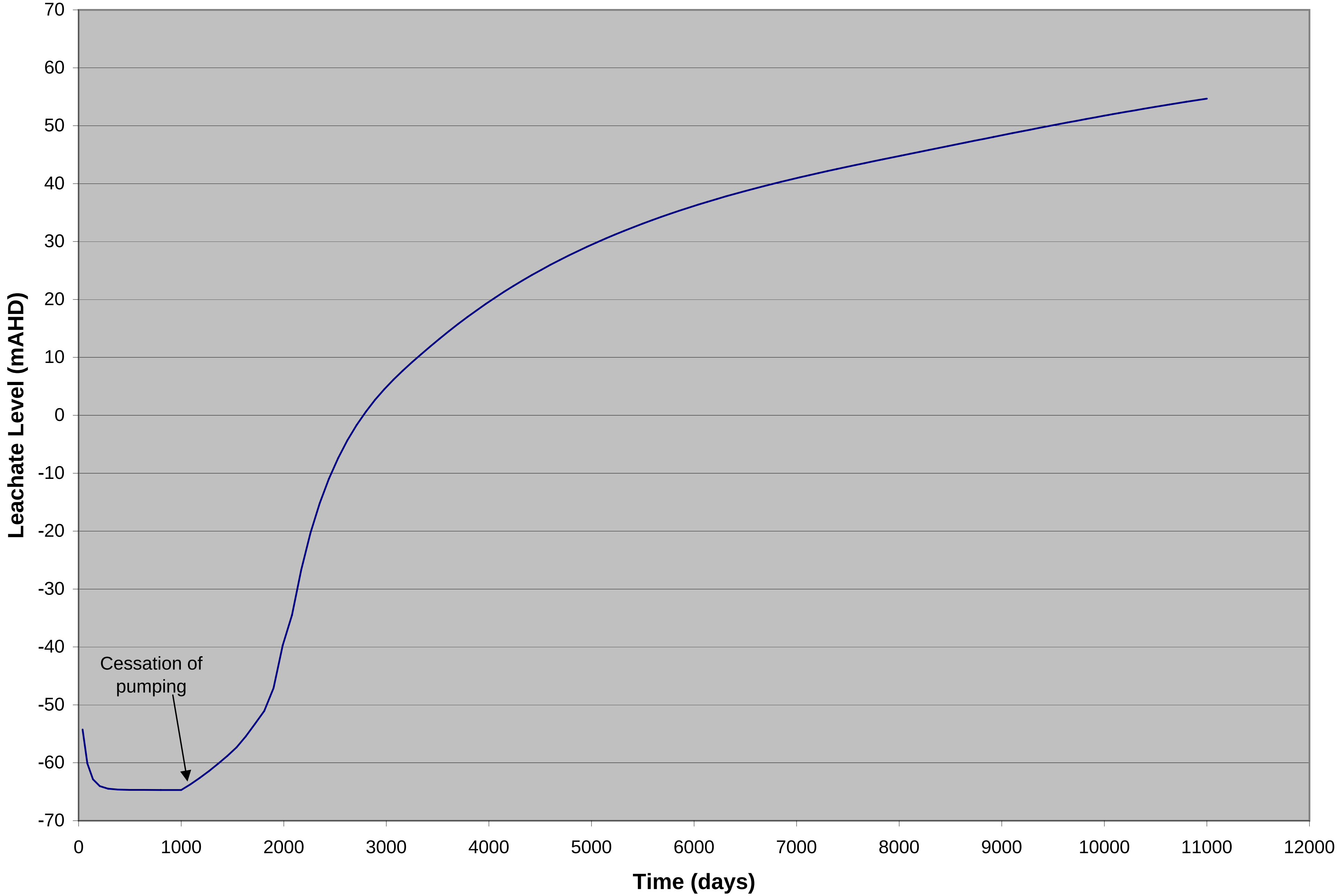


FIGURE 7.6: Predicted Leachate Level Rise, No Pumping

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

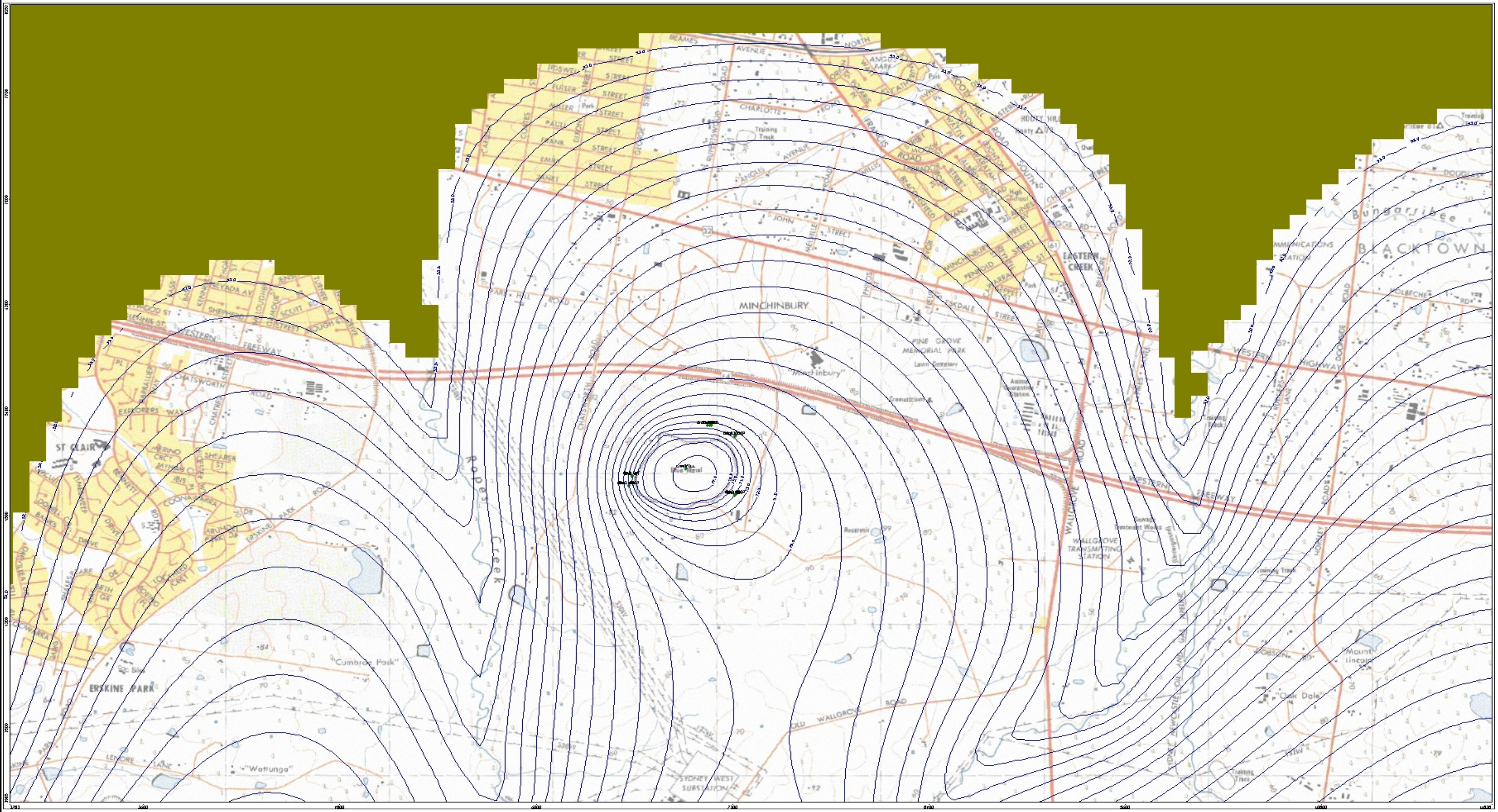


FIGURE 7.7: Predicted Groundwater Contours, Final Conditions (Full Repressurisation - 1m Intervals)

Project: Detailed Hydrogeological Investigation and Assessment

Location: Proposed Light Horse Landfill Site, Eastern Creek

Client: Dial A Dump Industries Pty Ltd

Project No: BJ07

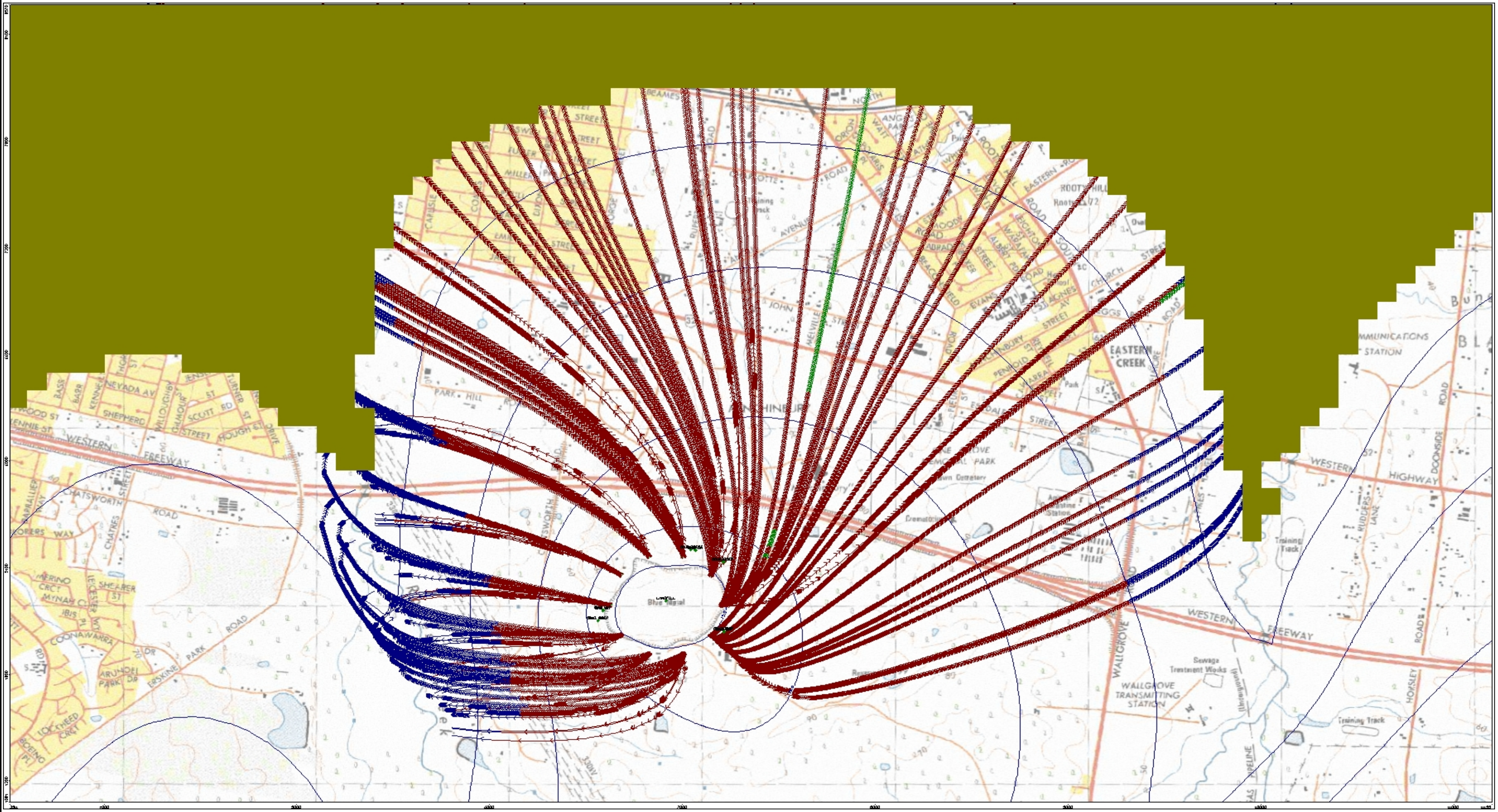


FIGURE 7.8: Particle Tracking Results, Full Repressurisation

Project: Detailed Hydrogeological Investigation and Assessment
Location: Proposed Light Horse Landfill Site, Eastern Creek
Client: Dial A Dump Industries Pty Ltd
Project No: BJ07

IGGC

Appendix A
J&K Fracture Mapping Report

Jeffery and Katauskas Pty Ltd

CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
ABN 17 003 550 801

PO BOX 976, NORTH RYDE BC NSW 1670
Tel: 02 9888 5000 • Fax: 02 9888 5003
Email: engineers@jkgroup.net.au

27 April 2009
Ref:18724ZR4let

Alexandria Landfill Pty Ltd
PO BOX 1040
MASCOT NSW 1460

ATTENTION: Mr Christopher Biggs

Dear Sir

MAPPING OF EXISTING FRACTURES WITHIN QUARRY ROCK FACES
EXISTING QUARRY, ARCHBOLD ROAD, EASTERN CREEK

1. INTRODUCTION

Acting on the commission received from James Duchenne (email dated 17 February 2009) the undersigned visited the above site on 24 March 2009 with Ian Grey (Ian Grey Groundwater Consulting Pty Ltd (IGGC)), James Duchenne (Dial A Dump Industries) and a representative of Crux Surveying Pty Ltd (Crux) also in attendance. The purpose of the site visit was to identify fractures within the rock mass exposed in the quarry faces, record their orientation, dip direction, continuity and extent, and to assess the current seepage rates from the fractures.

We have been provided with the following information:

- A copy of a report entitled 'Archbold Road, Eastern Creek: Groundwater and Salinity Assessment for Proposed Quarry Rehabilitation Project and Developable Land' (Ref. BJ07/Rp010 Rev E, dated May 2007) prepared by IGGC.
- A copy of a report entitled 'Light Horse Business Centre, Eastern Creek, NSW, Australia: Groundwater Assessment' (Ref. 0071234RP-FINAL, dated August 2008) prepared by ERM.

The purpose of this letter is to summarise the results of our mapping of fractures within the existing quarry slopes and faces and to provide additional commentary on the recorded fracture pattern within the quarry in relation to the geological structure within and surrounding the site.

2. ASSESSMENT PROCEDURE

An Associate level engineering geologist carried out a geotechnical mapping survey of defects (fractures) within the quarry, in conjunction with Ian Grey (IGGC). During the site visit, the defect locations were marked up for subsequent location by the surveyor. Crux located the identified defects using optical surveying techniques on 2 April 2009 and presented them on a plan (Drawing Number 81212AAH-1 Issue 3) dated 16 April 2009. In addition Crux provided a composite survey plan (Drawing Number 81212AAH-2 Issue 3) dated 16 April 2009 which included current survey data and previous survey data provided to Crux by DADI. The defect locations are presented on the attached Figure 1.

The defects described in Section 3 below, and indicated on the attached Figure 1, have been measured by hand held inclinometer and tape measure techniques and hence are only approximate. In addition, some features were located over the upper portions of quarry faces and so lengths and heights of such defects were estimated from the quarry floor or the haul road. The location recorded by Crux was either the defect itself or a location transferred down to a readily accessible location on the quarry floor or haul road.

3. DESCRIPTION OF DEFECTS

A summary table of defect descriptions is presented in the attached Table A. With regard to the information presented in Table A we note the following:

- The description of the defects is based on AS 1726-1993 'Geotechnical Site Investigations'.

- With regard to the reported estimated seepage rates there is no industry standard for presentation of such information. Typically, information regarding seepage rates and their descriptors is project specific and we have adopted a similar approach; our estimation of seepage rates is presented in the notes at the foot of the table. In the field, the seepage rate was visually assessed as often the rate of seepage and volume of water was too low to accurately record and/or was issuing from a defect that was inaccessible.

4. COMMENTS ON RECORDED DEFECTS

4.1. Geological Setting

The 1:100,000 Geological Map of Penrith, Sheet 9030 and the accompanying manuscript published by the Geological Survey of New South Wales Department of Minerals and Energy (dated 1991) indicates that that the quarry has exploited an igneous diatreme (Number Jv17 – the Minchinbury Diatreme)

A diatreme represents an explosive intrusive geological event where a column of molten igneous rock was injected into country rock (at the subject site sub-horizontally bedded Bringelly Shale generally consisting of an interbedded sequence of shales and sandstone) and fed a volcano at the surface. In this instance, the diatreme is characterised by a volcanic breccia comprising fine grained basaltic tuff with large inclusions of basaltic rock and country rock with an approximately ovoid shape (in plan).

The volcano has long since been eroded and the only recent geomorphologic expression of the volcano was the hill which formed the Minchinbury Trig. Station prior to quarry activities commencing at the site.

Research into the relevant published geological data for the site indicates that:

- Previous studies have identified vertically bedded Bringelly Shale 'dragged down a ring fault surrounding the diatreme'.
- The igneous bodies within and surrounding the site occur within a 20km wide zone trending approximately east-west and coincides with the Lachlan River Lineament which the Jurassic age igneous activity appears to have used as a pathway to the surface.

4.2. Pattern of Defects

The geological setting of the site would suggest that defects recorded within the quarry follow a pattern orientated approximately parallel to the perimeter of the diatreme, i.e. influenced by the ring fault feature defining the diatreme perimeter.

The attached Figure 1 provides a diagrammatic representation of the orientation of defects recorded within the quarry. In general, it is clear that the majority of defects are orientated approximately parallel to the perimeter of the quarry which may reasonably be assumed to follow the margin of the diatreme. In addition, a small number of the defects appear to be orientated approximately perpendicular to the adjacent diatreme margin.

The intrusive nature of the diatreme would result in the ring fault around the diatreme cutting across any previous defects (fractures, faults etc) within the country rock. On this basis, the above described defect pattern would suggest the following:

- Defects within the site and orientated parallel to the margins of the diatreme would not extend outside of the site.
- Defects within the site and orientated perpendicular to the margins of the diatreme would be expected to connect to, and terminate at, the ring fault.
- Defects present within the country rock prior to the intrusion (i.e. outside the site) would be expected to connect to, and terminate at, the ring fault.

4.3. Influence of Defects on the Hydrogeological Regime of the Site

The rock types within and surrounding the quarry may be regarded as being of at least low permeability and in most cases 'impermeable'. The permeability of any rock mass is determined by the number, size, orientation, extent and connectivity of the defects within the rock mass.

With regard to hydrogeological regime affecting the site, the above described defect pattern would suggest that groundwater contained within the surrounding country rock would flow towards the site along the defects and be intercepted by the ring fault at the margin of the diatreme. From the margin of the diatreme, groundwater seepage into the quarry would only occur along defect planes where there was connectivity with the ring fault.

The ERM report has assessed the site to be affected by a 'steep regional groundwater gradient'. If there were substantial connectivity between the surrounding groundwater regime and the quarry defects then the steep nature of the groundwater gradient would be expected to manifest itself as discrete areas of concentrated and high volume groundwater seepage at specific defect planes. No such evidence was apparent during the latest site visit or the previous site visits the undersigned has carried out for the purposes of assessment of the stability of quarry slopes, completed on 20 August 2004, 7 December 2005, 20 and 26 February 2008 and 2 March 2009.

We also note that the assessed seepage rates recorded along discrete defect planes within the quarry were generally of low volumes; typically at or below 0.1litres/sec and rarely approaching 1 litre/sec. Further, the fieldwork was completed after a period of intermittent and occasionally heavy rainfall that had lasted for over a week.

5. GENERAL COMMENTS

It is possible that the subsurface soil, rock or groundwater conditions encountered during construction may be found to be different (or may be interpreted to be different) from those inferred from our surface observations in preparing this report. Also, we have not had the opportunity to observe surface run-off patterns during heavy rainfall and cannot comment directly on this aspect. If conditions appear to be at variance or cause concern for any reason, then we recommend that you immediately contact this office.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. Copyright in this report is the property of Jeffery and Katauskas Pty Ltd. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

Should you have any queries regarding this report, please do not hesitate to contact the undersigned.

Yours faithfully
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD

Paul Roberts
Associate

Agi Zenon
Senior Associate

Attachments

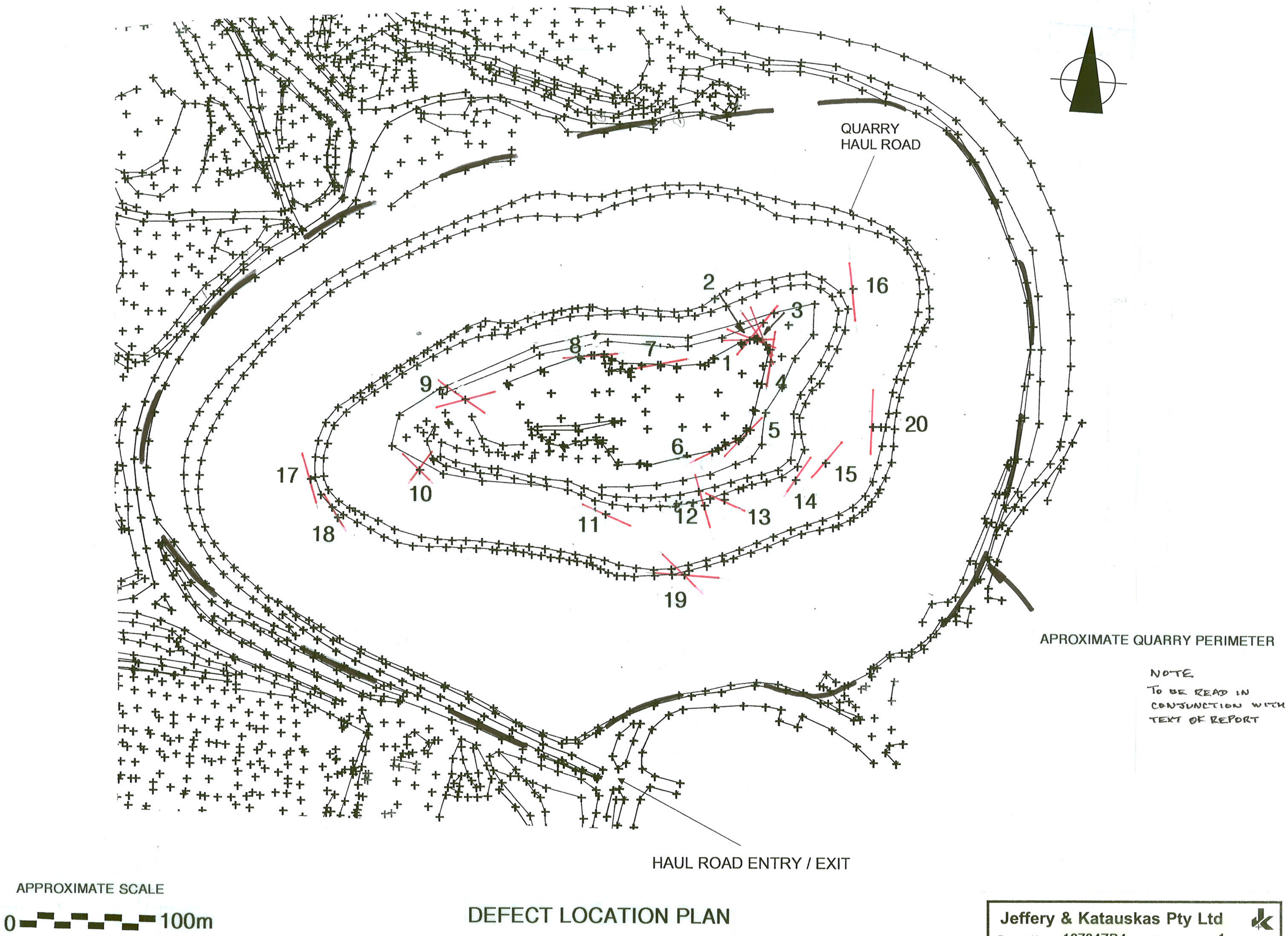
TABLE A: SUMMARY OF RECORDED DEFECTS

FIGURE 1: DEFECT LOCATION PLAN

TABLE A SUMMARY OF RECORDED DEFECTS

DEFECT LOCATION	ORIENTATION AND DIP DIRECTION	DESCRIPTION	SEEPAGE
1	035°. Sub-vertical	Un, S. Extends for a vertical height of 8m from the base of the quarry	Slight seepage from 4m above the quarry base.
2	265°. Dip maximum 70° down to 175° 105°. Dip 25° down to 195°	Un, S, sub-parallel defects spacing approx. 0.3m. Extends for a vertical height of 8m from the base of the quarry and a horizontal extent of 10m. P, S, sub-parallel defects spacing approx. 1m. Maximum lateral persistence approx. 10m	Slight seepage from 25° dipping defects and at intersection with steep defects over an area of face of approx 8m height from the quarry base and 10m width.
3	335°. Dip maximum 30° down to 245° 320°. Dip 80° down to 240°.	P, S, calcite mineralisation on defect plane. Located 8m above quarry face. Maximum lateral persistence approx. 5m. P, S. Extends for a vertical height of 8m from the base of the quarry.	Slight seepage from 30° dipping defect discharging down fractured quarry face.
4	005°. Dip 45° down to 275°.	P, S, calcite mineralisation on defect plane. Located approx. 1m above quarry face and maximum lateral persistence approx. 15m.	Intermittent slight seepage along defect plane.
5	040°. Dip 50° down to 310°.	Un, S. Located 6m above quarry base and extends for a horizontal distance of approx. 5m.	No seepage evident.
6	060°. Dip 80° down to 330°. 060°. Dip 10° down to 330°.	P, S. Defects intersect approx. 20m above quarry base and extend for horizontal distance of approx. 25m.	No seepage evident.
7	075°. Dip 60° down to 165°.	P, S and Un, S. Located approx. 8m above quarry base and extend for horizontal distance of approx. 50.	Intermittent slight seepage over this area of the quarry face
8	260°. Dip maximum 5° down to 170°	P, S. Sub-parallel defects approx. 0.5m spacing extend over horizontal area of approx. 10m from quarry base to a height of approx. 8m above quarry base.	No seepage evident.
9	070°. Dip 48° down to 160°. 300°. Dip 35° down to 030°.	Un, S, calcite mineralisation on defect plane. Extend over 8m height up from base of quarry P, S, calcite mineralisation on defect plane. Extend over 8m height up from base of quarry.	Slight seepage.
10	130°. Dip 30° down to 040°. 030°. Dip 75° down to 120°.	P, S. Sub-parallel defects approx 0.3m to 1.0m spacing. Extends over 15m length of quarry face from base of quarry. P, S. Extend approx. 40m vertical height from base of quarry to haul road above.	No seepage evident. Intermittent slight seepage
11	110°. Dip 45° down to 020°	P, S, calcite mineralisation on defect plane. Sub-parallel defects approx. 1m horizontal spacing. Extend over approx. 50m length of face from base of haul road to approx. 4m above the haul road. Similar defects present approx. 10m to 20m above haul road	At haul road level - slight seepage down face from defect plane. 10m to 20m above haul road – strong seepage.
12	340°. Sub-vertical.	P, S, calcite mineralisation on defect plane. Sub-parallel defects approx. 2.0m horizontal located approx. 6m above haul road and extend east to defect location 15.	No seepage evident.
13	110°. Dip 40° down to 020°.	Un, S. Defect located with extremely weathered breccia zone.	No seepage evident.
14	030°. Sub-vertical.	P, S. Defect defines extremely weathered and fractured area approx. 0.5m maximum width.	Slight seepage over 15m height from base of haul road.
15	035°. Sub-vertical.	P, S. Sub-parallel defects, 0.2m to 0.3m horizontal spacing over a 10m wide zone extending approx. 10m up from haul road.	Moderate seepage.
16	350°. Dip 60° down to 260°. 350°. Dip 5° down to 080° and 260°. 055°. Sub-vertical.	P, S, calcite mineralisation on defect plane. Sub-parallel defects approx. 1m horizontal spacing. Defects maximum persistence approx. 5m. Defects intersect with sub-horizontal defect forming a fractured zone of approx. 30m width and extending a maximum vertical height of 15m above the haul road. P, S extends at least 100m to west – evident within bench face below.	Slight seepage. Dry
17	340°. Dip 80° down to 070°. 160°. Dip max. 10° down to 070°.	P, S and Un, S. Defects intersect. Steep defect extends approx. 6m above haul road. Low angle defect extends over approx. 30m horizontal distance.	Intermittent slight seepage over the extent of the low angle defect (approx. 30m horizontal distance).
18	325°. Dip 80° down to 055°.	Un, S shear zone, thinly laminated approx. 150mm maximum width with calcite veins. Extends vertical height of at least 25m from haul road, trace evident on quarry face adjacent to bench above.	No seepage evident.
19	310°. Dip 80° down to 040°. 270°. Dip 80° down to 360°.	P, S and Un, S. Zone approx. 15m wide and extends approx. 8m height above haul road and comprises extremely weathered breccia.	Strong seepage.
20	360°. Dip 30° to 45° down to 270°.	Un, S. Major defect plane over upper portion of eastern quarry face, extends over approx. 60m vertical height and similar horizontal distance.	No seepage evident.

NOTES
Defect Descriptions: P: planar. Un: undulating. S: smooth
Estimated Seepage Rates: Slight: max. 0.01 litres/sec. Moderate: >0.01 to 0.1 litres/sec. Strong: >0.1 to 1 litres/sec.



DEFECT LOCATION PLAN

Jeffery & Katauskas Pty Ltd
Report No. 18724ZR4 Figure No. 1

Appendix B
Core Photographs

Core Hole 1 31-38 m core intervals

Core Hole 1 38-46 m core interval

Core Hole 1 46-54 m core interval

Core Hole 1 54-62 m core interval

Core Hole 1 62-66 m core interval

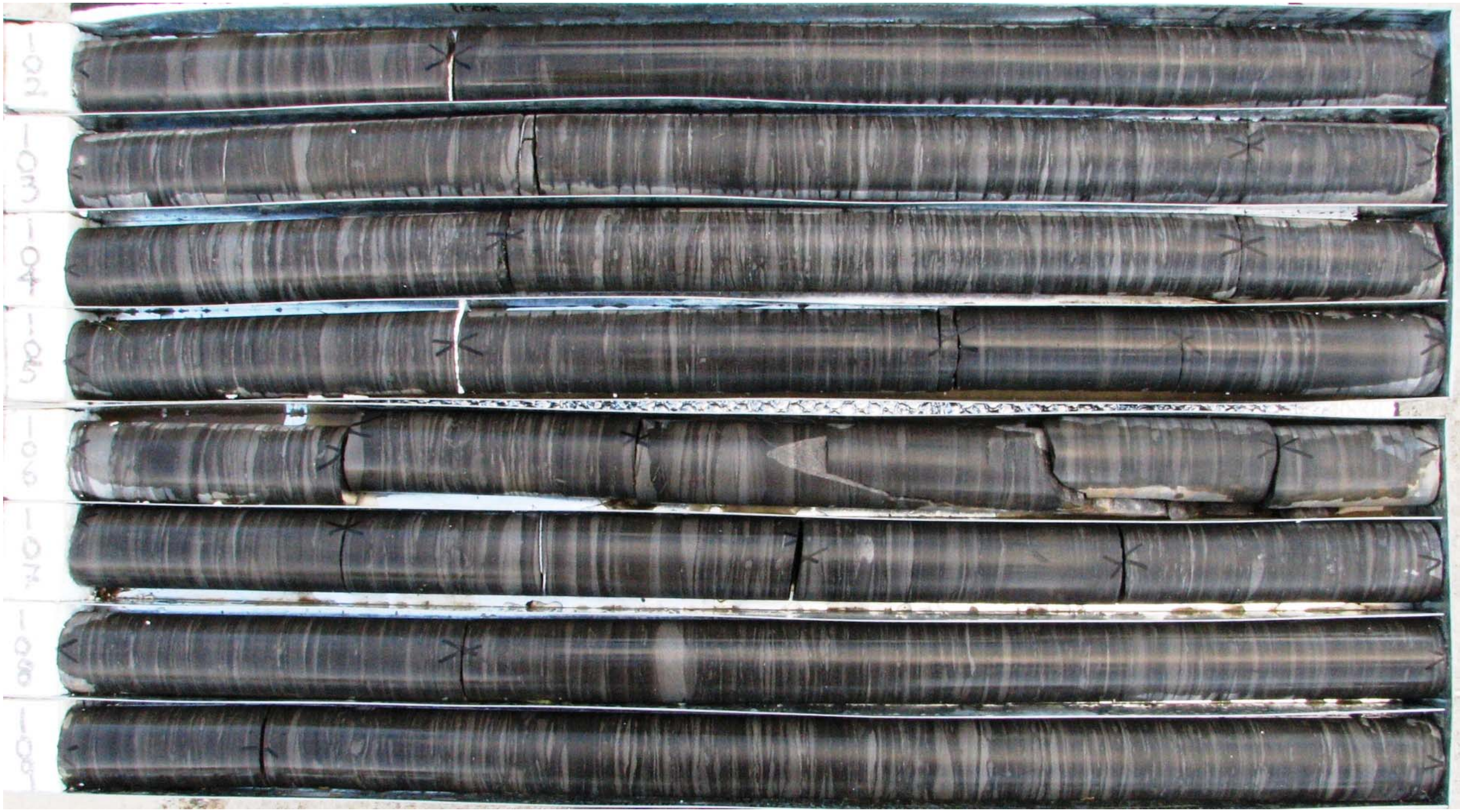
Core Hole 1 66-74 m core interval

Core Hole 1 74-78 m core interval

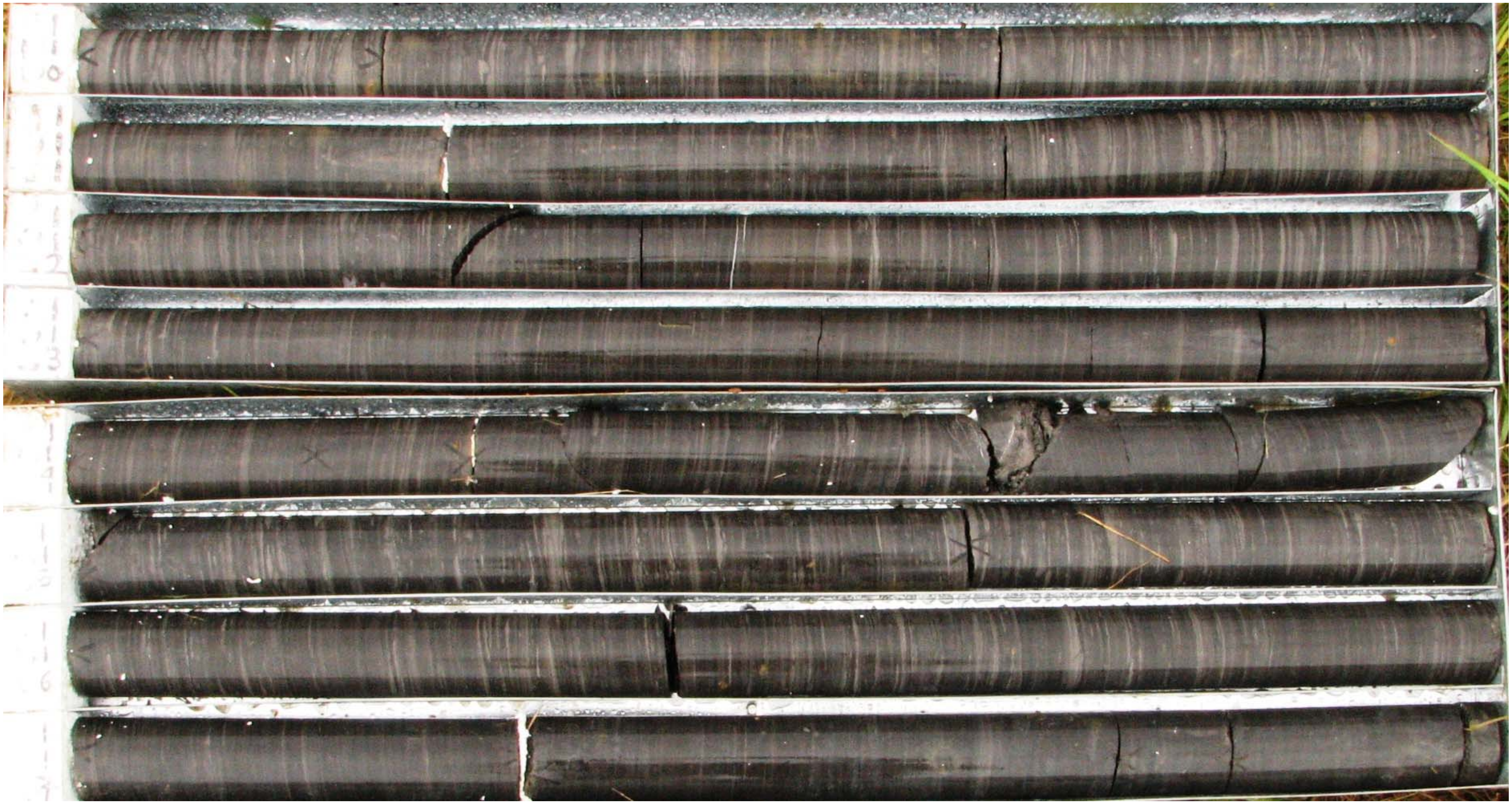
Core Hole 1 78-86 m core interval

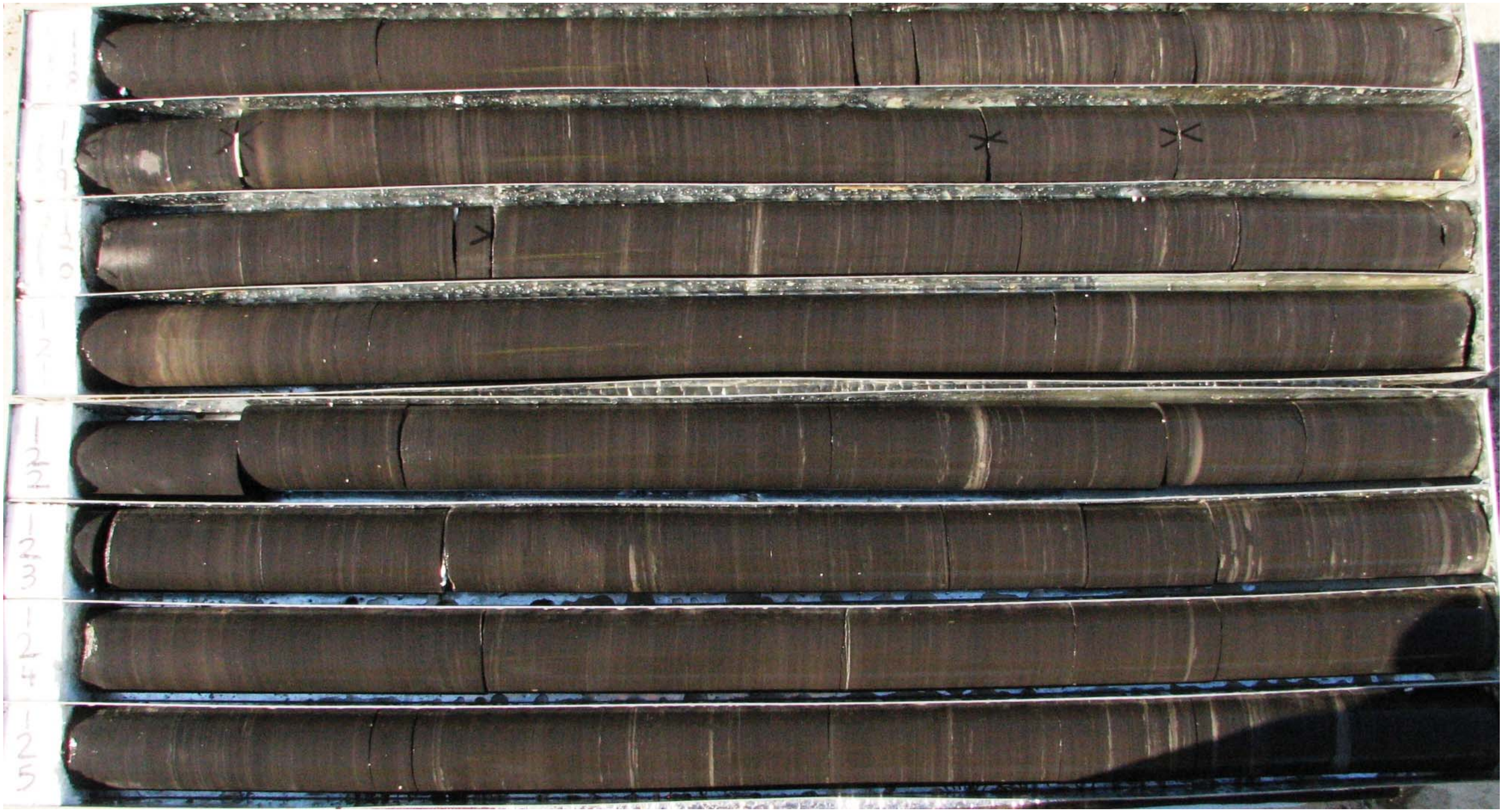
Core Hole 1 86-94 m core interval. Note the increase in lamination down hole and transition from siltstone to shale.

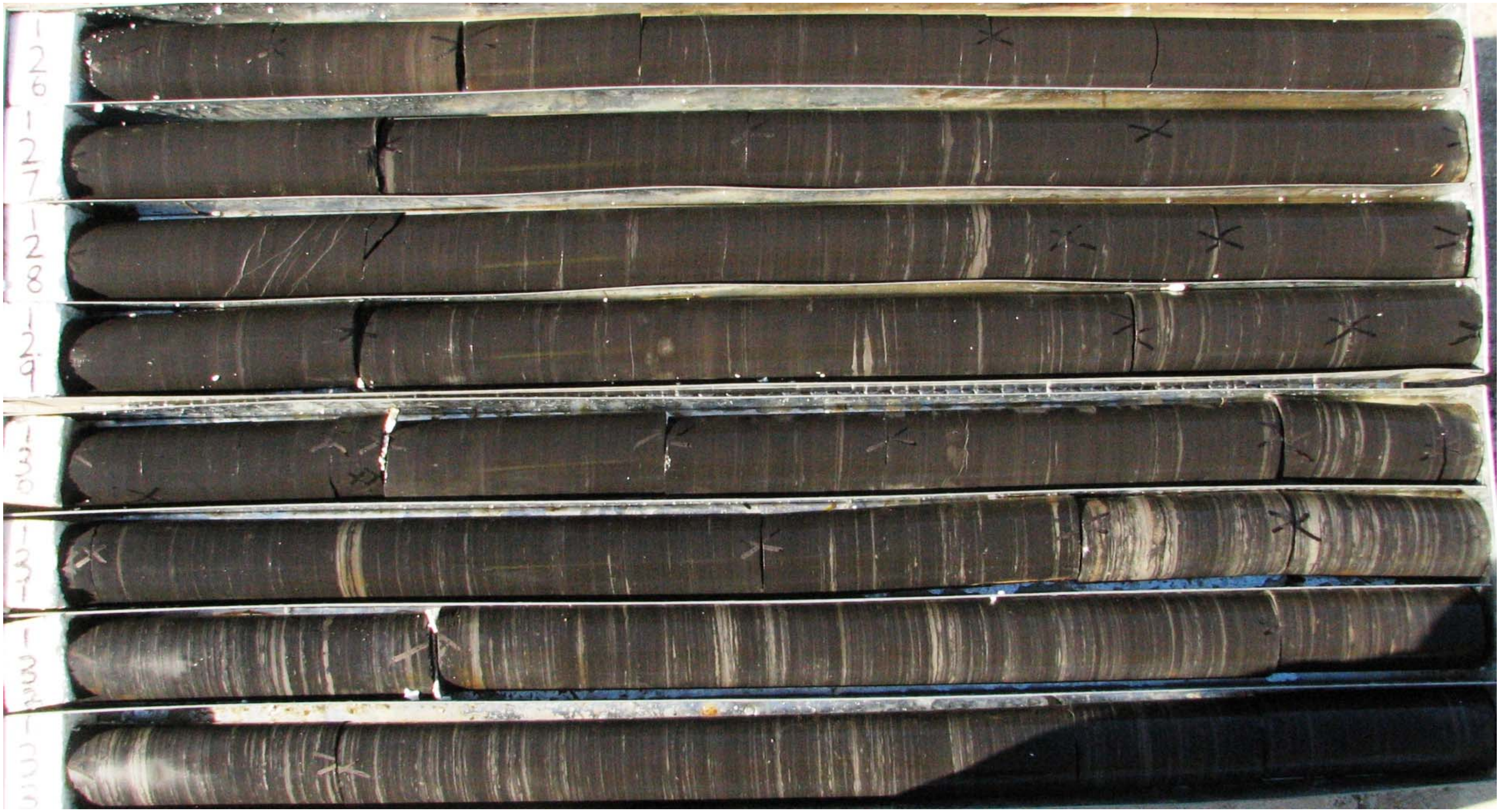
Core Hole 1 94-102 m core interval. The shale is separated by the sandstone unit ~94-96 m.

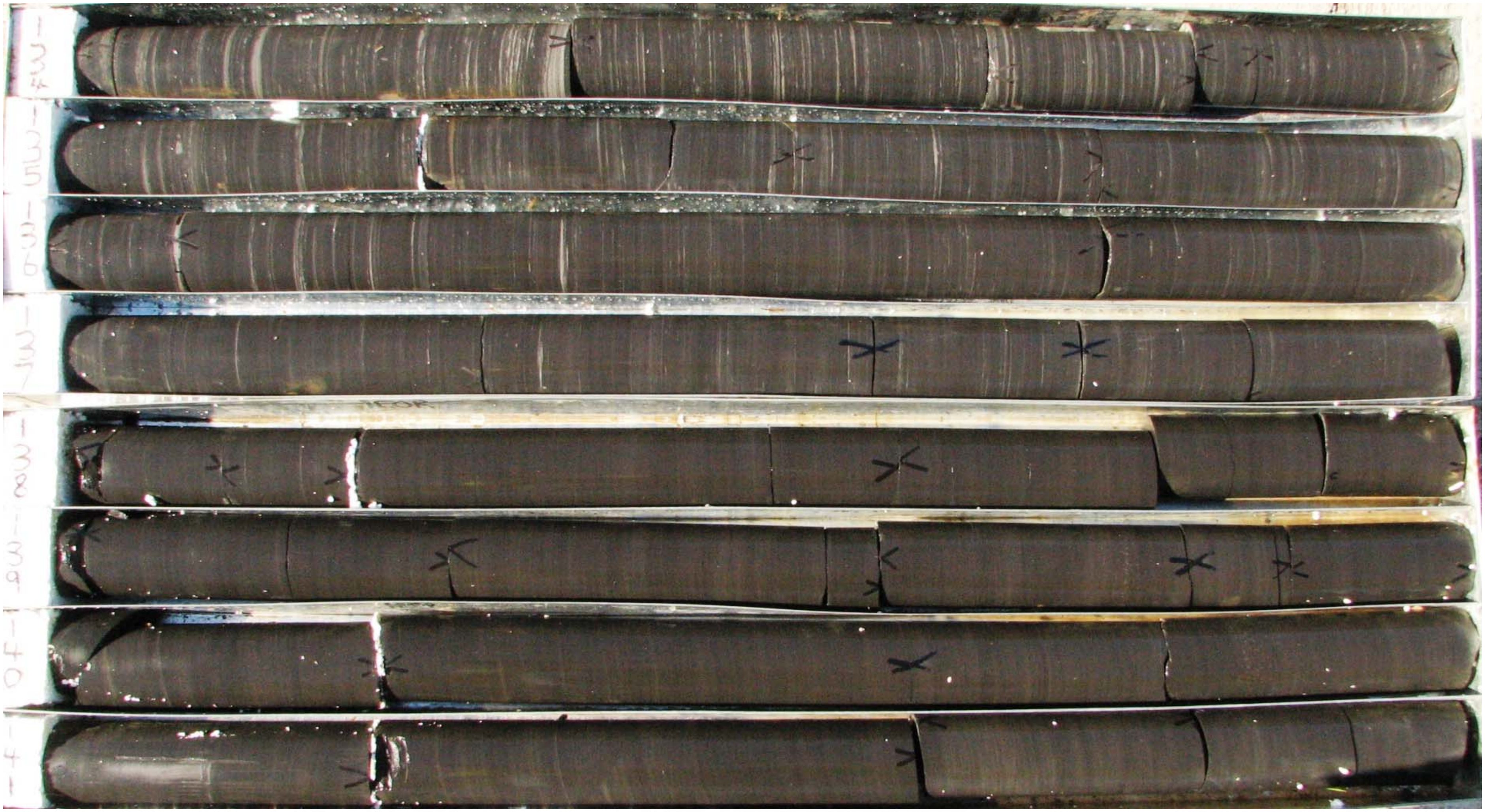


Core Hole 1 102-110 m core interval. Carbonaceous shale, continuing down hole.


Core Hole 1 110-118 m core interval. Carbonaceous shale.


Core Hole 1 118-126 m core interval. Carbonaceous shale.


Core Hole 1 126-134 m core interval. Carbonaceous shale.



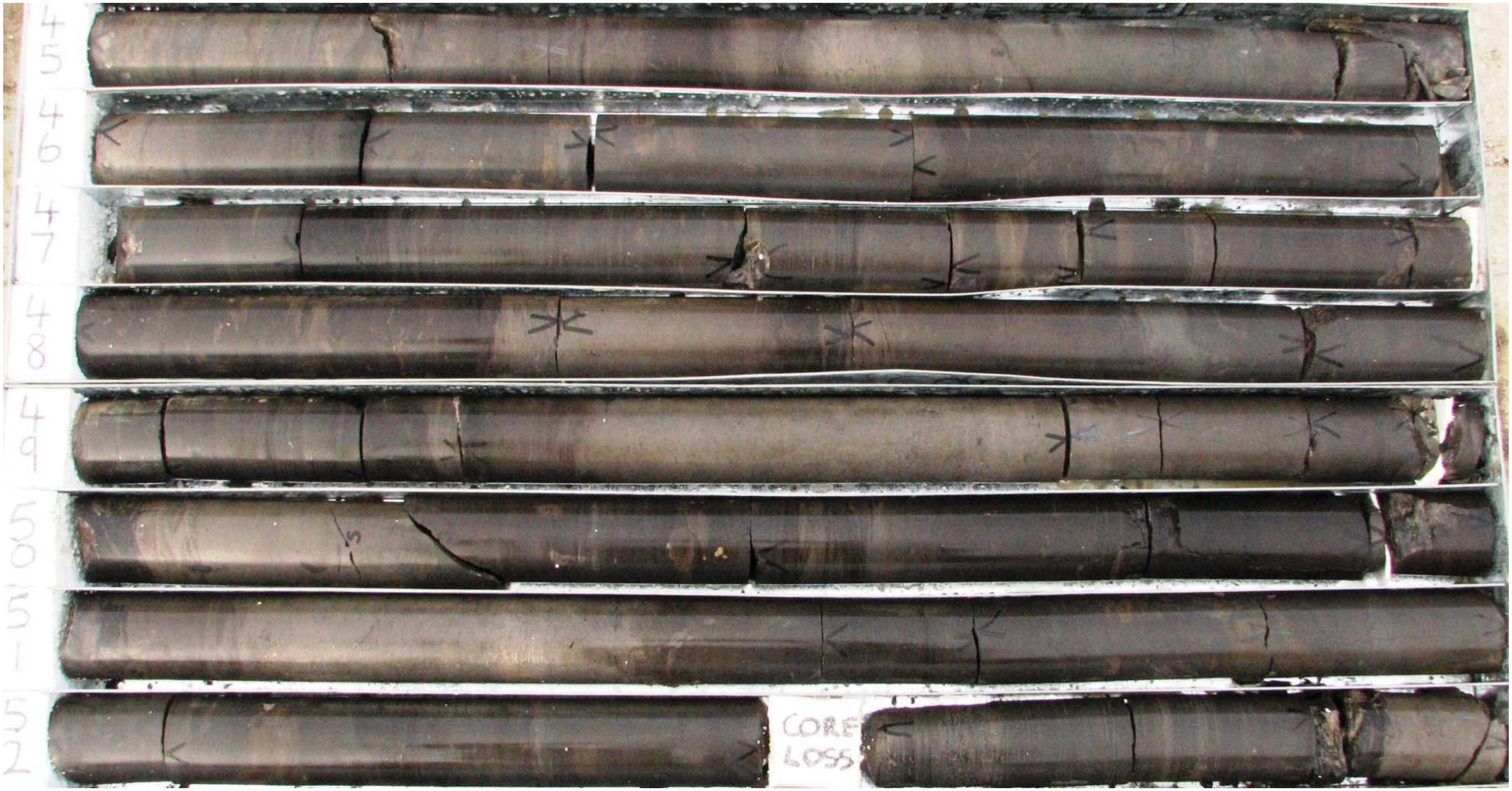
Core Hole 1 134-142 m core interval. Carbonaceous shale.

Core Hole 1 142-150 m (End of Hole) core interval. Carbonaceous shale and sharp contact with underlying coarse sandstone.

CORE HOLE 2

Core 2 30-37 m. Note that this hole starts coring in siltstone, intersecting the sandstone at 34.8 m, corresponding to the sandstone intersected in Core 1 from 31 m.

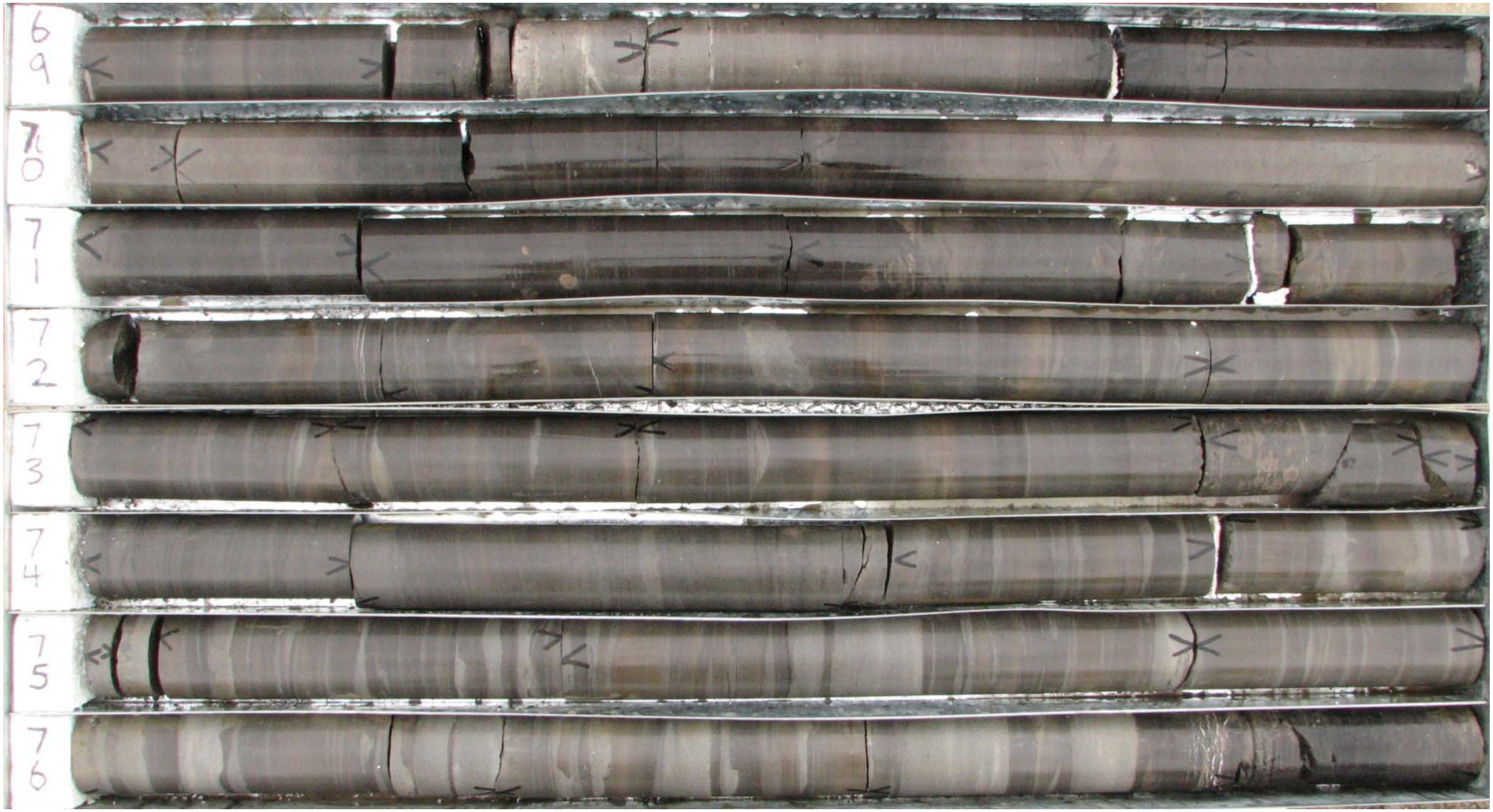
Core2 37-44 m Sandstone that undergoes a transition downhole to sandstone interbedded with siltstone by 39.26 m. The fine sandstone unit is thinner than that observed at the top of Core1. Note the interval of conglomerate at 43.95-44.56 m.



Core 2 45-53 m. Note the dark carbonaceous nature of much of the siltstone. Several fractures at 30-60 oLCA are noted around 50.2 m.

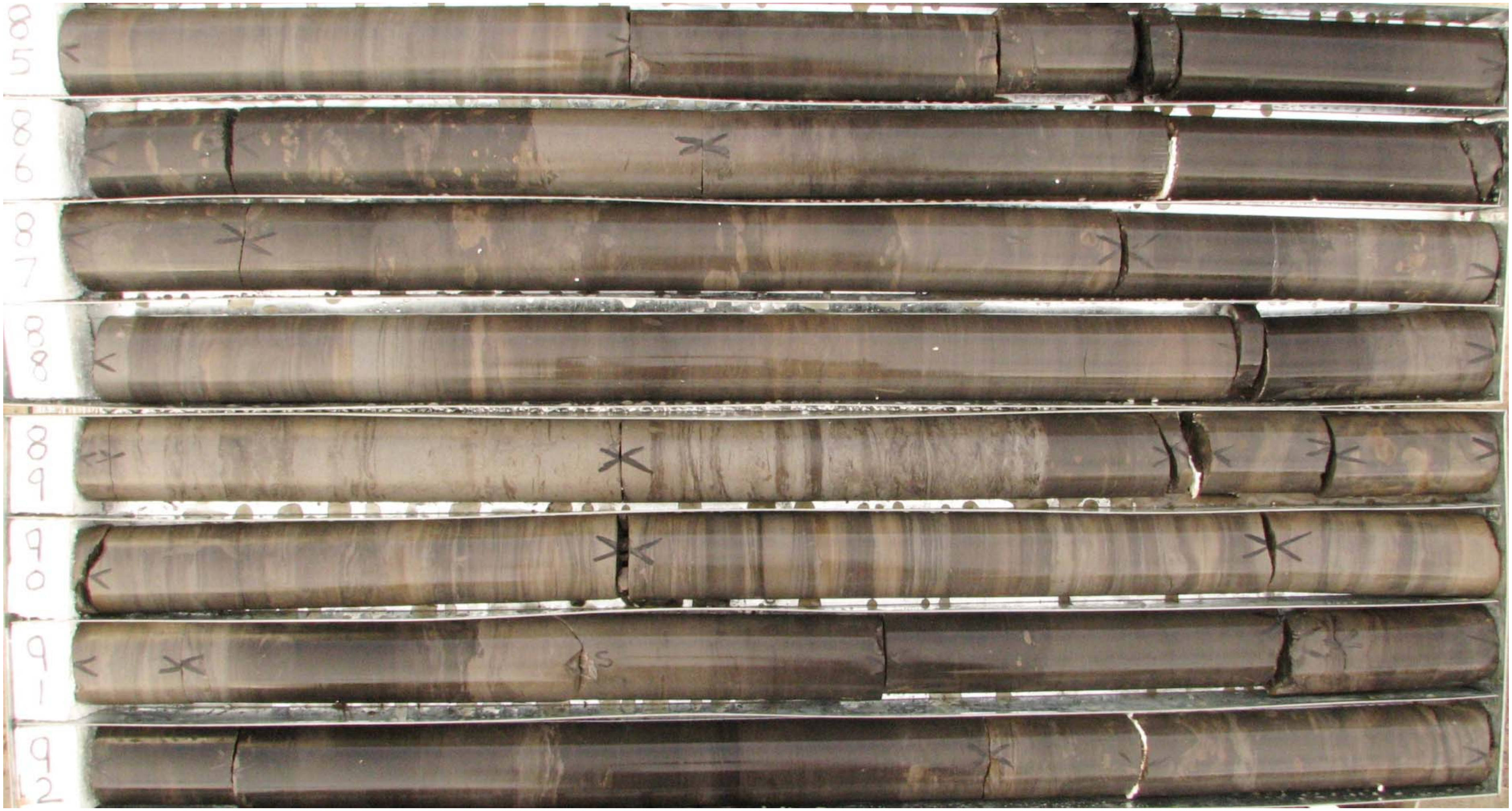
Core 2 53-61 Banded variably carbonaceous siltstone

Core 2 61-69 m



Core2 69-77 – Note blacker more carbonaceous siltstone units

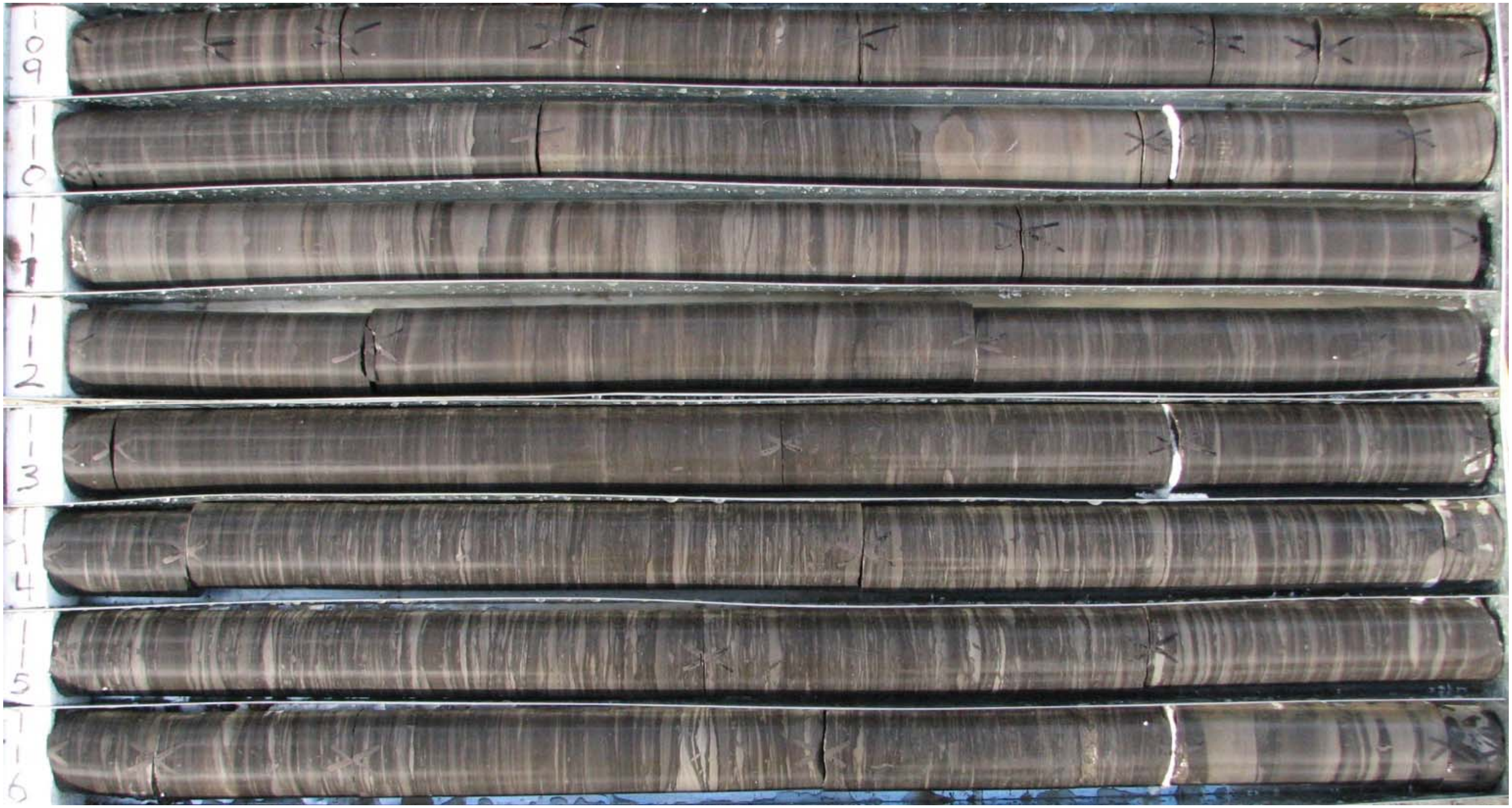
Core 2 77-85 m Note broken zone near 84 m



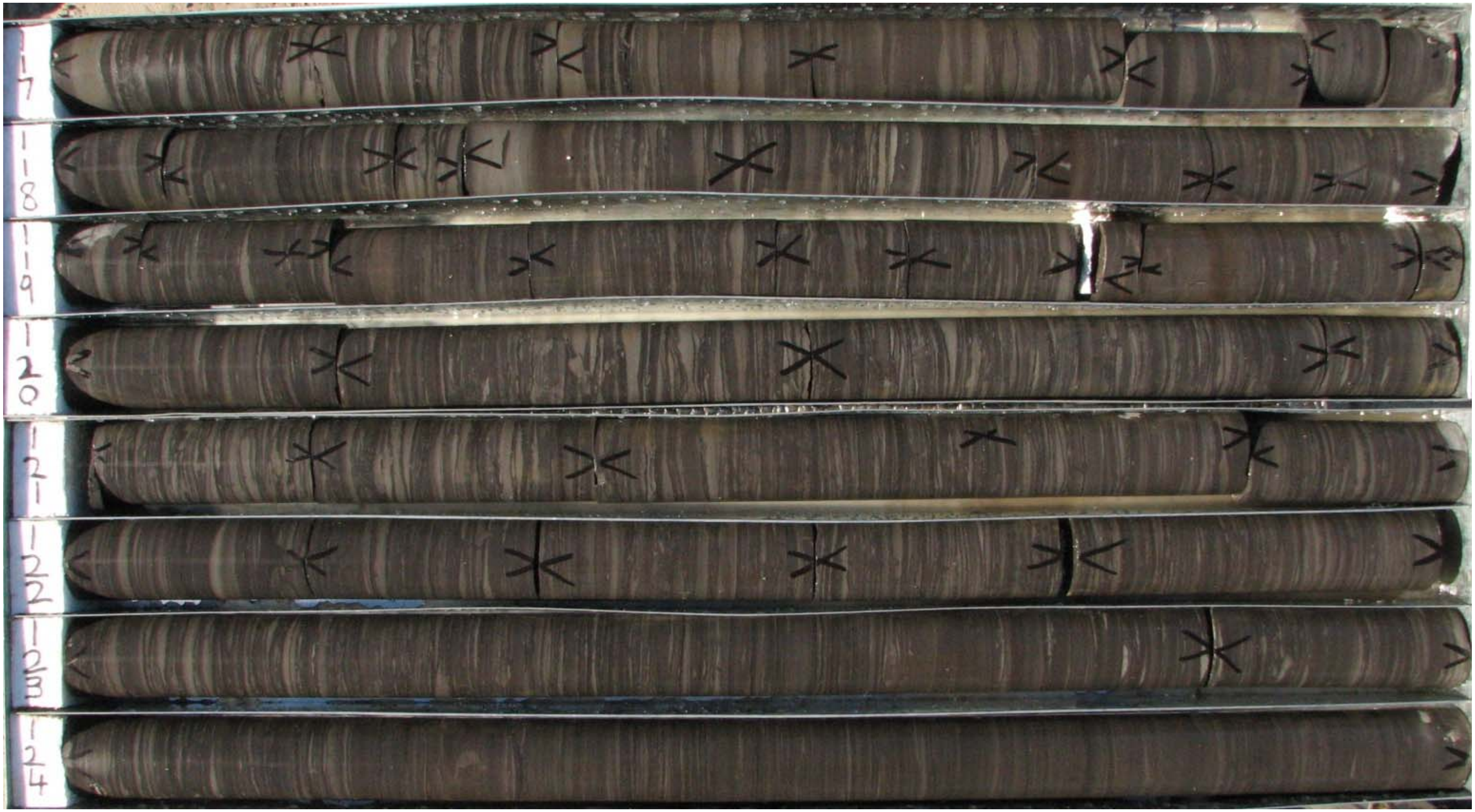
Core 2 85-93 m Note shear fracture marked with S

Core 2 93-101 m. Note fractures marked with S – open shear planes with striations. Note this interval shows more laminated siltstone, transitional to shale, overlying a fine sandstone interval

Core 2 101-109 Laminated shale with sandstone intervals below sandstone from 100.89 m. Note the lack of fractures in the core (excluding drilling induced fractures). The shale becomes more carbonaceous and has less sandy interbeds down hole.



Core 2 109-117 Laminated shale. Note the lack of fractures in the core and the dark carbonaceous character



Core 2 117-125 m. Laminated shale with grey fine sandstone interbeds. Sandstone content decreasing down hole. Note that fractures are essentially entirely drilling induced, with sticks of core a metre or more long recovered.

Core 2 125-133 m. Laminated carbonaceous shale, with sand content decreasing down hole and laminations becoming finer.


Core 2 133-141 Carbonaceous siltstone unit, with mm-scale laminations and core breaking along bedding planes.

Core 2 141-149 m Carbonaceous shale – variably laminated. Minor carbonate filled sub-mm fractures at 70-90^{o}LCA opened during drilling.

Core 2 149-150 m. Carbonaceous shale with well developed banding and laminations. Note that the coarse sandstone intersected in Core 1 at 146 m was not intersected in this hole.

Appendix C
Core Recovery Data and Bore Logs

INTERVAL LOG
QUARRY PROJECT
Date: 12/05/09-22/05/09

Borehole BH10d
Sheet 1 of 1
Logged by: MRB

Hole #	Depth from	Depth to	Interval	Recovered m	Core recovery %	Comments	Loss/gain in run	Cumulative loss
Core 1	31.1	34.1	3	2.98	99.33	Check initial drill runs	-0.02	-0.02
Core 1	34.1	37.1	3	3.02	100.67	Very solid sandstone interval - induced fractures	0.02	0
Core 1	37.1	40.1	3	2.985	99.50		-0.015	-0.015
Core 1	40.1	43.1	3	2.985	99.50		-0.015	-0.03
Core 1	43.1	46.1	3	3.015	100.50		0.015	-0.015
Core 1	46.1	49.1	3	2.995	99.83		-0.005	-0.02
Core 1	49.1	50.1	1	1.07	107.00		0.07	0.05
Core 1	50.1	51.1	1	0.72	72.00		-0.28	-0.23
Core 1	51.1	54.1	3	2.96	98.67		-0.04	-0.27
Core 1	54.1	57.1	3	2.97	99.00		-0.03	-0.3
Core 1	57.1	60.1	3	3.06	102.00		0.06	-0.24
Core 1	60.1	63.1	3	2.98	99.33		-0.02	-0.26
Core 1	0	66.1	3	3	100.00		0	-0.26
Core 1	66.1	69.1	3	3	100.00		0	-0.26
Core 1	69.1	70.1	1	0.97	97.00	No obvious core loss in this zone	-0.03	-0.29
Core 1	70.1	72.1	2	1.93	96.50		-0.07	-0.36
Core 1	72.1	75.1	3	2.94	98.00		-0.06	-0.42
Core 1	75.1	78.1	3	2.985	99.50	Minor grinding	-0.015	-0.435
Core 1	78.1	80.1	2	2.08	104.00		0.08	-0.355
Core 1	80.1	81.1	1	0.94	94.00		-0.06	-0.415
Core 1	81.1	84.1	3	2.94	98.00		-0.06	-0.475
Core 1	84.1	87.1	3	2.905	96.83		-0.095	-0.57
Core 1	87.1	90.1	3	2.99	99.67		-0.01	-0.58
Core 1	90.1	93.1	3	2.995	99.83		-0.005	-0.585
Core 1	93.1	96.1	3	3	100.00		0	-0.585
Core 1	96.1	99.1	3	3.022	100.73		0.022	-0.563
Core 1	99.1	100.1	1	1.06	106.00		0.06	-0.503
Core 1	100.1	102.1	2	1.925	96.25		-0.075	-0.578
Core 1	102.1	105.1	3	3.01	100.33		0.01	-0.568
Core 1	105.1	108.1	3	3.015	100.50		0.015	-0.553
Core 1	108.1	110.1	2	1.93	96.50		-0.07	-0.623
Core 1	110.1	111.1	1	1.06	106.00		0.06	-0.563
Core 1	111.1	114.1	3	3.015	100.50		0.015	-0.548
Core 1	114.1	117.1	3	3.005	100.17		0.005	-0.543
Core 1	117.1	120.1	3	2.98	99.33		-0.02	-0.563
Core 1	120.1	123.1	3	2.98	99.33		-0.02	-0.583
Core 1	123.1	126.1	3	3.015	100.50		0.015	-0.568
Core 1	126.1	129.1	3	2.955	98.50		-0.045	-0.613
Core 1	129.1	130.1	1	1.025	102.50		0.025	-0.588
Core 1	130.1	132.1	2	2.045	102.25		0.045	-0.543
Core 1	132.1	135.1	3	3.01	100.33		0.01	-0.533
Core 1	135.1	138.1	3	2.965	98.83		-0.035	-0.568
Core 1	138.1	140.1	2	2.01	100.50		0.01	-0.558
Core 1	140.1	141.1	1	1	100.00		0	-0.558
Core 1	141.1	144.1	3	3	100.00		0	-0.558
Core 1	144.1	147.1	3	2.95	98.33	Sandstone	-0.05	-0.608
Core 1	147.1	150.1	3	3.025	100.83		0.025	-0.583
END OF HOLE								

INTERVAL LOG
QUARRY PROJECT
Date: 28/05/09-12/06/09

Borehole BH12d
Sheet 1 of 1
Logged by: MRB

Hole #	Depth from	Depth to	Interval	Recovered m	Core recovery %	Comments	Loss/gain in run	Cumulative loss
Core 1	30.1	33.1	3	2.92	97.33		-0.08	-0.08
	33.1	36.1	3	3.05	101.67		0.05	-0.03
	36.1	39.1	3	2.975	99.17		-0.025	-0.055
	39.1	40.1	1	0.875	87.50	Lost remaining section of core back in hole	-0.125	-0.18
	40.1	42.1	2	1.86	93.00		-0.14	-0.32
	42.1	45.1	3	3.03	101.00		0.03	-0.29
	45.1	48.1	3	2.98	99.33		-0.02	-0.31
	48.1	50.1	2	1.98	99.00		-0.02	-0.33
	50.1	51.1	1	0.935	93.50	Loss	-0.065	-0.395
	51.1	54.1	3	2.945	98.17	Loss - overdrill 0.06m	-0.055	-0.45
	54.1	57.1	3	2.975	99.17		-0.025	-0.475
	57.1	60.1	3	2.93	97.67		-0.07	-0.545
	60.1	63.1	3	2.94	98.00		-0.06	-0.605
	63.1	66.1	3	3	100.00		0	-0.605
	66.1	69.1	3	2.905	96.83	Core loss down hole	-0.095	-0.7
	69.1	70.1	1	1.015	101.50		0.015	-0.685
	70.1	72.1	2	2.085	104.25		0.085	-0.6
	72.1	75.1	3	2.935	97.83		-0.065	-0.665
	75.1	78.1	3	3.01	100.33		0.01	-0.655
	78.1	80.1	2	2.015	100.75		0.015	-0.64
	80.1	81.1	1	0.96	96.00		-0.04	-0.68
	81.1	84.1	3	2.96	98.67		-0.04	-0.72
	84.1	87.1	3	2.955	98.50		-0.045	-0.765
	87.1	90.1	3	3	100.00		0	-0.765
	90.1	93.1	3	2.97	99.00		-0.03	-0.795
	93.1	96.1	3	2.995	99.83		-0.005	-0.8
	96.1	99.1	3	2.97	99.00		-0.03	-0.83
	99.1	100.1	1	0.975	97.50		-0.025	-0.855
	100.1	102.1	2	2.065	103.25		0.065	-0.79
	102.1	105.1	3	3.005	100.17		0.005	-0.785
	105.1	108.1	3	2.96	98.67		-0.04	-0.825
	108.1	110.1	2	2.08	104.00		0.08	-0.745
	110.1	111.1	1	0.9	90.00		-0.1	-0.845
	111.1	114.1	3	2.985	99.50		-0.015	-0.86
	114.1	117.1	3	3.04	101.33		0.04	-0.82
	117.1	120.1	3	2.996	99.87		-0.004	-0.824
	120.1	123.1	3	2.97	99.00		-0.03	-0.854
	123.1	126.1	3	3.03	101.00		0.03	-0.824
	126.1	129.1	3	3.005	100.17		0.005	-0.819
	129.1	130.1	1	1.005	100.50		0.005	-0.814
	130.1	132.1	2	1.97	98.50		-0.03	-0.844
	132.1	135.1	3	3	100.00		0	-0.844
	135.1	138.1	3	3.02	100.67		0.02	-0.824
	138.1	140.1	2	1.995	99.75		-0.005	-0.829
	140.1	141.1	1	1.03	103.00		0.03	-0.799
	141.1	144.1	3	3.04	101.33		0.04	-0.759
	144.1	147.1	3	2.95	98.33		-0.05	-0.809
	147.1	150.1	3	3.05	101.67		0.05	-0.759
END OF HOLE								

COMPOSITE WELL LOG

Well No: BH10d

IGGC
PO Box 248
Newtown
NSW, 2042
Australia
Tel: (+61) (02) 9029 2995
Fax: (+61) (02) 9519 0905

Client: Light Horse Business Centre Pty Ltd **Project:** Lighthorse Landfill Site Eastern Creek

Commenced: 12/05/2009
Completed: 11/06/2009
Drilled: Terratest
Logged By: Murray Brooker

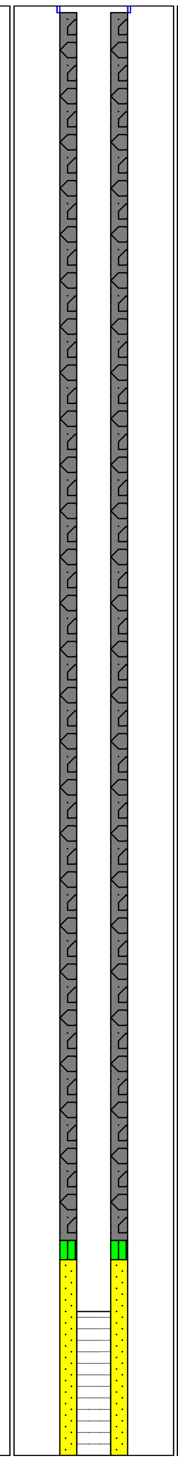
Method: RC
Fluid: Air
Bit Record: 6"

Area: Southwestern
East: 298563
North: 6258104
Elevation:

Static Water Level:

Date:

Depth (mbgl)	Geology	Graphic Log	Lithological Description	Field Notes	Well Completion Diagram	Well Completion Notes
0			Oxidised SST & SSL: Weakly oxidised sandstone and minor interbedded siltstone			Class 18 PVC/monument/lockable cap
-10			Interbedded SST & SSL: Interbedded siltstone and fine sandstone, generally with beds < 1 m thick.			
-20						
-30						
-40						Grouted Annulus
-50						Class 18 PVC pipe
-60						
-70						
-80						
-90			Shale & SST: Relatively rapid upper transition from siltstone to shale, which is carbonaceous and interbedded with fine sandstone.			
-100						
-110						
-120						
-130						Bentonite Seal
-140						Sand Pack
-150			Coarse SST: Coarse quartz sandstone, with grey laminations			Screened Interval 135.3-150.3 m



COMPOSITE WELL LOG

Well No: BH11i

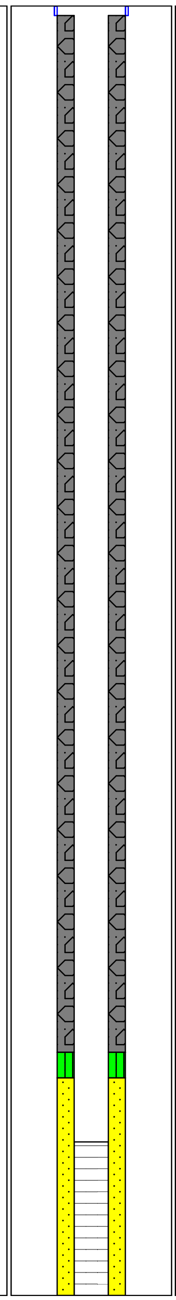
IGGC
PO Box 248
Newtown
NSW, 2042
Australia
Tel: (+61) (02) 9029 2995
Fax: (+61) (02) 9519 0905

Client: Light Horse Business Centre Pty Ltd **Project:** Lighthorse Landfill Site Eastern Creek

Commenced:	12/06/2009	**Method:**	RC	**Area:**	Southwest
Completed:	15/06/2009	**Fluid:**	Air	**East:**	298566
Drilled:	Terratest	**Bit Record:**	6"	**North:**	6258082
Logged By:	Murray Brooker			**Elevation:**	

Static Water Level: **Date:**

Depth (mbgl)	Geology	Graphic Log	Lithological Description	Field Notes	Well Completion Diagram	Well Completion Notes
0			Oxidised SST & SSL: Weathered fine sandstone and interbedded siltstone			Class 18 PVC/monument/lockable cap
-10			Interbedded SST & SSL: Interbedded siltstone and fine sandstone, generally with beds < 1 m thick.			
-20						
-30						
-40						Grouted Annulus
-50						Class 18 PVC pipe
-60						
-70						
-80						Bentonite Seal
-90			Shale & SST: Relatively rapid upper transition from siltstone to shale, which is carbonaceous and interbedded with fine sandstone.			Sand Pack Screened Interval 88-100 m
-100						



COMPOSITE WELL LOG

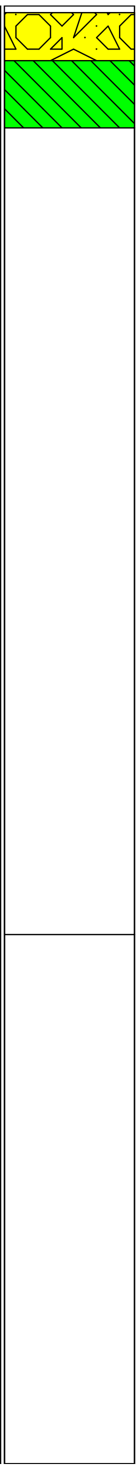
Well No: BH12d

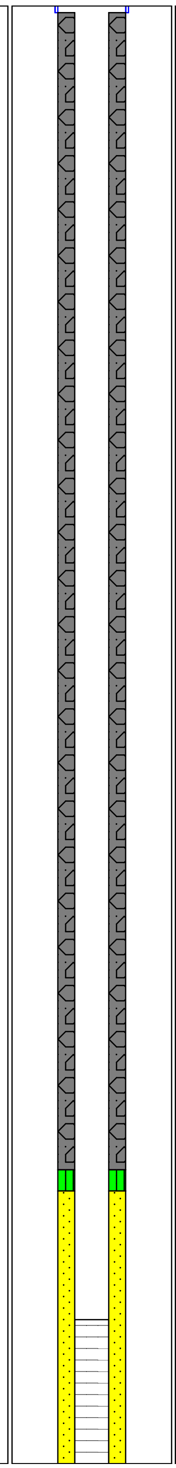
IGGC
PO Box 248
Newtown
NSW, 2042
Australia
Tel: (+61) (02) 9029 2995
Fax: (+61) (02) 9519 0905

Client: Light Horse Business Centre Pty Ltd **Project:** Lighthorse Landfill Site Eastern Creek

Commenced:	25/05/2009	**Method:**	RC	**Area:**	
Completed:	23/06/2009	**Fluid:**	Water - Core	**East:**	299226
Drilled:	Terratest	**Bit Record:**	6"	**North:**	6258462
Logged By:	Murray Brooker			**Elevation:**	
Static Water Level:				**Date:**	

Depth (mbgl)	Geology	Graphic Log	Lithological Description	Field Notes	Well Completion Diagram	Well Completion Notes
0			Fill: Shale rich fill as part of bund around quarry pit			Class 18 PVC/monument/lockable cap
-10			Clay: Oxidised clay and weathered siltstone			
-20			Interbedded SST & SSL: Interbedded siltstone and fine sandstone, generally with beds < 1 m thick.			
-30						
-40						Grouted Annulus
-50						Class 18 PVC pipe
-60						
-70						
-80						
-90						
-100			Shale & SST: Relatively rapid upper transition from siltstone to shale, which is carbonaceous and interbedded with fine sandstone.			
-110						
-120						Bentonite Seal
-130						
-140						Sand Pack
-150						Screened Interval 136-151 m





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NSW, 2042
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Tel: (+61) (02) 9029 2995
Fax: (+61) (02) 9519 0905

COMPOSITE WELL LOG

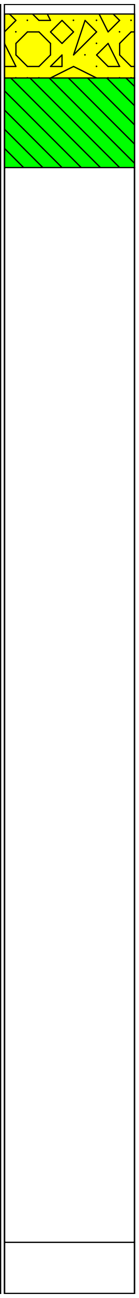
Well No: BH13i

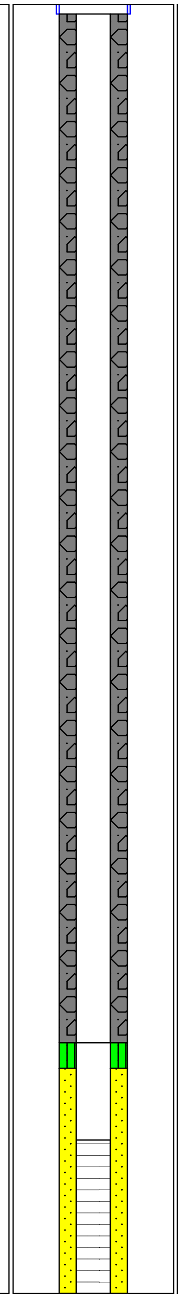
Client: Light Horse Business Centre Pty Ltd **Project:** Lighthorse Landfill Site Eastern Creek

Commenced:	17/06/2009	**Method:**	RC	**Area:**	North side
Completed:	17/06/2009	**Fluid:**	Air	**East:**	299201
Drilled:	Terratest	**Bit Record:**	6"	**North:**	6258470
Logged By:	Murray Brooker			**Elevation:**	

Static Water Level: **Date:**

Depth (mbgl)	Geology	Graphic Log	Lithological Description	Field Notes	Well Completion: Diagram	Well Completion: Notes
0			Fill: Shale rich fill as part of bund around quarry pit			Class 18 PVC/monument/lockable cap
-5			Clay: Oxidised clay and weathered siltstone			
-12			Interbedded SST & SSL: Interbedded siltstone and fine sandstone, generally with beds < 1 m thick.			
-40						Grouted Annulus
-50						Class 18 PVC pipe
-80						Bentonite Seal
-90						Sand Pack
-96			Shale & SST: Relatively rapid upper transition from siltstone to shale, which is carbonaceous and interbedded with fine sandstone.			Screened Interval 88-100 m
-100						





Depth scale: 0, -10, -20, -30, -40, -50, -60, -70, -80, -90, -100