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Report on
Soil, Water & Leachate Management Plant

Light Horse Business Centre
Eastern Creek

Prepared for
Alexandria Landfill Pty Ltd

Project 46950
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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

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Table of Contents

Page

List of Figures

List of Appendices

List of Drawings in Appendix A

List of Drawings in Appendix B

Report on Soil, Water & Leachate Management Plan
Light Horse Business Centre, Eastern Creek

1. Introduction

ACN 114 843 453 Pty Ltd (also referred to as the Light Horse Business Centre or LHBC) proposes to operate a Resource Recovery/Materials Processing Centre and a solid waste landfill site at the former Pioneer Quarry located off Old Wallgrove Road at Eastern Creek ("The Site").

The landfill will be located within a former breccia quarry and its geological and hydraulic characteristics are described in, amongst others, the following reports:

- Archbold Road, Eastern Creek: Groundwater and Salinity Assessment for Proposed Quarry Rehabilitation Project and Developable Land, Ian Grey Groundwater Consulting May 2007,
- Report on Preliminary Contamination Assessment Stockpiled Material and General Land Quality, Douglas Partners, April 2006.

A water balance and a Soil, Water and Leachate Management Plan (SWLMP) for the site is required for the construction phase of the project and for the operation of the site, based in part on the surface water report modelled by Martens Consulting Engineers and on the projected filling plan. The information has been compiled herein and the relevant reports on which the SWLMP is based are appended.

This SWLMP is required under the following planning conditions issued by the Department of Planning associated with their Major Project Assessment dated 22 November 2009:

"The Proponent shall prepare and implement a Soil, Water and Leachate Management Plan for the site to the satisfaction of the Director-General. This plan must:

a) be submitted to the Director-General for approval prior to construction;

b) be prepared by a suitably qualified and experienced expert;

c) be prepared in consultation with the DECC and Council; and

d) include:

   - a site water balance;
   - an erosion and sediment control plan;
   - a stormwater management scheme;
   - a surface water, groundwater and leachate monitoring programme; and
   - a surface water, groundwater and leachate response plan."

The structure of this report reflects the respective planning conditions associated with the SWLMP. In addition, background information is provided on the planning condition relating to the Leachate Management System given its relation to the SWLMP. This SWLMP draws from a number of reports prepared by various consultants relating to specific facets of the overall SWLMP. Appended reports prepared by others have been summarised in this report. Technical review and endorsement by DP of calculations, designs and recommendations presented in referenced or appended reports prepared by others was not part of DP's scope of work. DP had no input to these reports and takes no responsibility for their content.

The report has been prepared with reference to the following published guidance:

- EPA, 1996. Environmental Guidelines: Solid Waste Landfills.
- Landcom, 2004. Managing Urban Stormwater: Soils and Construction ("Blue Book"). Fourth Edition.
- DEC, 1997. Draft Managing Urban Stormwater: Council Handbook.

The site layout is shown on Drawing 1, Appendix A.

2. Operational Areas

In order to establish a background to the proposed site operations, the "operational areas" are described as follows.

Sector A – Processing Area approximately 194,000 m^2
Area 1 - Clean collection area in Sector A comprises the Materials Processing Centre (MPC) shed and the stockpile areas including the internal roads, the workshop, office, weighbridge area and car parks.

Area 2 – Leachate area comprising green waste storage, green waste processing activities and MPC work floor areas (exposed).

The separation of Areas 1 and 2 will be ensured through:

- Delineation of the green waste areas;
- Preventing leachate from escaping the green waste areas;
- Preventing clean operational waters from entering the green waste areas;
- By the use of appropriate bunding and grading of Sector A.

Note: The processing area (Sector A) including the MPC, administration, workshop weighbridge buildings and car parks drains in the manner and direction described in GMW Drawings (7328_006 to 7328_0010) presented in Appendix A. Further details are also shown on Jones Nicholson Drawing 090669 H01 presented in Appendix A.

Sector B – Landfill Area
The total catchment over the landfill is approximately 280,000 m^2.

Area 1 – Clean operational area. This is defined as an area having no exposed waste and that has been capped with intermediate cover material (300 mm of suitable cover material or as required by the

Environmental Protection Licence for the site). The clean operational area of the landfill is expected to be graded towards a sump or pond. Water collected therein will be pumped for appropriate reuse (mostly dust suppression around the site) or to stormwater as required.

Area 2 – Leachate area. This is an area where water comes into contact with exposed waste or daily cover. All surface water run off from that area will be treated as leachate and collected in a sump. Following treatment of leachate waters it will then be discharged through to sewer. The daily tipping area is expected to be approximately 450 m^2 – 1000 m^2 with a working landfill face of approximately 4,000 m^2.

Sector B is the area to be filled progressively in a cellular manner as set out in chapter 3.4.6 (Figure 3.5) of the EA (ERM, 2008). The filling plan is summarised as follows:

Filling Plan
Initial filling will commence in the south-western corner of the quarry base at the shallowest point and proceed north and west in a series of landfill cells towards the north-western corner. Working in cells, benching and vehicle movements will be accommodated around the active tipping area. The initial lift is expected to be incremented to a height of 10 m. Once the north-western corner is reached, the filling area will proceed east and continue back to the southern side of the pit. This will constitute a 'windscreen wiper' formation. This process will be repeated until filling reaches the eastern end of the quarry at which time the total lift throughout the base of the quarry is expected to reach the initial 10 m. Filling will then occur in the same manner in the opposite direction and with subsequent lifts to be approximately 10 m (ERM, 2008).

In accordance with Benchmark Technique No. 33 (BT33) (EPA, 1996), the active tipping area will be covered at the end of each working day with 150mm of cover material or other type of landfill daily cover as approved by the Office of Environment and Heritage (OEH). Cover material will be either VENM or Alternative Daily Cover material approved by the OEH. Daily cover will be scraped off everyday prior to filling starting and will be topped up as required to allow that recycled daily cover material will diminish with each scrape-off event. Concept landfill plans have been prepared based on the maximum fill rate of 700,000 tonnes per annum.

Landfill Capping Schedule

Sector B
Daily and Intermediate Cover
The active tipping area of approximately 450 m^2 – 1000 m^2 in Sector B is the only part of the site in which waste is open to the environment. The active tipping face is to be covered at the end of every day with approved covers or with 150 mm of cover material as per BT33 (EPA, 1996). This is to be scraped back daily prior to tipping re-commencing on each subsequent day.

Once tipping is complete in any area of Sector B (i.e. no further tipping will occur for at least 90 days) intermediate cover is to be laid and compacted. The capping material shall comprise 300mm of VENM as per BT33 or other suitable engineering material approved by the OEH for that purpose.

Final Capping
Benchmark Technique No. 28 (BT28) (EPA, 1996) seeks to addresses site capping and revegetation as follows:

- prevention of pollution of water by leachate;
- prevention of landfill gas emissions;
- assurance of the quality of design, construction and operation
- minimisation of landfill space;
- prevention of degradation of local amenity.

LHBC has taken the above as a guide for provision of a suitable containment system designed to prevent the spread of contaminants from the landfill. LHBC may however amend this plan in line with any updates or changes of policy concerning landfill capping at the time it is required.

This provides a barrier to the migration of water and gas, promotes sound land management and conservation, and prevents hazards whilst protecting local amenity. Further, LHBC will ensure that capping remains effective through long term monitoring of groundwater (as described in this SWLMP) and landfill gas.

3. Site Water Balance

The planning condition for the site water balance states the site water balance must:

- include details of all water extracted, transferred, used and/or discharged by the development;
- identify the source of all water collected or stored on the site, including rainfall, stormwater and groundwater; and
- describe the measures that would be implemented to minimise water use on site.

The following subsections of the report seek to address the above planning condition. The information provided derives from reports prepared previously by others. DP has not completed detailed checks of calculations, designs and recommendations reported by others and summarised herein.

Water extracted, transferred, used and/or discharged by the development is summarised in the following table.

Table 1: Summary of Site Water Balance

Water	Usage / Storage	Anticipated Volume
Sector A – Processing Area		
Water demands	Toilets, irrigation, dust suppression, wheel wash	33,900kL per year^
Runoff	Runoff (mean rainfall year)	193,900kL per year**
Roof runoff	Total storage 1,580kL	meets 100% of demands
On-site detention basins (OSD)	Two basins to handle runoff, Basin One, Basin Two	One 3,400kL capacity to outlet weir Two 3,100kL capacity to outlet weir
Dust suppression	OSD water for dust suppression	meets 100% of demand
Sector B – Landfill Area		
Groundwater ingress	Generated by groundwater inflow	<$3m^3$ per day*
Leachate generation / pumping rates	Leachate pumping rates are likely to vary throughout the life of the landfill	estimated maximum required 500kL per day^^
Leachate storage tanks	Sequencing batch reactors (SBR) for leachate treatment	550kL per day based on 7 to 9 hour treatment time
Sediment basin	Sediment basin for "clean water" in quarry	4,362kL capacity^
Leachate disposal	Four sequencing batch reactors (SBR) for leachate treatment prior to discharge to sewer	320kL decanting capacity for each sequence
Leachate disposal	Sewer Discharge under Trade Wastewater Consent	Max Discharge – TBC

Source: ^Martens (2011)
**DADI (2011)
*IGGC, (2009)
^^ERM, (2008)

3.1 Site Water Balance

This section presents the results of the site water balance using actual monthly rainfall and estimated leachate production rates for average and 90th percentile conditions. The information summarised herein is based largely on Martens (2011), ERM (2008) and IGGC (2009).

Calculation of a leachate water balance for a landfill site involves estimation of the various inputs and outputs to and from the waste mass, and allows the potential leachate production rate to be assessed. Water balances are commonly used in landfill site design, particularly in the sizing of cells to minimise leachate production.

In the case of LHBC, the site is not yet in operation and potential leachate production rates can only be estimated.

The leachate water balance for the site for any given time period is described by the following equation:

Output (pumped leachate) = Input – Change in Storage

Inputs

The liquid inputs to the landfill site are as follows:

- infiltration of rainfall directly into waste;
- infiltration of rainfall through the landfill cap;
- infiltration of stormwater runoff from off-site and on-site areas;
- groundwater ingress (breccia fractures);
- liquid waste inputs; and
- miscellaneous other sources, such as infiltration of dust suppression water etc.

Liquid waste inputs are assumed to be negligible for the site given that no liquid waste will be received.

Miscellaneous other sources of liquid and moisture in the waste stream are also assumed to be negligible. This includes water used for dust suppression. Spraying for dust suppression only occurs during dry weather to prevent dust generation from dry surfaces, and infiltration of spray waters is expected to be negligible. Stormwater inflows into the landfill site from offsite sources are also expected to be minor.

The site wheel wash will be a sealed, recirculating design, and does not contribute any water to the landfill area. Direct infiltration of rainfall into the waste mass is considered to be the most important input, because of the limited potential for evaporation and the large active area.

Groundwater ingress from fractures in the Breccia is thought to be minor. A proportion of the groundwater inflow will be intercepted and pumped to stormwater via two systems as per the EA (ERM, 2008).

Outputs

Losses of water from the waste are as follows:

- Direct evaporation from waste surface;
- Direct evaporation from hard-surfaced areas;
- Run-off and discharge to stormwater from hard-surfaced areas;

- Evapo-transpiration from capped areas;
- Absorption by received waste; and
- Leachate disposal to treatment plant and or sewer.

Leakage
Leakage into underlying strata is expected to be negligible at Eastern Creek because of the inward hydraulic gradient in the perched and fractured rock aquifers.

Absorptive Capacity
Absorptive capacity of the waste received is estimated at 50 litres per cubic metre, approximately two-thirds of that typical for compacted domestic waste. The expected waste input rate is 30,000 tonnes per month, equivalent to 450,000 m^3 per year (excluding daily cover material).

3.1.1 Water Balance Methodology & Concept

A daily water balance analysis modified after Storm (2008) (DADI pers. comm. Oct 2011) was used to determine the feasibility of the proposed rain and stormwater harvesting scheme and in particular the effects of various storage sizes for stormwater harvesting along with changes to demand. The water balance utilised flows generated using a simple runoff calculation using historical rainfall data, analysed for various rainfall patterns including dry, mean and wet rainfall years.

The purpose for modelling dry, mean and wet years was to assess the performance of various tank sizes given the changes to rainfall patterns. It is noted that with the potential effects of climate change and the current trend of dry rainfall patterns, the need to consider lower annual rainfalls for rain and stormwater harvesting reuse schemes is becoming more and more necessary. In addition, any excess stormwater produced (especially during wet season periods) need to be considered for the management of on-site surface waters.

A concept diagram for the proposed re-use scheme on site is shown in Figure 1 below.

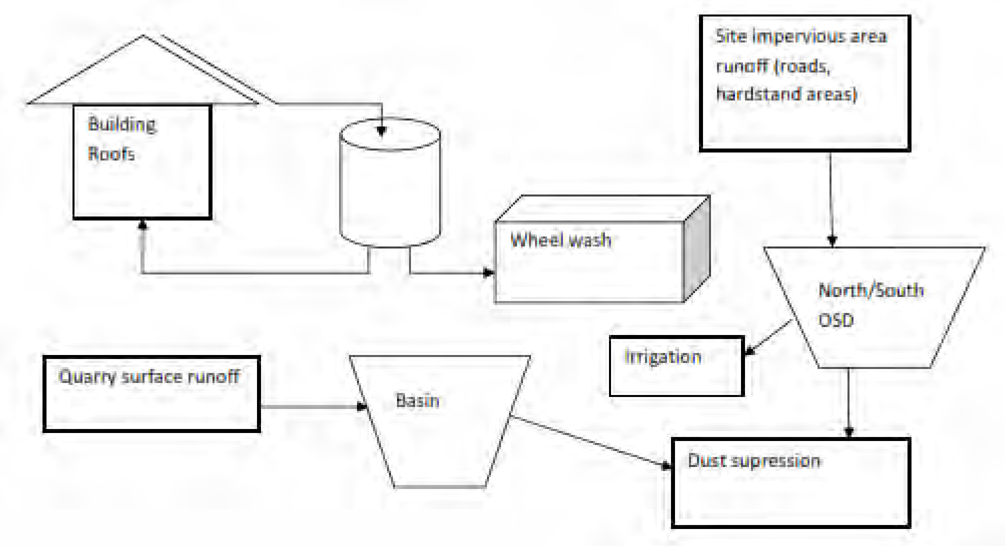

Figure 1: Concept Diagram for Proposed Re-Use Scheme

Modelling Inputs

Rainfall

Data from St Clair (BOM station #67102) was used. Seventeen years of daily rainfall data (1985 – 2002) was assessed to determine a dry, median and wet rainfall sequence for use in the water balance model.

Table 2: Modelled Rainfall

	Modelled Average rainfall and years (mm)	Prospect (long term average) (mm)
Dry	553 (1994/1995/2001/2002)	562
Median	851 (1987/1989/1991)	831
Wet	1104 (1986/1987/1988/1989/1990)	1183

Harvestable areas

The proposed roof and stormwater reuse scheme can harvest runoff from the operational area catchment. This is conservative (under-estimates area available) and excludes the proposed green waste area.

Table 3: Harvestable Areas

Precinct	Area (ha)	Initial loss (mm)
Building roofs	0.85	1
Catchment runoff	21	5
Quarry	26.5	10

Pre-Water demands

The demands for harvested water for reuse includes toilet flushing, dust suppression, sprinklers (irrigation) and the wheel wash.

Table 4: Water Demands

Annual Demand (Ml/yr)				Modelling assumptions
	Dry years	Mean years	Wet years	
Toilet	0.4	0.4	0.4	34 staff on site x 6 flushes/day x 4.5L
Dust suppression	25.8	24.1	24.0	Average application = 80kL/day (assumes no application if daily rainfall exceeds 2mm)
Sprinklers	9.7	9.1	9.0	Average application = 30kL/day (assumes no application if daily rainfall exceeds 2mm)
Wheel wash	0.3	0.3	0.3	Water use = 25kL/month
Total	36.2	33.9	33.7	

Results - Catchment runoff

The actual runoff that can be harvested for reuse will not be the entire volume generated due to losses from the system, and is dependent on storage behaviour (i.e. if the storage volume reaches 100% capacity, overflows will occur rather than further collection).

Table 5: Runoff Summary

Rainfall Scenario	Potential Runoff Generated (ML/yr)		
	Dry	Median	Wet
Building roof	4.25	6.6	8.7
Quarry	44.9	73.2	236.8
Catchment runoff	62.7	114.1	200.2
Total	111.8	193.9	445.7

Rain tanks and building roofs

Overall tank storage volumes of up to 1,580kL would meet all of the site's toilet flushing and wheel wash demands for the dry, median and wet rainfall scenarios.

Surface runoff from clean operational area of the RRF

Surface runoff from the internal roads/hardstand areas and remaining site operational area will be collected. Runoff from these areas will be directed towards the OSD basins which will include a storage component and be drawn down for reuse on site following storm events.

An additional five tanks with a capacity of 50kL each have been provided for, to specifically pump water from the OSDs for storage and re-use to prevent overflow.

A water balance was prepared for the water demand scenario of:

- Dust suppression for watering carts + truck on-board reservoirs (80kL/day) and spray mists/sprinkler system for irrigation or dust suppression (30kL/day).
  - Note: it is assumed that the water quality will be of adequate standard for reuse and will not pose a risk to human or environmental health.
- It was also assumed that on days where daily rainfall exceeds 2mm there is no demand for dust suppression.

Current indicative basin size in the site drawings (Appendix B of the Martens Report) allows for approximately 5,000kL from Basins 1 and 2 combined, which should meet all of the assumed water demand for dust suppression and irrigation combined.

Surface Runoff from Quarry – Sector B
Captured runoff in the quarry basin will be used for dust suppression via water carts. The available water volume for reuse from the basin will vary depending on rainfall and the stage of landfill operation, as the basin size is intended to increase in proportion to the capped landfill catchment area and runoff from quarry walls as required.

In practice the basin size may vary in relation to the area of capped landfill that is its catchment (at a rate of 165 m^3/ha). For this reason it was modelled separately to the storage options within the OSD basin.

Runoff collected from these areas will be suitable for reuse such as dust suppression if it has not come into contact with waste.

Summary of Storage Volumes
Each building should have its own rainwater tank (minimum 10kL volume) to harvest roof water runoff for reuse including toilet flushing and wheel wash top up. Tank storage capacity totalling approximately 1.5 million litres has been provided for on site.

The OSD storage proposed for the operational area is of sufficient volume to contain the 1 in 2 year storm event, 1 in 10 year storm event, and 1 in 100 year storm event with storm durations between 25 and 540 minutes. By use of additional depth in the basin (nominal 0.5m in indicative basin sizes supplied) to act as storage for reuse on-site. It is anticipated that drawdown will occur regularly for dust suppression (water carts and sprinkler) and irrigation.

The proposed sediment basin in the quarry has been sized using the Blue Book (approximately 165 m^3/ha) and can be drawn down following storm events for dust suppression (water carts).

Evaporation and Runoff
Evaporation has been simulated by applying a factor to the total rainfall depending on the type of surface.

In the case of Sector A and the clean pond in Sector B the factor applied represents both evaporation and runoff, because these areas drain to stormwater collection and disposal systems.

Low infiltration rates for Sector A which will be concrete or bitumen surfaces graded to fall towards gross pollutant traps (GPTs).

This is a highly simplified approach to evaporation, and will result in overestimated evaporation during the winter months and during wet years, and underestimated evaporation during the summer (although annual values should be accurate as long as the calibration is reliable). However, a more sophisticated approach would require detailed analysis of daily rainfall and evaporation data, and possibly disaggregation of rainfall events into hourly rates. Information would also be needed on cap/daily cover thickness & construction and other factors.

Given the uncertainties involved in the type of analysis and the heterogeneous nature of landfilled waste, a more sophisticated approach is not justified, even if sufficient data were available. Calibration using actual data for rainfall and leachate generation provides confidence that the overall approach is robust.

Leachate Storage *In Situ*
The waste mass within LHBC site provides a potential storage for leachate. The base of the pit is at around minus 150 mAHD, and the current area of the base of the quarry is approximately 13,000 m^2. The total volume in storage has no direct bearing on the water balance calculations; however the change in storage volume over the period under consideration is significant.

Leachate levels can only be measured at the sump, and there is a high degree of uncertainty with estimated changes in storage because conditions within the waste mass are poorly understood (including performance of the drainage system, leachate levels away from the sump, effective porosity etc).

3.1.2 Groundwater Ingress and Capture & Leachate Generation

Estimating the groundwater ingress into the quarry void has been considered by previous reports (ERM, 2008; IGGC, 2009; and Red Earth Geosciences, 2009) in the context of the water balance. This section outlines the previous studies and evaluates the significance of groundwater ingress on the water balance.

Groundwater ingress into the site from apparent fractures in the Breccia as have been identified in the report of Jeffrey & Katauskas (ref: Appendix K of the EA) occurs primarily along the west / Northern boundary of the site, at a level of between 0 and minus 6 mAHD.

The rate of ingress has previously been estimated as 125 kL/day (ref: Page 17, Report by IGGC Pty Ltd, Appendix C of the EA).

Recent assessment by IGGC Pty Ltd using a water logger produced the following report Groundwater Inflow Assessment, Former Hanson Quarry (IGGC, 2009). The findings of the report are summarised in the subsequent report entitled Proposed Light Horse Landfill Site, Eastern Creek: Detailed Hydrogeological Investigation and Assessment (IGGC, 2009). The latter report is presented in Appendix D. The findings of the report are summarised herein.

Data Collection
The quarry is in the process of being dewatered by pumping of water from the pond located in the quarry base. Pumping was routinely undertaken during quarry operation with anecdotal information from quarry staff indicating an average accumulation rate of around 125kL/day or 125m^3/day (IGGC,

2006). A substantial proportion of this accumulation is expected to result from rainfall and this is also supported by anecdotal information.

Quarrying ceased in 2006 at which time pumping also ceased for a period of around 18 months. Hanson Construction Materials Pty Ltd re-commenced pumping in late 2008 for use in dust suppression for its crushing and stockpiling activities. The quarry pond is almost completely dewatered. The pumping rate during dewatering is estimated at 30L/s.

Pumping was suspended between the 5th February and the 11th February 2009 to allow monitoring of the rate of water level rise. Prior to suspension, two pressure transducers with data loggers (referred to hereafter as “loggers”) were placed in a length of well screen for protection and lowering into a sump hole in the quarry floor. A barometric pressure logger was left in the site office to allow correction of data for barometric variations. On 11 February 2009, the loggers were retrieved and the data downloaded.

Data collected by the loggers were corrected for barometric variations and graphed to allow analysis. A graph by IGGC showing the full record from both loggers is shown below in Figure 2.

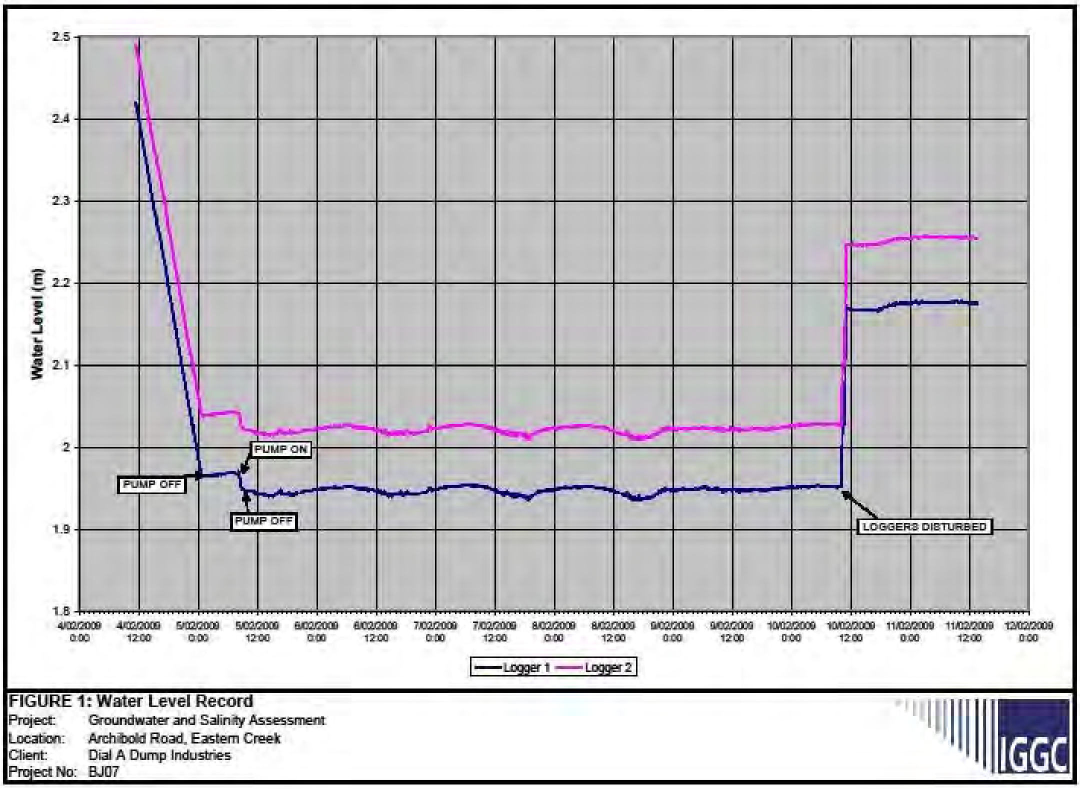

Figure 2: Graphed Data-logger Level Data

The following conclusions were made by IGGC :

- 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; and
- 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 6.

Table 6: Summary of Climate Data (to 9am on date given)

Date	Day	Rain to 9am (mm)	Evaporation to 9am (mm)	Net Gain (mm)
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	1.6
11/02/2009	Weds	5.6	1.1	4.5
	TOTAL	8.8	42.9	-34.1

Starting with the water level on 5 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.

The quarry pond was estimated to have a surface area of around 3,600m^2 during the monitoring period (pers. comm., DADI). A check calculation was performed using the estimated pump rate (30L/s) and the observed rate of decline during pumping (0.8m/day).

This indicates an effective pond area of 3,240m^2, and the estimate of 3,600m^2 will therefore give a slightly conservative results. The IGGC 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 0.049m over 6 days, equivalent to 0.008m/d. Based on the estimated pond area of 3,600m^2 this indicates a net volume gain of 29.4m^3/d.

According to IGGC 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 125m^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 1.75×10^{-6} m/d to 8.7×10^{-6} m/d with a calculated inflow of around 2m^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 and localised higher hydraulic conductivity zones associated with fracturing.

In the long term operation of the proposed landfill IGGC advocates allowing the leachate level to rise as waste is placed, with a final level maintained at an appropriate margin below the regional groundwater level (RL 50mAHD) 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 3m^3/day.

Recommendations
Analysis of pond water level data collected during a period without dewatering indicates a maximum groundwater inflow rate to the quarry pond of 29.4m^3/d (IGGC, 2009). This is consistent with previous assessments, confirms that the groundwater contribution is very low, and comprises a small proportion of the total water input, the majority being due to rainfall.

The hydraulic conductivity of the deep formation has been estimated to be 1.01×10^{-10}m/s and 2.03×10^{-11}m/s based on slug test calculations at BH01 and BH03 (Red Earth Geosciences, 2009). Packer testing has indicated the hydraulic conductivity to be 1.5×10^{-8}m/s for BH10d and 8.1×10^{-9}m/s for BH12d based on the geometric mean of packer tests conducted whilst drilling (IGGC, 2009). The packer tests showed some indication of a general decrease in hydraulic conductivity with depth (refer to Figure 6.3 in IGGC, 2009). Based on these results an engineered compacted clay liner with a design hydraulic conductivity of 1×10^{-9}m/s will not reduce groundwater inflow significantly. Localised grouting of active fracture zones should be considered if a reduction in the groundwater inflow rate is required to assist in leachate management, and experience with grouting of fractures for tunnelling projects in the Sydney basin shows this approach to be effective.

The long-term groundwater inflow rate is expected to reduce over time due to the reduced hydraulic gradient as the leachate levels build up in the pit. On this basis and subject to further investigations and data to be obtained as recommended by Red Earth Geosciences (2009), it appears that groundwater inflow is likely to be a negligible factor in water balance calculations.

Leachate Water Balance (ERM, 2008)
The leachate water balance was presented in Appendix D of the EA (ERM, 2008). The leachate water balance is summarised herein. It was anticipated that the ‘best case’ infilling rate of the landfill was approximately 400,000 tonnes/year (estimated to be 235,000 m^3/year), however, under ‘worst case’ conditions the infilling rate is likely to approximate 700,000 tonnes/year (estimated to be 413,000 m^3/year). Under best case conditions it is anticipated that the pit cavity will be infilled within 65 years, this will shorten to approximately 26 years under worst case infilling conditions.

DECC requested that a spreadsheet based model be developed by ERM to assess the required discharge rates for leachate; the likely leachate water elevations in the landfill; the required leachate surface storage; and the anticipated discharge rate to sewer. A spreadsheet-based model was developed by ERM in accordance with the Draft Environmental Guidelines: Landfilling (DECC, 2008) as per DECC recommendations and included the following parameters:

- Monthly time steps over a period of 100 years;
- The incorporation of 90th percentile wet years at year 1 and at 10 year intervals. Average rainfall conditions were used for the remaining years;
- Groundwater inflow to the pit of $2m^3$/day;
- A surface area of the landfill base of $12,000m^2$ and a maximum surface area of $265,000m^2$;
- In accordance with the Draft Environmental Guidelines: Landfilling (DECC, 2008) there has been an assumption that 50% of rain falling on the temporary capping at the surface of the landfill becomes leachate while the remaining rainfall runs off as surface water. Following this it is assumed that 10% of rain falling on the landfill cap after closure becomes leachate; and
- The spreadsheet model was also designed to incorporate the infilling procedure outlined above.

The key ERM data relevant to this document are summarised follows:

- The design of the infilling system will allow separation of surface water run-off from the sides of the landfill from the rain falling directly onto the landfill waste and infiltrating to become leachate. This will significantly reduce the volume of leachate generated;
- Table 7 below summarises the conservatively estimated volumes of surface water and leachate generated within the landfill. Based on this, leachate generated was anticipated to range between 45 and $872m^3$/day, with and average of $241m^3$/day;
- In order to maintain groundwater elevations at acceptable levels within the landfill pumping rates from the landfill will be required to range between $250m^3$/day and $500m^3$/day;
- Providing that pumping rates do not fall below $241m^3$/day, the landfill will be able to be used as a storage facility during times of high rainfall. This will allow a constant flow rate to be achieved from the leachate collection system and will negate the need for surface storage capacity for leachate pumped from the landfill;
- At the completion of the landfill and subsequent capping, leachate generation is likely to fall below $90m^3$/day. Due to the potentially poor ability of the regional groundwater system to absorb this volume of leachate there is potential for leachate elevations to eventually rise above the regional groundwater elevation and begin recharging the shallow perched groundwater system. Post landfill monitoring will help to quantify this process, however, there is potential for ongoing pumping to be required to prevent impact to receptors in potential hydraulic contact with the landfill.

The results presented in Table 7 represent the results for a ‘best case’ landfill filling rate. These results were found not to change significantly under worst case conditions.

Table 7: Surface Water & Leachate Generation Estimates (ERM, 2008)

	Surface Water Inflow (m^3/day)	Leachate Generation (m^3/day)	Total Inflow (m^3/day)
Minimum	209	45	254
10th Percentile	238	119	357
Average	385	241	626
90th Percentile	507	374	881
Maximum	1,003	872	1,875

Peak Leachate Generation

The leachate storage/injection trench for the new tipping area should provide sufficient storage capacity to deal with a 1 in 20 year ARI 24-hour storm. The potential volume of leachate or contaminated run off volume generated during such an event is given in Table 8.

Table 8: Peak Leachate Generation, Active Tipping Face (4,000 m^2)

	1:20 year ARI (m^3 /day)	1:20 year ARI (L/s)
Rainfall Intensity	9.27 mm/hr	9.27 mm/hr
Volume Generated – Active Tipping Area	890	10.3

This indicates leachate generation of 890m^3 during a 1 in 20 year 24-hour storm event, with a flow rate of 10.3 L/s. A leachate collection trench should be designed with a capacity of 890m^3 minus the effective infiltration rate.

Should the capacity of Leachate collection system become inadequate, a longer trench lengths will be constructed to aid in infiltration.

3.1.3 Uncertainty in the Water Balance Calculations

The recalibrated water balance for LHBC is considered to be reliable, and the similarity in results between the current version and the previous calibration gives an additional degree of confidence. There is inevitably a degree of uncertainty in such assessments, and the main areas of uncertainty for this site are:

- Groundwater inflow;
- Leachate storage (note that this does not apply to the long term average leachate generation estimates); and
- Proportion of infiltration.

Recalculation of the water balance on an annual basis has been suggested, and this approach should be adopted to allow regular refinement of the process and to allow changes in site conditions to be taken into account.

3.1.4 Haul Road Stormwater Collection System

As indicated previously, it is proposed that the Haul road from the lip of the Quarry to its base will be graded with a fall towards the perimeter quarry wall at the base of which will be formed a dish drain. Further details are outlined on page 69 of the EA (ERM, 2008).

The stormwater pond will receive stormwater runoff from the dish drain on the haul road.

The volume flowing from the dish drain to the stormwater pond will be estimated using a “V” notch weir and water level logger and total potable water use is also measured, but the groundwater component will only be able to be measured during dry weather.

Aspects of the monitoring programme are aimed at verification of base data for the SWLMP and quantification of leachate that will be required to be discharged from site (a requirement of the site development approval).

4. Erosion & Sediment Control Plan

The planning condition for the erosion and sediment control plan states:

The erosion and sediment control plan must:

- be consistent with the requirements in the latest version of Managing Urban Stormwater: Soils and Construction (Landcom);
- identify the activities on site that could cause soil erosion and generate sediment; and
- describe what measures would be implemented to:
    - minimise soil erosion and the transport of sediment to downstream waters, including the location, function and capacity of any erosion and sediment control structures; and
    - maintain these structures over time.

The following subsections of the report seek to address the above planning condition. The information primarily derives from reports prepared previously by others and from information in Managing Urban Stormwater: Soils and Construction 4th Ed. (Landcom, 2004) – the Blue Book.

Activities on the site that could cause soil erosion and generate sediment during the construction have been identified in the Consolidated Stormwater Management Plan (Martens, 2011) presented in Appendix C. The report outlines the measures to be implemented to minimise soil erosion and the transport of sediment to downstream waters, including the location, function and capacity of any erosion and sediment control structures and to maintain these structures over time.

Furthermore, erosion and sediment control measures are inextricably linked to the general Surface Water Management Plan presented in Section 5. The erosion and sediment control plan is therefore primarily covered under the Surface Water Management Plan in Section 5.

4.1 Construction Phase

In summary, during the construction phase of the project the following sediment and soil erosion management measures will be implemented:

- Prior to major surface disturbance graded contour drains, diversion channels, catch drains, sediment traps and basins will be constructed in order to allow water flows to pass through the disturbed areas without mixing with unfiltered run off from the disturbed areas;
- Silt fences and hay bales will be installed where required downstream of disturbed areas, base of embankments, existing drainage lines, earthworks stockpiles;
- All vehicles exiting the site will, if required, travel through a wash down area to limit tracking of dirt;
- Exposed construction areas will undergo regular wet downs to limit sediment erosion and aid in dust suppression. Construction areas include but are not limited to embankment and excavation areas, stockpile areas, site facility and storage areas and temporary work areas;
- On going earthworks will be protected by temporary berms and drains to prevent the scouring of unconsolidated earthworks;
- Where prompt revegetation cannot be completed, implement erosion control measures including silt fencing until revegetation is completed;
- Sediment loaded water may be treated (flocculation) at stormwater sumps before discharge to detention basins.
- Velocities in drainage system will be limited by implementing sediment barriers in order to minimise possible scouring and to encourage precipitation of particulates in run off;
- Access track will be provided where practicable, along the toe of embankments to allow access for maintenance;
- Vegetation will be maintained in and adjacent to drainage lines;
- Pits and sumps will be cleared of silt build up following large storm events;
- Sedimentation basins will be kept in a drawn down state by preferential use of the water quality if required; and
- Wash out concrete delivery vehicles and wash down plant items a minimum of 20m from stormwater drainage systems and natural water course.

Other management measures:

- A detailed site inspection will be conducted after a significant rain event to confirm that erosion control safeguards are working effectively;
- Monitoring and testing of water quality if required.
- Inspection of silt fences regularly to confirm that they are not partially buried and still in good condition
- Conducting regular inspection of water management safeguards and complete checklist; and
- Fuelling and servicing all plant and equipment on a safe area away from any water course.

The proposed sediment and erosion management measures during the construction phase are depicted Drawing 2, Appendix A. Temporary drains attaching to specific areas of construction are not shown as it will be assessed on a task basis and will change according to area of work. Blue Book (Landcom) diagrams are presented on Drawing 1, Appendix B and shows the construction methodology for proposed sediment and erosion management measures.

5. Surface Water Management Plan

The planning condition for the stormwater management scheme (surface water management plan) states:

The stormwater management scheme must:

- Be consistent with the guidance in the latest version of Managing Urban Stormwater: Council Handbook (DEC, 1997); and
- Include the detailed plans for the proposed surface water management system.

The following subsections of the report seek to address the above planning condition. The information primarily derives from reports prepared previously by others and from information in draft Managing Urban Stormwater: Council Handbook (DEC, 1997) and Managing Urban Stormwater: Soils and Construction 4th Ed. (Landcom, 2004).

5.1 Background

The Consolidated Stormwater Management Plan (Martens, 2011) is presented in Appendix C. The report outlines plans for the proposed surface water management system. Key aspects of the report are reproduced herein.

Part of the analysis required for successful development of the Resource Recovery Facility (RRF) and Landfill Facility includes planning of surface water management for the site. As water is both an input and output (waste product) of site activities, site planning needs to adopt an integrated approach to water management.

The key issues concerning site surface water (stormwater) management comprise:

- Segregation and management of ‘clean water’ (water from operational areas) and ‘dirty water’ runoff (i.e. leachate), or
- Water that has come into contact with mixed wastes, green and timber wastes and uncovered landfilled wastes);
- Erosion and sediment control including protection of the drainage system from sediment influx;
- Quarry pit/haul road water management;
- Water quality control; and
- Provision of adequate on-site detention for the proposed operations.

Additionally, the Precinct Plan and Engineering Guide to Development require that pipe sizes be based on a 20 year ARI design flow and that the major drainage system be designed to safely convey the critical 100 year event under normal operating conditions.

Surface runoff generated on-site will fall into two categories:

- 'clean water' (not leachate) – available for reuse (following roof water collection in rainwater tanks or runoff from clean operational areas which may require treatment for sediment only), and
- 'dirty water' (leachate) – generated from the base of the landfill, green waste areas and run off that has come into contact with mixed wastes, green and timber wastes and uncovered landfilled wastes.

Given the recent and impending changes to climate (including pronounced drought conditions), it is intended that the site remains as independent as possible of external water sources, and that the potential for off-site impacts to local receiving waters is minimised.

5.2 Delineation of Surface Water Catchments

Surface water management of the site is based on the principle of separation of the site into different areas, according to the activities undertaken in each area and the treatment/disposal requirements for surface water runoff arising from these. Table 9 describes the proposed land uses and water management requirements for the Sectors nominated by LHBC.

Table 9: Land Use and Water Management Requirements

Land Use	Water Management Requirements
Sector A: Area 1 - Clean operational areas (hard fill sorting/processing) including roads, car parks aprons and building surrounds	Discharge to stormwater with sediment control and monitoring
Area 2 – Green waste areas / MPC work floor	Grading for surface water towards sump for either reuse in recirculation of green waste or to treatment plant prior to discharge to sewer
Sector B: Area 1 – Areas subject to intermediate cover and other inactive areas	Temporary capping or final capping Reuse for dust suppression or discharge through to stormwater
Area 2 – Active tipping area – consisting of daily active face and movement of work face with daily cover (leachate generation)	Treatment and/or discharge to trade waste system

At commencement of operations there will be only two main areas: the clean operational areas draining to the stormwater system, and the "dirty" area comprising the active tipping area.

During the first stage and first lift of waste (10m) there will be insufficient depth for creation of a clean stormwater pond in Sector B and all water collected in the base of the pit during this period will be treated as leachate (reference EA section 3.4.6 (ERM, 2008)).

After the first ten metres of lift there is expected to be a sufficient depth of landfilled material in order to create the Storm Water Pond.

Site Area Separation
The division of the site into Sectors with clearly defined water control systems aims to achieve the following:

- Minimise leachate generation by preventing clean water entering the active tipping area (Sector B, Area 2); and
- Prevents pollution of stormwater by ensuring that run off from the processing area, roads and car parks (Sector A, Area 1), and the inactive parts of Sector B, are differentiated by clear physical barriers and that the water are appropriately managed so as to meet compliance with stormwater standards.

Where appropriate, particularly in Sector B earth banks or drainage gullies used for delineation of clean and dirty areas, will be constructed with maximum batter grades of 2(H):1(V), and a minimum height or depth of 300mm.

Earth banks at the perimeter or lip of the Quarry are to be created by the placement of compacted road base placement of a geotextile, stabilised by compaction and followed by establishment of grass cover.

Earth bank construction is shown in an excerpt from the “Blue Book” (Landcom, 2004) presented in Appendix B.

5.3 Soil and Water Management

5.3.1 General

Site soil and water management will be required throughout the life of the project. The SWLMP will adhere to the following principles:

- It is proposed to direct all operational area (hardstand clean) surface runoff (excluding water managed within the quarry pit) towards the “Quarry” catchment (note this reference is a Blacktown City Council (“BCC”) designation and does not refer to the Pit);
- Sediment-laden stormwater from the materials stockpile area will be directed through permanent sediment capture sumps or mini-basins along surface drainage to intercept sediment prior to reduce sediment ‘slugs’ reaching the GPT. Site grading is to be used to direct sediment-laden drainage away from hardstand areas;
- The MPC work floor and green waste areas will be diverted to sewer;
- Truck access to and from the unsealed areas are to be stable and designed to prevent influx of run-on and escape of untreated flows where possible;
- Runoff from site operational areas of the RRF is to be directed through treatment devices (sediment traps and low-flow wetland treatment) and OSD for reuse prior to release to the site’s

drainage network. Overland flow paths for flows in excess of the design event are to follow natural drainage lines to the west of the site;

- Treatment devices around the site would provide sediment capture, gross pollutants where necessary, and must also be capable of capturing oil and fuel spills. Proprietary devices such as CDS, Humeceptor or similar can be selected and designed in consultation with the manufacturer to accommodate the required treatment;

The treatment devices proposed for soil and water management are:

- Small sediment sumps or mini-basins along swales to trap sediment ‘slugs’ if entrained in stormwater flow;
- Sediment traps, e.g. proprietary gross pollutant trap (GPT) (i.e. CDS) or baffled settlement tank capable of retaining gross pollutants, sediment, oils and grease;
- Within OSD basin: allowance for wet storage component, as a low-flow wetland for low-flow water quality treatment to remove fine suspended sediments as well as nutrients; and
- Energy dissipation in the OSD basin settling basin for pre-treatment before entry to the OSD basin will provide further attenuation and capture of sediment that may reach the detention basin.

5.3.2 Stockpile and Green Waste Area

Sediment controls installed within the materials stockpile area will be maintained to prevent clogging and to prevent excessive sediment and nutrients entering the drainage system. These controls are to include:

- Small sediment sumps or mini-basins along swales to trap sediment ‘slugs’ if entrained in stormwater flow. Treatment through a GPT or baffled sediment settlement underground tank at the drainage outlet of these two areas,
- Protection of drains within these areas using:
- Vehicle exclusion areas;
- Stabilisation or lining of drains;
- Check-devices such as sediment sumps or mini-basins approximately every 50 metres to attenuate flows and encourage sediment dropout;

Regular maintenance of drains and sediment traps will be undertaken to reduce loads within the system. Runoff within the MPC work floor / green waste collection area is to be managed as leachate.

5.3.3 Resource Recovery Facility

Surface runoff from the operational areas of the (Resource Recovery Facility) RRF at surface will be managed separately from runoff generated in the quarry pit and haul road.

Sources of stormwater runoff from the operational area include:

- Building roofs – workshop, MPC/ WTS, administration building and weighbridge shed – clean;
- Roads, car parks and other hardstand areas – clean, containing sediment;

- MPC work floor/green waste stockpiles – dirty (to be directed to sewer);
- Materials stockpiles/drop off zones– clean, containing sediment.

Vehicle entry points for MPC work floor, green waste and materials stockpile / drop-off areas are to be located to minimise uncontrolled runoff and sediment release outside these areas. Overland flow paths around the site are to remain stable in 100 year critical flows.

Runoff collected from the clean or sediment-only areas will be reused on site, for uses including building internal uses (toilet flushing), wheel wash facility, dust suppression (via water carts) and irrigation/dust suppression from sprinkler systems around the site.

Run off from areas (other than the green waste areas and the MPC floor) of the RRF and stockpile/drop-off zones is considered to be “clean operational waters” but runoff from these areas will be subject to treatment (sediment removal) prior to reuse.

Clean runoff from roofs will be primarily collected in rainwater tanks for reuse on-site.

Runoff from other parts of the operational area (e.g. roads, open areas away from stockpiles and buildings) will also be considered clean runoff and suitable for treatment and reuse on-site. This water will be directed to the OSD basin.

Stormwater runoff will be conveyed by a combination of major and minor drainage systems, as shown in Appendix B, including:

- An underground piped system with provision for overland flow in swales and along roads;
- Stormwater detention and pollution control structures, and
- The natural drainage systems including creeks and overland flow.

Blacktown City Council requirements are that piped networks are designed to convey 1 in 20 year flows without surcharge. Drainage overflows (greater than 1 in 100yr flows) from both these areas will be discharged away from the quarry pit via overland flow paths.

5.4 Stormwater Discharge Arrangements

Stormwater discharge arrangements proposed by LHBC are as follows:

If required water retained in OSD1 and OSD2, it may be pumped back to the 250KL tank holding capacity for use around the site in dust suppression measures.

The quality of the water released (if any) should be in accordance with the site’s Environment Protection Licence. Typically the licence will only permit discharge once the water in storage has been tested to ensure it complies with specified water quality standards for discharge.

Water quality monitoring from OSD1 and OSD2 is proposed to be carried out after a rain event or at intervals required by OEH and to ensure Compliance with Council’s policy for a suite of indicator parameters (including ammonia). The monitoring requirements are discussed in Section 6 of this report.

5.5 Peak Stormwater Generation

The stormwater management system for the site will be designed to deal with runoff generated under a range of rainfall conditions.

Volume 2B of the 'Blue Book' for Waste Landfills recommends that stormwater drains and storages be designed to ensure separation of clean stormwater from water that has come into contact with waste, and that surface water collected from cleared, non-vegetated areas be treated in accordance with stormwater guidelines.

Council guidelines require post-development peak flows to match pre-development peak flows up to the 100yr storm events. The model was run for the 2 year, 10 year and 100 year ARI storm events for 25 to 540 minutes to derive the required OSD volumes.

DRAINS software modelling allows the user to optimise OSD volume requirements. A 1 in 2, 10 and 100 year ARI storm with durations of 25 minutes to 9 hours were modelled by Martens (2011) to check discharge calculations for peak flow hydrographs.

Assumptions
The operational area (including berms) was modelled in DRAINS and incorporated an area of 19.44 ha. The operational area was divided into two separate catchments to reduce the total anticipated basin size.

OSD 1 catchment is the northern section of the operational area with a modelled area of 10.34 ha.

OSD 2 catchment occupies the southern section of the operational area with a modelled area of 9.10 ha.

The catchments were considered to be 100% pervious in the pre-development model and 13.80 ha was considered 100% impervious post-development.

These assumptions would result in conservative estimates for flow and OSD storage requirements.

Results
Peak flows from the site operational areas were calculated by Martens using DRAINS for the pre-development and post-development scenarios.

This was used to calculate the required OSD storage volume to prevent downstream hydraulic impacts as a result of site development and allow matching of pre- and post-development flows off site. Table 2 and 3 (Martens, 2011 (page 13)) shows the results of peak flow modelling.

5.6 On-Site Detention Basin (OSD) Storage Volume

Based on the OSD modelling results presented in Martens Report (Appendix B), the OSD basin storage volume of 6,000m^3 is sufficient for the proposed operational area.

5.7 Dam Safety Committee Requirements

The New South Wales Dam Safety Committee (DSC) *Risk Management Policy Framework for Dam Safety* (2006) has been reviewed for requirements and criteria for risk assessment. Among other goals, the DSC states that its mission is to develop and implement effective policies and procedures for regulation of dam safety. In general, dam safety is initially determined through a risk assessment that uses the probability of failure per dam in one year (with probabilities ranging from 10^{-7} to 10^{-3}) and the number of fatalities that would occur as a result of dam failure.

For this site, the proposed OSD basin sizes are 3,400m^3 and 2,600m^3, which is smaller than several of the existing dams at the Eastern Creek Precinct. Generally basins will be constructed so that maximum water levels will be at most one metre above existing downstream ground levels, overland flow travels across rural land towards Ropes Creek.

Flows from either basin could be classed as “slow and shallow” in relation to overland flow paths, non-defined drainage lines allowing flow dispersion, and relatively long overland flow paths over un-occupied land to the nearest defined drainage line.

In a Probable Maximum Flood the dam will have already overtopped from a smaller 1:100 event as part of its design. In a PMF event, the volume of catchment flows from further up the Ropes Creek catchment, beyond the site, are likely to be having a greater impact at this point in the catchment, in which the contribution of any (unlikely) dam failure would be negligible.

As a result, these factors contribute to a negligible risk and the Dam Safety Committee has confirmed that the OSDs do not need to be prescribed.

5.8 Groundwater and Stormwater Reuse

Primary dust suppression will be carried out by the operator using a network of:

- Spray mists and sprinkler systems for crushing, grinding and chipping operations;
- Spray mists on all material stockpiles;
- Spray mists and sprinkler systems on the perimeter berms
- Wetting of vehicles with potentially dusty loads, prior to unloading
- Wheel wash for all vehicles travelling off site
- Water carts operated as required;
- Use of onboard reservoirs on site dump trucks to allow wetting whilst in motion

The spray and sprinkler systems are supplied with potable water via five storage tanks each of 450kL capacity located next to the green waste area.

Captured clean groundwater and stormwater are stored in these storage tanks, and reused on site for manual dust suppression (using water trucks) and materials processing. During dry periods the system will be topped up with mains water.

5.9 Stormwater Drainage and Sediment Control

Effective stormwater drainage of all areas is required to minimise flow into the quarry pit and thereby reduce the potential for infiltration through the landfill cap into the waste mass, and to prevent ponding of stormwater, which could impact on landfilling operations.

To meet these requirements, capping and drainage measures have been implemented as outlined below.

Sector A
The processing area will be provided with a compacted surface, or concrete/tarmac or a bitumen concrete mix as appropriate. In addition, the surface will be laid with adequate falls to ensure effective shedding of stormwater. A minimum target fall of 1% has been applied.

Stormwater runoff from this area drains to one of two drainage systems:

- Perimeter drains and sumps will be constructed to convey stormwater from the clean operational area;
- These will comprise concrete dish drains along the north-south section, leading to sumps and from there an underground pipe directing stormwater to GPT1 for transfer to storage tanks or discharge to OSD1 (north);
- Overflow from the sump at GPT flows westward into an overland swale which will be constructed with a target fall of 1% to 5% and which is lined with geofabric, clay and rocks to limit infiltration;
- Sedimentation ponds and check dams will be constructed at intervals to control flow and encourage settlement of suspended solids, in accordance with Blue Book guidelines (Landcom, 2004);
- Sediment control measures will also been constructed around the stormwater discharge point, and will comprise a check dam and a double layer of geotextile-wrapped filter bales;

Excerpts from the Blue Book showing construction details of drainage and pollution control measures are provided in Appendix B.

Monitoring of stormwater quality at OSD1 (north) and OSD2 (south) is recommended to be undertaken during rainfall events, with a target of four monitoring events per year for the first year of operation, to ensure that sediment control is adequate.

Sector B
Inactive areas of Sector B are to be capped (temporary capping), and runoff directed to the stormwater pond in Sector B. The active tipping area and the inactive capped areas are to be separated by compacted earth banks.

After the first lift of 10 metres in which all water into the base of the pit will be treated as leachate the site will normally operated with an active tipping face area of approximately 450 - 1000 m^2, which is to be capped each day.

Until the first ten metres depth of filling is completed the area taken to leachate generating is treated as being 4,000 m^2. Thereafter the 'open' or exposed active filling area prone to generation of leachate will be not more than 4,000 m^2 per month.

Sector B is to be contoured to prevent ponding of stormwater in the 'dirty' tipping areas and to direct surface runoff to the stormwater pond at the north-eastern end of the Sector or as changed over time based on landfilling direction.

The surface of filled areas in Sector B which is not to be active for more than three months will be capped with compacted clean fill material to minimise infiltration and allow all-weather vehicular access.

Stormwater is to be collected in the stormwater pond, which also acts as a sedimentation pond. Stormwater from the pond will be pumped to the stormwater drainage system in Sector A for disposal and/or storage for on-site reuse as required.

Due to the proposed landfill location being within the existing quarry pit, sediment control per se of the landfill area is not essential as the risk of environmental damage from sedimentation is low within the quarry pit itself for those areas where water reuse does not take place. Rather, the primary aim of a collection basin within the quarry pit is to assist in controlling the volume of stormwater runoff that comes into contact with waste or the active landfill area (hence minimising leachate generation). Reuse of this water was also reviewed in a water balance model for its ability to meet demand for dust suppression, to maximise reuse potential.

Volume 2B of the 'Blue Book' for Waste Landfills states that sediment basins and water storages should not be located on landfilled areas. However, the unavoidable constraint of being within the quarry pit, and the need to manage runoff effectively within the pit, necessitates the use of temporary stormwater controls and storage within the quarry pit.

The use of suitable grading and bunding and inclusion of a leachate trench to separate leachate from stormwater from capped areas within the landfill is also necessary to minimise surface water flows into active landfill areas. Erosion across capped areas and sediment influx into any temporary storage at capped areas must also be accommodated.

Forward planning for the location and size of the basin is important for effective runoff and sediment control. Its location should be determined at the development of each landfill lift, taking into account that a sealed basin area is necessary to prevent infiltration, and that it is not possible to excavate through capping and back into landfilled materials.

Initial shaping or grading of capped / covered areas is necessary to allow for a suitable placement for the basin to create a catchment with a low point designed into the intermediate capped areas, to drain away from the active tip face / daily cover areas and allows placement of a liner for a basin without disturbing existing capped material.

Basin Sizing

Basin calculations were undertaken in accordance with the Blue Book for the quarry pit (26.5 ha). The maximum total basin volume based on the total quarry pit footprint (including settling zone and sediment zone) that may be required is approximately 4,362.5 m^3 which equates to 165 m^3 per hectare of catchment area, which may include quarry walls that drain into the pit.

Assumptions and spreadsheets used for sediment basin sizing including rainfall percentiles are presented in Appendix B of the Storm Report (Appendix C) and include the use of 5-day, 80th percentile rainfall and 2-month sediment accumulation.

Sediment influx can be reduced by including a controlled, stabilised inlet to the basin and installing and maintaining effective erosion controls around the haul road outlet and around the boundary of the basin.

A series of basins may be installed to capture flows from sub-catchments of the quarry depending on available space within the quarry. The sub-basins will need to meet minimum storage requirements of 165 m3/ha of catchment draining to each basin.

Based on the basin sizing assumptions used, drawdown of water within the basin would need to occur within 5 days of a storm event occurring, to follow the basin design requirements and also to minimise the time that water is stored at the landfill area.

Water collected in the basin should be used initially for in-pit dust control or other uses requiring water in the pit area. Basin(s) may be drawn down by the water carts for dust suppression purposes or used in dump truck on-board reservoirs.

Guidelines for the construction of the stormwater basin within the landfill area

The designated area covers approximately 4362.5m^3. The proposed outline design comprises wet earth basins: a relatively small first pond to allow preliminary settlement and/or filtration of coarse sediments; followed by a full-size main pond to allow full settlement.

The main objective of the preliminary settlement pond is to reduce clean-out requirements for the main pond. The typical design and construction of a wet earth basin is shown on the excerpt from the "Blue Book" presented in Appendix B.

The basin will be lined using a seam welded HDPE membrane to prevent infiltration of stormwater into the subgrade and underlying waste mass. Discharge of water from the basin is to be by pumping from a floating pump intake after field testing to confirm acceptable water quality.

The sizing of the preliminary basin is considered not critical, but should be as large as is practicable. Construction of any HDPE lined basins should be undertaken as follows:

- Design of the appropriate geometry and dimensions;

- Placement of a protection layer comprising uniform compacted fill, with the need for a geotextile protective layer considered depending on detailed ground conditions;
- Placement of the HDPE liner and welding of the sheets, with appropriate post construction inspection and quality assurance to ensure suitability of the final liner; and
- Placement of a protective layer as required, such as a sacrificial coarse sand layer and/or a geotextile. This layer is designed to minimise the risk of damage to the HDPE membrane during sediment removal, and is particularly important for the preliminary pond.

Construction of any compacted clay lined basins should be undertaken as follows:

- Design of the appropriate geometry and dimensions;
- Sourcing of clay suitable for engineered compacted clay liner;
- A maximum 300mm thick, loose lift is recommended. A minimum of 95% standard compaction is recommended. Soil placement with a density of less than 95% of standard compaction should be rejected. No more than 5% of density values can be below 95% as long as these failures are not concentrated in one lift or in one area. Moisture content should be maintained at -1% to +3% of optimum moisture content (OMC).
- The liner shall be placed with a scraper-pan or trucks and then distributed with a dozer or grader or equivalent.
- The successive lifts shall be compacted with a sheepsfoot roller. The sheepsfoot roller studs shall be sufficiently long to fully penetrated the loose layer and knead successive lifts. Compaction equipment weighing 17 tonne or more is recommended.
- Appropriate post construction inspection and quality assurance is recommended to ensure suitability of the final basin liner.

Basin Operation

The main stormwater basin is designed to provide containment of storm events, and as such should be maintained empty or with a low water level. Stormwater should therefore be pumped to the discharge point after each storm event or reused on site, after sufficient time has passed to allow settlement and testing.

Regular cleaning-out of sediment is required to prevent excessive build-up, and gauges should be installed to allow monitoring of sediment accumulation and to allow ready identification of the top of the sediment storage zone. The effectiveness of the sedimentation pond system at achieving the required water quality is depended on a number of factors, including the flow rate, sediment size distribution and dispersibility of sediments.

Use of flocculants may be considered should sediment settlement be inadequate or unacceptably slow. These include gypsum, which is relatively insoluble and must be dosed across the entire water surface to be effective, polyaluminium chloride, and proprietary block flocculants.

Occasions are likely to arise where the pond becomes full prior to adequate settlement being obtained. Under these conditions, excess stormwater will need to be discharged to the leachate drainage system until conditions are suitable for the stormwater pond to be emptied. Adequate buffer storage capacity should be maintained in the leachate drainage systems and waste mass to allow for this eventuality.

Additional Sediment and Erosion Control Measures
Prevention of sediment generation is the most effective method of stormwater quality control, and provision of erosion and sediment control measures across the catchment area is therefore a critical aspect of stormwater management. Control measures include silt fences, vegetation of inactive areas and maintenance of appropriate surface gradients.

6. Surface Water, Groundwater & Leachate Monitoring Programme

The planning condition for the surface water, groundwater and leachate monitoring programme states:

The surface water, groundwater, and leachate monitoring programme must:

- be generally consistent with the guidance in Benchmark Techniques 4, 5, 6, 7 and 8 of Appendix A of the EPA *Environmental Guidelines for Solid Waste Landfills* (1996, or the relevant sections of the latest version of the guideline); and
- include:
  - baseline data;
  - details of the proposed monitoring network; and
  - the parameters for testing and respective trigger levels for action under the surface water, groundwater and leachate response plan (see below).

The following subsections of the report seek to address the above planning condition.

6.1 Monitoring Plan - General

Monitoring would be implemented to assist in verification of the current water management plan and future water balance calculations, including the following:

- Leachate volume pumped from the sump and discharged to sewer (monthly);
- Leachate quality discharged to sewer (every 22 days);
- Volume of groundwater and clean operational water pumped out from within the pit (monthly);
- Leachate levels within the main sump (daily);
- Groundwater levels in the piezometers surrounding the quarry (quarterly);
- Groundwater quality from piezometers (quarterly);
- Stormwater quality for reuse on site (4 rain events per annum).

The monitoring programme frequency is summarised in Table 10.

Table 10: Proposed Water Management Monitoring Programme

Monitoring Type	Frequency
Leachate volume pumped from sump/discharged to sewer	Monthly
Leachate quality monitoring	Every 22 days according to Sydney Water requirements
Leachate level in sump	Daily, reported on a quarterly basis
Clean operational water within the pit/ discharged volume	Monthly
Groundwater boreholes level	Quarterly
Stormwater quality (run-off and groundwater) reused on site	Four rain events per annum
Groundwater quality	Quarterly
Surface water quality (Ropes Creek)	Quarterly

6.2 Groundwater Monitoring Programme

Benchmark Techniques No. 4 and 5 (BT4, BT5) outline the requirements for a groundwater monitoring network and monitoring programme. The groundwater monitoring programme should effectively monitor and report groundwater character, and ensure early detection and reporting of possible pollution of groundwater.

A comprehensive hydrological investigation of the site and the surrounding groundwater regime has been conducted and is presented in Appendix D. DP has also noted the findings of the peer review of the IGGC hydrogeological reports by Dr Boyd Dent of Red Earth Geosciences (Red Earth Geosciences, 2009; 2010).

Quarterly monitoring will utilise the existing monitoring well network comprising nested piezometers presented on Figure 5.2 of the IGGC report (Appendix D). A suitably qualified environmental consultant must complete all groundwater monitoring and reporting.

Groundwater levels will be recorded in the piezometers surrounding the quarry using an electronic dip meter or whistle. The groundwater monitoring consultant should give consideration to the piezometer depths and analytes (e.g. oxygen sensitive analaytes) when selecting the groundwater sampling method. The sampling method should be consistent for successive monitoring events to maximise data comparability.

A set of groundwater environmental indicator parameters for groundwater quality are presented in Table 11. It is noted that the actual parameters required under the routine monitoring programme will be stipulated on the Environment Protection Licence and may vary slightly from the table below. The parameters in the table below are extracted from BT5 (EPA, 1996).

Table 11: Indicator Parameters

Chemical Determinand	Analytical Detection Limit (µg/L)
Electrical Conductivity	1 mS/m
Ph	0.1 pH unit
Redox Potential	1 Eh
Temperature	0.1
Adsorbable organic halides (AOX)	10
Alkalinity	1000
Ammonia	50
Calcium	5000
Chloride	5000
Fluoride	500
Iron	500
Manganese	50
Magnesium	5000
Nitrate	100
Total phenolics	50
Potassium	5000
Sodium	5000
Sulphate	5000
Total organic carbon (TOC)	50

Quarterly groundwater sampling must include adequate field and laboratory quality control samples including duplicates, trip spikes, trip blanks and equipment rinsate samples.

6.2.1 Establishing Baseline Data

Establishment of baseline groundwater quality and baseline concentration ranges prior to the commencement of landfilling activities at the site is required.

Two groundwater sampling events were completed at the site by ERM. The initial sampling event was completed between 7 and 9 November 2007 and 22 and 23 November 2007. The second sampling event was completed between 20 February and 30 March 2008.

It is considered that the initial rounds of groundwater monitoring will also be suitable for inclusion into a baseline dataset. The determination of what data is suitable to retain in the baseline dataset should be made by the monitoring consultant (hydrogeologist). It is anticipated that at least data from the initial two quarterly monitoring events would be suitable to use in the baseline dataset.

The monitoring consultant will need to tabulate the previous ERM data in addition to subsequent data deemed suitable for inclusion in the baseline dataset. It would be prudent to collect as much quarterly monitoring data prior to commencement of landfilling activities as practicable to establish a robust baseline dataset that includes seasonal variability.

6.2.2 Reporting of Groundwater Monitoring

Groundwater data will be summarised, graphed and interpreted to determine trends, assess impacts against the baseline water quality at the site, and evaluate any exceedances of relevant published guideline values (e.g. ANZECC (2000)).

Trigger levels for action should be based on the variance from baseline range concentrations (yet to be established) with respect to naturally occurring groundwater constituents. Trigger values for anthropogenic contaminants (e.g. AOX or total phenolics) should be based on detection of the contaminants in groundwater above the analytical detection limit, as this may indicate that leachate is impacting the groundwater. Further comparison of detected concentrations against relevant published guideline values (e.g. ANZECC (2000)) should be undertaken in order to evaluate the overall risk posed by naturally occurring and anthropogenic contaminants in the groundwater.

In the event of the monitoring consultant (hydrogeologist) detects a possible failure of the leachate containment system, a groundwater assessment programme should be established to determine the extent of that failure. This would form part of a groundwater action plan or water contamination remediation plan as required under Benchmark technique No 9 (BT9). The formulation of the action plan will depend on the nature and extent of the groundwater contamination. Further information on a groundwater response plan is presented in Section 7.2 of this SWLMP report.

6.3 Surface Water Monitoring Programme

The surface water monitoring programme must be able to demonstrate that surface water is not polluted by the landfill. Surveyed monitoring points will be established at the entry and discharge points of the OSD basins. The locations will be monitored a minimum of four times per year during a rain event.

If the surface water monitoring programme detects water pollution, the occupier should follow the procedures outlined in the Water Contamination Remediation Plan to investigate surface water pollution.

Stormwater from the clean area of Sector A and the clean area of Sector B discharges via GPT1 and GPT2 to the OSD basins via swales.

The quality of the water released (if any) should be in accordance with the site Environment Protection Licence. Typically, the licence will only permit discharge once the water in storage has been tested to ensure it complies with specified water quality standards for discharge. The water quality criteria in Table 12 are suggested based on the requirements of the draft OEH site licence and on ANZECC (2000) criteria.

Table 12: Proposed Stormwater Quality Criteria for Discharge

Analyte	Unit	Proposed Criterion
Ammonia	mg/L	0.9[1]
pH	pH Units	6.5 to 8.5[2]
Dissolved Oxygen	% Saturation	80-110%[2]
Oil & Grease	mg/L	10[3]
Suspended Solids	mg/L	50[3]
Total Organic Carbon	mg/L	10[3]
Lead	mg/L	0.0034[1]
Phenol	mg/L	0.32[1]
Total Nitrogen	µg/L	350[4]
Total Phosphorous	µg/L	25[4]

Notes: 1: ANZECC (2000) Default Trigger Values, Toxicants
2: ANZECC (2000) Criteria for Environmental Stressors
3: Typical DEC discharge water quality criteria applied for industrial and/or landfill sites in Sydney
4: Blacktown City Council Water Quality and Quantity Monitoring Programme 2008 – 2012

The OSD basins, swales and GPTs will be inspected daily as part of the daily Site Environmental Inspection to be undertaken by the site Operational Manager. Sediment ponds must be maintained in a manner that ensures these retain an appropriate freeboard to minimise the potential for any turbid discharge. The health of the wetland plants must be maintained to ensure water quality control. Depth indicators will be installed and maintained within the OSD basins, that indicate the required freeboard to be maintained. If daily inspections reveal the build up of sediment, or that the health of the wetlands is failing, immediate maintenance will be undertaken. Otherwise, maintenance on the OSD basins will occur every three months.

All results of maintenance and monitoring will be available for inspection on site by the OEH, and will be reported in the annual report to the OEH in accordance with the Environment Protection Licence

6.4 Leachate Monitoring Programme

The leachate monitoring programme will involve three components:

- Monitoring of the leachate level in the primary (basal) sump;
- Monitoring of the leachate level in successive sumps;
- Monitoring leachate quality prior to discharge to sewer; and

- Monitoring leachate generation volumes to calibrate the leachate water balance.

Leachate level monitoring in the basal sump and the active successive sump will be done on a daily basis. Level monitoring will be used to evaluate the connectivity of leachate accumulation at the base of the landfill relative to perched leachate that accumulates in the active successive sump. It will also be used to monitor the proposed storage of leachate in the landfill (i.e. recovery of the leachate head to 70 – 80m above the quarry base.

Leachate disposal for the treated leachate is as trade waste. Industrial customers need to meet the conditions of Sydney Waters trade waste criteria. Monitoring of leachate water quality for discharge to sewer will be done in accordance with the requirements of the Trade Waste Agreement with Sydney Water.

Monitoring of pumped leachate volumes will be done using flow meters. The monitoring data will be used to further refine the water balance based on actual generation rates.

7. Surface Water, Groundwater & Leachate Response Plan

The planning condition for the surface water, groundwater and leachate response plans states:

The surface water, groundwater and leachate response plan must:

- include a protocol for the investigation, notification and mitigation of any exceedances of the respective trigger levels; and
- describe the array of measures that could be implemented to respond to any surface or groundwater contamination that may be caused by the development.

The following subsections of the report seeks to address the above planning condition.

7.1 Surface Water Response Plan

In the event of any identified contamination the following steps will be taken:

- The water will be re-sampled and retested as soon as possible;
- If the indication of contamination persists, the flow will be contained, i.e. the discharge point will be closed;
- OEH will be notified;
- A Water Remediation Plan, suited to the particular circumstances, will be put into place, to the satisfaction of Council and the OEH.

7.2 Groundwater / Leachate Response Plan

The primary objective of groundwater contaminated remedial plan is to ensure that the escape of leachate does not continue to contaminate groundwater quality following its detection. In order to achieve this objective, an individual plan relating to groundwater and surface water will need to be prepared upon detection of any anomalies in the groundwater quality.

As discussed in the groundwater and surface water management and controls that in the event of any identified contamination in groundwater the following steps will be taken. Initially the OEH will be informed within 24 hours of the exceedance and within 14 days in writing and steps will be taken to re-sample from the locations which showed the exceedance of the established environmental trigger levels.

Re-sampling results will determine if an adverse trend is developing, or whether the initial exceedances were isolated incidents or spurious readings. Once a trend has been established which indicates deteriorating groundwater quality then a suitable groundwater remediation action plan will be developed and notification of environmental harm made to the OEH.

Detailed plans cannot be provided until the nature of the problem has been identified. Proposals for voluntary groundwater remediation (i.e. the groundwater remediation action plan) will be forwarded to the OEH for agreement.

Results of the monitoring programme, details of any required action plans and implementation of the remediation programme and its results will be provided in the annual report as specified in the site license.

8. Conclusions on Soil, Water & Leachate Management Plan

This report describes the proposed soil, water and leachate management practices for the recycling premises and the landfill premises at the LHBC site. These are based on a variety of reports prepared by others. It includes stormwater management for the proposed separation of the site into clean operational (draining to the stormwater system or used in dust suppression) and leachate areas (draining to a leachate re-injection trench).

The estimated average leachate generation rate can be expected to vary between 45m^3/day and a maximum of 872m^3/day (ERM, 2008). However due to the storage capacity provided by the waste mass and the separation of the site areas for collection of clean water within the pit for the purposes of landfill mitigation measures e.g. dust suppression, the total leachate generation is expected to be at most 500m^3 per day. The proposed site leachate management and disposal system is considered adequate to deal with these volumes, and includes excess capacity and buffering storage to deal with higher short-term flows during wet weather periods.

Filling Plan

The current filling plan involves tipping in the active area of the landfilling premises. Filling and capping of filled areas is being carried out progressively. Filling at the Site is estimated to take over 20 years, based on proposed waste input rates. A new filling plan will be produced at least six months before filling is due to be completed.

Groundwater Monitoring Network

The proposed groundwater monitoring network is based on the hydrogeological conditions of the site. The proposed monitoring network is considered to be adequate to enable satisfactory monitoring of groundwater conditions and to determine the effects of the landfill site on the surrounding groundwater systems.

Hydraulic Containment and Potential for Leachate Migration

Operation at the site is proposed to be on the basis of hydraulic containment, whereby leachate is maintained at a level below that in the surrounding groundwater systems, in order to maintain an inward hydraulic gradient. The system will be based on the concept design proposed by Earth2Water comprising compartmentalised waste cells and partial landfill lining (i.e. a basal, followed by two intermediate liners with no side wall liner).

DP understands from Alexandria Landfill Pty Ltd that the OEH has suggested that leachate may be stored in the landfill provided leachate elevations are maintained below the base of the regional groundwater levels (to maintain an inward head gradient). Based on the regional groundwater elevations observed at the site, this would suggest a potential leachate elevation approximating to a level 70 – 80m above the pit base, which in turn suggests that there is potential from time to time to operate at pumping rates well below $500m^3$.

However, average pumping rates above $250m^3$/day are likely to be required to conservatively ensure an adequate inward hydraulic gradient towards the landfill. The proposed leachate treatment system, being able to treat double that amount, should be adequate to deal with this daily treatment and pumping average. Additional sequencing batch reactors (SBR) could be used to supplement the system based on the actual volumes of leachate generated if required.

Review of Data

The data gathered from the monitoring programme associated with implementation of this report should be reviewed on a regular basis. Changes to the overall system including monitoring requirements / frequency would be recommended based on the data reviews.

9. References

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ERM, 2008. Light Horse Business Centre, Volume 1 & 2, Environmental Assessment Report, Final Report.

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Martens, 2011. Consolidated Stormwater Management Plan, Light Horse Business Park, Eastern Creek, NSW

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

Red Earth Geosciences, 2010. Review of Additional Hydrogeological Investigations - Development of Disused Quarry at Eastern Creek, NSW.

Storm Consulting 2008. Site Surface Water Management Plan.

10. Limitation of this Report

Douglas Partners (DP) has prepared this report for this project at Light Horse Business Centre off Old Wallgrove Road, Eastern Creek, in accordance with DP's proposal SYD090902 dated 5 October 2009 and acceptance received from Christopher Biggs dated 14 October 2009. The work was carried out under DP Conditions of Engagement. This report is provided for the exclusive use of the Alexandria Landfill Pty Ltd for the specific project and purpose as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

Whilst DP has prepared this report, completing a detailed technical review of reports prepared by others was not part of DP's scope of work, and in this regard, DP is not formally endorsing the calculations, designs or recommendations provided by others (e.g. ERM, IGGC, Martens and Storm Consulting) summarised herein unless expressly stated.

This report must be read in conjunction with all of the attached notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

Douglas Partners Pty Ltd

About this Report

Douglas Partners

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Copyright

This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

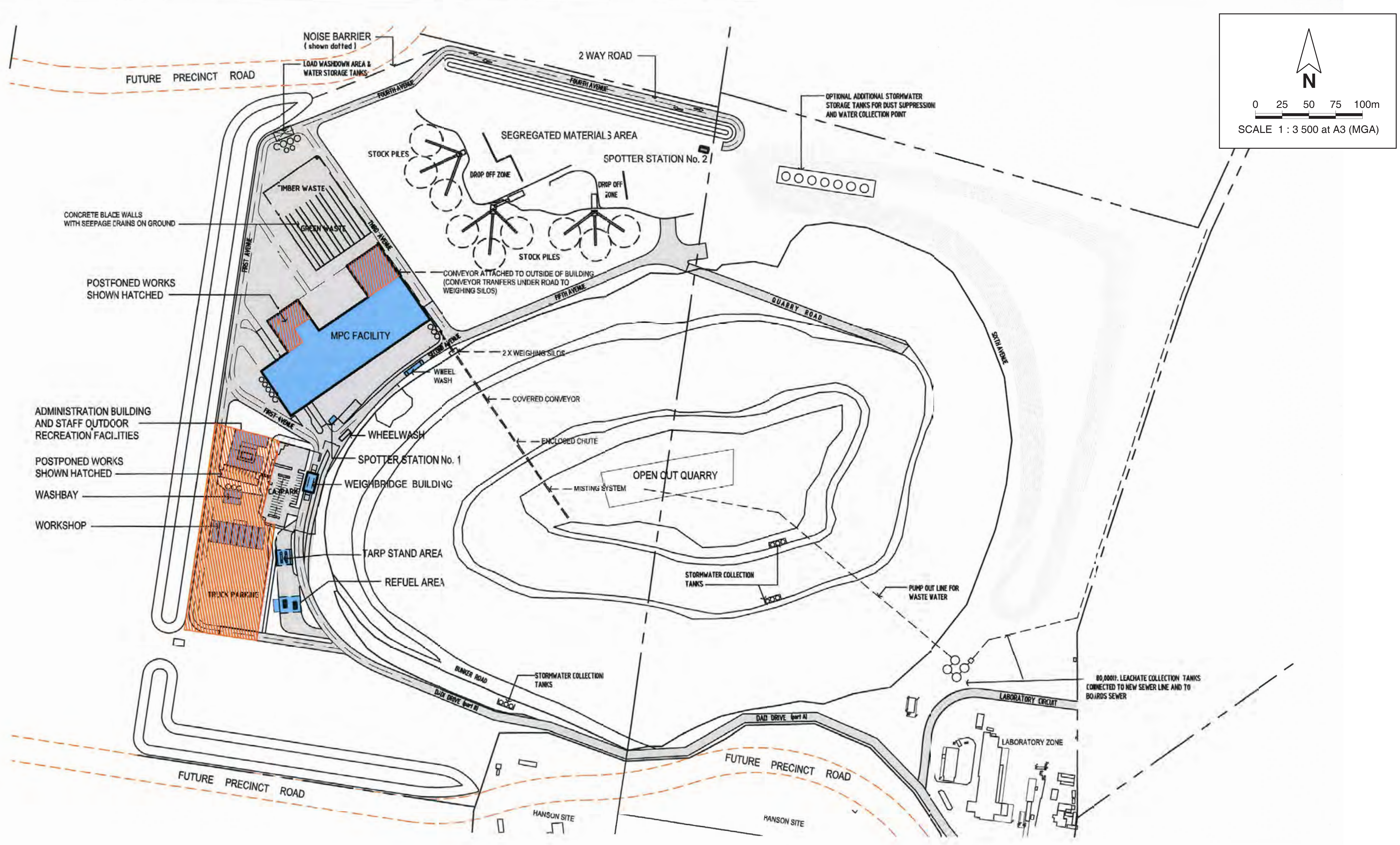
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Appendix A

Drawings



AXIS architectural

SOURCE: Axis Architectural, Dwg No. 090507/DI-EP-MP A101-8/A, March 2010.

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Brisbane, Cairns, Canberra, Darwin, Gold Coast, Melbourne, Minto
Newcastle, Perth, Sydney Sunshine Coast, Townsville, Wollongong, Wyong

Title: SOIL, WATER AND LEACHATE MANAGEMENT PLAN - LIGHT HORSE BUSINESS CENTRE, EASTERN CREEK, NSW

PROPOSED SITE LAYOUT

Client: ALEXANDRIA LANDFILL PTY LTD		Office: PERTH	
Drawn By: J. RUSSELL	Scale: 1 : 3 500	Project No: 46950	Revision: A
Approved By: --		Date: 10-10-11	**DRAWING 1**

Mon 10 Oct 11
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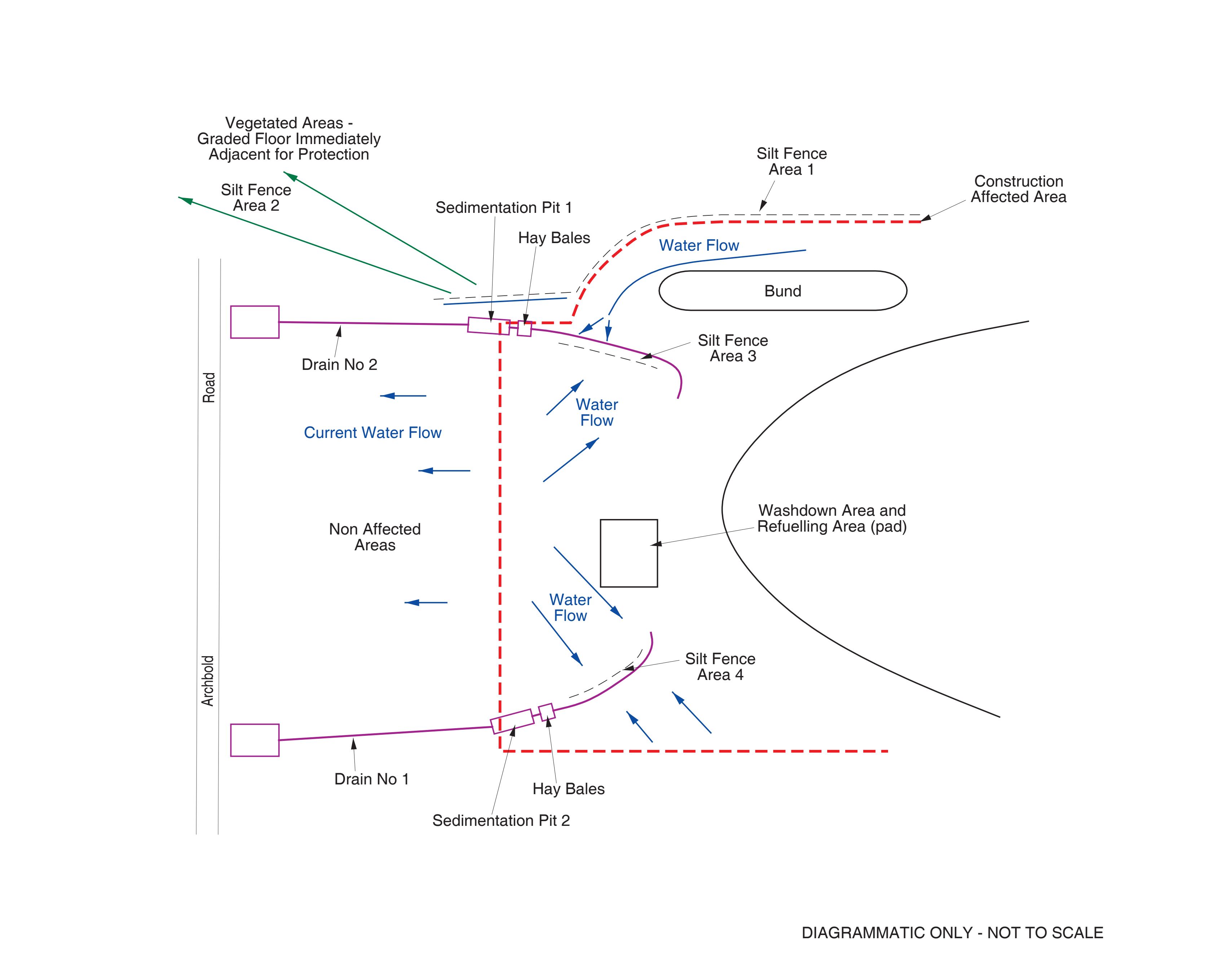
PINPOINT CARTOGRAPHICS (08) 9277 7763 46950-SWLMP-f02.dgn Wed 03 Feb 10



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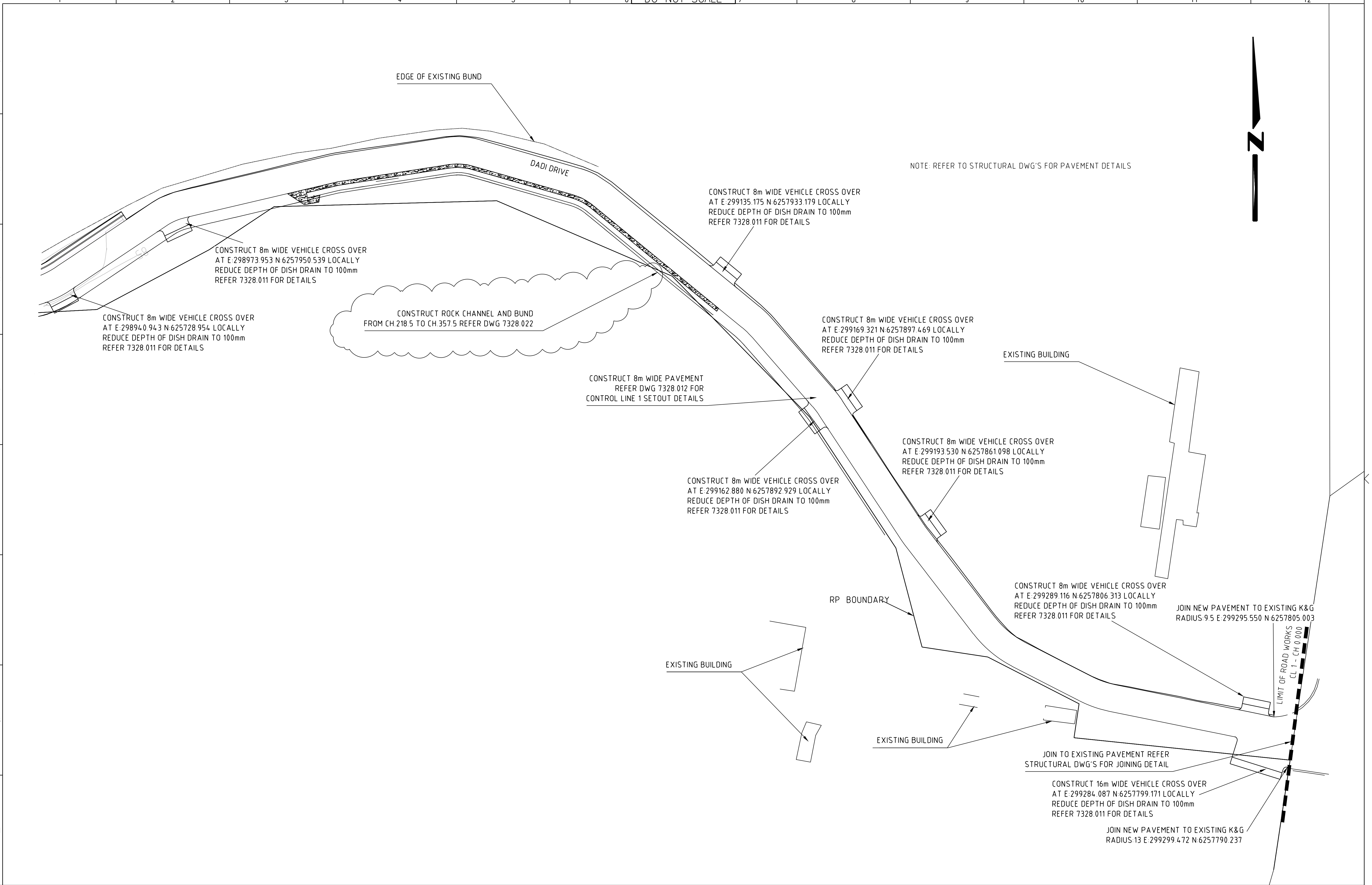
Brisbane, Cairns, Canberra, Darwin, Gold Coast, Melbourne, Minto
Newcastle, Perth, Sydney Sunshine Coast, Townsville, Wollongong, Wyong

Title: SOIL, WATER AND LEACHATE MANAGEMENT PLAN - LIGHT HORSE BUSINESS CENTRE, EASTERN CREEK, NSW

CONSTRUCTION PHASE EROSION AND SEDIMENT CONTROL

Client: ALEXANDRIA LANDFILL PTY LTD			Office: PERTH
Drawn By: J. RUSSELL	Scale: N/A	Project No: 46950	**DRAWING 2**
Approved By: --		Date: 21-12-09	Revision: A

DO NOT SCALE



No.	Revision – Revise on CAD do not amend by hand	Checked	Approved	Date	M/Film
10	CONCRETE DRAIN REMOVED			14/07/11	
9	CROSS OVER AMENDED			07/07/11	
8	CL 1 ALIGNMENT AMENDED			05/04/11	
7	VEHICLE CROSS OVER NOTE AMENDED			21/02/11	
6	DRIVEWAY CROSS OVERS ADDED			11/02/11	

Notes

STATUS

GMW

DIAL A DUMP
KEEPING AUSTRALIA CLEAN

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Scale 0 10 20 30m
SCALE 1:500 @ A1

Drawn	E.HEUGH	Designed	E.HEUGH
Drafting Check	A.CHOONG	Design Check	R.JARJEES
Approved	RAAD JARJEES		

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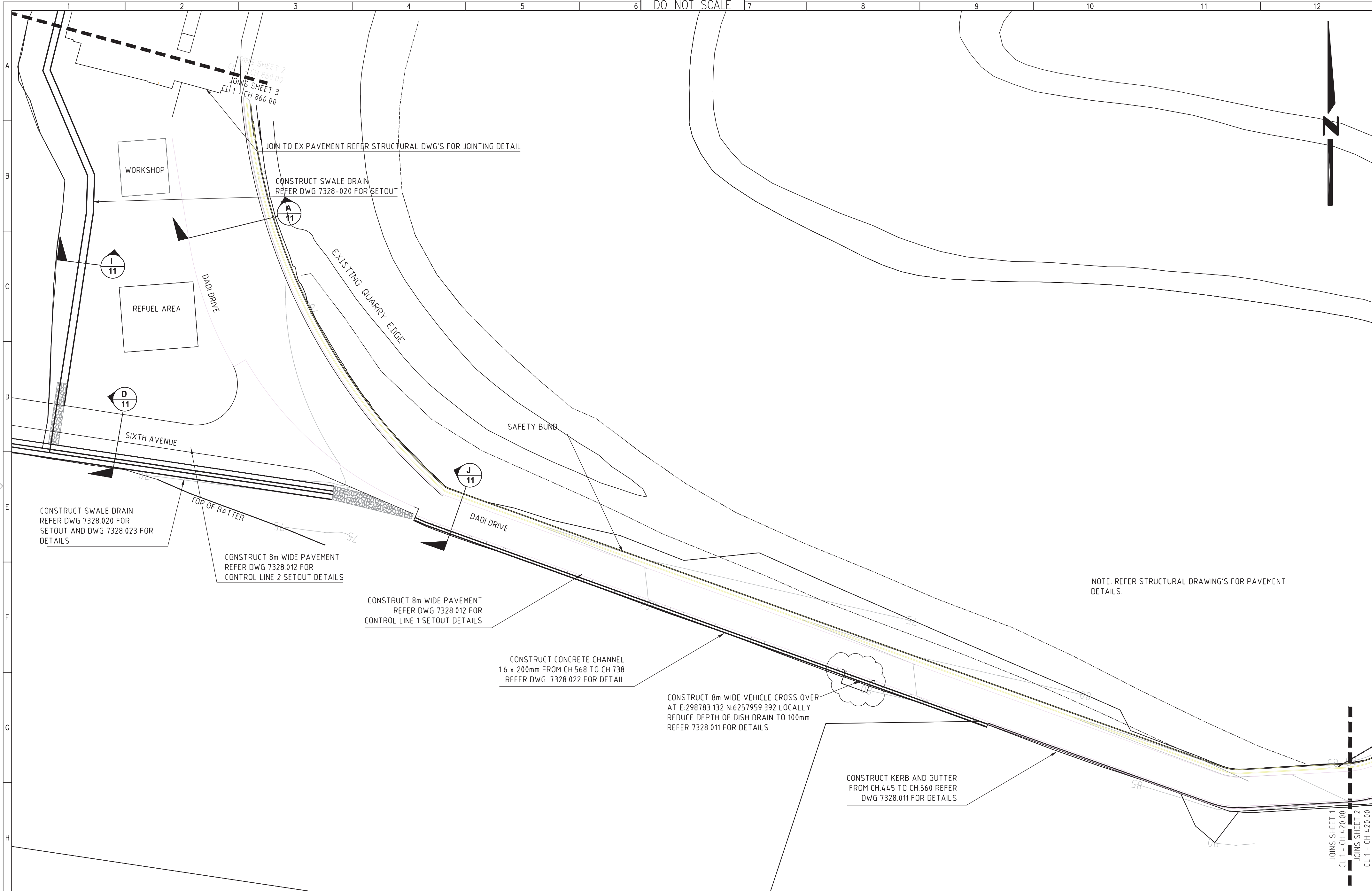
Client and Job: GMW URBAN
DIAL A DUMP – LIGHT HORSE BUSINESS CENTRE

Title: CIVIL
GENERAL WORKS & SERVICES SHEET 1 OF 5

Drg. No. 7328–006 | Size A1 | Rev 10

Plot Date: CAD File Name:



No.	Revision – Revise on CAD do not amend by hand	Checked	Approved	Date	M/Film
5	DRAINAGE CHANGED			5/10/10	
9	CROSS OVER AMENDED			19/04/11	
8	CROSS OVER NOTE AMENDED			21/02/11	
7	DRIVEWAY CROSSOVERS ADDED			11/02/11	
6	DADI DRIVE LEVEL CHANGE			05/01/11	

Plot Date: CAD File Name:

Notes

STATUS

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Scale 0 10 20 30m
SCALE 1:500 @ A1

Drawn	E HEUGH	Designed	E HEUGH
Drafting Check	A CHOONG	Design Check	R JARJEES
Approved	RAAD JARJEES		

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Client and Job: GMW URBAN
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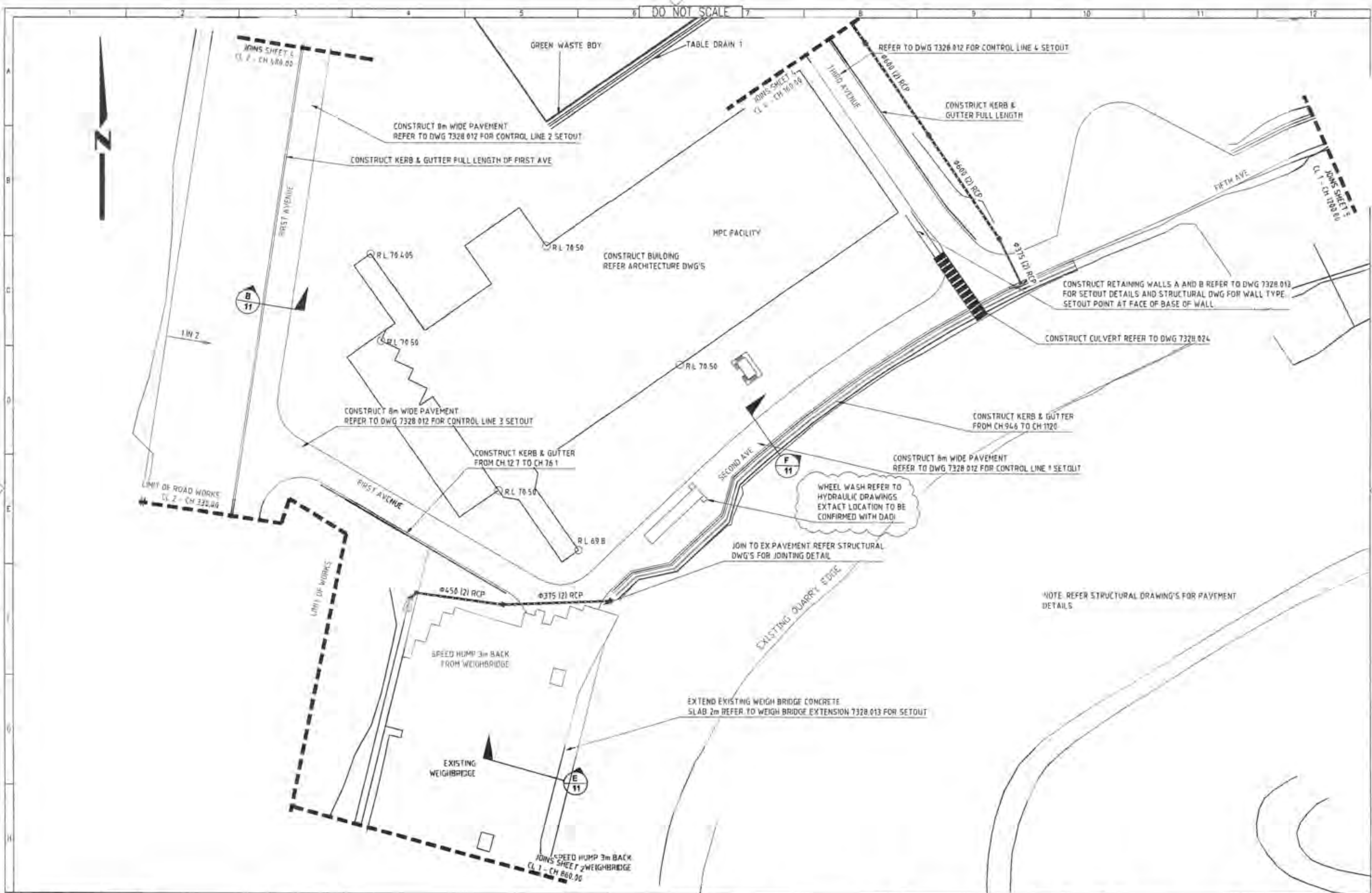
Title: CIVIL
GENERAL WORKS & SERVICES – 2 OF 5

Drg. No. 7328–007 | Size A1 | Rev 9



No.	Revision	Checked	Approved	Date	W/TIM
5	WHEEL WASH NOTE AMENDED SPOT LEVELS CHANGED			11/02/11	
4	DRAINAGE REMOVED			3/10/10	
3	FOR CONSTRUCTION			07/07/10	
2	PRELIMINARY ISSUE REVISION			29/05/10	
6	WHEEL WASH AMENDED			04/02/11	

Plot Date: CAD File Name:

GMW

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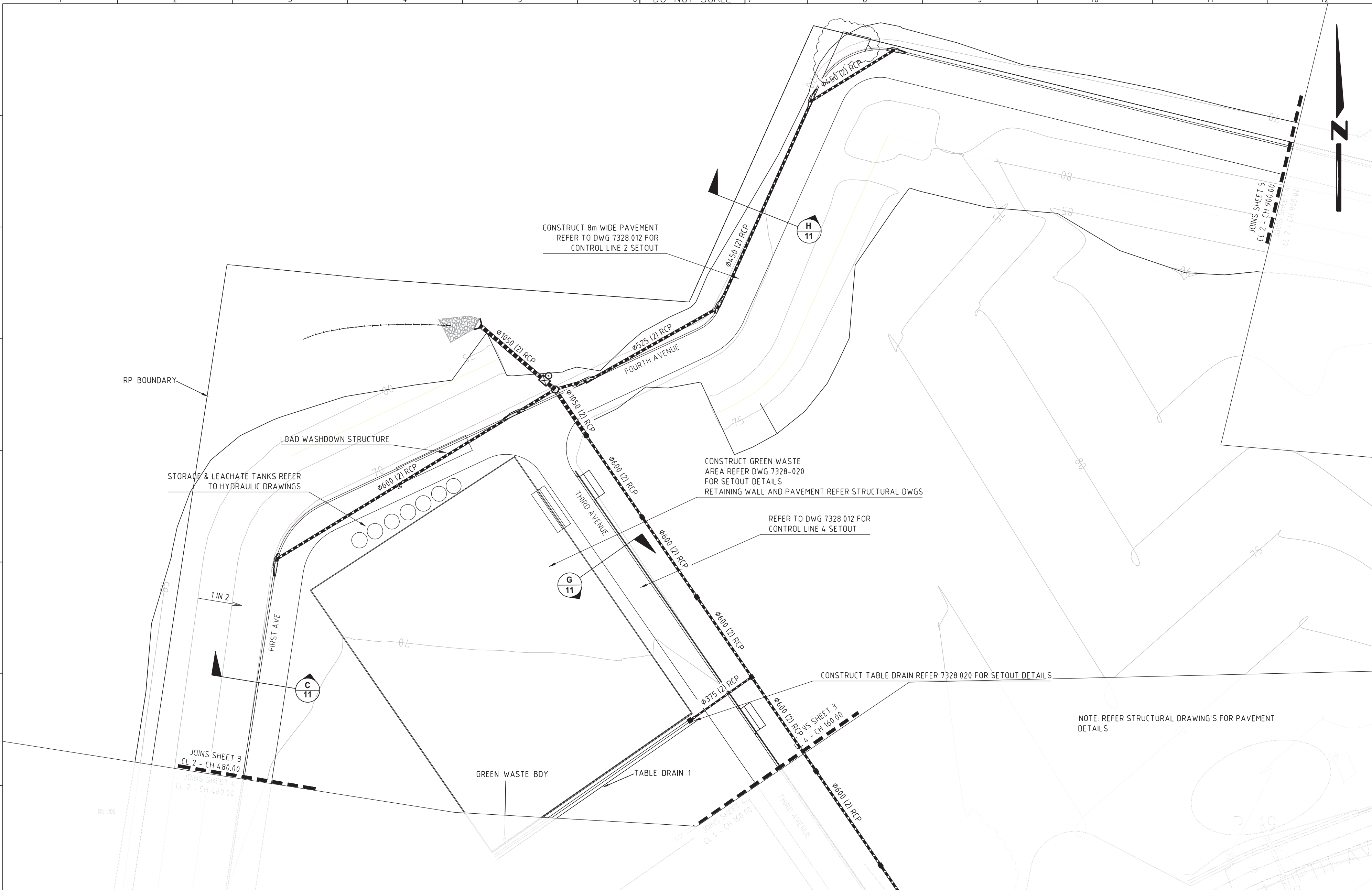
CIVIL
GENERAL WORKS & SERVICES – 3 OF 5

Drg. No. 7328-008 | A1 | 5



No.	Revision	Checked	Approved	Date	M/Film
10	VEHICLE CROSS OVER AMENDED			11/07/11	
9	LOAD WASHDOWN STRUCTURE LOCATION AMENDED			30/06/11	
8	TRUCK WASH NOTE AMENDED			07/07/10	
7	FOURTH AVE ALIGNMENT CHANGE			24/11/10	
6	FOURTH AVE LEVEL CHANGE			22/10/10	

Revision – Revise on CAD do not amend by hand

Notes

STATUS

Plot Date:

CAD File Name:

GMW

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Scale 0 10 20 30m
SCALE 1:500 @ A1

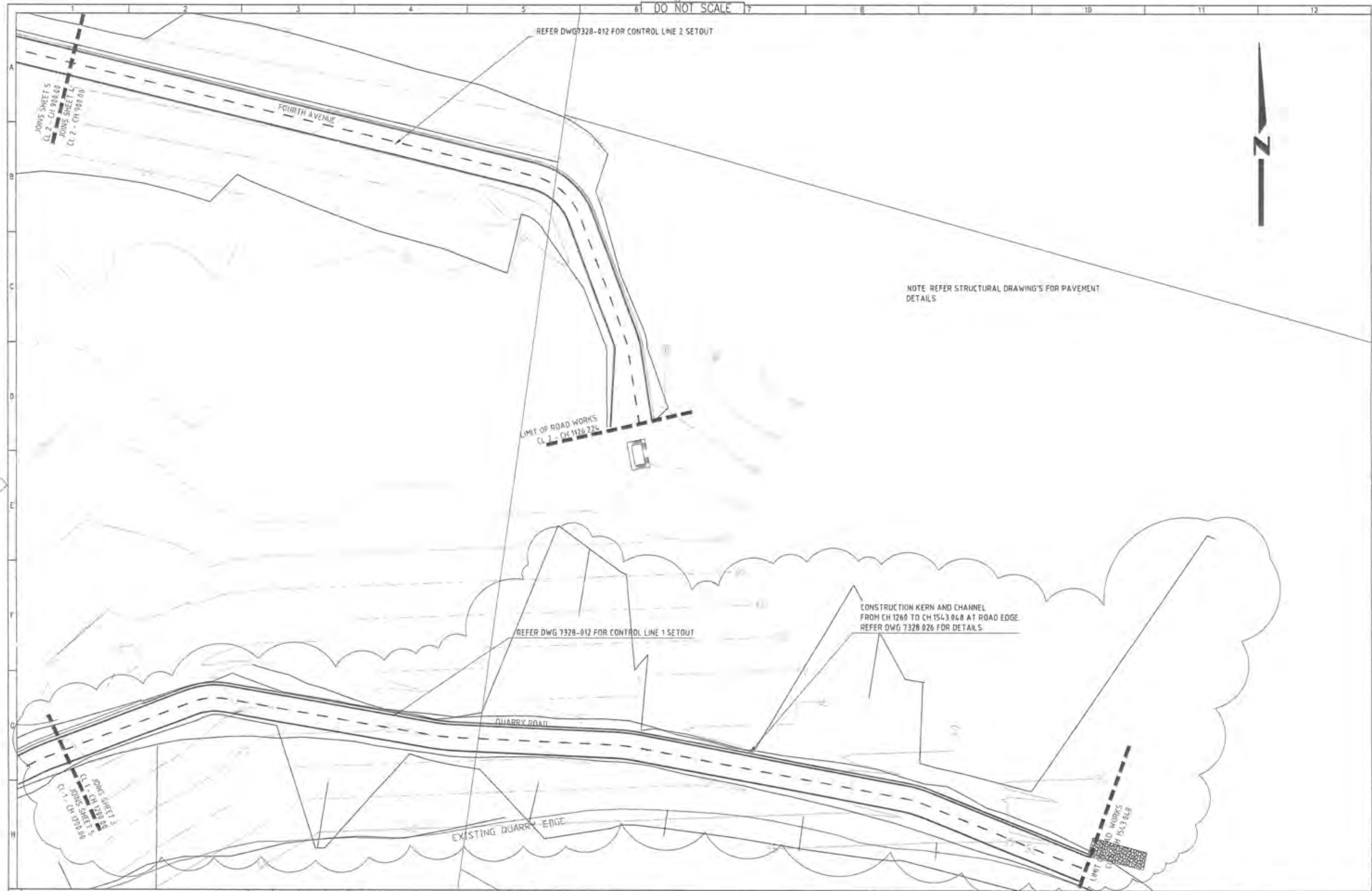
Drawn	E.HEUGH	Designed	E.HEUGH
Drafting Check	A.CHOONG	Design Check	R.JARJEES
Approved	RAAD JARJEES		

This Drawing must not be used for Construction unless signed as Approved

Client and Job: GMW URBAN
DIAL A DUMP – LIGHT HORSE BUSINESS CENTRE

Title: CIVIL
GENERAL WORKS & SERVICES – 4 OF 5

Drg. No. 7328-009 | Size A1 | Rev 10



No.	Revision	Checked	Approved	Date
5	BERM AMENDMENT			5/10/10
4	FOURTH AVENUE LEVEL CHANGE			30/08/10
3	FOR CONSTRUCTION			07/07/10
7	FIFTH AVE AND QUARRY ROAD AMENDED			04/07/11
6	FOURTH AVE LEVEL CHANGE			22/10/10

Plot Date: CAD File Name:

GMW

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SCALE 1:500 @ A1

Drawn: C.HEUGH Designed: C.HEUGH
Drafting Check: A.CHOONG Design Check: R.JARVIES
Approved: RAAD JARJEES

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Client: GMW URBAN
Job: DIAL A DUMP - LIGHT HORSE BUSINESS CENTRE

Title: CIVIL
GENERAL WORKS & SERVICES - 5 OF 5

Dwg. No. 7328-010 | Size A1 | Rev. 7

LIGHT HORSE BUILDING CENTRE

EASTERN CREEK

Job No. 090669

HYDRAULIC ABBREVIATIONS

TYPE	DESCRIPTION	TYPE	DESCRIPTION
AAV	AIR ADMITTANCE VALVE	OF	OVERFLOW
B	BASIN	ORG	OVERFLOW RELIEF GULLY
CA	COMPRESSED AIR	P	PENETRATION
CO	CLEAR OUT	PLV	PRESSURE LIMITING VALVE
CS	CLEANERS SINK	PRV	PRESSURE REDUCING VALVE
CW	COLD WATER	RL	REDUCED LEVEL
DF	DRINKING FOUNTAIN	RPZD	REDUCED PRESSURE ZONE DEVICE
DST	DRAINAGE STACK	RV	RELIEF VENT
DW	DISHWASHER	SK	SINK
EWS	EMERGENCY EYE WASH/ SHOWER	SHR	SHOWER
FFL	FINISHED FLOOR LEVEL	SMH	SEWER MANHOLE
FH	FIRE HYDRANT	SRM	SEWER RISING MAIN
FHR	FIRE HOSE REEL	SS	SEWER STACK
FW	FLOOR WASTE	ST	STOP TAP
HL	HIGH LEVEL	SV	STOP VALVE (ISOLATION VALVE)
HT	HOSE TAP	TD	TUNDISH
HWU	HOT WATER UNIT	TMV	THERMOSTATIC MIXING VALVE
IL	INVERT LEVEL	TW	TRADE WASTE
IPMF	INDUCT PIPE MICA FLAP	U.N.O.	UNLESS NOTED OTHERWISE
KS	KITCHEN SINK	U	URINAL
LL	LOW LEVEL	VP	VENT PIPE
NG	NATURAL GAS	WC	TOILET SUITE (WATER CLOSET)
NTS	NOT TO SCALE		

HYDRAULIC SYMBOLS

TYPE	DESCRIPTION	TYPE	DESCRIPTION
H01	DRAWING REFERENCE SHEET		GULLY PIT
	WATER METER WITH STOP VALVE		EXTERNAL HOSE TAP
	BL VALVE		HOT WATER UNIT I
BT	BOUNDARY TRAP		PIPECUT
BTFW	BASKET TRAP FLOOR WASTE		GRATED INLET PIT
	PIPE END CAP		COVERED PIT
C/O	CLEAR OUT		PUMP
	CONTINUATION		RELIEF VALVE
1 S2	DETAIL REFERENCE		PIPE RISER
FHR 40	FIRE HYDRANT DIRECTIONAL		REDUCED PRESSURE ZONE DEVICE
TD	TUNDISH	SMH	SEWER MAN HOLE
C	PIPE DROPPER		SOLENOID VALVE
	FIRE BRIGADE BOOSTER VALVE I		THERMOSTATIC MIXING VALVE
FH	FIRE HYDRANT		URINAL CONTROL I
FHR	FIRE HOSE REEL		STRAINER
F	FILTER		STOP VALVE
	PIPE FLOW		INTERNAL HOSE TAP
	FLOOR WASTE		INTERNAL TAP WITH STOP VALVE
	CHECK VALVE		CONTROL PANEL
ABE 9KG	9.0KG POWDER PORTABLE EXTINGUISHER (ABE) AND ASSOCIATED SIGNAGE	ABE 4.5KG	4.5KG POWDER PORTABLE EXTINGUISHER (ABE) AND ASSOCIATED SIGNAGE

GENERAL NOTES

1. ALL WORKS TO BE INSTALLED IN ACCORDANCE WITH AUSTRALIAN STANDARDS AND LOCAL GOVERNING AUTHORITY REQUIREMENTS.
2. ALLOW TO COORDINATE ALL PIPE-WORK WITH ALL STRUCTURAL COMPONENTS AND OTHER BUILDING SERVICES. ALLOW ALL OFFSETS, BENDS AND DIVERSIONS AS MAY BE REQUIRED.
3. ALLOW TO PAY ALL FEES AND CHARGES AS NECESSARY TO COMPLETE THE WORK.
4. THESE DRAWINGS ARE TO BE READ IN CONJUNCTION WITH THE HYDRAULIC SPECIFICATION, ARCHITECTURAL, STRUCTURAL AND OTHER SERVICES DOCUMENTATION.
5. THE LOCATION AND SET OUT OF ALL FITTINGS, FIXTURES AND FLOOR-WASTES IS TO BE FULLY COORDINATED WITH THE ARCHITECTS DETAIL DOCUMENTATION.
6. ALLOW TO EXTEND ALL VENTS VERTICALLY THROUGH ROOF AND DISCHARGE WITH A WEATHERPROOF FLASHING AND COWL.
7. PROVIDE CONCEALED 'P-TRAP' DISCHARGE PIPES TO ALL BASINS AND SINKS. REFER TO DETAIL.
8. PIPING SHALL BE A MINIMUM OF 20MM EXCEPT FOR THE LAST 2M TO A SINGLE FIXTURE WHICH MAY BE 15MM.

CW — CW — CW
COLD WATER PIPE

HW — HW — HW
HOT WATER PIPE

S — S
SEWER PIPING

FS — FS
MISTING SYSTEM PIPE WORK

FHR — FHR
FIRE HOSE REEL PIPING

FH — FH
FIRE HYDRANT PIPING

RWR — RWR
RAIN WATER REUSE

AMDT	DATE	BY	DESCRIPTION	AMDT	DATE	BY	DESCRIPTION
F	08.02.11	JC	CONSTRUCTION ISSUE				
E	26.11.10	JC	CONSTRUCTION ISSUE				
D	23.11.10	JC	CONSTRUCTION ISSUE	G	28.06.11	JC	CONSTRUCTION ISSUE

CIVIL • STRUCTURAL • BUILDING SERVICES

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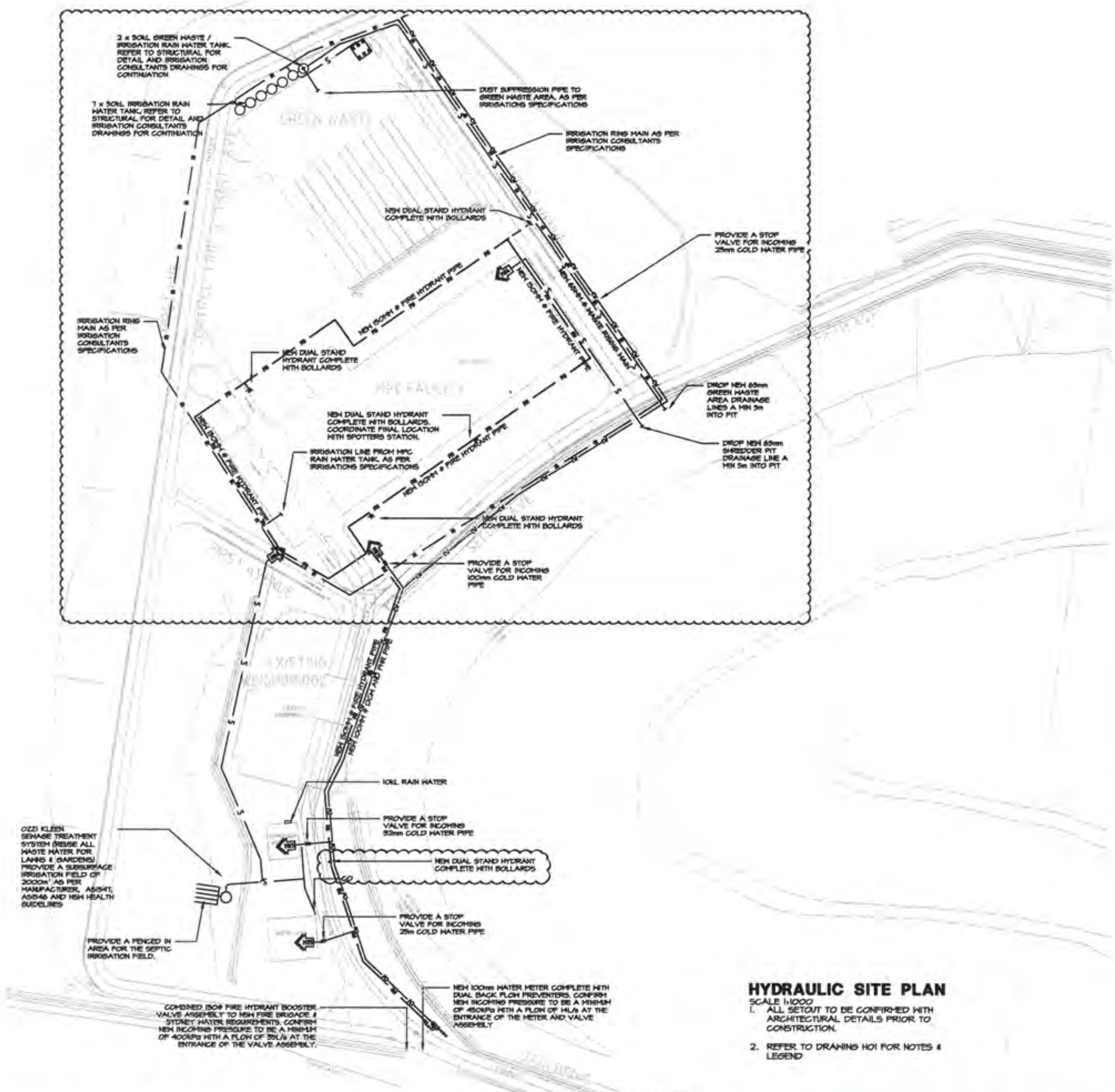
DESIGN : JC
DRAWN : JC
DATE : 07.06.10
DRG SIZE : A1
SCALE : AS NOTED
PROJECT MGR : GC

HYDRAULIC DESIGN LEGEND AND NOTES

LIGHT HORSE BUSINESS CENTRE
OLD WALGROVE ROAD
EASTERN CREEK
DIAL A DUMP

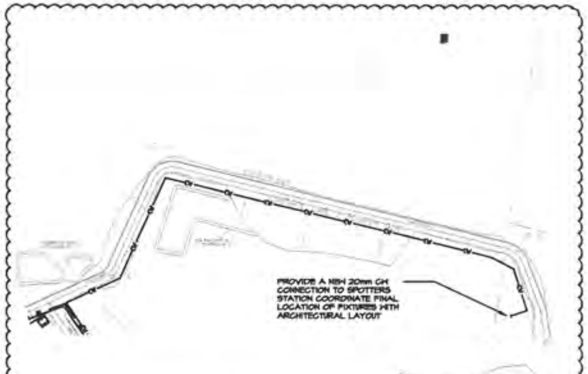
090669
H01 G



HYDRAULIC SITE PLAN

SCALE 1:1000

1. ALL SETOUT TO BE CONFIRMED WITH ARCHITECTURAL DETAILS PRIOR TO CONSTRUCTION.
2. REFER TO DRAWING H01 FOR NOTES & LEGEND



ENLARGED HYDRAULIC SITE PLAN

SCALE 1:2000

1. ALL SETOUT TO BE CONFIRMED WITH ARCHITECTURAL DETAILS PRIOR TO CONSTRUCTION.
2. REFER TO DRAWING H01 FOR NOTES & LEGEND

AMDT	DATE	BY	DESCRIPTION	AMDT	DATE	BY	DESCRIPTION
I	09.02.11	JC	CONSTRUCTION ISSUE				
H	01.02.11	JC	CONSTRUCTION ISSUE	K	25.02.11	JC	CONSTRUCTION ISSUE
G	25.11.10	JC	CONSTRUCTION ISSUE	J	15.02.11	JC	CONSTRUCTION ISSUE

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DESIGN: JC
DRAWN: JC
DATE: 04.06.10
DRG SIZE: A1
SCALE: AS NOTED
PROJECT NO: QC

HYDRAULIC SITE PLAN

LIGHT HORSE BUSINESS CENTRE
OLD WALGROVE ROAD
EASTERN CREEK
DIAL A DUMP

090669
H02 K

THESE DRAWINGS SHOULD BE READ IN CONJUNCTION WITH MARTENS CONSULTING ENGINEERS CONSOLIDATED STORMWATER MANAGEMENT PLAN P1103002JR01V05

Issue	Description	Date
A	ISSUED FOR APPROVAL	01-12-11

Client: DIAL A DUMP

Scale Bar: 0 5 10 15 20 25m
1 : 300

Scales: 1 : 300
Grid: MGA
Height Datum: AHD

Drawn: MM
Designed: AMc
Checked:
Approved:

File

Project: LIGHT HORSE BUSINESS PARK EASTERN CREEK

Title: NORTHERN OSD BASIN LAYOUT PLAN

Civil Engineers and Project Managers
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Suite 702, 6A Glen Street
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Status: PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

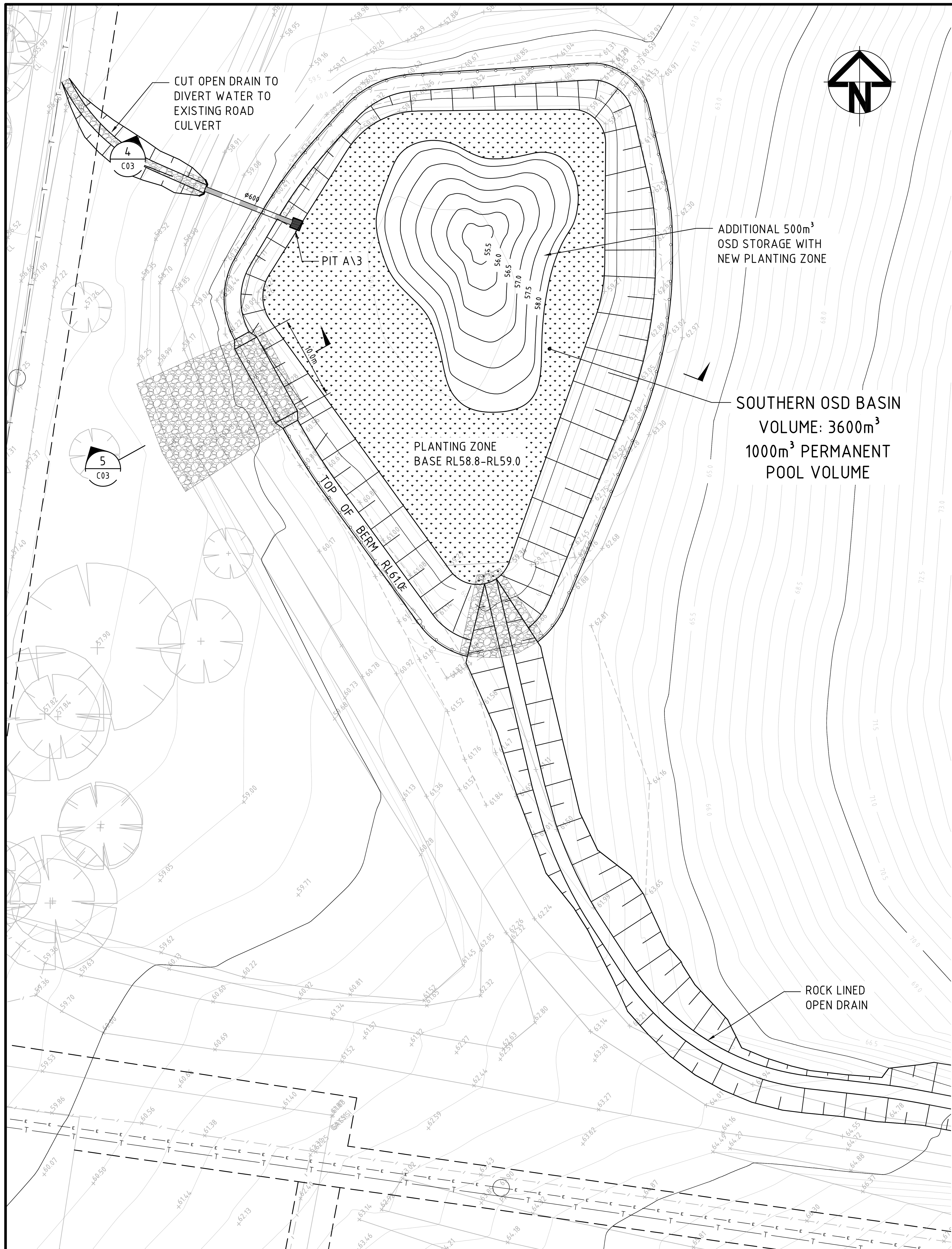
A1

Drawing No. C01 | Project No. 10-16 | Issue A

100mm on Original

F:\10-16 Dial A Dump\Drgs\Civil\Final\C01 Northern Basin Plan.dwg

V1



EMBANKMENT CONSTRUCTION SEQUENCE

1. STRIP VEGETATION AND TOPSOIL FROM EMBANKMENT AREA AND STOCKPILE TOPSOIL FOR LATER USE. CUT BACK AREA TO FIRM GROUND.
2. CONSTRUCT EMBANKMENT IN PRESENCE OF QUALIFIED AND EXPERIENCED GEOTECHNICAL ENGINEER IF NOT ROCK.
3. IN THE CASE WHERE THE EMBANKMENT AREAS SLUSH, GROUTING AND DENTAL CONCRETE MAY BE REQUIRED, AS DIRECTED BY A QUALIFIED AND EXPERIENCED GEOTECHNICAL ENGINEER.
4. COMPACT CLAY STABILIZED WITH GYPSUM (3% BY DRY MASS, MINIMUM) AS APPROVED BY A QUALIFIED AND EXPERIENCED GEOTECHNICAL ENGINEER INTO THE CUT-OFF TRENCH OF LAYERS NOT EXCEEDING 150mm LOOSE THICKNESS TO A DRY DENSITY EQUIVALENT TO 98% OF THAT DETERMINED BY STANDARD COMPACTION (AS 1289.5.1.1) AND AT A MOISTURE CONTENT OF -2% TO +2% OF OPTIMUM MOISTURE CONTENT.
5. GYPSUM STABILIZED NATURAL SOILS EXPOSED IN EMBANKMENT AREA WITH MINIMUM 3% GYPSUM BY DRY MASS AND COMPACT AS FOR #4. ALL TO THE APPROVAL OF A QUALIFIED AND EXPERIENCED GEOTECHNICAL ENGINEER.
6. CONSTRUCT BODY OF EMBANKMENT WITH CLAYEY MATERIAL WON FROM SITE. COMPACT THE CLAYEY MATERIAL APPROVED BY A QUALIFIED AND EXPERIENCED GEOTECHNICAL ENGINEER IN LAYERS NOT EXCEEDING 150mm THICKNESS TO A DRY DENSITY EQUIVALENT TO 98% OF THAT DETERMINED BY STANDARD COMPACTION (AS 1289.5.1.1) AND AT A MOISTURE CONTENT OF -2% TO +2% OF OPTIMUM MOISTURE CONTENT. MOST IMPORTANTLY, IF SHRINKAGE CRACKS OCCUR, AS DIRECTED BY A QUALIFIED AND EXPERIENCED GEOTECHNICAL ENGINEER.
7. OVERFILL THE EMBANKMENT AND TRIM OFF, SO THAT THE ENTIRE BODY OF THE EMBANKMENT IS COMPACTED.
8. TRIM THE EMBANKMENTS BATTERS TO THE OVERFILLED MATERIAL, STABILIZE THE UPSTREAM CLAY BATTERS WITH WELL MIXED GYPSUM (3% BY DRY MASS, MINIMUM) AND COMPACT TO MIN. 98% STD -2% TO +2% OMC.
9. PLACE ROCK RIP-RAP AS SHOWN.
10. RECOVER TOPSOIL FROM STOCKPILE AND SPREAD OVER EMBANKMENT AND CUT BATTERS (A THIN COVER OF TOPSOIL ONLY HAS BEEN NOMINATED). ONLY LIGHTLY TRACK-ROLL THE TOPSOIL AND THEN LANDSCAPE IN ACCORDANCE WITH THE LANDSCAPE AREA DRAWINGS.
11. WATER AND FERTILIZE LANDSCAPE AS REQUIRED BY CLIMACTIC CONDITIONS TO ENSURE THE LANDSCAPE IS SUCCESSFUL.
12. AT THE COMPLETION OF WORK WRITTEN CONFIRMATION & CERTIFICATION IS TO BE PROVIDED FROM A QUALIFIED & EXPERIENCED GEOTECHNICAL ENGINEER THAT THE EMBANKMENTS HAVE BEEN CONSTRUCTED IN ACCORDANCE WITH THESE DRAWINGS.

BULK EARTHWORKS NOTES

1. ORIGIN OF LEVELS: REFER SURVEY NOTES
2. STRIP ALL TOPSOIL/ORGANIC MATERIAL FROM CONSTRUCTION AREA AND REMOVE FROM SITE OR STOCK PILE AS DIRECTED BY SUPERINTENDENT.
3. EXCAVATED MATERIAL TO BE USED AS STRUCTURAL FILL PROVIDED THE PLACEMENT MOISTURE CONTENT OF THE MATERIAL IS +/- 2% OF THE OPTIMUM MOISTURE CONTENT IN ACCORDANCE WITH AS 1289.5.7.1.
4. COMPACT FILL AREAS AND SUBGRADE TO NOT LESS THAN:

LOCATION	STANDARD DRY DENSITY (AS 1289 E 5.1.1.)
UNDER BUILDING SLABS ON GROUND	98%
UNDER ROADS AND CARPARKS	98%
LANDSCAPED AREAS UNLESS NOTED OTHERWISE	98%

5. FOR NON COHESIVE MATERIAL, COMPACT TO 75% DENSITY INDEX.
6. BEFORE PLACING FILL, PROOF ROLL EXPOSED SUBGRADE WITH AN 8 TONNE (MIN) DEADWEIGHT SMOOTH DRUM VIBRATORY ROLLER TO DETECT THEN REMOVE SOFT SPOTS (AREAS WITH MORE THAN 2mm MOVEMENT UNDER ROLLER).
7. FREQUENCY OF COMPACTION TESTING SHALL BE NOT LESS THAN :-
   (A) 1 TEST PER 200m³ OF FILL PLACED PER 300mm LAYER OF FILL.
   (B) 3 TESTS PER VISIT
   (C) 1 TEST PER 1000m² OF EXPOSED SUBGRADE
   TESTING SHALL BE "LEVEL 1" TESTING IN ACCORDANCE WITH AS 3798 (1996).
8. FILLING TO BE PLACED AND COMPACTED IN MAXIMUM 150mm LAYERS
9. NO FILLING SHALL TAKE PLACE TO EXPOSE SUBGRADE UNTIL THE AREA HAS BEEN PROOF ROLLED IN THE PRESENCE OF AT & L AND APPROVAL GIVEN IN WRITING THAT FILLING CAN PROCEED.

EROSION AND SEDIMENT CONTROL NOTES:

GENERAL INSTRUCTIONS

1. THE SITE SUPERINTENDENT/ENGINEER WILL ENSURE THAT ALL SOIL AND WATER MANAGEMENT WORKS ARE LOCATED AS DOCUMENTED.
2. ALL WORK SHALL BE GENERALLY CARRIED OUT IN ACCORDANCE WITH
   a. LOCAL AUTHORITY REQUIREMENTS
   b. EPA REQUIREMENTS
   c. MANAGING URBAN STORMWATER - SOIL & CONSTRUCTION VOLUME 1, (2004) LANDCOM
3. MAINTAIN THE EROSION CONTROL DEVICES TO THE SATISFACTION OF THE SUPERINTENDENT AND THE LOCAL AUTHORITY.
4. WHEN STORMWATER PITS ARE CONSTRUCTED, PREVENT SITE RUNOFF ENTERING UNLESS SEDIMENT FENCES ARE ERECTED AROUND PITS.
5. CONTRACTOR IS TO ENSURE ALL EROSION & SEDIMENT CONTROL DEVICES ARE MAINTAINED IN GOOD WORKING ORDER AND OPERATE EFFECTIVELY. REPAIRS AND OR MAINTENANCE SHALL BE UNDERTAKEN AS REQUIRED, PARTICULARLY FOLLOWING STORM EVENTS.

LAND DISTURBANCE

6. WHERE PRACTICAL, THE SOIL EROSION HAZARD ON THE SITE WILL BE KEPT AS LOW AS POSSIBLE. TO THIS END, WORKS SHOULD BE UNDERTAKEN IN THE FOLLOWING SEQUENCE:
   (A) INSTALL A WIND FENCE ALONG THE BOUNDARIES AS SHOWN ON PLAN. REFER DETAIL.
   (B) INSTALL A SEDIMENT FENCE ALONG THE BOUNDARIES AS SHOWN ON PLAN. REFER DETAIL.
   (C) CONSTRUCT STABILISED CONSTRUCTION ENTRANCE TO LOCATION AS DETERMINED BY SUPERINTENDENT/ENGINEER. REFER DETAIL.
   (D) INSTALL SEDIMENT BASIN AS SHOWN ON PLAN
   (E) INSTALL SEDIMENT TRAPS AS SHOWN ON PLAN.
   (F) UNDERTAKE SITE DEVELOPMENT WORKS IN ACCORDANCE WITH THE ENGINEERING PLANS. WHERE POSSIBLE, PHASE DEVELOPMENT SO THAT LAND DISTURBANCE IS CONFINED TO AREAS OF WORKABLE SIZE.

EROSION CONTROL

7. DURING WINDY WEATHER, LARGE, UNPROTECTED AREAS WILL BE KEPT MOIST (NOT WET) BY SPRINKLING WITH WATER TO KEEP DUST UNDER CONTROL.
8. FINAL SITE LANDSCAPING WILL BE UNDERTAKEN AS SOON AS POSSIBLE AND WITHIN 20 WORKING DAYS FROM COMPLETION OF CONSTRUCTION ACTIVITIES.

SEDIMENT CONTROL

9. STOCKPILES WILL NOT BE LOCATED WITHIN 5 METRES OF HAZARD AREAS, INCLUDING LIKELY AREAS OF CONCENTRATED OR HIGH VELOCITY FLOWS SUCH AS WATERWAYS.
10. ANY SAND USED IN THE CONCRETE CURING PROCESS (SPREAD OVER THE SURFACE) WILL BE REMOVED AS SOON AS POSSIBLE AND WITHIN 10 WORKING DAYS FROM PLACEMENT.
11. WATER WILL BE PREVENTED FROM ENTERING THE PERMANENT DRAINAGE SYSTEM UNLESS IT IS RELATIVELY SEDIMENT FREE, I.E. THE CATCHMENT AREA HAS BEEN PERMANENTLY LANDSCAPED AND/OR ANY LIKELY SEDIMENT HAS BEEN FILTERED THROUGH AN APPROVED STRUCTURE.
12. TEMPORARY SOIL AND WATER MANAGEMENT STRUCTURES WILL BE REMOVED ONLY AFTER THE LANDS THEY ARE PROTECTING ARE REHABILITATED.

OTHER MATTERS

13. ACCEPTABLE RECEPTORS WILL BE PROVIDED FOR CONCRETE AND MORTAR SLURRIES, PAINTS, ACID WASHINGS, LIGHT-WEIGHT WASTE MATERIALS AND LITTER.
14. ANY EXISTING TREES WHICH FORM PART OF THE FINAL LANDSCAPING PLAN WILL BE PROTECTED FROM CONSTRUCTION ACTIVITIES BY:
   (A) PROTECTING THEM WITH BARRIER FENCING OR SIMILAR MATERIALS INSTALLED OUTSIDE THE DRIP LINE
   (B) ENSURING THAT NOTHING IS NAILED TO THEM
   (C) PROHIBITING PAVING, GRADING, SEDIMENT WASH OR PLACING OF STOCKPILES WITHIN THE DRIP LINE EXCEPT UNDER THE FOLLOWING CONDITIONS.
      (I) ENCROACHMENT ONLY OCCURS ON ONE SIDE AND NO CLOSER TO THE TRUNK THAN EITHER 1.5 METRES OR HALF THE DISTANCE BETWEEN THE OUTER EDGE OF THE DRIP LINE AND THE TRUNK, WHICH EVER IS THE GREATER
      (II) A DRAINAGE SYSTEM THAT ALLOWS AIR AND WATER TO CIRCULATE THROUGH THE ROOT ZONE (E.G. A GRAVEL BED) IS PLACED UNDER ALL FILL LAYERS OF MORE THAN 300 MILLIMETRES DEPTH
      (III) CARE IS TAKEN NOT TO CUT ROOTS UNNECESSARILY NOR TO COMPACT THE SOIL AROUND THEM.

CONCRETE NOTES

1. ALL WORKMANSHIP AND MATERIALS SHALL BE IN ACCORDANCE WITH AS 3600 CURRENT EDITION WITH AMENDMENTS, EXCEPT WHERE VARIED BY THE CONTRACT DOCUMENTS & PITTWATER COUNCIL SPECIFICATIONS.
2. CONCRETE QUALITY
   ALL REQUIREMENTS OF THE CURRENT ACSE CONCRETE SPECIFICATION DOCUMENT 1 SHALL APPLY TO THE FORMWORK, REINFORCEMENT AND CONCRETE UNLESS NOTED OTHERWISE.

ELEMENT	AS 3600 F'c MPa AT 28 DAYS	SPECIFIED SLUMP	NOMINAL AGG. SIZE
VEHICULAR BASE	32	60	20
KERBS, PATHS, AND PITS	25	80	20

   - CEMENT TYPE SHALL BE (ACSE SPECIFICATION) TYPE SL
   - PROJECT CONTROL TESTING SHALL BE CARRIED OUT IN ACCORDANCE WITH COUNCIL SPECIFICATIONS.
3. NO ADMIXTURES SHALL BE USED IN CONCRETE UNLESS APPROVED IN WRITING BY AT & L.
4. CLEAR CONCRETE COVER TO ALL REINFORCEMENT FOR DURABILITY SHALL BE 40mm TOP AND 70mm FOR EXTERNAL EDGES UNLESS NOTED OTHERWISE.
5. ALL REINFORCEMENT SHALL BE FIRMLY SUPPORTED ON MILD STEEL PLASTIC TIPPED CHAIRS, PLASTIC CHAIRS OR CONCRETE CHAIRS AT NOT GREATER THAN 1m CENTRES BOTH WAYS. BARS SHALL BE TIED AT ALTERNATE INTERSECTIONS.
6. THE FINISHED CONCRETE SHALL BE A DENSE HOMOGENEOUS MASS, COMPLETELY FILLING THE FORMWORK, THOROUGHLY EMBEDDING THE REINFORCEMENT AND FREE OF STONE POCKETS. ALL CONCRETE INCLUDING SLABS ON GROUND AND FOOTINGS SHALL BE COMPACTED AND CURED IN ACCORDANCE WITH R.T.A. SPECIFICATION R83.
7. REINFORCEMENT SYMBOLS:
   N DENOTES GRADE 450 N BARS TO AS 1302 GRADE N
   R DENOTES 230 R HOT ROLLED PLAIN BARS TO AS 1302
   SL DENOTES HARD-DRAWN WIRE REINFORCING FABRIC TO AS 1304

   NUMBER OF BARS IN GROUP — BAR GRADE AND TYPE
   17 N 20 250
   NOMINAL BAR SIZE IN mm — SPACING IN mm

   THE FIGURE FOLLOWING THE FABRIC SYMBOL SL IS THE REFERENCE NUMBER FOR FABRIC TO AS 1304.
8. FABRIC SHALL BE LAPPED IN ACCORDANCE WITH THE FOLLOWING DETAIL:

   25 MIN — LAP TWO WIRES

STORMWATER DRAINAGE NOTES

1. PIPES TO BE INSTALLED TO TYPE HS1 SUPPORT IN ACCORDANCE WITH AS 3725 (1989) IN ALL CASES BACKFILL TRENCH WITH SAND TO 300mm ABOVE PIPE. WHERE PIPE IS UNDER PAVEMENTS BACKFILL REMAINDER OF TRENCH TO UNDERSIDE OF PAVEMENT WITH SAND OR APPROVED GRANULAR MATERIAL COMPACTED IN 150mm LAYERS TO MINIMUM 98% STANDARD MAXIMUM DRY DENSITY IN ACCORDANCE WITH AS 1289 5.2.1. (OR A DENSITY INDEX OF NOT LESS THAN 75)
2. CARE IS TO BE TAKEN WITH LEVELS OF STORMWATER LINES. GRADES SHOWN ARE NOT TO BE REDUCED WITHOUT APPROVAL.
3. GRATES AND COVERS SHALL CONFORM TO AS 3996.
4. AT ALL TIMES DURING CONSTRUCTION OF STORMWATER PITS, ADEQUATE SAFETY PROCEDURES SHALL BE TAKEN TO ENSURE AGAINST THE POSSIBILITY OF PERSONNEL FALLING DOWN PITS.
5. ALL EXISTING STORMWATER DRAINAGE LINES AND PITS THAT ARE TO REMAIN ARE TO BE INSPECTED AND CLEANED. DURING THIS PROCESS ANY PART OF THE STORMWATER DRAINAGE SYSTEM THAT WARRANTS REPAIR SHALL BE REPORTED TO THE SUPERINTENDENT/ENGINEER FOR FURTHER DIRECTIONS.

FENCING AND SECURITY

BOTH BASINS ARE TO BE FENCED WITH 1.8m HIGH MAN PROOF FENCE GALVANISED POSTS AT 3m cts. PROVIDE 4.8m WIDE GATE TO ALLOW ACCESS. GATES TO BE LOCKED AT ALL TIMES. EXACT LOCATION OF FENCE AND GATE TO BE DETERMINED ON SITE. PROVIDED HINGED ONEWAY FLOOD FLAP AT WEIR LOCATIONS.

Issue	Description	Date
A	ISSUED FOR APPROVAL	01-12-11

Client: DIAL A DUMP

Scale Bar: 0 5 10 15 20 25m — 1 : 300

Scales: 1: 300
Grid: MGA
Height Datum: AHD

Drawn: MM
Designed: AMc
Checked:
Approved:

Project: LIGHT HORSE BUSINESS PARK EASTERN CREEK

Title: SOUTHERN OSD BASIN LAYOUT PLAN

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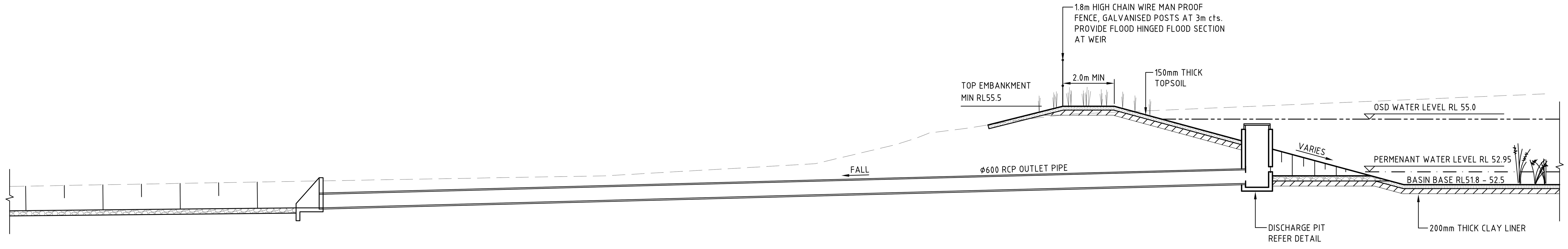
Status: PRELIMINARY ONLY NOT TO BE USED FOR CONSTRUCTION | A1

Drawing No. C02 | Project No. 10-16 | Issue A

File: F:\10-16 Dial A Dump\Drgs\Civil\Final\C02 Southern Basin Plan.dwg

100mm on Original

V1



SECTION 3
1 : 100 C01

BASIN STORAGE TABLE

NORTHERN BASIN

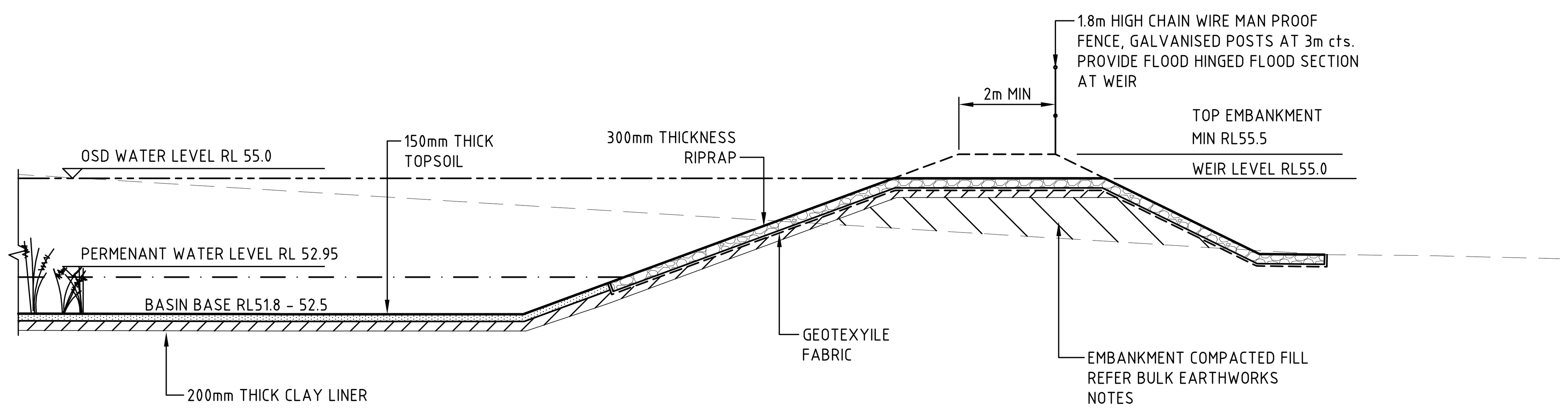
PERMANENT WATER VOLUME	500m³
OSD VOLUME	3400m³
TOTAL VOLUME	3900m³

NORTHERN BIO-FILTRATION BASIN

FILTER AREA	450m²
EXTENDED DETENTION AREA	450m²
EXTENDED DETENTION DEPTH	400mm
FILTER DEPTH	500mm
FILTER MEDIA PERMEABILITY	200mm\HOUR (SPEC) 100mm\HOUR (ULTIMATE)

SOUTHERN BASIN

PERMANENT WATER VOLUME	1000m³
OSD VOLUME	2600m³
TOTAL VOLUME	3600m³



SECTION 2
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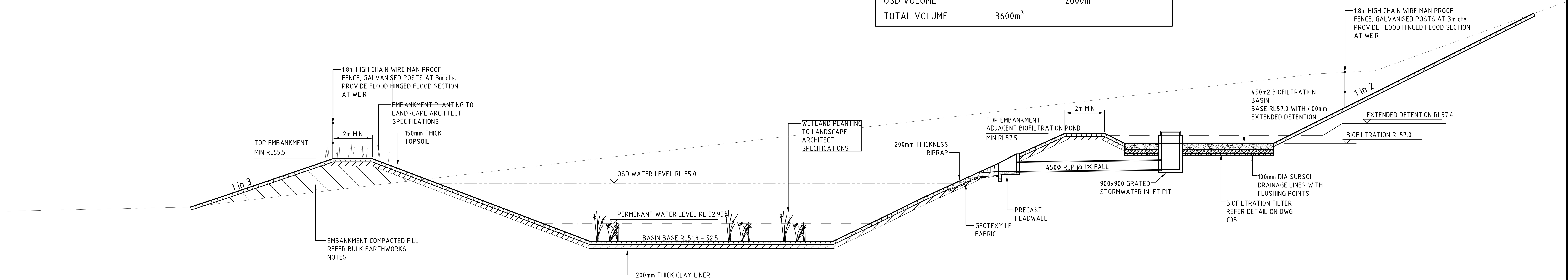

SECTION 1
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Issue	Description	Date
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Client: DIAL A DUMP

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Grid: MGA
Height Datum: AHD
Drawn: AM
Designed: AM
Checked:
Approved:
File:

Project: LIGHT HORSE BUSINESS PARK EASTERN CREEK

Title: NORTHERN BASIN SITEWORKS SECTIONS

Civil Engineers and Project Managers
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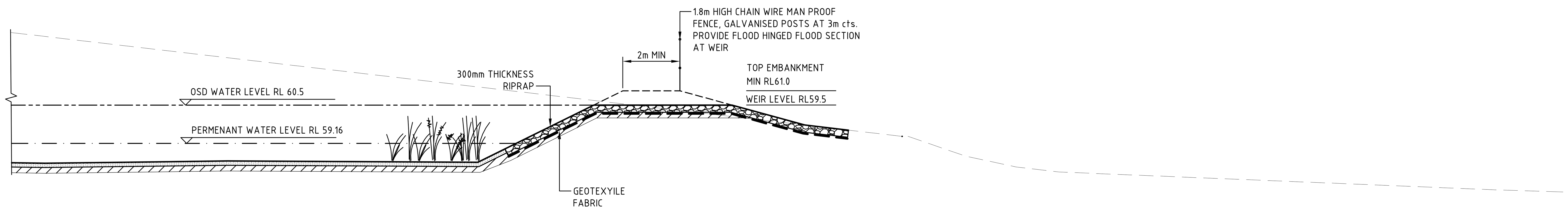
Suite 702, 6A Glen Street
Milsons Point NSW 2061
ABN 96 130 882 405
Tel: 02 8920 2466
Fax: 02 9922 5102
www.atl.net.au
info@atl.net.au

Status: APPROVAL — NOT TO BE USED FOR CONSTRUCTION
A1
Drawing No. C03 | Project No. 10-16 | Issue A



SECTION 5
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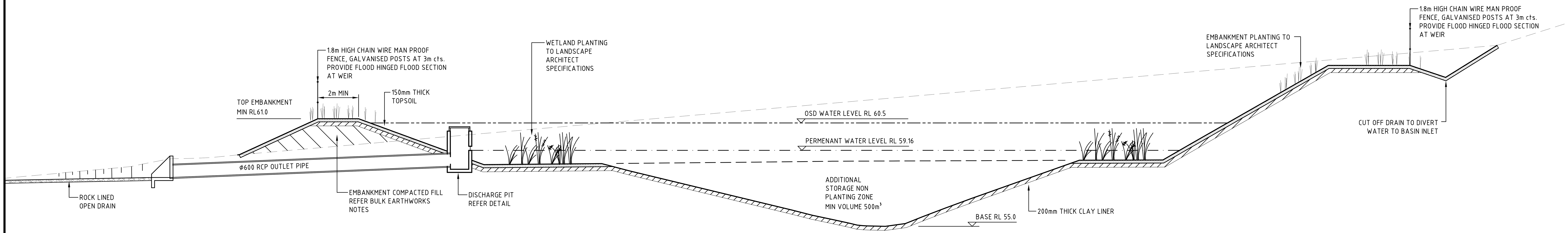
C02



SECTION 4
1 : 100
C02

Issue	Description	Date
A	ISSUED FOR APPROVAL	01-12-11

Client: DIAL A DUMP

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Scales: 1:100 @ A1, 1:200 @ A3
Grid: MGA
Height Datum: AHD
Drawn: AM
Designed: AM
Checked:
Approved:
File:

Project: LIGHT HORSE BUSINESS PARK EASTERN CREEK

Title: SOUHERN BASIN SITEWORKS SECTIONS

Civil Engineers and Project Managers
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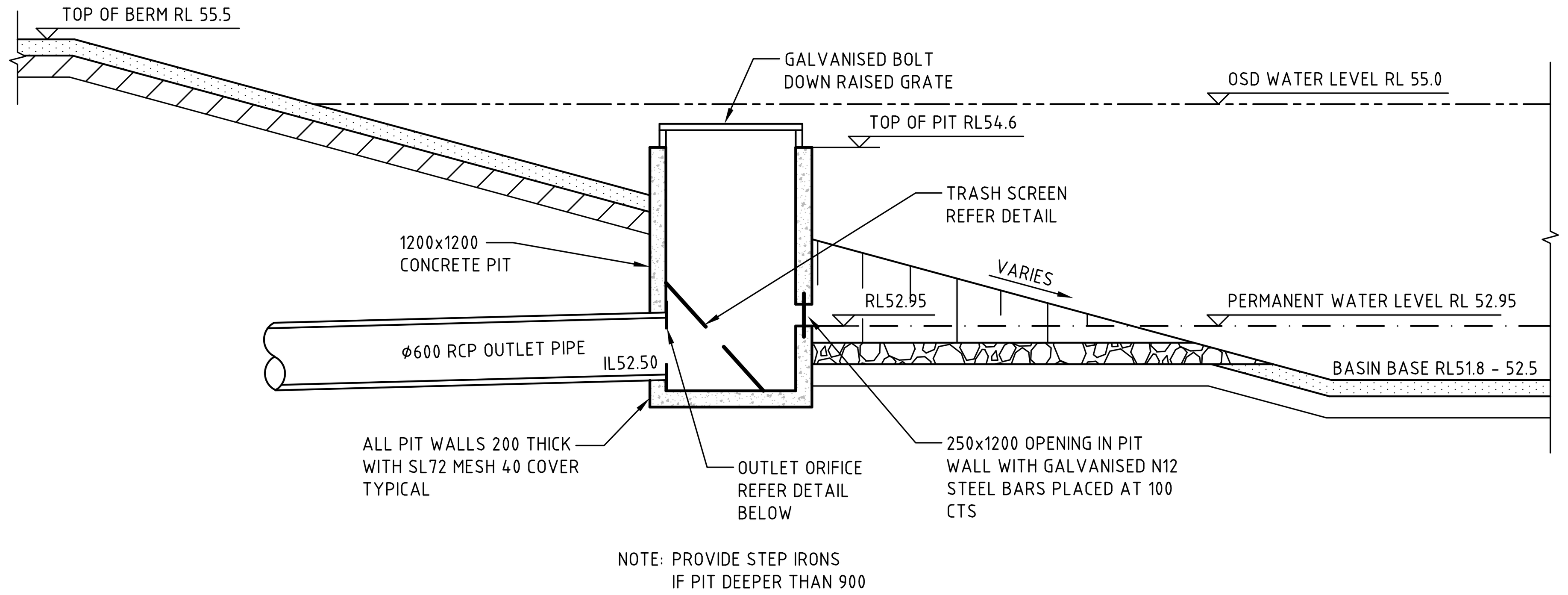
Status: APPROVAL
NOT TO BE USED FOR CONSTRUCTION
A1

Drawing No.	Project No.	Issue
C04	10-16	A

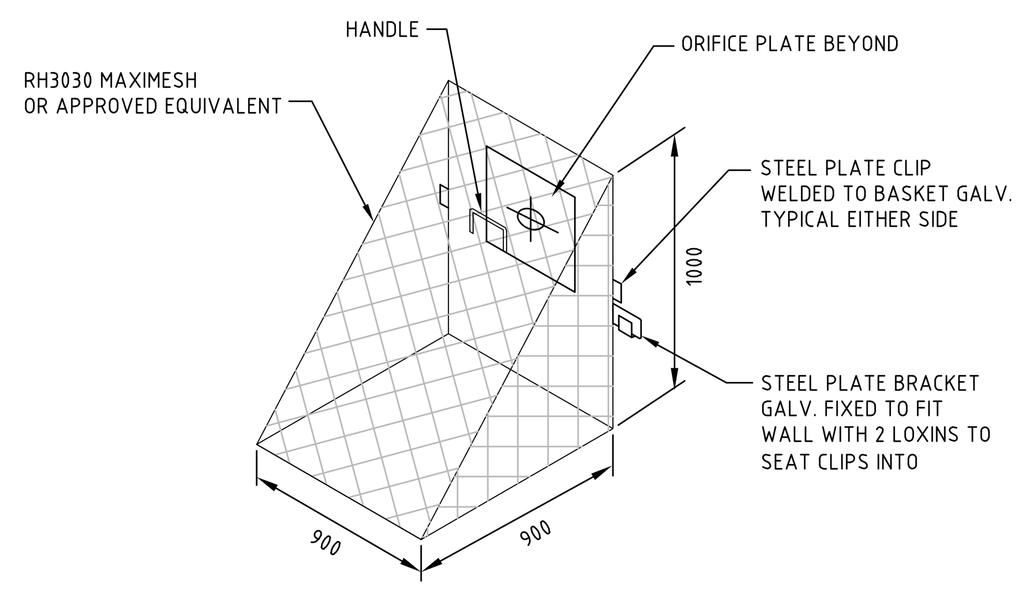
100mm on Original

F:\10-16 Dial A Dump\Drgs\Civil\Final\C04 Basin Sections.dwg
V1



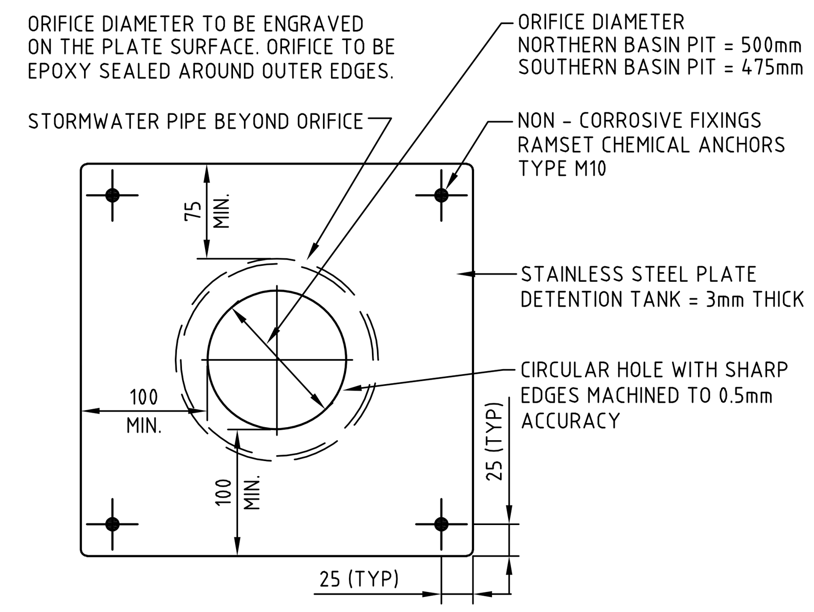
NORTHERN BASIN PIT A\1 DETAIL

SCALE 1:50



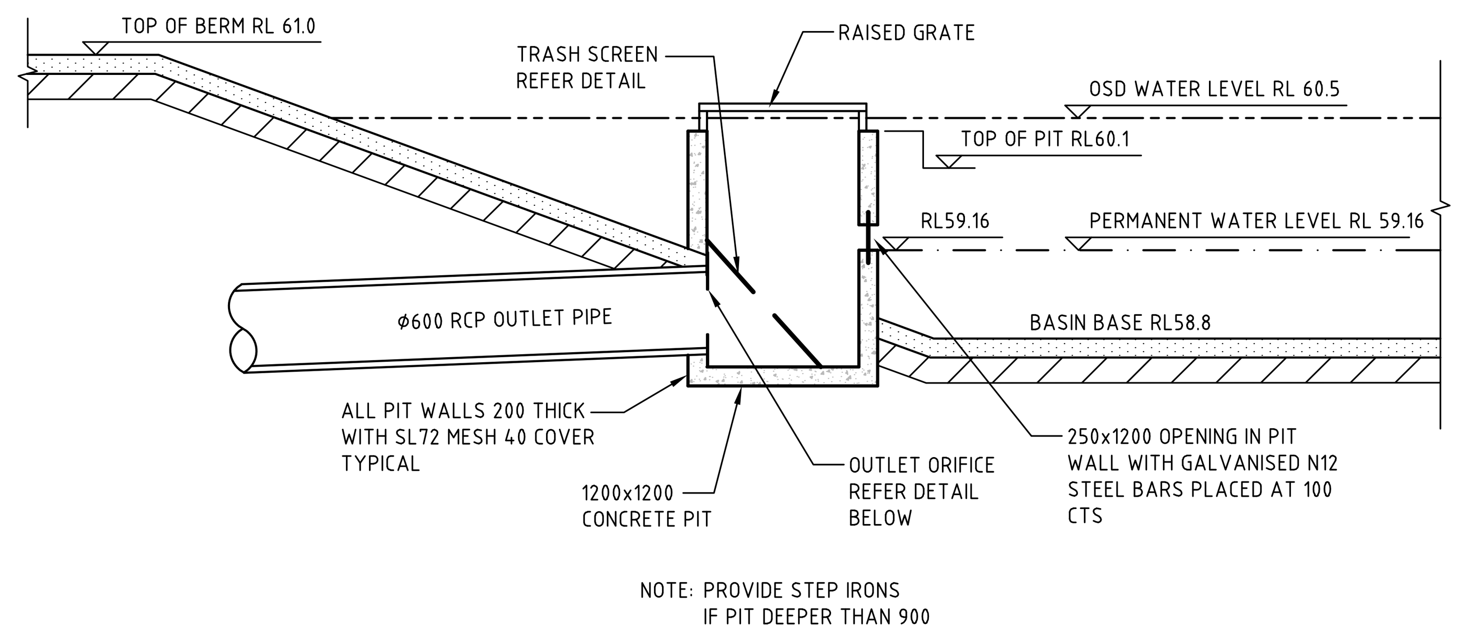
TRASH SCREEN DETAIL

SCALE 1:10



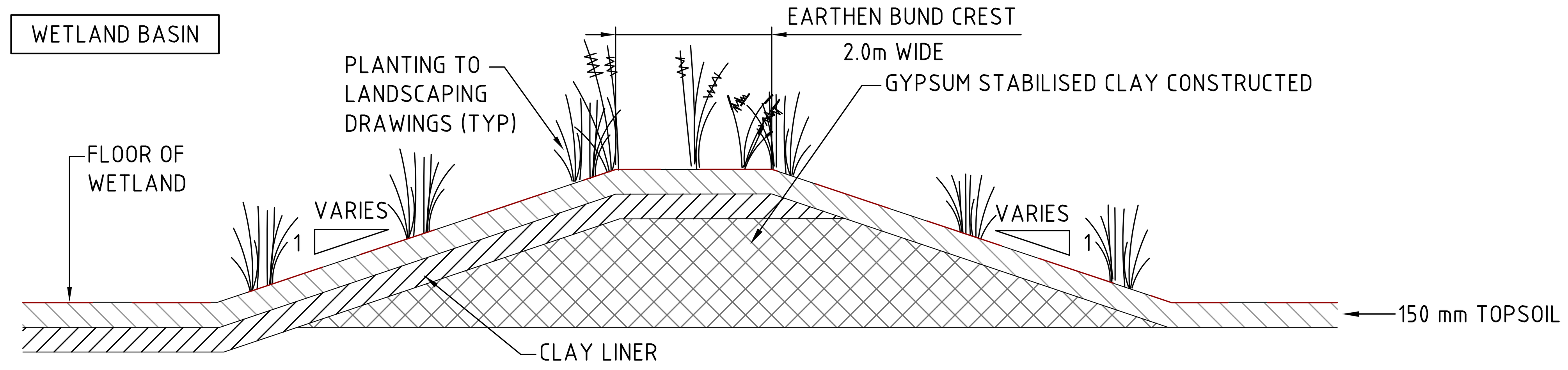
ORIFICE PLATE DETAIL

SCALE 1:5



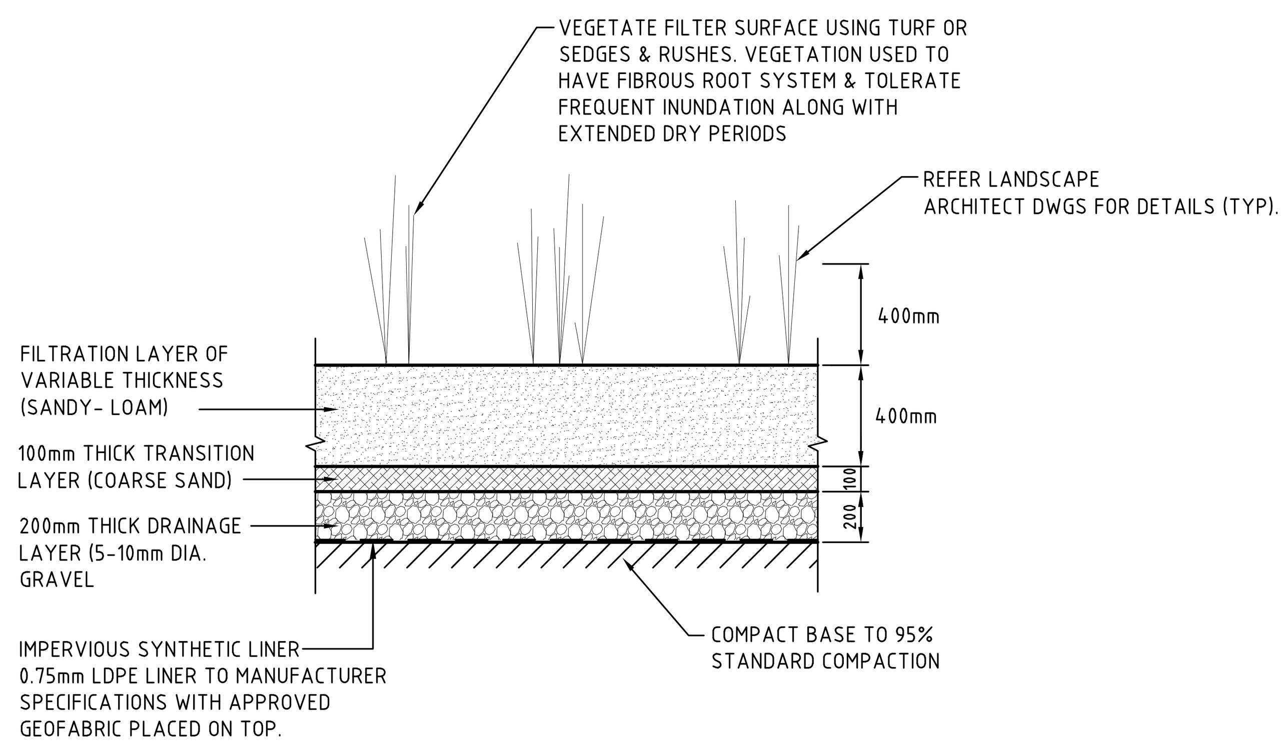
SOUTHERN BASIN PIT A\3 DETAIL

SCALE 1:50



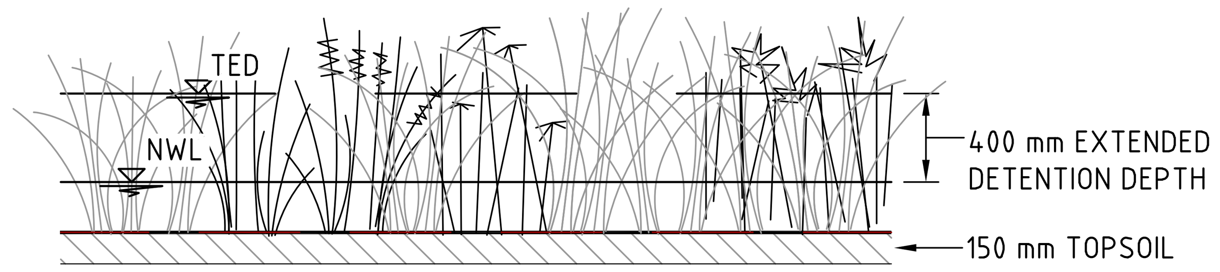
WETLAND FLOW DEFLECTOR / DIVIDER TYPICAL DETAIL

NOT TO SCALE



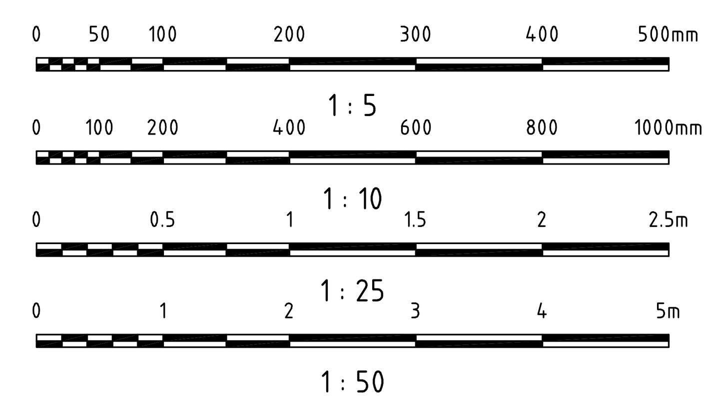
BIOFILTRATION FILTER DETAIL

SCALE 1:25



WETLAND TYPICAL DETAIL

NOT TO SCALE



Issue	Description	Date
A	ISSUED FOR APPROVAL	01-12-11

Client: DIAL A DUMP

Scale Bar

Scales	AS SHOWN	Drawn	MM
		Designed	AMc
Grid	MGA	Checked	
Height Datum	AHD	Approved	

File

Project: LIGHT HORSE BUSINESS PARK EASTERN CREEK

Title: BASIN TYPICAL DETAILS

Civil Engineers and Project Managers
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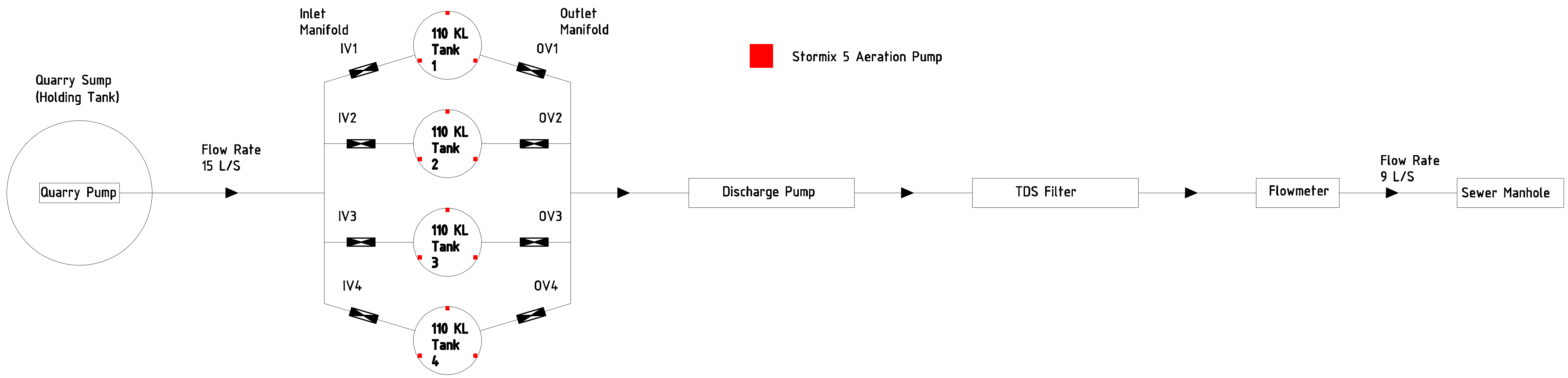
Fax: 02 9922 5102
www.atl.net.au
info@atl.net.au

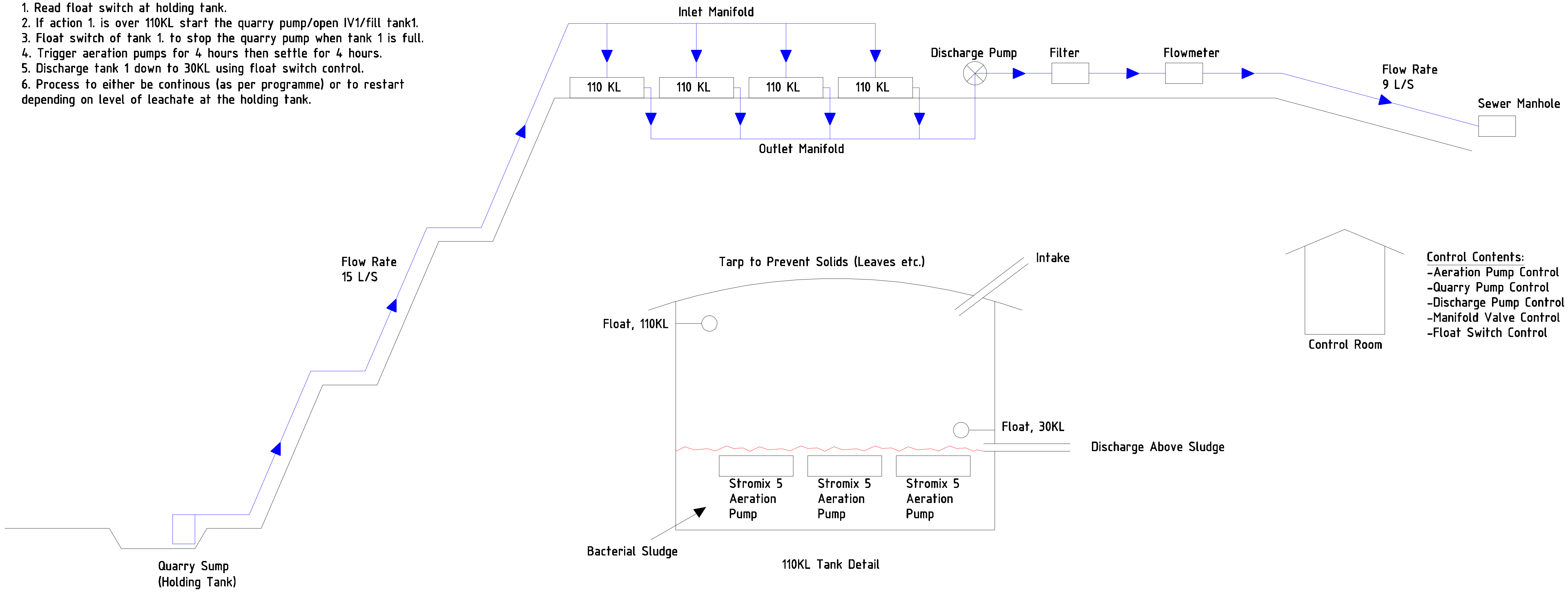
Status: PRELIMINARY ONLY
NOT TO BE USED FOR CONSTRUCTION

A1

Drawing No.	Project No.	Issue
C05	10-16	A



1. Read float switch at holding tank.
2. If action 1. is over 110KL start the quarry pump/open IV1/fill tank1.
3. Float switch of tank 1. to stop the quarry pump when tank 1 is full.
4. Trigger aeration pumps for 4 hours then settle for 4 hours.
5. Discharge tank 1 down to 30KL using float switch control.
6. Process to either be continous (as per programme) or to restart depending on level of leachate at the holding tank.



RevNo	Revision note	Date	Signature	Checked

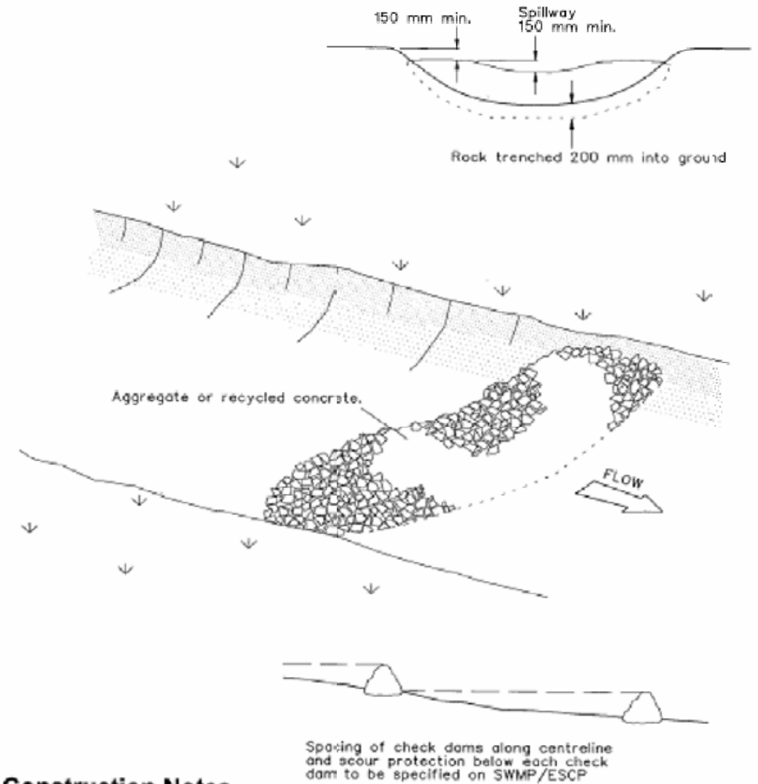
Designed by	Checked by	Approved by - date	Filename	Date	Scale
MD	IM			28/7/11	NTS

LEACHATE PUMPING

	Edition	Sheet
DS146_1	0	1/1

Appendix B

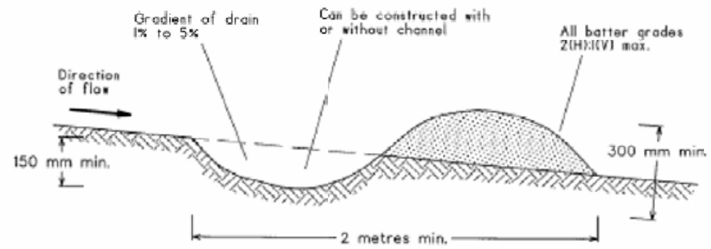
Blue Book Drawings



Construction Notes

1. Check dams can be built with various materials, including rocks, logs, sandbags and straw bales. The maintenance program should ensure their integrity is retained, especially where constructed with straw bales. In the case of bales, this might require their replacement each two to four months.
2. Trench the check dam 200 mm into the ground across its whole width. Where rock is used, fill the trenches to at least 100 mm above the ground surface to reduce the risk of undercutting.
3. Normally, their maximum height should not exceed 600 mm above the gully floor. The centre should act as a spillway, being at least 150 mm lower than the outer edges.
4. Space the dams so the toe of the upstream dam is level with the spillway of the next downstream dam.

ROCK CHECK DAM **SD 5-4**



NOTE: Only to be used as temporary bank where maximum upslope length is 80 metres.

Construction Notes

1. Build with gradients between 1 percent and 5 percent.
2. Avoid removing trees and shrubs if possible - work around them.
3. Ensure the structures are free of projections or other irregularities that could impede water flow.
4. Build the drains with circular, parabolic or trapezoidal cross sections, not V shaped.
5. Ensure the banks are properly compacted to prevent failure.
6. Complete permanent or temporary stabilisation within 10 days of construction.

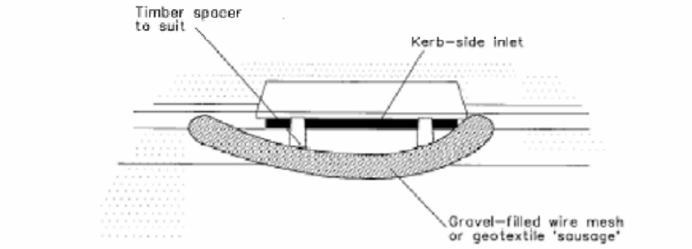
EARTH BANK (LOW FLOW) **SD 5-5**

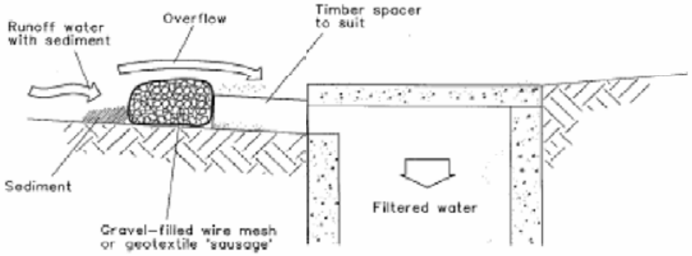
Douglas Partners
Geotechnics•Environment•Groundwater
Brisbane, Cairns, Canberra, Darwin, Gold Coast, Melbourne, Minto
Newcastle, Perth, Sydney Sunshine Coast, Townsville, Wollongong, Wyong

Title: SOIL, WATER AND LEACHATE MANAGEMENT PLAN - LIGHT HORSE BUSINESS CENTRE, EASTERN CREEK, NSW

SOURCE: Landcom (2004) Managing Urban Stormwater: Soils and Construction

Client: ALEXANDRIA LANDFILL PTY LTD		Office: PERTH	
Drawn By: J. RUSSELL	Scale: N/A	Project No: 46950	Revision: A
Approved By: --		Date: 26-11-09	**APPENDIX B-1**



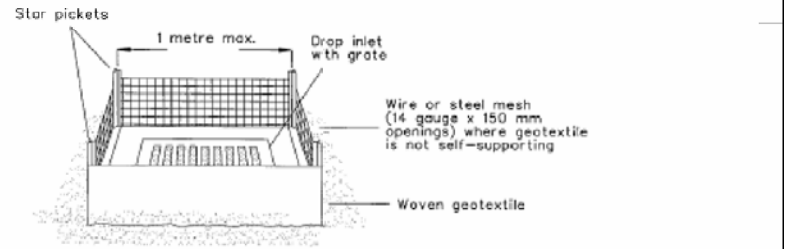


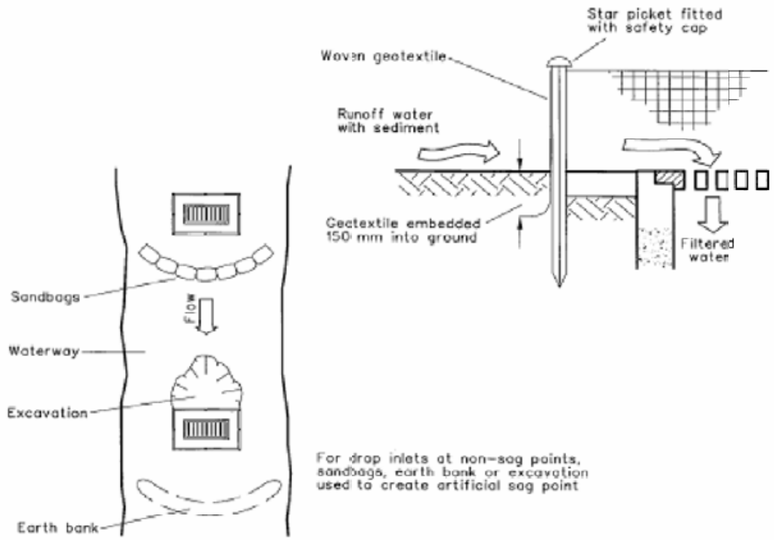
NOTE: This practice only to be used where specified in an approved SWMP/ESCP.

Construction Notes

1. Install filters to kerb inlets only at sag points.
2. Fabricate a sleeve made from geotextile or wire mesh longer than the length of the inlet pit and fill it with 25 mm to 50 mm gravel.
3. Form an elliptical cross-section about 150 mm high x 400 mm wide.
4. Place the filter at the opening leaving at least a 100-mm space between it and the kerb inlet. Maintain the opening with spacer blocks.
5. Form a seal with the kerb to prevent sediment bypassing the filter.
6. Sandbags filled with gravel can substitute for the mesh or geotextile providing they are placed so that they firmly abut each other and sediment-laden waters cannot pass between.

MESH AND GRAVEL INLET FILTER **SD 6-11**





Construction Notes

1. Fabricate a sediment barrier made from geotextile or straw bales.
2. Follow Standard Drawing 6-7 and Standard Drawing 6-8 for installation procedures for the straw bales or geofabric. Reduce the picket spacing to 1 metre centres.
3. In waterways, artificial sag points can be created with sandbags or earth banks as shown in the drawing.
4. Do not cover the inlet with geotextile unless the design is adequate to allow for all waters to bypass it.

GEOTEXTILE INLET FILTER **SD 6-12**

Douglas Partners
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Brisbane, Cairns, Canberra, Darwin, Gold Coast, Melbourne, Minto
Newcastle, Perth, Sydney, Sunshine Coast, Townsville, Wollongong, Wyong

Title: SOIL, WATER AND LEACHATE MANAGEMENT PLAN - LIGHT HORSE BUSINESS CENTRE, EASTERN CREEK, NSW

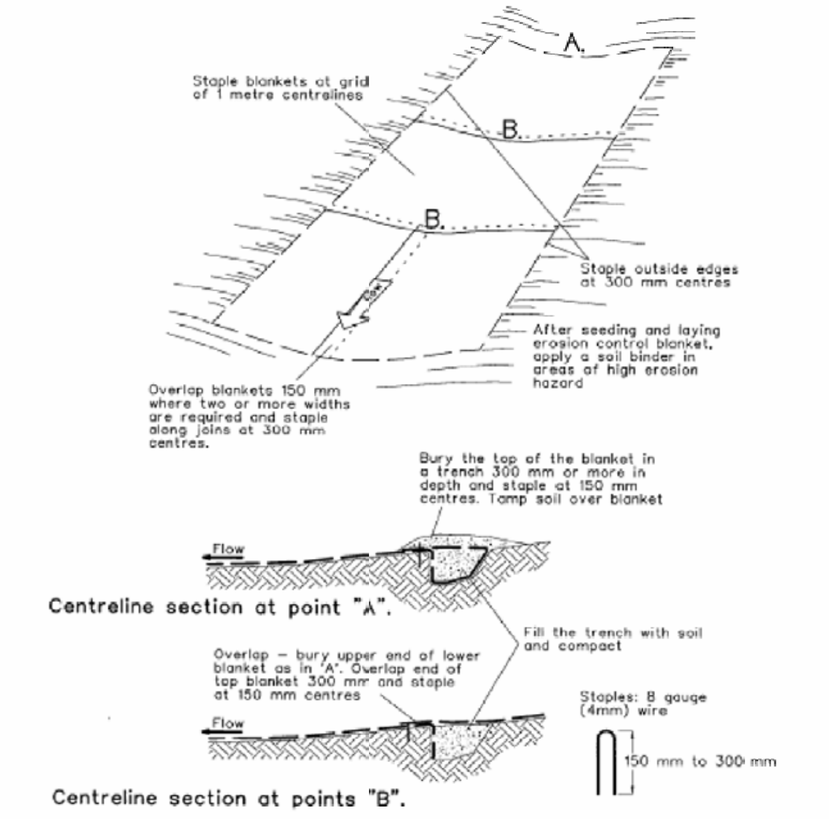
SOURCE: Landcom (2004) Managing Urban Stormwater: Soils and Construction

Client: ALEXANDRIA LANDFILL PTY LTD | Office: PERTH

Drawn By: J. RUSSELL | Scale: N/A | Project No: 46950 | Revision: A

Approved By: -- | Date: 26-11-09

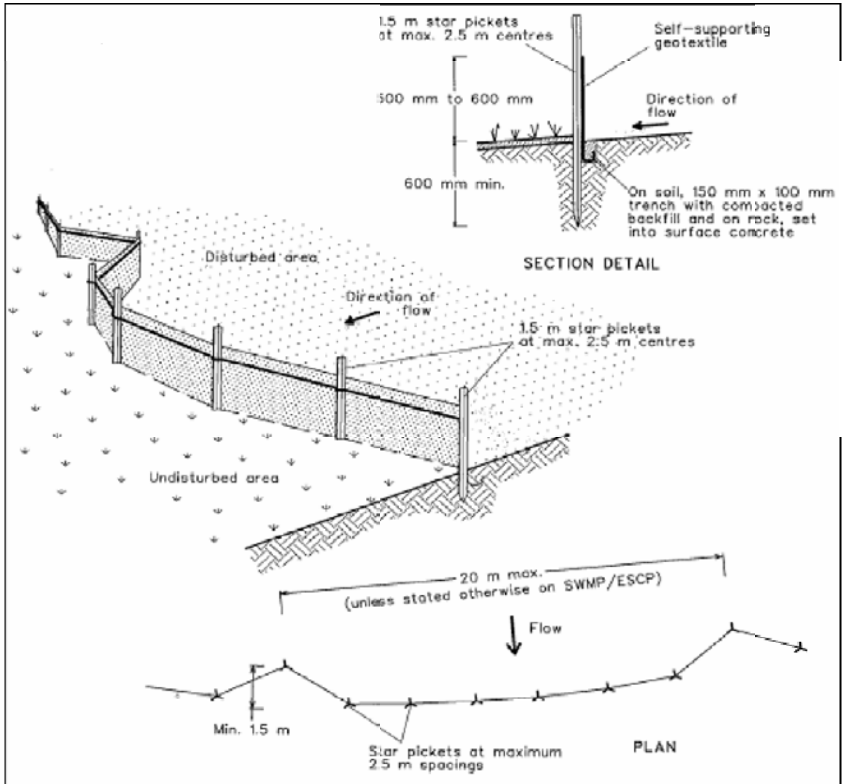
APPENDIX B-2



Construction Notes

1. Remove any rocks, clods, sticks or grass from the surface before laying matting
2. Ensure that topsoil is at least 75 mm deep.
3. Complete fertilising and seeding before laying the matting.
4. Ensure fabric will be continuously in contact with the soil by grading the surface carefully first.
5. Lay the fabric in "shingle-fashion", with the end of each upstream roll overlapping those downstream. Ensure each roll is anchored properly at its upslope end (Standard Drawing 5-7b).
6. Ensure that the full width of flow in the channel is covered by the matting up to the design storm event, usually in the 10-year ARI time of concentration storm event.
7. Divert water from the structure until vegetation is stabilised properly.

RECP : CONCENTRATED FLOW **SD 5-7**



Construction Notes

1. Construct sediment fences as close as possible to being parallel to the contours of the site, but with small returns as shown in the drawing to limit the catchment area of any one section. The catchment area should be small enough to limit water flow if concentrated at one point to 50 litres per second in the design storm event, usually the 10-year event.
2. Cut a 150-mm deep trench along the upslope line of the fence for the bottom of the fabric to be entrenched.
3. Drive 1.5 metre long star pickets into ground at 2.5 metre intervals (max) at the downslope edge of the trench. Ensure any star pickets are fitted with safety caps.
4. Fix self-supporting geotextile to the upslope side of the posts ensuring it goes to the base of the trench. Fix the geotextile with wire ties or as recommended by the manufacturer. Only use geotextile specifically produced for sediment fencing. The use of shade cloth for this purpose is not satisfactory.
5. Join sections of fabric at a support post with a 150-mm overlap.
6. Backfill the trench over the base of the fabric and compact it thoroughly over the geotextile.

SEDIMENT FENCE **SD 6-8**

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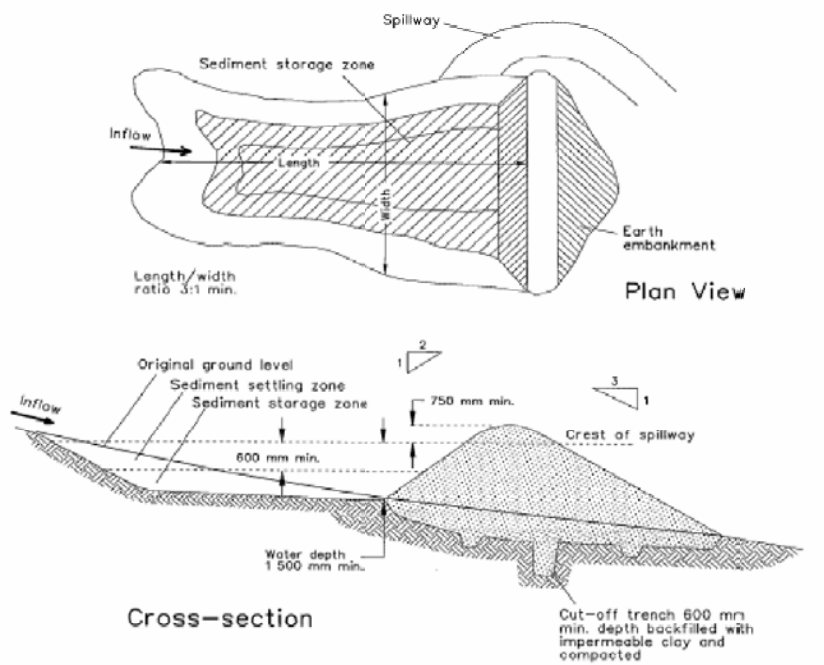
Brisbane, Cairns, Canberra, Darwin, Gold Coast, Melbourne, Minto
Newcastle, Perth, Sydney Sunshine Coast, Townsville, Wollongong, Wyong

Title: SOIL, WATER AND LEACHATE MANAGEMENT PLAN - LIGHT HORSE BUSINESS CENTRE, EASTERN CREEK, NSW

SOURCE: Landcom (2004) Managing Urban Stormwater: Soils and Construction

Client: ALEXANDRIA LANDFILL PTY LTD		Office: PERTH	
Drawn By: J. RUSSELL	Scale: N/A	Project No: 46950	Revision: A
Approved By: --		Date: 26-11-09	**APPENDIX B-3**



Construction Notes

1. Remove all vegetation and topsoil from under the dam wall and from within the storage area.
2. Construct a cut-off trench 500 mm deep and 1,200 mm wide along the centreline of the embankment extending to a point on the gully wall level with the riser crest.
3. Maintain the trench free of water and recompact the materials with equipment as specified in the SWMP to 95 per cent Standard Proctor Density.
4. Select fill following the SWMP that is free of roots, wood, rock, large stone or foreign material.
5. Prepare the site under the embankment by ripping to at least 100 mm to help bond compacted fill to the existing substrate.
6. Spread the fill in 100 mm to 150 mm layers and compact it at optimum moisture content following the SWMP.
7. Construct the emergency spillway.
8. Rehabilitate the structure following the SWMP.

EARTH BASIN - WET

(APPLIES TO 'TYPE D' AND 'TYPE F' SOILS ONLY)

SD 6-4

Douglas Partners

Geotechnics•Environment•Groundwater

Brisbane, Cairns, Canberra, Darwin, Gold Coast, Melbourne, Minto

Newcastle, Perth, Sydney, Sunshine Coast, Townsville, Wollongong, Wyong

Title: SOIL, WATER AND LEACHATE MANAGEMENT PLAN - LIGHT HORSE BUSINESS CENTRE, EASTERN CREEK, NSW

SOURCE: Landcom (2004) Managing Urban Stormwater: Soils and Construction

Client: ALEXANDRIA LANDFILL PTY LTD			Office: PERTH
Drawn By: J. RUSSELL	Scale: N/A	Project No: 46950	**APPENDIX B-4**
Approved By: --		Date: 26-11-09	Revision: A

Appendix C

Martens Consulting Engineers (2011),
Consolidated Stormwater Management Plan

Dial A Dump Industries Pty Ltd

martens consulting engineers

Consolidated Stormwater Management Plan

Light Horse Business Park, Eastern Creek, NSW

ENVIRONMENTAL

WATER

WASTEWATER

GEOTECHNICAL

CIVIL

PROJECT MANAGEMENT

P1103002JR01V05
October 2011

Limitations Statement

The sole purpose of this report and the associated services performed by Martens & Associates Pty Ltd is to prepare a Consolidated Stormwater Management Plan in accordance with the scope of services set out in the contract / quotation between Martens & Associates Pty Ltd and Dial A Dump Industries Pty Ltd (hereafter known as the Client). That scope of works and services were defined by the requests of the Client, by the time and budgetary constraints imposed by the Client, and by the availability of access to the site.

Martens & Associates Pty Ltd derived the data in this report primarily from a number of sources which may include for example site inspections, correspondence regarding the proposal, examination of records in the public domain, interviews with individuals with information about the site or the project, and field explorations conducted on the dates indicated. The passage of time, manifestation of latent conditions or impacts of future events may require further examination / exploration of the site and subsequent data analyses, together with a re-evaluation of the findings, observations and conclusions expressed in this report.

In preparing this report, Martens & Associates Pty Ltd may have relied upon and presumed accurate certain information (or absence thereof) relative to the site. Except as otherwise stated in the report, Martens & Associates Pty Ltd has not attempted to verify the accuracy of completeness of any such information (including for example survey data supplied by others).

The findings, observations and conclusions expressed by Martens & Associates Pty Ltd in this report are not, and should not be considered an opinion concerning the completeness and accuracy of information supplied by others. No warranty or guarantee, whether express or implied, is made with respect to the data reported or to the findings, observations and conclusions expressed in this report. Further, such data, findings and conclusions are based solely upon site conditions, information and drawings supplied by the Client *etc*. in existence at the time of the investigation.

This report has been prepared on behalf of and for the exclusive use of the Client, and is subject to and issued in connection with the provisions of the agreement between Martens & Associates Pty Ltd and the Client. Martens & Associates Pty Ltd accepts no liability or responsibility whatsoever for or in respect of any use of or reliance upon this report by any third party.

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1	DoP Submission	Final	10.03.2011	1E,1P,1H	1H	1P	
2	DoP Revised Final	Final	20.06.2011	1E,1P,1H	1H	1P	
3	OSD Assessment	Final	19.07.2011	1E,1P,1H	1H	1P	
4	Water Quality Assessment and OSD amendments	Final	22.09.2011	1E,1P,1H	1H	1P	
5	Respond to BCC 11/10/2011	Final	15.11.2011	1E,1P,1H	1H	1P	

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All enquiries regarding this project are to be directed to the Project Manager.

Contents

1 Introduction

1.1 Overview

Martens and Associates has been commissioning by Dial-A-Dump Industries (the client) to prepare a consolidated Stormwater Management Plan for the site.

1.2 Scope

This document shall:

- Reassess stormwater quantity controls through modelling of constructed OSD basins to determine their adequacy to achieve site objectives.
- Reassess stormwater quality outcomes achieved by the proposed treatment train, including the OSD basins, to confirm their adequacy and compliance with Blacktown City Council policy.
- Document the final design stormwater solution for the site.

1.3 Site Areas

1.3.1 Fill Pad – 'Area D'

'Area D' lies to the north of the quarry pit and is bound by a conservation area to the west, the M4 motorway to the north and a vacant future industrial lot to the east. Prior to recent works, the area was a grassed paddock. It has since been filled and no formal use is proposed. The area shall revegetate with grass.

1.3.2 Operational Area

The operational area includes hardstand area and buildings and is located directly west of the quarry pit. The area is bound by the conservation area and Area D to the north, a large earth bund and OSD basins beyond to the west (see Section 1.3.3) and cleared grasslands to the south.

Within this area, waste is to be processed for recycling or disposal to landfill.

Design layout of the operational area is provided in previously submitted Storm Consulting documentation. Final layout plan is reproduced in Attachment A.

Processing of waste within the MPC building will ensure that waste does not come into contact with stormwater.

To the extent that water may be used within the MPC building, basal grates ensure that drainage is wholly captured within the building, with excess overflow being directed to the leachate collection system.

Processing of green waste will take place in the designated green waste area at the northwestern corner of the operational area. This area is concreted and equipped with a sump, submersible (float switch activated) pump and co-located collection tanks. Concrete berms at the entrance and exit points to a height of 30cm ensure that the bunded concrete green waste area is capable of containing stormwater from a 12 hour, 1 in 100 year storm event. In the event that the pump failed, the area has been designed and equipped with overflow pipes directing surplus leachate to the in-pit leachate collection system. All runoff from the green waste area is either recycled or discharged to the quarry pit thence trade waste sewer connection.

The greater part of the operational area will drain to the northern GPT and thereafter to the northern bioremediation basin, wetland and OSD pond. The remainder will drain towards the southern GPT and southern OSD, or to the quarry and be captured by the in-pit stormwater collection pond.

Sediment control will be provided by the use of GPTs (CDS units) and open swales, bioremediation basin and wetlands constructed within OSDs.

1.3.3 Western Section

This area lies to the west of the operational area, and contains the OSD basins servicing the operational area. There is no formal landuse proposed and the area shall be allowed to regrass.

1.4 Past Documentation

Previously prepared stormwater documentation and its relevance to this management plan is summarised as follows:

- LandPartners, 2011 *'Area D Finished Surface Contours'*

  Provides proposed finished contours for 'Area D'. As no formal use of the areas is proposed at this stage, no OSD is required.

  The Precinct plan adopted by Blacktown City Council foreshadows the construction of a regional detention basin at or near the north eastern corner of the Conservation area.

  Currently a temporary sediment pond occupies this area and, in the future when Area D is developed, that future application will address details of the permanent stormwater detention facility.

- Storm Consulting, 2008 *'Site Surface Water Management Plan'*

  This document (Attachment C) addresses the main waste processing area and details drainage requirements and, now superceded, water quality controls and OSD measures.

  OSD and water quality modelling by Storm (2008) has been superseded by Martens and Associates (2011) modelling.

- Storm Consulting, 2009 *'Light Horse Business Centre Pavement Setout and Drainage Plans'*

  We understand these plans have been prepared on the basis of more detailed modelling of site hydrology and are relied on for catchment details.

- G R Hawkes and Associates (2010) '*Volume Capacity Southern Basin'* and '*Volume Capacity North Basin'*.

  We understand these plans provide the most accurate survey of the existing north and south basin. These plans (Attachment B) are relied on to determine 'as built" basin volumes for DRAINS hydraulic modelling.

- NSW Dam Safety Committee Advice (2010)

  The Landowner has obtained confirmation from the NSW Dam Safety Committee that the OSDs in the position and at the volumes as shown are not prescribed structures. This advice is provided as Attachment D.

2 Southern Riparian Zone

2.1 Overview

Site Image Landscape Architects Pty Ltd was commissioned by the client to prepare a Riparian Zone Management Plan covering the areas located in Lot 3 DP 1145808. This area, as part of SEPP 59, has been dedicated to conservation. It contains an intermittent watercourse which runs the length of this boundary.

At the time of inspection by Site Image, this creek was found to be highly affected by erosion and sedimentation. Riparian vegetation was limited and mainly consisted of noxious weeds. An unauthorized diversion trench had been temporarily constructed to divert overflow waters from neighbouring sedimentation dams while creek works were carried out.

2.2 Reinstatement of Riparian Zone

The following management plan has been approved by DECCW and DoP and works have subsequently been carried out at the site.

- Sediment and erosion control measures were installed as required. Trees requiring protection during works identified.
- Fill material and sediment within the watercourse was removed and used to fill the diversion trench and reinstate the ground level in that area.
- The watercourse was reinstated to reflect its original channel form.
- Channel was lined with rocks and gravel to address future scouring and erosion.
- Topsoil was replaced utilising material stockpiled onsite.
- The banks, restored watercourse and other areas affected by restoration works were revegetated by spray seeding of suitable native grasses. This extends 10m either side of the creek from the creek centreline.
- Regular weed control is being undertaken.

The Precinct Plan stipulates a 40 metre buffer from top of bank on each side of a scheduled watercourse. The riparian zone abutting the unnamed tributary to Ropes Creek is located in Lot 3 DP 1145808. This is not a project area, and no development works are proposed for this area. It is currently vacant and not cultivated.

The Precinct Plan stipulates a 10 metre set back from the Upper Angus Creek area. This riparian area is located on neighbouring land to Lot 4 DP 1145808 owned by Sumy Pty Ltd and similarly no works or development is proposed for this area.

3 Surface Water Quantity

3.1 Overview

This section provides results of hydraulic modelling of constructed basins. It assesses the existing structures adequacy to reduce post-development discharges to equal or less than pre-development discharges for the 2, 10 and 100 year ARI storms. It provides the final stormwater quantity solution for the development.

We note that hydraulic modelling results reported in this document supersedes results provided in Storm (2008) assessment. Requirements are detailed in Sections 3.3 to 3.8.

Subsequent to the submission of a previous version of this report (Version 4 dated 22.09.2011), Blacktown Council advised they required changes to input parameters in the DRAINS model. While the original input parameters are considered adequate and appropriate it was agreed, in subsequent consultation with Council, that remodelling using updated initial and continuing loss inputs would be completed.

3.2 General OSD Requirements

3.2.1 Operational Area

OSD is required for the proposed Operational Area which is subject to change in landuse resulting in an increase in impervious area.

3.2.2 'Area D and the Western Section'

OSD is not required for 'Area D' and the 'Western Section'. On completion of construction, these areas will regrass and therefore will not change hydrologically from the pre-development situation. Area D and the remainder of the western section therefore do not require OSD at this stage. Ultimately, it is assumed these areas shall undergo development for industrial/commercial purposes at which stage OSD shall be provided in accordance with preliminary requirements.

3.3 Modelling

DRAINS modelling was used to analyse site hydrology and confirm that existing OSD structures satisfy water quantity objectives outlines in Section 3.1. Input parameters were varied between modelling detailed in Version 4 of this report and the now reported figures by increasing initial and continuing losses for the pre-development model and for the undeveloped 'bund' catchments to 15 mm and 2.5 mm/hr respectively. It is noted that Council's original requested initial loss of 25 mm was considered unacceptable and, in consultation with Council, it was agreed 15 mm would be used for modelling purposes.

3.4 Catchment Areas

The pre-development and post-development catchment areas used in DRAINS modelling are summarised in Table 1 and shown in Attachment E.

Table 1**:** DRAINS catchment summary

DRAINS Scenario	Catchment ID	Area (ha)	% Impervious	% Pervious	Existing Receiving Basin
Pre-Development	North Catchment	10.67	0	100	NA
	South Catchment	8.68	0	100	NA
Total		**19.35**			
Post-Development	North Stockpile Cat	4.27	100	0	North Basin
	North Operational Cat	3.90	100	0	
	North OSD Bund	2.17	0	100	
	South Operational Cat	5.63	100	0	South Basin
	South OSD Bund	3.47	0	100	
Total		**19.44**			

3.5 DRAINS Results

Results of pre- and post-development DRAINS modelling for 1 in 2, 10 and 100 year ARI storms are summarised in Table 2 (north OSD basin) and Table 3 (south OSD basin). Storm durations of 25 minutes to 9 hours (540 minutes) are assessed.

Table 2: North OSD Basin - Summary of DRAINS results (total flows) for 25-540 minute duration storm flows for design storm events

Duration (mins)	Q_2 Pre-development peak flow (m^3/s)	Q_2 Developed peak flow (m^3/s)	Q_2 Difference (m^3/s)	Q_{10} Pre-development peak flow (m^3/s)	Q_{10} Developed peak flow (m^3/s)	Q_{10} Difference (m^3/s)	Q_{100} Pre-development peak flow (m^3/s)	Q_{100} Developed peak flow (m^3/s)	Q_{100} Difference (m^3/s)
25	0.51	0.46	-0.05	1.31	0.57	-0.74	2.64	1.47	-1.17
30	0.57	0.45	-0.12	1.48	0.58	-0.90	2.83	1.60	-1.23
60	0.97	0.51	-0.46	1.91	0.65	-1.26	3.13	2.40	-0.73
90	1.13	0.52	-0.61	1.99	0.66	-1.33	3.09	2.52	-0.57
120	1.03	0.51	-0.52	2.03	0.66	-1.37	3.20	2.66	-0.54
180	0.76	0.49	-0.27	1.59	0.64	-0.95	2.56	1.62	-0.94
270	0.96	0.49	-0.47	1.59	0.64	-0.95	2.26	1.82	-0.44
360	0.82	0.48	-0.34	1.23	0.63	-0.6	1.74	1.47	-0.27
540	0.72	0.47	-0.25	1.08	0.61	-0.47	1.53	1.22	-0.31

Table 3: South OSD Basin - Summary of DRAINS results (total flows) for 25-540 minute duration storm flows for design storm events

Duration (mins)	Q_2 Pre-development peak flow (m^3/s)	Q_2 Developed peak flow (m^3/s)	Q_2 Difference (m^3/s)	Q_{10} Pre-development peak flow (m^3/s)	Q_{10} Developed peak flow (m^3/s)	Q_{10} Difference (m^3/s)	Q_{100} Pre-development peak flow (m^3/s)	Q_{100} Developed peak flow (m^3/s)	Q_{100} Difference (m^3/s)
25	0.45	0.25	-0.20	1.15	0.38	-0.77	2.31	1.02	-1.29
30	0.50	0.25	-0.25	1.27	0.39	-0.88	2.40	1.19	-1.21
60	0.82	0.31	-0.51	1.62	0.45	-1.17	2.61	1.84	-0.77
90	0.96	0.33	-0.63	1.69	0.47	-1.22	2.62	1.94	-0.68
120	0.88	0.32	-0.56	1.74	0.47	-1.27	2.68	2.06	-0.62
180	0.64	0.30	-0.34	1.36	0.45	-0.91	2.12	1.20	-0.92
270	0.79	0.31	-0.48	1.32	0.45	-0.87	1.87	1.50	-0.37
360	0.67	0.31	-0.36	1.00	0.46	-0.54	1.42	1.24	-0.18
540	0.58	0.34	-0.24	0.88	0.47	-0.41	1.25	1.17	-0.08

Results demonstrate existing OSD basins provide a reduction in downslope storm flows from post-development to pre-development for all storm durations modelled for the 2, 10 and 100 year ARI events. OSD basins therefore achieve objectives set in Section 3.1 for stormwater quantity.

3.6 OSD Storage Capacities

Stage storage relationships were determined for the OSD basins based on G R Hawkes and Associates (2010) survey (Attachment B). The capacities from G R Hawkes are summarised in Table 4 with allowance for 500 kL permanent pool volume in the northern basin and 1000 kL in the southern - of which 500 kL is to be constructed in addition to volumes surveyed.

Table 4: OSD Capacity Details

Basin	Level (mAHD)	Measured Area (m^2)	GR Hawkes Volume[1]
North Basin	52.0	227	0
	52.9	800	500[2]
	53.0	1018	545
	54.0	1415	1763
	55.0	1830	3384
	55.24 (1% TWL)		3838
	55.5	2012	4349
South Basin	59.15	1040	1000[2]
	60.0	1687	2257[2]
	60.5	1857	3142[2]
	60.70 (1% TWL)		3519
	61.0	2025	4115[2]

Note: [1] Volume based on survey provided by G R Hawkes and Associates with [2]assumed increase of 500 kL for Southern Basin; 500kL / 1000 kL of permanent pool volume is provided below outlet invert of the Northern and Southern basins respectively.

Based on Table 4, the required OSD volumes of the North Basin and South Basin were determined to be 2.9 ML (3.4 ML including retained 0.5 ML) and 2.6 ML (3.1 ML including retained 1.0 ML) respectively at the invert of the outlet weir.

3.7 Outlet Structures

Table 5 provides details of the outlet structures modelling in DRAINS and required for the OSD basins to achieve adequate retention of flow.

Table 5: OSD Outlet Details

	Low Level Outlet			Weir Data	
Basin	**Type**	**Diameter (mm)**	**Centre Elevation (mAHD)**	**Crest Length (m)**	**Crest Level (mAHD)**
North Basin	Orifice	500	53.2	10	55.0
South Basin	Orifice	475	59.4	10	60.5

3.8 Conclusion

DRAINS modelling undertaken by Martens and Associates (2011) concluded that OSD basins as recommended are adequate (with appropriate outlets) to provide a reduction in downslope storm flows from post-development to pre-development for both the 'north' and 'south' catchments.

4 Water Quality Management

4.1 Overview

Water quality MUSIC modelling of the operational area has been previously undertaken by Storm Consulting (2008). Due to modification to OSD basin/wetland configuration, Blacktown City Council have requested in recent correspondence (August 12, 2011) that this modelling be reassessed.

Subsequent to the assessment as completed in Version 4 of this report Council has requested further modifications and amendments to the submitted MUSIC model. These requests and responses are discussed below.

4.2 Blacktown City Council Water Quality Submission

BCC requested a number of model changes, responses to which are provided below:

- Re-use volumes assumed in modelling are 110 kL/day drawn from the two basins. This figure has been determined in consultation with the site operator (annual site usage of 40 ML) and is assigned in the model as 60 kL/day and 50 kL/day from the North and South basins respectively. This estimate is based on existing comparable operators and anticipated site processes and takes into account periods when usage will be lower due to rainfall etc.

  For the purposes of modelling it is assume that the usage is evenly spread through the year as this is the simplest means to enter this parameter to MUSIC. The figure of 110 kL/day reflects 40 ML/year spread across the year, it is acknowledged that on some days a greater amount shall be used and on others a lesser amount.

- Modify treatment of OSD capacity in wetland treatment nodes – BCC considers that inclusion of the extended detention depth (OSD) in the MUSIC model shall result in over estimation of pollutant removal. Analysis of DRAINS inflow / outflow hydrographs indicates that the inclusion of the OSD volume shall result in detention of water for periods comparable to that modelled by MUSIC using input parameters relied on. The inclusion of this as a component of the model is therefore considered reasonable.

Notwithstanding Martens' position that inclusion is appropriate, remodelling without this parameter shows that the change to the final treatment train efficiency is less than 1 % for nutrients (i.e. resolution of reporting) and at most 1 % for suspended solids (northern wetland). To satisfy Council's request amended modelling has assumed a nominal 0.25 m extended detention depth.

- Impervious percentage – the areas of the site to be used for stockpiling aggregates was treated differently between the water quality and water quantity assessment in Version 4 of this report. The stockpiles were considered pervious for the frequent lower intensity water quality storms and impervious for the higher intensity rarer OSD events.

  While this rationale is considered sound and reasonable, for the purposes of conservatism and to address Council's request, the stockpile area shall be treated as impervious in this revision of the assessment.

- Rainfall - Runoff and catchment generation rates – the modelling has been updated to reflect BCC requested input parameters.

4.3 Blacktown City Council Water Quality Objectives

Blacktown City Council's *Stormwater Quality Control Policy* (2005) water quality objectives are adopted as the site's water quality objectives. The following pollutant reduction objectives are set out for comparing the post-development untreated versus treated scenarios:

- 90% reduction in gross pollutants;
- 80% reduction in suspended solids; and
- 45% reduction in nitrogen and phosphorus.

4.4 Treatment Train

The proposed water quality treatment train for the site is:

1. Rainwater Tanks -

   Water balance modelling undertaken by Storm (2008) calculated that 40kL of tank capacity is required to capture roofwater runoff for reuse in toilet flushing and other site uses not addressed by stormwater harvested from OSD basins.

2. Gross Pollutant Traps (GPT) -

   Tank overflow, surface runoff from the remaining operational areas and parts of the site draining west are directed through gross pollutant traps (GPTs).

3. Bioremediation Area

   In addition to the GPTs and water quality wetlands already constructed on-site (basins not 'fitted out' as wetlands at this stage) it is required that a bioremediation basin be constructed to further treat runoff in the northern catchment.

   The required basin shall have the following specifications:

   - Filter area: 450 m^2
   - Extended Detention Area: 450 m^2
   - Extended detention depth : 400 mm
   - Filter Depth: 500 mm
   - Filter media permeability: 200 mm/hour for specification and 100 mm/hr ultimate (as used in modelling to reflect blockage of media over time)
   - Constructed with a system of underdrains discharging into the OSD basin / wetland system.

4. Water Quality Wetland

   GPT discharge is directed to one of 2 wetlands to be used for OSD and stormwater treatment/reuse ('OSD North' and 'OSD South'). As earlier outlined Dial-a-dump confirms an annual average daily flow of 110 kL/day of water is required for site uses (we assume water is preferentially drawn from the northern basin). This volume is in excess of water used from roof water tanks.

   The southern basin is to be deepened at the western side to provide an additional 500 kL of storage capacity below the current invert. The design basin capacity therefore shall be increased to 1000 kL below the outlet invert to maximise the stored water for re-use during dry periods. The northern basin shall remain as built (other than inlet / outlet structures and wetland planting).

4.5 Modelling Methodology

The treatment train efficiency of the trains leading to the outlets of the north and south basins was assessed using MUSIC V5.

4.6 Model Inputs

MUSIC model inputs were sourced as follows:

- A 6-minute daily time step climate file was used for the revised MUSIC modelling. This file was supplied by Blacktown City Council.
- Rainfall – Runoff parameters as provided by Council.
- OSD volumes are based on G R Hawkes and Associates (2010) survey (Attachment B) as amended by this report.

4.7 MUSIC Results

MUSIC model results are provided in Table 6 with catchments and MUSIC model layout provided in Figure 1 (Attachment F).

Table 6: Treatment train pollutant reduction rates modelled in MUSIC.

Parameter	Northern Basin (%)	Southern Basin (%)
Total Suspended Solids	87.3	84.0
Total Phosphorous	69.8	73.2
Total Nitrogen	45.3	45.0
Gross Pollutants	100	100

4.8 Conclusion

Pollutant reductions achieved project water quality objectives (Section 4.3) and therefore Blacktown City Council's requirements demonstrating that the proposed treatment train is adequate for the treatment of site runoff to an appropriate standard.

5 Conclusion

5.1 Water Quantity

DRAINS modelling undertaken by Martens and Associates (2011) confirms OSD basins specified are adequate to achieve post-development storm flows less than pre-development flows for all events required by Blacktown City Council.

5.2 Water Quality

Outcomes from MUSIC modelling confirm that water quality pollutant reduction rates achieved by the proposed treatment train and are in accordance with relevant local policy and are therefore adequate.

5.3 Water Quality Monitoring

Council's Stormwater Quality Control Policy (2005) has now been superseded and quarterly testing is unlikely to demonstrate compliance with Council policy requirements.

Accordingly, it will be necessary to monitor storm events to determine the overall pollutant load reductions. A monitoring and testing program will be implemented to the satisfaction of OEH that tests a certain agreed number of storms at the inlet to the treatment train and at the discharge point.

6 References

LandPartners (2011) *'Area D Finished Surface Contours'*

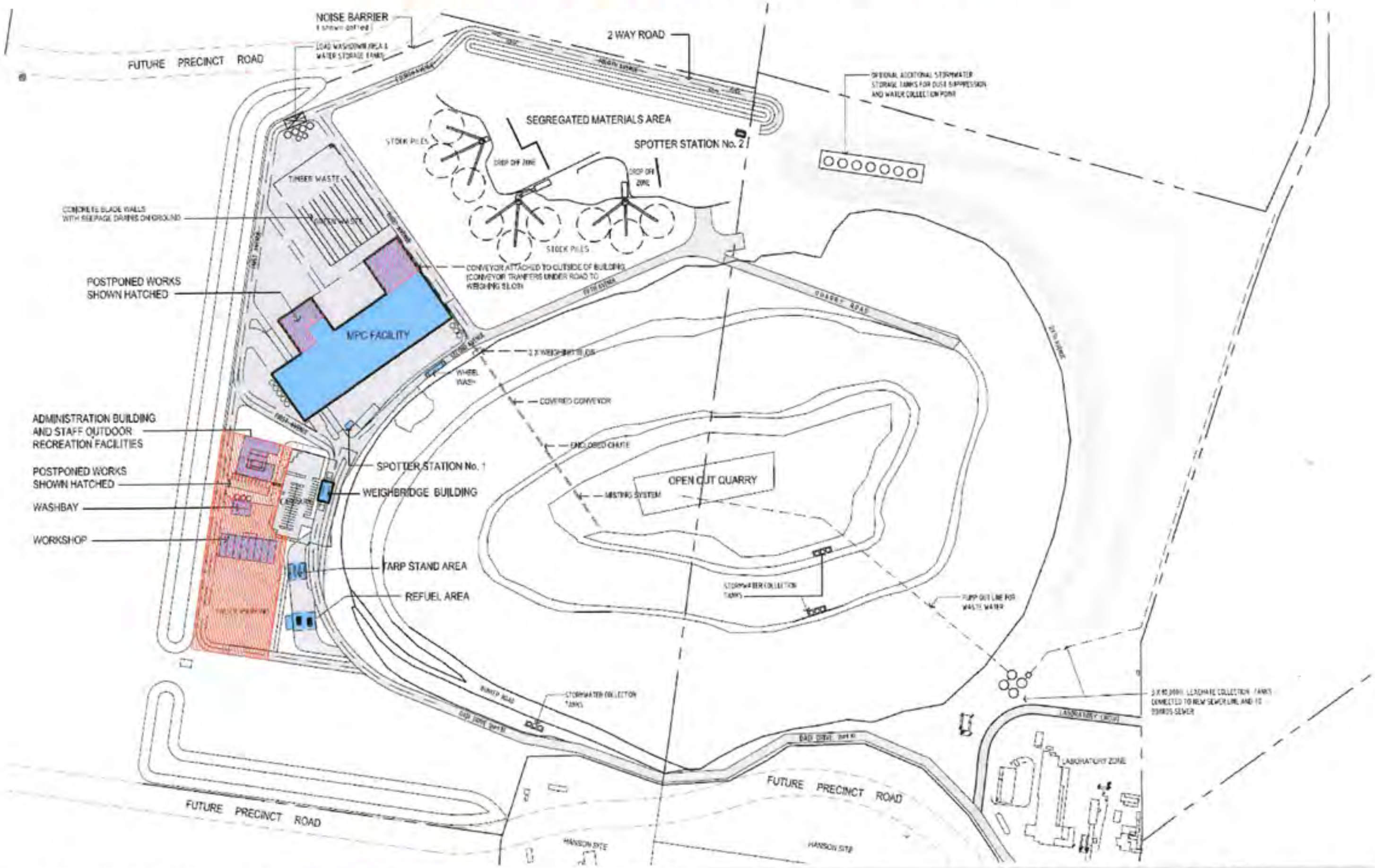
Storm Consulting Pty Ltd (2008) '*Site Surface Water Management Plan'*

Storm Consulting Pty Ltd (2009) *'Light Horse Business Centre Pavement Setout and Drainage Plans'*

G R Hawkes and Associates (2010) '*Volume Capacity Southern Basin'* and '*Volume Capacity North Basin'*

7 Attachment A – Operational Area Layout (Storm Consulting, 2009)

APPENDIX 1 - SITE PLAN & BUILDING ELEVATIONS



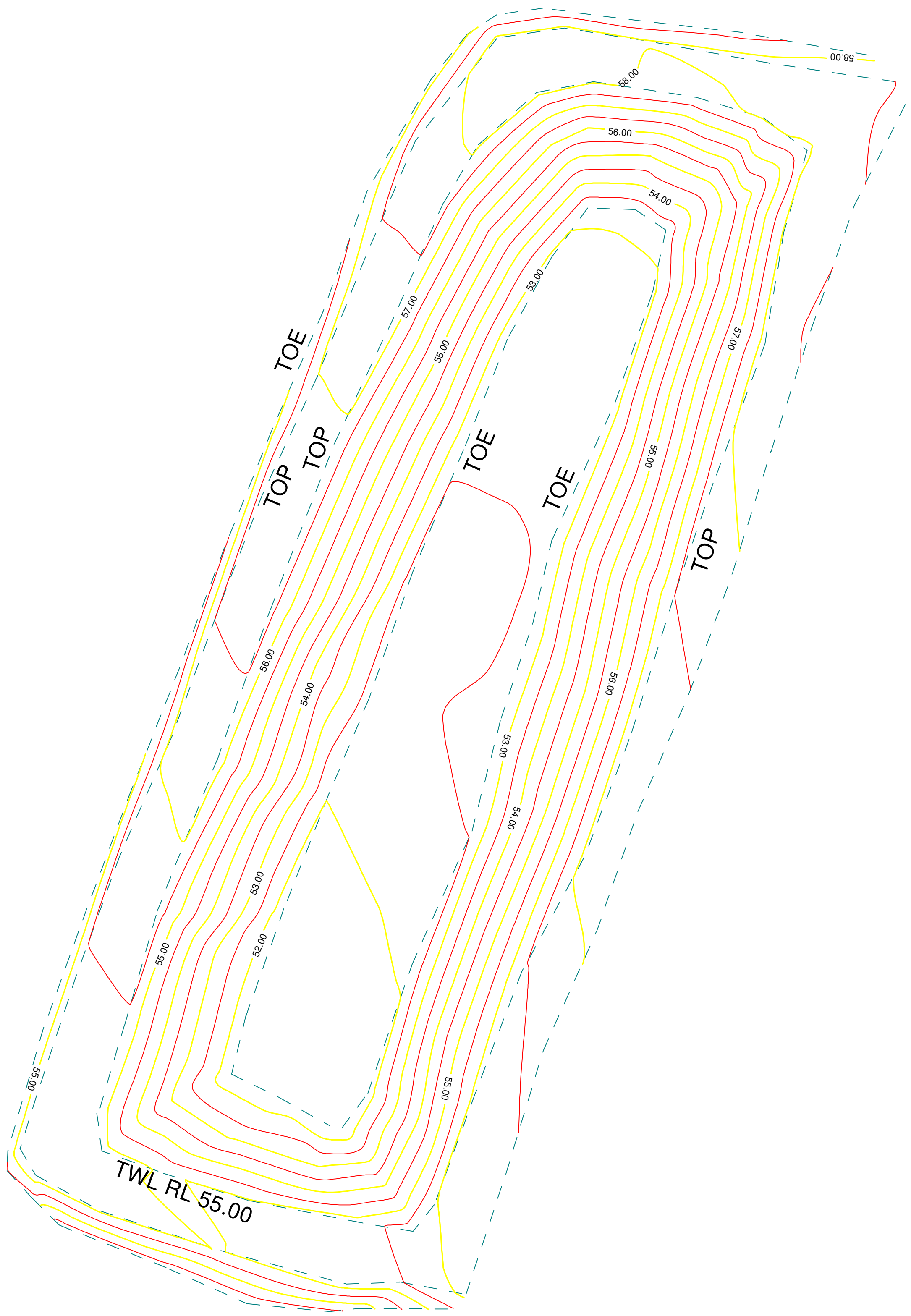
LIGHT HORSE BUSINESS CENTRE
DADI - MATERIALS PROCESSING FACILITY
DIAL A DUMP INDUSTRIES
OLD WALGROVE ROAD, EASTERN CREEK, NSW

DIAL A DUMP
KEEPING AUSTRALIA CLEAN

AXIS architectural
2 / 107 Cronulla Street, Cronulla NSW 2230
p + 02 9523 7858 e + david@axisarchitects.com.au
nominated architect - David McDonald NSW ARB -7997

SITE LAYOUT PLAN
OVERALL SITE
MARCH 2010
1:3000 scale
090507/ DI-EP-MP A 101 - 8 / A

8 Attachment B – G R Hawkes and Associates (2010) OSD Basin Survey



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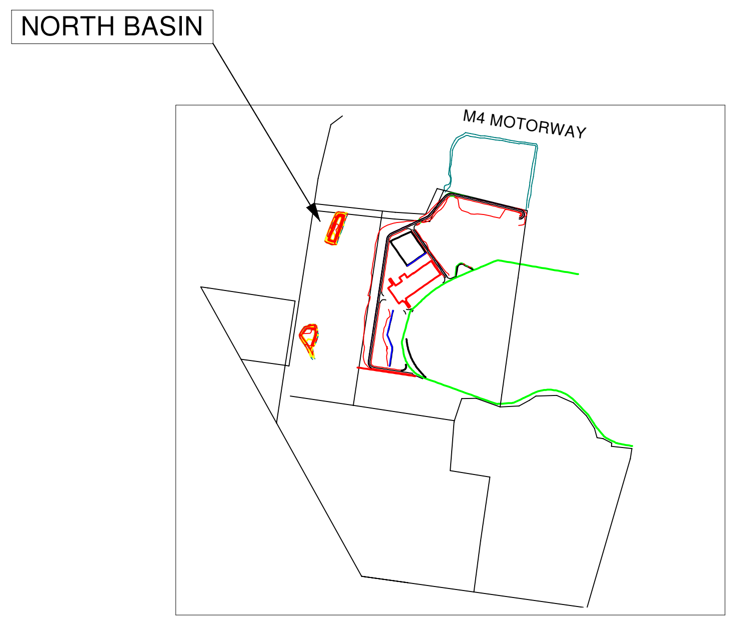


Revision	Date	Description	Drawn By	Approved By
B	13/01/11	Amendments: Volume	AJ	GH
A	1/09/10	Detail Survey	PE	GH

Surveyed By:
GRH Hawkes & Associates Pty Limited
E-Mail: grh@grh.com.au | Mob: 0419 249 667
Web: www.grh.com.au

Title:

VOLUME CAPACITY NORTH BASIN
DIAL A DUMP
LIGHT HORSE BUSINESS CENTRE
EASTERN CREEK

Scale Bar: 0 3 6 9 12 15 30

Surveyed By:	Job No:	Contour Interval:
AJ	1084	0.50
Print Scale:	**Projection:**	**Datum:**
1: 500 @ A3	MGA	AHD

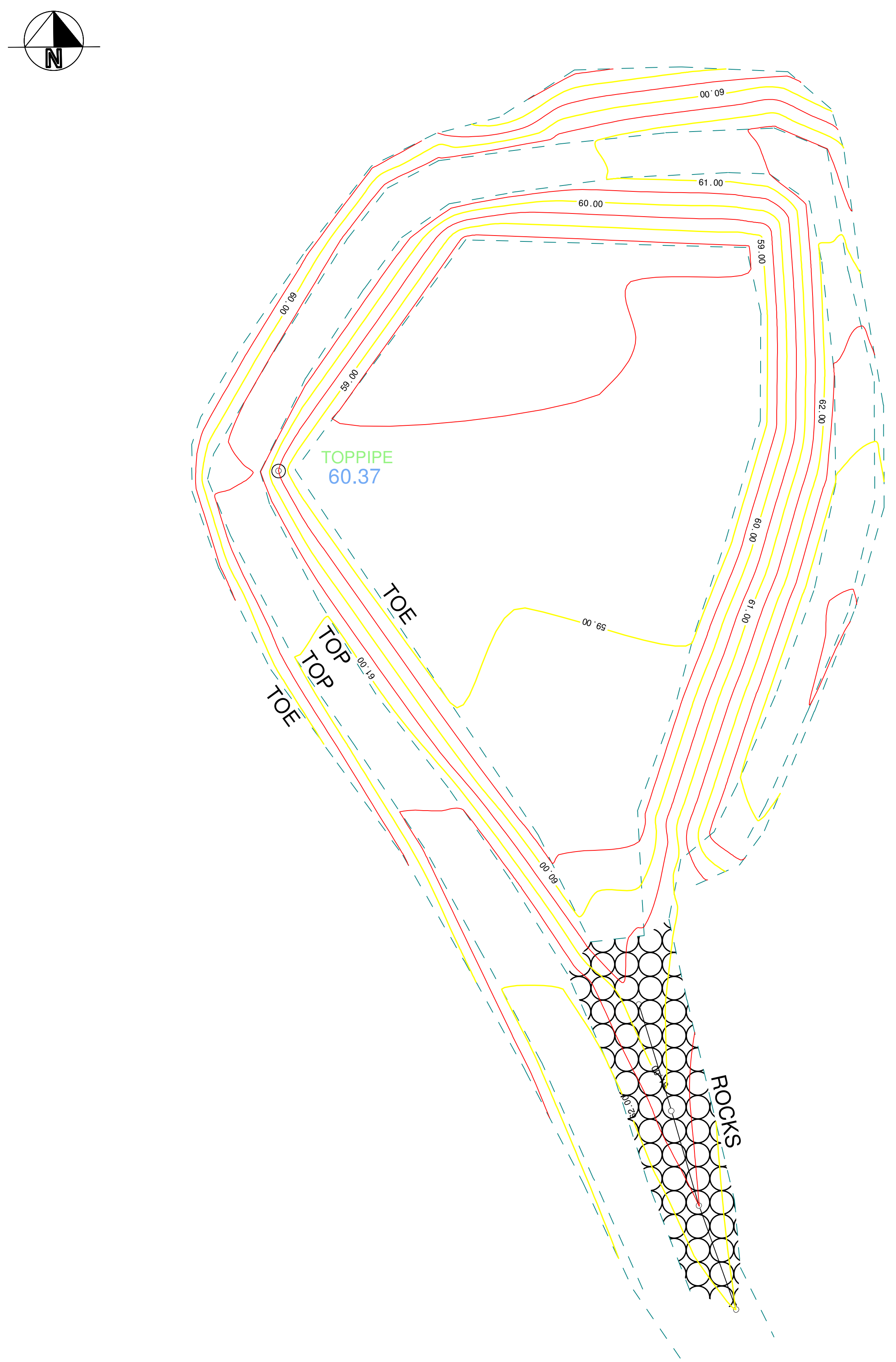
Notes:
#Volume was calculated to Top of Water Level = RL 55.00

Computer File:
S:\GRH\GRH JOB Documents\PE 1084 EASTERN CREEK FILL AREA 26 08 10

Surveyed For:
DIAL A DUMP INDUSTRIES
KEEPING AUSTRALIA CLEAN

Rev: B

Page: 1 - 1



VOLUMES:	
=========	
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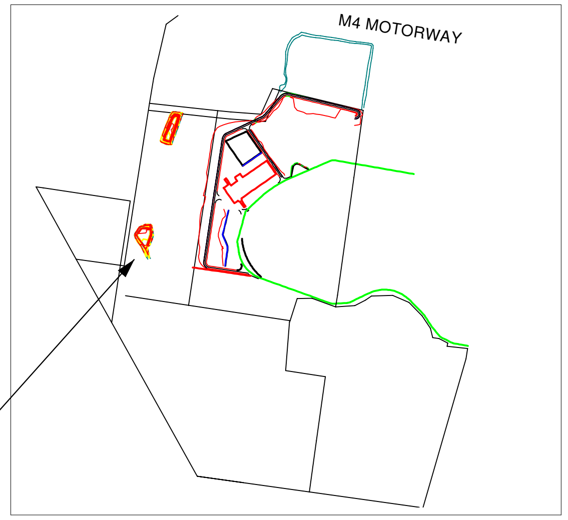

SOUTH BASIN

Revision	Date	Description	Drawn By	Approved By
B	13/01/11	Amendments: Information Removed	AJ	GH
A	1/09/10	Detail Survey	PE	GH

Surveyed By:

GRH Hawkes & Associates Pty Limited

E-Mail: grh@grh.com.au | Mob: 0419 249 667
Web: www.grh.com.au

Title:

VOLUME CAPACITY SOUTH BASIN
DIAL A DUMP
LIGHT HORSE BUSINESS CENTRE
EASTERN CREEK

Scale Bar: 0 3 6 9 12 15 30

Surveyed By:	Job No:	Contour Interval:
PE	1084	0.50
Print Scale:	**Projection:**	**Datum:**
1: 500 @ A3	MGA	AHD

Notes:

#Volume was calculated to top of outlet pipe.

#Volume is approximate only.

#Toe of basin was approximated due to the water in the basin at time of survey.

Computer File: S:\GRH\GRH JOB Documents\

Surveyed For: DIAL A DUMP INDUSTRIES KEEPING AUSTRALIA CLEAN

Rev: B

Page: 1 - 1

9 Attachment C – Site Surface Water Management Plan (Storm Consulting, 2008)

STORM_CONSULTING

Site Surface Water Management Plan

Report Prepared for:
Alexandria Landfill Pty Ltd

November 2008
Project No. 717

Prepared by:
STORM CONSULTING PTY LTD

_ANALYSIS
_CONSENSUS
_DESIGN
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Document Verification

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Description	Light Horse Waste Facility – Surface Water Assessment	
Client Contact	Chris Biggs, DADI	

	Name	Signature	Issue:	Date
Prepared by	Georgia de Biasi / Nicola Lo		A B C D	15/12/07 07/02/08 27/3/08 24/11/08
Checked by	Brad Farr / Nicola Lo			
Issued by	Georgia de Biasi			
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Issue to:	Date	No. Copies	Date	No. Copies	Date	No. Copies		
Jodi Kelehear (ERM)	15/12/07	PDF	7/2/08	PDF				
Rene Bakeri					27/3/08	PDF		
Chris Biggs (DADI)							24/11/08	PDF

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EXECUTIVE SUMMARY

This Surface Water Management report was prepared as part of the Environmental Assessment for the Light Horse Business Centre which is proposed to include the construction and operation of a resource recovery facility and a landfill facility. It was revised following a request for further information. The Project site is located at Eastern Creek in western Sydney and comprises four separate parcels of land, identified as Lot 2 DP 262213, Lot 1 DP 400697, Lot W DP 419612, and Lot 10, DP 241859. Storm has prepared a Surface Water Management Plan (SWMP) taking into account Blacktown City Council's stormwater management objectives and also the Director-General's Requirements for the Project.

The SWMP addresses erosion and sediment control, water quantity and water quality.

In summary, the proposed stormwater management system includes:

- Each building should have its own rainwater tank (min. 10kL volume) to harvest roof water for reuse on site, including for toilet flushing and wheel wash top up;
- Runoff generated from the Materials Processing Centre and green waste area are to be diverted to sewer and managed accordingly;
- Stormwater runoff from the other operational areas of the site will be treated through a gross pollutant trap prior to discharge to a combined on-site detention (OSD) basin with wetland treatment for water quality
- Additional volume is allowed in the OSD / treatment basin for irrigation water storage; drawdown would occur regularly for irrigation and dust suppression.
- The proposed OSD storage requirements for the operational area is 370m^3/Ha (5500m^3 based on 14.8ha impervious area) and has been designed to manage peak flows up to the 1 in 100yr ARI storm event.
- A sprinkler system is proposed to be located along the berms and utilised for both dust suppression and irrigation purposes.
- Stormwater runoff control within the quarry pit is to be used to assist in reducing leachate volumes. A collection basin is proposed which can be drawn down following storm events for reuse for dust suppression by water carts.

TABLE OF CONTENTS

APPENDICES

APPENDIX A

Surface Water Management Plan

APPENDIX B

Blue Book calculations

APPENDIX C

Model outcomes – RAFTS

APPENDIX D

Model – MUSIC layout

1.0 INTRODUCTION

1.1. Background

This is a revised report commissioned by Alexandria Landfill Pty Ltd to provide additional information to that supplied in STORM_CONSULTING's *Site Surface Water Management Plan* dated February 2008.

ThaQuarry Pty Ltd and ACN 114 843 453 Pty Ltd seek project approval for the construction and operation of resource recovery facility (including a materials processing centre (MPC) and waste transfer station (WTS)), and a Class 2 inert and solid waste landfill at Eastern Creek, in Sydney's west. Project approval is sought under Part 3A of the *Environmental Planning and Assessment Act, 1979*. The application process is to be managed on behalf of both parties by ThaQuarry Pty Ltd under the project name Light Horse Business Centre.

STORM_CONSULTING was commissioned by Environmental Resources Management Australia Pty Ltd (ERM) on behalf of the proponent to prepare a site Surface Water Management Plan for the Project, as part of the overall Environmental Assessment. This report has been revised to include additional information and amended data following a request for additional information from Blacktown City Council.

The site's location is shown in Figure 1-1. It is within the Blacktown City Council (BCC) Local Government Area. The Pioneer Quarry previously operated at the site. It has now reached the end of its economic life and all quarrying activities at the site ceased in September 2006, though the quarry void remains.

State Environmental Planning Policy No 59 – Central Western Sydney Economic and Employment Area (SEPP 59) applies to a number of landholdings in western Sydney, including the Project site, which lies within the Eastern Creek Precinct of the SEPP 59 lands.

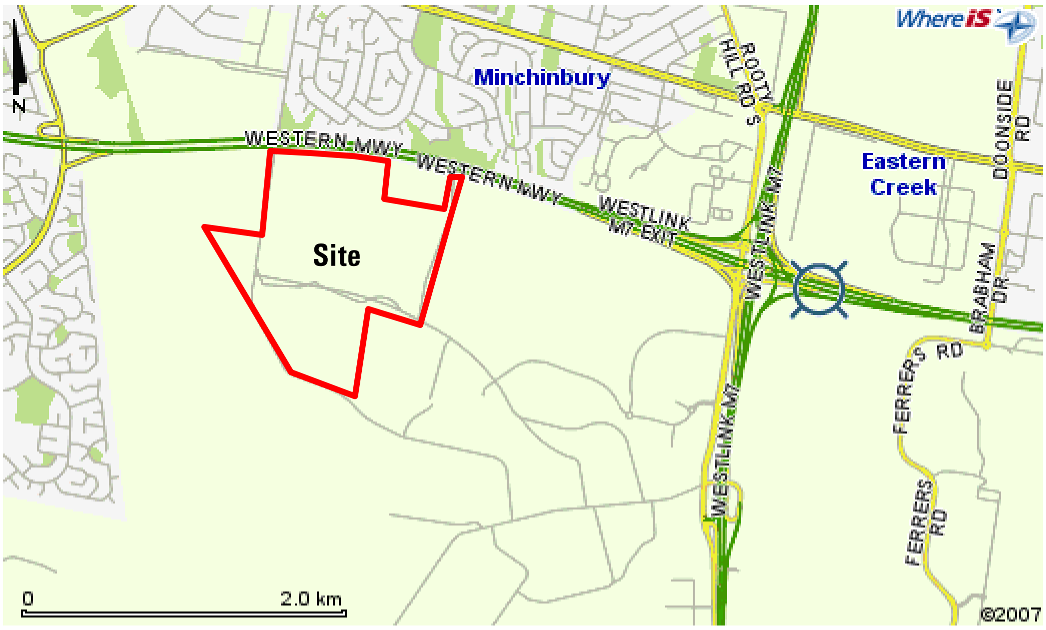


Figure 1-1: Site Location

1.2. Development Overview

For the purposes of this assessment, the area of operations has been divided into two areas, termed the operational area and the quarry area. The operational area is depicted in Figure 1-2 and will be bounded by berms to the north, west and south, and by the quarry pit to the east. It will incorporate the following features:

- Access and internal roads;
- Car and truck parking areas;
- Administration and workshop buildings;
- Weighbridge and associated building;
- Materials Processing Centre (MPC) and Waste Transfer Station (WTS);
- Green waste processing/stockpile area; and
- Drop-off zone and materials stockpile areas.

The quarry area refers to the existing quarry pit, which is the main feature of the site. It is proposed to become a licensed class 2 inert and solid waste landfill.

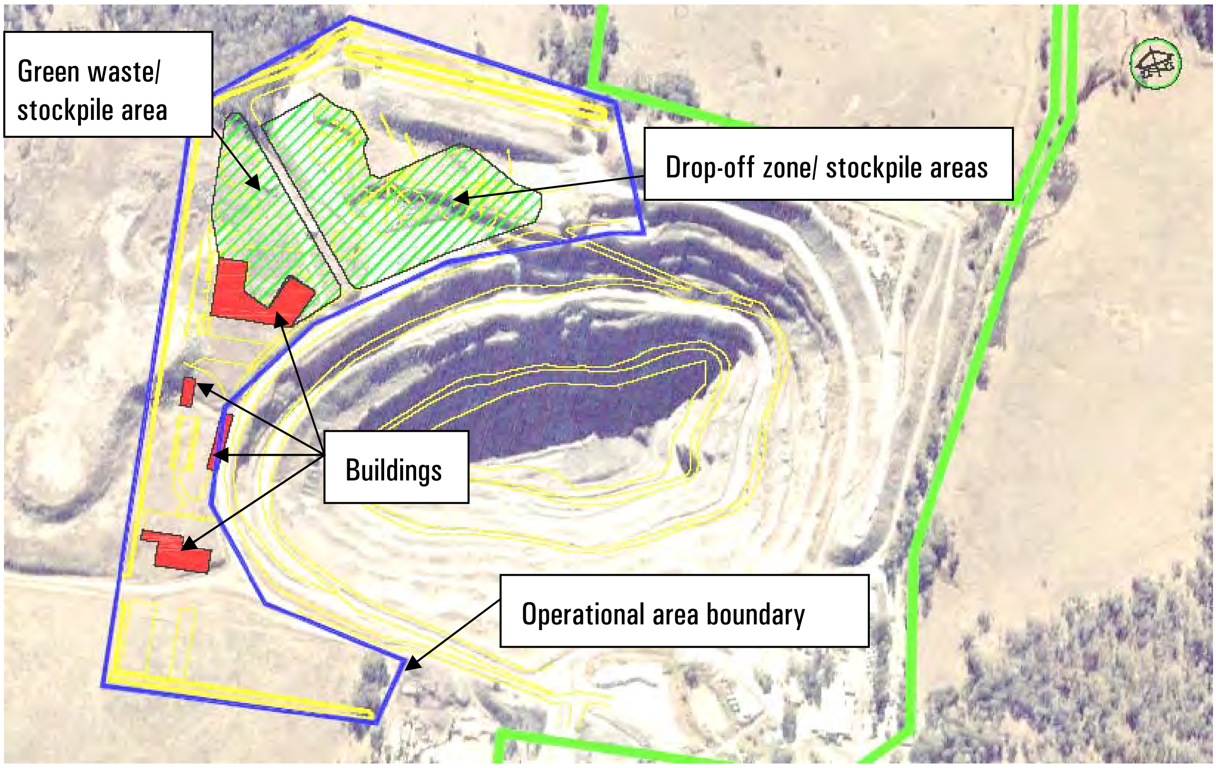


Figure 1-2. Site layout

1.3. Project Scope

STORM's scope of works for this surface water assessment report included:

- Preparation of site catchment plans and justification for any proposed redistribution between catchment areas, as defined in the Precinct Plan;
- Development of a concept stormwater drainage plan, including provision of water sensitive urban design (WSUD) elements where possible;
- Preparation of a soil and water management plan in accordance with Landcom's (2004) *Managing Urban Stormwater- Soils and Construction* – "The Blue Book";

- Stormwater detention calculations to determine relevant details of basins and drainage works;
- Water balance/ water management for wet, dry and average years, including water requirements (quantity, quality and sources) and proposed stormwater and wastewater disposal, including type, volumes, proposed treatment and management methods and reuse options;
- Identification of the quantity and physico-chemical properties of potential water pollutants and the risks they pose;
- Review of flood reports;
- Preparation of a Surface Water Assessment Report.

1.4. Planning Controls and Policy Objectives

1.4.1. Director-General's Requirements

The Director-General's (DG's) requirements issued on 22 June 2006 require a detailed assessment of specified key issues. In this report STORM addresses the DG's requirements for surface water, which are included in the Soil and Water category of Key Issues. These include:

- Surface water impacts;
- Stormwater management, including detailed consideration of any potential offsite drainage and flooding impacts;
- Erosion and sediment control;
- Salinity, in the context of site surface water management.

Other items identified in the DG's requirements (including groundwater, soil contamination, and other aspects of salinity at the site) are beyond the scope of this report and have been addressed in the Environmental Assessment Report prepared by ERM (2008).

Where necessary, STORM has consulted with Blacktown City Council with respect to regulatory requirements.

1.4.2. Eastern Creek Precinct Plan

The *Eastern Creek Precinct Plan – Stage 3 has* been prepared in accordance with the provisions of SEPP 59 for land identified as Release Area 3 within the Eastern Creek Precinct (inclusive of the Project site). The draft Precinct Plan was adopted by Council on 7 December 2005, and came into force on 14 December 2005. It outlines the provisions relating to development of the Stage 3 Release Area, to ensure the SEPP aims are met.

This report aims to ensure the Project meets the relevant Environmental, Urban Amenity, Engineering and Economic objectives as set out in Section 5.5 of the Precinct Plan.

1.4.3. Other Relevant Documents

Other documents considered in the preparation of this report include:

- Institution of Engineers (2000) *Australian Rainfall and Runoff*;
- Blacktown City Council (2005) *Engineering Guide for Development*;
- Landcom (2004) *Managing Urban Stormwater: Soils and Construction Volume 1*, 4th Edition and Volume 2B – Waste Landfills (currently available as a draft for consultation only);
- Blacktown City Council (2005) *Stormwater Quality Control Policy P01100*;
- SMEC (2004) *SEPP59 Landholder Group Eastern Creek Precinct Plan Stormwater Management Strategy*.

2.0 SITE DESCRIPTION

2.1. Location and Land Use

The site (refer Figure 2-1) covers an area of approximately 122ha and comprises 4 lots:

- Lot 2, DP262213
- Lot 10, DP241856
- Lot 1, DP400697
- Lot W, DP419612

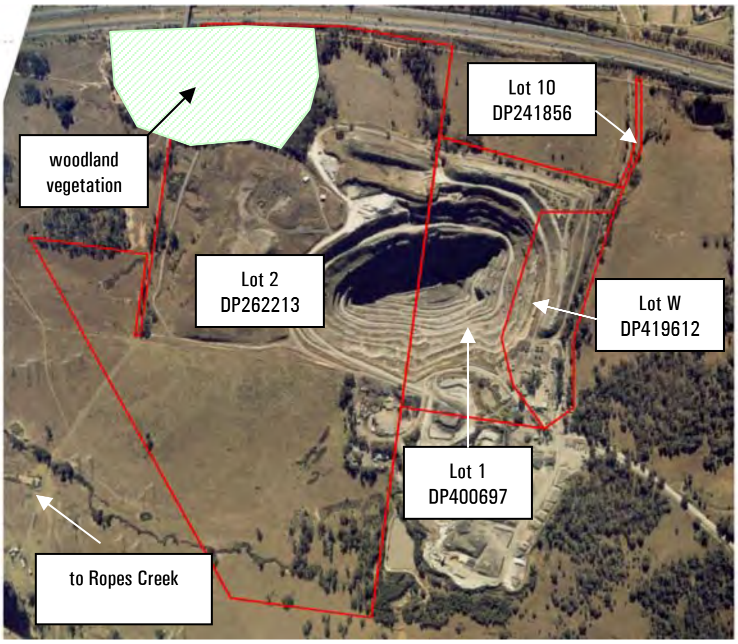


Figure 2-1: Site Boundaries

It is noted that the development footprint as assessed for this report will be restricted to the central portion of the site, as indicated on Figure 1-2.

The site is largely cleared of vegetation and is generally undeveloped beyond the existing quarry pit with associated overburden stockpiles. It is bounded by the M4 motorway to the north, a tributary of Ropes Creek to the south, Archbold Road to the west and open paddocks and the Hanson Asphalt Batching Plant and Hanson yard ('Hanson site') to the east.

In the area proposed for development, a low ridge divides the northern and north-western portions of the site. Native vegetation is largely limited to sparse trees in the north east, far south and west of the site, in addition to an area of remnant woodland in the north west of the site.

2.2. Catchments, Hydrology and Drainage

General overland flow direction across the site is to the north-west and ultimately reaches Ropes Creek approximately 1km west of the site. Ropes Creek flows northwards and is located along the western boundary of the Precinct with a total catchment area of approximately 127Ha. There is an ephemeral drainage line in the northern portion of the site that flows west towards Ropes Creek. To the south of the quarry and beyond the extents of the proposed site operations, overland drainage is generally south to south-west towards a tributary of Ropes Creek.

The site surface water drainage network is characterised by wide, flat and generally poorly defined drainage lines, which is fairly typical of drainage in western Sydney, where low topographic relief and meandering drainage lines dominate the natural landscape.

The Eastern Creek Precinct comprises nine major catchments as identified in Figure 10 of Council's Employment Lands Precinct Plan (2005), with the site located across four of the main catchments (refer Figure 2-2 and Table 2-1):

- Catchment 1: Quarry Catchment;
- Catchment 2: Quarry North Catchment;
- Catchment 3: Upper Angus Creek Catchment; and
- Catchment 6: Ropes Creek Tributary Catchment.

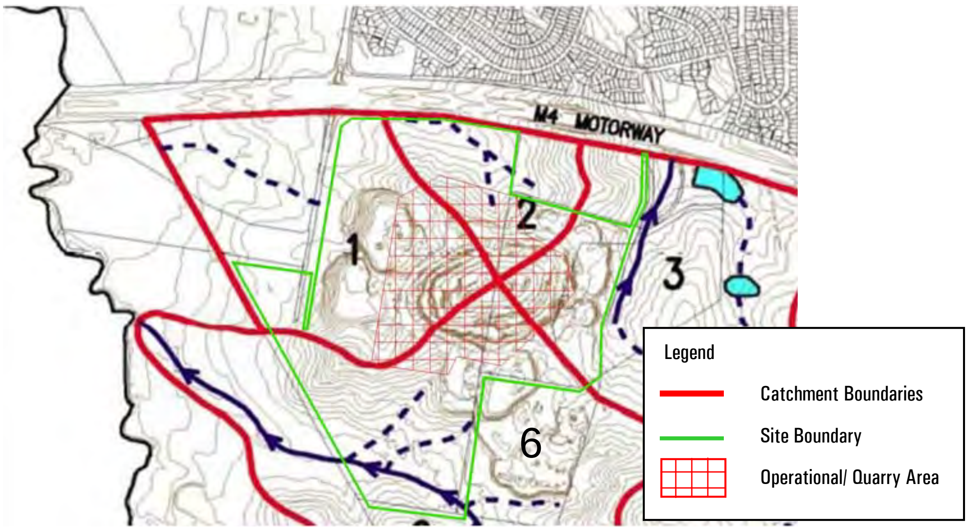


Figure 2-2: Catchment Areas (source: BCC, 2005)

Quarry Catchment (sub-catchment Ropes Creek)

The Quarry Catchment is located within the northwest corner of the Employment Lands Precinct, immediately south of the M4 Motorway. The total catchment area is approximately 72Ha, and drains to Ropes Creek via the existing contours on the site. A large portion of the current catchment runoff is reduced due to the presence of the quarry.

The site area that falls within the boundaries of the Quarry Catchment is approximately 41Ha (including part of the quarry).

Quarry North Catchment

The Quarry North Catchment is approximately 28Ha and is located immediately south of the M4 and east of the Quarry Catchment. The quarry void would intercept a large portion of the runoff from this catchment. Stormwater from this catchment drains through culverts located under the M4, to the area north of the M4.

The area of the Quarry North Catchment included within the site boundaries is approximately 19Ha.

Upper Angus Creek Catchment

The Upper Angus Creek Catchment is located in the northern section of the Precinct and has an area of 89Ha and drains northwards beneath the M4 Motorway.

The site area that lies in the Upper Angus Creek Catchment is 16Ha. It consists primarily of the quarry, whilst the remaining area is not subject to any development proposed under this DA.

Ropes Creek Tributary Catchment

Ropes Creek Tributary flows from east to west. There is a small farm dam located near the top of its catchment. Some signs of erosion are present near the dam.

The site area that falls within the Ropes Creek Tributary Catchment comprises quarry and undeveloped lands of approximately 44Ha.

Table 2-1: Site Catchment

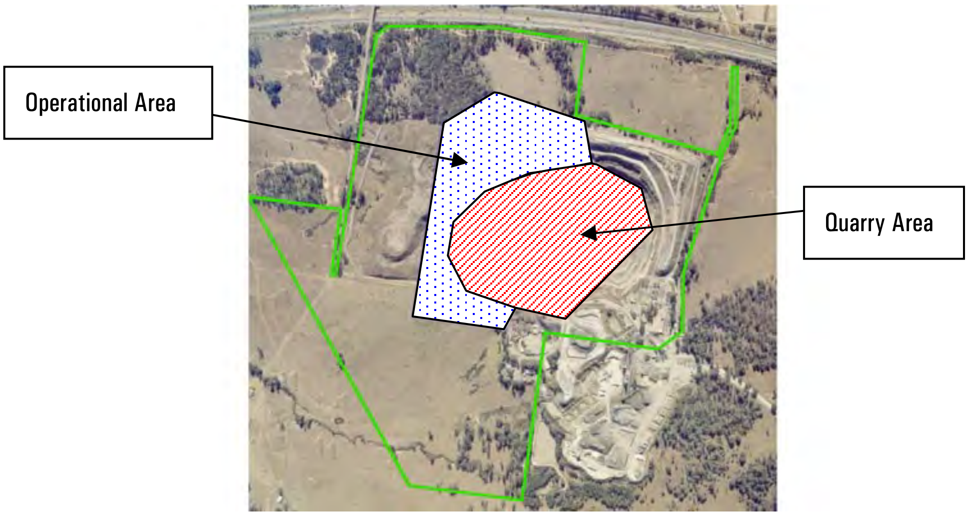
Catchment Number	Catchment Name	Site Area in Catchment (Ha)	Total Catchment Area (ha) [1]
1	Quarry Catchment	41	72
2	Quarry North Catchment	19	28
3	Upper Angus Creek Catchment	17.6	89
6	Ropes Creek Tributary Catchment	44	127
	TOTAL	121.6	316

[1] Source: Blacktown City Council Eastern Creek Precinct Plan 2005

For the purposes of modelling for this surface water assessment, the development area has been divided into two catchment areas. It has been assumed that the remaining site area will retain its current characteristics (pre-development state). The two developable areas comprise:

1) Operational area of the resource recovery facility (RRF), 14.8ha
2) Quarry area, approximately 26.5Ha.

Surface water flows from these developed areas will be managed to discharge towards the west to the Quarry catchment. This is discussed further in Section 3.5.



3.0 SURFACE WATER MANAGEMENT

3.1. Background

Part of the analysis required for successful development of the Resource Recovery Facility (RRF) and Landfill Facility includes planning of surface water management for the site. As water is both an input and output (waste product) of site activities, site planning needs to adopt an integrated approach to water management.

The key issues concerning site surface water management comprise:

- Segregation and management of 'clean' (water from operational areas) and 'dirty' runoff (i.e. leachate, or water that has come into contact with mixed wastes, green and timber wastes and uncovered landfilled wastes);
- Erosion and sediment control including protection of the drainage system from sediment influx;
- Quarry pit/haul road water management;
- Water quality control; and
- Provision of adequate on-site detention for the proposed operations.

Additionally, the *Precinct Plan* and *Engineering Guide to Development* require that pipe sizes be based on a 20 year ARI design flow and that the major drainage system be designed to safely convey the critical 100 year event under normal operating conditions.

Surface runoff generated on-site will fall into two categories

1) 'clean' (not leachate) – available for reuse (following roof water collection in rainwater tanks or runoff from clean operational areas which may require treatment for sediment only), and

2) 'dirty' (leachate) – generated from the base of the landfill, green waste areas and run off that has come into contact with mixed wastes, green and timber wastes and uncovered landfilled wastes.

Given the recent and impending changes to climate (including pronounced drought conditions), it is intended that the site remains as independent as possible of external water sources, and that the potential for off-site impacts to local receiving waters is minimised. The site layout for stormwater management is presented in Appendix A.

3.2. Soil and Water Management

Appendix A contains the Soil and Water Management Plan.

3.2.1. General

Site soil and water management will be required throughout the life of the project. The SWMP will adhere to the following principles:

- It is proposed to direct all operational area (hardstand clean) surface runoff (excluding water managed within the quarry pit) towards the Quarry catchment;
- Sediment-laden stormwater from the materials stockpile area will be directed through permanent sediment capture sumps or mini-basins along surface drainage to intercept sediment prior to reduce sediment 'slugs' reaching the GPT. Site grading is to be used to direct sediment-laden drainage away from hardstand areas;
- The MPC work floor and green waste area is to be diverted to sewer;

- Truck access to and from the unsealed areas are to be stable and designed to prevent influx of run-on and escape of untreated flows where possible;
- Runoff from site operational areas (as defined in Section 3.3) is to be directed through treatment devices (sediment traps and low-flow wetland treatment) and OSD for reuse prior to release to the site's drainage network. Overland flow paths for flows in excess of the design event are to follow natural drainage lines to the west of the site;
- Treatment devices around the site would provide sediment capture, gross pollutants where necessary, and must also be capable of capturing oil and fuel spills. Proprietary devices such as CDS, Humeceptor or similar can be selected and designed in consultation with the manufacturer to accommodate the required treatment;
- The treatment devices proposed for soil and water management are:
  - Small sediment sumps or mini-basins along swales to trap sediment 'slugs' if entrained in stormwater flow;
  - Sediment traps, e.g. proprietary gross pollutant trap (GPT) (CDS, Humeceptor or similar) or baffled settlement tank capable of retaining gross pollutants, sediment, oils and grease;
  - Within OSD basin: allowance for wet storage component, as a low-flow wetland for low-flow water quality treatment to remove fine suspended sediments as well as nutrients.
- Energy dissipation in the OSD basin settling basin for pre-treatment before entry to the OSD basin will provide further attenuation and capture of sediment that may reach the detention basin.

3.2.2. Stockpile and green waste areas

Sediment controls installed within the materials stockpile area will be maintained to prevent clogging and to prevent excessive sediment and nutrients entering the drainage system. These controls are to include:

- Small sediment sumps or mini-basins along swales to trap sediment 'slugs' if entrained in stormwater flow;
- Treatment through a GPT or baffled sediment settlement underground tank at the drainage outlet of these two areas,
- Protection of drains within these areas using:
  - vehicle exclusion,
  - stabilisation or lining of drains,
  - check-devices such as sediment sumps or mini-basins approximately every 50 metres to attenuate flows and encourage sediment dropout.
- Regular maintenance of drains and sediment traps to reduce loads within the system.

Runoff within the MPC work floor/ green waste collection area is to be managed as described in Section 3.2.

The green waste area, MPC floor, and materials stockpile area are graded away from the quarry to reduce the risk of overflows entering the quarry/landfill area.

3.3. Operational Areas

Surface runoff from the operational area will be managed separately from runoff generated in the quarry pit and haul road. Sources of stormwater runoff from the operational area include:

- Building roofs – workshop, MPC/ WTS, administration building and weighbridge shed – clean;
- Roads, car parks and other hardstand areas – clean, containing sediment;
- MPC work floor/ green waste stockpiles – dirty (to be directed to sewer);

- Materials stockpiles / drop off zones– clean, containing sediment.

Runoff collected from the clean or sediment-only areas will be reused on site, for uses including building internal uses (toilet flushing), wheel wash facility, dust suppression (*via* water carts) and irrigation/dust suppression from sprinkler systems around the site. (A water balance which utilises runoff generated on-site and estimated demands for the above uses have been developed and are discussed in Section 5.0.)

Drainage from the MPC work floor/green waste area is to remain in a 'closed loop' system with connections only to sewer. Drainage from this area is not considered in detail in this report.

Run off from other areas of the MPC/ WTS and stockpile/drop-off zones is considered to be "clean operational waters" but runoff from these areas will be subject to treatment (sediment removal) prior to reuse. Clean runoff from roofs will be collected in rainwater tanks for reuse on-site. Runoff from other parts of the operational area (e.g. roads, open areas away from stockpiles and buildings) will also be considered clean runoff and suitable for treatment and reuse on-site. This water may be directed to the OSD basin or storage tanks on-site (location to be determined), subject to satisfactory water quality.

Stormwater runoff will be conveyed by a combination of major and minor drainage systems, as shown in Appendix A, including:

- An underground piped system with provision for overland flow in swales and along roads;
- Stormwater detention and pollution control structures, and
- The natural drainage systems including creeks and overland flow.

BCC requirements are that piped networks are designed to convey 1 in 20 year flows without surcharge. Drainage overflows (greater than 1 in 100yr flows) from both these areas will be discharged away from the quarry pit via overland flow paths. Alternatively if required, overflows will be treated and sent to sewer (at an increased capacity if required).

Vehicle entry points for MPC work floor, green waste and materials stockpile / drop-off areas are to be located to minimise uncontrolled runoff and sediment release outside these areas.

Overland flow paths around the site are to remain stable in 100 year critical flows.

3.4. Quarry Pit / Landfill

The in pit haul road will be graded towards the quarry wall. The haul road will be graded towards the quarry wall and will follow a dish drain along the length of the road to a sediment basin proposed for the base of the quarry. Small check dams (e.g. sand bags or aggregate material approx 50mm diameter) located along the dish drain will assist in controlling flow velocities and erosion. At the base of the haul road, a temporary settlement sump is to be installed (e.g. concrete tank or temporary basin lined with geotextile and rock that can be moved as required) to slow down flows and to allow sediment to drop out prior to diversion to a clean area for pumping out (during initial 10m lift) or diversion to the in-pit basin (later stages of landfill management)

A storage basin will be required in the quarry pit to collect clean runoff from quarry walls, haul road and capped landfill areas. This basin is to be progressively relocated throughout landfilling, however no basin is proposed for the first 10m landfill lift.

Runoff collected from these areas will be suitable for reuse if it has not come into contact with waste, and it is expected that the water carts will be able to draw from the sediment basin and reuse this water for dust suppression on haul roads.

3.4.1. Quarry Pit Storage Basin

Managing Urban Stormwater: Soils and Construction, Vol 2B (Waste Landfills) (NSW DECC, Draft, 2007) acknowledges that stormwater from the areas of the landfill that have daily, intermediate or final cover applied can be directed to the sediment basin/s for treatment, rather than managing this water as leachate. Only stormwater which has come into contact with waste or other leachate needs to be managed as leachate, therefore water could be transported out of the pit basin for dust suppression, stockpile watering and similar on site activities within the site's operational area outside of the pit.

1) Initial Stage

The proposed stormwater basin in the quarry will not be placed at the first 10 metres of lift. During the initial stages where the first 10 metres of landfill lift is placed, stormwater influx to the landfilling areas is to be minimised using a sump and high rated pump to capture water from the sides of the quarry. All water falling on the landfill area itself is to be treated as leachate during the first 10m lift.

2) Later Stages

Due to the proposed landfill location being within the existing quarry pit, sediment control *per se* of the landfill area is not essential as the risk of environmental damage from sedimentation is low within the quarry pit itself. Rather, the primary aim of a collection basin within the quarry pit is to assist in controlling the volume of stormwater runoff that comes into contact with waste or the active landfill area (hence minimising leachate generation). Reuse of this water was also reviewed in a water balance model (Section 5.4.1) for its ability to meet demand for dust suppression, to maximise reuse potential.

Volume 2B of the 'Blue Book' for Waste Landfills (draft for consultation only) states that sediment basins and water storages should not be located on landfilled areas. However, the unavoidable constraint of being within the quarry pit, and the need to manage runoff effectively within the pit, necessitates the use of temporary stormwater controls and storage within the quarry pit.

The use of suitable grading and bunding and inclusion of a leachate trench to separate leachate from stormwater from capped areas within the landfill is also necessary to minimise surface water flows into active landfill areas. Erosion across capped areas and sediment influx into any temporary storage at capped areas must also be accommodated.

Forward planning for the location and size of the basin is important for effective runoff and sediment control. Its location should be determined at the development of each landfill lift, taking into account that a sealed basin area is necessary to prevent infiltration, and that it is not possible to excavate through capping and back into landfilled materials. Initial shaping or grading of capped/covered areas is necessary to allow for a suitable placement for the basin to create a catchment with a low point designed into the intermediate capped areas, to drain away from the active tip face / daily cover areas and allows placement of a liner for a basin without disturbing existing capped material.

3) Basin Sizing

Basin calculations were undertaken in accordance with the Blue Book for the quarry pit (26.5Ha).

The maximum total basin volume based on the total quarry pit footprint (including settling zone and sediment zone) that may be required is approximately 4,362.5m^3 which equates to 165m^3 per hectare of catchment area, which may include quarry walls that drain into the pit. Assumptions and spreadsheets used for sediment basin sizing including rainfall percentiles are presented in Appendix B and include the use of 5-day, 80th percentile rainfall and 2-month sediment accumulation.

Table 3-1 presents the basin data.

Table 3-1. Quarry pit basin information

Detail	Basin Option 1
Volume per hectare runoff capture (m^3/ha)	165
Size (m^3) – for 26.5ha area quarry footprint	4,362
Rainfall – overflows downstream (landfill protection)	5-day, 80th percentile (16.5mm)
Dust suppression uses - % demand met at full basin size	Refer Table 5-5

Sediment influx can be reduced by including a controlled, stabilised inlet to the basin and installing and maintaining effective erosion controls around the haul road outlet and around the boundary of the basin.

A series of basins may be installed to capture flows from sub-catchments of the quarry depending on available space within the quarry. The sub-basins will need to meet minimum storage requirements of 165m^3/Ha of catchment draining to each basin.

Based on the basin sizing assumptions used, drawdown of water within the basin would need to occur within 5 days of a storm event occurring, to follow the basin design requirements and also to minimise the time that water is stored at the landfill area.

Water collected in the basin should be used initially for in-pit dust control or other uses requiring water in the pit area. Basin(s) may be drawn down by the water carts for dust suppression purposes or used in dump truck on-board reservoirs.

3.5. Flooding

A review of SMEC's *Eastern Creek Precinct Plan Stormwater Management Strategy (2004) – Appendix B Hydraulic Analyses* was undertaken to inform the flooding assessment. Within the site boundaries, there is only one distinct overland flow path identified, which is in the Quarry North catchment. Flows in other catchment areas are not affected by the proposed activities.

Peak flows from the site into to the Quarry catchment drainage will be detained in OSD storage (refer to Section 4.0) to match pre-development flow levels. No drainage is proposed to be directed to the Quarry North catchment. No changes to the existing flooding regime are anticipated.

3.6. Proposed Changes to Catchment Drainage

The catchment boundaries in the Eastern Creek Precinct area as set out in BCC's Precinct Plan are based on old topographic boundaries which have been extensively modified since the construction of the quarry. Drainage in the remaining portion of the modified Quarry catchment (west) has been modified due to the presence of large overburden banks that act to redirect and prevent smaller flows from draining easily through the Quarry catchment area to Ropes Creek.

The change in catchment boundaries during site operations was assessed using site plans and proposed operational catchment areas. BCC has advised that all site flows may be directed west to the Quarry catchment. A RAFTS hydrological model was used to assess catchment flows as a result of the proposed drainage design at the site (see Chapter 4).

4.0 ON-SITE DETENTION (OSD)

4.1. Background

A series of regional detention basins are proposed in BCC's Eastern Creek *Precinct Plan*. One of these regional detention basins is proposed to be located at the site, within the Quarry North Catchment adjacent to the northern site boundary and the M4 Motorway. Another is proposed to the west of the site towards Ropes Creek.

Discussions with Council indicated that these regional basins were still subject to investigation therefore site basin(s) would be required for any proposed development in the interim.

This section presents the results of site specific OSD modelling.

4.2. Methodology

An XP-RAFTS computer model was generated to replicate pre- and post-development flows for the operational area which is subject to change in land-use following construction for the proposed operational area, to calculate OSD volume requirements. This was based on the assumption that the remaining site area will not change form or characteristics from the pre-development situation, and hence, any flows generated in these areas will remain the same as for the pre-development scenario.

Council guidelines require post-development peak flows to match pre-development peak flows up to the 100yr storm events. The model was run for the 2 year and 100 year ARI storm event to derive the required OSD volumes.

XP-RAFTS software allows the user to optimise OSD volume requirements with the use of a storage node receiving flows from the subject catchment. A two-stage discharge (2yr and 100yr) was modelled to check preliminary discharge calculations for peak flow hydrographs.

4.3. Assumptions

The operational area (including berms) was modelled in XP-RAFTS and incorporated an area of 14.8Ha. The operational area was divided into two separate catchments to reduce the total anticipated basin size. Basin 1 catchment is the northern section of the operational area with a modelled area of 10.03ha. Basin 2 catchment occupies the southern section of the operational area with a modelled area of 4.74ha.

The catchments were considered to be 100% pervious in the pre-development model and 100% impervious post-development. These assumptions would result in conservative estimates for flow and OSD storage requirements.

Other XP-RAFTS modelling assumptions are documented in Table 4-1.

Table 4-1: RAFTS modelling criteria for on-site detention determination

Parameter	Pre-development	Post-development
Initial Loss/Continuing Loss (assumes wet antecedent conditions and is a conservative approach)	15mm/3mm	5mm/1mm
Roughness value across site	0.04	0.02
Proportion impervious (%)	0	100

4.4. Results

Peak flows from the site operational areas were calculated using RAFTS for the predevelopment and post-development scenarios. This was used to calculate the required OSD storage volume to prevent downstream hydraulic impacts as a result of site development and allow matching of pre- and post-development flows off site. Table 4.4 shows the results of peak flow modelling.

Table 4-2: Results for OSD modelling

Catchment	ARI	Pre-development Peak Flows (m^3/s)	Post-development Peak Flows without detention (m^3/s)	Post-development Peak Flows with detention (m^3/s)	Required OSD Storage Volume (m^3)
Basin 1 Catchment	100 yr	1.156	5.277	1.110	3,900
	2yr	0.416	2.643	0.403	
Basin 2 Catchment	100yr	0.605	2.511	0.621	1,600
	2yr	0.215	1.249	0.206	

Staged discharge was initially determined using the orifice equation to estimate an orifice outlet diameter, then modelled in several iterations to ensure that pre-development and post-development flows and hydrographs were as close as possible for the 2 year and 100 year ARI. Charts showing pre- and post-development hydrographs and basin hydrographs are presented in Appendix C.

4.5. Discussion

Based on the OSD modelling results presented in Table 4-2, an OSD basin storage volume of 5,500m^3 is required for the proposed operational area. The quarry area itself will not require detention storage and following final completion and capping of the landfill, drainage from the area is to be diverted around detention storages. In the event that there is a change in impervious area, an OSD volume of 370m^3/Ha may be adopted based on the modelling in this report.

4.6. Dam Safety Committee requirements

The New South Wales Dam Safety Committee (DSC) *Risk Management Policy Framework for Dam Safety* (2006) was reviewed for requirements and criteria for risk assessment.

Among other goals, the DSC states that its mission is to develop and implement effective policies and procedures for regulation of dam safety. In general, dam safety is initially determined through a risk assessment that uses the probability of failure per dam in one year (with probabilities ranging from 10^{-7} to 10^{-3}) and the number of fatalities that would occur as a result of dam failure. An appropriate dam safety assessment would need to be undertaken at the relevant detailed design stage for the OSD basin.

For this site, the proposed OSD basin sizes are 3,900m^3 and 1,600m^3, smaller than several of the existing dams at the Eastern Creek Precinct. Generally basins will be constructed so that maximum water levels will be at most 1 metre above existing downstream ground levels, overland flow travels across rural land towards Ropes Creek.

Flows from either basin could be classed as “slow and shallow” in relation to overland flow paths, non-defined drainage lines allowing flow dispersion, and relatively long overland flow paths over un-occupied land to the nearest defined drainage line.

Moreover, STORM notes that the intended construction of a much larger regional detention basin in the vicinity of the proposed OSD basins. The larger OSD basin may present higher risks than that proposed for this site for the operations phase of the development, and will also require scrutiny particularly as the structure is intended to be in place through the long term.

In a Probable Maximum Flood the dam will have already overtopped from a smaller 1:100 event as part of its design. In a PMF event, the volume of catchment flows from further up the catchment beyond the site are likely to be having a greater impact at this point in the catchment, in which the contribution of any (unlikely) dam failure would be negligible.

As a result, these factors are likely to contribute to a negligible risk.

5.0 WATER CYCLE MANAGEMENT

5.1. Water Balance Methodology and Concept

A daily water balance analysis was used to determine the feasibility of the proposed rain and stormwater harvesting scheme and in particular the effects of various storage sizes for stormwater harvesting along with changes to demand. The water balance utilised flows generated using a simple runoff calculation using historical rainfall data, analysed for various rainfall patterns including dry, mean and wet rainfall years.

The purpose for modelling dry, mean and wet years was to assess the performance of various tank sizes given the changes to rainfall patterns. It is noted that with the potential effects of climate change and the current trend of dry rainfall patterns, the need to consider lower annual rainfalls for rain and stormwater harvesting reuse schemes is becoming more and more necessary. In addition, any excess stormwater produced (especially during wet season periods) need to be considered for the management of on-site surface waters.

A concept diagram for the proposed re-use scheme on site is shown in Figure 5-1 below.

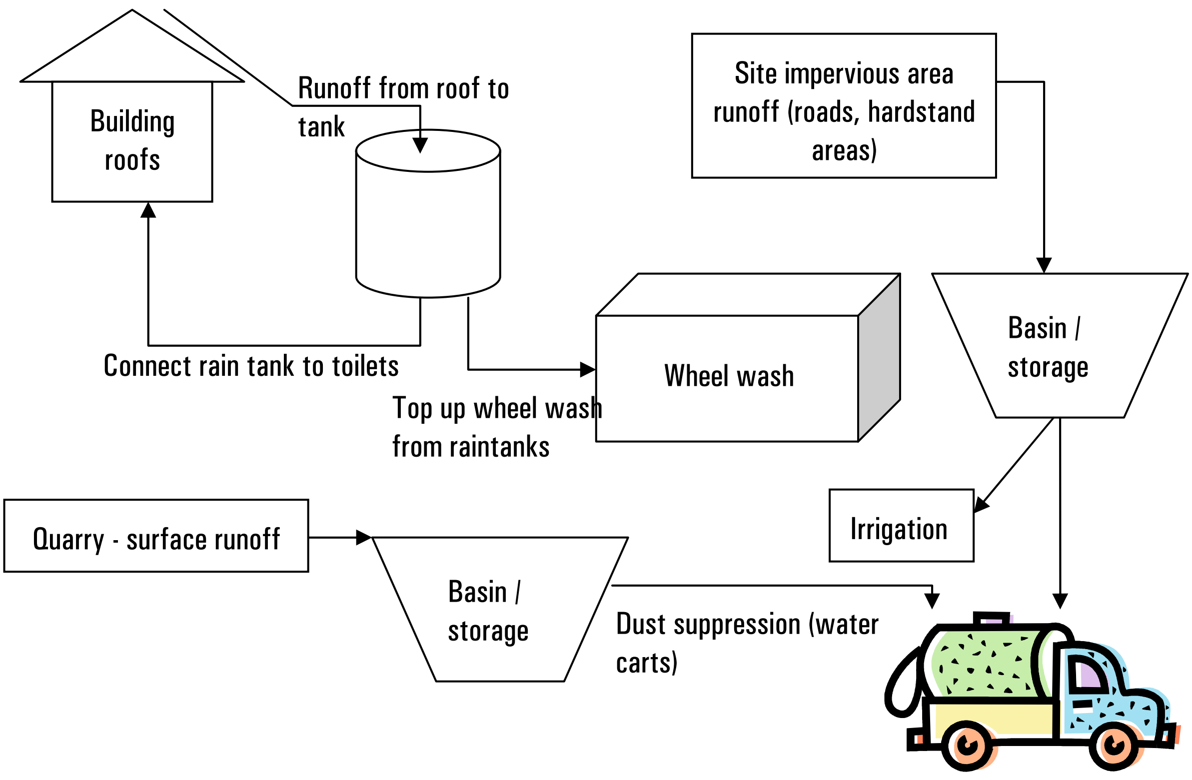


Figure 5-1: On-site water reuse concept

5.2. Modelling Inputs

5.2.1. Rainfall

Data from St Clair (BOM station #67102) was used in this analysis. Seventeen years of daily rainfall data (1985 – 2002) was assessed to determine a dry, median and wet rainfall sequence for use in the water balance model.

The following dry, median and wet year rainfalls were derived, and compared against long term averages for Prospect.

Table 5-1: Rainfall Records

	Annual Rainfall (mm)	
	Modelled average rainfall and years	Prospect (long term average)
Dry	553 (1994 / 1995 / 2001/ 2002)	562
Median	851 (1987 / 1989 / 1991)	831
Wet	1104 (1986 / 1987 / 1988 / 1989 / 1990)	1183

Note: median rainfall at St Clair is below Sydney Observatory's average of 1162mm/yr.

5.2.2. Harvestable areas

The proposed roof and stormwater reuse scheme can harvest runoff from the operational area catchment, comprising the areas identified in Table 5-2. This is conservative (under-estimates area available) and excludes the proposed green waste area.

Table 5-2: Harvestable Areas

Precinct	Area (Ha)	Initial loss (mm)
Building roofs	0.6	1
Remaining Site Operational Area	13.1	5
Quarry	26.5	10

5.2.3. Water demands

The demands for harvested water for reuse includes toilet flushing, dust suppression, sprinklers (irrigation) and the wheel wash. Estimated water demands used in the water balance model are presented in Table 5-3 below.

Table 5-3: Modelled Demands

	Annual Demand (ML/yr)			Modelling Assumptions
	Dry years	Mean years	Wet years	
Toilets	0.4	0.4	0.4	34 staff on-site x 6 flushes/day x 4.5L/flush
Dust suppression	25.8	24.1	24.0	Average application = 80kL/day (assumes no application if daily rainfall exceeds 2mm)
Sprinklers (irrigation)	9.7	9.1	9.0	Average application = 30kL/day (assumes no application if daily rainfall exceeds 2mm)
Wheel wash	0.3	0.3	0.3	Water use = 25kL/month
TOTAL	**36.2**	**33.9**	**33.7**	

5.3. Results - Catchment runoff

Based on harvesting stormwater from 13.1ha operational catchment area, calculations undertaken by STORM for a dry, median and wet year sequence provide the runoff volumes shown in Table 5-4. The actual runoff that can be harvested for reuse will not be the entire volume generated due to losses from the system from overflows, and is dependent on storage behaviour (i.e. if the storage volume reaches 100% capacity, overflows will occur rather than further collection). The performance of varying storage volumes is presented in Section 5.4.

Table 5-4: Potential Runoff Generation

Rainfall Scenario	Potential Runoff Generated (ML/yr)		
	Dry	Median	Wet
Building Roofs	3.0	4.7	6.2
Quarry	39.1	71.2	124.9
Remaining Site Operational Area	44.9	73.2	236.8
TOTAL	**87.0**	**149.1**	**367.9**

5.4. Storage sizing

The water balance model was set up to determine the amount of runoff generated from the catchment under the various rainfall scenarios, with the aim of assessing the performance of various storage sizes.

5.4.1. Raintanks and building roofs

Figure 5-2 demonstrates the results of capturing roof runoff from buildings and reusing it for internal uses (toilet flushing i.e. 0.9kL/day) and topping up of the wheel wash facility (1kL/day).

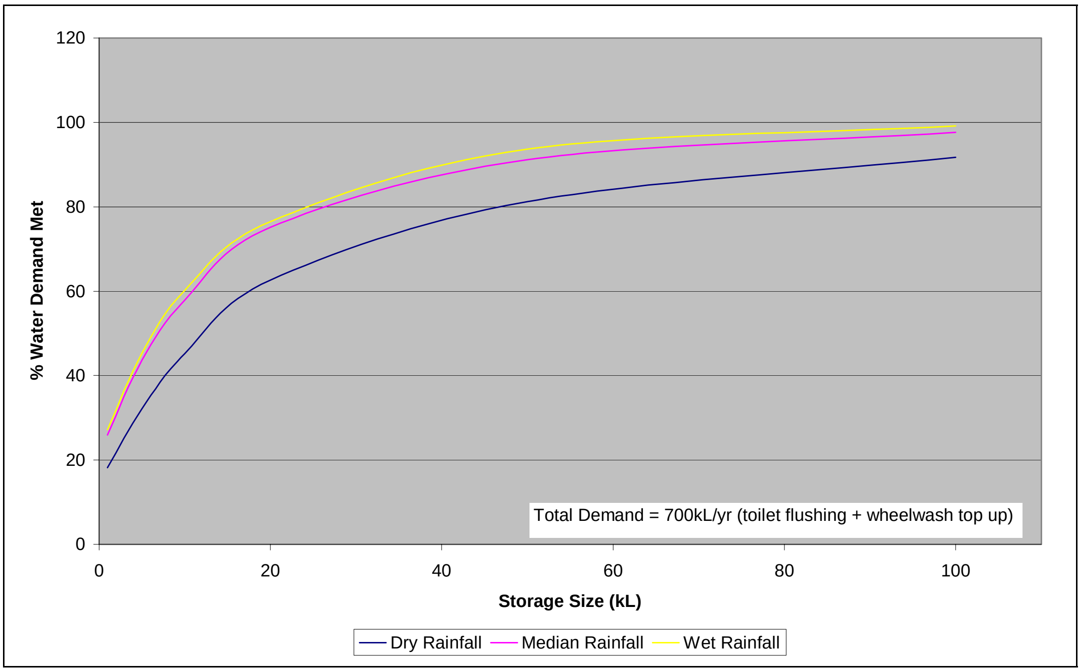


Figure 5-2: Roofwater reuse for toilet flushing + wheelwash

Based on Figure 5-2, overall tank storage volumes of up to 40kL would meet over 75% of the site's toilet flushing and wheel wash demands for the dry, median and wet rainfall scenarios e.g. a 40kL storage volume would meet 88% of the 700kL demand under median rainfall conditions. It is recommended that each of the four buildings on-site install a 10kL tank (minimum) to maximise potential roof runoff collection for reuse.

5.4.2. Surface runoff from operational area

There is opportunity to collect surface runoff from the internal roads/hardstand areas and remaining site operational area. Runoff from these areas may be directed towards the OSD basins which are proposed to include a storage component and be drawn down for reuse on site following storm events. A water balance was prepared for the water demand scenario of:

- Dust suppression for watering carts + truck on-board reservoirs (40kL/day) and spray mists / sprinkler system for irrigation or dust suppression (30kL/day).

Note: it is assumed that the water quality will be of adequate standard for reuse and will note pose a risk to human or environmental health.

It was assumed that on days where daily rainfall exceeds 2mm there is no demand for dust suppression.

A range of reuse storage volumes (within the OSD basin, as additional storage to OSD volume) under dry, median and wet rainfall scenarios were modelled.

Figure 5-3 shows the volume of rainwater supplied for a range of storage volumes under a dry, median and wet rainfall scenario. Figure 5-4 shows potential water supply and percent water demand met for dust suppression and sprinkler irrigation on site. As storage volume increases, the ability of the storage supply to meet demand will increase.

Current indicative basin size in the site drawings (Appendices A & B) allows for approximately 1000kL from Basins 1 and 2 combined, which would meet approximately 55% of the assumed water demand for dust suppression and irrigation combined. If required the storage volume could be increased at the detailed design stage.

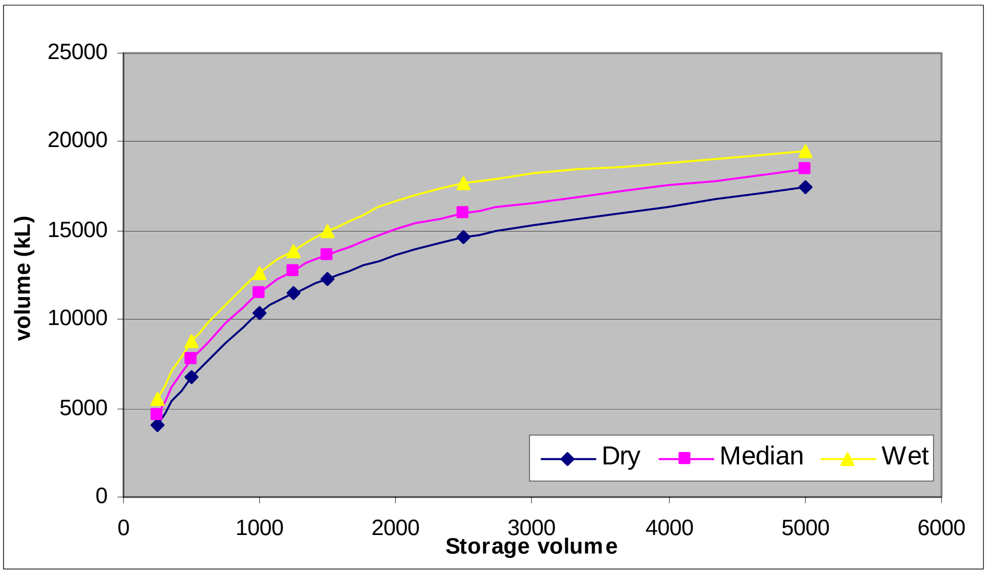


Figure 5-3. Rainwater volume supplied based on storage volume, kL/yr

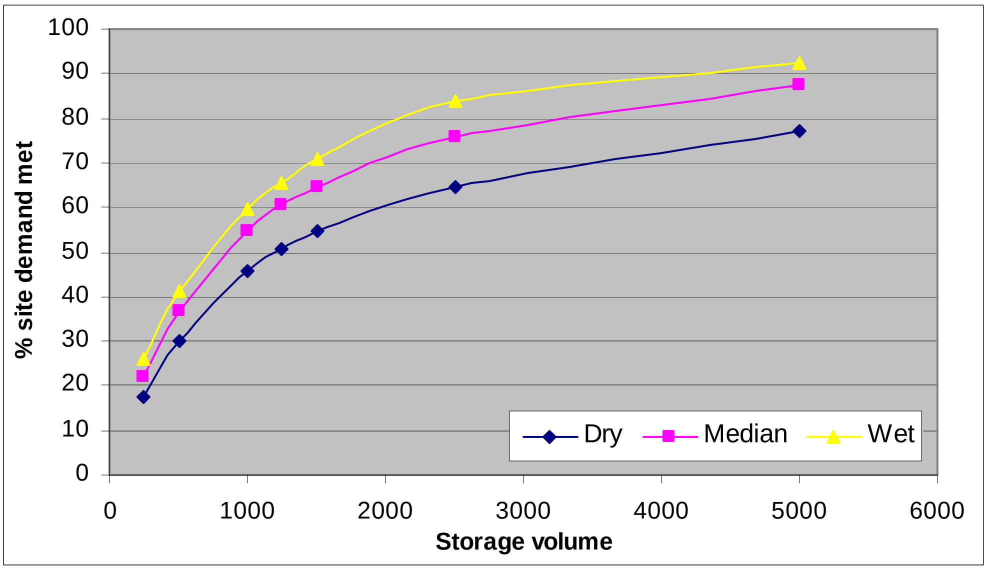


Figure 5-4. Percent water demand met annually, dust suppression and irrigation only

5.4.3. Surface runoff from quarry

Captured runoff in the quarry basin will be used for dust suppression via water carts. The available water volume for reuse from the basin will vary depending on rainfall and the stage of landfill operation, as the basin size is intended to increase in proportion to the capped landfill catchment area and runoff from quarry walls as required.

The modelled volume of reuse for dust suppression per day was 40kL/day.

Table 5-5 shows the per cent demand met from a basin sized to capture runoff from the 26.5ha quarry area. In practice the basin size may vary in relation to the area of capped landfill that is its catchment (at a rate of 165m^3/ha). For this reason it was modelled separately to the storage options within the OSD basin.

Runoff collected from these areas will be suitable for reuse such as dust suppression if it has not come into contact with waste.

Table 5-5. Quarry basin reuse - % demand met

Rainfall scenario	Basin (4,362m^3)	
	Total water demand ML/yr	% water demand met
Dry	12.97	72%
Median	12.07	87%
Wet	12.13	91%

5.5. Summary of Storage Volumes

The following recommended storage volumes are based on the analysis above:

- Each building should have its own rainwater tank (min. 10kL volume) to harvest roof water runoff for reuse including toilet flushing and wheel wash top up;
- The OSD storage proposed for the operational area is of sufficient volume (min. 370m^3/Ha) to contain the 1 in 2 year storm event 1 in 100yr storm event and by use of additional depth in the basin (nominal 0.5m in indicative basin sizes supplied) to act as storage for reuse on-site. It is anticipated that drawdown will occur regularly for dust suppression (water carts and sprinkler) and irrigation.
- The proposed sediment basin in the quarry has been sized using the Blue Book (approx. 165m^3/Ha) and can be drawn down following storm events for dust suppression (water carts).

6.0 WATER QUALITY

6.1. Water Quality Management

The stormwater management controls for the site including water quality management measures are presented in Appendix A.

6.1.1. Pollutant Treatment Priorities

Table 1 in BCC's Stormwater Quality Control Policy (2005) presents treatment priorities for a range of pollutants generated from various land uses. The proposed development is deemed industrial and as such the pollutant treatment priorities are identified in Table 6-1, based on Table 1 in the Policy. The Policy also states that for developments on sites greater than 5Ha, the pollution treatment methods selected must treat all pollutants cited with emphasis on the first three priority pollutants.

Table 6-1 Pollutant Treatment Priorities for Industrial Areas

Development Type	Litter (Gross Pollutants)	Coarse Sediment	Nutrients	Fine Sediment	Hydrocarbons, Motor Spirit, Oil & Grease
Industrial	3	4	5	1	2

Table 6-2 outlines the pollutant retention criteria for development sites, based on Table 2 in BCC's (2005) Stormwater Quality Control Policy. MUSIC modelling (refer to Section 6.2) was undertaken to assess the effectiveness of the proposed treatment system based on the information in this table.

Table 6-2 Pollutant Retention Criteria

Pollutant	Description	Retention Criteria for Development Sites
Fine Sediment	Contaminant particles 0.1mm diameter or less	50% of the total annual load
Hydrocarbons, Motor Spirit, Oil & Grease		Whichever is greater: 1. 90% of the total annual load; or 2. Total discharge from site of TPH [1] < 10mg/L at all times.
Litter (gross pollutants)	Trash litter and vegetation larger than 5mm	90% of the total annual load
Coarse sediment	Contaminant particles between 0.1mm and 5mm diameter	80% of the total annual load
Nutrients	Total phosphorus and total nitrogen	45% of the total annual load for each nutrient

Notes: 1. TPH – Total petroleum hydrocarbons

Table 6-3 is based on Table 3 in BCC's (2005) Stormwater Quality Control Policy and outlines the qualitative operational objectives for new developments, and how the proposed Stormwater Management Plan meets these objectives.

Table 6-3. Water Quality Management Objectives

Pollutant/Issue	Management Objectives	SMP addresses objectives
1. Runoff volumes and flow rates	Impervious areas are not to be directly connected to the stormwater drainage system unless uncontrolled property runoff needs to be constrained	OSD to be utilized to address runoff from site developed areas including road drainage and other paved areas.
2. Stormwater quality	Reuse of stormwater for non-potable uses maximised	Yes (addressed in Section 5.0)
	Vegetated flow paths or similar are to be used to connect impervious areas to the stormwater system	A vegetated wetland (end of line system) in each OSD basin will be used to treat stormwater runoff prior to discharge to the environment. Where feasible in detailed design, rock-lined or grass swales adjacent to berms will direct site operational area runoff to treatment/OSD basin. Sediment/Gross pollutant traps and low-flow treatment through wetland (as wet storage component of OSD basin) to be utilized for operational areas
	Use of stormwater infiltration 'at source' where soil types allow.	Infiltration will occur for smaller storm events ARIs. Soil types on site (heavy clays) inhibit use of infiltration for larger ARIs. Site use not conducive to stormwater infiltration as WQ control.
3. Riparian vegetation and aquatic habitat	Protect and maintain (i.e. no demonstrated adverse impact on) natural drainage features [1]. All natural (or modified) drainage channels within the site that possess either: • base flow • defined bed and/or banks • locally occurring native riparian vegetation are to be protected and maintained.	Drainage paths within site catchments are poorly defined with no base flow. Nil to very little native riparian vegetation is present at drainage paths within site. There are no modifications proposed for existing riparian vegetation and aquatic habitat at drainage paths in the Quarry North

	'Natural channel designs' should be adopted in lieu of floodways in areas in areas where there is no natural (or unmodified) channel.	and Quarry site catchments
4. Flow	Natural flow paths, discharge points and runoff volumes from the site should not be altered. Frequency of bank-full flows should not increase as a result of development. Generally, no increase in the 2-yr and 100-yr ARI peak flows.	Key discharge points for site are to be maintained or will remain unaffected by site development. OSD is proposed to maintain peak discharges at pre-development levels. Staged discharge modelling undertaken for 2yr &100 yr events
5. Amenity	Multiple uses of stormwater facilities to the degree compatible with other management objectives.	OSD aligns with requirements for onsite detention in Eastern Creek Precinct. Site OSD needs could be integrated with regional detention basin in future following assessment. No clashes with other management objectives Stormwater harvesting and reuse to reduce potable water demands and peak runoff volumes.
6. Natural bushland	No demonstrated adverse impact from stormwater into urban bushland area.	No stormwater to be directed to new discharge points including bushland areas. Stormwater from site operational areas is directed through sediment trap, wetland and OSD prior to release at existing discharge points.
Notes: 1. Wetlands, watercourses and riparian corridors.		

6.1.2. Maintenance and Monitoring

The developer will be responsible for the maintenance of the proposed stormwater controls. A maintenance plan will be developed during the detailed design phase. In general, the maintenance plan should allow for:

- Regular visual inspection of the stormwater treatment measures, for example on a monthly basis and after rain events;
- OSD Basin and GPT cleaning program – more frequently as site settles from development and then based on results of regular visual inspections. Cleaning generally to consist of:
  - Sediment and weed removal from the OSD basin and its associated sediment control/stilling basin, and
  - checking integrity of in-pit stormwater basin, plus sediment removal

- (Optional) water sampling of OSD basin and in-pit stormwater collection basin, e.g. on a quarterly basis for the first year of the basin's operation as each basin is developed, then 6-monthly in following years, to ensure reused/released water is of suitable quality for end-use, e.g. in irrigation equipment (if necessary can refer to ANZECC guidelines and relevant NSW guidance;

A maintenance and monitoring check-sheet should be developed that allows for the data entry, location of stormwater management device on site (e.g. based on a map with numbered locations), type of inspection (visual, water sampling, etc), outcome (e.g. all clear, device needs cleaning), actions taken, and any follow up required.

Site salinity management with reference to water collected within the quarry pit is addressed in Section 6.3.

The quality of the water released (if any) should be in accordance with the site's Environment Protection Licence. Typically the licence will only permit discharge once the water in storage has been tested to ensure it complies with specified water quality standards for discharge. Sampling requirements may include total suspended solids (TSS), Electrical conductivity, Turbidity, Ammonia, Biochemical Oxygen Demand (BOD), Total Nitrogen and Total Phosphorus.

6.2. MUSIC modelling

6.2.1. Methodology

The MUSIC model was chosen to model water quality. This model has been released by the Cooperative Research Centre for Catchment Hydrology (CRCCH) and is a standard industry model for this purpose. MUSIC (the Model for Urban Stormwater Improvement Conceptualisation) is suitable for simulating catchment areas of up to 100 km^2 and utilises a continuous simulation approach to model water quality.

By simulating the performance of stormwater management systems, MUSIC can be used to determine if these proposed systems and changes to land use are appropriate for their catchments and are capable of meeting specified water quality objectives (CRC 2002). The water quality constituents modelled in MUSIC and of relevance to this report include Total Suspended Solids (TSS), Total Phosphorus (TP) and Total Nitrogen (TN).

Only the site operational area and quarry area were modelled as these undergo the greatest change in land use. The post-development model was used to compare the pollutant loads generated from the proposed development with, and without treatment controls.

The pollutant retention criteria set out in BCC's Stormwater Policy were used as a basis for assessing the effectiveness of the selected treatment trains.

The layout of the MUSIC model is presented in Appendix D.

6.2.2. Assumptions

Rainfall

Rainfall data was obtained from the Bureau of Meteorology (BOM) for Prospect Dam (BOM station #67109) the closest station to the site with continuous rainfall data. Meteorological data from 1984 - 2004 (slightly above average rainfall conditions) was used in the model in an attempt to replicate climatic conditions typical of the site.

The MUSIC User Manual (CRCCH, 2004) suggests that the time-step should not be greater than the time of concentration of the smallest sub catchment, but consideration should also be given to the smallest detention time of treatment nodes in the system. To accurately model the performance of the treatment nodes, a daily time step was chosen.

Soil Properties

Various parameters are required to be entered in MUSIC regarding soil properties. The soil profile of the existing site is fairly uniform and soil parameters were set to default Sydney values throughout the modelling.

Event Mean Concentrations

The default MUSIC Event Mean Concentration (EMC) values have been adjusted to reflect more recent data available by Duncan (2004) for specific land uses such as roads, roofs and urban areas. The parameter concentrations adopted are shown in Table 6-4.

Table 6-4: MUSIC Modelling EMCs

Land Use1	EMC (mg/L)		
	TSS	TP	TN
Roofs	20	0.13	2
Other site areas	270	0.5	2.2

[1] Fletcher, T., Duncan, H., Poelsma, P. & Lloyd, S. (CRC, 2004)

Proposed Treatment Method

Treatment measures modelled include gross pollutant traps, treatment through a wetland system and inclusion of stormwater runoff reuse.

6.2.3. Results

The results of the post-development model are shown in Table 6-5. The reduction rate is expressed as a percentage and compares the post-development pollutant loads **without** treatment versus post-development loads **with** treatment. When a positive reduction percentage is achieved there is a net decrease in pollutant loads as a result of development. The development can then be considered to have a beneficial effect. However, if a negative reduction percentage occurs then there is an increase in pollutant loads in that particular post-development scenario.

Table 6-5: Flow and Pollutant Load Reductions

Parameter	A Post-Development Results (without treatment controls)	B Post-Development Results (with treatment controls)	Reduction % (A-B)/A
Flow (ML/yr)	64.80	42.7	34.1
Total Suspended Solids (kg/yr)	15600.00	770	95.1
Total Phosphorus (kg/yr)	29.70	6.33	78.7

Total Nitrogen (kg/yr)	141.00	69.4	50.8
Gross Pollutants (kg/yr)	2100.00	0	100

The model results (Table 6-5) indicate that pollutant load reductions for Total Suspended Solids, Total Phosphorus and Total Nitrogen will meet BCC's Stormwater Quality Control Policy (2005).

The detention storage for the proposed wetland can be wholly contained within the basin in addition to OSD. Site drawings in Appendix A show the indicative cross section for OSD-wetland configuration with provision for storage and reuse.

6.2.4. Discussion

Based on the water quality modelling undertaken, measures including gross pollutant traps, treatment through a wetland system and stormwater runoff reuse will enable stormwater discharged from the site to be treated to a standard that meets water quality objectives as set out by Council.

It is considered that other pollutants such as hydrocarbons are not expected to cause any significant impacts on site under every day operations. Under extreme circumstances (e.g. a petrochemical spill during refuelling), operational management plans will be in place which identify strategies for remediation. Selection of a suitable GPT will allow some oils and grease to be retained.

Other measures (under the Environmental Management Plan) would include a covered and bunded area being provided for any refuelling (and materials storage) facilities on the site. Bunds should be capable of containing the full storage volume of the container plus an additional 10%.

6.3. Salinity

There is presently no visible indication of salinity at the ground surface around the site. The *Precinct Plan* suggests that adverse impacts on salinity would be expected if the groundwater level were to be raised significantly over a period of time. In this way, contributing factors may include prolonged flooding, removal of deep-rooted vegetation, over-irrigation, disruption of natural drainage lines, stormwater infiltration and leaky pipes. Some areas of the site may be more susceptible to developing soil salinity problems due to their geology.

The pit is likely to have contributed to some extent to lowering the groundwater table in the vicinity of the site by creating a groundwater 'sink' (IGGC) and this may result in the possibility that saline drainage from sub-soils and bedrock will reach the quarry pit and walls and contribute to saline runoff collected in the pit.

Please refer to the IGGC report for more information about salinity in groundwater and saline groundwater impacts.

6.3.1. Site Water Management for Salinity

Water quality in the proposed temporary sedimentation basin located on progressively capped areas of the landfill within the pit is to be assessed as per monitoring requirements. If salinity or TDS results for water quality in the basin proves too saline for site irrigation or related surface uses, its use is to be restricted to suitable areas of the site, e.g. dust suppression within and around the quarry pit.

In general, the Department of Primary Industries (Agriculture) recommend that acceptable salinity levels for pasture (assuming that any irrigated areas at the site will primarily be turf) are in the order of 2200μS/cm before growth begins to be affected.

Potential impacts on salinity will be managed in the following ways:

- Modifications to existing site vegetation are kept to a minimum. Most site development is to occur on existing disturbed or cleared areas adjacent to the quarry pit;
- Minimise additions to groundwater table by avoiding waterlogged areas and over-irrigation; and
- The proposed OSD basin serving the proposed development, at a depth of approximately 3m below existing ground surface, is not likely to intercept potentially saline groundwater.

7.0 SUMMARY

7.1. Summary of Stormwater Management Measures

Water on site is to be managed according to the goals and methods outlined in Table 7.1 and takes into account site needs and BCC requirements. The general layout for site surface water management is presented in Appendix A.

Table 7-1: Stormwater Management

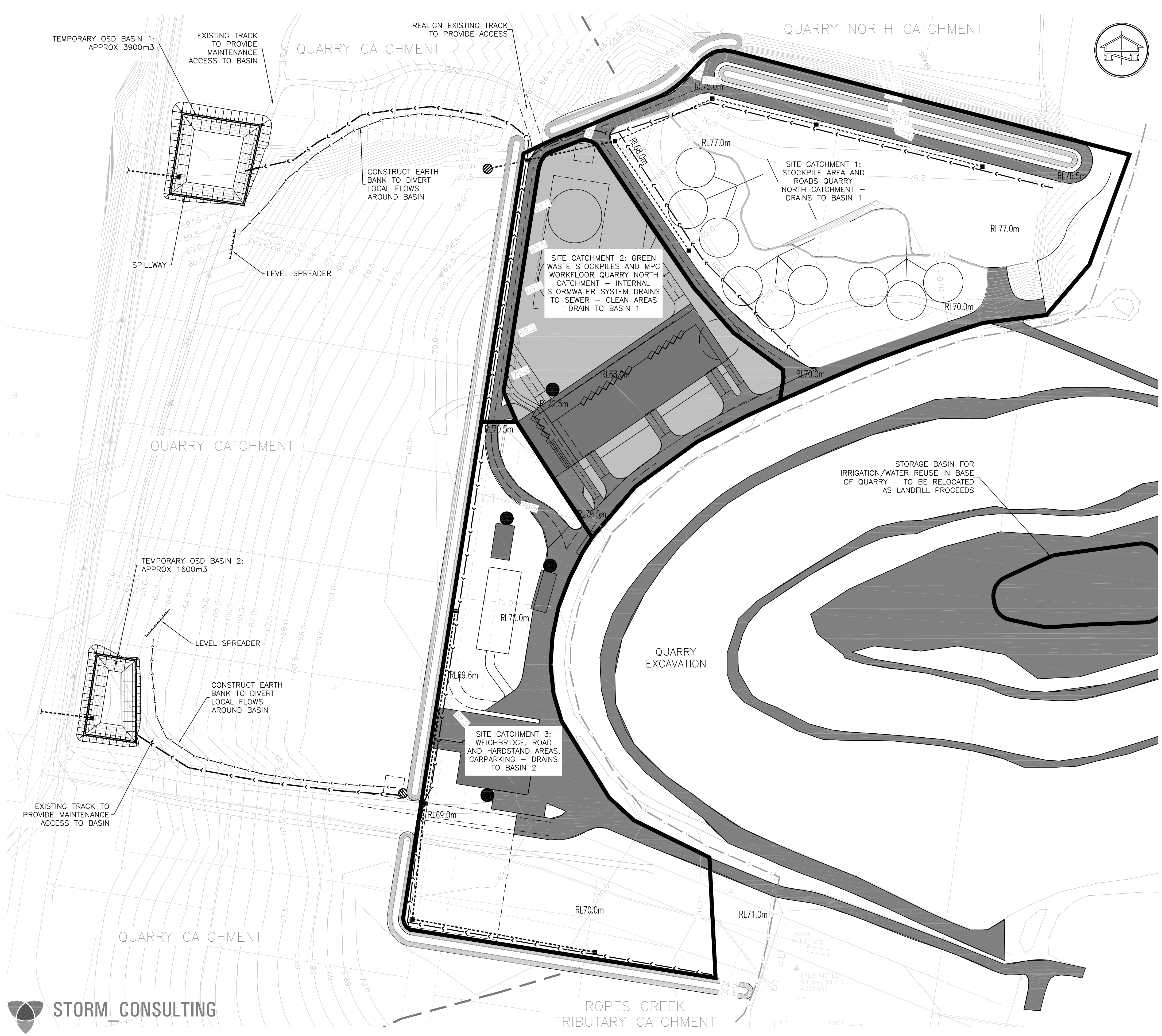
Stormwater Management Measure	Management goal	Methods
Site Stormwater Drainage System	Piped and open drainage structures to convey the major and minor storm events (1 in 100 year and 1 in 20 year respectively) to storage and reuse facilities or off-site as required, via on-site treatment and/or detention facilities where necessary.	All clean or sediment-only surface runoff to be directed to two detention basins in the Quarry catchment, *via* constructed drainage. See Section 3.1.1 for definitions of clean and dirty runoff. Piped networks will be designed to convey 1 in 20 year flows without surcharge. The MPC work floor/green waste stockpile area is to be directed to sewer. Drainage overflows (greater than 1 in 100yr flows) from both these areas is directed away from the quarry pit via overland flow paths. Vehicle entry points for MPC work floor / green waste and materials stockpile & drop-off areas to be located to minimise uncontrolled runoff and sediment release outside these areas. Overland flow paths around the site are to remain stable in 100 year critical flows.
On-site Detention	On-site detention is required to match post-development flows with pre-development flows.	OSD is required to match post-development flows with pre-development flows from the developed operational area (14.8Ha). The remaining site area flows will not be detained as there will be no change in land use in these areas. The required OSD volume to contain 1 in 100 year flows from the 14.8Ha surface operational area is 5500m^3.
Stormwater Management	Minimise generation of leachate and contaminated runoff	All clean surface runoff to be diverted to two OSD basins (operational area flows) or an in-pit basin (quarry area flows). Surface run-on to and from or sediment-generating operational areas at the surface, and to the quarry pit, is to be minimised through the use of diversion bunds and site grading. This will include grading the site such that all surface runoff up to the 100 year (or design) event is directed away from quarry pit. Drainage from the MPC work floor/green waste area is to be connected to sewer. Runoff from the stockpile/drop-off area is to be managed as clean surface stormwater, with additional sediment control. The MPC work floor/green waste area will be bunded to

		prevent stormwater entering the area.
Water Quality and Reuse	Treat, store and reuse runoff on site where possible	Any runoff water coming into contact with waste as defined in Section 3.1 is to be treated as leachate. Reuse purposes include: 1. toilets and other building internal uses 2. outdoor uses including dust suppression, stockpile management, irrigation. See Chapter 5, Water Cycle Management for further discussion. Water management practices for incidental waste storage (e.g. office waste storage areas, 240L 'Otto' bins), vehicle wash down areas and materials storage areas to follow Appendix D in BCC *Stormwater Quality Control Policy*.
Reduce mains water demand.	Roof water is to be captured in rainwater tanks for reuse on site. A network of water storages will be located on site to provide water supply to the facility as determined by the water balance model and site demands.	Harvested roof water to be used in buildings for appropriate end-uses (e.g. toilet flushing and localised irrigation). Roof water will also be used to top up the wheel wash. Stormwater harvested from the OSD basin would be used for dust suppression, irrigation around the berms, stockpile management and other non-potable water uses. Water from proposed quarry pit detention basin installed after the initial 10m lift to be reused for dust suppression, in water carts on haul roads or in dump truck on-board reservoirs. See Chapter 5, Water Cycle Management
Site Monitoring	Monitor water quality and drainage systems	Periodic checking and maintenance of site drainage and water quality controls to be undertaken to reduce likelihood of drain blockage and overflows. To ensure water quality is suitable for equipment used in irrigation or stockpile spraying/management, monitoring of water quality may take place by sampling from the site OSD basin, and if necessary from the proposed in-pit basin.

8.0 REFERENCES

- Blacktown City Council (2005) Eastern Creek Precinct Plan – Stage 3
- Blacktown City Council (2005) Engineering Guide for Development
- Blacktown City Council (2005) Stormwater Quality Control Policy P01100
- Institution of Engineers (2000) Australian Rainfall and Runoff
- Landcom (2004) Managing Urban Stormwater: Soils and Construction Volume 1, 4th Edition
- NSW DECC (2007) Managing Urban Stormwater: Soils and Construction Volume 2B – Waste Landfills (currently available as a draft for consultation only)
- SMEC (2004) SEPP59 Landholder Group Eastern Creek Precinct Plan Stormwater Management Strategy

APPENDIX A
Surface Water Management Plan



LEGEND

- INDICATIVE STORMWATER PIT LOCATION
- INDICATIVE STORMWATER PIPE LOCATION
- OVERLAND FLOW/ SWALE DIRECTION
- OVERLAND FLOW SWALE TO DETENTION BASIN
- GROSS POLLUTANT TRAP
- DESIGN CONTOURS (0.5m INTERVAL)
- MIN 10kL RAINTANKS FOR BUILDINGS (INTERNAL REUSE – TOILET FLUSHING, LOCAL IRRIGATION, TOP UP OF WHEEL WASH)
- INDICATIVE FUTURE OVERLAND FLOW SWALE FROM FINAL LANDFILL FORMATION

NOTES

1. ALL ROADS AND CONTSTRUCTED PLATFORMS MINIMUM GRADE 0.5%.
2. ALL CONSTRUCTED PLATFORMS AND ROADS TO BE DRAINED BY PIT/PIPE DESIGNED FOR 20 YEAR ARI CAPACITY, AND OPEN SWALE DESIGNED FOR 100 YEAR ARI CAPACITY.
3. ALL SWALES AND CULVERT OUTLETS TO BE ARMOURED/VEGETATED AS PER DETAILED DESIGN.
4. ALL AREAS TO DRAIN TO DETENTION BASINS TO THE WEST OF THE SITE AS PER COUNCIL INSTRUCTIONS.
5. IRRIGATION/ DUST SUPPRESSION LINE TO BE INSTALLED ALONG BERMS AND AS DETAILED BY HYDRAULIC ENGINEER.
6. TYPICAL SECTION OF DETENTION BASIN SHOWN ON DRAWING D02.
7. BASIN VOLUME GIVEN IS IN ADDITION TO WETLAND PERMANENT STORAGE AND REUSE STORAGE.
8. REFER TO DRAWING D02 FOR EARTH BANK DETAIL AND LEVEL SPREADER DETAIL

SITE CATCHMENTS

	CATCHMENT 1 [INCLUDES CRUSHED MATERIAL STOCKPILE AREA]	CATCHMENT 2 [GREEN WASTE STOCKPILE AREA AND MPC WORKSHOP]	CATCHMENT 3 [WEIGHBRIDGE, HARDSTAND AREAS, OTHER BUILDINGS]
TOTAL AREA	6.5HA	3.3 HA	4.8 HA
TEMPORARY OSD BASIN VOL (BASED ON 400m3/Ha WITH STAGED DISCHARGE OUTLETS AT BASIN)	3,900m3 FOR TOTAL AREA OF CATCHMENTS 1 AND 2. VERY CONSERVATIVE OSD VOLUME AS MAJORITY OF RUNOFF FROM CATCHMENT 2 UP TO 100 YEAR EVENT WILL BE COLLECTED AND PUMPED TO LEACHATE TREATMENT SYSTEM		1,600m3

ISSUE	REVISION	BY	CKD	DATE	ISSUE	REVISION	BY	CKD	DATE
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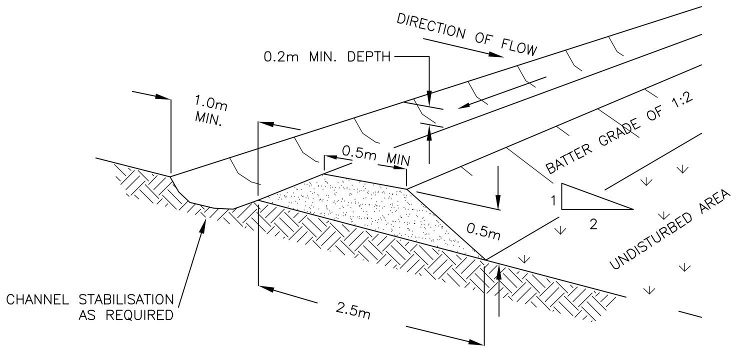
DOCUMENT PURPOSE: ADVANCED COPY ☐ FOR CLIENT APPROVAL ☐ FOR DA ☒ FOR CC ☐ FOR TENDER ☐ FOR CONSTRUCTION ☐

COORD. ORIGIN: | DATUM: | DRAWN: BF | DESIGNED: BF | SCALE: 1:1500 | DATE: NOV 2008 | CHECKED: NL

CLIENT: Alexandria Landfill Pty Ltd, 32 Burrows Road, Alexandria NSW 2015, (02) 9519 9999

LIGHT HORSE BUSINESS CENTRE
STORMWATER MANAGEMENT PLAN
GERERAL ARRAGEMENT AND LAYOUT

DRAWING: P01
SHEET: 1 OF 3
A1



EARTH BANK DETAIL

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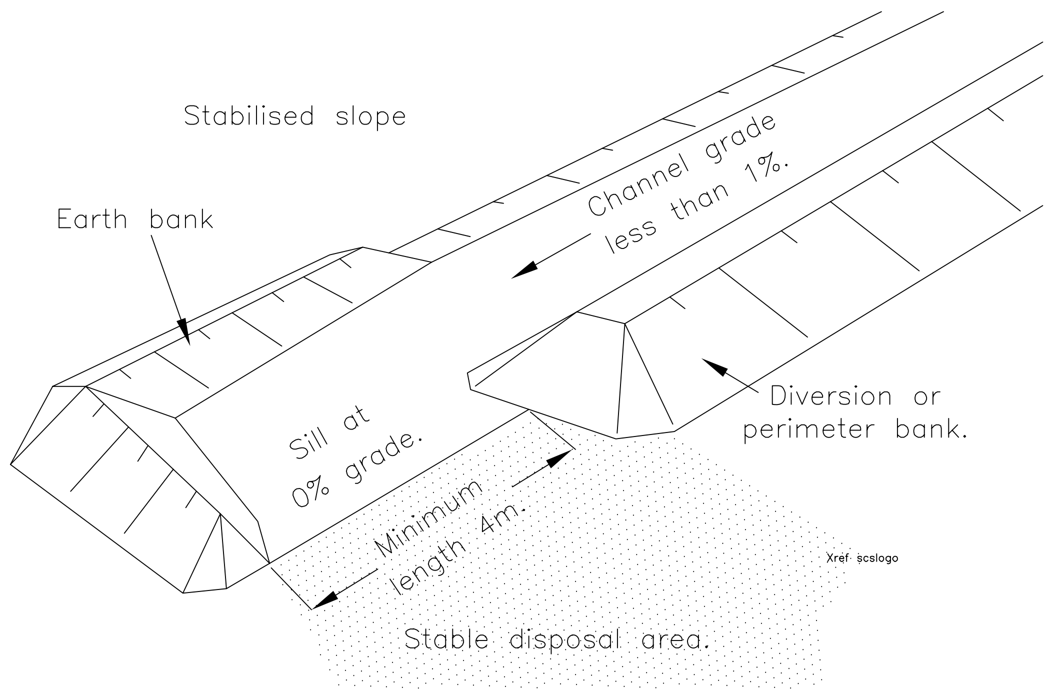
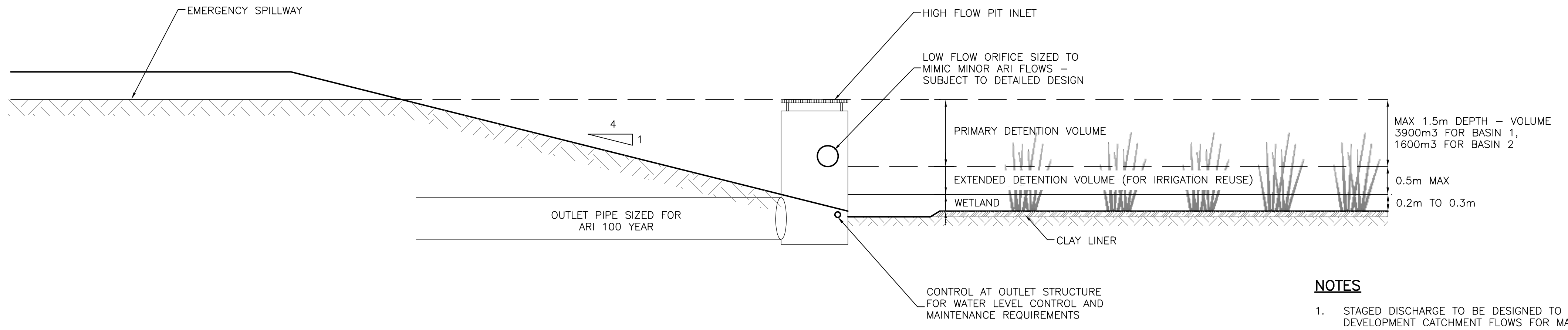


LEVEL SPREADER

NOT TO SCALE

EARTH BANK CONSTRUCTION NOTES

1. CONSTRUCT AT A GRADIENT BETWEEN 1 AND 5%
2. AVOID REMOVING TREES AND SHRUBS IF POSSIBLE – WORK AROUND THEM
3. ENSURE THE STRUCTURES ARE FREE OF PROJECTIONS OR OTHER IRREGULARITIES THAT COULD IMPEDE WATER FLOW.
4. BUILD THE DRAINS WITH CIRCULAR, PARABOLIC OR TRAPEZOIDAL CROSS SECTIONS, NOT V-SHAPED.
5. ENSURE THE BANKS ARE PROPERLY COMPACTED TO PREVENT FAILURE.
6. COMPLETE PERMANENT OR TEMPORARY STABILISATION WITH 10 DAYS OF CONSTRUCTION.
7. WHERE DISCHARGING TO ERODIBLE LANDS, ENSURE THEY OUTLET THROUGH A PROPERLY CONSTRUCTED LEVEL SPREADER.
8. CONSTRUCT LEVEL SPREADER AT A GRADIENT OF LESS THAN 1%
9. WHERE POSSIBLE, ENSURE THEY DISCHARGE WATERS ONTO EITHER STABILISED OR UNDISTURBED DISPOSAL SITES WITH THE SAME SUBCATCHMENT AREA FROM WHICH THE WATER ORIGINATED. APPROVAL MIGHT BE REQUIRED TO DISCHARGE INTO OTHER SUBCATCHMENTS.



SCHEMATIC OF DETENTION BASIN AND STAGED DISCHARGE OUTLET

SCALE 1:50

NOTES

1. STAGED DISCHARGE TO BE DESIGNED TO MATCH PRE AND POST DEVELOPMENT CATCHMENT FLOWS FOR MAJOR AND MINOR ARI RAINFALL EVENTS UP TO 100 YEAR.
2. PRIMARY DETENTION VOLUME TO CATER FOR UP TO 100 YEAR ARI.
3. EXTENDED DETENTION (REUSE) VOLUME TO BE INCLUDED DEPENDING ON FINAL WATER BALANCE AND REUSE REQUIREMENTS – SUBJECT TO DETAILED DESIGN.
4. WETLAND DESIGN AND PLANTING SUBJECT TO DETAILED DESIGN.

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CLIENT: Alexandria Landfill Pty Ltd, 32 Burrows Road, Alexandria NSW 2015, (02) 9519 9999

LIGHT HORSE BUSINESS CENTRE
STORMWATER MANAGEMENT PLAN
OSD TYPICAL SECTION

DRAWING: D02
SHEET: 2 OF 3
A1

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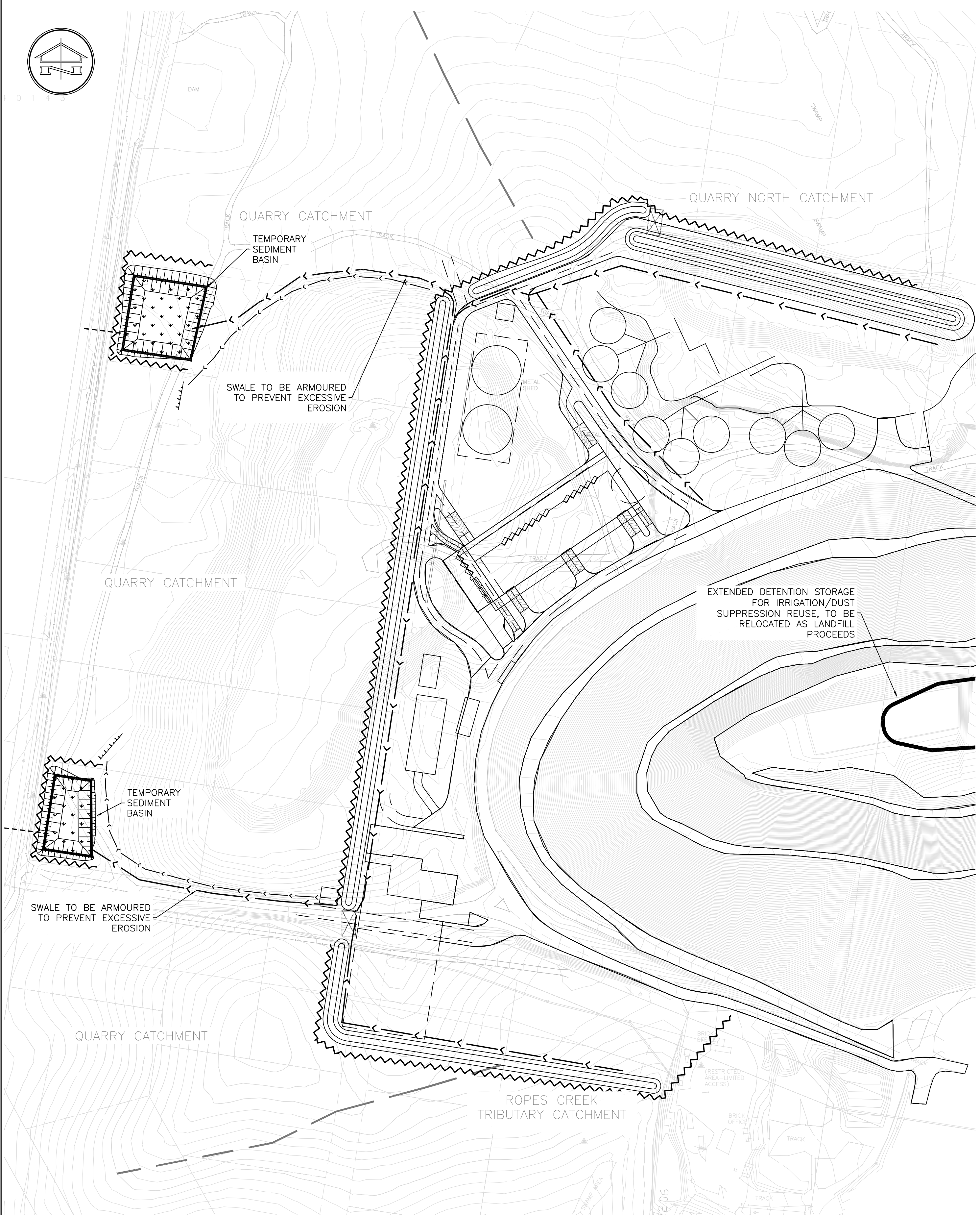


GENERAL REQUIREMENTS

THE FOLLOWING SOIL AND WATER MANAGEMENT PLAN (SWMP) HAS BEEN DEVELOPED IN GENERAL ACCORDANCE WITH LANDCOM (2004) – MANAGING URBAN STORMWATER: SOILS AND CONSTRUCTION, OTHERWISE KNOWN AS "THE BLUE BOOK". THE CONTRACTOR SHALL BE AT ALL TIMES RESPONSIBLE FOR TAILORING THE SOIL AND WATER MANAGEMENT PLAN TO SUIT SITE CONDITIONS. AS CONSTRUCTION PROGRESSES THE CONTRACTOR SHALL AMEND THE SOIL AND WATER MANAGEMENT PLAN ACCORDINGLY. IT IS THE CONTRACTOR'S RESPONSIBILITY ALL TIMES TO ENSURE THAT THE SOIL AND WATER MANAGEMENT MEASURES COMPLY WITH THE REQUIREMENTS OF THE BLUE BOOK.

SITE ESTABLISHMENT

PRIOR TO THE COMMENCEMENT OF EARTHWORKS ON THE SITE THE FOLLOWING SHALL BE UNDERTAKEN AS A MINIMUM:

1. ERECT SAFETY FENCING WITH SIGNAGE CLEARLY INDICATING THAT THE SITE IS A CONSTRUCTION ZONE AND ACCESS IS RESTRICTED AS DEEMED NECESSARY.
2. ERECT CLEARLY VISIBLE BARRIER FENCING AT LOCATIONS SHOWN OR IF NOT SHOWN AT THE DISCRETION OF THE SITE SUPERINTENDENT TO ENSURE TRAFFIC IS CONTROLLED AND TO PROHIBIT UNNECESSARY SITE DISTURBANCE.
3. INSTALL STABILISED SITE ACCESS IN ACCORDANCE WITH DRAWING SD6-14 AT EACH SITE ACCESS POINT TO PREVENT CONSTRUCTION EQUIPMENT FROM CARRYING SEDIMENT OFF THE SITE ONTO SURROUNDING ROADS.
4. INSTALL SEDIMENT AND EROSION CONTROL DEVICES IN ACCORDANCE WITH THE CONSTRUCTION DETAILS SPECIFIED IN THIS DRAWING SET AND/OR THE REQUIREMENTS OF THE 'BLUE BOOK'.

CONSTRUCTION

5. TOPSOIL, FROM ALL AREAS TO BE DISTURBED, SHALL BE STRIPPED PRIOR TO CONSTRUCTION OF ANY WORKS AND STOCKPILED AND LATER RESPREAD TO AID REVEGETATION IN LANDSCAPED AREAS. TOPSOIL SHALL BE STOCKPILED IN WINDROWS OUTSIDE OF MAJOR FLOW AREAS.
6. ALL DRAINAGE WORKS SHALL BE CONSTRUCTED AND STABILISED AS EARLY AS POSSIBLE DURING DEVELOPMENT.
7. ALL TAIL-OUT DRAINS SHALL BE GRASSED AND TRAPEZOIDAL IN SECTION. HAY BALES SHALL BE PLACED AS A SEDIMENTATION CONTROL DEVICE WHERE REQUIRED.
8. ALL DISTURBED AREAS SHALL BE REVEGETATED AS SOON AS THE RELEVANT WORKS ARE COMPLETED. TOPSOIL SHALL BE AMELIORATED AND COMPOSTED TO LANDSCAPE ARCHITECTS SPECIFICATIONS.
9. INLET FILTERS WILL BE INSTALLED WHERE SHOWN TO PREVENT WATER FROM DIRECTLY ENTERING THE PERMANENT DRAINAGE SYSTEM UNLESS IT IS RELATIVELY SEDIMENT FREE. IF THE LOCATION OF INLET FILTERS ARE NOT SHOWN ON THE PLAN THEIR LOCATION SHALL BE AT THE DISCRETION OF THE SUPERINTENDENT.

STOCKPILES

10. SPOIL AND TOPSOIL STOCKPILES SHALL BE LOCATED NO CLOSER THAN 2m (PREFERABLY 5m) FROM EXISTING VEGETATION, CONCENTRATED WATER FLOW, ROADS AND HAZARD AREAS.
11. IF STOCKPILES ARE TO BE IN PLACE FOR LONGER THAN 10 DAYS THEN THEY SHALL BE STABILISED BY COVERING WITH MULCH OR WITH TEMPORARY VEGETATION.
12. STOCKPILES SHALL BE IN WINDROWS NO HIGHER THAN 2m HIGH AND SHALL HAVE BATTER SLOPES NO STEEPER THAN 1 IN 2. AN EARTH BANK SHALL BE INSTALLED ON THE UPSLOPE SIDE AND SEDIMENT FENCING SHALL BE INSTALLED ALONG THE LENGTH OF THE DOWNSLOPE SIDE ON ANY STOCKPILE.

MAINTENANCE

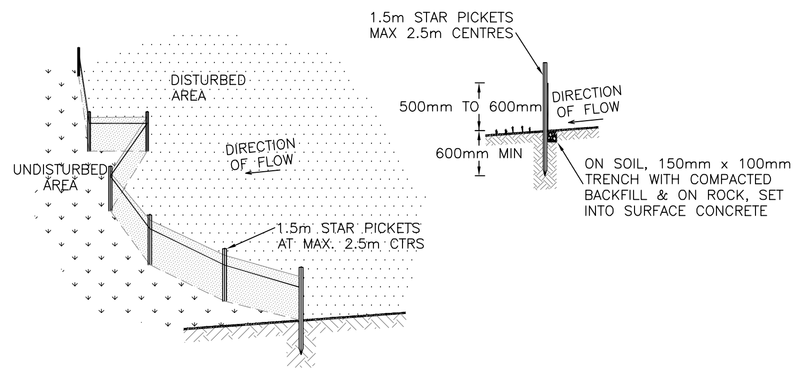
13. ALL SEDIMENT BASINS AND TRAPS SHALL BE CLEANED WHEN THE STRUCTURES ARE A MAXIMUM OF 60% FULL OF SOLID MATERIALS (INCLUDING DURING THE MAINTENANCE PERIOD) AND DISPOSED OF IN A MANNER THAT PREVENTS FURTHER POLLUTION OF THE SITE.
14. TEMPORARY SEDIMENT AND EROSION CONTROL DEVICES WILL BE RETAINED UNTIL AFTER THE LANDS THEY ARE PROTECTING, ARE COMPLETELY REHABILITATED.
15. THE CONTRACTOR WILL INSPECT THE SITE AT LEAST WEEKLY OR AFTER ANY STORM EVENT AND WILL:
   - ENSURE THAT DRAINS OPERATE PROPERLY AND TO EFFECT ANY NECESSARY REPAIRS;
   - REMOVE SPILLED SAND OR OTHER MATERIALS FROM HAZARD AREAS (E.G. LANDS CLOSER THAN FIVE METRES FROM AREAS OF LIKELY CONCENTRATED OR HIGH VELOCITY FLOWS ESPECIALLY DRAINS, WATERWAYS AND PAVED AREAS);
   - REMOVE TRAPPED SEDIMENT WHENEVER LESS THAN DESIGN CAPACITY REMAINS WITHIN THE STRUCTURE;
   - ENSURE REHABILITATED LANDS HAVE EFFECTIVELY REDUCED THE EROSION HAZARD AND TO INITIATE UPGRADING OR REPAIR AS APPROPRIATE;
   - CONSTRUCT ADDITIONAL EROSION AND/OR SEDIMENT CONTROL WORKS AS REQUIRED;
   - MAINTAIN EROSION AND SEDIMENT CONTROL MEASURES IN A FULLY FUNCTIONING CONDITION UNTIL ALL EARTHWORK ACTIVITIES ARE COMPLETED AND THE SITE IS REHABILITATED; AND
   - REMOVE TEMPORARY EROSION AND SEDIMENT CONTROL STRUCTURES AS THE LAST ACTIVITY IN THE REHABILITATION PROGRAM.

SEQUENCE OF WORKS:

1. INSTALL SOIL AND WATER MANAGEMENT MEASURES AS DETAILED.
2. CONSTRUCT EARTHWORKS
3. CONSTRUCT DRAINAGE STRUCTURES
4. CONSTRUCT ROADS
5. REHABILITATE SITE AND REMOVE MANAGEMENT DEVICES.
6. CLEAN SEDIMENT BASINS AND CONSTRUCT AS OSD BASINS AS PER DETIALED DESIGN.

OTHER:

2. DETENTION BASINS TO BE CONSTRUCTED AT COMMENCEMENT OF PROJECT AND USED AS SEDIMENT BASINS UNTIL CONSTRUCTION OF OPERATIONAL AREA IS COMPLETE.
3. SEDIMENT BASINS TO BE CONVERTED TO OSD AT THE COMPLETION OF CONSTRUCTION AND WHEN SITE HAS BEEN STABILISED.
4. MAIN BUNDS AROUND SITE TO BE FORMED AT COMMENCEMENT OF EARTHWORKS.
5. ALL SITE SEDIMENT AND EROSION CONTROL MEASURES TO BE REVIEWED AS EARTHWORKS PROCEED.



SEDIMENT CONTROL FENCE DETAIL

NTS

SEDIMENT FENCE CONSTRUCTION NOTES

1. CONSTRUCT SEDIMENT FENCES AS CLOSE AS POSSIBLE TO BEING PARALLEL TO THE CONTOURS OF THE SITE, BUT WITH SMALL RETURNS AS SHOWN IN THE DRAWING TO LIMIT THE CATCHMENT AREA OF ANY ONE SECTION. THE CATCHMENT AREA SHOULD BE SMALL ENOUGH TO LIMIT WATER FLOW IF CONCENTRATED AT ONE POINT TO 50 LITRES PER SECOND IN THE DESIGN STORM EVENT, USUALLY THE 10YR EVENT.
2. CUT A 150mm DEEP TRENCH ALONG THE UPSLOPE LINE OF THE FENCE FOR THE BOTTOM OF THE FABRIC TO BE ENTRENCHED.
3. DRIVE 1.5m LONG STAR PICKETS INTO THE GROUND AT 2.5m INTERVALS (MAX) AT THE DOWNSLOPE EDGE OF THE TRENCH. ENSURE ANY STAR PICKETS ARE FITTED WITH SAFETY CAPS.
4. FIX SELF-SUPPORTING GOETEXTILE TO THE UPSLOPE SIDE OF THE POSTS ENSURING IT GOES TO THE BASE OF THE TRENCH. FIX THE GEOTEXTILE WITH WIRE TIES OR AS RECOMMENDED BY THE MANUFACTURER. ONLY USE GEOTEXTILE SPECIFICALLY PRODUCED FOR SEDIMENT FENCING. THE USE OF SHADE CLOTH FOR THIS PURPOSE IS NOT SATISFACTORY.
5. JOIN SECTIONS OF FABRIC AT A SUPPORT POST WITH A 150mm OVERLAP.
6. BACKFILL THE TRENCH OVER THE BASE OF THE FABRIC AND COMPACT IT THOROUGHLY OVER THE GEOTEXTILE.

LEGEND:

- PROPOSED SWALE (AND DIRECTION OF FLOW)
- PROPOSED SEDIMENT FENCE
- DESIGN SURFACE CONTOURS (0.5m INTERVAL)
- STABILISED SITE ACCESS
- SEDIMENT BASIN LOCATION

LAND USE	LIMITATIONS	COMMENTS
CONSTRUCTION AREAS	DISTURBANCE TO BE NO FURTHER THAN FIVE (5) AND PREFERABLE TWO (2) METRES FROM THE EDGE OF ANY ESSENTIAL ENGINEERING ACTIVITY AS SHOWN ON THE PLAN	ALL SITE WORKERS WILL CLEARLY RECOGNISE THESE ZONES THAT, WHERE APPROPRIATE, ARE IDENTIFIED WITH BARRIER FENCING (UPSLOPE) AND SEDIMENT FENCING (DOWNSLOPE) OR SIMILAR MATERIALS
ACCESS AREAS	LIMITED TO A MAXIMUM WIDTH OF TEN (10) METRES	THE SITE MANAGER WILL DETERMINE AND MARK THE LOCATION OF THESE ZONES ONSITE. ALL SITE WORKERS WILL CLEARLY RECOGNISE THEIR BOUNDARIES THAT, WHERE APPROPRIATE, ARE MARKED WITH BARRIER MESH, SEDIMENT FENCING, OR SIMILAR MATERIALS.

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COORD. ORIGIN: | SCALE: 1:2000 | DATUM: | DATE: NOV 2008 | DRAWN: BF | DESIGNED: BF | CHECKED: NL

CLIENT: Alexandria Landfill Pty Ltd, 32 Burrows Road, Alexandria NSW 2015, (02) 9519 9999

LIGHT HORSE BUSINESS CENTRE
STORMWATER MANAGEMENT PLAN
EROSION AND SEDIMENT CONTROL PLAN

DRAWING: P03
SHEET: 3 OF 3
A1

APPENDIX B

Blue Book calculations

Note: These "Detailed Calculation" spreadsheets relate only to high erosion hazard lands as identified in figure 4.6 or where the designer chooses to use the RUSLE to size sediment basins. The "Standard Calculation" spreadsheets should be used on low erosion hazard lands as identified by figure 4.6 and where the designer chooses not to run the RUSLE in calculations.

1. Site Data Sheet

Site Name: Light Horse Business Centre

Site Location: Blacktown City Council

Precinct: Eastern Creek

Description of Site: Quarry pit, steep walls w likely low sediment yield now, require clean water run-on capture from intermediate capped landfill area & wall runoff where nec to reduce leachate generation

Site area	Site						Remarks
Total catchment area (ha)	26.5						
Disturbed catchment area (ha)	26.5						

Soil analysis

% sand (faction 0.02 to 2.00 mm	10						Soil texture should be assessed through mechanical dispersion only. Dispersing agents (e.g. Calgon) should not be used
% silt (fraction 0.002 to 0.02 mm)	20						
% clay (fraction finer than 0.002 mm)	70						
Dispersion percentage	30.0						E.g. enter 10 for dispersion of 10%
% of whole soil dispersible	24						See Section 6.3.3(e)
Soil Texture Group	D						See Section 6.3.3(c), (d) and (e)

Rainfall data

Design rainfall depth (days)	5						See Sections 6.3.4 (d) and (e)
Design rainfall depth (percentile)	80						See Sections 6.3.4 (f) and (g)
x-day, y-percentile rainfall event	25						See Section 6.3.4 (h)
Rainfall intensity: 2-year, 6-hour storm	10.1						See IFD chart for the site

RUSLE Factors

Rainfall erosivity (*R* -factor)	2250						Automatic calculation from above data
Soil erodibility (*K* -factor)	0.038						RUSLE data can be obtained from Appendixes A, B and C
Slope length (m)	100						
Slope gradient (%)	5						
Length/gradient (*LS* -factor)	1.35						
Erosion control practice (*P* -factor)	1.3	1.3	1.3	1.3	1.3	1.3	
Ground cover (*C* -factor)	1	1	1	1	1	1	

Calculations

Soil loss (t/ha/yr)	150						
Soil Loss Class	1						See Section 4.4.2(b)
Soil loss (m^3/ha/yr)	115						
Sediment basin storage volume, m^3	520						See Sections 6.3.4(i) and 6.3.5 (e)

2. Storm Flow Calculations

Peak flow is given by the Rational Formula:

$$Qy = 0.00278 \times C_{10} \times F_Y \times I_{y,\,tc} \times A$$

where:

- Q_y is peak flow rate (m^3/sec) of average recurrence interval (ARI) of "Y" years
- C_{10} is the runoff coefficient (dimensionless) for ARI of 10 years. Rural runoff coefficients are given in Volume 2, figure 5 of Pilgrim (1998), while urban runoff coefficients are given in Volume 1, Book VIII, figure 1.13 of Pilgrim (1998) and construction runoff coefficients are given in Appendix F
- F_y is a frequency factor for "Y" years. Rural values are given in Volume 1, Book IV, Table 1.1 of Pilgrim (1998) while urban coefficients are given in Volume 1, Book VIII, Table 1.6 of Pilgrim (1998)
- A is the catchment area in hectares (ha)
- $I_{y,\,tc}$ is the average rainfall intensity (mm/hr) for an ARI of "Y" years and a design duration of "tc" (minutes or hours)

Time of concentration (t_c) = $0.76 \times (A/100)^{0.38}$ hrs (Volume 1, Book IV of Pilgrim, 1998)

Note: For urban catchments the time of concentration should be determined by more precise calculations or reduced by a factor of 50 per cent.

Peak flow calculations, 1

Site	A (ha)	tc (mins)	Rainfall intensity, I, mm/hr						C_{10}
			$1_{yr,tc}$	$5_{yr,tc}$	$10_{yr,tc}$	$20_{yr,tc}$	$50_{yr,tc}$	$100_{yr,tc}$	
	26.5	28	36.2	60	68	78	91	101	0.85

Peak flow calculations, 2

ARI (yrs)	Frequency factor (F_y)	Peak flows						Comment
						5		
		(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m3/s)	
1 yr,tc	0.8	1.813						Page D-11 BCC eng guidelions for development
5 yr,tc	0.95	3.569						
10 yr,tc	1	4.258						
20 yr,tc	1.05	5.129						
50 yr,tc	1.15	6.553						
100 yr,tc	1.2	7.589						

4. Volume of Sediment Basins, *Type D* and *Type F* Soils

Basin volume = settling zone volume + sediment storage zone volume

Settling Zone Volume

The settling zone volume for *Type F* and *Type D* soils is calculated to provide capacity to contain all runoff expected from up to the y-percentile rainfall event. The volume of the basin's settling zone (V) can be determined as a function of the basin's surface area and depth to allow for particles to settle and can be determined by the following equation:

$$V = 10 \times C_v \times A \times R_{x\text{-day, y-\%ile}} \ (m^3)$$

where:

10 = a unit conversion factor

C_v = the volumetric runoff coefficient defined as that portion of rainfall that runs off as stormwater over the x-day period

$R_{x\text{-day, y-\%ile}}$ = is the x-day total rainfall depth (mm) that is not exceeded in y percent of rainfall events. (See Sections 6.3.4(d), (e), (f), (g) and (h)).

A = total catchment area (ha)

Sediment Storage Zone Volume

In the detailed calculation on Soil Loss Classes 1 to 4 lands, the sediment storage zone can be taken as 50 percent of the settling zone capacity. Alternately designers can design the zone to store the 2-month soil loss as calculated by the RUSLE (Section 6.3.4(i)(ii)). However, on Soil Loss Classes 5, 6 and 7 lands, the zone must contain the 2-month soil loss as calculated by the RUSLE (Section 6.3.4(i)(iii).

Place an "X" in the box below to show the sediment storage zone design parameters used here

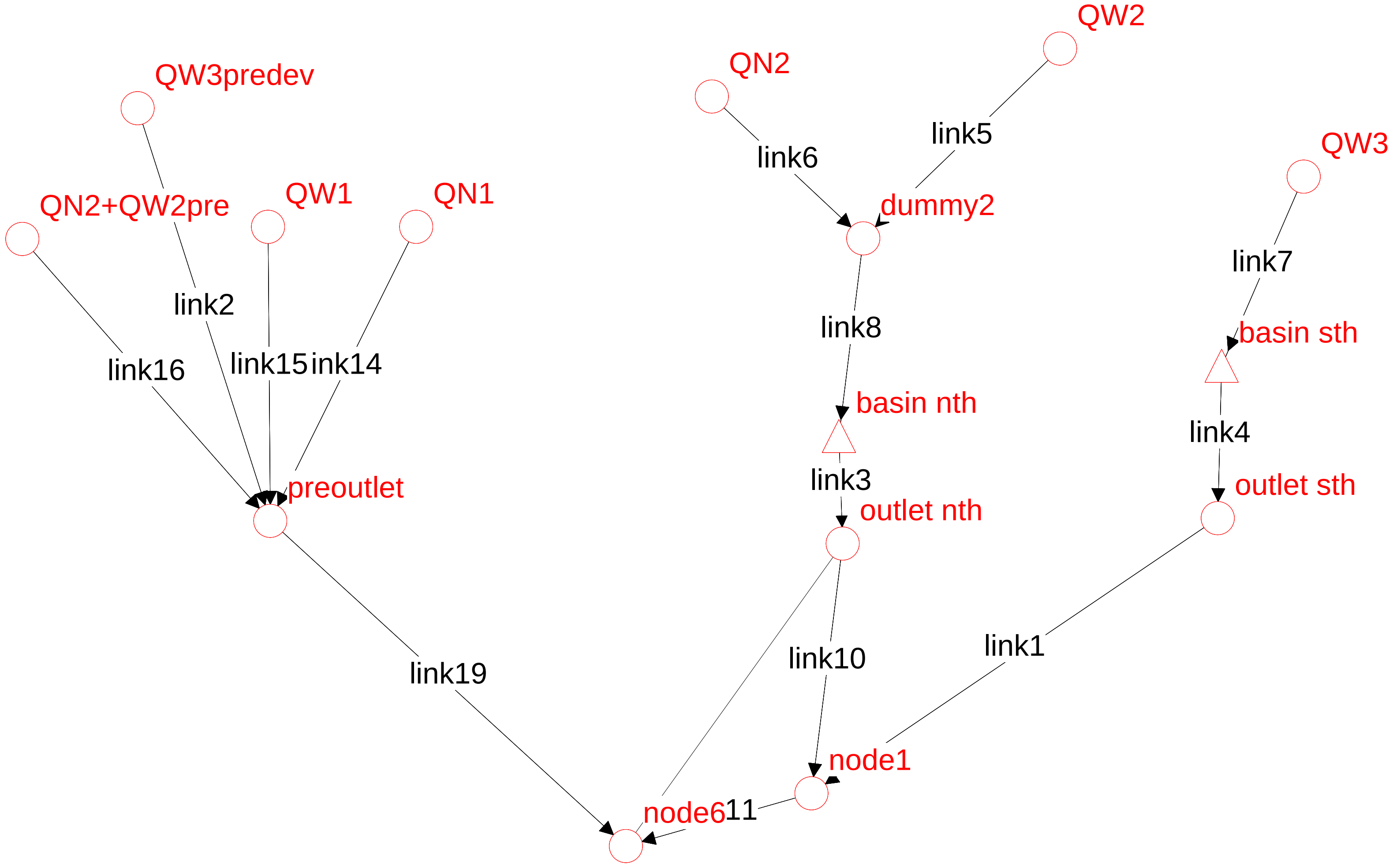
	50% of settling zone capacity,
X	2 months soil loss calculated by RUSLE

Total Basin Volume

Site	C_v	$R_{x\text{-day, y-\%ile}}$	Total catchment area (ha)	Settling zone volume (m^3)	Sediment storage volume (m^3)	Total basin volume (m^3)
	0.58	25	26.5	3842.5	520	4362.5

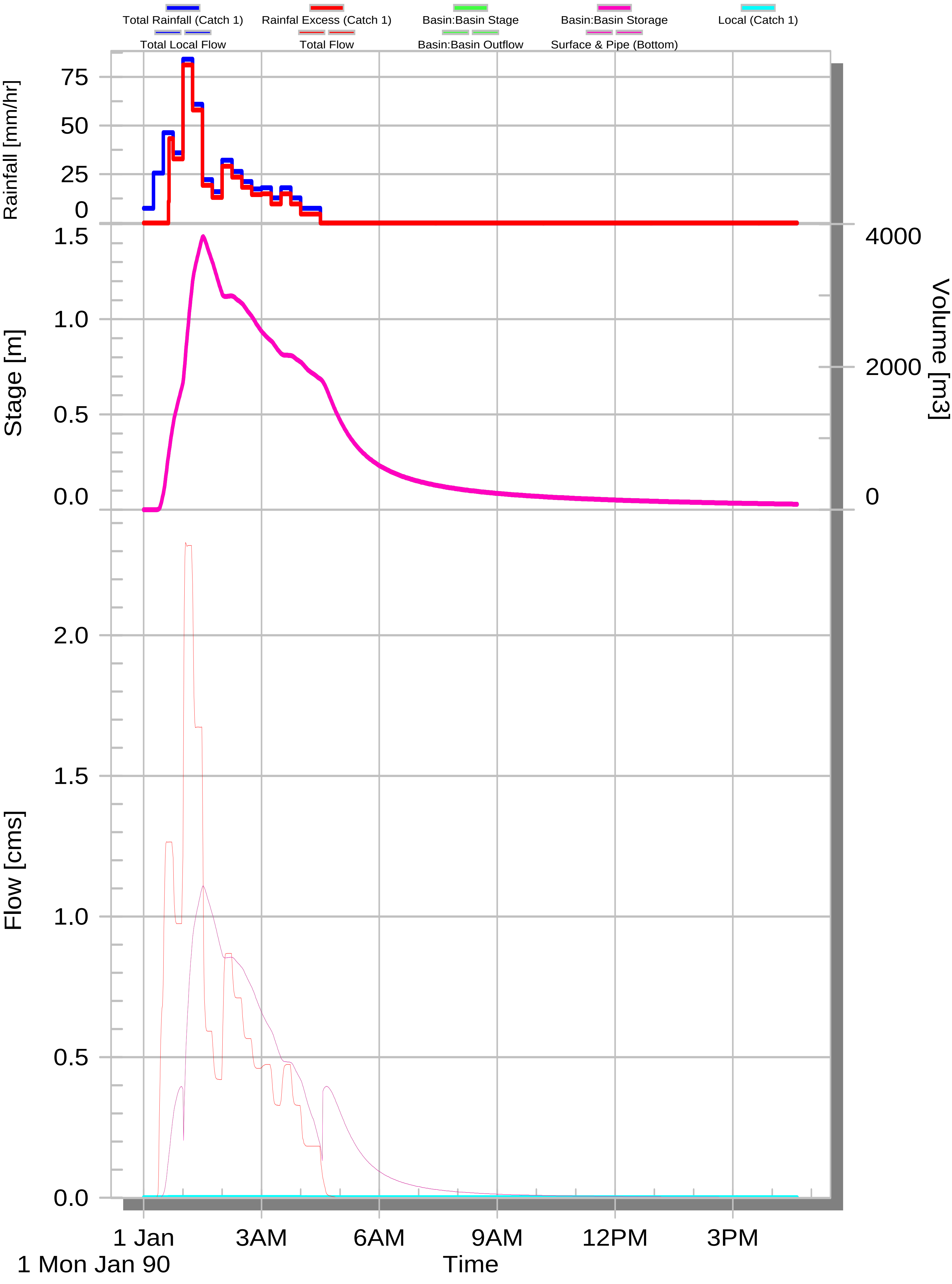
APPENDIX C

Model outcomes – RAFTS


QW3predev
QN2+QW2pre
QW1
QN1
link2
link16
link15
ink14
preoutlet
link19
QN2
QW2
link6
link5
dummy2
link8
basin nth
link3
outlet nth
link10
QW3
link7
basin sth
link4
outlet sth
link1
node1
node6
11

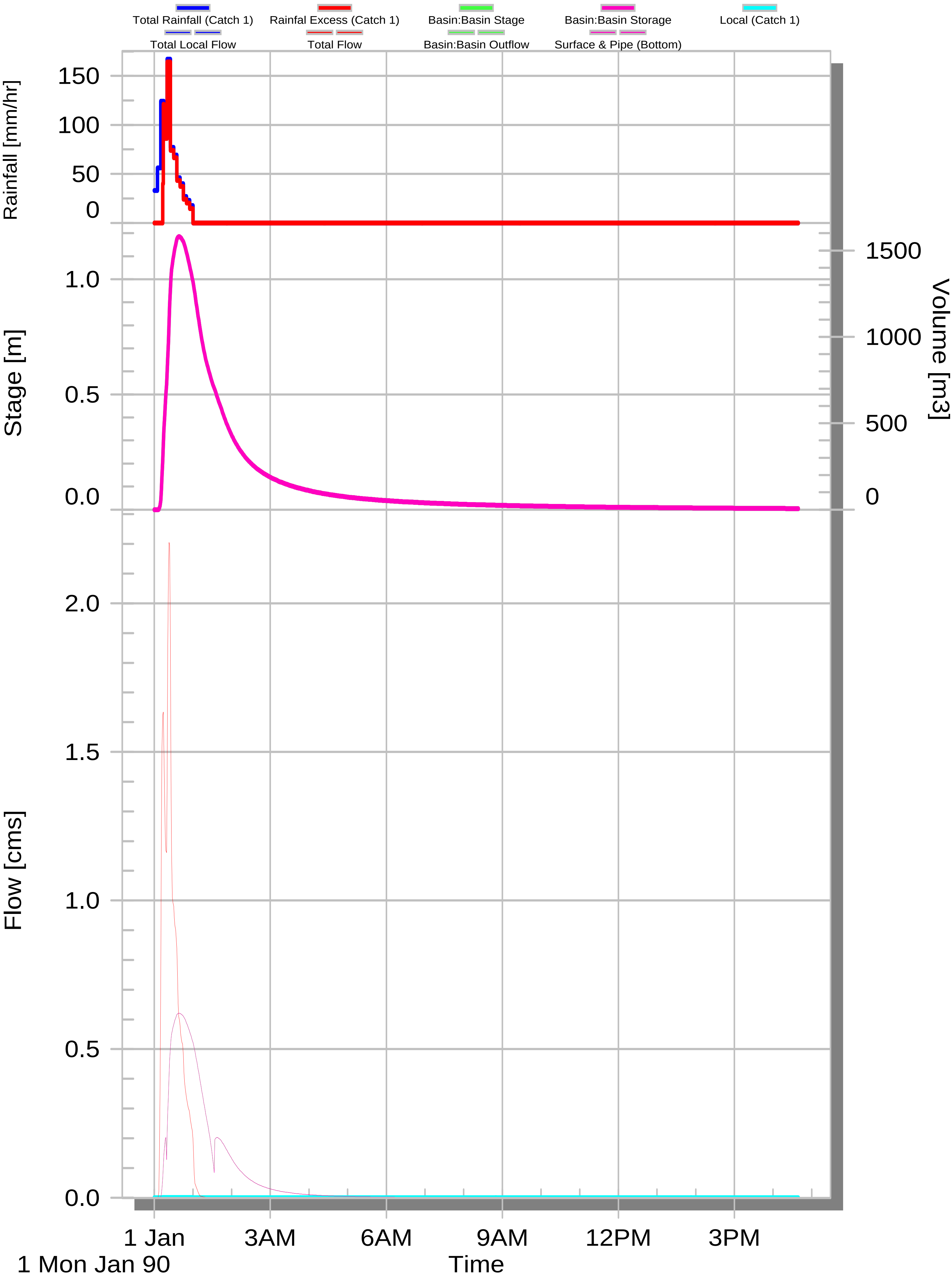
basin 1

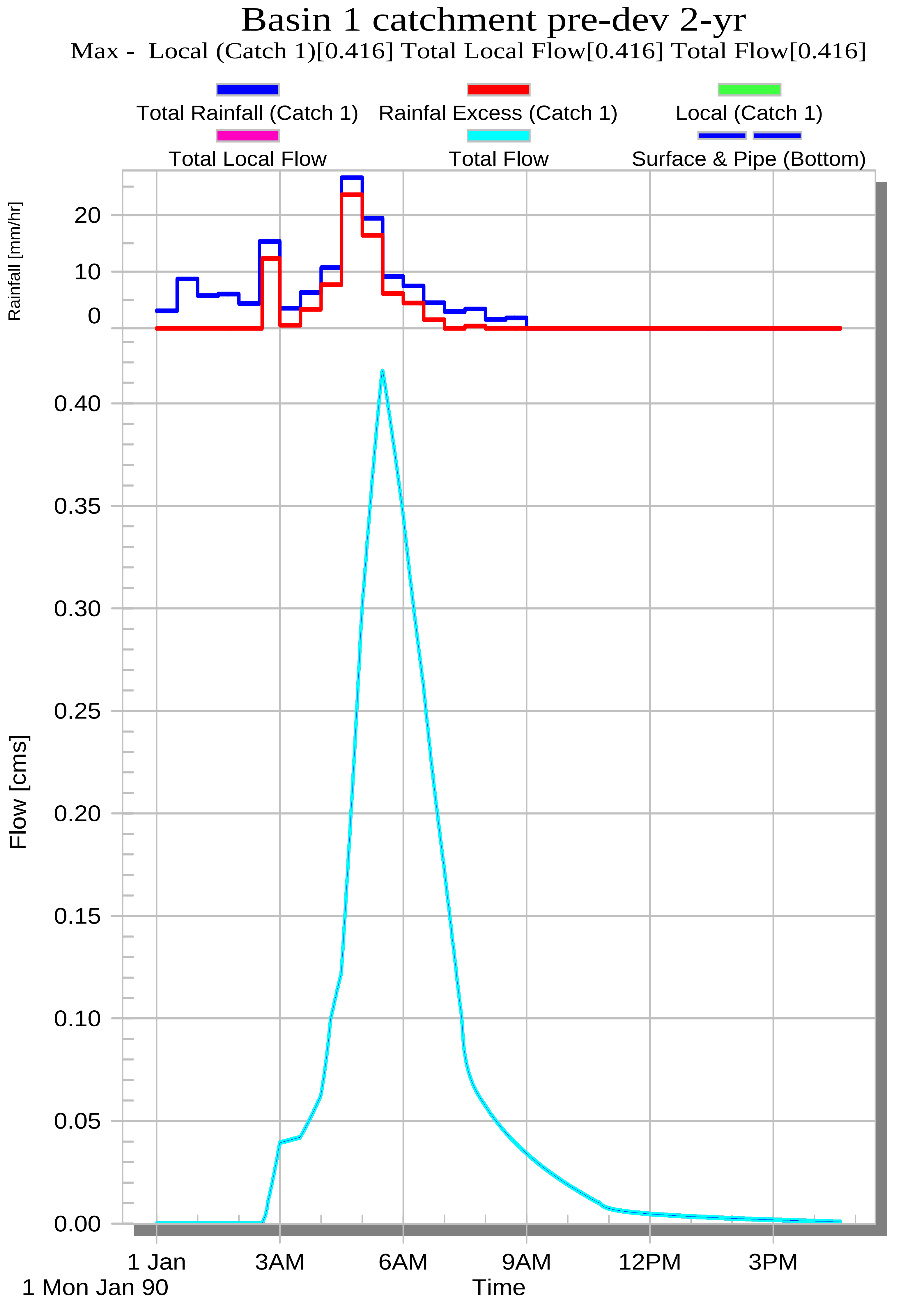
Max - Basin:Basin Stage[1.434] Basin:Basin Storage[3825.2] Local (Catch 1)[0.000] Total Local Flow[0.000] Total Flow[2.331]



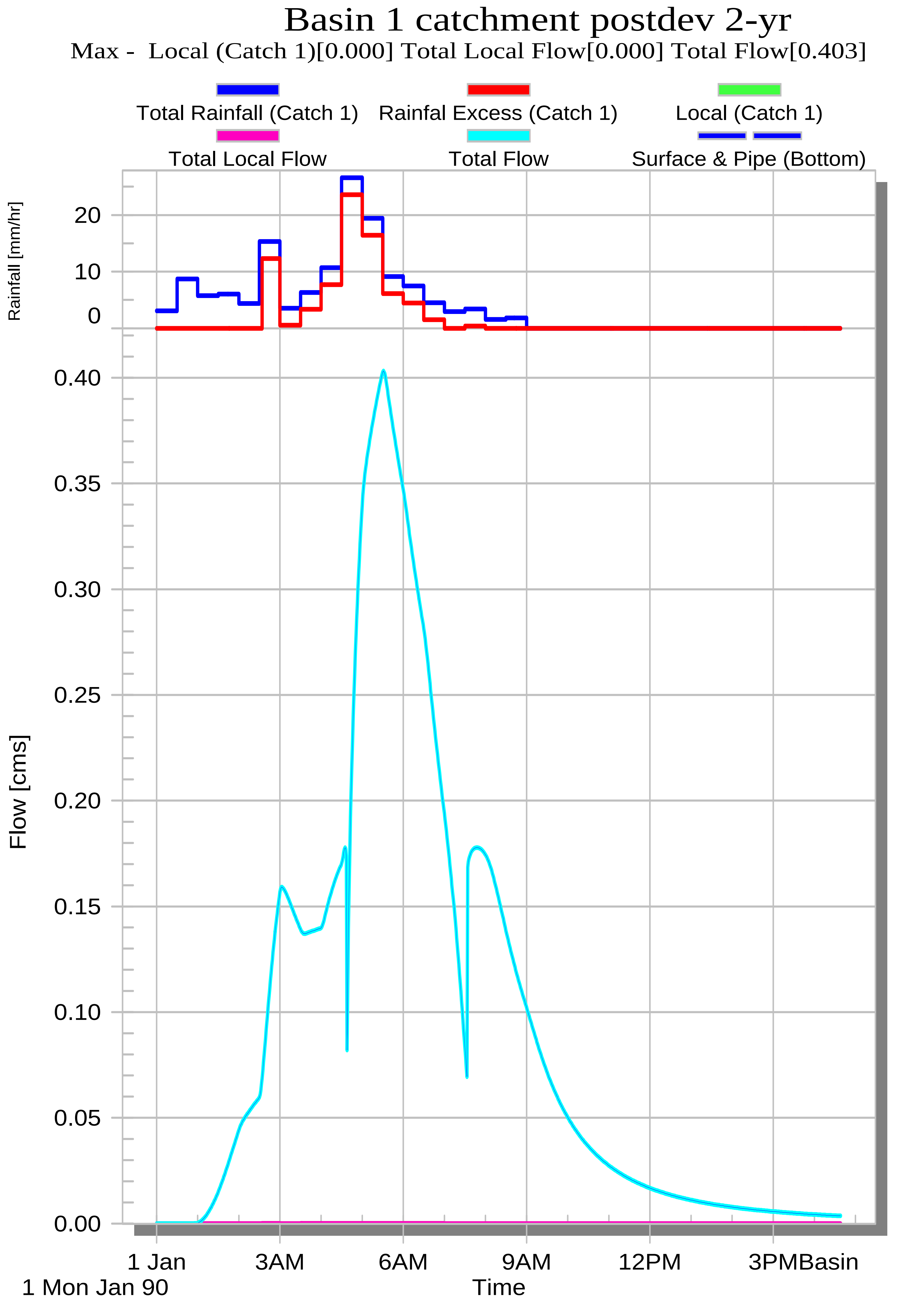
basin 2

Max - Basin:Basin Stage[1.186] Basin:Basin Storage[1580.7] Local (Catch 1)[0.000] Total Local Flow[0.000] Total Flow[2.205]

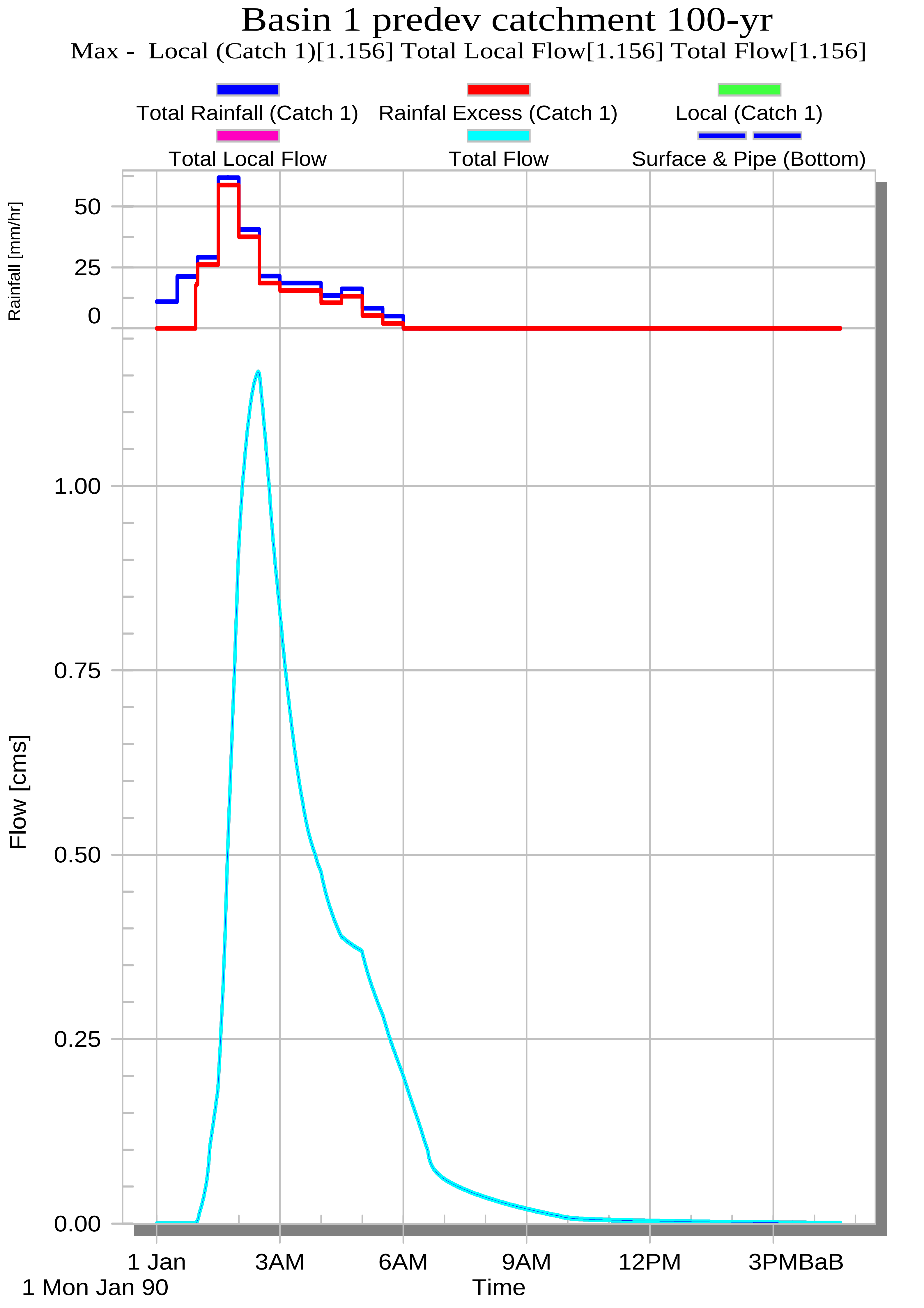




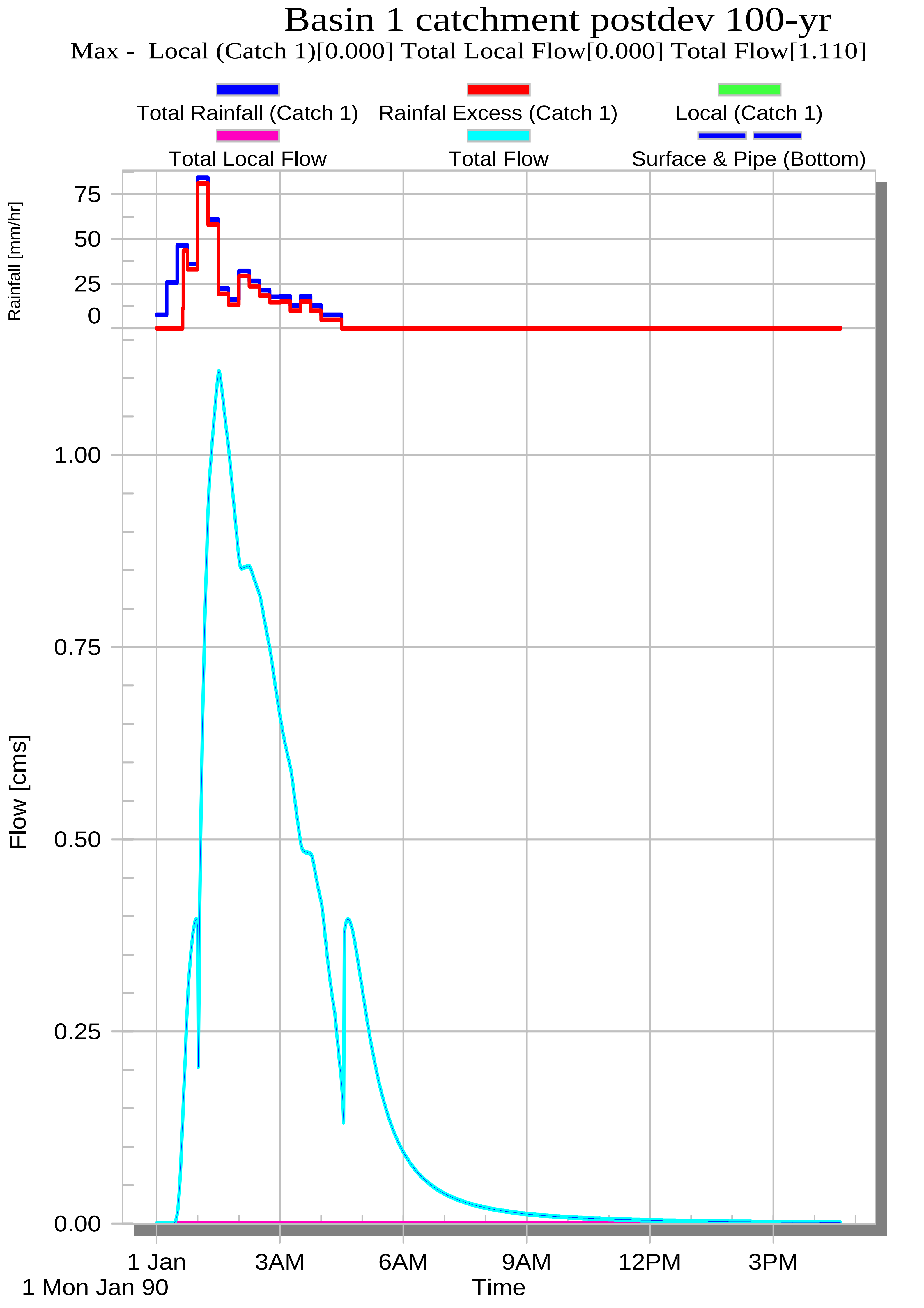
Basin 1 catchment pre-dev 2-yr
Max - Local (Catch 1)[0.416] Total Local Flow[0.416] Total Flow[0.416]
Total Rainfall (Catch 1)
Rainfal Excess (Catch 1)
Local (Catch 1)
Total Local Flow
Total Flow
Surface & Pipe (Bottom)
Rainfall [mm/hr]
20
10
0
Flow [cms]
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
1 Jan
3AM
6AM
9AM
12PM
3PM
1 Mon Jan 90
Time

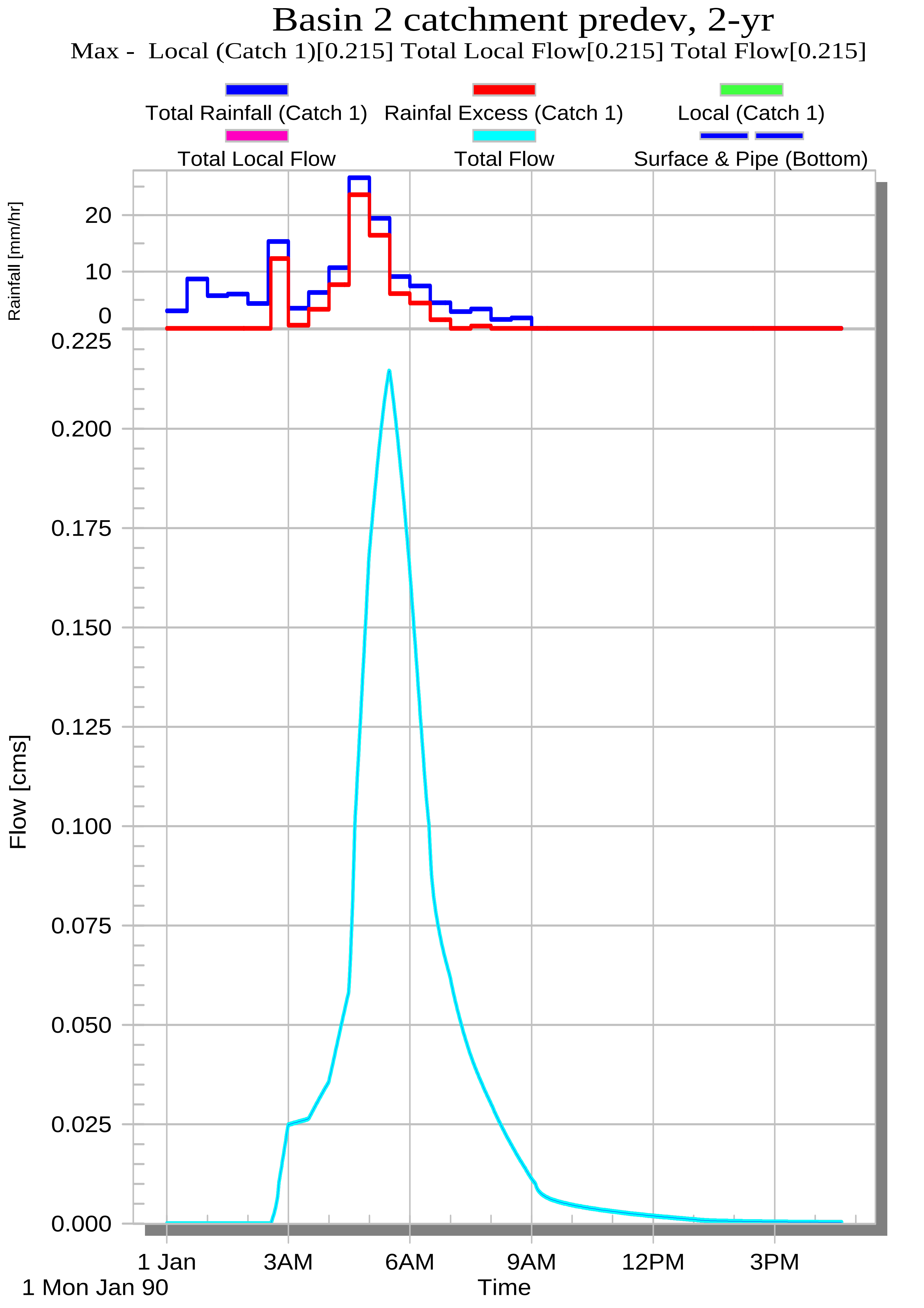


Basin 1 catchment postdev 2-yr
Max - Local (Catch 1)[0.000] Total Local Flow[0.000] Total Flow[0.403]
Total Rainfall (Catch 1)
Rainfal Excess (Catch 1)
Local (Catch 1)
Total Local Flow
Total Flow
Surface & Pipe (Bottom)
Rainfall [mm/hr]
20
10
0
Flow [cms]
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
1 Jan
3AM
6AM
9AM
12PM
3PMBasin
1 Mon Jan 90
Time

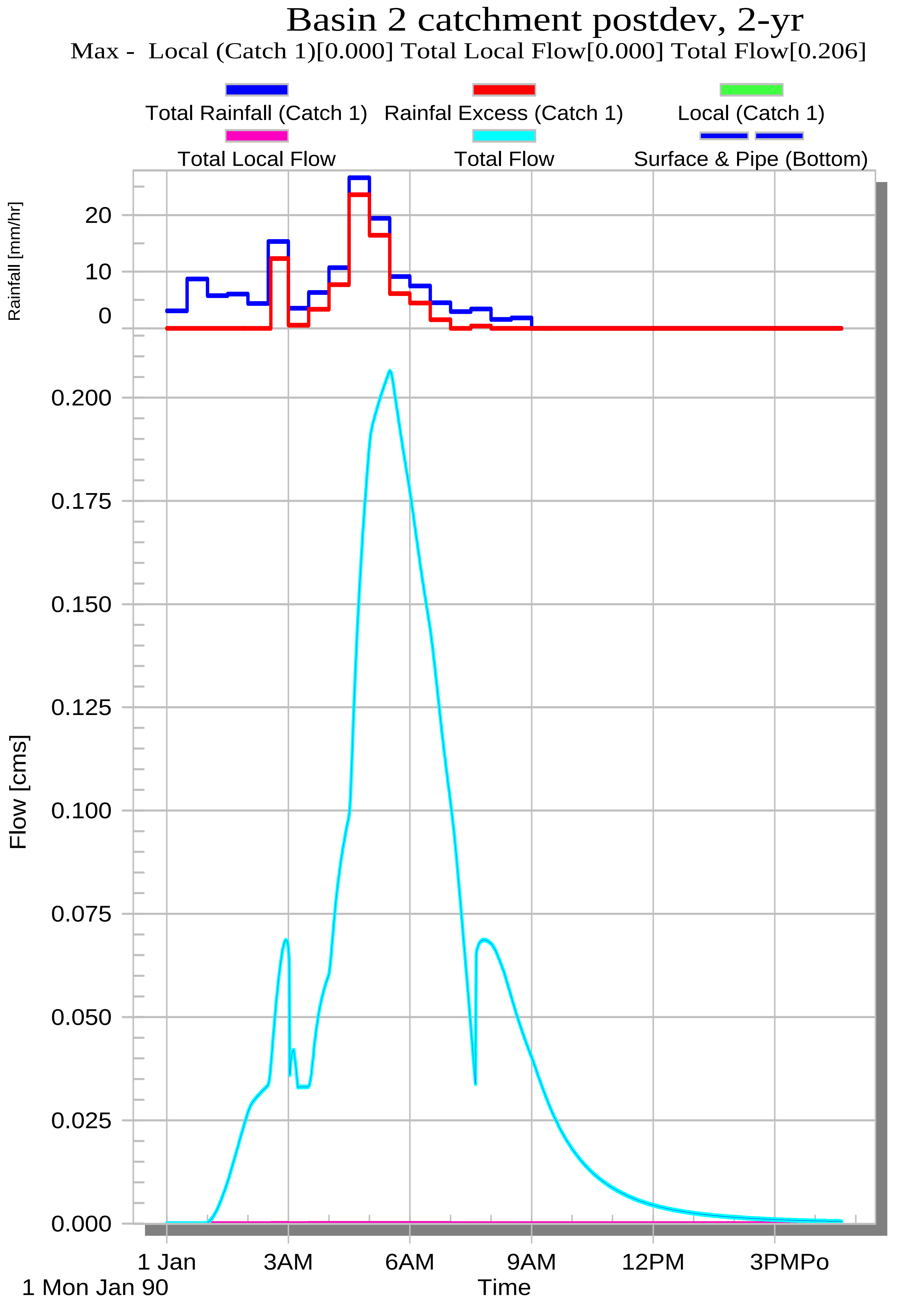


Basin 1 predev catchment 100-yr
Max - Local (Catch 1)[1.156] Total Local Flow[1.156] Total Flow[1.156]
Total Rainfall (Catch 1)
Rainfal Excess (Catch 1)
Local (Catch 1)
Total Local Flow
Total Flow
Surface & Pipe (Bottom)
Rainfall [mm/hr]
50
25
0
Flow [cms]
1.00
0.75
0.50
0.25
0.00
1 Jan
3AM
6AM
9AM
12PM
3PMBaB
1 Mon Jan 90
Time


Basin 1 catchment postdev 100-yr
Max - Local (Catch 1)[0.000] Total Local Flow[0.000] Total Flow[1.110]
Total Rainfall (Catch 1)
Rainfal Excess (Catch 1)
Local (Catch 1)
Total Local Flow
Total Flow
Surface & Pipe (Bottom)
Rainfall [mm/hr]
75
50
25
0
Flow [cms]
1.00
0.75
0.50
0.25
0.00
1 Jan
3AM
6AM
9AM
12PM
3PM
1 Mon Jan 90
Time



Basin 2 catchment predev, 2-yr
Max - Local (Catch 1)[0.215] Total Local Flow[0.215] Total Flow[0.215]
Total Rainfall (Catch 1)
Rainfal Excess (Catch 1)
Local (Catch 1)
Total Local Flow
Total Flow
Surface & Pipe (Bottom)
Rainfall [mm/hr]
20
10
0
0.225
0.200
0.175
0.150
0.125
0.100
0.075
0.050
0.025
0.000
Flow [cms]
1 Jan
3AM
6AM
9AM
12PM
3PM
1 Mon Jan 90
Time


Basin 2 catchment postdev, 2-yr
Max - Local (Catch 1)[0.000] Total Local Flow[0.000] Total Flow[0.206]
Total Rainfall (Catch 1)
Rainfal Excess (Catch 1)
Local (Catch 1)
Total Local Flow
Total Flow
Surface & Pipe (Bottom)
Rainfall [mm/hr]
20
10
0
Flow [cms]
0.200
0.175
0.150
0.125
0.100
0.075
0.050
0.025
0.000
1 Jan
3AM
6AM
9AM
12PM
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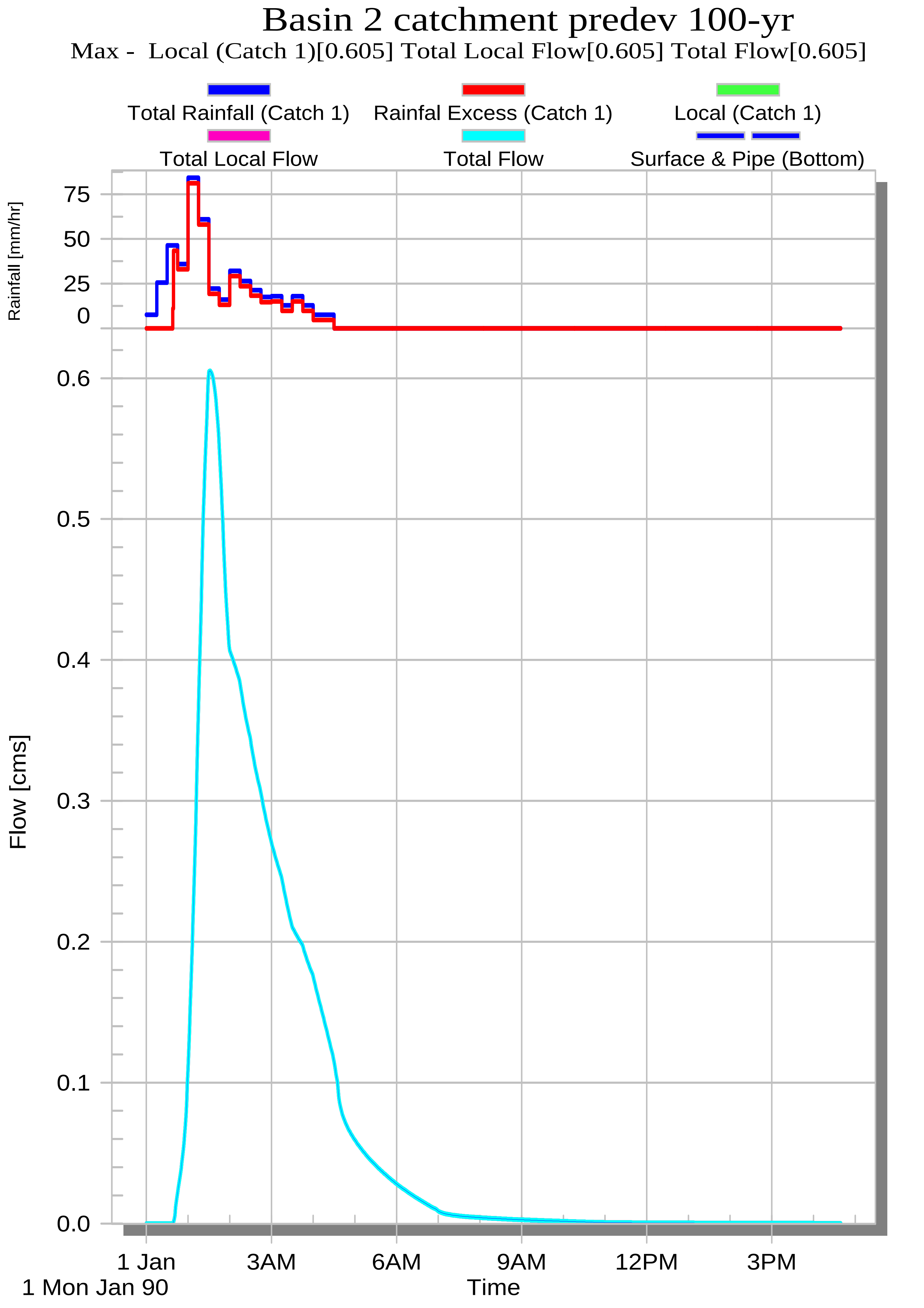
1 Mon Jan 90
Time



Basin 2 catchment predev 100-yr
Max - Local (Catch 1)[0.605] Total Local Flow[0.605] Total Flow[0.605]
Total Rainfall (Catch 1)
Rainfal Excess (Catch 1)
Local (Catch 1)
Total Local Flow
Total Flow
Surface & Pipe (Bottom)
Rainfall [mm/hr]
75
50
25
0
Flow [cms]
0.6
0.5
0.4
0.3
0.2
0.1
0.0
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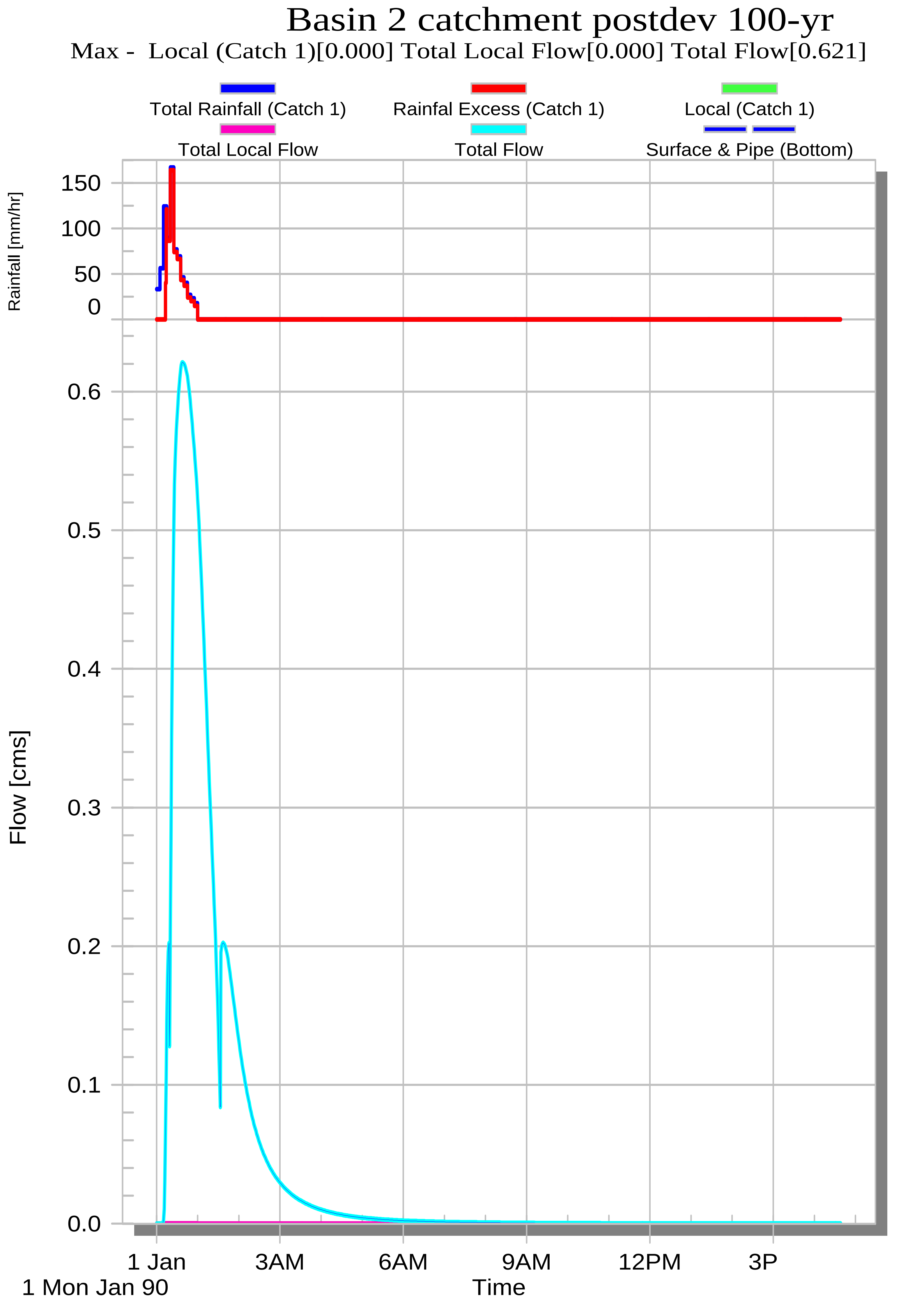
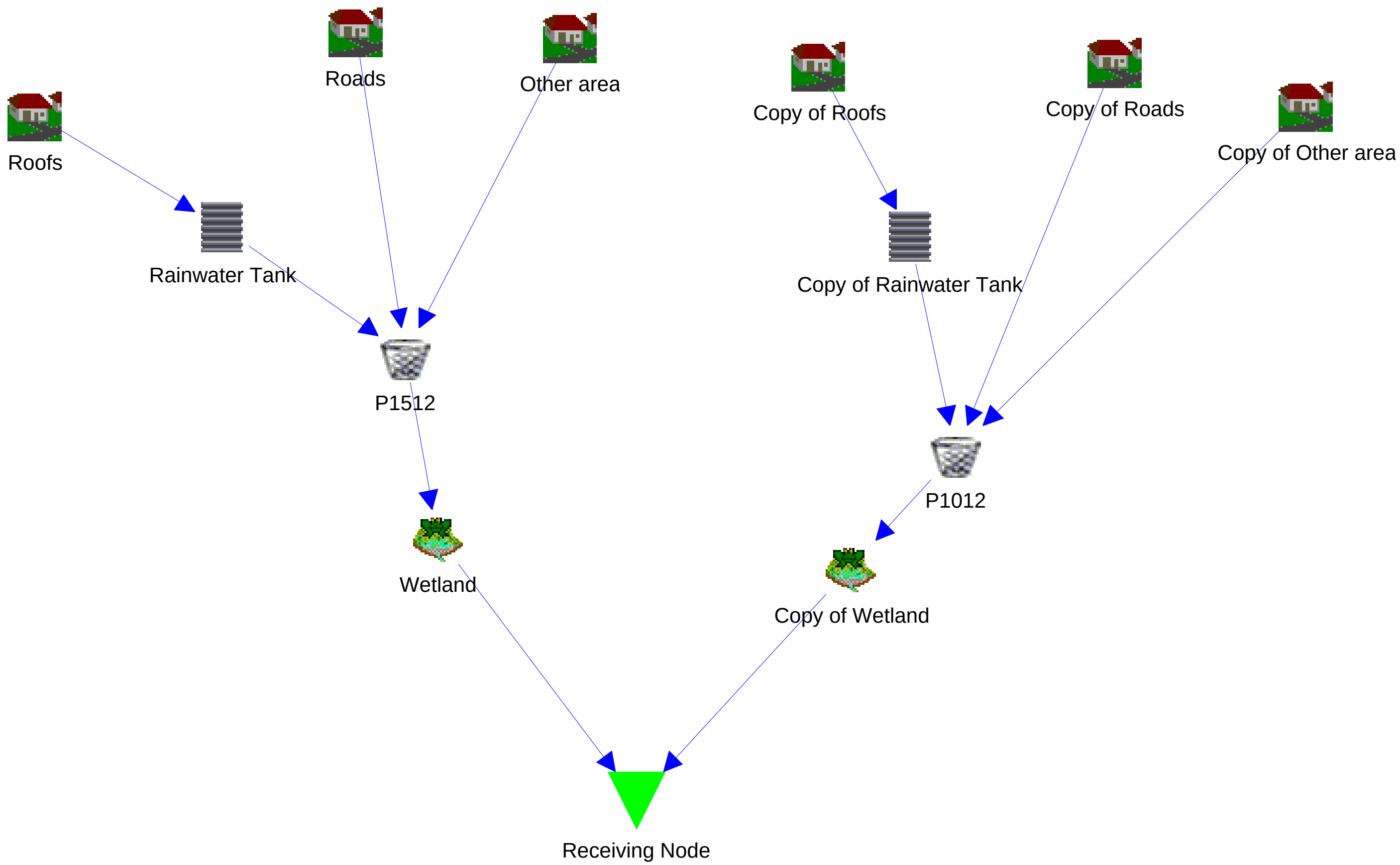
3AM
6AM
9AM
12PM
3PM
1 Mon Jan 90
Time



Basin 2 catchment postdev 100-yr
Max - Local (Catch 1)[0.000] Total Local Flow[0.000] Total Flow[0.621]
Total Rainfall (Catch 1)
Rainfal Excess (Catch 1)
Local (Catch 1)
Total Local Flow
Total Flow
Surface & Pipe (Bottom)
Rainfall [mm/hr]
150
100
50
0
Flow [cms]
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1 Jan
3AM
6AM
9AM
12PM
3P
1 Mon Jan 90
Time

APPENDIX D

Model – MUSIC layout


Roads
Other area
Copy of Roofs
Copy of Roads
Copy of Other area
Roofs
Rainwater Tank
Copy of Rainwater Tank
P1512
P1012
Wetland
Copy of Wetland
Receiving Node

10 Attachment D – NSW Dam Safety Committee Advice (2010)

Megan Bowling

From: Charles Navaratne <charles@damsafety.nsw.gov.au>
Sent: Thursday, 16 June 2011 1:02 PM
To: Jacqueline Brauman
Subject: RE: D1 Form

Hi Jackie,

As per your data, the OSD's are located below ground level.
Hence DSC would not prescribe them.

Regards

Charles Navaratne
Small Dams Engineer
NSW Dams Safety Committee
Level 3, 10 Valentine Av.
Parramatta NSW 2150

Phone 02-98957848
Fax 02-98957354

From: Jacqueline Brauman [mailto:JacquelineBrauman@dadi.com.au]
Sent: Thursday, 16 June 2011 12:29 PM
To: charles@damsafety.nsw.gov.au
Subject: FW: D1 Form

Hi Charles,

Further to our telephone conversation earlier this morning about OSD sizes at Eastern Creek, I have attached a D1 form and drawings for each OSD.

I have also attached the Blacktown Council's Concept Masterplan with crosses marking the OSD locations. The natural slope is to the west.

I would be pleased if you could advise immediately whether the OSDs need to be prescribed.

Thanks for your help.

Regards,
Jacqueline Brauman | Solicitor
Dial A Dump Industries | Keeping Australia Clean |32 Burrows Road Alexandria NSW 2015
Contact | P: (02) 9519-9999| F: (02) 9516-5559 |W: www.dadi.com.au

---

From: Charles Navaratne [mailto:charles@damsafety.nsw.gov.au]
Sent: Tuesday, 30 March 2010 3:05 PM
To: Steve Baxter
Subject: RE: D1 Form

Hi Steve,

11 Attachment E – DRAINS Catchment Plan

martens

MARTENS & ASSOCIATES PTY LTD
Sustainable Solutions
Environmental - Geotechnical - Civil
Hydraulic - Wastewater Engineers

6/37 Leighton Place
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CLIENT: DIAL A DUMP INDUSTRIES PTY LTD

THIS PLAN MUST NOT BE USED FOR CONSTRUCTION UNLESS SIGNED AS APPROVED BY PRINCIPAL CERTIFYING AUTHORITY
All measurements in mm unless otherwise specified.

TITLE: PRE DEVELOPMENT CATCHMENT PLAN - LIGHT HORSE BUSINESS PARK, EASTERN CREEK, NSW

PROJECT MANAGER: AN	DRAWING NUMBER: P1103002JD02V03		
DESIGNED: SL	DATUM: mAHD	SHEET 1 OF 2 SHEETS	
DRAWN: SL	HORIZONTAL RATIO: 1:1500 @ A1 1:3000 @ A3		
REVIEWED: AN	VERTICAL RATIO: 1:1500 @ A1 1:3000 @ A3	PAPER SIZE: A1 / A3	

REV.	DESCRIPTION	DATE	ISSUED
1	ATTACHMENT OF CONSOLIDATED STORMWATER MANAGEMENT PLAN	19.07.2011	SL
2	FINAL CONSOLIDATED STORMWATER MANAGEMENT PLAN	20.09.2011	AN
3	AMENDED FINAL CONSOLIDATED STORMWATER MANAGEMENT PLAN	21.09.2011	AN

martens
6/37 Leighton Place
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All measurements in mm unless otherwise specified.

TITLE: OSD CATCHMENT PLAN - LIGHT HORSE BUSINESS PARK, EASTERN CREEK, NSW

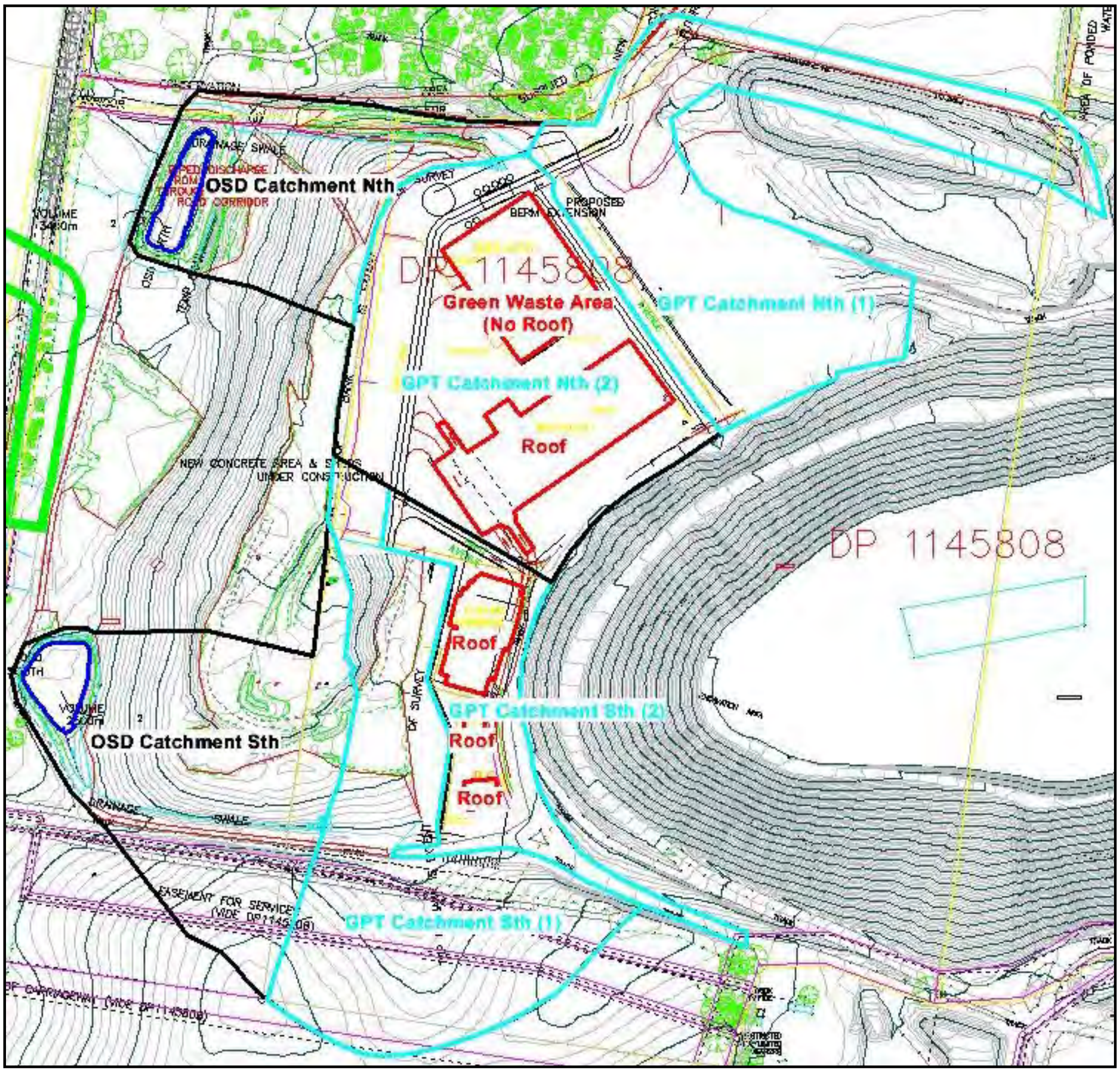
PROJECT MANAGER: AN

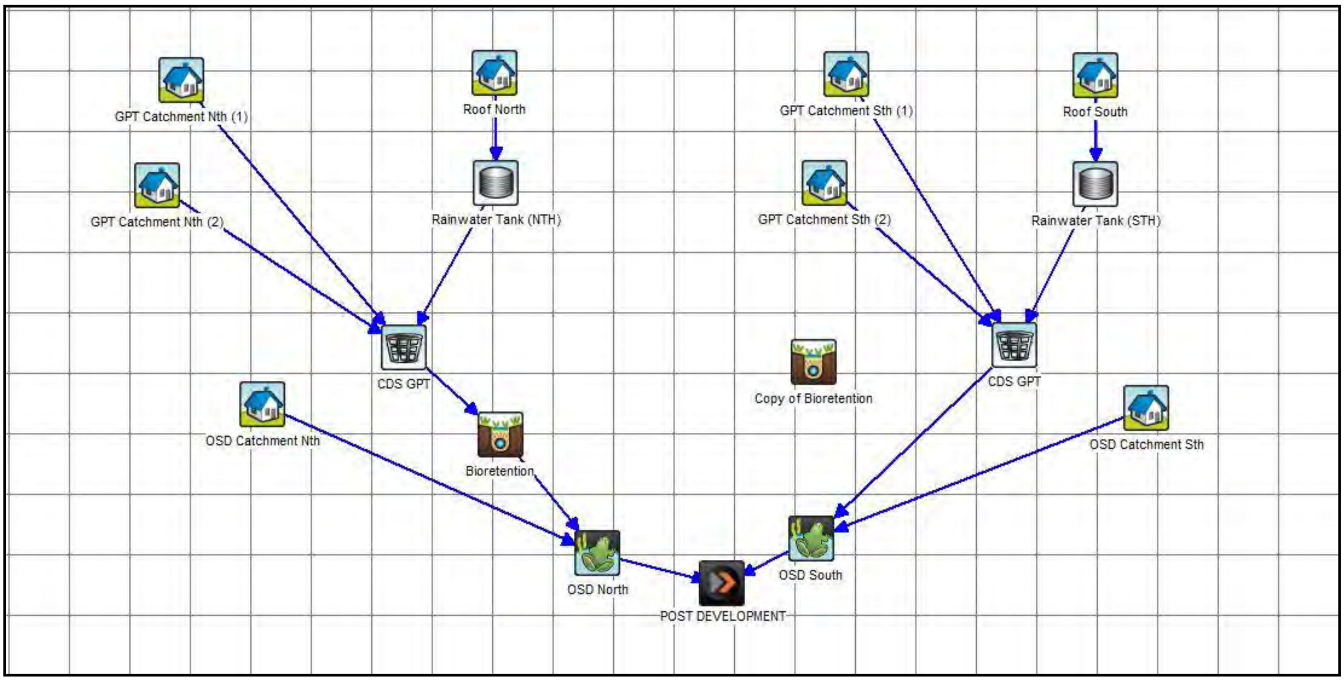
DRAWING NUMBER: P1103002JD02V03

DESIGNED: SL
DRAWN: SL
REVIEWED: AN
DATUM: mAHD
HORIZONTAL RATIO: 1:1500 @ A1, 1:3000 @ A3
VERTICAL RATIO: 1:1500 @ A1, 1:3000 @ A3
SHEET 2 OF 2 SHEETS
PAPER SIZE: A1 / A3

REV.	DESCRIPTION	DATE	ISSUED
1	ATTACHMENT OF CONSOLIDATED STORMWATER MANAGEMENT PLAN	19.07.2011	SL
2	FINAL CONSOLIDATED STORMWATER MANAGEMENT PLAN	22.09.2011	AN
3	AMENDED FINAL CONSOLIDATED STORMWATER MANAGEMENT PLAN	21.09.2011	AN

12 Attachment F – MUSIC Model Layout





Martens & Associates Pty Ltd ABN 85 070 240 890		Environment | Water | Wastewater | Geotechnical | Civil | Management	
Drawn:	MLB	MUSIC Model Catchments and Layout	FIGURE 1
Approved:	AN		
Date:	26.10.2011		
Scale:	NA		Job No: P1103002

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

BJ07/Rp040 Rev B
September 2009

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

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Author: .... Ian Grey

Signed: ....

Date: ....15th September 2009

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

....

Contents

List of Tables

List of Figures

List of Appendices

Executive Summary

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.

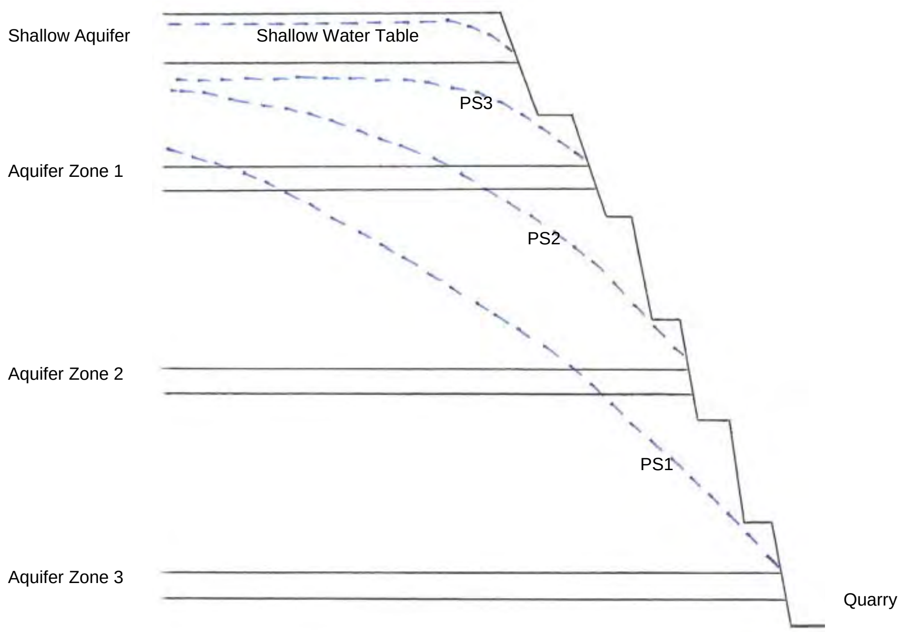
Previous assessment of the hydraulic conductivity of the deep shale strata surrounding the quarry derived from slug tests indicated values of $1.75x10^{-6}$ m/d to $8.7x10^{-6}$ m/d with a calculated inflow of around 2 m^3/day (ERM, 2008). This is around an order of magnitude below the observed inflow 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.

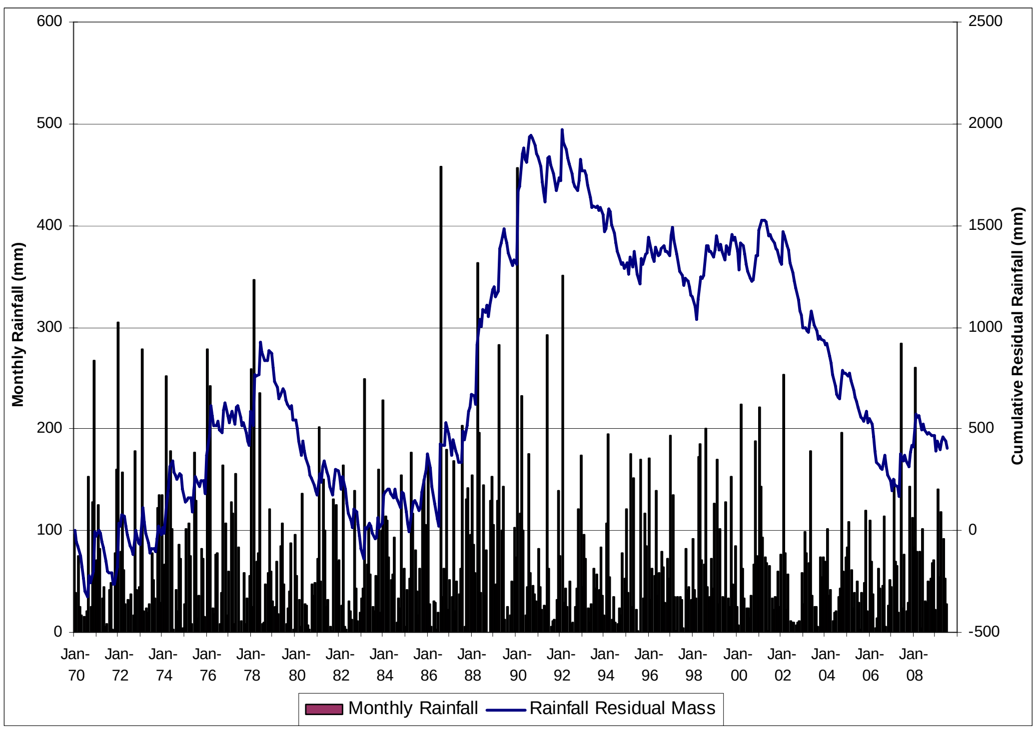
4.6.2 Long-Term Rainfall Data

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;

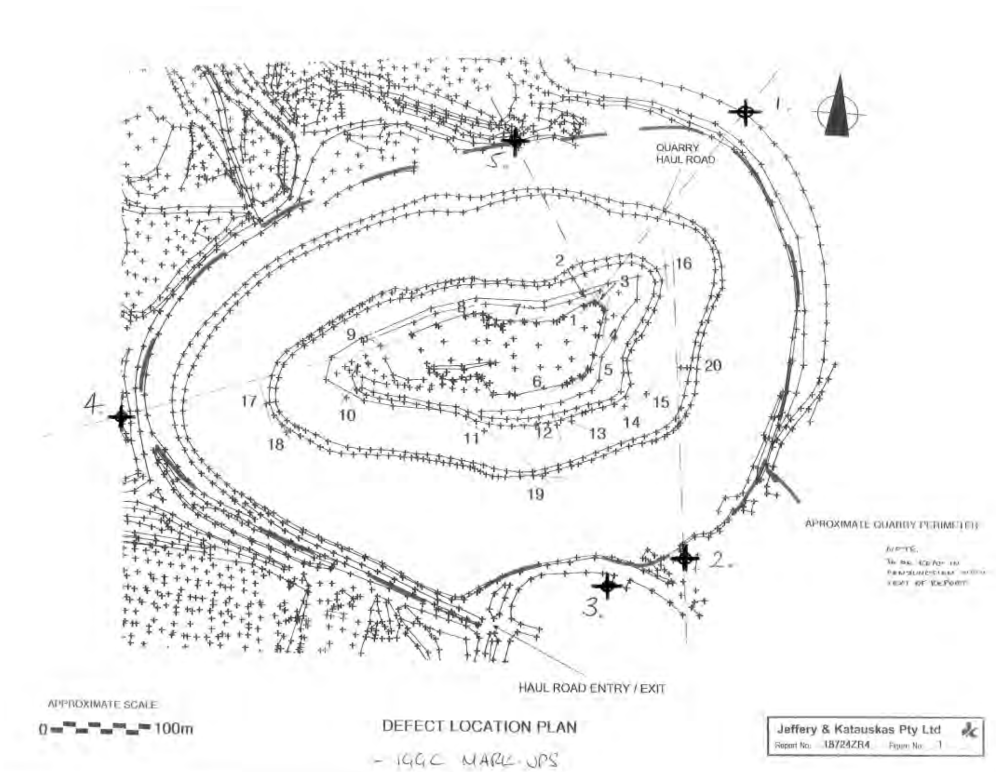
3. Southern area due to the presence of the strongest seepage. It should be noted, however, that it is IGGC's view that this seepage is associated with leakage from the quarry dewatering system over a long period (pipes, transfer pumps and surface channel) rather than reflecting true groundwater discharge;

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

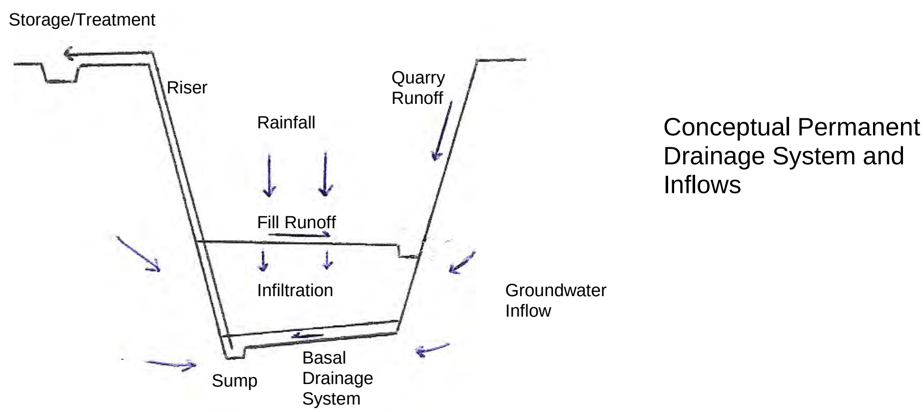
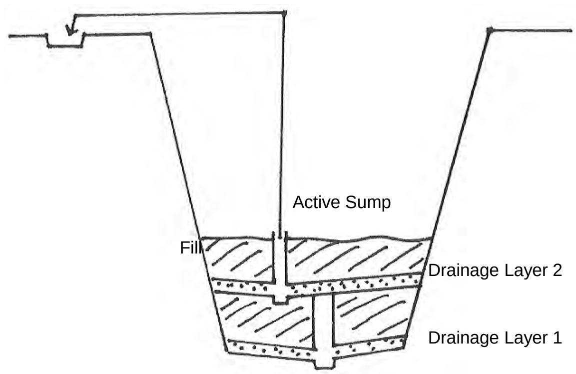
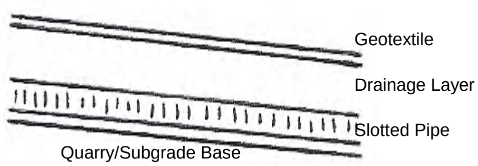


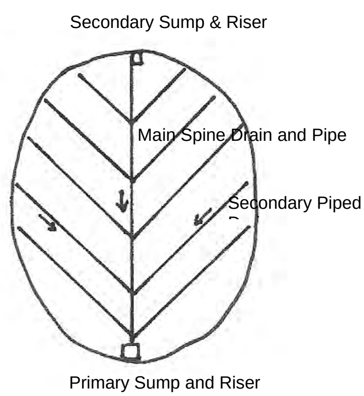
Figure 8.1: Water Collection System – Conceptual Design (continued)



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

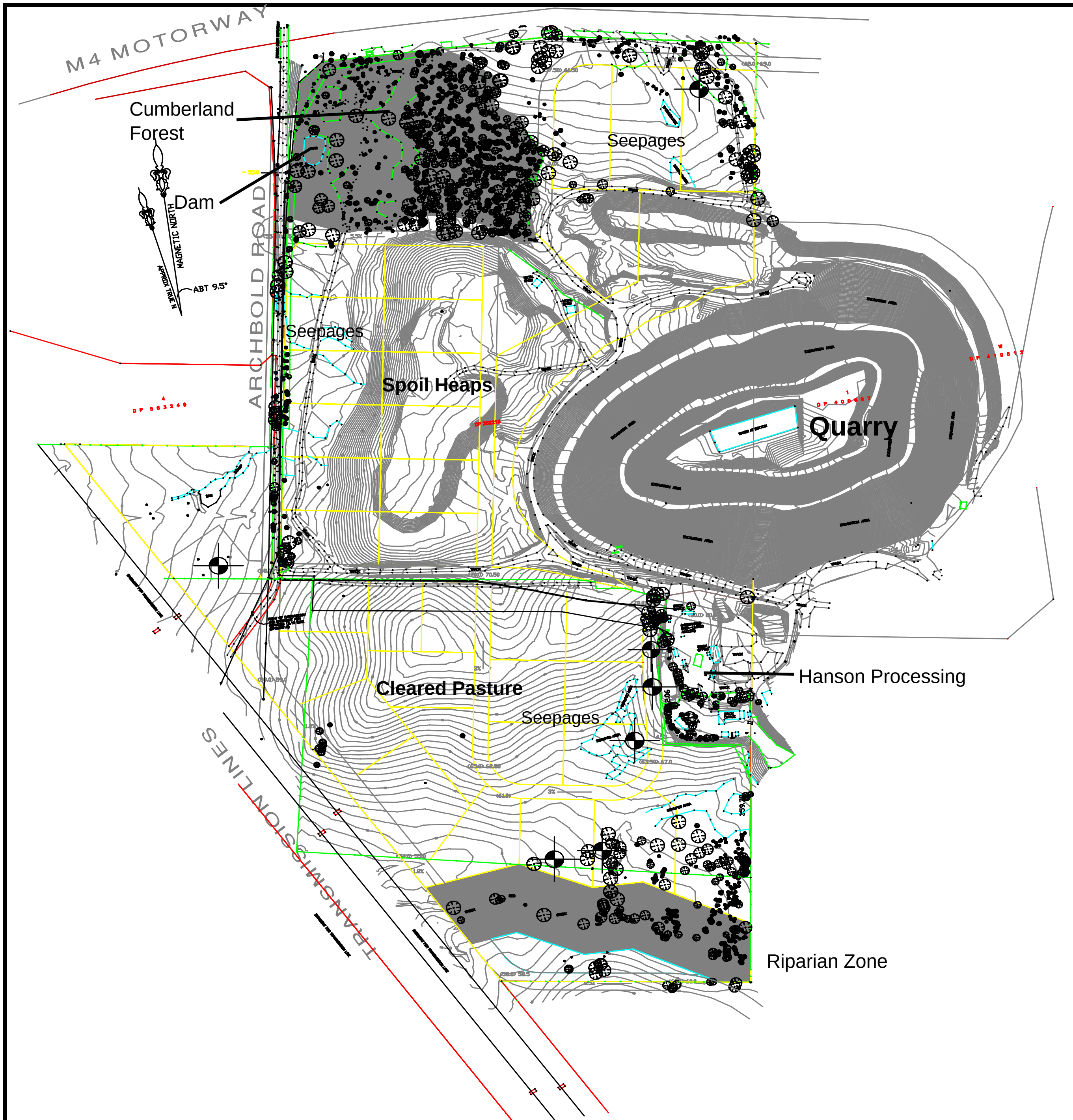

FIGURE 4.2: Site Features

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

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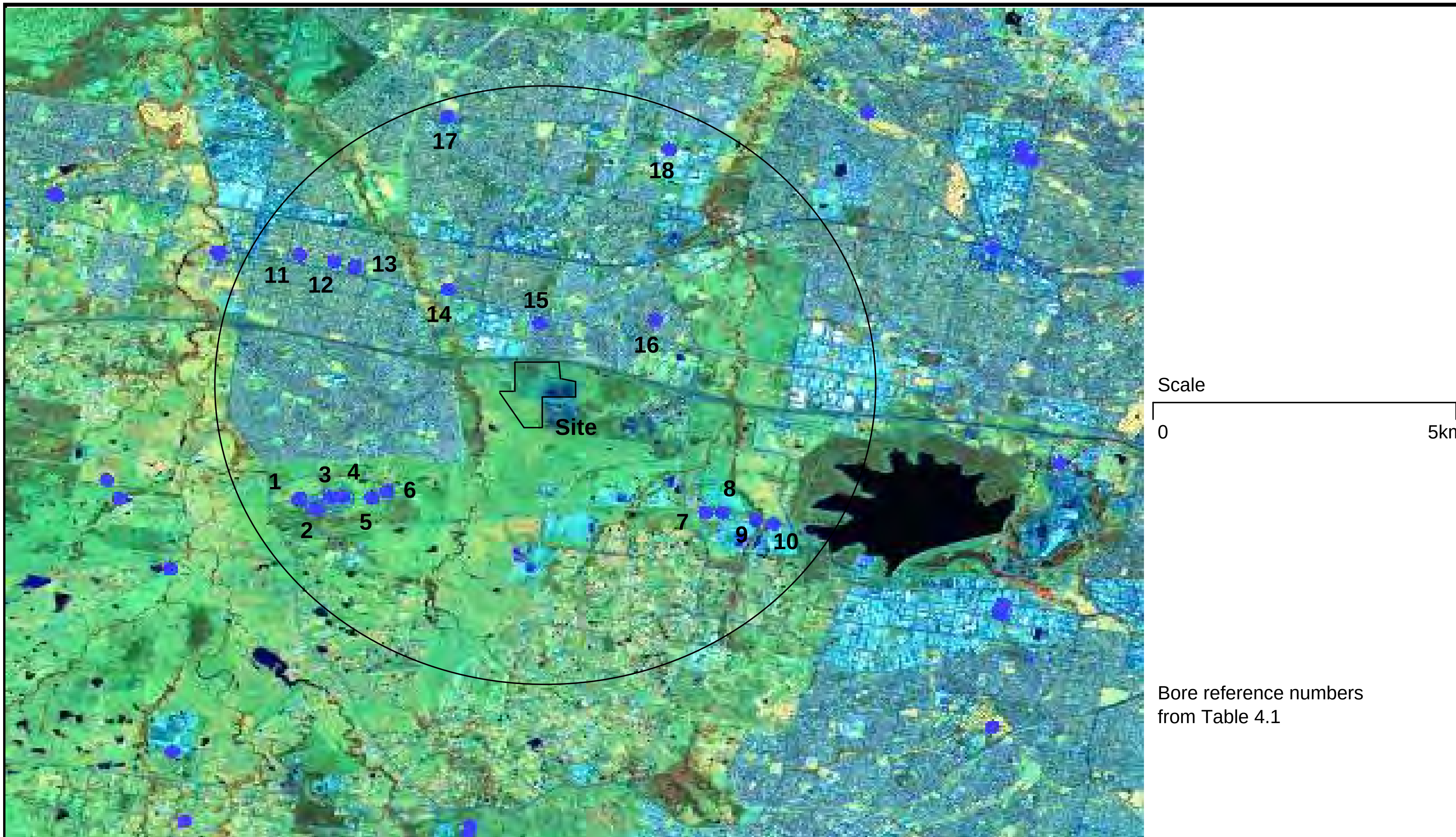

FIGURE 4.3: Locations of Registered Bores

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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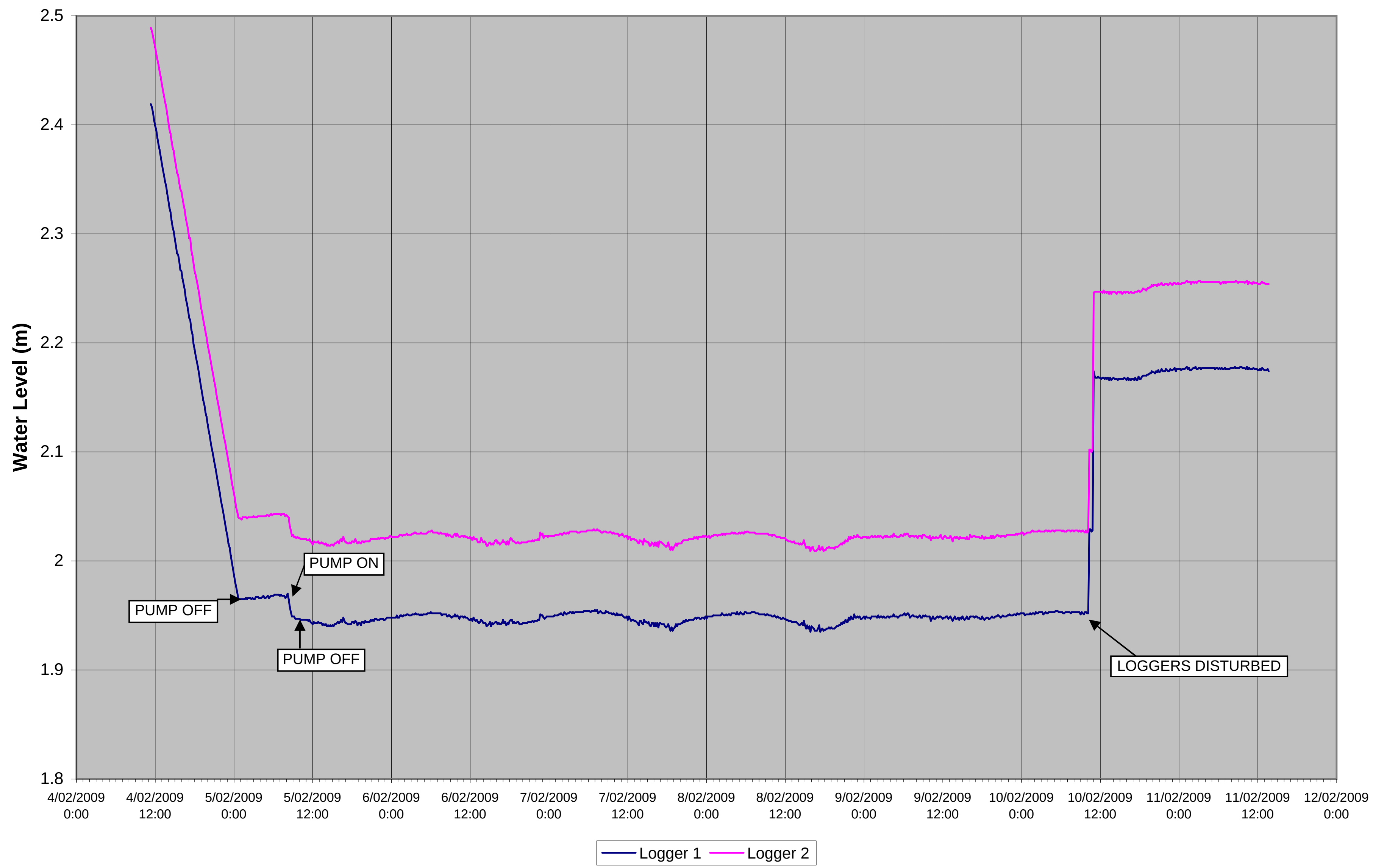

FIGURE 4.4: Quarry Pond Water Level Record

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

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

IGGC

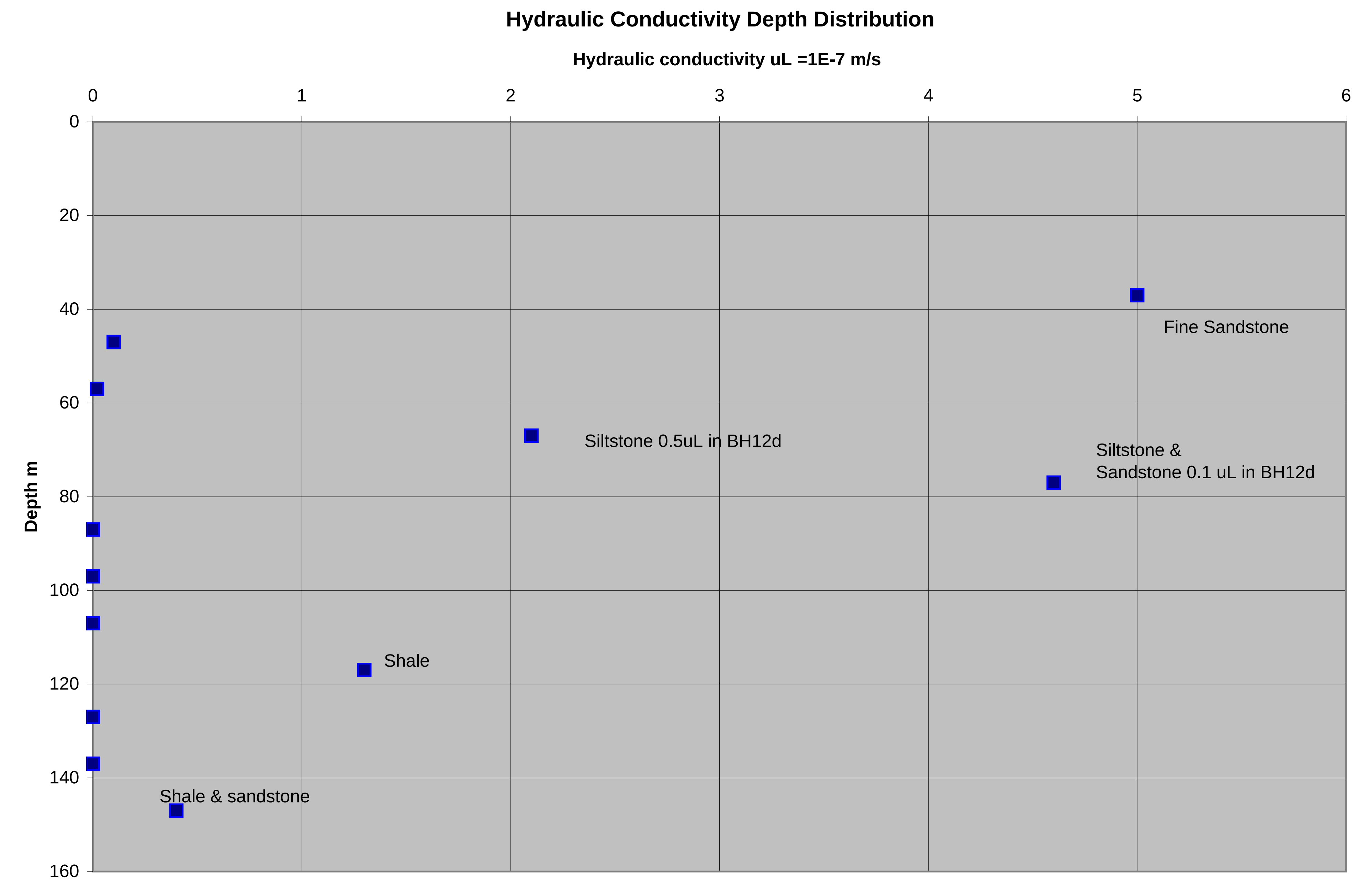

FIGURE 6.1a: Hydraulic Conductivity Depth Distribution from Packer Test Results - BH10d

Project: Detailed Hydrogeological Investigation and Assessment

Location: Proposed Light Horse Landfill Site, Eastern Creek

Client: Light Horse Business Centre Pty Ltd

Project No: BJ07

IGGC

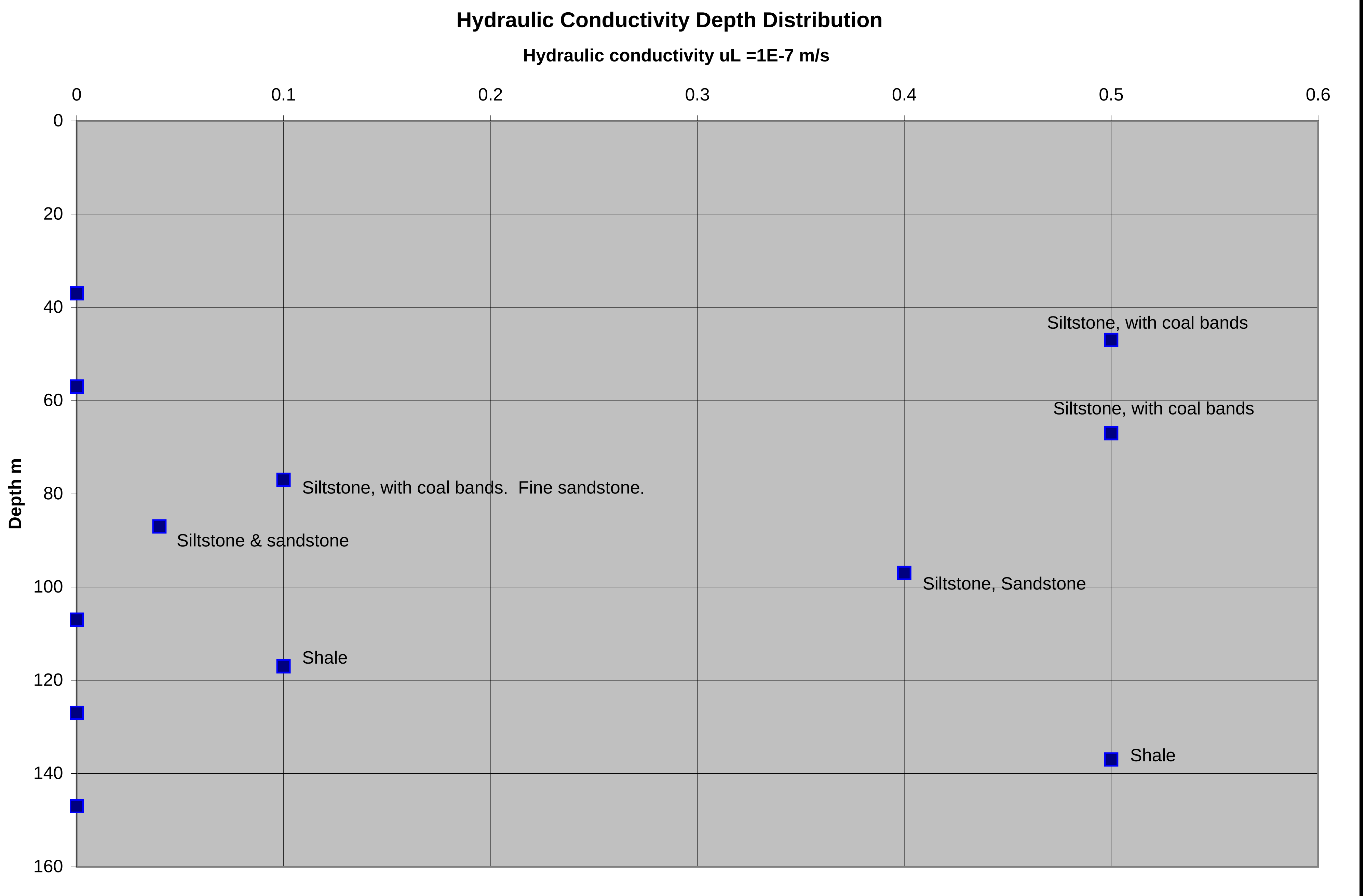

FIGURE 6.1b: Hydraulic Conductivity Depth Distribution from Packer Test Results - BH12d

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

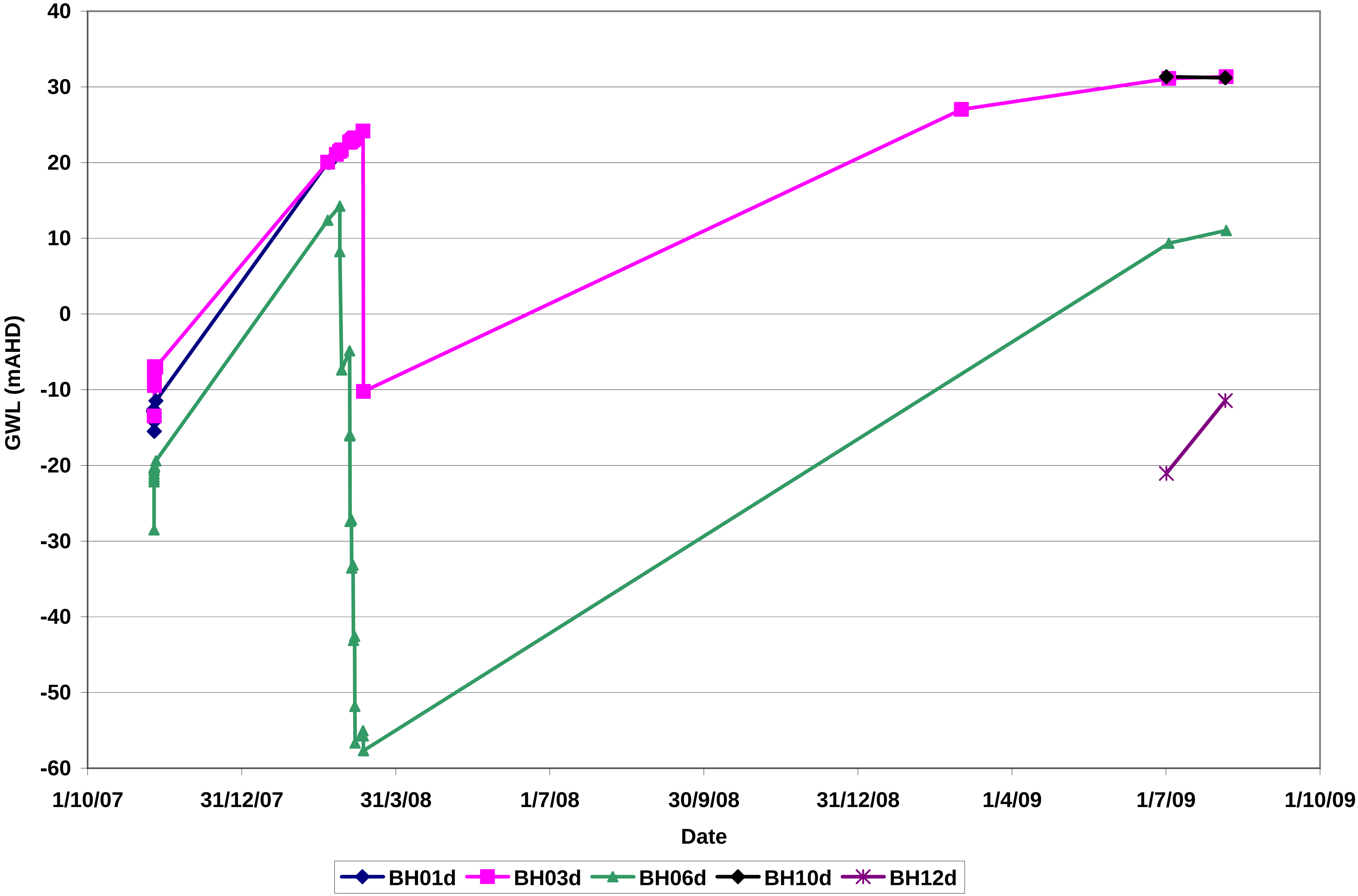

FIGURE 6.2a: Groundwater Levels - Deep Bores

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

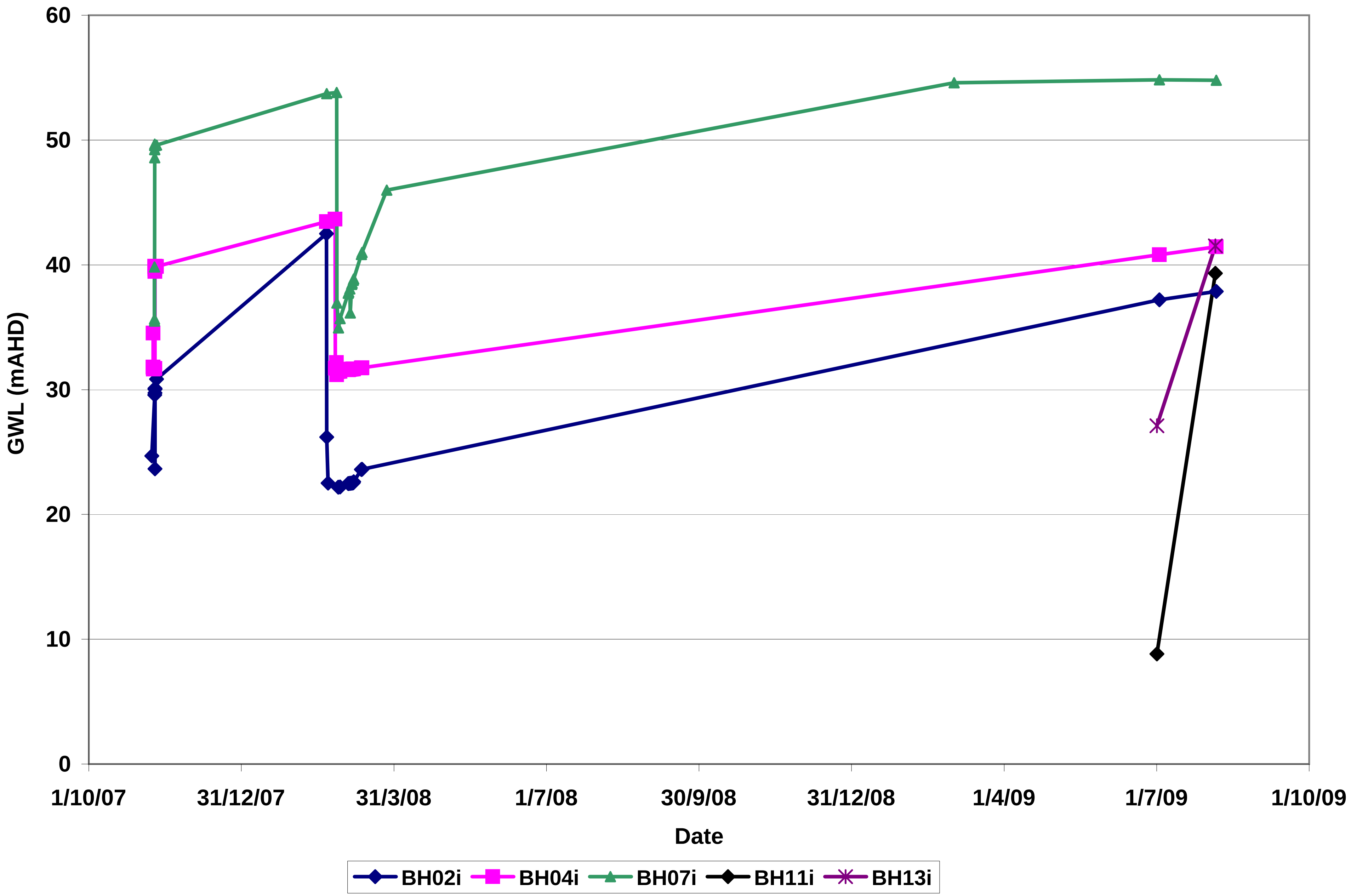

FIGURE 6.2b: Groundwater Levels - Intermediate Bores

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

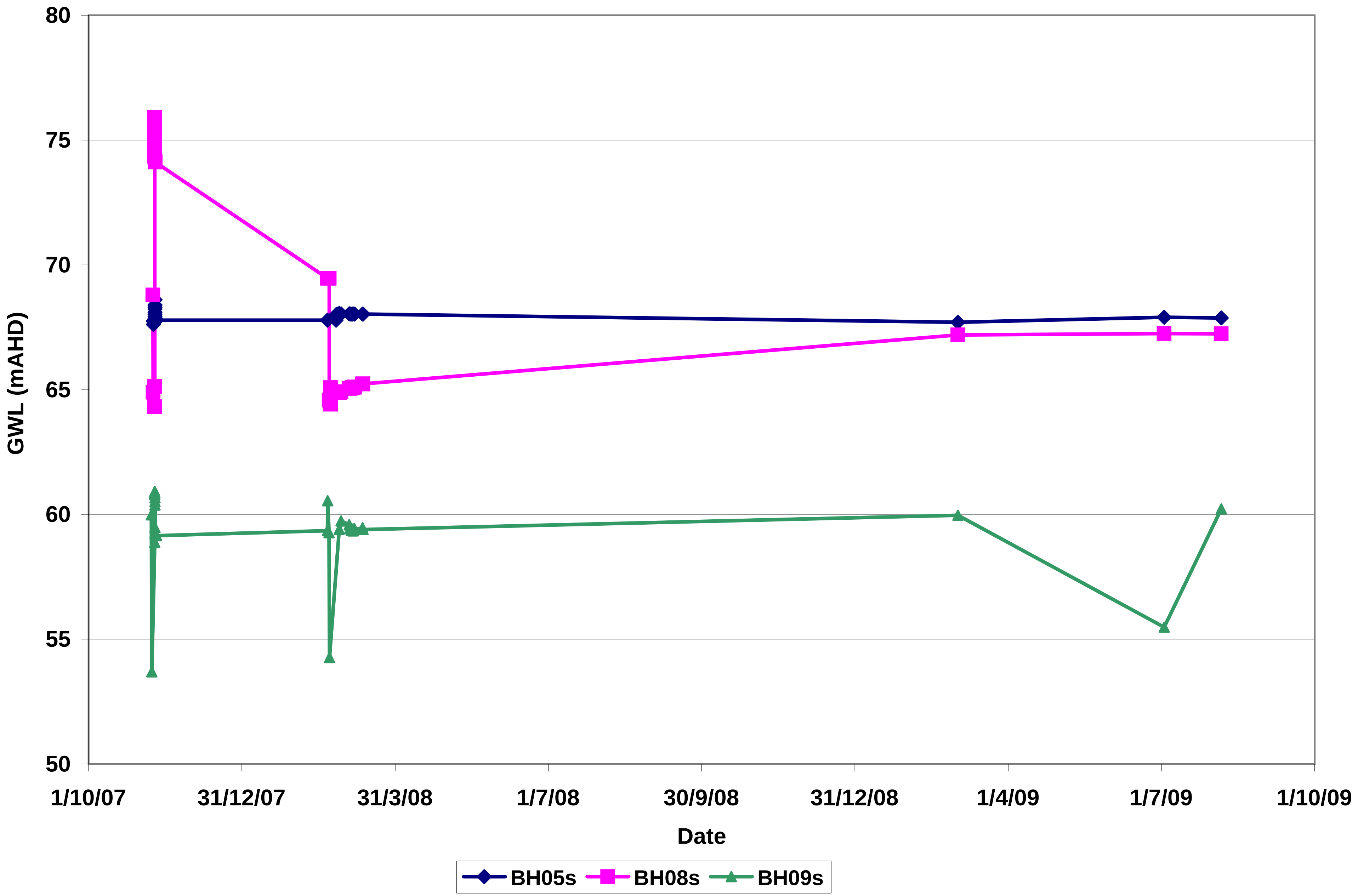

FIGURE 6.2c: Groundwater Levels - Shallow Bores

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

IGGC

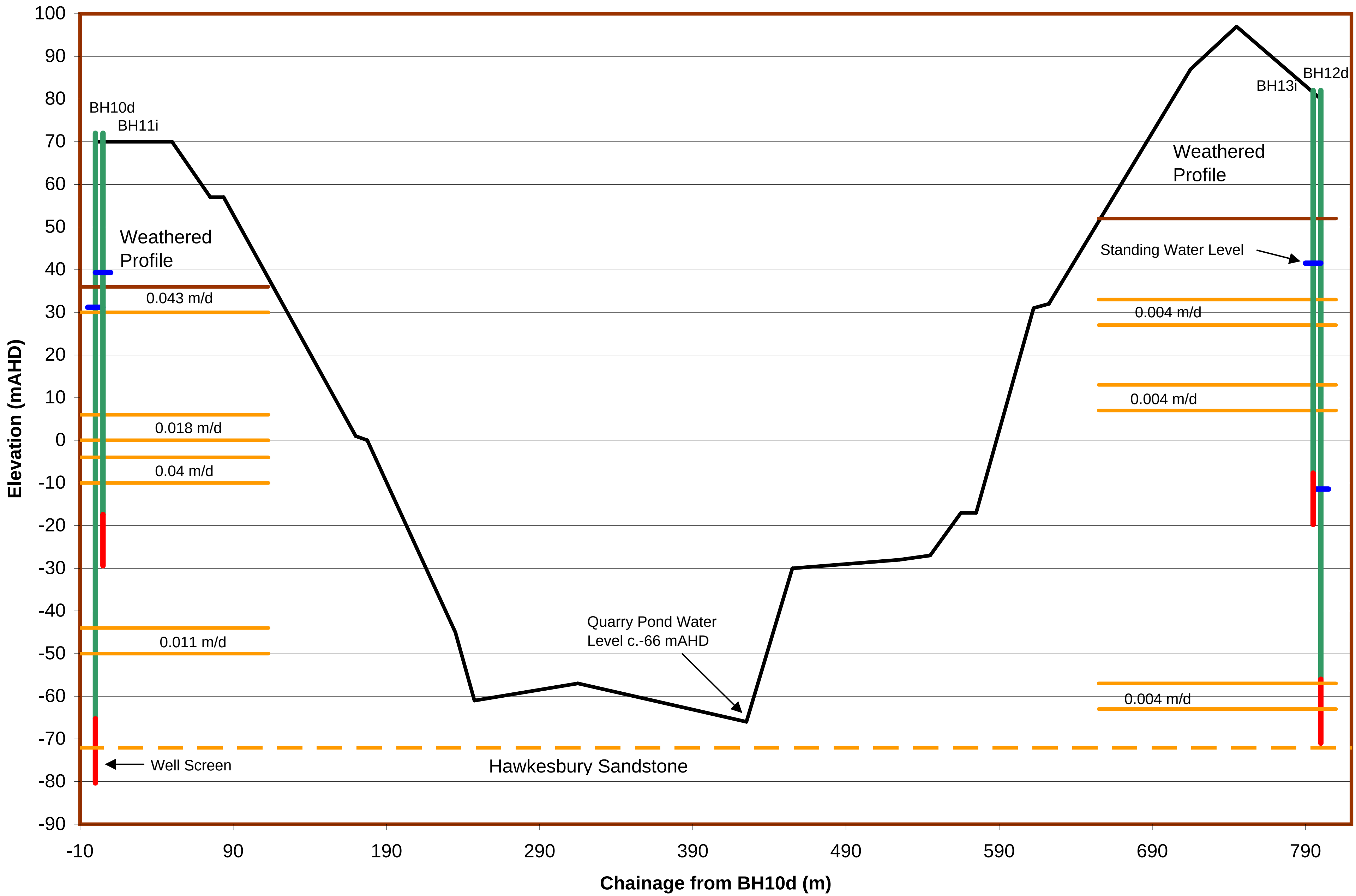

FIGURE 6.3: Detailed Hydrogeological Cross Section

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

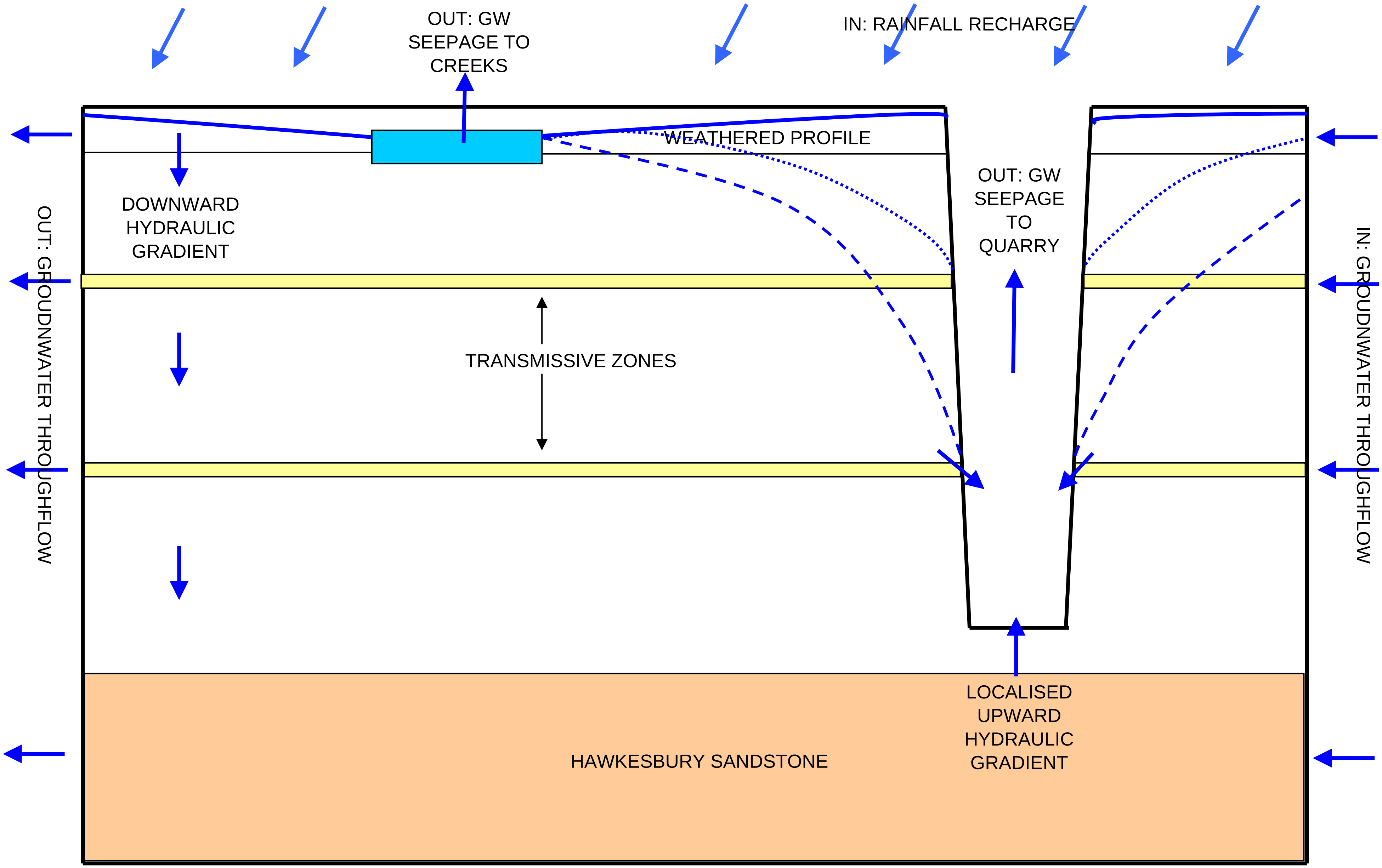

FIGURE 7.1: Conceptual Hydraulic Model

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

IGGC

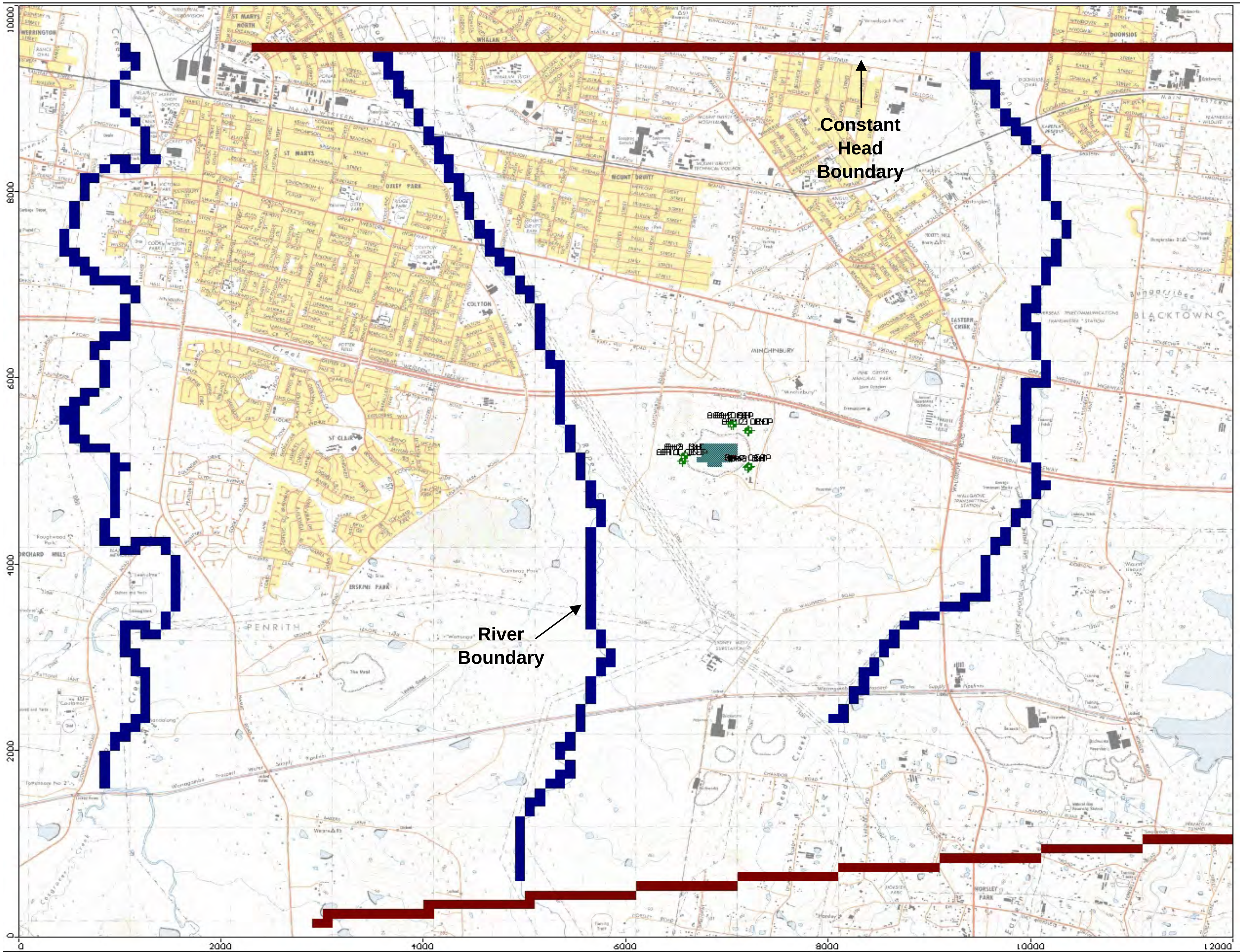

FIGURE 7.2: Model Extent and Boundaries

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

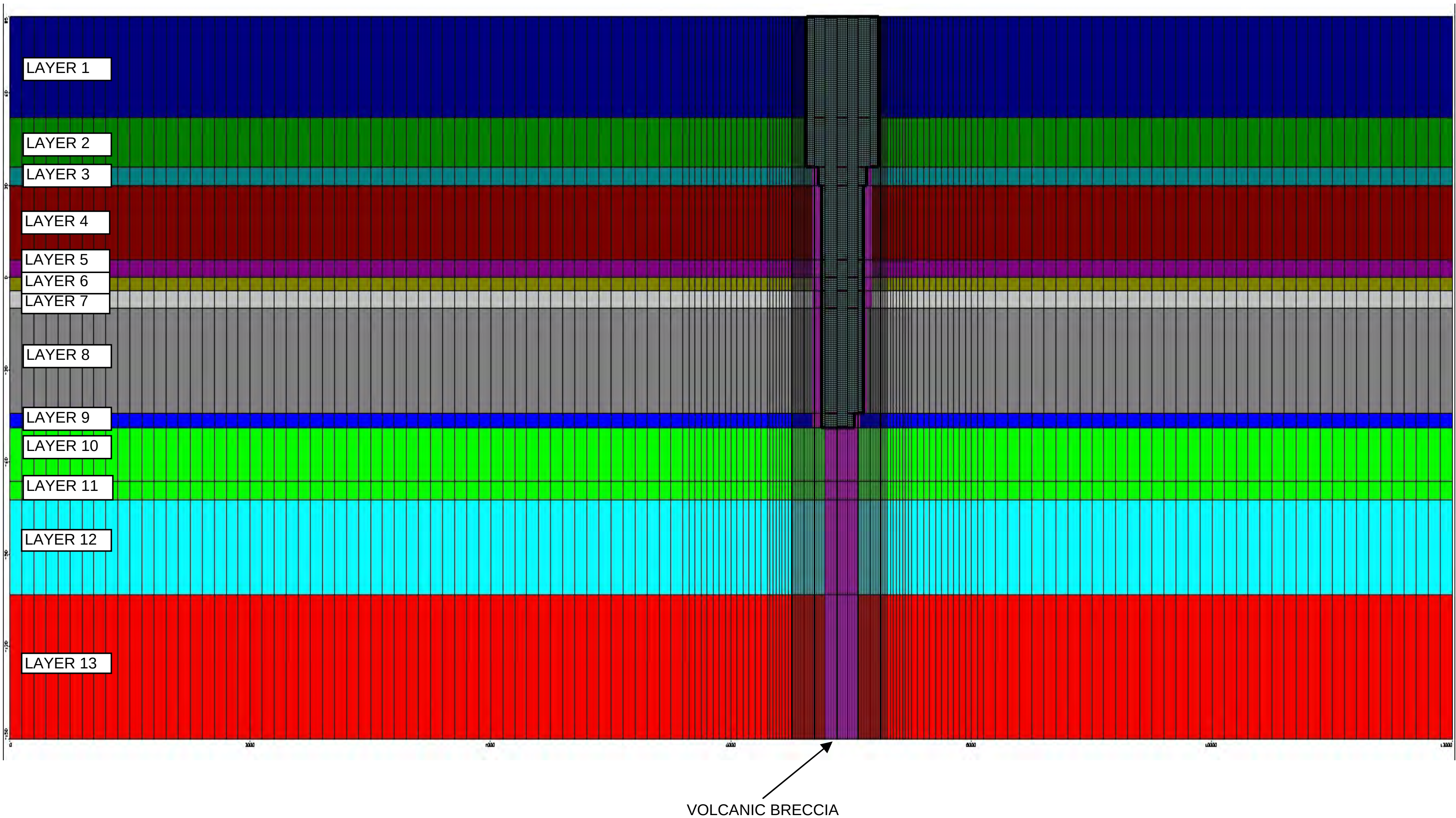

FIGURE 7.3: Model Geological Profile

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

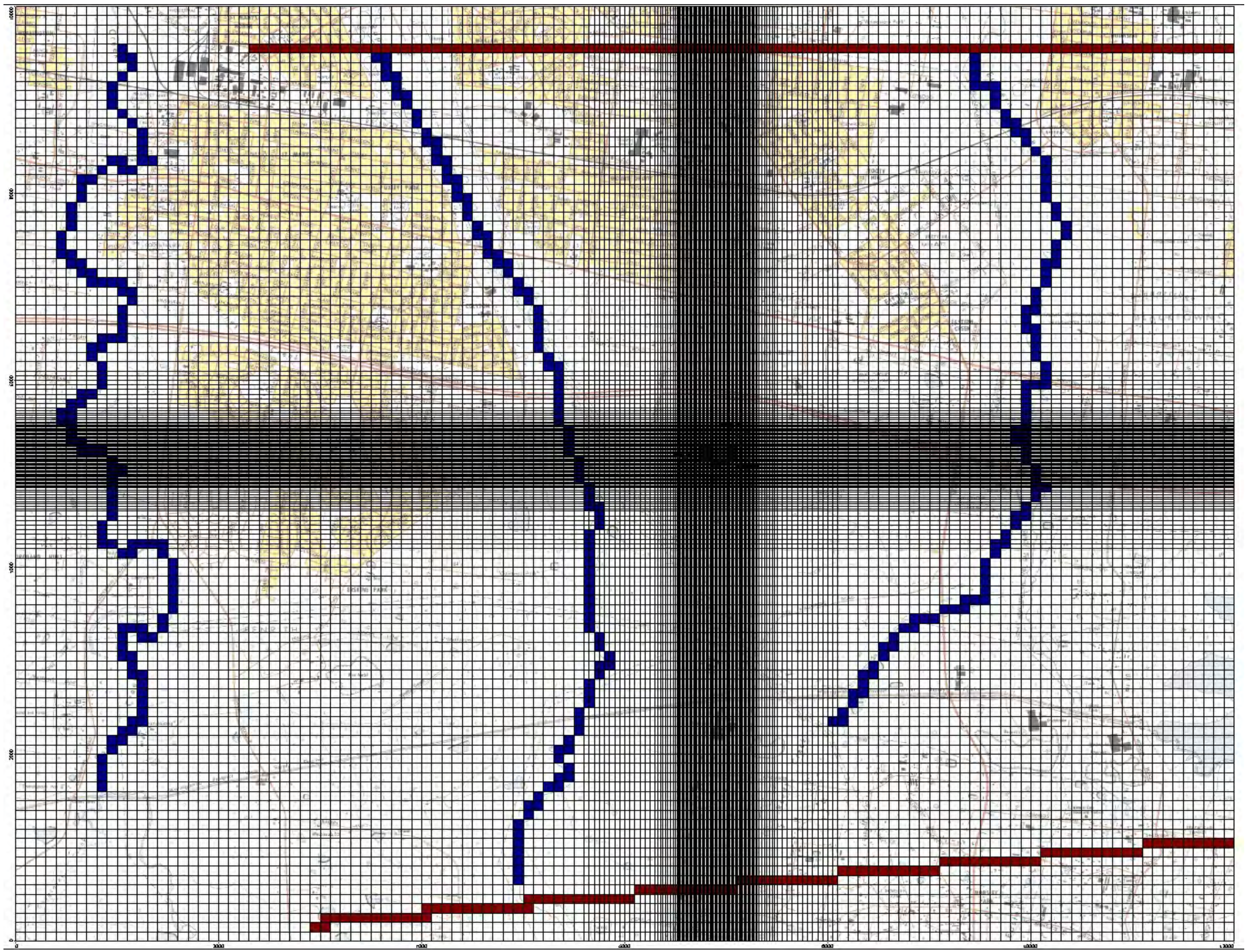

FIGURE 7.4a: Model Grid - Entire Model

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

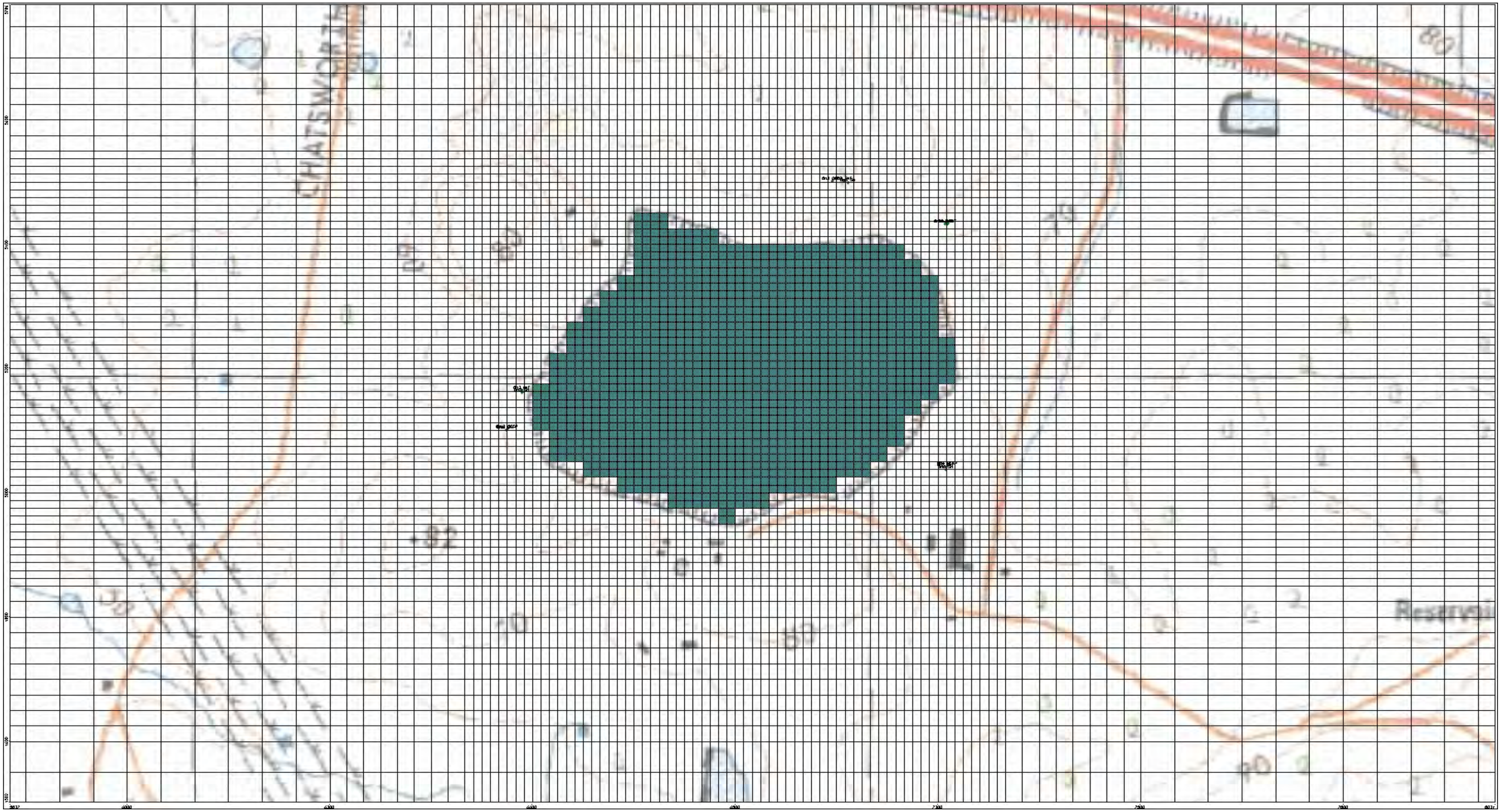

FIGURE 7.4b: Model Grid - Quarry Area

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

IGGC

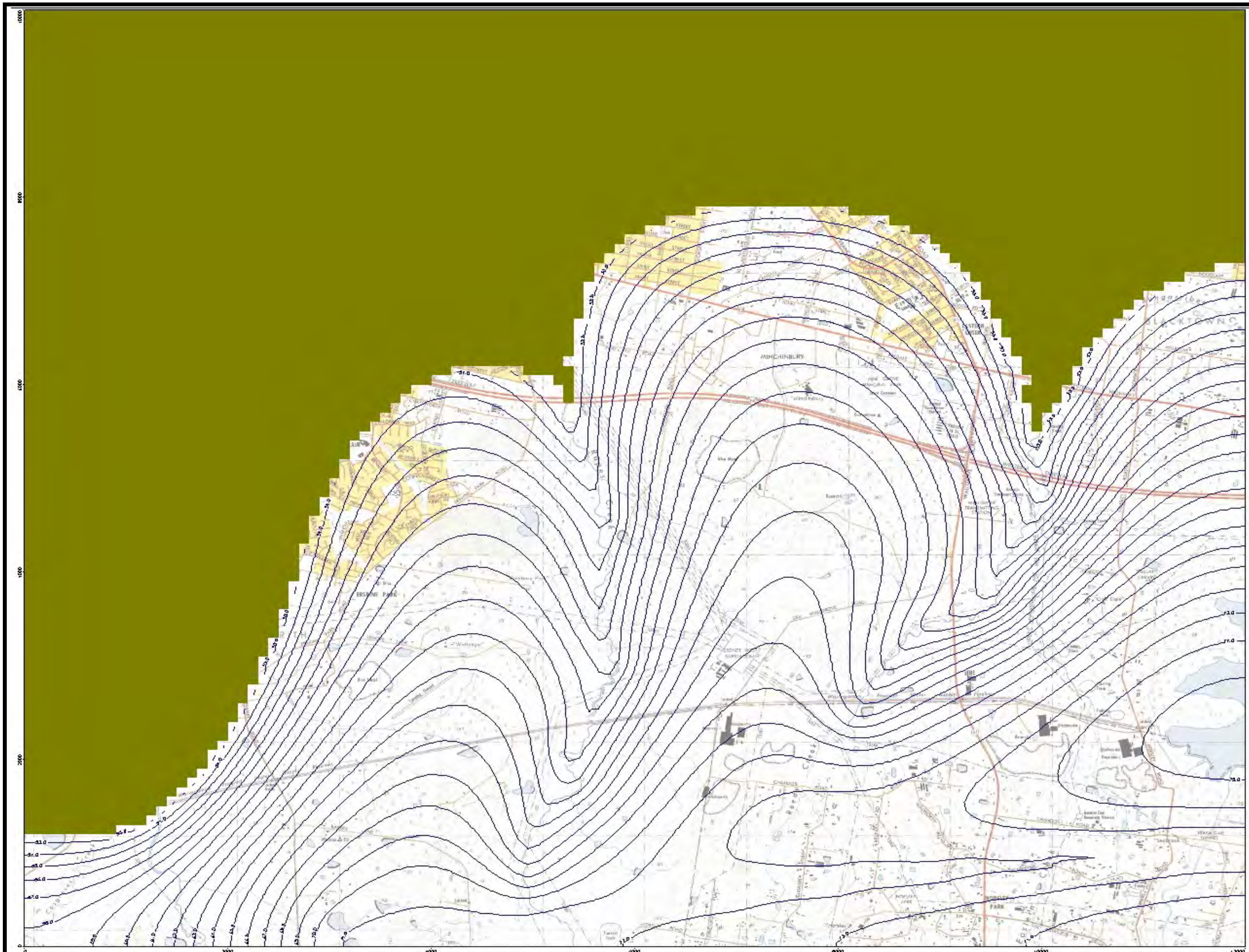

FIGURE 7.5: Simulated Shallow Groundwater Contours, Pre-Quarry Conditions (1 m Intervals)

Project: Detailed Hydrogeological Investigation and Assessment

Location: Proposed Light Horse Landfill Site, Eastern Creek

Client: Dial A Dump Industries Pty Ltd

Project No: BJ07

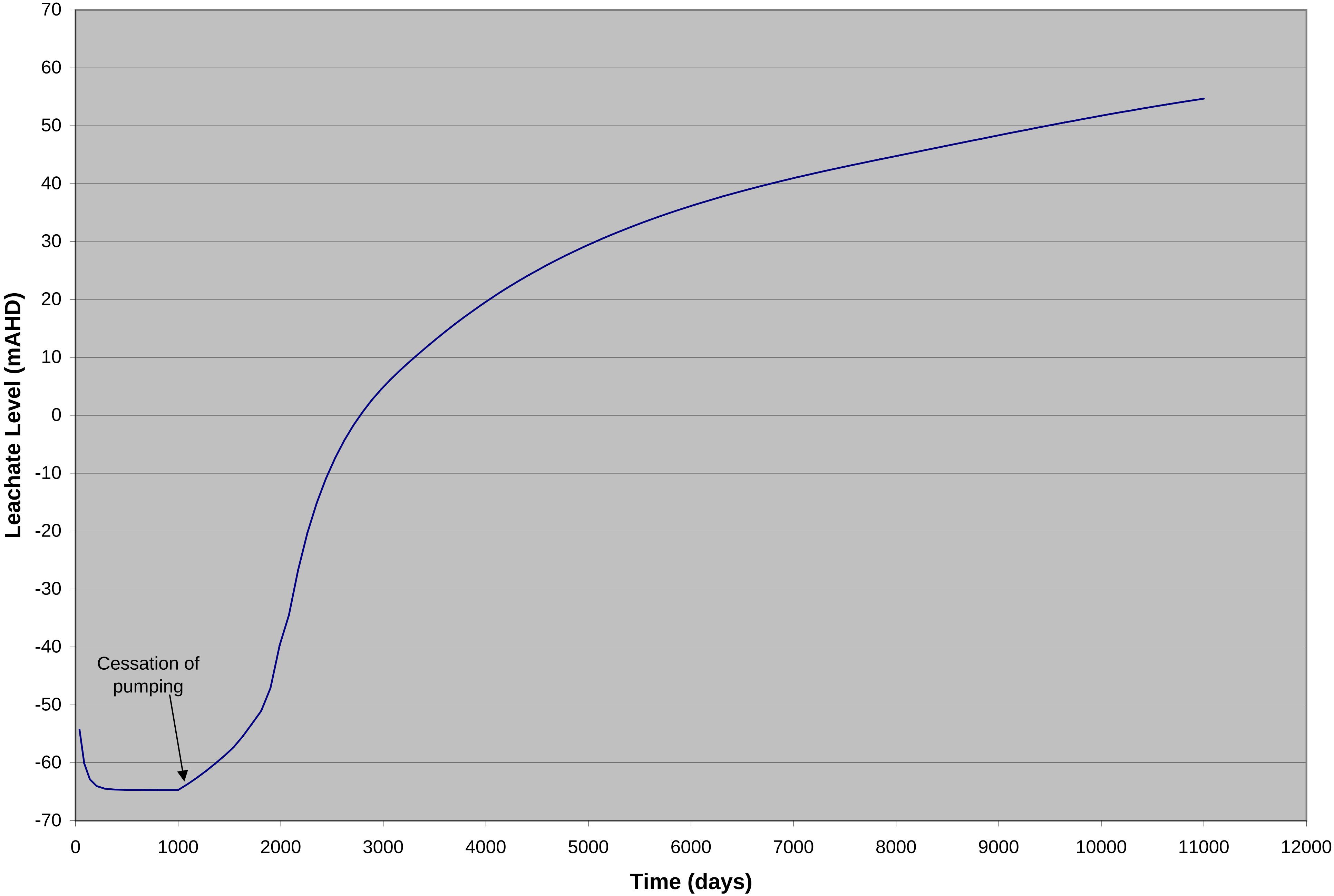

FIGURE 7.6: Predicted Leachate Level Rise, No Pumping

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

IGGC

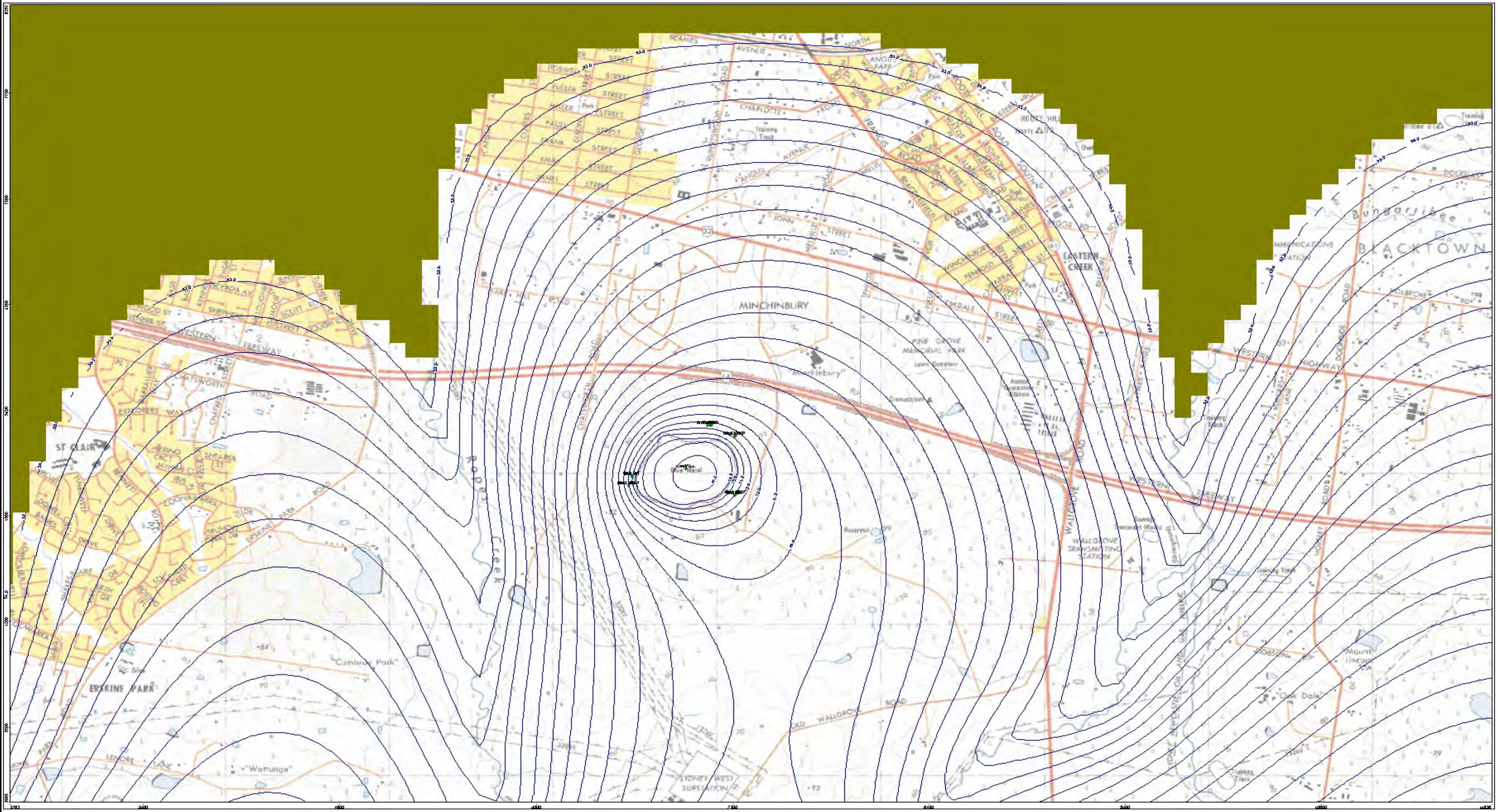

FIGURE 7.7: Predicted Groundwater Contours, Final Conditions (Full Repressurisation - 1m Intervals)

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

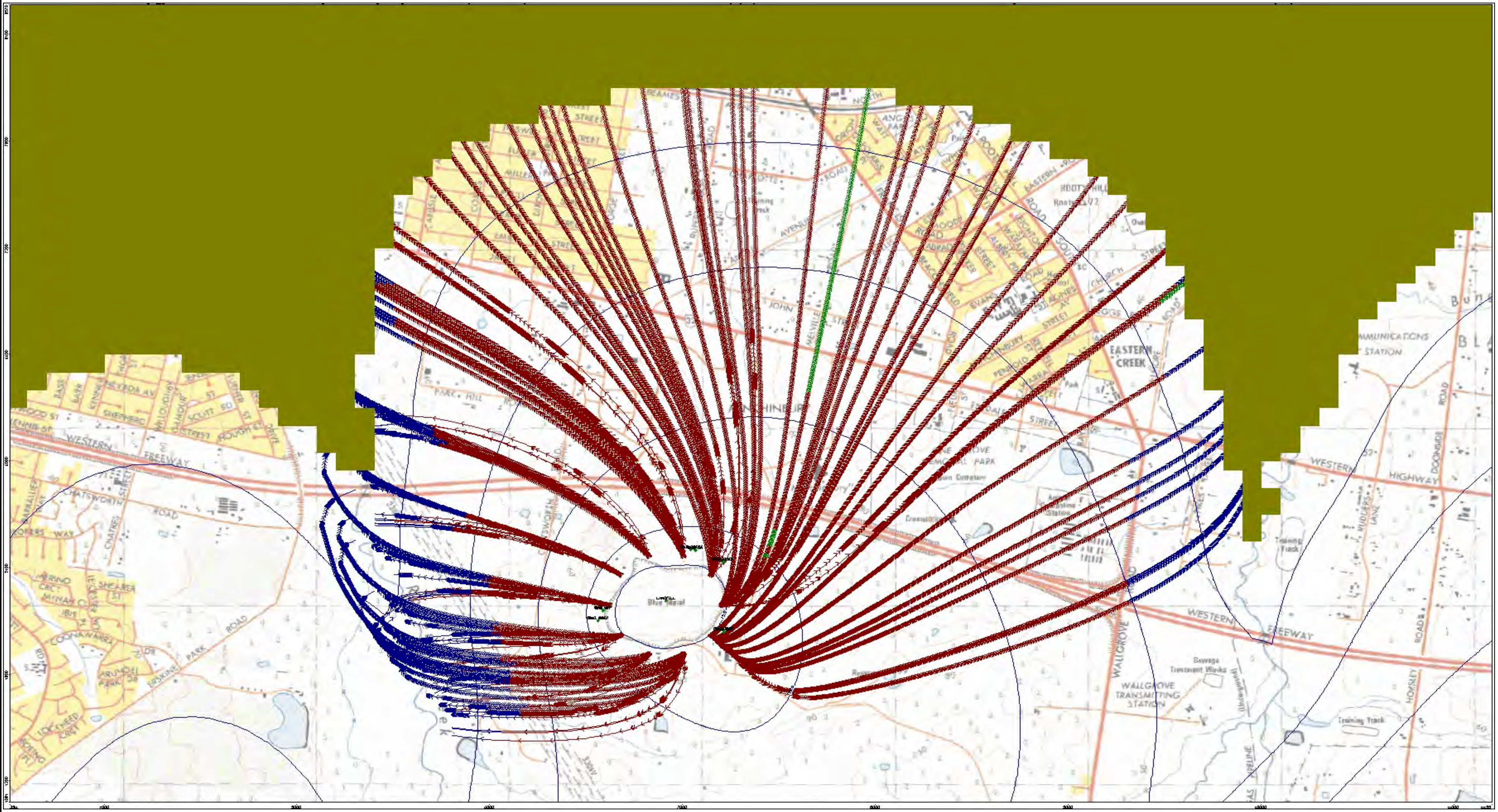

FIGURE 7.8: Particle Tracking Results, Full Repressurisation

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

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
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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**

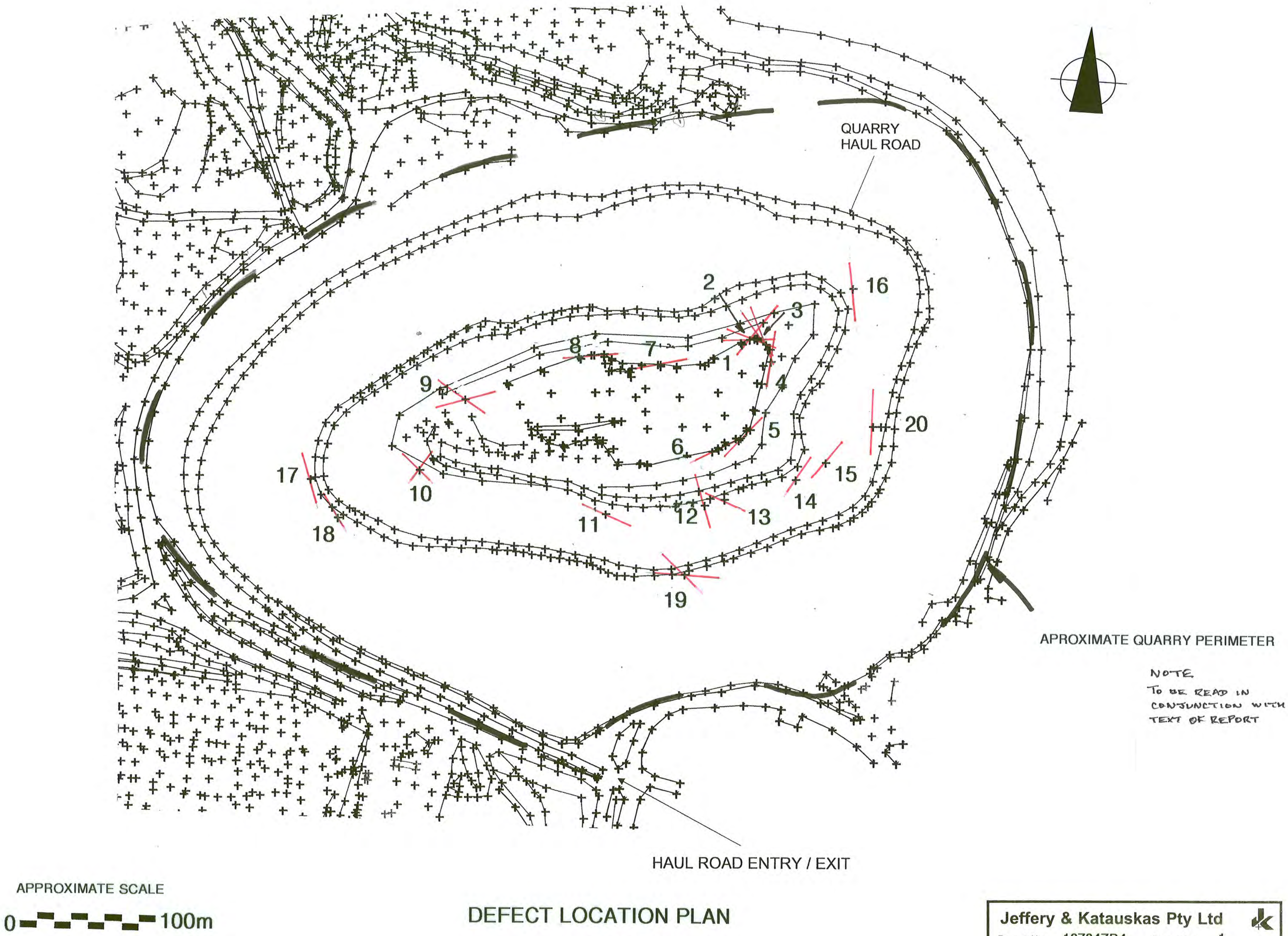
TABLE A SUMMARY OF RECORDED DEFECTS

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

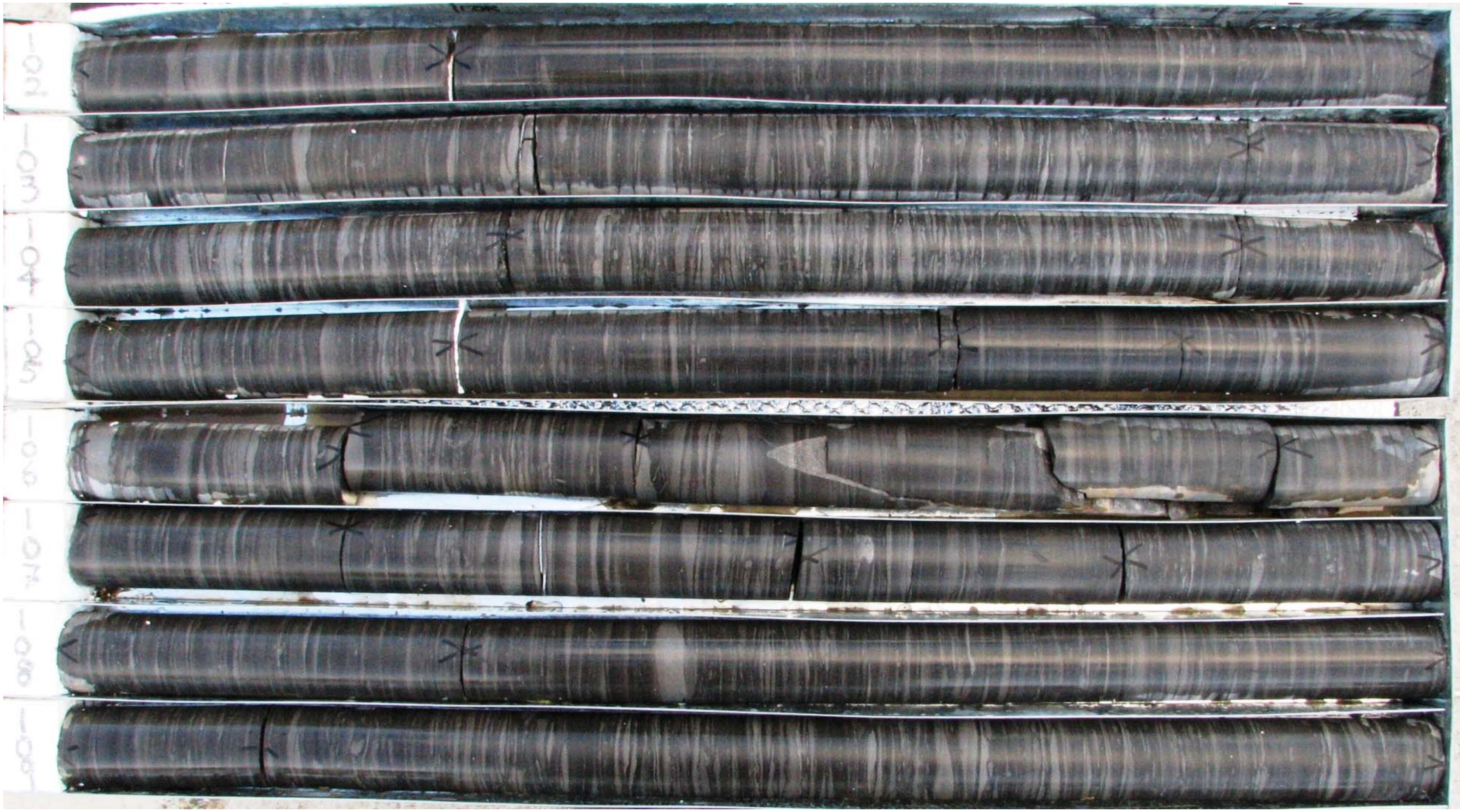
Core Hole 1 66-74 m core interval

Core Hole 1 74-78 m core interval

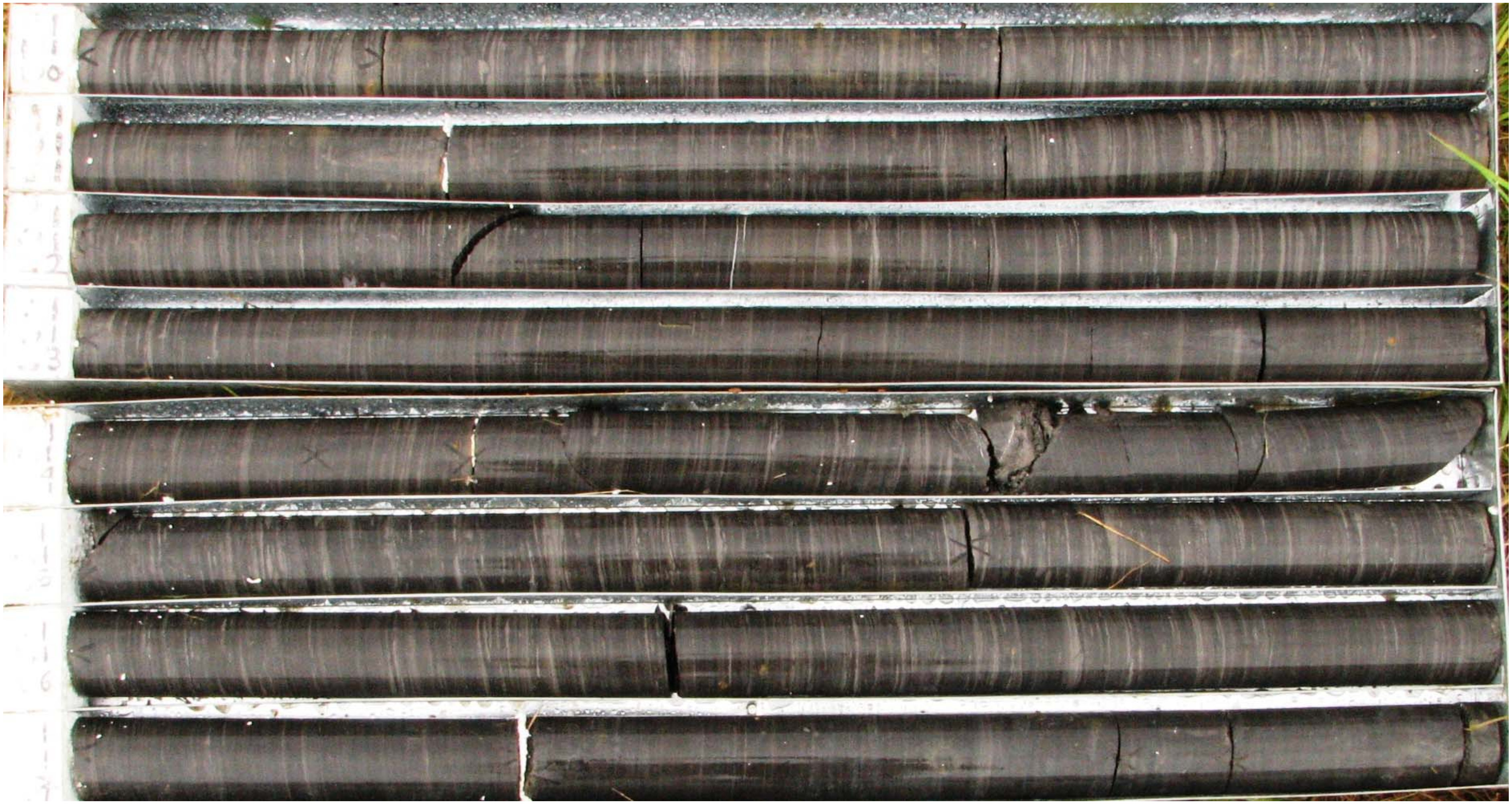
Core Hole 1 78-86 m core interval

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

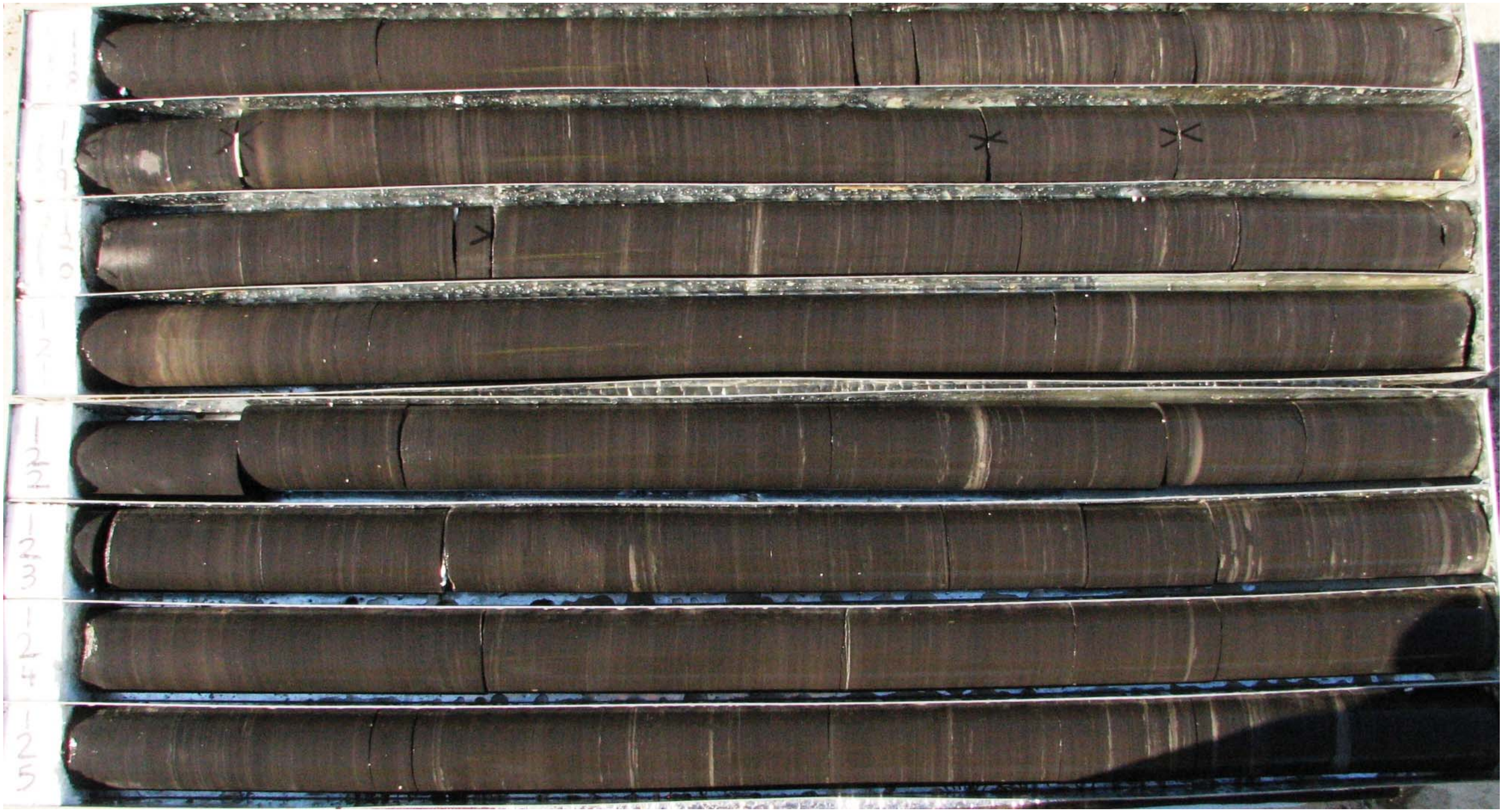
Core Hole 1 94-102 m core interval. The shale is separated by the sandstone unit ~94-96 m.



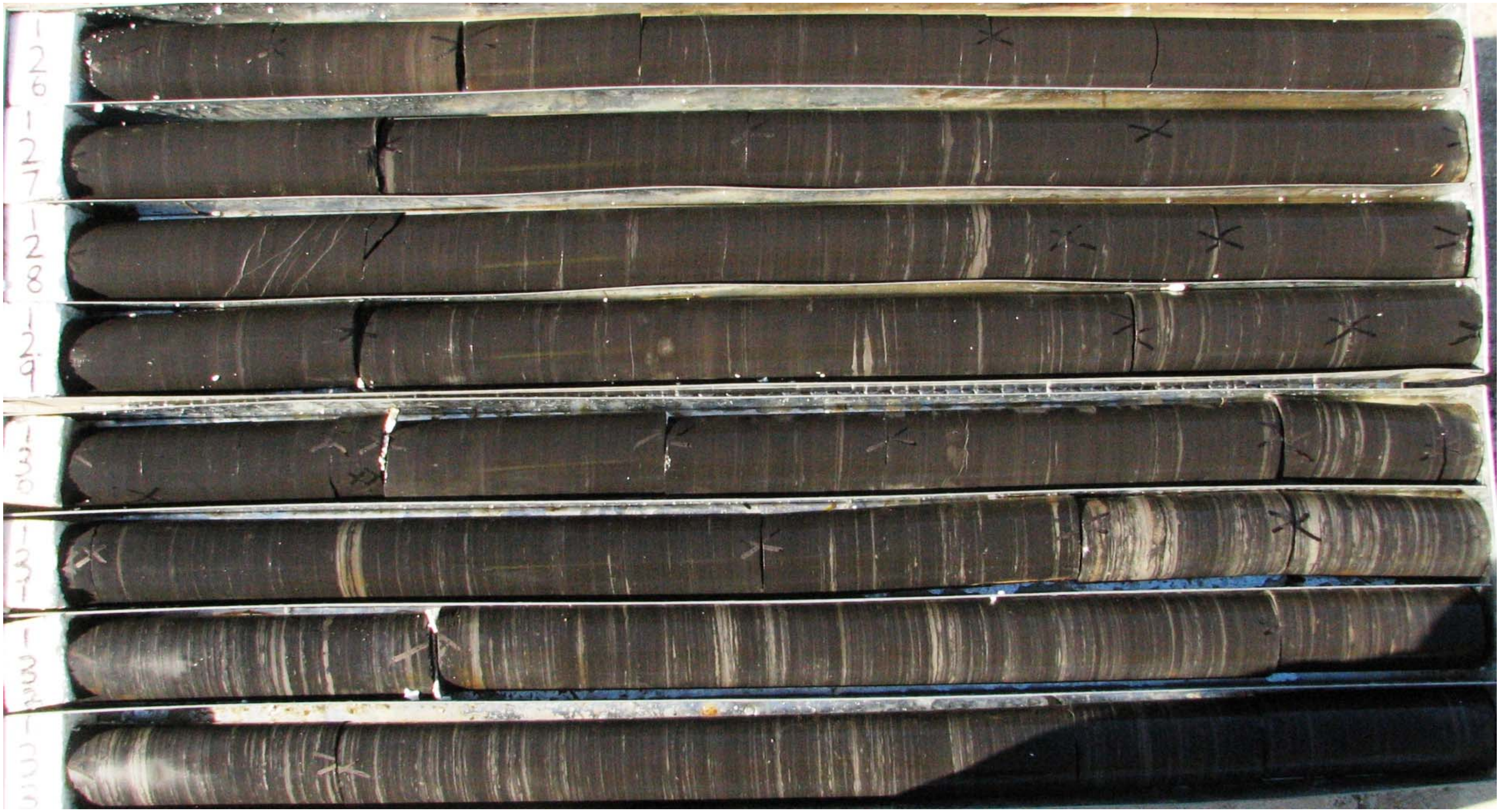
Core Hole 1 102-110 m core interval. Carbonaceous shale, continuing down hole.



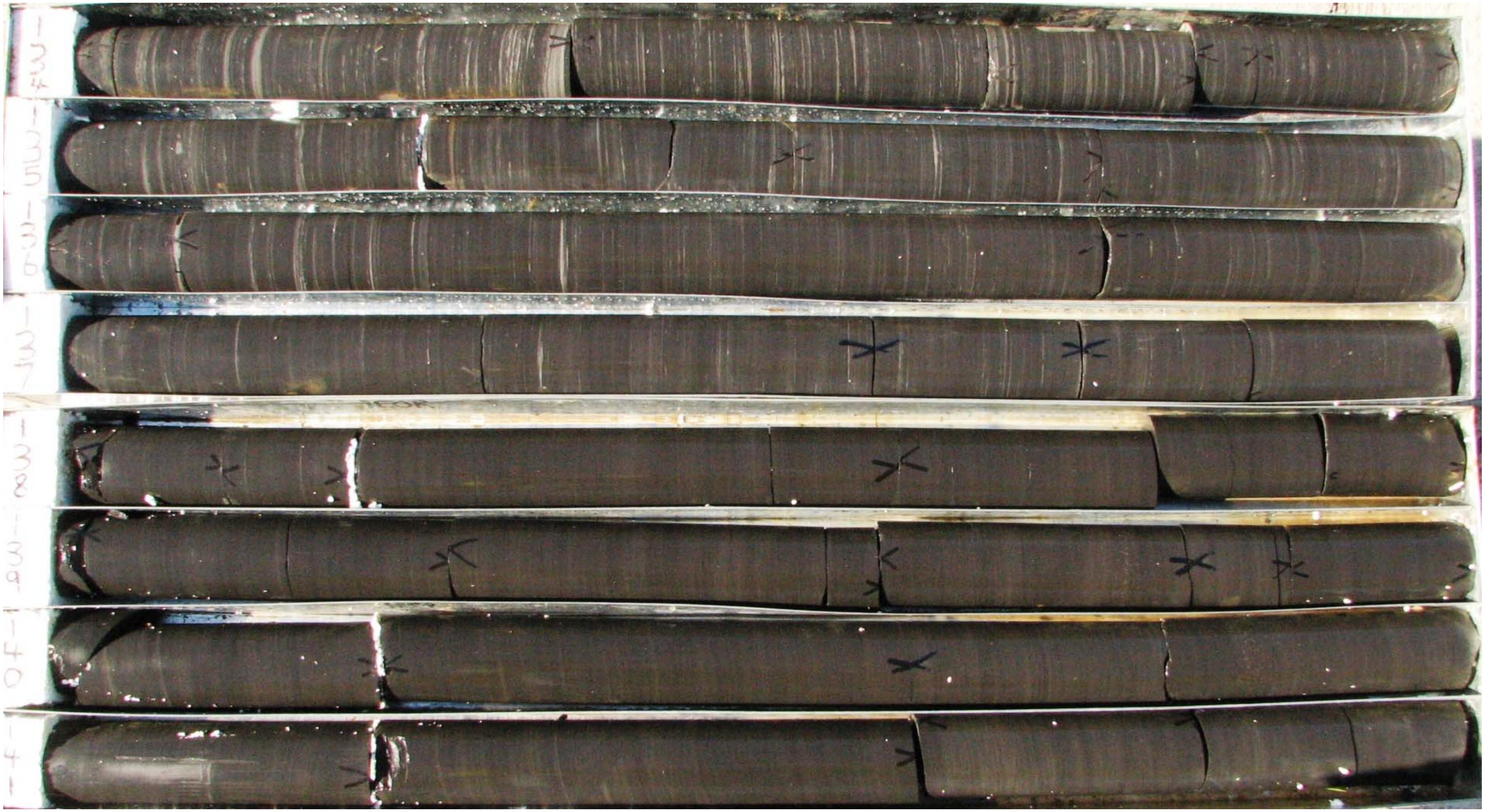
Core Hole 1 110-118 m core interval. Carbonaceous shale.



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



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



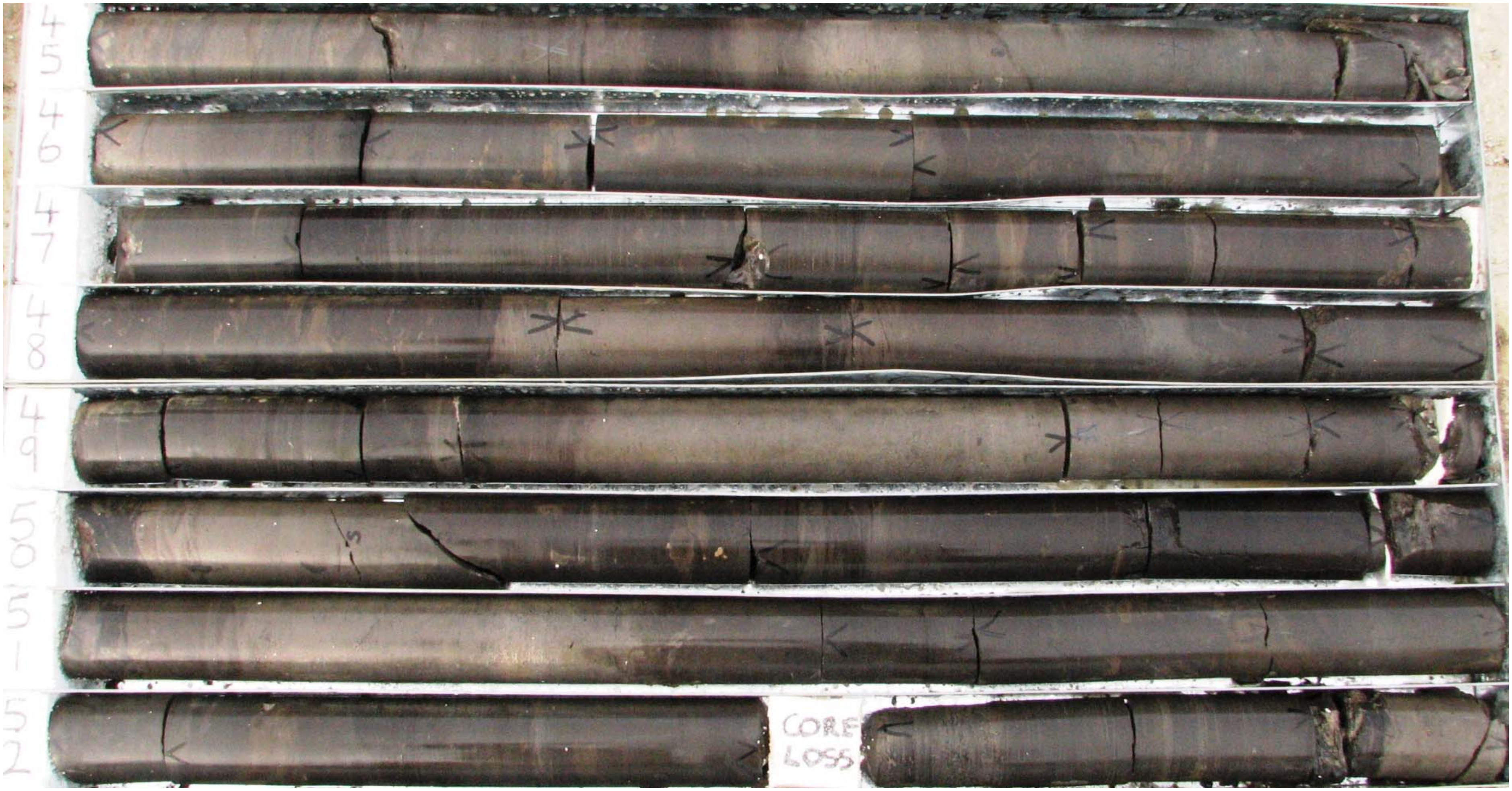
Core Hole 1 134-142 m core interval. Carbonaceous shale.

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

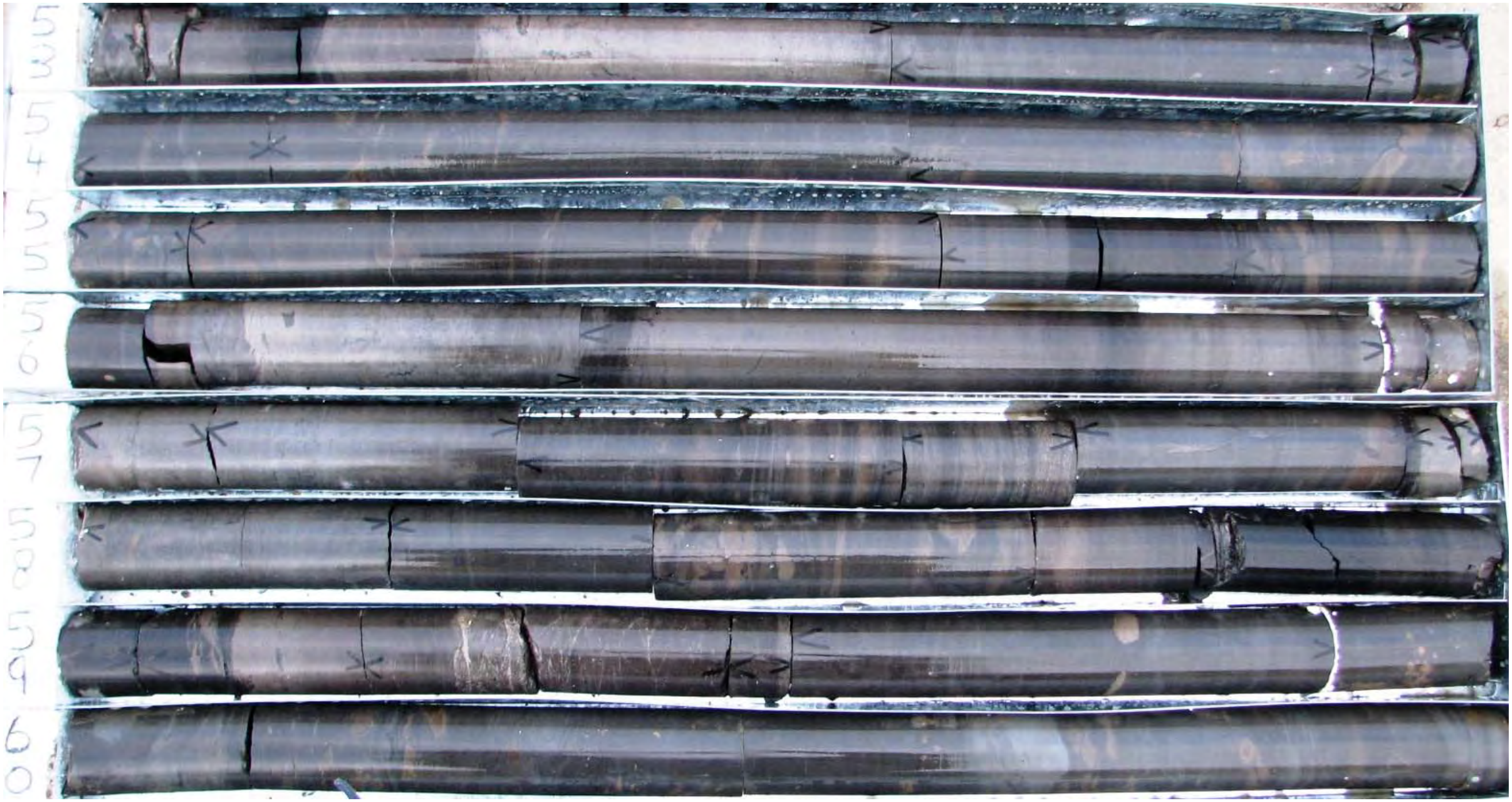
CORE HOLE 2

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.

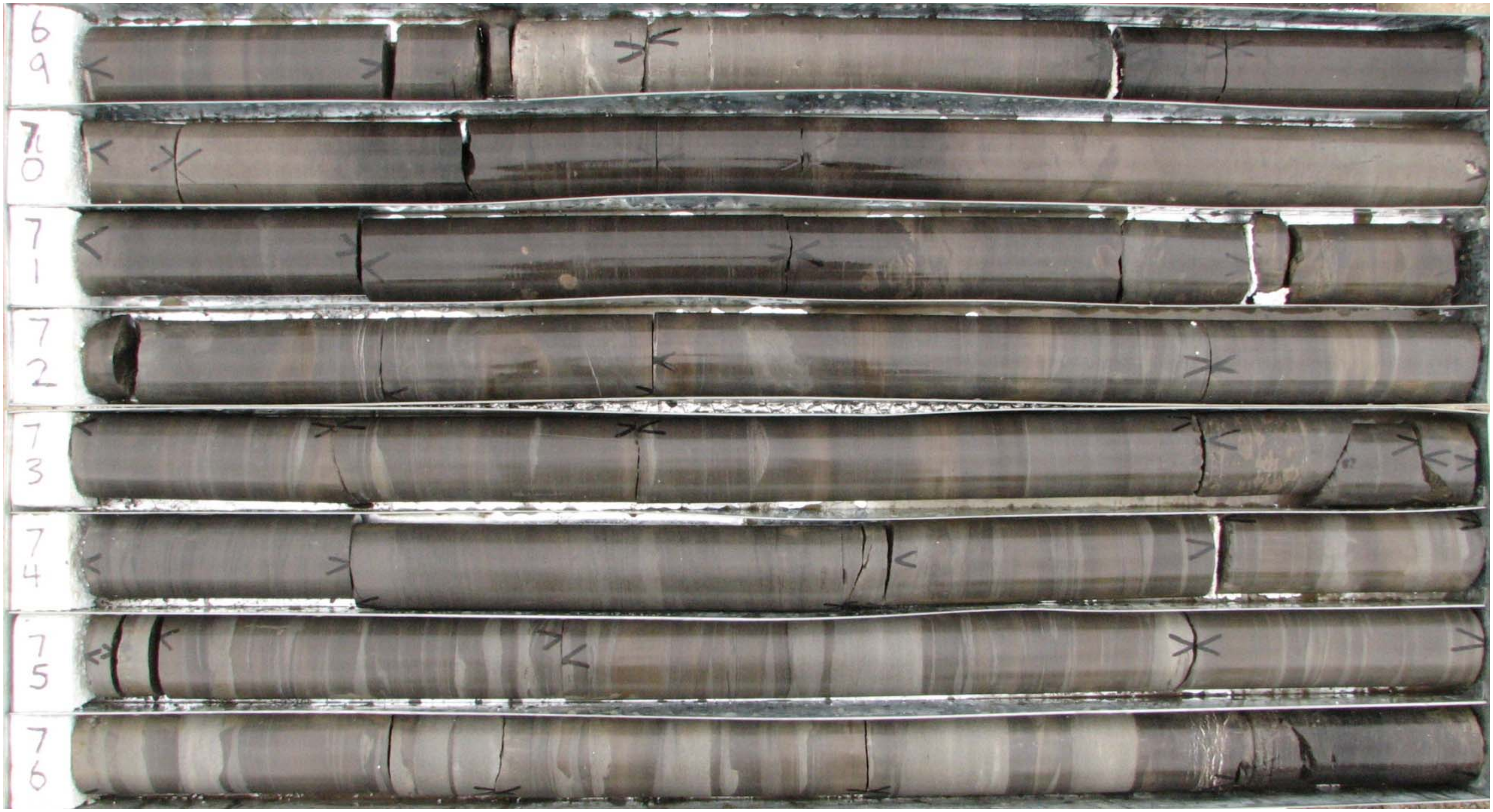


Core 2 45-53 m. Note the dark carbonaceous nature of much of the siltstone. Several fractures at 30-60 oLCA are noted around 50.2 m.



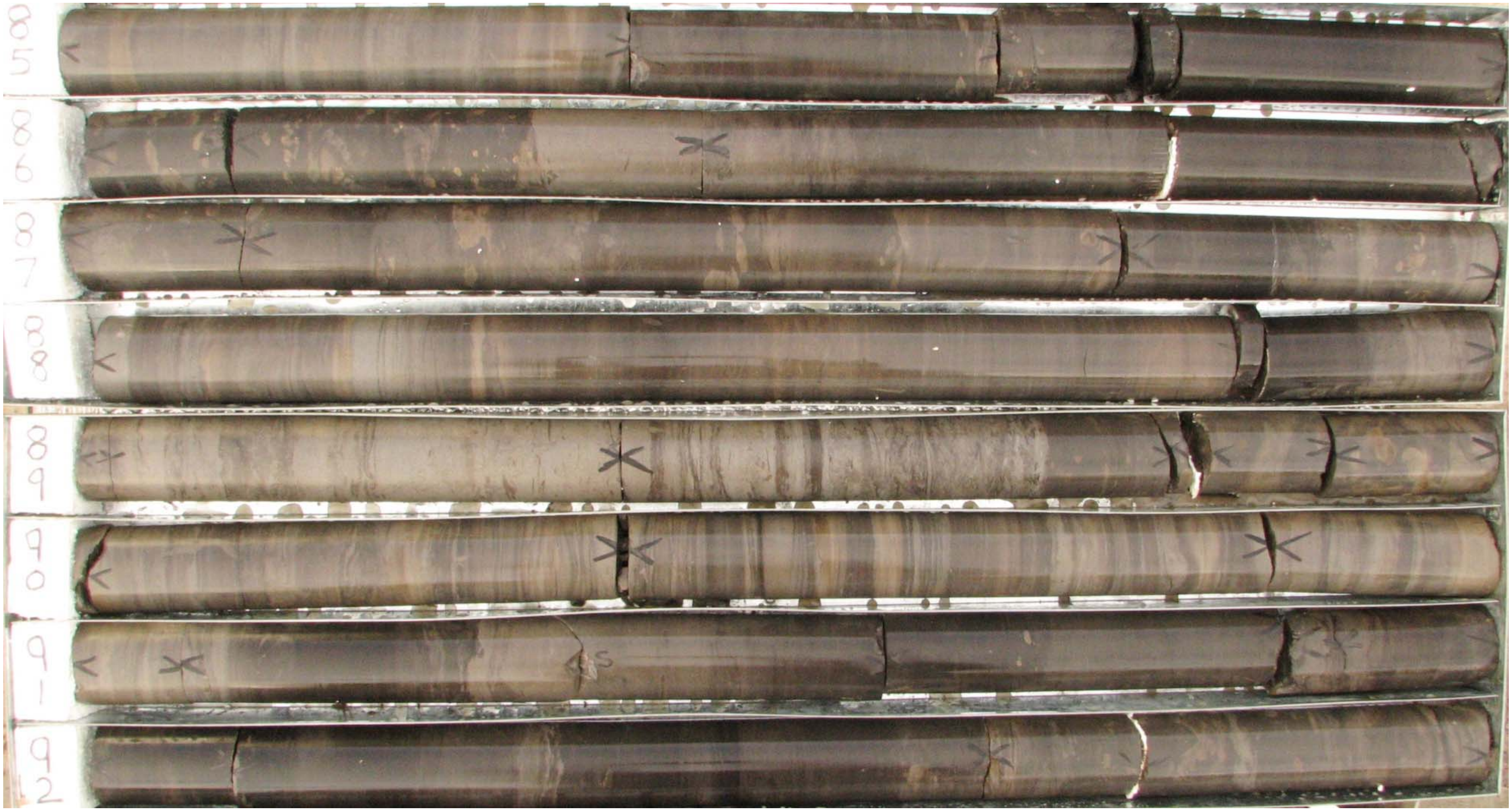
Core 2 53-61 Banded variably carbonaceous siltstone

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

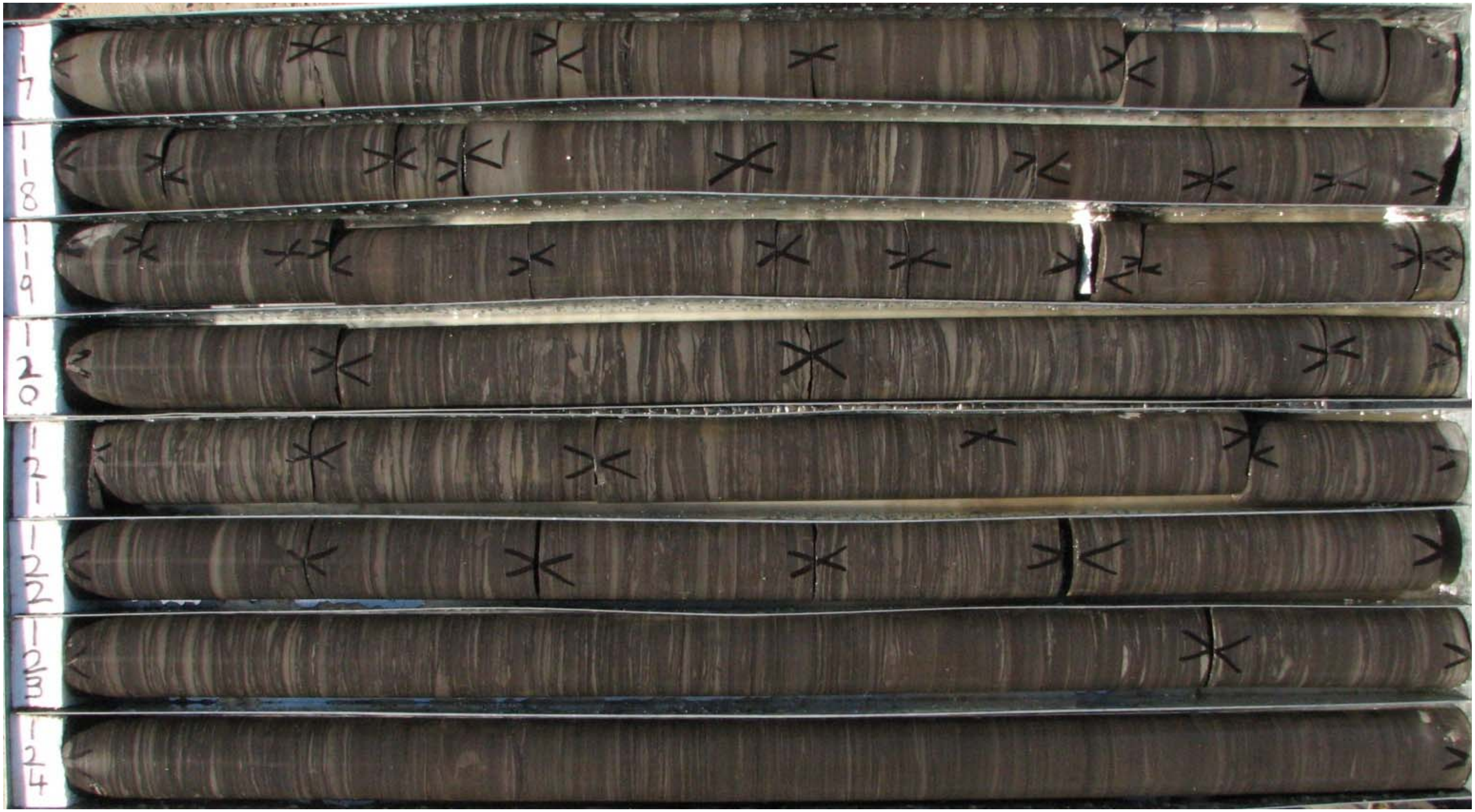


Core 2 85-93 m Note shear fracture marked with S

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								

IGGC

COMPOSITE WELL LOG

Well No: BH10d

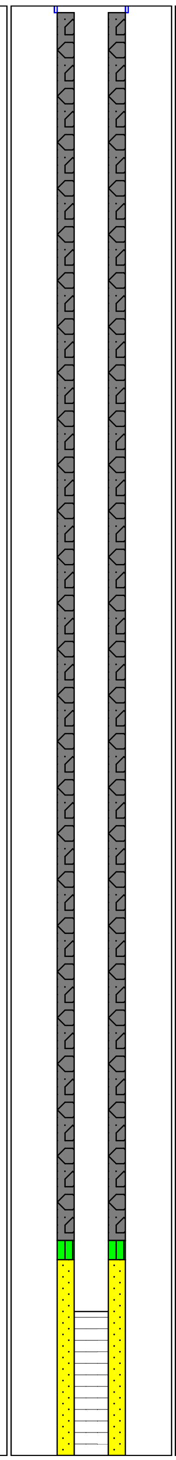
IGGC
PO Box 248
Newtown
NSW, 2042
Australia
Tel: (+61) (02) 9029 2995
Fax: (+61) (02) 9519 0905

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

Commenced:	12/05/2009	**Method:**	RC	**Area:**	Southwestern
Completed:	11/06/2009	**Fluid:**	Air	**East:**	298563
Drilled:	Terratest	**Bit Record:**	6"	**North:**	6258104
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: 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

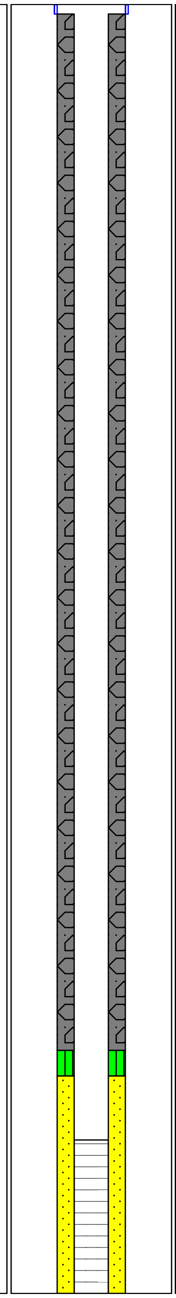
IGGC
PO Box 248
Newtown
NSW, 2042
Australia
Tel: (+61) (02) 9029 2995
Fax: (+61) (02) 9519 0905

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.			
-40						Grouted Annulus
-50						Class 18 PVC pipe
-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						



IGGC
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NSW, 2042
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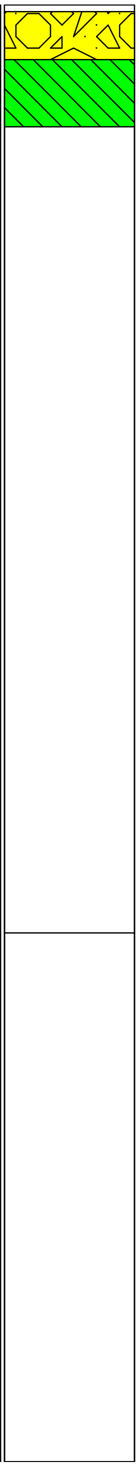
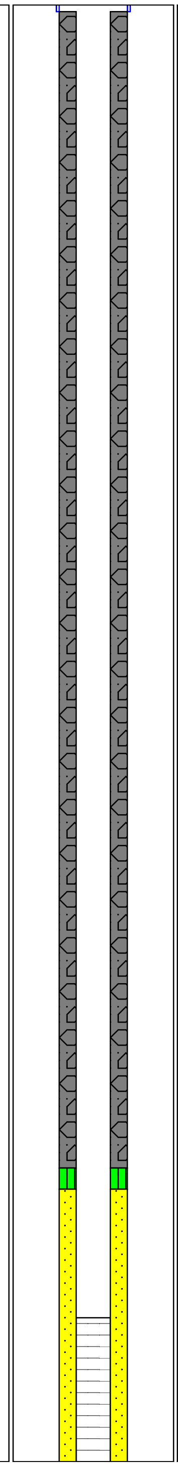
COMPOSITE WELL LOG

Well No: BH12d

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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COMPOSITE WELL LOG

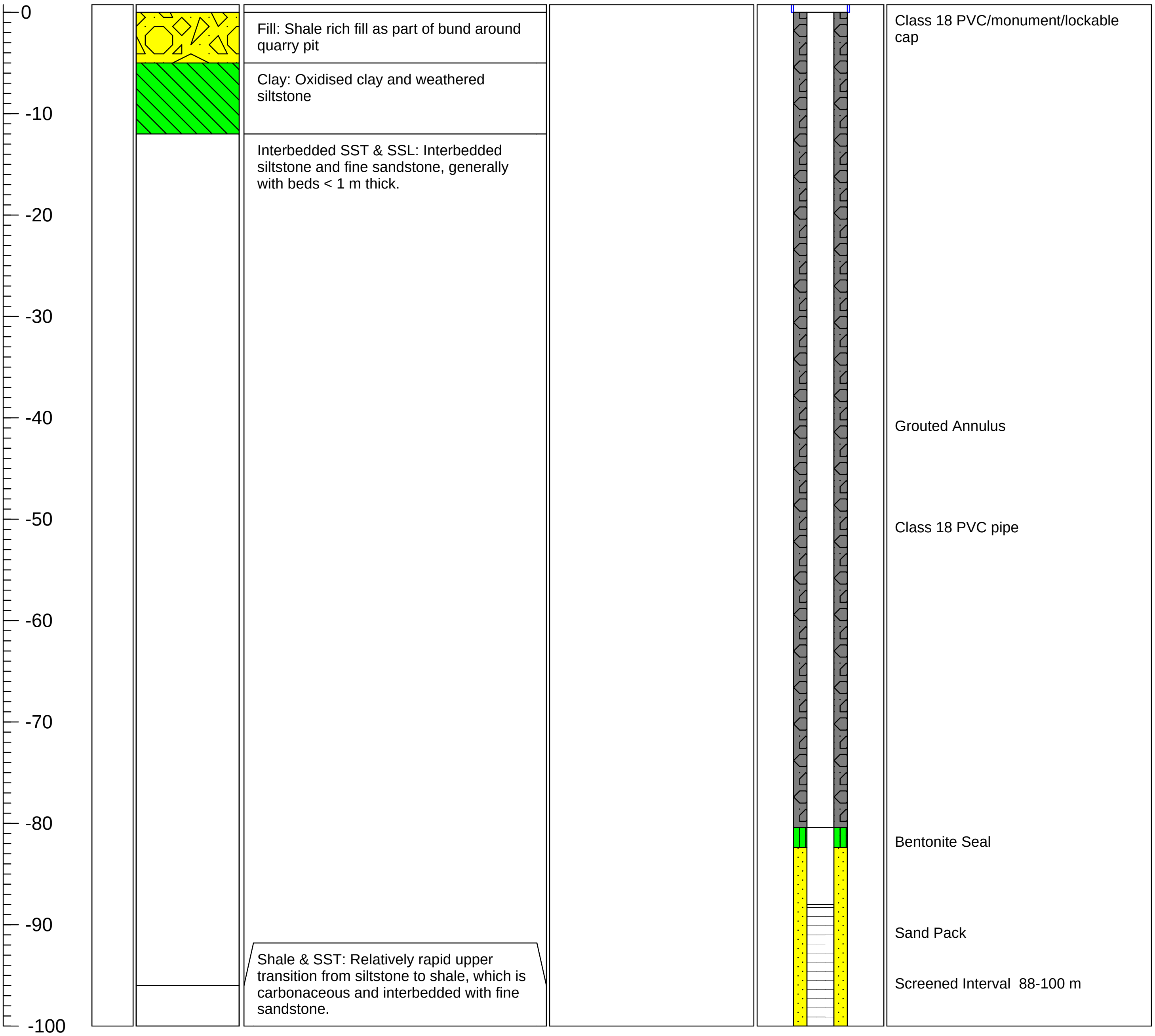
Well No: BH13i

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)	Lithological Description	Field Notes	Well Completion Notes
0			Class 18 PVC/monument/lockable cap
0–5	Fill: Shale rich fill as part of bund around quarry pit		
5–12	Clay: Oxidised clay and weathered siltstone		
12–96	Interbedded SST & SSL: Interbedded siltstone and fine sandstone, generally with beds < 1 m thick.		Grouted Annulus (to ~81 m); Class 18 PVC pipe
~81			Bentonite Seal
~83–100			Sand Pack
88–100			Screened Interval 88-100 m
96–100	Shale & SST: Relatively rapid upper transition from siltstone to shale, which is carbonaceous and interbedded with fine sandstone.		



Appendix D
Packer Testing Photographs

LIGHTHORSE LANDFILL SITE PACKER TESTING AND PIEZOMETER INSTALLATION PHOTOGRAPHS

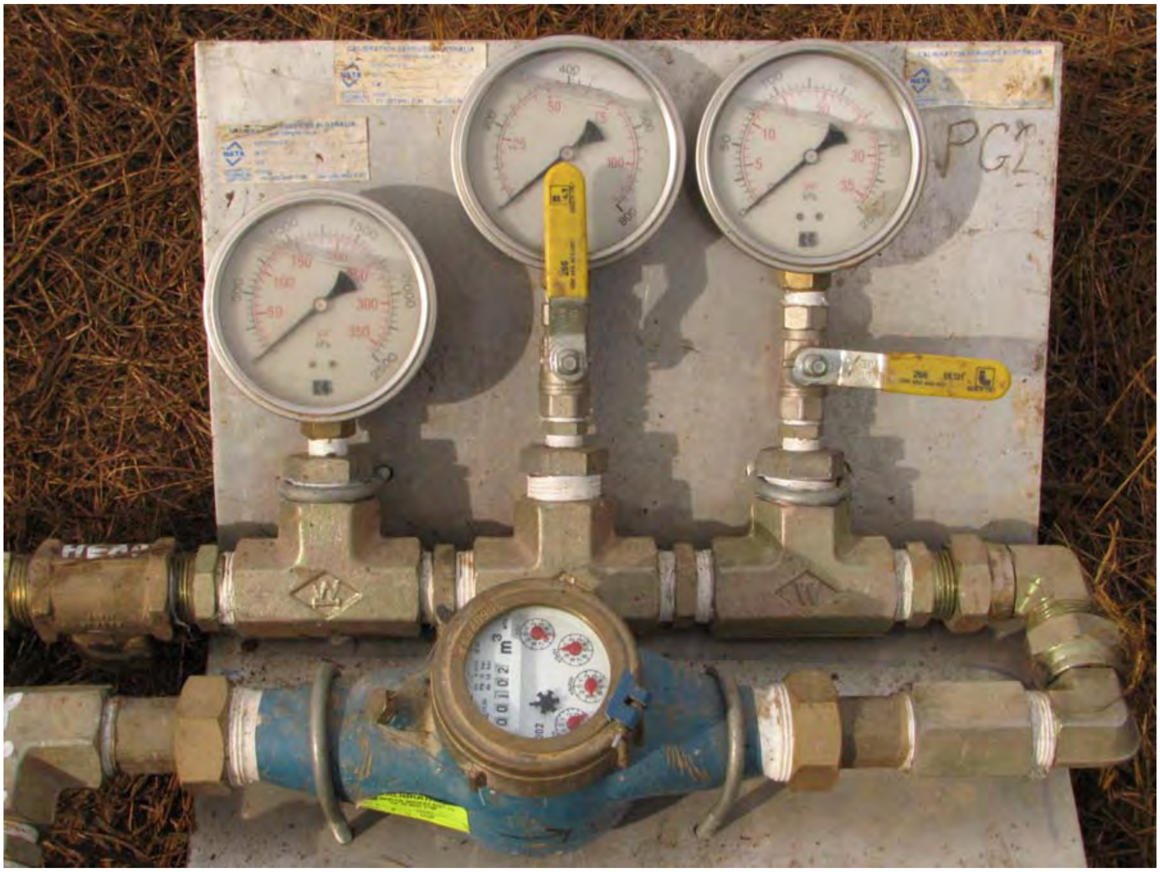
Flushing dirty water with cuttings into pits prior to packer testing

Packer on rack with pump and 200 litre water tank at rear

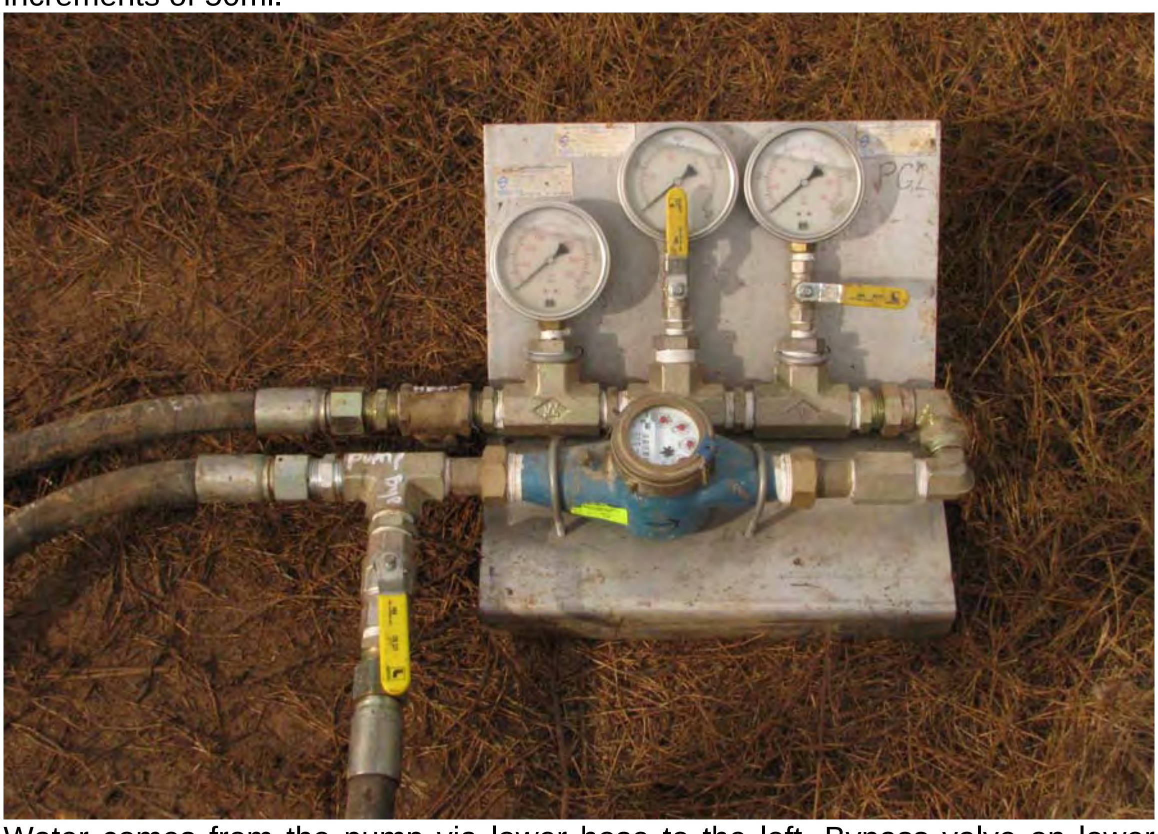
Compressed air source to inflate the packer down hole – packer inflation hose is visible on the reel

Feeding packer air hose down hole with the packer assembly. The air hose is taped to the cable suspending the packer.

Packer setup with packer suspended by cable and the compressed air inflation hose passing through the fitting at the top of the rods. Water under pressure from the pump is supplied via the hose fitting on the right.



Pressure gauges for measurement located 10 and 20 cm above ground level. Gauges are oil damped. The low metre in the foreground records water flow to increments of 50ml.



Water comes from the pump via lower hose to the left. Bypass valve on lower hose allows control of flow (diverting water back to tank). Water flows through the meter and out the upper hose to the rods and packer

The pump and drum used to hold clean water for packer testing, after the hole has been flushed with water from pits to remove silt that might prevent sealing of the packer.


Core holes where packer testing was conducted were reamed to a 150 mm diameter using an open hole hammer. The hole was then flushed prior to piezometer installation.


Drill cuttings from hammer drilling. Cuttings were collected on a metre basis and geologically logged for comparison with the core holes and to assess areas of groundwater inflow, together with the driller's observations.

Holes were flushed with water and with biodegradable foam, to remove remaining cuttings, prior to filling holes with water and installing the piezometers

Clean municipal supply water was used to fill drill holes prior to installation of piezometers.

Piezometer materials for installation. Class 19 screwed PVC was used, with plastic centralisers used every 9 to 12 metres to keep pipe central in the hole. A lifting sub was used to lower pipe into the hole.


DIAL-A-DUMP
PLANT HIRE
9519-9999
PLANT No. 8001

Appendix E
Packer Test Analyses

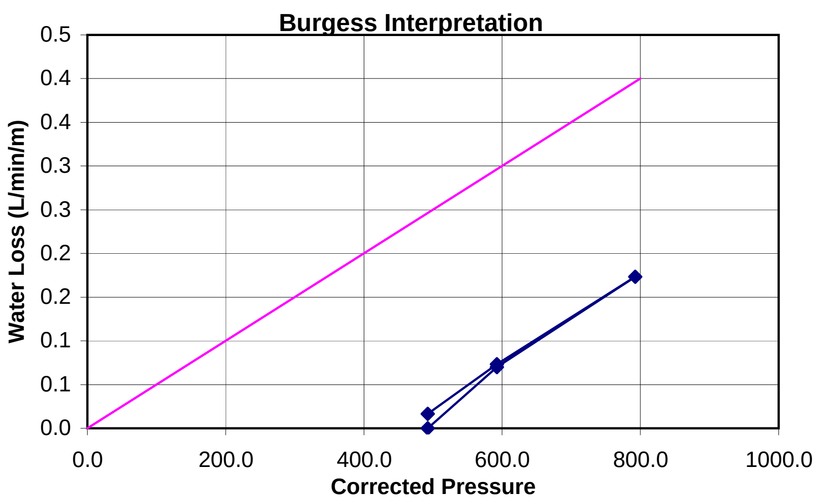
BOREHOLE WATER PRESSURE TEST

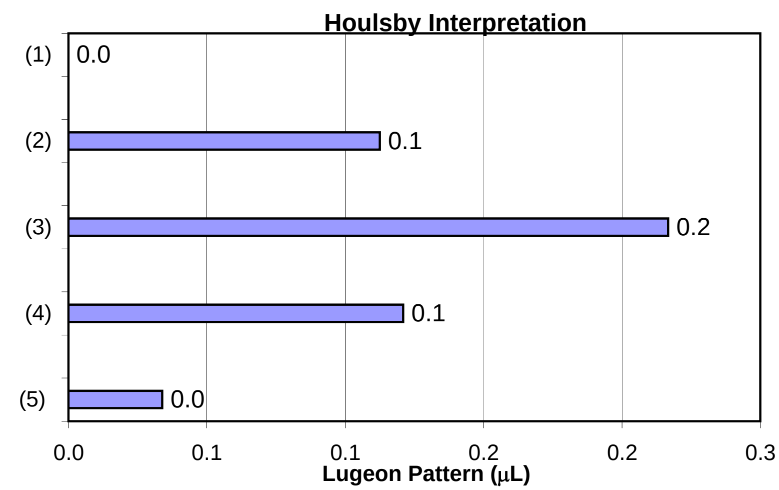
Job No: **BJ07**
Office: **Syd**

Client: **LHL** Location: **Eastern Creek** Reduced By: **IG** **Borehole no:**
Project: **Light Horse Landfill** Test Date: **13-May-09** Checked By: **MB** **BH10d**

Test Details: Packer Type: **End** Borehole Diameter: **100** Depth from: **34.00 m**
Type of Pump: Borehole Inclination: **90** Depth to: **40.00 m**
Pressure Gauge & Serial No: Borehole Azimuth: **0 0** Test Depth Interval: **6.00 m**
Pressure gauge height: **0 m** Depth to g'water: **40 m** Static Pressure: **392.4 kPa** Test Midpoint (m): **37.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter)			Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
		Start (L)	Finish (L)	Total (L)				
5	100	880.2	880.2	0	0.00	0.0	492.4	0.00
5	100	880.2	880.2	0	0.00	0.0	492.4	0.00
5	200	880.2	882.3	2.1	0.42	0.0	592.4	0.07
5	200	882.3	884.2	1.9	0.38	0.0	592.4	0.06
5	400	884.2	889.4	5.2	1.04	0.0	792.4	0.17
5	400	889.4	894.5	5.1	1.02	0.0	792.4	0.17
5	200	894.5	896.6	2.1	0.42	0.0	592.4	0.07
5	200	896.6	898.8	2.2	0.44	0.0	592.4	0.07
5	100	898.8	899.3	0.5	0.10	0.0	492.4	0.02
5	100	899.3	899.8	0.5	0.10	0.0	492.4	0.02





Interpreted Burgess Permeability : **1 μL**
Based on Burgess, 1983 5E-07 m/d

Interpreted Houlsby Permeability : **<0.5 μL**
Based on Houlsby, 1976 m/d

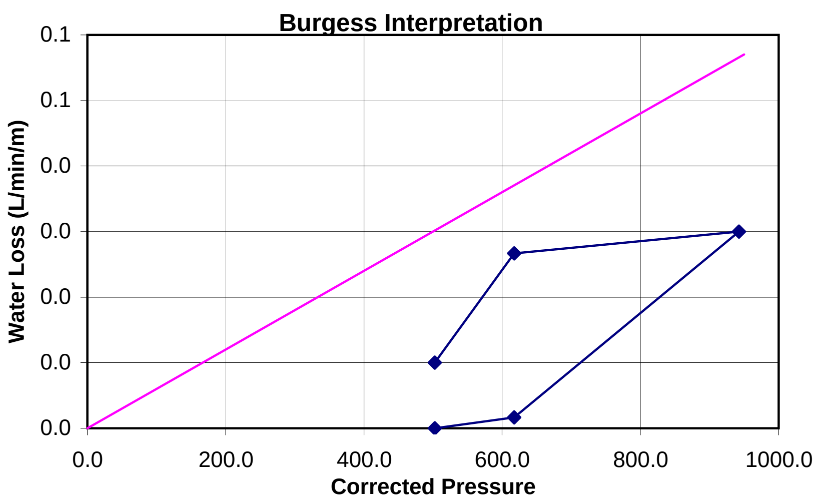
BOREHOLE WATER PRESSURE TEST

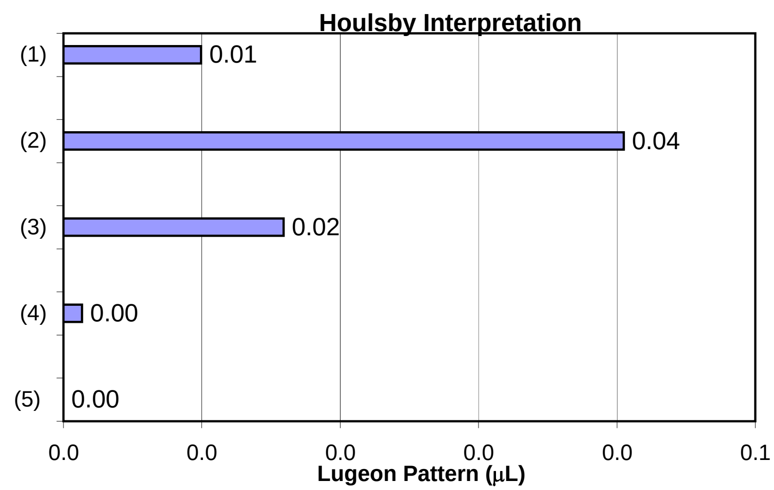
Job No: **BJ07**
Office: **Syd**

Client: **LHL** Location: **Eastern Creek** Reduced By: **IG** **Borehole no:**
Project: **Light Horse Landfill** Test Date: **13-May-09** Checked By: **MB** **BH10d**

Test Details: Packer Type: **End** Borehole Diameter: **100** Depth from: **44.00 m**
Type of Pump: Borehole Inclination: **90** Depth to: **50.00 m**
Pressure Gauge & Serial No: Borehole Azimuth: **0 0** Test Depth Interval: **6.00 m**
Pressure gauge height: **0 m** Depth to g'water: **40 m** Static Pressure: **392.4 kPa** Test Midpoint (m): **47.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	110	916.8	916.8	0	0.00	0.0	502.4	0.00
5	110	916.8	917.1	0.3	0.06	0.0	502.4	0.01
5	225	921.7	922.4	0.7	0.14	0.0	617.4	0.02
5	225	922.4	923.2	0.8	0.16	0.0	617.4	0.03
5	550	929.7	930.6	0.9	0.18	0.0	942.4	0.03
5	550	930.3	930.3	0	0.00	0.0	942.4	0.00
5	225	930.3	930.35	0.05	0.01	0.0	617.4	0.00
5	225	930.35	930.35	0	0.00	0.0	617.4	0.00
5	110	930.35	930.35	0	0.00	0.0	502.4	0.00
5	110	930.35	930.35	0	0.00	0.0	502.4	0.00





Interpreted Burgess Permeability : <0.5 µL m/d
Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 µL m/d
Based on Houlsby, 1976

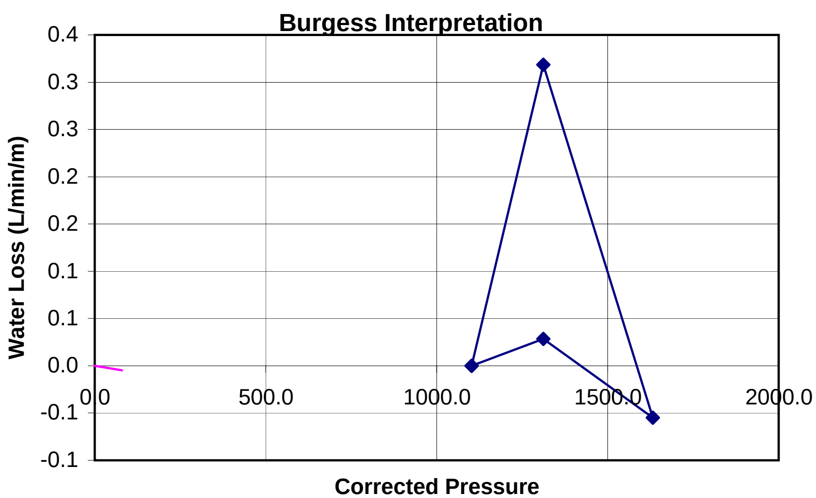
BOREHOLE WATER PRESSURE TEST

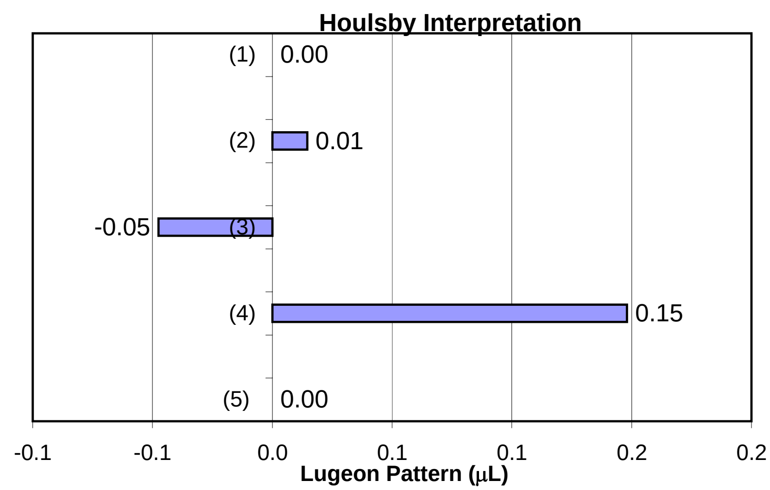
Job No:
Office:

Client: **Dial a Dump** Location: Reduced By: **MRB** **Borehole no: BH10d**
Project: **Lighthorse Landfill Site** Test Date: **14-May-09** Checked By:

Test Details: Packer Type: **Single** Borehole Diameter: **100** Depth from: **54.00 m**
Type of Pump: Borehole Inclination: **90** Depth to: **60.00 m**
Pressure Gauge & Serial No: **800kPa max** Borehole Azimuth: Test Depth Interval: **6.00 m**
Pressure gauge height: **0.1 m** Depth to g'water: **100 m** Static Pressure: **982.0 kPa** Test Midpoint (m): **57.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	120	951.7	951.7	0	0.00	0.0	1102.0	0.00
5	120	951.7	951.7	0	0.00	0.0	1102.0	0.00
5	330	959.45	960.3	0.85	0.17	0.0	1312.0	0.03
5	330	960.3	960.6	0.3	0.06	0.0	1312.0	0.01
5	650	964.65	963	-1.65	-0.33	0.0	1632.0	-0.05
5	650	963	960	-3	-0.60	0.0	1632.0	-0.10
5	330	957.7	967.25	9.55	1.91	0.0	1312.0	0.32
5	330	967.25	969.35	2.1	0.42	0.0	1312.0	0.07
5	120	979.35	979.35	0	0.00	0.0	1102.0	0.00
5	120	979.35	979.35	0	0.00	0.0	1102.0	0.00





Interpreted Burgess Permeability : <0.5 μL
Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 μL
Based on Houlsby, 1976

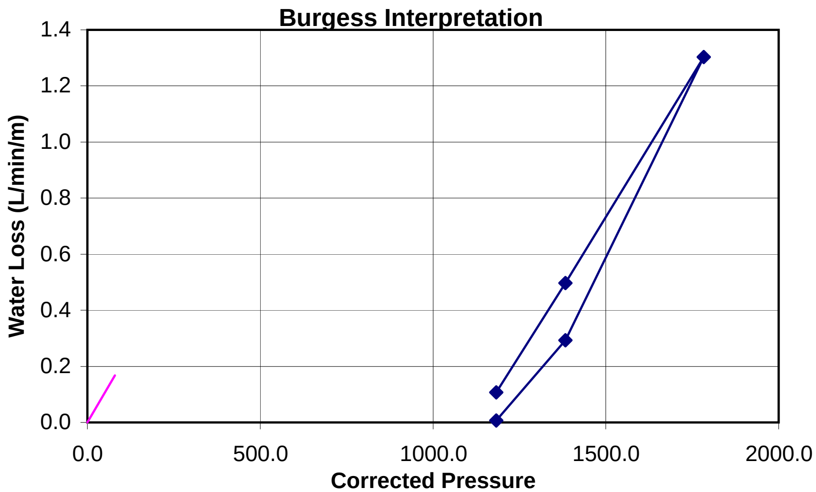
BOREHOLE WATER PRESSURE TEST

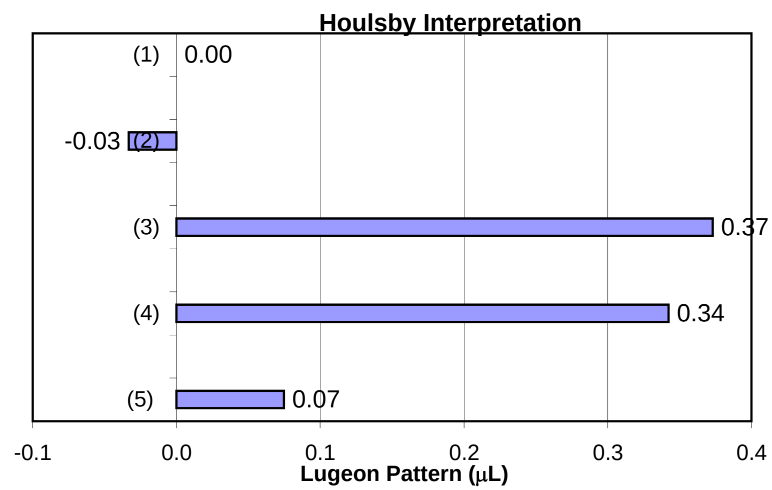
Job No:
Office:

Client: **Dial a Dump** Location:
Project: **Lighthorse Landfill Site** Test Date: **15-May-09**
Reduced By: **MRB**
Checked By:
Borehole no: BH10d

Test Details: Packer Type: **Single** Borehole Diameter: **0.1** Depth from: **64.00 m**
Type of Pump: Borehole Inclination: **90** Depth to: **70.00 m**
Pressure Gauge & Serial No: **800kPa** **PG2** Borehole Azimuth: Test Depth Interval: **6.00 m**
Pressure gauge height: **0.2 m** Depth to g'water: **100 m** Static Pressure: **983.0 kPa** Test Midpoint (m): **67.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	200	1995.55	1995.35	-0.2	-0.04	0.0	1183.0	-0.01
5	200	1995.35	1995.55	0.2	0.04	0.0	1183.0	0.01
5	400	2000.5	2009.3	8.8	1.76	0.0	1383.0	0.29
5	400	2009.3	1997.75	-11.55	-2.31	0.0	1383.0	-0.38
5	800	2002	2002.8	0.8	0.16	0.0	1783.0	0.03
5	800	2002.8	2041.9	39.1	7.82	0.0	1783.0	1.30
5	400	2043.2	2056.7	13.5	2.70	0.0	1383.0	0.45
5	400	2056.7	2071.6	14.9	2.98	0.0	1383.0	0.50
5	200	2071.6	2073.7	2.1	0.42	0.0	1183.0	0.07
5	200	2073.7	2076.9	3.2	0.64	0.0	1183.0	0.11





Interpreted Burgess Permeability : 2 µL
Based on Burgess, 1983

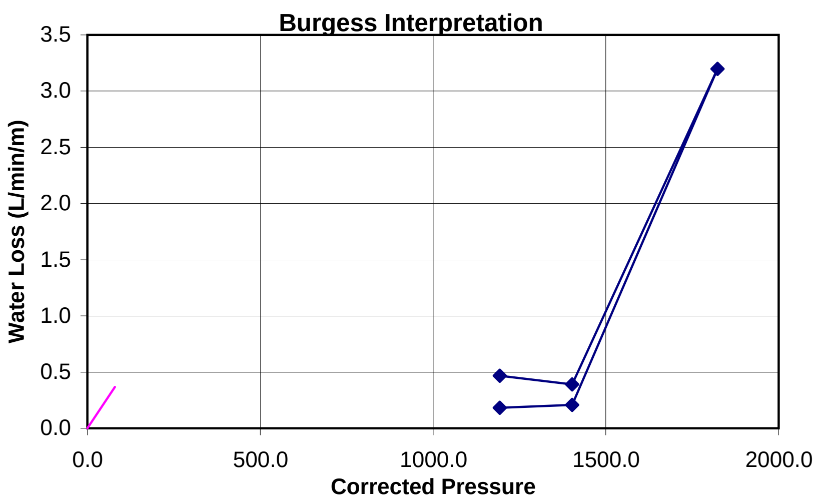
Interpreted Houlsby Permeability : <0.5 µL
Based on Houlsby, 1976

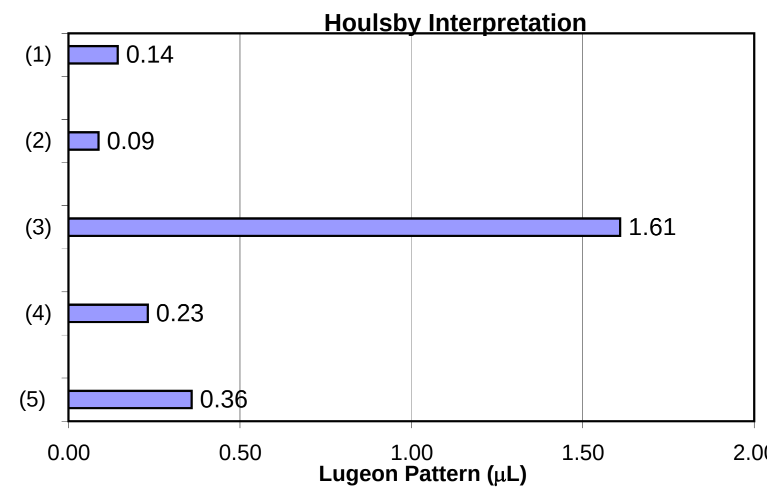
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client: **Dial a Dumpr**	Location:		Reduced By: **MRB**	**Borehole no:**
Project: **Lighthorse Landfill Site**	Test Date: **15-May-09**		Checked By:	**BH10d**
Test Details: Packer Type: **Single**		Borehole Diameter: **0.1**	Depth from:	**74.00 m**
Type of Pump:		Borehole Inclination: **-90**	Depth to:	**80.00 m**
Pressure Gauge & Serial No: **PG2**		Borehole Azimuth:	Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.2 m**	Depth to g'water: **100 m**	Static Pressure: **983.0 kPa**	Test Midpoint (m):	**77.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter)			Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
		Start (L)	Finish (L)	Total (L)				
5	210	112.6	117.45	4.85	0.97	0.0	1193.0	0.16
5	210	117.45	122.9	5.45	1.09	0.0	1193.0	0.18
5	420	124	130.2	6.2	1.24	0.0	1403.0	0.21
5	420	130.2	131.35	1.15	0.23	0.0	1403.0	0.04
5	840	148	228.1	80.1	16.02	0.0	1823.0	2.67
5	840	228.1	324	95.9	19.18	0.0	1823.0	3.20
5	420	343	354.7	11.7	2.34	0.0	1403.0	0.39
5	420	354.7	362.45	7.75	1.55	0.0	1403.0	0.26
5	210	362.45	376.45	14	2.80	0.0	1193.0	0.47
5	210	376.45	388.2	11.75	2.35	0.0	1193.0	0.39





Interpreted Burgess Permeability : 5 µL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 µL

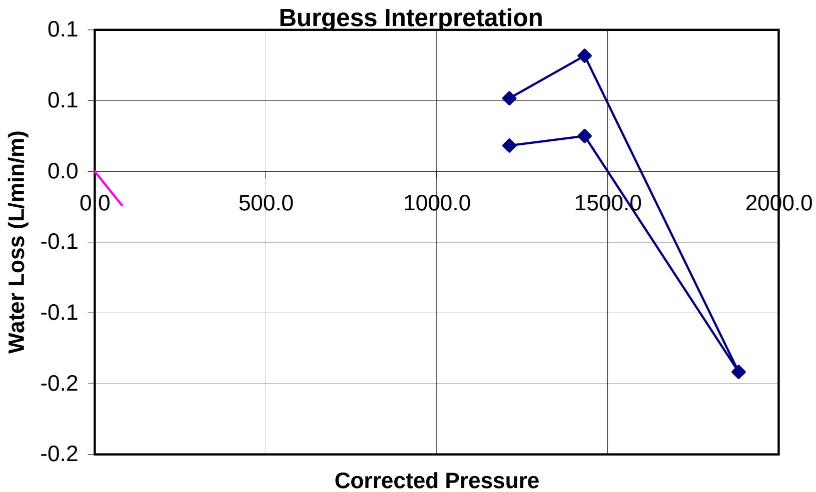
Based on Houlsby, 1976

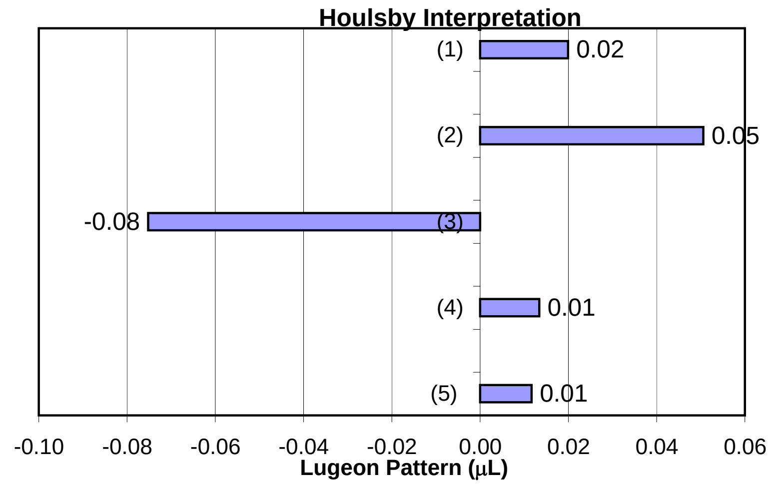
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client:	**Dial a Dump**	Location:		Reduced By: **MRB**	**Borehole no:**
Project:	**Lighthorse Landfill Site**	Test Date:	**18-May-09**	Checked By:	**BH10d**
Test Details: Packer Type:	**single**	Borehole Diameter:	**100 mm**	Depth from:	**84.00 m**
Type of Pump:		Borehole Inclination:	**-90**	Depth to:	**90.00 m**
Pressure Gauge & Serial No:	**3 Dial set**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.2 m**	Depth to g'water: **100 m**	Static Pressure:	**983.0 kPa**	Test Midpoint (m):	**87.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	230	446.15	446.05	-0.1	-0.02	0.0	1213.0	0.00
5	230	446.05	447.6	1.55	0.31	0.0	1213.0	0.05
5	450	450.75	452.65	1.9	0.38	0.0	1433.0	0.06
5	450	452.65	455.1	2.45	0.49	0.0	1433.0	0.08
5	900	563.4	559.15	-4.25	-0.85	0.0	1883.0	-0.14
5	900	563.4	559.15	-4.25	-0.85	0.0	1883.0	-0.14
5	450	558.65	559.4	0.75	0.15	0.0	1433.0	0.03
5	450	559.4	559.8	0.4	0.08	0.0	1433.0	0.01
5	230	559.8	560.1	0.3	0.06	0.0	1213.0	0.01
5	230	560.1	560.65	0.55	0.11	0.0	1213.0	0.02





Interpreted Burgess Permeability : <0.5 μL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 μL

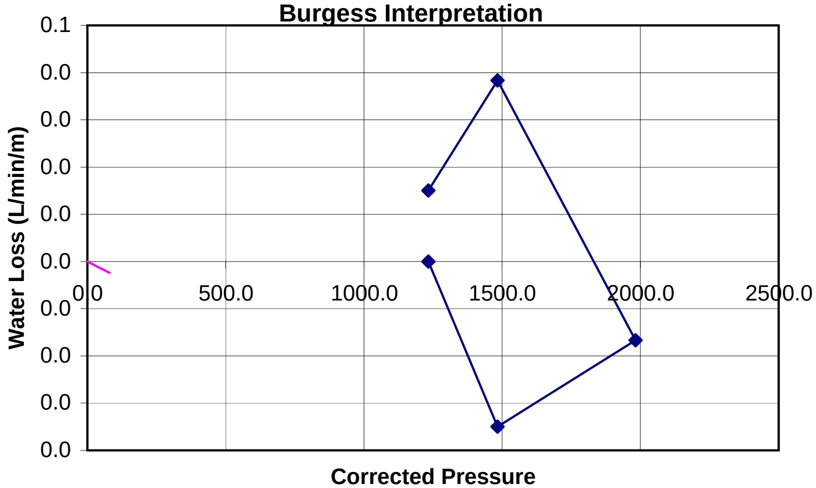
Based on Houlsby, 1976

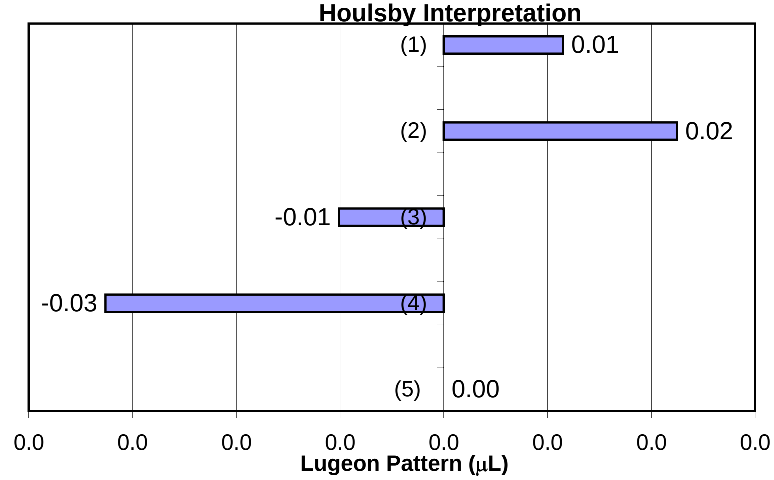
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client:	**Dial a Dump**	Location:		Reduced By: **MRB**	**Borehole no:**
Project:	**Lighthorse Landfill Site**	Test Date:	**19-May-09**	Checked By:	**BH10d**
Test Details: Packer Type:	**Single**	Borehole Diameter:	**100 mm**	Depth from:	**94.00 m**
Type of Pump:		Borehole Inclination:	**90**	Depth to:	**100.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.2 m**	Depth to g'water: **100 m**	Static Pressure:	**983.0 kPa**	Test Midpoint (m):	**97.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	250	592.25	592.7	0.45	0.09	0.0	1233.0	0.02
5	250	592.7	593.1	0.4	0.08	0.0	1233.0	0.01
5	500	597.8	598.65	0.85	0.17	0.0	1483.0	0.03
5	500	598.65	599.8	1.15	0.23	0.0	1483.0	0.04
5	1000	603.5	602.8	-0.7	-0.14	0.0	1983.0	-0.02
5	1000	602.8	602.3	-0.5	-0.10	0.0	1983.0	-0.02
5	500	600.7	599.65	-1.05	-0.21	0.0	1483.0	-0.04
5	500	599.65	597.8	-1.85	-0.37	0.0	1483.0	-0.06
5	250	592.2	592.2	0	0.00	0.0	1233.0	0.00
5	250	592.2	592.2	0	0.00	0.0	1233.0	0.00





Interpreted Burgess Permeability : <0.5 µL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 µL

Based on Houlsby, 1976

BOREHOLE WATER PRESSURE TEST

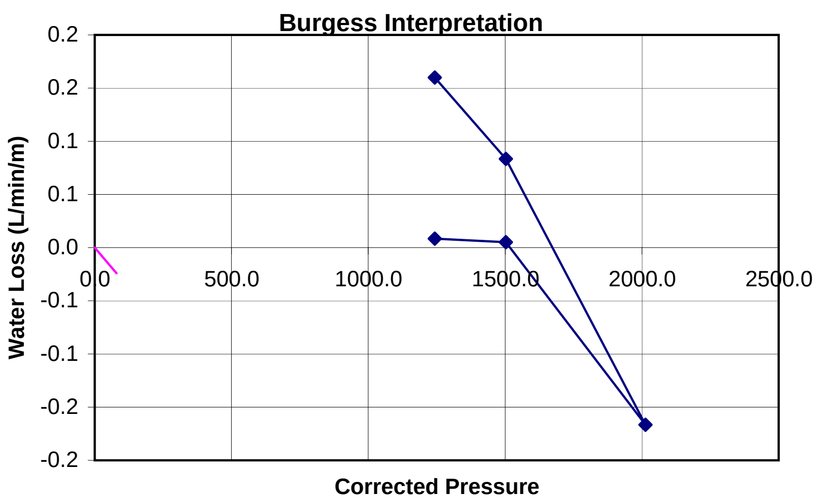
Job No:
Office:

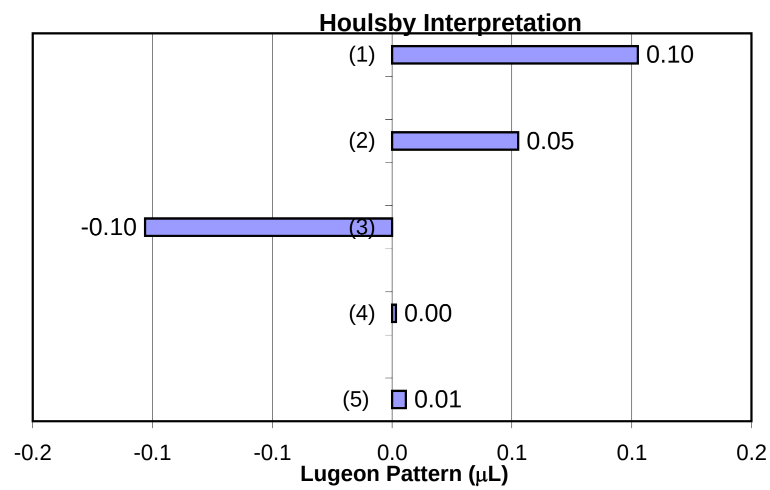
Client: **Dial a Dump**	Location:	Reduced By: **MRB**	**Borehole no:**
Project: **Light Horse Landfill Site**	Test Date: **19-May-09**	Checked By:	**BH10d**

Test Details:

Packer Type: **Single**	Borehole Diameter: **100**	Depth from:	**104.00 m**
Type of Pump: **Rotary surface**	Borehole Inclination: **-90**	Depth to:	**110.00 m**
Pressure Gauge & Serial No: **Analogue dial** **PG2**	Borehole Azimuth:	Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.2 m** Depth to g'water: **100 m**	Static Pressure: **983.0 kPa**	Test Midpoint (m):	**107.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	260	620.7	625.5	4.8	0.96	0.0	1243.0	0.16
5	260	625.5	628.35	2.85	0.57	0.0	1243.0	0.10
5	520	631.45	633.95	2.5	0.50	0.0	1503.0	0.08
5	520	633.95	636.2	2.25	0.45	0.0	1503.0	0.08
5	1030	643.2	638.2	-5	-1.00	0.0	2013.0	-0.17
5	1030	638.2	630.75	-7.45	-1.49	0.0	2013.0	-0.25
5	520	629.8	629.95	0.15	0.03	0.0	1503.0	0.01
5	520	629.95	629.95	0	0.00	0.0	1503.0	0.00
5	260	628.65	628.9	0.25	0.05	0.0	1243.0	0.01
5	260	628.9	629.08	0.18	0.04	0.0	1243.0	0.01





Interpreted Burgess Permeability : <0.5 μL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 μL

Based on Houlsby, 1976

BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client: **Dial a Dump** Location: Reduced By: **MRB** **Borehole no: BH10d**

Project: **Lighthores landfill site** Test Date: **20-May-09** Checked By:

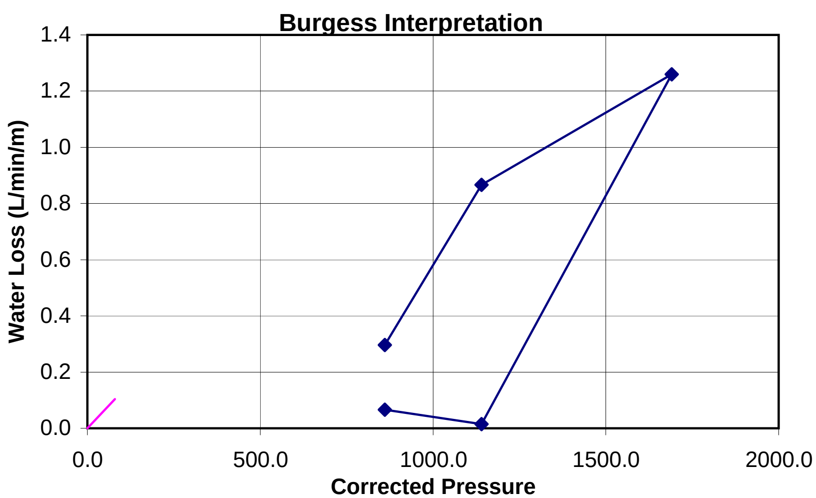
Test Details: Packer Type: **Single** Borehole Diameter: **0.1 m** Depth from: **114.00 m**

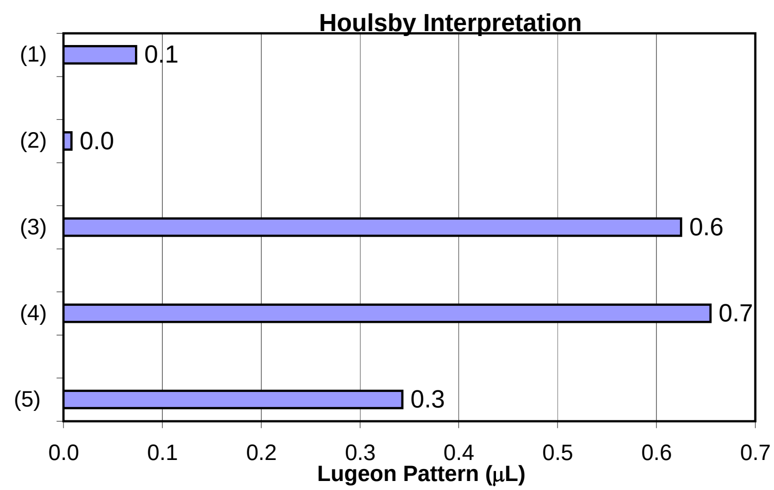
Type of Pump: Borehole Inclination: **-90** Depth to: **120.00 m**

Pressure Gauge & Serial No: **PG2** Borehole Azimuth: Test Depth Interval: **6.00 m**

Pressure gauge height: **0.2 m** Depth to g'water: **60 m** Static Pressure: **590.6 kPa** Test Midpoint (m): **117.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	270	656.4	658.4	2	0.40	0.0	860.6	0.07
5	270	658.4	660.2	1.8	0.36	0.0	860.6	0.06
5	550	666.35	666.8	0.45	0.09	0.0	1140.6	0.01
5	550	666.8	666.9	0.1	0.02	0.0	1140.6	0.00
5	1100	686.8	712.4	25.6	5.12	0.0	1690.6	0.85
5	1100	712.4	750.2	37.8	7.56	0.0	1690.6	1.26
5	550	752	778	26	5.20	0.0	1140.6	0.87
5	550	778	796.8	18.8	3.76	0.0	1140.6	0.63
5	270	795.7	804.5	8.8	1.76	0.0	860.6	0.29
5	270	804.5	813.4	8.9	1.78	0.0	860.6	0.30





Interpreted Burgess Permeability : 1 μL

Based on Burgess, 1983

Interpreted Houlsby Permeability : 1 μL

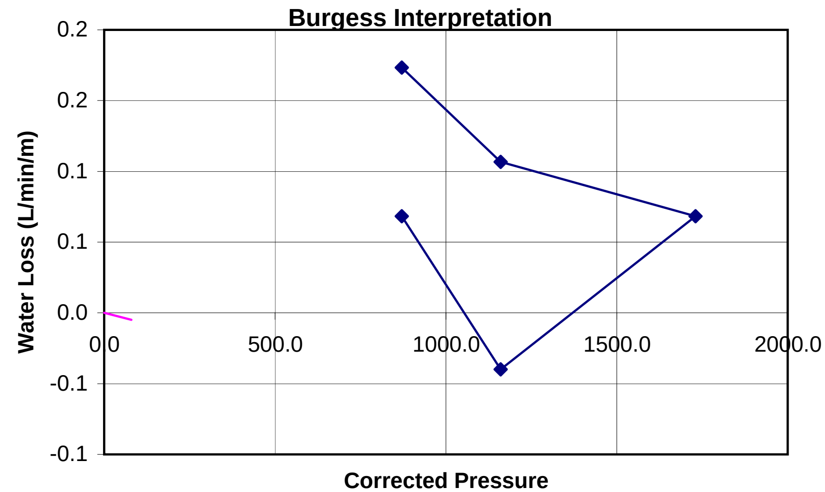
Based on Houlsby, 1976

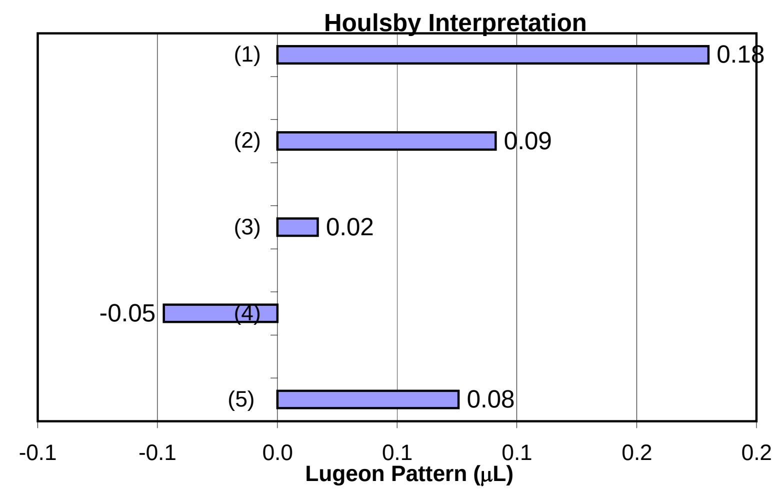
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client:	**Dial a Dump**	Location:		Reduced By: **MRB**	**Borehole no:**
Project:	**Light Horese Landfill Site**	Test Date:	**21-May-09**	Checked By:	**BH10d**
Test Details: Packer Type:	**Single**	Borehole Diameter:	**0.1**	Depth from:	**124.00 m**
Type of Pump:		Borehole Inclination:	**-90**	Depth to:	**130.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.2 m**	Depth to g'water: **60 m**	Static Pressure:	**590.6 kPa**	Test Midpoint (m):	**127.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	280	892.8	898	5.2	1.04	0.0	870.6	0.17
5	280	898	902.2	4.2	0.84	0.0	870.6	0.14
5	570	911.15	914.35	3.2	0.64	0.0	1160.6	0.11
5	570	914.35	917.5	3.15	0.63	0.0	1160.6	0.10
5	1140	929.1	931.15	2.05	0.41	0.0	1730.6	0.07
5	1140	931.15	930.85	-0.3	-0.06	0.0	1730.6	-0.01
5	570	932.4	931.2	-1.2	-0.24	0.0	1160.6	-0.04
5	570	931.2	929.1	-2.1	-0.42	0.0	1160.6	-0.07
5	280	927.65	929.7	2.05	0.41	0.0	870.6	0.07
5	280	929.7	931.6	1.9	0.38	0.0	870.6	0.06





Interpreted Burgess Permeability : <0.5 μL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 μL

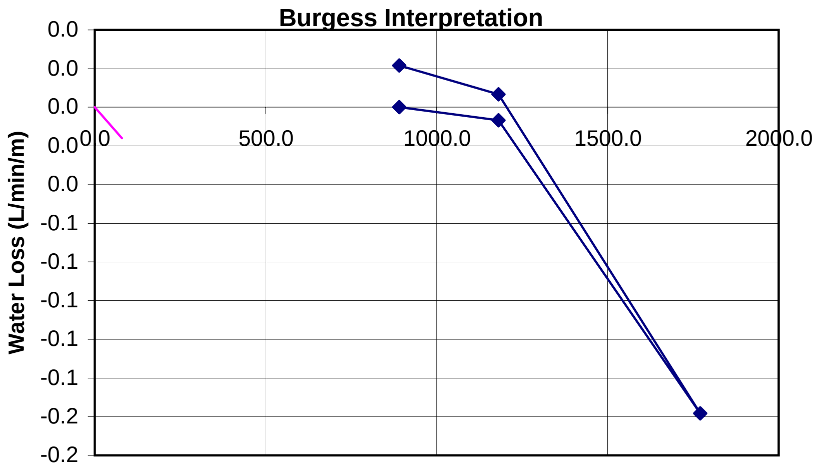
Based on Houlsby, 1976

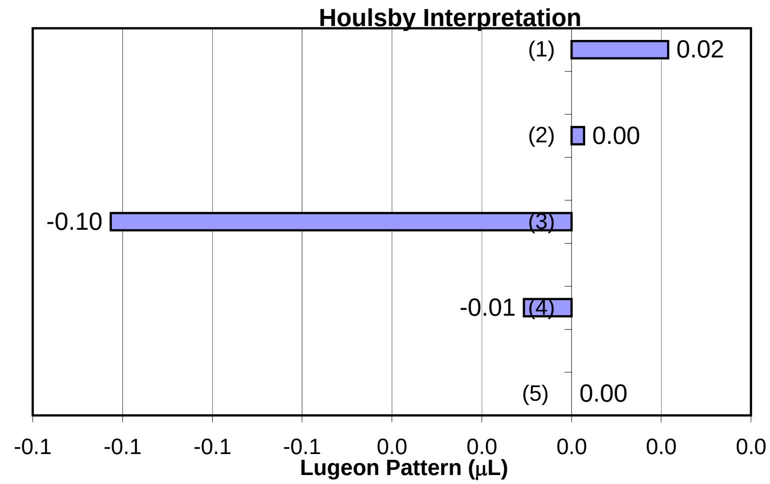
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client: **Dial a Dump**		Location:		Reduced By: **MRB**	**Borehole no:**
Project: **Light Horse Landfill site**		Test Date: **21-May-09**		Checked By:	**BH10d**
Test Details:	Packer Type: **Single**	Borehole Diameter: **0.1 m**		Depth from:	**134.00 m**
	Type of Pump:	Borehole Inclination: **-90**		Depth to:	**140.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.2 m**	Depth to g'water: **60 m**	Static Pressure:	**590.6 kPa**	Test Midpoint (m):	**137.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	300	948	948.65	0.65	0.13	0.0	890.6	0.02
5	300	951.5	952	0.5	0.10	0.0	890.6	0.02
5	590	952	952.2	0.2	0.04	0.0	1180.6	0.01
5	590	952.2	952.2	0	0.00	0.0	1180.6	0.00
5	1180	953.9	949.15	-4.75	-0.95	0.0	1770.6	-0.16
5	1180	949.15	943	-6.15	-1.23	0.0	1770.6	-0.20
5	590	942.65	942.45	-0.2	-0.04	0.0	1180.6	-0.01
5	590	942.45	941.9	-0.55	-0.11	0.0	1180.6	-0.02
5	300	940.9	940.9	0	0.00	0.0	890.6	0.00
5	300	940.9	940.9	0	0.00	0.0	890.6	0.00





Interpreted Burgess Permeability : <0.5 µL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 µL

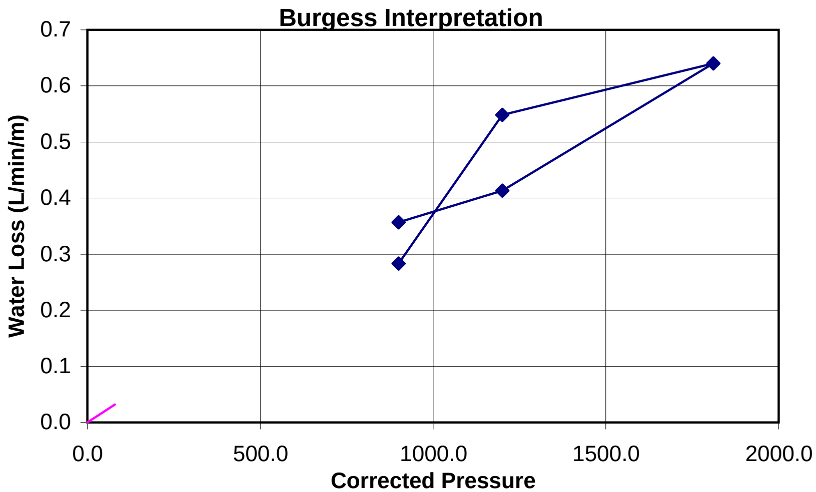
Based on Houlsby, 1976

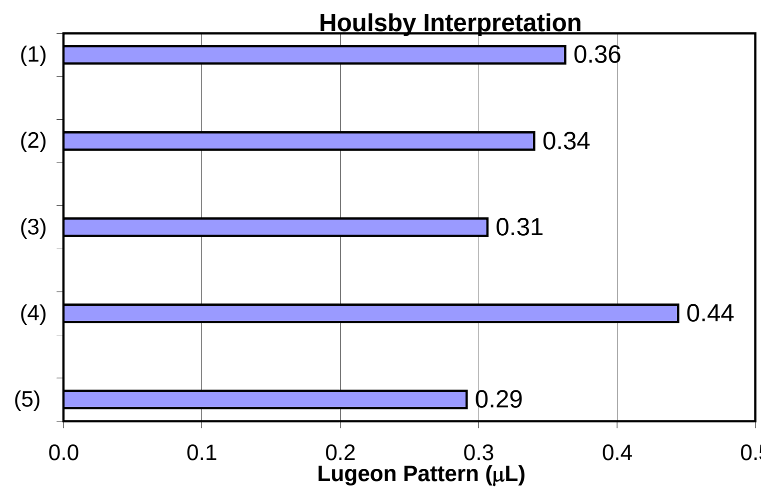
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client: **Dial a Dump**		Location:		Reduced By: **MRB**	**Borehole no:**
Project: **Lighthorse Landfill Site**		Test Date: **22-May-09**		Checked By:	**BH10d**
Test Details:	Packer Type: **Single**	Borehole Diameter: **0.1**		Depth from:	**144.00 m**
	Type of Pump:	Borehole Inclination: **-90**		Depth to:	**150.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.2 m**	Depth to g'water: **60 m**	Static Pressure:	**590.6 kPa**	Test Midpoint (m):	**147.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	310	102993.5	103004.2	10.7	2.14	0.0	900.6	0.36
5	310	103004.2	103013.1	8.9	1.78	0.0	900.6	0.30
5	610	103017.4	103029.5	12.1	2.42	0.0	1200.6	0.40
5	610	103029.5	103041.9	12.4	2.48	0.0	1200.6	0.41
5	1220	103269.6	103288.8	19.2	3.84	0.0	1810.6	0.64
5	1220	103288.8	103302.9	14.1	2.82	0.0	1810.6	0.47
5	610	103308.4	103324.9	16.45	3.29	0.0	1200.6	0.55
5	610	103324.9	103340.4	15.55	3.11	0.0	1200.6	0.52
5	310	103351.1	103359.6	8.5	1.70	0.0	900.6	0.28
5	310	103359.6	103366.9	7.25	1.45	0.0	900.6	0.24





Interpreted Burgess Permeability : <0.5 μL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 μL

Based on Houlsby, 1976

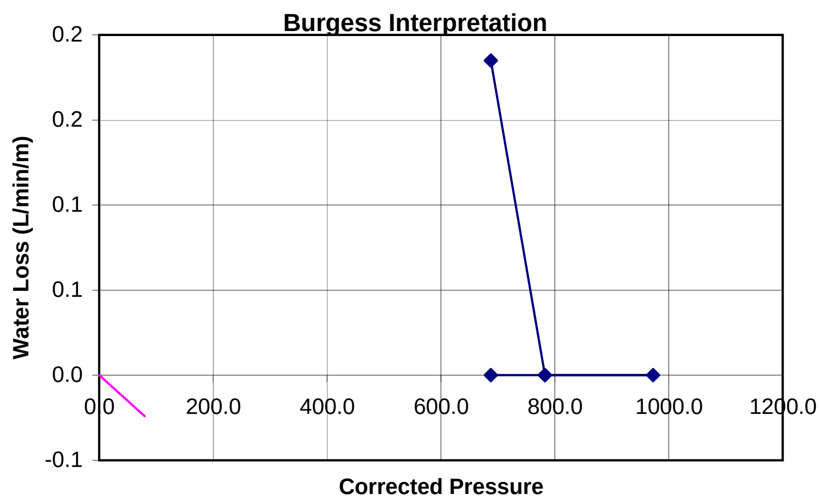
BOREHOLE WATER PRESSURE TEST

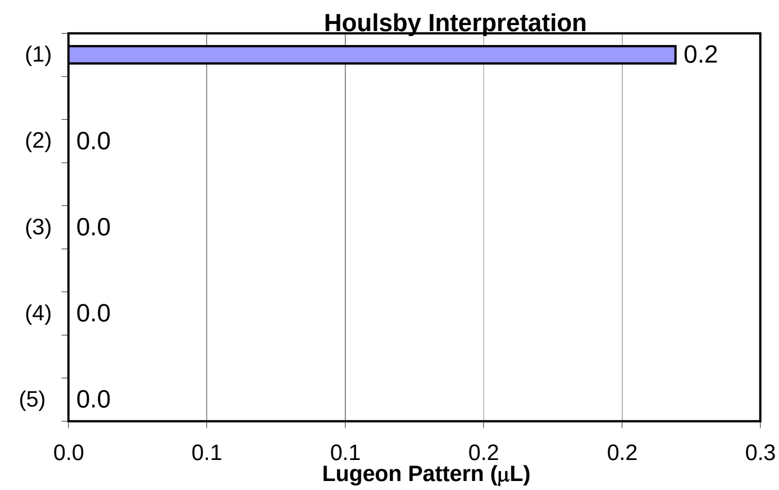
Job No:
Office:

Client: **Dial a dump** Location:
Project: **Light Horse Landfill Site** Test Date: **28-May-09**
Reduced By: **MRB**
Checked By:
Borehole no: BH12d

Test Details: Packer Type: **Single** Borehole Diameter: **93 mm** Depth from: **34.00 m**
Type of Pump: **Rotary** Borehole Inclination: **-90** Depth to: **40.00 m**
Pressure Gauge & Serial No: **PG2** Borehole Azimuth: Test Depth Interval: **6.00 m**
Pressure gauge height: **0.4 m** Depth to g'water: **60 m** Static Pressure: **592.5 kPa** Test Midpoint (m): **37.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter)			Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
		Start (L)	Finish (L)	Total (L)				
5	95	379.05	382.55	3.5	0.70	0.0	687.5	0.12
5	95	382.55	388.1	5.55	1.11	0.0	687.5	0.19
5	190	407	407	0	0.00	0.0	782.5	0.00
5	190	407	407	0	0.00	0.0	782.5	0.00
5	380	407.2	407.2	0	0.00	0.0	972.5	0.00
5	380	407.2	407.2	0	0.00	0.0	972.5	0.00
5	190	407.2	407.2	0	0.00	0.0	782.5	0.00
5	190	407.2	407.2	0	0.00	0.0	782.5	0.00
5	95	407.25	407.25	0	0.00	0.0	687.5	0.00
5	95	407.25	407.25	0	0.00	0.0	687.5	0.00





Interpreted Burgess Permeability : <0.5 μL
Based on Burgess, 1983

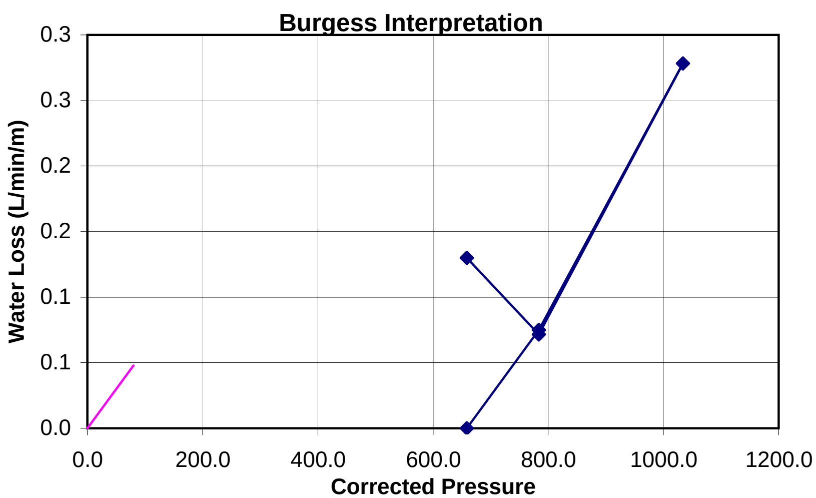
Interpreted Houlsby Permeability : <0.5 μL
Based on Houlsby, 1976

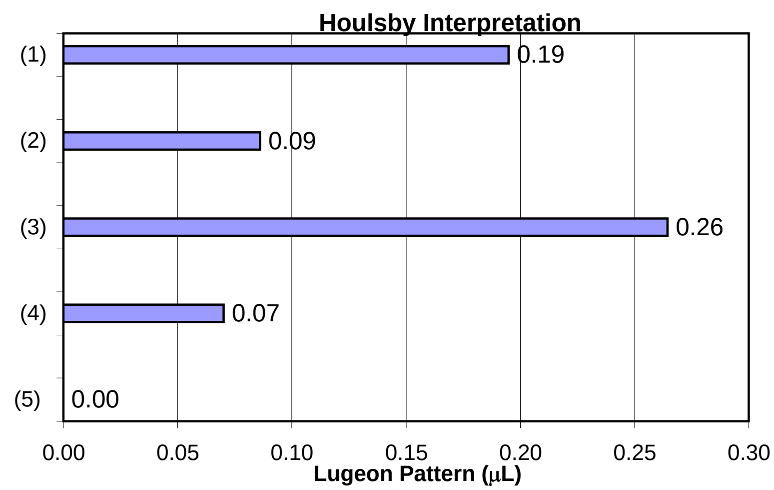
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client:	**Dial a Dump**	Location:		Reduced By: **MRB**	**Borehole no:**
Project:	**Lighthorse Landfill Site**	Test Date:	**28-May-09**	Checked By:	**BH12d**
Test Details: Packer Type:	**Single**	Borehole Diameter:	**93 mm**	Depth from:	**44.00 m**
Type of Pump:	**Rotary**	Borehole Inclination:	**-90**	Depth to:	**50.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.4 m**	Depth to g'water: **54 m**	Static Pressure:	**533.7 kPa**	Test Midpoint (m):	**47.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter)			Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
		Start (L)	Finish (L)	Total (L)				
5	125	439.4	443.3	3.9	0.78	0.0	658.7	0.13
5	125	443.3	447.1	3.8	0.76	0.0	658.7	0.13
5	250	476.6	478.5	1.9	0.38	0.0	783.7	0.06
5	250	478.5	480.65	2.15	0.43	0.0	783.7	0.07
5	500	484.15	492.5	8.35	1.67	0.0	1033.7	0.28
5	500	492.5	500.55	8.05	1.61	0.0	1033.7	0.27
5	250	502.15	504.4	2.25	0.45	0.0	783.7	0.08
5	250	504.4	505.45	1.05	0.21	0.0	783.7	0.04
5	125	505.45	505.45	0	0.00	0.0	658.7	0.00
5	125	505.45	505.45	0	0.00	0.0	658.7	0.00





Interpreted Burgess Permeability : 1 µL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 µL

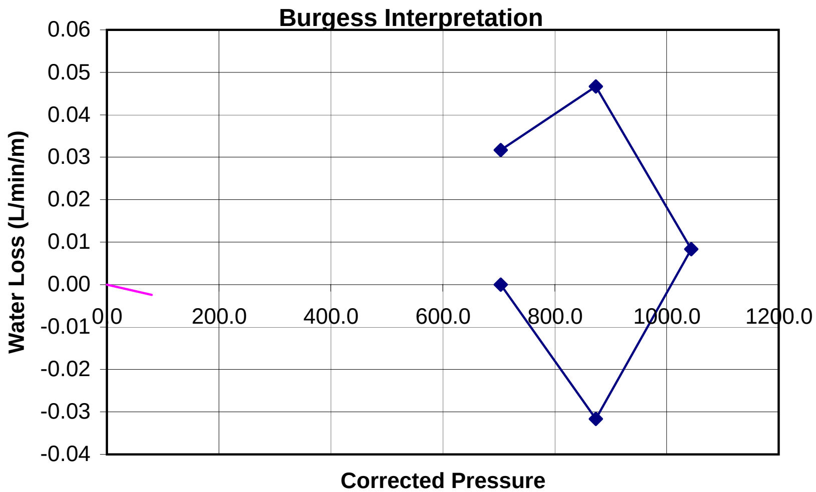
Based on Houlsby, 1976

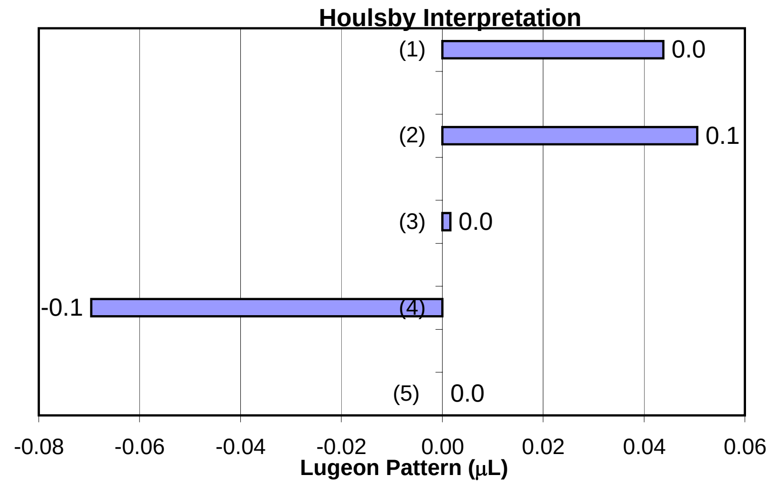
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client:	**Dial a Dump**	Location:		Reduced By: **MRB**	**Borehole no:**
Project:	**Lighthorse Landfill Site**	Test Date:	**29-May-09**	Checked By:	**BH12d**
Test Details: Packer Type:	**Single**	Borehole Diameter:	**93 mm**	Depth from:	**54.00 m**
Type of Pump:	**Rotary**	Borehole Inclination:	**-90**	Depth to:	**60.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.4 m**	Depth to g'water: **54 m**	Static Pressure:	**533.7 kPa**	Test Midpoint (m):	**57.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter)			Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
		Start (L)	Finish (L)	Total (L)				
5	170	560.5	561.45	0.95	0.19	0.0	703.7	0.03
5	170	561.45	562.35	0.9	0.18	0.0	703.7	0.03
5	340	568.6	570	1.4	0.28	0.0	873.7	0.05
5	340	570	571.25	1.25	0.25	0.0	873.7	0.04
5	510	574.4	574.25	-0.15	-0.03	0.0	1043.7	0.00
5	510	574.25	574.5	0.25	0.05	0.0	1043.7	0.01
5	340	574.25	573.3	-0.95	-0.19	0.0	873.7	-0.03
5	340	573.3	570.6	-2.7	-0.54	0.0	873.7	-0.09
5	170	568.6	568.6	0	0.00	0.0	703.7	0.00
5	170	568.6	568.6	0	0.00	0.0	703.7	0.00





Interpreted Burgess Permeability : <0.5 μL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 μL

Based on Houlsby, 1976

BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client: **Dial a Dump** Location: Reduced By: **MRB** **Borehole no: BH12d**

Project: **Lighthorse Landfill Site** Test Date: **01-Jun-09** Checked By:

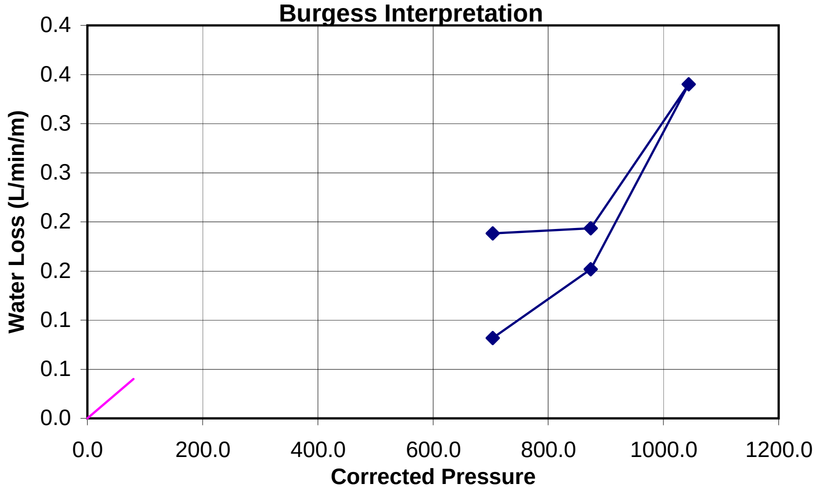
Test Details: Packer Type: **Single** Borehole Diameter: **93 mm** Depth from: **64.00 m**

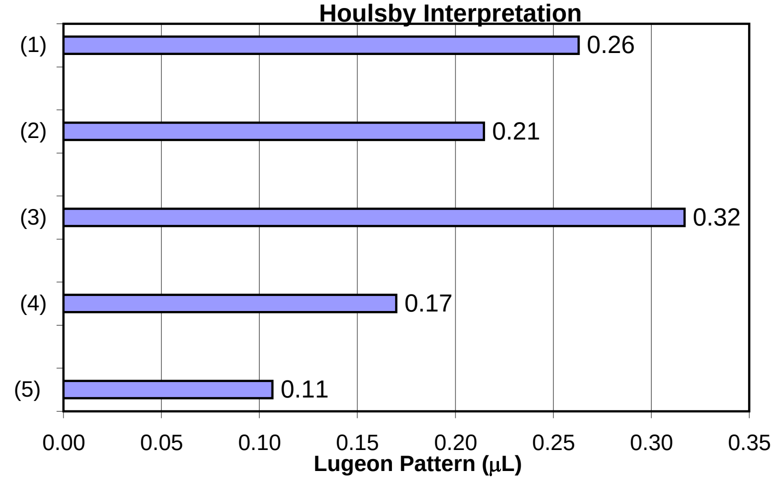
Type of Pump: **Rotary** Borehole Inclination: **-90** Depth to: **70.00 m**

Pressure Gauge & Serial No: **PG2** Borehole Azimuth: Test Depth Interval: **6.00 m**

Pressure gauge height: **0.4 m** Depth to g'water: **54 m** Static Pressure: **533.7 kPa** Test Midpoint (m): **67.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	170	616.3	621.95	5.65	1.13	0.0	703.7	0.19
5	170	621.95	627.4	5.45	1.09	0.0	703.7	0.18
5	340	628.35	634.15	5.8	1.16	0.0	873.7	0.19
5	340	634.15	639.6	5.45	1.09	0.0	873.7	0.18
5	510	641.6	651.8	10.2	2.04	0.0	1043.7	0.34
5	510	651.8	661.45	9.65	1.93	0.0	1043.7	0.32
5	340	661.45	666	4.55	0.91	0.0	873.7	0.15
5	340	666	670.35	4.35	0.87	0.0	873.7	0.15
5	170	670.35	672.4	2.05	0.41	0.0	703.7	0.07
5	170	672.4	674.85	2.45	0.49	0.0	703.7	0.08





Interpreted Burgess Permeability : 1 µL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 µL

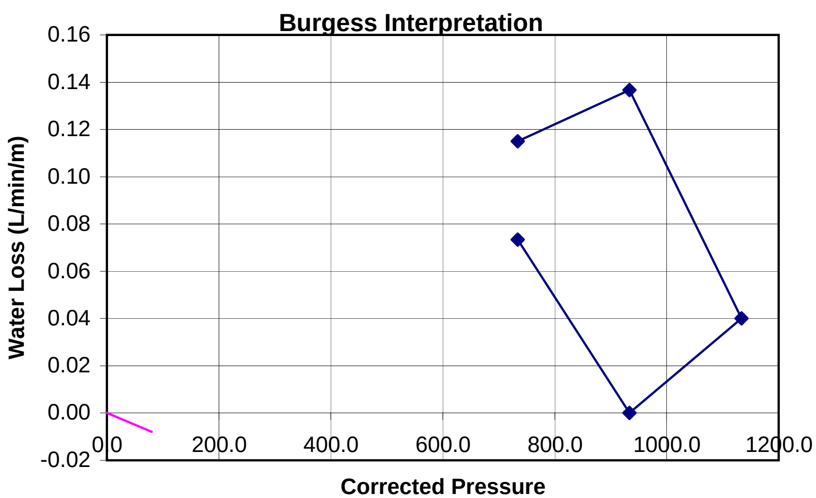
Based on Houlsby, 1976

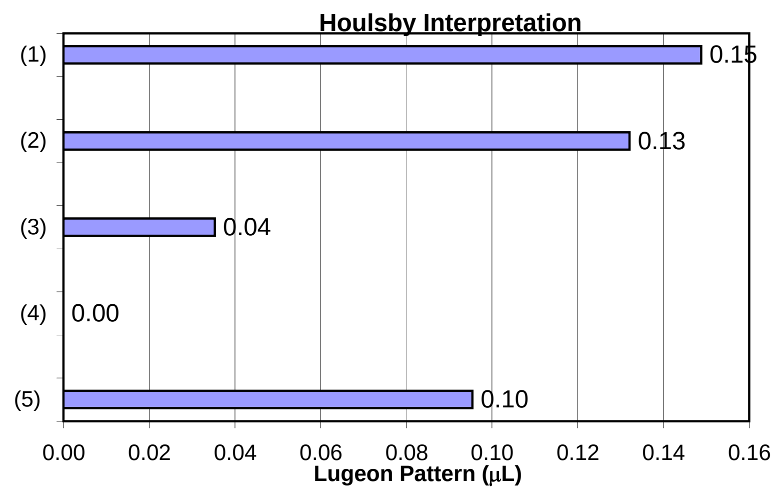
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client: **Dial a Dump**	Location:	Reduced By: **MRB**	**Borehole no:**
Project: **Lighthorse Landfill Site**	Test Date: **02-Jun-09**	Checked By:	**BH12d**
Test Details: Packer Type: **Single**	Borehole Diameter: **93 mm**	Depth from:	**74.00 m**
Type of Pump: **Rotary**	Borehole Inclination: **-90**	Depth to:	**80.00 m**
Pressure Gauge & Serial No: **PG2**	Borehole Azimuth:	Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.4 m** Depth to g'water: **54 m**	Static Pressure: **533.7 kPa**	Test Midpoint (m):	**77.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	200	792.05	795.5	3.45	0.69	0.0	733.7	0.12
5	200	795.5	798.6	3.1	0.62	0.0	733.7	0.10
5	400	816.7	820.8	4.1	0.82	0.0	933.7	0.14
5	400	820.8	824.1	3.3	0.66	0.0	933.7	0.11
5	600	843.4	844.6	1.2	0.24	0.0	1133.7	0.04
5	600	843.4	844.6	1.2	0.24	0.0	1133.7	0.04
5	400	838.1	838.1	0	0.00	0.0	933.7	0.00
5	400	838.1	838.1	0	0.00	0.0	933.7	0.00
5	200	835	837	2	0.40	0.0	733.7	0.07
5	200	837	839.2	2.2	0.44	0.0	733.7	0.07





Interpreted Burgess Permeability : <0.5 μL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 μL

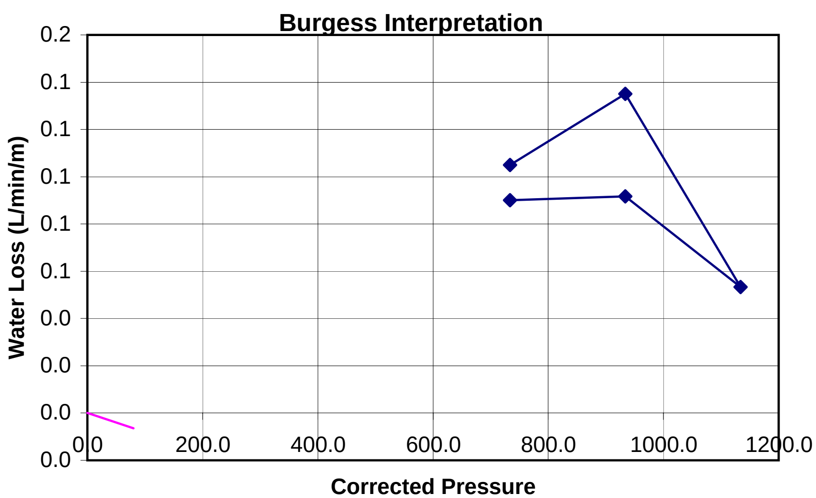
Based on Houlsby, 1976

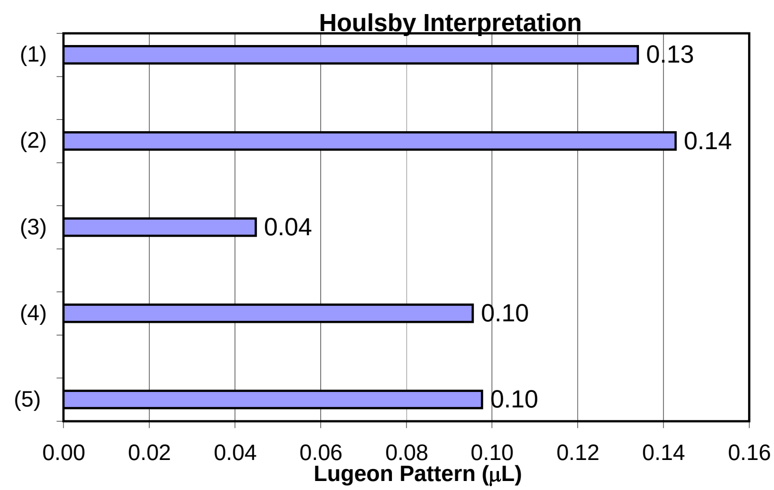
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client:	**Dial a Dump**	Location:		Reduced By: **MRB**	**Borehole no:**
Project:	**Lighthorse Landfill Site**	Test Date:	**02-Jun-09**	Checked By:	**BH12d**
Test Details: Packer Type:	**Single**	Borehole Diameter:	**93 mm**	Depth from:	**84.00 m**
Type of Pump:	**Rotary**	Borehole Inclination:	**-90**	Depth to:	**90.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.4 m**	Depth to g'water: **54 m**	Static Pressure:	**533.7 kPa**	Test Midpoint (m):	**87.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	200	884.25	887.4	3.15	0.63	0.0	733.7	0.10
5	200	887.4	890.15	2.75	0.55	0.0	733.7	0.09
5	400	897.55	901.6	4.05	0.81	0.0	933.7	0.14
5	400	901.6	905.55	3.95	0.79	0.0	933.7	0.13
5	600	910	911.6	1.6	0.32	0.0	1133.7	0.05
5	600	911.6	913.05	1.45	0.29	0.0	1133.7	0.05
5	400	912.9	915.65	2.75	0.55	0.0	933.7	0.09
5	400	915.65	918.25	2.6	0.52	0.0	933.7	0.09
5	200	916.5	918.1	1.6	0.32	0.0	733.7	0.05
5	200	918.1	920.8	2.7	0.54	0.0	733.7	0.09





Interpreted Burgess Permeability : <0.5 μL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 μL

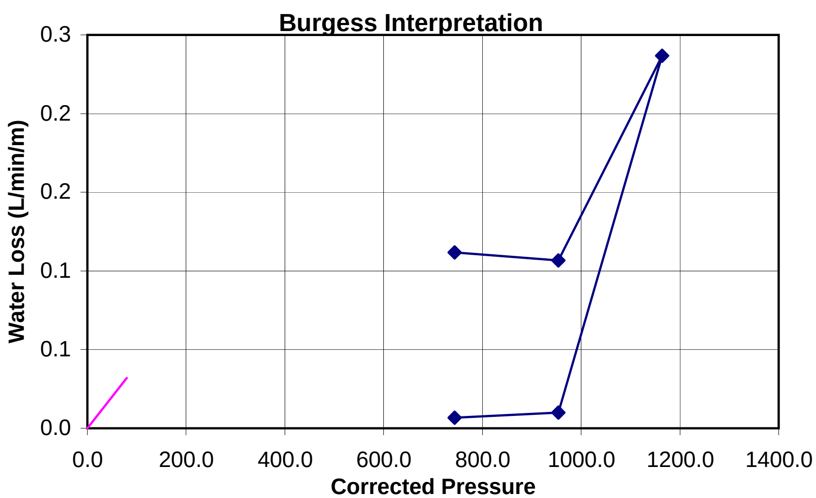
Based on Houlsby, 1976

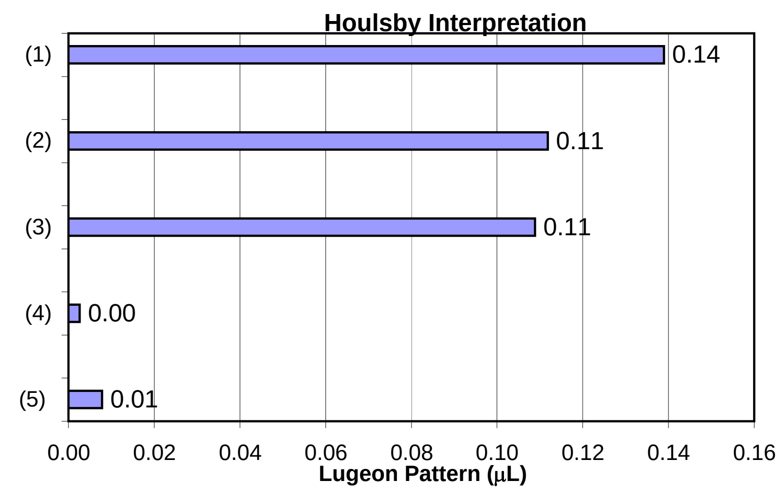
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client:	**Dial a Dump**	Location:		Reduced By: **MRB**	**Borehole no:**
Project:	**Lighthorse Landfill Site**	Test Date:	**04-Jun-09**	Checked By:	**BH12d**
Test Details: Packer Type:	**Single**	Borehole Diameter:	**93 mm**	Depth from:	**94.00 m**
Type of Pump:	**Rotary**	Borehole Inclination:	**-90**	Depth to:	**100.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height:	**0.4 m** Depth to g'water: **54 m**	Static Pressure:	**533.7 kPa**	Test Midpoint (m):	**97.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	210	3978.2	3981.55	3.35	0.67	0.0	743.7	0.11
5	210	3981.55	3984.4	2.85	0.57	0.0	743.7	0.09
5	420	3995.3	3998.5	3.2	0.64	0.0	953.7	0.11
5	420	3998.5	4001.7	3.2	0.64	0.0	953.7	0.11
5	630	4001.7	4008.8	7.1	1.42	0.0	1163.7	0.24
5	630	4008.8	4009.3	0.5	0.10	0.0	1163.7	0.02
5	420	4009.15	4009	-0.15	-0.03	0.0	953.7	-0.01
5	420	4009	4009.3	0.3	0.06	0.0	953.7	0.01
5	210	4007.4	4007.6	0.2	0.04	0.0	743.7	0.01
5	210	4007.6	4007.75	0.15	0.03	0.0	743.7	0.01





Interpreted Burgess Permeability : <0.5 μL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 μL

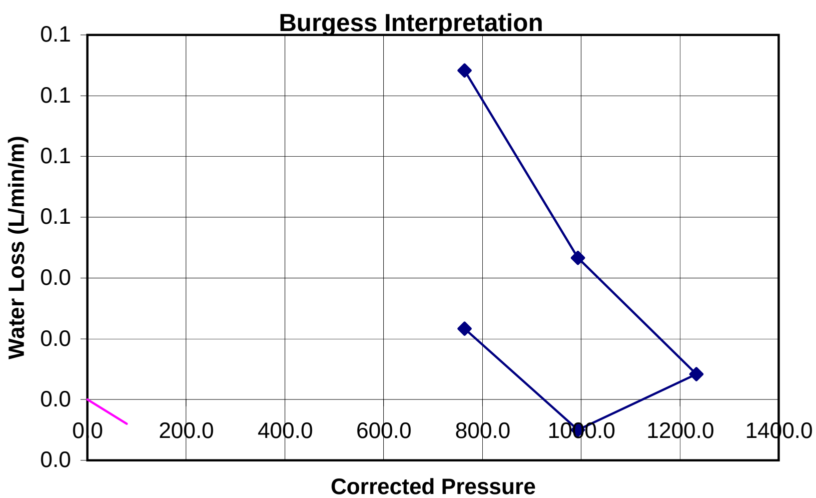
Based on Houlsby, 1976

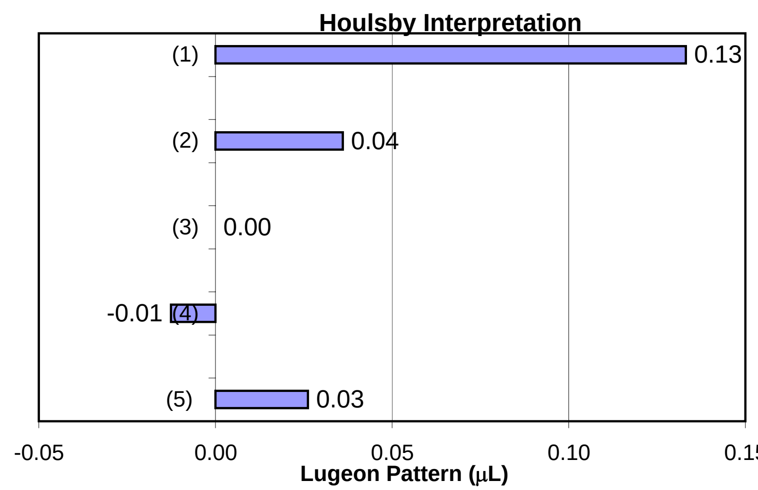
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client:	**Dial a Dump**	Location:		Reduced By: **MRB**	**Borehole no:**
Project:	**Lighthorse Landfill Site**	Test Date:	**05-Jun-09**	Checked By:	**BH12d**
Test Details: Packer Type:	**Single**	Borehole Diameter:	**93 mm**	Depth from:	**104.00 m**
Type of Pump:	**Rotary**	Borehole Inclination:	**-90**	Depth to:	**110.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height:	**0.4 m** Depth to g'water: **54 m**	Static Pressure:	**533.7 kPa**	Test Midpoint (m):	**107.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter)			Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
		Start (L)	Finish (L)	Total (L)				
5	230	165.9	169.15	3.25	0.65	0.0	763.7	0.11
5	230	169.15	172	2.85	0.57	0.0	763.7	0.09
5	460	179.1	180.5	1.4	0.28	0.0	993.7	0.05
5	460	180.5	181.25	0.75	0.15	0.0	993.7	0.03
5	700	185.8	186.05	0.25	0.05	0.0	1233.7	0.01
5	700	186.05	185.8	-0.25	-0.05	0.0	1233.7	-0.01
5	460	185.45	185.15	-0.3	-0.06	0.0	993.7	-0.01
5	460	185.15	184.7	-0.45	-0.09	0.0	993.7	-0.02
5	230	184.3	184.8	0.5	0.10	0.0	763.7	0.02
5	230	184.8	185.5	0.7	0.14	0.0	763.7	0.02





Interpreted Burgess Permeability : <0.5 μL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 μL

Based on Houlsby, 1976

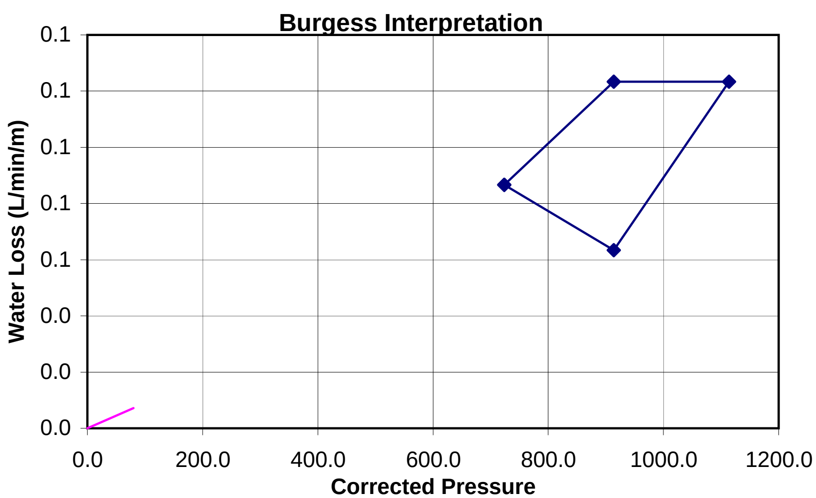
BOREHOLE WATER PRESSURE TEST

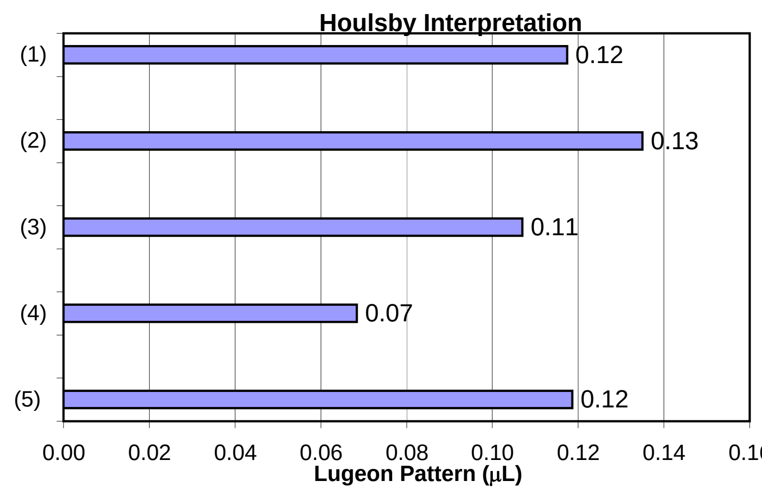
Job No:
Office:

Client: **Dial a Dump** Location: Reduced By: **MRB** **Borehole no:**
Project: **Lighthorse Landfill Site** Test Date: **09-Jun-09** Checked By: **BH12d**

Test Details: Packer Type: **Single** Borehole Diameter: **93 mm** Depth from: **114.00 m**
Type of Pump: **Rotary** Borehole Inclination: **-90** Depth to: **120.00 m**
Pressure Gauge & Serial No: **PG2** Borehole Azimuth: Test Depth Interval: **6.00 m**
Pressure gauge height: **0.4 m** Depth to g'water: **54 m** Static Pressure: **533.7 kPa** Test Midpoint (m): **117.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter)			Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
		Start (L)	Finish (L)	Total (L)				
5	190	403.2	405.7	2.5	0.50	0.0	723.7	0.08
5	190	405.7	408.3	2.6	0.52	0.0	723.7	0.09
5	380	413.6	417.3	3.7	0.74	0.0	913.7	0.12
5	380	417.3	421	3.7	0.74	0.0	913.7	0.12
5	580	428.75	432.45	3.7	0.74	0.0	1113.7	0.12
5	580	432.45	435.9	3.45	0.69	0.0	1113.7	0.12
5	380	435.8	437.7	1.9	0.38	0.0	913.7	0.06
5	380	437.7	439.55	1.85	0.37	0.0	913.7	0.06
5	190	437.6	440.2	2.6	0.52	0.0	723.7	0.09
5	190	442.65	445.2	2.55	0.51	0.0	723.7	0.09





Interpreted Burgess Permeability : <0.5 μL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 μL

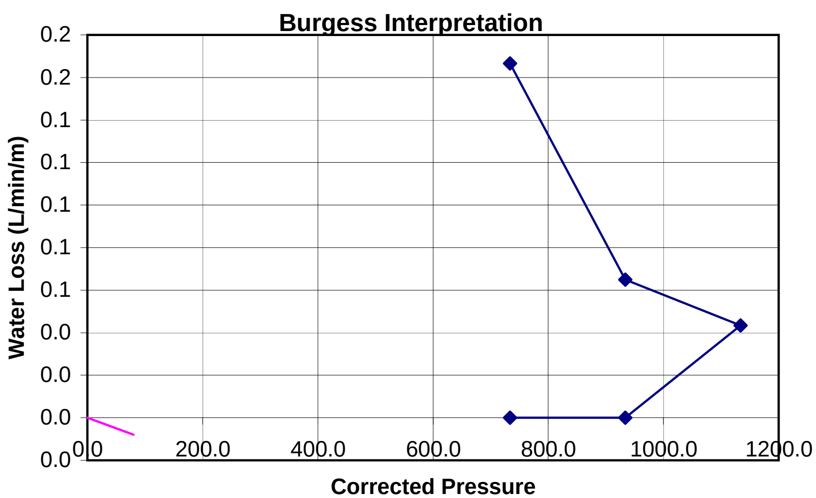
Based on Houlsby, 1976

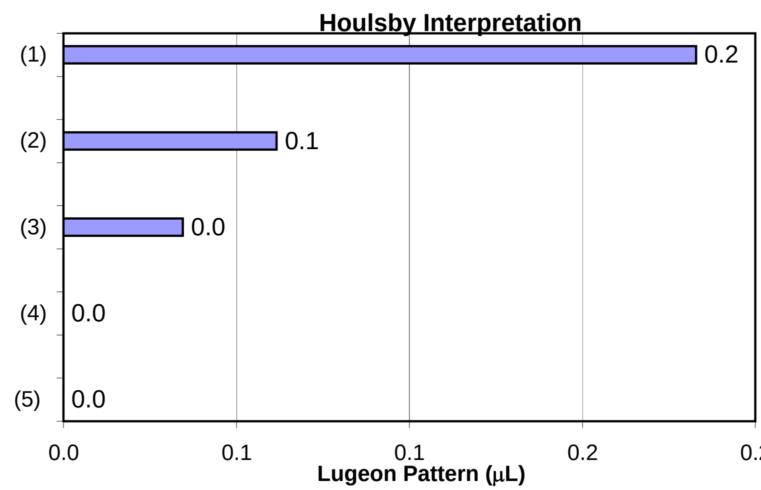
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client:	**Dial a Dump**	Location:		Reduced By: **MRB**	**Borehole no:**
Project:	**Lighthorse Landfill Site**	Test Date:	**10-Jun-09**	Checked By:	**BH12d**
Test Details: Packer Type:	**Single**	Borehole Diameter:	**93 mm**	Depth from:	**124.00 m**
Type of Pump:	**Rotary**	Borehole Inclination:	**-90**	Depth to:	**130.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.4 m**	Depth to g'water: **54 m**	Static Pressure:	**533.7 kPa**	Test Midpoint (m):	**127.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	200	526.1	531.1	5	1.00	0.0	733.7	0.17
5	200	531.1	534.15	3.05	0.61	0.0	733.7	0.10
5	400	545.2	547.15	1.95	0.39	0.0	933.7	0.06
5	400	547.15	548.65	1.5	0.30	0.0	933.7	0.05
5	600	551.6	552.9	1.3	0.26	0.0	1133.7	0.04
5	600	552.9	553.95	1.05	0.21	0.0	1133.7	0.04
5	400	552.55	552.55	0	0.00	0.0	933.7	0.00
5	400	552.55	552.55	0	0.00	0.0	933.7	0.00
5	200	552.2	552.2	0	0.00	0.0	733.7	0.00
5	200	552.2	552.2	0	0.00	0.0	733.7	0.00





Interpreted Burgess Permeability : <0.5 µL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 µL

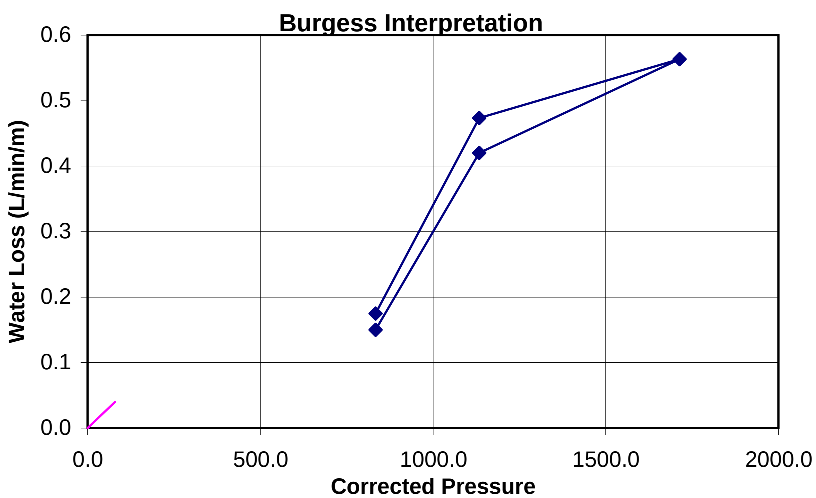
Based on Houlsby, 1976

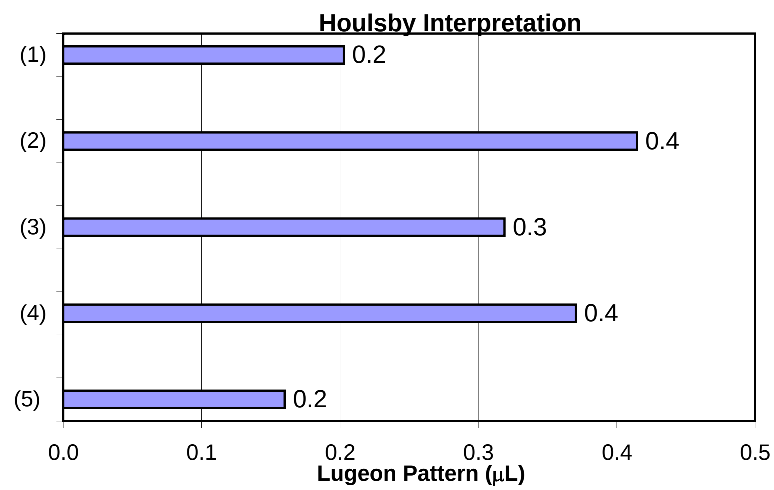
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client:	**Dial a Dump**	Location:		Reduced By: **MRB**	**Borehole no:**
Project:	**Lighthorse Landfill Site**	Test Date:	**10-Jun-09**	Checked By:	**BH12d**
Test Details: Packer Type:	**Single**	Borehole Diameter:	**93 mm**	Depth from:	**134.00 m**
Type of Pump:	**Rotary**	Borehole Inclination:	**-90**	Depth to:	**140.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.4 m**	Depth to g'water: **54 m**	Static Pressure:	**533.7 kPa**	Test Midpoint (m):	**137.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter) Start (L)	Finish (L)	Total (L)	Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
5	300	574.85	580.1	5.25	1.05	0.0	833.7	0.18
5	300	580.1	585	4.9	0.98	0.0	833.7	0.16
5	600	601.2	615.2	14	2.80	0.0	1133.7	0.47
5	600	615.2	629.4	14.2	2.84	0.0	1133.7	0.47
5	1180	650.6	667.5	16.9	3.38	0.0	1713.7	0.56
5	1180	667.5	683.4	15.9	3.18	0.0	1713.7	0.53
5	600	701.4	714	12.6	2.52	0.0	1133.7	0.42
5	600	701.4	714	12.6	2.52	0.0	1133.7	0.42
5	300	722.7	727.2	4.5	0.90	0.0	833.7	0.15
5	300	727.2	730.7	3.5	0.70	0.0	833.7	0.12





Interpreted Burgess Permeability : 0.5 µL

Based on Burgess, 1983

Interpreted Houlsby Permeability : <0.5 µL

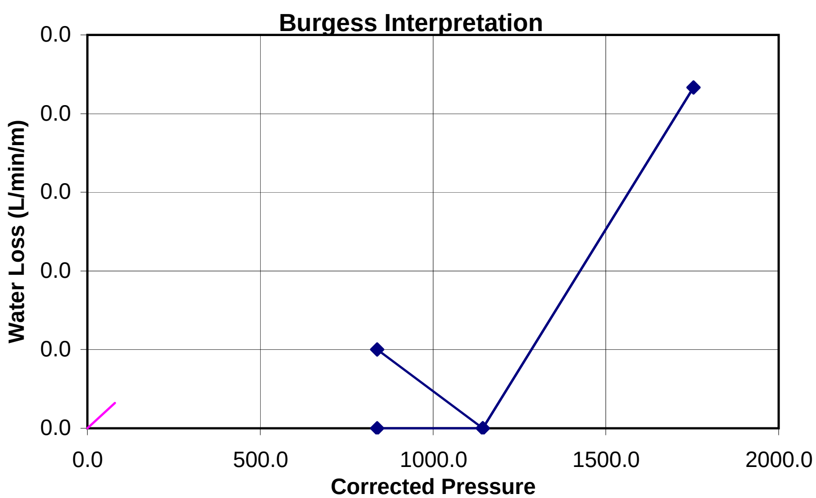
Based on Houlsby, 1976

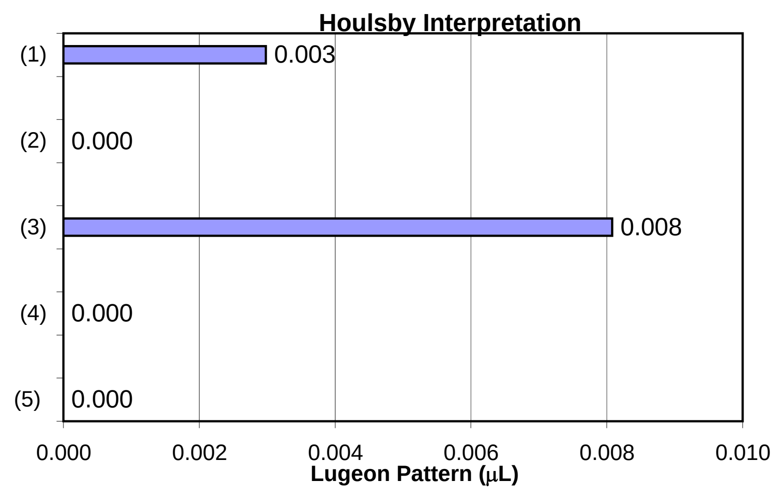
BOREHOLE WATER PRESSURE TEST

Job No:
Office:

Client:	**Dial a Dump**	Location:		Reduced By: **MRB**	**Borehole no:**
Project:	**Lighthorse Landfill Site**	Test Date:	**12-Jun-09**	Checked By:	**BH12d**
Test Details: Packer Type:	**Single**	Borehole Diameter:	**93 mm**	Depth from:	**144.00 m**
Type of Pump:	**Rotary**	Borehole Inclination:	**-90**	Depth to:	**150.00 m**
Pressure Gauge & Serial No:	**PG2**	Borehole Azimuth:		Test Depth Interval:	**6.00 m**
Pressure gauge height: **0.4 m**	Depth to g'water: **54 m**	Static Pressure:	**533.7 kPa**	Test Midpoint (m):	**147.00 m**

Time Interval (minutes)	Gauge Pressure (kPa)	Water Loss (litres - from flow meter)			Flow Rate (Litres/minute)	Head Loss (kPa)	Corrected Pressure (kPa)	Water Loss Rate (L/min/m)
		Start (L)	Finish (L)	Total (L)				
5	305	765.6	765.75	0.15	0.03	0.0	838.7	0.00
5	305	765.75	765.75	0	0.00	0.0	838.7	0.00
5	610	768.8	768.8	0	0.00	0.0	1143.7	0.00
5	610	768.8	768.8	0	0.00	0.0	1143.7	0.00
5	1220	774.55	774.75	0.2	0.04	0.0	1753.7	0.01
5	1220	774.75	775.4	0.65	0.13	0.0	1753.7	0.02
5	610	775.15	775.15	0	0.00	0.0	1143.7	0.00
5	610	775.15	775.15	0	0.00	0.0	1143.7	0.00
5	305	766.85	766.85	0	0.00	0.0	838.7	0.00
5	305	766.85	766.85	0	0.00	0.0	838.7	0.00





Interpreted Burgess Permeability : <0.5 µL

Based on Burgess, 1983

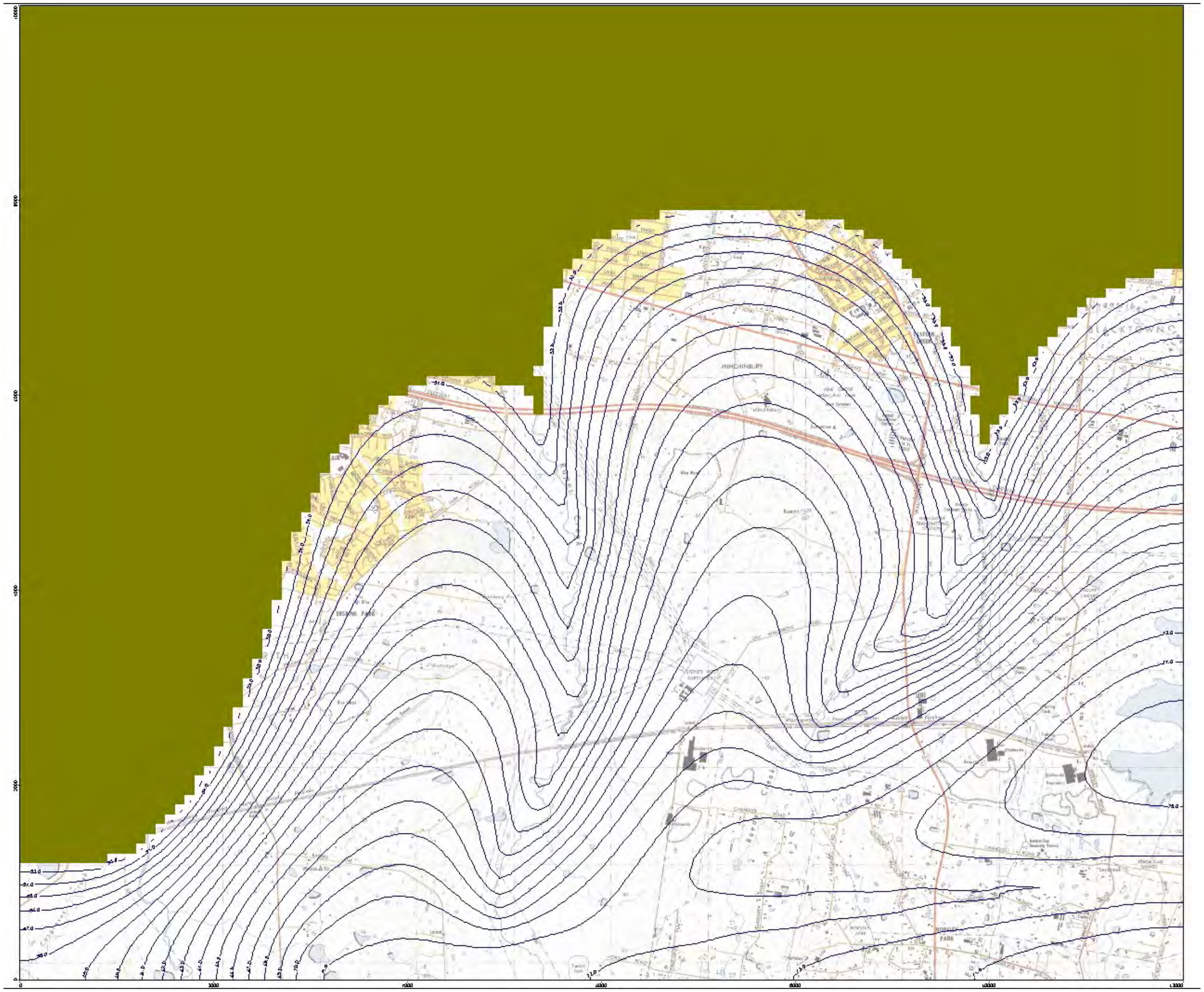
Interpreted Houlsby Permeability : <0.5 µL

Based on Houlsby, 1976

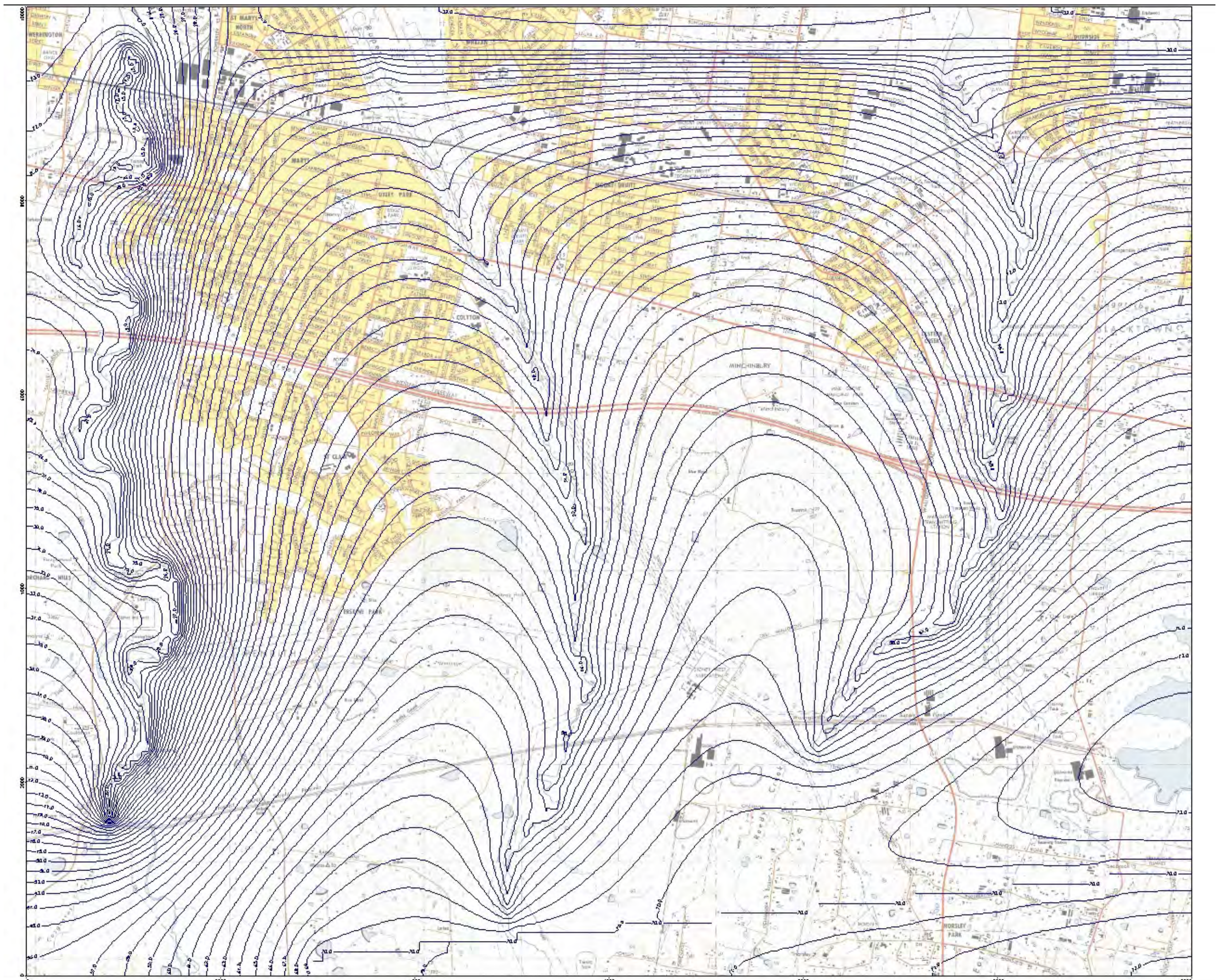
Appendix F
Numerical Model Outputs

Appendix F

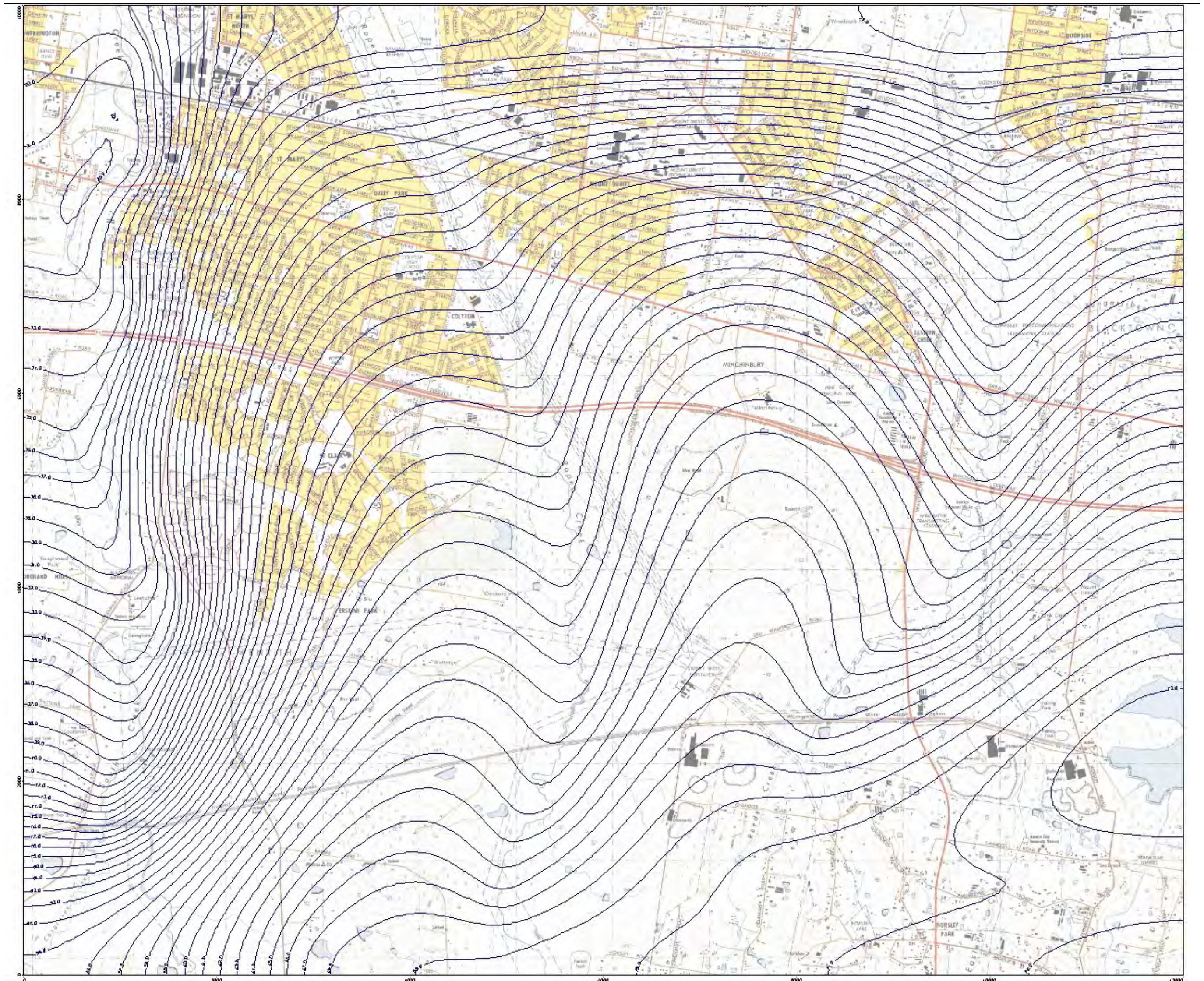
Pre-Quarry Conditions – Layer 1 – 1m Contour Intervals



Pre-Quarry Conditions – Layer 5 – 1m Contour Intervals



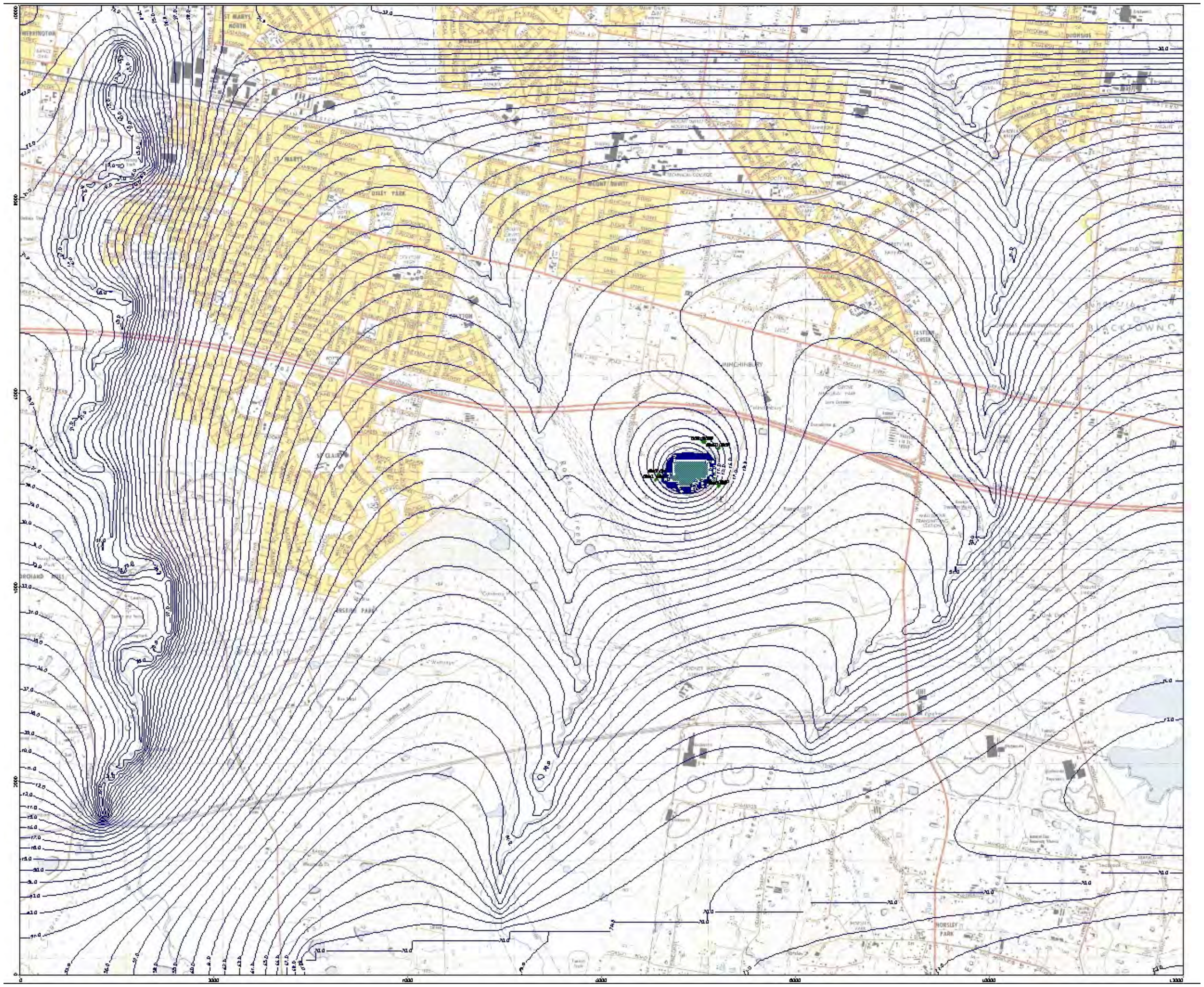
Pre-Quarry Conditions – Layer 10 – 1m Contour Intervals



Existing Conditions – Layer 1 – 1m Contour Intervals

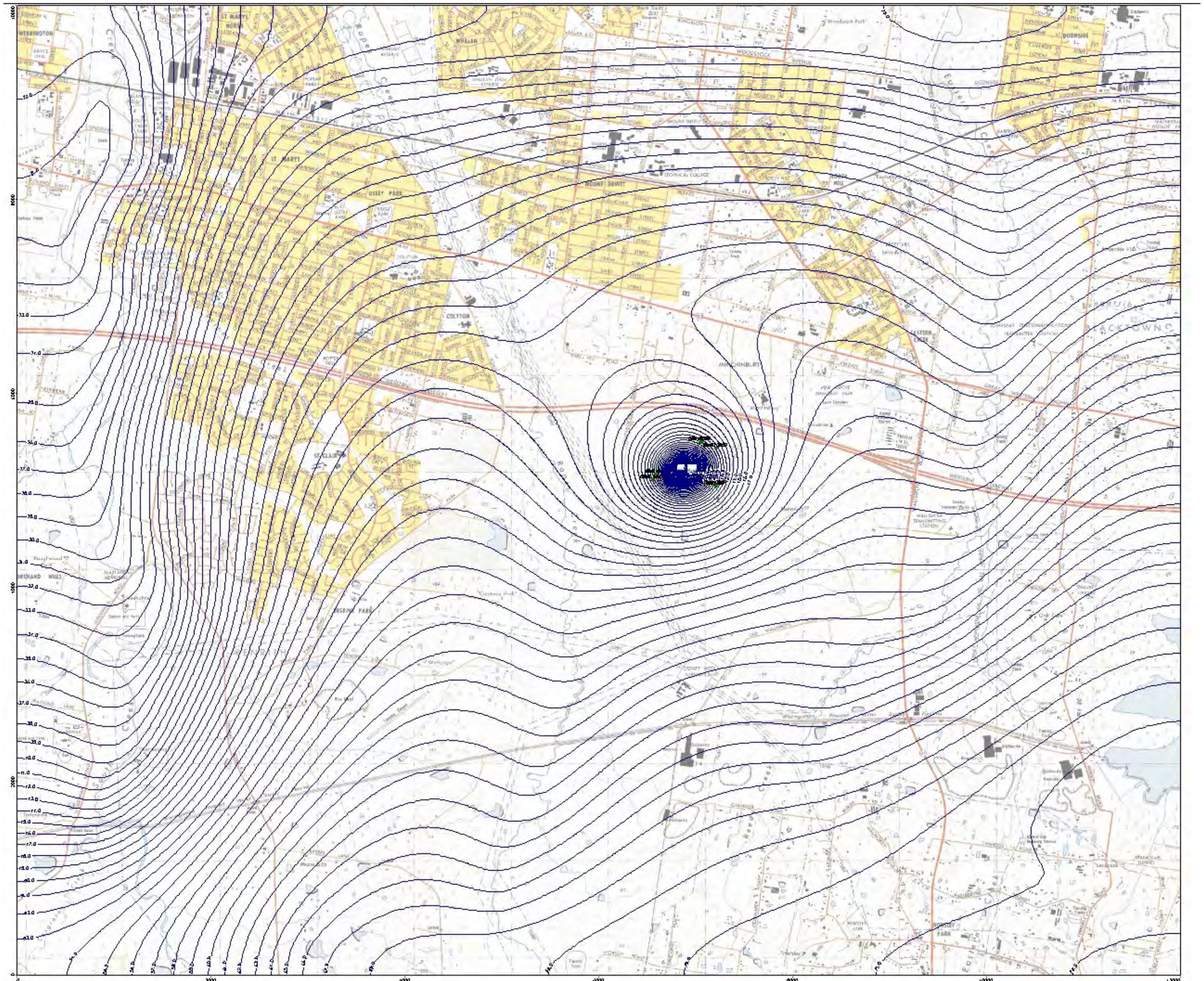
Appendix F

Existing Conditions – Layer 5 – 1m Contour Intervals

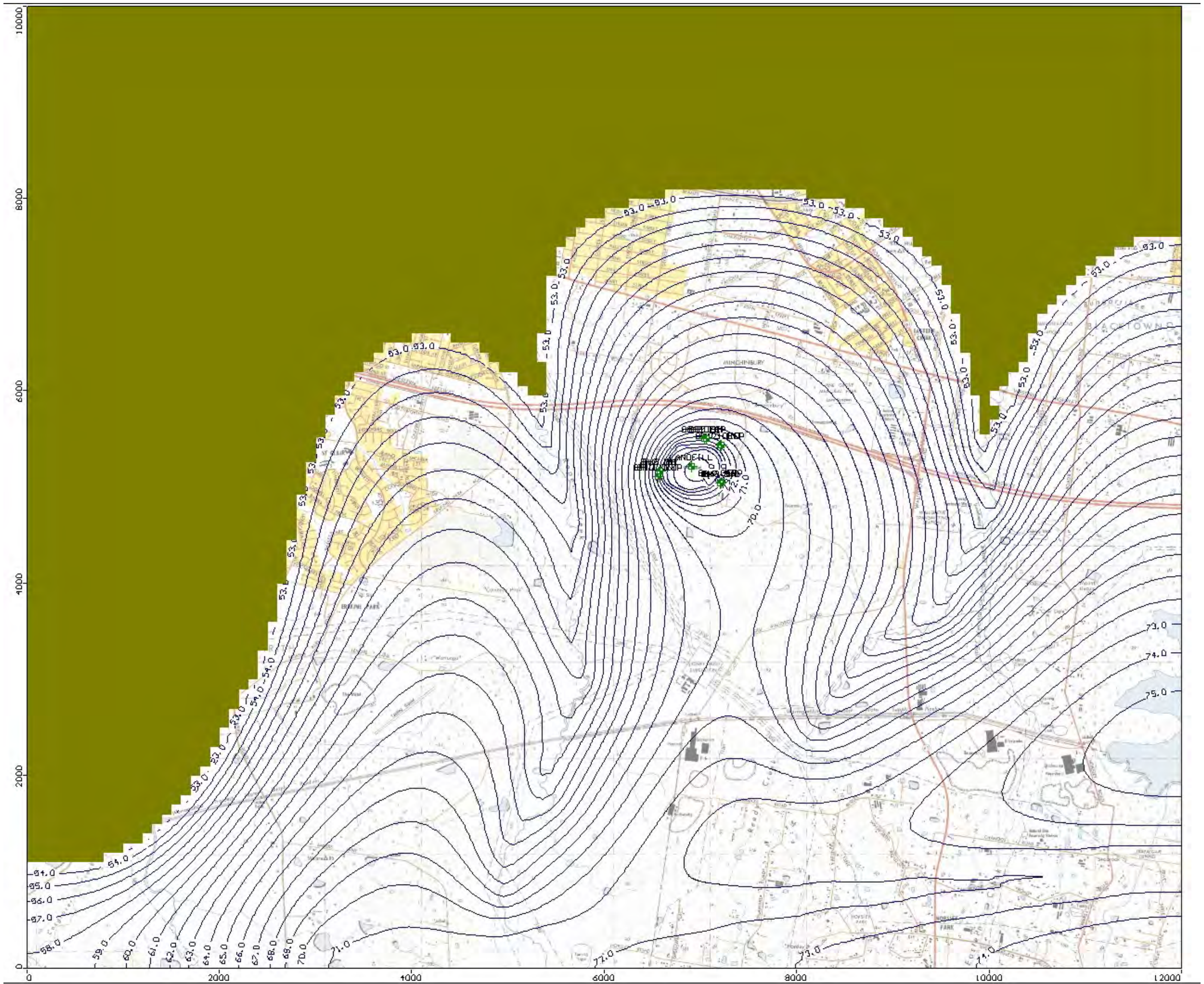


Appendix F

Existing Conditions – Layer 10 – 1m Contour Intervals

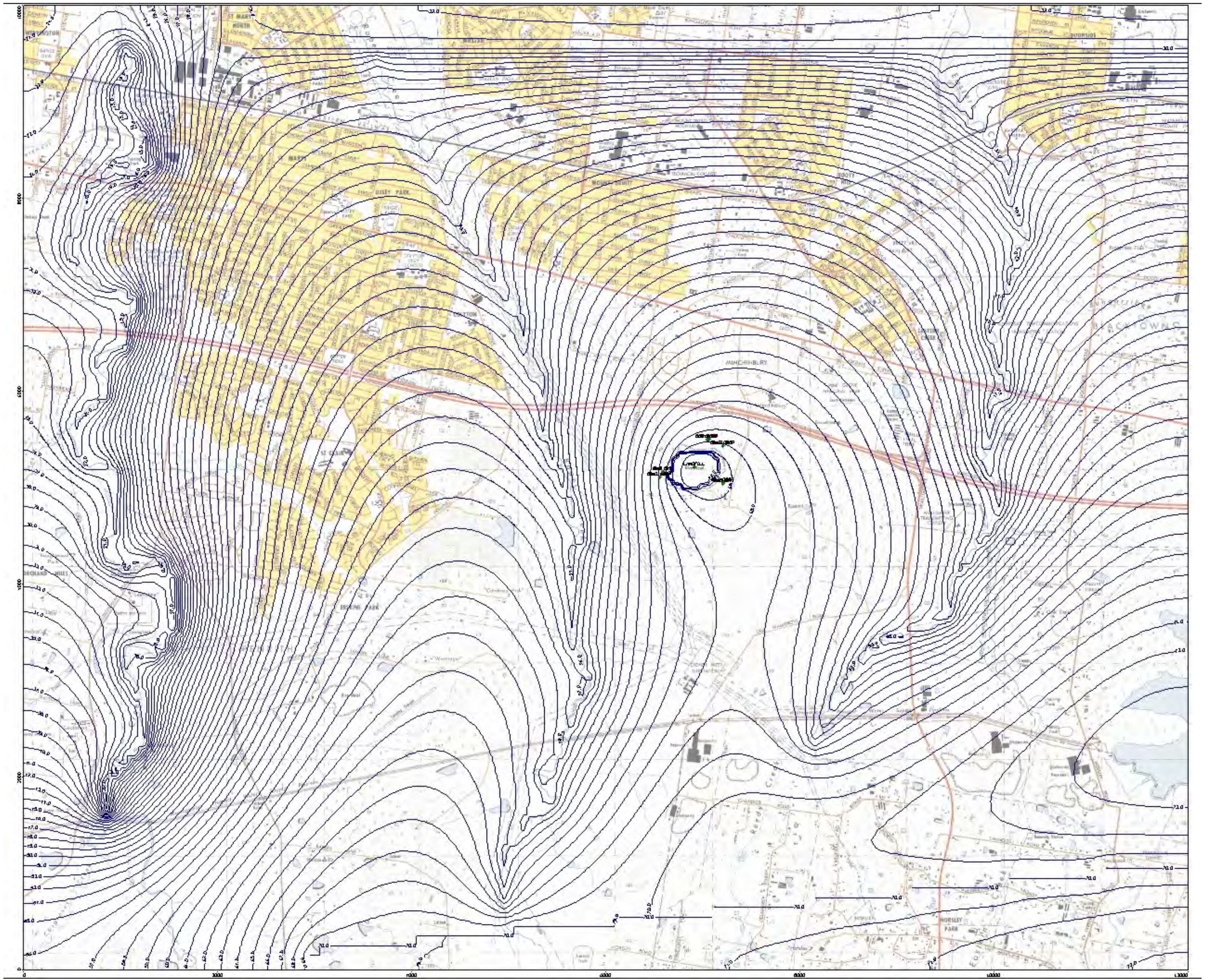


Final Repressurisation – Layer 1 – 1m Contour Intervals



Appendix F

Final Repressurisation – Layer 5 – 1m Contour Intervals



Final Repressurisation – Layer 10 – 1m Contour Intervals