CONCEPT DESIGN REPORT

STORM WATER MANAGEMENT

BELFAST PROJECT

SEPTEMBER 2011

REVISION 0

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VERIFICATION PAGE	Form 403.1 Rev 10/06

TITLE : CONCEPT DESIGN REPORT – STORM WATER MANAGEMENT AND DAMS

JG NO. : 002802	DATE : 19/09/2011	STATUS : Final

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SYNOPSIS :

This report covers the concept design of a new coal mine planned by Exxaro in the vicinity of Belfast to be used as part of the water use license application for the Belfast Project, Mpumalanga.

KEY WORDS :

Concept design report, storm water management, dam sizing and design, low level bridge, lining design, pans, spillways, culverts, drains.

QUALITY VERIFICATION

This report has been prepared under the controls established by a quality management system that meets the requirements of ISO9001: 2008 which has been independently certified by DEKRA Certification under certificate number 90906882

Verification	Capacity	Name	Signature	Date
By Author	Project Leader	CC le Roux		
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1. INTRODUCTION

Exxaro Resources Limited (Exxaro) appointed Jeffares & Green (Pty) Ltd (J&G) to undertake the conceptual design of a surface water runoff system for the proposed plant area of the Belfast Project.

The Belfast Project is located in Mpumalanga and approximately 10km southwest of Belfast on the farms Leeuwbank, Blyvooruitzicht and Zoekop.

Exxaro is evaluating the utilisation of its coal reserves at the site and has commissioned several studies to this effect. The conceptual design of surface water runoff is specifically required for the Water Use License Application (WULA) which would pave the way for further and more detailed studies to commence. Refer to Figure 1.

Figure 1: Site Location

2. PROJECT OBJECTIVES

The objective can be summarised as the development and design of a storm water drainage system in compliance with environmental and water management requirements and legislation as applicable to the mining industry in order to ensure a successful Water Use License Application (WULA).

The following main requirements must be met for compliance:

- Unpolluted water to be confined to a clean water system.
- Polluted water to be confined to a closed system (runoff and seepage).
- Polluted and unpolluted systems not to spill over more than once in 50 years.
- Systems to be fully serviceable for floods up to the 1:50 year.
- No infrastructure within the 1:100 year flood-line or within 100m from a water course, estuary etc.
- Minimum freeboard of 800mm above full supply level applicable to dams (unless otherwise specified in the relevant act).
- The effect of any watercourse diversions and runoff reductions to be minimised.
- To comply with Dam Safety Regulations.
- To comply with regulations on the use of water for mining, Government Notice 704.

3. EXECUTION METHODOLOGY

The execution methodology followed is:

- Gather available information:
  - Topography and digital terrain model (DTM).
  - Previous reports relating to surface water, water balance, hydrology etc.
  - Previous reports relating to environmental issues.
  - Relevant legislation.
  - Client specific requirements.
- Confirm scope of work:
  - Determine the size and layout of clean and dirty water drains and specify erosion protection.
  - Determine the size of Storm Water Dams, specify protection and design spillways.
  - Design low level crossing and determine impact on flood-lines.
  - Propose sewerage treatment plant type and size.
  - Design and determine the size of a Biofilter Dam.
  - Design Pollution Control Dams (certified dam engineer where appropriate).
  - Design linings appropriate to hazard.
  - Write report and compile drawings.
  - Design of pumping systems is excluded.
  - Design recommendations for Discard Facility in compliance to WULA regulations.
  - Stockpile design recommendations for compliance to WULA regulations.
- Site visits.
- Carry out geotechnical testing.
- Confirm plant layout.
- Provisional sizing and layout to be checked for space constraints and adjustments to be made if required.
- Design, drawings and report complete with options and recommendations.

- Internal review and submit to Exxaro.
- Exxaro to exercise options.
- Adjust, peer review and submit final documents to Exxaro.

4. LEGAL FRAMEWORK

The legal requirements as summarised in the Best Practice Guidelines issued by the Directorate: Resource Protection & Waste of the Department: Water Affairs and Forestry were referenced. The following sections of the Best Practice Guidelines are of specific relevance:

- G1 – Storm Water Management, Section 5 (DWAF-G1, Aug 2006)
- A2 – Water Management for Mine Residue Deposits, Section 5 (DWAF-A2, Jul 2008)
- A4 – Pollution Control Dams, Section 5 (DWAF-A4, Aug 2007)
- A5 – Water Management for Surface Mines, Section 4 (DWAF-A5, Jul 2008)

Of the Acts referred to in the Guidelines, the following form the backbone of the framework:

- National Water Act, 1998 (Act No. 36 of 1998)
- Government Notice No. 704, 4 June 1999 (Regulations on the use of water for mining)
- Government Notice R.1560 of 25 July 1986 (Dam Safety Regulations)
- National Environment Management Act, 1998 (Act No. 107 of 1998)

5. SITE DESCRIPTION

5.1 TOPOGRAPHY

The plant is situated on the western banks of the Klein-Komatirivier with a Return Water Dam on the eastern bank. The topography of the western bank slopes gently (<3%) towards the river and the eastern bank is more steep (8%) but then flattening off above the 1770 contour at the dam site.

At the time of the site visit, July 2011, the majority of the area was used for cattle grazing. The area is covered by grassland (Figure 2) and there is no evidence of recent crop cultivation except in the Discard Facility area. Refer to Figure 3.

Plantations of wattle, blue gum and pine trees have been planted to the north and east of the proposed site with portions of the Discard Facility and Plant areas occupied by stands of trees. Refer to Figure 3.

Figure 2: General Topography and Vegetation on Site

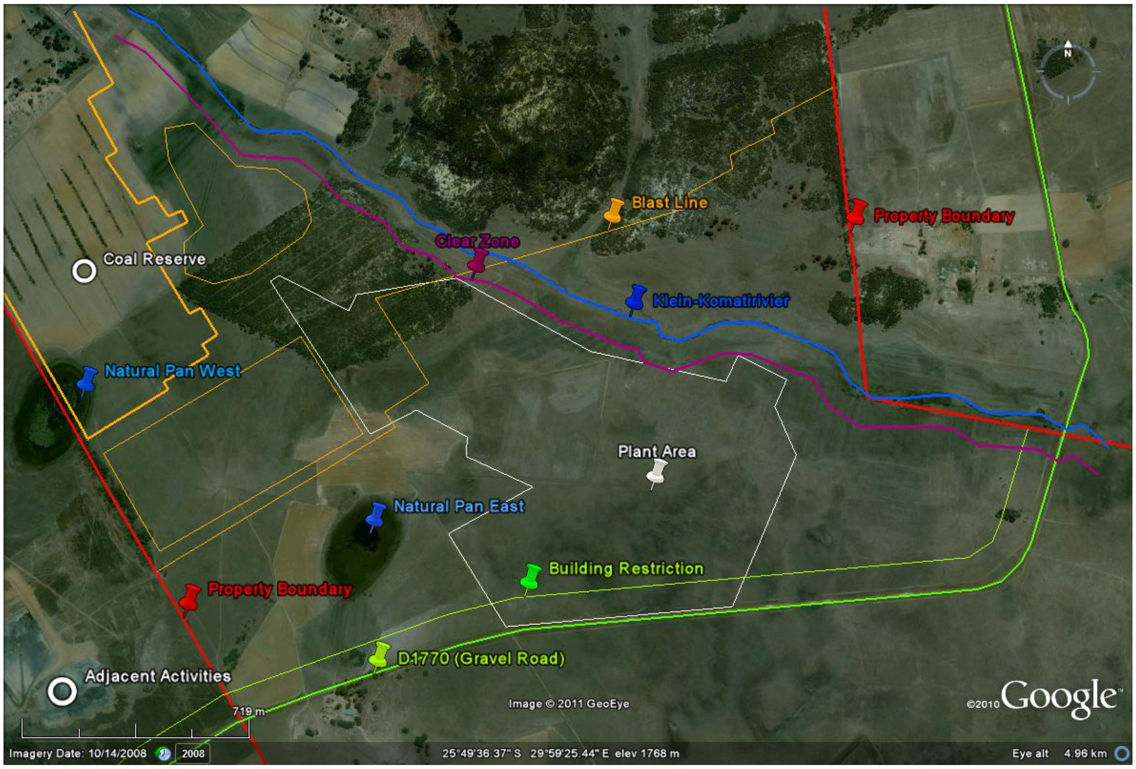
Figure 3: Google Image of Site

5.2 CONSTRAINTS

The plant area is restricted by (Figure 4):

- Coal reserves to the north and east (blast lines and mining operations).
- Klein-Komatirivier to the west (the 1:50 and 1:100 flood-lines and a 100m clear zone from the stream centre).
- Property boundary to the east.
- Provincial road to the south (building line restriction of 95m from the road centre line).
- Two pans to the west.

Figure 4: Plant Area Limitations



All the above is fixed with the exception of the building line. Following discussions with the regional office of the Roads Department, the building line restriction for this class of road can be relaxed to 16m measured from the edge of the road reserve. The road reserve for road D1770 is 25m and the effective distance is now 28,5m. A formal application is in process.

The original plant layout (Aurecon, Jun 2011) was revised to shift the plant layout as far as possible away from the wetland and flood-lines. This updated layout (Drawing 002802-BP-1) was used as the basis for the concept design.

6. SITE HYDROLOGY

6.1 RAINFALL RECORDS

The Roodepoort weather station (No. 0516554) records as provided by the South African Weather Service (SAWS) span a period of 98 years and is the closest to the Belfast Project site.

The same station and values were used in the report *Belfast Surface Water Assessment* (Golder and Associates, 2009).

The Mean Annual Precipitation for the Roodepoort station is 690mm and the 24hr rainfall depths for various recurrence intervals as presented in Table 1 were used.

Table 1: 24hr Rainfall depths

Recurrence Intervals (Years)	2	5	10	20	50	100	200
24 hr Rainfall Depth (mm)	58	77	90	104	123	137	153

A different method was applied by GCS in their report *Glisa Hydrological Study* (GCS, Jan 2011). Runoff data from the WR2005 database (WRC, 2008) was used in the report. The Glisa colliery is also owned by Exxaro and is west of Belfast. The MAP for Glisa is slightly higher at 714mm.

As the catchment area is in the headwaters of the Komati River and relatively small, preference was given to the historic rainfall data.

6.2 EVAPORATION DATA

The mean monthly pan evaporation rates for both the Belfast Project (Golder and Associates, 2009) and Glisa Colliery (GCS, Jan 2011) can be found in the Table 2 below:

Table 2: **Average Monthly Evaporation Rates**

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Tot
Belfast Pan Evaporation (mm)	138	138	156	164	140	138	104	91	75	81	102	124	1451
Glisa Pan Evaporation (mm)	189	169	163	122	106	88	93	129	175	195	185	200	1814

6.3 HYDROLOGY

6.3.1 CATCHMENTS AND FLOOD ESTIMATION

The Klein-Komatirivier catchment is in the upper reaches of the Komati River. At the position of the site, the total catchment area is approximately 23km^2 which is relatively small.

Floods can be defined in two ways depending upon the application:

- The 24hr runoff depth (section 6) is used to determine the sizes of Storm Water Dams where storage capacity (flood volume) is critical (DWA requirement).
- Flood Peak Methods (section 6.3.2) are used to calculate the sizes of drains, culverts, spillways and silt traps where peak flows exceed the 24hr runoff requirement. These methods relate to events irrespective of the duration of these events.

6.3.2 FLOOD PEAK METHODS

Methods can be broadly classified as statistical, deterministic or empirical. As the catchment area is relatively small, deterministic methods are deemed more appropriate.

It is good practice to use more than one method to get confidence in the flood peak value. The methods considered are discussed in the following paragraphs.

6.3.2.1 Rational Method

The Rational method is probably the most widely used method for the calculation of peak flows for small catchments. It was first proposed in 1851 by the Irish engineer Mulvaney (SANRAL, 2006). It is recommended for use in catchments up to 15km^2, but can be used by experienced engineers for much larger areas, especially where verified with another method.

6.3.2.2 Standard Design Flood

The Standard Design Flood method was developed by Alexander (Alexander, 2002) and is based on the Rational Method. It can be described as a numerically regionally calibrated version of the Rational Method, but is more robust and less site specific.

6.3.2.3 Unit Hydrograph

This method is recommended for areas between 15 and 5 000km^2 and is set out in detail in Report 1/72 of the Hydrological Research Unit, University of Witwatersrand (Witwatersrand, 1972). It is a time consuming method and problematic to apply to short storm durations.

6.3.2.4 Deterministic Method

The empirical peak flow calculations for rural areas as developed by Midgley and Pitman (SANRAL, 2006) were used. The results are likely to be less accurate and should be adjusted subjectively. It is based upon flow measurements and these are seldom available for catchments smaller than 10km^2 and usually only for catchments bigger than 100km^2.

6.3.3 FLOOD VOLUME ESTIMATION

Flood volumes are most accurately determined from flow stations for bigger catchments. Where not applicable, the unit hydrograph is recommended where the hydrograph shape is of importance. The Rational and SDF methods assume a triangular hydrograph. These methods are applicable to single storm events.

The requirements calls for 1:50 year 24hr runoff which is best calculated using statistical data from the SAWS.

6.4 LIMITATIONS

All hydrology calculations are based upon contour information (0,5m intervals) and a Lidar survey provided by Exxaro. No flow data is available close to this section of the Klein-Komatirivier, and estimated Manning "n" values for the river and banks were used in the calculations.

Topographical maps (1:50 000) of the Chief Directorate of Survey and Mapping were used for areas outside that covered by the Lidar survey.

7. SURFACE GEOLOGY

The dam sites are underlain by rock units of the Dwyka Group of the Karoo Supergroup and the area to the east is underlain by rock units of the Vryheid Formation.

Fieldwork was carried out in June 2011 and a detailed report can be found in Annexure G.

7.1 SUMMARY BY AREA

Below is a general summary of this report with corresponding paragraph letters for similar properties.

7.1.1 RETURN WATER DAM WEST

a) Test pits 1 to 10 carried out by hand auger to depths of between 0,20 and 1,05m.

b) DCP refusal between 0,15 and 1,00m with slow advance to 1,5m at one position.

c) Moist to wet, loose, gravelly, silty sand (colluvium) transitioning to honeycomb ferricrete.

d) Hardpan ferricrete observed in outcrops.

e) Standing water at lower end of site with little to no seepage higher up.

7.1.2 RETURN WATER DAM EAST

a) Test pits 11 to 19 were excavated by TLB to depths of between 0,15 and 1,15m.

b) DCP refusal between 0,06 and 0,87m.

c) Dry to moist, loose, gravelly, silty sand (colluvium) transitioning to honeycomb ferricrete or even hardpan ferricrete.

d) Hardpan ferricrete observed in outcrops and possibly underlain by weathered sandstone in places.

e) Slight ground water seepage in 2 test pits and none in the other 7 pits.

7.1.3 DISCARD AREA

a) Test pits 20 to 23 were excavated by TLB to depths of between 0,12 and 1,80m.

b) DCP refusal between 0,15 and 1,90m (apparatus limit).

c) Dry to moist, loose, gravelly, silty sand (colluvium) transitioning to honeycomb ferricrete in two pits and residual sandstone in the other two pits.

d) Hardpan ferricrete not observed although shallow at one test pit position.

e) No ground water seepage, but moisture does increase with depth.

7.2 PROBLEMATIC SOILS

7.2.1 CLAY AND HEAVE

The soils are predominantly sandy with low Plasticity Index (PI) if any and “low” heave potential (low clay content). PI varies from Non Plastic (NP) to 10.

7.2.2 COLLAPSIBLE SOILS

Further testing should be done as soils encountered in TP23 were described as “open ended”. This could be a potentially collapsible material. This test pit is in the Discard Facility area.

7.2.3 AGGRESSIVENESS

Soils were found to be highly aggressive towards concrete and highly corrosive towards metals. This nature of the soil must be taken into account in the design of any buried structures.

7.3 SUITABILITY OF SITES

This preliminary geotechnical investigation indicates that the sites are suitable for the construction of two main earth fill Return Water dams (East and West) provided that the recommendations are implemented:

- Move the Return Water Dam West in a southerly direction as far as possible.
- Take into account the aggressiveness of the soils for any ground touching structures.
- Conduct further investigations for detail design purposes.

8. CLEAN AND DIRTY WATER SEPARATION

8.1 THE REGULATION

In broad terms, Government Notice No. 704, 4 June 1999, inter alia requires the following:

Unpolluted water should be confined to a clean water system away from dirty water areas and polluted water inclusive of runoff and seepage should be confined to a closed system, not affecting clean water. The mentioned systems may overspill only once in 50 years and should remain serviceable (maintained) for this event.

8.2 SEPARATION SYSTEM

The plant area is situated next to the Klein-Komatirivier and two pans are located to the west of the plant (refer to Drawing 002802-BP- 1). Pans form in low lying areas that cannot drain freely. The challenge is not to contaminate these clean water areas and to divert any overflow to the river although the natural flow path is through a dirty area.

The proposed system is for diverting water as close as possible to its natural drainage path while still complying with the maximum of 1:50 year spillage requirement. This drainage path through the plant should be designed such that it can accommodate the runoff and that there is no possibility for accidental or otherwise contamination with polluted water that may occur during the operational phase of the mine.

The proposed catchment areas of the clean and dirty water systems are indicated on Figure 5 below. Refer to Drawing 002802-BP- 2 for more detail.

8.3 DESIGN APPROACH

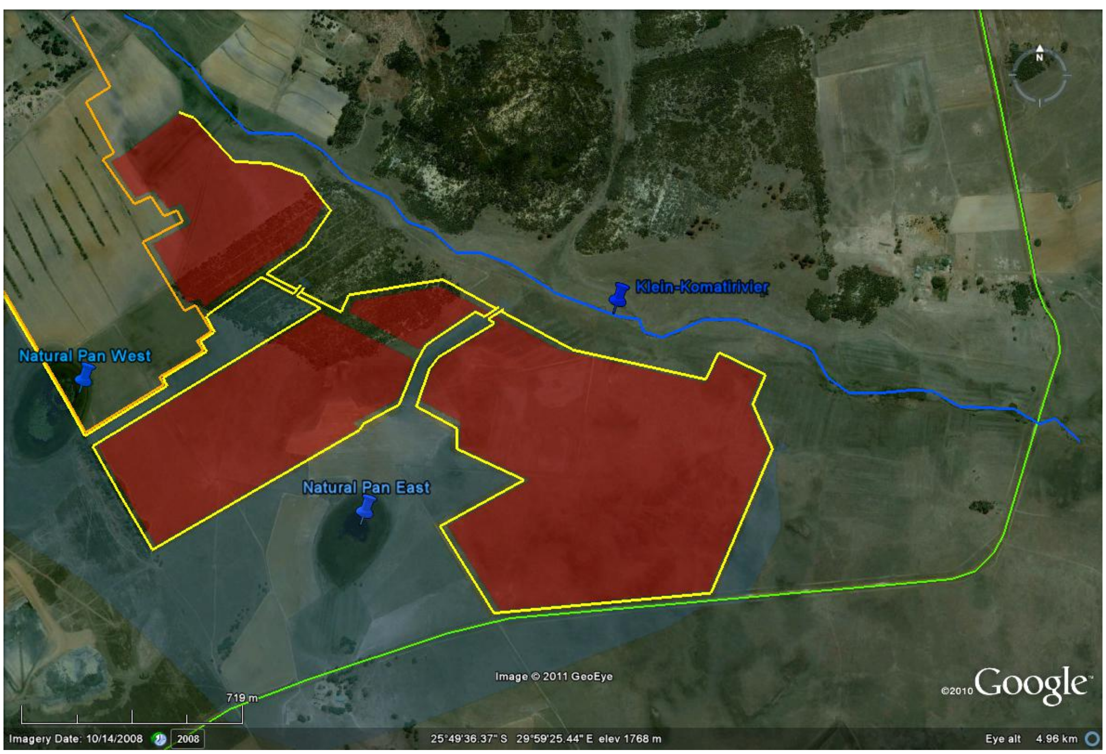
8.3.1 SYSTEM PERIMETER

In compliance with the regulations, the outer perimeter of the systems is designed to ensure there is no contamination within a 50 year recurrence period. The outer perimeter consists of a combination of open drains and berms as well as roads with safety berms and side drains.

The cut-off drains and berms that form this perimeter can be seen as yellow lines on Figure 6 below.

Figure 5: Clean (Blue) and Dirty Water (Red) Catchment Areas

Figure 6: Cut-Off Drains and Berms (Yellow lines)



8.3.2 INNER SYSTEM

The inner system need not be designed for the 1:50 year recurrence as it is contained within the outer perimeter. The inner system design is a function of safety, operational and maintenance requirements.

The minimum recurrence interval used for the design is 10 years although a lower figure can be used. This recurrence interval is deemed appropriate as maintenance requirements necessitate bigger culverts and drains, and the additional cost is minimal.

During detail design, these assumptions should be revisited as critical areas may require adjustments. Critical areas include high value, high operational risk as well as areas where safety can be an issue.

9. CLEAN WATER SYSTEM

9.1 PANS

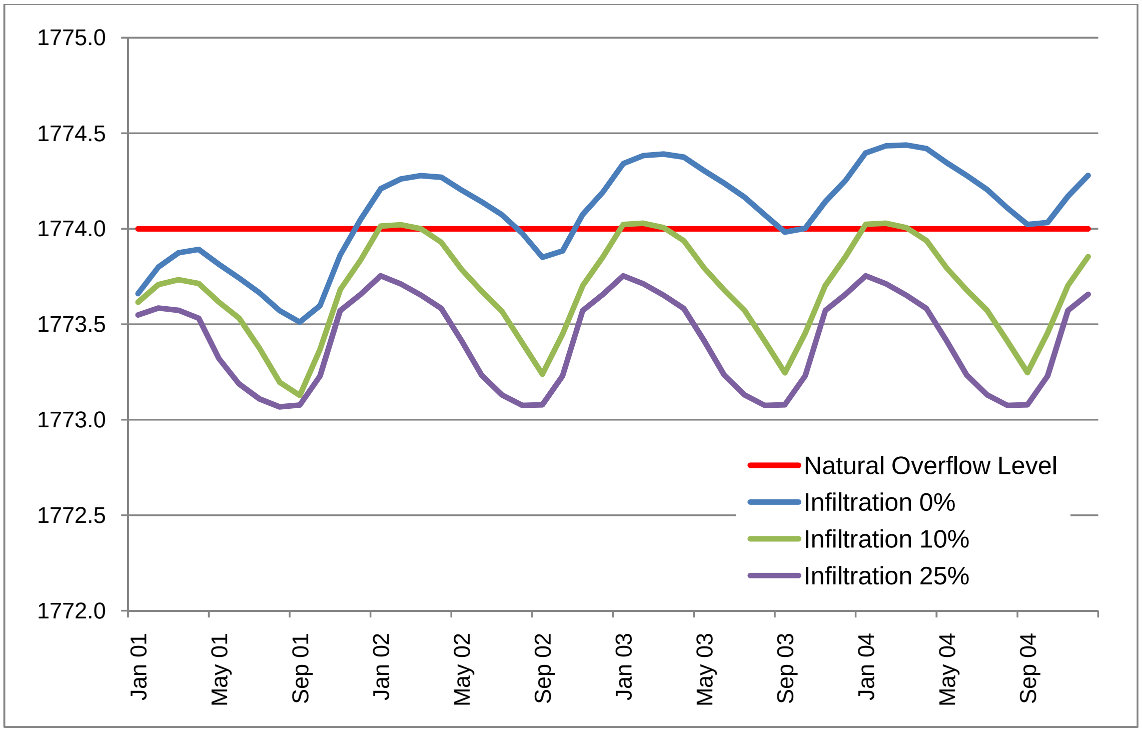
The clean water system incorporates two Natural Pans located to the west of the plant (Figure 7) referred to as Natural Pan East and Natural Pan West. Refer to Drawing 002802-BP- 2 to see the pans in relation to the clean and dirty water areas as well as the Mining Area.

Figure 7: Natural Pan Positions

9.1.1 NATURAL PAN EAST

The Natural Pan East has a relatively low natural overflow level and a simulation was done based upon average precipitation, pan evaporation and a variable infiltration rate. The results in Figure 8 confirm that it is appropriate to assume that the pan may be full prior to the 1:50 year event.

Figure 8: Natural Pan East – Balance Model



9.1.2 NATURAL PAN WEST

The Natural Pan West is intersected by the property boundary as well as the mining perimeter as evident on Figure 7. There are neighbouring mining related activities taking place to the west of the pan. To mitigate his clash, the Mining Area can be reduced or the pan can be divided by a wall.

A modelling was carried out for the two scenarios:

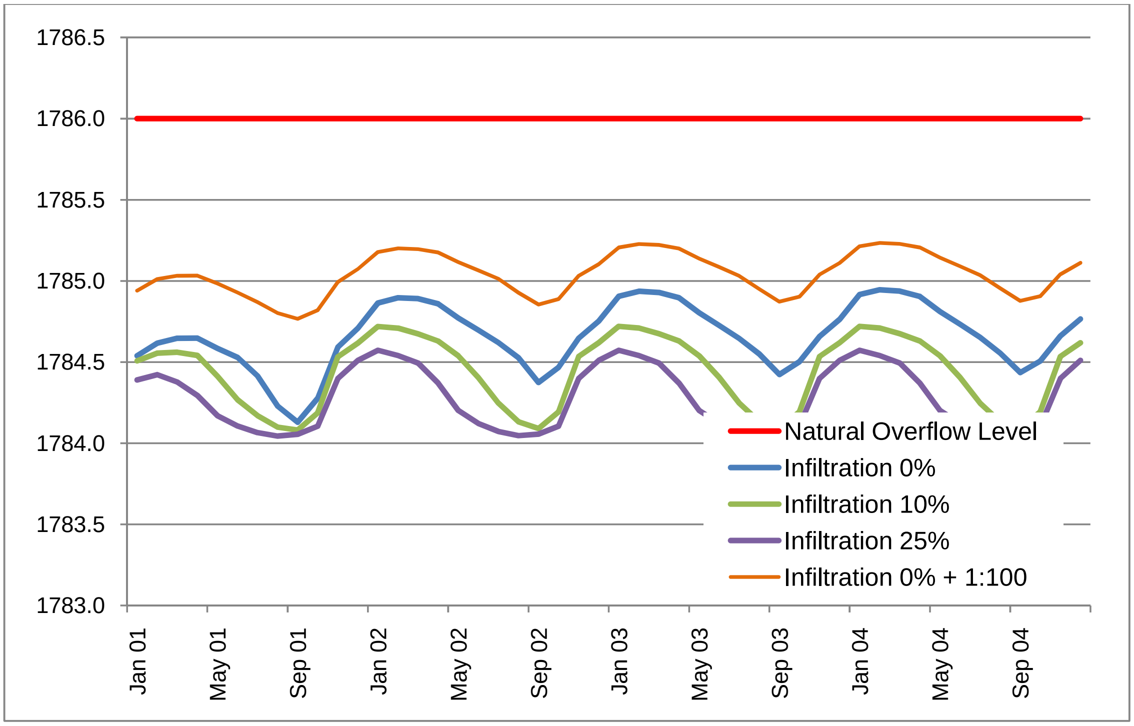
a) Undivided

The first scenario is the reduction of the Mining Area. The pan will remain in its natural state but coal reserves will be sterilized.

From Figure 9 it is clear that the pan will not overflow even if there is no infiltration. No overflow structure is required for this scenario.

The possibility of retaining the Natural Pan West was investigated, but this will result in the sterilization of coal reserves. Given the impact on the reserves and the neighbouring mining activities, this scenario is not regarded as desirable.

Figure 9: Natural Pan West Undivided – Balance Model

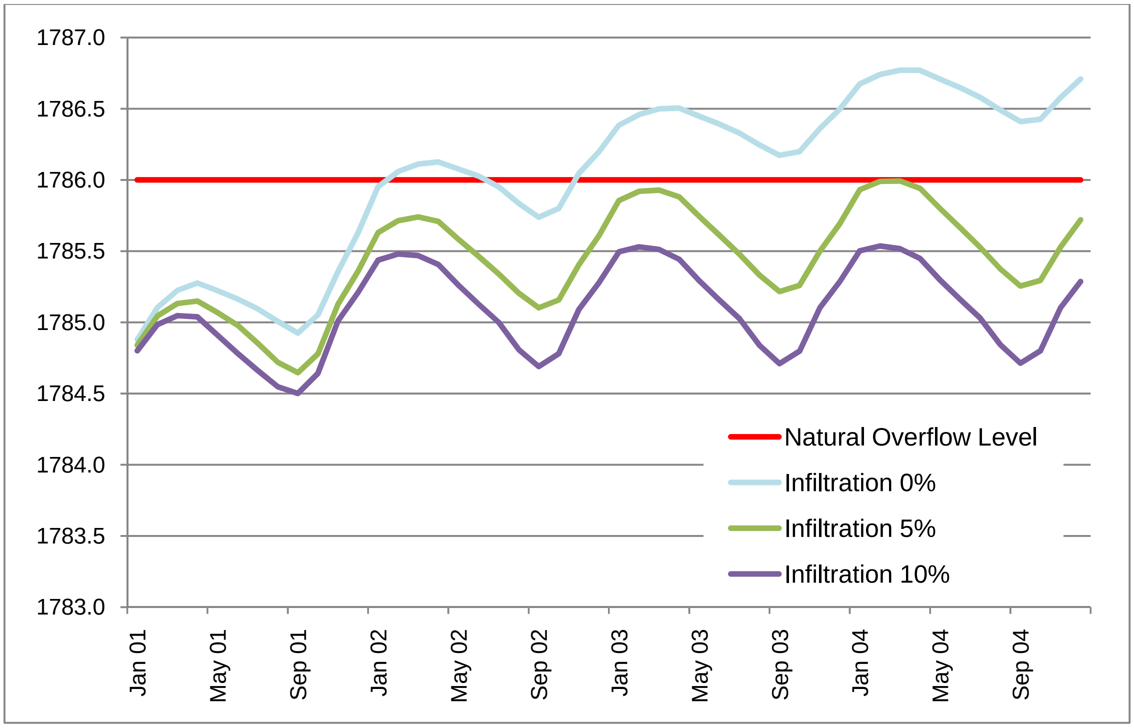


b) Divided

The second scenario is of the pan being divided by a wall on the property boundary. The eastern half will then fall in the Mining Area.

The model in Figure 10 confirms that the pan may be at the overflow level at the onset of the 1:50 year event (assuming infiltration is less than 5%) and an overflow should be provided.

Figure 10: Natural Pan West Divided – Balance Model



Scenario b) was assumed for the concept design. This entails the construction of a wall with a freeboard of 800mm minimum and a spillway and drain to cater for a 1:50 year event. It has the advantage of not sterilizing coal reserves and any overflow would be diverted as clean water to the Klein-Komatirivier.

9.1.3 DISCHARGE

The runoff from the clean water areas is conveyed by a system of drains and culverts through or around the dirty areas.

Where clean water has to be diverted through the dirty areas, it is protected by a berm on all sides of sufficient height to prevent cross contamination for a 50 year recurrence event. Refer to nodes C4, C5a up to C5b on Drawing 002802-BP- 1.

10. DIRTY WATER SYSTEM

10.1 DIVISIONS

Given the topography of the area and to reduce risk, the dirty water system is divided into three separate areas:

- Hard and Soft Stockpile Area (D1)
- Discard Facility (D2)
- Plant Area (D5)

Refer to Figure 5 for the clean and dirty water catchment areas. More detail can be found on Drawing 002802-BP- 2. Points D1, D2 and D5 indicate dams for collecting runoff.

10.2 SEDIMENTATION

A network of open drains collects water from the dirty areas and discharges the runoff into the Storm Water Dams.

To protect Storm Water Dams from sedimentation, silt traps are located as close as possible to the source of contamination. Some drainage lines may pass through more than one silt trap.

Silt traps are also provided next to all Storm Water Dams with a side overflow into the dams. Alternative arrangements can be considered during detailed design e.g. Figure 11 (DWAF-A4, Aug 2007).

10.3 HAUL ROAD DRAINAGE

Sections of the haul roads pass through clean water areas and to simplify the drainage network and to reduce maintenance requirements, berms and drains have been integrated into the roads.

Over sections, the roads are designed with superelevation discharging all runoff to the one side where it is collected in a side drain and drained away to dirty water systems. Superelevated sections should be kept short and be an extension of superelevation around a curve. Where this is not the case, the driver may feel uneasy and with regular maintenance, the tendency is for these areas to be graded back to a camber.

Berms and side drains form part of the safety measures of haul roads and is not an additional cost. When properly designed and maintained, they are extremely effective as part of the drainage network.

Refer to Drawing 002802-BP- 5 for a typical superelevated cross-section of a haul road.

The yellow lines in Figure 12 indicate the section of haul roads where a combination of superlevation, berms and channels are used to divert polluted runoff back to dirty areas.

Figure 11: Silt Trap Incorporated into Dam(DWAF-A4, Aug 2007)

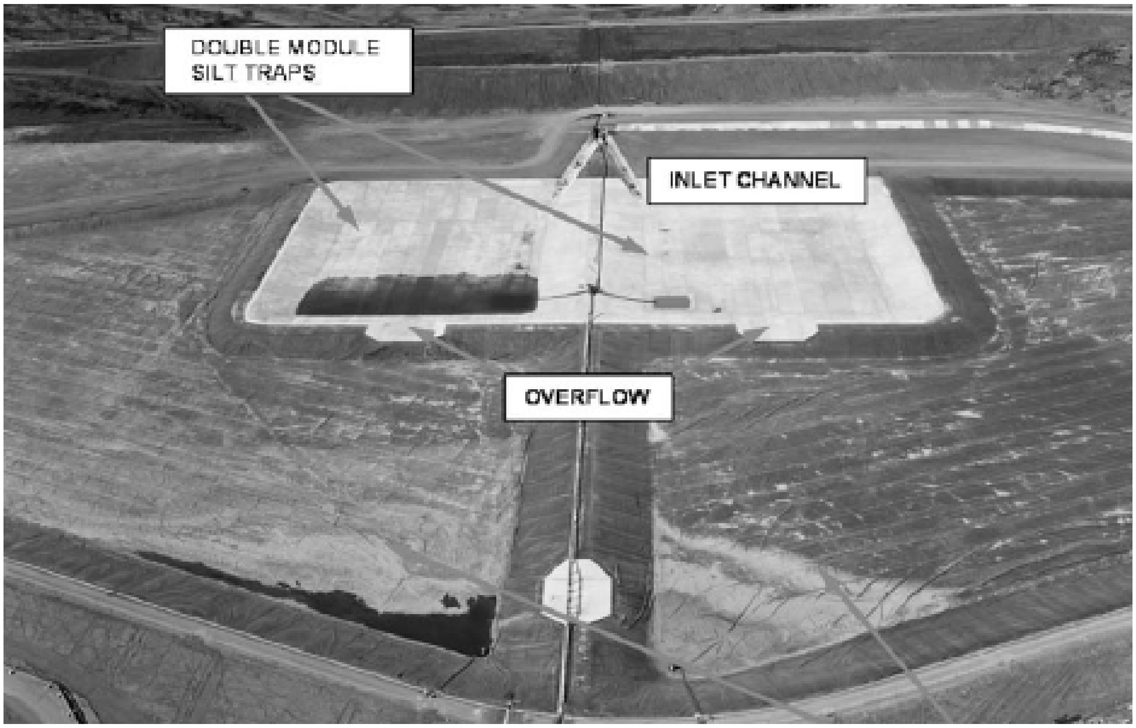
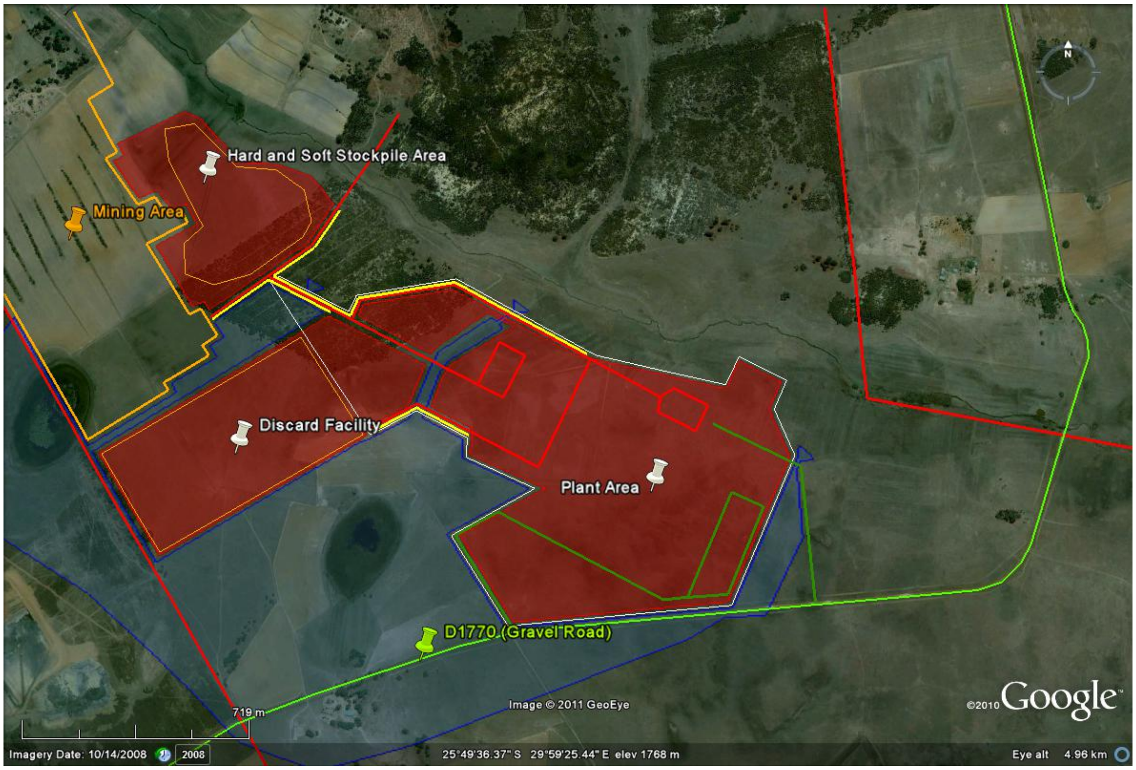


Figure 12: Superelevated Sections of Haul Road (Yellow Lines)



11. DRAIN AND CULVERT DESIGNS

11.1 DRAINS

11.1.1 DESIGN APPROACH

It is preferred to keep runoff as close to its natural state as possible. In a developed area this is not always possible as the vegetation and permeability of the catchment area is changed and may also change over time.

The following order of preference was applied:

- Maintain natural flow conditions.
- Provide wide shallow drains.
- Grass lined drains.
- Grass block lined drains.
- Concrete lined drains.

Energy dissipation is normally required where flow velocities become too high and need to be reduced to acceptable levels e.g. where drains discharge into natural areas.

11.1.2 DRAIN LINING MATERIALS

a) Natural

The area is mainly grassland with vlei conditions close to the Klein-Komatirivier and stands of trees at the discard area. The trees, wattle, blue gum and pine, will be cleared. The surface material (topsoil) consists predominantly of loose, intact, silty fine sand with relatively low clay content.

Based upon the above, a MAP of 690 and a clay content of between 6 and 15, the safe flow velocity (SANRAL, 2006) to prevent erosion is between 0,8 and 1,5m/s. As the MAP is on the upper limit, a figure of 1,2m/s was adopted.

b) Grass

Grass lined drains will be covered with topsoil after excavation. The type of grass covering can be selected to favour higher flow velocity conditions. A maximum allowable velocity of 1,5m/s was adopted (Kikuyu or NK37).

c) Grass Blocks

Grass blocks are individual concrete blocks with holes, stringed together to form a mattress through which vegetation can establish. As a concrete matrix exists around the roots of the vegetation, much higher velocities can be achieved. It is a flexible system and can easily be shaped to follow natural contours.

Two sizes of grass blocks were considered:

- Type 140 – weight of 140kg/m^2 and can be used for velocities up to 3m/s.
- Type 180 – weight of 180kg/m^2 and can be used for velocities up to 6m/s

The disadvantage of grass blocks is that it is more difficult to maintain where sedimentation may take place. For this reason, its use should rather be restricted to clean water systems or where water is not highly polluted with sediment.

More detail on a commercial product can be found in Annexure F.

d) Concrete

Concrete lined drains can be designed to withstand velocities of up to 8m/s (SANRAL, 2006) if heavily reinforced. Thin linings (60mm) can be used up to 2,5 m/s, but this is not practical in a mining environment. A maximum velocity of 5m/s was used.

e) Gabion Mattresses

These mattresses are not normally used for the lining of drains as they are expensive and difficult to clean when silted up. Their most appropriate application is for protection of banks, stilling basins, retaining walls, spillways etc.

They have been used in combination with the other linings, especially concrete, to dissipate energy.

Safe velocities for gabions are determined by their thickness and size of stone used. This should be addressed during the detail design phase.

11.1.3 DRAIN DESIGN

The drain designs were based upon empirical open channel flow formulae such as Manning and Chezy (Webber, 1985). The Manning formula is widely employed today and is particularly appropriate to the rough turbulent zone, the zone in which most channels operate. From flow calculations using the Manning n-value, the absolute roughness k-value was calculated and compared to the estimated roughness of the channel as a check.

Freeboard was calculated as per the Drainage Manual (SANRAL, 2006), for straight sections. Additional freeboard must be provided for curved sections. This detail should be determined during the detail design phase.

The calculations are attached in Annexure B. These calculations are cross-referenced to the catchment areas presented in Annexure A.

From the calculations as summarised in Annexure B, standard drain sections were identified. This selected was based upon:

- Lining type.
- Capacity required.
- Construction practicality.
- Accessibility (some drains crossed by vehicles).

The types are indicated on Drawing 002802-BP- 4 and their positions on Drawing 002802-BP- 1 and Drawing 002802-BP- 3.

Table 3: Typical Drain Sizes Implemented

Type	Lining	Depth	Bottom Width	Side Slope	Total Width
Type 1	Concrete - V	0.20m	0.0m	1:2.5	1.1m
Type 2a	Concrete – Trap.	0.50m	2.0m	1:2	4.1m
Type 2b	Concrete – Trap.	0.50m	1.0m	1:2	3.1m
Type 2c	Concrete – Trap.	0.20m	1.0m	1:10	5.1m
Type 3	Grass – Trap.	0.50m	2.0m	1:3	5.0m
Type 4a	Grass Block – Trap.	0.80m	2.0m	1:3	6.8m
Type 4b	Grass Block – Trap.	0.80m	5.0m	1:3	9.8m
Type 4c	Grass Block – Trap.	0.80m	8.0m	1:3	12.8m

11.2 CULVERTS

11.2.1 MINIMUM CULVERTS SIZES

Haul road traffic exceeds normal traffic loads on culverts and these culverts are best cast in situ. To simplify construction and to keep costs low, a single barrel size of 900x900mm was adopted and additional barrels of the same size added where required. A dimension of 900x900mm is the minimum recommended from a maintenance perspective for haul roads which are typically wider than 20m.

A minimum size of 900x600mm (900mm wide and 600mm high) was used for other roads as it will be easier to maintain in a mining environment. The minimum size adopted by most road authorities for box culverts is 600x600mm.

Pipe culverts are not recommended as additional cover over the culverts is required.

11.2.2 CULVERT CAPACITY DESIGN

Culverts were selected using inlet control as the restriction. Outlet control reduces capacity and sedimentation may take place inside the culvert. It is imperative that the outlet be maintained free of obstructions.

The culvert inlet and outlet geometry as well as erosion protection are indicated on Drawing 002802-BP- 11 and Drawing 002802-BP- 12 for haul road culverts.

A maximum Hw/D (head water depth / culvert depth) of 1.2 was used in the calculation, so no pressure will build up inside culverts.

The calculations are attached in Annexure D. These calculations are cross-referenced to the catchment areas presented in Annexure A. Pipe sizes are indicated for comparison only, and they are not recommended.

11.3 ENERGY DISSIPATION

Gabion boxes and mattresses have been used extensively for energy dissipation. Refer to Drawing 002802-BP- 5 and Drawing 002802-BP- 7 for details. Positions are indicated on Drawing 002802-BP- 1.

11.4 SILT TRAPS

A velocity of less than 0,8m/s (SANRAL, 2006) is required for silt to be deposited. The silt traps were designed as such to ensure this velocity is not exceeded. Silt traps were designed as long as possible, within reason, to ensure sufficient time for silt to be deposited.

Silt traps were placed in the plant area as indicated on Drawing 002802-BP- 3.

The silt trap at P27 is downstream of the Washing Plant, the Middlings Export and the Emergency Stockpile areas where a relatively high percentage of suspended solids can be expected.

A second silt trap is located at C12 to trap any solids spilled at the Export and Middling Bins and from vehicles on the concrete road.

The third silt trap is placed adjacent to the Storm Water Dam D5. It will trap suspended solids that may have passed through the previous two traps, as well as that from the Primary Crusher, ROM stockpile and Discard Bin (C6, C7 and C8). This silt trap decants longitudinally into D5. Refer to Drawing 002802-BP- 6 for a typical detail.

Two further silt traps are provided next to the Hard and Soft Stockpile Area (D1) and the Discard Facility (D2). Refer to Drawing 002802-BP- 1. These silt traps are similar in operation than the one at D5.

The dimensions are summarised in Table 4.

Table 4: Silt Trap Dimensions

Position	Width m	Length m	Depth m	Flow m^3/s	Velocity m/s
Dam 1 (D1)	3	20	1.2	4.2	<0.8
Dam 2 (D2)	3	20	1.2	6.2	<0.8
Dam 5 (D5)	5	40	1.2	6.9	<0.8
Plant (P27)	3	8	1.0 min	1.6	<0.8
Plant (C12)	3	8	1.0 min	0.8	<0.8

12. DAM DESIGNS

12.1 DAM CATEGORY CLASSIFICATION

Dams should adhere to the relevant dam safety criteria, based upon the safety risk and classification of the dam. The relevant dam safety regulations can be found in Government Notice R 1560 of 25 July 1986, (DWAF-A4, Aug 2007).

The dam category classification is based upon size and the potential hazard. Each category has its own conditions and requirements to be adhered to, and these are more stringent and comprehensive for large and high hazard potential dams.

In terms of the Guidelines (DWAF-A4, Aug 2007), Pollution Control Dams (PCD) should be classified in terms of the following tables:

Table 5: PCD Size Classification

Size Class	Maximum Wall Height in Metres
Small	More than 5 but less than 12m
Medium	Equal to or more than 12 but less than 30m
Large	Equal to or more than 30m

Table 6: Hazard Potential Classification

Hazard Potential Rating	Potential Loss of Life	Potential Economic Loss
Low	None	Minimal
Significant	Less than ten	Significant
High	More than ten	Great

Table 7: Category Classification of Dams with a Safety Risk

Size Class	Hazard Potential Rating		
	Low	Significant	High
Small	Category I	Category II	Category II
Medium	Category II	Category II	Category III
Large	Category III	Category III	Category III

All dams under consideration are classified as small as their wall height is less than 12m. The Hazard Potential Classification of the two biggest dams (Return Water Dam East and

Return Water Dam West) is Significant based upon the potential economic loss for the proposed mining operation. The Hazard Potential Classification of the other dams is Minimal.

Although only the Return Water Dams must be designed by an Approved Professional Person, it is recommended that all the dams be checked by such a person.

The Category Classification for the PCDs can be found in Table 8. Capacity calculation is dealt with under Section 12.7 and Freeboard determination under Section 12.2.

Table 8: Category Classification of Belfast Dams

No.	PCD Type	Volume Required[1]	Wall Height[2]	Freeboard[3]	Classification
D1	Storm Water Dam	11 800 m^3	3.1m	0.8m	Category I
D2	Storm Water Dam	17 100 m^3	3.3m	0.8m	Category I
D3	Process Water Dam	15 000m^3	3.5m	0.8m	Category I
D4	Emergency Slurry Dam	Not specified	Ground level	N.a.	N.a.
D5	Storm Water Dam	47 400m^3	7.9m	0.8m	Category I
D6	Biofilter Dam	30 000m^3 max	2.3m	0.8m	Category I
D7	Return Water Dam	230 000m^3	9.0m	1.5m	Category II
D8	Return Water Dam	230 000m^3	9.0m	1.5m	Category II
No.	Clean Water Types	Volume Required[1]	Wall Height[2]		Classification
PE	Natural Pan	N.a.	N.a.	N.a.	N.a.
PW	Divided Pan	64 800m^3	2.0m	0.8m	Category I

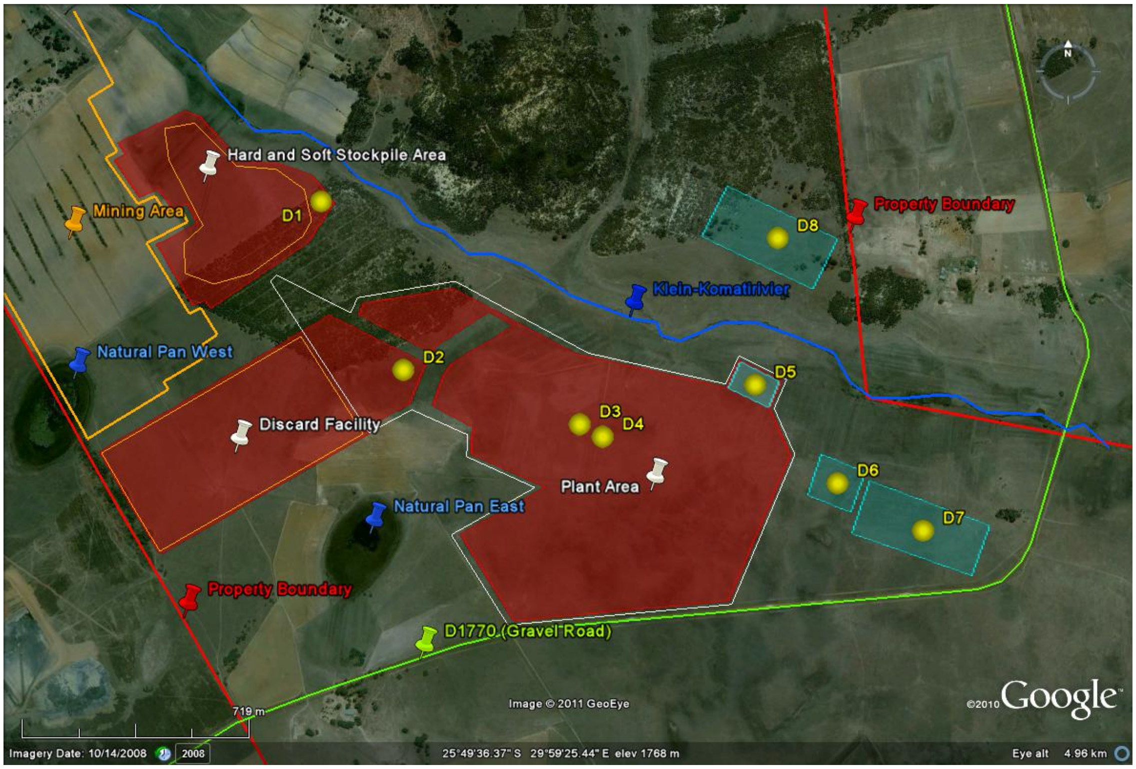
[1] *Excluding freeboard.*

[2] *Measured from crest to invert, including free board.*

[3] *Minimum Freeboard requirement.*

Refer to Figure 13 for the position of the dams (more detail on Drawing 002802-BP- 1).

Figure 13: Dam Positions



12.2 FREEBOARD DETERMINATION

In terms of the DWA regulations, a minimum freeboard of 0,8m is required above full supply level. This may not be sufficient for dams with a safety risk, and the most appropriate guideline is published by the South African National Committee on Large Dams (SANCOLD). Report No. 3, Safety Evaluation of Dams, Interim Guidelines on Freeboard for DAMS (SANCOLD, Aug 1990).

Freeboard determination is based upon design combinations including:

- Flood Outlets - *Not applicable.*
- Flood Surges and Seiches - *Relevant to bigger water masses.*
- Earthquake Wave - *Even in mildly seismic areas, detailed calculations are not warranted.*
- Land Slide Wave – *Not applicable.*
- Wind Wave Run-up and Wind Set-up - *The effect of wind is negligible as the biggest dam is less than 300m long. This is the minimum length where wind starts having an effect.*
- Recommended Design Flood - *The biggest two dams are Return Water Dams (230,000m^3 each) and will have pumped inlets and outlets. Design Floods and*

Flood Surges will have a minimal impact on Freeboard. The remaining dams are small (Table 8) and do not pose a safety risk w.r.t. freeboard.

The Interim Guidelines present a practical and simplified table for the circumstances explained above. The relevant data is reflected in Table 9.

Table 9: Simplified Practical Freeboard Guidelines

Category and Type of Dam	Total Freeboard[1]
Category I (Earthfill)	0,8m
Category II (Earthfill)	1,5m

[1] *Measured between design water level and non-overspill crest.*

Refer to Table 8 for a summary of Freeboard requirements for the Belfast dams.

12.3 OPERATIONAL REQUIREMENTS

The operational requirements as summarised in Table 10 were used as the basis for the design.

Table 10: Operational Requirements of Dams

PCD Types	Applicable Dams	Operational Requirement
Storm Water Dams	D1	Keep empty by: Evaporation, return to process water system, manage water quality dynamically and release into clean water system if quality complies. Source: Storm water runoff from Hard and Soft Stockpile Area.
Storm Water Dams	D2, D5	Keep empty by: Return to process water system. Source: Storm water runoff from Discard Facility and Plant Area.
Process Water Dam	D3	Operate at level to accommodate dirty water inflow, less outflow and losses and maintain required freeboard. Source: Pumped from Return Water Dams and other dirty areas.
Emergency Slurry Dam	D4	Evaporate excess fluid and remove to dump / discard facility. Source: Plant processing.
Biofilter Dam (Evaporation Dam)	D6	Operate at a level to accommodate inflow, less outflow, losses and maintain required freeboard by returning to process water should water quality comply with regulations. Sourced: Outflow from Sewage Package Plant.
Return Water Dams	D7,D8	Operate at level to accommodate dirty water inflow, less outflow and losses and maintain at required operational level. Operational level provides for direct rain collection and freeboard. Source: Pumped from Mining Areas
Clean Water Types	**Applicable Dams**	**Operational Requirement**
Natural Pan	PE	None. Pan will overflow naturally and follow clean water path and drain. Source: Clean storm water runoff.
Divided Pan	PW	None. Pan may overflow through spillway provided and follow clean water drain. Source: Clean storm water runoff.

12.4 SPILLWAYS

The dams are discussed below:

a) Storm Water Dams (D1, D2, D5)

These are provided with spillways to accommodate 1:100 year 24hr events as the timing of the event cannot be predicted or managed. The minimum freeboard is 0.8m as per Table 8. Dam D1 is to be provided with a gabion lined overflow.

b) Process Water Dam (D3)

This dam is similar in operation to the Return Water Dams discussed below. The spillway and volume has been designed to accommodate the 1:100 year events.

c) Emergency Slurry Dam (D4)

This facility should be maintained at a low level and no spillway is provided.

d) Biofilter Dam (D6)

A spillway is provided and sized to accommodate a 1:100 year event. The minimum freeboard is 0.8m as per Table 8.

e) Return Water Dams (D7, D8)

A combination of float valves and level sensors can be installed to shut-off pumps at a preset level. The dams are elevated above ground level so no storm water runoff can enter the dam. Provision has to be made for rainfall collected over the dam surface. The spillway has been designed to allow for a 1:100 year peak flow event combined with the peak rate of inflow from the pumps. The minimum freeboard is 1.5m as per Table 8. These are the only Category II dams.

f) Natural Pan East (PE)

No spillway is provided for the *Natural Pan East* as it will not overflow for a 1:50 or even a 1:100 recurrence interval (refer to section 9.1.1). The outflow will follow its natural path up to the diversion though the plant area.

g) Natural Pan West (PW)

An overflow is provided for the *Natural Pan West – Divided* as it is likely to overflow (refer to section 9.1.2). The overflow is designed for a 1:100 year event and constructed from gabions.

The spillway sizes are summarised in Table 11 below. The spillway calculation and sizes can be found in Annexure D and more detail can be seen on Drawing 002802-BP- 7.

Table 11: Spillway Sizes

No.	Dam Type	Type	Length	Design	Rate	Flow Depth
D1	Storm Water Hard and Soft	Gabion overflow	20m	1:100	5.4 m^3/s	0.31m
D2	Storm Water Discard and Plant	Side decanting	20m	1:100	8.95m^3/s	0.44m
D3	Process Water	Side decanting	5m	1:100	1.0 m^3/s	0.26m
D4	Emergency Slurry	Side decanting	2m	1:100	0.2m^3/s	0.16m
D5	Storm Water Discard and Plant	Side decanting	40m	1:100	15.03m^3/s	0.39m
D6	Biofilter	Side decanting	5m	1:100	1.58m^3/s	0.35m
D7,8	Return Water	Spillway	20m	1:100	5.4m^3/s	0.31m
PE	Natural Pan	Natural	N.a.	N.a.	N.a.	N.a.
PWb	Pan – Divided.	Gabion overflow	20m	1:100	8.0m^3/s	0.41m

12.5 DAM LININGS

Linings are discussed in detail in section 13. Table 12 below summarises which dams require lining.

Table 12: Lining of Dams

Lining and Hazard Classification	Applicable to Dams
Lined (hazardous liquid)	D2 to D8
Unlined (not hazardous or clean)	D1, PE and PW

12.6 GROUNDWATER AND DRAINAGE

Linings float when the groundwater is high and this then reduces the capacity of lined dams. Three methods or a combination thereof, are normally used to prevent floating:

- Subsurface drainage (below invert level)
- Ballast (stone, gravel and even tyres)
- Anchoring (could be problematic when dam levels are low)

Subsurface drainage is proposed as the main measure to prevent floating and this can be implemented in the following way:

- Network of shallow subsurface trench drains (herringbone pattern)

- Cut-off vertical drains, deep trench drains (normally around the perimeter)
- Layer subsurface drainage (permeable layer in a geotextile blanket)

Based upon the geotechnical investigation, the following measures are envisaged for the dams:

Table 13: Subsurface Drainage Measures for Dams

Dam	Envisaged Conditions	Wall and Lining[1]	Sub surface Drainage Method
D1	Intermediate to hard	Earth, HDPE lining of wall.	N.a.
D2	Intermediate to hard	Earth, HDPE lining	Blanket layer drain
D3	Intermediate to hard	Earth, HDPE lining	Blanket layer drain
D4	Intermediate to hard	Earth, concrete lining	Blanket layer drain
D5	Intermediate to hard, groundwater	Earth, HDPE lining	Blanket layer, vertical cut-off drain
D6	Soft to Intermediate to hard, groundwater	Earth, HDPE lining	Blanket layer, shallow trench and vertical cut-off drains
D7	Intermediate to hard, groundwater	Earth, HDPE lining	Blanket layer, vertical cut-off drain
D8	Intermediate to hard	Earth, HDPE lining	Blanket layer drain
PE	Natural Pan	N.a.	N.a.
PWb	Natural Pan, divided	Earth, HDPE lining of wall.	N.a.

[1] *Refer to section 13 for lining designs*

The subsurface drains discharge into a collection sump from where it should be pumped by means of automated level control. These sumps can also be used for water quality monitoring and can be an indication of possible lining leakage. The sizing of pumps and automation are to be part of the detail design phase.

12.7 CAPACITY DETERMINATION AND DAM SIZING

The dam capacities and sizes were determined on the following basis for the various types of dams:

a) Storm Water Dams (D1, D2, D5)

Dams designed to accommodate the 1:50 year 24hr event assuming they are empty at the onset of the event. Although not a requirement, additional provision was made for some degree of silting to take place (varies between 5 and 10% of dam capacity). Freeboard added to this level and flood level for 1:100 year peak runoff checked (overflow).

b) Process Water Dam (D3)

Capacity requirement of 15 000m^3 as per information provided (Aurecon, Jun 2011). The 1:50 year 24hr event over the surface area of the dam was added to the given capacity. Freeboard added to this level and flood level for 1:100 year peak runoff checked (overflow).

c) Emergency Slurry Dam (D4)

No capacity specified. It is an emergency facility linked to operational requirements. The maximum reasonable volume for the available space was calculated.

d) Biofilter Dam (D6)

A detailed discussion on sewage treatment and the sizing of the dam can be found in section 16.

e) Return Water Dams (D7, D8)

Capacity requirement of 230 000m^3 each as per information provided (Aurecon, Jun 2011). The 1:50 year 24hr event over the surface area of the dam was added to the given capacity. Freeboard added to this level and flood level for 1:100 year peak runoff checked (overflow).

f) Natural Pan East (PE)

For the modelling done in section 9.1.1, a depth-volume relationship was calculated from the available contours. The volume reflected is at the estimated overflow level.

g) Natural Pan West (PW)

For the modelling done in section 9.1.2, a depth-volume relationship was calculated from the available contours. This relationship was adjusted to reflect the division of the pan with a wall. The volume reflected is at the estimated overflow level for the divided pan.

The calculations can be found in Annexure C and a summary in Table 14.

Table 14: Dam Dimensions and Volumes

Dam	Width[1] m	Length[1] m	Crest Width m	Max. Wall Height[2] m	Volume to Spillway m^3	Actual Freeboard m
D1	85	105	3	3.1	11,840	1.1
D2	96	126	3	3.3	17,100	1.2
D3	89	109	3	3.6	16,060	1.1
D4	50	115	3	2.5	4,700	1.0
D5	103	142	5	7.8	47,360	1.2
D6	160	160	3	2.5	31 560	1.1
D7,8	140	370	5	9.3	234,320	1.8
PE	N.a.	N.a.	N.a.	N.a.	N.a.	N.a.
PWb	N.a.	N.a.	3	3.2	4,700	1.2

[1] *Measured between insides of crest*
[2] *Measured from invert to crest (inclusive of freeboard)*

12.8 DAM WALL DESIGN

12.8.1 GEOTECHNICAL INVESTIGATION

The geotechnical investigation at the location of the proposed dam sites (Annexure G) has revealed that both the Return Water Dams are underlain by colluvium, with some alluvium encountered at the Return Water Dam West site. The alluvium is underlain by pedogenic ferricrete which varies between honeycomb to hard pan ferricrete. The ferricrete was encountered between 0.3m and 0.9m below ground level. The material underlying the ferricrete was not proved but is anticipated to be residual to highly weathered sandstone.

No trail pits could be excavated at the Return Water Dam West site and soil samples from the Return Water Dam East site were tested. It is anticipated that the laboratory test results will be representative of the material encountered at both sites as the geology is fairly consistent.

The shear strength parameters for the colluvial material encountered at both sites, was determined by undertaking consolidated un-drained (CU) tri-axial tests with pore pressure measurements. The tests were undertaken on re-compacted samples that were first compacted to 95% Proctor density, and the samples were saturated prior to testing.

The test results are summarised in Table 15.

Table 15: Summary of Strength Test Results

Test Position	Depth m	Sample Preparation	Angle of Internal Friction (Phi)[1] Degrees	Cohesion (C)[1] kPa
T11	0.2-0.70	Remoulded	35.4	10.0
T15	0.25-0.75	Remoulded	34.1	12.8
T16	0.35-0.85	Remoulded	32.3	17.9

[1] *Angle of internal friction (Phi), Effective Strength*

12.8.2 DAM WALL DIMENSIONS

The height of the dam walls is determined by the topography of the sites and was taken to be 6m for the outside slope and 9m for the inside slope. The height difference means that the centre of the dams will need to be excavated below ground level to generate sufficient retention capacity and also to provide construction material

From the geotechnical investigation the maximum depth of excavation which was possible with a Tractor Loader Backhoe (TLB) was between 0.3m and 1.1m below ground level. As the required excavation will be to 3m below ground level, heavier excavation plant will be required to break through the hard pan ferricrete to reach the required depth. It is anticipated that residual sandstone grading into weathered sandstone will be present beneath the ferricrete horizon.

12.8.3 SLOPE STABILITY ANALYSIS

By lining the dams (as per section 13), the embankments can therefore be designed for the unsaturated case, as there will be no flow within the dam walls from the waste water contained within the dams. The use of a liner therefore obviates the need to provide an impermeable core to the dam walls and the embankments have been designed to be homogeneous, using in situ construction material.

Slope stability analyses for the proposed dam were undertaken using the software programme *SLIDE*, which is part of the *RocScience* suite of geotechnical programmes. Initial analysis showed that slopes up to 1 vertical to 2.5 horizontal (1:2.5) will be stable. However, the lining materials dictate a maximum slope of 1 vertical to 3.5 horizontal (1:3.5) to prevent slippage and membrane failure (refer to paragraph 13.4).

The inside slope was analysed at a slightly steeper 1:3 to accommodate an alternative lining solution that may cope with a steeper slope. The outside slope was analysed at 1:2. The crest of the dam is 5m, which allows for vehicular access and inspections to be carried out.

Conservative values of shear strength were used for the construction material, with an angle of internal friction taken as 30 degrees and a cohesion of 5kPa. The *Bishop and Morgernstern Price* method of analyses was used. The design has allowed for subsurface drainage beneath the dam to accommodate the high in-situ water table and to prevent water seepage into the embankment.

The plots of Stability analysis for the inner wall are presented in Figure 14 and that of the outer wall in

Figure 15.

Figure 14: Stability analysis for the inside slope of the dam wall

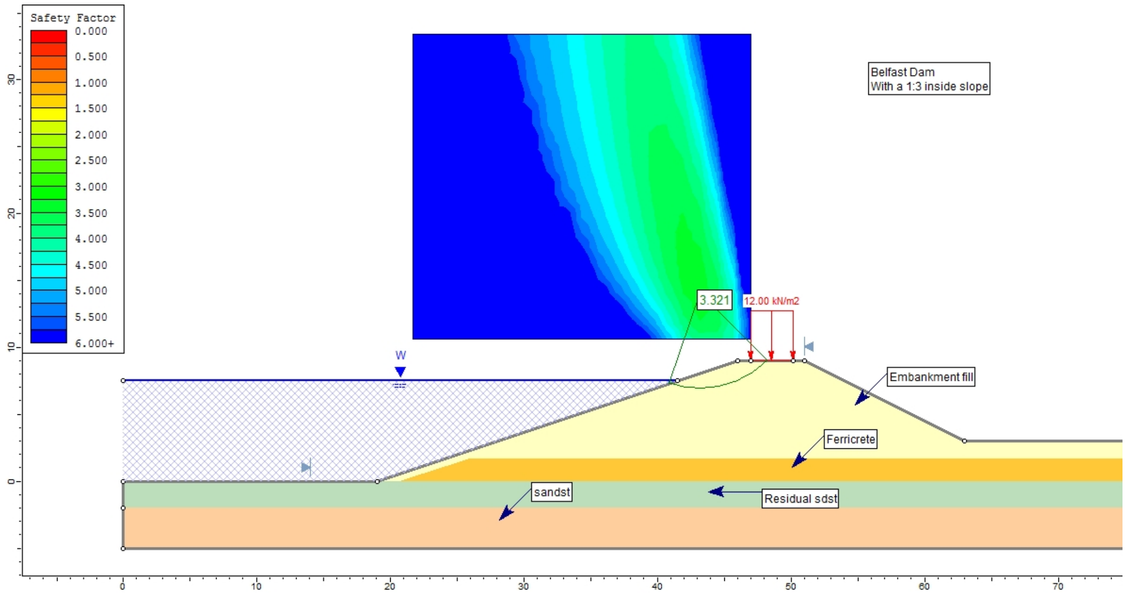
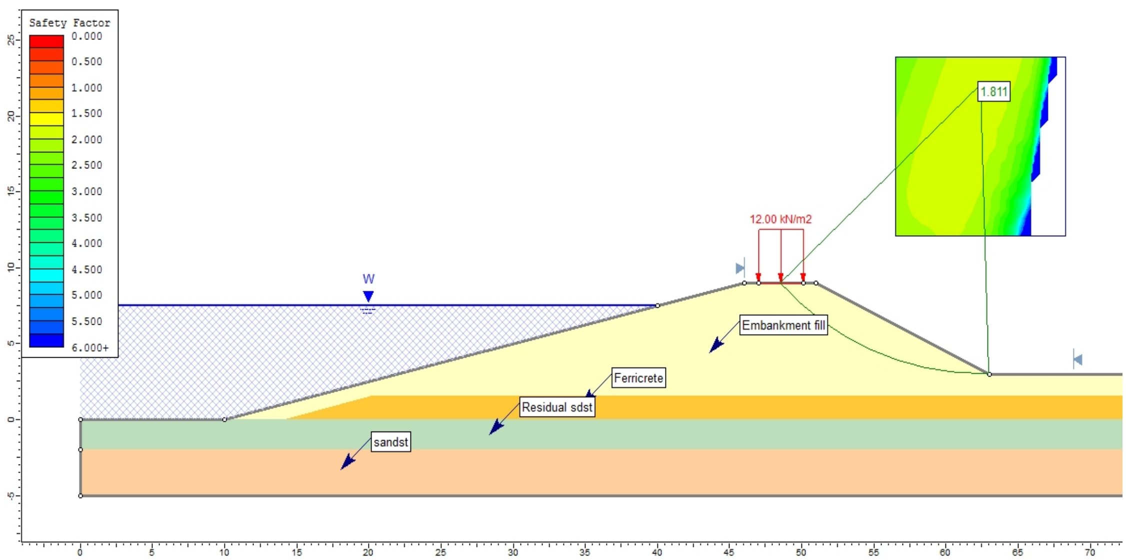


Figure 15: Stability analysis for the outside slope of the dam wall



As can be seen from Figure 14 and

Figure 15, the factors of safety against failure are 3.6 for the inside slope and 1.8 for the outside slope. Both values are above the recommended minimum value of 1.5.

12.8.4 CONSTRUCTION METHOD

The embankment foundation will need to be stripped of topsoil and organic matter prior to construction. The material for the embankment shall be placed at a minimum compaction of 95% Proctor at optimum moisture content. The material shall be placed such that the larger particle sizes, i.e. anything greater than 60mm in diameter shall be placed on the steeper outer slope. The colluvial material tested on site may need to be blended with the material excavated out of the excavation for the dam, assumed to be residual and weathered sandstone, and additional shear strength testing of the blended material undertaken to finalise the design. It is recommended that the outer slope of the dams be top soiled and grassed to reduce the risk of erosion.

12.8.5 CAUTION

Cognisance should be taken of the fact that the in situ soils that were tested in the laboratory are classified as “Very Highly Corrosive”, and concrete structure in contact with these soils will need to be designed accordingly.

13. LINING DESIGNS

13.1 INTRODUCTION

A number of dams are proposed for the Belfast Project and they are listed in Table 16 with regards to content (refer to Table 10 for more detailed operational requirements). Their positions are shown on Figure 16.

Table 16: Dam Content

No	Dam Description	Retention and Liquid
D1	Storm Water from Hard and Soft Stockpile Are	Keep empty, evaporate, release Could be dirty.
D2,5	Storm Water from Discard and Plant Areas	To be kept empty. Dirty water
D3	Process Water mainly from Return Water Dams	Maintained at operational level Dirty water
D4	Emergency Slurry Dam from plant operations	Evaporate liquid and remove Dirty slurry
D6	Biofilter Dam downstream of a package sewage treatment plant	Evaporate. Not suitable for release into clean water system.
D7,8	Return Water Dam	Maintained at operational level Dirty water
PE	Natural pan with clean storm water	Evaporate, infiltrate, overflow Clean water system.
PW	Divided pan with clean storm water	Evaporate, infiltrate, over flow Clean water system.

The lining systems will be specifically designed and installed to prevent contamination from entering into the underlying groundwater and to facilitate the storage, handling, re-use or disposal of the contained liquids.

The selection of the lining systems needs to take into account regulatory requirements, cost considerations, availability of materials, functional requirements, lifespan in terms of operating, chemical and climatic conditions, ease of installation and serviceability, i.e. posing the least risk in terms of short to long term leakage.

Existing site conditions such as the geology and nature of the soils associated with the site, the capacity and physical characteristics of the dams (e.g. magnitude of side slopes, depth of contained liquids) and the nature of the contained liquids also need to be assessed in the selection and design of the lining systems.

Figure 16: Dam Positions

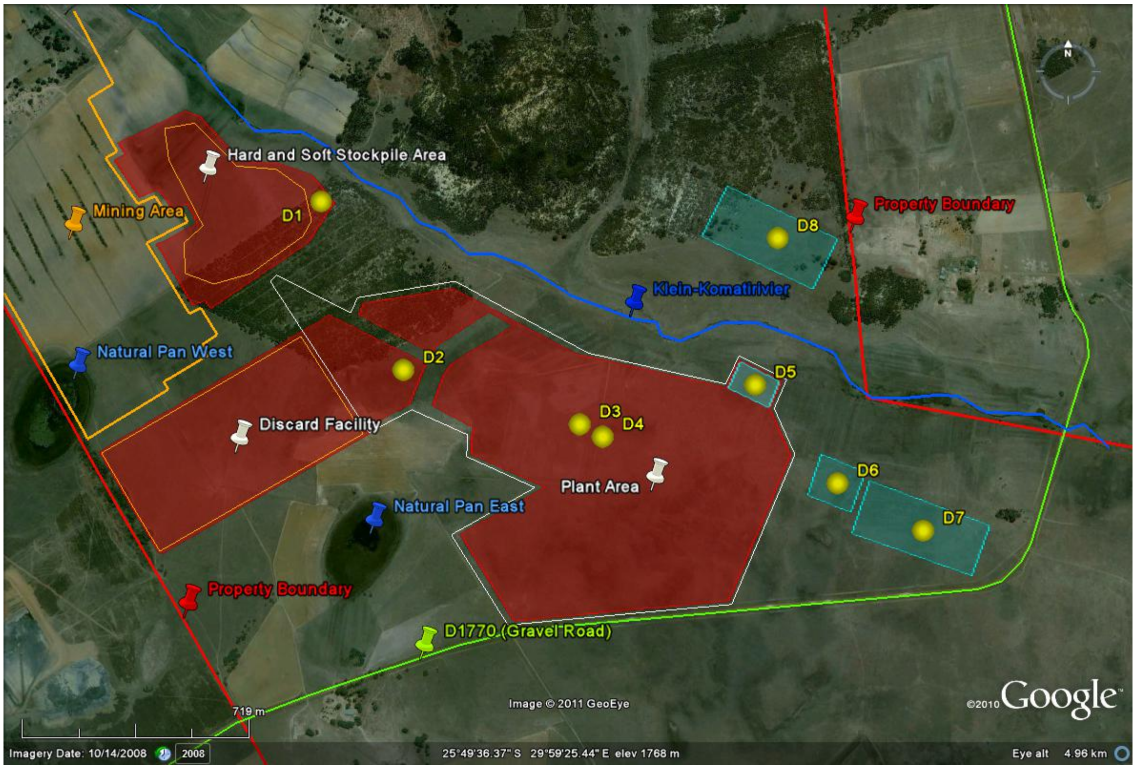


Figure 17: Typical Lined Dam (DWAF-A4, Aug 2007)

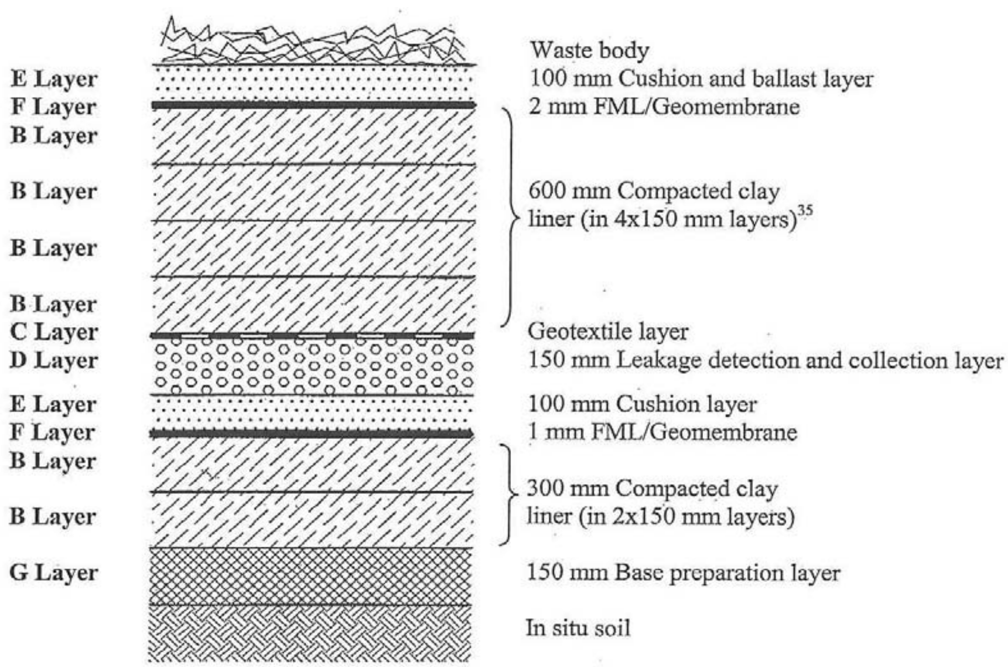
13.2 RETURN WATER AND PROCESS WATER DAMS (D3, D7 AND D8)

The two proposed Return Water Dams (East and West, 230,000m^3 each) are intended to store process water to be re-used in the mining operations. The depth of water stored will be approximately 7.5m with the dams maintained close to their full capacity for most of the time.

It is envisaged that the process water will generally have a low pH value which is reported could be as low as 2. Chemical results of water samples taken from other similar mines indicate that elevated total dissolved solids (TDS), sulphates and manganese could be expected (GCS, Jan 2011).

Employing the LC_{50} eco-toxicity criteria, i.e. the concentration at which a substance will kill 50% of aquatic animals tested, a liquid with a pH of less than 6 can be regarded as a "hazardous liquid" and a storage dam or lagoon should be effectively lined so as to contain the liquid in order to prevent environmental contamination. The anticipated low pH characteristic will classify the liquid as a moderate to high-risk substance, and the proposed liner design is accordingly based on the Department of Water Affairs Minimum Requirements for the design of Hazardous Waste Lagoons (DWA, 2005), (DWAF-A2, Jul 2008). The diagram below indicates the design criteria set out in the Minimum Requirements:

Figure 18: **Hazardous Waste Lagoons: Minimum Requirements for Liner Design**



Note: *It may be possible to treat the mine water through a process of blending and/or pH control and thus delist the liquid to a low-hazard classification which in turn may result in a reduced acceptable design standard. For purposes of this report, a precautionary approach has been adopted and a moderate-high hazard classification is assumed.*

Taking the DWA Minimum Standards as the basis for the liner design, three options were designed (Figure 19,

Figure 20 and

Figure 21). Subsurface drainage measures are not reflected on these figures.

- Option A is proposed where there is sufficient and suitable clay material in the area. For this project, there is uncertainty as to the quantity of clay available.
- Option B proposes the replacement of clay with a geosynthetic clay (mineral) liner (GCL), while still having a 300mm ballast layer on top of it to provide a confining pressure on the GCL.
- Option C excludes the 300mm ballast layer on top of the GCL as these dams will be operated at operational level all the time. Sufficient pressure from water.

Figure 19: **Alternative Lining for Moderate to High Risk Liquid – Option A**

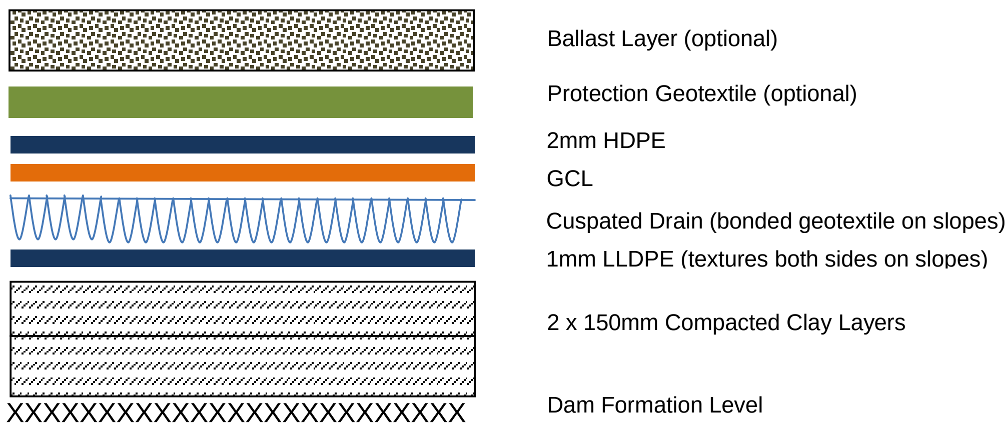


Figure 20: **Alternative Lining for Moderate to High Risk Liquid – Option B**

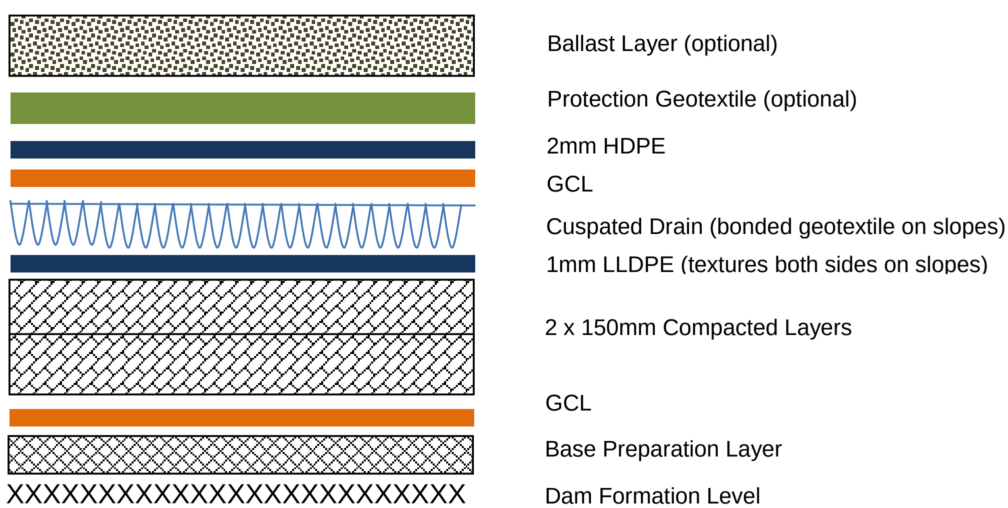
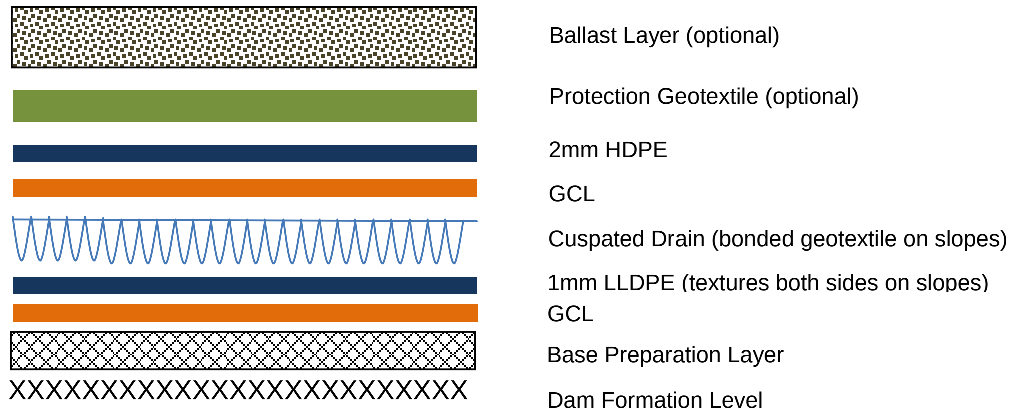


Figure 21: **Proposed Lining for Moderate to High Risk Liquid – Option C**



Note: Additional subsurface drainage layers may be required where groundwater is expected. Placement would be between GCL and Dam Formation Level.

The layers as proposed in the three options are discussed below (layers from formation level):

a) Dam Formation preparation includes trimming and compaction of constructed dam basin floor and embankment sides to construction specifications.

b) A Base Preparation Layer free from particles that may cause mechanical damage to the overlying liner.

c) Compacted Clay Layers (CCL's). This layer should serve as a protection layer to the LLDPE liner (particle size that does not cause mechanical damage) and as a preparation layer for the primary and secondary lining layers that follow. The permeability (water-tightness) of the two CCL layers is not considered a critical requirement in the context of the total lining design, and from a practical construction perspective, CCL's will always have limitations in achieving permeability requirements, irrespective of the average quality of the clay used.

d) A Geosynthetic Clay (mineral) Liner (GCL) consists of two layers of geotextiles with a thin layer of bentonite powder or granules sandwiched between the geotextiles and with the geotextiles needle-punched together to contain the bentonite. Normally, a 300mm ballast layer is placed on top of the GCL to provide a confining pressure to the GCL. The PH of the polluted water is not as important as the Ca content where bentonite is used. This is not considered a major concern based upon the available information(GCS, Jan 2011).

e) The LLDPE geosynthetic secondary liner should be textured on both sides where placed on the sloping embankments, and smooth on both sides where placed on the basin floor. Full manufacturing and construction quality assurance should be implemented.

f) The HDPE cuspated drain layer is a leakage collection layer and consists of a bonded geotextile applied to both sides where used on the sloping embankments. This layer should be drained by pipes to a collection sump for monitoring. Bigger dams, especially where there is no alternative storage, can be divided into section each with its own collection sump.

g) The primary composite liner comprises a GCL followed by a HDPE liner.

h) Geotextile protection layer protects against migration of particles, damage and can act as a drainage layer.

i) The top ballast layer protects the integrity of the primary liner. It is recommended that the ballast layer comprise of a 150mm layer of crushed stone aggregate on the sloping sides. Consideration can be made to using an alternative ballast material for the basin floor, such as motor car tyres. Consideration should be given regarding the low pH of the contained liquid when deciding on a suitable ballast material. Should it be required to periodically remove accumulated solids from the basin floor, a “hyson-cell” or similar cellular layer could be considered. The infill material to the cellular layer should take into account potential chemical attack from the low pH liquid.

13.3 STORM WATER DAMS (D2 AND D5)

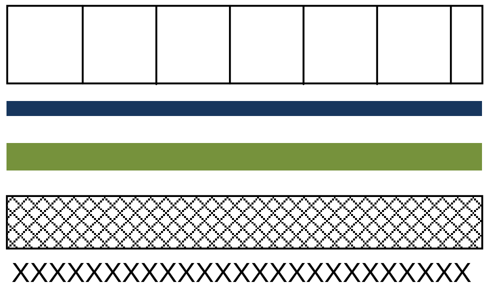
These two Storm Water Dams collect runoff from the Discard Facility and Plant Area. The classification of the water is considered to be of a low-risk hazard, but likely to fail the water quality requirements for open discharge into a receiving stream. The requirement is that these dams be lined.

These dams are not used for storage, but for collection and transfer to the Process Water system – seasonal runoff and for short periods of time. The risk of seepage and contamination is therefore much reduced and the liner design can be adjusted accordingly.

The design of the lining system should take into account the aggressive chemical characteristics of the soils, the wetting and drying cycles that will occur and the need to periodically clean the accumulated solids from the basin floors.

The following lining system is proposed:

Figure 22: Proposed Lining System for Storm water Dams



Note: Additional subsurface drainage layers may be required where groundwater is expected. Placement would be between Protection Geotextile and Dam Formation Level.

In addition to layers previously discussed:

a) A concrete-filled hyson-cell layer or concrete slab can be considered as protection to the geosynthetic LLDPE membrane and to facilitate periodic cleaning out of the dam.

13.4 STORMWATER DAM (D1)

This Storm Water Dam collects runoff from the Hard and Soft Stockpile Area. It is expected that this water may meet water quality requirements for open discharge into a receiving stream, but this cannot be assumed for planning and design purposes. The classification of the water is that of a low risk hazard.

Water should be tested and discharged into the clean water system or pumped to the dirty water system should it feel to meet the criteria. It may be desirable to give the water time for any suspended solid to settle.

For the reasons above, it is not recommended to line this dam.

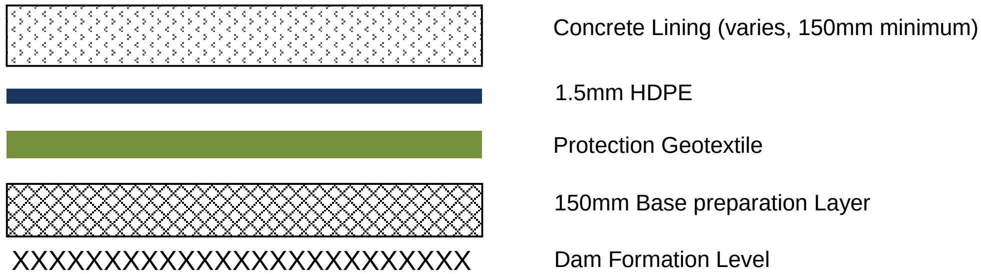
13.5 EMERGENCY SLURRY DAM (D4)

This Dam would be used in operational emergencies to store and dry out slurry. Front-end loaders would typically be used to transfer slurry and the lining system should be durable.

The proposed dam is to be concrete lined. Joints should be designed to be durable and movement between slabs should be restricted.

The following basic design is proposed:

Figure 23: Proposed Lining System for Emergency Slurry Dam



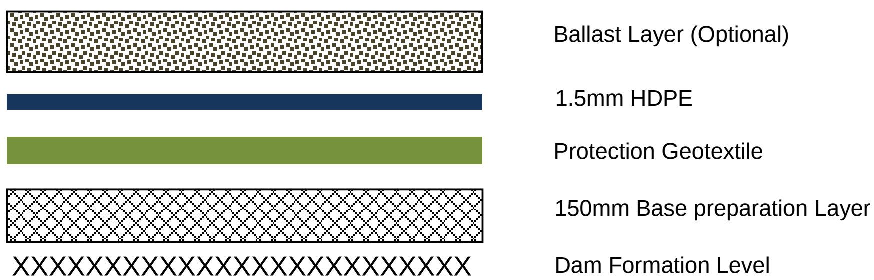
Note: Additional subsurface drainage layers may be required where groundwater is expected. Placement would be between Protection Geotextile and Dam Formation Level.

13.6 BIOFILTER DAM (D6)

This dam is used as an evaporation pond for effluent from the sewage treatment package plant. The final effluent from the package sewage treatment plant will be of the General Limit effluent standard. For discharge into the clean water system, effluent will have to comply with Special Limit quality since the mine is within a Special Limit catchment (i.e. All tributaries of the Komati River between Nootgedacht Dam and the confluent with the Sevenfonteinspruit, Table 3.3 Listed Resources, National Water Act of 1998).

The following lining system is proposed:

Figure 24: Proposed Lining System for Biofilter Dam



Note: Additional subsurface drainage layers may be required where groundwater is expected. Placement would be between Protection Geotextile and Dam Formation Level.

13.7 SLOPE OF LINING MATERIAL

The slope of lining material is determined by the interface friction between the liners. At steep slopes, there will be slippage leading to membrane failure. The preferred slope is 1:4 (1 vertical and 4 horizontal), but this could probably be increased to 1:3.5. The difference in slope is small and not critical in the dam wall design as shown with the slope stability analysis (section 12.8.3). It is recommended that this be further investigated during the detail design phase.

13.8 INSTALLATION

The design of the geosynthetc layers (for all proposed dams) must incorporate proper anchorage detailing for the prevention of slip and rupture failure on the side slopes. The quality of the materials used in the lining layers as well as the quality of the construction and installation of the lining layers is critical and the requirements of the accepted industry standards and specifications as well as any special requirements of the regulatory authorities should be strictly applied.

13.9 MAINTENANCE REQUIREMENTS

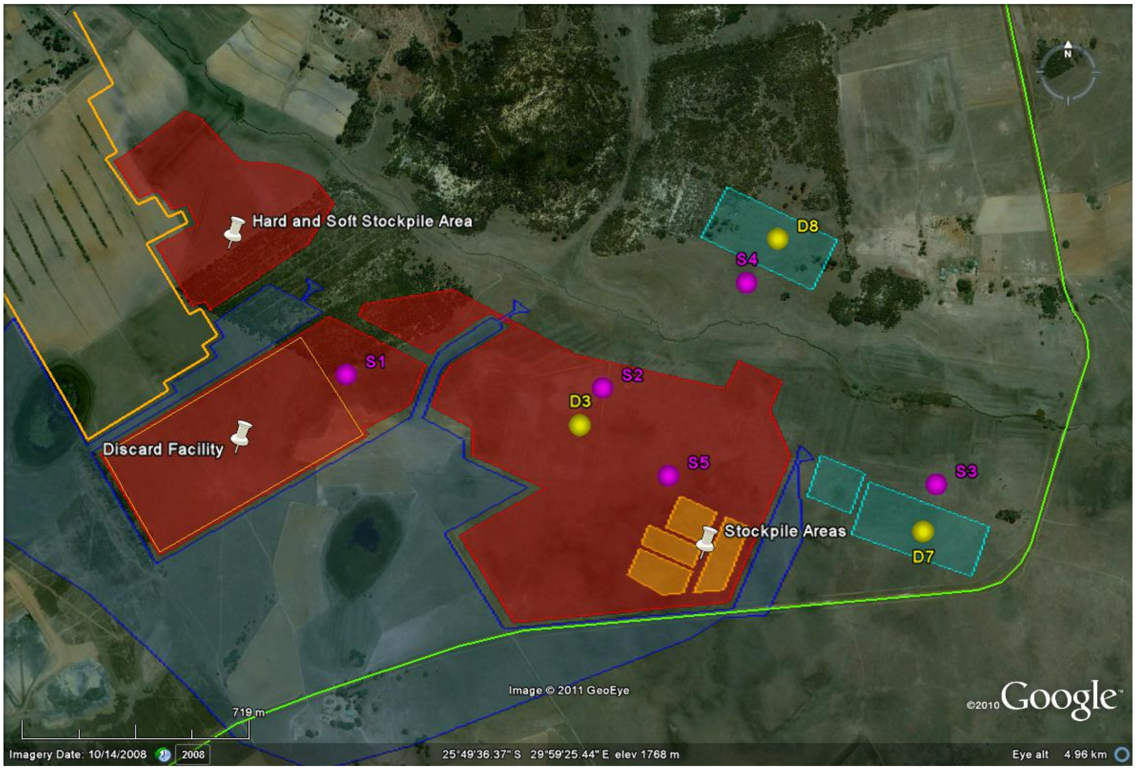
Periodic cleaning of the dams would be required although a 5-10% allowance was made for silt. Cleaning methods and structures designed during the detail design phase should be such that they do not compromise the proposed linings.

13.10 LEAKAGE DETECTION (D3, D7 AND D8)

Dams with high potential hazard liquid and dams operated at a full level should be monitored for leakage of the lining. The cuspated drain layer should be drained by pipes to a collection sump. The position of monitoring sumps is indicated on Figure 25 in purple. Sumps S3, S3 and S4 are applicable. Sumps S1 and S5 relate to seepage collection dealt with under Section 14.

Water from the sumps are to be pumped back to the Process Water System.

Figure 25: Position of Leakage Detection Sumps (Purple)



14. DISCARD DUMP AND STOCKPILE SEEPAGE PREVENTION

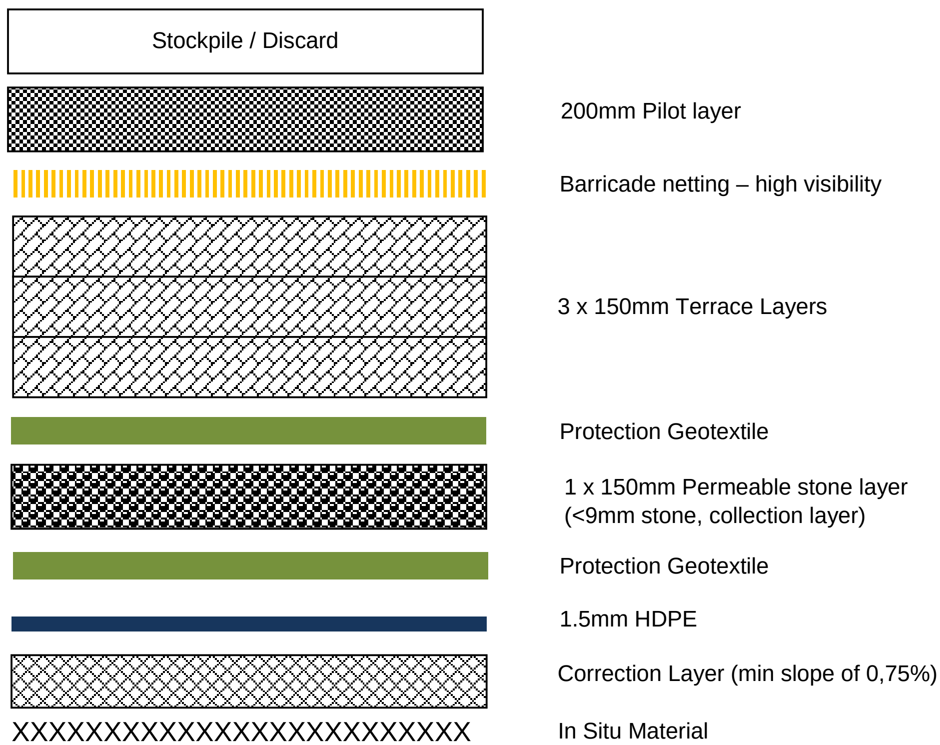
14.1 LINING DESIGNS

Runoff and seepage from these areas must be prevented from contaminating clean surface water and groundwater. It is preferred to intercept seepage before it enters the ground.

The proposed solution is to make use of a combination of an impermeable layer and a seepage collection layer. Special attention should be given to the edges of this system so contamination does not occur at this interface with the natural ground.

The proposed lining system is shown below:

Figure 26: Seepage Collection System



Note: Additional subsurface drainage layers may be required where groundwater is expected. Placement would be between 1.5mm HDPE and In Situ Material.

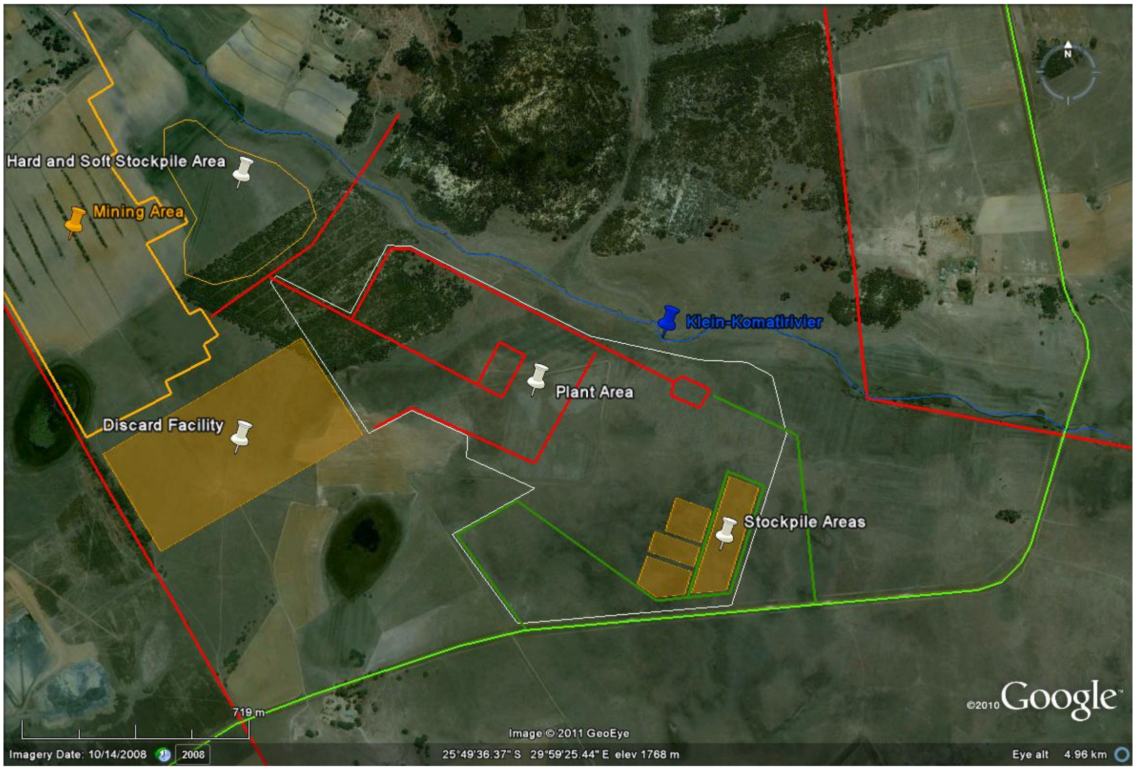
Figure 27 indicates the lined areas (shaded orange) for collection of seepage before contamination of groundwater. It includes the Discard Facility, Export Stockpile, Middlings Stockpile, Emergency Stockpiles and Export and Discard Bin areas. Refer to Drawing 002802-BP- 1 for the exact location.

14.2 SEEPAGE COLLECTION

Seepage is to be collected by a network of pipes e.g. herringbone pattern, and discharged to a collection sump to be pump by means of level control to the Process Water System.

Figure 26 (Section 13.10) and sumps S1 and S5 refer. S1 serves the Discard Facility Area and S5 the Middling and Export Stockpile Areas.

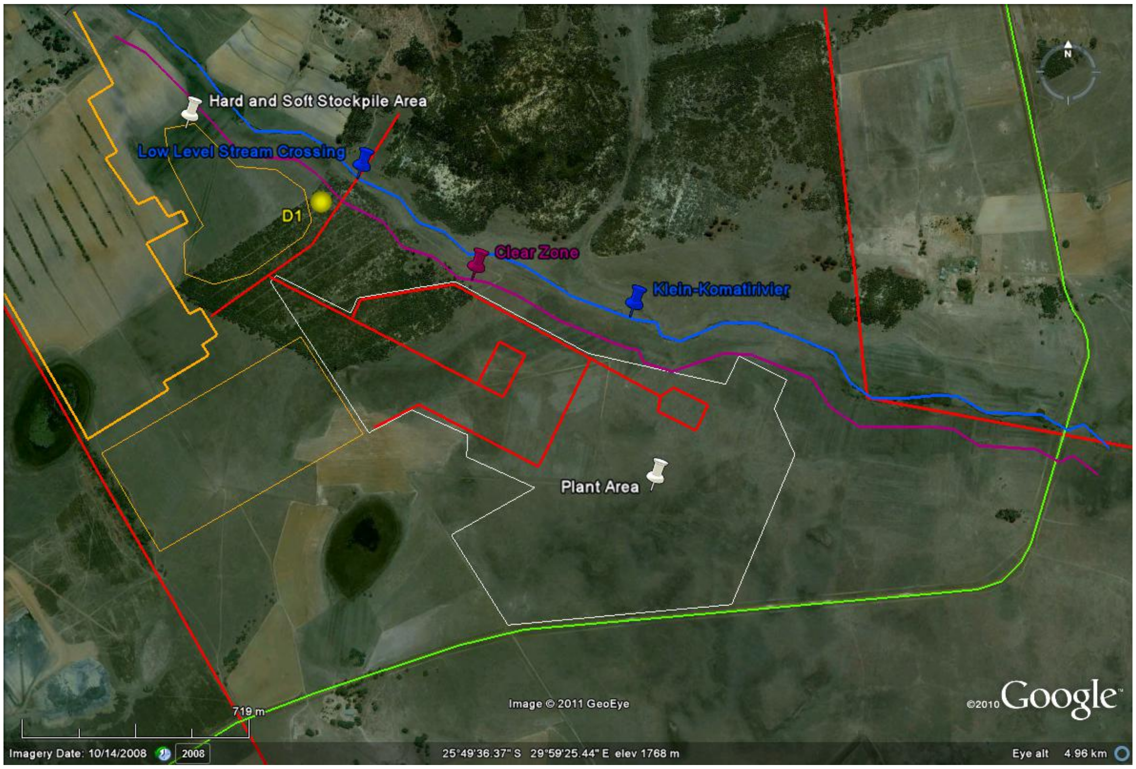
Figure 27: Lined Stockpile Areas (shaded orange)



15. LOW LEVEL STREAM CROSSING

The haul road crosses the Klein-Komatirivier northwest of the plant area (refer to Figure 28 for position). This position is close to and will replace an existing river crossing. The Surface Water Assessment Report (Golder and Associates, 2009) presents a flood-line modelling of this section of river before and after a proposed a low level structure consisting of 5/2mx1.8m (barrels / width x height) pre-cast culverts (Figures 8 and 9 of the said report).

Figure 28: Position of Low Level Stream Crossing



15.1 HYDROLOGY CALCULATIONS

The hydrology of the area was repeated with the following results as set out in Annexure A:

- Catchment area — $23km^2$
- Runoff coefficient C of between — 0.36 and 0.44
- Runoff peak for 1:50 — $76m^3/s$
- Runoff peak for 1:100 — $105m^3/s$

15.2 STRUCTURES INVESTIGATED

Three different culvert and road overflow combinations were used as presented in Annexure E. The smallest culvert opening considered was 1.5mx1.5m as smaller openings can get blocked easily with debris from these size storms. The flood levels immediately upstream of the structure were calculated as follows (refer to Annexure E for detailed calculations):

Table 17: Low Level Crossing Flood Levels

Culvert[1]	Road Overflow Width	1:50 Level	1:100 Level
5/2.0mx1.8m BC	32m	1762.80	1763.17
10/1.5mx1.5m BC	39m	1762.29	1762.64
14/1.5mx1.5m BC	39m	1761.77	1762.25
Streambed Level is 1760m			

[1] *Barrels/widthxheight*

From the above it can be seen that the flood levels for the 5/2.0mx1.8m is much higher than the other two options. The reason for this is the confinement of flow through a narrow but high opening.

The flood level is more sensitive to the number of barrels than road overflow width, as the total deck thickness is estimated to be 500mm. This deck thickness is due to structural strength requirements of the design haul vehicles. The deck thickness increases the headwater depth at the inlet forcing more water through the culvert.

At the position of the proposed low level crossing, the 100m clear zone restriction from the river is more critical than the 1:100 year flood-line level. The footprint of storm water dam D1 is therefore well away from the flood line and any possible undermining.

15.3 RECOMMENDED STRUCTURE

A lower and wider low level structure (1.5mx1.5m box culvert solution) is preferred as;

- this will reduce the average flow velocity,
- have a lower flood level and,
- will be closer to the conditions prior to development.

The additional cost of 14/1.5mx1.5m box culverts brings about little advantages and the 10/1.5mx1.5m box culverts are proposed.

As can be seen from the detailed calculations in Annexure E, “LowLevB” option, the structure will not overtop up to the 1:10 year event. Crossing the structure is possible up to the 1:50 year event.

Table 18: Proposed Low Level Structure

10/1.5m x 1.5m Box Culverts				
Property	**1:10 Year**	**1:20 Year**	**1:50 Year**	**1:100 Year**
Road Overtopping Width	0	39m	39m	39m
Max. Overtopping Depth	0	0.11m	0.19m	0.54m
Flood Peak Runoff Rate	38.6m^3/s	50.6m^3/s	76.0m^3/s	105.7m^3/s
Flood Level (Upstream)	1761.9	1762.2	1762.3	1762.6
Streambed Level	1760	1760	1760	1760
Average Through Velocity[1]	2.1m/s	2.0m/s	1.96m/s	1.8m/s
Pre-development Velocity	1.6m/s	1.7m/s	1.8m/s	2.0m/s

[1].*Note: The Average Through Velocity takes the headwater build-up into account as well, and is not the outlet velocity! The outlet velocity is to be reduced by energy dissipation to pre-development velocities as discussed in the section below.*

15.4 PROTECTION AND DETAILS

The streambed needs to be protected upstream and downstream of the culvert and it is recommended to use gabions as energy dissipaters. The length of road that will be flooded must also be protected and the road surface is to be constructed of concrete with anchors to tie gabion mattresses to that protect the side slopes. Refer to Drawing 002802-BP- 8 for more detail.

16. SEWERAGE TREATMENT PLANT AND BIOFILTER DAM

16.1 BIOFILTER DAM (D6) SIZING

The design brief entailed the conceptual design for an evaporation pond to dispose of the domestic sewage from the new mine. The evaporation pond will receive treated effluent from a package sewage treatment plant.

The final effluent from the package sewage treatment plant (General Limit effluent standard) cannot be discharged off site, as the mine is within a Special Limit catchment (i.e. all tributires of the Komati River between Nootgedacht Dam and the confluent with the Sevenfonteinspruit, Table 3.3 Listed Resources, National Water Act of 1998). It is rather onerous to achieve the Special Limit effluent quality. Some alternatives to this concept are included later.

The rationale for the sizing of the sewage treatment plant and the evaporation pond is as follows.

16.1.1 Sizing Of The Sewage Treatment Plant

The estimated domestic sewage flow for the facility was based on SANS 10252-2 (Table 9) as follows:

a) Office Staff :

225 people per day working 1 x 8hour shift, amounts to a total of 15.75m^3/d.

b) Labour:

225 people per day, including shower use (7 days a week), amounts to a total of 33.75m^3/d.

c) Canteen:

450 meals per day amounts to a total of 13,5m^3/d

Thus the estimated total sewage flow from the facility is 63m^3/d.

16.1.2 Sizing Of The Evaporation Pond

Table 19 shows the average evaporation data (GCS, Jan 2011) that was used as a basis for the sizing of the evaporation pond.

Table 19: Evaporation Data Used In The Modelling

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Tot
Pan Evaporation (mm)	189	169	163	122	106	88	93	129	175	195	185	200	1814
Precipitation (mm)	138	87	71	43	12	6	4	7	25	72	125	124	714
Net Evaporation (mm)	51	82	92	79	94	82	89	122	150	123	60	76	1100

The evaporation from the pond was modelled over a 4 year period using the average evaporation data and a consistent flow of $63m^3/d$. It was determined that an evaporation pond of 2.0ha will be adequate assuming an average depth of 1.5m. The results of the evaporation modelling on the dam volume are given in Figure 29.

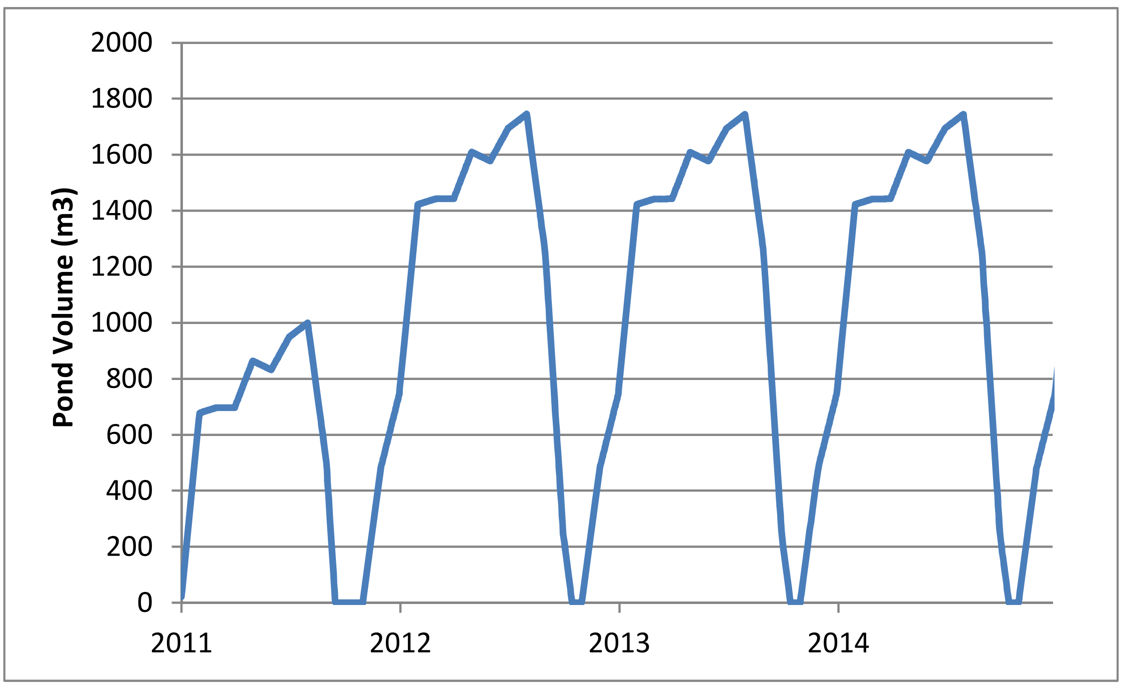


Figure 29: Result Of Pond Evaporation Modelling (1.5m Depth)

16.1.3 Package Sewage Treatment Plant

There are numerous commercial package sewage treatment plants available on the market in South Africa that are designed to treat to the General Limit effluent standard. Since the effluent is to be evaporated or irrigated, the General Limit effluent standard would be more than sufficient to comply with the standards set by the Department of Water Affairs.

It is therefore recommended that a package plant based on the extended aeration activated sludge process be used. The package plant should have facilities to store and possibly digest sludge in order to minimise the maintenance required by the mine.

16.2 ALTERNATIVE STRATEGIES

The following alternative strategies can be considered:

16.2.1 Effluent Irrigation

The effluent from the package sewage treatment plant would be of a quality that could be suitable for irrigation, but not for discharge off site. The quantity of $63m^3/d$ is above the limit of $50m^3/d$ which would allow this to be done in terms of a General Authorisation and as such an application for the registration of a water use must be submitted to DWA before the irrigation can commence. An area of between 2 and 3 hectares would probably be sufficient for this irrigation and could include the use of the water for dust suppression. However, the following restrictions would apply:

- the irrigation cannot take place below the 100 year flood line,
- the irrigation cannot take place less than 100 metres from a water resource or a borehole which is utilised for drinking water or stock watering; and
- the irrigation cannot take place if the land overlies a Major Aquifer

16.2.2 Grey Water Diversion And Low Flow Fittings

The shower water makes up approximately 30% of the total sewage flow and this fraction could be separated before the sewage treatment plant and irrigated. This would reduce the size of the evaporation pond by 30%. This shower water could irrigate between 0.5 and 1.0 hectares.

The calculation of the sewage flow is based on conventional sanitation fittings. Further reductions of the sewage flow could be achieved by means of low flow fittings, hold flush toilets and waterless urinals. This could thus result in a reduction in the required size of the evaporation pond by a further 10% to 30%.

16.2.3 Constructed Reedbed

An alternative to the package plant would be to construct a reedbed preceded by a large septic tank. This would have the advantage of not requiring electricity to operate and would significantly reduce the maintenance requirements. An area of between $2500m^2$ and $3000m^2$ would be required for a reedbed to treat the sewage from 450 people. This could be reduced if the grey water from the showers is diverted prior to the septic tank. The reedbed would have to be lined, preferably with a GCL and would be planted with a commonly available reed.

The final effluent would probably not meet General Limit standard effluent quality, but would be suitable for irrigation in terms of Clause 2.7 (1) of the National Water Act. The final effluent would however have to be disinfected before irrigation takes place. The septic tank would be sized for at least 24 hours hydraulic retention and it would be advisable to construct 2 x 32 m^3 parallel tanks so that one tank can be cleaned without shutting down the entire tank. Either a 2 or 3 compartment septic tank would be required, and a manual raked screen should be considered. Sludge would have to be removed from the septic tank once a year.

17. CONCLUSIONS

It is possible, as demonstrated, to comply with the requirements of the regulations without altering the plant layout dramatically. A slight rotation of the plant as well as a relaxation of the building line requirements improved the storm water management system.

Further in investigations need to be carried out such as:

- Surface geotechnical investigation where access was restricted.
- Borrow area identification (e.g. source of clay)
- Topographical survey

Structures (steel and concrete) in contact with the ground need to be designed to withstand the corrosive soil conditions.

Lining requirements may be relaxed once more accurate data is available as to the chemical composition and acidity of water in the area. The consumptions made are conservative.

Minor adjustments to the layouts may be required once more detailed information is available.

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Annexure A:
SITE HYDROLOGY CALCULATIONS

CATCHMENT HYDROLOGY

Project Number: **002802**
Project Title: **Belfast Stormwater**
Done by: **CCLR**
Date: **23 July 2011**
Spreadsheet by RLR

Ref	Catchment No		PW-50	PW-100	PE-50	PE-100	C2-50	C3-50	C4-50	C5-50	D1-50	D1-100	D2-50	D2-100	D3-50	D3-100	D4-50	D4-100	D5-10	D5-50	D5-100	D6-50
1	Approx. Culvert km	km																				
2	**SUMMARY**																					
3	Rational Method	m³/sec	7.959	11.806	8.650	12.830	7.917	0.915	8.730	12.130	4.485	5.396	6.474	8.947	0.841	1.035	0.154	0.190	4.521	10.136	15.034	1.286
4	SDF Method	m³/sec	9.461	12.183	10.249	13.198	8.981	1.062	9.903	13.356	4.206	5.416	6.236	8.385	0.317	0.408	0.058	0.075	3.971	9.075	11.686	0.485
5	Empirical Method	m³/sec	11.959	15.106	12.604	15.921	11.466	2.109	12.398	15.671	6.608	8.347	8.276	10.666	n.a.	n.a.	n.a.	n.a.	8.194	13.194	16.666	n.a.

Ref	Universal Input Data		PW-50	PW-100	PE-50	PE-100	C2-50	C3-50	C4-50	C5-50	D1-50	D1-100	D2-50	D2-100	D3-50	D3-100	D4-50	D4-100	D5-10	D5-50	D5-100	D6-50
6	**Universal Input Data**																					
7	Return Period (T)	years	50	100	50	100	50	50	50	50	50	100	50	100	50	100	50	100	10	50	100	50
8	Catchment Area (A)	km²	0.480	0.480	0.528	0.528	0.592	0.061	0.653	1.133	0.246	0.246	0.336	0.336	0.013	0.013	0.002	0.002	0.858	0.858	0.858	0.020
9	Main Channel length (L)	km	0.700	0.700	0.760	0.760	1.120	0.800	1.120	1.900	0.680	0.680	0.920	0.920	0.000	0.000	0.000	0.000	1.200	1.200	1.200	0.000
10	Mean Annual Rainfall (MAP)	mm	690	690	690	690	690	690	690	690	690	690	690	690	690	690	690	690	690	690	690	690

Steps	Rational Method Calculations (for areas smaller than 15 km²)		PW-50	PW-100	PE-50	PE-100	C2-50	C3-50	C4-50	C5-50	D1-50	D1-100	D2-50	D2-100	D3-50	D3-100	D4-50	D4-100	D5-10	D5-50	D5-100	D6-50
1	Catchment Area (A)	km²	0.480	0.480	0.528	0.528	0.592	0.061	0.653	1.133	0.246	0.246	0.336	0.336	0.013	0.013	0.002	0.002	0.858	0.858	0.858	0.020
2	Main Channel length (L)	km	0.7	0.7	0.76	0.76	1.12	0.8	1.12	1.9	0.68	0.68	0.92	0.92	0	0	0	0	1.2	1.2	1.2	0
3	Average Slope (10-85) (S)	m/m	0.020	0.020	0.022	0.022	0.016	0.015	0.016	0.017	0.010	0.010	0.026	0.032	-	-	-	-	0.004	0.004	0.004	-
4	Time of Concentration (Tc)	hours	0.227	0.227	0.234	0.234	0.355	0.281	0.355	0.524	0.292	0.292	0.253	0.234	0.150	0.150	0.150	0.150	0.614	0.614	0.614	0.150
5	Mean Annual Rainfall (MAR)	mm	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0
6	Region		Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland
7	Point Intensity (Pit)	mm	197	243	195	240	159	179	159	127	175	216	188	239	231	285	231	285	71	115	142	231
8	Area Reduction Factor (ARF)		100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
9	Average Rainfall Intensity (IT)	mm	197.05	242.60	194.67	239.67	158.91	178.74	158.91	127.24	175.43	215.98	187.79	239.35	231.44	284.94	231.44	284.94	71.08	115.19	141.82	231.44
10a	Dolomite Area	%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
10	Ct = αC1d+βC2*γC3		0.303	0.365	0.303	0.365	0.303	0.303	0.303	0.303	0.374	0.365	0.369	0.401	1.000	1.000	1.000	1.000	0.267	0.369	0.445	1.000
11	**Peak Flow for 1: 50 years (Qt)**	m³/sec	**7.959**	**11.806**	**8.650**	**12.830**	**7.917**	**0.915**	**8.730**	**12.130**	**4.485**	**5.396**	**6.474**	**8.947**	**0.841**	**1.035**	**0.154**	**0.190**	**4.521**	**10.136**	**15.034**	**1.286**

Steps	SDF Method Calculations (no limit on area size)		PW-50	PW-100	PE-50	PE-100	C2-50	C3-50	C4-50	C5-50	D1-50	D1-100	D2-50	D2-100	D3-50	D3-100	D4-50	D4-100	D5-10	D5-50	D5-100	D6-50
1	Basin number	no.	29	29	29	29	29	29	29	29	29	29	29	29	29	29	29	29	29	29	29	29
6a	Time of concentration (t)	minutes	14	14	14	14	21	17	21	31	18	18	15	14	9	9	9	9	37	37	37	9
6b	Mean annual daily maxima (M)	mm	66	66	66	66	66	66	66	66	66	66	66	66	66	66	66	66	66	66	66	66
6c	Audible Thunder (R)	days/yr	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11
6d	Point precipitation depth (Pt)	mm	45	52	46	53	54	49	54	62	50	58	47	53	37	42	37	42	42	65	75	37
7a	Area Reduction Factor (ARF)	%	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
7b	Catchment Rainfall for return period (T	mm	45.06	51.92	45.62	52.57	54.17	49.42	54.17	62.12	50.21	57.85	47.27	52.64	36.57	42.13	36.57	42.13	42.26	65.37	75.32	36.57
7c	Rainfall Intensity (It)	mm/hr	198.2	228.4	195.2	225.0	152.6	175.6	152.6	118.6	171.7	197.8	186.7	224.6	243.8	280.9	243.8	280.9	68.8	106.4	122.6	243.8
8d	Runoff coefficient Ct		0.358	0.400	0.358	0.400	0.358	0.358	0.358	0.358	0.358	0.400	0.358	0.400	0.358	0.400	0.358	0.400	0.242	0.358	0.400	0.358
9	**Peak Flow (Qt)**	m³/sec	**9.461**	**12.183**	**10.249**	**13.198**	**8.981**	**1.062**	**9.903**	**13.356**	**4.206**	**5.416**	**6.236**	**8.385**	**0.317**	**0.408**	**0.058**	**0.075**	**3.971**	**9.075**	**11.686**	**0.485**

Steps	Empirical Method Calculations (no limit on area size - prefer larger areas)		PW-50	PW-100	PE-50	PE-100	C2-50	C3-50	C4-50	C5-50	D1-50	D1-100	D2-50	D2-100	D3-50	D3-100	D4-50	D4-100	D5-10	D5-50	D5-100	D6-50
5	Veld Type		4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a
6	Catchment Parameters (C)		0.240	0.240	0.234	0.234	0.104	0.020	0.114	0.071	0.091	0.091	0.111	0.123	-	-	-	-	0.069	0.069	0.069	-
7	Kovacs Region		K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5
9	Constant Value for (Kt)		0.950	1.200	0.950	1.200	0.950	0.950	0.950	0.950	0.950	1.200	0.950	1.200	0.950	1.200	0.950	1.200	0.590	0.950	1.200	0.950
10	Peak Flow (Midgley & Pitman) (Qt)	m³/sec	11.959	15.106	12.604	15.921	11.466	2.109	12.398	15.671	6.608	8.347	8.276	10.666	-	-	-	-	8.194	13.194	16.666	-
11a	Peak Flow (Qrmf Kovacs factor 1)		100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
11b	Peak Flow (Qrmf Kovacs factor 2)		0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
11	Peak Flow (Kovacs) (Qrmf)	m³/sec	6.928	6.928	7.266	7.266	7.694	2.466	8.080	10.643	4.964	4.964	5.797	5.797	1.144	1.144	0.490	0.490	9.261	9.261	9.261	1.414
12	Qt/Qrmf ratios		1.726	2.180	1.735	2.191	1.490	0.855	1.535	1.472	1.331	1.682	1.428	1.840	-	-	-	-	0.885	1.425	1.800	-
12	**Peak Flow (Qt)**	m³/sec	**11.959**	**15.106**	**12.604**	**15.921**	**11.466**	**2.109**	**12.398**	**15.671**	**6.608**	**8.347**	**8.276**	**10.666**	**-**	**-**	**-**	**-**	**8.194**	**13.194**	**16.666**	**-**

CATCHMENT HYDROLOGY

Project Number: **002802**
Project Title: **Belfast Stormwater**
Done by: **CCLR**
Date: **23 July 2011**
Spreadsheet by RLR

Ref	Catchment No		D6-100	D7,8-50	D7,8-100	C10-50	C14-50	LL-2	LL-5	LL-10	LL-20	LL-50	LL-100	LL-200
1	Approx. Culvert km	km												
2	**SUMMARY**													
3	Rational Method	m³/sec	1.583	4.372	5.382	5.397	2.370	21.611	29.906	38.614	50.632	75.959	105.691	130.120
4	SDF Method	m³/sec	0.624	1.648	2.122	6.164	2.812	2.906	17.272	31.069	46.954	71.002	91.426	113.224
5	Empirical Method	m³/sec	n.a.	n.a.	n.a.	8.230	4.424	#N/A	#N/A	44.748	51.574	72.052	91.013	91.013

Ref	Universal Input Data		D6-100	D7,8-50	D7,8-100	C10-50	C14-50	LL-2	LL-5	LL-10	LL-20	LL-50	LL-100	LL-200
6	**Universal Input Data**													
7	Return Period (T)	years	100	50	100	50	50	2	5	10	20	50	100	200
8	Catchment Area (A)	km²	0.020	0.068	0.068	0.389	0.144	22.970	22.970	22.970	22.970	22.970	22.970	22.970
9	Main Channel length (L)	km	0.000	0.000	0.000	1.200	0.800	15.000	15.000	15.000	15.000	15.000	15.000	15.000
10	Mean Annual Rainfall (MAP)	mm	690	690	690	690	690	690	690	690	690	690	690	690

Steps	Rational Method Calculations (for areas sm		D6-100	D7,8-50	D7,8-100	C10-50	C14-50	LL-2	LL-5	LL-10	LL-20	LL-50	LL-100	LL-200
1	Catchment Area (A)	km²	0.020	0.068	0.068	0.389	0.144	22.970	22.970	22.970	22.970	22.970	22.970	22.970
2	Main Channel length (L)	km	0	0	0	1.20015	0.8001	15	15	15	15	15	15	15
3	Average Slope (10-85) (S)	m/m	-	-	-	0.022	0.025	0.010	0.010	0.010	0.010	0.010	0.010	0.010
4	Time of Concentration (Tc)	hours	0.150	0.150	0.150	0.331	0.231	3.184	3.184	3.184	3.184	3.184	3.184	3.184
5	Mean Annual Rainfall (MAR)	mm	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0	690.0
6	Region		Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland	Inland
7	Point Intensity (Pit)	mm	285	231	285	165	196	13	17	21	25	33	41	51
8	Area Reduction Factor (ARF)		100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
9	Average Rainfall Intensity (IT)	mm	284.94	231.44	284.94	164.97	195.59	12.75	16.78	20.66	25.43	33.48	41.22	50.74
10a	Dolomite Area	%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
10	Ct = αC1d+βC2*γC3		1.000	1.000	1.000	0.303	0.303	0.266	0.279	0.293	0.312	0.356	0.402	0.402
11	**Peak Flow for 1: 50 years (Qt)**	**m³/sec**	**1.583**	**4.372**	**5.382**	**5.397**	**2.370**	**21.611**	**29.906**	**38.614**	**50.632**	**75.959**	**105.691**	**130.120**

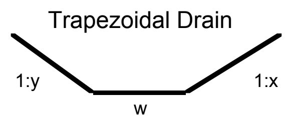
Steps	SDF Method Calculations (no limit on area :		D6-100	D7,8-50	D7,8-100	C10-50	C14-50	LL-2	LL-5	LL-10	LL-20	LL-50	LL-100	LL-200
1	Basin number	no.	29	29	29	29	29	29	29	29	29	29	29	29
6a	Time of concentration (t)	minutes	9	9	9	20	14	191	191	191	191	191	191	191
6b	Mean annual daily maxima (M)	mm	66	66	66	66	66	66	66	66	66	66	66	66
6c	Audible Thunder (R)	days/yr	11	11	11	11	11	11	11	11	11	11	11	11
6d	Point precipitation depth (Pt)	mm	42	37	42	53	45	29	49	64	79	99	114	129
7a	Area Reduction Factor (ARF)	%	100	100	100	100	100	100	100	100	100	100	100	100
7b	Catchment Rainfall for return period (T	mm	42.13	36.57	42.13	52.71	45.40	29.00	48.92	63.99	79.06	98.99	114.06	129.13
7c	Rainfall Intensity (It)	mm/hr	280.9	243.8	280.9	159.5	196.4	9.1	15.4	20.1	24.8	31.1	35.8	40.6
8d	Runoff coefficient Ct		0.400	0.358	0.400	0.358	0.358	0.050	0.176	0.242	0.296	0.358	0.400	0.438
9	**Peak Flow (Qt)**	**m³/sec**	**0.624**	**1.648**	**2.122**	**6.164**	**2.812**	**2.906**	**17.272**	**31.069**	**46.954**	**71.002**	**91.426**	**113.224**

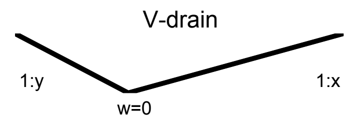
Steps	Empirical Method Calculations (no limit on		D6-100	D7,8-50	D7,8-100	C10-50	C14-50	LL-2	LL-5	LL-10	LL-20	LL-50	LL-100	LL-200
5	Veld Type		4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a	4&5a
6	Catchment Parameters (C)		-	-	-	0.070	0.062	0.017	0.017	0.017	0.017	0.017	0.017	0.017
7	Kovacs Region		K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5	K5
9	Constant Value for (Kt)		1.200	0.950	1.200	0.950	0.950	#N/A	#N/A	0.590	0.680	0.950	1.200	1.200
10	Peak Flow (Midgley & Pitman) (Qt)	m³/sec	-	-	-	8.230	4.424	#N/A	#N/A	44.748	51.574	72.052	91.013	91.013
11a	Peak Flow (Qrmf Kovacs factor 1)		100	100	100	100	100	100	100	100	100	100	100	100
11b	Peak Flow (Qrmf Kovacs factor 2)		0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
11	Peak Flow (Kovacs) (Qrmf)	m³/sec	1.414	2.608	2.608	6.235	3.795	47.927	47.927	47.927	47.927	47.927	47.927	47.927
12	Qt/Qrmf ratios		-	-	-	1.320	1.166	#N/A	#N/A	0.934	1.076	1.503	1.899	1.899
12	**Peak Flow (Qt)**	**m³/sec**	**-**	**-**	**-**	**8.230**	**4.424**	**#N/A**	**#N/A**	**44.748**	**51.574**	**72.052**	**91.013**	**91.013**

Annexure B:
DRAIN SIZING

UNIFORM OPEN CHANNEL FLOW - MANNING

Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**
	Spreadsheet by RLR





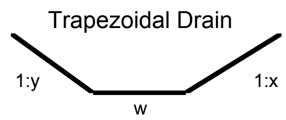
Simplified Target Values

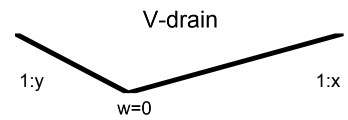
Lining and Roughness (Manning n)	Max. m/s	1:max
Concrete □(0.0140-0.005)	6.0	1
Armorflex 140 □(0.0300-0.150)	3.0	2
Armorflex 180 □(0.0300-0.150)	6.0	2
Grass Long □(0.0360-0.400)	1.8	4
Grass Short □(0.0320-0.200)	1.5	4
Max velocity to encourage silt depositing		0.8

Section	w (m)	y (m)	x (m)	Ground Slope	Type	Flow depth (m)	Manning n	Lining n-k	k (m)	Flow Width (m)	Wetted Area (m2)	Wetted Perim. (m)	Hydraulic Radius, R	Flow Q (m3/s)	Peak Flow (m3/s)	V (m/s)	Freeb Straight (m)	Drain Depth (m)	Notes	Traffic
	5.00	3.00	3.00	0.030		0.34	0.032		0.156	7.03	2.040	7.144	0.286	4.787		2.347	0.102	0.441		
	5.00	3.00	3.00	0.030		0.45	0.032		0.174	7.71	2.873	7.859	0.366	7.950		2.767	0.136	0.588		
C1-C30	5.00	3.00	3.00	0.030	Trapezoidal	0.57	0.032	Armorflex 140 (0.0300-0.150)	0.188	8.39	3.783	8.573	0.441	11.866	11.806	3.137	0.170	0.735	Energy dissipation rqd	None
	5.00	3.00	3.00	0.030		0.68	0.032		0.200	9.07	4.769	9.288	0.513	16.552		3.471	0.203	0.881		
	5.00	3.00	3.00	0.030		0.79	0.032		0.210	9.75	5.832	10.003	0.583	22.031		3.778	0.237	1.028		
	5.00	3.00	3.00	0.030		0.29	0.032		0.145	6.71	1.674	6.808	0.246	3.557		2.125	0.086	0.372		
	5.00	3.00	3.00	0.030		0.38	0.032		0.163	7.29	2.341	7.410	0.316	5.878		2.511	0.114	0.495		
C2-C5A	5.00	3.00	3.00	0.030	Trapezoidal	0.48	0.032	Armorflex 140 (0.0300-0.150)	0.177	7.86	3.063	8.013	0.382	8.730	8.730	2.851	0.143	0.619	Energy dissipation rqd	None
	5.00	3.00	3.00	0.030		0.57	0.032		0.189	8.43	3.838	8.615	0.446	12.119		3.157	0.171	0.743		
	5.00	3.00	3.00	0.030		0.67	0.032		0.199	9.00	4.669	9.218	0.506	16.057		3.439	0.200	0.867		
	8.00	3.00	3.00	0.030		0.27	0.032		0.145	9.62	2.381	9.709	0.245	5.048		2.121	0.081	0.351		
	8.00	3.00	3.00	0.030		0.36	0.032		0.163	10.16	3.272	10.279	0.318	8.256		2.523	0.108	0.468		
C5A-C5B	8.00	3.00	3.00	0.030	Trapezoidal	0.45	0.032	Armorflex 140 (0.0300-0.150)	0.178	10.70	4.211	10.848	0.388	12.130	12.130	2.880	0.135	0.585	Energy dissipation rqd	None
	8.00	3.00	3.00	0.030		0.54	0.032		0.190	11.24	5.200	11.418	0.455	16.659		3.204	0.162	0.703		
	8.00	3.00	3.00	0.030		0.63	0.032		0.201	11.78	6.237	11.988	0.520	21.836		3.501	0.189	0.820		
	2.00	3.00	3.00	0.030		0.13	0.032		0.098	2.79	0.316	2.835	0.112	0.397		1.254	0.040	0.172		
	2.00	3.00	3.00	0.030		0.18	0.032		0.112	3.06	0.445	3.113	0.143	0.658		1.480	0.053	0.229		
C3-C2	2.00	3.00	3.00	0.030	Trapezoidal	0.22	0.032	Grass Short (0.0320-0.200)	0.123	3.32	0.585	3.391	0.173	0.982	0.915	1.678	0.066	0.286	Energy dissipation rqd	None
	2.00	3.00	3.00	0.030		0.26	0.032		0.132	3.58	0.737	3.670	0.201	1.368		1.856	0.079	0.343		
	2.00	3.00	3.00	0.030		0.31	0.032		0.140	3.85	0.901	3.948	0.228	1.820		2.021	0.092	0.400		
	2.00	3.00	3.00	0.030		0.13	0.032		0.098	2.79	0.316	2.835	0.112	0.397		1.254	0.040	0.172		
	2.00	3.00	3.00	0.030		0.18	0.032		0.112	3.06	0.445	3.113	0.143	0.658		1.480	0.053	0.229		
C3-C2 Alternative	2.00	3.00	3.00	0.030	Trapezoidal	0.22	0.032	Armorflex 140 (0.0300-0.150)	0.123	3.32	0.585	3.391	0.173	0.982	0.915	1.678	0.066	0.286	Energy dissipation rqd	None
	2.00	3.00	3.00	0.030		0.26	0.032		0.132	3.58	0.737	3.670	0.201	1.368		1.856	0.079	0.343		
	2.00	3.00	3.00	0.030		0.31	0.032		0.140	3.85	0.901	3.948	0.228	1.820		2.021	0.092	0.400		
	2.00	3.00	3.00	0.030		0.10	0.032		0.087	2.62	0.237	2.650	0.089	0.257		1.083	0.031	0.134		
	2.00	3.00	3.00	0.030		0.14	0.032		0.100	2.82	0.330	2.866	0.115	0.423		1.282	0.041	0.178		
C29-C6	2.00	3.00	3.00	0.030	Trapezoidal	0.17	0.032	Grass Short (0.0320-0.200)	0.110	3.03	0.430	3.083	0.140	0.627	0.627	1.457	0.051	0.223		None
	2.00	3.00	3.00	0.030		0.21	0.032		0.119	3.23	0.538	3.300	0.163	0.868		1.615	0.062	0.267		
	2.00	3.00	3.00	0.030		0.24	0.032		0.127	3.44	0.652	3.516	0.185	1.147		1.760	0.072	0.312		
	2.00	3.00	3.00	0.008		0.31	0.030		0.114	3.86	0.909	3.962	0.229	0.983		1.082	0.074	0.384		
	2.00	3.00	3.00	0.008		0.41	0.030		0.126	4.48	1.340	4.615	0.290	1.696		1.266	0.099	0.513		
C6-D5	2.00	3.00	3.00	0.008	Trapezoidal	0.52	0.030	Armorflex 140 (0.0300-0.150)	0.135	5.10	1.835	5.269	0.348	2.623	2.623	1.429	0.124	0.641		None
	2.00	3.00	3.00	0.008		0.62	0.030		0.143	5.72	2.395	5.923	0.404	3.780		1.578	0.149	0.770		
	2.00	3.00	3.00	0.008		0.72	0.030		0.150	6.34	3.018	6.577	0.459	5.184		1.718	0.175	0.899		
	2.00	2.00	2.00	0.008		0.23	0.015		0.004	2.91	0.561	3.022	0.186	1.055		1.880	0.069	0.297		
	2.00	2.00	2.00	0.008		0.30	0.015		0.004	3.22	0.795	3.362	0.236	1.754		2.207	0.091	0.396		
6-D5 rnative)	2.00	2.00	2.00	0.008	pezoidal	0.38	0.015	ncrete 40-0.005)	0.004	3.52	1.052	3.703	0.284	2.623	2.623	2.494	0.114	0.495	nergy ation rqd	lone

UNIFORM OPEN CHANNEL FLOW - MANNING

Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**
	Spreadsheet by RLR





Simplified Target Values

Lining and Roughness (Manning n)	Max. m/s	1:max
Concrete □(0.0140-0.005)	6.0	1
Armorflex 140 □(0.0300-0.150)	3.0	2
Armorflex 180 □(0.0300-0.150)	6.0	2
Grass Long □(0.0360-0.400)	1.8	4
Grass Short □(0.0320-0.200)	1.5	4
Max velocity to encourage silt depositing	0.8	

Section	w (m)	y (m)	x (m)	Ground Slope	Type	Flow depth (m)	Manning n	Lining n-k	k (m)	Flow Width (m)	Wetted Area (m2)	Wetted Perim. (m)	Hydraulic Radius, R	Flow Q (m3/s)	Peak Flow (m3/s)	V (m/s)	Freeb Straight (m)	Drain Depth (m)	Notes	Traffic
C Alte	2.00	2.00	2.00	0.008	Trap	0.46	0.015	Co (0.014	0.004	3.83	1.331	4.043	0.329	3.666		2.753	0.137	0.594	E dissip	N
	2.00	2.00	2.00	0.008		0.53	0.015		0.004	4.13	1.635	4.384	0.373	4.889		2.991	0.160	0.693		
	2.00	3.00	3.00	0.016		0.15	0.032		0.103	2.88	0.357	2.926	0.122	0.347		0.972	0.039	0.185		
	2.00	3.00	3.00	0.016		0.20	0.032		0.117	3.17	0.505	3.234	0.156	0.578		1.146	0.052	0.248		
C18-C16	2.00	3.00	3.00	0.016	Trapezoidal	0.24	0.032	Grass Short (0.0320-0.200)	0.128	3.46	0.666	3.543	0.188	0.865	0.865	1.298	0.066	0.310		None
	2.00	3.00	3.00	0.016		0.29	0.032		0.138	3.76	0.843	3.851	0.219	1.209		1.435	0.080	0.372		
	2.00	3.00	3.00	0.016		0.34	0.032		0.146	4.05	1.033	4.160	0.248	1.613		1.562	0.093	0.435		
	1.00	10.00	10.00	0.016		0.05	0.015		0.004	1.96	0.071	1.965	0.036	0.066		0.922	0.014	0.062		
	1.00	10.00	10.00	0.016		0.06	0.015		0.004	2.28	0.105	2.286	0.046	0.113		1.081	0.019	0.083		
Middlings-C23	1.00	10.00	10.00	0.016	Trapezoidal	0.08	0.015	Concrete (0.0140-0.005)	0.004	2.60	0.144	2.608	0.055	0.176	0.176	1.223	0.024	0.104		Yes
	1.00	10.00	10.00	0.016		0.10	0.015		0.004	2.92	0.188	2.930	0.064	0.254		1.352	0.029	0.125		
	1.00	10.00	10.00	0.016		0.11	0.015		0.004	3.24	0.237	3.251	0.073	0.350		1.473	0.034	0.146		
	0.00	5.00	5.00	0.016		0.09	0.015		0.004	0.92	0.042	0.937	0.045	0.045		1.068	0.028	0.119		
	0.00	5.00	5.00	0.016		0.12	0.015		0.004	1.23	0.075	1.249	0.060	0.097		1.293	0.037	0.159		
Middlings-C23 Alternative	0.00	5.00	5.00	0.016	V-drain	0.15	0.015	Concrete (0.0140-0.005)	0.004	1.53	0.117	1.562	0.075	0.176	0.176	1.501	0.046	0.199		Yes
	0.00	5.00	5.00	0.016		0.18	0.015		0.004	1.84	0.169	1.874	0.090	0.286		1.695	0.055	0.239		
	0.00	5.00	5.00	0.016		0.21	0.015		0.004	2.14	0.230	2.186	0.105	0.432		1.878	0.064	0.279		
	1.00	5.00	5.00	0.016		0.05	0.015		0.004	1.53	0.067	1.539	0.043	0.070		1.042	0.016	0.069		
	1.00	5.00	5.00	0.016		0.07	0.015		0.004	1.70	0.095	1.719	0.055	0.117		1.226	0.021	0.092		
Middlings-C23 Alternative	1.00	5.00	5.00	0.016	Trapezoidal	0.09	0.015	Concrete (0.0140-0.005)	0.004	1.88	0.127	1.899	0.067	0.176	0.176	1.389	0.026	0.115		Yes
	1.00	5.00	5.00	0.016		0.11	0.015		0.004	2.06	0.162	2.078	0.078	0.248		1.536	0.032	0.137		
	1.00	5.00	5.00	0.016		0.12	0.015		0.004	2.23	0.199	2.258	0.088	0.334		1.672	0.037	0.160		
	1.00	2.00	2.00	0.016		0.15	0.015		0.004	1.60	0.195	1.672	0.117	0.394		2.016	0.045	0.195		
	1.00	2.00	2.00	0.016		0.20	0.015		0.004	1.80	0.281	1.896	0.148	0.662		2.359	0.060	0.260		
C23-P25	1.00	2.00	2.00	0.016	Trapezoidal	0.25	0.015	Concrete (0.0140-0.005)	0.004	2.00	0.376	2.120	0.177	1.000	1.000	2.661	0.075	0.326	Energy dissipation rqd	Yes
	1.00	2.00	2.00	0.016		0.30	0.015		0.004	2.20	0.481	2.344	0.205	1.412		2.934	0.090	0.391		
	1.00	2.00	2.00	0.016		0.35	0.015		0.004	2.40	0.596	2.568	0.232	1.900		3.186	0.105	0.456		
	1.00	10.00	10.00	0.005		0.06	0.015		0.004	2.15	0.090	2.154	0.042	0.051		0.569	0.015	0.072		
	1.00	10.00	10.00	0.005		0.08	0.015		0.004	2.53	0.135	2.538	0.053	0.090		0.667	0.020	0.096		
Emer-C24	1.00	10.00	10.00	0.005	Trapezoidal	0.10	0.015	Concrete (0.0140-0.005)	0.004	2.91	0.187	2.923	0.064	0.141	0.141	0.755	0.025	0.121		Yes
	1.00	10.00	10.00	0.005		0.11	0.015		0.004	3.30	0.247	3.307	0.075	0.206		0.835	0.030	0.145		
	1.00	10.00	10.00	0.005		0.13	0.015		0.004	3.68	0.313	3.692	0.085	0.285		0.910	0.035	0.169		
	1.00	2.00	2.00	0.016		0.19	0.015		0.004	1.77	0.268	1.865	0.144	0.621		2.315	0.058	0.251		
	1.00	2.00	2.00	0.016		0.26	0.015		0.004	2.03	0.391	2.153	0.182	1.057		2.703	0.077	0.335		
P25-C11	1.00	2.00	2.00	0.016	Trapezoidal	0.32	0.015	Concrete (0.0140-0.005)	0.004	2.29	0.530	2.441	0.217	1.615	1.615	3.046	0.097	0.419	Energy dissipation rqd	None
	1.00	2.00	2.00	0.016		0.39	0.015		0.004	2.55	0.686	2.730	0.251	2.303		3.358	0.116	0.503		
	1.00	2.00	2.00	0.016		0.45	0.015		0.004	2.80	0.858	3.018	0.284	3.131		3.647	0.135	0.587		
	1.00	10.00	10.00	0.016	al	0.05	0.015	5)	0.004	2.01	0.076	2.011	0.038	0.072		0.946	0.015	0.065		

UNIFORM OPEN CHANNEL FLOW - MANNING

Project Number:	**002802**
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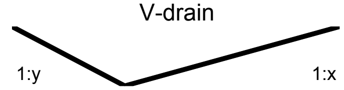
Trapezoidal Drain

V-drain



Simplified Target Values

Lining and Roughness (Manning	Max. m/s	1:max
Concrete □(0.0140-0.005)	6.0	1
Armorflex 140 □(0.0300-0.150)	3.0	2
Armorflex 180 □(0.0300-0.150)	6.0	2
Grass Long □(0.0360-0.400)	1.8	4
Grass Short □(0.0320-0.200)	1.5	4
Max velocity to encourage silt depositing		0.8

Section	w (m)	y (m)	x (m)	Ground Slope	Type	Flow depth (m)	Manning n	Lining n-k	k (m)	Flow Width (m)	Wetted Area (m2)	Wetted Perim. (m)	Hydraulic Radius, R	Flow Q (m3/s)	Peak Flow (m3/s)	V (m/s)	Freeb Straight (m)	Drain Depth (m)	Notes	Traffic
Exp-P26	1.00	10.00	10.00	0.016	Trapezoida	0.07	0.015	Concrete (0.0140-0.00	0.004	2.34	0.112	2.348	0.048	0.124	0.193	1.110	0.020	0.087		Yes
	1.00	10.00	10.00	0.016		0.08	0.015		0.004	2.68	0.154	2.685	0.057	0.193		1.255	0.025	0.109		
	1.00	10.00	10.00	0.016		0.10	0.015		0.004	3.01	0.202	3.022	0.067	0.280		1.388	0.030	0.131		
	1.00	10.00	10.00	0.016		0.12	0.015		0.004	3.35	0.255	3.359	0.076	0.386		1.512	0.035	0.153		
Stack-C13	1.00	10.00	10.00	0.014	Trapezoidal	0.09	0.015	Concrete (0.0140-0.005)	0.004	2.84	0.176	2.847	0.062	0.218	0.637	1.235	0.028	0.119		Yes
	1.00	10.00	10.00	0.014		0.12	0.015		0.004	3.45	0.273	3.463	0.079	0.395		1.449	0.037	0.159		
	1.00	10.00	10.00	0.014		0.15	0.015		0.004	4.06	0.388	4.078	0.095	0.637		1.643	0.046	0.199		
	1.00	10.00	10.00	0.014		0.18	0.015		0.004	4.68	0.522	4.694	0.111	0.951		1.823	0.055	0.239		
	1.00	10.00	10.00	0.014		0.21	0.015		0.004	5.29	0.674	5.310	0.127	1.343		1.993	0.064	0.279		
C13-C12	1.00	2.00	2.00	0.014	Trapezoidal	0.14	0.015	Concrete (0.0140-0.005)	0.004	1.57	0.183	1.637	0.112	0.335	0.848	1.831	0.043	0.185	Energy dissipation rqd	None
	1.00	2.00	2.00	0.014		0.19	0.015		0.004	1.76	0.262	1.850	0.142	0.563		2.145	0.057	0.247		
	1.00	2.00	2.00	0.014		0.24	0.015		0.004	1.95	0.350	2.062	0.170	0.848		2.420	0.071	0.309		
	1.00	2.00	2.00	0.014		0.29	0.015		0.004	2.14	0.448	2.275	0.197	1.194		2.668	0.086	0.371		
	1.00	2.00	2.00	0.014		0.33	0.015		0.004	2.33	0.554	2.487	0.223	1.605		2.898	0.100	0.432		
C13-C12 Alternative	1.00	5.00	5.00	0.014	Trapezoidal	0.12	0.015	Concrete (0.0140-0.005)	0.004	2.22	0.197	2.247	0.088	0.307	0.848	1.557	0.037	0.159	Energy dissipation rqd	Yes
	1.00	5.00	5.00	0.014		0.16	0.015		0.004	2.63	0.296	2.663	0.111	0.540		1.823	0.049	0.212		
	1.00	5.00	5.00	0.014		0.20	0.015		0.004	3.04	0.411	3.078	0.134	0.848		2.062	0.061	0.265		
	1.00	5.00	5.00	0.014		0.24	0.015		0.004	3.45	0.544	3.494	0.156	1.240		2.282	0.073	0.318		
	1.00	5.00	5.00	0.014		0.29	0.015		0.004	3.85	0.692	3.909	0.177	1.722		2.487	0.086	0.371		
C11,12-C9-D5	2.00	2.00	2.00	0.030	Trapezoidal	0.21	0.015	Concrete (0.0140-0.005)	0.004	2.82	0.495	2.918	0.170	1.749	4.309	3.537	0.062	0.267	Energy dissipation rqd	None
	2.00	2.00	2.00	0.030		0.27	0.015		0.004	3.09	0.697	3.224	0.216	2.899		4.159	0.082	0.356		
	2.00	2.00	2.00	0.030		0.34	0.015		0.004	3.37	0.918	3.529	0.260	4.319		4.705	0.103	0.445		
	2.00	2.00	2.00	0.030		0.41	0.015		0.004	3.64	1.158	3.835	0.302	6.015		5.196	0.123	0.534		
	2.00	2.00	2.00	0.030		0.48	0.015		0.004	3.92	1.416	4.141	0.342	7.996		5.646	0.144	0.622		
D5 STILLING	5.00	0.01	5.00	0.000	Trapezoidal	0.60	0.014	Concrete (0.0140-0.005)	0.002	8.01	3.902	8.659	0.451	2.749	6.932	0.705	0.125	0.725	Min. Size	Cleaning
	5.00	0.01	5.00	0.000		0.80	0.014		0.002	9.01	5.603	9.879	0.567	4.603		0.821	0.167	0.967		
	5.00	0.01	5.00	0.000		1.00	0.014		0.002	10.01	7.505	11.099	0.676	6.932		0.924	0.209	1.209		
	5.00	0.01	5.00	0.000		1.20	0.014		0.002	11.01	9.607	12.319	0.780	9.759		1.016	0.251	1.451		
	5.00	0.01	5.00	0.000		1.40	0.014		0.001	12.01	11.910	13.539	0.880	13.110		1.101	0.292	1.692		
P27 STILLING	3.00	0.01	5.00	0.000	Trapezoidal	0.60	0.032	Concrete (0.0140-0.005)	0.181	6.01	2.702	6.659	0.406	0.607	1.615	0.225	0.121	0.721	Min. Size	Cleaning
	3.00	0.01	5.00	0.000		0.80	0.032		0.199	7.01	4.003	7.879	0.508	1.045		0.261	0.161	0.961		
	3.00	0.01	5.00	0.000		1.00	0.032		0.213	8.01	5.505	9.099	0.605	1.615		0.293	0.201	1.201		
	3.00	0.01	5.00	0.000		1.20	0.032		0.224	9.01	7.207	10.319	0.698	2.327		0.323	0.241	1.441		
	3.00	0.01	5.00	0.000		1.40	0.032		0.234	10.01	9.110	11.539	0.789	3.191		0.350	0.281	1.681		
12 STILLING	3.00	0.01	5.00	0.000	rapezoidal	0.60	0.032	Concrete .0140-0.005)	0.181	6.01	2.702	6.659	0.406	0.319	0.848	0.118	0.120	0.720	Min. Size	Cleaning
	3.00	0.01	5.00	0.000		0.80	0.032		0.199	7.01	4.003	7.879	0.508	0.549		0.137	0.160	0.960		
	3.00	0.01	5.00	0.000		1.00	0.032		0.213	8.01	5.505	9.099	0.605	0.848		0.154	0.200	1.200		
	3.00	0.01	5.00	0.000		1.20	0.032		0.224	9.01	7.207	10.319	0.698	1.222		0.170	0.240	1.440		

UNIFORM OPEN CHANNEL FLOW - MANNING

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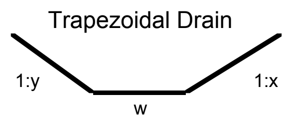
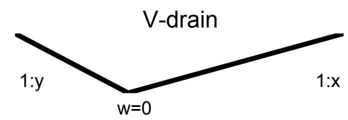




Simplified Target Values

Lining and Roughness (Manning n)	Max. m/s	1:max
Concrete (0.0140-0.005)	6.0	1
Armorflex 140 (0.0300-0.150)	3.0	2
Armorflex 180 (0.0300-0.150)	6.0	2
Grass Long (0.0360-0.400)	1.8	4
Grass Short (0.0320-0.200)	1.5	4
Max velocity to encourage silt depositing		0.8

Section	w (m)	y (m)	x (m)	Ground Slope	Type	Flow depth (m)	Manning n	Lining n-k	k (m)	Flow Width (m)	Wetted Area (m2)	Wetted Perim. (m)	Hydraulic Radius, R	Flow Q (m3/s)	Peak Flow (m3/s)	V (m/s)	Freeb Straight (m)	Drain Depth (m)	Notes	Traffic
C	3.00	0.01	5.00	0.000	T	1.40	0.032	(0	0.234	10.01	9.110	11.539	0.789	1.677		0.184	0.280	1.680		

Annexure C:
PANS AND DAMS

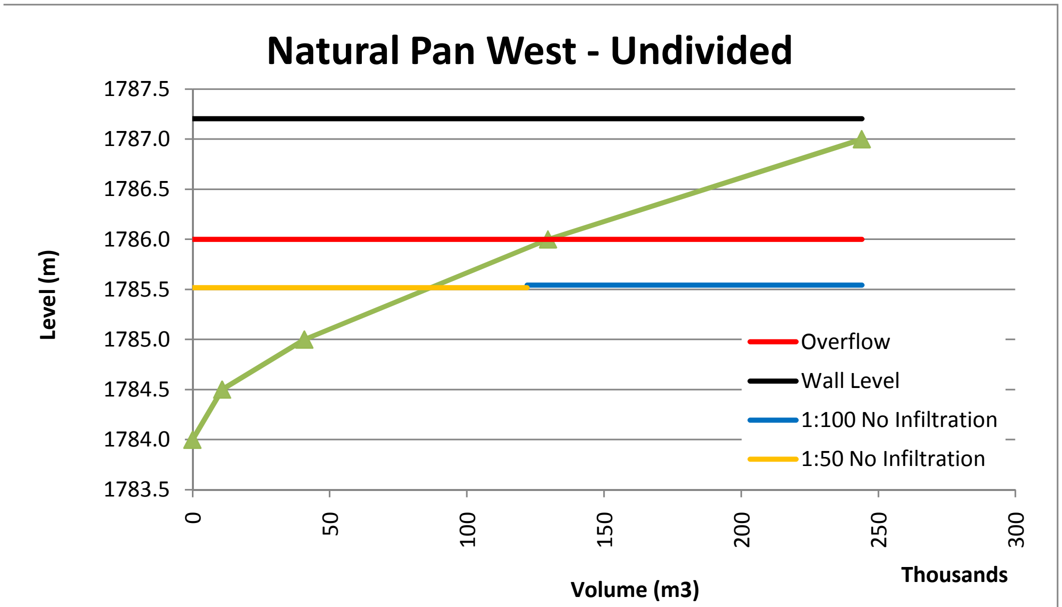
WATER BODIES Natural Pan West - Undivided

Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**
	Spreadsheet by RLR

Mean Annual Precipitation	690.000	mm
Mean Annual Evaporation	1450	mm
Runoff Coefficient max.	0.365	
Catchment Area	480,000	m2
Overflow Level	1786.0	m

24hr rainfall depths for different Recurrence Intervals						
2	5	10	20	50	100	200
58	77	90	104	123	137	153

Level, Area and Volume Relationship of Pan						
Level	m	1784.0	1784.5	1785.0	1786.0	1787.0
Depth	m	0.0	0.5	1.0	2.0	3.0
Surface Area	m2	0	43,200	76,800	100,800	128,000
Inc. Volume	m3	0	10,800	30,000	88,800	114,400
Cum. Volume	m3	0	10,800	40,800	129,600	244,000



Mean Annual Volume		120,888			
Infiltration		0%	25%	50%	75%	100%
Remaining Volume		120,888	90,666	60,444	30,222	0
Area for Balance Evap.		83,371	62,528	41,686	20,843	0
Balance Volume before Event		65112.8	28057.4	10421.4	5210.7	0.0
Balance Level before Event		1,785.27	1,784.79	1,784.48	1,784.24	1,784.00
Event	24h Volume	Level After Event				
5	13,490	1785.43	1785.01	1784.72	1784.63	1784.54
10	15,768	1785.45	1785.03	1784.76	1784.67	1784.58
50	21,550	1785.52	1785.10	1784.85	1784.77	1784.68
100	24,002	1785.54	1785.13	1784.89	1784.81	1784.72
200	26,806	1785.58	1785.16	1784.94	1784.85	1784.77

1786.0 *Indicate overflow conditions*

Peak Runoff (m3/s) for:	1:10	1:20	1:50	1:100	1:200
Rational Method	n.a	n.a	7.959	11.806	n.a
SDF Method	n.a	n.a	9.461	12.183	n.a
Empirical Method	n.a	n.a	11.959	15.106	n.a

11.806 *Indicates the most relevant method and recommended peak runoff rate*

Spillway	Unit	Dimension	Level	Freeboard	
Width and Level	m	30.000	1786.000	0.000	
Height and Level	m	1.204	1787.204	1.204	
1:50 Event	m	0.000	1786.000	1.204	
1:100 Event	m	0.404	1786.404	0.800	

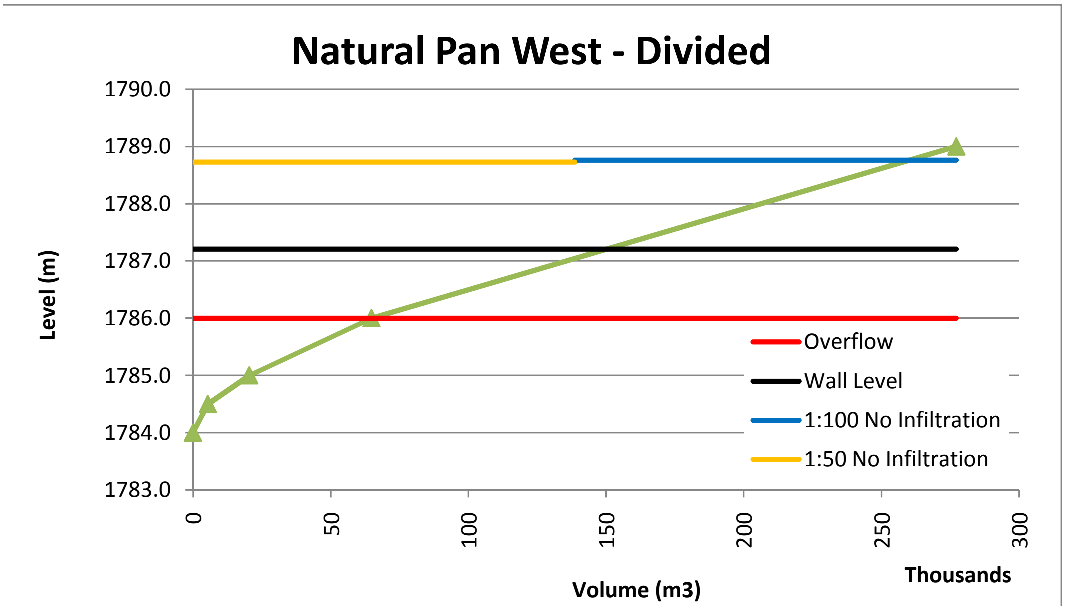
WATER BODIES   Natural Pan West - Divided

Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**
	Spreadsheet by RLR

Mean Annual Precipitation	690.000	mm
Mean Annual Evaporation	1450	mm
Runoff Coefficient max.	0.365	
Catchment Area	480,000	m2
Overflow Level	1786.0	m

24hr rainfall depths for different Recurrence Intervals						
2	5	10	20	50	100	200
58	77	90	104	123	137	153

Level, Area and Volume Relationship of Pan						
Level	m	1784.0	1784.5	1785.0	1786.0	1789.0
Depth	m	0.0	0.5	1.0	2.0	5.0
Surface Area	m2	0	21,600	38,400	50,400	91,200
Inc. Volume	m3	0	5,400	15,000	44,400	212,400
Cum. Volume	m3	0	5,400	20,400	64,800	277,200



Mean Annual Volume	120,888				
Infiltration	0%	25%	50%	75%	100%
Remaining Volume	120,888	90,666	60,444	30,222	0
Area for Balance Evap.	83,371	62,528	41,686	20,843	0
Balance Volume before Event	236443.3	127938.4	32556.4	5210.7	0.0
Balance Level before Event	1,788.42	1,786.89	1,785.27	1,784.48	1,784.00

Event	24h Volume	Level After Event				
5	13,490	1788.61	1787.08	1785.58	1784.94	1784.77
10	15,768	1788.65	1787.11	1785.63	1785.01	1784.85
50	21,550	1788.73	1787.20	1785.76	1785.14	1785.03
100	24,002	1788.76	1787.23	1785.81	1785.20	1785.08
200	26,806	1788.80	1787.27	1785.88	1785.26	1785.14

1786.0 *Indicate overflow conditions*

Peak Runoff (m3/s) for:	1:10	1:20	1:50	1:100	1:200
Rational Method	n.a	n.a	7.959	11.806	n.a
SDF Method	n.a	n.a	9.461	12.183	n.a
Empirical Method	n.a	n.a	11.959	15.106	n.a

7.959 *Indicates the most relevant method and recommended peak runoff rate*

Spillway	Unit	Dimension	Level	Freeboard	
Width and Level	m	20.000	1786.000	0.000	
Height and Level	m	1.207	1787.207	1.207	
1:50 Event	m	0.000	1786.000	1.207	
1:100 Event	m	0.407	1786.407	0.800	

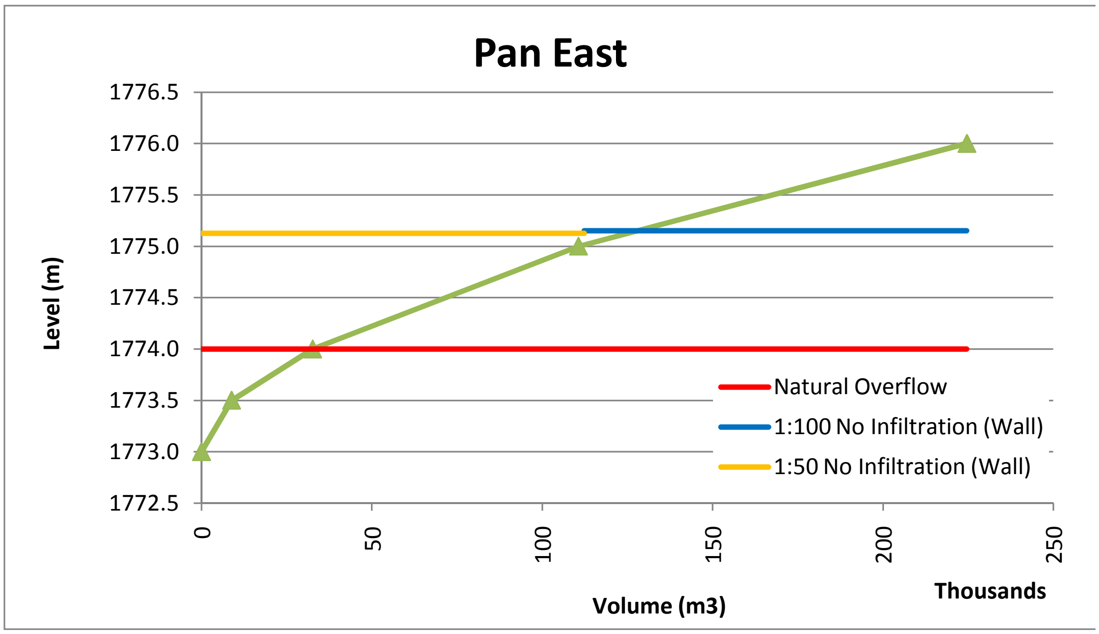
WATER BODIES Pan East

Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**
	Spreadsheet by RLR

Mean Annual Precipitation	690.000	mm
Mean Annual Evaporation	1450	mm
Runoff Coefficient max.	0.365	
Catchment Area	528,000	m2
Overflow Level	1774.0	m

24hr rainfall depths for different Recurrence Intervals						
2	5	10	20	50	100	200
58	77	90	104	123	137	153

Level, Area and Volume Relationship of Pan						
Level	m	1773.0	1773.5	1774.0	1775.0	1776.0
Depth	m	0.0	0.5	1.0	2.0	3.0
Surface Area	m2	0	35,200	60,000	96,000	132,000
Inc. Volume	m3	0	8,800	23,800	78,000	114,000
Cum. Volume	m3	0	8,800	32,600	110,600	224,600



Mean Annual Volume	132,977				
Infiltration	0%	25%	50%	75%	100%
Remaining Volume	132,977	99,733	66,488	33,244	0
Area for Balance Evap.	91,708	68,781	45,854	22,927	0
Balance Volume before Event	101301.0	51625.7	19024.5	5731.8	0.0
Balance Level before Event	1,774.88	1,774.24	1,773.71	1,773.33	1,773.00

Event	24h Volume	Level After Event				
5	14,839	1775.05	1774.43	1774.02	1773.75	1773.63
10	17,345	1775.07	1774.47	1774.05	1773.80	1773.68
50	23,705	1775.13	1774.55	1774.13	1773.93	1773.81
100	26,403	1775.15	1774.58	1774.16	1773.99	1773.87
200	29,486	1775.18	1774.62	1774.20	1774.03	1773.93

1774.0 *Indicate overflow conditions*

Peak Runoff (m3/s) for:	1:10	1:20	1:50	1:100	1:200
Rational Method	n.a	n.a	8.650	12.830	n.a
SDF Method	n.a	n.a	10.249	13.198	n.a
Empirical Method	n.a	n.a	12.604	15.921	n.a

8.650 *Indicates the most relevant method and recommended peak runoff rate*

Spillway	Unit	Dimension	Level	Freeboard
Width and Level	m	Natural overflow!		
Height and Level	m			
1:50 Event	m			
1:100 Event	m			

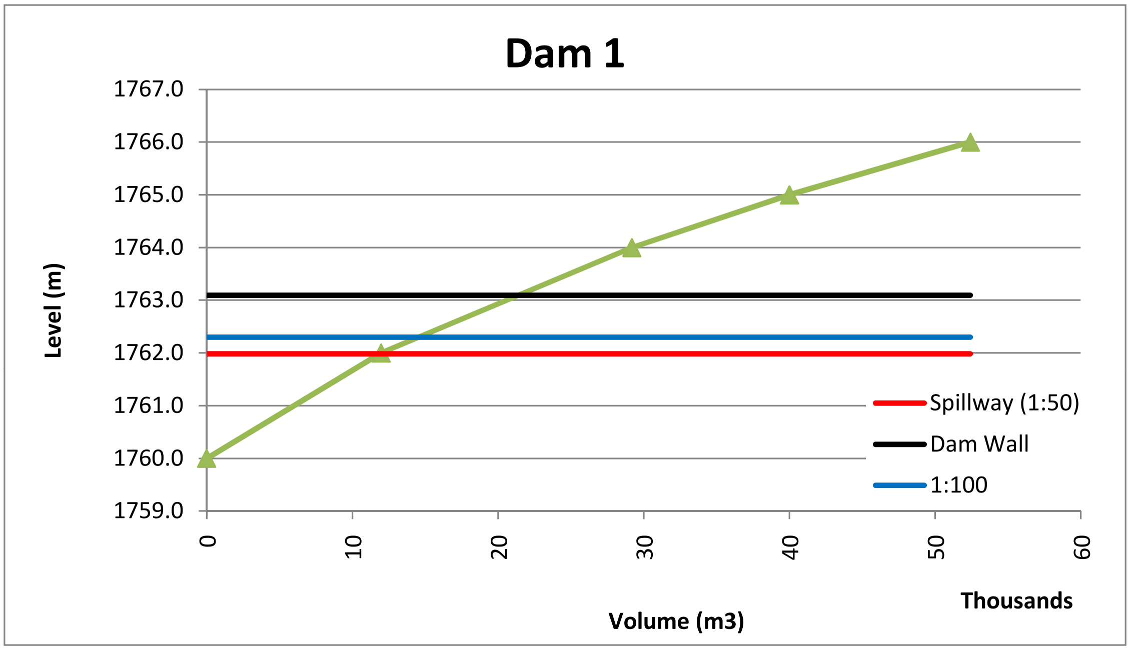
WATER BODIES **Dam 1**

Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**
	Spreadsheet by RLR

Mean Annual Precipitation	690.000	mm	**Dam Size**	Inside	Footprint
Mean Annual Evaporation	1450	mm	Floor Width	60.00	100.05
Runoff Coefficient max.	0.374		Floor Length	80.00	123.15
Catchment Area	246,400	m2	Sides 1:	4.00	2.00
Overflow Level	1762.0	m	Tot Depth / crest	3.10	3.00

24hr rainfall depths for different Recurrence Intervals						
2	5	10	20	50	100	200
58	77	90	104	123	137	153

Level, Area and Volume Relationship Dam						
Level	m	1760.00	1762.00	1764.00	1765.00	1766.00
Depth	m	0.0	2.0	4.0	5.0	6.0
Surface Area	m2	0	7,296	10,304	12,000	13,824
Inc. Volume	m3	0	11,968	17,216	10,816	12,416
Cum. Volume	m3	0	11,968	29,184	40,000	52,416



Allow for siltation in dam						
Start Silt Volume	m3	0.0	540.0	1000.0	1500.0	2000.0
Start Silt Level	m	1,760.00	1,760.09	1,760.17	1,760.25	1,760.33
% of Capacity	%	0.00%	4.55%	8.12%	11.70%	15.02%

Event	24h Volume	Level After Event				
5	7,086	1761.18	1761.27	1761.35	1761.43	1761.52
10	8,283	1761.38	1761.47	1761.55	1761.63	1761.72
50	11,320	1761.89	1761.98	1762.04	1762.10	1762.16
100	12,608	1762.07	1762.14	1762.19	1762.25	1762.31
200	14,081	1762.25	1762.31	1762.36	1762.42	1762.48

1762.0 *Indicate overflow conditions*

Peak Runoff (m3/s) for:	1:10	1:20	1:50	1:100	1:200
Rational Method	n.a	n.a	4.485	5.396	n.a
SDF Method	n.a	n.a	4.206	5.416	n.a
Empirical Method	n.a	n.a	6.608	8.347	n.a

5.396 *Indicates the most relevant method and recommended peak runoff rate*

Spillway	Unit	Dimension	Level	Freeboard	Volume
Width and Level	m	20.000	1761.982	0.000	11,837
Height and Level	m	1.114	1763.096	1.114	20,702
1:50 Event	m	0.000	1761.982	1.114	11,837
1:100 Event	m	0.314	1762.296	0.800	14,166

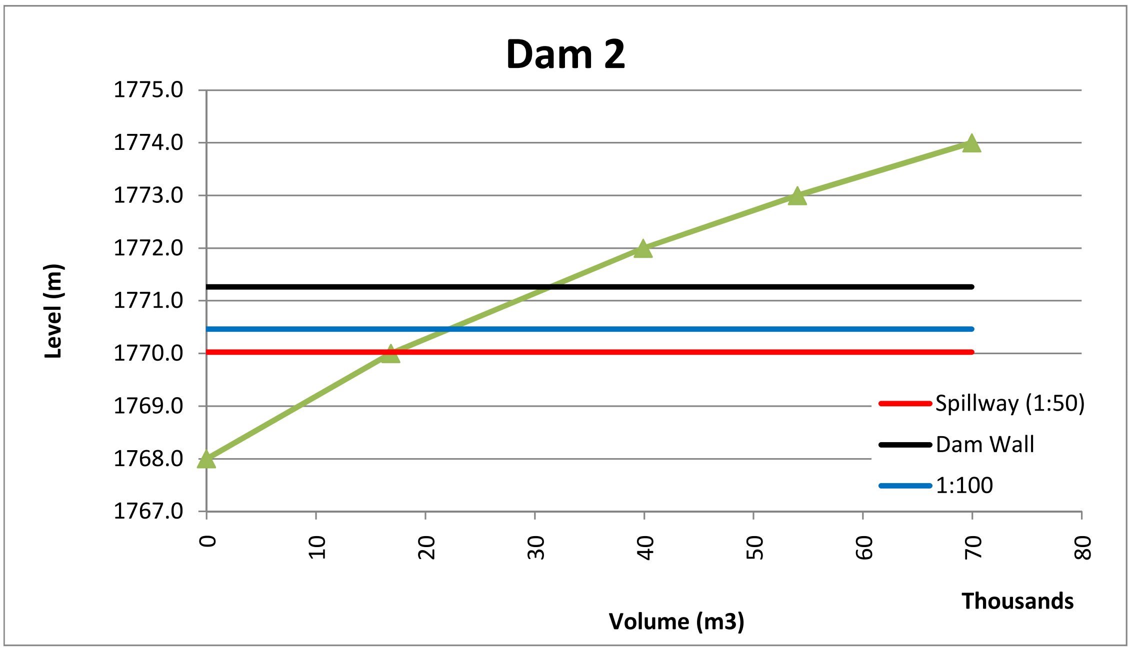
WATER BODIES Dam 2

Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**
	Spreadsheet by RLR

Mean Annual Precipitation	690.000	mm	**Dam Size**	Inside	Footprint
Mean Annual Evaporation	1450	mm	Floor Width	70.00	111.91
Runoff Coefficient max.	0.374		Floor Length	100.00	145.18
Catchment Area	336,000	m2	Sides 1:	4.00	2.00
Overflow Level	1770.0	m	Tot Depth / crest	3.26	3.00

24hr rainfall depths for different Recurrence Intervals						
2	5	10	20	50	100	200
58	77	90	104	123	137	153

Level, Area and Volume Relationship Dam						
Level	m	1768.00	1770.00	1772.00	1773.00	1774.00
Depth	m	0.0	2.0	4.0	5.0	6.0
Surface Area	m2	0	9,976	13,464	15,400	17,464
Inc. Volume	m3	0	16,848	23,056	14,096	15,936
Cum. Volume	m3	0	16,848	39,904	54,000	69,936



Allow for siltation in dam						
Start Silt Volume	m3	0.0	1700.0	2000.0	2500.0	3000.0
Start Silt Level	m	1,768.00	1,768.20	1,768.24	1,768.30	1,768.36
% of Capacity	%	0.00%	9.92%	11.47%	13.94%	16.27%

Event	24h Volume	Level After Event				
5	9,663	1769.15	1769.35	1769.38	1769.44	1769.50
10	11,295	1769.34	1769.54	1769.58	1769.64	1769.70
50	15,436	1769.83	1770.02	1770.05	1770.09	1770.14
100	17,193	1770.03	1770.18	1770.20	1770.25	1770.29
200	19,201	1770.20	1770.35	1770.38	1770.42	1770.46

1770.0 *Indicate overflow conditions*

Peak Runoff (m3/s) for:	1:10	1:20	1:50	1:100	1:200
Rational Method	n.a	n.a	6.474	8.947	n.a
SDF Method	n.a	n.a	6.236	8.385	n.a
Empirical Method	n.a	n.a	8.276	10.666	n.a

8.947 *Indicates the most relevant method and recommended peak runoff rate*

Spillway	Unit	Dimension	Level	Freeboard	Volume
Width and Level	m	20.000	1770.025	0.000	17,096
Height and Level	m	1.240	1771.265	1.240	30,659
1:50 Event	m	0.000	1770.025	1.240	17,096
1:100 Event	m	0.440	1770.465	0.800	21,625

WATER BODIES Dam 3

Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**
	Spreadsheet by RLR

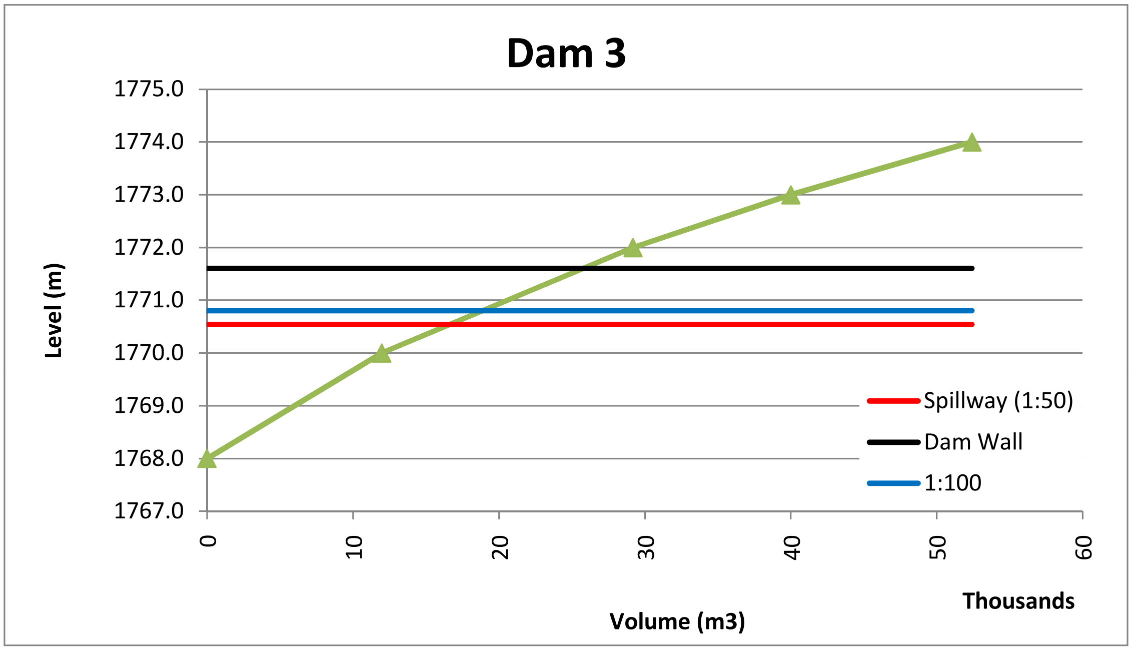
Mean Annual Precipitation	690.000	mm	**Dam Size**	Inside	Footprint
Mean Annual Evaporation	1450	mm	Floor Width	60.00	105.63
Runoff Coefficient max.	1.000		Floor Length	80.00	129.23
Catchment Area	13,081	m2	Sides 1:	4.00	2.00
Overflow Level	1770.5	m	Tot Depth / crest	3.60	3.00

24hr rainfall depths for different Recurrence Intervals

2	5	10	20	50	100	200
58	77	90	104	123	137	153

Level, Area and Volume Relationship Dam

Level	m	1768.00	1770.00	1772.00	1773.00	1774.00
Depth	m	0.0	2.0	4.0	5.0	6.0
Surface Area	m2	0	7,296	10,304	12,000	13,824
Inc. Volume	m3	0	11,968	17,216	10,816	12,416
Cum. Volume	m3	0	11,968	29,184	40,000	52,416



Allow for 15000 capacity plus 24hr event

Start Volume	m3	15000.0	15000.0	15000.0	15000.0	15000.0
Start Level	m	1,770.35	1,770.35	1,770.35	1,770.35	1,770.35
% of Capacity	%	90.31%	90.31%	90.31%	90.31%	90.31%

Event	24h Volume	Level After Event				
5	1,007	1770.47	1770.47	1770.47	1770.47	1770.47
10	1,177	1770.49	1770.49	1770.49	1770.49	1770.49
50	1,609	1770.54	1770.54	1770.54	1770.54	1770.54
100	1,792	1770.56	1770.56	1770.56	1770.56	1770.56
200	2,001	1770.58	1770.58	1770.58	1770.58	1770.58

1770.5 *Indicate overflow conditions*

Peak Runoff (m3/s) for:	1:10	1:20	1:50	1:100	1:200
Rational Method	n.a	n.a	0.841	1.035	n.a
SDF Method	n.a	n.a	0.317	0.408	n.a
Empirical Method	n.a	n.a	n.a.	n.a.	n.a

1.035 *Indicates the most relevant method and recommended peak runoff rate*

Spillway (for reference only)	Unit	Dimension	Level	Freeboard	Volume
Width and Level	m	5.000	1770.539	0.000	16,060
Height and Level	m	1.063	1771.602	1.063	25,306
1:50 Event	m	0.000	1770.539	1.063	16,060
1:100 Event	m	0.263	1770.802	0.800	18,201

WATER BODIES **Dam 4**

DO NOT INCLUDE!!!!!!
check levels

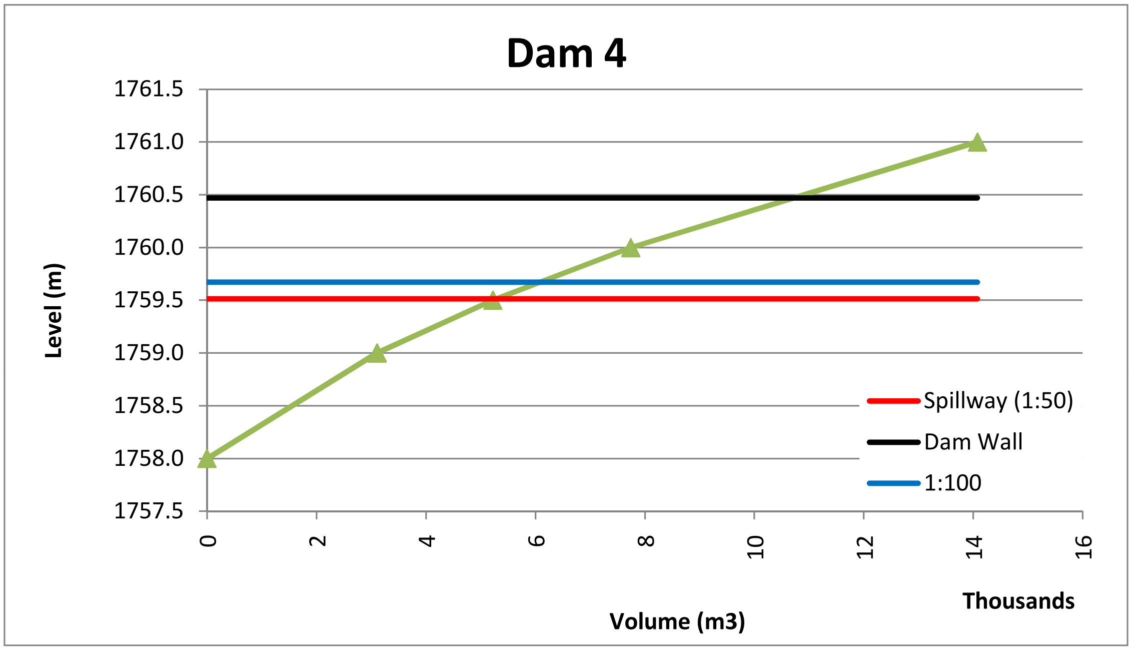
Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**

Spreadsheet by RLR

			Dam Size	Inside	Footprint
Mean Annual Precipitation	690.000	mm	Floor Width	30.00	121.41
Mean Annual Evaporation	1450	mm	Floor Length	80.00	115.65
Runoff Coefficient max.	1.000		Sides 1:	4.00	2.00
Catchment Area	2,400	m2	Ramp 1:	10.00	
Overflow Level	1759.5	m	Tot Depth / crest	2.47	3.00

24hr rainfall depths for different Recurrence Intervals						
2	5	10	20	50	100	200
58	77	90	104	123	137	153

Level, Area and Volume Relationship Dam						
Level	m	1758.00	1759.00	1759.50	1760.00	1761.00
Depth	m	0.0	1.0	1.5	2.0	3.0
Surface Area	m2	0	3,872	4,692	5,568	7,488
Inc. Volume	m3	0	3,108	2,117	2,520	6,332
Cum. Volume	m3	0	3,108	5,225	7,744	14,076



Allow for 5000 capacity plus 24hr event						
Start Volume	m3	5000.0	5000.0	5000.0	5000.0	5000.0
Start Level	m	1,759.45	1,759.45	1,759.45	1,759.45	1,759.45
% of Capacity	%	94.43%	94.43%	94.43%	94.43%	94.43%

Event	24h Volume	Level After Event				
5	185	1759.49	1759.49	1759.49	1759.49	1759.49
10	216	1759.50	1759.50	1759.50	1759.50	1759.50
50	295	1759.51	1759.51	1759.51	1759.51	1759.51
100	329	1759.52	1759.52	1759.52	1759.52	1759.52
200	367	1759.53	1759.53	1759.53	1759.53	1759.53

1759.5 *Indicate overflow conditions*

Peak Runoff (m3/s) for:	1:10	1:20	1:50	1:100	1:200
Rational Method	n.a	n.a	0.154	0.190	n.a
SDF Method	n.a	n.a	0.058	0.075	n.a
Empirical Method	n.a	n.a	n.a.	n.a.	n.a

0.190 *Indicates the most relevant method and recommended peak runoff rate*

Spillway (for reference only)	Unit	Dimension	Level	Freeboard	Volume
Width and Level	m	2.000	1759.514	0.000	4,698
Height and Level	m	0.957	1760.471	0.957	8,856
1:50 Event	m	0.000	1759.514	0.957	4,698
1:100 Event	m	0.157	1759.671	0.800	5,312

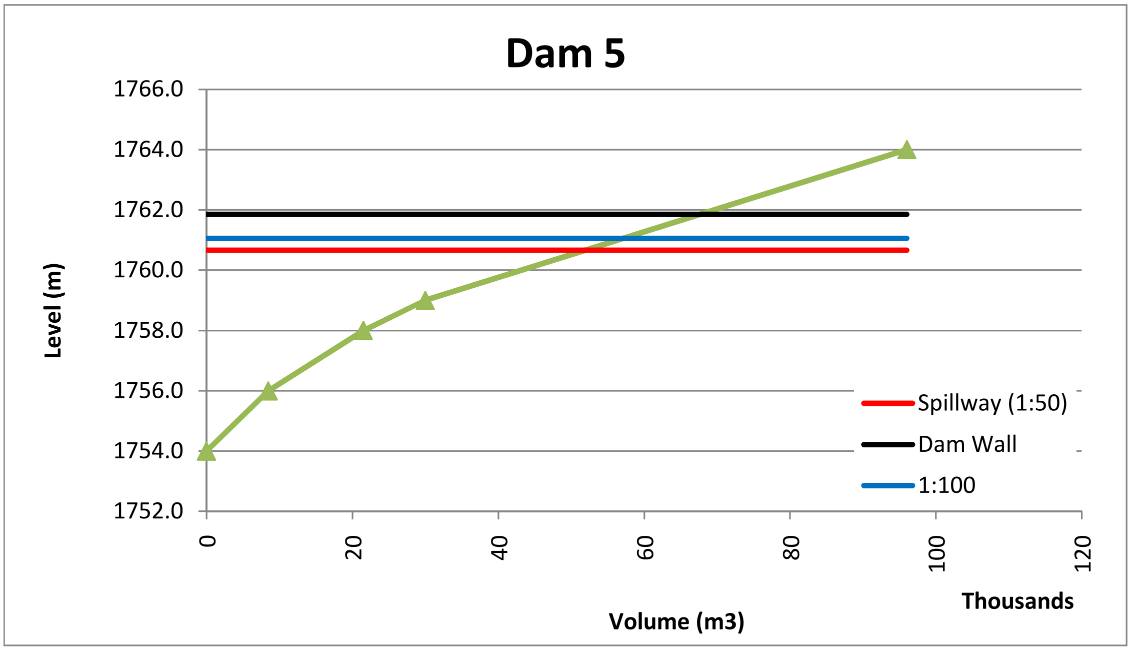
WATER BODIES   Dam 5

Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**
	Spreadsheet by RLR

			Dam Size	Inside	Footprint
Mean Annual Precipitation	690.000	mm	Floor Width	40.00	136.39
Mean Annual Evaporation	1450	mm	Floor Length	80.00	184.25
Runoff Coefficient max.	0.445		Sides 1:	4.00	2.00
Catchment Area	857,600	m2	Tot Depth / crest	7.85	5.00
Overflow Level	1760.7	m			

24hr rainfall depths for different Recurrence Intervals						
2	5	10	20	50	100	200
58	77	90	104	123	137	153

Level, Area and Volume Relationship Dam						
Level	m	1754.00	1756.00	1758.00	1759.00	1764.00
Depth	m	0.0	2.0	4.0	5.0	10.0
Surface Area	m2	0	5,376	8,064	9,600	19,200
Inc. Volume	m3	0	8,448	13,056	8,496	66,000
Cum. Volume	m3	0	8,448	21,504	30,000	96,000



Allow for siltation in dam						
Start Silt Volume	m3	0.0	5000.0	10000.0	15000.0	20000.0
Start Silt Level	m	1,754.00	1,755.18	1,756.24	1,757.00	1,757.77
% of Capacity	%	0.00%	9.63%	17.56%	24.22%	29.88%

Event	24h Volume	Level After Event				
5	29,386	1758.93	1759.33	1759.71	1760.09	1760.47
10	34,347	1759.33	1759.71	1760.09	1760.47	1760.84
50	46,941	1760.28	1760.66	1761.04	1761.42	1761.80
100	52,284	1760.69	1761.07	1761.45	1761.82	1762.20
200	58,390	1761.15	1761.53	1761.91	1762.29	1762.67

1760.7 *Indicate overflow conditions*

Peak Runoff (m3/s) for:	1:5	1:20	1:50	1:100	1:200
Rational Method	4.521	n.a	10.136	15.034	n.a
SDF Method	3.971	n.a	9.075	11.686	n.a
Empirical Method	n.a	n.a	13.194	16.666	n.a

15.034 *Indicates the most relevant method and recommended peak runoff rate*

Spillway	Unit	Dimension	Level	Freeboard	Volume
Width and Level	m	40.000	1760.662	0.000	47,355
Height and Level	m	1.192	1761.854	1.192	62,491
1:50 Event	m	0.000	1760.662	1.192	47,355
1:100 Event	m	0.392	1761.054	0.800	52,071

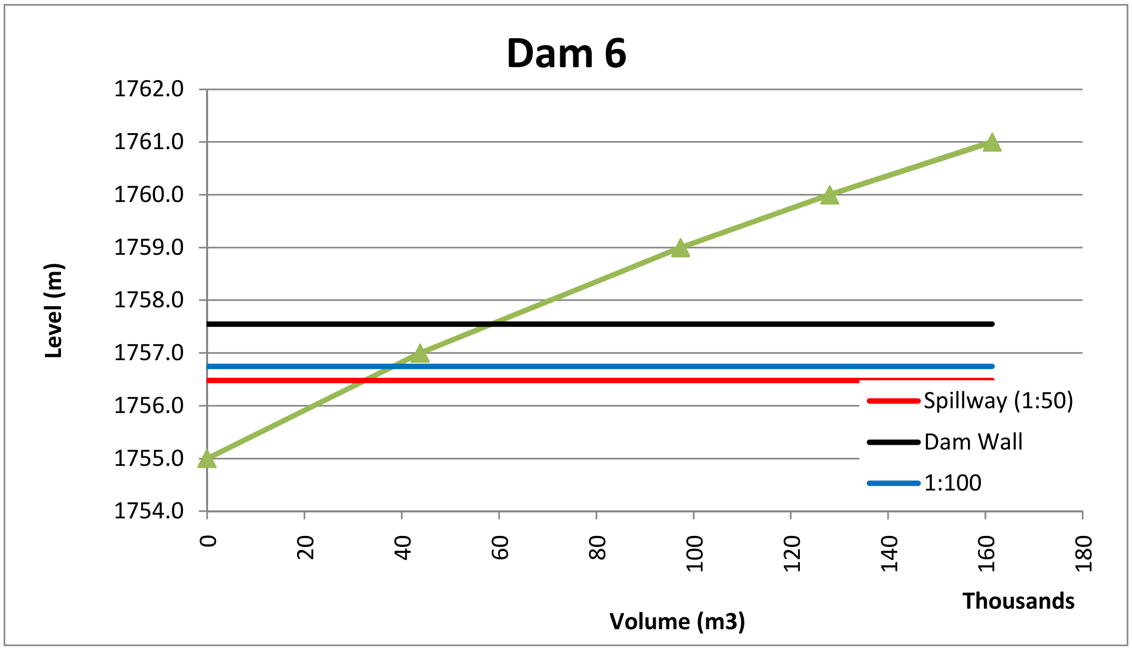
WATER BODIES Dam 6

Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**
	Spreadsheet by RLR

Mean Annual Precipitation	690.000	mm	**Dam Size**	Inside	Footprint
Mean Annual Evaporation	1450	mm	Floor Width	140.00	174.00
Runoff Coefficient max.	1.000		Floor Length	140.00	176.54
Catchment Area	20,000	m2	Sides 1:	4.00	2.00
Overflow Level	1756.5	m	Tot Depth / crest	2.55	3.00

24hr rainfall depths for different Recurrence Intervals						
2	5	10	20	50	100	200
58	77	90	104	123	137	153

Level, Area and Volume Relationship Dam						
Level	m	1755.00	1757.00	1759.00	1760.00	1761.00
Depth	m	0.0	2.0	4.0	5.0	6.0
Surface Area	m2	0	24,336	29,584	32,400	35,344
Inc. Volume	m3	0	43,808	53,536	30,656	33,376
Cum. Volume	m3	0	43,808	97,344	128,000	161,376



Allow for 30000 capacity plus 24hr event						
Start Volume	m3	30000.0	30000.0	30000.0	30000.0	30000.0
Start Level	m	1,756.37	1,756.37	1,756.37	1,756.37	1,756.37
% of Capacity	%	92.42%	92.42%	92.42%	92.42%	92.42%

Event	24h Volume	Level After Event				
5	1,540	1756.44	1756.44	1756.44	1756.44	1756.44
10	1,800	1756.45	1756.45	1756.45	1756.45	1756.45
50	2,460	1756.48	1756.48	1756.48	1756.48	1756.48
100	2,740	1756.49	1756.49	1756.49	1756.49	1756.49
200	3,060	1756.51	1756.51	1756.51	1756.51	1756.51

1756.5 *Indicate overflow conditions*

Peak Runoff (m3/s) for:	1:10	1:20	1:50	1:100	1:200
Rational Method	n.a	n.a	1.286	1.583	n.a
SDF Method	n.a	n.a	0.485	0.624	n.a
Empirical Method	n.a	n.a	n.a.	n.a.	n.a

1.583 *Indicates the most relevant method and recommended peak runoff rate*

Spillway (for reference only)	Unit	Dimension	Level	Freeboard	Volume
Width and Level	m	5.000	1756.482	0.000	31,557
Height and Level	m	1.063	1757.545	1.063	57,403
1:50 Event	m	0.000	1756.482	1.063	31,557
1:100 Event	m	0.263	1756.745	0.800	37,701

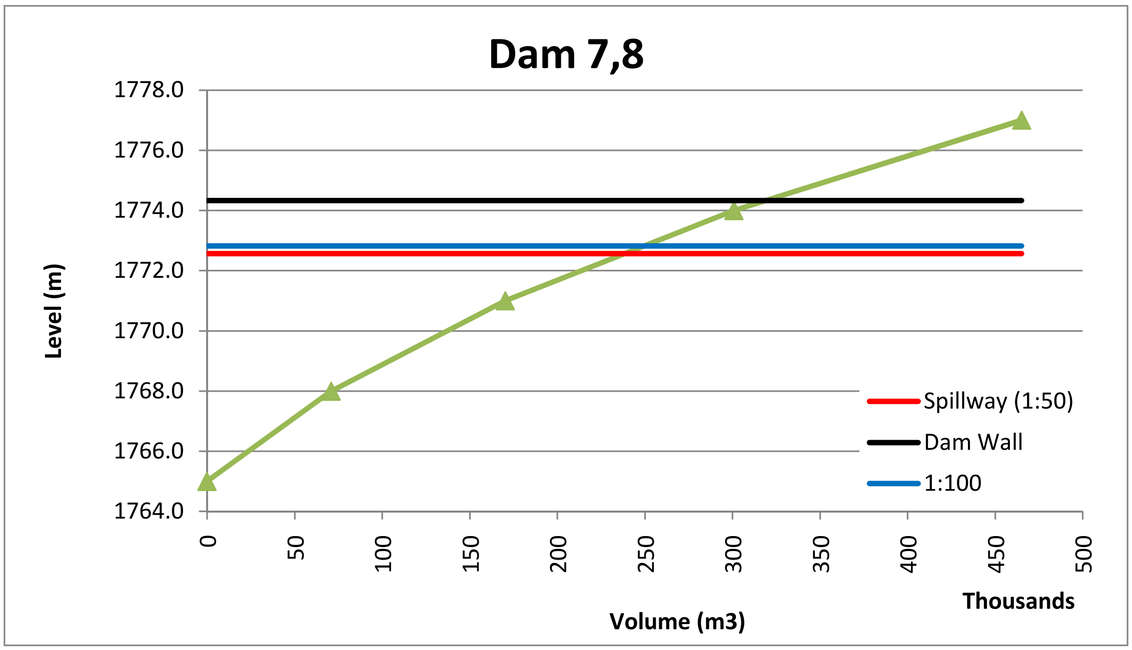
WATER BODIES Dam 7,8

Project Number:	**002802**
Project Title:	**Belfast Stormwater**
Done by:	**CCLR**
Date:	**23 July 2011**
	Spreadsheet by RLR

Mean Annual Precipitation	690.000	mm	**Dam Size**	Inside	Footprint
Mean Annual Evaporation	1450	mm	Floor Width	65.00	177.59
Runoff Coefficient max.	1.000		Floor Length	295.00	416.92
Catchment Area	68,000	m2	Sides 1:	4.00	2.00
Overflow Level	1772.6	m	Tot Depth / crest	9.33	5.00

24hr rainfall depths for different Recurrence Intervals						
2	5	10	20	50	100	200
58	77	90	104	123	137	153

Level, Area and Volume Relationship Dam						
Level	m	1765.00	1768.00	1771.00	1774.00	1777.00
Depth	m	0.0	3.0	6.0	9.0	12.0
Surface Area	m2	0	28,391	38,759	50,279	62,951
Inc. Volume	m3	0	70,917	99,429	130,533	164,229
Cum. Volume	m3	0	70,917	170,346	300,879	465,108



Allow for 230000 capacity plus 24hr event						
Start Volume	m3	230000.0	230000.0	230000.0	230000.0	230000.0
Start Level	m	1,772.37	1,772.37	1,772.37	1,772.37	1,772.37
% of Capacity	%	96.49%	96.49%	96.49%	96.49%	96.49%

Event	24h Volume	Level After Event				
5	5,236	1772.49	1772.49	1772.49	1772.49	1772.49
10	6,120	1772.51	1772.51	1772.51	1772.51	1772.51
50	8,364	1772.56	1772.56	1772.56	1772.56	1772.56
100	9,316	1772.59	1772.59	1772.59	1772.59	1772.59
200	10,404	1772.61	1772.61	1772.61	1772.61	1772.61

1772.6 *Indicate overflow conditions*

Peak Runoff (m3/s) for:	1:10	1:20	1:50	1:100	1:200
Rational Method	n.a	n.a	4.372	5.382	n.a
SDF Method	n.a	n.a	1.648	2.122	n.a
Empirical Method	n.a	n.a	n.a.	n.a.	n.a

5.382 *Indicates the most relevant method and recommended peak runoff rate*

Spillway (for reference only)	Unit	Dimension	Level	Freeboard	Volume
Width and Level	m	5.000	1772.563	0.000	234,319
Height and Level	m	1.763	1774.326	1.763	317,069
1:50 Event	m	0.000	1772.563	1.763	234,319
1:100 Event	m	0.263	1772.826	1.500	245,947

Annexure D:
CULVERT SIZING

CULVERT HYDRAULICS

Project Number: **002802**
Project Title: **Belfast Stormwater**
Done by: **CCLR**
Date: **23 July 2011**
Spreadsheet by RLR

Catchment No.		PWa-100	PWb-100	D1-100	D2-100	D3-100	D4-100	D5-100	D6-100	D7,8-100	PE50	C3-50	C4-50	C5-50	D5-50	D5-10
Position		C1	C1	D1	D2	D3	D4	D5	D6	D7,8	C2	C3	C4	C5	C6	C7
Road		n.a.	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.	Maint.	Maint.	Haul	Haul	Haul	Haul
Note		Spillway	Spillway	Spillway	Spillway	Spillway	Spillway	Spillway	Spillway	Spillway	Culvert	Culvert	Culvert	Culvert	Culvert	Culvert
Sub area m2		480,000	480,000	246,400	336,000	13,081	2,400	857,600	20,000	68,000	528,000	528,000	652,800	1,132,800	44,000	160,000
Q (m3/s)	m3/s	11.81	7.96	5.40	8.95	1.04	0.19	15.03	1.58	5.38	7.92	0.91	8.73	12.13	0.63	1.41
Hmax water	m	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	1.08	0.90	0.90	1.20	0.90	0.90
Pipe Culverts																
D (diameter.)	m	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.900	0.900	0.600	0.900	0.900	0.600	0.600
H/Dmax Pipes										1.00	1.20	1.20	1.20	1.20	1.20	1.20
So (%)										1.00	1.00	1.00	1.00	1.00	1.00	1.00
Barrels	No									18	7	3	10	11	2	4
H/D max										0.56	1.20	1.20	1.00	1.20	1.20	1.20
Q/barrel	m3/s									0.31	1.16	0.42	0.88	1.16	0.42	0.42
Actual Hw	m									0.49	1.06	0.56	0.89	1.05	0.57	0.60
Actual H/D										0.54	1.18	0.93	0.99	1.16	0.95	1.00
Hmax>D										OK	OK	OK	OK	OK	OK	OK
Throat Velocity	m/s									0.47	1.78	1.08	1.37	1.73	1.11	1.24
Box Culverts																
B (width)	m	30.000	20.000	20.000	20.000	5.000	2.000	40.000	5.000	20.000	0.900	0.900	0.900	0.900	0.900	0.900
D (depth)	m	0.400	0.400	0.400	0.600	0.600	0.600	0.600	0.600	0.600	0.900	0.600	0.900	0.900	0.600	0.600
H/Dmax Boxes		1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
Cb		0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Ch		0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60
Barrels	No	1	1	1	1	1	1	1	1	1	6	2	8	8	1	2
H/D max		1.20	1.20	1.20	0.83	0.83	0.83	0.83	0.83	0.83	1.20	1.20	1.00	1.20	1.20	1.20
Q/barrel	m3/s	15.62	10.42	10.42	10.85	2.71	1.08	21.70	2.71	10.85	1.58	0.86	1.18	1.58	0.86	0.86
Actual Hw	m	0.40	0.41	0.31	0.44	0.26	0.16	0.39	0.35	0.31	0.97	0.48	0.86	1.06	0.59	0.64
Actual H/D		1.01	1.02	0.78	0.73	0.44	0.26	0.65	0.58	0.52	1.08	0.80	0.95	1.18	0.98	1.06
Hmax>D		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Throat Velocity	m/s	0.97	0.98	0.86	1.02	0.79	0.61	0.96	0.91	0.86	1.51	1.06	1.42	1.58	1.18	1.22

CULVERT HYDRAULICS

Project Number: **002802**
Project Title: **Belfast Stormwater**
Done by: **CCLR**
Date: **23 July 2011**
Spreadsheet by RLR

Catchment No.		D5-10	D5-50	C10-50	D5-10	D5-10	D5-10	C14-50	D5-10	D5-10	D5-10	D5-10	D5-10	D5-10	D5-10	D5-10
Position		C8	C9	C10	C11	C12	C13	C14	C15	C16	C17	C18	C19	C20	C21	C22
Road		Haul	Access	Access	Berm	Berm	Conc.	Access	Maint.	Maint.	Maint.	Access	Access	Haul	Maint.	Haul
Note		Culvert	Culvert	Culvert	Culvert	Culvert	Culvert	Culvert	Culvert	Culvert	Culvert	Culvert	Culvert	Culvert	Culvert	Culvert
Sub area m2		67,200	302,640	388,800	183,760	96,480	72,480	144,000	15,360	98,400	79,200	12,000	67,200	14,400	38,400	6,720
Q (m3/s)	m3/s	0.59	4.31	5.40	1.61	0.85	0.64	2.37	0.13	0.86	0.70	0.11	0.59	0.13	0.34	0.06
Hmax water	m	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Pipe Culverts																
D (diameter.)	m	0.600	0.900	0.900	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600
H/Dmax Pipes		1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
So (%)		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Barrels	No	2	5	7	4	3	2	6	1	3	2	1	2	1	1	1
H/D max		1.20	1.00	1.00	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
Q/barrel	m3/s	0.42	0.88	0.88	0.42	0.42	0.42	0.42	0.42	0.42	0.42	0.42	0.42	0.42	0.42	0.42
Actual Hw	m	0.55	0.89	0.82	0.65	0.54	0.57	0.64	0.36	0.54	0.60	0.32	0.55	0.35	0.59	0.24
Actual H/D		0.92	0.99	0.92	1.08	0.90	0.95	1.07	0.61	0.90	1.00	0.53	0.92	0.59	0.98	0.39
Hmax>D		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Throat Velocity	m/s	1.04	1.35	1.21	1.43	1.00	1.13	1.40	0.48	1.02	1.23	0.37	1.04	0.45	1.19	0.21
Box Culverts																
B (width)	m	0.900	0.900	0.900	0.900	0.900	0.600	0.900	0.600	0.900	0.900	0.600	0.900	0.900	0.600	0.600
D (depth)	m	0.600	0.900	0.900	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600	0.600
H/Dmax Boxes		1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
Cb		0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Ch		0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60
Barrels	No	1	4	5	2	1	2	3	1	2	1	1	1	1	1	1
H/D max		1.20	1.00	1.00	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
Q/barrel	m3/s	0.86	1.18	1.18	0.86	0.86	0.57	0.86	0.57	0.86	0.86	0.57	0.86	0.86	0.57	0.57
Actual Hw	m	0.57	0.85	0.85	0.70	0.71	0.49	0.69	0.28	0.46	0.63	0.24	0.57	0.20	0.51	0.16
Actual H/D		0.95	0.94	0.94	1.17	1.18	0.82	1.15	0.46	0.77	1.06	0.39	0.95	0.34	0.85	0.27
Hmax>D		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
Throat Velocity	m/s	1.16	1.41	1.41	1.28	1.33	1.08	1.27	0.81	1.04	1.22	0.74	1.16	0.69	1.10	0.61

CULVERT HYDRAULICS

Project Number: **002802**
Project Title: **Belfast Stormwater**
Done by: **CCLR**
Date: **23 July 2011**
Spreadsheet by RLR

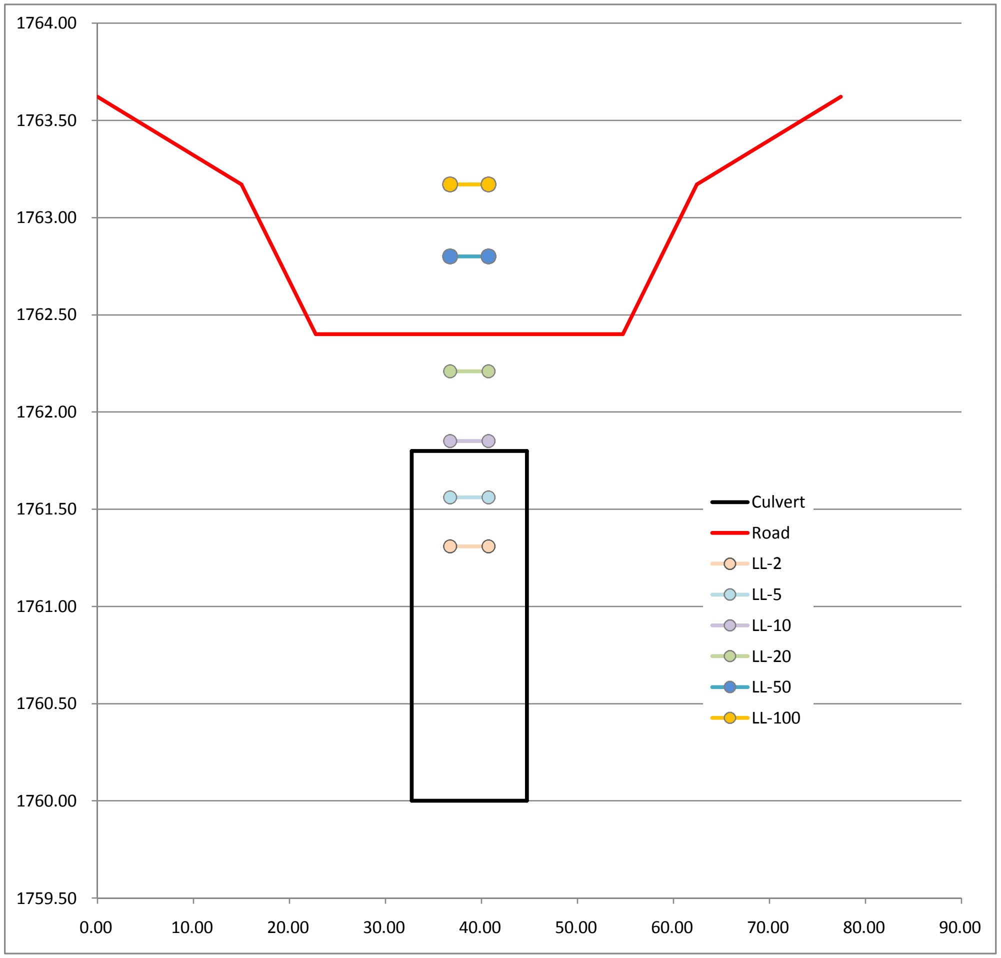
Catchment No.		D5-10	D5-10	D5-10	D5-10	D5-10	D5-10	D5-10	C10-50
Position		C23	C29	P24	P24Emer	P25	P26	P27	P28
Road		Conc.	Haul	Calculated peak runoff for drain sizing!					
Note		Culvert	Culvert						
Sub area m2		20,000	9,600	134,400	16,000	146,400	22,000	183,760	336,000
Q (m3/s)	m3/s	0.18	0.08	1.18	0.14	1.29	0.19	1.61	4.66
Hmax water	m	0.90	0.90						
Pipe Culverts									
D (diameter.)	m	0.600	0.900	0.000	0.000	0.000	0.000	0.000	0.000
H/Dmax Pipes		1.20	1.20						
So (%)		1.00	1.00						
Barrels	No	1	1						
H/D max		1.20	1.00						
Q/barrel	m3/s	0.42	0.88						
Actual Hw	m	0.42	0.25						
Actual H/D		0.70	0.28						
Hmax>D		OK	OK						
Throat Velocity	m/s	0.62	0.13						
Box Culverts									
B (width)	m	0.900	0.900	0.000	0.000	0.000	0.000	0.000	0.000
D (depth)	m	0.600	0.900						
H/Dmax Boxes		1.20	1.20						
Cb		0.90	0.90						
Ch		0.60	0.60						
Barrels	No	1	1						
H/D max		1.20	1.00						
Q/barrel	m3/s	0.86	1.18						
Actual Hw	m	0.25	0.16						
Actual H/D		0.42	0.17						
Hmax>D		OK	OK						
Throat Velocity	m/s	0.77	0.60						

Annexure E:
LOW LEVEL STRUCTURE

LOW LEVEL STRUCTURE

Project Number: **002802**
Project Title: **Belfast Stormwater**
Done by: **CCLR**
Date: **23 July 2011**
Spreadsheet by RLR

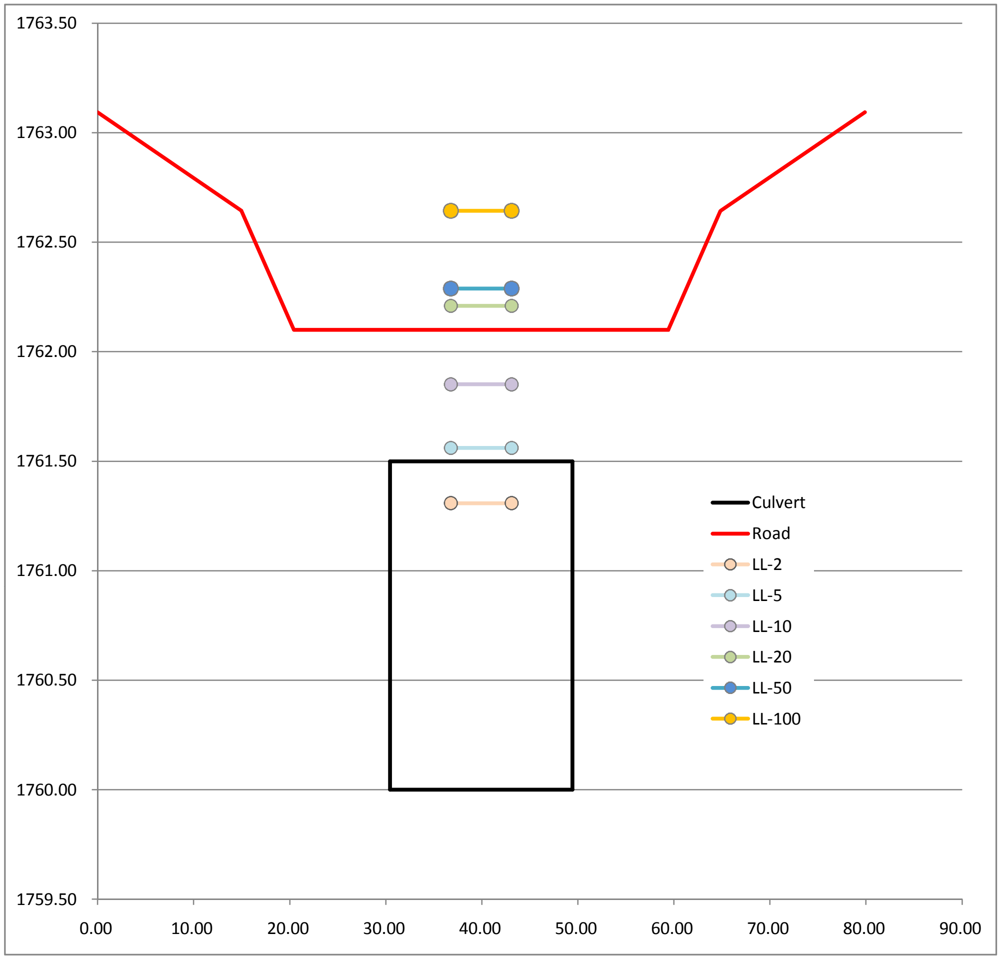
Catchment No.		LL-2	LL-5	LL-10	LL-20	LL-50	LL-100
Position		Opt A	Opt A	Opt A	Opt A	Opt A	Opt A
Road		Haul	Haul	Haul	Haul	Haul	Haul
Note		Crossing	Crossing	Crossing	Crossing	Crossing	Crossing
Sub area km2		22.97	22.97	22.97	22.97	22.97	22.97
Q req. (m3/s)	m3/s	21.61	29.91	38.61	50.63	75.96	105.69
H water	m	1.31	1.56	1.85	2.21	2.80	3.17
Q Total	m3/s	22.97	29.91	38.64	50.85	76.58	106.22
Q total > Q req.		OK	OK	OK	OK	OK	OK
Invert Level	m	1760.00	1760.00	1760.00	1760.00	1760.00	1760.00
Box Culverts							
B (width)	m	2.000	2.000	2.000	2.000	2.000	2.000
D (depth)	m	1.800	1.800	1.800	1.800	1.800	1.800
H/D max Boxes		1.20	1.20	1.20	1.20	1.20	1.20
Cb		0.90	0.90	0.90	0.90	0.90	0.90
Ch		0.60	0.60	0.60	0.60	0.60	0.60
Barrels	No	5	5	5	5	5	5
Actual H/D		0.73	0.87	1.03	1.23	1.56	1.76
Q total	m3/s	22.97	29.91	38.64	50.85	62.74	69.18
Q total > Q req.		OK	OK	OK	OK	Not OK!	Not OK!
Throat Velocity	m/s	1.76	1.92	2.09	2.30	2.24	2.18
Broad Crested Weir							
Height above Culvert Soffit	m	0.600	0.600	0.600	0.600	0.600	0.600
Road Length	m	32.0	32.0	32.0	32.0	32.0	32.0
H (above weir)	m	0.00	0.00	0.00	0.00	0.40	0.77
Q weir	m3/s	0.00	0.00	0.00	0.00	13.84	37.04
d (critical)	m	0.00	0.00	0.00	0.00	0.27	0.52
v (critical)	m	0.00	0.00	0.00	0.00	1.62	2.25
Throat Velocity	m/s	0.00	0.00	0.00	0.00	1.08	1.50
Total Volume							
Box Culverts	m3/s	22.97	29.91	38.64	50.85	62.74	69.18
Weir	m3/s	0.00	0.00	0.00	0.00	13.84	37.04
Total Volume	**m3/s**	**22.97**	**29.91**	**38.64**	**50.85**	**76.58**	**106.22**



LOW LEVEL STRUCTURE

Project Number: **002802**
Project Title: **Belfast Stormwater**
Done by: **CCLR**
Date: **23 July 2011**
Spreadsheet by RLR

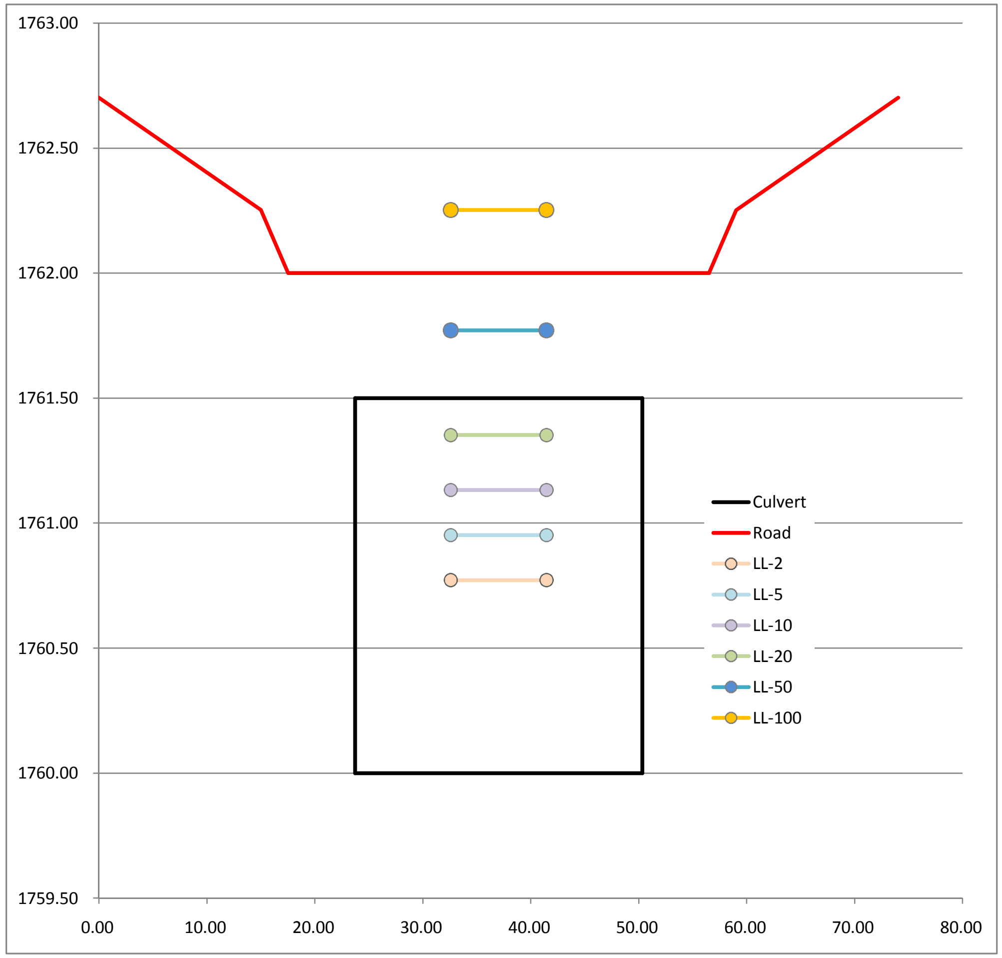
Catchment No.		LL-2	LL-5	LL-10	LL-20	LL-50	LL-100
Position		Opt B	Opt B	Opt B	Opt B	Opt B	Opt B
Road		Haul	Haul	Haul	Haul	Haul	Haul
Note		Crossing	Crossing	Crossing	Crossing	Crossing	Crossing
Sub area km2		22.97	22.97	22.97	22.97	22.97	22.97
Q req. (m3/s)	m3/s	21.61	29.91	38.61	50.63	75.96	105.69
H water	m	1.31	1.56	1.85	2.21	2.29	2.64
Q Total	m3/s	34.46	44.87	58.31	70.87	75.96	105.69
Q total > Q req.		OK	OK	OK	OK	OK	Not Ok!
Invert Level	m	1760.00	1760.00	1760.00	1760.00	1760.00	1760.00
Box Culverts							
B (width)	m	1.500	1.500	1.500	1.500	1.500	1.500
D (depth)	m	1.500	1.500	1.500	1.500	1.500	1.500
H/D max Boxes		1.20	1.20	1.20	1.20	1.20	1.20
Cb		0.90	0.90	0.90	0.90	0.90	0.90
Ch		0.60	0.60	0.60	0.60	0.60	0.60
Barrels	No	10	10	10	10	10	10
Actual H/D		0.87	1.04	1.23	1.47	1.53	1.76
Q total	m3/s	34.46	44.87	58.31	68.44	70.48	78.96
Q total > Q req.		OK	OK	OK	OK	Not OK!	Not OK!
Throat Velocity	m/s	1.76	1.92	2.10	2.06	2.05	1.99
Broad Crested Weir							
Height above Culvert Soffit	m	0.600	0.600	0.600	0.600	0.600	0.600
Road Length	m	39.0	39.0	39.0	39.0	39.0	39.0
H (above weir)	m	0.00	0.00	0.00	0.11	0.19	0.54
Q weir	m3/s	0.00	0.00	0.00	2.43	5.48	26.73
d (critical)	m	0.00	0.00	0.00	0.07	0.13	0.36
v (critical)	m	0.00	0.00	0.00	0.85	1.11	1.89
Throat Velocity	m/s	0.00	0.00	0.00	0.57	0.74	1.26
Total Volume							
Box Culverts	m3/s	34.46	44.87	58.31	68.44	70.48	78.96
Weir	m3/s	0.00	0.00	0.00	2.43	5.48	26.73
Total Volume	**m3/s**	**34.46**	**44.87**	**58.31**	**70.87**	**75.96**	**105.69**



LOW LEVEL STRUCTURE

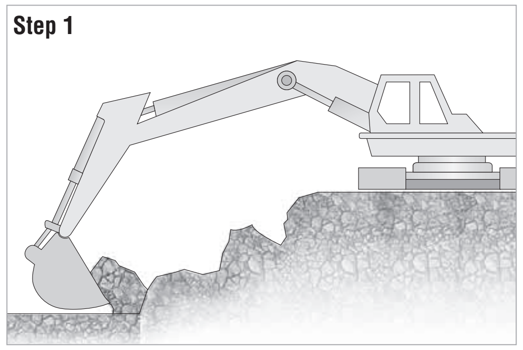
Project Number: **002802**
Project Title: **Belfast Stormwater**
Done by: **CCLR**
Date: **23 July 2011**
Spreadsheet by RLR

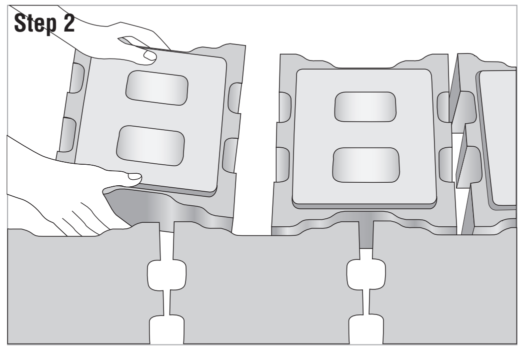
Catchment No.		LL-2	LL-5	LL-10	LL-20	LL-50	LL-100
Position		Opt C	Opt C	Opt C	Opt C	Opt C	Opt C
Road		Haul	Haul	Haul	Haul	Haul	Haul
Note		Crossing	Crossing	Crossing	Crossing	Crossing	Crossing
Sub area km2		22.97	22.97	22.97	22.97	22.97	22.97
Q req. (m3/s)	m3/s	21.61	29.91	38.61	50.63	75.96	105.69
H water	m	0.77	0.95	1.13	1.35	1.77	2.25
Q Total	m3/s	21.86	29.91	38.81	50.66	75.96	105.78
Q total > Q req.		OK	OK	OK	OK	OK	OK
Invert Level	m	1760.00	1760.00	1760.00	1760.00	1760.00	1760.00
Box Culverts							
B (width)	m	1.500	1.500	1.500	1.500	1.500	1.500
D (depth)	m	1.500	1.500	1.500	1.500	1.500	1.500
H/D max Boxes		1.20	1.20	1.20	1.20	1.20	1.20
Cb		0.90	0.90	0.90	0.90	0.90	0.90
Ch		0.60	0.60	0.60	0.60	0.60	0.60
Barrels	No	14	14	14	14	14	14
Actual H/D		0.51	0.63	0.75	0.90	1.18	1.50
Q total	m3/s	21.86	29.91	38.81	50.66	75.96	97.34
Q total > Q req.		OK	OK	OK	OK	OK	Not OK!
Throat Velocity	m/s	1.35	1.50	1.63	1.78	2.04	2.06
Broad Crested Weir							
Height above Culvert Soffit	m	0.500	0.500	0.500	0.500	0.500	0.500
Road Length	m	39.0	39.0	39.0	39.0	39.0	39.0
H (above weir)	m	0.00	0.00	0.00	0.00	0.00	0.25
Q weir	m3/s	0.00	0.00	0.00	0.00	0.00	8.44
d (critical)	m	0.00	0.00	0.00	0.00	0.00	0.17
v (critical)	m	0.00	0.00	0.00	0.00	0.00	1.29
Throat Velocity	m/s	0.00	0.00	0.00	0.00	0.00	0.86
Total Volume							
Box Culverts	m3/s	21.86	29.91	38.81	50.66	75.96	97.34
Weir	m3/s	0.00	0.00	0.00	0.00	0.00	8.44
Total Volume	**m3/s**	**21.86**	**29.91**	**38.81**	**50.66**	**75.96**	**105.78**

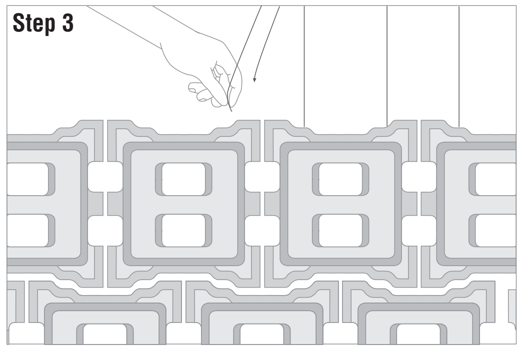


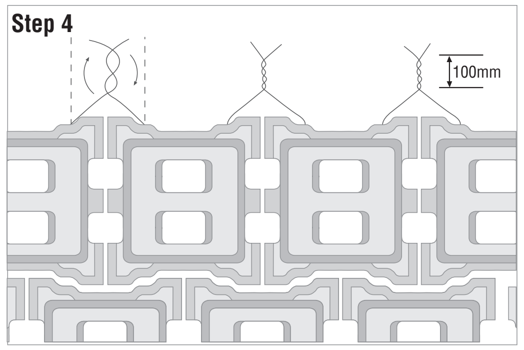
Annexure F:
GRASS BLOCK SYSTEM

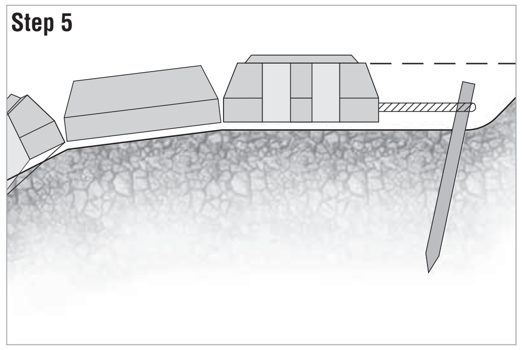
The 6 easy steps to site assembled Armorflex...

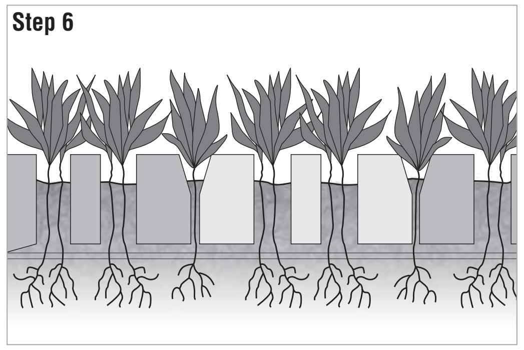












Step 1: ***Site preparation, excavation, trimming & compaction***
Prior to laying Armorflex, the base material must be profiled to line and level and should be compacted to a firm and even finish. Obstructions, such as roots and projecting stones should be removed as the quality of the preparation will be reflected in the finished surface. The angle of repose of the in situ material must not be exceeded. Maximum desired slope is 1:1,5

Step 2: ***Handling & placing by manual labour***
Armorflex loose block should be placed in a stretcher bond pattern to achieve the mechanical interlock. At areas such as culvert inlets and outlets, the blocks should be placed to allow for access to the cable ducts.

Step 3: ***Wiring up in situ***
The wire is easily pushed through the cable ducts in the blocks and secured as detailed in Step 4. The choice of wire will depend on the application. A 3,1 mm diameter galvanized fencing wire or a 5 mm diameter polyester rope can be used. In certain situations wiring up may not be necessary. Generally the wire will be threaded perpendicular to the flow.

Step 4: ***A final twist to the wire***
Galvanized wire can be twisted across the block joint for a length of minimum. 100mm or a suitable knot used on the polyester cable.

Step 5: ***Anchorage***
Armorflex placed on steep slopes may slide on the geotextile until the system has settled. Temporary or permanent anchorage can be achieved with steel or wooden pegs through the top cable loops.

Step 6: ***Finishing***
Armorflex subject to wave attack should be blinded with a sand/gravel mixture. Above normal waterline, the voids should be soiled and seeded to develop natural vegetation.

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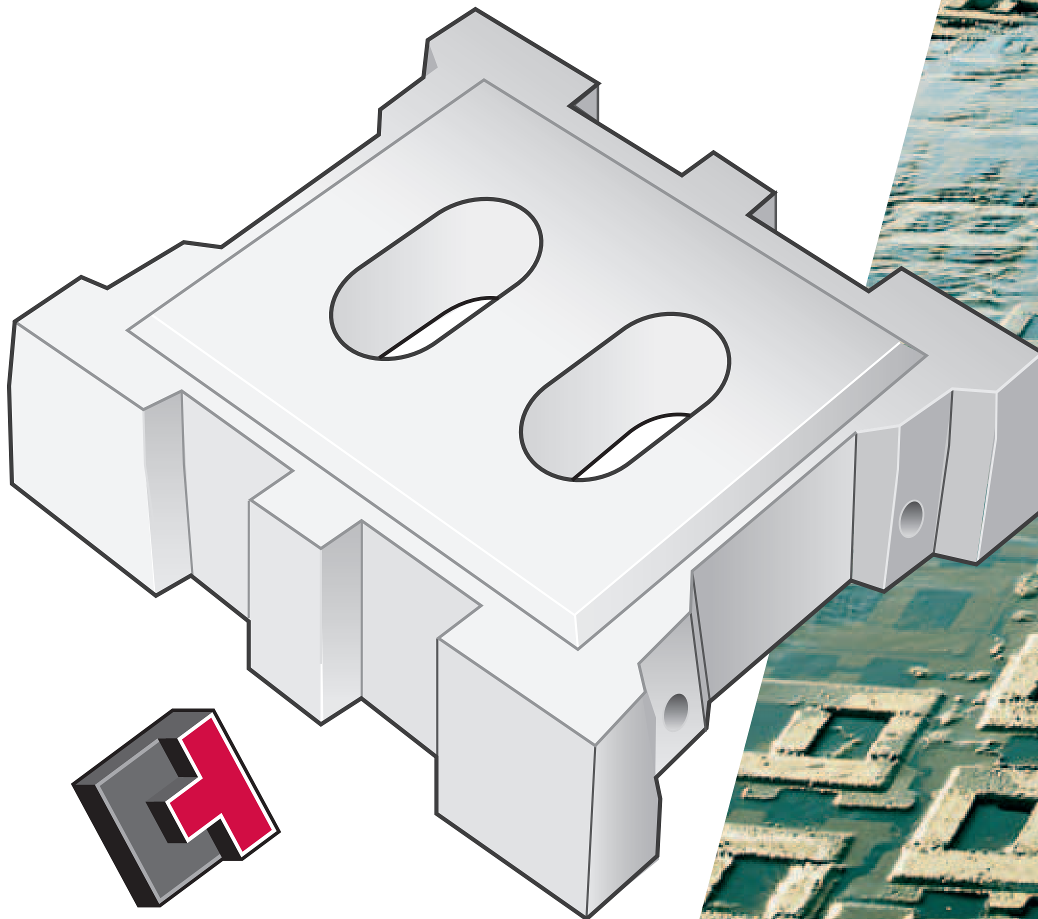
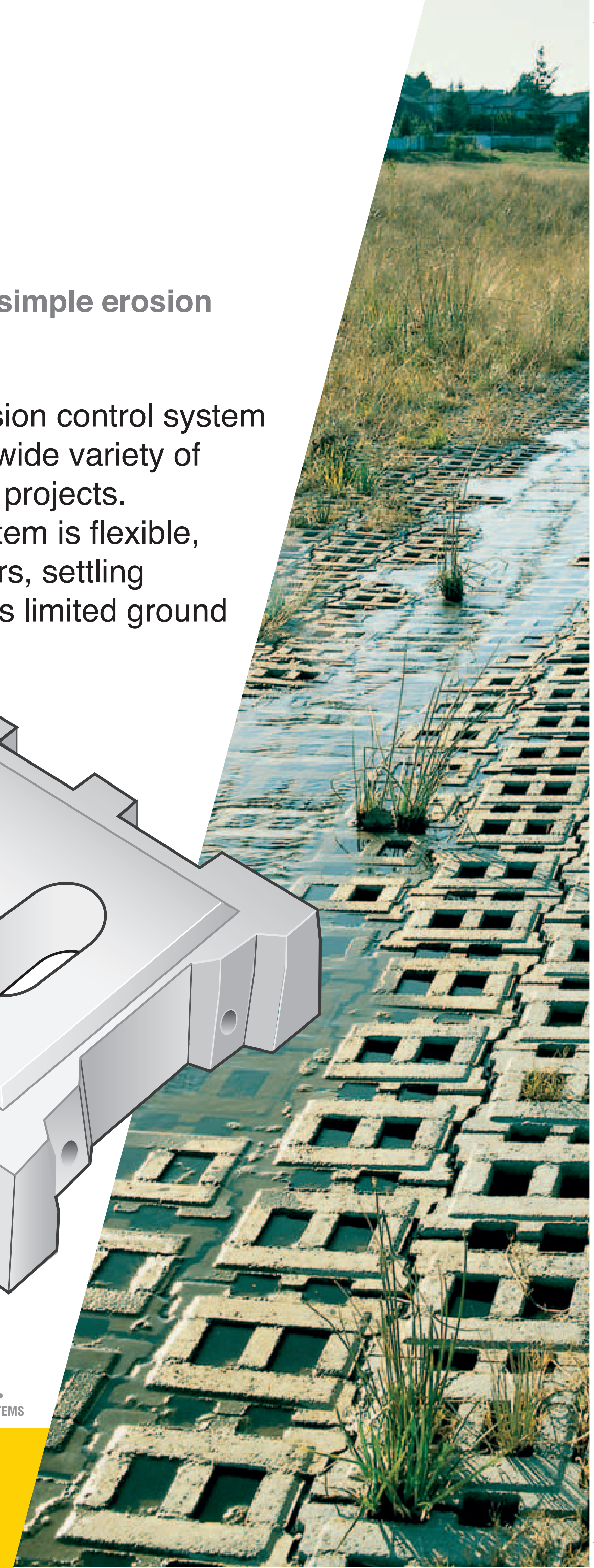
TECHNICRETE
Armorflex®

The engineered solution for simple erosion protection

Technicrete Armorflex® erosion control system provides an alternative for a wide variety of erosion control and drainage projects. **Technicrete Armorflex®** system is flexible, conforming to ground contours, settling without cracking, and requires limited ground preparation.



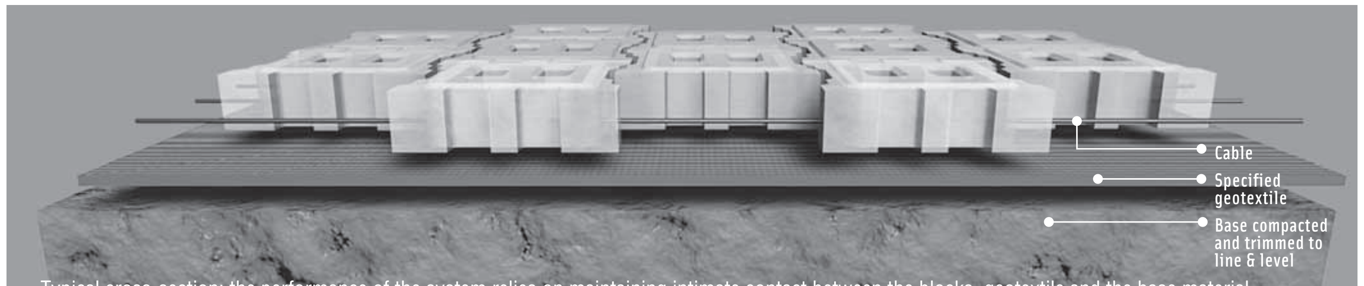


TECHNICRETE
quality support

CONCRETE ROOF TILES • **EROSION PROTECTION BLOCKS** • KERBS • MASONRY • PAVING • PREBAGGED PRODUCTS • RETAINING WALLS • STOPE SUPPORT SYSTEMS

Technicrete Armorflex®

When your project calls for protection that can withstand severe applications and climatic conditions, when it must be installed quickly with no in situ concrete, and even when it must be placed under water, ARMORFLEX is the engineered solution. Technicrete can provide design assistance and on site consultation if required.



Typical cross-section: the performance of the system relies on maintaining intimate contact between the blocks, geotextile and the base material.

The engineered solution

Civil Engineers throughout the world are continually faced with the problem of controlling erosion of coastal shorelines and inland waterways. Coastal areas frequently experience land loss and property damage resulting from the dynamic forces associated with wave attack, while inland waterways experience heavy currents which erode channel banks and beds, consequently resulting in unfavourable environmental conditions.

The ARMORFLEX Erosion Control System provides an engineered alternative for a wide variety of erosion control and drainage projects. The matrix of open cells and projections retain soils, relieve hydrostatic pressure and provide the perfect environment for establishing natural vegetation.

The ARMORFLEX system is flexible, conforming to ground contours, settling without fracture, and requires only limited ground preparation.

ARMORFLEX can be supplied palletized in loose block form for manual installation or in preformed mats for mechanical installation. The result is a stable protection designed to withstand high water velocities and wave attack with a finish that is environmentally acceptable.

Applications

Erosion control
ARMORFLEX provides defence against erosion in fast flowing streams and rivers. ARMORFLEX is particularly suitable for protection of rivers, estuaries, lakes, reservoirs and other areas subject to wave action. ARMORFLEX, with the stability of its specially designed blocks, provides flexible protection unaffected by subsidence and hidden by nature.

Roadways
ARMORFLEX provides an ideal heavy duty riding surface for temporary and permanent access roads, parking areas and stormwater drift crossings.

Drainage
ARMORFLEX provides an excellent lining for drainage channels. Bed and channel banks are stabilized against erosion caused by high velocities and the tendency of water to change the planned course of a channel. ARMORFLEX aprons at pipe inlets and outlets eliminate pipe undercutting that may lead to severe problems such as surrounding bank failure and siltation downstream. Other drainage applications include: ditch linings, spillways, headwalls, sediment basins and traps, pipe inlet protection, and protection of berms.

Characteristics

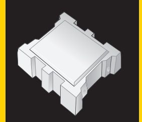
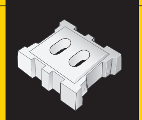
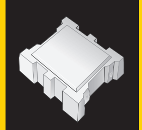
Stability
ARMORFLEX provides protection that acts as a single articulating mat to withstand the destructive forces of water. Where necessary, alternative weights and sizes of mats can be produced for special applications.

Flexibility
ARMORFLEX blocks are of a sophisticated design which allows the mat to remain flexible. The blocks are specially tapered to allow for this flexibility, maintaining minimum stress on the blocks. This facility enables ARMORFLEX to conform to contours even if settlement occurs after installation.

Filtration
ARMORFLEX mats are placed on a geotextile. The geotextile replaces graded filter materials for a more simplified installation. The permeability of the filter and blocks relieves hydrostatic pressure while its capacity for soil retention prevents leaching of materials through the installation.

Vegetation
ARMORFLEX, with stone filling in the cells, will greatly reduce the development of vegetal growth. When the cells are filled with topsoil, ARMORFLEX provides the perfect environment for the establishment of vegetation. Roots will penetrate the geotextile providing a permanent anchor for the installation.

Flow resistance
The ARMORFLEX matrix of open cells and projections create a surface with an engineered roughness. This surface roughness causes a loss of energy due to the formation of eddies within each open cell, thus reducing the potential for erosion. The Manning Roughness Coefficient, "n", of ARMORFLEX has a value ranging from 0.025 - 0.035, depending on the material filling the open cells and vegetal cover. ARMORFLEX 140 offers protection against flow velocities up to 3.5 m/s and ARMORFLEX 180 up to 5.5 m/s. Each project should however be carefully assessed to determine the correct specification and product size.

		Dimensions length x breadth x height (mm)	Normal plan size of block (mm)	No. of blocks (p/m²)	Weight of block (kg ave)	Unit weight (kg/m²)	Open area (%)	Vol. material to fill joints & voids (m³/m²)	Mat sizes (m)	Cable Factory assembled	In situ assemled	Vertical bending radius (m)
Armorflex 180		340 x 294 x 115	309 x 294	11	16.4	180	18	0.022	Standard 6.2 x 2.4 (20 x 8 blocks)	galvanised steel wire/synthetic rope	galvanised fencing wire/synthetic rope	0.5 min
Armorflex 205		340 x 294 x 115	309 x 294	11	19.2	205	8	0.008	Standard 6.2 x 2.4 (20 x 8 blocks)	galvanised steel wire/synthetic rope	galvanised fencing wire/synthetic rope	0.5 min
Armorflex 140		340 x 400 x 95	309 x 400	8	17.5	140	18	0.017	Standard 6.2 x 2.4 (20 x 6 blocks)	galvanised steel wire/synthetic rope	galvanised fencing wire/synthetic rope	0.5 min
Armorflex 165		340 x 400 x 95	309 x 400	8	20.6	165	8	0.009	Standard 6.2 x 2.4 (20 x 6 blocks)	galvanised steel wire/synthetic rope	galvanised fencing wire/synthetic rope	0.5 min

TECHNICRETE RESERVES THE RIGHT TO AMEND THE ABOVE SPECIFICATIONS WITHOUT NOTICE

Specifications: *Armorflex blocks consist of machine compressed concrete blocks which are either solid or with vertical holes and two horizontal cable ducts, depending on the application. The block shape is such that they interlock with each other transversely across the mat. The blocks have a partial taper to the sides which allow the system to articulate freely without disjointing. The partial taper encourages the ingress of fine granular particles into the joint between blocks.*

Annexure G:
GEOTECHNICAL REPORT

Jeffares & Green
ENGINEERING AND ENVIRONMENTAL CONSULTING

Experience - Quality - Integrity
Ageless Values

VERIFICATION PAGE

BELFAST MINE DAMS
GEOTECHNICAL REPORT
JULY 2011

JGI NO:	DATE:	REPORT STATUS:
2812	July 2011	FINAL

CARRIED OUT BY:	COMMISSIONED BY:
Jeffares & Green (Pty) Ltd P O Box 1109 Sunninghill 2157 Tel: +27 11 807 0660 Facsimile: +27 11 807 1607	Exxaro Resources
AUTHOR: Steven Bok	**CLIENT CONTACT PERSON:** Mr Daan Killian

SYNOPSIS:

Report presenting the results of a geotechnical investigation undertaken for the preliminary design of two earthfill storage dams.

KEY WORDS:

Exxaro, Belfast, Dams, Mpumalanga

QUALITY VERIFICATION

This report has been prepared under the controls established by a quality management system that meets the requirements of ISO9001: 2008 which has been independently certified by DEKRA Certification under certificate number 90906882/1

Quality Management ISO 9001:2008
▸ Voluntary participation in regular monitoring
DEKRA Certification
DEKRA certified

Verification	Capacity	Name	Signature	Date
By Authors	Eng. Geol.	S Bok	[signature]	27 JULY 2011
Checked by	Eng. Geol.	C Canahai	[signature]	27/07/2011
Authorised by	Director	P A Olivier	[signature]	27/7/11

BELFAST MINE DAMS

GEOTECHNICAL REPORT

BELFAST MINE DAMS

GEOTECHNICAL REPORT

1. INTRODUCTION

This report presents the results of a geotechnical investigation undertaken for the preliminary design of two lined earthfill storage dams at a proposed mine infrastructure site located near eMakhazeni (Belfast) in Mpumulanga. The dams will be required to store water pumped from the open cast mine and other contaminated runoff from the mine and each dam will have a storage capacity of approximately 230 000m^3.

The investigation included an initial assessment of the ground conditions at the proposed discard site.

The objectives of the investigation are as follows:

- Determine the geotechnical conditions at the proposed dam sites
- Locate suitable construction materials for the embankments (preferably within the dam footprints)
- Determine suitable design parameters (for both the in-situ soils and embankment materials) for input into the dam design

The field investigation was carried out between the 3rd and 7th of June 2011 and entailed the following:

- Excavation and profiling of 23 trial pits
- Driving of 23 Dynamic Cone Penetrometer tests
- Recovery of representative disturbed samples for laboratory testing

The interpretation of the overall subsurface conditions across the site is inferred, using professional judgment, from the interpolation and extrapolation of point information assimilated from the test positions. Given the relatively limited number of investigation points and the shallow excavation depths obtained using the available excavation methods, it is recommended that further investigations are undertaken for detailed design purposes.

2. AVAILABLE INFORMATION

The following information was available at the time of the site visit:

- Drawing titled "Belfast_wetlands_Golder_February_2011" showing wetland areas and the proposed layout of the dams and other mine infrastructure.

A directive that no mechanical excavation of trial pits may take place within the designated wetland areas was received from the client.

Jeffares & Green's Engineering Geologist was accompanied during the field investigations by Mrs Millicent Mkhwanazi from Exxaro.

3. SITE LOCATION

The proposed mine infrastructure site is located approximately 18 km south west of Belfast (by road). The proposed storage dams are located on the northern and southern side of a gentle sloping valley formed by a stream which runs in a rough east to west direction through the site. The dam on the northern side is designated the East Dam and the dam on the southern side the West Dam.

Locality Plans (Figures 1a and 1b) and a Site Layout Plan showing the position of the dams and the discard area are included below and overleaf.

Figure 1a: Locality Plan – Large Scale

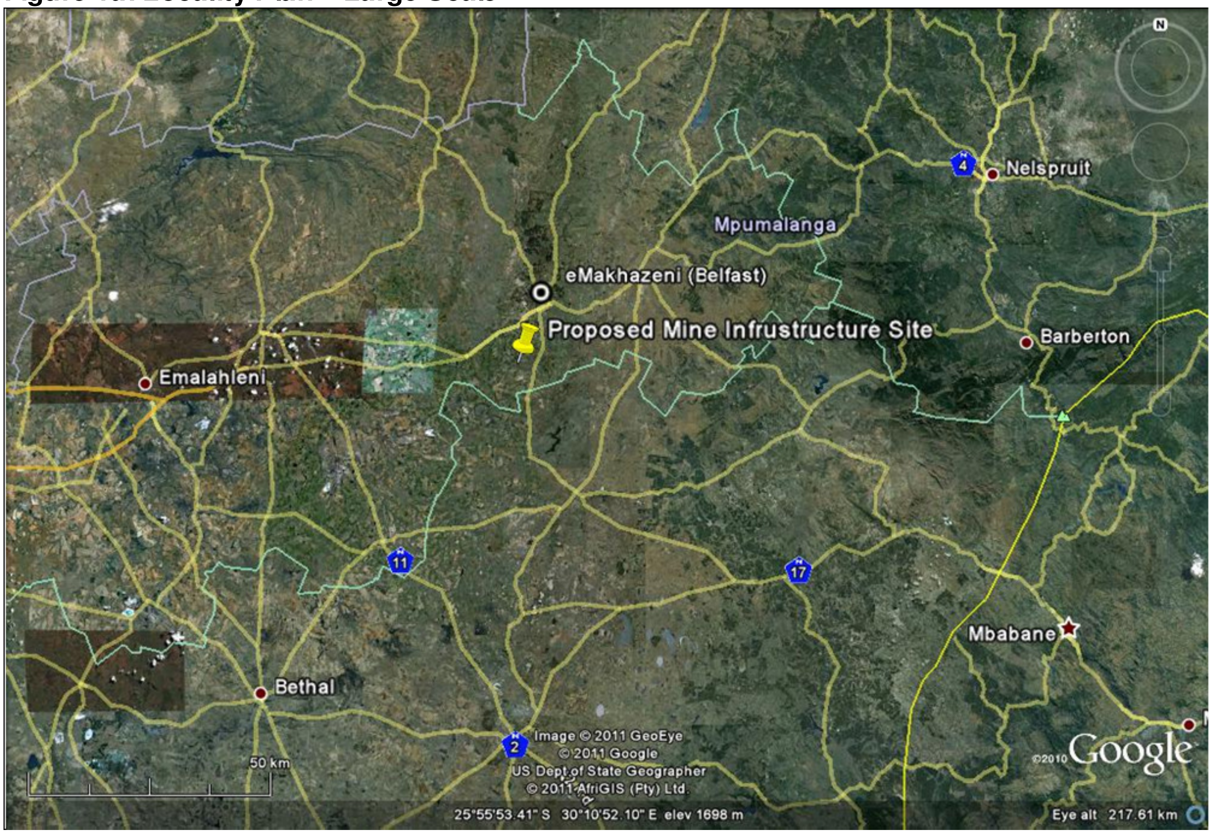


Figure 1b: Locality Plan – Medium Scale

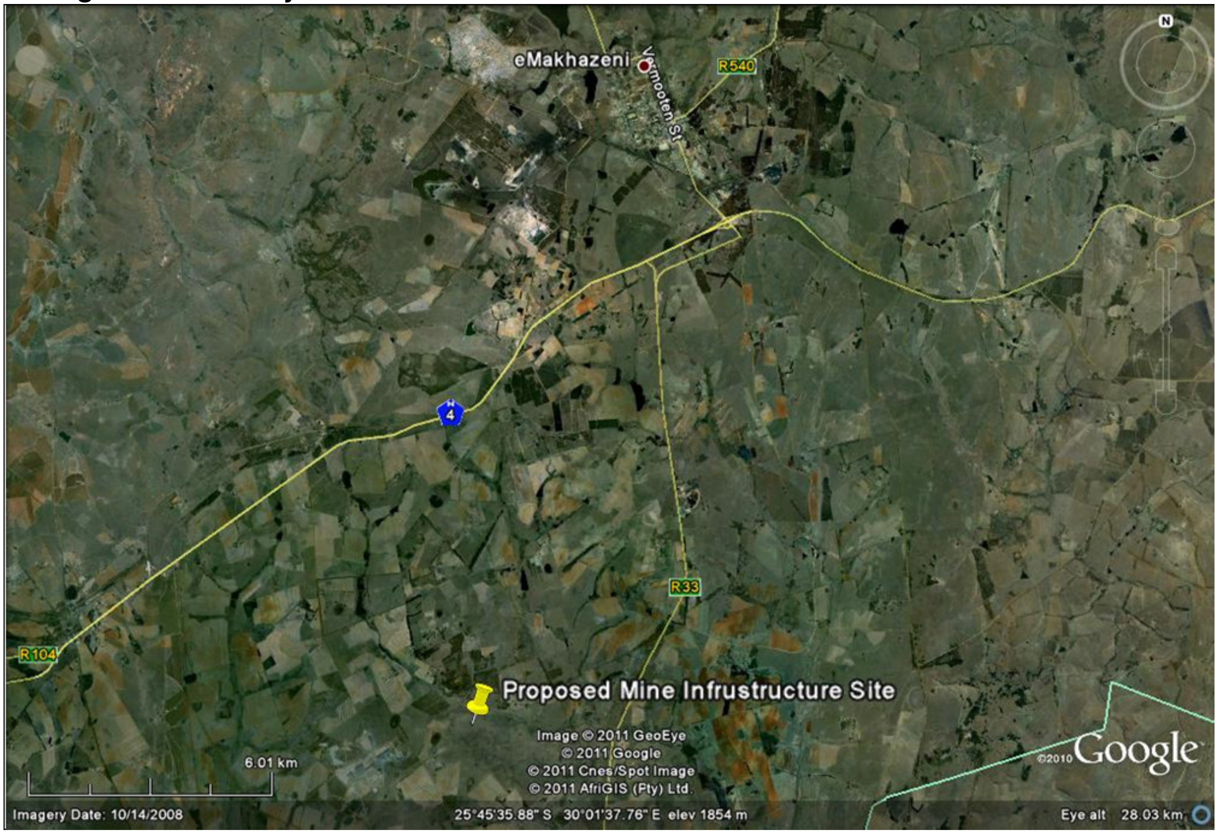


Figure 2: Site Layout Plan

3.1. Topography and Drainage

As mentioned above, the site is located within a gently sloping valley. The topography of the surrounding area is gently undulating.

The proposed footprint of the West Dam is located approximately 130 m south of the stream channel and the land slopes in a gentle northerly direction. The topography over the dam footprint ranges from a gently convex slope in the northern section of the footprint to a concave slope over the lower southern section. Poor drainage conditions were noted in many areas of the site. The northern section of the footprint indicated in Figure 3b appears to be located within the "permanent zone" of a wetland and standing water was observed at shallow depth below ground level in this area. Ground water seepage was also noted in the south western corner of the site.

The East Dam is located approximately 180 m north of the stream channel on elevated ground that slopes in a very gentle southerly direction. There are no drainage features on the site itself. However the flatter central section of the dam footprint is poorly drained and slight groundwater seepage was observed in TP19 excavated in this area.

The proposed Discard Area slopes in a general north easterly direction towards the stream. The south western boundary is located near the crest of a gentle ridge while the south north eastern section appears to overly a wetland formed by a drainage line running in a north easterly direction towards the stream.

3.2. Vegetation, Landuse and Existing Infrastructure

The natural vegetation of the area is grassland. Plantations of wattle, blue gum and pine trees have been planted to the north and east of the proposed mine infrastructure site and sections of the East Dam and the Discard Area are occupied by stands of trees.

The site is currently used for grazing animals and a section of the Discard Area is occupied by ploughed fields.

There is no existing infrastructure at the proposed dam sites. A number of fences traverse the Discard Area.

Old farmstead buildings occur to the east of the West Dam and graves were noted in this area. Piles of rocks were noted in the northern section of the East Dam site and the presence of graves should be investigated at this site.

Distinct piles of stones occur to the west of the drainage line on the western side of the East Dam and this area appears to be an old graveyard.

No underground infrastructure was encountered in any of the trial pits.

3.3. Access

The site is accessed via the "Eesterlingsfontein" road off the N4. Access to the West Dam and the Discard Area is obtained via tracks directly off the Eesterlingsfontein Road, as indicated in Figure 2. Access to the East Dam site is via various gravel roads and farm tracks from the north, as indicated in Figure 2. Sections of this access route will be difficult to traverse during the wet summer season and the route will need to be upgraded for construction vehicles.

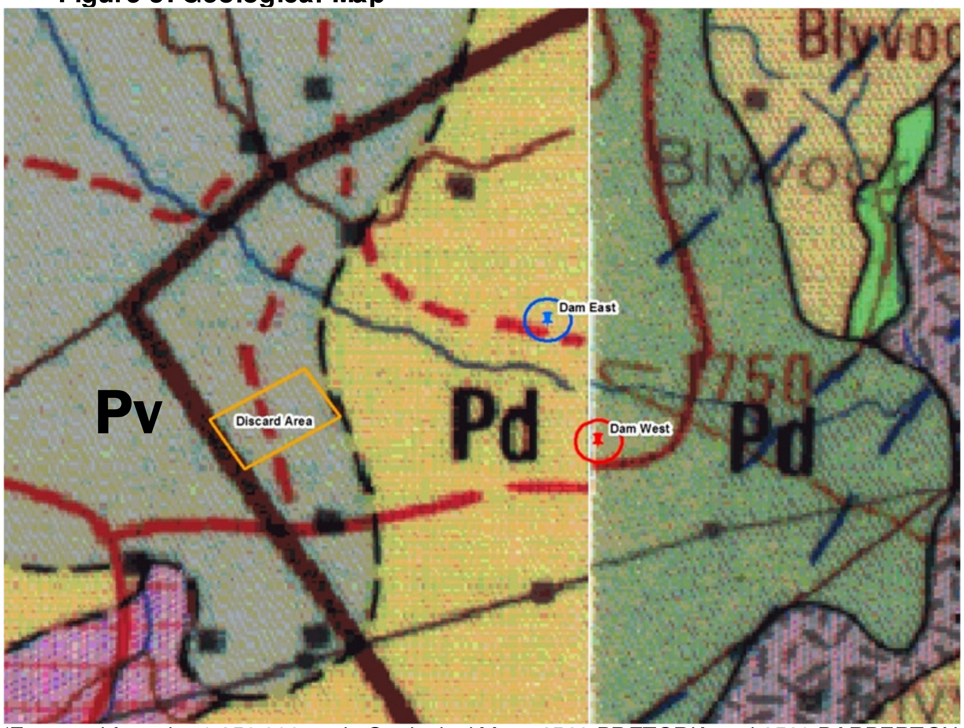
4. GEOLOGY

The 1:250 000 scale Geological Maps of the study area (2528 PRETORIA and 2530 BARBERTON) indicate that the two proposed storage dams are underlain by rock units of the Dwyka Group of the Karoo Supergroup. The Dwyka Group typically comprises diamictite, also known as tillite. However the group contains various rock types including the *stratified diamictite* facies containing mudrock, sandstone and conglomerate beds, the *conglomerate* facies and the *sandstone* facies consisting of fine- to medium- to coarse-grained sandstones (Johnson, *et. al*. 2006).

The proposed Discard Area is underlain by sedimentary rock units of the Vryheid Formation of the Ecca Group which also forms part of the Karoo Supergroup. The Vryheid Formation comprises quartzitic sandstone, pebbly and gritty sandstone, shale and coal measures.

A Geological Map of the area is given in Figure 3.

Figure 3: Geological Map



(Extracted from the 1:250 000 scale Geological Maps 2528 PRETORIA and 2530 BARBERTON published by the Council for Geoscience)

Legend:			
	Pd – Dwyka Group	Lithology: Shale	
	Pv – Vryheid Formation	Lithology: Sandstone, lesser shale, coal	} Ecca Group

5. CLIMATE

The climatic regime plays a fundamental role in the development of a soil profile. Weinert (1964), through his studies of weathering of basic igneous rocks, demonstrated that mechanical disintegration is the predominant mode of rock weathering in areas where his climatic "N-value" is greater than 5, while chemical decomposition predominates where the N-value is less than 5. Weinert's climatic N-value for the Belfast area is approximately 1,7. This implies that chemical decomposition is the dominant mode of weathering in the study area.

6. FIELDWORK

The fieldwork was undertaken on the 3rd, 6th and 7th of June 2011, during the drier winter season.

The approximate positions of the trial pits are shown on the Site Plans, Figures 3a to 3c overleaf. The test positions were recorded using a Garmin e-trex hand-held GPS.

All depths provided were measured from existing ground level at each test position.

6.1. Trial Pits

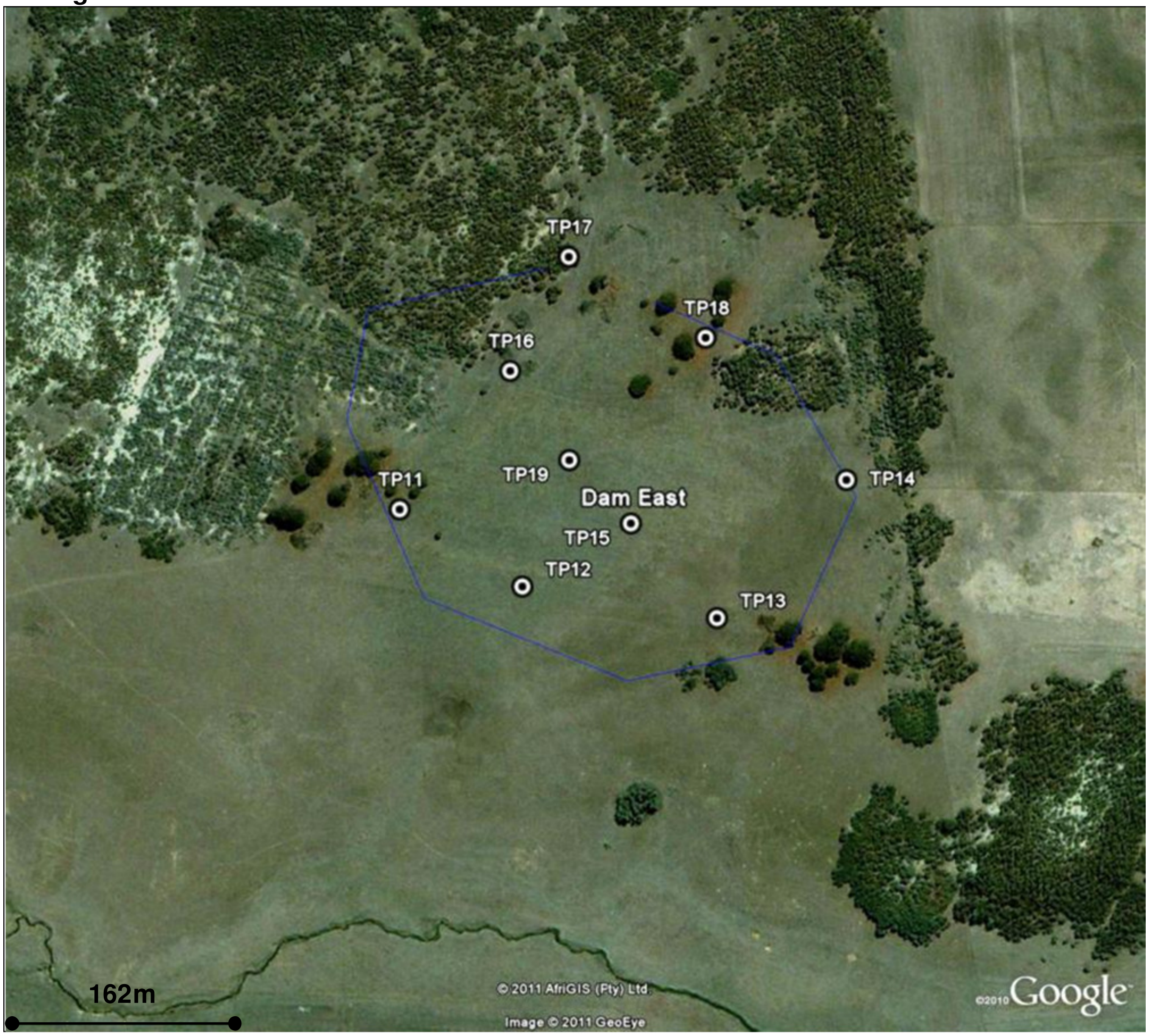
A total of 23 trial pits, designated TP1 to TP23, were excavated across the three sites. The trial pits were profiled immediately after excavation by our Engineering Geologist in accordance with the method of Jennings et al., (1973). The trial pits were loosely backfilled after profiling.

TP1 to TP10 were excavated by hand auger at the West Dam site to depths of between 0,20 and 1,05 m. TBL access was not permitted at the West Dam site as the site falls within the area designated as a wetland on the wetland map referenced in Section 2.

Figure 3a: West Dam Site

TP11 to TP19 were excavated by TLB at the East Dam site to depths of between 0,15 and 1,15 m. The test pits were terminated at the refusal depth of the TLB.

Figure 3b: East Dam Site



TP20 to TP23 were excavated by TLB at the Discard Area site to depths of between 0,12 and 1,80 m.

Figure 3c: Discard Area

The trial pit profiles are attached in Appendix A and photographs of the soil profiles are provided in Appendix C.

6.2. DCP Tests

Twenty three in-situ Dynamic Cone Penetrometer (DCP) tests, designated DCP1 to DCP23, were carried out adjacent to each of the trial pits.

The DCP apparatus consisted of a 10 kg weight falling from a drop height of 450 mm onto a string of rods with a 25 mm diameter end-cone with a 60 degree apex angle.

The DCP tests were advanced to depths of between 0,12 and 1,90 m below existing ground level. The DCP tests were terminated when the blow count exceeded approximately 50 blows per 300 mm, or refusal. The results have been used to derive, empirically, Estimated Allowable Safe Bearing Pressures (EASBP) for the soils. A non-cohesive soil profile has been assumed for the purposes of interpreting the DCP results as a predominantly sandy soil profile was encountered. The estimation of the EASBP's is based on Terzaghi's chart for allowable bearing pressures for less than 25 mm of settlement.

The DCP test results may also be used to obtain a rough estimate of the shear strength of the soils.

The blow counts obtained from the DCP tests indicate that the soils at shallow depth (overlying the ferricrete / weathered rock / gravel) typically have “loose” or “very loose” consistencies. Extremely low blow counts of 1 blow per 300 mm were recorded at DCP6 and DCP8 which were undertaken within the permanent zone of the wetland.

Refusal of the DCP probe is attributed to the presence of hard ferricrete, gravel (which typically occurred immediately above the ferricrete in the trial pits) or possibly weathered rock.

An aspect of DCP testing that should always be borne in mind is that the results are affected by the moisture content of the soil profile, as well as any gravel, concretions or boulders that may be struck. A dry soil horizon will provide higher consistencies than a similar test undertaken during the rainy season, when percolating water softens the subsoils. Moisture content should thus always be noted and made mention of in any DCP investigation. Soils within the proposed site were described as “moist” or “wet” and as such a significant reduction in strength with increasing moisture content is not expected.

The results of the DCP tests are included in Appendix B.

7. LABORATORY TESTING

The following laboratory tests were carried out on disturbed soil samples recovered from the trial pits:

	Refer to:
• Grading analyses and hydrometer tests	Appendix D1
• Atterberg limit and linear shrinkage determinations	Appendix D1
• Triaxial testing	Appendix D2
• Moisture density relationship (Standard PROCTOR)	Appendix D3
• Permeability testing (Falling Head Permeability)	Appendix D4
• Chemical testing	Appendix D5

Table 7.1: Grading and Atterberg Limit Determinations

Pit No	Depth (m)	Description	Particle Size %				Atterberg Limits %			GM	Heave Potential
			Clay	Silt	Sand	Gravel	LL	PI	LS		
TP3	0,10-0,80	Sandy fine gravel (colluvium)	5	10	33	51	NP	NP	0,0	1,92	Low
TP6	0,30-0,90	Slightly silty sand (alluvium)	6	15	70	10	NP	NP	0,0	1,20	Low
TP8	0,20-0,80	Silty clayey sand (alluvium)	18	17	62	3	25	12	5,0	0.83	Low
			18	18	61	2	23	10	5,0	0,80	Low
TP11	0,20-0,70	Sandy ferricrete gravel	2	10	22	66	NP	NP	0,0	2,26	Low
TP12	0,70-1,00	Silty gravelly sand (colluvium)	8	13	47	33	19	6	2,0	1,53	Low
TP15	0,25-0,75	Sandy gravel (various)	5	10	38	47	20	8	2,5	1,87	Low
TP16	0,35-0,85	Slightly silty sand (colluvium)	5	16	58	21	NP	NP	0,0	1,28	Low
TP17	0,20-0,50	Silty sand (colluvium)	6	15	65	14	NP	NP	0,0	1,22	Low
TP20	0,30-0,70	Silty sand (colluvium)	6	25	67	2	NP	NP	0,0	0,87	Low

Pit No	Depth (m)	Description	Particle Size %				Atterberg Limits %			GM	Heave Potential
			Clay	Silt	Sand	Gravel	LL	PI	LS		
TP20	1,05-1,20	Silty gravelly sand (colluvium)	5	14	37	43	20	6	2,5	1,76	Low
TP22	1,00-1,75	Silty sand (residual sandstone)	8	24	49	19	27	8	4,0	1,13	Low
TP23	0,95-1,80	Slightly clayey silty sand (residual sandstone)	11	27	58	5	26	10	4,0	0,82	Low

LL- Liquid Limit GM - Grading Modulus LS - Linear Shrinkage PI - Plasticity Index
Heave Potential – assessed according to the Van der Merwe method (Williams & Donaldson 1980)

The laboratory test results indicate that the colluvial soils are predominantly sandy or gravelly in composition low with PI's (Plasticity Index) of between non-plastic and 8.

The alluvial soil recovered from TP6 (0,30-0,90 m) was non-plastic while the alluvial soil recovered form TP8 (0,20-0,80 m) had a higher clay content and was moderately plastic with a PI of 10 to 12.

The residual sandstone soils recovered from TP22 (1,00-1,75 m) and TP23 (0,95-1,80 m) were predominantly sandy in composition and were moderately plastic with PI's of 8 to 10.

Table 7.2: Triaxial Test Results

Test Position	Depth (m)	Sample Preparation	Angle of internal Friction (Phi) Effective Strength) Degrees	Cohesion (C Effective Strength) kPa
TP11	0.20-0.70	Remolded	35,4	10,0
TP15	0.25-0.75	Remolded	34,1	12,8
TP16	0.35-0.85	Remolded	32,3	17,9

1) Triaxial tests Consolidated Undrained (CU) tests with pore water pressure measurements
2) Tests undertaken on disturbed samples re-compacted to 95% Proctor density
3) Samples saturated prior to testing
4) Specified normal stress: 50, 100, 200 kPa

Table 7.3: Moisture Density Relationship & Permeability Test Results

Pit No	Depth (m)	Optimum Moisture Content (%)	Maximum Dry Density Mod AASHTO (kg/m^3)	Coefficient of Permeability (m/s)
TP11	0.20-0.70	10,8	2105	$1,7 \times 10^{-8}$
TP15	0.25-0.75	11,9	2168	$2,6 \times 10^{-8}$
TP16	0.35-0.85	7,7	2080	$5,9 \times 10^{-8}$

1) Moisture density relationship undertaken using Standard Proctor compactive effort
2) Permeability Coefficient obtained using the Falling Head test method on disturbed samples re-compacted to 95% Proctor density under a load of 100 kPa

Chemical testing was undertaken on representative soil samples to determine the aggressiveness of the soils (and of the percolating groundwater) to concrete and steel. The aggressiveness of the soils to concrete was determined using the method developed by J. J.

Basson, which is described in Fulton's Concrete Technology (1994). The results of the analyses are presented in Appendix D5 and are summarised in Table 7.4.

Table 7.4: Chemical Analysis to Determine Aggressiveness to Concrete (Basson Index)

Pit No	Depth (m)	Soil Type	Aggressiveness Index (Nc corrected for stagnant conditions)	Aggressiveness
TP6	0,30-0,90	Slightly silty sand (alluvium)	1746	Very highly corrosive
TP11	0,20-0,70	Sandy ferricrete gravel	1526	Very highly corrosive
TP16	0,35-0,85	Slightly silty sand (colluvium)	1561	Very highly corrosive

Chemical analysis was undertaken to determine the corrosivity of the soils to buried metal and concrete. The chemical analysis results were used to determine the Langelier Index, the Ryznar Stability Index, Stability pH and Aggressiveness Index. The full chemical analysis results are included in Appendix D5 and the results are summarised in Table 7.5.

Table 7.5: Chemical Analysis to Determine Corrosivity

Pit No	Depth (m)	Chemical Analysis					Corrosiveness towards concrete & metals
		pH Stability	Langelier Index	Ryznar Index	Aggress-iveness Index	Cl / SO4 Corrosivity Index	
TP6	0,30-0,90	10,6	-4,9	15,5	6,5	2,4	Very highly corrosive
TP11	0,20-0,70	10,5	-4,2	14,6	7,3	1,9	Very highly corrosive
TP16	0,35-0,85	10,7	-4,5	15,2	7,0	3,8	Very highly corrosive

The chemical tests indicate that the soils are very highly corrosive towards concrete and metals. The corrosive nature of the soils must be taken into account for the design of buried structures.

8. GENERAL ASSESSMENT OF THE DAM AND DISCARD SITES

The ground conditions encountered in trial pits at each location are described below.

8.1. Soil and Rock Conditions – West Dam

The profile descriptions at the west dam are based on materials recovered from hand auger excavations. The excavation depths obtained with the hand auger was limited due to the presence of gravel and ferricrete at most test positions.

8.1.1. Colluvial Soils

Colluvial soils were encountered in all trail pits with exception of TP6 and TP8 and extended from surface to depths of between 0,20 and 1,05 m (average depth 0,53 m). The soil profile typically consisted of an upper horizon of brown to grey brown, loose, silty sand to gravelly sand extending to depths of approximately 0,10 to 0,25 m below ground level.

The upper colluvial soil in the higher-lying areas was typically underlain by pale orange to orange brown, loose, slightly gravelly silty sand.

The gravel content of the colluvial soils generally increased with depth and refusal of the hand auger on gravel occurred at many test positions.

8.1.2. Pedogenic Soils - Ferricrete

The accumulation of iron oxides and hydrates is a commonly occurring pedogenic phenomenon related to a varying water table. This process takes place between the limits of a fluctuating water table and results in the formation of mottles and hard concretions, often with dark brown or black centres. With time the concretions may coalesce, resulting in an open honeycomb structure (commonly known as honeycomb ferricrete), or a continuous sheet of cemented material, commonly known as hardpan ferricrete.

Honeycomb ferricrete was encountered at the base of TP2 (0,50 m) and TP3 (0,80 m) and loose ferricrete gavel was observed in TP10 between 0,80 and 1,05m below ground level.

Hardpan ferricrete was also noted in the vicinity of TP1 and occasionally over the higher-lying sections of the site.

8.1.3. Alluvial Soils

Alluvial soils are transported and deposited by flowing water. Alluvium was encountered in TP6 and TP8 from surface to the base of both trial pits at 0,90 m.

An upper horizon of dark grey brown, loose, silty sand with abundant organic matter was encountered in TP6 and TP8 to depths of 0,10 and 1,20 m, respectively. Light grey to light grey brown mottled orange, very loose slightly silty sand was encountered to depths of 0,30 m (TP6) and 0,50 m (TP8). Light grey to beige mottled light orange, loose slightly silty sand containing cobbles with depth was encountered below the aforementioned horizon in both trial pits.

Bedrock was not encountered at the West Dam site.

8.2. Soil and Rock Conditions – East Dam

The ground conditions encountered in the mechanically excavated trial pits are described below. The trial pits were excavated to the refusal depth of the TLB which ranged from 0,15 to 1,15 m (average refusal depth 0,65 m).

8.2.1. Colluvial Soils

Colluvial soils were observed in all trial pits from surface to depths of between 0,10 m to 1,00 m below ground level (average depth 0,47 m).

In areas of deeper soil cover the profile typically consisted of brown, loose, silty fine sand to gravelly silty fine sand underlain by pale orange to pale orange brown silty sand.

Colluvial gravel generally consisting of sandstone rock fragments and transported ferricrete gravel in a silty sand matrix was encountered TP12 (0,70-1,00 m), TP13 (0,10-1,55 m) and TP15 (0,40 – 0,65 m).

8.2.2. Pedogenic Soils - Ferricrete

Pedogenic soils in the form of hardpan ferricrete, honeycomb ferricrete, nodular ferricrete and ferruginised sand were observed beneath the colluvial soils in all nine trial pits.

The pedogenic soils observed in the trial pits have formed in a predominately sandy parent material that has the appearance of weathered sandstone rock in some of the trial pits.

8.3. Soil and Rock Conditions – Discard Area

Four trial pits were excavated at the proposed Discard Area. One trial pit (TP21) refused at shallow depth (0,10 m) on hardpan ferricrete while the remaining trial pits were advanced to depths of between 1,30 and 1,80 m.

8.3.1. Colluvial Soils

With the exception of TP21, broadly similar colluvial soils were observed to extend from surface to depths of between 0,95 and 1,20m. An upper horizon of light brown, loose, silty fine sand was observed from surface to between 0,30 and 0,35 m. This was underlain by pale orange to orange brown, loose, silty sand to a depth of 0,70 m in all three trial pits. A soil horizon containing gravel (interpreted to be a gravel marker) was observed in all three trial pits.

8.3.2. Pedogenic Soils - Ferricrete

Weakly cemented honeycomb ferricrete was encountered in TP20 between 1,20 m and the refusal depth of the TLB at 1,30 m.

The TLB refused on hardpan ferricrete at a depth of 0,12 m at TP21.

8.3.3. Residual Sandstone Soils

Residual soils are formed from the complete in-situ weathering of the underlying bedrock.

Residual sandstone soils described as pale orange, medium dense to dense, silty fine sand was observed in TP22 and TP23 to depths of 1,75 and 1,80 m, respectively.

Sandstone rock was not encountered in the trial pits.

8.4. Groundwater

Groundwater seepage or standing water was observed in the following trial pits:

West Dam Site
TP4 0,20 m (Moderate seepage)
TP6 0,00 m (Free standing water)
TP8 0,10 m (Free standing water)

East Dam Site
TP12 1,00 m (Slight seepage)
TP19 0,55 m (Slight seepage)

It must be noted that the investigation was undertaken during the dry winter season. Given the poor drainage conditions seepage is probable in other areas during the wetter summer season and after rainfall.

The groundwater levels and the subsequent rates of infiltration into excavations will vary seasonally.

8.5. Expansive, Collapsible and Dispersive soils

The soils encountered during the investigation were described as predominantly sandy in composition. The laboratory test results indicate that the soils have “low” heave potential and problematic ground conditions arising from expansive soils are not expected.

The colluvial silty fine sand encountered in TP23 (0,35 – 0,70 m) was described as “open-voided”. This texture is characteristic of a potentially collapsible soil. Further investigations should be designed determine the collapse potential of the soils.

Problems associated with dispersive soils are not anticipated.

9. GEOTECHNICAL ASSESSMENT

The project involves the construction of two earth embankment dams with wall heights of approximately 6 m. It is understood that the dams will be lined to prevent water loss. The design of the dam embankments had not been confirmed at the time report this report was compiled. The dam design will take into account the materials available on site.

The following broad assessment is provided for the two storage dams.

9.1. Foundations

There are two main geotechnical criteria for considering the foundations for a dam, firstly, the dam needs to be founded on competent material of sufficient strength so that settlements are limited and secondly the material beneath the dam must not allow for excessive seepage under of the dam, which has the effect of destabilising the dam and also results in water loss.

The investigation indicates that founding material with sufficient strength to support an earthfill dam embankment will be encountered at shallow depth at the East Dam site. Similar competent foundation material will is expected at shallow depth in the central and southern section of the proposed West Dam footprint. Very loose material was encountered in TP6 and TP8 in the northern section of the West Dam footprint. This material will have a very low shear strength and will be problematic as a founding medium.

Free standing water was observed at very shallow depth in TP6 and TP8 and compaction and construction activities in this area will require dewatering. It is therefore recommended that dam footprint is shifted in a southerly direction to avoid these problematic geotechnical conditions.

The permeability of the soils was found to be moderately permeable with permeability coefficients of between $1{,}7 \times 10^{-8}$ and $5{,}9 \times 10^{-8}$ m/s.

9.2. Clay Core

The majority of the natural materials encountered in the trial pits have a low clay content and appeared unsuitable for use in construction of a clay core. This assessment was confirmed by the laboratory test results and none of the soils tested had properties within the range generally considered suitable for clay core.

Given the geology of the area, obtaining sufficient natural material suitable for clay core material in close proximity to the project area will be problematic.

9.3. Embankment Material

The predominantly sandy and gravelly colluvial soils encountered in the trial pits will be suitable for embankment construction. Blending of these materials with the underlying ferricrete and weathered sandstone rock will improve the material shear strength properties and is recommended for construction. However, as discussed in Section 9.4, excavation into this material will require the use of heavy excavation plant or possibly blasting.

The strength properties of selected representative soil samples were obtained from triaxial testing on samples re-compacted to specifications anticipated for embankment construction. These results are considered suitable for preliminary design purposes. Given the shallow excavation depths achieved during this investigation it is recommended that further shear strength testing is undertaken on representative samples of the actual materials that will be used during construction.

9.4. Ease of Excavation

Soft excavation conditions are expected within the colluvial and residual soils. These soils were found to occur at shallow depth in most areas of the dam sites and were underlain by harder ferricrete or gravel. The average refusal depth of the hand auger at the West Dam site was 0,60 m and the average refusal depth of the TLB at the East Dam site was 0,65 m. “Intermediate” to “hard” excavation conditions are expected at an average depth of less than approximately 1.0 m at both sites.

Given the difficult excavation conditions it is recommended that further investigations are undertaken using large excavation plant such as a track-mounted excavator fitted with a rock bucket. Alternatively rotary core drilling may be considered.

10. CONCLUSIONS

This geotechnical investigation was undertaken to provide information for the preliminary design of two lined earthfill storage dams. The investigation indicates that the sites are suitable for construction of the storage dams, provided that the recommendations provided in this report are implemented. The recommendations include moving the West Dam footprint in a southerly direction to avoid problematic ground conditions that were encountered in the low-lying northern section of the proposed footprint.

Even though the equipment employed to excavate trial pits was only able to penetrate to limited depths, the investigation undertaken provides an adequate picture of the ground conditions for the purposes of preliminary design. It is recommended that further investigations are undertaken for detailed design purposes.

11. REFERENCES

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