RobertBirdGroup

Water Management

(Including Stormwater

& Soil Erosion Management Plan)

Proposed Stables Precinct Royal Randwick Racecourse, Randwick NSW.

Prepared For: Australian Jockey Club

September 2010

Job No.:10619

Report No: 10619-SW

benchmark IN QUALITY ISO 9001

JAS-ANZ

Report Issue Register

Rev. No.	Issue	Author	Project Engineer	Checked	Date
1	Draft Issue	Ian Harris	Ian Harris	Laurence Melville	August 2010
2	Final Issue	Ian Harris	Ian Harris	Laurence Melville	September 2010

Report Amendment Register

Rev. No.	Section & Page No.	Amendment
1	N/A	Original Issue
2	N/A	Final Issue

FINAL DRAFT ACCEPTED BY:

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References:

1. Stormwater Management Plan, Freeway North Business Park, Weakleys Drive, Beresfield, Rev E – PPK Environment & Infrastructure Pty Ltd – Dated Feb. 2002

2. Stormwater Management Plan – Stage 1 works – 2008 Upgrade prepared for the Australian Jockey Club, November 2008 – WMAwater.

3. Director-Generals requirements, New Stables Facility, Royal Randwick Racecourse, Randwick (MP 10_0098). Dated August 2010.

4. Preliminary Comments – Infiltration Rates to the Botany Aquifer at Randwick Racecourse, Water Research Lanoratory. Dated November 2007.

Design References:

- Managing Urban Stormwater: Soils & Construction (Landcom)
- Soil and Landscape Issues in Environmental Impact Assessment (DLWC)
- Floodplain Development Manual: The Management of Flood Liable Land (DIPNR).
- Floodplain Risk Management Guideline – Practical Consideration of Climate Change (DECC)
- Managing Urban Stormwater: Council Handbook. Draft (EPA).
- Managing Urban Stormwater: Treatment Techniques (EPA).
- Managing Urban Stormwater: Source Control. Draft (EPA).
- Managing Urban Stormwater: Harvesting and Reuse (EPA).

1.0 Executive Summary

This report has been prepared by Robert Bird Group to support a Part 3A application to be submitted under the Environmental Planning and Assessment Act, 1979, for a stables development on the Royal Randwick Racecourse site, Randwick, NSW.

The report has been created in response to the Director Generals Requirements (10_0098 dated 06 August 2010) in particular the section on soil and water.

The report addresses issues of flooding, site stormwater management, control of discharge from the site, water quality and water usage management.

The stormwater management philosophy discussed in this report has be prepared following dialogue with Randwick City Council's Development Engineering team and the principles have been agreed.

Flood levels have been obtained from a report prepared by WMAwater (Ref 2). The flood report demonstrates that the proposed development area is not located within the current 100 year flood envelope and as a result there will be no increase in flood levels caused by the proposed development. All building floor levels will have more than 500mm freeboard above the estimated 100-year flood levels and above overland flow paths.

A drainage diversion will be provided as part of the development. The diversion will retain the existing piped flows which are discharged into the existing site detention basin. In order to prevent additional flows discharging to the existing basin the pipe size for this diversion has been retained.

Underground piped stormwater systems will be provided to cater with all stormwater runoff from roofed and paved areas, with overland flow paths defined to cater for safe disposal of runoff in excess of the pipe system capacity.

Peak stormwater discharge from the site will be reduced by both the on-site retention of roofwater for re-use on site and a new detention pond positioned at the down stream corner of the development area. The on site detention will attenuate the post development peak discharge flows back to the pre development peak discharge flows. The existing detention ponds on site will provide further attenuation resulting in the total site discharge from the racecourse meeting with Council requirements.

Roofwater will be retained on site in above ground tanks each stable will have two tanks, each tank with a capacity of 30,000 litres. The water in these tanks will be used for washing the horses, the stable buildings and providing water for toilet flushing in the stable apartments.

Stormwater quality will be improved by the use of combined gross pollutant trap / oil and silt arrestor to provide filtration and sedimentation in accordance with Randwick City Council policies.

Development plans have been prepared showing the proposed extent of Stormwater Drainage and Erosion and Sedimentation Control works for the proposed development. These drawings are included in the Appendix to this report.

2.0 Introduction

It is proposed to develop part of the Royal Randwick Racecourse site with new stables and associated infrastructure. The stables will include six double storey stable buildings with a mezzanine level for strapper/ stable hand accommodation. Each stable will have two two storey mechanical horse walker rings, a new tie up stall area, a farrier kiosk, an equine pool, a waste collection area, a new subterranean infield access tunnel and associated vehicle parking areas.

This report forms part of the Project Application documentation to accompany the environmental assessment for the Part 3A application. Accordingly this report responds to the DGR's issued by the Director General on 06 August 2010.

This report has been prepared in support of this application. It combines the requirements for a Water Usage Report and a Stormwater Drainage Report, as these separate requirements have many elements in common.

3.0 Site Description

The location of the Stables Precinct ("the site") is at the south east corner of the Royal Randwick Racecourse site, Randwick, NSW. The development area has a total area of approximately 7 hectares.

The development area is located generally between Alison Road to the north, Wansey Road to the east and the racecourse to the south.

Current site levels fall from approximately RL 52.0m at the south boundary with Wansey Road to RL 39.7m at the north boundary with Alison Road and to RL 31.0m at the lowest point to the south west next to the racecourse. All levels are quoted to Australian Height Datum – (AHD). The land slopes more steeply at the boundaries with Alison Road and Wansey Road, where steep batters fall from the boundary with the road to the racecourse level (approximately 1 in 3).

The site is currently developed with a timber scraping yard with external bitumen, a training track and vegetated areas with scattered trees.

4.0 Flooding

WMAwater was previously commissioned by the Australian Jockey Club to investigate flood levels for the racecourse site (Ref. 2). The report indicates that the 100 year flood level in the existing site basins is RL 29.20m. The proposed development levels are in the region of RL 31.00m which is 1.80m above the 100 year flood levels; as such the site will not be affected by flood waters and is not within the flood envelope. The proposed development will not adversely affect the existing flood storage volumes of either the upstream or downstream areas.

Randwick City Council requires habitable areas to have a minimum freeboard of 500mm above the 100-year ARI flood level. The proposed floor levels for the development buildings are above RL 33.00 and are well above the 100 year flood level.

5.0 Stormwater Drainage

5.1 Catchment Areas

The internal catchment area of the site is divided up as follows.

Land Use	Area (ha)
Building Roof	1.59
Pavements	1.42
Non-developed area	2.51
Total	**5.52**

Table 1. Catchment areas

5.2 Rainfall Intensities

Rainfall intensities have been measured and collated by the Bureau of Meteorology over many years in order to determine the statistical relationship between rainfall of a particular intensity and the frequency of its occurrence. The probability that a particular intensity might be exceeded in a storm in any one year is denoted as its *Annual Exceedance Probability* (AEP). Thus an intensity which has an AEP of 1% has a probability of 0.01 of being exceeded in any one year. This may also be considered as the intensity that might be exceeded on average once every 100 years (the inverse of 0.01). This intensity can thus also be termed as the 100-year *Average Recurrence Interval* (ARI) intensity, and the runoff generated from this rainfall would be termed the *Q100 peak runoff*.

Randwick City Council has published rainfall intensity tables in their *Private Stormwater Code*. Accordingly, design rainfall intensities will be based on the Randwick City Council table. This table is attached in the Appendix to this report.

5.3 Existing Infrastructure

In it's current state there are a number of detention basins located within the site which act as detention systems for the site and for the current upstream catchments which feed into the site. The area of the proposed development on the site has limited in ground drainage but does have a number of channels which guide the upstream overland flow across the site to the existing detention basins.

There are two existing detention basins in the area of the proposed development. The Stormwater Management Plan prepared by WMAwater (Ref 1.) describes the process of water flowing over the site. In summary, the flows from the upstream catchments surcharge onto the site along Wansey Road and High Street, the flows are guided across the site to the initial basin using a collection of channels and in ground drainage. The first basin is located to the west of the 1400m shute. Overflow from this basin then flows to the secondary basin on the south west side of the 1400m shute. Discharge from this basin feeds into council's existing drainage line in High Street.

The proposed development will be located above the existing overland flow path of the water discharging from Wansey Road and as such allowance must be made for the diversion of this water through the proposed development to the existing basins.

5.4 Points of Discharge

It is proposed that all stormwater drainage from the proposed development will be discharged into a new detention basin located in the centre of the new Bull ring. The controlled discharge from this basin will be fed into the existing site detention basin to the south west of the 1400m shute.

The in ground drainage will discharge at a new headwall located within the new proposed basin at an RL of 30.350 and the basin will have storage enough to attenuate the development discharge to flows which would be passing into the existing basin prior to the construction of the development.

The peak 100-year discharge for the development has been calculated at 3.81cumecs however flowing attenuation the discharge to the existing detention basin will be restricted to 3.23 cumecs to match that of the pre development situation.

In addition to the main piped discharge there will also be a discharge to the aquifer located below the site. Investigations have been undertaken on site to confirm the hydraulic conductivity of the existing ground material, these results have indicated that it will be possible to discharge 0.01m3/s into the aquifer. The main location for this discharge will be through the detention basin as the stormwater runoff will be detained in this location allowing time for it to infiltrate the soil.

5.5 Roof Drainage

Stormwater from roofed areas will be collected by along the building eaves. Gutters and downpipes will typically cater for the peak 1 in 20 year ARI runoff, with provision for overflow in the event of blockage or extreme rainfall events.

Roof drainage will then be directed into rainwater harvesting tanks with the overflow being discharged into the in ground drainage system.

The roofwater drainage system of gutters, downpipes and in-ground pipework will be designed in accordance with AS/NZS 3500.3.2 *National Plumbing and Drainage Part 3.2: Stormwater Drainage – Acceptable Solutions*.

5.6 Surface Drainage

Stormwater runoff from all paved areas will be directed by pavement falls towards kerbs and gutters and collected by inlet pits at regular intervals.

Runoff falling on landscaped areas will primarily filter through the sandy ground material into the ground water below, however in major storm events allowance will be made to direct the overland flows to the in ground drainage with the overflow being directed to the proposed detention basin.

In truck pavements, grates to inlet pits and trench drains will be Class D (heavy duty) as defined in AS 3996 *Metal Access Covers, Road Grates and Frames*. Grates in other areas will be class B (medium Duty) as defined in AS 3996.

In areas where horse traffic is possible all grates will be fitted with horse proof grates in order to prevent potential damage to the horse's hooves.

5.7 In-Ground Drainage

In general, the in-ground drainage system will be designed to convey runoff from their respective catchments up to the peak 20-year ARI storm event.

This meets the requirements set by Randwick City Council, as set out in their *Private Stormwater Code.*

A preliminary design of the in-ground stormwater network has been carried out and shown on drawings 10619-SCK10 and 10619-SCK11 in the Appendix to this report. It should be noted that this design will be adjusted during detailed design, at which time surface levels will be finalised and conflicts between stormwater pipelines and other services resolved.

All in-ground stormwater drainage will be designed with full hydraulic calculations, including head losses at all junctions, demonstrating that the system can convey the design storms flows. The hydraulic gradeline (HGL) will be plotted on stormwater longitudinal sections which will show all crossing services. A minimum of 150mm freeboard above the calculated HGL level will be provided at all stormwater inlets.

5.8 Overland Flows

The existing overland flows from the site discharge across the race track into the infield and the down stream detention basins. The proposed development will retain this overland flow path effectively ensuring that the discharge to the external drainage systems and associated infrastructure is retained. Drawing 10619-SKC20 in the Appendix of this report shows the existing and proposed overland flow paths across the site.

In general, stormwater runoff in excess of the capacity of the piped drainage system will be conveyed across the site by overland flow.

Randwick City Council's Design Guidelines specify that design of drainage systems must take into account runoff from storms up to the 100-year design storm. The total 100-year runoff (Q100) from all internal catchments, assuming a typical time of concentration of 6.27 minutes, has been estimated as 3.57cumecs.

Flows from both the upstream catchment and the proposed development in excess of the piped drainage capacity will be directed towards the infield of the racetrack as per the existing condition. These flows will be directed through channels in a controlled manner as to not adversely effect the proposed development.

This has been documented on drawings 10619-SCK10 and 10619-SCK11 in the Appendix to this report.

5.9 On-Site Detention

Randwick City Council requires the peak stormwater discharge from development sites to be limited using on-site detention. The requirements are set out in their Private Stormwater Code. The code states that the flow discharging from the site should be attenuated to the PSD for the site. However as the proposed development form a small section of the larger site that is Royal Randwick Racecourse it is proposed that by attenuating the discharge for the proposed development back to that of the predevelopment then the existing site detention basins will have sufficient storage to attenuate the discharge flows to levels acceptable to Randwick City Council.

To attenuate the post development flows back to the pre development flows a new detention basin is proposed for the development. This basin will be located within the infield of the new bull ring and will have sufficient capacity to attenuate all design flows up to the 20 year design storm.

As indicated previously in this report due to the materials present in the ground of the proposed development (manly Botany Sands, refer Geotechnical Report for details) it has been deemed possible to take into account discharge into the existing aquifer due to infiltration. This discharge effectively reduces the volume of stormwater runoff which has potential to discharge into the existing site drainage system and as such reduces the required detention volume required to attenuate the developments discharge flows. These infiltration rates have been previously tested on site to confirm that this allowance will be possible. The test results indicate that the discharge rates into the aquifer due to a nominal 0.2m of head (storage depth in detention basin) will be in the region of 0.01m3/s. This discharge rate has been included in the storage calculations supplied in the Appendix of this report.

Results for the on site infiltration tests undertaken are included in the appendix of this report, these test results verify the infiltration rates used in the calculation of the detention basin volume. Further testing will be undertaken at the actual location of the detention basin to confirm that these infiltration rates can be achieved.

Calculations have been supplied in the Appendix of this report which shows that the required detention basin volume is 712m3. This will be supplied by creating a 0.30m deep detention basin as indicated on drawing 10619-SKC11 in the appendix of this report. Ponding depths in the basin have been summarised below.

Design Storm ARI (years)	Detention Volume Required (m3)	Detention Basin Water Depth (m)
5	405	0.120
10	530	0.158
20	712	0.213

Table 2. Detention Basin Depths

6.0 Water Quality

The pollution control system will be required to treat the runoff from a “3-month return period” storm. (This is defined as the storm intensity that is exceeded on average 4 times per year.) Since rainfall intensity data is only available for return periods of 1 year or greater, the “3-month return period” design rainfall intensity has been derived by extrapolation of the published data on a logarithmic scale.

The specified level of treatment for the design flows (the Treatable Flow Rate) will be as follows:

- Removal of at least 95% of all particles greater than 3mm in size (eg cigarette butts or bigger).
- Settlement of suspended finer particles by filtration and/or velocity reduction.
- Removal of at least 95% of all floating oils and greases.
- Trapping of all retained materials to prevent re-mobilisation in extreme storm events.

The proposed stormwater drainage system incorporates measures for the trapping and removal of pollutants near the points of discharge. These measures consist of a combined Gross Pollutant Trap / Oil and Silt Arrestor structure, servicing the drainage line collecting runoff from paved areas and roof areas.

The proposed structure will be an Ecosol in-line units of the RSF 4000 series, or equivalent. This will be located immediately upstream of the discharge point to the proposed detention basin as indicated on 10619-SKC11 included in the Appendix of this report.

Detailed calculations have been carried out and determined the required Treatable Flow Rate (TFR) 207 litres/second for the GPT. These calculations are included in the Appendix to this report. The suggested Ecosol unit that provide TFR well in excess of this requirement is the model RSF4450.

It is important to note that, in addition to the treatment of pollutants up to the Treatable Flow Rate by the GPT, the pollutant control measures will remove trash and sediment for all return periods.

Cleaning of the pollutant control structure will be by eduction equipment (ie suction pumps) mounted on a tanker truck. The pollution control structures are located to be accessible to tanker trucks from the truck hardstand or fire access road.

The frequency of access for maintenance will depend on the rate of pollutant collections, and will be based on monitoring of the system. An initial maintenance check should be carried out 6 months after completion. Experience at other distribution centres indicates that the time between successive cleanouts could be approximately 5 years.

7.0 Erosion and Sediment Control

Erosion and sediment controls will be provided during the construction phase in accordance with Council guidelines. An Erosion and Sedimentation Control Plan has been prepared for submission with this report, and is included in the Appendix to this report. The plan includes measures such as sediment fences at the downstream edges of all disturbed areas, diversion banks to prevent water egress into the site from upstream catchments, erosion control measures at all discharge locations and a truck shaker tray at each point of access to the work area. A Sedimentation basin has been provided, sized in accordance with the guidelines in the “Blue Book” - Managing Urban Stormwater - Soils and Construction (Landcom). Calculations are included in the Appendix to this report.

Final details of Erosion and Sediment Control measures will be documented in the drawings to be prepared for the Construction Certificate.

8.0 Rainwater Harvesting

Rainwater harvesting tanks will be provided for the roof water runoff from the proposed stable buildings. These tanks have been initially sized according to the following assumptions:

- 100 horses per stable building
- Assume that each horse is washed for 5 minutes twice per day.
- Assume that the hose washing the horse discharges 20l/minute
- 2 apartments per stable for caretakers
- Assume that there are basic amenities; hence usage is 150L/day (rainwater usage only).
- Assume the stable building is hosed down for 1 hour twice per week
- Calculations model 50 years of day by day use, utilising 50 years of day by day rainfall data (18,000+ continuous days) recorded at Sydney observatory
- Each stables has approximately a 1800m^2 roof

Water usage calculations suggest that the water usage for the development will be approximately 3,500L/day.

It is proposed that the installation of two tanks per stable building, each 30,000 litres, will provide the most efficient rainwater harvesting system. Each of these tanks will be located in the centre of the horse walking rings.

The overflow from the rainwater tanks will be discharged into the proposed in ground stormwater drainage system and conveyed to the site detention basin.

APPENDIX

Calculations - Water Quality

The pollution control system will be required to treat the runoff from a “3-month return period” from all paved areas.

Contributing catchment:

Roof Catchment	1.590 ha
Pavement Catchment	1.420 ha
Total paved catchment	**3.010 ha**

Rainfall Intensity:

Rainfall intensities have been published by Randwick City Council for return periods of 1 to 100 years. Time of concentration has been calculated at 5 minutes. Extrapolation is necessary to obtain rainfall intensity for return period less than 1 year. Refer to attached graph.

Runoff Coefficient:

Runoff coefficients are derived from the runoff coefficient for paved areas for 10-year return period, multiplied by a factor for return periods greater or less than 10 years. The runoff coefficient for 40% impermeable areas for 10-year ARI is 0.65. Australian Rainfall & Runoff published runoff coefficient multipliers for return periods between 1 and 100 years. Extrapolation is necessary to obtain runoff coefficient multiplier for return period less than 1 year. Refer to attached graph.

Summary of graph results:

Return Period (ARI) Years	Rainfall Intensity for Tc = 5.0 mm/h	Runoff Coefficient Multiplier
5	168	0.95
2	134	0.85
1	106	0.80
0.25 (3 month)	**56**	**0.68**

Calculated runoff therefore becomes as follows.

For GPT:

Q = C. I. A

= 0.68 x 0.65 x 56 x 3.010 / 360

= **0.207 cumecs**.

Pollutant Trap Selection:

Refer to attached excerpts from Ecosol GPT technical specification.

GPT

For Model RSF4450 the quoted Treatable Flow Rate is in the range 0.04-0.26 cumecs, for pipe sizes between 225mm and 900mm diameter.

The required standard is 0.207 cumecs.

Thus Ecosol RSF4450 should be OK for the office GPT.

Confirmation of treatment capacity at the design levels and slopes will be sought from manufacturer prior to Construction Certificate issue.

Other pollutant traps of similar capability would be acceptable.

19th November, 2007

Our Ref: WRL 07064:WAT:L071119.doc

BVN Architecture
Level 6, 11-31 York Street
SYDNEY NSW 2000

Attention: Peter Clarke

Dear Peter,

PRELIMINARY COMMENTS - INFILTRATION RATES TO THE BOTANY AQUIFER AT RANDWICK RACE COURSE

The Water Research Laboratory (WRL) has been engaged by BVN Architecture on behalf of the Australian Jockey Club to provide expert advice on groundwater related aspects of stormwater infiltration. This information is to be used by Webb McKeown and Associates (WMcK) as part of their stormwater assessment for the Randwick Racecourse site.

PRELIMINARY COMMENTS – GROUNDWATER CONDITIONS

Groundwater conditions on the racecourse are yet to be assessed. However, an indication of likely conditions can be provided based on regional reports (eg. Timms et al. 2006), and on monitoring bores to the south (UNSW Kensington campus) and to the north (test bores near Allison Road in Centennial Park).

The aquifer is composed of clean quartz sands and underlying silty sands with peat layers with a total sediment depth of about 15 m. The depth to groundwater may be between 1 and 5 m below the ground surface. Total porosity is likely to be 0.32 to 0.38; therefore with storage of 320 to 380 mm per 1 m of sand. Note that porosity within soil horizons may be reduced by organic matter.

A recharge coefficient of 30 to 97% has been reported for grassy sandy areas in the Botany aquifer based on groundwater flow modelling, although typical values are expected to be closer to 30%. Therefore, a 100 mm rainfall event can be expected to result in ~30 mm of recharge with a watertable increase of 86 mm (assuming a porosity of 0.35). It is noted that total infiltration through the soil zone (ie. losses at the ground surface) during such an event would be somewhat greater than 30 mm, particularly with dry antecedent conditions.

PRELIMINARY COMMENTS - INFILTRATION RATES

Infiltration and recharge processes

Infiltration is the process by which water on the ground surface enters the soil. The soil zone is defined by high organic content. Soils overlie sediments or regolith material of weathered rock. A brief overview of concepts is provided to clarify terminology (Figure 1).

Once water has infiltrated the soil, it can partially fill pore space in the vadose zone (ie. the semi-saturate zone), percolate down to the watertable (ie. the saturated zone), or be lost from the vadose zone. Recharge is the portion of infiltration that reaches the watertable.

The types of permeability or hydraulic conductivity that apply to each of these processes are shown in Figure 1. It is important that the measures of infiltration rate and/or hydraulic conductivity are specific to the depth, direction and saturation status relevant to the study or design criteria.

Specific measurements of hydraulic conductivity are required for various applications or design criteria as follows:

- Surface infiltration or losses - for storm water modelling.
- Vertical percolation rates at about 2-6 m depth – for design of buried percolation chambers, for both semi-saturated and saturated conditions.
- Saturated hydraulic conductivity in the horizontal direction - for determining the rate of groundwater flow, to determine the extent and timing of watertable mounding that occurs with recharge.

Factors that control infiltration and recharge rates include, but are not limited to the following:

- Types of soil and sediment
- Backfill if present on the site
- Vegetation cover
- Slope of ground surface
- Consolidation history of ground surface
- Antecedent soil moisture
- Preferential flow paths through holes and fractures etc.

Typical infiltration rates

Table 1 provides typical infiltration rates, assuming homogenous sediments and free draining conditions.

Table 1 Typical Infiltration Rates

Soil or sediment type	Rate (mm/hr)	Rate (m/s)
Consolidated clay	0.0004	1.1E-10
Heavy clay	0.036	1.0E-08
Medium clay	3.6	1.0E-06
Sandy clay	36	1.0E-05
Deep unconfined sands	180	5.0E-05
Dune sands	1800	5.0E-04
Shallow soil over rock - min	3.6	1.0E-06
Shallow soil over rock - max	36	1.0E-05

Source: ARQ (2006)

Pond seepage

The rate of seepage from existing depressions and ponds is unknown and there is limited information for estimating the volume of leakage. It is likely that seepage occurs from the sides of the depression as the pond base is probably covered with lower hydraulic conductivity sediments. It is anticipated that seepage could be significant under typical conditions when the watertable is at a depth greater than the base of the pond. However the hydraulic gradient may reverse under some conditions causing groundwater to flow into the pond.

To estimate the rate of leakage from a depression or detention pond, the following information would be necessary:

- Relative water levels in pond and groundwater and variations over time.
- Relative elevation of base of pond compared with the watertable.
- Area and circumference of the pond and stage-volume relationships.
- Vertical and lateral hydraulic conductivity of sediments.

A pond de-watering experiment at Centennial Park including detailed groundwater monitoring was undertaken by the WRL to test the relationship between pond leakage and groundwater levels (Timms et al. 2007; Turner et al. 2002). This experiment at Kensington Pond No. 2, adjacent to Allison Road demonstrated that there was a net loss of water from the pond (ie. groundwater recharge), with groundwater discharge occurring during very low stages in the pond. Groundwater discharge was therefore responsible for maintaining pond water levels during drought periods. It was concluded that groundwater was closely connected with ponds, and that pond sediments should remain in place to help maintain pond levels.

Gross estimates of total pond outflow were derived from the field data. During high pond water levels, groundwater flow velocities away from the pond were in the order of 0.5 m/day, based on measured horizontal hydraulic conductivity and hydraulic head differentials between the pond level and groundwater. Total leakage outflow from the pond was about 520 m^3/day during high pond levels were calculated based on these groundwater flow rates and assuming a 3 m saturated depth and ~300 m pond circumference. The leakage rate at high pond levels was about 7% of the total pond volume of 6,500 m^3 (full level, maximum depth 3.7). Therefore, ponds levels would fall significantly within days to weeks if there were no stormwater inflows or outflows.

Measurements of infiltration rates

No suitable site specific measurements of infiltration rate are available for stormwater modelling at the Randwick Racecourse. However, measurements on nearby UNSW Village Green and from similar coastal sandy aquifers in NSW could provide a rough guide. It is worth noting that field measurements of infiltration rates tend to be overestimated for sandy soils and underestimated for clayey soils (ARQ, 2006). Thus, field measurements for sandy soils tend to result in infiltration system designs that are “too small”.

Infiltration rates reported for coastal sand aquifers in NSW range considerably. Surface infiltration rates averaging about 60 mm/hour (1.7×10^{-5} m/s) have recently been observed by WRL in a sandy aquifer on a hillslope in Yamba. By contrast surface infiltration rates of 57 to 1252 mm/hour (1.6×10^{-5} to 3.5×10^{-4} m/s) have been reported at various points in a sandy catchment near Nelson Bay.

Surface infiltration measurements to the south of the Randwick Racecourse were reported by Dickson (2004) as part of design work for the percolation system at UNSW. The grass was covered with a geotextile membrane to prevent scour. Twin sheet metal dams (1.5 m^2 and 0.7 m^2 square with an inner area 0.5 m^2) were hammered 100 mm into the soil surface, and filled with water.

Initial infiltration rates by this method at two sites (T1 & T2) were ~83 mm/hour, with maximum rates of 95 mm/hour. Infiltration rates then declined rapidly at the T1 site and slowly at the T2 sites. A steady state rate of 5.9 mm/hour (1.6×10^{-6} m/s) was attained at T1 after 305 minutes of infiltration. At this site, a 50% decline in infiltration rate was observed within 5-10 minutes. However, at the T2 site the infiltration rate was 65 mm/hour (1.8×10^{-5} m/s) at 43 minutes, but was still declining when observations ceased.

These surface infiltration measurements are 1 to 2 orders of magnitude lower than expected values for sandy soil, although the reported measurement values are considered uncertain. The difference between the sites indicates heterogeneous subsurface conditions below the Village Green, but may also be subject to measurement errors. Duplicate measurements at each site were apparently not attempted.

Note also that continuous infiltration rates may be about 1 order of magnitude less than initial infiltration rates. Lower long term infiltration rates are consistent with the relatively small proportion of rainfall that recharges groundwater during an event (eg. 30 mm from a 100 mm event as discussed above).

Stormwater modelling and infiltration

It is noted that stormwater modelling in the Green Square area (Webb McKeown, 2007) used initial loss values of 5 mm for grassy areas for MIKE-Storm modelling. A soil type with moderate infiltration rate that was well-drained was selected with antecedent moisture of over 25 mm rainfall in the preceding 5 days. Model sensitivity testing indicated that peak flood levels were most dependent on rainfall, initial losses and antecedent moisture of all the model parameters. For example, increasing initial losses and changing antecedent moisture could reduce peak overland flows by up to 60%.

RECOMMENDATIONS

Initial surface infiltration rates on flat grassy areas of the Randwick Racecourse are likely to be about 36 mm/hour (1.0×10^{-5} m/s). However a range of infiltration rates that are at least an order of magnitude lower and higher than this value are recommended for stormwater modelling. Long term continuous infiltration rates are likely to be an order of magnitude lower than initial rates. Limited field measurements have indicated a 50% decline in surface infiltration rates within 5-10 minutes.

Site specific measurements using an appropriate methodology and testing duration are recommended to verify and refine these estimates.

Thank you for the opportunity to provide this preliminary information. Please do not hesitate to contact Wendy Timms on ph. 9949 4488 ext. 253, or myself should you wish to discuss or clarify any matters.

SUMMARY AND RECOMMENDATIONS

In summary, a range of infiltration rates may be applied for stormwater management as follows:

* Initial surface infiltration rates on flat grassy areas of the Randwick Racecourse are likely to be about 36 mm/hour (10-5 m/s). However a range of infiltration rates at least an order of magnitude lower and higher (10-6 to 10-4 m/s) are recommended for stormwater modelling.
* Steady state infiltration rates after major rainfall events are likely to be an order of magnitude lower than initial rates. A range of 0.6 to 60 mm/hour (1.7×10-7 to 1.7×10-5 m/s) is recommended for adoption, but is based on only one field measurement of 5.9 mm/hour during a double walled infiltration test,.
* The long term average infiltration rate into a grassy sandy surface is unknown, but is likely to be between 30 and 70% of rainfall. Surface infiltration is greater than the long term average recharge of 30% to the sandy grassland areas of Botany aquifer. Runoff (ie. no infiltration) occurs when rainfall intensity exceeds the infiltration capacity and the ground is saturated.
* Maximum seepage rates of 100-1000 mm/hour may occur during periods of high water level through a pond that maximises infiltration rates (eg. leaky sides and leaky base). Careful

design, construction and maintenance of such a pond would be required to achieve high infiltration performance.

It is noted that these gross estimates of infiltration rates are subject to considerable uncertainty and very limited data. Site specific measurements using an appropriate methodology and testing duration are recommended to verify and refine estimated infiltration rates. Furthermore, the capacity of the aquifer to receive additional stormwater infiltration would require a detailed groundwater assessment, possibly including groundwater flow modelling of the area.

Yours sincerely,

Brett Miller
Manager

cc:
Donna Hughes, Ben Noble
Webb McKeown & Associates
Level 2, 160 Clarence Street
Sydney NSW 2000

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Randwick City Council - OSD Mass Curve Analysis

Site Location:	Royal Randwick Race Course
Job Number:	10619

PRE-DEVELOPMENT ANALYSIS

Design Storm:	20	Year	C10	0.648629348
Site Area:	55200	m2		
Impervious Area:	8100	m2		
% Imp:	14.67	%		
Cy:	0.68624985			

Duration (mm)	Intensity (mm/hr)	Discharge Rate (L/s)	Volume (m3)
5	213	2241.29	672.39
6	200	2104.50	757.62
10	167	1757.26	1054.35
20	127	1336.36	1603.63
30	106	1115.38	2007.69
40	91.5	962.81	2310.74
50	81	852.32	2556.97
60	72.9	767.09	2761.52
70	66.6	700.80	2943.35
120	47.2	496.66	3575.97
180	35.9	377.76	4079.78
360	22.2	233.60	5045.75
720	13.9	146.26	6318.55
1440	9	94.70	8182.29
2880	5.81	61.14	10564.25

53

Design Storm:	20	Year	C10	0.766042391
Site Area:	55200	m2		
Impervious Area:	30100	m2		
% Imp:	54.53	%		
Cy:	0.81047285			
Infiltration Flow	0.01	m3/s		

Duration (mm)	Appendix B Intensity (mm/hr)	Discharge Rate (L/s)	Volume (m3)	Revised Discharge Rate (L/s)	Revised Volume (m3)	OSD Volume (m3)
5	213	2647.00	794.10	2637.00	791.10	118.71
6	200	2485.45	894.76	2475.45	891.16	133.54
10	167	2075.35	1245.21	2065.35	1239.21	184.86
20	127	1578.26	1893.91	1568.26	1881.91	278.28
30	106	1317.29	2371.12	1307.29	2353.12	345.43
40	91.5	1137.09	2729.02	1127.09	2705.02	394.28
50	81	1006.61	3019.82	996.61	2989.82	432.85
60	72.9	905.95	3261.41	895.95	3225.41	463.88
70	66.6	827.65	3476.15	817.65	3434.15	490.80
120	47.2	586.57	4223.28	576.57	4151.28	575.31
180	35.9	446.14	4818.29	436.14	4710.29	630.51
360	22.2	275.88	5959.12	265.88	5743.12	697.37
720	13.9	172.74	7462.32	162.74	7030.32	711.77
1440	9	111.85	9663.43	101.85	8799.43	617.14
2880	5.81	72.20	12476.56	62.20	10748.56	184.31

Randwick City Council - OSD Mass Curve Analysis

Site Location:	Royal Randwick Race Course
Job Number:	10619

PRE-DEVELOPMENT ANALYSIS

Design Storm:	10	Year	C10	0.648629348
Site Area:	55200	m2		
Impervious Area:	8100	m2		
% Imp:	14.67	%		
Cy:	0.648629348			

Duration (mm)	Intensity (mm/hr)	Discharge Rate (L/s)	Volume (m3)	
5	187	1859.84	557.95	
6	176	1750.43	630.16	
10	146	1452.06	871.24	
20	111	1103.97	1324.76	
30	91.6	911.02	1639.84	
40	79.1	786.70	1888.08	
50	70	696.20	2088.59	
60	63	626.58	2255.67	
70	57.4	570.88	2397.70	
120	40.7	404.79	2914.47	
180	30.9	307.32	3319.06	
360	19.2	190.96	4124.66	
720	12	119.35	5155.82	
1440	7.79	77.48	6693.98	
2880	5.03	50.03	8644.60	53

Design Storm:	10	Year	C10	0.766042391
Site Area:	55200	m2		
Impervious Area:	30100	m2		
% Imp:	54.53	%		
Cy:	0.766042391			
Infiltration Flow	0.01	m3/s		

Duration (mm)	Appendix B Intensity (mm/hr)	Discharge Rate (L/s)	Volume (m3)	Revised Discharge Rate (L/s)	Revised Volume (m3)	OSD Volume (m3)
5	187	2196.50	658.95	2186.50	655.95	98.00
6	176	2067.29	744.23	2057.29	740.63	110.47
10	146	1714.91	1028.95	1704.91	1022.95	151.71
20	111	1303.80	1564.56	1293.80	1552.56	227.80
30	91.6	1075.93	1936.68	1065.93	1918.68	278.84
40	79.1	929.11	2229.86	919.11	2205.86	317.78
50	70	822.22	2466.66	812.22	2436.66	348.07
60	63	740.00	2663.99	730.00	2627.99	372.32
70	57.4	674.22	2831.72	664.22	2789.72	392.02
120	40.7	478.06	3442.04	468.06	3370.04	455.57
180	30.9	362.95	3919.87	352.95	3811.87	492.81
360	19.2	225.52	4871.29	215.52	4655.29	530.63
720	12	140.95	6089.12	130.95	5657.12	501.29
1440	7.79	91.50	7905.70	81.50	7041.70	347.73
2880	5.03	59.08	10209.42	49.08	8481.42	-163.18

Randwick City Council - OSD Mass Curve Analysis

Site Location:	Royal Randwick Race Course
Job Number:	10619

PRE-DEVELOPMENT ANALYSIS

Design Storm:	5	Year	C10	0.648629348
Site Area:	55200	m2		
Impervious Area:	8100	m2		
% Imp:	14.67	%		
Cy:	0.61619788			

Duration (mm)	Intensity (mm/hr)	Discharge Rate (L/s)	Volume (m3)
5	168	1587.33	476.20
6	158	1492.84	537.42
10	131	1237.74	742.64
20	98.2	927.83	1113.40
30	81	765.32	1377.57
40	69.8	659.50	1582.79
50	61.6	582.02	1746.06
60	55.4	523.44	1884.38
70	50.6	478.09	2007.97
120	35.7	337.31	2428.61
180	27.1	256.05	2765.35
360	16.8	158.73	3428.62
720	10.6	100.15	4326.60
1440	6.87	64.91	5608.25
2880	4.44	41.95	7249.09

Design Storm:	5	Year	C10	0.766042391
Site Area:	55200	m2		
Impervious Area:	30100	m2		
% Imp:	54.53	%		
Cy:	0.727740272			
Infiltration Flow	0.01	m3/s		

Duration (mm)	Appendix B Intensity (mm/hr)	Discharge Rate (L/s)	Volume (m3)	Revised Discharge Rate (L/s)	Revised Volume (m3)	OSD Volume (m3)
5	168	1874.66	562.40	1864.66	559.40	83.20
6	158	1763.07	634.71	1753.07	631.11	93.68
10	131	1461.79	877.07	1451.79	871.07	128.43
20	98.2	1095.78	1314.94	1085.78	1302.94	189.54
30	81	903.85	1626.94	893.85	1608.94	231.36
40	69.8	778.88	1869.30	768.88	1845.30	262.51
50	61.6	687.37	2062.12	677.37	2032.12	286.07
60	55.4	618.19	2225.49	608.19	2189.49	305.11
70	50.6	564.63	2371.44	554.63	2329.44	321.48
120	35.7	398.37	2868.23	388.37	2796.23	367.62
180	27.1	302.40	3265.92	292.40	3157.92	392.58
360	16.8	187.47	4049.26	177.47	3833.26	404.64
720	10.6	118.28	5109.78	108.28	4677.78	351.19
1440	6.87	76.66	6623.44	66.66	5759.44	151.19
2880	4.44	49.54	8561.30	39.54	6833.30	-415.79

APPENDIX B

$\ln(I) = a + b_*[\ln(T)] + c_*[(\ln(T)]^2 + d_*[(\ln(T)]^3 + e_*[(\ln(T)]^4 + f_*[(\ln(T)]^5 + g_*[(\ln(T)]^6$

I = Intensity in millimetres per hourT = Time in hours

RETURN PERIOD YEARS	A	B	C	D	E	F	G
1	3.4891	-0.6061	-0.0526	0.00718	0.002790	-0.0001436	-0.0000884
2	3.7473	-0.6032	-0.0551	0.00755	0.002935	-0.0002021	-0.0000826
5	4.0142	-0.5942	-0.0621	0.00794	0.003458	-0.0002933	-0.0000816
10	4.1427	-0.5893	-0.0654	0.00801	0.003704	-0.0003259	-0.0000829
20	4.2896	-0.5856	-0.0686	0.00820	0.003947	-0.0003676	-0.0000827
50	4.4550	-0.5811	-0.720	0.00836	0.004209	-0.0004102	-0.0000828
100	4.5648	-0.5779	-0.0742	0.00831	0.004394	-0.0004228	-0.0000852

RAINFALL INTENSITY IN MM/HR FOR VARIOUS DURATIONS AND STORM RECURRANCE INTERVALS

		STORM RECURRANCE INTERVAL						
min	DURATION (HOURS)	1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
5	0.083	106.	134.	168.	187.	213.	246.	271.
6	0.100	98.8	126.	158.	176.	200.	231.	254.
10	0.167	80.9	103.	131.	146.	167.	193.	214.
20	0.333	59.5	76.5	98.2	111.	127.	149.	165.
30	0.500	48.5	62.6	81.0	91.6	106.	124.	138.
40	0.666	41.5	53.7	69.8	79.1	91.5	107.	120.
50	0.833	36.5	47.3	61.6	70.0	81.0	95.8	106.
60	1.000	32.8	42.4	55.4	63.0	72.9	86.1	96.0
70	1.166	29.8	38.4	50.6	57.4	66.6	78.6	87.7
120	2.000	21.0	27.3	35.7	40.7	47.2	55.8	62.3
180	3.000	16.0	20.7	27.1	30.9	35.9	42.4	47.3
360	6.000	9.96	12.9	16.8	19.2	22.2	26.1	29.2
720	12.000	6.30	8.14	10.6	12.0	13.9	16.4	18.3
1440	24.000	4.09	5.29	6.87	7.79	9.00	10.6	11.8
2880	48.000	2.65	3.42	4.44	5.03	5.81	6.83	7.61
4320	72.000	1.98	2.55	3.31	3.74	4.31	5.06	5.64

Royal Randwick _ Sed Basin Design	DESIGN	PAGE OF
	JOB No.	DATE

Designed to - Soils & Construction (LANDCOM)

Capacity of Basin for C Type Soils.

Design Storm Event = 3month ARI

Determining 3 Month Design Flows

$Q = (C_{0.25} \times {}^{t}I_{0.25} \times A) / 360$

where $C_{0.25}$ = Coefficient of runoff for 3month ARI (unitless)
${}^{t}I_{0.25}$ = Rainfall Intensity for 3 month ARI, $t$min storm duration (mm/hr)
A = Site Area. (Ha)

Co-Efficient of Runoff ⇒ Refer to graph showing plot of runoff coefficient

$C_{0.25} = f_{0.25} \times C_{10}$

$= 0.68 \times 0.65$
$= \underline{0.442}$

$C_{10} = 0.65$.
with ${}^{10}I_{1} = 63$mm/hr
$f = 0.0$

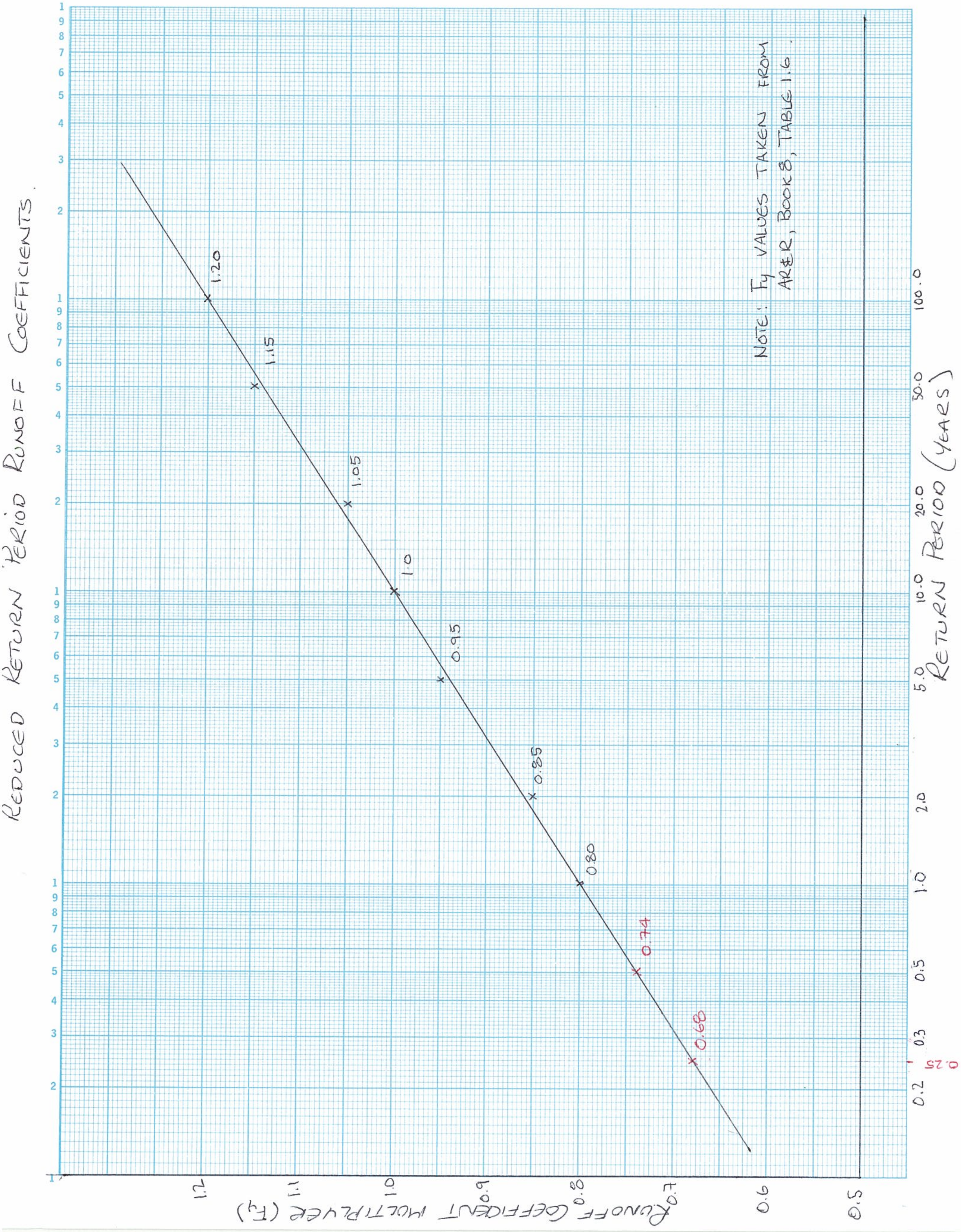
$f_{0.25}$ from graph
$f_{0.25} = 0.68$

Critical Storm Time of Concentration

From AR&R: $t = 6.94 (L.n^{*})^{0.6} / I^{0.4} . S^{0.30}$

where L = flow path length = 340m
n^{*} = roughness coefficient = ~~0.053~~ 0.010 (AR&R, Book 8, Table 1.4) (Bare Sand)
$I^{0.4}$ = Rainfall Intensity
S = slope = 0.086 m/m

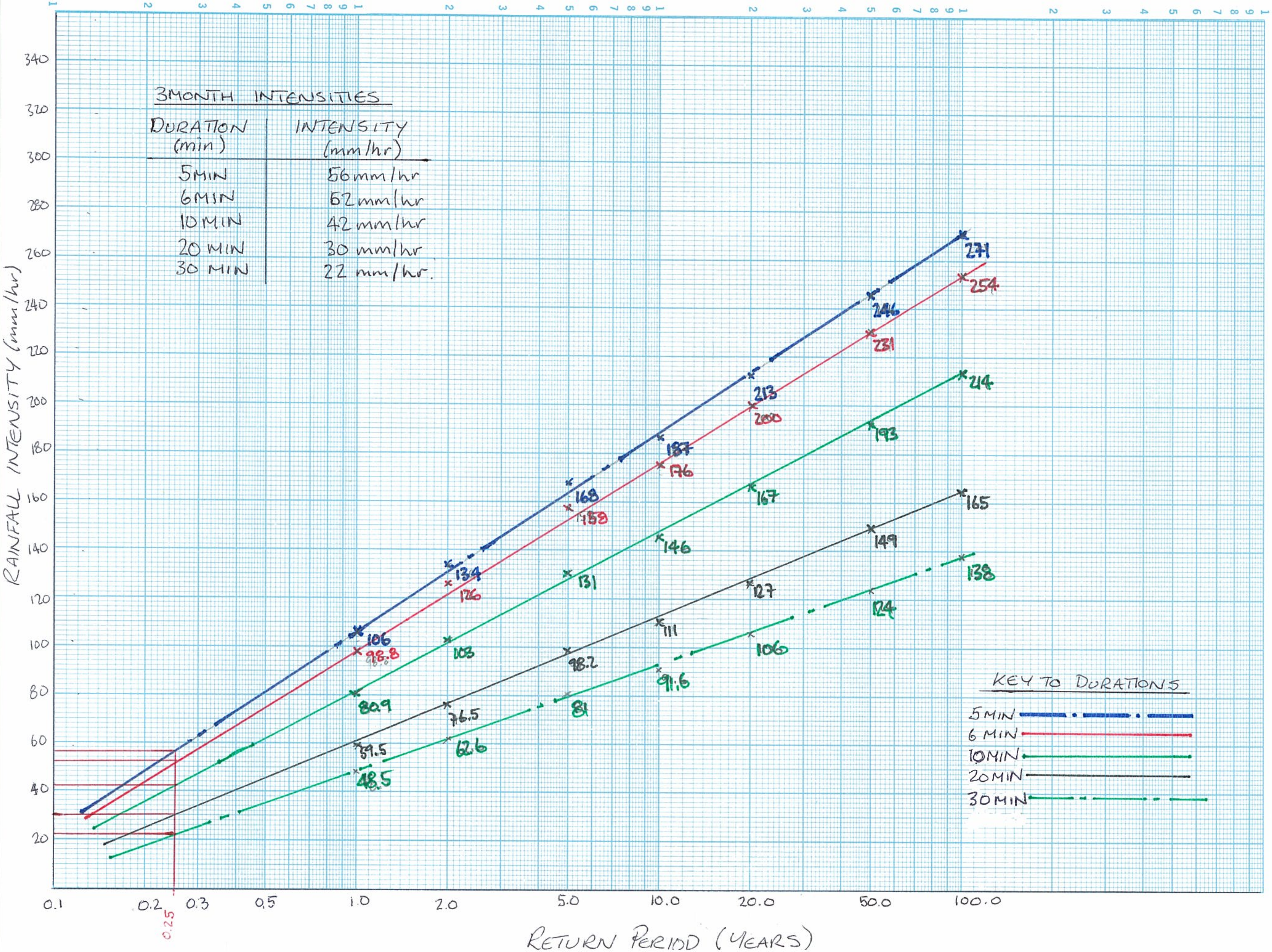
$t = 6.94(340 \times \cancel{0.053}\ 0.01)^{0.6} / I^{0.4} \times 0.086^{0.3}$
$14.46 = \cancel{39.34} / 0.479\ I^{0.4}$
$= \cancel{82.13}\ I^{0.4}$
$30.19\ I^{0.4}$


REDUCED RETURN PERIOD RUNOFF COEFFICIENTS.
NOTE: F_y VALUES TAKEN FROM AR&R, BOOK 8, TABLE 1.6.
RETURN PERIOD (YEARS)
100.0
50.0
20.0
10.0
5.0
2.0
1.0
0.5
0.3
0.25
0.2
RUNOFF COEFFICIENT MULTIPLIER (F_y)
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
1.20
1.15
1.05
1.0
0.95
0.85
0.80
0.74
0.68

GORMACK GRAPH PAPERS : CHRISTCHURCH N.Z. D411Y Log 4 cycle x mm

REDUCED RETURN PERIOD RAINFALL INTENSITIES

PROJECT - RANDWICK STABLES



3MONTH INTENSITIES

DURATION (min)	INTENSITY (mm/hr)
5MIN	56mm/hr
6MIN	52mm/hr
10MIN	42mm/hr
20 MIN	30 mm/hr
30 MIN	22 mm/hr.

ROBERT BIRD GROUP

RBP-DOC/2.9

Royal Randwick - Sed Basin Design	DESIGN	PAGE OF
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Rainfall Intensity Conversion

For 3 month ARI :

Duration (min)	Intensity (mm/hr)	$t . I^{0.4}$
5	56	25.02
6	52	29.14
10	42	44.60
20	30	77.96
30	22	103.30

for $t = \cancel{82.13}\ 30.19\ I^{0.4}$, t is between ~~20min~~ 6min & ~~30min~~ 10min

by iteration : t = ~~21.65~~ 6.27min mins.

Critical time of concentration = ~~21.65~~ 6.27min mins.

For purposes of calcs t_c = ~~20~~ 6 mins.

Rainfall Intensity = $^{\cancel{20}\ 6}I_{0.25}$ = ~~30~~ 52 mm/hr (chart).

Site Area = 67,290 m² ≐ 6.73 Ha.

Design Flow $Q_{0.25}$ = $(0.442 \times \cancel{30}\ 52 \times 6.73) / 360$

= ~~0.248 m³/s~~

0.430 m³/s.

Basin Surface Area

1) 4100 m² / m³/s = Surface Area

4100 × ~~0.248~~ 0.430 = ~~1,016 m²~~ 1763 m² ∴ assuming particles = 0.02 mm.

Length : width ratio

Length : width ratio to be 3:1 or greater

Using goal seek on EXCEL ⇒ Length = ~~55.21m~~ 72.73m

Width = ~~18.40m~~ 24.24m. } 3:1

Royal Randwick - Sed Basin Design	DESIGN	PAGE OF
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Settling Zone Depth.

Cross Sectional Flow Area $< 0.07\,m/s$.
for 1 year ARI flow.

1 Year Design Flow $= Q_1 = C_1 \times {}^6I_1 \times A$

where $C_1 = F_Y C_{10}$, ${}^6I_1 = 98.8\,mm/hr$
$= 0.8 \times 0.65$
$= 0.52$ $A = 6.73\,Ha$

$Q_1 = (0.52 \times 98.8 \times 6.73)/360$
$= \underline{0.961\,m^3/s}$

Flow Area $= Q_1 / A$ 0.961 / 0.07 = A m^3/s / m/s / m
$0.07A = 0.961 / A$
$0.961 / 0.07 = A$
$13.73\,m^2 = A$

Basin width = 24.240m

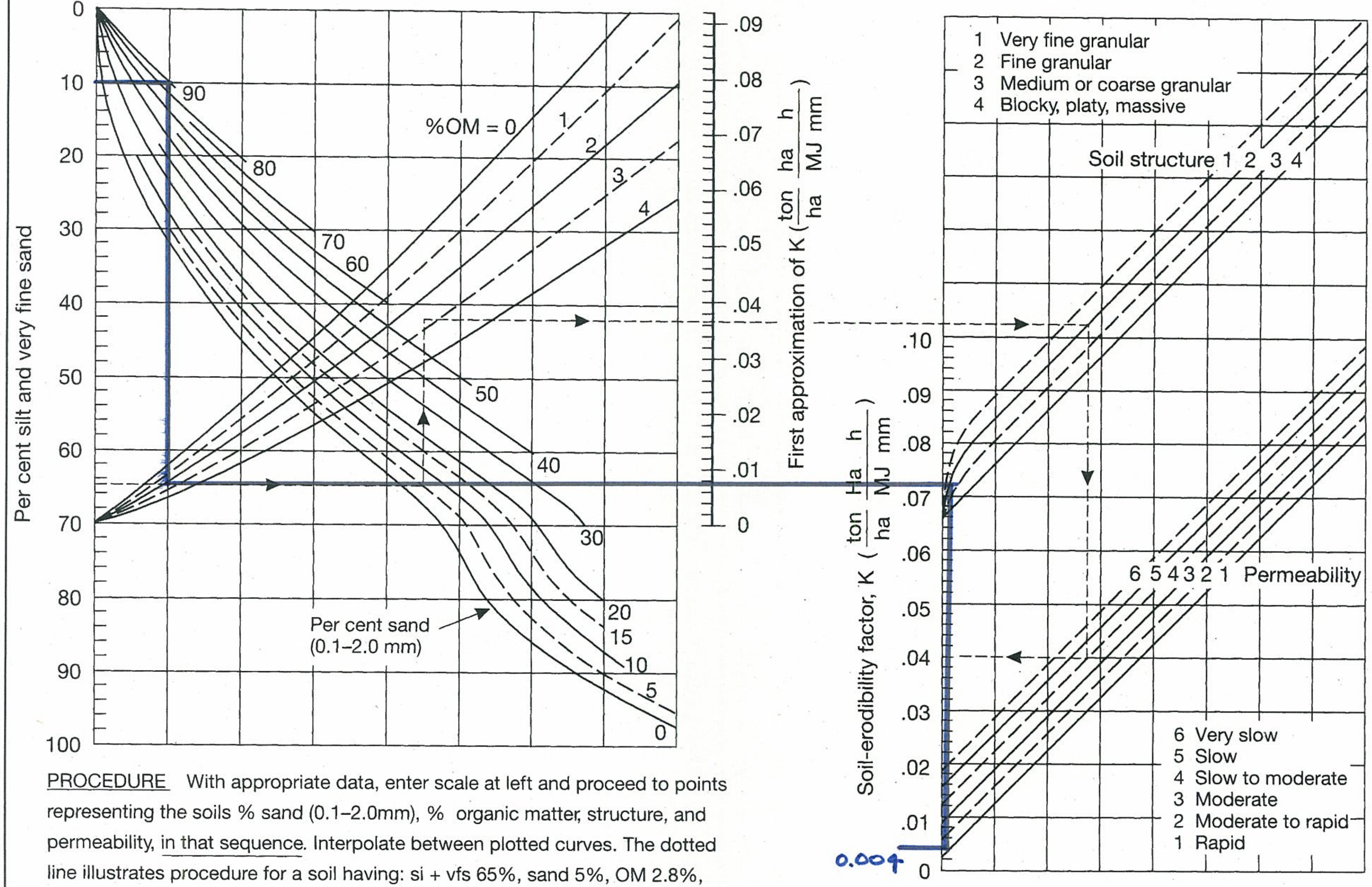
Depth $= 13.73\,m^2 / 24.24\,m$
$= \underline{0.566m} < 0.60m$ ∴ min depth $\underline{0.60m}$

Rainfall Erosivity Factor $R = 164.74\,(1.1177)^S S^{0.6444}$

where S = 2 year ARI, 6 hour event = 12.9mm/hr RCC.

$R = 164.74\,(1.1177)^{12.9} \times 12.9^{0.6444}$
$= 3596$

Refering to Section 4, figure 4.6, site has a high erosion risk. "Blue Book"



PROCEDURE With appropriate data, enter scale at left and proceed to points representing the soils % sand (0.1–2.0mm), % organic matter, structure, and permeability, in that sequence. Interpolate between plotted curves. The dotted line illustrates procedure for a soil having: si + vfs 65%, sand 5%, OM 2.8%, structure 2, permeability 4. Solution: $K = 0.04 \left(\frac{ton}{ha} \; \frac{ha}{MJ} \; \frac{h}{mm}\right)$.

Figure A3 Soil erodibility nomograph in SI units (Foster et al., 1981)

Royal Randwick – Sed Basin Design	DESIGN	PAGE OF
	JOB No.	DATE

Sediment Zone Sizing

$$\text{Sediment Zone} = \frac{0.17 \times A\,(R \times K \times LS \times 1.3 \times 1.0)}{1.3}$$

where A = 6.73 Ha
R = 3596
K = Soil Erodability Factor
LS = Slope Length / Gradient Factor (80m)

Soil Erodability

Texture grading – Fine Sand
Organic Matter – 2%
Soil Structure – 2
Profile Permeability – 1

All assumed.

Refer chart => K = 0.004

Slope Length / Gradient Factor

Table A1 – Appendix A — "Blue Book"

Slope Gradient = 8.6%

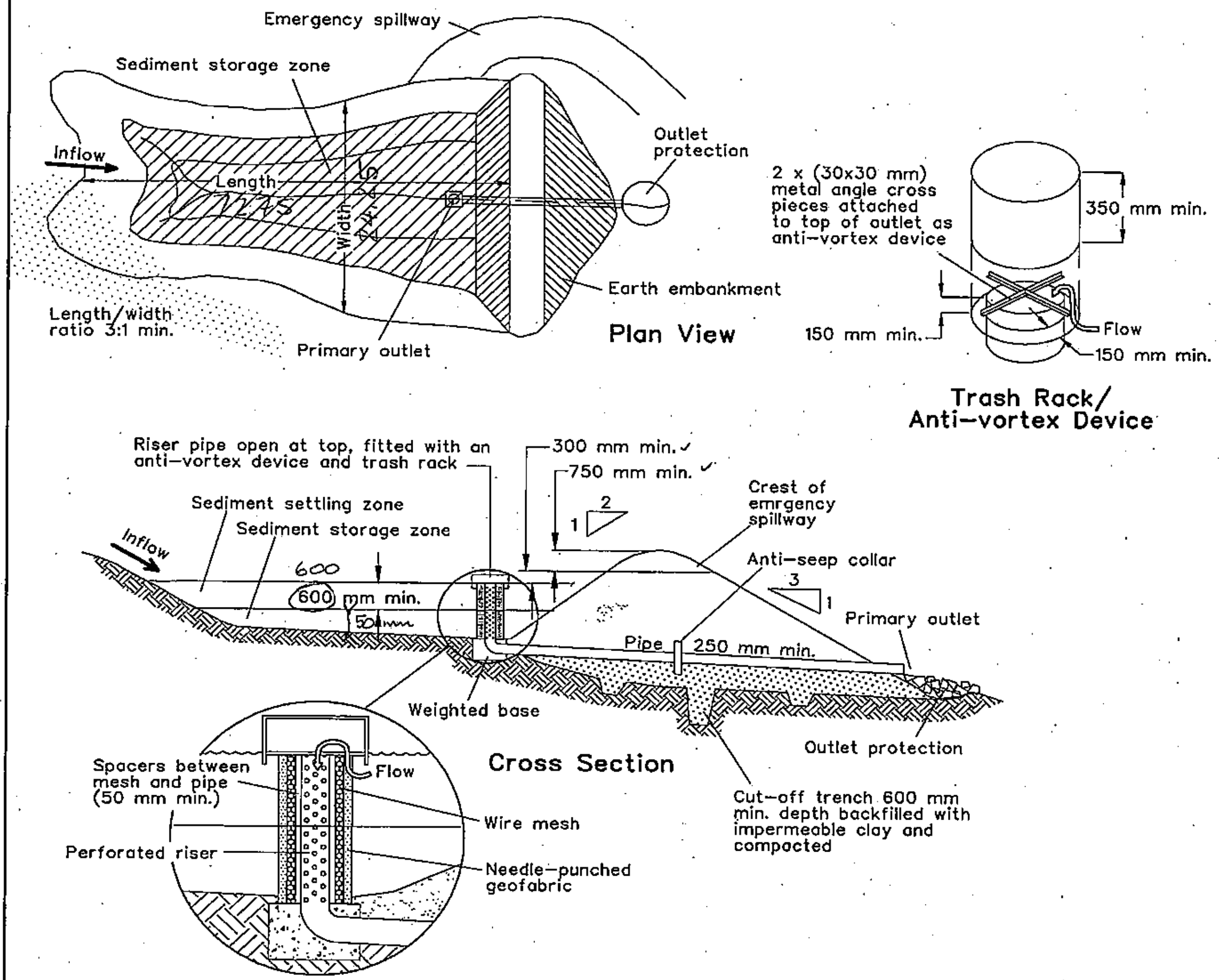
LS-Factor = ~~2.05~~ 2.28

$$\text{Sediment Zone} = \frac{0.17 \times 6.73\,(3596 \times 0.004 \times 2.28 \times 1.3 \times 1.0)}{1.3}$$

$= 37.52\ m^3$

Sediment Depth = 37.52 / 1763
= 0.021 m (20mm)

Total Basin Dimensions = 72.73 × 24.24 × 0.65m (1,146 m^3) vol.



Construction Notes

1. Remove all vegetation and topsoil from under the dam wall and from within the storage area.
2. Form a cut off trench under the centreline of the embankment 600 mm deep and 1,200 mm wide, extending to a point on the watercourse wall above the riser sill level.
3. Maintain the trench free of water and recompact the materials with equipment as specified in the SWMP to 95 per cent Standard Proctor Density.
4. Select fill according to the SWMP that is free from roots, wood, rock, large stone or foreign material.
5. Prepare the site under the embankment by ripping to at least 100 mm to help bond the compacted fill to the existing substrate.
6. Spread the fill in 100 mm to 150 mm layers and compact it at optimum moisture content following the SWMP.
7. Install the pipe outlet with seepage collars as specified in the SWMP and Standard Drawing 6-3b.
8. Form batter grades at 2(H):1(V) upstream and 3(H):1(V) downstream or as specified in the SWMP

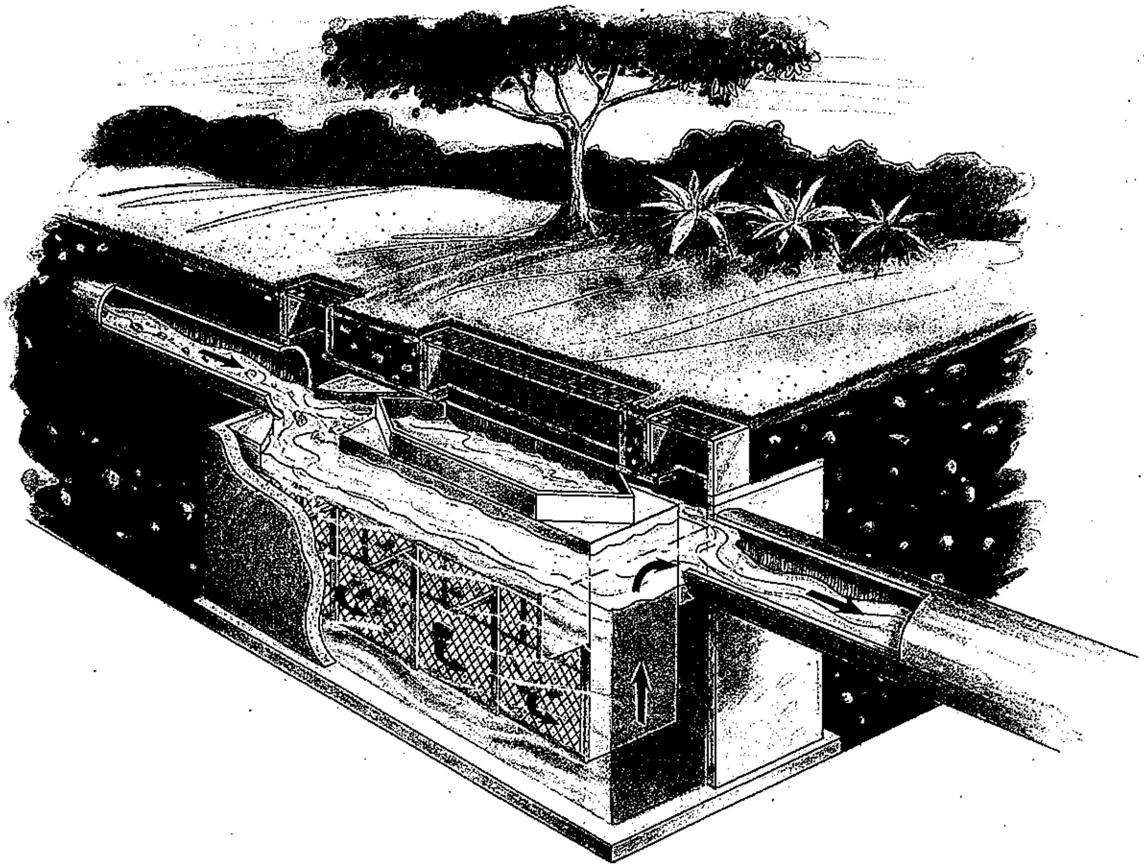
EARTH BASIN - DRY

(APPLIES TO 'TYPE C' SOILS ONLY)

SD 6-3

Ecosol™ RSF 4000

Solid Pollutant Filter/Oil and Grease Arrester



KEY FEATURES

Effective Pollutant and Litter Retention

- ☑ Captures more than 98% of gross pollutants > 3mm
- ☑ Collects up to 97% free oil and grease at design flow
- ☑ Captures more than 98% of sediment > 0.211mm
- ☑ No remobilisation or overtopping of captured pollutants
- ☑ Designed & managed hydraulics eliminates blockage risk
- ☑ Containment by design - small, medium, or large capacity

Tested and Proven Fail-Safe Overflow System

- ☑ Patented hydraulically-driven barrier
- ☑ Minimal head/hydraulic loss
- ☑ Unrestricted flow in extreme (flood) conditions
- ☑ Operates effectively in all flows and gradients
- ☑ Independently tested at a NATA-approved facility
- ☑ Meets EPA and ANZECC guidelines

Cost-Effective Maintenance

- ☑ Easily cleaned by vacuum
- ☑ Easy access to capture silo for cleaning & maintenance
- ☑ Pollutants are not handled during cleaning
- ☑ No risk to public safety and health
- ☑ Maintenance procedures within OHS&W guidelines
- ☑ De-watering facility removes all solids during cleaning
- ☑ Reduced costs as only solids transported to waste facility

Cost-Effective Design and Installation

- ☑ Simple design with no moving parts
- ☑ Safe installation procedures minimise public risk
- ☑ Small surface footprint with minimal aesthetic impact
- ☑ Lockable lids prevent access by the public
- ☑ Utilises space already used by the existing stormwater pipe
- ☑ Compact design & shallow depth reduces installation costs
- ☑ Made from durable and corrosive-resistant materials

Ecosol™
WASTEWATER FILTRATION SYSTEMS

Ecosol™ RSF 4000

Solid Pollutant Filter/Oil and Grease Arrester

The Ecosol **RSF 4000** is designed for use in stormwater drains. Incoming flows enter the capture silo and pass through the filtration mesh located below invert level on both sides of the silo before entering the clean chamber and then exiting the unit. The **RSF 4000** unit separates, collects, and retains more than 98% of gross pollutants down to at least 3mm, with a significant part of the collected pollutants smaller than 3mm. Independent tests show that the **RSF 4000** collects more than 98% of sediments with a median diameter greater than 0.211mm. Free oils and grease are retained in the outer channels below invert level by the use of two vertical baffles.

The key to the **RSF 4000** success is the design that forces a proportion of the filtered water back upstream along the by-pass channels and against the main directional flow. As this water meets the flows entering the inlet to the capture silo, a unique, patented hydraulically-driven barrier is created ensuring all flows up to the Treatable Flow Rate (TFR) are directed into the capture silo, thereby enhancing considerably the unit's capture efficiency.

As flows of greater magnitude enter the unit, the hydraulic barrier gradually breaks down and in major pipe discharges allows the excess flows to by-pass, without remobilising captured pollutants. However, the unit will continue to collect and filter flows at least equivalent to the TFR during these conditions.

When the capture silo is full, the water cannot pass through the mesh and the hydraulically-driven barrier can no longer be formed. Concurrently, the pollutants form a barrier across the mouth of the filtration unit, directing the incoming water into the two overflow by-pass channels, effectively eliminating the risk of flooding.

PERFORMANCE SPECIFICATIONS

Ecosol Unit Code	Inlet/Outlet Pipe Diameter[1]	Range of Treatable Flow Rates (TFRs)[2] (m^3/s)	Approximate External Dimensions (LxWxD from invert)[3] (mm)	HOLDING CAPACITIES		
				Solid Pollutants m^3	Free Oil and Grease Litres	Water Litres
RSF 4200	Up to 375mm	**Up to 0.051**	2200 x 900 x 750	0.23	268	667
RSF 4300	150 to 600mm	**0.03 - 0.12**	2700 x 1350 x 750	0.32	469	1,181
RSF 4450	225 to 900mm	**0.04 - 0.26**	3600 x 1650 x 1050	1.03	1,347	3,348
RSF 4600	300 to 1200mm	**0.13 - 0.47**	4580 x 1950 x 1350	2.43	2,994	7,211
RSF 4750	450 to 1350mm	**0.24 - 0.73**	5500 x 2250 x 1650	4.83	5,711	13,608
RSF 4900	600 to 1650mm	**0.42 - 1.05**	6570 x 2600 x 1950	8.30	9,576	22,768
RSF 41050	750 to 1800mm	**0.66 - 1.43**	7550 x 2900 x 2250	13.11	14,850	35,262
RSF 41200	900 to 2100mm	**0.96 - 1.87**	8250 x 3200 x 2550	19.52	21,793	51,698
RSF 41350	1050 to 2400mm	**1.26 - 2.37**	9500 x 3500 x 2850	27.70	30,578	72,495
RSF 41500	1200 to 2400mm	**1.68 - 2.93**	10470 x 3800 x 3150	37.94	41,491	98,317
RSF 41800	1350 to 2400mm	**2.50 - 4.21**	12420 x 4400 x 3950	65.33	70,452	166,836

Notes
[1] The unit can be sized to suit almost any type of pipe or box culvert
[2] The TFR varies dependent on the size and slope of the outlet pipe
[3] Unit dimensions can vary depending on vehicle load requirements

Ecosol also offers a comprehensive cleaning and maintenance service

Ecosol™
WASTEWATER FILTRATION SYSTEMS

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