Appendix L – Water Management Plan

Arafura Resources Limited

Nolans Project

Water Management Plan

March 2016

Document Status

Version	Author	Reviewer	Approved by	Date	Status

Amendments

Section	Details

Audit Summary

Date	Auditor	Details

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Abbreviations

Term	Description
ACM	Acid Consuming Material
AHD	Australia Height Datum
AMD	Acid and Metalliferous Drainage
AMDMP	Acid and Metalliferous Drainage Management Plan
ANC	Acid Neutralising Capacity
ANCOLD	Australian National Committee on Large Dams
ANZECC	Australian and New Zealand Environment and Conservation Council
ARI	Annual Recurrence Interval
ARMCANZ	Agriculture and Resource Management Council of Australia and New Zealand
ASLP	Australian Standard Leaching Procedure
AST	Above Ground Storage Tanks
BaP	Benzo[a]pyrene
BFD	Blind Field Duplicate
BOM	Bureau of Meteorology
BTEX	Benzene, Toluene, Ethylbenzene and Xylenes
CSM	Conceptual Site Model
DLRM	Department of Land and Resource Management
DME	NT Department of Mines and Energy
EIS	Environmental Impact Statement
EMP	Environmental Management Plan
EP	Evaporation Pond
ESC	Erosion and Sediment Control
ESCP	Erosion and Sediment Control Plan
FTSF	Flotation Tailings Storage Facility
GDE	Groundwater Dependent Ecosystem
GL	Gigalitre
HDPE	High Density Polyethylene
H_2SO_4	Sulphuric Acid
IFD	Intensity–Frequency–Duration
JORC	Joint Ore Reserves Committee
kg	Kilogram
L	Litre
LOM	Life of Mine
µg	Microgram
µS	Microsiemens
mg	Milligram
M&I	Measured and Indicated
ML	Mining Lease
ML	Megalitre
MMP	Mining Management Plan
MPA	Maximum Potential Acidity
NAF	Non-acid Forming
NAG	Net Acid Generation
NMD	Neutral Mine Drainage
NTEPA	Northern Territory Environmental Protection Agency
PAF	Potentially Acid Forming
PAH	Polycyclic Aromatic hydrocarbons
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PPL	Perpetual Pastoral Lease
RE	Rare Earths

Term	Description
REE	Rare Earth Elements
ROM	Run of Mine
RSF	Residue Storage Facility
S	Sulphur
SD	Saline Drainage
SOCS	Site of Conservation Significance
SSTV	Site Specific Trigger Value
SSGTV	Site Specific Groundwater Trigger Values
STP	Sewage Treatment Plant
ToR	Terms of Reference
TRH	Total Recoverable Hydrocarbon
WDL	Waste Discharge Licence
WMP	Water Management Plan
WRD	Waste Rock Dump
WSD	Water Storage Dam

1. Introduction

1.1 Purpose

The purpose of this Water Management Plan (WMP) is to address water management for the Nolans Rare Earths Project (Project) which includes the mine site, processing site, borefield and accommodation village. The Project phases include construction, mining, processing, rehabilitation and decommissioning. Currently the mine design is in the preliminary design stage with limited staging information available for the expansions throughout the Life of Mine (LOM).

This WMP has been developed to:

- Establish a systematic and planned series of monitoring events;
- Determine baseline conditions and operational impacts upon the surrounding environment; and
- Implement and assess rehabilitation goals.

This WMP forms part of the Environmental Management Plan (EMP) as part of the requirements of the NT EPA Terms of Reference (ToR) issued in May 2015 (NT EPA, 2015). The WMP is currently in draft form to facilitate environmental approvals and will be reviewed and updated prior to Project construction.

1.2 Objectives

The WMP objectives include:

- Establish surface water, stormwater, processing water (onsite water storage), groundwater and sediment sampling regimes and procedures; and
- Provide requirements for establishing groundwater site specific trigger values for the mine site and processing site.

1.3 Associated Documentation

The WMP forms part of the Environmental Management Plan. In addition, several sub-management plans, procedures and technical studies relate to the WMP as follows:

- Management Documentation
  - Surface Water Sampling Procedure (Appendix A)

    Field guide to sampling surface water bodies across the Project including procedures for sampling seepages and emergency overflow.
  - Sediment Sampling Procedure (Appendix B)

    Field guide to sampling sediment across the Project.
  - Groundwater Sampling Procedure (Appendix C)

    Field guide to purging and sampling groundwater across the Project.
  - Erosion and Sediment Control Plan (Appendix D)

    In addition to the WMP, an Erosion and Sediment Control Plan (ESCP) has been developed for the Project. The ESCP includes details of permanent and temporary erosion and sediment control methods and treatments to be implemented and is consistent with the International Erosion Control Association's Best Practice Publication.
  - Emergency Response Management Plan (Environmental Management Plan, Appendix F)

The ERMP summarises processes for managing environmental investigations across the project including the development of Sampling, Analysis and Quality Plan, Site Investigation and Incident Assessment Report requirements.

- Technical Studies
  – Hydrogeological Assessment (EIS Appendix K);
  – Surface Water Technical Report (EIS Appendix I); and
  – Acid and Metalliferous Drainage (AMD) Assessment and Management Plan (EIS Appendix L).

1.4 EIS ToR Requirements

The WMP has been developed in accordance with the Northern Territory Environmental Protection Authority (NT EPA) Terms of Reference for the Project. A summary to address ToR requirements and the provision of relevant sections of the WMP or other management documentation is provided in Table 1-1.

Table 1-1 Summary of ToR Requirements

ToR Requirement		WMP Management Summary
The WMP should include but not be limited to measures that avoid or minimise:	Project contamination of surface or groundwater resources.	The potential for contamination of surface water and groundwater has been established within a preliminary assessment of potential source-pathway-receptor. The potential source-pathway-receptor table is provided in Section 5.
	Impacts to water dependent ecosystems.	The riparian vegetation immediately adjacent to the mine area (upstream to the point of the diversion and downstream in Kerosene Camp Creek to about the confluence of Nolans Creek) is highly likely to be catastrophically impacted by the mining operations (i.e. riparian vegetation will die and not recolonise the area). Modelled drawdown from the Borefield may peak in the order of 1.5 m in the vicinity of Day Creek during extraction. The drawdown rebounds rapidly once extraction ceases. The current depth to groundwater is generally 20 m and it is considered likely that existing vegetation would be capable of extending root systems during the extraction period. Napperby Creek is approximately 18 km further west than Day Creek is from the Borefield and as such drawdown is significantly reduced. Drawdown peaks at 0.7 m at the end of mining and recovers to 0.1 m one thousand years post closure. The predicted drawdowns are negligible in the Lake Lewis area and are not likely to be measureable. However, the peak decreases in groundwater availability for evapotranspiration in the Lake Lewis area of the Southern Basins is 3% or 712 m^3/day (8 L/s) and this rebounds to approach steady state with a decrease of approximately 0.5 % or 103 m^3/day (1 L/s).

ToR Requirement		WMP Management Summary
	Impacts to existing users of bores and/or surface waterways.	The initial impacts on the groundwater system will be localised drawdown at the mine site (from dewatering) and Borefield (from production bore extractions). The old Alyuen Community water supply may be affected by the groundwater extraction from the mine. It should be noted that Alyuen community is now supplied from a new bore in the eastern section of the Southern Basins and it is predicted that no impact will occur to this supply. Laramba and Napperby groundwater supply (drinking water) situated on the western side of Day Creek north of the Reaphook Range is expected to experience a peak drawdown of approximately 1.5 m at the end of mining and decrease to 0.1 m one thousand years post closure.
	Exposure of sensitive biological receptors to contaminants or water of a poor quality that may be harmful. Release of contaminated Project waters or hazardous materials to the environment, including post-closure.	The Project is being established as a nil discharge site. Contingency measures in place will include: • ROM Pad and Low grade Ore Stockpile Pad constructed with impermeable bases (1x10-8 m/s) with drainage to stormwater retention ponds; • Flotation Tailings Storage Facility (FTSF) constructed with low permeability soil liner and seepage collection system; • Residue Storage Facility (RSF) constructed with HDPE/low permeability soil liner system, combined with basin drainage and a leakage collection and recovery system; • Evaporation Ponds (EPs) constructed with HDPE liner; • Fuel stored in self-bunded Above Ground Storage Tanks (ASTs); and • Chemical Storage Shed with internal bunding. Failure of primary containment controls within designated water containment structure basins and/or Processing infrastructure will be pumped into the pit to reduce the risk of uncontrolled discharge. The Project will be rehabilitated and the working strategy for the Flotation Tailings Storage Facility (FTSF), Residue Storage Facility (RSF) and Evaporation Ponds (EPs) includes revegetation, top flow diversion banks to a rock lined chute with energy dissipater and recessed rock pad (outlet structure) at the toe of the structures. The groundwater model indicates the pit is likely to become a long term groundwater sink (evaporation losses exceed inputs from runoff, precipitation and groundwater inflows). As a result, the pit water is expected to show continual increases in acidity, metals and salt concentrations over time through accumulation of solutes introduced via groundwater inflows, run-off and precipitation. However, the pit lake is not considered a contaminant risk to the local or regional aquifers as it will remain a terminal sink.
Measures to treat and manage domestic wastewater and sewage.		Wastewater (non-Processing water) from the accommodation village, processing site and mine site will be pumped to a Sewage Treatment Plant (STP) adjacent to the processing site or at the mine site. Treated effluent will be discharged to the evaporation ponds and sludge will be disposed offsite at an appropriately licenced facility.

<table>
<tr><th colspan="2">ToR Requirement</th><th>WMP Management Summary</th></tr>
<tr><td colspan="2">Measures to ensure treatment / neutralisation of hazardous materials to identified safe levels, before any controlled environmental release is considered.</td><td>The Project has been established as a nil discharge site. Contingency measures will include the installation of leak detection systems at the Flotation Tailings Storage Facility (FTSF), Residue Storage Facility (RSF) and Evaporation Ponds (EP).
Failure of primary containment controls within designated water containment structure basins and/or Processing infrastructure will be pumped into the pit to reduce the risk of uncontrolled discharge.</td></tr>
<tr><td colspan="2">Outline details of monitoring programs to be implemented throughout the life of the Project to determine effectiveness of the mitigation measures (Section 5.3.3), and to monitor for impacts to water resources from the Project.</td><td>Monitoring will comprise surface water (when available), sediment and groundwater. The monitoring program has been designed in accordance with the Multiple Before-After Control-Impact (MBACI) methodology.
A baseline will be established throughout the construction phase (24 months) for all elements of the monitoring program. It is intended to use the baseline to develop site specific trigger values for groundwater and provide reference points for surface water and sediment.
Monitoring programs are detailed in Section 6.</td></tr>
<tr><td colspan="2">Proposed monitoring should be described for leaks, spills or seepage of materials from pipelines, storage / disposal facilities (including tailings disposal facilities) and transport operations to identify impacts, should they occur, to local soils, aquifers, environments, workers and/or the general public.</td><td>Leaks and seepage of hazardous substances will be managed in accordance with the Hazardous Substances Management Plan. Spills or sabotage will be managed in accordance with the Emergency Response Management Plan (ERMP).
These management plans are provided within the Environmental Management Plan (EMP).</td></tr>
<tr><td rowspan="2">The monitoring programs should include relevant water quality target values based on appropriate guidelines and/or standards. The monitoring program should outline reporting procedures and contingencies that will be implemented in the event monitoring activities identify performance indicators have been triggered, or other water related hazard or emergency.</td><td>Methods to monitor the impacts of the Project on surface and groundwater quality and quantity during mine operations and beyond mine closure.</td><td>Monitoring locations have been established outside of the of the LOM footprint for surface water, sediment and groundwater across the Project. These locations will be utilised to establish a baseline, assess impacts during operation and confirm successful rehabilitation of the Project.
The locations include control sites (upstream / up gradient), adjacent site (point of discharge) and impact sites (downstream / downgradient).</td></tr>
<tr><td>Provisions to notify and respond to environmental and human health risks associated with water quality, or other water related emergency.</td><td>Statutory notification procedures for hazardous substances spills/leaks and/or environmental incidents are provided within the Hazardous Substances Management Plan and Emergency Response Management Plan within the EMP.
An environmental investigation procedure has been established for the response to incidents with the potential to cause environmental impacts. The procedure includes development of a Sampling, Analysis and Quality Plan (SAQP), Site Investigation and Incident Assessment Report (IAR) and is detailed within the ERMP.
Procedures for responses to structural failures, hazardous substances spills and vehicle incidents are provided within the ERMP.</td></tr>
</table>

ToR Requirement		WMP Management Summary
	Contingency plans to be implemented should monitoring identify an unacceptable impact.	Trigger values have been determined for sediment and groundwater across the Project including ANZECC Interim Sediment Quality Guideline (ISQG) for sediment and Stockwater, Drinking Water and Irrigation for groundwater. However, it is noted that groundwater sampling to date includes local values already exceeding several trigger values and therefore site specific trigger values will be established for groundwater prior to Project operation commencing. During operation, analytical results will be compared to trigger values. If the trigger values have been exceeded across three consecutive occasions (Section 6.2.3, 6.3.3 and 6.4.2) additional investigations will be undertaken to determine the root cause and assess additional management measures.
Erosion and Sediment Control Plan (ESCP) including permanent and temporary measures.		An ESCP has been established for the Project and is provided in Appendix D. The Plan covers construction (initial clearing), operation and closure scenarios.

1.5 Guidelines

The WMP has been developed with reference to the Australian and New Zealand Environment and Conservation Council (ANZECC) guidelines including the National Water Quality Management Strategy Documents as follows:

- Australian and New Zealand Guidelines for Fresh and Marine Water Quality (ANZECC/ARMCANZ, 2000a);
- Australian Guidelines for Water Quality Monitoring & Reporting (ANZECC/ARMCANZ, 2000b);
- Australian Drinking Water Guidelines (NHMRC/ARMCANZ) (2004); and
- Guidelines for Groundwater Protection in Australia (ANZECC/ARMCANZ, 2013).

2. Overview

2.1 Location

The Project is situated approximately 135 km northwest of Alice Springs and 13 km west of Aileron in the Northern Territory.

2.2 Operator Details

The Nolans Rare Earths Project is 100% owned by Arafura, an Australian stock exchange listed company. The project will be the company's first operational mine. A summary of the operator details are provided in Table 2-1

Table 2-1 Operator Details

Company	Arafura Resources Limited
Contact	Brian Fowler NT General Manager and Sustainability
Street address	18 Menmuir Street, Winnellie, NT 0820
Postal address	PO Box 37220, Winnellie, NT 0820
Phone	08 8947 5588
Fax	08 8947 5599
Email	bfowler@arultd.com
ABN	22 080 933 455
ASX code	ARU
Web	http://www.arultd.com/

2.3 Development Schedule

Arafura plans to mine, concentrate and chemically Process Rare Earths at the Nolans Site and transfer the Rare Earths intermediate product to an offshore refinery (Rare Earths Separation Plant) for final processing. The construction phase for the Project is expected to take 2 years followed by a 41 year mine production period. A summary of the Project development is provided in Table 2-2.

Table 2-2 Project Development Summary

Description	Unit	M&I Case[a]	LOM Case[b]
Construction	Months	30	
Mine Pit Stages	Number	6	7
Waste Rock Dumps	Number	6	
Flotation Tailings Storage Facility	Number	1	
Residue Storage Facility	Number	2	
Evaporation Pond	Cells	6	
Total Mine Life	Years	25	43
Effective Mine Production Period	Years	23	41
Rehabilitation and Closure	Years	-	-

Note: [a] M&I refers to the higher-confidence Measured and Indicated classifications of the Project's total inventory of Mineral Resources that, under certain assumptions, could be converted to economic Ore Reserves in accordance with the 2012 JORC code.

[b]LOM refers to the Project's total inventory of Mineral Resources, represented by Measured, Indicated and Inferred classifications in accordance with the 2012 JORC code.

2.4 Resource

The Project's Mineral Resources are comprised of two broad styles of Rare Earth-bearing mineralisation including Apatite and Calcsilicate. The two broad styles of mineralisation are further subdivided into six material type categories based on geological and mineralogical characteristics and metallurgical performance. The material types are detailed in Table 2-3.

Table 2-3 Material Types

Style	Type	Description	Proportion
Apatite (MT123)	1	Cream/green Apatite	17%
	2	Brown Apatite	7%
	3	Brown Apatite with kaolin and/or clay	21%
Calcsilicate (MT456)	4	Apatite and Allanite	9%
	5	Apatite, Allanite and Calcsilicate	44%
	6	Apatite, Allanite, Calcsilicate with kaolin and/or clay	2%

2.5 Background

2.5.1 Project and Surrounding Land Use

The Project is situated on the Aileron Perpetual Pastoral Lease (PPL 1097) held by Aileron Pastoral Holdings Pty Ltd. Napperby Perpetual Pastoral Lease is situated to the west of this lease (west of the borefield). The leases have been used for grazing cattle and extraction of stockwater since the late 1800's.

2.5.2 Communities

Aileron Roadhouse is the closest community to the Project situated approximately 12 km to the east of the mine site. Laramba is the closest community to the borefield situated approximately 30km from the western end of the borefield to the northwest. Additional small communities or outstations adjacent to the Project include:

- Aileron station – 4,078 km^2 cattle station within which the entire footprint of the Project is contained;
- Alyuen (Aileron) – a family outstation 130 km north of Alice Springs and 3 km south of the Aileron roadhouse (population approximately 20 people);
- Alkuptija (Gillians Bore) – a family outstation 3 km east of Stuart Highway and 75 km southeast of the Project (population approximately 20 people);
- Burt Creek (Rice's Camp) – a family outstation close to Stuart Highway and 85 km southeast of the Project (population approximately 15 people);
- Injulkama (Amburla) – a family outstation 56 km south of the Project and 100 km to the northwest of Alice Springs (population approximately 10 people);
- Laramba – key community due to its relative proximity to the borefield. Access to the community is by the Napperby Station Road, which runs west from the Stuart Highway. The community is located 83 km from the turnoff. Laramba is a large community of mostly Aboriginal people (population approximately 300 people);
- Napperby Station – cattle station with homestead located 50 km to the west of the mine site. The Station is owned and operated by the Chisholm family since 1948;

- Pine Hill (Anyumgyumba) – a family outstation located 35 km west of the Stuart Highway and approximately 27 km north westof the Project (population varies from 0-20 people);
- Pmara Jutunta (Six Mile) – a major community of approximately 200 people 45 km to the northeast of the Project and close to the Stuart Highway and Ti Tree community; and
- Ti Tree – a community located 170 km north of Alice Springs along the Stuart Highway (population approximately 280 people).

2.5.3 Topography

The mine site lies at the head of Kerosene Camp Creek valley on the north facing slopes of an east – west trending ridge of the Reynolds Range. The processing site is situated on the southern slopes of the same ridge. Topographic elevation is 886 m above sea level (m ASL) at Mt Boothby to the east of the mine site, and 1006 m ASL at Mt Freeling to the west. Most of the Kerosene Camp Creek valley floor at the mine site is typically between 650 and 700 m ASL, and longitudinal gradients along local creeks to the north and south of the ridge line are typically less than 0.5 percent, with steeper gradients of approximately 10 percent on isolated hills.

The accommodation village and borefield are relatively flat locations with poorly defined natural waterways and/or drainages.

2.5.4 Vegetation

A total of 14 vegetation communities were identified across the Project. These vegetation communities each display a degree of variation which is to be expected given the influence of differing geology, soils, hydrology, fire regimes and grazing pressures.

The dominant vegetation types at the Project are Mulga shrublands, which occur on alluvial fans and plains containing clayey red earths and Triodia hummock grasslands which grow on sandy plains.

In more fertile riparian areas and associated floodplains there is evidence of impacts associated with cattle grazing including weed invasion, reduction in ground cover species richness and soil erosion. In particular there is a high abundance of the introduced Buffel Grass (*Cenchrus ciliaris*).

2.5.5 Acid, Metalliferous and Saline Drainage Potential

A total of 200 stage one and 25 stage two Acid Metalliferous Drainage tests were undertaken. The results of static Net Acid Generation (NAG) tests indicate the majority of material is non-reactive and Non-Acid-Forming (NAF). One recorded sample was recorded as Potential Acid Forming (PAF) with a relatively low Maximum Potential Acidity (MPA) and low Acid Neutralising Capacity (ANC). Final kinetic NAG pH showed that single addition NAG pH is suitable for identifying PAF and reaction times are relatively slow. The tests ultimately indicated there was a very low risk of acid generation either during short-term storage of ore or long-term storage of waste rock.

The abundance of NAF and Acid Consuming Material (ACM) provides a conservative cut-off value of 0.3% Sulphur (S) or 10 kg/t Sulphuric Acid (H_2SO_4) for PAF material. Confirmatory field NAG testing will be carried out on samples with a sulfur content of greater than 0.15% S, unless pre-production testing provides sufficient information for a revised cut-off.

The 25 leachate tests indicate the majority of the waste rock was non-sulfidic and relatively benign, with small amounts of material with slightly elevated sulfur. Although neutralised by the excess ANC the material may, however, contain metals such as zinc that form soluble carbonates when their sulfide forms are oxidised and neutralised with carbonate minerals. Leachate salinity was low and fluoride was elevated in one sample, hence the risk of generation of saline or fluoride-rich leachate is low.

One sample of pegmatite produced Australian Standard Leaching Procedure (ASLP) leachate with a lead concentration 1,054 times greater than the average groundwater concentration. The critical leachate constituents appear to be aluminium and zinc which were consistently elevated against ambient groundwater concentrations. Site specific groundwater values will be developed for future assessments of potential impact.

Leachate from WRDs is considered unlikely to pose significant risks to the existing groundwater and/or ephemeral surface water systems at the Project.

Based on the geochemistry of the waste rock and ore, the risk of acid, metalliferous or saline drainage is low and the material can generally be managed as NAF waste. The Acid and Metalliferous Drainage Management Plan (AMDMP) will be implemented and provide contingency for management of the nominally <0.05 % PAF material.

The AMDMP is provided in Appendix L of the EIS.

2.5.6 Climate

Rainfall and Evaporation

The mean annual rainfall is approximately 316.7 mm, with a seasonal pattern of more summer rainfall than winter rainfall. Average monthly rainfall totals range from 4.7 mm in August to 65.8 mm in February. Average three-monthly rainfall totals range from 18.3 mm in June/July/August to 178.7 mm in December/January/February. However, any month can receive relatively large rainfall totals, or little or no rain at all.

Evaporation is greatest during months of higher mean rainfall with the highest average evaporation occurring in December and January at 375 mm. Rates of evaporation are significantly lower from May to August coinciding with lower mean rainfall and temperatures. The annual average evaporation is 3,000 mm, approximately 847% greater than the annual average rainfall of 316.7 mm.

The rainfall and evaporation rates are provided in Table 2-5.

Rainfall Statistics

Rainfall at the site is generally characterised by infrequent and intense rainfall events, single events can deliver > 50 mm within 24 hour. The Bureau of Meteorology Intensity–Frequency–Duration (IFD) indicates 305 mm for a 1 in 100 year, 72 hour rainfall event. A summary of the IFDs are provided in Table 2-4.

The PMP is the greatest depth of precipitation for a given duration meteorologically possible for a given size storm area (World Meteorological Organization, 1886). The Probable Maximum Precipitation (PMP) 72 hour storm depth is 1,100 mm at the Project.

Table 2-4 IFD Rainfall Depth (mm) (Source: BOM IFD AR&R87 Tool)

Duration	Return Period (Years)						
	1	2	5	10	20	50	100
5	5	6	9	10	12	15	17
6	5	7	10	12	14	17	19
10	7	10	14	16	19	24	27
20	11	15	21	25	30	37	43
30	14	19	27	32	38	47	54
1	19	25	37	44	53	66	76
2	24	32	47	57	69	86	100
3	26	35	53	64	79	99	115

Duration	Return Period (Years)						
	1	2	5	10	20	50	100
6	31	42	64	79	97	123	145
12	36	50	78	97	120	155	182
24	45	62	97	121	151	194	229
48	56	77	120	148	185	238	281
72	60	83	129	161	200	257	305

Temperature and Humidity

The Project area experiences hot and arid conditions. The hottest months are November to March, with monthly mean daily maximum temperatures above 35 ^{o}C, and monthly mean daily minimum temperatures not dropping below 18 ^{o}C. The coolest months are May to August, with monthly mean daily maximum temperatures remaining at or below 25.5 ^{o}C, and monthly mean daily minimum temperatures not rising above 9.5 ^{o}C.

The average humidity at the Project is 40% at 09:00 and 25% at 15:00, this is consistent across the year with monthly afternoon humidity readings being 15% lower than the morning. The highest levels of humidity are experienced in June at 53%. This coincides with lower temperatures occurring.

The temperature and humidity rates are provided in Table 2-5.

Wind

The winds at the Project are predominant south easterly wind direction throughout the year. The average wind speeds range from 2.50 to 3.17 m/s (9.0 to 11.4 km/h) with an annual average of 2.86 m/s (10.3 km/h).

The wind roses are provided in Figure 2-1 and speeds are summarised in Table 2-5.

Table 2-5 Summary of Climate Statistics (BoM 2015; Territory Grape Farm NT 1987-2014)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Rainfall (mm)												
Highest	280.4	**342.2**	109.2	151.7	136.3	53.8	34.2	39.4	96.6	56.8	119.2	119.2
95th percentile	159.0	**244.2**	96.9	89.9	100.1	48.7	21.3	26.9	41.7	51.3	81.4	109.9
Mean	62.4	**65.8**	21.9	18.0	23.3	8.7	4.9	4.7	10.3	15.3	30.9	50.5
5th percentile	3.8	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.9	**8.9**
Lowest	**2.2**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.0
Evaporation (mm)												
Total	**375**	300	290	210	150	125	145	180	200	300	350	**375**
Temperature (°C)												
Maximum[1]	**37.3**	36.2	34.3	30.5	25.5	22.2	22.5	25.3	30.5	33.3	35.6	36.3
Minimum[2]	**21.9**	21.6	19.5	14.6	9.5	6.2	5.2	7.1	12.1	15.6	18.8	21.1
Humidity (%)												
Mean 9 am	38	40	37	37	47	**53**	51	38	32	32	34	37

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean 3 pm	24	**28**	27	25	27	**28**	**28**	22	21	21	22	26
Wind (km/h)												
Mean 9 am	17.0	18.1	**19.7**	18.9	15.2	12.8	14.3	17.3	18.2	19.6	18.2	18.0
Mean 3 pm	15.8	**16.7**	16.6	14.9	14.2	13.5	14.0	16.0	15.5	14.8	14.1	14.5

Notes: [1] Monthly mean maximum temperature is the average of the available daily maxima for that month.

[2] Monthly mean minimum temperature is the average of the available daily minima for that month.

Highest values are indicated in bold.

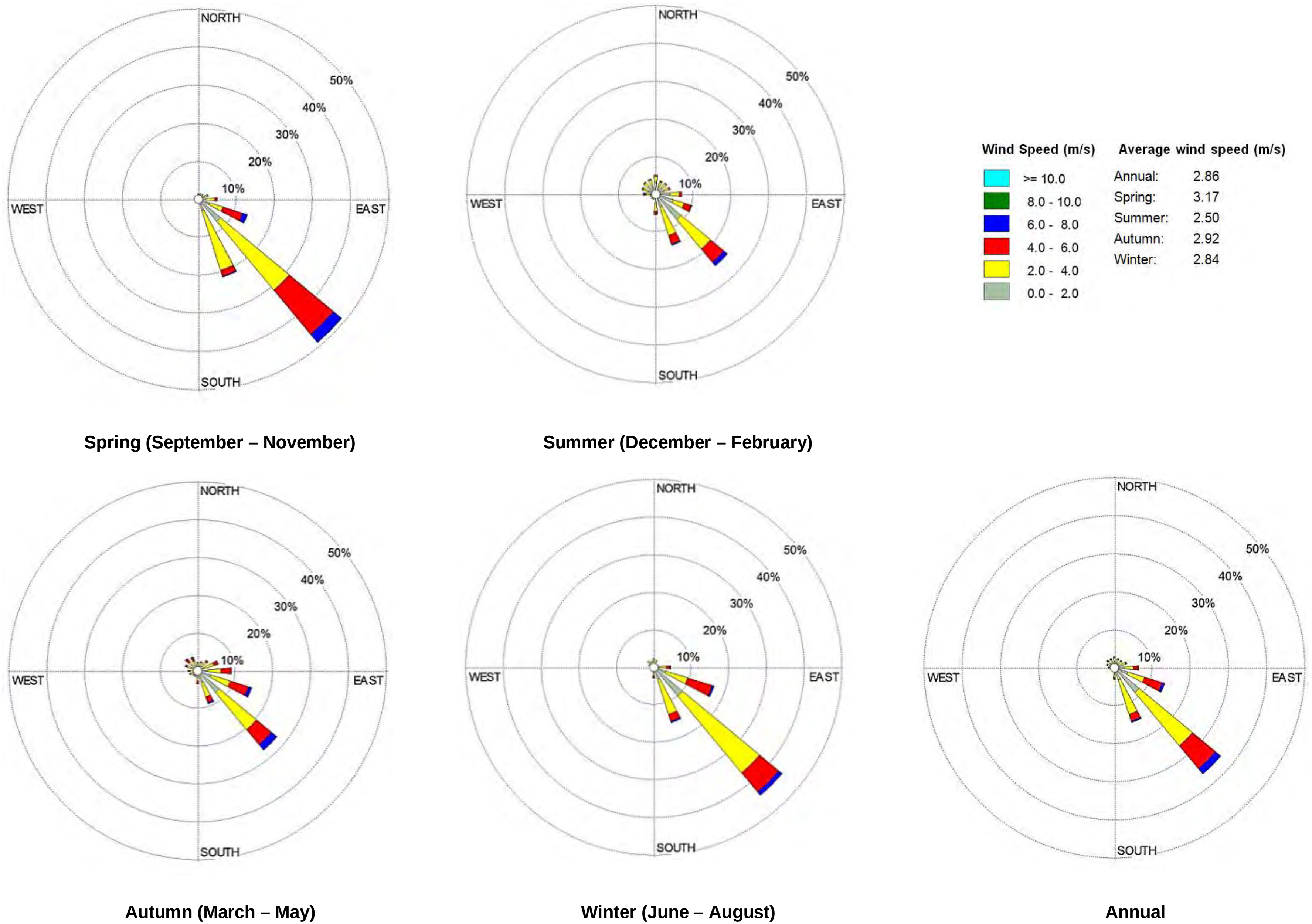


Figure 2-1 Annual and Seasonal Wind Roses (Source: BOM, Territory Grape Farm Station No. 015643)

3. Surface Water

3.1 Drainage

Semi-arid regions, such as the area in which the Project is located, are typically characterised by conditions in which actual evaporation closely matches rainfall and virtually all rainfall evaporates resulting in almost no surface runoff. Therefore, the occurrence of surface runoff and flows within local creeks is infrequent and only occurs during exceptional rainfall events associated with the occasional southward extension of the monsoon trough or periodic incursion of north-west cloud bands over the interior.

3.1.1 Mine Site

The mine site is situated within the Ti-Tree Basin which has a large catchment area.

Kerosene Camp Creek is the main creek which flows through the mine site, the creek flows from south to north across the Site. Kerosene Camp Creek is fed by several creeks across the mine site. The catchment area of Kerosene Camp Creek and its tributaries upstream of the mine site is approximately 18 km^2.

Nolans Creek joins Kerosene Camp Creek approximately 500 m downstream of the mine site northern boundary. However, Nolans Creek also transects the mine site flowing adjacent to the FTSF and between Waste Rock Dump 2 and 6. It has an upstream catchment of 26 km^2.

Kerosene Camp Creek joins the Woodforde River approximately 12 km from the mine site northern boundary. Woodforde River ultimately becomes a flood-out approximately 62 km north of the mine site northern boundary with the potential to discharge to the Hanson River.

The mine site surface water features are illustrated on Figure 3-1.

3.1.2 Processing Site

The processing site is situated on the head waters of the Southern Basin. This site has poorly defined natural waterways and/or drainages.

3.1.3 Accommodation Village

The accommodation village is situated on a plain with rocky hills nearby to the eastern side. This area has poorly defined natural waterways and/or drainages.

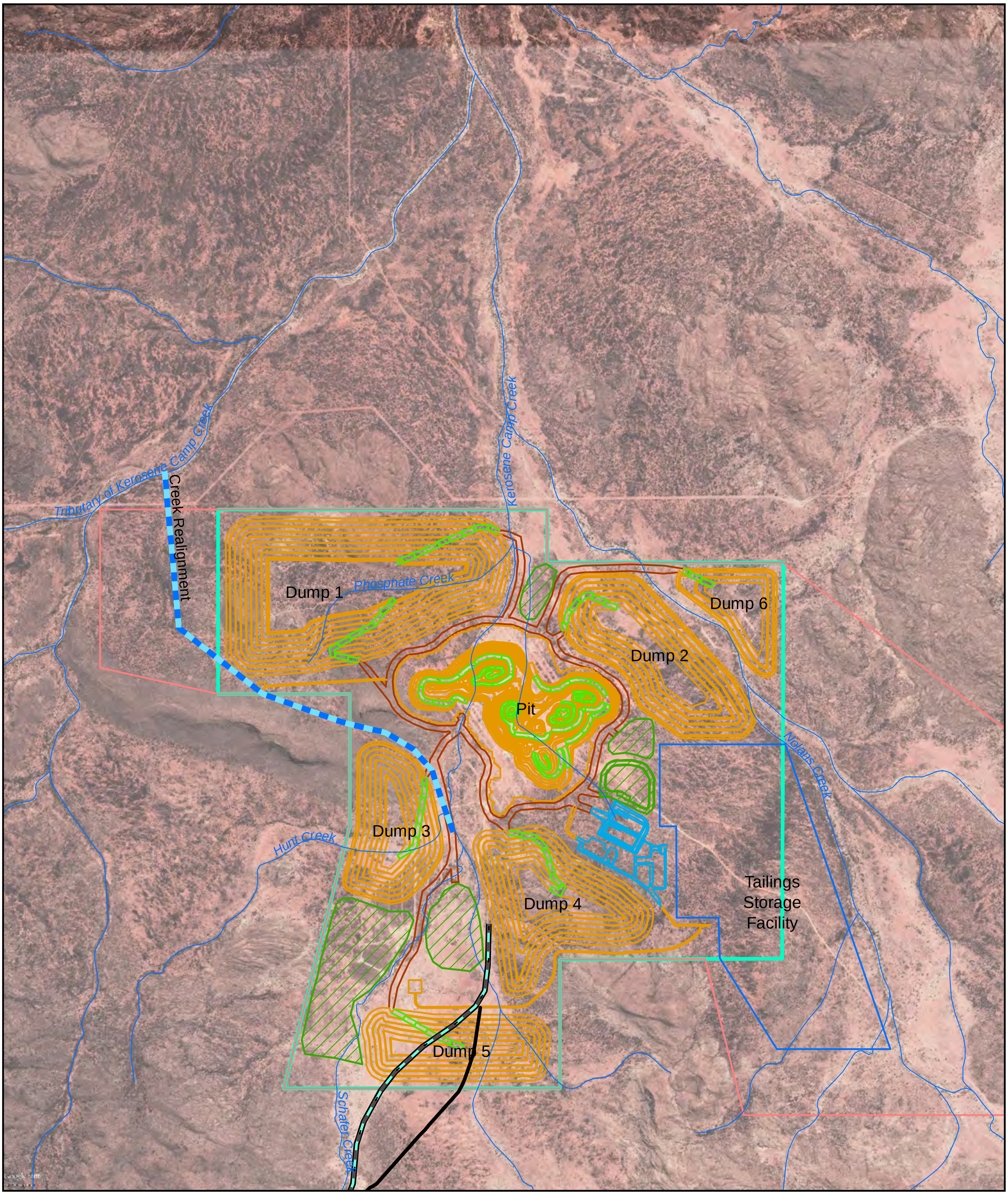
3.1.4 Borefield

The borefield is situated within the Southern Basin. No major creek/river system is present within the borefield itself. Day Creek is the closest watercourse situated approximately 10 km form the western end of the borefield and flows in a southerly direction.

Napperby Creek is the next closest watercourse located 30 km to the west of the borefield and also flows in a southerly direction directly into Lake Lewis.

Lake Lewis is approximately 42 km southwest of the borefield and considered a Site of Conservation Significance (SOCS No. 54) by the Department of Land Resource Management. It is considered to be of national significance in relation to providing:

- An important ecosystem for waterbird species, endemic and restricted range plant species; and
- Supporting a fish population during times of flood.



LEGEND

Powerline	Existing Gas Pipeline	Proposed Mine Site Boundary	ROM Pad
Water Supply Pipeline	Waterways	Topsoil Storage	
Diversion Channel	Roads	Mineral Lease	
Potable Water Pipeline	Haul Road/Access Tracks	Long Term Storage Stockpile	

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0 0.25 0.5 0.75 1

Kilometres

Map Projection: Universal Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 53

N

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Revision 0
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Mine Site Surface Water Features

Figure 3-1

G:\43\22301\GIS\Maps\4322301_1010_SurfaceWaterMonitoringLocations.mxd

Level 5, 66 Smith Street Darwin NT 0800 Australia **T** 61 8 8982 0100 **F** 61 8 8981 1075 **E** drwmail@ghd.com **W** www.ghd.com

Data source: GA - Roads, Waterways, Placenames, Placenames, Lakes (2015). ESRI - Shaded Relief (2009). Google Earth Pro - Imagery (Date extracted: 15/02/2016). ARL - Project Areas (2015). GHD - Monitoring Locations (2016). Created by: CM

3.2 Flood Risk

A flood assessment and modelling was undertaken for the mine site to establish potential flood related impacts. Recorded flow events at the mine site are infrequent and the probability of flood events occurring during the 41 year operation of the Project is summarised as follows:

- 1 in 10-year ARI flood event – 99%;
- 1 in 50-year ARI flood event – 58%; and
- 1 in 100-year ARI flood event – 35%.

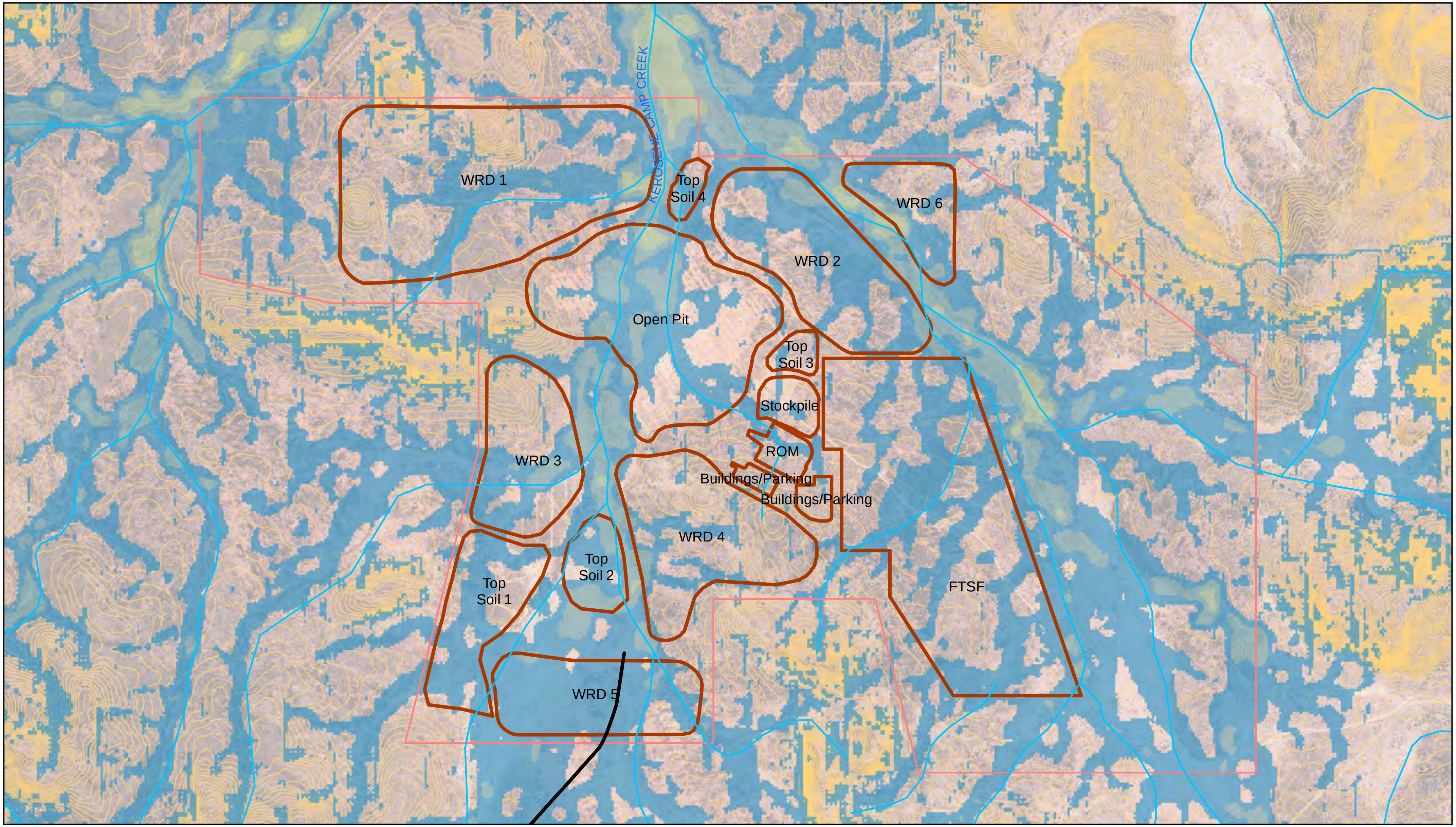
Modelling of the 1:100 ARI flood event for the current conditions (i.e. without LOM mine infrastructure present) and with proposed infrastructure was undertaken. Modelling included two separate types including:

- XP-RAFTS model to determine flood peak flows and velocity; and
- 2-D rain-on-grid flood modelling to establish extents and depth of floods.

The flood extents is provided in Figure 3-2 and Figure 3-3 and summarised in Table 3-1. Further details on the flood model are provided within the EIS, Appendix I. Flow diversion banks will be installed across the mine site to divert clean water away from infrastructure. The ROM Pad, Low grade Ore Stockpile and FTSF will be constructed to sufficient levels to mitigate flood potential from a 1:100 year ARI flood event.

Table 3-1 Modelled 1:100 ARI Flood Characteristics

Surface Water System	Location	Upstream Area (km^2)	Flood Event (1:100-year ARI)		
			Peak (m^3/s)	Velocity (m/s)	Depth (m)
Nolans Creek	Upstream mine site boundary	26.3	114	0.7	1.0
Nolans Creek	Downstream mine site boundary	29.6	120	0.5	1.0
Kerosene Camp Creek	Upstream mine site boundary	12.3	50	0.3	0.2
Kerosene Camp Creek	Proposed diversion inlet	20.0	86	0.8	2.0
Kerosene Camp Creek	Downstream mine site boundary	25.9	95	0.5	1.0
Kerosene Camp Creek	Downstream of confluence of Kerosene Camp Creek and Nolans Creek	58.0	232	0.5	1.3
Tributary to Kerosene Camp Creek	Proposed diversion outlet	46.0	184	0.7	3.0



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0 250 500 750 1,000

Metres

Map Projection: Universal Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 53

N

LEGEND

Existing Creek Channels
Proposed Haul Roads
2m Contours
Proposed Infrastructure

Flood Depth - Pre-mining (metres)

0.0 - 0.5
0.5 - 1
1.0 - 1.5
1.5 - 2
2.0 - 2.5
2.5 - 3
3.0 - 3.5
3.5 - 4

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Flood Depth (1:100 year)
Pre-mining

Figure 3-2

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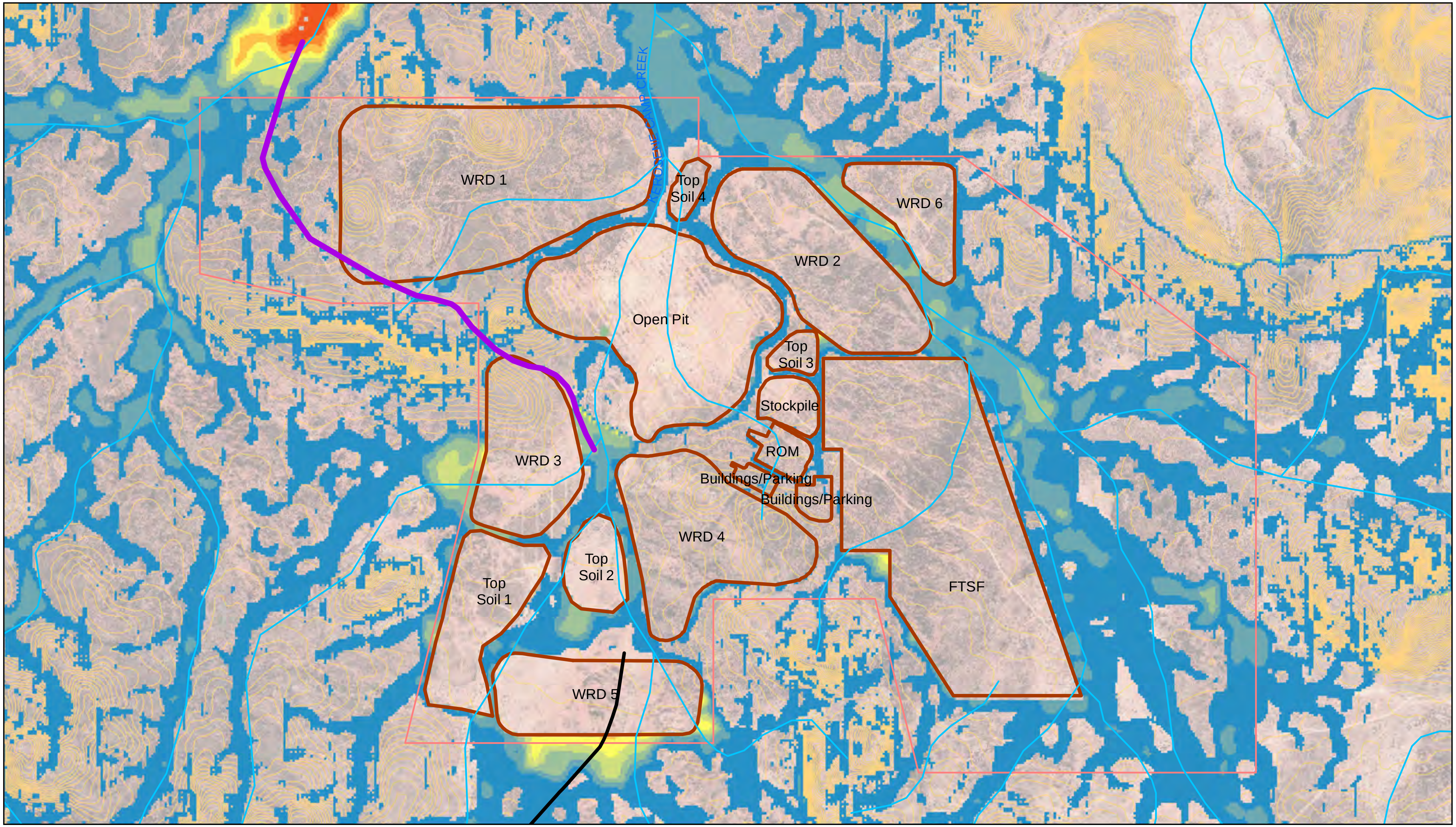
Level 5 66 Smith Street Darwin NT 0800 Australia T 61 8 8982 0100 F 61 8 8981 1075 E drwmail@ghd.com W www.ghd.com

Data source: Google Earth Pro - Imagery (Date extracted: 29/04/2015). ARL - Proposed Haul Roads, Existing Creek Channels, Proposed Infrastructure (2015). CSIRO - 2m Contours (derived from SRTM) (2015). Created by: CM



1:35,000 @ A4

0 250 500 750 1,000

Metres

Map Projection: Universal Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 53

N

LEGEND

Creek re-alignment
Existing Creek Channels
Proposed Haul Roads
2m Contours
Proposed Infrastructure

Flood Depth - Post-mining (Metres)

0 - 0.5
0.5 - 1
1.0 - 1.5
1.5 - 2
2.0 - 2.5
2.5 - 3
3.0 - 3.5
3.5 - 4

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Revision 0
Date 29 Apr 2016

Flood Depth (1:100 year)
Post mining

Figure 3-3

G:\43\22301\GIS\Maps\4322301_1012_FloodDepthPostMine.mxd

Level 5 66 Smith Street Darwin NT 0800 Australia **T** 61 8 8982 0100 **F** 61 8 8981 1075 **E** drwmail@ghd.com **W** www.ghd.com

Data source: Google Earth Pro - Imagery (Date extracted: 29/04/2015). ARL - Proposed Haul Roads, Existing Creek Channels, Proposed Infrastructure (2015). CSIRO - 2m Contours (derived from SRTM) (2015). Created by: CM

3.2.1 Infrastructure

Infrastructure with the ability to collect direct rainfall such as the FTSF, RSF and EP will be constructed with sufficient freeboard to collect a minimum of a 1:100 year 72 hour rainfall event (305 mm) whilst retaining sufficient additional freeboard to accommodate a PMP 72-hour storm rainfall event (1,100 mm).

Contingency management measures will include the installation of pumps and HDPE piping to transfer water to alternative storages (including the pit). A summary of Processing infrastructure, anticipated water quality and freeboard is provided in Table 3-2.

Table 3-2 Processing Infrastructure and Anticipated Water Quality

Infrastructure	Liner	Drainage	Water Classification	Rainfall Management	
				Freeboard	Contingency
Floatation Tailings Storage Facility	Low permeability soil liner.	Seepage collection system.	Elevated metal and metalloids.	100 year 72 hour ARI plus PMP 72 hour event.	Pump transfer to alternative storage/pit.
Residue Storage Facilities	High Density Polyethylene (HDPE)/low permeability soil liner.	Basin drainage and a leakage collection and recovery system.	Elevated metal and metalloids. Low pH.		
Evaporation Ponds		-			

3.2.2 Erosion and Sediment Control

An Erosion Sediment Control Plan (ESCP) has been prepared to provide high level strategy for the management of erosion and sediment across the Project areas. The ESCP covers the construction, operation and closure phases. ESCs will include the diversion of clean water around the site and installation of sediment, erosion and drainage controls.

Flow Diversion Banks

Flow diversion banks will be installed across the Project to divert clean water away from disturbed areas and therefore reducing management/reducing sediment basin capacities.

The FTSF and haul roads at the mine site will be constructed to levels above the 1:100 year 72 hour rainfall event. Culverts will be installed in haul roads to facilitate overland flow through the site and to facilitate diversion of flows into the creek realignment. Culvert design will be undertaken as part of the detailed design phase following the EIS.

Stormwater Retention Ponds

Surface water runoff from infrastructure will be collected by catch drains (with check dams) and transferred to stormwater basins. Water stored within the stormwater basins will be utilised for dust suppression, recycled or evaporated. Sediment basin water quality will be monitored across the Project. The frequency of tests will be reduced if water quality is consistent.

The basins will be dredged periodically to maintain capacity. Stormwater retention ponds will be designed in the detailed design phase.

3.3 Surface Hydrology

3.3.1 Watercourses

Kerosene Camp Creek is an ephemeral creek and flows through the centre of the mine site before joining the Woodforde River 12 km further to the north. Kerosene Camp Creek has a catchment area of ≈18 km^2 upstream of the mine pit.

Nolans Creek is a tributary of Kerosene Camp Creek and has a catchment area of 26 km^2 upstream of the confluence with Kerosene Camp Creek. It flows adjacent to the eastern boundary of the proposed Flotation Tailings Storage Facility and will pass between waste rock dumps 2 and 6 before it joins Kerosene Camp Creek.

Catchments upstream of creek crossing points along the access road from the Stuart Highway drain towards the Southern Basins and are typically less than 3 km^2, but with one catchment of about 12 km^2. Areas draining towards the Project are typically less than 1 km^2 in extent and channels are ill-defined with runoff likely to be dispersed across the south facing hillslope.

Semi-arid regions such as the area in which the Project is located are typically characterised by conditions in which actual evaporation during events closely matches rainfall and virtually all rainfall evaporates. Therefore, the occurrence of surface runoff and flows within local creeks is infrequent and only occurs during exceptional rainfall events associated with the occasional southward extension of a tropical monsoon trough or periodic incursion of north-west cloud bands over the interior of the continent.

Local creek beds tend to be mobile with deep sand deposition and banks that show signs of active erosion. Creek channels are typically 1.0 m deep with a base width of 5 m. Intense, short duration rainfall events can be expected to occur over the Project area and the relatively shallow depth of creek channels will lead to out-of-bank flow and possibly temporary and short-term flooding of adjacent areas.

3.3.2 Flow Records

Long-term gauging of flow in watercourses that traverse or flows near to the Project has been carried out at one location, namely Arden Soak Bore on the Woodforde River. A summary of gauging is provided in Table 3-3. This gauge is located approximately 26 km downstream of the mine site and comprises a water level gauge board in the sandy river bed.

A second gauge at Allungra Waterhole is located about 42 km to the east and is outside the catchment of the Project and its infrastructure.

The Arden Soak Bore and Allungra gauges both provide records of water level from which discharge can be calculated. Gauging of water levels is also carried out at a third location, Pine Hill on the Hanson River, which is situated 33 km to the west, also outside the catchment of the Project and its infrastructure. No flow records are available for this latter gauge.

Arden Soak Bore on the Woodforde River measures runoff from a catchment area (393 km^2) that is an order of magnitude greater than that subtended by the mine site (54 km^2). However, given the similarity of catchment conditions the recorded time series of water levels and discharge are likely to be indicative of the pattern of runoff (but not the magnitude) from catchments at the mine site.

An analysis of the flow record at Arden Soak Bore (Figure 3-4) confirms that flow events are relatively infrequent with only 25 percent of days during the 41 year record having a total daily flow greater than 3 ML (arbitrarily selected threshold discharge of 0.03 m^3/s). Runoff is most likely in months during the summer season, December to March (Figure 3-5). The low frequency of flow events suggests that only one or two flow events can be expected in most years (Figure 3-6).

The maximum recorded flow at Arden Soak Bore on Woodforde River is 206 m^3/s and occurred in January 2010 (Figure 3-7) with a measured water depth of 3.7 m. Whilst this flow was recorded 26 km downstream of the proposed Project it serves to show the relatively 'flashy' response and short duration of flow events for drainage systems in this region.

Both the flow frequency curve (Figure 3-4) and hydrograph of the maximum recorded flow event (Figure 3-7) at Arden Soak Bore on Woodforde River illustrate the absence of baseflow (surface flow sustained by groundwater). However, anecdotal evidence[1] states that during 2010 and 2011 (wet years) water drained out of the local hills for months and a 'soak' upstream of the mine site was wet most of the year. This suggests that surface runoff infiltrates into the alluvium of creek channels where it will form shallow groundwater flow moving down gradient along the creek channel.

The volume of surface runoff relative to locally recorded rainfall for the January 2010 event at Arden Soak Bore is estimated to be nine percent and indicates relatively high rainfall losses of over 90 percent. What proportion of this 'loss' infiltrates to a shallow aquifer and what proportion is lost to the atmosphere through evapotranspiration is uncertain but serves to confirm the typically low rate of surface runoff in the area.

Table 3-3 Hydrometric Gauges

Type	Gauge Number	Name	Latitude	Longitude	Record Start	Record End	Record Length (years)	Location Relative to Mine Project
River Flow	0280010	Woodforde River - Arden Soak	22.367	133.324	1974	open	41	Same river system 26 km downstream.
River Flow	0280004	Allungra Creek - Allungra Waterhole	22.689	133.631	1996	open	19	Different river system 42 km to east.
River Height	0280010	Woodforde River - Arden Soak Bore	22.367	133.324	1974	open	41	Same river system 26 km downstream.
River Height	0280004	Allungra Creek - Allungra Waterhole	22.689	133.631	1996	open	19	Different river system 42 km to east.
River Height	0280021	Hanson River At Pine Hill	22.367	133.025	1968	1977	9	Different river system 33 km to west.
Surface Water Quality	0280010	Woodforde River - Arden Soak Bore	22.367	133.324	1974	open	41	Same river system 26 km downstream.
Surface Water Quality	0280004	Allungra Creek - Allungra Waterhole	22.689	133.631	1996	open	19	Different river system 42 km to east.

[1] Nolans Feasibility Study – Preliminary Studies Project Drainage and Land Tenure, AMC Consultants, February 2015. Comments on report by K Hussey.

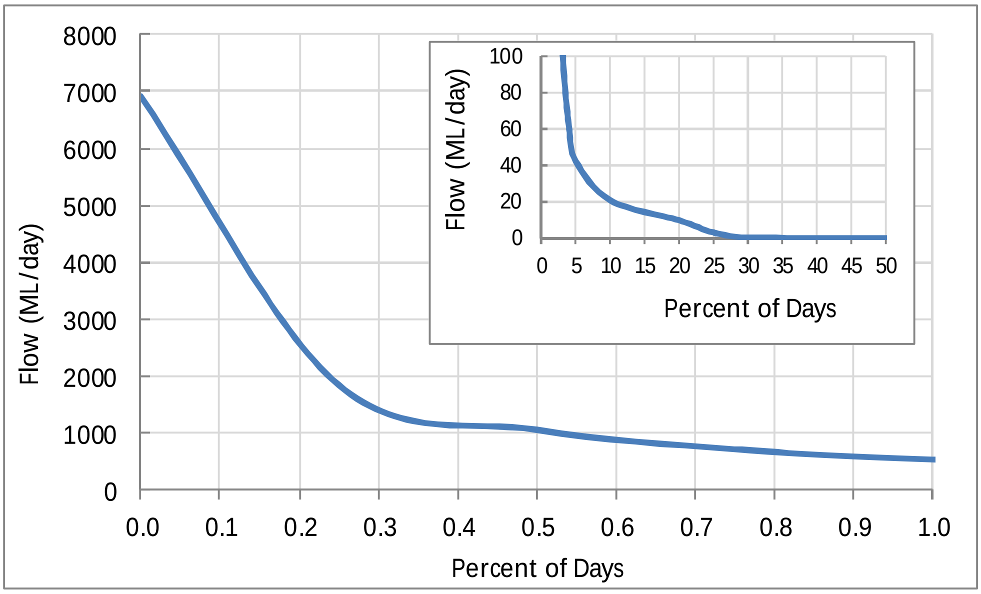


Figure 3-4 Flow frequency curve for Woodforde River at Arden Soak Bore

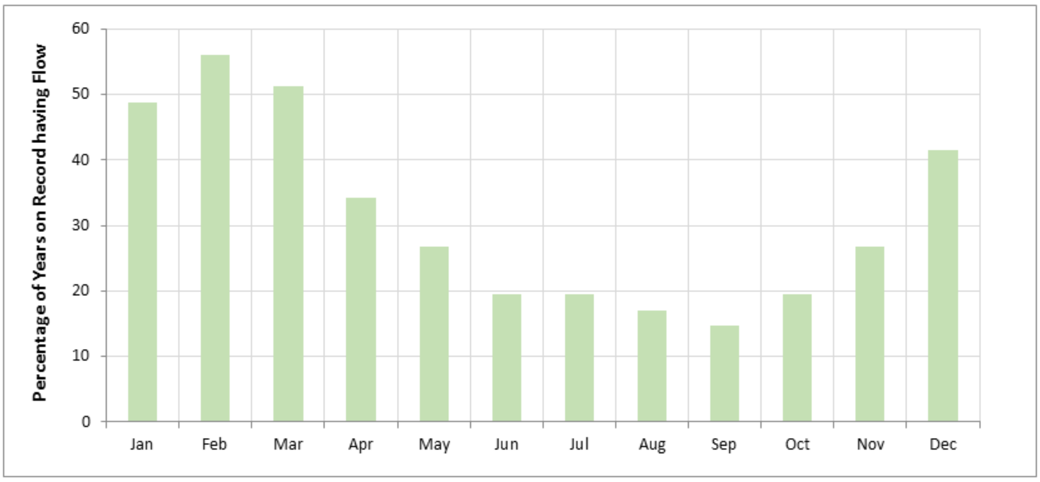


Figure 3-5 Occurrence of flow at Arden Soak Bore on Woodforde River

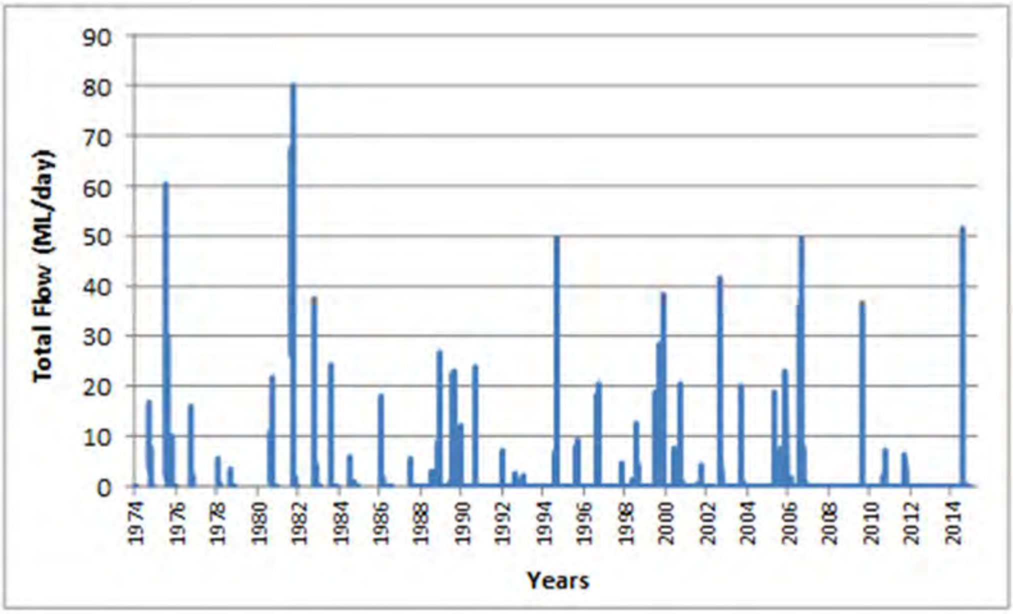


Figure 3-6 Time series of recorded flow at Arden Soak Bore on Woodforde River

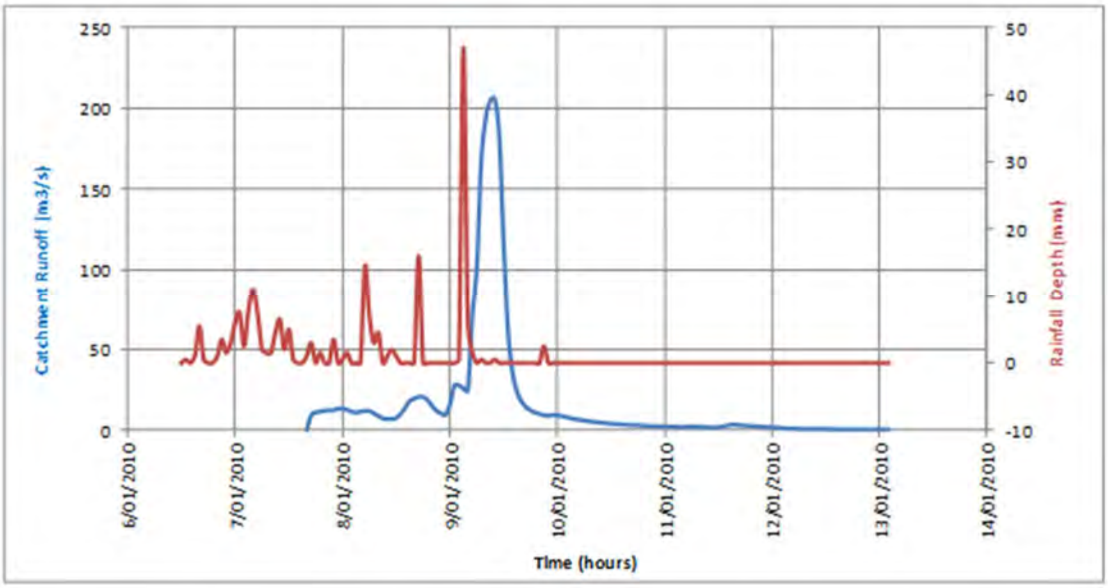


Figure 3-7 Maximum recorded flow at Arden Soak Bore on Woodforde River

4. Groundwater

4.1 Introduction

Groundwater extraction will occur at the Project via two methods including groundwater extraction from five production bores (RN18876, RN19027, RN19033, RN19037 and RN19039) within the Southern Basins and pit sump dewatering in the Ti-Tree Basin. The water supply will be utilised throughout the life of the Project (approximately 41 years) and following the closure of the Project pumping will cease. The pit will remain open and act as groundwater sink in perpetuity (evaporation losses exceed inputs from runoff, precipitation and groundwater inflows).

4.1.1 Groundwater Management

Groundwater will be managed at the Project in accordance with ANZECC/ARMCANZ (2013). Key principles from this guideline include management in accordance with fundamental economic and social principles including the consideration of current and future generational values.

Management of groundwater across the Project will be aligned with the six underlying principles including:

- Protection of specified Environmental Value

  The current and future landuse of the area is considered to be pastoral (cattle). Application of stock water and irrigation ANZECC values and reference to drinking water values will be used whilst local Site Specific Groundwater Trigger Values are established.

- Risk-based Approach

  The EIS risk assessment has assessed the risks posed by infrastructure and inform mitigation measures. Locations which are likely to provide a groundwater contamination source will be constructed to reduce risks to groundwater include:

  – ROM Pad and Stockpile Pad constructed with impermeable bases ($1x10^{-8}$ m/s) with drainage to stormwater sediment retention ponds;
  – FTSF constructed with low permeability soil liner and seepage collection system;
  – RSF constructed with HDPE/low permeability soil liner system, combined with basin drainage and a leakage collection and recovery system;
  – EPs constructed with HDPE liner;
  – Fuel stored in self-bunded Above Ground Storage Tanks (ASTs); and
  – Chemical Storage Shed with internal bunding.

- Polluter Pays Principle

  The site will be constructed, operated and rehabilitated in accordance with the *Mining Management Act* (MM Act). The MM Act requires the Proponent to report environmental data to assess and understand potential impacts from the Project to the Department of Mines and Energy (DME). In accordance with the MM Act, a security bond will be provided as part of the initial grant and maintaining Mine Authorisation. The security bond reinforces the polluter pays principle whereby the bond will be returned to the Proponent following successful rehabilitation or utilised by DME to complete rehabilitation (if the Proponent is not able to due to unforeseen circumstances).

- Intergenerational Equity

  Currently the predominate use for groundwater in the immediate vicinity of the mine site and processing site is for pastoral use (i.e. stock drinking water). The development of the Project will be undertaken with consideration of current and potential future generations of pastorists.

- Precautionary Principle

  Hydrogeological modelling has been undertaken to assess the potential impact of the Project on the surrounding environment. In accordance with the risk-based approach and implications of the polluter pays principal, the site will be operated under a precautionary principle.

- Ecologically Sustainable Development

  The Project will be managed in accordance with the above principals to promote ecologically sustainable development.

4.1.2 Hydrogeological Setting

Soils

A geotechnical assessment of the mine site was undertaken by Lycopodium Minerals in 2010 (Lycopodium Minerals, 2010). The assessment indicated soils at the mine site generally comprise clayey sand (colluvium) from surface to approximately 1m Below Ground Level (BGL). Laboratory testing undertaken on samples indicated 62% sand, 34% silt/clay and 4% gravel. A layer of gravelly sand to sandy gravel is present below the colluvium. The test pits met refusal on gneiss at depths ranging from 0.2 to 2.4 m BGL.

Geology

The basement geology at the Project can be summarised by three significant units including:

- Proterozoic Arunta Block granites and gneiss outcrop forming the bulk of the hills and ranges adjacent to the mine site (including Reynolds Range and Yalyirimbi Range) and basement rocks beneath the basins;
- Proterozoic Vaughan Springs Quartzite and Treuer Member outcropping as the Hann Range and Reaphook Hills as a distinct, almost linear feature across the southern plain, as isolated hills outcropping from the plain at the southern fringe of the Yalyirimbi Range and as basement rocks beneath only a minor section of the Southern Basins; and
- Ngalia Basin sedimentary rocks are also present, but comprise relatively little outcrop in the study area and form the basement of the majority of the Whitcherry Basin section of the Southern Basins.

It is recognised that the Arunta Block also contains multiple units other than granites and gneiss (i.e. schist, quartzite etc.) which may contain higher fracture permeability, but all Arunta Block rocks are collectively grouped as the hydrogeological unit ‘basement’ for the purpose of this assessment.

Only the mineralised areas of the ore deposit that contain primary porosity are considered in isolation as distinct aquifer. The rocks of the Vaughan Springs Quartzite and Treuer Member, as well as those of the Ngalia Basin are, like the units of the Arunta Block, collectively included in the hydrogeological unit ‘basement’.

Basins

The borefield and processing site are located within the Southern Basins and the borefield targets the Reaphook Palaeochannel. Groundwater within the basins generally flows westwards. The mine site and accommodation village are within the Ti-Tree Basin which also comprises palaeochannels. The divide between the basins is located at the ridge line to the north of the processing site.

Riparian Vegetation

Riparian vegetation (dominated by Eucalyptus camaldulensis, colloquially referred to as Red River Gums line the larger creeks and rivers across the region. Napperby Creek, Day Creek and Woodforde

River are generally dominated by Red River Gums. The vegetation has the potential to be groundwater dependent and utilise groundwater from depth (approximately 15 m BGL).

Declaration of Beneficial Uses

The mine site is located within the Ti-Tree Water Control District (WCD) within the Western Zone. WCDs are proclaimed arears where the Department of Land Resource Management (DLRM) have identified a need to manage water resources (surface and groundwater) to avoid stressing groundwater reserves, river flows or wetlands. DLRM manage the Ti-Tree WCD.

The Ti-Tree WCD covers approximately 14,000 km^2 within which the majority of water supplies for the 1000 people are sourced from groundwater. It was initially declared in 1983 and groundwater in the WCD continues to be utilised for agriculture, horticulture and stock and domestic water use (DLRM, 2016).

Mining is exempt from licencing under the *Water Act*. However, extraction and dewatering activities are governed under the *Mining Management Act* administered by the Department of Mines and Energy (DME) who have a memorandum of understanding with the DLRM to manage activities so they do not affect other water users.

The processing plant and accommodation village is outside the Ti-Tree Water Control District.

4.2 Groundwater Modelling

4.2.1 Model

A class 1 numerical groundwater model was developed using graphical user interface (GUI) GMS 10.1, MODFLOW-NWT with Upstream Weighting (UPW) and Newton (NWT) solver, Evaporation package (EVT1), General Head Boundary package (GHB1), Drains (DRN1) package, Recharge (RCH1) package and automated Parameter Estimation (PEST) packages. The model simulates aquifer conditions to develop a steady-state (existing) and predicts impacts from Project operations including 100 and 1000 years post closure.

The model was run for a 41 year mining operation using annualised groundwater extraction rate of 4.5 GL/year.

4.2.2 Groundwater System

The model used assumed rates for groundwater extraction stock bores, Ti-Tree Basin bore pumping for horticultural irrigation and community pumping (Pmara Jutunta, Alyuen and Laramba). The Project extractions included the development of a pit with associated pit dewatering and production bores in the borefield. Tertiary stresses included recharge (diffuse and direct) and evapotranspiration.

4.2.3 Groundwater Flow Regimes

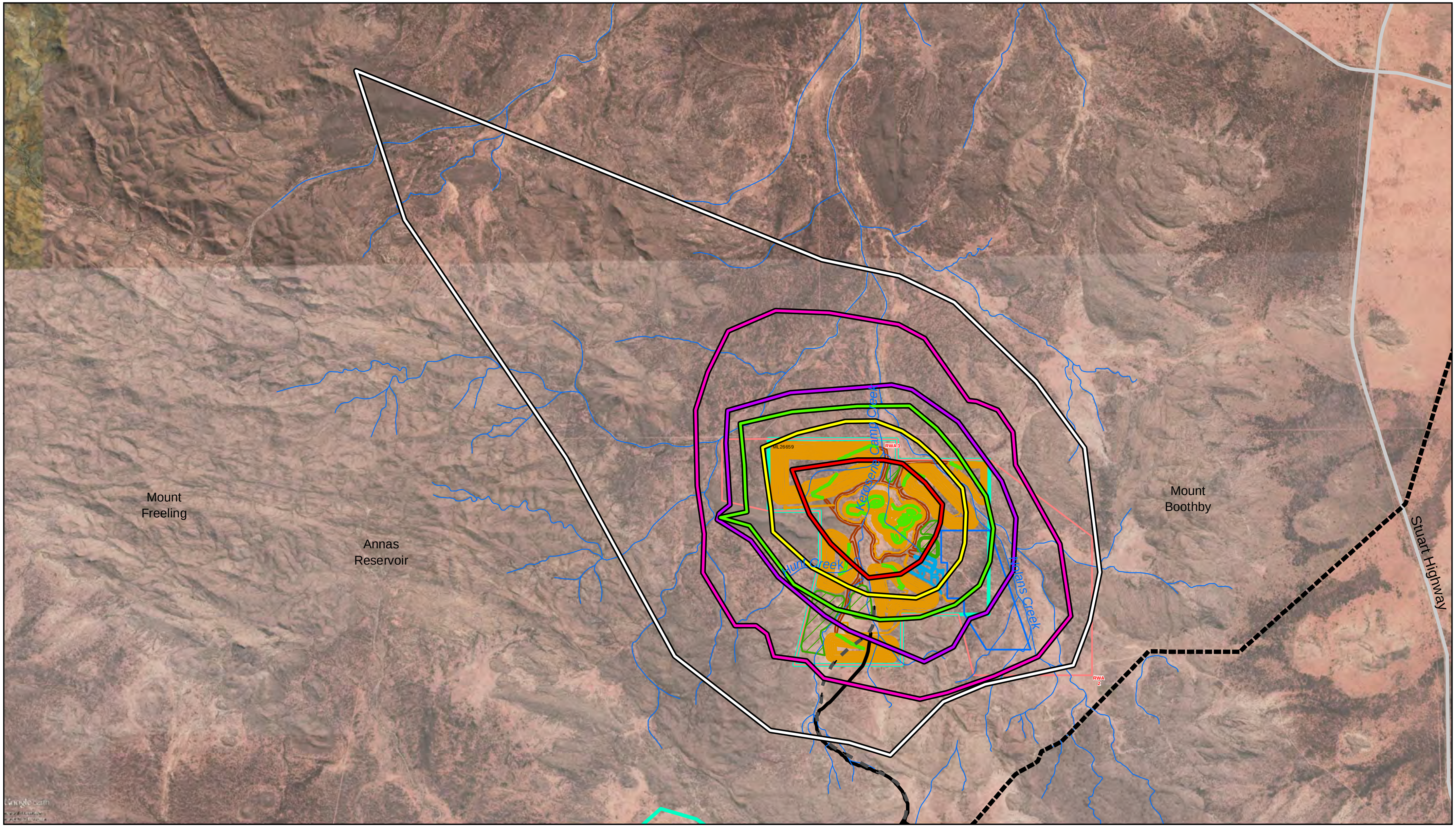
Regional

The model indicates groundwater flow at a regional scale displays no change. The Southern Basin continues to flow to the west and the Ti-Tree Basin continues to flow to the northeast.

Mine Pit Inflow

Immediately adjacent to the pit groundwater flow direction reverses and flows report to the pit. The extent of the groundwater reversal increases through the LOM due to the increase in depth of the pit (and required dewatering). At the modelled LOM, the majority of the groundwater beneath the mine site will return to the pit. Following closure, the extent of groundwater reversal continues to extend as the pit operates as a groundwater sink (i.e. evaporation losses exceed inputs from runoff, precipitation and groundwater inflows). The 100 year and 1,000 year post closure pit inflow contour extends beyond the Project lease boundary. The extent and associated periods for pit inflow contours are provided in Figure 4-1.

If mining were to cease prior to reaching the LOM pit shell, the extent of the groundwater reversal would still continue to laterally extend beyond the Project area due to the groundwater inflow to the pit and evaporation from the system (which exceeds precipitation)..



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0 1 2 3 4

Kilometres

Map Projection: Universal Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 53

N

LEGEND

- 2028: 10 years
- 2038: 20 years
- 2048: 30 years
- 2058: life of mine
- 2160: 100 years post closure
- 3060: 1000 years post closure
- Powerline
- Waterways
- Roads
- Haul Road/Access Tracks
- Proposed Mine Site Boundary
- Mineral Lease

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Pit Groundwater Inflow

Figure 4-1

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Level 5 66 Smith Street Darwin NT 0800 Australia T 61 8 8982 0100 F 61 8 8981 1075 E drwmail@ghd.com W www.ghd.com

Data source: Google Earth Pro - Imagery (Date extracted: 08/03/2016). ARL - Proposed Pipelines, Proposed Mine Site, Proposed Diversion Channel Options, Tailings Storage Facility (2015). Created by: CM

4.2.4 Drawdown

Mine Site

The drawdown of groundwater levels adjacent to the pit are due to the removal of groundwater from the system through evaporation and pit dewatering. The drawdown continues to laterally extend following the cessation of mining due groundwater inflow to the pit and evaporation from the system (which exceeds precipitation). Peak pit groundwater inflows are approximately 4,000 m^3/day (45 L/s) and post closure are 700 m^3/day (8 L/s).

The modelled 1m drawdown contour extends beyond the Project lease boundary approximately 500m at the LOM, 750m at 100 years post closure and 1.5 km at 1,000 years post closure. The drawdown contours are provided in the Hydrogeological Report (EIS, Appendix K).

Borefield

The extent of drawdown at the borefield is greater than the mine site which is generally due to the volume of water being removed from the aquifer. The drawdown at the epicentre of the borefield is 7 m (at SB022) excluding actual drawdown within operating bores. The drawdown-rebound is a typical log-linear response to aquifer pumping and recovers to 1 m drawdown approximately 100 years post closure.

4.2.5 Modelled Impacts

The hydrogeological model indicates that there will be drawdown due to pit dewatering and extraction at the borefield. Levels of drawdown range are dependent upon distance from the extraction site. The borefield generally recovers across the closure period but the drawdown associated with the pit continues to expand post closure due to pit acting as a groundwater sink. A summary of predicted impacts is provided in Figure 4-1.

Table 4-1 Summary of Modelled Impacts

Area	Impact
Mine Site	
Riparian Vegetation	The riparian vegetation immediately adjacent to the mine area (upstream to the point of the diversion and downstream in Kerosene Camp Creek to the confluence of Nolans Creek) is highly likely to be catastrophically impacted by the mining operations (i.e. riparian vegetation will die and not recolonise in the area).
Groundwater Users	Pine Hill Station bores (RN010759 and RN012624) are likely to experience an increased drawdown to 0.05 m by the end of mining and 0.1 m 1,000 year post closure. Groundwater availability for drinking water, stock, horticulture and viticulture within the Ti-Tree Basin is highly unlikely to be measurably impacted (i.e. less than 0.012 m predicted drawdown). Drawdown at the Aileron Station (homestead and roadhouse locations) is predicted to be impacted by mine dewatering and remain impacted beyond mine closure. Drawdown is likely to commence following the end of mining increasing from 0 m in approximately 2291 to about 0.7 m 1,000 years post closure. However, the water supply for Aileron Station is from the Sothern Basins and is detailed below.
Southern Basins	
Riparian Vegetation	Modelled drawdown from the borefield peaks in the order of 1.5 m in the vicinity of Day Creek during extraction. The drawdown rebounds rapidly once extraction ceases. The depth to groundwater currently is generally 20 m BGL and it is considered likely that current vegetation would be capable of extending root systems during the extraction period. Napperby Creek is approximately 18 km further west than Day Creek is from the borefield and as such drawdown is significantly reduced. Drawdown peaks at 0.7 m by the end of mining and recovers to 0.1 m 1,000 years post closure. The predicted drawdowns are negligible in the Lake Lewis area and are not likely to be measureable. However, the peak decreases in groundwater availability for evapotranspiration in the Lake Lewis area of the Southern Basins is 3% or 712 m^3/day (8 L/s) and rebounds to approach steady state levels in the order of approximately

Area	Impact
	0.5 % or 103 m^3/day (1 L/s).
Groundwater Users	The old Alyuen Community water supply bore may be impacted by groundwater extraction from the mine site. Alyuen Community is currently supplied from a new bore to the east of the project borefield which will not be impacted by the Project's development. Drawdown is likely to peak at the end of mining by 0.6 m and decrease to 0.4 m 1,000 years post closure. Laramba and Napperby groundwater supply (drinking water) situated on the western side of Day Creek north of the Reaphook Range is expected to experience a peak drawdown of 1.5 m at the end of mining and decrease to 0.1 m 1,000 years post closure.

4.2.6 Validation

The model requires temporal monitoring and flow gauges to be installed on extraction bores to facilitate validation and recalibrating of the model during Project operations. The monitoring program detailed in Section 6.3 has been designed to provide sufficient information to progress this model to a class 2 or class 3 in accordance with the Australian Groundwater Modelling Guidelines (Barnett et al, 2012).

4.3 Groundwater Quality

Groundwater sampling has been undertaken across the Project including 158 samples from a total of 71 bores. Samples were obtained using numerous opportunistic methods including but not limited to, during airlifts, through the use of existing infrastructure (submersibles, outlets and taps) as well as specific sampling from depth.

4.3.1 Electrical Conductivity

Groundwater chemistry has been sampled from several different aquifer types across the Southern Basins and Ti-Tree Basin. Electrical Conductivity (EC) measurements have been used to assess the quality of groundwater across the areas. EC is an indirect measurement of salt content and within the Project area is currently likely to be influenced predominately by:

- Chloride and sodium ions followed by carbonates, sulphates, magnesium, calcium and potassium;
- Land Use (irrigation);
- Interaction of basement aquifer (old groundwater) with overlying aquifer (fresh groundwater); and
- Mineralisation (i.e. heavy metals due to metallic ions).

Median values of EC range from approximately 900 to 3,700 µS/cm across the Project area. In general, EC values are higher at the mine site and processing site relative to the borefield as displayed in the minimum and maximum EC values in Figure 4-2 and Figure 4-3 respectively.

4.3.2 General Chemistry

Groundwater chemistry across the Project varies and is dependent upon the geological unit being screened. Initial groundwater chemistry indicates levels generally exceed stock water trigger values in certain analytes across the hydrogeological study area with the exception of the Southern Basins alluvial and calcrete screen bores which exceed drinking water trigger values. Stock water trigger values are the least conservative trigger values (i.e. the guideline/trigger values are generally orders of magnitude greater than drinking water trigger values).

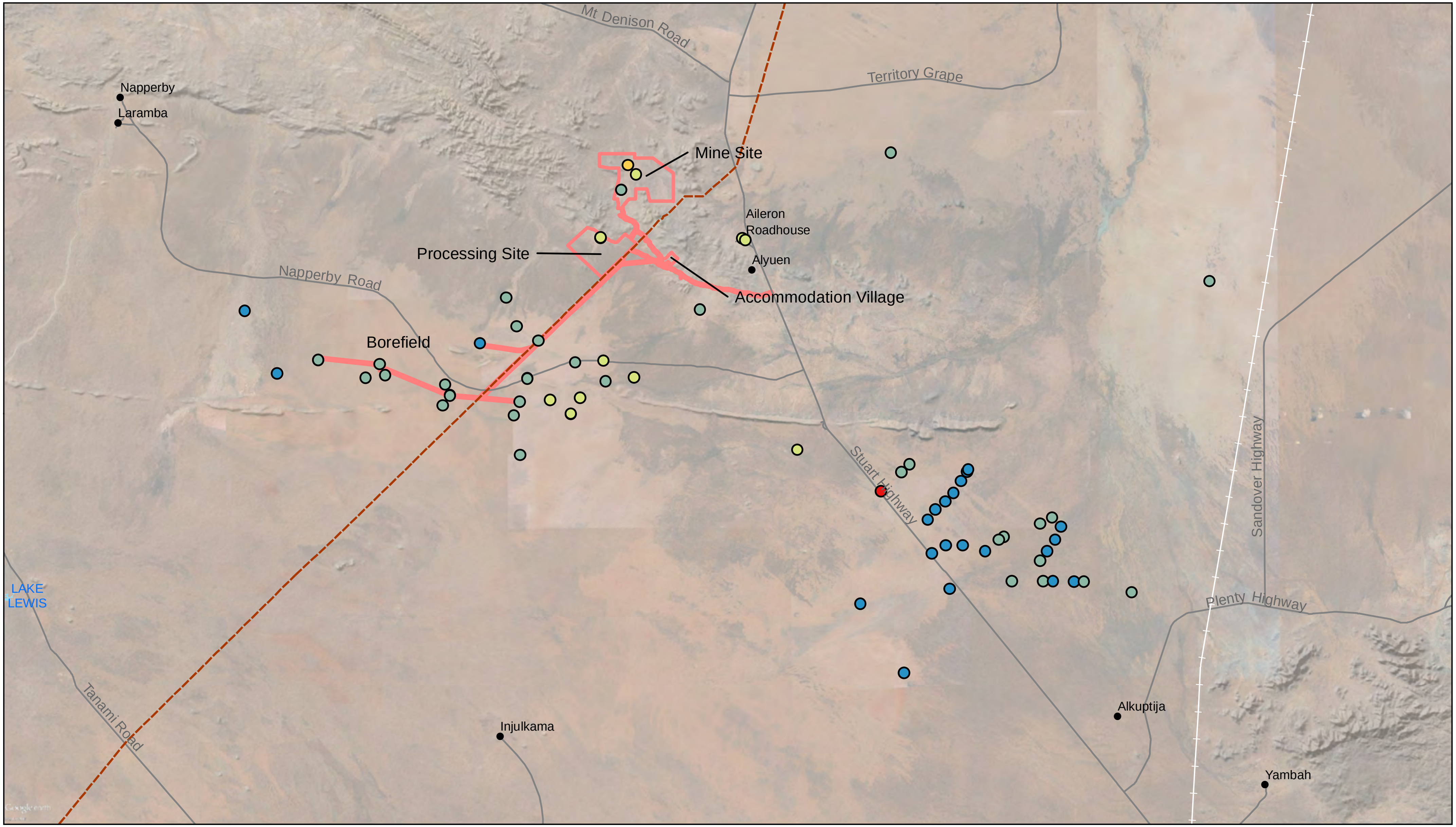
A summary providing an initial overview of current groundwater uses and analytes which exceeded associated trigger values are provided in Table 4-2 and additional data is located within the

Hydrogeological Report (EIS, Appendix K). Additional groundwater chemistry will be collected as part of this WMP and monitoring is detailed in Section 6.3.

Table 4-2 Chemistry Summary

Area	Current Groundwater Use			Exceedances
	Stock Water[1]	Drinking Water	Irrigation	
Mine Site	✓			Stock water exceedances of uranium, fluorine and mercury.
Ti-Tree Basin	✓	✓	✓	Drinking water exceedances of total dissolved solids, chlorine, sodium, uranium and nitrate. Irrigation exceedances of boron, uranium and fluorine.
Southern Basins (Basement)	✓			Stock water exceedances of uranium, fluorine, sulphate, selenium and total dissolve solids.
Southern Basins (Alluvial and Calcrete)	✓	✓		Drinking water exceedances of total dissolved solids, high pH, chlorine, sodium, uranium, sulphate, manganese, aluminium, nitrate, fluoride and selenium.
Southern Basins (Rheapook Channel)	✓	✓		Stock water exceedances of fluorine and selenium. Drinking water exceedances of total dissolved solids, high pH, chlorine, sodium, uranium, iron, sulphate, lead, antimony, nitrogen dioxide, fluoride and selenium.

Note: [1] ANZECC & ARCANZ (2000) Stock Watering Trigger Values.
[2] Australian Drinking Water Guidelines (ADWG 2011) Aesthetic and Health guideline.
[3] ANZECC & ARCANZ (2000) Irrigation - Long-term Trigger Values.



1:500,000 @ A4

0 5 10 15 20

Kilometres

Map Projection: Universal Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 53

N

LEGEND

Electrical Conductivity (uS/cm)

- 0 - 1500
- 1501 - 3000
- 3001 - 4500
- 4501 - 6000
- 6001 - 7500
- 7501 - 13500
- Gas Pipeline
- Major Roads
- Project Areas
- Waterbodies

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Nolans Project

Job Number	4322301
Revision	0
Date	29 Apr 2016

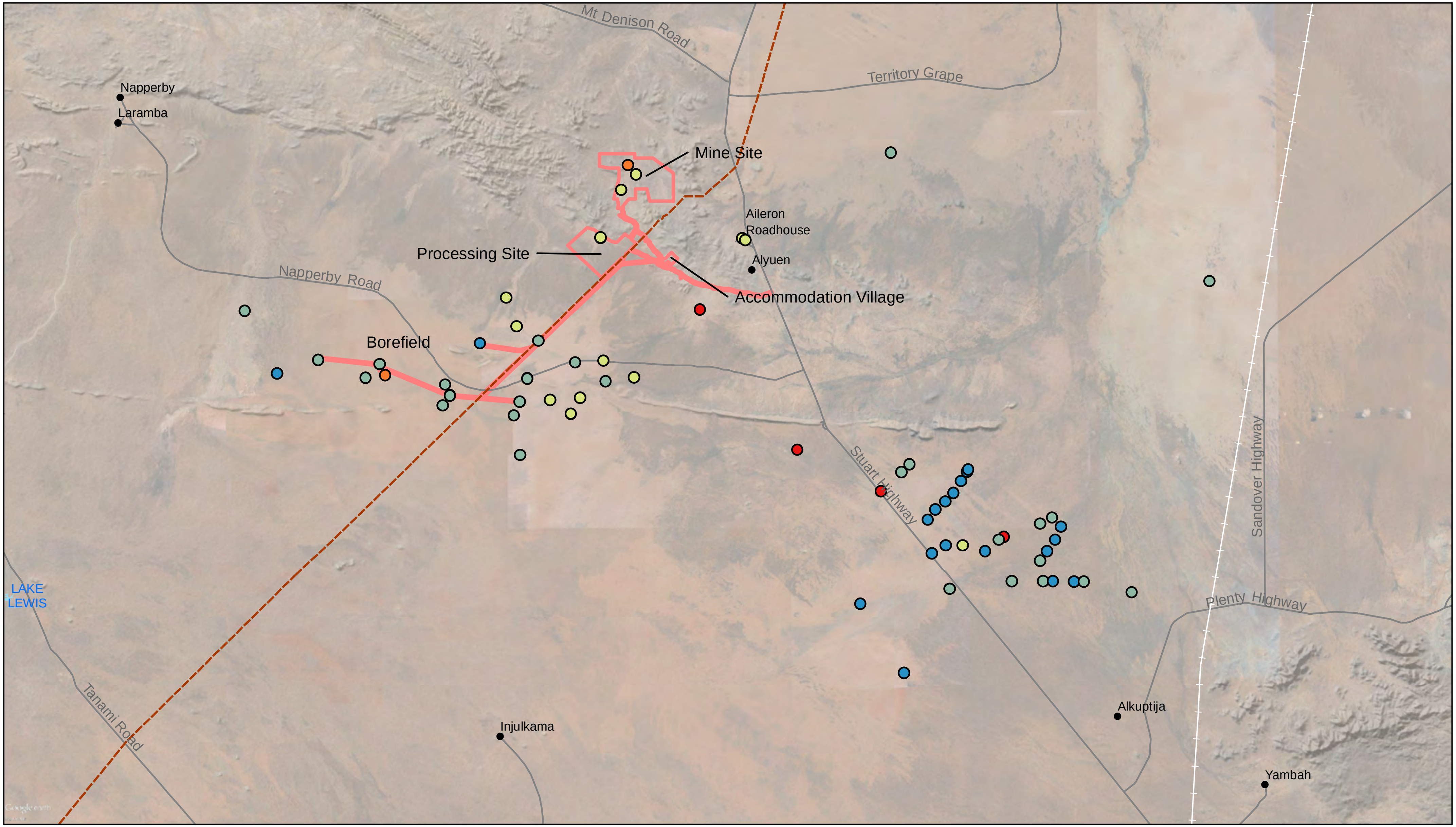
Electrical Conductivity (Min) of Groundwater Samples

Figure 4-2

G:\43\22301\GIS\Maps\4322301_1014_GroundSamplesElectricalConductivity.mxd

Level 5 66 Smith Street Darwin NT 0800 Australia T 61 8 8982 0100 F 61 8 8981 1075 E drwmail@ghd.com W www.ghd.com

Data source: Google Earth Pro - Imagery (Date extracted: 04/02/2015). Ride - Groundwater Data (2015). ARL - Project Areas (2015). GA - Placenames, Railways, Gas Pipeline, Major Roads, Waterbodies (2015). Created by: CM



1:500,000 @ A4

0 5 10 15 20

Kilometres

Map Projection: Universal Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 53

N

LEGEND

Electrical Conductivity (uS/cm)

- 0 - 1500
- 1501 - 3000
- 3001 - 4500
- 4501 - 6000
- 6001 - 7500
- 7501 - 13500
- Gas Pipeline
- Major Roads
- Project Areas
- Waterbodies

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Nolans Project

Job Number 4322301
Revision 0
Date 29 Apr 2016

Electrical Conductivity (Max) of Groundwater Samples

Figure 4-3

G:\43\22301\GIS\Maps\4322301_1015_GroundSamplesElectricalConductivityMax.mxd

Level 5 66 Smith Street Darwin NT 0800 Australia **T** 61 8 8982 0100 **F** 61 8 8981 1075 **E** drwmail@ghd.com **W** www.ghd.com

Data source: Google Earth Pro - Imagery (Date extracted: 04/02/2015). Ride - Groundwater Data (2015). ARL - Project Areas (2015). GA - Placenames, Railways, Gas Pipeline, Major Roads, Waterbodies (2015). Created by: CM

5. Preliminary Conceptual Site Model

5.1 General

A Conceptual Site Model (CSM) is a representation of site-related information regarding potential surface and groundwater impacts, receptors and potential exposure pathways.

The development of a preliminary CSM provides the framework of identifying potential source-pathway-receptor linkages and associated monitoring techniques. Once detailed site specific information (monitoring data) is evaluated, the CSM will be refined and used as a decision tool to identify requirements for contingency management measures.

5.2 Key Aspects

5.2.1 Surface Watercourses

Watercourses are generally dry and do no exhibit flows for the majority of the year. Flows only occur during exceptional rainfall events and can flood existing watercourse banks. No watercourses are present within 1 km of the processing site or accommodation village.

5.2.2 Monitoring Benchmarks

Groundwater chemistry generally exceeds assigned environmental values (stock water, irrigation water and drinking water ANZECC values). Site Specific Groundwater Trigger Values (SSGTV) will be established throughout the construction phase (24 months) at nested wells at the mine site and the processing site. The SSGTV will be used to provide a definitive dataset to be assessed against during operation, care and maintenance and/or rehabilitation.

5.2.3 Mine Pit Groundwater Inflow

The excavation of the pit below the groundwater table will commence the reversal of the groundwater flow direction causing it to report into the pit. The extent of the pit inflow contour laterally extends beyond all LOM mine site infrastructure 100 years post closure and continues to laterally extend to approximately 4 km radially from the pit 1000 years post closure. However, it should be noted that at any time leading up to the end of the 41 year operational life of the Project, there is potential for any seepage from the FTSF and/or WRDs to discharge away from the mine site.

Post closure the pit will act as a terminal groundwater sink (evaporation losses exceed inputs from runoff, precipitation and groundwater inflows). As a result, the pit water is expected to show continual increases in acidity, metals and salt concentrations over time through accumulation of solutes introduced via groundwater inflows, run-off and precipitation.

5.2.4 Nil Discharge

The mine site and processing site have been designed as a no discharge site, however during heavy rainfall events and under exceptional circumstances there is the potential for stormwater basins discharging. AMD potential is considered low and during these events the considerable dilution within adjacent creeks will substantially reduce potential contaminant concentrations. Sampling would be undertaken in accordance with the Discharges / Emergency Overflows procedure within the Surface Water Sampling Procedure in Appendix A. The sampling data will be used to assess if any additional management measures are required.

5.2.5 Borefield

The processing site is considered to pose a risk to groundwater contamination. During operation of the borefield, drawdown has the potential to impact local groundwater flow direction. It is hypothesised that groundwater beneath the processing site could ultimately end up within the borefield. However, following closure and recovery of the borefield the potential fate of groundwater from the processing site will be validated following recalibration of the model using monitoring data from operations. Given the distance of the borefield from the processing site it is predicted that any potential impacts would be very gradual. Monitoring near the processing site will allow early detection and mitigation measures to minimise potential impacts

Infrastructure at the processing site with the potential of being a groundwater contamination source includes seepage collection systems and impermeable bases ($1x10^{-8}$ m/s).

5.3 Potential Water Impacts

5.3.1 Contamination Sources

The Project has four key areas including the mine site, processing site, borefield and accommodation village of which two are considered as potential contamination sources. The CSM has been developed based on known and potential contamination sources during operation (i.e. the highest risk period).

Table 5-1 Potential Sources of Contamination

Source / Area	Contaminant of Potential Concern (CoPC)
Mine Site	
Mine Pit Walls	Metals, radioactive and rare earth elements, pH, electrical conductivity and sulfate.
ROM Pad	
Low grade Ore Stockpile	
WRDs	
FTSF	
Fuel Farm (diesel / petrol storage)	Metals (including cadmium, chromium, copper, lead and nickel) and hydrocarbons (BTEX, BaP, total PAH and TRH).
Landfill	Metals (including arsenic, cadmium, chromium, copper, lead, mercury, nickel and zinc).
Processing Site	
Evaporation Ponds	Metals, radioactive and rare earth elements, pH, electrical conductivity and sulfate.
Residue Storage Facility	
Fuel Farm	Metals (including cadmium, chromium, copper, lead and nickel) and hydrocarbons (BTEX, BaP, total PAH and TRH).
Hazardous Substance Storage	Sulphuric acid, hydrochloric acid, sodium hydroxide (caustic soda), sodium carbonate, carbonate, lime, barium chloride and hydrocarbons (BTEX, BaP, total PAH and TRH).

Note: BaP: Benzo[a]pyrene
BTEX: Benzene, Toluene, Ethylbenzene and Xylenes
TRH: Total Recoverable Hydrocarbon
PAH: Polycyclic Aromatic hydrocarbons

5.4 Potential Receptors

5.4.1 Current Land Use

The Project is currently utilised as a pastoral lease for cattle grazing with stockwater bores installed across the area. The mine site is situated within the Ti-Tree Water Control District which is used for irrigation, stockwater and drinking water. Power Water Corporation (PWC) drinking water supply for the Laramba community and Napperby Station is located west of the of the Project's borefield within the Southern Basins. Therefore, the current receptors are considered to be:

- Human Health:
  - Drinking water from Ti-Tree and Southern Basins.
- Environment:

- Irrigation and stockwater within the Ti-Tree and Southern Basins, groundwater recharge of Lake Lewis and overland flows into the Woodford River; and
- Potential Groundwater Dependent Ecosystems (GDE) at Napperby Creek, Day Creek and Woodforde River.

5.4.2 Proposed Land Use

The proposed use of the site is to construct and operate a mine and processing site. The mine will involve the mining of an open pit and construction of associated infrastructure. The processing site will require the construction of RSFs, EPs and power station. The potential receptors during the operation will be generally consistent with the current land use:

- Human Health:
  - Drinking water from Ti-Tree and Southern Basins.
- Environment:
  - Irrigation and stockwater within the Ti-Tree and Southern Basins, groundwater recharge of Lake Lewis and overland flows into the Woodford River; and
  - Potential Groundwater Dependent Ecosystems (GDE) at Napperby Creek, Day Creek and Woodforde River.

5.4.3 Future Land Use

Following the 2 year construction and 41 years of operation, the Project area will be rehabilitated and closed. At closure the Project area is considered likely to return to the original land use as a pastoral lease for cattle production. Water use at and surrounding the Project is likely to continue to be required for use as drinking water, irrigation and stockwater.

5.5 Potential Operation Source-Pathway-Receptor Linkages

Preliminary source, pathway and receptor linkages has been developed to establish potential linkages and is detailed below. A summary of potential source, pathway and receptor linkages is provided in Table 5-2. Monitoring will be undertaken to assess potential linkages and to inform additional management measures. The monitoring plan is provided in Section 6.

Table 5-2 Conceptual Site Model Summary

Source	Pathway	Receptor	Possible Link and Associated Management Measure(s)	Monitoring
Mine Site				

Source	Pathway	Receptor	Possible Link and Associated Management Measure(s)	Monitoring
Open Pit	Vertical migration of pit lake water into saturated zone and horizontal migration.	Woodforde River (via Nolans Creek, Schafer Creek or Hunt Creek) Ti-Tree Basin Pastoral Bores	Unlikely The pit will develop a cone of depression through operations, pit groundwater inflow and surface water captured in the pit will not be able to migrate. In addition, water contained within the pit will be pumped to be used within the Processing water circuit or to the FTSF. Post closure the pit will act as a terminal groundwater sink (evaporation losses exceed inputs from runoff, precipitation and groundwater inflows) which will ultimately collect any seepage from mine site infrastructure. In this situation, groundwater will not migrate from the pit.	Nested monitoring bores with quarterly sampling to assess potential impact against the groundwater baseline and up-gradient monitoring wells.
ROM Pad Low grade Ore Stockpile	Overland flow from stockpile bases entering Nolans Creek and ultimately infiltrating into the aquifer. Vertical migration through unsaturated zone into saturated zone and horizontal migration.	Woodforde River (via Nolans Creek, Ti-Tree Basin	Unlikely The ROM Pad and Stockpile Pad will be constructed with impermeable bases (1x10-8 m/s) with surface drainage captured within stormwater retention ponds. ROM Pad and Stockpile seepage to groundwater will be collected into the pit following approximately 15 years of operation (Figure 4-1).	Surface water sampling within watercourses during flow events. Sampling of discharges in accordance with the Emergency Overflow Procedure.
FTSF	Overflow from structure and entering Nolans Creek and ultimately infiltrating into the aquifer. Vertical migration through unsaturated zone into saturated zone and horizontal migration.		Unlikely FTSF constructed with capacity to capture a 100 year 72 hour ARI plus PMP 72 hour event. Contingency measures will be available to transfer excess water to alternative storage/pit. FTSF constructed with low permeability soil liner and seepage collection system. FTSF seepage to groundwater will be collected into the pit following approximately 100 years post closure (Figure 4-1).	Surface water sampling within watercourses during flow events. Sampling of discharges in accordance with the Emergency Overflow Procedure. Nested monitoring bores surrounding the FTSF with quarterly sampling to assess potential impact against the groundwater baseline and up-gradient monitoring wells.

Source	Pathway	Receptor	Possible Link and Associated Management Measure(s)	Monitoring
WRDs	Surface flow from bases entering adjacent creek system and ultimately infiltrating into the aquifer. Vertical migration through unsaturated zone into saturated zone and horizontal migration.		Possible Excess surface flows to report to stormwater retention ponds. It is unlikely that the WRDs will seep to groundwater because of the climatic conditions at site. If seepage does occur it will flow into the pit following approximately 30 years of operation with the exception of WRD 5. Seepage from WRD 5 will be collected into collection sumps pit following approximately 100 year post closure (Figure 4-1).	Surface water sampling within watercourses during flow events. Sampling of discharges in accordance with the Emergency Overflow Procedure. Nested monitoring bores near WRDs with quarterly sampling to assess potential impact against the groundwater baseline and up-gradient monitoring wells.
Fuel Farm	Vertical migration through unsaturated zone into saturated zone and horizontal migration.		Improbable Fuel stored in self-bunded Above Ground Storage Tanks (ASTs) and Fuel Inventory (Loss Management) monitoring in accordance with the Hazardous Substances Management Plan.	Spills or sabotage will be managed in accordance with the Emergency Response Management Plan (ERMP) including the Environmental Investigation Procedure.
Landfill	Seepage through the landfill vertically migrating through the unsaturated zone into the saturated zone and horizontal migration.		Unlikely No hazardous or toxic substances will be landfilled at the Project.	*tbc (dependent upon landfill location)*
Processing Site				
Evaporation Ponds Residue Storage Facility	Overflow flowing south and ultimately infiltrating into the aquifer. Vertical migration through unsaturated zone into saturated zone and horizontal migration.	Southern Basins Napperby Creek, Day Creek Lake Lewis (recharge)	Unlikely RSF to have a high density polyethylene (HDPE)/low permeability soil liner system, combined with basin drainage and a leakage collection and recovery system. EPs to have a HDPE liner. RSF and EPs constructed with capacity to capture a 100 year 72 hour ARI plus PMP 72 hour event. Contingency measures will be available to transfer excess water to alternative storage/pit.	Nested monitoring bores near the processing site including up-gradient, adjacent and down-gradient bores with quarterly sampling to assess potential impact against the groundwater baseline and up-gradient monitoring wells.
Fuel Farm	Spill or leak from structure flowing off site and ultimately infiltrating into the aquifer. Vertical migration through unsaturated zone into saturated zone		Improbable Fuel stored in self-bunded Above Ground Storage Tanks (ASTs) and Fuel Inventory (Loss Management) monitoring in accordance with the Hazardous Substances Management Plan.	Spills or sabotage will be managed in accordance with the Emergency Reponses Management Plan (ERMP) including the Environmental Investigation Procedure.

Source	Pathway	Receptor	Possible Link and Associated Management Measure(s)	Monitoring
Hazardous Substance Storage	and horizontal migration.		Unlikely Hazardous substances stored in a Chemical Storage Shed with internal bunding.	

Note: Scales of Likelihood include improbable, unlikely, possible and probable.

6. Monitoring Program

6.1 Multiple Before-After Control-Impact

Monitoring will be undertaken in accordance with the Multiple Before-After Control-Impact (MBACI) approach due to the large scale and potential for permanent and/or long term water related environmental impacts. The monitoring program has been designed to include:

- Control Sites: upstream / up gradient monitoring sites which monitor background concentrations. Multiple control sites will be utilised.
- Adjacent: monitoring points situated adjacent to potential point sources of contamination (i.e. locations storing Impacted Water, Processing Water), often called 'point of discharge'.
- Impact Site: downstream / downgradient monitoring sites. Multiple impact sites will be utilised.

All monitoring sites will be located and installed in locations where the future Project footprint will enable monitoring consistency throughout the Project life..

6.1.1 Sampling Periods

Water quality monitoring combines surface water, sediment and groundwater which will occur upstream/up gradient, adjacent and downstream/downgradient of the Project. The construction period will be used as the baseline period to capture a robust groundwater, sediment and surface water (where available) dataset.

The basis of each phase of monitoring is provided in Table 6-1.

Table 6-1 Sampling Period and Basis

Sampling Period	Duration	Basis
Baseline	30 month	Establish existing conditions at the Project for surface water, sediment and groundwater. Baseline period monitoring will be used to establish a definitive dataset from which potential impacts can be assessed during operation, care and maintenance and/or rehabilitation.
Operation	41 years	Assess monitoring data against baseline to determine if an impact has or is occurring. If significant differences between baseline and operation monitoring periods occur further management measures will be investigated and implemented as required.
Care and Maintenance	-	Assess Project impact to the surrounding environment through care and maintenance activities (i.e. minimal activities and/or management occurring). If significant differences between baseline and care and maintenance monitoring periods occur further management measures will be investigated and implemented as required.
Rehabilitation	-	Utilise baseline sampling data as the ultimate rehabilitation goal for groundwater, sediment and surface water.

6.1 Monitoring Summary

The monitoring program has been designed to capture both baseline conditions and assess potential impacts from the operation. Monitoring will ultimately be utilised to assess if the Project is impacting the surrounding environment and to inform rehabilitation goals. A summary of monitoring program is provided in Table 6-2 and Table 6-3.

The monitoring locations will be updated following finalisation of the mine, processing site and borefield design.

Table 6-2 Baseline Monitoring

Monitoring	Number of Locations	Matrix	Frequency / Date	
			Field Measurements	Field and Laboratory
Mine Site				
Surface Water	*tbc*	Water	-	Early flows and late flows#
Groundwater	*tbc*	Water	Biannual	Quarterly
Groundwater – up-gradient boundary bores MB101A and MB101B*	*tbc*	Water	Monthly	Monthly
Sediment	*tbc*	Sediment	-	Annually
Photopoint Monitoring	*tbc*	n/a	Six Monthly	-
Processing Site				
Groundwater	*tbc*	Water	Biannual	Quarterly
Groundwater – up-gradient boundary bores MB201A and MB201B*	*tbc*	Water	Monthly	Monthly
Borefield				
Groundwater	*tbc*	Water	Quarterly	Biannual
Production Wells	*tbc*	Water	Quarterly	Biannual

Note: # Due to the flow characteristics of the adjacent waterways sampling has been scheduled to be undertaken during flow events where a minimum of 0.1m flowing water is present.
* The groundwater up-gradient boundary bore will be monitored monthly throughout the 30 month baseline period to collate sufficient data to determine site specific trigger values for the mine site and processing site.

Table 6-3 Operational Monitoring

Monitoring	Number of Locations	Matrix	Frequency / Date	
			Field Measurements	Field and Laboratory
Mine Site				
Surface Water	*tbc*	Water	-	Early flows and late flows#
Stormwater Retention Ponds	*tbc*	Water	Monthly	Quarterly
Groundwater	*tbc*	Water	Quarterly	Quarterly
Sediment	*tbc*	Sediment	-	Annually
Photopoint Monitoring	*tbc*	n/a	Annual	-
Open Pit	*tbc*	Water	Monthly	Quarterly
Flotation Tailings Storage Facility	*tbc*	Water	Monthly	Quarterly
Processing Site				
Groundwater	*tbc*	Water	Quarterly	Quarterly
Stormwater Retention Ponds	*tbc*	Water	Monthly	Quarterly
Residue Storage Facilities	*tbc*	Water	Monthly	Quarterly
Evaporation Ponds	*tbc*	Water	Monthly	Quarterly
Borefield				
Groundwater	*tbc*	Water	Monthly	Quarterly
Accommodation Village				
Stormwater Retention Ponds	*tbc*	Water	Quarterly	Biannual

Note: # Due to the flow characteristics of the adjacent waterways sampling has been scheduled to be undertaken during flow events where a minimum of 0.1m flowing water is present.
***** Current design is being finalised, sampling will include one sample per cell.

6.2 Surface Water

6.2.1 Sampling Locations

The Project has been designed as a nil discharge site with any potential discharge from the Project considered to stay within the local vicinity of the Project footprint and infiltrating to groundwater.

Surface water sampling locations will be established across the Mine Site during the baseline period. The locations will be positioned within pre-existing waterways with control (upstream), adjacent and impact (downstream) sample locations.

The processing site is located on an alluvial fan in the upper catchment of the north eastern section of the Southern Basins and as such there are limited watercourses that can be monitored.

Stormwater sediment retention basins will be installed across all Project areas, the potential locations of stormwater basins are provided with in the Erosion and Sediment Control Plan. However, this is subject to change through the detailed design phase.

Samples will also be collected from the pit, FTSF, RSF and EPs. These samples will be used to characterise the water quality from these facilities and will be used as reference when assessing potential impacts to groundwater and/or unexpected overflows.

A summary of surface water sampling locations and sampling frequencies are provided in Table 6-4 and illustrated on Figure 6-1. The surface water sampling procedures including sampling to be undertaken in the case of an uncontrolled discharge are provided in the Surface Water Sampling Procedure (Appendix A).

Table 6-4 Surface Water Sampling Locations

Site ID	Coordinates (GDA94 Zone 53)		Type	Description	Sample Frequency	
	Easting	Northing			Baseline	Operation
Surface Water						
tbc	*tbc*	*tbc*	*tbc*	*tbc*	Early and late flows with a minimum of 0.1m flowing water.	
tbc	*tbc*	*tbc*	*tbc*	*tbc*		
Stormwater Sediment Retention Ponds						
tbc	*tbc*	*tbc*	*tbc*	*tbc*	-	Field measurements monthly Field and laboratory suite quarterly.
tbc	*tbc*	*tbc*	*tbc*	*tbc*		
Mine Pit and Processing Infrastructure						
Mine pit Sump	*tbc*	*tbc*	-	Mine pit sump water (i.e. groundwater inflow).	-	Field measurements monthly Field and laboratory suite quarterly.
Flotation Tailings Storage Facility	`	*tbc*	-	Flotation Storage Facility ponded water.		
Residue Storage Facilities 1	*tbc*	*tbc*	-	Residue Storage Facility ponded water.		
Residue Storage Facilities 2	*tbc*	*tbc*	-	Residue Storage Facility ponded water.		
Evaporation Pond 1	*tbc*	*tbc*	-	Evaporation Pond water.		
Evaporation Pond 2	*tbc*	*tbc*	-	Evaporation Pond water.		
Evaporation Pond 3	*tbc*	*tbc*	-	Evaporation Pond water.		

6.2.2 Sample Assay Suite

The sampling suite for surface water quality includes either field measurement or field and laboratory measurements. The suites are summarised as follows:

- Field Measurements

  Temperature, pH, Electrical Conductivity, Total Dissolved Solids, Turbidity and Oxidation Reduction Potential. Depth at location and photopoint monitoring (photos of sample location, upstream and downstream).

- Field and Laboratory Measurements

  In field: Temperature, pH, Electrical Conductivity, Total Dissolved Solids, Turbidity and Oxidation Reduction Potential. Depth at location and photopoint monitoring (photos of sample location, upstream and downstream).

  Laboratory analysis:

  – Total suspended solids (TSS);
  – Total hardness;
  – Total acidity and alkalinity;
  – Major ions ($CaCO_3$, CO_3, HCO_3, Ca, Mg, K, Na, Cl, SO_4, NO_3, NO_2);
  – Metals total and dissolved (0.45 µm field filtered[2]): Al, As, B, Ba, Cd, Co, Cu, Fe, Li, Pb, P, Mn, Hg, Mo, Ni, Rb, Se, Sr, Ag, U, Th and Zn;

Note these are indicative analytes and the final assay suite will be determined following review of baseline data.

6.2.3 Surface Water Trigger Values

Surface water trigger values are unable to be determined for the Project at this time due to the ephemeral nature of the creeks. Due to limited flow events to date, limited baseline data has been collected and trigger values will be determined as sufficient data becomes available.

Stormwater sediment retention ponds and processing infrastructure have been designed to capture and contain precipitation across the project site. The stormwater sediment retention basin capacities will be designed in the detailed project design phase. However, in the unlikely event of an uncontrolled discharge from the site, sediment in addition to surface water sampling (where available) will be undertaken. This will be assessed against any baseline data collected and same day control site samples.

6.2.4 Seepages

Water retention structures including stormwater sediment retention ponds, will be installed across the site to reduce potential impacts on the receiving environment from FTSF, RSF and EV. If seepage is identified during routine inspections the following will be undertaken:

1. Location and extent: a summary of the location of the seep will be recorded and indicated on map. The extent of the seep will be recorded including visible on the ground and surface water influence.
2. Volume: the volume of seepage will be recorded as an estimate in L/minute.
3. Duration: the duration including commencement and ceasing date will be recorded.
4. Photographs: a photographic log will be taken to visualise the seep.

[2] Samples for dissolved metals are field filtered using 0.45 µm Stericup filter.

5. Sampling – Field: field water quality of the seep will be undertaken.
6. Sampling – Laboratory: if sufficient water can be collected and/or the seep continues for three consecutive days a laboratory sample will be collected.

The frequency of inspection, measurement and sampling is to be determined at the initial identification of seepage and reviewed regularly.

6.2.5 Discharges / Emergency Overflows

Mine site infrastructure has the potential to overflow during significant rainfall events. In the event of a discharge (stormwater overflowing basins), the discharge water and receiving waterbodies will be sampled. Sampling will be undertaken daily during discharges including field and laboratory suites.

The standard surface water sampling procedures will be followed in addition to the following:

1. Location and extent: a summary of the location of the discharge will be recorded and indicated on map.
2. Volume: the volume of discharge will be recorded daily as an estimate in L/minute.
3. Duration: the duration including commencement and ceasing date will be recorded.
4. Photographs: a photographic log will be taken at discharge point and sample locations (discharge point, upstream and downstream).
5. Sampling – Field: daily field water quality of the discharge will be undertaken.
6. Sampling – Laboratory: daily laboratory sampling of discharge, upstream and downstream receiving environment locations.

To be determined during detailed design phase.

Figure 6-1 Surface Water Sampling Locations

6.3 Groundwater

6.3.1 Sampling Locations

Mine Site and Processing Site

Groundwater contamination has the potential to cause long term impacts. The distance between potential contamination sources such as the FTSF, RSF, ROM Pad and low grade stockpiles and monitoring locations potentially influences contaminant concentrations. Monitoring will be undertaken adjacent to the point of potential contaminant discharge and at boundary locations.

The mine site will include an array of nested monitoring bore locations across the mine site at selected sites to monitor potential contaminant sources. Monitoring will also occur at or near the mining lease boundary. The monitoring bores will be utilised to assess potential impact to groundwater quality and assist in validating the groundwater model.

The processing site will also include an array of monitoring locations near potential contaminant sources at the lease boundary. These monitoring bores will be used to confirm groundwater flow direction and monitor groundwater quality.

Site Specific Groundwater Trigger Values (SSGTVs) will be determined from up-gradient monitoring bores. The location of these monitoring bores will be finalised once detailed design of project infrastructure is complete.

A monitoring network will be established across the borefield to confirm and monitor aquifer performance. The network will act to safeguard the water supply for the life of the Project whilst ensuring that impacts are minimised. The monitoring bores will be utilised to assess potential impact to groundwater quality, to assist in validating the groundwater model and developing a class 2 or 3 groundwater model.

A summary of sampling locations are provided in Table 6-5 and illustrated on Figure 6-2 to Figure 6-4. The groundwater sampling methodology is provided within the Groundwater Sampling Procedure (Appendix C).

Table 6-5 Groundwater Sampling Locations

Site ID	Coordinates (GDA94 Zone 53)		Type	Description	Sample Frequency			
					Baseline		Operation	
	Easting	Northing			SWL	SWL and Laboratory*	SWL	SWL and Laboratory*
Mine Site								
MB101A	*tbc*	*tbc*	Boundary (up-gradient)	*tbc*	Automatic Loggers	Monthly	Automatic Loggers	Quarterly
MB101B								
tbc	*tbc*	*tbc*	*tbc*	*tbc*	Quarterly	Biannual	Quarterly	Quarterly
tbc								
Processing Site								
MB201A	*tbc*	*tbc*	Boundary (up-gradient)	*tbc*	Automatic Loggers	Monthly	Automatic Loggers	Quarterly
MB201B								
tbc	*tbc*	*tbc*	*tbc*	*tbc*	Biannual	Quarterly	Quarterly	Quarterly
tbc								
Borefield								
tbc	*tbc*	*tbc*	Monitoring Bores	*tbc*	Quarterly	Biannual	Quarterly / Automatic Logger	Biannual
tbc	*tbc*	*tbc*		*tbc*				
SB008	308109	7479250	Production Bore	*tbc*	Quarterly	Biannual	Quarterly	Biannual
SB015	301290	7479850		*tbc*				
SB021	294454	7482340		*tbc*				
SB025	288460	7483290		*tbc*				
SB027	304196	7484910		*tbc*				

Note: A = shallow monitoring well.
B = deep monitoring well.
* Radionuclides will be tested annually at representative bores only.

6.3.2 Sample Assay Suites

The sampling suite for surface water quality includes either field measurement or field and laboratory measurements. The suites are summarised as follows:

- Standing Water Level
- Standing Water Level and Laboratory Analysis:

  Standing water level and purged water quality characteristics including temperature, pH, electrical conductivity, total dissolved solids, turbidity and oxidation reduction potential.

  Laboratory analysis:

  – Total Suspended Solids, Total Hardness and Total Acidity and Alkalinity
  – Major ions ($CaCO_3$, CO_3, HCO_3, Ca, Mg, K, Na, Cl, SO_4, NO_2 NO_3)
  – Metals total and dissolved (0.45 µm field filtered[3]): Al, As, B, Ba, Cd, Co, Cu, Fe, Li, Pb, P, Mn, Hg, Mo, Ni, Rb, Se, Sr, Ag, U, Th and Zn.
  – Radionuclides (U-238, U-234, Th-230, Ra-226, Rn-222, Pb-210, Po-210, Th-232, Ra-228, Th-228). Radionuclides will be tested annually at representative bores only.

Note these are indicative analytes and the final assay suite will be determined following review of baseline data.

6.3.3 Groundwater Trigger Values

The Project is situated in a predominately undeveloped area, natural variation in groundwater quality exists and will be further established through a baseline assessment in order to understand the Project potential impact over its lifetime, in accordance with ANZEC/ARMCANZ (2013). The current level of understanding of groundwater properties does not allow the adoption or determination of appropriate trigger values such as ANZECC/ARMCANZ, as illustrated summarised in Section 4.3.2.

Additional baseline data is required to inform an effective monitoring program and to establish site specific trigger values. The length of baseline monitoring will be sufficient to establish natural variability of groundwater quality in the area. The baseline at the mine site and processing site will be established over a two year period (i.e. the construction phase). The data will be collected from up-gradient and boundary bores at both sites which will be monitored on a quarterly basis. The 80th percentile values will be utilised to develop Site Specific Groundwater Trigger Values (SSGTVs).

Adjacent land use and the ultimate end land use of the Project is likely to be a pastoral lease (cattle grazing), therefore in addition to the SSGTVs, groundwater monitoring data will be reviewed against ANZECC Stock Watering for closure and rehabilitation purposes.

Trigger Value Assessment

Groundwater monitoring results will be assessed against SSGTVs during operation as follows:

- Toxicants (Metals) – 95th percentile of concentration values across a quarter will be assessed against trigger values and reference site; and
- Physical and Chemical Stressor – median concentration values across a quarter will be assessed against the 80th percentile at the reference site.

If concentrations exceed trigger values as defined by the above criteria, investigations will be undertaken to identify management measures to be implemented if risks are considered to be significant.

[3] Samples for dissolved metals are field filtered using 0.45 µm Stericup filter.

Contingency plans specific to Acid Metalliferous Drainage (AMD) management at the site would need to be implemented if any AMD issues result in an exceedance of ground or surface water quality when assessed against site-specific trigger values. This approach would involve undertaking an investigation to identify ‘root cause’ whereby the causal link for the water quality exceedance would be determined. Adaptive management would then seek to implement an appropriate alternate management strategy to eliminate any future risk of a repeat, given the nature of the incident.

Trend Assessment

The principal objective of the monitoring programs will be to assess change over time. A trend analysis will be utilised to determine potential impact to groundwater and assess if the impact is increasing, decreasing or constant.

To be determined during detailed design phase.

Figure 6-2 Groundwater Sampling Locations – Mine Site

To be determined during detailed design phase.

Figure 6-3 Groundwater Sampling Location – Processing Plant

To be determined during detailed design phase.

Figure 6-4 Groundwater Sampling Locations – Borefield

6.4 Sediment

6.4.1 Sampling Locations

Sediment sampling will be undertaken when samples are available to augment the water quality sampling. The purpose of sediment sampling will be to characterise the quality of sediments within the flow channels. Sediment sampling locations are proposed to coincide with the above-mentioned surface water monitoring locations at the mine site.

A summary of sampling locations and suites are provided in Table 6-6 and illustrated on Figure 6-1. The sediment sampling procedures are provided in the Sediment Sampling Procedure (Appendix B).

Table 6-6 Sediment Sampling Locations

Site ID	Coordinates (GDA94 Zone 53)		Type	Description	Sample Frequency	
	Easting	Northing			Baseline	Operation
					Biannual	Annually

6.4.1 Sampling Assay Suite

The sampling suite for sediment includes photopoint monitoring and laboratory measurements as follows:

- Photopoint Monitoring
  - Photopoint Monitoring, depth of standing water, direction of flow, summary of ground conditions (sediments, debris and pollution impacts)
- Laboratory Analysis:
  - Electrical conductivity and pH.
  - Total organic carbon.
  - Major ions ($CaCO_3$, CO_3, HCO_3, Ca, Mg, K, Na, Cl, SO_4, NO_3)
  - Meals (Al, As, B, Ba, Cd, Co, Cu, Fe, Li, Pb, P, Mn, Hg, Mo, Ni, Rb, Se, Sr, Ag, U, Th and Zn.
  - Total Hardness and Total Acidity and Alkalinity.
  - Particle Size Distribution (sieve and hydrometer).

Note these are indicative analytes. The final suite will be determined following review of existing baseline data.

6.4.2 Sediment Trigger Values

Sediment has the potential to be a source for dissolved contaminants. In general, sediments represent a source of bioavailable contaminants to benthic biota and are therefore a threat to the aquatic food chain. However, the Projects location and the aforementioned limited aquatic fauna makes the use of ANZECC Interim Sediment Quality Guidelines (ISQG) inappropriate.

In some instances there are no guidelines for a specific contaminant due to an absence of baseline data. The recommended interim approach is to derive a value on the basis of natural background (control) concentration multiplied by an appropriate factor (ANZECC 2000 recommends a factor of 2).

Baseline Assessment

Sediment monitoring results will be assessed against baseline and reference sites to determine if there is a significant increase. A minimum of three baseline samples will be collected.

Trend Assessment

The principal objective of the monitoring programs will be to assess change over time. A trend analysis will be utilised to determine potential impact to groundwater and assess if the impact is increasing, decreasing or constant.

7. Monitoring Program - Quality Assurance and Quality Control

Quality Assurance (QA) involves all of the actions, procedures, checks and decisions, undertaken to ensure the representativeness and integrity of samples and accuracy and reliability of analytical results (National Environmental Protection Council, 1999). Quality Control (QC) involves protocols to monitor and measure the effectiveness of QA procedures.

The QA/QC procedures outlined in the following Sections are based on AS 5567.1 – 1998 and will be implemented during sampling.

7.1 Data Quality Indicators

To minimise the potential for unrepresentative data, the following Data Quality Indicators (DQIs) will be used to evaluate sampling techniques and laboratory analysis of collected samples:

- **Data representativeness** - expresses the degree which sample data accurately and precisely represents a characteristic of a population or an environmental condition. Representativeness is achieved by collecting samples in an appropriate pattern across the Project, and by using an adequate number of sample locations to characterise the site. Consistent and repeatable sampling techniques and methods are utilised throughout the sampling program.
- **Completeness** - defined as the percentage of measurements made which are judged to be valid measurements. The completeness goal is set as being sufficient valid data generated during the monitoring program. If there is insufficient valid data, then additional data are required to be collected.
- **Comparability** - is a qualitative parameter expressing the confidence with which one data set can be compared with another. This is achieved through maintaining a level of consistency in techniques used to collect samples and ensuring analysing laboratories use consistent analysis techniques and reporting methods.
- **Precision** - measures the reproducibility of measurements under a given set of conditions. The precision of the data is assessed by calculating the Relative Percent Difference (RPD) between duplicate sample pairs.

$$\mathrm{RPD}(\%) = \frac{|C_o - C_d|}{C_o + C_d} \times 200$$

Where Co = Analyte concentration of the original sample

Cd = Analyte concentration of the duplicate sample

  A nominal acceptance criteria of 30% RPD for field duplicates and splits for inorganics will be adopted, however it is noted that this will not always be achieved, particularly at low analyte concentrations.

- **Accuracy** - measures the bias in a measurement system. Accuracy can be undermined by such factors as field contamination of samples, poor preservation of samples, poor sample preparation techniques and poor selection of analysis techniques by the analysing laboratory. Accuracy is assessed by reference to the analytical results of laboratory control samples, laboratory spikes, laboratory blanks and analyses against reference standards. The nominal “acceptance limits” on laboratory control samples are defined as follows:
  - Laboratory spikes – 70-130% for metals/inorganics, 60-140% for organics;

– Laboratory duplicates – <30% for metals/inorganics, <50% for organics; and

– Laboratory blanks – <practical quantitation limit.

Accuracy of field works is assessed by examining the level of contamination detected in field and equipment blanks. Blanks should return concentrations of all organic analytes as being less than the practical quantitation limit of the testing laboratory.

The individual testing laboratories will conduct an internal assessment of the laboratory QC program; however the results will also be independently reviewed and assessed.

7.2 Summary of Data Quality Acceptance Criteria

Data quality acceptance criteria adopted for this Project are set out in Table 7-1. These are generally based on the minimum requirements detailed in the Australian Standard AS4482.1-2005.

Table 7-1 Data Quality Acceptance Criteria

Measurement	Sediment	Water	Frequency	Acceptance Criteria	
				RPD (%)	Recovery (%)
Quality control samples to be prepared or taken on site (field)					
Blind field duplicate (BFD) samples (primary laboratory)	Yes	Yes	1 in 20 samples collected or 1 per batch	30 or 50	-
Quality control samples to be prepared by laboratory					
Laboratory blanks	Yes	Yes	1 per batch	-	-
Laboratory duplicates	Yes		1 in 10 samples collected or 1 per batch (whichever is smaller)	30	-
Matrix spike recoveries	Yes		1 per batch	-	70 to 130
Laboratory control sample spike recoveries	Yes		1 per batch	-	70 to 130
Surrogate spikes	Yes	Yes	Each analysis done by GC-MS (all organics except TPH $_{C>10}$)		

Note: water includes surface and groundwater.

7.3 Field Program

All field work will be conducted with reference to the advisory note of the Department of Resources (2009) for the sampling of surface waters and groundwaters. Key requirements of these procedures are as follows:

- Decontamination procedures - including the use of new disposable gloves for the collection of each sample, decontamination of all multiple use sampling equipment between each sampling location (using a phosphate free detergent and potable water) and the use of dedicated sampling containers provided by the laboratory;
- Sample identification procedures - collected samples will be immediately transferred to sample containers of appropriate composition and preservation for the required laboratory analysis. All sample containers to be clearly labelled with a sample number, sample location, sample depth (for groundwater) and sample date. The sample containers are then transferred to an ice filled cooler for sample preservation prior to and during shipment to the testing laboratory;
- Chain of custody protocols - a chain-of-custody form is to be completed and forwarded to the testing laboratory with each discrete batch of samples; and
- Sample duplicate frequency - field duplicates (blind) to be collected and analysed at a rate not less than ten per cent (i.e. not less than one duplicate per ten primary samples).

7.3.1 Field Quality Control

Field quality control procedures will include the collection and analysis of the following:

- **Blind field duplicates (BFDs):** Comprise a single sample that is divided into two separate sampling containers. Both samples are sent anonymously to the primary Project laboratory. Blind duplicates provide an indication of the analytical precision of the laboratory, but are inherently influenced by other factors such as sampling techniques and sample media heterogeneity.

8. Site Water Management

8.1 Water Classifications

Four water types are anticipated to be present during the operation of the Project. These water types range in potential contaminant loading and associated management measures. The water types include:

- Clean Water

  Water which originates outside of the disturbance footprint / upstream / up gradient. This water may be diverted around the Project with the use of flow diversion banks and/or catch drains and allowed to discharge into the surrounding environment or maybe captured and re-used within the process.

- Sediment Laden Water

  Water originating on disturbed areas of the site but has not come into contact with pollutants or contaminants. In general, this would comprise water collected within the WRDs, Mining Services/Buildings, Processing Site, Power Plant and Accommodation Village stormwater basins.

- Impacted Water

  Water originating on disturbed areas of the site that has come into contact with ore or hazardous substances. In general, this would comprise water collected within the Open Pit, ROM Pad and Stockpile stormwater basins. This water will be used in the Processing circuit.

- Processing Water

  Water which has been in contact with or used within the Processing circuit. This includes water storage within the Flotation Tailings Storage Facility (FTSF), Residue Storage Facility (RSF) and Dense Media Separation Stockpile. Processing water will predominately evaporate with some recycling of water from the FTSF.

8.2 Water Management Systems

8.2.1 Potable Water Supply

Potable water will be sourced from the Borefield then treated (filtered and chlorinated) at the processing plant and distributed across the Project as required. At the accommodation village where a high demand is anticipated the water supply will be stored within an onsite tank with sufficient capacity to store two days' supply.

A temporary water supply will be established as an interim measure until the permanent potable water treatment plant is commissioned.

8.2.2 Sewerage

Wastewater from the accommodation village and non-processing wastewater from the processing site and mine site will either be pumped to a common sewage treatment plant located adjacent to the processing plant or to a second small treatment facility installed at the mine site. The pipelines will be located within defined road service corridors.

The Sewage Treatment Plant (STP) will be a package type unit providing the appropriate level of treatment. Treated effluent will be disposed of within the Evaporation Ponds and sludge will be pumped out and disposed of by a local (Alice Springs) contractor on a regular basis.

8.2.3 Processing Plant

Concentrate slurry from the concentrator will be pumped approximately 8 km to the processing site. The pipeline will run above ground within a bunded corridor. In the event of leaks or pipe failure, slurry will be captured within the bunded corridor and within event ponds located at significant low points along the 8 km alignment.

The processing plant and its associated residue storage facilities, namely Phosphate Residue, Impurity Removal Residue, Water Leach Residue and Sodium Sulphate are located west of the gas pipeline and on the south facing slopes above the Southern Basins.

8.3 Water Use

The peak raw water demand is projected to be 4,776.5 ML/y consisting of 4,418 ML/y, 91.5 ML/y and 267 ML/y for Processing water, potable water and dust suppression respectively. The demand for raw water will steadily ramp up over the construction and operations phase of the mine and is expected to peak in mine development stage 6 (approximately year 19 of the 43 year LOM period). The demands are summarised with additional detail within Table 8-1 and provided in Figure 8-1.

Table 8-1 Peak Operational Water Requirements

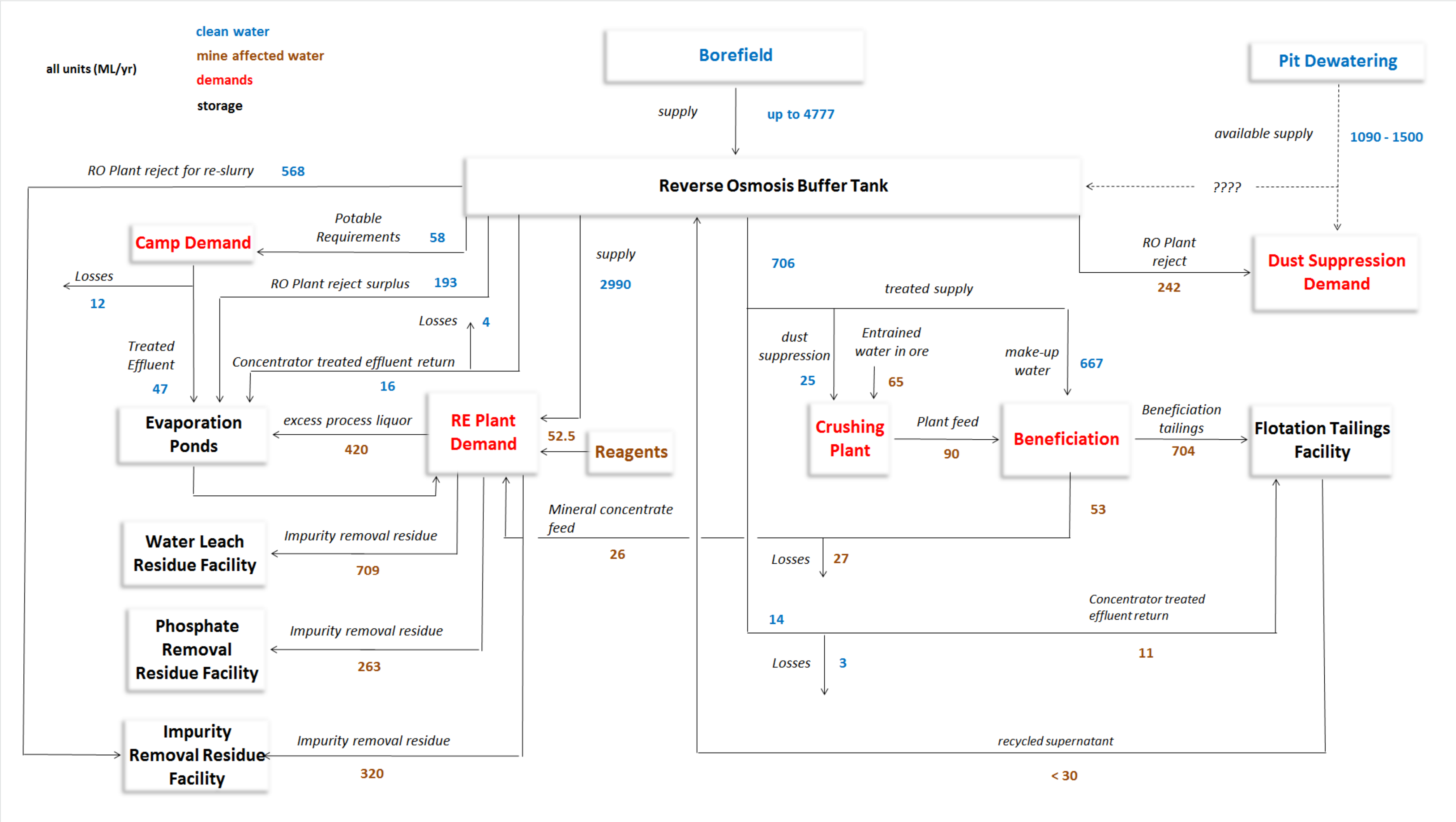
Area	Details	Water Requirement (ML/yr)
Processing Water	Beneficiation make-up water	667
	Process Plant	2,990
	RO Plant reject surplus	761
Potable Water	Accommodation camp	58
	MSA/concentrator	13.5
	Process Plant	20
Dust Suppression	Haul roads	242
	Crusher	25
	TOTAL	4,776.5

Note: Potable water includes 400 people (at 400 L/person/day) at the accommodation village, 205 people at the MSA/concentrator (at 180 L/person/day) and 300 people at the Process Plant (at 180 L/person/day)
Dust suppression includes the use of water carts 336 days a year.

8.3.1 Water Sources

The majority of water for the Project will be sourced from the Borefield situated approximately 25 km south west of the processing plant. The Borefield is located within the Southern Basins and will contain a number of production bores pumping approximately 4,777 ML/yr which matches the water use requirements detailed in Table 8-1.

Groundwater modelling of the mining operation indicates peak groundwater inflow will be approximately 46 L/s. Mine pit water will be collected in a sump and pumped for use at the crusher, dust suppression and the processing plant.



source: Nolans Project Infrastructure Engineering Cost Study. Lycopodium, February 2014. {Figure - Overall Nolans Site Water Balance Summary Rev C 5/11/2013}

Figure 8-1 Schematic of Water Demands

9. References

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GHD (2016a) Surface Water Sampling Procedure (Appendix A).

GHD, 2016b Sediment Sampling Procedure (Appendix B).

GHD, 2016c Groundwater Sampling Procedure (Appendix C).

GHD, 2016d Erosion and Sediment Control Plan (Appendix D).

GHD (2016e) Hydrogeological Assessment (EIS Appendix K).

GHD (2016f) Surface Water Technical Report (EIS Appendix I).

GHD (2016g) Acid and Metalliferous Drainage (AMD) Assessment and Management Plan (EIS Appendix L).

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Appendices

Appendix A – Surface Water Sampling Procedure

Arafura Resources Limited

Nolans Project

Surface Water Sampling Procedure

March 2016

Document Status

Version	Author	Reviewer	Approved by	Date	Status

Amendments

Section	Details

Audit Summary

Section	Details

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1. Introduction

The Water Management Plan (WMP) for the Nolans Project (Project) provides a summary of sampling requirements at the site. The WMP contains a commitment to collect data throughout the construction and operations phase, to assess the performance of water management on site. In order to facilitate consistency in sampling, and comply with quality assurance and control methodologies, a series of sampling procedures have been established including:

- Surface Water Sampling Procedure (this procedure);
- Groundwater Sampling Procedure; and
- Sediment Sampling Procedure.

1.1 Purpose

The primary objective of the surface water sampling procedure is to prevent contamination or alteration in water chemistry during sample collection. The collected sample should represent the physical, chemical and biological characteristics of surface water in the targeted water body as closely as possible.

1.2 Planning and Equipment

A number of factors must be considered during the field planning phase, prior to surface water sampling. These include consideration of ground condition at targeted locations and safety requirements. A summary of equipment and associated suppliers are provided in Table 1-1. All equipment in relation to surface water sampling should be ordered a minimum of four weeks prior to sampling.

Table 1-1 Summary of Planning

Timing	Details	Supplier
At least 4 weeks prior to sampling	**Order Lab Bottles** Laboratory bottles Eskies and Cool Bricks	*tbc*
	Hire / Maintenance Check Long arm sampler	Eco Environmental 6/509-511 South Rd, Ashford SA 5031 08 8293 3355 adelaide@ecoenvironmental.com.au Thermo Fisher Scientific 5 Caribbean Dv, Scoresby Vic 3179 03 9757 4377 RentalsAU@thermofisher.com
	Purchase 0.45µm Stericup filters Stericup vacuum pump Nitrile gloves Decon N	Eco Environmental 6/509-511 South Rd, Ashford SA 5031 08 8293 3355 adelaide@ecoenvironmental.com.au Thermo Fisher Scientific 5 Caribbean Dv, Scoresby Vic 3179 03 9757 4377 RentalsAU@thermofisher.com
1 day prior to sampling	**Calibrate** Water quality meter	-

2. Surface Water Sampling Procedure

2.1 Sampling Equipment

Surface water sampling requires the following:

- Surface Water Quality Sheet (Appendix A);
- Long arm sampler;
- Water quality meter (calibrated);
- 0.45µm water filters and suction pump;
- Eskies and Cool Bricks;
- Laboratory bottles;
- Nitrile Gloves;
- Decontaminated plastic or stainless steel bucket;
- Tool kit including screw drivers, tape measure and shovel; and
- Permeant marker.

2.2 Sampling Locations

There are a number of surface water sampling locations across the Project. These sampling locations are positioned to assess upstream, onsite and downstream impacts from the Project. Rising stage samplers have been installed in the Kerosene Camp Creek, the Nolans Creek and the western tributary of the Kerosene Camp Creek where the intended diversion will flow. Gauging stations have also been installed downstream of the processing site in drainage lines to capture run-off and understand baseline water quality.

The sampling program has been designed to assess Nolans Creek, Kerosene Camp Creek, Kerosene Camp Creek western tributary and other minor drainage lines in and around the Project.

A summary of sampling locations, frequency and suites are provided in Table 2-1 and illustrated on Figure 2–1.

Table 2-1 Surface Water Sampling Locations

Site ID	Coordinates (GDA94 Zone 53)		Type	Description	Sample Frequency	
	Easting	Northing			Baseline	Operation
Surface Water						
tbc	*tbc*	*tbc*	*tbc*	*tbc*	Early and late flows with a minimum of 0.1m flowing water.	
tbc	*tbc*	*tbc*	*tbc*	*tbc*		
Stormwater Sediment Retention Ponds						
tbc	*tbc*	*tbc*	*tbc*	*tbc*	-	Field measurements monthly Field and laboratory suite quarterly.
tbc	*tbc*	*tbc*	*tbc*	*tbc*		
Mine Pit and Processing Infrastructure						
Mine pit Sump	*tbc*	*tbc*	-	Mine pit sump water (i.e. groundwater inflow).	-	Field measurements monthly Field and laboratory suite quarterly.
Flotation Tailings Storage Facility	`	*tbc*	-	Flotation Storage Facility ponded water.		
Residue Storage Facilities 1	*tbc*	*tbc*	-	Residue Storage Facility ponded water.		
Residue Storage Facilities 2	*tbc*	*tbc*	-	Residue Storage Facility ponded water.		
Evaporation Pond 1	*tbc*	*tbc*	-	Evaporation Pond water.		
Evaporation Pond 2	*tbc*	*tbc*	-	Evaporation Pond water.		
Evaporation Pond 3	*tbc*	*tbc*	-	Evaporation Pond water.		

To be determined during detailed design phase.

Figure 2-1 Surface Water Sampling Locations

2.2.1 Surface Water Sampling Suite

The surface water sampling suite is provided below. Field measurements are to be collected using the water quality meter following stabilisation of parameters.

- Field Measurement
  - Temperature, pH, oxidation-reduction potential (ORP), electrical conductivity (EC), total dissolved solids (TDS), dissolved oxygen (DO) and turbidity
  - Level and Photopoint Monitoring (photos of sample location, upstream and downstream)
- Laboratory Analysis
  - Total suspended solids (TSS)
  - Total hardness
  - Total acidity and alkalinity
  - Major ions ($CaCO_3$, CO_3, HCO_3, Ca, Mg, K, Na, Cl, SO_4, NO_2, NO_3)
  - Metals total and dissolved[1]: Al, As, B, Ba, Cd, Co, Cu, Fe, Li, Pb, P, Mn, Hg, Mo, Ni, Rb, Se, Sr, Ag, U, Th and Zn

Note these are indicative analytes. The final suite will be determined following review of existing baseline data.

Sampling is to be undertaken in accordance with the sampling procedure provided in Section 3.

2.3 Sampling Frequency

Sampling will be undertaken in accordance with the frequency identified in Table 2-1 when sufficient water is available to collect a sample without sediments being disturbed. Surface water sampling will only be undertaken during flow events where a minimum 0.1m of flowing water is present.

2.3.1 Seepages

Water retention structures including stormwater retention basins, FTSF, RSFEP will be installed across the site to reduce potential impacts on the receiving environment. If seepage is identified during routine inspections the following will be undertaken:

1. Location and Extent: a summary of the location of the seep will be recorded and indicated on map (Figure 2–1). The extent of the seep will be recorded including the visible on ground and surface water influence
2. Volume: the volume of seepage will be recorded daily as an estimate (L/minute or L/day)
3. Duration: the duration including commencement and ceasing date will be recorded
4. Photographs: a photographic log will be taken to visualise the seep
5. Sampling – Field: field water quality of the seep will be undertaken. Additional field sampling will be undertaken upstream, at the seep and downstream at consistent location if the seepage extent is greater than 10 m
6. Sampling – Laboratory: if sufficient water can be collected and/or the seep continues for three consecutive days a laboratory sample will be collected.

The frequency of inspection, measurement and sampling is to be determined at the initial identification of seepage and reviewed regularly.

[1] Samples for dissolved metals are field filtered using 0.45 µm Stericup filter.

2.3.2 Discharges / Emergency Overflows

Mine Site infrastructure has the potential to overflow during significant rainfall events. In the event of a discharge (stormwater overflowing basins) the discharge water and receiving waterbodies will be sampled. Sampling will be undertaken daily during discharges including field and laboratory suites.

The standard surface water sampling procedures will be followed in addition to the following:

1. Location and Extent: a summary of the location of the discharge will be recorded and indicated on map including estimation of its extent.
2. Volume: the volume of discharge will be recorded daily as an estimate in L/minute.
3. Duration: the duration including commencement and ceasing date will be recorded.
4. Photographs: a photographic log will be taken at the Sample locations (discharge location, upstream and downstream).
5. Sampling – Field: daily field water quality of the discharge will be undertaken.
6. Sampling – Laboratory: daily laboratory sampling of discharge, upstream and downstream receiving environment locations.

3. Surface Water Sampling Procedures

3.1 Field Measurements

Surface water gauging is to be undertaken in accordance with the following:

1. Complete surface water quality sheet for location (Appendix A)
2. Water Quality Parameters

   Record field water quality parameters by either suspending the water quality meter within the water body or collecting a sample and placing into a clean bucket for measurements to be taken
3. Photographs

   Photographs of the sample location should be taken including the sampling point, upstream and downstream. Photographs to be logged into a filing system indicating site location and date.

3.2 Field and Laboratory Measurements

Surface water sampling is only to be undertaken during periods of flow greater than 0.1m deep. The process is to be undertaken in accordance with the following:

1. Complete surface water quality sheet for location (Appendix A)
2. Water Quality Parameters

   Record field water quality parameters by either suspending the water quality meter within the water body or collecting a sample and placing into a clean bucket for measurements to be taken
3. Photographs

   Photographs of the sample location should be taken including the sampling point, upstream and downstream. Photographs to be logged into a filing system indicating site location and date
4. Grab Sample

   Rinse long arm sampler container in the water body to be sampled three times. Place long arm sampler directly into water body, open end vertically down and fill with an arc motion with the bottle mouth facing upstream. Take care to avoid collecting surface films

   For waters less than half a metre in depth, collect a grab sample at half the water depth. For waters greater than half a metre in depth, a grab sample should be taken at 20 to 30 cm below the surface water

   – Field Filtering

     A total metals sample (not filtered) and a dissolved metal sample should be collected. The dissolved metal sample requires field filtration through a disposable 0.45 µm filter
5. Waste Disposal

   Excess surface water is to be returned to ground and all disposable sampling equipment used should be stored for disposal at the Process Site including filters
6. Electronic Transfer

   All water quality results, duplicate locations and Chain of Custody (CoC) are to be scanned and kept on file. The purging results are to be entered into the surface water database.

3.3 Sample Dispatch

Water samples have a high potential to deteriorate following collection. Samples are to be placed into onsite fridge pending dispatch to laboratory. At completion of the sampling round, bottles are to be packed into eskys and ice bricks placed on top of samples and transferred to Alice Springs haulage depot. Samplers are to contact the haulage companies and the laboratory to inform them of sample delivery and requirements to keep refrigerated.

The sampler is to inform the laboratory of sample postage and provide a completed CoC. A blank CoC is provided in Appendix A.

4. Discharge Notifications

Discharges from the Site will be assessed on a case by case basis to determine if formal notifications to the DME and NT EPA are required. All external communication of incidents will be signed and approved by Arafura Resources Management Team.

In general, if there is a discharge of contained/managed water from the Project (i.e. collapse of flood levees or overflow of stormwater retention basins) the DME and NT EPA will be notified. A summary of the notification requirements is provided in Table 4-1.

Table 4-1 Formal Notification Requirements

Entity	Trigger	Timeframe and Contact Details	Incident Reporting Details
Department of Mines and Energy (DME)	Incident which causes minor environmental impact with some minor actual or potential hard to the environment.	As soon as practicable. Mineral.Info@nt.gov.au	The Section 29 Notification of Environmental Incident Form requires the following details: • Site and operator details. • Location occurred and area impacted (GPS coordinates); • Date and time; • Description of incident • Emergency and remedial actions taken. • Nature of impact and severity; • Current situation; • Details of sampling undertaken; and • Notification status internally and externally. The form is to be signed by the HSEC Manager and/or General Manager. A blank form is provided in Appendix C.
Northern Territory Environment Protection Authority (NT EPA)	Incident which causes, or is threatening or may threaten to cause pollution resulting in material environmental harm or serious harm.	< 24 hrs post incident ntepa@nt.gov.au pollution@nt.gov.au	The Section 14 Incident Report Form requires the following details: • Incident causing or threatening to cause pollution; • Location occurred and area impacted; • Date and time; • How the pollution has occurred, is occurring or may occur; • Attempts made to prevent, reduce, control, rectify, investigation and/or clean up the pollution or resultant environmental harm caused or threatening to be caused by the incident; and • Operator details. The form is to be signed by the HSEC Manager and/or General Manager. A blank form is provided in Appendix D.

Qualifying triggers requiring submittal of Section 14 Incident Report to NT EPA are any of the following:

- is not trivial or negligible in nature; or
- consists of an environmental nuisance of a high impact or on a wide scale; or
- results, or is likely to result in $50,000 or more in taking action to prevent or minimise environmental harm or rehabilitate the environment; or results in actual or potential loss or damage to value of $50,000 or more of the prescribed amount (whichever is the greater).

Appendices

Appendix A – Surface Water Quality Sheet

SURFACE WATER QUALITY FIELD SHEET

Date:	Sampler:

FIELD PARAMETERS

Location ID	Time	pH	D.O (%)	ORP (mV)	E.C (µS/cm)	TDS (mg/L)	Temp (^{0}C)	Turbidity (NTU)	Salinity (ppt)	Comments (water flow, colour, suspended sediments)

Additional Comments

Appendix B – Chain of Custody Form

CHAIN OF CUSTODY

ALS Laboratory: please tick →

❑ADELAIDE 21 Burma Road Pooraka SA 5095
Ph: 08 8359 0890 E: adelaide@alsglobal.com

❑BRISBANE 32 Shand Street Stafford QLD 4053
Ph: 07 3243 7222 E: samples.brisbane@alsglobal.com

❑GLADSTONE 46 Callemondah Drive Clinton QLD 4680
Ph: 07 7471 5600 E: gladstone@alsglobal.com

❑MACKAY 78 Harbour Road Mackay QLD 4740
Ph: 07 4944 0177 E: mackay@alsglobal.com

❑MELBOURNE 2-4 Westall Road Springvale VIC 3171
Ph: 03 8549 9600 E: samples.melbourne@alsglobal.com

❑MUDGEE 27 Sydney Road Mudgee NSW 2850
Ph: 02 6372 6735 E: mudgee.mail@alsglobal.com

❑NEWCASTLE 5 Rose Gum Road Warabrook NSW 2304
Ph: 02 4968 9433 E: samples.newcastle@alsglobal.com

❑NOWRA 4/13 Geary Place North Nowra NSW 2541
Ph: 024423 2063 E: nowra@alsglobal.com

❑PERTH 10 Hod Way Malaga WA 6090
Ph: 08 9209 7655 E: samples.perth@alsglobal.com

❑SYDNEY 277-289 Woodpark Road Smithfield NSW 2164
Ph: 02 8784 8555 E: samples.sydney@alsglobal.com

❑TOWNSVILLE 14-15 Desma Court Bohle QLD 4818
Ph: 07 4796 0600 E: townesville.environmental@alsglobal.com

❑WOLLONGONG 99 Kenny Street Wollongong NSW 2500
Ph: 02 4225 3125 E: portkembla@alsglobal.com

CLIENT:

OFFICE:

PROJECT:

ORDER NUMBER:

PROJECT MANAGER: **CONTACT PH:**

SAMPLER: **SAMPLER MOBILE:**

COC emailed to ALS? (YES / NO) **EDD FORMAT (or default):**

Email Reports to (will default to PM if no other addresses are listed):

Email Invoice to (will default to PM if no other addresses are listed):

TURNAROUND REQUIREMENTS : (Standard TAT may be longer for some tests e.g.. Ultra Trace Organics)

☐ Standard TAT **(List due date):**

☐ Non Standard or urgent TAT **(List due date):**

ALS QUOTE NO.:

COC SEQUENCE NUMBER (Circle)

COC:	**1**	**2**	**3**	**4**	**5**	**6**	**7**
OF:	**1**	**2**	**3**	**4**	**5**	**6**	**7**

FOR LABORATORY USE ONLY (Circle)

Custody Seal Intact?	Yes	No	N/A
Free ice / frozen ice bricks present upon receipt?	Yes	No	N/A
Random Sample Temperature on Receipt:	˚C		
Other comment:			

RELINQUISHED BY:	**RECEIVED BY:**	**RELINQUISHED BY:**	**RECEIVED BY:**
DATE/TIME:	DATE/TIME:	DATE/TIME:	DATE/TIME:

COMMENTS/SPECIAL HANDLING/STORAGE OR DISPOSAL:

ALS USE	SAMPLE DETAILS MATRIX: SOLID (S) WATER (W)			CONTAINER INFORMATION		ANALYSIS REQUIRED including SUITES (NB. Suite Codes must be listed to attract suite price) Where Metals are required, specify Total (unfiltered bottle required) or Dissolved (field filtered bottle required).								Additional Information
LAB ID	SAMPLE ID	DATE / TIME	MATRIX	TYPE & PRESERVATIVE *(refer to codes below)*	TOTAL CONTAINERS									Comments on likely contaminant levels, dilutions, or samples requiring specific QC analysis etc.
					TOTAL									

Water Container Codes: P = Unpreserved Plastic; N = Nitric Preserved Plastic; ORC = Nitric Preserved ORC; SH = Sodium Hydroxide/Cd Preserved; S = Sodium Hydroxide Preserved Plastic; AG = Amber Glass Unpreserved; AP - Airfreight Unpreserved Plastic
V = VOA Vial HCl Preserved; VB = VOA Vial Sodium Bisulphate Preserved; VS = VOA Vial Sulfuric Preserved; AV = Airfreight Unpreserved Vial SG = Sulfuric Preserved Amber Glass; H = HCl preserved Plastic; HS = HCl preserved Speciation bottle; SP = Sulfuric Preserved Plastic; F = Formaldehyde Preserved Glass;
Z = Zinc Acetate Preserved Bottle; E = EDTA Preserved Bottles; ST = Sterile Bottle; ASS = Plastic Bag for Acid Sulphate Soils; B = Unpreserved Bag.

Appendix C – DME Section 29 Notification of Environmental Incident

Notification of an Environmental Incident

Section 29 of the Mining Management Act

**Forward completed form to: Mining Compliance Division,
Department of Mines and Energy**

Email: mineral.info@nt.gov.au (preferred) or Fax: (08) 89996527

PLEASE TYPE OR PRINT CLEARLY

Please ensure that you have read the Draft Guideline - Environmental incident reporting under Section 29 of the Mining Management Act (July 2012) [167kb]]

NAME OF MINING SITE		
NAME OF OPERATOR		
DATE & TIME OF INCIDENT		
NAME OF PERSON NOTIFYING		
POSITION/TITLE		
CONTACT PERSON		
CONTACT DETAILS	Business:	Mobile
	Fax:	E-mail:
INCIDENT LOCATION (use GPS co-ordinates, attach map, etc as appropriate)		
DESCRIPTION OF INCIDENT		
EMERGENCY & REMEDIAL ACTIONS TAKEN		

ENVIRONMENTAL DETAILS

NATURE OF IMPACT AND SEVERITY (Volume/ of spillage, area impacted, wildlife/vegetation/ erosion, etc) DME severity classification: 1 2 3 4 *Refer to pages 3 to 5 of the Draft Guideline - Environmental incident reporting under Section 29 of the Mining Management Act (July 2012) [167kb]]*	
CURRENT SITUATION (Potential / ongoing / ceased / etc)	
DETAILS OF ANY SAMPLES TAKEN (when / where / type / number / time for results /etc)	

OPERATOR INTERNAL REPORTING

Has the incident been reported internally? **YES / NO** If so, to whom	Name: Position:
Operator reference number (where applicable/available)	

HAS THE DEPARTMENT BEEN NOTIFIED EARLIER?	☐**YES** ☐ **NO**
WHO WAS NOTIFIED	
HOW (phone/email/fax)	
WHEN (date & time)	
BY WHOM	

Signed: ___________________________________ *Date:* _______________

NAME*:* ___________________________________

POSITION: _______________________________

OFFICE USE ONLY	
RECEIVED BY	
DATE	TIME

Appendix D – NT EPA Section 14 Incident Report

SECTION 14 INCIDENT REPORT (*Waste Management and Pollution Control Act*)

Date and Time of Notification:	
Person / Company:	
Incident:	

(a) the incident causing or threatening to cause pollution	
(b) the place where the incident occurred	
(c) the date and time of the incident	
(d) how the pollution has occurred, is occurring or may occur	
(e) the attempts made to prevent, reduce, control, rectify or clean up the pollution or resultant environmental harm caused or threatening to be caused by the incident	
(f) the identity of the person notifying the NT EPA	

GHD

Level 5, 66 Smith Street Darwin NT 0800
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T: (08) 8982 0100   F: (08) 8981 1075   E: drwmail@ghd.com.au

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Revision	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Rev.0	A Koscielski	G Metcalfe N Conroy		N Conroy		29/03/16

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Appendix B – Sediment Sampling Procedure

Arafura Resources Limited

Nolans Project

Sediment Sampling Procedure

March 2016

Document Status

Version	Author	Reviewer	Approved by	Date	Status

Amendments

Section	Details

Audit Summary

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1. Introduction

The Water Management Plan (WMP) for the Nolans Project (Project) provides a summary of sampling requirements at the site. The WMP has been designed to collect data throughout the construction and operations phase to assess the performance of water management onsite. In order to facilitate consistency in sampling, and comply with quality assurance and control methodologies, a series of sampling procedures have been established including:

- Surface Water Sampling Procedure;
- Groundwater Sampling Procedure; and
- Sediment Sampling Procedure (this procedure).

1.1 Purpose

Sediment sampling will be undertaken as a proxy for water quality due to the limited flows experienced at and surrounding the Project.

The primary objective of the sediment sampling procedure is to obtain a sample with minimal significant alteration in sample chemistry during collection. The collected sample should represent the physical, chemical and biological characteristics of sediment in the targeted drainage system as closely as possible.

1.2 Planning and Equipment

A number of factors must be considered during the field planning phase prior to sediment sampling. These include consideration of ground condition at targeted locations and safety requirements. A summary of equipment and associated suppliers are provided in Table 1-1. All equipment in relation to sediment sampling should be ordered a minimum of four weeks prior to sampling.

Table 1-1 Summary of Planning

Timing	Details	Supplier
>4 weeks prior to sampling	**Order Lab Bottles** Laboratory jars and large zip lock bags Eskies and Cool Bricks	tbc
	Purchase Nitrile gloves Decon N	Eco Environmental 6/509-511 South Rd, Ashford SA 5031 08 8293 3355 adelaide@ecoenvironmental.com.au Thermo Fisher Scientific 5 Caribbean Dv, Scoresby Vic 3179 03 9757 4377 RentalsAU@thermofisher.com

2. Sediment Sampling Procedure

2.1 Sampling Equipment

Sediment sampling requires the following:

- Chain of Custody (Appendix A)
- Stainless steel or plastic bucket and spade;
- 3 mm stainless steel sieve, 80 mesh sieve
- Eskies and Cool Bricks;
- Laboratory jars and large zip lock bags (capable of storing 1 kg of sediments);
- Nitrile Gloves;
- Decon N; and
- Permeant marker.

2.2 Sampling Locations

Sediment sampling will be undertaken when samples are available to augment the water quality sampling. The purpose of sediment sampling will be to characterise the quality of sediments within the flow channels. Sediment sampling locations are proposed to coincide with the above-mentioned surface water monitoring locations at the mine site.

A summary of sampling locations, frequency and suites are provided in Table 2-1 and illustrated on Figure 2–1. These sites are indicative and will be reviewed after completion of detailed design. Sites will continue to match surface water sampling locations.

Table 2-1 Sediment Sampling Locations

Site ID	Coordinates (GDA94 Zone 53)		Type	Description	Sample Frequency	
	Easting	Northing			Baseline	Operation
tbc	*tbc*	*tbc*	*tbc*	*tbc*	Annually	Annually
tbc	*tbc*	*tbc*		*tbc*		

To be determined during detailed design phase.

Figure 2-1 Sediment Sampling Locations

2.2.1 Sediment Sampling Suite

The sampling suite for sediment includes photopoint monitoring and laboratory measurements as follows:

- Photopoint Monitoring
  - Photopoint Monitoring, depth of standing water, direction of flow, summary of ground conditions (sediments, debris and pollution impacts)
- Laboratory Analysis:
  - Electrical conductivity and pH.
  - Total organic carbon.
  - Major ions ($CaCO_3$, CO_3, HCO_3, Ca, Mg, K, Na, Cl, SO_4, NO_3)
  - Meals (Al, As, B, Ba, Cd, Co, Cu, Fe, Li, Pb, P, Mn, Hg, Mo, Ni, Rb, Se, Sr, Ag, U, Th and Zn.
  - Total Hardness and Total Acidity and Alkalinity.
  - Particle Size Distribution (sieve and hydrometer).

Note these are indicative analytes. The final suite will be determined following review of existing baseline data.

2.3 Sampling Frequency

Sampling will be undertaken annually at the same time each year.

3. Sediment Sampling Procedure

3.1 Field Measurements

Sediment sampling is to be undertaken in accordance with the following:

1. Photographs

   Photographs of the sample location, upstream and downstream. Photographs to be logged into a filing system indicating site location and date.

2. Ground Conditions

   Summarise all ground conditions at the sampling location including the presence or absence of water, direction and volume of flow.

   Areas of discoloured sediments, polluted water, affected plant growth and animal populations (aquatic) and odours should be identified and mapped.

   Undertake a qualitative assessment of sediment loss or deposition that has occurred since the previous sampling event.

3.2 Sample Collection

Sediment samples are composite samples including five sub samples across the primary drainage channel. The collection of sediment samples will be undertaken in accordance with the following:

1. Ensure equipment is clean (rinse spade and mixing bucket with Decon N between samples)
2. Scrape away any organic matter on surface
3. Sample Collection

   Apply nitrile gloves and collect five 1 kg subsamples across the primary flow channel. The sample should be collected from surface to a maximum depth of 150 mm.

   Combine and mix the subsamples thoroughly within the decontaminated bucket and collect the following:

   – Three glass jars (laboratory jars);
   – Two 1 kg zip lock bag (particle size distribution and contingency sample).

4. Waste Disposal

   Excess sediment will be returned to ground at the side of the creek and all used disposable sampling equipment should be stored for disposal at the Processing Site.

5. Electronic Transfer

   All sediment quality results, duplicate locations and CoC are to be scanned and kept on file.

3.3 Sample Dispatch

Sediment samples have potential to deteriorate following collection. Samples are to be placed into onsite fridge pending dispatch to laboratory. At completion of the sampling round, samples are to be packed into eskys and ice bricks placed on top of samples and transferred to Alice Springs haulage depot. Samplers are to contact the haulage companies and laboratory to inform them of sample delivery and requirements to keep refrigerated.

The sampler is to inform the laboratory of sample postage and provide a completed Chain of Custody (CoC). An example CoC is provided in Appendix A.

Appendices

Appendix A – Chain of Custody Form

CHAIN OF CUSTODY

ALS Laboratory: please tick →

❑ADELAIDE 21 Burma Road Pooraka SA 5095
Ph: 08 8359 0890 E: adelaide@alsglobal.com

❑BRISBANE 32 Shand Street Stafford QLD 4053
Ph: 07 3243 7222 E: samples.brisbane@alsglobal.com

❑GLADSTONE 46 Callemondah Drive Clinton QLD 4680
Ph: 07 7471 5600 E: gladstone@alsglobal.com

❑MACKAY 78 Harbour Road Mackay QLD 4740
Ph: 07 4944 0177 E: mackay@alsglobal.com

❑MELBOURNE 2-4 Westall Road Springvale VIC 3171
Ph: 03 8549 9600 E: samples.melbourne@alsglobal.com

❑MUDGEE 27 Sydney Road Mudgee NSW 2850
Ph: 02 6372 6735 E: mudgee.mail@alsglobal.com

❑NEWCASTLE 5 Rose Gum Road Warabrook NSW 2304
Ph: 02 4968 9433 E: samples.newcastle@alsglobal.com

❑NOWRA 4/13 Geary Place North Nowra NSW 2541
Ph: 024423 2063 E: nowra@alsglobal.com

❑PERTH 10 Hod Way Malaga WA 6090
Ph: 08 9209 7655 E: samples.perth@alsglobal.com

❑SYDNEY 277-289 Woodpark Road Smithfield NSW 2164
Ph: 02 8784 8555 E: samples.sydney@alsglobal.com

❑TOWNSVILLE 14-15 Desma Court Bohle QLD 4818
Ph: 07 4796 0600 E: townesville.environmental@alsglobal.com

❑WOLLONGONG 99 Kenny Street Wollongong NSW 2500
Ph: 02 4225 3125 E: portkembla@alsglobal.com

CLIENT:

OFFICE:

PROJECT:

ORDER NUMBER:

PROJECT MANAGER: **CONTACT PH:**

SAMPLER: **SAMPLER MOBILE:**

COC emailed to ALS? (YES / NO) **EDD FORMAT (or default):**

Email Reports to (will default to PM if no other addresses are listed):

Email Invoice to (will default to PM if no other addresses are listed):

TURNAROUND REQUIREMENTS : (Standard TAT may be longer for some tests e.g.. Ultra Trace Organics)

☐ Standard TAT **(List due date):**

☐ Non Standard or urgent TAT **(List due date):**

ALS QUOTE NO.:

COC SEQUENCE NUMBER (Circle)

COC:	**1**	**2**	**3**	**4**	**5**	**6**	**7**
OF:	**1**	**2**	**3**	**4**	**5**	**6**	**7**

FOR LABORATORY USE ONLY (Circle)

Custody Seal Intact?	Yes	No	N/A
Free ice / frozen ice bricks present upon receipt?	Yes	No	N/A
Random Sample Temperature on Receipt:		˚C	
Other comment:			

RELINQUISHED BY:	**RECEIVED BY:**	**RELINQUISHED BY:**	**RECEIVED BY:**
DATE/TIME:	DATE/TIME:	DATE/TIME:	DATE/TIME:

COMMENTS/SPECIAL HANDLING/STORAGE OR DISPOSAL:

ALS USE	**SAMPLE DETAILS MATRIX: SOLID (S) WATER (W)**			**CONTAINER INFORMATION**		**ANALYSIS REQUIRED including SUITES** (NB. Suite Codes must be listed to attract suite price) Where **Metals** are required, specify **Total** (unfiltered bottle required) or **Dissolved** (field filtered bottle required).								**Additional Information**
LAB ID	**SAMPLE ID**	**DATE / TIME**	**MATRIX**	**TYPE & PRESERVATIVE** *(refer to codes below)*	**TOTAL CONTAINERS**									Comments on likely contaminant levels, dilutions, or samples requiring specific QC analysis etc.
				TOTAL										

Water Container Codes: P = Unpreserved Plastic; N = Nitric Preserved Plastic; ORC = Nitric Preserved ORC; SH = Sodium Hydroxide/Cd Preserved; S = Sodium Hydroxide Preserved Plastic; AG = Amber Glass Unpreserved; AP - Airfreight Unpreserved Plastic
V = VOA Vial HCl Preserved; VB = VOA Vial Sodium Bisulphate Preserved; VS = VOA Vial Sulfuric Preserved; AV = Airfreight Unpreserved Vial SG = Sulfuric Preserved Amber Glass; H = HCl preserved Plastic; HS = HCl preserved Speciation bottle; SP = Sulfuric Preserved Plastic; F = Formaldehyde Preserved Glass;
Z = Zinc Acetate Preserved Bottle; E = EDTA Preserved Bottles; ST = Sterile Bottle; ASS = Plastic Bag for Acid Sulphate Soils; B = Unpreserved Bag.

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		Name	Signature	Name	Signature	Date
Rev.0	A Koscielski	G Metcalfe N Conroy		N Conroy		29/03/16

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Appendix C – Groundwater Sampling Procedure

Arafura Resources Limited

Nolans Project

Groundwater Sampling Procedure

March 2016

Document Status

Version	Author	Reviewer	Approved by	Date	Status

Amendments

Section	Details

Audit Summary

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1. Introduction

The Water Management Plan (WMP) for the Nolans Project (Project) provides a summary of sampling requirements at the site. The WMP has been designed to collect data throughout the construction and operations phase to assess the performance of water management onsite. In order to facilitate consistency in sampling and comply with quality assurance and control methodologies, a series of sampling procedures have been established including:

- Surface Water Sampling Procedure;
- Groundwater Sampling Procedure (this procedure); and
- Sediment Sampling Procedure.

1.1 Purpose

The primary objective of groundwater sampling is to obtain a representative water sample with minimal significant alteration in water chemistry. The collected sample should represent the physical, chemical and biological characteristics of groundwater in the target unit as closely as possible.

1.2 Planning and Equipment

A number of factors must be considered during the field planning phase prior to groundwater sampling. These include consideration of access road conditions, safety requirements, the depth of groundwater and well construction (internal diameter and gravel pack). A summary of equipment and associated suppliers are provided in Table 1-1. All equipment in relation to groundwater sampling should be ordered a minimum of four weeks prior to sampling.

Table 1-1 Summary of Planning

Timing	Details	Supplier
>4 weeks prior to sampling	**Order Lab Bottles** Laboratory bottles Eskies and Cool Bricks	tbc
	Hire / Maintenance Check Low-flow pump Water level gauge or interface probe	Eco Environmental 6/509-511 South Rd, Ashford SA 5031 08 8293 3355 adelaide@ecoenvironmental.com.au Thermo Fisher Scientific 5 Caribbean Dv, Scoresby Vic 3179 03 9757 4377 RentalsAU@thermofisher.com
	Purchase 0.45µm Stericup filters Stericup vacuum pump Low-flow tubing Nitrile gloves Decon N	Eco Environmental 6/509-511 South Rd, Ashford SA 5031 08 8293 3355 adelaide@ecoenvironmental.com.au Thermo Fisher Scientific 5 Caribbean Dv, Scoresby Vic 3179 03 9757 4377 RentalsAU@thermofisher.com
1 day prior to sampling	**Calibrate** Water quality meter	-

2. Groundwater Sampling Procedure

2.1 Groundwater Sampling Events

The monitoring of groundwater at the Project is split into two types as detailed below:

- Standing Water Level Gauging

  Measurement of the standing water level relative to the internal well casings. A groundwater gauging field sheet is provided in Appendix A

- Groundwater Sampling

  Measurement of the standing water level, purging, recording water quality data and sampling. A groundwater purging sheet and Chain of Custody (CoC) sheet is provided in Appendix B and Appendix C respectively.

2.2 Sampling Equipment

Groundwater sampling requires the following:

- Groundwater Gauging Sheet (Appendix A), Groundwater Purging Sheet (Appendix B) and Chain of Custody sheet (Appendix C)
- Water level gauge or interface probe
- Water quality meter (calibrated)
- Low-flow sampling pump/equipment
- Disposable low-flow sampling tubes
- 0.45µm water filters and suction pump
- Eskies and Cool Bricks
- Laboratory bottles
- Nitrile Gloves
- Decontaminated plastic or stainless steel bucket
- Padlock keys and tools to remove well caps and
- Permeant marker.

2.3 Sampling Locations

Groundwater sampling location, number and frequency are indicative only and will be revised in line with completed detailed design

The sampling events and frequencies at each groundwater well are provided in Table 2-1. The groundwater well locations are provided in Figure 2–1 to Figure 2–3.

Table 2-1 Groundwater Sampling Locations

Site ID	Coordinates (GDA94 Zone 53)		Type	Description	Sample Frequency			
					Baseline		Operation	
	Easting	Northing			SWL	SWL and Laboratory*	SWL	SWL and Laboratory*
Mine Site								
MB101A	*tbc*	*tbc*	Boundary (up-gradient)	*tbc*	Automatic Loggers	Monthly	Automatic Loggers	Quarterly
MB101B								
tbc	*tbc*	*tbc*	*tbc*	*tbc*	Quarterly	Biannual	Quarterly	Quarterly
tbc								
Processing Site								
MB201A	*tbc*	*tbc*	Boundary (up-gradient)	*tbc*	Automatic Loggers	Monthly	Automatic Loggers	Quarterly
MB201B								
tbc	*tbc*	*tbc*	*tbc*	*tbc*	Biannual	Quarterly	Quarterly	Quarterly
tbc								
Borefield								
tbc	*tbc*	*tbc*	Monitoring Bores	*tbc*	Quarterly	Biannual	Quarterly / Automatic Logger	Biannual
tbc	*tbc*	*tbc*		*tbc*				
SB008	308109	7479250	Production Bore	*tbc*	Quarterly	Biannual	Quarterly	Biannual
SB015	301290	7479850		*tbc*				
SB021	294454	7482340		*tbc*				
SB025	288460	7483290		*tbc*				
SB027	304196	7484910		*tbc*				

Note: A = shallow monitoring well.
B = deep monitoring well.
* Radionuclides will be tested annually at representative bores only.

To be determined during detailed design phase.

Figure 2-1 Mine Site Groundwater Well Locations

To be determined during detailed design phase.

Figure 2-2 Processing Site Groundwater Well Locations (West)

To be determined during detailed design phase.

Figure 2-3 Borefield Groundwater Wells

2.3.1 Groundwater Sampling Suite

The groundwater sampling suite is provided below. Field measurements are to be collected using the water quality meter during the purging process. Following stabilisation of water quality parameters (see Section 3.1.2) laboratory samples are to be collected.

- Standing water level
- Standing water level and laboratory measurements

  Standing water level and purged water quality characteristics including temperature, pH, electrical conductivity, total dissolved solids, turbidity and oxidation reduction potential

  Laboratory analysis:

  – Total Suspended Solids, Total Hardness and Total Acidity and Alkalinity
  – Major ions ($CaCO_3$, CO_3, HCO_3, Ca, Mg, K, Na, Cl, SO_4, NO_2 NO_3)
  – Metals total and dissolved (0.45 µm field filtered[1]): Al, As, B, Ba, Cd, Co, Cu, Fe, Li, Pb, P, Mn, Hg, Mo, Ni, Rb, Se, Sr, Ag, U, Th and Zn.
  – Radionuclides (U-238, U-234, Th-230, Ra-226, Rn-222, Pb-210, Po-210, Th-232, Ra-228, Th-228). Radionuclides will be tested annually at representative bores only.

Note these are indicative analytes. The final suite will be determined following review of existing baseline data.

Sampling is to be undertaken in accordance with the sampling procedure provided in Section 3.

2.4 Sampling Frequency

Sampling frequency will be determined following detailed design and/or installation of bores and will be monthly, quarterly, biannually or opportunistically following rainfall. Drilling data shows the occurrence of groundwater away for the Nolans deposit is limited. An indicative frequency is provided in Table 2-1.

2.4.1 Environmental Incident Sampling

In the event that an incident occurs where a hazardous substance or chemical is discharged to the environment the EHS Manager will determine if an investigation is warranted based on severity of the incident, the requirements of the Hazardous Substances Management Plan and Emergency Response Plan.

[1] Samples for dissolved metals are field filtered using 0.45 µm Stericup filter.

3. Groundwater Sampling Procedures

3.1.1 Standing Water Level Gauging

Groundwater gauging is to be undertaken in accordance with the following:

1. Complete groundwater gauging sheet for each sample location;
2. Gauging

   Gauge water level relative to Top of Casing (TOC) using an electronic interface meter. The well cap should be removed and the well allowed to stabilise before measurements are made. Where possible, depth measurements should be recorded to the nearest 1 mm (i.e. 0.001 m).

   A groundwater gauging sheet is provided in Appendix A.

3.1.2 Groundwater Sampling Methodology

Groundwater sampling is to be conducted in accordance with the following:

1. Complete groundwater gauging sheet for location;
2. Gauging

   Gauge water level relative to Top of Casing (TOC) using an electronic interface meter. The well cap should be removed and the well allowed to stabilise before measurements are made. Where possible, depth measurements should be recorded to the nearest 1 mm (i.e. 0.001 m);
3. Decontamination

   Reusable sampling equipment such as the pump and cables should be decontaminated prior and at completion of sampling at each sample location. Decontamination can be undertaken by submerging the pump and cables in a mixture of Decon N / Decon 90 and water.
4. Pump Installation

   Insert pump into well with care to avoid excessive disturbance and re-suspension of sediment within the well. The pump intake should be suspended inside the well screen so as to minimise the volume of stagnant groundwater required to be purged and intercept the inflowing groundwater from the target formation.
5. Purging

   Commence purging of well, the aim of this process is to remove ‘stagnant’ groundwater from the well so that groundwater representative of the surrounding unit. Water quality parameters should be recorded at regular intervals (i.e. every 2 to 5 minutes or every 2 to 5 litres) on the groundwater gauging sheet (Appendix B).

   Parameters are to stabilise prior to sampling, they are consider stabilised when three consecutive readings are within the following limits:

   – 10% for Dissolved Oxygen;
   – ± 3% Electrical Conductivity;
   – 0.05 pH units for pH;
   – ± 0.2 °C for Temperature; and
   – ± 10 mV Redox.

   Contingency – No Parameter Stabilisation

   If after prolonged purging the parameters do not stabilise to within the specified limits, the original well and gravel pack volume should be calculated and ensure at least 3 well volumes of groundwater has been purged.

Contingency – Pumped Dry

Low yielding wells that are purged dry should be left to recover. Following recovery of groundwater levels in the well, sampling can proceed on the assumption that the groundwater represents inflow from the unit screened by the well. In this instance, measurement of stabilisation parameters should record a minimum of three consecutive readings prior to sampling.

6. Groundwater Sampling

   A groundwater sample should be collected after the measured parameters have stabilised. Commonly the purging device is used to sample the groundwater. Sampling should be undertaken so as to minimise the entry of air into the sample – run the outflow from the sampling device down the side of the container, rather than allowing it to cascade into the container.

   Once collected, groundwater samples should be labelled and stored in ice chilled cooler boxes. Samples should be kept out of the sun. Samples should be returned to the laboratory under Chain of Custody (COC) documentation as detailed in Section 3.2.

   – Field Filtering

     A total metals sample (not filtered) and a dissolved metal sample should be collected. The dissolved metal sample requires field filtration through a disposable 0.45 µm filter.

7. Waste Disposal

   Purged groundwater is to be pumped/tipped on to the ground and all used disposable sampling equipment should be stored for disposal at the Process Site including filters, tubing and bladders.

8. Electronic Transfer

   All purging results, duplicate locations and CoC are to be scanned and kept on file. The purging results are to be entered into the groundwater database.

3.2 Sample Dispatch

Sediment samples have potential to deteriorate following collection. Samples are to be placed into onsite fridge pending dispatch to laboratory. At completion of the sampling round bottles are to be packed into eskys and ice bricks placed on top of samples and transferred to Alice Springs haulage depot. Samplers are to contact the haulage companies and laboratory to inform of sample delivery and requirements to keep refrigerated.

The sampler is to inform the laboratory of sample postage and provide a completed Chain of Custody (CoC). An example CoC is provided in Appendix C.

4. Discharge Notifications

Discharges from the Site will be assessed on a case by case basis to determine if formal notifications to the DME and NT EPA are required. All external communication of incidents will be signed and approved by EHS Manager and/or General Manager.

In general, if there is a discharge of contained/managed water from the Project (i.e. collapse of flood levees or overflow of stormwater basins) the DME and NT EPA will be notified. A summary of the notification requirements is provided in Table 4-1.

Table 4-1 Formal Notification Requirements

Entity	Trigger	Timeframe and Contact Details	Incident Reporting Details
Department of Mines and Energy (DME)	Incident which causes minor environmental impact with some minor actual or potential hard to the environment.	As soon as practicable. Mineral.Info@nt.gov.au	The Section 29 Notification of Environmental Incident Form requires the following details: • Site and operator details. • Location occurred and area impacted (GPS coordinates); • Date and time; • Description of incident • Emergency and remedial actions taken. • Nature of impact and severity; • Current situation; • Details of sampling undertaken; and • Notification status internally and externally. The form is to be signed by the EHS manager and/or General Manager. A blank form is provided in Appendix D
Northern Territory Environmental Protection Authority (NT EPA)	Incident which causes, or is threatening or may threaten to cause pollution resulting in material environmental harm or serious harm.	< 24 hrs post incident ntepa@nt.gov.au pollution@nt.gov.au	The Section 14 Incident Report Form requires the following details: • Incident causing or threatening to cause pollution; • Location occurred and area impacted; • Date and time; • How the pollution has occurred, is occurring or may occur; • Attempts made to prevent, reduce, control, rectify, investigation and/or clean up the pollution or resultant environmental harm caused or threatening to be caused by the incident; and • Operator details. The form is to be signed by the EHS Manager and/or General Manager. A blank form is provided in Appendix E.

Qualifying triggers requiring submittal of Section 14 Incident Report to NT EPA are any of the following:

- is not trivial or negligible in nature; or
- consists of an environmental nuisance of a high impact or on a wide scale; or
- results, or is likely to result in $50,000 or more in taking action to prevent or minimise environmental harm or rehabilitate the environment; or results in actual or potential loss or damage to value of $50,000 or more of the prescribed amount (whichever is the greater).

Appendices

Appendix A – Groundwater Gauging Sheet

GROUNDWATER GAUGING FIELD SHEET

Date:	Sampler:

GROUNDWATER LEVELS

Standing water level is measured from the top of internal casing (TOIC).

Groundwater Well	Easting	Northing	Top of Internal Casing (m AHD)	Standing Water Level (m BTOIC)	Standing Water Level (m AHD)	Comments

Additional Comments

Appendix B – Groundwater Purging Sheet

GROUNDWATER PURGING AND SAMPLING FIELD SHEET

PROJECT DETAILS	Borehole ID:
Depth to Water Table Before Sampling (m TOC):	
Depth to Bottom of Casing (m TOC):	Date:
Depth to Water Table After Sampling (m TOC):	QA Collected:
Sampler:	Sample Method:

FIELD PARAMETERS (minimum of five)

Time	Volume (L)	D.O (%)	D.O (mg/L)	TDS (mg/L)	Turbidity (NTU)	E.C (us/cm)	pH	Eh (mV)	Temp (^0C)	Comments

Post Sample Parameters

Number of Bottles:	Comments:

Appendix C – Chain of Custody Form

CHAIN OF CUSTODY

ALS Laboratory: please tick →

❑ADELAIDE 21 Burma Road Pooraka SA 5095
Ph: 08 8359 0890 E: adelaide@alsglobal.com

❑BRISBANE 32 Shand Street Stafford QLD 4053
Ph: 07 3243 7222 E: samples.brisbane@alsglobal.com

❑GLADSTONE 46 Callemondah Drive Clinton QLD 4680
Ph: 07 7471 5600 E: gladstone@alsglobal.com

❑MACKAY 78 Harbour Road Mackay QLD 4740
Ph: 07 4944 0177 E: mackay@alsglobal.com

❑MELBOURNE 2-4 Westall Road Springvale VIC 3171
Ph: 03 8549 9600 E: samples.melbourne@alsglobal.com

❑MUDGEE 27 Sydney Road Mudgee NSW 2850
Ph: 02 6372 6735 E: mudgee.mail@alsglobal.com

❑NEWCASTLE 5 Rose Gum Road Warabrook NSW 2304
Ph: 02 4968 9433 E: samples.newcastle@alsglobal.com

❑NOWRA 4/13 Geary Place North Nowra NSW 2541
Ph: 024423 2063 E: nowra@alsglobal.com

❑PERTH 10 Hod Way Malaga WA 6090
Ph: 08 9209 7655 E: samples.perth@alsglobal.com

❑SYDNEY 277-289 Woodpark Road Smithfield NSW 2164
Ph: 02 8784 8555 E: samples.sydney@alsglobal.com

❑TOWNSVILLE 14-15 Desma Court Bohle QLD 4818
Ph: 07 4796 0600 E: townesville.environmental@alsglobal.com

❑WOLLONGONG 99 Kenny Street Wollongong NSW 2500
Ph: 02 4225 3125 E: portkembla@alsglobal.com

CLIENT:

OFFICE:

PROJECT:

ORDER NUMBER:

PROJECT MANAGER: **CONTACT PH:**

SAMPLER: **SAMPLER MOBILE:**

COC emailed to ALS? (YES / NO) **EDD FORMAT (or default):**

Email Reports to (will default to PM if no other addresses are listed):

Email Invoice to (will default to PM if no other addresses are listed):

TURNAROUND REQUIREMENTS : (Standard TAT may be longer for some tests e.g.. Ultra Trace Organics)

☐ Standard TAT **(List due date):**

☐ Non Standard or urgent TAT **(List due date):**

ALS QUOTE NO.:

COC SEQUENCE NUMBER (Circle)

COC:	**1**	**2**	**3**	**4**	**5**	**6**	**7**
OF:	**1**	**2**	**3**	**4**	**5**	**6**	**7**

FOR LABORATORY USE ONLY (Circle)

Custody Seal Intact?	Yes	No	N/A
Free ice / frozen ice bricks present upon receipt?	Yes	No	N/A
Random Sample Temperature on Receipt:		˚C	
Other comment:			

RELINQUISHED BY:	**RECEIVED BY:**	**RELINQUISHED BY:**	**RECEIVED BY:**
DATE/TIME:	DATE/TIME:	DATE/TIME:	DATE/TIME:

COMMENTS/SPECIAL HANDLING/STORAGE OR DISPOSAL:

ALS USE	**SAMPLE DETAILS MATRIX: SOLID (S) WATER (W)**			**CONTAINER INFORMATION**		**ANALYSIS REQUIRED including SUITES** (NB. Suite Codes must be listed to attract suite price) Where **Metals** are required, specify **Total** (unfiltered bottle required) or **Dissolved** (field filtered bottle required).								**Additional Information**
LAB ID	**SAMPLE ID**	**DATE / TIME**	**MATRIX**	**TYPE & PRESERVATIVE** *(refer to codes below)*	**TOTAL CONTAINERS**									Comments on likely contaminant levels, dilutions, or samples requiring specific QC analysis etc.
				TOTAL										

Water Container Codes: P = Unpreserved Plastic; N = Nitric Preserved Plastic; ORC = Nitric Preserved ORC; SH = Sodium Hydroxide/Cd Preserved; S = Sodium Hydroxide Preserved Plastic; AG = Amber Glass Unpreserved; AP - Airfreight Unpreserved Plastic
V = VOA Vial HCl Preserved; VB = VOA Vial Sodium Bisulphate Preserved; VS = VOA Vial Sulfuric Preserved; AV = Airfreight Unpreserved Vial SG = Sulfuric Preserved Amber Glass; H = HCl preserved Plastic; HS = HCl preserved Speciation bottle; SP = Sulfuric Preserved Plastic; F = Formaldehyde Preserved Glass;
Z = Zinc Acetate Preserved Bottle; E = EDTA Preserved Bottles; ST = Sterile Bottle; ASS = Plastic Bag for Acid Sulphate Soils; B = Unpreserved Bag.

Appendix D – DME Section 29 Notification of Environmental Incident

Notification of an Environmental Incident

Section 29 of the Mining Management Act

**Forward completed form to: Mining Compliance Division,
Department of Mines and Energy**

Email: mineral.info@nt.gov.au (preferred) or Fax: (08) 89996527

PLEASE TYPE OR PRINT CLEARLY

Please ensure that you have read the Draft Guideline - Environmental incident reporting under Section 29 of the Mining Management Act (July 2012) [167kb]]

NAME OF MINING SITE		
NAME OF OPERATOR		
DATE & TIME OF INCIDENT		
NAME OF PERSON NOTIFYING		
POSITION/TITLE		
CONTACT PERSON		
CONTACT DETAILS	Business:	Mobile
	Fax:	E-mail:
INCIDENT LOCATION (use GPS co-ordinates, attach map, etc as appropriate)		
DESCRIPTION OF INCIDENT		
EMERGENCY & REMEDIAL ACTIONS TAKEN		

ENVIRONMENTAL DETAILS

NATURE OF IMPACT AND SEVERITY (Volume/ of spillage, area impacted, wildlife/vegetation/ erosion, etc) DME severity classification: 1 2 3 4 *Refer to pages 3 to 5 of the Draft Guideline - Environmental incident reporting under Section 29 of the Mining Management Act (July 2012) [167kb]]*	
CURRENT SITUATION (Potential / ongoing / ceased / etc)	
DETAILS OF ANY SAMPLES TAKEN (when / where / type / number / time for results /etc)	

OPERATOR INTERNAL REPORTING

Has the incident been reported internally? **YES / NO** If so, to whom	Name: Position:
Operator reference number (where applicable/available)	

HAS THE DEPARTMENT BEEN NOTIFIED EARLIER?	☐**YES** ☐ **NO**
WHO WAS NOTIFIED	
HOW (phone/email/fax)	
WHEN (date & time)	
BY WHOM	

Signed: ___________________________________ *Date:* _______________

NAME*:* ___________________________________

POSITION: _______________________________

OFFICE USE ONLY	
RECEIVED BY	
DATE	TIME

Appendix E – NT EPA Section 14 Incident Report

SECTION 14 INCIDENT REPORT (*Waste Management and Pollution Control Act*)

Date and Time of Notification:	
Person / Company:	
Incident:	

(a) the incident causing or threatening to cause pollution	
(b) the place where the incident occurred	
(c) the date and time of the incident	
(d) how the pollution has occurred, is occurring or may occur	
(e) the attempts made to prevent, reduce, control, rectify or clean up the pollution or resultant environmental harm caused or threatening to be caused by the incident	
(f) the identity of the person notifying the NT EPA	

GHD

Level 5, 66 Smith Street Darwin NT 0800
GPO Box 351 Darwin NT 0801
T: (08) 8982 0100   F: (08) 8981 1075   E: drwmail@ghd.com.au

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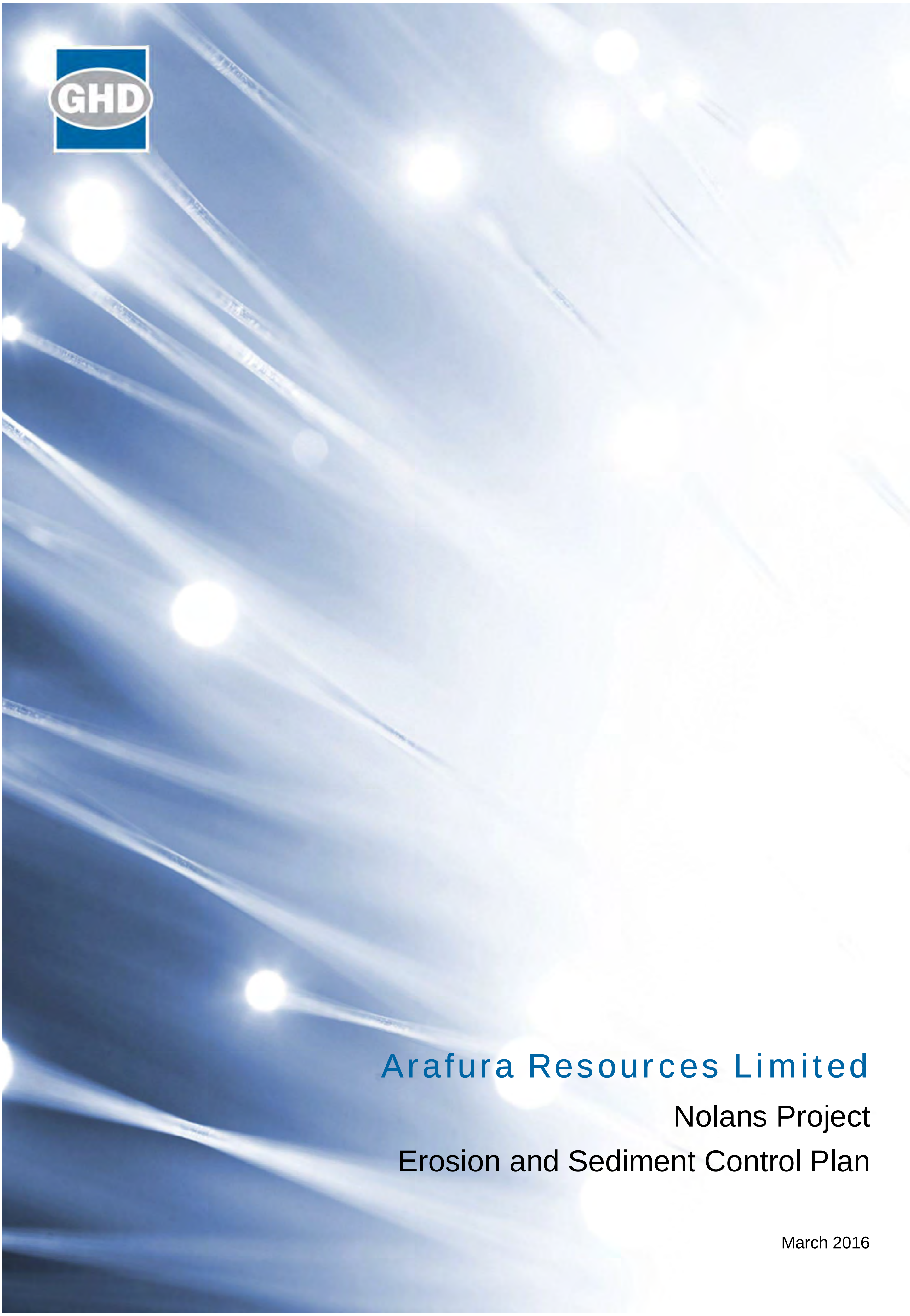
Document Status

Revision	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Rev.0	A Koscielski	G Metcalfe N Conroy		N Conroy		29/03/16

www.ghd.com

Appendix D – Erosion and Sediment Control Plan



Arafura Resources Limited

Nolans Project

Erosion and Sediment Control Plan

March 2016

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1. Introduction

1.1 Purpose

The Erosion and Sediment Control Plan (ESCP) forms part of the Water Management Plan (WMP) and has been prepared to:

- Provide a high level strategy for the management of erosion and sediment across the Project areas. The ESCP covers the construction, operation and closure phases; and
- Inform the reader/contractor of Erosion and Sediment Control (ESC) design philosophy, and to establish measures for installation and maintenance activities.

The ESCP has been aligned with the Mine Closure Plan (EIS, Appendix W). The Mine Closure Plan (MCP) is a live document that will evolve as new information is gathered and additional studies are undertaken to address any information shortfalls identified. However, closure goals will include the management of erosion and sediment with reference to this Plan and the principles detailed in Section 3.

1.2 Objectives

The objective of this ESCP is to minimise the potential for erosion and sedimentation during the construction, operation and closure phases through:

- Identifying and locating the controls required to divert stormwater runoff away from disturbed areas;
- Preventing release of sediment laden stormwater runoff from the disturbed areas into the surrounding environment, and
- Encouraging prompt rehabilitation of Project construction, operational and closure areas through appropriate revegetation strategies.

This ESCP is based on the following standards and guidelines:

- Best Practice Erosion and Sediment Control for Building and Construction Projects (November 2008), International Erosion Control Association (IECA) Australasia;
- Erosion and Sediment Control Guidelines Built Environment, the former Department of Natural Resources, Environment and the Arts, Northern Territory Government;
- Northern Territory Minerals Council (Inc.) and the Mines and Petroleum Management Division of the Northern Territory Government, 2004, TEAM NT: Technologies for Environmental Advancement of Mining in the Northern Territory: Toolkit, D.R. Jones and M. Fawcett, principal authors; and
- Erosion and Sediment Control Plans Fact Sheet, Land Management Unit, Natural Resources Division.

1.3 Report Structure

The structure of the report is summarised below:

- Introduction;
- Project Conditions;
- Erosion and Sediment Control Design Philosophy;
- Overview of Erosion and Sediment Control Measures; and

- Maintenance Requires.

2. Project Conditions

2.1 Topography

The mine site lies at the head of Kerosene Camp Creek valley on the north facing slopes of an east – west trending ridge of the Reynolds Range. The processing site is situated on the southern slopes of the same ridge. Topographic elevation is 886 m above sea level (m ASL) at Mt Boothby to the east of the mine site, and 1006 m ASL at Mt Freeling to the west. Most of the Kerosene Camp Creek valley floor at the mine site is typically between 650 and 700 m ASL, and longitudinal gradients along local creeks to the north and south of the ridge line are typically less than 0.5 percent, with steeper gradients of about 10 percent on isolated hills.

2.2 Climate

2.2.1 Rainfall and Evaporation

The mean annual rainfall is approximately 310 mm, with a seasonal pattern of more summer rainfall than winter rainfall. Average monthly rainfall totals range from 4.7 mm in August to 65.8 mm in February. Average three-monthly rainfall totals range from 18.3 mm in June/July/August to 178.7 mm in December/January/February. However, any month can receive relatively large rainfall totals, or little or no rain at all.

Potential evaporation is greatest in December and January at 375 mm and coincides with months when rainfall can be highest. Rates of potential evaporation are significantly lower from May to August coinciding with lower mean rainfall and temperatures. The annual average potential evaporation is approximately 3,000 mm, which far exceeds the annual average rainfall of 310 mm.

The rainfall and evaporation rates are provided in Table 2-2.

Rainfall Statistics

Rainfall at the Project is generally characterised by infrequent and intense rainfall events, single events can deliver > 50 mm within 24 hour. The Bureau of Meteorology Intensity–Frequency–Duration (IFD) indicates 305 mm for a 1 in 100 year, 72 hour rainfall event.

A summary of the IFDs are provided in Table 2-1.

Table 2-1 IFD Rainfall Depth (mm) (Source: BOM IFD AR&R87 Tool)

Duration	Return Period (Years)						
	1	2	5	10	20	50	100
5	5	6	9	10	12	15	17
6	5	7	10	12	14	17	19
10	7	10	14	16	19	24	27
20	11	15	21	25	30	37	43
30	14	19	27	32	38	47	54
1	19	25	37	44	53	66	76
2	24	32	47	57	69	86	100
3	26	35	53	64	79	99	115
6	31	42	64	79	97	123	145
12	36	50	78	97	120	155	182
24	45	62	97	121	151	194	229
48	56	77	120	148	185	238	281

Duration	Return Period (Years)						
	1	2	5	10	20	50	100
72	60	83	129	161	200	257	305

2.2.2 Temperature and Humidity

The Project area experiences hot and arid conditions. The hottest months are November to March, with monthly mean daily maximum temperatures above 35 °C, and monthly mean daily minimum temperatures not dropping below 18 °C. The coolest months are May to August, with monthly mean of daily maximum temperatures remaining at or below 25.5 °C, and monthly mean daily minimum temperatures not rising above 9.5 °C.

The average humidity at the Project is 40% at 09:00 and 25% at 15:00, consistent across the year with monthly afternoon humidity readings being 15% lower than the morning. The highest levels of humidity are experienced in June at 53%. This coincides with lower temperatures occurring.

The temperature and humidity rates are provided in Table 2-2.

2.2.3 Wind

The winds at the Project are predominant south easterly wind direction throughout the year. The average wind speeds range from 2.50 to 3.17 m/s (9.0 to 11.4 km/h) with an annual average of 2.86 m/s (10.3 km/h).

The wind roses are provided in Figure 2-1 and speeds are summarised in Table 2-2

Table 2-2 Summary of Climate Statistics (BoM 2016; Territory Grape Farm NT 1987-2014)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Rainfall (mm)												
Highest	280.4	**342.2**	109.2	151.7	136.3	53.8	34.2	39.4	96.6	56.8	119.2	119.2
95th percentile	159.0	**244.2**	96.9	89.9	100.1	48.7	21.3	26.9	41.7	51.3	81.4	109.9
Mean	62.4	**65.8**	21.9	18.0	23.3	8.7	4.9	4.7	10.3	15.3	30.9	50.5
5th percentile	3.8	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.9	**8.9**
Lowest	**2.2**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.0
Evaporation (mm)												
Total	**375**	300	290	210	150	125	145	180	200	300	350	**375**
Temperature (°C)												
Maximum[1]	**37.3**	36.2	34.3	30.5	25.5	22.2	22.5	25.3	30.5	33.3	35.6	36.3
Minimum[2]	**21.9**	21.6	19.5	14.6	9.5	6.2	5.2	7.1	12.1	15.6	18.8	21.1
Humidity (%)												
Mean 9 am	38	40	37	37	47	**53**	51	38	32	32	34	37
Mean 3 pm	24	**28**	27	25	27	**28**	**28**	22	21	21	22	26
Wind (km/h)												
Mean 9 am	17.0	18.1	**19.7**	18.9	15.2	12.8	14.3	17.3	18.2	19.6	18.2	18.0

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean 3 pm	15.8	**16.7**	16.6	14.9	14.2	13.5	14.0	16.0	15.5	14.8	14.1	14.5

Notes: [1] Monthly mean maximum temperature is the average of the available daily maxima for that month.

[2] Monthly mean minimum temperature is the average of the available daily minima for that month.

Highest values are indicated in bold.

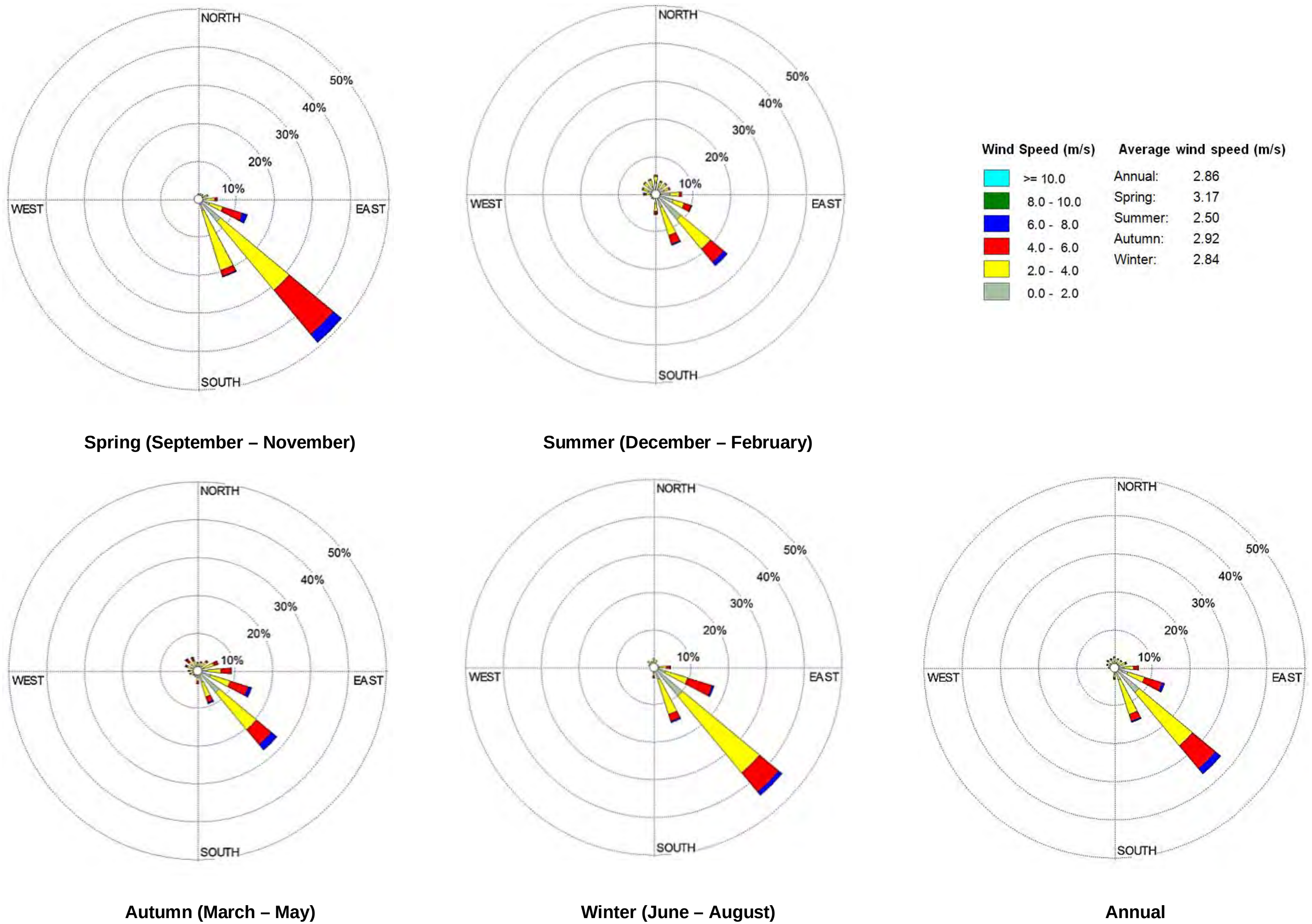


Figure 2-1 Annual and Seasonal Wind Roses (Source: BOM, Territory Grape Farm Station No. 015643)

2.3 Surface Hydrology

2.3.1 Watercourses

Kerosene Camp Creek is an ephemeral creek and flows through the centre of the mine site before joining the Woodforde River 12 km further to the north. Kerosene Camp Creek has a catchment area of approximately 18 km^2 upstream of the mine site.

Nolans Creek is a tributary of Kerosene Camp Creek and has a catchment area of 26 km^2 upstream of the mine site. It flows parallel to the eastern boundary of the proposed Flotation Tailings Storage Facility and will pass between Waste Rock Dumps 2 and 6 before it joins Kerosene Camp Creek.

Catchments upstream of creek crossing points along the access road from the Stuart Highway drain towards the Southern Basins and are typically less than 3 km^2, but with one catchment of about 12 km^2. Areas draining towards the Project are typically less than 1 km^2 in extent and channels are ill-defined with runoff likely to be dispersed across the south facing hillslope.

Semi-arid regions such as the area in which the Project is located are typically characterised by conditions in which actual evaporation during rainfall events closely matches rainfall and virtually all rainfall evaporates, resulting in almost no surface runoff. Therefore, the occurrence of surface runoff and flows within local creeks is infrequent and only occurs during larger rainfall events associated with the occasional southward extension of a tropical monsoon trough or periodic incursion of north-west cloud bands over the interior of the continent.

Local creek beds are mobile with deep sand deposition and banks that show signs of active erosion. Creek channels are typically 1.0 m deep with a base width of 5 m. Intense, short duration rainfall events can be expected to occur over the Project area and the relatively shallow depth of creek channels will lead to out-of-bank flow and possibly temporary and short-term flooding of adjacent areas.

2.3.2 Flow Records

Long-term gauging of flow in watercourses that traverse or flows near to the Project has been carried out at one location, namely Arden Soak Bore on the Woodforde River. A summary of gauging is provided in Table 2-3. This gauge is located approximately 26 km downstream of the mine site and comprises a water level gauge board in the sandy river bed.

A second gauge at Allungra Waterhole is located about 42 km to the east and is outside the catchment of the Project and its infrastructure. The NT Department of Land Resource Management is responsible for maintaining these gauging stations and records.

The Arden Soak Bore and Allungra gauges both provide records of water level from which discharge can be calculated. Gauging of water levels is also carried out at a third location, Pine Hill on the Hanson River, which is situated 33 km to the west, also outside the catchment of the Project and its infrastructure. No flow records are available for this latter gauge.

Arden Soak Bore on the Woodforde River measures runoff from a catchment area (393 km^2) that is an order of magnitude greater than that subtended by the mine site (54 km^2). However, given the similarity of catchment conditions the recorded time series of water levels and discharge are likely to be indicative of the pattern of runoff (but not the magnitude) from catchments at the mine site.

An analysis of the flow record at Arden Soak Bore (Figure 2-2) confirms that flow events are relatively infrequent with only 25 percent of days during the 41 year record having a total daily flow greater than 3 ML (arbitrarily selected threshold discharge of 0.03 m^3/s). Runoff is most likely in months during the summer season, December to March (Figure 2-3). The low frequency of flow events suggests that only one or two flow events can be expected in most years (Figure 2-4).

The maximum recorded flow at Arden Soak Bore on Woodforde River is 206 m^3/s and occurred in January 2010 (Figure 2-5) with a measured water depth of 3.7 m. Whilst this flow was recorded 26 km downstream of the proposed Project it serves to show the relatively ‘flashy’ response and short duration of flow events for drainage systems in this region.

Both the flow frequency curve (Figure 2-2) and hydrograph of the maximum recorded flow event (Figure 2-5) at Arden Soak Bore on Woodforde River illustrate the absence of baseflow (surface flow sustained by groundwater). However, anecdotal evidence[1] states that during 2010 and 2011 (wet years) water drained out of the local hills for months and a 'soak' upstream of the mine site was wet most of the year. This suggests that surface runoff infiltrates into the alluvium of creek channels where it will form shallow groundwater flow moving down gradient along the creek channel.

The volume of surface runoff relative to locally recorded rainfall for the January 2010 event at Arden Soak Bore is estimated to be nine percent and indicates relatively high rainfall losses of over 90 percent. What proportion of this 'loss' infiltrates to a shallow aquifer and what proportion is lost to the atmosphere through evapotranspiration is uncertain but serves to confirm the typically low rate of surface runoff in the area.

Table 2-3 Hydrometric Gauges

Type	Gauge Number	Name	Latitude	Longitude	Record Start	Record End	Record Length (years)	Location Relative to Mine Project
River Flow	0280010	Woodforde River - Arden Soak	22.367	133.324	1974	open	41	Same river system 26 km downstream.
River Flow	0280004	Allungra Creek - Allungra Waterhole	22.689	133.631	1996	open	19	Different river system 42 km to east.
River Height	0280010	Woodforde River - Arden Soak Bore	22.367	133.324	1974	open	41	Same river system 26 km downstream.
River Height	0280004	Allungra Creek - Allungra Waterhole	22.689	133.631	1996	open	19	Different river system 42 km to east.
River Height	0280021	Hanson River At Pine Hill	22.367	133.025	1968	1977	9	Different river system 33 km to west.
Surface Water Quality	0280010	Woodforde River - Arden Soak Bore	22.367	133.324	1974	open	41	Same river system 26 km downstream.
Surface Water Quality	0280004	Allungra Creek - Allungra Waterhole	22.689	133.631	1996	open	19	Different river system 42 km to east.

[1] Nolans Feasibility Study – Preliminary Studies Project Drainage and Land Tenure, AMC Consultants, February 2015. Comments on report by K Hussey.

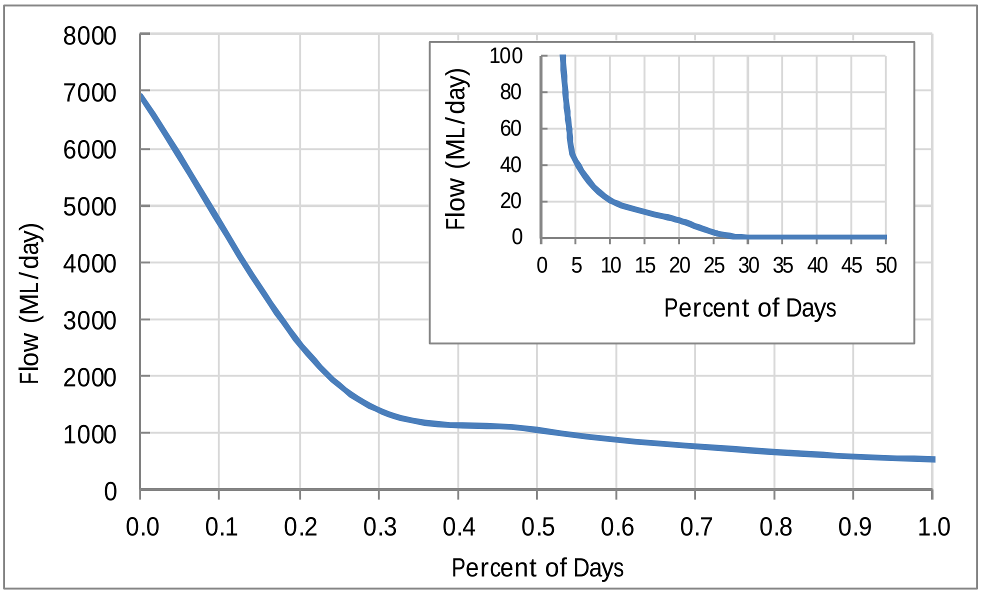


Figure 2-2 Flow frequency curve for Woodforde River at Arden Soak Bore

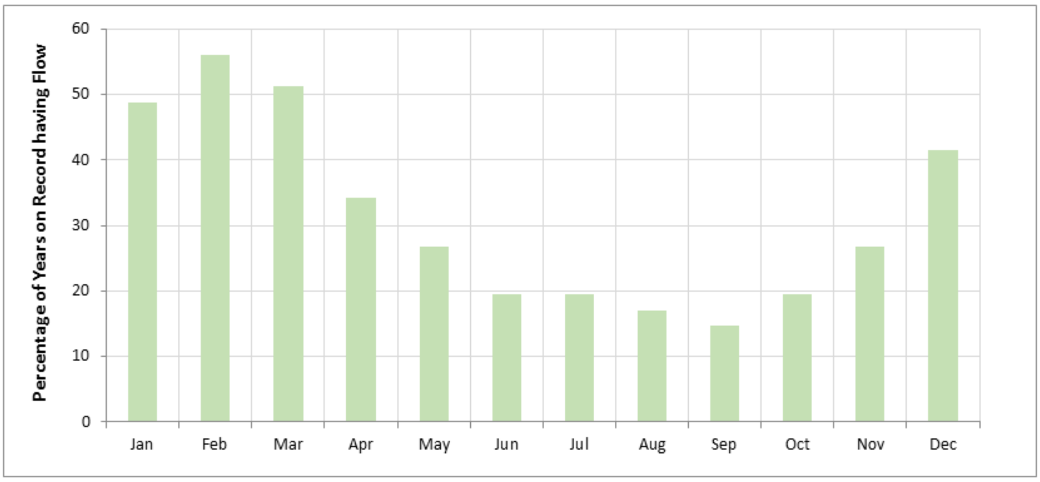


Figure 2-3 Occurrence of flow at Arden Soak Bore on Woodforde River

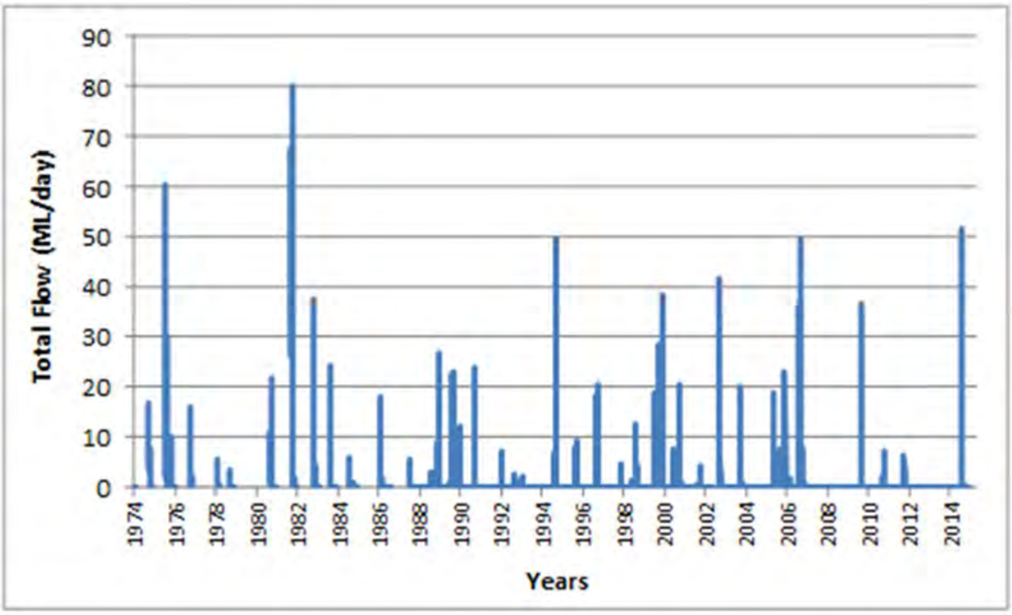


Figure 2-4 Time series of recorded flow at Arden Soak Bore on Woodforde River

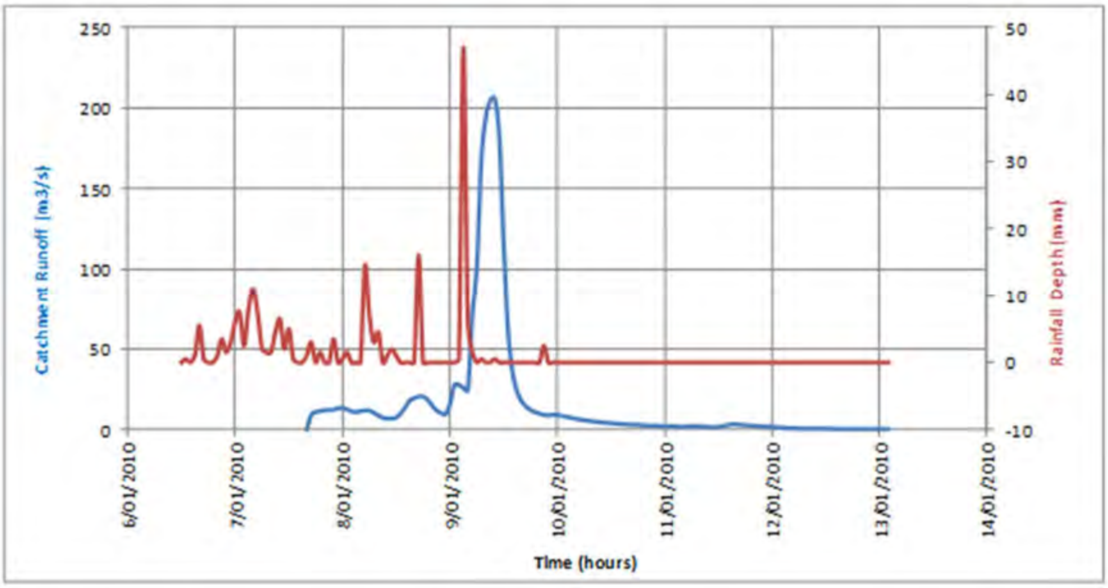


Figure 2-5 Maximum recorded flow at Arden Soak Bore on Woodforde River

2.4 Vegetation

The vegetation types that will be affected by the Project comprise 14 distinct communities and sub-communities which are presented in Table 2-4.

A maximum total of 4,161.56 ha may be required to be cleared for the Life of Mine (LOM) Project footprint with the major vegetation units being cleared including:

- Mulga shrubland on sandy red earths over tussock grasses – 33.92%
- Mulga shrubland on sandy red earths over tussock grasses / Mulga shrubland on sandy red earths over spinifex – 26.73% and
- Mixed woodland over tussock grasses – 15.79%.

Table 2-4 Vegetation Communities (Source: GHD, 2016a)

V*	Description	Area to be cleared (% of total)	Total (ha)
1	Riparian woodland along water courses and drainage channels	5.77%	239.96
2a	Mulga shrubland on sandy red earths over spinifex	0.14%	5.90
2b	Mulga shrubland on sandy red earths over tussock grasses	33.92%	1411.45
2c	Mulga shrubland on sandy red earths over chenopods	0.84%	34.82
3a	Mixed woodland over tussock grasses	15.79%	657.18
3b	Mixed woodland over spinifex	0.26%	10.97
3c	Mixed woodland over a highly disturbed understorey dominated by Cenchrus ciliaris	0.16%	6.46
4	Triodia schinzii hummock grassland on red clayey sands	0.00%	0.00
5	Hakea/Senna shrubland on calcareous alluvial plains and low rises	5.59%	232.49
6	Eucalyptus (mallee)/Acacia kempeana/Triodia shrubland on rocky slopes	1.44%	59.86
7	Acacia/Triodia shrubland on rocky outcrops	4.95%	205.99
8	Rocky gneiss or schist outcrops with no spinifex	0.01%	0.37
9	Acacia kempeana and/or Mulga shrubland on gravel	1.07%	44.44
10	Claypans with chenopods and herbs	0.00%	0.12
11	Cottonbush chenopod shrubland on highly erodible duplex soils	0.09%	3.55
12	Triodia basedowii hummock grassland on sand plains	2.53%	105.39

V*	Description	Area to be cleared (% of total)	Total (ha)
13	Senna shrubland on quartz	0.14%	5.96
14	Coolabah woodland on claypans	0.00%	0.00
2a/2b	Mulga shrubland on sandy red earths over tussock grasses / Mulga shrubland on sandy red earths over spinifex	26.73%	1112.43
2b/3a	Mulga shrubland on sandy red earths over tussock grasses / Mixed woodland over tussock grasses on alluvial plains	0.13%	5.23
3a/12	Mixed woodland over tussock grasses on alluvial plains / Cottenbush chenopod shrubland on highly erodible duplex soils	0.12%	5.05
3b/2b	Mixed woodland over spinifex on alluvial plains / Mulga shrubland on sandy red earths over tussock grasses.	0.32%	13.35
	Existing disturbance.	0.01%	0.59
	Total	100.00%	4,161.56

2.5 Soils

A geotechnical assessment of the mine site was undertaken by Lycopodium Minerals in 2010 (Lycopodium Minerals, 2010). The assessment indicated soils at the mine site generally comprise clayey sand (colluvium) from surface to approximately 1 m Below Ground Level (BGL). Laboratory testing undertaken on samples indicated 62% sand, 34% silt/clay and 4% gravel. A layer of gravelly sand to sandy gravel is present below the colluvium. The test pits met refusal on gneiss at depths ranging from 0.2 to 2.4 m BGL.

For the purpose of erosion and sediment control a precautionary approach has been undertaken and it is assumed that the soils are dispersive following disturbance.

3. Erosion and Sediment Control Design Philosophy

3.1 Introduction

Soil erosion can occur through several individual or combined processes. In general soil erosion is driven by water, wind or physical disturbance which causes particles to become displaced. It can be natural geological erosion or accelerated human-induced erosion.

A summary of erosion processes is provided in Table 3-1.

Table 3-1 Erosion Processes (IECA, 2009)

Aspect	Forms of Water Erosion	Description	Factors Affecting Erosion
Water Erosion	Splash Erosion (raindrop)	The spattering of soil particles caused by the direct impact of precipitation onto an exposed surface. The soil particles are typically moved distances up to 1 m when initially dislodged. These particles may subsequently be transported by surface water runoff. Splash erosion is minimised if soils/tailings/residue storage facilities have a water coverage greater than 2 mm.	The factors which affect water erosion include: • Soil with low surface cover; • Shallow surface soils overlying low permeable subsoils/rock; • Surface soils with high percentage of fine sand or silts; • Surface soils that are hard setting or have a surface crust; • Soils with low levels of organic matter; and • Soil with dispersive properties.
	Sheet Erosion (includes splash erosion)	Uniform removal of soil in thin layers from sloping land. Sheet erosion is minimised through stabilisation of surfaces through practices such as revegetation.	
	Rill Erosion	Rill erosion generally occurs by the removal of soil by water concentrated into small defined channels on sloping land. Rills can be up to 300 mm deep. Rill erosion is minimised through stabilisation of surfaces through practices such as revegetation.	
	Gully Erosion	Gully erosion is similar to rill erosion but produces deeper channels generally greater than 300 mm.	
	Tunnel Erosion	Tunnel erosion is the removal of subsoils in a sub-surface tunnel (i.e. out of sight). It generally occurs near gullies, creek lines or constructed embankments in dispersive soils or where a weak drainage path is already present.	
	Watercourse Erosion	Watercourses naturally transfer sediments downstream. However, a modification of stream banks often leads to instability and erosion.	
Wind Erosion	Surface Creep	Rolling and sliding of large particles (>1 mm) which are too heavy to be lifted in the air. The particle rolls and dislodges other soil particles by hitting into them.	The factors which affect wind erosion include: • Soil with low surface cover; • Dry and High/consistent wind environments; and • Soil characteristics including its binding potential and surface roughness.
	Saltation	Wind directly causing particles generally with a diameter of 0.1 to 0.5 mm to hop and bounce across the surface. The particle then dislodges other particles on impact.	
	Suspension	Movement of small dust (<0.1 mm) particles into the air. The particles can rise high above the ground and form severe dust storms.	

3.2 Treatment Measure Options and Selection Criteria

Selection of drainage, erosion and sediment treatment measures is based on the following principles of effective erosion and sediment control:

- Minimise the extent and duration of soil disturbance;
- Control the location and velocity of drainage flow;
- Minimise soil erosion initiated by wind, rain or concentrated flow;

- Minimise sediment flow from the Project;
- Promptly revegetate/stabilise all exposed and/or unstable soil surfaces; and
- Appropriately install, operate and maintain all ESC measures.

Erosion and sediment control fact sheets are provided in Appendix A.

3.2.1 Design Parameters

The extent and type of erosion control measures depends on the likelihood and intensity of expected rainfall and sheet flow. The treatments and approaches in this ESCP are divided into three categories of control measures including:

- Erosion Control Measures

  Erosion control design is based on average monthly rainfall ranging from 4.7 mm in August to 65.8 mm in February
- Drainage Control Measures

  Drainage control design is primarily based on the Average Recurrence Internal (ARI) of the design storm which is 1:100 year 72 hour rainfall event and
- Sediment Control Measures

  Sediment control design is based on the ARI design for a 1:100 year event 72 hour rainfall event with a rainfall depth of 305 mm (with the exception of WRD stormwater retention basins which will be designed during detailed design phase).

The ESC measures considered for the Project are briefly described in the following sections.

3.3 Erosion Control

Erosion control measures are to minimise movement and loss of sediments at the source. Typical measures include:

- Minimise the area and duration of disturbance;
- Minimise soil and stockpile erosion caused by wind and rain; and
- Minimise turbidity levels in stormwater runoff by minimising the exposure of soil to rain and stormwater flow.

3.3.1 Planning Construction/Clearance Works

The following measures are recommended during planning and preparation of construction works:

- Avoid placing stockpiles near boundary of clearings to limit the area of impact of runoff from Project;
- All Project stockpiles will be located in previously cleared and disturbed areas within the Project boundaries;
- No Project stockpiles will be established within the following areas without prior consultation with Project Management:
  - Ecologically or culturally sensitive area (refer to Biodiversity Management Plan, Cultural Heritage Management Plan and Ground Disturbance Permit System); or
  - Floodplains or within 20 m of a watercourse or drainage channel.
- Where possible, soils around the stockpiles will be re-levelled to minimise sedimentation;
- Disturbed areas will be stabilised progressively e.g. with vegetation/cellular confinement system/mulch during construction where necessary;
- Stockpiles will be sprayed as required by the watercart to minimise dust emissions on dry windy days; and

- Erosion and sediment control measures will not be removed until disturbed areas have been stabilised.

3.3.2 Flagging

Flagging will be installed at all locations to be cleared to ensure the areas are not over cleared. The maintenance of vegetation adjacent to clearing assists in reducing any surface water runoff volume and velocity.

3.3.3 Revegetation

Vegetation or revegetation of a Project provides:

- Physical protection against raindrop impact;
- Barrier between the earth and flow;
- Increased surface roughness that reduces erosive flow velocities; and
- Increased absorption of rainfall by the soil-profile, reducing the volume of runoff.

Revegetation will be carried out on disturbed soil surface that has the potential to erode and cause sediment movement into the surrounding environment during rain events. Revegetation is an effective long-term ESC measure. Ideally, plants should be native to the area, have good soil binding capability and compete successfully with weed species.

Topsoil will be collected during the disturbance and applied across areas to be revegetated. Vegetation cleared will be stockpiled during the clearing process and stored for use on exposed soil surfaces no longer required (i.e. road easements). The stockpiled vegetation will provide a protection layer to the seedbank allowing it to grow.

3.3.4 Gravelling

Gravelling provides a permanent erosion control from raindrop, wind and potential mud generation impacts. It is ideal for application on areas of broad, low gradient earth surfaces and can be used in high traffic volume areas. In general gravelling will be utilised at site offices/administration buildings, across the accommodation village and dedicated light vehicle parking areas.

Application of gravel to the Project compounds provides:

- Physical protection against raindrop impact;
- Barrier between the earth and flow;
- Increased surface roughness that reduces erosive flow velocities; and
- Increased absorption of rainfall by the soil-profile, reducing the volume of runoff.

Gravel should be approximately 20 – 75 mm hard, angular, weather resistant and evenly graded. It should be applied to a minimum of 50 mm thickness across the designated area. Reapplication of gravel will be undertaken as required following maintenance inspections.

Note: if gravel continually migrates off dedicated location a Cellular Confinement System (CCS) may be installed to restrict lateral displacement.

Specific Areas

Site offices/administration buildings, across the Accommodation Village and dedicated vehicle parking areas.

3.3.5 Cellular Confinement System

CCS are utilised to hold soils or gravels in place thereby restricting potential erosion. The material is an expandable, three dimensional open honeycomb mesh. The mesh is installed at flush or slightly below the existing ground surface. If it is to be installed on a slope steeper than 10% it will be anchored into a trench along the top of the treatment area (200 mm deep and 500 mm wide). CCS may be used at the Project when:

- Gravelling applied is migrating from dedicated location
- Gravelling is applied to an area which is not evenly graded
- Sandy river beds are encountered at temporary watercourse crossing: Fords and
- Steep slopes require vegetation to be established, the CCS can hold the soils in place.

3.3.6 Dust Control – Watercart

Ground conditions are generally dry and traffic movements and wind energy has the potential to erode unsealed tracks, haul roads and topsoil stockpiles. Watercarts will be used to suppress dust particles (generally 0.001 to 0.1 mm). In addition, watercarts will be utilised throughout the construction process to facilitate settlement of unsealed and sealed roads.

Specific Areas
Unsealed tracks, haulage roads and topsoil stockpiles.

3.3.7 Wind Breaks – Vegetation

Natural vegetation will be utilised as a wind break across the Project. Wind breaks act by providing a buffer and reducing wind velocity. Flagging will be used to ensure areas aren't over cleared.

Specific Areas
Across the whole of the Project.

3.3.8 Surface Roughening / Contour Ripping

Surface roughening on exposed or revegetated surfaces increases erosion protection of soil surfaces by increasing water infiltration, delaying the formation of rilling and reducing dust generation. In order to roughen surfaces machinery will be utilised (disks, tillers, spring harrows, ploughs or rippers).

Ridges will be installed along contours and perpendicular to the predominant wind direction (south easterly wind direction) where possible. In general ridges will be ripped to a depth of 600 to 900 mm in pairs approximately 2 to 6 m apart.

The installation will include the diversion of up-gradient stormwater runoff around the roughening areas. Following roughening/ridge installation the areas will be immediately seeded and mulched to optimise seed germination and growing conditions.

Specific Areas
Drill pads, unseal track rehabilitation, areas which have been over cleared and topsoil stockpiles.

3.4 Drainage Controls

Drainage control measures are principally used to:

- Divert 'clean' up-slope water around any soil disturbances; and
- Contain and transport potentially contaminated stormwater through disturbed areas to treatment measure(s), minimising contact with erodible soils.

Open channels are economical where large flows are to be carried and space is not restricted. Open channels provide for the continuous collection of surface runoff. Open unlined drains and v drains are generally not acceptable, all drains must be stabilised and preferably vegetated.

Bunds are utilised to divert or contain flow. Flow can be contained on a natural surface by a raised bank (bund) formed by raising compacted earth on the surface.

Channel or bund lining (e.g. rock, vegetation) is specified to protect the subsoil from erosion.

3.4.1 Flow Diversion Banks

Flow diversion banks are earth embankments which divert up-slope stormwater runoff from entering the disturbed area. Water collected by a flow diversion bank is transferred to a stable outlet structure (i.e. level spreader). The diversions are capable of containing dispersive subsoil due to the construction methodology not generally requiring the exposure of subsoils. Design considerations include:

- Discharge to a stable outlet;
- Drain sediment trap if the diverted water is expected to be contaminated;
- Not divert or concentrate flows onto an adjacent property;
- Sides of the bank are to be not steeper than 2:1 (H:V) slope and the completed bank must be at least 500 mm high.

Due to the duration of flow diversion banks at the Project they will be stabilised immediately following initial construction (seeded, mulched and CCS where appropriate).

Haul Roads will also be utilised as flow diversion banks across the mine site, specifically surrounding the LOM pit to restrict overland flows entering the mine site.

Specific Areas

Accommodation Village, Processing Site and Mine Site.

3.4.2 Catch Drains

Catch drains are channels excavated to divert flow around disturbed areas, and drain run-off away from erosion prone areas. Catch drains should be at least 300 mm deep and 1000 mm wide. They may be constructed across a slope to convey runoff at a non-erosive velocity. The channel may be combined with an embankment on the downslope side to increase its capacity. The drains intercept the sheet runoff and divert it at a non-scouring velocity to a stabilised outlet.

A typical gradient of a catch drain is 0.5% and may be as low as 0.25% or as high as 0.75%. As a general rule, the deeper the flow, the lower the maximum gradient. Use of rock-check dams can reduce the effective channel gradient of steeper channels by typically 5%.

Specific Areas

Mine Site: Waste Rock Dump perimeter drain, ROM Pad and crushing plant perimeter drains,

Processing Site: Processing plant facilities and Power Station perimeter drains.

Accommodation Village: Village perimeter drain.

3.4.3 Table Drains

Table drains are constructed adjacent to sealed and unsealed roads to provide a preferential pathway for drainage. Table drains receive sheet flow from the surrounding environment and the formed road. The drains should be at least 300 mm deep and flat bottomed. Where clearing allows the flat bed of the drain should be 2.5 m wide to facilitate maintenance activities.

Table drains will have check dams installed to reduce water velocities and will discharge into a diversion drain.

Specific Areas

Sealed and unsealed roads across the Project.

3.4.4 Diversion Drains

Diversion drains are constructed drainage channels which collect water from table drains and direct it to a suitable disposal area. Similar to a table drain they will be flat bottomed and will collect flows at grade. The drains are to discharge water via a level spreader or the final grade should be 0.2% for 30 m (i.e. 6 cm fall over 30 m). The positioning of diversion drains is site specific but generally should be at a maximum of 120 m at slopes up to 2% reducing to 15 m for slopes greater than 8%.

> **Specific Areas**
>
> Sealed and unsealed roads across the Project.

3.4.5 Check Dams

Check-dams are considered a drainage control. They control the flow velocity in channels. They are also effective at removing coarse sediments from stormwater flow.

Check-dams are placed at intervals within the channel to create ponding of flow along the channel's length, between the toe of the upstream dam and overflow-invert of the downstream dam. This reduces the flow velocity, decreasing scour of the channel and allows coarse sediments to settle. Design criteria includes:

- Dam centre to be at least 150 mm lower than the edges, and dam height limited in height to around 0.5 m. Greater heights require a larger rock-apron to dissipate energy of the overflow; and
- Maximum spacing between the dams occurs where the toe of the upstream dam is at the same elevation as the crest of the downstream dam.

> **Specific Areas**
>
> Situated across the Project within catch drains, table drains and flow diversion banks (upstream sides).

3.4.6 Level Spreaders

Level spreaders are typically constructed along the contour line and consist of a level rock protected entry, allowing concentrated flow to spread to a nominated flow width. Level spreaders are used on the outlet of diversion channels and basins to spread flow and convert concentrated flow into sheet flow. Key issues are noted below:

- Level spreader outlet grade must be less than 10% and ideally discharge should occur to areas of undisturbed land
- Typical maintenance, such as periodic checks, should be conducted to ensure that sediment build up and general erosion such as scouring or channel damage upstream and downstream of the spreader, does not occur and
- Protection of the outlet can be achieved using jute mesh, grass turf, rock or other appropriate stabilisers.

> **Specific Areas**
>
> Situated across the Project as outlets to flow diversion banks, diversion drains or rock lined chutes.

3.4.7 Temporary Watercourse Crossing: Fords

Temporary watercourse crossings are constructed at watercourses or major drainage lines to facilitate traffic movements during construction phases. The crossings are constructed and maintained in a manner which minimises impacts to watercourses and associated habitat value. The majority of temporary watercourse crossings at the Project will be on pre-existing crossings.

Key design details include:

- Constructed at the existing watercourse bed level on a straight section of the watercourse;
- Approach roads should be straight for a minimum of 10 m prior to the crossing and be stabilised;
- Road drainage will be installed to transfer flows away from the crossing; and
- Sandy riverbeds can be stabilised using a geogrid or cellular confinement system.

Specific Areas

Situated across the Project as required.

3.4.8 Rock Lined Chute

Chutes provide a stable pathway for the transfer of water from elevated surfaces such as rehabilitated Waste Rock Dumps, Flotation Tailings Storage Facility and Residue Storage Facility to ground level.

Key design details include:

- Surface drainage across an elevated structure to be directed toward chute(s);
- Installation of rock mattress or alternative stable landform at the base to control erosion;
- Chute to be designed with a safety factor of 1.5 (high risk structure); and
- Rock to be geochemically stable, durable and resistant to weathering.

Specific Areas

Waste Rock Dumps, Residue Storage Facility and Flotation Tailings Storage Facility (following final rehabilitation in line with closure strategies).

3.4.9 Energy Dissipater and Recessed Rock Pad (Outlet Structure)

Energy dissipaters provide outlet control for rock lined chutes to prevent undermining of the chute and control scour immediately downstream. The dissipater itself will be made of coarse riprap or rows of small concrete impact blocks to form as bed roughness and will lead into a recessed rock pad to allow sheet flow to the surrounding environment.

Specific Areas

Situated at the toe of rock lined chutes including the Waste Rock Dumps, Residue Storage Facility and Flotation Tailings Storage Facility (following final rehabilitation in line with closure strategies).

3.5 Sediment Controls

Sediment controls are measures that trap and retain sediments, thereby enabling removal of sediment from the stormwater flow. Where practical, sediment should be trapped close to its source, reducing break-down of soil particles and the release of dispersive clays (if present).

Sediment controls have the greatest maintenance requirements of ESC measures. A sediment control structure may not work properly if it does not have sediment-removal (maintenance) after a storm event.

3.5.1 Sediment Fences

Sediment fences provide physical filtration of sheet flow passing through the filter material, and allow settling of suspended sediments by the ponding of water behind the fence. Sediment fences typically:

- Consist of a filter fabric attached to a wire and post fence at a maximum height of 700 mm with an additional 200 mm (min) buried and compacted into an upstream trench;

- Should be constructed along a contour with turn-ups at either end to prevent runoff flowing around the fence;
- Are most effective for coarse-fraction sediments in sheet flows;
- Trap sediment larger than 0.14 mm and have little impact on fine silts;
- May be used in the control of sediment runoff from exposed land, unsealed roads, batters and stockpiles; and
- For large areas on moderate slopes, sediment fences may be placed at intervals down-slope with a catch-drain on its downstream side. This will contain sediments at the source and minimise concentration of flow.

Specific Areas

Topsoil stockpiles across the Project.

3.5.2 Stormwater Retention Basin

A stormwater retention basin is an effective system to trap and retain a wide range of sediment particle sizes down to 0.045 mm, depending on its hydraulic characteristics (retention time and flow-distribution). It is noted that:

- Stormwater retention basins are usually required when the disturbed area is greater than one hectare, the soils are dispersive and/or there is a need to control runoff turbidity;
- Stormwater retention basins should be located upstream of water bodies, bushland and major stormwater systems;
- Stormwater retention basins are sized to contain and slowly settle fine particles or to slow the flow's velocity allowing settlement of coarser particles during flow-through; and
- Both coarse sediment concentration and turbidity levels can be reduced.

Specific Areas

Waste Rock Dumps, ROM Pad, Mining Services, Processing Plant, Power Station and Accommodation Village.

4. Overview of Erosion and Sediment Control Measures

4.1 General

4.1.1 General Ground Disturbance

Ground disturbances will be undertaken in accordance with the Ground Disturbance Permit System. Ground disturbance includes all disturbances to natural ground including borrow pits, drill pads and infrastructure construction easements. Disturbances will be staged to reduce the area of exposed surfaces through the construction and operations phases. Clearing extents are based on the historical mean rainfall as follows:

- Six weeks for months with mean rainfall between 45 and 100 mm; and
- Eight weeks for months with mean rainfall below 45 mm.

A summary of the land clearing extents in terms of work activity is provided in Table 4-1.

Table 4-1 Summary of Allowable Clearing Extents

	Month											
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Rainfall (mm)												
Mean[a]	62.4	65.8	21.9	18.0	23.3	8.7	4.9	4.7	10.3	15.3	30.9	50.5
Clearing Extents												
No. Weeks	6	6	8	8	8	8	8	8	8	8	8	6

Note: [a] Mean rainfall from BoM 2015; Territory Grape Farm NT 1987-2014.

Process

Disturbances will only be undertaken to provide sufficient work/operation area for eight (mean monthly rainfall 45 to 100 mm) or six weeks (mean monthly rainfall <45 mm). This process restricts the exposure time for areas to be disturbed.

Disturbances will be managed in general accordance with the following:

1. Weed Removal

   The area will be surveyed to assess if the vegetation present comprises of any weeds. Weeds will be removed / treated in accordance with the Weed Management Plan prior to ground disturbances occurring. Weed removal is an important part of the process to ensure mulch does not assist in the distribution of weeds across the site

2. Vegetation Removal

   Vegetation will be cleared in a manner that minimises damage to any retained vegetation. Cleared vegetation will remain in stockpiles. The stockpiled vegetation will either be used during revegetation of the area or transferred to the mine site for storage/covering top soil stockpiles

3. Flow Diversion Bank

   Flow diversion banks will be installed to facilitate the diversion of clean water around the disturbance. The banks will have a level spreader to discharge

4. Topsoil Removal

   Following the installation of flow diversion banks, topsoil will be removed and stockpiled adjacent to the disturbance area. The topsoil will be utilised as part of the revegetation process for the disturbance or transferred to the Mine Site for storage within the topsoil stockpiles

5. Sediment Fence

   If the topsoil is intended to be reused at the area of ground disturbance, the stockpile is to be located within the disturbance area and associated flow diversion bank. In addition, a sediment fence is to be installed on the downgradient side of the stockpile

6. Surface Roughening / Contour Ripping

   The flow diversion bank will be removed / flattened. Roughening will then be undertaken to facilitate vegetation establishment and reduce potential for rill and gully erosion. Following roughening/ridge installation the areas will be immediately seeded and mulched to optimise seed germination and growing conditions and

7. Revegetation

   Where possible, areas will be revegetated immediately following the completion of works with native species. If revegetation is not established sufficiently due to gradient complications then a CCS may be installed to arrest soil movement and facilitate vegetation establishment.

4.1.2 Unsealed and Sealed Tracks

In accordance with the Ground Clearing Procedure and the clearing details provided in Section 4.1.1, clearing will only be undertaken to provide sufficient work area of eight (mean monthly rainfall 45 to 100 mm) or six weeks (mean monthly rainfall <45 mm).

The flow diversion banks installed as part of the clearing procedure will remain insitu until sufficient site drainage has been established. Tracks will be constructed to facilitate effective drainage with a targeted crossfall of 4% (1 in 25). Drainage will be installed adjacent to tracks including:

- Table Drains

  The drain collects drainage from the surrounding environment and road. The drains will be installed with check dams to reduce water velocities and

- Diversion Drains

  Diversion drains will be installed and pushed out into the surrounding environmental to facilitate the disposal/discharge of flows into the table drains. The drains are to discharge water via a level spreader or the final grade should be 0.2% for 30 m (i.e. 6 cm fall over 30 m).

Floodways

Due to the nature of the drainage within the project, floodways will be used where practicable to reduce the interruption of natural sheet flow and to avoid, as much a is practicable, the concentration of water flows.

Closure

At closure or when unsealed and sealed tracks become redundant across the Project the following will occur:

- Surface Roughening / Contour Ripping

  Roughening will be undertaken to facilitate vegetation establishment and reduce potential for rill and gully erosion. Following roughening/ridge installation the areas will be immediately seeded and mulched to optimise seed germination and growing conditions

- Revegetation

  Where possible, areas will be revegetated following the completion of works with native species, understanding that results may not be apparent until seasonal rainfall occurs. If revegetation is not established sufficiently due to gradient complications then a CCS may be installed to arrest soil movement and facilitate vegetation establishment.

4.2 Mine Site

4.2.1 Open Pit

Construction and Operation

Haul roads constructed across the mine site will also serve as flow diversion banks. The pit perimeter haul road will restrict overland flows entering the pit. The remaining haul roads across the site will have culverts or floodways' installed to facilitate overland flow through the site (design to be determined).

Closure

The flow diversion banks will become the ultimate pit abandonment bunds for the Project in accordance with the Western Australian Department of Industry and Resources, *Safety Bund Walls Around Abandoned Open Pit Mines*. The diversion banks will continue to operate as an impermeable core to the abandonment bund, however the following modifications will be made:

- Expanded to minimum dimension of 2 m height and 5 m wide base; and
- Developed with unweathered (geochemically stable) freely draining end dumped rockfill.

4.2.2 Waste Rock Dumps

Waste Rock Dumps (WRDs) will be constructed to store waste rock from mining activities. In general, WRDs are positioned in close proximity to Pit entry/egress points to reduce haulage distance and therefore fuel costs.

Geochemistry at the Project indicated the risk of acid, metalliferous or saline drainage is low and the material can generally be managed as Non Acid Forming waste. However, field testing procedures will be implemented during the operation to identify and appropriately manage any potential (estimate <1%) Potential Acid Forming (PAF) waste (EIS, Appendix L).

Construction and Operation

WRD construction will be staged across the development of the Project. In accordance with the Ground Clearing Procedure and the clearing details provided in Section 4.1.1, clearing will only be undertaken to provide sufficient work area of eight (mean monthly rainfall 45 to 100 mm) or six weeks (mean monthly rainfall <45 mm).

A low permeability base will be constructed for the WRDs. The construction material will be geochemically stable (inert) and have the ability to be compacted by traffic and track rolling. A quality assurance and quality control document will be established as part of the detailed design to ensure construction quality.

Catch drains will be constructed adjacent to the WRD base and act as a perimeter drain collecting surface flows and transferring them to a stormwater basin (sediment basin). The catch drains will be installed with check dams to reduce flow velocities. A bund will be positioned on the 'outside' of the catch drain to restrict other water sources entering the drain.

Stormwater basins will be positioned adjacent to the WRDs. The basins will be designed during the preliminary WRD design phase. Indicative locations of the stormwater basins are provided on Figure 4-1.

Closure

WRDs will reach capacity throughout the 41 year operational period and be closed and rehabilitated progressively. The rehabilitation of WRDs is described fully within the Mine Closure Plan (EIS, Appendix W). In general, it is planned that WRDs will remain on the surface rather than be backfilled into the pit and thereby sterilising future resources. The cover anticipated to be installed would be a dry cover, however details of specific cover design are not currently available.

Irrespective of specific cover design, the installation of erosion and sediment controls at the rehabilitated WRD structures will be priority in order to establish long term landform stability.

The following ESC measures will be implemented at WRDs to slow down the rates of erosion to small enough quantities / low enough rates that the receiving environment can absorb and assimilate them without adverse impacts. The measures include:

1. Dump design

   It is intended that the batter slopes used in the design of the waste dump will mimic the natural hill observed at site i.e. mesa shaped landforms. These typically have steeper upper batters and lower gradient lower slopes. This landform not only provides and more aesthetically natural appearance but the slope design tends to be more conducive to erosion minimisation. Stability testing will be done of the waste rock to determine batter slope prior to dump construction

2. Revegetation

   The cover will be revegetated with native species to minimise erosion of the WRD surface. If revegetation is not established sufficiently on batters a CCS may be installed to arrest soil movement and facilitate vegetation establishment

3. Flow Diversion Banks

   Flow diversion banks made from stabilised topsoil and/or geotextile will transfer flows around the top of the WRD to rock lined chute(s). The diversion banks primary use is to reduce the potential for rill and subsequent gully erosion of the batters

4. Rock Lined Chute

   Rock lined chutes may be installed at WRDs to transfer runoff water from the top of WRDs to ground level. The chutes primary role is to reduce the potential for erosion of the batters and transfer flows into an energy dissipater. It should however be noted that WRD design preference will be to manage runoff water from the top of the WRDs by establishing a store and release system that encourages infiltration along with perimeter bunding and inner swales (i.e. bunded cells) breaking up catchment areas across the top of WRDs

5. Energy Dissipater and Recessed Rock Pad (Outlet Structure)

   Energy dissipaters will collect flows from rock lined chutes. The structures are designed to significantly reduce flow velocity and therefore erosion potential. In order for flows to be discharged effectively to the surrounding environment a recessed rock pad will be utilised to reduce potential for scour.

4.2.3 Flotation Tailings Storage Facility

Construction and Operation

The Flotation Tailings Storage Facility (FTSF) will include several expansions during the life of the Project. In accordance with the Ground Clearing Procedure and the clearing details provided in Section 4.1.1, clearing will only be undertaken to provide sufficient work/operation area for eight (mean monthly rainfall 45 to 100 mm) or six weeks (mean monthly rainfall <45 mm).

The FTSF will be operated as a wet facility with a minimum of 0.5 m supernatant (water) on top of the tails throughout the operation. The water will reduce the potential for wind erosion on the tailings surface. The base and embankment will be impermeable with a target permeability of 5×10^{-8} m/s.

The FTSF has built in capacity to contain a 1 in 100-year ARI wet year rainfall whilst retaining a freeboard equivalent to the PMP – 72 hour storm event depth (1100 mm).

Closure

The Project will have one FTSF with a preliminary LOM design footprint of 245 ha and embankment height of 25.1 m. The FTSF will be operated through the life of the Project and closed and rehabilitated at closure. The rehabilitation of the FTSF is described fully within the Mine Closure Plan (EIS, Appendix W).

The FTSF is likely to be rehabilitated in a similar process to the WRDs and the ESC measures will include:

1. Revegetation

   The cover will be revegetated with native species to minimise erosion of the cap. If revegetation is not established sufficiently on batters a CCS may be installed to arrest soil movement and facilitate vegetation establishment

2. Flow Diversion Banks

   Flow diversion banks made from stabilised topsoil and/or geotextile will transfer flows around the top of the FTSF to rock lined chute(s). The diversion banks primary use is to reduce the potential for rill and subsequent gully erosion of the batters

3. Rock Lined Chute

   Rock lined chutes may be installed at FTSF to transfer runoff water from the top of the FTSF to ground level. The chutes primary role is to reduce the potential for erosion of the batters and transfer flows into an energy dissipater. It should however be noted that FTSF design preference will be to manage runoff water from the top of the FTSF by establishing a store and release system that encourages infiltration along with perimeter bunding and inner swales (i.e. bunded cells) breaking up catchment areas across the top of FTSF and

4. Energy Dissipater and Recessed Rock Pad (Outlet Structure)

   Energy dissipaters will collect flows from rock lined chutes. The structures are designed to significantly reduce flow velocity and therefore erosion potential. In order for flows to be discharged effectively to the surrounding environment a recessed rock pad will be utilised to reduce potential for scour.

4.2.4 ROM Pad

Construction and Operation

The Run-of-Mine (ROM) Pad will be constructed during the construction period to its LOM extents. The ROM Pad will have a raised compacted base with a target permeability of $5x10^{-8}$ m/s. A flow diversion bank will be installed along the perimeter of the ROM Pad to transfer flows to the stormwater basin and the pad itself will have a gentle gradient to the basin. The stormwater basin will be designed to capture a 1:100 year ARI 72 hour storm event of 305 mm (refer to Table 2-1). The basin will be impermeable with a target permeability of $5x10^{-8}$ m/s.

Closure

The ROM Pad base material has the potential to be geochemically impacted throughout the LOM and as such the closure methods are dependent upon the chemistry of the base at closure. The base will be considered chemically unsuitable if it exhibits levels of contaminants in excess of baseline, ecological trigger values (NEPM) or site specific guidelines for vegetation growth.

All or part of the ROM Pad which is considered geochemically unstable will be removed and disposed of with a WRD, TSF or RSF prior to the facility being capped. If the material is stable it will be removed and utilised as part of the closure cover on surrounding infrastructure.

The ground below the ROM Pad (original ground level) will be rehabilitated in accordance with the General Ground Rehabilitation described in Section 4.1.1.

4.2.5 Topsoils Storages

Topsoil storage areas have been identified to facilitate progressive rehabilitation and closure of the Project.

Construction and Operation

The construction phase will be managed such that topsoil is collected from all ground disturbances (through the Ground Disturbance Permit System – see Biodiversity Management Plan). The stockpiles will be placed in soil stockpile envelopes across the Project for efficient management. The topsoil stored within these areas will be utilised for progressive rehabilitation across the Project as required.

Stockpiles will be managed to ensure stability of topsoil is achieved within a minimum timeframe. During the establishment a sediment fence will be installed surrounding the down gradient area of the stockpile. The sediment fence will be removed following the establishment of vegetation (stability of the landform).

Temporary Closure

Topsoil stockpiles will be removed at the closure phase of the Project. However, following the construction phase of the Project a significant volume of topsoil will be stored onsite and require ESC measures to be installed. The measures to be used include:

1. Surface Roughening / Contour Ripping

   Roughening will occur across the top of the topsoil stockpile to facilitate vegetation establishment and reduce potential for rill and gully erosion. Following roughening/ridge installation the areas will be immediately seeded and mulched to optimise seed germination and growing conditions

2. Revegetation

   The top of the stockpile will be progressively revegetated with native species to minimise erosion. The revegetation will act as a trial for the closure of the Project and identify species which are likely to withstand the surrounding environmental conditions

   If revegetation is not established sufficiently on batters a CCS may be installed to arrest soil movement and facilitate vegetation establishment and

3. Flow Diversion Banks

   An up-gradient flow diversion bank will be installed with a level spreader outlet. If there are repeat occurrences of the stockpile toe eroding then a flow diversion bank will be extended along the length of the stockpile.

Closure

Topsoil stockpiles will have been removed and utilised across the Project prior to/during mine closure. Following removal, the areas will be rehabilitated in accordance with the general ground disturbance process (Section 4.1.1).

4.2.6 Mining Services

Construction and Operation

In accordance with the Ground Clearing Procedure and the clearing details provided in Section 4.1.1, clearing will only be undertaken to provide sufficient work area of eight (mean monthly rainfall 45 to 100 mm) or six weeks (mean monthly rainfall <45 mm).

Following the removal of vegetation, gravelling will be installed across the areas proposed for buildings and light vehicle parking. To reduce surface runoff from the surrounding areas flow diversion banks with level spreader outlets will be installed up gradient of gravelled areas.

A stormwater basin will be installed to collect overland flow from precipitation falling directly on the Mining Services compound/area. A catch drain will be installed downgradient of gravelled areas to transfer to stormwater basins. The stormwater basin has been designed to capture a 1:100 year ARI 72 hour storm event of 305 mm (refer to Table 2-1). The basin will be impermeable with a target permeability of 5×10^{-8} m/s.

Closure

Following the closure of operations it is likely several field offices will remain insitu to facilitate closure and rehabilitation processes. The gravelled areas, flow diversion banks and level spreaders will remain insitu until Mining Services is to be completely removed from the Project.

When rehabilitation of the Project is nearing completion and there is little or no requirement for Mining Services, it will be rehabilitated in accordance with the general ground disturbance process (Section 4.1.1).

4.2.7 Kerosene Camp Creek Realignment

Construction and Operation

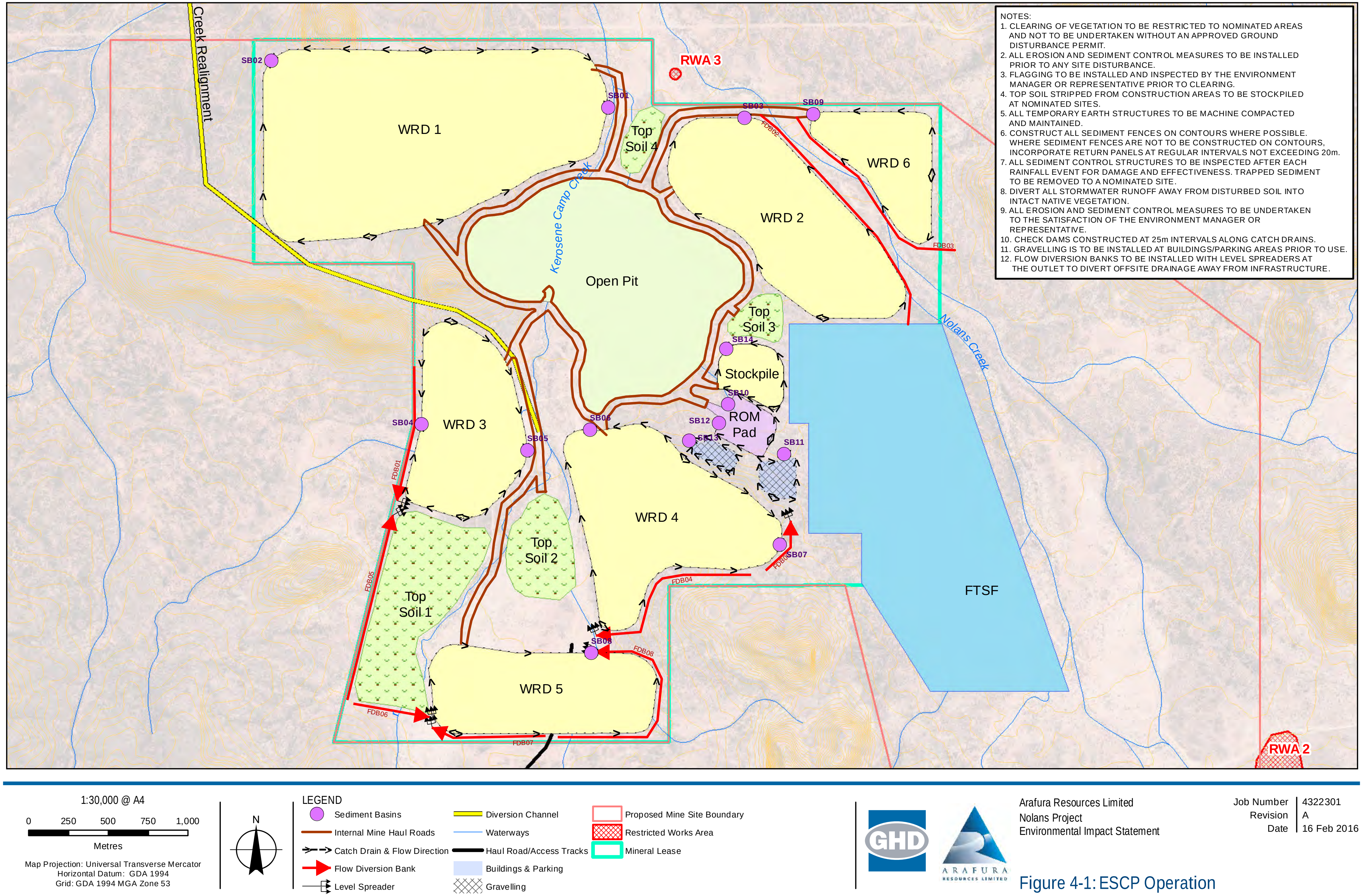
Kerosene Camp Creek will be permanently realigned to divert flows around the Mine Site. The realignment will be designed with the initial 200 to 400 m section steepened to improve flow conveyance and sediment transport. In addition, flood protection bunds will be installed to pass flows exceeding the design discharge. The preliminary design for the realignment includes:

- Steep sided channel with 3V to 1H batters to minimise both excavation volumes and the top footprint width of the realignment
- Inset channel with 1V to 1.5H banks and dimensions of 2 m deep with a 4 m wide invert that mimics the dimensions of the existing channel; and
- Benches, each 2 m wide, either side of the inset channel to provide an opportunity for vegetation to establish in proximity of the channel.

The design of the realignment is provided within the EIS documentation Kerosene Camp Creek Diversion – Concept (EIS, Appendix I).

Closure

The realignment will form a permanent surface water feature at the Project. The closure phase and potential modifications of the realignment will be addressed during the detailed design phase but are likely to include expansion of flood protection bunds to 1:1000 ARI and a high level spillway to maintain flood protection bund stability.



NOTES:
1. CLEARING OF VEGETATION TO BE RESTRICTED TO NOMINATED AREAS AND NOT TO BE UNDERTAKEN WITHOUT AN APPROVED GROUND DISTURBANCE PERMIT.
2. ALL EROSION AND SEDIMENT CONTROL MEASURES TO BE INSTALLED PRIOR TO ANY SITE DISTURBANCE.
3. FLAGGING TO BE INSTALLED AND INSPECTED BY THE ENVIRONMENT MANAGER OR REPRESENTATIVE PRIOR TO CLEARING.
4. TOP SOIL STRIPPED FROM CONSTRUCTION AREAS TO BE STOCKPILED AT NOMINATED SITES.
5. ALL TEMPORARY EARTH STRUCTURES TO BE MACHINE COMPACTED AND MAINTAINED.
6. CONSTRUCT ALL SEDIMENT FENCES ON CONTOURS WHERE POSSIBLE. WHERE SEDIMENT FENCES ARE NOT TO BE CONSTRUCTED ON CONTOURS, INCORPORATE RETURN PANELS AT REGULAR INTERVALS NOT EXCEEDING 20m.
7. ALL SEDIMENT CONTROL STRUCTURES TO BE INSPECTED AFTER EACH RAINFALL EVENT FOR DAMAGE AND EFFECTIVENESS. TRAPPED SEDIMENT TO BE REMOVED TO A NOMINATED SITE.
8. DIVERT ALL STORMWATER RUNOFF AWAY FROM DISTURBED SOIL INTO INTACT NATIVE VEGETATION.
9. ALL EROSION AND SEDIMENT CONTROL MEASURES TO BE UNDERTAKEN TO THE SATISFACTION OF THE ENVIRONMENT MANAGER OR REPRESENTATIVE.
10. CHECK DAMS CONSTRUCTED AT 25m INTERVALS ALONG CATCH DRAINS.
11. GRAVELLING IS TO BE INSTALLED AT BUILDINGS/PARKING AREAS PRIOR TO USE.
12. FLOW DIVERSION BANKS TO BE INSTALLED WITH LEVEL SPREADERS AT THE OUTLET TO DIVERT OFFSITE DRAINAGE AWAY FROM INFRASTRUCTURE.

Map Projection: Universal Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 53

Arafura Resources Limited
Nolans Project
Environmental Impact Statement

Job Number	4322301
Revision	A
Date	16 Feb 2016

Figure 4-1: ESCP Operation

G:\43\22301\GIS\Maps\4322301_455_MineSiteErosionSedControl.mxd

Level 5 66 Smith Street Darwin NT 0800 Australia T 61 8 8982 0100 F 61 8 8981 1075 E drwmail@ghd.com W www.ghd.com

Data source: Google Earth Pro - Imagery (Date extracted: 29/04/2015). ARL - Proposed Pipelines, Proposed Mine Site, Proposed Diversion Channel Options, Tailings Storage Facility (2015). Created by: CM

4.3 Processing Site

4.3.1 Process Plant

Construction and Operation

In accordance with the Ground Clearing Procedure and the clearing details provided in Section 4.1.1, clearing will only be undertaken to provide sufficient work area of eight (mean monthly rainfall 45 to 100 mm) or six weeks (mean monthly rainfall <45 mm).

Following the removal of vegetation, gravelling will be installed across the areas proposed for modular buildings and light vehicle parking. To reduce surface runoff from the surrounding areas, flow diversion banks with level spreader outlets will be installed up gradient of the gravelled areas.

A stormwater basin will be installed to collect overland flow from precipitation falling directly on the Processing Site compound/area. A catch drain will be installed downgradient of gravelled areas to transfer to stormwater basins. The stormwater basin will be designed to capture a 1:100 year ARI 72 hour storm event of 305 mm (refer to Table 2-1). The basin will be impermeable with a target permeability of $5x10^{-8}$ m/s.

Closure

The processing plant will be removed as part of the closure and rehabilitation activities. Following its removal the site will be rehabilitated in accordance with the general ground disturbance process (Section 4.1.1).

4.3.2 Power Station

In accordance with the Ground Clearing Procedure and the clearing details provided in Section 4.1.1, clearing will only be undertaken to provide sufficient work area of eight (mean monthly rainfall 45 to 100 mm) or six weeks (mean monthly rainfall <45 mm).

Following the removal of vegetation, gravelling will be installed across the areas proposed for modular buildings and light vehicle parking. To reduce surface runoff from the surrounding areas flow diversion banks with level spreader outlets will be installed up gradient of the gravelled area.

A stormwater basin will be installed to collect overland flow from precipitation falling directly on the power station compound/area. A catch drain will be installed downgradient of gravelled areas to transfer to stormwater basins. The stormwater basin will be designed to capture a 1:100 year ARI 72 hour storm event of 305 mm (refer to Table 2-1). The basin will be impermeable with a target permeability of $5x10^{-8}$ m/s. Closure

The power station will be removed as part of the closure and rehabilitation activities. Following its removal from the site it will be rehabilitated in accordance with the general ground disturbance process (Section 4.1.1).

4.3.3 Residue Storage Facility and Evaporation Pond

Construction and Operation

The Residue Storage Facilities (RSF) will be gradually expanded across the LOM. In accordance with the Ground Clearing Procedure and the clearing details provided in Section 4.1.1, clearing will only be undertaken to provide sufficient work/operation area for eight (mean monthly rainfall 45 to 100 mm) or six weeks (mean monthly rainfall <45 mm).

The RSF will be operated with a minimum of 0.5 m water maintained throughout the operation. The water will reduce the potential for wind erosion on the residue storage base. The base and embankments will be impermeable with a target permeability of $5x10^{-8}$ m/s.

The facilities will be constructed with sufficient freeboard to collect a minimum of a 1:100 year 72 hour rainfall event (305 mm) whilst retaining sufficient additional freeboard to accommodate a PMP 72-hour storm rainfall event (1,100 mm).

Closure

The RSF will be progressively rehabilitated throughout the Project once they reach operational capacity. The rehabilitation of the RSF is described fully within the Mine Closure Plan (EIS, Appendix W).

The RSF and Evaporation Pond will be capped with low permeability soil, a layer of waste rock and the ESC measures will include:

1. Revegetation

   The cover will be revegetated with native species to minimise erosion of the cap. If revegetation is not established sufficiently on batters a CCS may be installed to arrest soil movement and facilitate vegetation establishment

2. Flow Diversion Banks

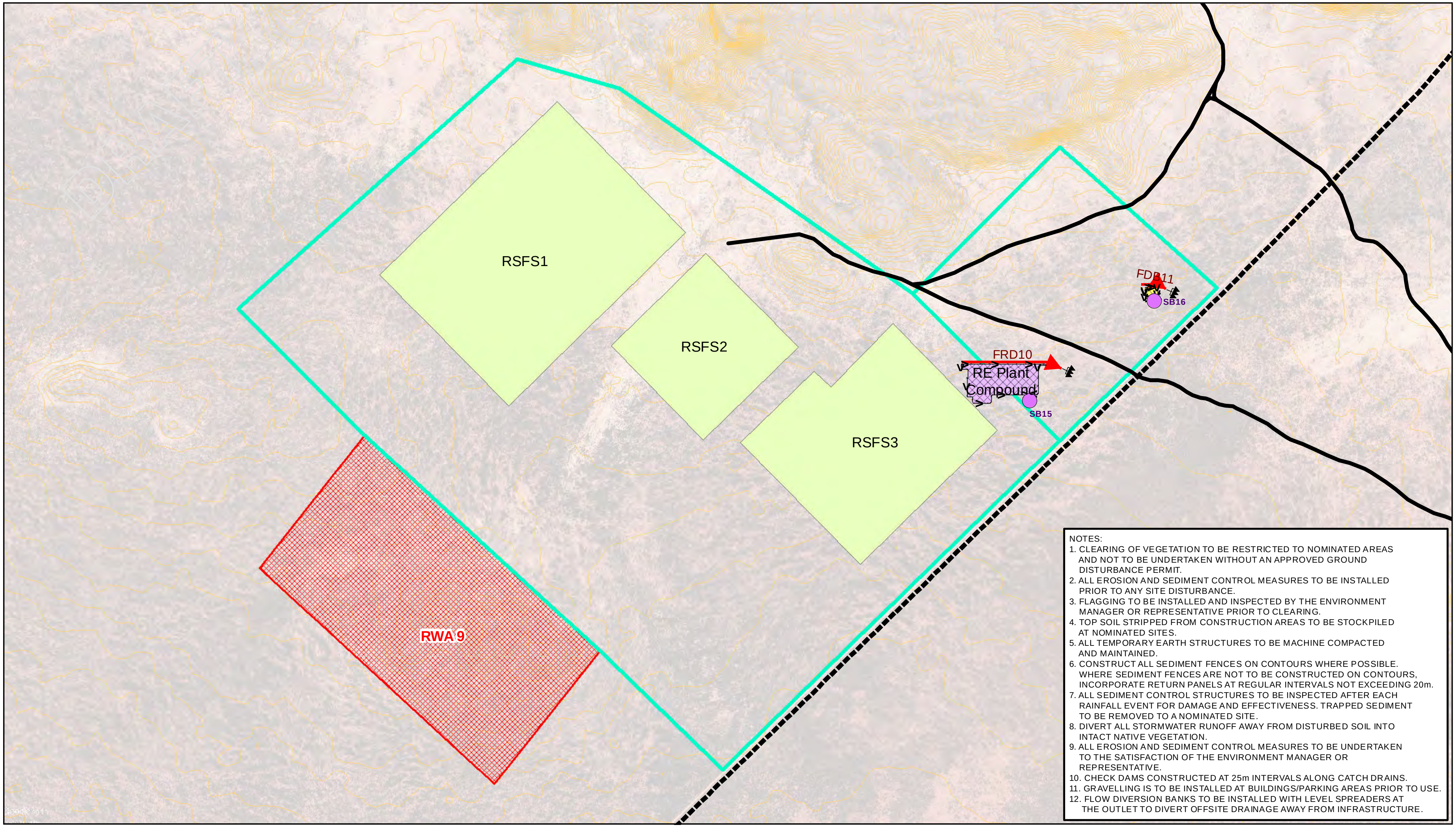
   Flow diversion banks made from stabilised topsoil and/or geotextile will transfer flows around the top of the facilities to rock lined chute(s). The diversion banks primary use is to reduce the potential for rill and subsequent gully erosion of the batters.

3. Rock Lined Chute

   Rock lined chutes may be installed at facilities to transfer runoff water from the top of the RSF to ground level. The chutes primary role is to reduce the potential for erosion of the batters and transfer flows into an energy dissipater. It should however be noted that RSF design preference will be to manage runoff water from the top of the RSF by establishing a store and release system that encourages infiltration along with perimeter bunding and inner swales (i.e. bund walls) breaking up catchment areas across the top of RSF and

4. Energy Dissipater and Recessed Rock Pad (Outlet Structure)

   Energy dissipaters will collect flows from rock lined chutes. The structures are designed to significantly reduce flow velocity and therefore erosion potential. In order for flows to be discharged effectively to the surrounding environment a recessed rock pad will be utilised to reduce potential for scour.



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9. ALL EROSION AND SEDIMENT CONTROL MEASURES TO BE UNDERTAKEN TO THE SATISFACTION OF THE ENVIRONMENT MANAGER OR REPRESENTATIVE.
10. CHECK DAMS CONSTRUCTED AT 25m INTERVALS ALONG CATCH DRAINS.
11. GRAVELLING IS TO BE INSTALLED AT BUILDINGS/PARKING AREAS PRIOR TO USE.
12. FLOW DIVERSION BANKS TO BE INSTALLED WITH LEVEL SPREADERS AT THE OUTLET TO DIVERT OFFSITE DRAINAGE AWAY FROM INFRASTRUCTURE.

1:35,000 @ A4

0 250 500 750 1,000

Metres

Map Projection: Universal Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 53

N

LEGEND

- Sediment Basins
- Catch Drain & Flow Direction
- Level Spreader
- Flow Diversion Bank
- Existing Gas Pipeline
- Access Road
- Indicative Power Station
- RE Plant Compound
- Residue Storage Facilities
- Gravelling
- Mineral Lease
- Restricted Works Area

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Arafura Resources Limited
Nolans Project
Environmental Impact Statement

Job Number	4322301
Revision	A
Date	16 Feb 2016

Figure 4-2: ESCP Operations

G:\43\22301\GIS\Maps\4322301_456_ProcessingSiteLayout.mxd

Level 5 66 Smith Street Darwin NT 0800 Australia T 61 8 8982 0100 F 61 8 8981 1075 E drwmail@ghd.com W www.ghd.com

Data source: Google Earth Pro - Imagery (Date extracted: 29/04/2015). ARL - Proposed Pipelines, Power Station, Powerline, Existing Gas Pipeline, Access Road, Treatment Plant Facilities, Mineral Lease, Restricted Works Area (2015). Created by: CM

4.4 Borefield

The ESC measures for the Borefield will be managed in accordance with the General Ground Disturbance and Unsealed Tracks measures described in Sections 4.1.1 and 4.1.2 respectively.

4.5 Accommodation Village

Construction and Operation

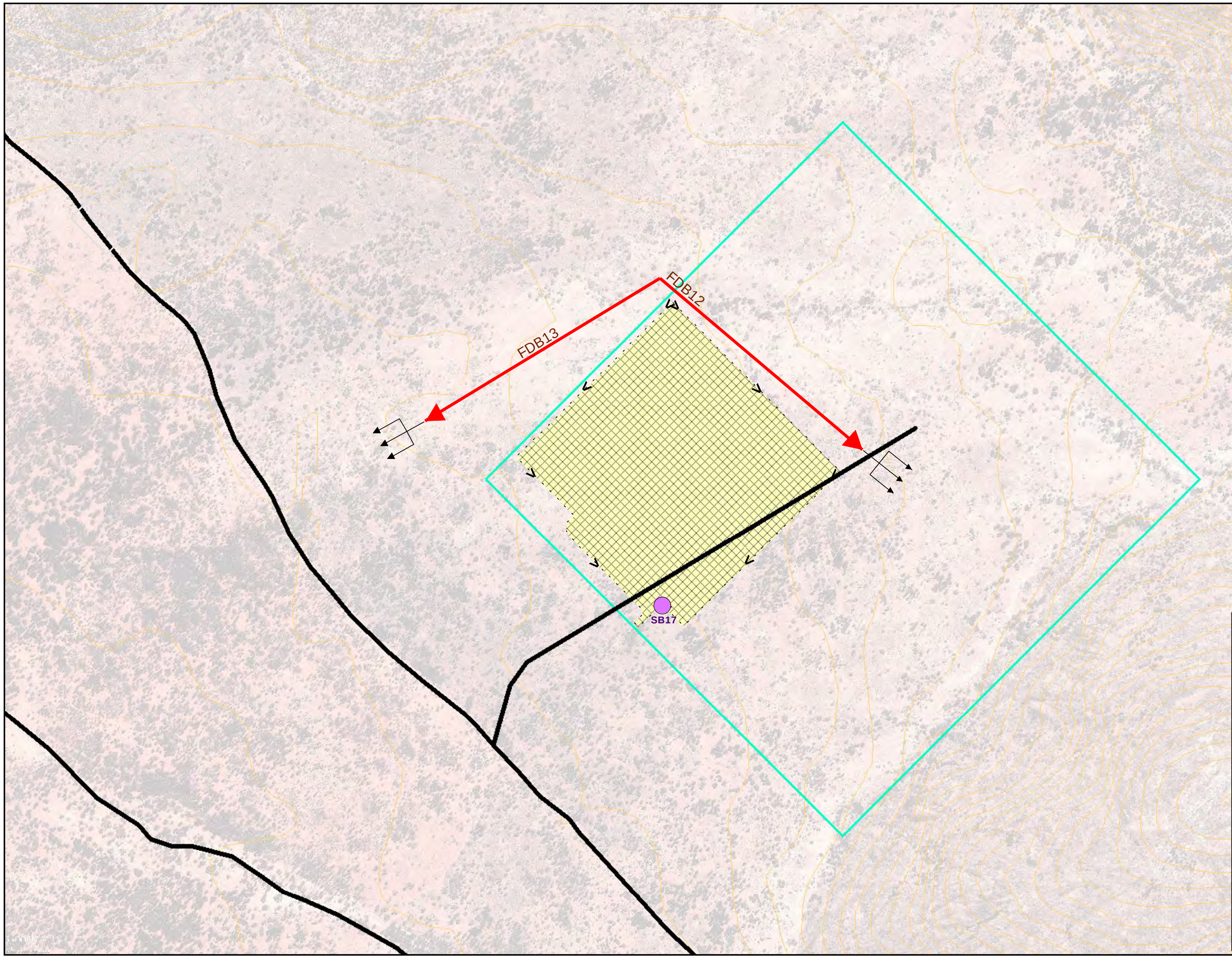
In accordance with the Ground Clearing Procedure and the clearing details provided in Section 4.1.1, clearing will only be undertaken to provide sufficient work area of eight (mean monthly rainfall 45 to 100 mm) or six weeks (mean monthly rainfall <45 mm).

Following the removal of vegetation, gravelling will be installed across the areas proposed for modular buildings and light vehicle parking. To reduce surface runoff from the surrounding areas flow diversion banks with level spreader outlets will be installed up gradient of the gravelled area.

A stormwater basin may be installed to collect overland flow from precipitation falling directly on the Accommodation Village compound/area. A catch drain will installed downgradient of gravelled areas to transfer to stormwater basins. The stormwater basin will be designed in the detailed design phase.

Closure

The Accommodation Village will be removed as part of the closure and rehabilitation activities. Following its removal from the site it will be rehabilitated in accordance with the general ground disturbance process (Section 4.1.1).



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10. CHECK DAMS CONSTRUCTED AT 25m INTERVALS ALONG CATCH DRAINS.
11. GRAVELLING IS TO BE INSTALLED AT BUILDINGS/PARKING AREAS PRIOR TO USE.
12. FLOW DIVERSION BANKS TO BE INSTALLED WITH LEVEL SPREADERS AT THE OUTLET TO DIVERT OFFSITE DRAINAGE AWAY FROM INFRASTRUCTURE.

1:10,000 @ A4

0 100 200 300 400

Metres

Map Projection: Universal Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 53

N

LEGEND

- Sediment Basins
- Catch Drain & Flow Direction
- Level Spreader
- Flow Diversion Bank
- Access Road
- Accommodation Village
- Mineral Lease
- Gravelling

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Figure 4-3: Accommodation Village

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Level 5 66 Smith Street Darwin NT 0800 Australia T 61 8 8982 0100 F 61 8 8981 1075 E drwmail@ghd.com W www.ghd.com

Data source: Google Earth Pro - Imagery (Date extracted: 29/04/2015). ARL - Accommodation Facility (2015). GHD - Sediment Basins, Catchment Drains and Flow Direction, Flow Diversion Bank, Gravelling (2015/16). Created by: CM

5. Maintenance Requirements

The ESCP for the proposed development is prepared with the following maintenance philosophy:

- Selection of mitigation measures requiring minimal regular maintenance or simple maintenance procedures; and
- Access must be provided if maintenance is required on any structure.

A maintenance program for the ESC measures is outlined in Table 5-1. A checklist will be developed that records maintenance problems likely to occur for each of the ESC measures adopted by the Project, and identifies the person responsible for implementing, maintaining, inspecting, repairing and modifying controls.

The inspection frequency will need to be adjusted according to the prevailing weather conditions, i.e. increased during wet periods and reduced during dry periods. Weekly inspections will be sufficient during minor runoff events. An inspection is required after any major runoff event.

Table 5-1 Erosion and Sediment Control Maintenance Requirements

Item	Inspection Frequency	Maintenance Frequency	Maintenance Activities
Erosion Control			
Flagging	As required; or Daily during clearances.	As required.	Identify any damage and re-establish flagging.
Revegetation (Seeds and Stockpiled Vegetation)	As required; or After a major rainfall event	When areas of mulch have been eroded or if vegetation does not establish in the required time.	Re-application of mulch and take action to prevent future damage. Assess if vegetation has established and to identify if any erosion, channelling or weed problems occur. Reseeding and weeding to maintain a dense, vigorous growth of vegetation. Vegetation and mulch will require reestablishment if less than 70% is present. Application of additional mulch as required. Maintenance of any upslope diversion channels or protective fences if installed.
Gravelling	As required; or After a major rainfall event	As required.	Check for continuous even cover and for rilling along the up-gradient slope edges. Replace gravel from the down-gradient location(s).
Cellular Confinement System	As required; or After a major rainfall event	As required.	Removal and reinstallation of system and/or growth media.
Dust Control – Watercart	n/a	n/a	n/a
Wind Breaks – Vegetation	*See revegetation*	n/a	n/a
Surface Roughening / Contour Ripping	As required; or After a major rainfall event	As required.	If rill erosion occurs through ridges the rills are to be filled.
Drainage Controls			
Flow Diversion	As required;	When slumps, wheel track damage or	Identify any damaged or eroded areas

Item	Inspection Frequency	Maintenance Frequency	Maintenance Activities
Bank	or After a major rainfall event	loss of freeboard has occurred. When litter or sediment has accumulated and filled 30% of the drain depth.	due to sediment accumulated in the channel, vehicular damage to the banks, settlement of banks and/or scour due to excessive flow velocity. Remove accumulated litter and sediment. Reform bund or channel banks to design grade.
Catch Drain	As required; or After a major rainfall event	Damage of the channel banks has occurred.	Identify any damaged or eroded areas due to sediment accumulated in the channel, vehicular damage to the banks, settlement of banks and/or scour due to excessive flow velocity. Remove accumulated litter and sediment. Reform bund or channel banks to design grade.
Table Drain	Biannual	When litter has accumulated or sediment has filled 30% of the drain depth.	Identify any damaged or eroded areas due to sediment accumulated in the channel, vehicular damage, settlement of banks and/or scour due to excessive flow velocity. Remove accumulated litter and sediment. Reform bund or channel banks to design grade.
Diversion Drain	Biannual	When litter has accumulated or sediment has filled 30% of the drain depth.	Identify any damaged or eroded areas due to sediment accumulated in the channel, vehicular damage, settlement of banks and/or scour due to excessive flow velocity. Remove accumulated litter and sediment. Reform bund or channel banks to design grade.
Check Dam	As required; or After a major rainfall event	When litter has accumulated or sediment has filled 30% of the drain depth.	Identify any damage or sediment build-up. Re-establish dams when sediment begins to flow through the structure. Remove accumulated litter and sediment.
Level Spreader	As required; or After a major rainfall event	When sediment build-up limits the spreader to function effectively Scouring of channel and vegetation damaged.	Identify any damage or sediment build-up causing concentration flow. Reformation of channel banks to design grade. Treat scouring or channel damage upstream of the spreader. Application of additional mulch or vegetation. Remove accumulated litter and sediment.
Temporary Watercourse Crossing: Fords	As required; or After a major rainfall event	When damage of CCS or excessive scour has occurred.	Debris trapped on or upstream of the crossing is removed. Identify and remediate any erosion upstream or downstream scour.
Rock Lined Chute	As required or after a major rainfall event	As required.	Check flow entry condition to ensure no flow is bypassing the chute(s). Check for inlet scour, piping or bank failures. Check whole of structure for rill or gully erosion to ensure chutes are operating efficiently.
Energy Dissipater and Recessed Rock Pad (Outlet Structure)	As required or after a major rainfall event	As required.	Identify any erosion around the edge of the pad and ensure rocks remain adequately recessed into the earth. Check for excessive displacement of

Item	Inspection Frequency	Maintenance Frequency	Maintenance Activities
			rocks and potential for reinstatement.
Sediment Controls			
Sediment Fence	As required; or After a major rainfall event	When sediment accumulates at the base of the control structure or when permeability is excessively reduced.	Identify any damage caused by on-site Project vehicles or excessive sediment movement. Remove accumulated litter and sediment. Reform sediment fence, take action to prevent future damage. Where fence is regularly damaged, reassess and reduce the area of inflow, install a second fence at least 1 m downslope of the existing fence.
Sediment Basin	As required; or After a major rainfall event	When litter has accumulated or sediment has filled 10% of the sediment basin volume as indicated by the marker post.	Remove accumulated litter and sediment, spreading it well away from drainage lines. Repair of any scouring damage to inlet and outlet and embankment vegetation. Pump-out of retained water to maintain capacity for subsequent inflow events.

5.1 Monitoring Checklist

A monitoring checklist will be maintained of all erosion and sediment control measures, with entries made as inspections are completed and after rainfall events on:

- Condition of ESC structures and stabilised surfaces;
- Repair of any damage to ESC structures; and
- Rainfall, including duration and times.

Corrective actions will be investigated and implemented within 24 hours where practicable where findings of the ESC monitoring indicate a non-conformance.

6. References

BoM, 2016 http://www.bom.gov.au/climate/averages/tables/cw_015643_All.shtml Accessed 14:49, 08/02/2016.

International Erosion Control Association (IECA) Australasia (November 2008) *Best Practice Erosion and Sediment Control for Building and Construction Projects*

Department of Industry and Resources, 1997. Safety Bund Walls Around Abandoned Open Mines – Guideline. (http://www.dmp.wa.gov.au/Documents/Safety/MSH_G_SafetyBundWallsAroundAbandonedMines.pdf)

GHD 2016a, Nolans Rare Earths Project Flora and Vegetation Assessment (EIS, Appendix M).

GHD 2016b, Nolans Bore EIS – Acid & Metalliferous Drainage (AMD) Assessment and Management Plan (EIS, Appendix L).

GHD 2016c, Nolans Bore Mine Closure Plan (EIS, Appendix W).

GHD 2016d, Kerosene Camp Creek Diversion – Concept (EIS, Appendix I).

Lycopodium Minerals 2010, Nolans Project Plant and Haul Road Geotechnical Report.

Appendices

Appendix A – Erosion and Sediment Control Fact Sheets

Source: Erosion and sediment control fact sheets have been provided downloaded from http://www.catchmentsandcreeks.com.au/M-fact_sheets-ESC.html. The fact sheets were produced by Catchments & Creeks Pty Ltd and represent current best practice.

Contents: Erosion control including:

- Revegetation
- Gravelling
- Cellular Confinement System
- Dust Control – Watercart
- Surface Roughening / Contour Ripping

Drainage controls including:

- Flow Diversion Bank: General
- Flow Diversion Bank: Earth
- Catch Drain
- Check Dam
- Level Spreader
- Temporary Watercourse Crossing: Fords
- Rock Lined Chute
- Energy Dissipater
- Outlet Structure (Recessed Rock Pad)

Sediment controls including:

- Sediment Fence
- Sediment Basin

Revegetation

Revegetation

EROSION CONTROL TECHNIQUE

Revegetation	✓	Temperate Climates	✓	Short-term	[1]
Non Vegetation		Wet Tropics	✓	Long-term	
Weed Control		Semi-Arid Zones	✓	Permanent	✓

[1] Temporary revegetation can be an effective form of erosion control, but it usually needs to incorporate *Light Mulching* in order to provide sufficient protection from raindrop impact erosion.

Symbol

Photo 1 – Turfing

Photo 2 – Fertiliser spreader and chisel plough

Key principles

1. Test the soils, and where required, adjust the soils before planting
2. The primary function of "temporary" vegetation, in association with mulching, is to achieve effective short-term erosion control through coverage of the soil surface, thus the effective percentage surface cover is the key performance measure.
3. Vegetative-based erosion control is primarily achieved through coverage of the soil. Root stabilisation of the soil structure is generally of secondary importance. However, the function of the roots becomes increasingly important as the surface slope increases.
4. The initial coverage of annual grasses in the weeks following seeding may not provide adequate erosion protection against raindrop impact because these grasses primarily grow vertically, thus providing only limited coverage of the soil surface. In such cases, mowing can increase the effective soil cover.

Design Information

Selecting the most suitable plant establishment techniques, appropriate species, seeding rates, planting densities, fertiliser types, watering rates, and maintenance techniques, requires the guidance of experts such as soil scientists, revegetation specialists, local bushland groups, and government extension officers.

Each of the various forms of soil erosion, whether initiated by wind, rain, or flowing water, are best controlled by different forms and/or combinations of vegetation. Table 1 outlines the types of vegetation most likely to be effective in the control of the various forms of soil erosion. Of course there are always exceptions to such generalisation.

Table 1 – Plant selection for the control of soil erosion

Erosion form	Primary vegetation	Secondary vegetation	Comments
Water induced:			
Raindrop impact	Ground covers, grasses, and living or dead organic matter	Trees, shrubs	• Ground covers need to quickly cover the soil surface (i.e. not just straight, vertical shoots—which is often the early growth characteristic of many annuals). • In this context, “grasses” includes living, dormant and dead grasses. • Trees contribute by suppling leaf and bark litter (mulch).
Sheet erosion	Ground covers, grasses		• Non-clumping, continuous ground cover is required.
Rill erosion	Ground covers, grasses		• Non-clumping, continuous ground cover is required.
Gully erosion	Ground covers, vetiver grass	Trees, shrubs, woody debris	• Vetiver grass can be used to form a vegetative sediment barrier. • Trees and shrubs may be required for bank stability.
Tunnel erosion			• Stabilisation of soil and control of water pathways are of primary importance. • Avoid deep-rooted or short-lived plants on water impoundment embankments.
Wave erosion	Reeds	Mangroves	• Critical locations include coastlines, rivers, lakes and dams. • Mangroves can struggle to deal with significant wave attack.
Gravity induced:			
Mass movement	Trees, vetiver grass	Shrubs	• Use of deep-rooted plants is critical.
Wind induced:			
Wind erosion	Ground covers	Tree, shrubs, mulches	• Trees can form windbreaks. • Aided by increased surface roughness.
Watercourse erosion:			
Refer to the *Instream Erosion Control* fact sheet, and Tables I14 to I15 (p. I.32 to I.34) in Appendix I – *Instream works*.			

ESTIMATING GROUND COVER

(i) Quadrat method

Materials:

- 50m tape measure
- $1m^2$ quadrat (a "quadrat" for these purposes being a 1m x 1m rectangular viewing grid)
- visual cover estimation template (Figure 1, otherwise refer to McDonald et al., 1990)
- notebook and pens

Procedure:

1. Locate sampling points at four evenly spaced points along a 50m transect.
2. Place the $1m^2$ quadrat on the ground with the nominated point at the centre. Identify all species rooted **within** the quadrat (if required), and estimate and record the percentage cover. Where required, record the percentage cover of each plant species. For the purpose of species identification, do not record plants rooted outside, but branching across, the quadrat. For purposes of total cover estimation, record all matter, plant (living or dead) and mulch, whether rooted inside or outside the quadrat.

(McDonald, R.C., Isbell, R.F., Speight, J.C., Walker, J. and Hopkins, M.S. 1990, *Australian Soils and Land Survey Field Handbook*, Inkata press, Melbourne)

(ii) Ellenbank Pasture Meter

The Ellenbank Pasture Meter consists of a weighted plate that compresses pasture, then measures the height of the compressed vegetation. Even though this procedure provides a good estimate of pasture density (for stock feed), it does **not** necessarily provide a good estimate of cover. It is noted that the bulk of the pasture may consist of tall, near-vertical stalks that provide limited protection against raindrop impact in comparison to shorter, near-horizontal dead or living stalks.

ESTIMATION OF TREE AND SHRUB DENSITY

Materials:

- 2 x 50m tape measures
- star pickets
- notebook and pens

Procedure:

At each sample site, mark the western end of a 50m transect with a star picket. Measure the tree and shrub densities using the Point-Centred Quarter method (Barbour et al. 1987), as described below.

1. Locate sampling points at the 0m, 25m and 50m points on the transect.
2. At each sample point, align two axes centred on the sample point. The axes follow the line of the transect, and a line perpendicular to the transect.
3. Within each quadrant formed by the axes, identify the closest tree and shrub. If the tree or shrub exceeds a distance of 50m, do not record it.
4. Measure the distance in metres to the closest tree, and to the closest shrub.
5. Record the species and estimate its height.
6. For each transect, average the distance measurements for trees (D_{ave}).
7. Calculate the average tree density (stems per hectare), $T_d = 10{,}000 / (2 \times (D_{ave})^2)$
8. Calculate the relative density of species, X =

   (Number trees of species, X) / (Total number of trees x average tree density)
9. Repeat Steps 6 to 8 for shrub species. Record the adopted classification of shrubs (e.g. all woody plants less than 6m tall, including tree saplings).

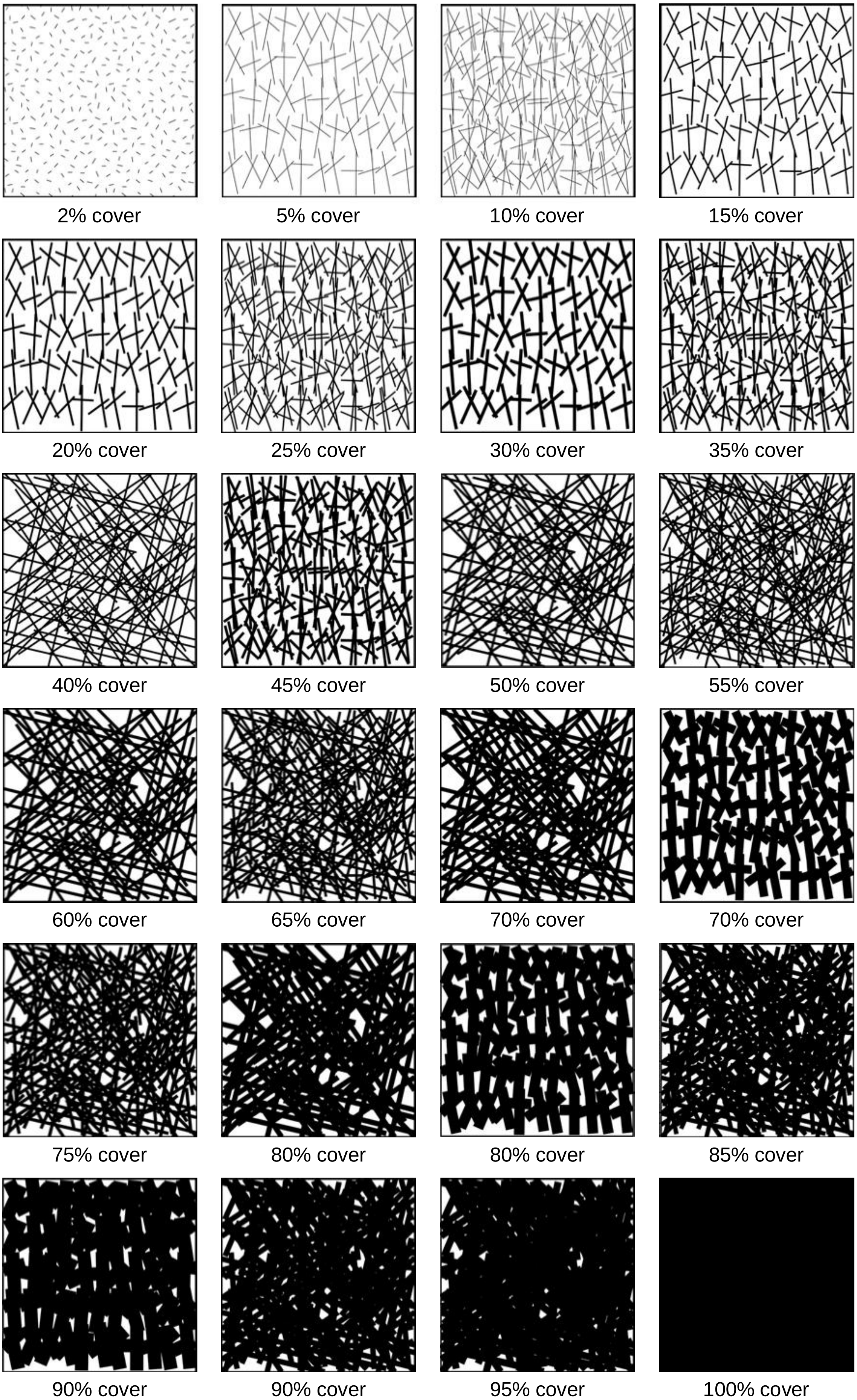


Figure 1 – Visual cover estimation template

Description

Establishment of temporary or permanent vegetation over exposed soil surfaces.

"Temporary seeding" is a process of providing a temporary grass cover during construction delays, or when final further soil disturbance is expected within a given area and short-term erosion control measures are deemed necessary.

Purpose

Site revegetation is performed for a number of reasons, including:

- Improve aesthetics
- Erosion control
- General ecological reasons including habitat, food source & shelter
- Stabilisation of shallow land slips
- Increase stormwater infiltration and reduce the volume of runoff
- Reduce rainfall impact energy
- Increase organic content of the soil
- Established vegetated buffer zones
- Reduce dust problems
- Filter sediment from sheet flow

Limitations

There are limits to the role vegetation alone can play in controlling erosion. Both soil strength and vegetation cover (including root system) can take years to develop to the required condition.

Usually not suitable in heavy traffic areas or on long slopes steeper than 2:1(H:V).

Advantages

In terms of ecologically sustainable soil protection, vegetation is the best long-term solution to wind and water induced erosion.

Most forms of vegetation are self-regenerating and to some degree, self maintaining.

Well-landscaped works are aesthetic and usually well received by the public.

Disadvantages

Long establishment time for most forms of vegetation, except turfing.

Subject to damage in heavy traffic areas.

Conflicts can exist between the choice of native and exotic species.

In some rural and semi-arid areas, watering costs can be high.

Usually requires a long maintenance period.

Common Problems

Poor site drainage can damage plant seeds and remove mulch cover.

Poor soil preparation can significantly limit the establishment, growth and erosion benefits of vegetation.

Many problems can initiate from inadequate soil testing and soil amendment.

Special Requirements

Usually requires guidance from local experts, such as local agronomists.

At least 70% ground cover (combined plant and mulch) is considered necessary to provide a satisfactory level of erosion control.

A mulch cover layer is usually required to control short-term erosion and provide good growing conditions. The mulching of exposed soils is generally recommended on all seeded areas, especially when the area contains: high clay content soils, dispersive soils, exposed subsoils, or during hot, dry weather (to limit soil moisture loss).

Requires suitable soil and soil conditioning.

Plant establishment requires a reliable water supply.

On some open grassed areas, slashing is recommended to reduce the excessive growth of the primary cover and also to remove immature seed heads. This is particularly important for summer plantings as regrowth can compete strongly for light and water with the secondary and tertiary cover species.

Long-term maintenance needs are usually inversely proportional to the degree of planning and quality of site preparation.

Site Inspection

Check effective percentage cover.

Check for damage to protective fencing.

Seed, seedlings and mulch may need re-application if the vegetation does not establish in the required time.

Look for displacement of mulch by wind or water.

Specifications for site revegetation vary considerably from site to site. Site supervisors should obtain site specific planting specifications.

Installation

1. Refer to approved plans for location, extent, and application details. If there are questions or problems with the location, extent, or method of application contact the engineer, landscape architect or responsible on-site officer for assistance.
2. Apply soil conditioners and fertiliser as specified on the approved plans. Rip the soil to a depth of 100 to 150mm to mix the components into the soil and to loosen and roughen the soil surface before seeding.
3. There should be sufficient soil depth to provide an adequate root zone. The depth to rock or impermeable layers such as hardpans should be 300mm or more, except on slopes steeper than 2:1(H:V) where such soil depth may not be feasible.
4. Ensure the soil pH is within the specified range.
5. Apply seed uniformly by hand or with a fertiliser spreader, drill-seeder, hydro-seeder, or other suitable equipment as specified.
6. When using broadcast-seeding methods, subdivide the area into workable sections and apply one-half the specified quantity of seed while moving back and forth across the area, making a uniform pattern. Then apply the second half in the same way, but moving at right angles to the first pass. Cover broadcast seed by raking or chain dragging; then firm the surface with a roller to provide good seed to soil contact.
7. Apply seed at the recommended rate, and disc or otherwise mechanically treat the surface to bring the seed into contact with the soil.
8. The seeded area should be mulched as specified in the approved plan.

Maintenance

1. During the construction phase, inspect the treated area fortnightly and after runoff-producing rainfall. Make repairs as needed.
2. Watering the vegetation periodically is essential, especially in the first 7 days after establishment. Use low-pressure sprays because high-pressure jets can wash away the seed and mulch cover.
3. Watering should start immediately after planting. Watering should comply with specifications provided with the approved plans. Generally watering should vary according to weather and soil conditions. A typical watering schedule may consist of the following:
   - 25 mm every second day for the first three waterings;
   - 25 mm twice a week for the next three weeks; and
   - 25 mm once weekly for a further two weeks.
4. Monitor site revegetation, particularly after rainfall, and appropriate maintenance and/or amendment to ensure that the revegetation is controlling erosion and stabilising soil slopes as required.
5. Where practicable, fill in, or level out, any rill erosion between plants. If excessive erosion occurs, then consider increasing the planting density, applying appropriate erosion control measures, or introducing alternative, non-clumping plant species.
6. Areas must be re-seeded and mulched if the vegetation fails to establish or is damaged by runoff or construction activities.
7. If the temporary vegetation cover or erosion control measure (e.g. mulch cover) should fail for any reason before establishment of the permanent vegetation cover, then it must be replaced with an appropriate type of cover sufficient to control soil erosion.
8. If the permanent vegetation should fail to establish or to adequately restrain erosion for any reason during the construction or maintenance period, the area should be revegetated or protected with other erosion control measures as appropriate.
9. In areas where the obtained vegetation cover is considered inadequate for erosion control, the affected area should be over-seeded and fertilised using half the originally specified rates, or as directed.

10. Maintain grass blade length at a minimum 50mm height within medium to high velocity drainage areas, and 20 to 50mm within low velocity flow paths.
11. Where necessary, or as directed by the site supervisor, slash the temporary crop/grass cover to allow the successful growth of the underlying permanent vegetation cover.
12. Control weed growth within 1m of immature trees for 6 to 12 months for fast growing species, and 18 to 20 months for slower growing species, or until the end of the specified maintenance period.
13. Where mulch is used to control weed growth, inspect and where necessary, renew at maintenance periods not exceeding 4 to 6 months.
14. Apply additional seed, mulch and/or soil conditioning as required. Mulches usually need to be maintained or renewed (as necessary) 2 to 3 times a year.
15. Inspect and where necessary repair protective fencing at maintenance periods not exceeding 1 month.
16. Re-firm plants loosened by wind-rock, livestock or wildlife.
17. Replace dead or severely retarded plants.
18. Prune any plants of dead or diseased parts. Cut off all damaged tree limbs above the tree collar at the trunk or main branch. Use several cuts including undercutting to avoid peeling bark from the healthy areas of the tree.
19. Dispose of cleared vegetation in an appropriate manner such as chipping or mulching, on-site burial, or off-site disposal. Cleared vegetation should not be dumped near a watercourse or on a floodplain where is could be removed by floodwaters. Vegetation should not be burnt on-site without specific approval from the local authority.
20. Repair damaged tree roots by cutting off the damaged areas and sealing them with an approved product. Spread moist topsoil over exposed roots.

Gravelling

Gravelling

EROSION CONTROL TECHNIQUE

Revegetation		Temperate Climates	✓	Short-Term	[2]
Non Vegetation	✓	Wet Tropics	✓	Long-Term	✓
Weed Control	[1]	Semi-Arid Zones	✓	Permanent	✓

[1] Refer to *Rock Mulching* fact sheet for weed control application.

[2] Can be used for short-term erosion control around the construction office area and car park.

Symbol

Photo 1 – Gravelling of construction access road

Photo 2 – Gravelling of construction site car park

Key Principles

1. Primarily used to control raindrop impact and mud generation, therefore the depth of cover, and percentage of fines (particles finer than 1mm) are critical.
2. Operational performance is governed by the control of raindrop impact erosion, dust and surface mud on trafficable areas.
3. Gravel is **not** a suitable material for the stabilisation of construction site entry/exit points; however it may be suitable for the formation of rock entry pads on some small building sites (e.g. those building sites with little or no soil/earth import or removal).

Design Information

Minimum 100% coverage of the soil surface.

Nominal aggregate (rock) size of 20 to 75mm.

Apply at a minimum thickness of 50mm, or at least twice the nominal aggregate size.

Allowable flow velocities for rock with a specific gravity of 2.6 are presented in Table 1.

The equivalent allowable shear stress, based on a critical Shield's parameter of 0.07 and a safety factor of 1.5, is provided in Table 2.

The assumed Manning's roughness for the gravel (used to determine the allowable flow velocity from the allowable shear stress) is presented in Table 3. This Manning's roughness is based on a $d_{50}/d_{90} = 0.8$ (i.e. a relatively uniform rock size). Note; d_{50} is the nominal rock size of which 50% of the rocks are smaller.

Hydraulic design of gravelled surface is only required if the surface is likely to be subjected to significant overland flow that could displace the gravel or otherwise cause erosion.

Table 1 – Allowable flow velocity (m/s) for various rock sizes [1,2]

Hydraulic radius (mm)	Nominal mean (d_{50}) rock size (mm)					
	20	**30**	**40**	**50**	**60**	**75**
50	0.86	0.85	0.83	0.81	0.80	0.80
75	1.02	1.05	1.05	1.03	1.02	0.99
100	1.14	1.19	1.21	1.21	1.20	1.18
150	1.29	1.40	1.45	1.47	1.48	1.48
200	1.39	1.53	1.61	1.66	1.69	1.71
300	1.51	1.70	1.83	1.91	1.98	2.03
500	1.65	1.89	2.06	2.19	2.30	2.42

[1] Based on a relative density of 2.6 (i.e. rock mass of 2.6 tonne/m^3)

[2] Applicable to slopes less than 5%. Caution if applied to slopes of 5 to 10%.

Table 2 – Allowable shear stress (N/m^2) for various rock sizes [1]

Hydraulic radius (mm)	Nominal mean (d_{50}) rock size (mm)					
	20	**30**	**40**	**50**	**60**	**75**
N/A	14.6	22.0	29.3	36.6	43.9	54.9

[1] Based on a critical Shield's parameter of 0.07 and a safety factor of 1.5.

Table 3 – Assumed Manning's roughness (n) of gravel [1]

Hydraulic radius (mm)	Nominal mean (d_{50}) rock size (mm)					
	20	**30**	**40**	**50**	**60**	**75**
50	0.027	0.034	0.040	0.046	0.052	0.060
75	0.025	0.029	0.034	0.038	0.043	0.049
100	0.023	0.027	0.031	0.034	0.038	0.043
150	0.022	0.025	0.028	0.030	0.033	0.037
200	0.021	0.024	0.026	0.028	0.030	0.033
300	0.021	0.023	0.024	0.026	0.028	0.030
500	0.021	0.022	0.024	0.025	0.026	0.028

[1] Based on a rock size distribution of d_{50}/d_{90} = 0.8.

Description

The stabilisation of broad, low gradient, earth surfaces using a mixture of relatively small size rock approximately 20 to 75mm in diameter.

The term *gravelling* normally refers to the application of a layer of gravel or aggregate on roads or car parks. It is generally not used to describe the use of small rocks as garden mulch (see *Rock Mulching)*.

Purpose

Primarily used in high traffic areas to reduce soil compaction and control raindrop impact and wind erosion.

Limitations

The small rock size limits its scour resistance resulting in a relatively low allowable shear stress.

Gravel should **not** be placed directly onto dispersible soils. Instead dispersive soil should be covered with a minimum 200mm layer of non-dispersive soil before placement of gravel.

Advantages

Produces a low cost, trafficable surface.

Gravelling the general construction office area and car park can significantly reduce the generation of mud during extended periods of wet weather.

Gravel roads generally experience less environmentally-damaging sediment runoff than dirt roads.

Disadvantages

Effective service life of a single application of gravel can be short, especially during wet weather and/or when placed on wet clayey soils.

The cost may not be easy to justify if recommended for placement over short-term construction access tracks.

Common Problems

Compression of the gravel into soft, clayey soils.

Special Requirements

Placement of the gravel on an appropriate geotextile can improve the service life of the gravelled surface.

Location

Light traffic access roads, car parks and general construction office area.

Site Inspection

Check even, continuous (100%) cover of earth.

Check if reapplication is required.

Check for rilling along the up-slope edges of the treated area, and the free passage of stormwater runoff across the gravel.

Performance Indicators

Application depth measured at random test locations.

Aggregate size, and particle size range measured using conventional particle size test procedures (if required).

Installation

1. Refer to approved plans for location, extent, and application details. If there are questions or problems with the location, extent, or method of application contact the engineer or responsible on-site officer for assistance.
2. Spread enough gravel to completely cover the surface of the soil at the density or thickness specified in the approved plans. If the application density is not supplied, then apply at a thickness of at least twice the mean rock size.
3. Make all necessary adjustments to ensure any run-on stormwater flow is allowed to pass freely across the treated area following its natural drainage path.

Maintenance

1. Inspect all treated surfaces fortnightly and after runoff-producing rainfall.
2. Check for rill erosion, or dislodgment of the gravel.
3. Replace any displaced gravel to maintain the required coverage.
4. If wash-outs occur, repair the slope and reinstall surface cover.
5. If the gravelling is not effective in containing the soil erosion it should be replaced, or an alternative erosion control procedure adopted.

Cellular Confinement System

Cellular Confinement Systems

EROSION CONTROL TECHNIQUE

Revegetation	[1]	Temperate Climates	✓	Short-Term	[2]
Non Vegetation	✓	Wet Tropics	✓	Long-Term	[2]
Weed Control		Semi-Arid Zones	✓	Permanent	✓

[1] Vegetation, such as grasses, can be established within the cells.

[2] Can be used for short-term erosion control, but is most commonly used as a permanent treatment.

Symbol CCS

Photo 1 – Cellular confinement system used to restrict gravel movement on a permanent car park

Photo 2 – Cellular confinement system used to retain soil and assist in the establishment of grass on a steep slope

Key Principles

1. Critical design parameters are the size and depth of the cells, choice of cell wall texture (smooth or rough, solid or perforated), type of anchorage system (applicable to slopes and concave surfaces), and the choice of infill material.
2. It is critical to ensure the top of the cellular confinement system (CCS) is set flush with, or slightly below, the adjacent terrain to avoid stormwater run-on water being diverted along the edge of the matrix.

Design Information

The following design information applies to applications not within a drainage channel. For use as a channel/chute lining, refer to the separate fact sheet within the *'Drainage Control'* section.

Step 1 Determine the type of cell wall: smooth, textured, or perforated.

Textured or perforated surfaces (Photo 4) are required when the honeycomb matrix is installed with a concave profile that may cause the matrix to lift from the ground. Perforated cell walls are required when it is necessary for water flow to pass laterally through the cell walls (Photos 5 & 6). This is usually required when the honeycomb matrix is to be grassed on slopes steeper than 10%. Textured surfaces are used with aggregate and concrete infill.

Step 2 Determine the design shear stress or average velocity resulting from the expected flow passing over the treated surface. Consult with, and/or review the manufacturer's design guidelines for sizing of cell depth and selection of infill material.

Cellular confinement systems are manufactured with smooth, textured, or perforated sidewalls. Each surface condition is used for a different purpose. The perforated, textured surfaces (Photo 4) are the most common and generally achieve most of the performance requirements.

Photo 3 – Smooth sidewall

Photo 4 – Textured and perforated sidewall

Cellular confinement systems allow topsoil to be placed and retained on steep slopes; however, it is important to recognise the long-term effects of retaining the plastic matrix within the ground surface, such as its impact on tree and shrub establishment, or the limitations on topsoil recovery if the slope is re-engineered.

Photo 5 – Installation of matrix on a road cutting

Photo 6 – Grass establishment on steep slope

Cellular confinement systems can be used with or without vegetation (Photos 6 & 8).

Photo 7 – Placement of filter cloth might not always be required

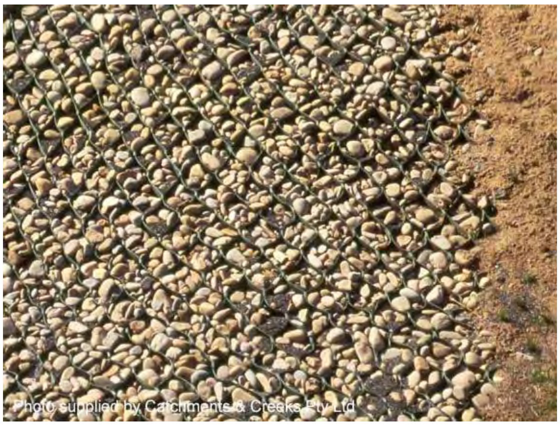


Photo 8 – Cellular confinement system used to hold aggregate on an embankment

Description

Expandable, three-dimensional, open honeycomb-like mesh manufactured from a synthetic material and filled on-site with soil, sand, small rocks, or low slump concrete.

Most products are formed from rigid plastic, while others use more flexible material.

Some products have pre-drilled drainage holes to allow the lateral movement of surface groundwater and to increase the shear resistance of the mesh to uplifting forces. The latter reduces the risk of the product lifting from the ground when placed over a concave surface.

Purpose

Can be used for the following purposes:

To hold topsoil and other loose material on steep slopes such as bridge abutments.

To restrict the lateral displacement of gravel or aggregate on car parks.

To improve the rubber-tired trafficability of sandy material (sand-confinement system).

As a form of earth or track stabilisation in muddy, boggy or sandy areas such as a temporary ford crossing of a dry, sandy river bed.

As a form of turf reinforcing.

As a form of earth reinforcing on very steep vegetated slopes—in which case the material is installed in a manner different from that discussed within this fact sheet.

Can also be used as a form of channel/chute lining (refer to appropriate channel-linings fact sheet).

Limitations

In areas of high flow velocity, cellular confinements systems in-filled with small rocks are generally not a suitable replacement the appropriate placement of suitable large rock.

Advantages

Can be used as an alternative to traditional rock armouring in areas that have limited supply of suitable large rock.

Long-term stability is not necessarily dependent on the establishment of vegetation; however, the incorporation of vegetation generally results in a significant increase in the allowable shear stress.

Easy to transport to the site.

Disadvantages

Displacement of the infill material can occur on an ongoing basis if not suitably stabilised with vegetation.

Common Problems

The mesh can lift from the channel surface if placed on a concave surface without being adequately anchored.

Can promote waterlogging of the ground unless adequate drainage exists.

Special Requirements

The mesh needs to be well anchored if placed on a concave surface.

Required good surface preparation with the removal of all major surface irregularities.

Location

Generally best used as permanent erosion control in environments where vegetation cannot be relied upon, such as arid and semi-arid areas and in heavily shaded areas.

Temporary ford crossings on dry, sandy river beds.

Site Inspection

Check for the displacement of infill material.

Ensure the cellular mesh is not lifting from the ground.

Ensure surface flow can freely enter the CCS-lined area. Check for water scour along the up-slope edges of the matrix.

Check for successful vegetation cover (if required).

Material

- Cellular confinement matrix: polyester non-woven material (flexible sidewalls), or high-density polyethylene (HDPE) (stiff cell wall).
- Tendons: steel cable, or bright, high-tenacity, industrial-continuous-filament polyester yarn woven into round braided cord.
- Anchors: wooden stakes, or 250 to 500mm steel J-pins. Wooden takes used only on mild slopes (<10%) as a temporary anchor during the placement of the infill material.
- Infill: topsoil, earth, aggregate or concrete. Maximum aggregate size no greater than 75% of the sidewall depth of the CCS matrix.

Installation

The following specification applies to use as temporary erosion control only.

1. Refer to approved plans for location, extent, and application details. If there are questions or problems with the location, extent, or method of application contact the engineer or responsible on-site officer for assistance.
2. Clear the treatment area of any debris that may interfere with placement of the cellular confinement system (CCS), or prevent good contact between the CCS matrix and the subgrade.
3. Ensure the surface is free of deep track marks of other features that may result in stormwater or groundwater passing in a concentrated under the CCS matrix.
4. Where necessary, establish up-slope drainage controls to limit run-on water that may disturb the matrix.
5. Shape and compact the subgrade surfaces to the shape and elevation shown on the Construction Drawings. When determining the elevation of the subgrade, ensure allowance is made for the thickness of the CCS matrix such that the top of the matrix will be flush with, or slightly below, the adjacent terrain.
6. Where necessary, excavate the subgrade such that when placed, the upper surface of the CCS matrix will be flush with, or slightly lower, than the adjacent terrain.
7. Remove any unstable subgrade, replace with suitable material and compact to achieve a stable surface.
8. If the material is to be placed on a slope steeper than 10%, then excavate an anchoring trench along the top of the treatment area 200mm deep and 500mm wide.
9. Where practical, roughen any excessively smooth, compacted subgrade to improve the eventual bonding between the subgrade and applied CCS matrix.
10. If specified, install the required geotextile underlay on the prepared surface, ensuring that required overlaps are maintained and that the upper edge of the geotextile is anchored (pinned) within the formed anchoring trench.
11. Spread out (expand) individual panels uniformly across the treatment area as specified by the manufacturer. Expand and stretch the panels down the slope instead of across the slope.
12. Along the top edge of the matrix anchor every other cell into the formed trench using steel J-pins.
13. On slopes steeper than 10%, anchor every other cell using steel J-pins at 2m intervals down the slope.
14. On slopes not steeper than 10%, anchor the individual panels along all four sides to prevent movement while placing infill.
15. Interleaf or overlap the edges of adjacent panels according to which sidewall profile abuts. In all cases, ensure that the upper surfaces of adjoining panel sections are flush at the joint and that adjoining cells are fully anchored (stapled).
16. Fill and compact the anchoring trench.
17. Fill the honeycomb panels mechanically or manually. Ensure earth fill and small aggregate (<75mm) is placed from a drop height not exceeding 1m, and large aggregate (>75mm) from a drop not exceeding 0.15m.
18. Place the infill evenly and such that when compacted, the fill will be level with the upper surface of the panel.
19. Lightly tamp or roll topsoil or earth fill, level aggregate fill with a plate tamper or mechanical (backhoe) bucket.

Additional specification for attachment of tendons for anchorage:

1. Feed pre-cut lengths of tendon material through the aligned holes in the cell walls of the matrix prior to expanding individual panels into position.
2. Tie off the ends of the tendons so that the knot cannot pass through the hole in the cell walls. Ensure the knots are tied to provide full tendon strength and will not slip when tensioned.
3. Attach restraining clips to the tendons at regular intervals to achieve the necessary load transfer.
4. Anchor the tendons and restraining clips with steel U-shaped or J-pins at 2m intervals. At each internal anchor location, form a loop in the tendon, insert the anchor, and drive into the subgrade.

Dust Control – Watercart

Dust Control

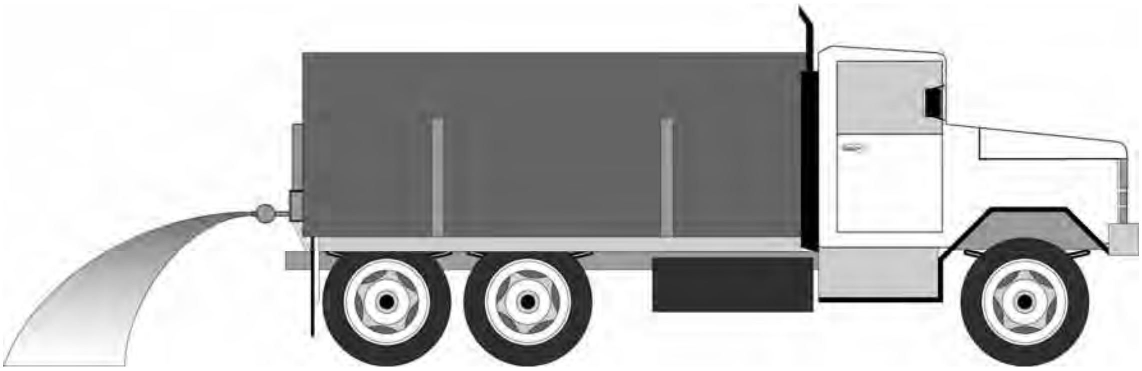
EROSION CONTROL TECHNIQUE

Revegetation	[1]	Temperate Climates	✓	Short-term	✓
Non Vegetation	[1]	Wet Tropics	✓	Long-term	[2]
Weed Control		Semi-Arid Zones	✓	Permanent	

[1] Treatment options can include temporary vegetation and non-vegetated treatment options.

[2] Most treatment options, excluding permanent revegetation, provide only short-term benefits.

Symbol



Key Principles

1. Potential adverse impacts of dust control products/chemicals on the environment (both short- and long-term) **must** not exceed the potential benefits achieved by their use, or any locally adopted measures of unacceptable environmental risk.
2. Critical design parameters include ability to control dust generation, suitability of the product to the work place conditions and the soil type.
3. Effectiveness and durability of most treatment measures depends on soil type, weather conditions, and frequency of disturbance (e.g. traffic movement).

Design Information

Dust control involves the suppression of dust particles generally in the range 0.001 to 0.1mm (1 to 100 microns). Much of the dust generated on construction sites is likely to be greater than 10 microns. Non-visible dust particles (less than 5 microns) are potentially the most harmful to human health.

Dust generation associated with wind erosion is normally controlled using one or more of the following techniques:

(i) Maintaining moist soil conditions (water trucks and sprinkler systems)

(ii) Chemical sealants placed over the soil surface (refer to *Soil Binders* fact sheet)

(iii) Surface roughening (refer to *Surface Roughening* fact sheet)

(iv) Revegetation (short- and long-term ground cover options)

(v) Wind breaks (e.g. retention of existing vegetation, or 60:40 fabric:opening shade cloth).

Dust problems can also be reduced by the following activities:

- Limiting the area of soil disturbance at any given time.
- Promptly replacing topsoil after completion of earthworks
- Programming works to minimise the life of soil stockpiles.
- Temporarily stabilising (e.g. vegetation or mulching) long-term stockpiles.
- Gravelling unsealed access and haul roads.
- Minimising traffic movements on exposed surfaces.
- Limiting vehicular traffic to 25kph.
- Retaining existing vegetation as wind breaks.

International Erosion Control Association (IECA, 1993) reports that:

- 30% soil cover will reduce soil losses by 80%.
- Roughening the soil to produce 150mm high ridges perpendicular to the prevailing wind can reduce soil losses by 80%.
- A small decrease in velocity can have a major impact in reducing wind erosion given that the erosive power of wind is proportional to the cube of the velocity.
- For wind barriers perpendicular to the wind, the width of the [protected] zone leeward of the barriers is around 8 to 10 times the height of the barrier.

Possible treatment options for dust are summarised in Table 1. A summary of dust suppressant agents is provided in Table 2. Discussion on the use of soil binders for dust control is provided in the *Soil Binders* fact sheet.

Table 1 – Dust control practices [1]

Site condition	Treatment options							
	Permanent vegetation	Mulching	Watering	Chemical surface stabiliser [2]	Gravel road [3]	Stabilised entry/exit pad	Haul truck covers	Minimise site disturbance
Areas not subject to traffic	✓	✓	✓	✓	✓			✓
Areas subject to traffic			✓	✓	✓	✓		✓
Material stockpiles			✓	✓				✓
Demolition areas			✓			✓	✓	
Clearing & excavation			✓	✓				✓
Unpaved roads			✓	✓	✓	✓	✓	
Earth transport					✓	✓		

[1] Sourced from: California Stormwater BMP Handbook – Construction (2003).

[2] Oil or oil-treated subgrade should not be used for dust control as this may migrate into downstream water bodies. It is also noted that surface stabilising chemicals (soil binder) may make the soil water repellent, possibly resulting in long-term revegetation problems.

[3] On long-term access and haul roads, the sealing of road with an application of 10mm single-coat bitumen seal can be more effective than the application of dust suppressants.

The following materials must not be used for dust suppression purposes:

- oil;
- landfill gas condensate;
- any contaminated leachate or stormwater when the use of such material is likely to cause unlawful environmental harm.

Table 2 – Summary of dust suppressant attributes [1]

Suppressant type	Typical attributes
Soil binders	• Refer to *Soil Binders* fact sheet
Chlorides: Calcium chloride ($CaCl_2$) Magnesium chloride ($MgCl_2$)	• Chloride compounds attract moisture from the air (hygroscopic) and attach themselves to soil particles if they are applied to wet soils • Less effective in dry climates • Ease of application, with 0 to 4 hours curing time • Can be applied when temperatures drop below freezing • Most suited to temperate and semi-humid conditions • Lose effectiveness in continual dry periods • Less effective than polymers during periods of heavy rainfall • Susceptible to leaching • Suitable for use on moderate surface fines (10–20%) • Not suitable on materials with a low-fines content • High fines content surfaces may become slippery in wet weather • Corrosive impacts associated with calcium chloride
Organic, non-bituminous: Calcium ligno-sulfonate Sodium ligno-sulfonate Ammonium ligno-sulfonate	• Ligno-sulfonate (lignin) is a by-product of the pulp-and-paper industry • React with negatively charged clay particles to agglomerate the soil • Perform well under arid conditions and in dry climates • Failures occur following rains • Susceptible to leaching by heavy rains • Suitable on high fines content (10–30%) in a dense graded material with nil loose gravel • Less effective on igneous, medium to low fines content materials and crushed gravels • High fines content surfaces may become slippery in wet weather • It is best to grade haul road to remove surface material, potholes, and corrugations before application of agent • Curing takes 4 to 8 hours
Petroleum-based products: Bitumen emulsion (slow-breaking non-ionic)	• Generally effective regardless of climate • Will pothole in wet weather and high traffic conditions • Suitable on materials with a low-fines content (<10%) • Non suitable where runoff could contaminate receiving waters
Electrochemical stabilisers: Sulfonated petroleum Enzymes	• Work over a wide range of climates • Suitable for clay materials but depends on clay mineralogy • Iron rich soils generally respond well • Least susceptible to leaching • Ineffective if surface is low in fines and contains loose gravel

[1] After UMA Engineering Ltd 1987, *Guidelines for Cost Effective use and Application of Dust Palliatives.* UMA Engineering Ltd, Ontario, Canada.

Water trucks and sprinkler systems

Water trucks have traditionally been used to control dust within construction sites, particularly on haul roads and for highway construction. The maintenance of moist soil conditions through watering remains a viable dust control measure.

The addition of wetting agents and polymer binders (refer to *Soil Binders* fact sheet) to the water can decrease both the water requirements and the required application frequency. Wetting agents can improve the depth and uniformity of the soil wetting process. Polymer binders improve the binding of individual soil particles, thus reducing dust generation even after drying of the soil surface. Dust suppressing agents can be applied by both water trucks and sprinkler systems.

Dust-suppressing fog and mist generators

High volume mist generating machines can be used to suppress airborne dust resulting from blasting operations. Large cannon-like systems can throw a mist some 250m to blanket the treatment area. On small sites, hydraulic atomising misting nozzles can be attached to sprinkler-like distribution system.

An ionic wetting agent can be added to the water to improve the performance of misting dust suppression systems.

Foaming agents

Foaming agent additives can be added to directional dust-suppressing sprinkler systems to apply a foam to the surface of conveyor belt materials to reduce dust resulting from crusher and material handling plants.

Vegetable oil based soil binders

Biodegradable vegetable oil based soil binders can be applied as a water-based emulsion to provide up to 3 months service life in heavy vehicular traffic areas.

Polymer based soil binders (refer to *Soil Binders* fact sheet)

Polymeric emulsion soil binders include: acrylic copolymers and polymers; liquid polymers of methacrylates and acrylates; copolymers of sodium acrylates and acrylamides; poly-acrylamide and copolymer of acrylamide; and hydro-colloid polymers.

In general terms, polymers can provide around 9 to 18 months service life if the treated area remain free of disturbance and traffic movement. On haul roads and permanent unsealed roads, polymer soil binders can be incorporated into road maintenance (grading and rolling) to improve surface stability and compaction.

Photo 1 – Dust generation on a construction site

Photo 2 – Dust control using a water truck

Surface Roughening / Contour Ripping

Surface Roughening

EROSION CONTROL TECHNIQUE

Revegetation		Temperate Climates	✓	Short-Term	✓
Non Vegetation	✓	Wet Tropics	✓	Long-Term	
Weed Control		Semi-Arid Zones	✓	Permanent	

Symbol

Photo 1 – Tracked vehicle walking up and down slope

Photo 2 – Corrugated (roughened) surface

Key Principles

1. Surface roughening is an erosion control technique of which the benefits can vary significantly from region to region, soil to soil, and climate to climate.
2. The appropriate application of surface roughening is possibly best resolved on a site by site basis. However, in most cases exposed soil surfaces should be left in a suitably roughened state, even if they are about to be vegetated or topped with another layer of soil.
3. In general, clayey soils should **not** be finished with a glassy smooth surface, especially if they are to be revegetated using such techniques as hydroseeding or hydromulching, or any of the hydraulically applied erosion control blankets.

Design Information

On exposed or recently vegetated surfaces, erosion protection can be increased by roughening the soil surface to increase water infiltration, delay the formation of rilling, and reduce dust generation. Surface roughening can be applied to both subsoils and topsoils, either before and/or after seeding.

A roughened soil surface is, however, not always desirable. In some cases it may be undesirable to promote the infiltration of water into the soil, such as stockpiled soil immediately prior it being used as embankment fill. Also, on steep slopes, loose surface soil can present an increased risk of sediment runoff, especially during periods of high rainfall intensity.

Table 1 provides general guidelines on the application of surface roughening to cut and fill slopes. This information must be applied in association with site specific geotechnical advice.

Table 1 – Typical application of surface roughening on slopes

Slope condition	Treatment
Cut slope steeper than 3:1(H:V)	• Stair-stepping with a vertical cut of 50 to 100mm can be used to aid in the anchorage of topsoil on steep slopes. • In situations where the stair-stepping is to be a permanent feature of the slope, the vertical cut should be less than 600mm high in soft material, or 1000mm high in rocky material. The width of each step should be greater than the cut height. Such stepping usually does not involve the subsequent placement of topsoil, and thus is only done on good, fertile subsoils, and rocky slopes that are not intended to be seeded. • The horizontal surface of each step should slope inwards towards the vertical face.
	• Grooving is generally limited to slopes less than 2:1. • Grooves should be at least 75mm deep, and not more than 400mm apart.
Fill slope steeper than 3:1(H:V)	• On slopes to be vegetated, ensure the face of the fill slope consists of firm, but not hard, fill 100 to 150mm deep; otherwise use grooving as described above. • On non-vegetated slopes (e.g. arid and semi-arid areas) achieve a soil compaction similar to natural slopes in the region.
Cut and fill slopes no steeper than 3:1(H:V)	• Application of shallow grooves/ploughing (along the contour) using normal tilling, discing, harrowing, or other suitable means. • Grooves should be spaced no less than 250mm, and not less than 25mm deep. • On slopes intended to be mown, ensure surface roughening is appropriate for the intended mowing procedures.
Sandy soils no steeper than 2:1(H:V)	• Roughen using tracked machinery (track walking).

(a) Stair-stepping:

Stair-stepping is achieved during the formation of cut slopes. It involves cutting the slope to form a series of steps formed along the contour. Each step slopes inward towards the slope to aid in the capture and pooling of water and seed.

Stair-stepping can be applied to very steep slopes to reduce the risk of topsoil slippage (Photo 6).

Photo 3 – Stair-stepping

Photo 4 – Slippage of topsoil from steep cut batter

(b) Track walking:

This is achieved by walking a tracked vehicle up and down the slope.

- Generally limited to a maximum 2:1 (H:V) slope.
- Best used on sandy soils that are not likely to compact under the weight of the vehicle.
- When used on some clayey soils, recessed track marks (similar to Photo 5) can be left in the soil resulting in the concentration of stormwater runoff.

Photo 5 – Wheel track marks down a slope potentially causing concentrated runoff

Photo 6 – Rilling down newly vegetated slope cutting through surface roughening

(c) Contour ploughing:

Contour ploughing involves the ripping of mild slopes using a chisel plough or similar tined implement.

- Plough depth of around 200mm is typical, but 300 to 350mm can be achieved with heavy duty tines.
- Typically used to prepare land surfaces prior to revegetation.
- Generally limited to slopes of less than about 10:1 (10% or approximately 6 degrees).

Photo 7 – Contour ploughing

Photo 8 – Contour ploughing

(d) Grooving:

Grooving involves the formation of a series of minor surface grooves aligned with the contour of a slope. These grooves can be formed using disks, tillers, spring harrows, chisel ploughs, scarifiers, rippers, or by attaching a serrated edge to a grader blade (commercially available attachment), the latter being useful when trimming road batters.

Grooves can also be formed by walking modified drum rollers up and down a slope. The drum rollers are modified by welding triangular sections to the drum (known also as "land imprinters").

(e) Contour furrowing:

Contour furrowing involves the construction of a series of small, level channels (furrows) designed to capture and hold rainwater on moderately steep land, thereby reducing runoff and the potential erosion hazard. The distance between the furrows depends on the soil type and slope. Contour furrowing is typically applied to moderately steep grazing land.

- The furrows generally penetrate at least 300mm, spaced 1 to 10m apart. It is usually carried out on hard packed soils to improve water infiltration, or on overburden immediately prior to topsoiling to assist bonding between the two soil layers.
- Contour furrowing should be employed only with extreme caution on dispersive soils. Always seek expert (soil science) advice.

Contour furrowing is generally not considered a part of *surface roughening*, instead it is a land management technique typically used in rural areas.

(f) Contour ripping:

Contour ripping is the formation of 600 to 900mm deep furrows along the contour of slopes. The deep furrows capture and infiltrate stormwater thus making best use of limited rainfall. In semi-arid areas subject to occasional heavy rainfall (e.g. parts of northern Australia), soil saturation following such heavy rain can lead to concentrated runoff down the slope damaging the rip lines, and potentially resulting in high sediment runoff (similar to Photo 6).

- Formed using machinery such as single or multi-tine ripper (600–900mm deep) attached to a heavy tractor or bulldozer.
- Typical ripping with two tines spaced about 1m apart. Each twin-furrow being spaced 2 to 6m apart depending on the slope grade.
- Generally limited to slopes of less than 6:1 (10 degrees).
- Generally limited to a maximum 3:1 (H:V) slope.
- Contour ripping should be employed only with extreme caution on dispersive soils. If soils are dispersive, then contour ripping may increase the erosion risk.

Contour ripping is generally not considered a part of *surface roughening*, instead it is a land management technique typically used in rural areas, and for mine site rehabilitation within arid and semi-arid areas.

Description

The roughening of exposed soil slopes with horizontal groves running across the slope. It is different from ‘contour furrowing’ and ‘contour ripping’, which are often used as a land management tools in rural areas.

Surface roughening can be achieved by a number of methods including walking a tracked vehicle up and down the slope.

It can also be produced by attaching a serrated edge to a grader blade (especially when trimming road batters), or by using a chisel plough, scarifier or ripper.

Purpose

Surface roughening can be used on exposed and recently seeded surfaces to:

- increase stormwater infiltration;
- delay the formation of rilling;
- reduce wind-induced soil erosion;
- promote faster seed germination within the dozer cleat marks by trapping and holding small pools of water, as well as seed and fertiliser.
- reduce runoff velocity (up to a given rainfall intensity, beyond which rilling may begin to occur resulting in concentrated, high-velocity flow)

Limitations

Each treatment method is limited to a different maximum bank slope.

Surface roughening produced by dozer track marks is generally best used on sandy soils. On clayey soils there is the risk of soil compaction leading to the formation of minor channel depressions that may concentrate runoff.

Advantages

The benefits of increased slope roughness include:

- increased retention of water on slopes;
- increased water infiltration into the soil;
- reduced runoff volume;
- reduced dust generation.

Inexpensive to implement, but may not be a cost-effective use of heavy machinery on a construction site.

Can improve the stabilisation of topsoil on steep slopes if surface roughening has been applied to the subsoil.

Aids in the establishment of vegetation by allowing water to collect and pool within the cleat marks (track walking).

Disadvantages

Generally of limited value during periods of heavy rainfall.

Questionable benefit on construction sites given the cost and effort of application.

Common Problems

Problems can occur once the soils are saturated and surface runoff begins to move down the slope across the grooves and furrows causing erosion.

Special Requirements

Immediately seed and mulch roughened areas to optimise seed germination and growing conditions.

Existing rutting and gullies should be filled or suitably contoured.

Up-slope runoff should be diverted around treated area if such run-on water is likely to cause erosion.

Seek expert (soil science) advice before deep ripping or furrowing land containing dispersive subsoils.

Site Inspection

Inspect the area for the formation of rill or gully erosion, and where necessary, repeat the surface treatment or improve up-slope drainage control.

Check the furrows/cleat marks are deep enough.

Check the furrows/cleat marks are aligned with the contour.

Application

1. Refer to approved plans for location, extent, and application details. If there are questions or problems with the location, extent, or method of application contact the engineer or responsible on-site officer for assistance.
2. Fill or suitably contour any existing rutting, rilling or gullies.
3. Suitably divert up-slope stormwater runoff around treated area as directed within the approved plans, or otherwise as directed by the site engineer.
4. Apply treatment to the area to the depth and frequency (spacing) specified on the approved plans, or otherwise as directed by the site engineer.
5. Immediately seed and mulch roughened areas to optimise seed germination and growing conditions.

Maintenance

1. During the construction period, inspect the treated area prior to forecast rainfall, daily during extended periods of rainfall, after significant runoff producing rainfall, or otherwise on a weekly basis.
2. Fill erosion rills slightly above the original grade, or regrade the slope as directed to remove the rills.

Flow Diversion Bank: General

Flow Diversion Banks Part 1: General

DRAINAGE CONTROL TECHNIQUE

Low Gradient	✓	Velocity Control		Short Term	✓
Steep Gradient		Channel Lining		Medium-Long Term	✓
Outlet Control		Soil Treatment		Permanent	[1]

[1] Flow diversion banks are not commonly used as permanent drainage structures.

Symbol → DB →

Photo 1 – Flow diversion bank down-slope of a future pipeline installation

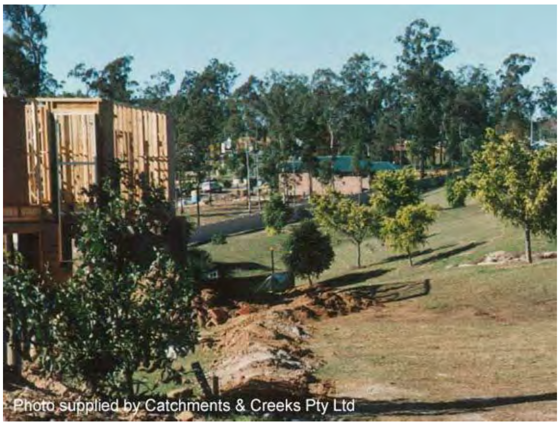


Photo 2 – Flow diversion bank up-slope of a building site

Key Principles

1. Key design parameters are the effective flow capacity of the structure, and the scour resistance of the embankment material.
2. The critical operational issue is usually preventing structural damage to the embankment as a result of high velocity flows or construction traffic.
3. Flow diversion banks are often favoured over *Catch Drains* in areas containing dispersive subsoil because their construction does not require exposure of the subsoils.

Design Information

Dimensional requirements of flow diversion banks and berms vary with the type of embankment. The recommended values are outlined in Table 1.

Table 1 – Recommended dimensional requirements of flow diversion banks/berms

Parameter	Earth banks	Compost berms [1]	Sandbag berms
Height (min)	500mm	300mm (450mm)	N/A
Top width (min)	500mm [2]	100mm (100mm)	N/A
Base width (min)	2500mm [2]	600mm (900mm)	N/A
Side slope (max)	2:1 (H:V)	1:1 (H:V)	N/A
Hydraulic freeboard	150mm (300mm) [3]	100mm	50mm

[1] Values in brackets apply to berms placed across land slopes steeper than 4:1 (H:V).

[2] Top width may be reduced in those non-critical situations in which overtopping will not cause excessive erosion and the banks are unlikely to experience damage from construction equipment.

[3] A minimum freeboard of 300mm applies to non-vegetated earth embankments.

Free standing earth embankments may be stabilised with rock, vegetation, or *Erosion Control Blankets*; however, unprotected topsoil embankments are also acceptable for short-term applications.

Maximum recommended spacing of flow diversion banks down long continuous slopes is provided in Table 2. The actual spacing specified for a given site may need to be less than that presented in Table 2 if the soils are highly susceptible to erosion, or if intense storm events are expected (i.e. northern parts of Australia during the wet season).

Table 2 – Maximum recommended spacing of flow diversion banks down slopes

Open Earth Slopes						Vegetated Slopes		
Slope	**Horiz.**	**Vert.**	**Slope**	**Horiz.**	**Vert.**	**Slope**	**Horiz.**	**Vert.**
1%	80m	0.9m	15%	19m	2.9m	< 10%	No maximum	
2%	60m	1.2m	20%	16m	3.2m	12%	100m	12m
4%	40m	1.6m	25%	14m	3.5m	15%	80m	12m
6%	32m	1.9m	30%	12m	3.5m	20%	55m	11m
8%	28m	2.2m	35%	10m	3.5m	25%	40m	10m
10%	25m	2.5m	40%	9m	3.5m	30%	30m	9m
12%	22m	2.6m	50%	6m	3.0m	> 36%	Case specific	

Photo 3 – Flow diversion berm used to minimise road runoff flowing down a steep, unstable section of the embankment

Photo 4 – Sandbag flow diversion berm used to minimise surface flow over a recently seeded embankment

Photo 5 – Earth flow diversion bank used to direct runoff towards the entrance of a *Slope Drain*

Photo 6 – Turf-lined flow diversion bank with grass-lined outlet chutes at regular intervals along the embankment

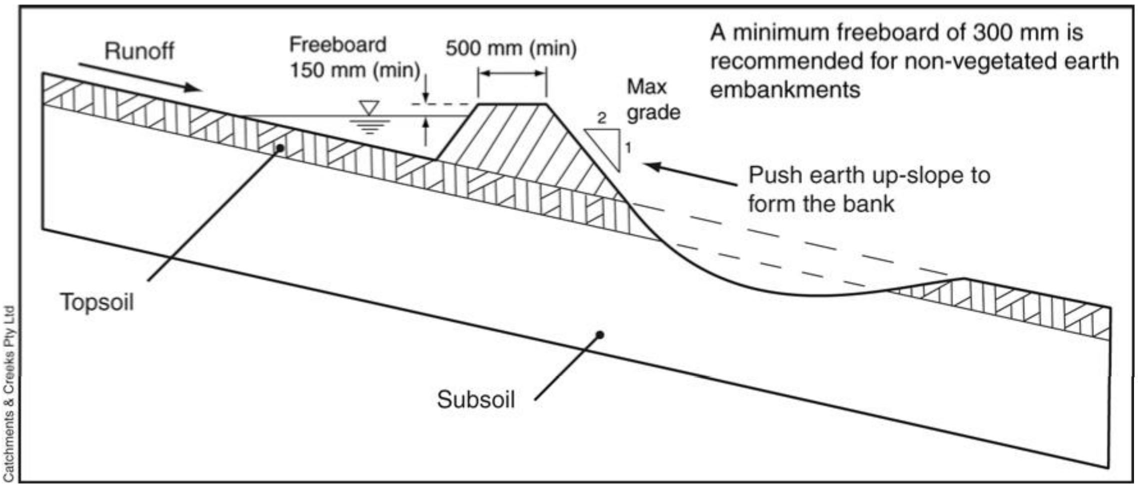
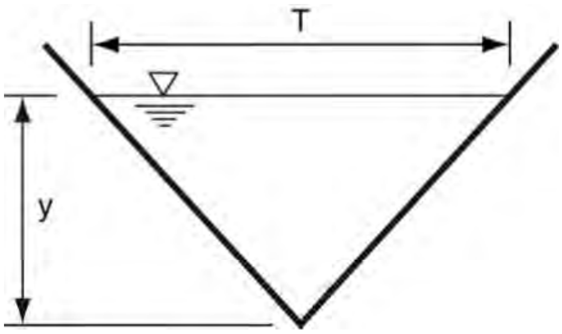
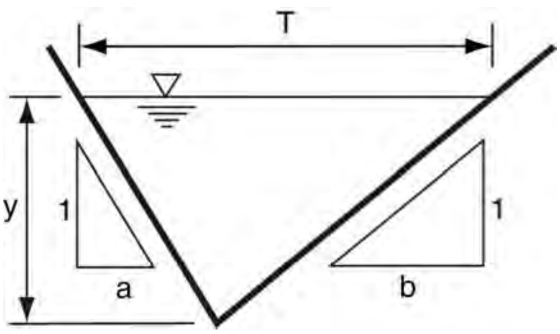


Figure 1 – Profile of 'back-push' bank

The hydraulic capacity of a flow diversion bank normally needs to be assessed on a case-by-case basis; however, the associated fact sheets *"Part 2: On earth slopes"* and *"Part 3: On grassed slopes"* provide the hydraulic capacity for drains with a standard triangular profile established on earth and grassed slopes respectively.

The geometric properties of triangular drainage channels formed by the construction of a flow diversion bank are provided in Table 3.

Table 3 – Geometric properties of triangular drainage profiles

<table>
<tr><td rowspan="3">Symmetrical or asymmetric V-drain:

</td><td>Area (A):
$$A = 0.5\,T\,y$$</td></tr>
<tr><td>Wetted perimeter (P):
$$P = \sqrt{T^2 + 4y^2}$$</td></tr>
<tr><td>Hydraulics radius (R):
$$R = \frac{Ty}{2\sqrt{T^2 + 4y^2}}$$</td></tr>
<tr><td rowspan="3">Asymmetric V-drain:
where flow top width, T = y(a + b)

</td><td>Area (A):
$$A = \left(\frac{a+b}{2}\right)y^2$$</td></tr>
<tr><td>Wetted perimeter (P):
$$P = y\left[\sqrt{(1+a^2)} + \sqrt{(1+b^2)}\right]$$</td></tr>
<tr><td>Hydraulics radius (R):
$$R = \frac{0.5(a+b)y}{\sqrt{(1+a^2)} + \sqrt{(1+b^2)}}$$</td></tr>
</table>

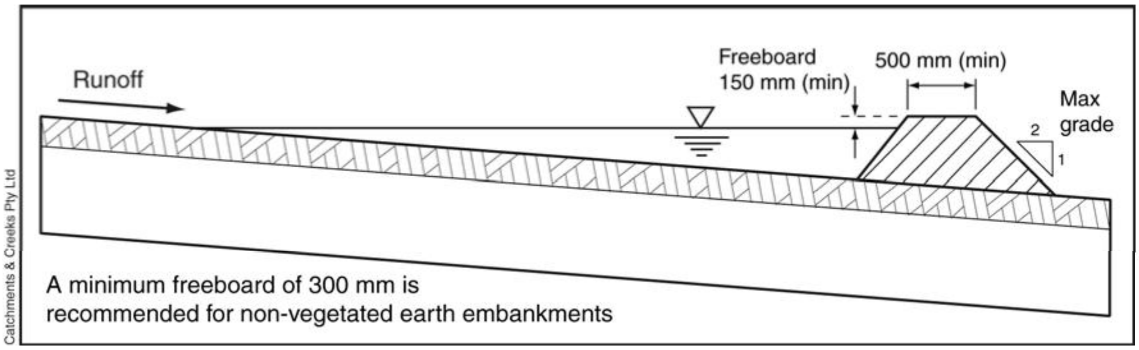


Figure 2 – Flow diversion bank formed from earth

Photo 7 – Flow diversion banks placed each side of drainage line passing through road construction site

Types of flow diversion banks:

The following provides a brief description of some of the flow diversion banks used within rural and construction land management.

Absorption bank	A level bank turned up at each end to promote water infiltration.
Back-push bank	A bank formed by moving in-situ earth up a slope.
Conventional bank	A bank formed by moving in-situ earth down thus forming an excavated drain up-slope of the bank. Also known as a 'catch bank'.
Diversion bank	A graded bank used to collect and divert water away from a soil disturbance, or to a dam, drainage channel, or sediment trap.
Graded bank	A bank constructed with a positive gradient to promote water movement.
Level bank	A bank constructed along a contour. Discharge usually occurs at each end of the bank.
Perimeter bank	A bank located along the upper or lower perimeter of a well-defined area, such as a building site, or along the top edge of a batter.
Trainer bank	A bank used to divert water away from unstable land.
Water-spreading bank	Banks used to collect and distribute surface runoff over an increased flow width. Typically used on low-gradient, marginal arable land.

Description

Flow diversion banks typically consist of a raised earth embankment normally placed along level or near level ground. Minor flow diversion berms can also be formed from tightly packed sandbags, or compost.

Short-term flow diversion banks can also be constructed from tightly packed straw bales. Such banks are often constructed prior to an impending storm.

The term *perimeter bank* is often used to describe an embankment constructed around the 'perimeter' of a work site. These are used to either prevent clean water entering the site, or to prevent the uncontrolled release of dirty water from a site.

The term *back-push bank* is used to describe an embankment formed by pushing in-situ soils up a slope to from an earth embankment.

Purpose

Flow diversion banks and berms are used as temporary drainage systems to:

- collect sheet runoff (clean or dirty) from slopes and transport it across the slope to a stable outlet (Photo 1);
- divert up-slope runoff around a stockpile or soil disturbance (Photo 2);
- divert stormwater away from an unstable slope (Photos 3 & 4);
- direct water to the inlet of a *Chute* or *Slope Drain* (Photos 5 & 6);
- control the depth of ponding around a sediment trap such as a stormwater drop (field) inlet.

Flow diversion banks can also act as a form of topsoil stockpile. Topsoil can be stripped from a site and used to form flow diversion banks either up-slope and/or down-slope of the soil disturbance (Photo 1). Such a practice can be very space effective when conducting 'strip' construction such as roadways and pipeline installation.

Limitations

Catchment area is limited by the allowable flow capacity of the diversion bank and the allowable flow velocity of the surface material.

Not used on slopes steeper than 10% (10:1).

Advantages

Quick to establish or re-establish if disturbed.

Generally inexpensive to construct and remove.

Allows for the management of stormwater flow without the need to excavate a drainage channel. This can be a significant advantage in areas that have highly erosive or dispersive subsoils.

Disadvantages

Can cause sediment problems and flow concentration if overtopped during a severe storm.

Can restrict the movement of equipment around the site.

Can be highly susceptible to damage by construction equipment.

Common Problems

Damaged by construction traffic.

Scour along the base of the embankment caused by excessive flow velocity or an unstable outlet.

Overtopping flows caused by the deposition of sediment up-slope of the bank.

Special Requirements

All flow diversion banks must have a stable outlet.

Flow diversion banks should be seeded and mulched if their working life is expected to exceed 30 days, or as required by the erosion control standard.

Banks should **not** be constructed of unstable, non-cohesive, or dispersive soil.

Location

When flow diversion banks are required and their locations are not shown on the approved plans, their location on the ground should be determined after taking into consideration the following:

- the bank must discharge to a stabilised outlet;
- the bank should drain to a sediment trap if the diverted water is expected to be contaminated with sediment;
- stormwater must not be unnaturally diverted or concentrated onto an adjacent property.

Site Inspection

Check for slumps, wheel track damage, or loss of freeboard.

Check for excessive sediment deposition.

Check for erosion along the bank.

Installation

1. Refer to approved plans for location, extent, and construction details. If there are questions or problems with the location, extent, or method of installation, contact the engineer or responsible on-site officer for assistance.
2. Clear the location for the bank, clearing only the area that is needed to provide access for personnel and equipment.
3. Remove roots, stumps, and other debris and dispose of them properly. Do not use debris to build the bank.
4. Form the bank from the material, and to the dimension specified in the approved plans.
5. If earth is used, then ensure the sides of the bank are no steeper than a 2:1 (H:V) slope, and the completed bank must be at least 500mm high.
6. If formed from sandbags, then ensure the bags are tightly packed such that water leakage through the bags is minimised.
7. Check the bank alignment to ensure positive drainage in the desired direction.
8. The bank should be vegetated (turfed, seeded and mulched), or otherwise stabilised immediately, unless it will operate for less than 30 days or if significant rainfall is not expected during the life of the bank.
9. Ensure the embankment drains to a stable outlet, and does not discharge to an unstable fill slope.

Maintenance

1. Inspect flow diversion banks at least weekly and after runoff-producing rainfall.
2. Inspect the bank for any slumps, wheel track damage or loss of freeboard. Make repairs as necessary.
3. Check that fill material or sediment has not partially blocked the drainage path up-slope of the embankment. Where necessary, remove any deposited material to allow free drainage.
4. Dispose of any collected sediment or fill in a manner that will not create an erosion or pollution hazard.
5. Repair any places in the bank that are weakened or in risk of failure.

Removal

1. When the soil disturbance above the bank is finished and the area is stabilised, the flow diversion bank should be removed, unless it is to remain as a permanent drainage feature.
2. Dispose of any sediment or earth in a manner that will not create an erosion or pollution hazard.
3. Grade the area and smooth it out in preparation for stabilisation.
4. Stabilise the area by grassing or as specified in the approved plan.

Flow Diversion Bank: Earth

Flow Diversion Banks Part 2: Earth slopes

DRAINAGE CONTROL TECHNIQUE

Low Gradient	✓	Velocity Control		Short Term	✓
Steep Gradient		Channel Lining		Medium-Long Term	✓
Outlet Control		Soil Treatment		Permanent	[1]

[1] Flow diversion banks are not commonly used as permanent drainage structures.

Symbol → DB →

Photo 8 – Flow diversion bank down-slope of a future pipeline installation

Photo 9 – Earth flow diversion bank used to direct runoff towards the *Slope Drain*

Key Principles

1. Key design parameters are the effective flow capacity of the structure, and the scour resistance of the embankment material.
2. The critical operational issue is usually preventing structural damage to the embankment as a result of high velocity flows or construction traffic.
3. Flow diversion banks are often favoured over *Catch Drains* in areas containing dispersive subsoil because their construction does not require exposure of the subsoils.

Design Information

The material contained within this fact sheet has been supplied for use by persons experienced in hydraulic design.

The recommended dimensional requirements of flow diversion banks are outlined in Table 1 (refer to the fact sheet: *Flow Diversion Banks, Part 1 – General*).

Recommended allowable flow velocities for open earth surfaces are provided in Table 4. The maximum flow velocity (i.e. the velocity most likely to cause erosion of the earth surface) is most likely to occur at the toe of the embankment where flow depth (y) is a maximum, as shown in Figure 3. In wide, shallow drains, such as typically occur adjacent flow diversion banks, the local flow velocity is dependent on the local flow depth rather than the hydraulic radius (R).

Table 5 presented the expected maximum flow velocity for various maximum flow depths and longitudinal channel gradients.

Tables 9 to 18 provide the expected flow capacity for flow diversion bank operating a various maximum flow depths on an open earth surface. These tables are based on an embankment side slope of 2:1 (H:V), and a Manning's roughness determined from Equation 1, but limited to a maximum value of, n = 0.2.

Note; flow capacity is presented in units of [L/s] in Tables 9 to 13, and units of [m^3/s] in Tables 14 to 18.

Table 4 – Allowable flow velocity for earth surfaces

Soil type	Allowable velocity	Comments
Extremely erodible soils	0.3m/s	• Dispersive clays are highly erodible even at low flow velocities and therefore must be either treated (e.g. with gypsum) or covered with a minimum 100mm of stable soil. • Highly erodible soils may include: Lithosols, Alluvials, Podzols, Siliceous sands, Soloths, Solodized solonetz, Grey podzolics, some Black earths, fine surface texture-contrast soils and Soil Groups ML and CL. • Moderately erodible soils may include: Red earths, Red or Yellow podzolics, some Black earths, Grey or Brown clays, Prarie soils and Soil Groups SW, SP, SM, SC. • Erosion-resistant soils may include: Xanthozem, Euchrozem, Krasnozems, some Red earth soils and Soil Groups GW, GP, GM, GC, MH and CH.
Sandy soils	0.45m/s	
Highly erodible soils	0.4 to 0.5m/s	
Sandy loam soils	0.5m/s	
Moderately erodible soils	0.6m/s	
Silty loam soils	0.6m/s	
Low erodible soils	0.7m/s	
Firm loam soils	0.7m/s	
Stiff clay very colloidal soils	1.1m/s	

Table 5 – Maximum flow velocity (toe of embankment) on earth surface (m/s) [1]

Flow depth	Gradient (S) along drain (%)									
	0.1	**0.2**	**0.4**	**0.6**	**0.8**	**1**	**2**	**4**	**6**	**8**
0.05	0.02	0.03	0.03	0.07	0.10	0.13	0.28	0.51	0.72	0.91
0.10	0.06	0.13	0.24	0.33	0.42	0.50	0.83	1.35	1.78	2.16
0.15	0.15	0.26	0.44	0.58	0.71	0.83	1.33	2.10		
0.20	0.23	0.38	0.62	0.81	0.99	1.14	1.80	2.79		
0.25	0.31	0.50	0.79	1.03	1.24	1.44	2.23			
0.30	0.38	0.61	0.96	1.24	1.49	1.72	2.64			
0.35	0.45	0.71	1.11	1.44	1.73	1.98				
0.40	0.52	0.82	1.27	1.63	1.95	2.24				
0.45	0.59	0.92	1.42	1.82	2.17					
0.50	0.65	1.01	1.56	2.00						

[1] Maximum flow velocity refers to the maximum local flow velocity, which would occur adjacent the toe of the flow diversion bank at the point of maximum flow depth. The velocity has been determined using Manning's equation based on a hydraulic radius (R) equal to the local flow depth (y), and Manning's roughness determined from Equation 1. These flow velocities are significantly greater than the "average" velocity within a given cross-section.

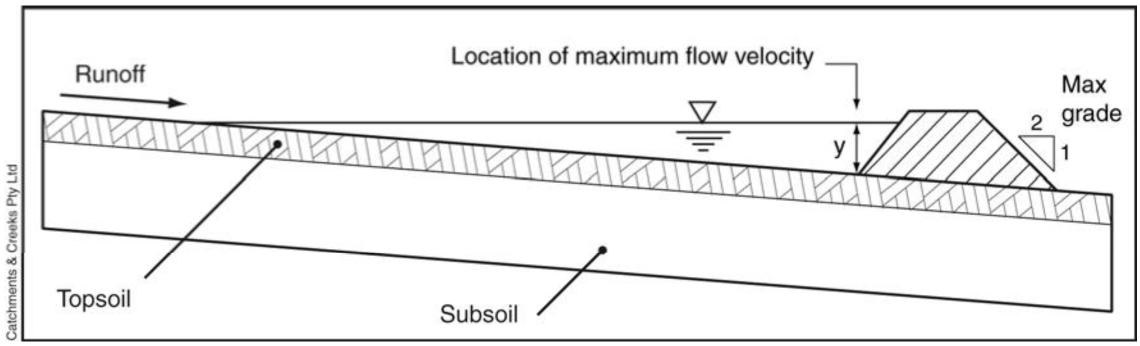


Figure 3 – Location of maximum flow velocity

Tables 6 to 8 provide typical Manning's n values.

Table 6 – Typical Manning's n roughness for deepwater flow conditions [1]

Channel description	Manning's roughness	Channel description	Manning's roughness
Smooth earth surface, few, if any, irregularities, sediment deposits, or loose surface material.	0.02	Concrete (smooth)	0.013
		Concrete (rough)	0.015
		Asphalt (smooth)	0.013
Slightly irregular earth surface with minor irregularities equivalent to that produced if an *Erosion Control Blanket* was pinned across the surface.	0.04	Asphalt (rough)	0.016
		Excavated open soil	0.020
		Gravel lined	0.025
		Earth with short grass	0.030

[1] Developed from Chow (1959)

Table 7 – Manning's roughness for various channel linings [1]

Material	Flow depth less than 150mm	Flow depth of 150 to 600mm	Flow depth greater than 600mm
Plastic sheeting	0.013		
Concrete	0.015	0.013	0.013
Asphalt	0.018	0.016	0.016
Straw (loose) covered with net	0.065	0.033	0.025
Jute net/mesh	0.028	0.022	0.019
Wood excelsior blanket	0.066	0.035	0.028
Turf Reinf. Mat – unvegetated	0.036	0.026	0.020
Turf Reinf. Mat – grassed	0.023	0.020	0.020

[1] Developed from Fifield (2001)

Table 8 – Manning's roughness for earth and lightly grassed surfaces [1]

R (m)	Drain slope in direction of flow (%)					
	1	**2**	**3**	**4**	**5**	**10**
0.05	0.100	0.070	0.059	0.053	0.049	0.040
0.10	0.043	0.037	0.034	0.032	0.031	0.027
0.15	0.034	0.030	0.028	0.027	0.026	0.024
0.20	0.030	0.027	0.025	0.025	0.024	0.022
0.25	0.028	0.025	0.024	0.023	0.023	0.021
0.30	0.026	0.024	0.023	0.022	0.022	0.020
0.40	0.024	0.022	0.022	0.021	0.020	0.020
0.50	0.023	0.022	0.021	0.020	0.020	0.020

[1] Values developed from Class E curve (Equation 1) for earth, burnt grass and lightly grassed surfaces (units of R [m] and S [m/m]). Note, minimum recommended Manning's roughness, n = 0.02. Caution use of Equation 1 for low values of hydraulic radius (negative values can occur).

Class E roughness:

$$n = \frac{R^{1/6}}{67.10 + 23.35\log_{10}(R^{1.4}.S^{0.4})} \quad \text{(Eqn 1)}$$

Hydraulic design of flow diversion banks:

Step 1 Determine the required design discharge based on the effective catchment area of the flow diversion bank.

Step 2 Determine the cross-sectional profile and surface condition. This fact sheet assumes the flow surface primarily consists of fully exposed or poorly vegetated earth.

Step 3 Determine the allowable flow velocity for the surface material from Table 4. Note; this is based on the surface of least scour resistance, whether the embankment or the adjacent slope.

Step 4 If the longitudinal gradient (S) of the drainage channel formed by the bank is known (i.e. set by site conditions), then determine the maximum allowable flow depth (y) from Table 5 given the allowable flow velocity determined in Step 3.

The maximum allowable flow depth (y) can also be determined directly from:

$$y = \frac{(n.V)^{3/2}}{S^{3/4}} \qquad \text{(S has units of m/m)}$$

If the longitudinal gradient of the drainage channel is not set by site conditions, then nominate a gradient from Table 5 based on a desirable maximum flow depth.

The maximum allowable longitudinal drainage gradient (S) can also be determined directly from:

$$S = \frac{(n.V)^{2}}{y^{4/3}} \qquad \text{(S has units of m/m)}$$

Step 5 Determine the Manning's roughness (n) from Tables 6 to 8, or Equation 1, as appropriate for the site conditions.

Step 6 Determine the cross-sectional flow area (A) and hydraulic radius (R).

Step 7 Determine the maximum allowable flow capacity (Q) of the flow diversion bank based on the values of n, A, R and S determined above.

Manning equation: $Q = (1/n)\, A\, R^{2/3}\, S^{1/2}$

Tables 9 to 18 provide flow capacities based on a simple triangular cross-sectional profile, an embankment side slope of 2:1 (H:V), and a Manning's roughness for open earth determined from Equation 1, but limited to a maximum value of, n = 0.2.

If the maximum flow capacity is less than the design discharge determined in Step 1, then it will be necessary to reduce the effective catchment area and design discharge of the flow diversion bank.

Alternatively, the scour resistance of the surface condition could be improved through appropriate erosion control measures, or the longitudinal gradient (S) of the drainage channel. Determine the required gradient (S) using Manning's equation.

$$S = \frac{(n.V)^{2}}{y^{4/3}} \qquad \text{(S has units of m/m)}$$

Step 8 Determined the required freeboard given the embankment type – refer to Table 1 in fact sheet: *Flow Diversion Banks, Part 1 – General*.

Step 9 Ensure suitable conditions exist (e.g. machinery access) to construct and maintain the embankment.

Step 10 Specify the overall dimensions of the flow diversion bank, including freeboard.

Step 11 Ensure the drainage embankment discharges to an appropriate, stable outlet.

Step 12 Appropriately consider all likely safety issues, and modify the embankment and/or surrounding environment where required.

Design example:

Design a short-term, non-vegetated, topsoil flow diversion bank capable of carrying a design discharge of $0.2m^3/s$ across a slope with a gradient of 5% (20:1) (note; this is the gradient of the land slope, not the drain slope). Both the topsoil embankment and the exposed earth surface is considered to be representative of erosion-resistant, loamy soil.

Step 1 The required design discharge is $0.2m^3/s$.

Step 2 Assume a simple triangular cross-sectional profile with fully exposed earth surface.

Step 3 From Table 4, adopt an allowable flow velocity, $V_{allow} = 0.7m/s$.

Step 4 A non-vegetated embankment is assumed, thus the recommended minimum freeboard is 300mm. This means for an embankment height of 500mm, the maximum flow depth (y) is 500 - 300 = 200mm. If is flow depth is insufficient, then an embankment height greater than 500mm should be considered.

Given y = 200mm, and $V_{allow} = 0.7m/s$, choose a longitudinal gradient (S) of 0.5% from Table 5.

Step 5 Hydraulic capacity will be determined from Table 12, therefore Manning's roughness will be based on Equation 1.

Step 6 There is no need to determine the cross-sectional flow area (A) and hydraulic radius (R) because the supplied design tables will be used. However, for demonstration purposes, given a maximum flow depth, y = 0.2m; embankment side slope, a = 2; and land slope, b = 20 (i.e. 5%); then:

$$A = 0.440m^2 \quad \text{(not final design)}$$

$$R = 0.099m$$

$$n = 0.05$$

Step 7 Given a maximum flow depth, y = 0.2m; land slope of 5%; and longitudinal drain slope, S = 0.5%; from Table 12 the maximum flow capacity (Q) is:

$$Q_{max} = 125L/s = 0.125m^3/s < 0.2m^3/s \quad \text{Problem!}$$

Thus the flow diversion bank will not have adequate flow capacity to carry the design discharge of $0.2m^3/s$.

Thus, try an increased maximum flow depth, y = 0.3m (thus the minimum embankment height will now be 0.3 + 0.3 = 0.6m).

From Table 5, try a drain slope, S = 0.2%

From Table 14, $Q_{max} = 0.252m^3/s > 0.2m^3/s$ OK

Step 8 From Table 1 the required minimum freeboard for a non-vegetated earth embankment is 300mm.

Step 9 Ensure suitable conditions exist (e.g. machinery access) to construct and maintain the embankment.

Step 10 Specify the overall dimensions of the flow diversion bank, including freeboard.

Embankment height of 600mm

Embankment side slopes of 2:1 (H:V)

Base width of embankment of 2900mm

Freeboard of 300mm

Longitudinal gradient of embankment of 0.2%

Step 11 Ensure the drainage embankment discharges to an appropriate, stable outlet.

Step 12 Appropriately consider all likely safety issues, and modify the embankment and/or surrounding environment where required.

Note, the allowable flow depth (y) is limited by the drain gradient and the allowable flow velocity.

Table 9 – Flow capacity (L/s) for flow diversion banks on earth surface [1]

Flow diversion bank on earth surface						Flow depth, y = 0.05m				
Land slope %	Gradient (S) along drain (%)									
	0.1	0.2	0.4	0.6	0.8	1	2	4	6	8
1	1.72	2.43	3.44	4.22	4.87	5.44	7.70	10.9	16.2	25.3
2	0.88	1.24	1.75	2.15	2.48	2.77	3.92	5.54	8.16	12.8
3	0.59	0.84	1.19	1.46	1.68	1.88	2.66	3.76	5.48	8.63
4	0.45	0.64	0.91	1.11	1.28	1.43	2.03	2.87	4.14	6.53
5	0.37	0.52	0.74	0.90	1.04	1.17	1.65	2.33	3.33	5.28
6	0.31	0.44	0.62	0.77	0.88	0.99	1.40	1.98	2.80	4.44
7	0.27	0.38	0.54	0.67	0.77	0.86	1.22	1.72	2.41	3.84
8	0.24	0.34	0.48	0.59	0.68	0.77	1.08	1.53	2.12	3.39
9	0.22	0.31	0.44	0.54	0.62	0.69	0.98	1.38	1.89	3.03
10	0.20	0.28	0.40	0.49	0.56	0.63	0.89	1.26	1.71	2.75
12	0.17	0.24	0.34	0.42	0.48	0.54	0.77	1.08	1.44	2.33
15	0.14	0.20	0.29	0.35	0.40	0.45	0.64	0.90	1.16	1.90
20	0.11	0.16	0.23	0.28	0.32	0.36	0.51	0.73	0.89	1.46
25	0.10	0.14	0.20	0.24	0.28	0.31	0.44	0.62	0.76	1.19
33.3	0.08	0.11	0.16	0.20	0.23	0.25	0.36	0.51	0.62	0.91
50	0.06	0.09	0.13	0.15	0.18	0.20	0.28	0.40	0.49	0.61

[1] NOTE: Flow rate is presented in units of litres per second, <u>not</u> m^3/s as used in Tables 14 to 18.

Table 10 – Flow capacity (L/s) for flow diversion banks on earth surface [1]

Flow diversion bank on earth surface						Flow depth, y = 0.10m				
Land slope %	Gradient (S) along drain (%)									
	0.1	0.2	0.4	0.6	0.8	1	2	4	6	8
1	10.9	15.5	21.9	33.4	50.4	66.7	140	262	366	460
2	5.56	7.87	11.1	16.8	25.5	33.8	70.8	133	186	234
3	3.77	5.34	7.55	11.3	17.2	22.8	47.9	89.8	126	158
4	2.88	4.07	5.76	8.57	13.0	17.3	36.4	68.4	95.8	121
5	2.34	3.31	4.68	6.91	10.5	14.0	29.5	55.5	77.8	97.9
6	1.98	2.81	3.97	5.80	8.87	11.8	24.9	46.9	65.8	82.8
7	1.73	2.44	3.46	5.01	7.67	10.2	21.6	40.7	57.2	72.0
8	1.54	2.17	3.07	4.42	6.77	9.03	19.2	36.1	50.7	63.8
9	1.39	1.96	2.77	3.95	6.08	8.11	17.2	32.5	45.7	57.5
10	1.27	1.79	2.54	3.58	5.51	7.37	15.7	29.6	41.6	52.5
12	1.09	1.54	2.18	3.01	4.67	6.25	13.4	25.3	35.6	44.9
15	0.91	1.28	1.82	2.45	3.82	5.13	11.0	21.0	29.5	37.2
20	0.73	1.03	1.46	1.87	2.95	4.00	8.69	16.6	23.4	29.5
25	0.62	0.88	1.24	1.52	2.42	3.30	7.26	13.9	19.7	24.9
33.3	0.51	0.72	1.02	1.25	1.88	2.59	5.80	11.2	15.9	20.2
50	0.40	0.56	0.80	0.98	1.29	1.82	4.26	8.40	12.0	15.2

[1] NOTE: Flow rate is presented in units of litres per second, <u>not</u> m^3/s as used in Tables 14 to 18.

Note, the allowable flow depth (y) is limited by the drain gradient and the allowable flow velocity.

Table 11 – Flow capacity (L/s) for flow diversion banks on earth surface

Flow diversion bank on earth surface						Flow depth, y = 0.15m				
Land slope %	Gradient (S) along drain (%)									
	0.1	0.2	0.4	0.6	0.8	1	2	4	6	8
1	32.2	71.4	157	232	301	364	640	1082	1452	1780
2	16.4	36.1	79.5	118	153	185	325	550	738	905
3	11.1	24.3	53.7	79.6	103	125	220	372	500	613
4	8.49	18.5	40.8	60.5	78.5	95.3	168	284	381	467
5	6.91	14.9	33.1	49.1	63.7	77.3	136	230	309	380
6	5.85	12.6	27.9	41.5	53.8	65.4	115	195	262	321
7	5.10	10.9	24.2	36.0	46.8	56.8	100	170	228	279
8	4.53	9.60	21.4	31.9	41.5	50.4	88.8	150	202	248
9	4.09	8.61	19.3	28.7	37.3	45.3	80.0	136	182	224
10	3.74	7.82	17.6	26.1	34.0	41.3	73.0	124	166	204
12	3.21	6.62	14.9	22.3	29.0	35.3	62.4	106	142	175
15	2.68	5.42	12.3	18.4	24.0	29.2	51.8	88.0	118	145
20	2.15	4.20	9.68	14.5	19.0	23.1	41.1	70.0	94.2	116
25	1.83	3.45	8.07	12.2	15.9	19.4	34.6	59.1	79.6	97.8
33.3	1.50	2.67	6.42	9.75	12.8	15.7	28.1	48.0	64.8	79.7
50	1.17	1.84	4.68	7.21	9.55	11.7	21.2	36.6	49.5	61.0

[1] NOTE: Flow rate is presented in units of litres per second, not m^3/s as used in Tables 14 to 18.

Table 12 – Flow capacity (L/s) for flow diversion banks on earth surface

Flow diversion bank on earth surface						Flow depth, y = 0.20m				
Land slope %	Gradient (S) along drain (%)									
	0.1	0.2	0.4	0.6	0.8	1	2	4	6	8
1	187	264	488	680	852	1011	1687	2747	3625	4398
2	94.8	134	248	345	433	514	857	1397	1843	2236
3	64.1	90.6	168	234	293	348	581	947	1249	1516
4	48.7	68.9	128	178	223	265	442	721	952	1155
5	39.5	55.9	104	145	181	215	359	586	774	939
6	33.4	47.2	87.6	122	153	182	304	496	655	795
7	29.0	41.0	76.1	106	133	158	264	431	570	691
8	25.7	36.3	67.5	94.2	118	140	235	383	506	614
9	23.1	32.7	60.8	84.9	107	127	212	345	456	554
10	21.0	29.7	55.4	77.4	97.2	115	193	315	416	506
12	17.9	25.4	47.3	66.2	83.1	98.8	165	270	357	433
15	14.8	21.0	39.2	54.9	69.0	82.0	137	225	297	360
20	11.7	16.5	31.0	43.5	54.8	65.2	109	179	237	287
25	9.8	13.8	26.1	36.6	46.2	54.9	92.4	151	200	243
33.3	7.81	11.1	21.0	29.7	37.4	44.6	75.2	123	163	199
50	5.76	8.15	15.8	22.4	28.3	33.8	57.4	94.6	125	153

[1] NOTE: Flow rate is presented in units of litres per second, not m^3/s as used in Tables 14 to 18.

Note, the allowable flow depth (y) is limited by the drain gradient and the allowable flow velocity.

Table 13 – Flow capacity (L/s) for flow diversion banks on earth surface

Flow diversion bank on earth surface						Flow depth, y = 0.25m				
Land slope %	Gradient (S) along drain (%)									
	0.1	0.2	0.4	0.6	0.8	1	2	4	6	8
1	439	621	1078	1464	1808	2123	3451	5513	7205	8691
2	223	315	548	744	919	1079	1754	2803	3664	4420
3	151	213	371	504	623	731	1189	1900	2484	2996
4	115	162	283	384	474	557	906	1448	1893	2284
5	93.2	132	229	312	385	453	736	1177	1539	1857
6	78.8	111	194	264	326	383	623	996	1302	1572
7	68.5	96.8	169	229	283	333	542	867	1134	1368
8	60.7	85.9	150	204	252	296	481	770	1007	1215
9	54.7	77.4	135	183	227	267	434	694	908	1096
10	49.9	70.5	123	167	207	243	396	634	829	1001
12	42.6	60.3	105	143	177	208	339	543	710	857
15	35.3	49.9	87.4	119	147	173	282	452	591	714
20	28.0	39.6	69.4	94.6	117	138	225	360	472	570
25	23.5	33.2	58.5	79.8	98.9	116	190	305	400	483
33.3	19.0	26.8	47.4	64.9	80.4	94.8	155	249	327	395
50	14.3	20.2	35.9	49.3	61.3	72.4	119	192	252	304

[1] NOTE: Flow rate is presented in units of litres per second, <u>not</u> m^3/s as used in Tables 14 to 18.

Table 14 – Flow capacity (m^3/s) for flow diversion banks on earth surface

Flow diversion bank on earth surface						Flow depth, y = 0.30m				
Land slope %	Gradient (S) along drain (%)									
	0.1	0.2	0.4	0.6	0.8	1	2	4	6	8
1	0.680	1.185	1.992	2.666	3.263	3.809	6.093	9.615	12.49	15.01
2	0.345	0.602	1.012	1.355	1.659	1.937	3.098	4.890	6.353	7.635
3	0.234	0.408	0.686	0.918	1.124	1.312	2.100	3.315	4.307	5.176
4	0.178	0.310	0.522	0.699	0.857	1.000	1.601	2.527	3.283	3.946
5	0.144	0.252	0.424	0.568	0.696	0.813	1.301	2.054	2.669	3.208
6	0.122	0.213	0.359	0.481	0.589	0.688	1.101	1.739	2.259	2.716
7	0.106	0.185	0.312	0.418	0.512	0.598	0.958	1.513	1.967	2.364
8	0.094	0.164	0.277	0.371	0.455	0.531	0.851	1.344	1.747	2.100
9	0.085	0.148	0.250	0.335	0.410	0.479	0.767	1.212	1.576	1.894
10	0.077	0.135	0.228	0.305	0.374	0.437	0.700	1.107	1.439	1.730
12	0.066	0.116	0.195	0.262	0.321	0.374	0.600	0.949	1.233	1.483
15	0.055	0.096	0.162	0.217	0.267	0.311	0.499	0.790	1.027	1.235
20	0.043	0.076	0.129	0.173	0.212	0.248	0.398	0.630	0.820	0.986
25	0.036	0.064	0.109	0.146	0.179	0.210	0.337	0.534	0.695	0.836
33.3	0.029	0.052	0.089	0.119	0.146	0.171	0.275	0.437	0.569	0.684
50	0.022	0.039	0.067	0.091	0.112	0.131	0.212	0.337	0.439	0.528

Note, the allowable flow depth (y) is limited by the drain gradient and the allowable flow velocity.

Table 15 – Flow capacity (m^3/s) for flow diversion banks on earth surface

Flow diversion bank on earth surface						Flow depth, y = 0.35m				
Land slope %	Gradient (S) along drain (%)									
	0.1	0.2	0.4	0.6	0.8	1	2	4	6	8
1	1.181	1.998	3.290	4.361	5.309	6.171	9.766	15.28	19.76	23.68
2	0.600	1.015	1.672	2.217	2.699	3.138	4.966	7.771	10.05	12.05
3	0.406	0.688	1.133	1.503	1.829	2.127	3.366	5.268	6.815	8.168
4	0.309	0.524	0.863	1.145	1.394	1.621	2.566	4.016	5.196	6.227
5	0.251	0.426	0.701	0.930	1.133	1.317	2.086	3.265	4.224	5.063
6	0.212	0.360	0.593	0.787	0.959	1.115	1.765	2.764	3.576	4.286
7	0.185	0.313	0.516	0.685	0.834	0.970	1.536	2.406	3.113	3.731
8	0.164	0.278	0.458	0.608	0.741	0.861	1.365	2.137	2.765	3.315
9	0.148	0.250	0.413	0.548	0.668	0.777	1.231	1.928	2.495	2.991
10	0.135	0.228	0.377	0.501	0.610	0.709	1.124	1.760	2.278	2.731
12	0.115	0.196	0.323	0.429	0.522	0.608	0.963	1.509	1.953	2.341
15	0.095	0.162	0.269	0.357	0.435	0.506	0.802	1.257	1.627	1.951
20	0.076	0.129	0.214	0.284	0.346	0.403	0.640	1.003	1.300	1.558
25	0.064	0.109	0.181	0.240	0.293	0.341	0.542	0.850	1.102	1.321
33.3	0.052	0.089	0.147	0.196	0.239	0.279	0.443	0.696	0.902	1.083
50	0.039	0.067	0.113	0.150	0.184	0.214	0.341	0.537	0.697	0.837

Table 16 – Flow capacity (m^3/s) for flow diversion banks on earth surface

Flow diversion bank on earth surface						Flow depth, y = 0.40m				
Land slope %	Gradient (S) along drain (%)									
	0.1	0.2	0.4	0.6	0.8	1	2	4	6	8
1	1.867	3.099	5.030	6.625	8.030	9.308	14.61	22.72	29.29	35.02
2	0.949	1.575	2.557	3.369	4.084	4.734	7.432	11.55	14.90	17.82
3	0.643	1.067	1.733	2.283	2.768	3.208	5.038	7.833	10.10	12.08
4	0.489	0.813	1.321	1.740	2.110	2.446	3.841	5.972	7.702	9.211
5	0.397	0.660	1.073	1.414	1.715	1.988	3.122	4.855	6.262	7.489
6	0.336	0.559	0.908	1.197	1.451	1.682	2.643	4.110	5.301	6.340
7	0.292	0.486	0.790	1.041	1.263	1.464	2.300	3.578	4.615	5.520
8	0.259	0.431	0.702	0.925	1.121	1.300	2.043	3.179	4.100	4.904
9	0.234	0.389	0.633	0.834	1.011	1.173	1.843	2.868	3.699	4.425
10	0.213	0.355	0.577	0.761	0.923	1.071	1.683	2.619	3.378	4.041
12	0.182	0.304	0.495	0.652	0.791	0.917	1.442	2.245	2.896	3.465
15	0.152	0.253	0.411	0.543	0.658	0.764	1.201	1.870	2.413	2.887
20	0.120	0.201	0.328	0.433	0.525	0.609	0.959	1.494	1.928	2.307
25	0.102	0.170	0.277	0.366	0.445	0.516	0.813	1.266	1.635	1.957
33.3	0.082	0.138	0.226	0.299	0.363	0.422	0.665	1.037	1.340	1.604
50	0.063	0.106	0.174	0.230	0.279	0.325	0.513	0.802	1.036	1.241

Note, the allowable flow depth (y) is limited by the drain gradient and the allowable flow velocity.

Table 17 – Flow capacity (m^3/s) for flow diversion banks on earth surface

Flow diversion bank on earth surface						Flow depth, y = 0.45m				
Land slope %	Gradient (S) along drain (%)									
	0.1	0.2	0.4	0.6	0.8	1	2	4	6	8
1	2.765	4.525	7.270	9.53	11.51	13.31	20.77	32.13	41.32	49.33
2	1.405	2.301	3.697	4.845	5.854	6.771	10.57	16.34	21.02	25.10
3	0.952	1.559	2.505	3.284	3.968	4.590	7.163	11.08	14.25	17.02
4	0.725	1.188	1.909	2.503	3.025	3.499	5.461	8.449	10.87	12.98
5	0.589	0.965	1.552	2.034	2.459	2.844	4.440	6.869	8.836	10.55
6	0.498	0.817	1.313	1.722	2.081	2.407	3.758	5.816	7.481	8.932
7	0.433	0.710	1.143	1.498	1.811	2.095	3.271	5.063	6.513	7.777
8	0.385	0.631	1.015	1.331	1.609	1.861	2.906	4.498	5.786	6.909
9	0.347	0.569	0.915	1.200	1.451	1.679	2.622	4.058	5.221	6.234
10	0.316	0.519	0.835	1.096	1.325	1.533	2.394	3.706	4.768	5.694
12	0.271	0.445	0.716	0.939	1.135	1.313	2.052	3.177	4.088	4.882
15	0.225	0.370	0.596	0.782	0.945	1.094	1.709	2.647	3.407	4.069
20	0.179	0.294	0.475	0.623	0.754	0.873	1.365	2.115	2.723	3.252
25	0.151	0.249	0.402	0.528	0.639	0.739	1.157	1.794	2.310	2.759
33.3	0.123	0.203	0.328	0.431	0.522	0.605	0.947	1.470	1.893	2.263
50	0.094	0.155	0.252	0.332	0.402	0.466	0.732	1.137	1.465	1.752

Table 18 – Flow capacity (m^3/s) for flow diversion banks on earth surface

Flow diversion bank on earth surface						Flow depth, y = 0.50m				
Land slope %	Gradient (S) along drain (%)									
	0.1	0.2	0.4	0.6	0.8	1	2	4	6	8
1	3.899	6.32	10.06	13.14	15.83	18.28	28.38	43.72	56.11	66.89
2	1.982	3.211	5.117	6.680	8.052	9.296	14.44	22.24	28.54	34.03
3	1.343	2.176	3.468	4.528	5.458	6.302	9.787	15.08	19.35	23.07
4	1.023	1.658	2.644	3.452	4.161	4.804	7.462	11.50	14.76	17.60
5	0.831	1.347	2.149	2.806	3.382	3.905	6.067	9.348	12.00	14.31
6	0.703	1.140	1.819	2.375	2.863	3.306	5.136	7.915	10.16	12.11
7	0.612	0.992	1.583	2.067	2.492	2.877	4.471	6.890	8.846	10.55
8	0.543	0.881	1.406	1.836	2.213	2.556	3.972	6.122	7.859	9.371
9	0.490	0.794	1.268	1.656	1.997	2.306	3.583	5.523	7.091	8.456
10	0.447	0.725	1.157	1.512	1.823	2.105	3.272	5.044	6.477	7.723
12	0.383	0.621	0.992	1.296	1.562	1.805	2.805	4.325	5.554	6.623
15	0.318	0.517	0.825	1.079	1.301	1.503	2.337	3.604	4.629	5.521
20	0.253	0.412	0.658	0.861	1.039	1.200	1.867	2.880	3.700	4.413
25	0.214	0.348	0.557	0.729	0.880	1.017	1.583	2.443	3.139	3.745
33.3	0.174	0.284	0.456	0.596	0.720	0.832	1.296	2.003	2.574	3.072
50	0.133	0.218	0.350	0.459	0.555	0.642	1.002	1.550	1.994	2.380

Catch Drain

Catch Drains Part 5: Rock-lined

DRAINAGE CONTROL TECHNIQUE

Low Gradient	✓	Velocity Control		Short Term	✓
Steep Gradient		Channel Lining		Medium-Long Term	✓
Outlet Control		Soil Treatment		Permanent	[1]

[1] The design of permanent catch drains requires consideration of issues not discussed within this fact sheet, such as maintenance requirements. This fact sheet should not be used for the design of permanent drains.

Symbol → CD →

Photo 13 – Rock lined catch drain

Photo 14 – Rock-lined catch drain

Key Principles

1. Catch drains typically have standardised cross-sectional dimensions. Rather than uniquely sizing each catch drain to a given catchment, standard-sized drains are used based on a maximum allowable catchment area for a given rainfall intensity.
2. The **maximum** recommended spacing of catch drains down slopes (Table 3, *Part 1 – General information*) is based on the aim of avoiding rill erosion within the up-slope drainage slope. It should be noted that the **actual** spacing of catch drains down a given slope may need to be less than the specified maximum spacing if the soils are highly erosive soils, or if rilling begins to occur between two existing drains.
3. The critical design parameters are the spacing of the drains down a slope, the maximum allowable catchment area, the choice of lining material (e.g. earth, turf, rock or erosion control mats), and the required channel gradient.

Design Information

The following information must be read in association with the general information presented in *Part 1 – General information*. The following design tables specifically address rock-lined catch drains of specific dimensions.

The design procedure outlined within this fact sheet has been developed to provide a simplified approach suitable for appropriately trained persons involved in the regular design of temporary catch drains. The procedure is just **one** example of how catch drains can be designed. Designers experienced in hydraulic design can of course, design a catch drain using the general principles of open channel hydrologic/hydraulic as outlined in IECA (2008) Appendix A – *Construction site hydrology and hydraulics*.

Common Problems

Rock-lined catch drains are often cut to the wrong dimensions (typically under-sized), and following placement of the rock the drains have insufficient available flow area to carry the design discharge.

Stormwater approaching the drain fails to enter the rock-lined section of the drain, but instead is deflected down the upper edge of the rock causing rill erosion (Figure 9). This problem most commonly occurs when the rock is placed such that is sits above the elevation of the surrounding ground. The problem can be avoided by ensuring that the drain is properly 'boxed out', and that the rocks sit flush with the surrounding soil (Figures 10 & 11).

Catch drains not discharging to a stable outlet either causing downstream erosion, or initiating erosion under the rock lining.

Special Requirements

The erodibility and dispersive nature of the local subsoils should be investigated before planning or designing any excavated drains.

Straw bales or other sediment traps should **not** be placed within these drains due to the risk of causing surcharging of the drain.

Catch drain should drain to a suitable sediment trap if the diverted water is expected to contain sediment. Clean water should divert around sediment traps.

The drain must have positive gradient along its full length to allow free drainage.

Sufficient space must be provided to allow necessary maintenance access.

Site Inspection

Check that the drain has a stable, positive grade along its length.

Check for a stable drain outlet.

Check if the associated flow diversion bank (if any) is free of damage, i.e. damage caused by construction traffic.

Check that the drain has the required width and depth to achieve the desired hydraulic capacity.

Check that the rock is not reducing the drain's required hydraulic capacity.

Check if rill erosion is occurring within the catchment area up-slope of the drain. If rilling is occurring, then the lateral spacing of the drains will need to be reduced. However, some degree of rill erosion should be expected if recent storms exceeded the intensity of the nominated design storm.

Inspect for evidence of water spilling out (overtopping) of the drain, or erosion down-slope of the drain.

Check for displacement of rock or exposure of the underlying earth.

Check for placement of filter cloth under the rock. Note; filter cloth may not be specified if the spaces between the rock are filled with soil and planted (typical only for permanent drains in certain climates).

Materials

Rock:

- All rock must be hard, weather resistant, and durable against disintegration under conditions to be met in handling, placement and operation.
- All rock must have its greatest dimension not greater than 3 times its least dimensions.
- The rock used in formation of the drain must be evenly graded with 50% by weight larger than the specified nominal rock size and have sufficient small rock to fill the voids between the larger rock. Dirt, fines, and smaller rock must not exceed 5% by weight.
- The diameter of the largest rock size should be no larger than 1.5 times the nominal rock size. Specific gravity to be at least 2.5.
- The colour of the riprap shall be [*insert*] and must be approved by the engineer. Once approved, the colour shall be kept consistent through the project.

Filter cloth:

- Geotextile fabric: heavy-duty, needle-punched, non-woven filter cloth, minimum 'bidim' A24 or equivalent.

Installation

1. Refer to approved plans for location, extent, and construction details. If there are questions or problems with the location, extent, or method of installation, contact the engineer or responsible on-site officer for assistance.

2. Prior to placement, all rocks must be visually checked for size, elongation, cracks, deterioration and other visible. The degree and thoroughness of such checking must be appropriate for the potential consequences associated with failure of the structure or purpose for which the material will be used.
3. Clear the location for the catch drain, clearing only what is needed to provide access for personnel and equipment for installation.
4. Remove roots, stumps, and other debris and dispose of them properly. Do not use debris to build the bank.
5. Remove all soft, yielding material; replace with suitable on-site material; compact to smooth firm surface.
6. Excavate the drain to the lines and grades shown on the approved plans. Over-cut the drain to a depth equal to the specified depth of rock placement such that the finished top surface will be at the elevation of the surrounding land. Placement of the rock lining must not reduce the drain's top width and depth as specified within the approved plans.
7. Grade the drain to the specified slope and form the associated embankment with compacted fill. Note that the drain invert must fall 10cm every 10m for each 1% of channel gradient.
8. Ensure the sides of the cut drain are no steeper than a 1.5:1 (H:V) slope and the embankment fill slopes no steeper than 2:1.
9. If the drain is cut into a dispersive (sodic) soil, then prior to placing filter cloth, the exposed dispersive soil must be covered with a minimum 200mm thick layer of non-dispersive soil prior to placement of filter cloth or rocks.
10. If a filter cloth underlay is specified, place the filter fabric directly on the prepared foundation. If more than one sheet of filter cloth is required to over the area, overlap the edge of each sheet at least 300mm, and secure anchor pins at minimum 1m spacing along the overlap.
11. Ensure the filter cloth is protected from punching or tearing during installation of the fabric and the rock. Repair any damage by removing the rock and placing with another piece of filter cloth over the damaged area overlapping the existing fabric a minimum of 300mm.
12. Placement of rock should follow immediately after placement of the filter layer. Place rock so that it forms a dense, well-graded mass of rock with a minimum of voids.
13. Place rock lining to the extent and depth indicated within the approved plans.
14. Ensure the rock is placed in an appropriate manner to avoid displacing underlying materials or placing undue impact force on the bedding materials.
15. Ensure the rock is placed with a minimum thickness of 1.5 times the nominal rock size (d50).
16. Ensure materials that are d50 and larger are positioned flush with the top surface with faces and shapes matched to minimise voids.
17. Ensure projections above or depressions under the specified top surface are less than 20% of the rock layer thickness. The average surface plane of the finished rock is defined as the plane where 50% of the tops of rocks would contact.
18. Ensure the completed drain has sufficient deep (as specified for the type of drain) measured from the drain invert (average surface plane along channel invert) to the top of the embankment. The average surface plane of the finished rock is defined as the plane where 50% of the tops of rocks would contact.
19. To the maximum degree practicable, the material between larger rock must not be loose or easily displaced by the expected flow.
20. After placement of the rock lining, ensure the drain has a constant fall in the desired direction free of obstructions.
21. Ensure the drain discharges to a stable outlet such that soil erosion will be prevented from occurring. Ensure the drain does not discharge to an unstable fill slope.

Maintenance

1. Inspect all catch drains at least weekly and after runoff-producing storm events and repair any slumps, bank damage, or loss of freeboard.
2. Closely inspect the outer edges of the rock protection. Ensure water entry into the rock-lined area is not causing erosion along the edge of the rock protection.
3. Carefully check the stability of the rock looking for indications of piping, scour holes, or bank failures.
4. Replace or reposition the surface rock such that the drain functions as required and the drain's required hydraulic capacity is not reduced.
5. Replace any displaced rock with rock of a significantly (minimum 110%) larger size than the displaced rock.
6. Ensure sediment is not partially blocking the drain. Where necessary, remove any deposited material to allow free drainage.
7. Dispose of any sediment or fill in a manner that will not create an erosion or pollution hazard.

Removal

1. When the soil disturbance above the catch drain is finished and the area is stabilised, the drain and any associated banks should be removed, unless it is to remain as a permanent drainage feature.
2. Dispose of any sediment or earth in a manner that will not create an erosion or pollution hazard.
3. Grade the area and smooth it out in preparation for stabilisation.
4. Stabilise the area by grassing or as specified within the approved plan.

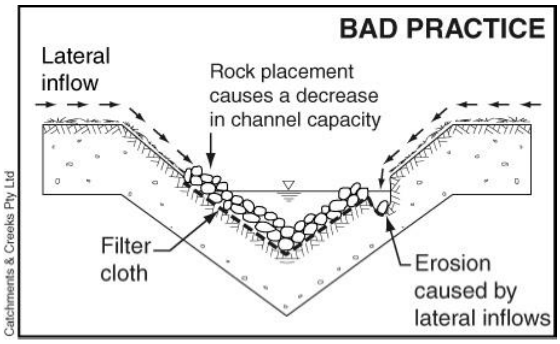


Figure 9 – Example of poor placement of rock

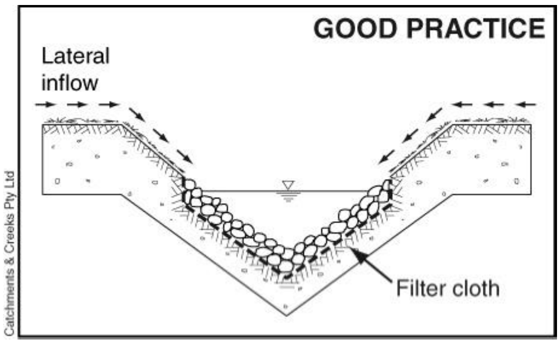


Figure 10 – Example of good placement of rock

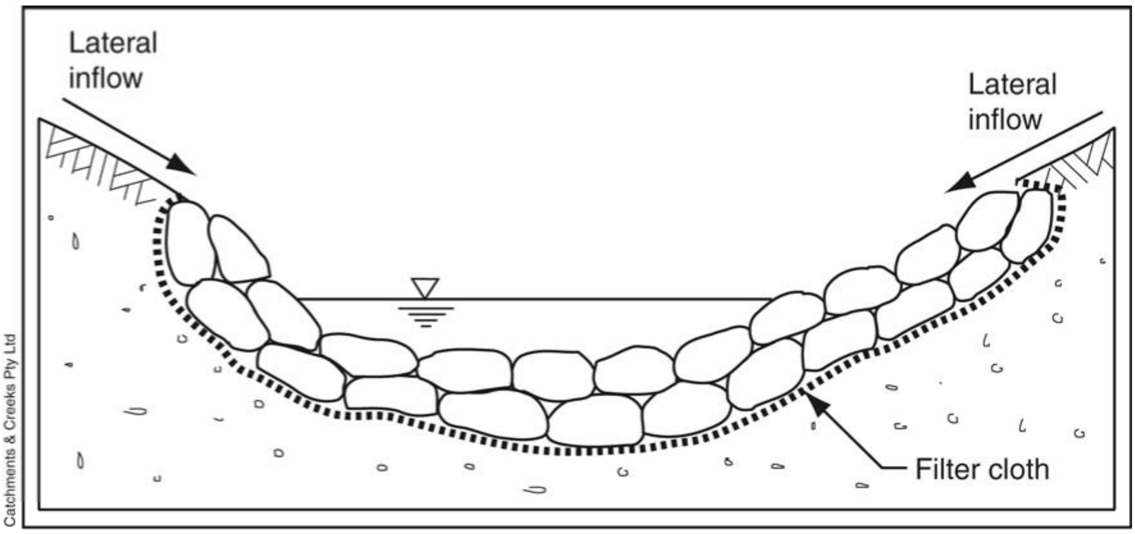


Figure 11 – Desired placement of rock within a catch drain such that lateral inflow has free entry into the drain

Hydraulic design of rock-lined catch drains (using the Rational Method approach):

Step 1 Choose the preferred rock size of the catch drain. This may be governed by the rock size readily available on-site, or based on past experience.

Step 2 Nominate the catch drain profile: parabolic or triangular (V-drain). Parabolic drains have a greater hydraulic capacity and are generally less susceptible to invert erosion. A triangular profile often best represents the profile of some table drains.

Step 3 Determine the required *Average Recurrence Interval* (ARI) of the design storm for the given catch drain (i.e. 1 year, 2 year, 5 year, etc. – refer to Table 4.3.1 in Chapter 4, or Table A1 in Section A2 of Appendix A). Note, if a locally adopted design standard exists, then the ARI must be determined from that standard.

Step 4 Determine the appropriate *time of concentration* (t_c) for the catch drain (refer to Step 4 in IECA 2008, Appendix A, Section A2).

If the time of concentration is unknown, then a conservative approach is to adopt a time of 5 minutes, then review later is necessary.

Step 5 Given the design storm ARI, and duration (t_c), determine the *Average Rainfall Intensity* (I, mm/hr) for the catch drain (refer to Step 6 in Section A2 of Appendix A).

To determine the average rainfall intensity it will be necessary to obtain the relevant *Intensity-Frequency-Duration* (IFD) chart for the given site location.

Step 6 Determine the *Coefficient of Discharge* (C) for the catchment contributing runoff to the catch drain (refer to Step 3 in IECA 2008, Appendix A, Section A2).

Note, it will be necessary to first determine the *Coefficient of Discharge* for a 10 year storm (C_{10}), and then the *Frequency Factor* (F_Y) for the nominated design storm frequency from Table A7 in Step 3, Section A2 of Appendix A, such that:

$$C = C_{10} \, . \, F_Y \leq 1.0$$

Step 7 Determine the *unit catchment area* (A*) of the catch drain. The unit catchment area is equal to the actual catchment area (A) times the coefficient of discharge (C).

Thus: $A^* = A.C$ (hectares)

Step 8 Given the design discharge (Q), or the rainfall intensity (I) and the unit catchment area (A*), determine the desired drain size and slope, and mean rock size (d_{50}) from Tables 45 to 53, or Tables 56 to 64 depending on the chosen drain profile. *It is noted that information provided in these tables are based on the use of angular, fractured rock with a specific gravity (s_r) of 2.4.*

Noting that: $Q(m^3/s) = (C.I.A)/360$

It is noted that in some cases the drain slope will be defined by site conditions.

If the slope of the drain varies along its length (which is often the case for table drains), then the catchment area may need to be determined at various locations along the length of the drain.

Step 9 If it is necessary to further analyse the catch drain, the allowable flow velocity (V) and Manning's roughness (n) for the catch drain can be determined from Tables 44 or 55, depending on the chosen drain profile.

The maximum channel slope presented in these tables represents the gradient beyond which the maximum allowable discharge is governed by the maximum allowable flow velocity. At lower gradient the maximum discharge for the drain is governed by the maximum allowable flow depth (Y).

Step 10 If necessary, the maximum allowable horizontal spacing of the catch drains down the slope can be determined from Table 3 (*Catch Drain, Part 1 – General information*).

Explanation of the design philosophy adopted within this fact sheet:

Given the cross-sectional dimensions of a given catch drain (A & R), its surface roughness (n), gradient (S), and required freeboard, it is possible (using Manning's equation) to determine the hydraulic capacity (Q) of the drain, as presented in Equation 1.

Manning's equation: $$Q = \frac{1}{n}.A.R^{2/3}.S^{1/2} \qquad \text{(Eqn 1)}$$

where: A = cross-sectional flow area of the catch drain

The Rational Method (Equation 2) can be rearrange to form Equation 3:

$$Q = (C.I.A)/360 \qquad \text{(Eqn 2)}$$

$$A.C = 360(Q / I) \qquad \text{(Eqn 3)}$$

where: A = catchment area (ha) of the catch drain (**not** the cross-sectional area of the drain)

If we define a new term called 'the unit catchment area' (A*) as the effective catchment area based on an **assumed** coefficient of discharge of unity (i.e. C = 1.0), then:

Maximum unit catchment area: $$A^* = 360(Q / I) \qquad \text{(Eqn 4)}$$

The relationship between flow velocity (V) and channel slope (S) is given by a modification of the Manning's equation (Equation 5):

$$V = \frac{1}{n}.R^{2/3}.S^{1/2} \qquad \text{(Eqn 5)}$$

For a given surface lining material we can determine the allowable flow velocity (V_{allow}). Therefore, for a given catch drain profile (represented by the hydraulic radius, R), and surface lining (represented by the Manning's roughness, n) we can determine the required drain slope (S) for a given allowable flow velocity. This information is presented in Tables 44 and 55. It is noted that at this channel slope, the maximum allowable flow velocity will be achieved when the channel is flowing at the maximum allowable flow depth (Y).

Also, for a given catch drain cross-sectional area (A), hydraulic radius (R), and maximum allowable flow velocity (V), we can determine the maximum allowable discharge (Q) from Equation 1. With this discharge, and the nominated design rainfall intensity (I), we can determine the maximum unit catchment area (A*) from Equation 4. This information is presented in Tables 45 to 53 for parabolic drains, and Tables 56 to 64 for drains with a triangular profile.

This means that Tables 45 to 53 and 56 to 64 are independent of location, and thus can be used anywhere in the world. Rainfall intensity, I (mm/hr) being the only parameter that is location specific.

The allowable flow velocities (V_{allow}) presented in Tables 44 and 55 are based on the use of angular, fractured rock with a specific gravity (s_r) of 2.4. Therefore, the maximum unit catchment areas presented in Tables 45 to 53 and 56 to 64 are conservative for rock with a specific gravity greater than 2.4, but are not conservative if rounded rock is used.

In order to determine the maximum allowable catchment area (A), it is necessary to determine the **actual** coefficient of discharge (C) for the adopted storm frequency (ARI), and catchment conditions (i.e. soil porosity). The maximum allowable catchment area (A) is determined from Equation 6.

Maximum allowable catchment area: $$A = A^*/C \qquad \text{(Eqn 6)}$$

Since the coefficient of discharge is always assumed to be less than or equal to unity, the maximum allowable catchment area (A) cannot exceed the maximum unit catchment area (A*).

If the actual catchment area is less than the calculated maximum catchment area (A) from Equation 6, then the catch drain can be constructed at a range of channel gradients (S):

$$S_{min} < S < S_{max}$$

where:

- S_{min} can be determined from Manning's equation based on the catch drain flowing full, but at a channel-full velocity less than the maximum allowable flow velocity;
- S_{max} can be determined from Manning's equation based on the catch drain flowing partially full, and at a velocity equal to the maximum allowable flow velocity.

For rock-lined channels the Manning's roughness can be determined from Equation 7.

$$n = \frac{(d_{90})^{1/6}}{26(1 - 0.3593^{(x)^{0.7}})} \qquad \text{(Eqn 7)}$$

where:
- X = $(R/d_{90})(d_{50}/d_{90})$
- R = Hydraulic radius of flow over rocks [m]
- d_{50} = mean rock size for which 50% of rocks are smaller [m]
- d_{90} = mean rock size for which 90% of rocks are smaller [m]

In 'natural' gravel-based streams the factor d_{50}/d_{90} is typically in the range 0.2 to 0.5, whereas in constructed channels in which imported graded rock is used, the ratio is more likely to be in the range 0.5 to 0.8. The results presented in Tables 44 to 53 and 55 to 64 are based on d_{50}/d_{90} = 0.67 which assumes that the maximum rock size is approximately equal to 1.5 times the mean rock size.

The allowable flow velocity for low gradient (i.e. S < 10%) rock-lined catch drains can be determined from Equation 8.

$$d_{50} = \frac{K_1 . V^2}{2.g.K^2(s_r - 1)} \qquad \text{(Eqn 8)}$$

where:
- V = average flow velocity [m/s]
- K_1 = 1.0 for angular (fractured) rock, or 1.36 for rounded rock
- g = acceleration due to gravity [m/s^2]
- K = 0.86 for highly turbulent flow as expected within a catch drain
- s_r = specific gravity of the rock (refer to Table 42)

Table 42 – Typical values of the specific gravity of rock

Rock type	Specific gravity of rock (s_r)
Sandstone	2.1 to 2.4
Granite	2.5 to 3.1 (typically 2.6)
Limestone	2.6
Basalt	2.7 to 3.2

Adopting the values, K_1 = 1.0, K = 0.86 and s_r = 2.4, Equation 8 can simplified to Equation 9:

$$d_{50} = 0.05\, V^2 \qquad \text{(Eqn 9)}$$

Note, Equations 8 and 9 are only applicable for small channels with a bed slope less than 10%.

Design example: Rock-lined catch drain

Design a long-term (> 24 months) rock-lined catch drain cut into a loam soil in Townsville with a catchment area of 2.0ha, and an average catchment land slope of 5%. The catch drain will be used to divert 'clean' water around a soil disturbance. The catchment consists of undisturbed, well-grassed, land, and the 'time of concentration' (t_c) for the catchment is known to be 20 minutes.

Step 1 A rock-lined drain has been nominated. A review of site conditions indicated that rock of mean (d_{50}) size of 150mm is readily available, and is thus chosen.

Step 2 Choose a parabolic drain profile.

Step 3 Nominate a 1 in 10 year ARI design storm from Table 4.3.1 (Chapter 4).

Step 4 The catchment time of concentration (t_c) is given as 20 minutes.

Step 5 Determine the average rainfall intensity: I = 134mm/hr for Townsville from Table A11 (Appendix A) for ARI = 10-year, and t_c = 20 minutes.

Step 6 Determine the coefficient of discharge (C_Y):

Given the catch drain's catchment area is open, undisturbed grass with medium permeability, 100% pervious surface area, and given that Townsville's 10 minute, 1-year rainfall intensity (${}^{1}I_{10}$) is 91.9mm/hr, the 10-year coefficient of discharge, C_{10} = 0.70 from Table A5 (Appendix A – *Construction site hydrology and hydraulics*).

Thus: $C = 0.7 \leq 1.0$ (OK)

also: $Q_{10} = (C.I.A)/360 = (0.7 \times 134 \times 2.0)/360 = 0.521 m^3/s$

Step 7 Calculate the unit catchment area (A*) for the catch drain:

$$A^* = A.C = 2.0 \times 0.7 = 1.4 ha$$

Step 8 Given the design discharge (Q) of $0.521 m^3/s$, a review Tables 45 to 50 indicates that it is unlikely that a Type-A or Type-B catch drain will have sufficient hydraulic capacity, thus a Type-C drain is chosen with dimensions: T = 3.0m, Y = 0.5m.

However, no design table is presented for a rock of size, d_{50} = 150mm. From Table 43 it can be determined that when flowing full (y = 0.5m), the hydraulic radius (R) is 0.310m, and the flow area (A) is $1.0m^2$. Also, from Table 44 the Manning's roughness (n) is 0.048, and the maximum allowable flow velocity (V_{allow}) of 1.73m/s is achieved at a channel gradient of 3.35%. At this channel gradient the hydraulic capacity of the drain (Q) can be determined as:

$$Q_{max} = V_{allow} . A = 1.73 \times 1.0 = 1.73 m^3/s \gg Q = 0.521 m^3/s$$

Thus the drain has more than adequate hydraulic capacity at a gradient of 3.35%

Step 9 Given the above results, it will be possible to construct a Type-C drain at a gradient less than 3.35%. Manning's equation can be used to determine the minimum gradient (S_{min}) for the rock-lined catch drain.

$$Q = \frac{1}{n} A.R^{2/3} S^{1/2} = \frac{1}{0.048} (1.0)(0.310)^{2/3} S^{1/2} = 0.521$$

therefore; $S_{min} = 0.00298 = 0.3\%$

Note, in the above equation, the term 'A' is the cross-sectional area of the catch drain at a depth of y = 0.5m (determined from Table 43), **not** the catchment area!

The **steepest** longitudinal gradient of the catch drain can also be determined from Manning's equation (Equation A16 in Appendix A); however, in this case the drain will be flowing partially full with a flow top width (T) less than 3.0m, and the flow depth (y) less than 0.5m. *(Note, the drain would still be constructed with the same standard overall physical dimensions specified for all Type-C catch drains.)*

Now, for a parabolic Type-C drain the numerical relationship between the flow top width (T) and the flow depth (y) is given by the following equation (Table 4):

$$y = 0.0556\,(T)^2$$

and the cross sectional area of flow (A) is given by (Table A30b, Appendix A):

$$A = 0.67(T.y) = 0.0371\,T^3 = Q/V = 0.521/1.73 = 0.301\text{m}^2$$

Therefore, the flow top width, T = 2.011m; the flow depth, y = 0.225m; and the hydraulic radius (R) can be determined from (Table A29b, Appendix A):

$$R = \frac{2\,T^2.\,y}{3\,T^2 + 8\,y^2} = \frac{2(2.011)^2 \times 0.225}{3(2.011)^2 + 8(0.225)^2} = 0.145\text{m}$$

The maximum catch drain slope is given by rearranging the Manning's equation:

$$S_{max} = 100 \text{ x } (V^2 \,.\, n^2)/R^{4/3} = 100 \text{ x } (1.73^2 \text{ x } 0.048^2)/0.145^{4/3} = 9.06\%$$

Therefore, the Type-C catch drain can be constructed at any longitudinal gradient between 0.3% (maximum flow depth) and 9% (maximum flow velocity), and still provide the required hydraulic capacity for the 1 in 10 year design storm. It is noted that constructing the drain at the steeper gradient will result in a construction site with maximum drainage capacity.

However, from the above analysis it was determined than a rock-lined drain with a gradient of 9% would achieve a flow of around $0.521\text{m}^3/\text{s}$ at a flow depth of 0.225m which is less than the maximum flow depth for a Type-B catch drain. This means that it now looks possible to construct a smaller Type-B catch drain rather than a Type-C drain.

Further analysis indicates that for a Type-B drain with dimensions (T = 1.8m, Y = 0.3m), mean rock size d_{50} = 150mm, and channel gradient of 9%, the following results can be achieved:

Manning's n = 0.061

Flow velocity = 1.60m/s < 1.73m/s OK

Peak discharge, Q_{max} = $0.577\text{m}^3/\text{s}$ < $0.521\text{m}^3/\text{s}$ OK

Thus a Type-B drain would be sufficient if a gradient of 9% can be achieved along its length, otherwise adopt a Type-C with a flatter channel gradient.

Table 43 – Dimensions of standard parabolic catch drains

Catch drain type	Max top width of flow (T)	Maximum flow depth (y)	Top width of formed drain [1]	Depth of formed drain	Hyd. rad. (R) at max flow depth	Area (A) at max flow depth
Type-A	1.0m	0.15m	1.6m	0.30m	0.094m	$0.100m^2$
Type-B	1.8m	0.30m	2.4m	0.45m	0.186m	$0.360m^2$
Type-C	3.0m	0.50m	3.6m	0.65m	0.310m	$1.000m^2$

[1] Top width of the formed drain assumes the upper bank slope is limited to a maximum of 2:1.

Table 44 – Hydraulic parameters of rock-lined, parabolic cross-section catch drains operating at maximum flow depth (Y)

Mean rock size, d_{50} (mm)	50	100	150	200	300
d_{90} (mm) [1]	75	150	225	300	450
Allowable flow velocity (m/s)	1.00	1.41	1.73	2.00	2.50
Type-A catch drain: width (T) = 1.0 m, depth (Y) = 0.15 m					
Manning's roughness 'n' [3]	0.042	0.066	0.088	0.108	0.148
Maximum channel slope (%)	4.09	[2]	[2]	[2]	[2]
Maximum channel slope (X:1)	24.4	[2]	[2]	[2]	[2]
Type-B catch drain: width (T) = 1.8 m, depth (Y) = 0.3 m					
Manning's roughness 'n' [3]	0.033	0.047	0.061	0.074	0.099
Maximum channel slope (%)	1.00	4.21	[2]	[2]	[2]
Maximum channel slope (X:1)	100.3	23.7	[2]	[2]	[2]
Type-C catch drain: width (T) = 3.0 m, depth (Y) = 0.5 m					
Manning's roughness 'n' [3]	0.029	0.039	0.048	0.058	0.075
Maximum channel slope (%)	0.39	1.43	3.35	6.32	[2]
Maximum channel slope (X:1)	258	70	30	15.8	[2]

[1] Based on a rock size distribution defined by d_{50}/d_{90} = 0.67, and specific gravity of 2.4

[2] Theoretical maximum channel slope exceeds 10% (1 in 10) for catch drain flowing at maximum allowable flow depth (Y).

[3] Manning roughness values (n) are only applicable to the drain flowing full, i.e. depth = Y.

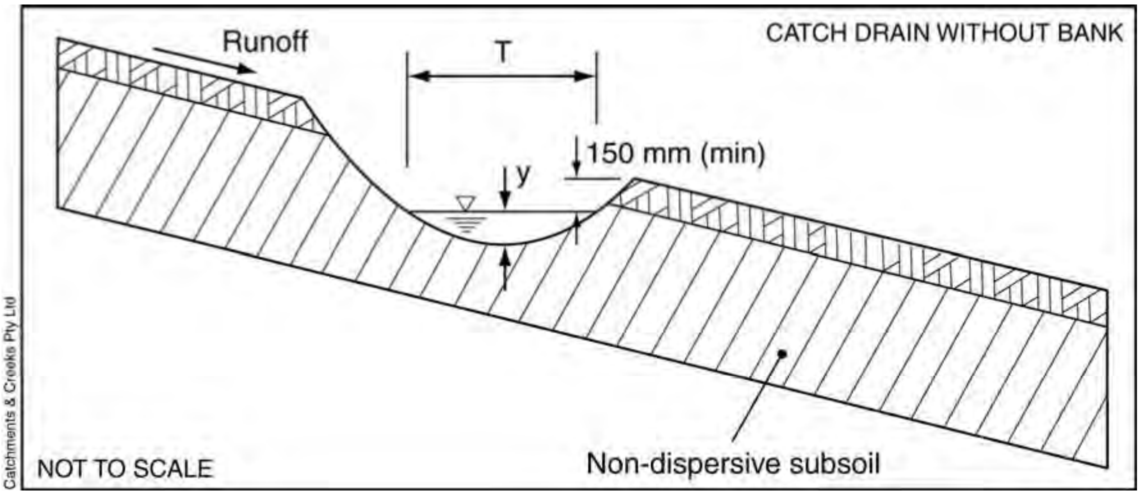


Figure 12(3) – Parabolic catch drain without bank

(Also refer to Part 1 of this fact sheet for diagram of parabolic catch drain with down-slope bank)

Table 45 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-A Catch Drain: Parabolic cross section										
Mean rock size (d_{50}) = 100 mm and s_r = 2.4					Variation in rock size (d_{50}/d_{90}) = 0.67					
Dimensions:				Flow top width = 1.0 m				Flow depth = 0.15 m		
Rainfall intensity (mm/hr)	Longitudinal slope of catch drain (%)									
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
20	0.569	0.804	0.985	1.137	1.271	1.393	1.504	1.608	1.706	1.798
25	0.455	0.643	0.788	0.910	1.017	1.114	1.204	1.287	1.365	1.438
30	0.379	0.536	0.657	0.758	0.848	0.929	1.003	1.072	1.137	1.199
35	0.325	0.460	0.563	0.650	0.727	0.796	0.860	0.919	0.975	1.027
40	0.284	0.402	0.492	0.569	0.636	0.696	0.752	0.804	0.853	0.899
45	0.253	0.357	0.438	0.505	0.565	0.619	0.669	0.715	0.758	0.799
50	0.227	0.322	0.394	0.455	0.509	0.557	0.602	0.643	0.682	0.719
55	0.207	0.292	0.358	0.414	0.462	0.506	0.547	0.585	0.620	0.654
60	0.190	0.268	0.328	0.379	0.424	0.464	0.501	0.536	0.569	0.599
65	0.175	0.247	0.303	0.350	0.391	0.429	0.463	0.495	0.525	0.553
70	0.162	0.230	0.281	0.325	0.363	0.398	0.430	0.460	0.487	0.514
75	0.152	0.214	0.263	0.303	0.339	0.371	0.401	0.429	0.455	0.479
80	0.142	0.201	0.246	0.284	0.318	0.348	0.376	0.402	0.426	0.450
85	0.134	0.189	0.232	0.268	0.299	0.328	0.354	0.378	0.401	0.423
90	0.126	0.179	0.219	0.253	0.283	0.310	0.334	0.357	0.379	0.400
95	0.120	0.169	0.207	0.239	0.268	0.293	0.317	0.339	0.359	0.379
100	0.114	0.161	0.197	0.227	0.254	0.279	0.301	0.322	0.341	0.360
105	0.108	0.153	0.188	0.217	0.242	0.265	0.287	0.306	0.325	0.342
110	0.103	0.146	0.179	0.207	0.231	0.253	0.274	0.292	0.310	0.327
115	0.099	0.140	0.171	0.198	0.221	0.242	0.262	0.280	0.297	0.313
120	0.095	0.134	0.164	0.190	0.212	0.232	0.251	0.268	0.284	0.300
125	0.091	0.129	0.158	0.182	0.203	0.223	0.241	0.257	0.273	0.288
130	0.087	0.124	0.152	0.175	0.196	0.214	0.231	0.247	0.262	0.277
135	0.084	0.119	0.146	0.168	0.188	0.206	0.223	0.238	0.253	0.266
140	0.081	0.115	0.141	0.162	0.182	0.199	0.215	0.230	0.244	0.257
145	0.078	0.111	0.136	0.157	0.175	0.192	0.208	0.222	0.235	0.248
150	0.076	0.107	0.131	0.152	0.170	0.186	0.201	0.214	0.227	0.240
155	0.073	0.104	0.127	0.147	0.164	0.180	0.194	0.208	0.220	0.232
160	0.071	0.101	0.123	0.142	0.159	0.174	0.188	0.201	0.213	0.225
165	0.069	0.097	0.119	0.138	0.154	0.169	0.182	0.195	0.207	0.218
170	0.067	0.095	0.116	0.134	0.150	0.164	0.177	0.189	0.201	0.212
175	0.065	0.092	0.113	0.130	0.145	0.159	0.172	0.184	0.195	0.205
180	0.063	0.089	0.109	0.126	0.141	0.155	0.167	0.179	0.190	0.200
185	0.061	0.087	0.106	0.123	0.137	0.151	0.163	0.174	0.184	0.194
190	0.060	0.085	0.104	0.120	0.134	0.147	0.158	0.169	0.180	0.189
200	0.057	0.080	0.098	0.114	0.127	0.139	0.150	0.161	0.171	0.180
210	0.054	0.077	0.094	0.108	0.121	0.133	0.143	0.153	0.162	0.171
220	0.052	0.073	0.090	0.103	0.116	0.127	0.137	0.146	0.155	0.163
230	0.049	0.070	0.086	0.099	0.111	0.121	0.131	0.140	0.148	0.156
240	0.047	0.067	0.082	0.095	0.106	0.116	0.125	0.134	0.142	0.150
250	0.045	0.064	0.079	0.091	0.102	0.111	0.120	0.129	0.136	0.144
Q (m^3/s)	**0.032**	**0.045**	**0.055**	**0.063**	**0.071**	**0.077**	**0.084**	**0.089**	**0.095**	**0.100**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 44).

Table 46 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-A Catch Drain: Parabolic cross section										
Mean rock size (d_{50}) = 200 mm and s_r = 2.4					Variation in rock size (d_{50}/d_{90}) = 0.67					
Dimensions:				Flow top width = 1.0 m				Flow depth = 0.15 m		
Rainfall intensity (mm/hr)	Longitudinal slope of catch drain (%)									
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
20	0.344	0.487	0.596	0.689	0.770	0.843	0.911	0.974	1.033	1.089
25	0.275	0.390	0.477	0.551	0.616	0.675	0.729	0.779	0.826	0.871
30	0.230	0.325	0.398	0.459	0.513	0.562	0.607	0.649	0.689	0.726
35	0.197	0.278	0.341	0.394	0.440	0.482	0.521	0.557	0.590	0.622
40	0.172	0.243	0.298	0.344	0.385	0.422	0.456	0.487	0.516	0.544
45	0.153	0.216	0.265	0.306	0.342	0.375	0.405	0.433	0.459	0.484
50	0.138	0.195	0.239	0.275	0.308	0.337	0.364	0.390	0.413	0.436
55	0.125	0.177	0.217	0.250	0.280	0.307	0.331	0.354	0.376	0.396
60	0.115	0.162	0.199	0.230	0.257	0.281	0.304	0.325	0.344	0.363
65	0.106	0.150	0.184	0.212	0.237	0.260	0.280	0.300	0.318	0.335
70	0.098	0.139	0.170	0.197	0.220	0.241	0.260	0.278	0.295	0.311
75	0.092	0.130	0.159	0.184	0.205	0.225	0.243	0.260	0.275	0.290
80	0.086	0.122	0.149	0.172	0.192	0.211	0.228	0.243	0.258	0.272
85	0.081	0.115	0.140	0.162	0.181	0.198	0.214	0.229	0.243	0.256
90	0.077	0.108	0.133	0.153	0.171	0.187	0.202	0.216	0.230	0.242
95	0.072	0.103	0.126	0.145	0.162	0.178	0.192	0.205	0.217	0.229
100	0.069	0.097	0.119	0.138	0.154	0.169	0.182	0.195	0.207	0.218
105	0.066	0.093	0.114	0.131	0.147	0.161	0.174	0.186	0.197	0.207
110	0.063	0.089	0.108	0.125	0.140	0.153	0.166	0.177	0.188	0.198
115	0.060	0.085	0.104	0.120	0.134	0.147	0.158	0.169	0.180	0.189
120	0.057	0.081	0.099	0.115	0.128	0.141	0.152	0.162	0.172	0.181
125	0.055	0.078	0.095	0.110	0.123	0.135	0.146	0.156	0.165	0.174
130	0.053	0.075	0.092	0.106	0.118	0.130	0.140	0.150	0.159	0.168
135	0.051	0.072	0.088	0.102	0.114	0.125	0.135	0.144	0.153	0.161
140	0.049	0.070	0.085	0.098	0.110	0.120	0.130	0.139	0.148	0.156
145	0.047	0.067	0.082	0.095	0.106	0.116	0.126	0.134	0.142	0.150
150	0.046	0.065	0.080	0.092	0.103	0.112	0.121	0.130	0.138	0.145
155	0.044	0.063	0.077	0.089	0.099	0.109	0.118	0.126	0.133	0.141
160	0.043	0.061	0.075	0.086	0.096	0.105	0.114	0.122	0.129	0.136
165	0.042	0.059	0.072	0.083	0.093	0.102	0.110	0.118	0.125	0.132
170	0.041	0.057	0.070	0.081	0.091	0.099	0.107	0.115	0.122	0.128
175	0.039	0.056	0.068	0.079	0.088	0.096	0.104	0.111	0.118	0.124
180	0.038	0.054	0.066	0.077	0.086	0.094	0.101	0.108	0.115	0.121
185	0.037	0.053	0.064	0.074	0.083	0.091	0.098	0.105	0.112	0.118
190	0.036	0.051	0.063	0.072	0.081	0.089	0.096	0.103	0.109	0.115
200	0.034	0.049	0.060	0.069	0.077	0.084	0.091	0.097	0.103	0.109
210	0.033	0.046	0.057	0.066	0.073	0.080	0.087	0.093	0.098	0.104
220	0.031	0.044	0.054	0.063	0.070	0.077	0.083	0.089	0.094	0.099
230	0.030	0.042	0.052	0.060	0.067	0.073	0.079	0.085	0.090	0.095
240	0.029	0.041	0.050	0.057	0.064	0.070	0.076	0.081	0.086	0.091
250	0.028	0.039	0.048	0.055	0.062	0.067	0.073	0.078	0.083	0.087
Q (m^3/s)	**0.019**	**0.027**	**0.033**	**0.038**	**0.043**	**0.047**	**0.051**	**0.054**	**0.057**	**0.060**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 44).

Table 47 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-A Catch Drain: Parabolic cross section										
Mean rock size (d_{50}) = 300 mm and s_r = 2.4					Variation in rock size (d_{50}/d_{90}) = 0.67					
Dimensions:				Flow top width = 1.0 m				Flow depth = 0.15 m		
Rainfall intensity (mm/hr)	Longitudinal slope of catch drain (%)									
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
20	0.252	0.357	0.437	0.505	0.564	0.618	0.667	0.714	0.757	0.798
25	0.202	0.285	0.350	0.404	0.451	0.494	0.534	0.571	0.605	0.638
30	0.168	0.238	0.291	0.336	0.376	0.412	0.445	0.476	0.505	0.532
35	0.144	0.204	0.250	0.288	0.322	0.353	0.381	0.408	0.432	0.456
40	0.126	0.178	0.218	0.252	0.282	0.309	0.334	0.357	0.378	0.399
45	0.112	0.159	0.194	0.224	0.251	0.275	0.297	0.317	0.336	0.355
50	0.101	0.143	0.175	0.202	0.226	0.247	0.267	0.285	0.303	0.319
55	0.092	0.130	0.159	0.183	0.205	0.225	0.243	0.259	0.275	0.290
60	0.084	0.119	0.146	0.168	0.188	0.206	0.222	0.238	0.252	0.266
65	0.078	0.110	0.134	0.155	0.174	0.190	0.205	0.220	0.233	0.245
70	0.072	0.102	0.125	0.144	0.161	0.177	0.191	0.204	0.216	0.228
75	0.067	0.095	0.117	0.135	0.150	0.165	0.178	0.190	0.202	0.213
80	0.063	0.089	0.109	0.126	0.141	0.154	0.167	0.178	0.189	0.199
85	0.059	0.084	0.103	0.119	0.133	0.145	0.157	0.168	0.178	0.188
90	0.056	0.079	0.097	0.112	0.125	0.137	0.148	0.159	0.168	0.177
95	0.053	0.075	0.092	0.106	0.119	0.130	0.141	0.150	0.159	0.168
100	0.050	0.071	0.087	0.101	0.113	0.124	0.133	0.143	0.151	0.160
105	0.048	0.068	0.083	0.096	0.107	0.118	0.127	0.136	0.144	0.152
110	0.046	0.065	0.079	0.092	0.103	0.112	0.121	0.130	0.138	0.145
115	0.044	0.062	0.076	0.088	0.098	0.107	0.116	0.124	0.132	0.139
120	0.042	0.059	0.073	0.084	0.094	0.103	0.111	0.119	0.126	0.133
125	0.040	0.057	0.070	0.081	0.090	0.099	0.107	0.114	0.121	0.128
130	0.039	0.055	0.067	0.078	0.087	0.095	0.103	0.110	0.116	0.123
135	0.037	0.053	0.065	0.075	0.084	0.092	0.099	0.106	0.112	0.118
140	0.036	0.051	0.062	0.072	0.081	0.088	0.095	0.102	0.108	0.114
145	0.035	0.049	0.060	0.070	0.078	0.085	0.092	0.098	0.104	0.110
150	0.034	0.048	0.058	0.067	0.075	0.082	0.089	0.095	0.101	0.106
155	0.033	0.046	0.056	0.065	0.073	0.080	0.086	0.092	0.098	0.103
160	0.032	0.045	0.055	0.063	0.071	0.077	0.083	0.089	0.095	0.100
165	0.031	0.043	0.053	0.061	0.068	0.075	0.081	0.086	0.092	0.097
170	0.030	0.042	0.051	0.059	0.066	0.073	0.079	0.084	0.089	0.094
175	0.029	0.041	0.050	0.058	0.064	0.071	0.076	0.082	0.086	0.091
180	0.028	0.040	0.049	0.056	0.063	0.069	0.074	0.079	0.084	0.089
185	0.027	0.039	0.047	0.055	0.061	0.067	0.072	0.077	0.082	0.086
190	0.027	0.038	0.046	0.053	0.059	0.065	0.070	0.075	0.080	0.084
200	0.025	0.036	0.044	0.050	0.056	0.062	0.067	0.071	0.076	0.080
210	0.024	0.034	0.042	0.048	0.054	0.059	0.064	0.068	0.072	0.076
220	0.023	0.032	0.040	0.046	0.051	0.056	0.061	0.065	0.069	0.073
230	0.022	0.031	0.038	0.044	0.049	0.054	0.058	0.062	0.066	0.069
240	0.021	0.030	0.036	0.042	0.047	0.051	0.056	0.059	0.063	0.066
250	0.020	0.029	0.035	0.040	0.045	0.049	0.053	0.057	0.061	0.064
Q (m^3/s)	**0.014**	**0.020**	**0.024**	**0.028**	**0.031**	**0.034**	**0.037**	**0.040**	**0.042**	**0.044**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 44).

Table 48 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-B Catch Drain: Parabolic cross section										
Mean rock size (d_{50}) = 100 mm and s_r = 2.4					Variation in rock size (d_{50}/d_{90}) = 0.67					
Dimensions:				Flow top width = 1.8 m				Flow depth = 0.3 m		
Rainfall intensity (mm/hr)	Longitudinal slope of catch drain (%)									
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
15	5.953	8.418	10.310	11.905	Allowable unit catchment areas presented in the columns below are limited by the allowable flow velocity					
20	4.465	6.314	7.733	8.929						
25	3.572	5.051	6.186	7.143						
30	2.976	4.209	5.155	5.953	5.389	4.729	4.227	3.858	3.585	3.330
35	2.551	3.608	4.419	5.102	4.619	4.053	3.623	3.307	3.073	2.854
40	2.232	3.157	3.866	4.465	4.041	3.547	3.170	2.893	2.689	2.497
45	1.984	2.806	3.437	3.968	3.592	3.153	2.818	2.572	2.390	2.220
50	1.786	2.526	3.093	3.572	3.233	2.837	2.536	2.315	2.151	1.998
55	1.623	2.296	2.812	3.247	2.939	2.579	2.306	2.104	1.956	1.816
60	1.488	2.105	2.578	2.976	2.694	2.364	2.114	1.929	1.793	1.665
65	1.374	1.943	2.379	2.747	2.487	2.183	1.951	1.780	1.655	1.537
70	1.276	1.804	2.209	2.551	2.309	2.027	1.812	1.653	1.537	1.427
75	1.191	1.684	2.062	2.381	2.155	1.892	1.691	1.543	1.434	1.332
80	1.116	1.578	1.933	2.232	2.021	1.773	1.585	1.447	1.345	1.249
85	1.050	1.486	1.819	2.101	1.902	1.669	1.492	1.362	1.265	1.175
90	0.992	1.403	1.718	1.984	1.796	1.576	1.409	1.286	1.195	1.110
95	0.940	1.329	1.628	1.880	1.702	1.493	1.335	1.218	1.132	1.052
100	0.893	1.263	1.547	1.786	1.617	1.419	1.268	1.157	1.076	0.999
105	0.850	1.203	1.473	1.701	1.540	1.351	1.208	1.102	1.024	0.951
110	0.812	1.148	1.406	1.623	1.470	1.290	1.153	1.052	0.978	0.908
115	0.776	1.098	1.345	1.553	1.406	1.234	1.103	1.006	0.935	0.869
120	0.744	1.052	1.289	1.488	1.347	1.182	1.057	0.964	0.896	0.832
125	0.714	1.010	1.237	1.429	1.293	1.135	1.015	0.926	0.861	0.799
130	0.687	0.971	1.190	1.374	1.244	1.091	0.976	0.890	0.827	0.768
135	0.661	0.935	1.146	1.323	1.197	1.051	0.939	0.857	0.797	0.740
140	0.638	0.902	1.105	1.276	1.155	1.013	0.906	0.827	0.768	0.714
145	0.616	0.871	1.067	1.232	1.115	0.978	0.875	0.798	0.742	0.689
150	0.595	0.842	1.031	1.191	1.078	0.946	0.845	0.772	0.717	0.666
155	0.576	0.815	0.998	1.152	1.043	0.915	0.818	0.747	0.694	0.645
160	0.558	0.789	0.967	1.116	1.010	0.887	0.793	0.723	0.672	0.624
165	0.541	0.765	0.937	1.082	0.980	0.860	0.769	0.701	0.652	0.605
170	0.525	0.743	0.910	1.050	0.951	0.835	0.746	0.681	0.633	0.588
175	0.510	0.722	0.884	1.020	0.924	0.811	0.725	0.661	0.615	0.571
180	0.496	0.702	0.859	0.992	0.898	0.788	0.705	0.643	0.598	0.555
185	0.483	0.683	0.836	0.965	0.874	0.767	0.685	0.626	0.581	0.540
190	0.470	0.665	0.814	0.940	0.851	0.747	0.667	0.609	0.566	0.526
200	0.446	0.631	0.773	0.893	0.808	0.709	0.634	0.579	0.538	0.499
210	0.425	0.601	0.736	0.850	0.770	0.676	0.604	0.551	0.512	0.476
220	0.406	0.574	0.703	0.812	0.735	0.645	0.576	0.526	0.489	0.454
230	0.388	0.549	0.672	0.776	0.703	0.617	0.551	0.503	0.468	0.434
240	0.372	0.526	0.644	0.744	0.674	0.591	0.528	0.482	0.448	0.416
Q (m^3/s)	**0.248**	**0.351**	**0.430**	**0.496**	**0.449**	**0.394**	**0.352**	**0.321**	**0.299**	**0.277**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 44).

Table 49 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-B Catch Drain: Parabolic cross section										
Mean rock size (d_{50}) = 200 mm and s_r = 2.4						Variation in rock size (d_{50}/d_{90}) = 0.67				
Dimensions:				Flow top width = 1.8 m				Flow depth = 0.3 m		
Rainfall intensity (mm/hr)	Longitudinal slope of catch drain (%)									
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
20	2.850	4.030	4.936	5.700	6.372	6.981	7.540	8.060	8.549	9.012
25	2.280	3.224	3.949	4.560	5.098	5.584	6.032	6.448	6.840	7.210
30	1.900	2.687	3.291	3.800	4.248	4.654	5.027	5.374	5.700	6.008
35	1.628	2.303	2.821	3.257	3.641	3.989	4.309	4.606	4.885	5.150
40	1.425	2.015	2.468	2.850	3.186	3.490	3.770	4.030	4.275	4.506
45	1.267	1.791	2.194	2.533	2.832	3.102	3.351	3.582	3.800	4.005
50	1.140	1.612	1.974	2.280	2.549	2.792	3.016	3.224	3.420	3.605
55	1.036	1.466	1.795	2.073	2.317	2.538	2.742	2.931	3.109	3.277
60	0.950	1.343	1.645	1.900	2.124	2.327	2.513	2.687	2.850	3.004
65	0.877	1.240	1.519	1.754	1.961	2.148	2.320	2.480	2.631	2.773
70	0.814	1.151	1.410	1.628	1.821	1.994	2.154	2.303	2.443	2.575
75	0.760	1.075	1.316	1.520	1.699	1.861	2.011	2.149	2.280	2.403
80	0.712	1.008	1.234	1.425	1.593	1.745	1.885	2.015	2.137	2.253
85	0.671	0.948	1.161	1.341	1.499	1.642	1.774	1.897	2.012	2.120
90	0.633	0.896	1.097	1.267	1.416	1.551	1.676	1.791	1.900	2.003
95	0.600	0.848	1.039	1.200	1.342	1.470	1.587	1.697	1.800	1.897
100	0.570	0.806	0.987	1.140	1.274	1.396	1.508	1.612	1.710	1.802
105	0.543	0.768	0.940	1.086	1.214	1.330	1.436	1.535	1.628	1.717
110	0.518	0.733	0.897	1.036	1.159	1.269	1.371	1.466	1.554	1.639
115	0.496	0.701	0.858	0.991	1.108	1.214	1.311	1.402	1.487	1.567
120	0.475	0.672	0.823	0.950	1.062	1.163	1.257	1.343	1.425	1.502
125	0.456	0.645	0.790	0.912	1.020	1.117	1.206	1.290	1.368	1.442
130	0.438	0.620	0.759	0.877	0.980	1.074	1.160	1.240	1.315	1.386
135	0.422	0.597	0.731	0.844	0.944	1.034	1.117	1.194	1.267	1.335
140	0.407	0.576	0.705	0.814	0.910	0.997	1.077	1.151	1.221	1.287
145	0.393	0.556	0.681	0.786	0.879	0.963	1.040	1.112	1.179	1.243
150	0.380	0.537	0.658	0.760	0.850	0.931	1.005	1.075	1.140	1.202
155	0.368	0.520	0.637	0.735	0.822	0.901	0.973	1.040	1.103	1.163
160	0.356	0.504	0.617	0.712	0.797	0.873	0.942	1.008	1.069	1.126
165	0.345	0.489	0.598	0.691	0.772	0.846	0.914	0.977	1.036	1.092
170	0.335	0.474	0.581	0.671	0.750	0.821	0.887	0.948	1.006	1.060
175	0.326	0.461	0.564	0.651	0.728	0.798	0.862	0.921	0.977	1.030
180	0.317	0.448	0.548	0.633	0.708	0.776	0.838	0.896	0.950	1.001
185	0.308	0.436	0.534	0.616	0.689	0.755	0.815	0.871	0.924	0.974
190	0.300	0.424	0.520	0.600	0.671	0.735	0.794	0.848	0.900	0.949
200	0.285	0.403	0.494	0.570	0.637	0.698	0.754	0.806	0.855	0.901
210	0.271	0.384	0.470	0.543	0.607	0.665	0.718	0.768	0.814	0.858
220	0.259	0.366	0.449	0.518	0.579	0.635	0.685	0.733	0.777	0.819
230	0.248	0.350	0.429	0.496	0.554	0.607	0.656	0.701	0.743	0.784
240	0.237	0.336	0.411	0.475	0.531	0.582	0.628	0.672	0.712	0.751
250	0.228	0.322	0.395	0.456	0.510	0.558	0.603	0.645	0.684	0.721
Q (m^3/s)	**0.158**	**0.224**	**0.274**	**0.317**	**0.354**	**0.388**	**0.419**	**0.448**	**0.475**	**0.501**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 44).

Table 50 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-B Catch Drain: Parabolic cross section										
Mean rock size (d_{50}) = 300 mm and s_r = 2.4					Variation in rock size (d_{50}/d_{90}) = 0.67					
Dimensions:				Flow top width = 1.8 m				Flow depth = 0.3 m		
Rainfall intensity (mm/hr)	Longitudinal slope of catch drain (%)									
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
20	2.135	3.020	3.698	4.271	4.775	5.230	5.649	6.040	6.406	6.752
25	1.708	2.416	2.959	3.416	3.820	4.184	4.520	4.832	5.125	5.402
30	1.424	2.013	2.466	2.847	3.183	3.487	3.766	4.026	4.271	4.502
35	1.220	1.726	2.113	2.440	2.728	2.989	3.228	3.451	3.661	3.859
40	1.068	1.510	1.849	2.135	2.387	2.615	2.825	3.020	3.203	3.376
45	0.949	1.342	1.644	1.898	2.122	2.325	2.511	2.684	2.847	3.001
50	0.854	1.208	1.479	1.708	1.910	2.092	2.260	2.416	2.562	2.701
55	0.776	1.098	1.345	1.553	1.736	1.902	2.054	2.196	2.329	2.455
60	0.712	1.007	1.233	1.424	1.592	1.743	1.883	2.013	2.135	2.251
65	0.657	0.929	1.138	1.314	1.469	1.609	1.738	1.858	1.971	2.078
70	0.610	0.863	1.057	1.220	1.364	1.494	1.614	1.726	1.830	1.929
75	0.569	0.805	0.986	1.139	1.273	1.395	1.507	1.611	1.708	1.801
80	0.534	0.755	0.925	1.068	1.194	1.308	1.412	1.510	1.601	1.688
85	0.502	0.711	0.870	1.005	1.123	1.231	1.329	1.421	1.507	1.589
90	0.475	0.671	0.822	0.949	1.061	1.162	1.255	1.342	1.424	1.501
95	0.450	0.636	0.779	0.899	1.005	1.101	1.189	1.271	1.349	1.422
100	0.427	0.604	0.740	0.854	0.955	1.046	1.130	1.208	1.281	1.350
105	0.407	0.575	0.704	0.813	0.909	0.996	1.076	1.150	1.220	1.286
110	0.388	0.549	0.672	0.776	0.868	0.951	1.027	1.098	1.165	1.228
115	0.371	0.525	0.643	0.743	0.830	0.910	0.983	1.050	1.114	1.174
120	0.356	0.503	0.616	0.712	0.796	0.872	0.942	1.007	1.068	1.125
125	0.342	0.483	0.592	0.683	0.764	0.837	0.904	0.966	1.025	1.080
130	0.329	0.465	0.569	0.657	0.735	0.805	0.869	0.929	0.986	1.039
135	0.316	0.447	0.548	0.633	0.707	0.775	0.837	0.895	0.949	1.000
140	0.305	0.431	0.528	0.610	0.682	0.747	0.807	0.863	0.915	0.965
145	0.295	0.417	0.510	0.589	0.659	0.721	0.779	0.833	0.884	0.931
150	0.285	0.403	0.493	0.569	0.637	0.697	0.753	0.805	0.854	0.900
155	0.276	0.390	0.477	0.551	0.616	0.675	0.729	0.779	0.827	0.871
160	0.267	0.377	0.462	0.534	0.597	0.654	0.706	0.755	0.801	0.844
165	0.259	0.366	0.448	0.518	0.579	0.634	0.685	0.732	0.776	0.818
170	0.251	0.355	0.435	0.502	0.562	0.615	0.665	0.711	0.754	0.794
175	0.244	0.345	0.423	0.488	0.546	0.598	0.646	0.690	0.732	0.772
180	0.237	0.336	0.411	0.475	0.531	0.581	0.628	0.671	0.712	0.750
185	0.231	0.326	0.400	0.462	0.516	0.565	0.611	0.653	0.693	0.730
190	0.225	0.318	0.389	0.450	0.503	0.551	0.595	0.636	0.674	0.711
200	0.214	0.302	0.370	0.427	0.477	0.523	0.565	0.604	0.641	0.675
210	0.203	0.288	0.352	0.407	0.455	0.498	0.538	0.575	0.610	0.643
220	0.194	0.275	0.336	0.388	0.434	0.475	0.514	0.549	0.582	0.614
230	0.186	0.263	0.322	0.371	0.415	0.455	0.491	0.525	0.557	0.587
240	0.178	0.252	0.308	0.356	0.398	0.436	0.471	0.503	0.534	0.563
250	0.171	0.242	0.296	0.342	0.382	0.418	0.452	0.483	0.512	0.540
Q (m^3/s)	**0.119**	**0.168**	**0.205**	**0.237**	**0.265**	**0.291**	**0.314**	**0.336**	**0.356**	**0.375**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 44).

Table 51 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-C Catch Drain: Parabolic cross section

Mean rock size (d_{50}) = 100 mm and s_r = 2.4	Variation in rock size (d_{50}/d_{90}) = 0.67

Dimensions:	Flow top width = 3.0 m	Flow depth = 0.5 m

Rainfall intensity (mm/hr)	Longitudinal slope of catch drain (%)									
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
15	28.36	Allowable unit catchment areas presented in the columns below are limited by the allowable flow velocity								
20	21.27									
25	17.02	15.66	11.56	9.34	8.04	7.07	6.35	5.83	5.33	5.00
30	14.18	13.05	9.63	7.79	6.70	5.89	5.29	4.86	4.45	4.17
35	12.16	11.19	8.26	6.67	5.74	5.05	4.53	4.17	3.81	3.57
40	10.64	9.79	7.23	5.84	5.02	4.42	3.97	3.65	3.33	3.12
45	9.45	8.70	6.42	5.19	4.47	3.93	3.53	3.24	2.96	2.78
50	8.51	7.83	5.78	4.67	4.02	3.53	3.17	2.92	2.67	2.50
55	7.74	7.12	5.25	4.25	3.65	3.21	2.89	2.65	2.42	2.27
60	7.09	6.53	4.82	3.89	3.35	2.95	2.65	2.43	2.22	2.08
65	6.55	6.02	4.45	3.59	3.09	2.72	2.44	2.24	2.05	1.92
70	6.08	5.59	4.13	3.34	2.87	2.52	2.27	2.08	1.91	1.79
75	5.67	5.22	3.85	3.11	2.68	2.36	2.12	1.94	1.78	1.67
80	5.32	4.90	3.61	2.92	2.51	2.21	1.98	1.82	1.67	1.56
85	5.01	4.61	3.40	2.75	2.36	2.08	1.87	1.72	1.57	1.47
90	4.73	4.35	3.21	2.60	2.23	1.96	1.76	1.62	1.48	1.39
95	4.48	4.12	3.04	2.46	2.12	1.86	1.67	1.54	1.40	1.32
100	4.25	3.92	2.89	2.34	2.01	1.77	1.59	1.46	1.33	1.25
105	4.05	3.73	2.75	2.22	1.91	1.68	1.51	1.39	1.27	1.19
110	3.87	3.56	2.63	2.12	1.83	1.61	1.44	1.33	1.21	1.14
115	3.70	3.41	2.51	2.03	1.75	1.54	1.38	1.27	1.16	1.09
120	3.55	3.26	2.41	1.95	1.67	1.47	1.32	1.22	1.11	1.04
125	3.40	3.13	2.31	1.87	1.61	1.41	1.27	1.17	1.07	1.00
130	3.27	3.01	2.22	1.80	1.55	1.36	1.22	1.12	1.03	0.96
135	3.15	2.90	2.14	1.73	1.49	1.31	1.18	1.08	0.99	0.93
140	3.04	2.80	2.06	1.67	1.44	1.26	1.13	1.04	0.95	0.89
145	2.93	2.70	1.99	1.61	1.39	1.22	1.09	1.01	0.92	0.86
150	2.84	2.61	1.93	1.56	1.34	1.18	1.06	0.97	0.89	0.83
155	2.74	2.53	1.86	1.51	1.30	1.14	1.02	0.94	0.86	0.81
160	2.66	2.45	1.81	1.46	1.26	1.10	0.99	0.91	0.83	0.78
165	2.58	2.37	1.75	1.42	1.22	1.07	0.96	0.88	0.81	0.76
170	2.50	2.30	1.70	1.37	1.18	1.04	0.93	0.86	0.78	0.74
175	2.43	2.24	1.65	1.33	1.15	1.01	0.91	0.83	0.76	0.71
180	2.36	2.18	1.61	1.30	1.12	0.98	0.88	0.81	0.74	0.69
185	2.30	2.12	1.56	1.26	1.09	0.96	0.86	0.79	0.72	0.68
190	2.24	2.06	1.52	1.23	1.06	0.93	0.84	0.77	0.70	0.66
200	2.13	1.96	1.45	1.17	1.00	0.88	0.79	0.73	0.67	0.62
210	2.03	1.86	1.38	1.11	0.96	0.84	0.76	0.69	0.64	0.60
220	1.93	1.78	1.31	1.06	0.91	0.80	0.72	0.66	0.61	0.57
230	1.85	1.70	1.26	1.02	0.87	0.77	0.69	0.63	0.58	0.54
240	1.77	1.63	1.20	0.97	0.84	0.74	0.66	0.61	0.56	0.52
Q (m^3/s)	1.182	1.088	0.803	0.649	0.558	0.491	0.441	0.405	0.370	0.347

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 44).

Table 52 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-C Catch Drain: Parabolic cross section										
Mean rock size (d_{50}) = 200 mm and s_r = 2.4						**Variation in rock size (d_{50}/d_{90}) = 0.67**				
Dimensions:				**Flow top width = 3.0 m**				**Flow depth = 0.5 m**		
Rainfall intensity (mm/hr)	**Longitudinal slope of catch drain (%)**									
	1.0	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**
15	19.09	27.00	33.07	38.18	42.69	46.76	Allowable unit catchment areas presented in the columns below are limited by the allowable flow velocity			
20	14.32	20.25	24.80	28.64	32.02	35.07				
25	11.45	16.20	19.84	22.91	25.61	28.06				
30	9.55	13.50	16.53	19.09	21.34	23.38				
35	8.18	11.57	14.17	16.36	18.29	20.04	19.08	17.36	16.07	14.86
40	7.16	10.12	12.40	14.32	16.01	17.54	16.70	15.19	14.07	13.00
45	6.36	9.00	11.02	12.73	14.23	15.59	14.84	13.50	12.50	11.56
50	5.73	8.10	9.92	11.45	12.81	14.03	13.36	12.15	11.25	10.40
55	5.21	7.36	9.02	10.41	11.64	12.75	12.14	11.05	10.23	9.46
60	4.77	6.75	8.27	9.55	10.67	11.69	11.13	10.13	9.38	8.67
65	4.41	6.23	7.63	8.81	9.85	10.79	10.28	9.35	8.66	8.00
70	4.09	5.79	7.09	8.18	9.15	10.02	9.54	8.68	8.04	7.43
75	3.82	5.40	6.61	7.64	8.54	9.35	8.91	8.10	7.50	6.93
80	3.58	5.06	6.20	7.16	8.00	8.77	8.35	7.60	7.03	6.50
85	3.37	4.76	5.84	6.74	7.53	8.25	7.86	7.15	6.62	6.12
90	3.18	4.50	5.51	6.36	7.11	7.79	7.42	6.75	6.25	5.78
95	3.01	4.26	5.22	6.03	6.74	7.38	7.03	6.40	5.92	5.47
100	2.86	4.05	4.96	5.73	6.40	7.01	6.68	6.08	5.63	5.20
105	2.73	3.86	4.72	5.45	6.10	6.68	6.36	5.79	5.36	4.95
110	2.60	3.68	4.51	5.21	5.82	6.38	6.07	5.52	5.11	4.73
115	2.49	3.52	4.31	4.98	5.57	6.10	5.81	5.28	4.89	4.52
120	2.39	3.37	4.13	4.77	5.34	5.85	5.57	5.06	4.69	4.33
125	2.29	3.24	3.97	4.58	5.12	5.61	5.34	4.86	4.50	4.16
130	2.20	3.12	3.82	4.41	4.93	5.40	5.14	4.67	4.33	4.00
135	2.12	3.00	3.67	4.24	4.74	5.20	4.95	4.50	4.17	3.85
140	2.05	2.89	3.54	4.09	4.57	5.01	4.77	4.34	4.02	3.71
145	1.97	2.79	3.42	3.95	4.42	4.84	4.61	4.19	3.88	3.59
150	1.91	2.70	3.31	3.82	4.27	4.68	4.45	4.05	3.75	3.47
155	1.85	2.61	3.20	3.69	4.13	4.53	4.31	3.92	3.63	3.36
160	1.79	2.53	3.10	3.58	4.00	4.38	4.17	3.80	3.52	3.25
165	1.74	2.45	3.01	3.47	3.88	4.25	4.05	3.68	3.41	3.15
170	1.68	2.38	2.92	3.37	3.77	4.13	3.93	3.57	3.31	3.06
175	1.64	2.31	2.83	3.27	3.66	4.01	3.82	3.47	3.21	2.97
180	1.59	2.25	2.76	3.18	3.56	3.90	3.71	3.38	3.13	2.89
185	1.55	2.19	2.68	3.10	3.46	3.79	3.61	3.28	3.04	2.81
190	1.51	2.13	2.61	3.01	3.37	3.69	3.52	3.20	2.96	2.74
200	1.43	2.02	2.48	2.86	3.20	3.51	3.34	3.04	2.81	2.60
210	1.36	1.93	2.36	2.73	3.05	3.34	3.18	2.89	2.68	2.48
220	1.30	1.84	2.25	2.60	2.91	3.19	3.04	2.76	2.56	2.36
230	1.25	1.76	2.16	2.49	2.78	3.05	2.90	2.64	2.45	2.26
240	1.19	1.69	2.07	2.39	2.67	2.92	2.78	2.53	2.34	2.17
Q (m^3/s)	**0.795**	**1.125**	**1.378**	**1.591**	**1.779**	**1.948**	**1.855**	**1.688**	**1.563**	**1.445**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 44).

Table 53 – Maximum allowable unit catchment area (A*, hectares)

Type-C Catch Drain: Parabolic cross section										
Mean rock size (d_{50}) = 300 mm and s_r = 2.4					**Variation in rock size (d_{50}/d_{90}) = 0.67**					
Dimensions:				**Flow top width = 3.0 m**				**Flow depth = 0.5 m**		
Rainfall intensity (mm/hr)	**Longitudinal slope of catch drain (%)**									
	1.0	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**
20	10.98	15.53	19.03	21.97	24.56	26.91	29.06	31.07	32.95	34.74
25	8.79	12.43	15.22	17.58	19.65	21.53	23.25	24.86	26.36	27.79
30	7.32	10.36	12.68	14.65	16.38	17.94	19.38	20.71	21.97	23.16
35	6.28	8.88	10.87	12.55	14.04	15.38	16.61	17.75	18.83	19.85
40	5.49	7.77	9.51	10.98	12.28	13.45	14.53	15.53	16.48	17.37
45	4.88	6.90	8.46	9.76	10.92	11.96	12.92	13.81	14.65	15.44
50	4.39	6.21	7.61	8.79	9.83	10.76	11.63	12.43	13.18	13.89
55	3.99	5.65	6.92	7.99	8.93	9.78	10.57	11.30	11.98	12.63
60	3.66	5.18	6.34	7.32	8.19	8.97	9.69	10.36	10.98	11.58
65	3.38	4.78	5.85	6.76	7.56	8.28	8.94	9.56	10.14	10.69
70	3.14	4.44	5.44	6.28	7.02	7.69	8.30	8.88	9.42	9.92
75	2.93	4.14	5.07	5.86	6.55	7.18	7.75	8.29	8.79	9.26
80	2.75	3.88	4.76	5.49	6.14	6.73	7.27	7.77	8.24	8.68
85	2.58	3.66	4.48	5.17	5.78	6.33	6.84	7.31	7.75	8.17
90	2.44	3.45	4.23	4.88	5.46	5.98	6.46	6.90	7.32	7.72
95	2.31	3.27	4.01	4.63	5.17	5.66	6.12	6.54	6.94	7.31
100	2.20	3.11	3.81	4.39	4.91	5.38	5.81	6.21	6.59	6.95
105	2.09	2.96	3.62	4.18	4.68	5.13	5.54	5.92	6.28	6.62
110	2.00	2.82	3.46	3.99	4.47	4.89	5.28	5.65	5.99	6.32
115	1.91	2.70	3.31	3.82	4.27	4.68	5.05	5.40	5.73	6.04
120	1.83	2.59	3.17	3.66	4.09	4.48	4.84	5.18	5.49	5.79
125	1.76	2.49	3.04	3.52	3.93	4.31	4.65	4.97	5.27	5.56
130	1.69	2.39	2.93	3.38	3.78	4.14	4.47	4.78	5.07	5.34
135	1.63	2.30	2.82	3.25	3.64	3.99	4.31	4.60	4.88	5.15
140	1.57	2.22	2.72	3.14	3.51	3.84	4.15	4.44	4.71	4.96
145	1.52	2.14	2.62	3.03	3.39	3.71	4.01	4.29	4.55	4.79
150	1.46	2.07	2.54	2.93	3.28	3.59	3.88	4.14	4.39	4.63
155	1.42	2.00	2.45	2.83	3.17	3.47	3.75	4.01	4.25	4.48
160	1.37	1.94	2.38	2.75	3.07	3.36	3.63	3.88	4.12	4.34
165	1.33	1.88	2.31	2.66	2.98	3.26	3.52	3.77	3.99	4.21
170	1.29	1.83	2.24	2.58	2.89	3.17	3.42	3.66	3.88	4.09
175	1.26	1.78	2.17	2.51	2.81	3.08	3.32	3.55	3.77	3.97
180	1.22	1.73	2.11	2.44	2.73	2.99	3.23	3.45	3.66	3.86
185	1.19	1.68	2.06	2.38	2.66	2.91	3.14	3.36	3.56	3.76
190	1.16	1.64	2.00	2.31	2.59	2.83	3.06	3.27	3.47	3.66
200	1.10	1.55	1.90	2.20	2.46	2.69	2.91	3.11	3.30	3.47
210	1.05	1.48	1.81	2.09	2.34	2.56	2.77	2.96	3.14	3.31
220	1.00	1.41	1.73	2.00	2.23	2.45	2.64	2.82	3.00	3.16
230	0.96	1.35	1.65	1.91	2.14	2.34	2.53	2.70	2.87	3.02
240	0.92	1.29	1.59	1.83	2.05	2.24	2.42	2.59	2.75	2.89
250	0.88	1.24	1.52	1.76	1.97	2.15	2.33	2.49	2.64	2.78
Q (m^3/s)	**0.610**	**0.863**	**1.057**	**1.221**	**1.365**	**1.495**	**1.615**	**1.726**	**1.831**	**1.930**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 44).

Table 54 – Dimensions of standard triangular V-drains

Catch drain type	Max top width of flow (T)	Maximum flow depth (y)	Top width of formed drain	Depth of formed drain	Hyd. rad. (R) at max flow depth	Area (A) at max flow depth
Type-AV	1.0m	0.15m	2.0m	0.30m	0.072m	0.075m^2
Type-BV	1.8m	0.30m	2.7m	0.45m	0.142m	0.270m^2
Type-CV	3.0m	0.50m	3.9m	0.65m	0.237m	0.750m^2

Table 55 – Hydraulic parameters of rock-lined, V-drain cross-section catch drains operating full, and at the maximum allowable channel slope

Mean rock size, d_{50} (mm)	50	100	150	200	300
d_{90} (mm) [1]	75	150	225	300	450
Allowable flow velocity (m/s)	1.00	1.41	1.73	2.00	2.45
Type-AV catch drain: width (T) = 1.0 m, depth (Y) = 0.15 m					
Manning's roughness 'n' [3]	0.047	0.076	0.102	0.127	0.175
Maximum channel slope (%)	7.53	[2]	[2]	[2]	[2]
Maximum channel slope (X:1)	13.3	[2]	[2]	[2]	[2]
Type-BV catch drain: width (T) = 1.8 m, depth (Y) = 0.3 m					
Manning's roughness 'n' [3]	0.036	0.053	0.070	0.086	0.116
Maximum channel slope (%)	1.71	7.69	[2]	[2]	[2]
Maximum channel slope (X:1)	58.5	13.0	[2]	[2]	[2]
Type-CV catch drain: width (T) = 3.0 m, depth (Y) = 0.5 m					
Manning's roughness 'n' [3]	0.030	0.043	0.054	0.066	0.087
Maximum channel slope (%)	0.63	2.50	6.06	[2]	[2]
Maximum channel slope (X:1)	159	40	16.5	[2]	[2]

[1] Based on a rock size distribution defined by d_{50}/d_{90} = 0.67, and specific gravity of 2.4

[2] Theoretical maximum channel slope exceeds 10% (1 in 10) for catch drain flowing at maximum allowable flow depth (Y).

[3] Manning roughness values (n) are only applicable to the drain flowing full, i.e. depth = Y.

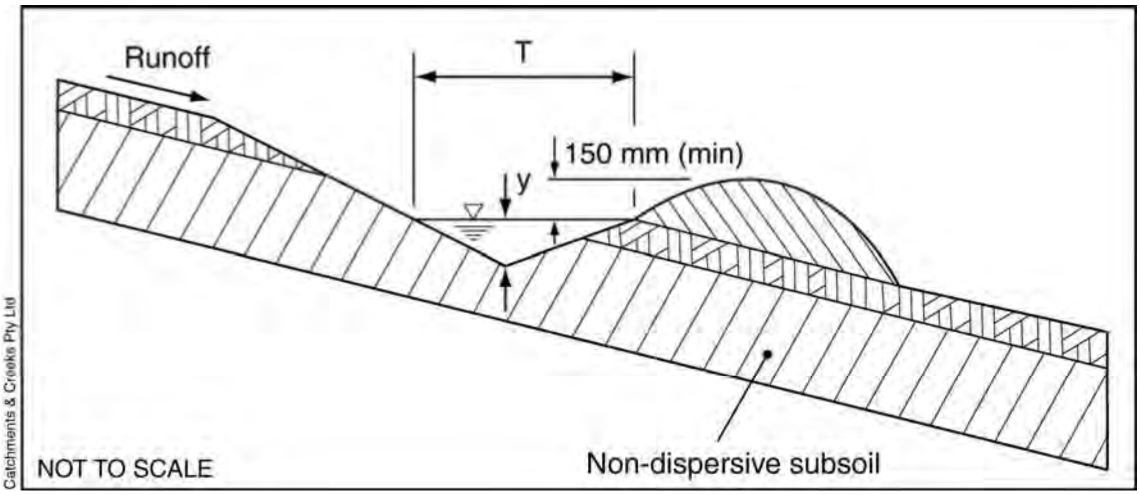


Figure 13(2) – Triangular V-drain with down-slope bank (from Part 1)

Table 56 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-AV Catch Drain: V-drain cross section										
Mean rock size (d_{50}) = 100 mm and s_r = 2.4					Variation in rock size (d_{50}/d_{90}) = 0.67					
Dimensions:				Flow top width = 1.0 m				Flow depth = 0.15 m		
Rainfall intensity (mm/hr)	**Longitudinal slope of catch drain (%)**									
	1.0	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**
15	0.410	0.579	0.709	0.819	0.916	1.003	1.084	1.158	1.229	1.295
20	0.307	0.434	0.532	0.614	0.687	0.752	0.813	0.869	0.922	0.971
25	0.246	0.348	0.426	0.491	0.549	0.602	0.650	0.695	0.737	0.777
30	0.205	0.290	0.355	0.410	0.458	0.502	0.542	0.579	0.614	0.648
35	0.176	0.248	0.304	0.351	0.392	0.430	0.464	0.496	0.527	0.555
40	0.154	0.217	0.266	0.307	0.343	0.376	0.406	0.434	0.461	0.486
45	0.137	0.193	0.236	0.273	0.305	0.334	0.361	0.386	0.410	0.432
50	0.123	0.174	0.213	0.246	0.275	0.301	0.325	0.348	0.369	0.389
55	0.112	0.158	0.193	0.223	0.250	0.274	0.296	0.316	0.335	0.353
60	0.102	0.145	0.177	0.205	0.229	0.251	0.271	0.290	0.307	0.324
65	0.095	0.134	0.164	0.189	0.211	0.232	0.250	0.267	0.284	0.299
70	0.088	0.124	0.152	0.176	0.196	0.215	0.232	0.248	0.263	0.278
75	0.082	0.116	0.142	0.164	0.183	0.201	0.217	0.232	0.246	0.259
80	0.077	0.109	0.133	0.154	0.172	0.188	0.203	0.217	0.230	0.243
85	0.072	0.102	0.125	0.145	0.162	0.177	0.191	0.204	0.217	0.229
90	0.068	0.097	0.118	0.137	0.153	0.167	0.181	0.193	0.205	0.216
95	0.065	0.091	0.112	0.129	0.145	0.158	0.171	0.183	0.194	0.204
100	0.061	0.087	0.106	0.123	0.137	0.150	0.163	0.174	0.184	0.194
105	0.059	0.083	0.101	0.117	0.131	0.143	0.155	0.165	0.176	0.185
110	0.056	0.079	0.097	0.112	0.125	0.137	0.148	0.158	0.168	0.177
115	0.053	0.076	0.093	0.107	0.119	0.131	0.141	0.151	0.160	0.169
120	0.051	0.072	0.089	0.102	0.114	0.125	0.135	0.145	0.154	0.162
125	0.049	0.070	0.085	0.098	0.110	0.120	0.130	0.139	0.147	0.155
130	0.047	0.067	0.082	0.095	0.106	0.116	0.125	0.134	0.142	0.149
135	0.046	0.064	0.079	0.091	0.102	0.111	0.120	0.129	0.137	0.144
140	0.044	0.062	0.076	0.088	0.098	0.107	0.116	0.124	0.132	0.139
145	0.042	0.060	0.073	0.085	0.095	0.104	0.112	0.120	0.127	0.134
150	0.041	0.058	0.071	0.082	0.092	0.100	0.108	0.116	0.123	0.130
155	0.040	0.056	0.069	0.079	0.089	0.097	0.105	0.112	0.119	0.125
160	0.038	0.054	0.067	0.077	0.086	0.094	0.102	0.109	0.115	0.121
165	0.037	0.053	0.064	0.074	0.083	0.091	0.099	0.105	0.112	0.118
170	0.036	0.051	0.063	0.072	0.081	0.089	0.096	0.102	0.108	0.114
175	0.035	0.050	0.061	0.070	0.078	0.086	0.093	0.099	0.105	0.111
180	0.034	0.048	0.059	0.068	0.076	0.084	0.090	0.097	0.102	0.108
185	0.033	0.047	0.058	0.066	0.074	0.081	0.088	0.094	0.100	0.105
190	0.032	0.046	0.056	0.065	0.072	0.079	0.086	0.091	0.097	0.102
200	0.031	0.043	0.053	0.061	0.069	0.075	0.081	0.087	0.092	0.097
210	0.029	0.041	0.051	0.059	0.065	0.072	0.077	0.083	0.088	0.093
220	0.028	0.039	0.048	0.056	0.062	0.068	0.074	0.079	0.084	0.088
230	0.027	0.038	0.046	0.053	0.060	0.065	0.071	0.076	0.080	0.084
240	0.026	0.036	0.044	0.051	0.057	0.063	0.068	0.072	0.077	0.081
Q (m^3/s)	**0.017**	**0.024**	**0.030**	**0.034**	**0.038**	**0.042**	**0.045**	**0.048**	**0.051**	**0.054**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 55).

Table 57 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-AV Catch Drain: V-drain cross section										
Mean rock size (d_{50}) = 200 mm and s_r = 2.4					**Variation in rock size (d_{50}/d_{90}) = 0.67**					
Dimensions:				**Flow top width = 1.0 m**				**Flow depth = 0.15 m**		
Rainfall intensity (mm/hr)	**Longitudinal slope of catch drain (%)**									
	1.0	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**
15	0.244	0.345	0.423	0.488	0.546	0.598	0.646	0.690	0.732	0.772
20	0.183	0.259	0.317	0.366	0.409	0.448	0.484	0.518	0.549	0.579
25	0.146	0.207	0.254	0.293	0.327	0.359	0.387	0.414	0.439	0.463
30	0.122	0.173	0.211	0.244	0.273	0.299	0.323	0.345	0.366	0.386
35	0.105	0.148	0.181	0.209	0.234	0.256	0.277	0.296	0.314	0.331
40	0.092	0.129	0.159	0.183	0.205	0.224	0.242	0.259	0.275	0.289
45	0.081	0.115	0.141	0.163	0.182	0.199	0.215	0.230	0.244	0.257
50	0.073	0.104	0.127	0.146	0.164	0.179	0.194	0.207	0.220	0.232
55	0.067	0.094	0.115	0.133	0.149	0.163	0.176	0.188	0.200	0.210
60	0.061	0.086	0.106	0.122	0.136	0.149	0.161	0.173	0.183	0.193
65	0.056	0.080	0.098	0.113	0.126	0.138	0.149	0.159	0.169	0.178
70	0.052	0.074	0.091	0.105	0.117	0.128	0.138	0.148	0.157	0.165
75	0.049	0.069	0.085	0.098	0.109	0.120	0.129	0.138	0.146	0.154
80	0.046	0.065	0.079	0.092	0.102	0.112	0.121	0.129	0.137	0.145
85	0.043	0.061	0.075	0.086	0.096	0.105	0.114	0.122	0.129	0.136
90	0.041	0.058	0.070	0.081	0.091	0.100	0.108	0.115	0.122	0.129
95	0.039	0.054	0.067	0.077	0.086	0.094	0.102	0.109	0.116	0.122
100	0.037	0.052	0.063	0.073	0.082	0.090	0.097	0.104	0.110	0.116
105	0.035	0.049	0.060	0.070	0.078	0.085	0.092	0.099	0.105	0.110
110	0.033	0.047	0.058	0.067	0.074	0.082	0.088	0.094	0.100	0.105
115	0.032	0.045	0.055	0.064	0.071	0.078	0.084	0.090	0.096	0.101
120	0.031	0.043	0.053	0.061	0.068	0.075	0.081	0.086	0.092	0.096
125	0.029	0.041	0.051	0.059	0.065	0.072	0.077	0.083	0.088	0.093
130	0.028	0.040	0.049	0.056	0.063	0.069	0.075	0.080	0.084	0.089
135	0.027	0.038	0.047	0.054	0.061	0.066	0.072	0.077	0.081	0.086
140	0.026	0.037	0.045	0.052	0.058	0.064	0.069	0.074	0.078	0.083
145	0.025	0.036	0.044	0.050	0.056	0.062	0.067	0.071	0.076	0.080
150	0.024	0.035	0.042	0.049	0.055	0.060	0.065	0.069	0.073	0.077
155	0.024	0.033	0.041	0.047	0.053	0.058	0.062	0.067	0.071	0.075
160	0.023	0.032	0.040	0.046	0.051	0.056	0.061	0.065	0.069	0.072
165	0.022	0.031	0.038	0.044	0.050	0.054	0.059	0.063	0.067	0.070
170	0.022	0.030	0.037	0.043	0.048	0.053	0.057	0.061	0.065	0.068
175	0.021	0.030	0.036	0.042	0.047	0.051	0.055	0.059	0.063	0.066
180	0.020	0.029	0.035	0.041	0.045	0.050	0.054	0.058	0.061	0.064
185	0.020	0.028	0.034	0.040	0.044	0.048	0.052	0.056	0.059	0.063
190	0.019	0.027	0.033	0.039	0.043	0.047	0.051	0.054	0.058	0.061
200	0.018	0.026	0.032	0.037	0.041	0.045	0.048	0.052	0.055	0.058
210	0.017	0.025	0.030	0.035	0.039	0.043	0.046	0.049	0.052	0.055
220	0.017	0.024	0.029	0.033	0.037	0.041	0.044	0.047	0.050	0.053
230	0.016	0.023	0.028	0.032	0.036	0.039	0.042	0.045	0.048	0.050
240	0.015	0.022	0.026	0.031	0.034	0.037	0.040	0.043	0.046	0.048
Q (m^3/s)	**0.010**	**0.014**	**0.018**	**0.020**	**0.023**	**0.025**	**0.027**	**0.029**	**0.031**	**0.032**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 55).

Table 58 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-AV Catch Drain: V-drain cross section										
Mean rock size (d_{50}) = 300 mm and s_r = 2.4					Variation in rock size (d_{50}/d_{90}) = 0.67					
Dimensions:				Flow top width = 1.0 m				Flow depth = 0.15 m		
Rainfall intensity (mm/hr)	Longitudinal slope of catch drain (%)									
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
15	0.178	0.251	0.308	0.355	0.397	0.435	0.470	0.502	0.533	0.562
20	0.133	0.188	0.231	0.266	0.298	0.326	0.352	0.377	0.400	0.421
25	0.107	0.151	0.185	0.213	0.238	0.261	0.282	0.301	0.320	0.337
30	0.089	0.126	0.154	0.178	0.199	0.218	0.235	0.251	0.266	0.281
35	0.076	0.108	0.132	0.152	0.170	0.186	0.201	0.215	0.228	0.241
40	0.067	0.094	0.115	0.133	0.149	0.163	0.176	0.188	0.200	0.211
45	0.059	0.084	0.103	0.118	0.132	0.145	0.157	0.167	0.178	0.187
50	0.053	0.075	0.092	0.107	0.119	0.131	0.141	0.151	0.160	0.168
55	0.048	0.069	0.084	0.097	0.108	0.119	0.128	0.137	0.145	0.153
60	0.044	0.063	0.077	0.089	0.099	0.109	0.117	0.126	0.133	0.140
65	0.041	0.058	0.071	0.082	0.092	0.100	0.108	0.116	0.123	0.130
70	0.038	0.054	0.066	0.076	0.085	0.093	0.101	0.108	0.114	0.120
75	0.036	0.050	0.062	0.071	0.079	0.087	0.094	0.100	0.107	0.112
80	0.033	0.047	0.058	0.067	0.074	0.082	0.088	0.094	0.100	0.105
85	0.031	0.044	0.054	0.063	0.070	0.077	0.083	0.089	0.094	0.099
90	0.030	0.042	0.051	0.059	0.066	0.073	0.078	0.084	0.089	0.094
95	0.028	0.040	0.049	0.056	0.063	0.069	0.074	0.079	0.084	0.089
100	0.027	0.038	0.046	0.053	0.060	0.065	0.070	0.075	0.080	0.084
105	0.025	0.036	0.044	0.051	0.057	0.062	0.067	0.072	0.076	0.080
110	0.024	0.034	0.042	0.048	0.054	0.059	0.064	0.069	0.073	0.077
115	0.023	0.033	0.040	0.046	0.052	0.057	0.061	0.066	0.070	0.073
120	0.022	0.031	0.038	0.044	0.050	0.054	0.059	0.063	0.067	0.070
125	0.021	0.030	0.037	0.043	0.048	0.052	0.056	0.060	0.064	0.067
130	0.020	0.029	0.035	0.041	0.046	0.050	0.054	0.058	0.061	0.065
135	0.020	0.028	0.034	0.039	0.044	0.048	0.052	0.056	0.059	0.062
140	0.019	0.027	0.033	0.038	0.043	0.047	0.050	0.054	0.057	0.060
145	0.018	0.026	0.032	0.037	0.041	0.045	0.049	0.052	0.055	0.058
150	0.018	0.025	0.031	0.036	0.040	0.044	0.047	0.050	0.053	0.056
155	0.017	0.024	0.030	0.034	0.038	0.042	0.045	0.049	0.052	0.054
160	0.017	0.024	0.029	0.033	0.037	0.041	0.044	0.047	0.050	0.053
165	0.016	0.023	0.028	0.032	0.036	0.040	0.043	0.046	0.048	0.051
170	0.016	0.022	0.027	0.031	0.035	0.038	0.041	0.044	0.047	0.050
175	0.015	0.022	0.026	0.030	0.034	0.037	0.040	0.043	0.046	0.048
180	0.015	0.021	0.026	0.030	0.033	0.036	0.039	0.042	0.044	0.047
185	0.014	0.020	0.025	0.029	0.032	0.035	0.038	0.041	0.043	0.046
190	0.014	0.020	0.024	0.028	0.031	0.034	0.037	0.040	0.042	0.044
200	0.013	0.019	0.023	0.027	0.030	0.033	0.035	0.038	0.040	0.042
210	0.013	0.018	0.022	0.025	0.028	0.031	0.034	0.036	0.038	0.040
220	0.012	0.017	0.021	0.024	0.027	0.030	0.032	0.034	0.036	0.038
230	0.012	0.016	0.020	0.023	0.026	0.028	0.031	0.033	0.035	0.037
240	0.011	0.016	0.019	0.022	0.025	0.027	0.029	0.031	0.033	0.035
Q (m^3/s)	**0.007**	**0.010**	**0.013**	**0.015**	**0.017**	**0.018**	**0.020**	**0.021**	**0.022**	**0.023**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 55).

Table 59 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-BV Catch Drain: V-drain cross section

Mean rock size (d_{50}) = 100 mm and s_r = 2.4 | Variation in rock size (d_{50}/d_{90}) = 0.67

Dimensions: | Flow top width = 1.8 m | Flow depth = 0.30 m

Rainfall intensity (mm/hr)	Longitudinal slope of catch drain (%)									
	1.0	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**
15	3.304	4.673	5.723	6.609	7.389	8.094	8.742	Allowable unit catchment areas presented below are limited by the allowable flow velocity		
20	2.478	3.505	4.292	4.956	5.542	6.070	6.557			
25	1.983	2.804	3.434	3.965	4.433	4.856	5.245			
30	1.652	2.337	2.862	3.304	3.694	4.047	4.371			
35	1.416	2.003	2.453	2.832	3.167	3.469	3.747			
40	1.239	1.752	2.146	2.478	2.771	3.035	3.278	3.324	2.992	2.715
45	1.101	1.558	1.908	2.203	2.463	2.698	2.914	2.955	2.660	2.413
50	0.991	1.402	1.717	1.983	2.217	2.428	2.623	2.659	2.394	2.172
55	0.901	1.274	1.561	1.802	2.015	2.207	2.384	2.417	2.176	1.975
60	0.826	1.168	1.431	1.652	1.847	2.023	2.186	2.216	1.995	1.810
65	0.763	1.078	1.321	1.525	1.705	1.868	2.017	2.045	1.841	1.671
70	0.708	1.001	1.226	1.416	1.583	1.734	1.873	1.899	1.710	1.551
75	0.661	0.935	1.145	1.322	1.478	1.619	1.748	1.773	1.596	1.448
80	0.620	0.876	1.073	1.239	1.385	1.518	1.639	1.662	1.496	1.358
85	0.583	0.825	1.010	1.166	1.304	1.428	1.543	1.564	1.408	1.278
90	0.551	0.779	0.954	1.101	1.231	1.349	1.457	1.477	1.330	1.207
95	0.522	0.738	0.904	1.043	1.167	1.278	1.380	1.400	1.260	1.143
100	0.496	0.701	0.858	0.991	1.108	1.214	1.311	1.330	1.197	1.086
105	0.472	0.668	0.818	0.944	1.056	1.156	1.249	1.266	1.140	1.034
110	0.451	0.637	0.780	0.901	1.008	1.104	1.192	1.209	1.088	0.987
115	0.431	0.610	0.747	0.862	0.964	1.056	1.140	1.156	1.041	0.944
120	0.413	0.584	0.715	0.826	0.924	1.012	1.093	1.108	0.997	0.905
125	0.397	0.561	0.687	0.793	0.887	0.971	1.049	1.064	0.957	0.869
130	0.381	0.539	0.660	0.763	0.853	0.934	1.009	1.023	0.921	0.835
135	0.367	0.519	0.636	0.734	0.821	0.899	0.971	0.985	0.887	0.804
140	0.354	0.501	0.613	0.708	0.792	0.867	0.937	0.950	0.855	0.776
145	0.342	0.483	0.592	0.684	0.764	0.837	0.904	0.917	0.825	0.749
150	0.330	0.467	0.572	0.661	0.739	0.809	0.874	0.886	0.798	0.724
155	0.320	0.452	0.554	0.640	0.715	0.783	0.846	0.858	0.772	0.701
160	0.310	0.438	0.537	0.620	0.693	0.759	0.820	0.831	0.748	0.679
165	0.300	0.425	0.520	0.601	0.672	0.736	0.795	0.806	0.725	0.658
170	0.292	0.412	0.505	0.583	0.652	0.714	0.771	0.782	0.704	0.639
175	0.283	0.401	0.491	0.566	0.633	0.694	0.749	0.760	0.684	0.621
180	0.275	0.389	0.477	0.551	0.616	0.674	0.729	0.739	0.665	0.603
185	0.268	0.379	0.464	0.536	0.599	0.656	0.709	0.719	0.647	0.587
190	0.261	0.369	0.452	0.522	0.583	0.639	0.690	0.700	0.630	0.572
200	0.248	0.350	0.429	0.496	0.554	0.607	0.656	0.665	0.598	0.543
210	0.236	0.334	0.409	0.472	0.528	0.578	0.624	0.633	0.570	0.517
220	0.225	0.319	0.390	0.451	0.504	0.552	0.596	0.604	0.544	0.494
230	0.215	0.305	0.373	0.431	0.482	0.528	0.570	0.578	0.520	0.472
240	0.207	0.292	0.358	0.413	0.462	0.506	0.546	0.554	0.499	0.453
Q (m^3/s)	**0.138**	**0.195**	**0.238**	**0.275**	**0.308**	**0.337**	**0.364**	**0.369**	**0.332**	**0.302**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 55).

Table 60 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-BV Catch Drain: V-drain cross section

Mean rock size (d_{50}) = 200 mm and s_r = 2.4	Variation in rock size (d_{50}/d_{90}) = 0.67
Dimensions: Flow top width = 1.8 m	Flow depth = 0.30 m

Rainfall intensity (mm/hr)	Longitudinal slope of catch drain (%)									
	1.0	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**
15	2.061	2.915	3.570	4.122	4.608	5.048	5.453	5.829	6.183	6.517
20	1.546	2.186	2.677	3.091	3.456	3.786	4.090	4.372	4.637	4.888
25	1.237	1.749	2.142	2.473	2.765	3.029	3.272	3.498	3.710	3.910
30	1.030	1.457	1.785	2.061	2.304	2.524	2.726	2.915	3.091	3.259
35	0.883	1.249	1.530	1.767	1.975	2.164	2.337	2.498	2.650	2.793
40	0.773	1.093	1.339	1.546	1.728	1.893	2.045	2.186	2.319	2.444
45	0.687	0.972	1.190	1.374	1.536	1.683	1.818	1.943	2.061	2.172
50	0.618	0.874	1.071	1.237	1.383	1.514	1.636	1.749	1.855	1.955
55	0.562	0.795	0.974	1.124	1.257	1.377	1.487	1.590	1.686	1.777
60	0.515	0.729	0.892	1.030	1.152	1.262	1.363	1.457	1.546	1.629
65	0.476	0.673	0.824	0.951	1.063	1.165	1.258	1.345	1.427	1.504
70	0.442	0.625	0.765	0.883	0.988	1.082	1.168	1.249	1.325	1.397
75	0.412	0.583	0.714	0.824	0.922	1.010	1.091	1.166	1.237	1.303
80	0.386	0.546	0.669	0.773	0.864	0.947	1.022	1.093	1.159	1.222
85	0.364	0.514	0.630	0.727	0.813	0.891	0.962	1.029	1.091	1.150
90	0.343	0.486	0.595	0.687	0.768	0.841	0.909	0.972	1.030	1.086
95	0.325	0.460	0.564	0.651	0.728	0.797	0.861	0.920	0.976	1.029
100	0.309	0.437	0.535	0.618	0.691	0.757	0.818	0.874	0.927	0.978
105	0.294	0.416	0.510	0.589	0.658	0.721	0.779	0.833	0.883	0.931
110	0.281	0.397	0.487	0.562	0.628	0.688	0.744	0.795	0.843	0.889
115	0.269	0.380	0.466	0.538	0.601	0.658	0.711	0.760	0.806	0.850
120	0.258	0.364	0.446	0.515	0.576	0.631	0.682	0.729	0.773	0.815
125	0.247	0.350	0.428	0.495	0.553	0.606	0.654	0.700	0.742	0.782
130	0.238	0.336	0.412	0.476	0.532	0.582	0.629	0.673	0.713	0.752
135	0.229	0.324	0.397	0.458	0.512	0.561	0.606	0.648	0.687	0.724
140	0.221	0.312	0.382	0.442	0.494	0.541	0.584	0.625	0.662	0.698
145	0.213	0.302	0.369	0.426	0.477	0.522	0.564	0.603	0.640	0.674
150	0.206	0.291	0.357	0.412	0.461	0.505	0.545	0.583	0.618	0.652
155	0.199	0.282	0.345	0.399	0.446	0.489	0.528	0.564	0.598	0.631
160	0.193	0.273	0.335	0.386	0.432	0.473	0.511	0.546	0.580	0.611
165	0.187	0.265	0.325	0.375	0.419	0.459	0.496	0.530	0.562	0.592
170	0.182	0.257	0.315	0.364	0.407	0.445	0.481	0.514	0.546	0.575
175	0.177	0.250	0.306	0.353	0.395	0.433	0.467	0.500	0.530	0.559
180	0.172	0.243	0.297	0.343	0.384	0.421	0.454	0.486	0.515	0.543
185	0.167	0.236	0.289	0.334	0.374	0.409	0.442	0.473	0.501	0.528
190	0.163	0.230	0.282	0.325	0.364	0.399	0.430	0.460	0.488	0.515
200	0.155	0.219	0.268	0.309	0.346	0.379	0.409	0.437	0.464	0.489
210	0.147	0.208	0.255	0.294	0.329	0.361	0.389	0.416	0.442	0.466
220	0.141	0.199	0.243	0.281	0.314	0.344	0.372	0.397	0.422	0.444
230	0.134	0.190	0.233	0.269	0.301	0.329	0.356	0.380	0.403	0.425
240	0.129	0.182	0.223	0.258	0.288	0.316	0.341	0.364	0.386	0.407
Q (m^3/s)	**0.086**	**0.121**	**0.149**	**0.172**	**0.192**	**0.210**	**0.227**	**0.243**	**0.258**	**0.272**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 55).

Table 61 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-BV Catch Drain: V-drain cross section										
Mean rock size (d_{50}) = 300 mm and s_r = 2.4						Variation in rock size (d_{50}/d_{90}) = 0.67				
Dimensions:				Flow top width = 1.8 m				Flow depth = 0.30 m		
Rainfall intensity (mm/hr)	Longitudinal slope of catch drain (%)									
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
15	1.529	2.162	2.648	3.058	3.419	3.745	4.045	4.324	4.587	4.835
20	1.147	1.622	1.986	2.293	2.564	2.809	3.034	3.243	3.440	3.626
25	0.917	1.297	1.589	1.835	2.051	2.247	2.427	2.595	2.752	2.901
30	0.764	1.081	1.324	1.529	1.709	1.872	2.022	2.162	2.293	2.417
35	0.655	0.927	1.135	1.310	1.465	1.605	1.734	1.853	1.966	2.072
40	0.573	0.811	0.993	1.147	1.282	1.404	1.517	1.622	1.720	1.813
45	0.510	0.721	0.883	1.019	1.140	1.248	1.348	1.441	1.529	1.612
50	0.459	0.649	0.794	0.917	1.026	1.123	1.213	1.297	1.376	1.450
55	0.417	0.590	0.722	0.834	0.932	1.021	1.103	1.179	1.251	1.319
60	0.382	0.541	0.662	0.764	0.855	0.936	1.011	1.081	1.147	1.209
65	0.353	0.499	0.611	0.706	0.789	0.864	0.933	0.998	1.058	1.116
70	0.328	0.463	0.567	0.655	0.733	0.802	0.867	0.927	0.983	1.036
75	0.306	0.432	0.530	0.612	0.684	0.749	0.809	0.865	0.917	0.967
80	0.287	0.405	0.497	0.573	0.641	0.702	0.758	0.811	0.860	0.907
85	0.270	0.382	0.467	0.540	0.603	0.661	0.714	0.763	0.809	0.853
90	0.255	0.360	0.441	0.510	0.570	0.624	0.674	0.721	0.764	0.806
95	0.241	0.341	0.418	0.483	0.540	0.591	0.639	0.683	0.724	0.763
100	0.229	0.324	0.397	0.459	0.513	0.562	0.607	0.649	0.688	0.725
105	0.218	0.309	0.378	0.437	0.488	0.535	0.578	0.618	0.655	0.691
110	0.208	0.295	0.361	0.417	0.466	0.511	0.552	0.590	0.625	0.659
115	0.199	0.282	0.345	0.399	0.446	0.488	0.528	0.564	0.598	0.631
120	0.191	0.270	0.331	0.382	0.427	0.468	0.506	0.541	0.573	0.604
125	0.183	0.259	0.318	0.367	0.410	0.449	0.485	0.519	0.550	0.580
130	0.176	0.249	0.306	0.353	0.394	0.432	0.467	0.499	0.529	0.558
135	0.170	0.240	0.294	0.340	0.380	0.416	0.449	0.480	0.510	0.537
140	0.164	0.232	0.284	0.328	0.366	0.401	0.433	0.463	0.491	0.518
145	0.158	0.224	0.274	0.316	0.354	0.387	0.418	0.447	0.474	0.500
150	0.153	0.216	0.265	0.306	0.342	0.374	0.404	0.432	0.459	0.483
155	0.148	0.209	0.256	0.296	0.331	0.362	0.391	0.418	0.444	0.468
160	0.143	0.203	0.248	0.287	0.320	0.351	0.379	0.405	0.430	0.453
165	0.139	0.197	0.241	0.278	0.311	0.340	0.368	0.393	0.417	0.440
170	0.135	0.191	0.234	0.270	0.302	0.330	0.357	0.382	0.405	0.427
175	0.131	0.185	0.227	0.262	0.293	0.321	0.347	0.371	0.393	0.414
180	0.127	0.180	0.221	0.255	0.285	0.312	0.337	0.360	0.382	0.403
185	0.124	0.175	0.215	0.248	0.277	0.304	0.328	0.351	0.372	0.392
190	0.121	0.171	0.209	0.241	0.270	0.296	0.319	0.341	0.362	0.382
200	0.115	0.162	0.199	0.229	0.256	0.281	0.303	0.324	0.344	0.363
210	0.109	0.154	0.189	0.218	0.244	0.267	0.289	0.309	0.328	0.345
220	0.104	0.147	0.181	0.208	0.233	0.255	0.276	0.295	0.313	0.330
230	0.100	0.141	0.173	0.199	0.223	0.244	0.264	0.282	0.299	0.315
240	0.096	0.135	0.166	0.191	0.214	0.234	0.253	0.270	0.287	0.302
Q (m^3/s)	**0.064**	**0.090**	**0.110**	**0.127**	**0.142**	**0.156**	**0.169**	**0.180**	**0.191**	**0.201**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 55).

Table 62 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-CV Catch Drain: V-drain cross section										
Mean rock size (d_{50}) = 100 mm and s_r = 2.4					Variation in rock size (d_{50}/d_{90}) = 0.67					
Dimensions:				Flow top width = 3.0 m				Flow depth = 0.50 m		
Rainfall intensity (mm/hr)	**Longitudinal slope of catch drain (%)**									
	1.0	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**
15	16.10	22.77	Allowable unit catchment areas presented in the columns below are limited by the allowable flow velocity							
20	12.08	17.08								
25	9.66	13.66	12.87	9.80	8.04	6.86	5.93	5.32	4.79	4.34
30	8.05	11.39	10.72	8.16	6.70	5.72	4.94	4.43	3.99	3.62
35	6.90	9.76	9.19	7.00	5.74	4.90	4.24	3.80	3.42	3.10
40	6.04	8.54	8.04	6.12	5.03	4.29	3.71	3.32	2.99	2.72
45	5.37	7.59	7.15	5.44	4.47	3.81	3.29	2.95	2.66	2.41
50	4.83	6.83	6.43	4.90	4.02	3.43	2.97	2.66	2.39	2.17
55	4.39	6.21	5.85	4.45	3.66	3.12	2.70	2.42	2.18	1.97
60	4.03	5.69	5.36	4.08	3.35	2.86	2.47	2.22	1.99	1.81
65	3.72	5.26	4.95	3.77	3.09	2.64	2.28	2.05	1.84	1.67
70	3.45	4.88	4.60	3.50	2.87	2.45	2.12	1.90	1.71	1.55
75	3.22	4.55	4.29	3.27	2.68	2.29	1.98	1.77	1.60	1.45
80	3.02	4.27	4.02	3.06	2.51	2.14	1.85	1.66	1.50	1.36
85	2.84	4.02	3.79	2.88	2.37	2.02	1.74	1.56	1.41	1.28
90	2.68	3.80	3.57	2.72	2.23	1.91	1.65	1.48	1.33	1.21
95	2.54	3.60	3.39	2.58	2.12	1.80	1.56	1.40	1.26	1.14
100	2.42	3.42	3.22	2.45	2.01	1.71	1.48	1.33	1.20	1.09
105	2.30	3.25	3.06	2.33	1.91	1.63	1.41	1.27	1.14	1.03
110	2.20	3.11	2.92	2.23	1.83	1.56	1.35	1.21	1.09	0.99
115	2.10	2.97	2.80	2.13	1.75	1.49	1.29	1.16	1.04	0.94
120	2.01	2.85	2.68	2.04	1.68	1.43	1.24	1.11	1.00	0.91
125	1.93	2.73	2.57	1.96	1.61	1.37	1.19	1.06	0.96	0.87
130	1.86	2.63	2.47	1.88	1.55	1.32	1.14	1.02	0.92	0.84
135	1.79	2.53	2.38	1.81	1.49	1.27	1.10	0.98	0.89	0.80
140	1.73	2.44	2.30	1.75	1.44	1.22	1.06	0.95	0.85	0.78
145	1.67	2.36	2.22	1.69	1.39	1.18	1.02	0.92	0.83	0.75
150	1.61	2.28	2.14	1.63	1.34	1.14	0.99	0.89	0.80	0.72
155	1.56	2.20	2.08	1.58	1.30	1.11	0.96	0.86	0.77	0.70
160	1.51	2.13	2.01	1.53	1.26	1.07	0.93	0.83	0.75	0.68
165	1.46	2.07	1.95	1.48	1.22	1.04	0.90	0.81	0.73	0.66
170	1.42	2.01	1.89	1.44	1.18	1.01	0.87	0.78	0.70	0.64
175	1.38	1.95	1.84	1.40	1.15	0.98	0.85	0.76	0.68	0.62
180	1.34	1.90	1.79	1.36	1.12	0.95	0.82	0.74	0.66	0.60
185	1.31	1.85	1.74	1.32	1.09	0.93	0.80	0.72	0.65	0.59
190	1.27	1.80	1.69	1.29	1.06	0.90	0.78	0.70	0.63	0.57
200	1.21	1.71	1.61	1.22	1.01	0.86	0.74	0.66	0.60	0.54
210	1.15	1.63	1.53	1.17	0.96	0.82	0.71	0.63	0.57	0.52
220	1.10	1.55	1.46	1.11	0.91	0.78	0.67	0.60	0.54	0.49
230	1.05	1.49	1.40	1.06	0.87	0.75	0.64	0.58	0.52	0.47
240	1.01	1.42	1.34	1.02	0.84	0.71	0.62	0.55	0.50	0.45
Q (m^3/s)	**0.671**	**0.949**	**0.894**	**0.680**	**0.558**	**0.476**	**0.412**	**0.369**	**0.332**	**0.302**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 55).

Table 63 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-CV Catch Drain: V-drain cross section										
Mean rock size (d_{50}) = 200 mm and s_r = 2.4					**Variation in rock size (d_{50}/d_{90}) = 0.67**					
Dimensions:				**Flow top width = 3.0 m**				**Flow depth = 0.50 m**		
Rainfall intensity (mm/hr)	**Longitudinal slope of catch drain (%)**									
	1.0	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**
15	10.52	14.88	18.23	21.05	23.53	25.78	27.84	29.77	31.57	33.28
20	7.89	11.16	13.67	15.79	17.65	19.33	20.88	22.32	23.68	24.96
25	6.31	8.93	10.94	12.63	14.12	15.47	16.71	17.86	18.94	19.97
30	5.26	7.44	9.11	10.52	11.77	12.89	13.92	14.88	15.79	16.64
35	4.51	6.38	7.81	9.02	10.09	11.05	11.93	12.76	13.53	14.26
40	3.95	5.58	6.84	7.89	8.82	9.67	10.44	11.16	11.84	12.48
45	3.51	4.96	6.08	7.02	7.84	8.59	9.28	9.92	10.52	11.09
50	3.16	4.46	5.47	6.31	7.06	7.73	8.35	8.93	9.47	9.98
55	2.87	4.06	4.97	5.74	6.42	7.03	7.59	8.12	8.61	9.08
60	2.63	3.72	4.56	5.26	5.88	6.44	6.96	7.44	7.89	8.32
65	2.43	3.43	4.21	4.86	5.43	5.95	6.43	6.87	7.29	7.68
70	2.26	3.19	3.91	4.51	5.04	5.52	5.97	6.38	6.77	7.13
75	2.10	2.98	3.65	4.21	4.71	5.16	5.57	5.95	6.31	6.66
80	1.97	2.79	3.42	3.95	4.41	4.83	5.22	5.58	5.92	6.24
85	1.86	2.63	3.22	3.71	4.15	4.55	4.91	5.25	5.57	5.87
90	1.75	2.48	3.04	3.51	3.92	4.30	4.64	4.96	5.26	5.55
95	1.66	2.35	2.88	3.32	3.72	4.07	4.40	4.70	4.99	5.25
100	1.58	2.23	2.73	3.16	3.53	3.87	4.18	4.46	4.74	4.99
105	1.50	2.13	2.60	3.01	3.36	3.68	3.98	4.25	4.51	4.75
110	1.44	2.03	2.49	2.87	3.21	3.52	3.80	4.06	4.31	4.54
115	1.37	1.94	2.38	2.75	3.07	3.36	3.63	3.88	4.12	4.34
120	1.32	1.86	2.28	2.63	2.94	3.22	3.48	3.72	3.95	4.16
125	1.26	1.79	2.19	2.53	2.82	3.09	3.34	3.57	3.79	3.99
130	1.21	1.72	2.10	2.43	2.72	2.97	3.21	3.43	3.64	3.84
135	1.17	1.65	2.03	2.34	2.61	2.86	3.09	3.31	3.51	3.70
140	1.13	1.59	1.95	2.26	2.52	2.76	2.98	3.19	3.38	3.57
145	1.09	1.54	1.89	2.18	2.43	2.67	2.88	3.08	3.27	3.44
150	1.05	1.49	1.82	2.10	2.35	2.58	2.78	2.98	3.16	3.33
155	1.02	1.44	1.76	2.04	2.28	2.49	2.69	2.88	3.06	3.22
160	0.99	1.40	1.71	1.97	2.21	2.42	2.61	2.79	2.96	3.12
165	0.96	1.35	1.66	1.91	2.14	2.34	2.53	2.71	2.87	3.03
170	0.93	1.31	1.61	1.86	2.08	2.27	2.46	2.63	2.79	2.94
175	0.90	1.28	1.56	1.80	2.02	2.21	2.39	2.55	2.71	2.85
180	0.88	1.24	1.52	1.75	1.96	2.15	2.32	2.48	2.63	2.77
185	0.85	1.21	1.48	1.71	1.91	2.09	2.26	2.41	2.56	2.70
190	0.83	1.17	1.44	1.66	1.86	2.04	2.20	2.35	2.49	2.63
200	0.79	1.12	1.37	1.58	1.76	1.93	2.09	2.23	2.37	2.50
210	0.75	1.06	1.30	1.50	1.68	1.84	1.99	2.13	2.26	2.38
220	0.72	1.01	1.24	1.44	1.60	1.76	1.90	2.03	2.15	2.27
230	0.69	0.97	1.19	1.37	1.53	1.68	1.82	1.94	2.06	2.17
240	0.66	0.93	1.14	1.32	1.47	1.61	1.74	1.86	1.97	2.08
Q (m^3/s)	**0.439**	**0.620**	**0.760**	**0.877**	**0.981**	**1.074**	**1.160**	**1.240**	**1.316**	**1.387**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 55).

Table 64 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-CV Catch Drain: V-drain cross section										
Mean rock size (d_{50}) = 300 mm and s_r = 2.4					**Variation in rock size (d_{50}/d_{90}) = 0.67**					
Dimensions:				**Flow top width = 3.0 m**				**Flow depth = 0.50 m**		
Rainfall intensity (mm/hr)	**Longitudinal slope of catch drain (%)**									
	1.0	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**9.0**	**10.0**
15	7.97	11.27	13.80	15.94	17.82	19.52	21.08	22.54	23.90	25.20
20	5.98	8.45	10.35	11.95	13.36	14.64	15.81	16.90	17.93	18.90
25	4.78	6.76	8.28	9.56	10.69	11.71	12.65	13.52	14.34	15.12
30	3.98	5.63	6.90	7.97	8.91	9.76	10.54	11.27	11.95	12.60
35	3.41	4.83	5.91	6.83	7.64	8.36	9.03	9.66	10.24	10.80
40	2.99	4.23	5.18	5.98	6.68	7.32	7.91	8.45	8.96	9.45
45	2.66	3.76	4.60	5.31	5.94	6.51	7.03	7.51	7.97	8.40
50	2.39	3.38	4.14	4.78	5.34	5.86	6.32	6.76	7.17	7.56
55	2.17	3.07	3.76	4.35	4.86	5.32	5.75	6.15	6.52	6.87
60	1.99	2.82	3.45	3.98	4.45	4.88	5.27	5.63	5.98	6.30
65	1.84	2.60	3.18	3.68	4.11	4.50	4.86	5.20	5.52	5.81
70	1.71	2.41	2.96	3.41	3.82	4.18	4.52	4.83	5.12	5.40
75	1.59	2.25	2.76	3.19	3.56	3.90	4.22	4.51	4.78	5.04
80	1.49	2.11	2.59	2.99	3.34	3.66	3.95	4.23	4.48	4.72
85	1.41	1.99	2.44	2.81	3.14	3.44	3.72	3.98	4.22	4.45
90	1.33	1.88	2.30	2.66	2.97	3.25	3.51	3.76	3.98	4.20
95	1.26	1.78	2.18	2.52	2.81	3.08	3.33	3.56	3.77	3.98
100	1.20	1.69	2.07	2.39	2.67	2.93	3.16	3.38	3.59	3.78
105	1.14	1.61	1.97	2.28	2.55	2.79	3.01	3.22	3.41	3.60
110	1.09	1.54	1.88	2.17	2.43	2.66	2.87	3.07	3.26	3.44
115	1.04	1.47	1.80	2.08	2.32	2.55	2.75	2.94	3.12	3.29
120	1.00	1.41	1.73	1.99	2.23	2.44	2.64	2.82	2.99	3.15
125	0.96	1.35	1.66	1.91	2.14	2.34	2.53	2.70	2.87	3.02
130	0.92	1.30	1.59	1.84	2.06	2.25	2.43	2.60	2.76	2.91
135	0.89	1.25	1.53	1.77	1.98	2.17	2.34	2.50	2.66	2.80
140	0.85	1.21	1.48	1.71	1.91	2.09	2.26	2.41	2.56	2.70
145	0.82	1.17	1.43	1.65	1.84	2.02	2.18	2.33	2.47	2.61
150	0.80	1.13	1.38	1.59	1.78	1.95	2.11	2.25	2.39	2.52
155	0.77	1.09	1.34	1.54	1.72	1.89	2.04	2.18	2.31	2.44
160	0.75	1.06	1.29	1.49	1.67	1.83	1.98	2.11	2.24	2.36
165	0.72	1.02	1.25	1.45	1.62	1.77	1.92	2.05	2.17	2.29
170	0.70	0.99	1.22	1.41	1.57	1.72	1.86	1.99	2.11	2.22
175	0.68	0.97	1.18	1.37	1.53	1.67	1.81	1.93	2.05	2.16
180	0.66	0.94	1.15	1.33	1.48	1.63	1.76	1.88	1.99	2.10
185	0.65	0.91	1.12	1.29	1.44	1.58	1.71	1.83	1.94	2.04
190	0.63	0.89	1.09	1.26	1.41	1.54	1.66	1.78	1.89	1.99
200	0.60	0.85	1.04	1.20	1.34	1.46	1.58	1.69	1.79	1.89
210	0.57	0.80	0.99	1.14	1.27	1.39	1.51	1.61	1.71	1.80
220	0.54	0.77	0.94	1.09	1.21	1.33	1.44	1.54	1.63	1.72
230	0.52	0.73	0.90	1.04	1.16	1.27	1.37	1.47	1.56	1.64
240	0.50	0.70	0.86	1.00	1.11	1.22	1.32	1.41	1.49	1.57
Q (m^3/s)	**0.332**	**0.470**	**0.575**	**0.664**	**0.742**	**0.813**	**0.878**	**0.939**	**0.996**	**1.050**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 55).

Catch Drains Part 2: Earth-lined

DRAINAGE CONTROL TECHNIQUE

Low Gradient	✓	Velocity Control		Short Term	✓
Steep Gradient		Channel Lining		Medium-Long Term	✓
Outlet Control		Soil Treatment		Permanent	[1]

[1] The design of permanent catch drains requires consideration of issues not discussed within this fact sheet, such as maintenance requirements. This fact sheet should not be used for the design of permanent drains.

Symbol → CD →

Photo 7 – Earth-lined catch drain

Photo 8 – Large rural catch drain (channel-bank)

Key Principles

1. Catch drains typically have standardised cross-sectional dimensions. Rather than uniquely sizing each catch drain to a given catchment, standard-sized drains are used based on a maximum allowable catchment area for a given rainfall intensity.
2. The **maximum** recommended spacing of catch drains down slopes (Table 3, *Part 1 – General information*) is based on the aim of avoiding rill erosion within the up-slope drainage slope. It should be noted that the **actual** spacing of catch drains down a given slope may need to be less than the specified maximum spacing if the soils are highly erosive soils, or if rilling begins to occur between two existing drains.
3. The critical design parameters are the spacing of the drains down a slope, the maximum allowable catchment area, the choice of lining material (e.g. earth, turf, rock or erosion control mats), and the required channel gradient.

Design Information

The following information must be read in association with the general information presented in *Part 1 – General information*. The following design tables specifically address earth-lined (or unlined) catch drains of specific dimensions.

The design procedure outlined within this fact sheet has been developed to provide a simplified approach suitable for appropriately trained persons involved in the regular design of temporary catch drains. The procedure is just **one** example of how catch drains can be designed. Designers experienced in hydraulic design can of course, design a catch drain using the general principles of open channel hydrologic/hydraulic as outlined in IECA (2008) "Best Practice Erosion and Sediment Control", Appendix A – *Construction site hydrology and hydraulics*.

Common Problems

If earth-lined catch drains discharge to an unstable outlet, then erosion (rilling) can occur downstream of the outlet. This erosion often migrates up into the earth drain causing erosion (rilling) along the channel invert.

The above problem is commonly experienced when the catch drain is cut into dispersive soils.

Invert erosion is also common when the exposed soils are very sandy and lack the necessary qualities of clay to bind the soil.

Damage to associated flow diversion bank (rutting) caused by vehicles.

Special Requirements

The erosion-resistance of the local subsoils should be investigated before planning or designing any unlined drains.

Straw bales or other sediment traps should **not** be placed within these drains due to the risk of causing surcharging of the drain.

Unlined catch drains need to be appropriately compacted, possibly by running a grader's wheels along the drain.

Catch drain should drain to a suitable sediment trap if the diverted water is expected to contain sediment.

Unlined drains usually have a very mild gradient, but it is still important to ensure there is a positive gradient along its full length.

Sufficient space must be provided to allow necessary maintenance access.

Site Inspection

Check that the drain has a stable, positive grade along its length.

Check for a stable drain outlet.

Check if the associated flow diversion bank (if any) is free of damage, i.e. damage caused by construction traffic.

Check that the drain has adequate hydraulic capacity given the catchment area (general observations based on past experience).

Check if rill erosion is occurring within the catchment area up-slope of the drain. If rilling is occurring, then the lateral spacing of the drains will need to be reduced. However, some degree of rill erosion should be expected if recent storms exceeded the intensity of the nominated design storm.

Inspect for evidence of water spilling out (overtopping) of the drain, or erosion down-slope of the drain.

Inspect for erosion along the bed (invert) of the drain. Investigate the reasons for any erosion before recommending solutions. Bed erosion can result from either excessive channel velocities, or an unstable outlet, which causes bed erosion (head-cut) to migrate up the channel.

Possible solutions to channel erosion include:

- reduce effective catchment area;
- increase channel width;
- increase channel roughness;
- stabilise bed with turf, mats or rock;
- stabilise the outlet.

Installation (Earth-lined)

1. Refer to approved plans for location, extent, and construction details. If there are questions or problems with the location, extent, or method of installation, contact the engineer or responsible on-site officer for assistance.
2. Clear the location for the catch drain, clearing only what is needed to provide access for personnel and equipment for installation.
3. Remove roots, stumps, and other debris and dispose of them properly. Do not use debris to build the bank.
4. Grade the drain to the specified slope and form the associated embankment with compacted fill. Note that the drain invert must fall 10cm every 10m for each 1% of required channel gradient.
5. Ensure the sides of the cut drain are no steeper than a 1.5:1 (H:V) slope and the embankment fill slopes no steeper than 2:1.
6. Ensure the completed drain has sufficient deep (as specified for the type of drain) measured from the drain invert to the top of the embankment.
7. Ensure the drain has a constant fall in the desired direction free of obstructions.
8. Ensure the drain discharges to a stable outlet such that soil erosion will be prevented from occurring. Specifically, ensure the drain does not discharge to an unstable fill slope.

Maintenance

1. Inspect all catch drains at least weekly and after runoff-producing storm events and repair any slumps, bank damage, or loss of freeboard.
2. Ensure fill material or sediment is not partially blocking the drain. Where necessary, remove any deposited material to allow free drainage.
3. Dispose of any sediment or fill in a manner that will not create an erosion or pollution hazard.

Removal

1. When the soil disturbance above the catch drain is finished and the area is stabilised, the temporary drain and any associated banks should be removed, unless it is to remain as a permanent drainage feature.
2. Dispose of any sediment or earth in a manner that will not create an erosion or pollution hazard.
3. Grade the area and smooth it out in preparation for stabilisation.
4. Stabilise the area by grassing or as specified within the approved site rehabilitation plan.

Hydraulic design of earth-lined catch drains (using the Rational Method approach):

Step 1 Choose the preferred surface condition of the catch drain (in this case an unlined drain). Also, determine the appropriate Manning's roughness for the nominated surface condition using Table 5.

Step 2 Determine the allowable flow velocity (V_{allow}) from Table 6.

Step 3 Nominate the catch drain profile: parabolic or triangular (V-drain). Parabolic drains have a greater hydraulic capacity and are generally less susceptible to invert erosion, but can be slightly more time-consuming to construct.

Step 4 Choose a trial catch drain size (flow top width 'T', and depth 'Y') from Table 7 (parabolic drains), or Table 12 (triangular drains).

Ultimately this may require an iterative process where various drain sizes are tested for hydraulic capacity using the following design steps.

Step 5 Determine the required longitudinal gradient (S%) for the catch drain using Tables 8 or 13, depending on the chosen drain profile.

Step 6 Determine the required *Average Recurrence Interval* (ARI) of the design storm for the given catch drain (i.e. 1 year, 2 year, 5 year, etc. – refer to Table 4.3.1 in Chapter 4, or Table A1 in Section A2 of Appendix A). Note; if a locally adopted design standard exists, then the ARI must be determined from that standard.

Step 7 Determine the appropriate *time of concentration* (t_c) for the catch drain (refer to Step 4 in IECA, 2008, Appendix A, Section A2).

It is usually sufficient to assume a 5-minute time of concentration (conservative approach), otherwise use the locally adopted hydrologic procedures for determining the time of concentration, or the procedures presented in Appendix A (IECA, 2008) if no preferred local procedure exists.

Step 8 Given the design storm ARI, and duration (t_c), determine the *Average Rainfall Intensity* (I) for the catch drain (refer to Step 6 in Section A2 of Appendix A).

To determine the average rainfall intensity it will be necessary to obtain the relevant *Intensity-Frequency-Duration* (IFD) chart for the given site location.

Step 9 Determine the *maximum unit catchment area* (A*) of the catch drain using Tables 9 to 11, or Tables 14 to 16 depending on the chosen drain type and profile.

The maximum unit catchment area (A*) is the maximum allowable catchment area based on a coefficient of discharge of unity (i.e. C = 1.0).

Step 10 Determine the actual *Coefficient of Discharge* (C) for the catchment contributing runoff to the catch drain (refer to Step 3 in IECA, 2008, Appendix A, Section A2).

Note, it will be necessary to first determine the *Coefficient of Discharge* for a 10 year storm (C_{10}), and then the *Frequency Factor* (F_Y) for the nominated design storm frequency from Table A7 in Step 3, Section A2 of Appendix A, such that:

$$C = C_{10} \,.\, F_Y \leq 1.0$$

Step 11 Determine the maximum allowable catchment area (A) for the catch drain based on the *Coefficient of Discharge* (C) determined in Step 10:

$$A = (A^*)/C \quad \text{(hectares)}$$

Step 12 Determine the maximum allowable horizontal spacing of the catch drains down the slope from Table 3 (*Catch Drain Part 1: General information*).

Step 13 If the desired catchment area of the catch drain (measured from the Erosion and Sediment Control Plan) is **greater** than the maximum allowable area determined in Step 11, that return to Step 4 and select a larger catch drain profile.

If the actual catchment area of the catch drain is **less** than the maximum allowable area determined in Step 11, then either return to Step 4 and select a smaller catch drain profile; or determine the minimum allowable drain slope (S_{min}) which is limited by the maximum allowable flow depth (Y), and maximum allowable drain slope (S_{max}) which is limited by the maximum allowable flow velocity V_{allow}.

Explanation of the design philosophy adopted within this fact sheet:

Given the cross-sectional dimensions of a given catch drain (A & R), its surface roughness (n), gradient (S), and required freeboard, it is possible (using Manning's equation) to determine the hydraulic capacity (Q) of the drain, as presented in Equation 1.

Manning's equation: $$Q = \frac{1}{n}.A.R^{2/3}.S^{1/2} \qquad \text{(Eqn 1)}$$

where: A = cross-sectional flow area of the catch drain

The Rational Method (Equation 2) can be rearrange to form Equation 3:

$$Q = (C.I.A)/360 \qquad \text{(Eqn 2)}$$

$$A.C = 360(Q / I) \qquad \text{(Eqn 3)}$$

where: A = catchment area (ha) of the catch drain (**not** the cross-sectional area of the drain)

If we define a new term called 'the unit catchment area' (A*) as the effective catchment area based on an **assumed** coefficient of discharge of unity (i.e. C = 1.0), then:

Maximum unit catchment area: $$A^* = 360(Q / I) \qquad \text{(Eqn 4)}$$

The relationship between flow velocity (V) and channel slope (S) is given by a modification of the Manning's equation (Equation 5):

$$V = \frac{1}{n}.R^{2/3}.S^{1/2} \qquad \text{(Eqn 5)}$$

For a given catch drain profile (represented by the hydraulic radius, R), and surface lining (represented by the Manning's roughness, n) we can determine the required drain slope (S) for a given allowable flow velocity. This information is presented in Tables 8 and 13. It is noted that at this channel slope, the maximum allowable flow velocity (V_{allow}) will be achieved when the channel is flowing at the maximum allowable flow depth (Y).

Also, for a given catch drain cross-sectional area (A), hydraulic radius (R), and maximum allowable flow velocity (V), we can determine the maximum allowable discharge (Q) from Equation 1. With this discharge, and the nominated design rainfall intensity (I), we can determine the maximum unit catchment area (A*) from Equation 4. This information is presented in Tables 9 to 11 for parabolic drains, and Tables 14 to 16 for drains with a triangular profile.

This means Tables 9 to 11 and 14 to 16 are independent of location, and thus can be used anywhere in the world. Rainfall intensity, I (mm/hr) being the only parameter that is location specific.

In order to determine the maximum allowable catchment area (A), it is necessary to determine the **actual** coefficient of discharge (C) for the adopted storm frequency (ARI), and catchment conditions (i.e. soil porosity). The maximum allowable catchment area (A) is determined from Equation 6.

Maximum allowable catchment area: $$A = A^*/C \qquad \text{(Eqn 6)}$$

Since the coefficient of discharge is always assumed to be less than or equal to unity, the maximum allowable catchment area (A) cannot exceed the maximum unit catchment area (A*).

If the actual catchment area is less than the calculated maximum catchment area (A) from Equation 6, then the catch drain can be constructed at a range of channel gradients such that:

$$S_{min} < S < S_{max}$$

where:

- S_{min} can be determined from Manning's equation based on the catch drain flowing full, but at a channel-full velocity less than the maximum allowable flow velocity;
- S_{max} can be determined from Manning's equation based on the catch drain flowing partially full, and at a velocity equal to the maximum allowable flow velocity.

Design example: Earth-lined catch drain

Design a temporary (< 6 months) catch drain in Townsville with a desired length of 50m across an open silty loam soil slope where the average land slope is 10%. The catch drain will be used to collect and transport 'dirty' water from a disturbed catchment (active construction site).

Step 1 Choose the preferred surface condition of the catch drain (in this case an earth-lined drain) and the associated Manning's (n) roughness. From Table 5 the expected surface condition is probably best represented by a Manning's n = 0.04.

Step 2 Nominate an allowable flow velocity (V_{allow}) of 0.6m/s from Table 6.

Step 3 Choose a parabolic drain profile.

Step 4 Initially try a Type-A catch drain with dimensions: T = 1.0m, Y = 0.15m.

Step 5 Determine the required longitudinal gradient (S) from Table 8 as S = 1.34% for an unlined parabolic drain with V_{allow} = 0.6m/s and n = 0.04.

Step 6 Nominate a 1 in 2 year ARI design storm from Table 4.3.1 (Chapter 4).

Step 7 Initially select a time of concentration (t_c) of 5 minutes.

This initial estimate of the time of concentration can be reviewed later after the spacing and length of the catch drain is finalised.

Step 8 Determine the average rainfall intensity: I = 148mm/hr for Townsville from Table A11 (Appendix A) for ARI = 2-year, and t_c = 5 minutes.

Step 9 Determine the maximum allowable unit catchment area as A* = 0.146ha from Table 9, given V = 0.6m/s, and I = 148mm/hr.

Step 10 Determine the coefficient of discharge (C_Y):

Given the catch drain's catchment area is open, disturbed soil with expected low permeability (due to soil compaction), and given that Townsville's 10 minute, 1-year rainfall intensity (${}^{1}\mathrm{I}_{10}$) is 91.9mm/hr, the 10-year coefficient of discharge, C_{10} = 0.70 from Table A5 (Appendix A – *Construction site hydrology and hydraulics*).

Determine the frequency factor, F_Y = 0.85 for the 1 in 2-year ARI storm from Table A7 (Appendix A of IECA, 2008).

Calculate the effective coefficient of discharge (C) for the 1 in 2-year event using Equation A4 (Appendix A):

$$C = C_2 = F_Y .C_{10} = 0.85 \times 0.70 = 0.60 \leq 1.0 \text{ (OK)}$$

Step 11 Calculate the maximum allowable catchment area (A) for the catch drain:

$$A = (A^*)/C = 0.14/0.60 = 0.245\text{ha}$$

Step 12 Determine the maximum allowable spacing of the catch drains down the catchment slope to be 25m from Table 1 (*Catch Drain, Part 1: General information*) given the catchment slope of 10%.

Now, the question indicates that the desired length of the drain is 50m, this means the catchment area of the drain is 25 x 50 = 1250m^2, or 0.125ha (assuming a rectangular shape), which is less than the maximum allowable catchment area of 0.245ha determined in Step 11. Therefore, this area is OK.

Because the actual catchment area (0.125ha) is significantly less than the maximum allowable catchment area (0.245ha), this mean that the catch drain could be constructed at a longitudinal gradient slightly flatter or steeper than the 1.34% determined in Step 5.

Step 8a For demonstration purposes, a better estimate of the time of concentration may be determined as follows:

Using the Friend's equation (Equation A5 in Appendix A); choose a Horton's n of 0.028 for an open soil catchment slope, an overland flow path length (L) of 25m (i.e. the spacing between catch drains determined in Step 12), and a catchment slope (S) of 10%. (Note, this is not the same term (S) previously presented as the drain slope.) Thus, the travel time of the initial overland sheet flow is:

$$t = (107\ n\ L^{0.333})/S^{0.2} = (107 \times 0.028 \times 25^{0.333})/(10^{0.2}) = 5.5 \text{ minutes}$$

Based on an assumed flow velocity of 0.6m/s, the travel time of flow down the 50m long catch drain is (note, in this case L = length of the drain, **not** the length of the overland flow path):

$$t = L/(V \times 60) = 50/(0.6 \times 60) = 1.4 \text{ minutes}$$

Therefore, the total travel time from the top of the catchment area is:

$$t_c = 5.5 + 1.4 = 6.9 \text{ minutes (say, 7 minutes)}$$

Determine the effective average rainfall intensity for a 1 in 2-year, 7 minute storm in Townsville from Table A11 (Appendix A) as, I = 135mm/hr. This intensity would result in an allowable unit catchment area, A* = 0.160ha from Table 9, and a maximum catchment area, $A = A^*/C_Y = 0.160/0.56 = 0.286\text{ha} > 0.125\text{ha}$, OK

Step 5a Given that the actual catchment area is significantly less than the maximum allowable catchment area, the catch drain can be constructed at:

- a flatter gradient (S_{min} < 1.34%) limited by the maximum flow depth of 0.15m; or
- a steeper gradient (S_{max} > 1.34%) limited by the allowable velocity of 0.6m/s.

To determined **flattest** allowable gradient for this catch drain, first calculate the design 1 in 2-year flow rate at the end of the 50m long catch drain.

$$Q = C\ I\ A/360 = (0.6 \times 135 \times 0.125)/360 = 0.028\text{m}^3/\text{s}$$

The minimum channel gradient (S_{min}) can be determined from Manning's equation:

$$Q = 0.028 = (1/n).A.R^{2/3}.S^{1/2} = (1/0.04)(0.100)(0.094)^{2/3}.S^{1/2}$$

$$S_{min} = 0.29\%$$

The **steepest** longitudinal gradient of the catch drain can also be determined from Manning's equation (Equation A16 in Appendix A); however, in this case the drain will be flowing partially full with a flow top width (T) less than 1.0m, and the flow depth (y) less than 0.15m. *(Note, the drain would still be constructed with the same standard overall physical dimensions specified for all Type-A catch drains.)*

For a Type-A drain the numerical relationship between 'T' & 'y' is given in Table 4:

$$y = 0.15\ T^2$$

and the cross sectional area of flow (A) is given by (Table A30b, Appendix A):

$$A = 0.67(T.y) = 0.1\ T^3 = Q/V = 0.028/0.6 = 0.0465\text{m}^2$$

Therefore, the flow top width, T = 0.775m; the flow depth, y = 0.090m; and the hydraulic radius (R) can be determined from (Table A29b, Appendix A):

$$R = \frac{2\,T^2.y}{3\,T^2 + 8\,y^2} = \frac{2(0.775)^2 \times 0.090}{3(0.775)^2 + 8(0.090)^2} = 0.058\text{m}$$

The maximum catch drain slope is given by rearranging the Manning's equation:

$$S_{max} = 100(V^2\ .\ n^2)/R^{4/3} = 100 \times (0.6^2 \times 0.04^2)/0.058^{4/3} = 2.57\%$$

Therefore, the Type-A catch drain can be constructed at any longitudinal gradient between 0.29% (maximum flow depth) and 2.57% (maximum flow velocity), and still provide the required hydraulic capacity for the 1 in 2 year design storm. It is noted that constructing the drain at the steeper gradient will result in a construction site with maximum drainage capacity.

Table 5 – Typical Manning's roughness for unlined, shallow catch drains [1]

Manning's n	Drain condition
0.02	• Smooth soil surface. • Surface of the drain is compacted to produce a firm, smooth surface using a hand-operated compactor. • There are few, if any, irregularities, sediment deposits or loose soil.
0.04	• Slightly irregular soil surface. • Surface of the drain is compacted to produce a firm smooth surface possibly by running a rubber wheel along the drain. • The drain has minor irregularities equivalent to that produced if an *Erosion Control Blanket* or jute mesh was pinned over the surface.
0.06	• Moderately irregular soil surface. • Surface of the drain is cut by a mechanical blade with little or no soil compaction. Alternatively the bed of the drain has been significantly disturbed by construction traffic. • The drain has irregularities equivalent to a recessed layer of 100mm rock, or the drain contains several clods of loose soil per metre length.

Note: [1] Shallow water drains with a flow depth less than 0.4 metres.

Table 6 – Allowable flow velocity for earth-lined drains

Type	Description	Allowable velocity	Comments
Open earth (unlined channels)	Extremely erodible soils	0.3m/s	• Dispersive clays are highly erodible even at low flow velocities and therefore must be either treated (e.g. with gypsum) or covered with a minimum 100mm of stable soil. • Highly erodible soils may include: Lithosols, Alluvials, Podzols, Siliceous sands, Soloths, Solodized solonetz, Grey podzolics, some Black earths, fine surface texture-contrast soils and Soil Groups ML and CL. • Moderately erodible soils may include: Red earths, Red or Yellow podzolics, some Black earths, Grey or Brown clays, Prarie soils and Soil Groups SW, SP, SM, SC. • Erosion-resistant soils may include: Xanthozem, Euchrozem, Krasnozems, some Red earth soils and Soil Groups GW, GP, GM, GC, MH and CH.
	Sandy soils	0.45m/s	
	Highly erodible soils	0.4 to 0.5m/s	
	Sandy loam soils	0.5m/s	
	Moderately erodible soils	0.6m/s	
	Silty loam soils	0.6m/s	
	Low erodible soils	0.7m/s	
	Firm loam soils	0.7m/s	
	Stiff clay very colloidal soils	1.1m/s	

Table 7 – Dimensions of standard parabolic catch drains

Catch drain type	Max top width of flow (T)	Maximum flow depth (y)	Top width of formed drain [1]	Depth of formed drain	Hyd. rad. (R) at max flow depth	Area (A) at max flow depth
Type-A	1.0m	0.15m	1.6m	0.30m	0.094m	0.100m^2
Type-B	1.8m	0.30m	2.4m	0.45m	0.186m	0.360m^2
Type-C	3.0m	0.50m	3.6m	0.65m	0.310m	1.000m^2

[1] Top width of the formed drain assumes the upper bank slope is limited to a maximum of 2:1.

Table 8 – Required longitudinal gradient (%) for unlined, parabolic catch drains

Manning's roughness (n)	Allowable flow velocity along catch drain (m/s)									
	0.3	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.5**	**2.0**
	Type-A catch drain: flow top width (T) = 1.0 m and flow depth (Y) = 0.15 m									
Smooth soil n=0.02	0.084	0.15	0.23	0.34	0.46	0.75	0.93	1.34	2.10	3.73
Rough soil n=0.04	0.34	0.60	0.93	1.34	1.83	3.02	3.73	5.36	8.38	14.9
Very rough soil n=0.06	0.75	1.34	2.10	3.02	4.11	6.79	8.38	12.1	18.9	33.5
	Type-B catch drain: flow top width (T) = 1.8 m and flow depth (Y) = 0.3 m									
Smooth soil n=0.02	0.034	0.060	0.094	0.14	0.18	0.30	0.38	0.54	0.85	1.50
Rough soil n=0.04	0.14	0.24	0.38	0.54	0.74	1.22	1.50	2.17	3.39	6.02
Very rough soil n=0.06	0.30	0.54	0.85	1.22	1.66	2.74	3.39	4.88	7.62	13.5
	Type-C catch drain: flow top width (T) = 3.0 m and flow depth (Y) = 0.5 m									
Smooth soil n=0.02	0.017	0.030	0.048	0.069	0.093	0.15	0.19	0.27	0.43	0.76
Rough soil n=0.04	0.069	0.12	0.19	0.27	0.37	0.62	0.76	1.10	1.71	3.05
Very rough soil n=0.06	0.15	0.27	0.43	0.62	0.84	1.39	1.71	2.47	3.86	6.85

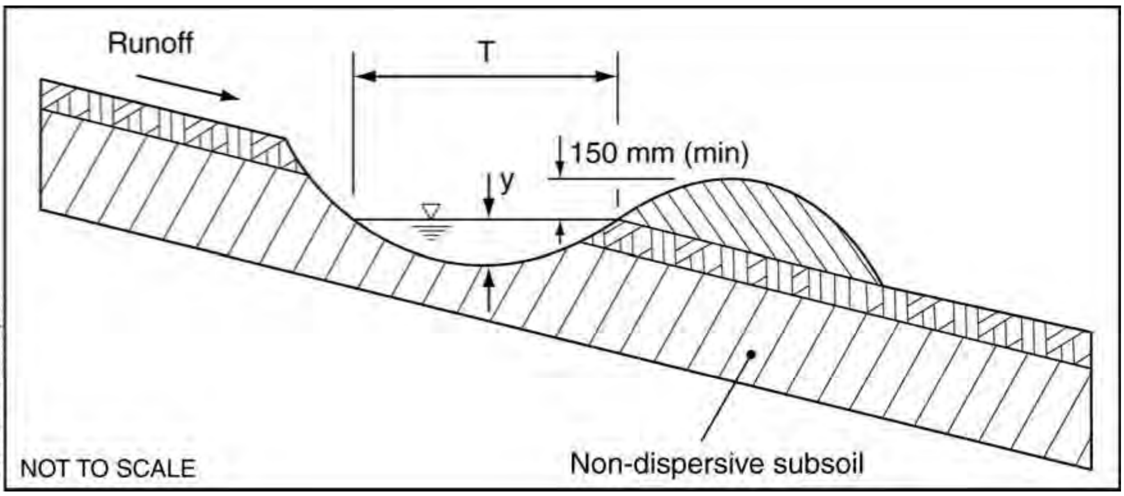


Figure 4(1) – Parabolic catch drain with bank

Table 9 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-A Catch Drain: Parabolic cross section										
Dimensions:				**Flow top width = 1.0 m**				**Flow depth = 0.15 m**		
Rainfall intensity (mm/hr)	**Allowable flow velocity along catch drain (m/s)**									
	0.3	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.5**	**2.0**
15	0.720	0.960	1.200	1.440	1.680	1.920	2.160	2.400	3.600	4.800
20	0.540	0.720	0.900	1.080	1.260	1.440	1.620	1.800	2.700	3.600
25	0.432	0.576	0.720	0.864	1.008	1.152	1.296	1.440	2.160	2.880
30	0.360	0.480	0.600	0.720	0.840	0.960	1.080	1.200	1.800	2.400
35	0.309	0.411	0.514	0.617	0.720	0.823	0.926	1.029	1.543	2.057
40	0.270	0.360	0.450	0.540	0.630	0.720	0.810	0.900	1.350	1.800
45	0.240	0.320	0.400	0.480	0.560	0.640	0.720	0.800	1.200	1.600
50	0.216	0.288	0.360	0.432	0.504	0.576	0.648	0.720	1.080	1.440
55	0.196	0.262	0.327	0.393	0.458	0.524	0.589	0.655	0.982	1.309
60	0.180	0.240	0.300	0.360	0.420	0.480	0.540	0.600	0.900	1.200
65	0.166	0.222	0.277	0.332	0.388	0.443	0.498	0.554	0.831	1.108
70	0.154	0.206	0.257	0.309	0.360	0.411	0.463	0.514	0.771	1.029
75	0.144	0.192	0.240	0.288	0.336	0.384	0.432	0.480	0.720	0.960
80	0.135	0.180	0.225	0.270	0.315	0.360	0.405	0.450	0.675	0.900
85	0.127	0.169	0.212	0.254	0.296	0.339	0.381	0.424	0.635	0.847
90	0.120	0.160	0.200	0.240	0.280	0.320	0.360	0.400	0.600	0.800
95	0.114	0.152	0.189	0.227	0.265	0.303	0.341	0.379	0.568	0.758
100	0.108	0.144	0.180	0.216	0.252	0.288	0.324	0.360	0.540	0.720
105	0.103	0.137	0.171	0.206	0.240	0.274	0.309	0.343	0.514	0.686
110	0.098	0.131	0.164	0.196	0.229	0.262	0.295	0.327	0.491	0.655
115	0.094	0.125	0.157	0.188	0.219	0.250	0.282	0.313	0.470	0.626
120	0.090	0.120	0.150	0.180	0.210	0.240	0.270	0.300	0.450	0.600
125	0.086	0.115	0.144	0.173	0.202	0.230	0.259	0.288	0.432	0.576
130	0.083	0.111	0.138	0.166	0.194	0.222	0.249	0.277	0.415	0.554
135	0.080	0.107	0.133	0.160	0.187	0.213	0.240	0.267	0.400	0.533
140	0.077	0.103	0.129	0.154	0.180	0.206	0.231	0.257	0.386	0.514
145	0.074	0.099	0.124	0.149	0.174	0.199	0.223	0.248	0.372	0.497
150	0.072	0.096	0.120	0.144	0.168	0.192	0.216	0.240	0.360	0.480
155	0.070	0.093	0.116	0.139	0.163	0.186	0.209	0.232	0.348	0.465
160	0.068	0.090	0.113	0.135	0.158	0.180	0.203	0.225	0.338	0.450
165	0.065	0.087	0.109	0.131	0.153	0.175	0.196	0.218	0.327	0.436
170	0.064	0.085	0.106	0.127	0.148	0.169	0.191	0.212	0.318	0.424
175	0.062	0.082	0.103	0.123	0.144	0.165	0.185	0.206	0.309	0.411
180	0.060	0.080	0.100	0.120	0.140	0.160	0.180	0.200	0.300	0.400
185	0.058	0.078	0.097	0.117	0.136	0.156	0.175	0.195	0.292	0.389
190	0.057	0.076	0.095	0.114	0.133	0.152	0.171	0.189	0.284	0.379
200	0.054	0.072	0.090	0.108	0.126	0.144	0.162	0.180	0.270	0.360
210	0.051	0.069	0.086	0.103	0.120	0.137	0.154	0.171	0.257	0.343
220	0.049	0.065	0.082	0.098	0.115	0.131	0.147	0.164	0.245	0.327
230	0.047	0.063	0.078	0.094	0.110	0.125	0.141	0.157	0.235	0.313
240	0.045	0.060	0.075	0.090	0.105	0.120	0.135	0.150	0.225	0.300
250	0.043	0.058	0.072	0.086	0.101	0.115	0.130	0.144	0.216	0.288
Q (m^3/s)	**0.030**	**0.040**	**0.050**	**0.060**	**0.070**	**0.080**	**0.090**	**0.100**	**0.150**	**0.200**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 8).

Table 10 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-B Catch Drain: Parabolic cross section

Dimensions:	Flow top width = 1.8 m				Flow depth = 0.3 m					
Rainfall intensity (mm/hr)	**Allowable flow velocity along catch drain (m/s)**									
	0.3	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.5**	**2.0**
15	2.592	3.456	4.320	5.184	6.048	6.912	7.776	8.640	12.960	17.280
20	1.944	2.592	3.240	3.888	4.536	5.184	5.832	6.480	9.720	12.960
25	1.555	2.074	2.592	3.110	3.629	4.147	4.666	5.184	7.776	10.368
30	1.296	1.728	2.160	2.592	3.024	3.456	3.888	4.320	6.480	8.640
35	1.111	1.481	1.851	2.222	2.592	2.962	3.333	3.703	5.554	7.406
40	0.972	1.296	1.620	1.944	2.268	2.592	2.916	3.240	4.860	6.480
45	0.864	1.152	1.440	1.728	2.016	2.304	2.592	2.880	4.320	5.760
50	0.778	1.037	1.296	1.555	1.814	2.074	2.333	2.592	3.888	5.184
55	0.707	0.943	1.178	1.414	1.649	1.885	2.121	2.356	3.535	4.713
60	0.648	0.864	1.080	1.296	1.512	1.728	1.944	2.160	3.240	4.320
65	0.598	0.798	0.997	1.196	1.396	1.595	1.794	1.994	2.991	3.988
70	0.555	0.741	0.926	1.111	1.296	1.481	1.666	1.851	2.777	3.703
75	0.518	0.691	0.864	1.037	1.210	1.382	1.555	1.728	2.592	3.456
80	0.486	0.648	0.810	0.972	1.134	1.296	1.458	1.620	2.430	3.240
85	0.457	0.610	0.762	0.915	1.067	1.220	1.372	1.525	2.287	3.049
90	0.432	0.576	0.720	0.864	1.008	1.152	1.296	1.440	2.160	2.880
95	0.409	0.546	0.682	0.819	0.955	1.091	1.228	1.364	2.046	2.728
100	0.389	0.518	0.648	0.778	0.907	1.037	1.166	1.296	1.944	2.592
105	0.370	0.494	0.617	0.741	0.864	0.987	1.111	1.234	1.851	2.469
110	0.353	0.471	0.589	0.707	0.825	0.943	1.060	1.178	1.767	2.356
115	0.338	0.451	0.563	0.676	0.789	0.902	1.014	1.127	1.690	2.254
120	0.324	0.432	0.540	0.648	0.756	0.864	0.972	1.080	1.620	2.160
125	0.311	0.415	0.518	0.622	0.726	0.829	0.933	1.037	1.555	2.074
130	0.299	0.399	0.498	0.598	0.698	0.798	0.897	0.997	1.495	1.994
135	0.288	0.384	0.480	0.576	0.672	0.768	0.864	0.960	1.440	1.920
140	0.278	0.370	0.463	0.555	0.648	0.741	0.833	0.926	1.389	1.851
145	0.268	0.358	0.447	0.536	0.626	0.715	0.804	0.894	1.341	1.788
150	0.259	0.346	0.432	0.518	0.605	0.691	0.778	0.864	1.296	1.728
155	0.251	0.334	0.418	0.502	0.585	0.669	0.753	0.836	1.254	1.672
160	0.243	0.324	0.405	0.486	0.567	0.648	0.729	0.810	1.215	1.620
165	0.236	0.314	0.393	0.471	0.550	0.628	0.707	0.785	1.178	1.571
170	0.229	0.305	0.381	0.457	0.534	0.610	0.686	0.762	1.144	1.525
175	0.222	0.296	0.370	0.444	0.518	0.592	0.667	0.741	1.111	1.481
180	0.216	0.288	0.360	0.432	0.504	0.576	0.648	0.720	1.080	1.440
185	0.210	0.280	0.350	0.420	0.490	0.560	0.630	0.701	1.051	1.401
190	0.205	0.273	0.341	0.409	0.477	0.546	0.614	0.682	1.023	1.364
200	0.194	0.259	0.324	0.389	0.454	0.518	0.583	0.648	0.972	1.296
210	0.185	0.247	0.309	0.370	0.432	0.494	0.555	0.617	0.926	1.234
220	0.177	0.236	0.295	0.353	0.412	0.471	0.530	0.589	0.884	1.178
230	0.169	0.225	0.282	0.338	0.394	0.451	0.507	0.563	0.845	1.127
240	0.162	0.216	0.270	0.324	0.378	0.432	0.486	0.540	0.810	1.080
250	0.156	0.207	0.259	0.311	0.363	0.415	0.467	0.518	0.778	1.037
Q (m^3/s)	**0.108**	**0.144**	**0.180**	**0.216**	**0.252**	**0.288**	**0.324**	**0.360**	**0.540**	**0.720**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 8).

Table 11 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-C Catch Drain: Parabolic cross section										
Dimensions:				**Flow top width = 3.0 m**				**Flow depth = 0.5 m**		
Rainfall intensity (mm/hr)	**Allowable flow velocity along catch drain (m/s)**									
	0.3	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.5**	**2.0**
15	7.200	9.600	12.000	14.400	16.800	19.200	21.600	24.000	36.000	48.000
20	5.400	7.200	9.000	10.800	12.600	14.400	16.200	18.000	27.000	36.000
25	4.320	5.760	7.200	8.640	10.080	11.520	12.960	14.400	21.600	28.800
30	3.600	4.800	6.000	7.200	8.400	9.600	10.800	12.000	18.000	24.000
35	3.086	4.114	5.143	6.171	7.200	8.229	9.257	10.286	15.429	20.571
40	2.700	3.600	4.500	5.400	6.300	7.200	8.100	9.000	13.500	18.000
45	2.400	3.200	4.000	4.800	5.600	6.400	7.200	8.000	12.000	16.000
50	2.160	2.880	3.600	4.320	5.040	5.760	6.480	7.200	10.800	14.400
55	1.964	2.618	3.273	3.927	4.582	5.236	5.891	6.545	9.818	13.091
60	1.800	2.400	3.000	3.600	4.200	4.800	5.400	6.000	9.000	12.000
65	1.662	2.215	2.769	3.323	3.877	4.431	4.985	5.538	8.308	11.077
70	1.543	2.057	2.571	3.086	3.600	4.114	4.629	5.143	7.714	10.286
75	1.440	1.920	2.400	2.880	3.360	3.840	4.320	4.800	7.200	9.600
80	1.350	1.800	2.250	2.700	3.150	3.600	4.050	4.500	6.750	9.000
85	1.271	1.694	2.118	2.541	2.965	3.388	3.812	4.235	6.353	8.471
90	1.200	1.600	2.000	2.400	2.800	3.200	3.600	4.000	6.000	8.000
95	1.137	1.516	1.895	2.274	2.653	3.032	3.411	3.789	5.684	7.579
100	1.080	1.440	1.800	2.160	2.520	2.880	3.240	3.600	5.400	7.200
105	1.029	1.371	1.714	2.057	2.400	2.743	3.086	3.429	5.143	6.857
110	0.982	1.309	1.636	1.964	2.291	2.618	2.945	3.273	4.909	6.545
115	0.939	1.252	1.565	1.878	2.191	2.504	2.817	3.130	4.696	6.261
120	0.900	1.200	1.500	1.800	2.100	2.400	2.700	3.000	4.500	6.000
125	0.864	1.152	1.440	1.728	2.016	2.304	2.592	2.880	4.320	5.760
130	0.831	1.108	1.385	1.662	1.938	2.215	2.492	2.769	4.154	5.538
135	0.800	1.067	1.333	1.600	1.867	2.133	2.400	2.667	4.000	5.333
140	0.771	1.029	1.286	1.543	1.800	2.057	2.314	2.571	3.857	5.143
145	0.745	0.993	1.241	1.490	1.738	1.986	2.234	2.483	3.724	4.966
150	0.720	0.960	1.200	1.440	1.680	1.920	2.160	2.400	3.600	4.800
155	0.697	0.929	1.161	1.394	1.626	1.858	2.090	2.323	3.484	4.645
160	0.675	0.900	1.125	1.350	1.575	1.800	2.025	2.250	3.375	4.500
165	0.655	0.873	1.091	1.309	1.527	1.745	1.964	2.182	3.273	4.364
170	0.635	0.847	1.059	1.271	1.482	1.694	1.906	2.118	3.176	4.235
175	0.617	0.823	1.029	1.234	1.440	1.646	1.851	2.057	3.086	4.114
180	0.600	0.800	1.000	1.200	1.400	1.600	1.800	2.000	3.000	4.000
185	0.584	0.778	0.973	1.168	1.362	1.557	1.751	1.946	2.919	3.892
190	0.568	0.758	0.947	1.137	1.326	1.516	1.705	1.895	2.842	3.789
200	0.540	0.720	0.900	1.080	1.260	1.440	1.620	1.800	2.700	3.600
210	0.514	0.686	0.857	1.029	1.200	1.371	1.543	1.714	2.571	3.429
220	0.491	0.655	0.818	0.982	1.145	1.309	1.473	1.636	2.455	3.273
230	0.470	0.626	0.783	0.939	1.096	1.252	1.409	1.565	2.348	3.130
240	0.450	0.600	0.750	0.900	1.050	1.200	1.350	1.500	2.250	3.000
250	0.432	0.576	0.720	0.864	1.008	1.152	1.296	1.440	2.160	2.880
Q (m^3/s)	**0.300**	**0.400**	**0.500**	**0.600**	**0.700**	**0.800**	**0.900**	**1.000**	**1.500**	**2.000**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 8).

Table 12 – Dimensions of standard triangular V-drains

Catch drain type	Max top width of flow (T)	Maximum flow depth (y)	Top width of formed drain	Depth of formed drain	Hyd. rad. (R) at max flow depth	Area (A) at max flow depth
Type-AV	1.0m	0.15m	2.0m	0.30m	0.072m	0.075m^2
Type-BV	1.8m	0.30m	2.7m	0.45m	0.142m	0.270m^2
Type-CV	3.0m	0.50m	3.9m	0.65m	0.237m	0.750m^2

Table 13 – Required longitudinal gradient (%) for unlined, triangular cross-section V-drains

Manning's roughness (n)	Allowable flow velocity along catch drain (m/s)									
	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.5	2.0
	Type-AV catch drain: flow top width (T) = 1.0 m and flow depth (Y) = 0.15 m									
Smooth soil n=0.02	0.12	0.21	0.33	0.48	0.66	0.86	1.08	1.34	3.01	5.36
Rough soil n=0.04	0.48	0.86	1.34	1.93	2.63	3.43	4.34	5.36	12.1	21.4
Very rough soil n=0.06	1.08	1.93	3.01	4.34	5.91	7.72	9.76	12.1	27.1	48.2
	Type-BV catch drain: flow top width (T) = 1.8 m and flow depth (Y) = 0.3 m									
Smooth soil n=0.02	0.048	0.086	0.13	0.19	0.26	0.34	0.44	0.54	1.21	2.15
Rough soil n=0.04	0.19	0.34	0.54	0.78	1.06	1.38	1.74	2.15	4.85	8.61
Very rough soil n=0.06	0.44	0.78	1.21	1.74	2.37	3.10	3.92	4.85	10.9	19.4
	Type-CV catch drain: flow top width (T) = 3.0 m and flow depth (Y) = 0.5 m									
Smooth soil n=0.02	0.025	0.044	0.068	0.10	0.13	0.17	0.22	0.27	0.61	1.09
Rough soil n=0.04	0.10	0.17	0.27	0.39	0.53	0.70	0.88	1.09	2.45	4.36
Very rough soil n=0.06	0.22	0.39	0.61	0.88	1.20	1.57	1.99	2.45	5.52	9.81

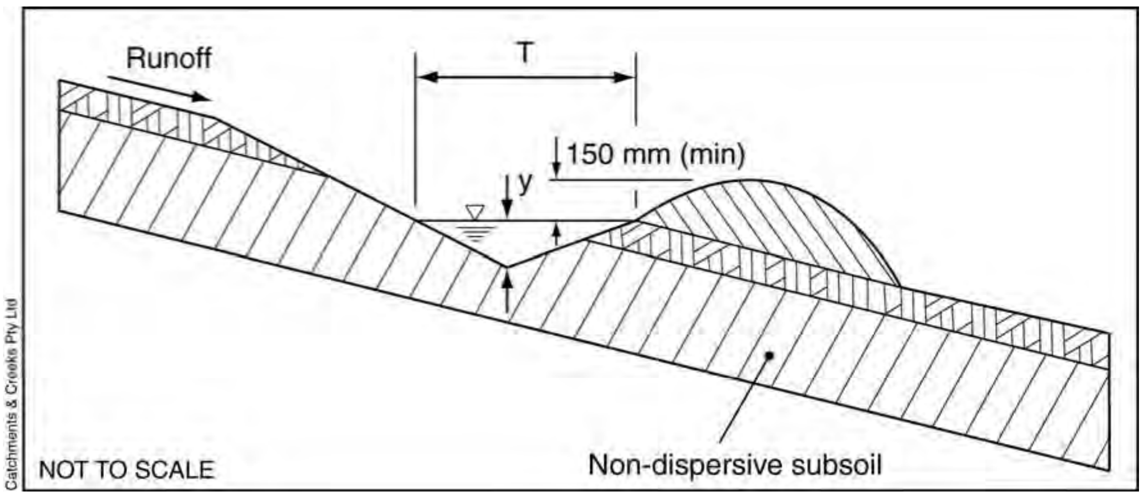


Figure 5(2) – Triangular V-drain with down-slope bank

Table 14 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-AV Catch Drain: V-drain cross section										
Dimensions:				**Flow top width = 1.0 m**				**Flow depth = 0.15 m**		
Rainfall intensity (mm/hr)	**Allowable flow velocity along catch drain (m/s)**									
	0.3	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.5**	**2.0**
15	0.540	0.720	0.900	1.080	1.260	1.440	1.620	1.800	2.700	3.600
20	0.405	0.540	0.675	0.810	0.945	1.080	1.215	1.350	2.025	2.700
25	0.324	0.432	0.540	0.648	0.756	0.864	0.972	1.080	1.620	2.160
30	0.270	0.360	0.450	0.540	0.630	0.720	0.810	0.900	1.350	1.800
35	0.231	0.309	0.386	0.463	0.540	0.617	0.694	0.771	1.157	1.543
40	0.203	0.270	0.338	0.405	0.473	0.540	0.608	0.675	1.013	1.350
45	0.180	0.240	0.300	0.360	0.420	0.480	0.540	0.600	0.900	1.200
50	0.162	0.216	0.270	0.324	0.378	0.432	0.486	0.540	0.810	1.080
55	0.147	0.196	0.245	0.295	0.344	0.393	0.442	0.491	0.736	0.982
60	0.135	0.180	0.225	0.270	0.315	0.360	0.405	0.450	0.675	0.900
65	0.125	0.166	0.208	0.249	0.291	0.332	0.374	0.415	0.623	0.831
70	0.116	0.154	0.193	0.231	0.270	0.309	0.347	0.386	0.579	0.771
75	0.108	0.144	0.180	0.216	0.252	0.288	0.324	0.360	0.540	0.720
80	0.101	0.135	0.169	0.203	0.236	0.270	0.304	0.338	0.506	0.675
85	0.095	0.127	0.159	0.191	0.222	0.254	0.286	0.318	0.476	0.635
90	0.090	0.120	0.150	0.180	0.210	0.240	0.270	0.300	0.450	0.600
95	0.085	0.114	0.142	0.171	0.199	0.227	0.256	0.284	0.426	0.568
100	0.081	0.108	0.135	0.162	0.189	0.216	0.243	0.270	0.405	0.540
105	0.077	0.103	0.129	0.154	0.180	0.206	0.231	0.257	0.386	0.514
110	0.074	0.098	0.123	0.147	0.172	0.196	0.221	0.245	0.368	0.491
115	0.070	0.094	0.117	0.141	0.164	0.188	0.211	0.235	0.352	0.470
120	0.068	0.090	0.113	0.135	0.158	0.180	0.203	0.225	0.338	0.450
125	0.065	0.086	0.108	0.130	0.151	0.173	0.194	0.216	0.324	0.432
130	0.062	0.083	0.104	0.125	0.145	0.166	0.187	0.208	0.312	0.415
135	0.060	0.080	0.100	0.120	0.140	0.160	0.180	0.200	0.300	0.400
140	0.058	0.077	0.096	0.116	0.135	0.154	0.174	0.193	0.289	0.386
145	0.056	0.074	0.093	0.112	0.130	0.149	0.168	0.186	0.279	0.372
150	0.054	0.072	0.090	0.108	0.126	0.144	0.162	0.180	0.270	0.360
155	0.052	0.070	0.087	0.105	0.122	0.139	0.157	0.174	0.261	0.348
160	0.051	0.068	0.084	0.101	0.118	0.135	0.152	0.169	0.253	0.338
165	0.049	0.065	0.082	0.098	0.115	0.131	0.147	0.164	0.245	0.327
170	0.048	0.064	0.079	0.095	0.111	0.127	0.143	0.159	0.238	0.318
175	0.046	0.062	0.077	0.093	0.108	0.123	0.139	0.154	0.231	0.309
180	0.045	0.060	0.075	0.090	0.105	0.120	0.135	0.150	0.225	0.300
185	0.044	0.058	0.073	0.088	0.102	0.117	0.131	0.146	0.219	0.292
190	0.043	0.057	0.071	0.085	0.099	0.114	0.128	0.142	0.213	0.284
200	0.041	0.054	0.068	0.081	0.095	0.108	0.122	0.135	0.203	0.270
210	0.039	0.051	0.064	0.077	0.090	0.103	0.116	0.129	0.193	0.257
220	0.037	0.049	0.061	0.074	0.086	0.098	0.110	0.123	0.184	0.245
230	0.035	0.047	0.059	0.070	0.082	0.094	0.106	0.117	0.176	0.235
240	0.034	0.045	0.056	0.068	0.079	0.090	0.101	0.113	0.169	0.225
250	0.032	0.043	0.054	0.065	0.076	0.086	0.097	0.108	0.162	0.216
Q (m^3/s)	**0.023**	**0.030**	**0.038**	**0.045**	**0.053**	**0.060**	**0.068**	**0.075**	**0.113**	**0.150**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 13).

Table 15 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-BV Catch Drain: V-drain cross section										
Dimensions:				Flow top width = 1.8 m				Flow depth = 0.3 m		
Rainfall intensity (mm/hr)	Allowable flow velocity along catch drain (m/s)									
	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.5	2.0
15	1.944	2.592	3.240	3.888	4.536	5.184	5.832	6.480	9.720	12.960
20	1.458	1.944	2.430	2.916	3.402	3.888	4.374	4.860	7.290	9.720
25	1.166	1.555	1.944	2.333	2.722	3.110	3.499	3.888	5.832	7.776
30	0.972	1.296	1.620	1.944	2.268	2.592	2.916	3.240	4.860	6.480
35	0.833	1.111	1.389	1.666	1.944	2.222	2.499	2.777	4.166	5.554
40	0.729	0.972	1.215	1.458	1.701	1.944	2.187	2.430	3.645	4.860
45	0.648	0.864	1.080	1.296	1.512	1.728	1.944	2.160	3.240	4.320
50	0.583	0.778	0.972	1.166	1.361	1.555	1.750	1.944	2.916	3.888
55	0.530	0.707	0.884	1.060	1.237	1.414	1.591	1.767	2.651	3.535
60	0.486	0.648	0.810	0.972	1.134	1.296	1.458	1.620	2.430	3.240
65	0.449	0.598	0.748	0.897	1.047	1.196	1.346	1.495	2.243	2.991
70	0.417	0.555	0.694	0.833	0.972	1.111	1.250	1.389	2.083	2.777
75	0.389	0.518	0.648	0.778	0.907	1.037	1.166	1.296	1.944	2.592
80	0.365	0.486	0.608	0.729	0.851	0.972	1.094	1.215	1.823	2.430
85	0.343	0.457	0.572	0.686	0.800	0.915	1.029	1.144	1.715	2.287
90	0.324	0.432	0.540	0.648	0.756	0.864	0.972	1.080	1.620	2.160
95	0.307	0.409	0.512	0.614	0.716	0.819	0.921	1.023	1.535	2.046
100	0.292	0.389	0.486	0.583	0.680	0.778	0.875	0.972	1.458	1.944
105	0.278	0.370	0.463	0.555	0.648	0.741	0.833	0.926	1.389	1.851
110	0.265	0.353	0.442	0.530	0.619	0.707	0.795	0.884	1.325	1.767
115	0.254	0.338	0.423	0.507	0.592	0.676	0.761	0.845	1.268	1.690
120	0.243	0.324	0.405	0.486	0.567	0.648	0.729	0.810	1.215	1.620
125	0.233	0.311	0.389	0.467	0.544	0.622	0.700	0.778	1.166	1.555
130	0.224	0.299	0.374	0.449	0.523	0.598	0.673	0.748	1.122	1.495
135	0.216	0.288	0.360	0.432	0.504	0.576	0.648	0.720	1.080	1.440
140	0.208	0.278	0.347	0.417	0.486	0.555	0.625	0.694	1.041	1.389
145	0.201	0.268	0.335	0.402	0.469	0.536	0.603	0.670	1.006	1.341
150	0.194	0.259	0.324	0.389	0.454	0.518	0.583	0.648	0.972	1.296
155	0.188	0.251	0.314	0.376	0.439	0.502	0.564	0.627	0.941	1.254
160	0.182	0.243	0.304	0.365	0.425	0.486	0.547	0.608	0.911	1.215
165	0.177	0.236	0.295	0.353	0.412	0.471	0.530	0.589	0.884	1.178
170	0.172	0.229	0.286	0.343	0.400	0.457	0.515	0.572	0.858	1.144
175	0.167	0.222	0.278	0.333	0.389	0.444	0.500	0.555	0.833	1.111
180	0.162	0.216	0.270	0.324	0.378	0.432	0.486	0.540	0.810	1.080
185	0.158	0.210	0.263	0.315	0.368	0.420	0.473	0.525	0.788	1.051
190	0.153	0.205	0.256	0.307	0.358	0.409	0.460	0.512	0.767	1.023
200	0.146	0.194	0.243	0.292	0.340	0.389	0.437	0.486	0.729	0.972
210	0.139	0.185	0.231	0.278	0.324	0.370	0.417	0.463	0.694	0.926
220	0.133	0.177	0.221	0.265	0.309	0.353	0.398	0.442	0.663	0.884
230	0.127	0.169	0.211	0.254	0.296	0.338	0.380	0.423	0.634	0.845
240	0.122	0.162	0.203	0.243	0.284	0.324	0.365	0.405	0.608	0.810
250	0.117	0.156	0.194	0.233	0.272	0.311	0.350	0.389	0.583	0.778
Q (m^3/s)	0.081	0.108	0.135	0.162	0.189	0.216	0.243	0.270	0.405	0.540

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 13).

Table 16 – Maximum allowable unit catchment area (A*, hectares) [1]

Type-CV Catch Drain: V-drain cross section										
Dimensions:				**Flow top width = 3.0 m**				**Flow depth = 0.5 m**		
Rainfall intensity (mm/hr)	**Allowable flow velocity along catch drain (m/s)**									
	0.3	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.5**	**2.0**
15	5.400	7.200	9.000	10.800	12.600	14.400	16.200	18.000	27.000	36.000
20	4.050	5.400	6.750	8.100	9.450	10.800	12.150	13.500	20.250	27.000
25	3.240	4.320	5.400	6.480	7.560	8.640	9.720	10.800	16.200	21.600
30	2.700	3.600	4.500	5.400	6.300	7.200	8.100	9.000	13.500	18.000
35	2.314	3.086	3.857	4.629	5.400	6.171	6.943	7.714	11.571	15.429
40	2.025	2.700	3.375	4.050	4.725	5.400	6.075	6.750	10.125	13.500
45	1.800	2.400	3.000	3.600	4.200	4.800	5.400	6.000	9.000	12.000
50	1.620	2.160	2.700	3.240	3.780	4.320	4.860	5.400	8.100	10.800
55	1.473	1.964	2.455	2.945	3.436	3.927	4.418	4.909	7.364	9.818
60	1.350	1.800	2.250	2.700	3.150	3.600	4.050	4.500	6.750	9.000
65	1.246	1.662	2.077	2.492	2.908	3.323	3.738	4.154	6.231	8.308
70	1.157	1.543	1.929	2.314	2.700	3.086	3.471	3.857	5.786	7.714
75	1.080	1.440	1.800	2.160	2.520	2.880	3.240	3.600	5.400	7.200
80	1.013	1.350	1.688	2.025	2.363	2.700	3.038	3.375	5.063	6.750
85	0.953	1.271	1.588	1.906	2.224	2.541	2.859	3.176	4.765	6.353
90	0.900	1.200	1.500	1.800	2.100	2.400	2.700	3.000	4.500	6.000
95	0.853	1.137	1.421	1.705	1.989	2.274	2.558	2.842	4.263	5.684
100	0.810	1.080	1.350	1.620	1.890	2.160	2.430	2.700	4.050	5.400
105	0.771	1.029	1.286	1.543	1.800	2.057	2.314	2.571	3.857	5.143
110	0.736	0.982	1.227	1.473	1.718	1.964	2.209	2.455	3.682	4.909
115	0.704	0.939	1.174	1.409	1.643	1.878	2.113	2.348	3.522	4.696
120	0.675	0.900	1.125	1.350	1.575	1.800	2.025	2.250	3.375	4.500
125	0.648	0.864	1.080	1.296	1.512	1.728	1.944	2.160	3.240	4.320
130	0.623	0.831	1.038	1.246	1.454	1.662	1.869	2.077	3.115	4.154
135	0.600	0.800	1.000	1.200	1.400	1.600	1.800	2.000	3.000	4.000
140	0.579	0.771	0.964	1.157	1.350	1.543	1.736	1.929	2.893	3.857
145	0.559	0.745	0.931	1.117	1.303	1.490	1.676	1.862	2.793	3.724
150	0.540	0.720	0.900	1.080	1.260	1.440	1.620	1.800	2.700	3.600
155	0.523	0.697	0.871	1.045	1.219	1.394	1.568	1.742	2.613	3.484
160	0.506	0.675	0.844	1.013	1.181	1.350	1.519	1.688	2.531	3.375
165	0.491	0.655	0.818	0.982	1.145	1.309	1.473	1.636	2.455	3.273
170	0.476	0.635	0.794	0.953	1.112	1.271	1.429	1.588	2.382	3.176
175	0.463	0.617	0.771	0.926	1.080	1.234	1.389	1.543	2.314	3.086
180	0.450	0.600	0.750	0.900	1.050	1.200	1.350	1.500	2.250	3.000
185	0.438	0.584	0.730	0.876	1.022	1.168	1.314	1.459	2.189	2.919
190	0.426	0.568	0.711	0.853	0.995	1.137	1.279	1.421	2.132	2.842
200	0.405	0.540	0.675	0.810	0.945	1.080	1.215	1.350	2.025	2.700
210	0.386	0.514	0.643	0.771	0.900	1.029	1.157	1.286	1.929	2.571
220	0.368	0.491	0.614	0.736	0.859	0.982	1.105	1.227	1.841	2.455
230	0.352	0.470	0.587	0.704	0.822	0.939	1.057	1.174	1.761	2.348
240	0.338	0.450	0.563	0.675	0.788	0.900	1.013	1.125	1.688	2.250
250	0.324	0.432	0.540	0.648	0.756	0.864	0.972	1.080	1.620	2.160
Q (m^3/s)	**0.225**	**0.300**	**0.375**	**0.450**	**0.525**	**0.600**	**0.675**	**0.750**	**1.125**	**1.500**

[1] Catchment areas are based on the drain being formed at the required longitudinal gradient (Table 13).

Check Dam

Check Dams

DRAINAGE CONTROL TECHNIQUE

Low Gradient	✓	Velocity Control	✓	Short Term	✓
Steep Gradient		Channel Lining		Medium-Long Term	✓
Outlet Control		Soil Treatment		Permanent	[1]

[1] Though not generally considered as permanent structures within drainage channels, rock check dams have been used in stormwater treatment swales to improve retention time and increase sedimentation. Permanent rock check dams can also be used to form a stable, terraced invert within mild-sloping (<10%) table drains. Permanent checks dams, however, can cause mowing problems.

Symbol (refer to Table 2)

Photo 1 – Sandbag check dams

Photo 2 – Rock check dam

Key Principles

1. The primary function of check dams is to control flow velocities within unlined drains. Most check dams, however, will also trap small quantities of sediment, thus allowing these structures to act as both *drainage* and *sediment* control devices.
2. Sediment control does **not** have to be considered a performance objective in all cases.
3. Hydraulic performance is governed by the height and spacing of the dams. The spacing of check dams down a drain varies with the slope of the drain and the height of each dam.
4. It is critical to ensure the check dams do not cause flow to unnecessarily spill out of the drain possibly resulting in flooding or erosion problems.
5. The crest of the check should be curved such that flow first spills over the centre of the dam. Use of a flat crest profile can cause erosion (rilling) down the banks of the drain.

Design Information

Table 2 provides guidance on the attributes and typical usage of various types of check dams, it is summarised in Table 1.

Table 1 – Summary of technique selection

Type of check dam	Typical conditions of use
Fibre rolls, Triangular & Sandbag check dam	• Drains less than 500mm deep
Rock check dam	• Drains more than 500mm deep
Compost-filled bags	• Situations where velocity control and enhanced stormwater treatment (filtration and adsorption) is required

Table 2 – Typical use of the various types of check dams

Technique	Code	Symbol [1]	Attributes and typical usage
Fibre rolls	FCD	→ FCD →	• Biodegradable (jute/coir) logs. • Used in wide, shallow drains where the logs can be successfully anchored down. • Used in locations where it is desirable to allow the fibre roll to integrate into the vegetation, such as vegetated channels. • Can be used as a minor sediment trap.
Rock check dams	RCD	→ RCD →	• Constructed from 150 to 300mm rock. • Best used **only** in drains at least 500mm deep, with a gradient less than 10%. • Should only be used in locations where it is known that they will be removed once a suitable grass cover has been established. • Can also be used as a minor sediment trap.
Recessed rock check dams	RRC	→ RRC →	• Constructed from minimum 200mm rock. • Used in wide, shallow, high velocity channels to prevent uncontrolled gully erosion during the revegetation period. • These are specialist hydraulic structures requiring specialist knowledge for their proper usage.
Sandbag check dams (including compost-filled bags)	SBC	→ SBC →	• Sandbags are typically filled with sand, aggregate, gravel, or compost. • Compost filled bags are considered to provide improved water treatment through filtration and adsorption. This system included compost-filled *Filter Socks*. • Typically used in drains less than 500mm deep, with a gradient less than 10%. • These check dams are typically small (in height) and therefore less likely to divert water out of the drain. • Can be used as a minor sediment trap.
Stiff grass barriers	SGB	▬ SGB ▬	• Requires long establishment times. • Typically used as a component of long-term gully stabilisation in rural areas. • Most suited to sandy soils. • Can be used as a minor sediment trap.
Triangular ditch checks	TDC	→ TDC →	• Manufactured from re-useable, porous, solid frame, PVC mesh. • Commonly used to stabilise newly formed, wide, shallow drains. • Used in drains with less than 10% gradient. • Can be used as a minor sediment trap.

[1] The check dam symbol is usually not used on ESC plans; instead the use of check dams is normally specified within technical notes listed on the plans. A table may be included within the ESCP to provide details on the type of check dam used at specific locations within the site.

Typical maximum channel gradient of 10% (1 in 10). Preference should be given to the use of a suitable channel lining if the drain or chute is steeper than 10%.

Check dams are spaced down the drain such that the crest of the check dam is level with the toe of the immediate upstream check dam (as shown in Figures 1).

Maximum recommended crest height of around 500mm. Check dams with a height exceeding 500mm should be checked for hydraulic stability.

Maximum slope of the face of rock check dams is 2:1 (H:V). For check dams higher than 500mm, the slope of the **downstream** face may need to be significantly flatter than a 2:1.

The crest of the check dam should be curved such that flow first spills over the centre of the dam. Ideally, the crest of each dam should be at least 150mm lower than the bank elevation at the outer edges of the structure.

The purpose of a curved crest profile is to:

- minimise the quantity of water bypassing around the edge of the check dam; and
- to concentrate flow into the centre of the channel.

Use of a flat crest profile can cause erosion (rilling) down the banks of the drain.

For sandbag check dams placed in shallow profile drainage channels, such as some table drains, it may be necessary to remove one or two sandbags from the centre of the structure (refer to Photo 3) to promote flow at the centre of the drain. The sandbags may also need to be placed in a curved (concave) horizontal profile to minimise flow bypassing around the ends of the dam (this can also be seen in Photo 3).

Check dams should not be used to control erosion within drains formed from dispersive soil (Photos 9 & 10). In such cases, the exposed dispersive soil should be covered with non-dispersive soil, then stabilised with an appropriate channel liner.

In circumstance where the use of check dams could cause such a significant reduction in the drain's hydraulic capacity to force water out of the drain resulting in either traffic safety issues (table drains) or flooding of adjacent properties, then the design options are:

- select an appropriate channel lining such that the use of check dams within the drain will no longer be required;
- perform an appropriate hydraulic analysis on the check dams to ensure that adequate hydraulic performance of the drain is maintained (refer over-page for guidance on such hydraulic analysis).

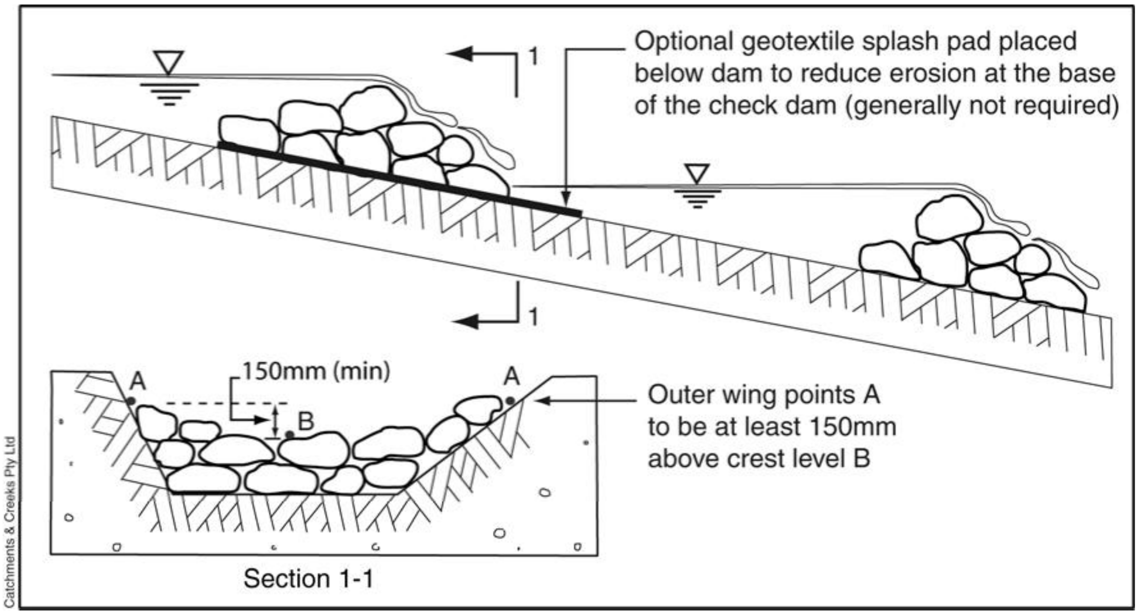


Figure 1 – Profile of temporary check dams

Photo 3 – Sandbag check dam

Photo 4 – Fibre rolls

Photo 5 – Triangular ditch checks

Photo 6 – Stiff grass barrier (background)

Photo 7 – Poor placement of rocks, note rocks are higher in centre of check dam

Photo 8 – Retained rock check dams can interfere with ongoing mowing

Photo 9 – Typical erosion problem when placed in dispersive soil

Photo 10 – Typical erosion problem when placed in dispersive soil

Erosion control at toe of check dams:

Erosion downstream of each check dam will be minimised if the dams are correctly spaced such that the crest of each dam is level with the toe of the nearest upstream dam.

Where necessary, the risk of erosion at the toe of each check dam may reduced by constructing each check dam on a sheet of geotextile fabric (e.g. filter cloth or woven fabric) that extends downstream of the dam a distance at least equal to the height of the dam (Figure 1).

Hydraulic design:

In general, a hydraulic analysis is not normally performed on check dams as their use should be restricted to those locations where they are unlikely to cause hydraulic problems. However, in circumstance where use of check dams could cause either traffic safety issues (table drains) or flooding of adjacent properties, then a hydraulic analysis will be required.

As a quick check, Table 3 can be used to assess the hydraulic capacity of a proposed check dam. Table 3 provides the maximum discharge for a given maximum water level (H) and check dam width (W). The table is based on a check dam with a **flat crested**, trapezoidal weir profile with side slopes of 1 in 2 (Figure 2) using Equation 1.

$$Q = 1.7\,W\,H^{1.5} + 2.5\,H^{2.5} \qquad \text{(Eqn 1)}$$

Table 3 – Assumed hydraulic capacity of check dam [1] (m^3/s)

Allowable upstream head (H) metres	Check dam flat crest width (W) metres				
	1.0	**1.5**	**2.0**	**2.5**	**3.0**
0.1	0.06	0.09	0.12	0.14	0.17
0.2	0.20	0.27	0.35	0.43	0.50
0.3	0.40	0.54	0.68	0.82	0.96
0.4	0.69	0.90	1.12	1.33	1.55
0.5	1.05	1.35	1.65	1.95	2.25
0.6	1.49	1.89	2.28	2.68	3.07
0.7	2.03	2.53	3.02	3.52	4.02
0.8	2.66	3.27	3.88	4.48	5.09
0.9	3.39	4.11	4.84	5.57	6.29
1.0	4.22	5.07	5.92	6.77	7.62

[1] Hydraulics is based on a flat crested, trapezoidal weir profile with a side slope of 2:1 (H:V).

If the side slopes of the drainage channel is not 2:1 (H:V), then the appropriate weir equation is:

$$Q = 1.7\,W\,H^{1.5} + 1.26\,m\,H^{2.5} \qquad \text{(Eqn 2)}$$

where:

- Q = Discharge passing over the check dam (m^3/s)
- W = Crest width of the check dam crest (m)
- H = Upstream water head relative to the crest of the check dam (m)
- m = Channel side slope, m:1 (H:V)

Both Equations 1 and 2 assume a flat crested weir profile; however, it is a requirement that check dams must have a curved crest with a minimum 150mm depression (Figure 1). Thus, Equations 1 and 2, and Table 3, all <u>overestimate</u> the hydraulic capacity of check dams. Therefore, a conservative design approach is required.

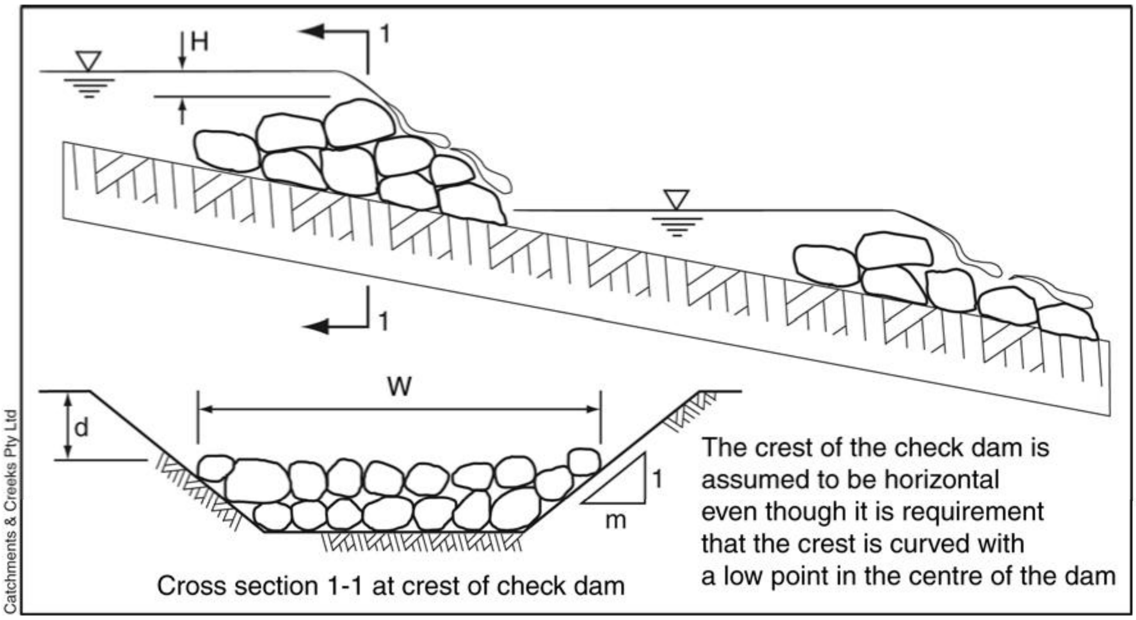


Figure 2 – Assumed check dam profile for Equations 1 and 2

Design example 1:

Determine the maximum allowable height of rock check dams placed along a channel that has a base width of 1.0m and side slopes of 3:1 (m:1). The total depth of channel is 0.7m and the required flow rate is $0.4m^3/s$. (note; this is the required allowable flow rate during the operational phase of the check dams, which may be different from that specified for design of the drain, especially if the drain is a permanent structure).

Solution:

The difficulty here is that the crest width of the check dam (W) will vary with the height of the dam, which is the variable that we are trying to determine. Therefore we will need to answer this question using a trial and error process.

As a first guess, try the maximum recommended check dam height of 0.5m. This means the maximum allowable upstream head (H) is 0.7 - 0.5 = 0.2m.

Thus the check dam crest width is:

$$W = \text{(bed width of channel)} + 2.\text{(side slope, m)}.\text{(height of check dam)}$$

$$W = 1.0 + 2(3)(0.5) = 4\text{m}$$

Using Equation 2, the maximum allowable discharge (i.e when H = 0.2m) is:

$$Q = 1.7\,W\,H^{1.5} + 1.26\,m\,H^{2.5} = 1.7(4)(0.2)^{1.5} + 1.26(3)(0.2)^{2.5} = 0.68m^3/s > 0.4m^3/s$$

Therefore the available hydraulic capacity of $0.68m^3/s$ is greater than the required hydraulic capacity of only $0.4m^3/s$, thus the check dam height will be limited to the maximum recommended height of 0.5m.

Design example 2:

Determine the maximum allowable flow rate (Q) for a check dam in a drainage channel with side slopes of 2:1; check dam crest width, W = 2m; and maximum allowable upstream hydraulic head, H = 0.4m.

Solution:

Given the side slope is 2:1 (H:V), we can use Table 3 to answer this question. From Table 3 it can be seen that the maximum allowable flow rate is around, $Q = 1.12m^3/s$ (note, Table 3 overestimates the available hydraulic capacity if the check dam has a curved, U-shaped crest).

Stiff grass barriers:

Stiff grass barriers (Figure 3) are typically used as a component of long-term gully stabilisation in rural areas. The most common grass species is the sterile form of vetiver zizanoides.

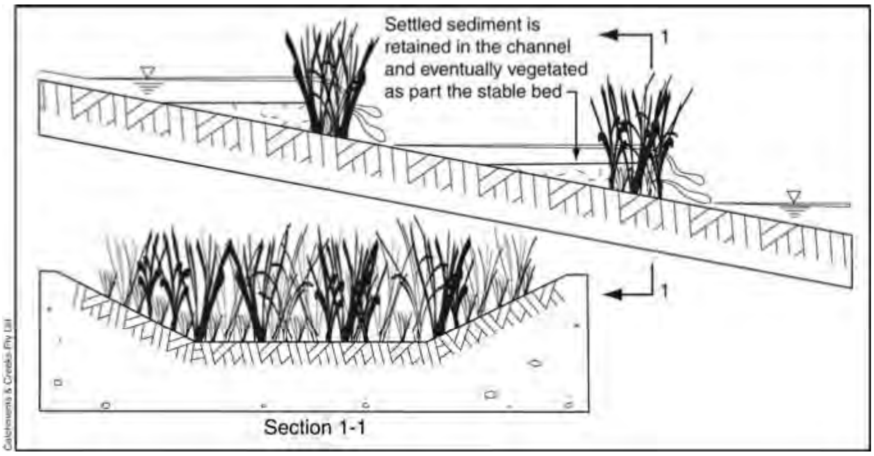


Figure 3 – Stiff grass barriers

Recessed rock check dams:

Recessed rock check dams can be used to:

- Control flow velocities in wide, shallow channels (typically less than 500mm deep) where other types of check dams, such as sandbags, are expected to wash away. In such cases the check dams are partially recessed into the channel bed.
- Control flow velocities and erosion in high velocity channels where a large rock size (greater than 300mm) is required, but the channel is too shallow to accommodate such rocks being placed directly on the channel bed. In such cases the check dams are partially recessed into the channel bed.
- Limit potential future gully erosion within constructed waterways and vegetated drainage channels. In such cases the rocks are recessed into the bed of the channel so that the top of each check dam is just below the bed of the channel (Figure 4).

In this latter case, the recessed rock checks (these are technically not 'dams') are used as an 'insurance policy' against possible future channel erosion, especially during the vegetation establishment phase when the channel roughness is significantly less than the assumed ultimate condition. The intension is to limit the extent and depth of any channel erosion between each recessed check structure. If erosion does not occur, then the check dams remain buried and incorporated into the stable channel profile.

Following installation of the recessed rock checks, the rocks are covered with soil (including the filling of all voids) and vegetated to fully incorporate the rock into the channel.

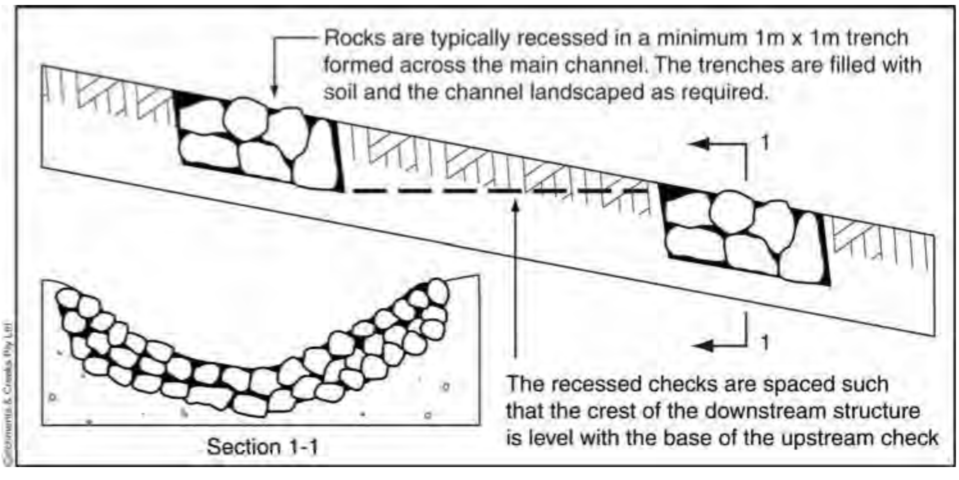


Figure 4 – Fully recessed rock check dams

Description

Check dams can be constructed from semipervious or impervious materials, typically rock or sandbags filled with a variety of porous materials.

Check dams should **not** be constructed from straw bales.

Rock check dams may be recessed into the channel bed to allow the use of larger sized rock, and/or to limit the crest height of the dams.

Purpose

Used to reduce flow velocity and the resulting erosion within:

- temporary, open earth channels;
- permanent vegetated channels during the plant establishment phase.

Check dam can also provide limited sediment trapping ability, but usually as a secondary function.

Limitations

Check dams are normally limited to mild sloping channels less than 10% grade.

Typical maximum height of 500mm.

Generally not used in watercourses. Instead, consider the used on *Sediment Weirs, Rock Filter Dams*, or formally designed rock weirs or drop structures.

Should not be placed directly on dispersive soils, or within drains cut into dispersive soils.

Advantages

Quick and inexpensive to install.

Low maintenance.

Disadvantages

Rock check dams can cause damage to grass cutting equipment if not removed from the channel after vegetation has been established (Photo 8).

Common Problems

Hydraulic problems often occur when rock check dams are specified in shallow drains.

Erosion can occur around the edges of the check dams, especially if installed with a flat crest.

Inappropriate spacing of the dams. This usually results from inadequate installation information supplied on the ESCPs.

Special Requirements

If soils are highly erosive (but not dispersive), then consider the use of an underlying geotextile skirt placed under each check dam (Figure 1).

Appropriate care must be taken to prevent failure caused by water undermining or bypassing round the dams.

Site Inspection

Check for invert erosion within the channel being stabilised with check dams.

Ensure the type of check dam is appropriate for the flow conditions and type of drainage channel.

Ensure the crest is below the height of the outer wings of the dams (refer to Figure 1).

Ensure the dams are appropriately spaced.

Materials

- Rock: 150 to 300mm nominal diameter, hard, erosion resistant rock. Smaller rock may be used if suitable large rock is not available.
- Sandbags: geotextile bags (woven synthetic, or non-woven biodegradable) filled with clean coarse sand, clean aggregate, straw or compost.

Installation

1. Refer to approved plans for location and installation details. If there are questions or problems with the location or method of installation, contact the engineer or responsible on-site officer for assistance.
2. Prior to placement of the check dams, ensure the type and size of each check dams will not cause a safety hazard or cause water to spill out of the drain.
3. Locate the first check dam at the downstream end of the section of channel being protected. Locate each successive check dam such that the crest of the immediate downstream dam is level with the toe of the check dam being installed.
4. Ensure the channel slope is no steeper than 10:1 (H:V). Otherwise consider the use of a suitable channel liner instead of the check dams.
5. Construct the check dam to the dimensions and profile shown within the approved plan.

6. Where specified, the check dams shall be constructed on a sheet of geotextile fabric used as a downstream splash pad.
7. Each check dam shall be extended up the channel bank (where practicable) to an elevation at least 150mm above the crest level of the dam.

Maintenance

1. Inspect each check dam and the drainage channel at least weekly and after runoff-producing rainfall.
2. Correct all damage immediately. If significant erosion occurs between any of the check dams, then check the spacing of dams and where necessary install intermediate check dams or a suitable channel liner.
3. Check for displacement of the check dams
4. Check for soil scour around the ends of each check dam. If such erosion is occurring, consider extending the width of the check dam to avoid such problems.
5. If severe soil erosion occurs either under or around the check dams, then seek expert advice on an alternative treatment measure.
6. Remove any sediment accumulated by the check dams, unless it is intended that this sediment will remain within the channel.
7. Dispose of collected sediment in a suitable manner that will not cause an erosion or pollution hazard.

Removal

1. When construction work within the drainage area above the check dams has been completed, and the disturbed areas and the drainage channel are sufficiently stabilised to restrain erosion, all temporary check dams must be removed.
2. Remove the check dams and associated sediment and dispose of in a suitable manner that will not cause an erosion or pollution hazard.

Level Spreader

Level Spreaders

DRAINAGE CONTROL TECHNIQUE

Low Gradient	✓	Velocity Control		Short Term	✓
Steep Gradient	[1]	Channel Lining		Medium-Long Term	✓
Outlet Control	✓	Soil Treatment		Permanent	✓

[1] Level spreaders can release sheet flow down steep slopes, but the level spreader itself must be constructed across a level gradient.

Symbol

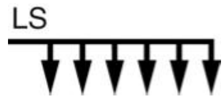


Photo 1 – Diversion drains (centre) collect stormwater from roadside table drains, then releases the water as sheet flow via a level spreader

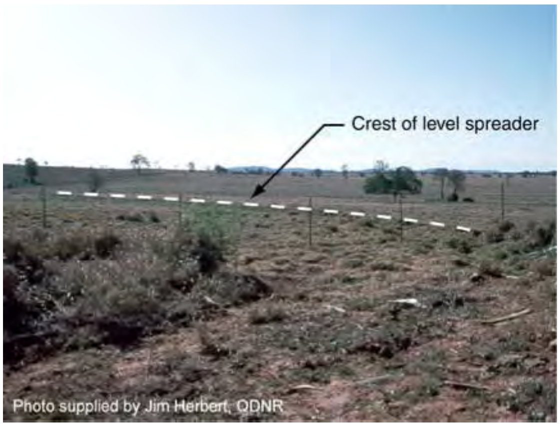


Photo 2 – Level spreader established to discharge stormwater from a diversion drain into the roadside property

Key Principles

1. Flow must be released from the level spreader as *sheet flow*.
2. Flow must be released over a stable, well-grassed surface that will maintain suitable flow conditions down the slope.
3. Critical design parameter is the length of the outlet sill.
4. Critical operational parameter is the level construction of the outlet sill.

Design Information

The length of the outlet sill (weir) of the level spreader is governed by the design discharge, and the allowable flow velocity of the down-slope area.

Allowable flow velocity for grassed surfaces can be determined from Table 1.

Minimum dimension can be determined from Tables 2 and 3.

Minimum sill length is 4m.

Maximum sill length is 25m. If a longer sill length is required, then the inflow must be spilt and released through more than one level spreader.

Up-slope channel grade should not exceed 1% for the last 6m before entering the level spreader.

Discharge must release evenly along a level surface (sill) of 0% cross gradient.

Caution the use of a design discharge exceeding 0.85m ^{3}/s.

Caution the release of water onto grass slopes steeper than 10%.

Table 1 – Allowable flow velocity (m/s) for grassed surfaces [1]

Percentage grass cover	Gradient of grass surface (%)									
	1	**2**	**3**	**4**	**5**	**6**	**8**	**10**	**15**	**20**
70% [2]	2.0	1.8	1.7	1.6	1.6	1.5	1.5	1.4	1.3	1.3
100% [3]	2.0	2.0	2.0	2.0	2.0	2.0	2.0	1.9	1.8	1.7
Poor soils [3]	1.5	1.4	1.3	1.2	1.2	1.1	1.1	1.1	1.0	0.9

[1] Maximum allowable flow velocity limited to 2.0m/s due to shallow water flow and resulting high shear stress. High flow velocities are allowable on reinforced grass.

[2] 70% cover would be typical for most grasses recently established by seed, but only when there is sufficient plant establishment time.

[3] 'Poor soils' refers to the soil's high erosion potential, such as dispersive clays (Emerson Class 1 and 2) such as sodic, yellow and red soils. Unstable, dispersible clayey sands and sandy clays, such as yellow and grey massive earths formed on sandstones and some granites. Highly erodible soils may include: lithosols, alluvials, podzols, siliceous sands, soloths, solodized solonetz, grey podzolics, some black earths, fine surface texture-contrast soils, and Soil Groups ML and CL.

Table 2 – Level spreader sill length – metres per unit discharge (m per m³/s) [1]

Land slope (%)	Allowable down-slope velocity over well grassed surface (m/s)						
	1.0	**1.2**	**1.5**	**1.8**	**2.0**	**2.2**	**2.5**
1.0	3.5*	2.5*	1.6*	1.1*	0.9*	0.8*	0.6*
2.0	5.2	3.8*	2.5*	1.8*	1.4*	1.2*	0.9*
3.0	6.6	4.8	3.2*	2.3*	1.8*	1.5*	1.2*
4.0	7.7	5.6	3.8*	2.7*	2.2*	1.8*	1.4*
5.0	8.7	6.3	4.3*	3.1*	2.5*	2.1*	1.6*
6.0	9.5	7.0	4.7	3.4*	2.8*	2.3*	1.8*
7.0	10.3	7.6	5.2	3.7*	3.1*	2.6*	2.0*
8.0	11.0	8.2	5.6	4.0*	3.3*	2.8*	2.2*
9.0	11.8	8.7	6.0	4.3*	3.5*	3.0*	2.4*
10.0	12.4	9.2	6.3	4.6*	3.8*	3.2*	2.5*
Caution the release of water onto grass slopes steeper than 10%.							
15.0	15.2	11.3	7.8	5.7	4.8	4.0*	3.2*
20.0	17.4	13.1	9.1	6.7	5.6	4.7	3.7*
25.0	19.4	14.6	10.3	7.6	6.3	5.3	4.3*
33.3	22.1	16.8	11.9	8.8	7.4	6.2	5.0
50.0	26.6	20.3	14.5	10.8	9.1	7.8	6.3

* Sill length limited to minimum 4m for discharges less than $0.85m^3/s$.

Design example:

Design a level spreader to release a flow rate of $0.5m^3/s$ down a 10% slope containing a good (70%) grass cover on moderately erodible soil.

Solution:

From Table 1, choose a maximum flow velocity of 1.4m/s as best representative of a good grass cover on a moderately erodible soil.

From Table 2, select a sill width per unit flow rate of $7.3m/m^3/s$.

Therefore, the sill length would need to be 0.5 x 7.3 = 3.65m < 4m (minimum).

Conclusion, specify a sill length of 4m.

The minimum sill lengths presented in Table 2 have been determined assuming a Manning's roughness for 50-150mm (Class D) grassed surfaces based on Equation 1. The sill length is sensitive to the selection of Manning's roughness. Variations between Table 2 and other published design tables for is due to variations in the assumed Manning's roughness, which is highly variable depending on the type and length of grass, and local growing conditions.

Class D roughness: $$n = \frac{R^{1/6}}{51.24 + 20.77\log_{10}(R^{1.4}.S^{0.4})} \quad \text{(Eqn 1)}$$

Table 3 – Minimum dimension of level spreader

Discharge (m^3/s)	Entrance width (m)	Depth (m)	End width (m)
0 to 0.28	3.0	0.15	0.9
0.29 to 0.57	4.9	0.18	0.9
0.58 to 0.85	7.3	0.21	0.9

Construction of a level spreader may require formation of flow control banks as shown in Figures 1 to 3.

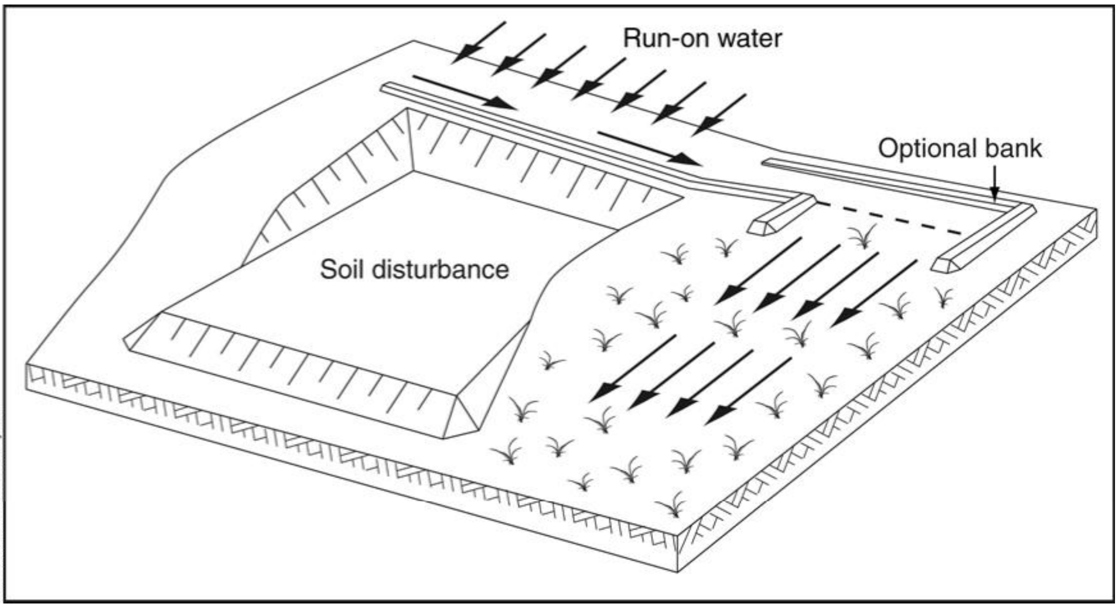


Figure 1 – Example of a level spreader used for flow diversion around a soil disturbance

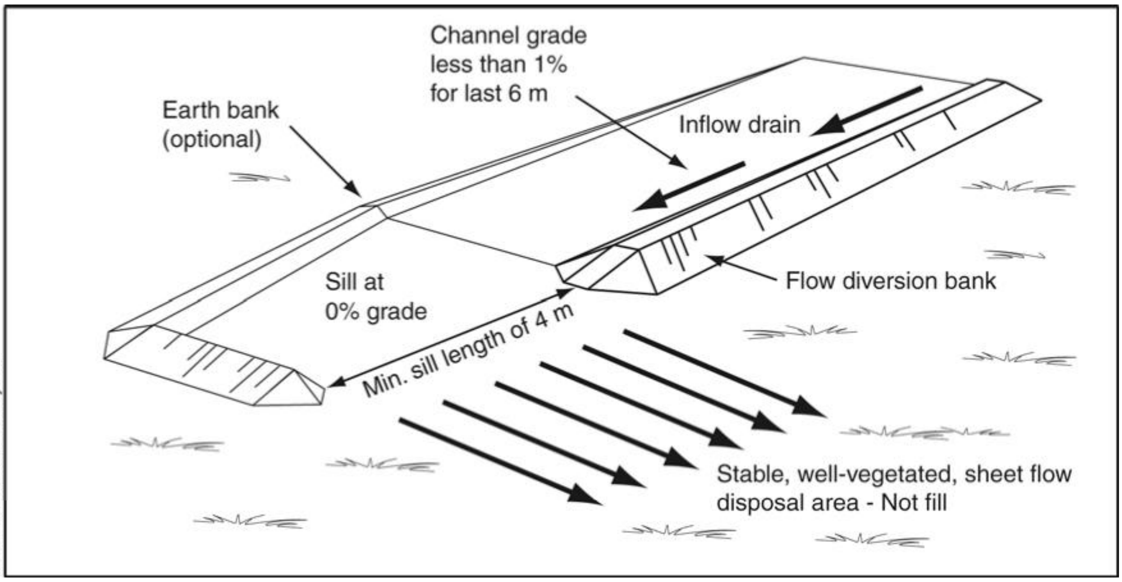


Figure 2 – Typical layout of level spreader

Description

Level spreaders consist of a level, grassed, side-flow weir (i.e. water discharges at 90 degrees to the inflow direction) constructed along the contour.

Purpose

Used to allow concentrated inflow to be released as *sheet flow* down a stable, vegetated slope.

Can be used as an outlet for *Catch Drains* and *Flow Diversion Banks*.

Level spreaders are commonly used in rural areas to discharge stormwater from roadside table drains into an adjacent property (Photos 1 & 2).

Limitations

Minimum sill length of 4m.

Maximum sill length of 25m.

Maximum discharge of around 0.85m^3/s.

Must only be used where the outflow can be discharged to an undisturbed, stable, grassed surface.

Construction traffic should be prohibited from the area of the level spreader.

Not suitable for highly erosive soils, dispersive soils, or soils with poor vegetation cover.

Advantages

Inexpensive to construct and maintain.

Disadvantages

Can be difficult to construct the outlet sill to the required precision.

May require a considerable width of undisturbed land.

May require the land to be free of trees, shrubs and other surface irregularities to avoid local erosion problems.

Common Problems

The most common problems result from damage to the outlet sill either from erosion, sedimentation, or stock.

Other problems can result from water flow concentrating below the level spreader due to the existence of a concave surface, vehicular tracks, or uneven vegetation cover.

Special Requirements

Outlet area must be free of depressions that may concentrate the outflow.

Extra erosion protection using jute mesh, *Erosion Control Mats*, turf, rock etc. may be required at the sill (Figure 4).

Generally constructed by dozers no larger than D5 or equivalent.

Extreme caution must be exercised when attempting to discharge *sheet flow* down a steep gradient (>10%) to ensure that the sedimentation or damage to the outlet sill does not concentrate the outflow.

Site Inspection

Check for sediment build-up on the sill, or the concentration of outflow.

Check for erosion down-slope of the sill.

Installation

1. Refer to approved plans for location, dimensions and construction details. If there are questions or problems with the location, dimensions, or method of installation contact the engineer or responsible on-site officer for assistance.
2. Wherever practical, locate the level spreader on undisturbed, stable soil.
3. Ensure flow discharging from the level spreader will disperse across a properly stabilised slope not exceeding 10:1 (H:V) and sufficiently even in grade across the slope to avoid concentrating the outflow.
4. The outlet sill of the spreader should be protected with erosion control matting to prevent erosion during the establishment of vegetation. The matting should be a minimum of 1200mm wide extending at least 300mm upstream of the edge of the outlet crest and buried at least 150mm in a vertical trench. The downstream edge should be securely held in place with closely spaced heavy-duty wire staples at least 150mm long.
5. Ensure that the outlet sill (crest) is level for the specified length.
6. Immediately after construction, turf, or seed and mulch where appropriate, the level spreader.

Maintenance

1. Inspect the level spreader after every rainfall event until vegetation is established.
2. After establishment of vegetation over the level spreader, inspections should be made on a regular basis and after runoff-producing rainfall.
3. Ensure that there is no soil erosion and that sediment deposition is not causing the concentration of flow.
4. Ensure that there is no soil erosion or channel damage upstream of the level spreader, or soil erosion or vegetation damage downstream of the level spreader.
5. Investigate the source of any excessive sedimentation.
6. Maintain grass in a health condition with no less than 90% cover unless current weather conditions require otherwise.
7. Grass height should be maintained at a minimum 50mm blade length within the level spreader and downstream discharge area, and a maximum blade length no greater than adjacent grasses.

Removal

1. Temporary level spreaders should be decommissioned only after an alternative stable outlet is operational, or when the inflow channel is decommissioned.
2. Remove collected sediment and dispose of in a suitable manner that will not cause an erosion or pollution hazard.
3. Remove and appropriately dispose of any exposed geotextile.
4. Grade the area and smooth it out in preparation for stabilisation.
5. Stabilise the area as specified on the approved plan.

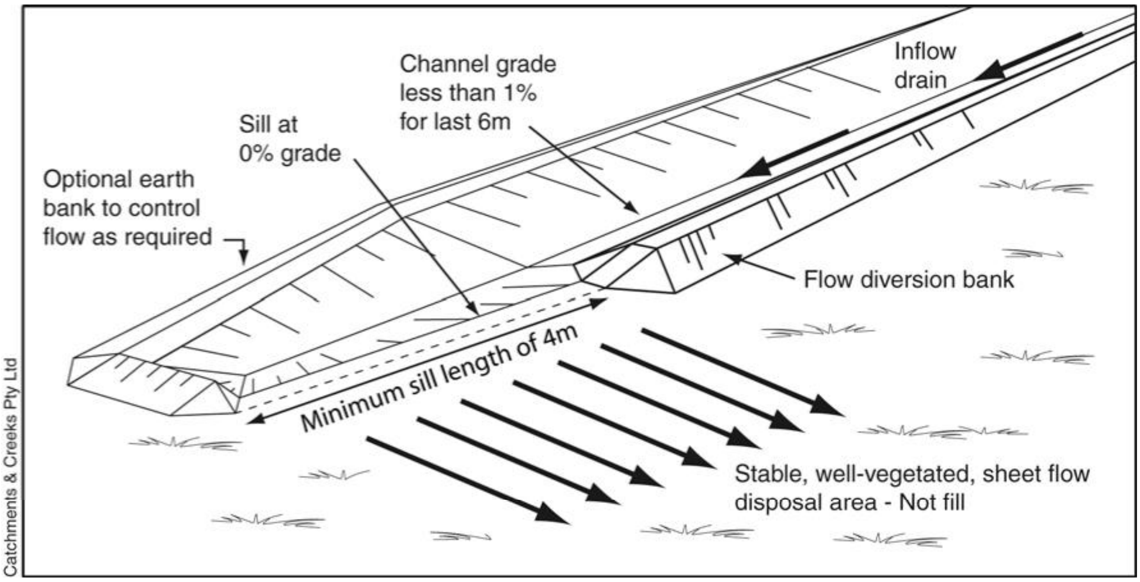


Figure 3 – Alternative level spreader layout

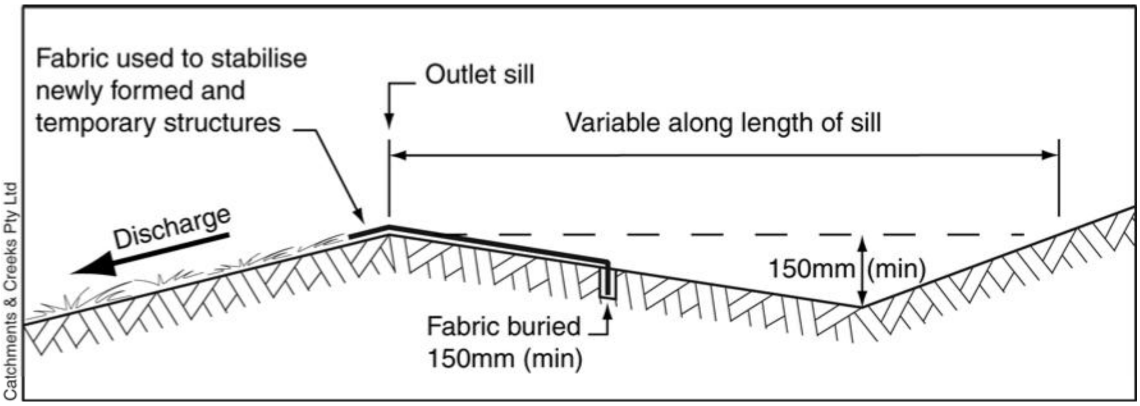


Figure 4 – Cross-sectional profile of end sill

Temporary Watercourse Crossing: Fords

Temporary Watercourse Crossing: Fords

DRAINAGE CONTROL TECHNIQUE

Low Gradient		Velocity Control		Short Term	✓
Steep Gradient		Channel Lining		Medium-Long Term	✓
Outlet Control		Soil Treatment		Permanent	[1]

[1] This fact sheet does not discuss all the issues requiring consideration in the design of permanent ford crossings.

TFC

Symbol

Photo 1 – Ford crossing of sandy creek

Photo 2 – Ford crossing of alluvial (gravel) stream

Key Principles

1. Significant bank damage can occur during construction of the access ramps; therefore, extreme care must to be taken to minimise such damage.
2. It is important to minimise the risk of sediment-laden runoff from the access ramps being allowed to discharge directly into the watercourse without passing through an appropriate sediment trap or vegetative filter.
3. Critical design parameter is the stability of the road surface crossing the streambed. Typically, ford crossings are only suitable for dry-bed, alluvial streams (i.e. streams with a bed primarily consisting of sand, gravel, and/or cobbles), or rocky streambeds.
4. Critical operational issue is the minimisation of harm to the watercourse, including sediment accumulation on the streambed.

Design Information

Temporary ford crossings require very little hydraulic design because they effectively make use of an existing stable streambed.

Ideally, the road surface should follow the natural cross-sectional profile of the streambed; however, it may be necessary for safety reasons to fill any deep holes with individually placed rock.

Most of the hydraulic design will be directed to appropriately managing stormwater runoff from the approach roads and access ramps cut down the stream banks.

Approach roads:

Approaches to the crossing should be stabilised and should have overland flow diversions to prevent runoff from entering the stream directly from the access road.

Ideally, the approach road should be straight for at least 10m each side of the crossing and should desirably cross the watercourse at right angles.

Where appropriate, access ramps should be stabilised with geotextile overlaid with minimum 150mm rock.

The watercourse should not have a base flow greater than 75mm in depth over the crossing. However, in most cases ford crossing should only be used to cross 'dry' channel beds.

If a temporary crossing is to be made of a wide, sandy riverbed, then a suitable, trafficable road surface may be formed by stabilising the riverbed with either a geogrid (refer to the fact sheet on *Geosynthetics*), or *Cellular Confinement System* (refer to separate fact sheet).

Photo 3 – Geogrid

Photo 4 – Cellular confinement system

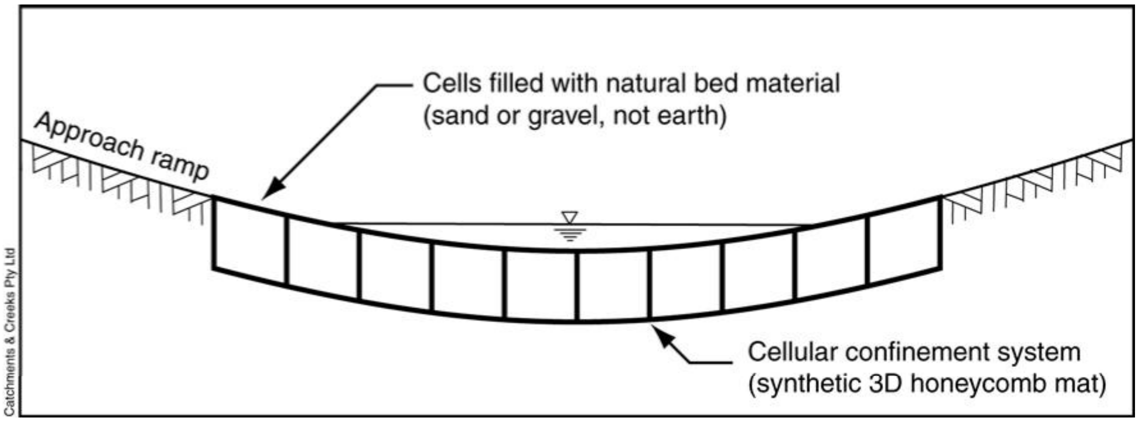


Figure 1 – Stabilisation of a temporary ford crossing in sandy bed stream using a *Cellular Confinement System*

On medium to high-speed roads, the access ramps usually need to be placed along a relatively straight alignment for safety reasons. In such cases, good vegetation coverage is highly desirable on the recessed banks to avoid erosion caused by turbulent eddies. Another benefit of this layout is that the recessed ramps help to create low velocity backwater areas that can be used by fish migrating upstream during flood events as resting areas.

If access ramps need to be cut into the channel banks (Figure 2 and Photo 6), and these ramps cannot be cut perpendicular to the channel, then wherever practicable align the ramps such that they fall to the waterway in an upstream direction. The reason for this is to minimise bank erosion caused by eddies resulting from flood flows moving past access ramps that are cut into the channel banks. Pointing the ramps upstream will usually allow a gradual expansion of the stream flow followed by a sudden contraction of the flow at the ramp (which is the preferred hydraulic condition).

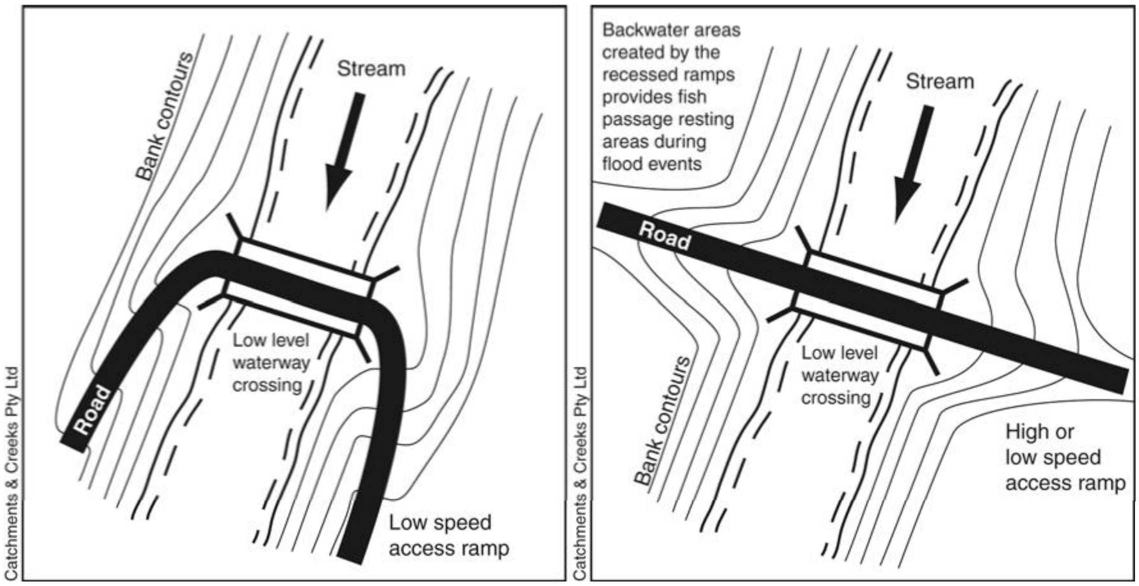


Figure 2 – Preferred alignment of access ramps

It is noted that if an access ramp's design results in a sudden expansion in the channel width, then eddies may form in the water during flood events and these eddies can then move downstream to locations where they can cause bank erosion. To avoid such erosion problems, sudden expansions in flow should be avoided.

The use of concrete to stabilise ford crossings should be avoided if crossing an alluvial stream because the fixed concrete slab can interfere with the natural downstream movement of the bed material.

Photo 5 – Temporary gravel-stabilised ford crossing of clay-based creek

Photo 6 – Ford crossing of newly constructed stormwater drain

Legislative Requirements:

Legislative requirements, permits and approvals vary from state to state, and region to region. Typical permit and approval requirements include:

- Approval for works within a watercourse (typically a department of water resources or natural resources)
- Approval for disturbance to bed, banks, or riparian vegetation.
- Approval for works that may interfere with fish passage (typically a fisheries authority)

Description

A ford is a shallow place in a stream where the bed may be crossed by traffic. By definition the crossing is a natural bed level.

Purpose

Used to provide very low traffic volume, dry weather access to a construction site. Fords are generally impassable during wet weather.

Fords provide a useful and affordable means of crossing dry creeks that have a solid rock bed, or wide sandy riverbeds during the dry season.

Limitations

Typically only suitable for crossing dry-bed, alluvial streams (i.e. streams with a bed primarily consisting of sand, gravel, and/or cobbles), or rocky streambeds.

They must only be used for very low traffic volumes.

Temporary ford crossing provide limited value during periods of significant stream flow.

Advantages

In the right environment, a ford crossing can cause the least amount of disturbance to the channel due to the absence of any construction works within the channel except the formation of access ramps.

Ford crossings are generally cheaper than causeways, culverts or bridges.

Fords potentially have the least impact on fish passage.

Disadvantages

Temporary ford crossings are generally impassable during periods of significant stream flow.

When used in a flowing stream, any sediment from the wheels of vehicles will be discharged directly into the stream.

Location

Ideally, temporary ford crossing should be located on a straight section of a watercourse, well downstream of a sharp bend.

In any case, all crossings should be located in an area that will cause the least overall disturbance, especially to those areas that are required to remain in a 'natural' state.

Site Inspection

Temporary ford crossings should be inspected with great care because they can promote the discharge of sediment directly into a stream and can cause significant environmental harm during construction, flood events and during decommissioning.

Check for erosion cause by overland flow moving down the approach ramps.

Check for appropriate flow diversions on the approach ramps to direct sediment runoff into a suitable sediment trap or grass filter.

Installation

1. Prior to commencing any works, obtain all necessary approvals and permits required to construct the temporary watercourse crossing, including permits for the disturbance of bank vegetation, aquatic vegetation (e.g. mangroves) and any temporary instream flow diversion barriers or sediment control measures.
2. Refer to approved plans for location and construction details. If there are questions or problems with the location or method of installation, contact the engineer or responsible on-site officer for assistance.
3. Ensure that the location of the crossing will not interfere with future construction works.
4. Prior to significant land clearing or construction of the approach ramps, establish all necessary sediment control measures and flow diversion works (instream and off-stream as required), clearing only those areas necessary for installation of these measures.
5. To the maximum degree practicable, construction activities and equipment shall not operate within open flowing waters.
6. Maintain clearing and excavation of the watercourse bed and banks to a minimum. Initially clear only the area necessary to allow access for construction. Clear the remainder of the approach ramps only when adequate drainage and sediment controls are in place.
7. If flow diversion systems cannot be installed, then conduct bank excavations by pulling the soil away from the channel.

8. Where practicable, construct the crossing perpendicular to the channel.
9. Where practicable, the approach ramps should be straight for at least 10m and should be aligned with the crossing.
10. Stabilise the streambed crossing as required for the given bed conditions, expected stream flow, and vehicular traffic.
11. Depressions in the rock bed should be filled with clean, graded rock.
12. Where practicable, direct stormwater runoff from the approach ramps into stable drains, adjacent vegetation, or appropriate sediment traps to minimise the release of sediment into the watercourse.
13. Take all reasonable measures to prevent debris and construction material from entering the watercourse, especially any still or flowing water.
14. If highly erosive soils are detected, then appropriately stabilise such soils as soon as practicable.
15. Appropriately stabilise disturbed watercourse banks.
16. Finish construction and stabilisation of the approach ramps each side of the crossing.
17. If it is not practicable to stabilise the access ramps against erosion, then install flow diversion banks across the width of each access ramp adjacent the top of the channel bank, and at regular intervals down the ramps (as required) to prevent or minimise sediment-laden runoff flowing directly into the watercourse.

Maintenance

1. Temporary watercourse crossings should be inspected weekly and after any significant change in stream flow.
2. Debris trapped on or upstream of the crossing should be removed.
3. Repair any damage caused by construction traffic. If traffic has exposed bare soil, stabilised as appropriate.
4. Check for erosion of abutments, channel scour, or rock displacement. Make all necessary repairs immediately.
5. Check for excessive erosion on the approach ramps.
6. Check the conditions of any flow diversion channels/banks and the operating conditions of associated sediment traps.

Removal

1. Temporary watercourse crossings should be removed, or the area appropriately rehabilitated, as soon as possible after alternative access is achieved or the crossing is no longer needed.
2. If the removal of the crossing is required, then remove all specified materials and dispose of in a suitable manner that will not cause an erosion or pollution hazard.
3. Restore the watercourse channel to its original cross-section, and smooth and appropriately stabilise and revegetate all disturbed areas.

Rock Lined Chute

Chutes Part 5: Rock linings

DRAINAGE CONTROL TECHNIQUE

Low Gradient		Velocity Control		Short-Term	✓
Steep Gradient	✓	Channel Lining		Medium-Long Term	✓
Outlet Control	[1]	Soil Treatment		Permanent	[2]

[1] Chutes can act as stable outlet structures for *Catch Drains* and *Flow Diversion Banks*.

[2] The design of permanent chutes may require consideration of issues not discussed here.

Symbol

Photo 17 – Permanent, rock-lined batter chute

Photo 18 – Permanent, rock-lined batter chute

Key Principles

1. The critical design components of a chute are the flow entry into the chute, the maximum allowable flow velocity down the face of the chute, and the dissipation of energy at the base of the chute.
2. The critical operational issues are ensuring unrestricted flow entry into the chute, ensuring flow does not undermine or spill out of the chute, and ensuring soil erosion is controlled at the base of the chute.
3. Most chutes fail as a result of water failing to enter the chutes properly. It is critical to control potential leaks and flow bypassing, especially at the chute entrance.

Design Information

The material contained within this fact sheet has been supplied for use by persons experienced in hydraulic design.

The following information must be read in association with the general information presented in *Part 1 - General information*.

Part 5 of this fact sheet addresses design issues associated with rock-lined chutes.

Tables 43 and 44 provide design, mean rock size (rounded up to the next 0.1m unit) for a safety factor of 1.2 and 1.5, based on Equation 3. Additional rock-sizing tables (for flatter slopes and higher flow rates) are provided in the separate ‘Chute & Channel Linings’ fact sheet on *Rock Linings*.

Equation 3 can also be used for sizing rock on the sides (banks) of the chute provided the bank slope (relative to the horizontal) does not exceed a gradient of 2:1. Rock size should be increased 25% for bank slopes of 1.5:1.

Equation 3 represents the recommended design formula for sizing rock on the bed of chutes.

$$d_{50} = \frac{1.27 . SF . K_1 . K_2 . S_o^{0.5} . q^{0.5} . y^{0.25}}{(s_r - 1)} \qquad \text{(Eqn 3)}$$

where:

- d_{50} = nominal rock size (diameter) of which 50% of the rocks are smaller [m]
- K_1 = correction factor for rock shape
  = 1.0 for angular (fractured) rock, 1.36 for rounded rock (i.e. smooth, spherical rock)
- K_2 = correction factor for rock grading
  = 0.95 for poorly graded rock ($C_u = d_{60}/d_{10} < 1.5$), 1.05 for well graded rock ($C_u > 2.5$), otherwise $K_2 = 1.0$ ($1.5 < C_u < 2.5$)
- q = flow per unit width down the embankment [m^3/s/m]
- s_r = specific gravity of rock
- S_o = bed slope = tan(θ) [m/m]
- SF = factor of safety (refer to Table 37)
- y = depth of flow at a given location [m]

Table 37 – Recommended safety factor for use in determining rock size

Safety factor (SF)	Recommended usage	Example site conditions
1.2	• Low risk structures. • Failure of structure is most unlikely to cause loss of life or irreversible property damage. • Permanent rock chutes with all voids filled with soil and pocket planted.	• Embankment chutes where failure of the structure is likely to result in easily repairable soil erosion. • Permanent chutes that are likely to experience significant sedimentation and vegetation growth before experiencing high flows. • Temporary (<2yrs) spillways with a design storm of 1 in 10 years of greater.
1.5	• High risk structures. • Failure of structure may cause loss of life or irreversible property damage. • Temporary structures that have a high risk of experiencing the design discharge while the voids remain open (i.e. prior to sediment settling within and stabilising the voids between individual rocks).	• Waterway chutes where failure of the chute may cause severe gully erosion and/or damage to the waterway. • Sediment basin or dam spillways located immediately up-slope of a residential area or busy roadway where an embankment failure could cause property flooding or loss of life. • Spillways and chutes designed for a storm frequency less than 1 in 10 years.

Design unit flow rate (q), flow velocity (V), and flow depth (y):

Wherever practical, the unit flow rate 'q' ($m^3/s/m$), flow velocity 'V' (m/s), and flow depth 'y' (m) used to determine rock size should be based on the 'local' conditions (e.g. the unit flow rate at a given location within the chute cross-section, or the depth-average flow velocity at a given location), rather than a value averaged over the full cross-section.

Rock type, size and grading:

The rock should be durable and resistant to weathering, and should be proportioned so that neither the breadth nor the thickness of a single rock is less than one-third its length. Generally, crushed (angular) rock is more stable than rounded stone.

Typical relative densities of various types of rock are provided in Table 38.

Table 38 – Typical relative density (specific gravity) of rock

Rock type	Relative density (s_r)
Sandstone	2.1 to 2.4
Granite	2.5 to 3.1, commonly 2.6
Limestone	2.6
Basalt	2.7 to 3.2

The maximum rock size should generally not exceed twice the nominal (d_{50}) rock size.

Table 39 provides a typical rock size distribution for use in preliminary design. Table 39 is provided for general information only, it does not represent a recommended design specification.

Table 39 – Typical distribution of rock size [1]

Rock size ratio	Assumed distribution value
d_{100}/d_{50}	2.00
d_{90}/d_{50}	1.82
d_{75}/d_{50}	1.50
d_{65}/d_{50}	1.28
d_{40}/d_{50}	0.75
d_{33}/d_{50}	0.60
d_{10}/d_{50}	> 0.50

[1] Wide variations in the rock size distribution can occur unless suitably controlled by the material contract specifications.

Thickness of rock protection:

The thickness of the rock protection should be sufficient to allow at least two overlapping layers of the nominal (d_{50}) rock size.

The thickness of rock protection must also be sufficient to accommodate the largest rock size.

In order to allow at least two layers of rock, the minimum thickness of rock protection (T) can be approximated by the values presented in Table 40.

Table 40 – Minimum thickness (T) of rock lining

Min. Thickness (T)	Size distribution (d_{50}/d_{90})	Description
1.4 d_{50}	1.0	Highly uniform rock size
1.6 d_{50}	0.8	Typical upper limit of quarry rock
1.8 d_{50}	0.67	Recommended lower limit of distribution
2.1 d_{50}	0.5	Typical lower limit of quarry rock

Backing material or filter layer:

Non-vegetated armour rock must be placed over a layer of suitably graded filter rock or geotextile filter cloth (minimum bidim A24 or the equivalent). The geotextile filter cloth must have sufficient strength and must be suitably overlapped to withstand the placement of the rock.

Armour rock that is intended to be vegetated by appropriately filling all voids with soil and pocket planting, generally will not require an underlying filter layer, unless the long-term viability of the vegetation is questioned due to possible high scour velocities, or limited natural light or rainfall conditions.

If the soils adjacent to the rock surface are dispersive (e.g. sodic soils), then prior to placing the filter cloth or filter layer, the exposed bank must first be covered with a layer of non-dispersive soil (Figure 11). The typically minimum thickness of non-dispersive soil is 200mm, but preferably 300mm.

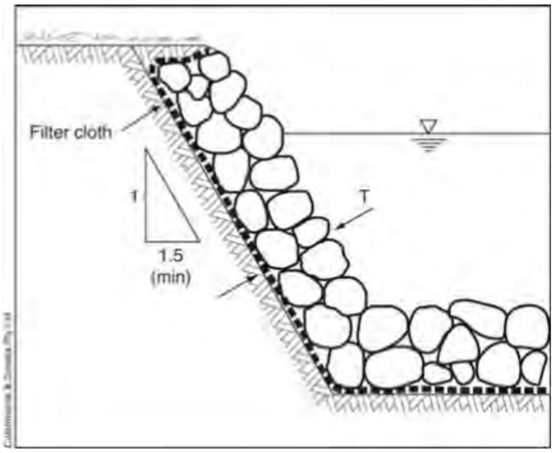


Figure 10 – Rock placement (without vegetation) on non-dispersive soil

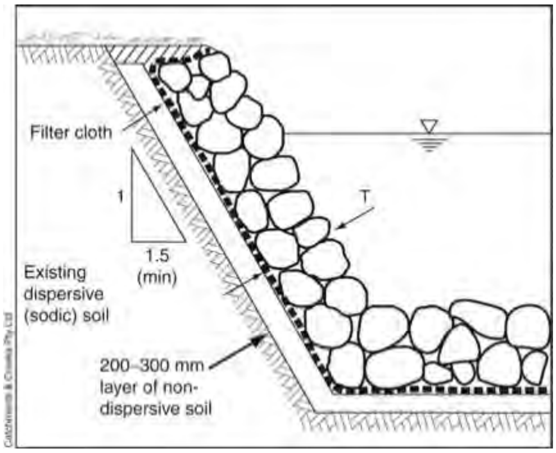


Figure 11 – Rock placement (without vegetation) on dispersive soil

Maximum bank gradient:

The recommended maximum desirable side slope of a large rock-lined chute is 2:1(H:V); however, side slopes as steep as 1.5:1 can be stable if the rock is individually placed rather than being bumped.

Typical angles of repose for dumped rock are provided in Table 41.

Table 41 – Typical angle of repose for rock

Rock shape	**Angle of repose (degrees)**	
	Rock size >100mm	**Rock size >500mm**
Very angular rock	41^{o}	42^{o}
Slightly angular rock	40^{o}	41^{o}
Moderately rounded rock	39^{o}	40^{o}

Placement of vegetation over the rock cover:

Vegetating rock-lined chutes can significantly increase the stability of these drainage structures, but can also reduce their hydraulic capacity. Obtaining experienced, expert advice is always recommended before establishing vegetation within drainage structures.

Manning roughness of rock-lined surfaces:

The Manning's (n) roughness for rock-lined surfaces can be determined from Table 42 or Equation 4.

Table 42 – Manning's (n) roughness of rock-lined surfaces

	$d_{50}/d_{90} = 0.5$				$d_{50}/d_{90} = 0.8$			
d_{50} =	**200mm**	**300mm**	**400mm**	**500mm**	**200mm**	**300mm**	**400mm**	**500mm**
R (m)	Manning's roughness (n)				Manning's roughness (n)			
0.2	0.10	0.14	0.17	0.21	0.06	0.08	0.09	0.11
0.3	0.08	0.11	0.14	0.16	0.05	0.06	0.08	0.09
0.4	0.07	0.09	0.12	0.14	0.04	0.05	0.07	0.08
0.5	0.06	0.08	0.10	0.12	0.04	0.05	0.06	0.07
0.6	0.06	0.08	0.09	0.11	0.04	0.05	0.05	0.06
0.8	0.05	0.07	0.08	0.09	0.04	0.04	0.05	0.06
1.0	0.04	0.06	0.07	0.08	0.03	0.04	0.05	0.05

The roughness values presented in Table 42 have been developed from Equation 4. Equation 4 (Witheridge, 2002) was developed to allow estimation of the Manning's n of rock lined channels in shallow water.

$$n = \frac{d_{90}^{\ 1/6}}{26\,(1 - 0.3593^{(X)^{0.7}})} \qquad \text{(Eqn 4)}$$

where: X = $(R/d_{90})(d_{50}/d_{90})$

R = Hydraulic radius of flow over rocks [m]

d_{50} = mean rock size for which 50% of rocks are smaller [m]

d_{90} = mean rock size for which 90% of rocks are smaller [m]

For 'natural' rock extracted from streambeds the relative roughness value (d50/d90) is typically in the range 0.2 to 0.5. For quarried rock the ratio is more likely to be in the range 0.5 to 0.8.

Placement of rock:

It is important to ensure that the top of the rock surface is level with, or slightly below, the surrounding land surface to allow the free entry of water including lateral inflows (if required) as shown in Figure 13.

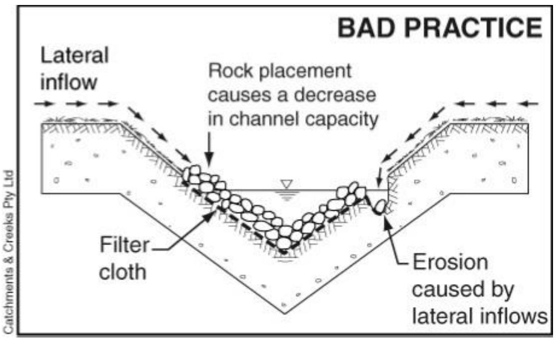


Figure 12 – Incorrect placement of rock causing loss of flow area and erosion along the outer limits of the rock

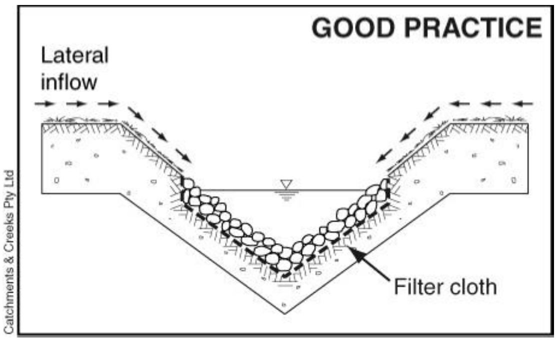


Figure 13 – Rock recessed into the soil to allow the free entry of lateral inflows

Most failures of rock-lined hydraulic structures are believed to occur as a result of inappropriate placement of the rock, either due to inadequate design detailing, or poorly supervised construction practices. Rock-lined chutes are usually most vulnerable to damage in the first year or two after placement while the voids remain open and free of sedimentation.

Where appropriate, permanent rock-lined chutes should be topped with a light covering of soil and planted to accelerate the integration of these structures into the surrounding environment. Revegetation is not however always advisable, and should be assessed on a case-by-case basis.

Photo 19 – Rock-lined spillway with well-defined crest profile

Photo 20 – Rock-lined spillway with poorly defined crest profile

Photo 21 – Placement of the rock <u>on</u> the soil can result in erosion problems if significant lateral inflows occur

Photo 22 – In this example, placement of the rock has resulted in the rock-lined table drain being higher than the road shoulder

Photo 23 – Rounded rock can be significantly less stable than angular, fractured rock, especially when placed on steep slopes

Photo 24 – Placement of a few large, anchor rocks down a steep slope will not help stabilise adjacent, under-sized rocks, and will likely cause flow diversion

Hydraulic design of rock-lined chutes:

Step 1 Determine the design discharge (Q) for the chute.

Step 2 Determine the slope (S) of the chute from the site geometry. The chute should be straight, with no bends or curves, from the crest to the base of the chute.

Step 3 Nominate the chute profile: e.g. trapezoidal or rectangular.

Step 4 Determine the maximum allowable approach flow depth, 'H' (relative to the inlet crest) upstream of the chute's inlet for the nominated design discharge.

Where necessary, design and specify appropriate *Flow Diversion Banks* or the like to appropriately control the approach flow and prevent any water bypassing the chute.

Step 5 Determine the required inlet geometry of the chute using an appropriate weir equation.

If the approach channel (the channel immediately upstream of the chute's crest) is short, then the relationship between the upstream water level (H) and discharge (Q) can be determined from one of the weir equations presented in Table 1 (*Part 1 – General information*). Tables 2 to 4 (Part 1) provide specific H–Q information for various chute profiles.

If the approach channel is long, and friction loss within this channel is likely to be significant, then an appropriate backwater analysis may be required.

Step 6 Ensure the entrance to the chute is suitably designed to allow the free flow of water into the chute (i.e. flow is not diverted along the up-slope edge of the rocks).

Where necessary, detail appropriate measures to control scour at the entrance to the chute (see Part 1 of this fact sheet, including Figure 3).

Step 7 Determine the design unit flow rate (q). This can be estimated by dividing the design discharge by the bed width determined in Step 5.

Step 8 Determine the likely density (specific gravity, s_r), and a size distribution (d_{50}/d_{90}) of the rock to be used on the chute.

Step 9 Using Manning's equation, or Tables 43 and 44, determine the uniform flow depth (y) and required size of the rock size (d_{50}) for the chute.

Manning's equation: $Q = A.V = (1/n)\,A\,.\,R^{2/3}.\,S^{1/2}$

Additional rock-sizing tables (for flatter slopes and higher flow rates) are provided in the separate fact sheet – *Rock linings*.

Step 10 Specify the required depth of the chute, being the greater of:

(i) 300mm (unless a lower depth is supported by expected flow conditions);

(ii) 0.67(H) plus minimum freeboard of 150mm; ('H' determined from Step 4)

(iii) the uniform flow depth (y) plus a minimum freeboard of 150mm, or the equivalent of the flow depth, whichever is smaller.

Step 11 Design the required outlet energy dissipation structure at the base of the chute.

For the design of the outlet structure, refer to Part 1 of this fact sheet or the fact sheet on *Outlet Structures*.

The 'local' uniform flow velocity (V) down the chute can be estimated by dividing the design unit flow rate (q) by flow depth (y). This flow velocity will be slightly greater than the average flow velocity, which is equal to the total discharge (Q) divided by the total flow area (A).

Design example: rock-lined chutes:

Design a rock-lined chute suitable to carry a discharge of $1m^3/s$ on a 3:1 slope with a maximum allowable upstream water level (H) of 300mm.

Step 1 Design discharge given as $1.0m^3/s$.

Step 2 The chute slope is given as, S = 33% (3:1).

Step 3 Try a trapezoidal profile with side slopes of 2:1

Step 4 The maximum allowable approach flow depth is given as, H = 0.3m

Step 5 Table 3 (Part 1) indicates that for an approach flow depth, H = 0.3m, a bed width of b = 3.2m (interpolated) is required to allow the design discharge of $1.0m^3/s$ to enter a trapezoidal chute with side slopes of 2:1

Step 6 To control water movement and erosion at the chute entrance, specify on the plans that the rock must be suitably recessed into the ground to allow the unrestricted entry of water.

Flow diversion banks will need to be constructed each side of the chute entrance to direct water into the chute with minimum height of, H + 0.3m = 0.3 + 0.3 = 0.6m

To control soil erosion near the entrance, the rock will extend a distance of 5(H) = 1.5m upstream of the crest. Otherwise, suitable erosion control matting shall be placed over the soil and overlapping the upstream edge of the rock lining.

Step 7 As a first trial, the unit flow rate can be estimated by dividing the design discharge by the bed width determined in Step 5.

Trial unit flow rate, q = Q/b = 1.0/3.2 = $0.313m^2/s$ (approximation)

Step 8 Assume rock is available with a specific gravity, s_r = 2.6, and a size distribution, d_{50}/d_{90} = 0.5

Step 9 Given the estimate unit flow rate of $0.313m^2/s$, the chute slope of 3:1, Table 43 indicates that the required mean rock size, d_{50} = 300mm.

Even though Table 43 is applicable for rock with a specific gravity of 2.4, thus the results are considered conservative for rock with a specific gravity of 2.6.

If it is assumed that this rock size is available on the site, then the bed width, b = 3.2m obtained in Step 5 appears suitable.

Step 10 From Table 43 the uniform flow depth is expected to be 0.19m (interpolated); however there is expected to be significant variation in this depth due to turbulence.

The required depth of the chute should be the greater of:

(i) 300mm;

(ii) 0.67(H) plus freeboard of 150mm = 0.67(300) + 150 = 351mm;

(iii) y + 150mm = 190 + 150 = 340mm.

Thus, choose a total chute depth, Y = 350mm.

Step 11 Design of outlet structure as per Part 1 – 'General Information':

Given the flow depth, y = 0.19m; the local uniform flow velocity can be estimated as, V = q/y = 0.313/0.19 = 1.65m/s.

Given that the flow approaching the outlet structure is less than 200mm in depth, and the velocity is less than 2m/s, Table 5 (Part 1) indicates a rock size of 100mm; however, choose the same 300mm rock as used on the face of the chute.

Table 6 (Part 1) indicates a length of rock protection, L = 2.1m.

Table 7 (Part 1) indicates a dissipation basin recess depth, Z = 0.12m

The flow top width at the base of the chute, T = b + 2my = 3.2 + 2(2)0.19 = 3.96m

From Figure 6 (Part 1), W_1 = 3.96 + 0.6 = 4.56m, and W_2 = 3.96 + 0.4(L) = 4.8m

Let W_1 = 4.6m and W_2 = 4.8m

Table 43 – Flow depth [1], y (m) and mean rock size, d_{50} (m) for SF = 1.2

Safety factor, SF = 1.2			Specific gravity, s_r = 2.4			Size distribution, d_{50}/d_{90} = 0.5		
Unit flow rate (m^3/s/m)	Bed slope = 5:1		Bed slope = 4:1		Bed slope = 3:1		Bed slope = 2:1	
	y (m)	d_{50}	y (m)	d_{50}	y (m)	d_{50}	y (m)	d_{50}
0.1	0.09	**0.10**	0.09	**0.10**	0.09	**0.20**	0.09	**0.20**
0.2	0.15	**0.20**	0.14	**0.20**	0.14	**0.20**	0.14	**0.30**
0.3	0.19	**0.20**	0.19	**0.20**	0.19	**0.30**	0.18	**0.30**
0.4	0.23	**0.30**	0.23	**0.30**	0.23	**0.30**	0.22	**0.40**
0.5	0.27	**0.30**	0.27	**0.30**	0.26	**0.40**	0.26	**0.40**
0.6	0.31	**0.30**	0.30	**0.40**	0.30	**0.40**	0.29	**0.50**
0.8	0.37	**0.40**	0.37	**0.40**	0.36	**0.50**	0.35	**0.60**
1.0	0.43	**0.40**	0.42	**0.50**	0.42	**0.60**	0.41	**0.70**
1.2	0.49	**0.50**	0.48	**0.50**	0.47	**0.60**	0.46	**0.70**
1.4	0.54	**0.50**	0.53	**0.60**	0.52	**0.70**	0.51	**0.80**
1.6	0.59	**0.60**	0.58	**0.70**	0.57	**0.70**	0.56	**0.90**
1.8	0.64	**0.60**	0.63	**0.70**	0.62	**0.80**	0.60	**1.00**
2.0	0.68	**0.70**	0.67	**0.70**	0.66	**0.90**	0.65	**1.00**
3.0	0.89	**0.90**	0.88	**1.00**	0.87	**1.10**	0.85	**1.30**
4.0	1.08	**1.00**	1.07	**1.20**	1.05	**1.30**	1.02	**1.60**
5.0	1.26	**1.20**	1.24	**1.30**	1.22	**1.50**	1.19	**1.80**

[1] Flow depth is expected to be highly variable due to whitewater (turbulent) flow conditions.

Table 44 – Flow depth [1], y (m) and mean rock size, d_{50} (m) for SF = 1.5

Safety factor, SF = 1.5			Specific gravity, s_r = 2.4			Size distribution, d_{50}/d_{90} = 0.5		
Unit flow rate (m^3/s/m)	Bed slope = 5:1		Bed slope = 4:1		Bed slope = 3:1		Bed slope = 2:1	
	y (m)	d_{50}	y (m)	d_{50}	y (m)	d_{50}	y (m)	d_{50}
0.1	0.10	**0.20**	0.10	**0.20**	0.10	**0.20**	0.10	**0.20**
0.2	0.16	**0.20**	0.16	**0.20**	0.15	**0.30**	0.15	**0.30**
0.3	0.21	**0.30**	0.21	**0.30**	0.20	**0.30**	0.20	**0.40**
0.4	0.25	**0.30**	0.25	**0.40**	0.25	**0.40**	0.24	**0.50**
0.5	0.29	**0.40**	0.29	**0.40**	0.28	**0.50**	0.28	**0.50**
0.6	0.33	**0.40**	0.33	**0.40**	0.32	**0.50**	0.31	**0.60**
0.8	0.40	**0.50**	0.40	**0.50**	0.39	**0.60**	0.38	**0.70**
1.0	0.47	**0.60**	0.46	**0.60**	0.45	**0.70**	0.44	**0.80**
1.2	0.53	**0.60**	0.52	**0.70**	0.51	**0.80**	0.50	**0.90**
1.4	0.58	**0.70**	0.58	**0.80**	0.57	**0.90**	0.55	**1.00**
1.6	0.64	**0.70**	0.63	**0.80**	0.62	**0.90**	0.60	**1.10**
1.8	0.69	**0.80**	0.68	**0.90**	0.67	**1.00**	0.65	**1.20**
2.0	0.74	**0.80**	0.73	**0.90**	0.72	**1.10**	0.70	**1.30**
3.0	0.97	**1.10**	0.96	**1.20**	0.94	**1.40**	0.92	**1.70**
4.0	1.17	**1.30**	1.16	**1.50**	1.14	**1.70**	1.11	**2.00**
5.0	1.36	**1.50**	1.34	**1.70**	1.32	**1.90**	1.29	**2.30**

[1] Flow depth is expected to be highly variable due to whitewater (turbulent) flow conditions.

Common Problems

Severe erosion problems if rocks are placed directly on dispersive soil. To reduce the potential for such problems, dispersive soils should be covered with a minimum 200mm layer of non-dispersive soil before rock placement.

Failure of rock-lined chutes due to the absence of a suitable filter cloth or aggregate filter layer beneath the primary armour rock layer.

Weed invasion of the rock protection can become unsightly. The control of weed growth can be an expensive, labour intensive exercise.

Rill erosion can occur along the upper edge of the rock if they are not properly set into the soil.

Severe rilling along the sides of the chute can be caused by splash or lateral inflows being deflected by the edge of the chute.

Erosion at the base of the chute caused by inadequate energy dissipation.

Special Requirements

An underlying geotextile or rock filter layer is generally required unless all voids are filled with soil and pocket planted (thus preventing the disturbance and release of underlying sediments through these voids).

The upper rock surface should blend with surrounding land to allow water to freely enter the channel.

Flow Diversion Banks are often required to direct flows into the chute.

Good subsoil drainage and foundations are required to stabilise the chute lining.

Site Inspection

Check flow entry conditions to ensure no bypassing, undermining, sedimentation or erosion.

Check for piping failure, scour holes, or bank failures.

Check for erosion around the outer edges of the treated area.

Ensure the chute is straight.

Ensure the rock size and shape agrees with approved plan.

Check the thickness of rock application and the existence of underlying filter layer.

Check for excessive vegetation growth that may restrict the channel capacity.

Ensure the outlet is appropriately stabilised.

Installation (chute formation)

1. Refer to approved plans for location and construction details. If there are questions or problems with the location or method of installation, contact the engineer or responsible on-site officer for assistance.
2. Ensure all necessary soil testing (e.g. soil pH, nutrient levels) and analysis has been completed, and required soil adjustments performed prior to planting.
3. Clear the location for the chute clearing only what is needed to provide access for personnel and equipment for installation.
4. Remove roots, stumps, and other debris and dispose of them properly.
5. Construct the subgrade to the elevations shown on the plans. Remove all unsuitable material and replace with stable material to achieve the desired foundations.
6. If the chute is temporary, then compact the subgrade to a firm consistency. If the chute is intended to be permanent, then compact and finish the subgrade as specified within the design plans.
7. Avoid compacting the subgrade to a condition that would prevent the turf from bonding with the subgrade.
8. Ensure the sides of the chute are no steeper than a 1.5:1 (H:V) slope.
9. Ensure the completed chute has sufficient deep along its full length.
10. Ensure the chute is straight from its crest to the toe of the chute.
11. On fill slopes, ensure that the soil is adequately compacted for a width of at least one metre each side of the chute to minimise the risk of soil erosion, otherwise protect the soil with suitable scour protection measures such as turf or erosion control mats.
12. Place and secure the turf as directed.
13. Install an appropriate outlet structure (energy dissipater) at the base of the chute (refer to separate specifications).
14. Ensure water leaving the chute and the outlet structure will flow freely without causing undesirable ponding or scour.
15. Appropriately stabilise all disturbed areas immediately after construction.

Materials

- Rock: hard, angular, durable, weather resistant and evenly graded with 50% by weight larger than the specified nominal rock size and sufficient small rock to fill the voids between the larger rock. The diameter of the largest rock size should be no larger than 1.5 times the nominal rock size. Specific gravity to be at least 2.5.
- Geotextile fabric: heavy-duty, needle-punched, non-woven filter cloth, minimum 'bidim' A24 or equivalent.

Installation (rock placement)

1. Over-cut the channel to a depth equal to the specified depth of rock placement such that the finished rock surface will be at the elevation of the surrounding land.
2. Rock must be placed within the channel as specified within the approved plans, including the placement of any specified filter layer.
3. If details are not provided on the rock placement, then the primary armour rock must be either placed on:

- a filter bed formed from a layer of specified smaller rock (rock filter layer);
- an earth bed lined with filter cloth;
- an earth bed not lined in filter cloth, but only if all voids between the armour rock are to be filled with soil and pocket planted immediately after placement of the rock.

4. If a rock/aggregate filter layer is specified, then place the filter layer immediately after the foundations are prepared. Spread the filter rock in a uniform layer to the specified depth but a minimum of 150mm. Where more than one layer of filter material has been specified, spread each layer such that minimal mixing occurs between each layer of rock.
5. If a geotextile (filter cloth) underlay is specified, place the fabric directly on the prepared foundation. If more than one sheet of fabric is required to over the area, overlap the edge of each sheet at least 300mm and place anchor pins at minimum one metre spacing along the overlap.
6. Ensure the geotextile fabric is protected from punching or tearing during installation of the fabric and the rock. Repair any damage by removing the rock and placing with another piece of filter cloth over the damaged area overlapping the existing fabric a minimum of 300mm.
7. Where necessary, a minimum 100mm layer of fine gravel, aggregate or sand should be placed over the fabric to protect it from damage.
8. Placement of rock should follow immediately after placement of the filter layer. Place rock so that it forms a dense, well-graded mass of rock with a minimum of voids.
9. Place rock to its full thickness in one operation. Do not place rock by dumping through chutes or other methods that cause segregation of rock sizes.
10. The finished surface should be free of pockets of small rock or clusters of large rocks. Hand placing may be necessary to achieve the proper distribution of rock sizes to produce a relatively smooth, uniform surface. The finished grade of the rock should blend with the surrounding area. No overfall or protrusion of rock should be apparent.
11. Immediately upon completion of the channel, vegetate all disturbed areas or otherwise protect them against soil erosion.
12. Where specified, fill all voids with soil and vegetate the rock surface in accordance with the approved plan.

Maintenance

1. During the construction period, inspect all chutes prior to forecast rainfall, daily during extended periods of rainfall, after significant runoff producing storm events, or otherwise on a weekly basis. Make repairs as necessary.
2. Check for scour or dislodged rock. Repair damaged areas immediately.
3. Closely inspect the outer edges of the rock protection. Ensure water entry into the channel or chute is not causing erosion along the edge of the rock protection.
4. Investigate the cause of any scour, and repair as necessary.
5. Carefully check the stability of the rock looking for indications of piping, scour holes, or bank failures.
6. Replace any displaced rock with rock of a significantly (minimum 110%) larger size than the displaced rock.
7. Ensure sediment is not partially blocking flow entry into the chute. Where necessary, remove any deposited material to allow free drainage.
8. Dispose of any sediment in a manner that will not create an erosion or pollution hazard.
9. 9. When making repairs, always restore the chute to its original configuration unless an amended layout is required.

Removal

1. When the soil disturbance above the chute is finished and the area is stabilised, the chute and any associated flow diversion banks should be removed, unless it is to remain as a permanent drainage feature.
2. Dispose of any materials, sediment or earth in a manner that will not create an erosion or pollution hazard.
3. Grade the area in preparation for stabilisation, then stabilise the area as specified in the approved plan.

Energy Dissipater

Energy Dissipaters

DRAINAGE CONTROL TECHNIQUE

Low Gradient		Velocity Control	✓	Short Term	✓
Steep Gradient		Channel Lining		Medium-Long Term	✓
Outlet Control	✓	Soil Treatment		Permanent	[1]

[1] The design of permanent energy dissipaters may require consideration of issues not discussed within this fact sheet. Obtaining expert hydraulic advice is always recommended.

Symbol (not applicable)

Photo 1 – Rock mattress lined basin spillway and energy dissipater

Photo 2 – Rock lined basin spillway and energy dissipater

Key Principles

1. Energy dissipation must be contained within a suitably stabilised area, therefore it is essential for the designer to be able to control the **location** of the hydraulic jump.
2. The key performance objectives are to control soil erosion associated with the energy dissipater, and present structural damage to the chute, culvert or spillway.

Design Information

Energy dissipation is usually required to achieve one or more of the following:

- prevent the undermining of the outlet, chute or spillway;
- control of bed scour immediately downstream of the energy dissipater;
- control of bank erosion well downstream of the structure caused by an 'outlet jet', if such jetting is possible at the structure.

Bank erosion downstream of pipe outlet is likely to result from the effects of an outlet jet if:

- tailwater levels are above the centre of a pipe outlet (which causes the jet to float); and
- the flow velocity at the outlet exceeds the scour velocity of the bank material; and
- the distance between the outlet and the opposing bank is less than approximately 10 times the equivalent pipe diameter for a single outlet, or 13 times the equivalent pipe diameter for a multi-cell outlet.

The control of *bed scour* is usually achieved by the development of a thick, low velocity, boundary layer usually through the introduction of erosion resistant bed roughness (e.g. rock).

Downstream bank erosion is usually controlled by breaking-up the outlet jet through the energy dissipating effects of a hydraulic jump, plunge pool, or impact structure.

Bed friction outlets

These energy dissipaters use coarse riprap or rows of small concrete impact blocks as a form of bed roughness to retard the outlet flow. This bed roughness can help spread the flow and develop an effective boundary layer thus reducing the potential for downstream bed scour. If favourable tailwater conditions exist, these outlets can also induce a hydraulic jump to aid in energy dissipation.

Bed friction outlet structures exhibit only minimal control over 'floating' outlet jets. They are therefore most effective when operating under low tailwater conditions.

For design guidelines, refer to the separate fact sheet on *Outlet Structures*.

Photo 3 – Rock pad outlet structure

Photo 4 – Rock pad outlet structure

Hydraulic jump energy dissipaters

These energy dissipaters that rely of the formation of a hydraulic jump and are usually best used to control high velocity flows confined within rectangular or near-rectangular channels. These structures usually require well-regulated tailwater conditions to prevent the hydraulic jump from being swept downstream of the stabilised energy dissipation zone.

To control the location of the hydraulic jump, the outlet pond can be recessed into the bed of the channel forming a recessed energy dissipation pool. Generally these dissipation pools need to be designed to be free draining to avoid permanent ponding and prevent mosquito breeding.

If a hydraulic jump is required to be formed downstream of a chute, then the crest of the chute must be flat, and the chute's cross-section must be as close to *rectangular* as is possible to produce near-uniform, 1-dimentsional flow conditions. Trapezoidal chutes with flat side slopes can cause highly 3D flow conditions resulting in the formation of an ineffective hydraulic jump.

Hydraulic jump energy dissipaters are usually **not** effective downstream of piped outlets because jetting from the pipe can prevent an effective hydraulic jump from forming.

Photo 5 – Hydraulic jump type energy dissipater on sediment basin spillway

Photo 6 – Hydraulic jump dissipater downstream of detention basin outlet

Plunge pool energy dissipaters

Plunge pools can be an effective way of dissipating energy and controlling bed scour. However, in order to be effective the outlet jet **must** be allowed to free fall into the pool. Therefore, low tailwater conditions are required. Under high tailwater conditions (i.e. when a floating outlet jet is formed) plunge pool designs are relatively ineffective. Though distinguished from hydraulic jump dissipaters, most plunge pool dissipaters effectively act as 'confined' hydraulic jumps.

Concrete or other hard-lined plunge pool dissipaters should be free draining to avoid the formation of stagnant water. Many standardised plunge pool dissipater designs can be successfully modified to avoid long-term ponding by introducing a narrow, open notch within the end sill.

Plunge pool dissipaters can be highly dangerous hydraulic structures resulting in severe head injuries to persons being swept through the structure.

Photo 7 – Rock-lined plunge pool energy dissipater

Photo 8 – Note use of impact blocks to stabilise the location of the hydraulic jump

Stepped spillways

Stepped spillways dissipate energy as the flow passes down the face of the spillway (chute), as well as allowing the formation of a hydraulic jump at the base of the spillway. Each step can operate under conditions of either a plunging jet (nappe flow regime), or as a fully or partially formed hydraulic jump.

Under high flow conditions, the water can begin to skim over the individual steps (skimming flow regime) greatly reducing energy dissipation down the face of the spillway. Once skimming flow conditions are fully developed, the spillway begins to behave like an unstepped spillway.

For design guidelines, refer to *Hydraulic design of stepped channels and spillways*, H. Chanson, Report CH43/94, February 1994, Department of Civil Engineering, The University of Queensland, Brisbane.

Photo 9 – Gabion lined stepped spillway on a stormwater detention basin

Photo 10 – Stepped chute acting as an outlet structure for a table drain

Impact structures

These structures contain impact walls, blocks or columns to break-up the jet and induce highly turbulent flow. They are generally very effective at dissipating flow energy from medium to high velocity outlets where control of the outlet jet is required. The control of bed scour immediately downstream of the outlet structure usually requires the use of additional riprap protection.

The height of impact blocks is usually set equal to the height of the incoming jet. In the case of culverts and stormwater outlets, this means a height equal to the height of the culvert or pipe.

These are some of the most dangerous of all the hydraulic structures. Their design and use must only be managed under the supervision of suitably trained experts.

Photo 11 – Baffled spillway

Photo 12 – Impact block energy dissipater

Design Information

Warning, energy dissipater can represent a significant safety risk to persons swept into the flow. In circumstance where a person could be swept into such danger, safety issues <u>must</u> be given appropriate consideration.

Energy dissipaters are usually major hydraulic structures requiring design input from experienced hydraulics specialists. This fact sheet does not provide sufficient information to allow energy dissipaters to be designed by inexperienced persons.

Photo 13 – Spillways <u>must</u> have a well-defined profile to fully contain the flow

Photo 14 – A suitable energy dissipater <u>must</u> be constructed at the base of the spillway

Design of rock mattress or concrete-lined energy dissipation pools:

The following design procedure and tables are provided as a guide only. This design information requires interpretation and application by experienced hydraulic design professionals.

Hydraulic design requires the estimation of flow depth, velocity, and Froude number at the base of the chute or spillway.

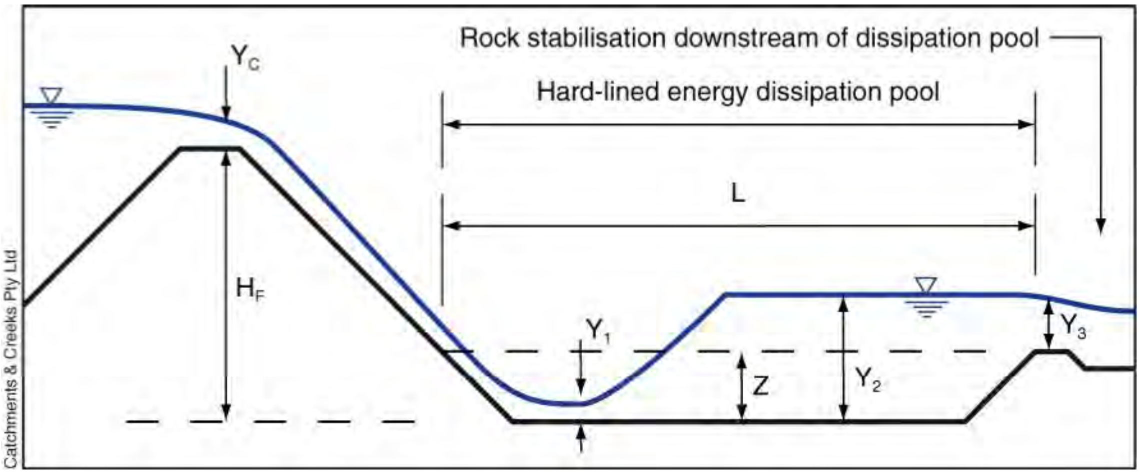


Figure 1 – typical profile of recessed, hard-lined energy dissipation pool located at the base of chute or spillway

Design steps:

1. Determine the flow depth (y_1), velocity (V_1) and Froude number (F_1) at the base of the chute for the design discharge.

$$F_1 = \frac{V_1}{\sqrt{g.y_1}} \qquad \text{(Eqn 1)}$$

2. For flow conditions where $F_1 > 1$ (i.e. supercritical flow) and where the resulting hydraulic jump can be represented by 1-dimensional hydraulic analysis (i.e. a regular hydraulic jump contained within a rectangular channel), calculate the sequent depth (y_2) associated with the hydraulic jump.

$$y_2 = \frac{y_1}{2}\left(\sqrt{(1+8F_1^{\,2})} - 1\right) \qquad \text{(Eqn 2)}$$

3. Determine the probable tailwater conditions including water level and flow depth (y_3) downstream of the recessed energy dissipation pool. This downstream flow depth should not be less than the critical flow depth (y_c).

4. Determine the recess depth of the energy dissipation pool.

$$Z = y_2 - y_3 \qquad \text{(Eqn 3)}$$

5. Calculate the desired length of the energy dissipation pool (L). Two equations can be used to determine this pool length, these equations are presented below as Equations 4 and 5.

$$L = 6y_2 \qquad \text{(Eqn 4)}$$

$$L = 6.9(y_2 - y_1) \qquad \text{(Eqn 5)}$$

An **approximate** length of the dissipation pool can be determined from Table 1 for an energy dissipation pool containing a standard, rectangular hydraulic jump. It is noted that hydraulic jumps formed within trapezoidal channels can be unpredictable in their shape and stability, potentially resulting in an increased length of the required energy dissipation basin.

Table 2 provides an **estimate** of the recess depth (Z) based on a downstream flow depth (y_3) equal to the critical flow depth (y_c). **Tables 1 and 2 should be used for preliminary design purposes only.**

Table 1 – Approximate length, L (m) of an energy dissipation pool containing a standard, rectangular hydraulic jump [1]

Unit flow (m^2/s)	Chute fall upstream of energy dissipater, H_F (m)									
	0.2	**0.3**	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**4.0**	**5.0**
0.01	0.4	0.4	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9
0.02	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.1	1.2	1.3
0.05	0.8	1.0	1.1	1.3	1.5	1.6	1.7	1.8	1.9	2.0
0.10	1.2	1.3	1.5	1.9	2.1	2.2	2.4	2.5	2.7	2.9
0.15	1.4	1.6	1.9	2.3	2.5	2.7	2.9	3.0	3.3	3.5
0.20	1.6	1.8	2.1	2.6	2.9	3.1	3.3	3.5	3.8	4.0
0.25	1.8	2.1	2.4	2.9	3.2	3.5	3.7	3.9	4.2	4.5
0.30	2.0	2.2	2.6	3.2	3.5	3.8	4.1	4.3	4.6	4.9
0.35	2.2	2.4	2.8	3.4	3.8	4.1	4.4	4.6	5.0	5.3
0.40	2.3	2.6	3.0	3.6	4.1	4.4	4.7	4.9	5.3	5.6
0.45	2.4	2.7	3.1	3.8	4.3	4.7	4.9	5.2	5.6	6.0
0.50	2.6	2.9	3.3	4.0	4.5	4.9	5.2	5.5	5.9	6.3
1.00	3.6	4.0	4.6	5.6	6.3	6.8	7.3	7.6	8.3	8.8
1.50	4.4	4.9	5.6	6.8	7.6	8.3	8.8	9.3	10.0	11.0

[1] Length of energy dissipation pool is based on an average of $6y_2$ and $6.9(y_2 - y_1)$, with y_1 based on a smooth chute (i.e. minimal friction loss), and y_2 determined from Table 5. Data is presented for preliminary design purposes only.

Table 2 – Approximate recess depth, Z (m) for an energy dissipation pool containing a standard, rectangular hydraulic jump [1]

Unit flow (m^2/s)	Chute fall upstream of energy dissipater, H_F (m)									
	0.2	**0.3**	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**4.0**	**5.0**
0.01	0.04	0.05	0.06	0.07	0.08	0.09	0.10	0.10	0.11	0.12
0.02	0.06	0.06	0.08	0.10	0.11	0.12	0.13	0.14	0.16	0.17
0.05	0.08	0.09	0.11	0.15	0.17	0.19	0.20	0.22	0.24	0.25
0.10	0.10	0.12	0.15	0.20	0.23	0.26	0.28	0.29	0.32	0.35
0.15	0.12	0.14	0.18	0.24	0.27	0.30	0.33	0.35	0.39	0.42
0.20	0.14	0.16	0.20	0.26	0.31	0.34	0.37	0.40	0.44	0.47
0.25	0.15	0.18	0.22	0.29	0.34	0.38	0.41	0.44	0.48	0.52
0.30	0.16	0.19	0.23	0.31	0.37	0.41	0.44	0.47	0.52	0.57
0.35	0.17	0.20	0.25	0.33	0.39	0.43	0.47	0.50	0.56	0.60
0.40	0.17	0.21	0.26	0.35	0.41	0.46	0.50	0.53	0.59	0.64
0.45	0.18	0.22	0.27	0.37	0.43	0.48	0.52	0.56	0.62	0.67
0.50	0.19	0.23	0.29	0.38	0.45	0.50	0.55	0.59	0.65	0.71
1.00	0.24	0.29	0.37	0.50	0.59	0.67	0.73	0.78	0.87	0.95
1.50	0.28	0.34	0.43	0.58	0.69	0.78	0.86	0.92	1.03	1.12

[1] Recess depth is based on a downstream flow depth (y_3) equal to the critical flow depth, and y_1 based on a smooth chute (i.e. minimal friction loss). Data is presented for preliminary design purposes only.

Design of rock protection downstream of hydraulic jump energy dissipaters:

Equation 6 is the recommended equation for sizing rock placed within the zone of highly turbulent water immediately **downstream** of the end sill of an **energy dissipater** (i.e. not within the main energy dissipation zone).

$$d_{50} = \frac{(0.081).V^{2.23}}{(s_r - 1)} \qquad \text{(Eqn 6)}$$

where: d_{50} = nominal rock size (diameter) of which 10% of the rocks are smaller (m)

V = local, depth-average flow velocity immediately downstream of the end sill (m/s)

Design of rock-lined energy dissipation pools:

The following design procedure and tables are provided as a guide only. This design information requires interpretation and application by experienced hydraulic design professionals.

An **estimation** of the recess depth (relative to the downstream water level) of a rock-lined energy dissipation pool can be determined from Equation 7.

$$Z + y_3 = 4.75 \frac{(H_F)^{0.2} q^{0.57}}{(d_{90})^{0.32}} \qquad \text{(Eqn 7)}$$

where: Z = Recess of energy dissipation pool relative to downstream ground level (m)

y_3 = depth of flow downstream of the energy dissipation pool at design flow (m)

H_F = fall in chute or spillway upstream of the energy dissipater (m)

q = design unit flow rate (m^2/s)

d_{90} = rock size, lining the dissipation pool, of which 90% of rocks are smaller (m)

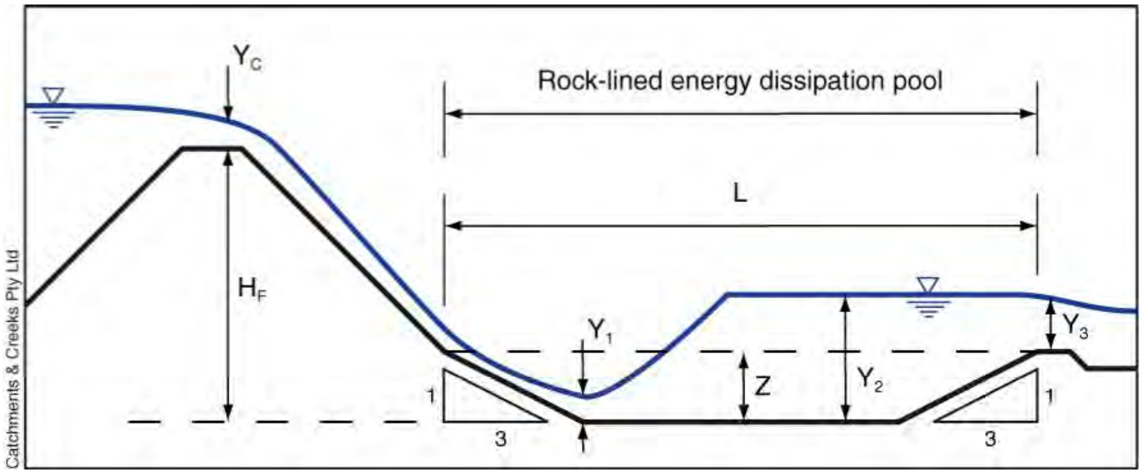


Figure 2 – Typical profile of rock-lined energy dissipation pool

The length of the dissipation pool (L) may be based on the same design procedures presented for a rock mattress or concrete-lined dissipation pool presented in the previous section.

Tables 3 to 5 provide an estimation of the recess depth (Z) for a mean rock size (d_{50}) of 200, 300 and 500mm, based on a rock size distribution, $d_{50}/d_{90} = 0.5$.

In circumstances where the energy dissipater is located downstream of a smooth channel surface (e.g. a concrete-lined chute or spillway), then the rocks located within the first quarter (minimum) of the dissipater basin should be grouted in place to avoid displacement. The displacement of loose rocks located immediately downstream of a smooth channel surface is partially caused by the changing boundary layer conditions from the smooth upstream channel to the rough, rock-lined basin.

Caution: trapezoidal chutes can result in the formation unstable, three-dimensional hydraulic jumps that may not dissipate energy as efficiently as rectangular chutes.

Table 3 – Approximate operating water depth within an energy dissipation pool ($Z + y_3$) lined with mean (d_{50}) 200mm rock, with rock size distribution, d_{50}/d_{90} = 0.5

Unit flow (m^2/s)	Chute fall upstream of energy dissipater, H_F (m)									
	0.2	**0.3**	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**4.0**	**5.0**
0.005	0.23	0.24	0.27	0.31	0.34	0.36	0.37	0.39	0.41	0.43
0.01	0.33	0.36	0.40	0.46	0.50	0.53	0.55	0.57	0.61	0.64
0.02	0.50	0.54	0.60	0.68	0.74	0.79	0.82	0.85	0.90	0.94
0.04	0.74	0.80	0.89	1.02	1.10	1.17	1.22	1.27	1.34	1.40
0.06	0.93	1.01	1.12	1.28	1.39	1.47	1.54			
0.08	1.09	1.19	1.31	1.51						
0.10	1.24	1.35	1.49							
0.15	1.57									

Table 4 – Approximate operating water depth within an energy dissipation pool ($Z + y_3$) lined with mean (d_{50}) 300mm rock, with rock size distribution, d_{50}/d_{90} = 0.5

Unit flow (m^2/s)	Chute fall upstream of energy dissipater, H_F (m)									
	0.2	**0.3**	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**4.0**	**5.0**
0.005	0.20	0.21	0.24	0.27	0.30	0.31	0.33	0.34	0.36	0.38
0.01	0.29	0.32	0.35	0.41	0.44	0.47	0.49	0.50	0.53	0.56
0.02	0.44	0.47	0.52	0.60	0.65	0.69	0.72	0.75	0.79	0.83
0.04	0.65	0.70	0.78	0.89	0.97	1.03	1.07	1.11	1.18	1.23
0.06	0.82	0.88	0.98	1.13	1.22	1.29	1.35	1.40	1.48	1.55
0.08	0.96	1.04	1.15	1.33	1.44	1.52				
0.10	1.09	1.18	1.31	1.51						
0.15	1.37	1.49								

Table 5 – Approximate operating water depth within an energy dissipation pool ($Z + y_3$) lined with mean (d_{50}) 500mm rock, with rock size distribution, d_{50}/d_{90} = 0.5

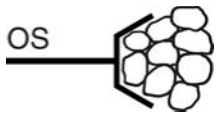
Unit flow (m^2/s)	Chute fall upstream of energy dissipater, H_F (m)									
	0.2	**0.3**	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**4.0**	**5.0**
0.005	0.17	0.18	0.20	0.23	0.25	0.27	0.28	0.29	0.31	0.32
0.01	0.25	0.27	0.30	0.34	0.37	0.40	0.41	0.43	0.45	0.47
0.02	0.37	0.40	0.44	0.51	0.55	0.59	0.61	0.64	0.67	0.70
0.04	0.55	0.60	0.66	0.76	0.82	0.87	0.91	0.94	1.00	1.05
0.06	0.69	0.75	0.83	0.96	1.04	1.10	1.15	1.19	1.26	1.32
0.08	0.82	0.88	0.98	1.13	1.22	1.29	1.35	1.40	1.49	1.55
0.10	0.93	1.00	1.11	1.28	1.39	1.47	1.54			
0.15	1.17	1.27	1.40	1.61						
0.20	1.38	1.49								
0.25	1.56									

Outlet Structure (Recessed Rock Pad)

Outlet Structures

DRAINAGE CONTROL TECHNIQUE

Low Gradient		Velocity Control	✓	Short Term	✓
Steep Gradient		Channel Lining		Medium-Long Term	
Outlet Control	✓	Soil Treatment		Permanent	



Symbol

Photo 1 – Temporary *Slope Drain* with rock stabilisation at inlet and outlet

Photo 2 – Temporary rock mattress outlet structure at end of a *Chute*

Key Principles

1. The primary performance objectives generally relate to minimising the risk of soil erosion at the outlet, and preventing excessive undermining of the pipe and/or headwall.
2. Critical design parameters are the mean rock size (d_{50}) and length of rock protection (L).
3. The recommended rock sizing design charts/tables are based on the acceptance of some degree of rock movement (rearrangement) following initial storm events.
4. Critical construction issues relate to the provision of suitable rock (size and density), suitable pad dimensions (width, length and depth), and suitably recessing/integrating the rocks into the outlet channel to allow outflows to pass evenly over the rocks.

Design Information – General:

The design procedures presented in this fact sheet are not appropriate for the design of energy dissipaters for Sediment Basin spillways. Designers are advised to always seek expert hydraulic advice regarding the appropriate use of the material supplied within this fact sheet.

The following information is appropriate for the design of loose rock outlet structures for small Slope Drains (300/375mm diameter) and minor batter Chutes (<300mm flow depth).

Recommended mean (d_{50}) rock sizes and length (L) of rock protection for ***Slope Drain*** outlets are presented in Tables 2 and 3 for smooth and rough internal sidewall pipes respectively.

Recommended mean (d_{50}) rock sizes and length (L) of rock protection for minor batter ***Chute*** outlets are presented in Tables 4 and 5. These rock sizes are based on information presented within ASCE (1992) rounded up to the next 100mm increment, with a minimum rock size set as 100mm.

The thickness of the rock pad should be based on at least two layers of rock. This typically results in a minimum thickness as presented in Table 1.

Table 1 – Minimum thickness (T) of rock pad

Min. Thickness (T)	Size distribution (d_{50}/d_{90})	Description
1.4 d_{50}	1.0	Highly uniform rock size
1.6 d_{50}	0.8	Typical upper limit of quarry rock
1.8 d_{50}	0.67	Recommended lower limit of distribution
2.1 d_{50}	0.5	Typical lower limit of quarry rock

[1] d_{50} = nominal rock size (diameter) of which 50% of the rocks are smaller (i.e. the mean rock size).

Design Information – Outlet structures for Slope Drains:

Table 2 – Mean rock size (mm) and length (m) of rock pad outlet structure for smooth internal sidewall slope drain

Pipe diameter: 300 and 375mm							Smooth internal sidewall: n = 0.01						
Pipe slope (X:1)	Pipe discharge (L/s)												
	30	40	50	60	70	80	100	120	140	160	180	200	220
10	150	150	150	150	150	150	200	200	200	200	200	300	300
8	150	150	150	150	150	150	200	200	200	200	300	300	300
7	150	150	150	150	150	150	200	200	200	300	300	300	300
6	150	150	150	150	150	200	200	200	300	300	300	300	300
5	150	150	150	150	200	200	200	200	300	300	300	300	300
4	150	150	150	200	200	200	200	300	300	300	300	300	300
3	150	150	200	200	200	200	300	300	300	300	300	300	300
2	150	200	200	200	200	300	300	300	300	300	400	400	400
1	200	200	300	300	300	300	300	400	400	400	400	400	400
L [1]	**1.1**	**1.2**	**1.5**	**1.5**	**1.5**	**1.5**	**1.7**	**2.0**	**2.0**	**2.0**	**2.1**	**2.1**	**2.5**

[1] Recommended minimum length (m) of rock pad outlet structure.

Table 3 – Mean rock size (mm) and length (m) of rock pad outlet structure for rough internal sidewall slope drain

Pipe diameter: 300 and 375mm							Rough internal sidewall: n = 0.03						
Pipe slope (X:1)	Pipe discharge (L/s)												
	30	40	50	60	70	80	100	120	140	160	180	200	220
10	150	150	150	150	150	150	150	150	150	150	150	150	150
8	150	150	150	150	150	150	150	150	150	150	150	150	150
7	150	150	150	150	150	150	150	150	150	150	150	150	150
6	150	150	150	150	150	150	150	150	150	150	150	150	150
5	150	150	150	150	150	150	150	150	150	150	150	150	150
4	150	150	150	150	150	150	150	150	150	150	150	150	200
3	150	150	150	150	150	150	150	150	150	150	200	200	200
2	150	150	150	150	150	150	150	150	200	200	200	200	200
1	150	150	150	150	150	150	200	200	200	200	300	300	300
L [1]	**1.6**	**1.8**	**1.9**	**2.1**	**2.2**	**2.3**	**2.5**	**2.6**	**2.8**	**2.9**	**3.1**	**3.2**	**3.3**

[1] Recommended minimum length (m) of rock pad outlet structure.

Technical Note – Development of Tables 2 and 3

Many of the rock sizing charts traditionally presented for the design outlet structures can attribute their origins to the published work of Bohan (1970). This research work was based on low gradient flow conditions where the pipe is flowing full just upstream of the outlet, and during low tailwater conditions, the flow passed through critical depth at or near the outlet of the pipe. Such flow conditions are not consistent with the high-velocity, partial-full flow expected at the base of a slope drain.

The rock sizes and pad lengths presented in Tables 2 and 3 have been determined by firstly determining the partial-full, supercritical flow velocity expected at the base of a slope drain for a given discharge, internal pipe roughness, and slope gradient. Secondly an equivalent pipe diameter was determined that would have a full-pipe discharge and velocity equivalent to that determined above. Using this equivalent pipe diameter and actual discharge velocity, the design charts presented by Bohan for low tailwater conditions were used to determine the required mean rock size and length of rock protection. The rock sizes where then rounded up to the nearest 100mm rock size, with a minimum rock size set as 150mm.

The typical layouts of a rock pad for a *Slope Drain* is shown in Figure 1. The rock pad should straight and align with the direction of the pipe outlet.

If the width of the rock pad is governed by the width of the receiving channel, then the rock protection should ideally extend up the banks of the channel to a height no less than the central elevation of the pipe outlet, but no more than the expected depth of flow.

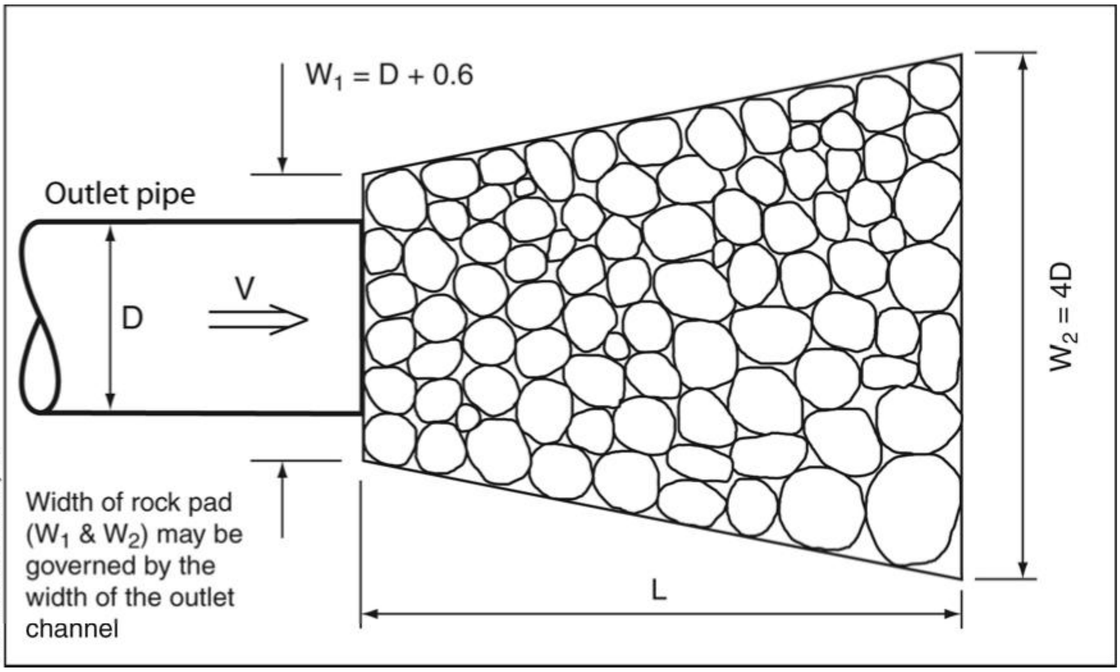


Figure 1 – Typical layout of a rock pad for a single pipe outlet (plan view)

The outlet structure for *Slope Drains* should be constructed at a level grade, or a gradient equal to that of the receiving channel.

The surface level of the downstream end of the rock pad should be level with the invert of the receiving channel, i.e. the rocks should be recessed into the outlet channel to minimise the risk of erosion around the outer edges of the rock pad.

The placement of filter cloth under the rock pad is generally considered optional for temporary outlet structures placed at the end of *Slope Drains*.

Design Information – Outlet structures for temporary drainage Chutes:

Table 4 – Mean rock size, d_{50} (mm) for batter *Chute* outlet protection [1]

Depth of approach flow (mm) [2]	Flow velocity at base of *Chute* (m/s)						
	2.0	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**
50	100	100	100	200	200	200	300
100	100	100	200	200	300	300	400
200	100	200	300	300	400	[3]	[3]
300	200	200	300	400	[3]	[3]	[3]

[1] For exit flow velocities not exceeding 1.5m/s, and where growing conditions allow, loose 100mm rock may be replaced with 75mm rock stabilised with a good cover of grass.

[2] Flow depth is based on the maximum depth, not the average flow depth.

[3] Consider using 400mm grouted rock pad, or a rock-filled mattress outlet.

The outlet pad lengths provided in Table 5 are suitable for temporary, rock-lined outlet structures only. These rock pad length will not necessarily fully contain all energy dissipation and flow turbulence; therefore, some degree of scour may still occur downstream of the outlet structure.

For permanent structures, or concrete-lined energy dissipaters, the length of the dissipater should be based on the estimated length of the resulting hydraulic jump. Also, in circumstances where the outlet structure is located downstream of a smooth surface chute, e.g. concrete-lined chutes, then the rocks should be grouted in place to avoid displacement.

Table 5 – Recommended length, L (m) of rock pad for batter *Chute* outlet protection [1]

Depth of approach flow (mm)	Flow velocity at base of *Chute* (m/s)						
	2.0	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**
50	1.0	1.5	2.1	2.6	3.1	3.6	4.2
100	1.3	2.0	2.7	3.4	4.1	4.8	5.5
200	2.1	2.7	3.4	4.3	5.2	6.1	7.0
300	2.7	3.6	4.3	4.8	5.8	6.8	7.9

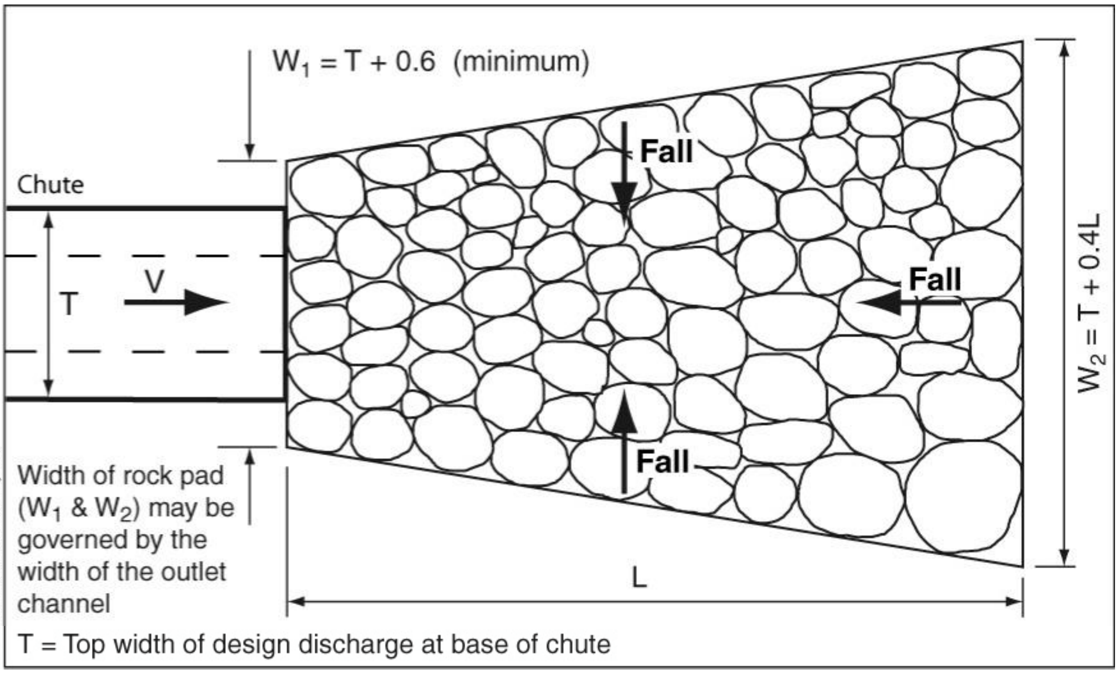


Figure 2 – Typical layout of a recessed rock pad for a *Chute* (plan view)

As indicated in Figures 2, 3 and 4, outlet structures for minor batter *Chutes* should be recessed below the surrounding ground level to promote effective energy dissipation. The recommended recess depth (Z) can be determined from Table 6.

Table 6 – Recommended recess depth, Z (m) for batter *Chute* outlet protection

Depth of approach flow (mm)	Flow velocity at base of *Chute* (m/s)						
	2.0	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**
50	0.13	0.20	0.28	0.36	0.43	0.50	0.60
100	0.14	0.23	0.32	0.42	0.50	0.60	0.70
200	0.12	0.21	0.31	0.42	0.50	0.60	0.70
300	0.07	0.16	0.25	0.35	0.44	0.55	0.65

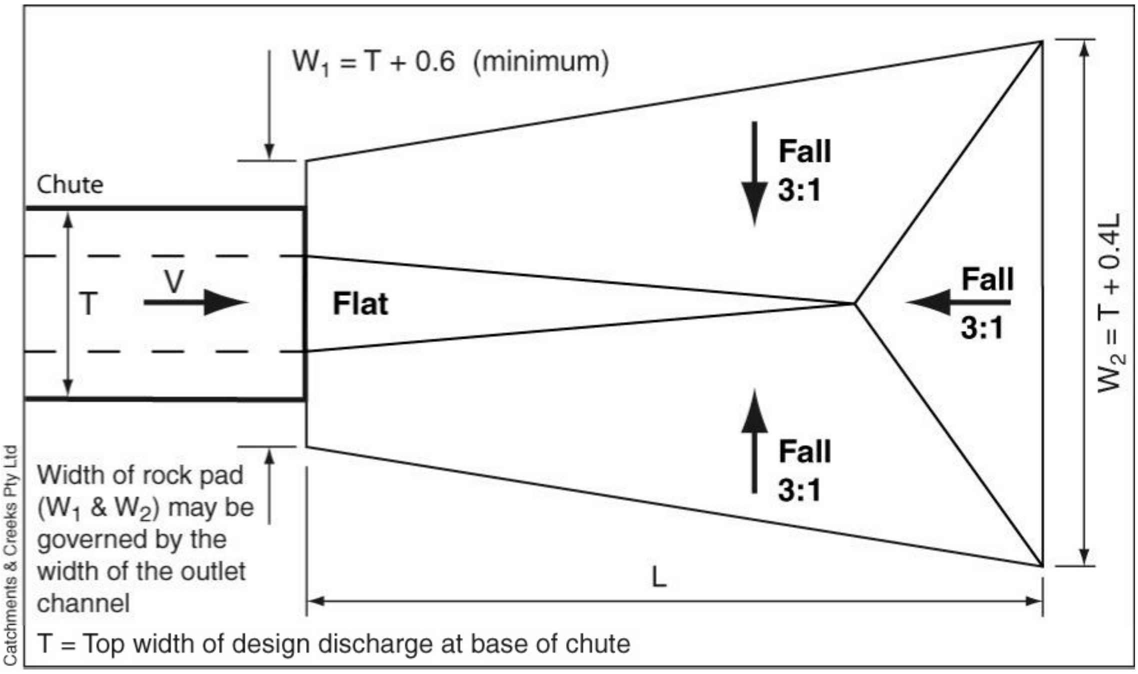


Figure 3 – Typical arrangement of recessed outlet structure for minor *Chutes*

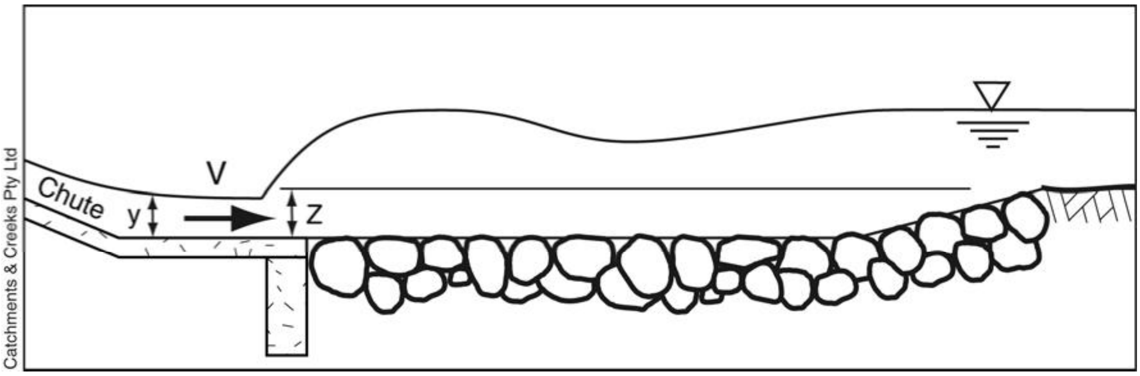


Figure 4 – Typical profile of recessed outlet structure for minor *Chutes*

References:

ASCE 1992, *Design and construction of urban stormwater management systems*. ASCE Manuals and Reports of Engineering Practice No. 77, and Water Environment Federation Manual of Practice FD-20, American Society of Civil Engineers, New York.

Bohan, J.P. 1970, *Erosion and riprap requirements at culvert and storm-drain outlets*. Research Report H-70-2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Design example 1 – Slope drain outlet structure:

Design the outlet protection for a temporary slope drain with a diameter (D) of 300mm, smooth internal sidewall, and design discharge of 100L/s.

Solution

Given D = 300mm and Q = 100L/s, the recommended mean rock size as obtained from Table 2 is d_{50} = 300mm and L = 1.7m.

Upstream width of the rock pad, W1 = D + 0.6 = 0.9m (see Figure 1).

Downstream width of the rock pad, W2 = 4D = 1.2m

If it is assumed that the largest rock is likely to be around 1.5 times the size of the average rock size, i.e. d_{50}/d_{90} approximately equals 0.67, then from Table 1 we can obtain the required depth of rock protection as, T = 1.8(d_{50}) = 0.54m. In any case, a minimum of two layers of rock should be specified on the construction plans.

Design example 2 – Chute outlet structure:

Design the outlet protection for a temporary, trapezoidal chute lined with filter cloth on a 3:1 batter slope with a base width of 1.0m, side slopes of 2:1, and design discharge of 600L/s.

Solution

Adopting a Manning's roughness of, n = 0.022 for the filter cloth, the flow conditions at the base of the chute can be determined from Manning's equation as:

Discharge, Q = 0.6m^3/s

Manning's roughness, n = 0.022 (based on an expected flow depth > 0.1m)

Channel slope, S = 0.333 (m/m)

Bed width, b = 1.0m

Channel side slope, m = 2:1

Flow depth, y = 0.1m

Flow top width, T = b + 2my = 1.4m

Hydraulic radius, R = 0.083m

Velocity, $$V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.022}(0.083)^{2/3}(0.333)^{1/2} = 5.0\text{m/s}$$

From Table 4 the mean rock size, d_{50} = 200mm

From Table 5 the length of the rock pad, L = 3.4m

From Table 6 the recommended recess depth, Z = 0.42m

From Figure 3 the upstream width of the rock pad, W1 = T + 0.6 = 2.0m

From Figure 3 the downstream width of the rock pad, W2 = T + 0.4L = 2.8m

If it is assumed that the largest rock is likely to be around 1.5 times the size of the average rock size, i.e. d_{50}/d_{90} approximately equals 0.67, then from Table 1 we can obtain the required depth of rock protection as, T = 1.8(d_{50}) = 0.36m. In any case, a minimum of two layers of rock should be specified on the construction plans.

Note, the symbol 'T' has traditionally been used for both the depth of rock protection (as in Example 1), and the top width of flow (as in Example 2).

Description

The term *Outlet Structure* refers to a wide range of outlet control devices including rock pads, rock mattress aprons, and various impact-type energy dissipaters.

The standard outlet structure consists of a pad of medium sized rock placed at the outlet of *Slope Drain*, *Chute*, stormwater pipe or culvert.

Purpose

Used to control soil erosion adjacent to the outlet and to dissipate flow energy.

Limitations

These rock pads are generally ineffective in controlling erosion caused by high-velocity outlet ‘jetting’ occurring during high tailwater conditions.

Advantages

Quick to install.

The rock can often be retained as a permanent erosion control measure.

Disadvantages

If the rock is not appropriately recessed into the surrounding soil, erosion can occur around the edge of the rock pad.

Common Problems

Inadequate rock size.

Inadequate length, width or depth of rock protection.

Rock not recessed into the channel bed.

Erosion along the outer edge of the rock pad caused by lateral inflows (i.e. water flowing towards the outlet from a location other than the pipe).

Special Requirements

Important to recess the rock so that the top of the rock pad is level with the surrounding earth surface.

The rock should extend downstream until non-erosive flow conditions are achieved. In some cases this may require the rock protection to be extended beyond standards outlet dimensions determined from the attached design tables.

Location

Rock pad outlet structures are constructed downstream of temporary *Chutes* and *Slope Drains*, as well as permanent stormwater outlets and culverts.

Site Inspection

Check for erosion around the edge of the rock pad.

Ensure the rocks are adequately recessed into the earth.

Check for excessive displacement of rocks. Some degree of rock movement should be expected, especially immediately downstream of the pipe or concrete apron.

Check for excessive sediment deposition.

Materials (Rock pads)

- Rock: hard, angular, durable, weather resistant and evenly graded with 50% by weight larger than the specified nominal rock size and sufficient small rock to fill the voids between the larger rock. The diameter of the largest rock size should be no larger than 1.5 times the nominal rock size. Specific gravity to be at least 2.5.
- Geotextile fabric: heavy-duty, needle-punched, non-woven filter cloth, minimum 'bidim' A24 or equivalent.

Installation (Rock pads)

1. Refer to approved plans for location and construction details. If there are questions or problems with the location, dimensions or method of installation contact the engineer or responsible on-site officer for assistance.
2. The dimensions of the outlet structure must align with the dominant flow direction.
3. Excavate the outlet pad footprint to the specified dimension such the when the rock is placed in the excavated pit the top of the rocks will be level with the surrounding ground, unless otherwise directed.
4. If the excavated soils are dispersive, over-excavated the rock pad by at least 300mm and backfill with stable, non-dispersive material.
5. Line the excavated pit with geotextile filter cloth, preferably using a single sheet. If joints are required, overlap the fabric at least 300mm.
6. Ensure the filter cloth is protected from punching or tearing during installation of the fabric and the rock. Repair any damage by removing the rock and placing with another piece of filter cloth over the damaged area overlapping the existing fabric a minimum of 300mm.
7. Ensure there are at least two layers of rocks. Where necessary, reposition the larger rocks to ensure two layers of rocks are achieved without elevating the upper surface above the pipe invert.
8. Ensure the rock is placed in a manner that will allow water to discharge freely from the pipe.
9. Ensure the upper surface of the rock pad does not cause water to be deflected around the edge of the rock pad.
10. Immediately after construction, appropriately stabilise all disturbed areas.

Maintenance

1. While construction works continue on the site, inspect the outlet structure prior to forecast rainfall, daily during extended periods of rainfall, after significant runoff producing rainfall, and on at least a weekly basis.
2. Replace any displaced rock with rock of a significantly (minimum 110%) larger size than the displaced rock.

Removal

1. Temporary outlet structures should be completely removed, or where appropriate, rehabilitated so as not to cause ongoing environmental nuisance or harm.
2. Following removal of the device, the disturbed area must be appropriately rehabilitated so as not to cause ongoing environmental nuisance or harm.
3. Remove materials and collected sediment and dispose of in a suitable manner that will not cause an erosion or pollution hazard.

Sediment Fence

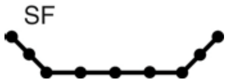
Sediment Fence

SEDIMENT CONTROL TECHNIQUE

Type 1 System		Sheet Flow	✓	Sandy Soils	✓
Type 2 System		Concentrated Flow	[1]	Clayey Soils	[2]
Type 3 System	✓	Supplementary Trap		Dispersive Soils	

[1] Not recommended in areas of concentrated flow—refer to *U-Shaped Sediment Traps*.

[2] Very limited capture of fine clay particles, but still useful for trapping sand and silt.

Symbol 

or 

Photo 1 – Installation of a sediment fence

Photo 2 – Sediment fence located down-slope of multi-dwelling building site

Key Principles

1. Primarily used to collect coarse sediments. Sediment fences have a poor capture rate of the finer sediment particles, thus operators should not expect to see any significant change in the colour or turbidity of water passing through the fence.
2. Treatment is primarily achieved through gravity-induced ‘settlement’ resulting from the temporarily ponding of sediment-laden water up-slope of the fence. ‘Filtration’ is only a secondary function of the fabric, if at all.
3. Critical to the effectiveness of a sediment fence is the ‘surface area’ of the pond that forms up-slope of the fence. Therefore, sediment fences need to be installed such that the total surface area of ponding up-slope of the fence is maximised.
4. Optimum performance can be achieved by installing the fence in a manner that allows water to pond either:
   - uniformly along the fence (i.e. a fence located along a line of constant elevation); or
   - at regular intervals along the fence (i.e. a fence installed at a slight angle to the slope, but with regular ‘returns’ installed along the length of the fence).
5. Woven and composite fabrics perform slightly different tasks and their selection depends on site conditions.
6. Though often referred to as ‘silt fences’, a sediment fence is unlikely to trap significant quantities of fine silts (< 0.02mm), thus the term is considered an inappropriate description.
7. A sediment fence in its standard installation is only suitable for the treatment of ‘sheet’ flows. If concentrated flow exist, such as in a minor drain, then a *U-Shaped Sediment Trap*, or other more appropriate sediment trap should be used.

Design Information

Table 1 provides the recommended **maximum** slope length up-slope of a sediment fence.

Table 1 – Recommended maximum slope length up-slope of a sediment fence on non-vegetated slopes [1]

Batter slope			Horizontal spacing (m)	Vertical spacing (m)
Percentage	Degrees	(H):(V)		
1%	0.57	100:1	60 [2]	0.6 [2]
2%	1.15	50:1	60	1.2
4%	2.29	25:1	40	1.6
6%	3.43	16.7:1	32	1.9
8%	4.57	12.5:1	28	2.2
10%	5.71	10:1	25	2.5
15%	8.53	6.67:1	19	2.9
20%	11.3	5:1	16	3.2
25%	14.0	4:1	14	3.5
30%	16.7	3.33:1	12	3.5
40%	21.8	2.5:1	9	3.5
50%	26.6	2:1	6	3.0

[1] Maximum recommended spacings is based on minimising the risk of rill erosion on low to moderately erodible soil. In areas of highly erodible soil, the slope length may need to be reduced.

[2] Recommended maximum slope length above a sediment fence is 60m.

The maximum slope lengths presented in Table 1 for land slopes steeper than 2% may be represented by Equation 1.

Maximum **horizontal** slope length (m) = 100/(batter slope (%)) $^{0.64}$ (Eqn 1)

The allowable flow rate per meter length of sediment fence should, wherever possible, be determined from actual fabric testing. However, the actual flow rate at any point in time will depend on the degree of sediment blockage of the fabric.

In the absence of testing data, preliminary design flow rates can be obtained from Table 2.

Table 2 – Typical as-new and design flow rates for sediment fence fabric [1]

Depth up-slope of fence (m)	'As new' flow rate (L/s/m)		'Design' flow rate (L/s/m) [2]	
	Woven fabrics	Composite	Woven fabrics	Composite
0.2	2.6	4.8	1.3	2.4
0.4	5.6	10.6	2.8	5.3
0.6	9.0	17.8	4.5	8.9
0.8	12.6	26.2	6.3	13.1

[1] Flow rates are based on simplified test results that may not extrapolate well to actual field conditions.

[2] Suggested 'design' flow rates are based on an assumed 50% sediment blockage of the fabric.

Technical Note:

Australian Standards indicate that the flow rate through geotextiles for a given hydraulic head can be determined by extrapolating the measured flow rate at a hydraulic head of 100mm. Such analysis is **not** appropriate for woven fabrics such as sediment fence fabric. Hydraulic performance must be determined by appropriate physical testing at or above the required hydraulic head.

(a) Choice of fabric

Woven fabrics (Photo 3) are generally preferred on large sites when the service life is expected to extend over several storm events. Composite fabrics (Photo 5) are generally preferred on small soil disturbances such a building sites, or when the sediment fence is the last line of defence prior to the runoff discharging from the site or entering a water body.

Table 3 provides guidance on the selection of the preferred sediment fence fabric.

Table 3 – Preferred use of sediment fabrics

Fabric type	Preferred conditions of use
Woven fabrics	• Large sites when the service life is expected to extend over several storm events. • Up-slope of a Type 1 or Type 2 sediment trap.
Composite non-woven fabrics with a woven backing	• Small soil disturbances such a building sites. • When the sediment fence constitutes the last line of defence up-slope of a water body.

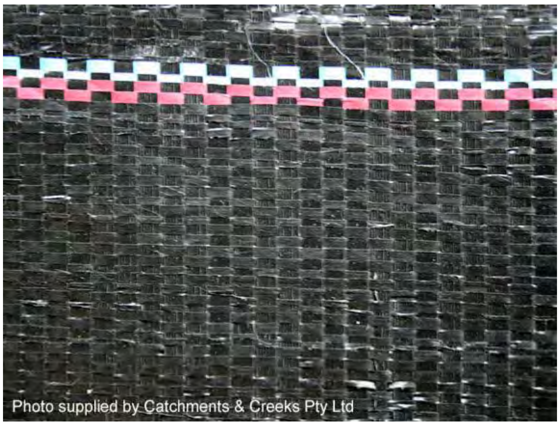


Photo 3 – Traditional woven sediment fence fabric

Photo 4 – Shade cloth MUST NOT be used

Composite fabrics, incorporating a non-woven fabric with woven fabric backing, typically have a higher flow rate (when first installed) due to the additional needle punching required to 'sew' the two fabrics together.

Composite fabrics are installed with the woven fabric as the down-slope face of the fence.

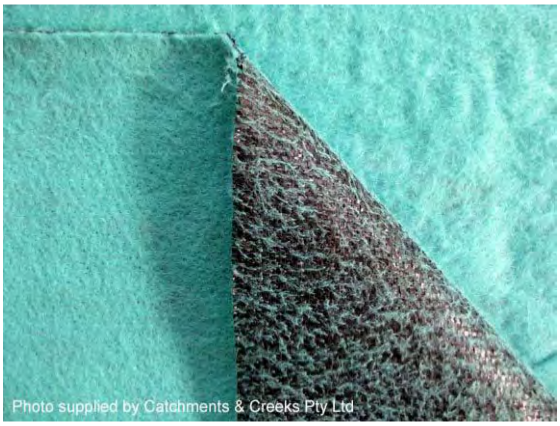


Photo 5 – Composite fabric with the woven (black) backing being the down-slope face of the sediment fence

Photo 6 – Filter cloth MUST NOT be used unless used in the construction of a 'Filter Fence' adjacent to a stockpile

Sediment fence fabric must be manufactured from either woven UV-stabilised polyester or polypropylene fabric, or a non-woven geotextile reinforced with a UV-stabilised polyester or polypropylene mesh.

Table 4 provides the recommended material properties of woven fabrics.

Table 4 – Recommended woven sediment fence material property requirements

Material property	Test method	Units	Typical value
Flow rate	AS 3706.9	L/s/m^2 (under 100 mm head)	15
Wide strip tensile strength	AS 3706.2	kN/m	10 both directions
Pore size (EOS) (O_{95})	AS 3706.7	mm x 10^{-3}	< 250
Mass per unit area	AS 3706.1	gsm	90
UV resistance	AS 3706.11	% retained (672 hours)	
Width	–	mm	730–910

Table 5 provides the recommended material properties of composite fabrics.

Table 5 – Recommended composite sediment fence material property requirements

Material property	Test method	Units	Typical value
Flow rate	AS 3706.9	L/s/m^2 (under 100 mm head)	145
Wide strip tensile strength	AS 3706.2	kN/m	17 both directions
Pore size (EOS) (O_{95})	AS 3706.7	mm x 10^{-3}	110
Mass per unit area	AS 3706.1	gsm	225
UV resistance	AS 3706.11	% retained (672 hours)	
Width	–	mm	730–910

(b) Location of a sediment fence

Wherever practical, the sediment fence should be installed along the contour, thus maintaining sheet flow conditions across the fence. If located at an angle to the contour, the fence needs to be installed with regular 'returns' to avoid water concentrating along the fence. Even if the fence is located along the contour, the use of regular returns is still recommended (refer to Figure 1).

The maximum spacing of fence 'returns' should be 20m if the fence is installed along the contour, or 5 to 10m (depending on slope) if located at an angle to the contour (Figure 2).

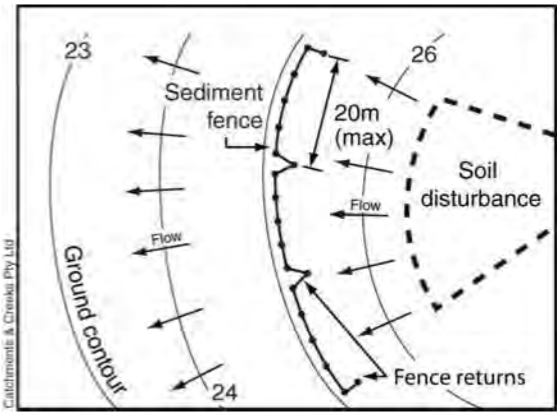


Figure 1 – Fence installed along the contour

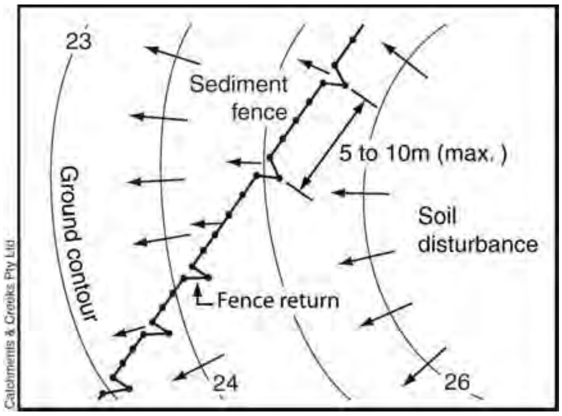


Figure 2 – Fence install down a slope

Wherever practical, allow at least 4.5m between the sediment fence and a single-storey building; 7.5m between the fence and a multiple-storey building; and at least 2m between the fence and the toe of a fill slope or stockpile (Figure 3).

A double sediment fence (Figure 4, Photo 8), or sediment fence with up-slope straw bale (Photo 7) can be used to reduce the risk of shifting fill damaging the fence.

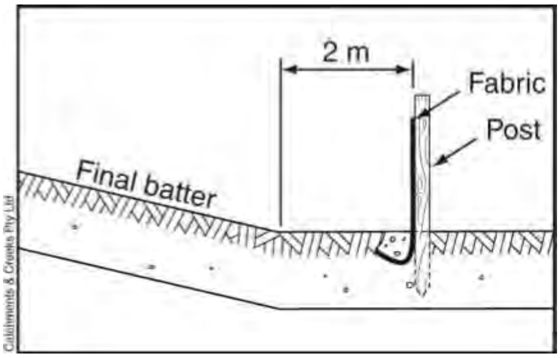


Figure 3 – Fence installation at base of slope

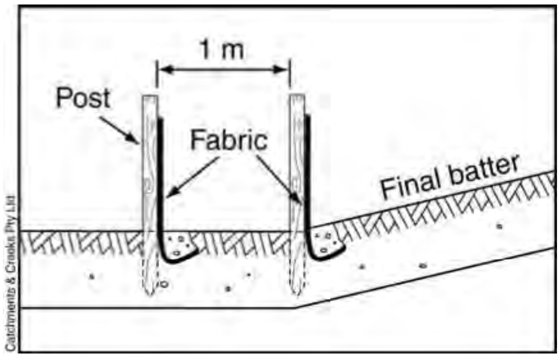


Figure 4 – Double sediment fence installed at the based of a fill slope

Photo 7 – Use of straw bales to prevent direct contact of stockpiles with the fence

Photo 8 – Double sediment fence

(c) Installation of a sediment fence

At least 300mm of fabric must be buried in either a 200mm trench (Figure 8, Photo 13), or under a continuous 100mm high layer of sand or aggregate (Photo 15), but **not** earth.

Straw bales can be placed up-slope of the fence (Figure 9) to retain settled sediment away from the fabric, thus improving the ease of ongoing maintenance (i.e. sediment removal). Alternatively, a small trench can be formed along the contour, up-slope of the fence.

Both ends of the fence should be turned up the slope to minimise the risk of flow bypassing around the ends of the fence (Figure 5, Photo 21).

Support posts should be spaced no greater than 3m if the fence is supported by a top support wire or weir mesh backing (Figure 7), otherwise no greater than 2m (Figure 6). The recommended maximum spacing of support posts is summarised in Table 6.

Table 6 – Maximum spacing of support post

Maximum post spacing	Installation condition
2m	No support wire or backing mesh.
3m	Support weir attached along top of the fabric at 1m intervals. Wire mesh or PVC safety mesh backing.

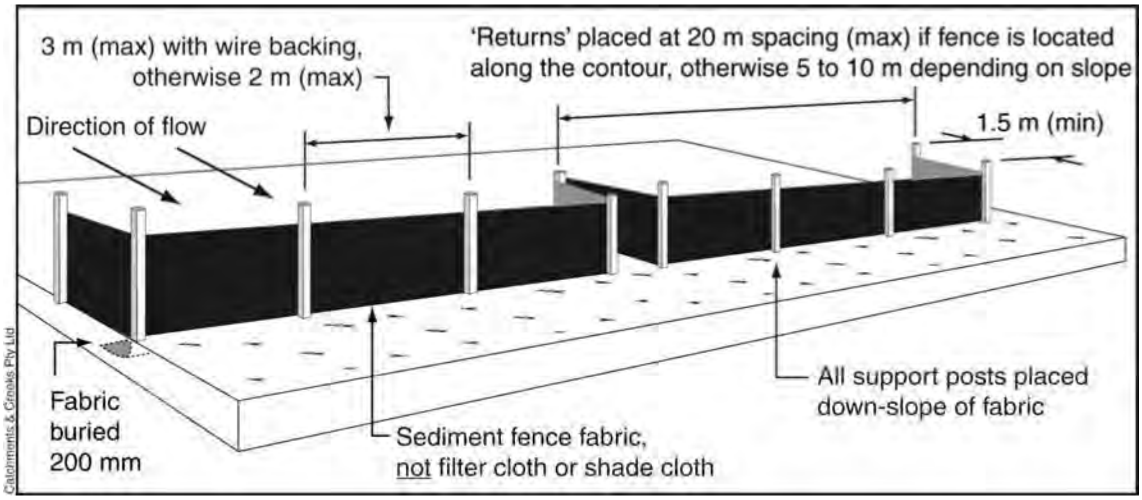


Figure 5 – Typical installation of a sediment fence

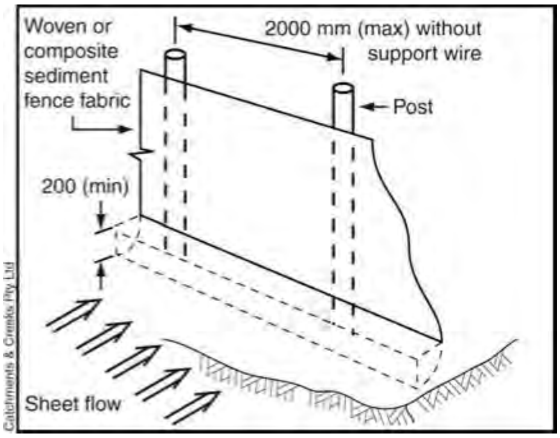


Figure 6 – Installation of a sediment fence without wire backing

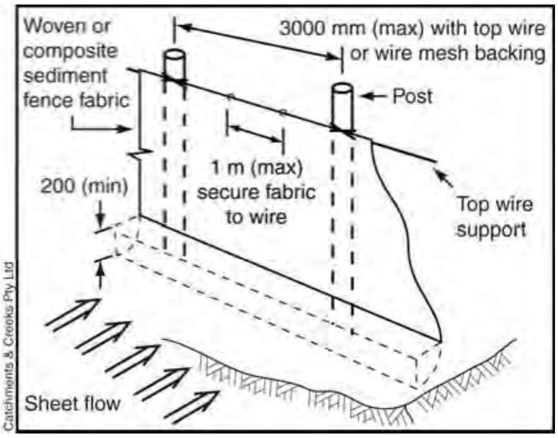


Figure 7 – Installation of a sediment fence with top wire support

Wherever possible, construct the sediment fence from a continuous roll. To join fabric either attach each end to individual stakes (Figure 10), holding the stakes together, rotate the stakes 180 degrees, then drive the two stakes into the ground; or overlap the fabric to the next support post (Figure 11).

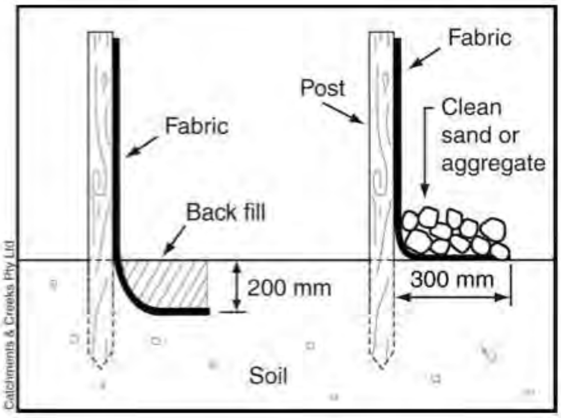


Figure 8 – Anchoring the fabric

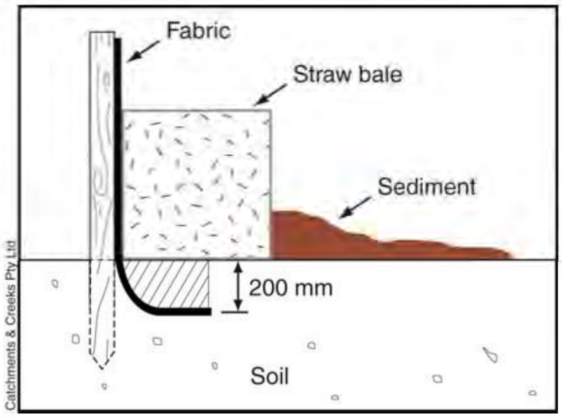


Figure 9 – Use of straw bales as a fabric-sediment separator

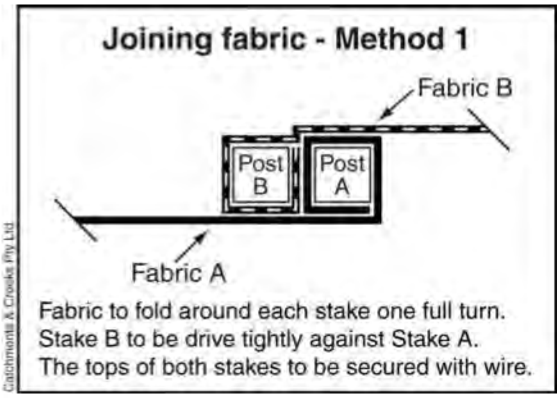


Figure 10 – Joining fabric (Option 1)

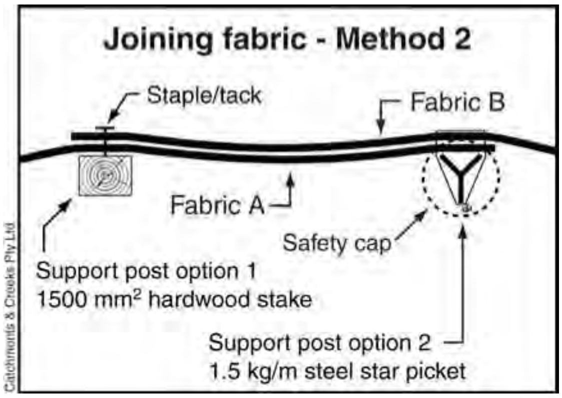


Figure 11 – Overlapping fabric (Option 2)

Photo 9 – Sediment fence placed along the contour

Photo 10 – Use of fence 'returns'

Photo 11 – Placement of fence off the contour with regular fence 'returns'

Photo 12 – Alternative design of a fence 'return'

Photo 13 – Trenching the fabric

Photo 14 – Inappropriate installation

Photo 15 – Bottom of fabric buried under aggregate

Photo 16 – Inappropriate use of sand to bury the fabric

Photo 17 – Straw bales placed up-slope of fence to separate sediment and fabric

Photo 18 – Inappropriate installation of the posts up-slope of the fabric

Photo 19 – Inappropriate junction

Photo 20 – Gaps in fence are not allowed

Photo 21 – Installation without backing weir/mesh

Photo 22 – Installation with top wire support

Photo 23 – Installation with weir mesh

Photo 24 – Installation using fence support

Photo 25 – Installation with safety fencing used as support

Photo 26 – A sediment fence braced for possible high flows

Photo 27 – Example of tacking

Photo 28 – Safety cap on a steel stake

Photo 29 – Flow diversion by fence

Photo 30 – No end return

Photo 31 – Damage by shifting fill

Photo 32 – Fence placed too close to fill

Photo 33 – Evidence of hydraulic wash-out under fence caused by poor trenching

Photo 34 – Sediment not removed after storm

Photo 35 – Flow allowed to bypass the fence

Photo 36 – Spill-through weirs must not discharge onto unstable land

(d) Use of spill-through weirs

Where appropriate, spill-through weirs can be installed into the fence to reduce hydraulic pressure and reduce the risk of hydraulic failure.

The required width (W) of the spill-through weir depends on the nominated design flow rate. The weir flow equation for a rectangular spill-through weir is provided below as Equation 2, as well as tabulated in Table 7.

$$Q = 1.7\, W\, H^{3/2} \qquad \text{(Eqn 2)}$$

where: Q = Design flow rate (usually 0.5 times the 1 in 1 year ARI peak discharge) [m^3/s]
W = Weir width [m]
H = Hydraulic head = height of upstream water level above weir crest [m]

Table 7 – Flow rates passing over a spill-through weir (m^3/s)

Hydraulic head, H (m)	Spill-through weir width, W (m)									
	0.3	**0.5**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**3.5**	**4.0**	**4.5**
0.10	0.016	0.027	0.054	0.081	0.108	0.134	0.161	0.188	0.215	0.242
0.15	0.030	0.049	0.099	0.148	0.198	0.247	0.296	0.346	0.395	0.444
0.20	0.046	0.076	0.152	0.228	0.304	0.380	0.456	0.532	0.608	0.684
0.25	0.064	0.106	0.213	0.319	0.425	0.531	0.638	0.744	0.850	0.956
0.30	0.084	0.140	0.279	0.419	0.559	0.698	0.838	0.978	1.12	1.26
0.35	0.106	0.176	0.352	0.528	0.704	0.880	1.06	1.23	1.41	1.58
0.40	0.129	0.215	0.430	0.645	0.860	1.08	1.29	1.51	1.72	1.94

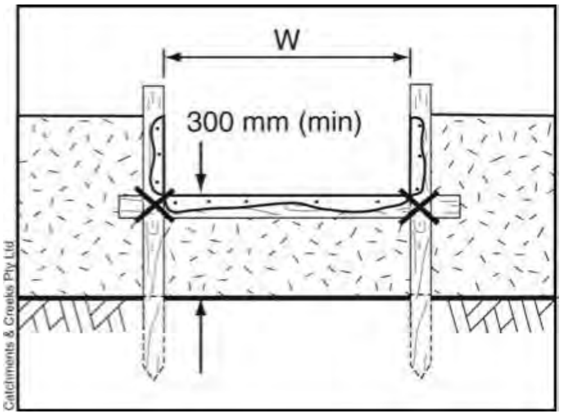


Figure 12 – Spill-through weir profile

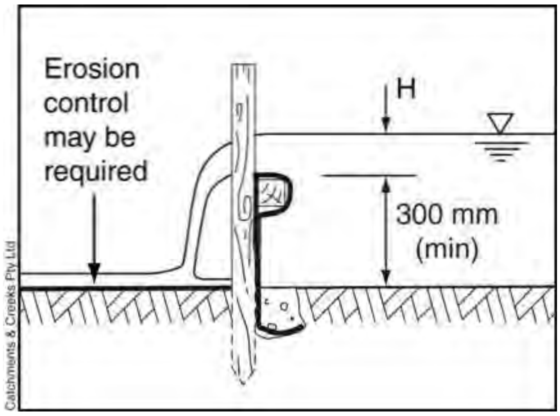


Figure 13 – Side profile of a spill-through weir

Photo 37 – Spill-through weir (down-slope side) with rock splash pad

Photo 38 – Spill-through weir with outlet chute

Photo 39 – Spill-through weir with up-slope aggregate filter

Photo 40 – Inappropriate placement of fence and installation of spill-through weir

If large sediment flows are expected, then a *Coarse Sediment Trap* can be used as an outlet structure for a sediment fence as shown in Figure 14. However, in most circumstances, a more elaborate outlet system would be required such as a Type 1 or Type 2 sediment trap.

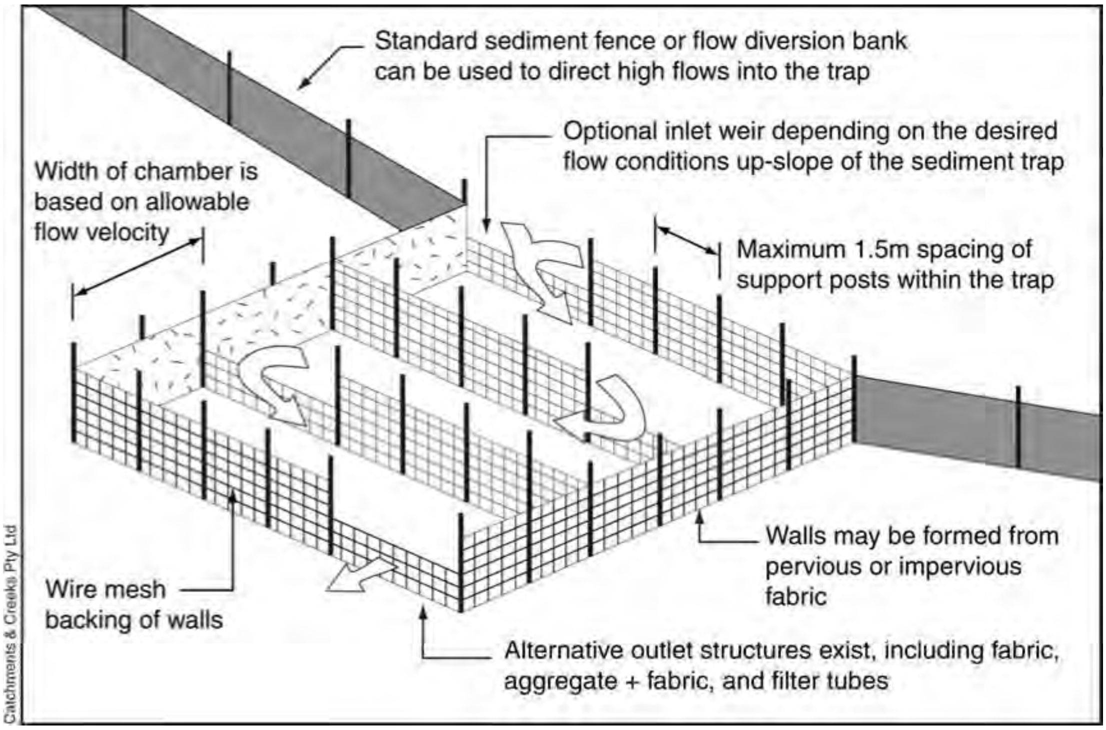


Figure 14 – Coarse sediment trap outlet structure

Description

A sediment fence consists of specially manufactured woven fabric attached to support posts. The typical height of the fence is around 600 to 700mm.

Most sediment fences are self-supporting; however, in appropriate circumstances the fence may be attached to an existing porous structure such as a property fence.

The fabric may be manufactured from either woven fabric, or a composite of woven and non-woven fabrics. The incorporation of a woven fabric is essential for the control of water flow needed to allow adequate temporary ponding up-slope of the fence.

Purpose

Used as a Type 3 sediment trap on small catchments, or as a supplement to Type 1 or 2 sediment traps on large catchments.

Limitations

Though often referred to as a 'silt fence', these Type 3 sediment traps have little impact on fine silts (< 0.02mm).

A sediment fence in its standard installation is only suitable for the treatment of 'sheet' flows. If concentrated flow exist, such as in a minor drain, then a *U-Shaped Sediment Trap*, or other more appropriate sediment trap should be used.

Most fabrics have an effective service life of around 6 months (check with manufacturer or distributor).

Advantages

Reasonably easy to install.

Has the ability to control sediment runoff close to the source of the erosion.

Disadvantages

Time-consuming to install, which often results in poor installation.

Easily damaged by construction equipment and shifting earth (Photos 31 & 32).

Can cause the concentration of stormwater runoff if poorly located, or installed.

Sediment fences are one of the most missed used sediment control devices, usually because they are either not installed in appropriate locations, or are installed in a manner that does not allow adequately water ponding up-slope of the fences.

Common Problems

If not installed along the contour, a sediment fence can result in flows being deflected along the fence (Photo 29).

If the ends of the fence are not turned up the slope, water and sediment can pass around the end of the fence (Photo 30).

If gaps exist in the fence (Photos 19 & 20), then water is prevented from ponding up-slope of the fence, thus sedimentation does not occur.

Excessive spacing between support posts is a common problem. In extreme cases this can result in the fabric sagging close to the ground.

Fabric not adequately connected to the support posts or backing wire.

The bottom of the fabric not adequately buried into the ground or under a suitable layer of sand or aggregate. If such fences are subjected to significant storms, the bottom of the fence can 'blow-out' causing erosion down-slope of the fence (Photo 33).

Spill-through weirs may not have been installed in large catchments or areas of high rainfall, thus increasing the risk of flow damage to the fence.

Crest of spill-through weir set too close to the ground (should be at least 300mm above ground level).

Crest of spill-through weir is set above the ground level at the ends of the fence, thus allowing flow bypassing rather than discharge over the weir (Photo 40).

Special Requirements

Woven fabrics are generally preferred on large sites when the service life is expected to extend over several storm events. Composite fabrics are generally preferred on small soil disturbances such a building sites, or when the sediment fence is the last line of defence prior to the runoff entering a water body.

Ideally, the sediment fence should be installed along the contour, thus maintaining sheet flow conditions across fence. If located across the contour, the fence should be installed with regular 'returns' to avoid water concentrating along the fence.

At least 300mm of fabric must be buried in either a 200mm trench, or under a continuous, 100mm high layer of sand or aggregate, but not earth.

Straw bales can be placed up-slope of the fence to retain bulk sediment away from the fabric, thus improving the ease of sediment removal. Alternatively, a small trench can be formed along the contour, up-slope of the fence. However, in all cases the aim should be to minimise high sediment flows so that such fence modifications become the exception, not the rule!

Where appropriate, spill-through weirs can be installed into the fence to reduce hydraulic pressure and reduce the risk of hydraulic failure.

Location

Install along the contour wherever possible.

Allow at least 4.5m between the fence and single-story buildings; 7.5m between the fence and multiple-story buildings; and at least 2m between the fence and the toe of a fill slope or stockpile.

Site Inspection

Ensure the sediment fence will adequately pond water up-slope of the fence.

Ensure the fabric is adequately buried.

Check the spacing of support posts/stakes.

Check for excessive sediment deposition.

Investigate the source of excessive sediment deposits.

Ensure the selection of appropriate fabric (i.e. woven or composite).

Check for damage to the fabric.

Check for erosion down-slope of any spill-through weirs.

Ensure the fence is not concentrating or diverting flows in an undesirable manner.

Materials

- Fabric: polypropylene, polyamide, nylon, polyester, or polyethylene woven or non-woven fabric, at least 700mm in width and a minimum unit weight of 140GSM. All fabrics to contain ultraviolet inhibitors and stabilisers to provide a minimum of 6 months of useable construction life (ultraviolet stability exceeding 70%).
- Fabric reinforcement: wire or steel mesh minimum 14-gauge with a maximum mesh spacing of 200mm.
- Support posts/stakes: $1500mm^2$ (min) hardwood, $2500mm^2$ (min) softwood, or 1.5kg/m (min) steel star pickets suitable for attaching fabric.

Installation

1. Refer to approved plans for location, extent, and required type of fabric (if specified). If there are questions or problems with the location, extent, fabric type, or method of installation contact the engineer or responsible on-site officer for assistance.
2. To the maximum degree practical, and where the plans allow, ensure the fence is located:
   (i) totally within the property boundaries;
   (ii) along a line of constant elevation wherever practical;
   (iii) at least 2m from the toe of any filling operations that may result in shifting soil/fill damaging the fence.
3. Install returns within the fence at maximum 20m intervals if the fence is installed along the contour, or 5 to 10m maximum spacing (depending on slope) if the fence is installed at an angle to the contour. The 'returns' shall consist of either:
   (i) V-shaped section extending at least 1.5m up the slope; or
   (ii) sandbag or rock/aggregate check dam a minimum 1/3 and maximum 1/2 fence height, and extending at least 1.5m up the slope.
4. Ensure the extreme ends of the fence are turned up the slope at least 1.5m, or as necessary, to minimise water bypassing around the fence.
5. Ensure the sediment fence is installed in a manner that avoids the concentration of flow along the fence, and the undesirable discharge of water around the ends of the fence.
6. If the sediment fence is to be installed along the edge of existing trees, ensure care is taken to protect the trees and their root systems during installation of the fence. Do not attach the fabric to the trees.
7. Unless directed by the site supervisor or the approved plans, excavate a 200mm wide by 200mm deep trench along the proposed fence line, placing the excavated material on the up-slope side of the trench.

8. Along the lower side of the trench, appropriately secure the stakes into the ground spaced no greater than 3m if supported by a top support wire or weir mesh backing, otherwise no greater than 2m.
9. If specified, securely attach the support wire or mesh to the up-slope side of the stakes with the mesh extending at least 200mm into the excavated trench. Ensure the mesh and fabric is attached to the up-slope side of the stakes even when directing a fence around a corner or sharp change-of-direction.
10. Wherever possible, construct the sediment fence from a continuous roll of fabric. To join fabric either:
    - (i) attach each end to two overlapping stakes with the fabric folding around the associated stake one turn, and with the two stakes tied together with wire (Method 1); or
    - (ii) overlap the fabric to the next adjacent support post (Method 2).
11. Securely attach the fabric to the support posts using 25 x 12.5mm staples, or tie wire at maximum 150mm spacing.
12. Securely attach the fabric to the support wire/mesh (if any) at a maximum spacing of 1m.
13. Ensure the completed sediment fence is at least 450mm, but not more than 700mm high. If a spill-though weir is installed, ensure the crest of the weir is at least 300mm above ground level.
14. Backfill the trench and tamp the fill to firmly anchor the bottom of the fabric and mesh to prevent water from flowing under the fence.
15. If it is not possible to anchor the fabric in an excavated trench, then use a continuous layer of sand or aggregate to hold the fabric firmly on the ground.

Additional requirements for the installation of a spill-through weir

1. Locate the spill-through weir such that the weir crest will be lower than the ground level at each end of the fence.
2. Ensure the crest of the spill-through weir is at least 300mm the ground elevation.
3. Securely tie a horizontal cross member (weir) to the support posts/stakes each side of the weir. Cut the fabric down the side of each post and fold the fabric over the cross member and appropriately secure the fabric.
4. Install a suitable splash pad and/or chute immediately down-slope of the spill-through weir to control soil erosion and appropriately discharge the concentrated flow passing over the weir.

Maintenance

1. Inspect the sediment fence at least weekly and after any significant rain. Make necessary repairs immediately.
2. Repair any torn sections with a continuous piece of fabric from post to post.
3. When making repairs, always restore the system to its original configuration unless an amended layout is required or specified.
4. If the fence is sagging between stakes, install additional support posts.
5. Remove accumulated sediment if the sediment deposit exceeds a depth of 1/3 the height of the fence.
6. Dispose of sediment in a suitable manner that will not cause an erosion or pollution hazard.
7. Replace the fabric if the service life of the existing fabric exceeds 6-months.

Removal

1. When disturbed areas up-slope of the sediment fence are sufficiently stabilised to restrain erosion, the fence must be removed.
2. Remove materials and collected sediment and dispose of in a suitable manner that will not cause an erosion or pollution hazard.
3. Rehabilitate/revegetate the disturbed ground as necessary to minimise the erosion hazard.

Sediment Basin

Sediment Basins

SEDIMENT CONTROL TECHNIQUE

Type 1 System	✓	Sheet Flow		Sandy Soils	✓
Type 2 System		Concentrated Flow	✓	Clayey Soils	✓
Type 3 System		Supplementary Trap		Dispersive Soils	[1]

[1] Settlement of dispersive soils may be achieved through the flocculation of 'wet' sediment basins.

Symbol

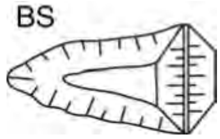


Photo 1 – Example of a 'dry' (Type C) sediment basin

Photo 2 – Example of a 'wet' (Type F/D) sediment basin

Key Principles

1. Sediment trapping can be achieved by both particle settlement within the settling pond (all basin types), and by the filtration of minor flows passing through the aggregate or geotextile filter (dry basins).
2. For continuous flow basins (i.e. dry basins) the critical design parameter for optimising particle settlement is the 'surface area' of the settling pond. For plug flow basins (i.e. wet basins) the critical design parameter is the 'volume' of the settling pond.
3. The critical design parameter for the filtration process (dry basins) is the design flow rate for water passing through the filter, which is related to the depth of water (hydraulic head), and the surface area and flow resistance of the filter.
4. Even if a basin is full of water, it can still be effective in the removal of coarse sediment from any flows passing through the basin. Therefore, unlike permanent stormwater settling ponds, high flows resulting from storms in excess of the 'design storm' should **not** be bypassed around a construction site sediment basin.

Design Information

This fact sheet summaries design requirements for three types of temporary sediment basins, Type C for coarse-grained soils, Type F for fine-grained soils, and Type D for dispersive soils. Detailed design procedures are provided in Appendix B of the IECA (Australasia) "Best Practice Erosion and Sediment Control" document.

Sediment basins should be designed and operated in a manner that produces near clear-water discharges (i.e. total suspended solids concentrations not exceeding 50mg/L) during non-overtopping events, especially following periods of light rainfall.

Summary of design requirements

Table 1 provides a summary of the recommended design requirements.

Table 1 – Summary of sediment basin design requirements

Parameter	Type C basin	Type F & D basins
Soil characteristic	Less than 33% of soil finer than 0.02mm and no more than 10% of soil dispersive.	**Type F:** More than 33% of soil finer than 0.02mm. **Type D:** More than 10% of soil dispersive, or where turbidity control is essential.
Settling pond sizing, surface area (A_s), or settling volume (V_s)	$A_s = 3400\ H_e\ (Q)$ Q = 0.5 times 1 in 1yr flow	$V_s = 10\ R_{(Y\%,\ 5\text{-day})}\ C_v\ A$
Length to width ratio	Hydraulic efficiency factor (H_e) is reduce with increasing length to width ratio	L:W of 3:1 is highly desirable
Minimum depth of settling zone	0.6m	0.6m
Sediment storage volume	100% of settling volume	50% of settling volume
Use of inlet chamber	Desirable if length to width ratio is less than 3:1, or if inflow is concentrated with high flow velocity.	
Internal baffles	Desirable if length to width ratio is less than 3:1	
Use of outlet chamber	Essential if skimmer pipe outlet system is employed	Use depends on type of outlet system adopted
Control inflow conditions	Used to control erosion at inlets and, where practicable, ensure the inflow pipe invert is above the spillway crest elevation.	
Pre-treatment pond	Used to reduce the cost and frequency of de-silting operations.	
Primary outlet	Ensure choice of outlet system is compatible with basin type.	
Emergency spillway minimum design capacity	Less and 3 month design life: capacity of 1 in 10 year ARI. 3 to 12 months design life: capacity of 1 in 20 year ARI. Greater than 12 months design life: capacity of 1 in 50 yr ARI.	
Elevation from top of riser pipe outlet to spillway crest	300mm (min)	N/A
Freeboard from maximum pond water level to top of virgin soil bank	150mm (min)	150mm (min)
Freeboard from maximum pond water level to top of fill embankment	300mm (min)	300mm (min)
Minimum freeboard along spillway chute	300mm (min)	300mm (min)
Minimum embankment crest width	2.5m	2.5m
Maximum gradient of access ramp	6:1	6:1
Chemical flocculation	As required to satisfy water quality objectives.	**Type F:** As required to satisfy water quality objectives. **Type D:** Essential

Design procedure:

Step 1 – Assess the need for a sediment basin

Sediment basins are recommended for any sub-catchment with a catchment area exceeding 2500m^2 and an estimated soil loss rate that exceeds the equivalent of 150t/ha/yr.

Step 2 – Selection of the required type of basin

Selection of the type of sediment basin is governed by the site's soil properties as outlined in Table 2.

Table 2 – Selection of basin type

Soil and/or catchment conditions [1]	Basin type
Less than 33% of soil finer than 0.02mm (i.e. d_{33} > 0.02mm) and no more than 10% of soil dispersive. [2]	Type C basin
More than 33% of soil finer than 0.02mm (i.e. d_{33} < 0.02mm) and no more than 10% of soil dispersive. [2]	Type F basin
More than 10% of soil dispersive [2], or when a Stormwater Management Plan (SMP), or adopted Water Quality Objectives (WQOs) specify strict controls on turbidity levels and/or suspended solids concentrations for discharged waters.	Type D basin

[1] If more than one soil type exists on the site, then the most stringent criterion applies (i.e. Type D supersedes Type F, which itself supersedes Type C).

[2] The percentage of soil that is dispersive is measured as the combined decimal fraction of clay (<0.002mm) plus half the percentage of silt (0.002–0.02mm), multiplied by the dispersion percentage.

Step 3 – Determine the basin location

Sediment basins should be located within a sub-catchment so as to maximise its overall sediment trapping capabilities of that sub-catchment. Issues that need to be given appropriate consideration include:

- Locate all basins within the relevant property boundary.
- Locate sediment basins above the 1 in 5 year ARI flood level. Where this is not practicable, then all reasonable efforts should be taken to maximise the flood immunity of the basin.
- Avoid locating a basin in an area where adjacent construction works may limit the operational life of the basin.
- Ensure basins have suitable access for maintenance and de-silting.

Step 4 – Diversion of 'clean' water around a basin

Up-slope 'clean' water should be diverted around the sediment basin to decrease the size and cost of the basin and increase its efficiency. The adopted flow diversion systems may need to be modified for each stage of construction as new areas of land are first disturbed, then stabilised.

'Clean' water is defined as water that has not been contaminated within the property, or by activities directly associated with the construction/building works.

Step 5a – Sizing of the settling pond, Type C basins

The settling pond within a Type C sediment basin is divided (horizontally) into two zones: the upper *settling zone* and the lower *sediment storage zone* as shown in Figure 1.

The minimum volume of the upper settling zone is defined by Equation 1.

$$A_s = 3400\ H_e\ (Q) \qquad \text{(Eqn 1)}$$

where: A_s = Surface area of settling pond at the base of the settling zone [m^2]

H_e = Hydraulic efficiency correction factor

Q = design flow rate [m^3/s]

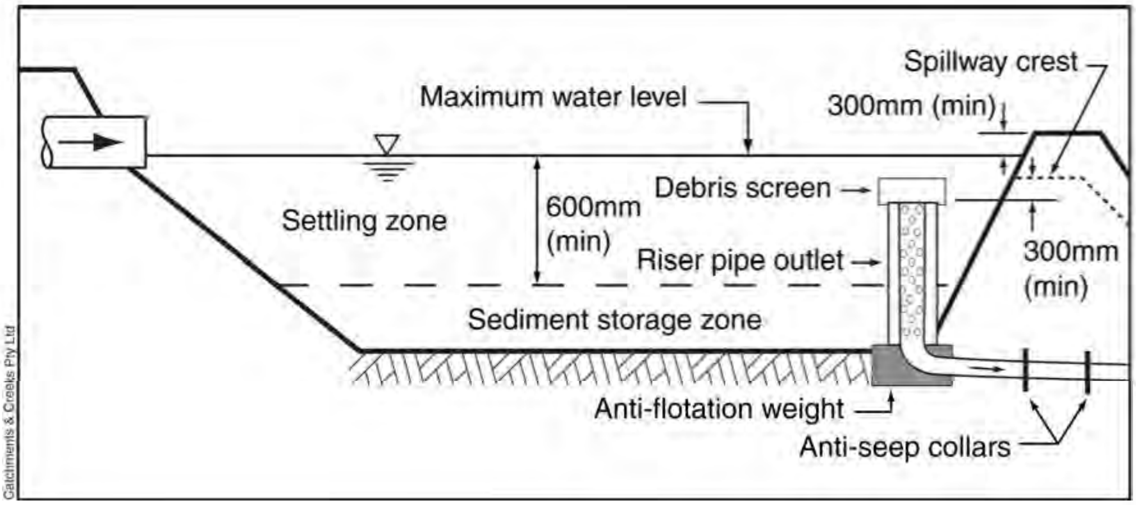


Figure 1 – Type C *Sediment Basin* with riser pipe outlet (long section)

Unless otherwise required by a regulatory authority, the design flow rate (Q) for a Type C sediment basin must be 0.5 times the peak discharge for the 1 in 1 year ARI storm.

The minimum recommended depth of the settling zone is 0.6m, or L/200 for basins longer than 120m (where L = effective basin length). Settling zone depths greater than 1m should be avoided if settlement velocities are expected to be slow.

The desirable minimum length to width ratio of 3:1, otherwise internal baffles need to be used wherever practicable to prevent short-circuiting of flows.

The hydraulic efficiency correction factor (H_e) depends on flow conditions entering the basin and the shape of the settling pond. Table 3 provides recommended values of the hydraulic efficiency correction factor.

Where available space does not permit construction of the ideal sediment basin, then a smaller basin may be used; however, erosion control and site rehabilitation standards need to be appropriately increased to a higher standard to compensate.

A Type C sediment basin that is less than the ideal size should be considered either a Type 2 or Type 3 sediment trap based on the effective particle settlement capabilities.

Table 3 – Hydraulic efficiency correction factor (H_e)

Flow condition within basin	Effective [1] length:width	H_e
Uniform or near-uniform flow conditions across the full width of basin. [2] For basins with concentrated inflow, uniform flow conditions may be achieved through the use of an appropriate inlet chamber arrangement.	1:1	1.2
	3:1	1.0
Concentrated inflow (piped or overland flow) primarily at one inflow point and no inlet chamber to evenly distribute flow across the full width of the basin.	1:1	1.5
	3:1	1.2
	6:1	1.1
	10:1	1.0
Concentrated inflow with two or more separate inflow points and no inlet chamber to evenly distribute flow across the full width of the basin.	1:1	1.2
	3:1	1.1

[1] The effective length to width ratio for sediment basins with internal baffles is measured along the centreline of the dominant flow path.

[2] Uniform flow conditions may also be achieved in a variety of ways including through the use of an inlet chamber and internal flow control baffles.

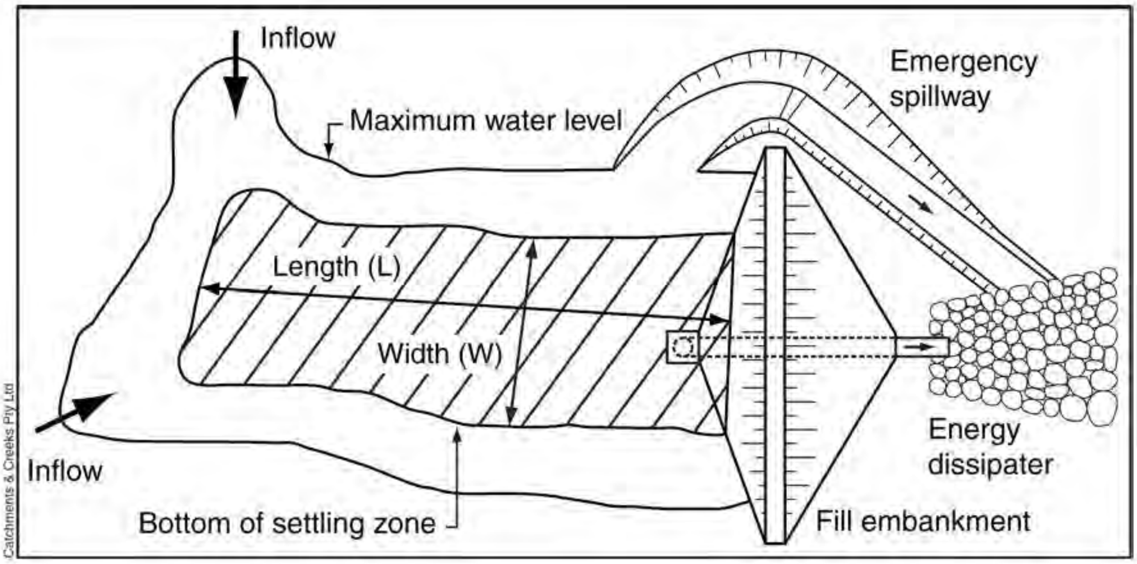


Figure 2 – Type C sediment basin with riser pipe outlet (plan view)

Step 5b – Sizing of the settling pond, Type F & D basins

The settling pond within a Type F or Type D sediment basin is divided (horizontally) into two zones: the upper *settling zone* and the lower *sediment storage zone* as shown in Figure 3.

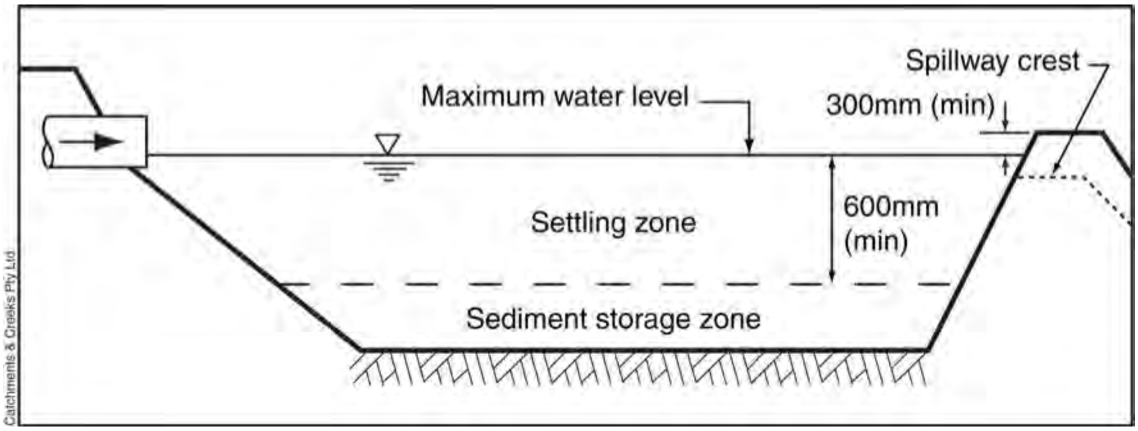


Figure 3 – Settling zone and sediment storage zone

The minimum volume of the upper settling zone is defined by Equation 2.

$$V_s = 10 . R_{(Y\%,5\text{-day})} . C_v . A \qquad \text{(Eqn 2)}$$

where: V_s = volume of the settling zone [m^3]

$R_{(Y\%,5\text{-day})}$ = Y%, 5-day rainfall depth [mm]

C_v = volumetric runoff coefficient

A = effective catchment surface area connected to the basin [ha]

The regulatory authority should provide the required design rainfall probability (Y%) and rainfall depth (R) for a given location.

The minimum recommended depth of the settling zone is 0.6m, or L/200 for basins longer than 120m (where L = effective basin length). Settling zone depths greater than 1m should be avoided if settlement velocities are expected to be slow.

The desirable minimum length to width ratio of 3:1 is recommended for Type F and Type D basins. The length to width ratio is important for Type F and Type D basins because they operate as continuous-flow settling ponds (as per Type C basins) once flow begins to discharge over the emergency spillway.

Step 6 – Determination of sediment storage volume

The sediment storage zone lies below the settling zone as defined in Figure 4. The recommended sediment storage volume can be determined from Table 4.

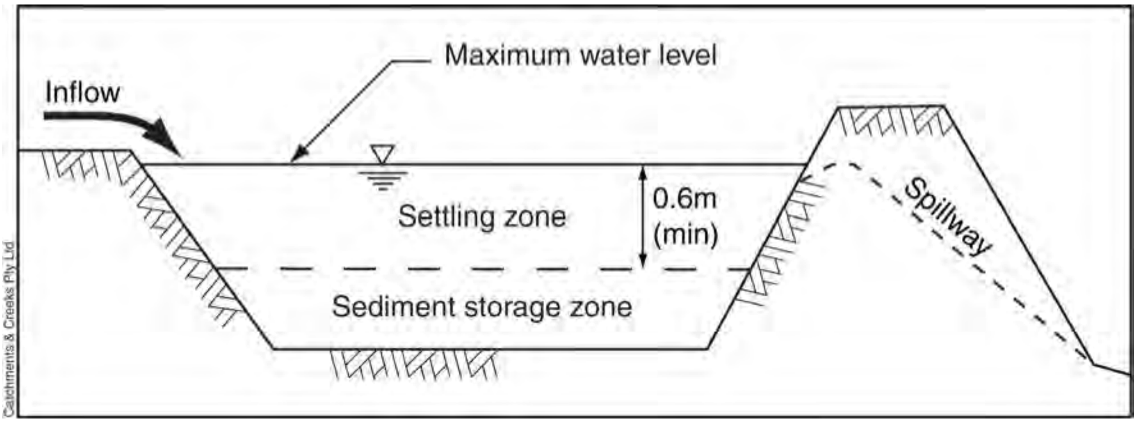


Figure 4 – Settling zone and sediment storage zone

Table 4 – Sediment storage volume

Basin type	Sediment storage volume
Type C	100% of settling volume
Type F and Type D	50% of settling volume

Alternatively, the volume of the sediment storage zone can be determined by estimating the expected sediment runoff volumes over the desired maintenance period, typically not less than 2 months.

Step 7 – Select internal and external bank gradients

Recommended bank gradients are provided in Table 5.

Table 5 – Suggested bank slopes

Slope (H:V)	Bank/soil description
2:1	Good, erosion-resistant clay or clay-loam soils
3:1	Sandy-loam soil
4:1	Sandy soils
5:1	Unfenced Sediment Basins accessible to the public
6:1	Mowable, grassed banks.

All earth embankments in excess of 1m in height should be certified by a geotechnical engineer/specialist as being structurally sound for the required design criteria and anticipated period of operation.

If public safety is a concern, and the basin is not to be fenced (not recommended), and the basin's internal banks are steeper than 5:1(H:V), then at least one bank should be turfed a width of at least 2m from top of bank to the toe of bank to allow egress during wet weather.

Step 8 – Design of flow control baffles

Baffles may be used for a variety of purposes including:

- energy dissipation (inlet chambers);
- the control of short-circuiting (internal baffles);
- minimising sediment blockage of the low-flow outlet structure (outlet chambers).

The need for flow control baffles should have been established in Step 5a based on the basin's length to width ratio. Both inlet baffles (inlet chambers) and internal baffles can be used to improve the hydraulic efficiency of the basin, and thus reduce the size of the settling pond through modification of the hydraulic efficiency correction factor.

(a) Inlet chambers

Flow control baffles or similar devices may be placed at the inlet end of a sediment basin to form an inlet chamber (Figures 5 & 6). These chambers are used to reduce the adverse effects of inlet jetting caused by concentrated, point source inflows. The objective of the inlet chamber is to produce near-uniform flow conditions across the full width of the settling pond.

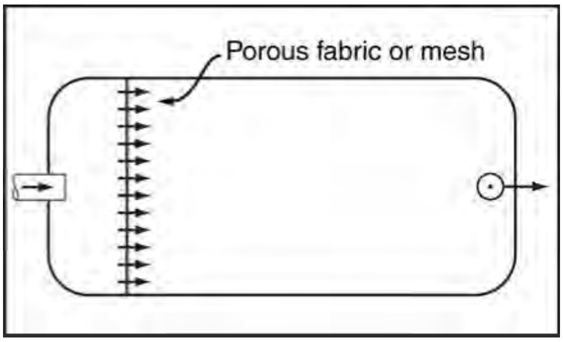


Figure 5(a) – Porous barrier inlet chamber

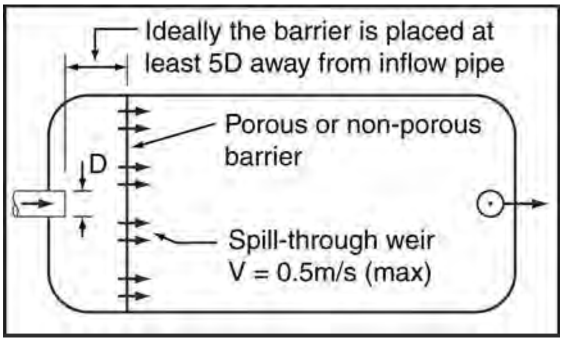


Figure 6(a) – Alternative inlet chamber design

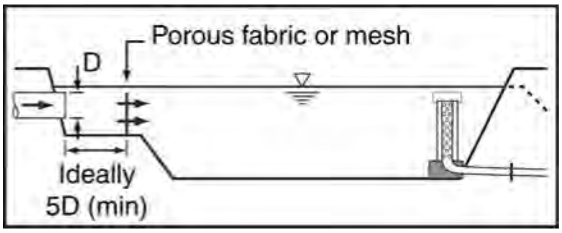


Figure 5(b) – Typical layout of inlet chamber with opposing inlet pipe (Type C basin)

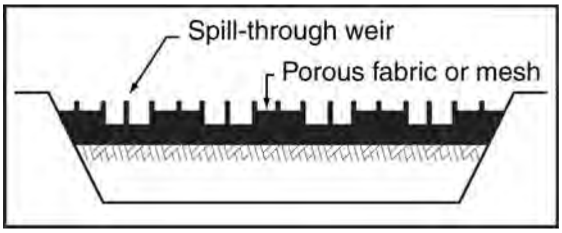


Figure 6(b) – Barrier with multiple spill-through weirs

The use of an inlet chamber is usually governed by the need to adopt a low hydraulic efficiency correction factor (H_e) in Step 5a. The incorporation of inlet baffles should be given serious consideration within Type C basins if the expected velocity of any concentrated inflows exceeds 1m/s.

Table 6 summaries the design of various inlet chambers.

Table 6 – Design of various inlet chambers

Baffle type	Discussion
Shade cloth	Typical spacing between support posts is 0.5 to 1.0m.
Perforated fabric	Heavy-duty plastic sheeting or woven fabric perforated with approximately 50mm diameter holes at approximately 300mm centres. Typical spacing between support posts is 0.5 to 1.0m.
Barrier with spill-through weirs	A porous or non-porous barrier constructed across the full width of the settling pond.

(b) Internal baffles

Internal baffles are used to increase the effective length-to-width ratio of the basin. Figure 7 demonstrates the arrangement of internal flow control baffles for various settling pond layouts.

If internal baffles are used, then the flow velocity within the settling pond must not exceed the sediment scour velocity of 0.2m/s for 0.02 to 0.10mm critical particle diameters respectively.

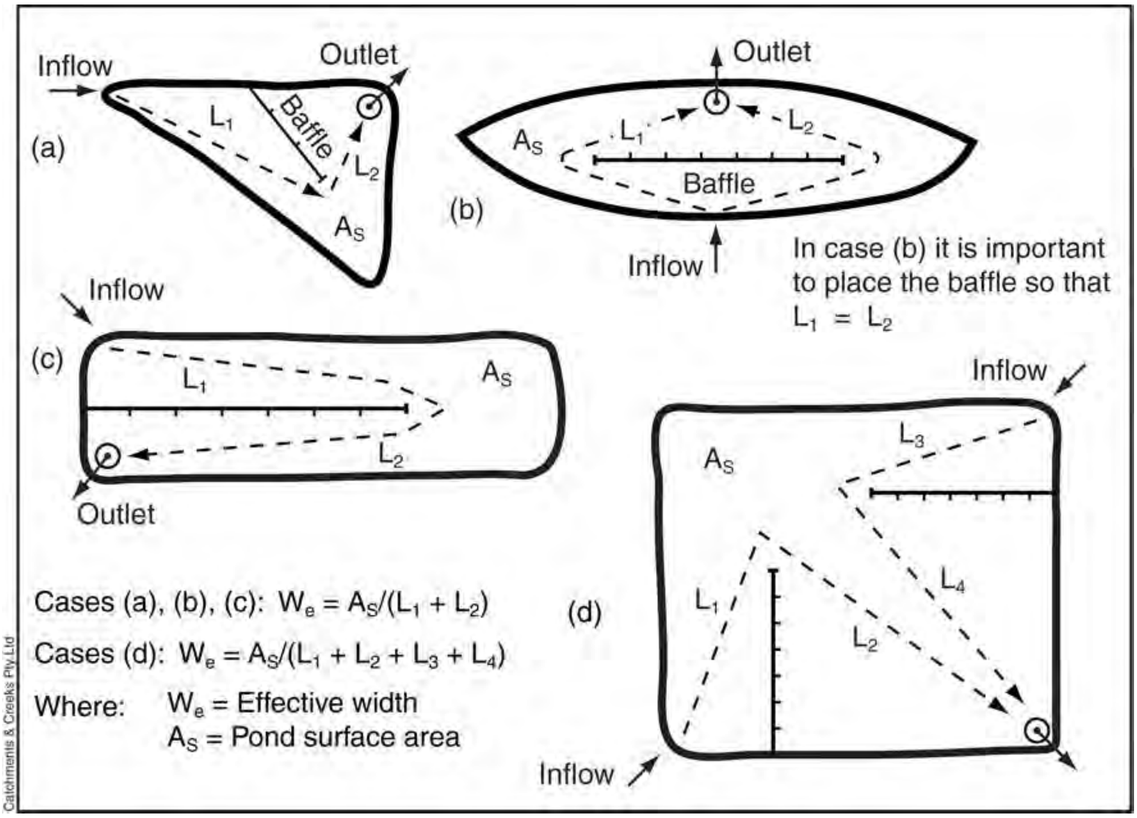


Figure 7 – Typical arrangement of internal flow control baffles (after USDA, 1975)

The crest of these baffles should be set level with, or just below, the crest of the emergency spillway. This is to prevent the re-suspension of settled sediment during severe storms (i.e. flows in excess of the basin's design storm should be allowed to overtop these baffles).

Photo 3 – Inlet chamber can also act as mixing chambers for the addition of flocculants

Photo 4 – Internal baffle extending the flow path between the basin inlet (left) and the basin outlet (right)

(c) Outlet chambers

Outlet chambers (Figures 8 & 9) are used to keep the bulk of the settled sediment away from certain low-flow outlet systems, particularly riser pipe outlets and flexible skimmer pipe outlets.

The use of an outlet chamber is mandatory when a flexible skimmer pipe outlet system is employed (Photo 6).

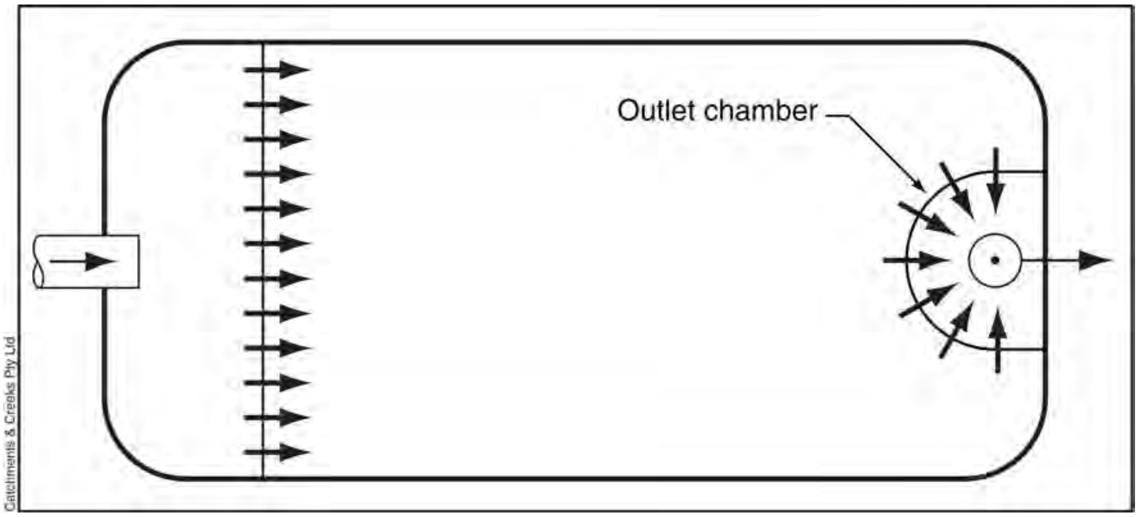


Figure 8 – Typical arrangement of outlet chamber (plan view)

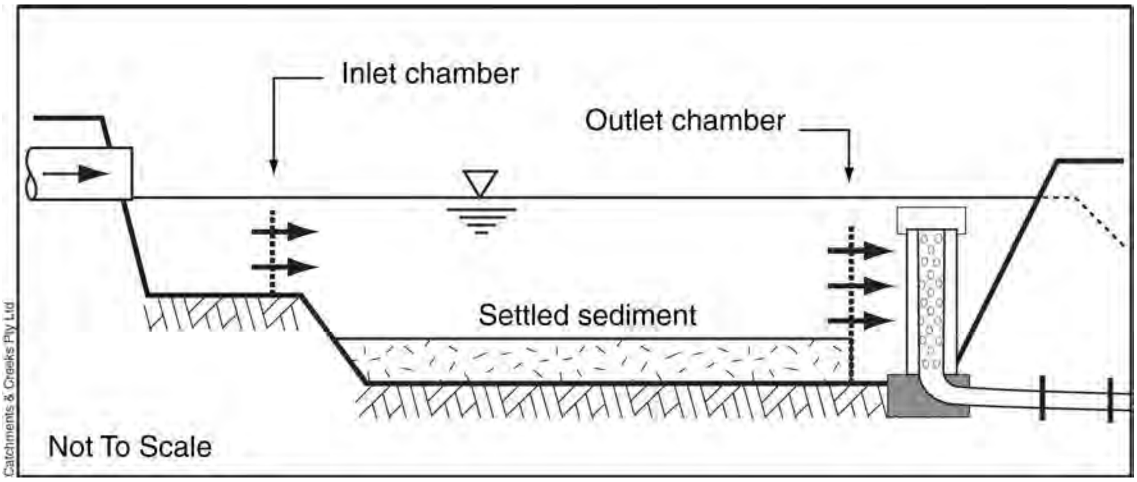


Figure 9 – Typical arrangement of outlet chamber (long section)

Photo 5 – Outlet chamber

Photo 6 – Outlet chamber surrounding a flexible skimmer outlet device

Step 9 – Design of the basin's inflow system

Surface flow entering the basin should not cause erosion down the banks of the basin. If concentrated surface flow enters the basin (e.g. via a *Catch Drain*), then an appropriately lined *Chute* (Photo 7) will need to be installed at each inflow point to control scour.

If flow enters the basin through pipes, then wherever practicable, the pipe invert should be above the spillway crest elevation to reduce the risk of sedimentation within the pipe. Submerged inflow pipes must be inspected and de-silted (as required) after each inflow event.

Constructing an appropriately designed pre-treatment pond or inlet chamber (Step 8) can be used to both improve the hydraulic efficiency of the settling pond, and reduce the cost and frequency of de-silting the main settling pond.

Where space is available, the construction of an inlet (pre-treatment) pond (Figure 10) or inlet chamber (Step 8) can significantly reduce the cost of regular de-silting activities for large and/or long-term basins.

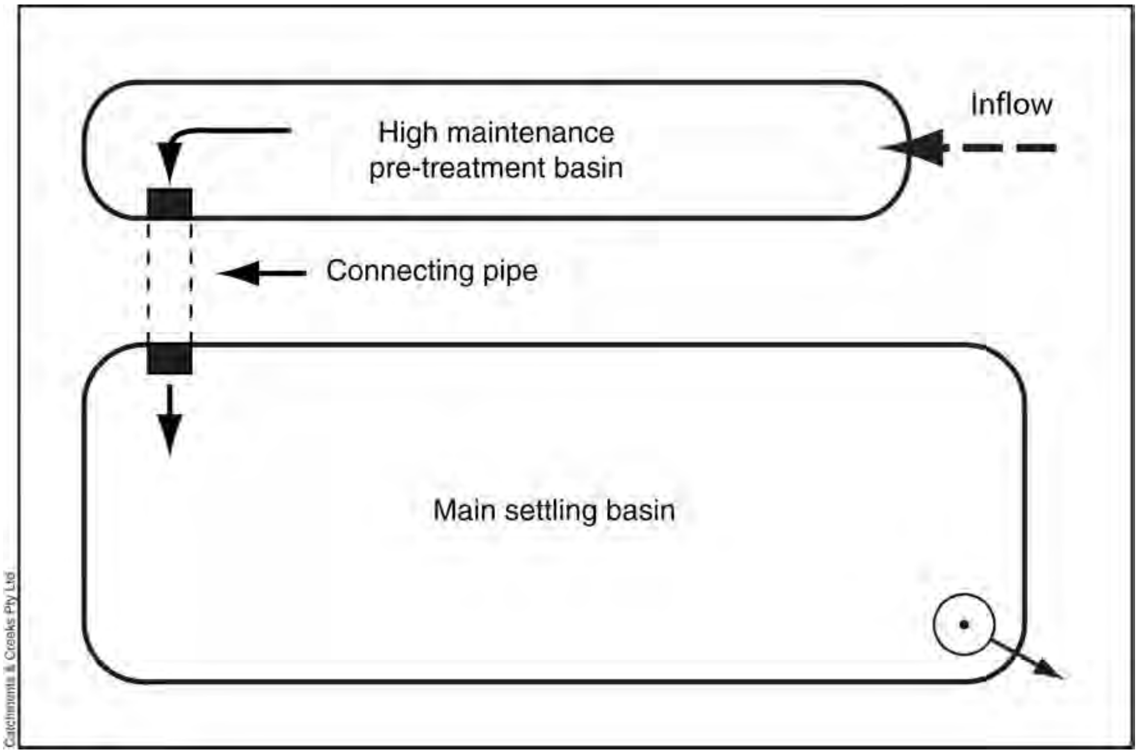


Figure 10 – Pre-treatment inlet pond

Photo 7 – Stabilise inflow chute

Photo 8 – Litter screen placed on inlet of permanent sediment basin

Step 10 – Design of the primary outlet system

Dry basins (Type C only) require a formal outlet system in the form of either a riser pipe outlet or floating skimmer system (Photo 11). Gabion wall, *Rock Filter Dam*, and *Sediment Weir* outlet systems are **not** recommended unless a Type 2 sediment retention system has been specified.

The hydraulics of a Type C basin's primary outlet system must ensure that the peak water level is at least 300mm below the crest of the emergency spillway during the basin's nominated design storm.

Wet basins (Type C, F or D) usually require a pumped outlet system. Alternatively, if a piped outlet exists, then a flow control valve must be fitted to the outlet pipe to allow full control of the basin discharge (note, Type C basins can be operated as wet basins with pumped outlets).

Photo 9 – Twin riser pipes in the process of being installed

Photo 10 – Riser pipe with aggregate filter and trash screen

Photo 11 – Skimmer outlet system

Photo 12 – Skimmer pipes must be protected from sediment build-up

Photo 13 – Low-flow sand filter outlet system on a permanent sediment basin

Photo 14 – Sand filter outlet system during installation

Step 11 – Design of the emergency spillway

All elevated sediment basins (i.e. not fully recessed below natural ground, Photo 17) require the construction of a formally designed emergency spillway. Spillways are critical engineering structures that need to be designed by suitably qualified persons.

The minimum design storm for sizing the emergency spillway is defined in Table 7.

Table 7 – Recommended design standard for emergency spillways

Design life	Minimum design storm ARI
Less than 3 months operation	1 in 10 year
3 to 12 months operation	1 in 20 year
Greater than 12 months	1 in 50 year
If failure is expected to result in loss of life	Probable Maximum Flood (PMF)

The crest of the emergency spillway is to be at least:

- 300mm above the primary outlet (if included);
- 300mm below a basin embankment formed in virgin soil;
- 450mm below a basin embankment formed from fill.

In addition to the above, design of the emergency spillway must ensure that the maximum water level within the basin during the design storm specified in Table 7 is at least:

- 300mm below a basin embankment formed from fill;
- 150mm plus expected wave height for large basins with significant fetch length.

The approach channel can be curved upstream of the spillway crest, but must be straight from the crest to the energy dissipater as shown in Figure 11. The approach channel should have a back-slope towards the impoundment area of not less than 2% and should be flared at its entrance, gradually reducing to the design width at the spillway crest.

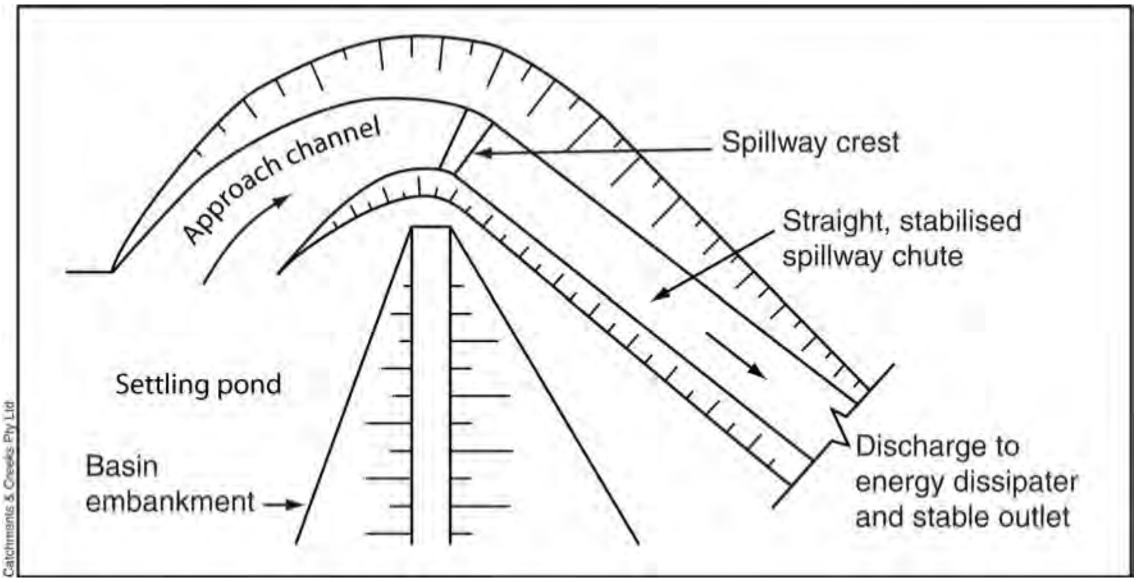


Figure 11 – Emergency spillway cut into virgin soil to side of fill embankment

All reasonable and practicable efforts must be taken to construct the spillway in virgin soil (Photo 16), rather than within a fill embankment (Photo 15). Placement of an emergency spillway within a fill embankment can significantly increase the risk of failure.

The downstream face of the spillway may be protected with grass, concrete, rock, rock mattresses, or other suitable material as required for the expected maximum flow velocity. Grass-lined spillway chutes are generally not recommended for sediment basins due to their long establishment time and relatively low scour velocity.

Photo 15 – Emergency spillway located within the fill embankment

Photo 16 – Emergency spillway located within virgin soil to the side of the embankment

For rock and rock mattress lined spillways, it is important to control seepage flows through the rocks located across the crest of the spillway. Seepage control is required so that the settling pond can achieve its required maximum water level prior to discharging down the spillway. Concrete capping of the spillway crest (Photo 18) can be used to control excess seepage flows.

Photo 17 – Fully recessed basin with natural ground forming the spillway

Photo 18 – Rock-lined spillway—note concrete sealing of the spillway crest

It is important to ensure that the spillway crest has sufficient depth and width to fully contain the nominated design storm peak discharge. Photo 20 shows a spillway crest with inadequate depth or flow profile.

Photo 19 – De-watering pipe intake <u>must</u> not rest on the basin floor

Photo 20 – Spillway crest with inadequate depth or profile

A suitable energy dissipater will be required at the base of the spillway. The recommended hydraulic freeboard down the spillway chute is 300mm to contain the turbulent whitewater.

Step 12 – Determination of the basin's overall dimensions

If a sediment basin is constructed with side slopes of say 3:1 (H:V), then the basin may be 5 to 10m longer and wider than the length and width of the settling pond determined in Step 5. It is important to ensure the overall dimensions of the basin can fit into the available space.

The minimum recommended embankment crest width is 2.5m, unless justified by hydraulic/ geotechnical investigations.

Where available space does not permit construction of the ideal sediment basin, then a smaller basin may be used; however, erosion control and site rehabilitation measures must be increased to an appropriate higher standard to compensate. If the basin's settling pond surface area/volume is less than that required in Step 5, than the basin must be treated as a Type 2 or Type 3 sediment control system.

Step 13 – Locate maintenance access (de-silting)

Sediment basins can either be de-silted using long-reach excavation equipment operating from the sides of the basin, or by allowing machinery access into the basin. If excavation equipment needs to enter directly into the basin, then it is better to design the access ramp so that trucks can be brought to the edge of the basin, rather than trying to transport the sediment to trucks located at the top of the embankment. Thus a maximum 6:1 (ideally 10H:1V) access ramp will need to be constructed.

If the sediment is to be removed from the site, then a suitable sediment drying area should be made available adjacent to the basin, or at least somewhere within the basin's catchment area.

Step 14 – Define the sediment disposal method

Trapped sediment can be mixed with on-site soils and buried, or removed from the site. If sediment is removed from the site, then it should be de-watered prior to disposal. De-watering must occur within the catchment area of the sediment basin.

Step 15 – Assess need for safety fencing

Construction sites are often located in publicly accessible areas. In most cases it is not reasonable to expect a parent or guardian of a child to be aware of the safety risks associated with a neighbouring construction site. Thus fencing of a sediment basin is usually warranted even if the basins are located adjacent to other permanent water bodies such as a stream, lake, or wetland.

Responsibility for safety issues on a construction site ultimately rests with the site manager; however, each person working on a site has a duty of care in accordance with the State's work place safety legislation. Similarly, designers of sediment basins have a duty of care to investigate the safety requirements of the site on which the basin is to be constructed.

Photo 21 – Sediment basin with poor access for de-silting operations

Photo 22 – Temporary fencing of a construction site sediment basin

Step 16 – Define the rehabilitation requirements for the basin area

The Erosion and Sediment Control Plan (ESCP) needs to include details on the required decommissioning and rehabilitation of the sediment basin area. Such a process may involve the conversion of the basin into a component of the site's permanent stormwater treatment network.

On subdivisions and major road works, construction site sediment basins often represent a significant opportunity for conversion into either: a detention/retention basin (Photo 23), bioretention system, wetland, or pollution containment system. In rural areas, basins associated with road works are often constructed within adjacent properties where they remain under the control of the landowner as permanent farm dams.

In some circumstances it will be necessary to protect newly constructed permanent (future) stormwater treatment devices from sediment intrusion during the construction phase. With appropriate site planning and design the protection of these permanent stormwater treatment devices is generally made easier if the sediment basin is designed with a pre-treatment inlet pond (Figure 10). The pre-treatment pond can remain as a coarse sediment trap during the maintenance and building phases, thus protecting the newly formed wetland or bioretention system located within the basin's main settling pond.

Continued operation of the sediment basin during the building phase of subdivisions (i.e. beyond the specified maintenance phase) is an issue for negotiation between the regulatory authority and the land developer on a case-by-case basis.

During the construction, decommissioning, rehabilitation, or reconstruction of a sediment basin, the basin area including settling pond, embankment and spillway, must be considered a construction site in its on right. Thus, these works must comply with normal drainage, erosion, and sediment control standards. This means that appropriate temporary sediment control measures will be required down-slope of the sediment basin during its construction and decommissioning.

Upon decommissioning of a sediment basin, all water and sediment must be removed from the basin prior to removal of the embankment (if any). Any such material, liquid or solid, must be disposed of in a manner that will not create an erosion or pollution hazard.

Photo 23 – Permanent sediment basin within residential estate

Photo 24 – Sediment basins converted to permanent stormwater treatment ponds on highway project

Under normal circumstances, a sediment basin must not be decommissioned until all up-slope site stabilisation measures have been implemented and are appropriately working to control soil erosion and sediment runoff in accordance with the specified ESC standard. This may require the basin to be fully operational during part of the maintenance and operational phases.

If an alternative, permanent, outlet structure is to be constructed prior to stabilisation of the up-slope catchment area, then this outlet structure must not be made operational if it will adversely affect the required operation of the sediment basin during the construction phase.

Step 17 – Specification of the basin's operational procedures

Sediment basins can be operated as either 'dry' or 'wet' basins as described below.

- Dry basins are free draining basins that allow water to commence discharging from the low-flow outlet system as soon as water enters the basin.
- Wet basins are designed to retain sediment-laden water for extended periods allowing adequate time for the gravitational settlement of fine sediment particles. Operation of these basins may be assisted through the use of chemical flocculants. Ideally these basins are not drained until a suitable water quality is obtained within the basin.

Type F and Type D basins must be operated as wet basins with the settled/treated water decanted from the basin as soon as a suitable water quality is achieved. Thus, as soon as conditions allow, the basin must be maintained in either a dry-bed condition, or with a water level no greater than the top of the sediment storage zone.

On each occasion when a Type F or Type D basin cannot be de-watered prior to being surcharged by a following rainfall event, the operator must record such an event and report it to the appropriate regulatory authority. Where appropriate, alternative operating procedures may need be adopted in consultation with the regulatory authority in order to achieve optimum environmental protection.

A Type C basin may be operated as either a dry basin or wet basin; however, when operated as a wet basin, the settled water does not necessarily need to be decanted from the settling pond after achieving the desired water quality. This means that the water can be retained on-site for revegetation purposes and dust control.

A Type C basin operating in a wet condition is still sized in accordance with the design requirements for a normal Type C basin; however, a low-flow drainage system is not necessarily incorporated into the basin, thus potentially saving significant construction and maintenance costs.

Table 8 provides a summary of the attributes of the various operational conditions.

Table 8 – Attributes of various types of *Sediment Basins*

Attribute	Type C dry basin	Type C wet basin	Type F and Type D wet basins
Soil type within catchment	Sandy soils	Sandy soils	Clayey or dispersive soils
Critical design parameter	Surface area at base of settling zone	Surface area at base of settling zone	Volume of settling zone
Desirable water level condition before a storm	Empty	Any condition	No greater than top of sediment storage zone
De-watering system	Low-flow piped drainage system (riser pipe)	Pumping	Pumping
Chemical flocculation	Only if specified water quality objectives fail to be achieved	Only if specified water quality objectives fail to be achieved	As necessary, but usually required for Type D basins

Type F and D basins are typically designed for a maximum 5-day cycle, that being the filling, treatment and discharge of the basin within a maximum 5-day period. In some tropical regions this may not be practicable, and either a shorter or longer time frame may be required. The use of a shorter time period usually requires application of fast acting coagulants, which may require a much higher degree of environmental management compared to gypsum.

Appropriate coagulation of sediment basins is required if the contained water does not achieve a specified water quality standard, usually 50mg/L. In cases where a poor discharge water quality is achieved, a Type C basin may need to be operated as if it was a Type F or Type D basin in order to satisfy specified water quality objectives.

Certification of Sediment Basin Construction

BASIN IDENTIFICATION CODE/NUMBER: . . . .

LOCATION: . . . .

Legend: ✓ OK ✗ Not OK N/A Not applicable

Construction:

Item	Consideration	Assessment
1	Sediment basin located in accordance with approved plans.	. . . .
2	Embankment material compacted in accordance with specifications.	. . . .
3	Critical basin and spillway dimensions and elevations confirmed by as-constructed survey.	. . . .
4	Required freeboard adjacent embankments and spillway confirmed by as-constructed survey.	. . . .
5	Placement of rock on chute and upstream face of spillway in accordance with design details and standards.	. . . .
6	Placement of rock within energy dissipation zone downstream of spillway in accordance with design details and standard.	. . . .
7	All other sediment basin requirements in accordance with design details and standards.	. . . .
8	As-constructed plan prepared for basin and spillway.	. . . .

INSPECTION OFFICER . . . . **DATE** . . . .

SIGNATURE . . . .

Geotechnical:

Item	Consideration	Assessment
9	Suitable material used to form all embankments.	. . . .
10	Appropriate compaction achieved in embankment construction (if observed).	. . . .
11	No foreseeable concerns regarding stability or construction of the basin and spillway.	. . . .

INSPECTION OFFICER . . . . **DATE** . . . .

SIGNATURE . . . .

Description

A purpose built dam designed to collect and settle sediment from sediment-laden runoff. It usually consists of a settling pond, a low-flow drainage or manual decant system, and a high-flow emergency spillway.

Purpose

Sediment basins generally perform two main functions: firstly the settlement of coarse-grained sediment particles (e.g. sand and coarse silt) from waters passing through the basin, and secondly either:

(i) the filtration of fine sediments (e.g. fine silt and clay) from waters passing through the filtration system attached to the low-flow outlet;

(ii) the settlement of fine-grained particles from those waters retained within the basin following a storm event.

Limitations

Generally used on catchments greater than 0.25ha.

The installation of a sediment basin does not replace the need for appropriate on-site drainage and erosion control measures.

Sediment basins operating in a free-draining mode (dry basins) have limited control over turbidity, especially that resulting from dispersive soils, unless chemical treated.

Advantages

Sediment basins need be designed and operated in a manner that produces near-clear water discharge (i.e. total suspended solids concentrations not exceeding 50mg/L), especially following periods of light rainfall.

It is the ability of sediment basins to reduce turbidity levels (wet basins) that allows these Type 1 sediment traps to significantly reduce the potential ecological harm caused by urban construction.

Even when a basin is full of water, it can still be effective in the removal of coarse sediment from any flows passing through the basin.

Very high capture rate for coarse sediments.

Can be an effective control of fine sediment and turbidity during the frequent 'minor' storms if suitably operated.

Can be converted into a permanent wetland or detention basin for ongoing stormwater management after completion of the construction phase.

Disadvantages

Chemical dosing of basins can be difficult to automate.

Basins are difficult and expensive to relocate if the construction or drainage layout changes.

Decommissioned and backfilled sediment basins generally attract lower land values and are best integrated into open space areas, or the site's permanent stormwater management system.

Common Problems

Sediment blockage of free-flow outlet systems (dry basins).

Difficulties in repairing the low-flow outlet system once sediment blockage has occurred.

Inadequate room made available to construct the sediment basin.

Poor access available to maintain the basin.

Poor hydraulic design and/or construction of the emergency spillway.

Special Requirements

The sediment trapping efficiency of a sediment basin can be increased by:

- reducing the energy (jetting) of inflow;
- construction basins as close as practicable to the ideal 10:1 length to width ratio;
- avoiding bends in the flow path of water through the settling pond that may cause secondary currents and dead-water areas;
- avoiding wind shear on large basins that could cause secondary currents and sediment re-suspension.
- where practicable, operating both wet and dry basins in a manner that allows them to fully drain before the next significant inflow event, thus allowing settled sediment to 'cement' together on the bed of the basin.

Early integration of the basin into the construction phase is essential.

Avoid constructing sediment basins in dispersive soils. Where this is unavoidable, the basin should be lined with a non-dispersive or treated soil, especially the banks.

Overland flow should enter the basin via a stabilised chute. It should not be allowed to cause erosion down the banks of the settling pond.

Medium to high velocity piped inflows may require an energy dissipater or inlet baffle to break-up the inflow jet.

Internal baffles may be required in 'dry' basins to improve the movement of water through the settling pond.

An outlet baffle or barrier may be required to reduce the build-up of sediment and mud around the primary outlet filter (dry basins).

Sediment basins should be fenced if a public safety risk exists.

Location

Basins need to be located such that they intercept runoff from the largest possible portion of the disturbed site.

Basins generally should not collect runoff generated from off-site areas.

Must be located so that construction and maintenance access is available.

Where practicable, an area of level land should be available adjacent to the basin to allow de-watering of excavated sediment.

Preferably located above the 1 in 5 year flood level if located on or near a watercourse floodplain.

Allow room between the toe of the embankment and the downstream property boundary for provision for safety fencing, the spillway outlet, and all necessary energy dissipation measures.

Site Inspection

Check the dimensions of the basin.

Check for scouring around, or damage to the inlets and outlets.

Check for damage to the emergency spillway and displacement of rocks.

Check the level of sediment build-up.

Check all internal and external banks for erosion.

Check the measures introduced to control inflow jetting (wet and dry basins).

Check for trash build-up on inlet screens.

Check for water passing through earth embankments that could lead to a piping failure and bank collapse.

Materials

- Earth fill: clean soil with Emerson Class 2(1), 3, 4, or 5, and free of roots, woody vegetation, rocks and other unsuitable material. Soil with Emerson Class 4 and 5 may not be suitable depending on particle size distribution and degree of dispersion. Class 2(1) should only be used upon recommendation from geotechnical specialist. This specification may be replaced by an equivalent standard based on the exchangeable sodium percentage.
- Riser pipe: minimum 250mm diameter.
- Spillway rock: hard, angular, durable, weather resistant and evenly graded rock with 50% by weight larger than the specified nominal (d_{50}) rock size. Large rock should dominate, with sufficient small rock to fill the voids between the larger rock. The diameter of the largest rock size should be no larger than 1.5 times the nominal rock size. The specific gravity should be at least 2.5.
- Geotextile fabric: heavy-duty, needle-punched, non-woven filter cloth, minimum 'bidim' A24 or equivalent.

Construction

1. Notwithstanding any description contained within the approved plans or specifications, the Contractor shall be responsible for satisfying themselves as to the nature and extent of the specified works and the physical and legal conditions under which the works will be carried out. This shall include means of access, extent of clearing, nature of material to be excavated, type and size of mechanical plant required, location and suitability of water supply for construction and testing purposes, and any other like matters affecting the construction of the works.
2. Refer to approved plans for location, dimensions, and construction details. If there are questions or problems with the location, dimensions, or method of installation, contact the engineer or responsible on-site officer for assistance.
3. Before starting any clearing or construction, ensure all the necessary materials and components are on the site to avoid delays in completing the pond once works begin.

4. Install required short-term sediment control measures downstream of the proposed earthworks to control sediment runoff during construction of the basin.
5. The area to be covered by the embankment, borrow pits and incidental works, together with an area extending beyond the limits of each for a distance not exceeding five (5) metres all around must be cleared of all trees, scrub, stumps, roots, dead timber and rubbish and disposed of in a suitable manner. Delay clearing the main pond area until the embankment is complete.
6. Ensure all holes made by grubbing within the embankment footprint are filled with sound material, adequately compacted, and finished flush with the natural surface.

Cut-off trench:

7. Before construction of the cut-off trench or any ancillary works within the embankment footprint, all grass growth and topsoil must be removed from the area to be occupied by the embankment and must be deposited clear of this area and reserved for topdressing the completing the embankment.
8. Excavate a cut-off trench along the centre line of the earth fill embankment. Cut the trench to stable soil material, but in no case make it less than 600mm deep. The cut-off trench must extend into both abutments to at least the elevation of the riser pipe crest. Make the minimum bottom width wide enough to permit operation of excavation and compaction equipment, but in no case less than 600mm. Make the side slopes of the trench no steeper than 1:1 (H:V).
9. Ensure all water, loose soil, and rock are removed from the trench before backfilling commences. The cut-off trench must be backfilled with selected earth-fill of the type specified for the embankment, and this soil must have a moisture content and degree of compaction the same as that specified for the selected core zone.
10. Material excavated from the cut-off trench may be used in construction of the embankment provided it is suitable and it is placed in the correct zone according to its classification.

Embankment:

11. Scarify areas on which fill is to be placed before placing the fill.
12. Ensure all fill material used to form the embankment meets the specifications certified by a soil scientist or geotechnical specialist.
13. The fill material must contain sufficient moisture so it can be formed by hand into a ball without crumbling. If water can be squeezed out of the ball, it is too wet for proper compaction. Place fill material in 150 to 250mm continuous layers over the entire length of the fill area and then compact before placement of further fill.
14. Place riser pipe outlet system, if specified, in appropriate sequence with the embankment filling. Refer to separate installation specifications.
15. Unless otherwise specified on the approved plans, compact the soil at about 1% to 2% wet of optimum and to 95% modified or 100% standard compaction.
16. Where both dispersive and non-dispersive classified earth-fill materials are available, non-dispersive earth-fill must be used in the core zone. The remaining classified earth-fill materials must only be used as directed by *[insert title]*.
17. Where specified, construct the embankment to an elevation 10% higher than the design height to allow for settling; otherwise finished dimensions of the embankment after spreading of topsoil must conform to the drawing with a tolerance of 75mm from the specified dimensions.
18. Ensure debris and other unsuitable building waste is not placed within the earth embankment.
19. After completion of the embankment all loose uncompacted earth-fill material on the upstream and downstream batter must be removed prior to spreading of topsoil.
20. Topsoil and revegetate/stabilised all exposed earth as directed within the approved plans.

Spillway construction:

21. The spillway must be excavated as shown on the plans, and the excavated material if classified as suitable, must be used in the embankment, and if not suitable it must be disposed of into spoil heaps.
22. Ensure excavated dimensions allow adequate boxing-out such that the specified elevations, grades, chute width, and entrance and exit slopes for the emergency spillway will be achieved after placement of the rock or other scour protection measures as specified in the plans.
23. Place specified scour protection measures on the emergency spillway. Ensure the finished grade blends with the surrounding area to allow a smooth flow transition from spillway to downstream channel.
24. If a synthetic filter fabric underlay is specified, place the filter fabric directly on the prepared foundation. If more than one sheet of filter fabric is required, overlap the edges by at least 300mm and place anchor pins at minimum 1m spacing along the overlap. Bury the upstream end of the fabric a minimum 300mm below ground and where necessary, bury the lower end of the fabric or overlap a minimum 300mm over the next downstream section as required. Ensure the filter fabric extends at least 1000mm upstream of the spillway crest.
25. Take care not to damage the fabric during or after placement. If damage occurs, remove the rock and repair the sheet by adding another layer of fabric with a minimum overlap of 300mm around the damaged area. If extensive damage is suspected, remove and replace the entire sheet.
26. Where large rock is used, or machine placement is difficult, a minimum 100mm layer of fine gravel, aggregate, or sand may be needed to protect the fabric.
27. Placement of rock should follow immediately after placement of the filter fabric. Place rock so that it forms a dense, well-graded mass of rock with a minimum of voids. The desired distribution of rock throughout the mass may be obtained by selective loading at the quarry and controlled dumping during final placement.
28. The finished slope should be free of pockets of small rock or clusters of large rocks. Hand placing may be necessary to achieve the proper distribution of rock sizes to produce a relatively smooth, uniform surface. The finished grade of the rock should blend with the surrounding area. No overfall or protrusion of rock should be apparent.
29. Ensure that the final arrangement of the spillway crest will not promote excessive flow through the rock such that the water can be retained within the settling basin an elevation no less than 50mm above or below the nominated spillway crest elevation.

Establishment of settling pond:

30. The area to be covered by the stored water outside the limits of the borrow pits must be cleared of all scrub and rubbish. Trees must be cut down stump high and removed from the immediate vicinity of the work.
31. Establish all required inflow chutes and inlet baffles, if specified, to enable water to discharge into the basin in a manner that will not cause soil erosion or the re-suspension of settled sediment.
32. Install a sediment storage level marker post with a cross member set just below the top of the sediment storage zone (as specified on the approved plans). Use at least a 75mm wide post firmly set into the basin floor.
33. If specified, install internal settling pond baffles. Ensure the crest of these baffles is set level with, or just below, the elevation of the emergency spillway crest.
34. Install all appropriate measures to minimise safety risk to on-site personnel and the public caused by the presence of the settling pond. Avoid steep, smooth internal slopes. Appropriately fence the settling pond and post warning signs if unsupervised public access is likely or there is considered to be an unacceptable risk to the public.

Maintenance of Sediment Basin

1. Inspect the sediment basin during the following periods:
   (i) During construction to determine whether machinery, falling trees, or construction activity has damaged any components of the sediment basin. If damage has occurred, repair it.
   (ii) After each runoff event. Inspect the erosion damage at flow entry and exit points. If damage has occurred, make the necessary repairs.
   (iii) At least weekly during the nominated wet season (if any) otherwise at least fortnightly.
   (iv) Prior to, and immediately after, periods of 'stop work' or site shutdown.
2. Clean out accumulated sediment when it reaches the marker board/post, and restore the original storage volume. Place sediment in a disposal area or, if appropriate, mix with dry soil on the site.
3. Do not dispose of sediment in a manner that will create an erosion or pollution hazard.
4. Check all visible pipe connections for leaks, and repair as necessary.
5. Check all embankments for excessive settlement, slumping of the slopes or piping between the conduit and the embankment; make all necessary repairs.
6. Remove all trash and other debris from the basin and riser.
7. Submerged inflow pipes must be inspected and de-silted (as required) after each inflow event.

Removal of Sediment Basin

1. When grading and construction in the drainage area above a temporary sediment basin is completed and the disturbed areas are adequately stabilised, the basin must be removed or otherwise incorporated into the permanent stormwater drainage system. In either case, sediment should be cleared and properly disposed of and the basin area stabilised.
2. Before starting any maintenance work on the basin or spillway, install all necessary short-term sediment control measures downstream of the sediment basin.
3. All water and sediment must be removed from the basin prior to the dam's removal. Dispose of sediment and water in a manner that will not create an erosion or pollution hazard.
4. Bring the disturbed area to a proper grade, then smooth, compact, and stabilise and/or revegetate as required to establish a stable land surface.

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