COWAL GOLD OPERATIONS
EROSION AND SEDIMENT CONTROL MANAGEMENT PLAN

July 2023

Revision Status Register

Section/Page/ Annexure	Revision Number	Amendment/Addition	Distribution	DPE Approval Date
All	ESCMP-O (September 2003) Document No. 684736	Original Erosion and Sediment Control Management Plan (ESCMP)	DLWC and EPA	September 2003
Annexure	ESCMP Amendments (i) to (v) (December 2004) Document No. 684887	Amendments to include additional water management and temporary erosion and sediment control measures prior to development of the CGO, and to remove contained water storage D7.	DLWC and EPA	21 December 2004
Addendum	Addendum dated December 2009 Document No. 684889	Revised to reflect Development Consent as modified on 28 August 2009 (in relation to introduction of the saline groundwater borefield within ML 1535).	DECCW, DoP	10 March 2010
Addendum	Addendum dated February 2015 Document No. 00653984	Revised to reflect Development Consent as modified on 22 July 2014.	DPE	21 March 2016
All	ESCMP02-A (July 2017) Document No. 00856137	Revised to reflect Development Consent as modified on 7 February 2017. Incorporates all addenda and annexures.	DPE	28 August 2018
All	ESCMP dated February 2019 Document No: ESCMP 02-B (964952-001)	ESCMP revised in accordance with DA 14/98 condition 9.1(c)(v) to reflect Development Consent as modified on 4 October 2018 and 23 August 2019.	DPE	-
All	ESCMP dated March 2022	ESCMP revised to reflect outcomes of approval of Mod16 to DA 14/98 and SSD 10367 for the Underground Development approved by DPE on 30 September 2021.		6 April 2023
All	ESCMP dated June 2023	Minor updates in accordance with Development Consent condition DA14/98 9.1(c)(i)	DPE	TBC

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1 INTRODUCTION

The Cowal Gold Operations (CGO) is located approximately 38 kilometres (km) north-east of West Wyalong in New South Wales (NSW) (Figure 1). Evolution Mining (Cowal) Pty Limited (Evolution) is the owner and operator of the CGO. All mining activity occurs within Mining Lease (ML) 1535 and ML 1791.

Development Consent no. 14/98 (DA 14/98) for the CGO (including the Bland Creek Palaeochannel Borefield water supply pipeline) was granted by the Minister for Urban Affairs and Planning under Part 4 of the *Environmental Planning and Assessment Act, 1979* (EP&A Act) on 26 February 1999. Development Consent (DA 2011/64) for the operation of the Eastern Saline Borefield was granted by the Forbes Shire Council on 20 December 2010.

The Minister for Planning granted approval for the *Cowal Gold Operations Underground Development Project as* State-significant Development No. 10367 (SSD 10367) under Section 4.38(2) of the EP&A Act on 30 September 2021 and to modify DA 14/98 through *Modification No.16* (herein referred to as Mod 16) under Section 4.55(2) of the EP&A Act. SSD 10367 was modified on 7 November 2022, to reflect minor changes in the underground mining method, through Mod 1 (Optimisation Modification).

DA 14/98 generally allows:

- Mining operations until 2040.
- Ore processing at a rate of 9.8 Mtpa.
- Tailings and waste rock emplacement on site.
- Operation of a range of ancillary mining infrastructure.

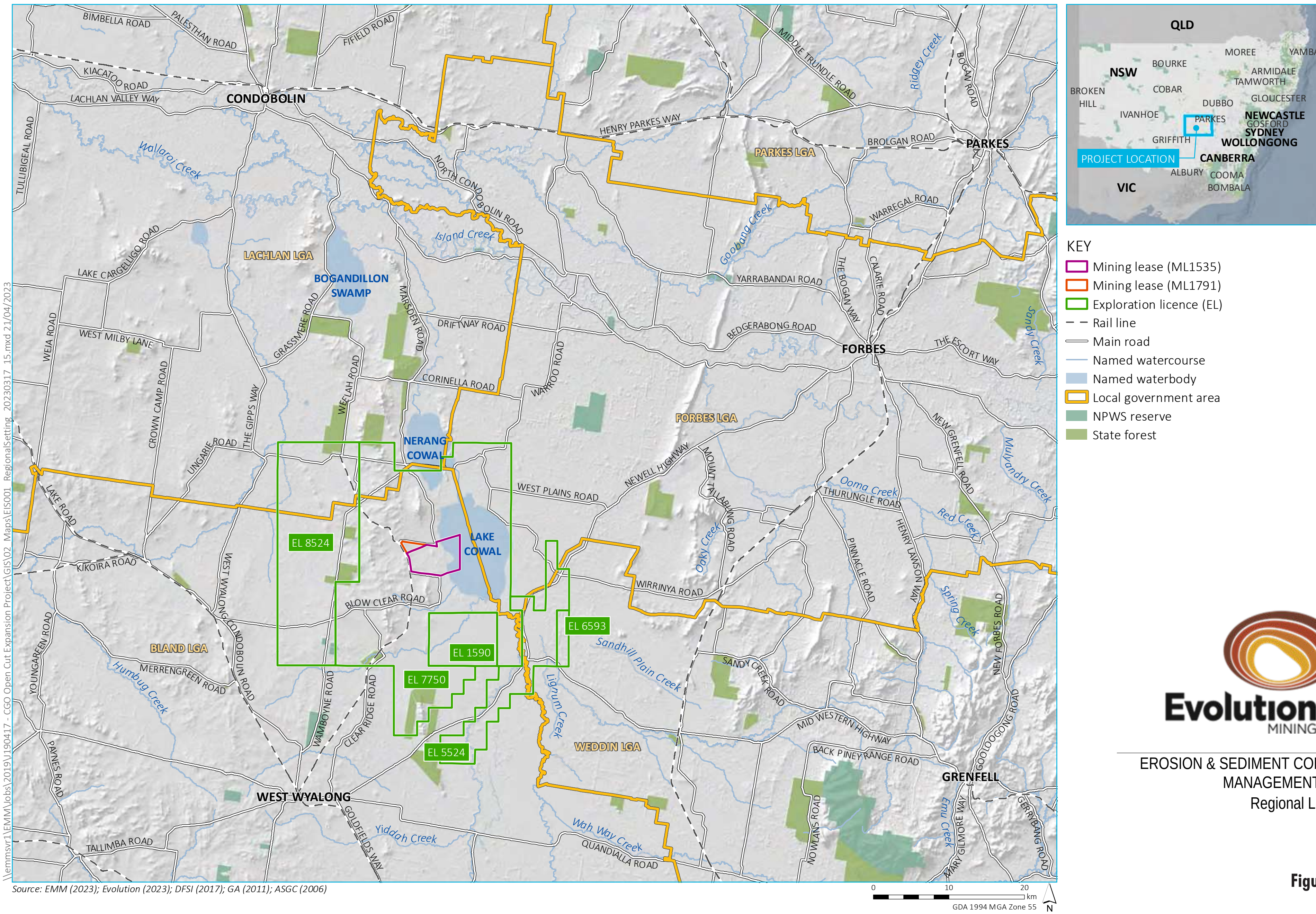
SSD 10367 generally allows:

- Underground stope mining until 2040.
- Backfilling the stopes with cemented paste made from tailings.
- Development of ancillary infrastructure including a box-cut to the underground mine and a paste fill plant.

The general arrangement of the approved CGO is provided in Figure 2.

This ESCMP has been developed as a best practice approach for responsible CGO water management (Commonwealth of Australia, 2016), and has been developed in-line with best practice erosion and sediment control measures listed in *Managing Urban Stormwater Soils and Construction Volume 1* (Landcom, 2004), *Managing Urban Stormwater Soils and Construction Volume 2E – Mines and quarries* (NSW Department of Environment and Climate Change [DECC], 2008), and the International Erosion Control Association (IECA) Australasian guidelines, *Best Practice Erosion & Sediment Control* guidelines (IECA, 2008).

This revised ESCMP has been prepared to reflect the conditions of DA 14/98 and SSD 10367, as approved on 30 September 2021 and 7 November 2023, and supersedes all former versions of the ESCMP. Copies of the approved development consents for Mod 16 and SSD 10367 are available on Evolution's website (www.evolutionmining.com.au).



Source: EMM (2023); Evolution (2023); DFSI (2017); GA (2011); ASGC (2006)

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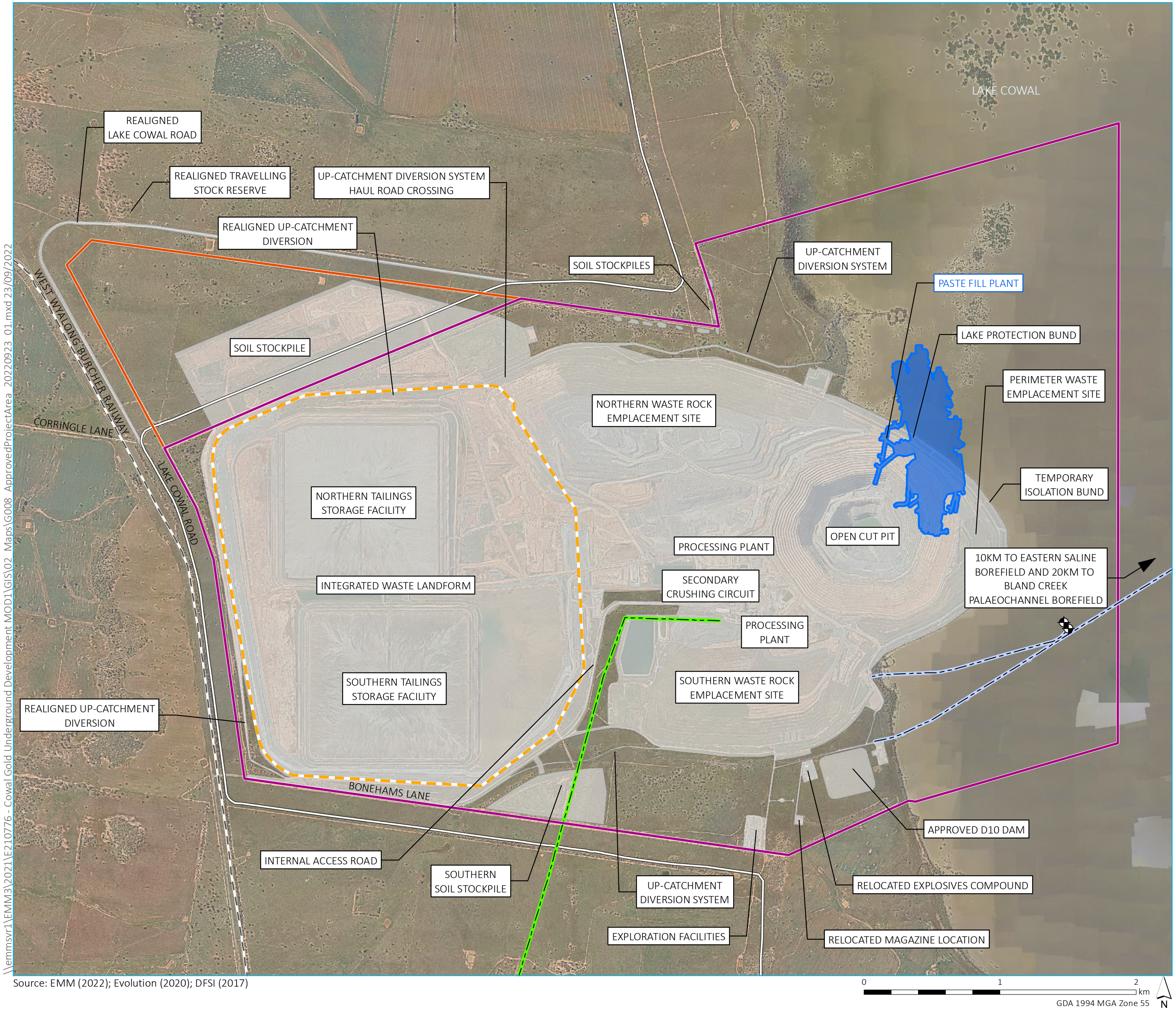
EROSION & SEDIMENT CONTROL MANAGEMENT PLAN
Regional Location

Figure 1



Source: EMM (2022); Evolution (2020); DFSI (2017)

KEY
- Approved underground development
- Mining lease (ML1535)
- Mining lease (ML1791)
- DA14/98 approved surface disturbance
- Indicative integrated waste landform perimeter
- Electricity transmission line
- Water supply pipeline
- Rail line
- Main road
- xxx Approved underground development elements
- xxx Approved surface elements

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EROSION & SEDIMENT CONTROL MANAGEMENT PLAN

General Arrangement of Approved CGO

Figure 2

1.1 OBJECTIVES

The objective of this ESCMP is to minimise the adverse impacts of soil erosion at the CGO through the effective design and implementation of sediment control strategies throughout the life of the CGO. This includes:

- the protection of water quality in Lake Cowal (via the separation of flows into the Internal Catchment Drainage System and the Up-catchment Diversion System);
- the prevention of sediment-laden runoff from the mine site; and
- compliance with regulatory conditions of approval

The preparation of this ESCMP is a requirement of DA 14/98 condition 3.5(a)(iii) (Section 2.1.1) and also addresses certain Conditions of Authority for ML 1535 (Section 2.2).

1.2 SCOPE

In accordance with the requirements for this ESCMP under DA 14/98 condition 3.5(a) (Section 2.1), this ESCMP:

- will be applied to all mine-related activities within ML 1535 and ML 1791, water supply borefields, pipelines and civil infrastructure (roads, tracks and travelling stock reserves) modified as a result of the approval of the activities at the CGO from time to time;
- details of temporary and permanent sediment and erosion control systems to be used during both mine construction and operation, including for earthworks associated with landscaping;
- details of salinity management; and
- includes a programme for reporting on the effectiveness of the sediment and erosion control systems and performance against objectives contained in this ESCMP.

The remainder of this ESCMP has the following structure:

Section 2	Discusses the relevant legislation and associated conditions of approval, Australian Standards and industry guidelines.
Section 3	Details the specific erosion and sediment control systems for the mine area, Saline Borefield, relocation of the Travelling Stock Reserve (TSR) and the realignment of Lake Cowal Road and the Bland Creek Palaeochannel borefield and pipeline.
Section 4	Provides details for management of salinity issues.
Section 5	Details soil management measures including soil stripping scheduling, techniques and stockpile management.
Section 6	Provides an overview of the CGO's rehabilitation principles and objectives.
Section 7	Summarises stakeholder consultation and complaint receipt processes.
Section 8	Outlines CGO's auditing requirements and review requirements for this ESCMP.
Section 9	Details the CGO's reporting requirements for the performance of erosion and sediment control systems .
Section 10	References cited in this document.

Appendices.

2 STATUTORY REQUIREMENTS

This ESCMP has been prepared in response to the requirements of various conditions of approval under State legislation. Other relevant policies and guidelines are also outlined in the sections below.

2.1 ENVIRONMENTAL PLANNING AND ASSESSMENT ACT 1979

2.1.1 DA 14/98

DA 14/98 condition 3.5(a) is a key influence on the ESCMP (see Table 1).

Table 1
DA 14/98 Requirements for this Plan

Condition		Corresponding Section of this Plan
3.5	***Prevention of Soil Erosion***	
	The Applicant shall prepare and implement the following plans to the satisfaction of the Planning Secretary:	
	(a) an erosion and sediment control management plan for the site which shall include, but not be limited to:	
	(i) details of temporary and permanent sediment and erosion control systems to be used during both mine construction and operation, including for earthworks associated with landscaping;	Sections 3, 4, 5 and 6
	(ii) details of salinity management; and	Section 4
	(iii) a program for reporting on the effectiveness of the sediment and erosion control systems and performance against objectives contained in the approved erosion and sediment control management plan, and EIS; and	Section 9

In addition to DA 14/98 condition 3.5(a) above, other development consent conditions relevant to this ESCMP include:

- DA 14/98 condition 3.4(c)(iv), which requires the preparation of a Biodiversity Offset program including the detailed description of the measures to control erosion.
- DA 14/98 condition 9.1(b), which requires the preparation of an Annual Review. This condition is addressed in Section 9.2.
- DA 14/98 condition 9.1(c), which outlines the revision requirements for the CGO's environmental management plans, strategies and programs, including this ESCMP. This condition is addressed in Section 8.2.
- DA 14/98 condition 9.1(d), which requires the establishment of a Community Environmental Monitoring and Consultative Committee (CEMCC). This condition is addressed in Section 7.1.
- DA 14/98 condition 9.2(a), which requires Independent Environmental Audits to be conducted. These conditions are addressed in Section 8.1.1.
- DA 14/98 condition 9.3, which outlines incident and non-compliance reporting requirements. This condition is addressed in Sections 9.3 and 9.4.
- DA 14/98 condition 9.4(a)(v), which requires the maintenance of a complaints register. This condition is addressed in Section 7.2.

2.1.2 SSD 10367

With its focus primarily on underground activities, SSD 10367 has only a few relevant conditions to this plan. They are:

- Condition B8 (Table 1) which requires that:
  Storages are suitably designed, installed and maintained to ensure no discharge of mine water or sediment-laden water outside the ICDS

- Condition C9, which requires the preparation of an Annual Review. This condition is addressed in Section 9.2.
- Condition C5, which outlines the revision requirements for the CGO's environmental management plans, strategies and programs, including this ESCMP. This condition is addressed in Section 8.2.
- Condition A11, which requires the establishment of a Community Environmental Monitoring and Consultative Committee (CEMCC). This condition is addressed in Section 7.1.
- Condition C11, which requires Independent Environmental Audits to be conducted. These conditions are addressed in Section 8.1.1.
- Conditions C7 and C8 which require incident and non-compliance reporting requirements. These conditions are addressed in Sections 9.3 and 9.4.
- Condition C14(a)(ix), which requires the maintenance of a complaints register. This condition is addressed in Section 7.2.

2.2 MINING ACT 1992

The principal regulatory instrument under the Mining Act 1992 are the conditions of authority on CGO's mining leases; ML 1535 and ML 1791. Those on ML 1535 include requirements relevant to soil erosion and pollution prevention, maintenance of roads and rehabilitation requirements. The Authority for ML 1535 also includes reporting requirements, including preparation of a Mining Operations Plan (MOP) and Annual Review (AR). Relevant Conditions of Authority for ML 1535 include:

Prevention of Soil Erosion and Pollution

14. Operations must be carried out in a manner that does not cause or aggravate air pollution, water pollution (including sedimentation) or soil contamination or erosion, unless otherwise authorised by a relevant approval, and in accordance with an accepted Mining Operations Plan. For the purpose of this condition, water shall be taken to include any watercourse, waterbody or groundwaters. The lease holder must observe and perform any instructions given by the Director-General in this regard.

This Condition of Authority is addressed in Sections 3 to 6 of this ESCMP.

Rehabilitation

12. (a) Land disturbed must be rehabilitated to a stable and permanent form suitable for a subsequent land use acceptable to the Director-General and in accordance with the Mining Operations Plan so that:

- *there is no adverse environmental effect outside the disturbed area and that the land is properly drained and protected from soil erosion.*
- *the state of the land is compatible with the surrounding land and land use requirements.*
- *the landforms, soils, hydrology and flora require no greater maintenance than that in the surrounding land.*
- *in cases where revegetation is required and native vegetation has been removed or damaged, the original species must be re-established with close reference to the flora survey included in the Mining Operations Plan. If the original vegetation was not native, any re-established vegetation must be appropriate to the area and at an acceptable density.*
- *the land does not pose a threat to public safety.*

(b) Any topsoil that is removed must be stored and maintained in a manner acceptable to the Director-General.

13. The lease holder must comply with any direction given by the Director-General regarding the stabilisation and revegetation of any mine residues, tailings or overburden dumps situated on the lease area.

These Conditions of Authority are addressed in Sections 3 to 6 and 9 of this ESCMP.

Mining Operations Plan

25(4) The Plan must present a schedule of proposed mine development for a period of up to seven (7) years and contain diagrams and documentation which identify

...

(i) Surface and groundwater management systems including monitoring (including integrated erosion and sediment controls);

This Condition of Authority is addressed in Sections 3 to 6 and 9 of this ESCMP.

2.3 OTHER LEGISLATIVE REQUIREMENTS

The following Acts contain provisions relevant to erosion and sediment control for the CGO:

Soil Conservation Act, 1938

Under this Act, where the Commissioner of the Soil Conservation Service of NSW considers that any act (or failure to carry out an act) has caused or is likely to cause soil erosion or land degradation, the Commissioner may issue a notice upon the owner or occupier of the land requiring the owner or occupier to abstain from doing the act or to carry out the act as specified in the notice. It is an offence to fail to comply with the notice.

Water Management Act, 2000

Under the *Water Management Act, 2000,* water use, drainage activities and floodplain management should avoid or minimise land degradation, including soil erosion, compaction, geomorphic instability, contamination, acidity, waterlogging, decline of native vegetation or, where appropriate, salinity, where possible, land should be rehabilitated.

Local Land Services Act, 2013

The *Local Land Services Act* 2013, provides a regulatory framework for native vegetation and land management activities in NSW. The *Local Land Services Act, 2013* also establishes a statutory corporation (i.e. Local Land Services) with responsibility for management and delivery of local land services in the social, economic and environmental interests of the State in accordance with any State priorities for local land services. The role of Local Land Services is to provide programs and advisory services associated with agricultural production, biosecurity, natural resource management and emergency management. The CGO is located within the Riverina Local Land Services division.

Protection of the Environment Operations Act, 1997

The *Protection of the Environment Operations Act, 1997* (PoEO Act) and the Protection of the Environment Operations (General) Regulation, 2009 set out the general obligations for environmental protection for development in NSW, which is regulated by the Environment Protection Authority (EPA).

Offences and duties under the PoEO Act relevant to this ESCMP include a person who pollutes any waters is guilty of an offence (s120). Substances that may cause water pollution includes any ashes, soil, earth, mud, stones, sand, clay or similar inorganic matter (Schedule 5 of the Protection of the Environment (General) Regulation, 2009). It is a defence in a prosecution under section 120 that the pollution was regulated by an environment protection licence, the conditions of which had not been contravened.

Under section 148 of the PoEO Act, a duty is imposed on certain persons to notify the EPA or local council where a pollution incident occurs in the course of an activity so that material harm to the environment is caused or threatened. The persons upon whom the duty is imposed include the person carrying on the activity and the occupier of the premises on which the incident occurred.

2.4 POLICIES AND GUIDELINES

Presented below is a summary of government policies and guidelines that may be of relevance to this ESCMP. These documents are referenced within the relevant sections of this ESCMP where appropriate. It is the responsibility of Evolution employees and its contractors to maintain up to date versions of these documents on file and be cognisant of their content.

Wetlands Policy, 2010

The *NSW Wetlands Policy, (DECCW, 2010)* aims to minimise any further loss or degradation of wetlands and to restore degraded wetlands through the improvement of wetland management by providing guidance and support to wetland managers, including individual land holders. Goals and principles relevant to this ESCMP are that water entering natural wetlands will be of sufficient quality so as not to degrade the wetlands, and land use and management practices that maintain or rehabilitate wetland habitats and processes will be encouraged. The Policy is addressed by the implementation of the erosion and sediment control systems outlined in Section 3 to 6 of this ESCMP.

A Resource Guide for Local Councils: Erosion and Sediment Control, Department of Environment and Conservation (DEC), 2006

This document provides for the implementation of erosion and sediment control measures, the preparation of erosion and sediment control plans, the management of stormwater and outlines principles and techniques for revegetating sites during development. Diagrams, formulae and tables are given including for the design and construction of sediment basins, traps and filters, and banks and channels linings. Design information relevant to this ESCMP is dealt with further in the body of the plan. An extract of this handbook is provided in Appendix B.

Managing Urban Stormwater – Soils and Construction Volume 1, Landcom, 2004

The manual focuses on the minimisation of erosion and prevention of sediment movement off site during the construction phase of land and building development. Relevant to this ESCMP, the manual:

- assists in identifying and addressing the various soil and water management issues at the design concept stage;
- outlines the preparation of erosion and pollution control plans;
- describes soil and water management aspects of land clearing and topsoil stripping, stockpiling and reuse in rehabilitation; and
- provides information on control of soil erosion sheet and concentrated flow and subsoil drainage in respect to construction and maintenance of site drainage works.

Design information relevant to this ESCMP is dealt with further in the body of the plan. An extract of this manual is provided in Appendix C.

Managing Urban Stormwater – Soils and Construction Volume 2E: Mines and Quarries, DECCW, 2008

Volume 2E of the Managing Urban Stormwater manual guides the user in the application of the principles and practices of erosion and sediment control described in Volume 1 (Landcom, 2004) to mines and quarries, specifically by:

- outlining an erosion and sediment control strategy development approach;
- summarising the considerations of mine and quarry design relevant to operational erosion and sediment control;
- providing information on rehabilitation relevant to minimising site erosion;
- providing guidance on applicable erosion and sediment control techniques at mines and quarries; and
- outlining the documentation strategy for erosion and sediment control plans.

Best Practice Erosion and Sediment Control Guidelines, IECA, 2008

The IECA's *Best Practice Erosion and Sediment Control Guidelines* contain clearly defined erosion and sediment control guidelines, recommended standards, erosion and sediment control plan development procedures, erosion and sediment control techniques, field guides and guidance on the design, construction and operation of sediment basins.

Guidelines for Erosion and Sediment Control on Building Sites, DLWC, 2001
This handbook provides a guide to best practice to reduce storm water pollution from building sites. The guidelines provide for legislative requirements, suggested erosion and sediment controls, the preparation of erosion and sediment control plans and soil and water management plans and a number of fact sheets providing for soil erosion and sediment prevention and control, clean-up and rehabilitation.

***Draft Guidelines for the Design of Stable Drainage Lines on Rehabilitated Minesites in the Hunter Coalfields, Department of Infrastructure, Planning and Natural Resources (DIPNR), undated* (the Draft Guidelines)**
The Draft Guidelines provide for the long-term stability of drainage lines on mine sites. Drainage lines require the application of control structures to mitigate against erosion and sediment discharge. The Draft Guidelines outline the elements of drainage design that include specific erosion control techniques and revegetation of areas adjacent to the drainage lines to control soil erosion of spill over areas. A copy of the Draft Guidelines is provided in Appendix E.

Erosion and Sediment Control on Unsealed Roads (NSW Office of Environment and Heritage, 2012)
This guide provides practical guidance on soil erosion and sediment control practices to minimise the amount of sediment entering waterways and for routine maintenance of unsealed roads using best practice standards.

Guide to Road Design (Austroads)
The *Austroads Guide to Road Design* is intended to provide designers with a framework that promotes efficiency and economy in design and construction, and consistency and safety for road users.

The Lake Cowal Road realignment (Section 6.2) was constructed in accordance with the *Austroads Guide to Road Design*, under DA 14/98 condition 7.1(b).

Controlled Actions on Waterfront Land Guidelines for Laying Pipes and Cables in Watercourses on Waterfront Land
This guideline provides the design and installation considerations for the laying of pipes and cables on waterfront land. In general, the guideline details measures that should be implemented such that disturbance associated with the laying of cables and pipes across a watercourse or on waterfront land is minimised, and rehabilitation of disturbed areas restores bed and bank stability and the integrity of any existing vegetation on the waterfront.

2.5 LOCAL AND REGIONAL MANAGEMENT PLANS

The *Cowal Gold Project Commission of Inquiry Commissioners Report* (COI) (Commissioners of Inquiry for Environment and Planning, 1999) stated that as part of its ongoing consultative process, the Applicant must ensure that the development remains compatible with the Floodplain Management Plan, the Lake Cowal Land and Water Management Plan, the Bland Creek Land and Water Management Plan, and the Mid Lachlan Regional Vegetation Plan (Commissioners of Inquiry for Environment and Planning, 1999), all of which were in draft form at the time of writing of the COI report.

Evolution will continue to consider the aims of these plans, where practical to facilitate a whole of catchment management objective.

Presented below is a summary of vegetation, land and water management plans relevant to the CGO. These documents are referenced within the relevant sections of this ESCMP where appropriate.

Lachlan River (Jemalong Gap to Condobolin) Floodplain Management Plan, NSW Office of Environment and Heritage and NSW Office of Water, 2012

The *Lachlan River (Jemalong Gap to Condobolin) Floodplain Management Plan* defines the requirements for managing floodwaters within floodplains. The floodplain management principle relevant to this ESCMP seeks to minimise the velocity of flood flow to avoid erosion or increased siltation in a flood event. The plan also provides a list of recording measures to be undertaken after a flood event and specifies that any floodplain management measures implemented must avoid or minimise land degradation, including soil erosion.

Jemalong Land and Water Management Plan, Steering Committee, 2000

The *Jemalong Land and Water Management Plan* (Jemalong Land and Water Management Plan Steering Committee, 2000) provides for the alleviation of land and water degradation, improvement of natural resource management and sustainability of agriculture and the environment in the Jemalong Irrigation District. Relevant to this ESCMP, the plan recommends the remediation of any degraded lands and the reduction of water erosion and sedimentation to reduce salinisation of land and water bodies including Lake Cowal and to protect Lake Cowal wetlands.

Performance indicators include reductions in visible salinity and improved health of wetlands and remnant vegetation. Routine soil salinity surveys are recommended to indicate whether adverse effects on soil and water quality by current land management practices are being minimised. Performance indicators are to be monitored in accordance with a programme based on Jemalong Irrigation Limited's Irrigation Corporation Water Management Works Licence and the CGO Environment Protection Licence (EPL). Issues relating to the effect of erosion and sedimentation on wetlands are proposed to be addressed in more detail in a *Jemalong Irrigation District Wetlands Management Plan*.

Lake Cowal Land and Water Management Plan, Australian Water Technologies Pty Ltd, 1999

This plan identifies environmental issues relevant to Lake Cowal including evidence of soil erosion, declining soil structure and increasing salinity. It recommends that land holders, including Evolution, restrict new scrub clearance to minimise impacts on soil structure and encourage the revegetation of the Lake and margins to slow the rate and volume of runoff, thereby reducing sedimentation of the Lake. It also suggests that landholders participate in, and monitor, trials to develop management options for improving vegetation cover and revegetation.

Lachlan Catchment Action Plan (2013-2023), Lachlan Catchment Management Authority, 2013

The *Lachlan Catchment Action Plan (2013 – 2023)* is a strategic regional plan that builds on the past and addresses the key future natural resource management issues for the catchment (Lachlan Catchment Management Authority [LCMA], 2013). The Lachlan Catchment Action Plan sets out to deliver triple bottom line outcomes for the five social-ecological systems in the catchment and in doing so maintain or improve their resilience (LCMA, 2013).

The CGO sits within the Lachlan Plains social-ecological system. There are no specific 'Priority catchment scale targets' to maintain or increase the Lachlan Plains social-ecological system. However, general strategic priorities for the greater catchment area of relevance to the ESCMP include targets T17 and T18, viz,

> *T17. Improved water quality*
>
> *T18. Improved condition of riparian areas, wetlands, flood dependent ecosystems and groundwater dependant ecosystems*

The construction and use of sediment and erosion control systems at the CGO (both temporary and permanent) are considered to address Target 17 of the *Lachlan Catchment Action Plan (2013-2023)*. The erosion and sediment control strategies are presented in Sections 3 to 6 of this ESCMP and will be designed to reduce sediment and erosion surface water runoff from within the CGO (ML 1535 and

ML 1791). These strategies are considered consistent with target T17 and T18 of the *Lachlan Catchment Action Plan (2013-2023)*.

The revegetation strategies for the CGO in particular the New Lake Foreshore described in the CGO's Rehabilitation Management Plan and CWMP are considered to be consistent with target T18 of the *Lachlan Catchment Action Plan (2013-2023)*.

Bland Creek Catchment Plan, Bland Creek Catchment Committee, 2002

The Bland Creek Catchment covers an approximate area of 940,950 hectares and is the southern portion of the Lachlan Catchment containing Lake Cowal. There are six sub-catchments within the Bland Creek Catchment, of which the CGO is located in the north of the Barmedman sub-catchment.

The *Bland Creek Catchment Plan* (Bland Catchment Committee, 2002) is divided into two stages. Stage one was developed to address local natural resource and land management issues and an economic analysis of the issues, presenting a 'no-plan' scenario. The Catchment Plan developed by stage one has collated existing information on the Catchment, provides an overview of the Catchment's physical features and socio-economic status and gives an account of the extent and severity of each issue at a sub-catchment level. Stage two was developed from the findings of stage one and provides a Catchment Action Plan, in which priorities are set for future 'on-ground' activities.

The Bland Creek Catchment Action Plan identifies a number of proposed actions to overcome threats and barriers to natural resource management. Actions are provided for soils, water, native vegetation and biodiversity, and salinity.

3 EROSION AND SEDIMENT CONTROL SYSTEMS

3.1 MINE AREA

Details of erosion and sediment control systems for the mine site area (i.e., ML 1535 and ML 1791), for both the construction and operation phases of the CGO are provided in the following subsections. These systems have been developed to meet the objectives of this ESCMP (Section 1.1) include monitoring, ameliorative responses and maintenance activities.

The CGO's initial construction phase included the establishment of major components of the CGO's water management system (i.e. construction of the Temporary Isolation Bund, Lake Protection Bund, Internal Catchment Drainage System, Up-catchment Diversion System and pit dewatering system), process plant and tailings storage facilities. With the exception of the ongoing IWL construction, the CGO is currently in its Operational Phase, which includes all activities associated with resource extraction and development of the approved project.

The CGO has been operational since 2005 however, periodically, parts of the site will be subject to new construction activities such as the new surface facilities associated with the Underground Development approved in September 2021. Further developments of various scales and duration are expected to occur from time to time until the mine eventually closes.

3.1.1 Landscape, Topography and Soils

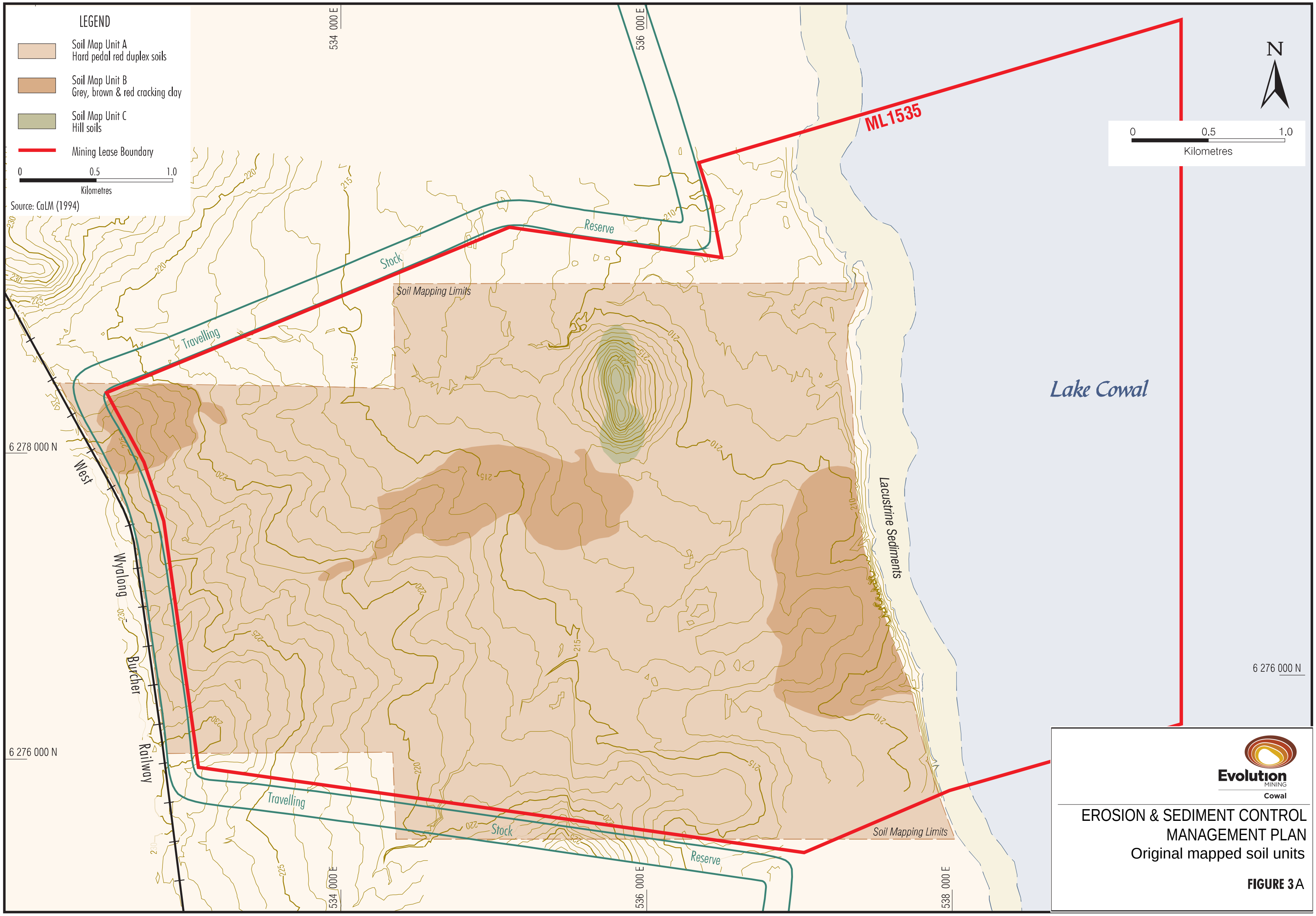
The general landscape of the approved CGO (Figure 2) area is flat to very gently undulating land with occasional rocky outcrops that slope from west to east towards Lake Cowal. It is entirely located within the Bland Creek catchment.

The original suitability assessment of soil types at the CGO was included in North's 1998 EIS (North Limited, 1998a). Appendix B1 (CALM, 1994) and Appendix B2 of the EIS (Resource Strategies, 1997). This assessment included descriptions of soil types, general characterisation of suitable stripping depths for revegetation/rehabilitation activities and formulation of soil stripping and soil stockpiling strategies. The major soil types identified (CALM, 1994) were:

- hard pedal red duplex soils;
- grey, brown and red cracking clays;
- hill soils; and
- lacustrine (lake) sediments.

The soils map units at the CGO are shown on Figure 3.

Descriptions of the soil profile are included in the Soil Stripping Management Plan (SSMP). Soil management measures implemented at the CGO as described in the SSMP are summarised in Section 5 of this ESCMP.



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EROSION & SEDIMENT CONTROL MANAGEMENT PLAN

Original mapped soil units

FIGURE 3A

HAL-02-07 CGO MP 2017_ESC MP_001B

3.1.1 Relevant Standards and Design Criteria

General design criteria for the erosion and sediment control structures used at the CGO are presented below and are largely derived from *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C). Other references are included in Sections 3 to 6 and 10 of this ESCMP. Where there is any contradiction between these other guidelines and the guideline *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004), the latter has taken precedence in the preparation of this document.

Design Average Recurrence Interval

The design average recurrence interval (ARI) used to size individual components of the erosion and sediment control systems presented in this ESCMP is shown in Table 2.

Table 2
Suggested Design Average Recurrence Intervals for Erosion and Sediment Control Measures in Urban Areas

Control Measure	Estimated Design Life	
	0 – 12 Months	> 12 Months
	Design ARI (Years)**	
Diversion Bank	1 – 10	*
Level Spreader	1 – 10	*
Waterway	1 – 10	*
Sediment Basin Primary Outlet	1 – 5	*
Sediment Basin Emergency Outlet	10 – 20	*
Sediment Trap	1 – 5	*
Outlet Protection	1 – 20	*
Grade Stabilisation Structure	1 – 20	*
Detention Basin Primary Outlet	1 – 5	*
Detention Basin Emergency Outlet	10 – 20	*
Waterway Diversion	1 - 2	*

Source: Managing Urban Stormwater – Soils and Construction (Landcom, 2004).

* Full design required in accordance with major/minor concept (Institute of Engineers Australia, 1987, revised 2001).

** Designs should generally comply with the upper limit and the ranges specified in the table, unless the consequences of failure are considered low.

Rainfall

Rainfall intensity for the sizing of components of the erosion and sediment control systems uses the method outlined in *Australian Rainfall and Runoff* (Institute of Engineers Australia, 1987, revised 2001) (provided in Appendix D) (or as revised) and *Technical Handbook No. 5 Design Manual for Soil Conservation Works* (Soil Conservation Service of NSW, 1982) (Appendix A) using Intensity-Frequency-Duration (IFD) data. Table 3 contains IFD data for Lake Cowal.

Table 3
Average Storm Recurrence Interval (mm/h) – Lake Cowal

Duration	Average Storm Recurrence Interval (years)						
(minutes)	**1**	**2**	**5**	**10**	**20**	**50**	**100**
5	59.54	78.38	106.20	123.03	145.87	177.40	202.62
6	55.68	73.28	99.23	114.91	136.20	165.59	189.09
7	52.47	69.04	93.42	108.16	128.17	155.77	177.85
8	49.73	65.42	88.48	102.41	121.33	147.42	168.28
9	47.35	62.28	84.20	97.43	115.40	140.19	160.00
10	45.26	59.52	80.44	93.06	110.20	133.84	152.73
11	43.40	57.07	77.10	89.17	105.58	128.20	146.27
12	41.74	54.87	74.10	85.69	101.44	123.15	140.49
13	40.23	52.89	71.39	82.54	97.70	118.59	135.27
14	38.86	51.08	68.93	79.68	94.29	114.44	130.52
15	37.61	49.42	66.68	77.06	91.19	110.65	126.18
16	36.45	47.90	64.61	74.66	88.33	107.16	122.20
17	35.39	46.50	62.69	72.44	85.69	103.95	118.52
18	34.40	45.20	60.92	70.38	83.25	100.97	115.11
20	32.62	42.85	57.73	66.68	78.85	95.61	108.98
25	29.05	38.15	51.34	59.25	70.04	84.87	96.71
30	26.34	34.57	46.48	53.62	63.35	76.73	87.40
35	24.18	31.73	42.63	49.16	58.06	70.30	80.05
40	22.43	29.42	39.50	45.53	53.76	65.07	74.07
45	20.97	27.50	36.89	42.51	50.17	60.71	69.09
50	19.72	25.86	34.67	39.94	47.13	57.01	64.87
55	18.65	24.45	32.76	37.73	44.51	53.83	61.24
60	17.71	23.22	31.10	35.80	42.23	51.05	58.07
75	15.18	19.86	26.50	30.44	35.83	43.21	49.08
90	13.36	17.46	23.20	26.60	31.26	37.64	42.69
Duration	**Average Storm Recurrence Interval (years)**						
(hours)	**1**	**2**	**5**	**10**	**20**	**50**	**100**
2.0	10.89	14.20	18.77	21.45	25.14	30.18	34.17
3.0	8.14	10.58	13.87	15.78	18.43	22.03	24.87
4.0	6.61	8.58	11.18	12.68	14.77	17.61	19.83
5.0	5.63	7.29	9.46	10.71	12.44	14.80	16.64
6.0	4.94	6.38	8.25	9.32	10.82	12.84	14.42
8.0	4.01	5.18	6.66	7.50	8.68	10.27	11.51
10.0	3.42	4.40	5.64	6.33	7.32	8.64	9.67
12.0	3.00	3.86	4.92	5.52	6.36	7.50	8.38
14.0	2.70	3.47	4.42	4.95	5.71	6.72	7.51
16.0	2.46	3.16	4.02	4.50	5.19	6.11	6.82
18.0	2.26	2.91	3.70	4.14	4.77	5.61	6.27
20.0	2.10	2.70	3.43	3.84	4.42	5.20	5.81
22.0	1.97	2.52	3.21	3.59	4.13	4.86	5.42
24.0	1.85	2.37	3.01	3.37	3.88	4.56	5.09
36.0	1.38	1.77	2.24	2.50	2.87	3.37	3.76
48.0	1.11	1.42	1.79	2.00	2.30	2.69	3.00
60.0	.93	1.19	1.50	1.67	1.92	2.25	2.50
72.0	.80	1.02	1.28	1.43	1.64	1.92	2.14

Source: Gilbert and Associates Pty Ltd (September 2003).
mm/h – millimetres per hour.

The depth of rain generated by a rainfall event is determined by multiplying the rainfall intensity of a given ARI event by the duration of the event. For example a 1 in 100 year rainfall event of 48 hours duration corresponds to a rainfall intensity of 3 mm/h, which in turn corresponds to 144 millimetres (mm) of rainfall.

Runoff

Peak flow calculations for components of the erosion and sediment control systems are made using a combination of the Statistical Rational Method (SRM), and the Deterministic Rational Method (DRM) as outlined in Chapter 1, Section 2.2 of the *Technical Handbook No. 5 Design Manual for Soil Conservation Works* (Soil Conservation Service of NSW, 1982) (Appendix A).

The SRM estimates time of concentration as a function of catchment size, and the coefficient of runoff is determined as a function of catchment size, locality and annual exceedance probability (AEP). This method is used on “natural” or untreated catchments to estimate peak discharge. The DRM is used to estimate peak discharge from treated catchments. Time of concentration for the DRM is estimated using flow routing procedures which estimate flow time for individual structures. The runoff coefficient for the DRM is a function of land use, relief, depression storage effects, infiltration and soil factors, and the design AEP, and will be modified using an area correction factor based on catchment size.

Culverts

All culverts are designed and constructed in accordance with details provided in Chapter 5.3.5 of *Managing Urban Stormwater – Soils and Construction Volume 1* (Landcom, 2004) (Appendix C). Culvert crossing capacities are designed in accordance with the methods stated above. Culverts under roads are designed and constructed in accordance with the Austroads *Guide to Road Design.*

Culvert design and construction takes into account a range of factors including purpose, location and materials available. The design storm event is selected based upon an assessment of catchment characteristics and consequence of failure. At a minimum, culverts are designed to convey the 1 in 10 year ARI critical duration flow (i.e. minimum criterion for waterways contained in Table 2).

Sediment Basins

Sediment basins are designed in accordance with procedures detailed in Section 6.3.3 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C). Design particle size and USLE “K” factor are determined from particle size analysis and laboratory testing previously conducted by CALM in 1994.

Attachment 7.1 of Appendix B of the EIS (North Limited, 1998a) provides further advice in relation to the application of USLE “K” factors (CALM, 1994):

> ***Erodibility (USLE K factor)***
>
> *The Unified Soil Loss Equation includes a soil erodibility factor known as K (Wischmeier and Smith 1978). K factor is a derived index of the susceptibility of a soil to sheet and rill erosion. The formula used to derive the K factor is USLE modified for Australian conditions and based on that used in SOILOSS (Rosewell and Edwards 1988) with profile permeability modified to follow that used by Soil and Water Conservation Society (1993).*
>
> *Silt and fine sand percentages are derived from fine earth fraction particle size results.*
>
> *Note: K factor of itself has a very limited value unless used in conjunction with all other factors in USLE.*

Attachment 7.3 of Appendix B of the EIS (North Limited, 1998a) indicates that USLE “K” factors range from 0.006 to 0.051 across soil map units A, B, C and K (CALM, 1994). These values correspond to low to moderate erodibility (Attachment 7.3 of Appendix B of the EIS [North Limited, 1998a]). The data presented in Appendix B of the EIS (North Limited, 1998a) is taken into consideration by the design hydrologist when determining sediment basin containment capacities.

As presented in Section 3.1.1 below, containment storages D1 to D10 are, and will continue to be, sized to contain rainfall events ranging from a 1 in 100 year, 48 hour rainfall event to a 1 in 1000 year, 48 hour rainfall event. As a minimum, all other sediment basins have been designed to contain the runoff generated by the 75th percentile 5-day rainfall event as suggested in Section 6.3.3 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) for Type F soils.

The C_{10} coefficient of runoff used in the Rational Method to estimate average discharge in designing sedimentation basins would be calculated using Table 3.2 in *Urban Erosion and Sediment Control Handbook* (CALM, 1992) (Appendix B) and equations 14.11 and 14.12 in *Australian Rainfall and Runoff* (Institute of Engineers Australia, 1987, revised 2001) (Appendix D) (or as revised).

Diversion Bank Channels and Grassed Waterways

Diversion bank channels and grassed waterways have been designed in accordance with details provided in Chapter 2 of *Design Manual for Soil Conservation* Works *Technical Handbook No. 5* (Soil Conservation Service of NSW, 1982) (Appendix A) and Section 5.4.4 of *Managing Urban Stormwater - Soils and Construction* (Landcom, 2004) (Appendix C).

Sodicity

Investigations into the dispersivity/stability of CGO soil types (Knight Piesold, 1995) indicated moderate dispersion potential. Further tests conducted using lake water and saline groundwater indicated stability of all water containment structures (including tailings embankments and the temporary isolation bund) would not be compromised by soil dispersion (North Limited, 1998a). Attachment 7.3 of Appendix B of the EIS (North Limited, 1998a) indicates that USLE “K” factors range from 0.006 to 0.051 across soil map units A, B, C and K (CALM, 1994).

Attachment N4 of Appendix N of the EIS (North Limited, 1998a) contains the following in relation to potential soil loss:

> *The estimate of soil loss due to erosion has been based on the Universal Soil Loss Equation. It is estimated that after rehabilitation the total soil loss from the TSF, waste emplacements and perimeter bund should not exceed 450 tonnes per year. Expressed as an average unit rate this is equivalent to 0.7 tonnes per hectare per year, which is considered to be low. A loss of 1 mm depth of surface soil would equate to about 14 tonnes per hectare.*

Measures that have been adopted to mitigate potential impacts on soils at the CGO include (North Limited, 1998a):

- minimising the area disturbed by the CGO and restricting access to non-disturbed areas;
- ripping and rehabilitation of hardstand areas and roads no longer required for access;
- avoidance of soil stripping operations during particularly wet or dry periods, minimising compaction during soil excavation and movement and the use of ameliorants where required (e.g. gypsum application to dispersive soils);
- use of silt fences and temporary sediment traps to minimise sediment movement;
- use of diversion banks, channels and rip-rap structures to divert surface water around disturbed areas and control runoff velocity;
- maintaining soil stockpile slopes at or below maximum acceptable angles to resist erosion;
- constructing all access roads at an appropriate slope along the contour, where practicable;
- the use of spoon drains, table drains and concrete culverts to control surface runoff from access roads; and
- leaving the more saline and dispersive soil horizons *in-situ* beneath mine landforms, where possible.

3.1.2 Construction Phase

The initial construction phase of the CGO has been completed and involved the development of major infrastructure components of the mine in preparation for actual mining of the orebody including (but not limited to):

- internal main access road;
- boundary fences;
- ore stockpile and process plant area;
- soil stockpiles;
- internal mine roads;
- contractors' areas;
- borrow pits;
- earthworks associated with landscaping (earth mounds);
- Up-catchment Diversion System;
- Internal Catchment Drainage System; and
- Lake Isolation System (temporary isolation bund, lake protection bund and perimeter waste emplacement).

The major components of the water management system during the construction pre-production phase included the following:

(i) Up-catchment Diversion System;

(ii) Lake Isolation System (including the temporary isolation bund and lake protection bund);

(iii) Internal Catchment Drainage System ;

(iv) Integrated Erosion, Sediment and Salinity Control System; and

(v) Pit Dewatering System.

The Up-catchment Diversion System, Lake Isolation System (including temporary isolation bund, lake protection bund and perimeter waste emplacement) and Internal Catchment Drainage System are described in Section 3.1.2.2 of this ESCMP. The integrated erosion, sediment and salinity control system is described throughout this ESCMP. The pit dewatering system is described in detail in the CGO Water Management Plan (WMP).

Nine contained water storages (D1 to D9) (Figure 4) have been constructed at the CGO to date, with contained water storage D10 yet to be built. A summary of the function, design criteria and capacity of each of the CGO contained water storages is provided in Table 4.

Table 4
Summary of Contained Water Storages

Storage Number	Catchment/Function	Design Criteria	Approximate Storage Capacity (ML)**
Contained Water Storage D1 (Existing)	Runoff from northern perimeter of the northern waste rock emplacement and the IWL. Collected water is pumped to D6.	Runoff from contributing catchment resulting from a 1 in 100 year ARI rainfall event of 48 hours duration	57.8
Contained Water Storage D2 (Existing)	Runoff/seepage from run-of-mine (ROM) and low grade stockpile areas from the northern waste rock emplacement and IWL areas, the southern and eastern batters of the IWL and other areas within the Internal Catchment Drainage System. Collected water is pumped to D6 or D9.	Runoff from contributing catchment resulting from a 1 in 100 year ARI rainfall event of 48 hours duration	198.2
Contained Water Storage D3 (Existing)	Runoff from perimeter catchment surrounding the open pit and the perimeter waste rock emplacement areas. Collected water is pumped to D6.	Runoff from contributing catchment resulting from a 1 in 100 year ARI rainfall event of 48 hours duration	38.1
Contained Water Storage D4 (Existing)	Runoff from the southern perimeter of the southern waste rock emplacement. Collected water is pumped to D6 or D9.	Runoff from contributing catchment resulting from a 1 in 100 year ARI rainfall event of 48 hours duration	62.3
(Process Plant Contained Water Storage) D5A	Process plant area drainage collection. Water is pumped to D6.	Runoff from a 1 in 1,000 year ARI storm of 48 hours duration	78.6
(Process Plant Contained Water Storage) D6 (Existing)	Process water supply storage. Main source of process plant make-up water requirements.	Runoff from a 1 in 1,000 year ARI storm of 48 hours duration above normal operating level	19.3
(Contained Water Storage) D8B (Existing)	Runoff from southern waste rock emplacement, the batters of the southern TSF and other areas within the ICDSInternal Catchment Drainage System. Water is pumped to D9.	Runoff from contributing catchment resulting from a 1 in 100 year ARI rainfall event of 48 hours duration	4330.4
(Process Plant Contained Water Storage) D9 (Existing)	Process water supply storage. Storage for raw water. Water is pumped to D6. Some water used for TSFs lift construction.	Runoff from a 1 in 1,000 year ARI storm of 48 hours duration above normal operating level	726730
(Process Plant Contained Water Storage) D10 (not yet constructed)	Process water supply storage. Storage for raw water. Water is pumped to D9.	Runoff from a 1 in 1,000 year ARI storm of 48 hours duration above normal operating level	1,6371,500

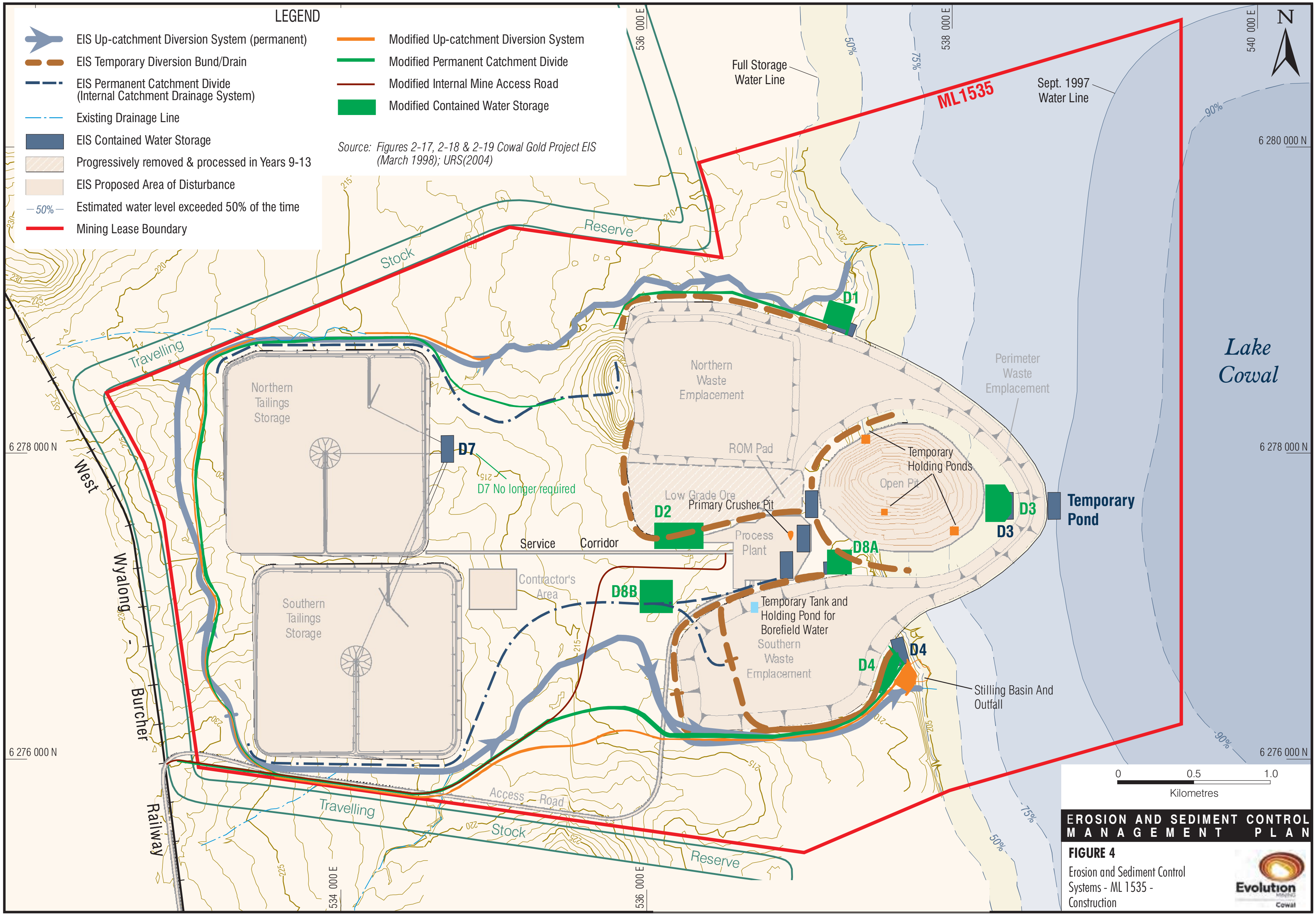
Source: After Hydro Engineering & Consulting (2018).

** Calculated from as-built plans and confirmed by Evolution (HEC, 2018).

ML megalitres.

The potential for erosion and sediment migration in site runoff is high during any construction phase. Given the CGO's proximity to Lake Cowal and the ecological status of the lake, prevention of sediment-laden runoff from the mine site discharging into the lake was, and continues to be, the primary objective of the erosion and sediment control system.

Specific sediment control protection works are put in place prior to initiation of any earthworks at the CGO. Immediately upon completion of earthworks, batter stabilisation and revegetation of disturbed areas commenced.



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The CGO water management system includes both permanent features that will continue to operate post-closure (e.g. diversions of surface water around the site, creation of new catchment divides and isolation of the lake from the open pit), and temporary structures (servicing the life of mine requirements only) (North Limited, 1998a).

3.1.2.1 Details of Temporary Erosion and Sediment Control Systems

Soil conservation and water management features that were implemented during the construction phase, where practicable, included (Resource Strategies, 1997):

- *Use of silt fences and sediment traps to minimise soil movement.*
  Temporary sediment traps and sediment filters are installed downslope of soil stockpiles where the potential for significant sediment migration was identified in accordance with details provided in Sections 3.4.2 and 3.4.3 of the *Urban Erosion and Sediment Control Handbook* (CALM, 1992) (Appendix B) Straw bales are considered more appropriate where a degree of ponding/energy loss is required, while sediment fences are suited to low energy flows where filtration was the primary objective.

- *Use of diversion banks, channels and rip-rap structures to divert surface water around disturbed areas and control runoff velocity.*
  Temporary diversion banks are designed and constructed upslope of disturbance areas where the potential for significant runoff from the upslope undisturbed area to the disturbed area is expected so as to minimise runoff to soil stockpile areas in accordance with details provided in Section 3.3.4.2 of the *Urban Erosion and Sediment Control Handbook* (CALM, 1992) (Appendix B).

  Rip-rap structures are designed and constructed to stabilise concentrated water flow paths where the potential for erosion of flow paths by high energy flows is identified in accordance with details provided in Chapter 5.4.5 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C). Rip-rap structures were typically used to dissipate energy where flow paths traversed relatively steep slopes and/or as waters exit constructed channels and enter natural drainage lines.

The CGO disturbance areas have been minimised by using the footprints of other approved disturbance areas for temporary stockpiling of soils during the CGO mine life (SSMP, Section 4.4).

Soil stockpiles from soil stripping operations at the CGO have been constructed in accordance with details provided in Chapter 4.3.2 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C).

Long-term topsoil stockpiles are constructed up to 3 m in height with slopes at a maximum acceptable angle to resist erosion. Subsoil stockpiles vary in height as determined by storage volumes and available space within the footprint of approved disturbance areas. Soil stockpile batters are generally revegetated in accordance with details provided in Section 3.3.6 of the *Urban Erosion and Sediment Control Handbook* (CALM, 1992) (Appendix B).

Measures to control wind-borne dust and soil sediment runoff from soil stockpiles are presented in the CGO Air Quality Management Plan (AQMP). Measures that are implemented during any construction phase include the following:

- avoiding stripping and placement of soil stockpiles during particularly wet or dry periods whenever possible;
- watering of soil stockpiles during construction when conditions indicated the need (i.e. dry conditions when excessive dust is being generated);

- the stabilisation of completed soil stockpiles and monitoring the revegetation of soil stockpiles (either unassisted or assisted, via the application of seed, fertiliser and water) to promote cover and, as a consequence, erosion control;
- use of silt fences and sediment traps to minimise soil movement;
- construction of soil stockpiles of appropriate height and batter angles; and
- minimisation of runoff to soil stockpile areas by using diversion channels or banks to divert surface water around soil stockpiles.

Monitoring and maintenance activities that is undertaken during any construction phase are described in Section 3.4 of this ESCMP.

- *Constructing all access roads at an appropriate slope along the contour, where practicable.*
  Internal mine roads were constructed at an appropriate slope along the contour (with the exception of roads within the pit limits) in order to avoid steep grades where practicable. The length of unavoidable steep grades was minimised.

- *The use of spoon drains, table drains and concrete culverts to control surface runoff from access roads*.
  Runoff from the road surface was transferred via road side table drains. All drains were designed and constructed in accordance with Section 3.1.1 of this ESCMP and details provided in Chapters 5.4.3 and 5.4.4 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C).

  Where the internal mine roads traverse natural drainage lines, culverts were designed and constructed where the potential for significant upslope runoff was identified in accordance with Section 3.1.1 of this ESCMP and details provided in Chapter 5.3.5 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C). Culvert crossing capacities were designed as detailed in Section 3.2 of this ESCMP.

- *Ripping and rehabilitation of roads no longer required for access.*
  Decommissioning of the roads no longer required for access involved the ripping, topsoiling and revegetation of the disturbed areas with a cover crop in accordance with details provided in Chapter 7 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C). Rehabilitation was undertaken progressively as described in Section 6 of this ESCMP.

3.1.2.2 Details of Permanent Erosion and Sediment Control Systems

The permanent water management systems/structures which have been constructed for the CGO (thereby providing permanent erosion and sediment control systems) include the following:

(a) Up-catchment Diversion System;

(b) Internal Catchment Drainage System; and

(c) Lake Isolation System (temporary isolation bund, lake protection bund and perimeter waste emplacement).

These permanent water management systems/structures as constructed during the CGO construction phase are shown on Figure 4 and will continue to operate post-closure (e.g. diversions of surface water around the site, creation of new catchment divides and isolation of the lake from the open pit), albeit as modified as a result of approved operations (Section 3.1.3). The operational arrangement of permanent water management systems is also outlined in Figure 4.

(a) Up-catchment Diversion System

The Up-catchment Diversion System is designed to convey upper catchment water around the western edge of the CGO area (near the existing tailings storages) and into the existing drainage lines to the north and south of the CGO site (North Limited, 1998a). The system is designed to manage peak discharge from enhanced "green house" 1 in 1,000 year ARI rainfall event (Gilbert and Sutherland, 1997).

The Up-catchment Diversion System maintains the natural flows of surface runoff to Lake Cowal from the up-catchment (North Limited, 1998b).

Further details are provided in the CGO WMP.

<u>Erosion and Sediment Control System</u>

The erosion and sediment control strategy for the construction of the Up-catchment Diversion System required specific sediment control protection works to be put in place prior to initiation of other earthworks. The key element was the installation of sediment control measures (capable of intercepting and retaining on-site sediment) prior to any surface disturbance associated with construction of the Up-catchment Diversion System occurred (Gilbert and Sutherland, 1997).

The erosion and sediment control measures included the installation of silt fences and hay bale weirs downslope of all disturbed areas (Gilbert and Sutherland, 1997). Temporary sediment traps and sediment filters were installed downslope of all disturbed areas where the potential for significant sediment migration was identified in accordance with details provided in Sections 3.4.2 and 3.4.3 of the *Urban Erosion and Sediment Control Handbook* (CALM, 1992) (Appendix B).

In order to reduce the erosion risk during construction of the Up-catchment Diversion System, dispersive soils (e.g. Gilgai areas commonly associated with Soil Map Unit B [Figure 3]) were excavated and replaced with non-dispersive soils where the potential for significant sediment migration was identified (North Limited, 1998a).

All soils stripped during the construction of the CGO have been managed in accordance with the procedures presented in Section 8 and the SSMP. As described in Section 8.3 of this ESCMP, soils may be treated with gypsum to reduce dispersiveness during stockpiling (North Limited, 1998a). As described in the SSMP and Section 5.2 of this ESCMP, infill testing of soil profiles will be undertaken to confirm the precise depths of soil in a particular area (North Limited, 1998a) and any requirements for amelioration at the time of soil stockpiling.

Immediately upon completion of construction and earthworks associated with the Up-Catchment Diversion System, batter stabilisation and revegetation of disturbed areas was undertaken. As indicated in the former *Mid-Lachlan Regional Vegetation Management Plan – Mid Lachlan Vegetation Guides – Section 3.4 Soils and Soil Conservation,* establishment and/or regeneration of native vegetation in and alongside gullies and creeks is perhaps the most effective use of vegetation for erosion control. Vegetative stabilisation was undertaken generally in accordance with details provided in Chapter 7 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C). Rehabilitation was undertaken progressively.

The Up-catchment Diversion System was designed and rehabilitated generally in accordance with the *Draft Guidelines for the Design of Stable Drainage Lines on Rehabilitated Mine Sites in the Hunter Coal Fields* (DLWC, undated) (Appendix E).

Monitoring and maintenance activities for the Up-catchment Diversion System that were undertaken during the construction phase are described in Section 3.4.

(b) Internal Catchment Drainage System

The Internal Catchment Drainage System is a permanent water management feature which will operate during and after the life of mine. The Internal Catchment Drainage System components constructed during the CGO construction phase included a series of contained water storages (D1 to D9) which were used to contain runoff and sediment from different construction and operational areas, and store water for use in process operations (Figure 4) (North Limited, 1998b).

A key objective of the Internal Catchment Drainage System is that during mining operations water collected within the Internal Catchment Drainage System containment storages is retained within CGO's enclosed water catchment and directed to the Process Water Storage (D6) (Figure 4).

(c) Lake Isolation System

The original Lake Isolation System was designed and constructed to hydrologically isolate and provide a barrier between the open pit and Lake Cowal during CGO development, mining and post-closure. The Lake Isolation System consists of three components that combine to provide a permanent arc around the open pit area, *viz. the* temporary isolation bund, lake protection bund and perimeter waste rock emplacement. A cross section of the Lake Isolation System is shown on Figure 5. Future developments of the Lake Isolation System may or may not include a perimeter waste rock emplacement.

3.1.2.3 Monitoring and Maintenance

Weekly inspections will be conducted during the course of construction works to ensure that erosion and sediment controls have been installed and performing effectively (i.e. preventing erosion and containing sediment) to minimise erosion and sediment migration. Site inspections will be undertaken by systematically walking around the site to relevant works areas and recording:

- condition of the erosion and sediment control structures;
- maintenance requirements (if necessary) including instructive actions;
- volume of sediment removed (e.g. from sediment basins to retain capacity requirements); and
- sediment disposal locations.

Stockpile areas will also be inspected and where the potential for excessive sediment migration (including dust generation) is identified, remedial/response measures will be enacted as described in the SSMP.

Inspections of sediment control structures and monitoring of water quality and inspections following rainfall events of 20 mm or more in a 24 hour period (within 48 hours of the event) are conducted during construction activities. Rainfall is monitored by the onsite meteorological station and recorded in accordance with DA 14/98 condition 6.2. Sediment control structures aree inspected for capacity (i.e. sediment build-up consequently reducing capacity), structural integrity (i.e. stability) and effectiveness (i.e. sediment containment).

A comprehensive water quality monitoring programme (including total suspended solids, total dissolved solids, electrical conductivity, major cations/anions and other key parameters) is maintained in accordance with the Surface Water, Groundwater, Meteorological and Biological Monitoring Programme (SWGMBMP) as required by DA 14/98 condition 4.5(a). Data from water quality monitoring (notably total suspended solids (TSS) indicating settlement of the sediment is occurring) and inspections (sediment build-up) is used to assess the success of sediment control structures and evaluate whether ameliorative or maintenance action is required.

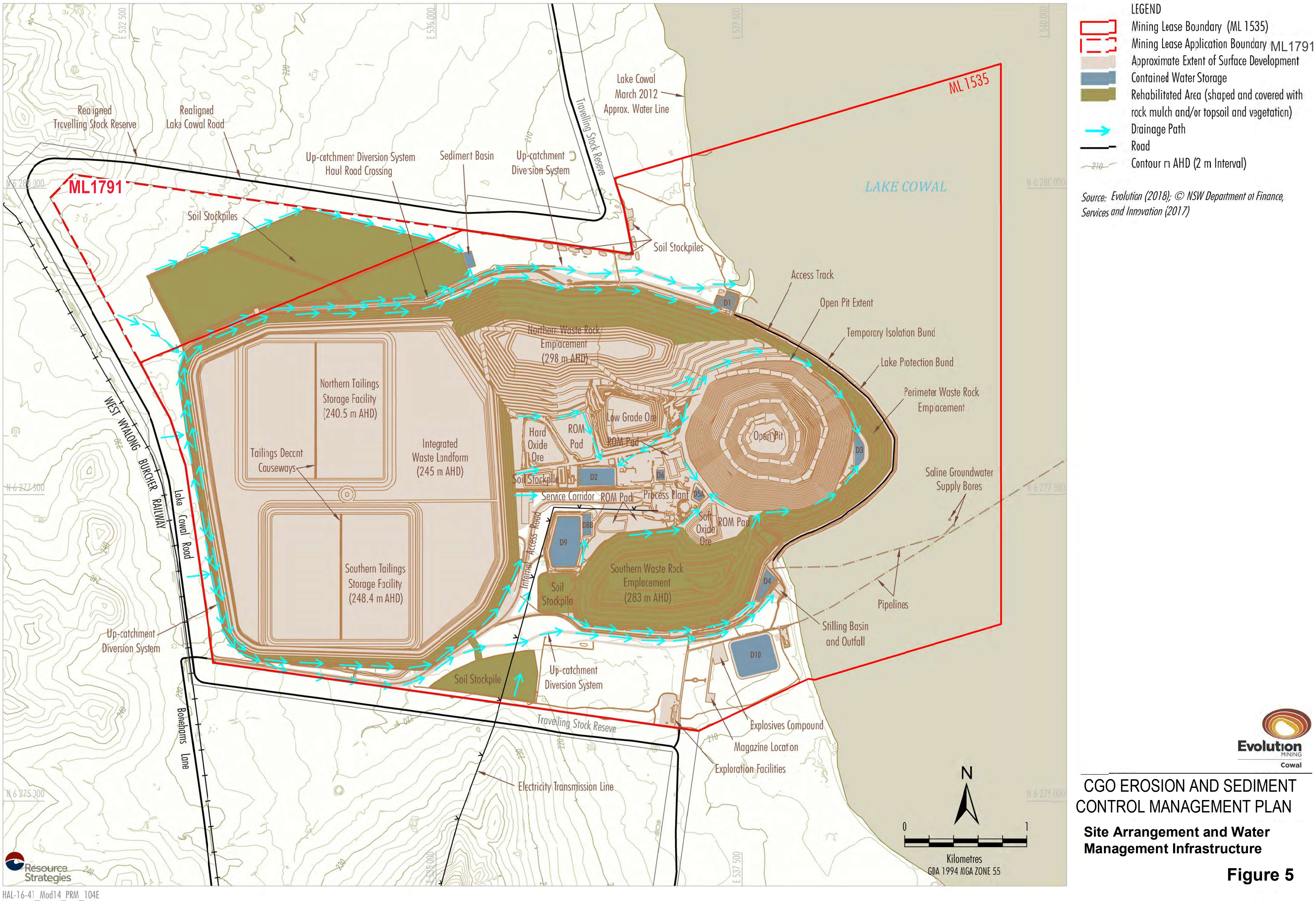
Ameliorative responses and maintenance activities typically used during any construction phase include (from Chapter 8.2 of *Managing Urban Stormwater – Soils and Construction* [Landcom, 2004] [Appendix C]):

- cleaning out of containment structures, diversion drains, etc. where sediment/sand/soil/vegetation builds up;
- repairing of areas of erosion (e.g. lining with a suitable material which may include use of grasses, plastic, geotextile, rock, concrete);
- further application of seed/fertiliser in areas of minor soil erosion and/or inadequate vegetative establishment; and
- installation of additional erosion and sediment control structures.

Ameliorative responses and maintenance activities as described above are conducted generally in accordance with details provided in Chapter 8 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C).

Water quality monitoring of sediment control structures (suspended solids, electrical conductivity and pH) occurs following overflow events (i.e. rainfall events greater than the prescribed design criteria – Section 3.4). Details of monitoring of the Up-catchment Diversion System, contained water storages and temporary storages (e.g. sediment basins) are contained in the SWGMBMP. Generally, water quality monitoring is undertaken monthly in these storages and drains.

Monitoring results are entered into a database and included in the Annual Review.



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CGO EROSION AND SEDIMENT CONTROL MANAGEMENT PLAN

Site Arrangement and Water Management Infrastructure

Figure 5

3.1.3 Operational Phase

3.1.3.1 Temporary Erosion and Sediment Control Systems

Temporary erosion and sediment controls for the approved operational phase of the CGO are presented below.

(a) Soil Stockpiles

The temporary systems installed during the construction phase (Section 3.1.2) for soil stockpiles within the Up-catchment Diversion System will continue to function during the operational phase of the CGO. Ongoing monitoring and maintenance will be undertaken as described in Section 3.4.

Additional soil stockpiles will be constructed both in the south of ML 1535, and in north of ML 1535 and in the south of ML 1791 (Figure 4). These soil stockpiles would be located outside of the Up-catchment Diversion System. As such, prior to placement of soil, upslope runoff would be directed around the stockpile area via a system of diversions/drains/bunds.

Runoff from the northern soil stockpile areas would be captured by toe drains (or other suitable structures) and directed to a sediment basin constructed at the eastern boundary of the northern stockpile within ML 1535 (Figure 4). Runoff from the southern soil stockpile area would be captured by toe drains and directed to the Up-catchment Diversion System (also facilitated by an existing internal access road located east of the soil stockpile) and collected by the Stilling and Outfall Basin located to the south-east of the southern waste rock emplacement (Figure 4).

The upslope stockpile diversions and the northern soil stockpile sediment basin would be constructed, operated and maintained in accordance with Landcom's (2004) *Managing Urban Stormwater – Soils and Construction Volume 1* and the Department of Environment and Climate Change (DECC) (2008) *Managing Urban Stormwater – Soils and Construction Volume 2E Mines and Quarries*. In addition, the construction and management of the soil stockpiles will be undertaken in accordance with the measures detailed within the SSMP, WMP and Section 5.

3.1.3.2 Details of Permanent Erosion and Sediment Control Systems

Details of permanent erosion and sediment controls used in the operational phase of the CGO are presented below.

(a) Up-catchment Diversion System

As described in Section 3.1, the Up-catchment Diversion System is a permanent feature designed to direct drainage around the CGO site.

Any future realignment of the Up-catchment Diversion System will be constructed in accordance with the same long-term design criteria as for the existing Up-catchment Diversion System (Table 4) and the design concepts as described in the WMP. Rehabilitation will be undertaken progressively as described in Section 9 of this ESCMP.

Ongoing monitoring and maintenance of the Up-catchment Diversion System including modified sections will be undertaken as described in Section 3.4.

(b) Internal Catchment Drainage System

The Internal Catchment Drainage System comprises a series of cut-off drains and catchment dams that capture potentially contaminated (or sediment-laden) water that falls as rainfall or collected as seepage within the disturbance area of the CGO. After it is collected, the water is pumped to a process water dam for reuse.

When the CGO is finally decommissioned and rehabilitated, the water within the Internal Catchment Drainage System will be directed to any remaining open pit void(North Limited, 1998a).

A decommissioning strategy for the temporary components of the Internal Catchment Drainage System (i.e. the contained water storages) has been developed in accordance with DA 14/98 condition 4.4(b) and is presented in the WMP.

The strategy is during closure, the contained water storages (i.e. D1 to D10) would be dewatered and liners removed. Alternatively, the contained water storages may be retained for local landholder use upon agreement by Evolution and in consultation with Department of Industry – Lands and Water (DoI L&W) and the DRG.

Rehabilitation concepts for the CGO including the Internal Catchment Drainage System are described in the RMP.

(c) Open Pit

The Lake Isolation System (Section 3.1) provides a permanent barrier around the open pit area (Gilbert and Sutherland, 1997), and prevents erosion and sediment flow toward the lake (Figure 4).

The catchment area draining to the open pit during operations will be restricted to the pit itself and to a small perimeter area enclosed by an external bund. Water management structures will be installed to divert water from other areas of the site outside of this bund to site runoff collection ponds (i.e. D3). The pit will also act as a final water containment point in extreme rainfall events (North Limited, 1998a).

The open pit design includes water management structures (face seepage collection drains) and in-pit sumps in the floor of the pit. The in-pit sumps will be sized to store runoff from a medium sized (1 in 10 year) rainfall event (North Limited, 1998a).

Sediment fluxes are expected to reduce as a result of the CGO. Whilst the soil loss rate is likely to be higher from the disturbance areas than under pre-mining conditions, the final void will intercept a large proportion of runoff, preventing a large proportion of suspended sediment from entering the lake (North Limited, 1998b).

(d) Tailings Storages/IWL

<u>Erosion and Sediment Control System</u>

An interceptor drain will be constructed under the outer lower embankment of the IWL to capture any runoff or seepage through the embankment. Water collected in these drains is directed to external sumps and then pumped directly back to the plant. External areas outside the IWL will either run into these interceptor drains and or water runoff will make its way to D1 and D2 (Table 4).

Erosion control of the tailings storages/IWL during operations is designed so that the outer batters are constructed with surface roughness (through addition of cover such as rock mulch or pasture cover) and with reverse grading berms which longitudinally “drain” to low depressions within the berms. Coupled with a deep cover (of subsoil and topsoil) on the top surfaces, the IWL landform will eventually act to absorb and store rainfall.

<u>Rehabilitation Objectives</u>

The rehabilitation objectives for the IWL are (Evolution, 2018):

- to establish permanently stable landforms;
- during operations, stabilise batters so that they provide minimal habitat value for bird life (i.e. rock mulch or pasture cover);
- post-operations, to establish vegetative communities (including Eucalypt and Riverine woodland species and understorey species such as Rush species and grass species) which are suited to the hydrological features and substrate materials of the landforms;

- post-operations, to establish vegetation communities (including native and/or endemic Eucalypt Woodland, shrubland and grassland species) similar to those remnants in the surrounding landscape which are suited to the substrate materials and slope of the embankments; and
- to exclude grazing and agricultural production.

Consistent with the rehabilitation concepts for the outer batters of the waste rock emplacements, benign primary waste rock mulch will be incorporated into the rehabilitation cover system for the outer batters of the IWL to provide long-term stability, control surface water runoff downslope and reduce erosion potential. The rock mulch will be cross-ripped with gypsum-treated topsoil along the contour of the slope to create ‘troughs and banks’ to further minimise the potential for erosion downslope and enhance vegetation establishment within the troughs. Revegetation would be undertaken with select native and/or endemic Eucalypt Woodland, shrubland and grassland species suited to the slope and substrate materials of the embankment. The depth of soil cover applied would be informed by rehabilitation trial results.

Rehabilitation of the IWL top surface will only be undertaken at the completion of deposition of tailings. The top surface of the IWL would form a low, internally draining landform, with drainage affected by controlled placement of cover materials and a number of shallow swales. The IWL surface would form lower and upper internally draining catchments to minimise surface water runoff from the top surface down the batters of the IWL.

(e) Northern and Southern Waste Rock Emplacements

Waste rock will be placed in a continuous waste emplacement surrounding the open pit consisting of three areas:

- northern waste rock emplacement;
- southern waste rock emplacement; and
- perimeter waste rock emplacement (part of the Lake Isolation System - Section 3.1) of this ESCMP).

Rehabilitation Objectives

The rehabilitation objectives for the waste rock emplacements are to (Evolution, 2018):

- stabilise batter slopes with rock armour (primary waste rock mulch) to control surface water runoff downslope and reduce erosion potential in the long-term;
- provide a stable plant growth medium able to support long-term vegetation growth, including native and/or endemic Eucalypt Woodland, shrubland and grassland species suited to slope and elevated positions similar to those remnants in the surrounding landscape; and
- exclude grazing and agricultural production.

Based on the results of rehabilitation trials and research conducted to date, the rehabilitation cover system for the outer batters of the waste rock emplacements will include benign primary waste rock mulch to provide long-term stability, control surface water runoff downslope and reduce erosion potential. The rock mulch will be cross-ripped with gypsum-treated topsoil along the contour of the slope to create ‘troughs and banks’ to further minimise the potential for erosion downslope and enhance vegetation establishment within the troughs.

The approved operations include the processing of mineralised material and would therefore remove the mineralised material stockpile as a component of the northern waste rock emplacement landform (dependent on market conditions). Should the mineralised material emplacement remain as a final landform, the rehabilitation objectives and concepts for the waste rock emplacements described in this section would be applied to the mineralised material emplacement and temporary ore stockpiles.

<u>Erosion and Sediment Control System</u>
The northern and southern waste rock emplacements have been designed to meet the long-term goal of containing potentially saline seepage generated from waste rock emplacement areas during operation and post-closure.

Drainage on the top surfaces of the waste rock emplacements would be managed via a series of small shallow basins (depressions) and would include a rehabilitation cover system that absorbs rainfall and comprises woodland vegetation (Evolution, 2018). The use of depressions would be aimed at maximising internal drainage without creating permanent ponding during normal and heavy rainfall events. A bund around the perimeter of the top surfaces of the waste rock emplacement would also be constructed to provide a contained catchment and minimise surface water runoff from the top surface down the batters.

(f) Lake Isolation System
As described in Section 3.1.2.2, the Lake Isolation System has been designed to hydrologically isolate and provide permanent mutual protection between the open pit and Lake Cowal during CGO development, mining and post-closure.

Temporary Isolation Bund
The temporary isolation bund was designed as a short-term feature that was used to isolate the pit from the lake during the construction phase while the lake protection bund was constructed. Accordingly, since the lake protection bund was constructed and revegetated, the isolation function of the temporary isolation bund has been superseded and breached in several locations to allow water flow (North Limited, 1998a). However, the bulk of the temporary isolation bund structure has been retained throughout operations due to the added protection provided from wave erosion and the provision of diverse constructed habitat features.

Lake Protection Bund
The lake protection bund forms part of the new lake foreshore (Figure 5).

Rehabilitation of the New Lake Foreshore will be an iterative process and revegetation species will continue to be selected in consideration of:

- Lake Cowal's hydrological regime (wetting and drying cycles);
- species occurring in relevant reference sites (including lake and slope woodland communities);
- species performance during revegetation trials; and
- suitability to substrate conditions.

Subject to these parameters, species may be selected from the following vegetative suites:

- fringing lake vegetation on foreshore batters (i.e. Eucalypt dominated woodland including River Red Gum [*Eucalyptus camaldulensis*], River Cooba [*Acacia stenophylla*], Wilga [*Geijera parviflora*], Kurrajong [*Brachychiton populneus*], Green Wattle [*Acacia deanei*] and Grey Box [*Eucalyptus microcarpa*]); and
- freshwater habitats (i.e. Foxtail [*Austrostipa densiflora*], Rush, Cane Grass [*Eragrostis australasica*] and Lignum).

Revegetation concepts and methods for the New Lake Foreshore are described in detail in the CWMP and RMP.

Erosion and sediment control for the lake protection bund will be affected by the temporary isolation bund. As described above, during the decommissioning phase, the temporary isolation bund will be reworked and breached once the lake protection bund is stabilised and revegetated (i.e. once stabilisation measures are effectively performing to prevent erosion and contain sediment).

Ongoing monitoring and maintenance is described in Section 3.4.

Perimeter Waste Rock Emplacement

The first batter of the perimeter waste rock emplacement is now part of the new lake foreshore (Figure 5). As stated earlier, any future redevelopment of the Lake Isolation System may or may not include a waste rock emplacement.

Erosion and sediment control for the perimeter waste rock emplacement is linked to the temporary isolation bund and lake protection bund. Consistent with the rehabilitation concepts for the outer batters of the waste rock emplacements and IWL, benign primary waste rock mulch has been incorporated into the rehabilitation cover system for the outer batters of the perimeter waste rock emplacement to provide long-term stability, control surface water runoff downslope and reduce erosion potential. The rock mulch has been cross-ripped with topsoil along the contour of the slope to create 'troughs and banks' to further minimise the potential for erosion downslope and enhance vegetation establishment within the troughs.

Ongoing monitoring and maintenance of the perimeter waste rock emplacement is described in Section 3.4.

Earthworks Associated with Landscaping

As described in Section 3.3.2 of this ESCMP, the earth mounds constructed for landscaping/screening purposes will remain as permanent features. Vegetative stabilisation will be generally undertaken in accordance with details provided in Chapter 7 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C). Rehabilitation will be undertaken progressively as described in Section 9 of this ESCMP.

Ongoing monitoring and maintenance is described in Section 3.4.

3.1.3.3 Monitoring and Maintenance

During any future construction phases, regular inspections of erosion and sediment control systems will be conducted by the Sustainability Manager (or delegate) to ensure the systems are performing as designed and that any revegetation areas have properly established. Site inspections will be undertaken by systematically walking around the construction areas to relevant areas and recording:

- condition of the erosion and sediment control structures;
- maintenance requirements (if necessary), including instructive actions;
- volume of sediment removed (e.g. from sediment basins to retain capacity requirements); and
- sediment disposal locations.

Stockpile areas will also be inspected and where the potential for excessive sediment migration (including dust generation) is identified, remedial/response measures will be enacted as described in the SSMP.

Inspections of sediment control structures and monitoring of water quality and inspections following rainfall events of 20 mm or more in a 24 hour period (within 48 hours of the event) will be conducted during the operational phase by the Sustainability Manager (or delegate). Rainfall events will be monitored as part of the meteorological monitoring programme and recorded in accordance with DA 14/98 condition 6.2. Sediment control structures will be inspected for capacity (i.e. sediment build-up consequently reducing capacity), structural integrity (i.e. stability) and effectiveness (sediment containment).

A comprehensive water quality monitoring programme (including total suspended solids, total dissolved solids (TDS), electrical conductivity (EC), major cations/anions and other key parameters) will be undertaken in accordance with the SWGMBMP and DA 14/98 condition 4.5(a) across the site. Data from monitoring of water quality (i.e. low total suspended solids [TSS] indicating settlement of the sediment is occurring) and inspections (sediment build-up) will be used to assess the design of sediment control structures and will be used to evaluate necessary ameliorative responses and maintenance activities (North Limited, 1998a).

Ameliorative responses and maintenance activities may include (from Chapter 8.2 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004)) (Appendix C):

- cleaning out of containment structures, diversion drains, etc. where sediment/sand/soil/vegetation build up;
- repairing of areas of erosion (e.g. lining with a suitable material which may include use of grasses, plastic, geotextile, rock, concrete);
- further application of seed/fertiliser in areas of minor soil erosion and/or inadequate vegetative establishment; and
- installation of additional erosion and sediment control structures.

Ameliorative responses and maintenance activities as described above will be conducted generally in accordance with details provided in Chapter 8 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C).

Maintenance of soil conservation works and revegetated areas is undertaken periodically as part of the rehabilitation programme (Sections 8 and 9 of this ESCMP). Monitoring of previously stabilised and revegetated areas associated with the lake isolation system is part of the rehabilitation programme. Ameliorative responses and maintenance activities as detailed above are carried out when problems are identified.

Water quality monitoring of sediment control structures (including TSS, EC and pH) occurs following overflow events (i.e. rainfall events greater than the prescribed design criteria – Section 3.2 of this ESCMP). Details of monitoring of the up-catchment diversions, contained water storages and temporary sediment storages are contained in the SWGMBMP. Generally, water quality monitoring is undertaken monthly in these storages and drains.

Monitoring results are entered into a database and reported in the Annual Review as described in the SWGMBMP.

3.2 SALINE BOREFIELD (ML 1535)

CGO's saline groundwater supply borefield is situated within ML 1535 and supplies low quality for CGO's process water needs. In accordance with DA 14/98 condition 4.2(a)(i) and (ii), the pipelines from the saline groundwater supply borefield to the site were constructed to the satisfaction of the Department of Primary Industries – Fisheries and the Office of Water; and specifically, laid in such a way as to not impede the passage of fish or other animals, interfere with flood behaviour or the passage of boats and vehicles.

As the pipeline is buried, the erosion and sediment control systems are limited to ongoing monitoring and maintenance activities described in Sections 3.2.1 and 3.2.2 below.

3.2.1 Monitoring and Control Systems

In accordance with DA 14/98 condition 4.2(a)(iii), the water supply pipeline was installed with an automatic shut-down device so water pumping is immediately stopped in the event of any pipe rupture. The water supply will not be restarted until the rupture is located and repaired. The saline groundwater supply borefield is only operated when Lake Cowal is dry and, in this way, negates the risk of any potential impacts on lake water quality (Gilbert and Associates, 2008).

3.2.2 Monitoring and Maintenance

Maintenance activities will be conducted generally in accordance with details provided in Chapter 8 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C).

Maintenance of revegetated lake bed areas will be undertaken periodically as part of the rehabilitation programme (Section 6).

3.3 BOREFIELD AND PIPELINES

CGO's four production bores in the Bland Creek Palaeochannel are located approximately 20 km to the east north-east of the CGO site (Figure 6).

The borefield reticulation system includes (North Limited, 1998a):

- a break pressure/balancing storage after the final bore; and
- a buried 600 mm (approximately) diameter pipeline to the CGO.

The Eastern Saline Borefield is located approximately 10 km east of Lake Cowal's eastern shoreline, and north-east of the Bland Creek Palaeochannel borefield (Figure 6). It has five production bores.

In general, borefields operate in drier times and are rested in wetter times when the site water supply would make-up the supply from this source. Water from the Eastern Saline Borefield is pumped via the Bland Creek Palaeochannel borefield water supply pipelines to the CGO.

As the existing pipeline is buried, the erosion and sediment control systems are limited to ongoing monitoring and maintenance activities during lake dry conditions.

The water supply pipeline from the Bland Creek Palaeochannel Borefield (up to Bore 4) has been duplicated and has similar surveillance and control systems as the original. Construction of the new pipeline and rehabilitation of disturbance areas within the existing pipeline corridor was undertaken in consideration of the DoI L&W's *Controlled Activities on Waterfront Land - Guidelines for Laying of Pipes and Cables in Watercourses on Waterfront Land*, and in accordance with the CGO's RMP, SSMP and WMP.

Decommissioning of the water supply pipelines will be undertaken in accordance with the CGO's Strategy for the Decommissioning of Water Management Structures and Long-term Management of Final Void and Lake Protection Bund described in the WMP.

3.4 MONITORING AND MAINTENANCE

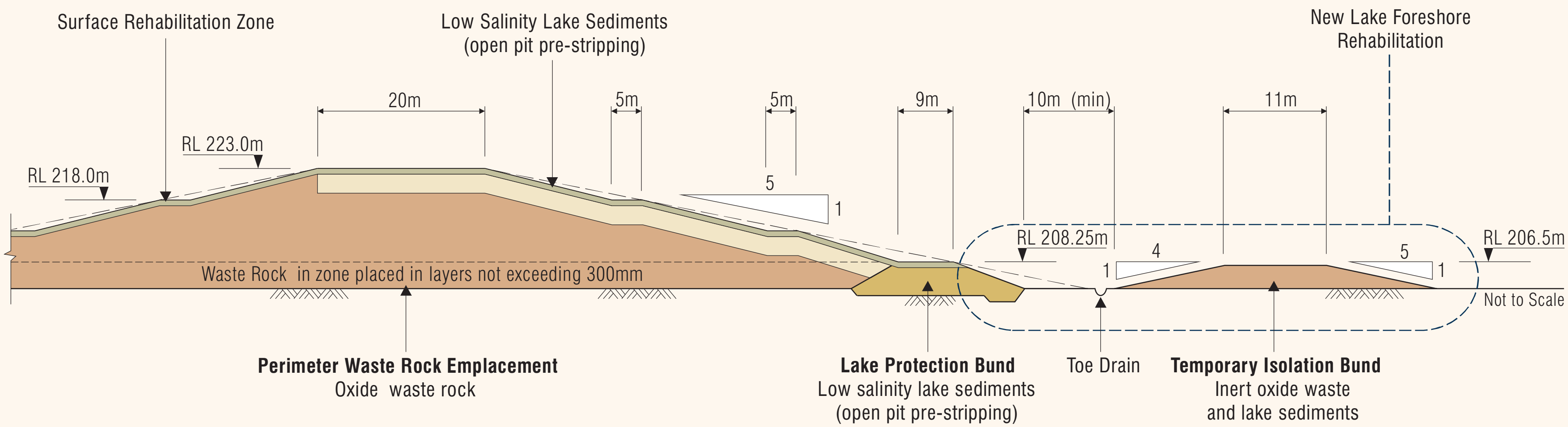
Once construction of the new pipeline was completed, inspections of the pipeline corridor were undertaken by the Sustainability Manager (or delegate) as required to ensure revegetation areas along the pipeline corridor have properly established. Inspections are undertaken systematically to record:

- condition of any temporary erosion and sediment control structures temporarily retained after construction; and
- maintenance requirements (if necessary) including instructive actions or supplementary seeding.

Ameliorative measures for the borefields and pipelines may include (from Chapter 8.2 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C):

- cleaning out of containment structures, diversion drains, etc. where sediment/sand/soil/vegetation builds up;
- repairing of areas of erosion (e.g. lining with a suitable material which may include use of grasses, plastic, geotextile, rock, concrete);
- further application of seed/fertiliser in areas of minor soil erosion and/or inadequate vegetative establishment; and
- installation of additional erosion and sediment control structures.

Maintenance activities as described above are conducted generally in accordance with details provided in Chapter 8 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C). Maintenance of revegetated areas is undertaken periodically as part of the rehabilitation programme (Section 6).



EROSION AND SEDIMENT CONTROL MANAGEMENT PLAN

FIGURE 6

Lake Isolation System
Construction Details

Evolution Mining Cowal

HAL-02-07 CGO MP 2017_ESC MP_002A

4 SALINITY MANAGEMENT

Many saline areas in NSW (e.g. inland salt pans, brackish streams, coastal salt marshes and naturally saline soils) are the result of entirely natural processes. Most newly salinised areas however, are often the result of particular land use practices, such as over-clearing, river regulation, irrigation or the cultivation of crops (DLWC, 2003b).

Salinity is often divided into several different categories including:

(a) Dryland Salinity;

(b) Irrigation Salinity;

(c) Urban Salinity;

(d) River Salinity; and

(e) Industrial Salinity.

Irrigation salinity (i.e. due to over-irrigation of farmland, inefficient water usage and poor drainage) and urban salinity (i.e. due to dryland salinity processes and over-watering of towns and urban areas) are not relevant to the CGO.

The three different types of salinity relevant to the CGO (i.e. dryland, river and industrial salinity) are described in the following subsections and include details of salinity management measures, where relevant. These measures are provided to meet the objectives relating to salinity management of the relevant local and regional management plans (e.g. *Floodplain* Management Plan Steering Committee, 2000], *Lake Cowal Land and Water Management Plan* [Australian Water Technologies Pty Ltd, 1999], the *Bland Creek Catchment Plan* [Bland Creek Catchment Committee, 2002] and *Lachlan Catchment Action Plan (2013-2023)* [Lachlan Catchment Management Authority, 2013]) described in Section 2.5.

4.1 DRYLAND SALINITY

Dryland salinity is the build-up of salt in the soil, usually as a result of a rising water table. Evaporation of saline water at the soil surface tends to concentrate salts to the point where they affect the environment (DLWC, 2003b).

In rural areas, dryland salinity associated with rising water tables is often caused by the loss of deep-rooted perennial trees, shrubs and grasses. The loss of vegetation is often caused by factors such as clearing, soil acidity, overgrazing or erosion (DLWC, 2003b).

The Lake Cowal Foundation (2013) states that Lake Cowal is under threat by dryland salinity induced by poor catchment management practices. The lake (outside of ML 1535) is subject to grazing and cropping below the lake full storage line when the lake recedes in dryer times (Lake Cowal Foundation, 2013). No grazing of areas occurs within ML 1535 and ML 1791.

Measures that have been adopted to manage the factors that affect dryland salinity as discussed above include:

- *Minimising the areas disturbed by the CGO components and restricting access to non-disturbed areas*
  Clearing of areas for the CGO components during the construction and operational phases will be limited and clearly delineated, where appropriate, with barrier mesh (upslope) and sediment fencing (downslope) or similar materials in accordance with details provided in Chapter 4.2 of *Managing Urban Stormwater – Soils and Construction* (Landcom, 2004) (Appendix C). Unrestricted vehicular plant access to undisturbed areas will not be permitted. Vegetation in close proximity to works areas will be demarcated with flagging tape (or similar) so as to prevent disturbance. All employees

undertaking the site induction/training programme will be made aware of the importance of remaining within the defined works areas.

- *Identification of saline soils (infill testing) and selective soil resource management*
  As described in the SSMP and summarised in Section 8.1 of this ESCMP, infill testing of soil profiles will be undertaken to confirm the precise depths of soil and any requirements for amelioration at the time of soil stripping. Where possible, the more saline and dispersive soil horizons are left *in-situ* beneath mine landforms (North Limited, 1998a). Topsoil and subsoil resources are stripped and stored in separate stockpiles comprising topsoil; low salinity subsoil; gypsum treated subsoil; and native seed topsoil. Stripped non-saline soils are stored in dedicated stockpile areas for re-use during construction and progressive rehabilitation works. Low salinity topsoils/soils are selectively placed on the outer face of the waste rock emplacement. Stripped soils not suitable for construction or rehabilitation are placed in the waste rock emplacements (North Limited, 1998a) or ameliorated as described within the SSMP.

- *Identification of low salinity construction material (construction fill testing) and selective resource management*
  Fill used for construction is tested for geochemical and geotechnical suitability prior to construction commencing. Suitable fill will be sufficiently impermeable, low dispersion, low salinity and not acid forming. Where possible, the more saline and dispersive soil horizons are left *in-situ* beneath mine landforms (North Limited, 1998a). The lake protection bund has been constructed with low salinity lake sediments.

- *Fencing ML 1535 and ML 1791 to restrict stock and prevent overgrazing and erosion*
  A fence has been constructed around the perimeter of ML 1535 and ML 1791 (except for the portion of the shared boundary) . The fence is currently inspected on a weekly basis for integrity and pest livestock by the Sustainability Manager (or delegate).

- *Implementation of appropriate erosion and sediment control systems and ongoing monitoring and maintenance*
  Erosion and sediment control systems as described in Sections 3, 4, 5 and 6 will be implemented to minimise erosion and the loss of vegetation and hence minimise salinity. The monitoring and maintenance activities for erosion and sediment controls described in Sections 3.4 is also undertaken during the life of the CGO in order to manage salinity related issues. Specifically, water quality monitoring of water contained in sediment control structures (i.e. suspended solids, electrical conductivity and pH) will occur following overflow events (i.e. rainfall events greater than the prescribed design criteria – Section 3.1.1). Details of monitoring of the up-catchment diversions, contained water storages and temporary storages are contained in the SWGMBMP. Generally, water quality monitoring is undertaken monthly in these storages and drains.

4.2 RIVER SALINITY

Surface water runoff from areas of dryland, irrigation and urban salinity may flow into creeks and rivers, raising their salinity. According to the Murray-Darling Basin Commission Salinity Audit, salinity is likely to rise to high levels in future in the Bogan, Castlereagh, Lachlan, and Macquarie and Namoi Rivers (DLWC, 2003b).

Measures adopted for the CGO to manage river salinity include:

- *Containment and management of saline surface water runoff*
  Soil and waste rock characterisation programmes have identified materials to be disturbed during the construction and development of CGO that have the potential to generate salinity (CALM, 1994; Resource Strategies, 1997; EGi, 1997). Accordingly, the CGO water management strategy incorporates design elements to contain surface runoff or seepage likely to contain increased salt concentrations (Gilbert and Sutherland, 1997). This includes some waste rock and subsoil

materials that have moderately to highly saline runoff and seepage that has the potential to retard revegetation and affect local water resources if not controlled. The CGO's water management systems therefore incorporates interception drains (toe drains) and collection storages (temporary sediment basins and contained water storages) around all stockpile areas, waste rock emplacements, tailings emplacements and structures containing saline materials. Water that accumulates in these storages is captured for reuse in the process water supply. Furthermore, contained water storages D1 and D4 are fitted with pumps capable of transferring the first flush of initial captured runoff waters (typically saline and containing sediment) from the outer batters of the northern and southern waste rock emplacements and IWL to the designated process water contained water storages.

- *Open pit/final void salinity sink*
  During CGO operations, water will accumulate within the open pit as a result of controlled drainage of surface water runoff during wet weather and groundwater inflows (Evolution, 2018). This water is managed in accordance with a pit dewatering programme described in the WMP. A large proportion of runoff is directed to the final void will intercept, preventing sediment from entering Lake Cowal. As a result, salt loads entering the lake from the CGO are expected to marginally decrease post-mining as salt will also be trapped by the void (North Limited, 1998b).

  In accordance with DA 14/98 condition 4.4(b), a strategy for the long-term management of the final void has been developed in consultation with the DoI L&W, EPA, Division of Resources and Geoscience and the Community Environmental Monitoring and Consultative Committee (CEMCC), and to the satisfaction of the Planning Secretary. The strategy is described in the WMP and includes long-term monitoring of the water quality in the final void and the stability of the void walls.

4.3 INDUSTRIAL SALINITY

Industrial salinity is often related to industrial processes, whereby salt is concentrated in the water used over time. Mine workings often need to manage saline water from groundwater seepage and from rainwater coming into contact with saline mined materials. Abandoned mines are also a major source of salinity in some areas (DLWC, 2003ab).

Measures to be adopted for the CGO to manage industrial salinity include:

- *Isolation of saline groundwaters by the open pit.*
  Isolation of groundwaters within the Internal Catchment Drainage System (Section 3.1.2.2) will be achieved by virtue of the permanent groundwater sink that will be formed as a result of open pit dewatering (Gilbert and Sutherland, 1997). Waters that accumulate within the open pit will be managed in accordance with a pit dewatering programme described in the WMP. The effect of the open pit will be to locally depress the groundwater table (potentiometric surface) such that all groundwater movement in the surrounding area will be towards the void (Gilbert and Sutherland, 1997). This effect will also reduce salinity associated with the rising groundwaters which is the greatest long-term threat to the values of the lake and the surrounding area (Williams, 1993 and ERIC, 1996 in Australian Water Technologies Pty Ltd, 1999). More recent groundwater modelling by Coffey (Coffey 2020a,b) predicts no overspill and no connection between the lake and the mine void.

- *Containment of potentially saline seepage generated from waste rock emplacement and IWL areas*
  The northern and southern waste rock emplacements have been designed to meet the long-term goal of containing potentially saline seepage generated from waste rock emplacement areas during operation and post-closure. Construction will involve surface preparation works to facilitate the direction of any permeating waters towards the open pit. The existing topography of the footprint has been altered by stripping topsoil and subsoil from the waste rock emplacement footprints. In addition, oxide waste rock has been placed and compacted (using haul truck movements) across

the footprints. The resulting basement for the emplacements slopes towards the open pit to provide drainage control. Any waters permeating through the emplacement are expected to be intercepted by this layer and preferentially flow towards the open pit (North Limited, 1998a).

Similar to the existing tailings storage facilities, the IWL will be constructed with a low permeability basement layer to control seepage. There are a series of interceptor drains and fingers under the rock embankment designed to capture stormwater that permeate through the rock embankment. This water is directed to an external sump and sent back to the mill for reuse. This water is also tested periodically for contaminants. A detailed description of the IWL design is provided in the WMP.

- *Containment and management of saline surface water runoff*
  As described above to manage river salinity, the CGO's water management strategy incorporates interception drains (toe drains) and collection storages (temporary sediment basins and contained water storages) around all stockpile areas, waste rock emplacements and IWL to contain saline materials.

- *Final void management and monitoring*
  As described above to manage river salinity, a strategy for the long-term management of the final void has been developed and will include long-term monitoring of the water quality in the final void.

5 SOIL MANAGEMENT

The principal strategy in soil resource management at the CGO is to strip suitable soil resources from all proposed disturbance areas within ML 1535 and ML 1791 and directly replace on rehabilitation areas or store in dedicated stockpiles for re-use during progressive rehabilitation works.

The strategies/objectives for management of the CGO soil resources include (Evolution, 2018):

- characterisation of the suitability of the material for rehabilitation purposes prior to stripping;
- soil resources are stripped, stored selectively and managed according to their suitability for rehabilitation purposes;
- sufficient subsoil and stable topsoil are available for rehabilitation purposes;
- progressive rehabilitation of final landforms is conducted as soon as practicable after completion of the landforms or when areas are no longer required; and
- soil resources are stripped and stored in such a manner that their long-term viability is maintained.

5.1 SOIL STRIPPING SCHEDULING

Disturbance areas are stripped progressively, to reduce potential erosion and sediment generation, and to minimise the extent of topsoil stockpiles and the period of soil storage.

Prior to soil stripping, testing of soil profiles is undertaken to confirm the precise depths of suitable soil and any requirements for amelioration at the time of soil stockpiling. Stakes may be used to delineate soil boundaries and to identify suitable stripping depths for equipment operators.

Soil stripping scheduling and details regarding the volume of topsoil proposed to be stripped will continue to be provided in the MOP.

The SSMP provides further detail.

5.2 SOIL STRIPPING PRACTICES

5.2.1 Pre-Stripping Activities

Cultural Heritage Inspection

Soil stripping activities within ML 1535 and ML 1791 (including preliminary soil testing) are managed to comply with the existing requirements of the NPWS Section 87 permits and Section 90 consents granted under the *National Parks and Wildlife Act, 1974* (NPW Act). These permits and consents allow both the collection of visible artefacts prior to soil stripping and also the collection of unknown artefacts that may be contained within the soil profile. In accordance with Condition 11 of the Section 87 permit, "*all areas where soil stripping occurs shall be further inspected following this operation in the event that datable materials might be revealed*".

An Aboriginal Heritage Impact Permit (AHIP) application for the ML 1791 area has been lodged with the NSW Office for Environment and Heritage under Section 90A of the NPW Act. Once the AHIP has been approved, all disturbance activities within ML 1791 will be undertaken in accordance with requirements of the AHIP.

A comprehensive Ground Disturbance Permit process is in place at the CGO. All land disturbance activities will only take place in approved surface disturbance areas. The permit clearly defines the location and nature of the earthworks activity, with steps required to enable ground survey by Wiradjuri monitors and/or an archaeologist. There are two steps to the process:

- A surface cultural heritage clearance survey, which is designed to inspect the relevant land and identify surface objects from which a representative sample would be collected.

- A sub-surface cultural heritage clearance survey, which allows for inspection once the grass layer has been removed. Typically at the CGO, this is accomplished by grading several centimetres of topsoil to enable identification of objects and other items (kept with the soil) for collection and storage.

The rationale for this is based on the original archaeological assessments, where it was deemed necessary to verify that no sites or features might be unwittingly destroyed. Standard test-pit excavation by hand was not considered a feasible strategy to deliver this certainty for Wiradjuri.

In accordance with DA 14/98 condition 3.2(c), a Vegetation Clearance Protocol and Threatened Species Management Protocol have been developed for the CGO and are detailed in the Flora and Fauna Management Plan. The clearance protocols outline measures to be undertaken prior to clearing of vegetation within the CGO. Measures include:

- pre-clearance survey for flora, including a targeted survey for any threatened species recorded in the mine site area; and
- preliminary and secondary fauna habitat assessments.

Prior to undertaking soil stripping, the Vegetation Clearance Protocol and Threatened Species Management Protocol will be employed.

5.2.2 Stripping Activities

As required by DA 14/98 condition 3.2(a)(i), the removal of trees and other vegetation from the mine site is restricted to the approved disturbance areas only. Once cleared of woody vegetation, soils are typically stripped using a grader, scraper or bulldozer. Scrapers may be used to strip soils where areas become too large for effective dozer or grader stripping.

Soil is stripped by firstly removing grasses. This is followed by the separate removal of topsoil and subsoil, where required. Stripped soils will be either directly replaced on rehabilitation areas or stored in separate topsoil and subsoil stockpiles where practicable. Where rehandling is necessary, this will typically be undertaken using excavators and dump trucks.

The SSMP provides further detail.

5.3 SOIL STOCKPILE MANAGEMENT

In accordance with DA 14/98 condition 3.5(b)(i), the general protocol for the management of soil stockpiles is provided below. It includes soil handling measures that optimise the retention of soil characteristics (in terms of nutrients and micro-organisms) favourable to plant growth:

- leave the surface of the completed soil stockpiles in a "rough" condition to help promote water infiltration and minimise erosion prior to vegetation establishment;
- deep-rip soil stockpiles and seed (if necessary) to maintain soil organic matter levels, soil structure and microbial activity;
- treat soil stockpiles with gypsum to reduce dispersiveness during stockpiling;
- install signposts for all soil stockpiles with the date of construction and type of soil; and
- record details of all soil stockpiles on a site database which includes the location and volume of each stockpile and the stockpile maintenance records (e.g. ameliorative treatment, weed control, seeding) (Section 5.3).

Where practicable, soil is stripped from one area and immediately transferred to an active rehabilitation area for direct placement. This reduces the size of soil stockpiles and optimise soil fertility for rehabilitation (Section 6).

Following construction, and if adequate unassisted revegetation has not occurred, soil stockpiles are sown with suitable annual or select grass and legume species to maintain soil condition for future revegetation/rehabilitation works to minimise erosion and wind-blown dust and discourage opportunistic weed growth. Soil treatment/amelioration methods undertaken as necessary prior to the use of soil for rehabilitation.

6 REHABILITATION

The approved CGO rehabilitation philosophy is to operate as a non-intrusive land user and to create stable rehabilitated landforms that increase the areas of endemic vegetation in the mine area and the status of land-lake habitats (Evolution, 2018). This philosophy has led to the rehabilitation principles and objectives as described below.

The rehabilitation programme will be undertaken in accordance with the following general principles (Evolution, 2018):

- The rehabilitation of landforms is to be progressive (where possible) and conducted in accordance with approved, verified plans.
- Final landforms are to be stable in the long-term and include native and/or endemic vegetation characteristic of remnant vegetation within the surrounding landscape.
- Native and/or endemic groundcover, understorey, and tree species are to be cultivated and used in the rehabilitation programme.
- Rehabilitation concepts are to be flexible to allow for adjustments, based on investigations, to improve the rehabilitation programme.
- The annual rehabilitation programme and budget is to be prepared by a site team incorporating senior management representatives.

The rehabilitation objectives for the CGO include (Evolution, 2018):

- The water quality of Lake Cowal is not detrimentally affected by the new landforms.
- Revegetating the new landforms with selected native and/or endemic vegetation that is suited to the physiographic and hydrological features of each landform, and which expand on the areas of remnant endemic vegetation in the surrounding landscape.
- Designing final landforms so that they are stable and include revegetation growth materials that are suited to the landform and support self-sustaining vegetation.
- The placement (wherever possible) of soils on final landforms to enable the progressive establishment of vegetation.
- The expansion of habitat opportunities for wetland and terrestrial fauna species. This includes the design and implementation of rehabilitation works at the New Lake Foreshore in a manner consistent with the NSW Wetlands Policy (Department of Environment, Climate Change and Water, 2010).
- The selection of revegetation species in accordance with accepted principles of long-term sustainability (e.g. genotypic variation, vegetation succession, water/drought tolerances).
- Grazing of land within ML 1535 and ML 1791 to be excluded during operations and during rehabilitation of the site. At lease relinquishment, rehabilitated final landforms are conserved (with grazing excluded), with some areas suitable for grazing surrounding the rehabilitated final landforms.

As described in Section 3.1.2.2 and consistent with the approved CGO Rehabilitation Proposal described in the *Cowal Gold Operations Underground Development Project EIS* (Evolution, 2020), rehabilitation concepts for the outer batters of the waste rock emplacements, IWL and the lake protection bund will include rock armouring with primary waste rock mulch to provide long-term slope stability, control surface water runoff downslope and reduce erosion potential.

Rehabilitation concepts and measures are described in detail in the CGO's RMP. Progressive rehabilitation works and proposed soil stripping works and areas are detailed and progressively updated in the MOP as required by the Conditions of Authority for ML 1535. As described in Section 2.2 of this ESCMP, a MOP is prepared to reflect the near-term development plans at the CGO, in accordance with the requirements of the DRG's *ESG3: Mining Operations Plan (MOP) Guidelines*.

7 COMMUNITY CONSULTATION

7.1 COMMUNITY ENVIRONMENTAL MONITORING AND CONSULTATIVE COMMITTEE

A CEMCC has been set up for the CGO in accordance with DA 14/98 condition 9.1(d) and SSD 10367 condition A11. Condition 9.1(d) is reproduced below:

9.1 Environmental Management

(d) Community Environmental Monitoring and Consultative Committee

(i) The Applicant shall establish and operate a Community Environmental Monitoring and Consultative Committee (CEMCC) for the Coal Gold Operations to the satisfaction of the Planning Secretary. This CEMCC must:

- *be comprised of an independent chair and at least 2 representatives of the Applicant, 1 representative of BSC, 1 representative of the Lake Cowal Environmental Trust (but not a Trust representative of the Applicant), 4 community representatives (including one member of the Lake Cowal Landholders Association);*
- *be operated in general accordance with the Department's Community Consultative Committee Guidelines: State Significant Projects (2019 or its latest version); and*
- *monitor compliance with conditions of this consent and other matters relevant to the operation of the Cowal Gold Operations during the term of the consent.*

Note: The CEMCC is an advisory committee. The Department and other relevant agencies are responsible for ensuring that the Applicant complies with this consent.

(ii) The Applicant shall establish a trust fund to be managed by the Chair of the CEMCC to facilitate the functioning of the CEMCC, and pay $2000 per annum to the fund for the duration of gold processing operations. The annual payment shall be indexed according to the Consumer Price Index (CPI) at the time of payment. The first payment shall be made by the date of the first Committee meeting. The Applicant shall also contribute to the Trust Fund reasonable funds for payment of the independent Chairperson, to the satisfaction of the Planning Secretary

As required by DA 14/98 condition 9.1(d)(i), the CEMCC comprises an independent chair and representatives of the BSC, Forbes Shire Council, Lachlan Shire Council, Lake Cowal Foundation, the Wiradjuri Condobolin Corporation, two Evolution representatives and four community representatives including one from the Lake Cowal Landholders Association.

The CEMCC provides opportunities for members of the community to attend CEMCC meetings to discuss specific issues relevant to them, including any concerns relating to erosion and sediment control. A landholder can make a request to the CEMCC regarding a particular issue, or the landowner can register a complaint in the complaints register. Landowners who register complaints may be invited to join in discussion of the issue at the next CEMCC meeting.

Items of discussion at these meetings will include mine progress, reporting on environmental monitoring, complaints, rehabilitation activities and any environmental assessments undertaken.

The CEMCC meets quarterly and the minutes from CEMCC meetings are provided on Evolution's website (www.evolutionmining.com.au).

7.2 COMPLAINTS REGISTER AND RECORDS

A process for the handling of complaints is provided below in accordance with the requirements of CGO's EPL and DA 14/98 conditions and to facilitate prompt and comprehensive responses to any community concerns.

As required by EPL Condition M6.1, a dedicated Community Complaints Line has been established (via phone [02] 6975 3454 or email community.cowal@evolutionmining.com.au) that is available 24 hours, seven days a week for community members who have enquiries or who wish to lodge complaints in relation to Evolution's activities at the CGO. A complaints register will be maintained by the Sustainability Manager (or relevant equivalent) in accordance with EPL condition M5 and will be made available on Evolution's website in accordance with DA 14/98 condition 9.4(a)(v).

Information recorded in the complaints register with respect to each complaint will include:

- the date and time of the complaint;
- the method by which the complaint was made;
- any personal details of the complainant which were provided by the complainant or, if no such details were provided, a note to that effect;
- the nature of the complaint;
- the action taken by Evolution in relation to the complaint, including any follow-up contact with the complainant; and
- if no action was taken by Evolution, the reasons why no action was taken.

The record of a complaint will be kept for at least four years after the complaint was made. The record will be available for inspection by the EPA.

8 AUDITING AND REVIEW

8.1 EXTERNAL AUDITS

8.1.1 Independent Environmental Audit

An Independent Environmental Audit will be conducted in accordance with DA 14/98 condition 9.2(a) and SSD 10367 condition C11. Condition 9.2(a) is reproduced below:

9.2 Independent Auditing and Review

(a) Independent Environmental Audit

(i) By the end of July 2016, and every 3 years thereafter, unless the Secretary directs otherwise, the Applicant shall commission and pay the full cost of an Independent Environmental Audit of the development. This audit must:

- *be prepared in accordance with the Independent Audit Post Approval Requirements (2020 or as amended from time to time);*
- *be led and conducted by a suitably qualified, experienced and independent team of experts (including ecology and rehabilitation experts, and in field's specified by the Planning Secretary) whose appointment has been endorsed by the Secretary;*
- *be carried out in consultation with the relevant agencies, BSC and the CEMCC;*
- *assess whether the development complies with the relevant requirements in this consent, and any strategy, plan or program required under this consent; and*
- *recommend appropriate measures or actions to improve the environmental performance*

This process provides a mechanism by which environmental management and monitoring at the CGO can be assessed against relevant Development Consent, mining lease and licence conditions.

8.2 REVIEW OF THIS ESCMP

In accordance with condition 9.1(c) of DA 14/98, this ESCMP will be reviewed within three months of:

- the submission of an Annual Review under Condition 9.1(b);
- the submission of an non-compliance or incident notification under Condition 9.3(a) or 9.3(b);
- the submission of an audit under Condition 9.2(a);
- the approval of any modification to the conditions of this consent; or
- any direction of the Planning Secretary under Condition 1.1(b) of this consent.

Where this review leads to revisions of this plan, then within six weeks of the review, the revised ESCMP will be submitted for the approval of the Planning Secretary of the DPE (unless otherwise agreed with the Planning Secretary). The revision status of this ESCMP is indicated after the title page of this ESCMP.

This ESCMP will be made publicly available on Evolution's website (www.evolutionmining.com.au) in accordance with DA 14/98 condition 9.4(a)(iii). A hard copy of the ESCMP will also be kept at the CGO.

9 REPORTING

9.1 EROSION AND SEDIMENT CONTROL SYSTEMS REPORTING

In accordance with DA 14/98 condition 3.5(a)(iii), the effectiveness of the erosion and sediment control systems and the performance of those systems are reported against the objectives contained in the ESCMP to:

- control of the movement of sediment and salinity migration from areas disturbed by mining and construction activities; and
- maintain downstream (lake) water quality.

The programme for reporting on the effectiveness and performance of the erosion and sediment control systems includes:

- Maintaining a site erosion, sediment and salinity database recording the condition of erosion and sediment control systems, maintenance requirements (where maintenance has been conducted) including instructive actions, and how/when the instructive actions had been implemented. The database will be maintained by the environmental department.
- Ongoing monitoring and review of water quality results from the SWGMBMP (i.e. total suspended solids).
- Reporting of the site erosion, sediment and salinity performance and water quality monitoring results in the Annual Review.

The programme enables the assessment of the effectiveness and performance of the erosion and sediment control systems against the objectives contained in the ESCMP by:

- recording the erosion and sediment control systems to control of the movement of sediment and salinity migration from areas disturbed by mining and construction activities; and
- monitoring the downstream (lake) water quality (as part of the SWGMBMP).

9.2 ANNUAL REVIEW

In accordance with condition 9.1(b) of DA 14/98 and condition C9 of SSD 10367, Evolution prepares an Annual Review to report on the environmental performance of the CGO by the end of March each year, or other timing as may be agreed by the Secretary of the DPE. The Annual Review is made publicly available on Evolution's website (www.evolutionmining.com.au) in accordance with DA 14/98 condition 9.4(a)(vii). The Annual Review also addresses the Annual Review requirements of ML 1535 Condition of Authority 26.

9.3 INCIDENT NOTIFICATION AND REPORTING

An incident is defined in DA 14/98 as:

> *A set of circumstances that causes or threatens to cause material harm to the environment.*

SSD 10367 uses a similar, albeit slightly different, definition.

In accordance with DA 14/98 condition 9.3(a) and SSD 10367 condition C7, Evolution will notify the Planning Secretary in writing via the Major Projects website (https://pp.planningportal.nsw.gov.au/major-projects) immediately after becoming aware of any incident related to the CGO. Evolution will provide the DPE with a detailed report on the incident in accordance with the conditions.

In addition, in accordance with EPL 11912 Condition R2, Evolution will notify the EPA (and all other relevant authorities) of incidents causing or threatening material harm to the environment immediately after becoming aware of the incident. Evolution will provide written details of the notification to the EPA within seven days of the date on which the incident occurred. Evolution maintains a record of/and report on any cyanide-related incidents.

9.4 NON-COMPLIANCE NOTIFICATION AND REPORTING

A non-compliance is defined within DA 14/98 as:

> *An occurrence, set of circumstances, or development which is a breach of this consent but is not an incident.*

Again, SSD 10367 has a similar, albeit slightly different, definition for a non-compliance.

In accordance with DA 14/98 condition 9.5(b) and SSD 10367 condition C8, Evolution will notify the DPE in writing via the Major Projects website (https://pp.planningportal.nsw.gov.au/major-projects) within seven days after becoming aware of any non-compliance. Evolution will provide in writing a detailed report of the non-compliance which identifies, the development application number for the CGO, the development consent condition of which the CGO is non-compliant, the way in which the CGO does not comply and the reason for the non-compliance. Evolution will also provide details around any actions which have been or will be taken to address the non-compliance.

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APPENDIX A

DESIGN MANUAL FOR SOIL CONSERVATION WORKS TECHNICAL HANDBOOK NO. 5 (Soil Conservation Service of NSW, 1982)

SOIL CONSERVATION SERVICE OF N.S.W.

DESIGN MANUAL FOR SOIL CONSERVATION WORKS

TECHNICAL HANDBOOK NO. 5

Edited by

J.M. Aveyard.

Design Manual For Soil Conservation Works
Technical Handbook No. 5

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FIGURES:

PREFACE

Soil conservation involves the formulation of land use and land management strategies to protect the soil against the erosive forces of wind and water.

This soil protection may involve the planting of windbreaks to reduce the erosive force of the wind, the improvement of pastures and the adoption of specific grazing management practices to maintain vegetative ground cover. In urban and mining areas the revegetation, rehabilitation and stabilisation of denuded and disturbed soil is involved. In agricultural areas the use of pastures in rotation with crops provides indirect protection for the soil by maintaining soil fertility and structural stability. The adoption of crop management practices such as those which involve cultivation on the contour, retention of crop residues or direct sowing into previously uncultivated land also provides essential soil protection.

More direct soil protection is provided by soil conservation structures such as dams, banks, channels or special works to divert and control potentially erosive runoff. The layout of crops and pastures in strips generally across the direction of water flow, on low sloping land, is also an effective runoff control measure.

This manual deals with the design of soil conservation structures commonly used in New South Wales. It has been prepared as a tool for field soil conservationists to enable them to design structures to meet most needs.

For each of the design problems commonly encountered by soil conservationists the manual provides sufficient technical theory to allow a general understanding of, and confidence in, the design procedures. Graphs, formulae, tables and worked examples should ensure that any officer can confidently design structures that are adequate for their purpose.

The manual deals only with the design of structures. It does not contain information on survey, layout or the construction of works. Where applicable it has been written so as to augment the use of the <u>Design Sheets</u> currently in use throughout the Service.

However, it is not intended or expected that the manual will be adequate in all situations. In complex or high risk circumstances, individual officers should contact the appropriate Research Service Centre, Regional Soil Conservationist or through their regional office a Service Engineer, for advice and help.

The manual has been written for general use mainly in the Eastern and Central divisions of the state. Because of this and because of the meagre rainfall and runoff records over much of inland New South Wales a number of generalised assumptions have been necessary in the design procedures. Consequently, it is strongly recommended that soil conservationists make maximum use of their own field observations to assess local variations in rainfall intensity, runoff, the performance of existing works and the occurence of soil erosion to augment the data given.

The text has been compiled by a group of soil conservationists and is based on the draft design manual for soil conservation works published in February, 1982. The names of the group and their main involvement are listed below;

J. M. Aveyard - Convenor, Editor, Chapters 2 and 3

R. P. Simpson - Chapters 1, 2 and 4.

L. M. Jackson - Chapters 2, 5 and 6.

K. E. McPhee - Chapter 2.

R. D. Lang - Chapter 2.

In addition, acknowledgement is made to the many other soil conservationists who prepared formal comment on the 1982 draft. In particular the help of Mr. R. J. Crouch in the preparation of Chapter 5 and Mr. S. Nastasi in the preparation of Chapters 1, 2, and 4 is acknowledged.

The manual has been prepared in a loose leafed format with each chapter designated by a specific tab. Within chapters each section has been written as an independent unit and the figures, tables and references which apply to it have been grouped, on separate sheets, at the end of the section.

CHAPTER 1 - RAINFALL AND RUNOFF

Section 1 - Rainfall

Section 2 - Runoff

Section 1 - Rainfall

Section 1 - Rainfall

1.1 Introduction to Rainfall Estimation

To design a structure for runoff control, it is necessary to determine design parameters for the structure. Two of the parameters required for the design of soil conservation structures are the maximum rate of, or the volume of runoff which the structure must successfully convey.

Estimates of surface runoff depend upon a knowledge of two things.

1. An estimate of the rate of rainfall.

2. An estimate of how much of this rainfall becomes runoff.

This section is primarily concerned with the first process, estimating the rate of rainfall.

The rate of rainfall is usually referred to as the rainfall intensity and is generally expressed in millimetres of rain per hour (mm/h). The length of a storm is called its duration, and the relationship between average rainfall intensity and duration has been found to be of the form indicated in figure 1.1.1.

The average frequency of occurence of a specific rainfall event is expressed as the annual exceedance probability (A.E.P). This is the probability that a given amount (or depth) of rainfall per unit time will be reached or exceeded each year. The A.E.P is expressed as a percentage.

In this section rainfall estimation is based on intensity-frequency duration curves. Four base maps of isopleths of rainfall intensity for durations of 12 hours and 72 hours and 50% A.E.P and 2% A.E.P for New South Wales are given in figures 1.1.2 -1.1.5. These figures are used to determine the four key rainfall intensities for a particular location which, together with the zone constant and interpolation methods, uniquely determine a full set of intensity-frequency duration curves.

1.2 Intensity-Frequency-Duration Estimation

The method of intensity-frequency-duration estimation presented in this section is based on the method presented in Australian Rainfall and Runoff (Inst. Aust. Eng., 1977).

The steps in the method are:

1) Determine the required storm duration and annual exceedance probability (A.E.P). The storm duration will be determined by the method of flow estimation chosen. For example, storm duration will equal time of concentration for the Rational

Method. (see section 2.2.3 of this chapter), for A.E.P. it will be determined by the type of structure, the intended life of the structure and the consequence of failure.

2) If the required storm duration is greater than 12 hours and the A.E.P is between but not equal to 2% or 50%, use the maps in figures 1.1.2 - 1.1.5 to estimate the 12 hour, 2% storm; the 12 hour, 50% storm; the 72 hour 2% storm; and the 72 hour 50% storm. Estimates for centres between isopleths can be interpreted by linear interpolation.

3) Plot the four values on figure 1.1.6. Note that in figure 1.1.6 the rainfall intensity axis is a linear scale and the designer can select rainfall intensity values appropriate to the problem. Join the two 12 hour values and the two 72 hour values and read the 12 hour and 72 hour duration intensities for the required A.E.P from a straight interpolation line.

The two values read from figure 1.1.6 are then plotted on figure 1.1.7 and a straight line drawn. The required intensity for a given duration can then be read.

If the required A.E.P is equal to either 2% or 50%, then only the 12 hour and 72 hour intensities are required for that A.E.P and figure 1.1.7 can be used to read the required intensity.

4) For storm durations less than 12 hours, the 12 hour 50% and 12 hour 2% intensities are estimated from figures 1.1. and 1.1.3.

The intensity for the required A.E.P (12 hour) storm is estimated using the interpolation diagram, figure 1.1.6.

5) If the storm duration is between 1 hour and 12 hour the required intensity can be estimated by.

$$I_t = \left(A \left(\frac{1.798}{t+0.576} - 0.143 \right) + 1 \right) \times I_{12} \quad \text{(equation 1.1.1}$$

Where A = a constant read from figure 1.1.8.

t = the rainfall duration (hours).

I_{12} = the 12 hourintensity (mm/h).

I_t = the t hour intensity (mm/h).

6) If the storm duration is less than 1 hour, the 1 hc intensity is estimated using the above equation and the requir intensity is calculated from:

$$I_m = \left(0.309 + \frac{49.586}{m+11.767}\right) I_{60} \qquad \text{(equation 1.1.2)}$$

Where m = the duration in minutes.

I_{60} = the one hour (mm/h) intensity.

I_m = the m hour intensity (mm/h).

1.3 Worked Examples

1.3.1 Example 1

Determine the 12 hour 5% A.E.P rainfall intensity for a site on Wild Cattle Creek in the Clarence River Valley.

1) Location and Catchment Area

   Topographic map - Brookland, scale 1:31680
   Grid reference of site - 868488
   Approx. catchment centroid
   -Latitude 30 degrees 17' (or 30.28 degrees)
   -Longitude 152 degrees 46' (or 152.77 degrees)

2) From figure 1.1.2 - 12 hour, 50% A.E.P = 7.0 mm/h
   From figure 1.1.3 - 12 hour, 2% A.E.P = 17.0 mm/h

3) Interpolating in figure 1.1.6 - 12 hour, 5% AEP = 14.4 mm/h

1.3.2 Example 2

Determine the 24 hour 5% A.E.P. rainfall intensity for the site of example 1.

1) From example 1- 12 hour, 5% A.E.P. = 14.4 mm/h

2) From figure 1.1.5- 72 hour, 50% A.E.P = 2.5 mm/h
   From figure 1.1.4- 72 hour, 2% A.E.P = 6.5 mm/h

3) Interpolating in figure 1.1.6- 72 hour, 5% A.E.P = 5.5 mm/h

4) Using figure 1.1.7 and interpolating
   24 hour, 5% A.E.P. = 10.0 mm/h

1.3.3 Example 3

Determine the 2.8 hour 5% A.E.P. for the catchment of the above examples.

1) From figure 1.1.2- 12 hour, 50% A.E.P = 7.0 mm/h
From figure 1.1.3- 12 hour, 2% A.E.P = 17.0 mm/h

12 hour, 5% A.E.P = 14.4 mm/h

2) Using equation 1.1.1. with A=2.95 (from figure 1.1.8)

$$I_{2.8} = \left(2.95\left(\frac{1.798}{2.80+0.576} - 0.143\right) + 1\right) 14.8$$

$= 2.15 \times 14.4$
$= 31.0$ mm/h

1.4 References

Hudson, N. (1971) Soil Conservation B.T. Batsford Ltd, London.

The Institution of Engineers Australia (1977) Australian Rainfall and Runoff Inst. Of Eng., Australia.

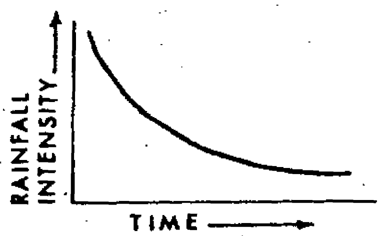

Figure 1·1·1 General relationship between rainfall intensity and duration. (from Hudson, 1971.)

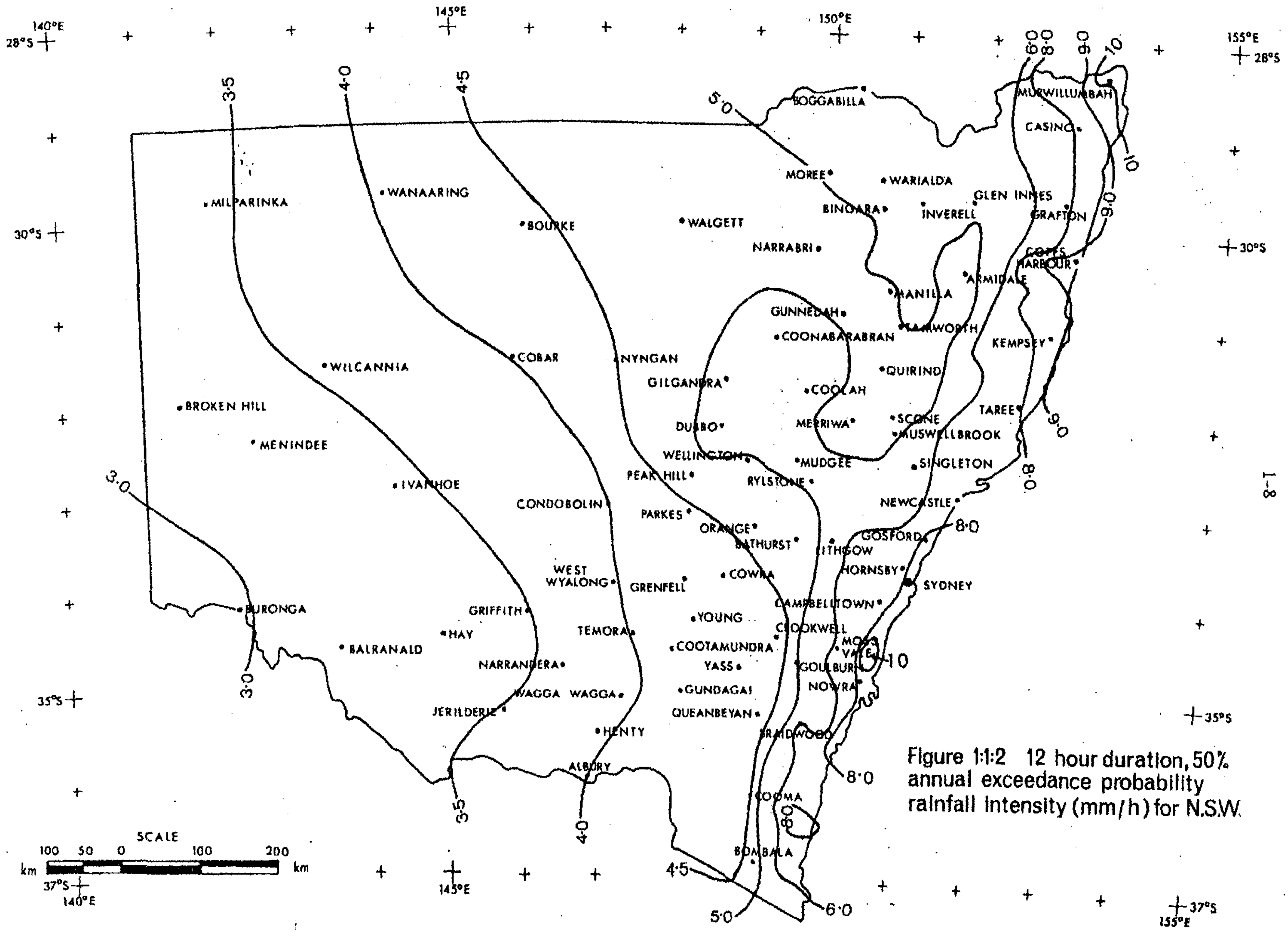

Figure 1:1:2 12 hour duration, 50% annual exceedance probability rainfall intensity (mm/h) for N.S.W.

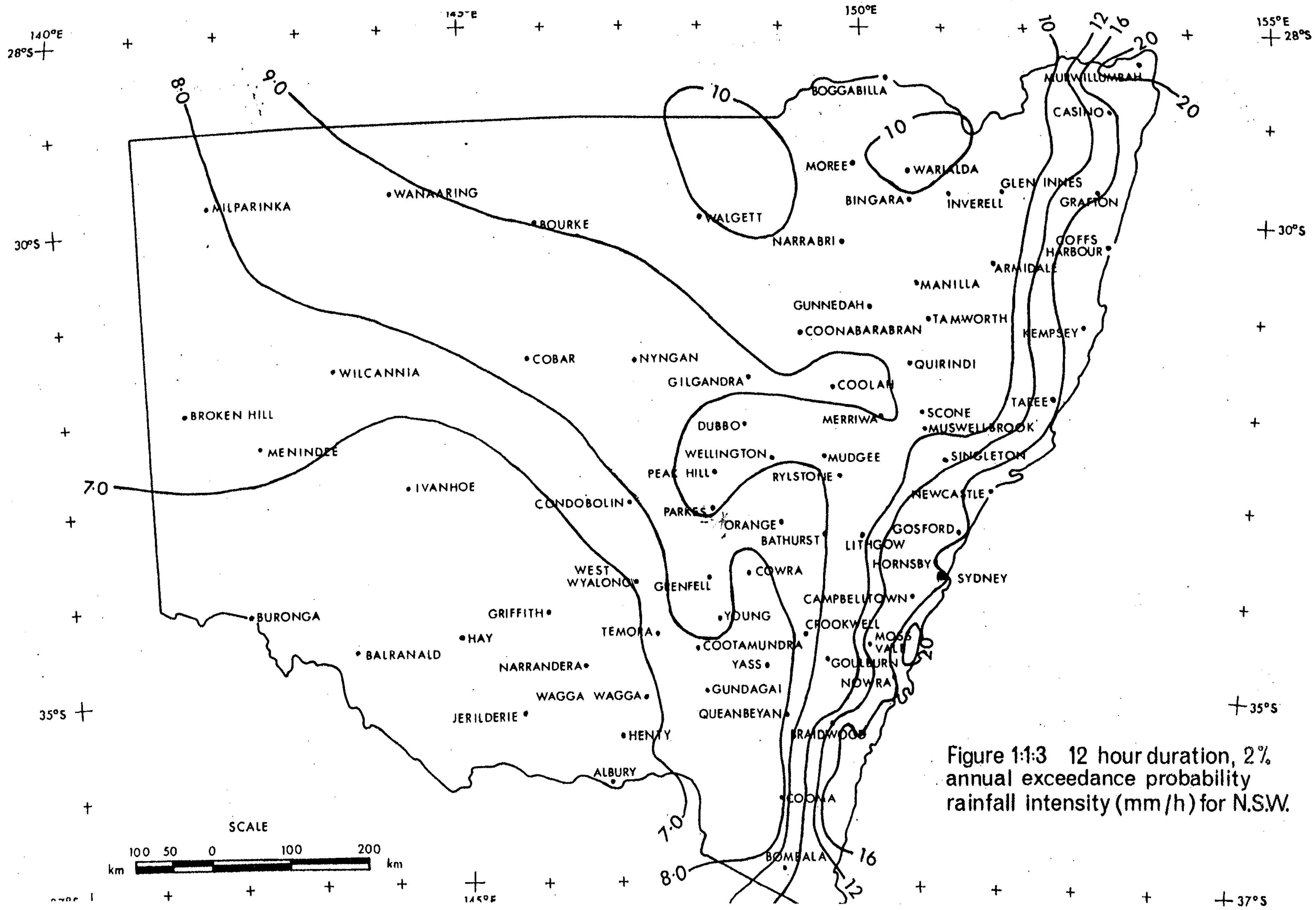

Figure 1.1.3 12 hour duration, 2% annual exceedance probability rainfall intensity (mm/h) for N.S.W.

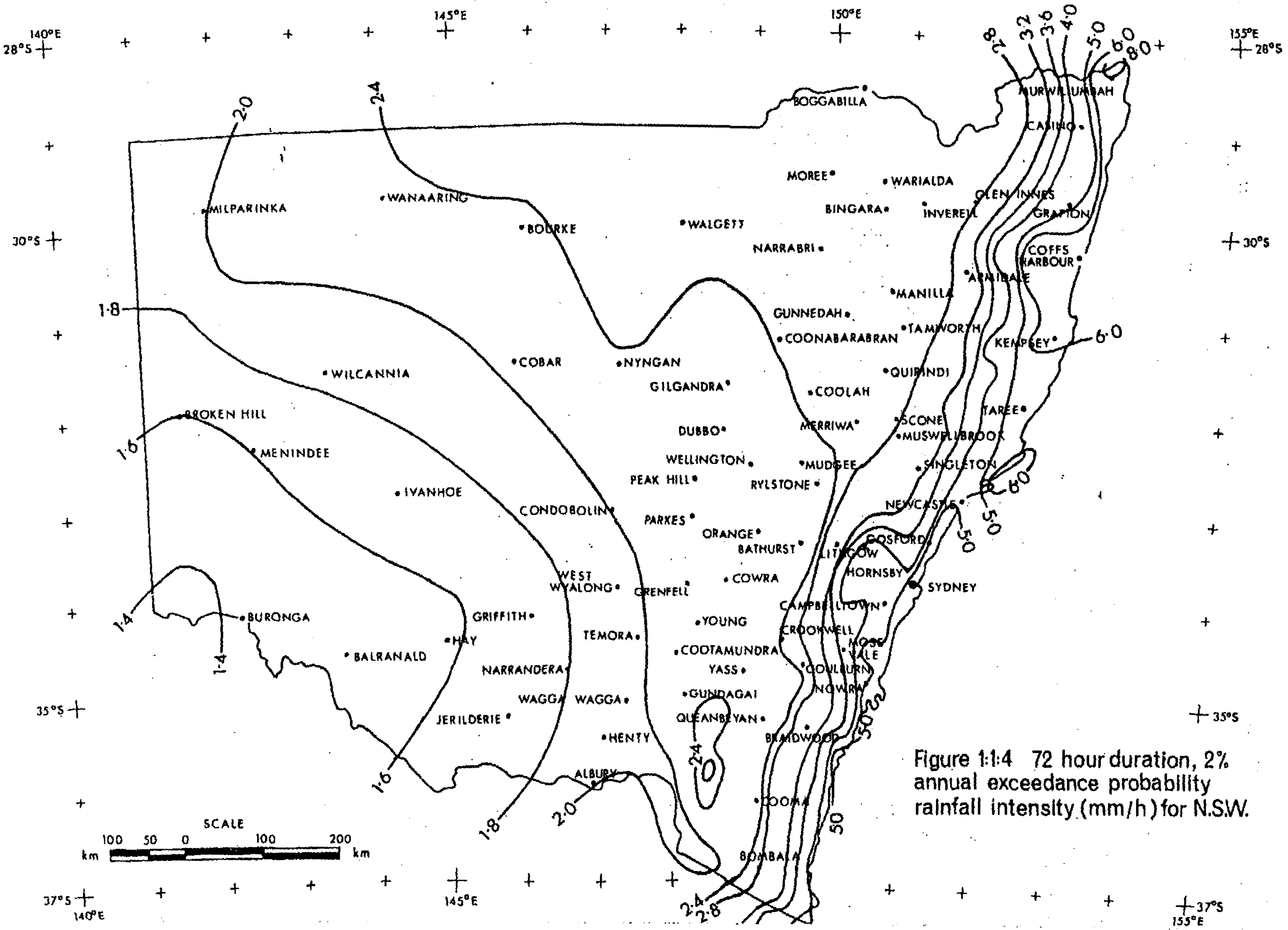

Figure 1.1.4 72 hour duration, 2% annual exceedance probability rainfall intensity (mm/h) for N.S.W.

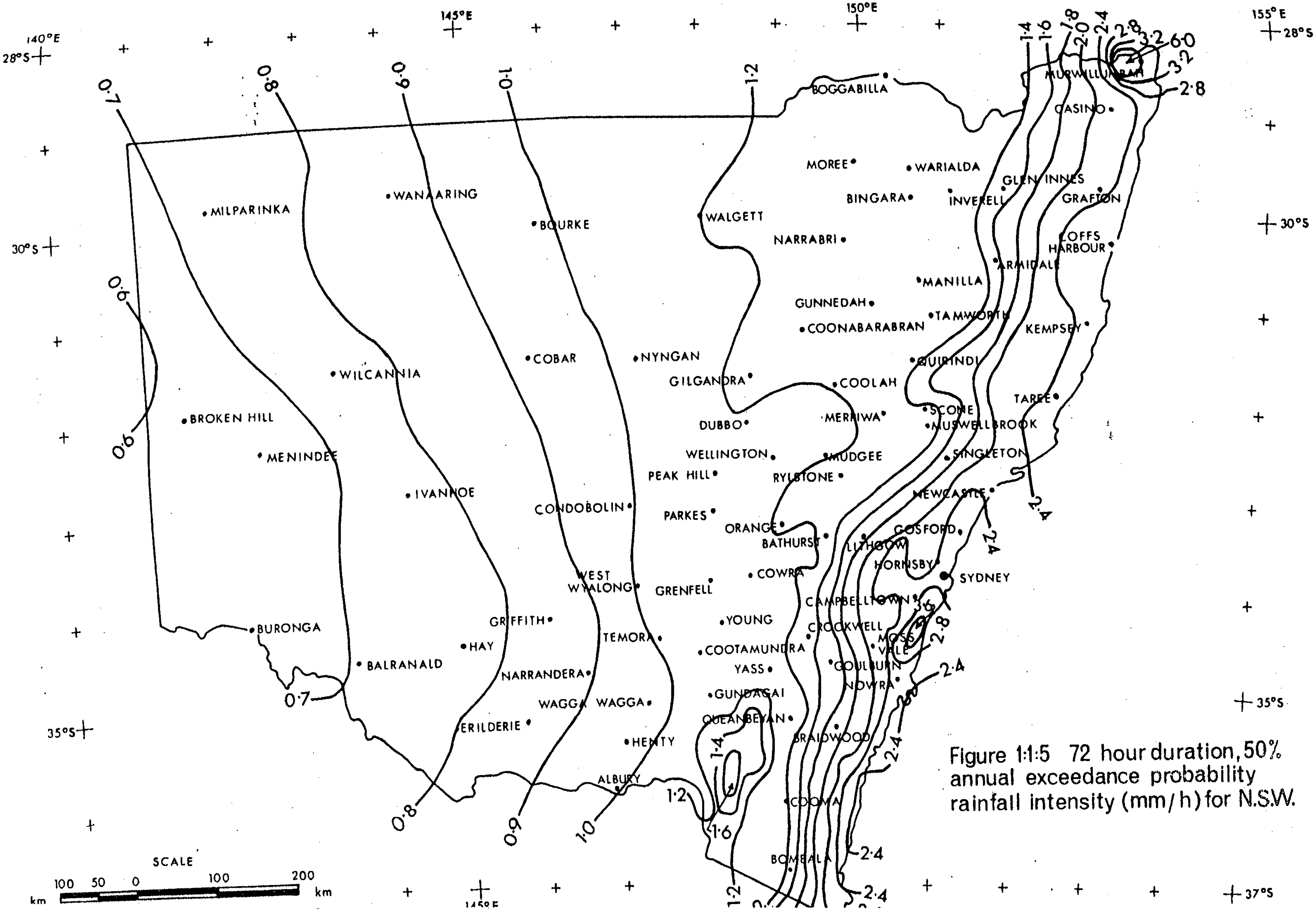

Figure 1:1:5 72 hour duration, 50% annual exceedance probability rainfall intensity (mm/h) for N.S.W.

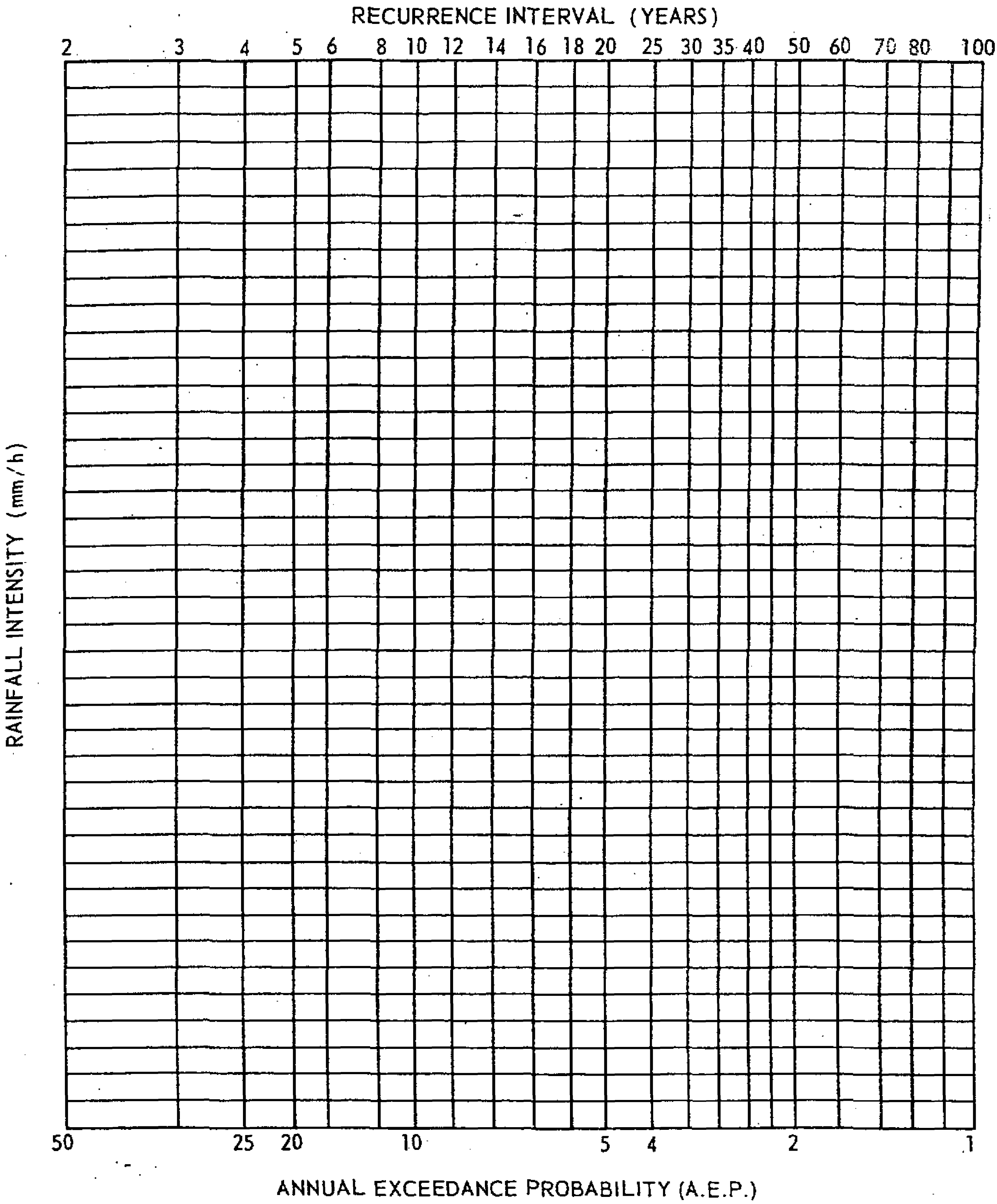

Figure 1.1.6 Interpolation diagram to determine rainfall intensity for various recurrence intervals.

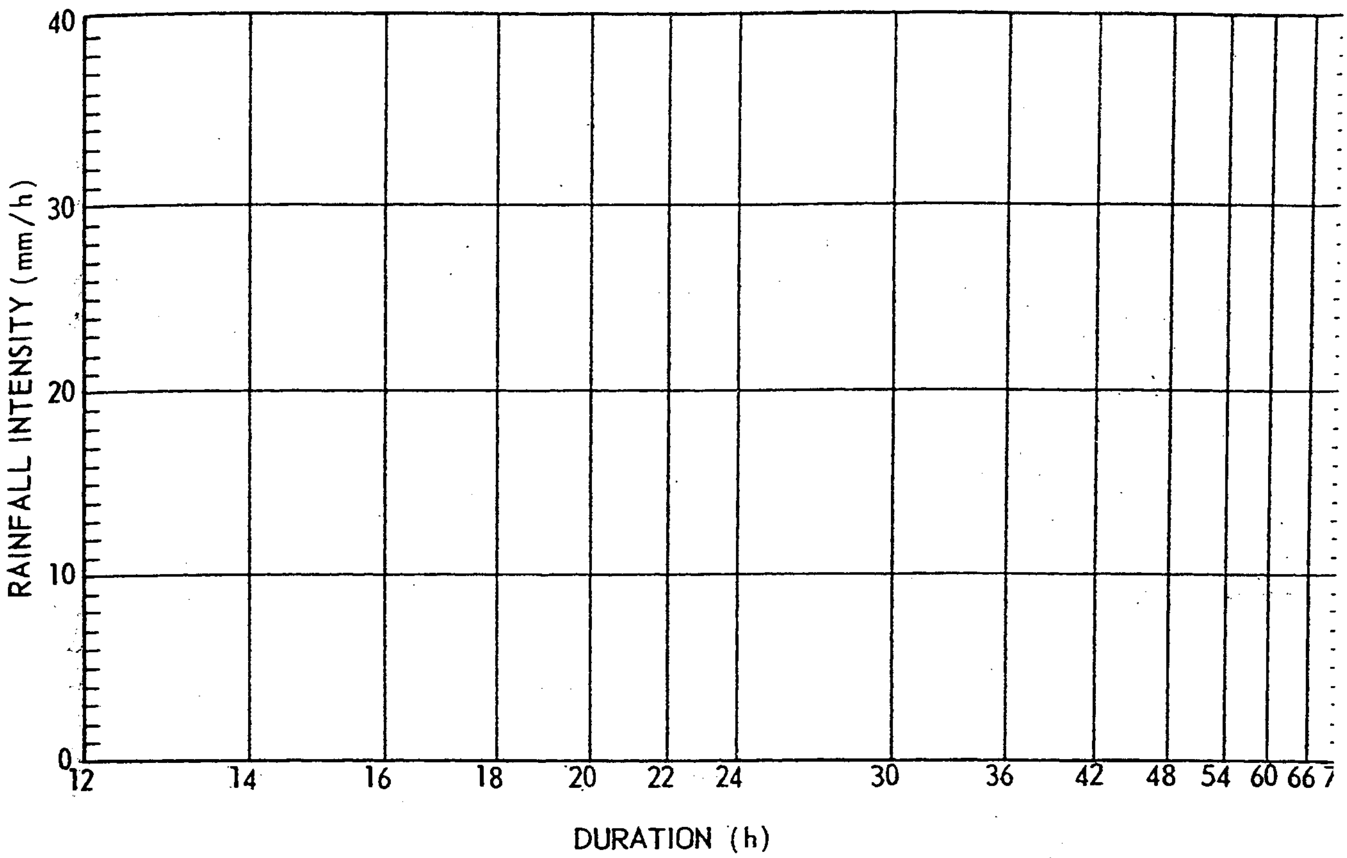

Figure 1.1.7 Interpolation diagram to determine rainfall intensity for various durations between 12 and 72 hours.

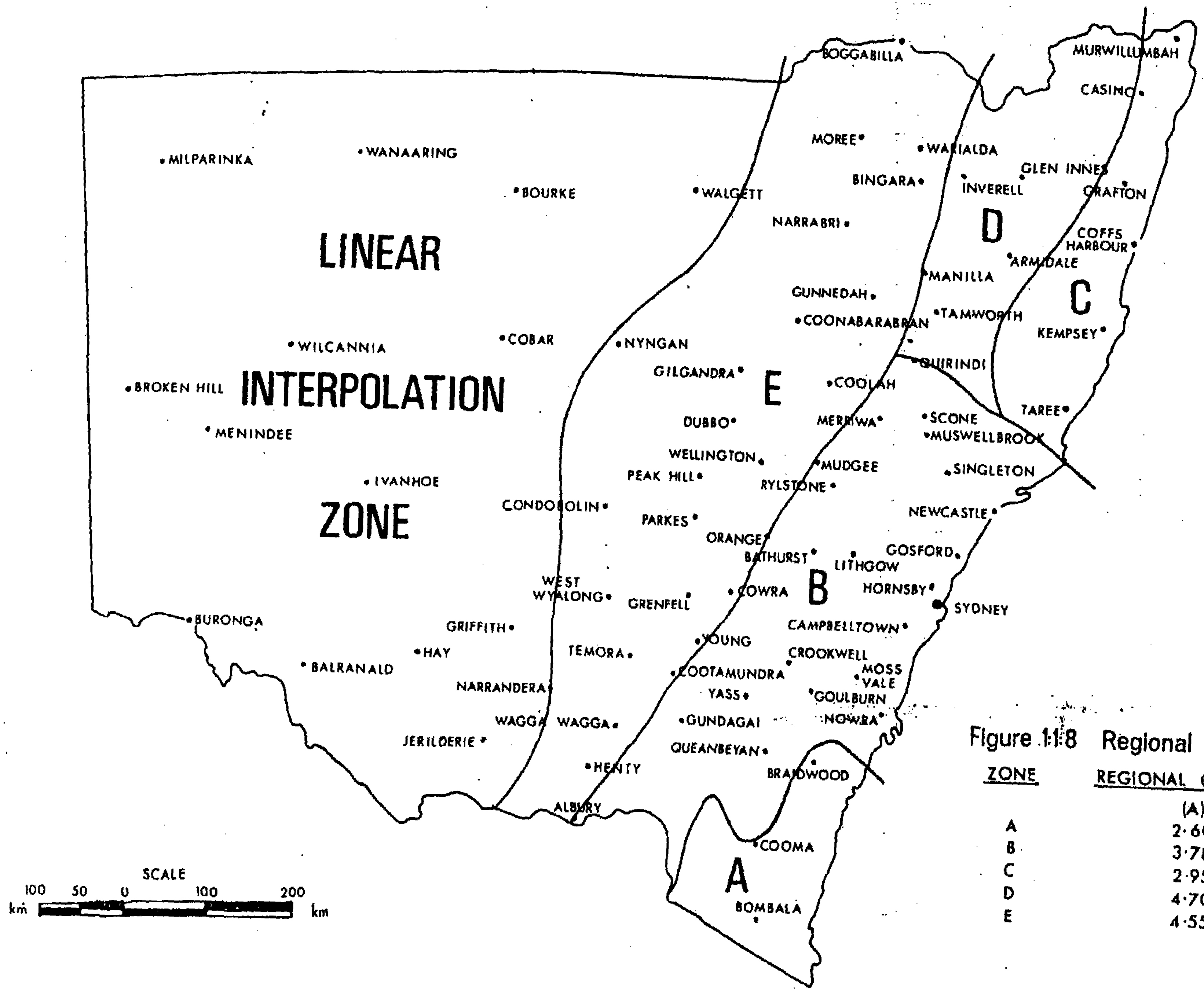

Figure 1.18 Regional Constants

ZONE	REGIONAL CONSTANT (A)
A	2·60
B	3·78
C	2·95
D	4·70
E	4·55

Section 2 - Estimation of Runoff

Section 2 Estimation of Runoff

The design of soil conservation earthworks and other structures requires an estimate of the likely amounts of runoff which will be produced from the catchments in which the structures are to be placed.

Runoff is that portion of rainfall which makes its way towards stream channels and hence to creeks and rivers in surface or subsurface flow. For design purposes, runoff is defined as surface flow only.

The design of soil conservation dams, banks, channels and waterways to handle natural surface flow involves the estimation of peak rates of runoff and runoff volumes.

In terms of soil conservation design the factors affecting runoff production are rainfall and catchment characteristics.

The most important features of the rainfall are its intensity and duration. These factors determine the rate of input of water into the catchment and the total amount of water received.

Among the catchment characteristics that affect runoff production are catchment area, shape, slope, topography, soils and land use.

2.1 Selection of Design Flood

It is generally not appropriate to analyse the economic feasibility of individual soil conservation structures, and as a result the choice of design flow for the structure must be made arbitrarily.

Two methods are available to guide the designer in choosing the design A.E.P for a structure. The first method involves a judgement based on experience, usually guided by a table of A.E.P's for a range of structure types. A list of suggested A.E.P's is given in tabular form below.

Structure	Suggested A.E.P(%)
Graded bank	20 - 10
Diversion bank	10 - 5
Waterway	10
Flume	5 - 2
Soil conservation dam	5 - 2

However where a structure has individual importance, (such as a large dam) or it's failure may have safety implications (a chute carrying water from a road causeway), an assessment of the risk of failure should be made. This risk can be defined as the probability that the design flow of the structure will be exceeded at least once during the design life of a structure.

The design life of a structure may be determined by some definite period, such as the operation of a mine, or by economic circumstances.

The A.E.P of a design flow for a structure with an intended design life (ℓ_n), and with a desired probability (P) of not failing within that life is:

$$A.E.P = \frac{-\ell_n\ (1-P)}{L}$$

Where: ℓ_n = natural logarithm

Alternatively, the risk of failure can be found for a given design life and A.E.P.

$$P = 1 - 2.718\ (-L/A.E.P)$$

Example: A structure is intended to have a design life of 20 years, with a 10% risk of failure during that life. What A.E.P should be chosen for design.

$$A.E.P = \frac{-\ell_n\ (1 - 0.10)}{20}$$

$= 0.005$

i.e. 0.5%

If a risk of failure of 50% is adopted:

$$A.E.P. = -\ell_n\ \frac{(1-0.5)}{20}$$

$= 0.035$

i.e. 3.5%

2.2 Runoff Estimation Methods

2.2.1 Statistical Rational Method

A design procedure is described below for flood estimation for rural catchments smaller than 250 km^2 in eastern New South Wales using flood data from 284 catchments and a statistical interpretation of the Rational Method (Pilgrim & McDermott, 1982).

This procedure is recommended for use by soil conservationists.

This approach, using a statistical interpretation of the Rational Method, is recommended because the derivation of data exactly matches the way in which the method is used in design. The approach is also an efficient form of regional flood frequency analysis and application of the method is simple and familiar to most designers.

The flood magnitudes of various annual exceedance probabilities (A.E.P) were obtained by frequency analysis of maximum monthly floods recorded on 284 catchments in eastern New South Wales and adjacent areas in Victoria and Queensland. Most catchments were smaller than 250 km^2 and had at least ten years of records. The flood data were carefully checked to ensure the greatest possible accuracy of the data base.

Rainfall intensities were obtained from the generalised procedures in Australian Rainfall and Runoff which must also be used in the application of the method.

Critical rainfall duration was determined by the formula;

$$Tc = 0.76A^{0.38} \qquad \text{(equation 1.2.1)}$$

This formula was derived from data from 96 catchments.

Runoff coefficients for each A.E.P were then derived as the ratio of flood magnitude to rainfall intensity. Ten per cent A.E.P coefficients C(10) were mapped for eastern New South Wales and contours of design values drawn. No significant relationships with other catchment characteristics could be found, and the scatter of the derived values from the design values given by the C(10) contours were shown to be consistent with the sampling errors inherent in the basic data. Average frequency factors were derived for each of three regions to enable evaluation of design runoff coefficients for A.E.P's to 1%.

For arid and semi arid western New South Wales, insufficient stream flow data are available for the development of firm design relationships. However, an approximate design method was developed based on estimates of bankfull discharge at sixty-eight sites (Pilgrim and McDermott, 1982). This is discussed more fully in section 2.2.2.

The steps in the design procedure using the Statistical Rational Method are similar to those in the currently used applications of the Rational Method. The steps are set out below;

1. The location of the approximate catchment centre is determined from a topographic map and the catchment area, A(km^2) determined.

2. The critical duration of the design rainfall Tc (hours) is calculated from:

   $Tc = 0.76\ A^{0.38}$ (equation 1.2.1)

   where A = catchment area (km^2)

3. Extract parameters for the estimation of the design rainfall
   - zone and zone factor (figure 1.1.8, section 1)
   - 12 hour, 50% A.E.P rainfall intensity (figure 1.1.2, section 1)
   - 12 hour, 2% A.E.P rainfall intensity (figure 1.1.3, section 1).

4. Calculate the design rainfall intensity for the design duration and the selected A.E.P (A.E.P(Y)). This is detailed in section 1 of this chapter and involves calculation of the 50% and 2% rainfalls of the design duration T (hours).

5. Determine the 10% runoff coefficient C(10) for the site from figure 1.2.1.
   For the region east of the line on figure 1.2.1 joining Ashford, Tamworth, Bathurst,Yass, Tumut and Jingellic, values of the 10% coefficient are read directly from the map.
   West of this line, design values are read directly from the map for catchments larger than 100 km^2, with a minimum recommended value of 0.10.
   For catchments smaller than 100 km^2, the value on the map should be used if it is greater than 0.40. If it is less than 0.40, the map value should be multiplied by

   $(100/A)^{0.15}$ (where A is in km^2) but, with a maximum value of 0.40. However, if this adjusted value is less than 0.20, a design value of 0.20 should be used for the 10% coefficient.
   These design values for the western region must be regarded as approximate, and can be used for the region between the line specified above and the boundary of the rainfall zone E and linear interpolation zone on figure 1.1.8. This western boundary corresponds approximately to a line joining the towns of Mungindi, Nyngan, Condobolin, Narrandera and Tocumwal.

6. For the design A.E.P(Y), determine the runoff coefficient C(Y).
   C(Y) = FFy. C(10)
   Where the frequency factor FFy is found from table 1.2.1 for the zone in which the catchment is located.

7. The design flood magnitude Q(Y) m^3/s is calculated by the Rational Method formula given below;

   Q(Y) = 0.278 C(Y%). I(T, Y%), A (equation 1.2.2)

8. If the catchment centroid is within 25 km of a zone boundary, steps 3 to 7 are carried out assuming that the site is in one zone and then the other, and the design flood magnitude is then calculated by linear interpolation given below;

$$Q = \frac{Q_{z1} + Q_{z2}}{2} + \frac{D_{z1}}{50} (Q_{z1} - Q_{z2}) \quad \text{(equation 1.2.3)}$$

   Where:
   Qz1 = flood magnitude in the first zone
   Qz2 = flood magnitude in the second zone
   Dz1 = distance of the catchment centroid from the zone boundary of the second zone (km)

Worked Examples - Statistical Rational Method

Four examples of the calculation of design flood magnitudes are given below. The first involves a catchment in a rainfall zone in the eastern part of eastern New South Wales, and the second is for the same catchment but for a 2% A.E.P involving evaluation of a frequency factor for the runoff coefficient that depends on I (12,2%) as well as the rainfall zone.

The third example illustrates the procedure where the site is within 25 km of a zone boundary.

The fourth example involves determination of 10% runoff coefficients only for small catchments at a site in the western part of eastern New South Wales. The remainder of the procedure for calculating the design flood magnitudes in this case would be identical with those in other examples.

Example 1

It is required to estimate the 5% A.E.P design flood for a site on Wild Cattle Creek in the Clarence River Valley.

1. Location and catchment area:
   Topographic map - Brookland, scale 1:31680
   Grid reference of site - 868488
   Approximate catchment centroid
   -Latitude 30 degrees 17' (or 30.28 degrees)
   -Longitude 152 degrees 46' (or 152.77 degrees)
   Catchment area = 31 km^2

2. Critical duration of design rainfall

   $$Tc = 0.76\ A^{0.38}$$
   $$= 0.76\ (31.0)^{0.38}$$
   $$= 2.80\ h$$

3. Parameters for estimation of design rainfall:
   figure 1.1.8 - (zone C) A = 2.95
   figure 1.1.2 - I(12,50%) = 9.0 mm/h
   figure 1.1.3 - I(12,2%) = 17.0 mm/h

4. Calculation of design rainfall intensity
   T = 2.80 hrs
   I (2.80,50%) = 19.4 mm/h
   I (2.80, 2%) = 36.6 mm/h

   Interpolating in figure 1.1.7- I(2.80, 5%) = 32.0 mm/h

5. 10% A.E.P runoff coefficient C(10%)
   figure 1.2.1 C(10%) = 1.20

6. Runoff coefficient of design A.E.P of 5%
   table 1.2.1 for (zone C) and A.E.P = 5%

   FFy = 1.08

   then C(5%) = FFy .C(10%)
   = 1.08 x 1.20
   = 1.30

7. Design flood magnitude
   Q(5%) = 0.278.C(5%). I(2.80,5%).A
   = 0.278 x 1.30 x 32.0 x 31.0
   = 360 m^3/s

Example 2

The problem is the same as for example 1 except it is required to estimate the 2% A.E.P design flood.

Steps (1) to (5) are the same as above, except that the rainfall intensity is I(2.80, 2%) = 36.6 mm/h.

6. Runoff coefficient of design A.E.P. of 2%
table 1.2.1 (for zone C), $FFy = 1.67 - 0.37 \log_{10} I(12,2\%)$
$= 1.67 - 0.37 \log_{10} 17.0$
$= 1.67 - 0.46$
$= 1.21$

7. Then $C(2\%) = FFy.\ C(10\%)$
$= 1.21 \times 1.20$
$= 1.45$

8. Design flood magnitude
$Q(2\%) = 0.278\ .C(2\%).\ I(2.80,2\%).A$
$= 0.278 \times 1.45 \times 36.6 \times 31.0$
$= 458\ m^3/s$

Example 3

It is required to estimate the 20% A.E.P design flood for a site on Omadale Creek in the Hunter River Valley.

1. Location and catchment area:
Topographic map - Ellerston, scale 1:31686
Grid reference of site - 300584
-Latitude 31 degrees 52' or (31.87 degrees)
-Longitude 151 degrees 18' or (151.30 degrees)
Catchment area = 104 km^2

2. Critical duration of design rainfall

$Tc = 0.76\ A^{0.38}$
$= 0.76\ (104)^{0.38}$
$= 4.44$ hours

3. Parameters for estimation of design rainfall:

figure 1.1.8 - Zone D, but very close (5km) to boundary of Zone B

As the catchment centroid is within 25 km of the zone boundary, calculations are carried forward for both zone D and zone B.

figure 1.1.8 - A = 4.70 (zone D)
= 3.78 (zone B)
figure 1.1.2 - I (12,50%) = 5.0 mm/h
figure 1.1.3 - I (12, 2%) = 9.5 mm/h

4. Calculation of design rainfall intensity:

T = 4.44 h	Zone D	Zone B
I (4.44,50%)	10.1 mm/h	9.1 mm/h
I (4.44, 2%)	19.1 mm/h	17.2 mm/h
Interpolating on figure 1.1.7		
I(4.44,20%)	12.9 mm/h	11.6 mm/h

5. 10% A.E.P runoff coefficient C(10):
figure 1.2.1
C(10%) = 0.22

6. Runoff coefficient of design A.E.P = 20%
From table 1.2.1

FFy	Zone D	Zone B
C(20%) = FFy .C(10)	0.82 x 0.22 = 0.18	0.86 x 0.22 = 0.19

7. Design flood magnitude for each zone:

For Zone D, $Q(20\%) = 0.278.\ C(20\%).\ I(4.44,20\%).A$
$= 0.278 \times 0.18 \times 12.9 \times 104$
$= 67.1\ m^3/s$

For Zone B, $Q\ (20\%) = 0.278 \times 0.19 \times 11.6 \times 104$
$= 63.7\ m^3/s$

8. Interpolation of design flood in boundary zone:
The distance (Dz1) of the catchment centroid from the boundary into Zone D is 5 km.

equation 1.2.3 then gives

$$Q(20\%) = \frac{(Q(20\%)D + Q(20\%)B)}{2} + \frac{Dz1}{50}\ (Q(20\%)D - Q(20\%)B$$

$$= \frac{67.1 + 63.7}{2} + \frac{5}{50}\ (67.1 - 63.7)$$

$$= 65.4 + 0.1 \times 3.4$$

$$= 65.7\ m^3/s\ (\text{say } 66\ m^3/s)$$

Example 4

It is required to estimate the 10% A.E.P design flood runoff coefficients for rural catchments of various sizes at Forbes in central New South Wales.

1. Catchment location
   -Latitude 33.4 degrees
   -Longitude 148 degrees

2. 10% A.E.P runoff coefficient from figure 1.2.1

   C(10%) = 0.12

3. The determination of the design C(10%) values appropriate to catchments of different sizes is illustrated in tabular form below.

Catchment Area (km^2)	Multiplying factor $(100/A)^{0.15}$	Calculated C(10%) value	Adopted C(10%) value
>100	-	-	0.12
10	1.41	0.17	0.20
1.0	2.00	0.24	0.24
0.1	2.82	0.34	0.34
0.01	3.98	0.48	0.40

4. Calculation of discharges:

The remainder of the procedure for calculating the design flood magnitudes would be identical with those in the previous examples.

2.2.2 The Bankfull Method for Western N.S.W.

Of the 284 gauged catchments used in developing the Statistical Rational Method, only three were located in the western two thirds of New South Wales. These catchments had short lengths of record and to obtain a better indication of flood potential over a wider area of the west, bankfull discharge estimates can be used.

Bankfull discharge estimates can only give an approximate indication of the flood potential of a catchment. The principle underlying their application is that the channel cross-section is formed mainly by certain flood flows. Frequent small flows have low erosive power, and very large floods have high erosive power but, occur so infrequently as to have relatively little effect on the stream cross-section. There is some <u>dominant</u> flood discharge of intermediate frequency which has greatest effect on the channel section which is conveyed just within the stream banks. The A.E.P of this discharge is 40%.

The method for estimation of design floods entails three steps.

These are:-

1. Estimate the bankfull discharge Q_b for the catchment

   $Q_b = 0.44\ (AS_e)^{0.69}$ (equation 1.2.4)

   where A = catchment area (km^2)

   S_e = equal area slope, which is the slope of a line drawn through the outlet site and intersecting the longitudinal profile of the main stream such that the area enclosed above the profile is equal to that below the profile (m/km).
   The A.E.P. of this discharge is 40%.

2. The frequency factor FFby appropriate to the design return period is determined from table 1.2.2 up to a maximum of 5% A.E.P.

3. The design flood discharge is calculated as;

   $Q(Y\%) = FF_{by} \times Q_b$

   Where $Q(Y\%)$ = flood magnitude (m^3/s) of the design A.E.P (Y%)

   FF_{by} = frequency factor from table 1.2.2

   Q_b = bankfull discharge (m^3/s)

The procedure is applicable to catchments up to $250km^2$ in undulating and hilly areas in western New South Wales. It is probably not applicable to very flat areas with characteristics different to those of the catchments from which the method was derived.

Even though this method contains considerable uncertainties no alternative design data method, based on data, is available and the procedure provides at least a guide to design floods in the absence of any other information.

Worked example - Bankfull Discharge Method

(a) Location of stream cross section of interest is on Box Creek at Cobar.

(b) Catchment area = 15.0 km^2

Main stream slope (equal area) Se = 3.2 m/km calculated from the 1:100000 scale map sheet

(c) Estimation of bankfull discharge

Ideally the site should be visited to carry out the bankfull estimate by site survey, as described in section 9.6 of Australian Rainfall and Runoff. In the normal case without a site survey, the bankfull discharge is estimated by equation 1.2.4.

$$Q_b = 0.44\ (AS_e)^{0.69} \qquad \text{(equation 1.2.4)}$$

$$= 0.44\ (15.0 \times 3.2)^{0.69}$$

$$= 6.4\ m^3/s$$

(d) Estimation of flood magnitudes for other A.E.P's.

As the catchment is in undulating to hilly country, the frequency factors from table 1.2.2 can be used to calculate these flood magnitudes from the 40% A.E.P bankfull flow, as listed in table 1.2.3.

2.2.3 Rational Method

The Rational Method is another way of estimating peak discharge. It has now largely been superceded by the Statistical Rational Method. It is used widely because of its relative simplicity and because it requires little hydrological data. However, it is stressed that the peak discharge obtained is very rough and is not recommended for final estimates.

The Rational Method is deficient in two main items. Firstly it neglects channel storage effects and secondly it assumes a uniform rainfall intensity. The Method can only be expected to give accurate results where these deficiencies are not of great consequence. This will generally be on small catchments (<500 ha) and where the variation of rainfall intensity during the critical duration is small. Also the difficulty of estimating accurate values of C and tc at present may lead to appreciable errors especially for rural catchments.

The Rational Method can be expressed by the formula

$$Q = \frac{CIA}{360} \qquad \text{(equation 1.2.5)}$$

Where Q = peak discharge (m^3/s)
C = runoff coefficient
A = catchment area (ha)
I = rainfall intensity (mm/h with the selected A.E.P and duration equal to the catchment's time of concentration tc (minutes))

The Rational Method assumes the peak rate of discharge from a catchment will be caused by a storm of duration just long enough for all parts of the catchment to contribute simultaneously to flow past the site. This is called the time of concentration, and is the time required for water to flow from the most remote part of the catchment to the outlet.

<u>Time of concentration</u> is determined by either of two methods. Selection of the appropriate method depends on the presence or absence of significant runoff diversion systems within the catchment.

(i) If the catchment flow paths remain unaltered time of concentration is estimated by the method of Bransby and Williams.

The Bransby and Williams formula is

$$t_c = \frac{FL}{A^{0.1}S^{0.2}}$$

Where t_c = time of concentration (min)
F = a factor of proportionality related to units used for the area of the catchment (92.7 when area is in hectares)
L = mainstream length (km)
A = catchment area (ha)
*S = mainstream slope (%)

$$\text{*slope} = \frac{\text{difference in elevation between highest and lowest points of catchment (m)}}{\text{length of main stream (m)}} \times \frac{100}{1}$$

Figure 1.2.2 can be used to solve the Bransby and Williams equation.

(ii) In any system of banks, runoff is diverted by well defined channels. In this case, estimates must be made for the time of overland and channelised flow.

Time of concentration is determined by the formula:

$$t_c = t_o + t_{ch}$$

where
t_c = time of concentration (min)
t_o = time of overland flow (min)
t_{ch} = time of channelised flow (min)

Figures 1.2.3 and 1.2.4 enable time of concentration to be calculated.

It is important to remember three points when determining time of concentration.

*Overland flow occurs as a thin sheet of water spread more or less evenly over the surface and is restricted to short distances rarely exceeding sixty metres. For distances exceeding this, the time of flow is estimated for channelised conditions. The velocity of flow can be assumed to be 1.0 m/s.

*Time of concentration following introduction of soil conservation works will differ from time of concentration of an untreated area. The design time of concentration should be for the treated catchment.

*In a system involving many different structures, e.g. a system of banks feeding a waterway, several trial times of concentration should be calculated. The longest time of concentration should be selected, as this allows for the whole area to be contributing at that instant.

<u>The rainfall intensity</u> (I) is calculated using the methods outlined in section 1.

<u>The runoff coefficient</u> (C) is the ratio of the peak rate of runoff to the mean rainfall intensity. The value of C is affected by rainfall intensity and annual exceedance probability, land use practices which influence infiltration, surface relief, and catchment storage.

The coefficient is estimated from table 1.2.4 and is the <u>sum</u> of the five values. Table 1.2.4 is adapted from Turner (1960), and includes data from the untreated catchment at Wagga Wagga Soil Conservation Research Service Centre. The various components are defined below:

<u>Land Use</u> This category best defines the main land use applying at the time of the design peak flow.

Relief This is the average catchment or stream channel slope. It has already been derived to compute time of concentration.

Depression Storage This defines the degree of potential catchment storage.

Infiltration and Soil Factors Classify the soil to its principal profile form using the Northcote Factual Key and select the appropriate category.

In catchments larger than 500 ha the runoff coefficient will be reduced due to variability in rainfall and storage. For these larger areas, the appropriate correction factor selected from table 1.2.5 should be applied to the runoff coefficient derived from table 1.2.4.

The steps in the procedure for estimating peak discharge using the Rational Method are set out below.

The following catchment characteristics, which can be taken from a map or aerial photograph, are required:

- catchment area
- measurement of stream channel length and difference in catchment elevation; or the overland and channelised flow components for complex situations.

1. Decide if the catchment is complex or simple in relation to runoff diversion systems. Select the appropriate method for determining time of concentration.

2. Select the appropriate annual exceedance probability.

3. Determine rainfall intensity

4. Determine the runoff coefficient from table 1.2.4 and correct for area with table 1.2.5

5. Substitute values into equation 1.2.5 to obtain peak discharge (Q) in m^3/sec.

Example:

Determine the 5% A.E.P peak discharge for a 100 hectare, untreated, arable catchment with an average slope of 5% and a main channel length of 1200 metres. The soils are moderately erodible duplex soils. The catchment is located at Wellington, New South Wales.

1. Untreated catchment, use figure 1.2.2 to determine t_c = 32 minutes

2. 5% A.E.P.

3. Rainfall intensity from section 1
   I = 77 mm/h

4. Determine runoff coefficient from table 1.2.4

*arable land with regular rotations	0.15
*average catchment slope 5%	0.00
*length of defined channel flow greater than overland flow	0.05
*duplex soils	0.15
*annual exceedance probability	0.15
	0.50

5. Determine area correction factor from table 1.2.5 = 0.87

6. Substitute into equation 1:2:5

$$Q = \frac{0.50 \times 77 \times 100}{360} \times 0.87 = 9.30 \text{ m}^3/\text{s}$$

2.2.4 Trickle Flows

Trickle flows are the persistent low volume flows associated with the recession limb of a hydrograph and catchment base flow.

They are particularly important in Tablelands and near Slopes environments where flows can persist for weeks after a flood event. Under these persistently waterlogged conditions death of vegetation and subsequent failure of grassed spillways and flumes is prevalent. The problem can be alleviated by using trickle pipes to carry the flow away from susceptible areas.

An estimate of the magnitude of this flow is required to allow economic selection of pipe sizes. A number of methods are available.

(i) Graphical Method: Figure 1.2.5 has been drawn using data from Wagga Wagga Soil Conservation Research Service Centre. It relates catchment area, annual rainfall, and discharge. It should only be used in southern New South Wales
The procedure is to enter the graph with the specified area in hectares, move up to the applicable annual rainfall in millimetres, then across to read the discharge in litres per second.

(ii) Other Methods: In areas where trickle flows persist for many weeks, an approximate estimate of flow can be obtained using a bucket of known volume and a stopwatch to actually time the rate of flow.

2.2.5 Estimation of Runoff Volume

Volume of runoff can be estimated by use of the following formula:

$$V = 90\ Q_p t_c$$

where V = runoff volume (m^3)
Q_p = peak flow (m^3/s)
t_c = time of concentration (min)

2.3 References

The Institution of Engineers Australia (1977) — Australian Rainfall and Runoff. Inst. Eng. Australia

Turner, A.K. (1960) — Rainfall Losses in Relation to Runoff for Small Catchments Jour. Inst. Eng. Aust. 32 1:6

Pilgrim D.H.; McDermott, G.E — Design Floods for Small Rural Catchments in Eastern New South Wales. A.W.R.C Tech. Paper No. 73.

TABLE 1.2.1

DESIGN VALUES OF AVERAGE FREQUENCY FACTORS $FF_Y = C(Y)/C(10)$ FOR REGIONS IN N.S.W.

Region (ARR) Rainfall Zones	Frequency Factors FF_Y ANNUAL EXCEEDANCE PROBABILITY						
	99%	50%	20%	10%	5%	2%	1% (Very approx. only)
Central and Southern Coastal (A & B)	0.60	0.70	0.86	1.00	1.13	$1.75-0.371\log_{10}I(12,2)$	$2.28-0.701\log_{10}I(12,2)$
Northern Coastal (C)	0.60	0.73	0.88	1.00	1.08	$1.67-0.371\log_{10}I(12,2)$	$2.20-0.701\log_{10}I(12,2)$
Eastern Interior & N. Tablelands (D + E)	0.49	0.62	0.82	1.00	1.18	$1.84-0.371\log_{10}I(12,2)$	$2.42-0.701\log_{10}I(12,2)$

Table 1.2.2 Frequency factors for bankfull discharge to obtain floods of other return periods - Western N.S.W.

A.E.P(%)	Frequency factor (FF_{by})
99.9	0.42
50	0.84
40	1.00
20	1.58
10	2.33
5	3.29

Table 1.2.3 Estimated flood discharges of various return periods for example of flood Design Method

Annual Exceedance Probability (AEP) %	Estimated flood magnitude $Q(y)$ (m^3/s)
99.9	2.7
50	5.4
40	6.4
20	10.0
10	15.0
5	21.0

Table 1.2.4 ESTIMATION OF RUNOFF COEFFICIENT

Catchment Characteristics	Runoff Producing Characteristics			
	Extreme	High	Moderate	Low
LAND USE	Continuous arable cultivation 0.20	Arable land with regular rotations 0.15	Hard Grazing 0.05	Ligh grazing retired land, forest, 0.00
RELIEF	Average catchment slope greater than 20% 0.10	Average catchment slope 11% - 20% 0.05	Average catchment slope 5% - 10% 0.00	Average catchment slope 0% - 5% 0.00
DEPRESSION STORAGE	Steep watercourses. Negligible catchment storage. Predominantly channelized flow 0.10	Some overland flow. Length of natural defined channel flow greater than overland flow 0.05	Some catchment Storage in banks and furrows. Length of channel flow similar to length of overland flow 0.00	Significant catchment storage in banks and furrows. Overland flow lengths greater then defined channel length 0.00
	Use modified Bransby Williams method		Use complex catchment method	
INFILTRATION AND SOIL FACTORS	No effective soil cover. Either solid rock or shallow lithosol soils 0.25	Duplex soils with hard-setting surfaces, and Uf soils 0.15	Gradational soils duplex soils with non/hard-setting surfaces, and Um and Ug soils 0.10	Deep sands and gravel deposits, and Uc soils 0.05

A.E.P.						
	1% 0.40	2% 0.30	5% 0.15	10% 0.10	20% 0.05	30% 0.00

Table 1.2.5 Area Correction Factors for the Runoff Coefficient

Area (Hectares)	Correction Factor
100	0.87
200	0.83
300	0.81
400	0.79
500	0.78
600	0.76
700	0.75
800	0.74
900	0.73
1000	0.72

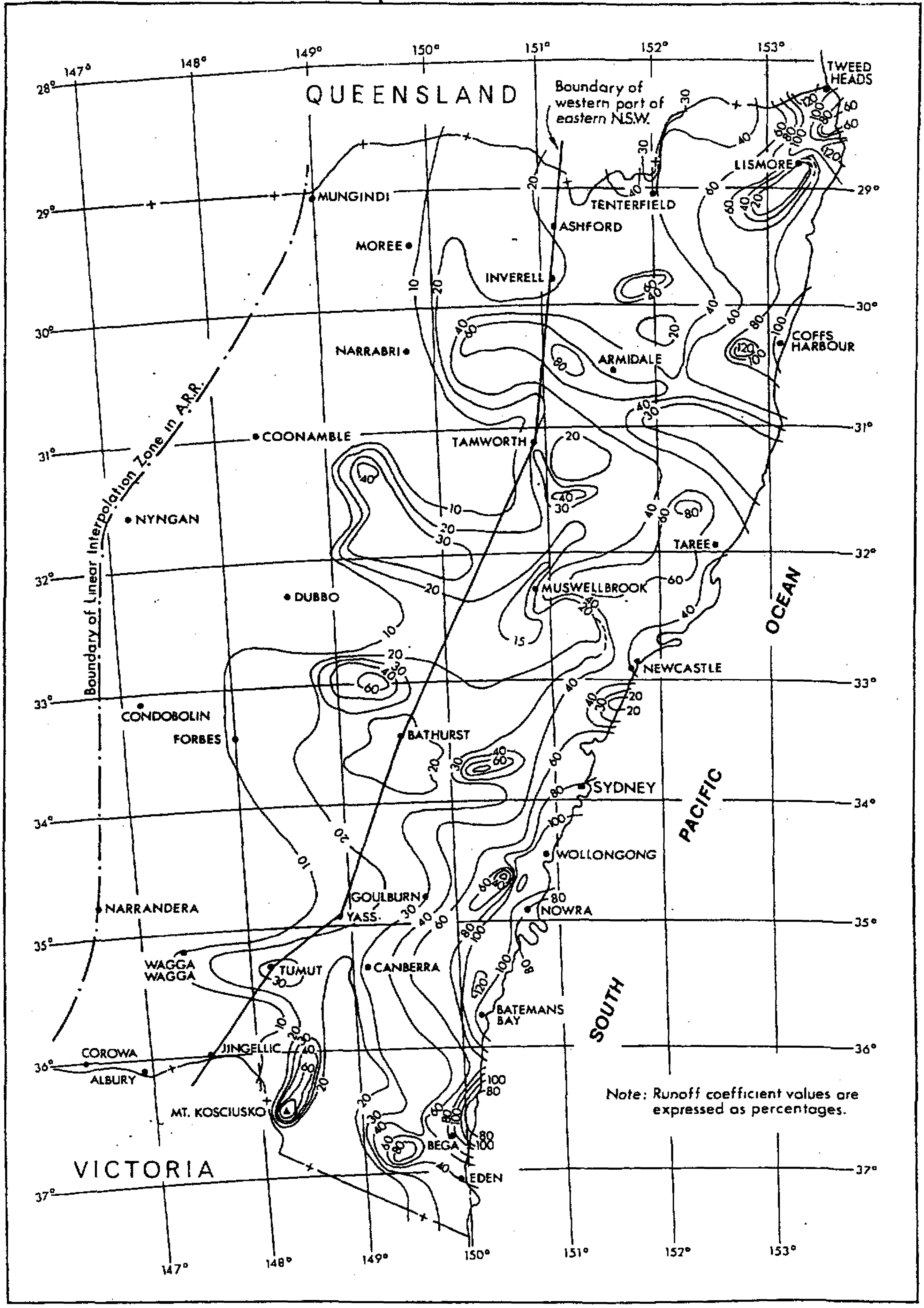

Figure 1:2:1 Map of design values of 10-year runoff coefficients-C(10)

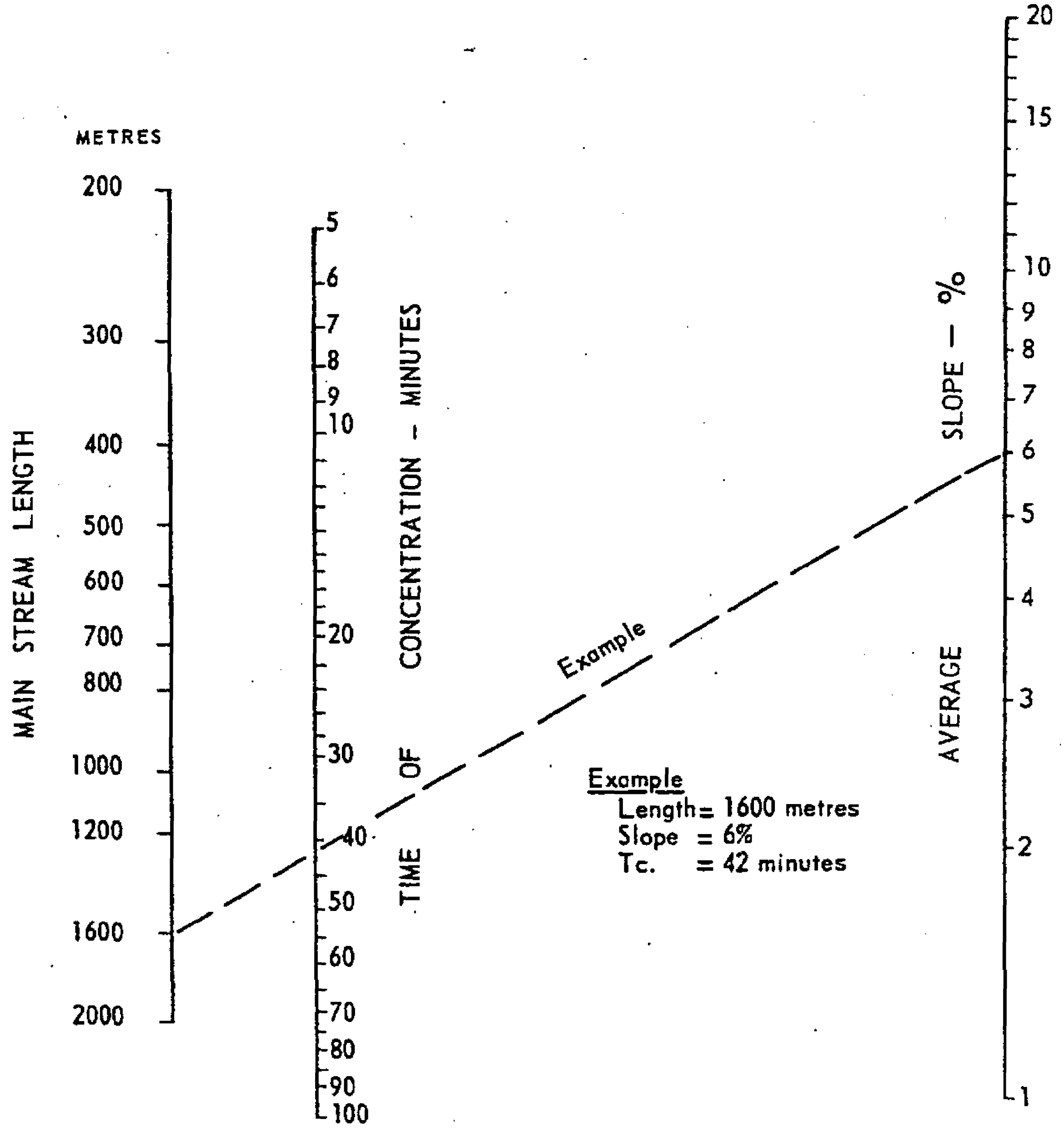

Figure 1:2:2 Nomogram for determination of time of concentration for catchments (modified Bransby Williams method).

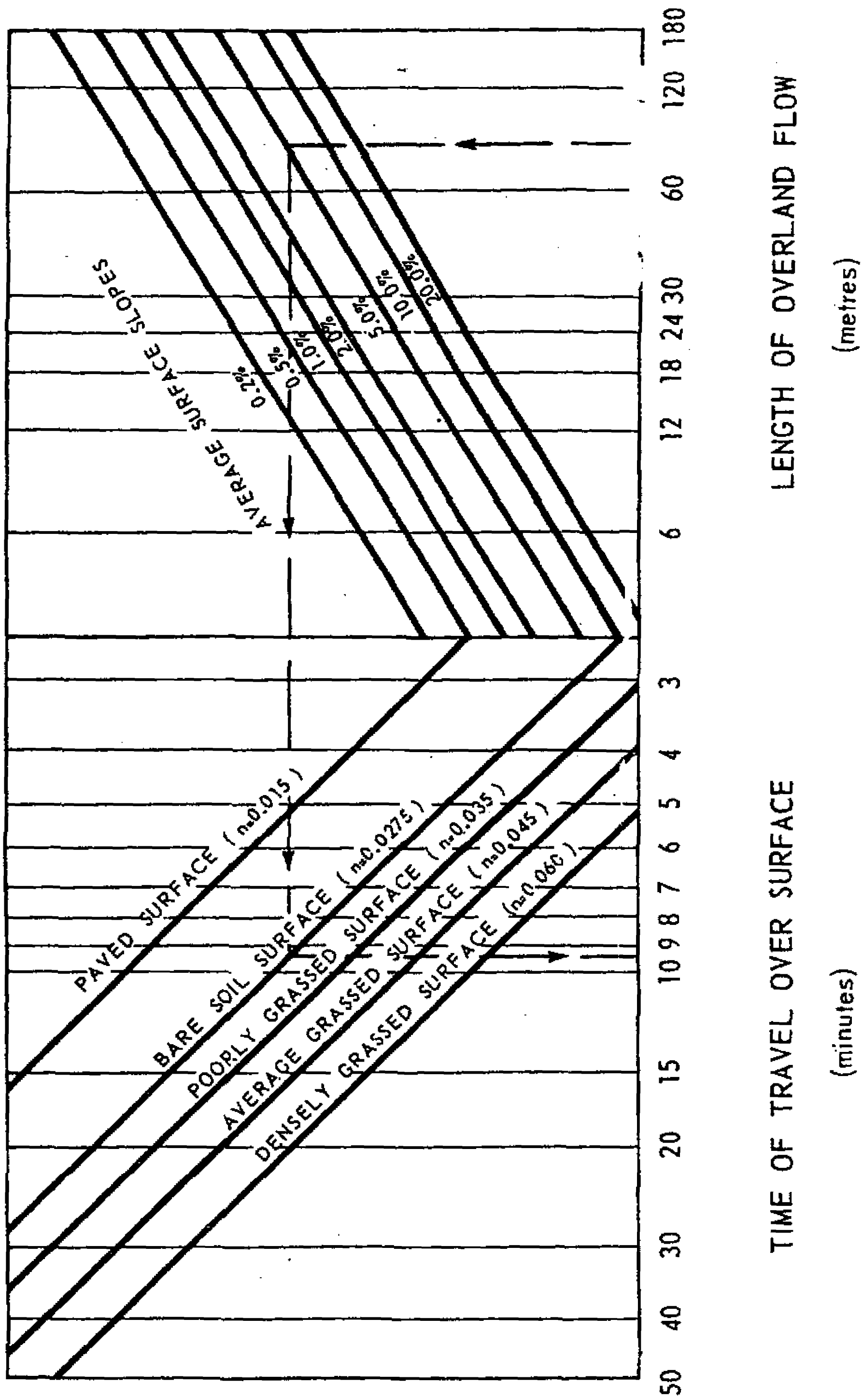

Figure1·2·3 Nomogram for determination of time of overland flow.

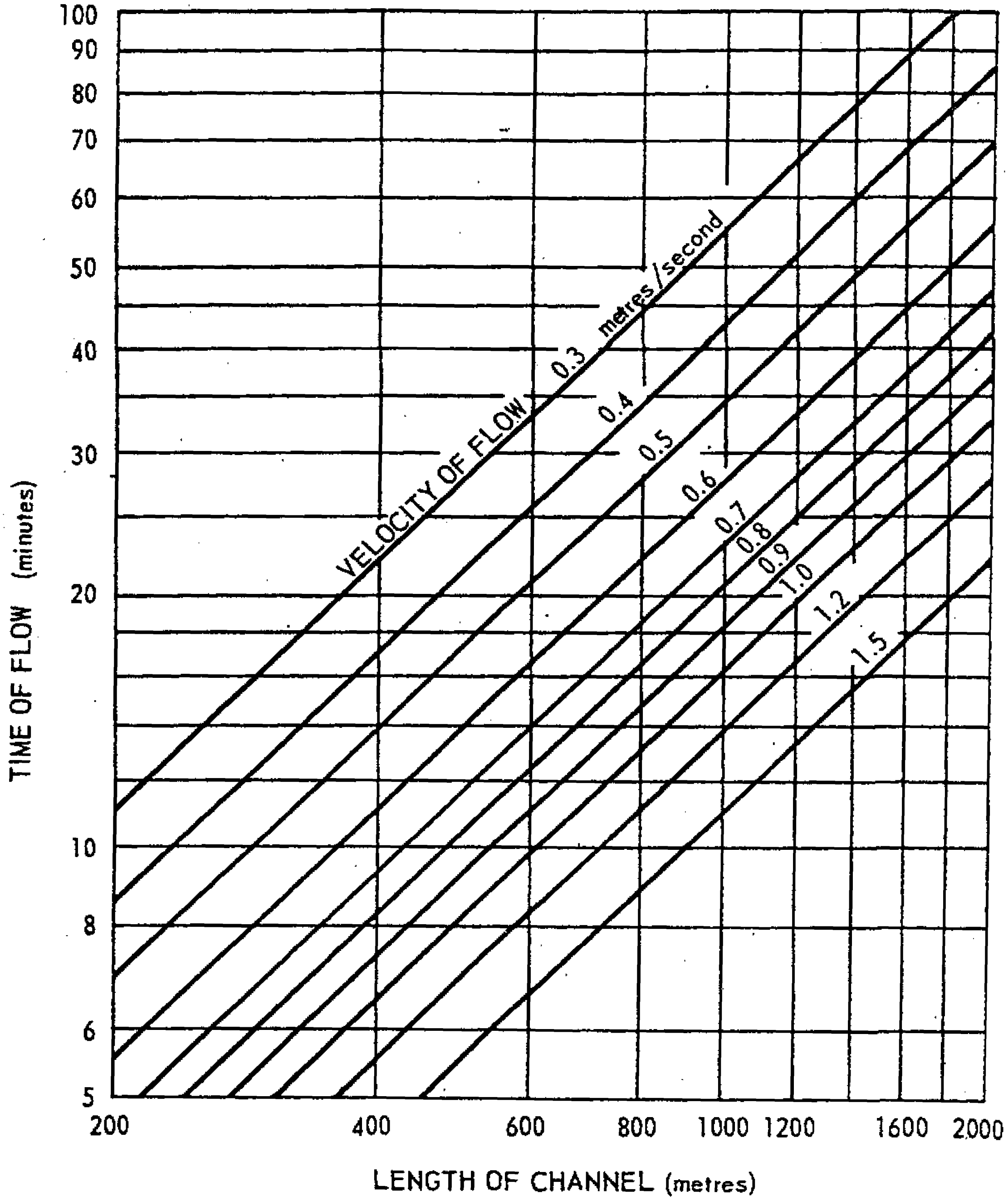

Figure 1:2:4 Nomogram for determination of time of channelised flow

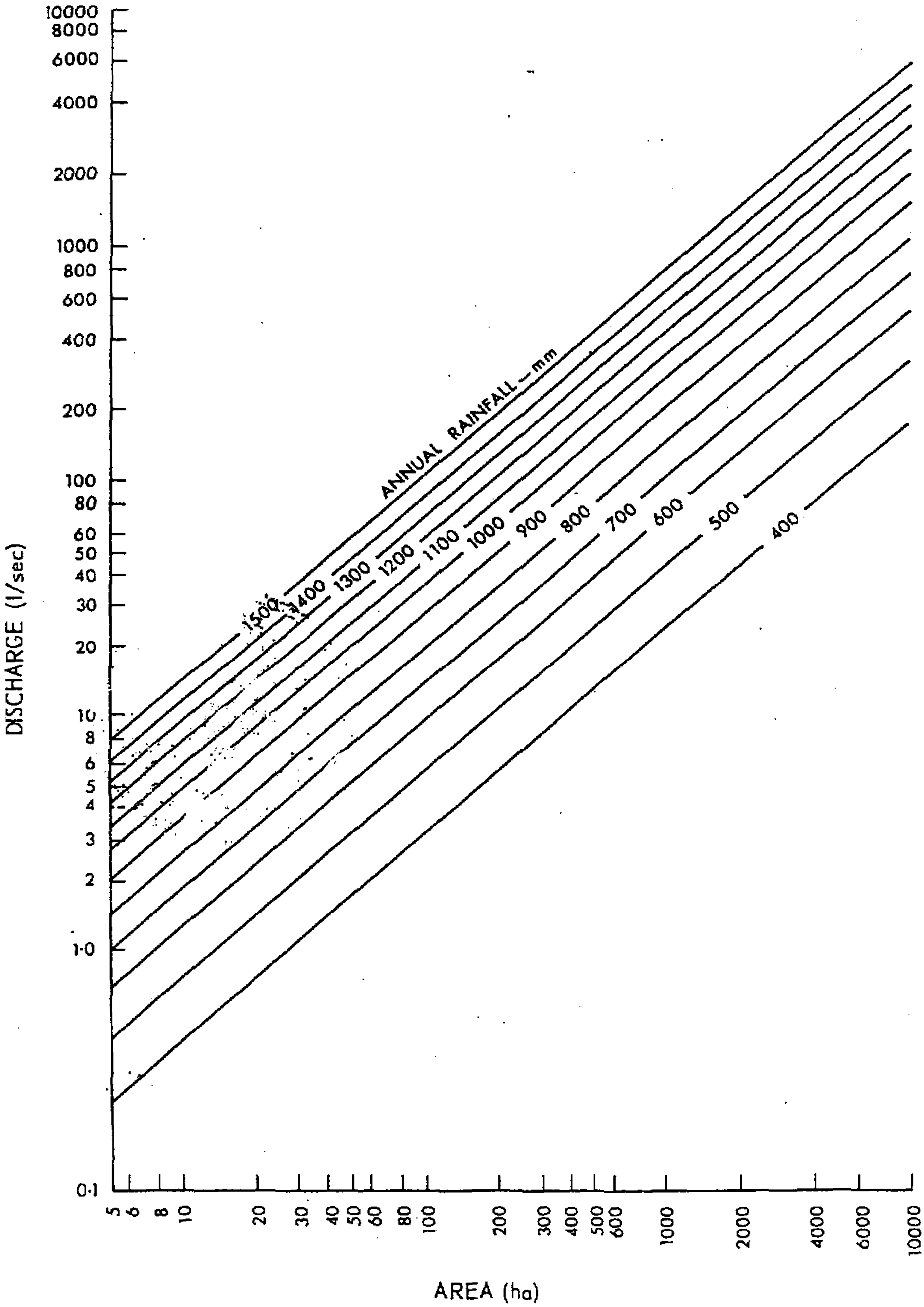

Figure 1:2:5 Graphical method for determining trickle flow discharge for area and annual rainfall.

CHAPTER 2 - BANKS AND WATERWAYS

Section 1 - Velocity of Flow and Manning's Formula

Section 1 Velocity of Flow and Manning's Formula

A system of earthworks for soil erosion control is planned so that runoff can be conveyed in channels. For an effective design to be prepared an understanding of the flow of water in open channels is therefore necessary.

Water flow in open channels is characterised by the exposure of a free surface to atmospheric pressure. Flow calculations must account for the fact that the position of the water surface is likely to change with time, the cross-sectional area and the slope of the channel.

The movement of water in an open channel can be either laminar or turbulent. However, the relatively large dimensions of most channels in soil conservation earthworks, combined with the low viscosity of water, make laminar flow extremely unusual.

The roughness of an open channel can vary considerably and the proper selection of the roughness coefficient is important in the analysis of open channel flow.

Water flowing in open channels is acted upon by all the forces that affect pipe flow (Chapter 4) with the addition of gravitational and surface tension forces that are the direct consequence of the free surface.

1.1 Types of Channel Flow

Open channel flow can be classified according to the change in flow depth with respect to time and the channel cross-sectional area occupied by the flow.

The types of open channel flow are:-

1. Steady flow
   - (a) Uniform flow
   - (b) Nonuniform flow
     - (1) Gradually varied flow
     - (2) Rapidly varied flow
2. Unsteady flow
   - (a) Uniform flow
   - (b) Nonuniform flow

(1) Gradually varied unsteady flow

(2) Rapidly varied unsteady flow

Steady Flow and Unsteady Flow: Flow is steady if depth and velocity at a given cross section are constant during the time interval considered. The flow is unsteady if the depth and velocity at a given cross section change with time.

In most open channel problems in soil conservation, flow is considered to be steady. However, in floods and surges which are typical examples of unsteady flow, the stage of flow changes instantaneously as the waves pass by and the time element becomes important in the design of control structures.

Uniform Flow and Nonuniform Flow: Open channel flow is uniform if the depth and velocity are the same at every section of the channel. A uniform flow may be steady or unsteady, depending on whether or not the depth changes during the time period being considered.

Steady uniform flow is the basic type of flow treated in open channel hydraulics.

In unsteady uniform flow the water surface fluctuates from time to time while remaining parallel to the channel bottom, which is practically impossible.

Flow is nonuniform if the depth of flow changes along the length of the channel. Nonuniform flow may be either steady or unsteady.

Nonuniform flow may be classed as either rapidly or gradually varied. The flow is rapidly varied if the depth changes abruptly over a comparatively short distance, otherwise it is gradually varied. Examples of rapidly varied flow are the hydraulic jump and the hydraulic drop.

1.2 Manning's Formula

The velocity of flow in the channels of soil conservation earthworks must be such that soil movement is negligible. The formula commonly used for calculation of velocities in channels is Manning's formula.

This formula expresses mean velocity of flow as a function of channel roughness, hydraulic radius and the slope of the energy gradient. The equation has been derived from experimental data.

Manning's formula is:

$$V = \frac{R^{2/3} S^{1/2}}{n}$$

where V = mean velocity of flow (m/s)
R = hydraulic radius (m)

$$= \frac{\text{cross-sectional area of flow}}{\text{wetted perimeter (m)}} \quad (m^2)$$

The wetted perimeter is the length of the line of contact between the water and the channel.

S = slope of the energy gradient (approximately the slope of the channel bed)(m/m))

n = coefficient of roughness

A nomogram for the solution Manning's formula is given in figure 2.2.6 (section 2 of this Chapter)

1.2.1 Roughness Coefficient

The computed discharge for any given channel or pipe will only be as reliable as the estimated value of n used in making the computation.

In the case of pipes and lined channels, specific information on the size and shape of the cross section, the alignment of the pipe or channel, and the type and condition of the material forming the wetted perimeter is known. These data permit estimation of n values within reasonably well defined limits.

(a) Bare Earth Channels

In natural and excavated earth channels various factors which affect the retardance to flow make the estimation of n more difficult. These factors are:-

<u>Physical Roughness</u> The type of material forming a channel bottom and sides will affect n. Fine particles on smooth, uniform surfaces result in relatively low values while coarse materials, such as gravel or boulders, and pronounced surface irregularity cause higher values.

<u>Cross Section</u> Gradual and uniform increase or decreases in the cross-sectional size of a channel will not significantly affect the value of n. Abrupt changes in size or the alternating of small and large sections call for the use of a larger value.

<u>Channel Alignment</u> Channels which do not meander or which have relatively large radii offer comparatively low resistance to flow. Severe meandering with the curves having relatively small radii will significantly increase the value of n.

<u>Silting or Erosion</u> Silt deposition or active erosion will result in channel variation of one form or another, so that the value of n will tend to increase.

<u>Obstructions</u> Deposits of any type of debris will increase the value of n. Clearly, the degree of effect is dependent on the number, type and size of the obstructions.

In addition, the value of n in a natural or constructed earth channel varies with the season and from year to year. It is not a fixed value.

All of the above factors should be studied and evaluated with respect to the kind of channel, the degree of maintenance, the seasonal requirements and the season of the year when the design storm normally occurs, as a basis for selecting the value of n.

Typical values for the roughness coefficient are given in table 2.2.6 (section 2 of this Chapter).

(b) <u>Vegetated Channels</u> In vegetated channels the roughness coefficient will vary greatly depending on the depth of flow and degree of vegetative retardance.

Very shallow flows encounter a maximum resistance because the vegetation is upright in the flow. Intermediate depths of flow bend over and submerge some of the grass and resistance drops off sharply as more and more vegetation is submerged.

Research in America has allowed vegetation to be grouped into five retardance categories designated A to E. Table 2.2.7 (section 2 of this Chapter) gives details of this classification while figure 2.2.5 (section 2 of this Chapter) shows the n-VR curves for the five retardance categories. The product VR, velocity multiplied by hydraulic radius, has been found to be a satisfactory index of channel retardance for soil conservation design purposes.

In the past it has been common practice to use n = 0.04 for vegetated waterway design. In many channels this may be satisfactory. However, for long, large channels where refinements in design may result in lower construction costs n should be estimated more accurately.

1.3 Hydraulic Efficiency of Channel Cross Sections

A typical uniform flow problem in the design of artificial channels is the economical proportioning of the cross section since the cost of constructing a channel is roughly proportional to its cross-sectional area.

A channel is designed with the given roughness coefficient and slope to carry a certain discharge. If cross-sectional area is to be minimum, then velocity of flow must be a maximum. Thus, from Manning's formula, the hydraulic radius must also be a maximum. If the cross-sectional area is fixed then the problem reduces to minimising the wetted perimeter. That is the ideal cross section should be a semi-circle.

It can also be shown that the best trapezoidal shape is that which approximates most closely to a semi-circle, in that a semi-circle having its centre in the surface, can be inscribed in the trapezoid.

The best shape for a rectanglular channel is that for which the width is twice the depth.

However, flow resistance is not the only consideration in the design of a channel. The possibility of erosion and the ease of construction of the channel must also be considered.

During low flows, sediment may be deposited in broad-bottom trapezoidal channels which may result in meandering of higher flows and development of turbulence and eddies which will result in damage to vegetation. Triangular channels reduce sedimentation, but high velocity flows may scour the bottom of the waterway.

Parabolic sections most closely resemble sections found in natural waterways and since the centre is low, small flows will be carried with less meandering than in a trapezoidal channel. However, a trapezoidal channel is generally easier to construct than a parabolic channel.

Batter grades should be such that they facilitate crossing with farm equipment.

1.4 Velocity of Flow

The design velocity for a channel is in fact only an average velocity over the cross section of that channel.

The ability of vegetation to resist the erosive force of runoff is limited. The permissible velocity of flow in a channel is dependent upon the type, condition, and density of vegetation and the erodibility of the soil.

Uniformity of vegetative cover is very important, as the stability of the most sparsely vegetated area controls the stability of the whole channel. Permissible velocities for tussock forming plants and other nonuniform cover are lower than those for the uniform cover provided by prostrate plants. Tussock forming plants tend to produce nonuniform flows with high localized erosion hazards.

Permissible velocities are also influenced by channel slope because steeper channels produce increasing turbulence with intense localized erosion hazard.

Table 2.2.3 (section 2 of this Chapter) gives maximum permissible velocities for bare soil channels.

Table 2.2.4 (section 2 of this Chapter) gives maximum permissible velocities for vegetated channels. These maximum permissible velocities have been based on an erodibility index of the surface soil as determined by Charman, 1978. Table 2.1.1 gives erodibility index values for a range of soil properties (Charman, 1978).

1.5 References

Charman, P.E.V. (1978) Soils of New South Wales: Their Characterisation, Classification and Conservation.
S.C.S. of N.S.W., Sydney. Tech Handbook No. 1.

Schwab, G.O.; Frevent, R.K..; Edminster, T.W., Barnes, K.W. (1981)
Soil and Water Conservation Engineering (3rd Edition); John Wiley and Sons

Table 2.1.1
Erodobility Index Values for a Range of Soil Properties
(after Charman, 1978)

Texture Group	Structure Grade	Horison Depth	Dispersibility: None	Dispersibility: Moderate	Dispersibility: High
Sand	apedal	<0.2m	1.3 (M)	-	-
		0.2-0.4m	0.9 (L)	-	-
		>0.4m	0.9 (L)	-	-
Sandy Loam	apedal	<0.2m	1.2 (M)	2.4 (H)	-
		0.2-0.4m	0.8 (L)	1.7 (M)	-
		>0.4m	0.8 (L)	-	-
	weakly pedal	<0.2m	2.7 (H)	5.4 (E)	-
		0.2-0.4m	1.9 (M)	3.8 (V)	-
		>0.4m	1.5 (M)	-	-
Loam	apedal	<0.2m	1.3 (M)	2.5 (H)	-
		0.2-0.4m	0.9 (L)	1.8 (M)	-
		>0.4m	0.7 (L)	-	-
	weakly pedal	<0.2m	2.5 (H)	5.1 (E)	-
		0.2-0.4m	1.8 (M)	3.6 (V)	-
		>0.4m	1.5 (M)	-	-
	peds evident	<0.2m	2.8 (H)	5.6 (E)	-
		0.2-0.4m	2.0 (H)	-	-
		>0.4m	2.0 (H)	-	-
Clay Loam	apedal	<0.2m	1.2 (M)	2.4 (H)	-
		0.2-0.4m	0.9 (L)	1.7 (M)	-
		>0.4m	0.7 (L)	-	-
	weakly pedal	<0.2m	2.3 (H)	4.6 (E)	-
		0.2-0.4m	1.5 (M)	3.0 (V)	-
		>0.4m	1.2 (M)	-	-
	peds evident	<0.2m	2.9 (H)	5.7 (E)	-
		0.2-0.4m	2.0 (H)	4.0 (E)	-
		>0.4m	1.6 (M)	-	-
Light Clay	weakly pedal	<0.2m	2.0 (H)	4.0 (E)	6.0 (E)
		0.2-0.4m	1.6 (M)	3.3 (V)	4.9 (E)
		>0.4m	1.6 (M)	3.3 (V)	4.9 (E)
	peds evident	<0.2m	1.7 (M)	3.5 (V)	5.2 (E)
		0.2-0.4m	1.4 (M)	2.8 (H)	4.2 (E)
		>0.4m	1.4 (M)	2.8 (H)	4.2 (E)
	highly pedal	<0.2m	2.0 (H)	4.0 (E)	-
		0.2-0.4m	1.6 (M)	3.3 (V)	-
		>0.4m	1.6 (M)	3.3 (V)	-
Medium to Heavy Clay	weakly pedal	<0.2m	1.4 (M)	2.8 (H)	4.3 (E)
		0.2-0.4m	1.2 (M)	2.3 (H)	3.5 (V)
		>0.4m	1.2 (M)	2.3 (H)	3.5 (V)
	peds evident	<0.2m	2.1 (H)	4.2 (E)	6.4 (E)
		0.2-0.4m	1.7 (M)	3.5 (V)	5.2 (E)
		>0.4m	1.7 (M)	3.5 (V)	5.2 (E)
	highly pedal	<0.2m	2.0 (H)	4.0 (E)	6.0 (E)
		0.2-0.4m	1.6 (M)	3.3 (V)	4.9 (E)
		>0.4m	1.6 (M)	3.3 (V)	4.9 (E)

L - low M - moderate H - high V - very high E - extreme

N.B. Borderline values are included in the higher category.

Section 2 - Banks

Section 2 - Banks

Banks are the most commonly used structures for soil erosion control and prevention in the grazing and agricultural areas of New South Wales. With complementary land management practices they enable the landuser to fully realise the potential of the land while maintaining its stability.

Banks are used to intercept, contain or divert runoff across slopes at a safe velocity. Banks outlet into formed waterways or natural water disposal areas. The purpose of each bank is to increase the time of concentration of runoff and to control its volume and velocity.

This section discusses the types of banks and their use in soil conservation as well as the design of banks and bank systems.

2.1 Function of Banks

Banks vary in shape and size to suit a wide range of conditions and requirments. Some of the functions of banks are:

* Reduce the length of slope: The positioning of banks at predetermined intervals on sloping land breaks up the slope into shorter lengths thus reducing the depth of runoff flow and consequently its velocity.

* Increase the time of concentration: This is achieved by lowering runoff velocities and increasing the length of travel for water flow along bank channels. This reduces peak discharge.

* Increased absorption: Level absorption banks are most effective in more permeable soils in grazing lands. The runoff retained in the bank channel is allowed to infiltrate into the soil. Increased time of concentration also encourages greater absorption.

* Diversion of flow: Banks can be used to divert runoff from an eroded area to a safe disposal area.

* Water spreading: Banks can be designed to spread runoff so that the erosive energy of the water is dissipated and some irrigation is achieved.

* Water harvesting: Water collected in banks can be transported to a water storage structure.

2.2 Bank Types

A wide range of bank types has evolved over the years as described below.

2.2.1 Graded Banks

Graded banks are the most frequently constructed banks in New South Wales. They are designed to intercept and divert runoff. The channel grade must be such that water is carried at a velocity that will not cause channel scouring or sedimentation. Graded banks may have bare soil or vegetated channels.

Graded banks are preferred in arable lands where they reduce the incidence of runoff pondage and subsequent production loss in bank channels. They can also be used on grazing land to some extent.

A number of types of graded banks are used;

(a) Broad Based Banks are low profile banks with a smooth cross section and gentle batters which are well compacted. They are used extensively on arable lands on the Slopes and Plains of New South Wales and can be constructed on slopes up to 6 per cent. Batter grades are commonly 6:1 (1 vertical to 6 horizontal) and the bank dimensions are chosen according to the farming machinery used by the landholder. Usually, the design requirements of the bank are different to its final size owing to these farming machinery requirements.

The advantages of broad based banks are:-

* less susceptible to failure due to soil tunnelling and cracking
* the entire bank and channel can remain in production
* weeds may be controlled more easily
* vehicle access and fencing over banks is improved
* the banks are aesthetically more appealing

The main disadvantages is that they are more costly to construct than ridge type banks.

(b) Semi-Broad Based Banks are used widely on arable lands of greater that 6 per cent slope and on lower slope grazing lands. Their batter grades range from 6:1 to 4:1 and batter lengths are within the range of 3.0 m to 4.0 m.

The excavated batter (No. 1 batter) channel and uphill batter of the bank (No. 2 batter) are constructed to facilitate farming operations. The downhill bank batter (No. 3 batter) may be farmed on lower slope lands. However, on steeper lands (greater than 10 per cent slope) the No. 3 batter is frequently short in length (2 - 2.5 m) and steep in grade (2:1 to 3:1).

These banks are a compromise between the broad based bank and the ridge type bank. They have some major advantages over ridge type banks. These are:-

* some or all of the bank area may be farmed
* weeds are easier to control
* vehicle access is possible
* the banks are aesthetically more appealing.

(c) Ridge Type Banks are narrow based banks having more or less triangular cross sections. They are used widely in the Tablelands and Slopes of New South Wales on slopes greater than 10 per cent. Their batter grades range from 4:1 on the No. 1 batter to 1.5:1 on the No. 2 and No. 3 batters. These banks may be 0.90 m high or higher and have the capacity to carry large volumes of runoff.

Ridge type banks cannot be cultivated with normal machinery and should only be used where land slope is so great that there is no alternative bank type which can be used or where large volumes of runoff are to be carried.

The advantages of this type of bank are their large channel capacity and lower construction cost.

The disadvantages are:-

* susceptible to failure due to soil tunnelling and cracking
* weed control is difficult
* vehicle access and stock movement are restricted
* stock tracks frequently reduce bank freeboard
* the banks are not aesthetically pleasing

(d) Diversion Banks are used in specific locations to divert relatively large amounts of runoff from one location to another. Consequently, they must be carefully designed.

(e) Trainer Banks are used where runoff is to be directed away from an undesirable location or toward a natural grassed waterway. These banks are constructed so that water will flow on natural vegetation and only be guided by the bank. The grade of these banks may be quite steep because they are often located directly down or partly across a slope.

2.2.2 Absorption Banks

Absorption banks are typically used on grazing land where the soil is highly permeable. They are designed to retain runoff by impounding a calculated amount of water in a level bank channel with a turn-up at each end. Water depth is varied by the height of the bank outlet.

Absorption banks can be most effective in increasing the time of concentration in a catchment and in reducing surface water flows over unstable water disposal areas. They should not be constructed in arable situations, on dispersible soils and on soils prone to cracking. This is to avoid inconvenience to the farmer or the risk of earthwork failure.

2.2.3 Level Banks

Level banks are constructed along the true contour with level channels which discharge on either or both ends. Level banks differ from absorption banks in that the bank outlets are not raised and the volume of water detained is minimal. A level channel will reduce the rate of water flow and thus increase the time of concentration in a catchment.

2.2.4 Waterspreading Banks

These are special-purpose banks commonly in use for waterspreading. There are three variations:

(a) Gap Spreader Bank

A level bank constructed to allow water to be spread by a series of gaps in the bank. Excavation from below creates a level channel into which water flows through the gaps, and from which it spreads evenly over land downslope. Such banks are normally used in marginal arable areas, where large amounts of runoff are to be controlled and soils are relatively impermeable.

(b) Gap Absorption Spreader Bank

A series of related absorption banks on the same contour which allow water to be spread via the gaps between them. Excavation from below creates level channels into which runoff water flows through the gaps when the banks are full, and from which it spreads evenly over land downslope. Such banks are normally used in marginal arable areas.

(c) Diversion Spreader Bank

A bank that collects water from high runoff producing areas and diverts and spreads out the flow to increase the production from a limited area downslope. Such banks normally consist of a diversion bank and a gap spreader bank joined by a continuous channel. They are particularly suitable for use in marginal arable areas.

The gap spreader section of the banks are constructed with excavation on the lower side. The water outlets into the excavation area and spills out evenly along a level sill on the downstream edge of the borrow area.

2.3 Bank Spacing

Banks should be spaced so that interbank rilling is minimised so that bank channel capacity is not reduced by siltation. Correct bank spacing is particularly important on arable land because cultivation increases the erosion hazard.

On grazing land, where vegetative cover usually provides better soil protection, bank spacing criteria can be more flexible and are mainly determined by site suitability.

The main factors determining correct bank spacing are:-

* Rainfall erosivity

* Soil type - soil texture and soil structure influence infiltration and soil erodibility

* Length of slope - long lengths of slope are more likely to result in channelised water flow

* Degree of slope - this will affect runoff velocity and the development of channelised water flow

* Change of slope - where slope varies along the bank length the change in bank spacing is inversely proportional to the change in slope. This is most important on low slope lands.

* Land use - grazing is a less intense land use than cultivation. The intensity of land use will affect bank spacing.

* Degree of existing erosion - banks should be closer together if soil erosion has already occurred.

* Land management - this may have a significant effect on bank spacing. For example, in some circumstances conservation farming will enable banks to be spaced further apart. This becomes a matter of individual assessment for soil conservationists in consultation with the landuser.

In practice, slight adjustments to calculated bank spacing can be made to reduce management problems, to be more economical and result in a more effective system of banks.

Instances where adjustments may be necessary and permissible are as follows:-

* to avoid acute angles and excessive intersection of fences and access roads

* to avoid buildings, stockyards, windmills and troughs, fodder and shade trees, power and telephone poles, underground cables and water pipes

* to provide a stable outlet into the disposal area

* to blend in with existing structures, e.g. a dam or an existing bank

When deciding bank spacing sound knowledge and experience of the local district will be invaluable. Consequently, district bank spacings have not been included in this Manual due to the variable bank spacing criteria from district to district.

2.3.1 Calculation of Bank Spacing

In general bank spacing is determined from the following formula (Stewart, 1955) :

$$HI = \frac{K}{\sqrt{S}}$$

$$VI = \frac{K\sqrt{S}}{100}$$

Where HI = horizontal interval (m)
VI = vertical interval (m)
K = bank spacing factor
S = slope (%)

As a general guide recommended bank spacing factors for regularly cultivated arable soils in the wheat belt of New South Wales are given in table 2.2.1.

Bank spacing factors for other areas, and for differing soil and land use conditions can be determined from table 2.2.2. The modifying factor for annual rainfall accounts for the general change in rainfall intensity (and consequently erosivity) with decreasing or increasing annual rainfall. Thus, the general decline in rainfall intensity towards the drier western areas of the wheat belt allow bank spacing to be increased.

After evaluating K, bank spacing may be determined by substitution in the above equations or it can be determined graphically from figures 2.2.1 or 2.2.2.

2.4 Bank Channels

2.4.1 Channel shape

Channel shape may be triangular, parabolic or trapezoidal. A trapezoidal channel shape is most frequently used because of the ease of its construction. However, all channels will approach a parabolic shape over time and this is the most hydraulically efficient.

Triangular shaped channels should be avoided.

Figure 2.2.3 depicts the various channel shapes and formulae used in determining channel cross-sectional area.

2.4.2 Channel stability

Banks are designed to be grade stable and to operate within specified permissible velocities of flow for both bare soil and vegetated channel conditions.

Bare soil channels are most commonly designed particularly where low peak flows are anticipated and cultivation is expected or encouraged. Table 2.2.3 indicates permissible velocities of flow for bare soil conditions.

Vegetated channels are normally only considered where special circumstances such as high peak discharge prohibits the use of bare soil channels. Vegetated channels will withstand greater velocities of flow and thus the bank cross-sectional area required is reduced. Table 2.2.4 indicates permissible velocities of flow for vegetated channels.

2.4.3 Channel Dimension Considerations

The minimum channel width is often determined by the width of the construction equipment. This width is commonly 3 metres. The increasing trend to wideline farming machinery makes it desirable that the length of all bank batters and the channel bottom width be the same to suit machinery size.

2.5 Bank Design

2.5.1 Graded Banks

The procedure for determining the specifications of graded banks described below is suited to both bare earth and vegetated channels. The procedure also caters for trapezoidal, triangular and parabolic cross sections. The steps are as follows:

1. Determine the discharge using the method described in Chapter 1.

2. Determine from table 2.2.3 or 2.2.4 the maximum permissible velocity.

3. Calculate the required cross-sectional area from the formula.

   $$A = \frac{Q}{V} \qquad \text{(equation 2.2.1)}$$

   Where:
   A = cross sectional area of the channel (m^2)
   Q = peak discharge (m^3/s)
   V = velocity of flow (m/s)

4. Select the desired channel shape (trapezoidal, triangular or parabolic) and from figure 2.2.4 nominate suitable dimensions to allow one of the following equations to be solved

   Trapezoidal channel $B = \frac{A - ZD^2}{D}$ (equation 2.2.2)

   Triangular channel $D = \left(\frac{A}{Z}\right)^{1/2}$ (equation 2.2.3)

   Parabolic channel $T = \frac{A}{0.67D}$ (equation 2.2.4)

   Where:
   B = bottom width (m)
   A = area of channel (m^2)
   Z = batter grade
   D = depth of flow (m)
   T = top width (m)

5. Calculate the wetted perimeter (P) from the equations

   Trapezoidal channel $P = B + 2D\sqrt{Z^2 + 1}$ (equation 2.2.5)

   Triangular channel $P = 2D\sqrt{Z^2 + 1}$ (equation 2.2.6)

   Parabolic channel $P = T + \frac{8D^2}{3T}$ (equation 2.2.7)

6. Calculate hydraulic radius (R)

$$R = \frac{A}{P} \qquad \text{(equation 2.2.8)}$$

7. For a bare earth channel select a suitable roughness coefficient from table 2.2.6.

For a vegetated channel, where the roughness coefficient is a function of vegetal retardance, velocity of flow and hydraulic radius, determine the roughness coefficient from table 2.2.7 and figure 2.2.5.

8. Using figure 2.2.6, or the following equation, solve Manning's equation for slope

$$S = \left(\frac{V_n}{R^{2/3}}\right)^2 \qquad \text{(equation 2.2.9)}$$

<u>Worked Example</u>

Design a graded bank with a vegetated channel. The design discharge is estimated at 5 m^3/sec and the maximum permissible velocity is 1.2 m/sec

1. Cross sectional area $= \frac{5.0}{1.2}$

$= 4.17\ m^2$

2. A trapezoidal shape is selected for the channel. From figure 2.2.4 a batter grade of 6:1 and bank height of 0.8 metres would give a batter length considered reasonable for farming (4.8m). This bank height provides for a depth of flow of 0.55 metres.

3. From equation 2.2.2 calculate bottom width

$$B = \frac{4.17 - 6\ (0.55)^2}{0.55}$$
$$= 4.28\ m$$

4. From equation 2.2.5 calculate the wetted perimeter

$$P = 4.28 + 2 \times 0.55\sqrt{37}$$
$$= 10.97\ m$$

5. From equation 2.2.8 calculate hydraulic radius

$$R = \frac{4.17}{10.97}$$
$$= 0.38\ m$$

6. The degree of retardance is determined from table 2.2.7 as class D (low retardance). With a VR of 0.456 (1.2 x 0.38) the roughness coefficient is determined from figure 2.2.5 as 0.035.

7. From equation 2.2.9 or figure 2.2.6 the required slope of the channel is 0.0065m/m (0.65%).

2.5.2 Level and Absorption Banks

In the design of level and absorption banks the critical steps are estimation of the volume of runoff and the design of a bank of sufficient capacity to store that runoff. The steps are as follows;

(1) Determine the peak discharge as described in Chapter 1

(2) Determine runoff volume as described in section 2.2.5 in Chapter 1.

(3) Calculate the required cross-sectional area from the formula.

$$A = \frac{Qv}{L} \qquad \text{(equation 2.2.10)}$$

where A = cross-sectional area of channel (m^2)

Q_v = volume of runoff (m^3)
L = bank length (m)

(4) From figure 2.2.4 select the desired batter grade and depth of water.

(5) From the appropriate equation 2.2.2, 2.2.3 or 2.2.4 determine the remaining channel dimensions.

Worked Example

Design a level bank with a trapezoidal channel where the peak discharge is calculated at 0.4 m^3/sec and the time of concentration is 20 minutes. The bank length is 500 metres.

(1) Runoff volume = 90 x 20 x 0.4

= 720 m^3

(2) From equation 2.2.10
cross sectional area $= \frac{720}{500}$

$= 1.44\ m^2$

(3) From figure 2.2.4 a depth of water of 0.3 metres and batter grade of 4:1 is selected. This requires a bank height of 0.5 metres.

(4) From equation 2.2.2
Base width $= \frac{1.44 - 4(0.3)^2}{0.3}$

$= 3.6\ m$

Where absorption banks are constructed, the capacity is increased by the water inundating areas above the excavated channel. This storage can be significant on low slopes and, to avoid significant over-design of the bank, it should be added to the channel capacity.

The area of additional storage can be determined from the following equation;

$$A = \frac{50D^2}{S} - \frac{ZD^2}{2} \qquad \text{(equation 2.2.11)}$$

where A = cross-sectional area of additional storage (m^2)
D = depth of water in channel minus the depth of excavation (m)
S = slope of area to be inundated (%)
Z = grade of excavated batter

Worked example

Determine the capacity of an absorption bank with a trapezoidal channel constructed on a 1.5% slope with the following dimensions

Length - 600 metres
Battergrades - 4:1
Depth of water stored in channel - 0.5 m
Depth of excavated channel - 0.3 m
Base width of channel - 3 m

(1) From figure 2.2.3
Area of channel $= 3 \times 0.5 + 4\ (0.5)^2$

$= 2.5\ m^2$

(2) From equation 2.2.11
Area of addition storage $= \frac{50 \times (0.2)^2}{1.5} - \frac{4(0.2)^2}{2}$

$= 1.25 \text{ m}^2$

(3) Total storage area $= 2.5 + 1.25$

$= 3.75 \text{ m}^2$

(4) Storage capacity $= 3.75 \times 600$

$= 2250 \text{ m}^3$

2.6 Bank Outlets

Bank outlets should allow water to spread and move downslope over a wide area. The use of a level sill outlet will spread the outflow. The aim is to prevent water flowing directly downslope in a concentrated stream that is liable to cause rilling. The length of level sill required can be determined from table 2.2.5.

Where banks discharge into stable sections of gullies or adjacent to flow lines, bank outlets are relatively independent of each other.

When a series of banks dispose water onto an unconfined area the top bank should be extended so that runoff does not enter the bank below. This procedure is followed down the slope resulting in a wider spread of runoff and elimination of excessive loading on lower banks.

2.7 Bank Freeboard

Freeboard is needed to enable a bank to function under adverse conditions, such as peak discharges in excess of bank capacity, to allow for bank settlement, other loss of bank height due to stock or machinery movement and increased depth of flow due to vegetal retardance.

A minimum freeboard of 50 per cent of the design depth of flow, with a minimum figure of 0.20 metres, should be added to the designed depth of channel flow.

2.8 Bank Length

The maximum bank length will be governed by the capacity of the bank. Care must be taken to ensure that design discharge does not exceed bank capacity.

Generally, graded banks with a height of 0.6 metres at the recommended spacings have sufficient capacity to allow for lengths up to 1000 metres.

2.9 References

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Schwab, G.O. et al (1981) Soil and Water Conservation Engineering, (3rd edition) J. Wiley and Sons, New York.

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Quilty, J.A. (1972) Soil Conservation Structures for Marginal Arable Areas - Gap Absorption and Gap Spreader Banks, J. Soil Cons. N.S.W. 28 116:130

Table 2.2.1 - Bank Spacing Factors (K) for Specific Localities

Inverell	110
Tamworth	140
Gunnedah	150
Wellington	160
Cowra	180
Wagga	200
Moree	140
Warialda	120

Table 2.2.2 - Method for Estimating Bank Spacing Factor K
(after Logan 1968)

1. Base value of K - select value of closest locality from table 2.2.1.

2. Adjust base value of K by the following:

(i) Soil Erodibility - determine topsoil erodibility using Hamilton's Erodibility Index (E.I.) (Charman 1978)

Erodibility of Topsoil	E.I.	Adjust Base Value K by
Extreme	4.0	- 20%
Very High	3.0 - 4.0	- 10%
High	2.0 - 3.0	No change
Moderate	1.0 - 2.0	+ 15%
Low	1.0	+ 30%

(ii) Existing Erosion

Badly eroded areas................. - 10%
Permanently grassed or stable areas....................... No change

(iii) Land Use

Grazing only....................... + 35%

Cultivation	Cropping frequency	*
Occasional	5	+ 20%
Frequent	3 - 5	+ 10%
Regular	3	No change

(iv) Annual Rainfall

400 mm	....	+ 15%
500 mm	....	+ 10%
600 mm	....	No change
700 mm	....	- 10%
800 mm	....	- 15%

(v) Conservation Farming

Stubble burnt................	- 10%
Stubble incorporated.........	No change
Stubble mulched	+ 10%
No tillage..................	+ 20%

N.B. Estimate K to the nearest 10 units.

* cropping frequency is defined as

$$\frac{\text{length of rotation in years}}{\text{number of years cropped}}$$

Table 2.2.3 - Maximum Permissible Velocities for Bare Soil Channels

Erodibility		Maximum Permissible
Assessment	Index Value	Velocity (m/sec)
Extreme	4.0	0.3
Very High	3.0-4.0	0.4
High	2.0-3.0	0.5
Moderate	1.0-2.0	0.6
Low	0-1.0	0.7

Table 2.2.4 - Maximum Permissible Velocities for Vegetated Channels (m/sec)

Cover Type*	Channel Slope (%)	Erodibility Index Assessment				
		0-1.0	1.0-2.0	2.0-3.0	2.0-4.0	>4.0
Kikuyu and other	0-5	2.6	2.4	2.3	2.2	2.0
dense, high growing,	5-10	2.5	2.3	2.2	2.1	1.9
prostrate perennials	>10	2.4	2.2	2.1	2.0	1.8
Couch and other low	0-5	2.1	2.0	1.9	1.7	1.5
growing, prostrate	5-10	2.0	1.9	1.8	1.6	1.4
perennials	>10	1.9	1.8	1.7	1.5	1.3
Perennial improved	0-5	1.7	1.6	1.4	1.2	1.0
pastures	5-10	1.6	1.5	1.3	1.1	0.9
	>10	1.5	1.4	1.2	1.0	0.8
Native tussocky						
grasses	0-5	1.4	1.2	1.0	0.8	0.6
sparse high growing	5-10	1.3	1.1	0.9	0.7	0.5
legumes self-						
regenerating annuals**						

* The velocities shown for each cover description assume good (i.e. >80 per cent) cover conditions.

** Tussocky grassed slopes of >10 per cent gradient are not recommended for vegetated channels because of the channelising effect such vegetation has on flow conditions.

Table 2.2.5- Sill Lengths For Bank Outlets For Conditions of Low Vegetal Retardance

(After Logan 1968)

(per m^3/s discharge in metres)

Slope below sill	Velocity of Flow ** (m/s)					
%	1.0	1.2	1.5	1.8	2.0	2.5
1	4.0	2.9	1.9	1.4	1.0	0.7
2	5.3	4.0	2.6	1.9	1.5	1.0
3	6.7	4.9	3.2	2.3	1.9	1.3
4	7.1	5.5	3.9	2.8	2.3	1.4
5	8.3	6.4	4.5	3.1	2.6	1.7
6	9.1	6.9	4.8	3.5	2.9	1.9
7	10.0	7.5	5.2	3.7	3.1	2.0
8	11.1	8.3	5.6	4.0	3.3	2.1
10	12.5	9.2	6.7	4.7	3.6	2.4
15	14.3	10.4	7.4	5.6	4.5	3.1
20	16.7	11.9	8.4	7.0	5.6	3.6

** Velocity of flow should be selected from table 2.2.4 and refers to the area onto which the bank discharges.

CHAPTER 2 - BANKS AND WATERWAYS

Table 2.2.6 - Values of Manning's Roughness Coefficient for Natural Channels

Type		Description	Coefficient
A	MINOR STREAM (Surface width of flood stage less than 30m)		
	1.	Fairly regular section	
		(a) Some grass and weeds, little or no brush	0.030-0.035
		(b) Dense growth of weeds, depth of flow significantly greater than weed height	0.035-0.050
		(c) Some weeds, light brush on banks	0.035-0.050
		(d) Some weeds, heavy brush on banks	0.050-0.070
		NOTE: For trees within channel, with branches submerged at high stage, increase above values by 0.010 to 0.020.	
	2.	Irregular sections, with pools, slight channel meander. Increase values given in 1(a)-(d) by 0.010 to 0.020.	
	3.	Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage	
		(a) Gravel bed, cobbles and few boulders	0.040-0.050
		(b) Cobble bed with large boulders	0.050-0.070
B.	MAJOR STREAMS (Surface width at flood stage greater than 30m)		
	The coefficient value is less than that for minor streams of similar description, because banks offer less effective resistance		
		(a) Regular section with no boulders or brush	0.025-0.060
		(b) Irregular and rough section	0.035-0.100
C	FLOOD PLAINS		
	1.	Pasture, no brush	
		(a) Short grass	0.030-0.035
		(b) High grass	0.035-0.050
	2.	Cultivated areas	
		(a) No crop	0.030-0.040
		(b) Mature row crops	0.035-0.045
		(c) Mature field crops	0.040-0.050
	3.	Brush	
		(a) Scattered brush, heavy weeds	0.050-0.070
		(b) Light brush and trees	0.060-0.080
		(c) Medium to dense brush	0.100-0.160
	4.	Trees	
		(a) Clear land with tree stumps, no sprouts	0.040-0.050
		(b) Same as (a) but with heavy growth of sprouts	0.060-0.080
		(c) Heavy stand of timber, little undergrowth, flood stage below branches	0.100-0.120
		(d) Same as above, but with flood stage reaching branches	0.120-0.160

(from Australian Rainfall and Runoff, 1977)

Table 2.2.7 - Classification of Vegetal Cover by Degree of Retardance

Degree of Retardance	Stand	Height of Vegetation
A - Very high retardance	Good	Greater than 75 cm
B - High retardance	(Good (Fair	28 - 60 cm Greater than 75 cm
C - Moderate retardance	(Good (Fair	15 - 25 cm 28 - 60 cm
D - Low retardance	(Good (Fair	5 - 15 cm 5 - 25 cm
E - Very low retardance	All	Less than 5 cm

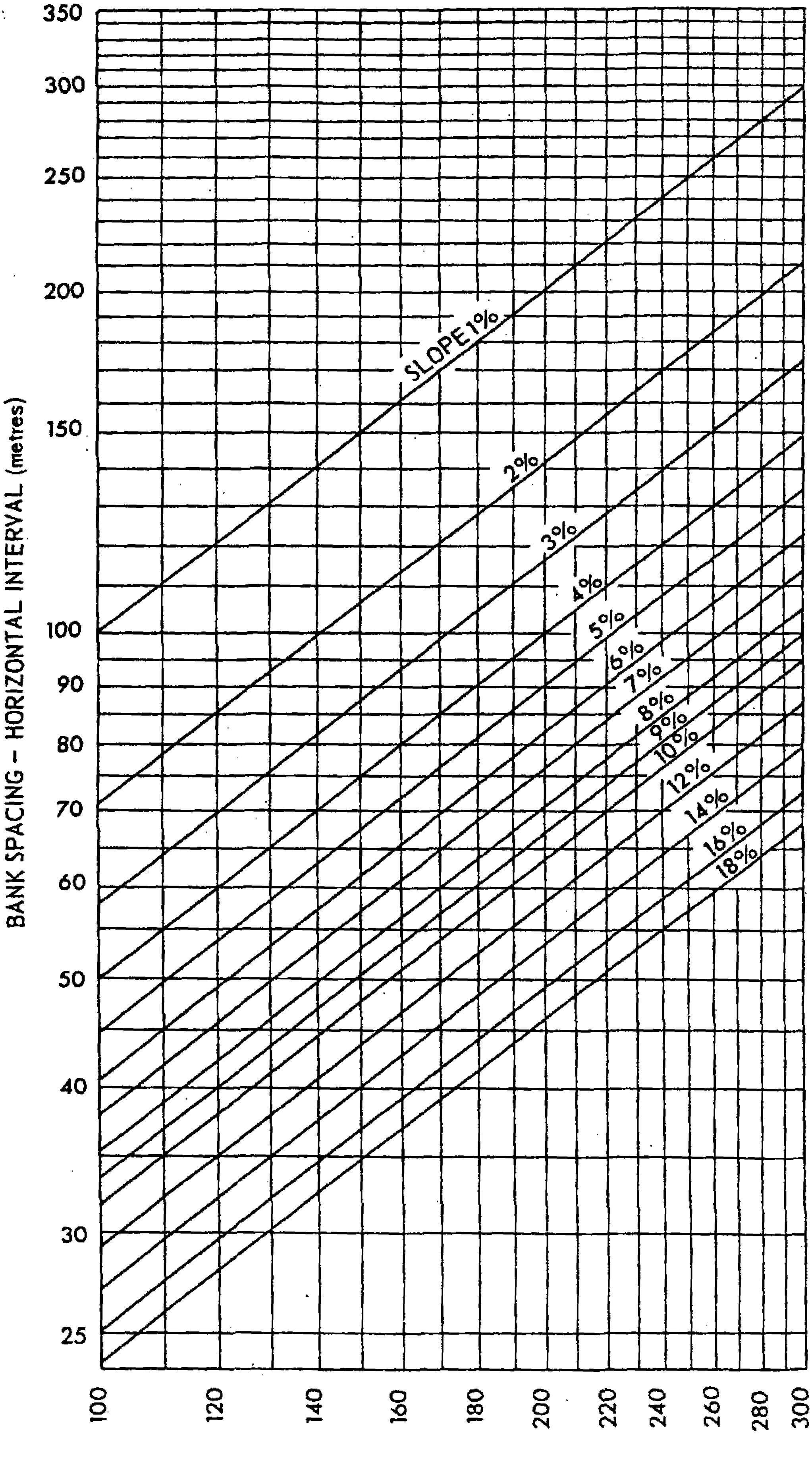


Figure 2:2:1 Horizontal interval for banks for various values of K.

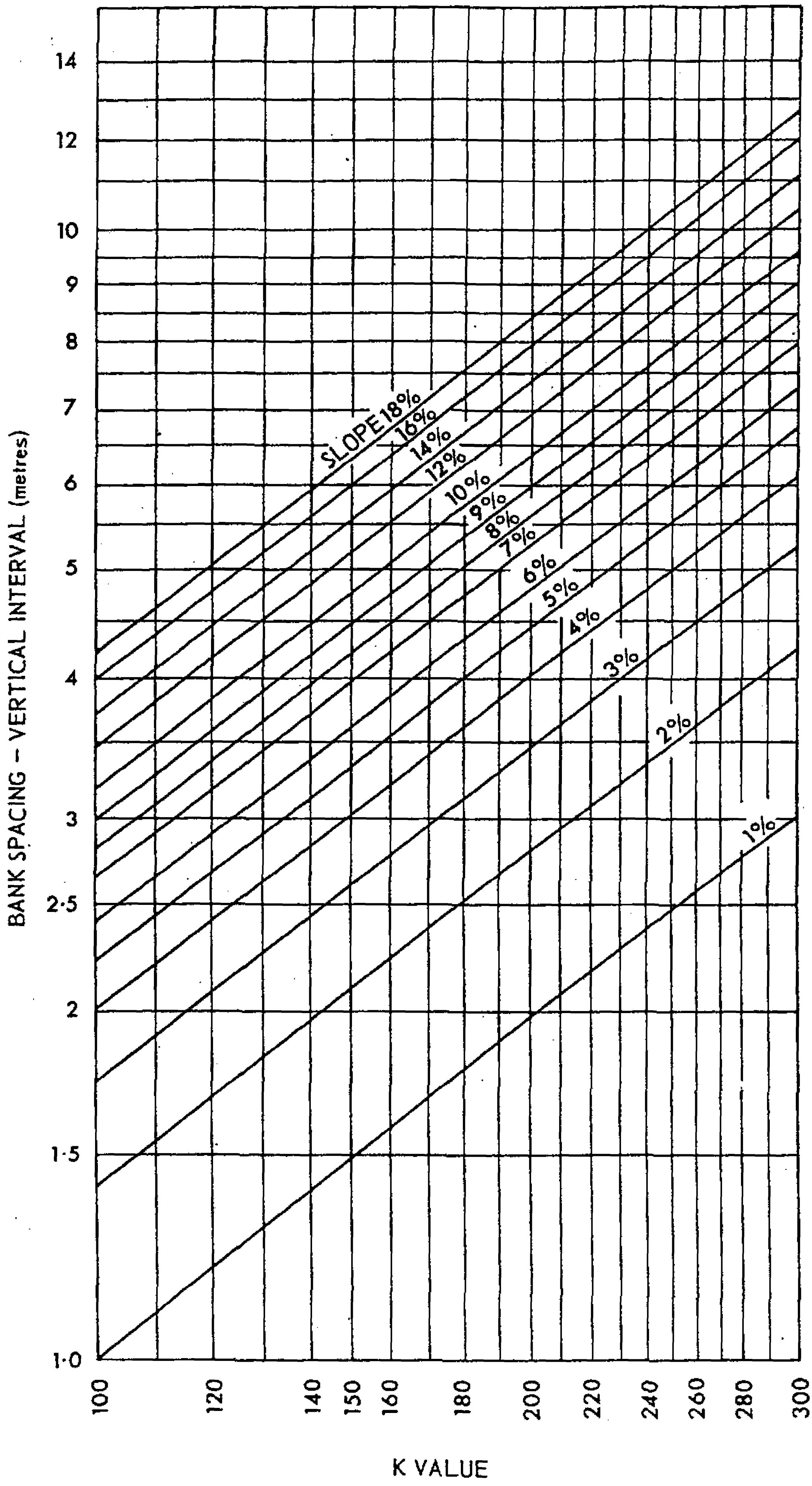
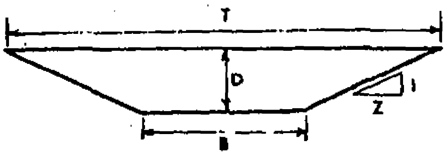
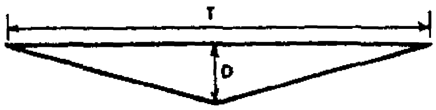


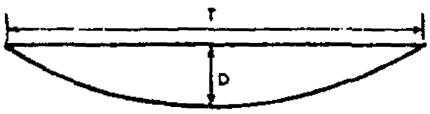
Figure 2·2·2 Vertical interval for banks for various values of K



Tropezoidal Cross Section



Triangular Cross Section



Parabolic Cross Section

	CROSS SECTIONAL AREA A	WETTED PERIMETER P	TOP WIDTH T
Tropezoidal Cross Section	$BD + ZD^2$	$B + 2D\sqrt{Z^2+1}$	$B + 2DZ$
Triangular Cross Section	ZD^2	$2D\sqrt{Z^2+1}$	$2DZ$
Parabolic Cross Section	$\frac{2}{3}TD$	$T + \frac{8D^2}{3T}$	$\frac{A}{0.67D}$

$Z = \frac{T}{4D}$

Figure 2:2:3 Channel cross sections and formulae (Redrawn from Schwab, et. al. 1981).

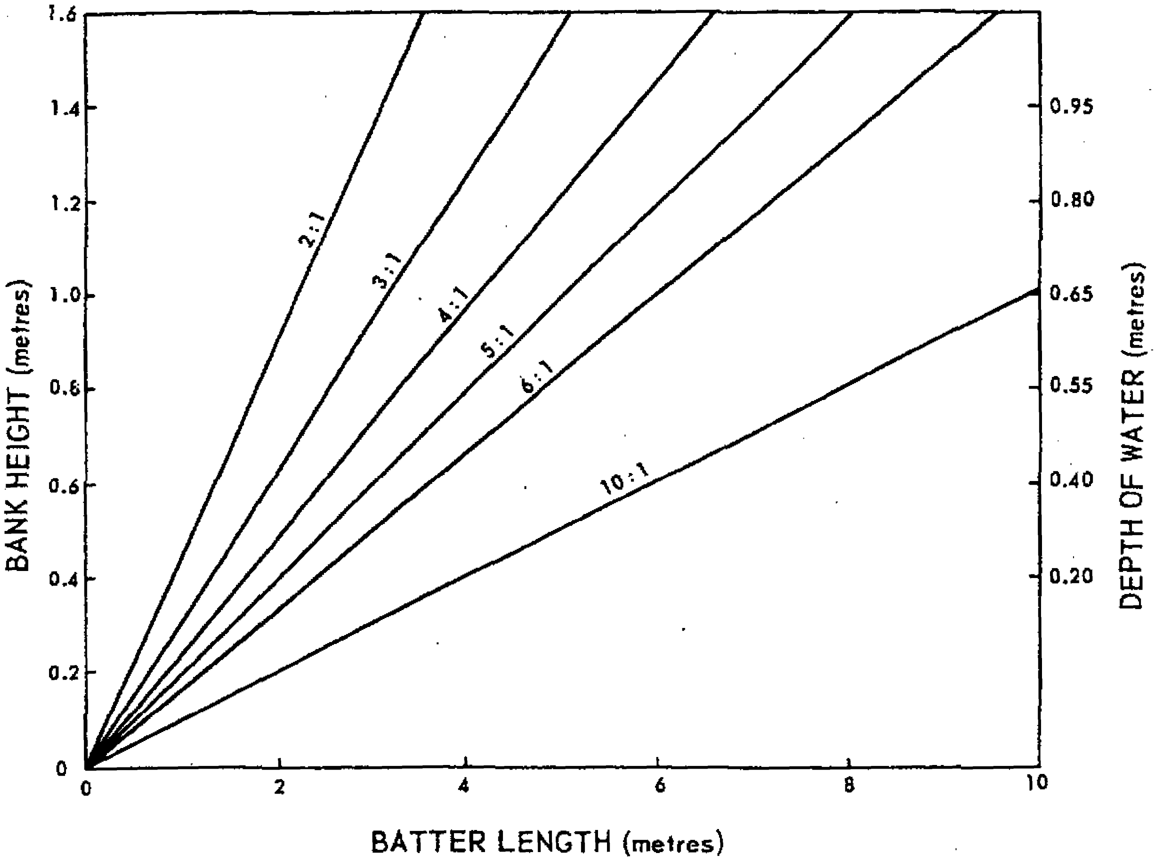


Figure 2:2:4 Selection of suitable bank dimensions.

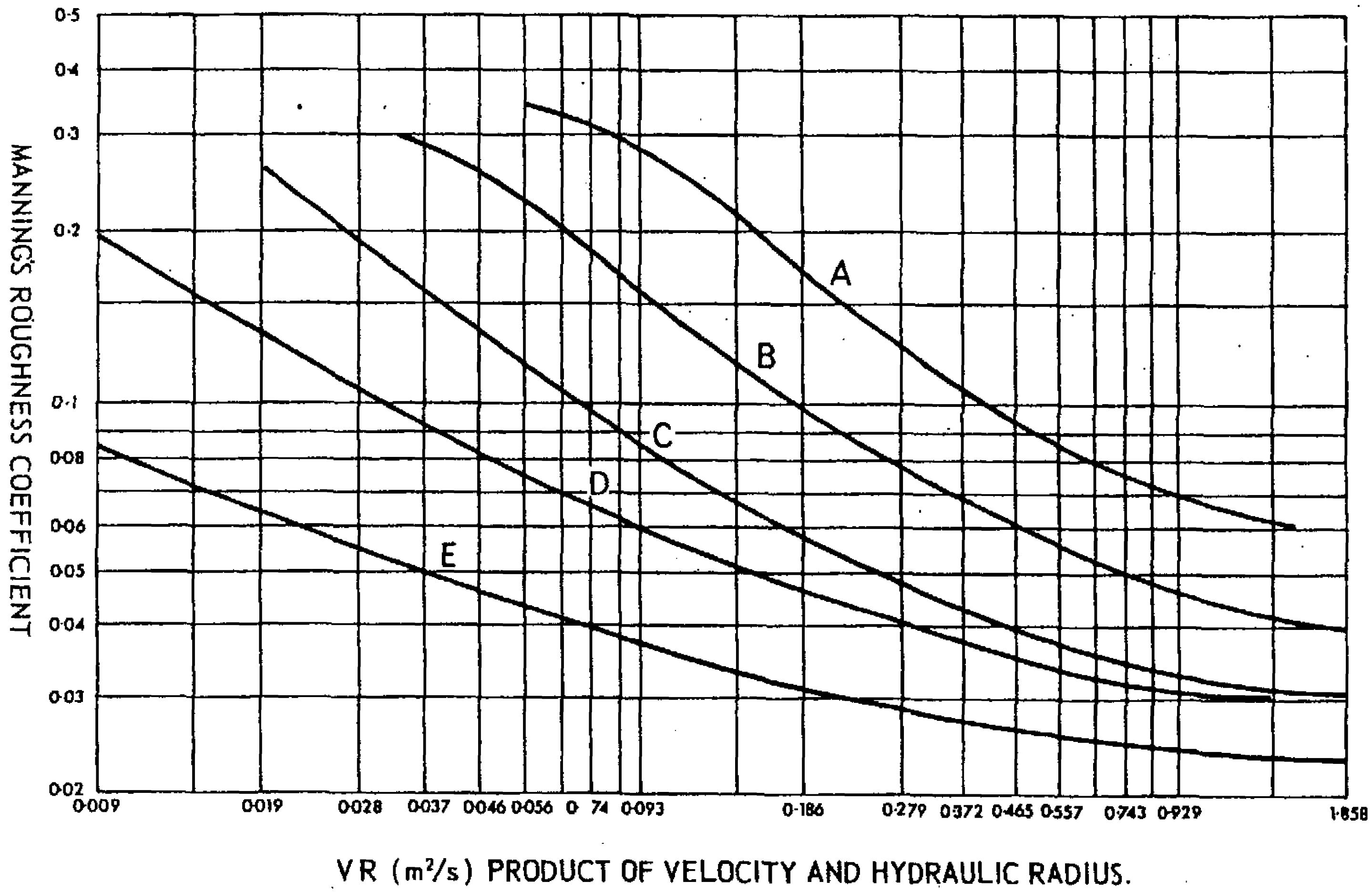


Figure 2·2·5 The behaviour of Manning's 'n' in grassed channels for different degrees of vegetal retardance.

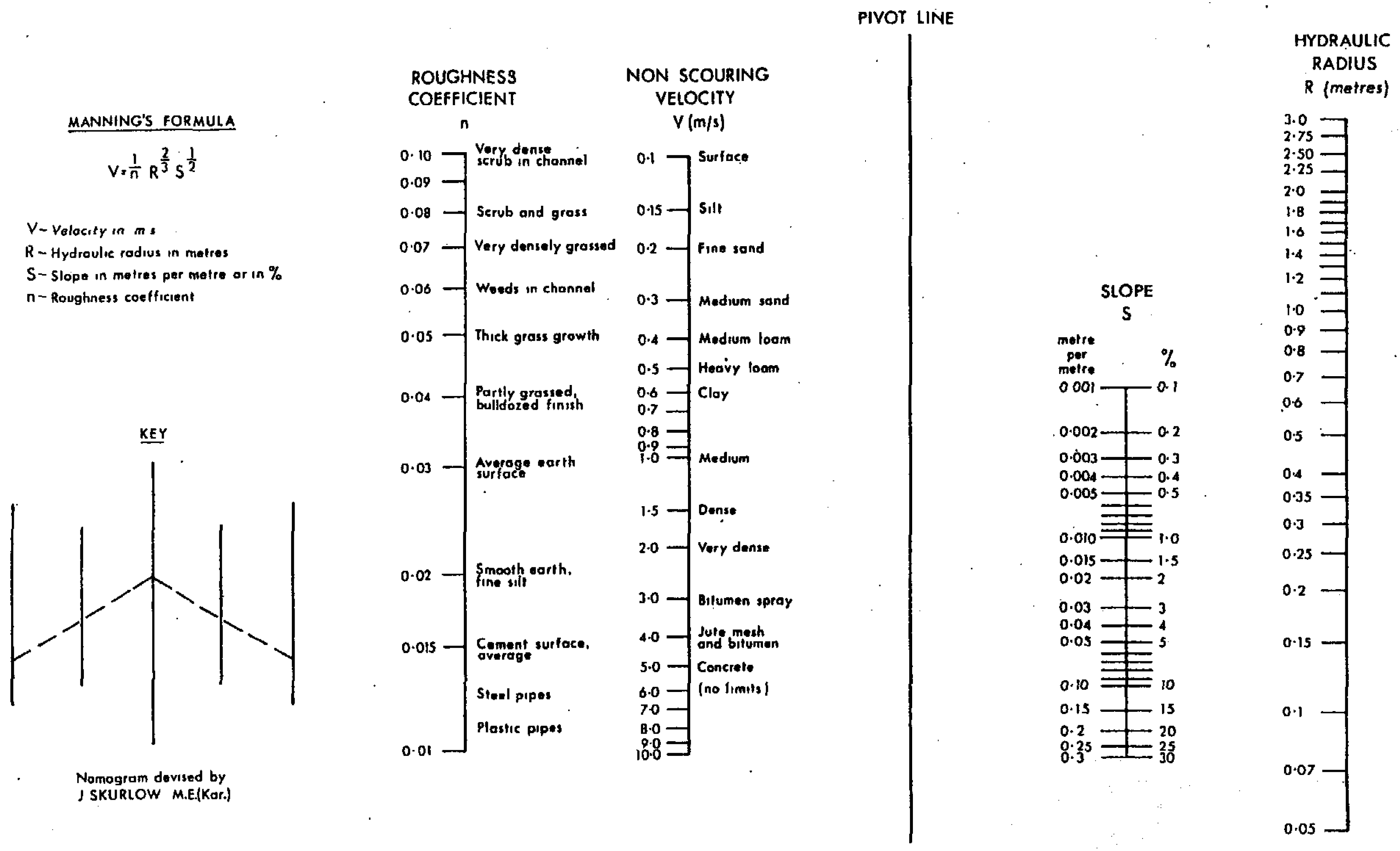


Figure 2:2:6 Nomogram for Manning's formula for velocity of flow in channels and pipes.

Section 3 - Waterways

Section 3 - Waterways

3.1 General

The design of a grassed waterway must satisfy two conditions:

1. The velocity of flow must not exceed the maximum permissible velocity for the slope, soil type and vegetation of the site. Scouring is most likely to occur when vegetation and channel roughness are a minimum.

2. The depth of flow must not exceed the height of the retaining banks. This is most likely to occur when vegetation and channel roughness are at a maximum.

Mannings formula is used to calculate the velocity of flow in open channels. Discussion of this formula and the variation of the roughness coefficient, n, with variation in vegetation is given in section 1 of this Chapter.

To determine the dimensions of a waterway, the following information is required:-

* The design peak discharge of the waterway
* The land slope
* The cross-sectional shape
* The retardance classes for the two roughness conditions
* The maximum permissible velocity of flow

The design peak discharge of the waterway is determined using the method outlined in Chapter 1. The peak discharge into the waterway should be estimated for each bank (or pair of banks) using the cumulative area of the banks to that point. When calculating the peak discharge two factors to be considered are;

1. The design exceedance probability for the waterway should exceed the design exceedence probability for the bank system feeding it.

2. The time of concentration will be the longest flow time to the outlet of the waterway that can be determined .

   It is necessary to calculate and compare times of concentration for each bank and to add the time of travel down the waterway until the outlet is reached. For long waterways, any increase in peak discharge downslope will theoretically result in an increase in the downslope width of the waterway. However it is usual practice to design and construct waterways with a constant width as determined by the calculated discharge at the base of the waterway. Although this may result in a small loss of cultivated land it will facilitate waterway fencing and paddock cultivation.

The natural land slope determines the maximum grade of the waterway. Occasionally, a waterway may be constructed across the natural land slope, this then becomes the waterway slope.

The cross-sectional shape of a waterway is determined by the construction technique. Waterways which have been externally constructed have a trapezoidal cross-sectional shape. Internally constructed waterways can be either trapezoidal or parabolic in cross-sectional shape. Internally constructed waterways are typically used on deep fertile cracking soils to confine at least part of the flow below the ground surface. This technique tends to decrease the risk of waterway failure as a result of cracks forming in the retaining banks. However, the technique must not be used where the exposure of poor quality soil in the excavated channel will inhibit rapid and complete revegetation of the channel.

Parabolic waterways resemble natural channels and usually suffer less damage from trickle flows. Trapezoidal channels tend to revert to a parabolic cross-section.

Selection of cross-sectional shape will dictate which dimensions need to be specified, for example;

(a) for parabolic waterways the depth and top width need to be known;

(b) for trapezoidal waterways the depth and bottom width are required.

The retardance classes for the expected minimum and maximum vegetative cover are required in order to satisfy the basic waterway design conditions and can be determined from table 2.3.1.

The maximum permissible velocity of flow is determined from table 2.3.2.

3.2 Procedure for Location of Waterways

The location of constructed waterways must be given careful consideration. Some standard principles apply which have to be adhered to at all times, however there are other instances where options can be exercised.

3.2.1 Standard Principles:

* Property Treatment: Waterways must be located so as to complement existing or proposed works.

* Bank Length: The position and number of waterways on a property must be such that bank lengths are kept within the limits suitable for the district.

* Top Elevation: Waterways must extend up the slope to a point where they can receive discharge from designed top banks and flow from outside catchments.

* Illegal Diversion: Waterways must be located so that the discharge from them leaves the property at exactly the same natural exit that flows left the property before earthworks were carried out.

* Concentration of Flow: Precautions must be taken when positioning the lower end of a waterway to ensure that undue concentration of discharge onto boundary fences is avoided.

* Cross Slope: Cross slope of a waterway is the difference in height from one side of the waterway to the other measured at right angles to the retaining banks. The ideal situation is nil cross slope. It is essential that cross slope, if any, be kept to a minimum and is within the limits acceptable for the soil type, ground slope, expected depth of flow and vegetative cover in each situation. Generally this figure should not exceed 15 cm.

* Bends: Bends in waterways should be avoided. Where this is not possible they must not be acute. Sharp bends may cause fencing problems and undue stress on the retaining banks and waterway surface.

* Workability: Waterways become a permanent feature of the landscape so their location must be compatible with the management and operation of the property. They should, where possible, be located to suit existing or proposed features such as fences, access roads and watering points. Particular attention is warranted to avoid cultivation bays that are narrow, triangular or have sharp corners.

3.2.2 Other Considerations

When determining the location of waterways, and all other aspects are equal, the following factors should be given consideration.

* Soil Type: Sites should be selected on the most stable and fertile soil types. These provide better opportunities for achieving well vegetated waterway surfaces that are capable of carrying concentrated flows.

* Slope: Preference should be given to lower slopes. This has the advantage of reducing velocity of flow down the waterway and hence, a narrower waterway is needed. However, construction costs will be higher, because of the additional waterway length the low slope option will usually involve.

* Vegetation Conditions: Whenever possible waterways should be located on stable, natural grassed areas rather than cultivated areas.

* Trees: Avoid situations where trees are growing on the proposed waterway surface or where a dense tree population is adjacent to the proposed waterway. Trees on a waterway surface will cause turbulence and most trees will compete with other ground cover for moisture and nutrients.

* Internal Fences: Preference should be given to locating waterways adjacent to or mid-way between existing internal fences. Waterways located adjacent to existing fence lines reduce waterway fencing costs and the native pasture species present along the fence line assists in revegetation of the waterway surface. Waterways located mid-way between existing fences provide equal bays of a workable size on either side of the waterway.

3.3 Design

The solution of Manning's equation for vegetated channels such as waterways is complex.

Where solutions are obtained manually a number of assumptions are made to simplify the calculations. In particular, for broad flat waterway channels the depth of flow is assumed to be equivalent to R, the hydraulic radius. The depth of flow is then easily calculated for a constant slope and roughness co-efficient, given a maximum permissible velocity of flow.

Cross-sectional dimensions of the waterway are subsequently obtained from the relationships:-

$$A = \frac{Q}{V}$$

where A = cross-sectional area (m^2)

Q = design peak discharge (m^3/s)

V = maximum permissible velocity of flow (m/s)

and;

$$W = \frac{A}{D}$$

where W = waterway width (m)

A = cross-sectional area (m^2)

D = depth of flow (m) for the selected vegetation retardance class.

The minimum height of the retaining bank is obtained by adding to the depth of flow an amount equivalent to freeboard. The freeboard allowance is usually 50 per cent of the depth of flow with a minimum figure of 0.20 m.

In practice, the calculation of waterway dimensions is simplified by the use of computers, tables or nomographs and design sheets. Computer programs have been produced by the Soil Conservation Service of N.S.W. (Jackson, 1981) and the Soil Conservation Authority of Victoria (Findlay and Ellul, 1976).

Two procedures for the calculation of waterway dimensions are presented. Firstly, tables from which waterway dimensions can be read directly and secondly, design sheets, which can be used to facilitate the calculation of waterway dimensions are given.

(a) Tabular Procedure

Tables 2.3.3 - 2.3.8 are taken from Findlay (1979) using the principles outlined by Findlay and Ellul (1976). Waterway width and depth are given for a flow of 1.0 m^3/sec. The maximum permissible velocities have been calculated for conditions of minimum retardance. Depth of flow and width of the waterways have been calculated for conditions of maximum retardance. The width required for other flow rates is calculated by multiplying the width in the table by the required flow rate.

Tables are presented for both trapezoidal and parabolic cross-sections, and for three combinations of minimum and maximum vegetative conditions most likely to be encountered by soil conservationists in N.S.W., *viz*,

Minimum E to Maximum C
Minimum D to Maximum C
Minimum D to Maximum B.

The following example demonstrates how to use these tables to design a waterway.

Worked Example

Determine the specifications of a waterway with a parabolic cross-sectional shape, and layout as shown in figure 2.3.1. The land slope is 6 per cent and the soil is of low erodibility (Erodibility Index 0-1.0). A good stand of couch grass will be established and maintained by slashing.

1. From table 2.3.1 the retardance class when couch has been slashed and is, say, 5 cm high is D. The retardance class when the couch may be 15 cm high or higher is C.

2. From table 2.3.2 the maximum permissible velocity for couch grass on low erodible soil on a 6 per cent slope is 2.0 m/s.

3. Using the Statistical Rational Method, the peak discharge at X and Y were calculated to be 3.0 m^3/s and 3.5 m^3/s, respectively.

   Note: The times of travel to point Y for the bank systems A, B and C are less than for bank D. Therefore 40 minutes (bank D) is used as the time of concentration when calculating peak discharge for point Y.

4. Waterway design at X:-

   Using table 2.3.7 (parabolic waterways, D-C retardance classes), horizontally opposite "Grade = 6%" in the column designated "Permissible velocity = 2.0 m/s" find:-

   $$T = 2.9 \text{ m (per 1.0 } m^3/s \text{ of discharge)}$$

   $$D = 0.28 \text{ m}$$

   Multiply the interim top width answer by 3.0 (the calculated discharge at X) to obtain:-

   $$T_x = 8.7 \text{ m}$$

   Therefore a waterway with a parabolic cross-section and top width of 8.7 m will carry a flow of 3.0 m^3/s at a maximum velocity of 2.0 m/s when vegetal retardance is at a minimum, and a depth of 0.28 m when vegetal retardance is at a maximum.

5. Waterway design at Y:-

Using the interim waterway width result already obtained from table 2.3.7, multiply T = 2.9 by 3.5 (the calculated discharge at Y) to obtain:-

$$T_y = 10.2 \text{ m}$$

Therefore a waterway with a parabolic cross-section and top width of 10.2 m will carry a flow of 3.5 m^3/s at a maximum velocity of 2.0 m/s when vegetal retardance is at a minimum, and a depth of 0.28 m when vegetal retardance is at a maximum.

Freeboard must be added to depth to obtain the retainer bank height.

6. Thus the waterway would be built with a parabolic cross-section and top width of 10 m. To safely carry a depth of flow of 0.28 m the minimum retainer bank height would be 0.50 m (0.28 + 0.20).

(b) Design Sheets Procedure

A series of design sheets were designed by Hedberg and Adamson (Soil Conservation Service of N.S.W.) to assist with calculating the dimensions of soil conservation earthworks.

The design sheet for waterways (design sheet D.S:W) is shown together with nomograms (figures 2.3.2 and 2.3.3) for the solution of Mannings formula for conditions of high and low vegetal retardance.

Example

Using the specifications given for the previous waterway example, the use of the design sheets to calculate waterway dimensions is demonstrated on the page headed "Worked Example".

3.4 References

Findlay, G.H.; Ellul, G.A. (1976)

The application of computers to prepare design tables for grassed waterways and flumes. Aust. J. Agric, Eng. 5 20-24.

Findlay, G.H. (1979) Design of structures for drainage and erosion control. In R.J. Garvin, M.R. Knight and T.J. Richmond "Guidelines for Minimising Soil Erosion and Sedimentation from Construction Sites in Victoria". (S.C.A., Kew).

Jackson, L.M. (1981) Programmable calculators for the design of earthworks. Narrabri Soil Cons. Dist. Tech. Bull. No. 81/1.

Table 2.3.1 - Classification of Vegetatal Cover by Degree of Retardance

Degree of Retardance	Stand	Height of Vegetation
A - Very high Retardance	Good	Greater than 75 cm
B - High retardance	(Good	28 - 60 cm
	(Fair	Greater than 75 cm
C - Moderate retardance	(Good	15 - 25 cm
	(Fair	28 - 60 cm
D - Low retardance	(Good	5 - 15 cm
	(Fair	5 - 25 cm
E - Very low retardance	All	Less than 5 cm

Table 2.3.2 - Maximum Permissible Velocities for Vegetated Channels (m/sec)

Cover Type*	Channel Slope (%)	Erodibility Index Assessment				
		0-1.0	1.0-2.0	2.0-3.0	2.0-4.0	>4.0
Kikuyu and other	0-5	2.6	2.4	2.3	2.2	2.0
dense, high growing,	5-10	2.5	2.3	2.2	2.1	1.9
prostrate perennials	>10	2.4	2.2	2.1	2.0	1.8
Couch and other low	0-5	2.1	2.0	1.9	1.7	1.5
growing, prostrate	5-10	2.0	1.9	1.8	1.6	1.4
perennials	>10	1.9	1.8	1.7	1.5	1.3
Perennial improved	0-5	1.7	1.6	1.4	1.2	1.0
pastures	5-10	1.6	1.5	1.3	1.1	0.9
	>10	1.5	1.4	1.2	1.0	0.8
Native tussocky						
grasses	0-5	1.4	1.2	1.0	0.8	0.6
sparse high growing	5-10	1.3	1.1	0.9	0.7	0.5
legumes self-						
regenerating annuals**						

* The velocities shown for each cover description assume good (i.e. >80 per cent) cover conditions.

** Tussocky grassed slopes of >10 per cent gradient are not recommended for vegetated channels because of the channelising effect such vegetation has on flow conditions.

Table 2.3.3 Trapezoidal waterways and chutes: Minimum vegetation 'E' to maximum vegetation 'C'

Grade (%)	W	D	W	D	W	D	W	D	W	D	W	D	W	D	W	D
0.25	3.8	0.43	2.3	0.55	1.5	0.71	0.7	0.92								
0.5	5.6	0.31	3.6	0.38	2.4	0.46	1.4	0.57	0.8	0.68	0.5	0.83				
1.0	8.4	0.22	5.6	0.26	3.9	0.31	2.4	0.37	1.6	0.42	1.2	0.50	0.9	0.57	0.6	0.68
2.0	12.2	0.17	8.3	0.19	5.9	0.21	3.8	0.24	2.6	0.28	2.0	0.32	1.6	0.36	1.2	0.41
4.0	17.4	0.13	12.0	0.14	8.8	0.16	5.7	0.18	4.0	0.20	3.2	0.22	2.7	0.26	2.0	0.30
6.0	22.2	0.11	14.9	0.12	11.1	0.13	7.2	0.14	5.1	0.16	4.1	0.17	3.4	0.19	2.6	0.21
8.0	24.9	0.10	17.3	0.11	12.8	0.12	8.5	0.13	6.0	0.14	4.9	0.15	4.1	0.16	3.1	0.18
10.0	27.9	0.09	19.3	0.10	14.2	0.11	9.6	0.11	6.9	0.12	5.6	0.13	4.7	0.14	3.6	0.16
15.0	34.7	0.08	23.7	0.09	17.5	0.09	11.8	0.10	8.6	0.10	7.0	0.11	5.9	0.12	4.6	0.13
20.0	39.6	0.07	27.2	0.08	20.4	0.08	13.7	0.09	10.0	0.09	8.3	0.09	6.9	0.10	5.4	0.11
25.0	44.0	0.07	30.3	0.07	22.8	0.07	15.5	0.08	11.3	0.08	9.4	0.09	7.8	0.09	6.2	0.10
Permissible velocity (m/s)	0.8		1.0		1.2		1.5		1.8		2.0		2.2		2.5	

W = Width (m). D = Depth of flow (m).

Notes:
1. Widths calculated on a flow rate of 1 m^3/s. For other flow rates, multiply width by required flow rate.
2. Required width has been determined for Type 'E' vegetation.
3. Depth of flow has been determined for Type 'C' vegetation.
4. Add freeboard to depth of flow to obtain required waterway bank height.
5. Based on batter slope 2 horizontal : 1 vertical.

Table 2.3.4 Trapezoidal waterways and chutes: Minimum vegetation 'D' to maximum vegetation 'C'

Grade (%)	W	D	W	D	W	D	W	D	W	D	W	D	W	D	W	D
0.25	2.5	0.56	1.6	0.67	1.0	0.92	0.5	1.22								
0.50	3.7	0.37	2.4	0.45	1.7	0.55	1.0	0.70	0.6	0.90	0.4	1.15				
1.0	5.4	0.27	3.6	0.31	2.5	0.36	1.6	0.45	1.1	0.56	0.8	0.66	0.7	0.76	0.4	0.90
2.0	7.7	0.19	5.3	0.22	3.9	0.25	2.5	0.30	1.8	0.34	1.4	0.40	1.2	0.45	0.8	0.52
4.0	11.0	0.14	7.6	0.16	5.6	0.18	3.8	0.20	2.7	0.23	2.2	0.27	1.8	0.30	1.4	0.34
6.0	13.2	0.12	9.2	0.13	6.8	0.15	4.7	0.16	3.4	0.18	2.8	0.20	2.3	0.22	1.8	0.25
8.0	15.1	0.11	10.6	0.12	7.9	0.13	5.5	0.14	4.1	0.15	3.3	0.17	2.8	0.19	2.1	0.21
10.0	16.8	0.10	11.7	0.11	8.9	0.12	6.1	0.13	4.5	0.14	3.7	0.15	3.2	0.17	2.4	0.18
15.0	19.9	0.09	14.2	0.09	10.7	0.10	7.5	0.11	5.5	0.11	4.6	0.12	3.9	0.13	3.1	0.14
20.0	22.4	0.08	16.1	0.08	12.3	0.09	8.6	0.09	6.4	0.10	5.3	0.11	4.5	0.12	3.6	0.12
25.0	24.8	0.07	17.8	0.08	13.6	0.08	9.6	0.09	7.2	0.09	6.0	0.10	5.1	0.10	4.1	0.11
Permissible velocity (m/s)	0.8		1.0		1.2		1.5		1.8		2.0		2.2		2.5	

W = Width (m). D = Depth of flow (m).

Notes:

1. Widths calculated on a flow rate of 1 m^3/s. For other flow rates, multiply width by required flow rate.
2. Required width has been determined for Type 'D' vegetation.
3. Depth of flow has been determined for Type 'C' vegetation.
4. Add freeboard to depth of flow to obtain required waterway bank height.
5. Based on batter slope 2 horizontal : 1 vertical.

Table 2.3.5 Trapezoidal waterways and chutes: Minimum vegetation 'D' to maximum vegetation 'B'

Grade (%)	W	D	W	D	W	D	W	D	W	D	W	D	W	D	W	D
0.25	2.5	0.64	1.6	0.77	1.0	0.97										
0.50	3.7	0.47	2.4	0.53	1.7	0.63	1.0	0.75								
1.0	5.4	0.35	3.6	0.38	2.5	0.43	1.6	0.49	1.1	0.59	0.8	0.69				
2.0	7.7	0.25	5.3	0.28	3.9	0.31	2.5	0.35	1.8	0.39	1.4	0.43	1.2	0.48	0.8	0.53
4.0	11.0	0.19	7.6	0.21	5.6	0.23	3.8	0.25	2.7	0.27	2.2	0.29	1.8	0.32	1.4	0.35
6.0	13.2	0.17	9.2	0.18	6.8	0.19	4.7	0.21	3.4	0.23	2.8	0.24	2.3	0.26	1.8	0.28
8.0	15.1	0.15	10.6	0.16	7.9	0.17	5.5	0.19	4.1	0.20	3.3	0.21	2.8	0.23	2.1	0.24
10.0	16.8	0.14	11.7	0.15	8.9	0.16	6.1	0.17	4.5	0.18	3.7	0.19	3.2	0.21	2.4	0.22
15.0	19.9	0.13	14.2	0.14	10.7	0.15	7.5	0.16	5.5	0.17	4.6	0.18	3.9	0.19	3.1	0.20
20.0	22.4	0.12	16.1	0.13	12.3	0.14	8.6	0.15	6.4	0.15	5.3	0.16	4.5	0.17	3.6	0.18
25.0	24.8	0.11	17.8	0.12	13.6	0.12	9.6	0.13	7.2	0.14	6.0	0.15	5.1	0.15	4.1	0.16
Permissible velocity (m/s)	0.8		1.0		1.2		1.5		1.8		2.0		2.2		2.5	

W = Width (m). D = Depth of flow (m).

Notes:
1. Widths calculated on a flow rate of 1 m^3/s. For other flow rates, multiply width by required flow rate.
2. Required width has been determined for Type 'D' vegetation.
3. Depth of flow has been determined for Type 'B' vegetation.
4. Add freeboard to depth of flow to obtain required waterway bank height.
5. Based on batter slope 2 horizontal : 1 vertical.

Table 2.3.6 Parabolic waterways: Minimum vegetation 'E' to maximum vegetation 'C'

Grade (%)	T	D	T	D	T	D	T	D	T	D	T	D	T	D	T	D
0.25	4.5	0.63	2.7	0.76	1.8	0.91										
0.50	6.9	0.45	4.3	0.52	3.0	0.61	1.7	0.76	1.1	0.94						
1.0	10.7	0.33	6.6	0.37	4.6	0.42	2.8	0.50	1.9	0.60	1.5	0.67	1.2	0.75	0.8	0.97
2.0	16.2	0.24	10.2	0.27	7.1	0.30	4.4	0.34	3.0	0.40	2.3	0.43	1.9	0.49	1.4	0.55
3.0	20.4	0.21	13.0	0.22	9.1	0.25	5.7	0.28	3.9	0.31	3.1	0.34	2.6	0.38	1.9	0.43
4.0	24.1	0.19	15.4	0.20	10.8	0.22	6.8	0.24	4.7	0.27	3.7	0.29	3.1	0.32	2.3	0.36
6.0	30.0	0.16	19.6	0.17	13.9	0.18	8.7	0.20	6.0	0.22	4.8	0.24	4.0	0.26	3.0	0.28
8.0	35.3	0.15	23.1	0.15	16.4	0.16	10.4	0.18	7.2	0.19	5.7	0.20	4.8	0.22	3.6	0.24
10.0	40.7	0.13	26.2	0.14	18.4	0.15	11.9	0.16	8.3	0.17	6.6	0.18	5.4	0.20	4.1	0.21
Permissible velocity (m/s)	0.8		1.0		1.2		1.5		1.8		2.0		2.2		2.5	

T = Top Width (m). D = Depth of flow (m).

Notes:
1. Widths calculated on a flow rate of 1 m^3/s. For other flow rates, multiply width by required flow rate.
2. Permissible velocities have been determined for Type 'E' vegetation.
3. Widths and depths have been determined for Type 'C' vegetation.
4. Add freeboard to depth of flow to obtain required waterway bank height.

Table 2.3.7 Parabolic waterways: Minimum vegetation 'D' to maximum vegetation 'C'

Grade (%)	T	D	T	D	T	D	T	D	T	D	T	D	T	D	T	D
0.25	2.8	0.77	1.7	0.95	0.9	1.43										
0.50	4.0	0.53	2.6	0.63	1.9	0.75	1.1	0.99								
1.0	6.0	0.38	3.9	0.43	2.8	0.50	1.8	0.61	1.2	0.76	0.9	0.89				
2.0	8.5	0.28	5.7	0.31	4.1	0.35	2.7	0.41	1.9	0.48	1.5	0.53	1.3	0.59	0.9	0.71
3.0	10.5	0.23	7.1	0.26	5.2	0.29	3.4	0.33	2.4	0.38	1.9	0.42	1.6	0.46	1.2	0.53
4.0	12.1	0.21	8.2	0.23	6.0	0.25	4.0	0.28	2.9	0.33	2.3	0.35	1.9	0.38	1.5	0.44
6.0	14.8	0.18	10.1	0.19	7.3	0.21	5.0	0.23	3.7	0.26	2.9	0.28	2.4	0.31	1.9	0.34
8.0	16.8	0.16	11.7	0.17	8.6	0.19	5.8	0.20	4.2	0.22	3.4	0.24	2.8	0.27	2.2	0.29
10.0	19.2	0.15	13.1	0.16	9.8	0.17	6.6	0.18	4.8	0.20	3.9	0.22	3.3	0.24	2.5	0.26
Permissible velocity (m/s)	0.8		1.0		1.2		1.5		1.8		2.0		2.2		2.5	

T = Top Width (m). D = Depth of flow (m).

Notes:

1. Widths calculated on a flow rate of 1 m^3/s. For other flow rates, multiply width by required flow rate.
2. Permissible velocities have been determined for Type 'D' vegetation.
3. Widths and depths have been determined for Type 'C' vegetation.
4. Add freeboard to depth of flow to obtain required waterway bank height.

Table 2.3.8 Parabolic waterways: Minimum vegetation 'D' to maximum vegetation 'B'

Grade (%)	T	D	T	D	T	D	T	D	T	D	T	D	T	D	T	D
0.25	3.2	0.96	1.9	1.15	1.1	1.45										
0.50	4.7	0.70	3.0	0.79	2.0	0.95	1.2	1.12								
1.0	6.9	0.52	4.5	0.57	3.2	0.64	2.0	0.74	1.4	0.88	1.0	1.04				
2.0	10.2	0.38	6.7	0.42	4.9	0.46	3.0	0.52	2.1	0.59	1.6	0.65	1.4	0.72	1.0	0.80
3.0	12.6	0.32	8.3	0.36	6.0	0.39	3.9	0.43	2.7	0.48	2.1	0.52	1.8	0.56	1.3	0.63
4.0	14.5	0.29	9.7	0.32	7.0	0.34	4.6	0.37	3.3	0.41	2.6	0.44	2.1	0.48	1.6	0.53
6.0	17.6	0.25	12.0	0.27	8.7	0.29	5.8	0.31	4.1	0.34	3.3	0.36	2.7	0.39	2.1	0.42
8.0	20.4	0.22	13.9	0.24	10.2	0.26	6.8	0.28	4.9	0.30	3.9	0.32	3.3	0.34	2.5	0.36
10.0	22.4	0.21	15.6	0.22	11.4	0.24	7.7	0.25	5.5	0.27	4.5	0.29	3.7	0.31	2.9	0.33
Permissible velocity (m/s)	0.8		1.0		1.2		1.5		1.8		2.0		2.2		2.5	

T = Top Width (m). D = Depth of flow (m).

Notes:

1. Widths calculated on a flow rate of 1 m^3/s. For other flow rates, *multiply* width by required flow rate.
2. Permissible velocities have been determined for Type 'D' vegetation.
3. Widths and depths have been determined for Type 'B' vegetation.
4. Add freeboard to depth of flow to obtain required waterway bank height.

'WORKED EXAMPLE'

SOIL CONSERVATION SERVICE OF N.S.W.

WATERWAY DESIGN

Design Sheet: D.S: W
Local File No:
R.O. File No:

APPLICANT:

ADDRESS:

WATERWAY No. or LOCATION: Waterway XY

PEAK DISCHARGE (Q)

Waterway Catchment Area = 72 ha Peak Discharge (Q) = 3·5 m^3/s (1)

VELOCITY (V)

Vegetation cover on Waterway is couch ; Waterway slope is 6 %

From Table 2:3:2 maximum permissible VELOCITY = 2·0 m/s (2)

HYDRAULIC RADIUS (R) – For Waterways R = D (depth of flow)

a) Low Retardance (R_L). Where from D.S:W.2 V = 2·0 m/s and waterway slope is 6 %

Then from Figure 2:3:2 – LOW RETARDANCE DEPTH (D) is 0·18 m (3)

b) High Retardance (R_H). Where from D.S:W.2 V = 2·0 m/s and waterway slope is 6 %

Then from Figure 2:3:3 – HIGH RETARDANCE DEPTH (D) is 0·3 m (4)

WATERWAY BANK HEIGHT (Minimum)

From D.S:W.4 High Retardance Depth = 0·3 m + Freeboard (generally 50%: 0·2m minimum)

Then MINIMUM WATERWAY BANK HEIGHT = 0·50 m

WATERWAY CROSS-SECTIONAL AREA (A) from $A = \frac{Q}{V}$

From D.S:W.1 where Q = 3·5 m^3/s and from D.S:W.2 where V = 2·0 m/s

∴ WATERWAY CROSS-SECTIONAL AREA (A) = 1·75 m^2 (5)

WATERWAY WIDTH (W) (with Low Vegetal Retardance) = $\frac{A}{D}$ (Low Retardance)

From D.S:W.5 where A = 1·75 m^2 and from D.S:W3 where Low Retardance Depth = 0·18 m

Then WATERWAY WIDTH (W) = 9·7 m

COMMENTS: Waterway to be fenced

WATERWAY SPECIFICATIONS

WIDTH	=	10 m
WATERWAY BANK HEIGHT (min)	=	0·50 m
SLOPE	=	6 %
VEGETATION COVER		couch

Prepared by:

Date:

SOIL CONSERVATION SERVICE OF N.S.W.

WATERWAY DESIGN

Design Sheet: D.S: W
Local File No: ______
R.O. File No: ______

APPLICANT: ______

ADDRESS: ______

WATERWAY No. or LOCATION: ______

PEAK DISCHARGE (Q)

Waterway Catchment Area = ______ ha Peak Discharge (Q) = ______ m^3/s (1)

VELOCITY (V)

Vegetation cover on Waterway is ______ ; Waterway slope is ______ %

From Table 2:3:2 maximum permissible VELOCITY = ______ m/s (2)

HYDRAULIC RADIUS (R) – For Waterways R = D (depth of flow)

a) Low Retardance (R_L). Where from D.S:W.2 V = ____ m/s and waterway slope is ______ %

Then from Figure 2:3:2 – LOW RETARDANCE DEPTH (D) is ______ m (3)

b) High Retardance (R_H). Where from D.S:W.2 V = ____ m/s and waterway slope is ______ %

Then from Figure 2:3:3 – HIGH RETARDANCE DEPTH (D) is ______ m (4)

WATERWAY BANK HEIGHT (Minimum)

From D.S:W.4 High Retardance Depth = ______ m + Freeboard (generally 50%: 0·2 m minimum)

Then MINIMUM WATERWAY BANK HEIGHT = ______ m

WATERWAY CROSS-SECTIONAL AREA (A) from $A = \frac{Q}{V}$

From D.S:W.1 where Q = ____ m^3/s and from D.S:W.2 where V = ____ m/s

∴ WATERWAY CROSS-SECTIONAL AREA (A) = ______ m^2 (5)

WATERWAY WIDTH (W) (with Low Vegetal Retardance) = $\frac{A}{D}$ (Low Retardance)

From D.S:W.5 where A = ____ m^2 and from D.S:W3 where Low Retardance Depth = ____ m

Then WATERWAY WIDTH (W) = ______ m

COMMENTS: ______

______

______

WATERWAY SPECIFICATIONS

WIDTH	=	______ m
WATERWAY BANK HEIGHT (min)	=	______ m
SLOPE	=	______ %
VEGETATION COVER		______

Prepared by: ______

Date: ______

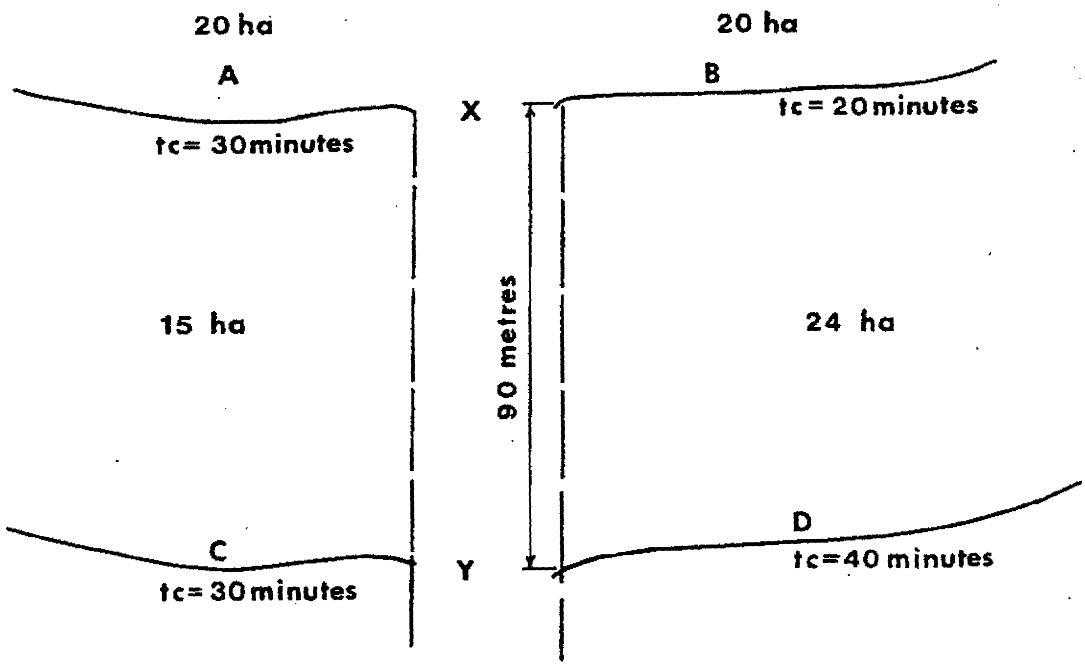


Figure 2·3·1 Waterway and bank layout - worked example in text.

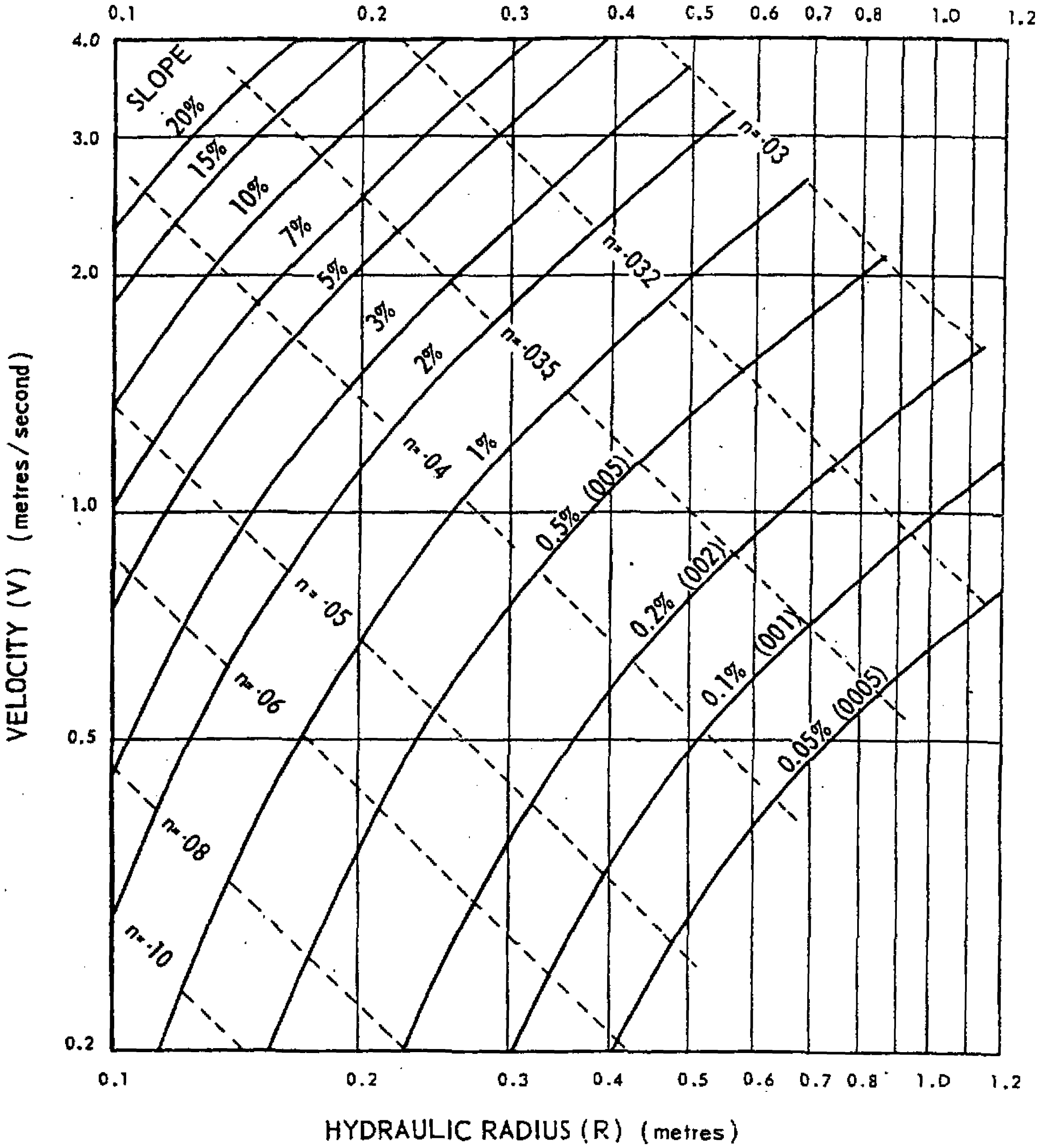


Figure 2·3·2 Solution of Mannings formula for vegetated channels of low vegetal retardance.
(good stand, 5-15 cm high or fair stand, 5-25 cm high)

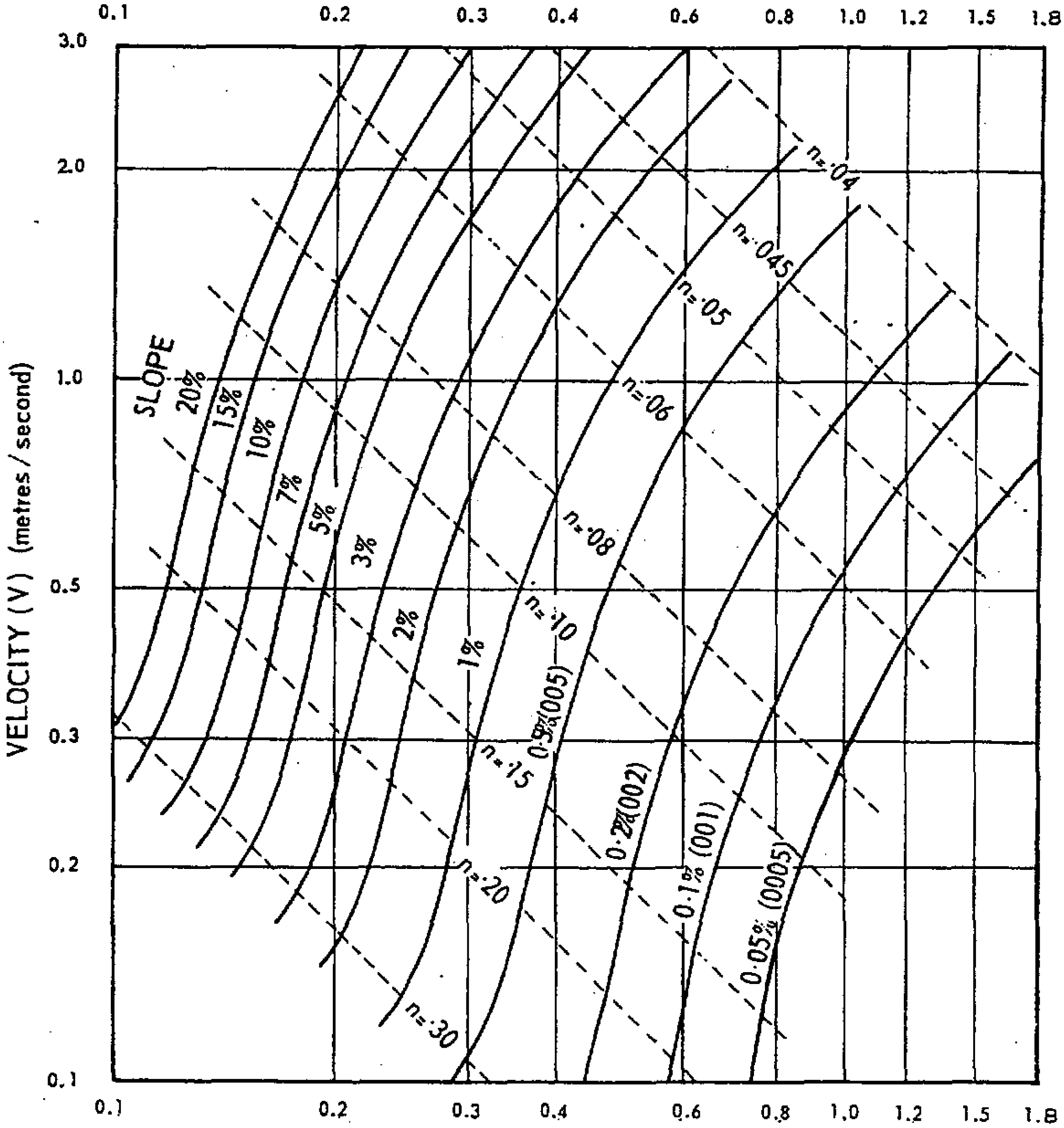


Figure 2:3:3 Solution of Mannings formula for vegetated channels of high vegetal retardance.
(good stand, 27-30 cm high or fair stand, 30 cm high)

CHAPTER 3 - DAMS FOR EROSION CONTROL

A dam is a barrier, embankment or excavated earth structure, generally built in or near a flowline, which has the primary function of impounding water for storage. The stored water is used for such purposes as stock watering, domestic supply irrigation and firefighting.

Dams used for water conservation on the farm are normally built of compacted soil and are called earth dams.

Dams may also be used in conjunction with the diversion of water, creation of hydraulic head, sediment retention or for soil erosion control. When their primary function is for gully erosion control the term gully control structure is used. When their primary function is for sediment retention the term sediment basin is used.

3.1 Consideration in the Location of Dams for Erosion Control

As with all storage structures the storage ratio (ratio of volume of water stored to volume of earth moved) is important. Optimum ratios are achieved with minimum volume walls on low slope sites. The higher the ratio the cheaper the cost per unit of stored water. However, as the structures are primarily for erosion control, a number of other factors need to be taken into account when locations are considered.

3.1.1 Location of Gully Head

In long gullies, location of a dam actually in the gully itself may be the only option available. Where there is a definite gully head with overfall erosion extending the gully upslope, the structure is frequently sited to drown the overfall. In such cases, advantage is taken of potential storage capacity in the gully.

Such a site may, however, have disadvantages. The length of time taken to prepare the wall area may be such that any advantages are outweighed. It may be difficult to obtain sufficient earth for the wall and much of the available material may be unsuitable. Too great a depth of water stored against the wall may result and this is highly undesirable because this will increase the risk of structure failure especially in unsuitable soil.

A structure located above a gully head keeps runoff away from the overfall which can then be battered down to a stable slope. Any excess runoff may be taken away by a diversion bank or dropped safely into the bottom of the gully by means of a flume. Whether the structure is located above or below an overfall will depend on the relative advantages and disadvantages mentioned above previously.

3.1.2 Location away from a Gully

In some instances it is preferable to locate the structure to one side of the gully or flow line and divert all or part of the flow into it. This may be so where the catchment is large and where difficulty would be experienced in obtaining sufficient strength in the wall or sufficient spillway capacity. In other places the gully may have such a high silt load that the life of a structure would be very limited if placed in the gully.

3.1.3 Soil Type

The extent to which the soil at a proposed structure site should be sampled as an aid to locating the structure depends on the variability of the soil.

All soil variability at the proposed site must also be accounted for prior to construction. Decisions can then be made as to the general suitability of the soil for the proposed structure and the placement of the soil within the embankment.

It is important to understand that a sample is only a minute representation of the proposed embankment. Therefore the samples taken must be as representative as possible. A suitable method of site sampling is described by Tregenza (1970). Ideally sampling should be done on an intensive grid basis, but this is usually impractical.

As a general rule if 2 or 3 holes reveal a relatively uniform soil type, then only one site need be chosen for sampling. If the three holes reveal three different soil materials then further holes should be dug until the full variability of the soil is known.

Sampling should include all soil horizons to be used as construction material. This will normally include the upper and lower B horizons and where necessary the C horizon.

3.2 Dimensions of Dams for Erosion Control

3.2.1 Capacity

(a) Square and Rectangular Structures

The best way of estimating the capacity of square and rectangular structures is to use the following formula:

$$V = \frac{H}{6} (T + B + 4M)$$

Where : V = capacity (m^3)
H = height or depth (m)
T = top area (m^2)
B = bottom area (m^2)
M = middle area (m^2)

(b) Dams in a Gully

The capacity of these structures can be determined then by carrying out a volume check on completion of construction or by drawing the structure on a contour plan of the site.

The method used by the Service involves calculation of the capacity from a contour survey of the structure, as described by Booth (1963).

3.2.2 Freeboard

Freeboard is the difference in height between the spillway level and the embankment crest. It must be sufficient to prevent overtopping under conditions of design surcharge and wave action.

Freeboard = Surcharge + Wave Action + Clearance

a) Surcharge

Surcharge is the depth to which water, stored as temporary flood storage, rises above the top water level whilst the spillway is discharging, that is, water stored above the outlet of the spillway (figure 3.1).

Surcharge depends on the nature of the inflow hydrograph, storage depth characteristics of the structure and the hydraulic characteristics. The location of the spillway is also important.

b) Wave Action

Wave action is due to the influence of winds and is dependent on the fetch. When the fetch is more than 600 metres and especially when it lies in the direction of prevailing winds consideration should be given to special forms of protection.

Figure 3.5 may be used to calculate wave action.

c) Clearance

Clearance is a standard dimension to provide a margin of safety and shall be not less than 0.5 metres.

d) Settlement

Settlement results from the consolidation of the material within the embankment.

If the settlement is not considered in the design of a embankment, hydraulic failure of the structure may occur due to decrease in the designed freeboard.

The following figures give a general guide for a gross settlement allowance in small earth embankments i.e., both dry and saturation settlements (Charman, 1978).

Rolled Fill	5%
Scraper placed without rolling	8%
Dozer placed without rolling	10%

The majority of small dams for erosion control require a one metre freeboard which includes a surcharge allowance of between 300-500 mm.

Allowance for settlement should be made during construction, with the construction crest being the appropriate percentage higher than the design crest.

After settlement the structure should then have a level crest.

3.2.4 Embankments

(a) Embankment Dimensions

The top of the embankment must have sufficient strength to prevent the structure failing when it is filled to the surcharge level.

Top width can be determined from the following formula:

$$T = \frac{H}{5} + 1.5$$

Where
T = Top width (m)
H = Settled height of embankment (m)

Recommended top widths are given in table 3.1

Base width of the embankment can be determined by the following formula:

$$B = T + H\,(X + Y)$$

Where:
B = Base width (m)
T = Top width (m)
H = Height of embankment (m)
X = Upstream batter 1:X
Y = Downstream batter 1:Y

Base widths for various heights of embankments with 3:1 upstream and 2.5:1 downstream batters are given in table 3.2.

(b) Embankment Batters

The slope of the embankment batters depends primarily on the depth of water stored and the type of construction material.

For a *homogeneous* embankment the batters recommended are:-

Height of Stored Water	Slopes	Soil Classification							
		GC	GM	SC	SM	CL	ML	CH	MH
0 - 3 m	Upstream Downstream	2.5:1 2:1	2.5:1 2:1	2.5:1 2:1	2.5:1 2:1	2.5:1 2:1	2.5:1 2:1	3:1 2.5:1	3:1 2.5:
3 - 6 m	Upstream Downstream	2.5:1 2.5:1	2.5:1 2.5:1	2.5:1 2.5:1	2.5:1 2.5:1	2.5:1 2.5:1	2.5:1 2.5:1	3:1 3:1	3:1 3:1
6 - 10 m	Upstream Downstream	3:1 2.5:1	3:1 2.5:1	3:1 3:1	3:1 3:1	3:1 3:1	3:1 3:1	3.5:1 3:1	3.5: 3:

If storage of water is the object of the structure, materials classified as GW, GP, SW and SP should not be used. Organic material such as OH, OL and Pt soils should not be use in any structure.

The above soils have been classified in accordance with t Unified Soil Classification System:-

GW - Well graded gravels
GP - Poorly graded gravels
GM - Silty gravels
GC - Clayey gravels
SW - Well graded sands
SP - Poorly graded sands
SM - Silty sands
SC - Clayey sands
ML - Inorganic silts with low to medium plasticity
CL - Inorganic clay of low to medium plasticity
MH - Inorganic silt of high plasticity
CH - Inorganic clay of high plasticity
OL - Organic silt of low to medium plasticity
OH - Organic clay of high plasticity
Pt - Peat and highly organic soils

Notes:-

(i) The above batters are for embankments on stable foundations.

(ii) If the embankment is to be constructed on steep slopes consideration should be given to flattenin the downstream batter.

(iii) Where embankments are likely to have a portion of the downstream batter submerged by floodwaters, batters not steeper than 3:1 should be used.

(iv) When a mixture of various materials is proposed, the flatter batters indicated by the above table should be adopted.

(c) Embankment Volume

For a structure with a straight embankment, an upstream batter of 3:1 and a downstream batter of 2:1 the volume of earth in the embankment is given by the following formula, which has been metricated from the formula given by Booth and Leet (1966).

$$V = 0.1755\ H\ (2B + L)\ (5.5H + 2T)$$

For straight walled structures with an upstream batter of 3:1 and downstream batter of 2.5:1 the formula becomes:

$$V = 0.1755\ H\ (2B + L)\ (5H + 2T)$$

and where both upstream and downstream batters are 3:1 the formula is:

$$V = 0.1755\ H\ (2B + L)\ (6H + 2T)$$

Where:

V = Volume of earth in the embankment (m^3)

H = Height of the embankment (m)

T = Top width of the embankment (m)

L = Length of the embankment at the top (m)

B = Length of the embankment at the base (m)

These formulae are presented graphically in figures 3.2, 3.3 and 3.4 for an embankment top width of 2.5 metres and range of batter grades.

For hillside structures with three sided embankments the only way to accurately determine the volume of earth in the embankment is by measurement following construction.

An approximation of the volume of earth is given by the following formula (Burton, 1965):

$$V = 1/3\ L_2\ B_2 + 1/3\ L_3 + B_3 + B_1 + L_1$$

Where:

V = Volume of earth in the embankment (m^3)
L_1 = Length of the front embankment (m)

B = Maximum cross-sectional area of the front embankment (m^2)

L_2 & L_3 = Length of the side embankments (m)

B_2 & B_3 = Maximum cross-sectional area of the side embankments (m)

3.3 References

Booth, I.C. (1963) Technical Notes on Surveying and Mapping. Part VI Estimation of Capacity, J. Soil Cons. N.S.W. 19 59:68.

Booth, I.C.; Leet, N.T. (1966) Technical Notes on Surveying and Mapping. Part VIII Estimation of Volume in Walls of Gully Control Structures, J. Soil Cons. N.S.W. 22 77:85

Burton, J.R. (1965) Water Storage on the Farm, Bulletin No. 9, Vol. 1. - Water Research Foundation of Australia.

Charman, P.E.V. (1978) Soils of New South Wales: Their Characterisation, Classification and Conservation, S.C.S. of N.S.W., Sydney Tech. Handbook No. 1.

Redford, J. Design of Storages for Farm Water Supplies, Technical Manual No. 3. Wat Resources Commission, N.S.W.

Tregenza, G. (1970) Sampling for Soil Conservation Earthworks, Technical Memorandum, Gunnedah Research Centre.

Table 3.1 - Recommended Top Widths for Earth Embankments of Soil Conservation Dams

Height (m)	Top Width (m)
Up to 3	2.1
3.1 - 4	2.3
4.1 - 5	2.5
5.1 - 6	2.7
6.1 - 7	2.9
7.1 - 8	3.1
8.1 - 9	3.3
9.1 - 10	3.5
10.1 - 11	3.7
11.1 - 12	3.9

Table 3.2 - Base Dimensions for Embankments with 3:1 Upstream and 2.5:1 Downstream Batters

Height	Base Width (m)	Distance from centre of embankment (m)	
		Upstream(m)	Downstream(m)
2	13.5	7.25	6.25
3	19.5	10.50	9.00
4	25.5	13.75	11.75
5	31.5	17.00	14.50
6	37.2	20.10	17.10
7	43.0	23.25	19.75
8	49.0	26.50	22.50
9	54.7	29.60	25.10
10	60.5	32.75	27.75

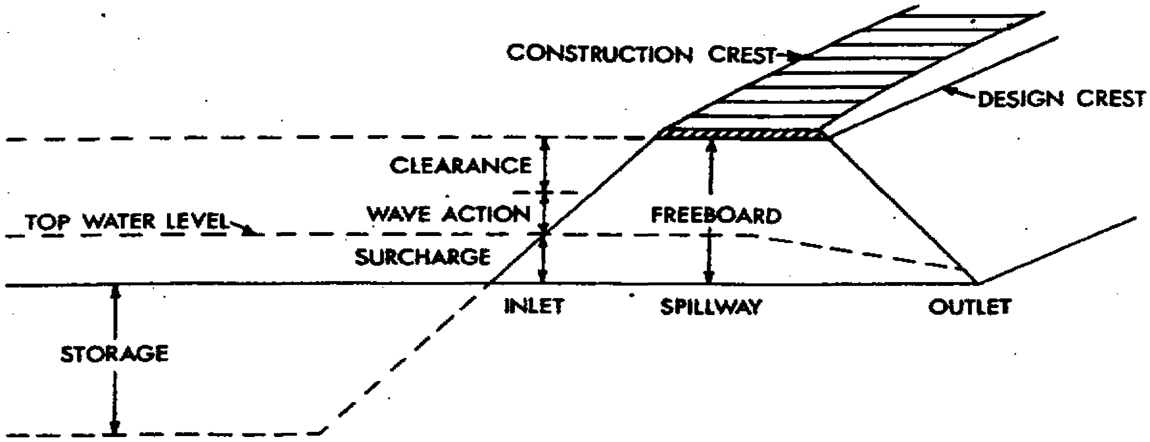


Figure 3:1 Diagram showing freeboard in an earth dam.

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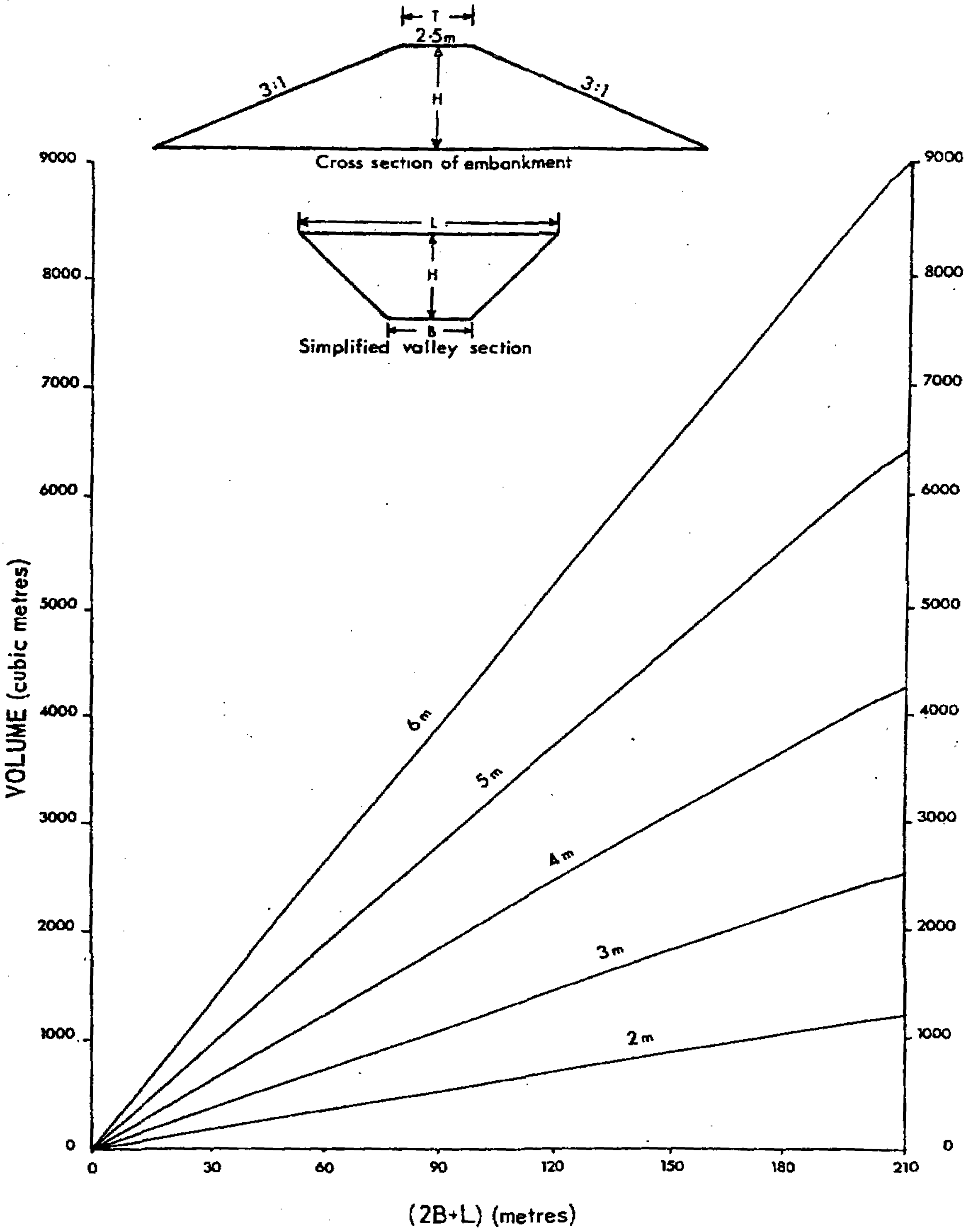


Figure 3:2 Estimation of volume in the embankment of a soil conservation dam.

T- 2·5m Batter - U/S 3:1 D/S 3:1

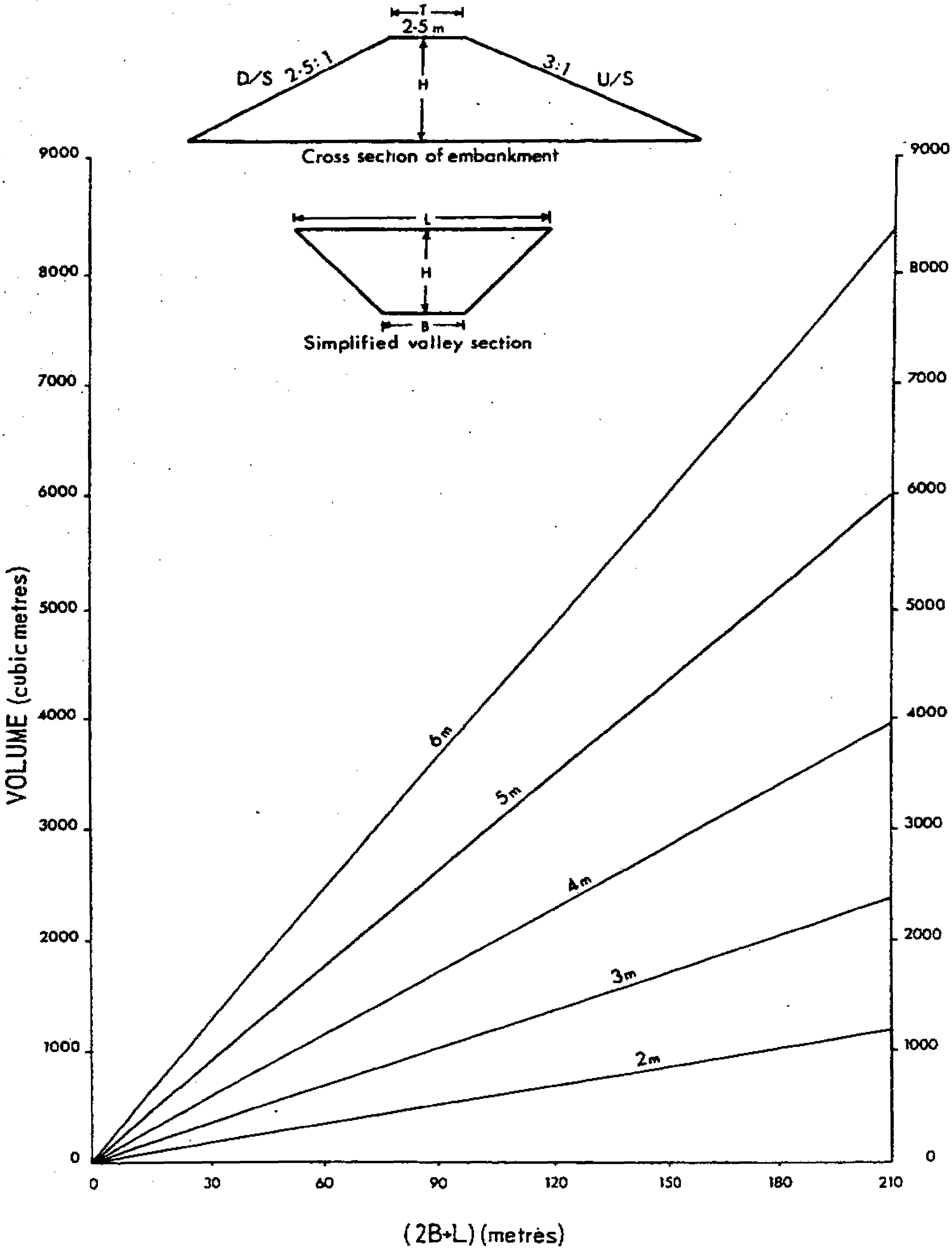


Figure 3·3 Estimation of volume in the embankment of a soil conservation dam.

T = 2·5m Batter = U/S 3:1 D/S 2·5:1

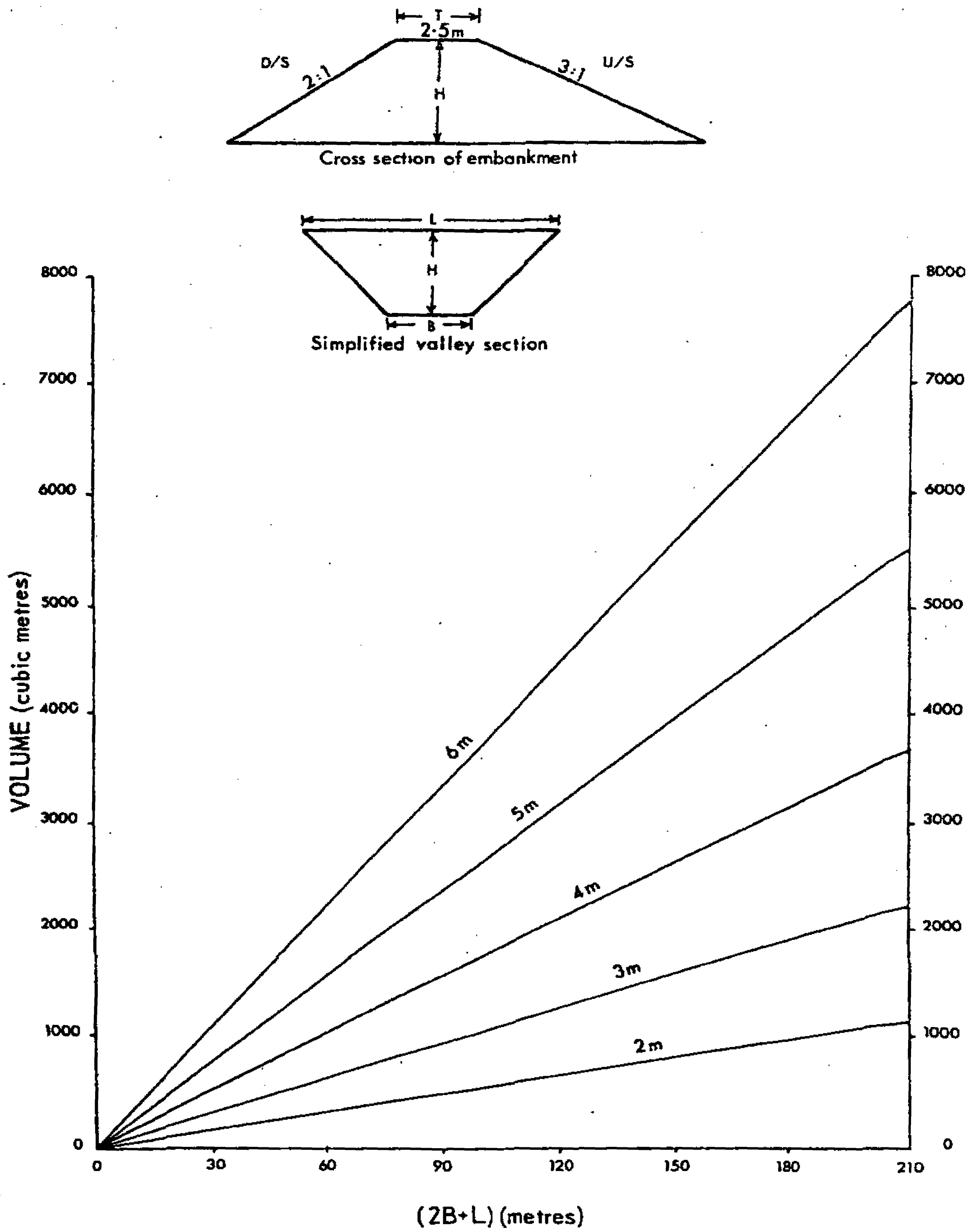
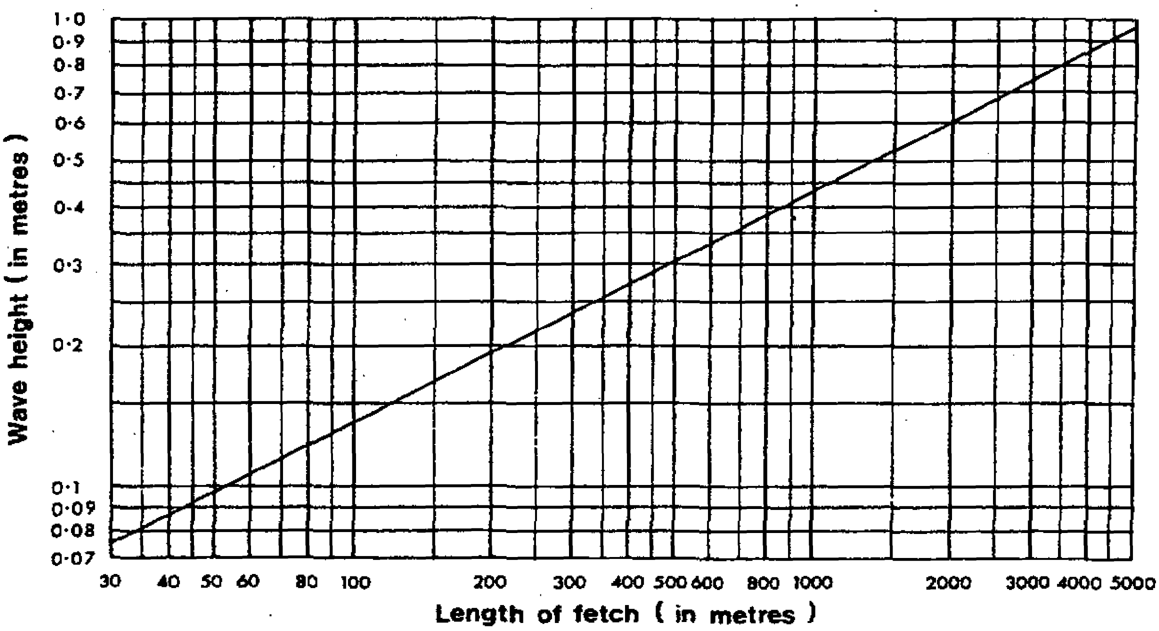


Figure 3·4 Estimation of volume in the embankment of a soil conservation dam.

T=2·5m Batter = U/S 3:1 D/S 2:1



From Hawksley's formula
$H = 0{\cdot}0138\sqrt{L}$ metres

Figure 3:5 Estimation of wave height for earth dams.

CHAPTER 4 - SPILLWAYS

Section 1 - Grass Spillways

Section 1 - Grass Spillways

An adequate spillway is essential to ensure the outlet stability of soil conservation banks and dams, and to safely convey water to lower levels.

Design of grass spillways requires knowledge of the peak design flow, which is usually obtained using the Statistical Rational Method (Chapter 1). If the spillway is designed for the outlet of a dam, the dam is assumed to be full at the time of the design flow.

Two tables are presented for spillway designers. Table 4.1.1 provides outlet widths for a range of flows, assuming a maximum surcharge depth of 0.5 m, and a maximum permissible flow velocity of 2.5 m/s.

Table 4.1.2 provides sill lengths at the spillway outlet for a range of flows and downstream slopes (return slopes).

The dimensions given in table 4.1.1 and table 4.1.2 are for well grassed conditions. In other cases spillways must be wider.

Sill length should not be more than 1.5 times the dam outlet width. If this occurs then adjust the dam outlet width.

For example, if the downstream slope is greater than 16%, and the design flow is $6 m^3/s$, then the sill length would need to be 30 metres, and the first estimate of dam outlet width would be 11 metres. In this case the dam outlet width should be increased to 20 metres.

Table 4.1.1 - Recommended Outlet Widths - Grass Spillways

Discharge (m^3/s)	Outlet width (m)
Up to 3	5.5
4	7.5
5	9.0
6	11.0
7	12.5
8	14.5
9	16.5
10	18.5
11	20.0
12	22.0
13	23.5
14	25.5
15	27.5

Table 4.1.2 - Recommended Sill Lengths - Grass Spillways

Discharge (m^3/s)	Downstream slope (m)										
	24%	22%	20%	18%	16%	14%	12%	10%	8%	6%	4% or less
Up to 3	20	19	18	16	15	13	12	10	9	7	6
4	27	25	23	22	20	18	16	14	12	9	8
5	34	31	29	27	25	22	20	17	14	11	10
6	40	38	35	32	30	27	24	21	17	14	12
7	47	44	41	38	35	31	28	24	20	16	14
8	54	50	47	43	39	36	32	28	23	19	16
9	60	56	53	49	44	40	36	31	26	21	17
10	67	63	58	54	49	45	40	35	29	23	19
11	74	69	64	59	54	49	44	38	32	26	22
12	80	75	70	65	59	54	48	41	35	28	24
13	87	81	76	70	64	58	52	45	38	30	26
14	94	88	82	75	69	62	56	48	41	33	28
15	100	94	87	81	74	67	59	52	44	35	30

Section 2 - Pipe Spillways.

Section 2 - Pipe Spillways.

The benefit of dams for soil conservation lies in their ability to temporarily store storm runoff, and reduce the peak flow rate from a catchment. However the available temporary storage in a dam may be quite small if the dam is almost full when storm runoff occurs.

A larger temporary storage volume may be guaranteed by installing a pipe through the embankment to reduce the permanent storage level of the dam to the inlet level of the pipe. The pipe must then be designed to carry flows for storm events with annual exceedance probabilities (A.E.P) up to the design A.E.P of the pipe.

Pipe flow is determined by pipe diameter, pipe length, the pressure difference between the pipe inlet and outlet, the roughness of the pipe walls, as well as a range of other factors including inlet geometry or pipe bends.

The number of factors involved in determining pipe flow makes pipe design very complex. A very simplified procedure is presented in this Section.

2.1 Design

Tables 4.2.1 to 4.2.4 present flow rates for concrete, asbestos cement, plastic and corrugated metal pipes, for a range of pipe lengths (L) and pressure heads (H) and pipe diameters (D).

The pressure head is the difference between the upstream water level and the downstream water level if the downstream end of the pipe is submerged.

If the pipe is discharging into free air, the head is the difference between the upstream water level and the pipe outlet level.

2.2 - Worked Example

A soil conservation dam with the dimensions shown in figure 4.2.1 is designed for a maximum discharge of 0.5 m^3/s. Determine the pipe diameter required to convey the design flow. Assume the pipe to be concrete.

From Table 4.2.1

(1) Select a 375 mm diameter pipe - Capacity = 0.517 m^3/s: This is greater than 0.50 m^3/s so,

(2) Select a 300 mm diameter pipe - Capacity = 0.309 m^3/s: This capacity is too small so,

The correct pipe size is 375 mm.

Table 4.2.1 - Flow Rates (m^3/s) Through Concrete Pipes

Pipe Diameter (D) (mm)	100	150	225	300	375	450	525	600	675	750	825	900
(L) = 10 m												
(H) = 1 m	0.016	0.042	0.108	0.207	0.338	0.501	0.697	0.925	1.186	1.479	1.803	2.160
2	0.023	0.060	0.153	0.293	0.478	0.709	0.986	1.309	1.677	2.091	2.551	3.056
3	0.028	0.073	0.188	0.359	0.586	0.869	1.208	1.603	2.054	2.561	3.124	3.743
4	0.032	0.085	0.217	0.414	0.676	1.003	1.395	1.851	2.372	2.958	3.608	4.322
5	0.036	0.095	0.243	0.463	0.756	1.122	1.560	2.070	2.653	3.307	4.034	4.832
6	0.039	0.104	0.266	0.507	0.828	1.229	1.709	2.268	2.906	3.623	4.419	5.293
7	0.042	0.112	0.287	0.548	0.895	1.327	1.846	2.449	3.139	3.913	4.773	5.718
8	0.045	0.120	0.307	0.586	0.956	1.419	1.973	2.619	3.355	4.183	5.102	6.112
(L) = 20 m												
(H) = 1 m	0.012	0.034	0.091	0.178	0.298	0.450	0.635	0.852	1.101	1.383	1.697	2.044
2	0.017	0.048	0.128	0.252	0.422	0.637	0.898	1.205	1.558	1.956	2.401	2.891
3	0.021	0.059	0.157	0.309	0.517	0.780	1.100	1.476	1.908	2.396	2.941	3.541
4	0.025	0.068	0.181	0.357	0.597	0.901	1.270	1.705	2.203	2.767	3.396	4.089
5	0.028	0.076	0.203	0.399	0.667	1.008	1.421	1.906	2.464	3.094	3.797	4.572
6	0.030	0.083	0.222	0.438	0.731	1.104	1.556	2.088	2.699	3.390	4.159	5.008
7	0.033	0.090	0.240	0.473	0.790	1.193	1.681	2.255	2.915	3.661	4.493	5.410
8	0.035	0.096	0.257	0.505	0.844	1.275	1.797	2.411	3.117	3.914	4.803	5.783
(L) = 40 m												
(H) = 1 m	0.009	0.026	0.071	0.145	0.248	0.382	0.548	0.745	0.975	1.237	1.531	1.857
2	0.013	0.037	0.101	0.205	0.351	0.541	0.775	1.055	1.379	1.750	2.166	2.628
3	0.016	0.045	0.124	0.251	0.430	0.663	0.950	1.292	1.690	2.143	2.653	3.219
4	0.018	0.052	0.143	0.290	0.497	0.765	1.097	1.492	1.951	2.475	3.064	3.718
5	0.020	0.058	0.160	0.325	0.556	0.856	1.226	1.668	2.182	2.768	3.426	4.157
6	0.022	0.063	0.176	0.356	0.609	0.938	1.344	1.828	2.391	3.032	3.753	4.554
7	0.024	0.069	0.190	0.384	0.658	1.013	1.451	1.974	2.582	3.275	4.054	4.919
8	0.026	0.073	0.203	0.411	0.703	1.083	1.552	2.111	2.761	3.502	4.334	5.259

Table 4.2.1 - Flow Rates (m^3/s) Through Concrete Pipes - Continued

Pipe Diameter (D) (mm)	100	150	225	300	375	450	525	600	675	750	825	900
(L) = 70 m												
(H) = 1 m	0.007	0.020	0.057	0.118	0.205	0.321	0.466	0.641	0.847	1.084	1.353	1.654
2	0.010	0.029	0.081	0.167	0.291	0.454	0.659	0.907	1.199	1.534	1.915	2.340
3	0.012	0.035	0.099	0.205	0.356	0.556	0.808	1.111	1.468	1.880	2.346	2.867
4	0.014	0.041	0.115	0.236	0.411	0.643	0.933	1.284	1.696	2.171	2.709	3.311
5	0.016	0.045	0.128	0.264	0.460	0.719	1.043	1.435	1.897	2.428	3.030	3.703
6	0.017	0.050	0.140	0.290	0.504	0.787	1.143	1.573	2.078	2.660	3.319	4.056
7	0.019	0.054	0.152	0.313	0.544	0.851	1.235	1.699	2.245	2.873	3.585	4.382
8	0.020	0.057	0.162	0.335	0.582	0.909	1.320	1.816	2.400	3.072	3.833	4.685
(L) = 100 m												
(H) = 1 m	0.006	0.017	0.049	0.102	0.179	0.282	0.412	0.570	0.759	0.977	1.225	1.505
2	0.008	0.024	0.069	0.144	0.253	0.399	0.583	0.808	1.074	1.383	1.734	2.130
3	0.010	0.030	0.085	0.177	0.310	0.489	0.715	0.990	1.316	1.694	2.125	2.610
4	0.012	0.034	0.098	0.204	0.359	0.565	0.826	1.143	1.520	1.957	2.455	3.014
5	0.013	0.038	0.110	0.229	0.401	0.631	0.923	1.279	1.700	2.188	2.745	3.371
6	0.015	0.042	0.120	0.250	0.439	0.692	1.012	1.401	1.862	2.397	3.007	3.693
7	0.016	0.046	0.130	0.271	0.475	0.747	1.093	1.513	2.012	2.590	3.249	3.989
8	0.017	0.049	0.139	0.289	0.508	0.799	1.168	1.618	2.151	2.774	3.471	4.264

(L) = Pipe Length

Table 4.2.2 - Flow Rates (m^3/s) Through Asbestos Cement Pipes

Pipe Diameter (mm)		100	150	225	300	375	450	525	600
(H) = Pressure Head									
(L) = Pipe Length									
L = 10 m	(H) = 1 m	0.014	0.039	0.102	0.197	0.325	0.486	0.679	0.904
	2	0.020	0.055	0.144	0.279	0.460	0.687	0.960	1.279
	3	0.025	0.067	0.177	0.342	0.563	0.842	0.176	0.567
	4	0.029	0.078	0.204	0.395	0.651	0.972	1.358	1.809
	5	0.032	0.087	0.228	0.441	0.727	1.087	1.518	2.023
	6	0.035	0.095	0.250	0.483	0.797	1.190	1.663	2.216
	7	0.038	0.103	0.270	0.522	0.861	1.286	1.797	2.393
	8	0.041	0.110	0.289	0.558	0.920	1.374	1.921	2.559
L = 20 m	(H) = 1 m	0.011	0.030	0.083	0.166	0.281	0.428	0.608	0.820
	2	0.015	0.043	0.118	0.236	0.398	0.606	0.860	1.159
	3	0.019	0.053	0.144	0.289	0.487	0.742	1.053	1.420
	4	0.022	0.061	0.167	0.333	0.563	0.857	1.216	1.640
	5	0.024	0.068	0.186	0.373	0.629	0.958	1.360	1.834
	6	0.027	0.075	0.204	0.408	0.689	1.050	1.489	2.009
	7	0.029	0.081	0.221	0.441	0.745	1.134	1.609	2.170
	8	0.031	0.086	0.236	0.471	0.796	1.212	1.720	2.319
L = 40 m	(H) = 1 m	0.008	0.023	0.065	0.133	0.229	0.356	0.514	0.704
	2	0.011	0.032	0.091	0.188	0.324	0.504	0.727	0.995
	3	0.014	0.040	0.112	0.230	0.397	0.617	0.891	1.219
	4	0.016	0.046	0.129	0.265	0.459	0.713	1.029	1.408
	5	0.018	0.051	0.145	0.297	0.513	0.797	1.150	1.574
	6	0.020	0.056	0.158	0.325	0.562	0.873	1.260	1.725
	7	0.021	0.061	0.171	0.351	0.607	0.943	1.361	1.863
	8	0.023	0.065	0.183	0.375	0.649	1.008	1.455	1.992
L = 70 m	(H) = 1 m	0.006	0.018	0.051	0.106	0.187	0.294	0.430	0.596
	2	0.009	0.025	0.072	0.151	0.265	0.417	0.609	0.843
	3	0.011	0.031	0.088	0.185	0.324	0.511	0.746	1.032
	4	0.012	0.036	0.102	0.213	0.374	0.590	0.862	1.192
	5	0.014	0.040	0.114	0.239	0.419	0.659	0.963	1.333
	6	0.015	0.044	0.125	0.261	0.459	0.722	1.055	1.460
	7	0.016	0.047	0.135	0.282	0.496	0.780	1.140	1.578
	8	0.017	0.050	0.145	0.302	0.530	0.834	1.219	1.687
L = 100 m	(H) = 1 m	0.005	0.015	0.043	0.091	0.162	0.257	0.377	0.526
	2	0.007	0.021	0.062	0.129	0.229	0.363	0.534	0.744
	3	0.009	0.026	0.075	0.159	0.281	0.445	0.654	0.911
	4	0.010	0.030	0.087	0.183	0.324	0.514	0.756	1.053
	5	0.012	0.034	0.097	0.205	0.362	0.575	0.845	1.177
	6	0.013	0.037	0.107	0.225	0.397	0.630	0.926	1.289
	7	0.014	0.040	0.115	0.243	0.429	0.680	1.000	1.393
	8	0.015	0.043	0.123	0.259	0.459	0.727	1.069	1.489

Table 4.2.3 - Flow Rates (m^3/s) in Plastic Pipes

(H) = Pressure Head
(L) = Pipe Length

Pipe Diameter (mm)		80	100	150	175	195	225	300
L = 10 m	(H) = 1 m	0.010	0.017	0.044	0.064	0.081	0.112	0.213
	2	0.014	0.024	0.063	0.090	0.115	0.159	0.301
	3	0.017	0.030	0.077	0.110	0.141	0.195	0.369
	4	0.020	0.034	0.089	0.127	0.163	0.225	0.426
	5	0.022	0.038	0.100	0.142	0.182	0.252	0.476
	6	0.024	0.042	0.109	0.156	0.200	0.276	0.522
	7	0.026	0.045	0.118	0.169	0.216	0.298	0.564
	8	0.028	0.048	0.126	0.180	0.231	0.319	0.602
L = 20 m	(H) = 1 m	0.008	0.013	0.036	0.052	0.068	0.095	0.186
	2	0.011	0.019	0.051	0.074	0.096	0.135	0.263
	3	0.013	0.023	0.063	0.091	0.118	0.165	0.323
	4	0.015	0.027	0.072	0.105	0.136	0.191	0.373
	5	0.017	0.030	0.081	0.118	0.152	0.214	0.417
	6	0.019	0.033	0.089	0.129	0.167	0.234	0.457
	7	0.020	0.035	0.096	0.139	0.180	0.253	0.493
	8	0.022	0.038	0.103	0.149	0.193	0.270	0.527
L = 40 m	(H) = 1 m	0.006	0.010	0.028	0.041	0.053	0.076	0.153
	2	0.008	0.014	0.039	0.058	0.076	0.108	0.217
	3	0.010	0.017	0.048	0.071	0.093	0.132	0.266
	4	0.011	0.020	0.056	0.082	0.107	0.153	0.307
	5	0.013	0.022	0.062	0.092	0.120	0.171	0.344
	6	0.014	0.024	0.068	0.101	0.132	0.187	0.377
	7	0.015	0.026	0.074	0.109	0.142	0.203	0.407
	8	0.016	0.028	0.079	0.116	0.152	0.217	0.435
L = 70 m	(H) = 1 m	0.004	0.008	0.022	0.032	0.043	0.061	0.126
	2	0.006	0.011	0.031	0.046	0.061	0.087	0.178
	3	0.007	0.013	0.038	0.056	0.074	0.107	0.218
	4	0.009	0.015	0.044	0.065	0.086	0.123	0.252
	5	0.010	0.017	0.049	0.073	0.096	0.138	0.282
	6	0.011	0.019	0.054	0.080	0.105	0.151	0.309
	7	0.011	0.020	0.058	0.086	0.114	0.163	0.334
	8	0.012	0.022	0.062	0.092	0.122	0.175	0.358
L = 100 m	(H) = 1 m	0.004	0.006	0.019	0.028	0.036	0.053	0.109
	2	0.005	0.009	0.026	0.039	0.052	0.075	0.155
	3	0.006	0.011	0.032	0.048	0.064	0.092	0.190
	4	0.007	0.013	0.037	0.056	0.073	0.106	0.219
	5	0.008	0.015	0.042	0.062	0.082	0.118	0.245
	6	0.009	0.016	0.046	0.068	0.090	0.130	0.269
	7	0.010	0.017	0.050	0.074	0.097	0.140	0.290
	8	0.010	0.019	0.053	0.079	0.104	0.150	0.311

Table 4.2.4 - Flow Rates (m^3/s) in Corrugated Metal Pipe

(H) = Pressure Head
(L) = Pipe Length

Pipe Diameter (mm)	(H)	300	375	450	600	900	1200	1500	1800
L = 10 m	(H) = 10 m 1 m	0.133	0.236	0.370	0.739	1.876	3.542	5.732	8.441
	2	0.189	0.333	0.523	1.045	2.654	5.010	8.106	11.937
	3	0.231	0.408	0.641	1.280	3.250	6.136	9.928	14.620
	4	0.267	0.471	0.740	1.478	3.753	7.085	11.464	16.882
	5	0.298	0.527	0.828	1.652	4.196	7.921	12.817	18.875
	6	0.327	0.577	0.907	1.810	4.596	8.677	14.040	20.676
	7	0.353	0.623	0.979	1.955	4.964	9.372	15.165	22.333
	8	0.377	0.666	1.047	2.090	5.307	10.020	16.212	23.875
L = 20 m	(H) = 1 m	0.102	0.183	0.294	0.608	1.624	3.169	5.243	7.844
	2	0.144	0.259	0.416	0.860	2.297	4.482	7.415	11.093
	3	0.176	0.317	0.509	1.053	2.814	5.489	9.082	13.586
	4	0.203	0.366	0.588	1.216	3.249	6.339	10.487	15.688
	5	0.227	0.410	0.657	1.360	3.632	7.087	11.725	17.540
	6	0.249	0.449	0.720	1.489	3.979	7.763	12.844	19.214
	7	0.269	0.485	0.778	1.609	4.298	8.385	13.873	20.753
	8	0.287	0.518	0.831	1.720	4.594	8.964	14.831	22.186
L = 40 m	(H) = 1	0.075	0.137	0.223	0.474	1.326	2.679	4.553	6.952
	2	0.106	0.194	0.315	0.670	1.875	3.789	6.439	9.832
	3	0.130	0.237	0.386	0.821	2.296	4.641	7.886	12.042
	4	0.150	0.274	0.446	0.948	2.652	5.359	9.106	13.905
	5	0.167	0.306	0.499	1.060	2.965	5.992	10.181	15.546
	6	0.183	0.336	0.546	1.161	3.248	6.563	11.153	17.030
	7	0.198	0.362	0.590	1.254	3.508	7.089	12.046	18.395
	8	0.212	0.387	0.631	1.340	3.750	7.579	12.878	19.665
L = 70 m	(H) = 1 m	0.058	0.106	0.174	0.376	1.082	2.242	3.891	6.048
	2	0.081	0.150	0.246	0.532	1.531	3.171	5.503	8.553
	3	0.100	0.184	0.302	0.651	1.875	3.884	6.740	10.475
	4	0.115	0.213	0.348	0.752	2.165	4.485	7.783	12.096
	5	0.129	0.238	0.390	0.841	2.420	5.014	8.701	13.523
	6	0.141	0.260	0.427	0.921	2.651	5.493	9.532	14.814
	7	0.152	0.281	0.461	0.995	2.863	5.933	10.296	16.001
	8	0.163	0.301	0.493	1.063	3.061	6.342	11.006	17.106
L = 100 m	(H) = 1 m	0.049	0.090	0.148	0.321	0.937	1.967	3.453	5.424
	2	0.069	0.127	0.209	0.454	1.325	2.782	4.884	7.670
	3	0.084	0.156	0.256	0.556	1.623	3.407	5.982	9.394
	4	0.097	0.180	0.296	0.642	1.874	3.934	6.907	10.848
	5	0.109	0.201	0.331	0.718	2.096	4.398	7.723	12.128
	6	0.119	0.220	0.362	0.787	2.296	4.818	8.460	13.286
	7	0.129	0.238	0.391	0.850	2.479	5.204	9.137	14.350
	8	0.137	0.254	0.418	0.908	2.651	5.563	9.768	15.341

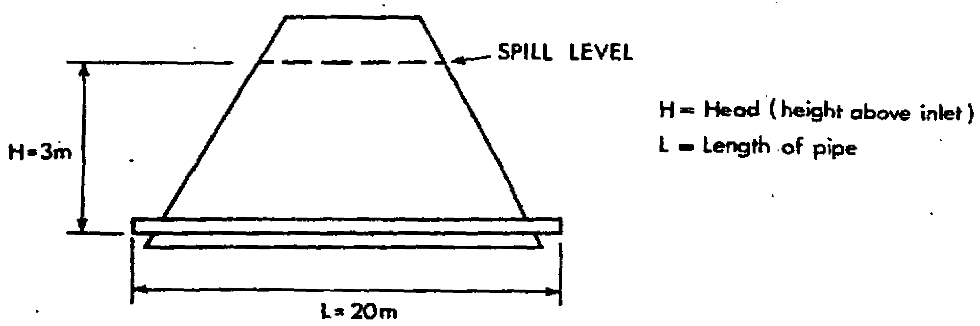


Figure 4:2:1 Soil conservation dam dimensions for pipe spillway design - worked example in section 2.2

Section 3 - Chute Spillways

Section 3 - Chute Spillways

Chute spillways can be used as dam spillways or to control gully head movement. The design of chute spillways requires the estimation of a design flow, and the sizing of the spillway inlet and stilling basin. The general elements of a chute spillway are shown in figure 4.3.1.

Many chute spillways have failed because of inadequate provision for energy dissipation in the stilling basin. The main objectives of a chute are to safely carry water from one level to another, and to adequately dissipate the energy gained during the drop. The two elements of the chute spillway are the chute and the stilling basin, and both elements require equal care in their design.

The procedure for design is:

1. Estimate the storm flow of the required probability and use this flow to calculate the design flow.

Instead of calculating the size of the chute based on actual catchment discharge and then adding a fixed amount of freeboard, the estimated catchment runoff is adjusted so that the resulting design flow includes freeboard in the design. This method ensures that the freeboard has a constant factor of safety throughout the structure.

It is important to note that design flow is used only in calculating the size of the structure, and is not used to calculate real depth of flow in the downstream channel. The equation for design flow is:

$$Q_D = (1.2 + 0.013\ D_z)\ Q_R \qquad \text{(equation 4.3.1)}$$

Where:
Q_D = the chute spillway design flow (m^3/s)
D_z = height of the drop (m)
Q_R = the R% A.E.P storm flow

2. Choose the inlet width or inlet wall height of the structure. Whichever one is chosen the other is fixed by one of the following equations:

$$W = \frac{Q_D}{1.7 \times H_I^{1.5}} \qquad \text{(equation 4.3.2)}$$

Where:
W = Inlet width (m)

H_I = Inlet wall height (m)

$$H_I = \left(\frac{Q_D}{1.7 \times W} \right)^{2/3} \quad \text{(equation 4.3.3)}$$

There are various reasons for choosing either inlet width or inlet wall height. For example, it may be advantageous to match the chute width to the downstream channel, or in some cases, in flat areas, it may be economical to limit inlet wall height, and thus reduce the length of earth training banks. Whichever the case, the choice is left to the discretion of the designer, provided the inlet equations are satisfied.

3. Calculate the critical flow at the crest of the spillway. With reference to figure 4.3.1 the depth and velocity at position 1 (i.e., at the crest) are:

$$d_1 = \frac{(Q_D^2)^{1/3}}{(w^2 g)^{1/3}} \quad \text{(equation 4.3.4)}$$

$$V_1 = \frac{Q_D}{w.d_1} \quad \text{(equation 4.3.5)}$$

$$H_1 = d_1 + D_z + \frac{V_1^2}{2g} \quad \text{(equation 4.3.6)}$$

Where:

d_1 = Water depth at position 1 (m)

V_1 = Water velocity at position 1 (m^2/s)

H_1 = Total energy (m)

g = Acceleration due to gravity ($9.81\ m/s^2$)

4. Calculate the energy of normal flow in the downstream channel to indicate how much energy needs to be dissipated in the structure.

Mannings n is chosen from table 4.3.1, and the normal flow in the downstream channel is calculated using Mannings equation (figures 4.3.2 or 4.3.3). Then equation 4.3.8 is used to calculate the total energy.

There are two important things to note. Firstly, the equations use the actual catchment discharge, and secondly, the choice of Mannings coefficient n will determine the safety factor in the outlet design. That is, a low estimate of n will provide a conservative design. Note also that the channel is assumed to be trapezoidal.

The water depth in the downstream channel is read fro figures 4.3.2 or 4.3.3 and velocity and energy are given by:

$$V_4 = \frac{Q_R}{Bd_4 + Z_D 4^2} \qquad \text{(equation 4.3.7)}$$

$$H_4 = d_4 + \frac{V_4^2}{2g} \qquad \text{(equation 4.3.8)}$$

Where:

V_4 = Water velocity in downstream channel (m/s)

H_4 = Water height in downstream channel (m)

5. Calculate the energy loss required in the structure fro the energy at the crest and the energy in the downstream channel

$$H_L = H_1 - H_4 \qquad \text{(equation 4.3.9)}$$

Where:

H_L = Energy Loss (m)

6. The dimensions of the structure can now be calculate using figures 4.3.4. and 4.3.5. The depth and length of th stilling basin are calculated from:

$$D_B = d_3 + \frac{V_3^2}{2g} - d_4 - \frac{V_4^2}{2g} \qquad \text{(equation 4.3.10)}$$

$$L_B = 4.2\ (d_3 - d_2) \qquad \text{(equation 4.3.11)}$$

7. Structure dimensions such as sideslopes and apron widt are also dependent on site conditions and should be chose accordingly.

Worked Example

A chute is to be designed for a drop of 3 metres. Th downstream gully is 3 metres wide with a side slope of 1:1 and longitudinal slope of 1%. Mannings n for the gully is estimate as 0.035. The ten year recurrence flow has been calculated a 2.7 m^3/s.

The solution to this problem is given in the following spillway design sheets.

Where:

D_B = Depth of stilling basin (m)

L_B = Length of stilling basin (m)

CHUTE SPILLWAY DESIGN SHEET

Example

1. INPUT DATA

a) Catchment Discharge (m^3/s) — $Q_R = 2{\cdot}7\ m^3/s$ (10 yr)

b) Drop Height (m) — $D_z = 3m$

c) Downstream Channel :

Width (m) — $B = 3m$

Slope (m/m) — $S = 0{\cdot}01\ m/m$

Side Slope (1:z) — $z = 1$

Manning's 'n' — $n = 0{\cdot}035$

2. DESIGN DISCHARGE

$Q_D = (1{\cdot}2 + 0{\cdot}013 D_z)\ Q_R =$

$$Q_D = (1{\cdot}2 + 0{\cdot}013 \times 3) \times 2{\cdot}7$$
$$= 3{\cdot}345$$

say $3{\cdot}35\ m^3/s$

3. INLET DIMENSIONS

either: $H_I = \left(\dfrac{Q_D}{1{\cdot}7.W}\right)^{2/3}$

or: $W = \dfrac{Q_D}{1{\cdot}7 H_I^{\,1{\cdot}5}} =$

In this case, choose chute width to suit gully width (ie3m)

$$\therefore\ h_I = \left(\frac{3{\cdot}35}{1{\cdot}7 \times 3}\right)^{2/3}$$
$$= 0{\cdot}76m$$

4. FLOW AT INLET

$d_1 = \left(\dfrac{Q_D^2}{9{\cdot}8\,W^2}\right)^{1/3} =$

$$d_1 = \left(\frac{3{\cdot}35^2}{9{\cdot}8 \times 3^2}\right)^{1/3}$$
$$= 0{\cdot}50\ m$$

$V_1 = \dfrac{Q_D}{W.d_1} =$

$$V_1 = \frac{3{\cdot}35}{3 \times 0{\cdot}5}$$
$$= 2{\cdot}23\ m/s$$

$H_1 = d_1 + D_z + \dfrac{V_1^2}{19{\cdot}6} =$

$$H_1 = 0{\cdot}5 + 3 + \frac{(2{\cdot}23)^2}{19{\cdot}6}$$
$$= 3{\cdot}75m$$

5. FLOW AT OUTLET

$$\frac{Q_R n}{B^{2.67} S^{0.5}} = \qquad \frac{2.7 \times 0.035}{3^{2.67} \times 0.01^{0.5}} = 0.050$$

from fig. 4·3·2 or 4·3·3

$$\frac{d_4}{B} = \qquad \frac{d_4}{B} = 0.17$$

$$\therefore d_4 = \qquad \therefore d_4 = 0.51\,m$$

$$\therefore V_4 = \frac{Q_R}{d_4 B + d_4^2 z} = \qquad V_4 = \frac{2.7}{0.51 \times 3 + 0.51^2 \times 1} = 1.51\,m/s$$

$$H_4 = d_4 + \frac{V_4^2}{2g} = \qquad \text{so: } H_4 = 0.51 + \frac{(1.51)^2}{19.6}$$

$$= 0.63\,m$$

6. HEAD LOSS

$$H_L = H_1 - H_4 = \qquad H_L = 3.75 - 0.63$$

$$= 3.12$$

7. FLOW DEPTHS

$$\frac{H_L}{d_1} = \qquad \frac{H_L}{d_1} = \frac{3.12}{0.5} = 6.24$$

from fig. 4·3·4

$$\frac{d_2}{d_1} = \qquad \frac{d_2}{d_1} = 0.237$$

$$\therefore d_2 = \qquad \text{then } d_2 = 0.1185$$

$$\text{say } d_2 = 0.12\,m$$

from fig. 4·3·5

$$\frac{d_3}{d_2} = \qquad \frac{d_3}{d_2} = 11.8$$

$$\therefore d_3 = \qquad \therefore d_3 = 1.398$$

$$\text{say } d_3 = 1.40\,m$$

8. STILLING BASIN DEPTH

$$V_3 = \frac{Q_D}{W.d_3}$$

$$D_B = d_3 + \frac{V_3^2}{2g} - H_4$$

$$V_3 = \frac{3\cdot35}{3\times1\cdot40} = 0\cdot80 \text{ m/s}$$

$$D_B = 1\cdot42 + \frac{0\cdot80^2}{19\cdot6} - 0\cdot63$$

$$= 0\cdot82 \text{ m}$$

9. STILLING BASIN LENGTH

$$L_B = 4\cdot2\,(d_3 - d_2)$$

$$L_B = 4\cdot2\,(1\cdot4 - 0\cdot12)$$

$$= 5\cdot376$$

to the nearest 0·5

$$L_B = 5\cdot5 \text{ m}$$

STRUCTURAL DIMENSIONS

Chute Width	:	$W = 3$ m
Inlet wall heights	:	$H_I = 0\cdot8$ m
Chute wall heights	:	$d_1 = 0\cdot5$ m
Stilling Basin wall height	:	$d_3 = 1\cdot45$ m
Stilling Basin depth	:	$D_B = 0\cdot82$ m
Stilling Basin length	:	$L_B = 5\cdot5$ m

Table 4.3.1 - Value of Manning's Roughness Coefficient for Natural Streams

TYPE		DESCRIPTION	COEFFICIENT
A	MINOR STREAM (Surface width of flood stage less than 30 m)		
	1.	Fairly regular section	
		(a) Some grass and weeds, little or no brush	0.030-0.035
		(b) Dense growth of weeds, depth of flow significantly greater than weed height	0.035-0.050
		(c) Some weeds, light brush on banks	0.035-0.050
		(d) Some weeds, heavy brush on banks	0.050-0.070
		NOTE: For trees within channel, with branches submerged at high stage, increase above values by 0.010 to 0.020.	
	2.	Irregular sections, with pools, slight channel meander. Increase values given in 1(a)-(d) by 0.010 to 0.020.	
	3.	Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerge at high stage	
		(a) Gravel bed, cobbles and few boulders.	0.040-0.050
		(b) Cobble bed with large boulders.	0.050-0.070
B	MAJOR STREAMS (Surface width at flood stage greater than 30 m)		
	The coefficient value is less than that for minor streams of similar description, because banks offer less effective resistance		
		(a) Regular section with no boulders or brush	0.025-0.060
		(b) Irregular and rough section	0.035-0.100
C	FLOOD PLAINS		
	1.	Pasture, no brush	
		(a) Short grass	0.030-0.035
		(b) High grass	0.035-0.050
	2.	Cultivated areas	
		(a) No crop	0.030-0.040
		(b) Mature row crops	0.035-0.045
		(c) Mature field crops	0.040-0.050
	3.	Brush	
		(a) Scattered brush, heavy weeds	0.050-0.070
		(b) Light brush and trees	0.060-0.080
		(c) Medium to dense brush	0.100-0.160
	4.	Trees	
		(a) Clear land with tree stumps, no sprouts	0.040-0.050
		(b) Same as (a) but with heavy growth of sprouts	0.060-0.080
		(c) Heavy stand of timber, little undergrowth, flood stage below branches	0.100-0.120
		(d) Same as above, but with flood stage reaching branches	0.120-0.160

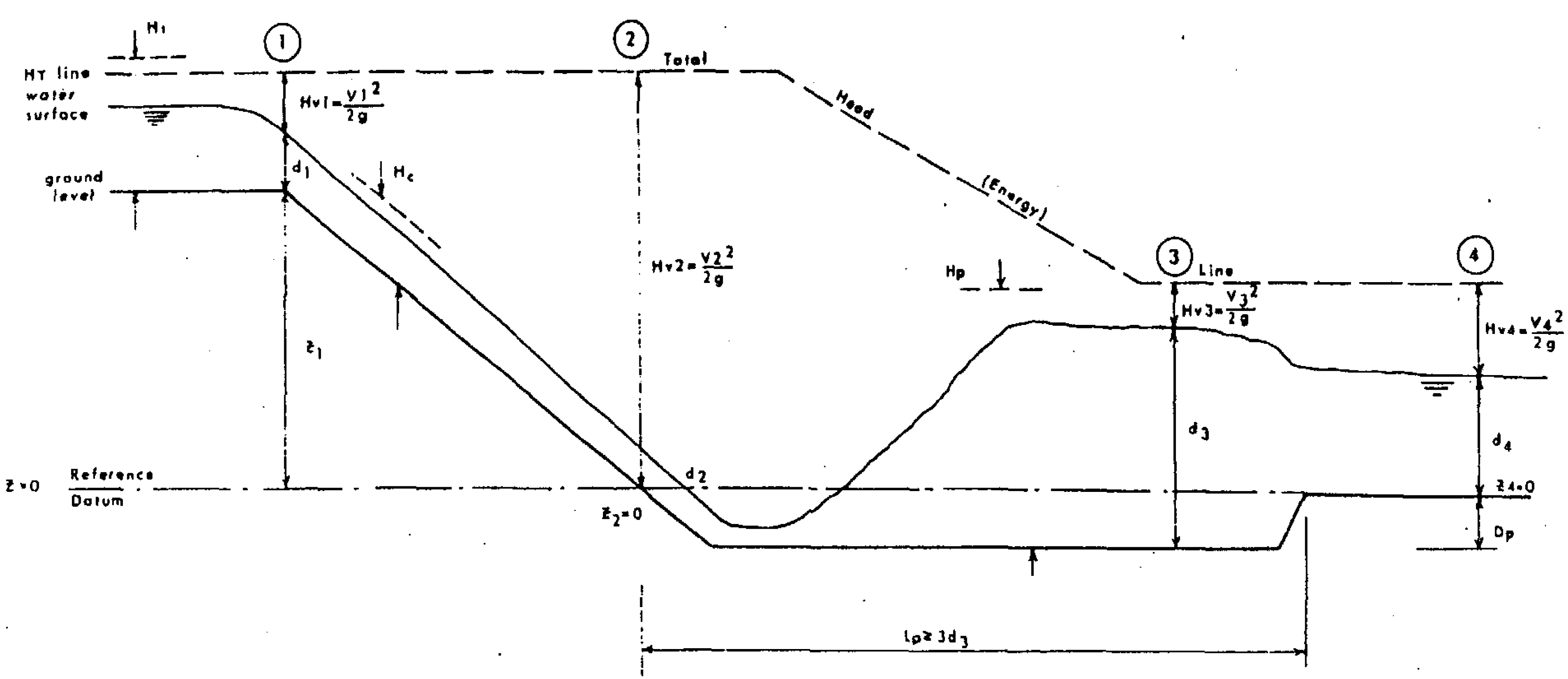


Figure 4:3:1 Longitudinal section of flow through a typical chute spillway.

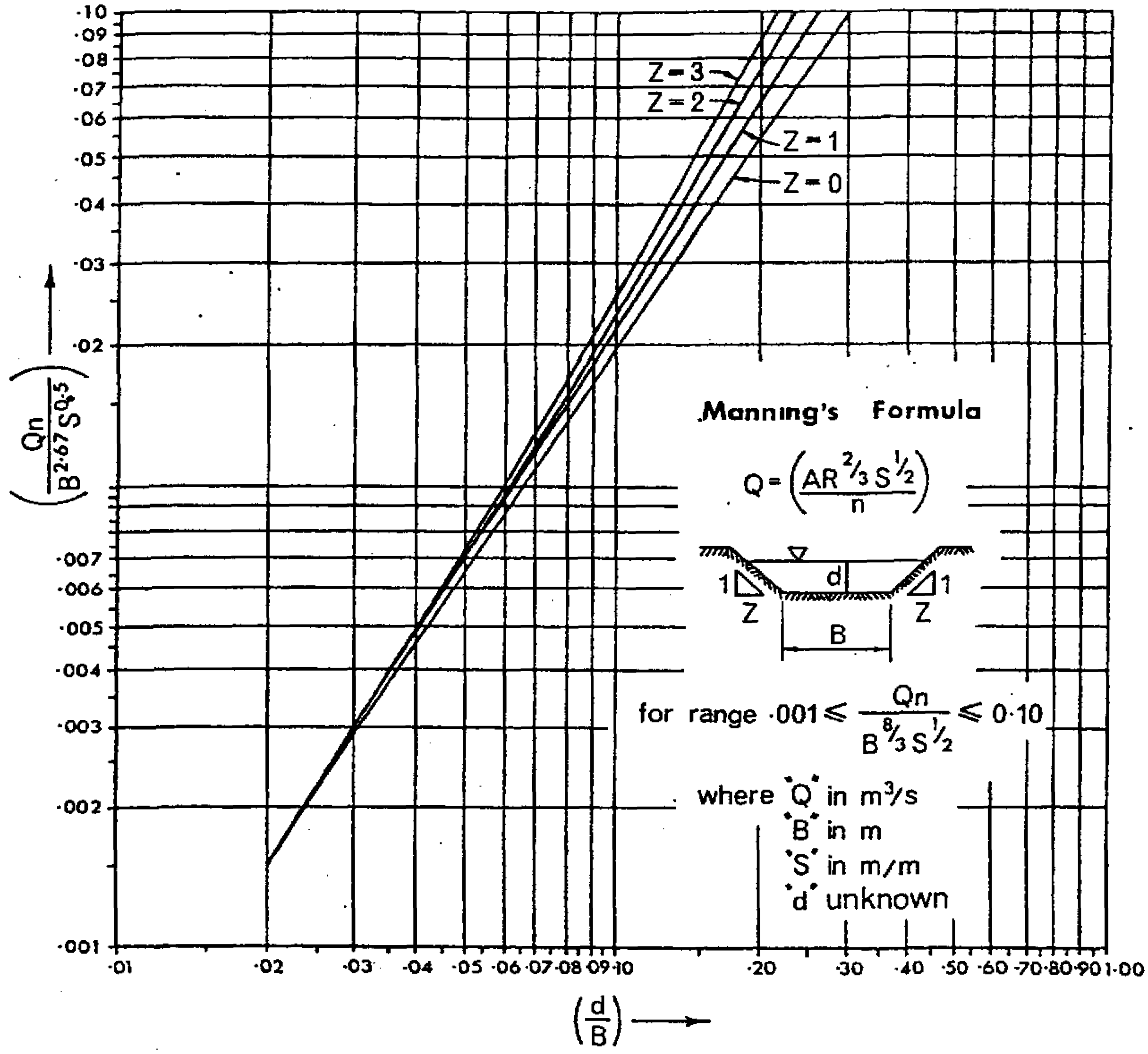


Figure 4·3·2 Solution of flow depth by Manning's formula.

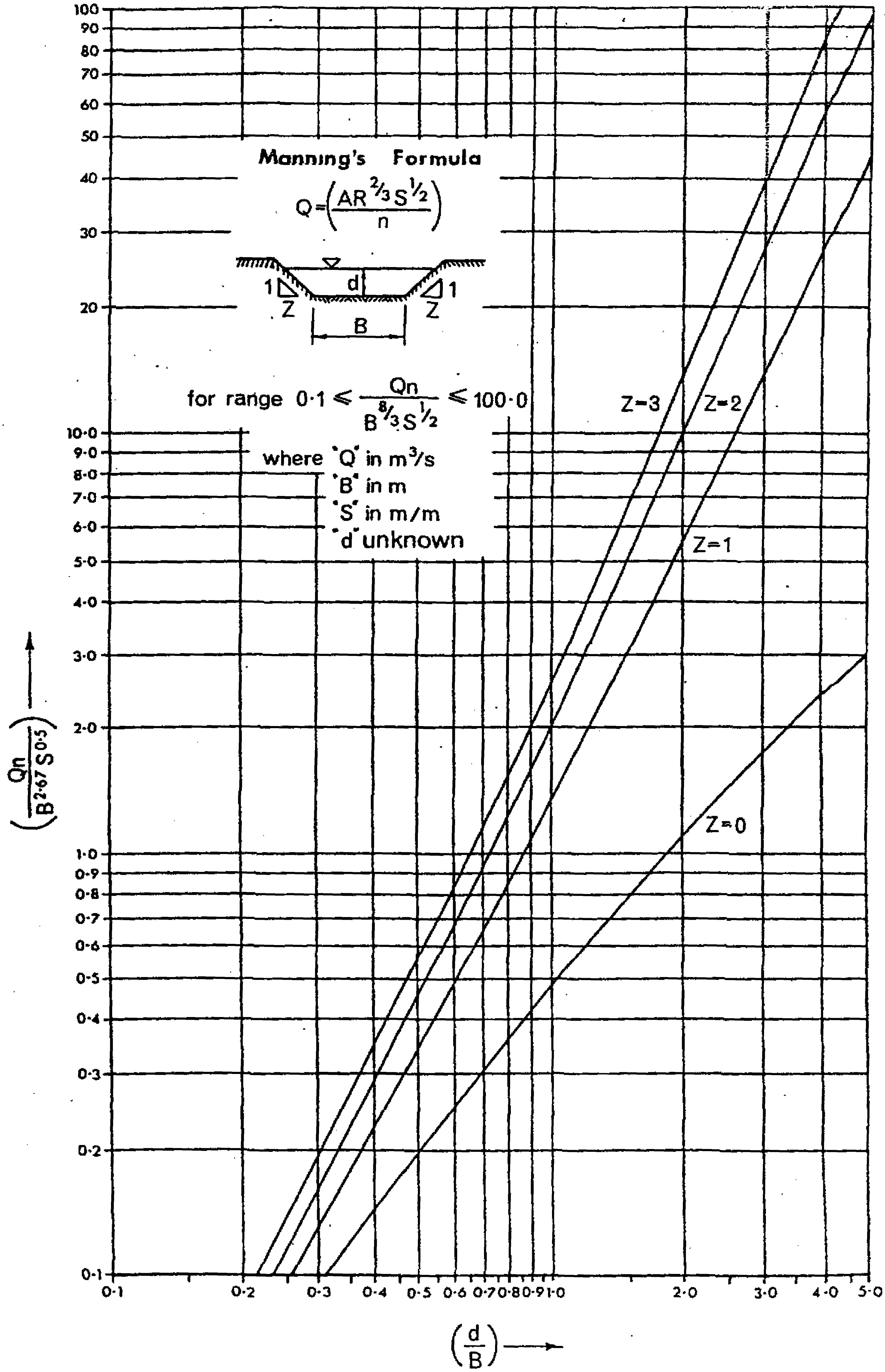


Figure 4·3·3 Solution of flow depth by Manning's formula.

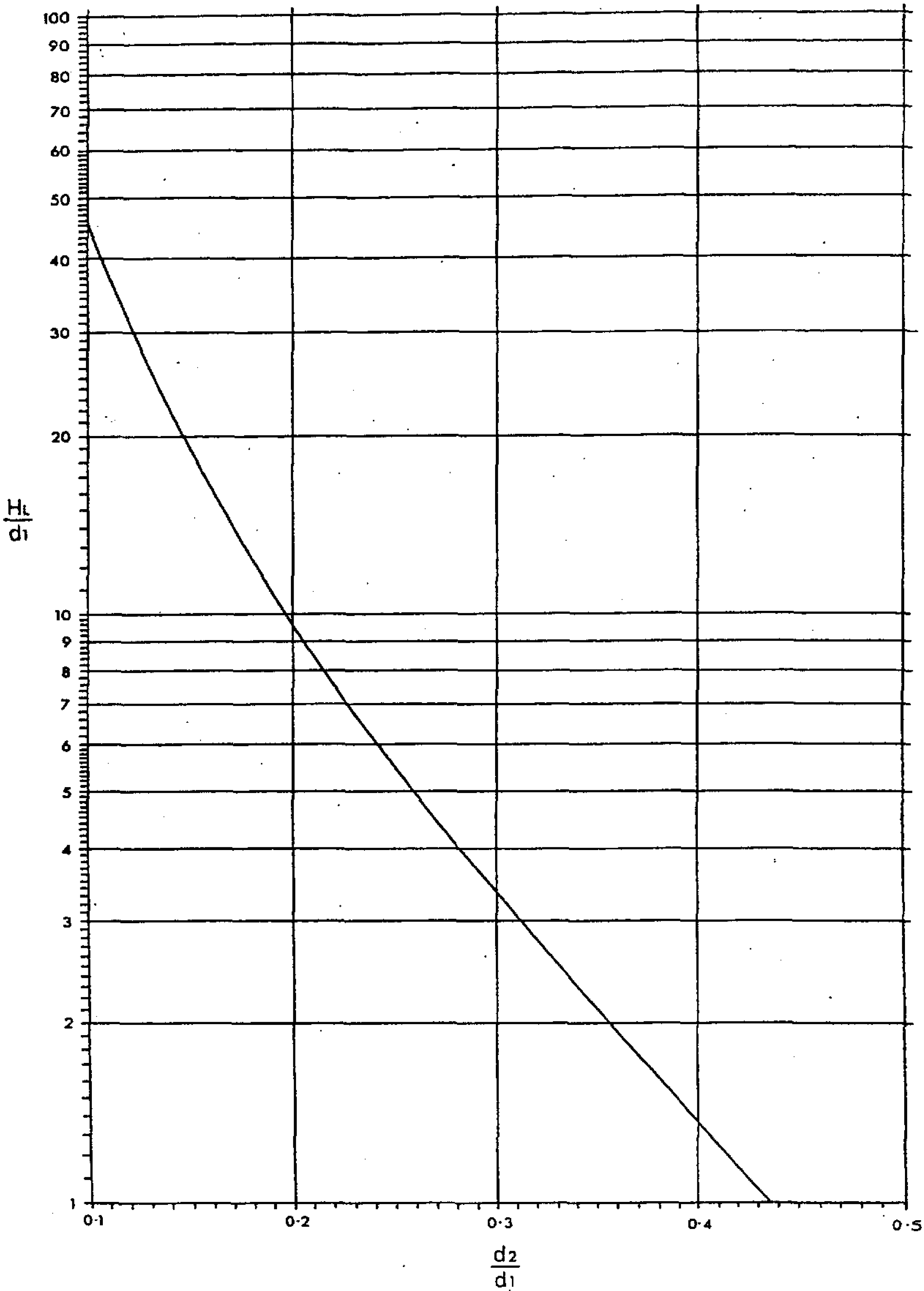


Figure 4:3:4 $\frac{H_L}{d_1}$ vs $\frac{d_2}{d_1}$

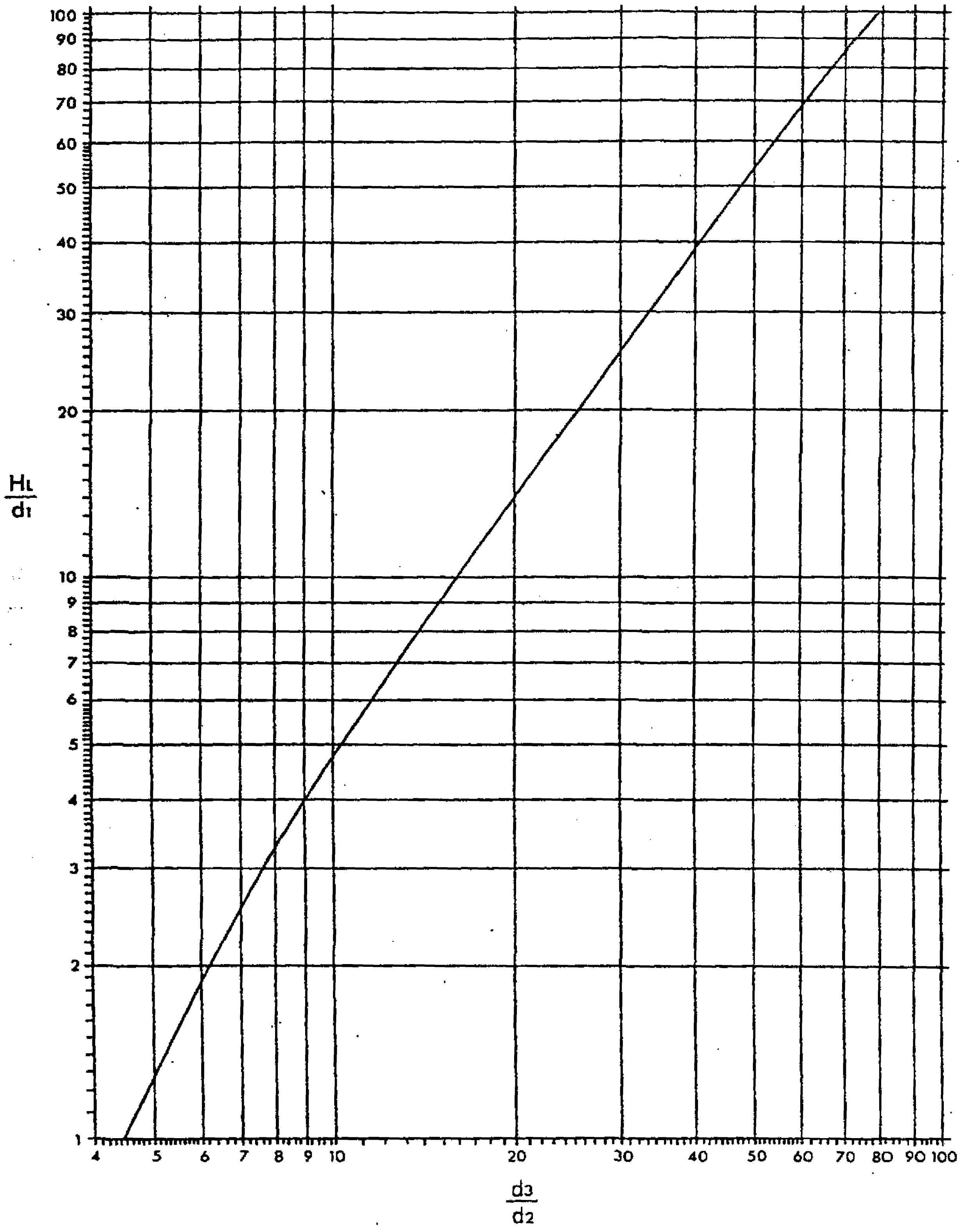


Figure 4.3.5 $\frac{H_L}{d_1}$ vs $\frac{d_3}{d_2}$

Section 4 - Drop Structures

Section 4 - Drop Structures

The straight drop structure illustated in figure 4.4.1 may be installed to achieve erosion control and convey water to lower levels.

The inlet capacity of a drop structure can be estimated by the weir formula:

$$W = \frac{Q_D}{1.7 \times H_I^{1.5}}$$

where:

Q_D = the design flow (m^3/s)

H_I = the inlet wall height (m)

W = the drop structure width (m)

Based on experimental data the flow geometry at straight drop spillways can be described by functions of the drop number (D), which is defined as

$$D = \frac{q^2}{g \quad z^3}$$

where q = the average discharge per unit width of the crest of overfall (m^3/s)

g = the acceleration due to gravity (9.81 m/s^2)

Z = the height of the drop (m)

The functions are (refer to figure 4.4.1):

$$L_d \Delta Z = 4.30\ D^{0.27} \quad \text{(equation 4.4.1)}$$

$$y_p \Delta Z = 1.00\ D^{0.22} \quad \text{(equation 4.4.2)}$$

$$y_u \Delta Z = 0.54\ D^{0.425} \quad \text{(equation 4.4.3)}$$

$$y_2 \Delta Z = 1.66\ D^{0.27} \quad \text{(equation 4.4.4)}$$

$$L_j = 6.9(y_2 - y_u) \quad \text{(equation 4.4.5)}$$

$$\text{(Basin Length (Lg)} = L_d + L_j \quad \text{(equation 4.4.6)}$$

$$n = y_{2/6}$$ (equation 4.4.7)

where:

L_d = the drop length (m) ie, the distance from the drop wall to the position of depth y_u

y_u = the depth (m) at the toe of the nappe or the beginning of the hydraulic jump

y_2 = the tailwater depth (m) sequent to define yu

For a given height and discharge q per unit width of the fall crest, the sequent depth Y_2 and the drop length L_d can be computed by equations 1 and 3 above.

In the above discussion it is assumed that the length of the spillway crest is the same as the width of the approach channel.

If the crest length is less than the width of the approach channel, the contraction at the ends of the spillway notch will cause the nappe to land beyond the stilling-basin sidewalls and the concentration of high water velocities at the centre of the outlet may cause additional scour in the downstream channel.

It is therefore important to design the approach properly by shaping the approach channel to reduce the effect of end contractions. The simplest form of such a structure, known as the box inlet drop structure, is simply a rectangular box open at the top and at the downstream end.

4.2 Worked Example

Determine the dimensions of a drop structure with a crest width (w) = 4m, height of drop (Z) = 2m Q = 11.6m^3/s and the average discharge per unit width of the crest of overfall

q = 2.9m^3/s/m

1. Drop Number

$$D = \frac{q^2}{g \quad Z^3}$$

$$= 2.90^2/9.8x2^3$$

$$= 0.107$$

2. $L_d \Delta Z = 4.30\ D^{0.27}$

$L_d = 4.30\ (0.107)^{0.27} \times 2$

$= 4.70$ m

3. $y_p \Delta Z = 1.00\ D^{0.22}$

$y_p = 1.00\ (0.107)^{0.22} \times 2$

$= 1.22$ m

4. $Y_u \Delta Z = 0.54\ D^{0.425}$

$Y_u = 0.54\ (0.107)^{0.425} \times 2$

$= 0.42$ m

5. $y_2 \Delta Z = 1.66\ D^{0.27}$

$y_2 = 1.66\ (0.107)^{0.27} \times 2$

$= 1.81$ m

6. $L_j = 6.9\ (Y_2 - y_u)$

$= 6.9\ (1.81 - 0.42)$

$= 9.6$ m

Total basin length (L_b) = 4.70 + 9.60
= 14.3 m (say 14 m)

Short basins can be achieved using standard laboratory tested stilling basins such as the U.S.B.R. basin or the SAF stilling basin. Details of these basins are set out in most standard texts on the subject.

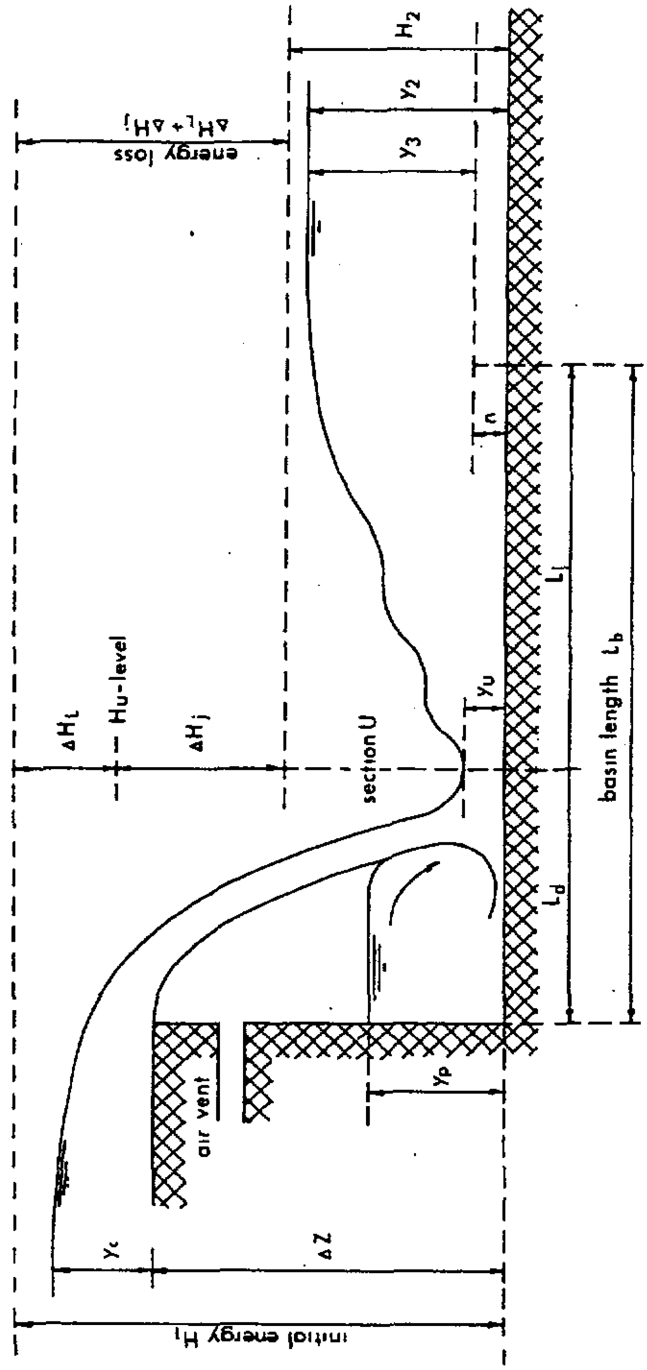


Figure 4.4.1 Longitudinal section through a straight drop structure.

Section 5 - References

Section 5 - References

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CHAPTER 5 - GRADE STABILISATION STRUCTURES

Grade stabilising structures are used to provide short steep erosion resistant zones where earth channels (including gullies) are used to convey runoff, and the natural channel slope will be unstable due to erosive flow velocities. This allows a lower, nonerosive grade to be used for the remainder of the channel.

These structures are commonly used to maintain stability in urban drainage ways where the increased discharge due to urbanisation would produce erosive flow velocities. They can also be used for gully stabilisation to reduce sediment transport and promote floor, side and head stability.

Grade stabilising structures can be broadly separated into;
* permanent structures usually built with rock, concrete or steel sheet piling and,
* temporary structures made of wire netting, brush or loose rock.

5.1 Permanent Structures

These structures have the following application:

(a) In constructed waterways or channels, where site restrictions such as available width or steep grade require that structures be used to achieve a stable flow velocity.

(b) At gully entry points:- where the erosion resistant surface of a concrete chute, pipe or verandah flume, is used to protect steep slopes.

(c) At isolated points in a drainage system:- where the protection of a specific cross section or site is required. For example, where a pipe, cable or road crosses an active gully and stabilisation of the entire system is not required.

5.2 Temporary Structures

Temporary structures are used to stabilise gully floors and promote vegetation establishment in gullies that are an integral part of the drainage system. They can be used to raise and widen gully floors, so reducing flow velocity to enable the establishment of vegetation. Their use is limited to situations where well established vegetation will ultimately provide sufficient stability.

Temporary structures are usually built as low weirs and are cheap to install. They may be pervious or impervious and can be constructed from rock, wire netting, brush or any other suitable material. To be effective they must be stayed to prevent overturning, good contact must be made with the gully floor and sides to prevent undermining, and side protection must extend above the expected flow heights. A sketch of a netting structure found to work effectively in the Central Tablelands is shown in figure 5.1.

5.3 Design.

Grade stabilising structures are often constructed in series. The stability of each structure depends on those lower down the drainage line. Expensive permanent structures therefore require design by a competent engineer. Consequently, only the design of temporary structures is considered here.

The main considerations in design are the crest capacity, spacing and height of these structures.

5.3.1. Crest Capacity

The crest or inlet capacity is determined from the weir formula given in Chapter 4, section 4.

5.3.2 Spacing

In a gully the desired result is that each structure will fill with sediment, so that sediment from one structure extends upstream to the toe of the next. This promotes the development of a stable natural stilling pool that is less likely to undermine the structure. Spacing is therefore a function of channel grade and structure height.

A stable channel grade depends on a number of factors including discharge, sediment size, and sediment supply. Although a number of formulae have been produced to predict channel grade. Most are very site specific and cannot be used indiscriminately.

Heede and Mufich (1973) suggest use of the following formula, which can be used as a guideline.

$$S = \frac{H}{K. \sin \theta}$$

Where:

S = Structure spacing (m)
H = Crest height (m)
θ = Grade of original gully floor
K = Constant - K = 0.3 when Tan < 2
K = 0.5 when Tan > 2

Alternatively, where a stable channel grade can be determined by measuring the grade of stable sections of the gully with similar dimensions, structures can be spaced to achieve that grade.

Another approach is to build the lowest structure in the system first and delay the construction of each subsequent structure until the one immediately below fills with sediment. By locating a structure on the sediment deposited in a lower structure the gully floor can be raised.

5.3.3 Height

The higher the structure the longer the spacing and the fewer the number of structures required. Actual structure height however, is restricted by the cross-sectional area of the channel, resistance of the structure to overturning and dissipation of energy below the crest. Because these structures are temporary and will fail, ultimate stability depends on vegetation. A height of up to 0.6m has been found to be satisfactory in most situations. Higher structures (up to 1m) may be effective in small catchments (10 ha) where a secure deep rooted vegetative cover can be maintained in the drainage line.

5.4 References

Crouch, R. J., Dawson, A.J., Henry, R.A. and O'Brien, W.J. (1984) Low Cost Weirs for Gully Stabilization. Conference on Agricultural Engineering, Bundaberg, The Institution of Engineers, Australia.

Heede, B.H. (1978) Designing Gully Control Systems for Eroding Watersheds. Environmental Management 2:509-522.

Heede, B.H., and Mufich, J.G. (1973) Functional Relationship and Computer Program for Structural Gully Control. J. Environmental Management 2:1-49.

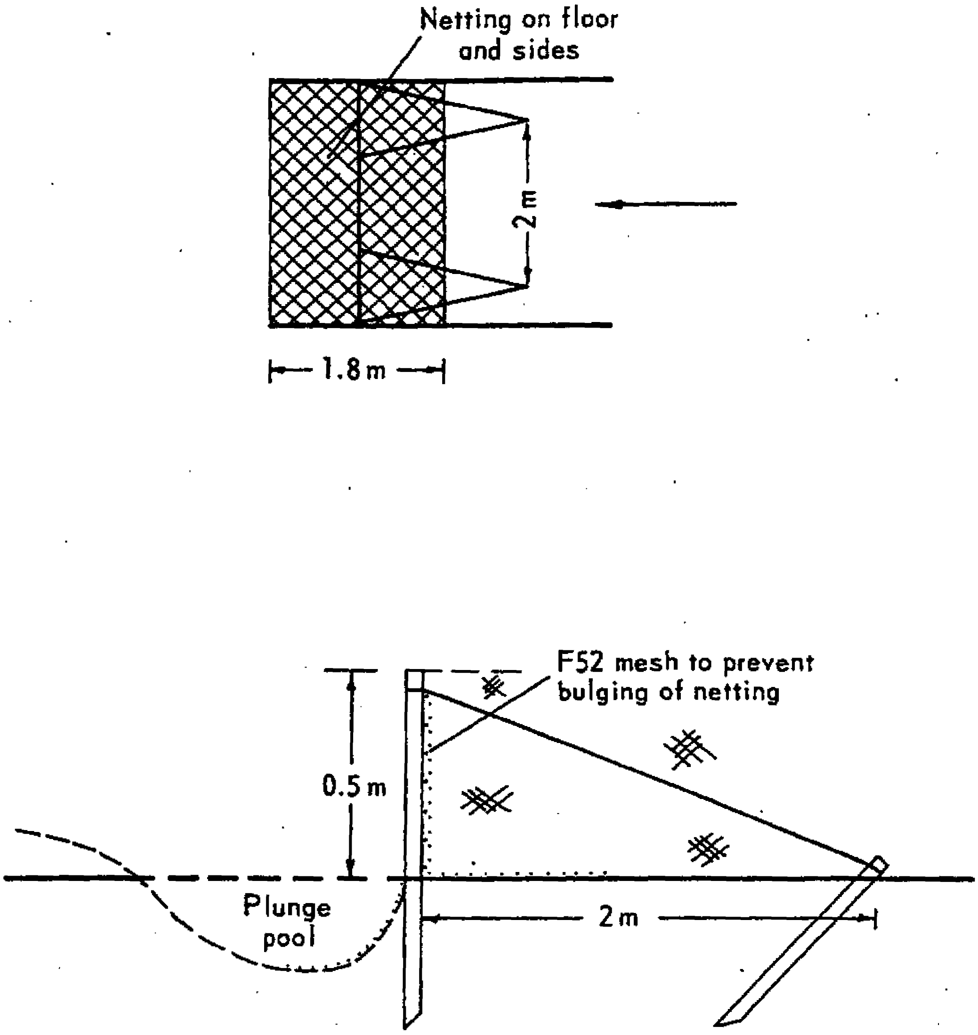

Figure 5:1 Diagrammatic representation of a steel post and netting temporary grade stabilisation structure used in the Central Tablelands of N.S.W.

CHAPTER 6 - STRIP CROPPING

Strip cropping is a land management system used to reduce soil erosion caused by the action of wind or water. The principle of strip cropping involves growing a rotation of crops in strips at right angles to the direction of wind or water flow. A rotation is adopted where high retardant strips separate and protect strips in a low retardant, erosion susceptible state.

6.1 Application and Types

The majority of strip cropping systems implemented in New South Wales are for the control of water erosion. In this situation there are two applications.

(a) As means of complementing the effectiveness of structural earthworks on slopes above 2%.

(b) On low slope areas (less than 1%) where catchments are ill defined and conventional earthworks are neither justified nor appropriate.

The satisfactory treatment of erosion on areas with slopes between 1% and 2% usually requires a combination of strip cropping and strategically located diversion banks.

There are three main types of strip cropping used to control water caused erosion.

(a) Contour strip cropping: Layout and tillage follow the contour and only very small deviations of strip boundaries from the exact contour are permissible.

(b) Buffer strip cropping: The planting or retention of permanent grass strips between or above strips of crops grown in a regular rotation.

(c) Field strip cropping: Sometimes referred to as parallel or strip cropping. This involves strips of a uniform width positioned across the slope but not necessarily following the exact direction of the contour. The design procedures referred to in this chapter apply to field strip cropping. This is the type recommended on the low slopes of the northern wheat belt.

6.2 Design Procedures

Design criteria for strip cropping have been developed primarily through the subjective monitoring of existing systems, with only limited research having been undertaken. Consequently, the procedures described can only be considered as guidelines.

They may be varied in the light of differing experiences and the availability of more objective data. However, since the ability of strips to control erosion is partly determined by the kind and degree of ground cover, and therefore subject to variation in seasonal conditions, it is unlikely that design procedures will be quantified to the same extent as for other forms of erosion control works.

6.2.1 Strip Widths

The width of a strip is determined by a number of factors but is generally restricted to a range of 20 to 100 metres.

Excessively narrow strips should be avoided as the spreading and retardence effect of the vegetation will be insufficient to achieve the necessary reduction in flow velocity. In reality, farmers are usually reluctant to accept as practical a strip width where this is a problem.

The optimum width for a given flow is where the back water effect of a high retardant strip extends to the top of the preceding unprotected strip. Slope is the main determinant of strip width and table 6.1 gives a general guide.

A nomogram (figure 6.1) has been developed which uses ground slope, catchment area and the length of the strip over which the flow can be spread as factors to determine strip width. Further factors to be considered in the adjustment of this figure are:-

(a) Management practices: The adoption of conservation tillage practices is essential for the satisfactory performance of a strip cropping system. Any practice that significantly reduces the amount of ground cover and soil disturbance from that achieved under a stubble mulching system will require a decrease in strip width.

(b) Types of crops grown: Crops such as sorghum provide a higher level of erosion control than crops where ground cover is minimal, eg. sunflowers, soybeans. Standing wheat stubble gives the greatest soil protection and provides the highest sediment trapping efficiency (Marschke, 1984).

(c) Previous erosion history: The stabilisation of existing depressions and other surface irregularities will require a reduction in strip width. In many cases, it will be necessary to undertake gully fill and levelling before the system can be implemented.

(d) Soil erodibility and rainfall intensity: The design criteria have been developed on the black clay soils of the northern NSW wheat belt.

(e) Width of farm machinery: With implements such as sowing equipment and boom sprays it is desirable to eliminate the necessity for wasteful overlap on the last run of each strip. In consultation with the farmer, a reasonable allowance for overlap per run should be decided on to determine the effective implement width. To provide for finishing a strip at the same end as was commenced, an even number of implement widths are frequently requested by farmers.

6.2.2 Strip Direction

With strip cropping generally confined to areas of low slope, a contour survey over an aerial photograph enlargement is used to determine the direction of the strips. A contour interval of 1 metre is satisfactory for this purpose.

The initial task is to establish a straight line that best approximates the contour, known as the line of best fit or key line (figure 6.2). Where a line crosses two or more contours the slope of the line can be calculated (off-contour tolerance). As a guide the off-contour tolerance should not exceed 0.3%. On very low slopes (less than 0.5%) the pattern of water flow is frequently not at right angles to the surveyed contours. In this instance it is appropriate to locate the strips at right angles to the observed flows. (Towler and Palmer, 1985).

Where it becomes necessary to change the direction of the strips the angle between the two key lines is bisected to form a pivot line (figure 6.3). This maintains the strip width on either side of the pivot line and allows for continuous farming of the strip around a turning point.

To ensure that satisfactory turning of machinery is possible a minimum angle of 130 degrees between strips is recommended. However, strict adherence to this criteria can result in difficulty with settling on a practical design and the minimum angle is often negotiated with the farmer.

The angle can be reduced by using two pivot lines. In this case the pivot lines and strip lines must form an isosceles triangle. The correct base angles of the triangle, between the stripline and the pivot lines, can be determined from the following formula (figure 6.4).

$$B = 45^{\circ} + 0.25\,\Theta$$

where B = correct base angles
Θ = Angle between striplines if only one pivot lined was used.

6.3 Crop Rotations

Rotations suitable for strip cropping are many and varied. A rotation must comply with a farmer's individual needs, but a few points to be considered are:-

(a) The principle of using high retardant strips to provide protection for strips of low retardance should be observed.

(b) The inclusion of a vigorous summer growing plant such as sorghum can provide a substantial backwater effect in the preceding strip.

(c) If the rotation requires a long fallow, particular care with weed control is required to maintain adequate ground cover and minimise soil disturbance. Stubble should be retained and herbicides used where possible.

(d) Opportunity cropping using strategies such as no tillage can be incorporated into a rotation to take advantage of abnormal seasonal conditions.

A number of possible rotations are illustrated in table 6.2.

6.4 Strip Cropping and Waterways

A situation commonly arises where a constructed waterway extends onto low slope arable land where strip cropping is a recommended soil conservation practice. In this case there are three options.

(a) Continue the waterway with the same capacity to a disposal area. Construction costs and property boundaries are the major factors in determining whether this option is feasible.

(b) Continue the waterway at a reduced capacity to cater for smaller flows only, with the remainder spreading onto surrounding areas protected by the strips. Residual flow waterways are useful where catchments flow at a low rate for long periods and waterlogging can be a problem.

(c) Terminate the waterway and rely on vegetation to spread the flow to a width where velocity is non-erosive. To assist in the initial spreading, an area of permanent grass is generally established as a buffer between the end of the waterway and the top strip. This is rarely feasible on slopes greater than 0.5%.

The procedure for determining the required width of spread relies on Manning's equation and is illustrated in the following example.

Example

A waterway with a design capacity (Q) of 20 m^3/s is to be spread onto a strip cropping system. The slope is 0.5%, the roughness coefficient (n) is 0.03 (bare earth) and the maximum permissible velocity (V) is 0.6 m/s. Calculate the required spread width (W).

$$A = \frac{Q}{V}$$

$$= 33.3 \ m^2$$

$$R = \frac{(Vxn)^{1.5}}{S^{0.75}}$$

$$= 0.13 \ m$$

In this case hydraulic radius = depth of flow

$$W = \frac{A}{D}$$

$$= 256 \ m$$

The critical, but as yet unknown factor, is for a given slope and vegetal retardance the distance a flow must travel before a specified width is achieved.

The ability of the grassed buffer to perform satisfactorily may be improved by the construction of a level sill along the width of the buffer. By locating the sill partway down the buffer siltation of the sill channel will be reduced.

Another option that can be used in addition to the level sill is to flare the last section of the waterway (figure 6.5). Alternatively, by tapering the last section, flows can be progressively spilt from the waterway into a permanently grassed area (figure 6.6).

6.6 References

Marschke, G.W.(1984) No Tillage Crops Production in Northern New South Wales. Proc. of Northern No-Till Project Team Meeting, Tamworth.

Towler, R.; Palmer, J. (1985) Strip Cropping in Conservation Farming, S.C.S of N.S.W.,Sydney, p80-86.

Table 6.1 - Estimated Strip Cropping Strip Widths as Influenced by Land Slope

Land Slope %	Estimated Strip Width (m)
0.5	90
1.0	70
1.5	50
2.0	P30

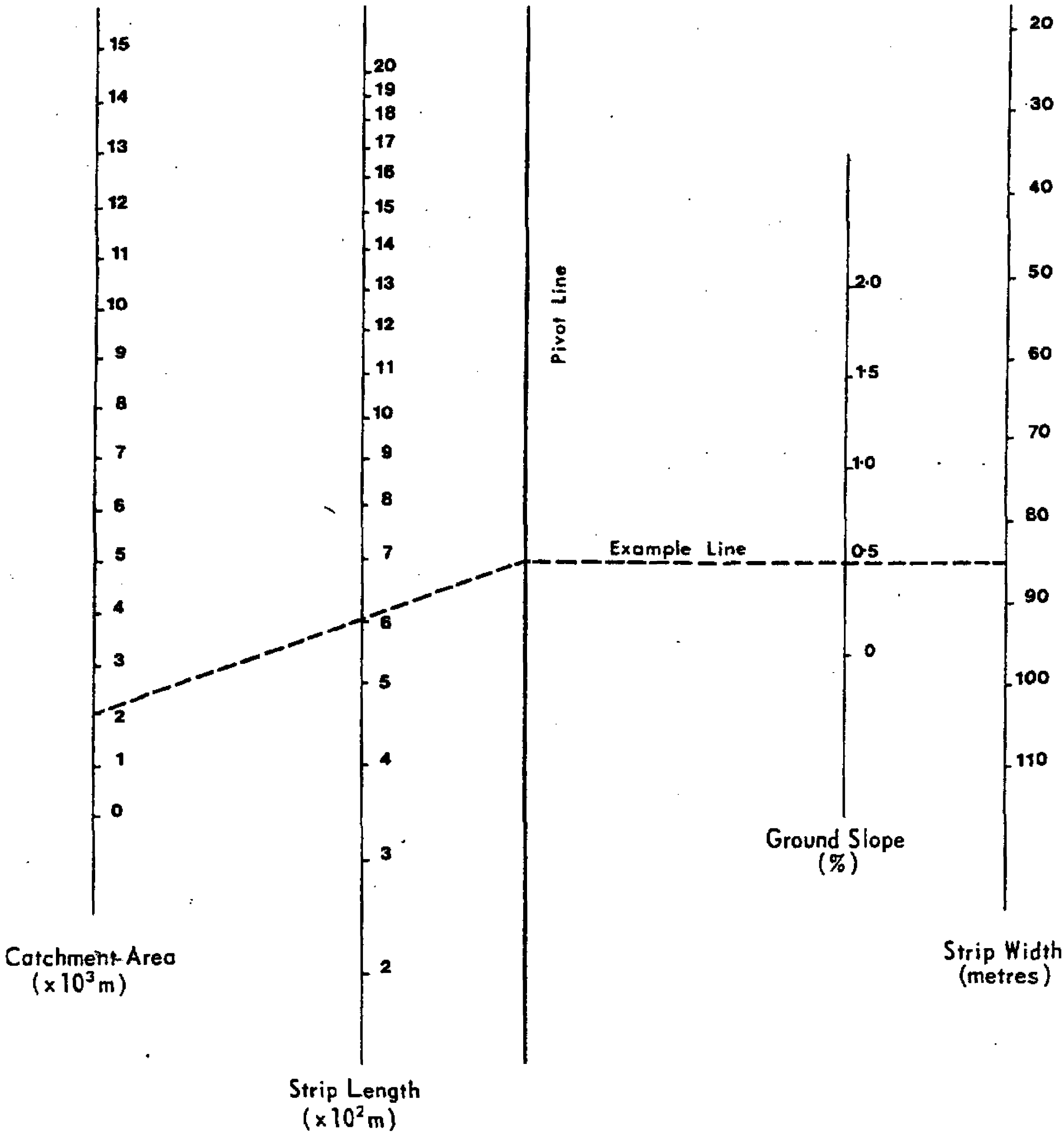

Figure 6·1 A nomogram for the estimation of strip widths

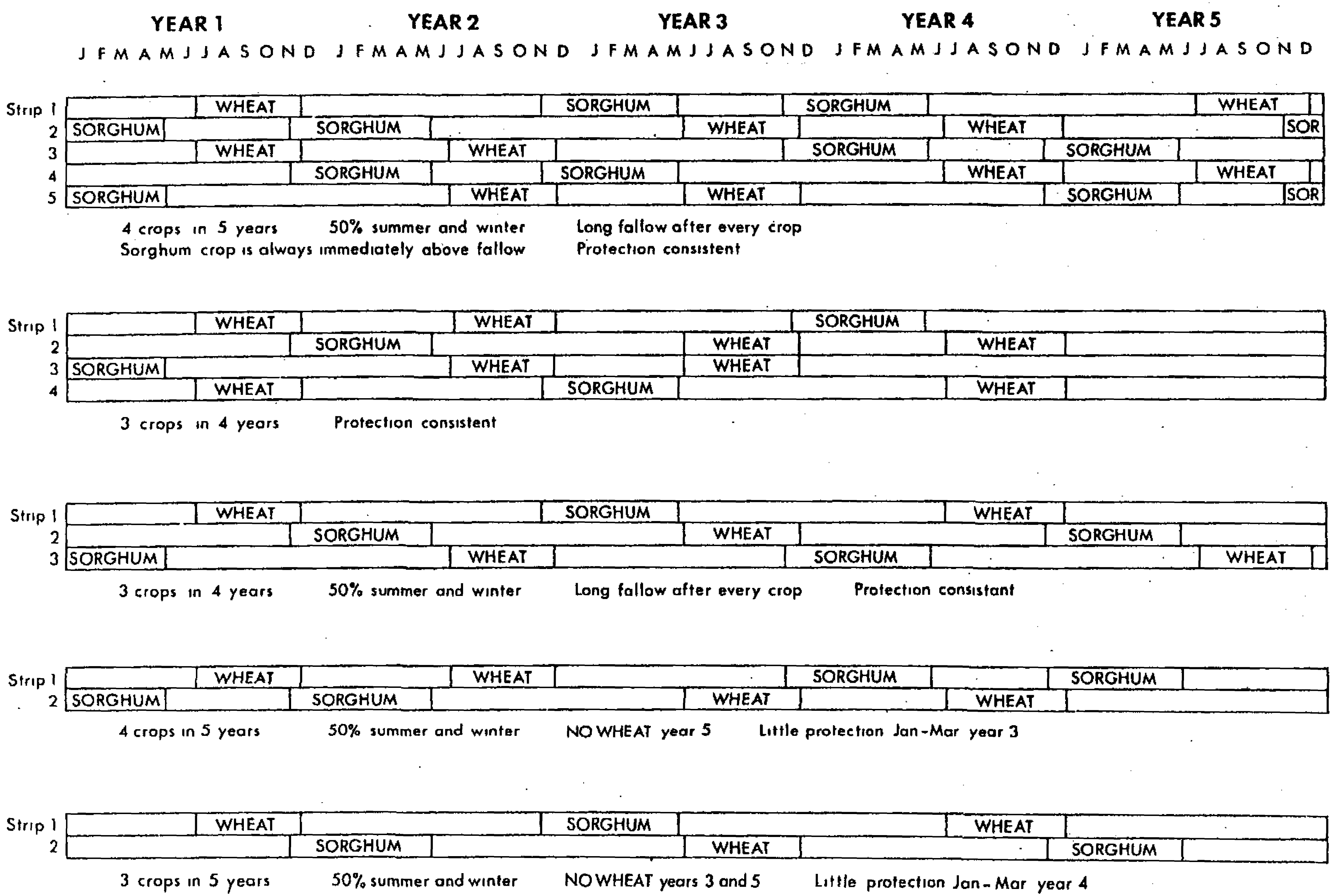

Table 6:2 Crop rotations for use in strip cropping.

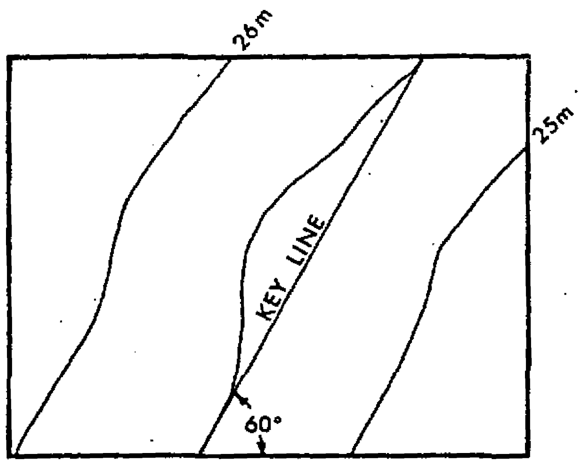

Figure 6:2 Location of the Key Line in strip crop planning.

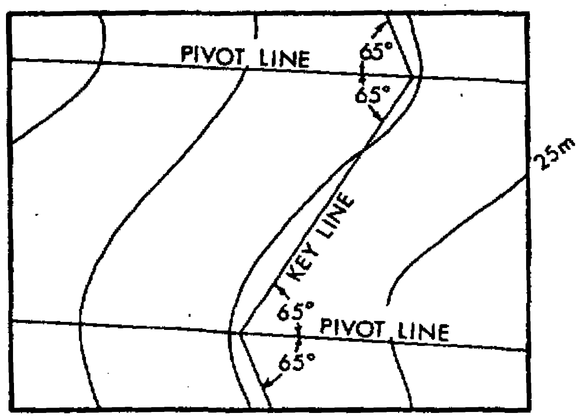

Figure 6:3 Location of a Pivot Line in strip crop planning.

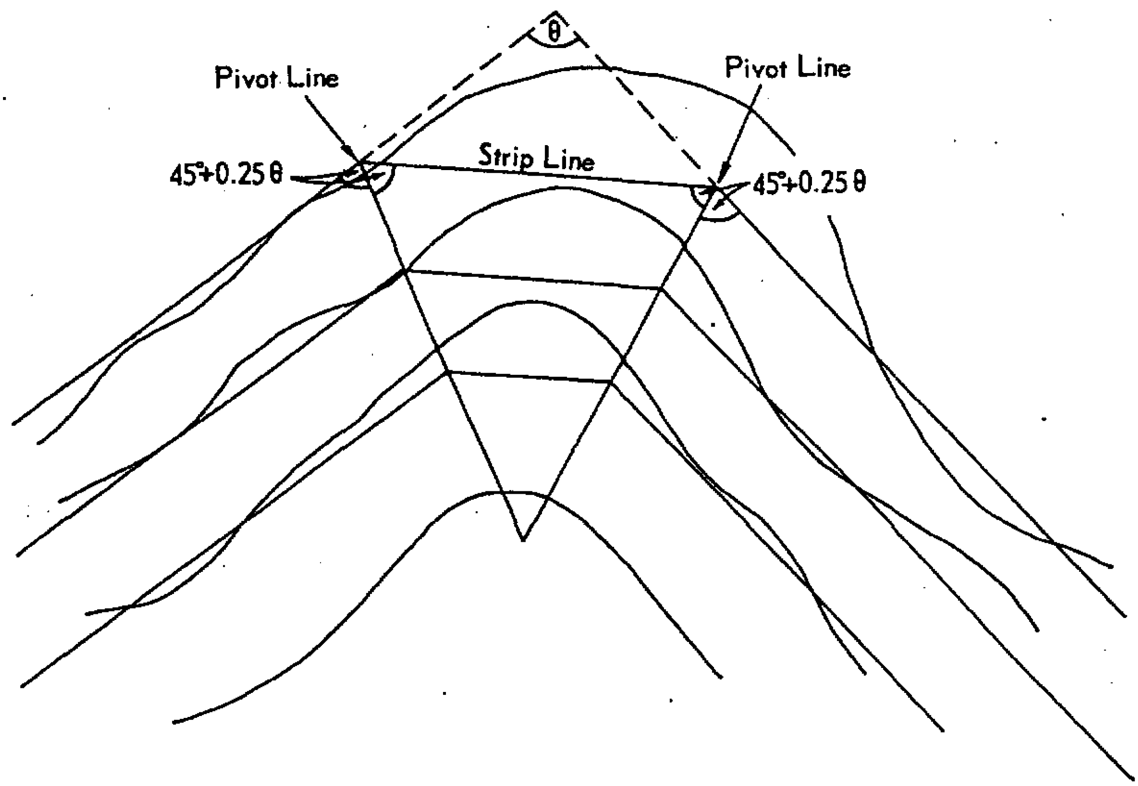

Figure 6:4 Method for reducing the angle between strips in strip crop planning using two Pivot Lines.

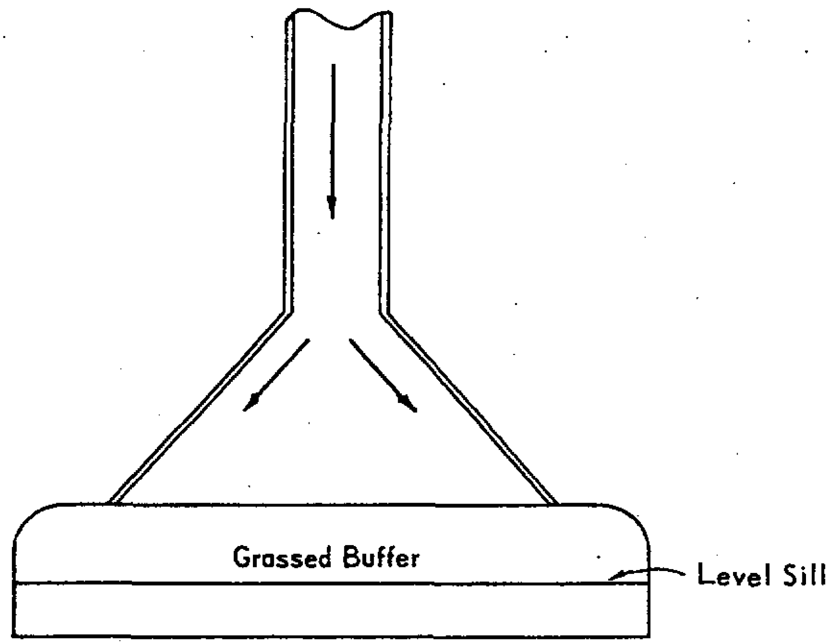

Figure 6·5 Diagram of a waterway flared to assist water spreading into a grassed buffer.

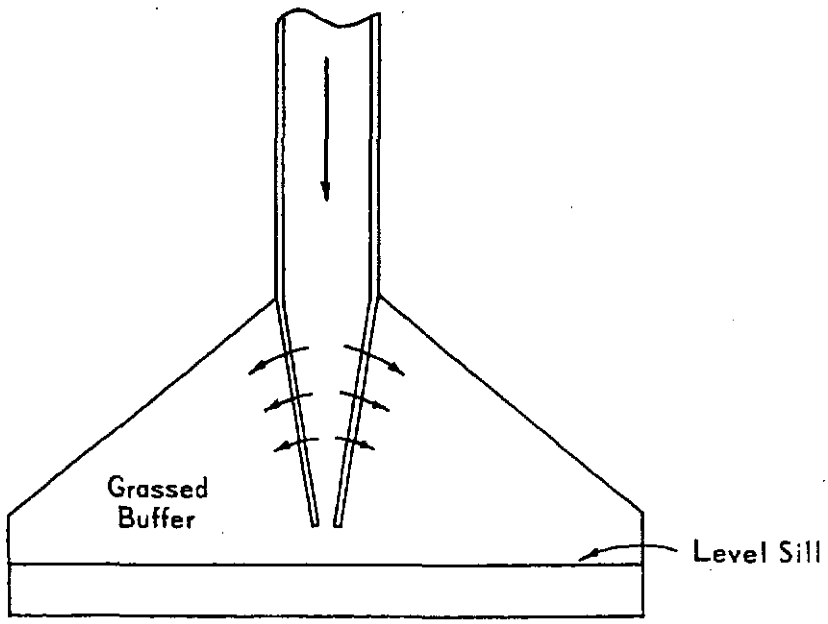

Figure 6·6 Diagram of a waterway tapered to assist waterspreading into a grassed buffer.

APPENDIX B

CHAPTER 3 FROM "URBAN EROSION AND SEDIMENT CONTROL HANDBOOK" (Department of Conservation and Land Management, 1992)

3. CONTROL MEASURES

3.1 INTRODUCTION

Some erosion and sediment generation during the construction stage of urban development is inevitable. Sound planning can reduce the potential for erosion, but the use of control measures will always be necessary to minimise the impact of any development, both on-site and off-site.

These control measures can comprise a variety of construction practices, structural or vegetative measures, or soil stabilisation techniques. To achieve the best results, these different types of measures should complement each other in an integrated development program.

Control measures may be temporary or permanent. Temporary measures are utilised for varying short periods of time, but basically for the duration of the construction period. Permanent measures are intended to remain in place until well after construction is complete.

Whether temporary or permanent, there are usually critical erosion and sediment control measures that should be the first items constructed when development work begins. Only after these are operational should clearing and excavations work begin on the building site. No matter which measures are selected and implemented, they must be properly maintained in order to remain functional.

The balance of this section is set out as follows:

- Section 3.2 Stormwater management principles. Basic design criteria for all control measures.
- Section 3.3 Specific design, construction and maintenance consideration for various erosion control measures and practices.
- Section 3.4 Specific design, construction and maintenance considerations for various sediment control measures and practices.

The measures referred to in the following sections represent the minimum design standards for erosion and sediment control works. Compliance with these standards does not relieve the designer of the responsibility to apply sound professional judgement to protect the health, safety and welfare of the general public. Additionally, since these are minimum standards, special site conditions and environmental constraints may require a greater level of protection than would normally be required under these guidelines.

3.2 STORMWATER MANAGEMENT

3.2.1 INTRODUCTION

Stormwater management is the basis of erosion and sediment control.

The urbanizing process is associated with major changes to the hydrological characteristics of a developing catchment. The substitution of paved surfaces for pervious areas, the channelling of flows and the introduction of piped drainage networks all serve to decrease the flow travel time and consequently increase the peak flow rates from an affected catchment. Similarly, but on a smaller scale, the disturbance associated with development sites can also result in considerably greater peak flows and velocities.

The management of stormwater must be viewed from a systems perspective. With any designed control measure, whether a piped system or say, a system of open channels, there will be elements of both erosion and sediment control. For example, a piped drainage system is effective in conveying high velocity flows through constricted areas and therefore can be seen as preventing channel erosion in the immediate vicinity. Similarly, a piped drainage system can be designed for a minimum velocity so as to be self cleansing and not subject to sedimentation.

On the other hand, where space allows, open channels can function to retard flood flows by utilizing lower velocities and effectively providing in-channel storage. Stormwater detention basins can also provide indirect erosion protection by reducing peak discharges through the use of "concentrated" storages.

From an overall systems viewpoint therefore, it is important to realise that many different stormwater management techniques (both structural and of a site planning nature) can be employed to provide the most appropriate solution to a site specific problem. Each has its own element of erosion and sediment control, and the wise application of the appropriate technique will result in an environmentally sound and cost-effective project.

The following sections provide guidance in the application of hydrologic and hydraulic principles to the design and construction of control measures on development sites and in urbanizing areas. The guidance offered is not prescriptive and it is always incumbent upon the designer to adopt suitable standards to meet site conditions and safety requirements where the consequences of failure can be severe.

3.2.2 STORMWATER MANAGEMENT PRINCIPLES FOR EROSION AND SEDIMENT CONTROL

The basic principles of managing stormwater within an urban development site are:

- *Control the flow volume.*
- *Control the flow path.*
- *Reduce runoff velocity.*

The achievement of these principles involves the integrated diversion, concentration, spreading or retention of stormwater runoff, while minimising erosion and the resultant generating of sediment.

The following aspects of stormwater planning, design and management should be incorporated into urban drainage design to ensure both short and long term environmental stability and wise landuse planning.

3.2.2.1 Natural Drainage System

Whenever possible, natural watercourses should be maintained and used to convey stormwater runoff. Channelling, straightening, lining and enclosing natural watercourses usually results in larger peak flows, and high drainage, erosion and sediment control costs downstream.

Maintaining the natural pre-development hydrological conditions within an urban catchment can have beneficial effects on groundwater recharge, the maintenance of water tables, runoff volumes and rates, as well as base flows during dry periods. In new developments, the retention of pervious surfaces and the minimising of areas of constructed, impervious surfaces, can help in this regard.

A variety of measures for detaining stormwater flows can be used to offset the unavoidable increases in flow volume or reductions in flow time as the result of urban development. Roof tops, parking lots, playing fields, swales, roadsides, ponds and stormwater pipes can provide storage capacity for stormwater runoff. Permanent or temporary detention basins can reduce peak runoff, aid groundwater recharge, provide stormwater pollution treatment, and lessen downstream flooding, erosion and sedimentation. (Section 3.3.12)

3.2.2.2 Catchment Planning

The total catchment area of any development site must be recognised as the logical unit for stormwater management planning. Urban drainage design for particular development sites must therefore be based on, and support, planning for the total catchment within which they occur.

The natural boundary for stormwater management planning for developments is the site catchment, which acts as a single integrated system during runoff events.

The diversion of all external runoff and of clean runoff from undisturbed areas within the site, to an alternative stable outlet, will protect those areas being developed. This strategy can greatly reduce the cost of the required structural erosion and sediment control measures, while maximising their efficiency.

3.2.2.3 Major/Minor Concept

An urban stormwater runoff system generally serves two purposes:

- The control of stormwater runoff to prevent or minimise damage to property, and physical injury and loss of life that may occur during or after a very infrequent or unusually severe storm.

- The control of stormwater to minimise inconvenience or the disruption of activity as a result of runoff from more frequent but less intensive storms. The drainage systems installed to achieve these purposes are usually referred to as the major and minor systems respectively.

The concept of these dual drainage/stormwater systems is equally applicable to the measures involved in engineering for erosion and sediment control. The minor system refers to structures capable of conveying minor flows and, in most urban situations, is a piped drainage system. The major system, on the other hand, refers to planned drainage routes which, during major storms, convey flows which exceed the capacity of the minor system.

Stormwater management for erosion and sediment control should address the major/minor concept. Any measure or structure should be checked for performance under a major flood flow so that either safe overflow routes are available (e.g. basin emergency spillways) or the structure is able to fail in a predictable or fail-safe manner. In this respect, the major/minor concept is equally applicable to site works consisting of say, contour banks or waterways, as the minor elements. (see Section 14.5.1 Australian Rainfall and Runoff (ARR), 1987)

3.2.2.4 Temporary Versus Permanent Measures

There are significant differences in design considerations, methods of construction and maintenance, and cost benefits between temporary and permanent control measures.

Temporary measures are designed to have a short life, typically for the duration of the construction period. They may, however, only have a lifespan of several days, or even overnight, where very short term protection is warranted. Because of their short life, they need not necessarily be designed to last for many years with minimal maintenance, nor built of highly durable materials. They must, however, receive regular maintenance during their period of use, to remain effective. Such measures may have a low initial cost but a relatively high maintenance costs if frequent or intense storms occur during the construction period.

Permanent measures are intended to remain in place until well after construction is complete and to function with a minimum of maintenance. They must be therefore be appropriately designed and constructed of durable materials, with their anticipated lifespan in mind.

It must be noted however, that the location of certain temporary structures, (e.g. in a critical flow path leading to an environmentally sensitive area) will require them to be designed to the same standards as permanent structures. Section 3.2.3.1 outlines design considerations linking the design life of a control structure and the probability of its design criteria being exceeded.

3.2.2.5 Control Measures to Modify Runoff and Erosion Rates

Standard soil conservation works such as contour bank systems act to reduce both runoff peaks and erosion rates. Peak runoff is reduced by the increased overall travel time afforded by the longer flow distance (around the contour compared with directly downslope) and the slower velocities along the bank channel. Similarly, sheet erosion can be reduced by modifying the slope lengths that are exposed. In terms of the Universal Soil Loss Equation (see Section 3.2.3.7) the "LS" factor (slope length) is reduced, thereby reducing the estimated sediment yield.

Other factors which can be modified to reduce erosion are the crop or cover factor (C) and the tillage factor (P). Goldman et al. (1986) show that by reasonable modification to the "LS, C & P" factors on an even 1:2 slope, the sediment yield can be reduced by around 90 per cent.

The employment of structural and surface treatment measures should therefore be combined in an optimal mix for various development site conditions to produce cost effective reductions in erosion and sedimentation on development sites.

3.2.2.6 Safety

Attention is drawn to the full discussion on public safety aspects in Section 14.10.4, ARR, 1987. The following points are some of the more important features that should be noted.

Safety requirements, particularly in regard to depth and velocity of flow, apply to the construction or alteration of all drainage channels where public access is likely. The depth x velocity criterion of 0.4 m^2/s for pedestrian safety, and 0.7 m^2/s for vehicle stability, should be adhered to. Where there is public access, grassed batter slopes should ideally be set at 1:6, but no steeper than 1:4.

With regard to sediment basins (particularly permanent basins) where public access is likely on the construction site, it is advisable to provide safety fencing when the total depth of water impounded, including surcharge, is greater than 1.4 m. Councils or agencies should seek legal opinion on this matter for specific circumstances.

Special attention should be given to emergency spillways to avoid sudden and catastrophic failure which may cause loss of life or property damage downstream. The operation of major basins should be checked under probable maximum precipitation events to ensure that any failure is in a gradual and predictably safe manner.

Where a waterway flows into a closed system, the incorporation of sloping grilles, steps and ladders may be necessary. For unavoidably steeply sided lined channels, the use of fencing and warning signs should be considered.

3.2.2.7 Legislative Requirements

Any alteration to a natural channel should be approached with care. Where the channel or waterway forms part of a trunk or creek system, certain legislation (e.g. Water Act, 1912; Rivers and Foreshores Improvement Act, 1948; Soil Conservation Service Act, 1938; etc.) may apply to the management and use of those stream channels.

For example, trees within twenty (20) metres of prescribed streams are protected under Section 21(c) of the Soil Conservation Act, 1938. If there are any doubts as to the status of a waterway, guidance should be sought from the Department of Water Resources or the Soil Conservation Service of N.S.W.

3.2.3 DESIGN CRITERIA

An estimation of runoff is an essential first step in the design of most control structures and surface treatment methods. Flow rates should be estimated by hydrological evaluation of the contributing area at a level of accuracy commensurate with the importance of the control measure.

3.2.3.1 Design Average Recurrence Interval - Rainfall

The design average recurrence interval (ARI) of the rainfall should be selected with a view to the estimated design life and the implied costs of failure of the structure or control measure (i.e. social, economic and environmental costs). In instances where the control measure is temporary (a design life of around one month, for example) a design ARI of 1 year may be appropriate. For larger and more permanent measures, the adoption of a lower probability ARI, say 50 or 100 years, should be considered. The method of assessment might therefore range from a simple subjective choice through to various levels of cost-benefit analysis where the costs of failure are significantly higher.

Equations 3.1 and 3.2 (from ARR, 1987) link the design ARI, the probability of exceedence and the design life of a control measure:

$$ARI = \frac{L}{-\ln(1-P)} \qquad \textbf{EQUATION 3.1}$$

or

$$P = 1 - e^{(-L/ARI)} \qquad \textbf{EQUATION 3.2}$$

where L = design life (yrs)
P = probability of exceedence (fraction)
ARI = design average recurrence interval (yrs)
e = Naperian base = 2.718
ln = natural logarithm

Examples:

1. **Equation 3.1** A diversion bank is expected to be in use on a construction site for a period of 2 months. The acceptable chance of exceedence (overtopping) is judged as 20 per cent. What is the design ARI for this structure ?

L = 2 months = $^2/12$ years
P = 0.20

so $$ARI = \frac{^2/12}{-\ln(1-0.20)} = 0.75 \text{ years}$$

Adopt design ARI of 1 year (min.)

2. **Equation 3.2** An emergency spillway on a sediment basin is designed for an ARI of 20 years. It is expected to have a design life of 2 years. What is the chance of exceedence in the period of 2 years ?

L = 2 years
ARI = 20 years

so P = 1 - e-2/20
= 0.10

That is, there is a 10 per cent chance of exceedence within the 2 year design life.

Table 3.1 gives suggested values of design ARI for a range of control measures. However, as mentioned above, where individual circumstances dictate it is incumbent upon the designer to chose a higher design standard if necessary.

TABLE 3.1

SUGGESTED DESIGN AVERAGE RECURRENCE INTERVALS FOR VARIOUS EROSION AND SEDIMENT CONTROL MEASURES IN URBAN AREAS

	Estimated Design		
	0-12 mths	12-48 mths	>48 mths
Control Measure	**Design ARI (years)**		
Diversion Bank	1-10	10-20	*
Level Spreader	1-10	10-50	*
Waterway	1-10	10-50	*
Sediment Basin			
- primary outlet	1-5	5-10	*
- emergency outlet	10-20	20-100	*
Sediment Trap	1-5	5-20	-
Outlet Protection	1-20	20-100	*
Grade Stabilising Structure	1-20	20-100	*
Detention Basin:			
- primary outlet	1-5	5-10	*
- emergency outlet	10-20	20-100	*

* Full design required in accordance with Major/Minor concept (see ARR, 1987 - Section 14.5).

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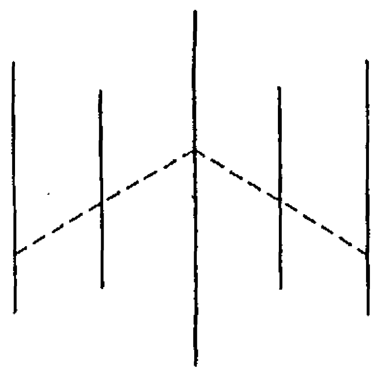
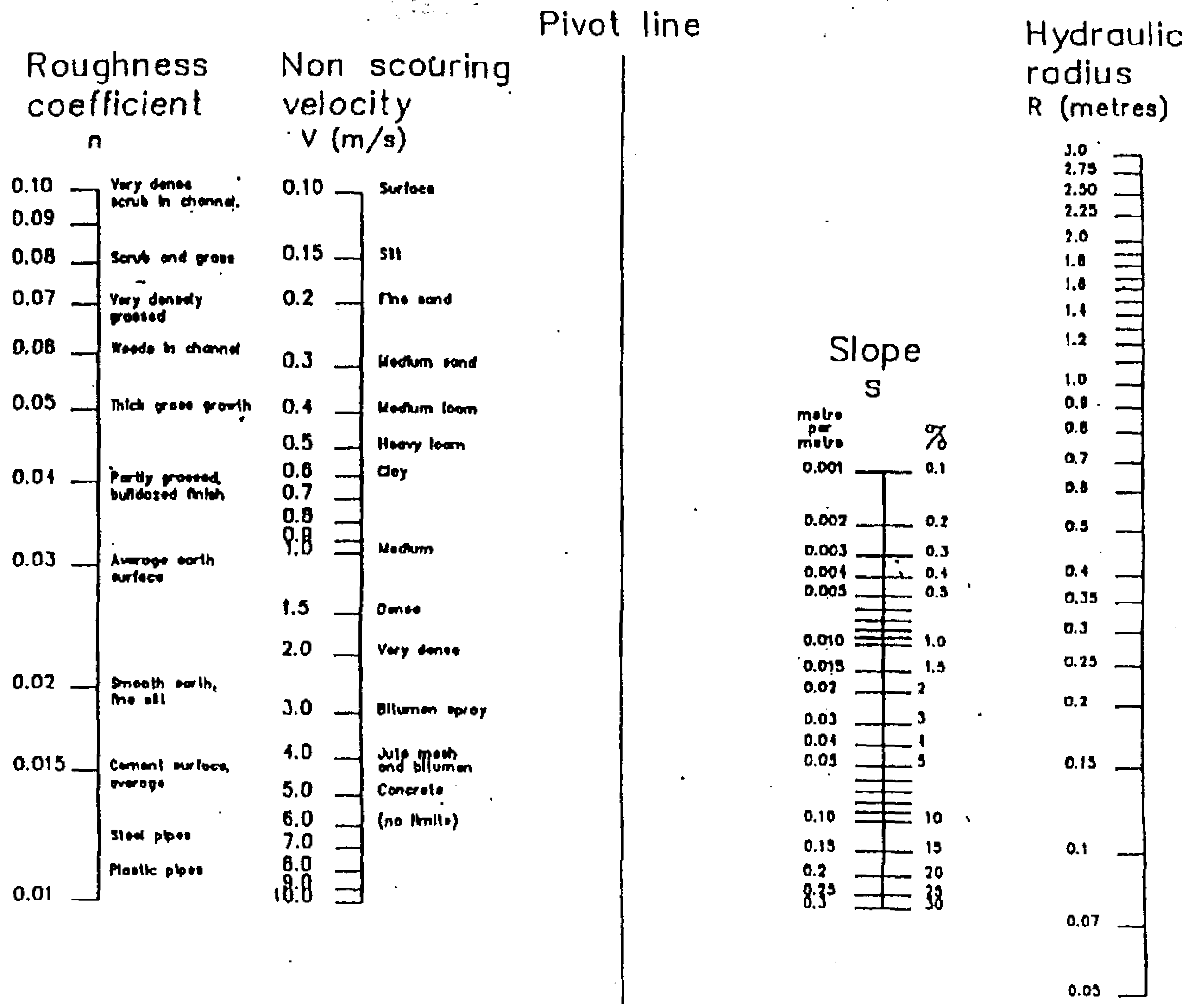
Manning's Formula

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

V–Velocity in m/s
R–Hydraulic radius in metres
S–Slope in metres per metre or in %
n–Roughness coefficient

KEY





Nomogram for Manning's formula for velocity of flow in channels and pipes.

Fig. 3.1

3.2.3.2 Selection of Runoff Estimation Method

For small catchments (e.g. less than 50 ha contributing area) use of the Probabilistic Rational Method as described in Australian Rainfall and Runoff (ARR), 1987 (Chapter 14) is suggested. For catchments larger than 50 ha the use of a runoff routing model may be advisable (i.e. synthetic hydrograph approach). Again, this size division is only arbitrary and depending on the importance of the control measure, the designer is ultimately responsible for selecting a method of appropriate accuracy. In many cases the relevant hydrological information (peak flows, volumes etc.) will be included with the engineering plans and calculations. The use of this information is generally encouraged as the consistency of data is likely to be enhanced.

Section 14.5.5 of AAR, 1987, (especially Technical Note 6) gives a worked example of the calculation of runoff using the Rational Method.

3.2.3.3 Intensity/Frequency/Duration (IFD)

In the use of the Probabilistic Rational Method, a design storm of duration corresponding to the time of concentration of the contributing area should be used to determine the rainfall intensity for the selected design ARI (frequency). The rainfall intensity is the average rate over the duration of the whole storm.

On the other hand, when runoff routing is used to determine peak flows and volumes, it is usually necessary to try several rainfall durations with corresponding temporal patterns. It is often found that peak flows and volumes result from storms of around 2 to 3 times the time of concentration of the catchment.

Time of concentration (t_c) may be determined by either of the following methods, (from AAR,1987) as appropriate:-

- For largely rural catchments in eastern N.S.W,

  $t_c = 0.76A^{0.38}$ where: A = catchment area (km^2)

  Guidance in the determination of rural "times of concentration"(t_c) is given in Section 14.5.5. of AAR, 1987.

- For urban catchments, the flow paths and times must be estimated from the top of the catchment (longest flow path) taking into account overland flow, channelled flow (rills, channels, gutters, etc.) and pipe flow components. Guidance in the determination of urban times of concentration is given in Section 14.5.5 of ARR, 1987.

The corresponding intensities and temporal patterns for the selected durations should be determined by the methods of Chapters 2 and 3, ARR, 1987.

3.2.3.4. Runoff

In the use of the Probabilistic Rational Method (Section 14.5.5., ARR, 1987) the following direction on runoff coefficients is given:

* **Existing Urban Areas:** use Figure 14.13, ARR, 1987, as appropriate for the specific locality concerned.

* **Undisturbed Areas:** use Figure 14.13, ARR, 1987, with impervious fraction =0.0.

* **Disturbed Areas:** use Table 3.2 for the estimation of the effective impervious fraction with Figure 14.13, AAR, 1987.

TABLE 3.2

RATIONAL METHOD C VALUES - DISTURBED SITES

Surface Type (For use with Figure 14.13 ARR 1987)	**Effective Impervious Fraction**
Bare packed soil	
smooth	0.25 - 0.55
rough	0.15 - 0.45
Temporary vegetation	
heavy soil	0.15 - 0.45
sandy soil	0.05 - 0.20
Established Pasture	
heavy soil	0.10 - 0.40
sandy soil	0.00 - 0.20

After Goldman et al. (1986)

For composite areas consisting of more than one surface type an area weighted mean runoff coefficient can be determined from Equation 3.3:-

$$C_{AVE} = \frac{I= 1,n\ \Sigma(C_iA_i + C_{i+1}\ A_{i+1}\ .....+ C_nA_n)}{I= 1,n\ \Sigma(\ A_i + A_{i+1}..................+A_n)} \qquad \textbf{EQUATION 3.3}$$

In the determination of runoff for large catchments, the use of a synthetic hydrograph method to derive peak flows and runoff volumes should be considered. Whilst the use of manual calculation methods is quite practicable, computer models (e.g. RORB, RAFTS, WBNM & BASIN) are available to facilitate the process.

Guidance on manual runoff routing calculations is given in Section 8.6 of ARR, 1987. Where computer models are available, and particularly where the work can be co-ordinated with other runoff calculations for engineering works, this approach is preferred. Section 14.2 of ARR, 1987 gives direction on the types of models and their application.

3.2.3.5 Design Procedure for Flows in Open Channels

The equation generally applied to open channel flow situations is the Manning Formula (from Henderson, 1966):

$$Q = \frac{AR^{2/3}\,So^{1/2}}{n} \qquad \text{EQUATION 3.4}$$

Where: Q = Flow rate (m^3/s) A = Flow area (m^2)
So = Bed slope n=Manning roughness coefficient
R = Hydraulic radius of flow area (m)
[$R = \frac{A}{P}$ where P = wetted perimeter of flow area (m)]

In addition, the Chezy Formula and the Colebrook-White Equation (Section 4.6, ARR, 1987) can be used with acceptable accuracy. Colebrook-White, which is also used to describe pipeflow friction, can be adapted by substituting 4R for the pipe diameter in the equation.

Table 3.3 gives a range of values for Manning's "*n*" for various surface and stream types. Nomograms for the solution of Manning's formula are, however, readily available. Figure 3.1 is an example (from Soil Conservation Service of N.S.W. 1982) which can be used to design for most situations including part-full pipe flow.

TABLE 3.3
VALUES OF MANNING'S - ROUGHNESS COEFFICIENT "*n*"

Glass, plastic, machined metal	0.010
Dressed timber, joints flush	0.011
Sawn timber, joints uneven	0.014
Cement plaster	0.011
Concrete, steel trowelled	0.012
Concrete, timber forms, unfinished	0.014
Untreated gunite	0.015-0.107
Brickwork or dressed masonry	0.014
Rubble set in cement	0.017
Earth, smooth, no weeds	0.020
Earth, some stones and weeds	0.025
Natural river channels:	
Clean and straight	0.025-0.030
Winding, with pools and shoals	0.033-0.040
Very weedy, winding and overgrown	0.075-0.150
Clean straight alluvial channels	$0.025d^{1/6}$

After Henderson, 1966

The design process commences by adopting a given gradient and trial section, computing R and entering the nomogram (Figure 3.1) from the RHS. By using the pivot line and the estimated Manning "n" value, the resulting velocity can be checked against the non-scouring value for a particular surface type. Conversely, an "n" value and velocity can be adopted for a selected surface and the slope and required cross-section determined from the RHS of the nomogram.

Freeboard is generally added to the calculated flow depth as 50 per cent of this depth, with a minimum freeboard of 0.20 m applying.

3.2.3.6 Permissible Flow Velocities in Vegetated Channels

Suggested permissible velocities of flow under various vegetative conditions are listed in Table 3.4.

TABLE 3.4

MAXIMUM PERMISSIBLE VELOCITIES FOR EARTH CHANNELS (M/SEC)

		Erodibility Assessment			
Cover	Channel Slope (%)	Low	Moderate	High	V.High
Bare soil	0-10	0.7	0.6	0.5	0.4
Kikuyu and other dense high-growing, prostrate perennials	0-5 5-10	2.6 2.5	2.4 2.3	2.3 2.2	2.2 2.1
Couch and other low-growing, prostrate perennials	0-5 5-10	2.1 1.6	2.0 1.5	1.9 1.3	1.7 1.1
Perennial improved pastures	0-5 5-10	1.7 1.6	1.6 1.5	1.4 1.3	1.2 1.1
Native tussocky grasses sparse legumes and annuals	0-5 5-10	1.4 1.3	1.2 1.1	1.0 0.9	0.8 0.7

From Soil Conservation Service, 1991

3.2.3.7 Estimating Soil Loss

Estimating soil loss from an urban construction site, in conjunction with estimating stormwater runoff, can provide valuable data for use in urban drainage design and the preparation of erosion and sediment control plans.

This data can assist the site planner to:-

- Avoid highly erodible areas where sediment generation will be difficult and/or costly to control.
- Select the most effective control measures for a particular site, by comparing the efficiencies of various measures.
- Estimate the volume of sediment storage needed in a sediment basin, thus avoiding undersizing or costly oversizing of basins.

Several methods of assessing soil loss have been developed. The Universal Soil Loss Equation (USLE) is an empirical model developed to estimate average annual soil loss from sheet and rill erosion. The equation is a simple arithmetic relation composed of five factors that influence erosion, and can be stated as follows (from Wischmeier and Smith, 1978):-

$$A = R \times K \times LS \times C \times P \qquad \textbf{EQUATION 3.5}$$

where

A =	annual rate of soil loss	[t/ha]
R =	rainfall erosivity factor (storm kinetic energy x 30 min. maximum rainfall intensity)	[MJ.mm/(ha.h.yr)]
K =	soil erodibility factor	[t.ha.h/(ha.MJ.mm)]
LS =	combined length/slope factor	[dimensionless]
C =	soil cover factor	[assumed 1.0 - dimensionless]
P =	soil conservation factor	[assumed 1.0 - dimensionless]

There are limitations to the use of the USLE in the urban context (Goldman et al, 1986), principally because:

* **it predicts average annual soil loss only,**
* **it estimates sheet and rill erosion, not gully erosion,**
* **it does not calculate sediment yield, only soil loss.**

Nevertheless, for small urban catchments where most of the eroded soil is delivered to a sediment basin, the USLE can be used as a conservative measure of potential sediment storage needs.

The Soil Conservation Service Technical Handbook No. 11 (SOILOSS) includes a computer program to assist in the use of the USLE. It details methods of determining the various factors used in the equation, particularly the critical soil erodibility factor

(K) where four options for determination are presented. SOILOSS has been optimised for New South Wales conditions.

For further guidance in the use of the USLE refer to the Soil Conservation Service Technical Handbook No. 11 and to Goldman et al. (1986).

3.2.3.8 Settling Velocities

The settling velocity of particles in suspension is fundamental to the design of sediment retention structures. Settling velocity is a function of size, shape and density of the particles, the density and turbulent state of the fluid, concentration, distribution and ionic strength of the suspension.

Settling velocity is determined by Stoke's Law, a theoretical relationship applicable to smooth spherical particles and ideal fluids.

In practice, to allow for the adverse effect of turbulence on basin efficiency, the sediment basin surface area formula (Goldman et al, 1986) is used:-

$$As = \frac{1.2Q}{V_s} \qquad \textbf{EQUATION 3.6}$$

where
- As = basin surface area (m^2)
- Q = runoff rate (m^3/sec)
- V_s = settling velocity for appropriate particle size (m/s)

Theoretical settling velocities for mineral particles, and corresponding sediment basin surface area requirement per unit discharge, as per this equation, are listed in Table 3.5.

Strongly aggregated soils will have a settling velocity range greater than that predicted by the dispersed particle size distribution. Aggregates of equivalent sizes to primary particles will have lower settling velocities due to reduced densities. Flocculation, due to high concentrations of very fine particles and/or high ionic strength, increases the settling velocity.

Where soils have a high clay or fine silt content, increasing the size - and thus the cost - of a basin will not bring about a proportional increase in basin efficiency. A balance between the cost-effectiveness of a certain basin size and the desire to capture very small soil particles (clays and fine silts) must be achieved.

TABLE 3.5

THEORETICAL PARTICLE SETTLING VELOCITIES AND SEDIMENT BASIN SURFACE AREA REQUIREMENTS

PARTICLE SIZE (mm)		SETTLING VELOCITY (m/sec)	BASIN SURFACE AREA REQUIRED ($m^2/m^3/sec$)
0.500	Coarse Sand	0.058000	20.7
0.200		0.020000	58.7
0.100	Fine Sand	0.007000	171.0
0.050		0.001900	635.0
0.020		0.000290	4101.0
0.010	Silt	0.000073	16404.0
0.005		0.000018	64617.0

after Goldman et al. 1986

It is therefore recommended that 0.02 mm be used as the design particle size for sediment basin design.

For coarse soils, where a larger design particle size may be more appropriate, particle size analysis data for subsoils from within the basin catchment area will be required if the design particle size of 0.02mm is to be amended. The appropriate design particle size (from Table 3.5) will be that for which at least 70% per cent of particles are coarser, with a minimum particle size of 0.02mm.

3.3 EROSION CONTROL MEASURES

Sediment can only be generated when soil erosion occurs. The prevention or minimising of soil erosion must therefore be the first priority of any erosion and sediment control strategy.

It is environmentally sound, easier, and more cost-effective to prevent erosion than to concentrate on trapping sediment from eroding areas. This applies particularly to areas where the soils have a high proportion of fine silts and clays, or are dispersible. Indirectly therefore, erosion control measures also control sediment.

These measures generally function by reducing the duration of soil exposure to erosive forces, either by holding the soil in place, or by shielding it. The staging of land shaping or regrading operations and immediate revegetation will help protect the soil surface from raindrop impact and minimise the area of soil exposed at any one time. The protection of the soil from surface runoff may be accomplished by interception, diversion and safe disposal of runoff in co-ordination with staged construction activities, designed land shaping methods, and the preservation of natural vegetation. Measures to be used include a variety of construction practices, structural controls and vegetative measures aimed at managing runoff at a non-erosive velocity, and the protection of disturbed soil surfaces.

Erosion control measures can either be temporary or permanent and ideally, used as primary techniques forming the basis of an erosion and sediment control strategy, reducing the reliance on, or need for, sediment control measures. Erosion control measures do, however, require regular maintenance to ensure their effectiveness.

The following measures and practices can be integral components of an effective erosion control program.

3.3.1. SITE MANAGEMENT

The way in which a construction or development site is managed can markedly influence the impact of that activity on the local environment.

Good site management, supervision and construction practice, in accord with the general principles of erosion and sediment control (Section 1.1) can bring improvements in the construction sequence. It can reduce the time spent on repairs and excessive maintenance of the site, its structures, and adjacent areas, thereby reducing costs to the developer and to the community.

The integration of the above principles with site management can be achieved if the following practices are implemented:-

- The contractor should appoint an appropriate supervisor or superintendent to be responsible for the implementation of the Urban Erosion and Sediment Control Plan. This person can ensure the integrity of control measures through regular inspections and maintenance, and by anticipating erosion control requirements throughout the construction sequence.

- A temporary construction exit (Section 3.4.4) should be installed at major exits subject to high traffic use where sediment could be carried onto adjacent road systems.

- All access to the site should be controlled and vehicles and plant kept to well defined haul roads, to avoid disturbing additional ground.

- All existing vegetation to be retained should be protected by fencing off or by defining those areas which will not be affected by construction.

- The construction sequence should allow for all permanent drainage, erosion and sediment control measures to be installed as soon as practical.

- All site topsoil should be retained and protected.

- Land shaping should aim to minimise slope lengths and gradients.

- Disturbed areas should be left in a roughened surface condition to reduce runoff velocity and increase infiltration.

- Temporary sediment trapping structures should be used as and where required.

- Soil stockpiles should be stored outside hazard areas such as drainage lines and protected from erosion.

- The development plan should specify the use of construction procedures which minimise the concentration of surface runoff and excessive velocities, which could cause erosion.

- The sequence of works should be both realistic and practical, so that progressive construction and stabilisation are feasible.

- Appropriate methods should be employed to prevent blowing dust creating an unacceptable hazard or nuisance on the site or down-wind.

- Prompt revegetation or mulching of disturbed areas should be encouraged.

- Site supervision should be flexible to accommodate changes in the construction sequence. New or modified temporary erosion and sediment control measures may be required if flow paths are changed, or when site works result in the temporary removal of a sediment trap or diversion channel.

- Advice on erosion and sediment control techniques is available from local offices of the Soil Conservation Service of NSW.

3.3.2 LAND SHAPING

Land shaping refers to the reforming of the ground surface to provide more suitable sites for buildings, service facilities or associated landuses, to improve surface drainage, or to develop road or pedestrian access. It involves cutting and filling to planned grades.

Development design should use the existing topography and natural features to avoid extremes, shaping only those areas which are to go under immediate construction. This will greatly assist in controlling erosion.

All land shaping should be in accordance with a plan co-ordinated with engineering layout and design. Based upon adequate topographic survey and soils investigation, this plan should show the location, slope gradient, and existing and proposed contours of the areas to be shaped. It should also show procedures for erosion control, batter stabilisation, surface and subsoil drainage, sediment control, and revegetation procedures. The plan should also include the scheduling and phasing of these practices.

There are a range of preferred measures in land shaping which should be incorporated in this plan:

- Standing vegetation which has not been identified for retention, as well as logs, stumps, rubbish and other vegetative matter which will interfere with land shaping or may affect the stability of fill, should be removed and stockpiled for disposal.

- Topsoil should be stripped and stockpiled for later re-spreading on all exposed areas when final shaping has been completed. Sediment control measures must be incorporated with any resulting stockpiles.

- Before stripping topsoil, reduce remaining vegetative cover by grazing, burning, or mowing (preferably with the removal of mown material). Excessive vegetative growth makes topsoil removal more difficult. Also, large quantities of green matter in stockpiles promotes chemical and biological degradation of plant material such as runners and root stocks, which would otherwise be a source of regrowth when topsoil is respread. (Section 4.3)

- Isolated large rocks should be removed from fill material. They may be a major threat to batter stability if the fill becomes saturated with water, and could initiate slope movement.

- Fill material should not be placed around, or up against, existing trees and shrubs.

- Fill material should not be placed adjacent to the bank of a channel or stream where it may cause bank failure, reduce the capacity of the stream, or provide a source of sediment.

- All fill should be compacted sufficiently for its intended use and as required to reduce sloughing, erosion or excessive saturation. It should be compacted in successive layers.

- Cut and fill batters should be formed to a stable slope consistent with soil properties, and be adequately protected from erosion by vegetation or by structural measures. (Section 3.3.3)

- Land shaping operations should leave the reshaped surface in a roughened condition to encourage infiltration and minimise runoff.

- Land shaping operations should not be carried out close to property boundaries unless adequate protection can be provided against erosion, slippage, subsidence or the settlement of the batters already formed.

- During land shaping, all disturbed areas which are not to be developed immediately should be temporarily protected from erosion by establishing vegetative cover, or by applying a surface mulch of hay or a chemical stabiliser such as bitumen emulsion (see Section 4.5).

3.3.3. BATTER STABILISATION

Batters are earth embankments or cuttings produced by cut and fill earthworks during land shaping (Figure 3.2). They represent a critical exposed soil surface which requires specific design and stabilisation considerations.

Batters must be designed to satisfy stability requirements, while allowing for future maintenance. They should be formed to a stable slope (consistent with site conditions) determined after investigations into topography, soil type and the presence of rock. Generally speaking, they should have a maximum gradient of:

* **1(vertical):2(horizontal) on soils with a low erosion hazard.**
* **1(v):3(h) on soils with a high erosion hazard.**
* **1(v):4(h) on soils with an extreme erosion hazard.**

For batters steeper than those described above, geotechnical advice should be sought to ascertain the capability of the soils for such use, and to outline appropriate stabilisation requirements. The following principles of batter construction and stabilisation should be incorporated in a development plan:-

- Provision must be made to prevent surface runoff damaging cut and fill batters. Catch drains, diversion banks and channels above and below batters, and berms within them, will intercept surface runoff and conduct it to safe disposal points. This will reduce the hazard of sheet erosion and batter slump.

- With cut batters, a catch drain or diversion bank should be constructed above the top of the cut prior to excavation. Temporary toe drainage should be maintained as the work progresses, with the permanent toe drainage installed when the final depth is reached.

- For fill batters, permanent toe drainage should be installed at an early stage and be discharged to a suitable outlet. At the completion of each work period, or at the onset of rain, a windrow of suitably compacted soil material should be constructed along the edge of the batter to prevent drainage water from passing down the fill slope. Permanent top drainage measures should be installed on completion of the filling operation.

- Chutes or pipe drops may be required at points along a catch drain or channel above a batter to allow safe disposal of runoff down the face of the batter. (Section 3.3.10)

- Berms or benches are recommended on batters with a vertical height greater than 5 m. The bench should be at least 2 m wide, but additional width may be necessary to allow for the movement of equipment used to establish and maintain vegetation on the batters. Berms should have a minimum longitudinal grade of 1 per cent if vegetated, or 0.5 per cent if paved. The maximum grades should be restricted to a level consistent with the maximum perishable velocity for the type of lining used. A maximum lateral slope of 10 per cent towards the toe of the upper batter should apply.

- Permanent diversion channels should be protected from scouring by appropriate lining or by sowing vegetative cover. (Section 3.3.9) In the latter case, jute mesh and bitumen emulsion can be incorporated to provide temporary protection while vegetation is establishing. (Section 4.5)

- Early stabilisation of exposed batters is essential. They should be adequately protected from erosion by vegetation within fourteen (14) days of their construction, or by structural measures.

  Techniques available for vegetative stabilisation of batters include hydroseeding, turfing, the use of hay and bitumen, hay and wire netting or erosion matting fabrics, applying bitumen only, and direct planting. (Section

4.5) Where possible, cut batters which are to be topsoiled should be scarified to a minimum depth of 100 mm prior to the placing of topsoil. Topsoil can be successfully spread and compacted using a heavy chain.

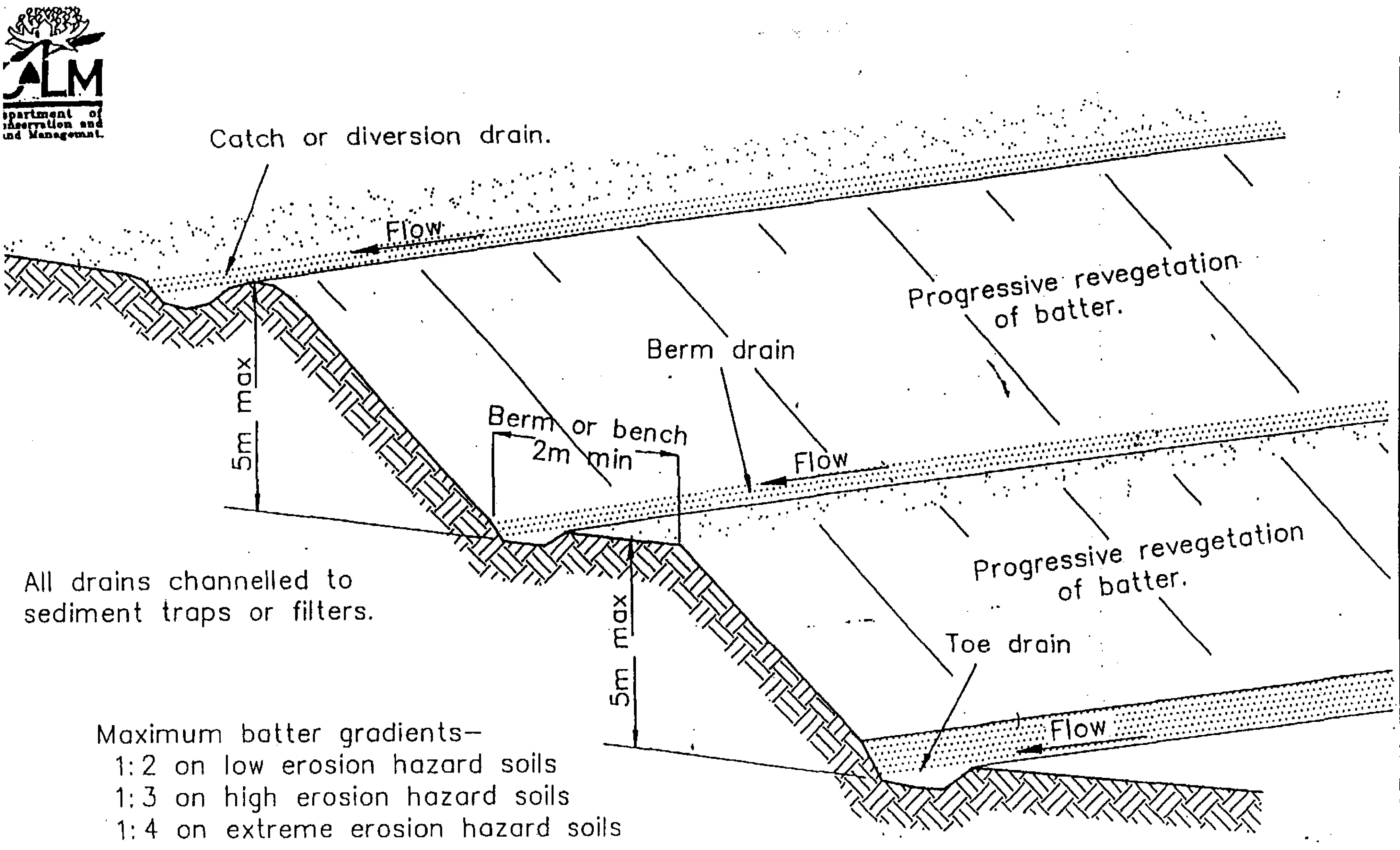
- Where batters are formed in soil materials which may be unstable, engineering techniques involving measures such as subsoil drainage and construction of retaining walls will often be required. In such circumstances, competent geotechnical engineering advice should be obtained before any shaping commences.

  If, in the above circumstances, rapid stabilisation is not critical, planting trees and shrubs with a deep, fibrous rooting system can be a suitable alternative to the use of engineering techniques. Deciduous species such as silver leaf poplar (*Populus alba*) and shrub willow (*Salix spp*) can be effective in some soil types. Their roots bind the soil and, if soil movement severs them they rapidly grow again. However, there can be some problems in controlling the suckering of these species, and penetration of their roots into nearby drains can also pose a hazard. Several native evergreen species (*Casuarina, Acacia, Melaleuca*, etc.) also have the potential to de-water slopes, as well as being very suitable for certain difficult soil types.

  More information on the management of development on unstable land is provided in Section 2.2.2.

- The maintenance of a healthy ground cover is desirable on batters stabilised by vegetation. Mowing, intermittent application of fertiliser and - where practical - irrigation, can all assist.

The functioning of drainage systems which are installed should be regularly checked. Batter sections which slump should be reshaped and stabilised as early as possible, before adjacent sections are adversely affected. Soil Conservation Service of N.S.W. (1992 - in press) details methods of batter stabilisation associated with road construction.



Source: Goldman et.al.

Batters should be formed to a stable slope and progressively revegetated.

3.3.4. BANKS AND CHANNELS

A bank is a constructed ridge or embankment of compacted earth. A channel is an excavated earth drainage ditch or path. Either individually or in combination, these structures are used to intercept and direct runoff water to a desired location. By doing so, they convert sheet flow to concentrated flow, and increase the time of concentration of runoff.

They can be used to:-

- Intercept and direct sediment laden runoff from disturbed areas to an appropriate sediment trapping structure.

- Intercept and direct clean runoff from above disturbed areas to a stabilised outlet at a non-erosive velocity.

- Reduce the flow path length and velocity of runoff flowing down a slope or along a graded right of way.

Under the Water Act (1912) it is illegal to divert water from one catchment to another, changing the discharge point. Care must therefore be exercised to ensure that runoff leaves a property at the same location before and after development work has been completed.

There are two major types of banks and/or channels:-

* Perimeter bank
* Diversion bank/channel

3.3.4.1 Perimeter Bank

This is a temporary earth bank located around the perimeter of construction sites or around disturbed areas within the site. It prevents sediment laden runoff from leaving a construction site or disturbed area, and prevents off-site runoff from entering. Stormwater runoff prevented from entering a disturbed site by a perimeter bank should be directed to a stable disposal area.

Perimeter banks are small-scale temporary structures, and therefore do not require formal design. They may be reconstructed daily to protect exposed areas as site grading proceeds. However, as they convert sheet flow to channel flow, they do increase the erosion potential in the channel. The following general design guidelines should therefore be observed to ensure their proper function.

An earth perimeter bank should only be used where the contributing catchment is 2 ha or less. It should remain in use until the disturbed area is permanently stabilised.

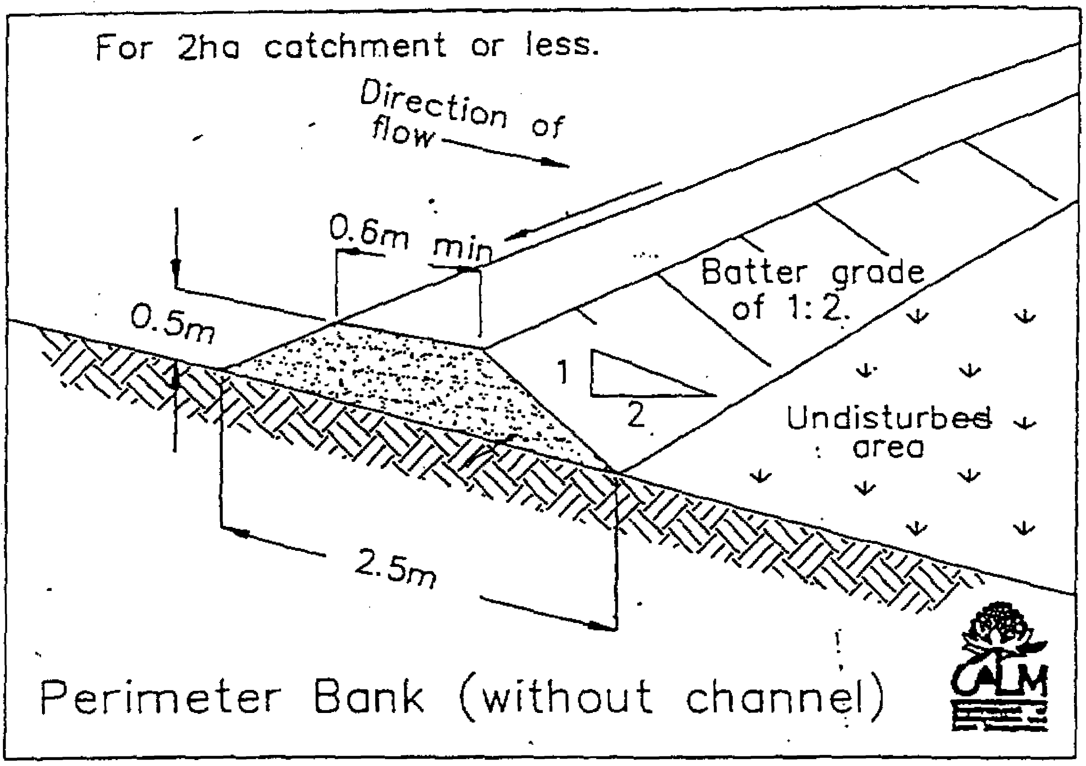


Fig. 3.3(a)

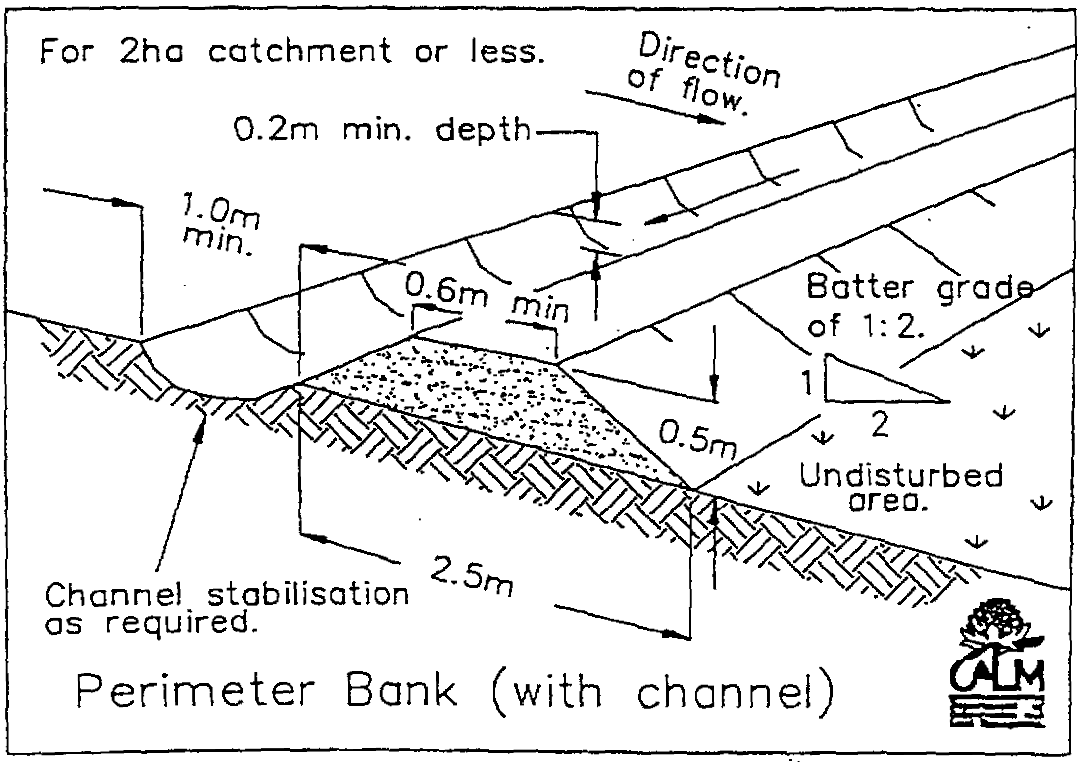


Fig. 3.3(b)

Where the contributing drainage area exceeds 2 ha, diversion banks are required to control runoff.

An earth perimeter bank may be formed using a grader or a front-mounted blade on a tractor or bulldozer. After the earth has been pushed up, it should be compacted by the machine. The resultant excavated channel upslope should be parabolic or trapezoidal in cross-section. An earth perimeter bank may also be constructed by importing soil material and forming a bank without an excavated channel. (Figure 3.3)

The compacted height of the bank should be at least 0.5m. Batter slopes should be no steeper than 1:2 and may have to be reduced to a much lower gradient at points where vehicle crossings are required. The top width of the bank should be at least 0.6m.

If an excavated channel is incorporated into the construction of the bank, the channel should be at least 0.2m deep with side slopes of 1:2 or flatter. The width of the channel should be at least 1.0m.

The grade of the bank and channel behind it will depend to some degree on site topography, but there should always be a positive grade of at least 0.5 per cent to the outlet point. If channel grade exceeds 5 per cent, special stabilising measures, such as the use of jute mesh, or lining with gravel, may be necessary to prevent scouring. (Section 3.3.9)

For banks draining disturbed areas, sediment laden runoff should be directed to a sediment trap or sediment basin. (Sections 3.4.2 and 3.4.1) Runoff diverted from protected or stabilised areas, on the other hand, may discharge directly onto a safe disposal area, or into a level spreader or a grade stabilising structure. (Sections 3.3.5 and 3.3.10)

Periodic inspections should be made of earth perimeter banks to repair damage caused by scour, sediment deposition, channel obstruction, excessive traffic or loss of freeboard.

3.3.4.2 Diversion Bank/Channel

A diversion channel is an earth channel with a minor ridge on its lower side constructed across the slope. It is designed to protect slopes or development works below it by intercepting surface runoff and diverting it to a stable outlet at a non-erosive velocity.

Where flows are too large to be contained by a simple channel, a diversion bank is constructed. The operating principle is the same, but it comprises a larger channel behind a substantial bank which is pushed up in the course of excavating this channel. Bank height and channel cross-section depend on the size of flows that must be contained.

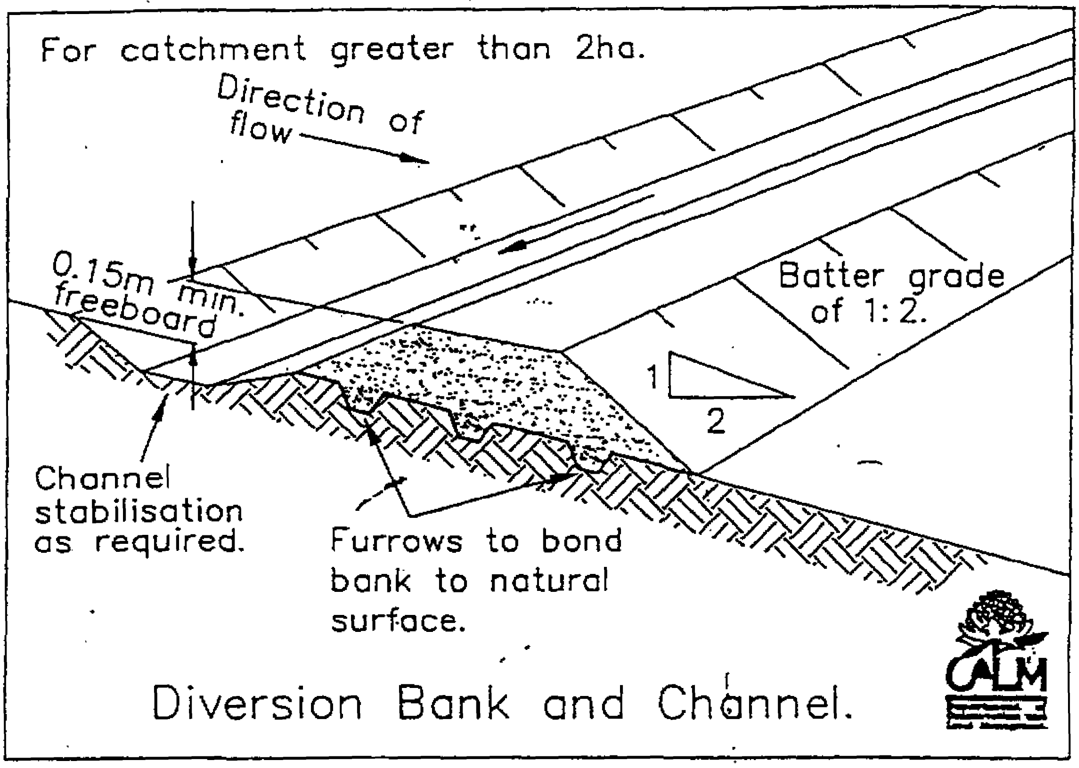

For catchment greater than 2ha.
Direction of flow
0.15m min. freeboard
Batter grade of 1:2.
1
2
Channel stabilisation as required.
Furrows to bond bank to natural surface.
Diversion Bank and Channel.

Fig. 3.4

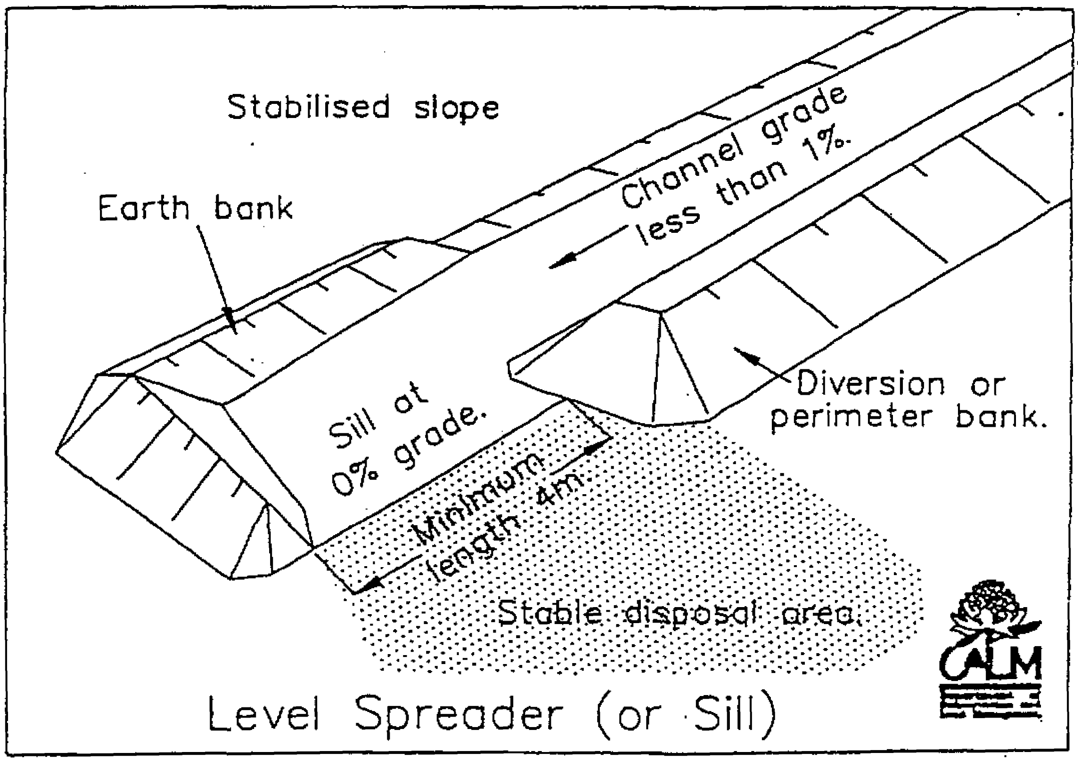

Stabilised slope
Earth bank
Channel grade less than 1%.
Sill at 0% grade.
Minimum length 4m.
Diversion or perimeter bank.
Stable disposal area.
Level Spreader (or Sill)

Fig. 3.5

Diversions may be temporary or permanent structures. They are used where perimeter banks are not viable, particularly where the contributing drainage area exceeds 2 ha.

Whether temporary or permanent, a formal design should be undertaken for all diversions. Section 3.2.3 and Tables 3.1 and 3.4 outline appropriate design criteria.

The average flow velocity in a vegetated channel should not exceed 2.5 m/sec and, if possible, should be kept below 2 m/sec if channel erosion is to be avoided. (Table 3.4)

The diversion channel should be parabolic or trapezoidal in shape. Its side slope should be flat enough to permit maintenance of the structure and its protective vegetative cover, but not steeper than 1:2. (Figure 3.4)

If a bank is constructed in conjunction with the diversion channel, it may be a ridge-type bank with steep batters rising to a narrow crest, or it may be broad-based with gentle batters rising to a gently rounded crest. The latter can be traversed by vehicles or machinery, and lends itself to the establishment and maintenance of grass cover on the batters and crest. Banks should have a minimum freeboard of 0.15m.

The size of these banks will vary according to the topography and the amount of runoff they are designed to control. Their design and construction are described in more detail in Soil Conservation Service Technical Handbook No.5.

For diversions draining disturbed areas, sediment laden runoff should be directed to a sediment basin or sediment trap. (Sections 3.4.1 and 3.4.2) Runoff diverted from protected or stabilised areas, on the other hand, may discharge directly onto a safe disposal area, or into a level spreader or a grade stabilising structure. (Sections 3.3.5 and 3.3.10). Outlets should be built and stabilised before constructing the diversion.

Diversion channels should be stabilised in accordance with the specifications for grassed waterways (Section 3.3.7) where grades and flow velocities are appropriate,. or as outlined in Section 3.3.9. Subsoil drainage and/or low flow pipes may be required in seepage areas to aid the establishment of suitable vegetation.

Periodic inspections should be made of these structures to repair damage caused by scour, sediment deposition, channel obstruction, excessive traffic, loss of vegetative cover, or loss of freeboard. Occasional mowing may be necessary to control rank growth and to maintain a healthy ground cover in channels.

A permanent diversion bank controlling runoff and protecting road development.

3.3.5. LEVEL SPREADER

A level spreader (often called a level sill) is an excavated outlet constructed at zero grade. It converts an erosive, concentrated flow of sediment free runoff into sheet flow, and discharges it at a non-erosive velocity onto an undisturbed area stabilised by vegetation.

Level spreaders may be used as outlets for diversion or perimeter banks or channels, where storm runoff has been intercepted and diverted to stable areas. They should be used only where the spreader can be constructed on undisturbed soil. The area directly below the spreader sill should be uniform in slope and well vegetated, allowing water to spread out as sheet flow. (Figure 3.5)

The cross-sectional area and length of the level spreader shall be at least sufficient to discharge the design flow from the selected frequency rainfall event. Section 3.2.3 and Table 3.1 outline appropriate design criteria.

The approach grade of the diversion or perimeter bank channel should not exceed 1 per cent for at least 6 m before it enters the spreader. Final discharge is over a level sill onto a stable undisturbed area. Extra protection by jute mesh, grass sod, rip-rap or some other appropriate stabiliser may be required below the sill area.

To maintain its efficient operation, particular attention should be paid to the sill to ensure that it remains stable and that a vigorous vegetative cover is maintained below it. The channel behind the sill may require periodic de-silting to retain an effective capacity.

3.3.6. REVEGETATION

Establishing vegetation quickly on all disturbed areas is the most effective means of preventing soil erosion. By effectively reducing runoff, vegetation minimises the erosion potential of a construction site and can therefore reduce the need for, and size of, structural control measures.

Seeding is often needed on graded or cleared areas and for stockpiled soil materials, to provide temporary surface protection during delays in construction, or until conditions are right for permanent revegetation. The progressive establishment of permanent vegetation applies to most urban sites as each construction activity is completed.

3.3.6.1. Temporary Seeding

This is the practice of sowing short term vegetation on critical disturbed areas to provide temporary soil stabilisation.

Regraded, cleared or disturbed lands, including stockpiled soil materials which will remain exposed to erosion for a period of 14 days or more, should be rendered erosion resistant as soon as possible. Temporary protection may be provided by seeding with cover crops or annual grass species. This applies even if the area will be subject to further regrading or subsequent construction. In some cases, alternative temporary control measures may be more appropriate.

This practice can significantly reduce the maintenance required for major controls elsewhere on the site, and reduce the need to import topsoil to replace that lost by erosion. The cost savings will usually far outweigh the cost of temporary seeding.

Although detailed revegetation guidelines are presented in Section 4, the following is a brief summary of the relevant aspects that should be addressed in any temporary seeding program:-

- **Site preparation:**
  Although final earthworks and regrading have usually not been completed for temporary seeding, any necessary erosion and sediment control measures should be installed prior to seeding to provide protection to the treated area.

- **Soil ameliorants:**
  For temporary seeding, fertiliser should be applied at the rate of approximately 250 kg/ha using 12:22:0 or similar fertiliser. Gypsum can be used to improve soil structure and reduce soil crusting. (See Section 4.5.3.2) Soils which are highly acid should be limed.

- **Seedbed preparation:**
  The area to be temporarily seeded should be cultivated, raked, scarified or deep ripped to ensure the soil surface is not hard, compacted or crusted and that an adequate seedbed exists.

- **Seeding:**
  The plants most adapted to temporary seeding are quick growing annual grasses and cereal cover crops (see Section 4). Seed should be applied uniformly over the area by direct drilling, broadcasting or by hydroseeding.

- **Mulching:**
  When temporary seeding is undertaken on critical sites, or sites with adverse soil conditions, or during periods of less than optimal seasonal conditions, immediate surface protection will be required. Mulch material should therefore be applied immediately after seeding and could include straw, wood chips, wood pulp, bitumen or other liquid binders, and erosion matting or netting.

3.3.6.2. Permanent Revegetation

This is the practice of planting permanent vegetation on all disturbed areas subject to erosion and where no further regrading or construction activity is proposed. A long-lived vegetative cover is required to provide long term soil surface stabilisation.

This practice can significantly reduce long term environmental degradation from construction sites, particularly downstream sedimentation. It will also reduce the level of on-going maintenance required for any permanent stormwater drainage measures installed.

Permanent vegetation can comprise annual and/or perennial grasses, legumes, ground covers, shrubs and/or trees. Species selection will depend on site factors and the proposed landuse of the area. (Section 4.2)

Detailed guidelines for permanent revegetation are outlined in Section 4.

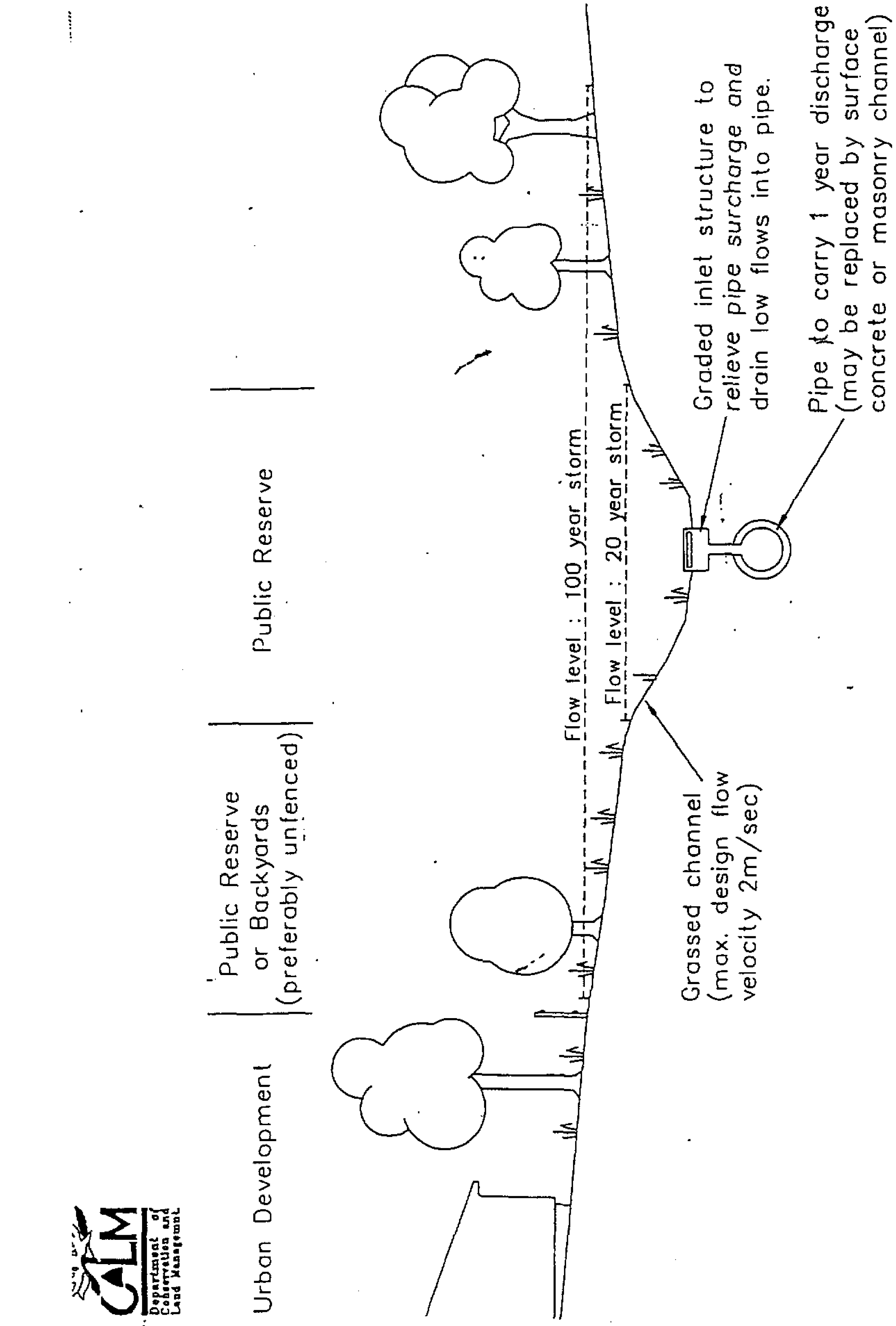
3.3.7 WATERWAYS

Waterways are natural or constructed open channels which function as drainage lines to collect and convey stormwater runoff through, and from, a development site.

The emphasis on the major/minor concept in urban stormwater management (Section 3.2.2.3) and the drainage master planning approach (Section 14.9, ARR, 1987) provides an opportunity for creative management of natural waterways by incorporating dual purpose detention structures and recreation areas at the development planning stage. (Section 3.3.12)

In their natural state, waterways are seldom straight, are of variable cross-section, and generally contain a wide range of vegetation types. From an aesthetic and conservation viewpoint, natural drainage lines should remain in a relatively unaltered state as natural channels provide lag in the form of channel and overbank storage, thus reducing runoff peaks. With the wise use of detention storages at appropriate sites, it is often practical to maintain the natural appearance and flow regime of a channel even though the upstream hydrology may have changed considerably.

Constructed waterways, on the other hand, are usually parabolic or trapezoidal in cross-section and can have varying surface treatments including grass, concrete low-flow channels or complete concrete lining. These waterways often function as trunk drainage links into which piped systems discharge. Urban development brings attendant increases in peak discharges, which has led to the past philosophy of streamlining natural channels to increase flow capacity. This process is ultimately self defeating as it merely relocates the problems further downstream. (Talbot, 1979) The wise use of detention storages - within the developing catchment at the planning stage - tends to remove this snowballing problem, thereby assisting to control costs.



Suggested Design for Grassed Waterway in an Urban Area. Fig. 3.6

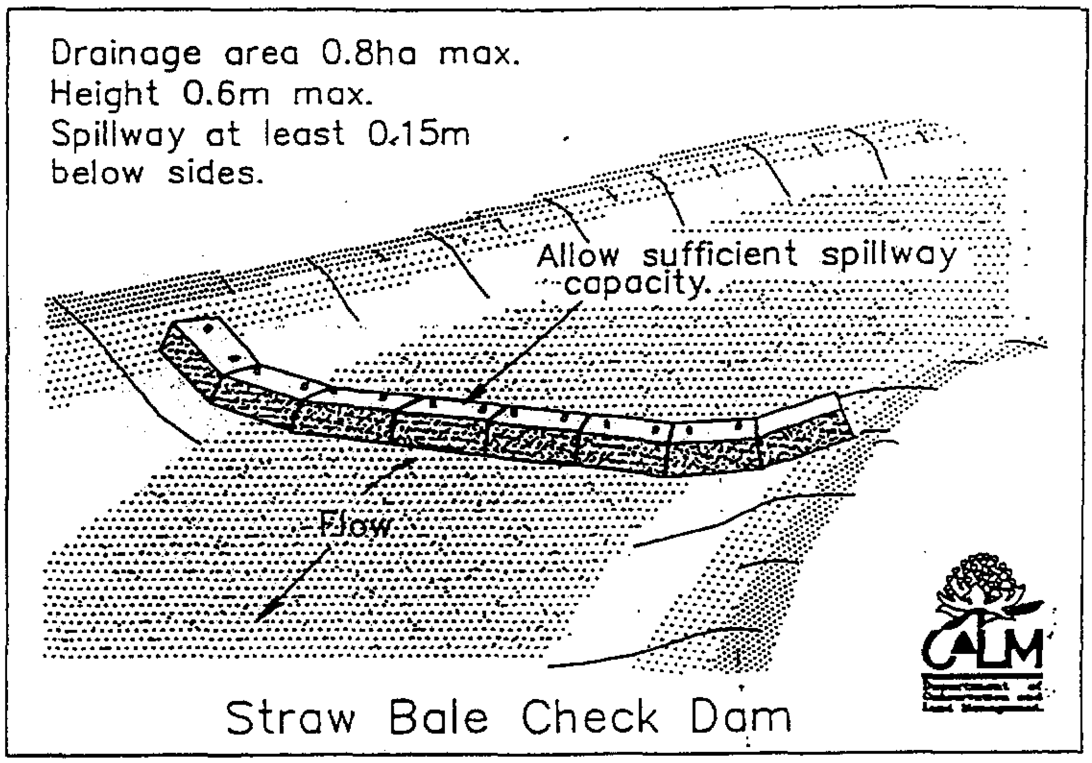


Fig. 3.7(a)

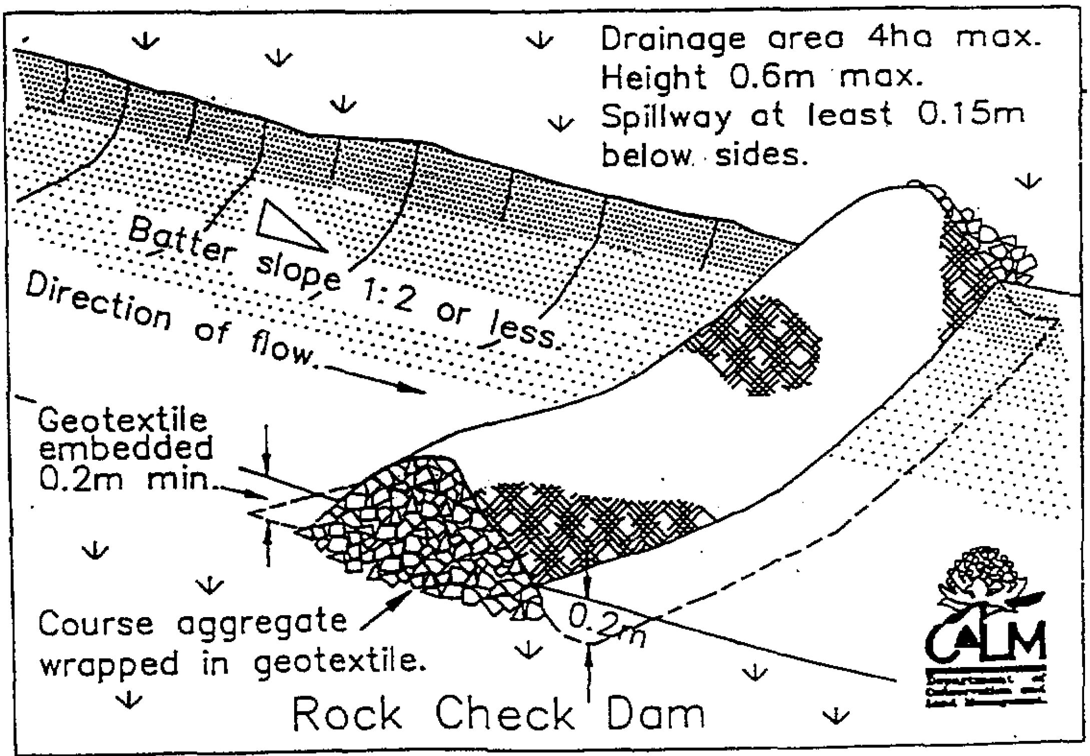
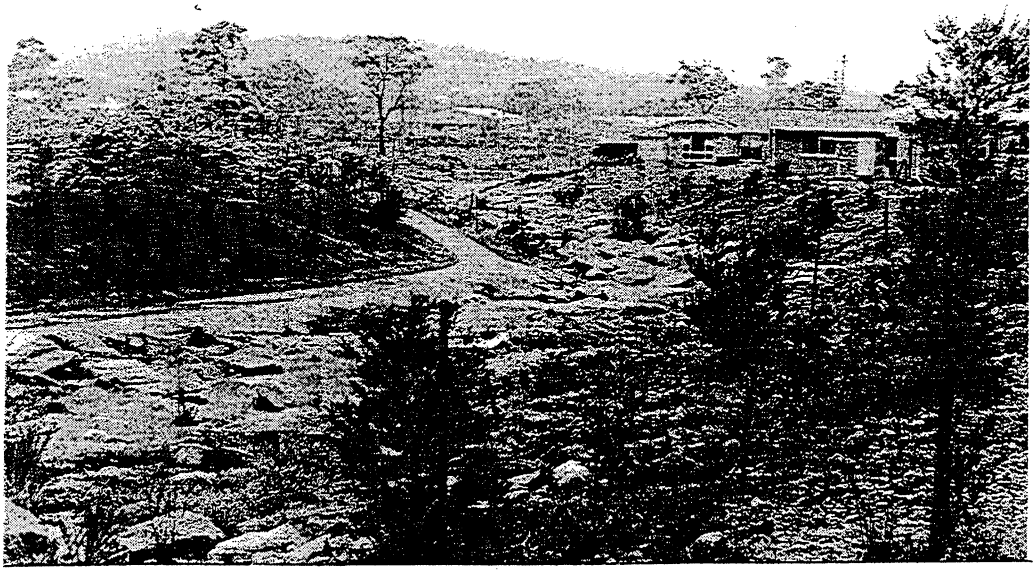
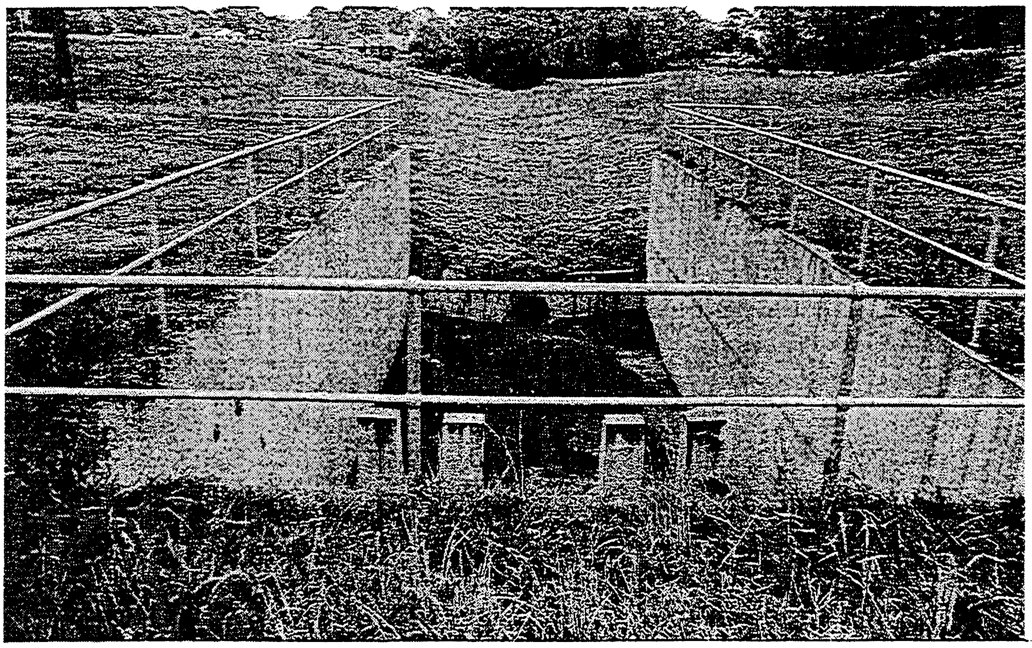


Fig. 3.7(b)



Landscaped urban waterway integrated with subdivision design, affording recreational amenity.



Grassed urban waterway with low flow pipe downstream of outlet protection structure.

Vegetated waterways have several advantages over stormwater pipes or fully lined channels, including:-

- Reduced installation costs.
- Reduced flow velocity, which generates less problems of flooding downstream.
- A greater degree of sediment settlement and filtration of pollutants.

Their major disadvantages are:-

- The increased land area which must be sacrificed to maintain channel capacity with the lower flow velocity.
- The long term maintenance requirements - particularly the mowing of vegetation.

A vegetated waterway can usually be considered where a well defined drainage depression or channel already exists in the natural landform, and where the catchment area is of the order of 5 ha or larger. The channel bed will generally have a grade less than 3 per cent. Where design velocities will prevent stable, non-scouring cross-sections, piping or a lined channel may be preferable to a vegetated waterway. (Section 3.3.9)

3.3.7.1 Design Considerations

Section 3.2.3 outlines general design criteria applicable to these structures. The following additional design considerations are more specific.

(i) Design Average Recurrence Interval

Depending on the site, the design ARI for a waterway can vary from 1:1 year for a short term construction site provision through to 1:100 year for the link of a trunk drainage system. In areas where the slope of adjoining land is less than 1 per cent, the design capacity might not be provided within the actual waterway formation, but should at least be available within the drainage easement containing the waterway. (Figure 3.5) In the case of a trunk drain, a minimum of 100 years is recommended. In some critical large trunk drains probable maximum precipitation events should be considered. (See Section 3.2.3 and Table 3.1) In all cases, trunk drains/waterways need to be designed in accordance with local government requirements and Section 14.6 of AAR, 1987.

(ii) Flow Velocities

Velocities within all sections of the channel should be non-erosive relative to the surface within the waterway. The average flow velocities in vegetated waterways should be limited to those outlined in Table 3.4, (Section 3.2.3). Drops or ledges should be checked to evaluate any potential instability from scour or plunge pool formation under estimated flow conditions.

In constructed waterways, the surface treatment can be selected to suit site constraints, particularly width and gradient. In the case of construction sites, temporary lining material (e.g. plastic sheet) may prove effective in conveying flows at increased velocities through confined areas.

As a general rule, it is advisable to adopt channel geometry which will result in subcritical flow conditions. Supercritical flow, particularly for temporary channels, will eventually cause deterioration and may give rise to safety problems. (See Section 14.10.4, ARR, 1987)

Ground cover and channel grade will influence acceptable flow velocities. Channels with stabilising cover comprising mostly of annuals or of sparse perennials will be subject to erosion at lower flow velocities than those with a cover of mat-forming species such as couch or kikuyu grass. Likewise, steeper channels are liable to scour at a lower flow regime than those with a gentle grade.

With rip-rap lined channels, selection of the appropriate rock size is important to protect against removal of the rock by the stormwater flow at design velocities. Goldman et al. (1986) gives guidance in the design procedure for rock lined channels.

Where smooth, impermeable linings (plastic, concrete, steel, etc.) are used, adequate outlet protection/energy dissipation should be incorporated at any discharge point into an unlined channel. (Section 3.3.11) Artificial roughening of the last section of lining may provide sufficient retardation to protect the receiving channel.

3.3.7.2 Construction of Vegetated Waterways

Trees, stumps and other debris which would obstruct flows should first be removed from the site of a proposed waterway. Topsoil should be removed and stockpiled prior to shaping the channel. This shaping should eliminate irregularities that would interfere with flows, and provide a stable cross-section.

Once the channel has been formed, topsoil should be re-spread and appropriate grasses sown or planted to establish cover.

Continuous trickle flows in a vegetated waterway (a typical urban situation) should be contained in a small pipe installed beneath the waterway, or in a masonry or concrete invert channel or a half-pipe following the centre of the waterway (Figure

3.6). Such continuous flows may otherwise erode the bed of the channel. The high nutrient content of this urban trickle flow also encourages invasion by weeds which choke suburban drainage channels and detract from the general appeal of the grassed easement. The resultant continual wetness also makes mowing and general maintenance difficult and may damage the stabilising grass cover, particularly if the base flow is saline.

If flows in a waterway can be controlled until it is stabilised, seeding and mulching may be used to establish vegetation. If this is not the case, it may be necessary to:

- Plant turf or grass sod to hasten the stabilisation process, or
- Cover seeded areas with a protective mulch such as erosion matting or straw and bitumen.

More detailed information on species selection and on the various stabilising and plant establishment techniques are presented in Section 4.

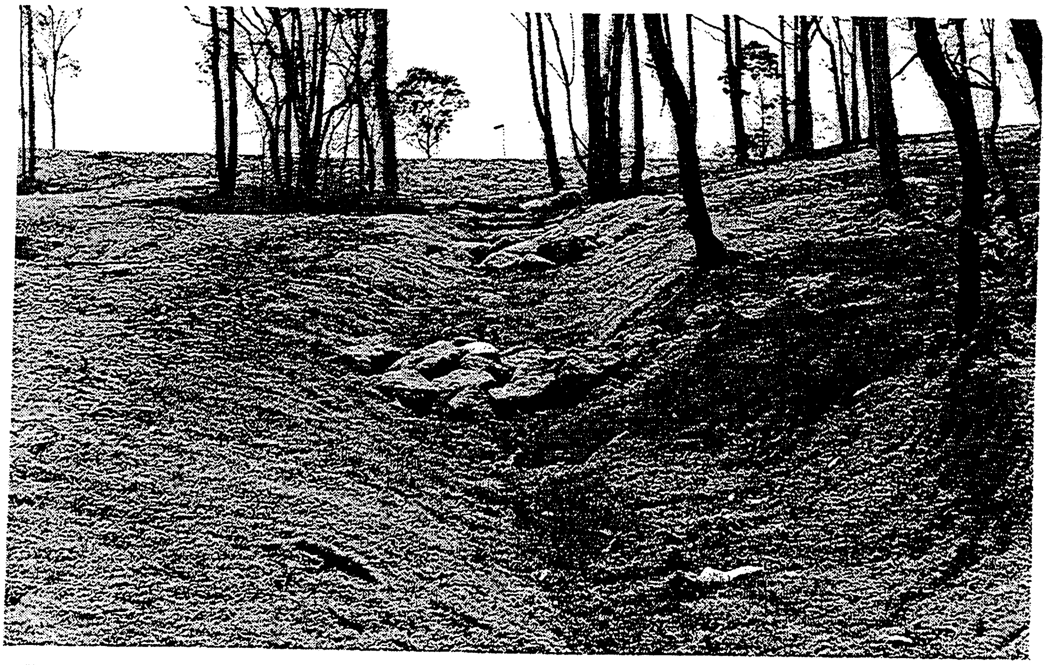
It is desirable to form and stabilise the waterway prior to significant development within the catchment, otherwise high flows stemming from this development will make stabilising more difficult. The potential value of the waterways in promoting filtration of sediment from construction sites may also be lost.

3.3.7.3 Maintenance of Vegetated Waterways

Regular mowing is desirable to control weeds and to maintain an effective cover so that the vegetation does not necessarily obstruct flows and cause flooding or an accumulation of stagnant pools. Maintenance applications of fertilizer may be necessary from time to time, to keep ground cover in a healthy condition.

If bare areas develop in a waterway, they should be resown and covered with a mulch of bitumen and hay, or planted with grass sod or turf. Failure to revegetate such areas quickly will lead to erosion of the bed.

Where bare areas occur because of salinity in surface flows or in subsurface seepage, salt tolerant species should be established on them, or corrective measures be applied as discussed in Section 4.6.



Check dams, made from rock or straw bales, afford temporary protection to these grassed urban waterways.

3.3.8 CHECK DAMS

A check dam is a small, temporary dam built across a swale, diversion channel or waterway. Its primary function is to reduce the velocity of flow in the channel and thus reduce erosion of the channel bed. The entrapment of sediment behind these structures is a secondary function.

Check dams can be used:

- To protect a grass lined channel during initial establishment of vegetation.
- As a substitute for channel lining in a temporary channel.

Although the materials and method of construction used are similar for check dams and sediment traps/filters, the primary functions of these structures differ depending on their location in the drainage network. Sediment traps and filters intercept runoff before it enters a channel and serve principally to capture sediment, while check dams slow the velocity of runoff within a channel.

Although no formal design is required for check dams, the following general design guidelines should be observed to ensure their proper function. (Figure 3.7)

The drainage area of the channel or waterway being protected should not exceed 4 ha except for straw bale check dams which should have drainage areas no greater than 0.8 ha. The maximum height of the check dam should be 0.6m and the centre should be at least 0.15m lower than the outer edges. The maximum spacing between the dams should be such that the toe of the upstream dam is at the same elevation as the top of the downstream dam. Rock protection may be required below the centre of the structure to dissipate overflows.

Check dams can be constructed by using any materials on the site which can withstand the flow of water. Rock, log and sandbag check dams can be the sturdiest, if these materials are correctly placed in position. Wire netting, woven brush and straw bales can also be used, but the random placement of trees and logs across a channel does not necessarily constitute an effective check dam.

Although check dams are not intended as sediment trapping devices, larger-sized particles will inevitably accumulate behind them. This sediment should be removed before it accumulates to one-half of the original height of the dam, and placed where it will not be washed back into the drainage system.

Check dams should be removed when they are no longer needed. In permanent waterways, check dams should be removed when a permanent lining is installed. If the permanent lining is grass, the check dam should be left in place until the grass has matured sufficiently to protect the waterway.

3.3.9 BANK AND CHANNEL LININGS

This section deals with measures to stabilise the bed and banks of watercourses, waterways or open earth channels to prevent further erosion, gully head migration, or a lowering of channel grade. Stability may be achieved either by reducing flow velocity or by providing a lining that will withstand high flow velocities.

In some constrained situations, open channel velocities may be well above the non-erosive values considered acceptable for an unlined or vegetated channel. (Sections 3.2.3.5 and 3.2.3.6) Similarly, where persistent seepage or trickle flows occur in a channel or waterway, it may be necessary to construct a low flow invert within the main channel to remove the continual wetness, while still allowing stabilisation by vegetation and the use of mowing equipment within the waterway.

Bank stabilisation and channel protection measures are therefore required where the capacity of a channel is exceeded as a result of changes in flow regime (e.g. resulting from urban development) where steep grades occur in a channel, or where runoff must be lowered directly from one elevation to another. They are used instead of, or in addition to, vegetative cover where such cover is subject to scour by erosive velocities, or where vegetation cannot be maintained because of pollutants in the streamflow.

Apart from stabilising the flow surface against erosive velocities, channel linings have several secondary characteristics which may influence the choice of specific lining material.

- The roughness of the lining material can be selected to control velocities (hence flow peaks) and depths of flow. This is a legitimate stormwater management procedure. At the junction of artificial and natural channels, it may be beneficial to provide roughening of the liner material to reduce velocities (matching boundary layers) to suit the unlined channel.

  By reducing velocity in a channel the resulting discharge peak is reduced by spreading it over a longer period. This is a fundamental characteristic of channel storage in hydrologic terms.

- As indicated above, smoother lining material can also be utilized to convey large flows through restricted or limited areas.

- Permeable lining materials can be used to encourage infiltration into the soil profile where this is desirable (e.g. groundwater recharge and vegetative growth).

- Impermeable lining materials can prevent infiltration where soil stability is a problem (e.g. areas with dispersive subsoils).

- Sheet lining materials can be used to strengthen and separate the surface of the waterway from underlying fine-grained material (e.g. geotextiles).

The choice of specific lining material will be indicated following consideration of the velocity of flow, economics, permanence, aesthetics, maintenance and other factors. Examples of the various lining materials commonly available include:

Permeable

Grass
Gravel and rock (incl. mattresses)
Geotextiles (usually combined with rock)
Jute mesh
Natural/synthetic erosion mattings
Sandbags

Impermeable

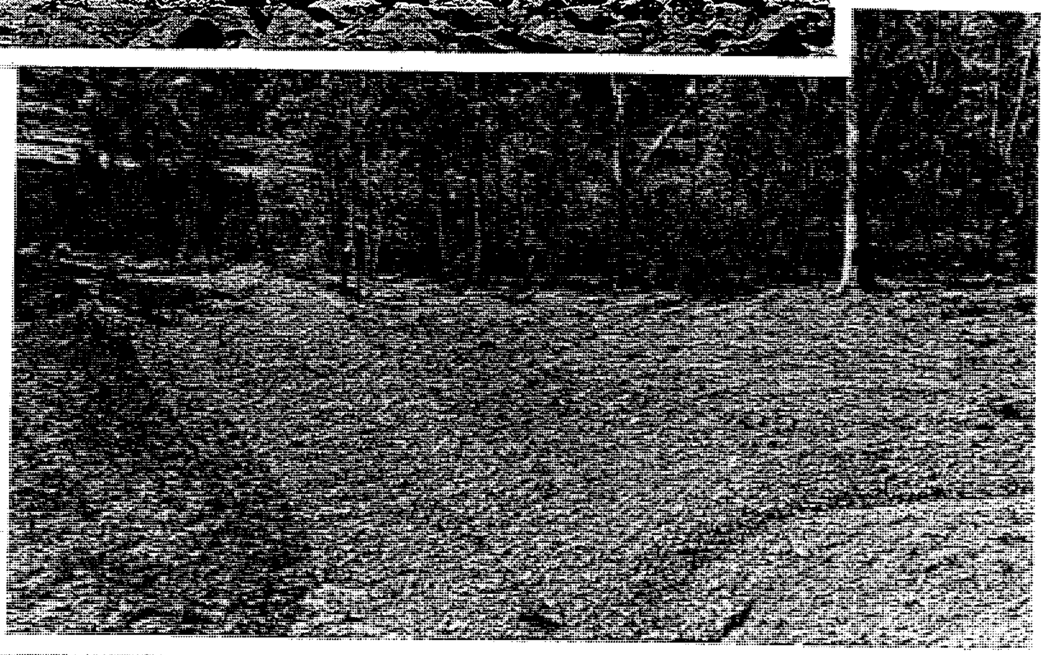
Concrete
Pressure grouted mattress
Asphalt/bitumen
Grouted rip-rap
Plastic sheeting
Half pipes
Concrete filled bags

Care should be taken in the use of impermeable lining material to ensure that water flow is prevented from getting beneath the liner and causing uplift. Suitable cut-off trenches should be adopted at the top and toe of any liner section. Similarly, it may be necessary to place weep holes or drainage blocks within the lining. Top of slope diversions or cutoff drains on filled areas are examples of where an impermeable channel liner should be used.

Geotextiles are commonly used as a filter beneath rip-rap and reno mattress channel linings. They provide the dual functions of reinforcement and separation.

Geotextiles may also be used as temporary channel liners but regular anchor points must be provided along the waterway to prevent significant flow beneath the fabric. Suitable applications are sandy, positive draining situations, but care should be taken in dispersive soil materials, where an impermeable fabric should be used. Alternative lining materials more suitable for temporary use include plastic sheeting, bitumen and sand bags.

All lining materials must be well keyed into the channel bed and/or bank at regular intervals, to resist undermining and displacement. Where bank lining is used, the material should extend up the bank to the level of maximum flow, or to a point where vegetation can be established to provide stability.



Temporary and permanent channel linings stabilise drainage channels against erosion.

3.3.10 GRADE STABILISING STRUCTURES

Grade stabilising structures are employed to provide erosion resistant controls in the bed of bare or vegetated earth channels. Their principal function is to pass the design flow from a higher to a lower elevation and to dissipate the excess energy in a controlled manner. A lower grade and a non-erosive velocity can then be adopted for the channel section below the structure.

These structures have special application on construction sites and urban developments, especially where increases in flow rate would produce active erosion in the bed of a previously stable channel or stream. They are also used to stabilise the bed of actively eroding gullies (usually in conjunction with a flume or gully control structure) to halt the headward erosion of the gully.

Grade stabilising structures usually take either of two forms:

- Flumes. These structures convey flows down an even gradient (usually between 1:20 and 1:5 slope). Flumes can have the same range of surface treatments or lining as waterways. However, special provisions may be necessary in the vicinity of the crest and the stilling basin, particularly in the case of grassed flumes.

- Drop Structures. These comprise a vertical drop constructed of suitable material and a stilling basin for energy dissipation.

In addition to the materials used to line waterways (Section 3.3.7) or banks and channels (Section 3.3.9) the following materials may also be used successfully in the construction of flumes and drop structures:

- Timber (e.g. form ply and railway sleepers).
- Galvanized roofing iron or sheet.
- Gabions.
- Conveyor belting.
- Sandbags (sand/cement mix).
- Half pipe (batter drain) or full pipes (pipe drop).
- Sheet piling.

When building grade stabilising structures, especially the weir apron on the higher level and the cut-off walls, special care should be taken during construction to ensure that undermining or outflanking of the structure does not occur. An energy dissipator may be required at the outlet to prevent erosion of the downstream channel. (Section 3.3.11)

3.3.10.1 Design Considerations

Section 3.2.3 outlines general design criteria applicable to these structures.

The following additional design considerations are of a more specific nature.

(i) Design Average Recurrence Interval

As the purpose of the structure is to provide a stable channel bed with only occasional damage, on which vegetation may establish, the design ARI should be selected with a view to the design life of the structure and the acceptable probability of failure. For structures on trunk systems and streams this should certainly be in the order of 1:100 years (see Section 3.2.3 and Table 3.1).

(ii) Crest Capacity

Crest capacity is given by the weir formula (from King, 1963):

$$Q_{cap} = C_w L H^{3/2} \qquad \textbf{EQUATION 3.7}$$

where:

where:	Q_{cap}	=	crest capacity (design flow) (m^3/s).
	C_w	=	weir coefficient (adopt 1.7 usually).
	L	=	weir length (across stream) (m).
	H	=	depth of flow at inlet (m).

This formula is used to select a weir length if the depth is fixed by some constraint, or conversely to select a depth (and hence an inlet bank height) if the weir length is fixed by site constraints.

(iii) Flume Design Procedure

The Soil Conservation Service Technical Handbook No.5 offers a design sequence based on a proportional adjustment to the calculated catchment discharge to give what is called the design discharge. Thereafter, no allowance is made for freeboard in any of the flume dimensions. In the case of a flume contracting towards the base, the sidewall height should be determined using the smaller dimension (bottom).

(iv) Drop Structure Design Procedure

The Soil Conservation Service Technical Handbook No. 5 offers a design procedure (after Henderson, 1966) which is based on experimental data and the use of a characteristic drop number. Donnelly and Blaisdell (1965) also provide design rules for these structures.

(v) Stability Considerations for Flumes

Depending on the material and the site conditions flumes should, as much as possible, be constructed in undisturbed ground. This may not necessarily apply to galvanized iron and sheet metal flumes. Where there is no option but to construct the flume in fill material, careful control of the compaction to at least 95 per cent maximum dry density (Proctor compaction) should ensure reasonable stability. In these cases, and particularly in the case of dispersive soils, the provision of appropriate cut-off walls and/or filter drains should be considered.

(vi) Stability Considerations for Drop Structures

Drop structures can be considered as gravity or cantilever structures. As such, they should be checked for stability against bearing failure, overturning and sliding, as applicable. In the case of a cantilever (retaining wall) type structure design loadings should take into account loading due to the estimated head of water (at surcharge) and the submerged weight of the sediment retained. Because of the complexity of the design process, structures which are over 1m in height should generally be designed by a practising engineer. The use of cut-off walls to control the seepage path may also be necessary.

3.3.11 OUTLET PROTECTION STRUCTURES

Outlet protection can consist of energy dissipating structures or the provision of erosion resistant channel sections at outfalls from stormwater pipes. Energy dissipating structures generally reduce energy by means of friction, impact and/or turbulence. They reduce the velocity of water leaving the artificial channel to a level consistent with the maintenance of stability in the natural channel.

Stilling basins remove energy by turbulence in the hydraulic jump. An outlet sump functions by a combination of impact and turbulence losses. Outlet aprons (i.e. below stormwater outlets) function primarily by increasing friction and spreading the concentrated flow, thus reducing the velocity. Artificially roughened sections of lined channels can also be used to reduce velocity before merging with an unlined section downstream.

The choice of specific energy dissipation technique will depend on flow conditions, economics and site area constraints.

3.3.11.1 Outfall Aprons and Stabilised Sections

Stabilised channel sections for energy dissipation should generally be designed and installed with the same care as channel linings. (Section 3.3.9) The channel section can take the form of rip-rap or reno mattress protection downstream of a culvert or concrete lined channel. The design of such sections should aim at providing a surface of sufficient roughness so that the normal flow depth developed on the stabilised channel section will be equal to that of the unlined section downstream. This flow should be allowed to develop over a length of lined channel which will ensure that the desired velocities are achieved at the junction with the unlined section.

The design of an outfall apron downstream of a stormwater pipe or other source of concentrated flow will depend to some extent on the estimated tailwater depth. Where this is less than $0.5d_o$ (where d_o = diameter of outlet) a condition of *minimum tailwater* is said to apply. Where the depth is greater than or equal to $0.5d_o$, a *situation of maximum tailwater* exists. Goldman et al. (1986) gives dimensions and rip-rap sizes for outfall aprons under both conditions.

3.3.11.2 Energy Dissipators

These structures come in many forms and the choice will ultimately depend on economics and the desirable permanence of the structure. (Figure 3.7)

As mentioned previously, energy dissipators function largely by impact and/or turbulence. Impact blocks, T junctions and vertical discharge sumps are examples of this type of structure. When these controls are used care should be taken with the downstream disposal of water. They should not discharge onto a steep or erodible slope, as further gullying and headward erosion will result.

Turbulence type dissipators generally require the formation of a hydraulic jump. This can be achieved at a change in grade (from steep to low slope) or by providing a purpose designed stilling basin. Section 3.3.10 gives direction on the sizing of stilling basins for both chutes and vertical drop structures. The aim of the design is to force a hydraulic jump to form within the basin under the design flow conditions. Under a variety of flows the stilling basin will therefore perform quite differently. However, formation of the jump should at least be contained within the area of stabilised channel, as excessive turbulence of the jump may cause severe downstream channel erosion and failure of the structure.

The Roads and Traffic Authority of New South Wales (1989) gives detailed design procedures for energy dissipators, while Henderson (1966) gives direction on the desirable features of stilling basin design. Goldman et al. (1986) also give some useful examples of dissipator types which seem to have direct relevance to construction sites and urban erosion works generally.

3.3.12 STORMWATER DETENTION MEASURES

Stormwater detention (sometimes referred to as stormwater retardation) involves the application of measures to delay the flow, and therefore the concentration, of stormwater runoff. When correctly designed, such measures reduce flooding by increasing the time of concentration of flow, so lowering peak discharges from a catchment. This in turn can reduce the channel capacity required downstream, and may reduce stabilising works required to prevent channel erosion. Stormwater detention measures may also promote settlement or filtration of pollutants, particularly sediment, which are carried in stormwater runoff.

Minor detention measures can be applied on individual residential, commercial or industrial allotments, in streets and parks, and in service areas such as parking bays. Major detention structures can be developed on trunk drainage systems provided the contributing catchment is not so large that the cost of building a safe storage structure outweighs the benefits to protected land downstream.

With larger structures, landform, geology and soils at the proposed construction site must be suitable for the development of an appropriate reservoir. If significant quantities of sediment will be entering the storage area, there must also be scope for periodic desilting.

Stormwater detention measures should be approached with caution on areas where land instability or high water table conditions are a problem. In such circumstances it is often more desirable to move stormwater from the affected area as quickly as possible in pipes or in lined channels, as the infiltration of water into the soil may aggravate the particular problems.

3.3.12.1 Minor Detention Measures

A wide range of measures can be applied at, or very close to, the source of runoff to steady the flow of water into the main stormwater drainage system. Some of the more common measures are itemised below but depending largely on the imagination of the designer, others may be applied.

- The use of waterways was discussed in Section 3.3.7. By accommodating flows at a lower velocity than that of conventional pipes or lined channels, they delay the concentration of stormwater.

- In similar vein, the use of grassed table drains instead of concrete kerbing as a drainage measure along subdivision roads will delay flows. A thick mat of grass will also trap some of the pollutants washed off the road surface that would otherwise reach streams.

- Where possible, minor channels and drainage depressions present in the natural landscape should be preserved to convey runoff to trunk stormwater systems, instead of using stormwater pipes. For example, where a minor depression passes through house yards, roof water may be directed into it from downpipes, rather than being discharged into street kerbs or diverted into stormwater pipes.

- Where natural depressions are not available, the discharge of roof drainage onto lawn areas rather than channelling it into an installed drainage system will delay its passage, and may also reduce its volume by allowing some to infiltrate the soil.

- On a flat roof, storage may be provided with a raised perimeter by constricting the downpipes. The roof must be engineered to a sufficient strength to support the maximum volume of water that can be stored, and an emergency outlet is essential to prevent overload. The development of rooftop gardens is another approach to this concept.

- Storage can be provided in parking lots by surface ponding. A low perimeter wall around a level lot or on the lower side of a sloping lot, with a constricted outlet at its lowest point, will delay the escape of runoff. This should provide only shallow ponding, so that parked cars do not suffer mechanical damage.

  Grass strips across the fall line of a parking lot, or grass collector channels for runoff, will also delay surface flows.

  On soils of high permeability, a certain amount of infiltration can be achieved by surfacing parking lots with porous asphalt or by impervious pavement punctured with regularly spaced holes. Alternatively, a gravel-filled trench may be installed along the lower side of a sloping allotment to collect surface water. A subsoil drain in its base receives surplus water that does not infiltrate the soil, and conducts this to the stormwater drainage system.

- Gravel or rubble seepage trenches, and oversized pipes with constricted outlets, can be used to provide underground storage of runoff, depending on local regulations.

- On highly permeable soils, infiltration basins may be excavated to store runoff from building sites, allowing it time to infiltrate the soil. This practice should only be undertaken where local groundwater or salinity problems will not be aggravate.

3.3.12.2 Major Retarding Measures

Large detention basins may be used for controlling the runoff from major catchments, while smaller basins may be used to control the runoff from individual development sites. These structures may be on-stream or off-stream and may be dry-bottomed or have an element of permanent pondage.

Stormwater detention basins can act as valid erosion control measures through their ability to modify downstream flow rates and velocities, limiting them to manageable values. By reducing peak flows, downstream structures (e.g. waterways and diversion channels) can be protected against scour.

All detention basins have the basic feature of storage volume obtained by the excavation of a basin, construction of a containing wall, or a combination of both. All have a primary outlet to discharge runoff at a controlled rate, and many have an emergency spillway to discharge overflows safely when their storage capacity has been exceeded.

They may also be designed as multiple use structures. In this context, wet ponds provide permanent storage to a certain level, and this water may serve for recreational use, as a fish and wildlife habitat, or as a water source for home garden use or for park and garden irrigation. Additional capacity is provided for flood storage and, where necessary, for sediment storage. If the catchment is yielding sediment to the pond, periodic desilting will be necessary. Dry storages may be used as lawns, playing fields, or landscaped gardens. Their beds may be used for agricultural purposes such as market gardening, if the user is prepared to carry the risk of a crop loss as a result of flooding.

Where a facility such as a sports oval, which is subject to intensive public use, performs as a dry storage, it may be desirable to install subsoil drainage beneath the storage area. This will hasten drying of the turf after stored runoff drains from the area.

It will not always be necessary to construct a wall for the specific purpose of impounding water. A constricted culvert through a road embankment will lead to pondage behind the embankment. A raised spectator area around a playing field may provide the earth embankment needed to temporarily contain runoff water on the field, should the outlet culvert be constricted.

Where the topography does not permit the construction of a containing embankment across a drainage line, storage may be provided by excavating a basin, diverting runoff or streamflow into it during a storm, and pumping out this water after the storm has passed. A stabilised entry chute should be provided to prevent scouring of the banks of the excavation. In some areas disused quarries may be suitable for this purpose.

To avoid regular desilting of a major basin, a smaller primary basin can be installed upstream from it to trap the bulk of coarse sediment before water flows into the main storage.

Although a detention basin can be employed either as a temporary or permanent structure, the design criteria are somewhat different from those used for sediment basins. There can also be opportunities for the creative design of multiple use structures such as sediment/detention basins or detention basins/artificial wetlands. Much will depend on the nature of the site and the specific erosion or pollution problem. However, it is essential that detention basins be designed and constructed in accordance with sound engineering practice.

3.3.12.3 Design Considerations for Detention Basins

Section 3.2.3 outlines general design criteria applicable to these structures. The following additional design considerations are of a more specific nature.

(i) Design Average Recurrence Interval

A minimum of a 10 year design ARI storm of critical duration (see Section 3.2.3 and Table 3.1) should be applied for the design of all parts of the structure subject to major storm through-flow (e.g. the emergency outlet plus primary outlet capacity). The design capacity of the primary outlet is usually selected to satisfy either flow capacity or velocity requirements for the downstream channel or pipe system.

For more permanent structures (design life > 4 years) and certainly for those subject to major floods, it is advised that at least the 1:100 year ARI standard be adopted. For those structures where the costs of failure are severe, performance under PMP (probable maximum precipitation) events should be checked for at least "fail-safe" operation.

(ii) Basin Size

Of the two approaches used for estimating detention basin volumes, the choice depends on the importance of the structure.

(a) Preliminary estimates and small sites.

The method advanced in ARR, 1987, Section 7.5.6 (Approximate Method) is recommended. This approach is based on a triangular approximation for the inflow and outflow hydrographs; the difference representing the amount of storage required.

(b) Large sites.

The estimation of storage volumes is best approached by using a runoff routing model (i.e. synthetic hydrograph approach) using a derived inflow hydrograph and an estimated height/storage/discharge relationship.

(iii) Outlet Design

(a) Primary outlet

This may consist of a drop inlet and pipe through the embankment, or an overflow spillway constructed as a chute. Both options should take account of scour at the outlet through the incorporation of energy dissipation structures. (Section 3.3.11) A minimum barrel size of 0.375m should be applied in most cases to cover the possibility of debris blockage and lack of maintenance. In the case of urban redevelopment basins (e.g. rooftops, parking areas) the use of smaller pipe sizes is feasible provide they include appropriate grates, to alleviate the debris problem.

Dual outletted "glory holes" can be designed to incorporate both the primary discharge as well as emergency aspects. This option might be explored in cases where site constraints may not allow the construction of an open channel type bywash.

(b) Emergency outlet

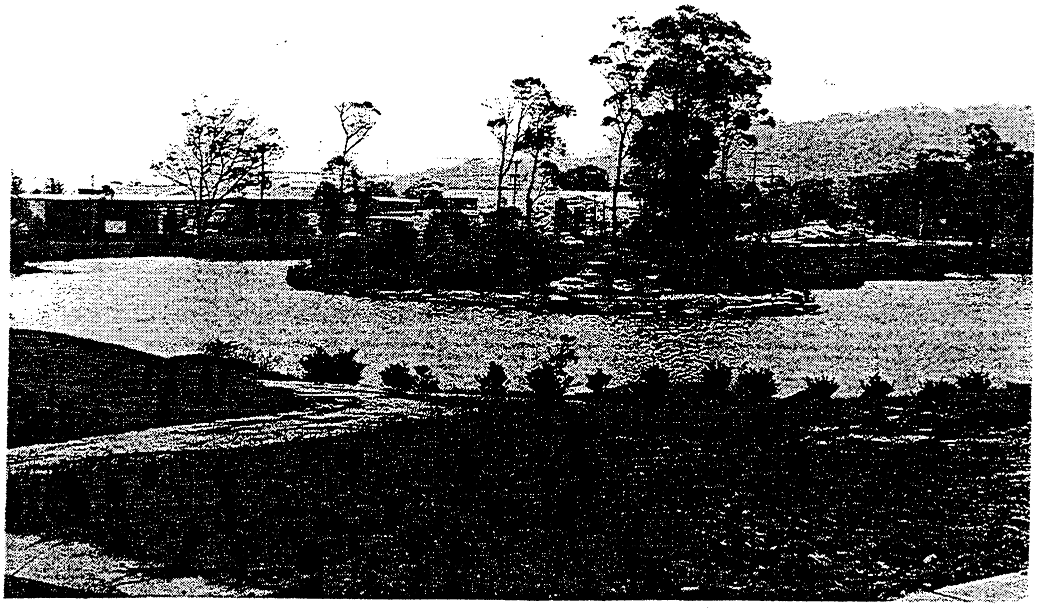
The width and gradient of the emergency outlet or spillway should be selected to limit velocities to those outlined in Table 3.4 (Section 3.2.3.6). The Soil Conservation Service of N.S.W. Technical Handbook No. 5 gives guidance in both erosion protection and energy dissipation structures. (see also Section 3.3.11)

(iv) Inlet Design

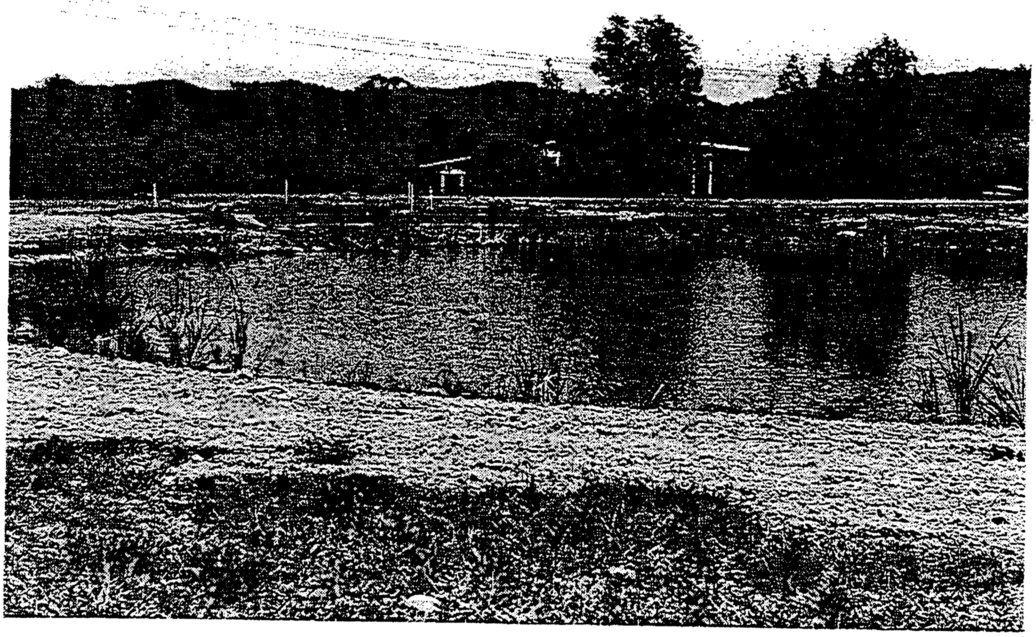
Entry to basins should be protected against erosion and scour as for sediment basins. This may be achieved by a level spreader sill - where the drop is not great - or by rip-rap protection on steeper inlets. In the case of stormwater pipes, discharge points can be protected by the use of rip-rap or suitably grated "bubble-up" pits. In many cases, a low flow pipe may be located beneath the basin and "bubble-up" pits can function as discharge points once the pipe pressurizes.

(v) Freeboard

For catchments of less than fifteen (15) ha a freeboard allowance of at least 0.75m should be provided between the design surcharge level in the basin and the top of the compacted embankment (allowing for settlement).



Detention basins can be designed with multi-purpose roles including trapping sediment, detaining stormwater and providing recreational and aesthetic amenity.



Calculation of freeboard for larger catchments, on the other hand, should include additional allowance for the following:

- *Surcharge*
- *Wave action*
- *Clearance*
- *Settlement*

For guidance in the calculation of these factors, see the Soil Conservation Service of N.S.W. Technical Handbook No. 5.

(vi) Multiple Use Basins

Basins may be designed with the dual purpose of trapping sediment and detaining stormwater. In this case, it may be possible to provide wet storage as a sedimentation volume within the main detention volume such that the settlement requirements for the selected particle size are achieved.

Where the surface area or volume requirement for effective sediment retention during construction exceeds that required for long term runoff detention, an artificial riser (e.g. gabion) incorporated with the primary outlet can provide temporary additional storage until the contributing catchment is stabilised.

Alternatively where a low flow pipe is incorporated into the basin design, temporary sediment detention capacity can be provided by blocking this low flow pipe at the basin's inlet surcharge pit. This will force all inflows to surcharge onto the basin floor facilitating the settlement of course and suspended sediments. Provided all outlets from the basin are afforded temporary sediment filtering capability, minor to moderate flows will be catered for. Regular desilting of the inlet surcharge pit will be critical after each storm event.

Basin design can be further extended to the incorporation of artificial wetlands within large detention storages, which can be designed to accept the first flush of stormwater (say, the first 5 mm) which generally carries the highest sediment/pollution load. This wetland storage should be sited in a quiescent part of the main storage, to prevent a significant disturbance of sediment and pollutants.

Guidelines on the design of wetlands are given in Part 8 of the State Pollution Control Commission publication "Pollution Control Manual for Urban Stormwater".

3.3.12.4 Maintenance of Detention Basins

Periodic desilting may be necessary, either of the main storage, or of a primary pond if one is provided upstream.

Any scouring within the earth embankment, grassed emergency spillway, or at the discharge point of the primary outlet, should be treated at an early stage. In the first two cases this should include sowing and mulching or by laying turf or grass sod, and in the latter case by installing rip-rap or by other appropriate energy dissipation measures. (Section 3.3.11)

3.4 SEDIMENT CONTROL MEASURES

The installation of appropriate erosion control measures will greatly reduce the quantity of soil eroded from a construction site. However, some erosion will be inevitable. Sediment control measures are therefore required to trap and retain sediment once soil has been eroded.

It is environmentally sound, easier and more cost-effective to prevent erosion than it is to concentrate on trapping sediment from eroding areas. This applies particularly to areas where the soils have a high proportion of fine silts and clays, or are dispersible.

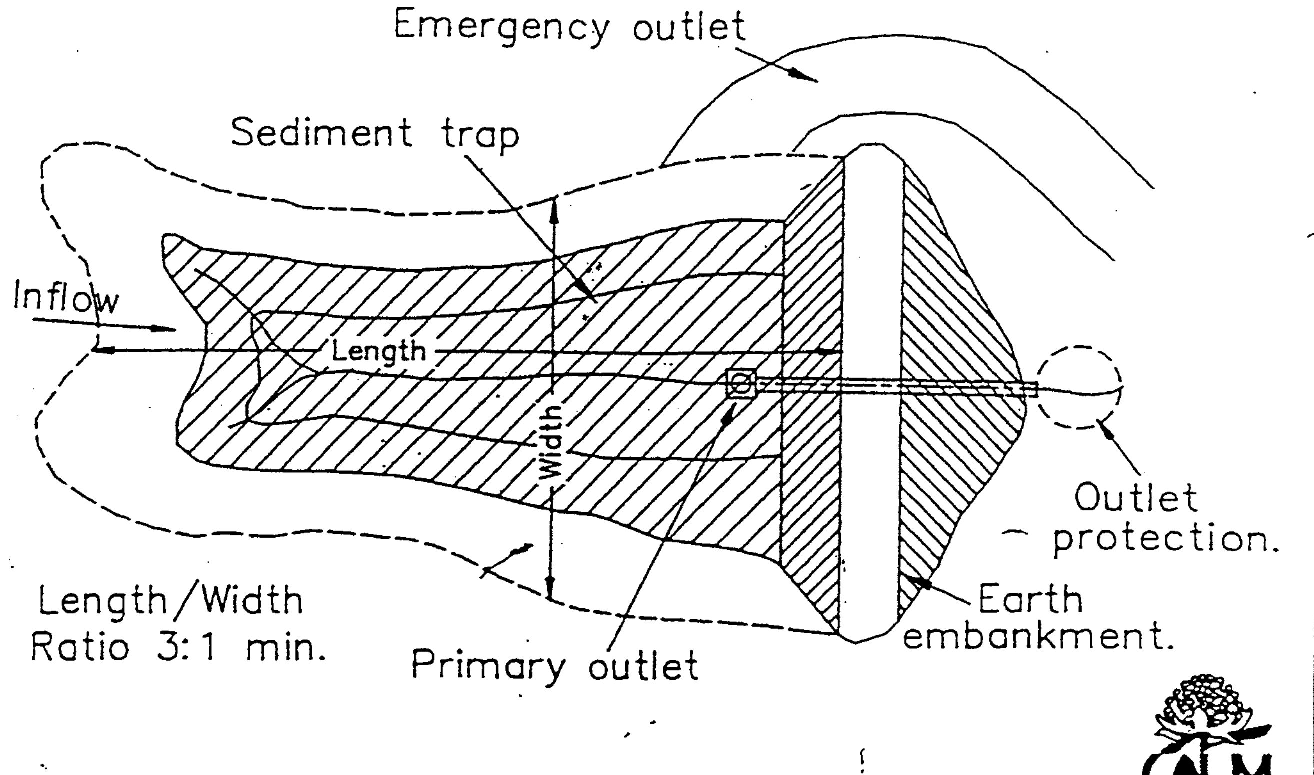
Sediment control measures, *therefore, only control sediment, they do not control* erosion. Their function is to trap eroded soil before it leaves the site and pollutes adjacent property or water bodies.

Sediment control measures are usually only temporary solutions until permanent measures, such as revegetation and paving, are in place. Ideally, they should be used as techniques to support or back-up a range of erosion control measures, not as the primary strategy to control sedimentation.

These measures usually function by slowing the velocity of runoff and allowing suspended soil particles to settle by gravity or filtration. They include such structures as sediment basins, sediment traps and sediment filters.

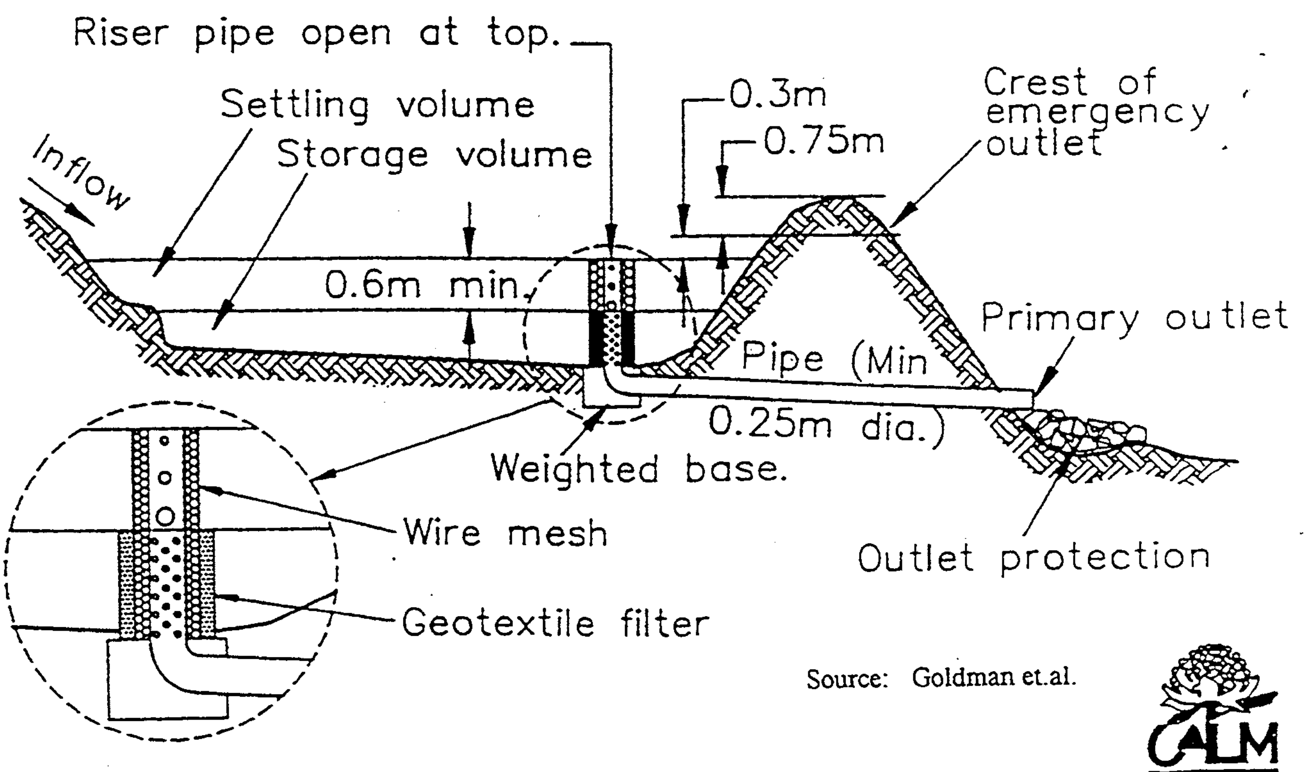
Sediment control measures require regular maintenance and cleaning. If too much sediment is allowed to accumulate in them, they will cease to function. Little or no settling will occur, and trapped sediment will be re-suspended and washed away.

The sediment control structures described in this section are intended as temporary measures for use during the construction stage of a project. Sediment basins, however, can also be designed as permanent structures. (Sections 3.2.3 and 3.3.12) Other permanent sediment control structures such as gross pollutant traps and wetlands, are discussed more fully in the State Pollution Control Commission publication "Pollution Control Manual for Urban Stormwater".



Plan View of Typical Sediment Basin

Fig. 3.8(a)



Cross Section of Typical Sediment Basin.

Fig. 3.8(b)

3.4.1 SEDIMENT BASINS

A sediment basin is a barrier or dam designed to intercept sediment laden runoff and retain the sediment. It is usually located on a drainage line below a construction site, or at some other stormwater collection point. It is fitted with a dewatering system and usually remains empty between rainfall events.

These basins trap sediment from developing areas where the drainage area is usually larger than 1 ha and concentrated flows exist. In so doing, they prevent sediment from clogging stormwater pipes and floodways, and reduce the environmental damage caused by sediment to vegetation and wildlife. They can also reduce the flooding which is associated with a reduction in a stream's capacity caused by deposited sediment.

Sediment basins are installed prior to development or construction activity on a site, and should remain in place until such activity has been completed and the land stabilised. They should be located away from busy construction areas at a point where they can trap a high proportion of polluted runoff which is not diluted by clean runoff from undisturbed watersheds. They should be sited where the terrain provides maximum storage benefits and where desilting is feasible.

A sediment basin does not replace on-site control measures such as perimeter banks, temporary revegetation or sediment traps at stormwater inlets. These measures retain a portion of the sediment carried in runoff, and the sediment basin is a final check to trap a significant part of the remaining sediment, before runoff discharges into stormwater mains or enters streams.

A typical sediment basin (Figure 3.8) will have:-

- Compacted earth, rock, or gabion embankments.
- Upstream storage provided by excavation.
- One or more inflow points carrying sediment laden runoff.
- A primary outlet.
- An emergency outlet or spillway.
- A basin dewatering device.
- Outlet protection to reduce erosion downstream.
- All-weather access for sediment removal.

Unlike the sediment trap, which is generally a minor facility achieved by modifying roadworks or drainage facilities during the construction phase, the sediment basin is a specifically sited and purpose designed structure.

Formal hydrologic and hydraulic design procedures must be applied.

A sediment basin may be either temporary or permanent in terms of design life. However, there is an over-riding need in both cases for adequate design procedures which should take account of:

- Peak flows (rates and volumes)
- A reasonable estimate of sediment yield from the contributing catchment.

Storage surface area is a critical design feature, and should be maximised within the site constraints.

The distance between the basin inlet and outlet should also be the maximum practical, to ensure optimum sediment trapping efficiency.

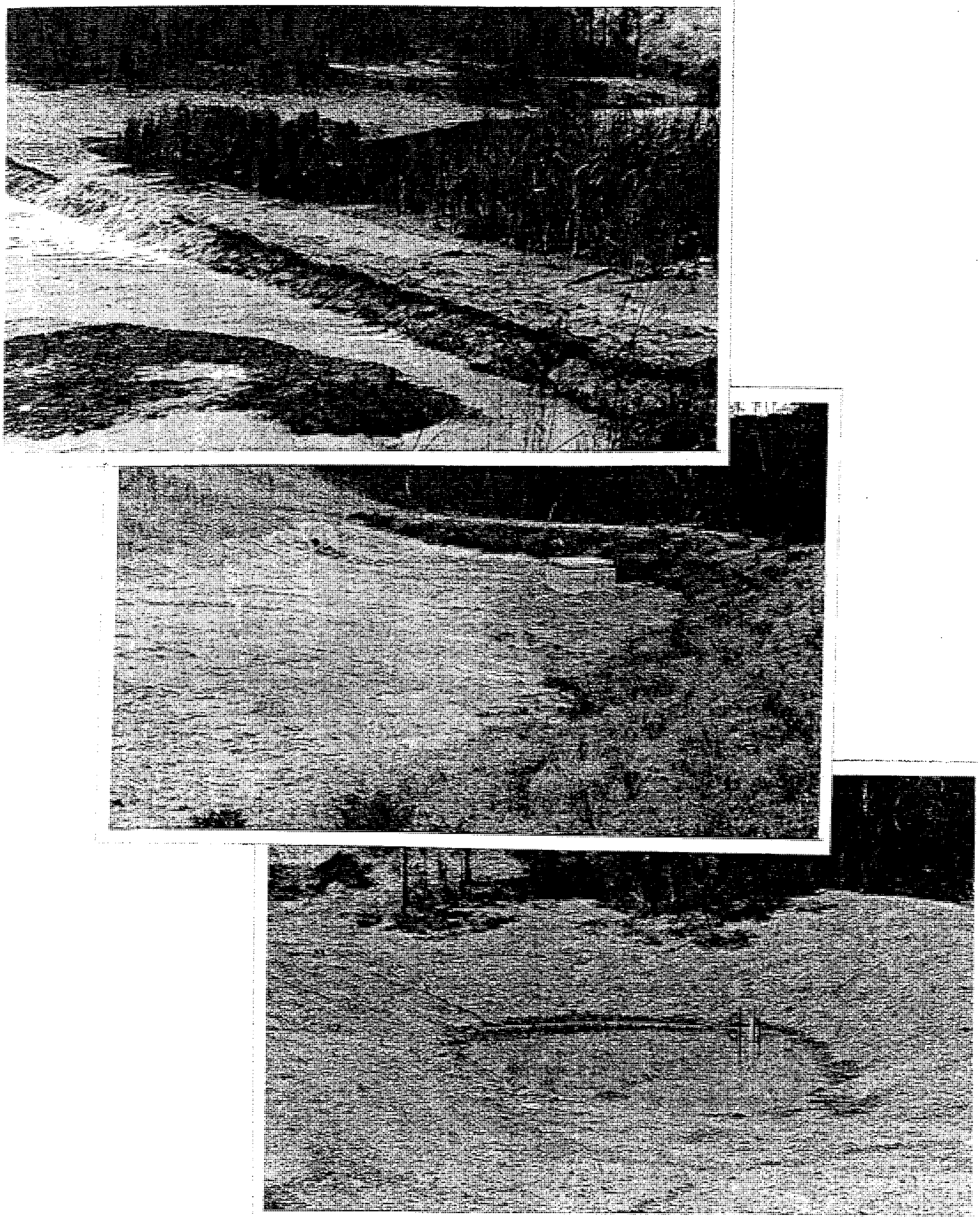
3.4.1.1. Design Considerations

Section 3.2.3 outlines general design criteria applicable to these structures. The following additional design considerations are of a more specific nature. Use of alternative design procedures to those recommended here should only be considered by relevant consent authorities if accompanied by full design details.

(i) Design Average Recurrence Interval

A minimum of a 10 year design ARI storm of appropriate time duration shall be selected for the design of all parts of the structure subject to major storm through-flow. (See Section 3.2.3 and Table 3.1 for the range of design ARI.) Notwithstanding this provision however, in cases where costs of exceedence may warrant (e.g. threat to property or human life) it is the responsibility of the designer to apply a higher design standard. Guidelines are available from the Dam Safety Committee (Department of Water Resources).

For the purposes of basin settling volume design, the 6 hour duration 10 year standard is recommended (after Goldman et al. 1986). Goldman supports the conclusion that it is generally more cost-effective to size basins using average runoff rates rather than peak flows, without significantly lowering basin efficiency.



Sediment basins can be temporary or permanent, constructed from compacted earth, rock or gabions, and should be installed prior to construction activity.

(ii) Basin Size

The size of any sediment basin depends on the erodibility of the soil within the catchment and the volume of water and sediment likely to enter the structure in a specific storm event.

The assessment of soil erodibility and particle size are critical considerations in the determination of basin size. Particle size analysis and soil erodibility data should be required for all developments to provide a sound basis for basin size evaluation.

The following design procedure based on the surface area formula, after Goldman et al. (1986) is recommended for basin sizing.

SPECIAL NOTE

This sediment basin design procedure assumes no other erosion and sediment control measures within the catchment.

The basin size criteria determined by this procedure therefore represent the maximum, desirable design parameters. where a range of alternative or supplementary control measures are to be incorporated within the catchment, these parameters may be reduced proportional to the cumulative sediment trapping effect of these alternative measures.

An erosion and sediment control plan is the only effective method of assessing the full impact of such alternative control measures on sediment basin size.

The surface area approach assumes free settling of rounded soil particles and the application of Stokes' Law. (Section 3.2.3.8) The basin surface area is determined as a function of the inflow rate and the target particle settling velocity. Basin length and width, settling depth and sediment storage volume can then be calculated.

The basin size design procedure is as follows:

Step 1: Calculate runoff rate using the 6 hour duration, 10 year ARI rainfall and the Rational Method. Allow for the anticipated types and conditions of different surfaces. (see Section 3.2)

Step 2: From wet sieve analysis or particle size analysis of representative soil samples, determine the design particle size, as per Section 3.2.3.8, and the percentage of particle sizes equal to or larger than this design particle size.

Step 3: Calculate the minimum basin surface area required, using the settling velocity of the design particle size, as per Table 3.5. (Section 3.2.3.8)

Step 4: Length of the basin is determined using the basin surface area and applying the criteria that the L/W ratio should not be less than 3:1. The settling depth should then be calculated as L/200, where L is the length of the basin, with a minimum depth of 0.6m being applied.

Step 5: Estimate the annual sediment yield. (In the absence of physical data, the USLE can be used for each length/slope/soil/cover category on the site (Section 3.2.3.7.). Convert the mass yield (t/ha) to volume yield (m^3) by dividing by the measured bulk density of the sediment (t/m^3); and then multiply volume yield by the area (ha) to get total sediment yield).

Step 6: Multiply the total sediment yield by the percentage of soil particles larger than the design particle size (from Step 2) to obtain effective sediment yield.

Step 7: Divide effective sediment yield by the minimum basin surface area (from Step 3) to get the storage depth.

Step 8: Total storage volume is obtained by multiplying total depth (settling depth plus storage depth) by minimum basin surface area.

Example

Data: An urban residential site of 4ha is being developed for medium density housing. All external runoff can be diverted, and all runoff within the site can be channelled to a single sediment basin. Site characteristics are:

- slope gradient (S) - 9%
- slope length (L) - 50m
- soil particles > 0.02mm - 70%
- 10yr 6hr rainfall intensity (I) - 23mm/hr
- rainfall erosivity factor (R) - 4600
- soil factor (K) - 0.03
- soil cover factor (C) - 1.0
- soil conservation factor (P) - 1.0
- runoff coefficient (C) - 0.7
- sediment bulk density (b.d.) - 1.2t/m^3.

Question: Calculate the required surface area, volume, shape and depth of the proposed sediment basin.

Solution:

Step 1: Average runoff (Q) $= \dfrac{C\,i\,A}{360} = \dfrac{0.7 \times 23 \times 4}{360} = 0.179 m^3/sec$

Step 2: Sieve analysis indicates 70% by weight of the soil is equal to or larger than 0.02mm (design particle size).

Step 3: Settling velocity of the 0.02mm particle is 0.00029m/sec. (Table 3.5)
The basin surface area required per unit discharge. (Table 3.5)
$= 4101 m^2/m^3/sec$
Actual basin surface area $= 4101 \times Q = 4101 \times 0.179 = 734 m^2$

Step 4: Minimum basin length:width = 3:1
For surface area of $734m^2$; L:W (3:1) = 48m:16m (approx.)
Settling depth (calculated) $= \dfrac{L}{200} = \dfrac{48}{200} = 0.24m$
(Minimum settling depth, 0.6m so adopt settling depth 0.6m)

Step 5: From the USLE;
Annual soil loss (A) = R x K x LS x C x P
Using SOILOSS (Soil Conservation Service Technical Handbook No.11) : A = 207t/ha/yr

Sediment volume/ha/yr $= \dfrac{A}{b.d.} = \dfrac{207}{1.2} = 173 m^3$
Total annual sediment yield $= 173 \times A = 173 \times 4 = 692 m^3$

Step 6: Effective annual sediment yield = 692 x % particles > 0.02mm
$= 692 \times 0.7 = 484 m^3$

Step 7: Storage depth = effective sediment yield (m^3) basin surface area (m^2)
$= \dfrac{484}{734} = 0.66m$

Step 8: Basin volume = total depth (m) x basin surface area (m^2)
$= (0.6 + 0.66) \times 734 = 925 m^3$

Summary: The proposed sediment basin should therefore have the following minimum size requirements:

- volume - $925m^3$
- surface area - $734m^2$
- total depth - 1.26m
  (storage depth - 0.66m)(settling depth - 0.60m)
- surface dimensions - 48m long x 16m wide (approx.)

To facilitate the process of designing and sizing sediment basins, while recognising the extreme variability of soil loss from construction sites and developing urban areas, a table has been prepared which recommends basin sizes for a range of construction conditions.

Table 3.6 outlines recommended basin volumes and minimum basin surface areas for the two geographic areas indicated, based on the above design procedure. This table *Assumes No Other Control Measures Within The Catchment*, and be used for all development sites up to fifteen (15) per cent average slope gradient.

SPECIAL NOTE

Where average slope gradients exceed fifteen (15) per cent in gradient or where more than ten (10) per cent of the site area exceeds fifteen (15) per cent in gradient, use of Table 3.6 is not recommended and a full basin design will be required.

The total volume of the sediment basin from the bottom of the sediment storage zone to the crest elevation of the primary outlet can be determined in accordance with Table 3.6. The total storage shall be made up of two thirds (2/3) settling volume at a minimum depth of 0.6 m and one third (1/3) sediment storage volume (i.e. storage below the settling or active volume). The minimum basin surface area at the crest elevation of the primary outlet can also be determined in accordance with Table 3.6.

To use this table, first determine the soil erodibility rating, or K factor, for the soil or soils within the site as outlined in Section 3.2.3.7. Table 3.6 is then used to allocate a basin volume and surface area based on the range of average slope gradients occurring within the site for the relevant geographic area.

The soil referred to in the soil erodibility rating used in Table 3.6 is the relevant soil surface that is, or will be, exposed to erosive forces. For most disturbed sites this will be the subsoil horizon, whereas within undisturbed contributing catchments the topsoil horizon will be most relevant.

The slope gradient referred to in Table 3.6 is the average slope gradient of the disturbed development site.

TABLE 3.6 (a)

SEDIMENT BASIN SIZES - NSW COASTAL ZONE

		Slope Gradient (%)				
Soil Erodibility (K factor)		0-5	5-7.5	7.5-10	10-15	>15
		Basin Volume (m^3/ha of disturbed catchment)				
Low	(0.02)	115	150	190	240	Detailed
Moderate	(0.03)	125	170	230	300	
High	(0.04)	135	195	270	365	design
Very High	(0.05)	145	220	310	430	
Min.Basin Surface Area (m^2/ha disturbed catchment)		155	170	185	200	required

SPECIAL NOTES

1. *Assuming no other control measures within the basin catchment)*

2. *USLE Approximation:*

R = 4 600, Slope length = 50 m, K as above C = 1.0,
P = 1.0, Wet bulk density of sediment = 1.2 t/m^3

3. *Basin Surface Area Formula Approximation:*

Runoff Coefficient (c) = 0.60 (0 - 5% gradient)
= 0.65 (5 - 7.5% gradient)
= 0.70 (7.5 - 10% gradient)
= 0.75 (10 - 15% gradient)
Runoff Intensity (I) = 23mm/hr (weighted average)
Proportion of sediment particles >0.02mm = 70%

TABLE 3.6 (b)

SEDIMENT BASIN SIZES - NSW TABLELANDS, SLOPES AND PLAINS

		Slope Gradient (%)				
Soil Erodibility (K factor)		0-5	5-7.5	7.5-10	10-15	>15
		Basin Volume (m^3/ha of disturbed catchment)				
Low	(0.02)	65	80	95	110	Detailed
Moderate	(0.03)	70	85	105	130	
High	(0.04)	75	90	115	145	design
Very High	(0.05)	100	110	120	130	
Min. Basin Surface Area (m^2/ha of disturbed catchment)		80	100	125	160	required

SPECIAL NOTES

1. Assuming no other control measures within the basin catchment)

2. USLE Approximation:

R = 1 250, Slope length = 50 m, K as above C = 1.0,
P = 1.0, Wet bulk density of sediment = 1.2 t/m^3

3. Basin Surface Area Formula Approximation:

Runoff Coefficient (c) = 0.60 (0 - 5% gradient)
= 0.65 (5 - 7.5% gradient)
= 0.70 (7.5 - 10% gradient)
= 0.75 (10 - 15% gradient)
Runoff Intensity (I) = 15mm/hr (weighted average)
Proportion of sediment particles >0.02mm = 70%

The basin volumes and surface areas suggested in Table 3.6 relate solely to the disturbed area of the contributing catchment, based on the assumption that all runoff from undisturbed areas within the catchment of the basin can be diverted to an approved stable outlet. **Where runoff from undisturbed areas cannot be diverted, additional capacity and surface area must be provided**. This should be equivalent to 50 per cent of the corresponding tabulated values, added to the total basin volume (settling volume + sediment volume) and minimum basin surface area for each additional hectare of contributing undisturbed catchment.

This is based on the runoff generating capacity of undisturbed versus disturbed catchments. Goldman et al (1986) show that for disturbed catchments the runoff co-efficient can generally range from 0.6 to 0.8, while for undisturbed grassland or bushland areas, it can generally range from 0.3 to 0.4 (a reduction of approximately 50 per cent). As both runoff rate and minimum basin surface area are directly proportional to the runoff co-efficient, for a storm of the same intensity, it is appropriate to use 50 per cent of the corresponding tabulated figures when determining the above basin criteria for runoff from undisturbed areas.

Factors influencing the estimation of soil loss in the USLE, and consequently the basin sizes and basin surface areas, can in fact be modified by on-site management. Such works can reduce considerably, and in some cases remove completely, the need for sediment retaining structures. For example, the alteration of exposed slope length can reduce the LS factor dramatically and so reduce the sediment storage requirement. Incorporation of such practices is best approached by the preparation of a formal Erosion and Sediment Control Plan.

Bearing in mind the coarse application of the basin surface area formula and the USLE as represented in the above table, there will be a great variation in the way the suggested basins perform. Under a range of storm and draw down scenarios, the basin will act to remove the various sizes of sediment with different efficiencies. The suggested basin sizes will, however, provide reasonable performance in most cases. Because of the range of possible operational situations it is necessary to specify a regular inspection/cleanout programme in the Erosion and Sediment Control Plan which will cover any contingencies that may arise.

In the case of a sensitive discharge area (e.g. wetlands) a full investigation including soil sampling and classification and design based on the acceptable sediment size and/or turbidity is recommended. In addition, sediment yield should be estimated by a specific application of the USLE using site-obtained soil data. The use of the USLE computer program SOILOSS is recommended to expedite this process and also help managers to evaluate site management strategies.

(iii) Basin Shape

Sediment basin geometry should incorporate the following features:-

(a) Length:width. The L/W ratio of the basin surface area at the crest elevation of the primary outlet should not be less than 3:1, where length is the distance from inlet to outlet. A baffle located near the inlet is recommended for all basins. Goldman et al. (1986) give examples of baffle dimensions.
Where restrictive site constraints prevent a L/W ratio of 3:1, a lesser ratio of 2:1 (minimum) may be permitted, provided design details for appropriate baffles which result in an **effective L/W ratio of 3:1** are approved for the site.

> ***The distance between the basin inlet and basin outlet should be the maximum practical, to ensure optimum sediment trapping efficiency.***

(b) Depth. The settling volume for basins should be two-thirds (2/3) of the volumes quoted in Table 3.6 or as determined in step 4 of the above design procedures. As a rule, the settling depth should be sized so that the basin length:settling depth ratio (L/D) is less than 200 with a minimum depth of 0.6m. The depth of the sediment storage volume is not critical, but will generally be determined by the dimensions of the settling volume above it.

(c) Surface Area. The minimum basin surface area should be as stated in Table 3.6 or as determined in Step 3 of the above design procedure.

(iv) Basin Outlet

(a) Primary outlet. The primary outlet may consist of a vertical riser pipe or drop inlet joined to a pipe through the embankment, an overfall spillway, or bywash constructed in one of the abutments. All options must be fully protected against erosion and scour.

It is recommended that, with water at the crest elevation of the emergency outlet, the primary outlet have the capacity to discharge the peak flow from the relevant design storm (see Table 3.1). If no emergency outlet is provided, it is recommended that the primary outlet have the capacity to discharge the peak flow from at least the 10-year frequency storm (See Table 3.1). The minimum size of the barrel for a pipe outlet should be 0.25m diameter.

The design of a drop inlet/riser should include allowance for uplift (buoyancy) forces on the structure in the form of a weighted concrete base.

The crest elevation of the primary outlet should be a minimum of 0.3m below the elevation of the control section of the emergency outlet. In the case of risers under 1.5m high, it is advisable to fit an anti-vortex type trash rack.

Outlet protection will be required to ensure that erosion of the embankment or downstream channel does not occur from outflows from the primary and/or emergency outlets.

(b) Dewatering. Provision for dewatering of the settling zone should be included in the design of the primary outlet. The dewatering facility should provide for the settling volume to be removed over an extended period (minimum 24 hours). This will ensure that basin efficiency is not adversely affected during smaller inflows, when less settling depth is available. If the primary outlet is a concrete chute, a separate dewatering structure will have to be provided.

The most effective method of dewatering is to use single or multiple holes in the riser within the settling zone. (Figure 3.9) Open, unfiltered holes within the sediment storage zone of the riser will only tend to re-suspend and scour trapped sediment and should be avoided. The settling zone holes can be easily sized by use of the orifice discharge formula, integrated with respect to the depth or head of water above the hole. The resulting expression for the area of the orifice is as follows (from Goldman et al,1986):

$$A_o = \frac{A_s\sqrt{2h}}{3600\,TC_d\sqrt{g}}$$

EQUATION 3.8

where:	A_o	=	surface area of orifice	(m^2)
	A_s	=	surface area of basin	(m^2)
	h	=	head of water above orifice	(m)
	T	=	dewatering time	(hrs)
	C_d	=	discharge coefficient	(adopt C_d = 0.60)
	g	=	gravitational constant	(9.806 m/s^2)

Where a single large hole is to be used for dewatering, it should be located at the base level of the settling zone.

To maximise sediment trapping efficiency, several holes of different sizes could be used within the settling zone, with the size of the holes graded vertically. That is, one hole is provided at the base level of the settling zone, with progressively larger holes provided higher up the riser.

When multiple holes are used throughout the settling zone, it becomes necessary to adjust the relative areas of each hole to provide a suitable overall dewatering period. This must take into account the variation in head over the dewatering time.

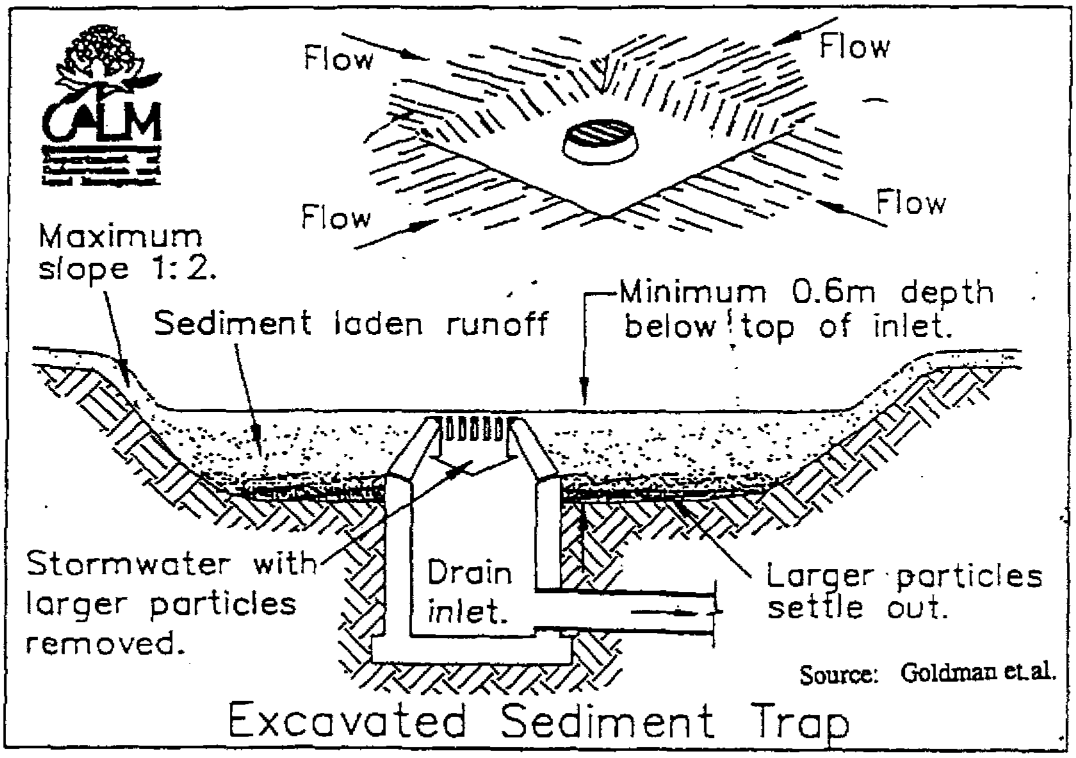

Flow
Flow
Flow
Flow
Maximum slope 1:2.
Sediment laden runoff
Minimum 0.6m depth below top of inlet.
Stormwater with larger particles removed.
Drain inlet.
Larger particles settle out.
Source: Goldman et.al.
Excavated Sediment Trap

Fig. 3.9

Dewatering holes in the settling zone should be covered with wire mesh (25 to 50 mm opening) or coarse gravel to prevent blocking by debris, but should not generally be covered by geotextiles or filter cloths. If such fabrics are to be used, they must be replaced after each storm event.

Dewatering of the sediment storage zone should be considered to facilitate basin cleanout. Dewatering can be achieved by using **filtered holes in the riser**, ideally at the base of the sediment storage zone. Wire mesh should be wrapped around and secured to the riser prior to attaching geotextile filter cloth, to increase the rate of water seepage into the riser.

Goldman et al. (1986) give details of suitable dewatering arrangements for sediment basins.

(c) **Emergency outlet.** The emergency outlet or spillway should have a minimum capacity equal to the peak flow from the relevant design storm. Section 2.3.2 and Table 3.1 outline design standards in this regard. Flow velocities in any outlet channel should generally be limited to those outlined in Table 3.4 (Section 3.2.3.6). The Soil Conservation Service Technical Handbook No. 5 gives guidance in both erosion protection and energy dissipation structures for such outlet works. (see also Section 3.3.11)

(v) Chemical Turbidity Treatment

Water levels in sediment basins should be kept as low as possible so that they are always ready to accept sediment laden runoff from the next rainfall event. The above mentioned guidelines (Section 3.4.1.2.iv(b)) suggest a minimum settling time of 24 hours to ensure a reasonable level of basin efficiency.

Water stored in sediment basins can be chemically treated to speed up the process of sediment deposition and to enable finer particle sizes to be trapped. Chemical treatment is the only practical approach on most development sites for the capture of particle sizes less than 0.02mm.

A suitable chemical for treating turbidity is gypsum, which does not change the pH of the water. Other chemicals (lime and alum) may be considered in special circumstances, but they do effect the pH. Gypsum should be used at a rate not less than 32 kg per $100m^3$ of basin capacity.

Gypsum may be mixed in a tank or tanker, or into a slurry in a concrete mixer, then distributed to several points in the basin by a spray or spreading technique. Spraying over the surface is preferred, but in-basin mixing may be utilized.

(vi) Basin Inlet

Basin inlet channels should be protected against erosion and scour. This may be achieved by the use of a level spreader where the drop is not great, or by rip-rap protection on steeper entries. A suitable geotextile can also be effectively used for protection of inlet channels. In the case of stormwater pipes, discharge points can be protected by the use of rip-rap or a concrete discharge sump (concrete pipe of suitable diameter). Ideally, points of entry should be combined and located so as to ensure the maximum travel distance to the primary outlet.

(vii) Freeboard

For compacted earth basins, with catchments of less than fifteen (15) ha, a freeboard allowance of at least 0.75m should be provided between the design surcharge level in the basin and the top of the compacted embankment.

For compacted earth basins, with catchments larger than fifteen (15) ha and for environmentally sensitive sites, **calculation of freeboard should include additional allowance for the following:-**

- ***surcharge***
- ***wave action***
- ***clearance***
- ***settlement***

Guidance on the calculation of these factors is given in the Soil Conservation Service Technical Handbook No. 5.

(viii) Sediment Removal

Sediment basins must be cleaned out when the sediment storage volume is full. At no time must the sediment level be permitted to significantly encroach into the sediment settling zone above. The cleanout operation should restore the original design dimensions and volume to the basin.

The elevation of the maximum allowable sediment level should be determined and indicated on the erosion and sediment control plan. The monitoring of sediment removal will be simplified if the riser is marked accordingly.

Dewatering of accumulated sediment is not essential but does facilitate the clean out of basins. (Figure 3.8 and 3.9)

Sediment removed from a basin must be stockpiled or disposed of in an area where it will not be subsequently eroded and transported into watercourses or drains.

3.4.2 SEDIMENT TRAPS

Sediment traps are temporary sediment control structures formed by excavation and/or an embankment to intercept sediment laden runoff and retain the sediment. They function by trapping sediment in runoff before it enters stormwater pipes or channels, and are usually located at inlets which receive runoff from only a small catchment.

Sediment traps should be located outside the area being graded and installed prior to the start of grading activities or the removal of existing vegetation. Locate traps to obtain maximum storage benefit from the terrain, for ease of periodic cleanout and disposal of the trapped sediment, and in a manner which will not divert flows should they fail, or interfere with construction activities.

Sediment traps can be constructed from a wide range of materials, depending on site conditions, the area or structures to be protected, and the catchment area. The following are the most common types.

3.4.2.1 Excavated Sediment Traps

An excavated sediment trap is in effect a small temporary sediment basin, designed to function in a drainage area of 2 ha or less. (Figure 3.9)

An excavated trap is simpler to build, and generally takes up less surface area than an embankment. In accordance with the requirements of Table 3.6 (Section 3.4.1.2) an appropriately sized excavation is made, and a suitable outlet provided.

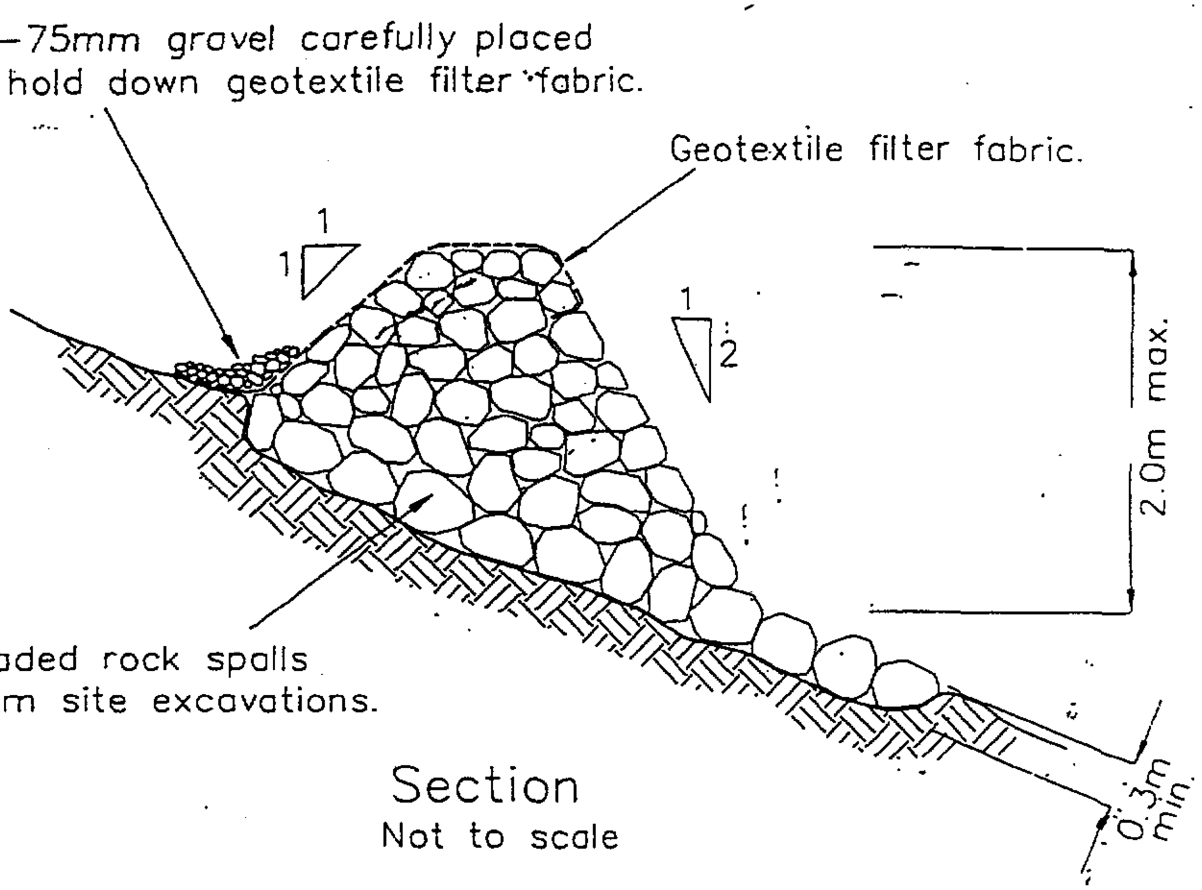
Temporary outlets can be a grassed spillway, gravel or stone, gravel with a straw bale core, or a pipe. Alternatively, permanent stormwater drainage inlets can be used as trap outlets, thus avoiding the need to construct separate temporary structures and preventing the erosion of downstream drainage channels.

On very steep or fragile areas, a stacked rock and geotextile filter fabric sediment trap may be more appropriate (Figure 3.10)

3.4.2.2. Stormwater Inlet Sediment Traps

Stormwater drainage systems often carry runoff before their drainage area is stabilised, and they can convey large amounts of sediment to streams or lakes. If erosion is extensive, the stormwater drain itself may clog and lose a major portion of its capacity. To prevent this, it is necessary to prevent sediment from entering the stormwater inlets.

Department of Conservation and Land Management



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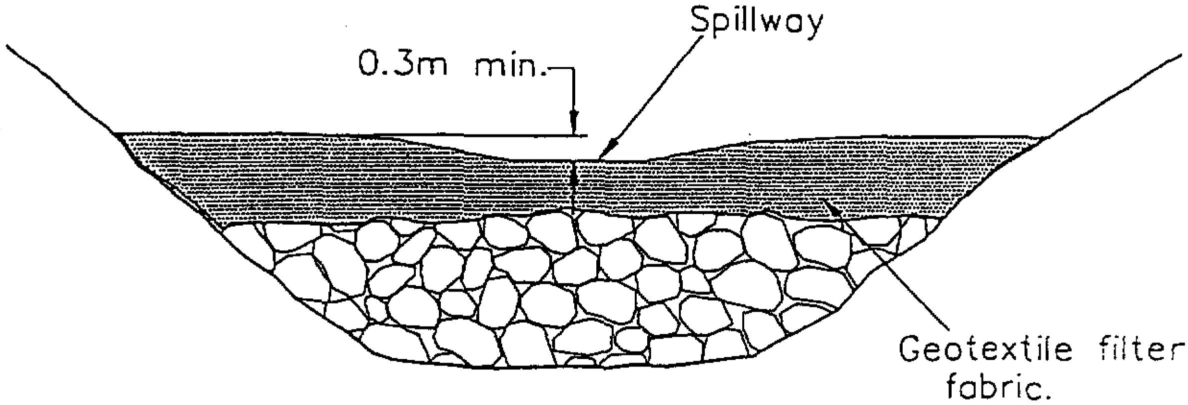

Front Elevation
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Stacked Rock Sediment Trap.

Fig. 3.10

The best way to do this is to stabilise the site with vegetation as quickly as possible, trap sediment near its source with sediment filters, and pave streets and install kerbs and gutters on schedule. As this is not always possible, inlet protection should be provided to reduce the sediment load entering the stormwater drainage system. Materials commonly used for this purpose include straw bales, geotextile filter fabric, gravel, and sand bags. Several types of inlet filters are described below. The choice of filter structure depends upon site conditions and the type of inlet being protected.

It is generally both convenient and cost-effective to construct the permanent stormwater drainage system at the beginning of a project. Certain inlets can then be used as the risers for sediment basins or traps. The area around the inlet can be excavated to form the storage area of the trap. The following inlet protection structures represent those most frequently used and recommended. The development of alternative innovative techniques, which can accomplish the some purpose, should be encouraged.

These structures are designed to keep sediment out of stormwater drain inlets where the drainage area is 0.4 ha or less, but do not have provision for sediment storage. Excavation of an area around the inlet to allow deposition of sediment will not only improve the structure's efficiency (Figure 3.10) and reduce frequency of maintenance, but may allow the structure to serve an area larger than 0.4 ha - subject to the requirements of Table 3.6 (Section 3.4.1.2)

(i) Straw Bale Drop Inlet Sediment Trap

A straw bale drop inlet sediment trap can be used where the inlet drains a relatively flat disturbed area (slopes no greater than 5 per cent) on which sheet flow occurs. Traps of this type should not be placed around inlets receiving concentrated flows such as those along major streets or roads. They are installed as follows (Figure 3.11):

1. Excavate a 0.1m deep trench the width of a straw bale around the inlet.

2. Place bales lengthwise around the inlet and press the ends of adjacent bales together. Orientate straw bales with the bindings around the sides of the bales rather than underneath.

3. Drive two wooden stakes (50 x 50 mm) or metal stakes of equivalent strength, through each bale to anchor it securely.

4. Backfill the excavated soil and compact it against the bales.

5. If gaps are present, wedge loose straw between the bales to prevent water from flowing between them.

The efficient life of this structure can be extended if a geotextile filter fabric is installed on the upstream side of the straw bales, as detailed in Section 3.4.3.3.
Straw bales should be inspected after each rain event for displacement, undercutting and overtopping, and repaired immediately.

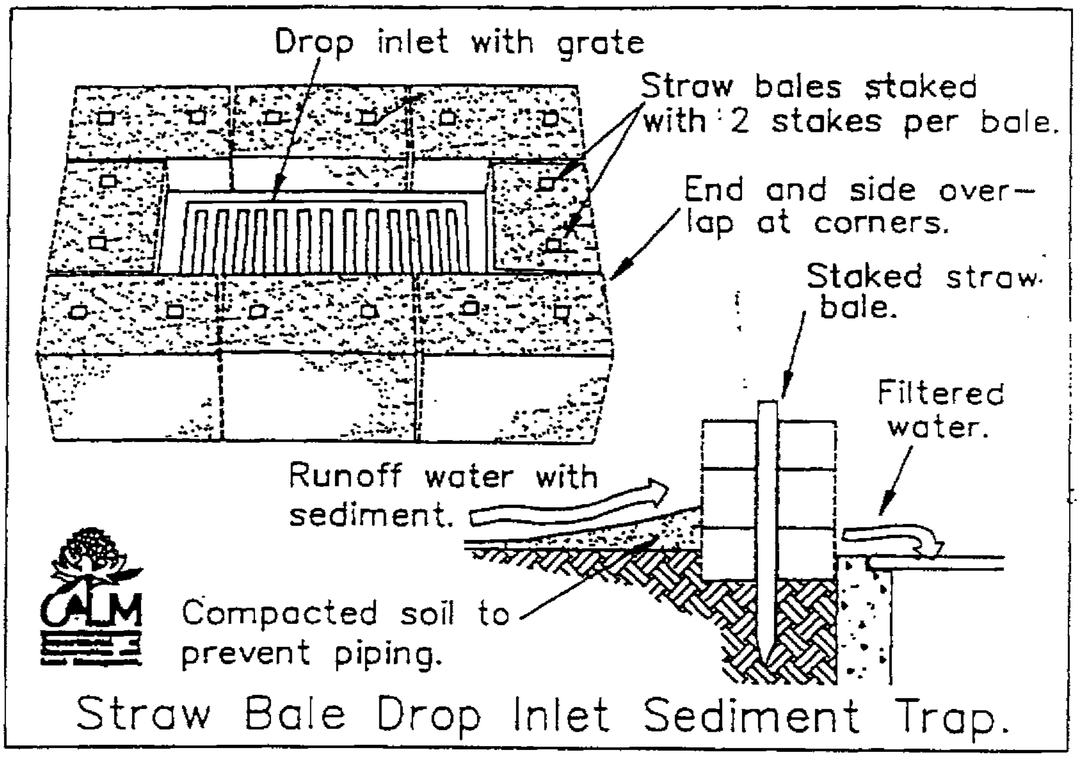
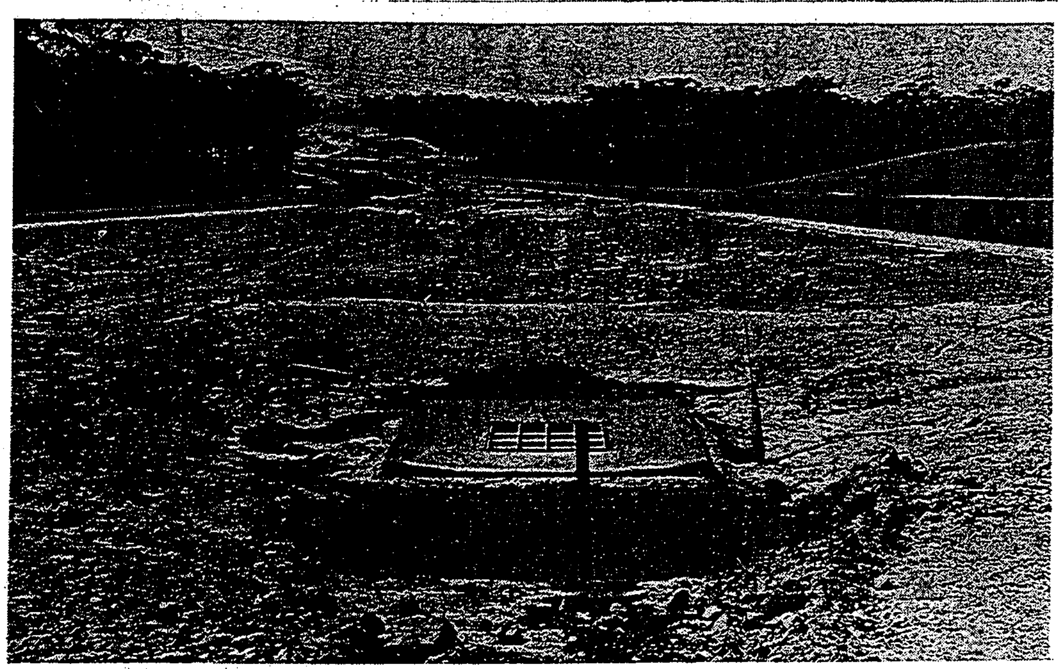
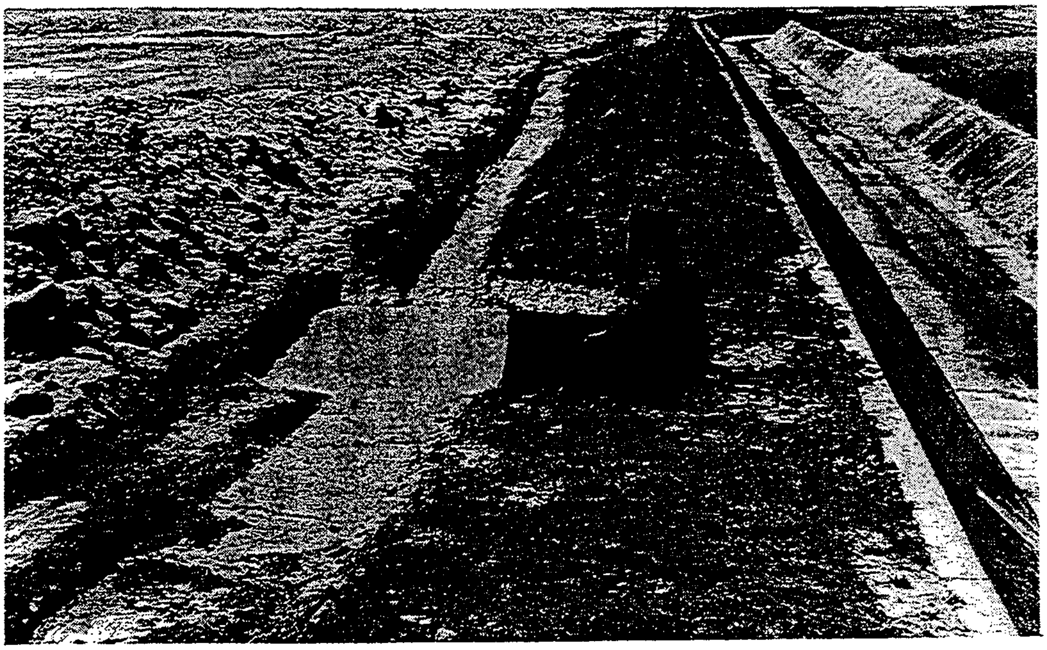


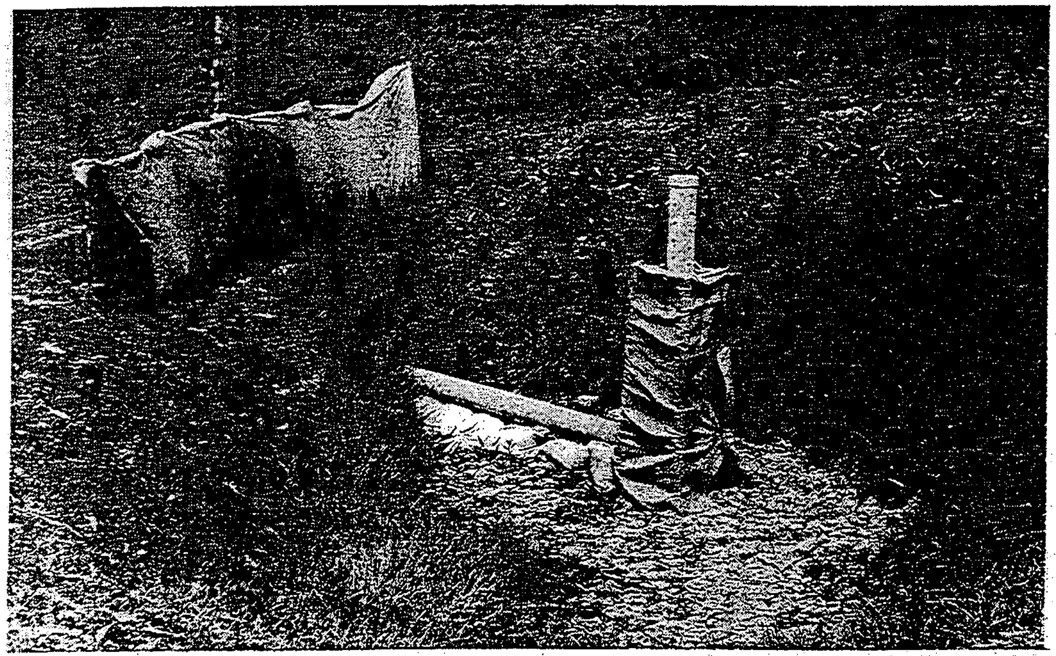
Fig. 3.11

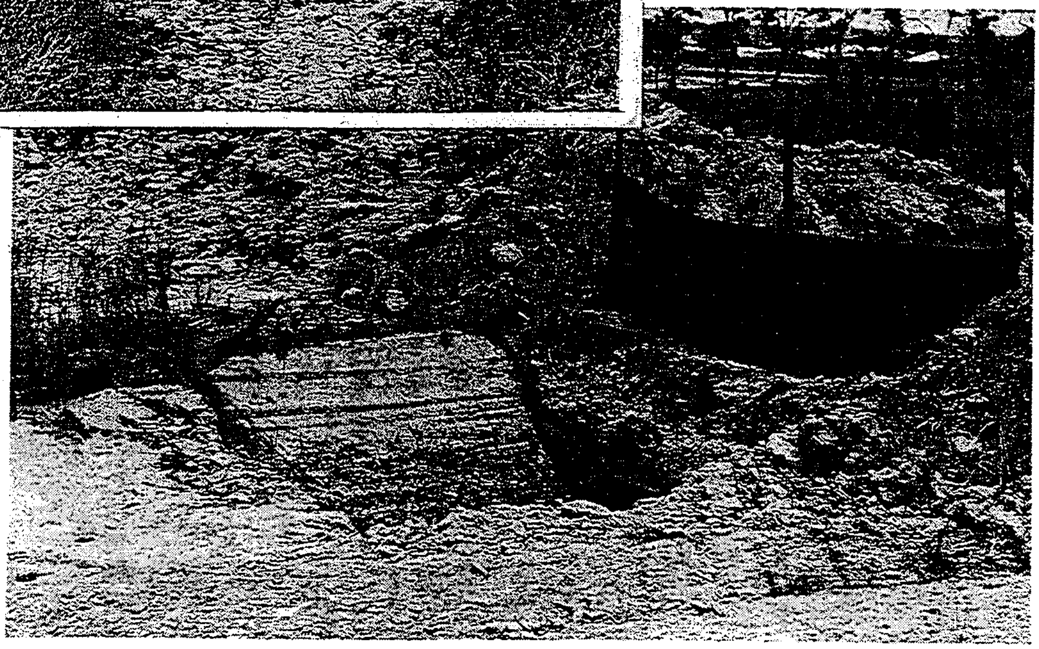
Source: Goldman et.al.



Excavated sediment traps can utilise existing stormwater drainage structures as trap outlets, or ....







.... a range of temporary filter outlets, including ....

(ii) Geotextile Filter Fabric Drop Inlet Sediment Trap

This is an alternative to the straw bale drop inlet sediment trap, and can be used in similar circumstances. However, the use of geotextile filter fabric increases the efficiency and effective life of this structure over that of the straw bale trap.
They are installed as follows (Figure 3.12):

1. Place stakes around the perimeter of the inlet a maximum of 0.9m apart and driven at least 0.3m into the ground. 50 x 50 mm wooden stakes or metal stakes of equivalent strength, at least 0.9m long, are recommended.

2. Excavate a trench approximately 0.1m wide and 0.2m deep around the outside perimeter of the stakes.

3. Staple an appropriate geotextile filter fabric to the stakes so that 0.2m of the fabric extends into the trench. Cut the fabric from a continuous roll to avoid more than one joint. Splice filter cloth at final joint, with a minimum 0.15m overlap, and securely fasten both ends to the support stake.

4. Backfill the trench and compact the soil over the fabric.

Filter fabric should be inspected after each rain event for undercutting, sagging or overtopping, and repaired immediately.

(iii) Wire Mesh and Gravel Drop Inlet Sediment Trap

This design can be used where heavy, concentrated flows are expected, such as at drop inlets in unpaved streets or major drainage channels. Because this trap has no means of handling overflows, it is likely to cause ponding, especially if sediment is not removed regularly. Therefore, it should not be used where an overflow would endanger an exposed fill slope, where ponding would interfere with traffic movement or construction work or damage adjacent structures or property. In locations where ponding would be a problem, use a block and gravel drop inlet structure (see (iv) below).

They are installed as follows (Figure 3.13):

1. Place heavy gauge wire mesh over the drop inlet so that the wire extends a minimum of 0.3m beyond each side of the inlet structure. Wire mesh with 12 mm openings should be used. If more than one strip of mesh is necessary, overlap the strips. For inlet pits larger than 0.6 x 0.6m, a supporting piece of reinforcing mesh should be placed under the wire mesh and extended 0.6m beyond the inlet opening.

2. Place a layer of 50 to 75 mm sized gravel over the wire mesh to a depth of at least 0.3m over the entire inlet opening. Extend the gravel beyond the inlet opening at least 0.5m on all sides.

3. If the gravel filter becomes clogged with sediment, the gravel must be pulled away from the inlet, cleaned and/or replaced.

If desired, geotextile filter fabric can be incorporated into the installation of this structure to improve filtering efficiency, in either of the following ways:

(i) Construct as above but replace the wire mesh with geotextile filter fabric held down by the gravel layer.

(ii) Replace both the wire mesh and the gravel with geotextile filter fabric with its edges entrenched 0.2m into the soil on all sides of the inlet. Backfill the trench and compact the soil.

(iv) Block and Gravel Drop Inlet Sediment Trap

This type of trap can be used where heavy flows are expected. It has an overflow mechanism to prevent excessive ponding around the structure.

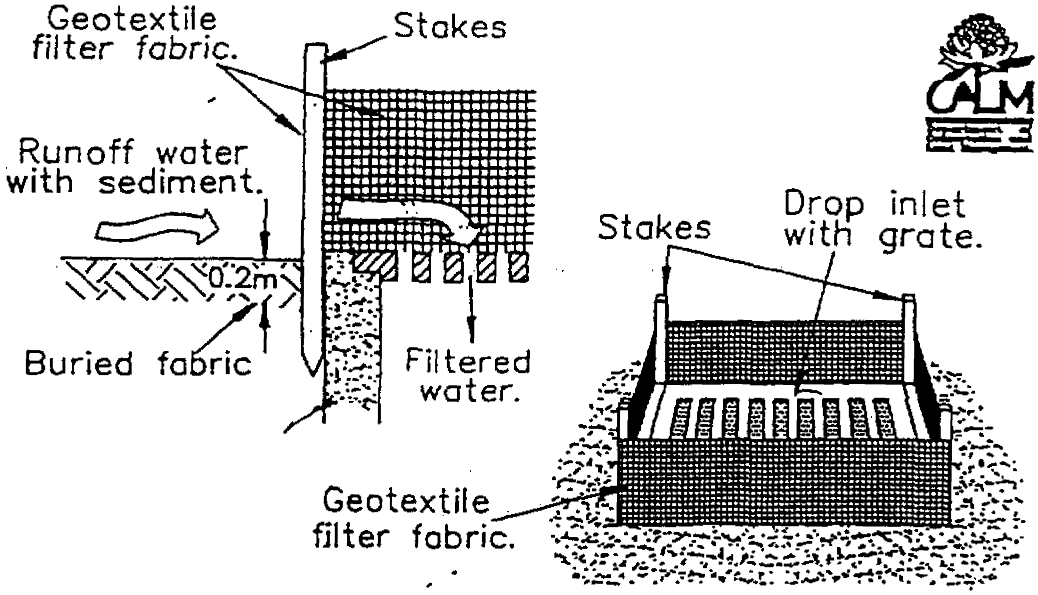
They are installed as follows (Figure 3.14):

1. Place concrete blocks lengthwise on their sides in a single row around the perimeter of the inlet, with the open ends facing outward, not upward. The ends of adjacent blocks should abut. The height of the trap can be varied, depending on design needs, by adding additional rows of blocks. The block wall should be at least 0.3m high but no greater than 0.6m.

2. Place wire mesh over the outside vertical face (open end) of the concrete blocks to prevent gravel from being washed through the blocks. Wire mesh with 12 mm openings should be used.

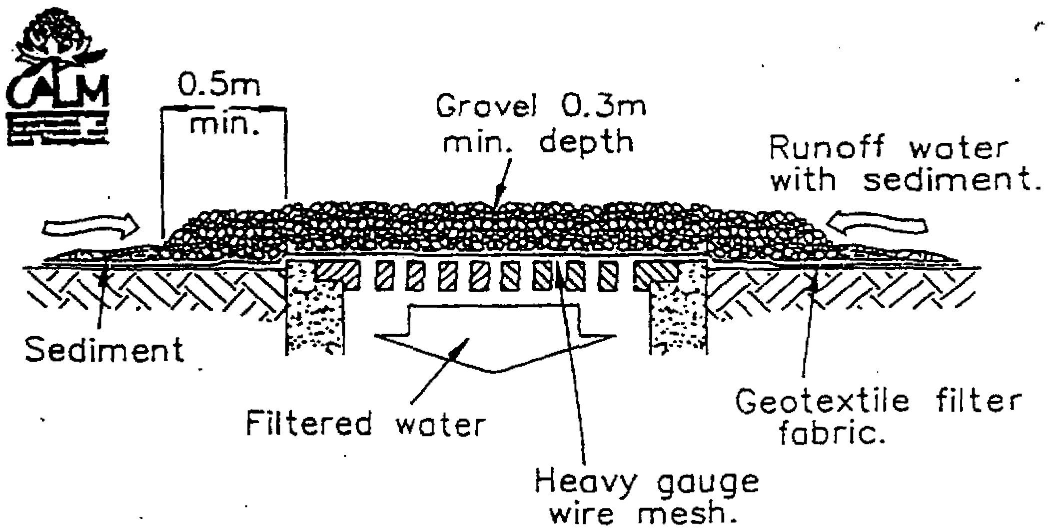
3. Place 50 to 75 mm sized gravel against the wire mesh, level with the top of the block wall.

4. If the gravel filter becomes clogged with sediment, the gravel must be pulled away from the blocks, cleaned and/or replaced.



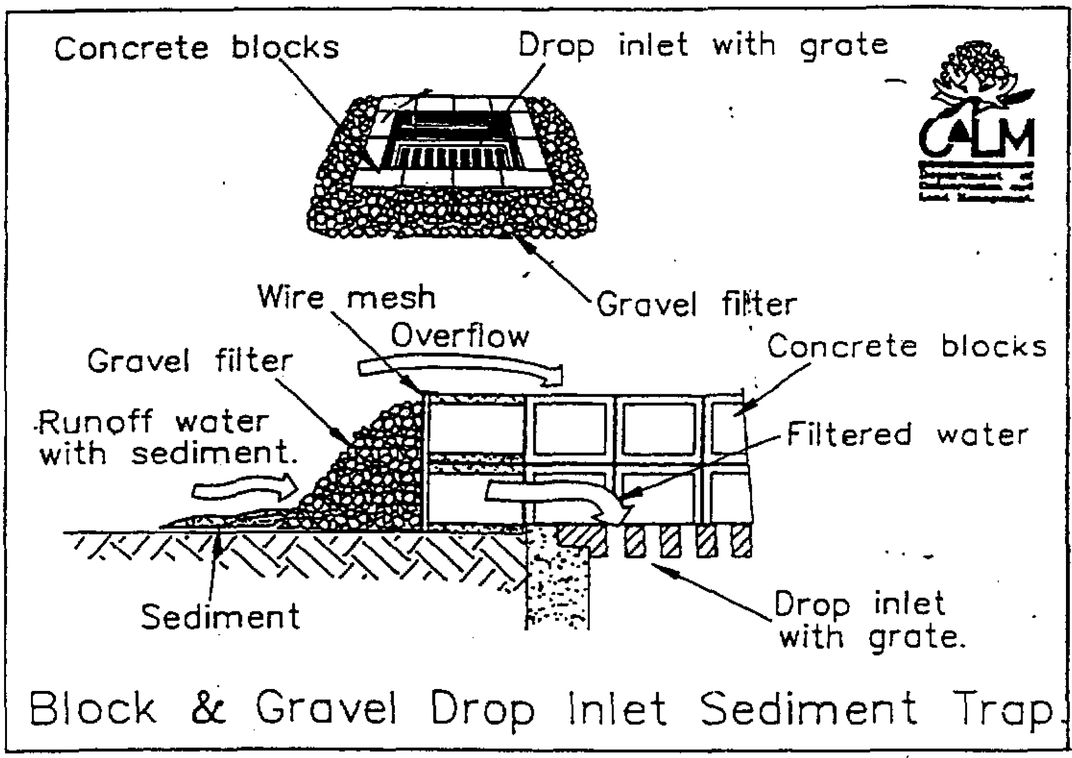
Geotextile Filter Fabric Drop Inlet Sediment Trap. Source: Goldman et.al.

Fig. 3.12

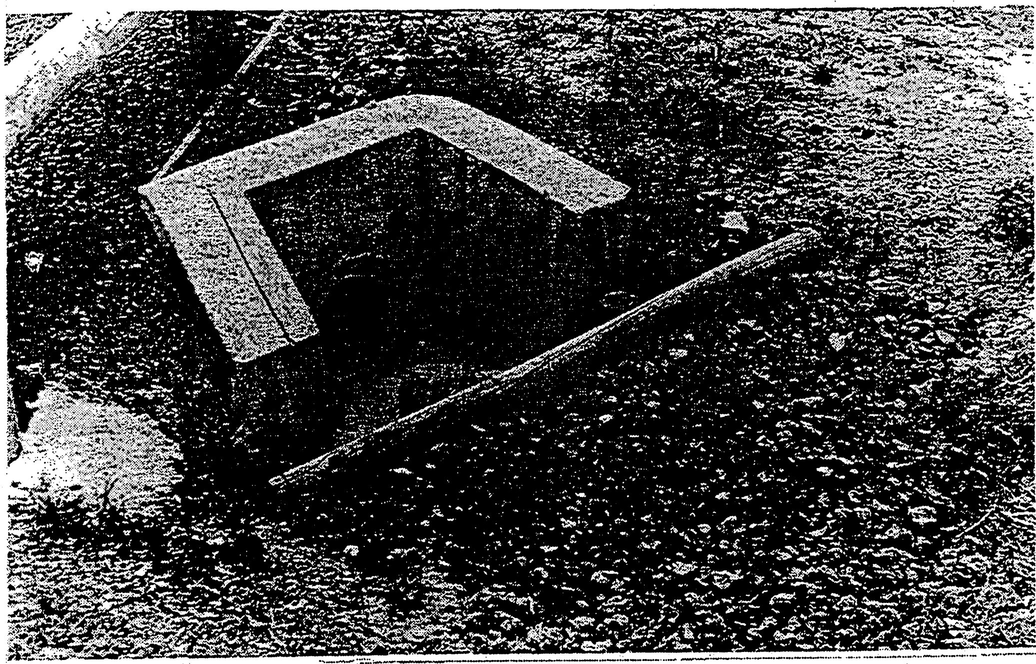


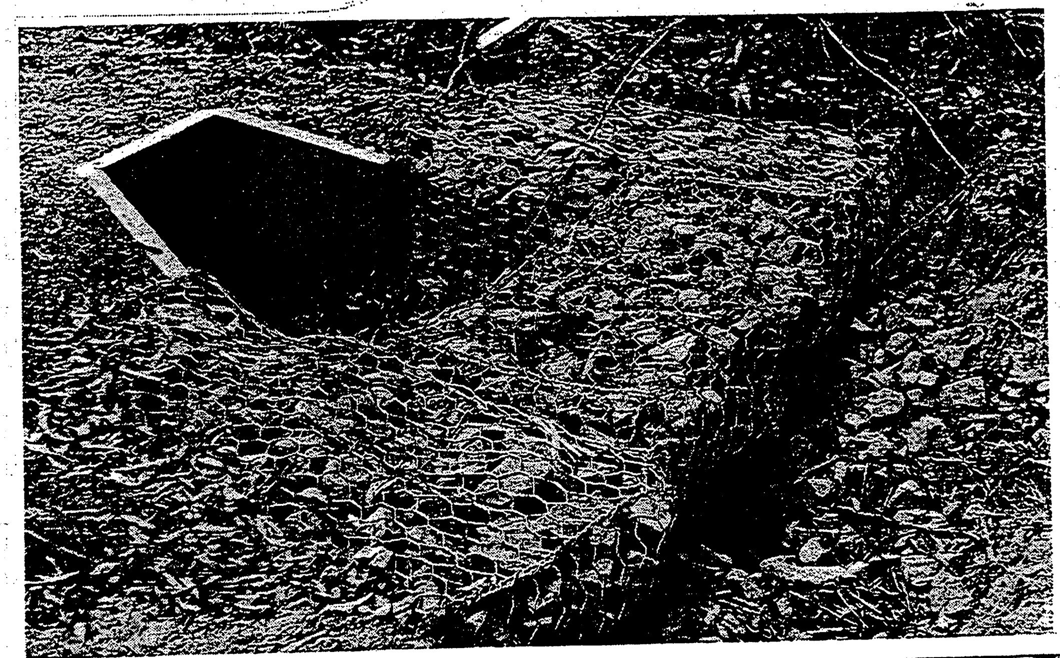
Wire Mesh and Gravel Drop Inlet Sediment Trap. Source: Goldman et.al.

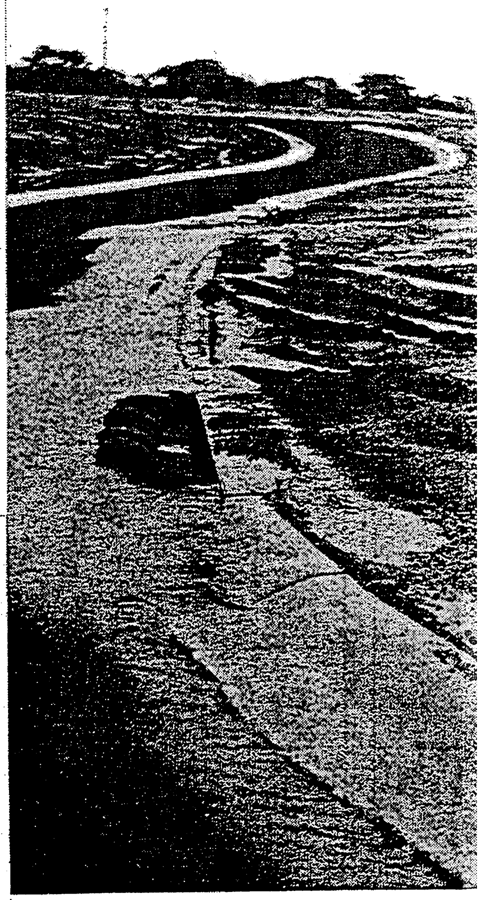
Fig. 3.13

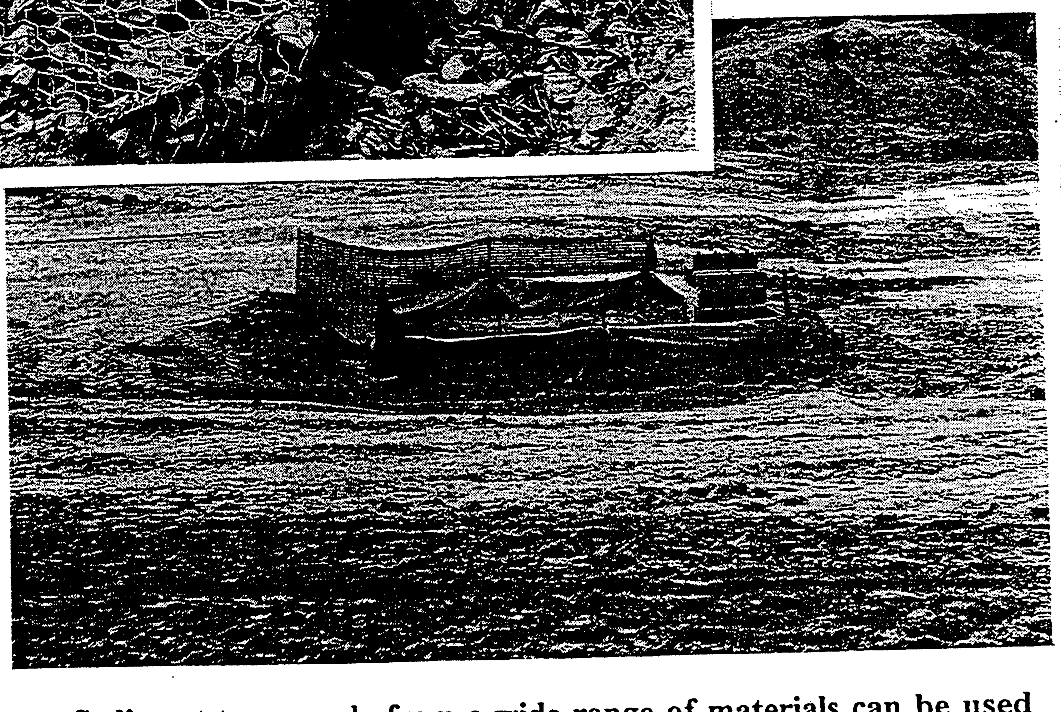


Source: Goldman et.al. Fig. 3.14









Sediment traps made from a wide range of materials can be used to protect stormwater inlets.

(v) Gravel Kerb Inlet Sediment Trap

A gravel kerb inlet sediment trap can be used at a kerb inlet if ponding in front of the inlet is not likely to cause inconvenience or damage to adjacent structures and unprotected areas. If ponding presents a problem, use a block and gravel kerb inlet sediment trap (see (vi) below).

They are installed as follows (Figure 3.15):

1. Place wire mesh with 12 mm openings over the kerb inlet opening so that at least 0.3m of wire extends beyond the top, bottom and sides of the inlet opening.

2. Place 50 to 75 mm sized gravel against the wire mesh to anchor the wire against the gutter, and cover the inlet opening completely.

3. If the gravel filter becomes clogged with sediment, the gravel must be pulled away from the block, cleaned and/or replaced.

A semi-portable variation of this structure, which is particularly suitable for overnight or short term use, is a cylinder of wire mesh and geotextile filter fabric filled with 50 to 75 mm sized gravel and placed against the kerb inlet. The cylinder should be securely sealed at sides and ends, and generally be 0.15 to 0.3m in diameter. This structure is simple to make and place on site and is easily cleaned, moved and reused. (Figure 3.16)

(vi) Block and Gravel Kerb Inlet Sediment Trap

A block and gravel kerb inlet sediment trap can be used at a kerb inlet if an overflow mechanism is needed to prevent excessive ponding in front of the inlet.

They are installed as follows (Figure 3.17):

1. Place two concrete blocks on their sides, perpendicular to the kerb at either end of the inlet opening, to serve as spacer blocks.

2. Place concrete blocks lengthwise on their sides across the front of the inlet and abutting the spacer blocks. The openings in the blocks should face outward, not upward.

3. Cut a 100 x 50 mm wooden stud the length of the curb inlet plus the width of the two spacer blocks. Place the stud through the outer hole of each spacer block to help keep the front blocks in place.

4. Place wire mesh over the outside vertical face (open ends) of the concrete blocks to prevent stones being washed through the blocks. Wire mesh with 12 mm openings should be used.

5. Place 50 to 75 mm sized gravel against the wire mesh to the level of the top of the concrete blocks.

6. If the gravel filter becomes clogged with sediment, the gravel must be pulled away from the blocks, cleaned and/or replaced.

Additional filtering efficiency can be achieved by placing a layer of geotextile filter fabric between the wire mesh and gravel.

A timber frame and formwork can be substituted for the concrete blocks provided the frame is strong enough (minimum 100 x 50 mm timber) and adequately anchored to the kerb to support the filtering medium.

(vii) Sandbag Kerb Inlet Sediment Trap

A sandbag kerb inlet sediment trap can be used at a kerb inlet on a paved street which receives relatively small runoff flows. Although simple to construct, it is not as effective as the inlet protection structures previously described and should therefore only be used for very temporary protection. Once the small catchment area behind the sandbags fills with sediment, further sediment-laden runoff will enter the storm drain directly. Therefore, sediment must be removed from these structures during or after every storm. Additional storage capacity can be obtained by constructing a series of sandbag barriers along a gutter so that each barrier traps only small amounts of sediment.

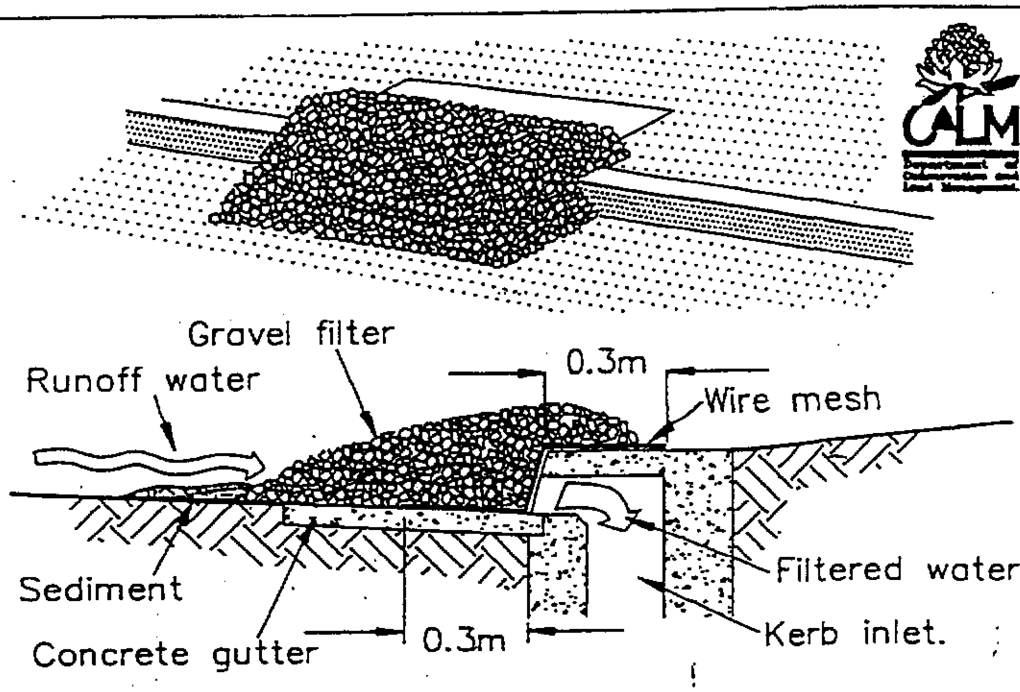
They are installed as follows (Figure 3.18):

1. Place a single layer of sandbags in a curved row out from the kerb, upstream of, and away from, the inlet. The row should be at least 2 m from the kerb inlet and should overlap onto the kerb as necessary to divert runoff through the barrier. It should extend into the street a sufficient distance to intercept runoff, but at least 0.9m.

2. Place additional layers of sandbags over the first, overlapping bags and packing them tightly together to minimise the space between them.

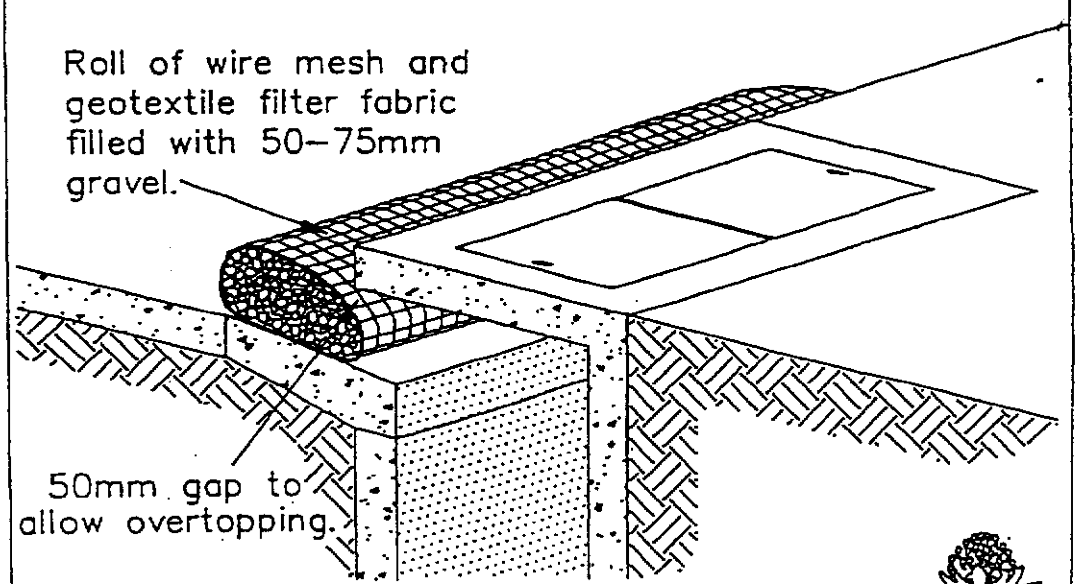
3. Leave a gap of one sandbag in the middle of the top row to serve as the spillway, ensuring the spillway is lower than the adjoining kerb.

4. Remove sediment when it reaches the top of the sandbag spillway and place it where it will not enter the stormwater drainage system.



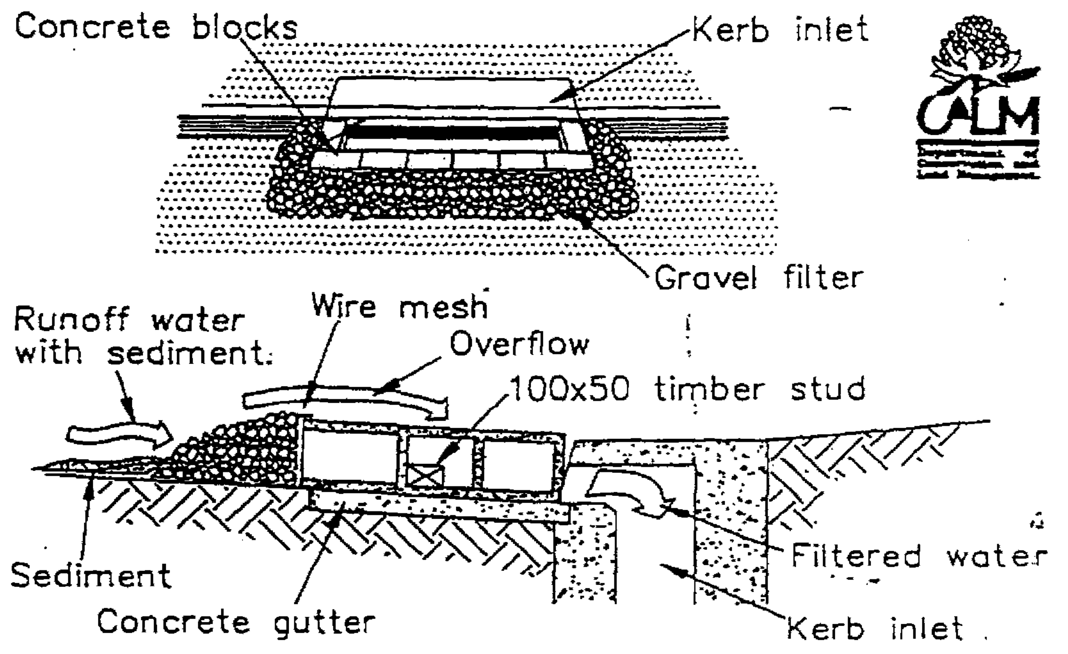
Gravel Kerb Inlet Sediment Trap.

Source: Goldman et.al. Fig. 3.15



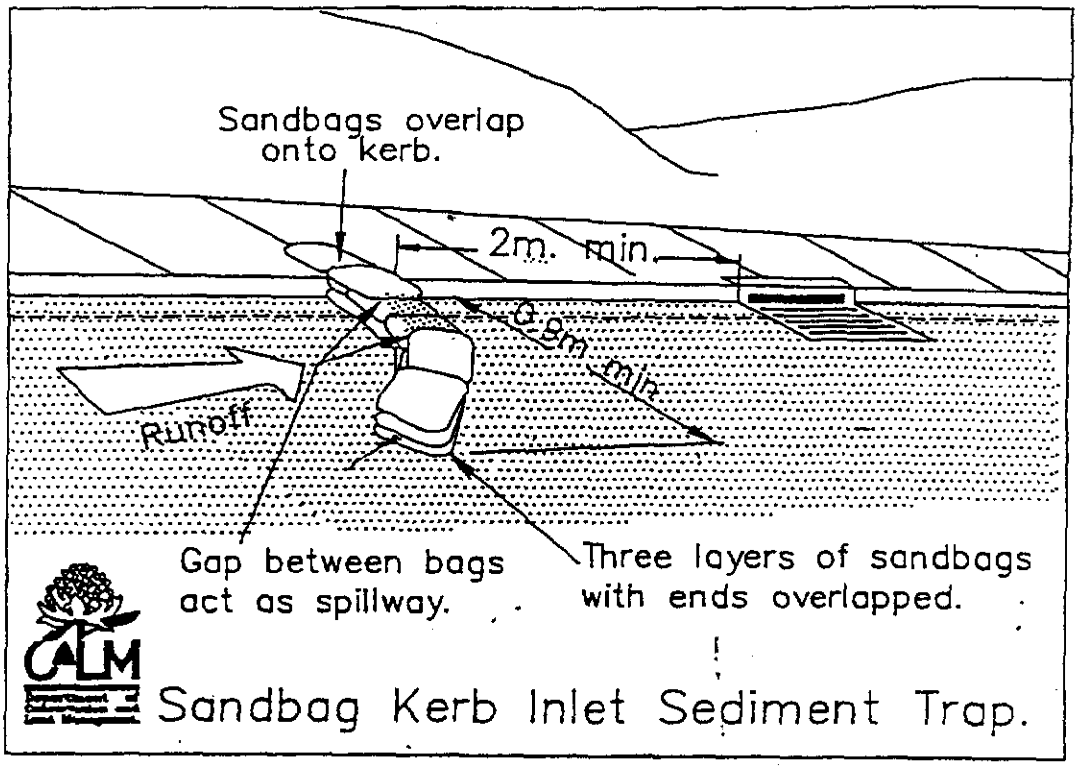
Portable Gravel Kerb Inlet Sediment Trap

Fig. 3.16



Block & Gravel Kerb Inlet Sediment Trap

Fig. 3.17



Source: Goldman et.al.

Fig. 3.18

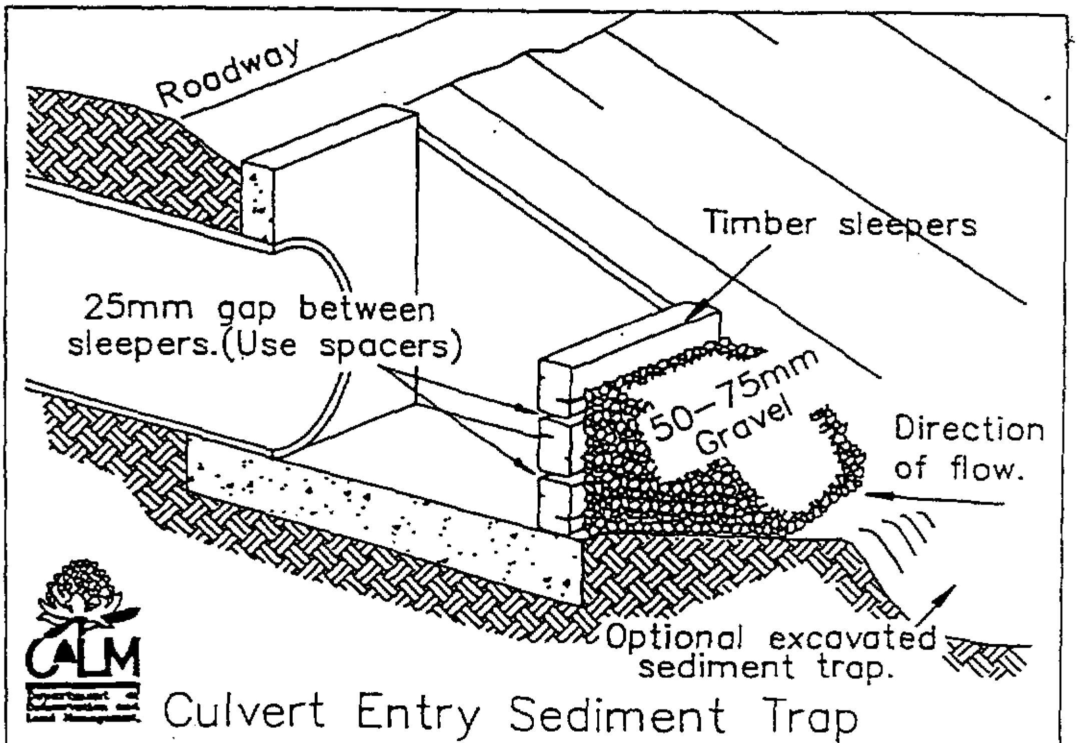


Fig. 3.19

(viii) Culvert Entry Sediment Trap

An existing road embankment and culvert can be converted for use as a temporary sediment trap. It can be used where heavy flows are expected, and incorporates an overflow mechanism to prevent excessive ponding.

They are installed as follows (Figure 3.19):

1 Place timber planks or sleepers (minimum 150 x 75 mm) across the front of the culvert entrance to form a timber wall, leaving a horizontal gap of 25 mm between each plank for filtered water to pass through. The height of the timber wall can be varied, depending on design needs, but should generally not exceed the diameter/height of the pipe/box culvert it is protecting.

Alternatively:

1(a) Build a wall of concrete blocks as follows:-

(a) Place two or more concrete blocks on their sides, perpendicular to the culvert entrance at either end of the entrance to serve as spacer blocks.

(b) Place concrete blocks lengthwise on their sides across the front of the culvert entrance and abutting the spacer blocks. The openings in the blocks should face outward, not upward.

(c) Cut a 100 x 75 mm wooden plank the length of the culvert entrance plus the width of the two spacer blocks. Place the plank through the outer hole of each spacer block to help keep the front blocks in place.

(d) The height of the trap can be varied, depending on design needs, by adding additional rows of blocks. However, the height should generally not exceed the diameter/height of the pipe/box culvert it is protecting.

2. Place 50 to 75 mm sized gravel against the timber/concrete block wall to act as a filter.

3. If the gravel filter becomes clogged with sediment, the gravel must be pulled away from the wall, cleaned and/or replaced.

Additional filtering efficiency can be achieved by placing a layer of geotextile filter fabric between the timber/concrete block wall and gravel.

(ix) Sump Pit Sediment Trap

On small, flat, construction sites with limited space for conventional sediment traps, and on sites where considerable excavation for foundations is involved, a temporary sump pit can be used to trap and filter water for disposal by pumping.

A sump pit comprises a perforated vertical standpipe placed in the centre of an excavated pit to collect water filtered through a layer of geotextile filter cloth and gravel. Water is then pumped from the centre of the pipe to a suitable discharge area.

They are installed as follows (Figure 3.20):

1. Excavate a pit, (dimensions are optional) in the path of any sediment laden runoff diverted from the disturbed area.

2. Place a pad of 50 to 75 mm sized gravel in the base of the pit to a depth of 0.3m.

3. Construct a standpipe by perforating 0.3 to 0.6m diameter corrugated or PVC pipe and placing it upright in the centre of the pit, on the gravel pad. The standpipe should extend to a height of 0.3 to 0.5m above the lip of the pit.

4. Backfill the pit around the standpipe with 50 to 75 mm sized gravel to the level of the lip of the pit.

Discharge of water pumped from the standpipe should be to a sediment trap or basin, or to a stabilised area. If water from the sump pit is pumped directly to a stormwater drainage system, geotextile filter fabric should be wrapped around the standpipe prior to backfilling with gravel, to ensure a clean discharge. It is recommended that wire netting be wrapped around and secured to the standpipe prior to attaching the filter cloth, to increase the rate of water seepage into the standpipe.

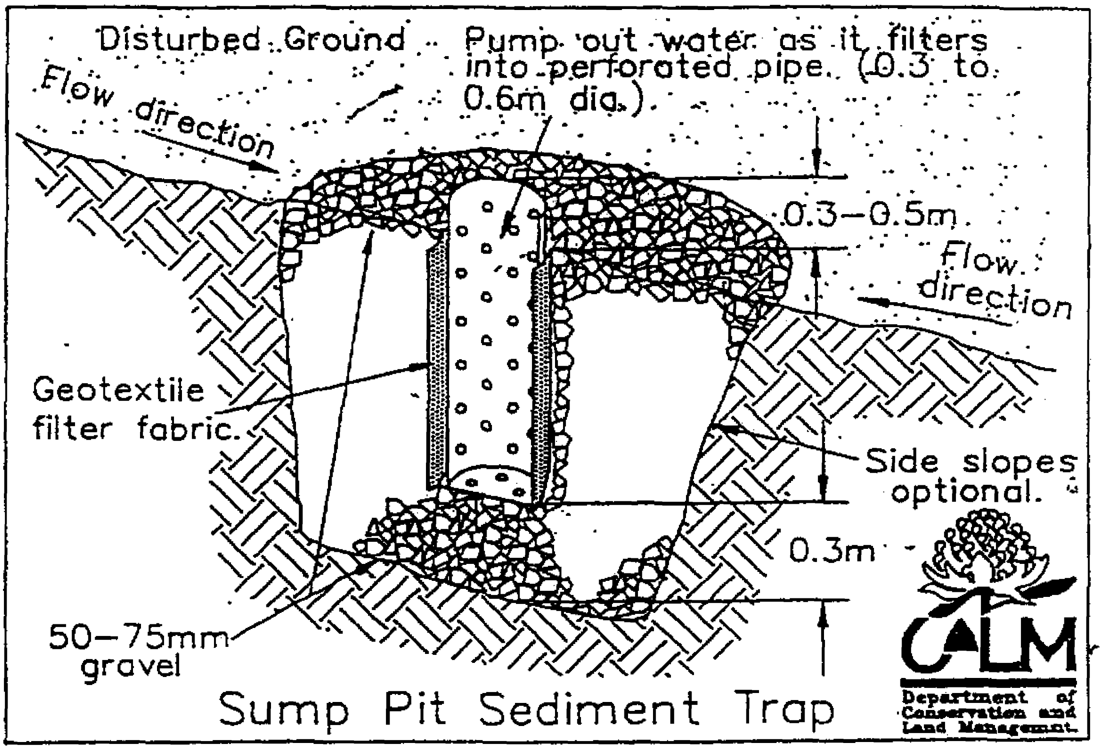

Disturbed Ground
Pump out water as it filters into perforated pipe. (0.3 to 0.6m dia.).
Flow direction
0.3–0.5m.
Flow direction
Geotextile filter fabric.
Side slopes optional.
0.3m
50–75mm gravel
Sump Pit Sediment Trap
CALM
Department of Conservation and Land Management

Fig. 3.20

3.4.3 SEDIMENT FILTERS

Sediment basins and traps function by impounding relatively large volumes of runoff from disturbed areas, particularly where the runoff is concentrated or channelled, and allowing sediment to settle out. Sediment filters, on the other hand, function by intercepting and filtering small volumes of runoff which mainly occur as sheet flow.

These structures are used:-

* *Below small areas of disturbance.*
* *Along the boundaries of a development.*
* *At the beginning of vegetative filter or buffer strips.*

Sediment filters have a useful life of only 3 to 12 months depending on the materials used. Straw bales last up to 3 months; sediment fences can function for 6 months or longer if sediment accumulations are removed. Sediment fences also trap a higher percentage of sediment than straw bale banks.

Sediment filters are relatively inexpensive and easy to install. However, they can only function to their design limits if particular care is taken with their location, installation and maintenance.

Should a sediment filter fail, sheet flow is changed into concentrated flow and serious damage can result, damage which may exceed that if no sediment filter had been installed.

There are four major types of sediment filters.

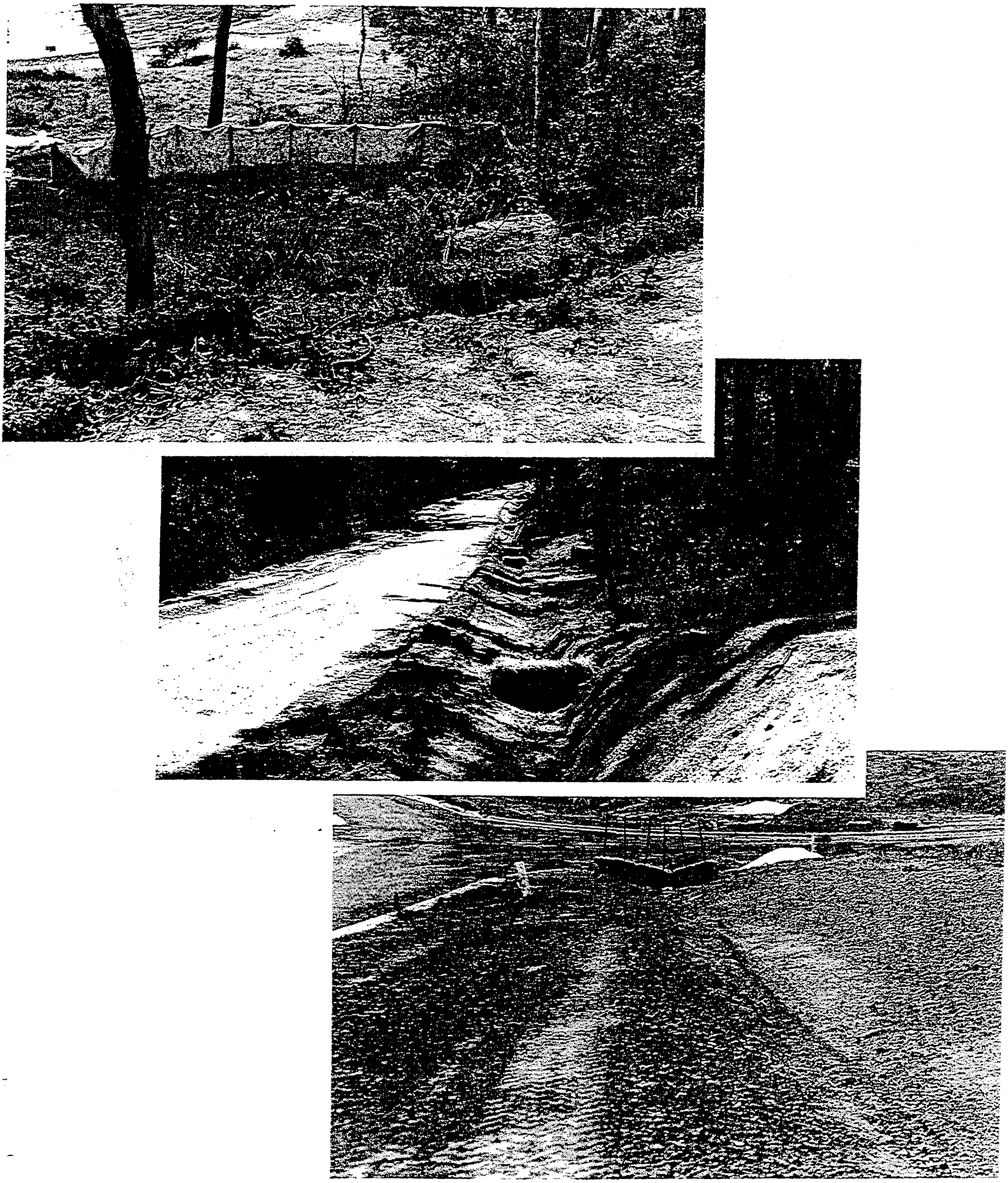
3.4.3.1 Straw Bale Sediment Filters

These are temporary structures made of straw bales laid end to end across the direction of flow.

Straw bale sediment filters may be used where:-

- *The area draining to the filter is 0.4 ha or less.*
- *The maximum slope gradient behind the filter is 1:2.*
- *The maximum slope length behind the filter is 40m.*

Detailed design will be required for any individual structures that exceed these general design parameters.



Straw bales, used alone or in conjunction with other materials, make effective temporary sediment filters in low flow situations.

They are installed as follows (Figure 3.21):

1. Excavate a 0.1m deep trench the width of a bale and the length of the proposed sediment filter. The filter should follow the slope contour. If the filter is at the toe of a slope, place it 1.5 to 2 m away from the slope if possible, to provide access for maintenance and to allow coarse sediment to drop out of suspension before it reaches the sediment filter.

2. Place straw bales lengthwise in the trench with their ends tightly abutting. Corner abutment is not acceptable. A tight fit is important to prevent sediment from escaping through the spaces between bales.

3. All bales must either be wire-bound or string-tied. Install bales so that bindings are orientated around the sides rather than along the tops and bottoms of the bales. If the binding is placed in contact with the soil, it will soon disintegrate and cause the bale to fall apart.

4. Straw bales, preferably cereal straw, should be used, not hay bales. Hay bales cost more than straw because they contain the edible portion of the grain and have a high leaf content. Therefore, they rot faster and require more frequent replacement.

5. Securely anchor each bale by driving at least two stakes through the bale. Drive the first stake in each bale toward the previously laid bale to force them together. Drive the stakes at least 0.6 m into the ground. Wooden stakes, (50 x 50 mm) or metal stakes of equivalent strength, are suitable. Staking is particularly crucial because, until they are thoroughly wet, the bales may float.

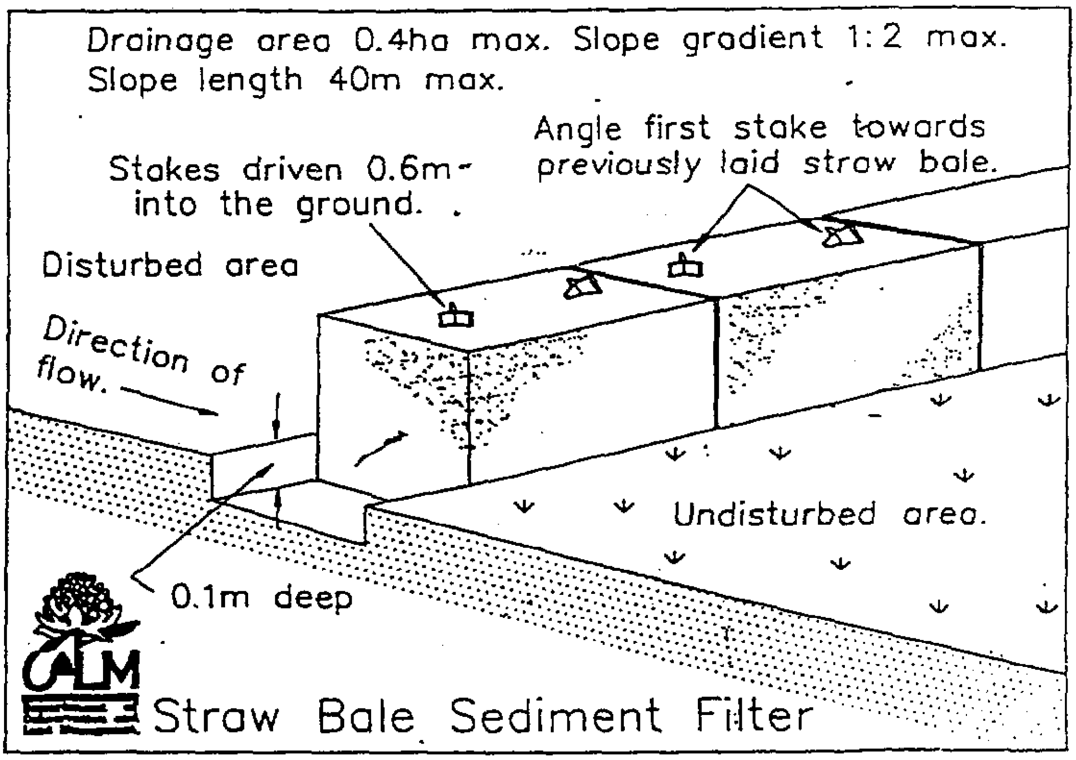
6. Fill any gaps between bales by wedging loose straw between them. Loose straw scattered over the area immediately uphill from a straw bale barrier tends to increase barrier efficiency. It is picked up by runoff and transported to holes in the barrier, which it tends to seal.

7. Backfill the trench with the excavated soil and compact it. The backfilled soil should conform to the ground level on the downhill side of the barrier, and should be built up to 0.1m above the ground on the uphill side of the bales.

8. Inspect and repair or replace damaged bales promptly. Straw bales typically deteriorate within 3 months when wet. Remove the straw bales when the upslope areas have been permanently stabilised.

Where the straw bale sediment filter crosses a swale, install the straw bales as described above, with the following additional requirements:-

- Place bales perpendicular to the flow.



Source: Goldman et.al. Fig. 3.21

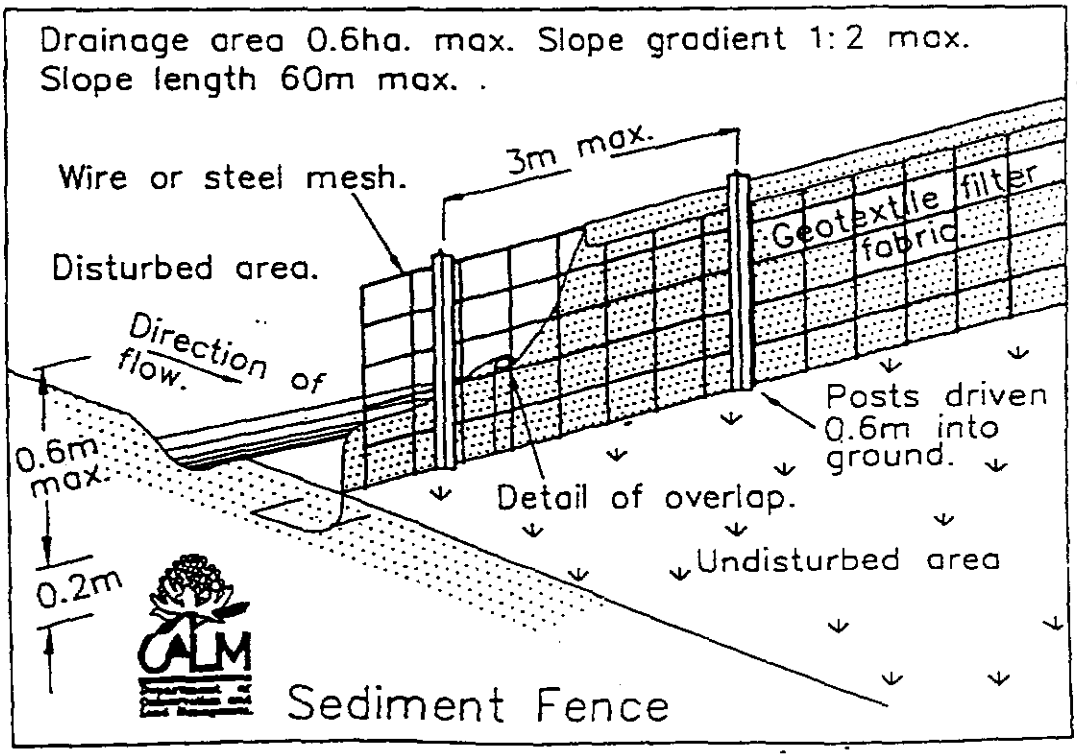


Fig. 3.22

- Extend the sediment filter to such a length that the bottoms of the end bales are at a higher elevation than the top of the lowest middle bale, to ensure that sediment-laden runoff will flow either through or over the barrier, but not around it. Rock placed below the middle bale will dissipate the energy of the falling water and reduce downstream erosion.

Straw bale sediment filters should be inspected after each rain event for displacement, undercutting and over-topping, and repaired immediately. Most straw bale filter failures are related to the following installation problems:

- Bales not staked firmly into the ground.
- Bales not trenched.
- Bales not butted tightly end-to-end.
- Insufficient space provided for sediment entrapment.
- Access for cleaning not provided.
- Bales displaced by equipment and not restored to their original position at the end of the day.
- Straw bale filter not centred in the flow path.

3.4.3.2 Sediment Fences

A sediment fence (often called a silt fence) is a temporary barrier of geotextile filter fabric, usually supported on wire or mesh fencing. It functions by retaining sediment on the site and reducing the runoff velocity across areas below it. The reduction in runoff velocity at the fence causes the majority of suspended soil particles to settle, with the fabric retaining further soil particles by filtration at its surface.

A sediment fence may be used where:

- ***The area draining to the fence is 0.6 ha or less.***
- ***The maximum slope gradient behind the fence is 1:2.***
- ***The maximum slope length behind the fence is 60 m.***

Detailed design will be required for any individual structures that exceed these general design parameters.

A sediment fence can last up to 6 months or longer, about twice as long as a straw bale sediment filter. A properly installed sediment fence is more effective than a straw bale sediment filter but also more costly. The greater effectiveness of the sediment fence is due to stronger construction, greater depth of ponding, and better installation practices. In addition, geotextile filter fabric allows fewer soil particles to pass through.

The selection of a geotextile filter fabric is based on soil conditions at the construction site and characteristics of the support fence. A filter fabric should be specified that retains the soil found on the construction site but will have openings large enough to permit drainage and prevent clogging.

Sediment fences are installed as follows (Figure 3.22):

1. Lay out a suitable fence line and set posts along its length. On slopes, align the fence along the contour as closely as possible. In small swales, curve the fence line upstream at the sides, to direct the flow toward the middle of the fence, with the sides higher than the centre.

   Space posts a maximum of 3 m apart and drive them at least 0.6m into the ground. If extra-strength or self-supporting fabric is used without the wire support fence, post spacing must not exceed 1.8 m. Posts for sediment fences can be either 0.1m diameter wood or equivalent 1.5 kg/m steel with a minimum length of 1.2 m. Steel posts must have projections to which the wire is fastened.

2. Excavate a trench approximately 0.1m wide and 0.2m deep along the line of posts and upslope from the barrier.

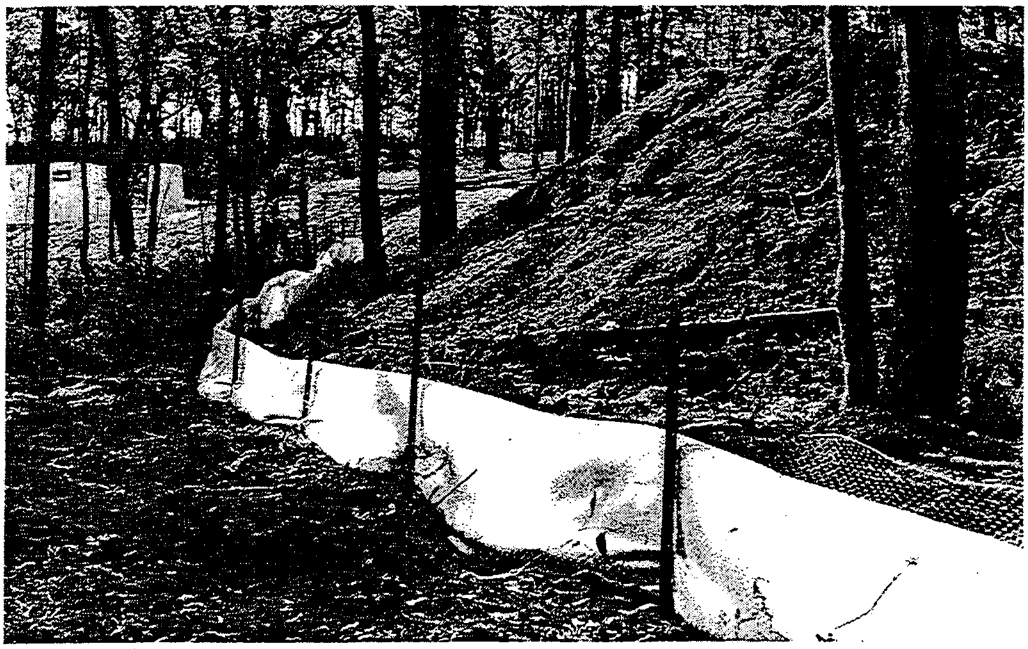
3. Fasten wire mesh securely to the upslope side of the posts. Use heavy-duty wire staples at least 2.5 cm long and tie wire. Extend the wire mesh 0.15m into the trench. Wire fence reinforcement for sediment fences must be a minimum of 14 gauge and have a maximum mesh spacing of 0.15m. (Note: When extra-strength or self-supporting fabric is used, the fence posts can be more closely spaced and the wire mesh omitted.)

4. Fasten the filter fabric to the uphill side of the fence posts, and extend it 0.2m into the trench. The height of the fence should not exceed 0.6 m. Do not staple fabric onto trees. Cut the filter fabric from a continuous roll to avoid the use of joints. When joints are necessary, splice the filter cloth at a support post, with a minimum 0.15m overlap, and securely fasten both ends to the post.

5. Backfill the trench over the toe of the fabric and compact the soil.

Sediment fences should be inspected after each rain event for undercutting, sagging and overtopping, and repaired immediately.

Sediment fences are effective temporary sediment filters that are easy to install and provide a very practical solution for most developments.



3.4.3.3 Straw Bale-Geotextile Fabric Sediment Filters

Straw bales and geotextile filter fabric can be used together to construct an effective sediment filter. The combination, although more expensive than either material used separately, compensates for the shortcomings of each. Straw bale sediment filters are frequently ineffective because they are not firmly staked and are not butted tightly together. When wrapped and secured with geotextile fabric, the bales have additional support and the gaps between them are covered with filter material.

Straw bale and geotextile sediment filters may be used where:-

- *the area draining to the filter is 0.5ha or less;*
- *the maximum slope gradient behind the filter is 1:2;*
- *the maximum slope length behind the filter is 50m.*

Detailed design will be required for any individual structures that exceed these general design parameters.

These structures are installed as follows (Figure 3.23):

1. Excavate a trench a few centimetres wider than the straw bales. Place the bales against the downslope side of the trench and anchor as described in Section 3.4.3.1.

2. Place appropriate geotextile filter fabric against the upstream face of the bales and extend it into the trench. Staple the fabric to the bales with 0.15 to 0.2m U-shaped wire pins.

3. Backfill the trench and compact the soil against the fabric and bales.

3.4.3.4 Vegetative Filter Strips

The maintenance of filter strips of existing vegetation adjacent to site boundaries, wetlands, streams and other areas of significant natural resource value, can aid in filtering stormwater runoff during and after the construction stage. This applies particularly where the vegetation comprises tall, dense grass, or ground covers on sites of low to medium slope. See Figure 3.24.

These filter strips may be contour strips located on long slopes to intercept overland flow, unmown bands across waterways, or natural swales to intercept channel flows, or buffer zones around stormwater inlets. The flatter and wider the strips, the more sediment they will retain.

The width of filter strip needed for such areas in order to provide adequate protection depends on the area to be protected, the vegetation type, the slope present, the ability of the soils in the buffer to absorb water, the size distribution of the incoming sediment, and the rate of runoff.

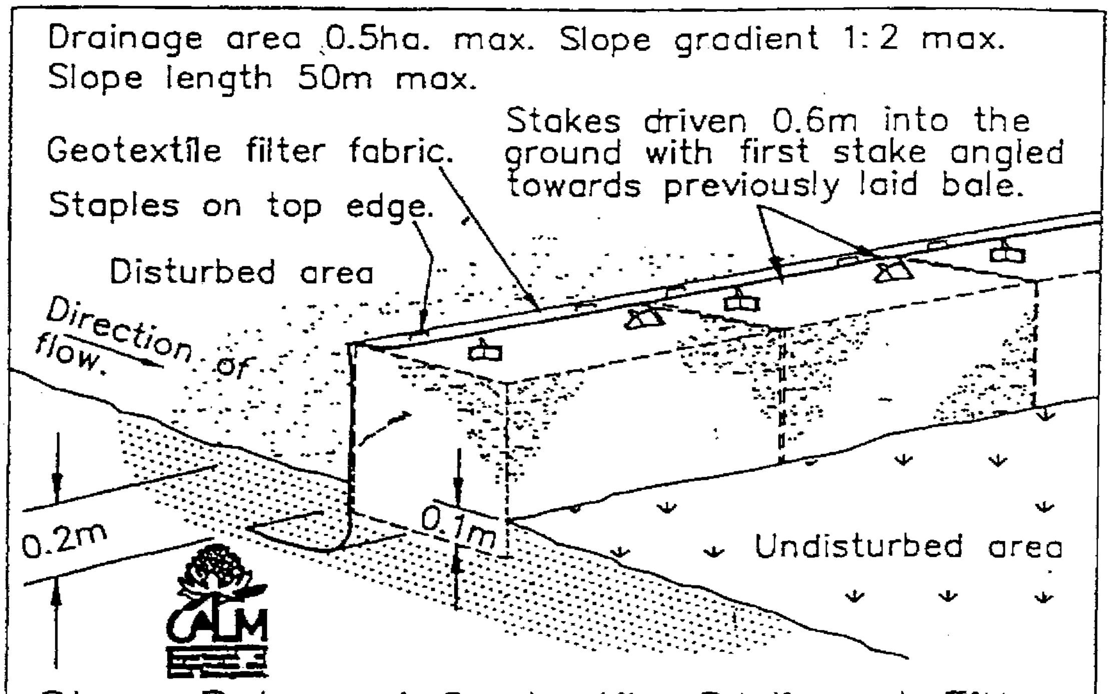
The National Capital Development Commission, (1988) has developed the following design criteria for determining minimum filter strip lengths for various slope gradients.

TABLE 3.7
VEGETATIVE FILTER STRIP LENGTH

Slope (%)	Minimum Filter Strip Length* (m)
2	15
4	20
6	30
8	40
10	50

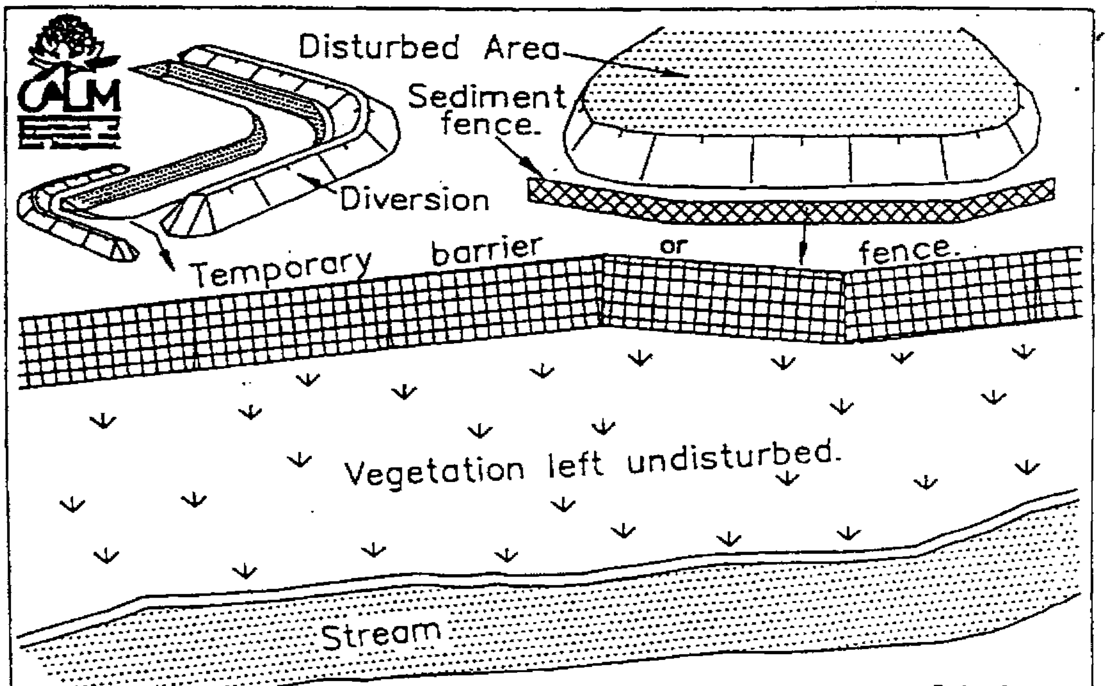
* measured down the slope

In areas where adequate vegetative filter strips do not exist, grasses may need to be planted to provide the necessary protection. The use of such planted strips can reduce the size of other sediment control measures that would otherwise be needed. Protective fencing is advisable.



Straw Bale and Geotextile Sediment Filter

Fig. 3.23



Vegetative Filter Strip.

Fig. 3.24

3.4.4 PORTABLE SEDIMENT TANK

Sediment laden water can be filtered through a portable sediment tank containing several compartments to trap and retain sediments, and which can allow chemical dosing.

Portable sediment tanks can be used on sites where excavations are deep and space is limited. This includes urban building sites, where the direct discharge of sediment laden water to stream and stormwater drainage systems is to be avoided.

The sediment tank should be located for ease of cleaning and the disposal of the trapped sediment, and to minimise interference with construction activities and pedestrian traffic.

The following formula (from National Capital Development Commission, 1988) should be used to determine the storage volume of the sediment tank :

Tank storage volume (m^3) = 20 x Pump discharge (l/sec)

This provides about 6 hours detention time. Multiple batches should be processed if additional clarification time is necessary. An example of a typical sediment tank is shown in Figure 3.25. Other container designs can be used provided the storage volume is adequate.

The tank can be chemically dosed with gypsum at a rate of 200 mg/l at the inlet to reduce the detention time. The treated water should not be pumped out until clarification occurs. The sediment tank should be completely cleaned out when it is no more than one third full of sediment.

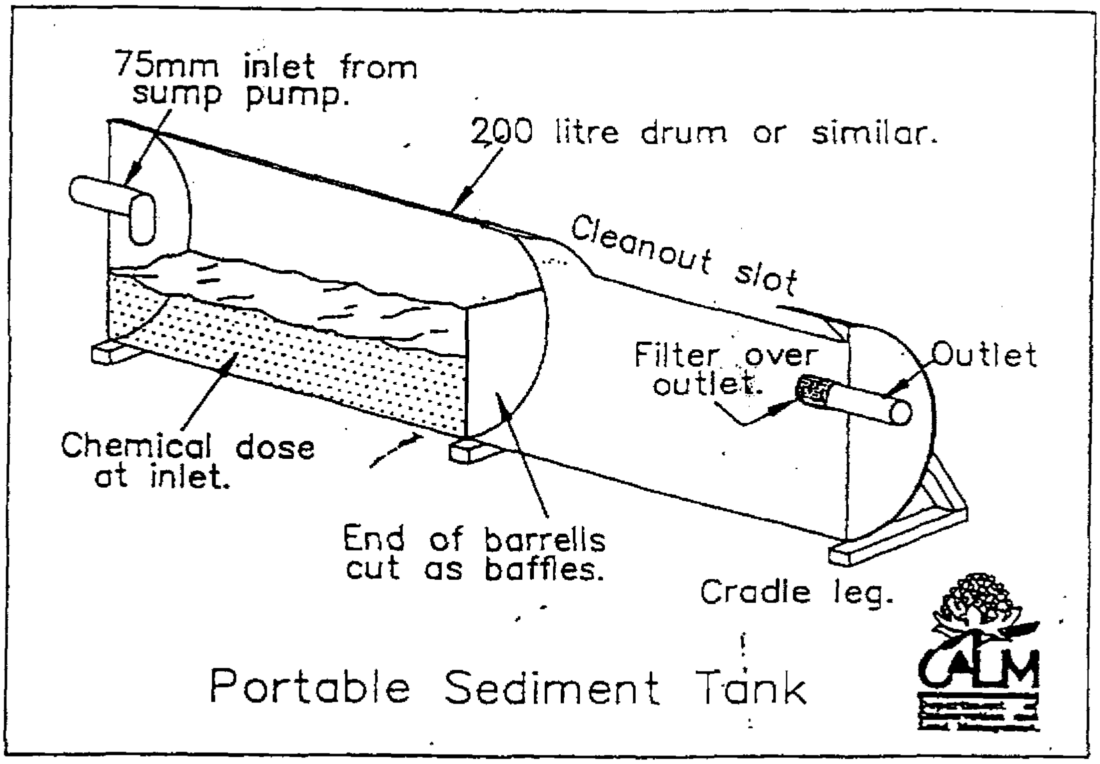


Fig. 3.25

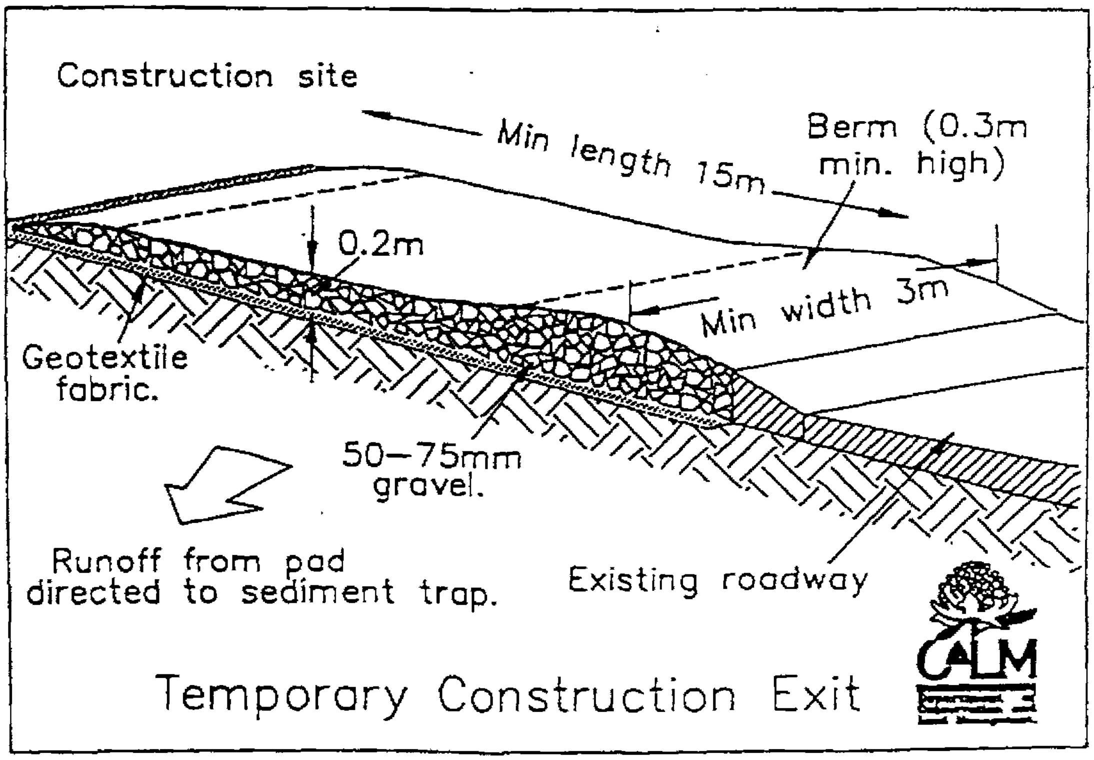


Fig. 3.26

3.4.5. TEMPORARY CONSTRUCTION EXIT

This comprises a pad of coarse gravel, occasionally with a concrete, steel or timber shaker ramp, located at exits from construction sites. It is designed to minimise the transport of sediment from construction sites onto public roads via the wheels and sides of vehicles.

When a site is dry, much of the soil is shaken from vehicles as they traverse this ramp. In wet weather, mud is hosed off on the ramp as vehicles leave the site.

The exit pad is constructed by placing a layer of geotextile filter fabric over the pad site and covering it with a layer of 50 to 75 mm sized gravel to a minimum depth of 0.2m. Its width should be no less than the full width of the exit point, and its length a minimum of 15m. (Figure 3.26)

All drainage from the exit pad should be directed into a sediment trap. A mountable berm (1:5 batters) may be required adjacent to the road footpath area, to prevent drainage directly onto the road.

Additional gravel may have to be added periodically, to maintain the correct functioning of the pad.

3.4.6. DUST CONTROL ON DISTURBED AREAS

Dust control measures should be applied to reduce surface and airborne movement of sediment blown from exposed areas of construction sites. Dust movement may create an unacceptable hazard or nuisance on the site or down-wind.

A variety of methods may be employed to provide temporary or permanent protection.

Vegetative Cover: The retention of existing trees and shrubs to act as a windbreak may afford valuable protection. Guidelines for the establishment of both temporary and permanent vegetative cover are provided in Sections 3.3.6 and 4.5.

Traffic Control: The control of traffic movement over a construction site will aid in dust control.

Tillage: Provided it is done at the correct moisture content, this practice roughens the soil and brings clods to the surface. It is an emergency measure to be used before wind erosion starts. Ploughing with a chisel-type implement, commencing at the windward side of the site, will produce the necessary change in surface conditions.

If the soil is very wet or very dry, tillage should not be carried out as it will damage soil structure and increase its susceptibility to erosion.

Mulches: The use of mulches to protect the soil surface and thereby prevent dust generation, is covered in Section 4.5.

Irrigation: Wetting the site surface is an emergency treatment which can be repeated when needed. Control of sediment laden runoff from over-watering should be closely monitored.

Barriers: Temporary barriers constructed from timber, synthetic fabrics, jute, straw bales, brush or similar materials can be used to control air currents and blowing soil. They should be placed at right angles to the prevailing wind and spaced at intervals equivalent to about 15 times their height.

Brush and jute fences have been used widely on dune areas to effectively reduce wind erosion and to trap moving sand (See Soil Conservation Service of N.S.W. (1990).

Stone: The eroding surface may be covered with crushed stone or gravel.

Chapter 4

EROSION CONTROL: MANAGEMENT OF SOILS

4. Erosion Control: Management of Soils

In this chapter, Section 4.1, Section 4.2 and Section 4.3 apply to all works where more than 250 square metres of land will be disturbed and requires the preparation of either an *ESCP* or a more detailed *SWMP* (Chapter 2). Section 4.4 applies only to sites where 2,500 square metres of land or more will be disturbed and a *SWMP* is required.

4.1 Introduction

The risk of erosion at land development sites is usually proportional to how much soil is exposed to erosive elements through loss of vegetative cover. Vegetation:

(i) binds the soil particles together and reduces the erosive effects of rain splash impact, surface water flow and wind;

(ii) can decrease wind and water velocity on the ground surface;

(iii) decreases the quantity of surface water runoff through an increase in interception water, depression storage and infiltration (figure 4.1);[1] and

(iv) helps retain sediment and nutrients on site.

The combined effect of these qualities of vegetation can result in a reduction in potential erosion to less than 1 percent of that with no vegetative cover (i.e. *C*-factors of less than 0.01 (Appendix A)). Other ground covers and soil surface protective measures can also reduce erosion, but to varying degrees of effectiveness (Section 7.4).

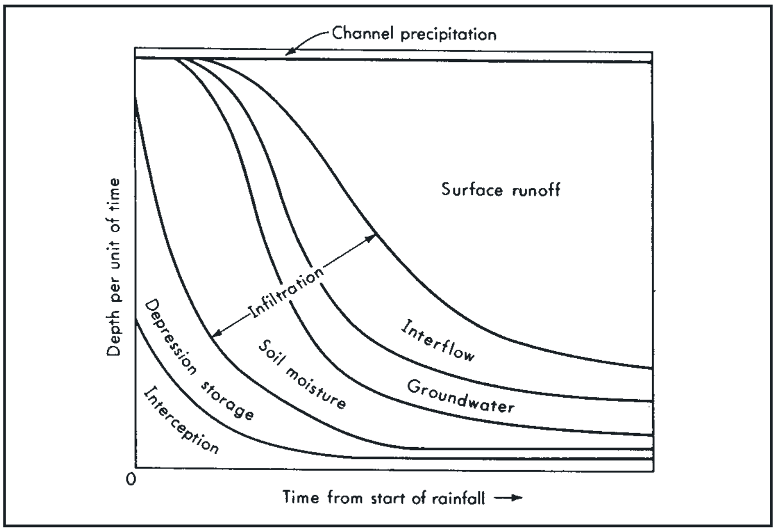


Figure 4.1 Division of storm rainfall into its component parts assuming constant precipitation (Morisawa, 1968)

1. Note:
   - soil structural decline and consequent reduction in soil permeability commonly results from loss of vegetative cover
   - increased infiltration does not necessarily result in rises in the watertable where corresponding increases in evapotranspiration from plant growth occurs
   - to avoid soil salinisation, the watertable needs to be closely monitored for saline content in areas used to increase infiltration or surface runoff.

4.2 Planning Considerations

(a) Where practicable, schedule the construction program so that the time from commencement of land disturbance activities to rehabilitation is less than six months. Restrict land disturbance to areas of workable size. Lands next to waterways should remain undisturbed for as long as possible and at least until the installation of culverts.

(b) Where possible, do not extend land disturbance activities beyond five (preferably two) metres from the edge of any essential construction activity other than in access areas (Table 4.1). These zones of restricted access might require clear identification with barrier mesh, sediment fencing, or other appropriate materials.

Table 4.1 Works limitations

Land use	Limitation	Comments
Construction areas	Disturbance to be no further than 5 (preferably 2) metres from the edge of any essential construction activity as shown on the engineering plans	All site workers should clearly recognise these zones that, where appropriate, are identified with barrier mesh (upslope) and sediment fencing (downslope), or similar materials
Access areas	Limited to a maximum width of 10 metres	The site manager should determine and mark the location of these zones on site. They can vary in position to best converse the existing vegetation and protect downstream areas while being considerate of the needs of efficient works' activities. All site workers should clearly recognise their boundaries which, where appropriate, are marked with barrier mesh, sediment fencing, or similar materials
Remaining lands	Entry prohibited except for essential thinning of plant growth	All site workers clearly recognise this land by marking boundary with barrier fence, etc.

4.3 Handling Soils

4.3.1 General erosion control guidelines

(a) Where possible and where more than 1,000 square metres of land are to be disturbed, ensure that slope lengths do not exceed 80 metres immediately before forecast rainfall or during shutdown periods. Any temporary diversions should outlet to stable discharge areas. On highly erodible lands, constructing earth banks (catch drains) at intervals of less than 80 metres might be necessary to reduce erosion hazards further.

(b) Where necessary, shorten steeper slopes through construction of mid-slope berms (figure 4.2) or other water diversion structures (Section 5.4.4).

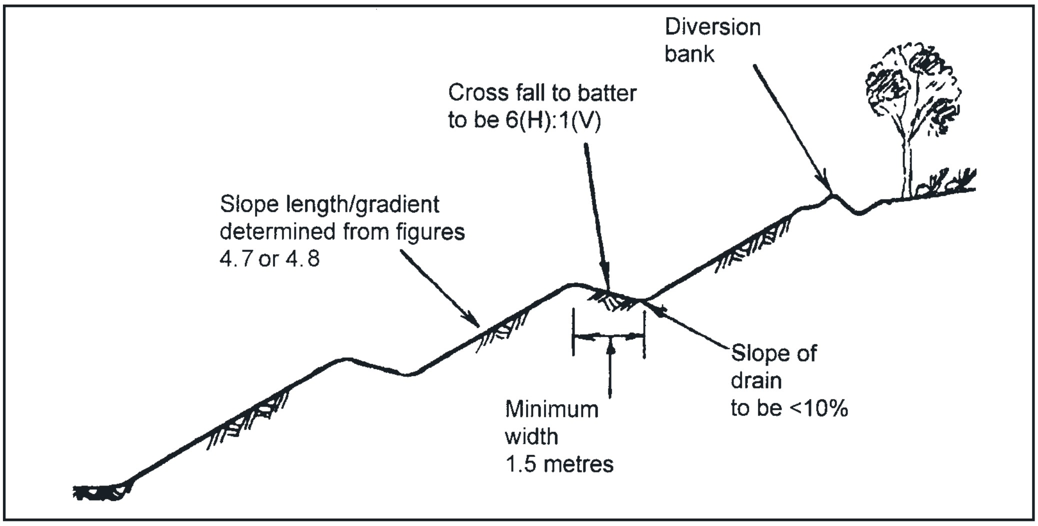


Figure 4.2 Construction of a berm drain.

4.3.2 Topsoil handling procedures

(a) Ideally, handle topsoil only when it is moist (not wet or dry) to avoid decline of soil structure.[2]

(b) Undertake stripping and stockpiling of topsoil immediately before starting bulk earthworks. Before stripping:

  (i) Identify and mark out those areas of vegetation or trees that are to be retained on the site. Commence clearing of trees and shrubs that are within the areas

2. Many types of topsoil are liable to be pulverised if they are too dry when handled and/or pug (set very hard in large clods when dry) if they are too wet when handled.

designated for berm drains, roadworks, interallotment drainage, etc. and stockpile the cut vegetative materials for mulching on site. Separate other debris including fence posts, wire, rocks, etc.; or alternatively

(ii) Slash or graze the site where vegetative growth is dense.[3]

(c) Where necessary, thin plant growth outside the construction zone by hand or rubber-tyred implement. Retain small branches, leaf litter and other residues as mulch.

(d) Take particular care when handling noxious plants that viable parts are not transported offsite by machines.

(e) See Section 5.3.4 regarding the construction of temporary culverts or causeways that might be necessary to cross drainage reserves during the stripping or stockpiling process.

(f) Strip topsoil usually to a depth of 100 to 150 millimetres.

(g) Where maintaining seed viability is desirable, ensure stockpiles of topsoil and leaf litter from remnant native bushland areas are no greater than 2 metres deep and kept weed-free. Structural decline in topsoil is likely in deeper stockpiles.

(h) Ensure stockpiles (Standard Drawing 4-1) are:

(i) constructed on the contour at least 2 (preferably 5) metres from hazard areas, particularly likely areas of concentrated water flows, e.g. waterways, roads, slopes steeper than 10 percent, etc.;

(ii) stabilised if they are to be in place for more than 10 days (Section 7.1.2 (d));

(iii) protected from run-on water by installing water diversion structures upslope (Section 5.4.4); and

(iv) formed with sediment filters placed immediately downslope to protect other lands and waterways from pollution (Section 6.3.7 (e)).

(i) Use topsoil on all lands to be rehabilitated by vegetative means.

(j) Normally, rehabilitate constructed slopes steeper than 2(H):1(V) by non vegetative means such as riprap.

(k) Before spreading topsoil, scarify the ground surface along the line of the contour to break any compacted and/or smooth materials[4] and enable key bonding of the materials to one another.[5] Do not apply topsoil to batters where keying is not possible (Standard Drawing SD 4-2).

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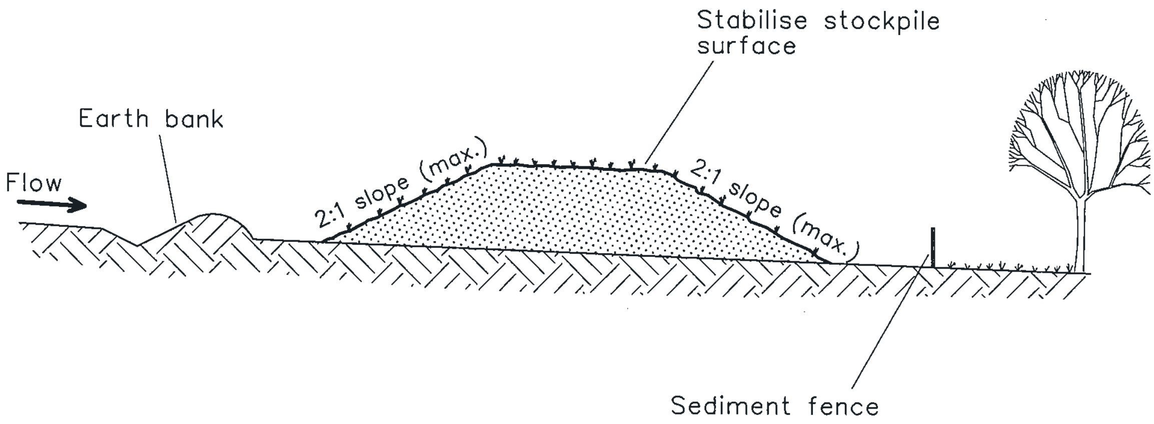
3. Consider stockpiling some or all of any slashed materials for later redistribution to the site as mulch (Section 7.4.1).
4. On constructed slopes less than about 3(H):1(V), scarify with tined implements or excavators to depths of 50 to 100 mm. On gradients steeper than about 3(H):1(V), chain or harrow to break any surface seals and fill any minor rills; alternatively, surfaces can be track-walked (using a crawler tractor driven up and down the slope leaving tread imprints parallel to the contour) (figure 4.3).
5. Keying binds topsoil and substrate layers and, so, mitigates the possibility of sheet erosion and/or creep or slump of topsoil; and enhances water infiltration to the upper subsoil layers, increasing moisture storage within the root zone.

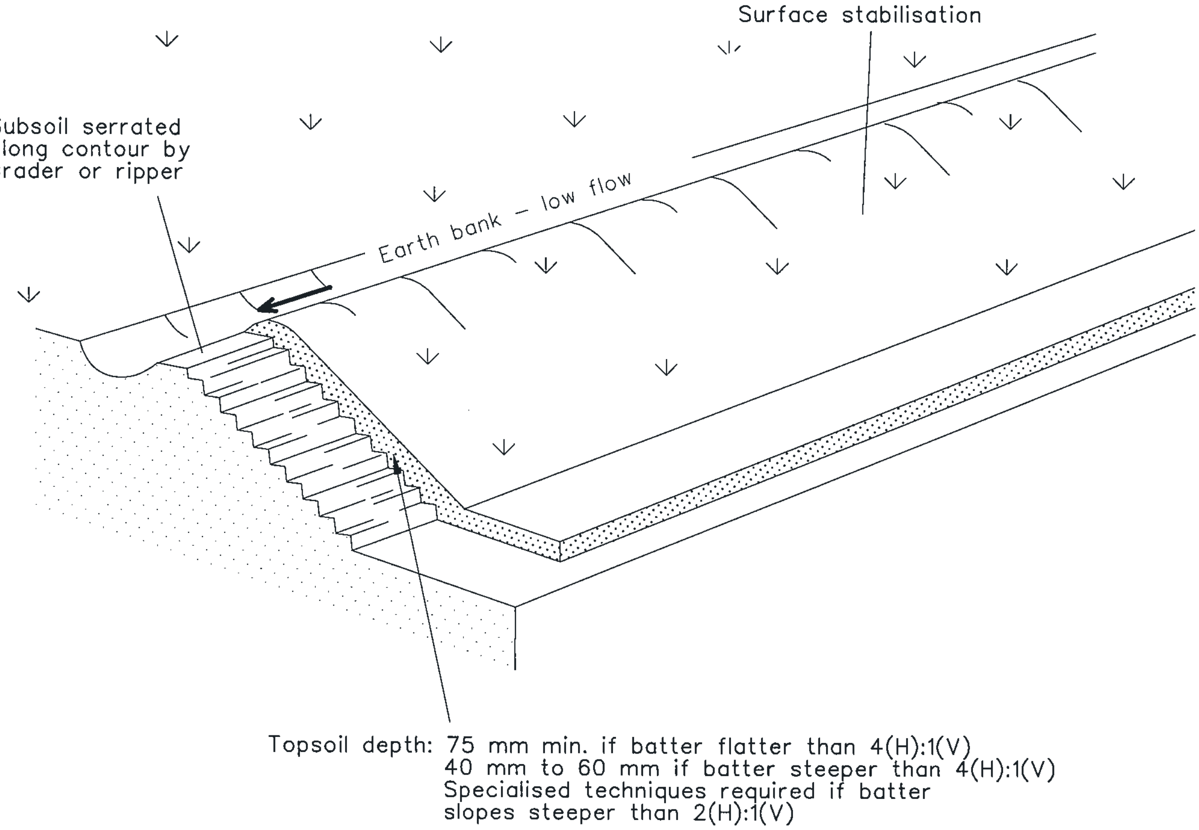


Construction Notes

1. Place stockpiles more than 2 (preferably 5) metres from existing vegetation, concentrated water flow, roads and hazard areas.
2. Construct on the contour as low, flat, elongated mounds.
3. Where there is sufficient area, topsoil stockpiles shall be less than 2 metres in height.
4. Where they are to be in place for more than 10 days, stabilise following the approved ESCP or SWMP to reduce the C-factor to less than 0.10.
5. Construct earth banks (Standard Drawing 5-5) on the upslope side to divert water around stockpiles and sediment fences (Standard Drawing 6-8) 1 to 2 metres downslope.

STOCKPILES

SD 4-1



Construction Notes

1. Scarify the ground surface along the line of the contour to a depth of 50 mm to 100 mm to break up any hardsetting surfaces and to provide a good bond between the respread material and subsoil.

2. Add soil ameliorants as required by the ESCP or SWMP.

3. Rip to a depth of 300 mm if compacted layers occur.

4. Where possible, replace topsoil to a depth of 40 to 60 mm on lands where the slope exceeds 4(H):1(V) and to at least 75 mm on lower gradients.

REPLACING TOPSOIL SD 4-2

(l) Apply topsoil to a depth of:

(i) about 40 to 60 mm on lands where the slope exceeds 4(H):1(V);[6] and

(ii) at least 75 mm on sites where the slope is less than 4(H):1(V).

(m) On completion of the respreading process, leave disturbed lands with a scarified surface[7] to inhibit soil erosion, encourage water infiltration and help with keying topsoil later (figure 4.3).

(n) Respread mulched vegetative material to provide soil stability on bare areas and particularly on those areas where landscape tree planting or bushland is to be established after works are complete.

(o) Select plant species that are consistent with the existing soil conditions at the site. Where practical, ensure the plants are consistent with any indigenous vegetation.

(p) Follow the stabilisation recommendations in Chapter 7.

Figure 4.3(a)
A track-walked slope
(see footnote 4)

6. Topsoil creep/slump is likely with greater topsoil depths, especially where keying is not satisfactory.
7. The practice (especially on batters) of leaving surfaces in a glazed condition with hard, smooth surfaces is not acceptable for several reasons, e.g. topsoils can slump, infiltration can be retarded, etc.

Figure 4.3(b) A track-walked slope (see footnote 4)

Figure 4.4 Phasing of rehabilitation is clearly evident on this batter

Figure 4.5 Recently rehabilitated lands at an urban subdivision near Sydney. Note the barrier mesh and sediment fencing still in position.

4.4 Special Considerations for *SWMPs*

4.4.1 Assessment of Erosion Hazard

(a) A simple procedure is provided to identify those sites of low erosion hazard, where the normal suite of erosion control measures, defined in earlier sections of this Chapter, is considered adequate.

(b) The potential erosion hazard associated with a specific site can be simply determined from figure 4.6, based on:

- the R-factor (rainfall erosivity) that relates to your site location, determined from the maps provided in Appendix B; and
- the typical upper slope gradient (measured in percent) of the site landform.

(i) Sites below the A-line on figure 4.6 have low potential erosion hazards and the standard erosion control measures defined in earlier sections of this Chapter are considered adequate. Planners of such sites need not undertake the tasks outlined in the remainder of Section 4.4[8].

(ii) Sites above the A-line have high potential erosion hazards and designers should apply the guidelines in Section 4.4.2, below.

8. Figure 4.6 has been derived assuming a typical maximum K-factor of 0.05, slope length of 80 metres, P-factor of 1.3 and C-factor of 1.0.

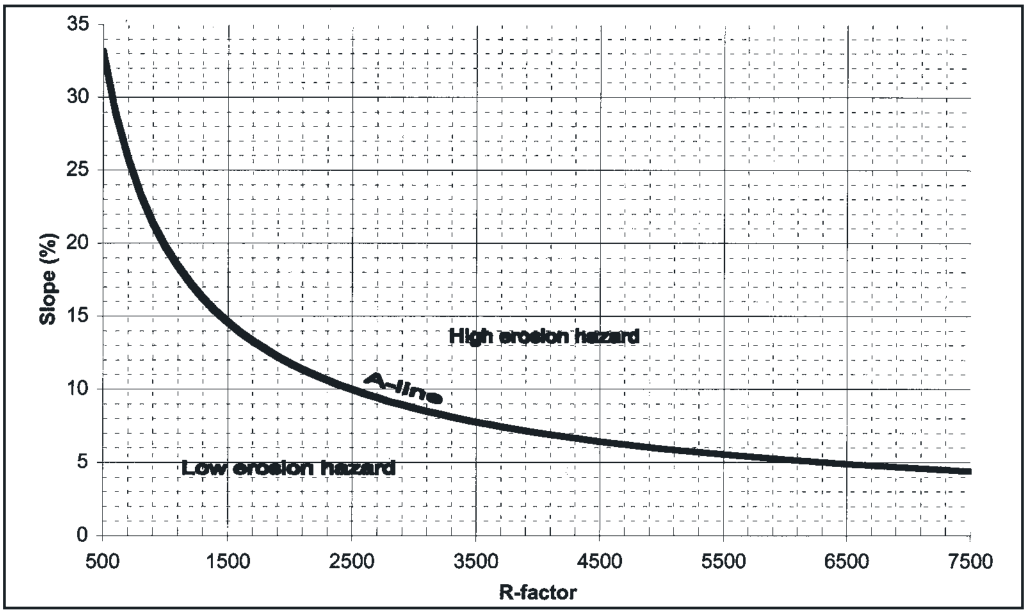


Figure 4.6 Assessment of potential erosion hazard

4.4.2 Management of Sites of High Erosion Hazard

Important Note: If you are not thoroughly familiar with the information on the Revised Universal Soil Loss Equation (RUSLE) presented at Appendix A, read it now.

(a) On constructed slopes ensure that slope length[9] and gradient relationships do not exceed those shown in figures 4.7 and 4.8.[10] Note that rehabilitating steeper slopes (>2.5(H):1(V)) by vegetative means can be difficult, especially where the soils are highly permeable, irrigation is not available and on sites with northerly or westerly aspects. The problem is greatest in the parts of New South Wales that are prone to extended periods where evaporation exceeds rainfall (i.e. nearly all areas except the North Coast and Snowy Mountains regions).

(b) Calculate the erosion hazard on all lands to be disturbed according to the Soil Loss Class (Table 4.2).[11] These classes should be based on local R, K and LS-factors with 80 metre slopes. Typical of most construction areas, also assume P-factors of

9. Here, slope length includes any batters and all upslope lands to a subcatchment boundary, either natural (e.g. crests) or built (e.g. roads or catch drains).
10. The graphs in figures 4.7 and 4.8 are based on LS = 750/1.3(R x K).
11. The Soil Loss Class has been used traditionally to compare apples with apples in relation to soil erosion hazard. Increasing the R-factor in the calculation of the Soil Loss Class by, say, factors of 1.4 and 2.3 where the receiving waters are highly or extremely sensitive, respectively, might be desirable. Highly and extremely sensitive receiving waters might include those requiring significant or comprehensive protection in Healthy Rivers Commission (2002), or those mapped as Classes P and S in SPCC (1980). Any increases should be considerate of the R-factor AEP (Appendix A), where the 20 percent AEP is suggested for highly sensitive waters and the 5 percent AEP for extremely sensitive waters.

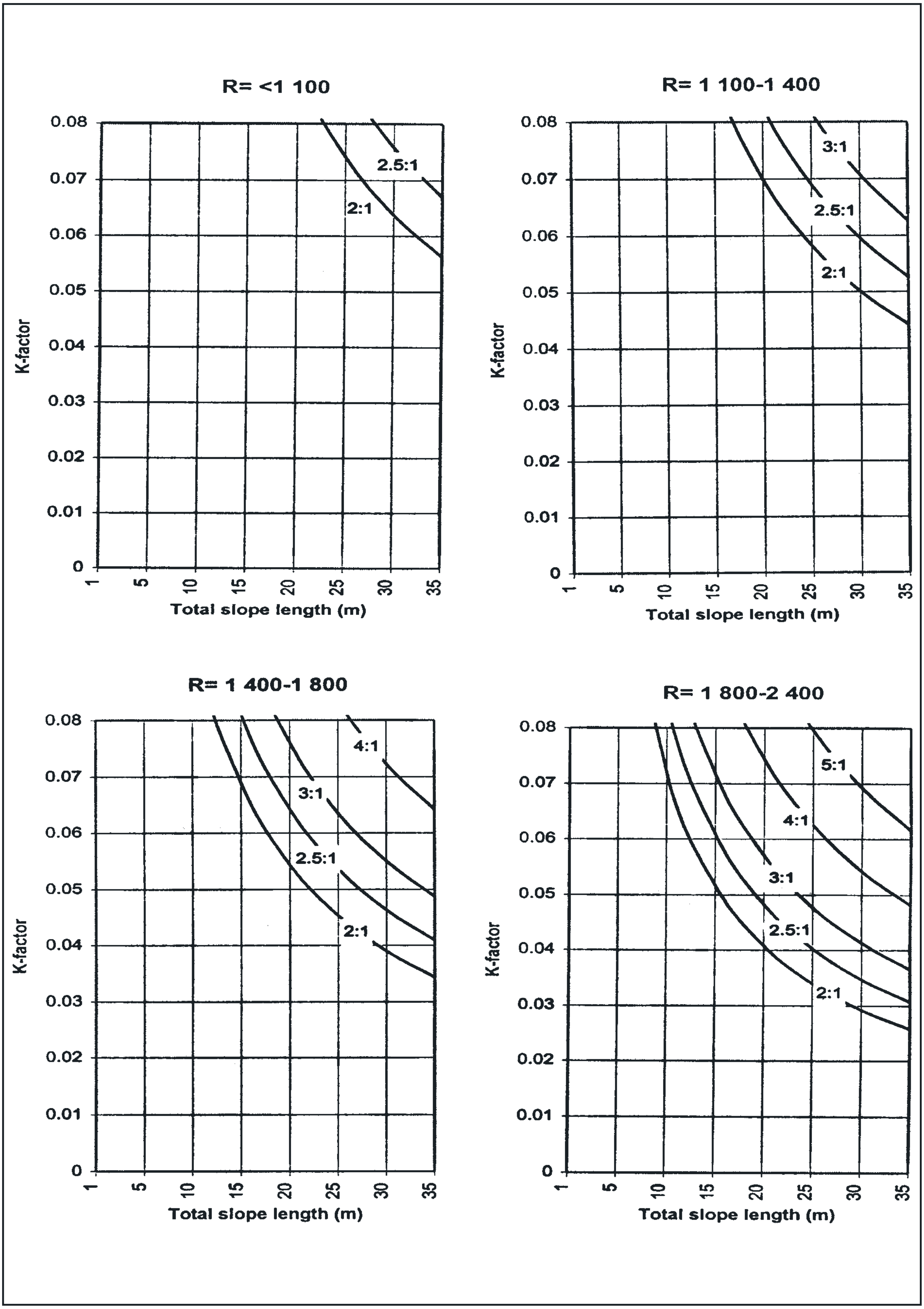


Figure 4.7 Maximum batter gradient (H:V) where the R-factor is 1,100, 1,400, 1,800 and 2,400 (adapted from Morse and Rosewell, 1993) (Appendix A)

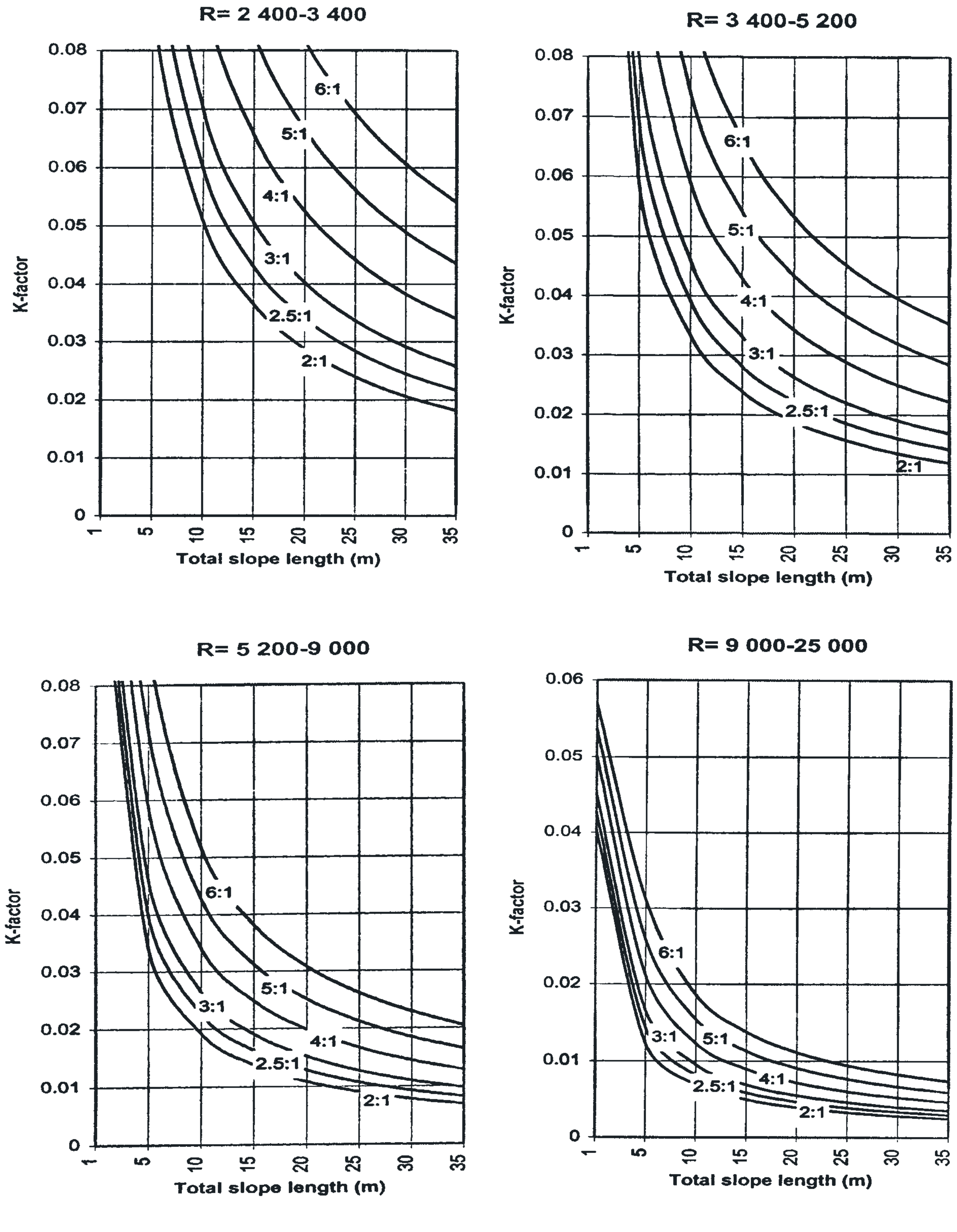


Figure 4.8 Maximum batter gradient (H:V) where the R-factor is 3,400, 5,200, 9,000 and 25,000 (adapted from Morse and Rosewell, 1993) (Appendix A)

Table 4.2 The Soil Loss Classes (adapted from Morse and Rosewell, 1996)

Soil Loss Class	Calculated soil loss (tonnes/ha/yr)	Erosion hazard
1	0 to 150	very low
2	151 to 225	low
3	226 to 350	low-moderate
4	351 to 500	moderate
5	501 to 750	high
6	751 to 1,500	very high
7	>1,500	extremely high

1.3 (i.e. the soils are hard and compact) and *C*-factors of 1.0 (i.e. no vegetative cover, probably being removed with a scraper). Planners can apply different slope lengths, P or *C*-factors if these are properly justified. Application of shorter slope lengths, for example, can put a site into a lower Soil Loss Class. However, the management of these variations should be clearly explained in any plans for erosion control (Chapter 2).

(c) Having identified the applicable Soil Loss Class, ideally schedule activities on highly sensitive lands to periods when rainfall erosivity is low. Highly sensitive lands occur:

(i) Always on Soil Loss Class 7 lands; and

(ii) At certain times of the year:

- on Soil Loss Classes 5 or 6 lands in all rainfall zones (figure 4.8)
- on Soil Loss Class 4 lands in Rainfall Zones 5 and 11.

Here, waterfront lands (Appendix I) should be regarded as Soil Loss Class 6 always.

Table 4.3 identifies those times of the year that do not contribute significantly to the rainfall erosivity (Appendix A) for different rainfall zones (figure 4.9). It shows those lands where land disturbance activities can be undertaken only with the application of special measures (marked "yes") and those where special measures are not required (marked "no"). Of course, this assumes that the regular suite of BMPs is installed as outlined elsewhere in these guidelines).[12]

(d) Where scheduling activities on highly sensitive lands to periods when rainfall erosivity is low is not possible or is impractical, ideally ensure that any disturbed lands have *C*-factors higher than 0.1 only when the 3-day forecast suggests that rain is unlikely. In this case, management regimes should be established that facilitate stabilisation within 24 hours should the forecast prove incorrect.[13]

(e) The kinds of other special erosion control measures that might be invoked in any erosion control plans are usually site specific and beyond the scope of these guidelines.

12. Efforts should be made to keep the calculated soil loss less than 50 tonnes per hectare in any one half month, i.e. the product of percentage average annual *EI* for any particular half month (Rosewell and Turner, 1992) and calculated average annual soil loss should be less than 50 tonnes. So, Table 4.4 shows that:
    - Special measures are required on Soil Loss Class 3 lands when more than 14 percent of the average annual *EI* normally occurs in a half month
    - Special measures are required on Soil Loss Class 4 lands when more than 10 percent of the average annual *EI* normally occurs in a half month
    - Special measures are required on Soil Loss Class 5 lands when more than 6 percent of the average annual *EI* normally occurs in a half month
    - Special measures are required on Soil Loss Class 6 lands when more than 3 percent of the average annual *EI* normally occurs in a half month
    - Special measures are always required on Soil Loss Class 7 lands.
13. *C*-factors of 0.1 can be achieved in various ways as shown at Appendix A, note especially figure A5, Table A3 and Table A4. For example, figure A5 shows that a *C*-factor of 0.1 can be achieved with a 60 percent grass cover where, previously, the soils were stripped or deeply cultivated; alternately, Table A3 shows it can be achieved temporarily by application of a hydraulic soil stabiliser.

Table 4.3 Lands where special erosion control measures apply

Zone	Soil Loss Class	Jan		Feb		Mar		Apr		May		Jun		Jul		Aug		Sep		Oct		Nov		Dec	
1	1-4	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	No	No	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	1-4	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes
	6	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes	Yes
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	1-4	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	No	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	1-4	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	No	No	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	1-3	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	4	No	No	No	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	No	No	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	6	No	No	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	1-4	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes
	6	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
7	1-4	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes
	6	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes	Yes	Yes
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
8	1-4	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes
	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes	Yes	Yes
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
9	1-4	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	6	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes	Yes	Yes
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
10	1-4	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes	Yes	Yes
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
11	1-3	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	4	No	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	6	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes	Yes
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
12	1-4	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No
	5	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes
	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No	No	No	No	Yes	Yes	Yes	Yes	Yes
	7	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

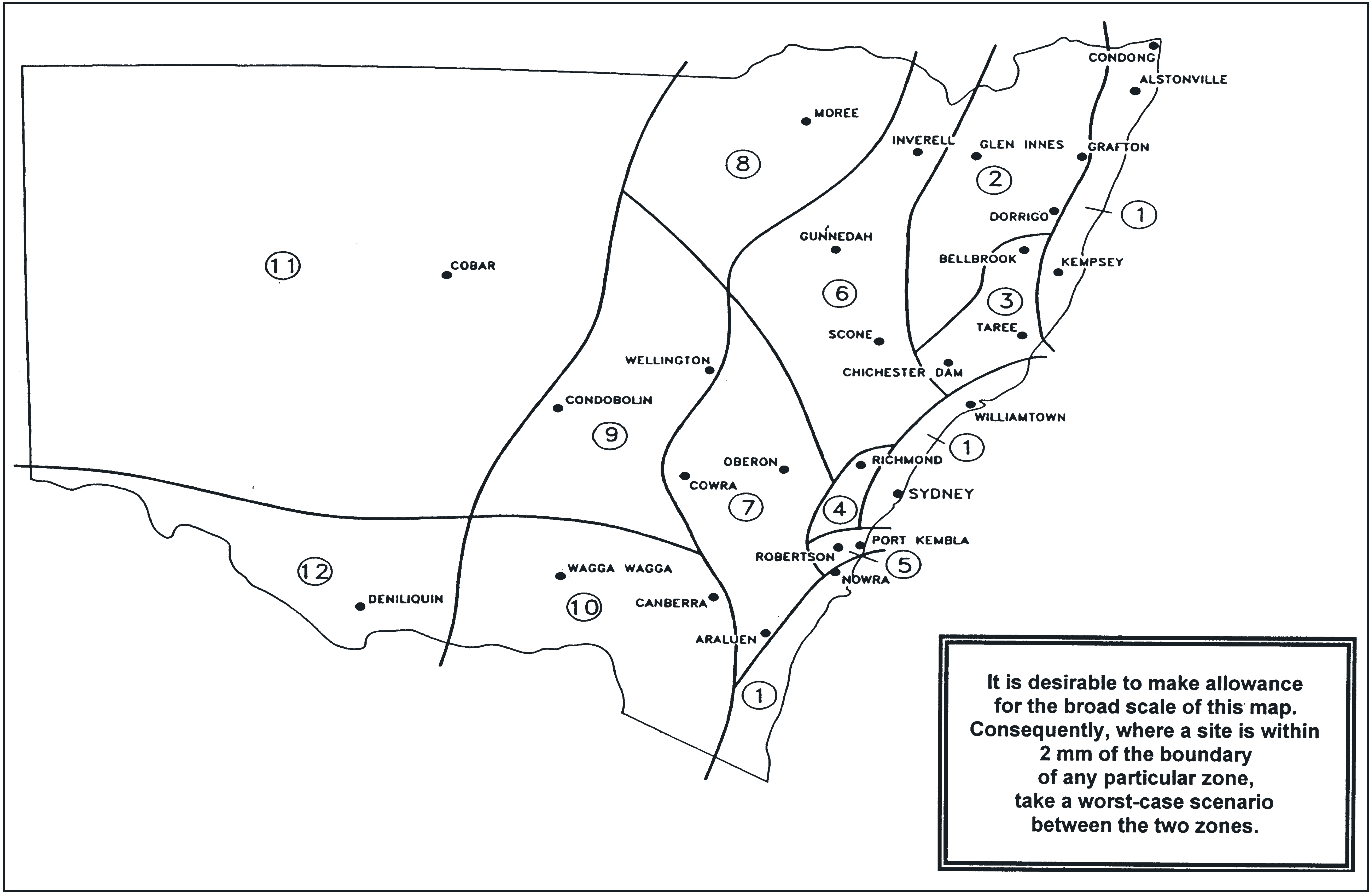


Figure 4.9 Rainfall distribution zones in New South Wales (Rosewell and Turner, 1992)

Chapter 5

EROSION CONTROL: MANAGEMENT OF WATER

5. Erosion Control: Management of Water

This chapter discusses erosion control through management of surface water flows. Being the pre-eminent consideration, works in and within core riparian zone are discussed first. Then, issues relating to sheet water flows and concentrated water flows outside the trunk drainage are addressed.

5.1 Introduction

(a) This chapter considers the selection, design and operation of water control structures that might be built during a development or land disturbance stage, including permanent drainage systems. Additional aspects are found in other texts (Chapter 10), including:
- *Managing Urban Stormwater: Council Handbook* (DEC, *in prep.*)
- *Managing Urban Stormwater: Treatment Techniques* (DEC – 2nd edition, *in prep.*)
- *Managing Urban Stormwater: Urban Design* (DEC, *in prep.*)
- *Constructed Wetlands Manual* (DLWC, 1988)
- *Australian Rainfall and Runoff* (AR&R) (Pilgrim, 1998)
- *Australian Rainfall Quality (draft)* (IEA, 2003).

(b) The process of land development often results in substantial modification to both the hydrological and topographical characteristics of a catchment. Note that:

(i) The hydrological characteristics usually undergo most change and result from:
- an increase in impervious surfaces, e.g., roofs and paved areas
- concentration of water through artificial drainage works
- alteration to the natural drainage pattern; and

(ii) The effect of these hydrological changes varies, but unless suitable control measures are in place, usually results in:
- a reduction in time of concentration
- a reduction in the duration of floods
- an increase in erosion hazards
- a considerable increase in the magnitude and frequency of peak flows.

(c) Erosion hazards and consequent risks of sediment pollution usually reach their highest levels during the land disturbance phase and the effects of soil and water management on disturbed sites are critical.

(d) Naturally, all works must comply with the various Acts (Appendix K), including:
- Water Act, 1912
- Soil Conservation Act, 1938
- Rivers and Foreshores Improvement Act (1948) (note Part 3A)
- Dam Safety Act, 1978
- Fisheries Management Act, 1994
- Protection of the Environment Operations Act, 1997
- Water Management Act, 2000.

5.2 Classification of Waterbodies Based on Objectives for Riparian Land

Riparian lands form the transition between terrestrial and aquatic environments.[1] As riparian environments are very diverse, defining a standard width for riparian lands is difficult. Nevertheless, three broad categories for riparian land are identified by the Department of Infrastructure, Planning and Natural Resources to reflect the relative importance of watercourses. Different management regimes apply to each of these categories (Table 5.1):

(a) Category 1 – environmental corridor[2]

Maximise the protection of terrestrial and aquatic habitats to:

- provide a continuous corridor for the movement of flora and fauna
- provide extensive habitats (and connectivity between habitat nodes) for terrestrial and aquatic fauna
- maintain the viability of native riparian vegetation
- manage edge effects at the riparian/urban interface
- provide bank stability
- protect water quality.

This is achieved by:

(i) where applicable, providing a continuous riparian corridor that links stands of remnant vegetation;

(ii) providing a "core riparian zone" (CRZ) with a minimum width of 40 metre from the top of the bank;

(iii) providing sufficient (additional) riparian corridor width based on geomorphological and environmental considerations;

(iv) providing a suitable environmental protection zoning to the riparian land that recognises its environmental significance;

(v) as far as practicable, restoring/rehabilitating the riparian zone by returning the vegetation, geomorphic structure, hydrology and water quality to the original (pre European) condition;

(vi) ensuring vegetation in the CRZ is at a density that would occur naturally (but see Section 5.3);

1. Riparian lands are those lands immediately next to and along or around waterbodies. They act as buffers and/or filters between the waterbodies and lands nearby. Also, they:
   – help with maintenance or improvement of water quality through stabilising banks
   – are important wildlife corridors that maintain biodiversity
   – improve aesthetic values, e.g. open space and the visual break up of lands.
2. Also note the importance of corridor connections between adjoining watercourses.

Table 5.1 Summary of Riparian Management Objectives

Minimum environmental objectives for riparian land	Category 1: Environmental corridor	Category 2: Terrestrial and aquatic habitat	Category 3: Bank stability and water quality
Delineate riparian zone on a map and zone appropriately for environmental protection	Yes	Yes	Not required
Provide a minimum core riparian zone width	40m from top of bank	20m from top of bank	No minimum – pipes last resort
Provide additional width to counter edge effects on the urban interface	10m	10m	Generally not required
Provide continuity for movement of terrestrial and aquatic habitat	Yes (including piered crossings)	Yes	Where appropriate
Rehabilitate/reestablish local provenance native vegetation	Yes	Yes	Where appropriate
Locate services outside the core riparian zone wherever possible	Yes	Yes	
Locate playing fields and recreational activities outside core riparian zone	Yes	Yes	
Treat stormwater runoff before discharge into riparian zone or the watercourse	Yes	Yes	Yes

(vii) placing services (power, water, sewerage, and water quality treatment ponds) outside the CRZ. Encroachment into the non core riparian area may be possible if the impact on riparian functions is minimal and integrity is maintained;

(viii) providing a suitable interface between the riparian area and urban development (roads, playing fields, open space) to minimise edge effects;

(ix) minimising the number of road crossings;

(x) maintain riparian connectivity by using peered crossings in preference to pipes or culverts

(xi) minimise the impact of walkways, cycleways and general access points by using ecologically informed design principles;

(xii) locating flood compatible activities (playing fields) outside the CRZ. (Encroachment into the riparian area may be possible if the impact on riparian functions is minimal and integrity maintained); and

(xiii) treating stormwater runoff before discharge into the riparian zone of the watercourse.

(b) Category 2 – Terrestrial and Aquatic Habitat

Maintain/restore as much as possible the natural functions of a stream to:

- maintain the viability of native riparian vegetation
- provide suitable habitat for terrestrial and aquatic fauna
- provide bank stability
- protect water quality.

This is achieved by:

(i) providing a CRZ with a minimum width of 20 metre from the top of the bank;

(ii) wherever possible, providing sufficient (additional) riparian corridor width based on geomorphological and environmental considerations;

(iii) as far as practicable, restoring/rehabilitating the riparian zone by returning the vegetation, geomorphic structure, hydrology and water quality of the original (pre European) condition;

(iv) ensuring vegetation in the CRZ is at a density that would occur naturally (but see Section 5.3);

(v) whenever possible, providing appropriate zoning that recognizes the environmental significance of the riparian land;

(vi) minimising the number of road crossings;

(vii) ensuring that road crossings are designed to maintain riparian connectivity;

(viii) providing a suitable interface between the riparian area and urban development (roads, playing fields, open space) to minimise edge affects;

(ix) minimising the extent of open parkland beside a stream;

(x) locating services (power, water, and sewerage water quality treatment ponds) outside the CRZ. Encroachment into the riparian area may be possible if the impact on riparian functions is minimised; and

(xi) treating stormwater runoff before discharge into the riparian zone or the watercourse.

(c) Category 3 – Bank Stability and Water Quality

Minimise sedimentation and nutrient transfer to:

- provide bank stability
- protect water quality
- protect native vegetation.

This is achieved by:

(i) where possible, emulating a naturally functioning stream;

(ii) where possible, providing opportunity for vegetated habitat refuges (terrestrial and aquatic);

(iii) using pipes or other engineering devices as a last resort; and

(iv) treating stormwater runoff before discharge into the riparian zone or the watercourse.

5.3 Works Within the CRZ

5.3.1 Introduction

(a) Works within the CRZ should maximise the retention of any existing native vegetation and minimise site disturbance.

(b) Where practical, ensure that constructed grassed batters have gradients no steeper than 6(H):1(V) if below the 2-year ARI flood level and following the recommendations in figures 4.3 and 4.4. Waterways and spillways should be designed following Section 5.4.

(c) Generally, the design for works in or near waterbodies should ensure the retention and enhancement of their natural functions and maintenance of fish passage. Design of permanent works should consider the following nine principles:

(i) *Interdependence* – all catchments should be considered as single functioning units with interchange between aquatic and riparian ecosystems, floodplains and tributaries;

(ii) *Individuality* – drainage designs should recognise that each catchment has individual characteristics with unique qualities;

(iii) *Continuity* – the linear nature of many ecosystems near waterbodies should be maintained as continuous corridors that allow living organisms to move and spread;

(iv) *Variety* – maintain the existing habitat variety (including any living and dead vegetation, and the litter layer, soil and landform) to ensure continued biodiversity;

(v) *Retention of existing habitats* – existing habitats should be retained because the original habitat cannot be redeveloped in the short term;

(vi) *Protection of potential habitat links* – degraded section of linear habitat systems should not be further degraded and, where possible, should be rehabilitated;

(vii) *Adding resilience* – where possible, existing or potential linkages between waterbody systems and habitat areas or sinks (e.g. parks, reserves, forests, etc.) distant from the waterbody should be protected and/or rehabilitated;

(viii) *Use of natural materials* – designs should incorporate indigenous vegetation propagated from seed collected from the local area and natural materials. Hard engineering designs should be applied only in exceptional circumstances; and

(ix) *Multi-disciplined approach* – the design approach should seek input from different professional disciplines to cover a wide range of multi-objective approaches.

(d) To protect and enhance various vegetation and ecological properties:

(i) Retain natural wetlands;

(ii) Where vegetation must be disturbed:

- first, assess the natural habitat and species to help design a reformed environment containing most of the features of a natural ecosystem
- avoid clearing aquatic and semi-aquatic plants
- investigate and incorporate suitable planting techniques for rehabilitation
- do not use invasive species in rehabilitation (e.g. kikuyu)
- include a broad range of endemic vegetation types (aquatic grasses, other groundcovers, shrubs and trees) in the species mix for permanent revegetation
- do not use herbicides where they might pollute the waterbodies; and

(iii) Overall design should include:

- a vegetated "core riparian zone" on the banks or shore according to the category of riparian land (see Section 5.2) or to the flood limit, whichever is greatest
- a diverse and stable environment in and near the waterbody
- an outer buffer zone of grass or appropriate pollution interception strategy (WSUD system, constructed wetland, or other approved system) to protect the waterbody and its riparian zone.

(e) When designing individual catchment configurations:

(i) Retain naturally functioning streams and, if necessary, engineer drainage lines to reflect natural functions, including the maintenance of fish passage;

(ii) Avoid the aggregation of several subcatchments to a common discharge point; and

(iii) Clearly define maintenance requirements before design starts to reduce needs/costs associated with clearing and mowing.

(f) Hay bales should never be used for sediment control where seed from them can wash into waterways or foreshore areas promoting weed growth. Straw bales consisting of crop stubble do not contain seed and do not have this problem. Both straw and hay bales are less durable than many other sediment control products, often considerably so.[3]

(g) Where works are to be undertaken within the 2-year flood level, measures should be incorporated that ensure the *C*-factors are always below 0.05 during possible erosion events. Further, measures to reduce the *C*-factors to this level should remain stable under concentrated water flow conditions where appropriate. Above all, works should not result in or be likely to cause sediment pollution, either directly or indirectly.[4]

(h) Critical aspects of in-stream works should be scheduled for forecasted dry weather periods.

(i) Where works occur in or close to watercourses, the Site Supervisor or someone nominated by him/her, should record in a notebook each day:
- the *C*-factor status at various positions along the watercourse
- publicised weather forecasts.

5.3.2 Protection of Riparian/foreshore/intertidal Areas

(a) Riparian/foreshore/intertidal areas support unique and delicate ecosystems and works here should be undertaken with the least impact possible. A special problem here is caused by unpredictable flood events or high tides that can wash equipment and materials away, polluting waters and, in some instances, causing navigational hazards. The following general management measures should be applied:

(i) Minimise land disturbance activities to those absolutely necessary to complete the works;

(ii) During construction activities, stockpile or store materials away from the 40-metre zone, where practical, and certainly outside the intertidal area;

(iii) Ensure that no damage occurs to watercourse or intertidal rocks and the organisms that live on them by equipment, machinery or any other activity;

3. Foreshore lands are between the high and low watermarks.
4. Works should be undertaken, preferably, in the period when the rainfall erosion index (*EI*) is likely to be low and lands in Soil Loss Classes 5, 6 and 7 can be disturbed (Table 4.2).

(iv) Note and avoid any areas of seagrasses or kelp; and

(v) Store suitable spill control materials in easily accessible locations at all times during construction works.

(b) Generally, the philosophies and techniques for erosion and sediment control on foreshore lands are similar to those applied to any site where erosion can occur (Section 5.3). However, where works are being undertaken close to waterways or intertidal zones, management measures should be designed to adapt to different influences.

(c) Wherever possible, erosion and sediment control measures should be located above high tide marks and the 2-year ARI flood to prevent impacts from concentrated water flows, or tidal or wave action.

(d) Barges are commonly used to get to waterfront sites during construction because of limited landward access. Management measures that relate to use of barges in the foreshore environment for construction purposes include:

(i) Only use barges where, in the context of the overall development, they offer the best environmental outcome;

(ii) In shallow water, only use self-propelled barges at suitably high tides where adequate clearances are available to prevent disturbances to seabeds and prevent damage to subtidal and intertidal rocks, kelp or seagrasses;

(iii) Where barges might collide with seawalls due to wave and wash conditions, use protective measures such as fenders or rubber tyres to prevent damages to the seawall.

(iv) Do not use screw piles in waters where a potential occurs to encounter rock;

(v) Floating sediment curtains should be deployed while materials and/or equipment are being transferred on and off barges to provide secondary containment for any spills; and

(vi) Materials being transported on and off barges must be adequately secured.

5.3.3 Works in Watercourses

(a) Apply the following general hydrological design guidelines:

(i) Maintain the "natural" channel and floodplain form. If a watercourse has been modified already, any channel works proposed should be based on designs that resemble the natural forms of that channel. In modified systems, the advice of a suitably qualified and experienced fluvial geomorphologist should be sought. In some cases, a natural system in a similar catchment nearby can be used as a guide for channel design.

(ii) In emulating the natural form, watercourse design should include floodplains, terraces and other features typical of the natural systems;

(iii) Where possible, runoff characteristics associated with high runoff coefficient land uses should be modified to as near to the natural condition as practicable before entering a watercourse. The aim is to try to mimic the predevelopment flow regimes.

(iv) Detention basins/wetland systems:
- should be designed to handle any increased runoff associated with the development so that no impact on watercourses occurs
- should be built away from:
    - watercourses
    - natural wetlands
    - where possible, remnant natural vegetation;

(v) Drainage discharge points (outlet pipes and spillways) should follow the requirements of Section 5.3.5, below.

(b) Apply the following geomorphic design guidelines to streams:

(i) Construction and maintenance activities should to be designed to avoid erosion of waterways through removal of vegetation or sediment from beds or banks;

(ii) Channel form, shape and cross-section at different sections of the river should mimic the natural stable conditions – channel dimensions should not be enlarged to cater for bigger flood events;

(iii) Design should be based on simulated stream hydrology to allow design of in-stream features, such as pool-riffle sequences, and not just peak flows;[5]

(iv) Watercourses should include characteristics typical of natural watercourses in the area. These might or might not include meanders, wet, low flow channels with pools and riffles, or bars and benches;

(v) The dimensions of watercourse cross-sections and in-stream features can be determined from the existing stream, if it is still present. Where it is not present, the nearest similar stream should be used as a guide;

(vi) Include pools and riffles (figure 5.1), where the length of reaches should be five to seven times the observed bank full width. Pool areas might need to be excavated with the removed sediment being used to build up the bank around the outside of a bend. Riffle areas can be restored with gravel or rock placed within the channel between the pools. The appropriate type of rock or gravel and its placement can be determined by viewing similar riffles upstream and

5. Riffles are shallow areas between pools. They are important for fish passage and aeration due to hydraulic jump (figure 5.1). Usually, pools are within the curves of meander bends while riffles are within straighter sections, but they can simply be depressions in streams. Mostly, pools have fine-grained sediment beds while riffles have rocky or gravely beds. Generally, pool-riffle pairs occur every five to seven channel widths along a stream channel.

Figure 5.1 Pool and riffle zone sequence

downstream, or in similar nearby intact meandering systems. Adjustments might need to be undertaken to the structural works after flood events. Note that meandering channels are never permanently stable and their shape will change in time;

(vii) In relation to riffle zones:

- calculate tractive forces for conditions to determine the minimum rock size for riffles where:
  - the average channel slope and bank full depths occur throughout the reach
  - critical flows are assumed to occur on riffle faces at 20(H):1(V) slopes
- ensure slopes do not exceed the 20(H):1(V) gradient
- ensure crests follow the average slope of the stream reach[6]
- ensure the largest boulders are placed on the channel bottom at the crest of the riffle while the smallest boulders are set aside until the downstream slope is nearly complete; and

6. Riffle crests are elevated pools that extend upstream to the midpoint of the upstream riffle slope. They form stilling basins that improve fish passage and reduce scour.

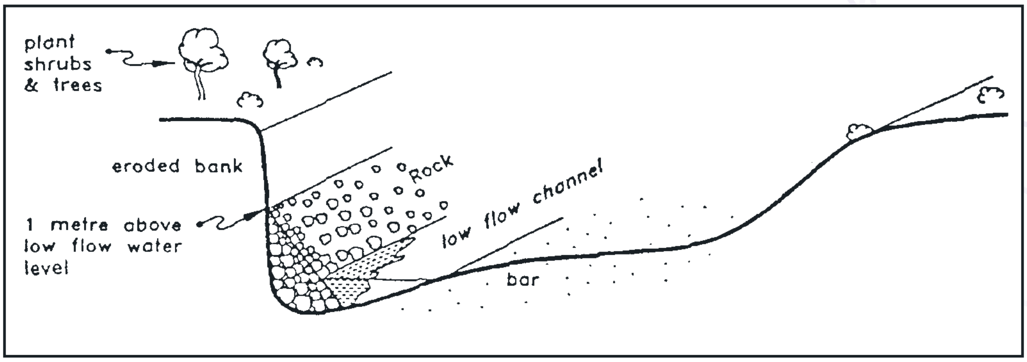


Figure 5.2 Use of rock to stabilise a stream bank (DLWC, 1994)

(viii) The average radius of the curvature of meander bends should be 2.4 times the bank full width.

(c) *Revetment and Vegetation*. Various techniques are available to help stabilise stream-banks. This one, which involves placing rock that cannot be washed away against the base of the bank to hold it in place (figure 5.2), must be designed by an appropriately skilled person. The rock needs to be hard, angular in shape, resistant to weathering and water action, and free from soil and vegetation. The practice is suitable for most situations except very narrow streams where the rock could block a large part of the channel. The procedure for implementing this technique is:

(i) Place the rock directly on the site, taking care to ensure that the range of sizes remains well mixed. If the eroded bank is high, form an access track to get to the base of the bank;

(ii) At 1-metre centres, plant:

- shrubs behind the wall and along the upper part and top of the bank
- trees along the top of the bank starting at a distance back from the bank equal to three times the bank height.

Establish groundcover over all areas at densities of four plants per square metre. Water all plants until they are established.

(d) *Planting the Common Reed, Phragmites australis*. With this technique, a dense growth of Phragmites australis is established along the bank. The reeds slow the flow and trap sediment, minimising scour and undercutting of the bank, while providing improved riparian and aquatic habitats. Groynes might be necessary if active erosion is already occurring. The technique is suitable for most situations where grazing control can be provided. However, it should not be used for very narrow streams (where flow might be obstructed) or where streams are perched above the floodplain and a breakout and course change is possible. *Phragmites australis* is effective because of its ability to grow in 1 to 2 metres of water and up steep banks above normal water level to provide bank protection against variable water levels. This technique is excellent for water quality improvement in tributaries and to build up the floor of gullies. The procedure for implementing this technique is:

(i) If necessary, construct groynes to provide bank stability;

(ii) Collect reeds for planting, either by transplanting them from an existing stand, harvesting seed and growing seedlings, or by taking cuttings in early spring. Plants might be available commercially;

(iii) Planting position and time depends on the bank shape and stream hydrology. Plant where and when moisture is consistent, without long periods of inundation of greater than 300 mm until the seedlings are established (generally one year). Plant at spaces of 300 to 500 mm if sufficient material is available, or in clumps. If bank shape allows, plant several rows; and

(iv) Use slow release fertiliser tablets in planting holes if the nutrient levels of the soils are low.

(e) *Bendway Weirs*. These should be designed by an appropriately experienced person. They are low level, upstream angled, stone sills attached to the outer banks of bends in smaller rivers or watercourses (Derrick, 1996). Typically, the weirs are:

- angled 10 to 25 degrees upstream (into the flow)
- built of well-graded stones with an upper weight limit of 300 to 450 kilograms;
- built in sets spaced 20 to 30 metres apart
- built 0.6 metres high at the stream end, rising to 1.2 metres at the bank end and keyed into the bank with lengths varying from one-quarter to one-half the width of the river at base flow.

They redirect water flowing over the weir at an angle perpendicular to the axis of the weir and break up the stream's strong secondary currents in bends. Consequently, flow is directed away from the outer bank of the bend towards the point bar reducing near bank velocities and erosion. Other benefits include improved aquatic habitats with the overall increase in stream depths, stable scour holes and velocity redistribution (even during low flow). In addition, pool-riffle regimes can be reestablished and any existing vegetation can be stabilised.

(f) *Reconstructing Vegetated Meander Bends in Straightened Channels*. This technique is used to restore the natural meandering streambed pattern where these have been artificially straightened or have eroded into straight channels. Consequently, it requires the reconstruction of meander bends and pool and riffle sequences, the shape and dimensions of which can be determined from aerial photographs, parish maps, portion plans, topographic maps, etc. Construction should begin upstream at the junction of the remaining intact meandering channel and the straight channel and continue downstream. Channel units that could need to be constructed include:

(i) The outside of curved banks of the meander bends. Some form of armouring should protect the toe of the bank until vegetation (grasses and shrubs planted with the structural works) becomes established. These works are placed outside the meander bends from just above the upstream point of inflection, around the bend, to just below the downstream point of inflection;

(ii) The inside of the bends (point bars). When flows start going around the new bends, point bars will naturally start to deposit on the insides of the bends; and

(iii) Pools and riffles.

5.3.4 Temporary Waterway Crossings

(a) Temporary waterway crossings are usually formed using culverts or pipes to carry flow under a raised gravel carriageway that allows vehicles to cross the stabilised waterway safely without causing damage and erosion.

(b) The following standard design criteria (Standard Drawing 5-1) should be followed:

(i) prohibit traffic until the access way is constructed;

(ii) Use a clean, rigid, non polluting aggregate or gravel (100 to 150 mm aggregate);

(iii) Support the gravel on needle-punched geotextile;

(iv) Have a minimum depth of 200 mm of gravel;

(v) Provide a 3-metre wide carriage way and sufficient length of culvert pipe to allow less than a 1(V):3(H) slope on side batters;

(vi) Provide a lower section to act as an emergency spillway in greater than the design storm events (Section 2.3.1 (e)); and

(vii) Ensure that culvert outlets extend beyond the toe of the fill embankments.

(c) The following maintenance issues should be noted:

(i) Keep the pipe culvert clear to avoid bypassing by storm flows less than the design storm event due to blockages from debris or sediment;

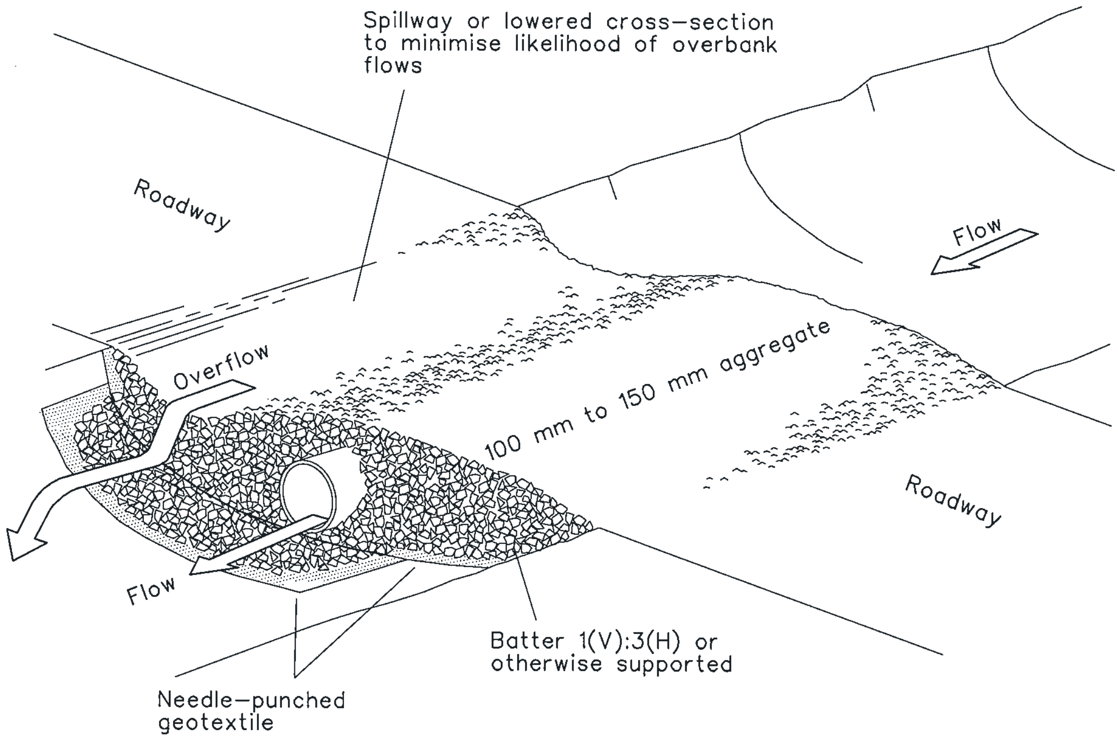
(ii) Recover gravel to maintain minimum depth of 200 mm;

(iii) Remove the crossing when it is no longer required; and

(iv) Rehabilitate the area following the vegetation management plan or other site rehabilitation plan.

(d) In addition, consideration should be given to the following limitations:

(i) Oils or other potentially hazardous materials should not be used as surface treatment;



Construction Notes

1. Prohibit all traffic until the access way is constructed.
2. Strip any topsoil and place a needle-punched textile over the base of the crossing.
3. Place clean, rigid, non polluting aggregate or gravel in the 100 mm to 150 mm size class over the fabric to a minimum depth of 200 mm.
4. Provide a 3-metre wide carriageway with sufficient length of culvert pipe to allow less than a 3(H): 1 (V) slope on side batters.
5. Install a lower section to act as an emergency spillway in greater than design storm events.
6. Ensure that culvert outlets extend beyond the toe of fill embankments.

TEMPORARY WATERWAY CROSSING

SD 5-1

(ii) Upstream flooding problems need to be assessed;

(iii) Downstream flooding in times of overtopping needs to be assessed;

(iv) Activities that might affect water quality should be addressed.

(v) Also, see Section 5.2 regarding special requirements for working closer than 40 metres from waterways.

5.3.5 Construction of Culverts and Bridges

(a) When working in creeks or rivers, careful planning is required to limit the impact of sediment pollution occurring because of works.

(b) Where possible, divert water (by pipe or bank) around culverts and/or bridges during construction so that the entire system is stable at least up to the 2-year ARI storm event. The formation of temporary dams and draining or pumping of water around the site to control polluted waters should be undertaken with all care.

(c) Where culverts intercept table drain flows, construct culvert headwalls with sufficient height and width to ensure flows in the drains do not bypass or overtop the inlet headwall. Culvert outlets should normally extend beyond the toe of fill embankments. Further, outlets should be designed and constructed so that they align evenly with surrounding landforms and do not protrude. Where possible, all outlets to watercourses should be of a natural form, using riprap packed with soil and planted with sedges, rushes and grasses as scour protection. The use of concrete headwalls should be avoided.

(d) Care should be exercised in the construction of sediment basins (Section 6.3.3) on waterways below these works. While sediment control is important, sometimes the effect of disturbance of waterways for basin construction can cause more damage than can be gained by their presence. Innovative techniques are encouraged, such as using floating booms, turbidity curtains and similar devices. Sediment basins should not be constructed in line on a watercourse.

(e) See Section 5.2 regarding special requirements for working closer than 40 metres from waterways.

5.4 Regular Site Drainage Works

5.4.1 General Recommendations

(a) Install site drainage works to convey stormwater safely through and away from the site, particularly those affecting possible erosion of soil and subsequent pollution by sediment and trash. Preferably, prioritise drainage works with the most important control measures installed first.

(b) Where possible, divert run-on water from lands upslope around the site while land disturbance activities are going on. Note Section 5.4.3, below, in relation to concentrating flows and "soft" outlets.[7]

(c) Direct water across the site at non erodible velocities in the design storm event (Section 2.3.1 (e)).[8]

(d) Water management programs should favour:

   (i) division of the site into smaller, more manageable catchments;

   (ii) installation of simple structures constructed from local materials;[9]

   (iii) taking advantage of permanent stormwater facilities that can double as temporary soil or water control measures; and

   (iv) the use of porous zones (e.g. grassed waterways) within the limitations of (e), below, to help natural assimilation of water pollutants and reduce runoff.

(e) Works should not:

- adversely affect upstream or downstream properties
- cause new seepage areas through a rise in the watertable[10]
- eliminate existing seepage areas that affect remnant natural ecosystems
- increase the hazards of dryland salinity within the catchment.

5.4.2 Sheet Flows

(a) Sheet erosion is the removal of a relatively uniform layer of soil from the land surface by rain-splash and/or water runoff. The energy in rainfall is often sufficient to loosen exposed soil particles that can be subsequently transported by water. Where practicable, reduce the sheet erosion hazard by:

   (i) protecting the ground surface with a cover of suitable vegetation or an erosion control product – note the influence of the *C*-factor on the erosion hazard (Appendix A);

   (ii) reducing the volumes of water flows; and

   (iii) reducing the velocities of water flows.

(b) Reduce runoff volumes by leaving scarified soil surfaces (or similar) or installing

7. Do not divert stormwater onto nearby lands without obtaining earlier written approval from the affected landholder.
8. This guideline applies to both sheet flows and where water is concentrated by works, such as catch drains, waterways, drop structures, outlet structures, etc.
9. Soft local rock might not be suitable for permanent structures.
10. Often watertables in eastern Australia are saline. Where these watertables are raised into the root zone, most plant species will die and the erosion hazard rises substantially.

contour banks to keep slopes less than 80 metres in rainfall events – note the influence of the LS-factor on the erosion hazard (Appendix A).

(c) Protect the ground surface by encouraging infiltration of water through progressive revegetation of the site (Chapter 7), installing various erosion control products (Appendix D), cellular confinement systems, etc. Note that:

(i) Under certain conditions, rolled erosion control products (RECPs) can exceed the sheer stress rating of rock rip rap and can be less expensive (Lancaster, *et al.*, 1997). A further benefit of RECPs is that they allow vigorous vegetation growth, offering a "softer", more aesthetically pleasing appearance. However, proper installation of RECPs is critical for their success (Standard Drawing 5-2); and

(ii) Cellular confinement systems are available with or without perforated sides. Perforated sides are preferred because they allow drainage and reduce the hydrostatic loads in the cells (Standard Drawing 5-3).

(d) Reduce water velocities by:

(i) keeping gradients as low as possible (Section 4.4.2 (a);

(ii) ensuring a good ground cover that promotes infiltration; and

(iii) installing banks upslope to divert flows away from the site (Section 5.4.4) and reducing slope length.

5.4.3 Concentrated Flows

(a) Waters are concentrated in drains for interception/diversion of surface or subsurface flows and to convey these to stable outlets, for example:

(i) to intercept offsite run-on water;

(ii) to intercept spring water, especially in areas with moderate or high hazards of landslip;

(iii) to divert water from cut or fill slopes;

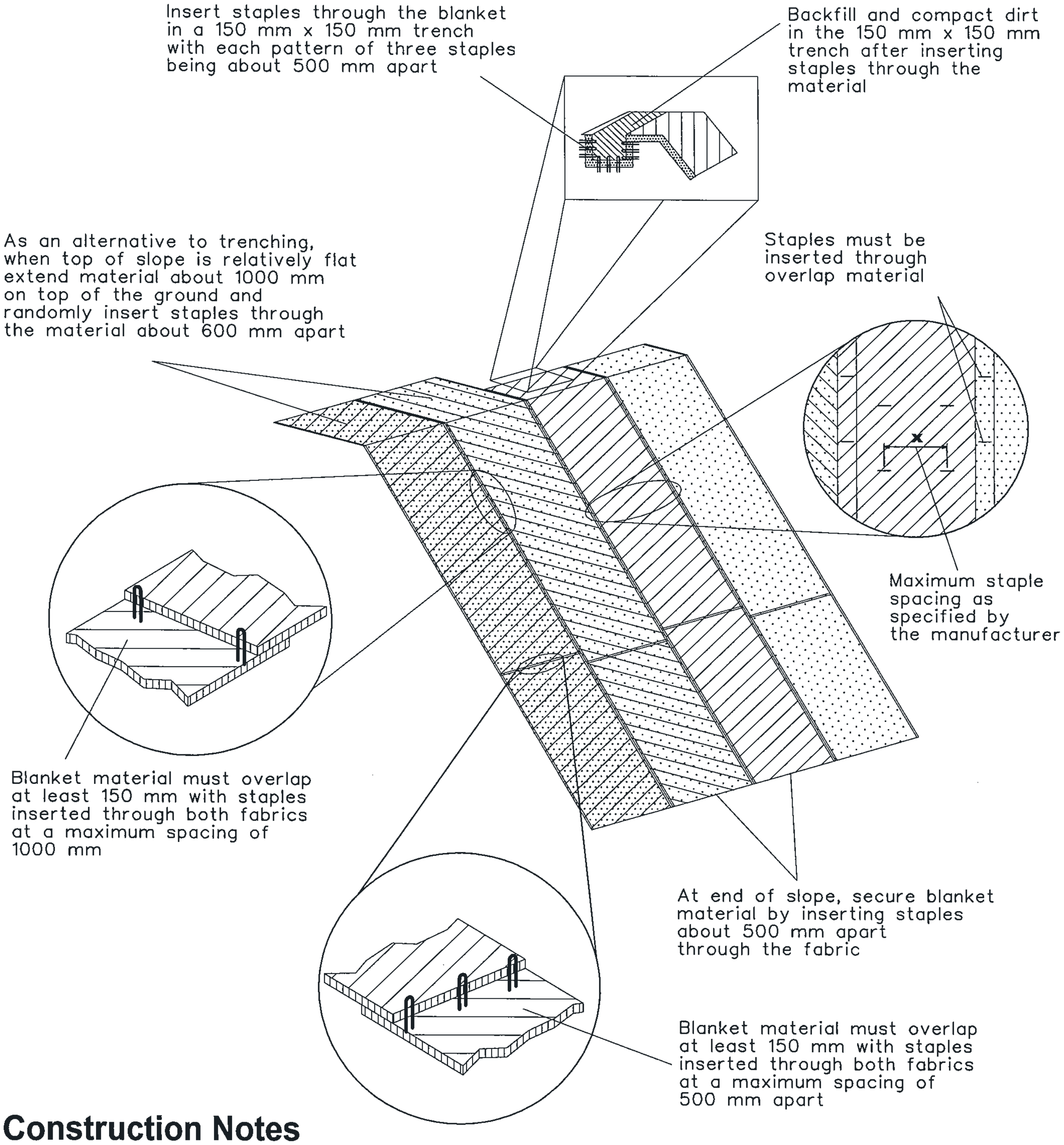
(iv) to shorten long lengths of slope, particularly on lands with high soil erosion hazards, such as earth batters, unpaved roads and newly seeded areas (Section 4.4.2 (a);

(v) to carry water down the face of a cut or fill slope; and

(vi) to provide the general stormwater conveyance system.

Choice of channel/drain type should depend on its purpose, materials available and cost.

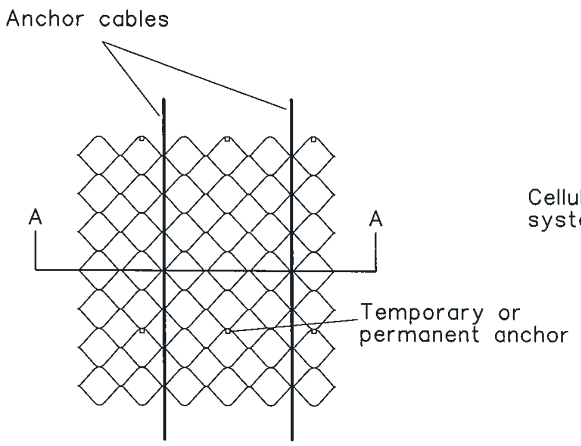
(b) Construct temporary channels/drains and their inlet and outlet works to convey water at least up to the design peak flow and remain stable, usually in the 10-year ARI time of concentration storm event. In some situations, designing it for flows from larger storm events might be necessary, e.g. to prevent damage of environmentally

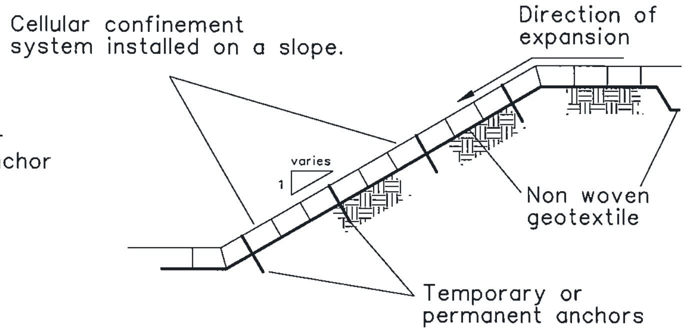


Construction Notes

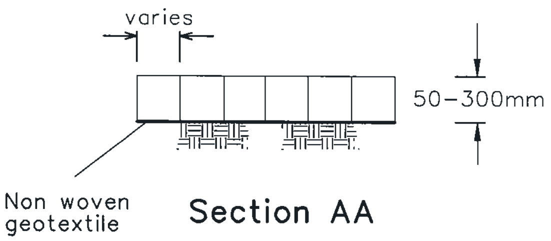
1. Remove any rocks, clods, sticks or grass from the ground surface before laying the matting.
2. Spread topsoil to at least 75 mm depth.
3. Where appropriate, complete fertilising and seeding on a properly prepared seedbed (Standard Drawing 7-1) before laying the matting.
4. Ensure the fabric can be continuously in contact with the soil by grading the surface carefully first.
5. Lay the matting in "shingle-fashion" with the ends of each upstream roll overlapping the next roll downslope.
6. Ensure sufficient staples are used to maintain a good contact between the soil and the matting.

RECP : SHEET FLOW **SD 5-2**

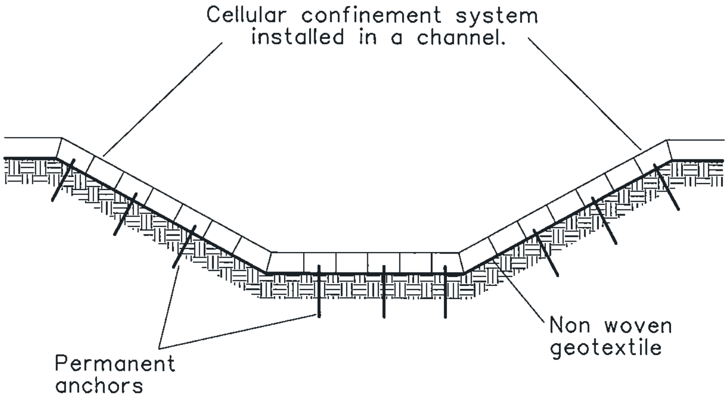




Slope Protection System



Section AA



Channel Protection System

Construction Notes

1. Undertake design only with the help of a suitably qualified geotechnical engineer.
2. Anchor systems on steep slopes to prevent sliding or movement under gravitational forces. This might include the use of high tensile, low creep cables made of polyester (not polypropylene), rope or steel wire.
3. Place thick, non woven geotextiles under the cellular confinement system to allow for lateral drainage.
4. Fill the cells with soil, rock or concrete depending on the application.

CELLULAR CONFINEMENT SYSTEMS **SD 5-3**

sensitive areas and/or where failure is likely to result in substantial loss of property or a danger to life.

(c) Also, design any permanent channels/drains and inlet and outlet works to convey water at least up to the design peak flow and remain stable, usually in the 10-year ARI time of concentration storm event. However, follow council's guideline where different to this.

(d) For all water conveyance structures, adopt the recommendations shown at Appendix F for calculation of peak flow runoff coefficients (C_{10}) where the lands are disturbed by removal of vegetation and topsoil (common on construction sites and mining sites). Where the lands are not so disturbed, apply the criteria shown in Pilgrim (1998).

(e) Reduce the erosive energy levels of concentrated water in constructed channels by:
- constructing channels/drains with a parabolic or trapezoidal cross-section (rather than V-shaped)
- widening the drain invert
- installing check dams (figure 5.3 and Standard Drawing 5-4)
- installing appropriate channel linings (Section 5.4.4 (d))
- installing energy dissipaters at outlets.

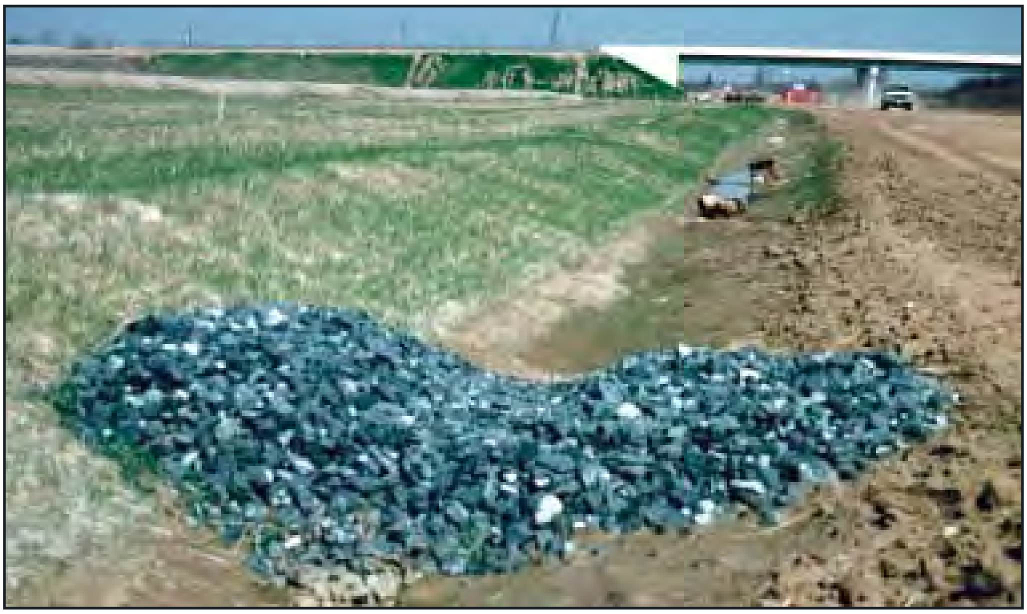


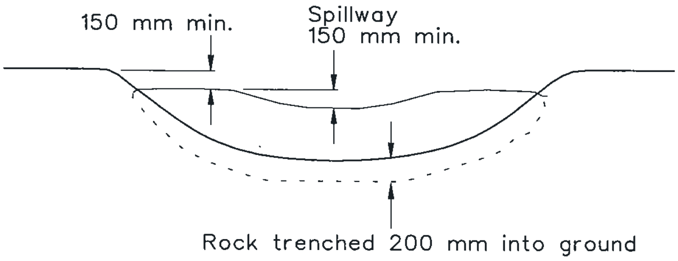
Figure 5.3(a) Use of rock check dams to reduce the erosive energy of flow

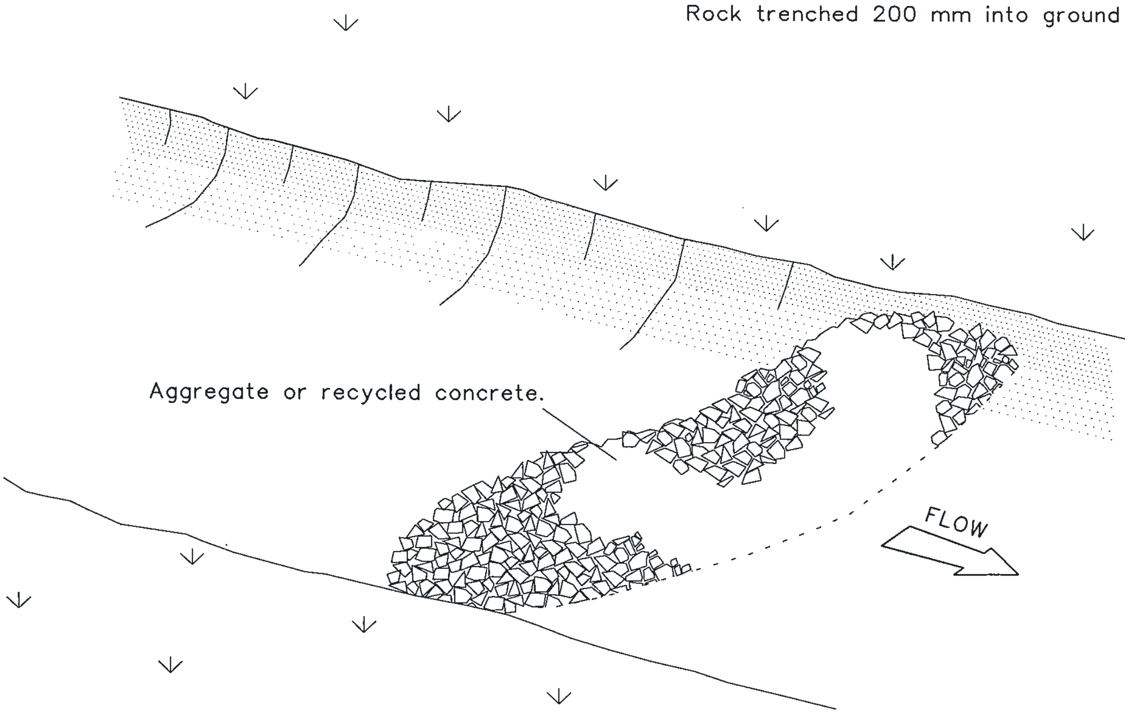
Figure 5.3(b)
Alternate check dam system. Where products like these are used, they should be installed with an appropriate erosion control blanket below them and covering at least 0.7 metres each side (c.f. SD 5-4)

(g) Outlets from all water conveyance structures should discharge water such that the erosion hazard to downslope lands and waterways is no greater than in the predevelopment condition up to the design storm event (Section 2.3.1 (e)). This can be achieved through use of water detention basins, waterways that increase the time of concentration, energy dissipaters, level spreaders, etc. Where permanent outlets discharge to watercourses, works should be of a "soft" design.

(h) Where appropriate, preparation for grassed waterway construction should involve:

- removal of trees, stumps and other debris that might impede flow of water or earthworks
- stripping and stockpiling of topsoil (Section 4.3.2)
- shaping the channel (to eliminate irregularities that would interfere with flow of water and provide a stable cross-section) and respreading topsoil
- applying appropriate ameliorants and/or fertilisers if rehabilitating by vegetative means (Section 7.2)
- applying ground covers.





Construction Notes

1. Check dams can be built with various materials, including rocks, logs, sandbags and straw bales. The maintenance program should ensure their integrity is retained, especially where constructed with straw bales. In the case of bales, this might require their replacement each two to four months.

2. Trench the check dam 200 mm into the ground across its whole width. Where rock is used, fill the trenches to at least 100 mm above the ground surface to reduce the risk of undercutting.

3. Normally, their maximum height should not exceed 600 mm above the gully floor. The centre should act as a spillway, being at least 150 mm lower than the outer edges.

4. Space the dams so the toe of the upstream dam is level with the spillway of the next downstream dam.

ROCK CHECK DAM

SD 5-4

(i) Where practical to do so, ensure that the design of permanent waterways mimics natural ecosystems. Use of rock and gravel, and planting out with local native vegetation can create waterways that are functional both for conveying discharges and as habitats and biological linkages. More information on the design of permanent waterways is contained in *Managing Urban Stormwater: Urban Design* (DEC, *in prep*.).

5.4.4 Temporary Water Diversion Structures

(a) Water diversion structures can be constructed from a wide range of substances, such as compacted earth, plastic sheeting, shotcrete, asphalt and pipes.

(b) As a first step in the design process, ensure that the route to be taken is considerate of all opportunities and constraints present at the site (Chapter 3). Select a route that avoids trees, preferably beyond their drip line, service infrastructure, etc.

(c) Where water diversion structures will outlet to downstream receiving waters (Section 5.3.5), ensure suitable water quality and quantity control structures are installed so that discharges meet the relevant guidelines. Ideally, discharge rates should mimic natural flows in terms of magnitude, seasonality, frequency and variability. Place any water quality and quantity control structures above the riparian zone, such as oil/grease interceptors, sediment traps/basins, litter traps, constructed wetlands and detention basins.

(d) Velocities should not exceed those recommended in Table 5.2 for the design storm event (Section 2.3.1 (e)).

(e) The design of *earth-based diversion structures* should not exceed those recommended in Table 5.1 in the design storm event (Section 2.3.1 (e)). The structures should be built following Standard Drawings 5-5 and 5-6. In addition, the following issues should be considered:

- additional lands that might be disturbed
- structures that need to be stabilised immediately
- downstream flow changes that need to be carefully assessed.

(f) Construct grassed waterways following processes that ensure an adequate root zone (Section 7.3). Note that:

(i) Where concentrated flows cannot be avoided immediately after planting, either initiate the grass lining with turf or where seed is used, protect the surface with a biodegradable mat; and

(ii) Ensure that grass cover is total and permanent, particularly in all areas of possible concentrated flow. Reestablishment might be necessary in bare areas, including replacement of any lost topsoil.

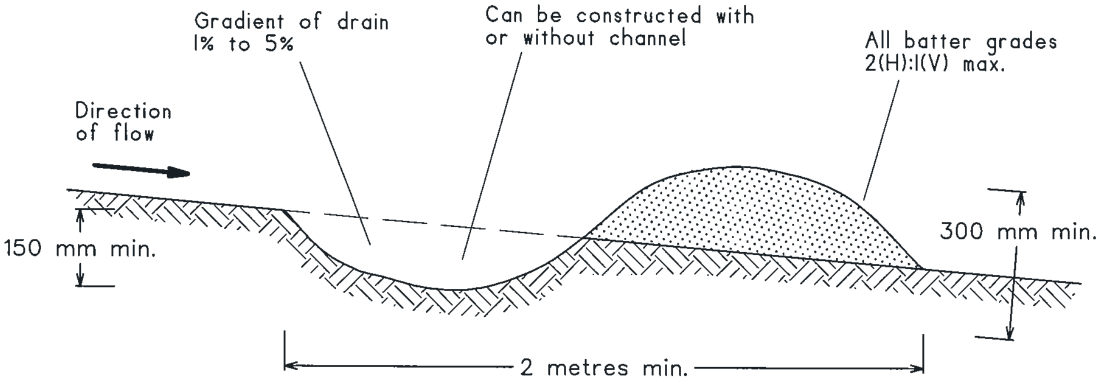
Table 5.2 Maximum Design Flow Velocities in Waterways (compiled from various sources)

Type	Material: Thickness (m)	Material: Aggregate size (mm)	Critical velocity (m/second)	
Gabions and reno mattresses	0.50	120-250	6.4	
	0.50	100-200	5.8	
	0.30	100-150	5.0	
	0.30	70-120	4.2	
	0.25	70-100	3.6	
	0.17	70-100	3.5	
	Weight each (kg)		Turbulent flow	Normal flow
Loose rock (assume 100 percent soil cover)	1,000		4.8	6.6
	500		4.2	5.7
	100		3.3	4.5
	50		2.8	3.8
	10		2.3	3.0
	Form			
Revetment mattresses	Storm mattress		>6.0	
	200 mm fp		6.0	
	125 mm fp		4.0	
	100 mm fp		2.0	

Assume that all soils with 10 percent or more dispersible fines have high erodibilities. Of those with less, soils with *K* -factors below 0.02 have low erodibilities, those between 0.02 and 0.045 have moderate erodibilities, while those above 0.045 have high erodibilities.

In addition, the figures here assume slope gradients of less than 10 percent and, where appropriate, good (>80 percent) ground cover. If good ground cover is not expected to be maintained properly (might die back seasonally or during short periods of drought) and is critical to the system, reduce all velocities by 1.0 metre per second. Alternately, seek the manufacurer's advice if these conditions are unlikely to be met.

Material	Critical velocity (m/second): Inundation <6 hours, Soil erodibility: Low	Moderate	High	Inundation <12 hours, Soil erodibility: Low	Moderate	High	Inundation <24 hours, Soil erodibility: Low	Moderate	High	Inundation <48 hours, Soil erodibility: Low	Moderate	High
High performance bonded plastic fibres (vegetated)	7.0	7.0	7.0	6.0	6.0	6.0	5.0	5.0	5.0	4.0	4.0	4.0
Plastic fibres with netting	5.0	5.0	5.0	4.3	4.3.	4.3	3.6	3.6	3.6	3.0	3.0	3.0
Mesh reinforced pregrown turf	3.0	2.7	2.4	2.6	2.3	2.0	2.3	2.0	1.8	2.0	1.8	1.6
Kiku yu	2.5	2.2	1.9	2.1	1.9	1.6	1.9	1.7	1.4	1.6	1.4	1.2
Jute or coir mesh (close weave, bitumen sprayed)	2.3	2.0	1.7	1.9	1.7	1.5	1.7	1.5	1.3	1.5	1.3	1.1
Coconut/ jute fibre mats	2.3	2.0	1.7	1.9	1.7	1.5	1.7	1.5	1.3	1.5	1.3	1.1
Couch , carpet grass , Rhodes grass , etc.	2.0	1.8	1.4	1.7	1.5	1.2	1.5	1.4	1.1	1.3	1.2	0.9
Bare soil	0.7	0.5	0.3	0.6	0.4	0.3	0.5	0.4	0.2	0.4	0.3	0.2

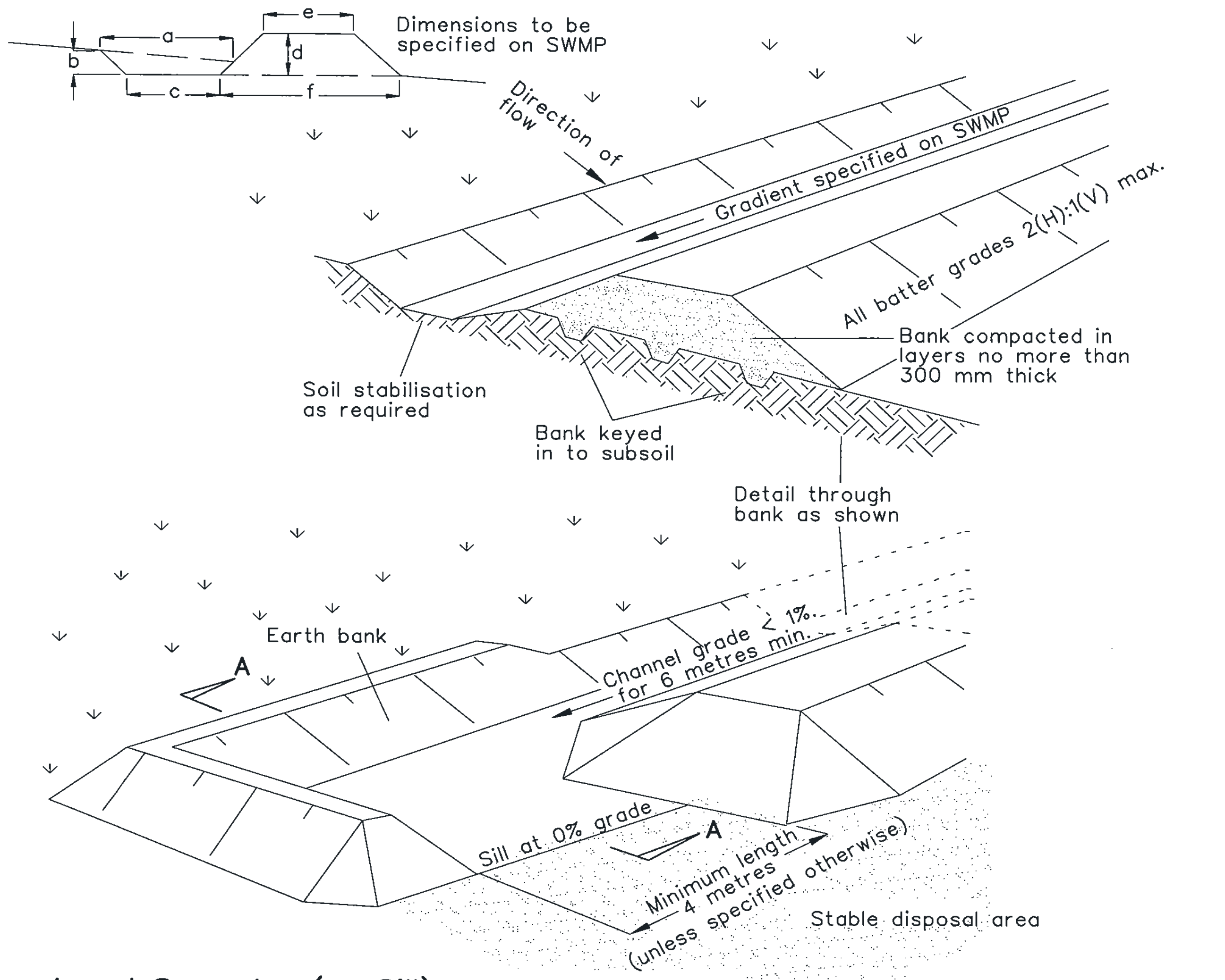


NOTE: Only to be used as temporary bank where maximum upslope length is 80 metres.

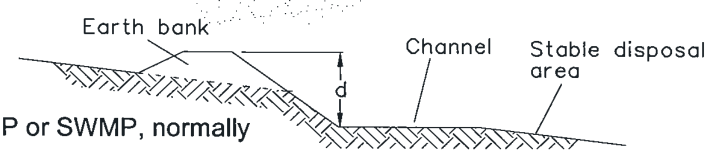
Construction Notes

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

EARTH BANK (LOW FLOW) — SD 5-5



Level Spreader (or Sill)



Section AA

Construction Notes

1. Construct at the gradient specified on the ESCP or SWMP, normally between 1 and 5 percent
2. Avoid removing trees and shrubs if possible - work around them.
3. Ensure the structures are free of projections or other irregularities that could impede water flow.
4. Build the drains with circular, parabolic or trapezoidal cross sections, not V-shaped, at the dimensions shown on the SWMP.
5. Ensure the banks are properly compacted to prevent failure.
6. Complete permanent or temporary stabilisation within 10 days of construction following Table 5.2 in Landcom (2004).
7. Where discharging to erodible lands, ensure they outlet through a properly constructed level spreader.
8. Construct the level spreader at the gradient specified on the ESCP or SWMP, normally less than 1 percent or level.
9. Where possible, ensure they discharge waters onto either stabilised or undisturbed disposal sites within the same subcatchment area from which the water originated. Approval might be required to discharge into other subcatchments.

EARTH BANK (HIGH FLOWS) — SD 5-6

(g) Grassed waterways should not be confused with grass swales. Rather than being simple water conveyance structures, grass swales are used to infiltrate stormwater to the watertable where the soils are Soil Hydrologic Groups A or B and the floor of the structure is at least two metres from any seasonal or permanent watertable. Grass swales should not be used to convey sediment-laden water because their infiltration abilities can be greatly reduced by it.

(h) Consideration might be given to the use of other *fabrics*, including jute mesh and other biodegradable blankets to line waterways. These can provide temporary protection to earth-based drains intended to be removed or upgraded within six months, or grassed waterways that have only recently been established from seed or runners. Nevertheless:

  (i) Ensure topsoil is at least 75 mm deep; and

  (ii) Install the fabric following Standard Drawing 5-7 after grading, seeding and fertilising is completed (Chapter 7) so that it is continuously in contact with the soil.[11]

(i) Gravel or rocks form one of the simplest kinds of linings (figure 5.4) and can be made to withstand most velocities in waterways if the proper sized materials are selected (Table 5.2). Where rocks are used, place them above a filter layer of suitable geotextile and, where necessary, properly graded layers of sand and gravel. Rocks are particularly useful in critical sections of waterways, such as bends and outlets. Gravel layers are useful for protecting any geofabric from tearing as the rocks are juggled into locking positions. Soil should be packed in all layers to:

- enable moisture transfer from the substrate to the waterway itself
- allow sedges and grasses to be planted in the rock voids.

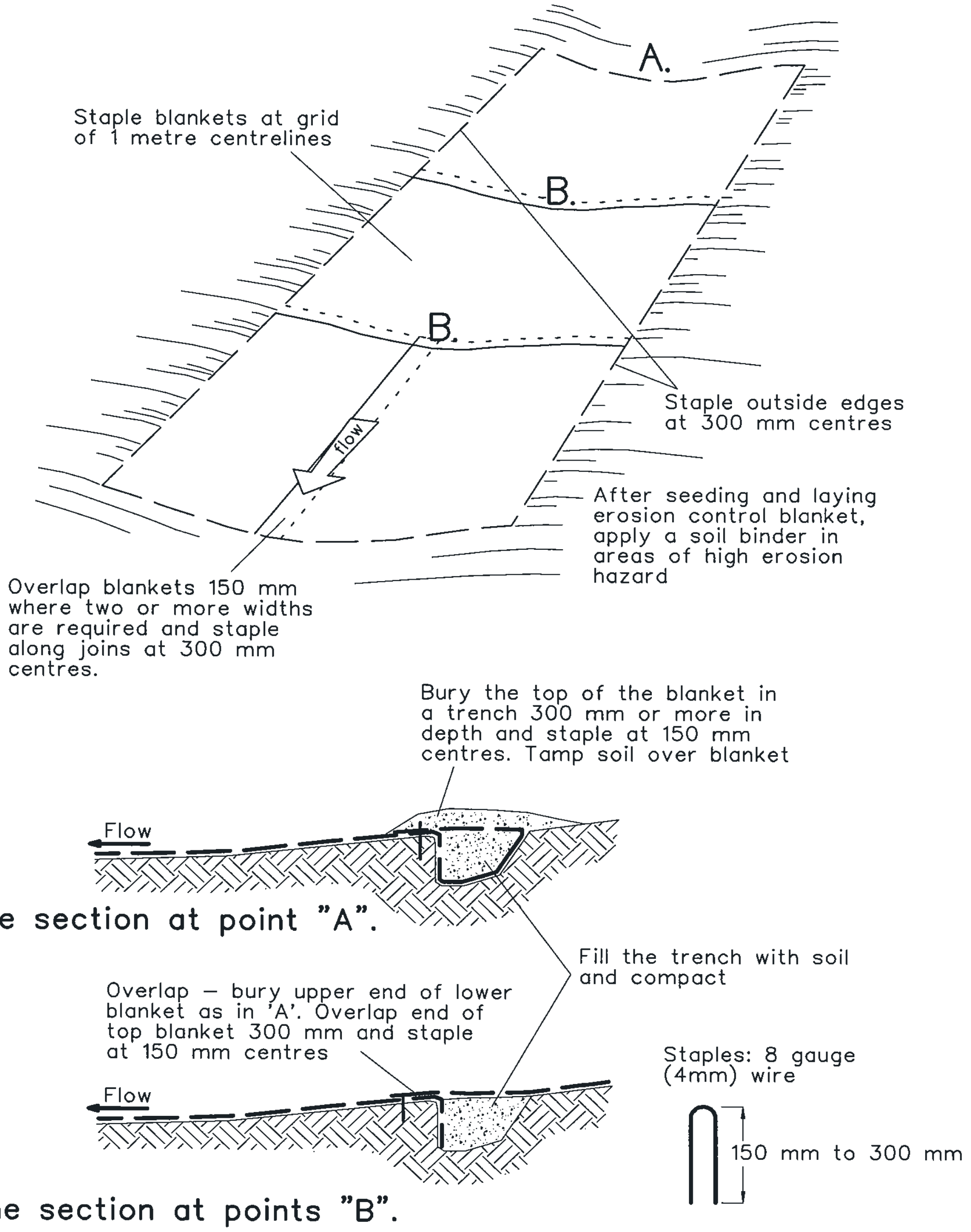
The roots of the vegetation will be protected by rocks and will enhance the locking effect. Also, vegetation will add to the ecological values and impart a more pleasing aesthetic effect.

(j) concrete, concrete-filled mattresses, shotcrete and asphalt can be used to line waterways, form chutes or to convert concentrated flow into sheet flow. Note that:

  (i) These structures should be used only in situations where ecological functions are non existent or have no potential. Even so, these materials should never be used in watercourses.

  (ii) Their impermeable, smooth surfaces usually result in higher velocities in drains and consequent erosion of natural waterways downstream unless protective measures are installed; and

11. In this situation, omit cover crop species from the mix because they have a higher retardance effect and might cause overtopping of the drain in large storm events.



Centreline section at point "A".

Centreline section at points "B".

Construction Notes

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

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

Figure 5.4
A rock-lined waterway

(iii) Concrete linings tend to be more durable than asphalt and require less maintenance.

(k) Chutes (figure 5.5) should have a minimum depth of 300-mm. In addition:

- bends should be avoided
- anchor lugs should be provided at a maximum of 3-metre intervals
- inlet and outlet sections should be at least 1.5 metres long
- energy dissipaters should be provided at the outlet to bring the flow to non erosive velocities.

Concrete chutes should never be used in watercourses. Use riprap instead.

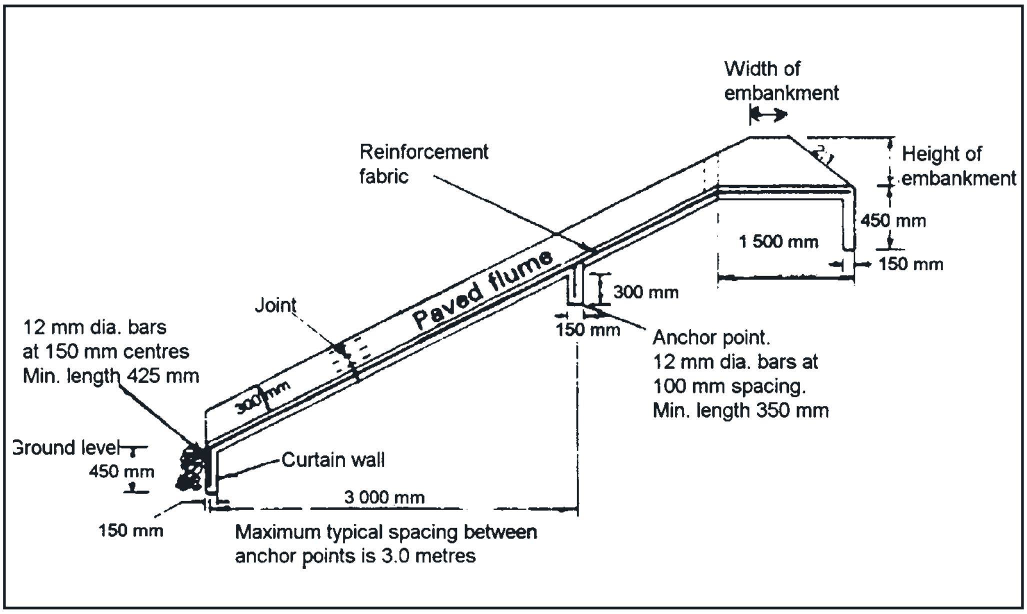


Figure 5.5 Cross-section of a typical paved chute (adapted from Virginia SWCC, 1980)

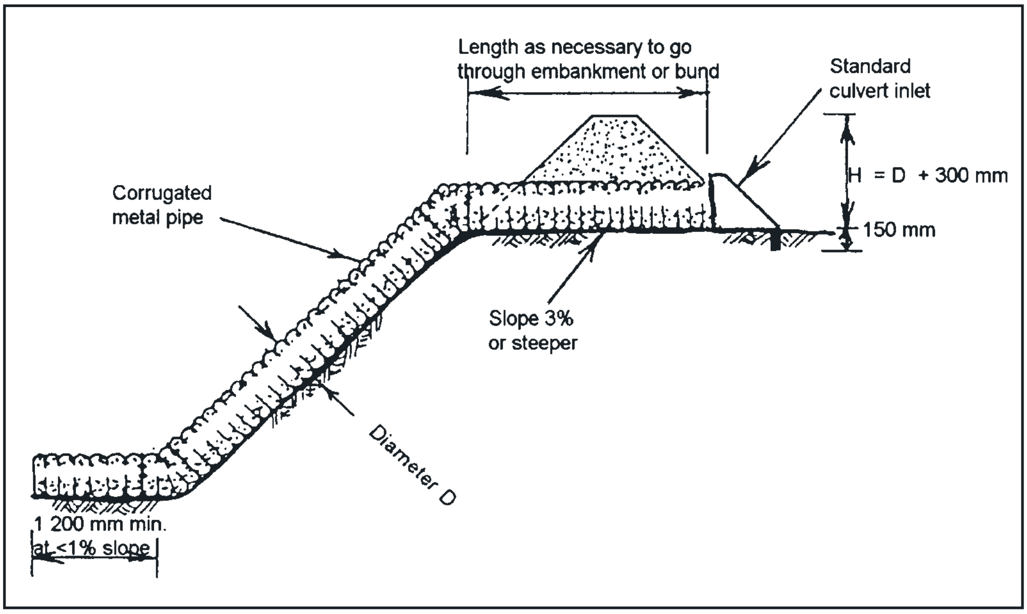


Figure 5.6 Cross-section of a typical pipe drain (adapted from USDA, 1975).

Figure 5.7 Use of heavy-duty flexible pipe to carry water down a batter

(l) Pipes. Half-round corrugated metal or concrete pipes or heavy-duty flexible pipes (figures 5.6 and 5.7) can be used as drains to transport water down steep slopes, e.g. batters. Install them progressively as construction proceeds and their design criteria should include an inlet section with a slope of greater than 3 percent. Ensure any embankment over the inlet of the pipe is a minimum of 300 mm higher than the soffit, and the soil around and under it compacted at least by hand and in half metre layers.

5.4.5 Energy Dissipaters and Outlet Protection

(a) Energy dissipaters can be used to mitigate erosion of drains and their outlets through reductions in water velocity. They are usually permanent structures and constructed with riprap, grouted riprap, gabions, recycled concrete or concrete. Where possible, concrete, grouted riprap, recycled concrete and mesh structures should not be used in watercourses. Rather, they should be constructed of riprap consisting of angular run-of-quarry durable rock and designed to mimic the natural function and appearance of the watercourse.

(b) Drains should discharge water to stable disposal areas including:

Figure 5.8
A riprap outlet on a steep slope.

- well-vegetated or otherwise stable lands (figure 5.8)
- other temporary diversions, drop-down structures[12] or culverts
- watercourses.

(c) Where structures outlet to a watercourse, ensure they:
- do not protrude beyond the stream bank and align evenly with it
- are placed at the invert level of the stream
- point downstream.

Further

(i) stockpile any excavated litter, topsoil and subsoils materials separately for later site rehabilitation;

(ii) if scour is likely, ensure the bed is properly protected (see below);

(iii) protect the opposite bank from scour as might be necessary, in consideration of the bank materials and the "jet" effect; and

(iv) if salinity is an issue, ensure the pipe/culvert specifications conform to Australian standards.

12. Usually formed where the catch drains cross depressions or other waterways.

(d) Sometimes, upstream control works can reduce velocities at stormwater outlets. This can have a positive effect by reducing the sizes of energy dissipaters below.

(e) Design criteria for energy dissipaters (Standard Drawing 5-8) should ensure:
- downslope conditions post development are not less stable than those prevailing predevelopment up to the design storm event (Section 2.3.1 (e))
- minimum maintenance requirements, preferably self-cleaning natures
- drainage by gravity when not in operation.

(f) Energy dissipaters can be categorised into groups where energy is either absorbed by impact through an obstruction in the flow path, and/or dissipated through a hydraulic jump in a large stilling pool. Riprap aprons at the outlet of a pipe or culvert of diameter D, should be at least 300 mm deep, 3D wide at the pipe end, underlain by needle-punched geotextile, and constructed according to the following:

(i) Decide whether minimum or maximum tailwater conditions apply using Q_{peak} in the receiving channel;[13]

(ii) Enter the appropriate chart (figures 5.9 and 5.10) to find riprap size and apron length; and

(iii) Where both minimum and maximum tailwater conditions apply, design the apron to cover both conditions (figure 5.11).

(g) Stabilise lands at and beyond outlets with a protective ground cover (Chapter 7). Within watercourses, ensure all rocks and cobbles in any permanent structures are packed with topsoil and planted with local native grasses, sedges and rushes. In disturbed areas in riparian zones (e.g. spoil areas, access tracks, etc.), plant local native shrubs, trees and groundcover species. Ensure these matters are properly addressed in a separate *Vegetation Management Plan* (Appendix I).

5.4.6 Subsoil Drainage

(a) Subsoil drains provide a means for controlled flow of water through the soil. They must be designed and installed only after any salinity issues have been addressed fully. Types of subsoil drains include:

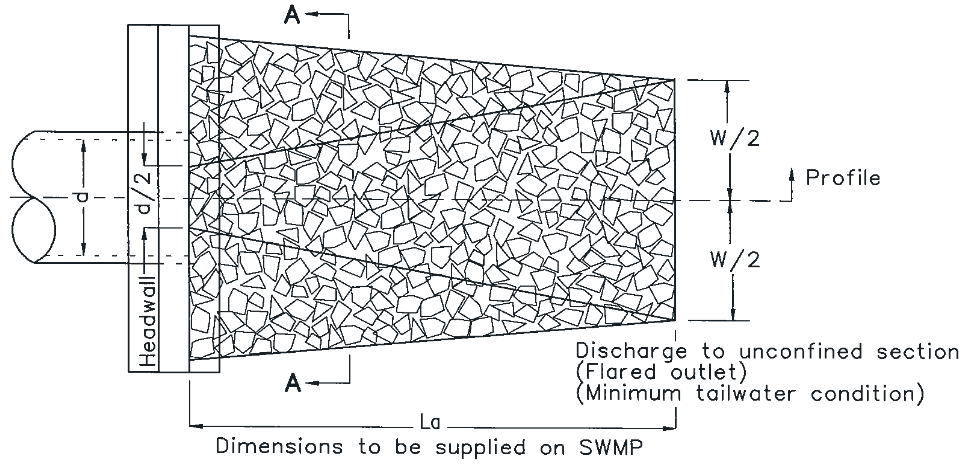
(i) strip drains, comprising geotextile filters over non corroding, rot-proof, plastic cores;

(ii) rubble drains; and

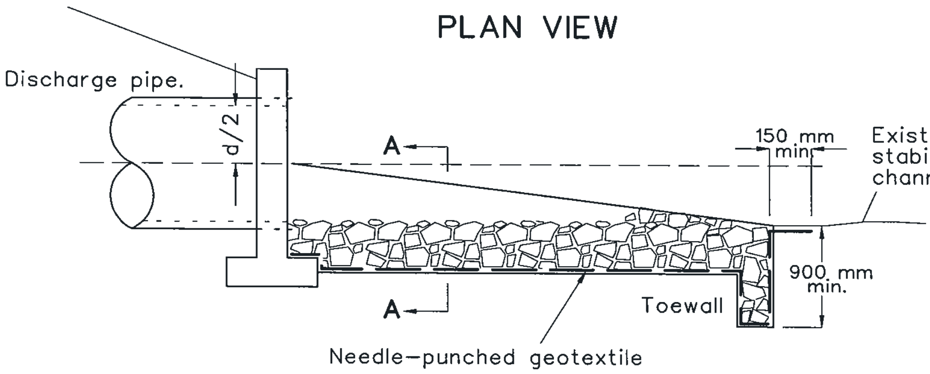
(iii) perforated or slotted pipes.

---

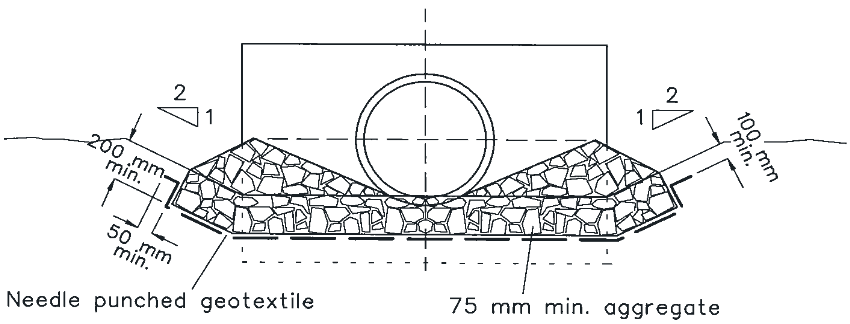
13. Calculate the depth of flow in the receiving channel and compare with the pipe diameter (Do) – minimum tail water conditions are <0.5Do, while maximum conditions are >0.5Do. Note that:
– Where a well-defined channel does not occur, minimum tail water conditions apply and the width of the downstream end of the apron should be equal to the pipe diameter plus the length of the apron
– Where a well-defined channel occurs immediately downstream, the width of the downstream end of the apron should be equal to the width of that channel.



PLAN VIEW



PLAN VIEW



CROSS SECTION AA

Construction Notes

1. Compact the subgrade fill to the density of the surrounding undisturbed material.
2. Prepare a smooth, even foundation for the structure that will ensure that the needle-punched geotextile does not sustain serious damage when covered with rock.
3. Should any minor damage to the geotextile occur, repair it before spreading any aggregate. For repairs, patch one piece of fabric over the damage, making sure that all joints and patches overlap more than 300 mm.
4. Lay rock following the drawing, according to Table 5.2 of Landcom (2004) and with a minimum diameter of 75 mm.
5. Ensure that any concrete or riprap used for the energy dissipater or the outlet protection conforms to the grading limits specified on the SWMP.

ENERGY DISSIPATER **SD 5-8**

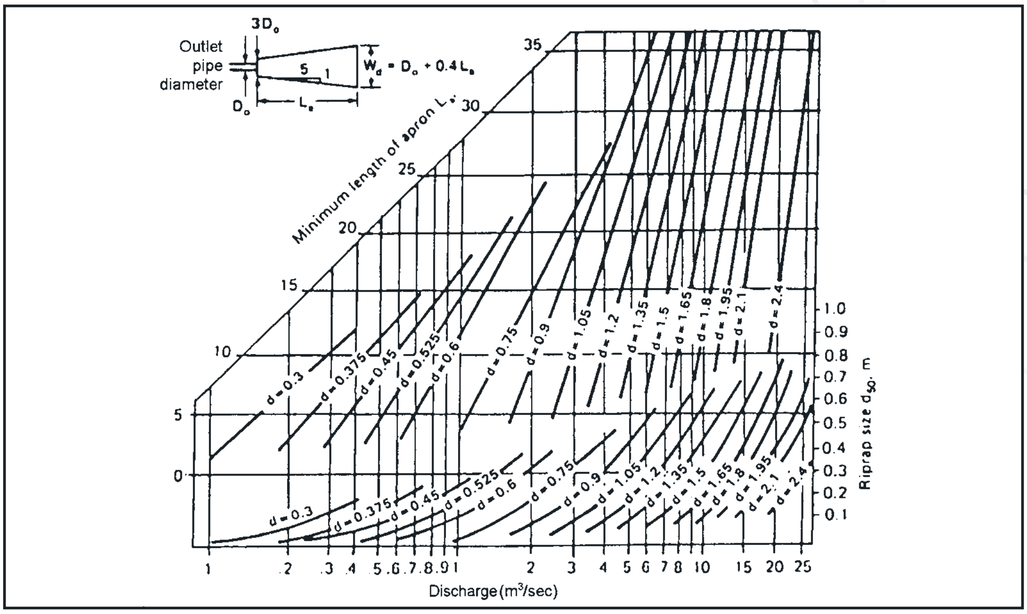


Figure 5.9 Design of riprap outlet protection - maximum tailwater conditions apply (MWRA, 1983)

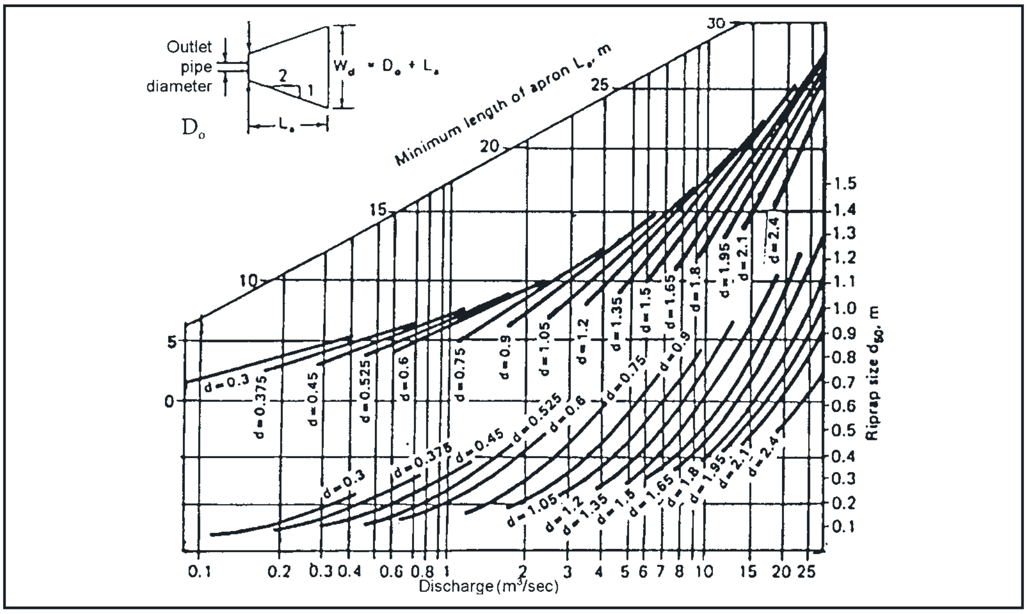


Figure 5.10 Design of riprap outlet protection – minimum tailwater conditions (MWRA, 1983)

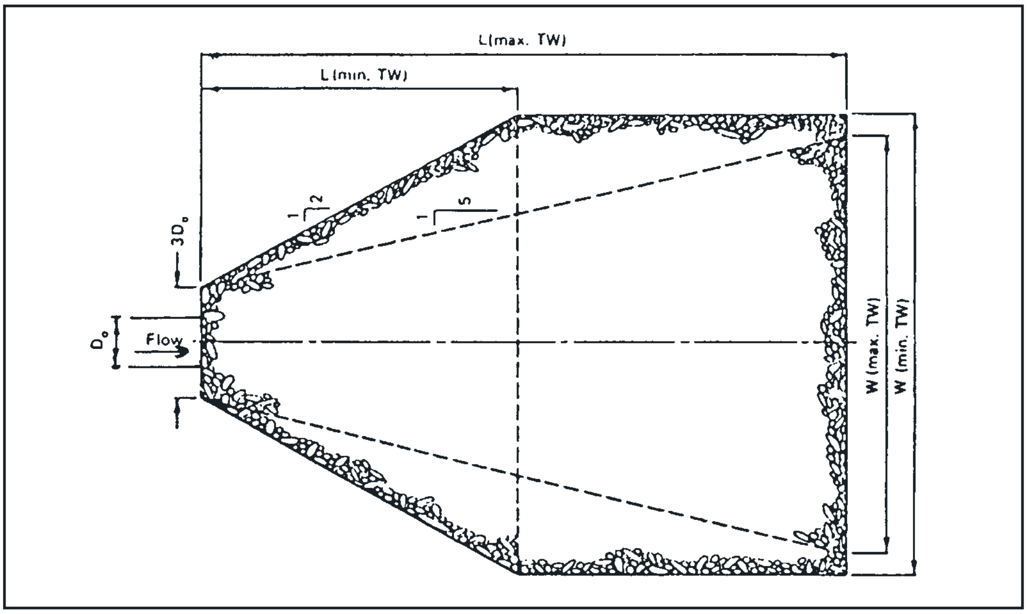


Figure 5.11 Riprap configuration for outlet protection under both maximum and minimum tailwater conditions (Bohan, 1970)

(b) Subsoil drainage can be installed to:

  (i) improve the soil environment for vegetative growth by regulating ground water flow, especially in grassed waterways and other low lying areas, recreation areas (such as ovals), and dry detention basins; and

  (ii) provide drainage of ground water on steep slopes to improve stability.

(c) Line rubble drains and perforated pipes with geotextile to act as both filters and separators. This is critical where permanence is required and the subgrade is finer than 0.02 mm and/or more than 10 percent of the whole soil material consists of dispersible fines (Section 6.3.3 (c) and following sections).

(d) When rubble drains or slotted pipes are used, they should be:

  (i) placed at a horizontal spacing that is consistent with the soil permeability and the desired surface amenity. As a general guide, place slotted pipes at a minimum depth of 0.6 metres below the soil surface while extending rubble drains to a depth of at least 0.6 metres; and

  (ii) ideally, designed to provide a minimum gradient of 1 percent.

Chapter 6
SEDIMENT AND WASTE CONTROL

6. Sediment and Waste Control

6.1 Introduction

6.1.1 Background

(a) This chapter has been written as an aid to professional designers to allow for better consideration of sediment pollution from disturbed lands such as urban construction sites, mining sites and the like. As with the rest of these guidelines, though, it does not consider farming lands. While the chapter's main focus is the control of erosion of soils and pollution from sediment and waste during the land disturbance phase, very brief discussion is also provided for long-term pollution control where this is appropriate.

(b) Stormwater treatment measures installed after construction is completed are not highlighted in this chapter. Specific information on this topic can be found in *Managing Urban Stormwater: Treatment Techniques* (EPA, 1997b; DEC, *in prep.*) and relating to such matters as:

- swales and "bioretention" measures
- infiltration measures
- constructed wetlands
- gross pollutant traps and trash racks
- litter baskets and litter booms
- oil/grit separators
- other treatment measures.

Additional information on constructed wetlands is provided by *The Constructed Wetlands Manual* (DLWC, 1998).

(c) Stormwater runoff from disturbed lands has become a major source of pollutants in New South Wales' waterways, particularly as some of the more marginal urban lands have been developed around Sydney, Newcastle and Wollongong. This chapter concentrates on diffuse or non point source pollution from land development activity.

(d) The form that diffuse pollution takes is usually determined by:

- climate, particularly local rainfall patterns
- geomorphic aspects and soil characteristics of the source area
- human and other activities in the catchment.

Contaminants are mainly transported by wind, rainfall or overland flow. Overland flow is especially relevant in areas with unprotected land surfaces or associated with particles that have settled on impervious surfaces.

(e) The quality of runoff also depends on the stage of works. For example:

(i) where much of the catchment lacks a suitable protective ground cover while development is going on, a potential exists for significant soil erosion of any exposed surfaces; and

(ii) where a significant proportion of the catchment has been developed and rates of runoff are high due to many impervious areas, a potential exists for pollution to downslope lands and waterways by nutrients, heavy metals, various chemicals and the like.

6.1.2 Pollutants

(a) The pollutants discussed and/or referred to in this document include:

(i) suspended solids, especially sediment, that might carry nutrients "piggyback" and reduce:
- light penetration in water and affect growth of aquatic plants
- suitability of habitats for some aquatic flora and fauna
- the aesthetic appeal of water;

(ii) nutrients that can promote the rapid growth of aquatic plants, particularly algae. Where algal growth becomes too dense:
- reduction occurs in light penetration of water
- oxygen levels of water can drop (see (iii), below)
- suitability for recreation, irrigation, etc. diminishes, particularly if toxic algae are present;

(iii) oxygen-demanding materials that deplete levels of dissolved oxygen in the water causing conditions to become anaerobic and, in turn, resulting in either decay or death of submerged plants and other benthic (bottom-dwelling) organisms.[1] These materials are usually measured in water as biochemical oxygen demand (BOD), and include biodegradable organic debris, such as decomposing litter and vegetation;

(iv) litter that is commonly washed from pavements and can be at high levels, especially in "first flush" waters; and

(v) microorganisms that frequently occur at high levels, especially in urban runoff, and associated with sewage/septic outfalls, animal faeces, soil, decaying vegetation and putrescible matter. Microorganisms that might make the water unsuitable for swimming or drinking are not discussed further here.

(b) Two features of sediment pollution are:

(i) studies at Lake Illawarra show that, where associated with new urban development for example, sediment pollution levels can be five to 20 times greater than those from developed urban areas and three to five times greater than those from undisturbed bushland; and

1. This situation often occurs when the available oxygen is used up by microorganisms that help in the decay process when plant materials die or an influx of other organic matter occurs. Various pollutants stored in sediment can be released under these conditions including phosphorus and some heavy metals. Some of these pollutants can promote further algal growth and a cycle is formed that is difficult to break.

(ii) many pollutants such as nutrients, are absorbed on and transported by suspended sediment particles, especially the finer (<0.005 mm) colloidal materials. Relatively coarse particles (>0.02 mm) hold very few pollutants.

Figure 6.1 Sediment deposition below a housing construction site. As a result, the bitumen road and kerb and gutter are barely distinguishable – only a drop inlet is clearly visible

Figure 6.2 Sediment resulting from unsatisfactory rehabilitation following roadworks

6.1.3 General Recommendations

(a) Where it is an option, erosion and sediment control at land development sites is the least expensive and the single most effective way to reduce water-borne pollution. It should be regarded as a major priority.

(b) Works should not cause either new seepage areas through rises in the watertable or pollution of aquifers.[2]

(c) Some aspects of sediment and nutrient management structures should not be deliberately combined, e.g. nutrient removal ponds (e.g. wetlands) should not be required to play a major role in removal of sediment. Nevertheless, each of these structures should be expected to remove various pollutants, but with varying degrees of effectiveness and with implications for design. However, nutrient removal ponds can be used for temporary sediment control during the construction phase when few nutrients are generated.

(d) The approach should be implemented completely to ensure that the desired degree of soil and water management is achieved. To be effective, it should address control/mitigation of pollution of suspended solids through reduction of soil erosion and minimisation of sediment pollution using:

   (i) drainage systems designed to minimise both the quantity of flow and its velocity, especially on unprotected land surfaces; and

   (ii) system controls that reduce the quantity of suspended solids reaching receiving waters, such as sediment retention basins, sediment traps and constructed wetlands.

(e) Be pragmatic in application of design criteria. Lower standards might be negotiable if the location and design of a particular structure meet a significant majority (e.g. >90%) of requirements and costs for the remaining parts are excessively high (in terms of capital and/or increasing the erosion hazard elsewhere). However, approval for any lower standards should be sought first with the consent authority. It is expected that application of lower standards will:

   (i) be offset through application of more stringent erosion controls, i.e. modification to BMPs elsewhere; and

   (ii) ensure that the viability of ecosystems downslope and aesthetic values are not compromised.

2. Rises in watertables through poor stormwater management practices are never encouraged and, especially, where the soils tend to have saline ground water. Many examples exist in New South Wales where extensive land degradation and damage to works have occurred because of what is now known as dryland salinity. While the principles of water sensitive urban design are strongly supported, the implications of encouraging water to infiltrate the ground should be thoroughly investigated always.

(f) Construct temporary works for control of pollution to be stable in runoff from the design storm event, usually taken to be the 10-year ARI time of concentration event. Design should take into consideration the implications of larger storm events and emergency spillways should be constructed where appropriate.

6.2 Waste Control

(a) Safe management of waste materials, such as paint, concrete slurry, acid, toilet effluent, cleared vegetation, garbage and various chemicals, should be applied at all land disturbance sites. The Protection of the Environment Operations Act, 1997 makes it an offence to allow any of the above materials to leak, spill or escape from the site or to place it where it might harm the environment.

(b) The following waste management practices should be applied:

(i) If possible at the design stage:
- design to standard sizes
- specify recycled and recyclable products
- specify reusable, repairable materials and fittings
- renovate or refurbish, rather than rebuild
- incorporate a composting area into the landscape design.

(ii) If practical during the demolition/extraction phase:
- separate reusable materials for reuse on or off site
- sell reusable materials to second-hand dealers
- reuse rock, topsoil and vegetation on the site
- stockpile materials for use elsewhere.

(iii) If practical during the construction phase:
- reuse materials from the demolition/extraction phase
- mulch and reuse as much green waste as you can
- order materials to size
- do not over order
- order precut or prefabricated materials
- purchase materials with minimal packaging
- separate reusable or recyclable materials from waste
- look out for and participate in recycling opportunities
- organise onsite sorting and/or collection systems for reprocessing
- train site workers not to damage or contaminate materials or off cuts so they can be reused elsewhere.

(iv) Store all possible pollutant materials well clear of any poorly drained areas, flood-prone areas, streambanks, channels and stormwater drainage areas. Such materials should be stored in a designated area, under cover where possible. Containment bunds should be constructed with provision for collection and restorage of any spilt material.

(v) Wash down materials and equipment away from the foreshore and intertidal areas.

(vi) Waste receptacles should also be placed away from the foreshore and intertidal areas. Where this is impractical, they can be placed there providing they are not overloaded and are watertight. Further, they should be provided with suitable waterproof covers for use during rain and site shutdown (e.g. weekends and nights) to prevent entry of water and vermin, and litter being blown around the area.

(vii) Ensure that runoff from polluted hard surfaces like roadways and vehicle hard stand areas is properly treated before discharge to the stormwater system.

(viii) Place staff facilities so that any effluent, including wash-down water, can be totally contained and treated within the site management area. Inform staff of the nature of each facility and of their obligation to use them.

(ix) For vegetation:

- preferably reuse waste materials onsite by chipping, mulching or composting, particularly where they can be used in rehabilitation programs
- otherwise:
  - gain necessary approvals from the local consent authority to remove it to an approved landfill; or
  - gain necessary approvals from the local council before burning waste in the open or by trench burning.

(x) Shift sorted waste only in an approved manner by means of suitable transport to licensed landfill sites.

(xi) Clear any bins of concrete and mortar slurries, paints, acid washings, light-weight waste materials and litter at least weekly or more frequently if they fill.

(xii) Provide designated waste collection areas with appropriate bunds or containers and maintain waste disposal and collection systems to operate within their capacity.

(xiii) Be careful in the disposal of used chemical containers and in the management of runoff and sediment loss from areas treated with pesticides (where possible use less hazardous or alternate material and comply with manufacturer's safety/usage directions).

(xiv) Dewatering activities should be closely monitored to prevent pollution in the form of sediment, toxic materials or petroleum products. Sediment controls should be established and testing of ground water should be undertaken before commencement of dewatering activities.

(xv) Prevent the discharge of pollutants to stormwater because of vehicle and equipment maintenance. This can involve using offsite maintenance facilities or undertaking work in designated and bunded areas only. Regular checks should be undertaken to ensure leaks and spills are rectified and cleaned immediately. Employees and subcontractors should be trained in this regard.

(xvi) Dispose of waste to landfill only as a last resort. Landfill sites and waste transfer stations will:

- require correct handling for dusty or hazardous wastes
- impose price penalties for mixed loads
- offer discounts for sorted wastes, such as bricks, metals and timber.

(xvii) Depending on the size of the job, the local council might require a waste management and minimisation plan. This will set out the type and volumes of any waste materials generated on the job, and explain reuse, recycle and disposal processes.

6.3 Sediment Control

6.3.1 Preamble

(a) Sediment retention basins are dams or impoundments designed to intercept sediment-laden runoff and retain most sediment and other materials, thereby protecting downstream waterways from pollution. The retention is generally achieved by the settling of the suspended sediment from the stormwater flow, combined with the interception of bedload material.

(b) Like other measures employed to control erosion and sediment at construction sites, sediment basins should be regarded as one component (or "carriage") in a properly planned "treatment train" (Section 1.2(b) (ii)). As previously stated, the control of soil erosion is the best way of minimising sediment pollution of receiving waters, an especially important principle with fine-grained soils. Sediment basins should be regarded like a fullback in a football team – a final control to be used when all others fail. However, as all construction activities inevitably disturb or expose soil materials, sediment basins (together with filters and traps) are frequently an important component of soil and water management on disturbed lands. As with a fullback, they are an essential part of the team or treatment train.

(c) The choice of sediment control measures is not limited to those nominated in this document. Creativity, combined with a sound understanding of the principles presented here, should allow other measures to be adopted to suit circumstances, provided those measures can be justified properly.

(d) Be aware that current legislation requires that the quality of runoff water leaving any work site must be of an acceptable standard and that this legislation does not make allowance for:

- particular difficulty with the site
- specific or general problems in carrying out the *ESCP/SWMP*
- whether the site manager is familiar with site-work management.

6.3.2 General Recommendations

The following issues should be considered in relation to the control of sediment:

(a) Design structures to minimise land disturbance.

(b) Pass any potential sediment-laden stormwater runoff through a trap or basin designed to minimise pollution to lands, waterways and services placed further downslope. Keep sediment as close to its source as possible.

(c) Where possible, do not construct sediment basins on line on a watercourse.

(d) Some small and/or flat sites might not warrant construction of a sediment basin, including those for which an *ESCP* (rather than a *SWMP*) is required (<2,500 square metres disturbed area). If in doubt, the average annual soil loss from the total area of land disturbance can be estimated (Appendix A). Where this is less than 150 cubic metres per year, the building of a sediment retention basin can be considered unnecessary. In such circumstances, alternate measures may be employed to protect the receiving waters.

(e) Design of sediment retention basins should ensure that water is not diverted from its intended flow path if structures become filled with sediment in the nominated storm event.

(f) Where practical, place sediment control structures:

  (i) so that only waters polluted because of site land disturbance activities enter them (i.e. waters from off-site sources and/or those from the site that are clean should be diverted elsewhere);

  (ii) off-line, so that trunk drainage carries only relatively clean water;

  (iii) away from normal construction operations (to minimise the need for regular repair); and

  (iv) upstream of any wet ponds, constructed wetlands or receiving waters.[3]

(g) Soil erosion and any resultant sediment pollution are largely dependent on storm size and nature of the soils. Ensure that the design:

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3. Here, "waters" mean:
   - street gutters
   - natural waterbodies, including lakes, lagoons or wetlands, rivers or streams, and tidal waters, e.g. bays, estuaries, inlets, etc.
   - constructed waterbodies, including lakes, wetlands, dams, ponds, waterways, channels or canals.

(i) allows adequate time for settling of the desired particle sizes; and

(ii) has adequate capacity to trap and store sediment eroded from the site.

(h) Where practical, do not decommission temporary sediment retention basins and traps until the works for which they were designed are completed and fully stabilised on more than 90 percent of the contributing catchment. Address the financial and other implications in the planning phase where this involves other developers or site operators.

6.3.3 Design of Sediment Basins

(a) When designing sediment retention basins, two principal criteria should be considered at the outset – the structures must have an ability to meet:

- appropriate water quality standards – discussed further below
- necessary structural integrity and stability standards.

Normally, sediment basins and their outlets should be designed to be stable in the peak flow from at least the 10-year ARI time of concentration event. However where individual circumstances dictate, adopting higher design standards for basin outlets might be necessary. Basins might need to be referred to the Dam Safety Committee if the wall is more than 15 metre high and/or failure could adversely affect the community's interests or the environment downstream. Further details on prescribed dams and surveillance requirements are at:

www.damsafety.nsw.gov.au/ftp/publications/pdf/dsc01.pdf.

(b) The effective design and operation of sediment retention basins from a water quality perspective depends primarily on the nature of the soil materials likely to be eroded and washed into them. Protection of downstream lands and waterways demands an approach to basin design that recognises the settling behaviour of different soil particles in water. Clearly, coarse-grained sediment will settle quicker than finer-grained sediment, whereas some clay particles seemingly never settle unaided.

(c) Soil materials that can erode and find their way to a sediment retention basin can be classified into three "texture" groups based on how effectively they are likely to settle:

- *"Type D"* soils that contain a significant proportion of fine (<0.005 mm) "dispersible" materials that will never settle unless flocculated[4]

4. Not all particles finer than 0.005 mm are dispersible. Clays are complex laminated structures comprising alternating sheets of alumina and silica. Typical structural groupings based on silica to aluminium ratios are:
   - 1:1 (kaolinite type) clays
   - 2:1 low expansion (illite) clays
   - 2:1 high expansion (montmorillonite, bentonite, smectite) clays.

   The structural grouping of the clay minerals can influence important soil properties, e.g. cation exchange capacity, shrink/swell potential, fertility and dispersibility. Illite structures can become dispersible if deformed while wet while montmorillonite structures tend to be dispersible always.

The other two categories cover those soils that are not dispersible:

- *"Type C"* soils, the bulk of which are coarse-grained (less than 33 percent finer than 0.02 mm) and will settle relatively quickly in a sediment retention basin
- *"Type F"* soils, the bulk of which are fine grained (33 percent or more finer than 0.02 mm) and require a much longer "residence" time to settle in a sediment retention basin.

(d) When deciding which soil type applies to a particular land disturbance activity, consider the following matters:

(i) Where soils of more than one type are present at a specific site, sediment basins should be designed to meet the most stringent criterion applicable. In this, note that soils that are essentially coarse-grained can be *Type D*.

(ii) No matter whether soils are classified as *Type D, Type C* or *Type F*, ensuring that pollution does not occur to downslope receiving waters is essential. To this end, treated discharge waters should not contain more than 50 milligrams per litre of suspended solids in the design rainfall event. More stringent requirements might be necessary in particularly sensitive environments or, where applicable, can be required by Council's stormwater management plan. Of course, all practical measures to reduce pollution should be taken for storm events beyond the design event.[5]

(e) Soils that are dispersible *(Type D)* and require flocculation (Appendix E) are those where more than 10 percent of the materials are dispersible. That is, where the percentage clay plus half the percentage silt (roughly the fraction <0.005 mm) multiplied by dispersion percentage (Ritchie, 1963) is equal to or greater than 10. [6]

(f) A simple field test is available that can eliminate the need for laboratory testing for dispersibility. This is the "field" Emerson Aggregate Test and is described further at Appendix E. However, soils that fail this test are not necessarily dispersible to the extent that flocculation is obligatory and laboratory analysis of dispersion percentage might still be required.

5. The actual discharge load should be considerate of the loads normally carried in the receiving waters, including those during and following storm events. Any fluvial processes within these waters will have reached equilibrium considerate of those loads. Reducing them significantly below these levels can cause streams to become "hungry" and erode their own bed and banks; while increasing them significantly can result in degradation to ecosystems.
6. Hazelton and Murphy (1992) provide a means of using Soil Texture Class as a guide to particle size distribution. Note that:
    - clays are finer than 0.002 mm (2 microns)
    - silts range from 0.002 to 0.02 mm (2 to 20 microns)
    - fine sands from 0.02 to 0.2 mm (20 to 200 microns)
    - coarse sands from 0.2 to 2.0 mm (200 microns to 2.0 mm).

    Most fine sands need a magnifying lens to see them.

(g) With dispersible and fine-grained soils, place far greater emphasis on erosion control measures to offset lower efficiencies achieved with sediment retention.

(h) The actual capacity of sediment retention basins is the sum of two components (Table 6.1):

(i) A settling zone, within which water is stored allowing the settlement of suspended sediment. The settling zone is designed to capture most sediment in a nominated design rainfall event and, in turn, a specific discharge water quality.

(ii) A sediment storage zone, where deposited sediment is stored until the basin is cleaned out – or for the life of the project where land disturbance is of a short duration (<two months).

(i) All sediment retention basins that might discharge sediment-laden stormwater more than once per year should have minimum length to width ratios of 3 to 1 to reduce short-circuiting and, preferably, at least 5 to 1. Figure 6.3 shows the influence of the ratio of the minimum flow-path length to the effective width on the apparent effectiveness of a sediment retention basin for one particular soil type. The actual effectiveness of a sediment retention basin must take into account its apparent effectiveness and the proportion of the design-size particles in the sediment load. Baffles can be employed to maximise the effective flow path within sediment basins as shown in figure 6.4.

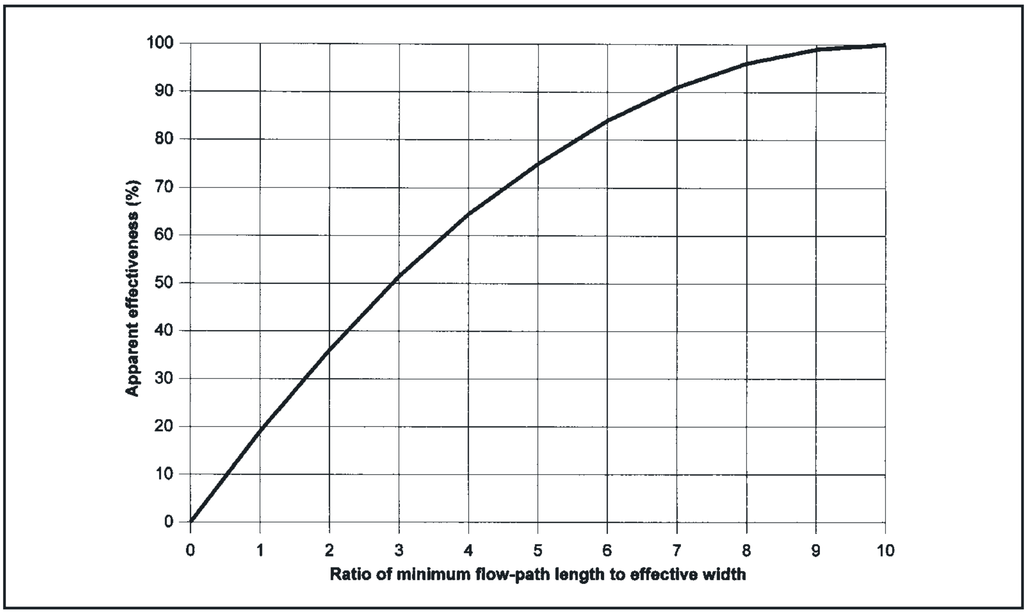

Figure 6.3 Apparent effectiveness of a sediment containment system (Fifield, 2001a)

Table 6.1 Summary of selected sediment basin types and design criteria

Soil Type	Soil characteristics	Treatment process	Basin design capacity	
			Settling zone	Sediment storage zone
Type D (dispersible)	10 percent or more of the soil materials are dispersible. Particle size is irrelevant	Aided flocculation in wet basins	Capacity to contain all runoff expected from the y percentile, x-day rainfall depth where, depending on the sensitivity of the receiving waters and/or the duration that the structure is in use: x is 2, 5, 10 or 20-days y is the 75th, 80th, 85th or 90th percentile	Normally taken as 50 percent of the capacity of the settling zone. However, it can be taken as two months soil loss as calculated by the RUSLE
Type C (coarse)	Less than 33 percent finer than 0.02 mm and less than 10 percent of the soil materials are dispersible	Rapid settling in wet or dry basins	Surface area of 4,100 m^2/m^3/sec in the 3-month ARI flow, minimum depth of 0.6m, and length:width ratio of >3:1	Normally taken as 100 percent of the capacity of the settling zone. However, it can be taken as two months soil loss as calculated by the RUSLE
Type F (fine)	33 percent or more of the particles are finer than 0.02 mm and less than 10 percent of the soil materials are dispersible	Slow settling in wet basins	Capacity to contain all runoff expected from the y percentile, x-day rainfall depth where, depending on the sensitivity of the receiving waters and/or the duration that the structure is in use: x varies between 2 and 20 days y is the 75th, 80th, 85th or 90th percentile	Normally taken as 50 percent of the capacity of the settling zone. However, it can be taken as two months soil loss as calculated by the RUSLE

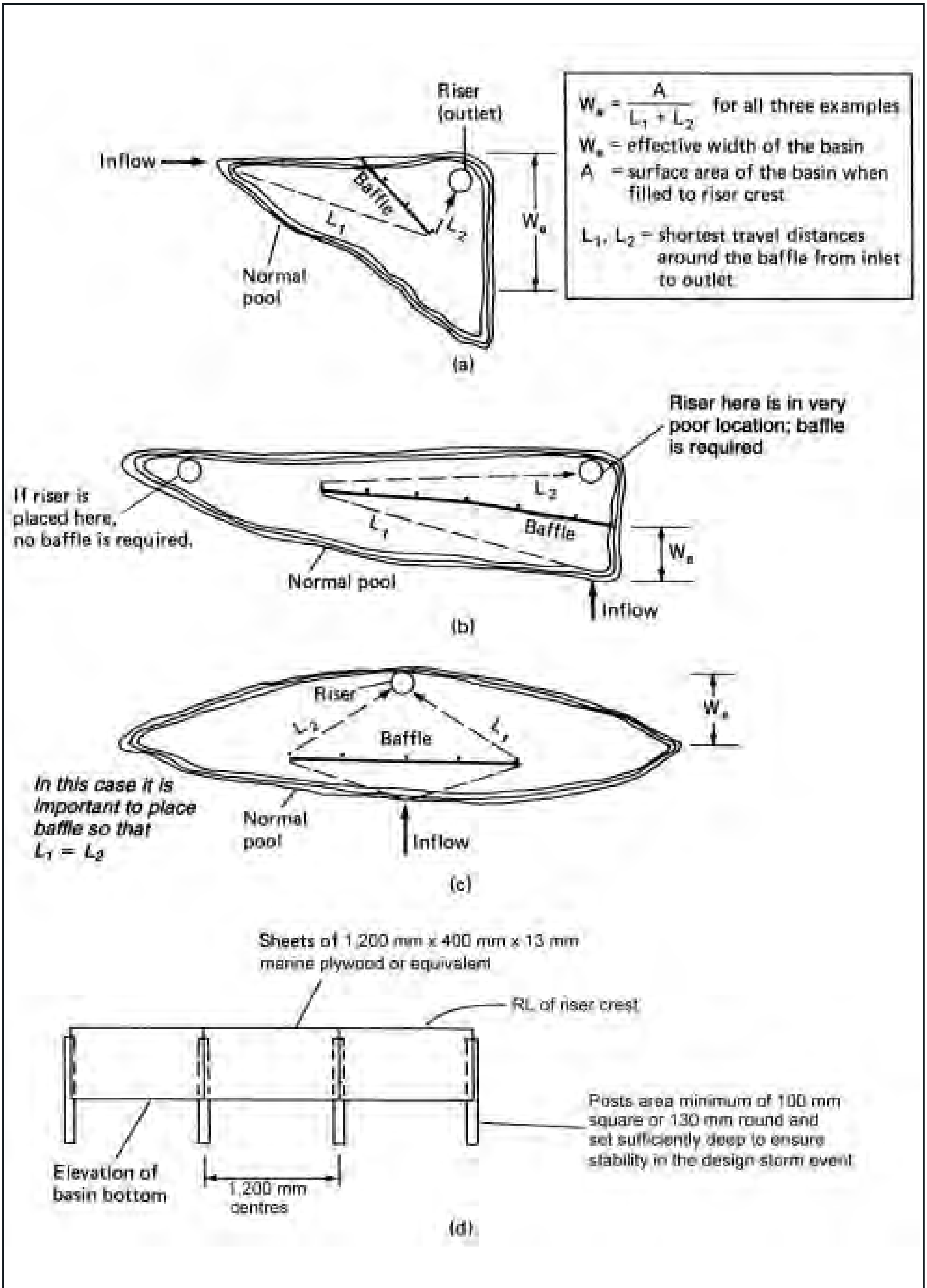

Figure 6.4 (a) to (c) – diagrammatic examples of placement of baffles in sediment basins to increase the ratio of length to width, and (d) – baffle detail, elevation (adapted from USDA, 1975)

(j) Sediment retention basins should be built to incorporate:

(i) a primary outlet designed:

- to have a capacity to pass the peak flow from the design storm event
- with an invert level at least 300 mm below any emergency outlet (where it is a separate structure);

(ii) an emergency spillway designed to have a capacity to pass the peak flow from the applicable storm event. Generally, this should be of open construction rather than a pipe outlet due to the risk of pipe blockages during high flows.[7] Nevertheless, any riser structures should be baffled or fitted with anti vortex devices;

(iii) internal batter gradients that are consistent with personal safety and, generally, within the following upper limits:

- where water depth is less than 150 mm when surcharging, 2.5(H):1(V) to 4(H):1(V) on earth structures and vertical on rock or gabion structures[8]
- where water depth is between 150 and 1,500 mm when unprotected and surcharging, a maximum slope of 5(H):1(V)[9]
- where water depth is between 150 and 1,500 mm when protected (e.g. fenced) and surcharging or greater than 1,500 mm:
    - 2.5(H):1(V) to 4(H):1(V) on earth structures[8]
    - 0.5(H):l(V) on rock gibber structures
    - 1(H):4(V) on gabion basket structures
    - 1(H): 4(V) on stacked (rough squared) rock structures;

(iv) appropriate outlet protection to ensure minimisation of scour as described in Section 5.3.3.

(k) Sediment basins can be constructed from earth, rock or suitable crushed concrete products where formed as aboveground ponds, or plastic or metal for underground tanks (see Standard Drawings SD 6-1, SD 6-2, SD 6-3, SD 6-4, SD 6-5). Choice of materials generally depends on constraints imposed by the design criteria, site conditions and local maintenance/cleaning criteria. Rock and gabion basket structures should be lined on the inside with a geotextile material (dry basins) to ensure removal of sediment particles from the system, or a suitable impermeable material for wet (*Type D* or *Type F*) basins.[10]

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7. If a piped outlet is adopted as an emergency spillway, appropriate measures should be employed to minimise the risk of blockages, and/or the pipe outlet should be significantly over designed to reduce the risk of blockage.
8. The actual gradient adopted depends on various soil characteristics.
9. The actual maximum gradient is determined by the “slipperiness” of the saturated sediment – whether or not a person can achieve a firm footing on it. Slippery sediments should have less steep gradients, in the order of 8:1 or even 10:1.
10. Geotextile can become blocked in time. So, dry basins can become wet if it is not replaced or reverse-flushed.

(l) The lower level of the settling zone should be identified with a peg or other marker that shows clearly the level above which the design capacity is available. Stored sediment should not encroach into the settling zone.

(m) Sediment removed from sediment basins should be disposed in places that will not result in a future erosion or pollution hazard (see Section 4.3.2 (h)). Note that fine and/or flocculated sediment removed from wet basins might require considerable time to dry to a level where it can be handled with relative ease.

6.3.4 Capacity of Basins for *Type D* and *Type F* Soils

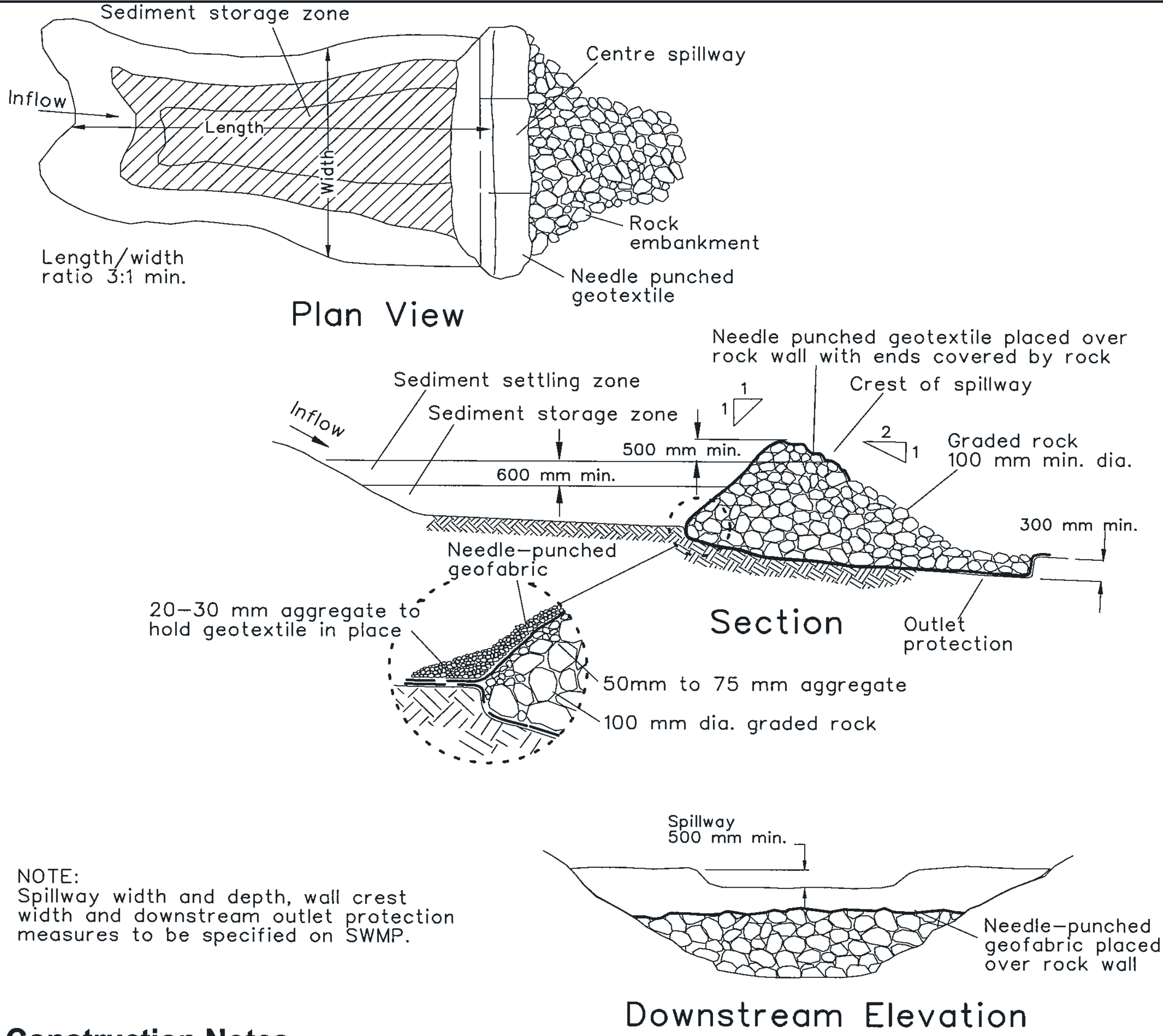
(a) Sediment retention basins for *Type D* and *Type F* soils are wet basins (SD 6-4 and SD 6-5). Their design criteria are the same. They differ only in management and, specifically, the recommended methods of dewatering and in the likely use of flocculants.

(b) The traditional approach to design of sediment retention basins is based on the settling of a design particle. However, this settling methodology is generally ineffective where the sediment contains significant quantities of fine (<0.02 mm) or dispersible materials because of very long settling times that require extraordinarily large structures and, even then, might not achieve the desired result. Both Type D and *Type F* soils are very fine and *Type D* soils are dispersible as well. So, generally, a total storm containment system is adopted here for a nominated design rainfall depth, a risk-based approach that is considerate of local daily rainfall patterns. Such basins are normally empty. They fill after rainfall events with water remaining in them long enough to be properly treated with settling agents such as gypsum (Appendix E). They are then pumped out or allowed to drain under gravity.

(c) However, other methodologies are available that enable constant flocculation of sediment-laden waters derived from *Type D* soils. One such method is described at Section E4.2 (Appendix E). Such basins do not require pumping out, so long as they can achieve an acceptable suspended solid's concentration of less than 50 milligrams per litre in the residence time provided in the design storm. Use of such alternative approaches should not vary the design capacity from that described here.

(d) A 5-day rainfall depth can be adopted as standard in the design of the settling zone where the soils being disturbed are *Type D* or *Type F*. This assumes that five days or less are required following a rainfall event to achieve effective flocculation if necessary, settling and subsequent discharge of the supernatant stormwater (Appendix E and Section 6.3.3(d)).

(e) In certain conditions, basins can be designed for rainfall depths and management periods of between 2 and 20 days, to accommodate a range of site constraints and opportunities that may be present :



NOTE:
Spillway width and depth, wall crest width and downstream outlet protection measures to be specified on SWMP.

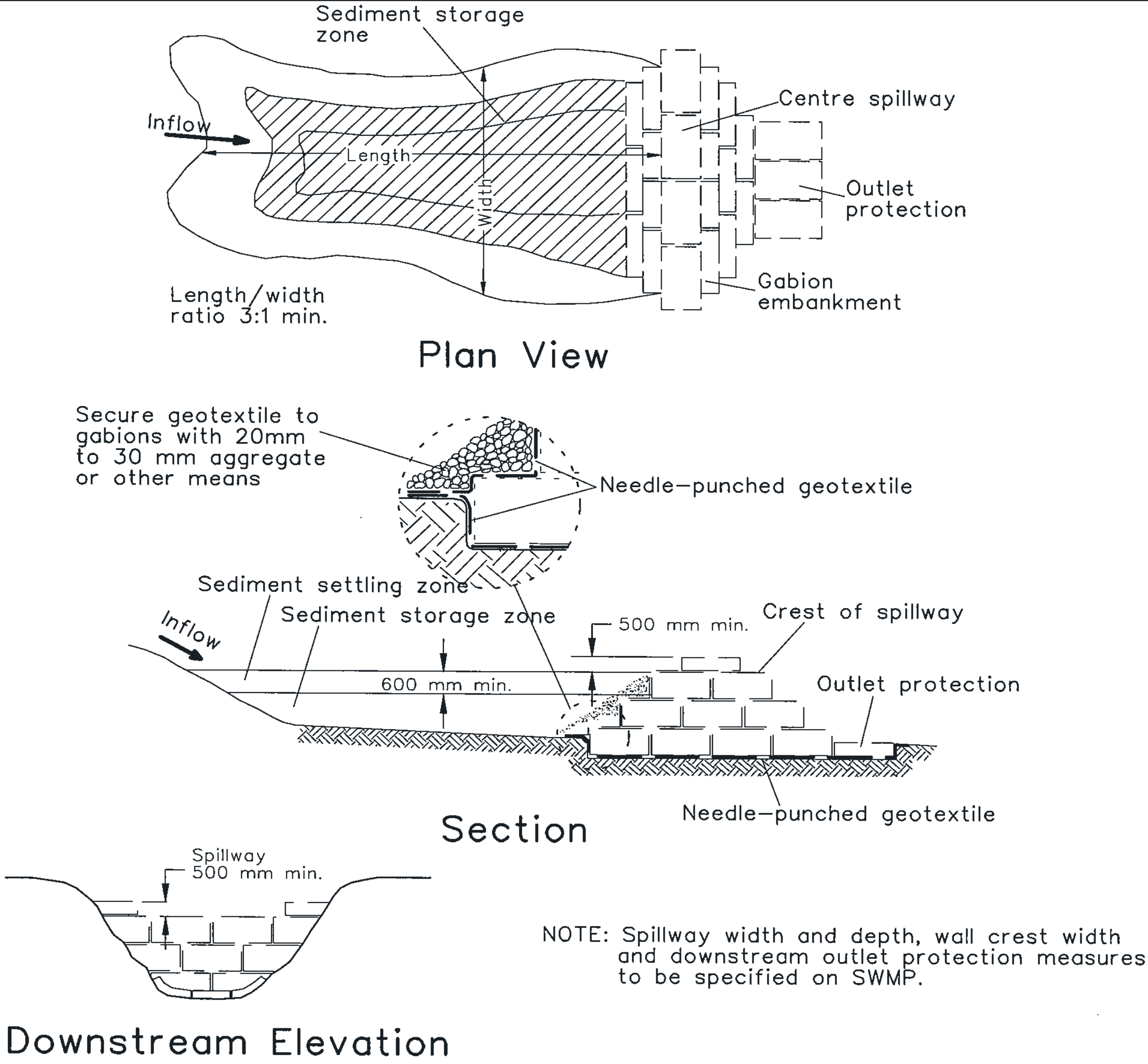
Construction Notes

1. Remove all vegetation and topsoil from under the dam wall and from within the storage area.
2. Excavate to 300 mm depth for base of the dam wall.
3. Line the excavation with a needle-punched geotextile allowing sufficient to line below the wall, and over the upstream rock and the spillway to 500 mm below the spillway exit on the downstream face.
4. Make up the wall profile and outlet protection with100 mm (min.) diameter graded rock. Spread a layer of 50 mm to 75 mm diameter aggregate over the upstream batter for a more even surface, and add 100 mm to 150 mm of 20 mm to 30 mm gravel over the 50 mm to 75 mm diameter aggregate.
5. Lay geotextile over the upstream batter and through the spillway, fixing in place with 100 mm rock.
6. Place a "Full of Sediment" marker to show when less than design capacity occurs and sediment removal is required.
7. Replace the upstream geotextile layer each time sediment is removed

ROCK SEDIMENT BASIN

(APPLIES TO 'TYPE C' SOILS ONLY)

SD 6-1



Construction Notes

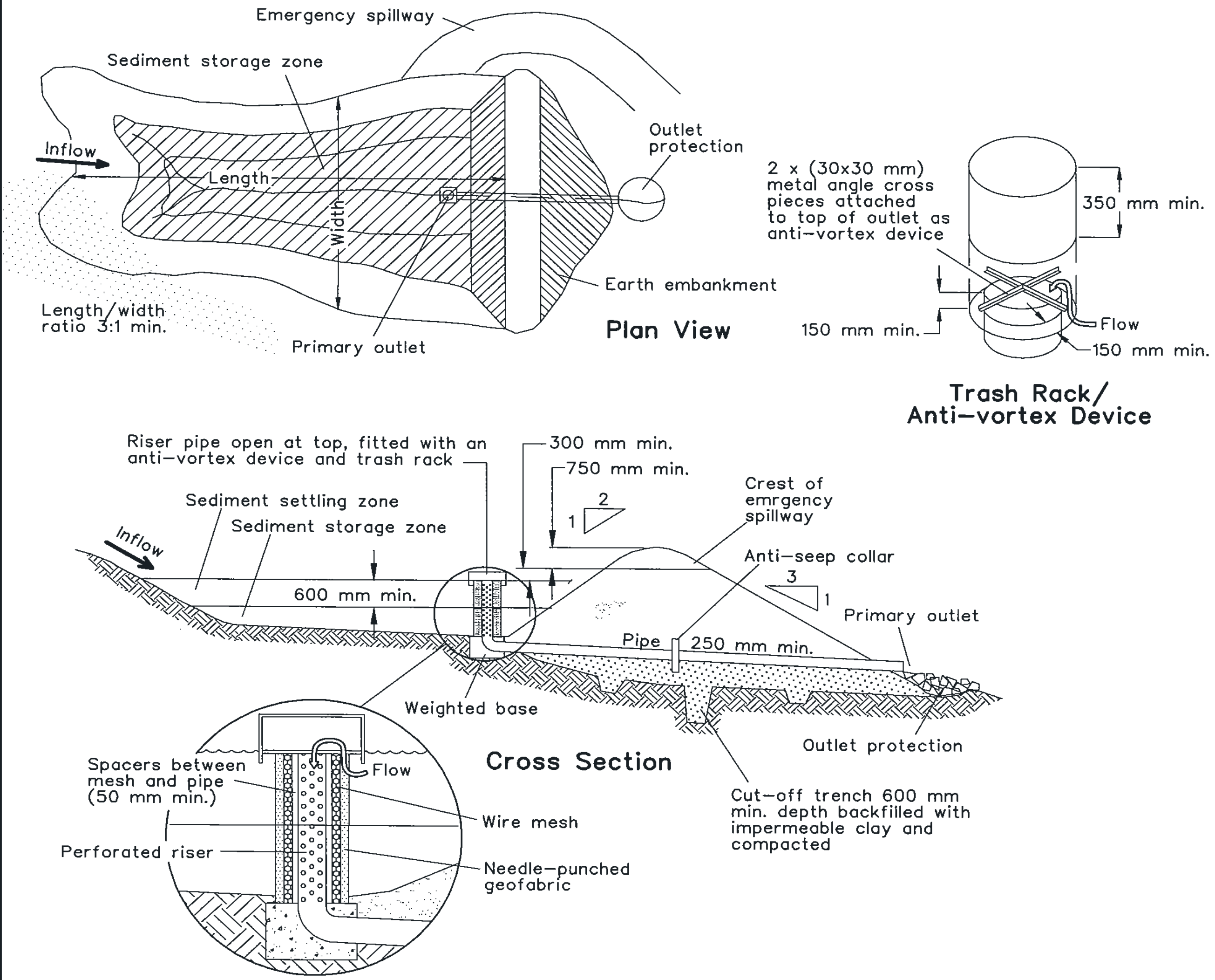
(Applies to Type C soils only)

1. Remove all vegetation and topsoil from under the dam wall and from within the storage area.
2. Excavate to 300 mm depth for the base of the dam wall and form a level platform for the gabions.
3. Line the excavation with a needle-punched geotextile allowing sufficient to line below the wall, and over the upstream gabions and spillway to 500 mm below the spillway exit on the downstream face.
4. Make up the wall profile and outlet protection with gabion units filled with graded rock as specified on the SWMP.
5. Construct a spillway 500 mm below the crest of the dam and for the width specified on the SWMP.
6. Lap the geotextile over the upstream face and through the spillway and fix it in place with the top row of gabions.
7. Cover the upstream face of the wall with 20 mm to 30 mm gravel and geotextile (Standard Drawing 6-2b)
8. Place a "Full of Sediment' marker to show when less than design capacity occurs and sediment removal is required.
9. Replace the upstream geotextile layer when sediment is removed if a dry basin is required.

GABION SEDIMENT BASIN

(APPLIES TO 'TYPE C' SOILS ONLY)

SD 6-2



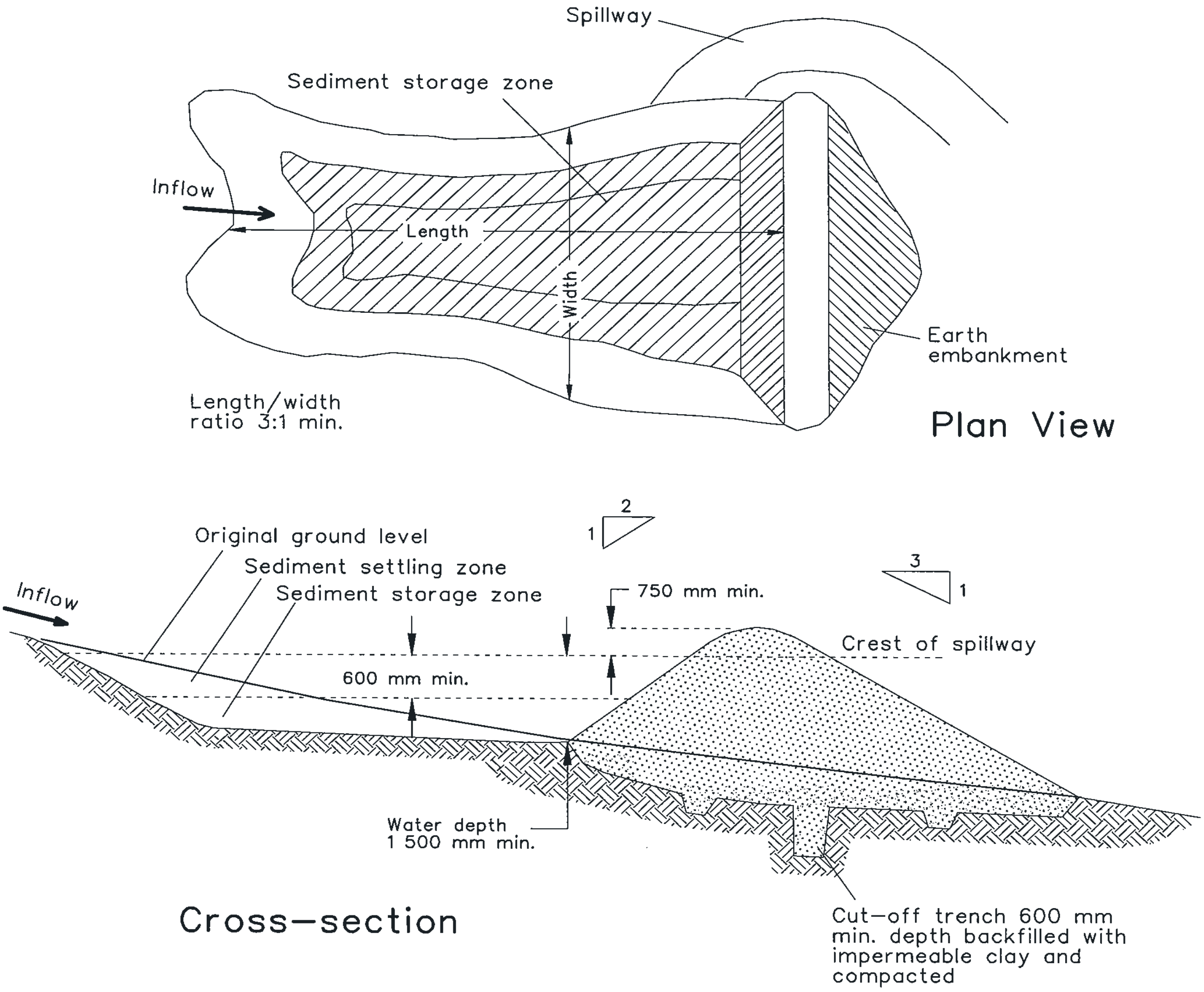
Construction Notes

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

EARTH BASIN - DRY

(APPLIES TO 'TYPE C' SOILS ONLY)

SD 6-3



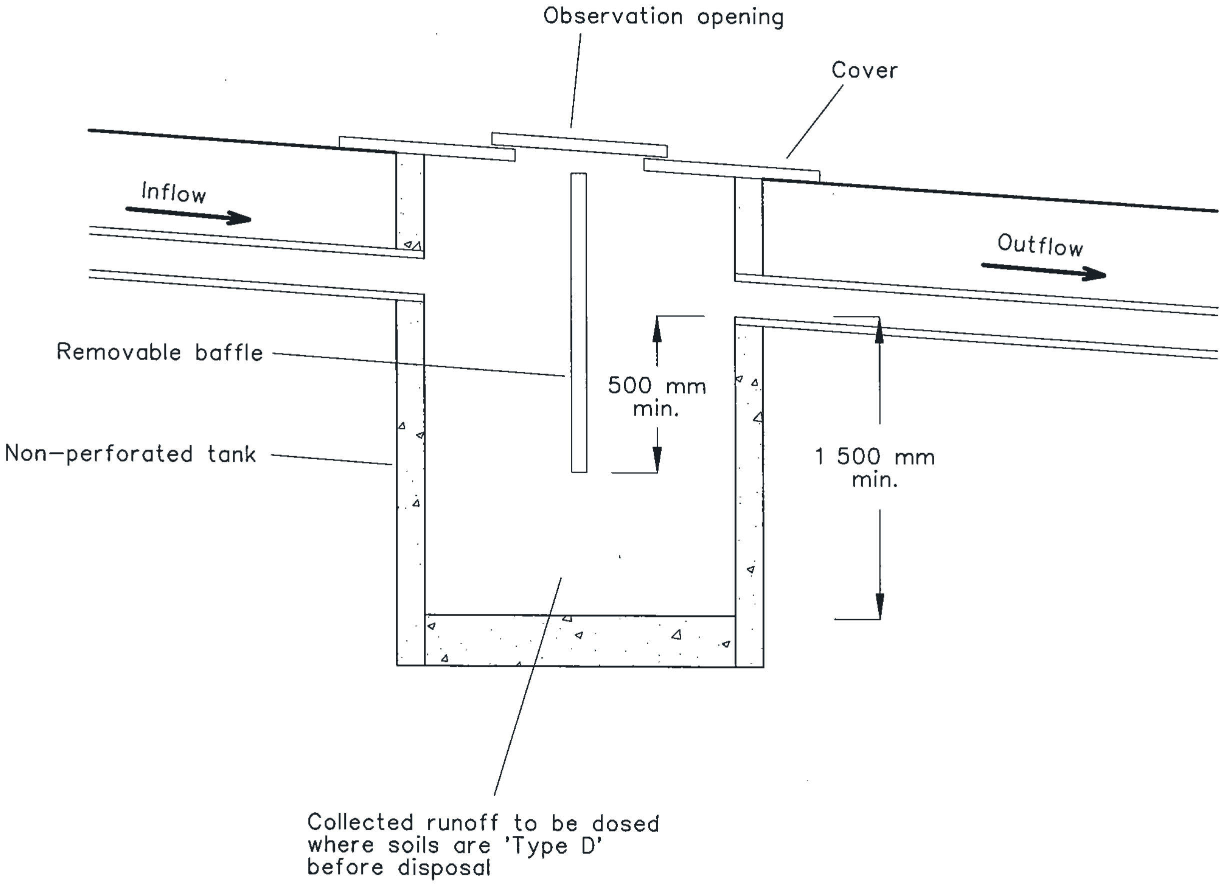
Construction Notes

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

EARTH BASIN - WET

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

SD 6-4



Construction Notes

1. Join the inlet to the stormwater, taking any suitable steps to remove bulky or coarse material before it can enter the tank.
2. Connect the outlet to a safe disposal area following the SWMP.
3. Install a removable baffle, central to the inflow/outflow and normal to the direction of flow, ensuring that it reaches 500 mm below the invert of the outlet pipe.
4. Install a cover over the pit with an observation port and access cover.

LINED TANK

SD 6-5

(i) Where the site area is insufficient to allow building structures as required for the y-percentile 5-day criterion, a 2, 3 or 4-day rainfall depth can be adopted providing flocculation, settlement and discharge can be achieved in that time. However, this will usually require the use of a special range of flocculants and specialised techniques that will achieve sufficiently fast settling (Section E4.2). Many such flocculants can cause environmental harm if not managed properly and the plans for sediment control must also include a detailed plan of management of these.

(ii) Where site conditions permit the construction of extremely large structures, a 6 to 20-day rainfall depth can be adopted. These large structures allow longer periods for reuse (e.g. dust suppression) or flocculation, settling and discharge.

(f) Unless Council's Stormwater Management Plan states differently:[11]

(i) on most sites the 75th percentile storm depth is recommended for use if the duration of disturbance is likely to be six months or less, while the 80th percentile storm depth is recommended if the duration of disturbance is likely to be more than six months;

(ii) where receiving waters are considered particularly sensitive, either by the development proponent/designer, local council or other consent authority, a higher level of protection can be provided, e.g.: the 80th percentile storm depth is recommended for use if the duration of disturbance is likely to be six months or less, while the 85th percentile storm depth is recommended if the duration of disturbance is likely to be more than six months.

Longer term land disturbances, such as waste depots, extractive sites and some road construction activities, warrant alternate levels of protection, as defined in relevant sections of Volume 2.

(g) Where space does not permit the use of structures designed for the 80th percentile or larger x-day rainfall depths and these are desirable, additional erosion controls can be considered instead, e.g. ensuring that the lands:

(i) are not in a condition of high erosion hazard during those half months when 5 percent or more of the average annual erosion index occurs (Table 6.2); or

(ii) have C-factors higher than 0.1 only when the 3-day forecast suggests that rain is unlikely.[12] In this case, management regimes should be established that facilitate rehabilitation within 24 hours should the forecast prove incorrect.

11. Note that increasing the design criteria from the 75th percentile, 5-day depth to the 90th percentile, 5-day depth more than doubles the size of the settling zone and sometimes triples it.
12. C-factors of 0.1 can be achieved in various ways as shown at Appendix A, note especially figure A5, Table A3 and Table A4. For example, figure A5 shows that a C -factor of 0.1 can be achieved with a 60 percent grass cover where, previously, the soils were stripped or deeply cultivated; alternately, Table A3 shows it can be achieved temporarily by application of a hydraulic soil stabiliser.

(h) Figure 6.5 shows the y-percentile 2, 5, 10 and 20-day rainfall depths (mm) for Sydney's Observatory Hill, while Tables 6.3a and 6.3b shows the 75th, 80th 85th, 90th and 95th percentile 2, 5, 10 and 20-day rainfall depths for 58 sites throughout New South Wales. Similar graphs to figure 6.5 are shown for 57 other sites throughout New South Wales at Appendix L. Rainfall depths at locations not shown, or for time periods not shown (between 2 and 20 days) can be estimated by interpolation or from graphs of rainfall depth estimated from the annual mean rainfall, whichever is the most conservative. Estimation of the 75th, 85th and 95th percentile 5-day rainfall depth from annual mean rainfall is shown in figure 6.6 and also at Appendix L for other locations and situations. Rainfall depths smaller than the 75th percentile depth are shown in figures 6.5 and 6.6 and at Appendix L for illustration purposes only and should not be used in design. Note that rainfall events smaller than 0.2 mm have been omitted in the calculations of all graphs.

(i) Normally, sediment basins where the soils are *Type D* or *Type F* are sized as follows:

V = settling zone + sediment storage zone (Table 6.1)

(i) The settling zone capacity designed to capture *Type D* and *Type F* soils can be determined from the y-percentile, 5-day rainfall depth, i.e.

Settling Zone $_{Type\ D/F}$ = 10 x Cv x A x R $_{(y\ \%ile,\ 5\ day)}$

where:

- 10 is a unit conversion factor
- C_v is a volumetric runoff coefficient, defined as that proportion of rainfall that runs off as stormwater[13]
- A is the catchment area of the basin (hectares)
- $R_{(y\ \%ile,\ 5\ day)}$ is the 5-day total rainfall depth (mm) that is not exceeded in y percent of rainfall events. This figure can be determined from Appendix L. Rainfall depths corresponding to management periods more and less than 5 days can be adopted, as site characteristics allow and as detailed previously

The volumetric runoff coefficient should be derived from Appendix F.

(ii) On lands of low erosion hazard (determined through the simple procedure described in Section 4.4.1) , the capacity of the sediment storage zones on *Type D* and *Type F* soils can be determined as either:

- 50 percent of the settling zone capacity, or
- two months soil loss as calculated with the RUSLE

13. This figure differs from the peak flow runoff coefficient used in the determination of peak flows according to Pilgrim (1998) (Appendix F). The volumetric coefficient of runoff is used for calculations for the sizing of sediment basins on Type D and Type F soils only.

Table 6.2 Percentage of average annual EI that normally occurs in the first and second half of each month for each Rainfall Zone (figure 4.9) (Rosewell and Turner, 1992)

Zone	Jan		Feb		Mar		Apr		May		Jun		Jul		Aug		Sep		Oct		Nov		Dec	
1	6	6	7	8	8	8	6	5	5	4	3	2	2	2	2	2	2	2	2	3	3	4	4	4
2	10	9	9	8	7	5	2	2	1	1	2	1	1	1	1	1	3	3	3	4	5	6	7	8
3	6	8	9	9	10	7	7	4	2	2	2	2	2	1	0	1	2	2	2	3	3	4	6	6
4	6	6	8	8	8	5	5	3	3	2	2	2	2	3	3	2	2	3	3	3	5	5	5	6
5	2	3	7	13	13	10	11	6	3	2	3	2	2	2	1	1	1	3	3	3	3	2	2	2
6	11	10	10	9	6	5	2	2	2	1	1	1	1	1	1	1	2	2	4	3	5	5	8	7
7	9	9	7	8	4	5	3	3	2	3	2	1	2	1	2	2	2	3	4	4	4	6	7	7
8	7	8	7	8	5	6	4	3	2	2	2	1	2	1	2	2	2	2	4	4	6	6	7	7
9	8	9	8	7	6	5	3	3	2	2	1	2	1	1	1	2	3	3	5	5	5	6	6	6
10	7	6	9	7	7	6	4	4	3	2	1	1	2	1	1	2	2	3	4	5	6	6	5	6
11	10	11	11	9	10	5	3	1	1	1	1	1	1	1	2	2	1	2	2	5	6	6	6	6
12	10	9	8	7	5	4	4	2	2	1	1	2	1	1	1	2	3	4	3	4	4	6	7	9

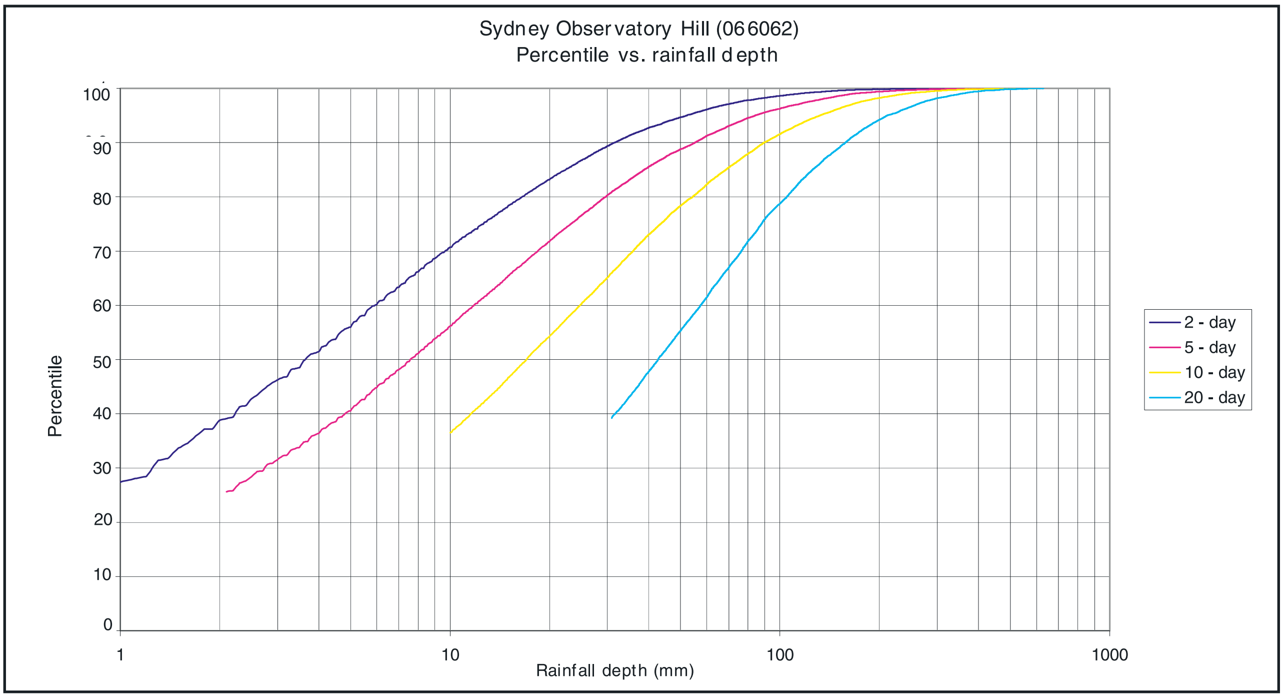


Figure 6.5 Y-percentile 2, 5, 10 and 20-day rainfall depths at Sydney's Observatory Hill

Table 6.3a 75th, 80th, 85th, 90th and 95th-percentile 2 and 5-day rainfall depths for 59 sites in New South Wales

Location	2-day rainfall depths (mm)					5-day rainfall depths (mm)				
	75th %ile	80th %ile	85th %ile	90th %ile	95th %ile	75th %ile	80th %ile	85th %ile	90th %ile	95th %ile
North Coast										
Coffs Harbour	18.3	23.6	31.8	44.4	70.8	33.6	42.7	55.8	74.9	117.6
Dorrigo	22.1	27.9	36.4	49.0	77.0	40.3	49.3	63.7	84.8	132.0
Grafton	14.0	17.8	22.9	31.2	48.9	23.3	29.0	37.2	50.1	75.4
Lismore	16.3	20.6	26.4	36.3	57.0	28.6	35.3	45.2	60.2	95.3
Port Macquarie	18.0	22.9	29.8	41.4	65.3	32.0	40.1	51.8	70.0	106.2
Taree	15.0	19.0	24.9	35.5	56.4	25.0	31.7	41.2	55.9	90.6
Tweed Heads	23.4	29.5	37.6	50.8	78.7	39.6	48.5	62.5	82.5	126.8
Central Coast/Hunter										
Cessnock	13.4	16.5	21.1	28.5	45.0	20.3	24.4	31.0	42.8	63.0
Gosford (Narara)	16.7	21.3	28.4	39.8	63.0	27.9	35.0	45.8	62.2	99.3
Nelson Bay	17.5	22.3	28.9	39.4	58.9	30.4	38.1	48.3	63.5	91.5
Newcastle	13.7	17.6	23.0	31.8	48.1	24.4	30.5	38.9	51.8	76.7
Scone	12.4	15.3	19.3	25.0	37.8	19.0	22.6	27.7	35.9	51.3
Wyong	16.8	20.8	26.9	37.2	58.8	26.8	33.8	43.2	58.7	90.1
Sydney/Blue Mountains										
Bankstown	11.4	14.5	19.6	27.0	42.0	19.4	24.4	31.5	42.6	66.6
Blacktown	12.0	15.0	20.3	28.0	43.6	19.0	24.6	32.2	43.2	70.8
Camden	13.6	16.8	21.6	29.2	44.8	20.2	25.1	32.0	43.4	66.3
Campbelltown	12.2	15.2	19.0	26.9	42.1	19.3	23.9	30.6	43.2	63.3
Hornsby	15.7	20.6	27.4	38.1	61.0	25.9	32.8	43.3	60.0	92.5
Katoomba	16.5	20.6	26.7	37.6	60.2	28.0	35.2	45.4	63.0	99.6
Lithgow	11.4	14.0	18.3	24.2	35.3	19.5	23.6	29.4	37.8	56.4
Liverpool	12.2	15.5	20.0	28.4	43.2	19.2	24.4	32.2	43.8	70.2
Mona Vale	19.0	23.6	29.2	38.7	62.0	29.0	35.2	44.0	61.2	92.0
Mosman	15.2	19.3	25.4	35.8	57.7	26.2	32.9	43.2	59.6	91.5
Parramatta North	11.7	15.2	20.6	28.2	45.5	20.3	25.8	33.1	45.8	74.1
Penrith	14.0	18.2	23.6	31.5	49.5	21.8	27.4	35.0	47.6	74.6
Richmond	10.2	13.5	18.0	24.9	39.2	17.5	22.4	29.5	39.7	61.4
Ryde	14.7	18.3	24.9	34.3	53.5	23.4	29.5	38.8	53.6	80.5
Springwood	15.5	20.1	25.9	35.0	55.6	25.2	31.4	40.4	55.0	84.1
Sutherland	15.0	18.8	24.9	34.8	55.0	23.4	29.7	38.9	54.6	85.1
Sydney 12.7		16.6	22.4	31.6	52.1	23.3	29.7	38.8	55.2	84.3
Wallacia	14.0	17.8	23.0	31.4	48.8	22.1	27.6	36.6	48.8	76.2
Wilberforce	11.4	14.9	19.8	27.7	46.4	19.8	24.6	33.2	46.7	69.4
Illawarra/South Coast										
Albion Park	16.5	21.1	27.9	39.1	67.4	25.2	31.8	41.9	59.8	101.2
Batemans Bay	13.7	17.8	24.1	34.2	54.9	22.1	28.0	37.4	52.4	84.4
Bega	12.6	16.1	21.3	30.5	51.1	19.5	24.6	32.5	46.2	77.2
Cooma	7.6	9.8	13.0	17.8	27.2	12.5	15.8	20.0	25.8	39.1
Helensburgh	23.1	28.7	38.1	53.0	81.3	35.6	45.0	57.4	78.2	124.6
Kiama	14.7	19.1	24.9	35.5	57.2	25.5	32.2	42.1	58.3	90.7
Kangaroo Valley	16.8	21.4	29.2	41.7	70.6	26.8	34.2	45.7	67.0	115.6
Mittagong	14.7	18.3	23.4	31.8	49.1	22.9	28.0	36.2	49.0	75.2
Robertson	15.8	20.3	27.9	38.2	67.3	28.4	36.0	46.1	67.3	113.0
Wollongong	13.8	18.0	24.8	36.6	61.3	25.4	33.0	43.5	60.8	95.6
Northern Tablelands and Northwestern Slopes										
Armidale	12.4	15.2	19.3	25.0	35.3	19.8	24.1	29.2	37.4	52.9
Gunnedah	14.2	17.3	21.3	27.7	39.2	20.0	24.1	30.2	38.4	53.0
Tamworth	15.2	18.3	22.2	27.7	39.6	21.6	25.2	30.8	39.2	54.2
Tenterfield	18.8	22.3	26.7	33.8	46.0	26.7	31.4	38.1	47.4	63.3
Central Tablelands and Central Western Slopes										
Bathurst	10.7	13.2	16.5	21.4	30.4	16.8	20.6	24.9	31.4	43.7
Cowra	12.0	14.7	18.0	22.9	32.8	18.1	21.6	26.1	32.5	44.9
Dubbo	12.7	16.0	20.2	26.1	36.0	18.8	22.8	28.4	35.6	50.7
Southern Tablelands and Southwestern Slopes										
Albury	11.8	14.4	17.4	22.4	31.6	20.0	23.7	28.4	35.2	45.2
Goulburn	7.8	10.0	13.2	18.0	27.4	14.2	17.8	22.2	28.6	40.8
Jindabyne	11.9	14.2	17.3	22.6	33.4	17.3	20.6	24.9	32.0	46.8
Queanbeyan	12.7	15.2	18.8	24.2	34.3	18.0	21.3	25.8	33.0	45.1
Wagga	9.2	11.4	14.4	19.3	27.6	15.6	18.8	23.4	29.4	40.2
Northwestern, Southwestern and Far Western Plains										
Bourke	11.7	14.6	18.3	24.8	35.6	15.3	19.0	23.9	30.9	44.5
Broken Hill	7.1	9.1	12.0	16.8	25.9	9.7	12.2	16.2	21.6	33.0
Griffith	9.5	11.7	14.0	18.5	26.2	13.8	16.4	20.6	25.4	34.6
Moree	12.6	15.8	19.3	25.1	36.8	18.0	21.9	26.8	36.3	51.4
Nyngan	12.2	15.2	19.1	25.6	37.3	16.5	20.4	25.8	33.8	47.8

Table 6.3b 75th, 80th, 85th, 90th and 95th-percentile 10 and 20-day rainfall depths for 59 sites in New South Wales

Location	10-day rainfall depths (mm)					20-day rainfall depths (mm)				
	75th %ile	80th %ile	85th %ile	90th %ile	95th %ile	75th %ile	80th %ile	85th %ile	90th %ile	95th %ile
North Coast										
Coffs Harbour	62.2	75.0	94.0	125.9	181.6	127.0	148.7	174.6	215.0	281.6
Dorrigo	71.2	87.5	108.8	141.4	213.1	142.2	169.1	203.5	257.8	366.2
Grafton	39.4	48.6	60.2	78.3	112.8	76.2	88.9	106.4	130.5	175.2
Lismore	49.8	60.2	75.3	100.4	148.1	98.0	115.6	138.2	176.2	242.0
Port Macquarie	57.9	70.6	88.4	115.0	159.9	116.6	135.4	158.6	193.0	249.0
Taree	43.5	53.5	68.0	91.8	135.8	85.6	101.6	124.6	157.7	207.0
Tweed Heads	67.0	81.0	99.3	130.8	186.8	124.6	147.0	178.6	219.2	288.5
Central Coast/Hunter										
Cessnock	31.0	38.3	48.3	62.3	87.3	57.0	67.0	80.1	99.0	133.4
Gosford (Narara)	49.2	59.7	76.5	100.7	148.6	96.1	113.4	137.0	173.4	239.2
Nelson Bay	53.3	63.8	77.7	99.2	136.7	100.2	117.0	135.2	165.4	219.0
Newcastle	43.9	52.8	64.6	83.3	113.6	85.6	98.6	115.3	139.5	182.4
Scone	28.7	33.9	41.2	52.0	70.6	50.6	57.9	67.7	82.8	109.2
Wyong	45.2	55.4	69.4	89.2	130.9	85.8	100.2	119.9	150.9	208.2
Sydney/Blue Mountains										
Bankstown	33.0	41.4	52.6	69.2	99.5	66.0	80.6	96.0	116.4	161.0
Blacktown	32.6	40.4	51.2	70.2	102.8	64.8	76.2	94.4	117.7	159.4
Camden	31.0	38.2	48.6	63.8	95.0	57.4	67.6	82.7	104.9	143.7
Campbelltown	32.0	39.2	49.9	64.9	100.3	61.6	71.1	87.4	118.7	149.9
Hornsby	44.4	54.9	71.1	91.9	139.4	86.2	100.8	122.8	156.7	214.9
Katoomba	50.1	62.2	78.4	103.8	157.6	101.6	121.4	146.4	183.4	260.0
Lithgow	32.9	38.9	47.5	60.7	84.4	63.9	74.0	86.6	104.4	134.3
Liverpool	33.2	41.0	52.4	70.4	102.0	66.4	79.0	95.8	118.6	156.8
Mona Vale	45.8	56.6	71.2	91.8	129.2	87.1	100.6	120.0	150.1	198.4
Mosman	47.0	57.5	72.8	95.8	137.4	93.2	110.2	131.0	160.9	218.6
Parramatta North	35.4	44.2	56.1	76.4	112.3	70.6	87.0	103.2	131.6	178.5
Penrith	33.8	41.7	52.9	71.4	104.9	61.7	74.1	91.3	118.4	160.4
Richmond	30.7	37.4	47.5	63.4	92.4	59.5	71.1	86.5	108.5	146.9
Ryde	38.4	48.0	61.4	80.6	116.8	72.6	87.0	105.4	130.3	185.3
Springwood	41.4	50.8	63.4	84.8	130.0	77.8	93.4	115.6	148.0	206.1
Sutherland	39.1	48.9	63.0	83.6	124.3	75.9	90.2	109.2	139.7	197.0
Sydney 43.6		54.5	68.6	89.5	132.3	88.0	105.0	125.9	158.1	211.2
Wallacia	35.6	43.6	55.4	75.4	113.1	66.4	81.1	98.7	125.8	173.5
Wilberforce	34.6	43.4	54.6	71.2	107.9	68.6	80.7	96.8	120.6	182.0
Illawarra/South Coast										
Albion Park	41.0	51.7	66.1	93.3	147.8	77.1	95.6	120.6	158.0	226.2
Batemans Bay	37.4	47.3	59.7	81.3	124.8	72.0	87.6	107.5	135.7	187.7
Bega	31.8	39.6	51.6	72.4	111.6	61.4	74.8	93.0	119.1	178.9
Cooma	21.2	25.0	31.0	40.4	59.2	40.6	48.0	57.4	72.2	92.4
Helensburgh	58.2	71.5	89.2	121.7	178.4	105.9	126.1	153.8	200.2	291.1
Kiama	46.0	56.0	71.6	93.1	136.5	90.7	106.7	130.1	161.3	220.7
Kangaroo Valley	45.7	58.8	76.4	107.5	166.4	92.8	112.8	141.3	181.6	254.5
Mittagong	37.1	45.1	56.4	74.6	110.4	69.2	82.1	99.6	122.8	164.6
Robertson	52.2	63.8	101.8	116.2	190.6	108.8	133.3	166.2	217.5	310.7
Wollongong	48.1	58.6	75.2	99.2	150.4	96.2	115.4	138.7	176.4	252.2
Northern Tablelands and Northwestern Slopes										
Armidale	32.7	38.3	45.7	56.8	76.5	60.5	68.0	79.2	93.9	119.5
Gunnedah	29.7	35.1	42.2	52.5	69.6	50.2	57.6	66.7	80.1	102.9
Tamworth	31.6	37.3	44.4	54.4	73.0	53.9	61.8	70.8	84.2	107.2
Tenterfield	39.6	46.1	54.4	66.0	88.6	67.6	76.5	89.4	105.4	128.0
Central Tablelands and Central Western Slopes										
Bathurst	27.2	31.6	37.8	46.3	60.7	49.5	55.7	63.4	74.0	94.3
Cowra	28.0	32.2	38.1	46.8	62.2	48.7	55.4	63.6	74.2	93.8
Dubbo	28.4	33.2	40.0	50.5	66.6	47.5	54.4	64.0	77.0	98.7
Southern Tablelands and Southwestern Slopes										
Albury	32.8	38.6	44.0	52.4	66.1	60.5	66.5	74.4	85.7	102.4
Goulburn	26.4	30.2	36.4	44.8	60.8	49.0	54.8	64.3	75.6	97.1
Jindabyne	26.4	31.2	37.6	47.8	64.3	48.2	55.4	64.7	76.7	96.4
Queanbeyan	26.9	31.5	37.6	45.8	62.3	47.0	53.0	61.6	73.7	93.2
Wagga	26.0	30.2	36.0	43.6	56.8	46.4	52.6	59.8	70.4	86.2
Northwestern, Southwestern and Far Western Plains										
Bourke	20.4	25.2	30.6	39.9	56.8	31.2	37.1	44.8	56.5	79.2
Broken Hill	13.4	16.8	21.4	28.9	41.4	21.8	26.7	32.4	40.6	59.7
Griffith	21.0	24.4	28.4	35.3	46.6	34.4	38.8	45.4	53.6	67.4
Moree	26.4	31.8	39.1	50.6	70.0	46.0	53.4	63.3	78.0	104.1
Nyngan	23.1	27.9	34.8	44.8	61.2	36.6	43.6	52.8	66.3	90.4

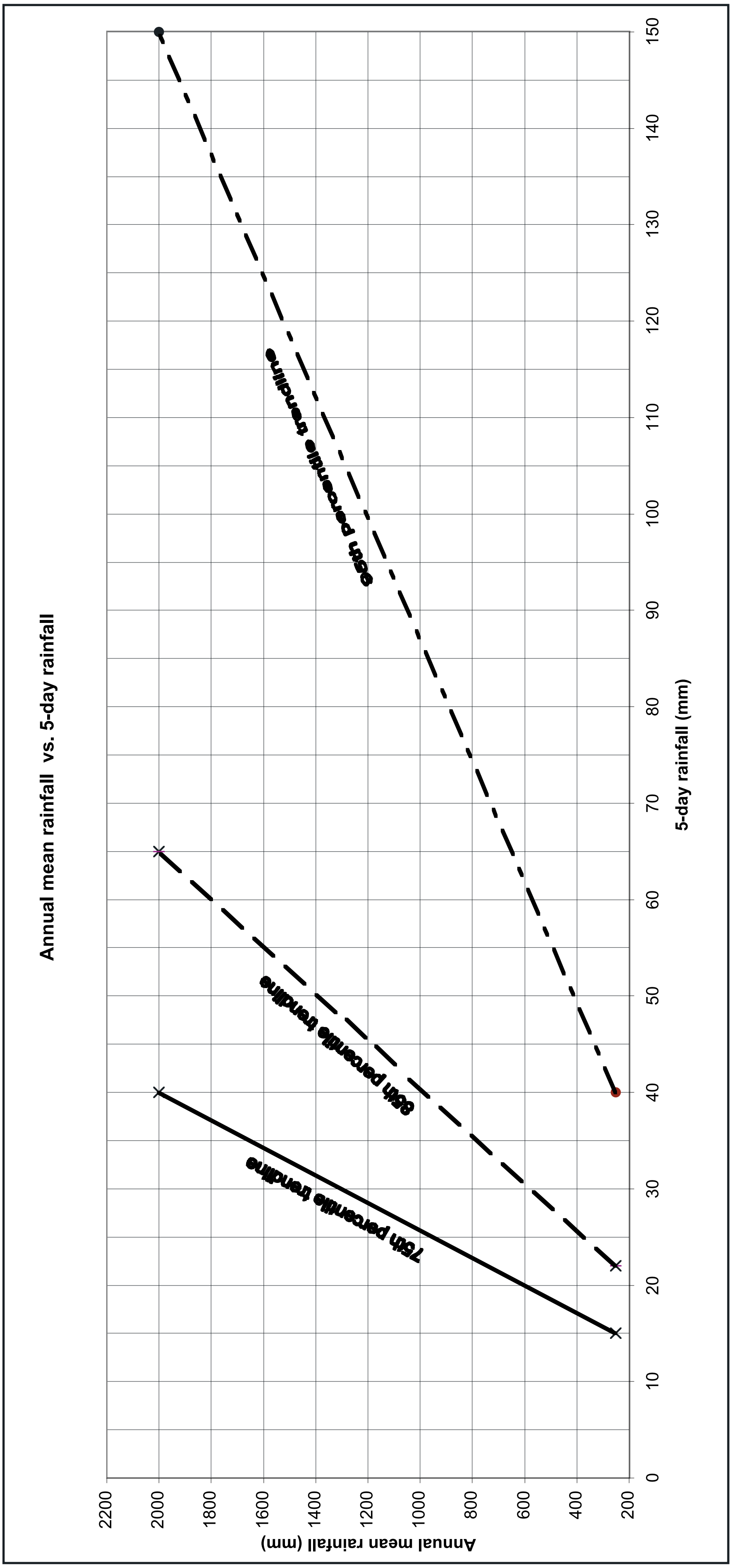


Figure 6.6 The 75th, 85th and 95th percentile 5-day rainfall depth estimated from the annual mean rainfall

On lands of high erosion hazard (typically Soil Loss Classes 5, 6 and 7), designers should always derive a more specific capacity based on two month soil loss as calculated with RUSLE, the following equation (Appendix A):

$$\text{Sediment Zone}_{\textit{Type F / Type D}} = \frac{0.17 \times A\,(R \times K \times LS \times 1.3 \times 1.0)}{1.3}$$

where:

0.17 = one sixth of the computed average annual soil loss

1.3 = the bulk density of the deposited sediment

A = the disturbed catchment area (hectares)

R and K are the RUSLE factors for the site

LS is the RUSLE factor for the site assuming an 80-metre slope length (other slope lengths can be chosen but should be properly justified in erosion control plans).

(iii) In cases where these criteria yield excessively large sediment storage zones, the application of one or more of the following options can help reduce their sizes:

- reducing the catchment areas
- reducing effective slope lengths
- increasing the frequencies of sediment removal.

Standard Worksheets for calculating the size of sediment basins on *Type F* and *Type D* soils are provided in Appendix J.

(j) Management of Sediment Basins on *Type D* and *Type F* Soils:

(i) With basins that capture runoff from *Type F* soils, stormwater in the settling zone should be drained or pumped out within that time period adopted in the design of the basin (5 days in most cases, but in the range of 2-20 days as site conditions allow) following rainfall if the nominated water quality targets can be met. Flocculation should be employed where extended settling is likely to fail to meet this objective within the nominated time period.

(ii) Because *Type D* soils contain a significant level of dispersible materials, dosing the captured stormwater with a chemical agent to facilitate settling and help manage the turbidity of discharged stormwater is necessary (Appendix E). For larger land disturbance activities, consideration should be given to establishing a site-specific relationship between suspended solids concentration (also reported as mg/L non-filtrable residue (NFR) and turbidity (measured in nephelometric turbidity units (NTU)) to allow a more rapid assessment of stormwater quality at the site. Samples collected for this purpose should be taken following a reasonable settling period.[14]

14. Most Australian streams naturally carry sediment loads at some time or another and have reached equilibrium under such conditions. Excessively reducing these loads for extended periods can cause such streams to become "hungry" and erode their own bed and banks.

(iii) Ensure only the clear (<50 mg/L suspended solids) supernatant waters are discharged from the settling zones of these structures. To help in this, use a floating inlet to any pump to reduce the opportunity for picking up any settled sediment

(iv) Sediment removed from sediment storage zones where the soils are *Types D* or *Type F* often requires a long time to dry out before it can be handled properly. Consideration should be given to this matter well before maintenance is required. Sediment must be stored, even temporarily, in ways that will not result in sediment pollution to downslope lands and waterways.

6.3.5 Capacity of Basins for *Type C* Soils

(a) Sediment retention basins on *Type C* soils can be wet or dry basins (SD 6-1, SD 6-2, SD 6-3, SD 6-4 and SD 6-5).

(b) The basic premise of sediment retention basins on *Type C* soils is that an acceptable discharge water quality can be achieved by providing a relatively short residence time for the settling of a design particle (usually 0.02 mm) (Table 6.1).

(c) The design storm event for basins on *Type C* soils is taken as the 3-month ARI flow, unless specified differently in the local Council's "Stormwater Management Plan". This design flow should be estimated for individual basin designs, but is commonly about half of the 1-year ARI flow unless the local consent authority specifies differently.[15]

(d) In determining peak rates of flow, the minimum critical duration (or time of concentration (tc)) likely to apply throughout the construction period should be adopted. Adopt the recommendations shown at Appendix F for calculation of peak flow runoff coefficients (C10) where the lands are disturbed by removal of vegetation and topsoil (common on building and road construction sites and mining sites). Where the lands are not so disturbed, apply the criteria shown in Pilgrim (1998).

(e) Three components need to be determined for the settling zone of a sediment basin on *Type C* soils, namely: the surface area, depth, and length:width ratio.[16]

(i) The basin surface area should be equal to or greater than the following:

- 4,100 square metres per cubic metre of discharge waters per second in the design storm event where the design particle is 0.02 mm[17]

---

15. Generally, more than 90 percent of average annual runoff occurs as flows with an ARI of three months or less.
16. Most laboratory studies that have investigated sediment basin design have occurred in environments where such factors as short-circuiting, turbulence, bottom scour, outlet design and temperature had minimal if any effect on their efficiencies. The reality is such an ideal basin is never constructed, despite very good intentions. Further, the particles being tested are usually perfectly spherical, have uniform densities and cannot interact with one another, factors that do not occur with real soils.
17. A basin surface area of 4,100 square metres is based on the equation A = 1.2 Q / Vs where A is the required basin surface area (m^2), Q is the peak flow rate in the design storm (m^3/sec) and Vs is the settling velocity of the design particle (0.00029 m/s for a particle of 0.02 mm diameter (Goldman *et al* (1986)).

- 635 square metres per cubic metre of discharge waters per second where it is 0.05 mm
- 170 square metres per cubic metre of discharge waters per second where it is 0.1 mm.

In most cases, the design particle should be taken as 0.02 mm. However, the larger sizes can be considered where at least 90 percent of those particles coarser than 0.02 mm are, in fact, coarser than 0.05 or 0.1 mm.

(ii) The depth of the settling zone should be at least 0.6 metres, sufficient to provide a cross-sectional flow area that limits flow velocities to values unlikely to scour settled sediment in a 1-year ARI flow, namely 0.07 metres per second for a particle of 0.02 mm diameter. If a less frequent storm event has been adopted (greater than the 1-year ARI) or if site constraints limit the depth of the sediment retention basin, a check should be made to ensure that the average flow velocity in the design storm event does not exceed the scour velocity for a particle of 0.02 mm diameter.[18]

(iii) Length:width ratios should be 3 to 1 or greater as discussed above at Section 6.3.3(i).

(iv) On lands of low erosion hazard, as determined by the simple procedure described in Section 4.4.1, the capacity of the sediment storage zones on Type C soils can be determined simply as 100 percent of the settling zone capacity. On lands of high erosion risk (typically Soil Loss Classes 5, 6 and 7), or as an alternative in any case, designers can derive a more specific capacity using the following equation (Appendix A):

$$\text{Sediment Zone}_{\mathit{Type\ F\ /\ Type\ D}} = \frac{0.17 \times A\ (R \times K \times LS \times 1.3 \times 1.0)}{1.3}$$

where:
0.17 = one sixth of the computed average annual soil loss
1.3 = the bulk density of the deposited sediment
A = the disturbed catchment area (hectares)
R and K are the RUSLE factors for the site
LS is the RUSLE factor for the site assuming an 80-metre slope length (other slope lengths can be chosen but should be properly justified in erosion control plans).

(v) On Soil Loss Classes 5, 6 and 7 lands, designers should derive the specific capacity using the above equation only.

(vi) In cases where these criteria yield excessively large sediment storage zones, the application of one or more of the following options can help reduce their sizes:

- reducing the catchment areas

18. Metcalf and Eddy (1991) suggest a scour velocity of 0.07 m/s for a particle of 0.02 mm diameter.

- reducing effective slope lengths
- increasing the frequencies of sediment removal.

Standard Worksheets for calculating the size of sediment basins on *Type F* and *Type D* soils are provided in Appendix J.

(f) Techniques for dewatering *Type C* sediment basins commonly involve the use of needle-punched geotextile, sand or gravel as mediums near the outlet. The use of sand or gravel as a filtering medium is not encouraged because it is more difficult to maintain. Geotextile should be placed:

  (i) on the upstream side of gabions (figure 6.7);

  (ii) on 50 to 75 mm aggregate (maximum batter gradient l.5(H):l(V)) placed on the upstream side of a wall constructed from local rock materials (figure 6.8); or

  (iii) around a perforated riser structure. An air gap is essential between the riser structure and the geotextile to ensure free drainage of "dry" basins. The geotextile should not be in close contact with the riser (SD 6-3).

(g) Whichever method is used, reverse flush or replace the geotextile each time sediment is removed from the basin to reduce the likelihood of the pores blocking and becoming essentially a wet basin. Consequently, do not sandwich the fabric between gabion baskets.

(h) Operation of basins on *Type C* soils should ensure that, where possible, water has drained from the settling zone by the beginning of the next storm event. This can be achieved with dry, above-ground basins, through use of geotextile filters or similar mechanisms.[10]

6.3.6 Infiltration sumps

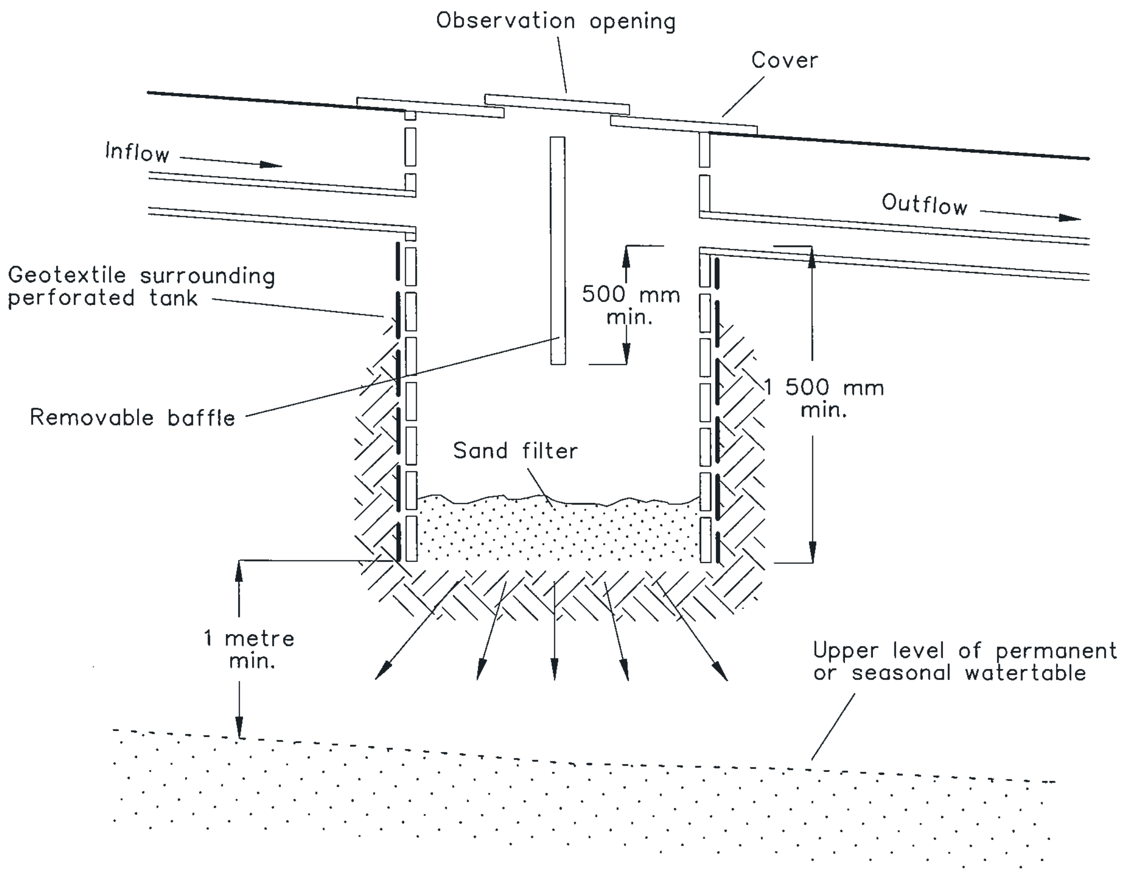
(a) Infiltration sumps (Standard Drawing SD 6-6) are subsurface facilities that collect stormwater for filtration of sediment and infiltration of stormwater to the watertable. They are used as an alternative to the regular sediment retention structures described above where:

- construction of regular structures can be impractical, (e.g. where most of the site will be disturbed for building purposes)
- the soils are well drained (Soil Hydrologic Groups A or B)
- the watertable (seasonal or permanent) is more than 2 metres below the floor of the structure (Chapter 3)
- addition of water to the watertable will not affect salinity levels – testing soils for the presence of soluble salts is essential (Chapter 3)
- groundwater cannot be contaminated
- catchment areas are very small.

Infiltration technology is not supported by the DIPNR unless the soils and groundwater have been properly assessed for their capability.

Figure 6.7 A sediment retention basin constructed on Type C soils with gabion baskets in very steep country. The structure has recently been converted to a constructed wetland.

Figure 6.8 A sediment retention basin constructed on Type C soils from local sandstone gibbers



Construction Notes

1. Join the inlet to the polluted supply taking any suitable step to remove bulky material before it can enter the sump.
2. Connect the outlet to a safe disposal area following the ESCP/SWMP.
3. Place a geotextile liner on the outside of the pit.
4. Install a removable baffle, central to the inflow/outflow and normal to the direction of flow, ensuring that it reaches 500 mm below the invert of the outlet pipe.
5. Install a cover over the pit with an observation port and access cover.

INFILTRATION SUMP

SD 6-6

(b) Generally, infiltration sumps are constructed from floorless plastic or metal tanks with perforated sides and surrounded with geotextile to filter the water before it enters the soil. The perforations should comprise at least 2.5 percent of the total surface area (preferably 7.5 percent), and be evenly placed to ensure water is delivered over the entire facility. Provide an aggregate bed at least 600 mm wide between the tank and the surrounding soil materials (aggregate coarser than the perforations).

(c) The capacity of the sump should accord with that for basins on *Type C* soils, but should have a settling zone depth greater than 1.5 metres.

(d) Provide an accessible observation point to allow estimates to be made about how quickly the tank will dewater following a storm event and to measure sediment levels. Inspect them after every storm event large enough to produce runoff. Pump them out 36 to 48 hours after each storm event if:

- fines block the geotextile and/or floor of the structure
- the infiltration rate drops below 15 mm per hour.

Of course, flocculation might be necessary first.

(e) Maintain design capacity always through regular removal of sediment.

6.3.7 Sediment Filters

(a) Sediment filters (also called sediment retention traps) are temporary measures used in mitigation of sediment pollution to downslope lands and waterways. They are relatively effective at retaining suspended solids coarser than 0.02 mm. Many finer particles and most soluble materials pass through them. They are simple to construct, relatively inexpensive and easily moved as development proceeds.

(b) Materials used in their construction include one or more of straw bales, woven geotextile, earth, rock or suitable crushed concrete products. Generally, actual choice is dependent on constraints imposed by the design criteria (including maintenance needs), availability of materials, cost and site conditions. Straw bales should not be used where they cannot be properly embedded into the ground unless alternative measures are taken to prevent polluted water passing under them.

(c) Place them to keep sediment as close to its source as possible.

(d) Maintain sediment filters so that no more than 30 percent of their design capacity is lost to accumulated sediment and construction materials are replaced when functionality is lost. Dispose of any waste material in an approved manner and where further pollution to downslope lands and waterways should not occur.

(e) Some filters are constrained by external design criteria, including sediment fences and straw bale barriers (Standard Drawings SD 6-7 and SD 6-8 and figure 6.9). They should be able to withstand the erosive forces from the design storm event, usually the 10-year ARI time of concentration event and, therefore, should not be placed in areas of concentrated water flows. Catchment areas of sediment fences can be constrained by building them along the contour with periodic small returns (figure 6.10) creating several subcatchments. Because these systems are prone to failure in relatively small storm events, subcatchment areas should be sufficiently small to constrain maximum flows to 50 litres per second in the design storm event should all water discharge at one point.

Figure 6.9 Sediment fence constructed below a fill batter – the sediment pollution is the result of a recent storm event

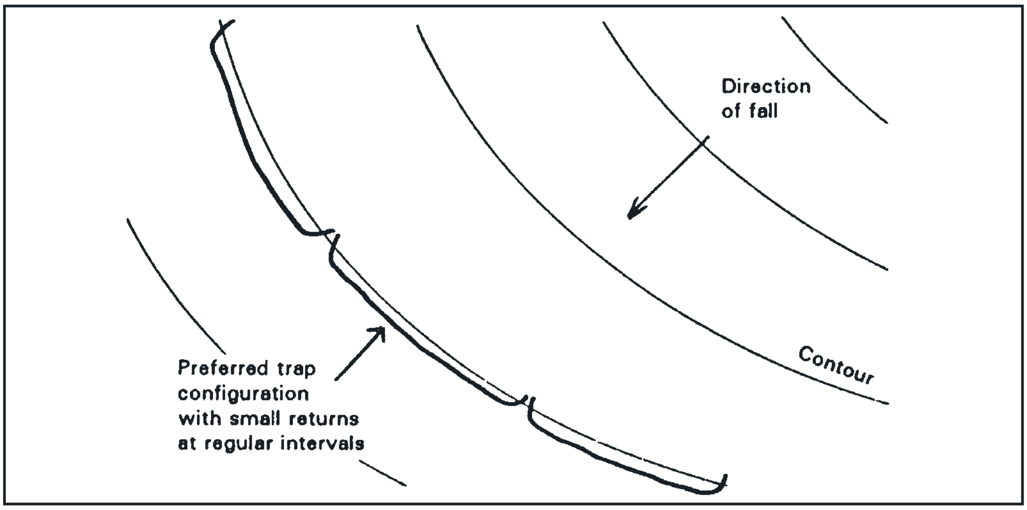
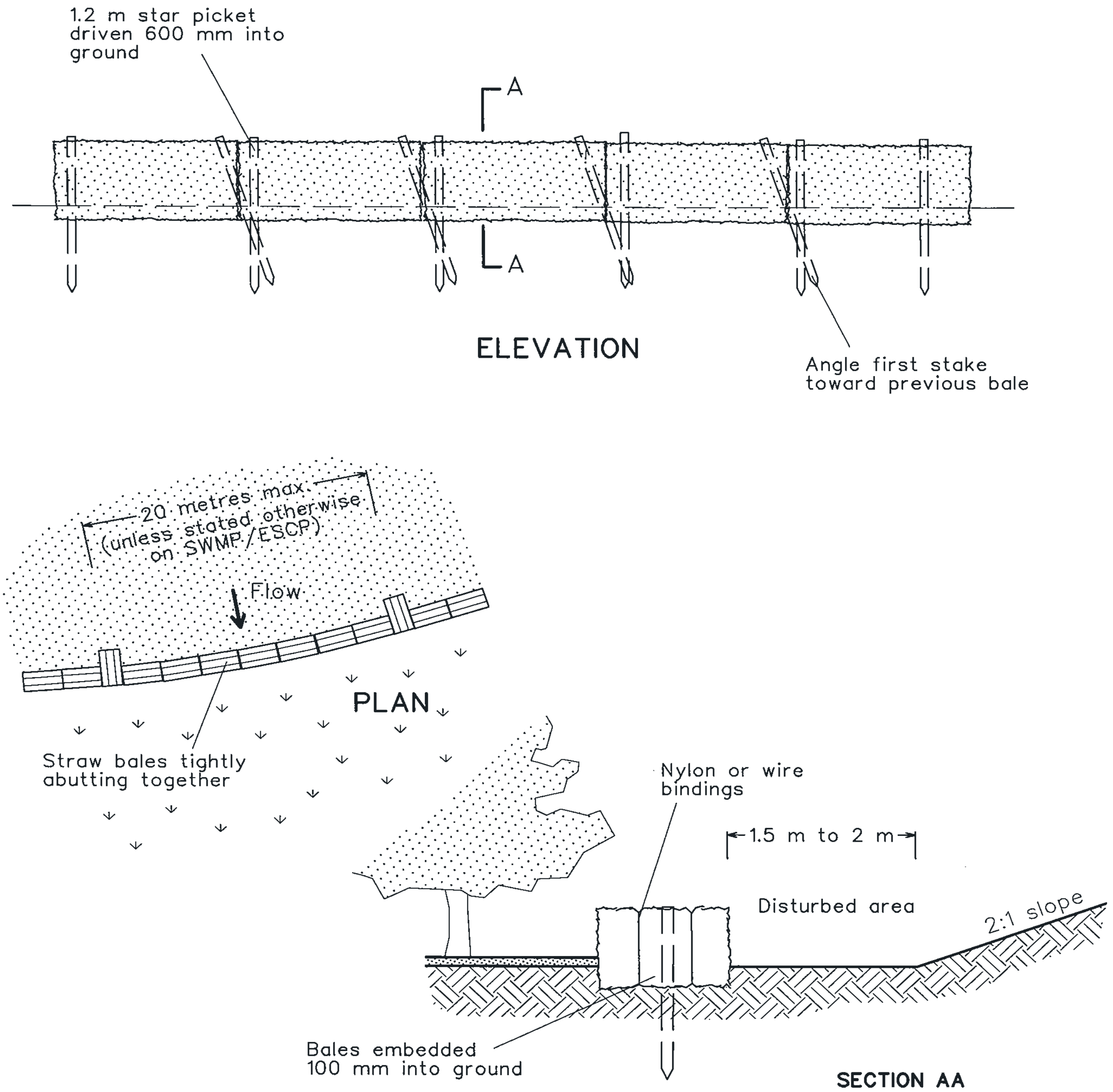


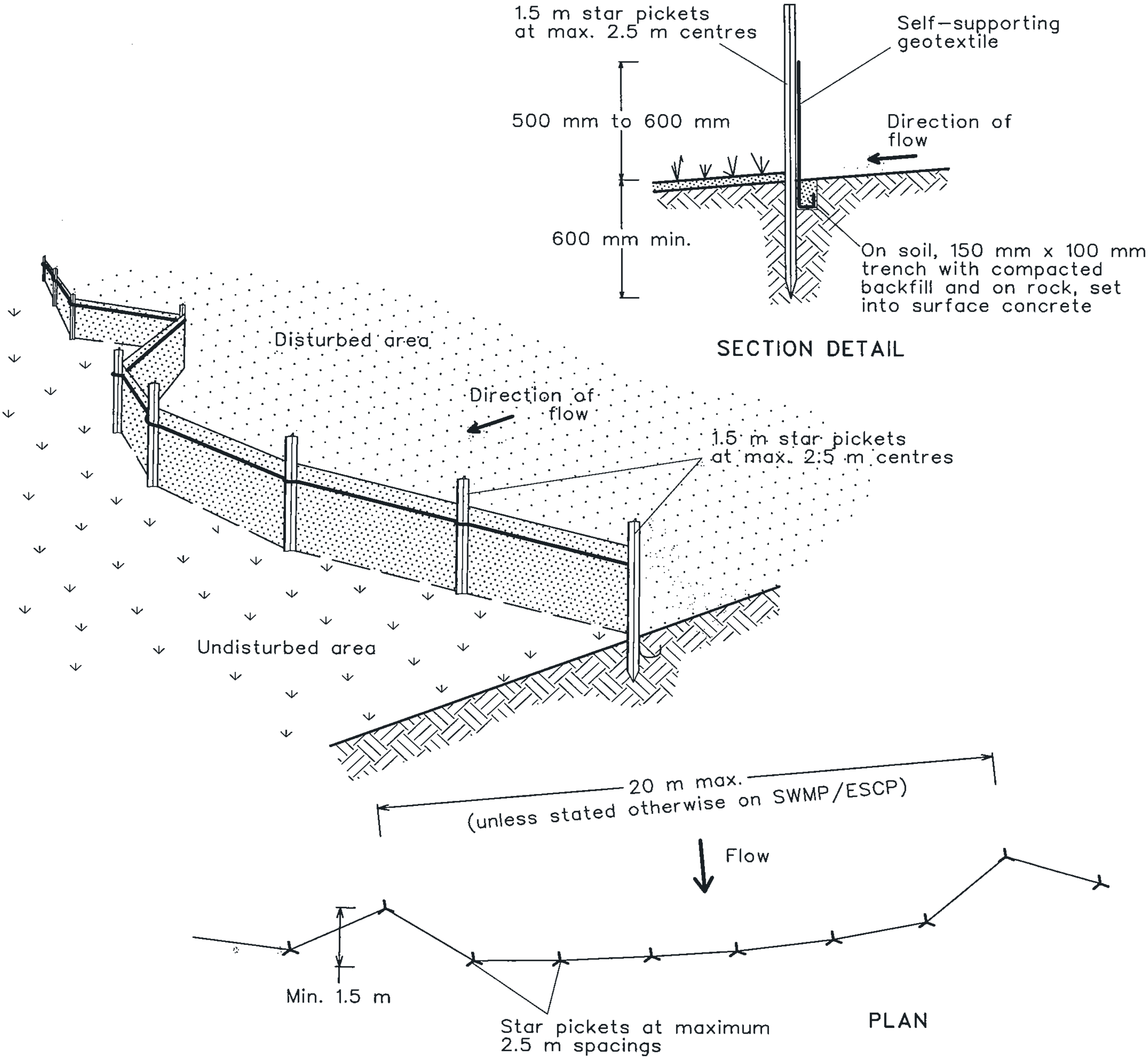
Figure 6.10 Preferred sediment fence configuration



Construction Notes

1. Construct the straw bale filter as close as possible to being parallel to the contours of the site.
2. Place bales lengthwise in a row with ends tightly abutting. Use straw to fill any gaps between bales. Straws are to be placed parallel to ground.
3. Ensure that the maximum height of the filter is one bale.
4. Embed each bale in the ground 75 mm to 100 mm and anchor with two 1.2 metre star pickets or stakes. Angle the first star picket or stake in each bale towards the previously laid bale. Drive them 600 mm into the ground and, if possible, flush with the top of the bales. Where star pickets are used and they protrude above the bales, ensure they are fitted with safety caps.
5. Where a straw bale filter is constructed downslope from a disturbed batter, ensure the bales are placed 1 to 2 metres downslope from the toe.
6. Establish a maintenance program that ensures the integrity of the bales is retained - they could require replacement each two to four months.

STRAW BALE FILTER **SD 6-7**



Construction Notes

1. Construct sediment fences as close as possible to being parallel to the contours of the site, but with small returns as shown in the drawing to limit the catchment area of any one section. The catchment area should be small enough to limit water flow if concentrated at one point to 50 litres per second in the design storm event, usually the 10-year event.

2. Cut a 150-mm deep trench along the upslope line of the fence for the bottom of the fabric to be entrenched.

3. Drive 1.5 metre long star pickets into ground at 2.5 metre intervals (max) at the downslope edge of the trench. Ensure any star pickets are fitted with safety caps.

4. Fix self-supporting geotextile to the upslope side of the posts ensuring it goes to the base of the trench. Fix the geotextile with wire ties or as recommended by the manufacturer. Only use geotextile specifically produced for sediment fencing. The use of shade cloth for this purpose is not satisfactory.

5. Join sections of fabric at a support post with a 150-mm overlap.

6. Backfill the trench over the base of the fabric and compact it thoroughly over the geotextile.

SEDIMENT FENCE

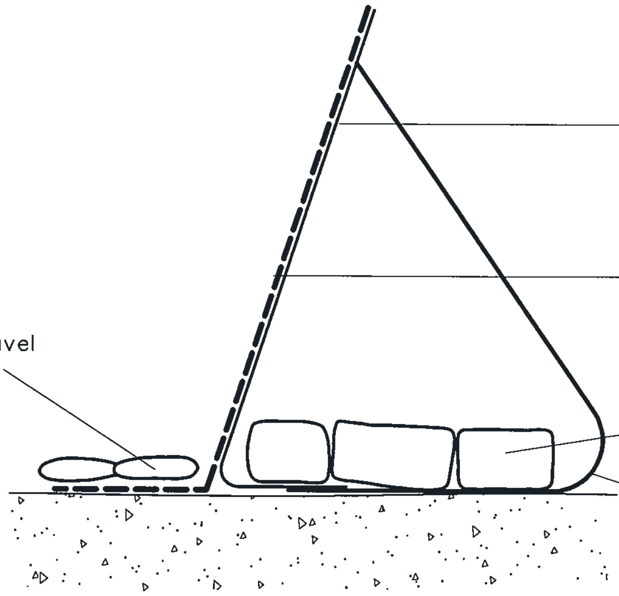
SD 6-8

(f) Special measures must be installed for trapping sediment in intertidal zones, e.g. sediment fences as shown in Standard Drawing SD 6-9.

(g) Floating sediment fences can be constructed below the intertidal zone, providing sufficient water depth is always available for the boom to float (Standard Drawing SD 6-10). They can be used to surround barges when transferring materials to and from the shore or when carrying out dredging activities. Maintenance of fixed of floating sediment fences should be undertaken only at low tides.

(h) Other filters are not constrained by external design criteria, including inlet filters (figure 6.11 and Standard Drawing SD 6-11 and SD 6-12). They are among the least effective of all BMPs at mitigating sediment pollution because their design does not take into account runoff volume. Their installation at any particular location is a matter for the site manager on a day-to-day basis as an informal part of the sediment control program and not normally detailed on any P1an. Nevertheless, they should be placed so that they are unlikely to divert water from its intended course in a very large storm event.

Figure 6.11 Filter roll at kerb-side sump



Construction Notes

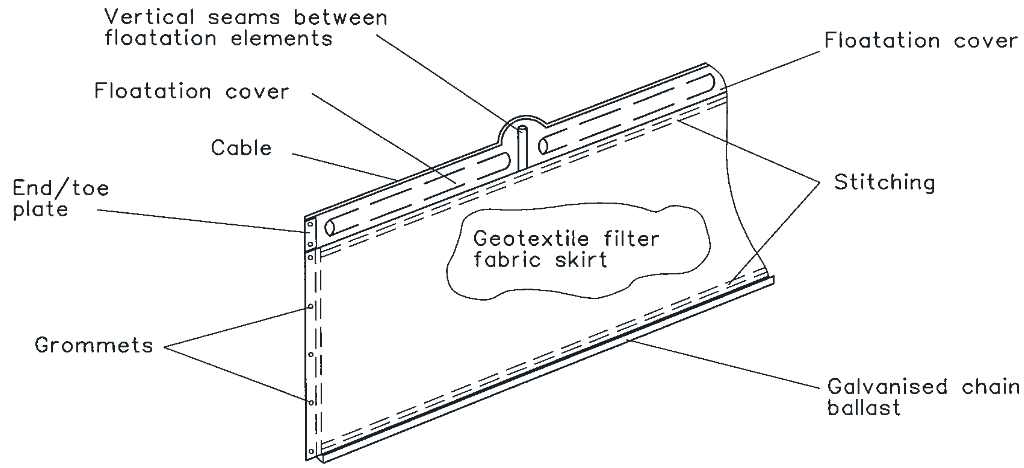
1. Install this type of sediment fence when use of support posts is not desirable or not possible. Such conditions might apply, for example, where approval is granted from the appropriate authorities to place these fences in highly sensitive estuarine areas.

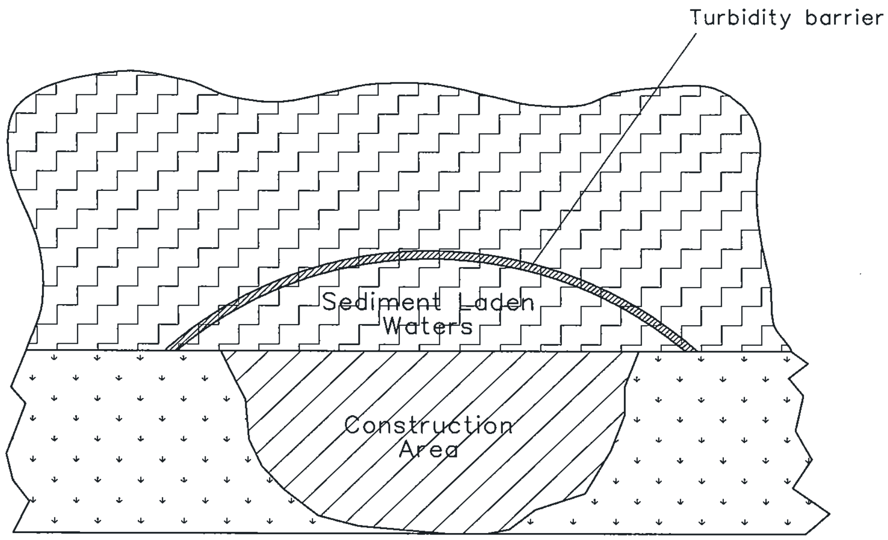
2. Use bent trench mesh to support the F82 welded mesh facing as shown on the drawing above. Attach the geotextile to the welded mesh facing using UV resistant cable ties.

3. Stabilise the whole structure with sandbag or rock anchoring over the trench mesh and the leading edge of the geotextile. The anchoring should be sufficiently large to ensure stability of the structure in the design storm event, usually the 10 year event.

ALTERNATIVE SEDIMENT FENCE

SD 6-9

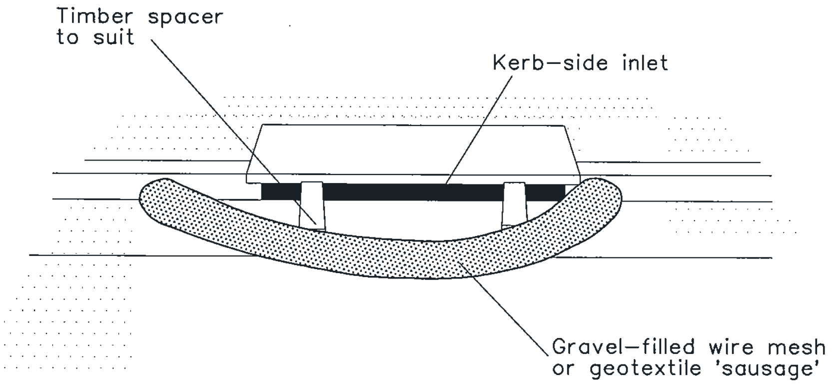


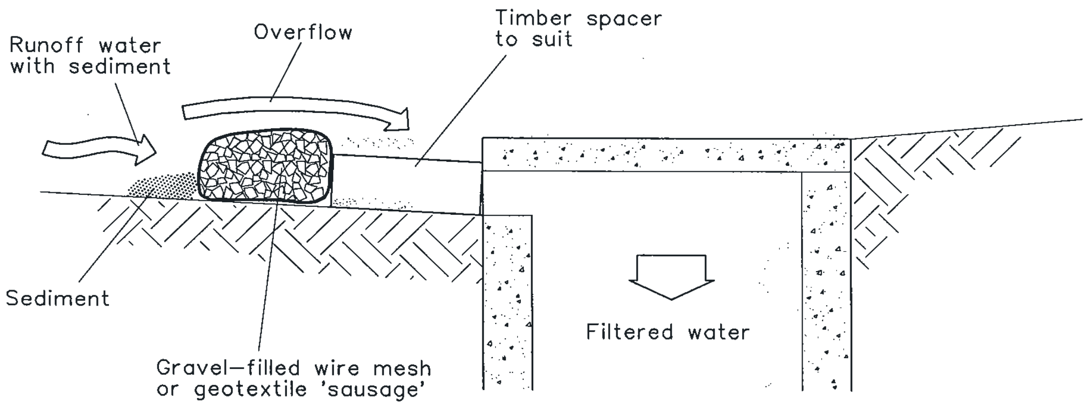


Construction Notes

1. Use turbidity barriers only where high flows are unlikely to remove accumulated sediment and/or move the curtain significantly.
2. Where the barrier is to remain in place for more than one month, ensure the floatation cover is a UV-resistant, durable material.
3. Use only closed cell foam or foam-filled PVC piping as floatation elements. Do not use unfilled pipes.
4. Use only woven or heat-set non woven geotextiles. Needle-punched, non woven geotextiles can become fouled with debris that fray and delaminate them as they move with the waves or currents.
5. Remove captured sediment before the barrier is decommissioned.
6. In tidal areas, ensure the barrier can rise and fall without being moved from its position.

TURBIDITY BARRIER **SD 6-10**



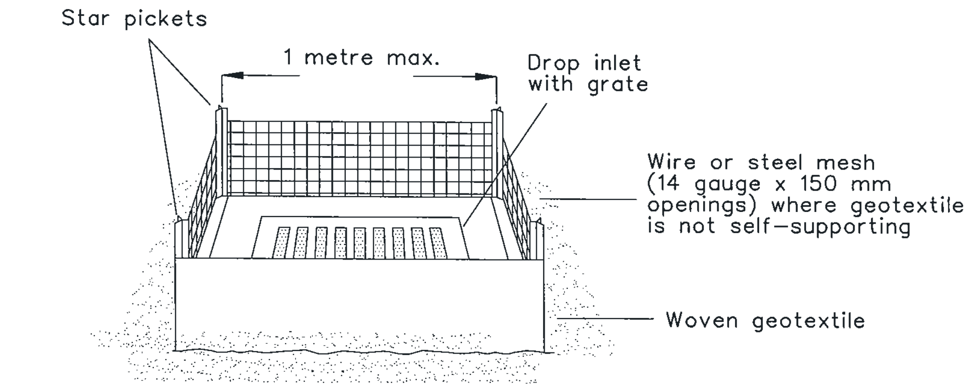


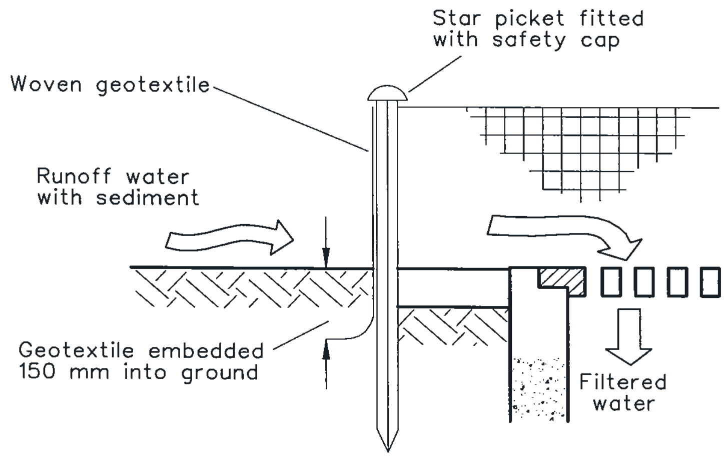
NOTE: This practice only to be used where specified in an approved SWMP/ESCP.

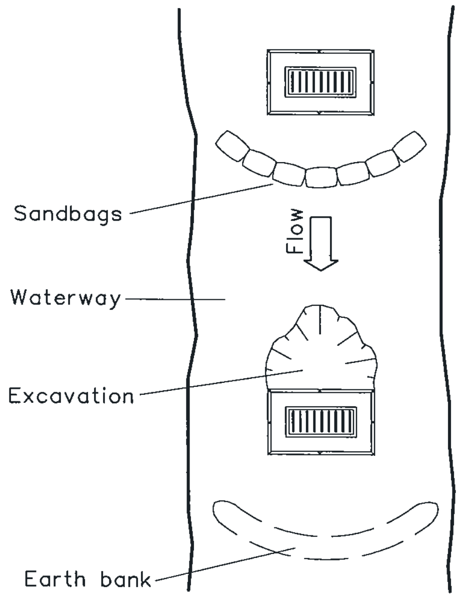
Construction Notes

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

MESH AND GRAVEL INLET FILTER SD 6-11







For drop inlets at non-sag points, sandbags, earth bank or excavation used to create artificial sag point

Construction Notes

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

GEOTEXTILE INLET FILTER

SD 6-12

(i) Developed areas, especially inner city areas with space constraints, need careful management of activities to prevent sediment pollution. This is particularly evident where building materials such as sand, fill material and topsoil, etc. are deposited near areas of concentrated water flows, e.g. on footpaths or side of the roads. In such instances, it is essential that stormwater flows in gutters and other surface drains are not impeded nor can they result in materials being washed into drainage systems. Downstream pit protection should be implemented and ongoing maintenance should be provided; pedestrian and vehicular safety and warning devices should be erected.

6.3.8 Filter Strips

(a) Strips of vegetation left or constructed downslope from earthworks provide a simple method of trapping coarse sediment in most storm events other than very large ones. This assumes that, where this vegetation is to be retained, it will have sufficient time to "recover" before the next load of sediment-laden water enters the site.

(b) The following factors should be considered in their design:

(i) the amount of sediment that might be stored in the area above the filter; and

(ii) the width of vegetation in the filter:

- required to filter coarse sediment (usually the upper section)
- required filter some of the finer sediment (usually the lower section).

(c) Native vegetation in riparian zones should not be used as filter strips. Only separate dedicated buffer zones upslope from riparian lands should be used.

(d) Karssies and Prosser (2001) suggest that the following amounts of sediment can be stored above the filter:

- where slopes are less than 4 percent, up to 50 tonnes per hectare per 100 metres length
- where slopes are between 4 and 7 percent, up to 15 tonnes per 100 metres length
- where slopes are between 8 and 10 percent, up to 10 tonnes per 100 metres length.

That significant amounts of sediment are stored in the area above the filter is shown in Figure 6.12.

(e) The suggested widths of grass filters for calculated values of annual soil losses (Appendix A) are in Table 6.4. Note that, generally, vegetated filter strips are most effective where the average annual soil losses are low and the sediment is relatively coarse; they are least effective where the calculated average annual soil losses are

Table 6.4 Recommended Grass Filter Strip Widths for Typical Values of Calculated Annual Soil Loss (Karssies and Prosser, 2001)

Soil loss (t/ha/yr)	Filter strip gradient (%)									
	1	2	3	4	5	6	7	8	9	10
1	2m	2 m	2 m	2m	2 m	2 m	2 m	2 m	2 m	2 m
2	2m	2 m	2 m	2m	2 m	2 m	2 m	2 m	2 m	2 m
5	2m	2 m	2 m	2m	3 m	3 m	3 m	4 m	4 m	4 m
10	2 m	2 m	2 m	5m	6 m	6 m	7 m	7 m	7 m	7 m
20	3 m	9 m	11 m	12 m	12 m	13 m	13 m	13 m	13 m	14 m
30	9 m	15 m	17 m	18 m	19 m	19 m	19 m	20 m	20 m	20 m
40	15 m	21 m	23 m	24 m	25 m	25 m	26 m	26 m	26 m	26 m
50	22m	28 m	NR	NR	NR	NR	NR	NR	NR	NR
60	28m	NR	NR	NR	NR	NR	NR	NR	NR	NR
70	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR

more than about 40 tonnes per hectare per year and/or the sediment is relatively fine.

(f) The best vegetation cover is one that provides a relatively uniform dense ground cover, e.g. sward-forming grasses about 150 mm high.

(g) A 400-mm wide grass strip can be installed next to a kerb to stabilise the interface between the kerb and footway (Standard Drawing SD 6-13). Also, it can provide worthwhile sediment trapping value in very small storm events.

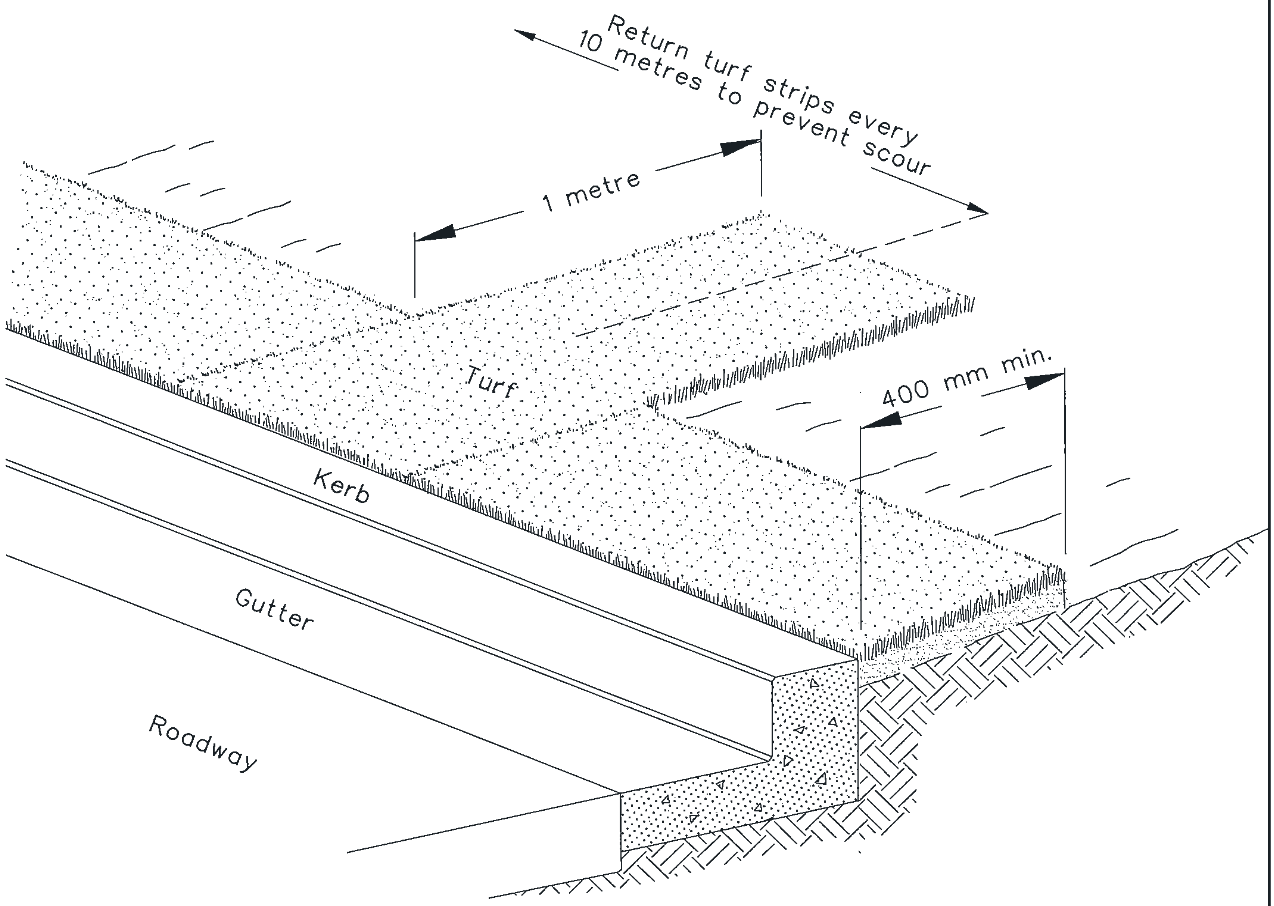
6.3.9 Stabilised Site Access

(a) Access to sites should be stabilised (Standard Drawing 6-14) to reduce the likelihood of vehicles tracking soil materials onto public roads and ensure all-weather entry/exit. Such areas should be at least 3 metres wide (or 2.4 metres per lane) and constructed with maximum 75 mm aggregate at least 15 metres long and 200 mm thick, underlain by needle-punched geotextile.

(b) It is very important that:

- surface water flows are diverted from the area
- the structures are placed so that bypassing them is not possible for vehicles
- they are maintained in an effective condition through removal of sediment and/or addition of extra aggregate.

(c) A variation on the design shown in SD 6-14 where tracking of sediment onto local public roads is likely to be a problem is the use of cattle grids installed under water (figure 6.13).



Construction Notes

1. Install a 400-mm minimum wide roll of turf on the footpath next to the kerb and at the same level as the top of the kerb.
2. Lay 1.4 metre long turf strips normal to the kerb every 10 metres.
3. Rehabilitate disturbed soil behind the

KERBSIDE TURF STRIP

SD 6-13

Figure 6.12 A kerbside turf strip showing sediment stored in the area above the filter.

Figure 6.13 A simple wash-down system.

(d) In addition, the following limitations should be considered:

(i) Wash-down areas and stabilised accesses require collection and treatment of waste water;

(ii) Ideally, both should be built on level areas; and

(iii) Supplementary, street sweeping on adjacent roads might still be required.

6.3.10 Control of Wind Erosion

(a) Research (Livingston, *et al.*, 1988) has shown that average dust emission rates of over 2.5 tonnes per hectare per month occur at urban construction sites.

(b) Various measures are available to minimise such emissions, including:

(i) limiting the area of lands exposed to erosive forces through:
- phasing works (Chapter 4)
- provision of a protective ground cover including mulches, vegetation (Chapter 7), organic binders or dust retardants
- keeping the ground surface damp (not wet)
- leaving the surface in a rough cloddy condition to increase roughness and slow surface wind speed;

(ii) limiting traffic movement on any disturbed areas;

(iii) applying a suitable hydraulic soil stabiliser to the soil surface to reduce the *C*-factor (Appendix A); and

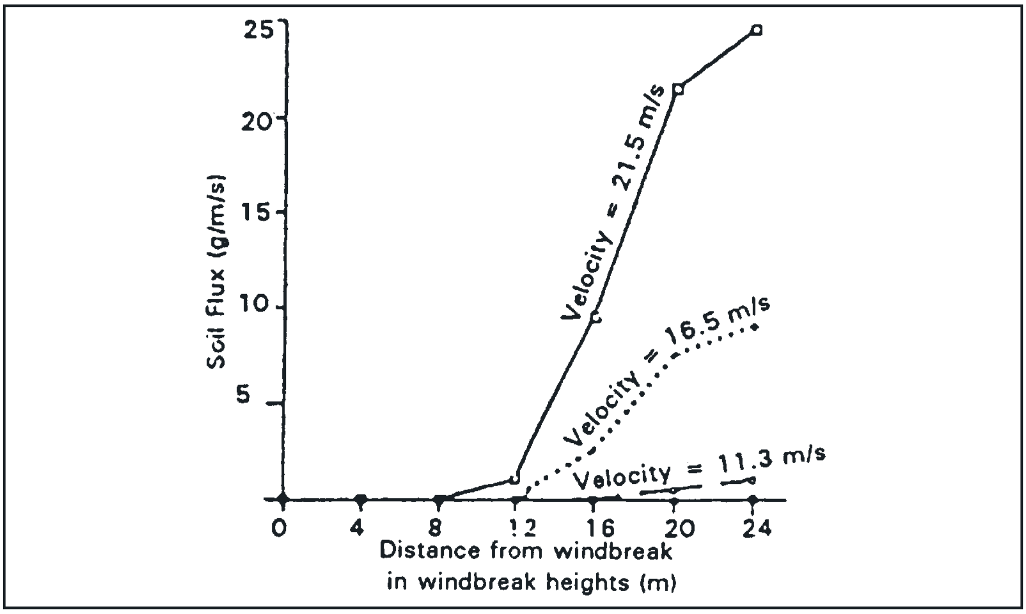


Figure 6.14 Effect of distance from windbreak on soil loss, wind blowing at less than 90° to the windbreak (Leys, 1991)

(iv) on building sites where 1,500 to 5,000 square metres are to be disturbed, installing a 40 percent porous, open-weave barrier fence (Standard Drawing SD 6-15) on the windward side.[19]

6.4 Constructed Wetlands

6.4.1 Preamble

(a) This section has been included to help decide:

- whether or not a constructed wetland might be required on a site
- where it might be placed
- its approximate capacity and dimensions.

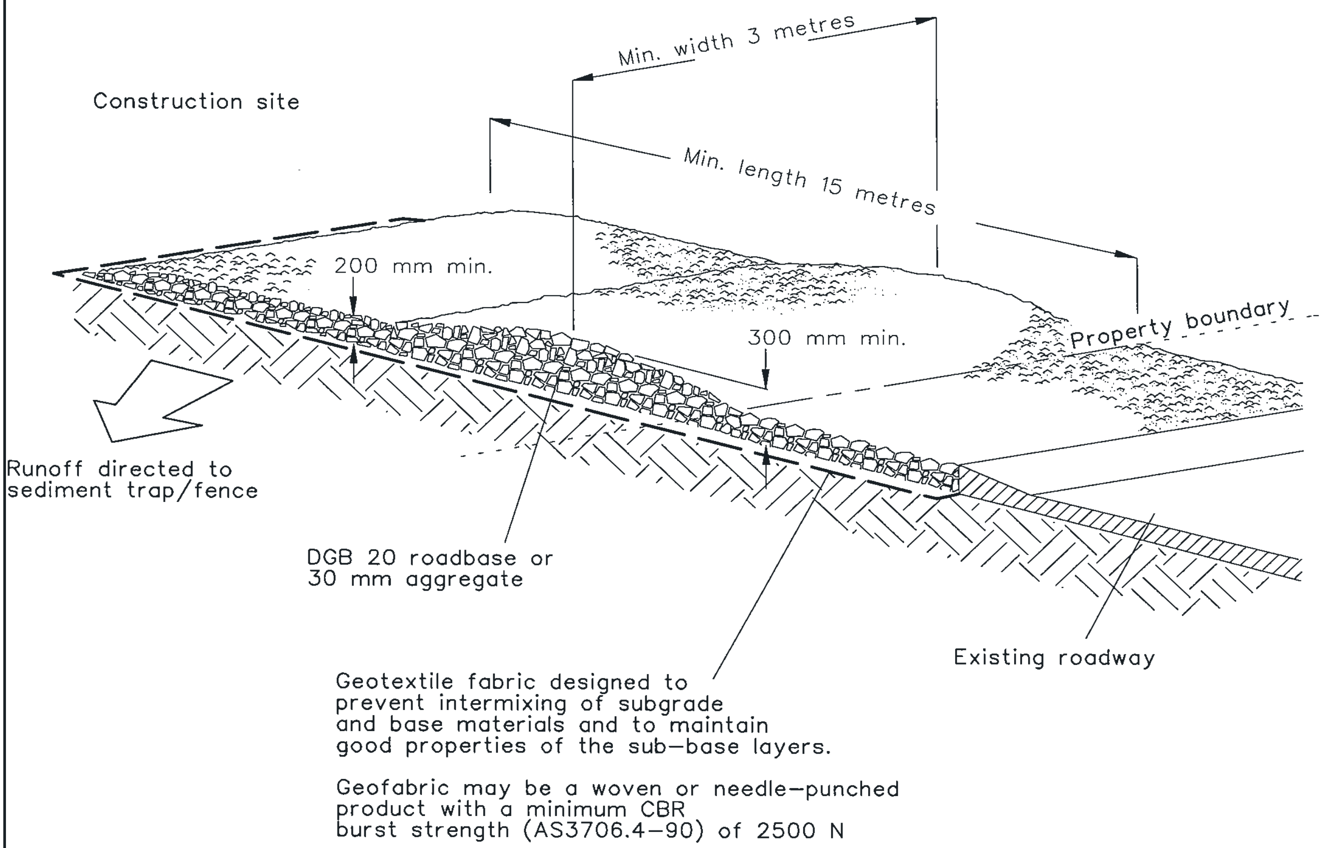
(b) Such information might affect choice, location and design criteria for BMPs to be used during the construction phase, e.g. sediment retention basins. Further, if a wetland is required on site after a land disturbance phase, its use as a temporary sediment retention basin during the disturbance phase should be considered. This section is not intended to aid the detailed design of constructed wetlands, nor does it discuss in sufficient detail their ecology, aesthetics or economics. Information on these topics can be found in DLWC (1998).

(c) Note that neither sediment basins nor wetlands should be built on line on watercourses.

(d) The design criteria and construction details for constructed wetlands are still in an evolutionary phase. Consequently, several national and State-based organisations are involved in the collection of information and investigations into various alternatives for improvement in water quality. Unfortunately, their task is made difficult by the scarcity and variability of existing Australian data. The information presented here should be interpreted in this light.

(e) Nevertheless, constructed wetlands should be considered, especially when undertaking the structure planning of new urban release areas. Their incorporation into infill developments should also be considered, although design opportunities are often limited. Constructed wetlands are most appropriate in areas where receiving water quality problems are, or are likely to result from high nutrient levels.

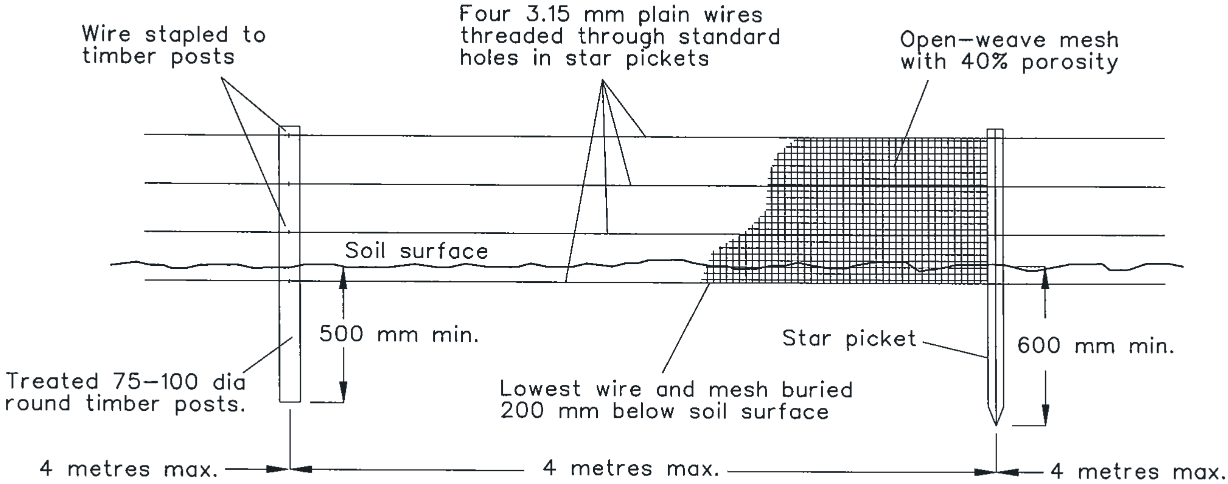
19. Such fences are effective to a distance of 15 times their height, assuming an acceptable soil flux of five grams per metre per second (figure 6:14). Sand and silt particles become airborne at surface wind speeds of about 10 metres per second (36 kilometres per hour) and exceed acceptable limits for airborne dust above 40 kilometres per hour.

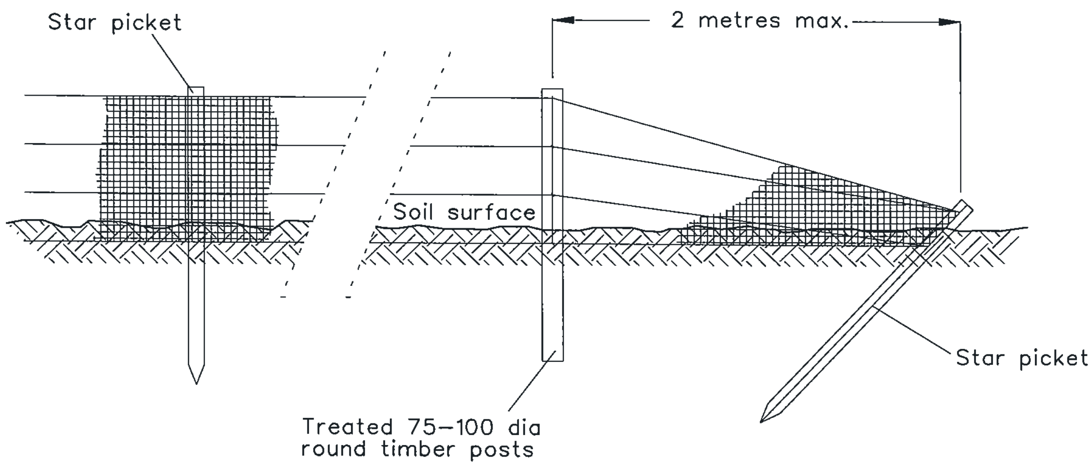


Construction Notes

1. Strip the topsoil, level the site and compact the subgrade.
2. Cover the area with needle-punched geotextile.
3. Construct a 200-mm thick pad over the geotextile using road base or 30-mm aggregate.
4. Ensure the structure is at least 15 metres long or to building alignment and at least 3 metres wide.
5. Where a sediment fence joins onto the stabilised access, construct a hump in the stabilised access to divert water to the sediment fence

STABILISED SITE ACCESS **SD 6-14**





Construction Notes

1. Install the fence to the height specified in the ESCP/SWMP.
2. Cut a channel 200 mm deep along the fence line.
3. Place wire and light resistant, open-weave polymer mesh with 40 percent porosity on the prevailing wind side of fence.
4. Fasten the mesh to all wires using ring fasteners at 100 mm to 150 mm intervals on top wire and 300 mm intervals on other wires.
5. Use one 75-mm to100-mm diameter treated round timber post every 20 metres.
6. Where star pickets are used, ensure they are fitted with safety caps.

CONTROL OF WIND EROSION **SD 6-15**

6.4.2 Introduction

(a) Constructed wetlands are purpose-built structures, predominantly constructed with natural materials of soil, water and biota. They mimic the desired processes and functions of natural systems to achieve specific objectives, such as retention or removal of various pollutants including nutrients, heavy metals, pathogens, hydrocarbons and colloidal particles. Typically, their design criteria aim to ensure that the levels of these pollutants after a land disturbance phase is complete are not worse than those before works began under average annual runoff conditions, especially on "greenfields" subdivisions.

(b) The term "constructed wetland" is considered as the total entity of a project and integrated with surrounding elements of the natural environment.

(c) Although using natural processes, constructed wetlands are not natural systems. Further, they require ongoing monitoring and management for continued performance over their design life.

(d) Before water enters a wetland, it is desirable to:

  (i) reduce sediment loads, particularly dispersible fines, organic debris and other floating materials;

  (ii) attenuate stormwater flows so that the wetland's retention time is not adversely affected; and

  (iii) attenuate stormwater velocities to ensure ecological viability, as scour and erosion can damage planted areas and re-suspend sediment leading to downstream pollution.

6.4.3 Planning

(a) General principles that should be adopted for constructed wetland projects include:

  (i) development with due consideration of any existing catchment or subcatchment stormwater management plan – they should be easily integrated with the local Council's stormwater planning process (see (b), below);

  (ii) construction off-line from watercourses and, preferably, outside the riparian zone;

  (iii) the application of a multi objective planning process (see (c), below);

  (iv) the use of a multi disciplinary team in the planning process that includes an ecological perspective and the likely eventual owner;

  (v) where possible, maintenance or improvement of the predevelopment water quality of the downstream receiving water body; and

  (vi) adoption of an operation and maintenance plan for the ongoing management of the wetland that considers both the biological and physical processes.

(b) Consider wetlands as part of a comprehensive stormwater management system that involves the whole catchment. Therefore, constraints should be assessed on a site-by-site basis (Chapter 3). Where these constraints limit opportunity for construction of wetlands at a particular site, consideration should be given to the control of pollution of nutrients, etc. further down the catchment (i.e. in a regional wetland).

(c) Constructed wetlands should be designed to meet multiple objectives to lengthen their effective life span and improve community usage of the area. Multiple objectives include:

- water quality improvements for various parameters
- wildlife habitats
- flood mitigation
- passive recreation
- visual amenities (landscape features)
- water supply (i.e. park irrigation)
- educational and research value.

(d) Use of constructed wetlands and their surroundings for active recreational purposes should be dependent on an assessment of the risk to human health and of the potentiall for vandalism. The DEC does not recommend water-based activities or fishing unless contaminants in the waters meet the relevant national water quality criteria (ANZECC, 2000 and Dunkerley, 1995). Note that birds can be a major source of faecal contamination, creating conflict between water quality and habitat objectives.

(e) Constructed wetlands for stormwater management are usually within the urban residential environment. Therefore, the local community often has a personal stake, particularly where they are to be retrofitted into an existing area. As a result, best management practice should include a community involvement program incorporating community awareness, consultation and, perhaps, participation (Brown, *et al.*, 1996). Community involvement programs can:

- engender community support and ownership for projects
- help incorporation of community wishes and concerns in the planning and design phases
- provide community education on environmental issues.

(f) Generally, the use of wetlands during the construction phase should not be necessary – during this phase, control of sediment pollution is the major issue. Control of pollution of materials other than sediment usually only becomes necessary after landscaping has started and traffic levels have increased (i.e. in the landscaping and post development phases). However, commissioning wetlands during the construction phase might be convenient if sediment pretreatment is provided.

6.4.4 Wetland Design

Configuration

(a) Where practicable, water entering wetlands should be relatively free of sediment. Pretreatment for sediment control is achieved before water enters the reed bed zone (containing emergent macrophytes) by designing a sedimentation zone. The sedimentation zone or sediment forebay removes coarse sediments from the water column through settling.

(b) Wetlands should be constructed only as offline systems. These are built outside the main flow channel, usually being fed by very small catchments where the drainage system in not of "watercourse" status. In some cases, a diversion structure that allows runoff from large storm events to bypass the system can be considered, but these are problematic in terms of maintaining the connectivity of the watercourse. Online systems built within the flow channel are not appropriate because all runoff flows through them and, consequently:

- their integrity can be damaged in large storm events
- they interrupt the stream or continuum, including the hydrology and the hydraulics, sediment transport and geomorphic processes
- they interrupt the wildlife corridor
- they replace a riparian/aquatic habitat with a wetland habitat.

Chapter 7

SITE STABILISATION

7. Site Stabilisation

7.1 Introduction

7.1.1 Background

(a) Stabilisation can be achieved with vegetation, paving, armouring or any other cover that protects the ground surface from erosive forces, i.e. reduces the C-factor to an acceptable level (Appendix A). It is essential on all disturbed lands where works are complete or in temporary abeyance to mitigate sediment pollution to downslope lands and waterways. This is because potential soil loss can often be reduced to about 1 percent or less of the prestabilisation level through the application of a suitable protective cover. In addition, stabilisation can improve the operational efficiency of the complete soil and water management program, and enhance the aesthetic values of the site. Nevertheless, sediment control works are necessary on all sites until stabilisation is complete.

(b) Sections elsewhere in these guidelines highlight the importance of giving priority to those BMPs that mitigate soil erosion in the first place rather than to those that clean up the mess downslope or at the catchment outlet. This is because the control of soil erosion is the simplest and most economical way of minimising sediment pollution.

(c) Vegetation is an ideal and usually inexpensive method of stabilisation because it reduces soil erosion hazards by:

- absorbing the impact of raindrops
- reducing volume and velocity of runoff
- binding the soil with roots
- protecting the soil from the erosive effects of wind.

(d) It is common practice to use annual species as a fast growing and highly effective temporary ground cover. However, these plants die within one season, providing almost no residual surface protection after about six or eight months. Where protection is required beyond six or eight months, using a mixture of perennial and annual species is best. While the perennial species are usually slower to establish, they will grow under the annual species and succeed them to provide a permanent surface protection.

(e) Effective revegetation is possible only where the factors necessary to promote and sustain plant growth levels are adequate, including sunlight, temperature, soil fertility and structure, and moisture levels.

(f) Where land disturbance activities occur in riparian zones or watercourses, prepare a separate *Vegetation Management Plan* (Appendix I). This plan is to cover all disturbed lands to at least 10 metres beyond the works. It should address revegetation, bush regeneration and weed control. It should ensure that previously stored topsoil is respread over disturbed lands and the litter layer is restored. Any imported topsoil must be weed free.

(g) If non indigenous plants are to be used as a temporary measure in natural areas, sterile hybrid species are preferred. Invasive species, such as Kikuyu and Rhodes Grass should not be used if not already common in the immediate vicinity.

7.1.2 General Principles

(a) Where practicable, schedule the land disturbance program so that the time from starting activities to completion of the final rehabilitation program is less than six months. Special erosion and sediment control measures should be considered where such staging of land disturbance activities is not possible. Here, rehabilitation is defined two ways, depending on the local rainfall erosivity:

  (i) In periods of expected low rainfall erosivity during the rehabilitation period, achieve a *C*-factor of less than 0.15 and keep it there by vegetation, paving, armouring, etc.[1] Low rainfall erosivity is a month with an erosivity of less than 100. The erosivity for a month at a location is calculated by:

  R-factor X percentage of annual EI occurring per month (Table 6.2 – derive the zone from figure 4.9).[2]

  (ii) In periods of moderate to high rainfall erosivity during the rehabilitation period, achieve a *C*-factor of less than 0.1 and set in motion a program that should ensure it will drop permanently, by vegetation, paving, armouring, etc. to less than 0.05 within a further 60 days. Of course, local water restrictions might affect this in drought times.

(b) In addition, schedule works above the 2-year ARI flood level so that the duration from the conclusion of land shaping to completion of final stabilisation is less than 20 working days. Where practical, phase works so that:

  (i) minimal lands are exposed to the forces of soil erosion at any one time; and

  (ii) site stabilisation measures are progressively installed throughout the development phase.

(c) However, where works are within the 2-year ARI flood level, ensure that the *C*-factors are higher than 0.1 only when the 3-day forecast suggests that rain is unlikely. In this case, management regimes should be established that facilitate rehabilitation within

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1. *C*-factors of 0.15 can be achieved in various ways as shown at Appendix A, note especially figure A5, Table A3 and Table A4. For example, figure A5 shows that:
   (i) A *C*-factor of 0.15 can be achieved with about 30 percent ground cover where the soils have not been disturbed recently and 50 percent cover where they have been disturbed (as at most construction sites);
   (ii) A *C*-factor of 0.05 can be achieved with about 55 percent and 70 percent cover on undisturbed and disturbed soils respectively.
2. Fortnightly *EI* data are available for some locations in New South Wales (Rosewell and Turner, 1992) (Table 7.1) and monthly data in Queensland (Rosenthal and White, 1980). Monthly estimates are available for some locations in South Australia (Yu and Rosewell, 1996) and Western Australia (McFarlane *et al*. 1986). A method for estimating half monthly values of erosivity from monthly data is provided by Renard *et al*. (1997).

24 hours should the forecast prove incorrect. Of course, this assumes that the regular suite of BMPs is installed as outlined elsewhere in these guidelines.

(d) While C-factors are likely to rise to 1.0 during the work's program, they should not exceed those given in Table 7.1.

Table 7.1 Maximum acceptable C-factors at nominated times during works.

Lands	Maximum C-factor	Remarks
Waterways and other areas subjected to concentrated flows (Section 5.2.3), post construction	0.05	Applies after 10 working days from completion of formation and before they are allowed to carry any concentrated flows. Also, note the requirements of Table 5.1 (Note: a C-factor of 0.05 can be acheived various ways, including with about 70% groundcover. See Appendix A, especially figure A5 and Tables A3 and A4)
Stockpiles (Section 4.2.2), post construction	0.10	Applies after 10 working days from completion of formation (Note: a C-factor of 0.10 is acheved with about 60% groundcover)
All lands, including waterways and stock-piles during construction	0.15	Applies after 20 working days of inactivity, even though works might continue later (Note: a C-factor of 0.15 can be acheived various ways, including with about 50% groundcover. See Appendix A, especially figure A5 and Tables A3 and A4)

(e) Successful revegetation of lands requires:

- availability of acceptable soil materials
- correct site preparation
- selection of the most suitable establishment technique
- selection of appropriate plant species, fertiliser(s) and ameliorant(s)
- application of sufficient water for germination and to sustain plant growth if rainfall is insufficient
- an adequate maintenance program.

Proper investigation of each of these matters on a site-specific basis is usually required.

(f) Investigate areas not satisfactorily revegetated to determine the reason for failure. Then undertake appropriate remedial action, including replacing any lost topsoil and resowing the site.

(g) Maintain any erosion and sediment control measures until all earthworks are completed and the site rehabilitated. Where appropriate, remove soil conservation structures as the last activity in the site stabilisation program.

7.2 Revegetation: Lands Subjected to Sheet Flow

7.2.1 Introduction

(a) On lands subjected to sheet flow, consider revegetation programs over two stages:

  (i) Primary revegetation, which normally does not include native species and is designed to reduce the erosion hazard to an acceptable level rapidly (figure 7.2); and

  (ii) Secondary revegetation, which might follow to create an aesthetically more pleasing environment through natural or artificial addition of permanent endemic/native species.[3]

(b) The landscape analysis for the site will identify a suitable strategy for revegetation, reflecting a specific theme such as mown grass, bushland regeneration, etc.

7.2.2 Primary Revegetation

(a) Primary revegetation usually includes the use of exotic species,[4] in particular pasture grasses.[5] In most cases:

  (i) use annuals where a quick, temporary cover is required (for up to about six months), and perennials for long term protection; and

  (ii) use warm season species where summer rainfall is dominant, and cool season species where winter rainfall is dominant and/or winters are cold.

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3. Primary revegetation might be omitted where the erosion hazard has been brought under control through use of mulches and/or various fabrics (Section 7.4.1). Exercise care in ensuring that:
   - materials are not toxic to the desired plant species
   - they are maintained until the secondary species are producing their own mulch (can be three to five years).
4. This recommendation does not hold in bushland areas if the use of exotic species is regarded as undesirable.
5. Exotic pasture species are preferred for primary revegetation because they are more easily established, provide rapid cover, and root growth quickly binds the soil surface. In addition, the seed is commercially available, seed viability is usually high and sowing methods are relatively simple. Legumes, in spite of the beneficial addition of nitrogen to the soil, are rarely used in urban revegetation programs because of their specialised management requirements.

Figure 7.1 Unsatisfactory site stabilisation has resulted in substantial quantities of sediment leaving the site

Figure 7.2 Primary revegetation of an earth batter with straw mulching (Section 7.4.1). Notice the sediment fence and barrier mesh to minimise soil erosion and sediment pollution

In the Sydney Region, Appendix G might help in the choice of plant species, fertilisers and ameliorants. Elsewhere, seek advice from local seed merchants or appropriate government departments for listings of species, sowing rates and fertiliser requirements suitable for rehabilitation.

(b) As the physical and chemical characteristics of many subsoil materials inhibit the establishment of plants, respreading of topsoil (Section 4.3) over the disturbed area is recommended. Avoid incorporation of subsoil material into the topsoil.

(c) Where practical to do so, a seedbed should be cultivated before sowing seed (Standard Drawing 7-1). This should include deep ripping to at least 300mm.

(d) Where possible, ensure any cultivation of the soil is parallel to the contour.

(e) Plants can be established (figure 7.3) by:

- (i) broadcasting, particularly on very small areas (<one hectare) or lands that are inaccessible to conventional implements;[6]
- (ii) conventional implements[7] including direct drilling or sod seeding to a depth of about 10 mm to 15 mm;[8]
- (iii) laying turf such as couch or kikuyu, particularly where immediate vegetative cover is required for stabili-sation or aesthetic reasons; and[9]
- (iv) hydraulic seeding, especially on steep or inaccessible areas (figure 7.2).

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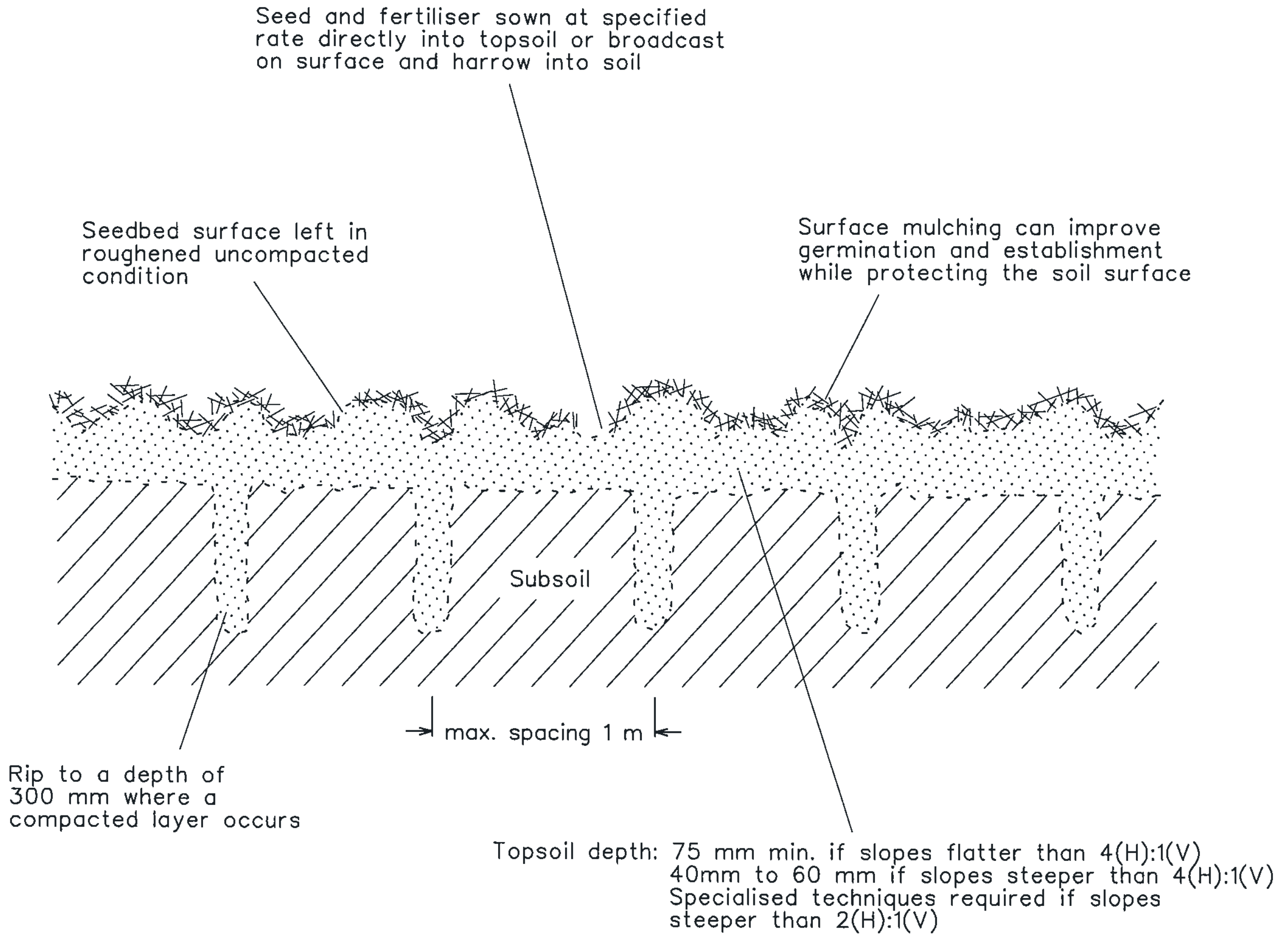
6. Add sand to the seed to help achieve an even spread. Where grasses are being established, harrow the surface immediately after the seed and fertiliser have been applied. With the establishment of bushland plant species, undertake the harrowing first.
7. Use of conventional implements is usually the most cost-effective method of establishing plants from seed. Plants with small seeds, grasses in particular, establish on a fine seedbed best. However, a relatively rough seedbed might be required where the soil is dispersible or the erosion risk is high (e.g. on Soil Loss Classes 5 to 7 lands). It can be formed by scarifying to a depth of about 50 mm to 75 mm. A rough seedbed is less likely to "surface seal" and will absorb moisture more readily (SD 7-1).
8. These methods are preferred and have three advantages:
   - fertiliser is placed below the soil surface reducing the possibility of it being washed into waterways
   - precision planting of seed and fertiliser is achieved (if appropriate)
   - higher germination rates usually occur.
9. Turf should be:
   - placed on a bed of fertilised topsoil of a minimum depth of 75 mm
   - laid parallel to the contour on sites with steep slope gradients
   - normal to direction of flow in waterways
   - under or over a pegged artificial mesh (e.g. a light polypropylene, UV stabilised mesh with about 20-mm openings) in areas of very high water velocity
   - rolled or tamped immediately as it is laid
   - where necessary, pegged to the soil at 1 to 2 metre centres, e.g. with 4 mm (No. 8 gauge) wire approximately 200 mm in length
   - watered immediately to enhance establishment
   - watered regularly for the first seven days or as required to effect establishment
   - mowed as required under the maintenance contract for the site.



Construction Notes

1. Loosen compacted soil before sowing any seed. If necessary, rip the soil to a depth of 300 mm. Avoid rotary hoe cultivation.
2. Work the ground only as much as necessary to achieve the desired tilth and prepare a good seedbed.
3. Avoid cultivation in very wet or very dry conditions.
4. Cultivate on or close to the contour where possible, not up and down the slope.

SEEDBED PREPARATION

SD 7-1

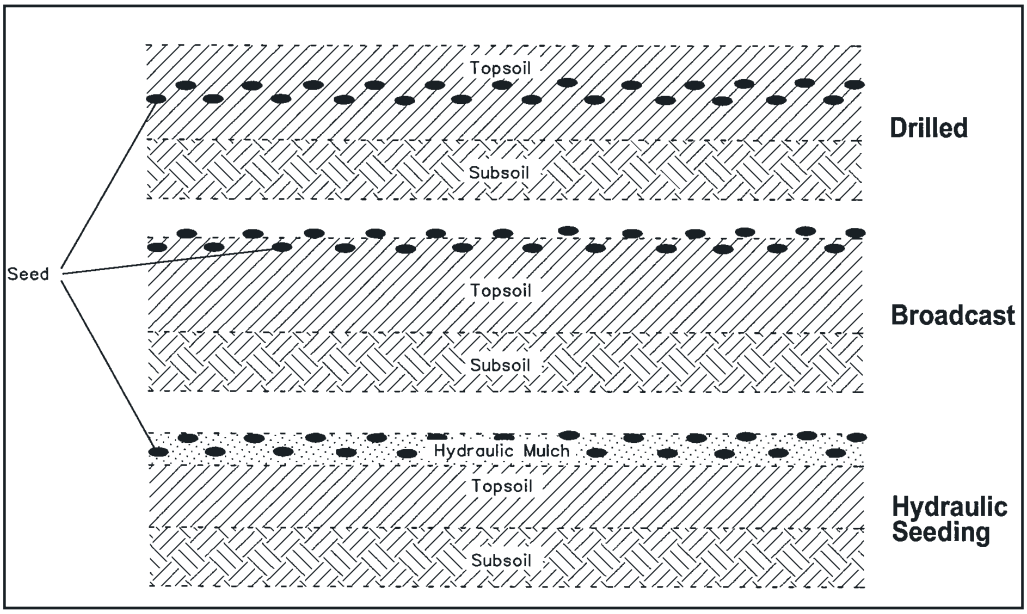


Figure 7.3 Seed placement with different sowing methods

(f) In addition to identifying the best species mix, establishing the best fertiliser and ameliorant mix for plant growth is essential.[10]

(g) Establishment of plants should not be attempted during hot, dry periods unless sufficient water can be applied artificially.

7.2.3 Secondary Revegetation

(a) Secondary revegetation normally follows the primary revegetation program, although species can be established at the same time.

(b) Include native species propagated to enhance ecological values and create an aesthetically pleasing environment after the soil erosion hazard has been reduced. Near watercourses, use native plants propagated from seeds collected from the local area.

(c) Establishment can be from seed, tube stock or invasion from the surrounding bushland. Where possible, choose species that will rapidly provide an adequate mulch to protect the ground surface from the forces of erosion.[11]

10. For optimum establishment and growth of exotic species, many soils require ameliorants to alter the soil pH and/or improve the soil structure. Details on specific requirements can be obtained from appropriate government departments or through laboratory testing of the soil materials.
11. Most exotic pasture species have difficulty competing with taller native plants when more than about 70 percent shading occurs.

(d) Effective establishment of native species, particularly those endemic to the region, usually requires an environment where the ongoing nutrient and moisture regimes are close to the natural status.[12] Avoid the use of fertilisers and/or ameliorants except in situations where they are likely to leach from the soil relatively quickly, and any modified conditions are unlikely to be retained. These factors are often critical when reestablishing or retaining native bushland downslope from an urban subdivision, for example, where drainage from well-tended gardens can create an environment where exotic plants can compete effectively with native plants.

(e) Where primary revegetation is omitted in bushland areas, protect the ground surface against erosion with mulch or a biodegradable blanket until adequate mulch is produced naturally by local plants (Section 7.4.1). This can also reduce weed competition, depending on depth or thickness.

7.3 Revegetation: Lands Subjected to Concentrated Flow

7.3.1 Introduction

(a) While most erosion control techniques are satisfactory under sheet flows conditions (e.g. wood chip mulches), special measures are essential under concentrated flow conditions. Some of these are identified in Table A4 (Appendix A). These special measures should be considered on all lands within the 10-year ARI flood level.

(b) Generally, use plants as a protective measure to bind the surface together. This is especially important in waterways because:

- (i) their growth pattern usually results in lower water velocities (therefore, increased time of concentration, lower peak flows and fewer outlet problems);
- (ii) they can filter pollutants from the water, including sediment and nutrients; and
- (iii) usually, they are aesthetically more pleasing than non vegetative materials.

(c) In most situations, use only non invasive exotic species for revegetation of lands subjected to concentrated water flow because:

- (i) seed is more likely to be available in commercial quantities when required; and
- (ii) they are more effective than native species in binding the soil material and reducing the soil erosion hazard.

12. Most native species have evolved adaptations to a harsh environment that often include soils deficient in nutrients, and/or that have extremes of pH, and/or that have extreme fluctuations in available moisture. Exotic species usually do not grow well in such environments without modification to one or more of these factors.

(d) Do not use reinforced turf near watercourses unless the mesh is biodegradable. If exposed, it can:
   - be a threat to wildlife
   - lead to mass failure of the turf if snagged.

7.3.2 Vegetation Establishment

(a) Establishment of plants on lands subjected to concentrated flows can be undertaken using similar methods to those described in Section 7.2.2 for Primary Revegetation.

(b) Areas of frequent channelised flow are best stabilised with reinforced turf.

(c) Where water flow is saline or relatively continuous, suitable species and channel treatment should be planned to avoid establishment failure and channel damage.

(d) Permanently established vegetated waterways should provide protection to the soil against the erosive action of flowing water as described in Section 5.3.3(c) and Table 5.2 in the design storm event (Section 2.3.1 (e)).

7.4 Special Considerations

7.4.1 Aids to Establishment

(a) Mulches:

   (i) usually provide a protective cover for the soil surface to prevent erosion of loose soil/fertiliser particles (Table A3, Appendix A), especially on lands with moderate to steep slopes, and help establishment of plants by reducing evaporation and increasing water infiltration;

   (ii) should be 20 mm to 40 mm thick – thicker mulches (75 mm to 100 mm) inhibit germination and can be applied to control weeds; and

   (iii) in bushland areas, should be:
   - comprised of local native species where available
   - maintained until the vegetative cover can provide adequate protection against the erosive forces (figure 7.4; cf. figure 7.3).

(b) Straw mulches:

   (i) are particularly effective where soils are dispersible, on sites with a high soil erosion hazard, or where soil moisture is likely to be inadequate for successful plant establishment (e.g. batters);

   (ii) such as wheat or oaten straw are suitable at about 250 bales per hectare, other than in bushland areas, and should be dry when applied and have a low leaf content;

(iii) should be free of non endemic seed in bushland areas; and

(iv) should be sprayed with an anionic bitumen emulsion at 2,500 litres per hectare or other suitable binder.

(c) Brush mulches:

(i) are preferred on lands where regeneration of native plants is wanted as it can provide an additional source of seed — of course, using endemic plant materials;

(ii) should be applied parallel to the contour; and

(iii) should be stockpiled with care since spontaneous combustion can occur.

Care should be exercised to ensure the cutting of local brush does not damage adjoining ecosystems. Brush should only be taken from approved cleared areas.

(d) Wood chip mulches are useful for weed control. Mulched street tree loppings and pine flakes are preferred from an ESD standpoint. Processed hardwood is ESD acceptable if taken from forest trimmings and not from primary forest timber trees.

(e) Biodegradable blankets (see Appendix D), including jute mesh and plant fibre matting, are alternatives to mulches and particularly useful in areas of high water concentration[13] Jute mesh should be sprayed with an anionic bitumen emulsion at about 1 to 3 litres per square metre for extra stability in areas where concentrated runoff might occur.

(f) Bitumen emulsion can be applied by itself as mulch and is suitable for areas where cool season plants will be sown and soil moisture is not a major constraint to plant establishment.

(g) Hydroseeding is particularly useful in the higher rainfall, coastal areas. Supplementary watering is advisable if weather conditions are unfavourable for germination or establishment. Include polymers or bituminous binders on steep lands. [14]

13. They can provide temporary protection to earth drains intended to be removed or upgraded within six months, or grassed waterways that have only recently been established from seed or runners (figure 4.4).
14. Hydroseeding involves the mixing of seed, fertiliser and a paper or wood pulp with water to form a slurry sprayed over the area to be revegetated. The seed generally sticks to the pulp that improves the microclimate for germination and establishment. Hydromulching is a different operation to hydroseeding in that it uses a higher rate of cellulose fibre to act as mulch by itself. Hydroseeding and straw mulching are normally concurrent operations and achieve superior results to hydromulching.

Figure 7.4 Lack of site stabilisation of a drainage line following installation of services

Figure 7.5 Effective mulching of a service easement using local brush

7.4.2 Saline Areas

(a) A Western Sydney Salinity Code of Practice has been prepared by the 13 relevant Councils through the Western Sydney Salinity Working Party. This is a useful document to help all those working in saline areas, but especially in western Sydney and should be consulted wherever salinity is expected, especially on soils derived from marine sediments such as the Wianamatta Shales. Read the Code together with the Salinity Hazard Map prepared by the Department of Infrastructure, Planning and Natural Resources.

(b) Salinity occurs when salts found in the soil or groundwater mobilise, allowing capillary rise and evaporation to concentrate salts at the ground surface. Usually, such movements are brought about by changes to the natural water cycle through:

- artificially adding water to the watertable, causing it to rise
- removing deep-rooted vegetation
- impeding subsoil drainage.

(c) Some developers in western Sydney have unwittingly contributed to the problem by following water-sensitive urban design principles developed in other places where salinity is not a problem. Specifically, they have followed procedures designed to encourage excessive infiltration to the watertable without a full appreciation of the consequences. Infiltration measures, while encouraged, should incorporate a subsurface drain and liner where infiltration to groundwater might exacerbate salinity problems.

(d) If the watertable within the root zone becomes saline, the vegetative cover is likely to die and expose the soil to erosive forces. Salinity also can affect built infrastructure, affecting detrimentally concrete, bricks and metal, and resulting in structural damage and unnecessary repair costs.

(e) Salinity problems are usually overcome by lowering the watertable through:

(i) reducing infiltration rates, e.g. lining waterways with impervious materials

(ii) improving drainage, e.g. installation of subsoil drains;

(iii) planting deep-rooting salt-tolerant plants to act as "pumps".

(f) Choose plant species for rehabilitated lands that are more tolerant of any likely high salt levels.[15]

(g) Where necessary, implement building controls and/or other engineering responses to salinity problems.

15. Including (from highly salt tolerant to moderately tolerant) puccinellia, tall wheat grass, couch, Wimmera rye grass, Rhodes grass, phalaris, strawberry clover and lucerne (Hamilton and Lang, 1978).

7.4.3 Maintenance

(a) Maintenance (Chapter 8) of both soil conservation works and revegetated areas is an essential part of any rehabilitation program and should be addressed in the *ESCP/SWMP*. It can include:

  (i) periodic application of water, especially in the first seven days from establishment on turfed areas and/or in hot, dry weather;

  (ii) further application of seed and fertiliser in areas of minor soil erosion and/or inadequate vegetative establishment; and

  (iii) regular mowing, especially in waterways, to control weeds and to maintain a cover that does not impede flows and cause flooding or accumulation of pools of stagnant water.

(b) Establish salt-tolerant species or apply other corrective measures where bare areas arise because of salinity in surface or ground water and soils.

(c) Control excessive vegetative growth through mowing, slashing or judicious use of herbicides. Do not mechanically grade vegetated waterways and road verges unless part of a further stabilisation program.[16]

16. Set mower height no lower than 75 mm above the ground surface.

Chapter 8
MAINTENANCE

8.1 Introduction

(a) Proper maintenance of soil and water conservation works plays a vital part in their management and operation. After a storm event, the effectiveness of the established controls can be readily seen with any shortcomings and damage.

(b) Always keep the potential hazards of soil erosion at the site and consequent sediment pollution to downslope areas to a minimum. This is always important, but especially before times when works are unlikely to proceed for any reason. Accordingly, the site manager should check the operation of all soil and water management works each day and initiate repair or maintenance as required.

(c) Current legislation requires the quality of run off water leaving each site to be of an acceptable standard. Penalties apply where pollution to downslope lands and waterways occurs. The law does not recognise:

- whether or not the site is difficult
- problems that might be encountered in implementing the plan
- whether or not you are familiar with good soil and water standards.

(d) An effective maintenance program should include ongoing modification to any *Plan* as development progresses. This is because such *Plans*:

(i) are usually based on a specific landform shape. However, as development proceeds, changes occur in slope gradients and drainage paths with their exact form frequently unpredictable before works begin; and

(ii) assume the site development works will proceed according to a specific set of engineering plans. However, these are often modified as part of the development process.

(e) Address ongoing maintenance of all permanent soil and/or water control structures in the planning phase. This is likely to be relevant, especially for some long-term works, where authority for maintenance passes from the developers/site operators and their contractors to, e.g. the local consent authority.

8.2 Maintenance Program

(a) Empty bins for concrete and mortar slurries, paints, acid washings, lightweight waste materials and litter at least weekly and otherwise as necessary. Dispose of any waste in an approved manner.

(b) Ensure proper drainage of the site. To this end:

(i) clean any catch drains, diversion banks, table drains, berm drains and drop-down structures (including inlet and outlet works) that have become blocked through sediment pollution, sand/soil/spoil being deposited in or too close to them, breached by vehicle wheels,[1] etc.;

(ii) check that drains are operating as intended (Section 5.4), especially that:
- no low points exist which can overtop in a large storm event[2]
- areas of erosion are repaired (e.g. lined with a suitable material[3] and/or velocity of flow is reduced appropriately through construction of small check dams or installing additional diversions upslope);

(iii) construct small additional earth diversions[4] at distances of less than 80 metres across the works to keep slope lengths short and dispose of water without causing channel erosion; and

(iv) regularly clean out sediment trapped behind sediment fences and other traps.

(c) Ensure removal of any sand/soil/spoil materials placed closer than 2 metres from hazard areas, such as waterways, gutters, paved areas and driveways. Provide protection to receiving waters from any such materials placed more than 2 metres from hazard areas by implementing the required soil and water management practices.

(d) Check that rehabilitated lands have established sufficient ground cover to reduce the erosion hazard effectively and initiate repair as appropriate (Chapter 7). Note that:

(i) periodic applications of water are essential, especially in the first seven days from establishment on turfed areas and/or in hot, dry weather; and

(ii) further applications of seed and fertiliser might be necessary in areas of minor soil erosion and/or inadequate vegetative establishment.

Establish salt-tolerant species or apply other corrective measures where bare areas arise because of salinity in surface or ground water.

(e) Control excessive vegetative growth through mowing,[5] slashing or judicious use of biodegradable herbicides. This is especially important with waterways to control weeds and to maintain a cover that does not impede water flow, thereby causing flooding or accumulation of pools of stagnant water. Do not grade existing waterways and road verges unless part of a further rehabilitation program.

(f) Do not dispose of cleared vegetation by open burning on site. Preferred disposal options include chipping or mulching for future rehabilitation purposes, unless the presence of weed seed or viable vegetation parts makes this not viable. Less preferred options include transport to a landfill facility, or trench-burning using licensed equipment.

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1. Redesigning any crossings to permit continued vehicle access without affecting the function of the drain might be necessary.
2. Either raise low points or, temporarily, line the downslope side with sandbags, straw bales, etc.
3. Including use of grass, plastic, geotextile, rock or concrete.
4. A single pass with a grader, constructing a diversion drain about 300 mm deep is usually adequate.
5. Set mower height no lower than 75 mm above the ground surface.

(g) Control emission of dust from unsealed roads and other exposed surfaces, such as unprotected earth or soil stockpiles, by use of surface sealants and/or water spray carts or other appropriate equipment. Keep the surfaces moist rather than wet.

(h) Keep all sediment detention systems in good, working condition. Ensure:

 (i) recent works have not resulted in the diversion of sediment-laden water away from them;

 (ii) degradable products (e.g. straw bales) are replaced as required;

 (iii) sediment is removed if the design capacity or less remains in the settling zone;

 (iv) retention basins on *Type C* soils have a minimum settling zone depth of at least 0.6 metres over two-thirds of the surface area when surcharging;

 (v) water in retention basins on *Type D* soils is treated with a flocculating agent following the requirements of Section 6.3 and Appendix E if the soils at the sediment source contain more than 10 percent dispersible materials.[6] Where basins require pumping out, the necessary dosing should occur within 24 hours of the conclusion of each storm event and the basin should be drained once suspended solids levels are less than 50 milligrams per litre, usually 36 to 48 hours later if gypsum is used. Longer or shorter treatment and dewatering periods may apply if rainfall events of duration other than 5 days has been adopted in the design of the basin;[7] and

 (vi) pollutants, sediment and/or waste removed from sediment basins, gross pollutant traps and trash racks are disposed in stabilised dumps where soil and water measures have been implemented to stop offsite movement of pollutants.

(i) To determine the effectiveness of any sediment retention basins, the consent authority might require the site manager to undertake sampling and subsequent analysis of non filterable residue (NFR) concentrations of waste water. Such sampling and analysis is likely to be required periodically or for a nominated period, usually the first three months after commissioning the basins.

(j) Dispose any pollutants removed from sediment basins in areas where further pollution to downslope lands and waterways should not occur.

(k) Construct additional erosion and/or sediment control works as might become necessary to ensure the desired protection is given to downslope lands and waterways, i.e. make ongoing changes to the Plan.

(l) Maintain erosion and sediment control measures until all earthwork activities are completed and the site rehabilitated.

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6. Where necessary, a suitably sized stockpile of flocculating agent should be kept onsite for the treatment of wastewater impounded in sediment retention systems.
7. Place a marker peg within each sediment retention basin to indicate the design capacity of the sedimentation zone and level above which capacity is available in the settling zone for containment of runoff.

(m) Temporary soil conservation structures/measures are to be removed and surfaces restored to the final landform as the last activity in the works program. Then, vegetative rehabilitation of these areas can begin following the requirements of the site rehabilitation/landscaping plan. First liaise with the relevant local government body where works:

- are likely to continue in the catchment and are not associated directly with the development
- include sediment retention basins.

This is to determine whether the local consent authority is prepared to take over control and responsibility for any such structures. Ongoing maintenance of sediment basins can be desirable where later works in the catchment not associated with this development are likely to produce sediment. If the local consent authority does agree to take such responsibility, the developer/site operator is expected to ensure that they are in good working order and design capacity is available.

(n) A self-auditing program should be established based on a Check Sheet (Table 8.1) developed for the specific site – note that every site will be different. A site inspection using the Check Sheet should be made by the site manager:

- at least weekly, and
- immediately before site closure, and
- immediately following rainfall events that cause runoff.

Undertake the self-audit by:

- walking around the site systematically (e.g. clockwise)
- recording the condition of every BMP employed
- recording maintenance requirements (if any) for each BMP
- recording the volumes of sediment removed from sediment retention systems, where applicable
- recording the site where sediment is disposed
- forwarding a signed duplicate of the completed Check Sheet to the project manager/ developer/ site operator for their information.

In particular, inspect:

- locations where vehicles enter and leave the site
- all installed erosion and sediment control measures, ensuring they are operating correctly
- areas that might show whether sediment or other pollutants are leaving the site or have the potential to do so
- all discharge points, to assess whether the erosion and sediment control measures are effective in preventing impacts to the receiving waters.

Tables 8.2, 8.3 and 8.4 are adapted from Fifield (2002b) and contain listings of suggested inspection guidelines that might apply. These listings are not intended to be complete and issues raised might vary from one site to another.

Keep a complete set of the Self-audit Check Sheets onsite and make them available to any officer of the local council, NSW DEC or other authorised person on request.

Table 8.1 Example of a Self-audit Check Sheet (part only)

Site Location:		
	Date Inspected:	
	Name:	
	Signature:	
BMP	Condition	Remarks
Basin 1	OK	No maintenance required
Basin 2	Contains Sediment (about 30 m^3)	Instructed J. Smith to remove it and dispose at the fill site
Silt fence 1	OK	No maintenance required
Silt fence 2	Breached for access	Instructed D. Brown to repair it
Etc.		

Table 8.2 Guidelines that might apply to inspection of structural measures

Sediment retention basins		
	·	has sediment settling zone sufficient capacity?
	·	is the outflow structure installed as illustrated in the *ESCP* or *SWMP* ?
	·	are the embankments protected against erosion?
Sediment filters		
Straw bales	·	are they installed in trenches?
	·	are they tightly abutting, with material stuffed between the bales?
	·	are they staked?
	·	has backfill material been placed on the upstream side?
	·	is runoff water running around, below, or between the bales?
Sediment fences	·	is the filter fabric buried in a trench and backfilled?
	·	are the stakes installed correctly with proper spacing?
	·	has sediment accumulated to within 300 mm of the top?
	·	is runoff water running around, below, or between the fabric joins?
Continuous berms	·	have the berms been installed correctly?
	·	is the fabric adequately stapled?
Other	·	are barriers causing local flooding problems?
Check dams		
Straw bales	·	are the bales staked and tight with each other?
	·	have the bales been installed in a trench and backfilled?
	·	will water be forced to run over a centre bale and not around the end bales?
	·	is the ground below where water flows over the bales eroding?
Rock	·	is the correct-size rock being used?
	·	will water flow over the middled instead of around the edges?
	·	Has movement of the rock occurred?
Drains/inlet protection		
Straw bales	·	are the bales staked and tight with each other?
	·	have the bales been installed in a trench and backfilled?
	·	will water be forced to run over a centre bale and not around the end bales?
	·	is the ground below where water flows over the bales eroding?
Filter fabric	·	is the filter fabric buried in a trench and backfilled?
	·	is it staked correctly with proper spacing?
	·	has sediment accumulated to within 300 mm of the top?
	·	is runoff water running around, below, or between the fabric joins?
Inserts	·	has the insert been installed correctly?
	·	will the insert prevent runoff water from entering the stormwater system?
	·	has sediment filled the structure? When will the sediment be removed?

Table 8.3 Guidelines that might apply to inspection of non-structural measures

Diversion and containment banks	·	are they protected against erosion?
	·	have they been constructed to control and divert anticipated flows?
	·	should the bottom be lined with any material to prevent erosion?
Slope drains	·	will runoff water be diverted into the pipe?
	·	does sufficient protection exist to prevent failure of piping?
	·	is the pipe anchored?
	·	does erosion protection exist where water charges?
	·	are they functioning in the manner they were designed?
Staging of construction	·	does all the ground need to be disturbed?
	·	how much land is being disturbed and how much can remain in vegetation?
Planting of perennial seed	·	are drill marks evident that are parallel or perpendicular to land contours?
	·	has seed tag been checked and the mixture verified?
	·	if seed was applied hydraulically, how much was used?
	·	if seed was broadcast, was the ground raked?
	·	what time of year was the seed planted?
	·	are weeds becoming established?
Planting of temporary, nursery, or cover crop	·	what type of seed was used?
	·	how long will the vegetation be in place before planting perennial grass?
	·	when was the seed planted?
Dry/hydraulic mulch	·	does the mulch cover 80-100% of the bare ground?
	·	if dry mulch is applied, how is it held in place?
	·	has wind removed the dry mulch and is this a problem?
Soil binder	·	what type of material was used?
	·	when was it applied?
	·	does the material still control erosion?
Hillside protection by RECP	·	is the material properly installed at the top?
	·	are sufficient staples used?
	·	does the material overlap along the edges?
	·	does the material need to be repaired?
Channel protection by ECBS, TRMS, and C-TRMS	·	is the material properly installed at the top?
	·	are sufficient staples used?
	·	is the material properly stapled or trenched along the edges?
	·	should a rock check structure be installed on top of the material?

Table 8.4 Guidelines that might apply to the control of wind-borne particles

Soil roughening	· how deep are the furrows? · are the furrows filling up with soil? · are the furrows perpendicular to the prevailing wind?
Wind barriers	· have they been installed perpendicular to what is accepted as the prevailing wind direction? · are they in need of repair or replacement? · have the structures been placed where maximum deposition of wind-borne particles can occur?
Vegetation	· is the ground bare? · how tall and/or dense is the vegetation?
Hydraulic mulch/soil binder	· has sufficient material been applied? · how long will the material be expected to control erosion? · has the material broken down, and is it still effective?

8.3 Post Construction Issues

(a) Whereas this manual deals in the main with the construction phase, there are some post construction issues that should be considered to ensure the construction phase is concluded in a responsible manner. This will apply to both subdivision works and building works.

(b) Issues to be considered include:

(i) Ensure revegetation and planting areas have been properly established, including areas occupied by all temporary erosion and sediment control structures.

(ii) Liaise with the local consent authority to determine whether the ownership and ongoing responsibility for any structures (e.g. sediment basins) can transfer to it.

(iii) Remove all treatment techniques or structures that are no longer required in a way that complies with:

- safety standards
- consent conditions
- requirements that sediment and other materials are disposed in an approved manner
- sound construction principles.

(iv) Ensure site access is returned to its original condition or approved final layout depending on site-specific circumstances.

(v) Transfer of any temporary works to permanent works. This might include the removal of sediment from a sediment basin that is to be transferred to the control of the local consent authority as either a permanent sediment basin or upstream section of a permanent wetland.

manual of the Queensland Department of Main Roads (1975).

In the design example in Technical Note 6, calculations are made both for full areas, and for partial areas based on the concentration times of impervious zones directly connected to the pipe system. This method may still underestimate flowrates, as the critical time for a sub-catchment may be other than the full-area or impervious area concentration times. However, the degree of under-estimation is usually quite small, and the procedure is simpler than the tangent check procedure.

(iii) Runoff Coefficients

Runoff coefficient C can be interpreted in different ways : as a ratio between runoff and rainfall volumes; as the ratio of their peak rates; or as the ratio of the runoff to rainfall frequency curves. In this last, "probabilistic" interpretation, the value of C does not relate to a particular storm. This concept is discussed in detail in Section 5.3.2. It covers the whole range of possible events, involving different combinations of rainfalls and antecedent conditions. Values have been derived for some medium-sized, gauged urban catchments in Australian capital cities by Aitken (1975), Pilgrim (1982) and others.

Many relationships have been proposed relating runoff coefficients to factors such as land-use, surface type, slope and rainfall intensity. The one given in Figure 14.13 is a composite relationship reflecting experience of drainage authorities and evidence from the few gauged urban catchments with suitable lengths of record. It should be used in preference to the runoff coefficient relationships given in previous editions of this publication.

Figure 14.13 relates the coefficient for a 10 year ARI, C_{10}, to the pervious and impervious fractions of the catchment, and to its rainfall climate, expressed through the 10 year ARI, 1 hour duration rainfall intensity, ${}^{10}I_1$.

The upper line represents conditions for areas where ${}^{10}I_1$ is 70 *mm/h* or greater, and the lower one is for areas where ${}^{10}I_1$ is 25 *mm/h* or lower. For areas where ${}^{10}I_1$ is between 25 and 70 *mm/h*, a line can be interpolated using the equations:

$$C_{10} = 0.9 \times f + C^1{}_{10} \times (1 - f) \qquad (14.11)$$

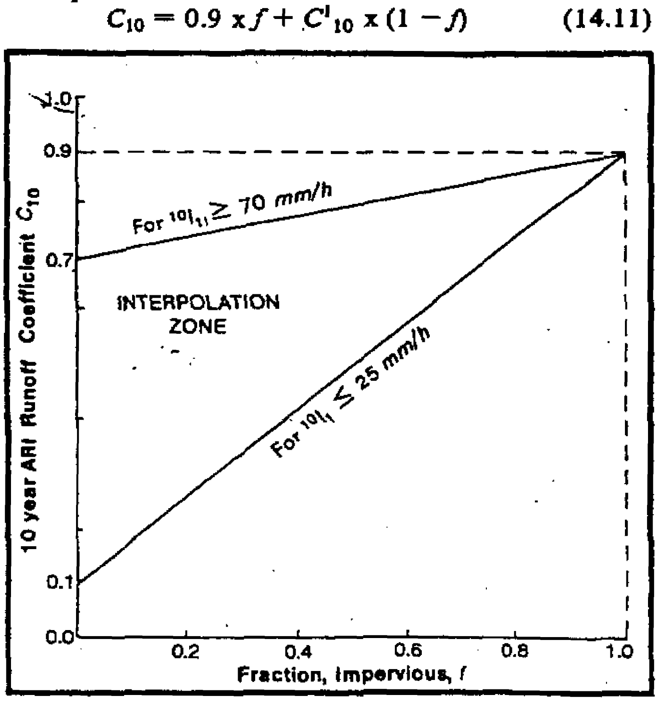


Figure 14.13 - Runoff Coefficients

and

$$C^1{}_{10} = 0.1 + (0.7 - 0.1) \times ({}^{10}I_1 - 25)/(70 - 25)$$
$$= 0.1 + 0.0133 \times ({}^{10}I_1 - 25) \qquad (14.12)$$

where C_{10} is the 10 year ARI runoff coefficient,
$C^1{}_{10}$ is the pervious area runoff coefficient, and
f is the fraction impervious (0.0 to 1.0).

Pervious area runoff coefficients range from 0.1 to 0.7, corresponding to the respective ${}^{10}I_1$ limits of 25 and 70 *mm/h*. These are likely to differ from coefficients derived in regional procedures for rural design flow estimation (such as those given in Chapter 5), due to different interpretations of the Rational Method, the different scales of catchment size and the different times of concentration.

For average recurrence intervals other than 10 years, the C_{10} value is multiplied by a frequency factor from Table 14.6: $C_y = F_y \cdot C_{10}$ (14.13)

Where runoff coefficients calculated from the above equations exceed 1.0, they should be arbitrarily set equal to 1.0.

Note that no allowance is made for slope or soil type. While it seems logical that they would affect runoff coefficients, there is little firm evidence to confirm this. To some extent, the effect of slope is incorporated in the time of concentration estimate. As for soil type, designers may make adjustments based on local evidence, if it is available.

The above relationships can be applied both to areas which are essentially homogeneous, and to those where pervious and impervious portions are intermixed. Where a catchment consists of portions which are significantly different, they should be separated and different C values applied.

(iv) Equivalent Impervious Areas

Design flowrates for pit inlets are calcuated for local contributing sub-catchments, while those for pipes are calculated for the accumulated areas draining through each pipe section or reach. Except for small property drainage systems, it is inappropriate to simply add the separate flows from each sub-catchment. This over-estimates flowrates, unless all sub-catchments have identical times of concentration. When times-to-peak differ, the added flows from a number of areas will have a maximum value less than the sum of the separate peaks.

A more accurate procedure is to accumulate equivalent impervious areas, products of C and A values for sub-catchments. Design flowrates can be calculated by multiplying total equivalent impervious areas by I values corresponding to the times of concentration at various points along a drainage line. Use of equivalent impervious areas also allows different zones within a sub-catchment to be combined. For example, if an area consists of three zones with land-uses denoted by a, b and c, the combined equivalent impervious area is:

$$(C.A)_{total} = C_a.A_a + C_b.A_b + C_c.A_c \qquad (14.14)$$

TABLE 14.6 - Frequency Factors for Rational Method Runoff Coefficients

ARI (years)	Frequency Factor, F_y
1	0.8
2	0.85
5	0.95
10	1.0
20	1.05
50	1.15
100	1.2

APPENDIX E
DRAFT GUIDELINES FOR THE DESIGN OF STABLE DRAINAGE LINES ON REHABILITATED MINESITES IN THE HUNTER COALFIELDS
(NSW Department of Land and Water Conservation, undated)

DRAFT GUIDELINES FOR THE DESIGN OF STABLE DRAINAGE LINES ON REHABILITATED MINESITES IN THE HUNTER COALFIELDS

NSW
DEPARTMENT OF LAND AND WATER
CONSERVATION

CONTENTS:

Purpose

- The establishment of long-term stability of drainage lines is an essential component of successful rehabilitation of minesites. Stable drainage lines protect the integrity of the post-mining landform and satisfy an important component of Ecological Sustainable Development for the minesite.
- Drainage of long steep slopes on emplacement areas or drainage of large catchment areas on rehabilitated minesites is now more common due to the increasing economic and physical constraints on minesites. These situations require mining companies to carefully plan their drainage systems in order to achieve stability both physically and ecologically in the long term.
- Suitable design is the starting point of establishing stable drainage lines on minesites. The purpose of this guideline is to provide direction at the design stage of the drainage program to maximise the opportunities for long term stability.
- The guideline identifies the elements of drainage design and assessment to achieve the necessary rehabilitation outcomes at mine closure.
- The guideline complements existing technical documented material on the topics of mine rehabilitation and drainage design, including the following key references:
    - Hannan, J.C. 1995 Mine Rehabilitation **-** *A Handbook for the Coal Mining Industry*. New South Wales Coal Association, Sydney.
    - Department of Housing. 1998. *"Managing Urban Stormwater, Soils and Construction"*. NSW Department of Housing, Sydney
    - Standing Committee on Rivers and Catchments Victoria. 1991. *Guidelines for Stabilising Waterways*. Rural Water Commission, Armadale, Victoria.
    - AR&R. 1998. *Australian Rainfall and Runoff, a Guide to Flood Estimation*. Institution of Engineers, Australia. Canberra.
    - Rutherfurd,I.D. Jerie, K & Marsh, N. 2000 *A Rehabilitation Manual for Australian Streams Vol 1 & 2.* Cooperative Research Centre for Catchment Hydrology. Land and Water Resources Research and Development Corporation.
    - Koehn, J.D. Brierley, G.J. Cant, B.L. & Lucas, A.M. 2001. *River Restoration Framework* Land and Water Australia.

Scope

- Mining development in NSW is required to establish long-term hydrologic stability on rehabilitated minesites. The scope of this guideline is to provide considerations for the <u>design</u> of stable drainage lines on minesites.
- Standard rehabilitation conditions of a mining lease include the requirements for prevention of soil erosion. Drainage lines are sensitive areas with a high risk of soil erosion which require special consideration and planning to ensure stability. Stable drainage lines are therefore an essential component of any rehabilitation plan which should integrate with the erosion and sediment control plan.
- Rehabilitation conditions for mining in NSW are determined under the Mining Act 1992 by the Department of Mineral Resources in consultation with the Department of Land and Water Conservation. This guideline identifies the process of agency assessment of drainage design for mine site rehabilitation.
- Drainage design is reviewed by these agencies sequentially during development approval in the EIS and during the mine life in the Mining Operations Plans (MOP). The MOP addresses in detail the design of the mine's water management and drainage system.
- This guideline applies to drainage lines and control structures on rehabilitated mine lands with catchments ranging from a few hectares to hundreds of hectares. Drainage lines are areas to which runoff water flows from adjacent areas of rehabilitation on minesites. Control structures apply to these drainage lines to mitigate against erosion and discharge of sediment. The design and assessment of large storage structures and dams on mine sites which are prescribable under the Dam Safety Act 1978 are not addressed in this guideline.
- A stable drainage system requires consideration of geomorphic considerations, design methodology, selection of appropriate input parameters, risk assessment of critical components in the drainage design and application of necessary construction and maintenance techniques. . The guideline avoids a prescriptive approach to drainage design. It presents the elements of drainage design and provides for flexibility in the selection of specific techniques which are dependent on site conditions and management practices.

- The design of drainage works should also required to consider the implications of other legislation and government policy including the Protection of the Environment Operations Act 1997 and the Water Management Act 1999.

Reference

- Mining Act 1992
- Soil Conservation Act 1938
- NSW Department of Mineral Resources. *Mining Operations: Environmental Policy Implementation Principles.* EDP04. September 1998.
- NSW Department of Mineral Resources. *Rehabilitation and Mine Closure* Environmental *Policy Implementation Principles.* EDP05. September 1998.

Design Process and Outputs

The planning of a drainage design system involves:

- stage 1: the development of a 'concept' drainage system and/or
- stage 2: the development of a 'detailed' drainage plan prior to its implementation.

The following details are adapted from Young, R. al et 1998.

Concept Development

The Concept Development can be part of strategic planning in order to provide a feasibility study of viability and estimates of costs or a formalised exercise for major project developments. This stage is generally part of the EIS/development application and the Mining Operation Plan process on plans 5 and 6. There is the opportunity to modify the mining plan to ensure that the drainage plan is feasible.

Stages:

- setting initial objectives for the drainage design
- identifying potential constraints within the drainage catchment area
- consideration of features of pre-mining drainage system, including the location of final discharge points exiting the boundary area and significant channel features (including locations of pools and riffles, channel meanders, channel slope)
- integrating drainage design into the mine plan and examining the range of design options available
- considering the identified constraints and modifying the drainage and/or mine plan
- developing preliminary design options for a drainage plan integrated with the mine plan

Outputs:

- conceptual plan indicating design components, catchment areas and any other relevant features including continuous drainage flow paths to their exit from the lease area or internal drainage to the final void.
- comparison with pre-mining drainage pathways and features
- identification of design constraints
- preliminary information for detailed design plan including estimates of catchment areas, drainage density, peak discharges, channel slopes and configuration(including hydraulic and habitat variability)
- preliminary location and design of any major instream structures
- assessment of feasibility regarding long term stability of the channel, including the requirement for bed control structures.

Detailed Concept Design

This stage is beyond the concept stage in the Mining Operation Plan process where drainage works will be implemented. The development of final landforms in the mining plan will trigger the requirement for drainage plans to be developed to this stage.

The detailed concept design develops the drainage concepts to the stage where the individual components of the drainage system are identified, assigned dimensions, documented on plans, costed and ready for subsequent implementation.

At the closure stage of the mine, the detailed design will provide essential support in the assessment of the final drainage system regarding its long-term stability and ecological sustainability.

Stages:

- refining objectives of the drainage design
- quantifying constraints through site and catchment investigations
- identifying drainage pathways and flow lines
- comparison with pre-mining drainage system for the area
- designing the drainage system to remain stable through projected storm events
- designing the channel to adequately provide sustainable habitats for instream fauna
- identifying individual components and quantifying dimensions prior to implementation

Outputs:

- a plan with detailed contours identifying the proposed channel configuration including:
    - lease boundaries
    - continuous drainage flow paths from upper catchment to the drainage exits
    - lease/property discharge points
    - important instream structural works
    - catchment boundaries of major drainage lines
    - hydraulic and instream habitat features
- a comparison with the pre-mining drainage system particularly for configuration, natural controls, discharge points, drainage density and peak discharges (see risk assessment)
- design details of peak discharges, runoff volumes and duration of flows for the following ARI flows:
    - 1:1 year ARI (5% for trickle flows),
    - 1:2 year ARI (for the dominant flows),
    - 1:100 year ARI
- details of relevant input data and assumptions used for all calculations
- stages of drainage works including details of any temporary works and "sacrifice" areas
- plan details and cross sections of typical channel sections and critical structures indicating:
    - cross sectional areas of the channel and spill-over areas adjacent to the channel banks
    - pool and riffle sequences
    - instream channel controls such as Large Woody Debris
    - rock bed controls which consider fish passage if applicable
    - general features of underlying soil material including details of any soils limitations regarding sodicity, salinity and wet strength
    - construction materials and
    - depths of flow for the different ARI peak discharges
    - the exit point and mitigating measures to reduce the impact of the constructed channel on any existing natural water courses
    - native riparian revegetation to take place along the reconstructed channels so that an adequate riparian zone can be established with emphasis placed on establishing a vegetation succession of ground covers through to trees. This will assist with the :
    - long term stability of the waterway
    - biodivesity both terrestrial and aquatic
    - complementary works where appropriate including:
    - specific erosion control techniques
    - revegetation of adjacent areas with suitable native grass and ground cover species specifically to control soil erosion of spill-over areas
- energy modifiers within the drainage system including:
    - energy dissipators, drop and bed control structures
    - Large Woody Debris such as Log Deflectors etc
    - geomorphic considerations including specific reference to the creation of pool and riffle sequences and adequate channel sinuosity and habitat variability.
- details for each critical structure including:
    - maintenance requirements
    - contingency plans to mitigate risk of failure in worst case scenarios and
    - preliminary information on monitoring requirements.
- risk assessment
    - refer to Australian Rainfall and Runoff 1999 – section 1 of Book 3
    - risk analysis of the drainage alterations to the pre-mining drainage system
    - risk analysis on critical structures in the drainage plan

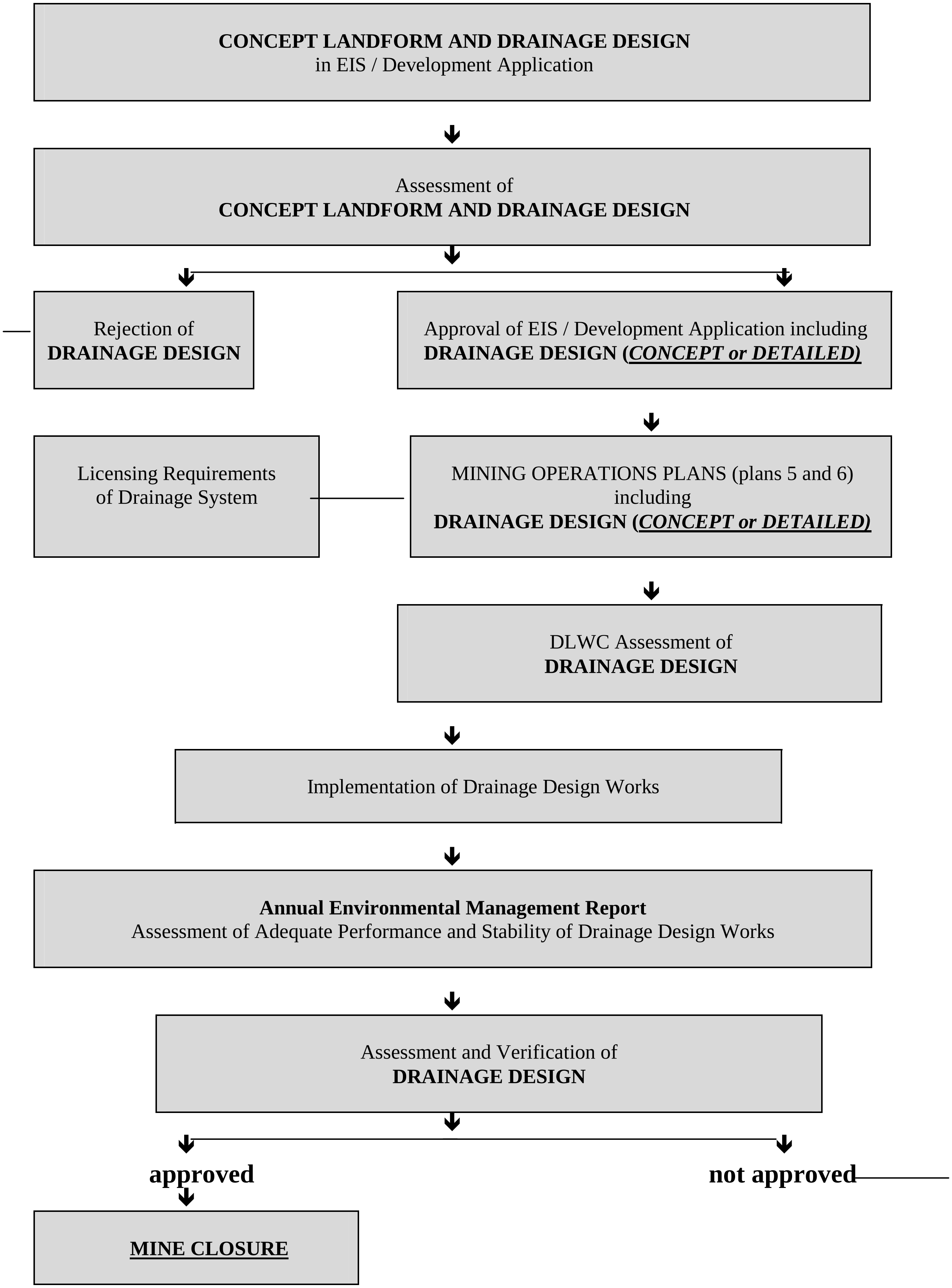
- estimate of likely maintenance requirements and costs

The above list provides a range of considerations for the design of drainage systems.

Assessment Process

The process indicated in Figure 1 below provides opportunities for review and assessment to ensure that proper consideration is given to the initial stage of drainage design. The review of drainage design plans will assess the adequacy of design detail and the basis for selecting different drainage options.

Figure 1. Assessment Process of Drainage Design Works



Requirements

The outputs of the design process for a minesite drainage system are required to:

- provide essential background information and plan details clearly indicating the location of all drainage system features at a suitable scale (maximum of 1:4,000)
- verify the sustainability and long term stability of the proposed drainage system for a range of conditions including:
    - the peak flow 1:100 yr ARI event
    - the dominant flow equivalent to the 1:2yr ARI event and
    - prolonged trickle flows equivalent to a continuous flow of 5% of the 1:1yr ARI event, particularly its impact on ground cover and soil stability within the drainage system
- assess the risk of any adverse impact on downstream drainage systems to ensure that it is minimised.
- include design parameters which conservatively reflect the potential site conditions and provide discharge calculations which meet an acceptable professional standard.
- identify the location of critical structures in the drainage design and provide background design calculations that verify adequate capacity for long term stability

include contingency in the drainage system to minimise damage where flow levels exceed design parameters

- assess the risk of failure of critical components of the drainage system and the impact of site limitations like unstable soil conditions and steep slopes
- minimise maintenance requirements within the drainage system without jeopardising long term stability
- provide detail appropriate for either:
    - a preliminary 'concept plan' or
    - a 'detailed plan' for the implementation of the final works.