



WSUD:

WATER SENSITIVE URBAN DESIGN

BASIC PROCEDURES FOR 'SOURCE CONTROL' OF STORMWATER

A Handbook for Australian practice

STUDENT EDITION – with Index



John R Argue (Editor)

ABOUT THE AUSTRALIAN WATER ASSOCIATION (AWA)

The Australian Water Association is Australia's largest water focussed association. As a not-for-profit, independent organisation, AWA brings together individuals, water utilities, consultancies, businesses, educational and government entities who share an interest in ensuring the sustainable management of Australia's water resources. With such a broad-based, multi-disciplinary membership, it could be expected that there is a diversity of opinions on how best to achieve this goal.

While Australians as a whole are learning the importance of more efficient and effective use of scarce water resources, it is often a harder exercise to convey appreciation of aesthetic, restful qualities of a babbling brook, a reeded pond or a fountain in cities and towns where the natural experience of running water is far removed.

Water sensitive urban design attempts to merge the goals of valuing, managing, and using all urban water resources (roof runoff, stormwater, groundwater and wastewater) to create a sustainable urban environment where humans can connect with nature while living in the city. This can be achieved by combining the skills of planners; community groups; developers; social scientists; building and landscape architects; environmental scientists; hydrologists; hydrogeologists; and civil, water and environmental engineers, all of whom share goals for sustainable water management similar to AWA.

AWA is delighted to offer its support to encouraging the adoption of the practical ideas and concepts described in this handbook for the planning and design of modern water (and human) sensitive cities and towns.

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ABOUT THE STORMWATER INDUSTRY ASSOCIATION (SIA)

Traditional urban water practice, as established in all developed countries over the past 150 years, has been characterised by an almost unquestioned acceptance of the twin notions that a) all operations involved in urban water supply, sewerage and drainage could remain functionally separated, and b) that the most efficient systems would be the largest in scale.

About 20 years ago these concepts started to be seriously questioned. In fact, as investigations have continued and experience gained, it has become now widely accepted that it is likely to be the almost exact opposites of these twin notions that will finally prove to be the more sustainable position.

The SIA was established as an Australia-wide organisation to take a lead in the development of a new generation of more efficient urban water systems based on these 'opposite' notions and to assist in the parallel development of a range of industries for their planning, construction, operation and regulation. Stormwater was an obvious starting point, but the Association has always recognised the need to take a 'total water cycle' approach to urban water matters. The Association takes great pride in what it has achieved in the relatively short time since its inception and the great support it has received from the public in its endeavours.

The term Water Sensitive Urban Design not only links water and urban design together, as they most obviously must be (but often are not), but it also links them with the somewhat provocative word 'sensitive'. We want our cities to be more than just functional, we want them to be happy, healthy and beautiful also. Water is life and Water Sensitive Urban Design is a reminder to its practitioners to take heed of all human values when addressing water as a basic of city living.

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Cover Picture :

Roof gardens on apartment buildings in Woolloomooloo, N.S.W.

WSUD : "Hold the rain where it falls!"

(Photograph : Mandy Argue and Uwe Dombrowski)



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UNIVERSITY OF SOUTH AUSTRALIA
in collaboration with
Stormwater Industry Association and Australian Water Association

WATER SENSITIVE URBAN DESIGN : BASIC PROCEDURES FOR ‘SOURCE CONTROL’ OF STORMWATER

A HANDBOOK FOR AUSTRALIAN PRACTICE
STUDENT EDITION

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ISBN 1 920927 18 2

February, 2013

First printing : November, 2004
Second printing : April, 2005
Third printing : February, 2007
Fourth printing : February, 2008
Fifth printing : February, 2009
Sixth printing : February, 2011
Seventh printing : February, 2013

FOREWORD

It is a great honour to have been asked to write the Foreword to this book. As a city planner and environmental designer, one of my strongest passions has been to find ways to use water as an integrating element to help develop more sustainable human settlements. In my searching for solutions to the challenge of creating more sustainable urban water systems I have benefited greatly from interactions with colleagues from different disciplines who share this passions. John Argue has been one of my valued collaborators in the task of translating Water Sensitive Urban Design from concept to reality.

The wisdom and insight presented in this, a very technical document, cannot be over-looked. The stormwater 'source control' approach to Water Sensitive Urban Design offered here is part of what can be recognised as a major shift in how we approach the planning, design and water infrastructure servicing of our cities and towns. The 'vision' of this approach is to find integrated solutions aligned with the ever-expanding quest of the sustainable cities movement, whether it be retrofitting old catchments or designing new settlements. This is not an insignificant challenge as our world becomes increasingly urbanised.

Fundamentally, the range of concepts, design approaches and technologies presented here, ask the designers to understand the site and its opportunities within a catchment repair context. In doing this, design solutions emerge that are site and catchment responsive. These solutions have a very simple starting point: a view that the rain that falls on our cities and towns is both a resource and something that needs to be managed with care to ensure that our urban water environments are protected and urban catchments are repaired.

The challenges of shifting to the Water Sensitive Urban Design approach presented in this book should not be under estimated. Our traditional urban water systems are designed and operate within complex and often conservative institutional, political and technical settings. In simple terms, Australians like their suburbs and like to keep their feet dry when it rains. Further, our urban development industry has become efficient at delivering that 'product' and local government, the owner of most of the stormwater facilities, is often conservative in its approach. But as communities begin to increasingly recognise the value and importance of water in urban environments, they are demanding that new systems and solutions be found.

While the barriers to innovation are significant, John Argue and his team have been able to capture the imagination of their clients and show through research, demonstration and evaluation that it is possible to find ways to help re-integrate our cities with 'place'. In doing this, the techniques and methods have been refined to assist designers to establish an urban hydrology that serves human needs as well as looking after ecological health.

As this research effort has been taken more and more into practice, the team has looked for ways to use these skills within trans-disciplinary teams. By seeking design solutions that integrate the skills of urban planning, ecological design, landscape and engineering, what has emerged are important 'case studies' of potentially far more sustainable patterns of human settlements. Water has helped to be an integrating element in the design process.

I know that this book fills a much-needed gap in the emerging practice of Water Sensitive Urban Design. I am confident that this contribution will have a significant and lasting influence on practitioners involved in designing, building and managing urban Australia, and probably beyond. The outcome will be better, more sustainable places to live that respect and celebrate 'place' and their associated water environments.

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September, 2004

TABLE OF CONTENTS

FOREWORD	i
PREFACE	viii
ACKNOWLEDGEMENTS	x
LAUNCH AND DEDICATION	xi
FUTURE DIRECTIONS	xi
PRINCIPAL DISTRIBUTORS	xi
DISCLAIMER	xi
ENDORSEMENTS AND AWARDS	xii
 1. INTRODUCTION : SOME BASIC CONCEPTS	 1
1.1 INTRODUCTION.....	1
1.2 DETENTION AND RETENTION OF STORM RUNOFF	1
1.2.1 Flood control.....	1
1.2.2 Other comparisons and potential of stormwater on-site retention (OSR).....	2
1.2.3 Detention technology : its role in contemporary stormwater management.....	4
1.3 RETENTION-BASED STORMWATER MANAGEMENT : WATER-SENSITIVE URBAN DESIGN (WSUD).....	5
1.3.1 Primary aims	5
1.3.2 Site-dependent options and opportunities	5
1.3.3 Runoff categories and treatment	5
1.3.4 Soils : permeability and water-reactivity	10
1.3.5 Aquifer water quality, potentiometric gradients and use	12
1.3.6 Bores : extraction rates and recharge rates.....	13
1.3.7 Aquifer recharge/retrieval protocol.....	13
1.3.8 OSR and site capability.....	14
1.4 WSUD – STORMWATER ‘SOURCE CONTROL’ : THE VISION AND ITS LIMITATIONS.....	15
1.4.1 Conventional urbanisation : typical consequences in a natural catchment	15
1.4.2 WSUD : planning for urbanisation in a natural or predominantly natural catchment.....	18
1.4.3 Retrofitting WSUD practices into an overdeveloped urban catchment	19
1.4.4 The problem of ‘oversizing’	20
1.4.5 WSUD – stormwater ‘source control’ : the limitations	20
 2. DEVICES AND SYSTEMS USED IN STORMWATER MANAGEMENT	 21
2.1 INTRODUCTION AND DIRECTORY	21
2.1.1 Selection	21
2.1.2 Hydrological design.....	21
2.1.3 Other design and option issues	21
2.2 ROOF SYSTEMS.....	21
2.3 “LEAKY” WELLS AND INFILTRATION TRENCHES (see Chapter 5).....	21
2.4 FILTER STRIPS.....	21
2.5 POROUS AND PERMEABLE PAVEMENTS	21
2.5.1 Porous pavements	21
2.5.2 Permeable pavements.....	21

2.6	SWALES AND BIO-RETENTION SWALES	21
2.7	TRASH RACKS, BASKETS AND BOOMS	21
2.8	CATCH BASINS AND “SOAK PITS”	21
2.8.1	Catch basins	21
2.8.2	“Soak pits”	21
2.9	WATER QUALITY IMPROVEMENT STRUCTURES.....	21
2.9.1	Water quality inlets	21
2.9.2	Oil/grit separators.....	21
2.9.3	Fine sediment removal/retention device	21
2.10	“FIRST FLUSH” TANK – PARTITIONING AND “FIRST FLUSH” STORMWATER RUNOFF	21
2.11	INFILTRATION BASINS.....	21
2.12	EXTENDED DETENTION BASINS	21
2.13	WET DETENTION BASINS	21
2.14	CONSTRUCTED WETLANDS.....	21
3.	THEORY, DATA AND SOME IMPLEMENTATION ISSUES.....	23
3.1	INFILTRATION OF STORM RUNOFF	23
3.1.1	Basic theory.....	23
3.1.2	‘Lifespan’ and the effect of sediment accumulation on permeability	24
3.1.3	Surface blockage and ‘lifespan’	25
3.2	SOIL PERMEABILITY AND PERCOLATION AT DEPTH.....	27
3.2.1	Percolation of storm runoff.....	27
3.2.2	Hydraulic conductivity and its measurement	28
3.3	RUNOFF VOLUME AND TIME BASE τ IN ‘SOURCE CONTROL’ TECHNOLOGY	30
3.4	AN EXCURSION INTO ‘CONTINUOUS SIMULATION’ MODELLING	31
3.4.1	Introduction : storm successions and the ‘design storm’ approach.....	31
3.4.2	‘Continuous simulation’ modelling : some background	31
3.4.3	Application of Figure 3.7 : the “soakaway” and ‘dry’ pond cases	33
3.4.4	Application of Figure 3.7 : the rainwater tank cases	36
3.4.5	Catchment-wide modelling and catchment performance	37
3.5	THE <i>MODIFIED</i> DESIGN STORM METHOD	38
3.5.1	Introduction	38
3.5.2	Emptying time – ‘natural’ drainage	39
3.5.3	Emptying time criteria – analyses.....	40
3.5.4	The <i>modified</i> design storm method.....	41
3.5.5	Catchment performance and the <i>modified</i> design storm method	42
3.6	RUNOFF COEFFICIENTS	43
3.7	RAINFALL.....	46
3.7.1	Rainfall and the ARI, Y = 0.25-year stormburst intensity	46
3.7.2	The rainfall data base : some current issues	46
3.8	ON-SITE RETENTION (OSR) TECHNOLOGY : SOME DO’S AND DON’TS	47
3.9	SOME EXAMPLES OF ON-SITE RETENTION PRACTICE.....	49

4.	GETTING STARTED WITH ‘SOURCE CONTROL’ IN WATER-SENSITIVE URBAN DESIGN	53
4.1	INTRODUCTION : FOUR ‘SOURCE CONTROL’ CATEGORIES.....	53
4.2	STORM RUNOFF MANAGEMENT IN CATEGORY 1 SYSTEMS.....	54
4.2.1	Three basic stream management scenario/strategies.....	54
4.2.2	A fourth strategy for stormwater management	56
4.2.3	The <i>design storm</i> critical duration : $T_{C(total)}$ and $T_{C(local)}$	56
4.2.4	The <i>design storm</i> in site drainage : on-site storage and the critical storm	57
4.2.5	‘Top end’ site drainage	58
4.2.6	Average recurrence interval (ARI) and installation components.....	61
4.3	CATEGORY 2 SYSTEMS	63
4.3.1	Strategy A and strategy B	63
4.3.2	Aquifer access or slow-drainage provision	64
4.4	CATEGORY 3 SYSTEMS	64
4.5	CATEGORY 4 SYSTEMS	65
4.6	COUNCIL RESPONSIBILITIES AND STRATEGIC PLANNING	66
4.6.1	The four networks	66
4.6.2	The four networks : management issues and ARIs.....	67
4.6.3	The ‘benchmark year’ concept.....	67
4.6.4	Summary of tasks for a stormwater management agency.....	68
4.6.5	The regime-in-balance strategy (see Section 4.2.1).....	69
4.6.6	The <i>yield-minimum</i> strategy (see Section 4.2.1).....	71
4.7	MIMICKING NATURAL CATCHMENT ‘LOW FLOWS’ IN THE URBAN LANDSCAPE.....	71
5.	RUNOFF QUANTITY CONTROL (Category 1 systems)	73
5.1	FOUR BASIC PROCEDURES.....	73
5.1.1	Introduction.....	73
5.1.2	The Procedure 1 cases.....	73
5.1.3	The Procedure 2 cases.....	75
5.1.4	Infiltration or ‘dry’ ponds	79
5.1.5	The Procedure 4 cases.....	80
5.2	EXAMPLES OF CATEGORY 1 INSTALLATIONS USING PROCEDURES 1B, 2A, 2B, 2C, 3, 1B/4A AND 2B/4B	83
5.3	SOME ADDITIONAL DESIGN CONSIDERATIONS.....	96
5.3.1	Design modification to meet emptying time criteria.....	96
5.3.2	Impact of site constraints and/or economics on design.....	96
5.3.3	Combination structures	96
5.3.4	High groundwater environments.....	96
5.4	SLOW-DRAINAGE PIPES : SOME ISSUES AND OPTIONS	96
5.5	ON-SITE RETENTION (OSR) IN CLAY SOIL SITES : THE FINAL OPTION	96
5.6	DETENTION-RETENTION : A LAST WORD	96

6.	ASPECTS OF STORMWATER QUALITY.....	97
6.1	URBAN STORMWATER PARAMETERS AND OTHER FACTORS	97
6.1.1	Chapter overview	97
6.1.2	Physical water quality parameters.....	97
6.2	CHEMICAL WATER QUALITY PARAMETERS	98
6.2.1	The nitrogen cycle	99
6.2.2	The phosphorous cycle	101
6.2.3	Heavy metals.....	101
6.2.4	Biological water quality parameters	102
6.3	CHARACTERISTICS OF URBAN STORMWATER POLLUTION	104
6.3.1	Effect of urbanisation on runoff quality.....	104
6.3.2	Sources of pollution	104
6.3.3	Factors influencing the degree of contamination	105
6.3.4	Contaminants and some of their impacts	106
6.3.5	Data on pollution concentrations in urban runoff	107
6.3.6	Conclusion	108
6.4	QUALITY STANDARDS FOR RECEIVING WATERS	112
6.5	QUALITY STANDARDS FOR AQUIFER RECHARGE	112
7.	STORAGES FOR POLLUTION CONTROL (Category 2 systems)	113
7.1	GENERAL BACKGROUND	113
7.2	DESIGN OF SIMPLE STRATEGY A SYSTEMS : PROCEDURE 5A	116
7.2.1	Introduction.....	116
7.2.2	PROCEDURE 5A : Strategy A design for sub-structure storages receiving cleansed runoff from paved surfaces	117
7.3	MORE COMPLEX STRATEGY A SYSTEMS : PROCEDURE 6A	119
7.3.1	Introduction.....	119
7.3.2	PROCEDURE 6A : Strategy A design for sub-structure storages.....	120
7.4	DESIGN OF SIMPLE STRATEGY B SYSTEMS : PROCEDURE 5B.....	121
7.4.1	Introduction.....	121
7.4.2	PROCEDURE 5B : Strategy B design for sub-structure storages receiving cleansed runoff from paved and other ground-level filter-integrated surfaces	122
7.5	MORE COMPLEX STRATEGY B SYSTEMS : PROCEDURE 6B.....	123
7.5.1	Introduction.....	123
7.5.2	PROCEDURE 6B : Strategy B design for sub-structure storages	123
7.6	INFILTRATION SYSTEMS : FURTHER DESIGN CONSIDERATIONS	124
7.6.1	Introduction.....	124
7.6.2	Flood control, pollution control interaction	125
7.6.3	Design of non-gravel retention devices	126
7.6.4	Design of Retention Trenches.....	127
7.6.5	Strategy A systems : consequences for ‘lifespan’ and maintenance	127
7.7	EXAMPLES : PROCEDURES 5A and 6A.....	128
7.7.1	Introduction.....	128
7.7.2	Example 7.1a : Use of Figure 7.1 graph (also Appendix C)	129
7.7.3	Example 7.1b : Procedure 5A (Strategy A)	129

7.7.4	Example 7.1c : Procedures 5A and 6A	130
7.7.5	Example 7.2 : Infiltration surface without ponding	131
7.7.6	Example 7.3 : “Soakaway” at Plympton Anglican Church.....	133
7.7.7	Example 7.4 : Design of Strategy B System.....	135
7.8	‘FILTER-STRIP’ SWALES.....	137
7.8.1	Introduction.....	137
7.8.2	‘Filter-strip’ swales : General description.....	138
7.8.3	Catchment area	138
7.8.4	Procedures 7 and 8 : Introduction	140
7.8.5	Specification	141
7.8.6	‘Lifespan’	142
7.8.7	Procedures 7 and 8 zones in Figure 7.7 – illustrations.....	143
7.8.8	‘Filter-strip’ swales in Australian environments.....	144
7.8.9	Design for flows greater than $Q_{0.25}$	145
7.8.10	Procedures 7 and 8 – the step-by-step process.....	145
7.8.11	Construction : some recommendations	147
7.9	‘FILTER-STRIP’ SWALES : ILLUSTRATIVE EXAMPLES	148
7.9.1	Introduction to Examples 7.5 and 7.6 : Swales for two residential streetscapes.....	148
7.9.2	Example 7.5 : Conventional development streetscape with swale.....	149
7.9.3	Example 7.6 : ‘Filter-strip’ swale for medium-density residential development.....	151
7.10	‘FILTER-STRIP’ SWALES : CONCLUDING COMMENT	153
8.	STORAGES FOR STORMWATER HARVESTING (Category 3 Systems)	155
8.1	INTRODUCTION	155
8.1.1	Current drainage practice	155
8.1.2	Chapter outline and qualification	155
8.2	ASPECTS OF RESIDENTIAL ROOF RUNOFF HARVESTING	156
8.2.1	Storage size – practical considerations	156
8.2.2	Considerations for maximising roof runoff capture.....	156
8.2.3	Appropriate uses of roof runoff	157
8.2.4	Roof runoff quality considerations	158
8.2.5	Practical considerations for RRHSs.....	160
8.3	AVERAGE ANNUAL YIELD PROCEDURES : BACKGROUND DISCUSSION	161
8.3.1	Approach.....	161
8.3.2	RRHS average annual yield model	161
8.3.3	Model requirements	161
8.4	AVERAGE ANNUAL YIELD PROCEDURES	166
8.5	INDICATIVE YIELDS FOR STATE AND TERRITORY CAPITALS	168
8.6	A REGIONAL STORMWATER HARVESTING MODEL	173
8.6.1	Introduction to Procedure 10	173
8.6.2	Figure 8.5 explained.....	173
8.6.3	Procedure 10	174
8.6.4	Illustrative examples	175

9.	CONCLUDING DISCUSSION	181
9.1	OVERVIEW	181
9.2	SOME ISSUES	181
9.2.1	Relationship of the Handbook to Current Australian Practice	181
9.2.2	Resistance to change.....	181
9.2.3	Differences in Northern, Intermediate and Southern Australian design outcomes	181
9.2.4	Pollution control in Australian urban catchments.....	181
9.3	THE NEED FOR CASE STUDIES.....	181
9.3.1	Stream management scenario/strategies	181
9.3.2	Detention/retention in the urban landscape.....	181
9.3.3	Rainwater tanks and flood control	181
9.3.4	Cost and policy matters	181
9.4	RESEARCH DIRECTIONS	181
9.4.1	Acceptable ‘emptying time’ for OSR devices	181
9.4.2	Detention/retention : spatial distribution.....	181
9.4.3	Problems of water retention and ‘reactive’ soils.....	181
9.4.4	Pollution control – fine sediments.....	181
9.5	CONCLUSION.....	181
	BIBLIOGRAPHY	183
	APPENDICES :	
A :	Measurement of hydraulic conductivity in soils for use in water-sensitive design	A1 – A6
B :	Rainfall I-F-D for typical Northern, Intermediate and Southern Australian locations and Perth	B1 – B6
C :	Pollution control/retention devices (“soakaways”) for five Australian climate zones	C1 – C3
D :	Design graphs for ‘filter strip’ swales for five Australian climate zones.....	D1 – D3
E :	Graphs for a regional stormwater harvesting model for five Australian climate zones.....	E1 – E3
F :	Tutorial exercises	F1 – F5
	INDEX	I1 – I

PREFACE

The seeds of this Handbook may be found in Chapter 3 of the ARRB publication, “Storm drainage design in small urban catchments” (Argue, 1986). But it was the fledgling research programme carried out in the late 1980s by staff and students of the former School of Civil Engineering (SA Institute of Technology, now University of South Australia) to find answers to many questions raised by ‘Chapter 3’, that provided the technical foundation and impetus for the present Handbook.

The early achievements of that programme, in particular the New Brompton Estate project, led in 1993 to creation of the Urban Water Resources Centre (UWRC), a research unit within the University of South Australia devoted to providing :

“...a focus for collaborative research, demonstration projects and education programmes aimed at improving the efficiency of urban water management while simultaneously increasing amenity and contributing to the achievement of ecologically sustainable development” (UWRC, 1993)

When Professor Wolfgang Geiger, a leading environmental engineering researcher from Essen University, Germany, visited Adelaide in the early 1990s, he was impressed by local progress in stormwater management and proposed a jointly authored design Handbook based on the German ATV (Abwassertechnische Vereinigung EV) procedures, modified to suit Australian practice and conditions.

About the same time, a group of environmental planners, engineers and landscape architects in Perth coined the term **water-sensitive urban design (WSUD)** to describe their thinking about total water cycle management in the urban landscape. The overlap between this concept and the research being conducted in Adelaide in the domain of stormwater ‘source control’ was striking. Clearly, the proposed Argue/Geiger document would be a valuable sub-set within the broader WSUD context.

A major submission was made to provide research infrastructure to support the programme of basic and applied research and demonstration projects. This was called “Watermark 21” – urban water cycle technology for the new century – and submitted to the appropriate national grants agency. The application was not successful, however the project was pursued in modified form.

For reasons beyond the control of either Wolfgang or myself, the original design Handbook proposal did not eventuate; however, many hours of discussion and review of draft material did take place in Essen and in Adelaide throughout the 1990s. The material in the present Handbook as well as its overall philosophy were strongly influenced by these most productive interactions. During this period, UWRC operated as a self-funded unit, generously supported by the University of South Australia, solely in the domain of innovative water cycle management technology.

On occasions, the group produced concept designs for clients, sometimes it operated on projects in collaboration with government agency or council staff, and sometimes with local consulting engineers, architects or builders. These projects provided opportunities to convert ‘in-house’ research concepts and findings into operating systems and, in the process, confront and overcome the inevitable practical problems posed by ‘real world’ situations. The Handbook encapsulates the wealth of experience gained through these most valuable interactions.

WSUD has gained wide recognition in Australia since the late 1990s when experience with constructed (stormwater) wetlands in Perth, Adelaide, Canberra and Melbourne, and success achieved with urban ASR (aquifer storage/recovery) and stormwater-based water cycle schemes in Adelaide and Newcastle (Newcastle City Council and University of Newcastle) were reported at national and international conferences: the AMCORD document (Dept of Housing and Regional Development, 1995) also dates from this period. This led to new initiatives in Melbourne (CRC for Catchment Hydrology) and Brisbane (WBM Oceanics and Brisbane City Council) in particular. WAWA, NSW Dept of Housing, CSIRO, SIA along with Melbourne Water, Brisbane City Council, the Lower Hunter Central Coast regional councils and Upper Parramatta River Catchment Trust (Sydney WSUD capacity building program) have all made major contributions to the field in support of its technical base or case studies or promoting WSUD principles through professional gatherings and conferences.

These achievements and activities are part of the early adoption phase of a new and quite revolutionary technology – the ‘paradigm shift’ referred to by some of its contributors. However, the great bulk of urban development carried out across the nation does so with scant regard for WSUD and for the principles of sustainability upon which it is based.

- CBD and residential roofs are still being directly-connected to street drainage networks despite many alternative solutions being available;
- car parking areas are still being constructed with hotmix surfaces though porous/permeable paving systems are readily available;
- drainage in new residential sub-divisions is still being installed using impervious pipes and channels in circumstances where swales would bring both flood security and a ‘softening’ of the urban landscape.

The Handbook provides the “how to” tools to support the wider adoption of the technology through its presentation of approaches to solving everyday problems of small-scale stormwater management – flood control, pollution control and stormwater harvesting. The procedures are simple yet soundly based in theory and practice, set out for the most part in step-by-step format. Lessons learned from over 20 case study installations have been incorporated. Material contained in the Handbook has been trialled over the past six years in undergraduate and postgraduate classes at the University of South Australia and duly modified as a consequence of student review and comment.

WSUD is committed to sustainability. It provides an urgent call to our post-industrial urban society to reclaim the seasons, to celebrate the presence of rainfall and sensitively managed runoff in the built environment and, by so doing, to protect the community and, at the same time, promote amenity and restore treasured waterways and their biotic communities. It is no accident, I believe, that the ‘Watermark’ demonstration projects have – without exception – produced community values and attitudes to ‘place’ undreamed of by those involved in the technical process of their creation.

The indigenous peoples who inhabited our great continent for tens of thousands of years before European settlement and who lived in sustained harmony with its bounty and vicissitudes, lived in spiritual union with and showed profound reverence for the land, the water and its fauna and flora. The cultural and technological differences brought by the ‘newcomers’ have resulted in a wide range of environmental problems for our nation that non-indigenous Australians, in particular, are being called upon to solve as a matter of urgency.

WSUD has, unquestioningly, a significant role to play in this process : its end result could well be a new appreciation for community values within our urban concentrations, hand-in-hand with a reverence for the land and water and for our native fauna and flora.

Users of this Handbook are challenged to embrace this vision and to apply the document’s procedures, guidelines and strategies not in a prescriptive way but, rather, to use them as building blocks for creating unique solutions that achieve not only their basic technical objectives but also goals of improved amenity – places of community harmony and spiritual enrichment – as well as increased quality of the natural environment.

John R. Argue

Adelaide

November, 2004

The past four years have seen many up-dates, additions and improvements made to the original text of the Handbook, mainly as consequences of interaction between participants and myself as Presenter at 29 two-day workshops conducted across the nation under the auspices of Engineering Education, Australia (EEA) between August 2005 and the end of 2008. A special *Student Edition* of the Handbook was developed for use in these workshops which are planned to continue in 2009 and beyond.

It is too early to declare how many ambitions of the Handbook as set out in the Preface, above, have been or are being met. Certainly, community awareness and concern about *water* have increased markedly over the last four years resulting from restrictions on use, Murray-Darling Basin issues affecting south-eastern Australia, desalination initiatives, recognition of climate change and interest in stormwater harvesting, wastewater treatment/re-use and environmental flows. Urban design that is water-sensitive provides solutions to many of the problems raised by these concerns, making the process of up-date, improvement and promotion of the Handbook procedures an on-going task of importance into the foreseeable future.

John R. Argue

Adelaide

February, 2009

ACKNOWLEDGEMENTS

The story which lies behind the production of this Handbook – theory development, experimentation, case example installations, monitoring, class trialling, text development – has involved many contributors, all of whom have responded to the opportunities inherent in 'source control' of stormwater with energy and commitment. The base of that support has been firmly located in South Australia, but a handful of agencies and personnel outside SA have also provided significant input and encouragement.

First to be recognised is University of South Australia – the primary source of support – particularly during years 1993 to 1999 when the bulk of the intellectual base of the project was laid. Instrumental in securing and maintaining this support were Mr. Bob Taylor of Techsearch and Mr. Jim Mitkas who acted as Business Manager for UWRC throughout the period. Members of the UWRC Advisory Board under the Chairmanship of Mr. B. C. ('Skip') Tonkin, AO – Kathryn Bellette, Leon Broster, Don Bursill, Jason Kuchel, Bill Lipp, Dr. Kevin Mills, Dr. Dennis Mulcahy, Professor Stephen Priest – also rendered sterling service, encouragement and sound advice. In more recent time the support of Professors Robin King, Ian Davey, Patrick James and Bill Richards is acknowledged with gratitude.

Mr. Bill Lipp in his role as Manager, Stormwater Section of Transport SA, deserves a special 'thank you' for his role in forging links between UWRC and municipal agencies, and for finding funds to bring collaborative projects to fruition. Other government agencies who provided significant help to and encouragement of field projects included South Australian departments responsible for, in particular, water resources, stormwater pollution control and planning/environment : their departmental names have changed over the period, but never their support. In this connection Messrs Murray Hutchesson and Peter Hoey deserve special mention. The Torrens and Patawalonga Catchment Water Management Boards and General Manager Alan Ockenden and Technical Manager Keith Downard are also recognised in this context.

Projects organised collaboratively with SA municipal agencies provided opportunities to convert theory into field installations and, thereby, to learn from experience: the councils of Charles Sturt, Marion, Burnside, West Torrens, Holdfast Bay, Campbelltown, Salisbury, Mount Barker, Mitcham and Whyalla were active in these respects. Success in these ventures led to collaborative projects interstate with Newcastle City Council, Wingecarribee Shire, Upper Parramatta River Catchment Trust (Dr. Stephen Lees) and Baulkham Hills Council (NSW Stormwater Trust). Waitakere City, New Zealand, may be added to this list as an overseas collaborator.

Commonwealth government support is also acknowledged. Federal funds were provided through a *Coasts and Clean Seas* grant for a permeable paving project leading to some of the data on which Table 3.1 is based.

Support for two significant stormwater 'source control' installations came from an unexpected source – the Adelaide Diocese of the Anglican Church led by former Archbishop, Dr. Ian George AO. Those involved in bringing these projects to fruition included Archdeacon John Collas, the Reverends Peter Simmons, Barry Davis and Peter Miller and the parishioners of Warradale and Plympton.

The concept of bringing the detailed design knowledge gained from all of these initiatives into a manual or handbook of practice under the heading of Water Sensitive Urban Design owes its origins to Mr. John Wood, former Executive Director of the Stormwater Industry Association. John recognised the need for something more than 'guideline' information readily available at that time. SIA's endorsement of the Handbook is due, largely, to representations made by Mr. Richard Clark, Chairman of the South Australian Chapter. Similar representations by Mr. Richard Marks within the ranks of Australian Water Association are also acknowledged with gratitude.

An early draft of the document was reviewed by some leading researchers and practitioners in the (Australian) field of urban drainage and 'source control' technology. These included Dr. Geoff O'Loughlin, Dr. Peter Coombes, Mr. Chris Tanner, Mr. Ian Rowbottom and Dr. Simon Beecham. Simon Beecham's review, in particular, resulted in the inclusion of a most valuable introductory section (written by Simon) for Chapter 6 : his contribution will be appreciated by all users of the Handbook. Ms. Megan Flint's input to Chapter 6 is also acknowledged.

There is a small army of supporters who have contributed their skills and encouragement to the Handbook project in a variety of ways. Particular thanks go to Cheryl and David Wilson for their skill in typing, formatting and in the graphics input to the final document; Carolyn Bellamy has handled administrative tasks associated with the Handbook with cheer and efficiency. Encouragement and contributions in many forms over the long period of the Handbook's development have been provided by Kim Read, OAM, Mark Brindal, MP, Hon. David Wotton, Lyn Parnell, Jay Hogan, Ken Schalk, Chris Tually, Dr Clive Jenkins, Dr Peter Mitchell, Frank Smith, Dr. Phil

Crawley, Dr. John Sabine, Rosalie Disney, Stephen West, Colin Pitman, Brian Taylor, Garry Moore, Sydney Wood and Rev. Tony Cox of the Green Worship Fellowship. Important, critical input to the Handbook came from undergraduate and post-graduate students of Urban Hydrology classes and Workshop participants at the University of South Australia since 1998.

Finally, there are the partners and families of the Principal Contributors – particularly Tina Pezzaniti and Jan Argue – who have maintained their belief in the value and importance of the Handbook project throughout its long gestation period. Their support, in no small measure, has contributed to the successful outcome of a project which, we trust, will lead to greater use of improved urban water management practices across the nation : we thank them sincerely for their patience and forbearing.

LAUNCH AND DEDICATION

The First Edition of the Handbook was launched by Hon. Dr. Don Hopgood, Chair, “Water Proofing Adelaide” Advisory Panel, at the Engineers Australia WSUD2004 “Cities as Catchments” Conference held 21 – 24 November, 2004, in Adelaide.

The Handbook was dedicated and commissioned by Reverends Barry Davis and Peter Miller in the Community Garden, Plympton Anglican Church, Adelaide, on Sunday 21 November, 2004.

FUTURE DIRECTIONS

It is the intention of the Principal Contributors to the Handbook to maintain the content of the document in as up-to-date form as possible. To this end, users are encouraged to provide information to UWRC on any aspects of the Handbook’s procedures, guidelines or strategies which produce interesting or unexpected outcomes or which raise issues of limited application. This feedback should be addressed to :

john.argue@unisa.edu.au

Corrections, alterations and improvements resulting from this and other feedback are incorporated into each **new printing** of the Handbook. The document should therefore be seen as a ‘work in progress’ and not a static, once-and-for-all-time manual.

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ENDORSEMENTS



Australian Water Association : November, 2004

Stormwater Industry Association : November, 2004



Department of Water, Western Australia : January 2008

AWARDS

Engineering Excellence – Engineering Reports
Engineers Australia, South Australia Division : 2005

Excellence in Stormwater Education
SIA National Awards : 2005

National Engineering Excellence Awards – Finalist
Engineers Australia : 2005

1. INTRODUCTION : SOME BASIC CONCEPTS

1.1 INTRODUCTION

The principles which govern the layout and detailing of storm drainage including site drainage as designed by leading practitioners has undergone greater change in the last 30 years than at any other period of similar length since wastewater collection and disposal technology (including stormwater) was introduced to Europe in the latter half of the 19th Century. For most of the 100 years prior to the 1960s, the philosophy of “collect and dispose of storm runoff as completely and as quickly as possible” dominated general practice throughout the developed world : its dire consequences are now being experienced in the older cities of Europe, UK and North America (Pearce, 2004). Little thought, if any, was given in that philosophy to :

- increased downstream flooding, and,
- pollution of receiving waterways.

The 1960s saw two significant changes in direction : the first was development of **detention** technology carried out at both individual premises and sub-division/catchment scales. The second change was the introduction of the major/minor philosophy for managing the bulk of storm runoff flows generated in urban landscapes (Wright-McLaughlin Engineers, 1969; Argue, 1986; I.E.Aust., 1987).

Australian practice has made wide use of detention basins in city-wide and municipal planning; on-site stormwater detention (OSD) has not been as universally accepted, but many Councils in Sydney and Melbourne as well as major cities such as Wollongong, Newcastle and Canberra, introduced OSD policies in the 1990s which apply, in particular, to cases of re-development (UPRCT, 2005).

1.2 DETENTION AND RETENTION OF STORM RUNOFF

1.2.1 Flood control

The primary goal of detention, whether practised at the scale of the individual residence or the suburb, is flood peak reduction achieved through temporary ponding. **Detention** refers to the holding of runoff for relatively short periods to reduce peak flow rates and later releasing it into natural or artificial watercourses to continue in the hydrological cycle as channel flow, evaporation, groundwater recharge, input to lakes and the ocean, etc. The volume of surface runoff involved in the temporary ponding process is relatively unchanged by it.

On-site stormwater retention (OSR) has been the logical next step in the progress of detention/retention technology. **Retention** refers to procedures and schemes whereby stormwater is held for relatively long periods causing it to continue in the urban water cycle via domestic use (in-house and outdoors), industrial uses and the natural processes of infiltration, percolation, evaporation, evapotranspiration, but not, usually, via direct discharge to natural or artificial watercourses (after Argue, 1986).

The differences between OSD and OSR are more than cosmetic with OSR installations being typically smaller (in volume) as the following illustration shows.

Consider a hypothetical, 60 lot sub-division in Parramatta, NSW, where each ‘quarter-acre’ block (1,000 m²) with house (300 m² roof) and 150 m² paved area, illustrated in Figure 1.1, is re-developed for dual-occupancy (500 m² roof area, 300 m² paved area). Hydrological modelling using the 1-hour, ARI, Y = 100-years design storm, is carried out on the sub-division to determine :

1. Runoff hydrograph for ARI, Y = 100-years 1-hour storm on each (present development) site.
2. Runoff hydrograph for ARI, Y = 100-years 1-hour storm on each re-developed site.
3. What peak flow rate would occur at the downstream end of the sub-division (60 sites) in the ARI, Y = 100-years, 1-hour storm, **before** re-development?
4. What peak flow rate will occur at the downstream end of the sub-division (60 sites) in the ARI, Y = 100-years, 1-hour storm, **after** re-development (without detention/retention measures)?

5. What SSR (site storage requirement) associated with an OSD device is needed on each re-developed site to ensure that the exit peak flow from the sub-division (60 sites) equals the exit peak flow before re-development?
6. What on-site volume of storage must be provided with an OSR device on each re-developed site to ensure that the exit peak flow from the sub-division (60 sites) equals the exit peak flow before re-development?

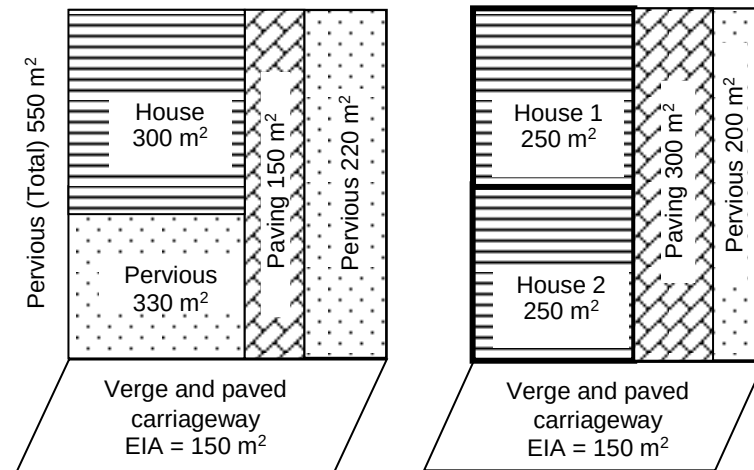


FIGURE 1.1 : Layouts of residential developed and re-developed allotments

Runoff hydrographs for the present and re-developed cases (items 1 and 2, above), are shown in Figure 1.2.1a. Hydrographs for the ARI, Y = 100-years storm applied to the 60 adjacent sites before and after re-development (items 3 and 4, above), are shown in Figure 1.2.2a.

The storage volume (SSR) required for an OSD device to achieve the required reduction in peak exit flow from the sub-division (illustrated in Figure 1.2.2a) is 28.2 m^3 per site (item 5, above). The volume of on-site storage required to achieve the same reduction using OSR devices is 18.3 m^3 per site (item 6, above). Additional graphical displays for the ARI, Y = 100-years 1-hour storm used in the illustration are included in Figure 1.2, in particular :

- hydrographs for re-developed site and re-developed site plus OSD device (Figure 1.2.1b);
- hydrographs for re-developed site and re-developed site plus OSR device (Figure 1.2.1c);
- hydrographs for 60 re-developed sites without and with OSD flood reduction (Figure 1.2.2b);
- hydrographs for 60 re-developed sites without and with OSR flood reduction (Figure 1.2.2c).

This illustration is little more than a ‘skirmish’ with the technical issues involved : Scott et al (1998) and Argue and Pezzaniti (2001) have explored the issue in much greater detail (see also Section 5.6). The re-development of 60 adjacent ‘quarter-acre’ lots is a most unlikely scenario, however, correct hydrological modelling of actual cases yields similar results.

Those seeking a logical rather than modelling explanation for the above outcome will find it in the fact that retention devices can withdraw some 20% – 30% of the flood wave from the drainage path. Comparison of Figures 1.2.2b and 1.2.2c reveals the benefits of this for downstream flood control.

1.2.2 Other comparisons and potential of stormwater on-site retention (OSR)

Cost : Accepting the outcome, above, as typical leads to the important conclusion that an OSR solution is likely to be the cheaper of the two options since the volume of storage required is less. But the cost advantage does not stop there. On-site retention devices (see Figures 1.3, 1.4 and 1.5) are simple and, in most circumstances, their depths are not limited – as OSD installations are – by street drainage invert levels. These advantages can result in significantly smaller capital outlays.

Single dwelling case

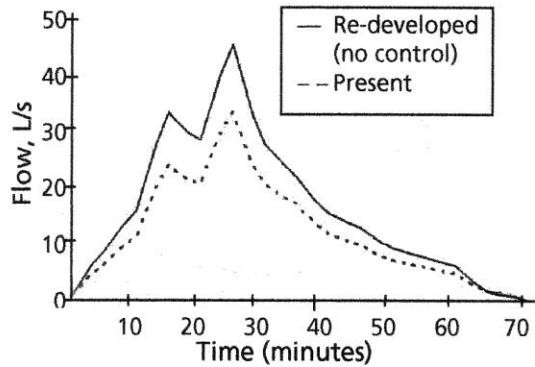


FIGURE 1.2.1(a) : Parramatta, N.S.W.
Hydrographs for present and re-developed
site 100-year ARI, 1-hour storm

60 dwellings case

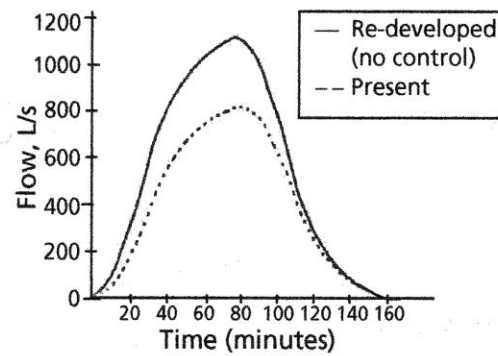


FIGURE 1.2.2(a) : 60 sites
each lagged by 1 minute

USING ON-SITE DETENTION, OSD

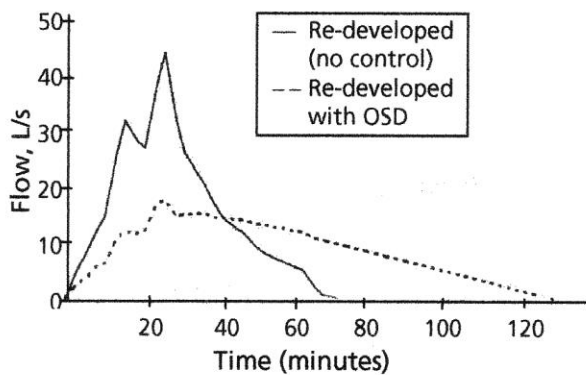


FIGURE 1.2.1(b) : Re-developed
with possible OSD solution

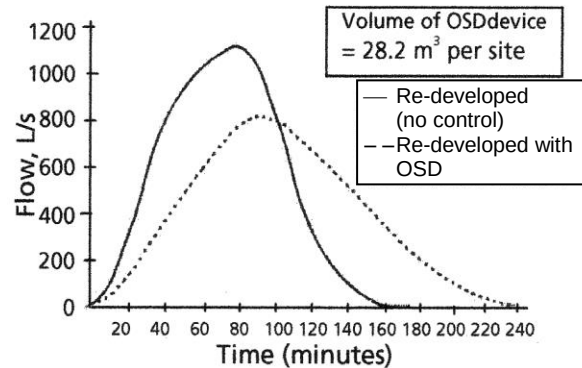


FIGURE 1.2.2(b) : 60 OSD sites
each lagged by 1 minute

USING ON-SITE RETENTION, OSR

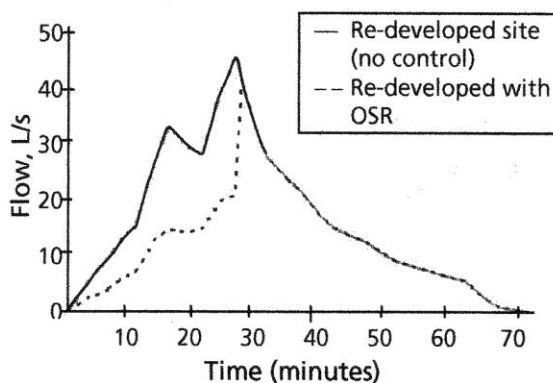


FIGURE 1.2.1(c) : Re-developed
with possible OSR solution

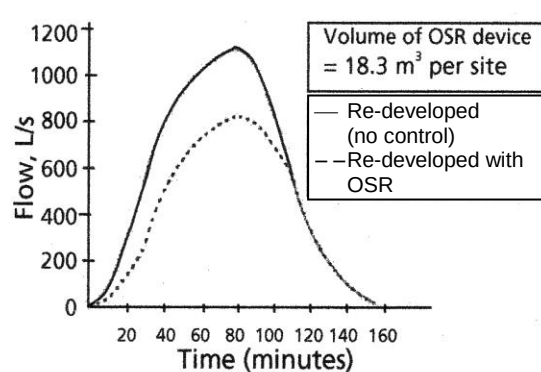


FIGURE 1.2.2(c) : 60 OSR sites
each lagged by 1 minute

Furthermore, and provided the guidelines on treatment of runoff passing to OSR devices are carefully followed, maintenance can also be significantly less.

Environmental benefits : Stormwater retention involves, firstly, the holding of storm runoff, including pollution, on site: this certainly reduces the flow of **site** contaminants downstream, but it does not reduce pollution loads originating on urban roads and highways passing to receiving waters. Secondly, such runoff management involves on-site collection and use of stormwater from roofs and other relatively clean surfaces. The consequences of both of these aspects, acting together, is **greater** concentration of pollution in the general (reduced) runoff flow passing towards urban streams, rivers and the sea than would otherwise occur. Pollution treatment devices and systems – GPTs, wetlands, etc. – intercepting this flow can therefore be made **smaller** than installations needed to treat the whole stormwater flow.

Harvesting of stormwater : Harvesting can be considered under the following categories :

- Roof runoff collected in rainwater tanks can be used in domestic hot water systems and for toilet flushing, etc. Roof runoff can be cleaner than some town water supplies, and when used in storage hot water systems, overcomes the problem of contamination by *E.coli*, *cryptosporidium* or *giardia*.
- Soil moisture maintenance in the vicinity of “leaky” devices such as wells, gravel-filled trenches, etc.; this moisture promotes growth of grass, shrubs and trees.
- Recharge of aquifers with cleansed water using ASR (aquifer storage and recovery) technology: this can be retrieved for irrigation, toilet flushing or industrial uses.
- Environmental flows: when cleansed stormwater is injected into an aquifer with a steep groundwater hydraulic gradient, retrieval of the recharged water is virtually impossible; the injected flow usually emerges as percolation into a local urban stream, thereby enhancing the stream environment and its biotic community.

1.2.3 Detention technology : its role in contemporary stormwater management

The case for introducing retention technology into Australian practice presented above should not be interpreted as signalling an end to the use of OSD and its complete replacement by OSR in urban catchments. There are at least four sets of circumstances where detention technology may be preferred to OSR in contemporary stormwater management practice :

- Situations where soil types or topography are unsuited to OSR; where allotment sizes are too small to permit required clearance distances being observed; where groundwater is saline and shallow; etc. (see Section 3.8).
- Catchment management scenarios where maximum storm runoff yield is sought from the urban landscape together with control of peak flood flows : this is an aspect of stormwater harvesting achieved through downstream wetlands associated with aquifer storage and recovery (ASR) schemes (see Section 4.2.1).
- Catchment management situations where the primary objective – perhaps even the sole objective – is flood control. There is no doubt that the simplest way to reduce flood peak flows in an isolated catchment – to achieve a target outflow – is through the use of a ‘bottom end’ detention basin. Certainly, the identical effect can be achieved through the use of distributed retention installations, but this option is often unattractive to municipal agencies who prefer the centralised solution (see Section 5.6).
- Detention techniques can also be employed to provide environmental flows in urban waterways through the medium, perhaps, of ‘slow-drainage’ (see Section 5.1.5).

1.3 RETENTION-BASED STORMWATER MANAGEMENT : WATER-SENSITIVE URBAN DESIGN (WSUD)

1.3.1 Primary aims

There are three primary goals of retention-based stormwater management :

1. to reduce storm runoff in terms of both peak flow and volume (flooding, channel-forming and environmental flows);
2. to minimise pollution conveyance from urban catchments to downstream waterways and receiving waters; and;
3. to harvest and use storm runoff to replace mains water use including some potable but mainly ‘second quality’ applications.

The first two goals are well understood and universal; the third has sprung from the pressing needs of urban communities looking for alternatives to ‘big system’ infrastructures delivering a single water product whose quality exceeds the requirements of many applications.

It follows that stormwater management practices, where they are incorporated as fully as possible into appropriate urban landscapes, may reproduce hydrological behaviours bearing close resemblance to those of the original forested catchments they replaced. Such simulation studies as have been carried out lend support to this assertion.

This is an exciting concept for planners and developers to entertain, as it carries with it the possibility that minor drainage lines with their associated fauna and flora habitats, normally sacrificed in the wake of the urban sprawl, may well find co-habitation alongside sensitively-designed urban developments. This concept is reviewed in greater detail in Section 1.4.

This is the vision of contemporary stormwater management or **water-sensitive urban design**, the terminology first used in Perth to describe this approach (Whelans et al, 1994; Wackernagel and Rees, 1996; Beecham, 2003; Mouritz et al, 2003) : how can it be realised?

1.3.2 Site-dependent options and opportunities

Opportunities for using rainwater tanks and/or (enlarged) eave gutter systems to store roof runoff exist at all sites, but in-ground storage of runoff, other than in impermeable rainwater tanks, raises a number of site capability issues which depend on the following properties :

- permeability of the soil, measured by its hydraulic conductivity, k_h ,
- nature of the soil in terms of its reaction to the temporary presence of water,
- geology of the site in terms of land slope, depth to bedrock, availability and types of aquifers and aquifer water quality,
- potentiometric gradient of water flowing through aquifers, where present.

Australian urban concentrations include a range of natural environments where interplay between these properties produces a number of zones whose capabilities vary from unlimited opportunities for storm runoff retention and retrieval for use, on the one hand, to total inability to apply this technology, on the other. There is a preponderance of the former over the latter. Some guidance on the types of devices and systems which may be used, and site capabilities, is presented in the following sub-sections.

1.3.3 Runoff categories and treatment

There are two basic types of water-retaining installations which may be used in on-site systems: simple in-ground devices such as “leaky” wells, gravel-filled trenches or pipe/gravel trenches, as illustrated in Figure 1.3 (also, strong plastic ‘milk crate’ and other types of cells) : these devices drain by natural percolation into the surrounding soil. Alternatively, the same simple device can drain directly (unconfined aquifer) or by means of a bore connection to an underlying confined aquifer, if present (see Figure 1.4a). Wherever on-site stormwater retention is practised, certain general guidelines should be observed in the way it is controlled or managed prior to entering a device. The first of these relates to roof runoff, the second to ground-level surface runoff.

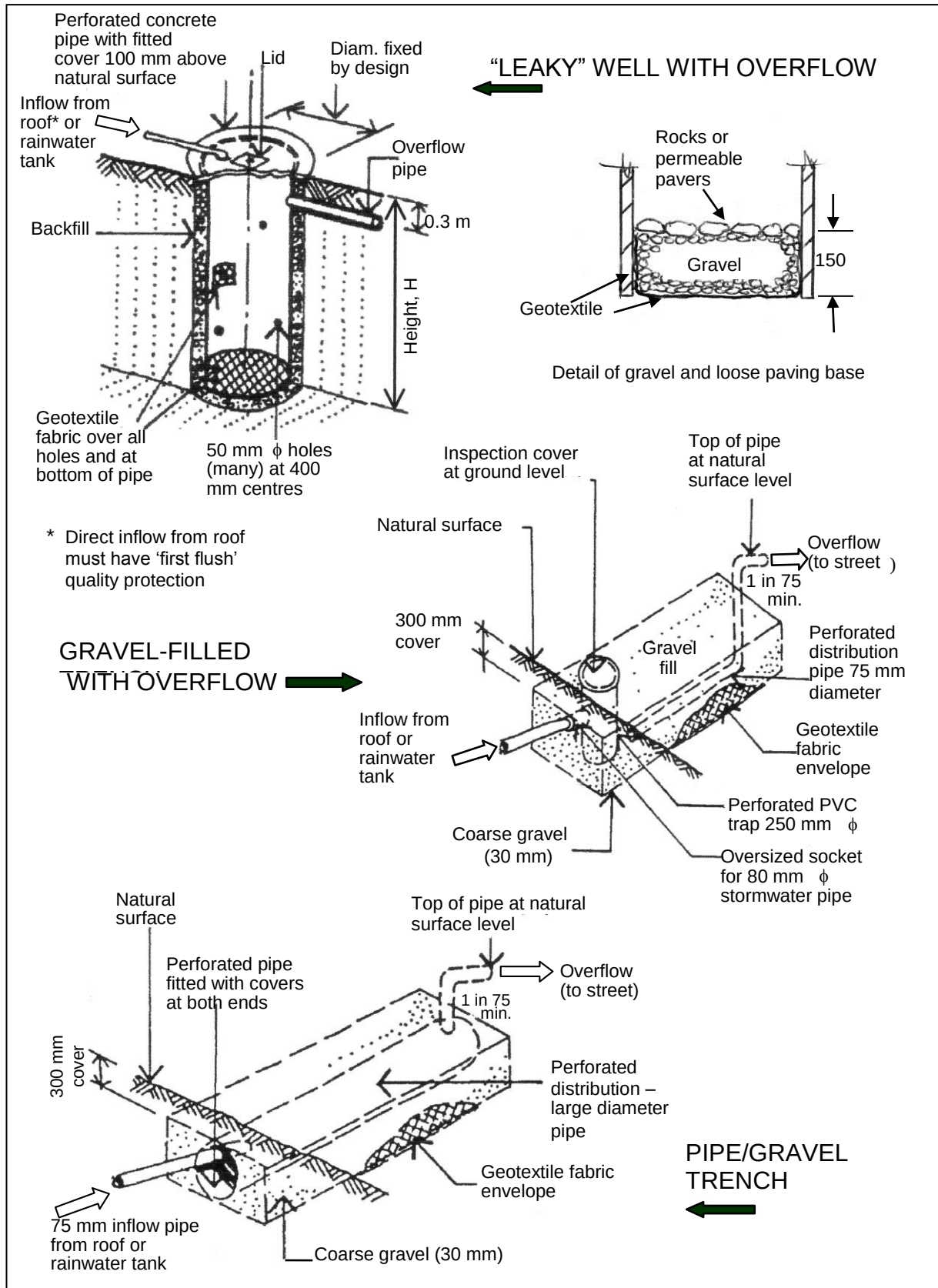
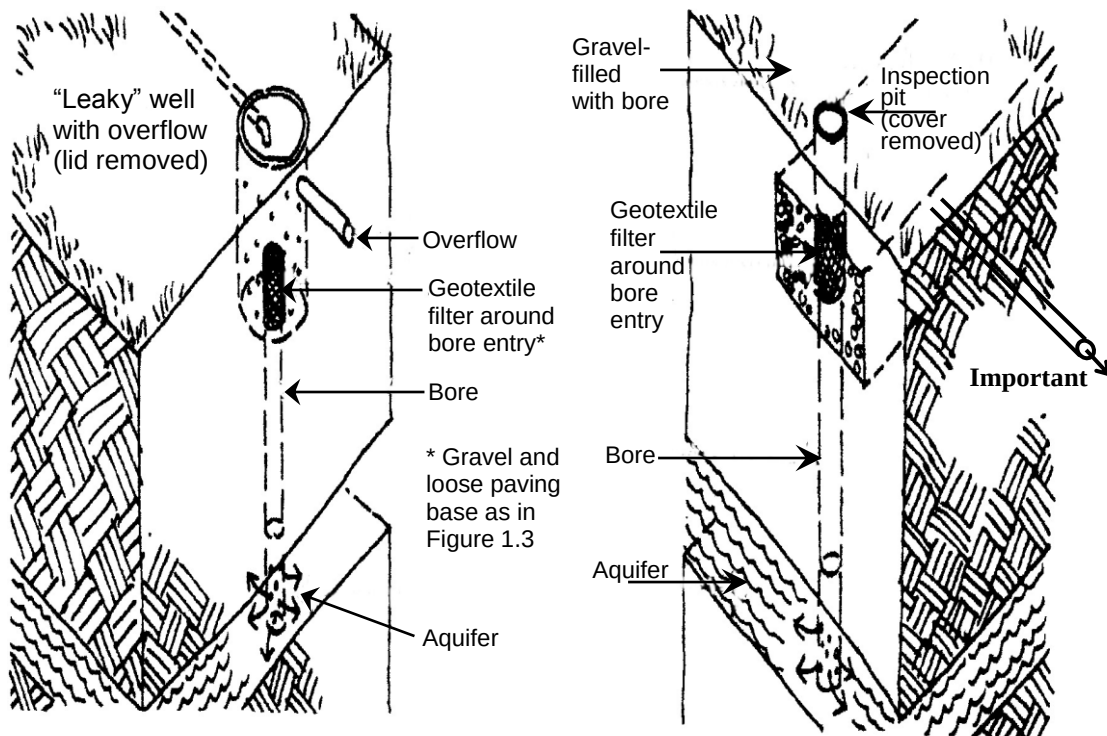
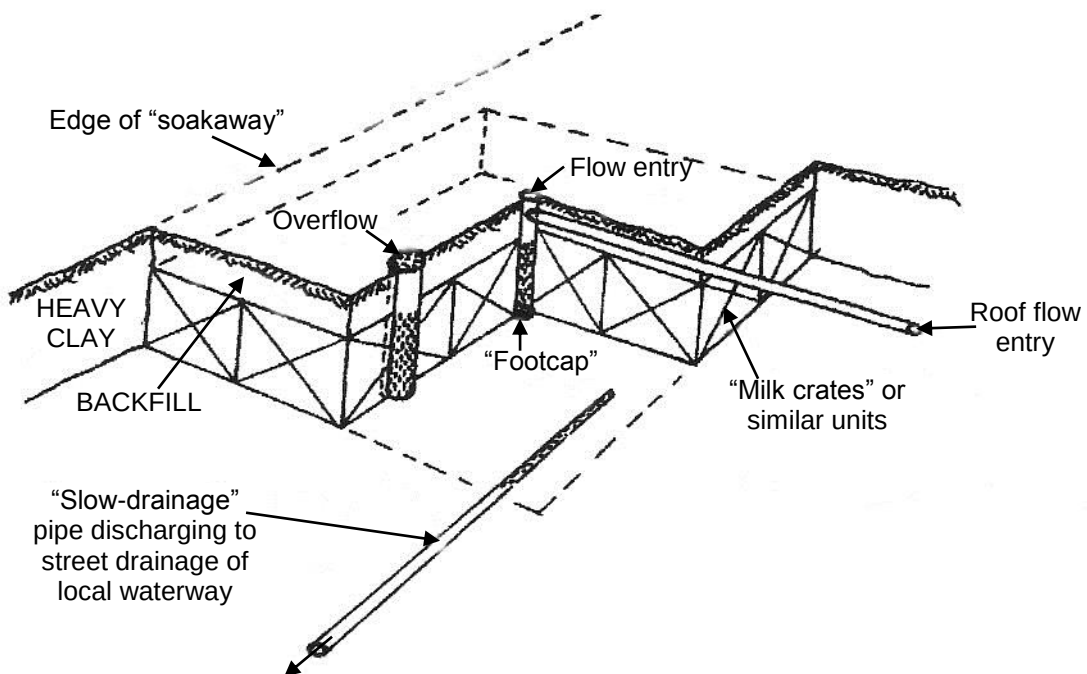


FIGURE 1.3 : ‘Simple’ devices for retaining stormwater



(a) Aquifer recharge from “leaky” devices : this technology has two main uses in water-sensitive urban design. The first is in stormwater harvesting, the second is to reduce device “emptying time” (see Procedure 4A, Chapter 5)



b) This illustration depicts the ‘limit’ of OSR technology in heavy clay soils where acceptable emptying time can only be achieved through the use of a slow-drainage pipe (see Procedure 4B and Section 5.4, Chapter 5)

FIGURE 1.4 : “Leaky” devices with aquifer access or ‘slow-drainage’ pipe to improve emptying performance

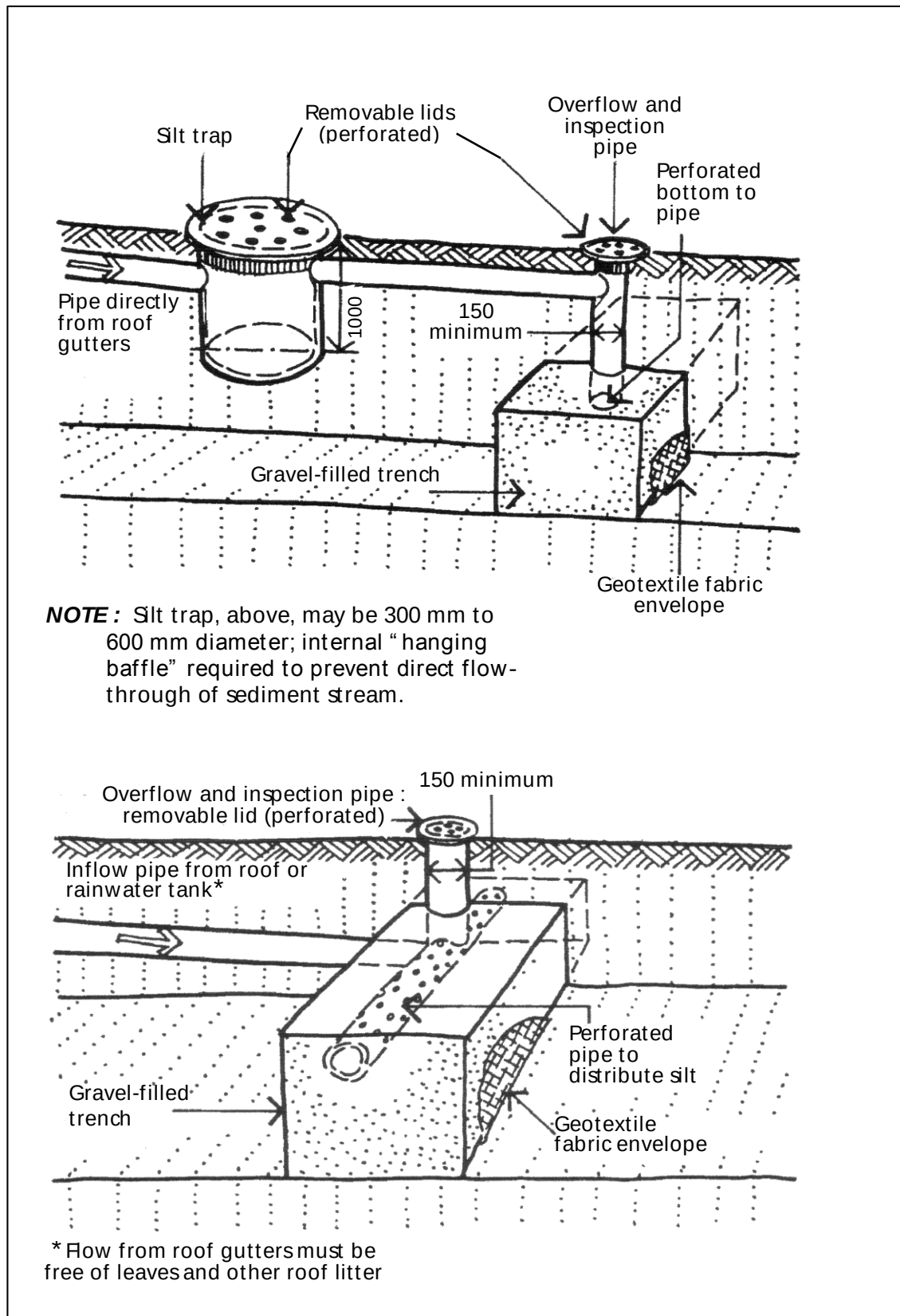


FIGURE 1.5 : Details of sediment management systems for roof (only) flows to gravel-filled trenches

Storm runoff from roofs, cleared of leaves and any other roof litter, may be passed into a rainwater tank(s) and the overflow piped directly to an in-ground device(s) of the type described above (Figures 1.3 or 1.4). Alternatively, roof runoff, cleared of roof litter and other gross pollutants may be passed **directly** to a “leaky” device, provided the entry details sketched in Figure 1.5 are observed. Some municipal and government housing agencies object to combined inspection/overflow pipes of the types illustrated in Figures 1.4 and 1.5 on two grounds :

- these elements may be responsible for perpetual ‘damp spots’ adjacent the overflows, and,
- these access points are vulnerable to environmental vandalism through the disposal of toxic (liquid) waste, for example sump oil.

The concern which forms the basis of the first objection can be largely dismissed but is reviewed in relation to four “soakaway” installations in Western Sydney in Section 5.5. However, the latter problem is particularly serious where direct access to aquifers is possible (as in Figure 1.4a). Where these valid concerns are felt, then appropriate action needs to be taken, such as locating devices in secure areas or incorporating the more formal overflows illustrated in Figure 1.3. This leads to the vital issue of sediment generation in urban catchments.

The most intensive investigation of the performance of infiltration facilities ever reported in the literature is the landmark study conducted in Maryland, U.S.A., in the late 1980s. The investigation followed legislative action by the State of Maryland requiring stormwater to be retained (‘source control’) on all developments and re-developments. The devices used to meet the requirements ranged from “leaky” wells to porous paving. Early indications of some poorly performing installations were followed by two surveys of randomly selected sites taken in the first six years of operation of the regulations. Some 200 devices/installations were sampled: the results showed **failure** rates ranging from 40% for “leaky” wells to 85% for porous paving. Sedimentation of facilities was reported as the most frequent cause (Pensyl and Clement, 1987; Lindsay et al, 1991).

Recognition of sedimentation and its potential for damage to WSUD installations is, undoubtedly, a first step in avoiding the mistakes revealed by the Maryland study. Australian practitioners have N.S.W. Department of Housing (1998) to thank for its excellent Handbook “Managing urban stormwater : soils and construction (3rd Edition)”, which includes procedures for calculating expected sediment loads for a wide range of field conditions (climate, soils, terrain, etc.) and practices for controlling sediment. The focus of the document is clearly on the construction phase of development/re-development.

The following data, based on North American literature (Wolman and Schick, 1962; Marsalek, 1992), are offered, not as a substitute for the N.S.W. Housing information, but simply to briefly convey the scale of the problem.

- **Atmospheric sediment** : The amount of sediment which can be expected to precipitate from the atmosphere onto the land surface in an urban catchment is at least 20 tonnes per km² per annum. This translates into 2 kg per annum per 100 m² of roof area;
- **Sediment supply from a fully-established suburb** : This ranges from 10 m³ per km² per annum to 50 m³ per km² per annum. The lower end of the range applies to high quality neighbourhoods where strategic action is taken to control sediment; the upper limit applies to neighbourhoods where no sediment control strategy is in place.
- **Sediment supply from suburb during construction phase** : This ranges from 7,000 m³ per km² per annum to 20,000 m³ per km² per annum. The observed sedimentation rate for small sites is even higher, and corresponds to 25 – 45 m³ per ‘quarter-acre’ block (1,000 m²) per annum. It follows that some 5 – 10 m³ of sediment can be expected to enter neighbourhood drainage paths from each developed/re-developed ‘quarter-acre’ building site during construction.

This information shows quite close agreement with results calculated using the N.S.W. Housing RUSLE model (construction sites) : the model does not address the problem of sediment supply from fully-established suburbs. It may be concluded that a neighbourhood facility designed for sediment storage over a 50-year, fully-established ‘lifespan’, is likely to fill well before the construction phase of the sub-division is completed (Argue, 2000).

These considerations lead to the following advice relating to the storage of surface runoff from any (surface) paved areas :

Storm runoff from paved areas including courtyards, walkways, driveways, carriageways, car parks, etc. **should under no circumstances be passed directly to devices of the types illustrated in Figures 1.3 or 1.4.** Such runoff, where it originates on relatively small areas, must be prepared for entry by passing it slowly (‘creeping flow’) across at least six metres of grass (Bren et al, 1997) or through a gravel/sand or sand/loam filter at least 100 mm thick and covered with grass or suitable ground cover (see St. Elizabeth Church Car Park, Section 3.9). Surface runoff from urban components such as car parks or suburban-scale areas should be passed through, perhaps, a succession of devices/systems called a **treatment train** before it is suitable for entry into aquifers, urban waterways (freshwater or marine) or rivers (see Sections 2.9.3 and Parfitt Square, Section 3.9).

The problem of ‘construction phase’ versus ‘fully-established phase’ in terms of sediment supply and control should be recognised and suitable provision made. This may lead to a two-phase approach, the first involving sediment being stored and, possibly, regularly removed during the construction process. The second phase is that of long-term management of sediment over the development’s well-established lifespan.

Reference should be made here to the variety of techniques described in Chapter 2 of this document or other guideline publications, e.g. Whelans et al, 1994; CSIRO, 1999; and the N.S.W. “blue book” (N.S.W. Dept. of Housing, 1998 and later editions), IEAust (2003).

1.3.4 Soils : permeability and water-reactivity

The soil types and surface geological conditions present in most landscapes range from aeolian soils (dune sands) in coastal zones, alluvial soils and sandy clays in stream outwash areas to, generally, medium clays and heavy clays. (There are also many areas where soils of various types provide shallow cover over rock.) The hydraulic conductivities of these soils range from around 5×10^{-4} m/s (dune sands) to 1×10^{-8} m/s (heavy clays). There are also “constructed soils” whose permeabilities may range from that of sand to values smaller than 1×10^{-8} m/s. The influence which soil permeability has on the type of on-site stormwater retention practice employed is profound :

Deep, confined or unconfined sands (homogeneous) : These sands are capable of accepting, via suitably designed devices, all roof runoff generated in small-moderate storms without overflow. Hydraulic conductivities range upwards from a minimum of 5×10^{-5} m/s. Installations constructed in these soils should recognise seepage during storms in their design, which may take the forms illustrated in Figures 1.3 and 1.5. The phenomenon of soil “heave” is not observed in unconfined sandy soils so water-retaining devices can be placed as close as 1.0 m from footings and property boundaries; a clearance of 2.0 m should be incorporated where sand is associated with a mantle of sandy clay. Infiltration or ‘dry’ ponds are well-suited to sandy soils.

Sandy clays (homogeneous) : The hydraulic conductivity of these soils ranges from 1×10^{-5} to 5×10^{-5} m/s; seepage “loss” from installations during storms is small but should, nevertheless, be taken into account in design. Devices used for on-site retention of stormwater in sandy clay soils can be of the types illustrated in Figures 1.3 and 1.5, where the soil mass is deep and homogeneous. In situations where the sandy clay covers a useable aquifer, installations of the types illustrated in Figures 1.4a may be used. Some “heave” may be expected in these soils: in-ground devices should show clearances to footings and property boundaries of not less than 2.0 m. Infiltration or ‘dry’ ponds are well-suited to sandy soils.

Medium clay soils (homogeneous) : The range of hydraulic conductivity values exhibited by these soils is 1×10^{-6} to 1×10^{-5} m/s. Seepage loss from installations during storms is therefore likely to be very small so that in-ground devices need to provide storage for almost the full design runoff volume, without overflow: trench-shaped installations are appropriate for these soils. Significant soil “heave” is characteristic of these clays: devices retaining water in these soils should therefore be placed at least 4.0 m from building footings and property boundaries. Installations of the types illustrated in Figures 1.3 and 1.5 may be used where the soil mass is deep and homogeneous. In situations where the clay covers a useable aquifer, installations of the types illustrated in Figure 1.4a may be used. Infiltration or ‘dry’ ponds constructed in these soils are not effective unless provided with a “soakaway” sub-structure.

Heavy clay soils (homogeneous) : These soils show hydraulic conductivity values in the range 1×10^{-8} to 1×10^{-6} m/s. Seepage “loss” from installations during storms is insignificant. Although success can be achieved using simple systems of the types illustrated in Figure 1.3, the most suitable shape is that of the “soakaway”, i.e. a shallow mattress with large plan area (see Figure 1.4b). Trenches constructed in these soils tend to have long emptying times (20 days or more); acceptable emptying times – one, two or three days – following moderate-large storms can often be achieved with some form of slow drainage. This may take the form of that provided by aquifer access or, in hilly terrain, a small diameter connection to a local stormwater main or to an urban waterway (see Figure 1.4b and Procedures 4A and 4B, Chapter 5). Soil “heave” similar in magnitude or greater than that observed in medium clay soils can be expected in these soils : clearance distances to footings and boundaries should therefore be not less than 5.0 m. Infiltration or ‘dry’ ponds constructed in these soils are not effective unless provided with a “soakaway” sub-structure possibly, also, with ‘slow-drainage’ assistance (see Procedure 4, Chapter 5).

Non-homogeneity : The properties, recommendations and guidelines set out above for the various naturally-occurring soil masses – sands to heavy clays – are all qualified as “...deep and homogeneous...”. The inconsistency of soils is well recognised in the literature of soil mechanics, in particular, differences in the permeability of a soil in the vertical and horizontal directions. Practitioners should be aware of such variability and seek advice from a geotechnical specialist as to how such soil conditions at the site of a project might affect detailed design.

Another aspect of non-homogeneity of soil at a site arises from the presence of sand/loam backfilling of service trenches for sewage and stormwater pipes, landlines, cabling, etc. Potential damage to footings as a consequence of these components creating unexpected pathways for retained water should be carefully considered and appropriate preventative action taken.

Constructed clay soils : There is a further class of soils encountered in stormwater management practice – those laid down according to specifications meeting the requirements of engineering design. These range from deep-filling of previously excavated sites to mantles constructed over landfill. Typical requirements call for heavy clay in layers 150 mm thick at optimum moisture content and rolled. The hydraulic conductivity of the resulting soil matrix is likely to fall within the range 1×10^{-10} to 1×10^{-8} m/s. These soils are virtually impervious; “soakaway” installations are preferred, or on-site retention devices of the types illustrated in Figures 1.3 and 1.5, but acceptable emptying – one, two or three days – following moderate-large storms requires some form of low-level drainage. This may take the form of that provided by aquifer access or, in hilly terrain, a small diameter connection to a local stormwater main or to an urban waterway (see Figure 1.4 and Procedures 4A and 4B, Chapter 5). Except where the engineering properties of such soil masses are known to be non-reactive, constructed soils may be expected to exhibit “heave”: clearance distances to footings and boundaries should therefore be not less than 5.0 m.

Sites with rock or shallow soil cover over rock : It has long been held that the presence of rock at a site precludes the use of techniques involving the retention and subsequent disposal, on site, of storm runoff. While this still holds true for site conditions where the rock is completely or nearly impervious, such as unweathered basalt, granite or shale, recent research has discovered rock cases – in particular, sandstone – which show similar permeabilities to “medium clay”, i.e. hydraulic conductivity values in the range 1×10^{-6} m/s to 1×10^{-5} m/s, and therefore suitable for OSR technology. The first reported results indicating this outcome were obtained in Adelaide, South Australia (van der Werf et al, 1999); these have been confirmed in tests conducted in Parramatta and Hornsby, NSW, (Argue, 2001). The presence of rock renders the possibility of footings disturbance most unlikely, however, the potential for “heave” to occur in the clay mantle, where this is present, should be recognised and a clearance distance of 2 m would seem to be a wise precaution in these circumstances.

Water-reactivity and ‘clearance’ : The clearance distances recommended above are a direct consequence of the potential for damage to footings, particularly domestic foundations, posed by soil “heave” (or swelling) near water-retaining devices draining “naturally”, i.e. without hydraulic assistance of the types illustrated in Figure 1.4. The recommendations are based on observations made at field installations in Adelaide, South Australia. These show maximum “heave” of 30 mm, approximately 2 m from the edges of devices constructed in the most water-reactive soils : swelling decreases to near zero at the extremity of the stated clearance distance.

A feasible alternative to observing clearance requirements is to ensure that OSR devices located in soils with high “heave” potential are designed to empty rapidly. It is inconceivable for the soil surrounding a gravel-filled trench, for example, to develop its full swelling potential if the device is “dry” one, two or even three days after filling. Figure 1.4 illustrates ways in which accelerated emptying can be achieved to take advantage of this phenomenon : Procedures 4A and 4B (Chapter 5) provide the design approach needed to meet this objective. With ‘accelerated’ emptying in place, it is suggested that devices could be located at, say, **half** the recommended distances from footings and property boundaries given above. In cases where even the modest seepage outflow into the parent soil that this suggestion entails is unacceptable, then the flood management benefits of OSR practice can still be realised through use of *extended detention*, as explained in Section 5.1.5. A storage device with impermeable boundaries and a **slow drainage** outlet would be needed.

Soils : a final word : The OSR devices of interest introduced in Section 1.3.3, namely “leaky” wells, trenches and “soakaways” of various types can be employed in all soil categories described in Section 1.3.4. However, the manner in which they perform their retention function – either using simple systems (illustrated in Figures 1.3 and 1.5) or through accelerated emptying between successive storms using systems of the types illustrated in Figure 1.4 – represent major concerns for the designer. It suffices at this stage of the Introduction merely to draw to the reader’s attention the range of soils for which simple solutions are possible, and those requiring more complex design approaches.

The point of changeover from the former to the latter is set at :

$$\text{hydraulic conductivity, } k_h = 1.0 \times 10^{-6} \text{ m/s.}$$

It will be noted (see above) that this value separates medium clay soils from those described as heavy. Further discussion of the issues raised here, and solutions, are set out in Chapter 5.

1.3.5 Aquifer water quality, potentiometric gradients and use

The qualities of water in Quaternary, Tertiary and fractured rock aquifers vary greatly with useable waters ranging from near tap water standard (less than 1,000 mg/L) to 5,000 mg/L (in Adelaide). [There are, also, groundwaters with salinities ranging up to greater than that of seawater (35,000 mg/L)]. It is recommended that no direct, potable use of these waters be considered in metropolitan areas.

Second quality, direct use of much of these resources is possible in industrial processes, such as cooling, washing-down operations and in toilet flushing, but the ambient salinity of most of it provides limited opportunity for use in long-term irrigation for house gardens. This opportunity can be significantly expanded through recharge using cleansed stormwater : localised mixing to sweeten saline waters, with subsequent retrieval, has the potential to lift the scope for employing this technology to much of these resources.

There are, however, many regions with limited opportunity for application of aquifer storage/recovery (ASR) technology : in these regions, natural waters are either far too saline to achieve irrigation-standard quality through mixing, or their potentiometric gradients are too steep. ASR technology may still be employed in the former of these cases, however, where a salt-tolerant use of the retrieved water is proposed, e.g. irrigation of special (salt-tolerant) plants and grasses, industrial applications, toilet flushing, etc.

The relationship between cleansed stormwater recharge, mixing and use is further complicated by localised gradients of the potentiometric surface. In locations where gradients are flat and ambient quality is better than 2,000 mg/L, storm runoff injected in the ‘wet’ season will be recoverable in the ‘dry’ season, providing potential for use in irrigation. [In Northern Australia (see Figure 3.8), the wet season is summer; the corresponding season in Southern Australia is winter. Dry seasons are winter in Northern Australia and summer in Southern Australia.]. There are other locations which have ambient water quality around 2000 mg/L, but steep potentiometric gradients : it is not possible to retrieve cleansed storm runoff recharged at these sites. This is because water injected in wet periods will have travelled too far downstream before the arrival of the following dry periods.

The above considerations should not be interpreted as meaning that aquifer recharge using stormwater is feasible only at those sites where recovery is advantageous, e.g. in irrigation. One of the main benefits of recharge of Quaternary, Tertiary and fractured rock aquifers using cleansed stormwater is provision of environmental flows in urban creeks and streams. This point is revisited in the Aquifer Recharge/Retrieval Protocol section, below.

1.3.6 Bores : extraction rates and recharge rates

Bore extraction rates vary from less than 0.5 L/s to 4.0 L/s (Quaternary aquifers) or greater per bore (Adelaide experience). These values represent long-duration extraction rates and are conservative when considered for short-term uses such as irrigation and other domestic and industrial/commercial purposes. It is recommended that site tests using an extraction bore and a second bore as observation well, be conducted as part of the exploratory process needed to establish an ASR system.

There is a rule-of-thumb link between extraction and recharge rates advised by Pavelic et al (1992) : the recharge rate is 0.5 times the extraction rate. This, again, is based on long-term performance and may therefore be considered conservative for the short-term periods of recharge likely to occur on most rainy days. Site testing of recharge capability is recommended in the early stages of project feasibility investigation.

1.3.7 Aquifer recharge/retrieval protocol

Broadly, two groundwater regimes in which recharge using cleansed stormwater may take place should be recognised. The first is where potentiometric gradients are ‘flat’ and groundwater movement is very slow (around 20 m per year). The second is where potentiometric gradients are steep and groundwater movement is relatively rapid, e.g. 300 m per annum.

In the regime of flat potentiometric gradients, it is essential that recharge and retrieval of groundwater be balanced on an annual basis : this ensures continued equilibrium of local potentiometric levels and also, sustainability of the resource. South Australian practice in this regard is to limit extractions in ASR schemes to “...not exceeding 80% of the volume artificially recharged...” (Northern Adelaide and Barossa CWMB, 2000). Even with such a management strategy in place, “mounding” of recharge water in unconfined aquifers, particularly where sandy soils are involved, can cause problems with foundations, flooding of cellars, salt damp, and ingress of groundwater into sewers. Severe mounding of highly saline groundwater can lead to surface salinisation – a consequence which must be avoided at all costs (see Section 3.8).

There are many ways in which the balance referred to above can be achieved. In cases of recharge using roof runoff at a large factory site, for example, or stormwater cleansed and collected from a large car park, the quantity involved is likely to justify, on economic grounds, installation of retrieval systems replacing mains water use in irrigation or certain factory processes such as cooling, washing-down, etc.

At the level of the domestic dwelling, such a recharge/retrieval system, while technically possible, is rarely economical. Encouragement of on-site retention of storm runoff in predominantly residential sectors of cities, however, can benefit local Councils who can maintain the required equilibrium of local potentiometric levels by extracting water for irrigation, thereby saving on mains water use.

In locations where potentiometric gradients are steep, usually in foothills or hilly terrain, on-site aquifer storage/recovery **of injected water** cannot be practised because of rapid movement of recharge downstream from injection sites. This does not preclude the use of aquifer-connected bore systems of the types illustrated in Figure 1.4a, but water injected into such systems is most likely to outflow ultimately into local creeks and urban streams. These are certainly not wasted flows : on the contrary, they represent most valuable contributions to achieving one important element of the vision for stormwater management referred to earlier (Section 1.3.1), namely, environmental flows and the return of urban drainage lines and associated fauna and flora to their natural state (see also Section 1.4). Great care needs to be exercised, however, in pursuing the strategy outlined here to ensure that injected flows do not create hazard or nuisance for developments between injection sites and the urban creeks (see Section 3.8).

A form of ASR (aquifer storage/recovery) is possible in locations where potentiometric gradients are steep, but, unlike conventional ASR schemes, the water withdrawn is different from the injected flow. In these cases, ambient aquifer water is extracted – matched in terms of quantity and quality to an appropriate use – and an equivalent (or 20% greater) quantity of cleansed water is injected, on an annual basis, meeting the requirements of the Protocol.

1.3.8 OSR and site capability

The capability of a site to retain stormwater and the constraints which must be observed in applying appropriate retention practices, depend on the site properties reviewed in the previous Sections. Their application to actual situations is illustrated, for the case of Adelaide's Patawalonga Catchment, in Figure 1.6 and for **two** of the zones in Table 1.1.

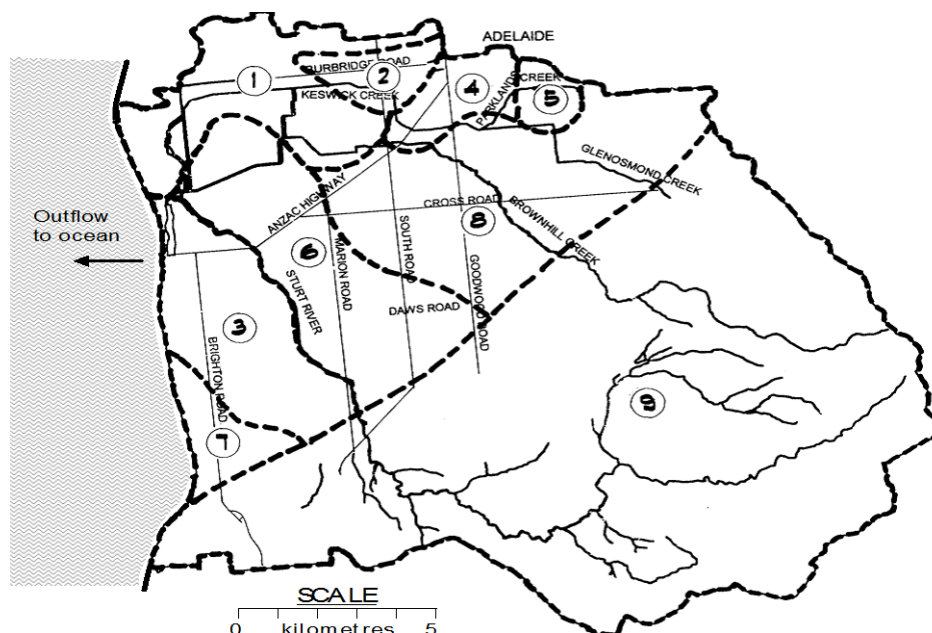


FIGURE 1.6 : Nine stormwater management zones of the Patawalonga Catchment (B C Tonkin, 1996)

TABLE 1.1
PATAWALONGA CATCHMENT ZONE GUIDELINES ON STORMWATER MANAGEMENT

PROPERTY	ZONE 1	-	ZONE 3	-
Soil Type	Alluvial soils, aeolian (dune) sands, some clay	-	Red-brown earths and podzolic soils north of Diagonal Road	-
Soil permeability	Hydraulic Conductivity: 5×10^{-5} m/s	-	Hydraulic Conductivity: 1×10^{-6} m/s, and 1×10^{-5} m/s	-
Clearances	Not less than 1.0 m to footings and boundaries	-	Not less than 4.0 m to footings and boundaries	-
Bores :		-		-
• Aquifer depths	Mainly 2 – 4 m, some deeper inland	-	Variable: 4.0 m to 10.0 m	-
• Water quality	Variable: salinity 500 – 3,000 mg/L	-	Variable: salinity 1,000 – 3,000 mg/L	-
• Extraction rate	Variable: 0.5 L/s to 4.0 L/s per bore	-	Variable: 0.5 L/s to 4.0 L/s per bore	-
• Recharge rate	Half extraction rate	-	Half extraction rate	-
Stormwater option	Simple devices or devices with bores (Figures 1.3, 1.4 and 1.5) may be used in all zones. See comment below			
Advised use	Variable: irrigation, industrial, toilet flushing, etc., depending on location	-	Variable: irrigation, industrial, toilet flushing, etc., depending on location	-
Recharge/retrieval protocol	Recharge/retrieval to be balanced on single allotment basis or within local collective system.	-	Recharge/retrieval to be balanced on single allotment basis or within local collective system.	-
Comments	The coastal sector of the zone is unconfined sand; inland sands may be covered by sandy-clay mantle (confined aquifers). Highly saline, deep clay patches should be avoided.	-	Potentiometric gradients, generally, suitable for above practices. Exception is foothills sector at southern extremity of zone where gradients are steep. Direct ASR is not feasible, but annual injection/extraction (80% requirement)* is acceptable option. Devices with bores support environmental flows in local streams.	-

* See Northern Adelaide and Barossa CWMB (2000).

1.4 WSUD – STORMWATER ‘SOURCE CONTROL’ : THE VISION AND ITS LIMITATIONS

1.4.1 Conventional urbanisation : typical consequences in a natural, coastal catchment

In its wild state, the natural (coastal) catchment is a system of drainage paths, creeks and streams well matched to the rainfall/runoff processes operating in that basin. Generally, less than 20% of storm rainfall is discharged from it as (annual) *surface* runoff. There is a second stream of flow from the natural catchment, that delivered by aquifers into creeks and waterways as ‘base flow’, long after the causative rainfall event. The *quantity* of discharge involved in this outflow is comparable to that delivered as surface runoff, but its exit into a local receiving domain or to the ocean takes place over a time scale measured, perhaps, in weeks, months, years or even decades (see Figure 1.7).

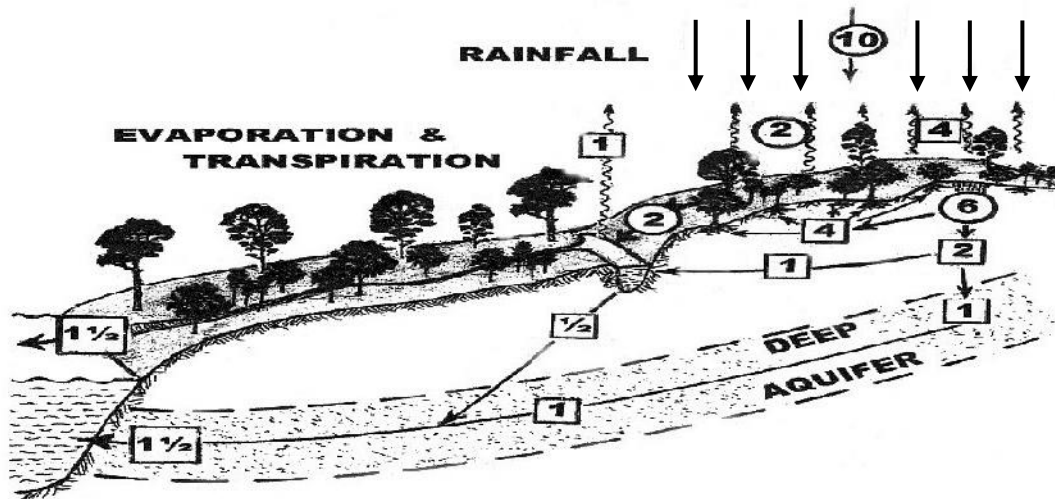


Figure 1.7 : Destiny of 10 units of *annual* rainfall input to a temperate zone, coastal forest catchment (notional values only)

The elements illustrated in Figure 1.7 present a ‘notional’ account of the fate of 10 units of **average annual rainfall input (AARI)**, shown circled) to a coastal forest catchment. The ‘first contact’ process is that of **interception** (including stemflow), which accounts for some two units (shown circled) of AARI: this stored water is temporarily retained before passing from the catchment as evaporation. Two units (shown circled) become surface runoff passing over the forest floor to enter ponds, small channels and streams. The remaining six units (shown circled) are infiltrated into the forest floor where two-thirds of it – four units (shown ‘boxed’) – is intercepted by tree roots and transpires; the two units (shown ‘boxed’) **not** taken up by tree roots is divided evenly between seepage to streams and lakes (one unit, ‘boxed’) and percolation to deep aquifers (one unit, ‘boxed’). **Half** of the surface runoff originating on the forest floor, i.e. one unit, ‘boxed’, evaporates in streams, ponds and lakes. A half unit (‘boxed’) of seepage passes from ponds and forest streams into the deep aquifer where it joins the aquifer stream – total 1½ units (‘boxed’) discharged to a local receiving domain or the ocean. Surface runoff flow to the ocean also equals 1½ units (‘boxed’) [see Walsh et al, 2004].

There are **two** sets of conditions that determine the ‘nature’ of a natural catchment waterway in terms of *cross section shape* and *ecosystem regime*, and **one** (set) which is responsible for delivering *effluent water quality*. These are, respectively :

natural stream morphology: the drainage paths of any forest basin have evolved over eons to establish waterway **shapes** which reflect, individually, their catchment area/shape and terrain (bed slope, soil, vegetation, etc) and their response to regional climate. The flow which exerts greatest influence on establishing waterway dimensions is that of ‘**bankfull**’ flow or some variant of it (Gippel, 2002; Doyle et al, 2007);

catchment average annual yield: this parameter, considered for any particular stream, is influenced by the same properties as determine stream *morphology* – area/shape of catchment, terrain and climate – and also the catchment’s response to frequent storm inputs – the ‘**low flow**’ regime. The characteristics of this regime determine the stream’s ecological state (Smakhtin, 2001; Lee et al, 2008);

efflux water quality : the primeval forest catchment provides home territory to a wide range of indigenous fauna and flora – including human – as well as temporary accommodation for migratory species. All of these elements lose ‘body products’ of one type or another – faeces, body fluids, feathers, skin, leaves, bark, branches etc. – and, with death, experience decay. [Duncan in Engineers Australia (2006) suggests pollutant concentrations for TSS, Total Phosphorus, Total Nitrogen and BOD of 80mg/L, 0.08 mg/L, 0.8 mg/L and 3 mg/L, respectively.] The catchment, including soil body, manages this pollution load using a wide range of natural ‘treatment train’ processes. Furthermore, the quality of efflux that results [Duncan suggests 15mg/L, 0.02 mg/L, 0.4 mg/L and 2 mg/L, respectively for the same pollutants] represents a level that contemporary best practice is ready to accept as the **benchmark for receiving waters** (Millis, 2003).

The scenario described here is represented in Figure 1.8a with a main stream providing capacity through its flood plain for the conveyance of all (surface) flood flows and base flows, as well as support for stream-associated indigenous fauna and flora in a plethora of biotic communities and riverine environments. The (natural) catchment exhibits a number of processes, apart from surface runoff and aquifer outflow recognised by the hydrologist – interception, detention, evaporation, transpiration, infiltration, percolation, soil moisture illustrated in Figure 1.7 – all of which can be sub-summed under the broad heading of storage/loss within the catchment or **retention** (of incident rainfall).

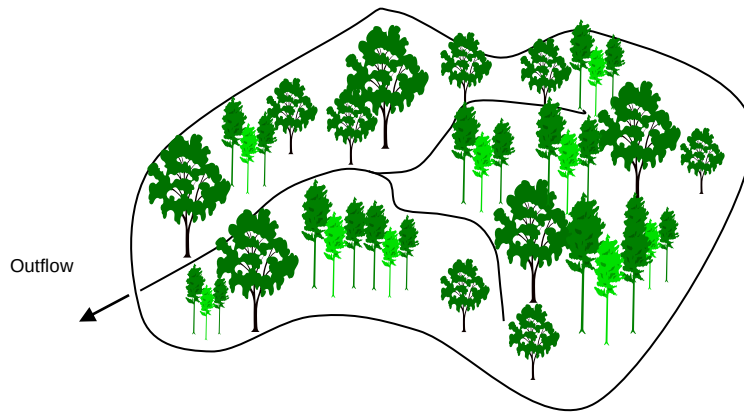
A further, important characteristic of natural catchments and one intimately linked with retention, is that of response or ‘**lag**’ time which is the delay evident between the period of rainfall input and the appearance of a portion of it as a surface runoff *flood wave* at the point of catchment discharge. This is a direct consequence of the surface-retardation/loss processes listed above – interception and detention in particular – which results in delayed passage of water flow through the catchment media of trees, understorey, forest floor humus, natural obstacles, surface depressions, etc. A high proportion of incident rainfall is retained in the catchment initially as soil moisture: the ultimate destiny of this component (of incident rainfall) can be take-up by forest vegetation roots leading to transpiration, or alternatively, deep percolation into aquifer strata and, hence, base flow supply to local streams (see Figure 1.7).

The first impact of development upon such a balanced system is usually the clearing of land for agriculture. This leads to change in the type of vegetation cover, e.g. from forest to grassland or crop, leading to slightly increased flood peak flows with attendant increased scour and erosion. Pollution loads – fertilisers, pesticides, etc. – originating in the catchment and carried in streams may be high depending on the type of agricultural activity involved (Lloyd et al, 2004).

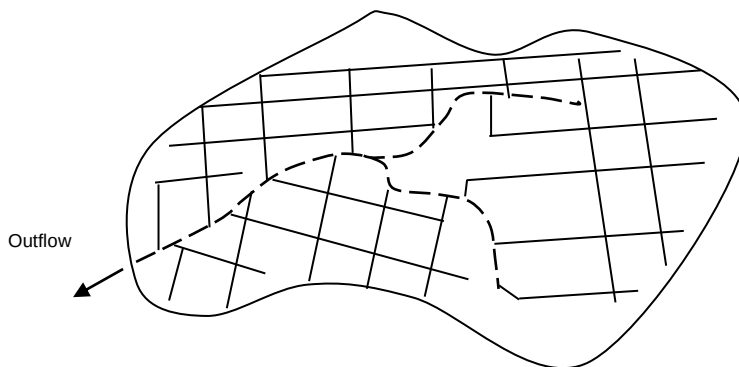
In the next phase of development – the conversion of agricultural land to low density urban landscape – a significant portion of the land is covered by impervious surface integral with the street drainage network and formal drainage channels. Some natural streams or portions of original waterways may be preserved in this stage of development. Greater surface runoff than observed in the pre-development catchment is characteristic of such catchments: this is universally recognised. What is often less appreciated is the effect that the presence of connected paved area has on catchment ‘lag’ time, referred to above. These two impacts – paved surfacing and *reduced* ‘lag’ time – account for, between them, the significant increases noted in flood runoff peaks compared to pre-development levels of (surface) outflow.

In what can be recognised as a further stage of urbanisation – ‘over-development’ - typically between 40% and 70% or more of the entire catchment is covered by roofs/paving or other relatively impervious surfaces, and runoff is conveyed rapidly by stormwater pipes and channels from points of origin in the catchment to its main stream(s). The natural creek branches are modified – straightened – and their original alignments used as the basis for a string of concrete pipes and channels, often contained within narrow municipal drainage easements (in some Australian jurisdictions, even this appreciation of the flood plain is absent) and development allowed to encroach, frequently, right to the boundaries of these easements. This process has two well-recognised consequences :

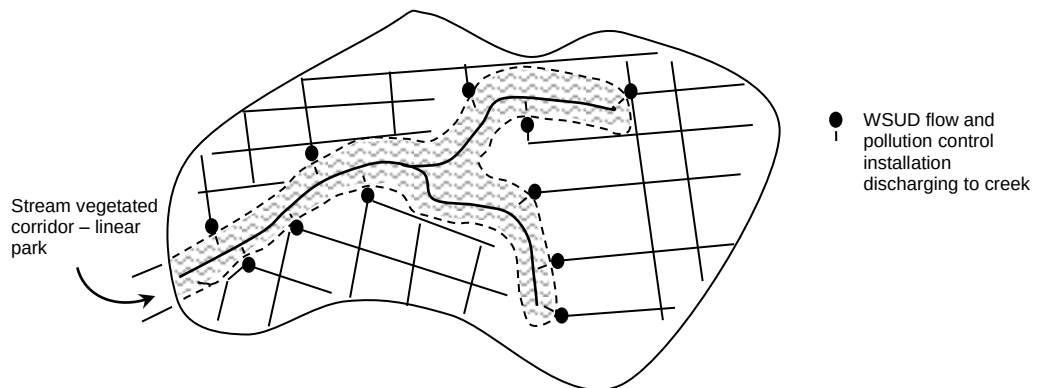
- greater surface runoff generated than occurred in the original, natural catchment; and,
- greatly reduced response (or ‘lag’) time, compared with that of the original catchment.



(a) Natural or substantially natural catchment



(b) Fully developed catchment with 'hard' flow paths and drainage lines



(c) Developed catchment with natural stream: WSUD in action

FIGURE 1.8 : Three stream scenarios as development proceeds in a natural catchment

In the case of over-developed catchments, riparian property owners in the valley “bottom lands” are likely to experience frequent inundation by floodwaters carrying high silt loads and high levels of coliform, phosphorous, nitrogen, heavy metals and petroleum hydrocarbons. Furthermore, because of rapid removal of stormwater from the urban landscape and consequent limited infiltration occurring during runoff events, supplementary irrigation is commonly undertaken to compensate for low soil moisture levels. Any remaining portions of natural streams and waterways of the valley may dry up completely during the low rainfall months (Southern and Central Australia) and members of their (stream) biotic communities which require year-round flow for their survival may vanish (after Argue, 1986; see King, 2003). This scenario is represented in Figure 1.8b.

1.4.2 WSUD: planning for urbanisation in a natural or predominantly natural catchment

Where the opportunity presents itself to plan the course of urbanisation in an undeveloped or partly-developed catchment, perhaps on the outskirts of a metropolis, then WSUD techniques can be employed to manage storm runoff in a truly water-sensitive manner. The primary principle upon which such planning must be based is for :

Stormwater generated on each site in the developed catchment – *above that occurring on the same site in the natural catchment* – to be fully **retained** for at least the duration of the **lag time** determined for the natural catchment. [This is referred to as the **regime-in-balance** strategy in later text, Section 4.2; *lag time* is the delay evident between the period of *excess* rainfall input to a catchment and its appearance as a surface runoff flood wave at the point of catchment discharge.]

The presence of pollutants of various types in the urban landscape – sediment, phosphorous, nitrogen, hydrocarbon – of course, must also be taken into account in the planning process, but the requirements of this aspect can be integrated into on-site stormwater management practices and/or into ‘final filter’ installations discharging to the urban streams.

Stormwater ‘source control’ retention may be provided by :

- Roof gardens, ‘green’ roofs and roof water storages.
- Rainwater tanks or similar storages.
- Surface infiltration at permeable surfaces including porous/permeable paving.
- Storage/percolation in “leaky” underground devices.
- Above-ground or underground storages or “leaky” devices with aquifer access and no recovery of water entering aquifers.
- Above-ground or underground storages or “leaky” devices with slow-drainage disposal (typically, 24 hours or longer emptying) to the stormwater drainage path or to a local waterway.
- ASR schemes providing recovered water available for various surface uses.

Application of this approach is illustrated in Figure 1.8c : it manifests itself in a developed catchment whose hydrological behaviour, in terms of surface runoff and underground seepage/aquifer processes, resembles that of the original basin or sub-catchment in its undeveloped state. It follows that the central drainage paths – where these processes have been applied – can exhibit not only hydraulic and hydrological similarity to the original waterways, but also, similarity of ecosystem behaviour – vegetation and habitats.

Some overseas practices, notably those of North America (Stephens et al, 2002; Gilliard, 2004) and UK (CIRIA, 2001), attempt to reproduce in their sensitively designed urban catchments the same runoff hydrographs as observed in or attributed to the original (natural) catchments, in *all incident storms* – *small, medium and large*. The quest for this goal introduces yet another parameter of importance, that of storm magnitude linked to a flood wave frequency indicator – “once in 2 years”, “once in 10 years”, “once in 100 years”, etc., called **average recurrence interval or ARI**. Success in achieving true hydrograph correspondence between the natural and developed catchments through the full range of ARIs is certainly an ideal and worthy objective, but one whose attainment in real world circumstances must be viewed with scepticism.

A practical compromise on this issue which delivers an ‘acceptable’ rather than ideal outcome is to base similarity of natural and developed catchment surface runoff performance on the selection of a ‘representative’ or *design storm* which combines the ‘lag’ time derived for the natural catchment with an adopted ARI of ‘median’ proportions – for example, “once in 5 years”, “once in 10 years”, “once in 20 years”, etc. Some practices adopt the ‘dominant’ or channel-forming flow ARI (1 – 2 years) for this focus (Dept of Water, WA, 2007; SEQ Healthy Waterways, 2007).

It follows that ‘enlightened’ WSUD development can be achieved in a natural catchment provided there is :

- **retention** of all (surface) runoff *above that occurring in the natural catchment*;
- maintenance of the **same ‘lag’ time** as occurs in the natural catchment;

- recognition of the need for pollution control installations ‘at-source’ and/or at waterway entry points, and,
- an appropriate **ARI** adopted leading to ‘acceptable’ runoff similarity (natural/developed catchments).

Realisation of these characteristics in the engineering of developed catchment infrastructure has the potential to deliver urbanisation outcomes that preserve natural waterway drainage paths along with their associated riverine and terrestrial environments and habitats in a practically sustainable manner. The second and fourth items in the above list, combined, form the basis of the **critical design storm** (see Section 4.2.3).

A major problem which the planner/designer must face in implementing schemes resulting from the above considerations, is that of sediment control during the construction phase of development. As explained in Section 1.3.3, it is possible – without due care – for a suburban water-sensitive installation designed for, say, 50 years service in ‘fully-established’ mode, to be filled with sediment before the construction operation has been completed (Argue, 2000). A two-phase solution to this problem may be called for : sediment storage/removal during and upon completion of construction, followed by long-term sediment management during the operating life of the development.

1.4.3 Retrofitting WSUD practices into an over-developed urban catchment

Opportunities for carrying out the type of development described in Section 1.4.2 are not the most common experience. It is more usual for a high level of urban development to have occurred, possibly to the extent referred to in Section 1.1, that is, serious flooding of catchment ‘bottom lands’ as well as pollution loads having significant impacts on downstream urban creeks and waterways including estuaries and marine foreshores. It is in these circumstances that WSUD practice (stormwater ‘source control’) meets its greatest opportunities and challenges.

The task of remediating an over-developed catchment calls for a two-step approach :

STEP 1 : Apply practices throughout the landscape aimed at **retaining as much surface runoff as possible** from every element of development or re-development taking place in the catchment; this process should be integrated with a strategy to incorporate pollution control installations within all modified sites and/or at points of discharge to urban waterways; and,

STEP 2 : restore the natural drainage path, as closely as practicable, to its pre-development state.

The process of **retention** (STEP 1) in the present case differs significantly from its counterpart in the natural catchment development process outlined above. Here, it is not sufficient to retain only the storm runoff **excess** (above that of the natural catchment) on a *site-by-site basis*, because development/re-development of sites occurs only on an opportunistic rather than ‘planned’ basis. It is therefore imperative that as much surface runoff as practicable be retained (for, at least, the duration of the critical design storm) in every development/re-development situation that arises in order to ‘make up’ for the numerous sites where no remedial action *in the short term* is likely. [This is referred to in later text as the **yield-minimum** strategy, Section 4.2.]

The greatest difficulty encountered by the practitioner attempting to implement this approach relates to established roadways. Techniques exist which can, given total community commitment, lead to retention measures being retrofitted into other urban classifications – residential (individual dwellings, housing clusters, high-rise apartment blocks), industrial sites, commercial/sporting complexes, etc. – as required by ‘enlightened’ re-development. However, retrofitting retention technology into an established roadway – short of complete or substantial reconstruction using permeable paving – presents, usually, insurmountable difficulties (including cost).

There are other components of the urban form in addition to roadways which may present similar difficulties, for example, sealed car park areas, multi-storey car parks, commercial/industrial estates, etc, where opportunities to modify existing building layouts to incorporate required stormwater retaining installations are severely limited. Or the cost of installations, were they to be constructed, may be prohibitive.

In circumstances such as these, it is possible to achieve the overall objectives of WSUD (stormwater ‘source control’), by a combination of **full retention** (retention of *all* surface runoff) in those components where this is possible, and **limited** retention in others. The process by which STEP 1 is implemented in an over-developed catchment has much the same basic profile as its earlier counterpart, and is :

- **retain as much (surface) runoff as possible** at each development/re-development site;
- employ (in design) the same catchment-wide **‘lag’ time** determined for the original catchment;
- recognise the need for pollution control installations ‘at-source’ and/or at waterway entry points, and,
- adopt an appropriate flood **ARI** for the catchment reflecting the consensus interests of its stakeholders.

The case for remediating an over-developed catchment through application of these actions as a first step, together with continuation of the process to embrace STEP 2 (restoration of the natural drainage path), requires high levels of commitment, patience and funding which should not be underestimated by any community setting out to “correct the errors of the past” (see Chandler, 2001). The ultimate outcome of such commitment is transformation of the catchment – illustrated in Figure 1.8b – towards the situation depicted in Figure 1.8c.

1.4.4 The problem of ‘oversizing’

It is tempting to interpret the suggestion made in Section 1.4.3 as meaning that ‘catch up’ retention capacity – by installing **above-full** retention – can be incorporated into all new developments/re-developments, allowing past failure (to install adequate retention capacity) to be corrected rapidly and progressively as development proceeds. Such **oversizing**, that is, the provision of greater capacity than called for at individual installations, must be avoided : **no greater capacity than that required to store all runoff generated at a site in the critical design storm** (see Section 1.4.2) **should be provided**.

A strategy based on correctly-sized installations, applied consistently to all new development/re-development cases in an overdeveloped catchment can, in time, deliver the ultimate overall goal, namely, runoff (volume) in the design storm event equal to that generated in the catchment before development – the **regime-in-balance** strategy referred to in Section 1.4.2. Catchment managers need to be patient, however, as steady progress is made towards this goal and not use ‘oversizing’ to achieve it in a compressed time scale.

1.4.5 WSUD – stormwater ‘source control’ : the limitations

It will be apparent from all that has been discussed in the course of Chapter 1, that stormwater ‘source control’ installations can be divided, broadly, into two main groups :

- above-ground or in-ground sealed systems; and,
- in-ground, including at-surface, “leaky” devices.

Opportunities for using the former are virtually inexhaustible and include rooftop rainwater storages, roof gardens, “green” roofs, enlarged gutter (eave) rainwater collectors, above-ground rainwater tanks and in-ground, sealed rainwater tanks. However, limitations must be recognised with certain classes of in-ground, “leaky” systems whose presence in unsuitable locations can lead to damage of buildings, in particular, domestic footings set in water-reactive soils, and situations where percolation from “leaky” systems will exacerbate problems associated with high or potentially rising groundwater. Problems of particular severity can arise where, in the latter case, the groundwater is highly saline. A more extensive discussion of such limitations may be found in Section 3.8.

Any review of limitations to the use of in-ground “leaky” retention practices must address the issue of soil permeability : it is widely held that locations where the permeabilities of soils are low to very low must be precluded from retention technology. Certainly, simple systems draining naturally (see Figure 1.3) cannot be successfully employed in such circumstances. But the techniques illustrated in Figure 1.4 open the way for on-site stormwater retention/detention – with, perhaps, 24-hour or longer emptying time incorporated into the design – in virtually the entire range of soil environments.

2. DEVICES AND SYSTEMS USED IN STORMWATER MANAGEMENT

INTRODUCTION AND DIRECTORY

This Chapter provides brief descriptions of a number of devices, systems and facilities used primarily in stormwater quality management : it is not comprehensive but intended, mainly, as an introduction to the field. There are numerous well-recognised publications in the pollution control domain, some produced in Australia, which describe various systems in great depth (e.g. Schueler et al, 1987; Ferguson, 1994; Debo and Reese, 2003; NSW EPA, 1999; CSIRO, 1999; CIRIA, 2001; Ontario Ministry of the Environment, 2003, France, 2003; Melbourne Water, 2005; Engineers Australia, 2006; Dept of Water WA, 2007). Practitioners requiring detailed information should consult these more comprehensive sources as well as information on commercially available devices and products obtainable from manufacturers' brochures and industry websites.

a. Selection

b. Hydrological design

c. Other design and option issues

ROOF SYSTEMS

“LEAKY” WELLS AND INFILTRATION TRENCHES (see Chapter 5)

FILTER STRIPS

POROUS AND PERMEABLE PAVEMENTS

2.5.1 Porous pavements

2.5.2 Permeable pavements

SWALES AND BIO-RETENTION SWALES

TRASH RACKS, BASKETS AND BOOMS

CATCH BASINS AND “SOAK PITS”

2.8.1 Catch basins

2.8.2 “Soak pits”

WATER QUALITY IMPROVEMENT STRUCTURES

2.9.1 Water quality inlets

2.9.2 Oil/grit separators

2.9.3 Fine sediment removal/retention device

“FIRST FLUSH” TANK – PARTITIONING AND “FIRST FLUSH” STORMWATER RUNOFF

INFILTRATION BASINS

EXTENDED DETENTION BASINS

WET DETENTION BASINS

CONSTRUCTED WETLANDS

[These sections omitted from the Student Edition]

See page xi for details of distributors of the full Handbook

3. THEORY, DATA AND SOME IMPLEMENTATION ISSUES

3.1 INFILTRATION OF STORM RUNOFF

3.1.1 Basic theory

Direct **infiltration** of storm runoff into a residential allotment topsoil surface, a permeable pavement, a grassed hard-standing area or into a well-vegetated loamy area specially intended for this purpose, represents a simple application of Darcy's Law as expressed in the following equation :

$$q = k_h \cdot I \quad (3.1)$$

where q = infiltration flow rate per unit surface area ($\text{m}^3/\text{s}/\text{m}^2 = \text{m/s}$)

k_h = hydraulic conductivity (m/s)

I = hydraulic gradient, $\frac{\Delta h}{\Delta \ell}$ (m/m).

As in all storm runoff infiltration and seepage analyses, the 'fully drained' condition is assumed :

$$I = \frac{\Delta h}{\Delta \ell} = 1.0 \quad (3.2)$$

Measurements made at many field sites and retention installations show this assumption to be, generally, satisfactory. The qualification which should be noted, however, is that $I = 1.0$ represents the limiting condition to which the hydraulic gradient tends during the wetting-up process. Field situations where high infiltration rates are observed in the initial stages of wetting-up are produced **not** by hydraulic gradients greater than 1.0 but, rather, as a consequence of water filling the void spaces between coarse particles.

Stormwater flow infiltration, being intermittent, frequently shows void-filling, as described above, before progressing to the fully drained condition. Design procedures which employ the $I = 1.0$ assumption are therefore, generally conservative.

It follows that direct infiltration of surface runoff into a facility specifically set aside for this purpose can be accomplished by matching the peak surface design runoff flow from the contributing catchment, Q_{peak} , to its capacity for infiltration, thus :

$$Q_{\text{peak}} = q \cdot A_i \quad (3.3)$$

where $q \equiv k_h$, the design hydraulic conductivity [see Eqns. (3.1) and (3.2) above],

and A_i = surface area available for infiltration.

The design of permeable paving surfaces – typically, concrete block pavers with vertical 'slots' or gravel-filled 'shafts' – is amenable to the above theory without modification, provided the overall or global value of hydraulic conductivity is known for the product and its underlying sub-structure system. Application of the (above) theory to hard-standing areas of the type illustrated in Figure 3.1a must incorporate a **blockage factor, Ψ** , to take account of the impervious surface area which is **not** subject to infiltration. A blockage factor, $\Psi = 0$ may be applied in cases where the paving is uniformly porous, and to permeable paving systems for which *overall hydraulic conductivity is known*.

However, the issue of greatest concern for designers of such porous/permeable surface systems, is the effect that **sediment accumulation** has on permeability (hydraulic conductivity, k_h) as the installations "age" in service: this important issue is addressed in Sections 3.1.2 and 3.1.3.

The matching of peak flow rate to infiltration capacity (with zero site overflow discharge) is a valid approach for minor drainage systems designed to accept the typical ARI, $Y = 2, 5$ or 10 years storm runoff. The technology has also been demonstrated beyond this range (to ARI, $Y = 20$ -years, 50-years and even 100-years events) with 'sheet' flow applied to residential allotment topsoil surfaces (Riley, 2001).



(a) : Example of block-reinforced grass used for vehicle hard-standing areas

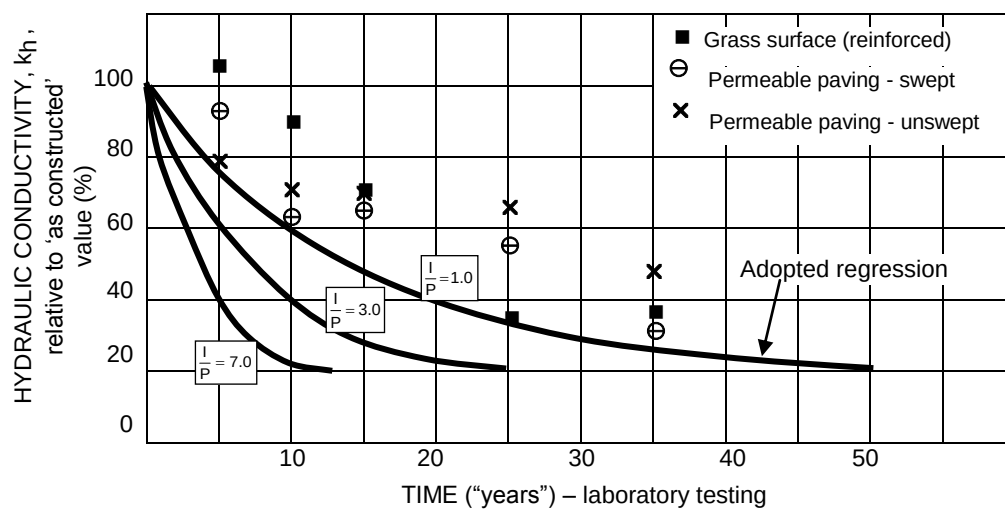
(b) : Hydraulic conductivity, k_h , deterioration trend curve for porous and permeable paving filters subject to (fine) sediment loading, based on laboratory results (after Rommel et al, 2001; UWRC, 2002a and b)

FIGURE 3.1 : Two aspects of stormwater infiltration into porous and permeable paving

3.1.2 'Lifespan' and the effect of sediment accumulation on permeability

The origins of sediment in the urban landscape and observed impacts of water-borne particulate matter on the performance of components of retention systems, were reviewed briefly in Section 1.3.3. The clear lesson which emerges from those considerations, particularly for systems of the type under discussion here – porous and permeable surfaces – is that they can be expected to show steadily reducing permeability (infiltration rate or hydraulic conductivity, k_h) as their years of service lengthen towards eventual reconstruction or reinstatement. The term '**lifespan**' is used to describe and quantify this phenomenon.

Reliable data on 'lifespan' or rates of permeability reduction in operating systems are either non-existent or anecdotal (Pratt, 1997): the studies by Rommel et al (2001) and UWRC (2002a and b) aimed to resolve these uncertainties. Testing by Rommel et al of a proprietary block paving system with internal geotextile filter layer used in a simulated (laboratory test rig) car park catchment element **with equal areas of impervious and permeable paving**, that is $I/P = 1.0$, showed a characteristic regression in filter permeability with time. [The simulation experiments applied one "year" equivalent of Adelaide, South Australia, rainfall plus sediment to the rig in each 7-hour period of testing.] The sediment application rate was more than 200 mg/L for the full period: this is a relatively high value for car parks and established

(but not construction phase) urban catchments. The graph of hydraulic conductivity normalised to ‘as constructed’ values, versus testing time in “years”, is presented in Figure 3.1b. Results from tests on an identical rig with grass (growing in a sand/gravel mixture : see Section 7.8.5) receiving the runoff, are also plotted in Figure 3.1b. ‘As constructed’ values for k_h were : 1.0×10^{-3} m/s for permeable paving and 2.5×10^{-4} m/s for the sand/gravel mixture.

The first important conclusion which can be drawn from Figure 3.1b is that permeability reduced to around 40% of ‘new’ product value over a period of some 30 – 35 (experimental) “years” : the adopted regression shows a drop to 20% of ‘as constructed’ value at 40 – 50 years. Taking this period as an acceptable ‘lifespan’ for porous/permeable paving systems leads to the following design guideline for surface infiltration, based on the adopted regression ($I/P = 1.0$) :

Guideline 1 : Design procedures for porous and permeable infiltration systems based on Eqn. (3.3) should adopt hydraulic conductivity, k_h , equal to **20% of ‘new’ product (or system) value for permeability** to ensure acceptable ‘lifespan’ performance.

Important qualification : This guideline applies to **fine** sediment input retained on an internal geotextile layer in the case of permeable paving (see Figure 2.3), and *on the vegetated surface* in the case of the porous system. A ‘lifespan’ of at least 40 years can be expected for an installation such as a car park with impervious-to-porous/permeable paving ratio, $I/P = 1.0$ (the geometrical ratio reported by Rommel et al and UWRC). It would be quite wrong to assume the same performance in a car park with significantly different impervious-to-porous/permeable ratio. For example, if I/P is increased to 3.0 thereby doubling the load on the filter system, then the ‘lifespan’ is reduced to half, as indicated by the derived curve in Figure 3.1b. Again, if I/P is increased to 7.0, ‘lifespan’ is not likely to be more than 12 years. These (latter) operating periods are not ‘long-term’, but may nevertheless be satisfactory in many applications and/or may be acceptable periods between major reconstruction or reinstatement of the paving system down to the level of the geotextile layer, inclusive. The 20% ‘new’ product value (of permeability) is recommended for universal use, but practitioners must recognise that the ‘lifespan’ which results is likely to vary depending on the I/P value of each case, as explained above.

3.1.3 Surface blockage and ‘lifespan’

Pollutant input to a porous or permeable paving system including coarse sediment, vegetative matter (leaves, blossoms, berries, etc.) as well as fine sediment is frequently **not** evenly distributed over the entire receiving surface. Instead, the pollutant supply may enter the paved area as a ‘line source’ along the edge of a carriageway or as a jet discharged from an upstream gutter. In either case, the porous/permeable surface where the flow makes first contact suffers the full impact of the pollutant stream, and may reach a situation of complete blockage by particulate matter at a relatively early stage in the life of the facility. The type of blockage referred to here is evidence of the porous/permeable system acting as a GPT (gross pollution trap). With time, the area affected spreads downstream as a ‘blockage front’.

In vegetated porous systems, this front is almost imperceptible, being obscured by the grass or other groundcover; furthermore, the filtering action of the grass coupled with regular removal of retained material through mowing, significantly diminish the impact of the “blockage front” in these systems. But, in surfaces finished with permeable paving, it may be quite apparent soon after commissioning. This should not cause alarm or be the signal to initiate major reinstatement works! The permeable paving system **downstream** will readily cope with the flow entering the facility perhaps for many years, provided it has been designed in accordance with Guideline 1 (and following qualification) above, and also, that the area sacrificed to blockage is located **in the uppermost segment of the porous/permeable paved surface**.

However, a new guideline is necessary to take account of this category of surface blockage, particularly where regular inspection and maintenance of the porous or permeable surface cannot be guaranteed and/or where no provision is made for interception of this type of particulate matter passing from the upstream, impervious catchment.

Guideline 2 : Porous and permeable infiltration systems receiving untreated runoff from contributing, ground-level impervious catchments, *and where regular inspection/maintenance cannot be guaranteed*, should have impervious to porous/permeable area ratios (I/P) not greater than the values given by Table 3.1, to ensure the indicated ‘lifespan’ performance.

TABLE 3.1
SURFACE BLOCKAGE : I/P RATIOS AND ADVISORY ‘LIFESPANS’*

Systems based on **permeable paving** : use ‘lifespan’ (table) values directly
 Systems based on **vegetated porous surfaces** : apply factor of 5 to tabulated ‘lifespan’ values

CATCHMENT DESCRIPTION	I/P RATIO 0	I/P RATIO 1.0	I/P RATIO 2.0	I/P RATIO 3.0	I/P RATIO 5.0	I/P RATIO 7.0
Parking bays in coastal suburb beside park (many trees)	lifespan : 10 years	lifespan : 5 years	lifespan : 3.3 years	lifespan : 2.5 years	lifespan : 1.7 years	lifespan : 1.2 years
Parking bays or car park in coastal suburb : site without trees	lifespan : 20 years	lifespan : 10 years	lifespan : 6.7 years	lifespan : 5 years	lifespan : 3.3 years	lifespan : 2.5 years
Parking bays or car park in average suburb : site with some trees	lifespan : 20 years	lifespan : 10 years	lifespan : 6.7 years	lifespan : 5 years	lifespan : 3.3 years	lifespan : 2.5 years
Parking bays or car park in average suburb : site without trees	lifespan : 40 years	lifespan : 20 years	lifespan : 13.3 years	lifespan : 10 years	lifespan : 6.7 years	lifespan : 5 years

* Based on observations of field installations in Adelaide over eight years period.

This Guideline (and Guideline 1, above) is derived from testing regimes which represented – in the field (and laboratory) – ‘established neighbourhood’ conditions, much less severe than ‘construction phase’ conditions. However, Guideline 2 should be interpreted, with respect to I/P ratios as involving flow contributed from **ground-level paved surfaces** only to establish values for I. In these circumstances, Guideline 1 still needs to be applied with full account taken of the **total** catchment contributing runoff, but Guideline 2 can be restricted to consideration of ground-level paved surfaces only and the contributing roof area can be ignored. Note that I/P = 0 corresponds to a fully paved porous/permeable site.

Another circumstance in which the I/P ratios employed in Guidelines 1 and 2 may differ is where special provision is made to trap coarse sediment before it passes onto a permeable surface. If a system capable of retaining all pollution components greater than 300 µm can be installed (upstream) to perform this duty, then design according to Guideline 1, only, is required. Of course, maintenance of the type(s) described by Dierkes et al (2002) will prolong the ‘life’ of these systems. Such maintenance, will also improve their appearance : this may be a determining factor where permeable paving is used in a high profile urban environment.

The same principle of I/P ratio recognition explained in **Important Qualification** following Guideline 1, above, explains the entries in Table 3.1 for I/P ratios other than 1.0. Also, it should be noted that ‘lifespan’, as it applies in Guideline 2, can be interpreted as the time gap between reconstruction/reinstatement operations affecting **the surface components only**. While inspection/assessment of the geotextile layer within a permeable paving system being upgraded in this way is strongly advised, it is quite possible that its replacement is unnecessary (Adelaide observation).

Total system design therefore involves use of an appropriate procedure, for example Procedure 1A or 1B (Section 5.1), applying Guideline 1 and relevant I/P information from Figure 3.1b, followed by Guideline 2 and advice from Table 3.1. This will normally result in **two** values for area in permeable systems where ‘lifespan’ is specified OR, two values of ‘lifespan’ where (I/P) ratios are fixed. The maintenance/reinstatement consequences of these alternatives should inform the decision which is taken in either of these two cases. Design of vegetated porous paving, on the other hand, does not involve such alternatives because the filtration process which takes place in these systems occurs at one level only – the grassed surface.

The task of reconstructing/reinstating permeable paving (surface) blocks at the end of a ‘lifespan’ period, while certainly labour-intensive, can be performed without wholesale lifting, cleaning, stockpiling and relaying. Unskilled workers removing, cleaning and replacing the blocks two or three at a time can reinstate a paved area at the rate of about one square metre per 45 minutes. Porous paving (grassed hard-standing areas), on the other hand, are much more difficult to reinstate. However, the growth process itself, as well as regular mowing of these systems, represents a form of ‘forced maintenance’ which accounts for the 5-fold multiplier advised for ‘lifespan’ periods in Table 3.1.

Sediment loads generated during local construction activity (see Section 1.3.3) are unpredictable other than in general terms, so that the best information obtainable on when remedial action such as reinstatement/reconstruction of an installation should be carried out comes from careful monitoring of the installation itself. In this respect it should be recognised that a paving installation is performing satisfactorily as a whole (system) if, in a design or significant storm event, the entire storm runoff is managed without overflow, even though failure (complete blockage) may be apparent in a limited area.

3.2

SOIL PERMEABILITY AND PERCOLATION AT DEPTH

3.2.1 Percolation of storm runoff

The disposal of storm runoff into percolation devices such as “leaky” wells or gravel-filled trenches draws on the same hydraulic theory as that used for surface infiltration. The modelling difficulties which arise in this case, however, stem from the more complex area through which the flow passes as it leaves the device on its outwards/downward path towards the watertable.

The basic augerhole case is considered first. The assumptions which lie at the heart of this derivation are that :

- a well-defined wetting front develops from the walls and floor of the augerhole as flow seeps from it and that the flow space occupied by this seepage is completely saturated;
- the seepage outflow from the augerhole is acting under ‘free draining’ conditions, i.e. hydraulic gradient, $I = 1.0$ applies;
- seepage from the augerhole, once it has commenced its journey through the soil matrix, does not return to the augerhole.

Applying these assumptions to outflow from a cylindrical augerhole with **low** watertable (Figure 3.2a) gives (Jonasson, 1984) :

Seepage outflow volume dV , from an augerhole, diameter $= 2r_o$, while depth changes from h to $(h - dh)$ in time dt is given by :

$$dV = \pi r_o^2 \cdot dh \quad (3.4)$$

Also, from Darcy’s Law, $dV = k_h I A_i dt$

where k_h = hydraulic conductivity

I = hydraulic gradient

A_i = area through which flow is percolating.

Hence,

$$dV = k_h I [\pi r_o^2 + 2\pi r_o h] dt \quad (3.5)$$

Equating (3.4) and (3.5) :

$$\pi r_o^2 \cdot dh = k_h I [\pi r_o^2 + 2\pi r_o h] dt \quad (3.6)$$

i.e.

$$dh = k_h I \left[1 + \frac{2h}{r_o} \right] dt = k_h I \left[\frac{r_o + 2h}{r_o} \right] dt \quad (3.7)$$

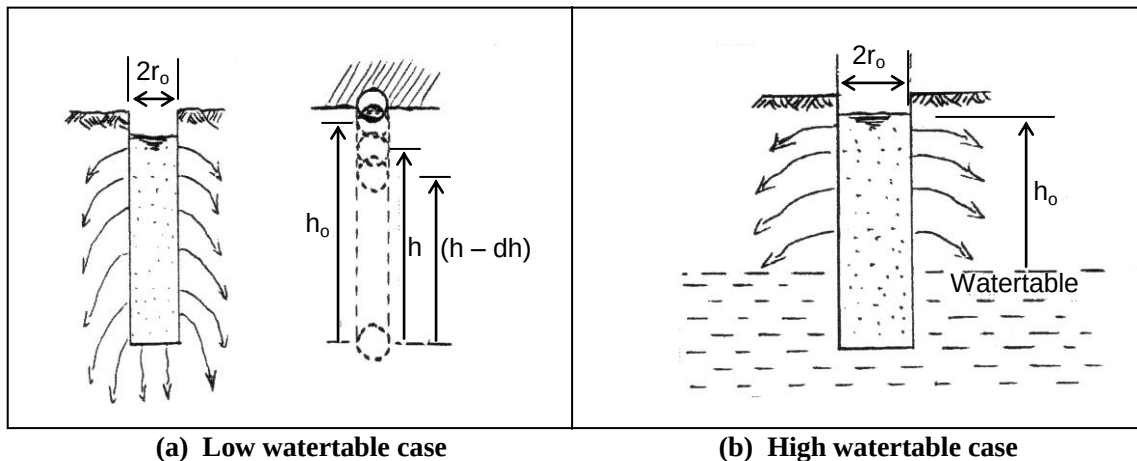


FIGURE 3.2 : Augerhole analysis

$$\text{i.e.} \quad k_h I \cdot dt = \left[\frac{r_o}{r_o + 2h} \right] dh = \frac{r_o}{2} \left[\frac{1}{\frac{r_o}{2} + h} \right] dh \quad (3.8)$$

$$\text{i.e.} \quad \frac{2k_h I}{r_o} \int_{t_1}^{t_2} dt = - \int_{h_1}^{h_2} \frac{dh}{\left(\frac{r_o}{2} + h \right)} \quad (3.9)$$

$$\text{i.e.} \quad \frac{2k_h I}{r_o} [t_2 - t_1] = - \left[\log_e \left(\frac{r_o}{2} + h \right) \right]_{h_1}^{h_2} \quad (3.10)$$

$$\begin{aligned} \text{i.e.} \quad \frac{2k_h I}{r_o} [t_2 - t_1] &= - \left[\log_e \left(h_2 + \frac{r_o}{2} \right) - \log_e \left(h_1 + \frac{r_o}{2} \right) \right] \\ &= \left[\log_e \left(h_1 + \frac{r_o}{2} \right) - \log_e \left(h_2 + \frac{r_o}{2} \right) \right] \end{aligned} \quad (3.11)$$

$$\begin{aligned} \text{i.e.} \quad k_h &= \frac{r_o}{2I(t_2 - t_1)} \log_e \left[\frac{h_1 + \frac{r_o}{2}}{h_2 + \frac{r_o}{2}} \right] \\ &= \frac{2.3r_o}{2I(t_2 - t_1)} \log_{10} \left[\frac{h_1 + \frac{r_o}{2}}{h_2 + \frac{r_o}{2}} \right] \end{aligned} \quad (3.12)$$

Assuming free draining seepage conditions, i.e. $I = 1.0$, gives :

$$\text{Hydraulic conductivity} \quad k_h = \frac{1.15r_o}{(t_2 - t_1)} \log_{10} \left[\frac{h_1 + \frac{r_o}{2}}{h_2 + \frac{r_o}{2}} \right] \quad (3.13)$$

A modified form of Eqn. (3.13) is used where the watertable is **high** and intersects the augerhole (Figure 3.2b). In this case, all heights, h , are measured **from the watertable surface** :

The analysis follows that presented above **except** that the πr_o^2 term is dropped from Eqn. (3.5), and subsequently; outflow is assumed to take place through the walls, only, of the borehole. The resulting expression for hydraulic conductivity is, therefore :

$$k_h = \frac{1.15r_o}{(t_2 - t_1)} \log_{10} \left[\frac{h_1}{h_2} \right] \quad (3.14)$$

The inverse augerhole equations, above, are used in two important ways in stormwater management technology. The first is in field testing to establish site hydraulic conductivity values (see Section 3.2.2, below); the second is in performance review studies such as those upon which the scale effect Moderation Factor, U , is based (see footnote, Section 5.1.2).

3.2.2 Hydraulic conductivity and its measurement

The performance of a practice or technique for managing stormwater by in-ground retention is influenced by the hydraulic conductivity of site soil. Five broad soil permeability classifications are recognised – these are :

Sandy soil :	$k_h > 5 \times 10^{-5} \text{ m/s}$
Sandy clay :	$k_h \text{ between } 1 \times 10^{-5} \text{ and } 5 \times 10^{-5} \text{ m/s}$
Medium clay and some rock :	$k_h \text{ between } 1 \times 10^{-6} \text{ and } 1 \times 10^{-5} \text{ m/s}$

Heavy clay : k_h between 1×10^{-8} and 1×10^{-6} m/s

Constructed clay : $k_h < 1 \times 10^{-8}$ m/s,

where k_h is the value of hydraulic conductivity determined by Jonasson's (1984) 'falling head' augerhole method, Eqns. (3.13) and (3.14), above. Brief descriptions of these five soil classifications are contained in Section 1.3.4.

Prior to conducting a falling head test it is important that the soil surrounding the borehole is **wet**. This is achieved by maintaining an inflow to the hole and checking its outflow rate until fairly constant. (A borehole of any diameter and depth may be used, but diameter = 125 mm and depth = 2.0 m is preferred : the borehole should be lined with 110 mm diameter perforated PVC pipe or equivalent.)

This process also provides the opportunity for obtaining a constant head soil permeability value based on the same theory presented in Section 3.2.1. In this case:

$$Q = q \cdot A_i = (k_h \cdot I) A_i \quad (3.15)$$

where $A_i = [\pi r_o^2 + 2\pi r_o h]$

hence $k_h = \frac{Q}{[\pi r_o^2 + 2\pi r_o h]}$

where Q = rate of topping-up of borehole (m^3/s). Site testing to obtain a satisfactory value of k_h should be repeated up to six times using 20-minute topping-up durations for each test.

When the borehole soil environment is satisfactorily wet following the constant head test, it is convenient to commence the falling head test by simply ceasing to supply further inflow to the bore. The characteristic h versus t curve, illustrated in Figure 3.3, results from subsequent observations.

The relevant basic equations from which the permeability value may be calculated are :

$$k_h = \frac{1.15r_o}{t_2 - t_1} \log_{10} \left[\frac{h_1 + \frac{r_o}{2}}{h_2 + \frac{r_o}{2}} \right] \text{ m/s} \quad \text{low water table case} \quad (3.13)$$

and $k_h = \frac{1.15r_o}{t_2 - t_1} \left[\log_{10} \frac{h_1}{h_2} \right] \text{ m/s} \quad \text{high water table case} \quad (3.14)$

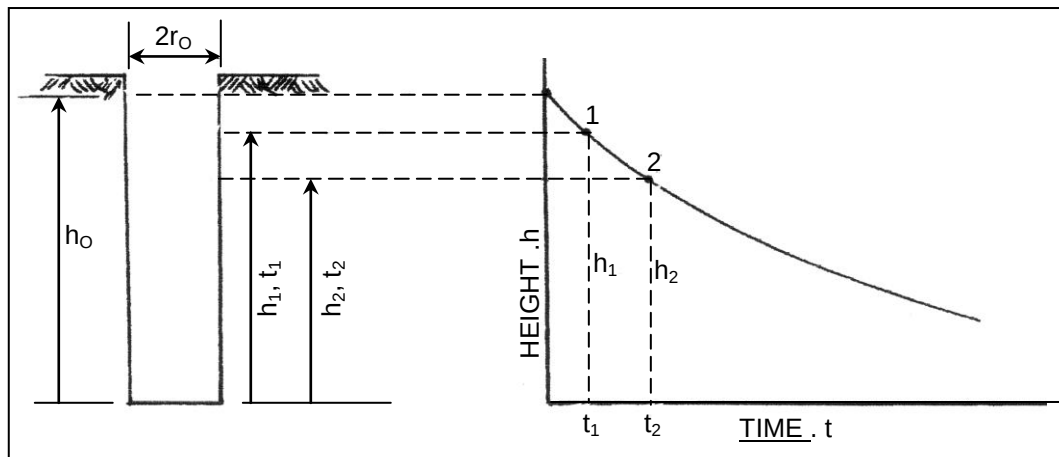


FIGURE 3.3 : Inverse augerhole test – definition of terms and typical "h" versus "t" relationship

A plastic, cylindrical float with attached measuring rod may be used to obtain readings of “h” as the water level drops. Time, t , is measured from the instant inflow to the bore ceases (following the constant head test).

The main disadvantage of hydraulic conductivity determination by a falling-head procedure is the (apparent) variation in k_h values which results when successive t - h observations are substituted into Eqns. (3.13) or (3.14). For this reason a standard 60 minute value for hydraulic conductivity, k_{60} , is used in clay soils. In sandy soils, the rate of fall of the water level may be quite rapid: in such cases the hydraulic conductivity is calculated for depth-time conditions around $h = 1.70$ m.

Appendix A includes a more comprehensive treatment of this topic and a Specification for hydraulic conductivity testing.

3.3 RUNOFF VOLUME AND TIME BASE τ IN ‘SOURCE CONTROL’ TECHNOLOGY

The broad class of underground, on-site retention installations described as “leaky” devices includes those illustrated in Figure 1.3 and installations of similar type which incorporate proprietary units specially manufactured to store water and release it slowly into the surrounding soil. Atlantis and Ausdrain (“milk crate”) cells and Everglas units are examples of the latter : infiltration or “dry” ponds are another member of this class of retention device. “Leaky” wells may be 2 – 3 metres deep; trenches have a maximum depth, typically, of 1.5 metres with 0.3 m of backfill cover; “dry” ponds – for safety reasons – are rarely deeper than 0.50 m.

These (in-ground) devices are effective in soils of high and medium permeability, but perform poorly in low permeability soils. Such soil conditions call for installations employing the same range of materials and proprietary products (gravel, slotted pipes, “milk crate” cells, etc.), but arranged in a mattress configuration, usually 0.3 – 0.5 m deep : the term “soakaway” is applied to these devices (see Figure 1.4b).

Engineering design of all types of installations referred to in this section depends on two main factors :

- the **volume**, ∇ , of stormwater runoff which **enters** the device during the design storm period of runoff; and,
- the **time base**, τ , of the design storm runoff hydrograph.

[The Handbook uses the ‘design storm’ method in some procedures and the ‘continuous simulation’ modelling approach in others. A comprehensive explanation of the issues involved in these uses is presented in Sections 3.4 and 3.5.]

There is, of course, a strong link between stormwater runoff volume ∇ and the design storm parameters, **storm** duration, T_C , and (site) **time of concentration**, t_c . This inter-relationship is illustrated in Figures 3.4a and 3.4b, and explained in greater depth in Section 4.2. These parameters and the relationships which exist between them, are important ingredients of the design procedures presented in Chapter 5.

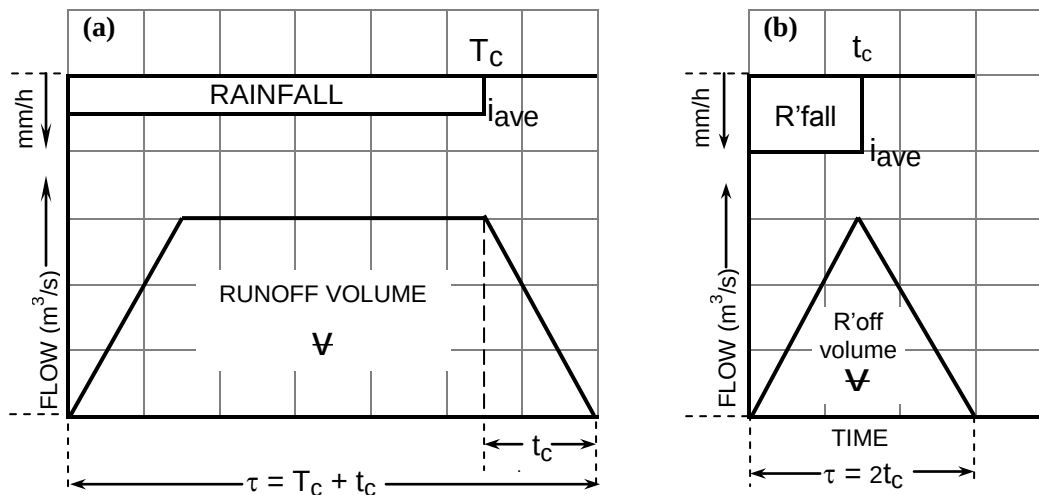


FIGURE 3.4 : Definition of ‘design storm’ parameters T_C , t_c , τ and ∇ for two catchment cases

3.4 AN EXCURSION INTO ‘CONTINUOUS SIMULATION’ MODELLING

3.4.1 Introduction: storm successions and the ‘design storm’ approach

Design procedures employed to create traditional street drainage networks have used the design storm approach assuming, typically, that runoff generated in the critical (design) event is unaffected by antecedent channel flow conditions or by floodwater held in temporary storage following a previous event (IEAust, 1999). The justification for this assumed behaviour was that flood wave movement through the urban landscape is so rapid that any remnants of a preceding event are unlikely to have a significant effect on the next storm runoff wave, even one of ‘design’ magnitude.

When **detention** techniques were introduced in the 1960s, the same assumption was invoked by the bulk of practitioners who ignored the changed conditions, even though the new technology involved significant elements of temporary ponding. Justification for ignoring the possible effects of interaction between storm successions was again based on the speed with which flood waves move, even though – in these new circumstances – they involved routing through temporary storage.

The introduction of **retention** technology puts the issue of storm successions and the inability of the design storm approach to deal satisfactorily with them, once more under the spotlight. The case against the design storm method – used widely in Australian and overseas contemporary practices (IEAust, 1999) – rests on two grounds :

1. users of the method are required to make professional judgements relating to the likely state of on-site storages – typically empty or half-full – at the commencement of the design storm, and,
2. design storm temporal patterns, determined by current analytical procedures are, in fact, “embedded” storm bursts : dimensions of storages determined from these (storm) profiles are therefore likely to be undersized (see Rigby et al, 2003).

Perhaps the strongest, comprehensive criticism of the design storm approach is contained in a Keynote Address delivered by Professor George Kuczera at the 2003 Engineers Australia Hydrology and Water Resources Symposium in Wollongong (Kuczera et al, 2003). Professor Kuczera concludes that there is “...already considerable evidence that specification of average initial conditions in the design storm approach is problematic and that the design storm approach can be seriously biased”. Professor Kuczera and other recognised researchers have proposed ‘continuous simulation’ modelling as the best remedy for these deficiencies (of the design storm method). These issues are explored for the case of initial loss estimation for flood studies in Heneker et al, 2003.

Given this authoritative assessment and the fact that many of the procedures proposed in later sections of the Handbook use a *modified* form of the design storm approach, it is, therefore, important that readers understand how the two design methods relate to each other and why the decision to *modify* the conventional design storm method was taken. The following content (Sections 3.4 and 3.5) provide an explanation of these matters. ‘Continuous simulation’ modelling is considered first, illustrated with aspects that are relevant to stormwater ‘source control’ in urban drainage systems.

3.4.2 ‘Continuous simulation’ modelling : some background

‘Continuous simulation’ modelling of runoff events using historical records, was introduced to the literature via the Stanford Watershed Model (Linsley and Crawford, 1960; Crawford and Linsley, 1962). Boughton (1966) was the first in Australia to use this form of modelling on a daily time-step basis; this was followed by Chapman (1968) using sub-daily time steps. More recent contributions of significance have been provided by Guo and Urbonas (1996) and Urbonas et al (1996) and by Dr. Tony Wong of the CRC in Catchment Hydrology, Monash University. The US and Wong’s work have focussed almost exclusively on problems of wetland ‘sizing’ and design, for example detention time (Somes and Wong, 2000). More recently, Coombes and Kuczera (2001) developed the PURRS program, based on ‘continuous simulation’ modelling and applied it to the design of rainwater tank systems.

‘Continuous simulation’ modelling uses historical rainfall records, typically, obtained from a long period – usually 20 years or more – together with a catchment rainfall/runoff model, to produce a continuous (rainfall) data-based streamflow record for the catchment – usually ungauged. While it is true that the resulting flow sequence is ‘constructed’ rather than ‘real’, its base in recorded rainfall information makes it

a valuable tool in seeking solutions to many runoff-related hydrological problems. It should be noted that the rainfall record used in the modelling observes the gaps (periods of no rainfall) between stormbursts as strictly as the stormbursts themselves.

Two applications of ‘continuous simulation’ modelling of interest to WSUD are explored here :

- in-ground “leaky” storages required to retain, on-site, storm runoff which has been cleansed through upstream filter/treatment units (see Figure 3.5a); and,
- rainwater storages receiving roof runoff: a household or industrial/commercial demand on the retained water is an essential element of the modelling process.

A simple general layout of the components which are common to both of these cases, is presented in Figure 3.5b.

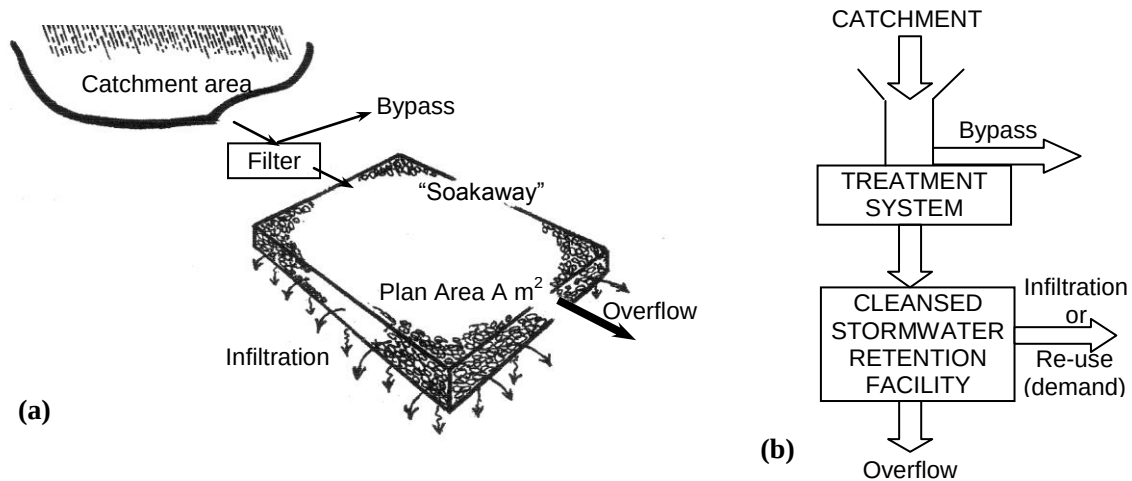


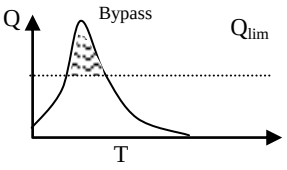
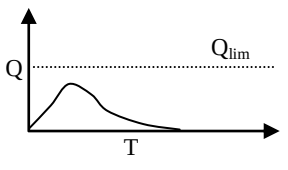
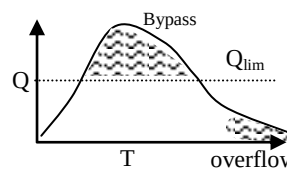
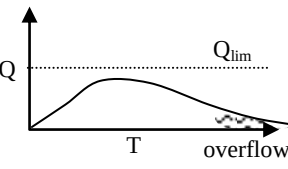
FIGURE 3.5 : Sketch of stormwater-retaining device and system schematic

It may assist the process of understanding these (two) applications if their essential components are recognised and the manner in which these relate to corresponding elements of other stormwater best practice installations involving storage is noted. The cases to which ‘continuous simulation’ modelling have been applied may be divided into four broad categories of systems :

1. **outlet control (only) with storage** : this includes detention basins with the full range of flood flows controlled at the basin outlet;
2. **formal inlet control (by design); storage, including on-site disposal; and overflow** : systems with inlets incorporating treatment/filter units with set flow capacities (hence bypass), before in-ground storage, typically with on-site abstraction by percolation, and overflow. This case is explored below;
3. **informal inlet control; storage (and on-site re-use); and overflow** : this covers rainwater tanks with upstream bypass of major flows taking place at roof guttering, before tank storage (with on-site abstraction for re-use) and overflow. This case is explored below;
4. **formal inlet control (by design); storage; and formal outlet control (by design)** : this represents the typical wetlands case with inlets (some treatment) having set flow capacities (hence bypass), before storage and formal outlet control (weir, ‘riser’, siphon, etc.).

There are four possible modes of operation of systems 2 and 3, above. These are illustrated in Figure 3.6.

The relationships between catchment area, storage, percolation rate, water (supply) demand, etc. which arise in the two cases (2 and 3, above), are, of course, quite different, however, the modelling steps involved in the process of arriving at correct relationships, have much in common. These are presented as a series of 13 steps in Table 3.2. A graphical outcome from the Table 3.2 procedure is presented in Figure 3.7.

RUNOFF EVENT TYPE WITH BYPASS	LOSS MODE : BYPASS AND/OR OVERFLOW	RUNOFF EVENT TYPE WITHOUT BYPASS	LOSS MODE : BYPASS AND/OR OVERFLOW
 High flow rate Low runoff volume	BYPASS and NO OVERFLOW*	 Low flow rate Low runoff volume	NO BYPASS and NO OVERFLOW
 High flow rate High runoff volume	BYPASS and OVERFLOW **	 Low flow rate High runoff volume	NO BYPASS and OVERFLOW**

* Retention storage antecedent condition may cause overflow to occur.

** Retention storage antecedent condition will affect overflow characteristics

Figure 3.6: Illustration of modes of operation of some on-site stormwater retention systems

3.4.3 Application of Figure 3.7 : the “soakaway” and ‘dry’ pond cases

Information from Figure 3.7 can be used directly to determine, for any small catchment in Adelaide described in terms of its equivalent impervious area and known site soil permeability, the dimensions of “soakaway” (unobstructed internal space) or ‘dry’ pond required to achieve a target retention efficiency within the range 30% – 90%. This information is part, only, of the full interpretation which can be made of the 13-step procedure (Table 3.2) described above relating to “soakaways”.

For example, it is possible, using Figure 3.7 and a simple multiplier, to dimension other retention device types, such as gravel-filled or ‘milk crate’ or combination gravel/pipe “soakaways” (see Figures 1.3 and 1.4b) receiving storm runoff from urban catchments located in the Adelaide climate zone. Of course the procedure is not limited to Adelaide climate conditions but can be applied using appropriate continuous rainfall data to any location.

The particular advantage of the procedure is its independence from the assumptions and potential errors inherent in the design storm method, as reviewed in Section 3.4.1. Its disadvantages include the limited rainfall (pluviograph) data directly available to carry out ‘continuous simulation’ modelling: directly useable data are available from only 600 stations across Australia. However, this disadvantage can be overcome using the DRIP program (Heneker et al, 2001), opening access to many of the 7,000 daily-read data sets available from Commonwealth Bureau of Meteorology sources. Other disadvantages of ‘continuous simulation’ modelling are the uncertainties which must surround any attempt to mathematically model catchment processes and in-ground retention installation (device) performance/behaviour, and the time and expertise required to undertake the analyses for individual cases.

For these reasons, a more general and simplified approach to the dimensioning of “soakaways” and ‘dry’ ponds is offered in the Handbook for locations in the Northern and Southern climate zones of Australia, and for three climate ‘bands’ covering Intermediate Australia. These (five) groupings are based on $i_{10,1}$ similarity. [$i_{10,1}$ is the ARI, $Y = 10$ years, 1-hour rainfall average intensity used in the AR&R – 1987 (IEAust, 1999) linear interpolation procedure to determine values of hydrological parameters for use in locations in Intermediate Australia (see Figure 3.8 and Eqn. 3.34)]. Five graphs derived using the 13-step procedure (Table 3.2), modified for special application to “soakaways” and ‘dry’ ponds, are presented in Appendix C. They form the basis for Procedure 5A in Section 7.2.

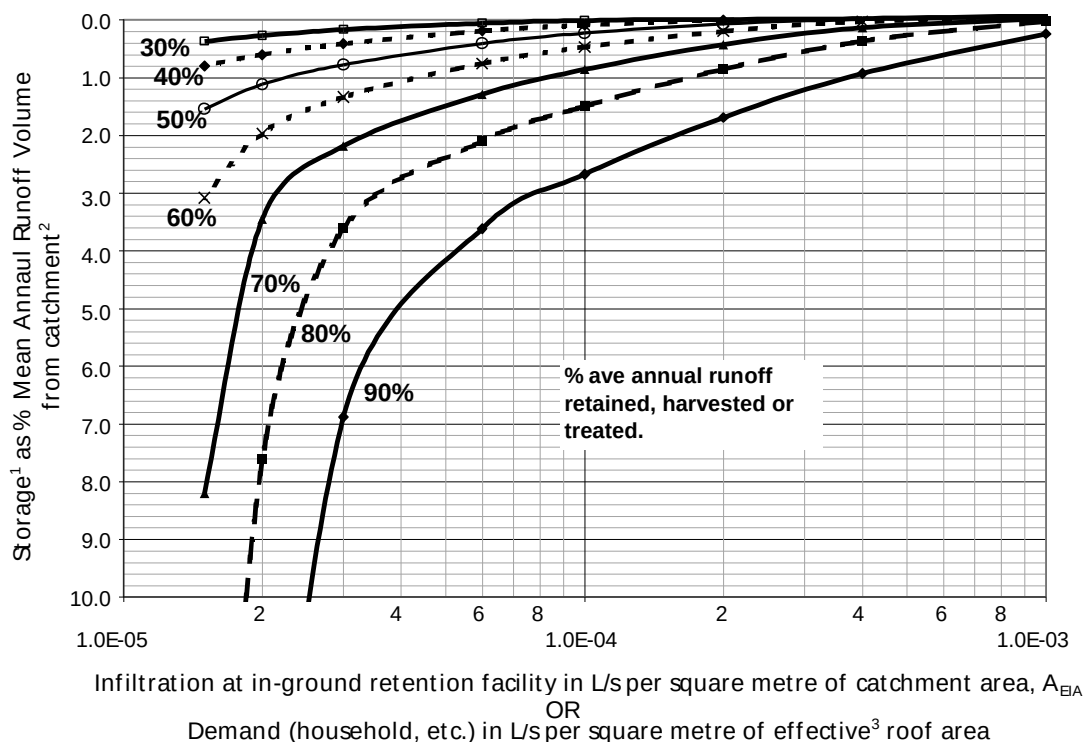
TABLE 3.2
BASIC STEPS IN TWO STORMWATER RETENTION CASE SCENARIOS

VARIABLES		
“LEAKY” DEVICES	RAINWATER TANKS	
A_{EIA} : catchment equivalent impervious area, m^2 H : device depth, m A_D : device plan area, m^2 R : volumetric retention efficiency, % k_h : hydraulic conductivity of host soil, m/s q : bypass in L/s per m^2 of catchment (A_{EIA}) q_{lim} : limit of bypass, q , for each R (see STEP 10) Q_{lim} : ($q_{lim} \times A_{EIA}$) for each value of R e_s : device void space ratio – 0.35 for gravel, etc. Q_I : cumulative volume of runoff which inflows to device/tank in Y years Q_B : cumulative volume of runoff which bypasses device/tank in Y years Q_O : cumulative volume of runoff which overflows device/tank in Y years $Q_{B/O}$: cumulative volume of runoff which bypasses or overflows device/tank in Y years $Q_{ret}\%$: percentage (total) of runoff which is retained by device or tank	A : roof area, m^2 V : rainwater tank volume, m^3 R : volumetric retention efficiency, % D : demand, L per household per day q_{20} : gutter/downpipes capacity flow expressed as L/s per m^2 of area A .	
“LEAKY” DEVICES	STEPS IN THE PROCESS	RAINWATER TANKS
Base mathematical model on equivalent impervious area, A_{EIA} : recommend $A_{EIA} = 1,000 m^2$	STEP 1 : Establish mathematical model of the catchment rainfall/runoff process.	Base mathematical model on roof area, A : recommend $A = 50 m^2$.
Nominate depth, H , and trial value of device plan area, A_D ; adopt a target value of R . Outflow in this case, is represented by percolation equal to $A_D \times k_h$, where k_h is a value selected within the soil range.	STEP 2 : Establish mathematical model of retention device for nominated dimensions and conditions of operation, in particular, outflow . Also nominate a target value for R .	Nominate volume, V , of rainwater tank; adopt a target value of R . Outflow in this case, is trial daily demand, D , made on water stored in the tank (50 L per day).
Bypass flow, q L/s per m^2 of area, A_{EIA} is assigned; all flow greater than ($q \times A_{EIA}$) bypasses. All flow reaching the retention device and exceeding its capacity, given by ($e_s \times A_D \times H$), overflows .	STEP 3 : Nominate a bypass condition applied upstream of the retention device and, also, an overflow condition.	Bypass , ($q_{20} \times A$) is determined by the capacity flow of the roof guttering/downpipes, typically, ARI = 20 years. All flow stored at any time in the rainwater tank above spill level – fixed by trial tank volume, V – overflows .
This step is applied to area, $A_{EIA} = 1,000 m^2$. ←	STEP 4 : Apply rainfall data (N -years record). Determine the cumulative (total) volume of runoff from the catchment over the N -years period, Q_I .	⇒ This step is applied to roof area, $A = 50 m^2$.
←	STEP 5 : Determine, for the values of parameters nominated in STEP 2 and conditions applied in STEP 3, the cumulative (total) volume of runoff which bypasses (Q_B) or overflows (Q_O) the retention device (in Y -years). Call this $Q_{B/O}$.	⇒
Increase trial value of Retention device plan area, A_D . Repeat STEPS 4, 5 and 6. ←	STEP 6 : Calculate the Retention percentage $Q_{ret}\% = 100 [(Q_I - Q_{B/O})/Q_I]\%$ and compare with target of R . If $Q_{ret}\%$ is less than R , return to STEP 2 and adopt larger value for the appropriate parameter. Repeat STEPS 4, 5 and 6 until $Q_{ret}\% = R$.	⇒ Increase trial value of daily demand, D . Repeat STEPS 4, 5 and 6.
Continued....		

TABLE 3.2 (continued)

“LEAKY” DEVICES	STEPS IN THE PROCESS	RAINWATER TANKS
Device depth, H (STEP 2); Soil type, k_h (STEP 2); Device plan area, A_D (STEP 6). Hence area ratio, $A_R = A_D/A_{EIA}$.	STEP 7 : Report adopted parameter values, condition, etc., following successful completion of STEP 6.	Tank volume, V (STEP 2); Demand, D_A (STEP 6).
Range of H : H = 0.30 m, 0.50 m, 0.75 m, 1.00 m, 1.50 m. Range of soils : $k_h = 1.0 \times 10^{-7}$ m/s; 1.0×10^{-6} m/s; 1.0×10^{-5} m/s; 1.0×10^{-4} m/s; 1.0×10^{-3} m/s. A value for area ratio A_R emerges for each (H, k_h) pair.	STEP 8 : Repeat STEPS 2 to 7 inclusive using the full range of installations of interest. Use the same bypass and overflow conditions set in STEP 3. Output from this step is a particular value for the ‘trial’ parameter nominated in STEP 2.	Range of tank sizes, V : 2.0 kL; 3.0 kL; 4.0 kL; 10.0 kL. A value for demand, D, emerges for each tank size.
Repeat STEPS 2 to 8, inclusive, with greater value of bypass capacity flow, q, set in STEP 3.	STEP 9 : Report parameter values, conditions, etc., from successful completion of STEP 8, for the target value of R set in STEP 2. [STEP 9A : It may be necessary to repeat STEPS 2 to 8 inclusive, for a different bypass condition from that set in STEP 3.]	Report final values (for retention efficiency, R) covering tank volumes 2 kL to 10 kL inclusive, hence demand, D – one value for each tank volume. Demand : 50, 60, 70, etc., L/day. Note : Bypass flow, q_{20} , used in the raintank analysis, is fixed for all cases; STEP 9A is therefore unnecessary.
Report final values covering ranges : H = 0.30 m to 1.50 m, incl.; $k_h = 1.0 \times 10^{-7}$ m/s to 1.0×10^{-3} m/s, incl.; q = 0.002 L/s etc., per $m^2 A_{EIA}$. It is discovered that different values of bypass flow, q, lead to different relationships between H and k_h for a given value of R. However, a limiting value, q_{lim} , can be identified : the relationships between depth, H, and k_h (permeability) associated with this limiting value are adopted as those which are unique to R.	STEP 10 : Report parameter values, conditions, etc., emerging from successful completion of STEP 9A.	N/A
Adopt R values 90%, 80%, 70%, 60%, 50%, 40%, 30%.	STEP 11 : Adopt a new target value for R in STEP 2 and repeat STEPS 2 to 10 inclusive, as appropriate.	Adopt R values 90%, 80%, 70%, 60%, 50%, 40%, 30%.
N/A	STEP 12 : Adopt a new value for roof area, A, in the raintank analysis; repeat STEPS 1 to 11 inclusive.	Adopt A = 100 m^2 , 150 m^2 , 200 m^2 , etc.
Report final values : H = 0.30 m to 1.50 m, incl.; $k_h = 1.0 \times 10^{-7}$ m/s to 1.0×10^{-3} m/s, inclusive; and q_{lim} ; R = 90%, 80%, 70% etc.	STEP 13 : Report parameter values, conditions, etc., emerging from successful completion of STEPS 11 or 12, as appropriate.	Report final values : R tanks : 2.00 to 10.0 kL, incl. Demand : 50, 60, 70 etc., L/day. Roof areas : 50, 100, 150 etc., m^2 . R = 90%, 80%, 70% etc.

Outcomes from STEP 13 – for both “leaky” devices and rainwater tanks – can be combined and presented on a **single** graph for the location to which the N-years record, employed in STEP 4, applies. The graph for Adelaide, South Australia, is presented in Figure 3.7.

**NOTES :**

- 1 : **Device** may be an unobstructed (internal volume) “soakaway”, a pond or a rainwater tank.
- 2 : **Catchment** may be a ground-level paved area or a roof.
- 3 : Effective roof area for rainwater tanks may be as low as 80% of nominal roof area (see Section 8.3.3).

FIGURE 3.7 : Relationship between infiltration, demand and retention storage – general case (Adelaide)

In-ground installations dimensioned according to the Table 3.2 procedure (and graphs similar to Figure 3.7) incorporate an outflow process which is entirely natural, namely percolation. Situations frequently arise in such systems where site or other requirements or constraints cannot be accommodated. In these circumstances it may be necessary to assist the emptying process by providing (additional) **slow-drainage** of stored water by systems similar to those illustrated in Figure 1.4b. Such provision is, in essence, the basis of on-site detention (OSD) practice, familiar to Australian urban drainage practitioners. Application of this technology to infiltration devices, however, differs significantly from OSD in the length of time during which the drainage process takes place. It is normal practice for detention installations to empty in periods of two to four hours; slow-drainage is calculated to take place over periods of 12 hours or longer.

Slow-drainage should only be considered after the preliminary design for an installation, dimensioned as explained in Section 7.2, is not practicable (because of site or other difficulties, referred to above). Details of how this aspect is managed in the design of pollution control retention installations are explained in Section 7.3.

3.4.4 Application of Figure 3.7 : the rainwater tank cases

Information from Figure 3.7 can also be used to determine, for any roof area in Adelaide and in-house daily (water) demand, the required **rainwater tank volume** needed to achieve a target retention efficiency within the range 60% – 90%. Of course the procedure is not limited to Adelaide climate conditions but can be employed using appropriate ‘continuous’ rainfall data applied to any location.

In a way which parallels the procedures outlined above (Section 3.4.3) for “soakaways” and ‘dry’ ponds, a more general and simplified approach is also offered in the Handbook for rainwater tank sizing. This is realised, also, for five climate zones or ‘bands’ in Australia and presented in Appendix E. They form the basis for Procedure 10 in Chapter 8.

3.4.5 Catchment-wide modelling and catchment performance

The task of identifying a critical flow peak and hence ‘stage’ condition (for a nominated ARI) at a particular location in an urban catchment – called a “point of singular interest” in the present discussion (see Section 4.2) – has been determined by practitioners in the past using a suite of design storm hyetographs applied to the catchment by way of an appropriate rainfall/runoff model. The outcome was a corresponding suite of hydrographs : the peak of the envelope curve enclosing the set of hydrograph maximum values gave the required peak flow (and hence ‘stage’), as well as **duration of the storm that was critical in the contributing catchment** (IEAust, 1999, Section 9.2.4).

This method, based on isolated, hypothetical ‘design’ storms, takes no account of the consequences of interaction between successive (real) storms. However, it has served more than a generation of urban drainage designers reasonably well, because its basic assumption – that there is no significant interaction between runoff generated in successive storms (because of rapid storm runoff wave movement in conventional urban catchments) – is reasonably true.

So what lies ahead for Australian practice if it is to take the ‘continuous simulation’ modelling path, only, from the ‘crossroads’ identified by Kuczera et al (2003)? First, a wealth of research and analysis development must occur before a procedure emerges which can truly take the place currently occupied by the design storm approach referred to above. To quote Hardy, Coombes and Kuczera (2004) : “These questions serve to motivate strongly the need for new modelling tools capable of multi-scale investigations in a continuous simulation framework. It will only be with the development of these new tools that more definitive and informative investigations of these and other questions can take place”. The PURRS program of Coombes and Kuczera (2001) is, undoubtedly, a major contribution towards this goal but more progress is needed before the goal is finally reached.

When it is (reached), its application into any typical municipal situation is likely to be complex and costly to produce and well beyond the up-front financial investment in design that a developer could – or should – be reasonably expected to bear in typical development circumstances. Argue and Pezzaniti (2004) predict that such ‘tailor-made’ (catchment-wide) packages, wherever they are developed, will be held by local government agencies, only, and that their use in the process of designing competent on-site works associated with development/re-development projects will take place as a collaborative exercise involving Council-owned software and consultants using the software on behalf of developers.

Although the advent of such models may put to rest the disquiet now harboured concerning the validity of the design storm method in the light of the earlier criticisms (Section 3.4.1), their widespread use – should that become a reality – is unlikely to yield any dramatically different outcomes when applied to general urban landscape flood study assignments requiring 20-years, 50-years, 100-years, etc., flood contours. The justification for this prediction is the relatively minor role played, typically, by non-channel storage in *normal* urban landscapes.

All of these considerations – currently (2004) the source of much professional discussion – would be matters of no great moment in a Handbook on basic stormwater ‘source control’ measures were it not for the fact that conversion of Australian practice exclusively into the domain of ‘continuous simulation’ modelling would involve the loss of a very useful catchment property, namely, catchment **critical storm duration** (see Section 4.2). This concept has no place in ‘continuous’ modelling but is a fundamental component of the design storm method, a *modified* form of which is used in some later procedures in this Handbook (see Section 5.1).

It need hardly be said that contemporary practice should be based on wise use of all available practices and technologies, recognising their advantages and their shortcomings. With this in mind, the following long-term approach to resolving the issues raised in the foregoing discussion is offered and represents, we believe, an acceptable solution leading to an urban drainage design practice which is both credible and easy for practitioners to apply.

STEP 1 : Use ‘continuous simulation’ modelling with a comprehensive rainfall/runoff model of the developing urban catchment of interest, as it is or was (the ‘benchmark year’ concept, Sections 4.2.1 and 4.6.3), together with streamflow frequency analysis, to determine the Q_{peak} versus frequency relationship and, hence, the target peak flow (and hence ‘peak stage’) condition at a “point of singular interest” (see Section 4.2.1) for a selected ARI = Y-years. Representatives of

all stakeholders in the catchment should reach consensus on these matters involving a fair balance between flood security, heritage, economic, environmental, etc. interests;

STEP 2 : Use the design storm approach [suite of design storm hyetographs, as in AR&R – 1987 (IEAust, 1999)] applied to the same rainfall/runoff model used in STEP 1, above, to determine the catchment **critical storm duration** for the same nominated ARI.

STEP 3A : Determine the dimensions of on-site flood control storages (detention or retention) by application of ‘continuous’ rainfall data to the comprehensive runoff model (developed and used in STEP 1) taking the sites to be developed or re-developed one at a time and using an iterative approach (‘trial and error’); successful on-site design for appropriate development/re-development, gives the same Q_{peak} *versus* frequency relationship for the catchment determined in STEP 1.

OR

STEP 3B : Base the design of on-site flood control (detention or retention) storages on use of the *modified* design storm method described below.

It is unlikely that the peak flows produced in STEPS 1 and 2 will be the same: that produced in STEP 1 should dominate. However, the critical storm duration – absent from STEP 1 – is an acceptable and valuable outcome from STEP 2, regardless of the difference in peak flows.

3.5 THE MODIFIED DESIGN STORM METHOD

3.5.1 Introduction

Professor Kuczera’s Keynote Address (Kuczera et al, 2003) places Australian practice at the crossroads with regard to its use of the design storm method and/or ‘continuous simulation’ modelling. The complexities of the latter as well as its uncertainties, reviewed in Section 3.4, taken with the familiarity which the design storm method enjoys across all national practices – not to mention its simplicity – provide considerable incentive to explore the criticisms which have been levelled against this method of design, and attempt to resolve them. If this can be achieved, then both methods can take their rightful and valued places in Australian practice.

So what are the substantive criticisms made of the design storm method? From Section 3.4.1 :

1. “users of the method are required to make arbitrary assumptions about the state of on-site storages – typically empty or half-full – at the commencement of the design storm; and,
2. design storm temporal patterns, determined by current analytical procedures are, in fact, ‘embedded’ storm bursts : dimensions of storages determined from these (storm) profiles are therefore likely to be undersized.”

Consideration and resolution of the **first** criticism is offered in the remainder of the present section; the **second** is addressed in Section 3.7.2.

The essence of the *first* criticism stems from uncertainty experienced by the designer concerning the initial volumetric status of any significant storage in an urban catchment following (unknown) previous rainfall – full?, empty?, part-full? etc. – as he/she commences to apply the appropriate design storm to the modelled catchment. [The separate issues of ‘shape’ and magnitude of the (design storm) hyetograph are aspects of the *second* criticism.]

However, this criticism **collapses** if the designer can be reasonably certain that the storage in question is, *in fact*, completely empty : this implies acceptance of some risk that it may **not** be the case. It is important to note that the discussion, following, applies to all types of retention storages *other than pollution control storages and rainwater tanks*. Some form of ‘continuous simulation’ modelling is mandatory in both of these cases (see Chapters 7 and 8).

The principal issue then becomes : what mechanisms operate naturally or can be introduced by design to retention storage devices and installations to ensure that they have a high likelihood of being empty prior to commencement of the (theoretical) design storm? Two candidates are suggested :

- natural water removal from storage installations by way of infiltration and/or percolation; and,
- ‘hydraulic’ means of abstraction such as bores transferring stored water to aquifers, or slow-drainage pipes releasing water into a nearby waterway or formal drainage path.

Both of these alternatives lead on to the concept of **emptying time** (‘ponding time’ or ‘drain time’ are employed in US terminology) : how long will it take for a retention device of given type and dimensions to empty from full under natural or ‘hydraulic’ emptying conditions? Theory associated with ‘natural’ emptying of typical systems is presented in Section 3.5.2; issues relating to aquifer access or slow-drainage provision (‘hydraulic’ emptying from storage) are explained in Section 3.5.4.

3.5.2 Emptying time – ‘natural’ drainage

The theory which leads to the basic emptying time formulae for typical retention devices is illustrated for the case of a “leaky” well, diameter D , height H :

$$\text{wetted area} = \pi D h + \frac{\pi D^2}{4} \quad (3.16)$$

Seepage outflow, Q , from the well when water depth equals “ h ”, assuming the fully drained condition is :

$$Q = k_h \left(\pi D h + \frac{\pi D^2}{4} \right) \quad (3.17)$$

Let water level drop Δh in time Δt :

then, seepage outflow in time Δt :

$$dV = Q \cdot dt = k_h \left(\pi D h + \frac{\pi D^2}{4} \right) \cdot dt \quad (3.18)$$

$$\text{also,} \quad dV = -\frac{\pi D^2}{4} \cdot dh \quad (3.19)$$

$$\text{Hence,} \quad k_h \left(\pi D h + \frac{\pi D^2}{4} \right) \cdot dt = -\frac{\pi D^2}{4} \cdot dh \quad (3.20)$$

$$\text{hence,} \quad k_h \left(h + \frac{D}{4} \right) \cdot dt = -\frac{D}{4} \cdot dh \quad (3.21)$$

$$\text{i.e.} \quad dt = \frac{-\frac{D}{4}}{k_h \left(h + \frac{D}{4} \right)} \cdot dh \quad (3.22)$$

$$\text{Integrating :} \quad \int_0^t dt = -\frac{D}{4k_h} \int_H^h \frac{dh}{\left(h + \frac{D}{4} \right)} \quad (3.23)$$

where H equals well-full depth.

$$\text{i.e.} \quad t = -\frac{D}{4k_h} \left[\log_e \left(h + \frac{D}{4} \right) \right]_H^h \quad (3.24)$$

hence, time for well depth to fall from H to h :

$$t_{H \rightarrow h} = -\frac{D}{4k_h} \left[\log_e \left(h + \frac{D}{4} \right) - \log_e \left(H + \frac{D}{4} \right) \right] \quad (3.25)$$

$$\text{i.e.} \quad t_{H \rightarrow h} = -\frac{D}{4k_h} \log_e \left[\frac{h + \frac{D}{4}}{H + \frac{D}{4}} \right] = -\frac{2.3D}{4k_h} \log_{10} \left[\frac{h + \frac{D}{4}}{H + \frac{D}{4}} \right] \quad (3.26)$$

hence, time for well to **empty**, i.e. $h = 0$:

$$T = -\frac{2.3D}{4k_h} \log_{10} \left[\frac{\frac{D}{4}}{H + \frac{D}{4}} \right], s \quad (3.27)$$

A corresponding expression may be derived for emptying time in a gravel-filled trench – it is :

$$T = -\frac{2.3Lb.e_s}{2k_h(L+b)} \log_{10} \left[\frac{Lb}{Lb + 2H(L+b)} \right], s \quad (3.28)$$

where L = trench length; b = trench width; H = trench depth; e_s = void space ratio $\left[\frac{\text{volume of voids}}{\text{total volume occupied}} \right]$. In “soakaways” or shallow trenches having large plan area, Eqn. (3.28) simplifies to :

$$T \approx \frac{H.e_s}{k_h}, s \quad (3.29)$$

Field measurement of infiltration devices in Adelaide, together with the observations of Lee and Taylor (1998), suggests that the latter three formulae **underestimate** emptying time, T , by a factor of about two. Hence, recommended formulae for emptying time are :

- “leaky” wells :
$$T = -\frac{4.6D}{4k_h} \log_{10} \left[\frac{\frac{D}{4}}{H + \frac{D}{4}} \right], s \quad (3.30)$$

- gravel-filled (or similar) trench :
$$T = -\frac{4.6Lb.e_s}{2k_h(L+b)} \log_{10} \left[\frac{Lb}{Lb + 2H(L+b)} \right], s \quad (3.31)$$

- “soakaways” :
$$T \approx \frac{2H.e_s}{k_h}, s \quad (3.32)$$

The latter formula can also be used for open water recession cases for example ‘dry’ ponds, by setting e_s , void space ratio, equal to 1.0.

3.5.3 Emptying time criteria – analyses

The emptying time concept is virtually unknown in current Australian practice (IEAust, 1999) but is a component of other practices. Not surprisingly, emptying time criteria, where they are used, vary. UK uses “50% empty 24 hours after cessation of rainfall” (Bettess, 1996); US practice, generally, uses “completely empty, 72 hours after cessation of rainfall”; Auckland City Council, New Zealand, has, possibly, the most conservative criterion – “completely empty, 24 hours after cessation of rainfall”.

The essence of statistical hydrology which can be brought to bear on the problem of emptying time recognises, firstly, that the **minimum** time gap which might be expected between two unconnected, random, meteorological events, varies inversely as the frequency of the events. Thus, it is hardly remarkable if two, unrelated, ARI, $Y = 1$ -year storm events occur at a particular location on consecutive days, but it would be very unexpected for two (unrelated) ARI, $Y = 100$ -years stormbursts to do so. Clearly, there must be some relationship (unknown) linking ARI and the minimum time likely between unconnected random events having the same average frequency of occurrence.

But historical records of stormbursts **do** show, not only ARI, $Y = 1$ -year (magnitude) rainfall events occurring on consecutive days, but also very rare occasions when ARI, $Y = 20$ -years, 50-years or even 100-years events show similar, apparent behaviour. However there is an important difference: the consecutive “1-year” events may be regarded as reasonably unconnected because they are relatively small; the likelihood of two “100-years” events occurring on consecutive days *and being unconnected* is remote. The phenomenon of their occurrence, when it does occur, is explained as two (internal) storm peaks on consecutive days as parts of a single storm having significantly greater magnitude than that of the “100-years” event, say, the once in 500-years or 1,000-years storm, and **not** two, independent occurrences.

The best way to produce a credible set of emptying time criteria for use with the design storm procedure is to apply ‘continuous simulation’ modelling to a range of catchments, each discharging to a retention (or detention) facility matched to the contributing catchment area and its physical characteristics, and taking account of the rainfall experienced at that catchment location. The link between size of installation (retention/detention facility), frequency of overflows (hence ARI) and emptying flow rate can be established: these data can then be interpreted into a relationship between ARI and emptying time, T . This process has not been carried out at the time of producing this Edition of the Handbook. However, the following interim relationship (Table 3.3), based on presently employed criteria is recommended for use pending the outcome of the study referred to above.

TABLE 3.3
INTERIM RELATIONSHIP BETWEEN ARI AND ‘EMPTYING TIME’

Ave Recurr. Interval (ARI), Y-years	1-year or less	2-years	5-years	10-years	20-years	50-years	100-years
Emptying time, T in days	0.5	1.0	1.5	2.0	2.5	3.0	3.5

3.5.4 The *modified* design storm method

The essence of the procedure offered as the *modified* design storm method has two basic elements :

- design of the required storage, volume ∇_d , by any conventional design storm method or by one of the (design storm) procedures presented in the Handbook; followed by,
- a check on emptying time, T , from the storage to ensure that it meets the appropriate criterion listed in Table 3.3 for the assigned ARI.

In simple retention systems (see Figure 1.3), such as “soakaways” and ‘dry’ ponds draining naturally – that is by infiltration or percolation – the check involves application of one of the formulae Eqns. (3.30), (3.31) or (3.32). Not all cases succeed and provision is made in the Handbook for this outcome : Procedure 4 (Section 5.1.5) enables the dimensions of simple devices and “soakaways” to be determined taking account of ‘hydraulic’ assistance (bore recharge or slow-drainage by pipeline) needed to meet the appropriate Table 3.3 criterion. [‘Slow-drainage’, the term used in the Handbook, is referred to as **extended detention** in US literature (US Dept. of Transportation, 1996)].

The Handbook advises users to design ‘dry’ ponds employing ‘natural’ emptying only and to avoid introducing ‘hydraulic’ assistance directly to these systems (see Section 5.1.4).

More detail on these matters is provided in the Handbook : the following notes are intended as an introduction only.

‘Natural’ drainage systems (emptying by infiltration or percolation) :

- STEP 1 :** design the retention storage device or installation, volume ∇_d , using the normal design storm approach for required ARI, critical storm duration, etc. (IEAust, 1999) or by Procedures 2 or 3 from the Handbook (see Chapter 5). Note that ∇_d is the volume of stored stormwater;
- STEP 2 :** check to ensure that outflow by ‘natural’ means will result in emptying from full in a time, T , approximating that given – for the appropriate design ARI – in Table 3.3. This will normally involve use of one of the formulae (3.30) or (3.31) or (3.32).

In the event of **failure** to meet the Table 3.3 criterion, then the designer may explore either or both of the options outlined below :

Sites where aquifer access is available :

- STEP 3A :** check that all required conditions in terms of recharge water quality, environmental impact and costs can be met, then conduct an exploration to determine (single) bore recharge capacity, q_r , at the site;

STEP 4A : design the retention storage device or installation according to Procedure 4A in the Handbook (Section 5.1.5) using one bore;

STEP 5A : calculate emptying time, T , [Eqn. (3.33), below]. The outcome must then be compared with the value given in Table 3.3 for the appropriate ARI. If the indicated emptying time is **exceeded**, then two, three or more bores should be considered [revisit Eqn. (3.33)] until a satisfactory result emerges.

Sites where aquifer access is NOT available : slow-drainage should be explored :

In field cases where ‘natural’ means of storage-emptying are insufficient to meet the criteria listed in Table 3.3 and aquifer access is denied, then recourse must be made to slow-drainage as the last available option other than emptying by pumping. This involves, simply, providing a pipeline from the storage to dispose of the retained water into a local waterway or formal drainage path.

STEP 3B : taking the value of “ T ” (converted to seconds) given for the appropriate ARI in Table 3.3, and the device (stored water) volume, V_d , calculated previously (see STEP 1, above), determine slow-drainage pipeline flow, Q_r :

$$Q_r = V_d/T \quad (3.33)$$

STEP 4B : design the retention storage device or installation according to Procedure 4B in the Handbook (Section 5.1.5) using Q_r as the release flow rate carried by the slow-drainage pipeline, but taking account of the conditions contained in the Special Note, following.

Special Note (slow-drainage case) :

A typical result for “ Q_r ” in STEP 2, above, is a flow less than 1.0 L/s, requiring a pipeline of extremely small diameter. The normal engineering design response to this result is to specify a pipe of some 90 mm size (unthrottled) on the ground that the target outflow rate will be easily achieved by such a conduit.

Certainly, with a 90 mm (unthrottled) pipe in place, the outflow, typically, will be considerably greater than required for satisfactory emptying, and the storage device or installation is likely to empty in much less time than set out in Table 3.3. In fact, the outflow rate, under these circumstances, may approach that of an uncontrolled site, defeating, totally, the purpose of having a retention storage. The criteria set out in Table 3.3 should therefore be interpreted – **in slow-drainage cases** – as target durations which should not be exceeded but which should, also, be **not significantly under-achieved**. The practical difficulty involved in balancing these seemingly conflicting demands is reviewed in Section 5.4 and a practical solution illustrated in Figure 5.6.

3.5.5 Catchment performance and the *modified* design storm method

The previous section on ‘continuous simulation’ modelling concluded with a discussion of the future direction that urban storm drainage design might take when analytical procedures have been developed to cope with the difficulty presently posed by catchment-wide modelling using the ‘simulation’ approach (Section 3.4.5). It offered a 3-step procedure, grounded in catchment-wide ‘continuous simulation’ modelling, but interpreted finally – at the scale of the individual site – in application of the complex catchment-wide ‘continuous simulation’ model (STEP 3A) **OR** the *modified* design storm method (STEP 3B). It remains for the latter approach to the design/dimensioning of on-site storages to be explained in the context of catchment-wide flood control objectives.

Consider the physical environment of a partly developed urban catchment which is assessed by consensus among all stakeholders to be operating **satisfactorily**. This implies that storm runoff in the Y -years ARI event (Y -years agreed by all interested parties) is accommodated by the existing drainage infrastructure with an *acceptable* level of inconvenience and flooding. This does not, typically, imply a “flood free” condition – although that could be the case if such a level of (flood) protection were to be demanded by stakeholders – but, more likely, acceptance of a trade-off between the financial burden of very high quality flood security and a modest level of community discomfort on rare occasions.

It follows (in this situation) that the surface runoff *volume* which passes from each component of the urban landscape to the existing drainage path **in the storm of critical duration** creates no unacceptable level of

nuisance in the community. Thus, the goal of flood management by the local government agency within whose jurisdiction the partly urbanised catchment lies should be to preserve this status indefinitely through application of appropriate policy instruments. In such a regulatory environment, it is possible for a high level of development and re-development to occur without overloading the existing drainage infrastructure : a vital condition of this scenario is competent maintenance – not upgrade – of the drainage infrastructure. The vision of “a high level of development/re-development” may, of course, be constrained by other (non-flood oriented) goals of stakeholders relating to issues of amenity, heritage, habitat, environmental values, biodiversity, etc.

Design of on-site works within the context described here becomes, then, simply a matter of ensuring that the ‘before and after’ runoff *volume* state of each component of development/re-development **in the storm of critical duration** remains the same. The various forms of on-site retention, including harvesting/re-use of the collected, cleansed stormwater (rainwater tanks, etc.) as well as opportunities for aquifer recharge and maintaining soil moisture, provide the means whereby this design objective can be achieved. The approach to design underpinning this framework for managed urban change is the *modified* design storm method.

What risk does the designer face in taking the path outlined above?

The design storm has characteristics determined through the process described in Section 3.4.5. If it were to occur on the catchment of interest, and **all on-site (designed) storages were empty** at its commencement, then equality of ‘before and after’ development/re-development runoff *volumes* would result and the overall system would perform as planned. However, if the storages were **not** empty at the commencement of the event, as consequences of the two (numbered) issues raised in Section 3.4.1, then runoff flows greater than those of the ‘before’ condition would result and the drainage system would undoubtedly overload. Adherence to the emptying time criteria set out in Table 3.3 offers the designer the best available security against this occurring.

The design storm is certainly a useful tool for drainage designers to use, but the likelihood of it ever occurring in a practical catchment case is remote, so the above scenario, in fact, is unrealistic. Critical flood conditions do occur – those that the stakeholders (see above) have identified as Q_{peak} (or peak ‘stage’) together with its associated ARI – but they result, typically, not from a design storm but from close sequences of small storms draining to storages which are not empty. Only correct ‘continuous simulation’ modelling is able to reveal these critical conditions.

This interplay between the *modified* design storm and ‘continuous simulation’ approaches is vital to a simple but credible urban drainage design practice. It may best be described as a design storm method calibrated to give the ‘continuous simulation’ outcome. To date (2004) only the “Heritage Mews” project in Western Sydney has been designed by both methods (see Section 3.9). The outcome showed the *modified* design storm method to require 7% greater storages than given by ‘continuous simulation’ modelling (Argue et al, 2003; Coombes et al, 2003).

3.6 RUNOFF COEFFICIENTS

Typical urban catchments in Australia have undergone a sequence of development/re-development phases in which primeval forests have been cleared, firstly, to make way for agricultural activity – crops or grazing. At a later stage, large-area farms located on the edges of urban concentrations were divided into small rural holdings of 5 – 20 acres (2 – 8 ha) apiece; these, in turn, were divided – typically – into quarter-acre allotment housing sub-divisions in the earliest phase of “greenfields” development, or given over to industrial estates.

The most recent stage of development/re-development which continues to take place following major planning changes on the part of State and Commonwealth governments in Australia to combat ‘urban sprawl’ (Dept. of Housing and Regional Development, 1995), is :

- “greenfields” developments in urban fringe areas with (housing) allotment sizes in the range 300 m² – 600 m²;
- “greenfields” industrial estates in urban fringe areas; and,
- urban infill involving re-development of old housing stock – typically quarter-acre blocks – as sites for two, sometimes three, residential units or townhouses.

The effect of the urban evolution, outlined above on the **structure** of upper-level soils in the regions of development/re-development has been profound, in particular, progressive loss of natural seepage paths along root systems, worm holes and animal burrows. The consequence is a very different response to rainfall: runoff coefficients for urban soils are greater than values for forested catchments. In Southern Australia (see Figure 3.8) the difference is not great, as Table 3.4 shows: but in Northern Australia the corresponding figure (runoff coefficient for pervious areas) is seven times its Southern Australian counterpart.

The explanation for this is not significantly different soils, but, rather, significantly different climates in which the critical conditions for flooding take place. Most **urban** flooding in Australia occurs as the consequence of cyclonic activity characterised by high intensity rainfall of relatively short duration in summer, typically December to March, inclusive. In Southern Australia, this occurs when the soil is in a hot/dry state resulting in low runoff rates; in Northern Australia, the cyclonic rain falls on catchments which may have experienced prolonged hot and wet conditions. The consequences are reflected in the basic information from “Australian Rainfall and Runoff” (IEAust, 1999) presented in Table 3.4.

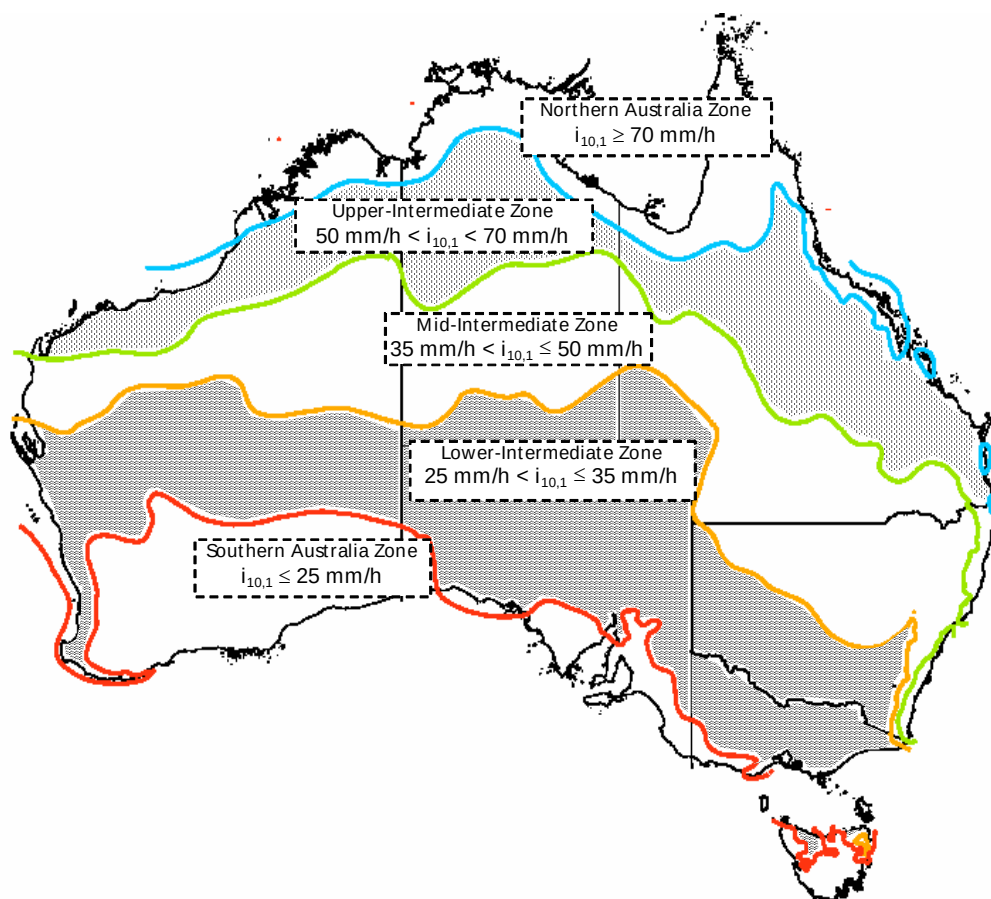


FIGURE 3.8 : Australian climate zones based on 10-year, 1-hour rainfall
Courtesy Bureau of Meteorology, Melbourne Office, gratefully acknowledged.

TABLE 3.4
BASIC RUNOFF COEFFICIENTS (C_{10})* FOR URBAN CATCHMENT SURFACES

SURFACE CLASSIFICATION	NORTHERN AUSTRALIA ZONE	SOUTHERN AUSTRALIA ZONE
Connected paved areas : • roadways and roofs	$C_{10} = 0.90$	$C_{10} = 0.90$
Unconnected paved areas and pervious areas : • mixed with paved areas as in residential land use; • major urban open space areas, parks, etc.	$C_{10} = 0.70$	$C_{10} = 0.10$

* C_{10} is the runoff coefficient for ARI, Y = 10-years storm conditions.

The explanation, above, for the sequence of development stages through which the urban landscape has evolved in the great bulk of cases, and the effect this has had on the values for pervious area runoff coefficient listed in Table 3.4, are relatively undisputed. What is less certain are the values of C_{10} which should be used in situations where the process is short-circuited and forest catchments are converted directly to urban development – typically residential in character – without passage through the agricultural phase. [Reference to Table 3.4 should, undoubtedly, be made for values of C_{10} to be applied to pervious areas integrated *within* such developments, despite their immediate pre-history. The reason for this is the totally destructive effects which land development processes have on soil structure in such circumstances.]

However, the interpretation of WSUD into such a scenario – in keeping with the principle explained in Section 1.4.2 – requires the designer to recognise the pre-development pervious area not as Table 3.4 presents it but, rather, as a **forest land use**. [AR&R – 1987 (IEAust, 1999) is silent on this issue.] This would see use of $C_{10} \cong 0.10$ for all (forest) catchment development cases, regardless of location in Australia. This value of runoff coefficient (for forest catchments) follows Canadian practice (Stephens et al, 2002; Gilliard, 2004).

The major Australian reference in the area of urban runoff (IEAust, 1999) provides a linear interpolation procedure based on the (location) ARI, $Y = 10$ -years, 1-hour stormburst intensity, for calculating paved and pervious area runoff coefficients for any place in Australia between the Northern and Southern zones (see Figure 3.8). Steps in the procedure to determine pervious area C_{10} for location X are as follows :

STEP 1 : Determine Northern Australia and Southern Australia values for the runoff coefficient of interest – C_N and C_S , respectively from Table 3.4;

STEP 2 : Determine the 10-year, 1-hour average rainfall intensity at location X, i.e. i_X ;

STEP 3 : Compute the required value for C_X :

$$\begin{aligned} C_X &= C_S + \frac{(i_X - 25)}{(70 - 25)}(C_N - C_S) \\ &= C_S + 0.022 (i_X - 25)(C_N - C_S) \end{aligned} \quad (3.34)$$

Example :

Find the runoff coefficient for pervious areas, design ARI = 10-years, i.e. $[C_{10}]_{\text{perv}}$ for Newcastle, N.S.W.

STEP 1 : $[C_{10}]_{\text{perv}}$ for Northern Australia = 0.70
 $[C_{10}]_{\text{perv}}$ for Southern Australia = 0.10
 (from Table 3.4)

STEP 2 : the 10-year, 1-hour, average rainfall intensity at Newcastle, $i_{(10,1), \text{N'castle}} = 49.6 \text{ mm/h}$

STEP 3 : $[C_{10}]_{\text{perv}}$ for Newcastle = $0.1 + 0.022 (49.6 - 25) (0.70 - 0.10) = 0.43$ (3.35)

One further step is required for the designer to be able to determine runoff coefficients in all practical situations. This relates to the conversion of C_{10} values into runoff coefficients for frequencies other than 10 years. The required **Frequency Conversion Factor, F_Y** , information is presented in Table 3.5 (Argue, 1986). The listed factors are applied directly to the appropriate C_{10} value(s) calculated by the procedure described above, with the over-riding constraint that the (derived) runoff coefficient should never exceed 1.0

TABLE 3.5
FREQUENCY CONVERSION FACTOR, F_Y
 (after Argue, 1986)

ARI (years)	1 and <1	2	5	10	20	40	60	80	100
Conversion Factor, F_Y	0.8	0.85	0.95	1.0	1.05	1.13	1.17	1.19	1.20

3.7 RAINFALL

3.7.1 Rainfall and the ARI, Y = 0.25-year stormburst intensity

Each of the design procedures included in the Handbook involves use of Australian rainfall information in one of its many available forms. Because of the simplicity of the procedures set out in Chapters 5 and 7, peak runoff flow can be determined, satisfactorily, from average stormburst intensity, i_{ave} , and runoff volume, V , can, similarly, be calculated using data from the basic IFD chart or table readily available from the Commonwealth Bureau of Meteorology or other sources for any location within Australia. Examples of these data sets are given in Appendix B representing Southern Australia, Intermediate Australia and Northern Australia (see Figure 3.8).

More detailed rainfall information required for ‘continuous simulation’ modelling in connection with raintank sizing (Procedure 9 in Chapter 8) calls for the use of daily rainfall records of long duration. These are also readily available, again from Bureau sources for over 7,000 stations where long-term records have been kept. No procedures included in the Handbook require users to employ small-interval, for example 6-minute or 15-minute, continuous rainfall information. Where such data bases are necessary to deliver a satisfactory outcome, the authors have undertaken the necessary modelling and produced results in forms (graphs and charts) which are directly useable by practitioners.

The only domain of rainfall information where the procedure in the Handbook departs, significantly, from current practice relates to the ARI, Y = 0.25-years rainfall (average) intensity, $i_{0.25}$, and its fraction or proportion of the ARI, Y = 1-year value. [The peak flow generated in a catchment in the ARI, Y = 0.25-years event is widely used in Australia as the basis for design of GPT installations, swales and stormwater quality improvement wetlands.] This relationship – in the interests of simplicity – needs to be ‘globalised’ and an acceptable average factor struck for universal use across the nation.

Australian practice to date has followed the lead of NSW Dept of Housing (1998) by setting $i_{0.25}$ equal to 0.25 times i_1 , the ARI, Y = 1-year rainfall (average) intensity. The factor 0.25, used in this context, is incorrect for Australia-wide use: examination of IFD data from stations across the continent shows the factor which needs to be applied to i_1 is 0.50 or an even greater value. The following simple relationship is, therefore, adopted in the Handbook and recommended for general use :

$$\text{0.25-years stormburst average intensity, } i_{0.25} = 0.50 \times i_1 \quad (3.36)$$

3.7.2 The rainfall data base : some current issues

“Embedded storms” : Recent years have seen disquiet expressed by some researchers concerning both the IFD (intensity-frequency-duration) data used in current Australian practice and, more particularly, the design temporal patterns recommended by AR&R – 1987 (IEAust, 1999). [See Rigby et al (2003).] The principal complaint centres on the claim that the analysis which has given us both data sets extracted “storms from within storms” (the so-called “embedded storms”) and that when these data are applied into the context of design, they lead to systematic *underestimates* of flows and, hence, underestimates of storages.

While the voracity of these concerns will be fully explored in the course of preparing the next edition of “Australian Rainfall and Runoff”, it is considered prudent to draw this issue to the attention of readers and, by so doing, to alert them to possible changes in practice which may ensue when these matters are resolved.

The procedures presented later in the Handbook and which employ the *modified* design storm approach, stand outside this controversy because they leave responsibility for estimating Q_{peak} or runoff volume, V , entirely in the hands of current Australian practice as set by Engineers Australia. It is recommended that practitioners take interest in the discussion outlined above, but in all matters of day-to-day design, follow the most up-to-date version of AR&R available from the professional body.

Continuous simulation : Another matter relating to rainfall which is currently under discussion in professional engineering circles, is the data base which should be used in ‘continuous simulation’ modelling. Two types of data are available :

- Long period daily rainfall records; and,
- Long period, short time increment (typically, 6 minutes, 15 minutes, etc.) continuous rainfall data.

Records of the first type are available from the Commonwealth Bureau of Meteorology for over 7,000 stations throughout the nation. These are used for sizing rainwater tanks in the spreadsheet procedure set out in Chapter 8.

The second category of data is available for some 600 Bureau stations: however it is possible to extend this data set to many of the 7,000 daily-reading stations using the DRIP program of Heneker et al (2001). Users of the Handbook are encouraged to proceed along these lines and follow the 13-step process described in Table 3.2 to obtain the required relationships for individual locations of interest. However, the authors have undertaken this task using recorded data for selected stations to produce design charts for pollution control installations (Appendix C) and rainwater storages (Appendix E) for climate groupings covering the entire Australian land mass. This is explained in Sections 7.2 and 8.6 leading to Procedures 5/6 and 10 respectively.

3.8 ON-SITE RETENTION (OSR) TECHNOLOGY : SOME DO'S AND DON'TS

1. **Unsuitable soils :** Don't put OSR devices ("leaky" wells, gravel-filled trenches, etc.) in soils which are predominantly "wind-blown" sands. This does not exclude well-compacted dune sands but loose, aeolian sands should be avoided. There are also clay (calcareous) soils which collapse when in direct contact with retained water and acid sulphate and 'sodic' soils which should also be avoided. Good practice demands that sites where 'source control' technology by retention is contemplated must be visited for permeability testing (see Section 3.2) and soil assessment to determine suitability.
2. **Clearance distances :** Don't put OSR devices closer to building footings or to boundaries than the recommended clearance distances. These are included with the soil classification information (five classes of soils) in Section 1.3.4. The consequences of ignoring this advice may be fracture of footings particularly domestic footings, and severe cracking of walls, both internal and external. The "Water-reactivity and 'clearance'" paragraph in Section 1.3.4 should, however, be noted.

The need to observe clearance distances to boundaries arises from the possibility that a neighbouring building may be placed on the boundary and be adversely affected by an OSR device placed next door.

3. **Rock and shale :** Don't put OSR devices in rock which has zero or near-zero permeability. This includes most non-sedimentary rock and some sedimentary rock such as shale. However, the issue of suitability should not be decided on the evidence of geological (map) information alone : permeability testing on site should be carried out and may reveal an apparently impermeable stratum to be severely weathered or fractured and, therefore, suitable for infiltration technology. The permeabilities of some sandstones have been found to be comparable to those of medium clays, encouraging the view that OSR technology can be considered in sandstone sites also (see "Sites with rock or shallow soil cover over rock", Section 1.3.4).
4. **Shallow soil cover over rock :** Great care must be exercised before applying OSR technology directly to shallow-soil sites. This is because of the likelihood that water stored on or near impervious bedrock will provide a stream of flow along the soil/rock interface and proceed down-slope. The plane of emergence of this interface can be predicted from study of the local geology : this needs to be checked and a conclusion reached as to its importance. If the plane is remote from dwellings or roads, etc., then a water-retaining device can be considered. However, if detailed exploration produces the possibility that emerging seepage will create nuisance or hazard for those downstream, then the prospect of using an OSR device in such circumstances should be abandoned.

The possibility that the underlying rock is permeable, severely weathered or fractured should be explored : a positive finding in this regard may lead to a situation similar to that addressed in item 3, above.

5. **Steep terrain** : Use of OSR devices in steep terrain is, generally considered unwise. British practice places a limit of 5% on the land-slope where on-site water-retention is recommended. The reasons for this limitation are not so much slope-dependent, but are related, rather, to the soil/rock conditions most likely to be encountered in steep terrain. These are reviewed in items 3 and 4, above. Recommended practice would therefore be to exclude water-retaining installations from steep-slope sites in the absence of thorough exploration of site and down-slope geology. A simple guideline is : soil depth (suitable soil) of at least 3 m should be available **throughout** a down-slope, developed hillside before OSR practice should be contemplated.
6. **Watertable interaction with OSR** : The presence of a high (unconfined) watertable certainly limits the potential for use of OSR devices but does not, of itself, preclude them. Provided groundwater levels are **stable**, apart from seasonal fluctuations, they can be associated with quite successful water-retention systems (see “High groundwater environments”, Section 5.3.4). Serious problems can arise, however, at sites where groundwater levels show a systematic rise explained by, perhaps, removal of forest vegetation or the presence of an artificial lake or some other global cause. The inclusion of OSR devices in such circumstances will only aggravate the situation, accelerating the inevitable “waterlogging” of the region. OSR systems should be avoided in any terrain which exhibits a rising water table, particularly where it is **highly saline**. This problem – salinisation – is particularly serious in parts of Western Sydney, Wagga Wagga, NSW, and south-west Western Australia (see WSROC, 2003).

Another problematical aspect of interaction between watertables and OSR practice is where components of the built environment such as basements and undercroft garages, intersect shallow aquifers. Construction of such components must take account of seepage encountered under these circumstances using, for example sump pumps. It would be quite unacceptable for additional water to be introduced into the aquifer through on-site stormwater retention practices applied in the same area. A possible solution is to install OSR devices draining directly to a **lower** aquifer, but it would be imperative that the standing water level (SWL) of the lower aquifer be significantly below the upper aquifer, **and that it remained so** indefinitely.

7. **Watertable affected by upstream OSR** : Care must be taken where groundwater levels meet the stability criterion, reviewed in item 6, above, but where future intended use of OSR devices, upstream, may lead to watertable rise. This situation is possible in valley “bottoms” below hillside developments where significant water-retention is intended. Again, thorough geological exploration of locations is called for to assess the likely impact of (upstream) OSR devices on valley floor watertables : outcomes may limit or even preclude on-site water-retention in some cases.
8. **Aquifer recharge/retrieval – annual balance** : The regime of “flat” potentiometric gradients associated, normally, with flat and gentle-grade landscapes is where ASR (aquifer storage and recovery) technology is most likely to be feasible. It is recommended that recharge exceed retrieval of groundwater by about 20% on an annual basis (see Northern Adelaide and Barossa CWMB, 2000) : this ensures continued equilibrium of local potentiometric levels and, also, sustainability of the resource.
9. **Water quality inflows to OSR devices** : Don’t put uncleansed stormwater runoff directly into sub-surface OSR devices such as “leaky” wells and gravel-filled trenches. The **only** site drainage which can be directly accepted is roof runoff, and even this component should be treated by passage through a rainwater tank with ‘first flush’ filter, or the devices illustrated in Figure 1.5. All leaf-matter must be strained from roof runoff before entering these devices. “Leaky” wells are more robust than gravel-filled trenches in this regard, since they can be more easily de-silted. Any sediment entering a gravel-filled trench is, virtually, locked in for the life of the installation (see “Runoff categories and treatment”, Section 1.3.3).

These cautions do not preclude general surface runoff from on-site retention practice, but the treatment train required for a significant installation must be carefully designed using grassed surfaces, vegetated strips, swales, sand filters, gravel-based reed beds, ‘treatment train’ tanks, geotextile ‘final filters’, etc. reviewed in Chapter 2. Three installations using these elements are described in Section 3.9.

3.9 SOME EXAMPLES OF ON-SITE RETENTION PRACTICE

New Brompton Estate Stormwater Management Project : This project, constructed in 1991, is centred on a small reserve 50 m × 45 m in the Adelaide suburb of Brompton in the City of Charles Sturt. The reserve is surrounded by 15 townhouses, the roof runoff of which is directed, via PVC pipes, to a gravel-filled trench located around three sides of the reserve (see Figure 5.2). Storm runoff collected in winter in the trench is conveyed to a central point where it enters a bore and is transferred to storage in a *Tertiary* aquifer, 30 m below surface level. In summer the stored water can be retrieved from the aquifer and used for irrigating the reserve, thereby saving on mains water normally used for this purpose. This is an example of small-scale aquifer storage and recovery (ASR) technology (Hopkins and Argue, 1994). **Maintenance** of the system at New Brompton Estate involves retrieving the mesh bucket lined with geotextile located at the base of each inlet standpipe (see Figure 5.2) and cleaning it. This operation, for the set of nine standpipes, requires a two-hour visit by the technician four times per year (since 1992). About 2 kg of sediment is removed from each bucket on these occasions. No other maintenance directly connected with the stormwater management and recharge system is needed. [The water retrieval system required for irrigating the reserve has not been installed.] The standpipe system, although satisfactory, is **not** recommended. The approach illustrated in Figure 2.1 is preferred : this is a zero maintenance system with a ‘lifespan’ of 20 years receiving an uninterrupted supply of roof-originating sediment (laboratory study result). ‘Lifespan’ of indefinite length – without maintenance – is possible for a gravel-filled trench where it receives roof runoff subject to ‘first-flush’ treatment or is overflow from a rainwater tank.

Parfitt Square Stormwater Management Project : Opened in 1997, Parfitt Square is a 0.6 ha recreational reserve in the City of Charles Sturt, Adelaide. The reserve contains a ‘treatment train’ of elements which strip sediment passing to it from 1 ha of mixed housing – old and new – and carriageways (see Figure 3.9). Runoff entering the park via car park A encounters, at B, a long, sediment-holding trough (0.5 m × 0.5 m section, 30 m long), followed by a 300 m² gravel-based reed area where surface runoff is further cleansed. Five wetland plants - *Marsillea drumondii*, *Centella asiatica* and *Viola hederacea* (ground covers) – and *Juncus kraussii* and *Typha orientalis* (wetland reeds) have demonstrated robust qualities in the harsh conditions of the reed bed. Next in line is a 100 m long, underground gravel trench with section 4 m × 1m (shown as C). This component terminates in four recharge bores (D) supplying the *Quaternary* aquifer at 12 m depth. Recharge takes place mainly during the winter rainy period; the stored water is recovered for summer irrigation of the reserve (irrigation bore, E). This is another example of small-scale ASR technology. The system was designed to control, without overflow, runoff in all events up to and including that corresponding to the “once in 100 years” event (Argue and Pezzaniti, 1999).

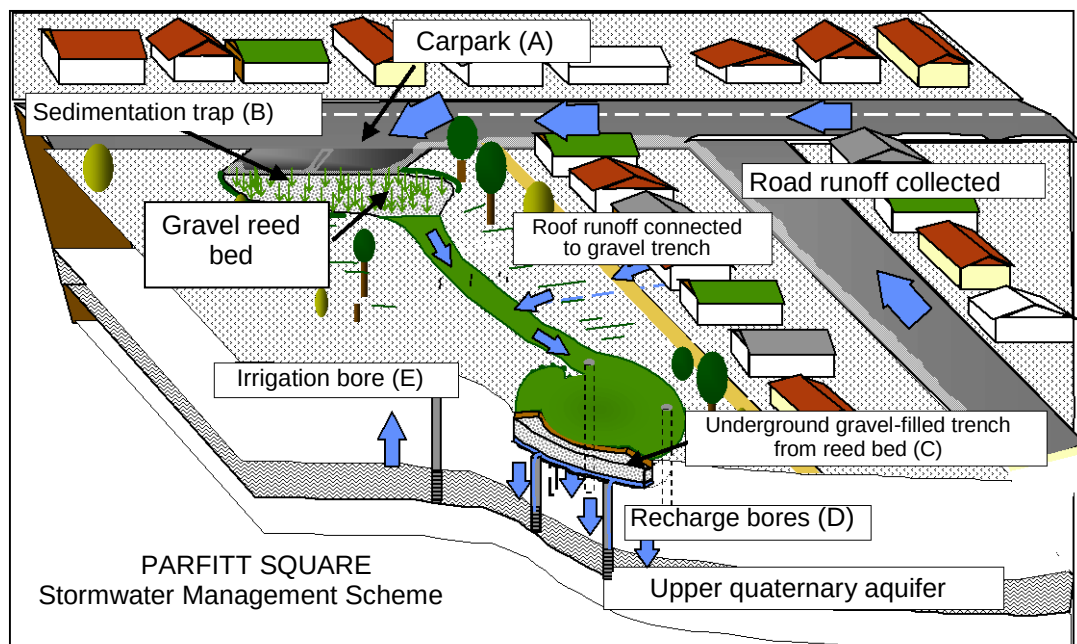


FIGURE 3.9 : “Parfitt Square” : general layout, looking North

Maintenance of the stormwater cleansing and recharge/retrieval system is concentrated on cleaning out the 30 m trough (B) where coarse sediment and other gross pollutants are deposited after each storm. A cleaning operation is mounted once in every two or three years. The gravel-based reed bed (see Section 7.8.5 for species) accumulates fine sediment which is retained within the spaces between the gravel – initial depth, 100 mm : when these spaces fill, another layer of gravel is introduced. The system has been constructed to retain, eventually, all of the fine sediment expected from the catchment over its 100 years ‘life’. Present indications (after six years service) are that the geotextile filter at the head of each bore (D) will need replacing about once in every 20 years.

St. Elizabeth Church Car Park : The car park at St. Elizabeth Church at Warradale, Adelaide (opened 1998), is part of a rainwater collection/treatment/irrigation system which includes tennis courts, the church building and landscaped area and the adjacent Council reserve. **All** surface runoff is retained on site: the design maximum storm is the “once in 100 years” event. In the case of the car park segment : all paved area runoff passes to a central, grassed ‘hard-standing’ area surfaced with “Grasspave”, a plastic ring-matrix product, where it is cleansed by the grass and filtered through 200 mm depth of sand/gravel mixture (see Figure 5.1 and Section 7.8.5). Beneath the filter is a gravel-filled “soakaway” which gathers the clean storm runoff and directs it to a bore where recharge is conveyed to a *Tertiary* aquifer 45 m below ground level. Stormwater generated on the tennis courts passes through a sedimentation tank before entering a gravel-filled trench which also conveys water to the bore; runoff from the church roof, after filtration, enters the same bore. In summer, water drawn from the aquifer through the recharge bore is used to irrigate the church landscaped area and portion of the Council reserve (Argue and Pezzaniti, 1999). **Maintenance** of the scheme at St. Elizabeth Church consists, almost entirely, of caring for the grassed hard-standing area – mowing, weeding, etc. – and providing adequate irrigation for the drought-tolerant grass (25 mm per month in summer). Apart from this, there is cleaning out of the commercial settling tank receiving runoff from the tennis courts. Sediment originating on the church and parish hall roofs is managed in the manner illustrated in Figure 2.1 and is expected to give at least 20 years service without reinstatement. Normal maintenance of the pumped retrieval system is experienced.

Plympton Church Outdoor Community Garden : The basis of the ‘passive’ irrigation facility at Plympton Anglican Church, Adelaide (completed 2001), is a “soakaway” (see Figure 5.3). Storm runoff passing to the “soakaway” is drawn from the main church buildings, an extensive paved area and the roofs of four church-associated residences. A sediment collection and ‘first flush’ filtration system for cleansing the runoff is located upstream of the sub-surface installation. The facility provides ‘passive’ irrigation of a 400 m² garden of native plants and drought-resistant grass planted on the backfill above the “soakaway”: water evaporating from the runoff stored among the gravel void spaces is applied directly to the roots of the native vegetation and grass (Argue and Pezzaniti, 1999). Evidence from this (trial) technology in the harsh South Australian climate suggests that the native vegetation and grass of the garden can be maintained indefinitely with minimal conventional (surface) irrigation. Further evidence of ‘passive’ irrigation withstanding severe drought conditions (spring/summer, 2002) was also observed in grass covering the “soakaways” constructed in Western Sydney (see Section 5.5). **Maintenance** of the Community Garden involves horticultural attention to the shrubs and grassed areas as required by any native garden, except for irrigation which is unnecessary. The scheme operates as a “waterless garden”, however, a sprinkling of water is sometimes applied to “green up” the grass ahead of community use of the site for weddings and other gatherings. The ‘first flush’ grassed area will in due course need to be replaced: its expected ‘lifespan’ is at least 15 years (see Section 7.8.6).

“Figtree Place”, Newcastle, N.S.W. : The Bus Station re-development in Central Newcastle (completed 1998) involves a 0.6 ha site which has been remediated after some decades of contamination by petrol hydrocarbons and other pollutants. The site is now home to 27 one-, two- and three-bedroom residential units called “Figtree Place” (see Figure 5.4). The Brief for the project called for a high level of water conservation to be incorporated into the ultimate design (Argue, 1997; Argue and Argue, 1998). The outcomes of the concept, planning and engineering phases of the project are :

all roof runoff is collected and stored, temporarily, in underground tanks before being used as base supply for (gravity) hot water systems and toilet flushing;

overflow from the rainwater tanks passes to gravel-filled trenches which temporarily store the excess before diverting it by percolation to the watertable (unconfined aquifer);

all surface runoff is directed to a “dry” pond where it is cleansed prior to seeping downwards to also recharge the groundwater;

groundwater beneath the site is used for open space and garden irrigation in summer, and to supply the entire water demand of the bus-washing facility in the adjacent Bus Station;

the system contains all rainfall events up to and including the “once in 50 years” storm in 83% of the re-developed area.

The water conservation performance of the residential segment of the re-development shows around 50% reduction in domestic water consumption and overall (including the bus-washing facility) some 60%; there is virtually “zero runoff” from 83% of the site (see Coombes et al, 2000).

“Heritage Mews”, Castle Hill, N.S.W. : This development (completed 2004) of a 3.2 ha “greenfields” site for townhouse accommodation in Western Sydney represents the first practical application of the **regime-in-balance** strategy (see Sections 4.2.1, 4.6.5 and 5.4). The terrain was characterised by shallow, heavy clay over rock and therefore unsuitable for infiltration as the primary mode of stormwater disposal. A Concept Design for the development which included 62 townhouses was prepared as a collaborative project between Dr. P. Coombes (University of Newcastle), Cardno BLH and UWRC staff (Argue et al, 2003; Coombes et al, 2003). Effective removal (of storm runoff) according to the time scale required – 24 hours – was achieved using 29 gravel-filled trenches and the ‘slow-drainage’ provisions of Procedure 4B (Section 5.1.5) : a special device was developed and incorporated with each trench to control outflow to the target value (see Figure 5.6). The inclusion of 62, 3.0 kL rainwater tanks – one with each dwelling – provided a significant component of retention within the development during storms, as well as replacement of some mains water. By these means, the development avoided using a ‘bottom end’ detention basin of plan area 2,500 m², the normal requirement of Council. The Developer was able to include six or seven housing lots which would otherwise have been allocated to the detention basin area. High standards of stormwater quality control were achieved using UniSA tanks (see Section 2.9.3).

4. GETTING STARTED WITH ‘SOURCE CONTROL’ IN WATER-SENSITIVE URBAN DESIGN

4.1 INTRODUCTION : FOUR ‘SOURCE CONTROL’ CATEGORIES

‘Source control’ of stormwater embraces each of the aims listed in Section 1.3.1, involving :

- control of storm runoff quantity;
- control of pollution conveyed in storm runoff;
- harvesting of cleansed storm runoff.

Unlike conventional storm drainage systems which tend to be focussed on the single issue of peak flow conveyance/reduction, water-sensitive stormwater management usually reflects some level of involvement with each of the domains listed above. For example, a gravel-filled trench (see Figures 1.3, 1.4 and 1.5) required to retain roof runoff from a domestic building should be designed, primarily, to control flooding but its long-term effective operation depends to a large extent on the adequacy of provision made for sediment collection/disposal. Furthermore, the process of emptying the device between storms can involve recharge/percolation/evapotranspiration mechanisms that have significant groundwater or soil moisture enhancement and therefore harvesting implications. More complex, on-site stormwater retention systems such as those described in Section 3.9, illustrate this “interconnectedness” even more strikingly.

The various devices and systems available to the designer for achieving the three aims listed above may be divided into four broad categories :

Category 1 Systems : Those whose primary function is to manage the quantity of storm runoff generated in contributing catchments. This *primary function* has two sub-sets : quantity control in relation to flood and channel-forming flows, and management of ‘low flows’ to maintain waterway bio-communities (fauna and flora). Management of pollution conveyed in the runoff in such cases should be recognised and addressed, but its influence on the dimensioning of the water-retaining devices and systems to achieve quantity control is secondary; stormwater harvesting has important links to both the flooding domain and that of environmental flows.

Examples of Category 1 installations include :

- infiltration or ‘treatment’ surfaces receiving paved area runoff;
- in-ground “leaky” devices receiving roof runoff;
- rainwater tanks dimensioned to achieve ‘low flow’ objectives.

Category 2 Systems : Those whose primary function is to achieve **high performance in containing or removing particulate and dissolved pollutant matter** from set percentages of annual average runoff flows, typically 90%, 80% 70%, 60%, generated in contributing urban catchments. Management of large storm runoff quantity must be recognised and addressed but its influence on detailed dimensioning of pollution control/treatment installations is secondary; similarly for stormwater harvesting.

Examples of Category 2 installations include :

- swales and sediment-retention tanks;
- ‘first flush’ pollution control installations;
- systems involving treatment surfaces with underlying (temporary) storage, such as permeable paving.

Category 3 Systems : Those whose primary function is to **harvest runoff** from contributing catchments. Design of stormwater harvesting systems may involve analysis of the frequency of failure to supply demand as well as ‘yield’ considerations which differ greatly from the peak flow, volume and bypass/overflow-oriented analyses required by Category 1 and 2 systems. The flood control and water quality management aspects of such installations, while significant and must be taken into account, are of secondary importance.

Examples of Category 3 installations include :

- rainwater tanks;
- aquifer storage and recovery (ASR) schemes;
- basins designed for amenity or water recreational activities.

Category 4 Systems : Those with **multi-purpose functions (none dominant)** drawn from the fields of flood control, pollution containment, stormwater harvesting and amenity. Dimensions and detailing for such devices and systems require two or all of these aspects to be considered separately and designs developed which either embrace all demands or represent a compromise between them.

Examples of Category 4 installations include :

- ASR schemes with surface detention storages providing flood control capability;
- swales designed for sediment retention and flood conveyance;
- rainwater tanks used in conjunction with a flood control strategy.

4.2 STORM RUNOFF MANAGEMENT IN CATEGORY 1 SYSTEMS

4.2.1 Three basic stream management scenario/strategies

Category 1 systems should be understood as components of channel networks gathering runoff from groups of sub-areas into branch and main drainage paths, typically of dendritic form (see Figure 4.1). Such catchments or basins should be managed by municipal agencies responsible for planning policies and strategies governing the orderly development of ‘greenfields’ and old occupied sites within their jurisdictions (see Section 4.6). These administrative activities are aided by a classification approach which considers some point “P” on the relevant catchment mainstream (see Figure 4.1), as a **location of singular interest** (flooding or channel-forming considerations). The interest may stem from three broad stream management scenarios and associated strategic objectives :

Scenario (a) : P is the site of an end-of-catchment harvesting scheme such as a wetland (on- or off-stream) where storm runoff, generated in the upstream urban landscape, is collected and cleansed prior to use. The objective of catchment management in this case is to maximise the quantity of water harvested in the catchment ‘bottom lands’ while, at the same time, ensuring that floodwaters (ARI = Y-years event) are contained within a defined flood plain : this objective is implemented at the site level in the **yield-maximum** strategy;

Scenario (b) : P is a recognised reference point in a catchment or sub-catchment in which urban development has occurred and/or is likely to occur under environmentally responsible local government or catchment-wide administrative control outside the borders of a defined flood plain (ARI = Y-years). The objective of catchment management in this case is to maintain the harmonious and synergistic relationship which exists between continuing urban development and ‘acceptable’ use of the flood plain for agricultural and amenity pursuits : this objective is implemented at the site level in the **regime-in-balance** strategy;

Scenario (c) : P is a recognised reference point in a catchment or sub-catchment in which poorly-controlled urban development has already intruded deeply into the flood plain with consequences of past serious flood damage and/or severe channel erosion with the potential for more trauma and environmental damage in the future. The objective of catchment management in this case is to establish and implement local government or catchment-wide strategies to improve the performance of the urban flood control infrastructure and to prevent further urbanisation of the flood plain (ARI = Y-years). This objective is achieved, in particular, by minimising the quantity of stormwater discharged from all new developments and re-developments under the **yield-minimum** strategy.

[Environmental flows: The bulk of text which follows under the Category 1 heading is devoted to the ‘high flows’ or *flooding* domain including channel-forming. The realm of ‘low flow’ hydrology contrasts sharply with this: indeed the two domains are, normally, mutually exclusive. ‘Low flow’ considerations lead to waterway environmental values [preservation of stream bio-communities (fauna and flora), etc.]. This topic is explored in Section 4.7, following, under the title “Mimicking Natural Catchment ‘low flows’ in the Urban Landscape”.]

These scenarios lead to three contrasting approaches to managing storm runoff in catchment components contributing runoff to and beyond point P. They are described as :

Yield-maximum strategy : This applies to case (a), above, where stormwater generated on all components of the upstream catchment is seen as a resource to be harvested at a central location (for example, a wetland). Retention of storm runoff using the ‘source control’ measures and devices introduced in Section 1.3 represents an approach which is in conflict with this objective and, therefore, to be avoided. The most effective way in which the aims of the **yield-maximum** strategy can be implemented is to use conventional storm drainage approaches – maximum paving, pipes, lined channels, etc. – to attract the greatest possible yield from the urban landscape. This has the potential to cause downstream flooding and degradation of waterways and receiving waters (see Section 1.1) : these consequences should and can be reduced through judicious use of detention techniques applied within the ARI = Y-years flood plain (UPRCT, 1999) together with wetland water quality improvement technology (see Sections 2.13 and 2.14). However, the warnings of Debo and Reese (1995) should not go unheeded (see also Nehrke and Roesner, 2004).

Regime-in-balance strategy : In this case, peak flow at point P (see Figure 4.1) for a specified average recurrence interval (ARI = Y-years), must be held at a target value as development/re-development takes place in the (upstream) contributing catchment. The ‘defined flood plain’ referred to in scenario (b), above, needs to be fixed by agreement between all interested stakeholders and should represent a realistic benchmark determined by consensus. [See the ‘benchmark year’ concept, Section 4.6.3.] A practical outcome from such consultation might be, for example, “... the 100-years flood level as it existed in 1980”. The essence of the strategy is its insistence that surface runoff *volume* passing from each catchment component in the defined Y-years event **following development/re-development** is the same as it was (from that same catchment component) **at the time of the adopted ‘benchmark’**. This approach was introduced in Section 1.4 : further explanation is offered in Section 4.6.5

The **regime-in-balance** strategy has two domains of application in long-term urban waterway planning. **“Greenfields” catchments :** equality of *before-and-after* surface runoff volumes – as defined above – in each element of future **development** ensures the continued presence of the existing (natural) waterways without engineering upgrade *for all time*. **Highly-developed catchments :** application of *before-and-after* volume equality in each element of **development/re-development** ensures the continued successful operation of existing (formal) waterways without the need for augmentation or upgrade *for all time* (see Section 3.5.5).

Yield-minimum strategy : This case is identical to that of the regime-in-balance strategy described above in terms of the Y-years peak flow target being maintained (not exceeded) as development/re-development takes place in the upstream catchment. However, the difference lies in recognition of the significantly smaller (flow) capacity of the ‘defined flood plain’ in the present case and, hence, the need for extreme measures to be instituted (for example, zero runoff from catchment components) to minimise runoff contributions from all or a high proportion of new development/re-development projects. Such action, suitably interpreted and implemented through catchment-wide planning instruments, can achieve the ultimate objective of matching the catchment’s (Y-years storm) response to the available flood plain capacity (see Section 1.4). A procedure for applying the strategy is explained in Section 4.6.6.

All (urban) catchments and sub-catchments within a municipal jurisdiction rarely, if ever, fall into one, only, of the above three scenario-strategies. The context of each waterway – particularly its flood history – is likely to place it far more in one category than either of the other two. For example, a stream may show satisfactory containment of floodwaters within its floodplain in the majority of events, but pose the threat of significant property damage in rare floods. This puts the waterway predominantly in scenario (b), above, and therefore a candidate for primary application of the **regime-in-balance** strategy. However, the evidence of damage risk in rare floods carries with it the likelihood of a deteriorating situation if not well managed, so a modest-level application of the **yield-minimum** strategy would also be appropriate. The resulting planning policies applicable to each portion of catchment or land use change would have to be determined by careful and systematic hydrological modelling using a modern computer package. [This process is the basis of the strategic direction discussed in Section 4.6.]

[Scenarios (a), (b) and (c) and their associated strategies, above, may seem to provide, between them, the basis for the full range of policy/technical tools needed to manage catchments experiencing the impacts of contemporary urbanisation. While they (the scenarios/strategies) are readily recognisable in the Australian environment – and, possibly, other developed nation contexts – they are by no means universal. A quite

different scenario/strategy system arises, for example, where managers of urban catchments set out to achieve correspondence of complete runoff hydrographs (**all** ARIs), not just peak flows and runoff volumes observed in pre-development catchments for a fixed critical storm duration and fixed ARI (as above). The need for this degree of streamflow control springs from a desire to re-establish stream conditions conducive to the annual cycle of spawning undertaken by salmon in Canadian and US Pacific North-west streams, seriously disturbed by catchment urbanisation. This approach is the goal of “Stormwater Planning : A Guidebook for British Columbia” (Stephens et al, 2002; see also Gilliard, 2004; Nehrke and Roesner, 2004). The aim of the British SUDS practice is similar (see CIRIA, 2001; CIRIA, 2007).

4.2.2 A fourth strategy for stormwater management

The three strategies for managing storm runoff described in Section 4.2.1 are proposed for use by municipal or catchment-wide water management agencies to control development/re-development requiring site-by-site (agency) approval as it occurs. A **fourth** strategy is also evident and widely employed in the Australian municipal context.

This is invoked where storm runoff from a general urban landscape is directed to an **end-of-pipe** stormwater quality improvement system prior to discharge into an environmentally-sensitive receiving water domain. Unlike the strategies associated with the listed scenarios [(a), (b) and (c), above], this (fourth) strategy cannot be readily interpreted into a site-by-site policy instrument in keeping with municipal needs to control progressive urbanisation (via the approval process) as it occurs.

4.2.3 The design storm critical duration : $T_{C(\text{total})}$ and $T_{C(\text{local})}$

The location P, defined above as a reference location of ‘singular interest’, may be part-way along a main drainage channel – natural or lined – as shown in Figure 4.1. The task of managing flow under any of the three stream scenarios [(a), (b) and (c), above] implies use – in the analytical process – of well-understood parameters such as contributing catchment area, runoff coefficient (or some other loss model), rainfall characteristics, etc. as well as **critical storm duration** at point P, introduced in Section 1.4.2. This latter parameter is defined, for the circumstances illustrated in Figure 4.1, as the (unique) storm duration which yields **the greatest peak discharge from the (upstream) catchment at point P for an adopted ARI = Y-years**. In the notes which follow, this critical duration is called $(T_C)_{\text{total}}$.

It should be noted that a different, larger, $(T_C)_{\text{total}}$ applies to catchment discharge point O, and smaller $(T_C)_{\text{total}}$ values at any points of singular interest upstream of P. Similar reasoning may be applied to point Q within a branch or ‘local’ sub-catchment (see Figure 4.1) leading to the use of $(T_C)_{\text{local}}$ in later notes.

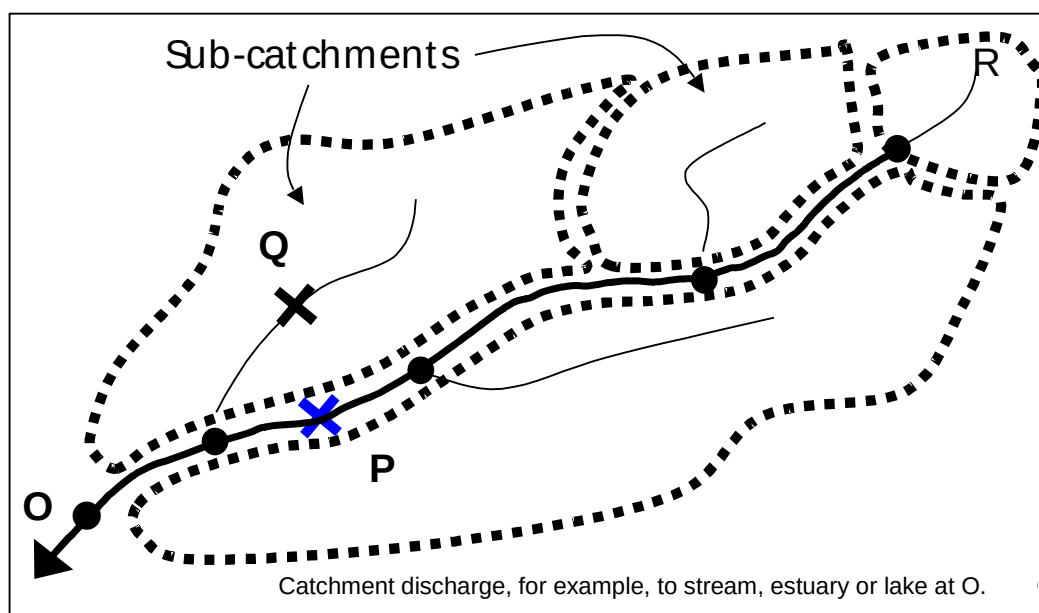


FIGURE 4.1 : Dendritic structure of a typical urban catchment

Values for $(T_C)_{total}$ and $(T_C)_{local}$, as described above, depend, of course, on the magnitudes of the respective catchments in which they are used. They are determined most effectively by applying a set of appropriate design storm patterns ($ARI = Y$ -years) to mathematical models of their contributing catchments and finding the storm duration giving the greatest Q_{peak} at the location(s) of interest (see Section 3.4.5).

Satisfactory values of $(T_C)_{total}$ and $(T_C)_{local}$ in keeping with ‘lag time’ considerations (Sections 1.4.2 and 1.4.3) and the *modified* design storm approach (see Section 3.5) used in this document can be found, however, by reference to the familiar ‘time of concentration’ concept explained in Section 5.3 of AR&R – 1987 (IEAust, 1999). Where uncertainty exists concerning the exact location of a point of singular interest or where good stream management throughout the catchment is intended, it is recommended that the longest (catchment) travel time to discharge point O be used. This is recognised as the catchment-wide approach which leads to devices and installations whose volumes, in the short term, may seem excessive but which has the advantage of providing the basis for a comprehensive strategy covering all catchment eventualities. Practitioners with long experience in urban catchment management, particularly flood control, recommend novices to err on the side of overestimating critical storm duration : there are sound reasons behind this advice.

Applying the outcomes of this discussion to the type of problem reviewed in Section 1.2 (urban re-development) within Scenario (b), above, for example, leads to the use of the **regime-in-balance** strategy to eliminate all of the **additional runoff volume** generated on re-developed sites in storms of critical duration ($ARI = Y$ -years). This opens the way for present development drainage capacity to serve the needs of re-developing urban landscapes with minimal, if any, expensive upgrades or duplication. Where re-development takes place within a catchment falling under Scenario (c), the **yield-minimum** strategy should be used to eliminate **all runoff volume** generated on re-developed sites, again, in storms of critical duration ($ARI = Y$ -years). This will give relief to the overstressed (existing) drainage network; persistence with this strategy may lead, eventually, to the matching of runoff flows to present development drainage capacity. [These matters are revisited in greater depth in Section 4.6.]

It must be appreciated that design based on this approach does not guarantee zero outflow from retention-based systems (OSR) in all $ARI = Y$ -years events. Y -years storms of duration **shorter** than ‘critical’ will (all) be fully contained, but **longer** (than ‘critical’) Y -years storms will produce overflow which must be catered for. However, such overflow, integrated with the general streamflow in these circumstances, produces channel peak flows at singular interest points such as P, Q, etc. which are **less** than Q_{peak} resulting from the Y -years storm of critical duration in the respective catchments.

4.2.4 The design storm in site drainage : on-site storage and the critical storm

The critical storm duration argument, above, can be extended “up” the catchment to discharge points of individual sub-areas enabling the potential for flooding at these points, as a consequence of storms of duration equal to **site time of concentration**, t_C , to be considered. Comparison of this potential threat against the economic and social consequences of flooding at, say, points P, Q, etc. in the complete catchment (see Figure 4.1), however, is likely to overwhelmingly favour the latter (i.e. P, Q, etc.) as the location of dominant concern in typical urban catchments. It follows that site drainage **storage devices** required under OSR (or OSD) policy applications, should be designed for $(T_C)_{total}$ or $(T_C)_{local}$ storm duration and **not** t_C . There is widespread misunderstanding about this, both nationally and internationally (Argue and Pezzaniti, 2004; Argue, 2005; Emerson et al, 2005).

While the use of $(T_C)_{total}$ or $(T_C)_{local}$, as appropriate, in the design of on-site storage devices satisfying catchment-wide flood control objectives is indisputable, use of these parameters to design **all** drainage elements on the site itself can lead to system failure and the potential for serious (site) flood damage. A clear distinction must therefore be drawn between design of site storages, as reviewed above, and design of the **entry** works leading to the storages and **exit** works leading from them (see Figures 1.3 and 1.5). Design of these components involves not only considerations of critical storm duration but also average recurrence interval, ARI. This issue is addressed in Section 4.2.6.

There are two unusual situations found in urban drainage systems which may be seen as departures from the theory presented above. They occur where surface runoff generated in isolated catchment elements passes out of the urban landscape without impacting in any way on neighbouring elements. Examples of the two circumstances are where controlled discharge from a relatively small catchment passes directly to :

- a waterway of great capacity – perhaps a tidal estuary, a lake or deeply incised channel,

OR

- a natural or purpose-built **sump** with zero overflow in all practical storm runoff circumstances. This is typically a groundwater recharge domain with large available storage and/or significant capability for into-ground water disposal.

Critical storm duration in the first case equals ‘time of concentration’, t_c ; marina developments are examples of this. But in the second case, it is incorrect to assume that the ‘sump option’ with critical storm duration, t_c , and ARI, $Y = 100$ years, covers “...all practical storm runoff circumstances”, quoted above. A true ‘sump’ must be capable of disposing of, easily, runoff from the contributing micro-catchment **in the full range of storm events** – durations 10 minutes to 72 hours for ARI, $Y = 100$ -years. Equations 5.7 to 5.9, inclusive, may be used to design such systems.

In summary, then, it is clear that where volume-dependent devices are to be designed as components of a catchment-wide stormwater management strategy, three values of critical storm duration have currency : $(T_c)_{total}$ or $(T_c)_{local}$ for the great bulk of on-site storage cases and t_c or site ‘time of concentration’ for the smaller number of applications reviewed above. The sump case requires the full range of storm durations to be considered at the level of ARI, $Y = 100$ -years.

The aspects of critical storm duration explored in the present and previous sections cover the great majority of urban catchment storm runoff cases encountered in normal practice (see the final four paragraphs of Section 4.2.6). Special circumstances do occur, however, in ‘top end’ portions of large catchments which can justify the use of smaller and therefore less costly OSR installations. Theory explaining this departure from mainstream practice is presented in the following sub-section.

4.2.5 ‘Top end’ site drainage

The considerations reviewed above relating to the determination of critical storm duration in ‘total’, ‘local’ and individual site situations are well-recognised and consistent with established hydrological design practice. The particular circumstances caused by the retention of runoff on **top end** sites in a design storm event, however, have important consequences which are additional to what has gone before. This novelty accounts for it being explored here in somewhat greater depth.

It is a frequent experience that basin-catchments of the type illustrated in Figure 4.1 are urbanised from their lower reaches (or ‘bottom lands’) upstream, leaving sub-catchments such as **R** till last. In the process, development encroaching close to or even into the floodplain occurs at low- or mid-catchment points, or a waterway crossing is constructed that is seriously restrictive when considered in the light of later, upstream development.

If the theory governing critical storm duration as explained above is applied, then design of components in sub-catchment **R**, where they are dominated by flooding concern at point **P** (see Figure 4.2), must be based on catchment travel time to **P**. This leads to OSR structures – “leaky” devices such as gravel-filled trenches, etc. – in sub-catchment **R** that must store the entire surface runoff in a critical design storm of, perhaps, long duration. The following theory shows this to be **unnecessary** and, further, that compliance with the criterion to preserve Q_{peak} at point **P** (or even lower it) with catchment development as it takes place, can be achieved with OSR structures of much lesser dimensions. The economic advantages of this approach are obvious.

It should be noted that the approach reviewed here is consistent with and useful in managing catchments falling into the (a) scenario-strategy, above (**yield-maximum** strategy). **However, it violates the principle of before-and-after runoff volume equality which is fundamental to the regime-in-balance strategy: examination of Figures 4.3(b) and (c) reveals why this is so. The approach is even less applicable in catchments where the yield-minimum strategy is employed to counteract the consequences of previous over-development.**

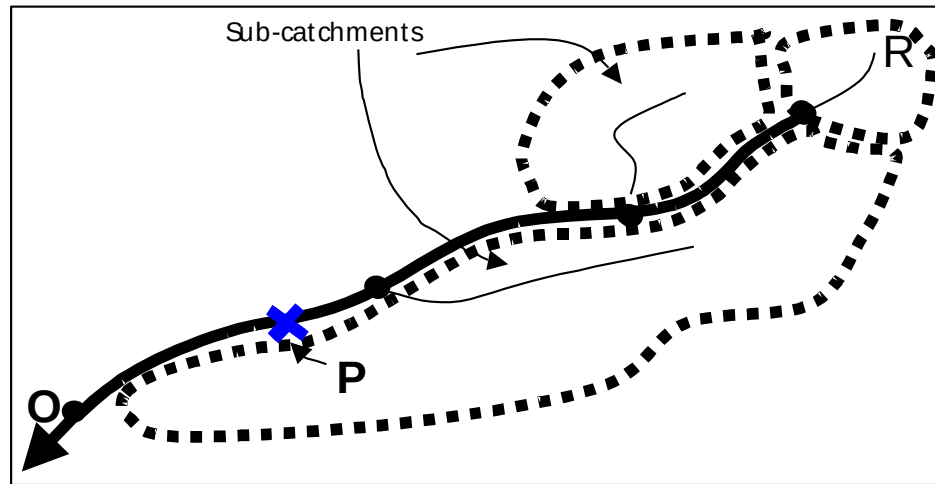


FIGURE 4.2 : Consideration of top-end sub-catchment R and those parts of the total catchment draining to P

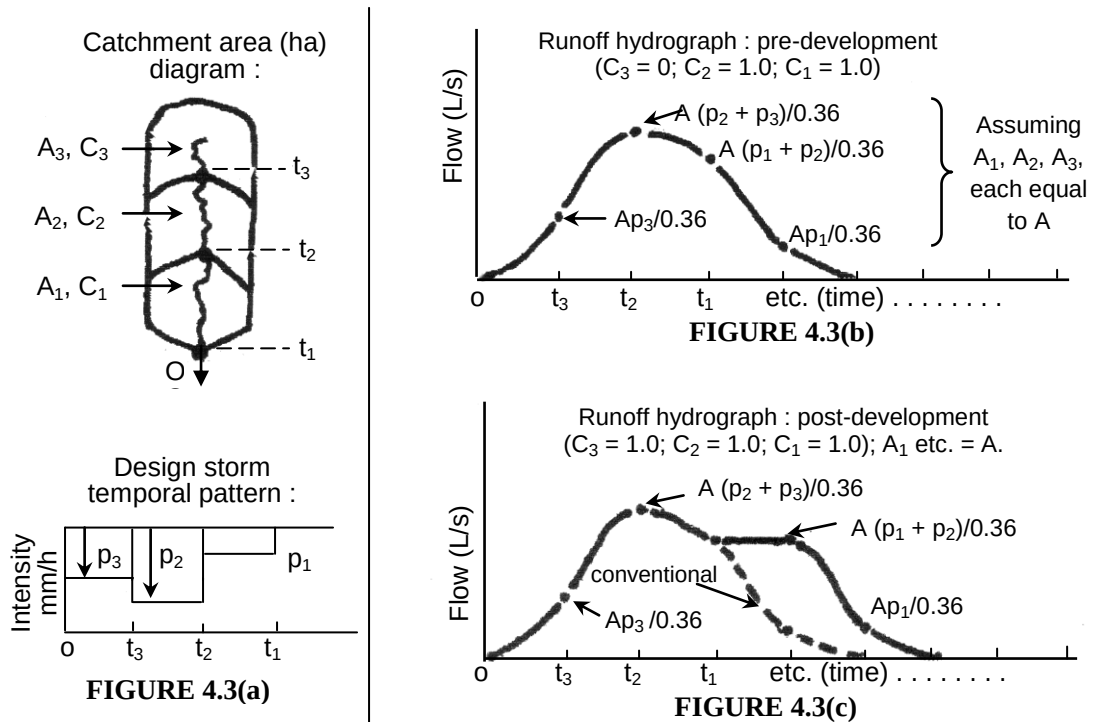


FIGURE 4.3 : Illustrations for hydrological analysis of ‘top-end’ sub-catchments

Consider the sub-catchments shown in Figure 4.3a having catchment travel times as follows :

Sub-catchment 3 (‘top end’) : area A_3 ha : internal travel time t_3 hours

Sub-catchment 2 (‘middle’ sub-area) : area A_2 ha : internal travel time $(t_2 - t_3)$ hours

Sub-catchment 1 (‘bottom end’) : area A_1 ha : internal travel time $(t_1 - t_2)$ hours

Catchment **overall** travel time to O is t_1 .

[Values for t_3 , t_2 and t_1 are determined by application (three times) of the methods referred to in Section 4.2.3, rather than – simply – flow travel time differences along the main drainage path.]

Runoff coefficients :

Area A₃, initially, C₃ = 0 (undeveloped);

Area A₂, fully developed, C₂ = 1.0;

Area A₁, fully developed, C₁ = 1.0.

Design storm temporal pattern :

$$\begin{array}{lcl} \text{Time } 0 \rightarrow t_3 : & \left. \begin{array}{l} p_3 \text{ mm/h} \\ p_2 \text{ mm/h typical pattern, } p_2 > p_3 > p_1 \\ p_1 \text{ mm/h} \end{array} \right\} & \\ \text{Time } t_3 \rightarrow t_2 : & & \\ \text{Time } t_2 \rightarrow t_1 : & & \end{array}$$

This leads to a runoff hydrograph (see Figure 4.3b) for which :

$$\begin{aligned} Q_{\text{peak}} &= (C_1 A_1 p_2 + C_2 A_2 p_3)/0.36 \text{ L/s} \\ &= A (p_2 + p_3)/0.36 \text{ L/s} \end{aligned} \quad (4.1)$$

if areas A₁, A₂ and A₃ are each considered equal to A ha.

With development of area A₃ (new runoff coefficient C₃ = 1.0) and if **full retention of only the runoff occurring in the first segment of the storm on area A₃, p₃ mm/h for time t₃ hours, is assumed** (stored volume = 10 A₃ p₃ t₃ m³), then the resulting hydrograph also has a peak value equal to A (p₂ + p₃)/0.36 L/s. The hydrograph includes a plateau of flow equal to A (p₁ + p₂)/0.36 L/s (see Figure 4.3c).

The importance of this conclusion for design of OSR facilities in sub-catchment A₃ is that if normal theory (see the Procedure 2 cases, Section 5.1.3) were followed and containment of the entire critical design storm runoff (duration t₁ hours) were provided, the required volume would be 10 A₃ (p₃ + p₂ + p₁) t₃ m³. Provision of this storage volume in sub-catchment A₃ would, of course, give rise to significantly smaller runoff *volume* at O (see Figure 4.3c) than would occur with the ‘first segment’ approach described above, but the **peak flow rates** of both hydrographs (at O) would be, in fact, the same.

It should be noted that, in terms of the quantities introduced earlier in this Section : duration t₃ corresponds to ‘time of concentration’ of area A₃; and, duration t₁ corresponds to (T_c)_{local} or (T_c)_{total}, depending on the extent, within the catchment, to which the ‘first segment’ approach is being applied. It should, further, be noted that the equality of segments assumed in Eqn. (4.1) represents a limiting case for satisfying equality of pre-development and post-development Q_{peak} flows at O. If A₃ < A₂ < A₁, then the post-development case shows a **lower** Q_{peak} which is a favourable outcome; if A₃ > A₂ > A₁ (an unusual catchment geometry), the opposite is true and the ‘first segment’ approach should **not** be used, because it leads to a **greater** Q_{peak} with development.

The same theory can be applied to ‘second segment’ cases but favourable outcomes in terms of pre- and post-development Q_{peak} flows at O are restricted to special circumstances requiring particular geometrical relationships between A₁, A₂ and A₃ as well as particular hyetograph temporal pattern distributions. The designer is advised, therefore, to either explore circumstances of ‘second segment’ development with great care to confirm peak flow equality/reduction, or employ conventional theory (Section 4.2.3) to base full retention storages on critical storm durations of (T_c)_{total}, (T_c)_{local} or t_c, as apply.

Design procedures which apply to the basic Category 1 installations, are set out in Chapter 5 under four main headings :

- infiltration or ‘treatment’ surfaces;
- “leaky” devices;
- infiltration or ‘dry’ ponds;
- infiltration devices with slow-release capability.

4.2.6 Average recurrence interval (ARI) and installation components

The question : “what ARI should be employed in a given set of design circumstances?”, is rarely, if ever, answered to the satisfaction of all concerned parties. Argue (1986) addressed this issue and offered a hierarchy of ARI and AEP (annual exceedance probability) values, applied to a wide range of land uses and community infrastructure components. This has been reproduced (modified) in Table 4.1.

In the earlier discussion, above, ARI = Y-years has been used in connection with the ‘defined flood plain’ and also in relation to the dimensions of on-site retention devices (Section 4.2.3). This should not be understood to mean equality of ARI value in the two cases quoted. Drawing together matters raised in the previous three sub-sections with those of ARI leads to the following summary (refer to Figures 1.3 and 1.5):

On-site entry works (Q_{peak}) : Most common applications (OSR devices as components of the minor drainage network) involve the use of design storm duration equal to t_c and ARI = Y-years, where “Y” is a value acceptable for minor system drainage in the range 2-years to 10-years inclusive. There are three vital elements of design that must be recognised with these (entry) systems :

- provision for bypass in events yielding runoff exceeding the entry works’ capacity. In these circumstances, accommodation for bypass of the ‘gap flow’ – the **difference** between the rare event flow (ARI, Y = 50-years, etc.) and that from the minor storm event (ARI, Y = 2-years, etc.) – must be provided. Storm duration equal to t_c should be used exclusively in **both** of these calculations;
- provision for conveying runoff generated in rare storms (ARI, Y = 50- or 100-years, etc.) of long duration [critical storm duration – $(T_c)_{\text{total}}$, $(T_c)_{\text{local}}$ etc.] in some stormwater management strategies (see Section 4.6). This requirement is explained in more detail below;
- a value of ARI, Y = 0.25-years is recommended for ‘entry’ sediment control units or gross pollution traps receiving runoff from ground-level paved surfaces. This aspect is discussed under Category 2 installations, Section 4.3. Storm duration equal to t_c should be used exclusively in these calculations.

On-site storage devices and installations (runoff volume) : Use design storm of duration $(T_c)_{\text{total}}$, $(T_c)_{\text{local}}$ or t_c as appropriate, and ARI = Y-years, where “Y” is consistent with the strategic approach adopted as municipal agency policy (see Section 4.6). ARI may therefore be in the minor system range (2- to 10-years), or Y = 50- to 100-years. Design of these components is examined in considerable detail in Chapter 5. Take careful note of Sections 4.2.4 and 4.6.1.

On-site overflow systems (Q_{peak}) : The maximum possible flow which these components are ever required to discharge is that determined as Q_{peak} for the entry works. It is therefore recommended that the same value be also adopted for exit works. This is a conservative but, nevertheless, acceptable solution to the design problem.

The range of ARI values included in this outline covers the entire spectrum of Australian practice with a design difficulty arising, apparently, in relation to different ARIs and different design storm durations applied to entry works, above. How can an on-site storage device with entry flows restricted to minor system limits (ARI, Y = 5-years, for example) possibly receive major system runoff flows (ARI, Y = 100-years) where this is required by Council policy?

The answer to this question comes from recognition of the significant differences in rainfall intensities, and therefore runoff peak flows, which occur between storms of duration t_c – used in entry works design – and those of duration $(T_c)_{\text{total}}$ or $(T_c)_{\text{local}}$ which determine storage requirements. For example, in Newcastle NSW, the t_c = 10-minutes ARI = 5-years storm intensity which might be used to design entry works, is 109 mm/h giving $Q_{\text{peak}} = 6.0$ L/s for a 200 m² domestic roof. The peak flow entering a device receiving runoff from the same roof in the $(T_c)_{\text{total}}$ = 60-minutes, ARI = 100-years event is 4.2 L/s, based on rainfall intensity, $i = 75$ mm/h. So the minor system design governs.

However, this is not always the case and the **greater** of the two calculated peak flows should be used for entry system dimensioning. Note that in the Newcastle, NSW, area, the 10-minute ARI, Y = 5-years storm intensity is smaller than that of the 20-minute ARI, Y = 100-years event (see Appendix B). Thus, if site drainage works (t_c = 10 minutes) were required for a development in Newcastle in a sub-catchment for

which $(T_C)_{\text{local}} = 20$ minutes, then the entry system for the works should be designed for the **major** system Q_{peak} with ARI, $Y = 100$ -years.

There is a further requirement of on-site drainage system behaviour which may sometimes be expected of OSR systems. This relates to performance **in all storms up to and including those of ARI = Y-years frequency**. At first sight this might appear to be covered by what has gone before, which is not the case. The difference lies in a change of focus from critical storm duration (ARI = Y-years) being central to the design process, to, frequency ARI = Y-years being central, untied to any particular storm duration.

The great majority of simple OSR systems such as those illustrated in Figures 1.3, 1.4 and 1.5, are unlikely to enter the domain under discussion here. But on-site retention installations which are integral with water features or have close interaction with the public, may well do so. Examples of such facilities include infiltration or ‘dry’ ponds which in normal times are used as open space recreation reserves, isolated roadway reserve swales, grassed hard-standing areas for motor vehicles, and sports fields integrated with OSR sub-structures.

Those responsible for financing and maintaining such installations – municipal, cultural, educational, commercial, etc. agencies – understand ‘failure’ with a recurrence interval of Y-years to mean “flooding, on average, once, **only**, in every Y-years.” To achieve this standard of protection requires design to take account of **all** storm durations, not just those which are critical in certain catchment circumstances. [The **sump** discussed in Section 4.2.4 represents the limit of this case with ARI, $Y = 100$ -years.]

Strictly, the class of problem discussed in these latter paragraphs lies not in Category 1 but, rather, in Category 4 because of its linking of two aspects of stormwater management – flood control and amenity. An illustration of this aspect of drainage design is presented in Example 5.5, Section 5.2.

TABLE 4.1
A LAND-USE/FLOOD FREQUENCY HIERARCHY FOR AN URBAN COMMUNITY
(after Argue, 1986)

FLOOD SECURITY LEVEL	CLASSIFICATION	DESCRIPTION OF COMPONENTS	DESIGN FLOOD FREQUENCY
VERY HIGH	Strategic I	Floor levels of hospitals, Civil Defence HQ	Design AEP 1 in 500
	Strategic II	Floor levels of police, ambulance and fire stations; water supply and wastewater treatment centres; electric power and gas supply stations. Floor levels of convalescent homes and community buildings which could operate as dormitory centres in great flood events.	Design AEP 1 in 200
HIGH	Dormitory I Dormitory II P/C/I* I	Floor levels of high density residential. Floor levels of low-medium density residential. Floor levels of essential food, pharmaceutical, retail and department stores; centres employing large labour force; community administration and education centres; centres for storage of rare artefacts; venues for entertainment, dining or popular indoor sports.	Design ARI 100-years
	P/C/I* II	Floor levels of factories and outlets supplying non-essential items; premises of businesses and institutions which involve small numbers of people; premises of sport or community activities infrequently used.	Design ARI 50-years
LOW	Open space I (and arterial roads)	Frontages of all units belonging to “very high” priority above; outdoor areas where rare artefacts are displayed or stored.	Design ARI 5 – 10 years
	Open space II (and local roads)	Frontages of all units belonging to “high” priority, above.	Design ARI 3 – 5 years
	Open space III	Other open space areas including general parks and outdoor recreation areas.	Design ARI 1 – 3 years

* Public/Commercial/Industrial

4.3 CATEGORY 2 SYSTEMS

4.3.1 Strategy A and strategy B

Pollution control in the urban landscape is normally implemented as a highly focussed activity using devices and systems ranging from trash racks and sedimentation basins in urban waterways to gross pollution traps (GPTs) used in pipe networks and minor drainage paths. These installations undoubtedly have a valid role to play in reducing gross pollution generated in medium to large catchments, but they do not – as stand-alone installations – provide the best option, necessarily, for stormwater ‘source control’ in urban design that is water-sensitive. There are two reasons for this :

1. The sediment-removing capabilities of GPTs do not extend, typically, to the fine sediments (size 100 μm and smaller) which cause greatest damage to aquatic (river and marine) ecosystems (Walker et al, 1997; Weisner et al, 1998).
2. GPTs play no part in reducing dissolved pollution concentrations frequently found in storm runoff from urban catchments.

It is possible – within the realm of ‘source control’ – to devise systems which satisfy **both** of these objections, however the issues raised by such design solutions are complex : they are reviewed in Sections 7.1. Suffice it to say at this point that systems which manage the full range of water-borne urban pollutants and which, finally, pass cleansed effluent downstream or to a receiving domain, represent valuable elements of any WSUD strategy : such elements fall into **Category 2**. They typically incorporate three elements – some form of ‘first contact’ filter system; capacity for temporarily storing (partly) cleansed storm runoff; and capability for percolating (infiltration) dissolved pollutants into the parent soil body. The *three* elements are illustrated in Figure 3.5a (see also Figure 1.7): the ‘best practice’ method for designing such systems – as explained in Section 3.4 – is through ‘continuous simulation’ modelling.

It is well-recognised (NSW Dept of Housing, 1998; CSIRO, 1999; Wong et al, 2000) that at least 95% of the total runoff (volume) generated annually – on average – in an urban catchment arises from flows equal to or smaller than the flood resulting from the storm of critical duration in a catchment and ARI, $Y = 0.25$ -years. Now, if it were assumed that all storm runoff, regardless of flood magnitude, carries the same pollution concentration, and, if it were further assumed that an installation or device ‘matched’ to the above limit is in place at the discharge point of a catchment and that it is effective in treating all flow passing through it, then it would be possible to guarantee at least 95% pollution control for the total (annual average) flow passing from the contributing catchment.

The assumption, above, relating to pollution concentration uniformity is erroneous : the true situation is for pollution concentrations in the larger flood flows to be usually **lower** (there are exceptions!) than those in small-medium flows, leading to a generally better than 95% performance. This consequence is further strengthened when it is recognised that much of the ‘first flush’ of pollution conveyed in runoff generated in greater-than-0.25-year events, is also treated. It is quite possible, in certain circumstances, for the ‘first flush’ of the 20-years or 50-years or even 100-years flood runoff from a small site to be treated in this way.

These considerations turn the suggested target of 95% pollution removal using on-site filtration at the discharge point of a catchment into an entirely feasible possibility. The linking of this goal with that of a modest contribution to flood-control by on-site retention (see above) of cleansed stormwater, is an objective well aligned to the principles of WSUD.

An **alternative** approach, but one which delivers an equally acceptable outcome, is that presented in Table 3.2 (“leaky” devices). In this case, bypass and overflow, **together** account for 10%, 20%, 30% and 40%, etc respectively, of catchment flow entering the system. Treatment of this flow **at the 90% retention level**, that is 10% (bypass + overflow), corresponds approximately to the 95% pollution removal level discussed above. This is the preferred *approach* of current Australian practice (Engineers Australia, 2006)

Both of the approaches described above are referred to as **Strategy A** pollution control in later sections of the Handbook.

Of course, the resources needed to achieve these objectives, in terms of required storage volume, may exceed the space available, or the cost of the required works may be beyond the means of agencies wishing to use the technology. In such cases, a second best option should be considered which amounts to using a

$Q_{0.25}$ entry filter matched to the site critical storm duration, t_c , followed by storage capacity sufficient to contain **the volume of the ‘first flush’ produced in the storm of critical duration, only**. This volume is much smaller than that required for Strategy A compliance. All runoff flows greater than $Q_{0.25}$ as well as all later flows (following the ‘first flush’) with their lesser pollution concentrations, would be bypassed.

The goal achieved in these circumstances is referred to as **Strategy B** in later sections of the Handbook and, although a significantly lower standard of pollution containment than Strategy A, is, nevertheless, an improvement on many current practices.

4.3.2 Aquifer access or slow-drainage provision

Storages required as the Strategy A outcome from the Table 3.2 “leaky” devices analysis, referred to above – and, to a much lesser extent, Strategy B cases – may be greater in plan area extent than the space available for them. [In this connection it should be noted that removal of **retained**, i.e. stored water, in the “leaky” devices analysis (Table 3.2), is entirely by ‘natural’ means (percolation) only.] In these circumstances, a steady release of some stored water throughout the filling (and later) stages, can lead to a decrease in the size (plan area) of the installation sufficient to achieve a satisfactory ‘fit’ in some cases.

The basis for modifying the initial design (given by the Table 3.2 analysis) in these circumstances is the same as that used to enable target *emptying times* to be achieved in the design of ‘leaky’ flood control installations, namely, ‘hydraulic’ means of stored water release into an accessible aquifer, or by slow drainage (see Section 3.5). This process is explained in Section 7.3: however, it includes a warning relating to use with runoff containing significant concentrations of dissolved pollution.

Design procedures for Category 2 installations are set out in Chapter 7 under three main headings :

- Simple pollution control (Strategy A and Strategy B) systems.
- More complex pollution control (Strategy A and Strategy B) systems.
- ‘Filter strip’ swale systems.

4.4 CATEGORY 3 SYSTEMS

The primary difference between the analyses which are used in the design of Category 1 and Category 2 systems and those of Category 3, relates to their data bases. The design process which produces Category 3 installations typically involves the use of historical volumetric data which may take the form of rainfall records (design of rainwater tanks) or monthly streamflows (water amenity installations) or annual rainfall (ASR schemes). These data, in each case, provide information on the **resource available**.

How the resource is used provides the other main ingredient in the design process. In the case of domestic rainwater tanks, this may be determined as a daily quantity but its magnitude will vary according to household membership and the range of uses to which the resource is put : with these matters resolved, use is also observed to vary diurnally and, possibly, seasonally. Design, taking all these variables into account, therefore, requires a trial-and-error approach to be taken to arrive at a final ‘match’ between water availability and use. For example, roof areas of arid region domestic dwellings may only be capable of collecting enough rainwater for household toilet flushing, whereas in a tropical location, raintank water may meet all domestic demand for toilet flushing, hot water and clothes washing and, possibly, some outdoor irrigation (see Chapter 8).

The same principles may be employed in fixing storage volumes for commercial/industrial plant using rainwater for processes such as cooling, slurry formation, washing or irrigation. Again, the feasibility of matching the various demands to the available resource can only be established by trial-and-error procedures using long-term rainfall data.

Similar procedures to those referred to above are employed to establish storage volumes for water amenity/recreation ponds and basins. In these cases, unlike rainwater tanks, account must be taken of various losses to the natural environment, in particular, evaporation from the water surface, leakage and ‘release policy’ to provide downstream environmental flows. In these cases direct use may be small compared to losses : these analyses may involve the use of monthly or daily inflow data.

The third broad category of stormwater harvesting subsumed under Category 3 is that of aquifer storage and recovery (ASR) schemes. Compared with rainwater tank and amenity basin dimensioning, these systems are ‘coarse’: the reason for this is their dependence on aquifer hydraulic processes which respond to water input/output stimuli much more slowly than surface water systems. The data base required for the design of ASR systems is little more than annual rainfall.

Harvesting which occurs as a by-product of on-site stormwater retention, for example “passive irrigation” in Example 5.3, Section 5.2, is **not** addressed in Chapter 8. The reasons for this are the secondary nature of such systems among retention practices, and the difficulties they pose in terms of analysis and quantification.

Design of ‘pure’ Category 3 systems is reviewed in Chapter 8, where consideration is given to harvesting of roof runoff: a secondary objective – but an important one – is the beneficial effect raintank presence in an urban catchment can have on local storm runoff peak flows.

4.5 CATEGORY 4 SYSTEMS

No additional types of analyses beyond those referred to in Sections 4.2, 4.3 and 4.4 are required to design devices and systems falling into Category 4 (see Section 4.1). This includes installations “..... with multi-purpose, approximately equal functions drawn from the fields of flood control, pollution containment, stormwater harvesting and amenity”. This gives rise to four different combination possibilities:

1. flood control and pollution containment (Problem 1);
2. flood control and stormwater harvesting (Problem 2);
3. pollution containment and stormwater harvesting (Problem 3);
4. flood control, pollution containment and stormwater harvesting (Problem 4).

It may be concluded from Sections 4.2 – 4.4, inclusive, that solutions to these problems of overlap may be solved, typically, as follows:

Problem 1 : Determine a design for $ARI = Y_{\text{flood}}$ using the flood control procedures in Section 5.1, hence volume V_f ; determine a pollution control storage volume, V_p (90% retention) from Chapter 7; compare the capacity of storage given by the flood control design, V_f , with that required for pollution control, V_p . If $V_f > V_p$ – which is usually the case (see Section 7.6.2) – accept design without storage modification; if $V_p > V_f$, increase capacity of flood control design to give storage required by V_p . [The design must include a required treatment train to ensure that pollution content up to the level of ARI, $Y = 0.25$ -year events, at least, can be contained including overflow.]

Problem 2 : The use of rain tank storage to achieve the joint objectives of *harvesting* and *flood control* raises complex issues which are reviewed in Section 8.7.

Problem 3 : Roof runoff is recognised as the cleanest stormwater source available in the urban landscape. Control of its pollution load – meeting the highest standards set in Chapter 7 – is achieved through the process of collection (including provision for bypass and ‘first flush’ treatment) and storage with use (refer to Chapter 8). The problem of pollution conveyed in general surface runoff is addressed in Chapter 2 and re-visited in Chapter 7.

Problem 4 : Solutions to problems involving all three domains of stormwater management are highly complex and require first contact management (‘collection’ as in Problem 3, above, or bypass and a treatment train) followed by storage (with use) and, possibly, provision for overflow. Each of the projects described in Section 3.9 represents a solution to ‘Problem 4’ situations. Designs of some elements of these projects can be found in Sections 5.2 and 7.7.

4.6 COUNCIL RESPONSIBILITIES AND STRATEGIC PLANNING

4.6.1 The four networks

Responsibility for policy decisions relating to the raft of issues reviewed in the foregoing sections rests firmly with councils or municipal agencies having oversight of the catchment-wide planning process. There are, broadly, four sets of catchment network conditions in which councils are required to make decisions about future development/re-development in their areas of jurisdiction. These are :

- **High-capacity major/minor networks :** These are associated with catchment-wide street drainage installations capable of satisfactorily controlling minor stormwater flows (2-years to 10-years, as appropriate), located within a matrix of well-defined overland flow paths providing acceptable freeboard to floors of residential and business premises in all flood flows up to and including the average recurrence interval (ARI), Y = 100-years critical storm event. The capabilities of both networks are such that they are unlikely to suffer loss of integrity in the face of future development/re-development provided it is well ordered and falls within the requirements of the governing local agency.

These networks represent ‘model’ systems for designers of new developments and re-developments. It is suggested that stormwater flows in such catchments and sub-catchments could be well managed using the **yield-maximum** strategy in selected, small sub-catchments with potential for catchment-based stormwater harvesting if desired, along with the **regime-in-balance** strategy applied generally (see Section 4.2.1). The suggested ARI used for design of future on-site water-retaining devices in this case should be in the range 50-years to 100-years.

- **Medium-capacity major/minor networks :** These are associated with catchment-wide or sub-catchment-wide drainage installations incapable of satisfactorily controlling minor stormwater flows, located within a matrix of well-defined overland flow paths providing acceptable freeboard to floors of residential and business premises in all flood flows up to and including the ARI, Y = 100-years critical storm event. The minor system network is likely to provide an even lower level of flood protection as development/re-development proceeds unless it is well ordered and controlled by the municipal agency. The major system network is sufficiently robust to maintain a high standard of performance in the face of future development/re-development provided it is well ordered and falls within the municipal agency policies and requirements.

It is suggested that stormwater flows in such catchments and sub-catchments could be well managed using, principally, the **yield-minimum** strategy focussed on the minor system. The suggested ARI used for design of on-site water-retaining devices in this case could be in the range 2-years to 10-years, as appropriate. [This process will lead to a reduction in the standard of major system flood protection towards ARI, Y = 50-years.] In time, it may be found that the performance of the minor system, following this strategy, improves significantly enabling the network to be upgraded to **high-capacity** status with corresponding strategy change to that of **regime-in-balance** with ARI, Y = 50-years to 100-years.

- **Low-capacity major/minor networks :** These are associated with catchment-wide or sub-catchment-wide installations capable of satisfactorily controlling minor stormwater flows, located within a matrix of poorly-defined overland flow paths resulting in progressively more serious flooding of floors of residential and business premises in all flood flows above the ARI, Y = 10-years critical storm event. The minor system network is sufficiently robust to maintain a satisfactory level of performance in the face of future development/re-development provided it is well ordered and falls within municipal agency requirements. The major system network is likely to deteriorate even further in performance as development/re-development proceeds leading to increased flooding of floors of residential and business premises in rare storm events unless it is significantly upgraded and controlled by the municipal agency policies and requirements.

It is suggested that the local government agency, in these circumstances, should institute structural changes to the urban landscape including property resumption/purchase if necessary to provide space for overland flow paths, coupled with application of the **yield-minimum** strategy with suggested ARI, Y = 100-years for design of all on-site water-retaining devices.

[This process will lead to an increase in the standard of nuisance flood protection offered by the minor system towards or even above the ARI, Y = 10-years level.] In time, it may be found that the performance of the major system improves significantly enabling the network to be upgraded to **medium-capacity** status with corresponding **yield-minimum** ARI change from 100-years to 2 – 10-years for all development/re-development. Subsequent status change to that of **regime-in-balance** with ARI = 50-years to 100-years may follow.

- **Poor-capacity major/minor networks :** These are associated with catchment-wide or sub-catchment-wide installations incapable of satisfactorily controlling minor stormwater flows, located within a matrix of poorly-defined overland flow paths resulting in progressively more serious flooding of floors of residential and business premises in all flood flows above the ARI, Y = 10-years critical storm event. The capabilities of both networks are likely to deteriorate even further in the face of any future development/re-development resulting in increased nuisance flooding and increased flooding of floors of residential and business premises unless both networks are significantly upgraded and controlled by municipal agency requirements.

It is suggested in this case that the local government agency undertakes a comprehensive review of the entire storm drainage system and takes action, including property resumption/purchase if necessary to provide space for overland flow paths. This action should be accompanied by application of the **yield-minimum** strategy with suggested ARI, Y = 100-years for design of all on-site water-retaining devices. In time, it may be found that the performances of both the major and minor networks, following this strategy, has improved significantly enabling the entire system to be upgraded to **medium-capacity** status with corresponding **yield-minimum** ARI change from 100-years to 2 – 10-years for all development/re-development. Subsequent status change to that of **regime-in-balance** with ARI = 50-years to 100-years may follow.

4.6.2 The four networks : management issues and ARIs

The four networks of flood management described above represent the full range of relationships possible between major and minor stormwater flow scenarios in the urban landscape. The history of flooding and/or the environmental performance (for example, with respect to channel erosion) of particular networks in significant storms of critical or near-critical duration will provide the data base needed by a municipal agency to divide catchments and sub-catchments into the four classifications.

With this determined, the local government agency can then institute a coherent strategy of management to maintain the status of well-performing networks and progressively upgrade those which are deficient. This (latter) action can be taken, not necessarily through expensive infrastructure upgrades using scarce community resources, but through policy instruments and the approval process. This should certainly be the case for networks described as “medium-capacity” and, possibly, for some classified as “low capacity”. Networks classified as “poor-capacity” (and some of “low-capacity”) are likely to need a concentration of available resources to pay for significant structural (including property resumption/purchase) improvement of the stormwater infrastructure. However, the classifications presented in Section 4.6.1 should aid this process.

Experienced practitioners may be surprised by the ARIs suggested for design of new installations (development and re-development) which appear to be outside minor system practice (typically, 2 – 10 years). For example, some **regime-in-balance** strategy applications may require on-site storages to be sized to ARI, Y = 100-years [**after** development *minus* **before** development (runoff volumes)]. These lead, usually, to storage volumes **less** than those given by the **yield-minimum** strategy with ARI, Y = 10-years.

4.6.3 The ‘benchmark year’ concept

A municipal agency responsible for managing the water resources and environmental qualities within a defined basin may employ the strategies outlined above as follows :

STEP 1 : **Classify** each catchment and sub-catchment within its jurisdiction according to the four drainage network categories described in Section 4.6.1. Acceptable average recurrence interval (ARI) and **critical storm duration** for each catchment or sub-catchment emerges from this process;

STEP 2 : **Identify** which of the three development/re-development strategies described in Section 4.2.1 should be applied to each catchment or sub-catchment. In cases where the overland flow path is grossly inadequate (‘low-’ and ‘poor-capacity’ networks), then separate strategies of resumption/purchase of properties should be applied.

It is simplest in terms of data acquisition and model validation etc., to conduct the analyses required under STEP 1 (see Section 4.6.4, following) to catchments and sub-catchments in their present states of development. But they (the analyses) need not be so constrained. It is quite reasonable – but more difficult – to adopt a stage of basin development earlier than ‘present’, at a time when the impact of development in terms of flooding or stream erosion in the critical (duration) storm is considered, retrospectively, to be ‘acceptable’. The time of this ‘acceptable performance’ – decided by consensus among the stakeholders – is termed the **benchmark year**.

It is important to recognise that the decision to choose the ‘present’ as the benchmark year fixes the time of ‘acceptable performance’ just as rigidly as if 1980 or 1990 were chosen. Hence, in applying **the regime-in-balance** strategy to design (see Section 4.6.5, following), the status of the site **before** development or re-development must be literally “as it was in the benchmark year”, **not** the time when the development/re-development happens to be taking place.

Comparison of the outcomes of hydrological analyses carried out for ‘present’ development and for the benchmark year will reveal, quite dramatically, how urbanisation has impacted, particularly where the time gap is a decade or more. For example, a major/minor network of interest may be classified (retrospectively) as having medium-capacity in the benchmark year; over the intervening period (to the present) this classification may have degraded to low-capacity. The correct strategic approach to be taken in such a situation would therefore be to assess present drainage networks and environmental qualities against the behaviour/performance of corresponding infrastructure in the benchmark year, and arrive at policy decisions accordingly.

4.6.4 Summary of tasks for a stormwater management agency

Primary task

Commission a comprehensive hydrological study as well as associated community action to achieve the following aims :

- determine a “point of singular interest” in each significant drainage unit (catchment or sub-catchment) where flood risk or environmental damage (channel scour) is readily assessable. This task involves gathering together the wealth of available information – formal records as well as anecdotal information – that is available in the community;
- determine peak flood flows (and hence peak stages) for a range of frequencies (ARIs) at each nominated point of singular interest;
- establish, through consensus among community stakeholders, acceptance of a limit of flooding and/or environmental damage together with an associated frequency (ARI), applicable to each point of singular interest. A decision involving ‘retrospectivity’ (the ‘benchmark year’) is permitted;
- review and classify the major/minor drainage networks in each component catchment and sub-catchment according to the four categories described in Section 4.6.1, together with appropriate ARIs, advised by the consensus decision, above;
- identify the critical storm duration appropriate to each component catchment and sub-catchment and its associated point of singular interest.

With these analyses, records and community decision-making in hand, the municipal agency can then proceed with the following tasks :

1. Determine which of the three scenarios/strategies (a, b or c) described in Section 4.2.1 most closely fits their situations and/or what balances between them should be observed. The **type** of hybrid policy which might emerge from such considerations is :

“**yield-minimum** strategy to be applied to all new housing on “greenfields” and previously undeveloped sites; **regime-in-balance** strategy to be applied to all other cases of development and re-development (see Section 4.2.1 and Examples 5.6 and 5.7)”.
2. List values of **critical storm duration** which should be applied in each catchment, $(T_C)_{total}$, and/or sub-catchment, $(T_C)_{local}$, identified in the basic hydrological investigation. As an interim measure, the familiar ‘time of concentration’ concept may be used, but this tends to under-estimate the critical storm duration determined by comprehensive hydrological analysis. Use of the appropriate Bransby Williams formula as explained in Sections 5.3 and 5.4 of “AR&R – 1987” (IEAust, 1987) provides simple yet acceptable estimates of $(T_C)_{total}$ or $(T_C)_{local}$.
3. Identify the values of average recurrence interval (ARI) which should be applied to drainage networks, taking account of the types of development which have taken place and are likely to take place in the area of jurisdiction. Clear policy decisions identifying council-adopted levels for both the minor and major systems should be made (see Section 4.2.6).
4. Identify acceptable emptying time criteria for application to “leaky” devices and installations temporarily storing stormwater on-site (see Table 3.3);
5. Determine policy on the two pollution containment strategies – Strategy A and/or Strategy B (see Section 4.3) – and how they should be interpreted on development/re-development sites. For example, it may be appropriate for Strategy A to be required in all development/re-development sites greater than 1.0 ha, and Strategy B in all small developments.
6. Determine if a catchment or sub-catchment-wide stormwater harvesting strategy should be followed. This requires detailed analyses of catchment characteristics linked to policies governing storm runoff management – retention or detention – as well as investigation of potential constructed wetland sites and ASR opportunities (see **yield-maximum** strategy, Section 4.2.1).

4.6.5 The regime-in-balance strategy (see Section 4.2.1)

Interpretation of the **regime-in-balance** strategy – as specified by Council – in relation to a new development or re-development involves design of the three main elements of on-site devices and installations (see Section 4.2.6) :

- entry works peak flow – based on site t_C ;
- OSR device storage volume – based on $(T_C)_{total}$, $(T_C)_{local}$ or t_C ;
- exit works peak flow : for simplicity, same as entry works peak flow.

The first and third of these elements may be determined by normal Rational Method procedures or more sophisticated techniques, as preferred, applied to the appropriate (site) contributing area, using council-advised ARI, local rainfall data, etc. The second element – on-site storage – must be designed to ensure that ‘before and after’ (development or re-development) runoff flows from the site are as close to equal as possible. This equality of flows should be interpreted in terms of **volume** of site runoff in the Y-years storm of critical duration as determined by council, **not** peak flow. Compliance involves the designer in obtaining the following information :

DEVELOPMENT (OF “GREENFIELDS” SITE)	OR	RE-DEVELOPMENT (OF DEVELOPED SITE)*
area of “greenfields” site, A_o		area of developed (present) site, A_{dev}
runoff coefficient for “greenfields” site, C_g		runoff coefficient for developed site, C_{dev}
runoff coefficient for “greenfields” site, C_{dev}		runoff coefficient for re-developed site, C_{red}
developed site ‘time of concentration’, t_c		re-developed site ‘time of concentration’, t_c
critical storm duration, T_C		critical storm duration, T_C
design average recurrence interval, Y		design average recurrence interval, Y
rainfall information for site location		rainfall information for location

* “Developed site” here means : the site as it was in the benchmark year (see Section 4.6.3).

These data are then applied into the following steps to determine the ‘before and after’ runoff volumes and their difference(s) :

- STEP 1 :** Identify the storm duration, T_C , which is **critical** for flooding at the location of singular interest, P, Q etc. in the ARI = Y-years event (see Sections 4.2.1, 4.2.3, 4.2.4 and Figure 4.1).
- STEP 2a :** Determine for any **development** application, the quantity (volume) of stormwater discharged from the undeveloped (“greenfields”) site in the ARI = Y-years design storm of duration T_C using parameters A_o , C_g , etc.
- STEP 2b :** Determine for any **re-development** application, the quantity (volume) of stormwater discharged from the developed (benchmark year) site in the ARI = Y-years design storm of duration T_C using parameters A_{dev} , C_{dev} , etc.
- STEP 3a :** Determine for any **development** application, the quantity (volume) of stormwater discharged from the developed site in the ARI = Y-years design storm of duration T_C using parameters A_o , C_{dev} , etc.
- STEP 3b :** Determine for any **re-development** application, the quantity (volume) of stormwater discharged from the re-developed site in the ARI = Y-years design storm of duration T_C using parameters A_{dev} , C_{red} , etc.
- STEP 4a :** For any **development** application, determine the **difference** in the volumes required under 2a and 3a, above : this becomes the parameter ∇ .
- STEP 4b :** For any **re-development** application, determine the **difference** in the volumes required under 2b and 3b, above : this becomes the parameter ∇ .
- STEP 5 :** Require the developer or re-developer to provide on-site stormwater retention facilities to fully retain* **without overflow**, the quantity ∇ , determined in STEPS 4a or 4b, respectively, above. Delivery of the calculated volume to the OSR installation or device takes place over a time period $\tau = (T_C + t_c)$ [see Section 3.3].

The volume ∇ and the runoff hydrograph time base τ are the primary parameters used to design OSR installations and devices for ‘source control’ stormwater management (Procedures 2, 3 and 4, Section 5.1). It is presumed in the procedure listed in the five steps, above, for the **re-development** case, that the (b) scenario described in Section 4.2.1 – that of a waterway in harmony with its urban environment – applies to development in the benchmark year.

* A small outflow downstream from a retention facility *during the filling process and subsequently* may occur – by design – in cases where Procedure 4 is employed (see Chapter 5).

4.6.6 The yield-minimum strategy (see Section 4.2.1)

The procedure for interpreting the **yield-minimum** strategy into design of OSR installations or devices in a changing urban landscape follows the same pattern set out for the **regime-in-balance** strategy presented in Section 4.6.5 with two important differences. These are :

Development (of “greenfields” site) case :

STEP 2a : $C_g = 0$ leading to a zero outcome for all cases in STEP 2a.

Re-development (of developed site) case :

STEP 2b : $C_{dev} = 0$ leading to a zero outcome for all cases in STEP 2b.

4.7 MIMICKING NATURAL CATCHMENT ‘LOW FLOWS’ IN THE URBAN LANDSCAPE

The quest to achieve mimicking of natural catchment behaviour in an urban landscape must incorporate the volume-equality (flooding) principle reviewed in Section 4.6.5. And it is tempting to assume that by satisfying this important principle, alone, an acceptable environmental flows regime will automatically follow. This is not so because the **frequency bases of the two cases are significantly different**: *upper flow (flood)* frequencies are focussed on 5-year, 10-year, 20-year etc ARIs; the *low flow regime* corresponds to peaks in the ARI range “3-months”, “2-months”, “1-month” etc associated with 90 – 70 percentile average annual flows. Furthermore, a focus on satisfying the (upper flow) volume-equality principle, alone, poses the threat of *reducing waterway discharges below the needs of environmental flows*. Recognition of this threat is of major importance since it is the ‘low flow’ regime which determines the fate of aquatic bio-communities – both fauna and flora (see Smakhtin, 2001; Lee et al, 2008; Walsh et al, 2009).

Research aimed at resolving the tension between the requirements of the two flow regimes – *upper flows* and *low flows* - is in its infancy, but the following brief analysis **focussed on ‘low flow’ mimicking considerations only** is offered as a pathfinding contribution to this domain of water-sensitive urban design.

The principal difference between natural and developed catchments is, of course, the replacement of pervious land surface with paving – roofs, carriageways, etc. On any developed site there is – in general – an area of pervious surface and an area of paving. We can assume that the hydrological behaviour of the *pervious* component is acceptably similar in both the natural and developed catchments. It therefore remains only for surface runoff from the *impervious* areas to be managed so that the **yield** (average annual runoff volume) and **spell characteristics** of each area are much the same in the developed catchment as they were (from the same area) in the pre-developed state (see Hewa et al, 2009). [A ‘spell’, in the present context, is a period of time during which successions of *low flows* (including ‘no flow’ episodes) passing from a catchment component, *fall below an identified ‘threshold’ value* (see below).]

If a developed catchment is to mimic natural (catchment) processes and behaviour, then it must actively simulate – as closely as practicable – **four rainfall destinies** of particular interest, illustrated in Figure 1.7 :

1. events which satisfy interception loss, only, and deliver *no input to the forest floor*;
2. events which satisfy interception loss and ‘take-up’ by tree roots, and deliver *no input below the root zone*;
3. events which satisfy interception loss and ‘take-up’ by tree roots, and also deliver *a significant seepage flow* to the catchment waterways (“base flow”) and to deep aquifers; and,
4. events which satisfy (1), (2) and (3), above, and deliver, in addition, *a small surface runoff*.

It is impossible to apply universal magnitudes to these levels of events because they vary so greatly between catchment types (vegetation, terrain, soils, etc) and climates across Australia. However, ‘notional’ values can be assigned to them in keeping with the spirit of the Figure 1.7 model as follows :

- around 20% of annual rainfall input, for item 1;
- around 40% of annual rainfall input for the *tree roots ‘take-up’* component of item 2;
- around 20% of annual rainfall input for the *seepage flow* component of Item 3; and,
- waterway contribution (surface and baseflow) up to a ‘threshold’ *low flow* peak value of, say, $Q_{1\text{ month}}, Q_{3\text{ months}}, Q_{1\text{ year}}$, etc. for Item 4.

The types of urban catchments in which the suggested ‘mimicking’ should take place are not universal. Inner-city and old suburbs with well-established, conventional stormwater infrastructures are specifically excluded on the ground that any presence of aquatic bio-communities requiring ‘low flows’ for their survival vanished decades ago. So the scope of the present initiative is ‘greenfields’ developments (see Section 1.4.2) and urban catchments where rejuvenation of drainage lines is a considered option (see Section 1.4.3). In both of these, residential development is likely to be the predominant if not the only land use. Furthermore, the residential roof represents a logical ‘first option’ for implementing a conscious strategy of low flow mimicking available in the urban landscape. How this might be accomplished is explained as follows :

Figure 1.1b presents the layout of a dual-occupancy allotment typical of those constructed in recent time in the suburbs and ‘greenfield’ estates of Australian main population concentrations. This layout has been used to explore its role in the mimicking process. The principal physical and occupancy characteristics of the layout – reduced to that of a **single** dwelling unit are :

- roof area, 250 m² for which *effective* (catchment) area is 80-90% or 210 m²;
- share of fronting carriageway/verge : 75 m² (equivalent impervious area);
- impervious area runoff coefficient (annual volumetric, incorporating initial loss) : $C = 0.75$;
- allotment (outdoor) **permeable** paving – driveway, outdoor living area, etc. : 150 m²;
- irrigated lawn/garden area : 110 m²;
- occupancy : 2.5 persons;
- daily (in-house) water use : 140 litres per person;

Application of the ‘continuous simulation’ modelling software for raintank sizing listed in Section 8.6.5 to these data for Australian climate zones produces the following results :

Tropical Northern Zone: It is impractical to use rainwater tanks in Darwin to achieve the ‘low flow’ mimicking objectives specified above because the required tanks would need to be immense. However, the goals can be achieved in Brisbane with tanks of capacity 25 – 30 kL where in-house demand *only* is met, and 13 kL where both in-house and outdoor irrigation is supplied. These storages are, also, greater than the typical householder is likely to tolerate within the confines of a medium-density residential allotment.

Intermediate Australia Zone: It is impractical (immense storages) to use rainwater tanks in Sydney to achieve ‘low flow’ mimicking objectives where in-house demand *only* is met from a rainwater tank. However, a tank of 22 kL capacity will deliver the goals of mimicking provided the harvested water is used for both in-house supply and outdoor irrigation. Full mimicking can be achieved in both Perth and Canberra with 16 kL and 2.8 kL tanks, respectively, meeting in-house *only* demand, and 12.5 kL and 2.7 kL storages, respectively, where both in-house and irrigation demands are catered for. Typical householders could not be expected to install rainwater tanks of the sizes indicated for Sydney or Perth, but ‘low flow’ mimicking based on tanks of acceptable size is feasible in Canberra.

Southern Australia: ‘Low flow’ mimicking in urban catchments, meeting the objectives listed above, is entirely feasible in all of the nation’s southern cities – in particular, Adelaide, Melbourne and Hobart (in addition to Canberra) – where rainwater tanks of acceptable size (2.0 – 2.5 kL) are required.

Some important conclusions may be drawn from this brief exploration of the potential for ‘low flow’ mimicking in ‘greenfields’ catchments and urban landscapes with rejuvenated waterways :

1. Mimicking objectives **can** be achieved through judicious choice of rainwater tank storages in Canberra and the southern Australian population concentrations;
2. Mimicking objectives **cannot** be achieved with domestic rainwater tanks of acceptable size in Australia’s high rainfall regions including Sydney and Perth; centralised facilities such as swales and mini-wetlands are needed to achieve the objectives in these locations;
3. Rainwater tanks with installed capacity greater than 2.5 kL in southern Australia (a not uncommon practice) are likely to impact adversely on environmental flows in local remnant waterways and rejuvenated streams.

5 RUNOFF QUANTITY CONTROL (Category 1 systems)

5.1 FOUR BASIC PROCEDURES

5.1.1 Introduction

There are three basic types of devices/installations used in runoff retention practice. These are :

1. **Infiltration or ‘treatment’ surfaces** (receiving runoff with no ponding) : design addressed under Procedure 1,
2. **“Leaky” devices and “soakaways”** with pre-treatment : design addressed under Procedure 2; and,
3. **Ponds with infiltration** : design addressed under Procedure 3.

A further element of runoff retention practice that deserves to be included among ‘basic’ procedures, is that relating to **slow-release** of water retained by the latter two types of devices/installations, introduced in Section 3.5.4. This component is included as Procedure 4.

Swales and wetlands are excluded from these basic, Category 1, procedures because their primary function is pollution control rather than quantity control. [Simple swales – called ‘filter strip’ swales in the Handbook – are considered in Chapter 7.] No consideration is given to constructed wetlands in the Handbook on the ground that they are, typically, beyond the scope of stormwater ‘source control’.

The procedures, following, for designing the three categories of devices listed above owe much to Professor Wolfgang Geiger and the German ATV (Abwassertechnische Vereinigung EV), as explained in the Preface. They are subdivided into two groups : those based on Q_{peak} (infiltration or treatment surfaces without ponding), and those based on **volume of surface runoff**, V (“leaky” devices and ponds with infiltration). In all cases where volume, V , dominates the design process, the analyses which follow lead to final formulations embracing the ‘device full’ assumption, namely :

*At the end of the period of surface runoff to the device in the design event (τ in Figure 3.4), the entire available storage is filled in readiness for the **emptying from full** process to commence.*

In fact, due to the combined effects of soil permeability, design rainfall intensity, critical storm duration, etc., this assumption is often invalid and the ‘device full’ stage is reached (and some emptying has taken place) **before** the end of the period of surface runoff, τ . Coincidence of the ‘device full’ condition with the end of the period of surface runoff used in the analyses is, therefore, **conservative** in such cases : more detailed mathematical modelling (runoff routing) is required to derive advantage from such departures from the simple approach presented below.

5.1.2 The Procedure 1 cases

The basis of the Procedure 1 cases is simple infiltration of storm runoff received from a paved or impervious area, such as a roof or bitumenised surface. They are included under Category 1 because quantity control (of peak flow rate) is identified as the primary objective; but, typically, pollution control is also involved. The widest potential use likely to be made of Procedure 1 is in the design of porous and permeable paving systems for car parks and roof areas draining directly to these spaces. The design process for such installations involves use of Guidelines 1 and 2 from Sections 3.1.2 and 3.1.3 respectively.

The principles which under-pin Procedure 1 are also employed in Chapter 7 of the Handbook to design ‘filter strip’ swales whose primary function is pollution control (Category 2 installations).

Two configurations of impermeable and porous/permeable paving need to be considered :

- those where the infiltration surface is *external* to the contributing impervious area; and,
- those where the infiltration (or treatment) surface is required to be *within* the total area.

Procedure 1A : Infiltration or treatment surface (without ponding) accepting runoff from roof or paved area, $A \text{ m}^2$, and external to it

The basic data requirements are (see Sections 4.2 and 4.6) :

- peak flow, $Q_{\text{peak}} \text{ m}^3/\text{s}$ passing to the infiltration or treatment surface, $A_s \text{ m}^2$; the analysis must take account of rainfall input to the surface A_s , itself;
- The choice of design storm duration, t_c , and ARI used to calculate Q_{peak} is the responsibility of the designer operating in consultation with Council;
- hydraulic conductivity of infiltration surface (porous or permeable paving), $k_h \text{ m/s}$;
- average rainfall intensity, $i \text{ mm/h}$ for design storm of duration t_c ;
- infiltration or treatment surface blockage factor, ψ ;
- paved area runoff coefficient, C .

$$\text{Total peak inflow into infiltration or treatment surface} = \frac{(CA)i}{10^4 \times 360} + \frac{A_s \cdot i}{10^4 \times 360} \text{ m}^3/\text{s}$$

$$\text{Flow (acceptance) capacity of surface} = (1-\psi) A_s \cdot k_h \text{ m}^3/\text{s}$$

hence,

$$\begin{aligned} \frac{(CA)i}{10^4 \times 360} &= (1-\psi) A_s \cdot k_h - \frac{A_s \cdot i}{10^4 \times 360} \\ &= A_s \left[(1-\psi) k_h - \frac{i}{3.6 \times 10^6} \right] \end{aligned}$$

hence,

$$\begin{aligned} A_s &= \frac{(CA)i}{3.6 \times 10^6 \left[(1-\psi) k_h - \frac{i}{3.6 \times 10^6} \right]} \\ &= \frac{Q_{\text{peak}}}{\left[(1-\psi) k_h \cdot U - \frac{i}{3.6 \times 10^6} \right]} \text{ m}^2 \end{aligned} \quad (5.1)$$

where $U = 1.0$.

* Soil hydraulic conductivity values are obtained from site tests on **small** test pits and boreholes, typically. When the results of these tests are applied to design infiltration surfaces or on-site stormwater retention devices, it is found that the systems or devices are “too big”, where site soil is clay, and “too small” where the soil is sandy (see Bressan, 1996). This observation has led to the introduction of a correction factor, **Moderation Factor, U**, which should be applied to hydraulic conductivity, k_h , in the formulae which follow :

In clay soils, $U = 2.0$ should be used; in sandy clay soils, $U = 1.0$; in sandy soils, $U = 0.5$;

$U = 1.0$ may also be used with **long-term** hydraulic conductivity, k_h , where this is known or estimated for the ‘lifespan’ of the system (see Sections 3.1.2 and 3.1.3).

Some additional points of explanation are :

- Q_{peak} in Eqn. (5.1) is calculated for the *total* contributing paved or roof surface area $A \text{ m}^2$, **only**;
- Area A_s determined from Eqn. (5.1) is the **(total) pervious** surface area of the facility, including – where applicable – space taken up by lattice blockwork, plastic rings, trees, etc. (see Figure 3.1a) : the factor $(1 - \psi)$ in Eqn. (5.1) takes account of the presence of these elements. In **porous or permeable paving** cases, area A_s is determined using an appropriate hydraulic conductivity, k_h , for example its long-term value (see Guideline 1, Section 3.1.2); systems designed according to this approach, use $\psi = 0$ in Eqn. (5.1).
- $U = 1.0$ is used for design, based on **long-term** hydraulic conductivity value, known or estimated (see Footnote, above);
- Every result given by Eqn. (5.1) should be followed by application of Guidelines 1 and 2 (Sections 3.1.2 and 3.1.3, respectively) to determine expected ‘lifespan(s)’. **Two** estimates arise in the case of permeable paving (geotextile layer and surface), and **one** in vegetated porous systems (surface only);
- See also the application of Procedure 1A or 1B in Example 7.2, Section 7.7.5.

Procedure 1B : Infiltration or treatment surface (without ponding) accepting runoff from paved area and located *within* the defined site area, $A \text{ m}^2$

The basic data requirements and theory correspond to those given for case 1A, above.

Total peak inflow into treatment surface

$$= \frac{(CA)i}{10^4 \times 360} \text{ m}^3/\text{s}$$

Flow (acceptance) capacity of treatment surface

$$= (1-\psi)A_s.k_h \text{ m}^3/\text{s}$$

hence,

$$\begin{aligned} A_s &= \frac{(CA)i}{3.6 \times 10^6 (1-\psi).k_h} \\ &= \frac{Q_{\text{peak}}}{(1 - \psi).k_h.U} \text{ m}^2 \end{aligned} \quad (5.2)$$

where $U = 1.0$.

- Q_{peak} in Eqn. (5.2) is calculated for the defined (total) **site** area $A \text{ m}^2$;
- All other points of explanation included under Procedure 1A, above, with respect to the factor $(1 - \psi)$, long-term value of k_h in porous/permeable paving systems, ψ in porous/permeable systems, U and use of Guidelines 1 and 2 (Sections 3.1.2 and 3.1.3), apply equally to Procedure 1B.

5.1.3 The Procedure 2 cases

The broad class of on-site retention installations described as “leaky” devices and “soakaways” includes two basic sub-categories :

- devices which provide unencumbered, well-type storage space; and,
- trench-shaped devices which provide storage space, part-occupied by materials such as gravel or plastic.

Each of these types of devices (see Figure 1.3) releases stored water into the surrounding soil medium by percolation; they also incorporate provision for overflows (see Figures 1.3 and 1.5) and, possibly, for slow-drainage (see Figure 1.4 and Procedure 4).

There is an important constraint or condition which must be observed in all Procedure 2 devices and installations, particularly those which involve gravel or sand as the water-retaining medium (see Section 1.3.3). This concerns the quality of water input, described as ‘cleansed water’. To be acceptable, this water should have TSS ≤ 20 mg/L **and** maximum particle size ≤ 20 μ m (see Section 7.1).

Procedure 2A : “Leaky” well with cleansed water inflow, typically, from a roof

The basic data requirements are (see Sections 4.2 and 4.6) :

- “Block” runoff volume, $\forall$ m³, passing to well from roof. Normal hydrological processes are used to estimate $\forall$; storm durations equal to $(T_c)_{total}$, $(T_c)_{local}$ and t_c should be considered (see “The Parameter, τ , and design storm duration”, Table 5.1);
- The choice of design ARI used to calculate $\forall$ is the responsibility of the designer operating in consultation with Council;
- hydraulic conductivity of soil, k_h m/s.

Design Assumptions :

- well is empty at commencement of inflow;
- well fills over time τ minutes (see Figure 3.4);
- percolation through floor is at full rate of k_h for period of τ minutes;
- percolation through walls is distributed hydrostatically, hence this component of outflow is **half** saturated (outflow) rate for period of τ minutes;
- the perforated wall offers no restriction to outflow;
- groundwater level is significantly below the floor of the well.

For perforated well, diameter D m, height H m :

$$\text{Inflow (volume) to well} = \forall \text{ m}^3$$

$$\text{volume of well} = \frac{\pi D^2}{4} \cdot H$$

$$\text{outflow from well floor} = \frac{\pi D^2}{4} \cdot k_h \cdot 60 \tau$$

$$\text{outflow from well wall} = \frac{1}{2} \left[\pi D H \cdot k_h \cdot \frac{60 \tau}{2} \right] = 15 \pi D H \cdot k_h \cdot \tau$$

Practical cases (particularly in clay soils) usually result in $D \approx H$: **this simplification is adopted.**

Hence, diameter of well, D :

$$D \approx \sqrt{\frac{\forall}{\frac{\pi}{4} (H + 120 k_h \cdot \tau)}} \text{ m}$$

Incorporating the Moderation Factor, U, this formula becomes :

$$D = \sqrt{\frac{\forall}{\frac{\pi}{4} \left(H + 120 k_h \cdot \tau \cdot U \right)}} \text{ m} \quad (5.3a)$$

where $U = 0.5$ for sandy soil; $U = 1.0$ for sandy clay; $U = 2.0$ for clay soil.

There is another form of this equation which is useful in designing “leaky” well installations :

$$H = \frac{4(\forall - 30\pi k_h \cdot \tau D^2 U)}{\pi D^2} \quad (5.3b)$$

Important Note : Every design application of Procedure 2A must be followed by a check on emptying time, T , in accordance with the requirements of the *modified* design storm method (see Section 3.5.4).

Procedure 2B : Trench, part-occupied with impervious material, pipes, etc., receiving cleansed water inflow

The basic data requirements and design assumptions correspond to those given for case 2A, above. For trench of length L , width b , and depth H , part-occupied with impervious material (gravel, plastic, perforated pipes, etc.) and providing void space, e_s , where :

$$e_s = \frac{\text{void space available}}{L b H}; \quad e_s \text{ is identical to porosity, } n, \text{ used in geotechnical engineering.}$$

Typical values of the ratio e_s are 0.35 for gravel and 0.95 for certain “milk crate” plastic units. Values of e_s for trenches part-occupied by perforated pipes range from 0.5 to 0.75 depending on pipe sizes and trench cross-section dimensions (see Dierkes et al, 2002).

Inflow volume to system = $\forall$ m³; storm durations $(T_C)_{\text{total}}$, $(T_C)_{\text{local}}$ and t_C should be considered (see “The Parameter, τ , and design storm duration”, Table 5.1) :

$$\text{volume of trench void space} = e_s \cdot LbH$$

$$\text{outflow from trench floor} = 60 Lb \cdot k_h \cdot \tau$$

$$\text{outflow from trench walls} = \frac{1}{2} \left[2(L + b) H \cdot k_h \cdot \frac{60\tau}{2} \right] = 30 (L + b) H \cdot k_h \cdot \tau$$

Ignoring outflow from ends of trench :

$$L = \frac{\forall}{\left[e_s \cdot bH + 60 k_h \cdot \tau \left(b + \frac{H}{2} \right) \right]} \text{ m}$$

Incorporating the moderating parameter, U , this formula becomes :

$$L = \frac{\forall}{\left[e_s \cdot bH + 60 k_h \cdot \tau \left(b + \frac{H}{2} \right) U \right]} \text{ m} \quad (5.4a)$$

where $U = 0.5$ for sandy soil; $U = 1.0$ for sandy clay; $U = 2.0$ for clay soil.

There are two other forms of this equation which are useful in designing trench systems :

$$H = \frac{\Psi - 60k_h.L\tau bU}{e_s Lb + 30k_h.L\tau U} \quad (5.4b)$$

$$b = \frac{\Psi - 30k_h.L\tau HU}{e_s LH + 60k_h.L\tau U} \quad (5.4c)$$

Important Note : Every design application of Procedure 2B must be followed by a check on emptying time, T, in accordance with the requirements of the *modified* design storm method (see Section 3.5.4).

Procedure 2C : “Soakaway” part-occupied with impervious material, pipes etc., receiving cleansed water inflow

Application of Procedure 2B in soils of very low permeability, notably heavy clays, yields trenches of impractical length. Such cases are better constructed as “soakaways”, that is, trenches of various types with length, L, approximately equal to width, b. Otherwise, the basic data requirements and theory correspond to those given for cases 2A and 2B, above.

Inflow volume to system = Ψ m³; storm durations $(T_C)_{total}$, $(T_C)_{local}$ and t_C should be considered (see “The Parameter τ , and design storm duration”, Table 5.1).

Volume of “soakaway” void space = $e_s.aH$

where a = plan area with $L \approx b$

Outflow from “soakaway” floor = $60 a.k_h.\tau$

Ignoring outflow from “soakaway” walls :

$$a = \frac{\Psi}{(e_s .H + 60 k_h .\tau)} \text{ m}^2$$

Incorporating the moderation factor, U, this formula becomes :

$$a = \frac{\Psi}{(e_s .H + 60 k_h .\tau .U)} \text{ m}^2 \quad (5.5a)$$

where $U = 0.5$ for sandy soil; $U = 1.0$ for sandy clay; $U = 2.0$ for clay soil.

The magnitude of “a” can be satisfied with any combination of $L.b = a$, but length, L, and width, b, should be as close as possible to being equal. There is another form of Eqn. (5.5a) which is useful in designing “soakaways” :

$$H = \frac{\Psi - 60k_h.a\tau U}{a e_s} \quad (5.5b)$$

Important Note : Every design application of Procedure 2C must be followed by a check on emptying time, T, in accordance with the requirement of the *modified* design storm method (see Section 3.5.4).

5.1.4 Infiltration or ‘dry’ ponds

Procedure 3 : Pond with infiltration, also called a ‘dry’ pond

The basic data requirements are (see Sections 4.2 and 4.6) :

- “Block” runoff volume, ∇m^3 passing to pond site; storm durations $(T_C)_{\text{total}}$, $(T_C)_{\text{local}}$ and t_C should be considered (see “The Parameter, τ , and design storm duration”, Table 5.1);
- the choice of design ARI used to calculate ∇ is the responsibility of the designer operating in consultation with Council;
- **long-term** hydraulic conductivity of pond floor soil, k_h m/s;
- pond design depth, d m, usually determined by safety considerations;
- critical storm duration for this case is expressed as $(\tau - t_C)$ minutes; this term effectively identifies the three **alternative** storm durations $(T_C)_{\text{total}}$, $(T_C)_{\text{local}}$ and t_C which should be considered;
- average rainfall intensity, i mm/h, for design storm;

Design assumptions :

- pond is empty at commencement of inflow;
- pond fills over time equal to τ minutes (see Figure 3.4);
- percolation through floor of pond, area A_p m^2 , is at full rate of k_h for period of τ minutes;
- percolation through wall of pond is negligible;
- pond design storage volume, ∇_p m^3 represents the maximum quantity of runoff which can be stored temporarily as ‘open water’ at the pond site;
- the area used to determine ∇ is the contributing catchment area, only, but the analysis of **total inflow to the pond must include rainfall on pond area A_p** , in addition to that on the contributing catchment area;
- groundwater level is significantly below the floor of the infiltration pond system.

For pond with surface area A_p m^2 and design storage volume ∇_p m^3 :

$$\text{Runoff volume entering pond} = \nabla$$

$$\text{Volume of pond} = A_p \cdot d, \text{ where } d = \text{design depth in metres}$$

$$\text{Outflow from pond floor} = A_p \cdot k_h \cdot 60\tau$$

$$\text{Rainfall volume entering pond} = A_p \cdot \frac{i \cdot (\tau - t_C)}{1000 \times 60}$$

The term $(\tau - t_C)$ effectively identifies the three alternative storm durations which should be considered in the design process (see Table 5.1).

Hence,

$$\nabla = A_p \cdot d - \frac{A_p \cdot i \cdot (\tau - t_C)}{60 \times 1000} + A_p \cdot k_h \cdot 60\tau = A_p \left[d + 60k_h \cdot \tau - \frac{i \cdot (\tau - t_C)}{6 \times 10^4} \right]$$

hence,

$$A_p = \frac{V}{\left[d + 60k_h \cdot \tau - \frac{i \cdot (\tau - t_c)}{6 \times 10^4} \right]} \text{ m}^2$$

Incorporating the moderation factor, U, this formula becomes :

$$A_p = \frac{V}{\left[d + 60k_h \cdot \tau \cdot U - \frac{i \cdot (\tau - t_c)}{6 \times 10^4} \right]} \text{ m}^2 \quad (5.6a)$$

where U = 1.0.

There is another form of this equation which is useful in designing “dry ponds” :

$$d = \frac{V}{A_p} - 60k_h \cdot \tau \cdot U + \frac{i \cdot (\tau - t_c)}{6 \times 10^4} \quad (5.6b)$$

Important Note : Every design application of Procedure 3 must be followed by a check on emptying time, T, in accordance with the requirement of the *modified* design storm method (see Section 3.5.4).

This technology requires pond site soil permeability to be in the range medium to high, that is $k_h \nless 1 \times 10^{-5}$ m/s which is the limit for sandy clay. Infiltration ponds of the type analysed here are **not recommended** in medium and heavy clays and constructed clays (see Section 1.3.4) if ‘natural’ drainage (percolation) is the only mode of emptying. This does not preclude the use of infiltration ponds in soils of low permeability, but such installations require (introduced) sand-bed bases 200 mm – 300 mm thick over “leaky” storage sub-structures (Procedures 2B, 2C, above), often with drainage assistance (see below) to enable emptying time criteria to be met.

5.1.5 The Procedure 4 cases

Introduction : There are two sets of circumstances which call for the application of Procedure 4. The **first** is where aquifer injection of cleansed stormwater contained in a “leaky” sub-surface device is proposed in order to meet a water resources objective (see Sections 1.3.5 – 1.3.7) : flood mitigation and/or water quality goals may also be involved. The **second** circumstance – more frequent than the first – arises where application of Procedures 2A, 2B or 2C, above, leads to designs which are very difficult or impossible to drain, by ‘natural’ drainage (percolation) alone, and satisfy required emptying time criteria (see Section 3.5.3).

The first case, called “Procedure 4A” in the following note, is an essential element in the design of aquifer recharge/retrieval (ASR) ‘source control’ schemes of the types described in Section 3.9 (see also Figure 1.4a).

The second case – “Procedure 4B” – might be the consequence of soil permeability being extremely low or site space limitations being too restrictive, or an emptying time requirement which is, perhaps, very conservative. In such circumstances it may be necessary to assist the emptying process by providing **slow-drainage** of stored water (see Figure 1.4b). Such provision is, in essence, the basis of on-site detention (OSD) practice: however, application of this technology to on-site retention (OSR) devices differs significantly in the length of time during which the slow-drainage process takes place. It is normal practice for OSD installations to empty in periods of four hours or less; “slow-drainage” is calculated to take place, purposefully, over periods of 12 hours or longer (see Table 3.3) : the practice has been recognised in US literature where it is referred to a “extended detention” (US Dept of Transportation, 1996). See Section 5.4.

Procedure 4 should **only** be acted upon after a preliminary design for an installation, dimensioned under the provisions of Procedures 2A, 2B or 2C, has failed to meet a required emptying time criterion and other options, e.g. changing the configuration of the installation or removing/using the retained water, including

ASR where possible, have been exhausted. This guideline applies equally to trenches and “soakaways” constructed beneath infiltration or ‘dry’ ponds located in soils of low permeability.

Procedure 4A : Slow-drainage with aquifer-access

This procedure involves the following steps :

- STEP 1 :** Establish by geological investigation or from otherwise available geotechnical data bases, the presence of a readily-accessible aquifer where a scheme for aquifer storage of cleansed storm-water without environmental damage can be established (see Figure 1.4a).
- STEP 2 :** Determine by field trial or otherwise a value for recharge rate per bore, q_r m³/s and, hence, the number of bores, n , required to empty the previously-designed device (Procedure 2A, 2B or 2C) in the required time (emptying time criterion, Table 3.3). Note that “ n ” is an integer. A recharge flow rate $Q_r = n \cdot q_r$ m³/s, should then be determined. [Recharge rate may be taken as 50% of bore yield (Pavlic et al, 1992)].
- STEP 3 :** Revisit the design of the “leaky” device (Procedures 2A, 2B or 2C), already prepared, using the recharge-corrected form of Eqns. 5.3a or b, 5.4a, b or c or 5.5a or b, as appropriate, listed below as Eqns. 5.7a or b, 5.8a, b or c; 5.9a or b, respectively. Application of these formulae leads to smaller installations with acceptable emptying times.

Procedure 4B : Slow-drainage with pipeline

The procedure involves the following steps :

- STEP 1 :** Determine by local terrain and property survey the potential for installing a slow-drainage pipeline with sufficient fall from the site to convey a small flow to a nearby storm drain or urban waterway (see Figure 1.4b).
- STEP 2 :** Determine the volume rate of flow needed to remove the stored water from the “leaky” device (already designed) in the required emptying time (emptying time criterion, Table 3.3). In the case of “leaky” devices this is water volume stored within the installation – taking account of gravel, etc. – divided by the required emptying time, T . This calculation leads, in each case, to a value for Q_r m³/s, the slow-drainage flow rate.
- STEP 3 :** Revisit the design of the “leaky” device (Procedures 2A, 2B or 2C) already prepared using the discharge-corrected form of Eqns. 5.3a or b, 5.4a, b or c; 5.5a or b, as appropriate, listed below as Eqns. 5.7a or b; 5.8a, b or c; 5.9a or b, respectively. Application of these formulae leads to smaller installations with acceptable emptying times.

Data requirements and design assumptions for the discharge-corrected equations :

- “Block” runoff volume, V m³, as per Procedure 2A, 2B or 2C as applicable; storm duration and parameter τ (see “The Parameter, τ , and design storm duration, Table 5.1), as applicable;
- design ARI as applicable;
- hydraulic conductivity of soil, k_h m/s, as applicable;
- device fills over time τ minutes (see Figure 3.4);
- percolation through floor is at full rate of k_h for period of τ minutes;
- exfiltration through walls and other assumptions, simplifications, etc., are the same as for corresponding analyses in Procedures 2A, 2B and 2C, above;
- groundwater level is significantly below the floor of the device in each case.

“Leaky” well with cleansed water inflow and outflow rate of q_r m³/s per well (see Procedure 2A) :

$$D = \sqrt{\frac{\Psi - 60q_r \cdot \tau}{\frac{\pi}{4}(H + 120k_h \cdot \tau \cdot U)}} \text{ m} \quad (\text{single bore case}) \quad (5.7a)$$

$$H = \frac{4(\Psi - 60q_r \tau - 30\pi D^2 k_h \cdot \tau U)}{\pi D^2} \text{ m} \quad (\text{single bore case}) \quad (5.7b)$$

Note that q_r may be replaced by Q_r in both equations 5.7a and 5.7b where slow-drainage by pipeline is employed.

Trench, part-occupied with impervious material, cleansed water inflow and outflow rate of Q_r m³/s (see Procedure 2B); note that $Q_r = n \cdot q_r$ may be an option to be considered :

$$L = \frac{\Psi - 60Q_r \cdot \tau}{\left[e_s \cdot bH + 60k_h \cdot \tau \left(b + \frac{H}{2} \right) U \right]} \text{ m} \quad (5.8a)$$

$$H = \frac{\Psi - 60Q_r \cdot \tau}{L(b e_s + 30k_h \cdot \tau U)} - \frac{60k_h \cdot \tau b U}{(b e_s + 30k_h \cdot \tau U)} \text{ m} \quad (5.8b)$$

$$b = \frac{\Psi - 60Q_r \cdot \tau}{L e_s H + 60L k_h \cdot \tau U} - \frac{30L k_h \cdot \tau H U}{L e_s H + 60L k_h \cdot \tau U} \text{ m} \quad (5.8c)$$

“Soakaway” part-occupied with impervious material, cleansed water inflow and outflow rate of Q_r m³/s (see Procedure 2C); note that $Q_r = n \cdot q_r$ may be an option to be considered :

$$a = \frac{\Psi - 60Q_r \cdot \tau}{(e_s \cdot H + 60k_h \cdot \tau \cdot U)} \text{ m}^2 \quad (5.9a)$$

$$H = \frac{\Psi - 60Q_r \cdot \tau}{a e_s} - \frac{60k_h \cdot \tau U}{e_s} \text{ m} \quad (5.9b)$$

where $U = 0.5$ for sandy soil; $U = 1.0$ for sandy clay; $U = 2.0$ for clay soils.

Important note : The discussion, above, of slow-drainage is confined to components of Procedure 2 (‘leaky’ wells, trenches and “soakaways”) and avoids Procedure 3 (infiltration or ‘dry’ ponds). This is because stormwater passing to a pond is unlikely to be ‘cleansed’, as required for all Procedure 2 devices. It is unacceptable environmental practice for aquifer access, applying Procedure 4A, or piped drainage (Procedure 4B) to be permitted **directly** to receiving domains with untreated water from a pond.

However, if treatment takes place within the pond ensuring that satisfactorily cleansed water, only, is admitted to the outlet, then the same design approach reviewed above for the trench and “soakaway” cases, namely catchment runoff volume Ψ replaced by $(\Psi - 60 Q_r \cdot \tau)$ in the corresponding equations [(5.6a) and (5.6b)], etc., etc., can be employed to yield appropriate outcomes.

A guideline on the standard of treatment required to achieve satisfactory cleansing is : subject to passage through two thicknesses of non-woven geotextile fabric before exit. This can be accomplished using a perforated collector/pipe system enclosed in a geotextile fabric sock. Regular inspection and maintenance is an essential ingredient of this solution.

TABLE 5.1
SUMMARY OF GUIDELINES FOR PROCEDURES 1A, 1B, 2A, 2B, 2C, 3 AND 4 : SOME
COMBINATION CASES

[Section 4.2 should be read and thoroughly understood before attempting to use this Table]

FACILITY TYPE AND CALCULATION PROCEDURE	PERCOLATION OUTFLOW TIME BASE, τ , MINS	DESIGN FREQUENCY	PRE-TREATMENT REQUIRED	PROVISION FOR HANDLING FLOWS EXCEEDING DESIGN
Infiltration surface receiving runoff from roof or paved surface, e.g. carriageway, car park, etc. (Eqns. 5.1, 5.2).	Not applicable : design is based on Q_{peak} for site t_c ; consult Guidelines 1 and 2, Sections 3.1.2 and 3.1.3. See Example 5.1, Section 5.2, this page; also Example 5.5, page 105.	Generally 2 – 10 years where focus of design is on minor system . ARIs normally associated with major system design – 50- to 100-years – are employed in some catchment cases (see Section 4.6.1).	NIL	Provide for overflow, that is, 'gap' between design Q_{peak} and peak flow for site t_c in ARI, Y = 50-years to 100-years event.
"Leaky" well, trench or "soak-away" receiving runoff from roof area or similar (Eqns. 5.3, 5.4 and 5.5).	The Parameter, τ, and 'design storm' duration τ is the time base of the runoff hydrograph which is taken to be the "period of filling"; this is considered also to define the period of percolation or drainage outflow from a water-retaining device or system during filling . The minimum value τ can take is $\tau = 2t_c$ where t_c = site time of concentration (see Figure 3.4b). However, concern for flooding in local drainage paths and in main streams where local flows enter, requires that site drainage be designed with a more global view in mind, explained in Section 4.2.3.		Requires 'first flush' or simple sediment removal/containment system as in Figs. 1.5 or 5.2.	It is important that provision be made for 'overflows' : these will occur in ALL cases except systems which have been designed to combat total catchment flooding in major, e.g. ARI, Y = 50-years to 100-years events with the yield-minimum strategy. Where site flooding, only, is the focus of design at, say, ARI, Y = 5-years, then a flow path with capacity equal to the 'gap flow' between Q_{100} and Q_5 must be provided.
Infiltration pond receiving runoff from small urban catchment, large paved area, etc. (Eqns. 5.6).	Typically, $\tau = (T_C + t_c)$ where T_C is local OR total catchment travel time (two values possible, see Figure 3.4). Three possible basic cases arise : $\tau = 2t_c$, site drainage case, or where t_c coincides with $(T_C)_{local}$; $\tau = [(T_C)_{local} + t_c]$ where local sub-area flooding, only, is to be considered; $(T_C)_{local}$ is, typically, local sub-area critical storm duration as defined in Section 4.2.3; $\tau = [(T_C)_{total} + t_c]$ where total catchment flooding, only, is to be considered; $(T_C)_{total}$ is critical storm duration as defined in Section 4.2.3.		Filter strip, bio-retention swale or similar upstream of pond is advised.	
Sub-structure (gravel-filled device) beneath treatment surface, infiltration pond, etc. (Eqns. 5.4 and 5.5). See, also Example 5.5, Section 5.2	The corresponding storm durations are t_c , $(T_C)_{local}$ and $(T_C)_{total}$ respectively. The dominant flooding consideration – site, local or total catchment – determines which of the three alternatives should be used in the design process.		Not applicable. Pre-treatment to pond or the pond/filter system itself provides all required pre-treatment.	
Combined "leaky" well/gravel-filled systems. Design of these requires trial solutions until match is achieved (see Combination Structures, Section 5.3.3) for runoff from roof area or similar (Eqns. 5.3, 5.4 and 5.5).			Requires 'first flush' or simple sediment removal/containment system as in Figs. 1.5 or 5.2.	

5.2 EXAMPLES OF CATEGORY 1 INSTALLATIONS USING PROCEDURES 1A, 2A, 2B, 2C, 3, 1A/4A AND 2B/4B

Example 5.1 : Infiltration surface without ponding

The case study used to illustrate this aspect of 'source control' of stormwater is St. Elizabeth Church car park at Warradale in Adelaide (see Section 3.9) illustrated in Figure 5.1. Determine the size of the central grassed hard-standing area. Surface reinstatement to take place less frequently than every 30 years; flooding ARI = 3 years (client requirements).

Details of the catchment draining to the central section of the car park :

- **total** car park area (40 car park spaces + carriageway, $C_{10} = 0.9$) 1,220 m²
- catchment description : car park in average suburb with some trees;
- site time of concentration, t_c 10 minutes
- design storm intensity (ARI = 3-years, $F_y = 0.9$, Table 3.5) 50 mm/h

Hence, design $Q_{peak} = 0.014 \text{ m}^3/\text{s}$.

Procedure 1B : Infiltration surface (without ponding) accepting runoff from paved area, A m², passing to :
an *internal* area, A_S :

required area,

$$A_S = \frac{Q_{\text{peak}}}{(1 - \psi) \cdot k_h \cdot U} \text{ m}^2 \quad (5.2)$$

where ψ = blockage factor (“Grasspave”, $\psi = 0.1$);

k_h = 2.5×10^{-4} m/s : hydraulic conductivity of ‘as constructed’ soil for grassed area;

k_h = **long-term** value of infiltration soil by Guideline 1, Section 3.1.2, is 5×10^{-5} m/s,

Hence $A_S = 311 \text{ m}^2$: this corresponds to $(I/P) = 2.9$, and ‘lifespan’ $\cong (5.2 \text{ years} \times 5) = 26$ years interpolated from Table 3.1 (“...average suburb with some trees”).

This is unsatisfactory. To meet minimum 30 years required by client, choose $I/P = 2.0$,

hence ‘lifespan’ $= (6.7 \text{ years} \times 5) = 33.5$ years, OK.

Hence, vegetated porous area $= \frac{1,220}{3} = 407 \text{ m}^2$.

[It should be noted that if the same result for A_S had been determined for a **permeable** paving system, then the ‘lifespan’ consideration process would proceed as follows :

$A_S = 311 \text{ m}^2$: this corresponds to $(I/P) = 2.9$;

Hence, ‘lifespan’ (Figure 3.1b) ≈ 27 years (geotextile layer) at 20% ‘as constructed’, and,

‘lifespan’ (Table 3.1) ≈ 5.2 years (surface blockage).

In the event of these estimates being unsatisfactory, a review and reconfiguration process similar to that outlined above would be needed to produce an acceptable outcome.]

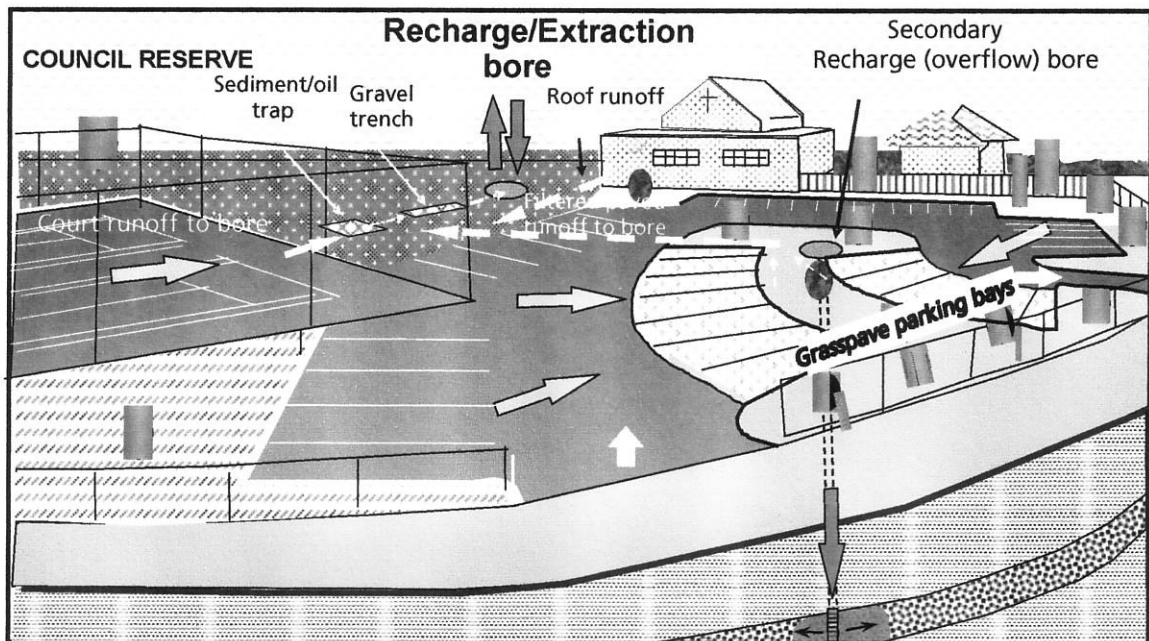


FIGURE 5.1 : General layout of St. Elizabeth Church car park

Example 5.2 : Gravel-filled trench receiving roof runoff

The New Brompton Estate, Adelaide, central reserve is the site of a “greenfields” development (modified project) whose aim is to retain roof runoff on site. The Council specification requires that the development be designed under the provisions of the **yield-minimum** strategy (see Section 4.2). The actual scheme is described in Section 3.9 and illustrated in Figure 5.2.

Details of the catchment (roof areas) draining to the gravel-filled trench bordering the reserve are as follows :

- total connected roof areas (and “greenfields” area) 1,200 m²
- site time of concentration, t_C 10 minutes
- critical storm duration, $(T_C)_{\text{total}}$, Council specification 60 minutes
- design storm intensity (ARI, Y = 10-years, Council specification) 26 mm/h

Hence, design runoff volume, $V = 28.1 \text{ m}^3$ (**yield-minimum** strategy, Council specification).

Procedure 2B : Trench, part occupied with impervious material, pipes, etc., with clear water inflow :

Required length,

$$L = \frac{V}{\left[e_s \cdot bH + 60k_h \cdot \tau \left(b + \frac{H}{2} \right) U \right]} \text{ m} \quad (5.4a)$$

- where
- e_s = void space ($e_s = 0.35$ for gravel)
 - b = trench width ($b = 0.9 \text{ m}$ in this case)
 - H = trench height ($H = 1.2 \text{ m}$ in this case)
 - k_h = soil hydraulic conductivity ($1 \times 10^{-6} \text{ m/s}$)
 - τ = time base of design storm runoff hydrograph ($\tau = 70 \text{ minutes}$ in this case)
 - U = 2.0 for clay soil.

Hence $L = 72.0 \text{ m}$. This must be followed by emptying time calculation :

$$\text{Emptying time, } T = -\frac{4.6Lbe_s}{2k_h(L+b)} \log_{10} \left[\frac{Lb}{Lb + 2H(L+b)} \right], \text{ s} \quad (3.31)$$

Hence, emptying time, $T = 4.7 \text{ days}$ (unsatisfactory, see Table 3.3).

In reality, the infill soil at New Brompton Estate, site of a former brick (clay) pit, was heavy clay laid in 150 mm layers and compacted with heavy equipment to a total depth of seven metres. Its measured hydraulic conductivity was $3 \times 10^{-9} \text{ m/s}$, a very low value. In its second year of operation a recharge bore receiving water from the trench (low level overflow) was installed: this provided the elements of an aquifer storage and recovery (ASR) scheme which ensured satisfactory emptying time; the project retains the high-level overflow to street drainage as originally planned.

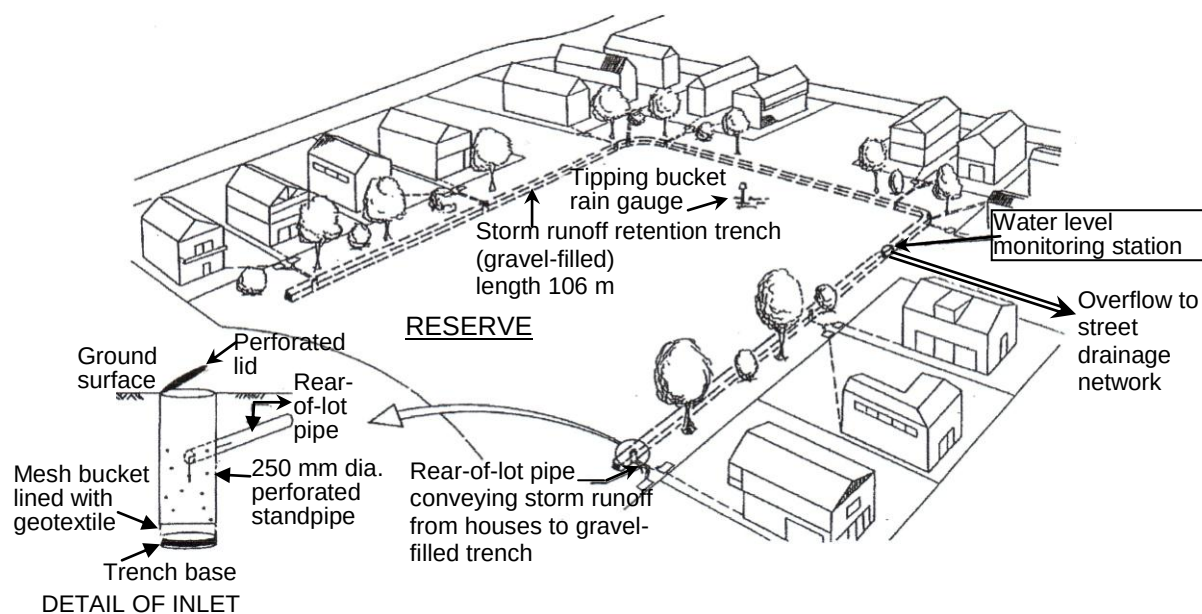


FIGURE 5.2 : Development at New Brompton Estate, showing gravel-filled trench and sediment retention standpipe

Example 5.3 : Gravel-filled “soakaway” and passive irrigation facility

The basis of the passive irrigation facility at Plympton Anglican Church is a perforated pipe and gravel “soakaway” (see Section 3.9) illustrated in Figure 5.3. The following design is prepared for the project considered as occupying a “greenfields” site. The Council specification requires that the development be designed under provisions of the **regime-in-balance** strategy (see Sections 4.2 and 4.6).

Original “greenfields” site :

- site area ($C_{10} = 0.10$) 2,640 m²
- site time of concentration, t_C 10 minutes
- critical storm duration, $(T_C)_{total}$, Council specification 30 minutes
- design storm intensity
[ARI = 100-years, ($F_Y = 1.2$, Table 3.5), Council spec.] 73 mm/h

Hence, design runoff volume = 11.6 m³ from “greenfields” site in 100-years design storm.

Developed site :

- site area 2,640 m²
- total connected roof areas 1,600 m²
- total connected paved area 360 m²
- total connected pervious areas ($C_{10} = 0.10$) 680 m²
- site time of concentration, t_C 10 minutes
- critical storm duration, $(T_C)_{total}$, Council specification 30 minutes
- design storm intensity
[ARI = 100-years, ($F_Y = 1.2$, Table 3.5), Council spec.] 73 mm/h

Hence, design runoff volume = 74.5 m³ from developed site in 100-years design storm.

Hence, design runoff volume, $V = (74.5 - 11.6) = 62.9$ m³ (**regime-in-balance** strategy, Council specification).

Procedure 2C : “Soakaway” part-occupied with impervious material, pipes, etc., with cleansed water inflow :

$$\text{required area, } a = \frac{\forall}{\left[e_s \cdot H + 60 k_h \cdot \tau \cdot U \right]} \text{ m}^2 \quad (5.5a)$$

where e_s = void space ($e_s = 0.50$ for perforated pipe/gravel system, see Procedure 2B, Section 5.1)

H = height (thickness) of soakaway (0.3 m in this case)

k_h = soil hydraulic conductivity (1×10^{-6} m/s); $U = 2.0$ for clay soil

τ = time base of design storm runoff hydrograph ($\tau = 40$ minutes in this case)

Hence a = **406 m²**. This must be followed by emptying time calculation :

$$\text{Emptying time, } T \approx \frac{2H \cdot e_s}{k_h}, \text{ s} \quad (3.32)$$

Hence emptying time, $T = 3.5$ days (satisfactory, see Table 3.3).

The installation at Plympton is enclosed in geotextile and overlaid by a 0.3 m thickness of loam supporting a garden of native plants and shrubs and drought-resistant grass. Surface watering was needed to establish the garden; it now receives normal rainfall and ‘passive’ irrigation (from below). The grassed area may receive a light “sprinkle” of mains water prior to use for outdoor ceremonies, otherwise no irrigation.

It is of interest to note how transfer of this illustration from Adelaide in Southern Australia to Brisbane in Northern Australia affects the design outcome. In this case – using the same basic data except for “greenfields” pervious area runoff coefficient, $C_{10} = 0.7$ (for Northern Australia, see Section 3.6), and Brisbane rainfall – the design runoff volume which must be accommodated is :

$$\forall = 24.9 \text{ m}^3 \text{ (regime-in-balance strategy, Council specification)}$$

leading to “soakaway” area, $a = 161 \text{ m}^2$ and emptying time of 3.5 days.

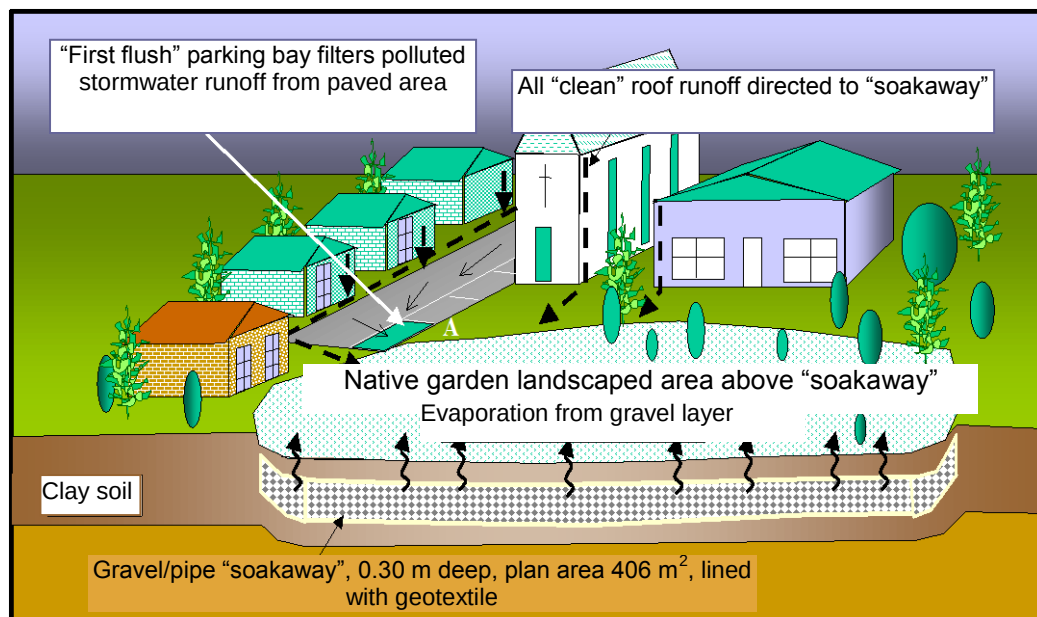


FIGURE 5.3 : Layout of Community Garden, Plympton Anglican Church, Adelaide

Example 5.4 : Infiltration or ‘dry’ pond

“Figtree Place” is an inner-city residential (27 units) re-development project completed in Newcastle, NSW, in 1998: its layout is illustrated in Figure 5.4 (see Section 3.9). The Council Specification required that the development be designed under the provisions of the **yield-minimum** strategy (see Sections 4.2 and 4.6). [The infiltration pond at “Figtree Place” is **not** a ‘sump’ referred to in Section 4.2.4, because it overflows in all storms greater than the 50-years critical storm event]

Details of the catchment (paved area only) draining to the recharge facility are as follows :

- total connected paved areas..... 1,900 m²
- site time of concentration, t_c 10 minutes
- critical storm duration, $(T_c)_{total}$, Council specification..... 60 minutes
- design storm intensity
[ARI = 50-years ($F_Y = 1.15$, Table 3.5) Council spec.] 67 mm/h

Hence, design runoff volume, $V = 127 \text{ m}^3$ (**yield-minimum** strategy, Council specification).

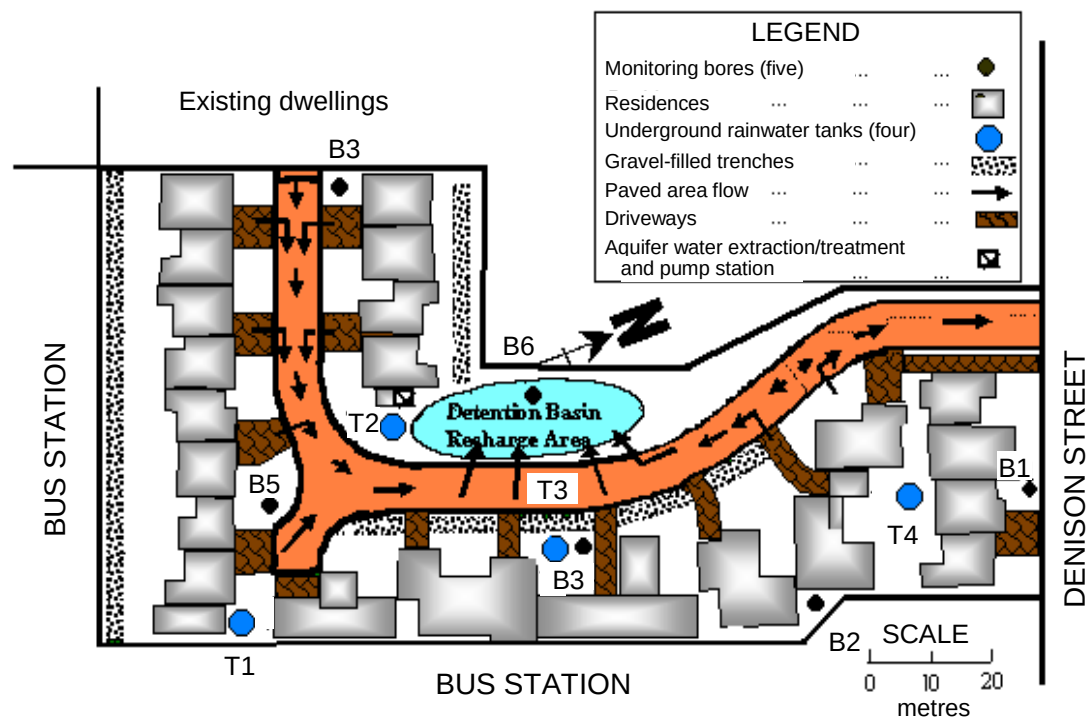


FIGURE 5.4 : “Figtree Place”, Newcastle : general plan and water-sensitive features

Procedure 3 : Pond with infiltration :

$$\text{required area, } A_p = \frac{V}{\left[d + 60 k_h \cdot \tau \cdot U - \frac{i \cdot (\tau - t_c)}{6 \times 10^4} \right]} \text{ m}^2 \quad (5.6a)$$

where d = design depth of pond (0.5 m in this case)

k_h = long-term hydraulic conductivity of pond surface soil ($5 \times 10^{-5} \text{ m/s}$) : $U = 1.0$

τ = time base of design storm runoff hydrograph ($\tau = 70$ minutes in this case)

i = rainfall intensity for design storm, duration 60 minutes, ARI = 50-years (67 mm/h).

Hence $A_p = 198 \text{ m}^2$. This must be followed by emptying time calculation :

$$\begin{aligned} \text{Emptying time, } T &= \frac{2d \cdot e_s}{k_h}, \text{ with } e_s = 1.0, \\ &= \frac{2d}{k_h}, \text{ seconds for open water body} \end{aligned} \quad (3.32)$$

Hence emptying time = 0.23 days (satisfactory, see Table 3.3).

The hydraulic conductivity of the general site soil at “Figtree Place” is less than that of the introduced soil used on the floor of the ‘dry’ pond. A sub-surface “soakaway” is therefore provided to ensure that the pond clears quickly following rainfall, enabling its other use as a recreation facility to be fulfilled (see Example 5.5, following).

Example 5.5 : Sub-structure “soakaway” beneath infiltration surface

This example re-visits Example 5.1: its aim is to dimension the gravel-filled sub-structure beneath the 407 m² car park infiltration or ‘treatment’ surface at St. Elizabeth Church. This is a prime example of the type of water-retaining facility whose overflow is understood by the public to occur, literally, “once only on average in 3-years”, not tied to site critical storm duration, t_c (see the final four paragraphs of Section 4.2.6). The design must therefore consider **all** storm durations with frequency ARI = 3-years to determine the optimum case.

Details of the catchment draining to the treatment surface and sub-structure are as follows :

- paved area (40 car park spaces + carriageway)..... 1220 m²
- grassed hard-standing area (15 car park spaces) 407 m²
- site time of concentration, t_c 10 minutes
- storm duration range considered
(ARI, Y = 3-years, $F_Y = 0.9$, Table 3.5) 10 mins to 3 days.

Procedure 4A : Establish the presence of a readily accessible aquifer; determine recharge rate per bore, q_r m³/s; use Eqn. (5.9a) :

$$a = \frac{\Psi - 60q_r \cdot \tau}{(e_s \cdot H + 60k_h \cdot \tau \cdot U)} \text{ m}^2 \quad (\text{single bore case}) \quad (5.9a)$$

where e_s = void space ($e_s = 0.35$ for gravel)
 H = “soakaway” height (use $H = 0.5 \text{ m}$ in this case)
 k_h = soil hydraulic conductivity ($1 \times 10^{-6} \text{ m/s}$); $U = 2.0$, clay soil;
 q_r = 0.0005 m³/s (given)
 τ = time base of design storm runoff hydrograph [$\tau = (t + 10 \text{ mins})$,
 where t = storm duration].

Tabulation to determine the optimum value of gravel-filled “soakaway” needed to give overflow literally once, only, in 3-years ($F_Y = 0.90$) for **all** storm durations :

Storm duration, t	Intensity, (mm/h)	Rainfall depth, d (mm)	$\tau = (t + t_c)$ (mins)	$\Psi = (0.81 \times 814 + 407)d$ (m ³)	$\Psi - 60 q_r \tau$ (m ³)	$e_s H + 60 k_h \tau U$ (m)	hence, a (m ²)
10 mins	50	8.3	20	8.85	8.25	0.177	46.5
...	...	...	...	...	...	...	...
...	...	...	...	...	...	...	...
3 hours	9.2	27.6	190	29.4	23.7	0.198	120
4 hours	7.6	30.4	250	32.4	24.9	0.205	121
6 hours	5.9	35.4	370	37.7	26.6	0.219	121
8 hours	4.9	39.2	490	41.8	27.1	0.234	116

The optimum area of “soakaway” required is 121 m², resulting from storm duration between 4 and 6 hours. This implies a gravel-filled trench, 0.5 m deep, under approximately 1/3 of the 407m² infiltration surface: its storage volume is 21.2 m³; the “soakaway” includes a single bore (see Figure 1.4a for sketch details). This must be followed by emptying time calculation :

$$\begin{aligned}
 \text{Emptying time, } T &= \frac{\text{stored volume}}{q_r} \\
 &= \frac{121 \times 0.5 \times 0.35}{0.0005} \\
 &= 41,650 \text{ s} \\
 &= 11.8 \text{ hours (see Table 3.3, satisfactory).}
 \end{aligned}$$

This design has been determined assuming the sub-structure to be a “soakaway”, not a gravel-filled trench. Strictly, the design should be reviewed using the trench formula, Eqn. (5.8a), in place of Eqn. (5.9a), inserting $H = 0.50$ m and $b = 2.0$ m, say, to determine a value for length L ; the **final** design should then be re-checked for emptying time. This process leads to $L = 59.6$ m (or plan area, $a = 119$ m²) and emptying time of 11.6 hours, very close to the “soakaway” solution.

The car park at St. Elizabeth Church has been designed with sufficient above-ground storage capacity to manage/dispose of all storm runoff generated on the site up to and including 100-years events covering the full range of storm durations (10 minutes to three days). It is therefore, for all practical purposes, a ‘sump’ (see Section 4.2.4). Its design required additional computations to those given above.

However, were this **not** the case and if overflow from the grassed hard-standing area were to find its way directly into the local (surface) drainage path *without ponding*, then an alternative design of the infiltration surface and the “soakaway” would have to be carried out. This (design) would have to consider the car park as a component of the catchment-wide stormwater management plan for, say, the **yield-minimum** strategy and ARI, $Y = 100$ -years required (see Sections 4.2 and 4.6). The infiltration surface (redesign of Example 5.1) in this case would be $A_S = 1024$ m² (Adelaide, 10 minutes, $i_{100} = 136$ mm/h); this configuration of the car park leads to an impervious-to-pervious ratio of 0.19 which easily satisfies Guidelines 1 and 2, Sections 3.1.2 and 3.1.3) and suggests service without maintenance for a long period of time. The revised “soakaway” calculations, under these altered circumstances, would be as follows :

Developed site :

- paved area (carriageway) 196 m²
- grassed hard-standing area (40 car parking spaces)..... 1,024 m²
- site time of concentration, t_c 10 minutes
- critical storm duration, $(T_c)_{\text{local}}$ (Council specification)..... 30 minutes
- design storm intensity [ARI = 100-years ($F_Y = 1.2$) Council spec.] 73 mm/h

Hence, total (stormwater) volume which must be managed on site *without overflow* :

$$\forall = 44.5 \text{ m}^3 \text{ from the site in the 100-years design storm.}$$

The gravel-filled “soakaway” has aquifer access, so Procedure 4A may be applied :

Procedure 4A : establish the presence of a readily accessible aquifer; determine recharge rate per bore, q_r m^3/s ; use Eqn. (5.9a) :

$$a = \frac{\forall - 60q_r \cdot \tau}{(e_s \cdot H + 60k_h \cdot \tau \cdot U)} \text{ m}^2 \quad (\text{single bore case}) \quad (5.9a)$$

where	e_s	=	void space ($e_s = 0.35$ for gravel)
	H	=	“soakaway” height (use $H = 0.5$ m in this case)
	k_h	=	soil hydraulic conductivity (1×10^{-6} m/s); $U = 2.0$ for clay;
	q_r	=	$0.0005 \text{ m}^3/\text{s}$ (given)
	τ	=	time base of design storm runoff hydrograph ($\tau = 40$ mins in this case).

Application of this formula to the case yields a “soakaway” plan area :

$$a = 241 \text{ m}^2.$$

This area **exceeds** that determined previously, therefore it would over-ride it *but only in the circumstances proposed in this addendum to the earlier example*. Note that emptying time for the alternative design is also satisfactory at 23.4 hours (see Table 3.3).

Important additional note :

Both analyses reviewed above provide dimensions – plan area or length, and depth – of the gravel-filled sub-structure required beneath the porous car park surface based on *hydrological*, only, considerations. In terms of total design of such installations, this represents half of the task. The other ‘half’ is that determined by the need for structural integrity of the facility under its expected vehicle loads : this activity should be carried out by someone competent in the design of road and highway pavements. The consequences of the two design directions, almost inevitably, are :

- two significantly different sets of basic dimensions for the installation (plan area and gravel depth); and
- two contrasting specifications for the gravel sub-structure.

Study of Example 5.5, above, reveals the hydrological requirements being satisfied by a gravel-filled sub-structure with depth $H = 0.50$ m located under less than half of the assigned ‘hard standing’ area. While the 0.50 m thickness of sub-structure may be harmonious with, or even exceed the needs of, structural integrity beneath the porous paved car park surface, it is certain that treatment of the *remaining* area (either case) without support from a gravel-filled sub-structure, would not. Competent structural design of a porous or permeable paved car park calls for substantial sub-structure support under the entire area of loading.

Then there is the issue of gravel Specification. The urban hydrologist may prefer a single-size gravel – to maximise void space; the pavement structural designer is likely to specify a graded mix of gravels for the sub-structure.

The apparent design conflict outlined here can be readily resolved by early consultation between the (design) parties. If the urban hydrologist is advised of the depth of (gravel) sub-structure needed to meet the structural requirements of the project, then this dimension becomes the value of “H” (“soakaway” height) used in the appropriate design procedure (Procedure 4A in the above illustrations). The hydrological design then becomes an exercise in checking whether the runoff volume, $\forall$, can be accommodated by a sub-structure system extending beneath the entire (car park) area. The issue of gravel specification is, also, not a cause of conflict. Graded gravel has a void space ratio which is little different from single-size gravel : $e_s = 0.30$ in this case is an adequate substitution for 0.35 normally used with single-size gravels.

Introduction to Examples 5.6 and 5.7 : OSR Devices for Developed/Re-developed Sites

The next two examples relate to the tasks of, firstly, providing a “leaky” well on-site stormwater retention device for a new, quarter-acre (1,000 m²) residential development and, secondly, designing a gravel-filled trench (OSR device) for the same site re-developed for dual (site) occupancy. The two examples use different soil types in order to extend the range of case study illustrations. Figure 1.1 is repeated here for ease of access.

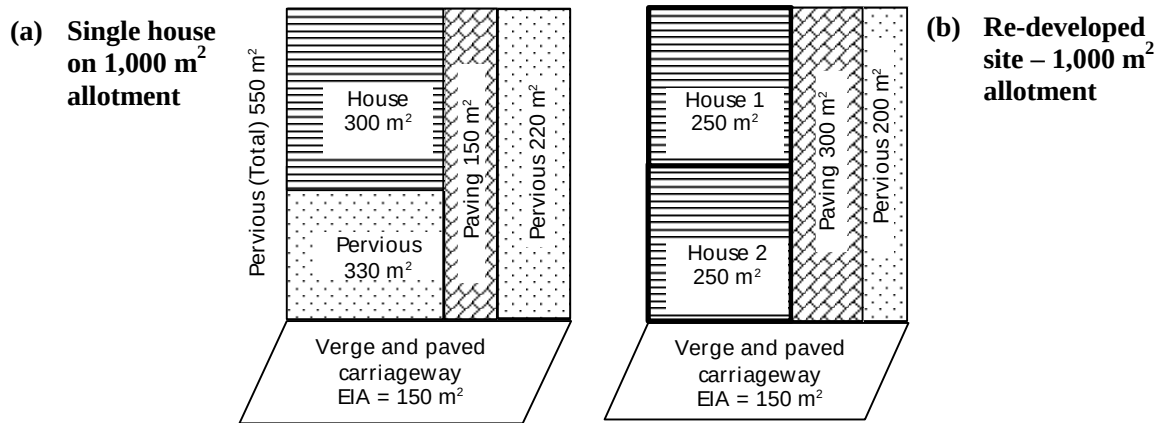


FIGURE 5.5 : Layout of typical development and re-development cases used in Examples 5.6 and 5.7

Example 5.6 : “Leaky” well for residence in new sub-division

The primary stormwater management device to be used on the new domestic site illustrated in Figure 5.5a is a “leaky” well receiving **all roof runoff** (only) from the residence. The allotment size is 1,000 m² and the site soil is sandy clay; the site is located in the Parramatta, NSW, region (Toongabbie rainfall). The complete site layout includes paved and pervious areas as well as footpath/nature strip and a segment of the fronting carriageway: the latter were used in the modelling reviewed in Section 1.2. The Council specification requires that the development be designed under the **yield-minimum** strategy (see Sections 4.2 and 4.6) with ARI, Y = 50 years.

Details of the roof area (only) draining to the “leaky” well are as follows :

- total connected roof area 300 m²
- site time of concentration, t_c 10 minutes
- critical storm duration, $(T_c)_{total}$, Council specification 90 minutes
- design storm intensity (ARI, Y = 50-years, Council specification) 50 mm/h

Hence, design runoff volume, $V = 22.5 \text{ m}^3$ (**yield-minimum** strategy, Council specification).

Procedure 2A : “Leaky” well with cleansed water inflow :

The runoff collected and retained on site is sourced from the roof area only.

$$D = \sqrt{\frac{V}{\frac{\pi}{4} \left(H + 120 k_h \cdot \tau \cdot U \right)}} \text{ m} \quad (5.3a)$$

[This formula includes the assumption, $D \approx H$]

where D = diam of “leaky” well.

H = height of well = 2.0 m

k_h = soil hydraulic conductivity (sandy clay)... 3×10^{-5} m/s

τ = $(T_C + t_C) = 100$ minutes

U = 1.0 for sandy clay.

Hence, $D = 3.48$ m (too big!).

Re-design, using $\nabla = 22.5/3 = 7.5$ m³ each

giving 3 wells, each 2.01 m diam..... **USE 3 × 2.10 m diam.**

Check emptying time :

$$T = -\frac{4.6D}{4k_h} \log_{10} \left[\frac{\frac{D}{4}}{H + \frac{D}{4}} \right] \text{ seconds} \quad (3.30)$$

= **15.3 hours** (satisfactory, see Table 3.3).

The three “leaky” wells should be located where they can receive **all** roof runoff; they should be placed at least 4 m (clear distance) apart and should be located at least 2.0 m from footings and property boundaries (see Section 1.3.4); they should have overflow provision to street drainage, if possible or, if not, be remote from down-slope properties.

Example 5.7 : Gravel-filled trench for re-developed residential site

A developed residential site identical to that shown in Figure 5.5a, except for soil type and council-specified conditions, is re-developed to provide two residential units as illustrated in Figure 5.5b. It is proposed that a gravel-filled trench be installed receiving storm runoff from the roof area only. The allotment size is 1,000 m² and the site soil is heavy clay, characteristic of the Parramatta region (Toongabbie rainfall). The Council specification requires that the re-development be designed under the **regime-in-balance** strategy (see Sections 4.2 and 4.6) with ARI, Y = 100 years. Note that OSR installations in heavy clay soils pose major challenges for the designer (see Section 1.3.4).

Original developed site :

- total connected roof area 300 m²
- total connected paved area..... 150 m²
- total connected pervious area ($C_{10} = 0.39$)..... 550 m²
- verge and paved carriageway (EIA)..... 150 m²
- site time of concentration, t_C 10 minutes
- critical storm duration, $(T_C)_{\text{total}}$, Council specification..... 60 minutes
- design storm intensity [ARI = 100-years ($F_Y = 1.2$) Council spec.]... 70 mm/h

Hence, design runoff volume = 60.0 m³ from **developed** site in 100-years design storm.

Re-developed site :

- total connected roof area..... 500 m²
- total connected paved area..... 300 m²
- total connected pervious area ($C_{10} = 0.39$)..... 200 m²
- verge and paved carriageway (EIA)..... 150 m²
- site time of concentration, t_C 10 minutes
- critical storm duration, $(T_C)_{total}$, Council specification..... 60 minutes
- design storm intensity [ARI = 100-years ($F_Y = 1.2$) Council spec.] 70 mm/h

Hence, design runoff volume = 73.0 m³ from re-developed site in 100-years design storm.

Hence, design runoff volume, $V = (73.0 - 60.0) \text{ m}^3 = 13.0 \text{ m}^3$ (**regime-in-balance** strategy, Council specification).

Procedure 2B : Trench, gravel filled, with cleansed water inflow :

Required length,

$$L = \frac{V}{\left[e_s \cdot bH + 60 k_h \cdot \tau \left(b + \frac{H}{2} \right) U \right]} \text{ m} \quad (5.4a)$$

The runoff collected and retained on site is sourced *from the roof areas only*.

L	=	length of trench
e_s	=	0.35
b	=	2.0 m
H	=	1.2 m
k_h	=	soil hydraulic conductivity (heavy clay)... $1 \times 10^{-7} \text{ m/s}$
τ	=	$(T_C + t_C) = 70 \text{ minutes}$
U	=	2.0 for heavy clay.

This leads to $L = 15.5 \text{ m}$ and emptying time, $T = 61 \text{ days}$ (very unsatisfactory, see Table 3.3).

Various options may be followed to arrive at a satisfactory solution. For example using plastic ‘milk crate’ units, say 0.4 m deep $\times$ 2.0 m wide (see Figure 1.4b) and $e_s = 0.95$, will give much the same plan area (34.2 m², 17.1 m trench) as the gravel-filled trench, but emptying time will be more than 70 days, which is still very unsatisfactory. The ‘limiting case’ for use of OSD technology in circumstances such as this, in fact, becomes a form of OSD technology with relatively long emptying time compared with those normally expected of OSD systems. Use is made of Procedures 4A or 4B to achieve a satisfactory outcome -

Procedure 4B : applied to the ‘milk crate’ device :

STEP 1 : Volume of water held in 17.1 m trench = $17.1 \times 2.0 \times 0.4 \times 0.95 \text{ m}^3 = 13.0 \text{ m}^3$; Calculate drainage flow, Q_r , to meet emptying time, $T = 24$ hours (very conservative criterion required by Council which is not bound to accept Table 3.3 suggestions!) :

$$Q_r = \frac{13.0}{24 \times 3600} = 0.00015 \text{ m}^3/\text{s}$$

STEP 2 : Determine L from :

$$L = \frac{V - 60Q_r \cdot \tau}{\left[e_s \cdot bH + 60k_h \cdot \tau \left(b + \frac{H}{2} \right) U \right]} \text{ m} \quad (5.8a)$$

$$\text{with } Q_r = 0.00015 \text{ m}^3/\text{s}, \text{ and } U = 2.0.$$

STEP 3 : $L = 16.2 \text{ m}$, and emptying time = 1.0 day (OK).

The trench should be located where it can receive **all** roof runoff; it may be placed **half** the recommended clearance distance (5.0m) from footings and property boundaries (see “Water-reactivity and ‘clearance’” in Section 1.3.4). It should have provision for overflow – meeting Council requirements – and a small diam pipe (e.g. 50 mm PVC) laid from floor level to street drainage or to an urban waterway (see Figure 1.4b, and Sections 3.5.4 and 5.4). If these are not available, a ‘final option’ is offered in Section 5.5.

Important comment : The re-development scenario reviewed here is common in the older residential suburbs of Australian cities. There are some important lessons which can be learned from a more detailed exploration of the proposed solution, above, as well as from different strategy circumstances which might be applied to the same case :

1. The (**regime-in-balance**) strategy employed in the illustration requires 13.0 m^3 of runoff to be retained on site in the design storm. Part, only, of the *total* storm runoff occurring on the (re-developed) roof area in the design storm (35 m^3) is collected to meet this target, yet this provision takes account of the OSR needs of the **entire** (re-developed) site, not just those of the enlarged roof area. This is not an unusual outcome and is repeated wherever the runoff volume generated on the (re-developed) roof area in the design storm is significantly greater than the “after-minus-before” stormwater volumes [$(73.0 - 60.0) \text{ m}^3$ in this case] determined for an altered site.
2. The interpretation of **regime-in-balance** strategy (Council requirement) employed in the illustration treats the ‘developed’ situation – the single house on the $1,000 \text{ m}^2$ allotment and the drainage infrastructure associated with it – as ‘acceptable’ development dating from the **benchmark year** (see Section 4.6.3). All instances of re-development in the region must therefore incorporate on-site works, such as the ‘milk crate’ trench used, above, to ensure that the existing (stormwater) infrastructure is not overloaded. Another Council, facing the same re-development scenario, might apply a more fundamental approach to its planning and insist that the benchmark year pre-dates **any** urban development in the region. [Council policy matching this approach may see the developed catchment main drainage line being returned to its pre-development alignment and (natural) channel form, as a long-term goal.] This interpretation of the **regime-in-balance** strategy leads to a requirement in the above illustration for 35 m^3 of on-site retention, not the 13.0 m^3 required in Example 5.7. It is interesting to note that this, also, can be achieved (just!), by collecting all of the roof, only, runoff from the re-development in the design storm.
3. In the event that Council strategy in relation to stormwater management were to favour the **yield-minimum** approach, then the quantity of runoff which would have to be retained on site in the design storm would, of course, be 73.0 m^3 (see above). It is not possible, in these circumstances, to solve the problem of on-site retention requirement sourcing the stormwater from the (re-developed) roof areas alone : these can only provide 35 m^3 of runoff in the design storm (see above). Designers must be vigilant to ensure that in circumstances such as these, they do not ‘oversize’ installations, that is, **provide greater storage capacity than the catchment component is able to deliver in the design storm** (see Section 1.4.4). Example 5.7 could only be solved as a yield-minimum problem by providing two elements of on-site storage – one receiving runoff from the roof areas (35 m^3 capacity), and, perhaps, two others retaining ground-level runoff, with joint holding capacity of 38 m^3 .

5.3 SOME ADDITIONAL DESIGN CONSIDERATIONS

The Procedures detailed above, together with the accompanying notes, guidelines and examples, should settle many of the issues relating to design storm duration, design average recurrence interval (ARI) and emptying time which need to be understood to calculate sizes for ‘source control’ water-retaining installations. There are a number of practical issues, however, which need to be clarified before the practitioner can proceed with confidence to design installations for any and every situation.

While the following considerations are not comprehensive, they offer some help in finding solutions to commonly encountered problems.

5.3.1 Design modification to meet emptying time criteria

5.3.2 Impact of site constraints and/or economics on design

5.3.3 Combination structures

5.3.4 High groundwater environments

5.4 SLOW-DRAINAGE PIPES : SOME ISSUES AND OPTIONS

5.5 ON-SITE RETENTION (OSR) IN CLAY SOIL SITES : THE FINAL OPTION

5.6 DETENTION-RETENTION : A LAST WORD

[These sections omitted from the Student Edition]

See page xi for details of distributors of the full Handbook

6. ASPECTS OF STORMWATER QUALITY

6.1 URBAN STORMWATER PARAMETERS AND OTHER FACTORS

6.1.1 Chapter overview

The quality of stormwater present in the urban environment varies from components which are potable – runoff from clean roof surfaces, for example – to runoff from major city traffic lanes and from heavy industry sites, which is likely to contain a mix of urban litter, vegetable matter, sediment and heavy metal contaminants – all in high concentrations. The task of managing this array of components to achieve the goals of WSUD related to water quality issues, requires that :

1. all important parameters and factors used to characterise water quality must be recognised;
2. the characteristics of the full range of urban stormwaters must be identified; and,
3. the question of limits or criteria for acceptance of stormwater elements into downstream uses or environments – with or without pre-treatment – must be answered.

This chapter is divided, broadly, between these three domains with a review of untreated water quality parameters, particularly those relevant to stormwater – physical, chemical and biological properties – examined first. This is followed by a review of the literature on urban stormwater pollution characteristics. The chapter concludes with a wealth of information on standards and guidelines – mainly in tabular form – which may be used to guide practitioners in their endeavours to achieve high standards of performance in urban design which is water-sensitive.

6.1.2 Physical water quality parameters

Temperature : The temperature of a water sample varies regionally, seasonally, diurnally (with night and day) and depending on shade-sunlight availability. In the context of water sensitive urban design, this is particularly relevant to the selection of underground or aboveground rainwater tanks. It affects the dissolved oxygen saturation levels of water, solubility of gases, aquatic fauna and algal growth rates and biological degradation rates (different bacteria species participate in degradation at different temperature ranges).

A phenomenon that may occur in rainwater tanks is thermal stratification whereby heat is absorbed throughout the surface layer of the water, with a cool, stratified layer below that does not allow mixing. This may result in an anoxic layer that permits the mobilisation of nutrients from the benthos layer.

Suspended solids : Suspended solids are defined as “the material that can be removed from a water sample by filtration under standard conditions” (Duncan, 1999). The solids may be organic (for example in the form of plant matter or algae), or inorganic (in the form of clay and silt emanating from increased erosion).

In urban environments, suspended solids are typically between 1 – 50 μm (Duncan, 1999) and may cause blockage to drainage pipes and culverts, for example. Increased turbidity may also result, reducing the light penetration through water and consequently decreasing the amount of light available for aquatic photosynthesis. Of equal concern are the other associated pollutants (such as hydrocarbons, heavy metals and phosphorous) that adsorb onto the surface of the suspended solids.

Turbidity : Turbidity is a measure of the quantity of light either absorbed or scattered by suspended material in water (Horner, 1999). The particle sizes rather than the quantity of sediment determines water sample turbidity. A water sample showing high turbidity may be characterised by having a small concentration of fine particles while another sample with lower turbidity has a higher concentrations of coarse suspended solids. High turbidity reduces the amount of light available for aquatic algae and flora photosynthesis.

Colour : A water sample that has temporary discolouration due to suspended particles has *apparent* colour, while one that has permanent discolouration due to dissolved solids has *true* colour. Many Australian waterways have a yellowish-brown hue due to the humic acid from decaying eucalyptus trees, for example. Water colour pollution may be of particular concern when the interaction of some dissolved

solids with chlorine reduces its effectiveness as a disinfectant or facilitates the formation of by-products that may have an objectionable taste or are carcinogenic (e.g. Trihalomethane –THM).

Odour and taste : Biological by-products, metals and minerals may produce an odour or taste in water (Horner, 1999). In the case of inorganic substances, the result is usually the production of taste without odour, while organic substances may cause both. For example, the biological decay of organisms sometimes causes a sulphuric (“rotten egg”) odour and/or taste to the water, alkaline water may have a bitter taste or metal and metallic salts may produce a bitter or salty taste.

6.2 CHEMICAL WATER QUALITY PARAMETERS

Oxygen : Dissolved oxygen (DO) is defined as the amount of free oxygen, or oxygen present excluding that which is bound by compounds such as H_2O , CO_3 and NO_3 , in a water sample. It is an essential component required for metabolic biological processes (DLWC, 1998).

The dissolved oxygen concentration in a sample is limited to its saturation point which is dependent on a number of factors including :

- temperature – as temperature increases, the DO saturation point decreases;
- atmospheric pressure - as atmospheric pressure increases, the DO saturation point increases;
- salinity – as total dissolved solids increases, the DO saturation point decreases.

The difference between the actual dissolved oxygen concentration in a water sample and the dissolved oxygen saturation point is referred to as the ‘dissolved oxygen deficit’.

The sources of dissolved oxygen in a water sample include :

- Algal photosynthesis : The rate of which is affected by the amount of light available for photosynthetic processes. Factors affecting the available light include the turbidity of the water, the position of the sun due to geographic location or change in seasons and the position of structures blocking sunlight penetration such as trees, buildings and fences.
- Atmospheric re-aeration : The dissolution of oxygen in the atmosphere to the water surface. This is influenced by the dissolved oxygen deficit (and thus water temperature) and the relative turbulence of the water surface (affected by the water depth and velocity as well as the presence of wind and hydraulic structures). Dissolved oxygen contained in incoming flows is also a source of dissolved oxygen in water bodies.

The sinks of dissolved oxygen include biochemical oxygen demand (BOD), sediment oxygen demand (SOD) and the dissolved oxygen deficit of incoming flows. Biochemical oxygen demand is the measure of the oxygen requirement of a mixed population of aerobic bacteria in oxidising the biodegradable organic matter in water (Horner, 1999). This is sometimes separated into carbonaceous BOD for all carbon oxidation and nitrogenous BOD for the oxygen required for nitrification. It is a process that ultimately increases the dissolved oxygen deficit. Sediment oxygen demand is the measure of oxygen required to oxidise sediments and organisms.

The testing of oxygen demand in waters may be unreliable if high concentrations of heavy metals exist as they inhibit the growth of bacteria (Schueler, 1987).

The combined effects of the oxygen sources and sinks described above are competing processes that are simultaneously adding and removing oxygen from water. Initially, oxygen-demanding wastes are oxidising usually at a faster rate than the combined actions of re-aeration. As the oxygen-demanding wastes are oxidised and the available dissolved oxygen is reduced, the oxidation rate gradually slows so that eventually re-aeration rate is greater than the oxidation (see Figure 6.1).

It is, however important to understand the limitations of the DO sag model. The model, for example does not take into account factors such as differential temperatures and light conditions between day, night and times of the year. It also does not take into account the effects of pollutants that are toxic to biota, such as heavy metals, that may affect oxidation or photosynthesis rates in water.

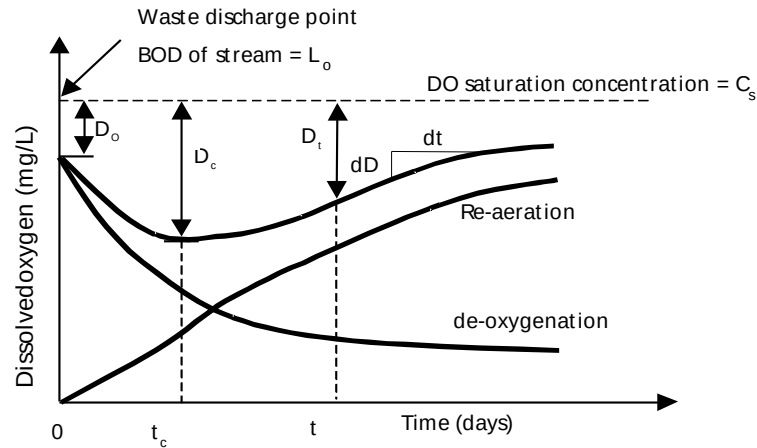


FIGURE 6.1 : Dissolved oxygen sag curve

Nitrogen : Nitrogen is the most common atmospheric gas and a major constituent of living organisms. In the aquatic environment, however, very little nitrogen is utilised (DLWC, 1998).

Nitrogen occurs in many forms, some of which are highly toxic with varying stabilities and oxidation rates. Some of the more important forms of nitrogen are :

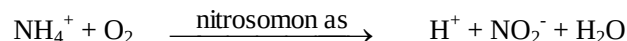
- Organic nitrogen (–N) : often in the form of urea, amino acids, or amines.
- Dissolved ammonia (NH_4^+) : the prevalent form of ammonia at pH7 (DLWC, 1998).
- Ammonia gas (NH_3) : the prevalent form of ammonia at pH12 (DLWC, 1998). At concentrations greater than 0.2 mg/L ammonia is toxic to aquatic life.
- Nitrite (NO_2^-) : an intermediate form in the nitrogen cycle that is toxic.
- Nitrate (NO_3^-) : essential nutrient for aquatic plant growth. In high concentrations (>10 mg/L), nitrates may cause the potentially fatal methemoglobinemia or “blue baby” syndrome.
- Nitrogen gas (N_2) : approximately 80% of atmosphere is nitrogen gas.

6.2.1 The nitrogen cycle

Organic Nitrogen : Amines are separated from particulate organic nitrogen (Org-N) that is produced from the expiration of biomass by heterotrophic bacteria, releasing ammonia nitrogen (NH_4^+). Particulate organic nitrogen that is not converted to ammonia in this manner settles to the benthos layer and may later be converted to ammonia by ‘benthic uptake’, which is the oxidation of ammonia from the benthos layer.

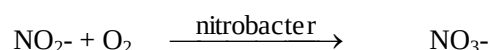
Ammonia Nitrogen : Sources of ammonia nitrogen include hydrolysis of organic nitrogen, benthic uptake and other external inputs including animal faeces and sewer overflows.

Nitrification : Once the above processes have formed ammonia, nitrification occurs in two steps. The first is the oxidation of ammonia to nitrite in an aerobic process by nitrosomonas bacteria.



An important aspect of nitrification is that the nitrosomonas bacteria that perform the oxidation must compete with heterotrophic bacteria for the oxygen required for these reactions. Thus dissolved oxygen availability is a dependent variable in the process. The other major dependent variables in the process are temperature and retention time (DLWC, 1998). Reduced alkalinity of the water may occur as the hydrogen ions (H^+) produced in this process react with naturally occurring carbonates.

The nitrite produced is then in turn oxidised by nitrobacter bacteria to nitrate form.



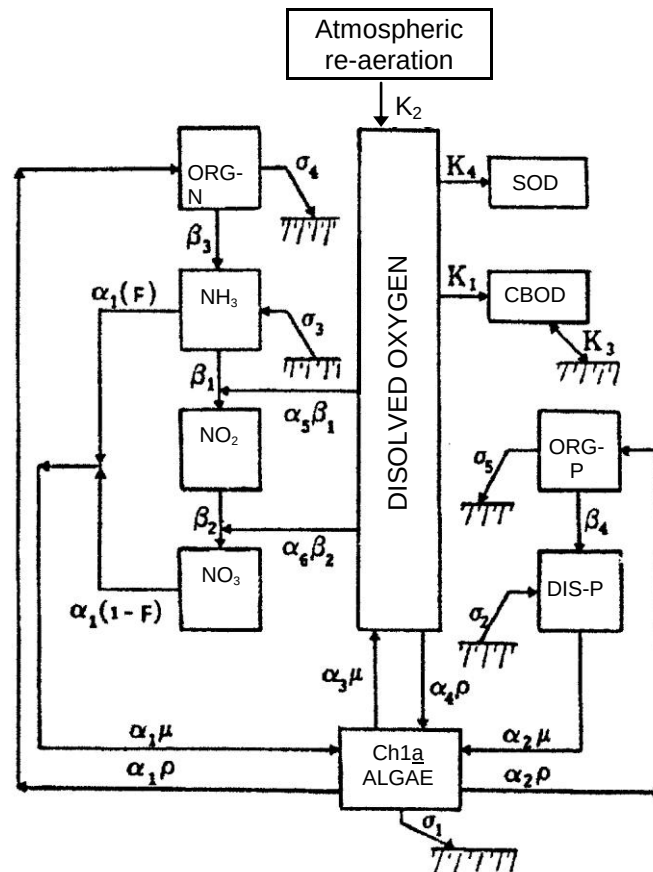
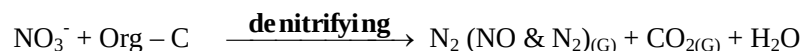


FIGURE 6.2 : Schematic view of the interrelations and oxygen processes used in QUAL2E (US EPA, 1987)

where	α_1	=	Fraction of algal biomass that is nitrogen (mg-N/mg-A)
	α_2	=	Fraction of algal biomass that is phosphorous (mg-P/mg-A)
	α_3	=	Benthos source rate for ammonia nitrogen (mg-N/ft ² /day)
	α_4	=	O ₂ production per unit of algal growth (mg-O/mg-A)
	α_5	=	O ₂ uptake per unit of NH ₃ oxidation (mg-O/mg-N)
	α_6	=	O ₂ uptake per unit of NO ₂ oxidation (mg-O/mg-N)
	β_1	=	Rate constant for the biological oxidation of NH ₃ to NO ₂ (day ⁻¹)
	β_2	=	Rate constant for the biological oxidation of NO ₂ to NO ₃ (day ⁻¹)
	β_3	=	Rate constant for the hydrolysis of organic-N to NH ₃ (day ⁻¹)
	β_4	=	Rate constant for the decay of organic-P to dissolved-P (day ⁻¹)
	μ	=	Algal growth rate (day ⁻¹)
	ρ	=	Algal respiration rate (day ⁻¹)
	σ_1	=	Algal settling rate (ft/day)
	σ_2	=	Benthos source rate for dissolved phosphorous (mg-P/ft ² /day)
	σ_3	=	Benthos source rate for ammonia nitrogen (mg-N/ft ² /day)
	σ_4	=	Organic nitrogen settling rate (ft/day)
	σ_5	=	Organic phosphorous settling rate (ft/day)
	F	=	Fraction of algal nitrogen uptake from the ammonia pool
	K ₁	=	Carbonaceous deoxygenation rate constant (day ⁻¹)
	K ₂	=	Re-aeration rate constant (day ⁻¹)
	K ₃	=	Rate of loss of BOD due to settling (day ⁻¹)
	K ₄	=	Benthic oxygen uptake (mg-O/ft ² /day).

Denitrification : Denitrification is the anoxic process of transformation of nitrate into gaseous forms in the benthos. It is utilised by denitrifying bacteria including *pseudomonas sp.* and is dependent on the presence of nitrate, anoxic conditions and a readily biodegradable carbon source.



where _(G) refers to a substance in its gaseous state.

Due to its dependence on anoxic conditions, denitrification generally occurs after sedimentation of the nitrate in the benthic layer. It also increases the acidity of the water.

Phosphorous : Phosphorous occurs in water in two forms – organic (e.g. nucleic acids, proteins) and inorganic dissolved phosphorous (polyphosphates often found in domestic detergents and orthophosphates that participate in the metabolism of organisms including algae).

Some phosphorous originates from natural sources such as biomass expiration and geological weathering and dissolution while most in the urban environment comes from sewer overflows, animal faeces, agricultural fertilisers and other effluent discharges including those from industry.

6.2.2 The phosphorous cycle

Organic Phosphorous : Organic phosphorous, from the sources listed above, may be transformed to inorganic dissolved phosphorous by bacterial decomposition, or may be adsorbed onto the surface of sediments that settle to the benthos layer.

Inorganic Dissolved Phosphorous : Aquatic biota including bacteria, algae, fungi and aquatic plants subsequently take up the phosphorous and assimilate it into their cells. In acidic conditions, insoluble phosphates of aluminium, calcium, iron or magnesium may be formed and precipitate with sediment to the benthos layer (DLWC, 1998). The phosphorous cycle is completed with the expiration of biota. This again provides organic phosphorous as a nutrient source.

pH : The pH parameter refers to the concentration of hydrogen ions in solution, with a value of 7 being neutral at normal temperatures. The pH quoted is actually the negative base 10 logarithm of the hydrogen ion activity ($-\log_{10} [\text{H}^+]$), with most raw water sources having a pH of between 6.5 to 8.5 (Duncan, 1999). A water body's pH level does not directly affect water quality, but it is a determining factor in almost every natural treatment process (Horner, 1999). Nitrifying bacteria, for example, operate at maximum efficiency in the range $7.5 < \text{pH} < 8.5$.

Levels of pH are usually determined using a pH meter that measures the voltage produced by a glass electrode in the specimen (the voltage produced in the electrode varies linearly with pH).

Alkalinity : The alkalinity parameter refers to the concentration of anions present that neutralise hydrogen ions in solution (acidity). Some of the anions present in natural waters include carbonates, bicarbonates, silicates, phosphates, sulphides and ammonia. The main water quality problem with alkalinity is that the reactions with cations can form precipitates. Alkalinity is expressed in milligrams per litre of CaCO_3 with typical concentrations in drinking water ranging from 5 to 125 mg/L.

The alkalinity of a water sample is determined by titrating the specimen using strong acids and measuring its pH as outlined above.

6.2.3 Heavy Metals

Heavy metals are all metals above calcium (Ca) in the periodic table of elements and have a specific weight higher than 5. They are often very toxic and, studies have shown (DePinto et al, 1980) that the density of benthos macroinvertebrates are related to sediment concentrations of several heavy metals. Because heavy metals are totally non-degradable, they accumulate in the aquatic environment over very long periods of time.

Some of the main heavy metals encountered in stormwater are described in more detail below.

Lead (Pb) : Occurs both in a particulate and dissolved form [but is mainly associated in the particulate form, adsorbed onto suspended solids in runoff (Duncan, 1999)] from sources such as petrol additives, motor vehicle tyre wear, industrial waste, soldered pipe joints and roof flashing. Lead bio-accumulates in the tissue of animals, plants and bacteria and if consumed in high concentrations by humans can cause developmental retardation in infants, brain damage in adults, convulsions, behavioural problems or in extreme cases, death.

Zinc (Zn) : An essential element in human growth, zinc is readily bio-accumulated in the tissue of aquatic animals and plants. Usually in the form of dissolved solids in runoff (Duncan, 1999) zinc will sometimes adsorb onto suspended solids and colloidal particles. Typical sources of zinc in the urban environment include motor vehicle tyre and brake pad wear, combustion of oils and the corrosion of various galvanised metals.

Copper (Cu) : Copper is an essential element aiding in human metabolism that in very large doses may lead to widespread irritation. Dissolved copper imparts undesirable taste and colour on drinking water and is also easily bio-accumulated in the aquatic environment. In the urban environment copper originates from motor vehicle tyre wear, corrosion of rooves, water pipes and from fungicides and pesticides.

Cadmium (Cd) : A highly toxic heavy metal, cadmium has been associated with food poisoning, cardiovascular disease and some cancers affecting humans. It accumulates mainly in the liver and kidneys of animals (including humans) and in the aquatic environment is found mainly in shellfish. In stormwater, cadmium is mostly present in dissolved form (Duncan, 1999). Sources of cadmium include motor vehicle tyre wear, combustion of oils, industrial waste, fertilisers, pesticides and the corrosion of galvanised metals.

Chromium (Cr) : Chromium occurs in two forms, trivalent (an essential mineral for human metabolism) and hexavalent (the predominant form in aerated waters that is associated with kidney and liver damage, gastrointestinal irritations and an increased risk of cancer). Chromium emanates from the wear of moving parts in engines, paints, ceramics, corrosion inhibitors, fertilisers and pesticides.

Iron (Fe) : Iron is an essential mineral for human nutrition that occurs in runoff in two main forms. The divalent (ferrous) state that is highly soluble and the trivalent (ferric) state that occurs in oxidising conditions that is only soluble at low pH levels. Iron is usually present adsorbed onto suspended solids with sources including the corrosion of motor vehicles, landfill leachate and the corrosion of service pipelines. In large doses, iron may be toxic to some fish and invertebrate species.

Hydrocarbons : Hydrocarbons are organic compounds of carbon and hydrogen that are typically present in roads and car park runoff in the petrochemical form originating from lubricating oils, petrols and hydraulic liquids. Some hydrocarbons, such as bitumen become more dense than water when affected microbially, but most remain less dense than water and as such form sheens on the surface of receiving waters. Some hydrocarbons have been shown to be toxic to aquatic fauna.

Approximately 70 - 75% of hydrocarbon oils display a strong attraction to suspended solids with suspended solids removal resulting in hydrocarbon concentration reduction. Methyl-tertiary-butyl-ether (MTBE), a common additive in unleaded petrol, is more soluble than most hydrocarbons while polynuclear aromatic hydrocarbons (PAHs) display extremely strong attractions to sediment.

Hydrocarbon degradation is facilitated by microbial and oxidative processes that have a biochemical oxygen demand.

6.2.4 Biological water quality parameters

Pathogens : A pathogen is a micro-organism (bacteria, virus, protozoa or helminth) that causes waterborne disease (DLWC, 1999). Typical sources of pathogens in the urban environment are animal (including human) faeces and other decaying biological matter. They may be suspended in the water or adsorbed onto the surface of particulate matter. Some of the typical pathogens found in the urban environment are listed in Table 6.1.

Bacteria : Due to the large variety of species of bacteria that require different procedures to identify, specific species are chosen as indicator organisms. Faecal pollution, for example may be monitored by detecting *Escherichia coli* (E. Coli) bacteria for long-term faecal pollution and faecal streptococci for short-term.

TABLE 6.1
TYPICAL PATHOGENS EXCRETED IN HUMAN FAECES
 [Masters, 1998]

PATHOGEN GROUP AND NAME	ASSOCIATED DISEASE
Virus	
Adenoviruses	Respiratory, eye infections
Enteroviruses :	
▪ Polioviruses	Aseptic meningitis, poliomyelitis
▪ Echoviruses	Aseptic meningitis, diarrhoea, respiratory infection
▪ Cocksackie viruses.....	Aseptic meningitis, herpangina, myocarditis
Hepatitis A viruses	Infectious hepatitis
Bacteria	
Salmonella typhi	Typhoid fever
Salmonella paratyphi	Paratyphoid fever
Other salmonellae	Gastroenteritis
Shigella sp.	Bacillary dysentery
Vibrio cholerae	Cholera
Other vibrios	Diarrhoea
Yersinia enterocolitica	Gastroenteritis
Protozoa	
Entamoeba histolytica	Amoebic dysentery
Giardia lamblia	Diarrhoea
Cryptosporidium	Diarrhoea
Helminth	
Ancylostoma duodenale	Hookworm
Ascaris lumbricoides	Ascariasis
Nymenolypis nana	Hymenolepiases
Necator americanus	Hookworm
Strongyloides stercoralis	Strongyloidiasis
Trichuris trichiura.....	Trichuriasis

Protozoa : There are over 50,000 species of protozoa, approximately one fifth of which are parasitic. They infect vertebrates and invertebrates and some are even parasitic in plants. Parasitic protozoa are, in general, small, have short generation times, high rates of reproduction and a tendency to induce immunity to reinfection in those hosts that survive. Protozoa include flagellates, amoeba and ciliates.

The most notorious protozoa in Australian waterways are giardia and cryptosporidium. *Giardia intestinalis*, which is common in the small intestine, is a parasite that has a pair of adhesive suckers which gives it a characteristic appearance. It attaches to the cells of the gut using the suckers and divides by binary fission, in this way huge numbers can build up in the intestine. Giardia are spread by resistant cysts, each of which contains a pair of parasites that causes disease symptoms such as diarrhoea, vomiting and loss of weight.

Cryptosporidium parvum is a parasite found in a range of vertebrates including cows, sheep, rodents, cats, dogs and humans. Many people are infected as children and develop a life-long immunity. Cryptosporidium cysts are so small and resistant that they are not removed by conventional water treatment. Cryptosporidium is a major problem in immuno-suppressed people, including AIDS sufferers and transplant patients.

Viruses : Viruses are parasitic organisms that are dependent on a living host for reproduction and long-term survival. Although they have a relatively short life span in water systems, they are capable of causing significant health risks in recently contaminated waters, while protozoa are unicellular organisms that can exist as parasites or independently. Few protozoan species are pathogenic and those that are may cause gastrointestinal infections in milder forms than that caused by bacteria.

Helminths : Commonly called parasitic worms, helminths originate from human or other animal waste and multiply under aerobic conditions.

6.3 CHARACTERISTICS OF URBAN STORMWATER POLLUTION

6.3.1 Effect of urbanisation on runoff quality

Urban stormwater is an extremely variable resource in terms of both quantity and chemical characteristics. It is defined as “pure rainwater runoff from urban areas including anything the rain carries along with it”. (NSW EPA, 2001)

Urbanisation compromises runoff water quality by making a wider range of pollutants more immediately available to surface runoff. Changes to the natural water balance as a result of urban activity exacerbate this effect.

In an area of undeveloped woodland the natural water cycle processes involve significant proportions of rainfall infiltrating the soil. Some is released back into the atmosphere by evaporation and transpiration and generally a small amount (10% – 20% of annual rainfall input) runs off into creeks or streams.

The activities of urbanisation turn this water balance around. Much less water finds its way into the soil, and the amount of water which runs off is significantly increased. Differences between undisturbed watershed and urban characteristics affect both the speed and volume of runoff events. Even small events of low intensity can result in runoff when natural landscape is replaced by high proportions of impervious area. Depending upon the land use, urbanisation will increase the runoff coefficient by up to 600% [based on undeveloped runoff coefficient of 0.15 increasing to 0.9, representing commercial or industrial development (Argue, 1986)]. As a result, runoff responds much more directly to rainfall. Discharges reach higher peaks more rapidly, resulting in higher flow/velocity runoff. While there is wide agreement that developed watersheds show an increase in peak discharge rate, the extent of this increase is less certain.

In the “Watershed Restoration Sourcebook”, prepared by the Department of Environmental Programs in Washington (Schueler, 1992b), it is claimed that the post-development peak discharge rate may increase by a factor of five times from the pre-development rate, while the National Water Quality Management Strategy (ANZECC 2000) and the Commonwealth EPA (1993) quote ten- and twenty-fold increases respectively.

The effects of these ‘quantitative’ changes have serious impacts on consequent water quality. Higher runoff velocity tends to increase mobilisation of accumulated pollution/sediment on road surfaces and parking areas, and intensifies erosion of soil from land surfaces and from streambeds and banks. Less rain is infiltrated where it falls and instead, it is conveyed to pipes and then on to urban creeks and waterways. This leads to accumulation and concentration of pollution, seriously degrading receiving waters and sensitive water-associated environments.

6.3.2 Sources of pollution

The potential exists for rainwater falling in urban areas to accumulate pollutants throughout the journey from atmosphere to final receiving waters. Even before it falls as rain, water vapour will “wash out” contaminants from the atmosphere. It has been observed that the deposition rate of atmospheric pollutants during dry periods is considerably less than wash out rates during precipitation events (Gutteridge, Haskins & Davey, 1981). Early Swedish studies (Malmquist, 1978; Goettle, 1978) reported that the atmospheric contribution to total pollutant concentrations in stormwater are considerable, for example, 20% organic matter, 70% total nitrogen, 25% total phosphorous and 7 – 40% heavy metals. In fact, in the case of nitrogen, rainfall concentrations have been reported to be higher than those in the stormwater flow itself, indicating that urban areas may be acting as a ‘sink’ for nitrogen (Randall and Grizzard, 1981). Clearly, this is a site specific phenomenon, related to the agricultural and industrial activity in the surrounding regions.

Most runoff from urban areas will contain pollution which has accumulated on roads and other impervious surfaces. Vehicular traffic is the most significant contributor to accumulated road contaminants. Old, poorly maintained vehicles are a particularly high source, as they will, in general, be characterised by higher oil and fluid leaks, leaded fuel usage and generally inefficient combustion. The following table (Table 6.2) from U.S. data (Maestri et al, 1985; US EPA, 1993) describes a number of highway-associated runoff contaminants and their particular sources. Some of these, such as de-icing salts and PCB spraying of highways, are not significant sources in Australian conditions.

TABLE 6.2
HIGHWAY RUNOFF CONSTITUENTS AND THEIR PRIMARY SOURCES

CONSTITUENTS	PRIMARY SOURCES
Particulate	Pavement wear, vehicles, atmosphere, maintenance
Nitrogen, phosphorous	Atmosphere, roadside fertiliser application
Lead	Leaded gasoline (auto exhaust), tyre wear (lead oxide filler material), lubricating oil and grease, bearing wear
Zinc	Tyre wear (filler material), motor oil (stabilising additive), grease
Iron	Auto body rust, steel highway structures (guard rails, etc.), moving engine parts
Copper	Metal plating, bearing and bushing wear, moving engine parts, brake lining wear, fungicides and insecticides
Cadmium	Tyre wear (filler materials), insecticide application
Chromium	Metal plating, moving parts, brake lining wear
Nickel	Diesel fuel and gasoline (exhaust), lubricating oil, metal plating, bushing wear, brake lining wear, asphalt paving
Manganese	Moving engine parts
Cyanide	Anti-caking compounds to keep de-icing salts granular
Sodium/Calcium chloride	De-icing salts
Sulphate	Roadway beds, fuel, de-icing salts
Petroleum	Spills, leaks or blow-by of motor lubricants, anti-freeze and hydraulic fluids, asphalt surface leachate
PCB	Spraying of highway rights of way, background atmospheric deposition, PCB catalyst in synthetic tyres.

Source : Maestri et al, 1985; US EPA, 1993

Sewer overflows and septic tank seepage are sources of faecal bacteria in urban stormwater runoff, as are animal and pet wastes: fines imposed on irresponsible pet owners are aimed at reducing the impact of this latter source. Tree-lined streets contribute significant loads of organic, oxygen demanding material by way of decaying vegetation. Gross litter accumulates on most urban pervious and impervious surfaces and is readily mobilised and added to the stormwater pollutant load.

Construction activity is a major source of pollution in developing areas contributing dust and a variety of sediment and suspended solids. Improper storage or spillage of toxic chemicals from industrial sites can contaminate stormwater runoff from these areas. Other sources of contamination include lawns, gardens and parks which may contribute pesticides, herbicides and fertilisers, and illegal dumping of waste.

6.3.3 Factors influencing the degree of contamination

The composition of stormwater runoff generated from an urban area will vary significantly from catchment to catchment. The processes involved in the generation of pollutant loads are complex and large spatial and temporal variations persist down to small scales. One factor in determining the quality of stormwater is clearly the presence or absence of various sources, as listed previously.

Given that at least some of these pollution sources are present, a number of other factors also contribute to the level of contamination contained in runoff during any particular event. Also, the proportion of impervious surfaces and the general topography will affect the velocity of stormwater conveyance. This, in turn, affects levels of pollutant mobilisation through erosion and re-suspension (or ‘wash-off’). Similarly, the duration and intensity of the rain plays a role in determining what percentages of those pollutants present will be suspended and hence transported. The time between storm events is significant in relation to some pollutants : for example, longer antecedent dry periods will allow more time for pollutant build-up on runoff surfaces.

Other considerations when assessing the degree of contamination include seasonal factors (e.g. the rate at which leaf litter accumulates on catchment surfaces), the presence of any pollution control measures already in place and the extent of local community interest in environmental and catchment issues.

Even within a given storm event there will be variation in the runoff water quality. The most recognised effect is that of the ‘first flush’, a feature of wash-off related to both storm and catchment characteristics. The first flush effect refers to a pollutant concentration peak which precedes the flow peak, and is most recognisable in small catchments. First flush effects have been observed for suspended solids, hydrocarbons and heavy metals as well as dissolved pollutants (Peterson and Batley, 1992). Results from a study of 13 sites in and around Melbourne confirmed higher concentrations in initial runoff flows. In heavy storms, up to 48% of pollutants by mass were found in the first 37% of the runoff (Gutteridge, Haskins & Davey, 1981). A first flush effect is not always significant, and in certain hydrological or catchment conditions may not be recognisable. For example, climatic regions characterised by frequent, low intensity storms which continually wash surfaces may present little evidence of first flush (Stockdale, 1991).

6.3.4 Contaminants and some of their impacts

Contaminants in urban stormwater can be categorised as :

- Gross litter
- Suspended solids
- Oil and grease
- Nutrients
- Oxygen demanding materials (organic matter)
- Bacteria/micro-organisms
- Heavy metals and other toxicants.

Gross litter comprises a significant portion of stormwater pollution. By definition, gross pollution as it is referred to here is everything greater than 4 mm in diameter, including paper, plastics, organic litter, packaging materials, glass and metal. The main impact of gross pollution is loss of aesthetic qualities and potential for injury when in contact with the runoff. Gross pollution may serve as a visual indicator for the many invisible contaminants most likely present when gross litter is a problem.

Suspended solid levels in urban runoff are predominantly due to inorganic soil particles. Suspended sediment causes a variety of adverse consequences ranging from increased turbidity and reduced light penetration to direct interference with aquatic and sediment dwelling (benthic) life. It is widely recognised that many other stormwater pollutants are associated with suspended material, particularly heavy metals. Contaminants become bound to the smaller sediments in particular and are thus transported in the stormwater from catchment areas to waterways.

Oil and grease derive mainly from motor vehicles, and hence are associated with roads, parking bays and service stations. Some of these hydrocarbon compounds are known to be toxic to aquatic life at relatively low concentrations. They may also exert a detrimental effect on aquatic life by interrupting the entry of oxygen as they form a film on the water surface. Aromatic hydrocarbons are present in most vehicle exhaust emissions, and have been identified as being carcinogenic. Once bound to sediments, these aromatic hydrocarbons present long term dangers due to their high toxicity.

Stormwater nitrogen and phosphorous levels, when in excess, can have significantly damaging impacts on receiving waters. Because the majority of nutrient load is present in a soluble form it is easily taken up by algae. When favourable conditions exist, algal blooms can result, severely depressing oxygen levels (during decomposition) and releasing toxins resulting in fish kills and potential human hazard (Schueler, 1987).

Bacteria levels in stormwater generally exist far in excess of water quality standards for drinking, irrigation or contact beneficial uses (Schueler, 1987; Dillon and Pavelic, 1996). Originating from human and animal faecal matter, the main indicator is faecal coliform bacteria. Levels are exacerbated by warm conditions and sewer overflows during large events.

A wide range of heavy metals can be present in urban runoff. Most are highly toxic at small concentrations and can accumulate in the body to threshold levels. Within stormwater they pose a threat to humans both via direct contact and via accumulation in the food chain. Metals found in stormwater include Arsenic, Cadmium, Chromium, Copper, Cyanide, Lead, Nickel and Zinc. Heavy metals must be chemically available to present a toxic hazard, though all metals, whether bound to sediments or not, are potentially toxic, and slight changes in sediment chemistry could mobilise previously harmless contaminants.

6.3.5 Data on pollution concentrations in urban runoff

Relatively little data are available for use in the prediction of stormwater pollution concentrations based on catchment characteristics. From existing data it is apparent that pollution concentrations are extremely variable in the urban setting (Dillon and Pavelic, 1996; Schueler, 1987; Commonwealth EPA, 1993). A number of studies have been carried out throughout Australia and the world which attempt to correlate stormwater pollution with land use and management factors. What follows is by no means a comprehensive review of existing studies in the area, but rather a small number of examples demonstrating the variety of results and hopefully highlighting the variability of conclusions.

Schueler (1987) writing for the Washington Metropolitan Water Resources Planning Board in 1987 prepared a manual in which he makes reference to a “Simple Method” for estimating pollutant export from urban development sites. The empirical approach is based on data coming from studies in Washington and throughout the U.S.A. True to its name, the method is used to determine pollutant export (mass) based on rainfall, runoff coefficients and a mean concentration of pollutant in urban environments (averaged from empirical studies). These mean concentrations of various pollutants, named “C” values, are based on statistical analysis of over 300 runoff events in Washington and over 2,300 events at 22 sites drawn from a much larger national database (Schueler, 1987). An interesting conclusion from the study was that there was no significant difference in average pollutant concentrations between the eight widely different urban sites measured within Washington D.C.: also, no consistent correlations between pollution concentrations and storm volume or intensity were found (Schueler, 1987). However, there were significant differences between pollutant levels in relatively recent urban development sites and those in older, poorly maintained urban neighbourhoods. Table 6.3 summarises results from these studies, categorised into broad watershed conditions.

A brief review of other studies carried out in America can be found in *New Techniques for Modelling the Management of Stormwater Quality Impacts* (Liesko et al, 1993).

Due to specific regional hydrology and climatic characteristics it may be inappropriate to simply apply international data concerning stormwater pollution for use in Australia. A review of stormwater quality studies performed by Dillon and Pavelic (1996) examined 14 published studies from 18 sites, two thirds of which were from Australia. They acknowledge that “concentrations across the sites vary significantly”, though still consider it useful to make some general comments in relation to existing guidelines.

Parameters which did not meet **drinking water** guidelines on all occasions include Ammonia, Aluminium, Cadmium, Chromium, Iron, Lead, Turbidity and Faecal Coliforms. Parameters which did not always meet **irrigation** guidelines include Cadmium, Copper, Iron, Lead, Zinc and Faecal Coliforms. A summary table of results for various parameters is reproduced in Table 6.4, detailing ranges and mean concentrations with comparison to recommended guidelines for drinking, irrigation and livestock. However, rather than providing firm conclusions, the greatest value of such a review is to “point out what we do not know about the quality of urban stormwater” (Dillon and Pavelic, 1996).

Another thorough review of Australian and international (mainly US) studies was provided by CSIRO in 1992, focusing primarily on roadway runoff quality. It was concluded that lead, zinc and copper are the pollutants of major concern, comprising 90 - 95% of all metals observed (Peterson and Batley, 1992). Once again however, there are large variations in pollutants, particularly heavy metals, apparent from analyses at different sites around the world, as emphasised in 6.5 showing comparative metal concentrations. Peterson and Batley suggest that these variations are strongly related to the range of catchment characteristics, site activities and traffic flows.

Cadmium levels are virtually invariant across the range of sites, which include light industrial, residential, commercial and agricultural catchments. This suggests the possibility of a more remote source, perhaps entering the stormwater via atmospheric wash-out. Zinc is generally present in the highest concentrations, and in particular is associated with commercial centres and busy intersections. Lead and copper also follow this trend, indicating motor vehicles as the major source. The two catchments with housing only presented comparatively low levels of all heavy metals.

TABLE 6.3
MEAN CONCENTRATIONS OF POLLUTANTS IN URBAN ENVIRONMENTS
 (Source : Schueler, 1987)

POLLUTANT	NEW SUBURBAN NURP SITES (WASH., DC) mg/L	OLDER URBAN AREAS (BALTIMORE) mg/L	CENTRAL BUSINESS DISTRICT (WASH., DC) mg/L	NATIONAL NURP STUDY AVERAGE mg/L	HARDWOOD FOREST (NORTHERN VIRGINIA) mg/L	NATIONAL URBAN HIGHWAY RUNOFF mg/L
Phosphorous :						
Total	0.26	1.08	-	0.46	0.15	-
Ortho	0.12	0.26	1.01	-	0.02	-
Soluble	0.16	-	-	0.16	0.04	0.59
Organic	0.10	0.82	-	0.13	0.11	-
Nitrogen :						
Total	2.00	13.6	2.17	3.31	0.78	-
Nitrate	0.48	8.9	0.84	0.96	0.17	-
Ammonia	0.26	1.1	-	-	0.07	-
Organic	1.25	-	-	-	0.54	-
TKN	1.51	7.2	1.49	2.35	0.61	2.72
COD	35.6	163.0	-	90.8	>40.0	124.0
BOD (5 day)	5.1	-	36.0	11.9	-	-
Metals :						
Zinc	0.037	0.397	0.250	0.176	-	0.380
Lead	0.018	0.389	0.370	0.180	-	0.550
Copper	-	0.105	-	0.047	-	-

The critical difficulty in using data such as these to establish and validate runoff and pollutant load models is the high number of model parameters required. In their summary of Australian research in this field, Allison and Chiew (1995) come to the following conclusion :

“It is conceivable that there may already be sufficient data to provide rough estimates of pollutant loads generated from large urban areas. However, because of the large variability in pollution characteristics, specific monitoring in the area of interest may always have to be carried out if detailed information on the pollution characteristics for the area is required.”

6.3.6 Conclusion

Urban areas inevitably generate considerable amounts of undesirable waste products, or pollutants, which contaminate runoff. Due to the nature of Australian conventional stormwater management systems being efficient at collection and local disposal, with little or no pre-treatment, these contaminants pose a growing threat to our immediate environment. Understanding the sources of pollution and the mechanisms which drive pollutant build-up, wash-off and transport will help focus attempts to alleviate the problem. To date, research has revealed the variable nature of urban stormwater contaminants. Heavy metals are a focal point, due to their relatively high concentrations and potential toxicity. Further research must pinpoint where critical pollutants are located within the ‘pollution column’ and hence suggest strategies for control. This will almost inevitably lead to changes at all levels of collection, transport and subsequent disposal or use of stormwater.

TABLE 6.4
TYPICAL CONCENTRATIONS OF CONTAMINANTS IN STORMWATER AND WASTEWATER AND
ASSOCIATED GUIDELINES FOR VARIOUS BENEFICIAL USES

PARAMETER	NWQMS GUIDELINES					STORMWATER		WASTEWATER		STORMWATER		WASTEWATER	
	DRINKING	RAW DRINKING	IRRIGATION	LIVESTOCK	RANGE	MEAN	RANGE	MEAN	% ANALYSES THAT MEET GUIDELINES				
									DRINKING	IRRIGATION	DRINKING	IRRIGATION	
pH	6.5 – 8.5	6.5 – 8.5	6 - 8.5 groundwater 6 – 9 surface water		6.7 – 8.5 ¹⁰	7.7	6.9 – 8.7 ¹⁰	7.6	100	100	90	100	
EC (µS/cm)	mg/L	mg/L	mg/L	mg/L	mg/L 197 – 8140 ⁷	mg/L 3250	mg/L	mg/L					
TDS	500 ^a	1000	Refer to Section 9.2.3 ANZECC 2000	Refer to Table 9.3.3 ANZECC 2000	44 – 208 ¹⁰	118	520 – 4940 ⁹	1950	100		0		
Calcium				1000	20 – 38.6 ³	26.8	21 – 236 ⁵	114					
Magnesium					3 – 15 ³	8.7	2.8 – 157 ⁵	59.2					
Sodium	180 ^a				12 – 116 ³	54	41 – 1540 ⁵	581	100		65		
Potassium					4.6 – 5.8 ³	5	8.1 – 37 ⁵	18.6					
Sulphate	250 ^a			1000	8 – 505 ⁴	146	5.5 – 152 ⁴	67	100		100		
Chloride		400	175 – 750		15 – 173 ⁵	62	38 – 494 ³	223	100		65		
Ammonia-N		0.01			0.03 – 8.8 ¹⁴	0.9	1.6 – 27.7 ¹¹	11.5	55		0		
Total-N					0.6 – 5.6 ¹⁵	2.1	6.1 – 44.2 ¹¹	20.3					
Nitrate-N		50 infants under 3 months, 100 otherwise		400	0.1 – 6.2 ¹³	1.5	0.1 – 19.5 ⁴	6.4	100		75		
Nitrite-N		3.0		30	0.1 – 0.3 ³	0.1	0.5 – 1.1 ³	0.9	100		65		
Phosphate-P			Refer to Table 9.2.19 ANZECC 2000		0 – 4.2 ¹⁰	0.5	2.3 - 9 ⁴	5.5					

TABLE 6.4 (cont.)

PARAMETER	NWQMS GUIDELINES				STORMWATER		WASTEWATER		STORMWATER			WASTEWATER	
	DRINKING	RAW DRINKING	IRRIGATION	LIVESTOCK	RANGE	MEAN	RANGE	MEAN	DRINKING	IRRIGATION	DRINKING	IRRIGATION	WASTEWATER
Aluminium	mg/L	0.2	5.0	0.5	0.3 – 1.4 ²	0.9	0.1 – 2.6 ⁸	0.5	0	100	50	100	100
Arsenic		0.007	0.5	5.0	0 – 0.003 ³	0.002	0.004 – 0.007 ²	0.006	100	100	100	100	100
Boron		4.0	B	5.0	0.08 – 0.17 ²	0.13	0.6 – 2.7 ²	1.7	100	100	0		
Cadmium		0.002	0.01 – 2 kg/ha ^b	0.01	0 – 0.011 ⁹	0.003	0 – 0.002 ¹¹	0.001	80	90	100	100	100
Chromium		0.05	0.01 – 1.0 ^b	1.0	0 – 0.58 ¹⁰	0.07	0 – 0.1 ¹¹	0.02	90	100	80	100	100
Copper		1.0	0.2 – 140 kg/ha ^b	0.5	0 – 0.48 ¹¹	0.08	0.001 – 0.12 ¹¹	0.04	100	90	100	100	100
Iron		0.3	0.2 – 10 ^b		2.4 – 7.3 ³	4.0	0.03 – 1.6 ⁹	0.5	0	0	80	80	80
Lead		0.01	2.0 – 260 kg/ha ^b	0.1	0 – 0.53 ¹¹	0.25	0 – 0.03 ¹¹	0.01	10	45	90	100	100
Manganese	0.1 ^a	0.5	0.2 – 10 ^b		0.04 – 0.11 ⁴	0.08	0.02 – 0.08 ¹⁰	0.04	100	100	100	100	100
Zinc	3 ^a		1.0 – 300 kg/ha ^b	20.0	0.02 – 5.8 ¹⁰	0.91	0.0 – 0.26 ¹¹	0.09	90	90	100	100	100
Suspended solids					6 – 476 ¹³	164	11 – 250 ¹⁰	65					
Turbidity (NTU)	5 ^a				12 – 34 ⁴	18			0				
Total organic carbon					5.8 – 61 ⁸	23.6	14 – 61 ⁸	37.5					
BOD					3 – 73 ¹⁰	23	8 – 80 ¹⁰	25					
?					0.2 – 0.45 ³	0.35	0 – 0.5 ⁸	0.3					
Coliforms	Per 100 mL	<10		100	4000 – 11700000 ³	3900000	1000 – 16500 ³	8200	0		0		
Faecal coliforms	0	0	1000	1000	0 – 600000 ¹⁰	150000			0	0			

^a aesthetic guideline^b specific to vegetation species^{f–15} number of analyses

Source : ANZACC, 2000; Dillon and Pavelic, 1996; NHMRC/ARMCANZ, 2001.

TABLE 6.5

**COMPARATIVE MEAN HEAVY METAL CONCENTRATIONS IN ROAD RUNOFF
OVERSEAS AND LOCALLY**

SITE	SEATTLE AREA, USA	ORLANDO, FLORIDA	GERMAN HIGHWAYS	MELBOURNE, VICTORIA	SYDNEY, NSW	YARRA RIVER, VICTORIA
METAL	MEAN CONCENTRATION, MICROGRAMS PER LITRE					
Arsenic	13	94	—	—	—	—
Cadmium	0.7	25	5	10	0.2	.33 - .49
Chromium	7	46	12	580	—	—
Copper	20	513	91	480	14	2.9 - 6.6
Lead	210	1580	203	690	20	2.2 - 16.1
Nickel	12	30	—	70	—	—
Zinc	120	423	430	5800	150	30 - 55

Note : “—” implies *not analysed*.

Source : Peterson and Batley, 1992

A summary of metal concentration data from thirteen Melbourne sites shown in Table 6 indicates a number of possible trends.

TABLE 6.6

**MEAN CONCENTRATIONS OF METALS IN RUNOFF AT DIFFERENT SITES
NEAR MELBOURNE**

SITE	1	2	3	4	5	6	7	10	13
METAL	MEAN CONCENTRATION, MICROGRAMS PER LITRE								
Lead	490	690	330	70	510	530	360	120	160
Zinc	5800	1100	1000	320	310	1230	710	340	210
Cadmium	11	9	9	10	9	10	10	9	10
Copper	480	120	45	50	59	91	18	27	26
Chromium	580	24	63	130	120	24	20	20	23
Nickel	20	31	69	10	30	22	17	28	22

Source : Gutteridge, Haskins & Davey, 1981

Site Descriptions :

1. Light Industrial
2. Major Traffic Junction
3. Mixed Urban - includes industrial and service stations
4. Mixed residential, light and heavy industry
5. Mixed industrial, commercial and residential
6. Residential and two shopping centres
7. Housing
10. Housing
13. Orchards and light grazing.

6.4 QUALITY STANDARDS FOR RECEIVING WATERS

Table 6.7: Guidelines for the protection of human consumers of fish and other aquatic organisms

Table 6.8: Summary of water quality Guidelines for recreational waters

Table 6.9: Summary for quality Guidelines for raw waters for drinking purposes subject to screening

Table 6.10: Summary of Guidelines for irrigation water quality

Table 6.11: Water quality Guidelines for livestock watering

6.5 QUALITY STANDARDS FOR AQUIFER RECHARGE

Table 6.12: Summary Guidelines for protection of aquatic ecosystems

[These sections omitted from the Student Edition]

See page xi for details of distributors of the full Handbook

7. STORAGES FOR POLLUTION CONTROL (Category 2 systems)

7.1 GENERAL BACKGROUND

Perhaps the pollution control or filtration system which the general public most easily recognises in the urban landscape is the trash rack, GPT (gross pollution trap), or sediment (collection) basin constructed in a local drainage path or waterway. Such installations tend to be one of the earliest initiatives taken by municipal authorities newly faced with the task of managing urban stormwater. While these components have a valid role to play in total catchment stormwater management, the complete absence of *storage* associated with them, even short-term detention storage, disqualifies them from inclusion – as isolated elements – among Category 2 systems (see Section 4.3.1). ‘At source’ stormwater pollution control as interpreted in this Chapter may involve some member of this group, typically a GPT or area of permeable paving with built-in filter, but it must also be associated with a component **retaining** the cleansed stormwater.

Chapter 5 of the Handbook is devoted to meeting the need, primarily, to control flood flows in the urban environment using on-site retention (OSR) practices. The approach which must be used to design systems intended, primarily, for *pollution* control differs from the procedures described in Chapter 5 in a number of significant respects. The main differences are explained as follows :

All Category 1 (flood control) systems must include some form of filter located upstream of the required storage : this element of design is explained comprehensively in Section 1.3.3. Category 2 (pollution control) systems also require filters located upstream of storages : a typical arrangement is illustrated in Figure 3.5a. **The capacity flows of the respective filters provide the single most important difference between Category 1 systems and those of Category 2.** Both are matched to the highest flows, Q_{peak} , likely to pass through them in their respective critical design events, but it is the significantly different events themselves which provide the distinction :

- Q_{peak} of a treatment unit placed upstream of a flood control installation (Category 1) is determined by catchment area and the appropriate ARI/critical storm duration which, in combination (interpreted as a rainfall intensity) produce the most demanding **entry** condition for the device or system. Critical storm duration is therefore (usually) equal to t_C , as previously discussed (Section 4.2.6), and ARI may be drawn from the range 2-years to 10-years for ‘minor’ system components and 50-years to 100-years where the storage is a ‘major’ system component. [There are rare circumstances in which $T_{C(total)}$ and t_C coincide]
- Q_{peak} of a pollution control treatment unit (Category 2) is determined by catchment area and rainfall intensity, also for t_C , and ARI = 0.25-years (Australian suggested practice, NSW Dept of Housing, 1998). Or it may arise from ‘continuous simulation’ modelling aimed at achieving retention of a specific percentage of annual runoff volume : in this case, the peak flow before bypass is called Q_{lim} (see Table 3.2). An essential characteristic of Category 2 installations is that, normally, they bypass all flow exceeding their Q_{peak} or Q_{lim} capacities (see Figure 3.6).

Illustrations of the former (Category 1 installations) include **infiltration surfaces** which fulfil a highly effective stormwater cleansing role : much greater flow capacities than $Q_{0.25}$ or Q_{lim} can be justified in these installations because of their amenity value, for example as car-parking areas (see Example 5.1, Section 5.2). The range of **“leaky” devices** shown in Figures 1.3 and 1.5 and Example 5.2, have provision for pre-treatment (rainwater tanks, filters or sediment traps) or they retain incoming sediment in cases where these loads are too small to affect performance over their expected lifespan periods : these devices also must have relatively high filter-entry flow capacities. **Infiltration ponds** (‘dry’ ponds) integrate within their flood control capability a strong treatment role provided by the presence of a pond where sediment can settle, underlain by a permeable filter bed : these, too, are capable of accepting flows of relatively high order (see Example 5.4). The ‘connectedness’ or overlapping of design objectives – **firstly**, flood control and, **secondly**, pollution control – referred to in Section 4.1 is clearly indicated by these illustrations.

The types of installations falling into Category 2 show a *reversal* of this priority with treatment of pollution emerging as the first design objective, and on-site storage of stormwater, as the second. The range of devices, systems, products and facilities available to achieve the first goal is extensive and growing : the available offerings are described in many current documents [Water and Rivers (WA), 1998; NSW Dept of Housing, 1998; CSIRO, 1999; Melbourne Water, 2005; Engineers Australia, 2006; Dept of Water, Western

Australia, 2007] and briefly reviewed in Chapter 2 of this Handbook. It is to this wealth of information and the accompanying performance data that designers must go to select the most appropriate, most cost-effective and most sustainable filter systems for their projects. Further detailed comment on this aspect of design/selection – beyond the following brief discussion - is outside the scope of the present chapter.

Systems used in the process of treating urban stormwater *at source* are of two basic types: **first**, gross Pollution collectors which filter leaves, cans, coarse sediment, etc. and which require regular cleaning, for example, after each major storm. These devices are widely used in Australia and are well known to all practitioners in the stormwater field. The **second** type employs the ‘treatment train’ concept – a succession of *different* measures – which can be effective, collectively, in reducing concentrations of pollutants ranging from sediment through to those (pollutants) which occur in urban runoff in soluble forms. The first element of treatment offered by such systems is that of *filtering* the sediment by slowing the velocity of water-borne particles through contact with vegetation or sand (or similar) or by ponding. [This also captures some pollutants – portions of the hydrocarbon, heavy metal and phosphorous load – which adsorb onto particle surfaces.] The succeeding elements of treatment involve (next) a gravel sub-structure or “soakaway” and then percolation, under gravity, through the parent soil body (see Figures 1.7 and 3.5a).

The high standards of stormwater treatment required of installations discharging *directly* to sensitive receiving (water) environments cannot be delivered by the **first** type described above, used alone. There are three reasons for this: first, they are incapable of capturing the very fine particulate matter which is harmful to aquatic ecosystems; secondly, they are not designed to remove dissolved pollution components – phosphorous, nitrogen and heavy metals – and, thirdly, the irregular maintenance schedules frequently employed with them can lead to degradation of retained material, converting it into point source toxic pollution. Certainly, efficient and effective GPTs are important elements in the “toolkit” of the stormwater manager, but they should be well maintained and integrated with additional treatment components in any competently planned facility.

This *additional treatment* can be provided through ‘treatment train’ systems incorporating, primarily, the gravel-filled (or similar) “soakaway” referred to above. [Porous and permeable paving includes such an element as sub-structure – see Figure 2.3.] Its role in treatment is similar to that of the *trickling filter* – well understood in wastewater treatment practice – and shows impressive performance in the removal of hydrocarbons, heavy metals and nutrients and reduction in chemical oxygen demand in stormwater (Pratt et al, 1998; Pratt et al, 2001; Napier and Jefferies, 2003; Sansalone and Teng, 2004; Bean et al, 2007). A further, important benefit of the “soakaway” can be its role – in appropriate circumstances – in passing any remaining dissolved pollutants into the parent soil for further treatment (Kotlar et al, 1996; Mottier et al, 2000; Detry et al, 2003; Weiss et al, 2006; Jefferies and Napier, 2008a and b). ‘Appropriate circumstances’ referred to here include clay soils, sandy clays and deep, fine sands but specifically excludes coarse, shallow sands with high groundwater tables (see Mikkelsen et al, 1997; Fischer et al, 2003).

Retention storages embraced by Category 2 **Strategy A** or **Strategy B** ‘source control’ systems (see Section 4.3.1), are predominantly **terminal** or off-line installations. The bulk of outflow from them (approximately 90% for Strategy A systems) passes, typically, as seepage into local parent soil, as explained above, rather than into a downstream drainage path. *Overflow* shown in Figure 3.5a represents only 3% – 5% of average annual runoff volume. In extreme circumstances (see Sections 7.3 and 7.5), some retained water *may* be passed to an aquifer or transferred by slow-drainage to a downstream waterway at very low flow rates. ‘Treatment train’ systems retain much of the pollution load passing to them, particularly sediment and products of bio-degradation: they are therefore subject, after some years of service (termed ‘lifespan’), to partial reconstruction or complete replacement. The first half of Chapter 7 is devoted, mainly, to solving the problem posed by **Strategy A** systems :

What ‘size’ of sub-structure should be provided – following initial (coarse) filtration – to achieve retention and hence substantial treatment of 90% average annual runoff from a specified catchment?

The analysis, following, employs ‘continuous simulation’ modelling to answer this question taking account of the two factors which dominate such dimensioning – climate and local soil type. The second half of Chapter 7 is devoted to application of the results of this enquiry to the particular case of swales receiving surface runoff from, in particular, residential streets.

Of the three stormwater management strategies listed in Section 4.2.1, only the **yield-minimum** option meets the requirements of the filter/sub-structure integrated systems reviewed above. Category 2 (**Strategy A**) pollution control installations are therefore designed for storage of 90% of annual average runoff volumes generated in their contributing developed catchments without any account being taken of runoff volumes generated in the same catchments in their pre-developed states.

An apparent departure from this principle occurs when installations fulfilling the dual purposes of flood control and pollution control are to be designed. In such cases, the **regime-in-balance** strategy may be employed to fulfil the requirements of flood control with ARI fixed by ‘minor’ (2 – 10 years) or ‘major’ (50 – 100 years) system demands. However, the pollution control element of the installation (Category 2 system) should still – even under these circumstances – be designed according to the **yield-minimum** strategy. The outcome, in terms of size of sub-structure storage volume, etc., then involves the comparison and final decision process discussed in Section 7.6.2.

The most effective way to determine the basic information needed to design retention sub-structures such as “soakaways” for pollution control installations in the full range of soil types encountered at a particular location, is to follow the 13-step ‘continuous simulation’ procedure, and following, set out in Section 3.4. The outcome, in such circumstances, is a set of curves similar to those shown as Figure 3.7, but peculiar to the location of interest. The availability of these curves enables the designer to carry out Procedure 5A, below, with the preferred data base for the particular location.

It is presumed in what follows, however, that practitioners will take advantage of the ‘generalised’ approach expressed in the set of graphs included as Figure 7.1 and in Appendix C based on the five climate zones illustrated in Figure 3.8. These require knowledge of, only, location (represented by $i_{10,1}$, rainfall intensity), soil type (measured or estimated **site** hydraulic conductivity, k_h), and physical characteristics of the contributing catchment, to design ‘simple’, Strategy A pollution control retention systems. The basic design process involves the three steps set out in Procedure 5A, Section 7.2.2.

In Summary (see Figure 3.5a) : A highly competent (Strategy A) stormwater pollution treatment/control system will consist, typically, of an upstream ‘coarse filter’ unit which may be of Type 1 or Type 2 :

- Type 1 :** device that uses processes drawn from the wide range of available technologies to retain gross pollution including coarse/medium sediment, and that delivers a **concentrated** or ‘point’ outflow, for example by pipe, OR,
- Type 2 :** device that retains water-borne sediment by means such as sand and/or geotextile (non-woven) filtration and that delivers a **distributed** outflow [“distributed” here means “from large plan area”. Type 2 is particularly applicable to porous/permeable paving systems: the pollution treatment/control performance of these can be evaluated in much the same way as described, following, for Type 1 systems.]

followed by a *retention* device whose dimensions (plan area, A_{avail} , and depth, H) have been determined by applying Procedure 5A. [A lesser standard of pollution treatment/control based on “first flush” considerations can be provided by applying Procedure 5B.]

In cases where a Type 1 coarse filter is used, the designer must consult an appropriate reference (local or regional information) for advice on the most suitable bioretention elements (media, vegetation, etc.) which should be incorporated into the device dimensioned by Procedure 5A. Common practice at this point (in the design process) is to evaluate the likely treatment performance of the system using the MUSIC package (eWater CRC, 2010). This evaluation is limited to the bioretention elements contained by the device, only, and cannot account for the *additional treatment* taking place as a consequence of “exfiltrate” percolation downwards through the parent soil body (see Figure 1.7). This (latter) component of treatment is far from trivial: an approximate or ‘notional’ evaluation for this can be formed from the literature (see previous page). It is recommended that the **two** evaluations are combined to yield an overall performance. Such a *total* treatment assessment/evaluation may be acceptable in many ‘real world’ circumstances.

The latter approach should **not** be considered in field situations where high or rising watertables are present (see Item 6, Section 3.8). In such cases, it is advised to incorporate as great a level of treatment as is possible within the retention device (media, vegetation, etc.) and to quarantine it from the parent soil mass using an impervious sheet (see Section 10.5, Engineers Australia, 2006).

7.2 DESIGN OF SIMPLE STRATEGY A SYSTEMS : PROCEDURE 5A

7.2.1 Introduction

The procedure starts by identifying the **climate band**, based on rainfall intensity, $i_{10,1}$, in which the location of interest lies : the graph-sets are found in Appendix C. The stormwater (cleansed) **retention efficiency**, **R = 90%** is recommended for all installations. This leads to the choice of an associated value for q_{lim} which is appropriate to the selected climate band. Taking this graph, the point determined by :

- site (modified) soil permeability (k_h), and,
- device-to-catchment (EIA) area ratio, A_R ,

is identified.

Three possible cases arise :

- Case 1 :** the plan area available for the on-site retention device is *larger* than that required by the installation of minimum recommended depth – $H_{min} = 0.30$ m.
- Case 2 :** The plan area available for the on-site retention device corresponds to a device depth, H , falling *within* the recommended range – $H = 0.30$ m to 1.50 m, inclusive.
- Case 3 :** The plan area available for the on-site retention device is *smaller* than that required by the installation of maximum recommended depth – $H_{max} = 1.50$ m.

Note that area available, A_{avail} , is considered to be, typically, **within** the contributing catchment area, A_{EIA} .

STEP 2 of Procedure 5A explains the course of action which the designer must follow in each case.

In STEP 3, the q_{lim} flow determined for the climate band is interpreted into a catchment-related flow through :

$$Q_{lim} = 0.9 \times A_{EIA} \times q_{lim} \text{ L/s} \quad (7.1)$$

The designer is then required to ‘match’ this capacity flow (before bypass) to a preferred and/or available proprietary or other filter unit or system. This process requires the designer to have available detailed technical information on a wide range of filters which might be used upstream of the retention device and which pass acceptably-treated stormwater to that device.

Where the circumstances of a particular proposed installation fall into Case 3, above, and where Strategy A compliance is mandatory, then the process moves to Procedure 6A, Section 7.3, intended for more complex solutions. **Certain cautions/warnings need to be heeded in making this transition.**

The process reviewed to this point is focussed on graphs, etc. (Figure 7.1 and Appendix C) derived from ‘continuous simulation’ modelling for “soakaway” devices filled with gravel. [The focus could have just as easily been placed on gravel/pipe combinations or, even, on ‘dry’ ponds.] The primary reason for this is recognition that the gravel-filled device is likely to be the most common form used, because of its ‘trickling filter’ capabilities. However, use of a simple formula (Eqn. 7.4) enables the design outcome determined for a gravel-filled “soakaway” to be converted into the dimensions of an in-ground device of any other type, including a ‘dry’ pond. A more comprehensive explanation of this process is contained in Section 7.6.3.

The steps of Procedure 5A are explained in the next section.

7.2.2 PROCEDURE 5A : Strategy A design for sub-structure storages receiving *filtered* runoff from paved surfaces

Basic data requirements :

- Location :** Identify climate group based on range of $i_{10,1}$ rainfall intensity and, hence, graph-set selected from Figure 7.1 or Appendix C.
- Value for R :** The value recommended for retention efficiency in all design cases addressed in the Handbook is **R = 90%** [graphical information for other values of R is available from the ARQ document (Engineers Australia, 2006)].
- Value for Q_{lim} :** Identify the corresponding value of limiting capacity flow before bypass, q_{lim} , and hence Q_{lim} (see Eqn. 7.1).
- Soil :** Site hydraulic conductivity, k_h m/s, and Moderation Factor, U, which converts site (borehole) test results into values appropriate for installations (see Footnote, Section 5.1.2). Clay soils : U = 2.0; sandy clays : U = 1.0; sandy soils : U = 0.5.

Equivalent paved area of contributing catchment : A_{EIA} m².

Available space for device (within A_{EIA}) : A_{avail} in m². Hence, $A_{avail}/A_{EIA} = A_R$.

Void space ratio of proposed retention device, e_s :

gravel-filled : $e_s = 0.35$; gravel-pipe combination : $e_s = 0.5$

‘milk crate’ cells : $e_s = 0.95$; ‘dry’ ponds : $e_s = 1.00$.

Preferred/available treatment units file : An inventory of effective filter devices and systems together with their capacity flow characteristics should be consulted to select a unit (see Figure 3.5a) suited to the particular catchment circumstances. Its design, including provision for **bypass** (see above), should be completed before Procedure 5A is commenced. The filter unit should have capacity, Q_{lim} , determined according to Eqn. (7.1).

The outflow from the (filter) device may be delivered as a concentrated entry (Type 1, see Section 7.1) or as a distributed entry from a porous/permeable paving surface (Type 2). In the latter case, attention must be given to the consequences for hydraulic conductivity, k_h , of sediment accumulation over the ‘lifespan’ of the paving (see Sections 3.1.2 and 3.1.3 and Procedure 1A or 1B, Section 5.1.).

Three-step process to determine device depth H and select treatment unit:

STEP 1 :

Enter Figure 7.1 (illustrative graph) or graph selected from Appendix C, at the adopted value of site hydraulic conductivity, **adjusted in accordance with Moderation Factor, U**;

Enter Figure 7.1 at the value of A_R as applies.

STEP 2 :

The point of intersection in the k_h versus A_R plane (Figure 7.1) identified in STEP 1 lies :

Case 1 : Above the family of curves (point “1” in Figure 7.1), which implies that the plan area available for the on-site retention device is **larger** than required by the installation of minimum recommended depth, namely $H_{min} = 0.30$ m.

In this event, the designer may use this value ($H_{\min} = 0.30$ m) for device depth, H , along with the corresponding **smaller** value of A_R given by the curve. In these circumstances the *adjusted* device plan area is called “ A_{avail} ” in STEP 3. The design process continues in STEP 3.

Case 2 : Within the family of curves (for example, point “2” in Figure 7.1), which implies that the plan area available for the on-site retention device can be matched to a depth, H , falling **between** the recommended limits – $H_{\min} = 0.30$ m to $H_{\max} = 1.50$ m, inclusive; in this event, A_{avail} retains its initial value in STEP 3. The design process continues in STEP 3. **It is strongly recommended that the process go no further than STEP 3, particularly where significant dissolved pollutants are considered likely in the catchment runoff. Heeding this caution implies selecting a design depth from the family of curves bounded by the recommended limits if at all possible. The consequence of adopting a solution arising from Case 3, following, is a level of full treatment less than that recognised as Strategy A. [The ARQ document (Engineers Australia, 2005) allows this option.] A further WARNING to this effect is contained in Section 7.3.**

Case 3 : Below the family of curves (for example, point “3” in Figure 7.1), which implies that the plan area available for the on-site retention device is **smaller** than required by the installation of maximum recommended depth, namely $H_{\max} = 1.50$ m. In this event, adopt $H_{\max} = 1.50$ m as the device depth and continue the design process in STEP 3 : A_{avail} retains its initial value. (This **must** be followed by Procedure 6A.) The designer may elect to use a value for H which is **smaller** than 1.50 m; the *process* to be followed is unaffected by this decision.

STEP 3 :

Report the dimensions and details of the filter device, including details of the limiting bypass flow, and of the retention system as plan area, A_{avail} , from STEP 2 and H (or $H_{\max}$), as applies, also from STEP 2. Provision for overflow must be included with these details.

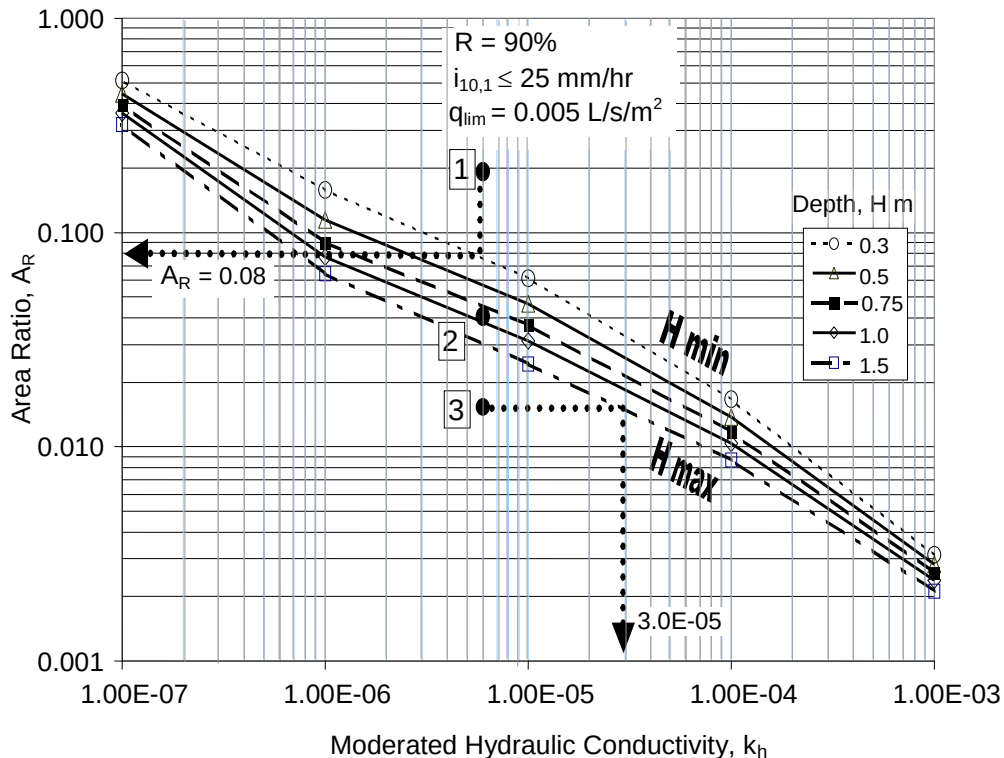


FIGURE 7.1 : Graph for retention efficiency, $R = 90\%$ (recommended value) for gravel-filled “soakaways” in Southern Australia (see Figure 3.8). “Soakaway” depth range : 0.30 – 1.50 m. Soils with moderated $k_h > 1.0 \times 10^{-4}$ m/s are, typically, unsuitable for treating *dissolved pollutants* in catchment runoff (see Mikkelsen et al, 1997; Fischer et al, 2003)

[The figures “1”, “2” and “3” relate to Example 7.1, Section 7.7.]

7.3 MORE COMPLEX STRATEGY A SYSTEMS : PROCEDURE 6A

7.3.1 Introduction

In the ‘simple’ systems reviewed above, water removal is by percolation to the host soil, **only**. More complex systems require the designer to consider ‘hydraulic’ abstractions made while the routing process – inflow/storage/percolation – is taking place.

In these situations, retention device **depth** required – to achieve retention and on-site abstraction of 90% average annual runoff – may be greater than the practical maximum (depth) offered in the families of curves provided in Figure 7.1 or Appendix C. This problem can be solved through **accelerated water removal** (from storage) using bores or drainage pipes specially designed for the purpose. (This approach is introduced in Section 3.5.4.) The quest for these solutions lies at the heart of Procedure 6A.

Procedure 6A commences with STEP 4. This is intended to convey the clear signal to the designer that problems of cleansed stormwater storage should be solved, first, as ‘simple’ systems (by the three steps of Procedure 5A) if at all possible before the move to a more complex solution is made. However, Procedure 6A also carries a WARNING: it should be avoided completely and design limited to Cases 1 and 2 (Procedure 5A) only, wherever a significant dissolved pollution load passing from the upstream filter is considered likely.

The issue facing the designer may be stated as follows : given the device plan area, A_{avail} , and depth equal to H_{max} – for the particular site condition identified in STEP 2 (Case 3) of Procedure 5A – what rate of (water) abstraction from the installation, *other than by percolation*, is needed to produce a successful system? [The designer may adopt a lesser depth than 1.5 m – say 1.0 m – in the following process.]

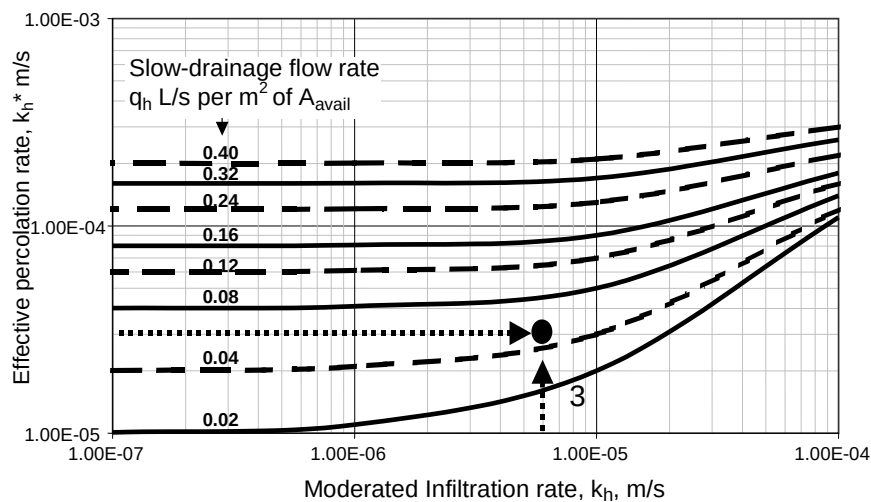


FIGURE 7.2 : Flow curves for accelerated water removal from in-ground retention devices

The two values, A_{avail} , and H_{max} , in any given set of circumstances, correspond to a **target** hydraulic conductivity value (‘to the right’ of point “3”) in the illustrative graph, Figure 7.1, which coincides with a successful outcome derived from the original ‘continuous simulation’ analysis. **The required, water (volume) removal outcome corresponding to this case can be achieved through ‘hydraulic’ drainage acting conjunctively with percolation.** This target permeability (‘to the right’) is called “**effective percolation rate, k_h^*** ”.

The percolation flow rate (host soil) beneath the retention device may be computed from the available data (site moderated k_h and plan area A_{avail}). Also, the *target* outflow rate may be calculated (from A_{avail} and k_h^*), so the required ‘hydraulic’ flow rate, q_h , can readily be found: it is the *difference* between the two calculated values. Values of this parameter for the full range of soil conditions are presented in Figure 7.2, where:

Horizontal axis : moderated hydraulic conductivity at site = $(k_h) \times$ Moderation Factor, U (used in STEP 1);

Vertical axis : effective percolation rate, k_h^* , stated in m/s - required with A_{avail} - obtained through the process described above and reported in STEP 4, following;

Family of curves (Figure 7.2) : recharge flow rate (or slow-drainage flow), q_h , stated in **L/s per m² of area, A_{avail}** , which must be abstracted from the device whenever it contains filtered stormwater. A factor of safety of 2 has been introduced in the process of obtaining the curves presented in Figure 7.2. **If this outflow contains significant dissolved pollutants it is considered unsuitable for discharge to aquifers or other sensitive (water) receiving environments.**

Recognition of the presence of an aquifer beneath a site where recharge of cleansed storm runoff is considered possible, should be followed by an environmental impact assessment as discussed in Procedure 4A, Section 5.1. If favourable, a review of available geological-geotechnical reports to ascertain its basic characteristics should be conducted – depth, standing water level, water quality, potentiometric gradient, local use (potable, irrigation, industry, etc.), likely yield per bore and, hence, potential rate of recharge, q_r . [Potential rate of recharge may be taken as 50% rate of yield (Pavelic et al, 1992).]

In situations where the required accelerated emptying is to be achieved by slow-drainage pipeline (see Procedure 4B, Section 5.1), the main restriction on the quantity of flow which can be abstracted is likely to be head difference between the in-ground device and the pipe outlet. This option is therefore unavailable in terrain which is very flat; but the flows involved – Q_{hA} calculated using Figure 7.2 and Eqn. (7.2) – are, typically, so small that this constraint is rarely preclusive.

Steps in the design procedure are set out in Procedure 6A, Section 7.3.2, commencing with STEP 4, recognising that the design process is a continuation from STEP 3 (Case 3) of Procedure 5A.

7.3.2 PROCEDURE 6A : Strategy A design for sub-structure storages with :

(a) aquifer access, or

(b) potential for ‘slow drainage’,

receiving filtered runoff from paved surfaces or from filter-integrated systems.

At this point in the design process, the filter unit has been selected and dimensioning of the installation as a ‘simple’ system (plan area A_{avail} and depth, H_{max}) – including provision for bypass (Q_{lim}) and device overflow – have been completed (STEPS 1-3 of Procedure 5A) before Procedure 6A is commenced.

STEP 4 :

Locate ‘point’ in illustrative graph, Figure 7.1, or on the appropriate graph from Appendix C chosen at the commencement of Procedure 5A, where A_R , given by A_{avail}/A_{EIA} , coincides with “soakaway” depth, $H = H_{max}$. [The ‘point’ corresponds to a significantly **higher** value of k_h than the (moderated) site value used in STEP 1, above.] This (new) value is called, “**effective percolation rate**”, k_h^* .]

Note : The designer may adopt a lesser value in place of $H_{max} = 1.50$ m, for example $H = 1.0$ m; this leads to a higher value of effective percolation rate, k_h^* , than that identified above. The procedure, below, is unaffected by this decision.

Locate ‘point’ in Figure 7.2 corresponding to :

- the **site** (moderated) hydraulic conductivity, k_h (horizontal axis); and,
- effective percolation rate, k_h^* (vertical axis);

Determine the ‘hydraulic’ flow rate, q_h , in L/s per m² of retention device plan area, A_{avail} , given by the family of curves.

STEP 5a (aquifer access) :

Calculate number of bores, “n”, needed, with natural percolation, to achieve the required (total) outflow rate : this requires ‘hydraulic’ flow, Q_{hA} , where :

$$Q_{hA} = [(A_{avail}) \times (q_h)] \text{ L/s} \quad (7.2)$$

and, $n \cdot q_r \geq Q_{hA}$, where q_r = (individual) bore recharge rate, L/s; hence number (integer) of bores, “n”.

OR

STEP 5b (slow-drainage pipeline) :

Calculate slow-drainage flow rate, Q_{hA} , by Eqn. (7.2).

Note : The *actual* flow rate in the slow-drainage pipeline may be any flow greater than Q_{hA} , however, if it is *much* greater, it may result in the outflow coinciding with the falling limb of the flood wave, causing the system to act more like a conventional OSD facility than its intended use as a retention device. (See Special Note in Section 3.5.4 and, also, Section 5.4.)

STEP 6 :

Report details of the **complete** installation including filter system (Type 1 or Type 2, as described in Section 7.1), retention device dimensions, and, from STEP 5 :

(a) “n” bores at q_r m³/s per bore; **OR** (b) ‘slow-drainage’ pipeline, flow Q_{hA} m³/s.

7.4 DESIGN OF SIMPLE STRATEGY B SYSTEMS : PROCEDURE 5B**7.4.1 Introduction**

Under **Strategy B** (see Section 4.3.1) pollution control using retention installations, only the ‘first flush’ of all storm runoff is treated and, hence, only the runoff volume which conveys this pollution to the device need be stored. Design of ‘simple’ systems following this approach, is the subject of Procedure 5B.

The above considerations – a focus on ‘first flush’ runoff – lead to site ‘time of concentration’, t_C , as the storm duration of significance used in the design process. The peak flow entering the device before bypass operates, Q_{lim} is set equal to $Q_{0.25}$ based on storm duration t_C (see Section 3.7.1). The design of in-ground, “leaky” installations receiving cleansed storm runoff, becomes a simple application of Eqn. (5.5a or b), as applies : in this case,

$$\tau = 2 t_C \text{ (see Figure 3.4).}$$

Volume, V , is set equal to the runoff volume generated in the ARI, $Y = 0.25$ -years storm of duration t_C , only.

The design process involves two steps which are applicable to both Type 1 and Type 2 treatment/filter units or systems (see Section 7.1) followed by retention installations.

An alternative to Procedure 5B, as reviewed above, is to use the technology of infiltration or ‘dry’ ponds for pollution control/retention in small urban catchments (see Section 2.11 and Procedure 3, Section 5.1.4). The procedure is based on the ‘design storm’ approach with application opportunities more limited than for “soakaways” : infiltration ponds are recommended for sandy and sandy-clay sites, only, in order to satisfy appropriate emptying time criteria. Nevertheless, these devices should not be overlooked by designers who should include them among alternative options in the initial stages of planning. An illustrative example of design, Example 7.4, is included in Section 7.7.

7.4.2 PROCEDURE 5B : Strategy B design for sub-structure storages receiving cleansed runoff from paved and other ground-level filter-integrated surfaces

In this case, the choice of treatment unit (see Figure 3.5a) has been made and its design, including provision for **bypass**, completed before Procedure 5B is commenced. The filter/treatment unit should have capacity, Q_{lim} , matched to the contributing catchment (critical storm duration, t_c) with ARI, $Y = 0.25$ -years.

The outflow from the treatment device may be delivered as a concentrated entry (Type 1, see Section 7.1) or as a distributed entry from a porous/permeable paving surface (Type 2). In the latter case, attention must be given to the consequences for hydraulic conductivity, k_h , of sediment accumulation over the ‘lifespan’ of the paving (see Section 3.1.2 and Procedure 1A or 1B, Section 5.1.)

STEP 1 :

Compute runoff volume $\forall \text{ m}^3$ from the contributing catchment in the ARI, $Y = 0.25$ -years storm for duration $t = t_c$, catchment ‘time of concentration’.

Given the space available for the “leaky” device, identify **available** plan area, $A_{avail} \text{ m}^2$ set equal to area “a” in Eqn. (5.5b, below); also identify void space ratio, e_s , of the selected “soakaway” type, hydraulic conductivity of the site soil, k_h , and the appropriate value of Moderation Factor, U ($U = 0.5$ for sand; $U = 1.0$ for sandy clay; $U = 2.0$ for clay).

With $\forall$, $a (= A_{avail})$, e_s , k_h and U assigned, determine “soakaway” depth, H , from :

$$H = \frac{\forall}{a \cdot e_s} - \frac{60}{e_s} \cdot k_h \cdot \tau \cdot U \quad (5.5b)$$

where $\tau = 2t_c$.

The depth H given by Eqn. (5.5b) should fall, for practical reasons, within the range 0.30 m to 1.50 m (see Figure 7.1). If H is **smaller** than 0.30 m, then the area A_{avail} is “too large” and a smaller area may be used for A_{avail} and STEP 1 repeated.

If Eqn. (5.5b) gives depth H **greater** than 1.50 m, then the area A_{avail} is too small for a ‘simple’ solution to emerge from Procedure 5B, and recourse to Procedure 6B is inevitable. In this circumstance, the value of depth H (greater than 1.50 m) goes **directly** to Procedure 6B, STEP 4. **Every effort should be made to avoid this design direction where significant dissolved pollutants are considered likely in the catchment runoff (see Section 7.2.2 – Case 2).**

STEP 2 :

Test the result (for depth H) from STEP 1 for acceptable emptying time using :

$$T \approx \frac{2H \cdot e_s}{k_h}, s \quad (3.32)$$

If $T \leq 12$ hours, then **accept** result as final value for depth H ;

If $T > 12$ hours, then **reject** result for depth H and proceed directly to Procedure 6B, STEP 4.

STEP 3 :

Report the dimensions and details of the filter device or system adopted, including details of the limiting bypass flow, and of the retention device using values for A_{avail} and H associated with the successful outcome (STEP 2). Provision for overflow must be included with these details. Re-design of a rejected system (STEP 2) takes place under Procedure 6B, Section 7.5.2, following, commencing with STEP 4.

7.5 MORE COMPLEX STRATEGY B SYSTEMS : PROCEDURE 6B

7.5.1 Introduction

Design of systems involving aquifer access or slow-draining pipes and meeting **Strategy B** requirements (see Section 4.3.1), that is, control of ‘first flush’ runoff and retention of the associated cleansed runoff volume, only, follows the same broad principles set out for Strategy A – complex systems – compliance, above (Sections 7.3.1 and 7.3.2). The main difference lies in the manner in which the aquifer recharge or slow-drainage flows are determined. In the present case, the flows are calculated on the basis of devices emptying in an ‘acceptable time’, set at 12 hours.

As in the parallel Strategy A procedure, recognition of the presence of an aquifer beneath a site where recharge of suitably cleansed storm runoff is considered possible, should be followed by an environmental impact assessment (see Figure 1.4a and Section 3.8). If favourable, a review of available geological-geotechnical reports to ascertain its basic characteristics should be conducted – depth, standing water level, water quality, potentiometric gradient, local use (potable, irrigation, industry, etc.), likely yield per bore and, hence, potential rate of recharge, q_r . [Potential rate of recharge may be taken as 50% rate of yield (Pavelic, et al, 1992).]

In situations where the required accelerated emptying is to be achieved by slow-drainage pipeline, the main restriction on the quantity of flow which can be abstracted is likely to be head difference between the in-ground device and the pipe outlet. This option is therefore unavailable in terrain which is very flat; but the flows involved – Q_{hB} calculated using Eqn. (7.3) – are, typically, so small that this constraint is rarely preclusive.

However, the WARNING contained in Section 7.3.1 relating to the likely presence of significant dissolved pollution in effluent passing from the upstream filter applies here also. Procedure 6B should therefore be avoided in these circumstances and design confined to the scope offered by Procedure 5B only.

The design is executed according to the steps of Procedure 6B, Section 7.5.2, following, commencing with STEP 4. This is intended to convey the clear signal to the designer that problems of cleansed stormwater runoff should be solved, first, as ‘simple’ systems (by the three steps of Procedure 5B) if at all possible before the move to a more complex solution is made.

7.5.2 PROCEDURE 6B : Strategy B design for sub-structure storages with :

(a) aquifer access, or

(b) potential for ‘slow drainage’,

receiving cleansed runoff from paved surfaces or ground-level filter-integrated systems.

At this point in the design process, the treatment unit has been selected and dimensioning of the installation as a ‘simple’ system – including provision for bypass and device overflow – has been completed (Procedure 5B) before Procedure 6B is commenced. The outcome of the previous design segment is either depth, H , greater than the recommended maximum of 1.50 m (see Procedure 5B, STEP 1) or **failure** to meet the required emptying time criterion (STEP 2). The design process continues :

STEP 4 :

Revisit the design previously prepared under Procedure 5B **and which produced a depth, $H > 1.50$ m or which failed the emptying time test** and determine, for this design, an acceptable emptying flow rate, Q_{hB} :

$$Q_{hB} = (a.H. e_s)/(12 \times 3600) \text{ m}^3/\text{s} \quad (7.3)$$

where the parameters a , H and e_s are identical to the values used in or resulted from STEP 1 (Procedure 5B).

STEP 5a (aquifer access) :

Calculate number of bores, “n”, required to recharge a flow of *at least* Q_{hb} (Eqn. 7.3) to the aquifer, that is:

$$n \cdot q_r \geq Q_{hb} \quad (7.3a)$$

where q_r = (individual) bore recharge rate, m^3/s ; hence number of bores, “n”, an integer.

OR

STEP 5b (slow-drainage pipeline) :

Calculate slow-drainage pipe flow rate, Q_{hb} , by Eqn. (7.3).

STEP 6a :

Re-evaluate depth H by direct substitution into Eqn. (5.9b) with parameter Q_r **replaced** by a flow given by $n \times$ (permissible recharge rate per bore), that is $(n \cdot q_r)$; this flow should be $\geq Q_{hb}$, determined in STEP 4. All other parameters remain the same as those used in or resulting from Procedure 5B, STEP 1. This process leads to a slightly shallower installation with emptying time less than 12 hours.

OR

STEP 6b :

Re-evaluate depth H by direct substitution into Eqn. (5.9b) with Q_r **replaced** by Q_{hb} , obtained from Eqn. (7.3). All other parameters remain the same as those used in or resulting from Procedure 5B, STEP 1. This process leads to a slightly shallower installation with emptying time less than 12 hours.

[While a value of Q_{hb} greater than that determined in STEP 4 is permissible, the excess should be small to achieve the purpose of ‘slow drainage’ (see Special Note, Section 3.5.4)].

STEP 7 :

Prepare a system design using values for a ($= A_{avail}$), H, n and q_r **or** Q_{hb} associated with the successful outcome from STEP 6.

7.6 INFILTRATION SYSTEMS : FURTHER DESIGN CONSIDERATIONS**7.6.1 Introduction**

Design of pollution control systems (Strategy A and Strategy B, see Section 4.3.1) are treated in Chapter 7 to this point as, effectively, **isolated** systems, i.e. cases addressing the joint tasks of filtration and treatment to achieve water quality improvement objectives only. A major focus is therefore on the provision of storage in sub-structures, often beneath treatment surfaces such as sand/gravel filters, permeable paving layers, etc. An important element of such systems is provision for bypass in events producing runoff flows greater than Q_{lim} (Strategy A systems) or $Q_{0.25}$ (Strategy B systems).

In practice, such system isolation is unusual : it is far more common for a comprehensive interpretation of ‘source control’ to be sought involving containment of flow quantity (flood control systems) as well as pollution control. Interaction between the two systems raises important design considerations and consequences, explored in the following sections.

Another issue of importance for the designer is how to adjust the information contained in the Strategy A graphs derived from ‘continuous simulation’ modelling – the illustrative graph, Figure 7.1 (and Appendix C) – for retention sub-structures **other** than gravel-filled “soakaways”. This raises two differences of importance :

- design of “**soakaways**” constructed from pipe/gravel materials or from ‘milk crate’ and other types of units, and design of “dry” ponds;
- design of gravel-filled **trenches** or trenches constructed from pipe/gravel materials or from ‘milk crate’ etc. units.

A further matter of uncertainty which is high on the agenda of every practitioner in the field of WSUD, is that of ‘lifespan’ of retention installations receiving storm runoff from urban catchments.

Each of these issues is considered below.

7.6.2 Flood control, pollution control interaction

In relation to **Strategy A** :

- 1) Current Australian practice [‘design storm’ approach, IEAust., 1987)] requires that **flood control** systems for specific sites be designed using a ‘critical storm duration’ which is, normally, related to the (local) catchment longest travel time; in other cases, this duration is set equal to site ‘time of concentration’, t_c . **Pollution control** system design procedures (5A and 6A) have no such focus in terms of storm duration because of their basis in ‘continuous simulation’ modelling. A ‘surrogate’ design storm duration can be determined for these systems: it tends to be longer than the typical urban catchment critical storm duration; also,
- 2) **Flood control** systems are, typically, designed with ARI = 2 to 10 years (‘minor’ networks) or 50 to 100 years (‘major’ networks), while **pollution control** systems (Procedures 5A and 6A) have no clearly-identifiable ARI : a ‘surrogate’ ARI of, perhaps, 0.25- to 1-year probably applies.

Consideration (1), above, results in storages required for flood control installations which are **smaller** than those required for pollution control systems, *if the Y-years frequency were taken as the same in both instances*. But this is not the case, as indicated by consideration (2), above. The significantly greater storm runoff volumes, $\forall$, generated in design storms used in flood control scenarios ($Y > 2$ -years), typically outweigh the corresponding $\forall$ resulting from ARI, $Y = 0.25$ - to 1-year events associated with pollution control installations. This is common but not universal, so the designer is advised to check each case to determine the dominant condition where problems involve both flood control and pollution control, which frequently is the case. While a universal ‘rule’ cannot be formed on this matter, the greater sub-storage volume is more likely to arise from flood control considerations than those of pollution containment. Illustrative Examples 7.2 and 7.3 (Section 7.7) shed more light on this issue.

In relation to **Strategy B** :

The storage volumes required for Strategy B cases (Procedures 5B and 6B) in normal circumstances are significantly smaller than those which result from Procedures 5A and 6A. The reason for this is the combination of short (design) storm duration – t_c , ‘time of concentration’ only – and low ARI, $Y = 0.25$ -years used in the ‘first flush’ procedure. It may therefore be reasonably assumed that any flood control design, correctly executed with ARI, $Y = 2$ -years or greater, will *inter alia* subsume the corresponding pollution control cleansed stormwater requirements of Strategy B.

In relation to **bypass** :

It is mandatory for an on-site installation falling within the ambit of **Type 1** – filter device with concentrated exit flow (see Section 7.1) – to incorporate bypass of all flows greater than Q_{lim} . Devices falling within this classification are usually proprietary products matched, by design/selection, to field circumstances.

However, **Type 2** installations – typically, with grass or permeable paving treatment surfaces designed according to Procedures 1A or 1B or 5A/6A – are not, necessarily, so limited. Indeed, it would be most unusual to impose a Q_{lim} bypass limitation, for example, on a permeable paving car park, to achieve purely pollution control objectives. This recognition, taken with flood/pollution control interaction discussed above, puts the **design** of porous/permeable paving systems primarily in the domain of flood control with pollution containment achieved as a subsidiary benefit. Study of Example 7.2, Section 7.7, should provide some valuable insights on this issue.

It may be concluded from this brief excursion into the realm of stormwater quantity (flooding) and quality management interaction, that **systems designed with flood control as the primary objective, rather than pollution containment, tend to achieve Strategy A or Strategy B compliance as a beneficial by-product of competent design.**

This does not mean that the detailing of Procedures 5A, 6A, 5B and 6B is without value – far from it – but it does mean that their **exclusive** use to design installations is likely to be unusual and their vital role in reviewing compliance with Strategy A or Strategy B objectives, as in Examples 7.2, 7.3 and 7.4 (Section 7.7), far more common. ‘Filter strip’ swales, described in Section 7.8, are members of the “unusual” class referred to here.

7.6.3 Design of non-gravel retention devices

One of the most important elements in the ‘continuous simulation’ modelling which led to Figure 7.1 (and the graphs of Appendix C) is that of water level drop with time – the recession relationship – of an operating retention device.

The relationship is revealed in Eqn. (3.32), emptying time :

$$T \approx \frac{2 H \cdot e_s}{k_h}, s \quad (3.32)$$

It is clear from this formula that emptying time of a gravel-filled “soakaway” is related, directly, to the product of device depth, H , and void space ratio, e_s . But this product represents, simply, the **absolute** depth of water contained in the device, that is, depth h_a , *unhindered by the presence of the gravel*.

Thus, the time taken for the water depth in a “soakaway” of any type to fall from

depth, $(h_a)_1$ to depth, $(h_a)_2$,

is **the same**, regardless of space taken up by solid material, provided depth h_a is set equal to the product of actual depth, h , and e_s . This reasoning may be applied equally to infiltration or ‘dry’ ponds : Eqn. (3.32) can be employed in the design of ponds so long as e_s is set equal to 1.0.

It follows that design of a “soakaway” of **any** type and, also, ‘dry’ ponds, can be determined from the sets of graphs referred to above provided a simple conversion between void space ratio, e_s , of the proposed device and $e_s = 0.35$, representing the gravel-filled case, is made (Eqn. 7.4).

It is recommended in these circumstances, that design of a “soakaway” other than gravel-filled, or of a ‘dry’ pond, be carried out in two steps :

STEP 1 :

Design as for a gravel-filled “soakaway” with the data appropriate to the particular case – location, soil type, catchment EIA, available space, etc., using Procedures 5A and 6A, as required.

STEP 2 :

Convert the final (gravel-filled) “soakaway” depth, H_{gravel} (Procedure 5A, STEP 3; or Procedure 6A, STEP 6, as applies), into a final ‘new’ device depth through the formula :

$$H_{\text{new}} = H_{\text{gravel}} \times [0.35/(e_s)_{\text{new}}] \quad (7.4)$$

An illustration of this process is to be found in Example 7.3, Section 7.7.

All other aspects of the design remain the same, except for the following qualifications applicable to infiltration or ‘dry’ ponds.

It is recommended that the full range of soils which is suitable for “soakaways” is not appropriate for infiltration ponds : these should be limited in application to sand and sandy clay sites only (see Procedure 3, Section 5.1.4). Another difference relates to upstream treatment : storage and the permeable bed of a pond provide, together, an effective sedimentation/filtration system. Upstream treatment, in the case of infiltration ponds, can therefore be limited to collection/retention of gross pollutants such as leaf matter, urban debris, etc. A vegetated swale is ideal for this duty (see Chapter 10, ARQ document, Engineers Australia, 2006).

A further difference between “soakaways” and ‘dry’ ponds relates to the application of the Procedure 6A or 6B component of design. Directly applied, this would lead to injection of ponded water into aquifers or removal of (pond) water by slow-drainage pipeline. Because of inadequate pre-treatment provided to retained water, this mode of abstraction practice *directly* is **not** recommended (see Important Note, Section 5.1.5).

7.6.4 Design of Retention Trenches

A ‘correct’ procedure for the design of retention trenches of various types – gravel-filled, ‘milk crate’, etc. – using the same ‘continuous simulation’ modelling approach which produced Figure 7.1, etc. – could certainly be developed. However, it would result in a folio of graphs – equivalent to Appendix C – for *each* trench width of interest. This has not been attempted : in its place is a simplified design procedure which is conservative and based on the existing Procedures 5A and 6A. It involves three steps :

STEP 1 :

Design as for a gravel-filled “soakaway” with the data appropriate to the particular case – location, soil type, catchment EIA, available space – using Procedures 5A and 6A, as required.

STEP 2 :

Accept the final (gravel-filled) “soakaway” depth, H_{gravel} , and plan area, A_{avail} (Procedure 5A, STEP 3, or Procedure 6A, STEP 6, as applies) in cases where a **gravel-filled** trench is sought; convert the device depth into a final ‘new’ device depth through Eqn. (7.4) in **all other cases**.

STEP 3 :

Arrange area A_{avail} in any (trench) plan layout that is suited to the location, keeping trench depth the same as given by STEP 2, above. Trenches designed according to these guidelines will tend to empty more rapidly than the equivalent “soakaways” upon which they are based.

7.6.5 Strategy A systems : consequences for ‘lifespan’ and maintenance

The origins of sediment in the urban landscape and observed impacts of water-borne particulate matter on the performance and ‘lifespan’ (years of service before reconstruction or reinstatement is necessary) of components of retention systems, were reviewed briefly in Section 1.3.3. The main lesson which emerges from these considerations is the need for great caution to be exercised by designers/practitioners in dealing with sediment mobilised in urban catchments following rainfall, particularly during the construction phase of any development.

Filter/treatment systems designed to achieve the Strategy A level of pollution control for any nominated retention efficiency such as $R = 90\%$ recommended in this Handbook, may be grouped into Type 1 or Type 2 applications (see above). The **former** includes the array of proprietary and public domain installations which collect/retain water-borne sediment and, therefore, require an ongoing commitment to cleaning, maintenance and eventual replacement. Some advice on the consequences of these aspects of GPTs is included in Chapter 2 and in Section 7.1.

Type 2 installations, on the other hand, are “lost sediment” systems by design, meaning that they collect/retain sediment within their structures for the duration of their effective ‘lifespans’ and are, therefore, maintenance-free as regards sediment removal. Of course, regular inspection and corrective maintenance such as mowing (grassed surfaces) or sweeping/vacuuming [block paving, see Dierkes et al

(2002)], to remove *surface*-retained sediment is strongly recommended, but the task of removing *internally*-retained particulate matter is not a required activity. This aspect makes them popular with both government and municipal agencies. The issue of their widespread use hinges, therefore, on the ‘lifespan’ which can be expected for such installations.[It is important to understand that the sediment accumulation reviewed here (below) is **not** that discussed in Section 3.1.3 for which jet-vacuuming is an effective antidote.]

This issue is explored in Section 3.1.2 where the findings of Rommel et al (2001) are reported. Rommel et al’s results can be re-interpreted to provide another insight to the problem at hand. Assuming that (see Urbanas and Stahre, 1993) :

- 1) 80% of the total pollution load is contained in the first 20% of runoff (the ‘first flush’); and,
- 2) the 200 mg/L used in the tests represents a typical ‘first flush’ (average) concentration for established residential neighbourhoods; and,
- 3) the design standard average recurrence interval likely to be used in design is ARI, Y = 2-years *at least*, is adopted,

then it can be shown that a ‘lifespan’ of at least 25 years without (**internal**) filter system replacement is attainable. This outcome, taken with the observation made in the previous section that “... systems designed with *flood control* as the primary objective, rather than pollution containment, tend to achieve Strategy A or Strategy B compliance as a beneficial by-product of competent design” offers the designer hope that such systems, using ARI values of 2-years and above, carry with them the assurance of (relatively) maintenance-free filter performance for periods of up to 25 years.

There is an important qualification incorporated into this line of reasoning which needs to be clearly understood, namely, that the suggested performance applies to “...established residential neighbourhoods...”. As pointed out by Argue (2000), poor on-site management of construction-phase sediment can produce complete blockage of a Type 2 filter system before it has even begun its intended ‘lifespan’ in an established urban setting.

7.7 EXAMPLES : PROCEDURES 5A and 6A

7.7.1 Introduction

It was pointed out in Section 7.6.1 that cases of isolated design, for example flood control, only, without stormwater treatment, or pollution control without associated flood control measures, are rare in practice. The examples which follow include a mix of illustrations, some involving dual objectives, others addressed to single goals. The aim of this section is not to present complete design detailing of case study installations but, rather, to provide practitioners with sufficient information to enable them to undertake such (complete) tasks drawing together aspects and components of design appropriate to the demands of particular situations.

Four illustrative examples are included :

- Example of the use of the Figure 7.1 graph (or Appendix C) in pollution control design or checking;
- Treatment surface designed, also, as a stormwater quantity management facility;
- “Soakaway” designed as a flood control facility and checked for Strategy A pollution control compliance;
- Residential sub-division with infiltration pond designed for treatment of ‘first flush’ pollution load.

7.7.2 Example 7.1a : Use of Figure 7.1 graph (also Appendix C)**Procedure 5A (Strategy A)**

Location : Adelaide, Southern Australia;
Soil : medium clay, $k_h = 3 \times 10^{-6}$ m/s; Moderation factor, $U = 2.0$;
Catchment : paved area, $A_{EIA} = 2,500 \text{ m}^2$;
Space available : $A_{avail} = 500 \text{ m}^2$; hence, $A_R = 0.20$;
Retention device : gravel-filled “soakaway”, $e_s = 0.35$.
Retention efficiency, R = 90%,

STEP 1 :

Moderated hydraulic conductivity :

$$k_h = (3 \times 10^{-6}) \times U = 3 \times 10^{-6} \times 2.0 = 6 \times 10^{-6} \text{ m/s};$$

Plan space ratio, $A_R = 0.20$;

Locate in Figure 7.1; see point “1” on graph.

This corresponds to a “soakaway depth significantly **less** than H_{min} . Therefore adjust “ A_{avail} ” to value corresponding to depth, $H = H_{min} = 0.30 \text{ m}$; (see ‘shift’ in Figure 7.1 to $A_R = 0.08$);

Hence, (new) $A_{avail} = 0.08 \times 2,500 \text{ m}^2 = 200 \text{ m}^2$.

STEP 2 : Case 1

“Soakaway” depth, $H = 0.30 \text{ m}$, from STEP 1.

STEP 3 :

Report conclusions, R = 90% :

$q_{lim} = 0.005 \text{ L/s per m}^2$ of A_{EIA} , hence, treatment system capacity flow :

$$\begin{aligned} Q_{lim} &= 0.9 \times A_{EIA} \times q_{lim} \text{ L/s} \\ &= 0.9 \times 2,500 \times 0.005 = \mathbf{11.25 \text{ L/s}}; \end{aligned} \tag{7.1}$$

Plan area of “soakaway” = 200 m^2 ; “soakaway” depth, $H = 0.30 \text{ m}$.

7.7.3 Example 7.1b : Procedure 5A (Strategy A)

Location : Adelaide, Southern Australia;
Soil : medium clay, $k_h = 3 \times 10^{-6}$ m/s; Moderation factor, $U = 2.0$;
Catchment : paved area, $A_{EIA} = 2,500 \text{ m}^2$;
Space available : $A_{avail} = 100 \text{ m}^2$; hence, $A_R = 0.04$;
Retention device : gravel-filled “soakaway”, $e_s = 0.35$.
Retention efficiency, R = 90%.

STEP 1 :

Moderated $k_h = 6 \times 10^{-6}$ m/s, as above;

Plan space ratio, $A_R = 0.04$;

Locate in Figure 7.1; see point “2” on graph.

This corresponds to a “soakaway depth of $H = 0.9$ m

STEP 2 : Case 2

“Soakaway” depth, $H = 0.90$ m.

STEP 3 :

Report conclusions :

$$q_{\text{lim}} = 0.005 \text{ L/s per m}^2 \text{ of } A_{\text{EIA}},$$

hence, treatment system capacity flow :

$$\begin{aligned} Q_{\text{lim}} &= 0.9 \times A_{\text{EIA}} \times q_{\text{lim}} \text{ L/s} \\ &= 0.9 \times 2,500 \times 0.005 = \mathbf{11.3 \text{ L/s}}; \end{aligned} \quad (7.1)$$

Plan area of “soakaway” = $\mathbf{100 \text{ m}^2}$; “soakaway” depth, $H = \mathbf{0.90 \text{ m}}$.

7.7.4 Example 7.1c : Procedures 5A and 6A

Location, soil, catchment, “soakaway” and retention efficiency and q_{lim} as for Example 7.1b, above.

Space available : $A_{\text{avail}} = 37 \text{ m}^2$; hence, $A_R = 0.015$;

STEP 1 :

Moderated $k_h = 6 \times 10^{-6}$ m/s, as above;

Plan space ratio, $A_R = 0.015$;

Locate in Figure 7.1; see point “3” on graph.

STEP 2 : Case 3

“Soakaway” depth, $H > H_{\text{max}}$ on $R = 90\%$ graph

$$q_{\text{lim}} = 0.005 \text{ L/s per m}^2,$$

hence,

$$\begin{aligned} Q_{\text{lim}} &= 0.9 \times A_{\text{EIA}} \times q_{\text{lim}} \\ &= 0.9 \times 2,500 \times 0.005 \text{ L/s} = \mathbf{11.3 \text{ L/s}}. \end{aligned} \quad (7.1)$$

Hence nominate “soakaway” depth, $H = \mathbf{H_{\text{max}} = 1.50 \text{ m}}$.

STEP 3 :

Report part-conclusions :

$$\begin{aligned} Q_{\text{lim}} &= 0.9 \times A_{\text{EIA}} \times q_{\text{lim}} \text{ L/s} \\ &= 0.9 \times 2,500 \times 0.005 = \mathbf{11.3 \text{ L/s}}; \end{aligned} \quad (7.1)$$

Plan area of “soakaway” = $\mathbf{37 \text{ m}^2}$;

“soakaway” depth, $H = \mathbf{1.50 \text{ m}}$.

Procedure 6A – STEP 4 (slow-drainage) :

Moderated $k_h = 6 \times 10^{-6}$ m/s, as previously;

Plan space ratio, $A_R = 0.015$, as in STEP 1; see point “3” on graph, Figure 7.1.

Locate ‘point’ (horizontal shift from “3”) in Figure 7.1, where $A_R = 0.015$ coincides with “soakaway” maximum recommended depth, $H_{\max} = 1.5$ m. This ‘point’ corresponds to :

$k_h = 3.0 \times 10^{-5}$ m/s, now called **effective percolation rate, k_h^*** .

Locate point “3” in Figure 7.2 corresponding to :

the **site** (moderated) hydraulic conductivity, $k_h = 6 \times 10^{-6}$ m/s (horizontal axis); and,

effective percolation rate, $k_h^* = 3.0 \times 10^{-5}$ m/s (vertical axis).

The value given by the family of curves is, say

$$q_h = 0.055 \text{ L/s per m}^2 \text{ of } A_{\text{avail}}$$

STEP 5 :

Calculate slow-drainage flow rate :

$$\begin{aligned} Q_{hA} &= [(A_{\text{avail}}) \times (q_h)] \text{ m}^3/\text{s} \\ &= [(37) \times (0.055)] \text{ L/s} = \mathbf{2.04 \text{ L/s}}. \end{aligned} \quad (7.2)$$

STEP 6 :**Report conclusions :**

$Q_R = 90\%$, as above, $= \mathbf{11.3 \text{ L/s}}$;

Plan area of “soakaway” $= \mathbf{37 \text{ m}^2}$;

“soakaway” depth, $H = \mathbf{1.50 \text{ m}}$;

Slow-drainage flow, $Q_{hA} = \mathbf{2.04 \text{ L/s}}$.

7.7.5 Example 7.2 : Infiltration surface without ponding

The case study used to illustrate this aspect of ‘source control’ of stormwater is based on the car park at St. Elizabeth Church, Warradale, SA, illustrated in Figure 7.3 (previously Figure 5.1), below.

Details of the catchment draining to the grassed (“Grasspave”) hard-standing area located **within** the total car park area, are as follows (see Example 5.1) :

Total area of car park (40 spaces + carriageway) $EIA = 1,220 \text{ m}^2$

site time of concentration, $t_c = 10$ minutes;

design storm intensity (ARI = 3-years, $F_y = 0.9$), $i = 50 \text{ mm/h}$;

hence, design $Q_{\text{peak}} = 0.014 \text{ m}^3/\text{s}$.

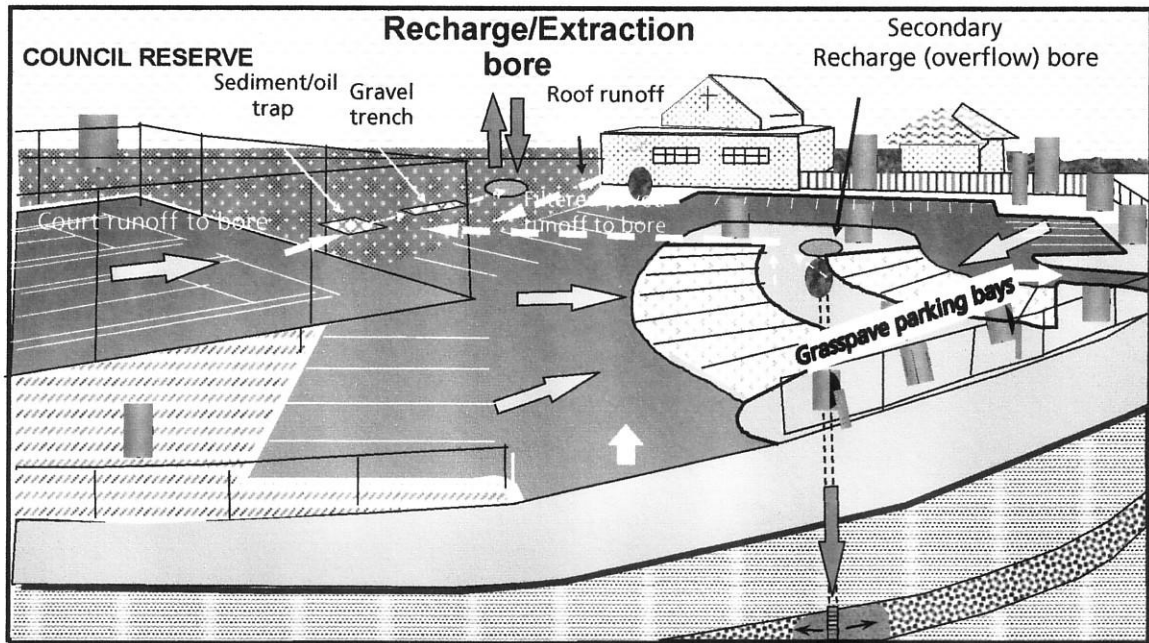


FIGURE 7.3 : General layout of car park at St. Elizabeth Church, Warradale, SA

Procedure 1B (Section 5.1.2) : infiltration surface (without ponding) accepting runoff from total area, A m^2 ; the infiltration surface is *internal*, that is located *within* the area set aside for the car park . Infiltration surface area $A_S = 407 \text{ m}^2$ (see Example 5.1, Section 5.2).

With plan area (equivalent to A_{avail} in the context of Procedures 5A and 6A), determined, the next step is to find the depth, H , of “soakaway” required **beneath** the treatment surface to accommodate cleansed storm runoff. This involves Procedure 5A :

- Location :** Warradale, Southern Australia;
- Soil :** medium clay, $k_h = 3 \times 10^{-6} \text{ m/s}$; Moderation factor, $U = 2.0$;
- Catchment :** paved area (total), $A_{\text{EIA}} = 1,220 \text{ m}^2$; it should be noted that although one-third of the total car park area is grassed, storm ‘runoff’ into the gravel-filled sub-structure beneath this component (by infiltration) will be close to 100%. It therefore behaves, effectively, as an impervious surface.
- Space available :** $A_{\text{avail}} = 407 \text{ m}^2$; hence, $A_R = 0.33$;
- Retention device :** gravel-filled “soakaway”, $e_S = 0.35$.
- Retention efficiency, $R = 90\%$.**

STEP 1 (Procedure 5A) :

Moderated hydraulic conductivity :

$$k_h = (3 \times 10^{-6}) \times U = 3 \times 10^{-6} \times 2.0 = 6 \times 10^{-6} \text{ m/s};$$

Plan space ratio, $A_R = 0.33$;

Locate in Figure 7.1 (**above** point “1” on graph used in Example 1a).

This corresponds to a “soakaway” depth significantly **less** than H_{min} . Adjust “ A_{avail} ” to value corresponding to depth, $H = H_{\text{min}} = 0.30 \text{ m}$; (see ‘shift’ in Figure 7.1 to $A_R = 0.08$);

$$\text{hence, (new) } A_{\text{avail}} = 0.08 \times 1,220 \text{ m}^2 = 98 \text{ m}^2.$$

STEP 2 : Case 1

“Soakaway” depth, $H = 0.30$ m, from STEP 1.

STEP 3 :**Report conclusions :**

$$q_{\text{lim}} = 0.005 \text{ L/s per m}^2 \text{ of } A_{\text{EIA}},$$

hence, treatment system capacity flow :

$$\begin{aligned} Q_{\text{lim}} &= 0.9 \times A_{\text{EIA}} \times q_{\text{lim}} \text{ L/s} \\ &= 0.9 \times 1,220 \times 0.005 = \mathbf{6.1 \text{ L/s}}; \end{aligned} \quad (7.1)$$

$$\text{Plan area of “soakaway”} = \mathbf{98 \text{ m}^2};$$

$$\text{“soakaway” depth, } H = \mathbf{0.30 \text{ m}}.$$

The practical implications of this outcome are :

- 407 m² of “Grasspave” surface must be provided to meet the storm runoff (quantity) control requirements, and to ensure the longevity of the pervious surface in the face of sediment accumulation;
- the volume required for storage of cleansed runoff (Strategy A requirement) delivered by the treatment surface to the in-ground “soakaway” (0.30 m deep), can be readily accommodated, occupying only **one-quarter** of the area of the treatment surface;
- the Q_{lim} capacity flow for Strategy A compliance – 6.1 L/s – is easily accommodated by the “Grasspave” treatment surface which has a capacity of more than 18 L/s;
- it is interesting to note that stormwater (quantity) control at the car park site (design computations are presented in Example 5.5, Section 5.2) requires a gravel-filled “soakaway”, 0.50 m deep, with bore, under one-third of the “Grasspave” hard standing area, but attention should be given to “Important additional note”, Example 5.5.

7.7.6 Example 7.3 : “Soakaway” at Plympton Anglican Church

Stormwater runoff from a large paved area and from six buildings at the Plympton Anglican Church in Adelaide, South Australia, passes, after cleansing, into a “soakaway” constructed from a combination of gravel and perforated pipes illustrated in Figure 7.4. Details of the catchment draining to the retention installation are as follows (see Example 5.3, Section 5.2) :

site area (total).....	2,640 m ²
total connected roof areas.....	1,600 m ²
total connected paved area	360 m ²
total connected pervious areas ($C_{10} = 0.10$)	680 m ²
site time of concentration, t_c	10 minutes
critical storm duration (Council specification)	30 minutes
design storm intensity [ARI = 100-years] (Council specification).....	73 mm/h

Procedure 2C (see Section 5.1.3) : “soakaway” part-occupied with impervious material, pipes, etc., with cleansed water inflow : required area, $a = 406 \text{ m}^2$ (see Example 5.3, Section 5.2).

[This example is parallel to Example 5.3, which is focussed on flood control design under the regime-in-balance strategy (Section 4.2.1). The present design review for pollution control, employs the yield-minimum strategy, even though regime-in-balance is specified for stormwater quantity management on the site. This apparent contradiction is explained in Section 7.1.]

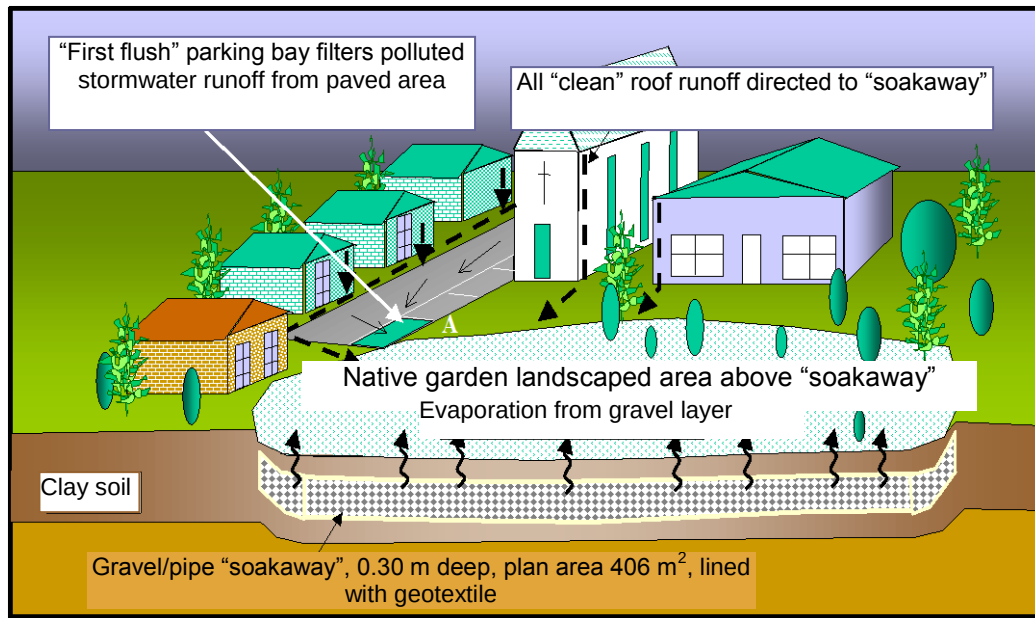


FIGURE 7.4 : General layout of site and “soakaway” at Plympton Anglican Church.

With plan area $a = 406 \text{ m}^2$ (equivalent to A_{avail} in the context of Procedures 5A and 6A), determined, the next step is to check the design to ensure that it complies with Strategy A pollution control/retention requirements under Procedure 5A :

Location : Plympton, Southern Australia;

Soil : medium clay, $k_h = 1 \times 10^{-6} \text{ m/s}$; Moderation factor, $U = 2.0$;

Catchment : paved area, $A_{\text{EIA}} = 2,028 \text{ m}^2$;

Space available : $A_{\text{avail}} = 406 \text{ m}^2$; hence, $A_R = 0.20$;

Retention device : gravel/pipe “soakaway”, $e_s = 0.50$.

Note : Procedure 5A is followed using information from Figure 7.1, arriving at a solution for the gravel-filled “soakaway” case. The conversion described in Section 7.6.3 is then carried out enabling the final check/comparison to be made for the pipe/gravel installation.

STEP 1 (Procedure 5A) :

Moderated hydraulic conductivity :

$$k_h = (1 \times 10^{-6}) \times U = 1 \times 10^{-6} \times 2.0 = 2 \times 10^{-6} \text{ m/s};$$

Plan space ratio, $A_R = 0.20$;

Locate in Figure 7.1 (**above** the curve for H_{min}).

This corresponds to a “soakaway” depth significantly **less** than H_{min} . Therefore adjust “ A_{avail} ” to value corresponding to depth, $H = H_{\text{min}} = 0.30 \text{ m}$;

$$\text{hence, (new) } A_{\text{avail}} = 0.12 \times 2,028 \text{ m}^2 = 243 \text{ m}^2.$$

STEP 2 : Case 1

“Soakaway” depth, $H = 0.30 \text{ m}$, from STEP 1.

STEP 3 :**Report conclusions :**

$$q_{\text{lim}} = 0.005 \text{ L/s per m}^2 \text{ of } A_{\text{EIA}},$$

$$\text{hence, treatment system capacity flow : } Q_{\text{lim}} = 0.9 \times A_{\text{EIA}} \times q_{\text{lim}} \text{ L/s} \quad (7.1)$$

$$= 0.9 \times 2,028 \times 0.005 = \mathbf{9.1 \text{ L/s;}}$$

$$\text{Plan area of "soakaway"} = \mathbf{243 \text{ m}^2};$$

$$\text{"soakaway" depth, H} = \mathbf{0.30 \text{ m.}}$$

Conversion step (see Section 7.6.3) :

The dimensions reported in STEP 3, above, apply to a Strategy A gravel-filled installation requiring inlet treatment flow capacity, $Q_{\text{lim}} = 9.1 \text{ L/s}$, plan area equal to 243 m^2 (or greater) and depth, $H = 0.30 \text{ m}$. The conversion, Eqn. (7.4), enables the gravel-filled depth to be converted to depth, H , for a pipe/gravel "soakaway" equal to :

$$H_{\text{new}} = 0.3 \times [0.35/0.50] = 0.21 \text{ m} \quad (7.4)$$

The practical implications of this design are :

- 406 m^2 of pipe/gravel "soakaway" of depth 0.30 m required for flood control (Example 5.3, Section 5.2) easily embraces the requirements for Strategy A, pollution control compliance, exceeding those requirements in terms of both plan area and device depth;
- the Q_{lim} capacity flow required for Strategy A compliance – 9.1 L/s – provides the basis for selecting/designing a treatment/filter unit or system to be located upstream of the in-ground device. In the case of the installation at Plympton, this takes the form of a "Grasspave" treatment area located at the end of the driveway; it can easily accommodate a flow of 9.1 L/s .

7.7.7 Example 7.4 : Design of Strategy B System

Stormwater runoff from a residential sub-division, plan area 3.0 ha , for which equivalent impervious area, $A_{\text{EIA}} = 1.50 \text{ ha}$, is required to be treated to Strategy B standard, that is '**first flush**' treatment, only (Council specification). Local soil is sandy-clay, so a "dry" pond appropriate to the pollution control/retention requirements of Council is to be designed.

Location :	Newcastle, NSW;	
Soil :	sandy-clay, $k_h = 3 \times 10^{-5} \text{ m/s}$; Moderation factor, $U = 1.0$;	
Catchment :	paved area, $A_{\text{EIA}} = 15,000 \text{ m}^2$;	
Time of concentration :	$t_c = 20 \text{ minutes}$;	
Rainfall intensity :	$i_{0.25} = 0.5 \times i_1$	(3.38)
	$= 0.5 \times 47.7 \text{ mm/h} = 24 \text{ mm/h}$	
Space available :	$A_{\text{avail}} = 250 \text{ m}^2$;	
Retention device :	infiltration or 'dry' pond.	

The design procedure is similar to Procedure 5B :

STEP 1 :

Determine runoff **volume** from catchment in the ARI = 0.25-year storm, duration, $t = 20$ minutes;

$$\begin{aligned}\forall &= C \times A_{EIA} \times 0.50i_1 \times t \\ &= 0.9 \times 15,000 \times (24/1,000) \times (20/60) = 108 \text{ m}^3.\end{aligned}$$

STEP 2 :

Determine **depth** of infiltration pond, d , from :

$$d = \frac{\forall}{A_p} - 60k_h \cdot \tau U + \frac{i(\tau - t_c)}{6 \times 10^4} \quad (5.6b)$$

$$\text{where } \forall = 108 \text{ m}^3;$$

$$A_p = 250 \text{ m}^2;$$

$$k_h = (3 \times 10^{-5} \text{ m/s}); U = 1.0;$$

$$\tau = 2t_c = 40 \text{ minutes, in this case;}$$

$$i_{0.25} = 24 \text{ mm/h}$$

$$\text{hence, } d = 0.37 \text{ m}$$

Check emptying time :

$$T \approx \frac{2H \cdot e_s}{k_h}, s \quad (3.32)$$

$$\text{where } H = d = 0.37 \text{ m}; e_s = 1.0;$$

$$\text{hence, } T = 6.9 \text{ hours – satisfactory for ARI, } Y = 0.25 \text{ years (see Table 3.3).}$$

Report :

‘Dry’ pond treating storm runoff from residential sub-division, area 3.0 ha, to Strategy B standard :

$$\text{pond area, } A_p = 250 \text{ m}^2;$$

$$\text{pond depth, } d = 0.37 \text{ m};$$

emptying time in ARI, $Y = 0.25$ years event :

$$T = 6.9 \text{ hours.}$$

Note: This design approach might be considered – in appropriate (soil) circumstances - as a ‘lesser’ alternative to the Filter Strip Swale option reviewed in the remaining sections of this chapter.

7.8 ‘FILTER-STRIP’ SWALES : GENERAL BACKGROUND

7.8.1 Introduction

Swales are shallow grassed channels – typically 0.30 to 0.50 m (maximum) deep, 5 to 6 m wide in residential streets – with longitudinal slopes less than 3% (see Chapter 2). They have wide application in water-sensitive urban design for three main reasons :

1. they can be instrumental in retaining runoff through bed infiltration;
2. they can be effective in retaining pollutants conveyed in stormwater; (Breen et al, 1997; Li et al, 2008) and,
3. they can fulfil a role in stormwater harvesting through soil moisture enhancement and, possibly, aquifer recharge and recovery (see Figure 2.4).

The full potential of swales therefore includes each of the primary goals of stormwater ‘source control’ as defined in Section 1.3. Clearly, this scope is very great and, in a handbook such as this, warrants an entire chapter : the following treatment is confined to ‘filter strip’ swales which extend Procedures 5 and 6 into a realm of stormwater management that presents WSUD with one of its greatest challenges – that of controlling pollution in residential streets (see Section 1.4.3). In terms of the four systems defined in Section 3.4.2 to which ‘continuous simulation’ modelling is applicable, ‘filter strip’ swales are closest to Number 2 – “formal inlet control (by design); storage, including on-site disposal; and overflow”. Because of their primary role – indeed their **only** ‘source control’ role – as pollution reservoirs, ‘filter strip’ swales earn a place among Category 2 systems described in Section 4.1 (see Section 7.6.2, above).

Like the bulk of catchment components embraced by Procedures 5 and 6, ‘filter strip’ swales handle flows exceeding Q_{lim} essentially as bypass. ‘Filter strip’ swales can therefore be expected to provide treatment for approximately 95% of average annual runoff (see Section 4.3.1). The configuration of a ‘filter strip’ swale in relation to a residential street carriageway is shown in Figure 7.5.

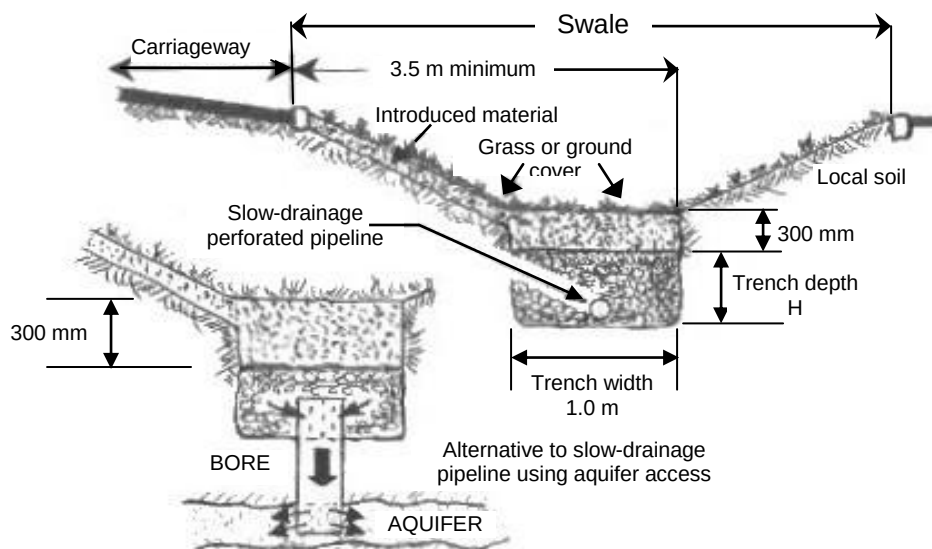


FIGURE 7.5 : Main components of a ‘filter strip’ swale (with sub-structure)

Swale systems of the type examined in this section abstract all flows up to the Q_{lim} limit – by design – and bypass all exceedances as open channel flow conveyed downstream within the boundaries of the swale, or perhaps channelled to the site of a ‘dry’ pond located within the streetscape (see final Note, Example 7.4). The process of abstraction is achieved through infiltration alone or by infiltration combined with sub-structure retention (gravel-filled trench or similar), and ‘hydraulic’ disposal to aquifers or local waterways (slow-drainage) if necessary. Application of the design procedures described earlier in this chapter to the particular case of ‘filter strip’ swales warrants the creation of two new design methods – Procedures 7 and 8 – parallel to Procedures 5 and 6 respectively. Before presenting details of the new procedures, however, it is appropriate to review the range of installations that result from their application.

7.8.2 ‘Filter-strip’ swales : general description

There are three broad types of ‘filter strip’ swales :

- (i) Swales whose cross-section includes a surface layer of sand and gravel mixture (100 mm thick) laid from the contributing carriageway edge to and across the bed of the excavated channel;
- (ii) Swales whose cross-section includes the layer of introduced material described in (i), above, plus a gravel-filled trench located beneath the swale invert;
- (iii) Swales whose cross-section includes the layer of introduced material and gravel-filled sub-structure, described in (i) and (ii), above, plus ‘hydraulic’ means of removing/conveying accumulated, cleansed stormwater either :
 - a) by direct disposal/recharge to a conveniently located aquifer; or,
 - b) by access to a ‘slow drainage’ perforated pipeline located in the base of the gravel-filled trench.

In this case the WARNING contained in Section 7.3.1 relating to the likely presence of significant dissolved pollution should be heeded and design limited to the scope provided by types (i) and (ii), above, only.

These components are illustrated in the compound ‘filter strip’ installation shown in Figure 7.5.

The use of these components as (i) only, or (i) and (ii) only, or all three, depends on the following factors :

- climate zone in which the swale is located;
- soil properties of the site;
- streetscape catchment area draining to the swale;
- availability of suitable aquifer (depth, permeability etc.) to receive recharge;
- availability of suitable receptor for cleansed stormwater.

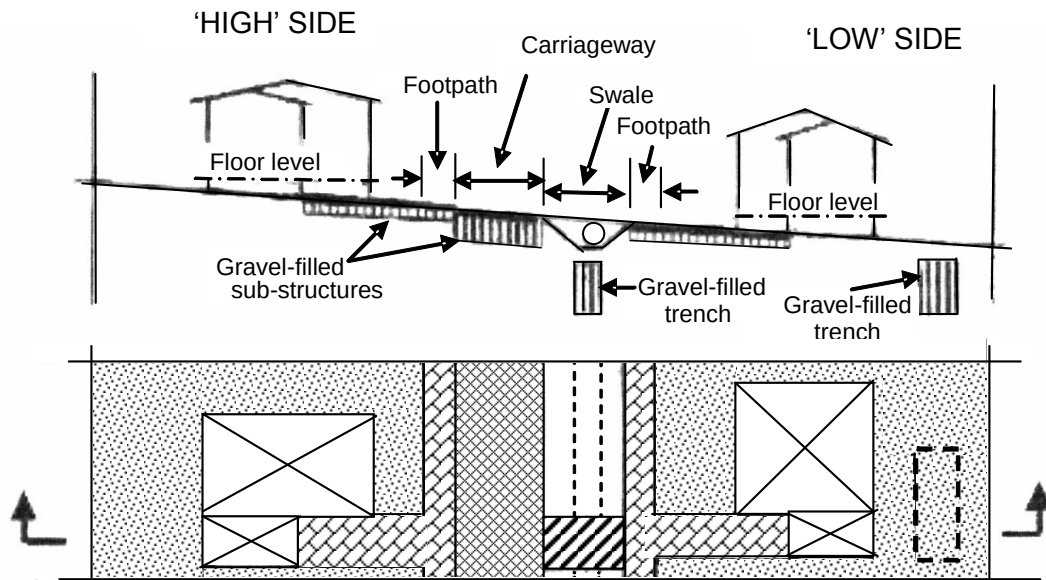
It is possible to prepare a comprehensive step-by-step procedure similar to those given for Procedures 5 and 6, but adapted for application to the particular case of ‘filter strip’ swales. However, this approach has been rejected in favour of information derived from ‘continuous simulation’ modelling, enabling ‘filter strip’ swales to be designed easily for the great bulk of streetscape scenarios (NB Section 7.10). This information is presented in a set of graphs appropriate to the five climate zones identified in Figure 3.8 and covering the range of native soils found in these zones with hydraulic conductivities from $k_h = 1 \times 10^{-7}$ to $k_h = 1 \times 10^{-3}$ m/s, inclusive.

7.8.3 Catchment area

The basic information (apart from climate zone and soil permeability) that the designer must bring to the task of planning a ‘filter strip’ swale using the graphs is : **catchment area (expressed as equivalent impervious area) draining to a typical 15 m segment of the proposed swale.** This raises some important issues about the management of storm runoff in the residential streetscape for which the design is being prepared. Figure 7.6 illustrates a strip of residential neighbourhood in which allotments are **15 m wide**. [In cases where average frontages are other than 15 m wide, a factor must be applied to reduce them to typical or ‘standard’ 15 m swale segments in order to use the graphs directly.]

Every effort should be made to minimise the area of paving directly draining to the swale and, hence, the quantity of runoff generated in the typical 15 m segment. Measures to achieve this include (see Figure 7.6) :

- backyard ‘soakaways’ in allotments remote from the swale streetscape but *up-slope* from it;
- permeable paving for allotment driveways including crossings;
- permeable paving for footpaths;
- permeable paving for the carriageway or (carriageway) edge strips.



15 m wide segment of residential street with swale

Within the segment there are six different surface covers or land uses :

- roof areas (house, garage) shown 
- allotment driveways and other outdoor 'hard' surfaces (possible porous/permeable paving) shown 
- pervious outdoor areas shown 
- footpath (possible permeable paving) shown 
- carriageway (possible permeable paving) shown 
- paved 'crossing' within swale (possible permeable paving) shown 

The remaining area within the segment is taken up by the swale.

In addition to these surface covers or land uses are sub-structures as follows :

- below allotment driveways and other 'hard' surfaces :
shallow, gravel-filled trench encased in geotextile shown 
- below footpath contiguous with driveway & carriageway on 'high' side; and below footpath (isolated) on 'low' side :
shallow, gravel-filled trench encased in geotextile shown 
- below carriageway on 'high' side :
deep, gravel-filled trench encased in geotextile shown 
- below invert of swale :
deep, gravel-filled trench encased in geotextile shown 
- 'Low' side roof runoff directed to deep, gravel-filled trench encased in geotextile shown 

FIGURE 7.6 : Swale streetscape segment with full stormwater retention options excluding rainwater tanks (see alternative layout in Figure 2.4)

The presence of rainwater tanks in such developments further assists the process of reducing the overall equivalent impervious area. It is difficult to satisfactorily quantify this in a given situation : it is therefore acceptable to consider this element as a ‘margin of safety’. By following this approach it is found that typical, medium-density residential streetscapes, yield **equivalent impervious areas between 300 m² and 500 m² per 15 m segment**. [This includes the *entire* filter strip area itself and the filter strip portion of the crossing (see Figure 7.5), which although partly pervious and partly paved, absorbs the full rainfall input and, therefore, acts as though it were fully paved.] The swale layouts which can be produced from the design graphs cover this range.

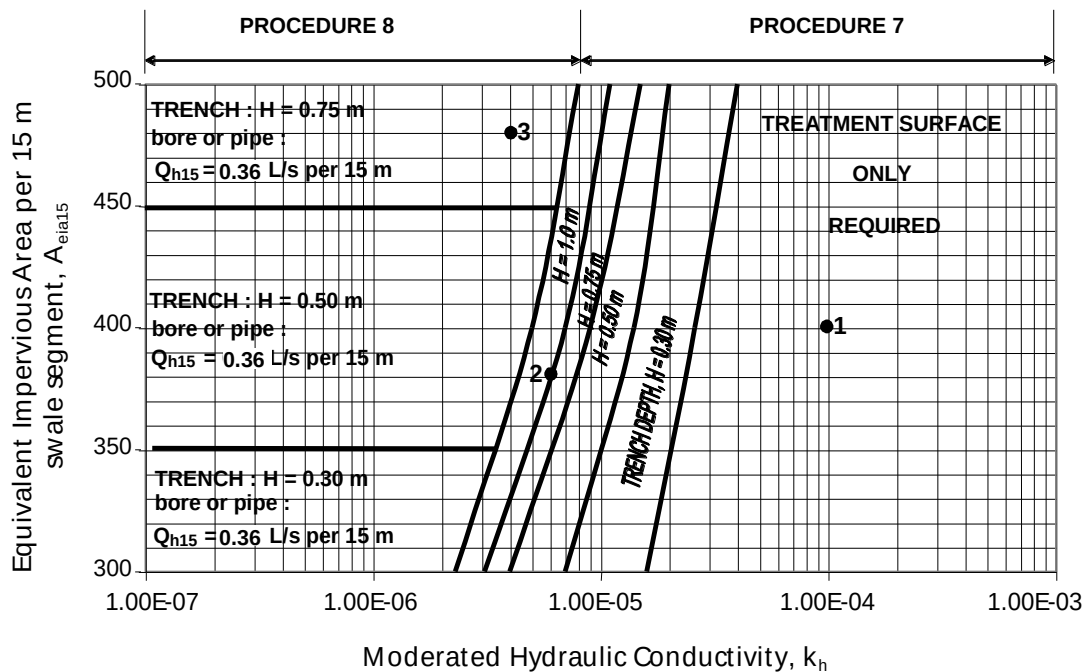


FIGURE 7.7 : Graph for design of ‘filter strip’ swales – Southern Australia ($i_{10,1}$ rainfall intensity range: up to and incl 25 mm/h). Soils with moderated $k_h > 1.0 \times 10^{-4}$ m/s are, typically, unsuitable for treating dissolved pollutants in catchment runoff (see Mikkelsen et al, 1997; Fischer et al, 2003)

7.8.4 Procedures 7 and 8 : introduction

The focus of Procedures 7 and 8 in any design task is the graph which has been prepared for the location’s climate zone. This is selected, as in Procedure 5A, on the basis of the $i_{10,1}$ value for the site, and its identification with one of the five climate zones illustrated in Figure 3.8.

The graph for the Southern Australia zone is Figure 7.7; graphs for the other zones are presented in Appendix D. Each graph (Catchment EIA vs soil moderated hydraulic conductivity) comprises three distinct areas (see Figure 7.7) :

- Swales which require a 100 mm layer of introduced sand/gravel, indicated as **treatment surface only** in each graph;
- Swales which require surface treatment and a gravel-filled (trench) sub-structure. The **width** of trench is 1.0 m in every case; the **depth** is read from the family of curves covering the range 0.3 m (minimum) to 1.0 m (maximum).

The information presented in these two areas of each graph is pertinent to the application of Procedure 7. The information derived from application of Procedure 8 is also shown in the graph and is necessary for :

- Swales which require surface treatment, and gravel-filled trench, as well as provision for disposal of cleansed stormwater into an aquifer, if available, or to a receiving drainage channel or waterway. The quantity of flow (L/s) per 15 m segment of swale (to be conveyed) can be read from the ‘bands’ set out under “Procedure 8” in each graph. **Use of Procedure 8 requires that significant dissolved pollution is absent (see Section 7.8.2).**

7.8.5 Specification

The **carriageway** delivering flow to the swale should show a fixed cross-slope towards the swale (no crown). A uniform longitudinal slope is not required, but changes in (longitudinal) grade – positive and negative, associated with ‘dips’ – should be avoided. The lower edge of the carriageway should be formed as a continuous concrete overflow kerb.

Introduced treatment surface : This component is responsible for filtering all surface runoff passing to the swale from the streetscape catchment. It is a mixture of coarse sand (propagating sand, used in the building industry, is preferred) and single-size gravel (20 mm to 30 mm) in a 100 mm thick layer underlain with non-woven geotextile fabric and **placed at optimum moisture content**. It is permissible for this mixture to be enriched with a small quantity of loam providing the base for drought-tolerant vegetation (grass or ground cover).

The minimum length of the treatment surface is 3.5 m measured from the edge of the roadway overflow kerb to the remote edge of the bed of the swale (see Figure 7.5). The grade of the introduced surface is typically 1 in 5 to 1 in 8. This grade as well as the length of the swale side-slope itself may be determined by conveyance capacity considerations: the swale must be designed to carry flood flows corresponding to ARI, Y = 50- to 100-years events in the streetscape catchment (see Section 7.8.9).

The swale’s side-slope remote from the carriageway plays no part in the filtering process. Its grade may match that of the near-carriageway slope or it may be steeper depending on channel geometry required for flood conveyance. It is permissible in appropriate circumstances for the remote slope to be steep and take the form of a “rockery” containing suitable plants and shrubs. This could be a useful and attractive option in circumstances where the space available for a swale is severely limited.

The graphs which have been prepared for use in Procedures 7 and 8 in Australian climate zones (Figures 7.7 and D1 – D4 in Appendix D), are based on treatment surface area of 42 m² corresponding to 12.0 m × 3.5 m (see Figure 7.5). This allows for a 3.0 m-wide driveway crossing in each 15 m length of swale segment, the most extreme case that is likely to occur.

Aspects of treatment surfaces reviewed above – slopes, geometry, construction, etc. – are of less importance to the success of a ‘filter strip’ swale than the landscaping and horticultural expertise which must be available. Indeed, it is recommended that ‘filter strip’ swales be ignored as a possible stormwater ‘source control’ measure unless this expertise including knowledge of local native and/or drought-tolerant vegetation is readily accessible. Even where these conditions of opportunity and expertise are met, it is probably best to ‘hasten slowly’ with the technology and gain experience with a relatively small demonstration section of swale before applying it in a wholesale manner.

The harsh growing environment presented by introduced sand/gravel as a base for vegetation, particularly in the semi-arid climate zone, or during prolonged dry spells in temperate Australia, calls for grass or ground cover capable of good performance in these conditions without continuing irrigation after establishment. Three ground covers – *Marsillea drumondii*, *Centella asiatica* and *Viola hederacea* – and two wetland reeds *Juncus kraussii* and *Typha orientalis* have demonstrated the required robust capabilities in Southern Australia and may give similar satisfactory performance in the wetter climatic environments of Intermediate and Northern Australia. Of course, locally available drought-tolerant species of grasses and ground covers should be sought to provide the vegetation cover required of ‘filter strip’ swales.

Gravel-filled (trench) sub-structure : This component is similar to the gravel-filled trenches reviewed among the Category 1 installations in Chapter 5 and illustrated in Figures 1.3, 1.4 and 1.5. The gravel should be sharp-edged and of uniform size, typically, 20 mm – 30 mm nominal size, encased in non-woven geotextile fabric. All trenches specified in the design graphs are 1.0 m wide. The depths covered in the design graphs (Figures 7.7 and D1 – D4, Appendix D) are 0.30 m, 0.50 m, 0.75 m, and 1.0 m : these are minimum depths for the particular design conditions in each case.

‘Hydraulic’ cleansed stormwater disposal : As explained above, two options may be explored for this component :

- (a) access (recharge) to an aquifer; or,
- (b) ‘slow-drainage’ to a local formal or natural waterway.

This option is required throughout Australia in all cases where native soils show very low permeability; in Northern Australia the range of requirement extends to soils of medium permeability. The flow given by Procedure 8 in the design graphs (Figures 7.7 and D1 – D4, Appendix D), is expressed in terms of **L/s per 15 m length of swale segment**. The lengths of typical streetscapes where ‘filter strip’ swales may be used represent multiples of 15 metre segments, side-by-side. The **total** flow which must be managed in such cases is therefore equal to the q_{h15} flow (per 15 m swale segment) multiplied by the number of 15 metre segments : this gives Q_{hA} . Where aquifer access is available and environmentally appropriate, the number of bores required and hence the spacing of bores along the swale alignment should be determined from a knowledge of the bore recharge rate, q_r . Hence, the number of bores :

$$n \geq \frac{Q_{hA}}{q_r}, \quad \text{where “n” is an integer.} \quad (7.5)$$

In the case of a slow-drainage pipeline draining cleansed stormwater to a formal or natural drainage water-way outside the streetscape, the perforated pipeline capacity at its discharge point should be at least Q_{hA} . The flows involved are relatively small compared to flows normally experienced in storm drainage networks and are unlikely to be greater than 120 L/s (Northern Australia zone, extreme streetscape case).

7.8.6 ‘Lifespan’

Stormwater generated in the catchment draining to a swale will convey runoff from a number of surfaces including roof areas, residential paved surfaces, footpath, carriageway, pervious areas, etc., some of which are relatively clean and others represent potential sources of pollution, particularly sediment. This latter component of runoff determines the speed with which the treatment surface (introduced material) blocks, leading, ultimately, to the need for its replacement or reinstatement. The term ‘lifespan’, introduced in Section 3.1.2, is applied to the time taken for progressive blocking to reach finality.

This process, as it takes place in ‘filter strip’ swales, is similar to the “blockage front” phenomenon described in Section 3.1.3 and quantified in Table 3.1. Information given in that table can be used to estimate ‘lifespan’ in these swales. It is related to :

- the **ground-level** impervious area, I , draining to a porous (or permeable) surface, such as a ‘filter strip’ swale;
- the area, P , of the filter strip receiving the runoff; and,
- the particular environmental conditions of the site – proximity to trees, wind-driven sand supply to the site from beaches, etc.

The first and second of these variables are expressed in the ratio (I/P). It is important to note that the value of “ I ” used in this ratio is **not** the same as A_{eia15} which includes roof areas. Also, “ I ” does not include ground-level areas devoted to porous or permeable paving.

The area of swale receiving runoff in all cases modelled to produce the graphs (Figures 7.7 and D1 – D4) is fixed at 42 m^2 ($3.5 \text{ m} \times 12.0 \text{ m}$) per 15 m segment, so that ratios of (I/P) may range from 1.0 or less, where permeable paving is used extensively up to 6.0 or more where little consideration, if any, has been given to retaining runoff from ground-level paved surfaces. Estimates of ‘lifespan’ can be made from Table 3.1 which shows a “times 5” factor for vegetated porous surfaces. In fact, the field observations at St. Elizabeth Church Car Park, Adelaide (see Section 3.9) which led to the recommended factor reported no significant ‘blocking’ after 6 years of service in an (I/P) = 40 environment! Use of the “times 5” factor (Table 3.1) leads to ‘lifespan’ estimates ranging from about 15 years up to 50 years in typical cases. Based on experience at St. Elizabeth Church, these estimates are likely to be conservative.

It is important for practitioners to realise that ‘lifespan’ values quoted here are derived from data for **established neighbourhood** conditions only. Poor monitoring/control of construction phase activities can reduce ‘lifespan’ to a fraction of the values indicated in the above discussion (see Section 1.3.3).

7.8.7 Procedures 7 and 8 and Figure 7.7 – Illustrations**ILLUSTRATION 1 – APPLICATION OF PROCEDURE 7**

Consider the requirements for a ‘filter strip’ swale in the following circumstances :

- Location : Adelaide, $i_{10,1} = 25$ mm/h, therefore Southern Australia zone – Figure 7.7.
- Local soil hydraulic conductivity, $k_h = 2 \times 10^{-4}$ m/s (sand); Moderation Factor, $U = 0.5$.
- Catchment EIA : 400 m^2 per 15 m length of swale segment, A_{eia15} .

Hence, Moderated hydraulic conductivity :

$$k_h = (2 \times 10^{-4}) \times U = 2 \times 10^{-4} \times 0.5 = \mathbf{1.0 \times 10^{-4} \text{ m/s}};$$

Catchment equivalent impervious area :

$$A_{\text{eia15}} = \mathbf{400 \text{ m}^2}.$$

Locate this data pair in Figure 7.7 at ‘1’.

Interpretation : The ‘filter strip’ swale requires treatment surface only comprising sand/gravel mixture, 100 mm deep, installed in the swale slope adjacent to the roadway reserve carriageway and extending 3.5 m (minimum) across the bed of the swale (see Figure 7.5).

ILLUSTRATION 2 – APPLICATION OF PROCEDURE 7 :

- Location : Adelaide, $i_{10,1} = 25$ mm/h, therefore Southern Australia zone – Figure 7.7.
- Local soil hydraulic conductivity, $k_h = 3 \times 10^{-6}$ m/s; Moderation Factor, $U = 2.0$.
- Catchment EIA : 380 m^2 per 15 m length of swale segment, A_{eia15} .

Hence, Moderated hydraulic conductivity :

$$k_h = (3 \times 10^{-6}) \times U = 3 \times 10^{-6} \times 2 = \mathbf{6.0 \times 10^{-6} \text{ m/s}};$$

Treatment equivalent impervious area :

$$A_{\text{eia15}} = \mathbf{380 \text{ m}^2}.$$

Locate this data pair in Figure 7.7 at ‘2’.

Interpretation : The ‘filter strip’ swale requires treatment surface as described in Illustration 1, above, with gravel-filled trench 1.0 m wide (all cases) and $H = 1.0$ m (from graph).

ILLUSTRATION 3 – APPLICATION OF PROCEDURE 8 :

- Location : Adelaide, $i_{10,1} = 25$ mm/h, therefore Southern Australia zone – Figure 7.7.
- Local soil hydraulic conductivity, $k_h = 2 \times 10^{-6}$ m/s; Moderation Factor, $U = 2.0$.
- Catchment EIA : 480 m^2 per 15 m length of swale segment, A_{eia15} .

Hence, Moderated hydraulic conductivity :

$$k_h = (2 \times 10^{-6}) \times U = 2 \times 10^{-6} \times 2 = \mathbf{4.0 \times 10^{-6} \text{ m/s}};$$

Catchment equivalent impervious area :

$$A_{\text{eia15}} = \mathbf{480 \text{ m}^2}.$$

Locate this data pair in Figure 7.7 at ‘3’.

Interpretation : This ‘filter strip’ swale requires treatment surface as described in Illustration 1, above, with gravel-filled trench 1.0 m wide and $H = 0.75$ m, and ‘hydraulic’ (additional) disposal at the rate of 0.36 L/s per 15 m swale segment, q_{h15} .

7.8.8 ‘Filter-strip’ swales in Australian environments

Application of Procedures 7 and 8, as set out above, in the five climate zones recognised in Figure 3.8 leads to a mix of swale outcomes. The ‘mix’ is determined not only by climate variation but also by the soil type in which the ‘filter strip’ swale is located and the presence or absence of aquifers.

Table 7.1 lists the likely outcomes for three of these zones for streetscape swales designed according to Procedures 7 and 8. It should be borne in mind that the design approach which underpins these procedures is **Strategy A**, described in Section 4.3.1. This represents a very high standard of pollution containment and carries with it the promise of relatively long ‘lifespan’, as reviewed above, before filter strip reinstatement might be required. This excludes regular inspection and maintenance of the treatment surface to keep it functioning including mowing, attending to ground cover and repairing scour channels or sheet erosion whenever these occur. The sub-structure system of gravel-filled trench and bore or slow-drainage pipeline – where these components are included – is virtually isolated from the impact of sediment and, therefore, can be expected to give service of indefinite length.

TABLE 7.1

‘FILTER STRIP’ SWALES PROVIDING A HIGH LEVEL OF POLLUTION CONTROL (STRATEGY A) IN SOUTHERN, MID-INTERMEDIATE AND NORTHERN AUSTRALIA

PARENT SOIL TYPE		ZONE		
		SOUTHERN AUSTRALIA	MID-INTERMEDIATE AUSTRALIA	NORTHERN AUSTRALIA
SANDY SOILS	Coarse*	Filter strip only required (no sub-structure)	Filter strip only required (no sub-structure)	Filter strip and trench sub-structure, 1.0 m deep
	Fine		Filter strip and trench sub-structure, 1.0 m deep	Filter strip, trench sub-structure, 1.0 m deep with ‘hydraulic’ assistance, $Q_{h15} = 3.6 \text{ L/s}^{**}$
SANDY CLAYS		Filter strip and trench sub-structure, 1.0 m deep	Filter strip, trench sub-structure, 1.0 m deep with ‘hydraulic’ assistance, $Q_{h15} = 1.6 \text{ L/s}^{**}$	Filter strip, trench sub-structure, 1.0 m deep with ‘hydraulic’ assistance, $Q_{h15} = 8.0 \text{ L/s}^{**}$
ALL CLAYS		Filter strip, trench sub-structure, 0.75 m deep with ‘hydraulic’ assistance, $Q_{h15} = 0.36 \text{ L/s}^{**}$	Filter strip, trench sub-structure, 1.0 m deep with ‘hydraulic’ assistance, $Q_{h15} = 1.6 \text{ L/s}^{**}$	Filter strip, trench sub-structure, 1.0 m deep with ‘hydraulic’ assistance, $Q_{h15} = 8.0 \text{ L/s}^{**}$

* See note re sites with coarse sand and significant **dissolved pollution** in Figure 7.7 and Appendix D.

** All ‘hydraulic’ assistance flows quoted are *per 15 m swale segment*. Warnings relating to injection of these flows into aquifers or waterways should be heeded (see Sections 7.8.2 and 7.8.4).

NOTE : Introduced filter strip sand/gravel mix must have hydraulic conductivity, k_h , at least $2.0 \times 10^{-4} \text{ m/s}$. Propagating sand with 20 mm gravel is suitable for this duty.

Outcomes of particular interest from Table 7.1 are :

- successful ‘filter strip’ swales can be constructed without gravel-filled sub-structures or ‘hydraulic’ assistance, in sandy soils in much of the southern regions of the continent;
- filter strips with introduced sand/gravel and gravel-filled trenches up to 1.0 m deep, only, are required in the coarse sands of Northern Australia, the fine sandy soils of much of Intermediate Australia and in the sandy clays of Southern Australia;
- the full ‘filter strip’ swale structure, including treatment surface, gravel-filled sub-structure and ‘hydraulic’ assistance, is required in all other cases;
- the quantity of cleansed stormwater which must be removed (“hydraulic assistance”) from the base of the formal structure varies from 0.36 L/s per 15 m swale segment in Southern Australia up to 8.0 L/s per segment in Northern Australia.

7.8.9 Design for flows greater than Q_{lim}

The basis for design of ‘filter strip’ swales by Procedures 7 and 8 is focussed on control of flows up to and including, *nominally*, those of ARI, $Y = 0.25$ years and **bypass** of all flows greater than this limit. Such (bypass) flows appear from time to time as surface flows exceeding the infiltration capacity of the filter strip, and follow moderate to large storm burst rainfall. The **total** flood flow involved, Q_Y , can be managed in a swale by considering it as, effectively, two flows, partitioned as follows :

- flow equal to $Q_{0.25}$ managed within the filter strip and its sub-structure; and,
- a “gap” flow of $(Q_Y - Q_{0.25})$ confined to the swale channel itself (see Procedure 7, STEP 5, following).

The former component is an element of design, adequately catered for within Procedures 7 and 8. The latter (component) can be managed as open channel flow passing to some downstream receiving domain such as a local drainage path or waterway. Its conveyance characteristics – depth, velocity, etc., – may be determined by reference to Manning’s formula or some similar, recognised hydraulic procedure such as Izzard’s triangular flow formulation (Argue, 1986).

It will be found that swale channels 0.30 m deep can accommodate major flood “gap” flows (ARI, $Y = 50$ -to 100-years) provided the street lengths are not excessive. In Southern Australia, residential street lengths up to 300 m can apply the technology; the corresponding limit in Northern Australia is about 200 m. Alternatively, it may be possible to collect the flow and pass it to a site set aside as a “dry pond” within the streetscape reserve or, perhaps, off-line : Procedure 3 (Section 5.1) could be applied to provide a satisfactory resolution to the problem of disposal in these circumstances.

7.8.10 Procedures 7 and 8 – the step-by-step process

The material presented above provides the basis for a step-by-step design process commencing with data requirements and climate zone selection leading to ‘filter strip’ swale design including sub-structure dimensioning and ‘hydraulic assistance’ flow detailing, where required, and also including hydraulic conveyance capacity considerations taking account of the flood flows for which the swale channel must be designed.

Preliminary considerations – site data, etc.

The procedure must commence with a clear description of the circumstances in which the design process is to take place. This involves identifying :

- The climate zone in which the streetscape is located.
- Components of the typical allotment and its portion of fronting roadway reserve draining to the swale. This list must identify, in particular, roof areas, paved surface areas and whether they are impervious, porous or permeable, pervious areas, etc.
- Physical properties of the streetscape – allotment frontage (typical), street length and longitudinal slope.
- Streetscape time of concentration, t_C . Hydrological (flood control) aspects of the design (STEP 5, below) come under the provisions of ‘site’ drainage (see Sections 4.2.4 and 4.2.6), in particular “entry works” which do not involve storage considerations. External critical storm durations – $T_{C(total)}$ and $T_{C(local)}$ – are therefore not relevant to swale design. Design intensity for t_C in the major storm set by local council requirements (ARI, $Y = 50$ to 100-years) should be listed.

PROCEDURE 7 – STEP 1 :

Determine equivalent impervious area of typical unit contributing runoff to the filter strip. This produces a total EIA for the typical allotment including a corresponding portion of the fronting roadway reserve and a portion of the swale corresponding to (allotment frontage length $\times$ 3.5 m), explained in Section 7.8.3. These data lead to determination of A_{eia15} , the equivalent impervious area which impacts on the filter strip component of the swale.

STEP 2 :

Identify climate zone of site based on $i_{10,1}$ rainfall intensity and select appropriate design graph from Figure 7.7 (Southern Australia) or from Appendix D (Figures D-1 to D-4).

Determine moderated hydraulic conductivity : $k_h \times$ (Moderation factor), as previously – see Section 7.7.

Locate ‘point’ in selected design graph corresponding to moderated hydraulic conductivity (X-axis) and A_{eia15} (Y-axis). The ‘point’ will fall either within the range covered by Procedure 7 or within the scope of Procedure 8 (see Section 7.8.7).

STEP 3 :

Determine required components of the filter strip – the treatment surface (always) **and** gravel-filled sub-structure, if applicable – and their (minimum) dimensions if and only if the ‘point’ falls within the scope of “Procedure 7” on the graph. Illustrations 1 and 2, Section 7.8.7, demonstrate this process. The plan area of the treatment surface – **42 m²** (minimum) – is universal and required in STEP 4, all cases.

However, if the ‘point’ lies within the domain of “Procedure 8”, all **dimension** information indicated in the graph is bypassed at this stage and re-visited under STEP 6, below. The design process for these cases continues, however, through STEPS 4 and 5 which are parts of both Procedures 7 and 8.

STEP 4 :

Estimate the ‘lifespan’ of the filter strip. This requires area I to be determined from the basic case data. Area I is the sum of EIAs of all *ground-level* components delivering runoff to the filter strip : roof areas and areas of porous or permeable paving are specifically excluded (see Section 7.8.6). From these data, a value of the parameter (I/P) is calculated and a value for ‘lifespan’ interpolated from Table 3.1. It is important in this operation to take note of the factor “5 times” which applies to vegetated treatment surfaces receiving stormwater.

STEP 5 : Swale hydraulic capacity (flood flow) considerations

This step involves calculating the major flood flow (design ARI in the range Y = 50- to 100-years) which must be conveyed by the streetscape swale without surcharge, as set by council requirements. This can be achieved by calculating the flood flow peak passing from the streetscape considered as a ‘site’ case **not** involving storage (see above, this section). The peak flow is determined :

$$Q_Y = \frac{(CA) i_Y}{0.36} \text{ L/s} \quad \text{with (CA) in hectares} \quad (7.6)$$

$$\text{where} \quad (CA) = \frac{A_{eia15} \times \text{the number of 15 m segments in the streetscape}}{10,000}.$$

The swale conveyance capacity is calculated using conventional open channel flow formulae such as Manning’s Equation or Izzard’s formulation (Argue, 1986). Q_Y must be significantly below the open channel (swale) capacity to ensure passage of the Y-years event without surcharge.

It should be noted that no allowance has been made in these calculations for flow abstracted into the filter strip during passage of a major flood in the streetscape. Account could be taken of this flow (nominally $Q_{0.25}$), but its omission is intentional and represents a ‘margin of safety’ which is considered to be warranted under circumstances of major flooding.

Another hydraulic constraint which should also be checked is that relating to culvert capacity at driveway crossings where these are used. The critical condition is likely to occur at the downstream extremity of the street where the entire flood flow, Q_Y , must pass through the last crossing. This installation should be designed as a culvert system with, possibly, multiple pipes. In this event, crossings upstream may have appropriately reduced numbers of pipe-culverts. Culverted crossings should incorporate rip-rap or gabion (scour) protection where the pipes discharge downstream.

In extreme circumstances where the presence of culverted crossings is likely to cause channel surcharge, such installations must give way to simple ‘low level’ driveways of the type illustrated in Figure 2.4.

The completion of STEP 5 concludes the design process for cases falling within the scope of Procedure 7. Those which have been shown (STEP 3) as “Procedure 8 cases” continue to STEP 6 where the issues of detailed dimensioning and setting of ‘hydraulic assistance’ flows are addressed.

STEP 6 : PROCEDURE 8 – revisit STEP 3

All ‘filter strip’ swale cases continuing to this stage comprise three components :

- Filter strip – 100 mm deep, sand/gravel mix, minimum 3.5 m length (see Figure 7.5).
- Gravel-filled sub-structure, 1.0 m wide (all cases), minimum depth, H, given by ‘point’ on the design graph noted in STEP 3.
- ‘Hydraulic assistance’, also indicated by the position of the ‘point’ plotted in STEP 3.

Illustration 3, Section 7.8.7 demonstrates this process.

Sufficient basic information is available to the designer through the six-step process, above, to enable him/her to complete detailed design of a ‘filter strip’ swale for any location in Australia.

7.8.11 Construction : some recommendations

Construction of a ‘filter strip’ swale should be integrated with construction of the carriageway with which it is associated. However, it should **under no circumstances** be called upon to operate in an environment of *unregulated* general building/construction activity that frequently follows completion of a carriageway. ‘Unregulated’ here means “inadequate sediment control associated with the off-site movement of construction equipment”. ‘Filter strip’ swales are capable of long service receiving and processing stormwater runoff from **established** residential street settings, only : the warning contained in the last paragraph of Section 7.6.5 should not go unheeded.

The most vulnerable (to physical damage) component of the ‘filter strip’ swale is undoubtedly the sand/gravel treatment surface, particularly during the period of its construction and sowing/planting with grass or selected ground cover. Coincidence of a significant storm with the sand/gravel laying and sowing/planting processes could result in erosion of sand in the immediate vicinity of the carriageway edging as well as ‘scouring off’ of upper level sand from the treatment surface itself. The first of these problems can be ongoing unless preventative action is taken; the second is temporary, until such time as the grass is established or the ground cover is thriving.

The most effective action which can be taken to prevent edge erosion in the vulnerable area – say the 0.5 m width beside the carriageway – is to provide some form of **porous** paving along this strip. There are a number of proprietary products which are suited to this duty including plastic ring-matrix and masonry systems (see Figure 3.1a; St. Elizabeth Church car park, Section 3.9). The essential criteria which should be sought in selecting a suitable product are strong grass integration with the system and a high level of porosity.

Some repair of the surface may, of course, be necessary, particularly where the construction schedule coincides with a large storm – say a “20-year” or larger event – and/or wherever a concentrated entry to the sloping carriageway surface from an upstream residence results in jet-impact on the swale surface matrix. The use of hay bales to break up such jet action is advised.

Clearly, it is better – if at all possible – for construction of such flood-sensitive components to take place when the risk of major storm rainfall occurring is minimum. In the case of Northern Australia, this is during the “dry” (winter) season. In Southern Australia, winter is also the preferred period as storm burst rainfall – while more frequent than in summer – is less intense; also, winter sowing avoids the intense heat of summer. Local climatic conditions of Intermediate Australia can be related, broadly, to these two extremes.

An alternative to sowing/planting is to use established (roll-out) grass to cover the entire or particularly vulnerable parts of the swale cross section. Provided the type of grass is well-suited to the climatic and irrigation conditions in which it is expected to survive, this is a satisfactory though more expensive solution.

There is one further aspect of construction and management of ‘filter strip’ swales, as described above, which is worthy of attention. This relates to use of the sloping swale surface for off-carriageway parking. This has been recognised as a major problem for municipal agencies responsible for maintaining swales. Solutions to date have been preventative, such as the border fencing illustrated in Figure 2.4. The sand/gravel mixture recommended for the filter strip in Procedure 7 and described in more detail above, provides a cost-effective alternative to this solution. The sand/gravel matrix with grass or ground-cover, forms a surface suitable for standing vehicles provided this use is rare. It is recommended, therefore, that irregular or occasional vehicle parking on swale slopes – constructed as outlined above – rather than regular use, could be tolerated without seriously impacting on the performance of the system. Notices advising offenders of the imposition of a fine for unlawful use of the filter strip, together with some vigilance, may be all that is needed to ensure minimal use of swales for car parking.

7.9 ‘FILTER-STRIP’ SWALES : ILLUSTRATIVE EXAMPLES

7.9.1 Introduction to Examples 7.5 and 7.6 : swales for two residential streetscapes

The following examples relate to the tasks of designing ‘filter strip’ swales for two streetscape scenarios in which the alternative types of development illustrated below are characteristic. Both allotments (Figure 7.8) are 1,000 m² in area (‘quarter acre’ block) with 30 m frontages. The first (Figure 7.8a), is typical of conventional development in the older suburbs of Australian cities; the second (Figure 7.8b), represents the same site area, this time accommodating two medium density residences. The two cases re-visit Examples 5.6 and 5.7.

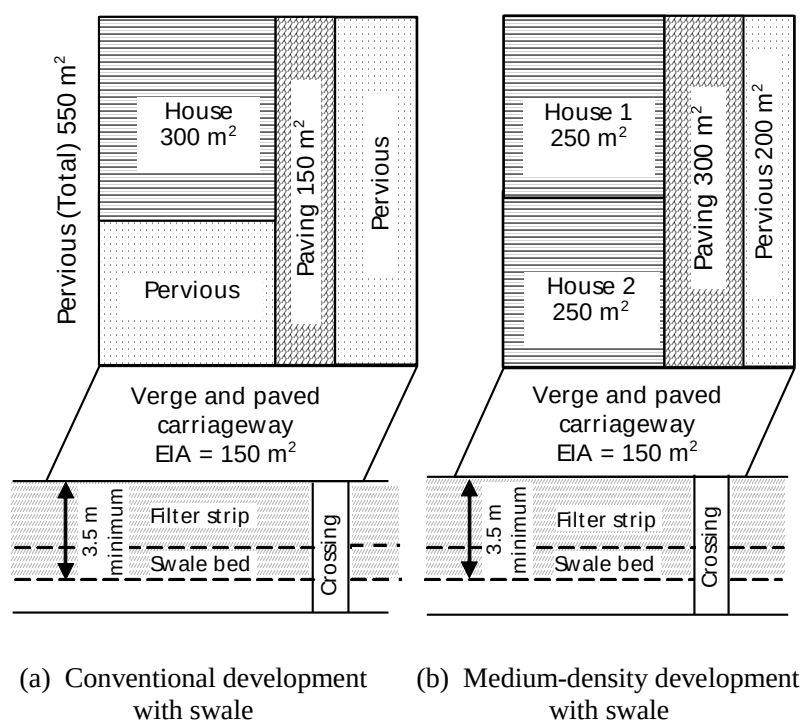


FIGURE 7.8 : Layout of typical residential streetscape developments used in Examples 7.5 and 7.6

7.9.2 Example 7.5 : Conventional development streetscape with swale

The task involved in designing a ‘filter strip’ swale streetscape in this case has two parts :

- design of the filter strip to trap pollutants conveyed in runoff from the carriageway; and,
- design of the swale to convey the full range of stormwater (flood) flows from ARI, $Y = 0.25$ -years up to $Y = 100$ -years.

In this example, the streetscape is 180 m long, located in an “...average suburb with some trees...” in Brisbane, Northern Australia. Other site and catchment characteristics, in particular, details of catchment components draining to the swale along each (typical) residential allotment frontage are :

- total connected roof area 300 m²
- allotment paved area (permeable paving) 150 m²
- allotment pervious area (runoff coeff, $C_{10} = 0.70$).....550 m²
- verge, footpath and carriageway (runoff coeff, $C_{10} = 0.90$).....150 m²
- soil k_h , hydraulic conductivity (sand)..... 5×10^{-4} m/s
- allotment frontage, 30 m; streetscape length, 180 m; longitudinal slope, 1.0 %
- streetscape time of concentration, t_c 25 minutes
- critical storm duration, $(T_c)_{total}$, Council specification.....90 minutes
- design storm intensity (ARI, $Y = 100$ -years, Council specification) ...173 mm/h (25 mins storm)

Notes :

- there is no runoff contribution to the swale from ‘low’ side residences (see Figure 7.6);
- the Q_{100} flood peak estimated for the streetscape is based on ‘time of concentration’, t_c , for the streetscape, not total or local critical storm duration, see Section 4.2.6 (“On-site entry works”).

PROCEDURE 7 : ‘Filter strip’ swale for conventional residential development (see Section 7.8.10)**STEP 1 :**

Determine EIA (equivalent impervious area) of typical unit contributing runoff to the swale :

- roof area = 300 m²
 - allotment paved area (permeable paving) = 0 m²
 - allotment pervious area = $550 \text{ m}^2 \times 0.7$ = 385 m²
 - verge and paved carriageway = 150 m²
 - filter strip – 30 m $\times$ 3.5 m (see Section 7.8.3)..... = 105 m²
- TOTAL..... = 940 m², 30 m frontage**

Hence, $A_{eia15} = 470 \text{ m}^2$ per 15 m (standard) swale segment

STEP 2 :

Identify climate zone of site based on $i_{10,1}$ rainfall intensity (= 71 mm/h),

hence Northern Australia, Appendix D, graph D-4.

Determine moderated hydraulic conductivity : $k_h \times (\text{Moderation factor})$

$$= 5 \times 10^{-4} \text{ m/s} \times (0.5) = 2.5 \times 10^{-4} \text{ m/s}$$

EIA per 15 m segment $A_{eia15} = 470 \text{ m}^2$.

STEP 3 :

Determine required components of filter strip and their dimensions (from Figure D-4) :

Filter strip (sand/gravel mix), 100 mm deep, underlain with geotextile, at least 3.5 m wide;

plan area – **42 m²**, minimum; this information is used in STEP 4, following.

Gravel-filled trench beneath swale bed, 1.0 m wide, **H = 0.75 m** (just!).

STEP 4 :

Determine ‘lifespan’ of filter strip system identified in STEP 3.

Ground-level surfaces contributing runoff to the filter strip are :

- allotment pervious area = $550 \text{ m}^2 \times 0.7$ = 385 m^2
- verge and paved carriageway = 150 m^2

TOTAL.....I = 535 m², 30 m frontage

$$\text{Hence, I/P ratio (ground-level runoff)} = \frac{535 \text{ m}^2 \div 2}{42 \text{ m}^2} = \mathbf{6.4}$$

Interpolating in Table 3.1 – “...average suburb with some trees” – and noting “times 5” factor for vegetated porous treatment surfaces, estimated ‘lifespan’ is 5×2.8 years = **14 years (minimum)**.

STEP 5 : Swale hydraulic capacity (flood flow) considerations

Determine (streetscape) Q_{100} flood flow and conveyance capacity of swale :

$$Q_{100} \text{ peak flow} = \frac{(CA)i_{100}}{0.36} \text{ L/s with (CA) in hectares,}$$

$$\text{hence, } \frac{(470 \times 12 \div 10,000) \times 173}{0.36} = \mathbf{271 \text{ L/s}}$$

Swale basic dimensions : depth = 0.30 m; bed width = 1.0 m; side-slope (filter strip) = 1 in 8;

minimum hydraulic radius = 0.18 m; bed slope, $S_o = 0.010$. **Manning’s equation, $Q = 0.38 \text{ m}^3/\text{s}$ or 380 L/s** (“n” = 0.06), see Argue (1986). **Result :** the swale will convey the Q_{100} peak flow without surcharge.

Driveway crossings (six) – culvert required to convey 271 L/s (end of street flow) :

4 × 225 mm diam concrete pipes (consult culvert handbook) will convey this flow. Normal rip-rap or gabion (scour) protection should be provided downstream of crossings. Number of pipe culverts required at crossings may be reduced upstream in proportion to contributing catchment.

Completion of STEP 5 concludes basic design of a ‘filter strip’ swale requiring application of Procedure 7, only, as determined by consultation of the relevant design graph, in this case Figure D-4.

7.9.3 Example 7.6 : ‘Filter strip’ swale for medium density residential development

The task involved in designing a ‘filter strip’ swale streetscape for this case has two parts :

- design of the filter strip to trap pollutants conveyed in runoff from the carriageway; and,
- design of the swale to convey the full range of stormwater (flood) flows from ARI, $Y = 0.25$ -years up to $Y = 100$ -years.

In this example, the streetscape is 180 m long and is located in an “...average suburb with some trees...” in Brisbane, Northern Australia. Other site and catchment characteristics, in particular, details of catchment components draining to the swale along each (typical) residential allotment frontage are :

- total connected roof area 500 m²
- allotment paved area (permeable paving) 300 m²
- allotment pervious area (runoff coeff, $C_{10} = 0.70$) 200 m²
- verge, footpath and carriageway (runoff coeff, $C_{10} = 0.90$) 150 m²
- soil k_h , hydraulic conductivity (medium clay) 3×10^{-6} m/s
- allotment frontage, 30 m; streetscape length, 180 m; longitudinal slope, 1.0 %
- streetscape time of concentration, t_c 25 minutes
- critical storm duration, $(T_c)_{total}$, Council specification 90 minutes
- design storm intensity (ARI = 100-years, Council specification) 173 mm/h (25 mins storm)

Notes :

- there is no runoff contribution to the swale from ‘low’ side residences (see Figure 7.6);
- the Q_{100} flood peak estimated for the streetscape is based on ‘time of concentration’, t_c , for the streetscape, not total or local critical storm duration, see Section 4.2.6 (“On-site entry works”).

PROCEDURE 7 : Design of filter strip components of swale

STEP 1 :

Determine EIA (equivalent impervious area) of typical unit contributing runoff to the swale :

- roof area = 500 m²
- allotment paved area (permeable paving) = 0 m²
- allotment pervious area = $200 \text{ m}^2 \times 0.7$ = 140 m²
- verge and paved carriageway = 150 m²
- filter strip (see above) = 105 m²

TOTAL..... = 895 m², 30 m frontage

Hence, $A_{eia15} = 448 \text{ m}^2$ for 15 m (standard) swale segment.

STEP 2 :

Identify climate zone of site based on $i_{10,1}$ rainfall intensity (= 71 mm/h),

hence Northern Australia, Appendix D, graph D-4.

Determine moderated hydraulic conductivity : $k_h \times (\text{Moderation factor})$

$$= 3 \times 10^{-6} \text{ m/s} \times (2.0) = 6.0 \times 10^{-6} \text{ m/s}$$

EIA per 15 m segment $A_{\text{eia15}} = 448 \text{ m}^2$.

STEP 3 :

Determine required components of filter strip and their dimensions (from Figure D-4). Although this case falls, clearly, into the domain of Procedure 8, the filter strip dimensions are universal. Hence :

Filter strip (sand/gravel mix), 100 mm deep, underlain with geotextile, at least 3.5 m wide,

plan area – **42 m²**, minimum.

This information is used in STEP 4, following.

STEP 4 :

Determine ‘lifespan’ of filter strip system identified in STEP 3.

Ground-level surfaces contributing runoff to the filter strip are :

- allotment pervious area = $200 \text{ m}^2 \times 0.7$ = 140 m^2
- verge and paved carriageway = 150 m^2

TOTAL.....I = 290 m², 30 m frontage

Hence, I/P ratio (ground-level runoff) = $\frac{290 \text{ m}^2 \div 2}{42 \text{ m}^2} = \mathbf{3.4}$

Interpolating in Table 3.1 – “...average suburb with some trees” – and noting factor of 5 for vegetated porous treatment surfaces, estimated ‘lifespan’ is 5×4.5 years = **22 years (minimum)**.

STEP 5 : Swale hydraulic capacity (flood flow) considerations

Determine (streetscape) Q_{100} flood flow and conveyance capacity of swale :

$$Q_{100} \text{ peak flow} = \frac{(CA)i_{100}}{0.36} \text{ L/s with (CA) in hectares,}$$

$$\text{hence, } \frac{(448 \times 12 \div 10,000) \times 173}{0.36} = \mathbf{258 \text{ L/s}}$$

Swale basic dimensions : depth = 0.30 m; bed width = 1.0 m; side-slope (filter strip) = 1 in 8;

minimum hydraulic radius = 0.18 m; bed slope, $S_o = 0.010$. **Manning’s equation, $Q = 0.38 \text{ m}^3/\text{s}$ or 380 L/s** (“n” = 0.06), see Argue (1986). **Result :** the swale will convey the Q_{100} peak flow without surcharge.

Driveway crossings (six) – culvert required to convey 271 L/s (end of street flow) :

4 × 225 mm diam concrete pipes (consult culvert handbook) will convey this flow.

STEP 6 : Procedure 8 – consult Figure D-4

Moderated hydraulic conductivity : $k_h \times (\text{Moderation factor})$

$$= 3 \times 10^{-6} \text{ m/s} \times (2.0) = \mathbf{6.0 \times 10^{-6} \text{ m/s}}$$

EIA per 15 m segment..... $A_{\text{eia15}} = 448 \text{ m}^2$.

Filter strip requirements with ‘hydraulic’ assistance :

- Filter strip (sand/gravel mix), 100 mm deep, underlain with geotextile, at least 3.5 m wide;

- Gravel-filled trench beneath swale bed, 1.0 m wide, $H = 0.75$ m (just !);
- ‘Hydraulic’ assistance (bore or slow-drainage pipeline) : $q_{h15} = 7.2$ L/s (flow per 15 m segment).

The latter requirement must be interpreted into the context of the site (12 segments).

Where aquifer access is available (and environmentally appropriate), the **total** ‘hydraulic’ flow for the streetscape, Q_{hA} , should be determined ($12 \times q_{h15} = 86.4$ L/s) and the number of bores, n , calculated from a knowledge of the permissible recharge rate per bore, q_r :

$$n \geq \frac{Q_{hA}}{q_r} \quad (7.5)$$

where “ n ” is an integer.

Alternatively, where aquifer access is denied, a slow-drainage (perforated) pipeline may be installed to convey the entire ‘hydraulic assistance’ flow to disposal outside the streetscape. In this case :

Total ‘hydraulic’ flow – as above, Q_{hA} – is 86.4 L/s; this can be conveyed in a 300 mm diam PVC pipe, perforated, except where it discharges from the streetscape into a downstream receiving domain.

Completion of STEP 6 concludes basic design of a ‘filter strip’ swale requiring application of Procedures 7 and 8 as determined by consultation of the relevant design graph, in this case Figure D-4.

7.10 ‘FILTER STRIP’ SWALES : CONCLUDING COMMENT

Practitioners seeking information and guidance from Sections 7.8 and 7.9 on design aspects of ‘filter strip’ swales, particularly those whose spheres of operation lie in the Mid-Intermediate to Northern climate zones of Australia, will discover the recommended layouts derived from Figures D2, D3 and D4 (Appendix D) to be unacceptably restrictive. This situation arises partly from the high level of treatment (of stormwater) adopted as the ‘target standard’ throughout Chapter 7, and partly from a particular concern about treatment of *dissolved pollutants* present in storm runoff.

These considerations lead directly to two restrictive recommendations being made in relation to swale layouts :

1. ‘filter strip’ swales are not advised for sites where sandy soils predominate (moderated hydraulic conductivity greater than 1.0×10^{-4} m/s); and,
2. use of Procedure 8 resulting in passage downstream of partly-treated stormwater – applicable, in particular, to clay and (some cases) sandy-clay sites – should be avoided wherever possible (see Sections 7.8.2 and 7.8.4).

The high standard of treatment referred to, namely, Strategy A (defined in Section 4.3), seeks to achieve substantial treatment of at least 95% of average annual runoff passing to the swale. [95% ‘substantial treatment’ corresponds to 90% retention, as explained in Section 4.3.1.] It is simply not possible to provide this level of treatment at typical sites in the Mid-Intermediate to Northern climate zones, inclusive, of Australia within a 3.5 m swale (effective) width – see Figure 7.5. The “Australian Runoff Quality” (ARQ) manual (Engineers Australia, 2006) has provided some relief for practitioners facing this difficulty by offering lower levels of retention – 80%, 70%, 60% – corresponding to levels of treatment significantly below the 95% ‘target standard’ of the present text. The “ARQ” manual also provides design directions for taking such action. [A recent, comprehensive review of the scope and performance of conventional bio-retention systems can be found in Davis et al, 2009.]

An alternative approach is to adopt Strategy B or ‘first flush’ treatment (see Section 4.3) set out in Sections 7.4.2 and 7.5.2. This is illustrated in Example 7.4, Section 7.7.7, and leads to a ‘dry’ pond solution which could be readily integrated into a typical residential streetscape where the local soil is characterised as sandy clay. To extend this alternative into the realm of streetscapes constructed in medium clay soils (moderated hydraulic conductivity, k_h , ranging from 1.0×10^{-6} m/s up to 1.0×10^{-5} m/s) is likely to require the presence of a “soakaway” sub-structure of the type described in Examples 7.2 and 7.3, Section 7.7.7. [A filter bed of introduced (sand) material laid on non-woven geotextile fabric would be required above the “soakaway”.] Study of these examples may lead to satisfactory design solutions for such cases in the Mid-Intermediate to Northern climate zones.

The particular exclusion listed as Item 1, above, is aimed at avoiding the quite inadequate treatment of dissolved pollution provided by (rapid) percolation of stormwater through sandy soils. While this restriction is of relatively minor importance in locations where clay soils dominate – the situation most commonly encountered across the Australian landmass – it poses major problems for those seeking high levels of treatment in regions where sandy soils are widespread, for example, in Perth, Western Australia. Evidence of recent eutrophication in the Swan-Canning and Peel-Harvey-Serpentine waterway systems has been linked to nutrients (phosphorus and nitrogen) carried in runoff from Perth suburbs and from the nearby city of Mandurah, respectively.

A potential solution to this problem is provided by the research of those who have discovered the impressive treatment capability observed in bed sediments of detention ponds (Datry et al, 2003; Fischer et al, 2003; Weiss et al, 2006) and in native clay soils (Jefferies and Napier, 2008). ‘Solutions’ based on these observations would involve interrupting the normal (percolation) flow paths of such areas with, for example, clay (or other suitable media) barriers or ‘treatment layers’. Evidence from the research suggests that substantial (pollution) concentration reductions can be achieved in remarkably short distances – of order 200 to 300 mm.

It might be argued that pollution concentrations – dissolved pollution, in particular – are likely to show significant variation across the urban land use spectrum, ranging from high levels at highway intersections to low or trivial levels in residential cul-de-sacs : such a finding could justify a relaxed view of the ‘restrictive recommendations’, above, being taken by practitioners.

The following information comes to light in pursuit of this enquiry (see Section 6.2.3 and Tables 6.3, 6.6 and 6.12) :

1. copper, zinc and hydrocarbons show significant differentiation – as suggested – across the land use spectrum;
2. lead, chromium, cadmium, nickel and iron, as well as the nutrients phosphorus and nitrogen, show poor differentiation across the land use spectrum;
3. lead and iron (predominantly ferric) are readily adsorbed onto particulate matter;
4. zinc, cadmium, chromium and nickel can occur in soluble form but are mainly adsorbed onto particulate matter;
5. phosphorous and nitrogen (particularly nitrate) occur in soluble form but, mainly, show adsorption to particulate matter;
6. competent sediment-collection systems (including filters) can account for the bulk of lead, iron, zinc, cadmium, chromium, nickel, phosphorus and nitrogen occurring in (new) residential neighbourhoods;
7. hydrocarbons can be readily broken down by microbial action in bio-retention or trickling filter systems (Pratt et al, 1998);
8. copper is readily accumulated by vegetated (filter) systems, particularly within the root zone;
9. concentrations of copper, chromium and nickel found in typical (new) residential neighbourhoods are acceptably close to background levels observed in forest catchments.

It follows that the mix of pollutants likely to pass downstream of a (new) residential street ‘filter strip’ swale via its underdrain (pipeline) may include TSS at very low concentrations together with soluble components of zinc, cadmium, chromium, nickel, phosphorus and nitrogen at low concentrations or concentrations acceptably close to those of background (forest) environments. Such an outcome – for the particular case of (new) residential street environments – can be claimed as achieving the stated objective of Category 2 systems – “...high performance in containing or removing particulate and dissolved pollutant matter” (Section 4.1).

A question remains: what characteristics of a residential street qualify it as “new” in this discussion? First: every effort should be employed in its planning to minimise surface runoff (see Figure 7.6). Second: it should be characterised by drought-tolerant (preferably native) vegetation requiring minimal, if any, fertiliser.

8 STORAGES FOR STORMWATER HARVESTING (Category 3 Systems)

8.1 INTRODUCTION

8.1.1 Current drainage practice

Discharge of roof and other surface runoff to the street drainage system is a common stormwater management practice in Australian cities. Although expedient for on-site drainage control, discharge of roof and allotment runoff directly to the street reflects a thoughtless attitude to the undesirable consequences of the practice, as reviewed in earlier chapters including :

- its significant contribution to street runoff peak flows, which increases the burden placed on drainage infrastructure;
- greatly increased **frequency** of flow for a specified flow rate : typically, development increases the frequency of ‘minor’ runoff events by at least an order of magnitude – this can markedly increase the rate of scour and erosion in receiving watercourses and degrade their ecological value;
- the greatly increased **volume** of catchment runoff, which can impact on the receiving water environment. Reduced volumes may also improve the efficiency of stormwater quality control measures, such as wetlands, due to increased retention time in such measures (or, in the case of wetlands, a smaller footprint to achieve the same level of treatment);
- the addition of pollutants such as sediment, animal faeces, fertilisers and pesticides – from domestic residences – and heavy metals, nutrients, hydrocarbons and other toxins from industrial properties;
- waste of a locally-available water resource.

The practice of diverting roof runoff directly or indirectly to the street represents, generally, poor water-sensitive urban (residential) design practice, except in the special circumstances of the **yield-maximum** strategy referred to in Section 4.2.1.

8.1.2 Chapter outline and qualification

This chapter explores, primarily, the potential of **Roof Runoff Harvesting Systems (RRHSs)** to harvest residential roof runoff, in particular, for use and thereby to minimise the waste of a potentially valuable resource which is a by-product of residential development. The term ‘rainwater tank’ is avoided as there is a growing number of alternatives to the traditional tank storages, for example enlarged (roof) gutter storages and roof runoff storage systems integrated with metal or polyethylene fencing, that also deserve consideration. In fact RRHSs should be considered as much more than a simple storage, but a ‘package’ system that matches roof-runoff harvest potential to the ‘best overall’ RRHS for the specific requirements and constraints of a given site. Rainwater tanks also play a role in stormwater quality improvement through their association with ‘first flush’ devices; furthermore, overflows provide cleansed water input to “leaky” on-site retention devices (see Section 5.1.3).

The focus of this chapter is on *small footprint* RRHSs, for example systems involving small rainwater storages located, typically, on individual allotments or integrated with a cluster of dwellings. The reason for this bias is the fact that the domestic sector makes greater demand on mains water supplies in Australian cities than all other sectors – industrial, commercial, etc. – combined. Any significant reduction in this demand can therefore be translated into major savings in the cost of water supply infrastructure as well as maintenance of these facilities.

Large (neighbourhood scale) rainwater storages providing supply to domestic consumers are considered impractical and usually unacceptable for general use in urban (residential) development, however, such units, integrated with industrial, commercial or, perhaps, educational sites, can be viable. The procedures for determining the capture/use performance of such facilities, developed later in this chapter, are equally applicable to these installations : only roof area, daily demand rate, and a handful of other parameters need be altered to produce a valid design.

The primary purpose of RRHSs is considered in this chapter to be for mains water replacement. However there is also potential for these components of urban infrastructure to co-incidentally reduce urban catchment stormwater volumes and mitigate street runoff peak flows. Investigations by Allen (1993), van der Wel (2000), Coombes et al (2002), Allen (2002), Kuczera and Coombes (2002), clearly demonstrate that RRHSs can be dimensioned to reduce peak flows resulting from ‘minor’ storm events, the magnitude of the reduction being locality dependent. Under some circumstances even quite small rainwater storages can reduce flood peaks for ARI, $Y = 5 - 10$ years flows, but they will have little or no effect in major storms such as the “50 or 100 years” events, unless used in conjunction with other flood mitigation measures such as large retention or detention storages.

Where RRHSs are **designed** to achieve street drainage peak flow mitigation as well as harvesting objectives they are more properly described as Category 4 systems (see Section 4.1). This topic is revisited in Section 8.7

8.2 ASPECTS OF RESIDENTIAL ROOF RUNOFF HARVESTING

8.2.1 Storage size – practical considerations

A common misconception is that RRHS storages must be very large in order to capture most of the roof runoff. It is true that large storages are required in areas without access to other supplies, where their role is to provide a high level of secure supply even during prolonged dry spells. However in urban areas, mains water is available as a ‘backup’ and security of supply is not a major concern.

The main consideration for RRHSs in urban areas is therefore not how large a tank must be to supply the demand 99-100% of the time, but how much roof runoff can be captured and put to beneficial use from a storage of ‘practical’ capacity. The focus is therefore on small storages of 1 to 5 kL capacity as these are suitable even for very small residential sites and should therefore be quite acceptable to the public. However, depending on the type of storage device, allotment size and individual householder preferences, storages of 5 kL to 10 kL may also be appropriate, so these are also considered.

Although storages larger than 10 kL may not be suitable for the bulk of contemporary domestic situations, even these have their place: for example, the ‘Healthy Home’ on the Gold Coast, uses a 22 kL concrete cistern installed below the low-set house.

8.2.2 Considerations for maximising roof runoff capture

In order to limit RRHS storage to the minimum whilst capturing as large an amount of roof runoff as possible, it is necessary to ensure there is as much free storage available as possible at the onset of rainfall. This can only be achieved by ensuring that water is regularly drawn from storage and that the rate at which it is abstracted is quite high, particularly during the rainfall season. Although a high rate of use might seem at odds with the water-conserving approach of WSUD there is a practical limit to the rate of domestic water use that can be predicated on factors such as household number and the purpose and pattern of water use. Failure to regularly draw water from storage will, of course, result in frequent overflows, which represent a loss of the resource through site runoff.

The two most popular uses of roof runoff in Australia are for drinking (typically, for tea- or coffee-making) and garden watering. The former provides almost negligible use (representing less than 1 percent of residential water demand) while the latter is predominantly a ‘dry season’ activity when small storages are likely to be empty. Furthermore, water use is not ‘guaranteed’ but relies on the householder’s volition to use tank water in preference to mains water. It is apparent, therefore, that neither drinking nor garden watering uses realise the full potential of this alternative supply. Despite these criticisms and recommendations that other uses be considered, it should be added that **any** use – from the water conservation and flood control points of view – is better than no use at all. Alternative methods that increase the use of roof runoff by a significant amount are discussed in the next section.

8.2.3 Appropriate uses of roof runoff

In harvesting terms, roof runoff is better used for in-house, year-round demand rather than for drinking or garden watering. The means for increasing roof runoff harvesting for in-house supply have been considered by a number of Australian researchers and at least one RRHS product designer. Four domains are recognised :

- gravity supply;
- hot water supply;
- all-house supply;
- all-house supply excluding drinking water.

Each of these is considered in the context of dual supplies, i.e. supported by mains or other reliable water supply, when the rainwater storage is empty.

Gravity supply : Roof runoff can be captured in an elevated tank (on a stand), or at roof (eaves) level by means of an enlarged gutter storage which comprises a screened box gutter that stores water at roof level for suitable gravity-fed uses such as toilet cistern supply. The advantage of such elevated storage is that it enables rainwater to be delivered under gravity (to some uses) as opposed to needing a pumped supply. Water can be supplied, for example, to lower-floor bathrooms, laundries and toilets of multi-storey units – although plumbing requirements for devices receiving a gravity fed (low pressure) water supply need attention. There were more than 100 single-storey residences in Sydney successfully using this system at the end of 2003. With enlarged gutter storage, the capacity of the gutter depends on its length and cross-sectional area, but for a typical installation will be in the order of 1 to 3 kL.

Hot water supply : Although in-house uses such as bathroom and laundry demand can make good use of roof runoff, a considerable cost disadvantage is incurred by the need to provide dual plumbing for mains and RRHS supplies. Allen (1993) suggested an alternative technique to reduce the amount of dual plumbing : plumbing roof runoff (via pumped supply) *and* mains supply to the gravity hot water service. Allen suggested that there will be an incidental advantage in that the water is ‘pasteurised’ in the hot water tank under normal operating temperatures, thereby improving the microbial quality of roof runoff supply. [Water quality is discussed in the next section].

Taking Adelaide as an example, model results have shown that rainwater tanks could provide 30 to 60 kL per year to households if they are plumbed into the hot water service, resulting in a 10 to 20 percent reduction in total domestic water use. In many other Australian (non tropical) cities the amount of water savings will be much greater than in Adelaide due to their higher rainfalls and more uniform seasonal distributions. Water use savings are discussed later in the chapter.

The Urban Water Resources Centre (University of SA), trialled the hot water service concept during 1995-96 at the ‘Intelligent Home’, an experimental house at Regent Gardens Estate, Adelaide. A 2 kL rainwater tank supplied roof runoff, via a pump, to a gravity-type hot water tank which was fitted with two float valves : one for activating the rainwater tank supply, when possible, and another, low-level float, that provided mains water (via another line) when the rainwater tank was empty.

Following the success of the trial, UWRC incorporated a variation of the design into the Concept Design for “Figtree Place”, a residential re-development in inner Newcastle (Argue, 1997). Computer modelling based on historical rainfall records suggested that a 2 kL storage per residential unit would supply nearly 50 percent of the combined hot water and toilet flushing needs for the site, based on assumed use rates for hot water and toilet flushing. At “Figtree Place”, hot water and toilet flushing are supplied from underground rainwater tanks (shared between clusters of, typically, four residential units, see Section 3.9). The tanks are topped up with mains water when the rainwater storage falls to a pre-set level, thereby avoiding the need for dual plumbing for the hot water service and/or toilets. Monitoring of “Figtree Place” by Dr. Peter Coombes, University of Newcastle, has proven the efficacy of the concept as a roof runoff harvesting technique. A 45 – 50 percent reduction in (in-house) mains water use has been achieved (Coombes et al, 1999; Coombes, 2002). Furthermore, water quality tests have proven the capability of the hot water supply concept to improve the microbial quality of roof runoff water.

All-purpose supply (including drinking supply) : RRHS storages can be plumbed directly into the household mains to provide *all* domestic uses. The main advantages of this system are :

- it maximises the capture and use of roof runoff;
- it minimises tank overflows; and
- it overcomes the difficulty and cost of dual plumbing.

Two perceived drawbacks are the risk of rainwater being back-siphoned into the public water supply system, and the potential health risks associated with using roof runoff for drinking water supply.

The potential risk of contaminating the mains supply is negated by ensuring that the installed system conforms to the regulatory requirements for rainwater tank-mains interconnections. This matter is the subject of Australian Standard AS 3900, which requires the installation of the ‘RZPD valve’ in mains/rainwater inter-connections. [This requirement was observed at “Figtree Place”.] Standards change from time-to-time and the local water supply authority should be consulted for up-to-date information on regulatory requirements and advice on mains/rainwater tank inter-connections.

The use of roof runoff for drinking and/or washing – both personal and clothes – requires a thoughtful evaluation on the part of the householder of the risks involved. These are discussed in Section 8.2.4. While there may be **potential** risks in using roof runoff for drinking and washing, the relative simplicity of connecting a RRHS – fitted with mains top-up supply – to supply the entire in-house needs is certainly attractive.

However, the possibility of contamination of the entire body of stored water as a consequence of some external cause – perhaps a dead animal on the roof or severe discolouration during leaf or blossom fall from a tree up-wind of the roof – should alert householders to the need for ‘instant’ alternative supply from mains sources.

A patented tank-mains diversion valve and associated components enabling a mains-rainwater tank inter-connection and pumped supply, which also incorporates automatic switch-over to the mains when the tank is empty or during a power failure (which would inactivate the pump) or as a consequence of contamination of the types outlined above, has recently been developed by an Australian company.

All-purpose supply (excluding drinking water) : Several alternative ‘all-purpose’ concepts that exclude the need to use roof runoff directly for drinking purposes are :

- all-purpose supply, but with bottled water used for drinking;
- all-purpose supply, but with householders storing water from the hot water supply and refrigerating it in preparation for drinking;
- mains top-up supply to the RRHS storage with a separate mains-water line to the kitchen. This approach has additional merit in being able to provide water inside the house (kitchen) even in the event of a pump malfunction, power blackout or period of rainwater tank contamination.

8.2.4 Roof runoff quality considerations

Roof runoff quality depends on many factors such as the local air environment, roof and storage tank material and whether the roof, gutter and storage tank are maintained in good condition. An excellent, detailed reference on factors that affect the quality of harvested roof runoff is the “Guidance on the Use of Rainwater Tanks” monograph by the National Environmental Health Forum (Cunliffe, 1998). The publication includes information on :

- rainwater tank microbial and chemical quality;
- Australian standards pertaining to RRHSs, including materials;
- roof and tank materials;

- installation considerations;
- procedures to improve water quality : maintenance and repair;
- components to improve water quality : inlet screens, ‘first flush’ diversion devices, filters, UV disinfection;
- chemical disinfection;
- mosquito control.

The monograph can be obtained from state and territory public health authorities and can (at the time of publication) be downloaded from some state health agency web sites. In addition to the monograph, the advice of state and local public health authorities can be sought for up-to-date information on regulatory and policy matters.

Microbiological considerations : Roof runoff that is stored in a rainwater tank or other storage device is quite likely to contain micro-organisms from one or more of a number of sources, for example :

- soil or leaf litter accumulated in roof gutters;
- faecal material deposited by birds, cats, mice, etc.

Most micro-organisms are completely harmless to humans and although there have been documented instances of gastrointestinal infections attributed to drinking rainwater, the risks appear to be extremely low. The National Environmental Health Forum monograph, which is endorsed by all Australian state and territory health authorities states : “the perception is that rainwater is safe to drink and this is probably true if it is clear, has little taste or smell and, importantly, that the source of water is from a well maintained tank and roof catchment system”.

In Adelaide, survey data show that roof runoff is the primary source of drinking water for about 28 percent of the population (Heyworth et al, 1998). This is the highest rate of use of any Australian state or territory capital and suggests that almost 300,000 persons in Adelaide use rainwater as the primary drinking supply. Most of these people prefer the taste of roof runoff water to the filtered mains water which is sourced from local catchments and from the River Murray.

A recent study on the health of South Australian children (Adelaide and rural communities) who drink raintank water, compared to those who drink mains water, found no greater incidence of illness among the children who drank tank water. In fact the study data suggest a slightly higher incidence of gastrointestinal illness among children exposed, exclusively, to mains water (Heyworth et al, 1999; Heyworth, 2001).

Chemical considerations : The National Environmental Health Forum monograph (Cunliffe,1998) provides information on chemical risks, including those associated with various roof materials, paints and coatings, industrial pollution and wood-burning stoves.

In general, the risks are extremely low even in urban areas. Nevertheless there can be exceptions : in Port Pirie, South Australia, where there is a long history of lead smelting with associated airborne pollution, lead levels in rainwater tanks have been detected at concentrations exceeding drinking water guidelines (Fuller et al, 1981; Body, 1986).

While state health authorities do not advocate the use of roof runoff for drinking where mains water is available, many provide information on ways to reduce the risks associated with drinking rainwater : for example, recommending that ‘first flush’ diversion devices be installed and emphasising good maintenance practices. Authorities may also provide warnings in relation to ‘at-risk’ persons such as the infirm or immuno-compromised persons. Boiling water before ingestion, of course, achieves disinfection, but over-night storage of water taken from the hot water system is a simple and energy-saving way to achieve the same objective.

It is **strongly** recommended that the National Environmental Health Forum monograph, and other information obtainable from state and territory public health authorities and local council authorities, be consulted before embarking on a proposal to collect roof runoff for drinking.

Health warning for underground tanks : There are documented cases where rainwater tanks installed below ground level have been contaminated by surface runoff. There is also the potential for accidental cross connection with other sources of water – even domestic wastewater or (in unsewered areas) septic tank water. In one instance 89 people supplied with drinking water from an underground tank became ill as a result of the tank being contaminated by overflow from a septic tank (Lester, 1992).

Monitoring at “Figtree Place” initially indicated microbial contamination of the subsurface rainwater tanks. This was caused by poor installation (sealing) of the tanks that enabled surface runoff and soil to enter the rainwater tanks. (Soil naturally contains some types of microbial organisms.) Fortunately, ‘pasteurising’ by the hot water services improved the quality of the water to a high standard (Coombes et al, 1999).

Surface runoff may potentially also contain traces of pesticides, fertilisers or other chemicals. Devices such as UV units and carbon filters may assist in reducing the health risk, but do not guarantee that water in subsurface tanks will be free of contamination.

It is **strongly** recommended that all storages and downpipes to storage devices be entirely above ground level, particularly where the water is used for drinking. If this is not possible, then all components should be subject to close and continued scrutiny. Advice on the need or otherwise for water quality monitoring should be sought from the relevant state health authority.

8.2.5 Practical considerations for RRHSs

General

There are many issues that require consideration when it comes to designing, operating and maintaining a RRHS :

- For new housing, ensure that all roof areas intended to drain to the storage device actually do so! Gutter support structures must be correctly designed and installed to ensure this occurs.
- It is often difficult to drain runoff from a large roof to a single storage device. In fact, because a single drainage point is often insufficient to prevent gutters from overflowing, it is common practice to drain roof runoff to two, three or more points around the roof perimeter. The use of a deep box gutter, previously described, avoids these problems and may allow excess roof runoff (exceeding the box gutter capacity) to drain to a single tank storage point by way of a downpipe to a conventional rainwater tank.
- Regularly check that all RRHS components operate satisfactorily;
- Where space is limited, alternative innovative storage techniques can be considered (separately or in combination) including :
 - ▷ modular ‘slim line’ rainwater tanks;
 - ▷ box gutter storage;
 - ▷ rainwater fences : a patented ‘ultra slim’ modular storage device that doubles as a fence. Modules can be connected to increase storage capacity (and length of fence).

Pressure supply system

- The possibility of pump noise should be considered when selecting and locating a pressure pump.
- Pump reliability : poorer quality pumps can be relatively cheap but may break down more frequently. In the longer term these may be less cost effective.
- It is recommended that the Master Plumbers Association be consulted in relation to aspects and advice on pressure pump supply systems.
- It may be prudent to use a plumber’s experience in installing pressure pump supply systems : contact the Master Plumbers Association for advice.

8.3 AVERAGE ANNUAL YIELD : BACKGROUND DISCUSSION FOR PROCEDURE 9

8.3.1 Approach

A daily water balance procedure which calculates daily inflows, outflows and changes in the volume of water in a RRHS storage device over a period of years, is the basis of estimating the average annual harvest potential (yield) from a RRHS.

Procedures based on time increments longer than one day are generally less suitable in that they provide a poorer estimate of the annual yield. Time periods of less than one day are unnecessarily complex for basic procedures and unlikely to provide a more accurate estimate of the average annual yield than a daily-based model. A comparison between rainwater tank yields determined by a daily rainfall model, and another using pluviometer data, was undertaken by Allen (1994). The latter approach involved modelling very short time steps (a few minutes) which, theoretically, should provide a more accurate estimate of annual yield than the daily model. However the yield estimates obtained from the models were almost identical (within 1 to 2 percent) indicating that daily rainfall is appropriate for estimating average annual yields.

This finding has been confirmed in more recent studies by UWRC researchers (Barton et al, 2003). There are also several disadvantages with using pluviograph data which relate to record accuracy and the relatively few stations with long-term pluviograph data compared to those providing daily data (600 stations compared with over 7,000 in Australia). However, use of the DRIP program (Henneker et al, 2001) has widened the opportunities for designing rainwater tank storages using time steps of short duration [6-minutes, 15-minutes, etc. (see Section 3.4.3)].

8.3.2 RRHS average annual yield model

Allen and Argue (1998) describe daily water balance procedures for two situations :

- RRHSs that do not incorporate a ‘low-level’ mains top-up supply to the storage; and
- RRHSs incorporating a low-level mains top-up supply, for example via a ‘low level’ float-activated valve that supplies mains water to a rainwater tank if the tank water level falls to a predetermined minimum.

Although these procedures differ in some respects, comparisons have shown that the difference in average annual yields calculated by the above procedures are very small (within several percent), which is well within the bounds of other uncertainties (such as the assumed average daily demand, discussed later). Consequently only water balance procedures for RRHSs **without** mains top-up supply are described in this Handbook. Further information on the procedures for RRHSs incorporating a low-level mains top-up supply, can be found in Allen and Argue (1998).

A simple daily RRHS model leading to Procedure 9 is based on a RRHS without mains top-up supply to the storage tank (see Section 8.4). Although the model is self-explanatory, it is strongly recommended that the following sections be read prior to using the model. [Software enabling use of Procedure 9 can be downloaded from the UniSA ‘water’ website: www.unisa.edu.au/water/UWRG/publication/ and follow the prompts.]

8.3.3 Model requirements

The information required to estimate the average annual yield is :

- historical daily rainfall record;
- effective roof area connected to the RRHS storage;
- RRHS storage capacity;
- RRHS assumed initial storage on the first day of the record;
- required daily demand.

Daily Rainfall Data

For urban areas, local historical daily rainfall data should be used. If these are not available, data from a nearby representative station with a similar seasonal and annual rainfall should be used. If unsure what data are available or appropriate, the Bureau of Meteorology should be contacted for advice. The data record should cover as long a period as possible – at least 10 years is recommended and preferably 20 years or more. If a requirement is also to calculate other, possibly useful information such as the ‘once in 50 years’ lowest annual yield, then a longer period of record will be needed. Long records of historical daily rainfalls are available for over 7,000 Australian locations and can be obtained, usually for a small fee, from Bureau of Meteorology offices. CSV or XLS format should be requested for the model accompanying this Handbook. The daily rainfall data should be requested from the Bureau of Meteorology in column format (which is standard) and should **include** days of no rainfall.

A check is recommended to ensure there are no gross errors in the data set. Daily rainfalls for each year can be summed for comparison with monthly or yearly totals, also obtainable from meteorological offices.

Often, daily rainfalls are not recorded on weekends (or public holidays), although the sum for the previous few days is usually recorded at 9.00 a.m. each Monday for the preceding three days from 9.00 a.m. Friday. This has minimal effect on the calculated average annual yield and if anything, provides a marginally conservative yield estimate. Occasionally there may be large gaps in a rainfall record due to loss of original records, temporary closure of the station, or other reasons. Where significant gaps occur, it is necessary to ascertain if it is due to a prolonged period without rain, or due to lost or incomplete data. If significant hiatus occur, consideration should be given to replacing the missing record with data from a suitable nearby station for the same period, or to excluding the ‘broken’ year’s rainfall altogether.

Effective Roof Area

The roof area connected to a RRHS storage is an important parameter affecting supply. As a rule, the connected area should be as large as practical to increase the likely capture. Other roofs including verandahs, garages, carports or sheds can be included in the total area also connected to the storage tank. It is not always feasible to connect the entire roof to a single rainwater storage unless it is below ground level – however, note the warning in Section 8.2.4 in relation to sub-surface storages. A better way to increase roof runoff capture may be to use an enlarged eave-gutter, described previously, the overflow from which is more easily connected to a single above-ground storage.

Not all rainfall striking the roof surface contributes to storage tank inflow : losses occur as a result of some uptake by semi-porous roof materials, sun and wind-induced evaporation, ponding in roof gutters, gutter overflows and gutter leaks. Even where impervious roof materials such as colour-bonded steel, galvanised iron or glazed tiles are used, the effective (connected) roof area may be only 80 to 90 percent of the nominal roof area. In determining harvesting potential for RRHSs, the National Environmental Health Forum monograph (Cunliffe, 1998) assumes a capture efficiency of 80 to 85 percent of the connected roof area.

Storage Capacity and Initial Storage

Initial storage refers to the assumed volume of water in storage at the beginning of the first day of the rainfall record being modelled. For small storages the assumption as to whether the storage is full, or empty, or between these extremes, has no significant effect on the estimated average annual yield performance. However where a model is used to assess the harvesting performance of extremely large storages the initial storage assumption may produce misleading estimates. For this reason, it is recommended that the initial stored volume should be assumed as :

- where the date of the first day of record occurs in the dry season : zero;
- where the date of the first day of record occurs in the rainy season : the full storage capacity, but not exceeding a maximum of 10 kL.

In locations which are not characterised by distinct “dry” and “wet” seasons, the initial storage should be set at half full.

Daily Demand – In-house

Daily demand (the rate of water use) can have a very significant influence on the yield. On a daily basis some types of in-house uses exhibit a high degree of variation, for example clothes washing, which might be undertaken on one or two days per week. However, in-house demand (for a given household) is generally fairly constant seasonally or annually so that the assumption of a constant daily demand is generally acceptable. Where data exist which indicate a significant seasonal influence in in-house water use – for example, higher use during summer – this should be accommodated in modelling.

The major factor influencing in-house water demand is the number of household occupants, which is often associated with dwelling type : townhouses, for example, are more likely to have an average occupancy of one to two persons, while large homes may have an average occupancy of three to four persons (two adults and one or two children) in new sub-divisions. It is therefore important to link average in-house daily water demand to the type of development or expected average number of occupants.

Unfortunately the rate of water use also varies considerably between Australian cities so that it is necessary not only to consider the average number of occupants per dwelling, but also location. Of the state and territory capitals, the highest per capita water uses occur in Darwin (very high), Hobart and Brisbane, and the lowest in Adelaide, Sydney and Melbourne. Canberra and Perth fall between these groups.

For most cities, detailed information on in-house water use patterns are not readily available. Potential sources of information are the local water authority and other state government agencies, the Australia Bureau of Statistics, university researchers and the Water Services Association of Australia (WSAA). Limited ‘indicative’ information is provided in the following sections for some state capitals.

Brisbane

Estimates of average in-house consumption by use type are summarised in Table 8.1. Unfortunately these data do not provide a breakdown for dwelling types or number of occupants. However a satisfactory estimate can be obtained by dividing the average in-house use by the average household occupancy for Brisbane to obtain per capita averages which can then be multiplied by the estimated average number of occupants for the dwelling type in question. It is suggested that the following average occupancy rates be used :

- Apartments and townhouses of one or two bedrooms : 2 persons. [Although apartments and townhouses may have an average occupancy of less than two persons, some evidence suggests that a higher per capita rate of in-house water use occurs in these developments. The assumption of two person occupancy is therefore recommended].
- Traditional large allotment residential housing with two bedrooms : 2.5 persons.
- Traditional large allotment residential housing with three bedrooms : 3 persons.

TABLE 8.1
TYPICAL DAILY WATER DEMAND (IN-HOUSE USE), BRISBANE

IN-HOUSE USE	AVERAGE DAILY DEMAND (L/DAY)
Shower/bath	193
Hand basin	28
Toilet	186
Laundry	135
Kitchen (incl. dish-washing)	44
Total In-house	586

Source : Urban Water Research Association of Australia (Jeppeson, 1996); data precede significant uptake of dual-flush toilet cisterns. Toilet flushing rates should be reduced by a factor of two where 6/3 litre dual flush toilets are installed – i.e. to 93 L/day.

Perth

An extensive 1986 survey by the Water Authority of Western Australia (WAWA, 1987) provided in-house water demand for Perth.

Table 8.2 shows the WAWA-derived relationships for in-house water demand for specified uses and numbers of occupants. The WAWA data were obtained by in-house metering, validated by homeowner surveys; consequently the data reflect in-house water use and in-house losses.

TABLE 8.2
DAILY WATER DEMAND AND NUMBER OF OCCUPANTS, PERTH

IN-HOUSE USE	QUANTITY FOR FIRST HOUSEHOLD OCCUPANT (L/DAY)	ADDITIONAL QUANTITY PER EXTRA PERSON (L/DAY)
Bathroom (incl. hand basin)	101.0	37.3
Toilet	81.8	31.4
Laundry	41.4	29.4
Kitchen (incl. dish-washing)	20.2	6.9
Total In-house	244.4	105.0

Source : Water Authority of Western Australia, 1987; data precede significant uptake of dual-flush toilet cisterns. Toilet flushing rates should be reduced by a factor of two where 6/3 litre dual flush toilets are installed.

Adelaide

Table 8.3 provides indicative information for in-house use for Adelaide dwelling types.

TABLE 8.3
ESTIMATED WATER USE BY DWELLING TYPE, ADELAIDE

IN-HOUSE USE	TOWNHOUSES	SMALL 'VILLA' HOMES	MODERATE SIZE NEW DWELLINGS	OLDER, 'THREE-BEDROOM' DWELLINGS
	LITRES PER DAY			
Bathroom (incl. hand basin)	110	110	220	180
Toilet	35	35	70	70
Laundry	65	65	130	100
Kitchen (incl. dish-washing)	40	40	50	50
Total In-house	250	250	470	400
Outdoor water use, L/day	70	160	400	600

Source : Adapted from Allen, 1993. Assumes low flow dual flush WCs for the first three dwelling categories and partial uptake of dual flush WCs for the fourth dwelling category. For the fourth category, reduce WC use to 50 L/day if 6/3 litre dual flush WCs are installed.

A more recent study of Adelaide domestic consumption by Barton (2003) provides general support for the values presented in Table 8.3.

Other State and Territory Capitals

In the absence of appropriate local information, the following 'indicative' in-house water use estimates will provide satisfactory RRHS yield estimates :

- Darwin : 1.5 times the Brisbane rate
- Sydney : 1.3 times the Adelaide rate
- Canberra : 1.3 times the Adelaide rate
- Melbourne : 1.2 times the Adelaide rate
- Hobart : 1.2 times the Adelaide rate.

For other locations the local water authority should be contacted for advice.

Daily Demand – influence of outdoor use

Outdoor demand is the most difficult component of all to estimate as it will vary considerably from house to house and year to year, as well as between geographic locations. Factors that influence garden watering include the type of vegetation (native or exotic), number of hot days (particularly days over 30°C), irrigated area, watering method (hand, sprinkler, dripper, automatic irrigation, etc.), socio-economic factors, mains water price, whether dwellings are owned or rented, and the householder's level of interest in gardening. In some areas, notably Perth, bore water is extensively used; there is likely to be much greater use of water where there is access to (free) bore water.

For many Australian cities the wet and dry weather periods are relatively distinct, particularly for cities such as Darwin and Brisbane. Because garden watering is largely a mid-dry season activity when rainfall is lowest, garden watering is less likely to influence RRHS yield (for small storages) than significant in-house consumptive uses, particularly for dwellings with small gardens. In the absence of other information, a conservative yield estimate for RRHSs designed to supply all-household demands can therefore be obtained by a number of methods such as :

- Assume no summer irrigation use : that is, modelling the in-house water use **only**. This is a simple approach which should give an accurate estimate of yield for locations where there is a pronounced wet-dry season, garden irrigated areas are quite small, and the RRHS storage is small, e.g. 1 to 2 kL capacity. The approach will provide a conservative estimate of yield for **any** situation where the rainwater storage is intended to supply water for all household demands including outdoor use.

OR

- Assume an average irrigation rate of 4 mm per day (4 litres/m²/day) per square metre of irrigated, **exotic** garden area (lawn plus garden beds) during the driest three months. This will enable garden use to be estimated for either a known or assumed irrigated garden (exotic) area. A daily water balance model can be established that incorporates the daily irrigation water use during the irrigation season (adding this to the constant in-house demand). This approach is most suited to estimating yield for sites where there is a less well-defined season, high garden water use and larger capacity storages.

OR

- Assume an average irrigation rate of 1 mm per day (1 litre/m²/day) per square metre of irrigated, **native or drought-tolerant** garden area (lawn plus garden beds) during the driest three months. This will enable garden use to be estimated for either a known or assumed low water-use garden area. Best practice management of native, drought-tolerant or Mediterranean-style vegetation involves one or two “soakings” per month totalling 25 – 30 mm. A daily water balance model can be established as outlined above.

Daily Demand – hot water use

The amount of water supplying domestic hot water needs can vary considerably between dwellings. In the (likely) absence of data, a conservative estimate of hot water use is to assume 50 percent of bathroom water use and 20 percent of kitchen and laundry use is supplied by the hot water service. The water demand for bathroom, laundry and kitchen uses should be determined from local information, where available, or Tables 8.1 to 8.3 with respect to Brisbane, Perth and Adelaide. For other state and capital cities, the rates should be estimated using the factors suggested in the preceding discussion. Estimates based on this approach are included in Table 8.5.

Significance of the selected demand rate

While the actual daily demands may in reality be very different from the estimated demand rates discussed in the preceding sections, in most cases this is unlikely to result in significant difference between the ‘actual’ and the ‘predicted’ average annual yield, providing that harvested roof water is used for all suitable consumptive purposes during the wet season. These include hot water supply, bathroom and/or toilet use as well as laundry uses. This is because, beyond a certain rate of daily water use, the average annual yield becomes less sensitive to daily water use. This is apparent from the charts of indicative yields for Australian state and Territory capitals presented in Section 8.5, which show that beyond a certain demand rate which varies between cities, an increased demand has increasingly less effect on the yield.

8.4 AVERAGE ANNUAL YIELD : PROCEDURE 9 – SITE MODEL

A daily water balance model is easily established using spreadsheet format such as Excel with calculations undertaken by cell formulae. An example layout of the first few rows of a spreadsheet-style calculation is shown (Table 8.4). Explanation of the column requirements is provided in the notes.

Each of the rows shown in Table 8.4 represents the water balance for one day and therefore these form only the first of thousands of rows of a spreadsheet model that should use at least 10 years of local daily rainfall record. Nevertheless, other than Column B (daily rainfall data) it is only necessary to set cell formulae in the first few rows of the record and then copy the “relative cell” formulae to the other rows. [To do so requires a basic knowledge of Excel or any other suitable spreadsheet program; the software referred to above (see Section 8.3.2) has built in cell formulae.] A sum for Columns D, Column F or Column I will provide, respectively :

- a sum for the entire record of rainwater tank inflow (effective roof runoff);
- overflow volume for the entire record;
- volume of the demand met by the RRHS for the entire record, i.e. summed yield for the entire record.

The summed values can be divided by the number of years of record to obtain average annual values. If desired, totals and averages for each year, or even for each season, are relatively easily extracted. Once the basic summary data are obtained, other parameters such as percentage of the required demand met by the RRHS, or percentage of days in which the tank storage can or cannot meet the full daily demand for the modelled use (from Column J), can be easily determined.

The model that is provided with the Handbook automatically calculates parameters such as average annual yield, percentage reduction in runoff to the street drainage system from the connected roof area, and percentage of the daily demand met by the RRHS.

PROCEDURE 9 : Method for determining the average annual yield

STEP 1 : Obtain the relevant daily rainfall record and adjust, if necessary, for missing data (see Section 8.3.3). Input this into the model provided and run the model for appropriate roof sizes and storage capacities as explained in the model.

OR

STEP 2 : Establish a tabular spreadsheet with headings similar to those shown in Table 8.4 and copy/paste the daily rainfall record into Column B.

STEP 3 : Establish the appropriate formulae in the cells in the spreadsheet. The effective roof area contributing to the RRHS, storage capacity, assumed daily demand (if necessary, varied seasonally), and other parameters must be considered as these need to be input into cell formulae. [This step requires a basic knowledge of the spreadsheet package – alternatively, the disc accompanying this Handbook includes the appropriate cell formulae and parameters in the RRHS model.]

STEP 4 : Sum the results in columns D, F and I over appropriate time periods, for example, the full length of record, or for each year or season, etc.

STEP 5: Repeat Steps 3 and 4 using other effective roof areas, storage capacity, assumed daily demand etc., if necessary. It is suggested that a number of storage capacity sizes be trialled.

TABLE 8.4

SIMPLE SPREADSHEET BASED MODEL FOR ESTIMATING RRHS YIELD AND OTHER PARAMETERS OF INTEREST

(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)	(J)
DAY IN RECORD	RAIN- FALL	INITIAL	INFLOW	SUM	OVER- FLOW	VOLUME	VOLUME	DEMAND	FLAG
1	0	2200	0	2200	0	2200	1960	240	1
2	0	1960	0	1960	0	1960	1720	240	1
3	0	1720	0	1720	0	1720	1480	240	1
4	2.8	1480	378	1858	0	1858	1618	240	1
5	51.8	1618	6993	8611	5611	3000	2760	240	1
etc.									

Column A : Day number in the record (1 = first day in the record).

Column B : Daily rainfall, in mm (preferably, the long-term historical record for the location).

Column C : Volume in rain tank (litres) at beginning of that day. Other than the first day of record (day 1) this value is the volume remaining in the rain tank at the end of the previous day (i.e. Column E minus Columns F and I, from the previous day). For the assumed volume for the first day of record, refer Section 8.3.2.

Column D : Tank inflow (litres) is the volume of rainfall (mm) from Column B, multiplied by the effective connected roof area (m²).

Column E : Sum (litres) of the initial tank storage volume (Column C) and day's inflow (Column D). Note this may exceed the tank capacity.

Column F : The tank overflow (litres). If the value in Column E is greater than the tank size, the overflow is the value of Column E minus the tank volume expressed in litres. Otherwise the value is 0 (no overflow).

Column G : Volume after overflow (litres). If no overflow occurred on this day (Column F value is zero), this value equals the value of Column E. If overflow did occur on this day (Column F value is greater than zero), then Column G value is the tank capacity.

Column H : Value (in litres) of Column G minus the daily demand (in litres). The value of Column H may be negative.

Column I : Volume of demand (litres) actually met by the tank during that day. If the volume in Column H is negative, the value of Column 9 is the value of Column G. If the value of Column H is greater or equal to zero, the value of Column I is the required daily demand.

Column J : Various "flags" can be devised in order to determine other parameters, if so desired. The flag shown in Column J, for example, may be set at a value of "1" if the full daily demand was met by the tank (i.e. value in Column 9 is the required daily demand), or a value of "0" if the full daily demand was not met by the tank. Analysis of these values for the entire record will indicate how many days the tank was able to supply the full daily demand (for the modelled type of demand).

8.5 INDICATIVE YIELDS FOR STATE AND TERRITORY CAPITALS

The charts presented in Figures 8.1 to 8.4 provide ‘first estimates’ of the long-term average annual yields of RRHSs for state and territory capitals. They should be considered as providing indicative estimates only of yield prior to obtaining location-specific daily rainfall data for modelling. The charts were developed using data obtained from a more sophisticated, but essentially equivalent approach to that outlined in Table 8.4. For each city the rainfall stations from which data were taken to develop the charts had greater than 50 years available data. It was assumed that the effective connected roof area (see Section 8.3.3) is 90 percent of the roof area connected to storage, i.e. for the first chart, Adelaide 50 m², the effective roof area is 45 m². Of particular interest are :

- the efficacy of small storages which, in many situations, will provide the same or almost the same yield as larger storages. This has significance for the practicality and cost-effectiveness of RRHSs;
- there is often a relatively pronounced ‘flattening’ of the curves for higher demands (typically, those exceeding about 300 L/day for most roof areas).

Demand rates should preferably be based on local data (if available) from the water supply authority, other sources, or estimated for the site in question. In the absence of such information Table 8.5 provides indicative average daily demands for in-house and hot water use for state and territory capitals, determined from Tables 8.1 to 8.3 and related discussion. The values in Table 8.5 can be used with the charts on the following pages to determine indicative average annual yields for various locations, storage sizes and connected roof areas.

TABLE 8.5

INDICATIVE DEMAND FOR VARIOUS DWELLING TYPES, STATE AND TERRITORY CAPITALS

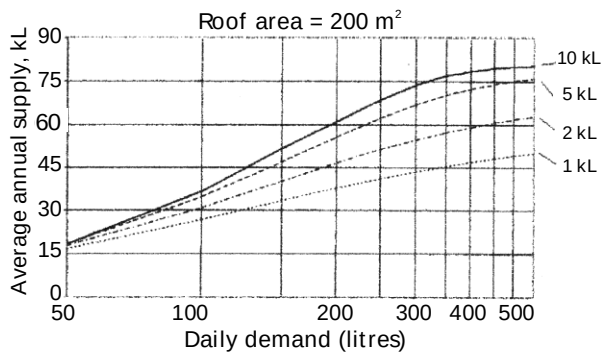
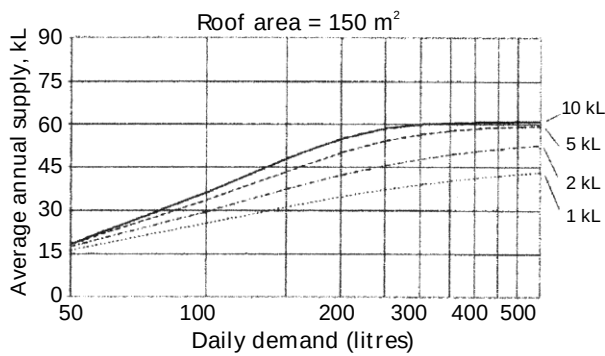
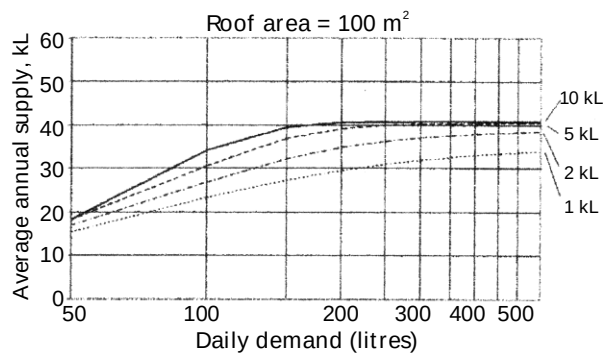
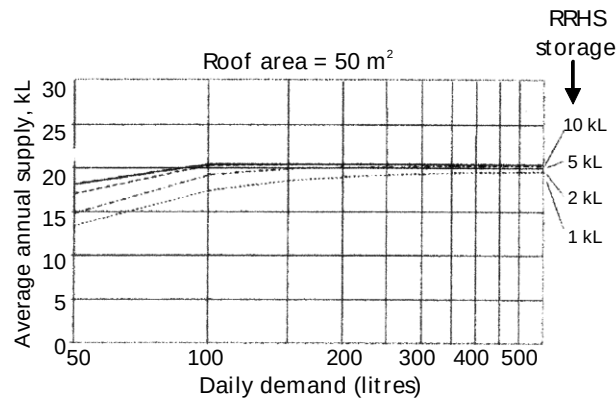
CITY	ALL IN-DOOR DEMANDS, L/DAY			HOT WATER ONLY DEMAND, L/DAY		
	TOWNHOUSE AND ‘VILLA’	MODERN DETACHED HOUSE	OLDER DETACHED HOUSE	TOWNHOUSE AND ‘VILLA’	MODERN DETACHED HOUSE	OLDER DETACHED HOUSE
Adelaide	250	470	400	75	145	120
Brisbane	420	835	630	105	210	155
Canberra	325	610	520	95	190	155
Darwin	630	1255	940	155	315	235
Hobart	300	565	480	90	175	145
Melbourne	300	565	480	90	175	145
Perth	350	560	455	90	140	115
Sydney	325	610	520	95	190	155

Note : Modern detached refers to new detached houses with an average occupancy of approximately four persons (two adults, two children); older detached houses have an assumed average occupancy of three persons.

Note that the chart for Darwin has a greater range of demand than the other cities due to the particularly high in-house water use noted for that city.

ADELAIDE

(Average annual rainfall : 545mm)



BRISBANE

(Average annual rainfall : 1128 mm)

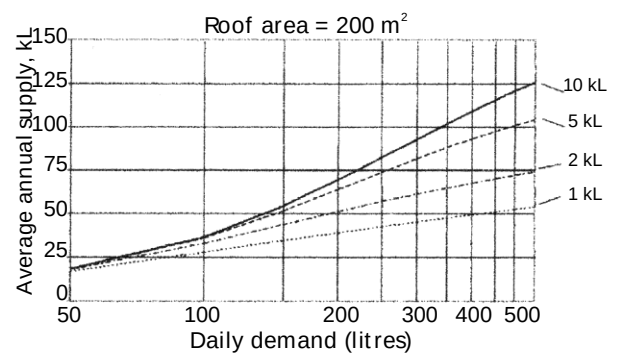
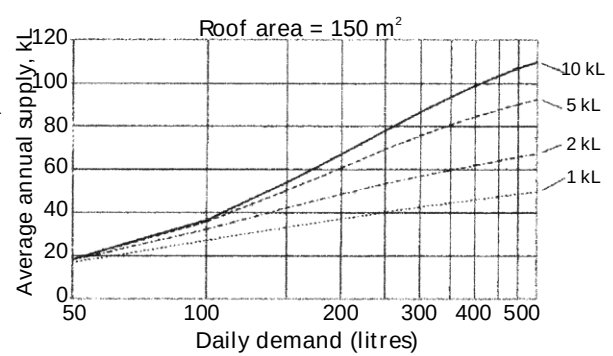
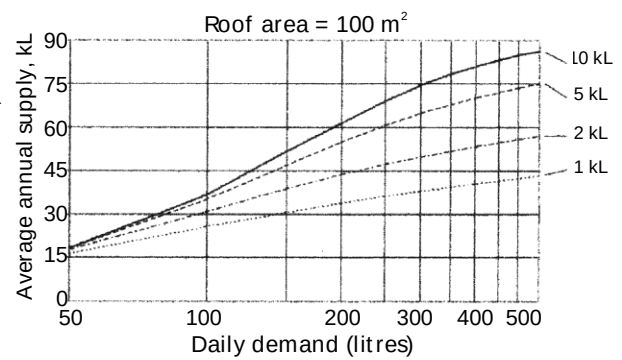
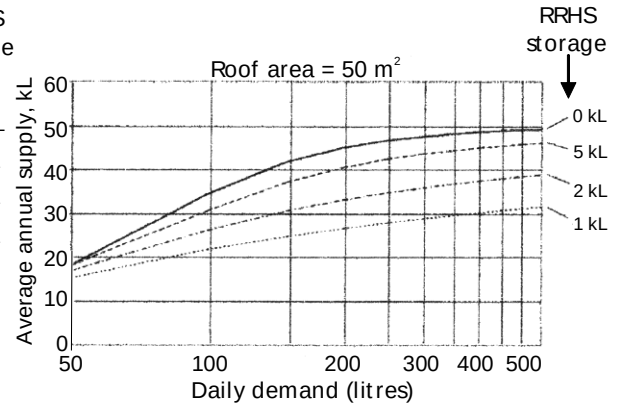


FIGURE 8.1 : Indicative Supply (Yield) from Roof Runoff Harvesting : State and Territory Capitals

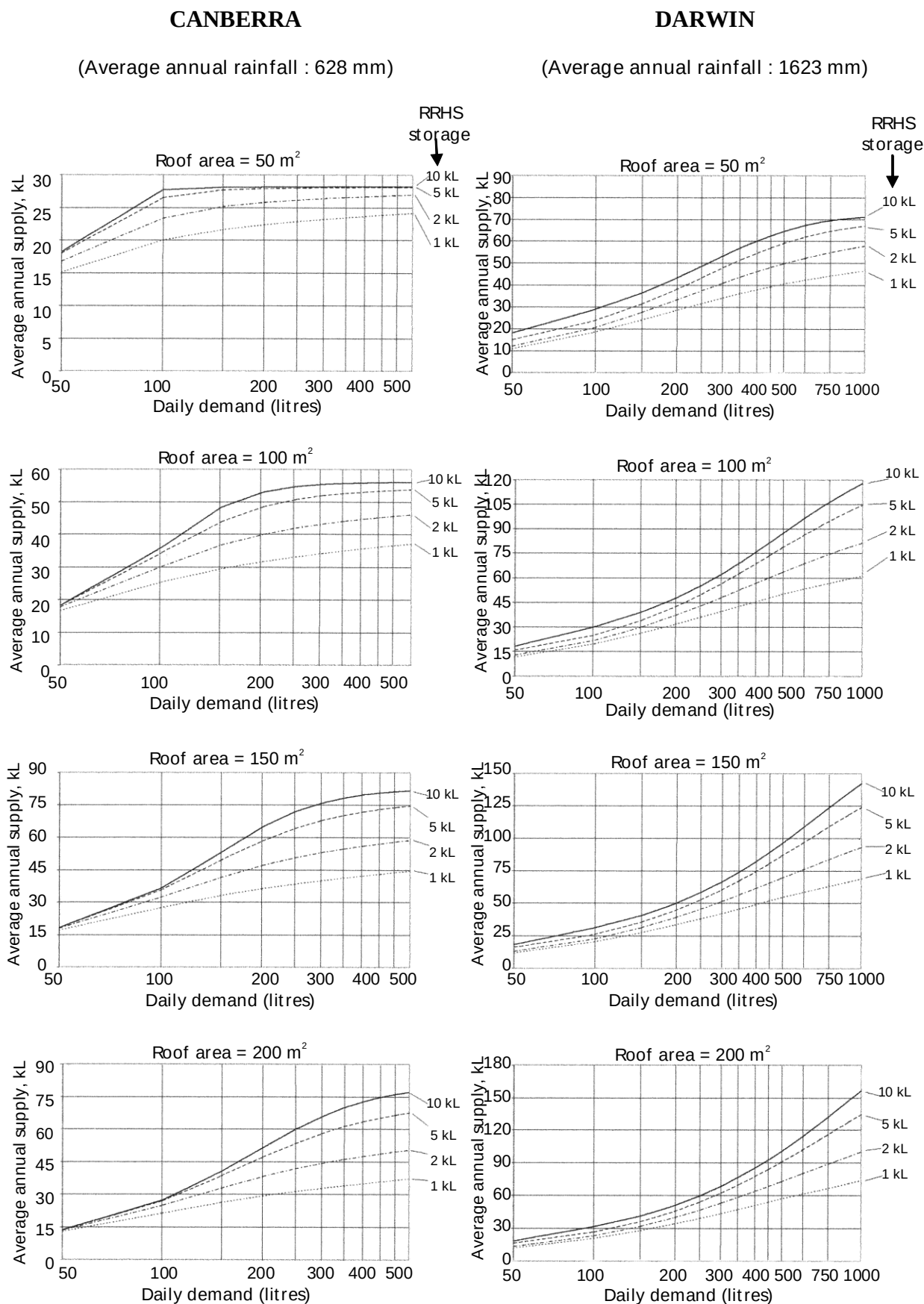


**FIGURE 8.2 : Indicative Supply (Yield) from Roof Runoff Harvesting :
State and Territory Capitals**

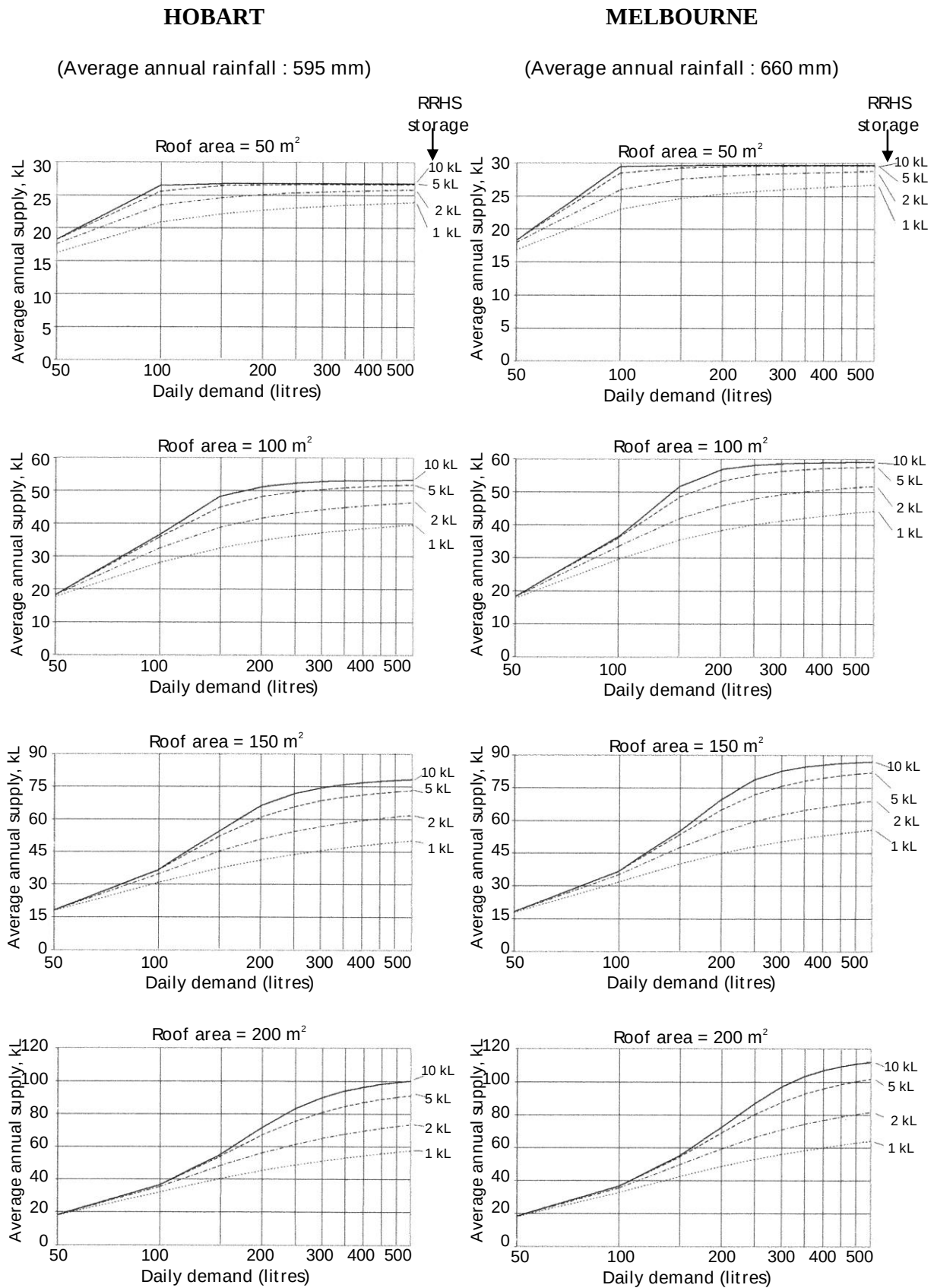
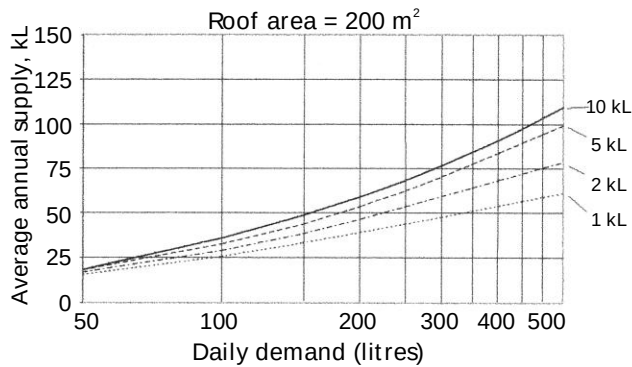
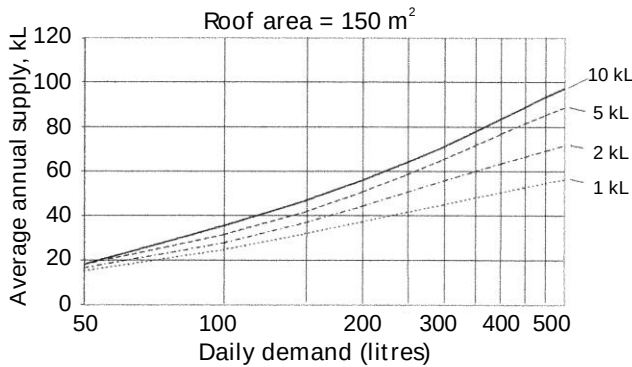
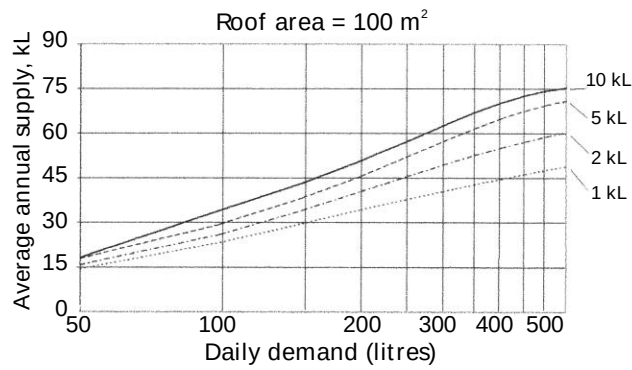
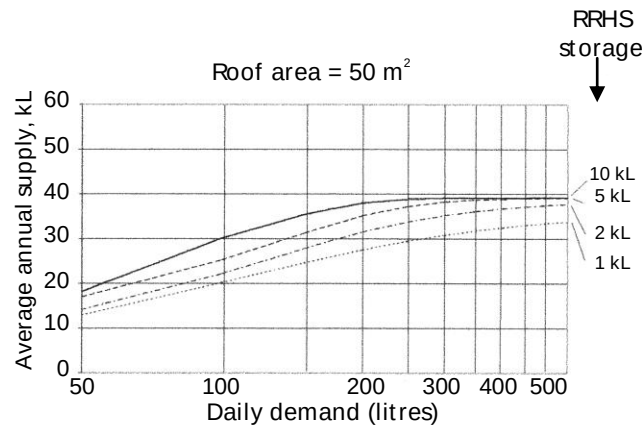


**FIGURE 8.3 : Indicative Supply (Yield) from Roof Runoff Harvesting :
State and Territory Capitals**

PERTH

(Average annual rainfall : 870 mm)



SYDNEY

(Average annual rainfall : 1240 mm)

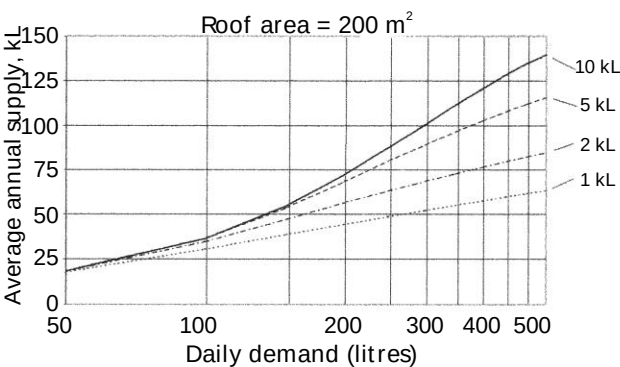
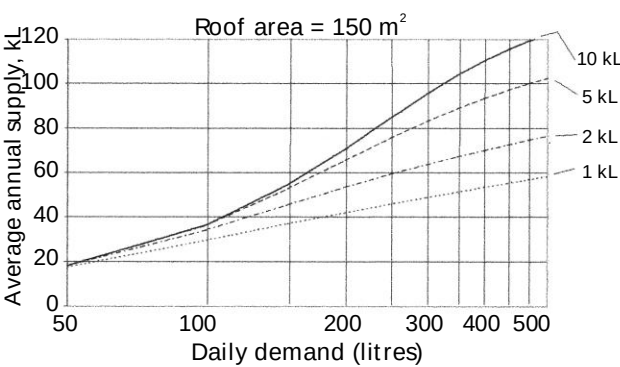
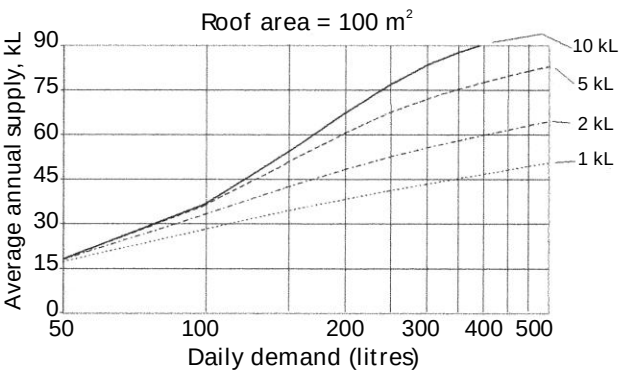
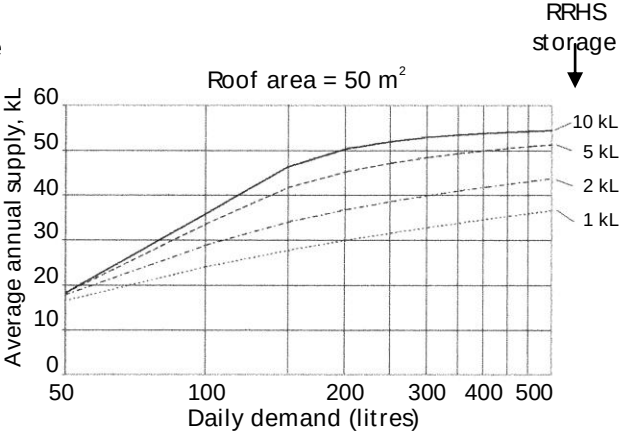


FIGURE 8.4 : Indicative Supply (Yield) from Roof Runoff Harvesting : State and Territory Capitals

8.6 A REGIONAL STORMWATER HARVESTING MODEL : PROCEDURE 10

8.6.1 Introduction

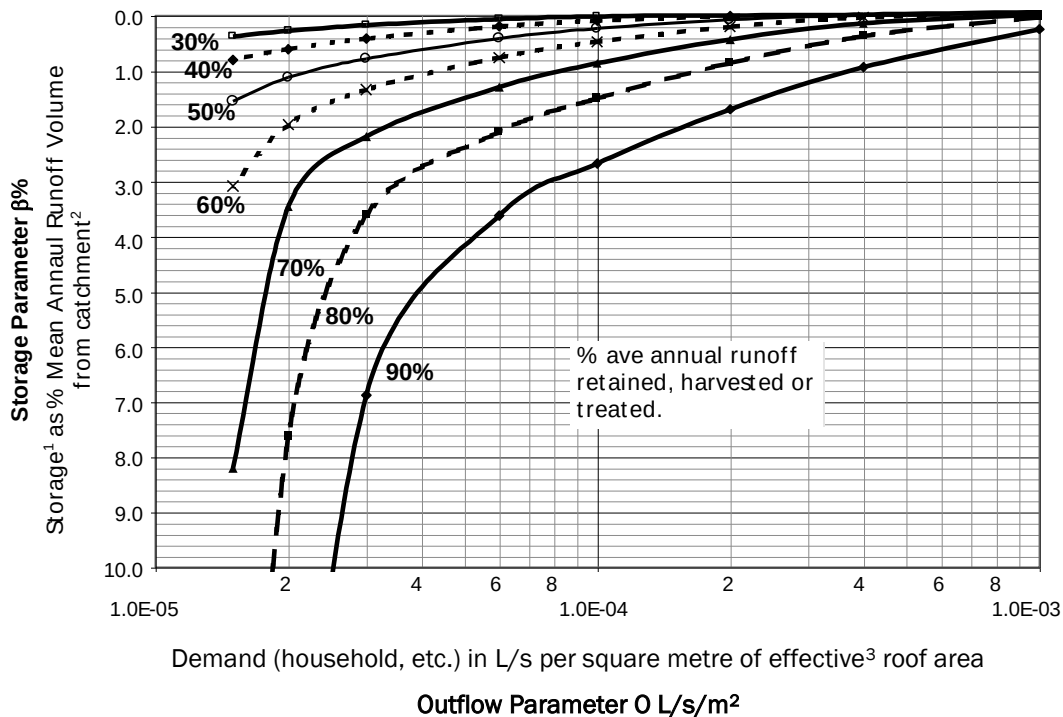
An alternative procedure to that reviewed above for fixing the sizes of rainwater and other storages can be developed from ‘continuous simulation’ modelling of the type introduced in Section 3.4.2 : results are presented in Figure 3.7, repeated here as Figure 8.5 (Southern Australia) for convenience. The process which lies behind the preparation of Figure 8.5 is explained in Table 3.2 and, for this reason, is not repeated here.

The design method associated with the use of this information is Procedure 10. It enables storage sizes for various harvesting devices and installations to be fixed **without** recourse to the continuous daily rainfall record for a particular location of interest which characterises Procedure 9. Procedure 10 provides general outcomes which are applicable in each of the climate zones identified in Figure 3.8. A graph for each zone is presented in Figure 8.5 or in Appendix E. Design by Procedure 10 is therefore simpler than by Procedure 9 but, because of its regional generality, is not as accurate.

8.6.2 Figure 8.5 explained

The independent variable (X axis) of Figure 8.5 represents **outflow, O**, from the storage device. This is (household or industrial) average daily demand – as reviewed in Section 8.3.3, above, expressed as “L/s/m² of **effective** roof area (EIA)”. *Effective* roof area is the actual catchment yielding runoff directly to the rainwater tank.

In the case of a storage pond, to which Procedure 10 can also be applied, **outflow, O**, is the sum of the three parameters – average daily extraction (for use), daily seepage quantity and daily average evaporation. The first of these parameters may be known; the second and third components must be estimated. This ‘sum’ must then be expressed as “L/s/m² of catchment area (EIA)” draining to the storage facility.



NOTES :

- 1 : **Device** may be a pond or a rainwater tank.
- 2 : **Catchment** may be a ground-level paved area or a roof.
- 3 : Effective roof area for rainwater tanks may be as low as 80% of nominal roof area (see Section 8.3.3).

FIGURE 8.5 : General stormwater harvesting design graph – Southern Australia

The dependent variable (Y axis) gives the **storage, β** , expressed in terms of *percentage of mean annual runoff volume* discharged from the catchment (EIA). This is, simply, mean annual rainfall, with an appropriate runoff coefficient applied typically 0.75, $\times$ catchment area. The factor 0.75, and not 0.90 or 0.95, is suggested because the great bulk of annual yield from a catchment results from rainstorms of small magnitude which show relatively high losses for individual events.

All of the graphs associated with Procedure 10 (Figures 8.5 and E-1 to E-4, Appendix E) have been derived for small catchment area cases – maximum 2.0 ha. They are therefore satisfactory for residential or industrial rainwater tank sizing, but are limited in their application to pond storages providing significant use/demand : they are appropriate for dimensioning ornamental ponds, for example.

The family of curves gives average annual retention (stormwater harvesting) at seven levels between 30% and 90% inclusive, and shows that for any given percentage of runoff harvesting, the size of storage required to deliver a fixed level of demand **decreases** as the demand itself **increases**. For some, this may be a counter-intuitive outcome.

8.6.3 Procedure 10

Preliminary considerations – location, catchment, etc.

The procedure must commence with a clear description of the circumstances of the particular application.

This involves defining :

- The **climate zone** in which the catchment is located.
- **Average annual rainfall** for the site of the installation.
- **Components** of the catchment draining to the installation. This list must identify, in particular, roof areas, ground-level areas and whether they are impervious, porous or permeable, or pervious, etc. This leads to A_{EIA} .
- **Outflow** from the device or installation (see above) converted to $L/s/m^2$ of equivalent impervious area, A_{EIA} .

PROCEDURE 10 – STEP 1 :

Determine equivalent impervious area of the catchment contributing runoff to the device (rainwater tank, storage pond, etc.). These data lead to determination of catchment area, A_{EIA} .

STEP 2 :

Identify climate zone of site based on $i_{10,1}$ rainfall intensity, and select appropriate design graph from Figure 8.5 (Southern Australia) or from Appendix E (Figures E-1 to E-4).

Identify **outflow, O** , as $L/s/m^2$ of equivalent impervious area, A_{EIA} .

Nominate a **target percentage** of (average) annual runoff to be harvested: available values – 30%, 40%, 50%, 60%, 70%, 80% and 90%.

Locate ‘point’ in selected design graph corresponding to the specified value of outflow (X-axis) and target harvesting percentage and identify corresponding **storage, β** , within the range 0 to 10%.

STEP 3 :

Calculate the average annual runoff (volume) from the contributing catchment. This is :

$$\nabla_A = (\text{average annual rainfall in metres}) \times 0.75 \times A_{EIA} \text{ m}^3 \quad (8.1)$$

Required storage volume is given by $\nabla_h = \beta \times \nabla_A \text{ m}^3 \quad (8.2)$

STEP 4 : Review the design volume, V_h

In the case of a rainwater tank, the volume, V_h , must be interpreted into a plan area and height, either of which may be considered unacceptable in terms of space available or appearance in certain circumstances leading to consideration of further (volume) options.

In the case of a pond intended for the dual purposes of providing water for daily use as well as amenity, a necessary compromise between acceptable depth (safety issues) and available space (plan area) may have to be reached requiring the exploration, also, of other volume options. This task is made difficult because evaporation from the pond cannot be satisfactorily estimated until its (final) plan area is known : only an approximation for this component of outflow can therefore be substituted into STEP 2.

Such situations involve returning to STEP 2 and repeating the design process using a different value of **target percentage** (harvesting) to arrive at a more acceptable outcome. This process is iterative and continues until such time as a satisfactory final solution or compromise is reached in each case.

This process of “... iteration and ... compromise ...” has been developed into a software package which is reviewed, briefly, in Section 8.6.5.

This terminates Procedure 10.

8.6.4 Illustrative examples**Example 8.1**

The storage volume of a rainwater tank is required for a residence in Brisbane ($i_{10,1} = 71$ mm/h) with effective roof area draining to the tank equal to 150 m^2 . The average annual rainfall at the Brisbane site is 1164 mm. Household estimated average daily water use from the rainwater tank is 350 litres.

Preliminary considerations

- The catchment is located in Brisbane – Northern Australia climate zone.
- Average annual rainfall is 1.164 m.
- Catchment area – *effective* roof area - $A_{EIA} = 150 \text{ m}^2$.
- Outflow based on household average daily water use (350 L/day) and roof area $A_{EIA} = 150 \text{ m}^2$ leads to outflow, $O = 2.70 \times 10^{-5} \text{ L/s/m}^2$.

PROCEDURE 10 – STEP 1 :

Catchment area, $A_{EIA} = 150 \text{ m}^2$.

STEP 2 :

The site location is Brisbane : climate zone – Northern Australia, design graph E-4.

Outflow, $O = 2.70 \times 10^{-5} \text{ L/s/m}^2$ of equivalent impervious area, A_{EIA} .

Target percentage of (average) annual runoff to be harvested : Trial 1, use 30%. Note that in the case of Northern Australia, it is not possible to harvest high percentages of annual (roof) runoff unless the daily quantity used is, similarly, large. In the present case – with only 350 L/s/m^2 of A_{EIA} being used – it is only possible to harvest around 30% of average annual runoff.

This gives (Figure E-4, Appendix E) **storage, $\beta = 3.6\%$** .

STEP 3 :

Average annual runoff (volume) from the contributing catchment is :

$$V_A = (1.164) \times 0.75 \times 150 \text{ m}^3 = 131 \text{ m}^3 \quad (8.1)$$

$$\text{Required storage volume is } V_h = 0.036 \times 131 \text{ m}^3 = 4.7 \text{ m}^3 \quad (8.2)$$

STEP 4 : Review the design volume, V_h

Rainwater tank volume 4.7 kL is acceptable.

This terminates Procedure 10.

Example 8.2

A storage gutter system (“Rainsaver” see Section 2.2) is required for a residence in Sydney ($i_{10,1} = 60$ mm/h) with effective roof area draining to the device equal to 200 m^2 ; the length of perimeter eave-guttering is 70 m. The average annual rainfall at the Sydney site is 1,226 mm. Household estimated average daily water use from the storage guttering is 350 litres.

Preliminary considerations

- The catchment is located in Sydney – Upper-Intermediate climate zone.
- Average annual rainfall is 1,226 mm.
- Catchment area – roof area – $A_{EIA} = 200 \text{ m}^2$.
- Outflow based on household average daily water use (350 L/day) and roof area $A_{EIA} = 200 \text{ m}^2$ leads to outflow, $O = 2.03 \times 10^{-5} \text{ L/s/m}^2$.

PROCEDURE 10 – STEP 1 :

Catchment area, $A_{EIA} = 200 \text{ m}^2$.

STEP 2 :

The site location is Sydney : climate zone – Upper-Intermediate, design graph E-3.

Outflow, $O = 2.03 \times 10^{-5} \text{ L/s/m}^2$ of equivalent impervious area, A_{EIA} .

Target percentage of (average) annual runoff to be harvested : Trial 1, **use 40%**.

This gives (Figure E-3, Appendix E) **storage, $\beta = 2.3\%$** .

STEP 3:

Average annual runoff (volume) from the contributing catchment is :

$$V_A = (1.226) \times 0.75 \times 200 \text{ m}^3 = 184 \text{ m}^3 \quad (8.1)$$

$$\text{Required storage volume is } V_h = 0.023 \times 184 \text{ m}^3 = 4.2 \text{ m}^3 \quad (8.2)$$

STEP 4 : Review the design volume, V_h

In the case of the residence in Sydney with 70 m length of eave-guttering, 4.2 m^3 of storage translates into $\frac{4,200}{70} = 60 \text{ L}$ per lineal metre. This is greater than the capacity available in the standard “Rainsaver” installation (23 L/m).

Return to **STEP 2** and select percentage of (average) annual runoff to be harvested as 30%. This leads to storage, **$\beta = 0.9\%$. Hence (STEP 3) :**

$$V_A = (1.226) \times 0.75 \times 200 \text{ m}^3 = 184 \text{ m}^3 \quad (8.1)$$

$$\text{Required storage volume is } V_h = 0.009 \times 184 \text{ m}^3 = 1.66 \text{ m}^3 \quad (8.2)$$

This storage volume distributed around 70 m of eave-guttering represents 23.6 L/m which corresponds, approximately, to the storage capacity of the standard “Rainsaver” unit.

This terminates Procedure 10.

Example 8.3

An ornamental pond is to be designed for the grounds at the rear of the church site illustrated in Figure 5.3, located in Adelaide: the ‘normal level’ in the pond will be fixed by harvesting (not flood control) considerations. The pond will receive all storm runoff from the catchment defined in Example 5.3. Outflow from the pond will include seepage into the native clay soil of the site and evaporation from the pond surface including an allowance for evapotranspiration from native water-plants. No additional use of the stored water is contemplated other than during periods of high rainfall when pond excess *up to the defined overflow level* (flood control) may be applied to open space irrigation.

Preliminary considerations

- The catchment is located in Adelaide – Southern Australia climate zone.
- Average annual rainfall is 0.58 m.
- Catchment :
 total connected roof area 1,600 m²
 total connected paved area 360 m²
 total connected pervious area ($C_{10} = 0.10$) 680 m²
- Outflow : for preliminary (trial) estimate, assume pond surface area, $A_p = 50 \text{ m}^2$:
 seepage = $1 \times 10^{-6} \text{ m/s} \times (\text{Moderation factor} = 2.0) \times (\text{pond area} = 50 \text{ m}^2)$ 8,640 L/day
 evaporation/transpiration (allow 3.0 m per annum, 50 m² pond) 410 L/day

PROCEDURE 10 – STEP 1 :

Catchment area, $A_{EIA} = 1,600 + 360 + (0.10 \times 680) = 2,028 \text{ m}^2$.

STEP 2 :

The site location is Adelaide : climate zone – Southern Australia, design graph Figure 8.5.

Outflow, O = $(8,640 + 410) = 9,050 \text{ L per day} = 0.1047 \text{ L/s}$

= $5.16 \times 10^{-5} \text{ L/s/m}^2$ of equivalent impervious area, A_{EIA} .

Note : Evaporation/transpiration is not constant throughout the year but is assumed so here to determine Outflow O . This approximation is acceptable because it represents less than 5% of O .

Target percentage of (average) annual runoff to be harvested: Trial 1, use 90%.

This gives (Figure 8.5) **storage, β** = 3.8%.

STEP 3:

Average annual runoff (volume) from the contributing catchment is :

$$V_A = (0.58) \times 0.75 \times 2,028 \text{ m}^3 = 882 \text{ m}^3 \quad (8.1)$$

$$\text{Required storage volume is } V_h = 0.038 \times 882 \text{ m}^3 = 33.5 \text{ m}^3 \quad (8.2)$$

STEP 4 : Review the design volume, V_h

Based on 90% stormwater harvesting, pond storage volume 33.5 m³ leads to pond depth, 0.67 m, for the 50 m² plan area pond. This may be considered too deep – for safety reasons – and a shallower depth preferred.

Return to STEP 2 and use harvesting percentage 80% instead of 90%. This leads to :

$$\text{storage, } \beta = 2.2\% \text{ and } V_h = 0.022 \times 882 \text{ m}^3 = 19.4 \text{ m}^3,$$

hence, pond depth, **$H = 0.39 \text{ m}$** , which is acceptable.

The review process in the case of an ornamental pond receiving surface runoff, must also take account of stormwater generated in minor and major storm events. This requires re-design of the facility as a ‘dry’ pond using Procedure 3, Section 5.1. Correct process with this calculation requires the storage available in the ornamental pond to be *ignored* (assumed always full) and the ‘dry’ pond to be located, effectively, above it. The computation (ARI, Y = 100-years design requirement, following the approaches presented in Examples 5.3 and 5.4, Section 5.2) leads to a design for the ‘dry’ pond (many options) :

$$A_p = 240 \text{ m}^2 \text{ with depth, } H = 0.30 \text{ m.}$$

This outcome could be interpreted into the site as an attractive landscaped area, 240 m² in extent and ‘sunk’ 0.30 m below natural surface (the ‘dry’ pond), within which the stormwater harvesting pond, 50 m² in area and a further 0.40 m deep, is located.

The following points should be noted in relation to this addendum (the ‘dry’ pond) :

- Inundation of the ‘dry’ pond area (surcharge from the stormwater harvesting pond) is likely to occur about four times per year because its capacity is closely associated with ARI, Y = 0.25-years.
- The design approach suggested above may be unacceptable on the grounds of cost and a design ARI significantly less than Y = 100 years required, for example ARI, Y = 5 years. In this event, consideration should be given to the provisions of the last four paragraphs of Section 4.2.6 and a design prepared taking account of the full range of storm durations from 10 minutes to 72 hours (see Example 5.5 in Section 5.2).

This concludes Procedure 10.

8.6.5 Rainwater tank design software package

It will be clear from Examples 8.1 to 8.3 inclusive that the process of using any of Figures 8.1 to 8.4 inclusive, or information from a graph such as Figure 8.5 (or any of the graphs from Appendix E), does not lead to a *direct* solution in any given practical case. As stated in Section 8.6.3, the design process involves iteration and compromise. A software package – publicly accessible – has been developed to aid this process. It is available from the Centre for Water Management & Reuse, University of South Australia website at : <http://www.unisa.edu.au/water/UWRG/publication/raintankanalyser.asp>

The primary use of the package is for matching a rainwater tank size (volume) to any given set of **domestic** circumstances (location, roof catchment area, daily demand, etc.) including irrigation if required, but it is equally capable of providing tank sizes (up to 20 kL capacity) associated with industrial settings. The software brings together elements of both Procedures 9 and 10. Its main features are :

- Provides, directly, a **suggested** tank size based on *hydrological* considerations; the user is also presented with information enabling an alternative, optimum *economic* size to be selected. The process is illustrated, directly, for each of the state and territory capital cities (nominal tank sizes).
- Includes capability for users to ‘paste in’ continuous (daily) rainfall data and local monthly irrigation demand for any location.
- Includes capability to fix tank storages for in-house only or outdoor only uses, or combination of in-house/outdoor uses.
- Each application of the software reports :
 - average annual in-house and irrigation demand in kL/annum;
 - **suggested** tank size (hydrological considerations) in kL;
 - average annual yield in kL/annum;
 - average number of days when less than full demand is provided;
 - average number of days with zero supply;
 - percentage of total demand supplied by *suggested* tank.

8.7 RAINWATER TANKS AND URBAN DRAINAGE : CONSEQUENCES FOR FLOOD CONTROL

It has long been held that rainwater tanks or similar installations, operated in the manner described in the foregoing sections of this chapter in an urban catchment, can result in significant reductions in stormwater peak flows generated in that catchment. Local government agencies have, on the other hand, been traditionally unimpressed by these claims and have, instead, maintained that reduction of peak flow through the presence of rainwater tanks cannot be relied upon because streamflow peaks are likely to coincide with the tanks being full and therefore incapable of providing benefit by way of detention storage. This ‘case’ is further strengthened when it is recognised that conventional use of rainwater tanks by past generations has been to supply minimal in-house demand, for example tea-making, with the consequence that rainwater tanks have remained full or nearly so for the greater part of the time.

Steadily growing population and profligate use of water to maintain exotic lawn and garden species, coupled with forecasts of adverse climate change, have led to the prospect of serious water shortages occurring in the foreseeable future in many of our major urban concentrations. Water restrictions imposed in the early years of the new century are evidence of this recognition. Two potential solutions are being widely canvassed – rainwater (or similar) on-site storages becoming mandatory, and conversion of the Australian garden culture from its preoccupation with exotic plants and lawn grasses to native and drought-resistant varieties (Argue and Barton, 2004).

In this climate of water conservation and cultural change, the types of devices and systems reviewed in Section 2.2 are likely to assume an importance in urban Australia of the future not seen since the days of the pioneers when the only water which could be relied upon was that available on-site – roof runoff, recycled water and groundwater, where available. It follows that future use of roof runoff collection/storage systems will not be limited to the trivial uses of the past, such as tea-making but, rather, to purposeful and strategic exploitation of the resource which it should be remembered is delivered “free” at good quality to every residence and commercial/industrial building in the nation. Earlier sections of Chapter 8 provide the basis for bringing about this change in rainwater collection/use; but its consequences for flooding have not been addressed.

So how is the predicted change likely to impact on flood management in the urban landscape?

There are complex issue here – interaction between two rainfall-dependent but significantly different phenomena (rainwater collection/use and catchment peak flows) – which can only be solved by analyses which have firm bases in the ‘continuous simulation’ modelling of both the collection/use aspect as well as catchment storm runoff flow estimation. Research has been undertaken in recent years on the problem [Allen (1993); van der Wel (2000); Coombes and O’Loughlin (2001), Allen (2002)] but has not satisfactorily answered the principal questions : What quantifiable effect can operating rainwater tanks have on catchment outflow peaks? And, in particular, can the presence of domestic rainwater tanks operated as outlined above be relied upon to absolve a municipal authority from the need to upgrade stormwater infrastructure as urban development/re-development takes place?

An exploration of these questions was conducted by Coombes (2002) who used ‘continuous simulation’ modelling to compare the volumes of conventional OSD tanks required on a residential allotment near Newcastle, NSW, to achieve flood mitigation objectives, against rainwater tank storages designed to satisfy allotment stormwater harvesting as well as (the same) flood mitigation objectives. ‘Continuous simulation’ was then applied to a proposed 39 ha sub-division, also near Newcastle, which incorporated allotment-sized rainwater tanks throughout. Stream flows computed for the proposed development showed reductions (below values determined for a ‘traditional’ drainage layout) of between 10% and 44%. Coombes described his analysis as an “initial foray” with improved performance likely through optimisation.

Pezzaniti (2003) investigated the consequences for flood mitigation of domestic rainwater tanks in a 50 ha urban catchment in Adelaide. He used ‘continuous simulation’ modelling based on over 100 years of rainfall records and took account of antecedent conditions in the rainwater tanks at the onset of critical design storm events. The capacity of the (existing) catchment drainage network was assessed to be ARI, Y = 2-years. The study showed that it was possible for this stormwater infrastructure to convey ARI, Y = 5-years peak flows if large (4.5 kL) rainwater tanks were fitted to each allotment and in-house water demand (from the tanks) was set at 300 L/day. This value of use is in the middle of the range for typical Adelaide households (see Table 8.3) and, therefore, quite feasible. Modelling of lower in-house daily demand by the

households did not result in any significant benefits for the drainage infrastructure. Also, small rainwater tanks (2 kL capacity) showed no drainage benefit at all.

A further, interesting outcome of the study related to the spatial distribution of rainwater tanks within the catchment. The same drainage infrastructure benefits in terms of peak flow conveyance could be achieved by applying rainwater tanks to only the upper two-thirds of the catchment. The economic benefit to a drainage authority of incorporating rainwater tanks into a new catchment is relatively small. However, the benefits of retrofitting tanks in the favourable circumstances reviewed above, namely 4.5 kL tanks with 300 L/day use, significantly outweigh the costs of upgrading existing drainage infrastructure in the face of widespread re-development.

Hardy, Coombes and Kuczera (2004) investigated the problem of rainwater tank impact on flood management in hypothetical, medium-density residential catchments in four Australian capital cities – Adelaide, Brisbane, Melbourne and Sydney. The catchments ranged in size from 2.5 ha to 10 ha. The developments incorporated street drainage networks matched to the critical ARI, $Y = 5$ -years design standard. Two domestic scenarios were investigated :

- Residences with no rainwater tanks (the “no tank” option); and,
- Residences with 5 kL rainwater tanks (the “tanks” option).

Rainwater abstracted from the tanks was applied to both indoor and outdoor uses in the model. Peak flows generated in the catchments were determined for design storms ranging from ARI, $Y = 0.25$ -years up to and including 100-years events; the storm duration range was 5-minutes up to 72-hours.

The study sought answers to two main questions :

1. What storage is available in a 5 kL rainwater tank prior to the arrival of a significant storm event? and,
2. What differences in peak flows can be expected between catchment outflows in the “no tanks” and “tanks” cases ?

Results of the modelling showed quite similar storage volumes being available in the rainwater tanks of the southern Australian cities (Adelaide and Melbourne) over the full range of ARIs normally associated with ‘minor’ and ‘major’ stormwater drainage systems. However, Brisbane (representing Northern Australia) showed indications of an ARI-dependent (direct) relationship with respect to available on-site storage. Results for Sydney (Upper-Intermediate climate zone) fell between these extremes.

Results (Hardy et al) from the second issue investigated, above, showed that rainwater tanks can be associated with significant peak flow reductions in the low rainfall regions of Australia, but not in the tropics. Outcomes for Intermediate Australia (represented by Sydney), again, fall between these extremes.

Collating all of these results, it may be concluded that the beneficial impact which the presence – and daily use – of rainwater stored in tanks is likely to have on flood peaks in the waterways of an urban catchment in Australia is related to :

- The common tank size and (daily) extraction rate;
- the spatial distribution of tanks within the catchment;
- whether new development (“greenfields”) or re-development of old housing stock is involved;
- the geographical location (climate) of the catchment.

This brief review serves to inform us that the use of rainwater tank storage in the context of flood management is complex and, certainly, not amenable to any simple rule, such as “5 kL rainwater tanks installed throughout an urban catchment will reduce main channel peak flows from ARI, $Y = 5$ -years, to ARI, $Y = 2$ -years”, highly desirable though this might be. It is our judgement that such a rule will never emerge for nation-wide application, although some guideline of this nature may be possible in specific locations under particular circumstances of urban development.

9 CONCLUDING DISCUSSION

9.1 OVERVIEW

The **vision** of stormwater management proposed in the Handbook, ranging from conservation of natural drainage lines and their associated fauna and flora ecosystems in forested catchments facing development, through to restoration of former drainage paths in catchments ‘lost’ to overdevelopment, was firmly set in Chapter 1 (see Section 1.4). The remaining chapters have provided the tools by which that vision can be realised. Indeed, it may be argued that this is the primary contribution which stormwater ‘source control’ can make to WSUD, not only because of its role in reducing the impacts of storm runoff, particularly in catchment ‘bottom lands’, but also because of the improved amenity and heightened community and environmental values which it brings.

9.2 SOME ISSUES

9.2.1 Relationship of the Handbook to Current Australian Practice

9.2.2 Resistance to change

9.2.3 Differences in Northern, Intermediate and Southern Australian design outcomes

9.2.4 Pollution control in Australian urban catchments

9.3 THE NEED FOR CASE STUDIES

9.3.1 Stream management scenario/strategies

9.3.2 Detention/retention in the urban landscape

9.3.3 Rainwater tanks and flood control

9.3.4 Cost and policy matters

9.4 RESEARCH DIRECTIONS

9.4.1 Acceptable ‘emptying time’ for OSR devices

9.4.2 Detention/retention : spatial distribution

9.4.3 Problems of water retention and ‘reactive’ soils

9.4.4 Pollution control – fine sediments

9.5 CONCLUSION

[These sections omitted from the Student Edition]

See page xi for details of distributors of the full Handbook

BIBLIOGRAPHY

- ALLEN, M. (1993). *An investigation of appropriate technology on-site water conservation, roof runoff supply and water re-use systems for application within the Adelaide Metropolitan area*. Master of Engineering thesis, School of Civil Engineering, University of South Australia.
- ALLEN, M.D. (1994). *Parametric studies for domestic rainwater tanks : comparison between pluviometric, daily and monthly data*. Consultancy for Engineering and Water Supply Department, South Australia.
- ALLEN, M.D. and ARGUE, J.R. (1998). *Stormwater management and rainwater use in Adelaide households*. Report, UWRC, University of South Australia.
- ALLISON, R.A. and CHIEW, F.H.S. (1995). *Monitoring of stormwater pollution from various land uses in an urban catchment*. Proc. I.E.Aust. 2nd Int'l Symposium on Urban Stormwater Management, Melbourne.
- ANZECC (2000). *Australian water quality guidelines for fresh and marine waters*. National Water Quality Management Strategy. AGPS, Canberra.
- ANZFA (2000). *Food standards code*. Australian and New Zealand Food Authority. AGPS, Canberra.
- ARGUE, J.J. and ARGUE, J.R. (1998). *Total stormwater management at "Figtree Place", Newcastle, NSW*. Proc. 3rd Novatech Conference on Innovative Technologies in Urban Stormwater Drainage, Graie, Lyon, June.
- ARGUE, J.R. (1986). *Storm drainage design in small urban catchments : a Handbook for Australian practice*. Aust. Road Research Board, Special Report, SR 34, South Vermont, Victoria.
- ARGUE, J.R. (1997). *Total stormwater management concept study for inner Newcastle : Stage 1 – concept study for Hamilton Bus Station site*. UWRC, University of South Australia, Adelaide.
- ARGUE, J.R. (2000). *Stormwater 'source control' design procedures for WSUD : some issues and examples*. Proc. 1st National Conf. on Sustainable Drainage Systems for Urban Areas, Melbourne Water, Melbourne, August.
- ARGUE, J.R. (2001). *Testing of infiltration at four sites in the Upper Parramatta River Catchment Trust Area – Progress Report Project 07-67610*. UWRC, University of South Australia, Adelaide, November.
- ARGUE, J.R. (2005). *Strategic planning for stormwater management in urban catchments*. Proceedings, 2005 Adelaide Public works Conf., Institute of Public Works, Australia, August, Adelaide.
- ARGUE, J.R. and BARTON, A. (2004). *Water-sustainability for Adelaide in 2020 based on stormwater : Options, Opportunities and Challenges*. Proc. Engineers Aust. WSUD 2004 Conference "Cities as Catchments", Adelaide, November.
- ARGUE, J.R., BOUBLI, D. and COOMBES, P.J. (2003). *Innovative water cycle management at a difficult site : "Heritage Mews" in Western Sydney*. Proc. 2nd National Conf. on SUDS, Coventry, June.
- ARGUE, J.R. and PEZZANITI, D. (1999). *Catchment 'greening' using stormwater in Adelaide, South Australia*. Water Science and Technology, Pergamon, Vol. 39, No. 2, pp 177 – 183.
- ARGUE, J.R. and PEZZANITI, D. (2001). *The five 'landmarks' of stormwater management in Adelaide*. Proc. "Water History" Symposium, Hydrological Society of SA, Goolwa, September.
- ARGUE, J.R. and PEZZANITI, D. (2004). *A strategic approach to managing storm runoff in urban/urbanising catchments*. Proc. 5th Novatech Conf. on Innovative Technologies in Urban Storm Drainage, Graie, Lyon, June.
- ARGUE, J.R. and PEZZANITI, D. (2006). *Domestic rainwater tanks: meeting the mains water 'shortfall'*. Proc. 30th Hydrology & Water Resources Symposium, Launceston, December.

- ATV (Abwassertechnische Vereinigung E V, 1990). *Construction and dimensioning of facilities for decentralised percolation of non-harmful polluted surface water*. Standard ATV – A 138E, Hennef, Germany.
- AUSTIN TEXAS (1988, 1990). *Austin 'first flush' filtration basin*. In Municipal Storm Water Management (Debo and Reese, 1995a), CRC Press LLC, Boca Raton, Florida.
- AWRC (1995). *Guidelines for groundwater protection in Australia*. Australian Water Resources Council and National Water Quality Management Strategy. AGPS, Canberra.
- BARTON, A. (2003). *A comparative analysis of the characteristics of domestic water use in metropolitan Adelaide*. Proc. of 2003 Regional Conf., Australian Water Association, Adelaide, August.
- BEAN, E.Z., HUNT, W.F. and BIDELESPACH, D.A. (2007). *Evaluation of four permeable pavement sites in Eastern North Carolina for runoff reduction and water quality impacts*. Journal of Irrigation & Drainage Engineering, ASCE, Vol. 133, No. 6, December 1.
- BEECHAM, S. (2003). *Water sensitive urban design – a technological assessment*. "Waterfall", Journal of the Stormwater Industry Assoc, Issue 17, Sydney.
- BETTES, R. (1996). *Infiltration drainage – manual of good practice*. Construction Industry Research and Information Assoc. CIRIA Report, 156, London.
- BODY, P. (1986). *The contamination of rainwater tanks at Port Pirie*. South Australian Department for Environment and Planning.
- BOND, P.C., PRATT, C.J. and NEWMAN, A.P. (1999). *A review of stormwater quantity and quality performance of permeable pavements in the UK*. Proc. 8th Int'l Conf. on Urban Storm Drainage, I. Joliffe and J. Ball (Eds.), I.E.Aust. IAHR/IAWQ, Sydney, August/September.
- BOUGHTON, W.C. (1966). *A mathematical model for relating runoff to rainfall with daily data..* Civil Engg Trans., I.E.Aust. CE30(4), pp 153 – 162.
- BREEN, L., DYER, F., HAIRSINE, P., RIDDIFORD, J., SIRRIWARDHENA, V. and ZIERHOLZ, C. (1997). *Controlling sediment and nutrient movement within catchments*. CRC for Catchment Hydrology, Industry Report 97/9, Melbourne.
- BRESSAN, F. (1996). *The relationship between hydraulic conductivity and borehole dimension in selected soils of the Greater Adelaide area*. B.Eng thesis, Faculty of Engineering, Civil Engineering Discipline, Univ. of South Australia.
- BUILDING RESEARCH ESTABLISHMENT (1991). *Soakaway design*. BRE Digest 365, London.
- CHANDLER, R. (2001). *Turbid relations : managing creeks in an ultra-urban city*. Proc. 4th Novatech Conf. on Innovative Technologies in Urban Storm Drainage, Graie, Lyon, June.
- CIRIA (2001). *Sustainable urban drainage systems (SUDS) : design manual for England and Wales*. Construction Industry Research and Information Association, Report C522, London.
- CIRIA (2007). *The SUDS (sustainable urban drainage systems) manual*. Construction Industry Research and Information Association, Report C697 ISBN 987-0-86017-697-8; 0-86017-697-5, London.
- CHAPMAN, T.G. (1968). *Catchment parameters for a deterministic rainfall-runoff model*. In Stewart, G.A. (Ed) Land Evaluation. Macmillan, Melbourne.
- CHARACKLIS, G.W. and WEISNER, M.R. (1997). *Particles, metals and water quality in runoff from a large urban watershed*. Journal of Environmental Engg, Vol. 123, No. 8, pp 753 – 759.
- COMMONWEALTH EPA (1993). *Urban stormwater – a resource too valuable to waste*. AGPS, Canberra.

- COOMBES, P., FROST, A., KUCZERA, G., O'LOUGHLIN, G. and LEES, S. (2003). *The impact of rainwater tanks in the Upper Parramatta River Catchment*. Australian Journal of Water Resources (Engineers Australia), Vol. 7, No. 2, Engineers Media, Crows Nest, NSW.
- COOMBES, P. and KUCZERA, G. (2001). *Rainwater tank design for water supply and stormwater management*. Stormwater Industry Assoc Regional Conference, Port Stephens, NSW, April.
- COOMBES, P., ARGUE, J. and KUCZERA, G. (2000). *"Figtree Place" : a case study in water sensitive urban development (WSUD)*. Urban Water, 1(4), Elsevier, London, pp 335 – 343.
- COOMBES, P., BOUBLI, D. and ARGUE, J. (2003). *Integrated water cycle management at the "Heritage Mews" development in Western Sydney*. Proc. 28th I.E.Aust. Hydrology and Water Resources Symposium, Wollongong, November.
- CRAWFORD, N.H. and LINSLEY, R.K. (1962). *The synthesis of continuous streamflow hydrographs on a digital computer*. Technical Report No. 12, Dept of Civil Engineering, Stanford University.
- CSIRO (1999). *Urban stormwater : best practice environmental management guidelines*. Victorian Stormwater Committee and Commonwealth Scientific & Industrial Research Organisation. ISBN 0643 06453 2, Melbourne.
- CUNLIFFE, D. (1998). *Guidance on the use of rainwater tanks*. National Environmental Health Forum Monographs – Water Series No. 3. Website : <http://www.dhs.sa.gov.au/pehs/publications/monograph-rainwater.pdf>.
- DATRY, T., MALARD, F. VITRY, L. HERVANT, F. and GILBERT, J. (2003). *Solute dynamics in the bed sediments of a stormwater infiltration basin*. Journal of Hydrology, Vol. 273, p 217-233.
- DAVIS, A.P., HUNT, W.F., TRAVER, R.G. and CLAR, M. (2009). *Bioretention technology : overview of current practice and future needs*. Proc. Journal of Environmental Engineering, ASCE, Vol. 135, No. 3, March 1.
- DAVIS, J.P., ROHRER, C.A. and ROESNER, L.A. (2006). *Effects of catchment modification on the flow frequency curve-modelling using SWMM*. Proc. ASCE World Environmental and Water Resources Congress, Omaha, Nebraska, USA.
- DEBO, T.N. and REESE, A.J. (2003). *Municipal stormwater management*. CRC Press LLC, ISBN0-87371-981-6, Boca Raton, Florida.
- DEBO, T.N. and REESE, A.J. (1995). *Determining downstream analysis limits for detention facilities*. Proc. Novatech Innovative Technologies in Urban Storm Drainage, Graie, Lyon.
- DELETIC, A. (2001). *Modelling of water and sediment transport over grassed areas*. Journal of Hydrology, Vol. 248, pp168-182.
- DEPARTMENT OF HOUSING and REGIONAL DEVELOPMENT (1995). *Australian model code on residential development (AMCORD)*. AGPS, Canberra.
- DEPARTMENT OF WATER, WESTERN AUSTRALIA (2007). *Stormwater Management manual for Western Australia*. Dept. of Water WA, Perth, August.
- DePINTO, J., YOUNG, T.C. and MARTIN, S.C. (1980). *Aquatic sediments*. Journal of Water Pollution Control Federation, Vol. 52, No. 6.
- DILLON, P.J. and PAVELIC, P. (1996). *Guidelines for the quality of treated effluent for injection into aquifers for storage and re-use*. Urban Water Research Assoc of Australia, Research Report No. 109, Melbourne.
- DIERKES, C., BENZE, W. and WELLS, J. (2002). *Sustainable urban drainage and pollution source control by infiltration*. "Waterfall", Journal of Stormwater Industry Association, Vol. 16, Spring, Sydney.

- DLWC (1998). *The constructed wetlands manual*. NSW Dept. of Land and Water Conservation, Sydney.
- DOYLE, M.W., SHIELDS, D., BOYD, K.F., SKIDMORE, P.B. and De WITT, D. (2007). *Channel-forming discharge selection in river restoration design*. ASCE Jour.of Hydraulic Engg, Vol. 133, No. 7, July 1.
- DUNCAN, H.P. (1999). *Urban stormwater quality – a statistical overview*. CRC for Catchment Hydrology, Melbourne.
- EMERSON, C.H., WELTY, C. and TRAVER, R.G. (2005). *Watershed-scale evaluation of a system of storm water detention basins*. Proc. ASCE Journal of Hydrologic Engineering, Vol. 10, No. 3, May 1.
- ENGINEERS AUSTRALIA (1999). *Australian rainfall and runoff – a guide to flood estimation*. This is the 3rd edition (1987) of “Australian Rainfall and Runoff – Volume 1” presented in eight booklets. ISBN 185 825 6878.
- ENGINEERS AUSTRALIA (2006). *Australian runoff quality*. Engineers Australia. Wong, T.H.F. (Editor), Canberra, November.
- ERICKSON, A.J., GULLIVER, J.S. and WEISS, P.T. (2007). *Enhanced sand filtration for stormwater phosphorus removal*. Journal of Environmental Engineering, ASCE, Vol. 133, No. 5, May 1.
- ESPINOSA, R., AL-FAYYAZ, T., ARGUE, J., JOHNSTON, L. and PEZZANITI, D. (2001). *High-performance sediment-retaining device for urban catchments : introducing the UniSatank*. Proc. Novatech Innovative Technologies in the domain of Urban Stormwater Drainage, Graie, Lyon, June.
- eWATER CRC (2010) *Model for urban stormwater improvement conceptualisation (MUSIC)*, ver. 4, computer program, eWater CRC, University of Canberra, ACR, Australia.
- FERGUSON, B.K. (1994). *Stormwater infiltration*. CRC Press, Boca Raton, Florida.
- FERGUSON, B.K. (1995). *Stormwater infiltration for peak-flow control*. ASCE Journal of Irrigation and Drainage Engineering, Vol. 121, No. 6, November/December.
- FISCHER, D., CHARLES, E.G. and BAEHR, A. L. (2003). *Effects of stormwater infiltration on quality of groundwater beneath retention and detention basins*. Journal of Environmental Engineering, Vol. 129, pp 464-471.
- FLETCHER, T., WONG, T. and BREEN, P. (2005). *Buffer strips, vegetated swales and bio-retention systems*. Chapter 9 of Australian Runoff Quality (Engineers Australia, 2005), Canberra, November.
- FRANCE, R.L. (2003). *Handbook of water sensitive planning and design*. CRC Lewis, France (Ed), Boca Raton, Florida.
- FULLER, C.A., MARTIN, T.J. and WALTERS, R.P. (1981). *Quality aspects of rainwater stored in domestic rainwater tanks (a preliminary study)*. Domestic Rainwater Tanks Working Party, South Australia.
- GEIGER, W.F. and DREISEITL, H. (1995). *Neue wege fur das regenwasser*. Oldenbourg, Munchen.
- GILLIARD, G. (2004). *A new solution for stormwater management*. Plan Canada, Spring edition, Vol. 44, No. 1, Robert Shipley (Ed.), Canadian Institute of Planners, Ottawa, Ontario, pp 29 –31.
- GIPPEL, C. (2002). *Geomorphic issues associated with environmental flow assessment in alluvial non-tidal rivers*. Aust. Journ of Water Resources, Vol. 5, No 1.
- GOETTLE, A. (1978). *Atmospheric contaminants – fallout and their effects on stormwater quality*. Progress in Water Technology, 10 (5/6), pp 455 – 467.
- GUO, C.Y. and URBONAS, B. (1996). *Maximised detention volume determined by runoff capture ratio*. ASCE Journal of Water Resources Planning and Management, Vol. 122, No. 1, January, pp 33 – 39.

- GUTTERIDGE, HASKINS and DAVEY (1981). *Characterisation of pollution in urban stormwater runoff*. AWRC Technical Paper No. 60, EPA of Victoria. AGPS, Canberra.
- HAMMER, T. (1973). *Effects of urbanisation on stream channels and stream flow*. Office of Water Resources Research, US Dept of Interior (Contract No. 14-31-0001-3406), Regional Science Research Institute, GPO Box 8776, Philadelphia, Penn, 19101, November.
- HARDY, M.J., COOMBES, P.J. and KUCZERA, G. (2004). *An investigation of estate level impacts of spatially distributed rainwater tanks*. Proc. Engineers Aust. WSUD 2004 Conference “Cities as Catchments”, Adelaide, November.
- HARSTON, E. and JOLIFFE, I. (1999). *Rationalisation of sediment basin sizing*. Proc. 8th International Conf. on Urban Storm Drainage, I.E.Aust. IAHR/IAWQ, Sydney, August/September.
- HATT, B.E., FLETCHER, T.D. and DELETIC, A. (2007). *Treatment performance of gravel filter media : implications for design and application of stormwater infiltration systems*. Water Research Vol. 41, Elsevier, pp 2513 – 2524.
- HENEKER, T.M., LAMBERT, M.F. and KUCZERA, G. (2001). *A point rainfall model for risk based design*. Journal of Hydrology, Vol. 247, 54 – 71.
- HENEKER, T.M., LAMBERT, M.F. and KUCZERA, G. (2003). *Overcoming the joint probability problem associated with initial loss estimation in design flood estimation*. Australian Journal of Water Resources (Engineers Australia), Vol. 7, No. 2, Engineers Media, Crows Nest, NSW.
- HEWA, G., ARGUE, J.R. and PEZZANITI, D. (2009). *Setting criteria for environmental and channel-forming flows for waterways in urbanising catchments*. Proc WSUD09 Conf. “Towards water sensitive cities and citizens”, Engineers Australia, Perth, May (in press).
- HEYWORTH, J.S., MAYNARD, E.J. and CUNLIFFE, D. (1998). *Who consumes what. Potable water consumption in South Australia*. Water 25(1), 9-13.
- HEYWORTH, J., GLONEK, G. and MAYNARD, E. (1999). *The prevalence of gastroenteritis amongst young children and the potential role of drinking water*. Proc. 18th Federal Convention, Aust. Water & Wastewater Assoc., Adelaide.
- HEYWORTH, J.S. (2001). *A diary study of gastroenteritis and tank rainwater consumption in young children in South Australia*. Proc. 3rd Int’l Rainwater Catchment Systems Conf., Mannheim, Germany, September, pp 141 – 148.
- HOPKINS, B. and ARGUE, J.R. (1994). *The New Brompton Estate stormwater management trial : first results*. Water Science and Technology, Vol. 29, No. 1-2, Pergamon, London.
- HORTON, R.E. (1993). *The role of infiltration in the hydrologic cycle*. Trans. American Geophysical Union, Vol. 14, pp 446 – 460.
- HORNER, S. (1999). *Water quality modelling manual*. Bullen and Partners, Bradford, UK.
- I.E.AUST. (1987). *Australian rainfall and runoff – a guide to flood estimation*. Institution of Engineers, Australia, Pilgrim, D. (Editor), Canberra.
- I.E.AUST. (1999). *Australian rainfall and runoff – a guide to flood estimation*. This is the 3rd edition (1987) of “Australian Rainfall and Runoff – Volume 1”, presented in eight booklets. ISBN 185 825 6878.
- I.E.AUST. (2003). *Australian runoff quality (DRAFT)*. Engineers Australia, Wong, T.H.F. (Editor), Canberra, June.
- JACOBS, R.R. (2001). *Development and evaluation of an improved infiltration device technology*. Report, UWRC, University of South Australia, Adelaide, June.

- JEFFERIES, C. and NAPIER, F. (2008a). *SUDS Pollution degradation - Report No 1*. Scotland & Northern Ireland Forum for environmental research (SNIFFER) Report UEUW01, Edinburgh, February.
- JEFFERIES, C. and NAPIER, F. (2008b). *SUDS Pollution degradation - Report No 2*. Scotland & Northern Ireland Forum for environmental research (SNIFFER) Report UEUW01, Edinburgh, October.
- JEPPESON, B. (1996). *Model guidelines for domestic greywater re-use for Australia*. Research Report No. 107, Urban Water Research Assoc. of Australia, Melbourne.
- JONASSON, A.S. (1984). *Dimensioning methods for stormwater infiltration systems*. Proc. 3rd Int'l Conf. on Urban Storm Drainage, IAHR/IAWQ, Gotenborg, June.
- KING, J. (2003). *Environmental flows for rivers : hydrologically definable, ecologically vital*. Keynote Address, 28th I.E.Aust. Hydrology and Water Resources Symposium, Wollongong, November.
- KOTLAR, E., TARTAKOVSKY, B., ARGAMAN, Y. and SHEINTUCH, M. (1996). *The nature of interaction between immobilised nitrification and denitrification bacteria*. Journal of Biotechnology Vol. 51, p 251-258.
- KUCZERA, G., LAMBERT, M., HENEKER, T., JENNINGS, S., FROST, A. and COOMBES, P. (2003). *Joint probability and design storms at the crossroads*. Keynote Address, 28th I.E.Aust. Hydrology and Water Resources Symposium, Wollongong, November.
- LEE, A., HEWA, G., PEZZANITI, D. and ARGUE, J. (2008). *Improving stream low flow regimes in urbanised catchments using water-sensitive urban design techniques*. Proc. Aust. Journ of Water Resources, Vol 12 No 2, Engineers Media, Crows Nest.
- LEE, G.F. and TAYLOR, S. (1998). *Development of appropriate stormwater infiltration BMPs : Part II – design of infiltration BMPs*. Proc. of Groundwater Protection Council Annual Forum 1998, Sacramento, California.
- LESTER, R. (1992). *A mixed outbreak of cryptosporidiosis and giardiasis*. May 1992. Update. A Quarterly Bulletin of Infectious Diseases. 1(1), 14 – 15. Health Department Victoria.
- LHCCREMS (2002). *Water smart model planning provisions*. Prepared by Planning Plus (Author I. Donovan) for Lower Hunter Central Coast Regional Environmental Management Strategy, Callaghan, October.
- LI, M-H., BARRETT, M.E., RAMMOHAN, P., OLIVERA, F. and LANDPHAIR, H.C. (2008). *Documenting stormwater quality on Texas highways and adjacent vegetated roadsides*. Journal of Environmental Engineering, ASCE, Vol. 134, No. 1, January, pp 48 – 59.
- LIEBMAN, M., BROWN, M., GARRAWAY, E. and JONES, C. (2004). Kiama CBD's stormwater treatment and re-use project. SIA 2004 Regional Conference, Shoalhaven, NSW, April.
- LIESKO, J.Z., WHITELY, H.R. and CORSI, R.L. (1993). *Quality of stormwater from residential areas*. In New Techniques for Modelling the Management of Stormwater Quality Impacts, W. James (Ed.), Lewis Publishers, London.
- LINDSAY, G., ROBERTS, L. and PAGE, W. (1991). *Stormwater management practice in Maryland : a second survey*. Maryland Dept. of Environment (Sediment and Stormwater Administration), Baltimore.
- LINSLEY, R.K. and CRAWFORD, N.H. (1960). *Computation of a synthetic streamflow record on a digital computer*. Publication No. 50, International Assoc. Scientific Hydrology, pp 526 – 538
- LLOYD, S., FLETCHER, T., WONG, T. and WOOTTON, R. (2001). *Assessment of pollutant removal in a newly constructed bio-retention system*. Proc. 2nd South Pacific Stormwater Conf., Auckland Regional Council, Auckland.

- LLOYD, S, FRANCEY, M. and SKINNER, L. (2004). *Cost of incorporating WSUD into greenfield site development and application of offset trading scheme*. Proc. Engineers Aust. WSUD Conf. "Cities as Catchments", Adelaide, November.
- LOWE, K.S. and SIEGRIST, R.L. (2008). *Controlled field experiment for performance evaluation of septic tank effluent treatment during soil infiltration*. Journal of Environmental Engineering, ASCE, Vol. 134, No. 2, February 1.
- MAESTRI, B., JOHNSON, F., BURCH, C.W. and DAWSON, B.L. (1985). *Management practices for mitigation of highway stormwater runoff pollution, Vol. III*. Quoted in Peterson and Batley, 1992.
- MALMQUIST, P. (1978). *Atmospheric fallout and street cleaning – effects on urban stormwater and snow*. Progress in Water Technology, 10 (5/6), pp 495 – 505.
- MARSALEK, J. (1992). *Overview of sediment issues in urban drainage*. Keynote Address, I.E.Aust. 1st International Symposium on Urban Storm Drainage, Sydney.
- MASTERS, G.M. (1998). *Introduction to environmental engineering and science*. Prentice Hall, Sydney.
- McCOMB, A.J. and LAKE, P.S. (1990). *Australian wetlands*. Angus & Robertson, North Ryde, NSW.
- MELBOURNE WATER (2005). *WSUD engineering procedures – stormwater*. CSIRO Publishing, Collingwood, Victoria.
- MIKKELSEN, P.S., HAFLIGER, M., OCHS, M., JACOBSEN, P., TJELL, J.C. and BOLLER, M. (1997). *Pollution of soil and groundwater from infiltration of highly contaminated stormwater – a case study*. Water Science and Technology, Pergamon, Vol. 36 (8-9), p 325-330.
- MILLIS, N.F. (2003). *Water quality relative to end use*. Australian Journal of Water Resources, Vol. 7, No 1.
- MOTTIER, V., BRISSAUD, F., NIETO, P. and ALAMY, Z. (2000). *Wastewater treatment by infiltration percolation: a case study*. Water Science and Technology, Pergamon, Vol. 41 (1), 77-84.
- MOURITZ, M., EVANGELISTI, M. and McALISTER, T. (2003). *Water sensitive urban design*. Chapter 4 of Australian Runoff Quality – DRAFT (Engineers Australia, 2003), Canberra, June.
- NAPIER, F. and JEFFERIES, C. (2003). *Heavy metals removal in Formpave block paving – report of a field study*. Report G25/2 prepared for Formpave Ltd. by Urban Water Technology Centre, University of Abertay, Dundee, Scotland, April.
- NEHRKE, S.M. and ROESNER, L.A. (2004). *Effects of design practice for flood control and BMPs on the flow-frequency curve*. ASCE Journal of Water Resources Planning & Management, March/April, pp 131 – 139.
- NEVILLE JONES & PARTNERS (1993). *Queensland urban drainage manual*. Prepared for Water Res Comm, LGEA of Queensland and Brisbane City Council, by Neville Jones & Partners and Australian Water Engineering, Brisbane.
- NHMRC/ARMCANZ (2001). *Australian drinking water guidelines*. National Health & Medical Research Council and Agriculture & Resource Management Council of Australia and New Zealand. AGPS, Canberra.
- NORTHERN ADELAIDE AND BAROSSA CWMB (2000). *Water allocation plan – Northern Adelaide Plains prescribed wells area*. Catchment Water Management Plan – Volume 3, December, Salisbury.
- N.S.W. DEPARTMENT OF HOUSING (1998). *Managing urban stormwater : soils and construction*. 3rd Edition, NSW Dept of Housing, Sydney.
- N.S.W. EPA (1996). *Managing Urban Stormwater : Strategic Framework – Draft 0 7 310 3810 X*. Chatswood, NSW.

- N.S.W. EPA (1997). *Managing Urban Stormwater : Treatment Techniques*. ISBN 07310 3886 X, Chatswood, NSW.
- N.S.W. EPA (1999). *Managing Urban Stormwater : Source Control DRAFT*. Chatswood, NSW, January.
- N.S.W. EPA (2001). *What is urban stormwater?* Webpage : http://www.epa/nsw/gov.au/stormwater/what_is/index.htm.
- O'LOUGHLIN, G. and STACK, R. (2001). *DRAINS user manual*. Watercom Pty. Ltd., Sydney.
- ONTARIO MINISTRY OF THE ENVIRONMENT (2003). *Stormwater management planning and design manual*. Available www.ene.gov.on.ca/envision/gp/4329eindex.htm. ISBN 0-7794-2969-9.
- PAVELIC, P., GERGES, N., DILLON, P. and ARMSTRONG, D. (1992). *The potential for storage and reuse of Adelaide's stormwater runoff using the upper Quaternary groundwater system*. Centre for Groundwater Studies, Report No. 40, Adelaide, April.
- PEARCE, F. (2004). *We can't hold back the water anymore*. New Scientist, 10 January, pp 26 – 29.
- PENSYL, K. and CLEMENT, P.E. (1987). *Results of the State of Maryland infiltration practice survey*. Maryland Dept. of Environment (Sediment and Stormwater Division), Baltimore.
- PETERSON, S.M. and BATLEY, G.E. (1992). *Road runoff and its impact on the aquatic environment : a review*. CSIRO Investigation Report CET/LH/IRO76, AGPS, Canberra.
- PEZZANITI, D., ARGUE, J. and JOHNSTON, L. (2003). *Detention/retention storages for peak flow reduction in urban catchments : effects of spatial deployment of storages*. Australian Journal of Water Resources (Engineers Australia), Vol. 7, No. 2, Engineers Media, Crows Nest, NSW.
- PITT, R., ROBERTSON, B., BARRON, P., AYYOUBI, A. and CLARK, S. (1999). *Stormwater treatment at critical areas : the multi-chambered treatment train (MCTT)*. USEP A wet weather flow management program, National Risk Management Research Laboratory. EPA/600/017. Cincinnati, Ohio, March.
- PITT, R. (2001). *Full-scale tests of the multi-chambered treatment train (MCTT)*. In Linking Stormwater BMP Designs and Performance to Receiving Water Impact Mitigation, B. Urbonas (Ed.), ASCE, Reston, Virginia.
- PRATT, C., NEWMAN, A. and BROWNSTEIN, J. (1996). *Bio-remediation processes within a permeable pavement : initial observations*. Proc. 7th Int'l Conf. on Urban Storm Drainage, Hannover, September.
- PRATT, C.J. (1997). *Design guidelines for porous/permeable pavements*. Proc. ASCE Engineering Foundation Conference "Stormwater Management – creating sustainable urban water resources for the 21st Century", Malmo, Sweden, September.
- PRATT, C.J., NEWMAN, A.P. and BOND, P.C. (1998). *Mineral oil bio-degradation within a permeable pavement : long-term observations*. Proc. 3rd International Conf. on Innovative Technologies in Urban Storm Drainage, Graie, Lyon, May, Vol. 1, pp 501 – 507.
- PRATT, C.J., NEWMAN, A.P. and COUPE, S.J. (2001). *Studies in the mechanisms of bio-remediation within permeable pavement constructions*. Proc. 4th International Conf. on Innovative Technologies in Urban Storm Drainage, Graie, Lyon, June.
- RANDALL, C.W. and GRIZZARD, T.J. (1981). *Comparison of pollutant mass loads in precipitation and runoff in urban areas*. Proc. 2nd International Conf. on Urban Storm Drainage, Urbana, Illinois.
- RIGBY, T., BOYD, M., RUSSO, S. and Van DRIE, R. (2003). *Storms, stormbursts and flood estimation : a need for review of the AR&R procedures*. Proc. 28th I.E.Aust. Hydrology and Water Resources Symposium, Wollongong, November.
- RILEY, S., (2001). Proc. SIA Workshop, "On-site retention : why? when? how?", Concord, NSW, 30 May.

- ROMMEL, M., RUS, M., ARGUE, J., JOHNSTON, L. and PEZZANITI, D. (2001). *Hydrological and 'lifespan' performances of "Formpave" permeable paving*. Proc. I.E.Aust./IPWEA International Public Works Conference, Perth, WA, August.
- SANSALONE, J. and TENG, Z. (2004). *In situ partial exfiltration of rainfall runoff I – quality and quantity attenuation*. ASCE Journ. of Environmental Engineering, Vol. 130, No 9, September 1.
- SCHUELER, T.R. (1987). *Controlling urban runoff : a practical manual for planning and designing urban best management practices*. Metropolitan Washington Council of Governments, Washington, DC.
- SCHUELER, T.R., KUMBLE, P.A. and HERATY, M.A. (1992). *A current assessment of urban best management practices – techniques for reducing non-point source pollution in the Coastal Zone*. Dept. of Environmental Programs, Metropolitan Washington Council of Governments, Washington, DC, March.
- SCHUELER, T.R. (1992a). *Design of stormwater wetland systems : guidelines for creating diverse and effective stormwater wetlands in the Mid-Atlantic Region*. Metropolitan Washington Council of Governments, Washington, DC.
- SCHUELER, T.R. (1992b). *Watershed restoration sourcebook – mitigating the adverse effects of urbanisation on streams*. Metropolitan Washington Council of Governments, Washington, DC.
- SCHUELER, T.R. (1995). *Site planning for urban stream protection*. Metropolitan Washington Council of Governments and Centre for Watershed Protection, Washington, DC.
- SCOTT, P., SANTOS, R. and ARGUE, J. (1998). *Performance, environmental and cost comparisons of OSD and OSR in re-developed catchments*. Proc. I.E.Aust. Symposium "HydraStorm '98", Adelaide, October.
- SEQ HEALTHY WATERWAYS PARTNERSHIP (2007). *South-east Queensland WSUD Guidelines (draft)*. Queensland Environment Protection Agency, Brisbane, November.
- SHACKEL, B. (2000). *Computer-based mechanistic methods for concrete block pavement design*. Proc. 6th International Conference on Concrete Block Paving, Tokyo.
- SHACKEL, B., BALL, J. and MEARING, M. (2003). *Using permeable Eco-paving to achieve improved water quality for urban pavements*. Proc. 7th International Conference on Concrete Block Paving.
- SIRIWARDENE, N.R., DELETIC, A. and FLETCHER, T.D. (2007). *Modelling of sediment transport through stormwater gravel filters over their lifespan*. Environmental Science & Technology, Vol. 41, No. 23, American Chemical Society, pp 8099 – 8103.
- SMAKHTIN, V. (2001). *Low flow hydrology: a review*. Journ. of Hydrology, Vol. 240, pp 147 – 186.
- SOMES, N.L. and WONG, T.H.F. (2000). *Detention time distribution of constructed stormwater wetlands*. Australian Journal of Water Resources (Engineers Australia), Vol. 4, No. 2, Engineers Media, Crows Nest, NSW.
- STANDARDS AUSTRALIA (2003). *Plumbing and drainage – Pt. 3; stormwater drainage*. Standards Australia International Limited, Sydney and Wellington, December.
- STEPHENS, K., REID, D. and GRAHAM, P. (2002). *Stormwater planning : a guidebook for British Columbia*. Greater Vancouver Regional District, Vancouver, BC
- STOCKDALE, E.C. (1991). *Freshwater wetlands, urban stormwater and non-point pollution control : a literature review and annotated bibliography*. 2nd Edition, Washington State Dept. of Ecology, Olympia.
- SWAMEE, P.K. and TYAGI, A. (1996). *Design of Class-I sedimentation tanks*. ASCE Journal of Environmental Engineering, Vol. 122, No. 1, January, pp 71 – 73.
- TONKIN, B.C. & ASSOC. (1996). *Comprehensive catchment water management plan : accompanying report*. Patawalonga Catchment Water Management Board, Adelaide, September.

- UPRCT (2004). *Water sensitive urban design technical guidelines for Western Sydney*. Final Report prepared by URS for Upper Parramatta River Catchment Trust, Parramatta, April.
- UPRCT (2005). *On-site stormwater detention Handbook*. 4th Edition, Upper Parramatta River Catchment Trust, Parramatta.
- URBONAS, B.R. (1999). *Design of a sand filter for stormwater quality enhancement*. Water Environment Research, Vol. 71, No. 1, January/February, Washington, DC, pp 102 – 113.
- URBONAS, B.R. and STAHERE, P. (1993). *Stormwater best management practices and detention for water quality, drainage and CSO management*. Prentice Hall, Eaglewood Cliffs.
- URBONAS, B.R., ROESNER, L.A. and GUO, C.Y. (1996). *Hydrology for optimal sizing of urban runoff treatment control systems*. Journal of Water Quality International, London, January/February.
- URRIOLA, H. (1999). *Roof gardens – an environmental asset*. “Waterfall”, Journal of Stormwater Industry Association Ltd, Issue 13, Spring, Sydney.
- US EPA (1987). *The enhanced stream water quality model QUAL2E and QUAL2E-UNCAS, documentation and users manual*. U S Environment Protection Agency, Washington, DC.
- US EPA (1993). *Pollution prevention fact sheet : bridge and roadway maintenance*. Criteria and Standards Division, U.S. Environment and Protection Agency, Washington, DC.
- US DEPT. OF TRANSPORTATION (1996). *Urban drainage design manual – hydraulic engineering circular No. 22*. U.S. Dept of Commerce – National Technical Information Service, Washington, DC, November.
- UWRC (1993). Brochure announcing establishment of Urban Water Resources Centre, Faculty of Natural and Built Environment, University of South Australia, Adelaide, April.
- UWRC (1999). *Arterial road runoff – quality, effects and control : a literature review*. Urban Water Resources Centre, Report to Transport SA (Ms A. Welsh), Authors – J. Argue, D. Pezzaniti, C. Jenkins, J. Gorrie, Adelaide, June.
- UWRC (2002a). Research into ‘effective life’ of permeable pavement source control installation. Study supported by Catchment Management Subsidy Scheme. Website : <http://www.unisa.edu.au/uwrc/permeable.htm>.
- UWRC (2002b). Study into permeable pavements for water quality improvement at Port Adelaide-Enfield Council car park. Study supported by Natural Heritage Trust. Website : <http://www.unisa.edu.au/uwrc/permeable.htm>.
- Van der WEL, B. (2000). *Roof runoff – maximising its use*. Proc. 10th World Water Congress, Melbourne, March.
- Van der WERF, E., ARGUE, J.R. and PEZZANITI, D. (1999). *Some unexpected results from infiltration tests in shallow clay over rock*. Proc. 8th Int’l Conf. on Urban Storm Drainage, I.E.Aust. IAHR/IAWQ, Sydney, August/September.
- WACKERNAGEL, M. and REES, W. (1996). *Our ecological footprint : reducing human impact on the Earth*. The new catalyst bioregional series, Vol. 9, Gabriola Island, BC, and Philadelphia, PA, New Society Publishers.
- WALKER, D., PASSFIELD, F., PHILLIPS, S., BOTTING, J. and PITRANS, H. (1997). *Stormwater sediment properties and land use in Tea Tree Gully, South Australia*. Proc. 17th AWWA Federal Convention, Melbourne.
- WALSH, C.J., LEONARD, A.W., LADSON, A.R. and FLETCHER, T.D. (2004). *Urban stormwater and the ecology of streams*. CRC for Freshwater Ecology and CRC for Catchment Hydrology, Canberra.

- WALSH, C.J., FLETCHER, T.D. and LADSON, A.R. (2009). *Retention capacity – a metric to link stream ecology and stormwater management*. Proc ASCE Journ of Hydrologic Engineering, Vol 14 No 4, April 1.
- WATER & RIVERS COMMISSION, W.A. (1998). *A manual for managing stormwater quality in Western Australia*. Water and Rivers Commission, Perth.
- WAWA (1987). *Domestic water study in Perth, Western Australia – Working Papers*. Water Authority of Western Australia, Perth.
- WEISNER, M., CHARACKLIS, G. and BREJCHOVA, D. (1998). *Metals and colloids in urban runoff*. In “Metals in Surface Waters” (Editors H. Allen, A. Garrison, G. Luther), Sleeping Ann Arbor Press, Chelsea, Michigan.
- WEISS, G.D., HONDZA, M. and SEMMENS, M. (2006). *Stormwater detention ponds modelling heavy metal removal by plant species and sediments*. ASCE Journal of Environmental Engineering, Vol. 132, No. 9, September 1.
- WHELANS and HALPERN GLICK MAUNSEL (1994). *Planning and management guidelines for water sensitive urban (residential) design*. Dept. of Planning and Urban Development, Water Authority of WA and Environment Protection Authority, Perth, WA, June.
- WOLMAN, M.G., and SCHICK, A.P. (1962). *Effects of construction on fluvial sediment in urban and suburban areas of Maryland*. Water Resources Research, Vol. 3.
- WONG, T.H.F. (2000). *Source control and water sensitive urban design*. Proc. 1st National Conf. on Water Sensitive Urban Design, Melbourne Water, Melbourne, August.
- WONG, T., WOOTTON, R., ARGUE, J. and PEZZANITI, D. (2000). *Bringing order to the pollution control industry : issues in assessing the performance of gross pollution traps*. I.E.Aust. Civil Engineering Transactions, Vol. CE 42, 2000.
- WONG, T.H.F., WOOTTON, R.M. and FABIAN, M D. (1995). *Investigation of the hydraulic performance of the ‘Pollutec’ continuous deflective separation gross pollution trap, Stage 2 report : operating characteristics*. Monash University, Dept. of Civil Engineering, Melbourne, October.
- WRIGHT-McLAUGHLIN, ENGINEERS (1969). *Urban storm drainage*. Criteria manual, volumes 1 and 2. Denver Regional Council of Governments, Denver, USA, March.
- WSROC (Western Sydney Regional Organisation of Councils, 2003). *Western Sydney salinity code of practice*. Prepared by R. Nicholson for Western Sydney Salinity Working Party, WSROC, March.
- WSUD in the Sydney Region (2003). *Water sensitive planning guide for the Sydney region*. Upper Parramatta River Catchment Trust, Parramatta.

APPENDIX A

MEASUREMENT OF HYDRAULIC CONDUCTIVITY IN SOILS FOR USE IN WATER-SENSITIVE DESIGN

INTRODUCTION

One of the most basic techniques included in ‘source control’ practices is that of retaining storm runoff in “leaky” devices such as perforated wells and gravel-filled trenches or “soakaways”. Use of these systems in sandy or sandy-clay soils is well established throughout the world. Their extension into less permeable clay soils needs to be approached with caution, but successful systems constructed in such soils are known in Europe, Japan and in South Australia.

The following notes have been compiled from experience in South Australia and in Auckland (“Stormwater Soakage Design Manual” by P. Nagels) : the latter reference is acknowledged with gratitude.

Soakage Report

Details to be covered in the Report include :

- weather conditions preceding and during the field test : the full test duration may spread over two or three days;
- soil profile : bore log with apparent permeability of the strata encountered, including water table;
- apparent long-term water table (if different from above);
- pre-soaking procedure : duration of pre-soaking.
- data from ‘constant head’ test (head $\approx h_0$) and calculation of hydraulic conductivity from these data;
- graph of ‘falling head’ test (time, t vs depth, h relationship);
- calculation of hydraulic conductivity at time = 60 minutes (clay soils) or depth = $0.85 h_0$ in sandy soils;
- plan of site showing bore positions in relation to buildings, existing soakholes, driveways, overland flow path, boundaries, contours or spot levels, any area intended for on-site retention, future site development, etc.;
- supply a dimensioned drawing showing details of the proposed “leaky” device or system;
- describe the consequences of the “leaky” device or system overflowing, i.e. effect on foundations, neighbouring properties, site stability, etc.; and,
- report signed and dated by either a civil engineer, civil engineering technician, or an engineering geologist experienced in soakage systems.

The bore hole

A minimum of two tests shall be carried out on each site. On large sites a minimum of one test should be carried out per 400 m^2 of site area. Each hole should correspond with the site position(s) of the proposed soakage devices.

Test holes of 100 mm diameter should be bored to a depth of 2.5 m in soil where rock is not encountered; otherwise, terminate drilling at the soil/rock interface. These depths correspond to the dimension, h_0 . The soil profile should be recorded as excavation proceeds and presented with the soakage report.

The hole should be prepared by carefully scratching the sides with a sharp-pointed tool to remove any smeared soil surfaces and to provide a natural soil interface through which water may infiltrate.

The hole must be lined with perforated or slotted PVC pipe (at least 20 holes per metre length of pipe). In sandy soil the perforations/slots should be covered (outside) with geotextile to prevent soil collapse into the bore through the slots or perforations. Any gap between the PVC liner and the wall should be filled with clean sand.

Boreholes drilled at sites where surfaces are waterlogged, indicating the presence of free surface water, should be protected from the entry of surface water during tests. This can be accomplished by forming a low soil mound 50 – 70 mm high around the top of the hole or driving a PVC “ring”, say 300 mm diameter, into the soil at the top of the hole. The ring should be symmetrical about the hole and present a 50 – 70 mm “wall” to surface runoff local to the borehole.

It is of vital importance to record groundwater level if encountered before the 2.50 m depth is reached. In heavy clay soils it is necessary to allow the hole to 'stand' for one or two days after drilling to ensure that the long-term groundwater level is clearly identified.

Pre-soak

After the long-term groundwater level has been established (where a watertable is encountered), the well should be filled with water and left to stand for 24 hours (clay soils). "Pre-soaking" in sandy soils consists of filling and allowing the borehole to drain three times.

Constant head test

The 'constant head' test seeks the flow rate of water into the bore sufficient to maintain water depth constant at h_0 . In fact, despite pre-soaking, there will be a gradual decline in the quantity of inflow required to maintain water depth **constant at h_0** .

In sandy soils, the flow rate is usually small but comfortably supplied from a one litre flask : the time taken to empty each one litre of input must be recorded.

In clay soils, the inflow rate is likely to be very small and the task of maintaining a constant water depth not easy to accomplish. An acceptable procedure for achieving a 'constant head' test result under these circumstances is to allow the water level to fall over a period of, say, 15 – 30 minutes and then to fill the hole to the initial level. The time (t_c) of water level fall should be noted as well as the Volume of water required (V_c) to restore the level.

Tests in both sandy and clay soils should be repeated at least four times.

Falling head test

The 'falling head' test follows immediately completion of the 'constant head' test and involves, simply, allowing the borehole to drain without further addition of water.

In the case of sandy soils, the rate of fall may be quite rapid and the task of recording water depth below the top of the PVC liner (used as datum) and the corresponding time, is a two-person operation. In such circumstances an acceptable depth vs time curve may be obtained from the average of three or more 'runs'. A light staff with scale supported by a float can be used to obtain more accurate readings of borehole water depth.

In clay soils the rate of fall may be very slow and early values hard to distinguish. This is of little consequence as the most critical time/depth conditions are those around $t = 60$ minutes, in particular, between $t = 45$ minutes and $t = 75$ minutes. Some 'scatter' of points is to be expected and a curve of best fit should be applied to the data. Again, the depth of water in the borehole can be obtained from measurements taken from the top of the PVC liner used as datum or, for greater accuracy, with a scale supported by a float. A typical t versus h curve is shown in Figure A-1.

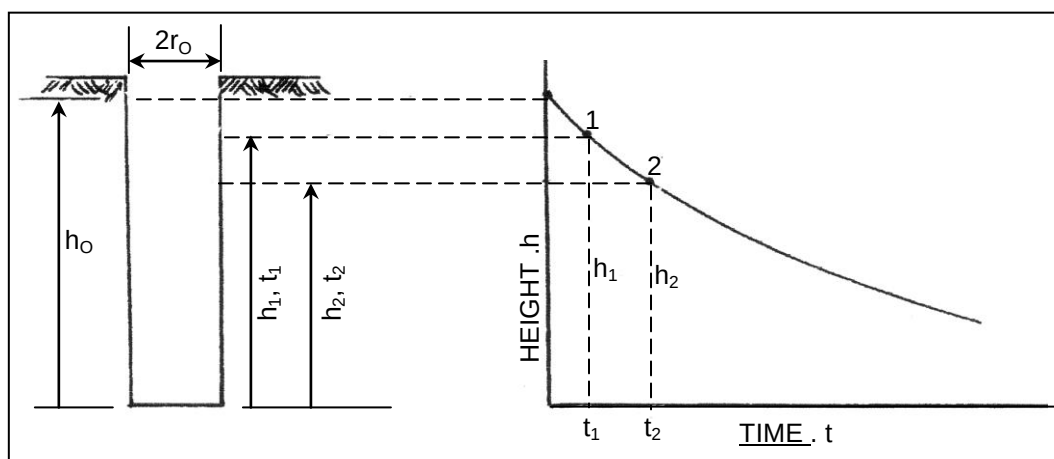


FIGURE A-1 : Inverse augerhole test – definition of terms and typical 'h' versus 't' relationship

ANALYSIS OF RESULTS

Constant head test : The hydraulic conductivity of soil given by the constant head test comes from calculating the average flow rate, $Q = V_c / t_c$ expressed in units of m^3 per second, and substituting this into the formula :

$$k_h = \frac{Q}{[\pi r_o^2 + 2\pi r_o h]}$$

where r_o = radius of the **borehole** (not PVC liner) in metres;

h_o = initial water depth in metres.

Falling head test : In this case, data from two water depth conditions taken from the ‘falling head’ curve illustrated in Figure A-1 are required. In the case of clay soils :

$$t_1 = 45 \text{ minutes} = 2,700 \text{ seconds; and, } t_2 = 75 \text{ minutes} = 4,500 \text{ seconds.}$$

The corresponding depths, h_1 and h_2 are measured in metres.

The values for t_1 , t_2 , h_1 and h_2 are substituted into the following expression :

$$\text{Hydraulic conductivity } k_h = \frac{1.15r_o}{(t_2 - t_1)} \log_{10} \left[\frac{h_1 + \frac{r_o}{2}}{h_2 + \frac{r_o}{2}} \right] \text{ m/s} \quad (3.13)$$

In the case of sandy soils which show very rapid fall in water level, two positions corresponding to t_1 and t_2 should be chosen such that the average of the corresponding values of h_1 and h_2 is about $0.85 h_o$. The same formula as above may be used, this time for the chosen values of t_1 and t_2 (in seconds) and corresponding values of h_1 and h_2 in metres.

Where the test borehole intercepts groundwater, the depths h_o , h_1 and h_2 are measured from the **long-term** level of the groundwater (see Figures A-2). Substitution t_1 , t_2 , h_1 and h_2 , similar to the above, is then made into the following formula :

$$\text{Hydraulic conductivity, } k_h = \frac{1.15r_o}{(t_2 - t_1)} \log_{10} \left[\frac{h_1}{h_2} \right] \quad (3.14)$$

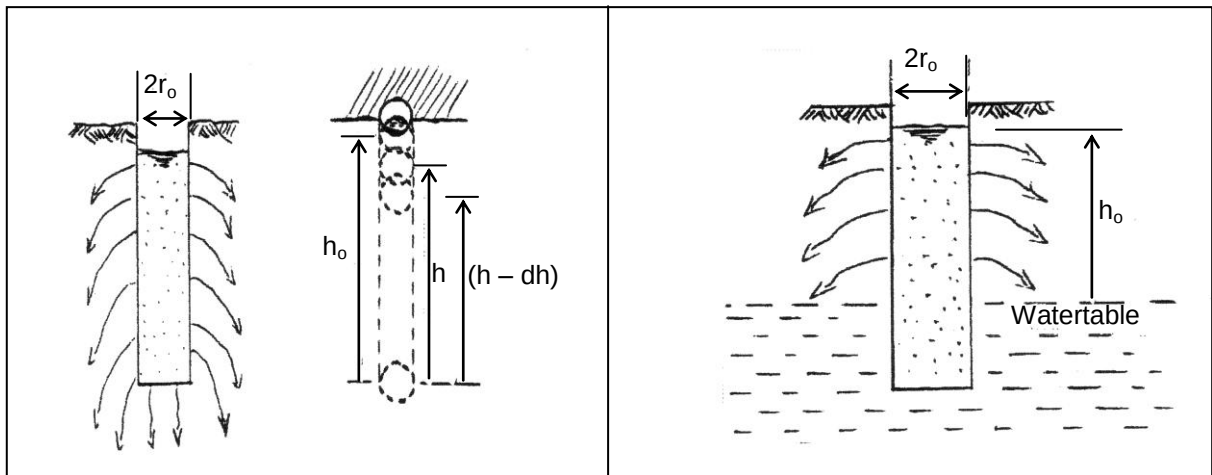


FIGURE A-2 : Boreholes with watertable intersection : definition of h_o and other water depths

HYDRAULIC CONDUCTIVITY RANGES FOR COMMON SOILS

The values of ' k_h ' derived from the above processes should fall into the following ranges for commonly encountered soils :

Sandy soil :	$k_h > 5 \times 10^{-5}$ m/s;
Sandy clay :	k_h between 1×10^{-5} and 5×10^{-5} m/s;
Medium clay :	k_h between 1×10^{-6} and 1×10^{-5} m/s;
Heavy clay :	k_h between 1×10^{-8} and 1×10^{-6} m/s.

APPENDIX B

RAINFALL I-F-D CHARTS FOR TYPICAL NORTHERN, INTERMEDIATE AND SOUTHERN AUSTRALIAN LOCATIONS

The Tables of IFD data included in Appendix B are representative only of rainfall in the stated locations, and are provided for illustrative purposes only and for use in tutorial exercises included in Appendix F. **Under no circumstances should these data be used for design purposes in the Adelaide, Newcastle or Brisbane regions.** Competent design requires the use of IFD data for specific locations with a resolution of 2.5 km, available from the Bureau of Meteorology or other competent sources. (See www.bom.gov.au/hydro/has/ifd.shtml and follow the link to IFD Request Form.)

TABLE B-1
RAINFALL INTENSITY DURATION DATA
ADELAIDE, SOUTH AUSTRALIA

Geographic Location : 34.9333° South; 138.6° East

AUSIFD Version 2.0

DURATION	1 YEAR ARI (mm/hour)	2 YEAR ARI (mm/hour)	5 YEAR ARI (mm/hour)	10 YEAR ARI (mm/hour)	20 YEAR ARI (mm/hour)	50 YEAR ARI (mm/hour)	100 YEAR ARI (mm/hour)
Minutes :							
5	42.9	58.0	81.0	98.0	121.0	155.0	186.0
6	40.0	54.0	75.0	91.0	112.0	144.0	172.0
7	37.6	51.0	70.0	85.0	105.0	135.0	161.0
8	35.5	47.9	67.0	80.0	99.0	127.0	151.0
9	33.8	45.5	63.0	76.0	94.0	120.0	143.0
10	32.2	43.3	60.0	72.0	89.0	114.0	136.0
11	30.8	41.5	57.0	69.0	85.0	109.0	129.0
12	29.6	39.8	55.0	66.0	81.0	104.0	124.0
13	28.5	38.3	53.0	64.0	78.0	100.0	119.0
14	27.4	36.9	51.0	61.0	75.0	96.0	114.0
15	26.5	35.6	49.1	59.0	72.0	93.0	110.0
16	25.7	34.5	47.5	57.0	70.0	89.0	106.0
17	24.9	33.4	46.0	55.0	68.0	86.0	103.0
18	24.2	32.4	44.6	53.0	66.0	84.0	99.0
19	23.5	31.5	43.3	52.0	64.0	81.0	96.0
20	22.8	30.6	42.1	50.0	62.0	79.0	94.0
25	20.2	27.1	37.1	44.4	54.0	69.0	82.0
30	18.2	24.4	33.6	39.9	48.7	62.0	73.4
35	16.7	22.3	30.6	36.3	44.3	56.3	66.6
40	15.4	20.6	28.1	33.4	40.7	52.0	61.0
45	14.4	19.2	26.1	31.0	37.8	47.9	57.0
50	13.5	18.0	24.4	29.0	35.3	44.7	53.0
55	12.7	17.0	23.0	27.3	33.2	41.9	49.5
Hours :							
1.0	12.0	16.1	21.7	25.8	31.3	39.6	46.6
1.5	9.4	12.5	16.8	19.9	24.0	30.2	35.5
2.0	7.9	10.5	14.0	16.4	19.8	24.8	29.1
3.0	6.1	8.1	10.7	12.5	15.1	18.8	21.9
4.0	5.1	6.7	8.9	10.4	12.4	15.4	17.9
5.0	4.4	5.8	7.7	8.9	10.7	13.2	15.4
6.0	4.0	5.2	6.8	7.9	9.4	11.6	13.5
7.0	3.6	4.7	6.1	7.1	8.5	10.5	12.1
8.0	3.3	4.3	5.6	6.5	7.8	9.6	11.1
9.0	3.1	4.0	5.2	6.0	7.2	8.8	10.2
10.0	2.88	3.76	4.86	5.62	6.67	8.20	9.47
11.0	2.71	3.54	4.57	5.28	6.26	7.68	8.86
12.0	2.57	3.35	4.32	4.98	5.90	7.23	8.34
14.0	2.29	3.00	3.86	4.45	5.27	6.45	7.44
16.0	2.08	2.72	3.50	4.03	4.77	5.84	6.74
18.0	1.91	2.49	3.21	3.69	4.37	5.35	6.17
20.0	1.77	2.31	2.97	3.42	4.04	4.94	5.70
24.0	1.54	2.01	2.59	2.98	3.52	4.31	4.96
36.0	1.13	1.48	1.90	2.18	2.58	3.15	3.62
48.0	0.90	1.18	1.51	1.73	2.04	2.49	2.87
60.0	0.75	0.98	1.25	1.43	1.69	2.06	2.37
72.0	0.64	0.83	1.06	1.22	1.44	1.75	2.02

The rainfall intensities shown above are calculated in accordance with Chapter 2, Australian Rainfall and Runoff – 1987 Edition.

AUS-IFD Ver.2.0, 2001 : <http://www.ens.gu.edu.au/eve/research/AusIfd/AusIfdVer2.htm>

TABLE B-2
RAINFALL INTENSITY DURATION DATA
NEWCASTLE, NEW SOUTH WALES

Geographic Location : 32.917° South; 151.8° East

AUSIFD Version 2.0

DURATION	1 YEAR ARI (mm/hour)	2 YEAR ARI (mm/hour)	5 YEAR ARI (mm/hour)	10 YEAR ARI (mm/hour)	20 YEAR ARI (mm/hour)	50 YEAR ARI (mm/hour)	100 YEAR ARI (mm/hour)
Minutes :							
5	88.0	113.0	142.0	159.0	181.0	210.0	232.0
6	83.0	106.0	133.0	149.0	170.0	197.0	218.0
7	78.0	100.0	126.0	141.0	160.0	186.0	206.0
8	74.0	95.0	119.0	133.0	152.0	177.0	195.0
9	71.0	90.0	114.0	127.0	145.0	169.0	186.0
10	68.0	87.0	109.0	122.0	139.0	162.0	178.0
11	65.0	83.0	105.0	117.0	134.0	155.0	171.0
12	63.0	80.0	101.0	113.0	129.0	149.0	165.0
13	60.0	77.0	97.0	109.0	124.0	144.0	159.0
14	58.0	75.0	94.0	105.0	120.0	140.0	154.0
15	57.0	72.0	91.0	102.0	117.0	135.0	149.0
16	55.0	70.0	89.0	99.0	113.0	131.0	145.0
17	53.0	68.0	86.0	96.0	110.0	128.0	141.0
18	52.0	66.0	84.0	94.0	107.0	124.0	137.0
19	51.0	65.0	82.0	91.0	104.0	121.0	134.0
20	49.3	63.0	80.0	89.0	102.0	118.0	130.0
25	44.1	56.0	71.0	80.0	91.0	106.0	117.0
30	40.1	51.0	65.0	72.0	83.0	96.0	106.0
35	37.0	47.3	60.0	67.0	76.0	88.5	97.5
40	34.3	43.9	56.0	62.0	71.0	82.0	91.0
45	32.2	41.2	52.0	58.0	66.0	77.0	85.0
50	30.3	38.8	49.0	55.0	63.0	73.0	81.0
55	28.7	36.8	46.5	52.0	59.0	69.0	76.0
Hours :							
1.0	27.3	35.0	44.2	49.5	57.0	66.0	73.0
1.5	21.1	27.1	34.3	38.4	43.9	51.0	57.0
2.0	17.6	22.5	28.5	32.0	36.6	42.6	47.2
3.0	13.5	17.3	21.9	24.6	28.2	32.9	36.4
4.0	11.1	14.3	18.2	20.4	23.4	27.3	30.3
5.0	9.62	12.4	15.7	17.7	20.3	23.7	26.3
6.0	8.54	11.0	14.0	15.7	18.0	21.1	23.4
7.0	7.72	9.91	12.7	14.2	16.3	19.1	21.2
8.0	7.07	9.08	11.6	13.1	15.0	17.5	19.4
9.0	6.55	8.41	10.8	12.1	13.9	16.3	18.0
10.0	6.11	7.86	10.1	11.3	13.0	15.2	16.9
11.0	5.74	7.38	9.45	10.6	12.2	14.3	15.9
12.0	5.43	6.98	8.93	10.1	11.6	13.5	15.0
14.0	4.96	6.39	8.19	9.24	10.6	12.4	13.8
16.0	4.59	5.91	7.6	8.58	9.88	11.6	12.9
18.0	4.29	5.52	7.11	8.04	9.26	10.9	12.1
20.0	4.03	5.2	6.7	7.58	8.74	10.3	11.4
24.0	3.62	4.67	6.04	6.84	7.89	9.27	10.3
36.0	2.83	3.66	4.76	5.41	6.26	7.37	8.23
48.0	2.36	3.06	3.99	4.55	5.27	6.22	6.95
60.0	2.04	2.64	3.46	3.95	4.58	5.42	6.06
72.0	1.79	2.33	3.06	3.5	4.06	4.81	5.39

The rainfall intensities shown above are calculated in accordance with Chapter 2, Australian Rainfall and Runoff – 1987 Edition.

AUS-IFD Ver.2.0, 2001 : <http://www.ens.gu.edu.au/eve/research/AusIfd/AusIfdVer2.htm>

TABLE B-3
RAINFALL INTENSITY DURATION DATA
BRISBANE, QUEENSLAND

Geographic Location : 27.4667° South; 153.0167° East

AUSIFD Version 2.0

DURATION	1 YEAR ARI (mm/hour)	2 YEAR ARI (mm/hour)	5 YEAR ARI (mm/hour)	10 YEAR ARI (mm/hour)	20 YEAR ARI (mm/hour)	50 YEAR ARI (mm/hour)	100 YEAR ARI (mm/hour)
Minutes :							
5	117.0	151.0	191.0	215.0	247.0	290.0	324.0
6	110.0	141.0	180.0	202.0	233.0	274.0	305.0
7	104.0	134.0	170.0	192.0	221.0	260.0	290.0
8	99.0	127.0	162.0	183.0	210.0	248.0	277.0
9	94.0	121.0	155.0	175.0	201.0	237.0	265.0
10	90.0	116.0	148.0	168.0	193.0	228.0	255.0
11	86.0	111.0	143.0	161.0	186.0	220.0	246.0
12	83.0	107.0	138.0	156.0	180.0	212.0	237.0
13	80.0	104.0	133.0	151.0	174.0	206.0	230.0
14	78.0	100.0	129.0	146.0	169.0	199.0	223.0
15	75.0	97.0	125.0	142.0	164.0	194.0	217.0
16	73.0	94.0	121.0	138.0	159.0	188.0	211.0
17	71.0	92.0	118.0	134.0	155.0	184.0	206.0
18	69.0	89.0	115.0	131.0	151.0	179.0	201.0
19	67.0	87.0	112.0	127.0	148.0	175.0	196.0
20	66.0	85.0	110.0	124.0	144.0	171.0	192.0
25	59.0	76.0	98.0	112.0	130.0	154.0	173.0
30	53.0	69.0	90.0	102.0	119.0	141.0	159.0
35	49.2	64.0	83.0	94.5	110.5	131.0	148.0
40	45.7	59.0	77.0	88.0	103.0	123.0	138.0
45	42.8	56.0	73.0	83.0	97.0	116.0	130.0
50	40.4	53.0	69.0	79.0	92.0	110.0	124.0
55	38.2	49.8	65.0	75.0	87.0	104.0	118.0
Hours :							
1.0	36.4	47.4	62.0	71.0	83.0	100.0	113.0
1.5	27.4	35.7	47.1	54.0	63.0	76.0	86.0
2.0	22.3	29.1	38.5	44.4	52.0	62.0	71.0
3.0	16.6	21.8	28.9	33.4	39.2	47.2	53.0
4.0	13.5	17.7	23.6	27.2	32.1	38.6	43.8
5.0	11.5	15.1	20.1	23.3	27.4	33.1	37.5
6.0	10.1	13.2	17.7	20.5	24.1	29.1	33.1
7.0	9.0	11.8	15.8	18.4	21.7	26.2	29.7
8.0	8.18	10.7	14.4	16.7	19.7	23.9	27.1
9.0	7.51	9.86	13.3	15.4	18.2	22.0	25.0
10.0	6.96	9.15	12.3	14.3	16.9	20.5	23.3
11.0	6.5	8.54	11.5	13.4	15.8	19.2	21.8
12.0	6.1	8.02	10.8	12.6	14.9	18.0	20.5
14.0	5.58	7.33	9.86	11.5	13.5	16.4	18.7
16.0	5.16	6.77	9.1	10.6	12.5	15.1	17.2
18.0	4.81	6.32	8.48	9.84	11.6	14.0	16.0
20.0	4.52	5.93	7.95	9.22	10.9	13.1	14.9
24.0	4.06	5.32	7.12	8.24	9.71	11.7	13.3
36.0	3.17	4.15	5.52	6.38	7.5	9.02	10.2
48.0	2.64	3.45	4.57	5.27	6.19	7.43	8.41
60.0	2.27	2.97	3.92	4.51	5.29	6.35	7.18
72.0	2.0	2.61	3.44	3.95	4.63	5.54	6.26

The rainfall intensities shown above are calculated in accordance with Chapter 2, Australian Rainfall and Runoff – 1987 Edition.

AUS-IFD Ver.2.0, 2001 : <http://www.ens.gu.edu.au/eve/research/AusIfd/AusIfdVer2.htm>

TABLE B-4
RAINFALL INTENSITY DURATION DATA
PERTH, WESTERN AUSTRALIA

Geographic Location : 31.95° South; 115.85° East

DURATION	1 YEAR ARI (mm/hour)	2 YEAR ARI (mm/hour)	5 YEAR ARI (mm/hour)	10 YEAR ARI (mm/hour)	20 YEAR ARI (mm/hour)	50 YEAR ARI (mm/hour)	100 YEAR ARI (mm/hour)
Minutes :							
5	59.0	78.0	103.0	120.0	145.0	181.0	213.0
6	55.0	73.0	95.0	111.0	134.0	167.0	196.0
7	52.0	68.0	89.0	104.0	124.0	155.0	181.0
8	48.7	64.0	84.0	97.0	117.0	145.0	170.0
9	46.2	61.0	79.0	92.0	110.0	137.0	159.0
10	44.0	58.0	75.0	87.0	104.0	129.0	151.0
11	42.0	55.0	71.0	83.0	99.0	123.0	143.0
12	40.3	53.0	68.0	79.0	94.0	117.0	136.0
13	38.7	51.0	65.0	76.0	90.0	112.0	130.0
14	37.2	48.8	63.0	73.0	87.0	107.0	124.0
15	35.9	47.1	61.0	70.0	83.0	103.0	119.0
16	34.8	45.5	58.0	67.0	80.0	99.0	115.0
17	33.6	44.0	56.0	65.0	77.0	95.0	111.0
18	32.6	42.6	55.0	63.0	75.0	92.0	107.0
19	31.7	41.4	53.0	61.0	72.0	89.0	103.0
20	30.8	40.2	51.0	59.0	70.0	86.0	100.0
25	27.2	35.4	44.9	52.0	61.0	75.0	86.0
30	24.4	31.7	40.1	46.0	54.0	66.0	76.0
35	22.3	28.9	36.4	41.6	48.9	59.5	68.5
40	20.5	26.6	33.4	38.0	44.6	54.0	62.0
45	19.1	24.7	30.9	35.1	41.1	49.9	57.0
50	17.8	23.0	28.8	32.7	38.2	46.2	53.0
55	16.8	21.7	27.0	30.6	35.7	43.2	49.3
Hours :							
1.0	15.9	20.5	25.4	28.8	33.6	40.5	46.2
1.5	12.3	15.8	19.5	22.1	25.7	30.9	35.2
2.0	10.2	13.1	16.2	18.2	21.2	25.0	28.9
3.0	7.83	10.1	12.3	13.9	16.1	19.2	21.9
4.0	6.49	8.31	10.2	11.4	13.2	15.8	17.9
5.0	5.61	7.18	8.75	9.81	11.3	13.5	15.3
6.0	4.98	6.36	7.75	8.67	10.0	11.9	13.5
7.0	4.5	5.75	6.99	7.81	9.02	10.7	12.1
8.0	4.12	5.27	6.39	7.14	8.23	9.78	11.1
9.0	3.82	4.88	5.91	6.6	7.6	9.02	10.2
10.0	3.57	4.55	5.51	6.14	7.08	8.39	9.48
11.0	3.35	4.28	5.17	5.76	6.63	7.86	8.97
12.0	3.17	4.04	4.88	5.43	6.25	7.41	8.36
14.0	2.87	3.66	4.43	4.95	5.7	6.77	7.64
16.0	2.63	3.36	4.08	4.56	5.26	6.25	7.07
18.0	2.44	3.12	3.79	4.24	4.9	5.83	6.6
20.0	2.28	2.91	3.55	3.98	4.6	5.48	6.2
24.0	2.02	2.59	3.17	3.55	4.11	4.91	5.57
36.0	1.54	1.98	2.43	2.74	3.19	3.83	4.35
48.0	1.26	1.62	2.0	2.27	2.64	3.18	3.63
60.0	1.07	1.38	1.71	1.94	2.26	2.73	3.13
72.0	0.93	1.2	1.49	1.7	1.99	2.4	2.75

Supply of these rainfall Intensity-Frequency-Duration data by Western Australia EPA is greatly appreciated.

APPENDIX C

POLLUTION CONTROL/RETENTION DEVICES (“soakaways”) FOR FIVE AUSTRALIAN CLIMATE ZONES

[Climate zones are based on $i_{10,1}$ rainfall intensities*]

See Figure 3.8

*** $i_{10,1}$ rainfall intensity is that given by I.E.Aust. (1987) for
ARI, Y = 10-years, duration 1-hour.**

Graph for Southern Australia is found in Figure 7.1 in Section 7.2.

$i_{10,1}$ is less than or equal to 25 mm per hour

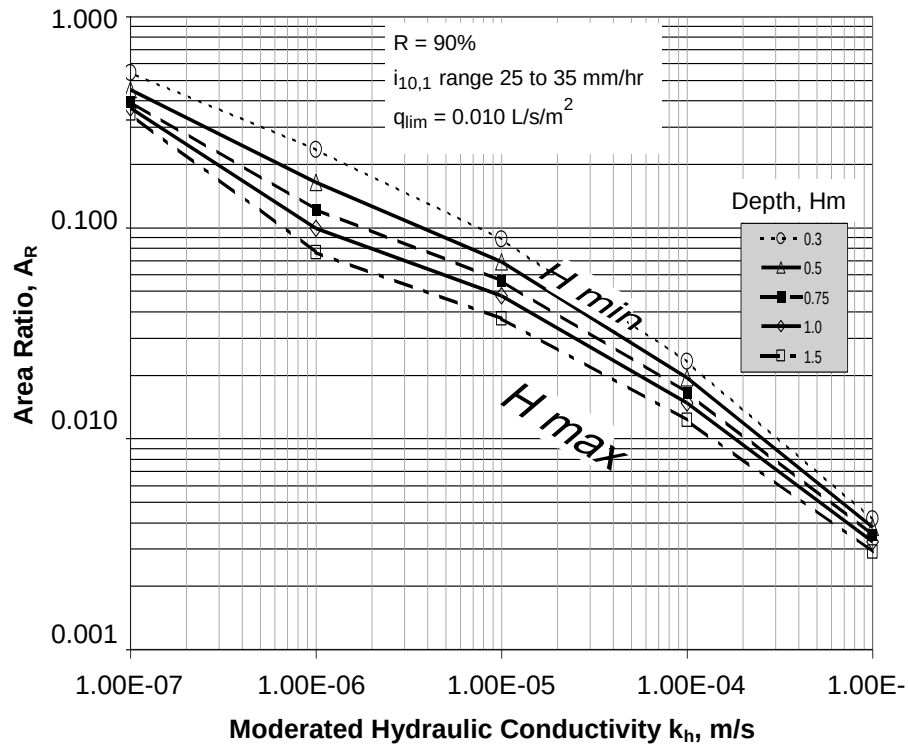


FIGURE C-1 : $i_{10,1}$ range, 25 mm/h to 35 mm/h – Lower-Intermediate Zone
 Soils with moderated $k_h > 1.0 \times 10^{-4} \text{ m/s}$ are, typically, unsuitable for treating *dissolved pollutants* in catchment runoff (see Mikkelsen et al, 1997; Fischer et al, 2003)

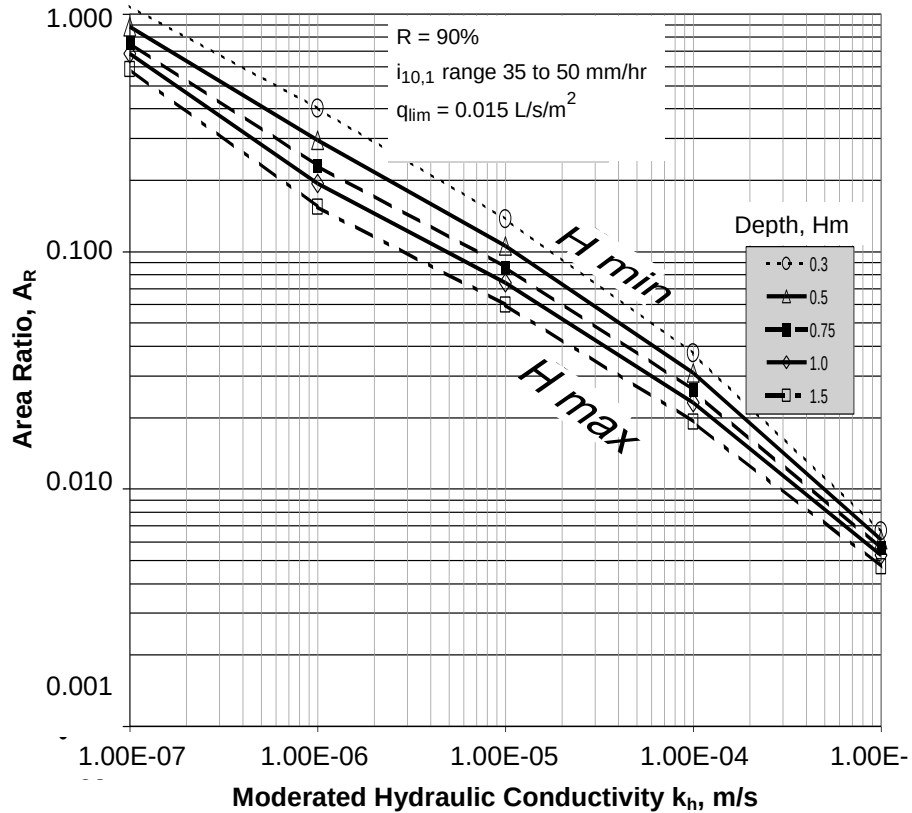


FIGURE C-2 : $i_{10,1}$ range, 35 mm/h to 50 mm/h – Mid-Intermediate Zone
 Soils with moderated $k_h > 1.0 \times 10^{-4} \text{ m/s}$ are, typically, unsuitable for treating *dissolved pollutants* in catchment runoff (see Mikkelsen et al, 1997; Fischer et al, 2003)

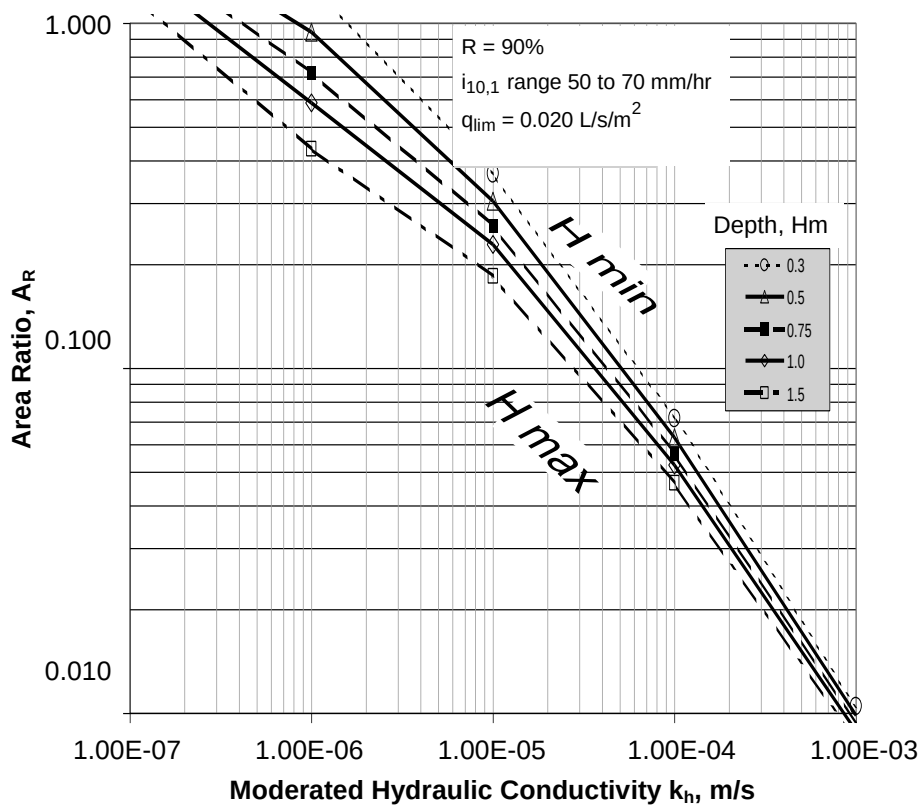


FIGURE C-3 : $i_{10,1}$ range, 50 mm/h to 70 mm/h – Upper-Intermediate Zone
Soils with moderated $k_h > 1.0 \times 10^{-4}$ m/s are, typically, unsuitable for treating *dissolved pollutants* in catchment runoff (see Mikkelsen et al, 1997; Fischer et al, 2003)

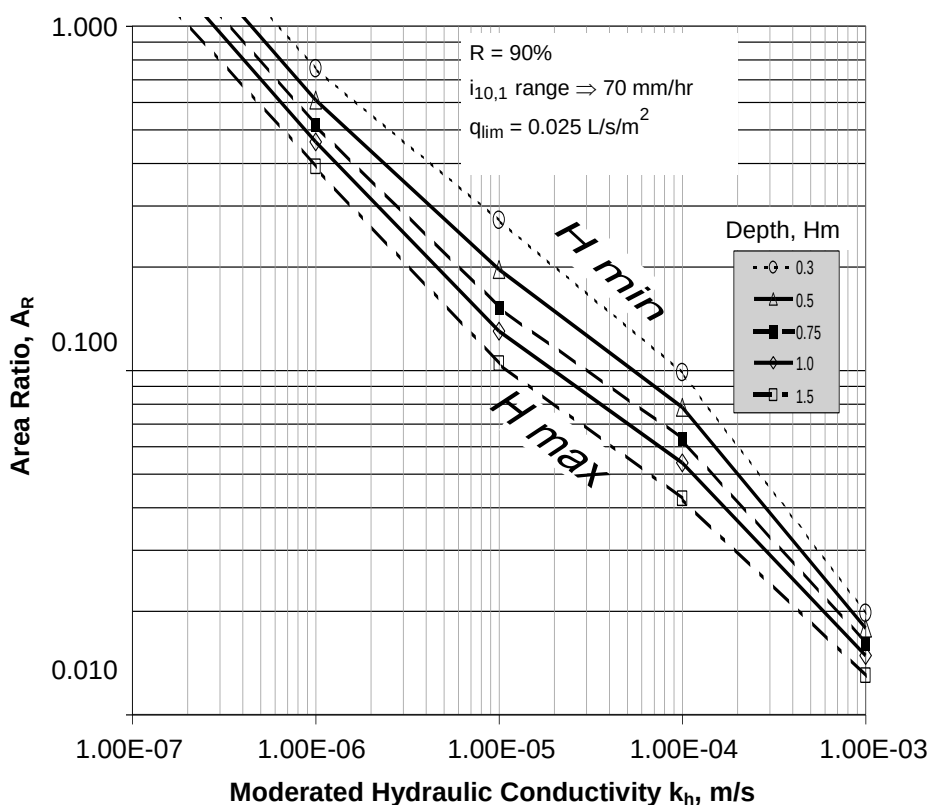


FIGURE C-4 : $i_{10,1}$ range, 70 mm/h and above – Northern Australia Zone
Soils with moderated $k_h > 1.0 \times 10^{-4}$ m/s are, typically, unsuitable for treating *dissolved pollutants* in catchment runoff (see Mikkelsen et al, 1997; Fischer et al, 2003)

APPENDIX D

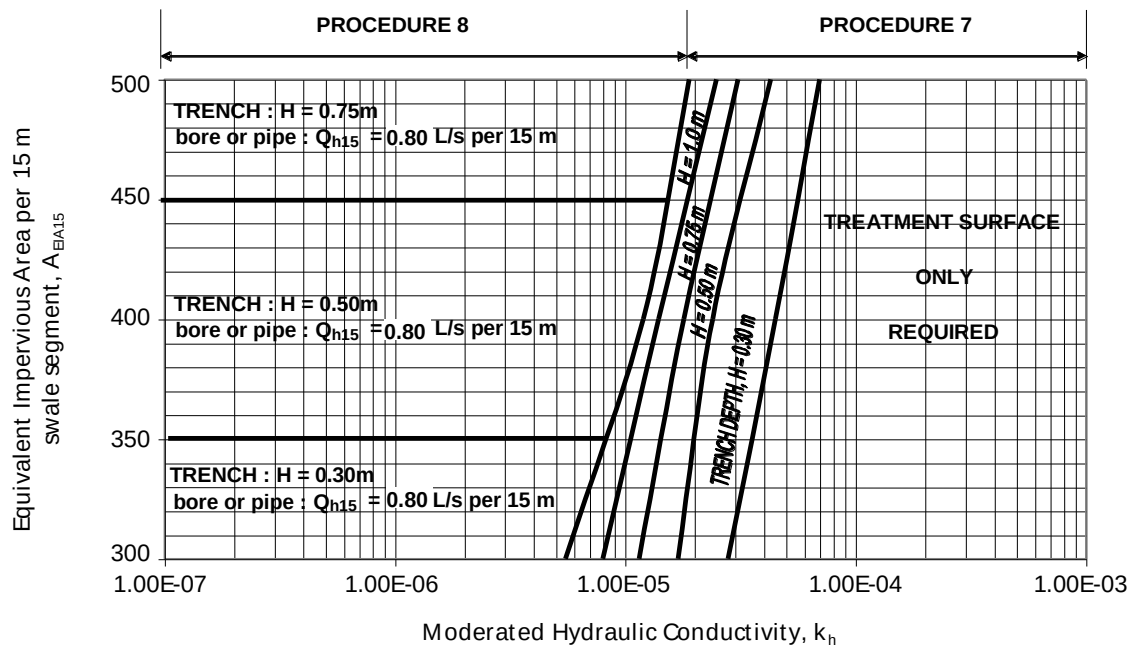
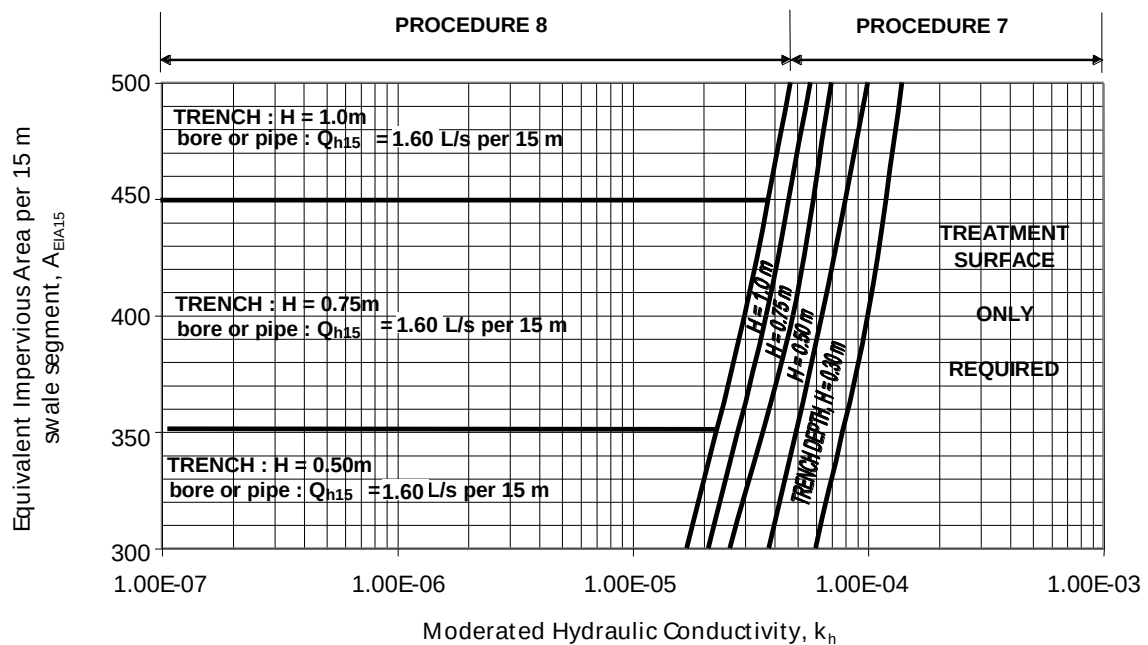
DESIGN GRAPHS FOR ‘FILTER STRIP’ SWALES FOR FIVE AUSTRALIAN CLIMATE ZONES

**[Climate zones are based on $i_{10,1}$ rainfall intensities*]
See Figure 3.8**

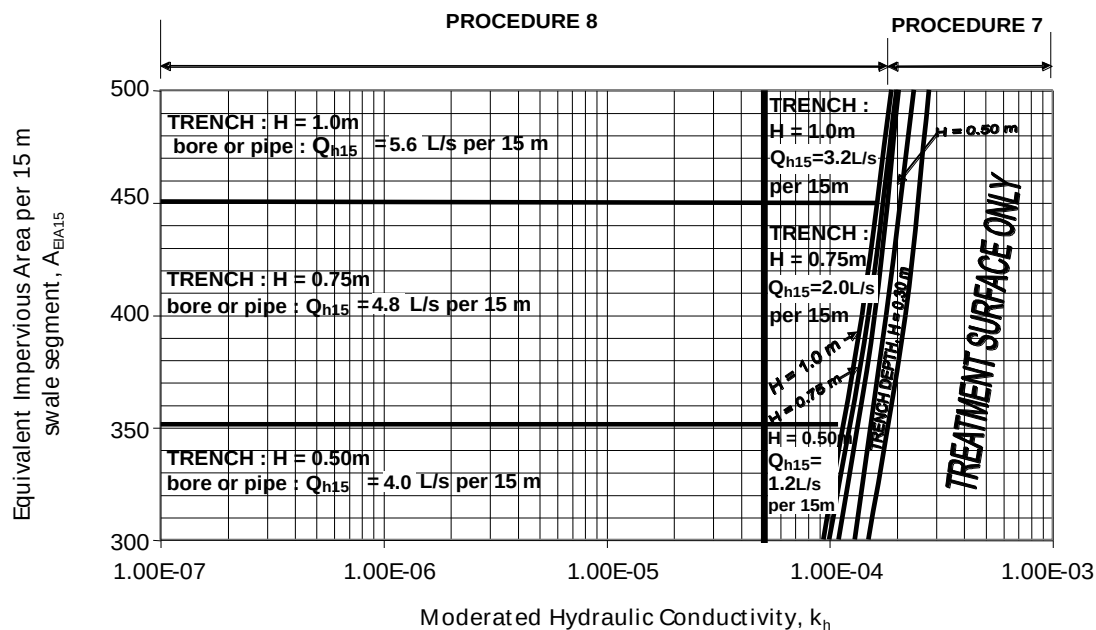
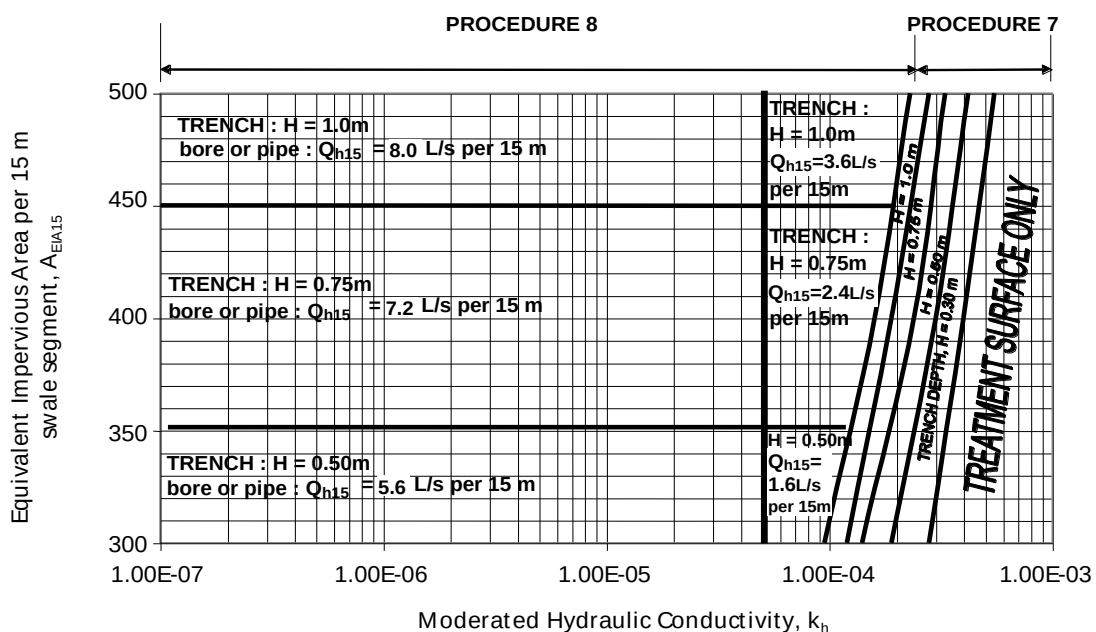
*** $i_{10,1}$ rainfall intensity is that given by I.E.Aust. (1987) for
ARI, Y = 10-years, duration 1-hour.**

Graph for Southern Australia is found in Figure 7.7 in Section 7.8.

$i_{10,1}$ is less than or equal to 25 mm per hour

FIGURE D-1 : $i_{10,1}$ range, 25 mm/h to 35 mm/h – Lower-Intermediate ZoneFIGURE D-2 : $i_{10,1}$ range, 35 mm/h to 50 mm/h – Mid-Intermediate Zone

Important Note: Soils with moderated $k_h > 1.0 \times 10^{-4}$ m/s are, typically, unsuitable for treating *dissolved pollutants* in catchment runoff (see Mikkelsen et al, 1997; Fischer et al, 2003)

FIGURE D-3 : $i_{10,1}$ range, 50 mm/h to 70 mm/h – Upper-Intermediate ZoneFIGURE D-4 : $i_{10,1}$ range, 70 mm/h and above – Northern Australia Zone

Important Note: Soils with moderated $k_h > 1.0 \times 10^{-4}$ m/s are, typically, unsuitable for treating *dissolved pollutants* in catchment runoff (see Mikkelsen et al, 1997; Fischer et al, 2003)

APPENDIX E

GRAPHS FOR A REGIONAL STORMWATER HARVESTING MODEL FOR FIVE AUSTRALIAN CLIMATE ZONES

[Climate zones are based on $i_{10,1}$ rainfall intensities*]

See Figure 3.8

*** $i_{10,1}$ rainfall intensity is that given by I.E.Aust. (1987) for
ARI, Y = 10-years, duration 1-hour.**

Graph for Southern Australia is found in Figure 8.5 in Section 8.6.

$i_{10,1}$ is less than or equal to 25 mm per hour

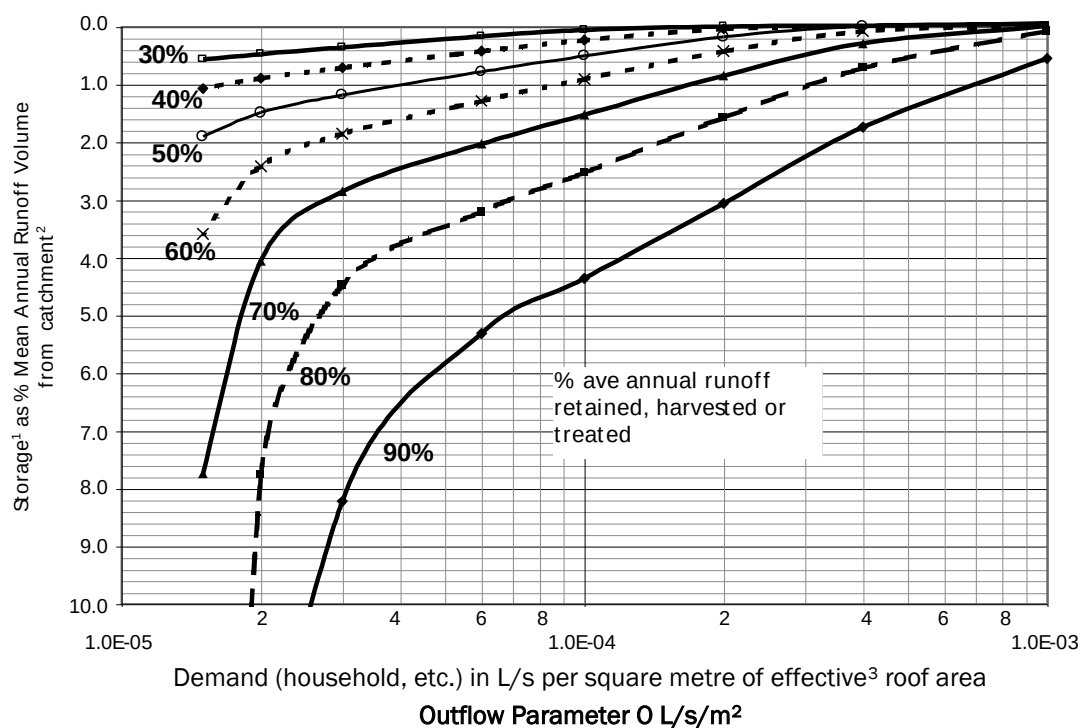


FIGURE E-1 : $i_{10,1}$ range, 25 mm/h to 35 mm/h – Lower-Intermediate Zone

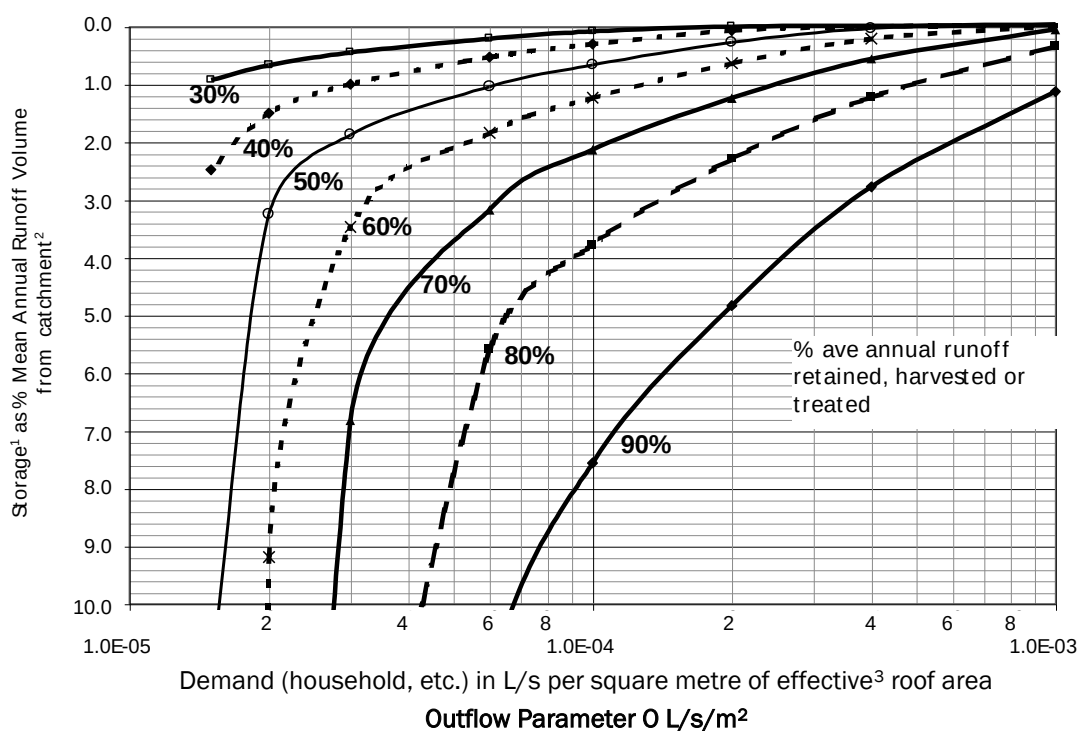
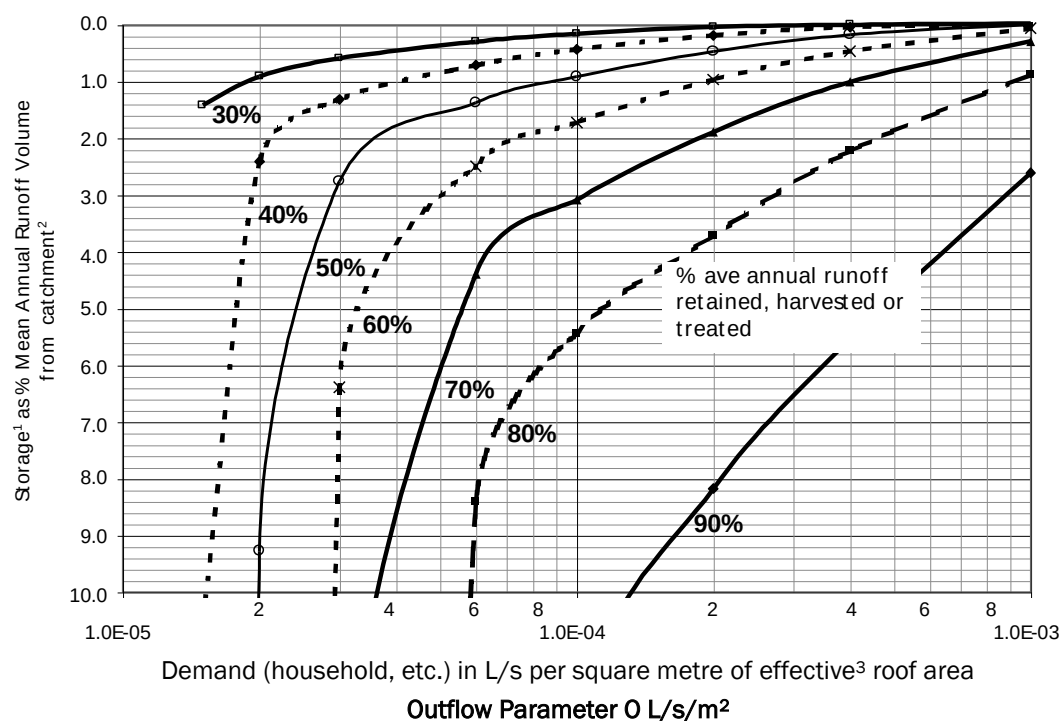
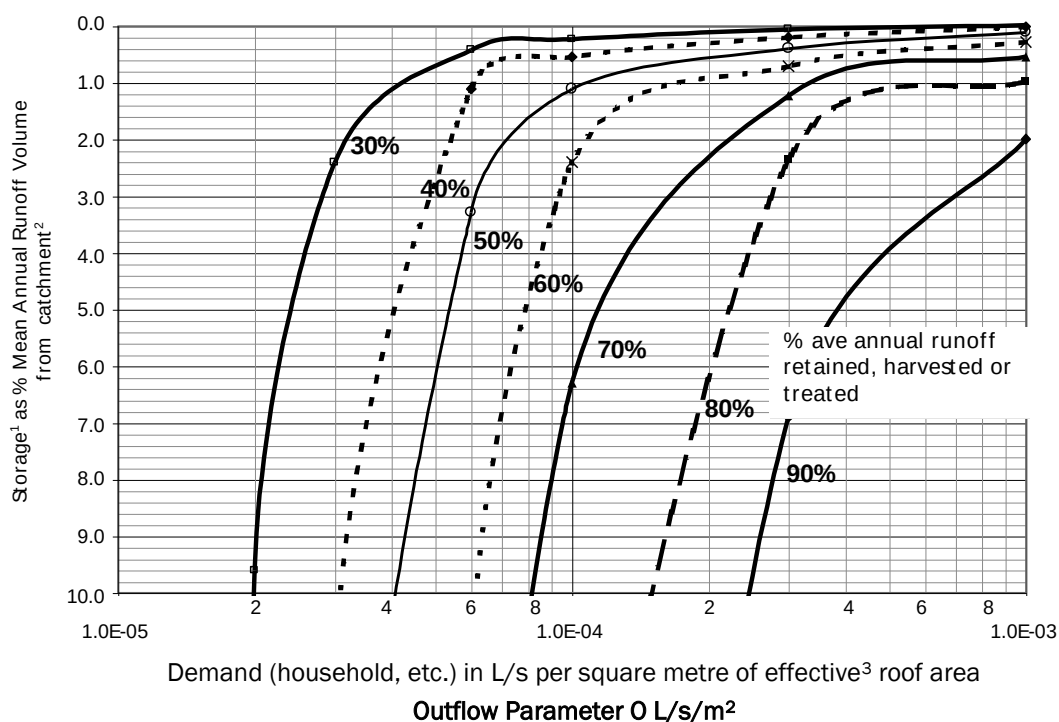


FIGURE E-2 : $i_{10,1}$ range, 35 mm/h to 50 mm/h – Mid-Intermediate Zone

- NOTES : 1 : Device may be a pond or a rainwater tank.
 2 : Catchment may be a ground-level paved area or a roof.
 3 : Effective roof area for rainwater tanks may be as low as 80% of nominal roof area (see Section 8.3.3).

FIGURE E-3 : $i_{10,1}$ range, 50 mm/h to 70mm/h – Upper-Intermediate ZoneFIGURE E-4 : $i_{10,1}$ range, 70 mm/h and above – Northern Australia Zone

- NOTES** : 1: **Device** may be a pond or a rainwater tank.
 2: **Catchment** may be a ground-level paved area or a roof.
 3: Effective roof area for rainwater tanks may be as low as 80% of nominal roof area (see Section 8.3.3).

APPENDIX F

TUTORIAL EXERCISES

TUTORIAL EXERCISES

EXAMPLE 1 :

The derivation of Equations 3.13 and 3.14, Section 3.2.1, includes one typographical error. Find and correct it.

EXAMPLE 2 :

Study the derivations of Equations 3.30, 3.31 and 3.32 in Section 3.5.2. Are there any typographical errors? If so find and correct them.

CHAPTER 5 : Procedures 1 – 4

One of the gravest errors which occurs in the design of storage-related flood control installations in Australia and, one suspects, overseas, arises from misunderstanding about the critical storm duration which should be used in each design context. This matter is addressed in the Handbook in Section 4.2. For this reason, the following exercises provide, typically, three ‘alternative’ critical storm durations : it is part of the design task to select which of these is (are) relevant to the exercise and which is (are) not.

EXAMPLE 3 :

Determine the area, A_s of a “Grasspave” vegetated porous treatment surface required for a car park with total area $A = 2,500 \text{ m}^2$, located in Newcastle; the treatment surface must be part of (included within) the car park area. The car park is to be designed for ARI, $Y = 5$ -years flood conditions. Also determine the expected ‘lifespan’ of the surface : the car park is located in an “average suburb – site with some trees” (see Table 3.1).

The alternative critical storm durations, in accordance with Section 4.2 of the Handbook are :

$$t_c = 10 \text{ minutes; } (T_C)_{\text{local}} = 60 \text{ mins; } (T_C)_{\text{total}} = 2 \text{ hours; ARI} = 5 \text{ years.}$$

The sand/gravel base for the “Grasspave” surface has “as constructed” hydraulic conductivity :

$$k_h = 2.5 \times 10^{-4} \text{ m/s.}$$

The blockage factor for “Grasspave” is $\Psi = 0.1$.

[Ans: $1,440 \text{ m}^2$; 65 years]

EXAMPLE 4 :

Design an on-site stormwater retention system for runoff from a roof, $A = 400 \text{ m}^2$, located, firstly in Perth and secondly in Brisbane : the **yield-minimum** strategy is to be applied (see Section 4.2.1 and Example 5.6 in Section 5.2). “Leaky” wells or a gravel-filled “soakaway” ($e_s = 0.35$) or “Atlantis” boxes ($e_s = 0.95$) may be used in soil of hydraulic conductivity $k_h = 1.6 \times 10^{-4} \text{ m/s}$ (Perth, sandy loam) and $1.0 \times 10^{-6} \text{ m/s}$ (Brisbane, clay). **Local** sub-area flooding is the main design consideration :

$$\text{site } t_c = 15 \text{ minutes, } (T_C)_{\text{local}} = 30 \text{ minutes; } (T_C)_{\text{total}} = 60 \text{ minutes; ARI} = 2 \text{ years.}$$

Use the following to start your design :

- the well effective height, $H = 2.30 \text{ m}$;
- “soakaway” depth, $H = 0.40 \text{ m}$;
- “Atlantis” boxes have alternative dimensions of 0.60 m or 0.40 m .

[Ans: Perth: $D = 1.50 \text{ m}$; $a = 13.6 \text{ m}^2$; $a = 8.14 \text{ m}^2$; Brisbane: 12 wells...; $a = 224 \text{ m}^2$; $a = 216 \text{ m}^2$]

HELPFUL HINT! The main purpose of this exercise (for Brisbane) is to show that ‘source control’ solutions using wells and trenches, only, sometimes lead to very impractical solutions. When you consider you have learned this lesson stop and go on to Example 5.

EXAMPLE 5 :

Repeat Example 4 (Brisbane), this time using the wells and trenches **with ‘slow-drainage’** to a local waterway to solve the problem (for guidance, see Section 5.1.5 and Example 5.7, Section 5.2).
[Ans: 2 wells, $D = 1.70$ m, $q_r = 0.060$ L/s; $a = 71$ m², $Q_r = 0.12$ L/s; $a = 27$ m², $Q_r = 0.12$ L/s]

EXAMPLE 6 :

Determine the plan area, A_p , of infiltration or “dry” pond required to accept, without overflow, storm runoff from a newly developed urban area, previously a 10.0 ha “greenfields” site, near the bottom of a minor catchment in Adelaide. The **regime-in-balance** strategy is to be applied (see Section 4.2.1 and Example 5.3 in Section 5.2). When fully developed, the 10 ha site will have 5.0 ha equivalent paved area and 5.0 ha open space. ARI, $Y = 100$ years is to be used. Downstream flooding of local and total catchment drainage paths is considered unlikely. Relevant critical storm durations are :

$$\text{site } t_C = 30 \text{ minutes; } (T_C)_{\text{local}} = 30 \text{ minutes and } (T_C)_{\text{total}} = 60 \text{ minutes.}$$

The soil at the pond site has long-term hydraulic conductivity, $k_h = 5.0 \times 10^{-5}$ m/s. Maximum pond depth, under design condition, $d = 0.50$ m.

EXAMPLE 7 :

The “Grasspave” treatment surface considered in Example 3 (Newcastle) is underlain by a gravel-filled “soakaway” ($e_s = 0.35$) which receives the flow, provides temporary storage and transfers the cleansed stormwater to an aquifer. Its plan area is the same as that of the treatment surface. The “soakaway” is to be designed for **yield-minimum** conditions (see Section 4.2.1 and Example 5.5 in Section 5.2). Flooding of the “soakaway” is required to be once, **only**, in every five years, on average.

- The site soil is clay for which $k_h = 1.0 \times 10^{-7}$ m/s.
- The recharge rate for the bore is estimated at $q = 0.5$ L/s.
- Determine a suitable depth, H , for the “soakaway”.
- Alternative design storm conditions are : $t_C = 30$ minutes; $(T_C)_{\text{local}} = 60$ minutes; $(T_C)_{\text{total}} = 2$ hours; ARI = 5 years.

HINT! You will find the Equation which you need to solve this exercise in Section 5.1.5.

EXAMPLE 8 :

Determine the area, A_s of a “Hydrapave” permeable paving treatment surface required for a Council car park with **total** area $A = 2,000$ m², located in Brisbane; the treatment surface must be part of (included in) the car park area. Average recurrence interval for the facility is ARI, $Y = 5$ -years. The following special circumstances must be taken into account in determining the area :

- The car park will be in service for a maximum of 10 years (after which time the area will become the site of a library building).
- The proposed (temporary) car park will also receive roof runoff from a Council building, area 1,000 m².
- “As constructed” hydraulic conductivity of “Hydrapave” permeable paving is 1.0×10^{-3} m/s.

The alternative critical storm durations are :

$$t_C = 10 \text{ minutes; } (T_C)_{\text{local}} = 60 \text{ mins; } (T_C)_{\text{total}} = 2 \text{ hours.}$$

CHAPTER 7 : Procedures 5 and 6

EXAMPLE 9 : Given the following data :

Location :	Adelaide, Southern Australia zone
Soil :	medium clay, $k_h = 3 \times 10^{-6}$ m/s; Moderation factor, $U = 2.0$
Catchment :	paved area, $A_{EIA} = 4,000 \text{ m}^2$
Space available :	$A_{avail} = 600 \text{ m}^2$
Retention device :	gravel-filled “soakaway”, $e_s = 0.35$.

Determine, for R = 90% and Strategy A compliance :

- treatment system capacity flow (bypass flow);
- recommended plan area of “soakaway”;
- “soakaway” depth, H.

EXAMPLE 10 : Given the following data :

Location :	Newcastle, Mid-Intermediate zone
Soil :	medium clay, $k_h = 3 \times 10^{-6}$ m/s; Moderation factor, $U = 2.0$
Catchment :	paved area, $A_{EIA} = 4,000 \text{ m}^2$
Space available :	$A_{avail} = 200 \text{ m}^2$
Retention device :	gravel-filled “soakaway”, $e_s = 0.35$.

Determine, for R = 90% and Strategy A compliance :

- treatment system capacity flow (bypass flow);
- recommended plan area of “soakaway”;
- “soakaway” depth, H.

EXAMPLE 11 : Given the following data :

Location :	Brisbane, Northern Australia zone
Soil :	medium clay, $k_h = 3 \times 10^{-6}$ m/s; Moderation factor, $U = 2.0$
Catchment :	paved area, $A_{EIA} = 4,000 \text{ m}^2$
Space available :	$A_{avail} = 80 \text{ m}^2$
Retention device :	gravel-filled “soakaway”, $e_s = 0.35$.

Determine, for R = 90% and Strategy A compliance :

- treatment system capacity flow (bypass flow);
- recommended plan area of “soakaway”;
- “soakaway” depth, H.

Provision for ‘slow-drainage’ may be made if the space available is too small for satisfactory emptying by ‘natural’ drainage. In this event, determine the magnitude of ‘slow-drainage’ which must be provided.

EXAMPLE 12 :

Stormwater runoff from an Adelaide residential sub-division, plan area 3.0 ha, for which equivalent impervious area, $A_{EIA} = 1.50$ ha, is required to be treated to Strategy B standard, that is, 'first flush' treatment, only (Council specification). Local soil is sandy-clay, so a 'dry' pond appropriate to the pollution control/retention requirements of Council is to be designed (ARI, $Y = 0.25$ -years). Given the following data :

Location :	Adelaide, SA	
Soil :	sandy-clay, $k_h = \times 10^{-5}$ m/s; Moderation factor, $U = 1.0$	
Catchment :	paved area, $A_{EIA} = 15,000 \text{ m}^2$	
Time of concentration :	$t_C = 20$ minutes	
Rainfall intensity :	$i_{0.25} = 0.5 \times i_1$	(3.38)
Space available :	$A_{avail} = 150 \text{ m}^2$.	

Determine :

- recommended plan area of 'dry' pond;
- recommended depth of 'dry' pond;
- emptying time for 'dry' pond – compare with interim criterion (Table 3.3).

WSUD INDEX – STUDENT EDITION

A

Aquatic ecosystems - water quality guidelines	112
Aquifers	
bores	13
potentiometric gradient	12
recharge, quality standards	80–82, 112
recharge/retrieval	13, 48
water quality	12
water quality standards	112
water, use of	12, 13, 49–51
Australian climate zones	
Lower-Intermediate	44
Mid-Intermediate	44
Northern Australia	44
Southern Australia	44
Upper-Intermediate	44
Average annual yield - harvesting	
daily demand - hot water use	165
daily demand - outdoor use	165
daily demand - use sensitivity	166
daily rainfall data	162
demand summary - Australian cities	168
effective roof area	162
in-house demand - Adelaide	164
in-house demand - Australian cities	164, 165
in-house demand - Brisbane	163
in-house demand - general	163
in-house demand - Perth	164
initial storage (model)	162
introduction to model	161
low-level 'top-up'	161
model requirements	161
ornamental pond example	177, 178
outdoor use - exotic vegetation	165
outdoor use - native vegetation	165
raintank examples	175, 176
regional model	173, 174
regional model - steps	174
regional yields - explanation	173, 174
software - access	161
software - access details	178
software package	178
South Australia - yields	173
steps in procedures	166, 167, 175–77
without low-level 'top-up'	161
yields for Adelaide	169
yields for Australian cities	168
yields for Brisbane	169
yields for Canberra	170
yields for Darwin	170
yields for Hobart	171
yields for Melbourne	171
yields for Perth	172
yields for Sydney	172
Average recurrence interval, ARI	
"once only in Y-years" case	61, 62, 89, 90
entry works	61
illustrations	1, 18
overflow (exit) systems	61
storage devices	61
urban land use hierarchy	62
use with installation components	61

B

Bacteria	21, 102, 103
Bacteria/micro-organisms	102, 106
'Benchmark year' concept	55, 67, 68
Beneficial uses - water quality guidelines	109, 110

C

Case studies, need for	
advantages of retention	181
cost and policy matters	181
introduction	181
raintanks and flooding	181
regime-in-balance cases	181
regulatory frameworks	181
yield-maximum cases	181
yield-minimum cases	181
Catch basins and 'soak pits'	
catch basins	21
'soak pits'	21
Category 1 systems	21, 53–62, 73–96
Category 2 systems	21, 53, 63, 64, 113–54
Category 3 systems	21, 53, 64, 65, 155–80
Category 4 systems	54, 65
Channel-forming flow	5, 18, 53, 54
Clay sites - the final design option	96
Coarse sediment	21
Complex treatment systems	
background	113–15
example	130, 131
further design considerations	124–28, 152, 153, 154
introduction, steps in Procedure 6A	119–21
introduction, steps in Procedure 6B	123, 124
Concluding discussion	
Category 1 measures, role of	181
Category 2 measures, role of	181
Category 3 measures, role of	181
'filter strip' swales	153, 154
strategic action, need for	181
vision of WSUD (stormwater)	181
WSUD - amenity dimension	181
Conclusion	181
Constructed wetlands	21
Contaminants - guidelines for beneficial uses	109, 110
'Continuous simulation' modelling	
13-step tabulation	34, 35
applications	21, 115, 166, 173
background	31
design storm method	31, 37
flow operation modes	33
future directions	37, 38
'in-ground' storages	32
modified design storm method	31, 37–43
outcome/example - rain tanks	36
outcome/example - 'soakaways'	36
outcomes - 'dry' ponds	33
outcomes - 'soakaways'	33
rainwater storages	32, 166, 173
references	31
'treatment train' elements	32, 114, 116
Critical storm duration	37, 56–58, 83
Critical storm duration – link to runoff volume	30

D

Darcy's Law	23
Design - 'first flush' pollution control systems	121–24
Design for high-level pollution control	116–21
Design storm method	
criticism of	31, 38
'hydraulic' abstractions	39, 41, 42, 80–82
<i>modified</i> design storm method	31, 37–43
'natural' drainage - emptying time	39, 40
summary/guidelines - all cases	83
Detention - wetlands	21
Detention of stormwater	1
Detention technology, role of	4
Detention/retention issues - a last word	96
'Dominant' flow	18, 53
Do's and don'ts - retention practice	
aquifer recharge/retrieval	48
clearance distances	47
clearance to rock and shale	47
shallow soils	47
steep terrain	48
unsuitable soils	47
water quality inflows to devices	48
watertable effects of OSR	48
watertable interaction	48
Drinking water - quality guidelines	112
'Dry' ponds - for rainwater harvesting	177, 178
'Dry' ponds (with infiltration)	
examples	88, 89, 135, 136
further design considerations	96
illustrations	21
summary/guidelines - all cases	83
theory formulation	79–82

E

Emptying time as 'target' value	42, 96
Emptying time formulae	40
Environmental flows	
four rainfall destinies	71
Intermediate Australia zone	72
'low flow' mimicking	71
Southern Australia zone	72
spell characteristics	71
Tropical Northern zone	72
Evolution of development	
effect on runoff coefficients	43–45
urbanisation from forest catchment origins	16, 17
Examples of on-site retention practice	
Figtree Place	50, 51
Heritage Mews	50, 51
New Brompton Estate	49
Parfitt Square	49
Plympton Church Community Garden, Adelaide	50
St. Elizabeth Church car park	50
Extended detention basins	21, 80

F

Figtree Place, Newcastle, NSW	50, 51, 88, 89, 157
'Filter strip' swales	
Australian climate zones	144
background	113–15, 137–45
Concluding discussion	153, 154
examples	148–51
further design considerations	147, 148, 152, 153, 154
steps in Procedures 7 and 8	145–47
Filter strips	21

'First flush' systems	
Austin tank	21
background	113–15, 121–24
example	135, 136
further design considerations	124–28
steps in Procedures 5B and 6B	121–24
'First flush' tanks	21
Fish consumption - water quality guidelines	112
Flat site/catchment slope	21
Flood control	
Category 1 systems	53–62, 73–96
interaction with pollution control	125
interaction with raintanks	179–81
primary aims of WSUD	5
summary/guidelines - all cases	83
Four stormwater 'source control' categories	
Category 1 - quantity management	53–62, 71–96
Category 2 - quality management	53, 63, 64, 113–54
Category 3 - harvesting	53, 64, 65, 155–80
Category 4 - mixed objectives	54, 65
Four urban (street) networks	
'benchmark year' concept	55, 67, 68
high-capacity (street) networks	66
low-capacity (street) networks	66
management issues and ARI	67
medium-capacity (street) networks	66
poor-capacity (street) networks	67
Fully- or over-developed catchments	17, 54, 55

G

Grass swales	21, 137–54
Gravel-based reed bed	21, 49
'Greenfield' catchments	
annual rainfall destiny	15
average annual yield	16
efflux water quality	16
environmental flows	16
'lag' time	16
stream morphology	15
stream scenarios	17
urbanisation stages	16
Gross litter	21, 106, 114, 115
Gross pollution traps	21, 113–15
Guidelines - raw waters for drinking	112
Guidelines - water for irrigation	109, 112
Guidelines - water for livestock	112
Guidelines for human consumers of fish	112
Guidelines for quality of recreational waters	112

H

Harvesting - consequences for flooding	179, 180
Harvesting (see Stormwater harvesting)	
Category 3 systems	53, 54, 64, 65, 155–80
interaction with flooding	179, 180
primary aims of WSUD	5
Heavy metal concentrations - various land uses	111
Heavy metals	21, 106, 114, 115, 154
Heavy metals and nutrients	114, 153, 154
Heritage Mews sub-division	43, 51, 96
High sediment input sites	9, 10, 21
High water table sites	21, 96
Highway runoff pollutants - primary sources	105
Human faeces - typical pathogens	103
Hydraulic conductivity	10–12, 23, 27–30
Hydrological design	
Category 1 systems	21, 53–62, 71–96
Category 2 systems	21, 53, 63, 64, 113–54

Category 3 systems.....	21, 53, 54, 64, 65, 155–80
Category 4 systems.....	21, 54, 65

I

Implementation of Handbook measures	
geographic/climate differences.....	181
relationship to other practices.....	181
resistance to change.....	181
sediment in urban catchments.....	9, 10, 21, 181
Infiltration basins	
examples.....	88, 89, 135, 136
further design considerations.....	96
illustrations.....	21
summary/guidelines - all cases.....	83
theory formulation.....	79–82
Infiltration of storm runoff	
augerhole analysis.....	27–30
basic theory.....	23
constructed clay.....	29
'falling head' analysis.....	29, 30
'fully drained' condition.....	23
heavy clay.....	29
hydraulic conductivity.....	23, 27–30
medium clay.....	28
rock.....	28
sandy clay.....	28
sandy soil.....	28
Infiltration ponds	
examples.....	88, 89, 135, 136
further design considerations.....	96
summary/guidelines - all cases.....	83
theory formulation.....	79–82
Infiltration surfaces	
examples.....	83, 84, 89–91, 131–33
external to paving - theory formulation.....	73–75
further design considerations.....	96
summary/guidelines - all cases.....	83
within paved area - theory formulation.....	75
Infiltration trenches	
examples.....	85, 86, 93–95
further design considerations.....	96
summary/guidelines - all cases.....	83
theory formulation.....	77, 78, 80–82
Irrigation water - quality guidelines.....	112

L

Land availability limitation.....	21
'Leaky' (infiltration) systems.....	21
'Leaky' wells - storage/percolation	
example.....	92, 93
further design considerations.....	96
summary/guidelines - all cases.....	83
theory formulation.....	75–77, 80–82
Lifespan of Strategy A and Strategy B systems.....	127, 128
Litter and gross pollution.....	21
Litter baskets and pits.....	21
Litter booms.....	21
Litter racks.....	21
Livestock watering - quality guidelines.....	112

M

Metro Washington Council of Governments.....	21
Moderation Factor, U.....	74
Modified design storm method	
aquifer access systems.....	41, 42, 80–82
basic elements.....	41

'emptying time'.....	39–41
emptying time (ARI) criteria.....	41
emptying time (US, UK, NZ) criteria.....	40
emptying time as 'target'.....	42, 96
'emptying time' formulae.....	40
explanation.....	38, 39
'hydraulic' abstractions.....	39
'natural' drainage systems.....	38–40
preamble.....	38
slow-drainage systems.....	42, 80–82
summary/guidelines - all cases.....	83
Multi-objective projects	
Category 4 systems.....	54, 65
examples.....	49–51
flood/harvesting objectives.....	65
flood/pollution management.....	65
flood/pollution/harvesting goals.....	65
pollution/harvesting objectives.....	65
MUSIC.....	115

N

Navigating the treatment measures.....	21
New Brompton Estate, Adelaide.....	49, 85, 86
'New' residential streets.....	137–54
Nutrients - pollution.....	99, 101, 106, 153, 154

O

Oil and grease.....	21, 106
Oil/grit separators.....	21
OSD systems.....	1–4, 36, 94, 96, 181
OSD/OSR comparisons	
cost.....	2
environmental benefits.....	4
graphical explanation.....	3
harvesting of stormwater.....	4
OSR systems.....	1–4, 36, 94, 96
Overseas WSUD-related practices.....	55, 56
Oxygen-demanding materials.....	21, 98, 106

P

Parfitt Square, Adelaide.....	49
Percolation of storm runoff	
augerhole analysis.....	27–30
constructed clay.....	11, 29
'falling head' analysis.....	29, 30
heavy clay.....	11, 29
hydraulic conductivity.....	10–12, 23, 27–30
medium clay.....	10, 28
rock.....	11, 28
sandy clay and sandy soil.....	10, 28
Permeable pavements.....	21, 23–26
Pervious paving	
blockage.....	23–26
'clogging'.....	23–26
design Guidelines 1 & 2.....	25, 26
example.....	83, 84
grass (porous) paving.....	24–26, 142
hydraulic conductivity.....	23–25
'lifespan'.....	24–26
permeable pavements.....	21, 23–26
porous pavements.....	21
porous paving.....	21, 23–26
screening tool.....	21
sediment accumulation.....	24–26
summary/guidelines - all cases.....	83
vegetated porous paving.....	26

Plympton Church Community Garden, Adelaide	50, 86, 87, 133–35
Pollutant removal screening tool	21
Pollutants in urban environments - concentrations	108
Pollution control/treatment	
accelerated water removal	119
aquifer access provision	64
Australian climate zones	44, 115, 116
bypass flow - Q_{lim}	116, 125
case examples - introduction	128
Category 1 and 2 systems compared	113
Category 2 systems	53, 63, 64, 113–54
chemical oxygen demand	114
complex Strategy A systems	119–21
complex Strategy B systems	123, 124
'continuous simulation' modelling	115, 116
conversion - "soakaways" to 'dry' ponds, etc.	116, 126, 127
conversion - "soakaways" to trenches	127
devices for sediment-only control	115
devices with concentrated outflows	115
devices with distributed outflows	115
dissolved pollutants	63, 114, 118–20, 123, 138, 140
environmental impact	21
'filter strip' swales - concluding comment	153, 154
'filter strip' swales - introduction	137
'first flush' pollution control/treatment	121
flood control and pollution control	125, 126
GPTs plus retention	63
hydrocarbons	114
hydrological design	21
interaction with flooding	179, 180
isolated systems	124
lifespan of control/treatment systems	127, 128
litter collection	113–15
location, importance of	21
maintenance, importance of	21, 49, 50, 114
nutrients and heavy metals	114
parent soil body, importance of	15, 16, 114
porous and permeable paving	114, 115
primary aims of WSUD	5
primary treatment	21
Procedure 5A	116–18
Procedure 5B	121, 122
Procedure 6A	119–21
Procedure 6A - aquifer access	120, 121
Procedure 6A - slow-drainage	120, 121
Procedure 6B	123, 124
Procedure 6B - aquifer access	123, 124
Procedure 6B - slow-drainage	123, 124
references	21, 113, 114
secondary treatment	21
sediment collection	9, 10, 21, 63, 113–15
selection of measures	21
simple Strategy A systems	116–18
simple Strategy B systems	121, 122
slow-drainage provision	64
Strategy A - examples	129–35
Strategy A (95% treatment) systems	63, 114, 116–21
Strategy A complex systems	119–21
Strategy B - example	135, 136
Strategy B ('first flush') systems	63, 64, 114, 121–24
Strategy B complex systems	123, 124
swale and sub-structure (sketch)	137
swale design - Procedures 7 and 8	145–47
swale designs across climate zones	144
swale graphs – illustrations of use	143
swale streetscapes - examples	148–51
swales - bypass and flood flows	145
swales - concluding comment	152, 153, 154

swales - construction recommendations	147, 148
swales - contributing catchment	138–40
swales - design - introduction	140
swales - disposal of part-treated stormwater	141, 142
swales - dissolved pollution	138, 140, 144
swales - general description	138
swales - introduced surface	138, 140, 141
swales - lifespan	142
swales - retention opportunities (illustration)	139
swales - specification	141, 142
swales - suggested vegetation	141
Swales, 'filter strip' - concluding comment	153, 154
swales, 'filter strip' - introduction	137
tertiary treatment	21
'trickling filter' process	114
Porous and permeable paving (see Pervious paving)	
example	83, 84
navigating the treatment measures	21
permeable pavements	21
porous pavements	21
summary/guidelines - all cases	83
theory and guidelines	23–26
Potential pollution re-mobilisation	21, 114
Primary treatment systems	21
Procedure 1A - infiltration surfaces	74
Procedure 1A - treatment surfaces	74
Procedure 1B - infiltration surfaces	75
Procedure 1B - treatment surfaces	75
Procedure 2A - 'leaky' wells	75–77
Procedure 2B - trench devices	75, 77, 78
Procedure 2C - 'soakaway' devices	75, 78
Procedure 3 - 'dry' ponds	79, 80
Procedure 3 - infiltration ponds	79, 80
Procedure 4A - slow-drainage aquifer access	80–82
Procedure 4B - slow-drainage pipeline case	80–82
Procedure 5A - simple treatment systems	116–18
Procedure 5B - simple 'first flush' systems	121, 122
Procedure 6A - complex treatment systems	119–21
Procedure 6B - complex 'first flush' systems	123, 124
Procedures 7 and 8 - swales ('filter strip')	137–48
Procedures 9 and 10 - rainwater tanks	155–75

Q

Quality standards - aquifer recharge	112
Quality standards for receiving waters	112

R

Rainfall intensity issues	
daily record data	45–47
'embedded' storms	45, 46
short time increment data	45–47
the ARI, $Y = 0.25$ -year case	45, 46
the DRIP programme	47
Rainwater tanks - regional model (see also Stormwater Harvesting)	
introduction	173
ornamental pond example	177, 178
preliminary considerations	174
raintank examples	175, 176
regional yields - explanation	173, 174
software - access details	178
software package	178
steps in procedure	175–77
Rainwater tanks - site model (see also Stormwater Harvesting)	
explanation of spreadsheet	167
introduction to spreadsheet	166

steps in procedures	166, 167
yields for Adelaide	169
yields for Australian cities	168
yields for Brisbane	169
yields for Canberra	170
yields for Darwin	170
yields for Hobart	171
yields for Melbourne	171
yields for Perth	172
yields for Sydney	172
Recreational waters - water quality guidelines	112
Re-development illustration	
data and outcomes	2
introduction - data	1
OSD/OSR comparisons	3
Regime-in-balance strategy	21, 54, 55
Research directions	
control of fine sediments	181
emptying time/ARI criteria	181
OSD/OSR systems - benefits	181
'reactive' soil problems	181
Residential streets	9, 66, 67, 137–54
Retention devices	6–8, 21
Retention devices in clay with high watertable	96
Retention of stormwater - definition	1
Roof systems	
eaves-gutter storage	21
'green' roofs	21
introduction	21
rainwater tanks	21
roof gardens	21
space-saving storages	21
Runoff coefficients	
ARI conversion factor table	45
connected paved areas	44
linear interpolation method	45
pervious areas	44
unconnected paved areas	44
Runoff volume determination	
basic theory	30
in-ground devices	30
time base, τ	30
volume, $\forall$	30

S

Sand filters	21
Secondary treatment systems	21
Sediment	
atmospheric	9
construction phase	9
from paved areas	10
from roofs	9
fully-established suburb	9
Sediment traps	21
Sedimentation basins	21
Shallow bedrock sites	21
Simple treatment systems	
background	113–15
examples	128–36
further design considerations	124–28, 152, 153, 154
introduction, steps in Procedure 5A	116–18
introduction, steps in Procedure 5B	121, 122
Site constraints screening tool	21
Site time of concentration, t_c	30
marinas, incised channels, etc.	58
summary/guidelines - all cases	83
t_c and runoff volume	30
use in design of infiltration surfaces	73–75, 83, 84

use in design of OSR storages	30, 57, 61
Slow-drainage - use of in runoff management	
examples	7, 89–91, 93–95
footnote	70
practical solution - pipeline case	96
theory formulation	80–82
'Soakaway' devices - storage/percolation	
examples	86, 87, 89–91, 96, 133–35
further design considerations	96
summary/guidelines - all cases	83
theory formulation	78, 80–82
Software package - rainwater tanks	178
Soils	
'clearance'	10, 11
constructed clays	11
deep sands	10
heavy clays	11
hydraulic conductivity	10–12
medium clays	10
non-homogeneity	11
permeability/reactivity	10
sandy clays	10
shallow, with rock	11
water reactivity	11
St. Elizabeth Church car park, Adelaide	50, 83, 84, 89–91
Steep site/catchment slope	21
Storm critical duration	
local catchment value, T_C	56, 57
t_c and runoff volume	30
T_C and runoff volume	30
total catchment value, T_C	56, 57
Storm critical duration, T_C	
site time of concentration, t_c	57, 58
summary/guidelines - all cases	83
the sump case - all durations	58
Storm runoff management	
aquifer access - example	89–91
aquifer access systems	7, 49–51
'benchmark year' concept	55, 67, 68
Category 1 systems	53–62, 73–96
design - clay sites - the final option	96
design - combinations of devices	83, 96
design - detention/retention options	96
design - 'device full' assumption	73
design - economic constraints	96
design - site constraints	96
design - slow-drainage issues/options	96
design to meet emptying time criteria	96
developed catchments	19, 20, 54, 55
devices in clay with high watertable	96
discharge-corrected equations	82
'dry' pond - example	88, 89
end-of-pipe systems	56
environmental flows	5, 54, 72
extended detention - example	95
extended detention systems	7, 80–82
four urban (street) networks	66
fully-developed catchments	19, 20, 54, 55
'greenfield' catchments	55
high-capacity (street) networks	66
infiltration surface - example	83, 84, 89–91
'leaky' well - example	92, 93
limitations of Procedure 3 cases	80
low-capacity (street) networks	66
medium-capacity (street) networks	66
Storm runoff management (cont.)	
Moderation Factor, U (footnote)	74
over-developed catchments	19, 20, 54, 55
overseas practice - North America	18, 55, 56

overseas practice - United Kingdom.....	55, 56
pavement/'soakaway' design interaction	91
poor-capacity (street) networks	67
primary aims of WSUD.....	5
Procedure 1 cases - introduction.....	73
Procedure 1A - infiltration surfaces.....	74, 75
Procedure 2 cases - introduction.....	75
Procedure 2A - 'leaky' wells	76, 77
Procedure 2B - trench devices	77, 78
Procedure 2B cases - introduction	77
Procedure 2C - 'soakaway' devices.....	78
Procedure 3 - 'dry' ponds	79, 80
Procedure 3 - pond with infiltration.....	79, 80
Procedure 4 cases - introduction.....	80
Procedure 4A - aquifer access	80
Procedure 4A/4B - limitations	82, 83
Procedure 4B - using pipeline	80
re-development.....	19, 20, 54, 55
re-development examples	1–4, 50, 93–95
regime-in-balance - 5-step table.....	70
regime-in-balance - introduction.....	69
regime-in-balance strategy.....	54, 55
retrofitting.....	19, 20, 54, 55, 71
retrofitting examples.....	1–4, 50, 93–95
roof sediment.....	8
'simple' installations.....	6
site time of concentration, t_c	57, 58
slow-drainage - example	95
'slow-drainage' systems	7, 70, 80–82
'soakaway' device - examples	86, 87, 133–35
'soakaway' device - illustrations	7, 96
storm <i>critical</i> duration	56–58
storms longer than <i>critical</i>	56–58
storms shorter than <i>critical</i>	56–58
sub-structure/'soakaway' - examples.....	89–91
summary/guidelines - all cases	83
the sump case - all durations	58
three stream scenario/strategies	54, 55
traditional approach.....	1
trench device - example.....	85, 93–95
void space ratio, e_s	77
water-retaining devices.....	5
yield-maximum strategy.....	54, 55
yield-minimum - tabulation.....	71
yield-minimum strategy	54, 55
Stormwater - concentrations of contaminants.....	109, 110
Stormwater harvesting	
aquifer storage/recovery (ASR).....	65
Category 3 systems.....	53, 54, 64, 65, 155–80
chemical considerations.....	159
consequences for flooding	179, 180
current drainage practice	155
domestic applications	64
flood relief - Northern/Southern effects.....	180
gravity supply	157
hot water supply	157
impact on flooding - references	179, 180
industrial applications.....	64
in-ground tanks - health issues	160
microbiological aspects	159
multiple tank harvesting	160
'passive irrigation'	65
references	156
roof runoff harvesting, RRHS	155–80
tank size - practical aspects	156
tanks - all purpose supply	158
tanks - box gutter storage.....	157, 160
tanks - modular.....	160
tanks - pumped systems.....	160
tanks - roof and mains supply	157
tanks - supply - non-drinking.....	158
tanks - 'ultra slim' (fence).....	160
the 'use' paradox.....	156
water quality - introduction.....	158, 159
Stormwater management agency, tasks of	
catchment-wide harvesting	69
commission hydrological study	68
decide <i>emptying time</i> criteria	69
determine scenario/strategies	69
determine storm <i>critical</i> durations	69
make policy decisions on ARIs.....	69
policy on Strategy A and/or B.....	69
Stormwater quality, aspects of	
biological parameters	102, 103
characteristics of urban pollution	104
chemical water quality parameters.....	98, 99
contaminants - main groups.....	106
contamination level - factors.....	105, 106
effects of urbanisation.....	17, 104
guidelines - aquatic ecosystems	112
guidelines - consumption of fish	112
guidelines - raw waters for drinking	112
guidelines - recreational waters.....	112
guidelines - water for irrigation	112
guidelines - water for livestock	112
heavy metals	101, 102
highway pollution sources	105
introduction.....	97
mean concentrations - US data.....	108
nitrogen cycle.....	99–101
pathogens in human faeces	103
phosphorous cycle.....	101
physical water quality parameters.....	97, 98
pollution concentrations - urban	107
quality standards - receiving waters.....	112
roads - heavy metal concentrations	111
sources of pollution.....	104, 105
standards - aquifer recharge	112
typical concentrations and use guidelines	109, 110
'wash-off' and 'first flush'	105, 106
Strategy A - design for 95% treatment.....	63, 114–21
Strategy B - 'first flush' treatment	63, 64, 114, 115, 121–24
Suspended solids.....	16, 21, 97, 106, 110, 112, 153, 154
Swales.....	21, 73, 137–54
Swales and bio-retention systems	21, 137–54
Swales, 'filter strip'	
background	113–15, 137–45
concluding comment.....	153, 154
examples	148–51
further design considerations	147, 148, 152, 153, 154
steps in Procedures 7 and 8.....	145–47

T

Tertiary treatment systems	21
Tidal range site	21
'Top end' site drainage	
economic advantage of	58–60
regime-in-balance strategy	58
theory of.....	59, 60
yield-maximum strategy	58
yield-minimum strategy.....	58

Total nitrogen	16, 21, 99–101, 105, 108, 112, 153, 154
Total phosphorous	16, 21, 101, 105, 108, 112, 153, 154
Trash racks	21, 113, 114
Treatment surfaces	
examples	83, 84, 89–91, 131, 132
further design considerations	96
summary/guidelines - all cases	83
theory formulations	82–84
Treatment systems - pollution control (see Pollution control/treatment)	
Category 2 systems	53, 63, 64, 113–54
interaction with flooding	179, 180
primary aims of WSUD	5
'Treatment train'	10, 21, 114
'Treatment train' elements	32
Trench devices - storage/percolation	
examples	85, 86, 93–95
further design considerations	96
summary/guidelines - all cases	83
theory formulation	77, 78, 80–82

U

Underground installation required	21
Urban environments - pollutant concentrations	108
Urban land use hierarchy and ARI	
community flood accommodation	62
factories, businesses, sport, etc.	62
food, pharmaceutical etc. stores	62
high-density residential	62
hospitals, civil defence HQ	62
medium-density residential	62
police, water supply, power, gas	62
Urbanisation, evolution of	
effect on runoff coefficients	43–45
stages in the process	16, 17

V

Vision of WSUD (stormwater)	
conventional development	1, 15–17
sustainable development	18–20, 181
Void space ratio, e_s	77

W

Wastewater - concentrations of contaminants	109, 110
Water quality improvement structures	
constructed wetlands	21
'dry' ponds	21
extended detention basins	21
fine sediment control devices	21
infiltration basins	21
oil/grit separators	21
UniSA tank	21
water quality inlets	21
wet detention basins	21
Water-sensitive urban design	
'oversizing' devices	20
planning for 'greenfield' development	18
primary aims	5
retrofitting WSUD practices	19
site options/opportunities	5
the limitations	20
Waterway management	53–56
Waterways and sustainable urbanisation	
existing infrastructure case	42, 43, 54, 55, 57
natural stream case	42, 43, 54, 55
Wetlands	21

Y

Yield-maximum strategy	21, 54, 55
Yield-minimum strategy	21, 54, 55