A BEST PRACTICE GUIDELINE FOR DESIGN FLOOD ESTIMATION IN MUNICIPAL AREAS IN SOUTH AFRICA

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WATER RESEARCH COMMISSION

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A BEST PRACTICE GUIDELINE FOR DESIGN FLOOD ESTIMATION IN MUNICIPAL AREAS IN SOUTH AFRICA

Report to the
WATER RESEARCH COMMISSION

by

URBAN FLOOD ESTIMATION WORKING GROUP

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The material contained in this publication is intended to assist Local Authority Officials and Private Practitioners to estimate design flood discharges and compute the associated water surface profiles. It is not intended as professional advice on specific applications. It is the responsibility of the user to determine the suitability and appropriateness of the material contained in this publication to specific applications. No person should act or fail to act on the basis of any material contained in this publication without first obtaining specific independent professional advice.

It is important to understand that flood discharges will change in response to changes in climate, as well as changes in the catchment, and that the water surface profile for any specific discharge will change in response to both long and short-term changes in channel and floodplain characteristics. Flood lines and floodwater levels must therefore be reviewed and possibly revised from time to time. The frequency of this revision must be determined by the Local Authority.

The 100-year recurrence interval is not a magic number. There is a significant probability that an event of this magnitude will be exceeded during the life of a project. It is therefore incumbent on the Local Authority, the Developer, and the Certifying Engineer to assess the appropriate level of risk and control development accordingly.

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EXECUTIVE SUMMARY

The requirements for design flood estimation in urban areas are changing largely as a result of the pressure placed on flood prone land by rapidly expanding urban populations. To address the facts that there are currently no design standards for urban flood estimation and risk assessment in South Africa, along with a decline in expertise in this regard at local municipality levels, it was proposed to extend the recent National Flood Studies Programme (NFSP) to focus on urban requirements and to develop a Best Practice Guideline specific to urban applications. The need for such a guideline was championed by the IMESA EXCO who then agreed a jointly funded project with the Water Research Commission (WRC). The team that undertook the project comprised specialists from the Universities of KwaZulu-Natal, Stellenbosch and the Witwatersrand, as well as the Central University of Technology, Free State and private practitioners.

The document commences with an introductory overview that gives guidance on how to use the guide and notes that there is a close relationship between the techniques used for design flood estimation and those used in the design of stormwater drainage and stormwater management systems. Although the literature on flood hydrology is extensively referenced in this guideline, it is regarded as a stand-alone document, giving sufficient information to enable the practitioner to estimate design floods in urban catchments. It is important to understand that these calculations are complex with a considerable degree of uncertainty, and hence should be undertaken only by suitably qualified and experienced practitioners.

The preliminary chapters, that discuss the legislation and principles applicable to design flood estimation, are followed by a detailed “road map” that guides the user through subsequent chapters.

This “road map” contained in Chapter 4 discusses the different approaches to flood estimation and differentiates between probabilistic methods, deterministic event-based methods, deterministic continuous simulation modelling and empirical methods. Guidance is given for the selection of the appropriate method for design flood determination. It is recognised that the range of uncertainty is great and advises that multiple methods of calculation be used.

Chapter 5 addresses the difficulty that practitioners often have in finding the necessary data on rainfall, measured streamflow data, catchment topography, land use, soil infiltration characteristics, etc. Most of the required information is contained in the chapter itself but, more importantly, there are numerous links to resources available in the public domain.

Probabilistic rainfall and flood frequency analyses are covered in Chapter 6, supplemented with information contained in appendices.

Event-based design flood estimation is discussed in Chapter 7 where the distinction is made between probabilistic, deterministic, and empirical methods, with recommendations made regarding the applicability of the different methods. A distinction is drawn between the use of discretised computer-based continuous simulation and single-event calculations, but it is noted that the principles and data requirements for these analyses are very similar that all computer-based discretised modelling is discussed in Chapter 8.

Although there are numerous programs available for computer modelling, the discussion in Chapter 8 is limited to the use of the US EPA's Stormwater Management Model (SWMM) given that this model is the most used in South Africa. SWMM is freely available in the public domain; hence, making it highly attractive to under resourced local authorities.

In almost all instances the estimated design flood peak discharges or hydrographs (inclusive of both volume and peaks) must be translated to a water surface elevation for the purpose of floodline and floodplain inundation determination. River hydraulics are therefore discussed in Chapter 9, with more detail given in the appendices.

It is recognised that the results of these calculations are inherently uncertain. Chapter 10 addresses this uncertainty and strongly advocates the need for the collection of data to allow calibration of the different modelling and calculation methods. This chapter also addresses the concepts of accuracy and consistency.

Appendices include a glossary of terms, more detail on applicable legislation including guidance on the content required in a report on floodline determination, further discussion on the hydrological characteristics of soils, an outline of the techniques used for the infilling of missing rainfall data, explanation of the quality codes in DWS flow data, some detail on the calculation procedures for probability distributions including worked examples including a description of the recently developed “Z-Set” plotting position, and more detail on river flood hydraulics, including guidance on the estimation of flow resistance.

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1. INTRODUCTION: FLOOD, RISKS AND DISASTER MANAGEMENT IN LOCAL AUTHORITIES

1.1 How to Use this Guide

This guideline is aimed at flood risk determination in Municipal catchments. It is intended to assist Municipal officials to carry out their flood management and stormwater design responsibilities. Hence the focus is much wider than flood line determination. The recommended design flood estimation methods are selected from the range of methods in use in South Africa as being the most suitable for urban catchments and Municipal requirements.

The selected methods are not ideally suited to large river catchments. Municipalities located on large rivers will need to look at other methods when determining flood lines for these main watercourses.

This guideline seeks to set out best practice for design flood estimation at Municipal level. In doing so it gives emphasis to the responsibilities and, therefore, accountability of the Municipal Manager regarding flood management planning and awareness and the expertise of officials and appointed external practitioners.

The guideline is comprehensive in content and, therefore, the primary reference for the user. The structure of the guideline is as follows:

- Legal overview from a Municipal perspective
- General principles of design flood estimation for Municipalities
- Recommended methods
- Information sources
- Worked examples
- Technical details

1.2 Scope for Flood Estimation

Design flood estimation is perhaps most frequently associated with the determination of flood lines, typically the 100-year flood line that is widely used for development control. There is, however, a much wider range of requirements in Municipal applications. The design of stormwater networks requires the determination of smaller events such as 1 year, 2 year and 5-year events. Design of attenuation systems may include mid-level events such as 10-year to 50-year events.

Events larger than the 100-year event may also be required for disaster management and planning. For example, in the United Kingdom the determination of the 1000-year flood event is now a requirement for planning and disaster risk analysis. It is also good practice to test the performance of facilities in events larger than their hydraulic design standard. This will include the likes of spillways, bridges and stormwater networks where hydraulic conditions during overtopping may present unacceptable or hazardous conditions. Running these tests may also influence the Design Engineers' approach to design flood estimation.

Urban design flood estimation is also adapting to support the transition to Water Sensitive Design (WSUD) and Sustainable Drainage Systems (SuDS). Here, emphasis is placed on managing urban stormwater as a resource and minimising the impact of urban development on the environment, often using the undeveloped, natural catchment condition as a reference base. In terms of flood estimation, this places focus on detention and retention within the drainage network, placing greater importance on understanding flood volume and antecedent conditions. It also reinforces the need to analyse storm events ranging from smaller frequent events to rare severe storm conditions.

1.3 Expert knowledge

It is incumbent on both practitioners and municipal officials managing stormwater systems and flood risk to develop sufficient understanding of the science of design hydrology and the implications of its application in the municipal space. Flood conditions arise from complex hydrological processes that can be analysed through the application of very simple empirical equations or by detailed modelling. It is expected that both practitioners and municipal officials have sufficient expert knowledge to judge the requirements of each DFE situation, and that they will know when to seek expert support. This guideline will help refine the understanding of good practice in South Africa, but it expects that users have the necessary foundational understanding of the subject matter.

1.4 Catchment Management and Planning

Changes in land use and land cover will change the flood response in a catchment. In particular, progressive development in a catchment, or initiatives for urban densification, will lead to increasing flood responses over time. Design flood estimates therefore reflect the land use in a catchment at a certain date, or some projected future scenario. Data sources and description of catchment conditions are an important part of the design flood estimation produced. It is also for this reason that design flood estimates and municipal flood lines need to be regularly reviewed and updated in support of strategic planning and development initiatives and other municipal responsibilities, such as disaster management. It is an important, but often neglected, Municipal responsibility to ensure that these data are kept up to date.

There needs to be a strong link between urban planning and stormwater management. Planners need to be aware of flood management strategies at a catchment scale so that, for example, development densities are not exceeded, and sufficient space is set aside for adequate management of increased stormwater runoff. This is increasingly important with increasing urban populations, plans for urban densification and the threats of climate change influences on storm events. Municipal Stormwater Departments need to take a leading role in communication with Planning Departments, setting out guidelines to assist planners as far as possible.

Similarly, Stormwater and Disaster Management Departments need to plan with foresight of land development objectives and must also be able to clearly communicate their stormwater and flood management strategies. This will include consistency of approach to design flood estimation to ensure continuity along watercourses as well as the frequency of updates. Again, the Stormwater Department's responsibility comes to the fore. As WSUD takes effect, these inter-departmental links in Municipalities will be reinforced.

1.5 Sources of Information

This guide seeks to provide as much information as possible as to where to find references or data necessary to carry out design flood estimation. All the links and references referred to are freely available unless stated otherwise.

In the past it was always considered part of good practice to visit a catchment before preparing a design flood estimate. Visual inspection of land cover conditions and even site measurements (e.g. soil auger sampling) would help refine hydrological parameters. With the increasing availability of spatially detailed information, it may be easy to develop a sense of accuracy in deriving hydrological parameters, reducing the inclination for further investigation and on the ground verification. It is the responsibility of municipal officials to enquire on the sources of information used and of the DFE practitioner to be able to defend it.

Very little of the information provided in this guideline is specific to individual Municipalities. For the reasons given above, proactive Municipalities will be developing their own local databases (e.g. flood records) and data sets (e.g. rainfall, flow records, topographic data, soil data) and Practitioners are advised to first approach the Local Authority for available information.

Municipalities also need to recognise the importance of maintaining their own register of infrastructure assets that may influence flood responses and to budget for the of building and maintaining of their own local records and data sets, which will lead to significant cost savings in the final more accurate designs. These include stormwater assets like drain networks and detention ponds, and the information stored should include dimensions, hydraulic performance (e.g. max capacity) and previous DFE calculations. This information will be useful in future flood assessments.

2. LEGAL AND REGULATORY REQUIREMENTS

2.1 Introduction

The legal requirements for future development within the South African environment are mainly a responsibility of Local Government and the Municipal Engineer is mostly tasked with ensuring that all requirements are met. It is, however, a multi-level responsibility which does not rest only with the Municipal Engineer, but at various levels of governance. In addition, the Engineer also carries a professional responsibility to ensure that all technical steps are taken to ensure a safe environment. In this regard, the attention of Municipal Engineers is drawn to the reality that, where they do not have the knowledge or capacity to ensure safety within the context of flooding, it remains their responsibility to apply proper professional judgement in appointing appropriately trained advisors to ensure a safe environment.

Within the context of this guideline, the main responsibility of Municipal Engineers is the determination of accurate flood lines to inform appropriate levels of development and to determine risk. The prediction of flooding is, however, not an exact science, with much professional judgement required in the process. In this context, Municipal Engineers are urged to ensure an approach where multiple flood determination methods are used and compared with each other to ensure the soundest solution.

Legislation related to a safe environment in the context of flooding can be found at various levels, starting with the Constitution, followed by National, Provincial and Local Government level. In most cases, the implementation of flood related legislation at Local Government level can be found in by-laws, as approved and adopted by the relevant Local Authorities.

2.2 National Legislation

2.2.1. Constitution

The Constitution (Act 108 of 1996) (RSA, 1996) provides the fundamental rights of all South African citizens. In the context of flood management, the Constitution gives emphasis to a safe and sustainable environment. Section 24 deals with the environment, and, amongst other issues, states that:

24. Everyone has the right—

(a) to an <u>environment that is not harmful</u> to their health or wellbeing; and

(b) to have the <u>environment protected</u>, for the benefit of present and future generations, through reasonable legislative and other measures that—

i. prevent pollution and ecological degradation;

ii. *promote conservation; and*

iii. *secure ecologically sustainable development and use of natural resources while promoting justifiable economic and social development.*

In the contexts of an "*environment that is not harmful to their health*", it can be argued that the intention is to ensure that all citizens do have a safe environment to live in, implying an environment that will among other aspects, also prevent citizens from being exposed to possible flooding. This is further supported by the requirement to "*secure ecologically sustainable development*". In this right exists a challenge to secure development, but at the same time ensure that it is a safe environment.

In Chapter 3, Section 41 of the Constitution, the need for cooperative governance stresses:

41. (1) All spheres of government and all organs of state within each sphere must—

(b) secure the well-being of the people of the Republic;

(c) provide effective, transparent, accountable and coherent government for the Republic as a whole;

(h) co-operate with one another in mutual trust and good faith by—

(ii) assisting and supporting one another;

(iii) informing one another of, and consulting one another on, matters of common interest;

(iv) co-ordinating their actions and legislation with one another;

The Constitution then goes further and allocates main functions and responsibilities to different levels of governance. The objectives of local government are highlighted in Section 152 of Chapter 7 and include, among others:

152. (1) The objects of local government are—

(c) to promote social and economic development;

(d) to promote a safe and healthy environment; and..

The functions are highlighted in Section 156:

156. (1) A municipality has executive authority in respect of, and has the right to administer—

(a) the local government matters listed in Part B of Schedule 4 and Part B of Schedule 5; and….

These schedules include functions such as the Environment, Disaster Management, Housing, Regional Planning and Development, Urban and Rural Development and Stormwater Management Systems in built-up areas. The Constitution also highlights the functions of Provinces, which include Provincial planning (Schedule 5, Part A).

To this end, Local Government is the main level of governance responsible for safe and sustainable development, therefore also the most likely level of governance to ensure proper flood management. The need for cooperative governance, however, stresses the importance of working together to ensure a safe environment.

2.2.2. National Water Act (Act 36 of 1998)

The purpose of the National Water Act (NWA) (RSA, 1998a) in relation to floods is highlighted in Section 2 of the Act:

> *The purpose of this Act is to ensure that the nation's water resources are ……. used, developed, …. managed and controlled in ways which take into account amongst other factors-*
>
> *(k) managing floods and droughts, and for achieving this purpose, to establish suitable institutions ……….*

Flood lines are dealt with in Part 3 of the NWA, with the objective to make information relating to floods and the potential risks, available to the public. The NWA clearly highlights the objective in Part 3 stating that:

> *Township layout plans must indicate a specific floodline. Water management institutions must use the most appropriate means to inform the public about anticipated floods, droughts or risks posed by water quality, the failure of any dam or any other waterworks or any other related matter.*

Section 144 of the NWA provides the requirements:

144. For the purposes of ensuring that all persons who might be affected have access to information regarding potential flood hazards, no person may establish a township unless the layout plan shows, in a form ***acceptable to the local authority*** *concerned, lines indicating the maximum level likely to be reached by floodwaters on average once in every 100 years.*

Section 145 emphasises the responsibilities:

145. (1) A water management institution must, at its own expense, make information at its disposal available to the public in an appropriate manner, in respect of -

(a) a flood which has occurred or which is likely to occur;

(e) levels likely to be reached by floodwaters from time to time;

A water management institution is defined in the NWA as "*a catchment management agency, a water user association, a body responsible for international water management or any person who fulfils the functions of a water management institution in terms of this Act.*" It therefore needs to be accepted that the implementation of Section 145 (1) is, according to the NWA, not a function of a Municipality.

In the absence of such a water management institution in the area of a municipality the responsibilities fall on the DWS to make this information available, but given the responsibility to ensure integrated development planning, and as such the responsibility for a safe and sustainable environment, of municipalities under the Water Service Act, it can be concluded that municipalities are responsible for at least the assessing of such information or alternatively the accessibility and availability of this information.

The NWA does, however, clearly state that no township (which is not clearly defined but can be assumed to be any urban development) can be developed, unless the 1 in 100-year flood line is indicated on the development plan, specifically to enable an assessment of risk in order to adhere to the Constitutional requirement of a safe environment.

2.2.3. Water Services Act (Act 108 of 1997)

The Water Services Act (RSA, 1997) provides the regulatory framework for water services institutions and defines all Municipalities as Water Service Authorities. Whilst Water Service Authorities' main function is to ensure access to water services, it must also be recognised that they are responsible for the approval of Water Service Development Plans, which form part of the Integrated Development Plan (IDP), thereby making Municipalities effectively also responsible for the approval of developments that need to be safe and sustainable, as highlighted in the Constitution.

2.2.4. Municipal Structure Act (Act 117 of 1998)

The main purpose of the Municipal Structure Act is to provide for the:

> "*establishment of municipalities in accordance with the requirements relating to categories and types of municipality; ………..to <u>define the types of municipality</u> that may be established within each category; to provide for an appropriate <u>division of functions and powers</u> between categories of municipalities*"

The 3 categories defined are category A (Metropolitan Municipalities), category B (Local Municipalities) and category C (District Municipalities). The category A Municipalities have all the functions allocated to Municipalities according to the Constitution (RSA, 1996), as listed in Part B of Schedule 4 and Part B of Schedule 5. A further division of functions between District and Local Municipalities is provided in Section 84 of the Municipal Structure Act.

Of note is the requirement of the Municipal Structure Act for Municipalities to compile an Integrated Development Plan (IDP), which, amongst other aspects, requires spatial planning and by default also a firm understanding of flooding.

2.2.5. Municipal Systems Act (Act 32 of 2000)

The key focus of the Municipal Systems Act (RSA, 2000) is to provide for:

> "*the core principles, mechanisms and processes that are necessary to enable municipalities to move progressively towards the <u>social and economic upliftment of local communities</u>, and ensure universal access to essential services that are affordable to all; ………; to provide for the manner in which <u>municipal powers and functions are exercised and performed;</u> …; to establish a simple and enabling <u>framework for the core processes of planning, performance management,</u> …….*"

The Municipal Systems Act sets the framework for the IDP and delegates the responsibility of the drafting process to the Executive Committee or Executive Mayor. It further tasks the Municipal Manager (MM) in Section 32 to submit the IDP to the MEC. The overall performance of the administration is also delegated to the MM in Section 51, with further functions highlighted in Section 53.

Since no appropriate planning (IDP) is possible without a clear understanding of the flooding regime in the area of responsibility to ensure a safe environment, the Municipal Systems Act clearly tasks the Executive Committee and the MM to ensure a reasonable understanding of the probability and likely extent of flooding.

Municipalities must:

- **Ensure a safe environment.**
- **Ensure that at least the 1:100-year flood line is shown on all development plans.**
- **Have a clear understanding of the flood regime before approving their IDP.**

2.2.6. National Building Regulations & Building Standards Act (Act 103 of 1977)

The National Building Regulations & Building Standards Act (NBRBSA) (RSA, 1977) burdens Municipalities with specific requirements related to flooding in both section 10 and 16 of the NBRBSA.

Section 10 (1):

If any building or earthwork-

(b) is being or is to be erected on a site which is subject to flooding or on a site which or any portion of which in the opinion of the local authority in question does not drain properly or is filled up or covered with refuse or material impregnated with matter liable to decomposition, such local authority may by notice in writing, served by post or delivered, prohibit the person erecting such building or earthwork or causing such building or earthwork to be erected from commencing or proceeding with the erection thereof or from so commencing or proceeding except on such conditions as such local authority may determine from time to time.

Section 16:

Report on adequacy of certain measures and on certain building projects

(1) The Minister, after consultation with the Administrator of a province in which the area of jurisdiction of a local authority is situated, may order such local authority to report to him on-

(a) the adequacy of measures in or in connection with buildings in its area of jurisdiction against fire, floods or other disasters and to make recommendations in order to remove any inadequacies in such measures;

Section 7 of the NBRBSA also provides some relevant background by stating that:

7(1)… If a local authority…

(b)(i) …is not so satisfied (that the building to which the application in question relates);

(bb) ….will probably or in fact be dangerous to life or property..

such local authority shall refuse to grant its approval in respect thereof and give written reasons for such refusal.

NBRBSA (RSA, 1977) requires the Municipality to be able to advise with regards to floods. No clear definition of “flood” is provided, but it is clear that all Municipalities do have the legal obligation to be aware of flood related events.

Thompson (2006) provides for a more practical approach in relation to the responsibilities of Municipalities and flooding, given that the NBRBSA is silent on the term flooding, linking the requirements of the NWA and the NBRBSA.

If the Municipality is satisfied that any building might be a danger to life or property, then it must refuse such building plan. Thompson (2006) stated that, since a building most probably will not be a danger to life if constructed (appropriately, meaning that the building can structurally withstand the flood) on an area subjected to flooding, floodwater might result in the building becoming a danger to life and can therefore be refused.

Municipalities must:

- **Ensure that any building/earthworks likely to cause damage due to flooding be managed appropriately to prevent flood damage or prevent the planned activity from being executed.**
- **Refuse an approval of building plans if such a planned building might be a danger to life or property due to its impact on flooding.**

2.2.7. Disaster Management Act (Act 57 of 2002)

The Disaster Management Act (RSA, 2002) allows for the establishment of structures at different levels (national, provincial, and municipal disaster management centres) to manage disasters in the country.

Section 15 deals with the functions and duties of the national disaster management centres and states that:

15 (1) The National Centre must, subject to other provisions of this Act, do all that is necessary to achieve its objective as set out in section 9, and, for this purpose-

(a) must specialise in issues concerning disasters and disaster management;

(b) must monitor whether organs of state and statutory functionaries comply with this Act and the national disaster management framework and must monitor progress with post-disaster recovery and rehabilitation;

(c) must act as a repository of, and conduit for, information concerning disasters, impending disasters and disaster management; ………

It also deals in Section 17 with a disaster management information system, which includes, amongst other things:

(1) The National Centre must act as a repository of, and conduit for, information concerning disasters and disaster management, and must for this purpose-

- *(a) collect information on all aspects of disasters and disaster management;*
- *(b) process and analyse such information;*
- *(c) develop and maintain an electronic database envisaged in subsection;*

Sections 30/44 deal with the functions of the Provincial/Municipal management centre, which also includes the need to "*act as a repository of, and conduit for, information concerning disasters, impending disasters and disaster management in the province / municipality*".

The need for the collection and management of information related to disasters, which includes floods, is clear. It can be concluded from these sections that the Disaster Management Act enforces cooperation amongst various levels of governance, with specific reference to the collection of data related to possible disasters which, of course, includes floods.

2.3 OTHER LEGISLATION

Following the various National Acts, a significant number of Provincial Acts and regulations by different spheres of government have been promulgated and are constantly being updated or amended. Provincial Acts such as the **Western Cape Land Use Planning Act** (Act 3 of 2014) mainly focus on the planning issues, in which floods frequently receive attention, based upon the need to develop a safe sustainable environment. The Western Cape Land Use Planning Act, for example, lists in Section 59 the principles of land use planning, *inter alia*:

59. (2) Land use planning is guided by the following principles of <u>spatial sustainability</u>:

(b) the sustained protection of the environment should be ensured by <u>having regard</u> to the following:

(iii) areas unsuitable for development, including <u>floodplains</u>, steep slopes, wetlands and areas with a high water table and landscapes and natural features of cultural significance;

The Neighbourhood Planning and Design Guide (RSA, 2019) summaries (see Figure 2-1) the complexity and integrated nature of policy and regulatory legal requirements, which include the **National Development Plan 2030** (NDP), **Integrated Urban Development Framework** (IUDF), **National Climate Change Response White Paper** and the **Spatial Planning and Land Use Management Act** (SPLUMA), with the **Town Planning and Township Ordinance**

(Ordinance 15 of 1986) providing the backbone for the development and implementation of the SPLUMA.

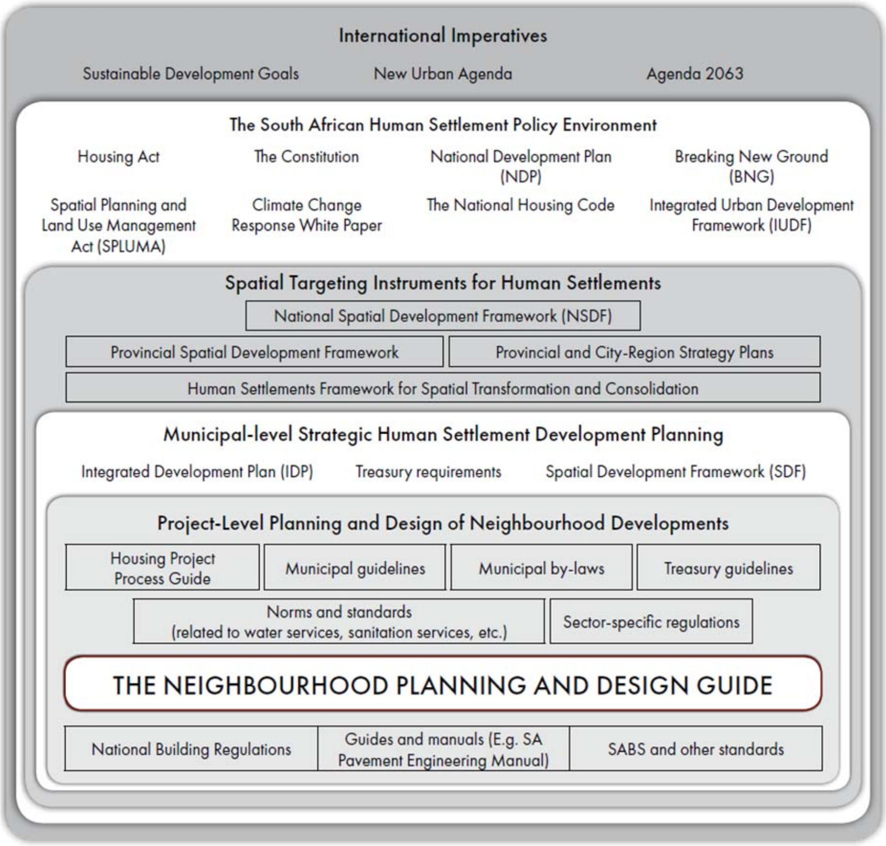

Figure 2-1: Planning Regulatory Environment (RSA, 2019)

Various other acts also have an impact on developments and in the context of floods, the key focus is on the engineer's role in ensuring that developments are planned and approved within a safe environment. A comprehensive guideline towards spatial planning has been provided in "Capacity Building Guidelines in Urban and Regional Planning. – A Guideline for Municipal Engineers and Engineering staff within Municipalities" (Jansen van Rensburg and Schoeman, 2016).

2.3.1. By-laws, Regulations and Strategic Documents

Acts, whether Provincial or National, are frequently followed by local regulations and by-laws governing a wide range of planning issues specific to a municipality. These are particularly important for aligning national legislation to local objectives. Where local by-laws and regulations relate to stormwater and flood management in a local municipality are available, it is critical that the design flood practitioner is made aware of this.

In many cases, the management of the legislative requirements is contained in strategic policy documents approved by applicable Councils such as the “Management of Urban and Stormwater Impact Policy” approved by the City of Cape Town (City of Cape Town, 2009).

This document, for example, contains the objectives and highlights the specific targets regarding floods to be considered in all planning processes, including the approval of building plans. The criteria are presented in the annexures of the policy and illustrated in Table 2-1.

Table 2-1: City of Cape Town Criteria for Sustainable Urban Drainage System Objectives (City of Cape Town, 2009)

SUDS OBJECTIVES		Greenfield Developments *and* Brownfield and Existing Development Sites located in catchments of sensitive receiving water systems	Brownfield and Existing Development Sites > 50 000 m^2	Brownfield and Existing Development Sites 4000 m^2 – 50 000 m^2 *and* Total impervious area (exist & new) > 15% of site	Brownfield and Existing Development Sites < 4000 m^2 *and* Total impervious area (exist and new) > 600m^2
CONTROL QUANTITY AND RATE OF RUNOFF	*Protect the stability of downstream channels*	24 hour extended detention of the1-year RI, 24h storm event	24 hour extended detention of the1-year RI, 24h storm event	Combination of on-site and regional off-site measures to achieve requirements as for development sites >50 000m^2	On-site runoff control measures not required but encouraged where practicable Regional off-site runoff control measures to be provided to achieve requirements as for development sites > 50 000m^2
	Protect downstream properties from fairly frequent nuisance floods	Up to 10-year RI peak flow reduced to pre-development level	Up to 10-year RI peak flow reduced to pre-development level		
	Protect floodplain developments and floodplains from adverse impacts of extreme floods	Up to 50-year RI peak flow reduced to existing development levels. Evaluate the effects of the 100-year RI storm event on the stormwater management system, adjacent property, and downstream facilities and property. Manage the impacts through detention controls and / or floodplain management	Up to 50-year RI peak flow reduced to existing development levels. Evaluate the effects of the 100-year RI storm event on the stormwater management system, adjacent property, and downstream facilities and property. Manage the impacts through detention controls and / or floodplain management		
		Developments adjacent to floodplains must adhere to the requirements of the Floodplain and River Corridor Management Policy			

This policy, in the case of the City of Cape Town, is further supported by the “Floodplain and River Corridor Management Policy” approved by the Council on 27 May 2009 (City of Cape Town, 2009b). Such a document provides valuable information regarding the technical challenges and the management principles used in the evaluation of developments exposed to possible flooding. The document, for example, relates the hazard risk associated with flooding with the level of expertise required to determine the flood lines, which then dictates the type of developments allowed by the Municipality. The classification of hazard in the context of flow depth and velocity associated with a particular flood/flow is illustrated in Figure 2-2 .

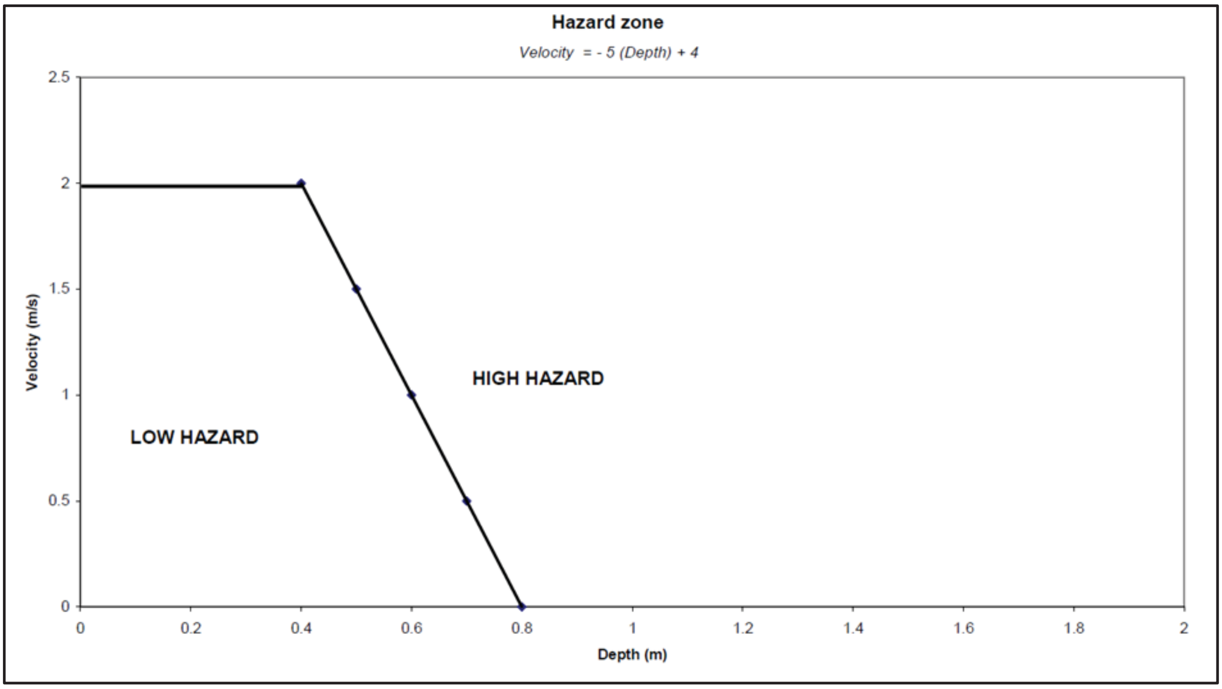

Figure 2-2: Flood Hazard Zones (City of Cape Town, 2009b)

The linkage between a specific type of development, the hazard level and the technical expertise required to determine the flood lines, is as an example illustrated in Table 2-2 .

The same policy process is also followed by other Municipalities, with eThekwini issuing by-laws (eThekwini, 2020a), which are supported by various strategy documents (Draft Flood Line and Stormwater Management Policy (eThekwini, 2020b), including the associated Flood line strategy – Annexure 1, Guidelines for Stormwater Drainage and Stormwater Management Systems – Annexure 2 and a Design Manual (eThekwini, 2018)). The latter all jointly providing the requirements for, amongst other issues, an understanding of floods and their impact on developments and how to deal with it in a practical way.

The legal requirement which must be adhered to by everyone in a specific municipal area is in most cases contained in the specific municipal by-laws. These by-laws can also contain a wide range of principles, which include, in the case of the City of Cape Town for example (see Appendix 2 for an example of the City of Cape Towns' by-law), aspects to protect their stormwater system, the prevention of flood risk and the studies required to establish flood lines. It also makes provision for penalties in the case of offences in terms of the by-law.

Table 2-2: Framework for the Assessment of Development Proposal (City of Cape Town, 2009b)

Table Shading Key	
Colour Coding	**Description**
Clear	Permitted
	Conditionally Permitted
	Not Permitted

Additional Requirement Key	
Code	**Requirement**
R1	A registered Engineering Professional must be engaged by the developer to satisfactorily demonstrate and certify that: • The activity / development will not materially increase flood hazards for other property owners or adversely affect flood behavior or the stability of river channels. • Any structure can withstand the forces and effects of flowing floodwaters, including scour of foundations, debris forces and buoyancy forces.
R2	Floors above 1:100 year flood level. Basements (non-habitable purposes) to be flood-proofed to 1:50-year flood level.
R3	Floors above 1:50 year flood level.
R4	A registered Environmental Professional (Aquatic Ecologist) must be engaged by the developer to determine the ecological buffer (if not available) and to satisfactorily demonstrate and certify that: • The activity / development will not negatively impact on the present condition of the watercourse or wetland OR • The activity or development will improve the condition of the watercourse or wetland from its present state

The land use / development / activity must be set back beyond the *greater* of the applicable floodplain zone / geomorphological or ecological buffer requirements

Land use / Development / Activity		Requirements and Conditions							
		Floodplain Zone (Flood Recurrence Interval in Years)						Ecological Buffer (Width in meters)	
Category	**Typical Examples**	**< 2**	**2–20**	**20–50**	**50-100**	**>100 (Note 1)**	***Explanatory Notes***	**Up to 75m (Note 2,3)**	***Explanatory Notes***
Industrial Development	Light, General, High Risk								
	Extractive (Mining)		R1	R1	R1				
Business Development	General				R2				
	Commercial (CBD)								
	Service Stations								
Residential Development	Formal				R2				
	Informal								
Community & Public Facilities	Hospitals, Clinics, Nursing Homes, Old Age Home								

Section 5 of the By-law (City of Cape Town, 2005) illustrated the intent of such a piece of legislation:

5. Prevention of flood risk

No person may, except with the written consent of the Council and subject to any conditions it may impose-

(a) obstruct or reduce the capacity of the stormwater system;

(b) change the design or the use of, or otherwise modify any aspect of the stormwater system which, alone or in combination with other existing or potential land uses, may cause an increase in flood levels or create a potential flood risk; or

(c) undertake any activity which, alone or in combination with other existing or future activities, may cause an increase in flood levels or create a potential flood risk.

In the case of the City of Cape Town, it is also the by-law that transfers the responsibility of the requirement to show the 1 in 100 year flood line on all building plans, as stipulated by the NWA, to the Developer as stated in Section 6:

6. Studies and assessments

(1) The conditions which the Council may impose in terms of Sections 3, 4, and 5, may include, but are not limited to -

(a) the establishment of flood lines,

(b) the undertaking of impact assessments, and

(c) environmental impact studies or investigations which may be required by any applicable environmental legislation.

(2) The costs of any study undertaken in terms of the provisions of subsection (1), will be for the account of the applicant.

The eThekwini by-laws (eThekwini, 2020a) use similar wording as those of the City of Cape Town under Section 5 to ensure a protected stormwater system:

5 (1) Subject to the written consent of the Municipality and to any conditions which the Municipality may impose, a person may not—

(g) change the design, the use of or modify any feature of the stormwater system which alone or in combination with other existing activities may cause an increase in flood levels or create a potential flood risk;

(h) undertake any activity which alone or in combination with other existing or future activities, may cause an increase in flood levels or create a potential flood risk;

The eThekwini by-laws (eThekwini, 2020a) also put the burden to indicating flood lines on the Developer as defined in Section 6 of the by-law:

6 (2) An approval of a development application is subject to—

(a) the submission by the developer of a stormwater management plan which is in accordance with the floodline and stormwater design requirements or guidelines as specified in the policy and in accordance with the requirements of regulation AZ4 of the National Building Regulations;

In the case of the City of Johannesburg, the Stormwater Management By-law (City of Johannesburg, 2010) defines a floodplain as:

"an area of land adjacent to a watercourse, or water body, with a catchment area exceeding 30 ha that will be inundated by floodwater on average once in a 100 years as determined by a professional engineer, on the basis that the minimum width of a floodplain is 32 m on each side of the centre line of the watercourse or water body"

The by-law of the City of Johannesburg then continues to set the requirements for a stormwater drainage plan stating that:

11. (1) A developer must in respect of any development site for which a permit is required in terms of section 7, prepare a stormwater drainage plan.

(a) A plan contemplated in subsection (1) must contain an analysis of the impact of stormwater quantity up to 500 m or a greater distance required by a notice in writing by an authorised official served on the developer concerned, downstream from the property on which the development site concerned is situated, which may result from the proposed development on that site and must contain features to mitigate such impact.

(b) For the purposes of paragraph (a), any existing and potential impact of stormwater, including - (ii) flooding;

The Stormwater Management By-law of the City of Johannesburg contains much more stormwater related detail (see Appendix 2 for an extract of the by-laws, as an example) than the strategy documents and design guidelines used by other large municipalities. In the case of the City of Johannesburg, the by-laws also defined the details required for an appropriate flood study / investigation, the required qualifications of the person who needs to do the studies and define a floodplain for example, not just only as the area below the 1:100-year floodline, but also any area within 32 m on either side of the centre line of the water course. In Section 38 of this by-law, for example, details are provided for stormwater quantity control, stating that:

38. (1) Subject to the provisions of subsection (2), the following requirements for stormwater quantity control apply:

(b) the post-development peak stormwater discharge rate from a development site for a 5- to 25-year recurrence interval design storm event of any duration from 0.25 to 24 hours, or any other design storm event stipulated by the Agency up to and including a 50-year design storm event, may not at any time exceed the pre-development peak stormwater runoff rate from that site for the same design storm event;

and

(d) If a proposed development will result in a discharge of stormwater to a closed natural depression that has a water surface area greater than 500 ha at overflow

elevation, the following requirements must be complied with for the purpose of an analysis contemplated in paragraph (c):

(i) the stormwater runoff hydrograph from a 100-year design storm event of any duration from 24 hours to seven days from the pre-development catchment area draining to a closed depression contemplated in paragraph (c), must be routed into that depression using only infiltration as outflow from the depression;

In smaller municipalities, like the Bergrivier Municipality (Bergrivier Municipality, 2009), the reference to flooding is limited to a section in the by-laws stating typically that:

4.(1) No person may, except with the written consent of the municipality,

(c) undertake any action that is likely to destroy, damage, alter, endanger or interfere with the free flow of water or the stormwater system, or the operation thereof, which action includes, but is not limited to—

(vi) changing the design or the use of, or otherwise modify any feature of the stormwater system which alone or in combination with other existing or potential land uses, <u>may cause an increase in flood levels or create a potential flood risk</u>; or……

Many Local Authorities however do not have any reference in any by-laws or policy documents to the requirement for the need to show flood lines, as required by the NWA.

Municipalities must:

- **Promulgate appropriate By-Laws, clearly indicating the requirements of any developments in relation with flooding.**
- **By-laws can refer to strategic policy documents, approved by Council, which contains the detail of all the requirements.**
- **By-laws/strategic documents must contain criteria for Flood hazard. Not only a flood line.**

2.3.2. Guidelines for the Development within a Floodline – DWAF

The Department of Water and Sanitation (previously known as the Department of Water Affairs and Forestry) compiled "Guidelines for the Developments within a Floodline" (GDF) in March 2007 (DWAF, 2007) to provide some interpretation of the NWA. It is the interpretation of the DWS in the GDF that a Developer needs to make the information regarding a 1 in 100-year flood line available and it is also interesting to note that the NWA does not exclude developments below the 1 in 100-year flood line, but merely requires that this information is made available.

The GDF states that it is "common knowledge" that no developments be allowed under the 1:100-year floodline level, but still provide some guidance in Table 2-3. This may limit effective land use planning and impact on land values which is important to municipalities.

The GDF also refers to Section 21 activities of the NWA, and to mining activities that may also be relevant to certain municipalities. Since Section 21 (c) [*impeding or diverting the flow of water in a watercourse*] and (i) [*altering the bed, banks, course or characteristics of a watercourse*] of the NWA requires a licence application process, the GDF suggest that no such activities are allowed without a licence:

a) Below the RMF level, if such activity results in a rise of water level that can have an adverse impact on adjacent properties.

b) Below the 1 in 100-year floodline, plus a 20 m buffer zone.

Table 2-3: Developments that may be considered in Different Zones (Guidelines for Developments within a Flood Line (GDF), DWAF, 2007)

Flood Risk Band	Development Description
<u>RMF to 1:100 year</u>	Any structures: • If the risk is pointed out to the occupants • If an adequate escape route exists
<u>1:100 year to 1:50 year</u>	• No structure that results in a loss of flood storage from the system • No fill, dykes, levees or beams intended to restrict the area of floodplain inundated • No structure not designed by a Structural Engineer to withstand the floodwater load • No ground floor on which people sleep at night • No sewer lines
<u>1:50 year to 1:20 year</u>	• No permanent structures, except bridges (this includes swimming pools, tennis courts, brickwork gazebos • Temporary structures that do not interfere with the function of the floodplain as an ecological corridor
<u>1:20 year to 1:10 year</u>	• Only ground level modifications that do not reduce the permeability of the floodplain soil
<u>Below the 1:10 year</u>	• Approved water abstraction facilities • Landscaping with very minor earthworks and planting with local indigenous riparian vegetation

2.3.3. The Neighbourhood Planning and Design Guide

To understand the full planning process within municipalities, reference must be made to "The Guidelines for Human Settlement Planning and Design" (Redbook) (CSIR, 2000) and "Sustainable Human Settlement Planning: Resource Book" (RSA, 2008). These guidelines have been replaced recently by "The Neighbourhood Planning and Design Guide" (RSA, 2019).

"The Neighbourhood Planning and Design Guide" (RSA, 2019) provides valuable input towards the planning framework for floods. The guide confirms the legal requirement of the indication of a 1:100-year flood line on all building plans in terms of the NWA, but adds that a more conservative approach is needed in some cases, indicating the need to use the RMF/PMF as guiding principle. The guide also suggests the use of Australian guidelines as reference where flood lines for specific services are needed, varying from the 1:200 to 1:500-year recurrence interval, depending upon the importance of the service linked to the building under review. Table 2-4 provides a summary of the proposed recurrence intervals needed for planning of various services.

Table 2-4: Design Recurrence Intervals of Essential Community Infrastructure (RSA, 2019)

Type of essential community service	Design Flood Recurrence Interval
Emergency Services (fire station	500 years
Emergency services (emergency shelter)	200 years
Emergency services (police station)	200 years
Hospital and health care services	500 years
Community facility (storage of valuable records or items of historical or cultural significance, e.g. galleries and libraries	200 years
Power station or renewable energy facility	500 years
Major electricity infrastructure (major switch yard)	500 years
Substations	200 years
Utility installation (water treatment plant)	200 years

Proposed minimum requirements:

- **Below 1:20 year flood lines:**
  - **No developments allowed, besides abstraction/discharge works.**
- **Between 1:20 and 1:50 year flood lines:**
  - **No permanent developments / structures, except bridges.**
- **Between 1:50 and 1:100 year floodline or 32 m either side from the centre of the watercourse, whichever is the largest):**
  - **No structure that results in a loss of flood storage from the system.**
  - **No fill, dykes, levees or beams intended to restrict the area of floodplain inundated.**
  - **No structure not designed by a Structural Engineer to withstand the floodwater load.**
- **No ground floor on which people sleep at night**

2.4 Reporting Standards

Within the context of the legislation discussed in this chapter, it is imperative that municipalities do take the necessary steps to ensure that reasonable precautions are taken to ensure a safe environment. One of these precautions is to ensure that all service providers/staff responsible for flood investigations do have a clear expectation of the content of the report on any flood studies required by the municipality. The typical content of a standard reports, using the Cities of Tshwane and the eThekwini Municipality as examples, are presented in Appendix 2.

2.5 Interpretation of Legislation

Flood lines are a critical element in the planning and development of infrastructure for all municipalities. It is clear from the legislation that at least the 1:100-year flood line needs to be determined and shown on any development plans. It can also be assumed, given the requirements from the Constitution related to cooperative governance and a need to provide for a safe environment, that local municipalities must ensure that they are well advised on the flood regime, not only for infrastructure planning, but also for disaster management risk assessments in their areas of jurisdiction. They must also ensure that the 1:100-year flood lines are clearly shown on all planning documents. This legal requirement is in almost all cases transferred to the developers (but the legal responsibility still rest with the municipality), mainly through by-laws and strategic documents. According to Thompson (2022) there exist a potential of an infringement of common-law rights, when organs of state, like municipalities, proceed with developmental plans that might be impacted by floods. Municipalities typically allow developers to proceed based on a written agreement and on condition that they adhere to the requirements set out in policy documents and by-laws. In a case of damages due to wrongful action related to stormwater, the challenge will be to proof that the municipality did not take “reasonable practical precautions” to prevent damages. This burden of proof clearly needs to be considered carefully by municipality in so far as it refers to the content of their by-laws or the reasonable steps followed, for example to accept a flood line derived by a specialist, taking all relevant circumstances into consideration or making sure that a flood line expressed the expected fully developed catchment conditions.

In a case of, for example, incorrect flood calculation methods used by the engineer / developer, that developer / engineer will be held responsible under common law rights. Similarly, the municipalities might be held responsible if they cannot proof that they did follow reasonable practices before accepting a developer’s plan.

Floods and the associated risk or hazard need to be determined by an experienced Engineer,

> Municipalities remain legally responsible to ensure safety and needs to be in a position to ensure that **all reasonable steps have been taken** (via By-laws and clear flood calculation reporting requirements) to ensure a safe environment when approving, or allowing work to be done, which expose property and/or lives to flood damages.

and the requirements and level of assessment must be defined and incorporated in strategic guidelines that need to be referred to in appropriate stormwater by-laws.

It is preferable that the number of technical details provided in the by-law are limited, and that the necessary details are rather provided in strategy documents that can be adapted and changed more easily than by-laws.

The application of flood determination methods is not an exact science and needs to be done with great care. Municipalities should consider the possibility of using levy income from development fees to contribute to a coordinated integrated flood determination approach for the full river system by a single service provider.

Regular (5 yearly) updates are required to ensure an active and applicable understanding of the flood regime.

Flood studies should not be limited to the legal required 1:100-year flood line, but should, in all cases, include the RMF, 1:50, 1:20, 1:10 and 1:5-year recurrent interval to allow for local preferences and requirement regarding the allowable activities/structure, taking the risk profile into consideration. In specific instances, as given in Table 2-4, determination of the 200 year and 500-year recurrence interval flood lines is also required.

Municipalities must:

- **Regularly update By-laws and or strategic documents ruling the requirements for developments related to possible flood hazards.**
- **Appoint experienced engineers to calculate flood lines and flood hazard zones.**
- **Ensure that the 1: 5 to 1:100 year, as well as the Regional Maximum Flood lines are shown on all development plans.**

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3. INTRODUCTION TO FLOOD ESTIMATION PRINCIPLES

3.1 Chapter Overview

Design flood estimation (DFE) is one of the cornerstones for flood risk management and limits the risk of the estimated design flood being exceeded. Indicative and certified flood lines, as well as hydraulic infrastructure, e.g. attenuation dams, culverts, bridges, channel widening, and flood bypass systems are examples of which require the estimation of design floods. Inherent in risk assessment is the combination of probability and consequence (impact). Typical consequences include the potential for hazard to human life, loss, or damage of property or to the environment, and possible long-term economic losses and societal impacts. Therefore, accurate and consistent DFEs that provide a balance between overdesign to avert the flood risk and under-design to minimise the costs and negative impacts, are critical input in the design of and management of hydraulic structures.

Historically, the demands for improved estimates have to some extent resulted in an increased understanding of the fundamental hydrological processes involved; however, the unavailability of data remained a challenge (Cordery and Pilgrim, 2000; Gericke, 2021; Smithers *et al.*, 2021) As a result, DFEs have relatively wide confidence bands due to the nature, occurrence, and frequency of flood events, along with the uncertainty associated in the flood estimates when using an array of methods as is currently advocated for practice in South Africa. In other words, despite the advancement in understanding and DFE methodologies, estimates are still somehow uncertain. As highlighted above, data are usually the limiting factor, particularly rainfall, runoff and catchment characteristics. In the past, practitioners were left to balance two conflicting demands:

(a) Time and cost for detailed scientific investigations and analyses.

(b) Taking short-cuts for a more time efficient and lower cost prediction.

The difference in application time and cost between the different methods can be considerable. Therefore, most of the DFE methods in use have been developed to provide practitioners a more defendable case for Option (b), with reliance placed on convention, professional judgement, experience, and precedents in similar DFEs. However, with increased availability of data and substantial improvements in computing capability, there has been a shift towards more comprehensive catchment analysis in urban catchments. Practitioners are now able to analyse catchment responses at high resolution, simulate runoff responses to a time series of rainfall data and perform sensitivity analyses, all of which go a long way to mitigate the uncertainties inherent in DFE. However, there is concern about the limited understanding of DFE and best practices amongst municipal officials and some practitioners. Liability for DFE will be directed to the design flood practitioner, with qualifications and registration status as defined by the local authority involved. In general, as detailed in Section 3.2, it is recommended that design flood practitioners should be either a professionally registered engineer (Pr Eng), technologist (Pr Tech Eng), or natural scientist (Pr Sci Nat), all with the required experience in engineering hydrology. The extent of liability is linked to the risk and consequence of flooding, as well as the impacts of the proposed flood management measures, all of which need to be considered in the estimation of design floods.

The municipality/local authority as an institution, and municipal officials, may not be exempted from liability in DFE. They are responsible for, amongst others, the appointment of suitable DFE professionals, the terms of reference given to the professionals, and ensuring continuity of the DFE methodologies applied along the same watercourses. Municipalities and their officials may also be liable for changes in downstream flood risk as a result of upstream development or land use changes in a catchment over time.

The limitations of many of the DFE methods commonly applied in South Africa are recognised and a National Flood Studies Programme (NFSP) has been initiated to update and modernise these methods (Smithers *et al.*, 2014). Hence, practitioners need to keep up to date with new developments to ensure they are applying acceptable best practice methods and thus limit their liability in the case of a failure of a hydraulic structure/system.

The following sections contain information covering the requirements, qualifications and experience required for flood estimation, concepts and design philosophy, flood risks, disaster and hazard mapping, sustainable drainage systems and water sensitive urban designs, catchment characteristics, and consistency in DFE approaches.

3.2 Requirements for DFE

The following requirements for flood estimation should be adhered to:

(a) The responsible person should be professionally registered (Pr Eng, Pr Tech Eng, or Pr Sci Nat) with appropriate experience in DFE. Hence, the appointment of a practitioner to undertake DFE based on price alone is inappropriate and could attract liability to the Local Authority

(b) The practitioner and Municipal Engineer should agree on the level of detail required in the analysis (e.g. an indicative estimate or certified design value). The application of multiple appropriate methods should be considered, and the results from the most appropriate (primary) method should be used to benchmark with other methods and quantify uncertainty.

(c) The selected primary method must be appropriate for the catchment size, complexity of land cover, soils, and hydraulic influences on the flood peak. This may highlight the need for a discretised or distributed catchment analysis.
(d) The Municipal Engineer must confirm the design flood standard (level of service) in each application and the associated residual flood risk (i.e. the risk for events larger than the design flood standard).
(e) The Municipal Engineer should consider disaster management implications after the outcome of (d) and communicate with the disaster management department.

The roles of design engineer and municipal engineer may be assigned to different individuals, in which case certification of a flood line should be signed by all involved and/or responsible professionals. Similarly, municipal officials responsible for catchment planning, flood management, and the appointment of practitioners should also be professionally registered and able to demonstrate the necessary competency and experience. Amongst other requirements, the municipal professional is required to either set and/or apply the standards for DFE in the relevant municipality.

3.3 Concepts and Design Philosophy

Various conventions and terminology, e.g. flood, flood hydrograph, flood plain and flood lines, have developed around the basic concepts that are applicable to DFE, all of which can be found in Alexander (2001). Given the importance of flood lines in terms of risk management and mitigation, these are detailed below.

Flood lines are used as a measure of flood risk along a watercourse. As such, they are useful for planning development and land use in floodplains. Unfortunately, in municipal areas, they have simply become development boundary lines, standardised at the 100-year annual exceedance probability (AEP) or return period (T), often without consideration of accuracy, consistency, risk, and associated land use options. Recently, there has been the emergence of two flood line standards (Brooker and Dunsmore, 2022):

(a) <u>Indicative flood line</u>: This is intended for general planning, but not for design or for development layouts (i.e. it is not in accordance with the National Water Act (NWA) requirement). This is generally derived by high level hydrological and hydraulic analyses as a "quick" assessment of flood risks.
(b) <u>Certified flood line</u>: As envisaged in the NWA, this is a flood line prepared and certified by expert practitioners as being based upon the best available information and practices.

The amount of effort in DFE will differ between these two standards. There is typically very little liability attached to (a), whilst certification (b) clearly attaches liability to the practitioner. Hence, the practitioner should be able to demonstrate due care and use of acceptable methods in preparing the flood estimates and flood lines and should indicate the level of guidance provided by the municipality. In the event of a claim, the municipality's role in the DFE may also be scrutinised.

3.4 Flood (Residual) Risks

Risk is a combination of probability and consequence. For example, it can be used to determine the design standard for a new land use or structure adjacent to a watercourse. If the consequences are potentially very high (e.g. loss of life), the exceedance probability of the design flood event is likely to be set very low (e.g. 0.5% or 1:200-year or even lower). If the consequences are low (e.g. shallow, slow moving flow depths in a car park), a higher exceedance probability of the design flood may be acceptable (e.g. 10% or the 10-year event). Hence, this implies that a consistent approach to DFE is adopted by a municipality. Even though there may be an inclination to adopt a more conservative estimate of the design flood if there are potentially more severe consequences concerned, this must be avoided as this would lead to an inconsistent approach towards DFE.

Hence, different design flood standards (level of service) are applied in municipal systems (e.g. 2-year & 5-year for stormwater networks, up to 100-year for detention facilities), and a municipality may consider extending the approach to other combinations of land use and hazard. This approach creates different opportunities for land use planning in a floodplain and allows the municipality to optimise land development along a watercourse. It suggests that simply adopting the 100-year design event as the standard development control line may not be in the best interest of the municipality, given that development control and risk to infrastructure may not be the only criteria. Typically, the municipality must also consider the ecological, urban aesthetic, and sociological benefits associated with the floodplain. Hence a municipality has the option to adjust the guideline standards to meet their own land use and risk criteria. As such is it recommended that municipalities clearly set out the design flood standards to be applied in their area of jurisdiction.

Once the standards are set, it is recommended that a consistent approach to DFE is adopted by a municipality. This is intended to minimise subjective influences on design flood estimates. Examples include the application of safety factors in the DFE process, or applying a standard other than that recommended to alter the DFE output. For example, the use of a Type 2 or Type 4 storm profile instead of the Type 3 profile recommended for the location, and where the flood peak estimates are lower or higher respectively. Any such adjustments or factors of safety need to be approved by the professional municipal practitioner. (Also see Chapter 10 on consistency vs accuracy.)

A DFE practitioner has an important role to play in this process and following best practice methods is essential. The practitioner will work closely with the municipality and planners to determine the best design flood policy, which may set a range of standards for different locations and land uses. Complete protection can never be guaranteed and there is always some residual risk of failure when events exceed the design flood standard. Policy decisions have to be made on the level of risk that is acceptable in a given situation. Examples of risk factors to be considered, include:

(a) Loss of life.

(b) Damage to structures.

(c) Interruption of transport and communications.

(d) Interruption of services (e.g. the isolation of communities during floods, cut-off from hospitals, water supply, sanitation, food, etc.).

3.5 Disaster Management and Hazard Mapping

Flood risk assessment can be enhanced by the preparation of flood hazard maps. Flood lines are usually presented as a simple line on a plan. While this may comply with NWA requirements, it provides little information on risk. In contrast, flood hazard mapping provides important information for both development control and disaster management.

Hazard conditions are those where potential negative impacts may occur and are an estimate of the consequences of an extreme flood event. Focus is typically limited to an analysis of flow depth and velocity, but hazard mapping should include indications of the rate of rise, duration and special hazards such as pseudo-islands that are potentially cut-off by flow in secondary channels. In addition, design guidelines are often only developed for people and vehicles, but structures and dwellings, particularly informal dwellings should be included. Software applications, e.g. HEC-RAS provides a real-time video representation of the expansion of inundated areas, which can be much more informative than static maps.

The broad principles for hazard management for people in floodplains are demonstrated in Figure 3-1. In riparian areas, where people are likely to be during a storm, a different design flood standard may be considered. These areas will include recreation sites or land uses such as car parks.

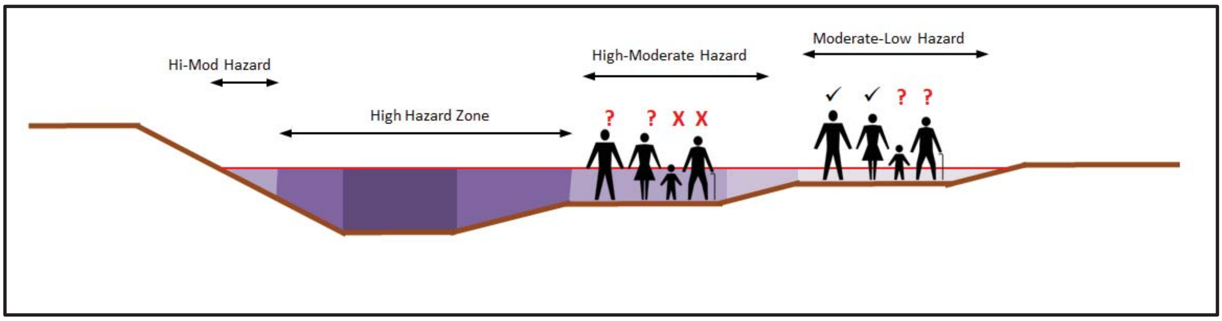

Figure 3-1: Principles for flood hazard zoning for risk to people (Dunsmore, 2022)

The Australian Institute for Disaster Resilience (AIDR) has prepared a series of technical guidelines that include a method for breaking down the floodplain based upon the varying combinations of velocity and depth associated with impacts on people, vehicles and buildings (AIDR, 2014).Figure 3-2 indicates flood hazard categories related to flood depth and velocity. A simplified version of this has been adopted in the City of Cape Town (Figure 2-2), but given the increasing development densities and complex land uses, including informally developed areas, it is expected that the standards for hazard management will need to be raised beyond the AIDR guidelines and such aspects as the rate of rise, flood duration and special hazards need to be included. DFE will need to support this development and full flood hydrograph analysis with attention to storm response timelines will be critical.

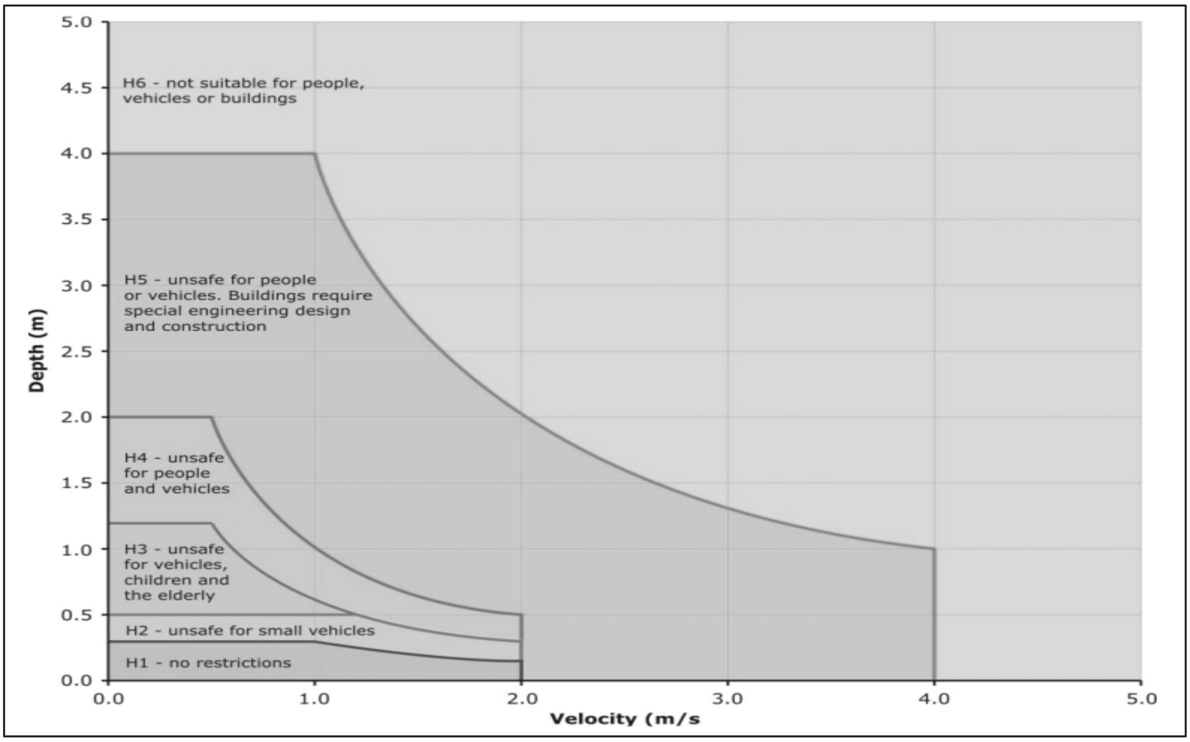

Figure 3-2: Flood hazard vulnerability curves adopted in Australian Disaster Resilience Guideline 7-3, 2014 (after McLuckie, 2014)

Most hydraulic analysis software packages, e.g. HEC-RAS, can map flow depths and velocities for hazard mapping. Figure 3-3 shows an example prepared in HEC-RAS. It presents the estimated 10-year flood conditions along an urban stream where both sports facilities and children's playing areas exist on the right bank. It indicates children will be at risk of being swept away in portions of the playing area. It also shows that adults will be in danger further downstream if attempting to recover a child. The flood hydrograph insert shows the flood response from the urban catchment to be "flashy," thereby increasing the risk of severe consequence if no suitable early warning system is in place.

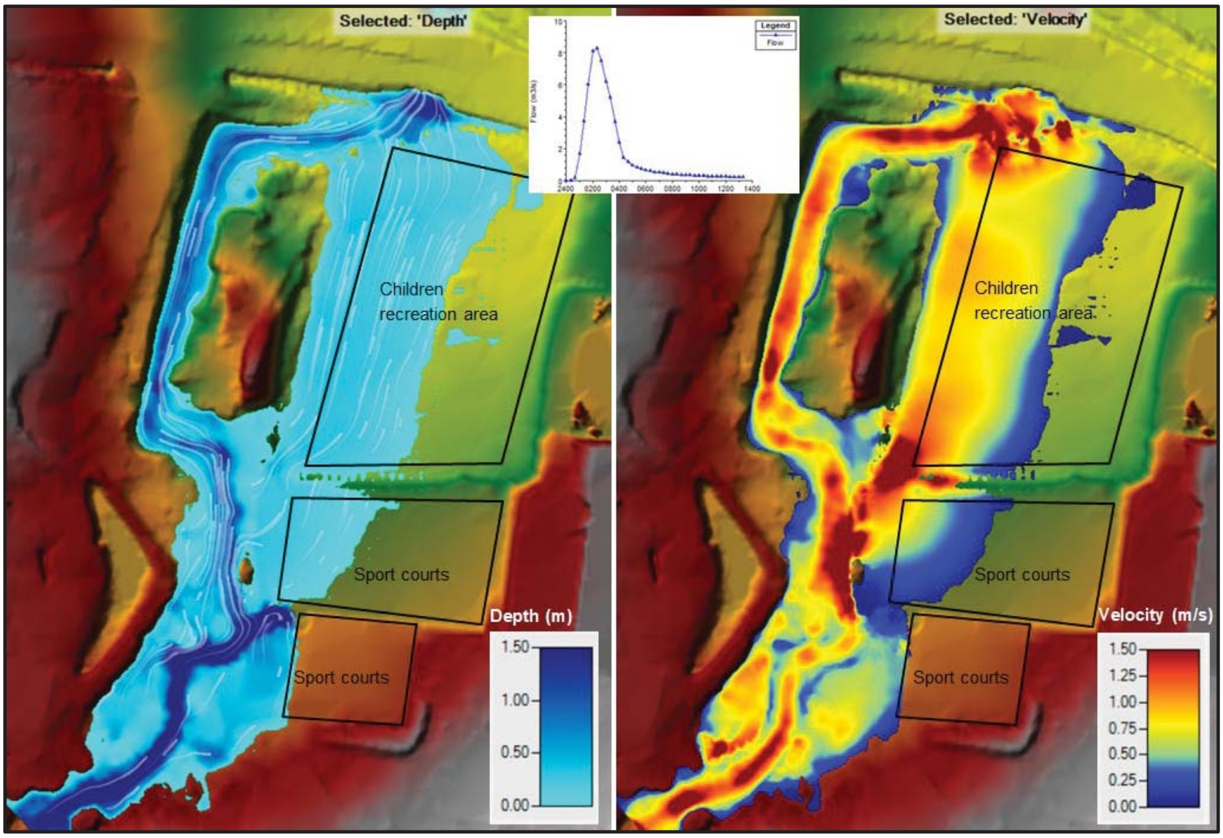

Figure 3-3: Example of flood depth and velocity analysis for hazard assessment (flow direction from bottom to top) (Dunsmore, 2022)

3.6 Sustainable Drainage Systems and Water Sensitive Urban Design

Sustainable Drainage Systems (SuDS, Armitage, 2013) and Water Sensitive Urban Design (WSUD, Armitage, 2014) are required for the planning and design of urban stormwater systems that seeks to mitigate the effects of impervious urban surfaces on receiving watercourses by mimicking rainfall-runoff responses from natural catchment surfaces. Hence, most facilities used in SuDS seek to reduce the storm runoff volume by directing runoff from impervious surfaces to “active” permeable spaces that enhance infiltration capacities. Such facilities include permeable paving, bioretention units, infiltration trenches, vegetated swales, etc. Although detention facilities may be currently considered part of conventional urban drainage, they are also important SuDS measures.

How SuDS affect DFE will depend on their storage capacity and their distribution across the catchment. It should not be assumed that SuDS will always reduce flood peaks. Like any system that relies on temporary storage to reduce peak flows, antecedent conditions are important. Additionally, their performance range is generally limited to low order (typically <10-year event, see below), and flood relief for higher order events will be limited.

WSUD measures include the range of SuDS indicated above and may also include harvesting measures that reduce the overall runoff from a site or urban catchment. Rainwater harvesting is one measure that is seeing wider application in South Africa. Other measures linked to stormwater harvesting (harvesting runoff in parts of the network) and aquifer recharge are less common but are expected to receive increasing attention in the future.

Assessment of the flood reduction effect of these systems will need to consider whether there is any reliable storage available at the start of flood producing storm events and if there are any hydraulic constraints in receiving the stormflow. Hydraulic constraints could be infiltration capacity or recharge capacity in the case of managed aquifer recharge wells. These aspects need to be carefully considered by the practitioner.

The latest Red Book published by the Department of Human Settlements (DHS, 2019), contains the guidelines shown in Table 3-1 and Figure 3-4, respectively. Note that, while volume control is assigned to design events up to the 2-year return period, the design of detention facilities is still required for events larger than this and volume control must be considered.

Table 3-1: Proposed design return periods for municipal stormwater management (DHS, 2019)

Return period (years)	Objective/component	Treatment
0.25 to 0.5	Interception storage, water quality volume including recharge volume.	None or good housekeeping or source or local controls or combinations.
0.5 to 2	Channel protection volume.	Source and local controls.
2 to 10	Flow control for minor storms.	Local and regional controls.
10 to 20	Flow control for major storms.	Roadway and regional attenuation.
> 20	No damage allowed.	Major design interventions.

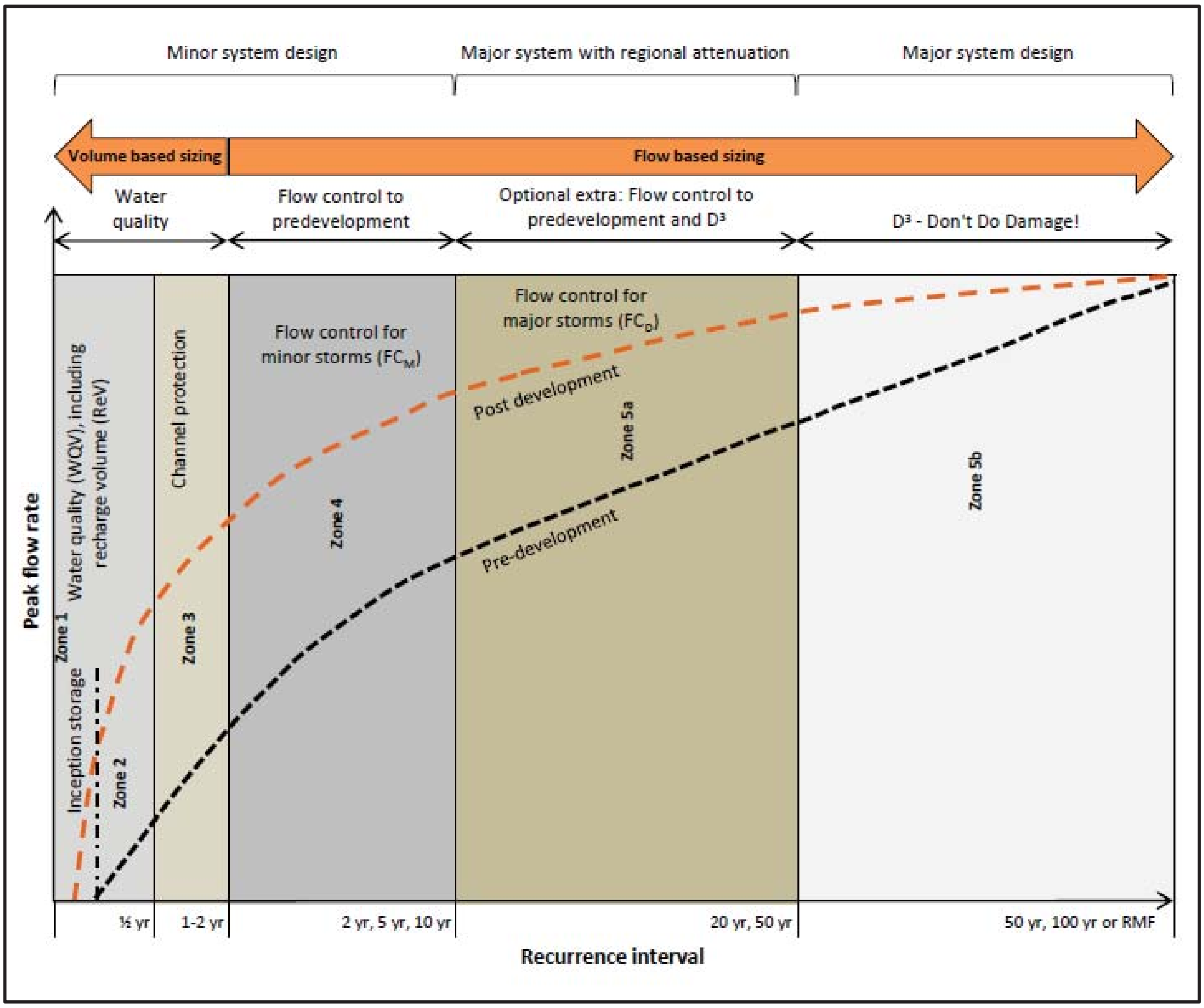

Figure 3-4: Conceptual stormwater design framework (after Armitage, 2014; DHS, 2019)

Figure 3-4 shows that the flood performance of SuDS facilities (source, local, and regional controls) is limited to events up to the 10-year storm event. Hence, the guide in Figure 3.4 is the best available for DFE. As a result, DFE practitioners should adopt the following as a conservative approach, while taking cognisance of the specific catchment conditions and being continuously aware of any new research findings:

(a) The effects of formal detention facilities in a catchment may be accounted for in the estimation of all design event probabilities. A conservative approach is to assume the effect of upstream attenuation to be negligible, but this can lead to overdesign and unnecessary cost, but there is the risk that municipalities lose sight of the value of integrated planning of urban networks, and so neglect maintenance. As high level planning guide, detention facilities comprising less than 4% of the total catchment surface area are unlikely to significantly reduce flood peaks and can be ignored (Görgens and McGill, 1990)

(b) For design events greater than 10 years, the effects of SuDS in a catchment can be ignored. Assume that runoff from all impervious areas reaches the drainage network within the duration of the design storm.

(c) For design events of 10 years and smaller, the impervious areas that are treated by SuDS measures may be regarded as pervious in the DFE. However, as a precaution, it is suggested that this may be done on a sliding scale, based upon an understanding of the SuDS in the catchment. For example:

i. For events up to and including the 2-year storm event, all treated impervious areas are considered “pervious”.
ii. For events between the 2 and 5-year storms, 50% of the treated impervious areas are considered “pervious”.
iii. For events between the 5 and 10-year storms, 25% of the treated impervious areas are considered “pervious”.

The application of SuDS in South Africa is still relatively new and experience of widespread SuDS at a catchment scale is limited. The above approach is only applicable to those impermeable areas of a catchment that are treated by SuDS. It is recommended that, if less than 10% of the impermeable areas of a catchment are treated by SuDS, the effect thereof on DFE may be ignored.

3.7 Catchment Characteristics

Deterministic DFE methods that are reliant on detailed catchment characteristics, are the main approaches to DFE presented in this guideline. Hence, determining catchment characteristics is an important part of minimising the overall uncertainty involved. The key principle is to measure accurately what can be measured, before using best judgement estimating the remaining characteristics.

Typical catchment characteristics include area, slope, shape, drainage system density, land cover, and the direction of the catchment slope relative to the direction of movement of severe rainfall producing weather systems. Data sets of spatially referenced information are available and are constantly improving in levels of detail. These data sets are well suited to hydrological analysis of large rural catchments, while being a useful reference in smaller urban catchments; however, land development activities can substantially change the hydrological characteristics of some of these parameters, e.g. changes in soil conditions and the effects of compaction. Factors affecting surface runoff detention may also not be evident from data sets or aerial imagery, stormwater assets, boundary walls, detention ponds, etc. Changes in conveyance systems (drains, streams, etc.) can affect catchment response times. For example, erosion of urban streams may increase channel conveyance efficiency and the backwater upstream of road crossings, may increase storage in the system.

Using data sets, even those referenced in this guideline, and other sources (e.g. Google Earth), may not replace site visits and field investigations. These do remain a standard part of best practice and it is left to practitioners to decide and report on the approaches used to determine catchment characteristics.

It should be noted that catchment and channel geomorphology (shape characteristics) can dominate other catchment characteristics in determining flood response time and peak discharges. For example, Gericke (2019) highlighted that shorter response times and higher peak flows are evident in similar-sized catchments characterised by lower shape factors, circularity ratios, and shorter centroid distances and associated higher elongation ratios, drainage densities and steeper slopes. Hence, particular attention is warranted on determining these characteristics. Please refer to Chapter 5, Section 5.6 for all the details related to catchment characteristics.

3.8 Consistency in Approach

Flood line continuity along a stream or river requires consistency in both DFE and hydraulic calculation methods, with the municipality being responsible for the oversight of both. DFE consistency refers to the application of the same method(s) and the same data sources. It is recommended that any departure from standard data sources is well motivated and preferably limited to reasons such as model calibration and site measurement. Applying factors of safety should be avoided. Guidelines for ensuring consistency in DFE approaches are presented in Chapter 4, Section 4.5, and discussed further in Chapter 10.

Factors that may change along the length of the stream or river should also be managed in a consistent manner. The example in Figure 3-5 represents a potential situation on an urban watercourse where flood lines may be prepared for different reaches at different times, and consistency in the DFE approach will be important. Nodes are identified at locations along the main watercourse. At each junction with a tributary there is a step increase in the contributing catchment area, while changes in catchment areas between the nodes are relatively small. Similarly, the time of concentration (T_C) will increase along the length of the main watercourse, implying the design rainfall conditions will change along the length of the watercourse. T_C and the catchment area should be analysed as a discretised system along the watercourse.

For example, in the determination of a flood line for Reach B-C, a practitioner may choose to apply the design peak flow or hydrograph at Node B to the hydraulic model for the flood line for Reach B-C. Alternatively the practitioner may select a more conservative approach and apply the design flood estimate at Node C to the hydraulic model for Reach B-C. Either approach may be defendable depending on the scale of the systems and the size of tributary catchments, but it is recommended that the municipality establishes a common approach along the entire watercourse.

Another example where problems can arise is at the junction of a large tributary that is analysed as a sub-catchment. The error is to assume the T-year event can simply be added to the T-year event on the main watercourse to represent the same T-year flood magnitude at the confluence. Technically this is incorrect, though the error may be small to negligible on small urban systems where the size of the design storm cell could easily cover the combined area of the two catchments and the design flood estimates could be produced by the same storm. However, on larger urban catchments where the storm cells for design event conditions are likely to be two separate storm systems, the likelihood of the coincident storms will not have the same return period as the initial T-year event. This approximation is reasonable if the contributing catchment area along the reach between Nodes B and C is small (<10%) in relation to the Tributary (C1) catchment area at Node C. This approach is recommended. However, the approach that determines separate *T*-year events for both Reach B-C and Tributary C1 and then simply combines the hydrographs at Node C, is not recommended.

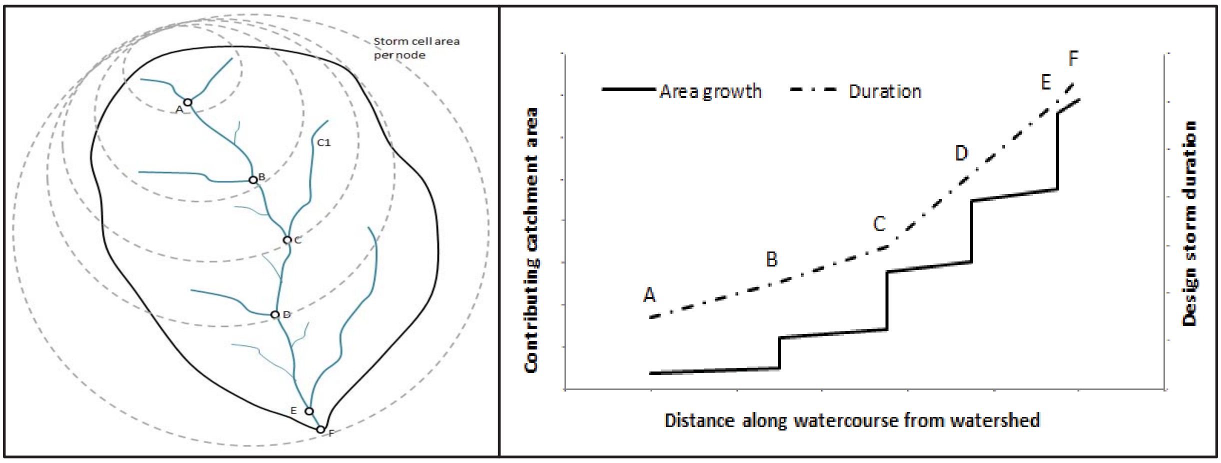

Figure 3-5: Application of variable storm durations and catchment areas in DFE (Dunsmore, 2022)

The municipality should confirm the approach to be adopted but this may rely upon the DFE practitioner to determine the location of the nodes along the watercourse. The municipality should also confirm the state of the catchment to be used in the DFE, i.e. present day condition of development, or some future state (e.g. as described in the municipal Integrated Development Plan). Further details are provided in Chapter 5, Section 5.5.

Finally, the municipality should also confirm whether the flood lines will be determined assuming a steady hydraulic state in the watercourse (i.e. only peak design discharge estimates required) or if a flood hydrograph (i.e. peak design discharge and runoff volume) needs to be routed along the watercourse. The latter is an important consideration where attenuation in the watercourse will lower the peak discharge rates as the flood hydrograph progresses downstream. Attenuation does occur naturally (in river reaches, floodplains and/or wetlands) or due to engineered infrastructure (attenuation ponds, dams, weirs, multiple bridge crossings, etc.). It should, however, be recognised that the lagging and attenuation of a hydrograph in a water course will retard the hydrograph, resulting in a longer critical storm duration.

The next chapter includes a "DFE Road Map" to provide an overview of the subsequent chapters in this best practice guideline which focusses on DFE, CSM and river hydraulics applicable to the design of hydraulic infrastructure in an urban environment.

3.9 Chapter References

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4. DESIGN FLOOD ESTIMATION: ROAD MAP

4.1 Chapter Overview

In selecting the most appropriate DFE methods, practitioners need to understand the methods, their assumptions, the data used in their derivation and the advantages and limitations associated with each method. Lack of knowledge or incorrect application of a method will contribute to professional liability in the event of a failure of a hydraulic structure or drainage system. Practitioners should thus be aware of the need to use best practice approaches, including updated or new methods of DFE when these are available for application in South Africa.

Deterministic methods are well suited to provide the output needed for municipal DFE (peak flow, volume, and duration) and can be applied in an event-based and/or CSM mode. The deterministic event-based methods recommended (see Section 4.5) for application in urban areas are the Rational Method (Section 7.2) and SCS-SA (Section 7.3) method. Deterministic CSMs are recommended for both event-based analysis and continuous simulation (Chapter 8) as they can include support for storm water designs and generally use sub-hourly time steps which is best suited to urban flood estimation, but their application may be limited by the availability of sub-daily rainfall data in which case event-based approaches can be considered. The only empirical method recommended for application is urban areas is the Regional Maximum Flood (RMF) method (Section 7.5), which provides an estimate of the maximum expected floods based on historical observed flood events.

Given the lack of DFE guidelines in South Africa, the application of more than one appropriate DFE method is recommended as good practice. The selection of the best flood estimate from the range of values obtained requires professional judgement and experience and can provide an indication of the uncertainty associated with the selected value.

4.2 Introduction

As illustrated in Figure 4-1, rainfall on a catchment is transformed into runoff from consisting of surface, subsurface and groundwater flows. The catchment characteristics define the nature of the transfer function, since rainfall losses occur as the catchment experiences a change in storage, while it absorbs (infiltration), retains, or attenuates (surface depressions or storage basins) and loses some of the rainfall through groundwater seepage and evaporation. Runoff therefore consists of two components: (i) stormflow (sometimes referred to as excess rainfall in flood studies) resulting from stormflow (surface and quick subsurface runoff), and

(ii) baseflows, which are releases from groundwater into streams. Methods for flood estimation do not generally consider groundwater flows.

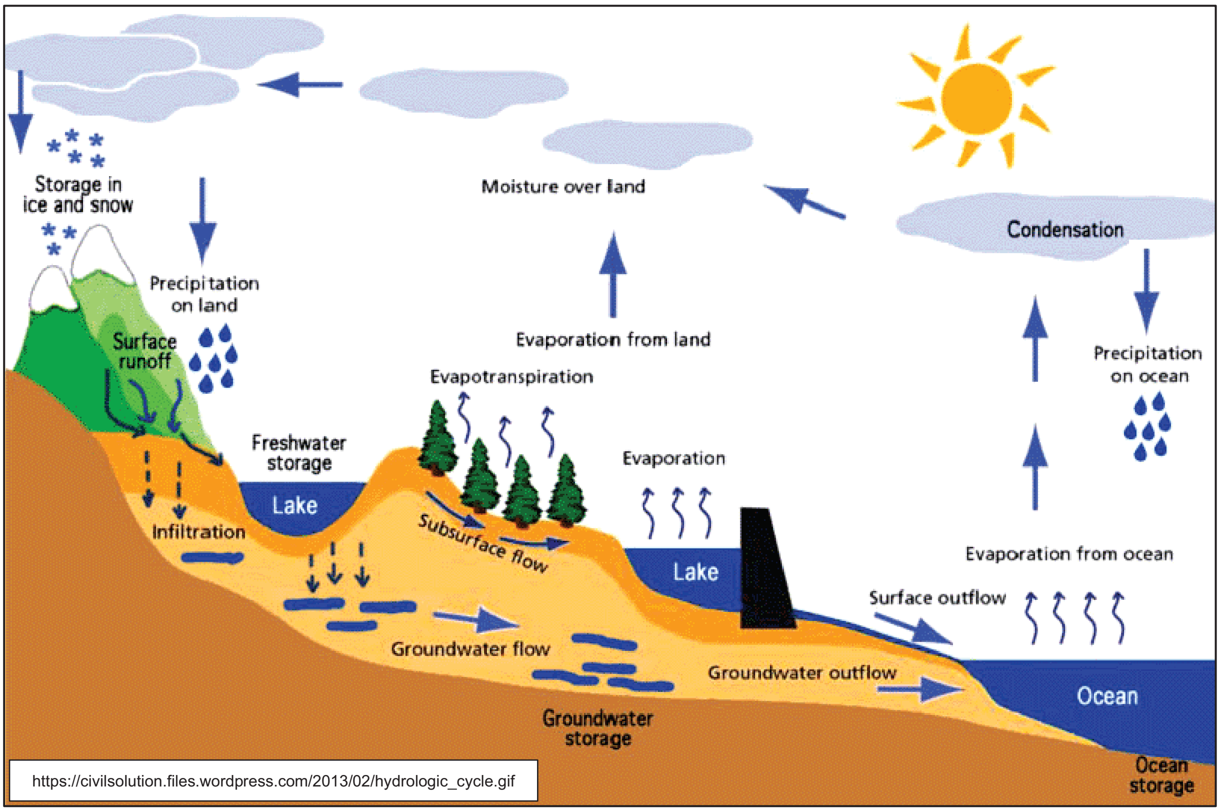

Figure 4-1: Hydrological cycle

Stormflow generation in catchments is highly variable both in time and space, depending not only on the input (amount and intensity of the rainfall), but also affected by catchment characteristics and conditions such as catchment wetness and response time. Consequently, catchment characteristics, antecedent moisture conditions, catchment response time and rainfall are regarded as fundamental input to all DFE methods in ungauged catchments. Errors in the estimates of these characteristics and attributes will directly impact on the accuracy of the estimated runoff volumes and peak discharges.

4.3 Road Map for Design Flood Estimation

The hydrological cycle as introduced above and shown in Figure 4-1, can be simplified for DFE into a conceptual framework consisting of three parts: (i) input, (ii) transfer function, and (iii) output. The latter "conceptual framework" is also used in this best practice guideline as a "DFE Road Map" to provide an overview of this chapter and the subsequent chapters focussing on DFE, CSM and river hydraulics applicable to the design of hydraulic infrastructure in an urban environment. Links to topics within the conceptual framework are summarised in Table 4-1.

Table 4-1: Conceptual Framework and Links for DFE

Conceptual Framework		Topic Description	Cross Reference
INPUT	Design Rainfall Estimation	Choice of Data Sources	Section 5.2
		Sources of Rainfall Data and Information	Section 5.3
		Rainfall time series data	5.3.1
		Infilling of missing data	5.3.2
		Design rainfall information	5.3.3
		Areal reduction factors	5.3.4
		PROBABILISTIC RAINFALL AND FLOOD FREQUENCY ANALYSIS	Chapter 6
	Observed Flow Data (Probabilistic methods)	Choice of Data Sources	Section 5.2
		Sources of Flow Data and Information	Section 5.4
		Primary flow data	Section 5.4.1
		Limitations of DWS flow data and stage-discharge rating tables	Section 5.4.2
		Flow data for flood frequency analysis	Section 5.4.3
		Anecdotal records	Section 5.4.4
TRANSFER FUNCTION	Catchment Characteristics	Catchment Management and Planning	Section 1.4
		Sustainable Drainage Systems and Water Sensitive Urban Design	Section 3.6
		Catchment Characteristics	Section 3.7
		Summary of Data Required for Methods	Section 4.7
		Climate Data	Section 5.5
		Catchment Geomorphology and Topography	Section 5.6
		Land Cover	Section 5.7
		Soils	Section 5.8
		Quality Control and Consistency Checks	Section 5.9
		Catchment Response Time	Section 5.10
OUTPUT	Design Flood Estimation	Approaches to Design Flood Estimation	Section 4.4
		Selection of Design Flood Estimation Methods	Section 4.5
		Selection of Design Flood Estimation Methods for Urban Applications	Section 4.6
		Summary of Data Required for Methods	Section 4.7
		PROBABILISTIC RAINFALL AND FLOOD FREQUENCY ANALYSIS	Chapter 6
		EVENT-BASED DESIGN FLOOD ESTIMATION	Chapter 7
		Rational Method	Section 7.2
		SCS-SA Method	Section 7.3
		PC-SWMM	Section 7.4
		RMF Methods	Section 7.5
		Event-based DFE Software	Section 7.6
		Performance of Methods	Section 7.7
		Emerging New Developments	Section 7.8
		COMPUTER MODELS AND CONTINUOUS SIMULATION MODELLING	Chapter 8
		Continuous Simulation Modelling (CSM)	Section 8.3
		Emerging New Developments	Section 8.6

4.4 Approaches to Design Flood Estimation

Standard approaches for DFE have been developed in many countries. These generally include the probabilistic (statistical) analyses of observed events where observed data are available, empirical methods, and deterministic rainfall-runoff modelling. Rainfall-runoff modelling can be broadly classified as either event-based DFE methods including discretised computer models using event design rainfall, or Continuous Simulation Modelling (CSM).

Event-based DFE methods are broadly categorised as probabilistic methods (which can be applied at-site on a regional scale), deterministic methods, or empirical methods (Cordery and Pilgrim, 2000; Rahman *et al.*, 2002; Van der Spuy and Rademeyer, 2010; Smithers, 2012).

(a) **Probabilistic methods** entail the frequency analysis of observed flood peak data that are adequate both in terms of record length and data quality.

(b) **Deterministic event-based methods** generally lump all heterogeneous catchment processes into a single event rainfall-runoff process to enable the estimation of the 1:*T*-year flood event from the 1:*T*-year rainfall event, with the catchment assumed to be at an "average condition."

(c) **Deterministic Continuous Simulation Models (CSM)** use a time series of rainfall as input to a continuous simulation of the rainfall-runoff process for user discretised catchment or hydrological response units, each with their own model parameters, Frequency analyses are performed on the simulated runoff.to extract the T year design events from the long duration hydrograph.

(d) **Empirical methods** are algorithms which generally relate peak discharge to catchment size and other physiographical and climatological indices.

CSM approaches use rainfall-runoff modelling to simulate the water balance on a continuous basis; thereby, eliminating the need for initial conditions and loss assumptions (Boughton and Droop, 2003; Smithers *et al.*, 2013).

Methods currently used for DFE in South Africa are summarised in the schematic overview shown in Figure 4-2.

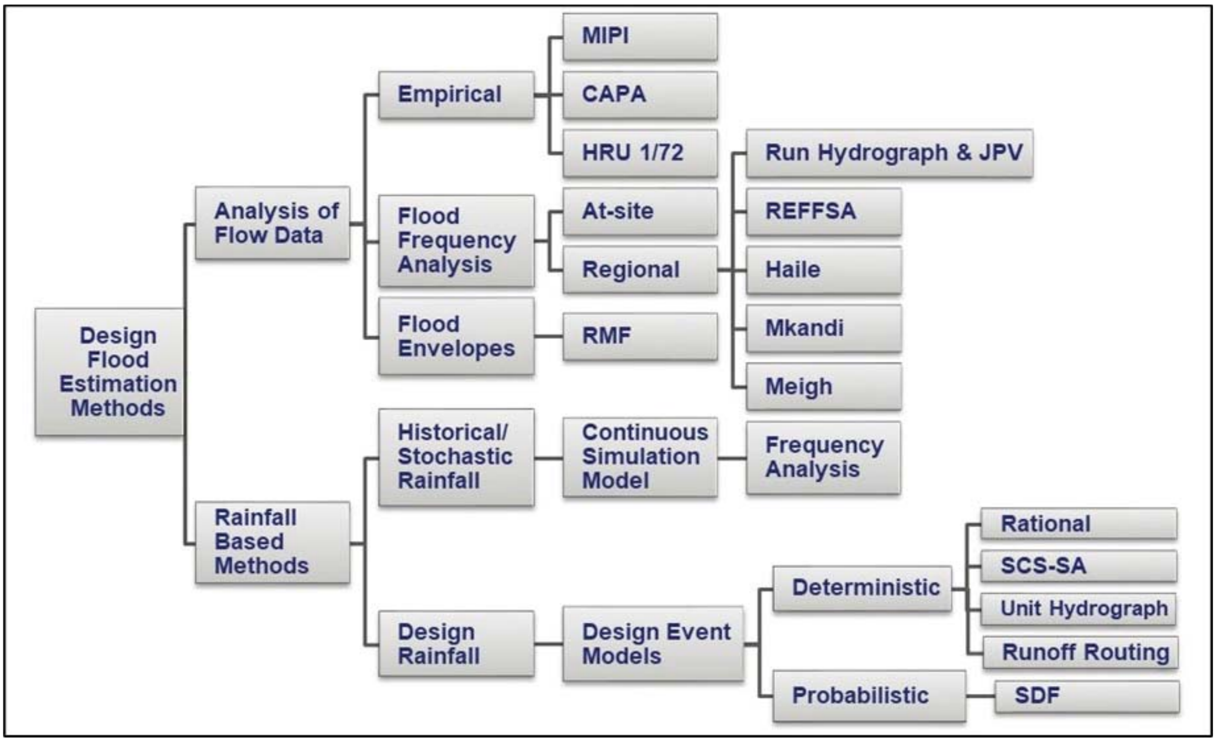

Figure 4-2: DFE methods used in South Africa (Smithers, 2012)

Many of the methods depicted in Figure 4-2 were developed in the late 1960s or early 1970s and are based on the approaches and data available at the time. As a result, there are still no universally applicable or legislated methods for DFE in South Africa and practitioners must use their experience and professional judgement in selecting appropriate methods for a specific design situation (Van der Spuy and Rademeyer, 2010). In addition, when selecting appropriate methods, practitioners need to understand the methods, their assumptions, the data used in their derivation and the advantages and limitations associated with each method.

The need for new approaches to DFE in South Africa has been highlighted (Alexander, 2002b; Smithers and Schulze, 2003; Görgens, 2007). This has resulted in the initiation of a National Flood Studies Programme (NFSP) by the South African Committee on Large Dams (SANCOLD) and the Water Research Commission (WRC). The NFSP is also supported in principal by the Department of Water and Sanitation (DWS) and the South African National Roads Agency Limited (SANRAL) (Smithers *et al.*, 2014). A summary of new research developments is contained in Section 7.8 for event-based DFE methods and in Section 8.6 for CSM. Practitioners should thus be aware of the need to use updated or new methods of DFE when these are available for application in South Africa. One source information on emerging methods is the National Flood Studies Program website and data portal www.waterresearchobervatory.org and https://data.waterresearchobservatory.org/.

4.5 Selection of Design Flood Estimation Methods

The South African DFE methods as shown in Figure 4-2, are extensively detailed in the literature, for example in SANRAL (2013) and Van der Spuy and Rademeyer (2010). However, there are currently no comprehensive guidelines on the selection of the best method(s) to use for particular situations in South Africa. In addition, there is no information available at a national scale on the accuracy or uncertainty associated with the application of these methods. For some methods, an assessment of performance has been done, as summarised in Chapter 7, Section 7.7 for event-based DFE methods and in Chapter 8, Section 8.5 for CSM.

In general, event-based DFE methods are suited for:

(a) Storm events where in-catchment storage is minimal.

(b) Flood peak estimation, where a steady state (constant flow) flood analysis is sufficient, e.g. flood lines in short reaches, and culvert sizing.

(c) Flood hydrograph estimation, where a dynamic state (variable flow) flood analysis is sufficient, e.g. the design of single detention facilities.

Any conditions outside the above, require increasing levels of assumptions with an associated increase of uncertainty.

In general, CSM is preferred and is better suited for:

(a) Planning and design of all flood conditions. The results become less reliable as the simulation period approaches the record length. There is a probability of about 64% that an event of recurrence interval equal to the record length is captured in the record, and a probability of about 90% that an event with a recurrence interval equal to half of the record length is captured in the record. It is therefore suggested that CSM is best suited for use up to a return period of approximately half of the length of the rainfall record (years).

(b) Design of multiple storage solutions.

(c) All catchment conditions, but particularly those with complex storage that will affect peak flow.

Both event-based methods and CSM are suited to distributed catchment analyses. This is important in heterogeneous catchments with varying topography, land cover and soils and/or where the drainage network results in significant attenuation of the flood peak.

In addition to the above, the following must be noted:

(a) Probabilistic methods provide the best estimate of a design flood provided there is an adequate record of sufficient length and accuracy. Streamflow records are available for many of the large rivers in the country and the data are available through the DWS (Chapter 5, Section 5.4). In municipalities where DWS flow-gauging stations are available, the use of probabilistic methods (Chapter 6) is recommended. Unfortunately, suitable streamflow records in municipal flow networks are rare in South Africa, but with the transition to WSUD programmes, more municipalities are likely to establish flow monitoring networks. Recently some of the metropolitan municipalities have established flow-gauging stations, for example, eThekwini has an extensive and well monitored network of stations, but the City of Johannesburg has only recently installed flow monitoring devices at a few locations.

(b) Deterministic methods offer moderate to reasonable levels of DFE certainty. The methods are based on physical systems and parameters are either directly measurable or derived from physical catchment attributes. This links flood estimations directly to site specific conditions and also allows for catchment sub-division that can improve confidence in heterogeneous catchments. The methods are also suitable for both event-based analyses and CSM. Deterministic methods typically have higher data input requirements and are subsequently generally less favoured by practitioners. However, generally most of the required data are linked to the physical attributes of the catchment or rainfall characteristics, requiring the practitioner to investigate all factors having an influence on the variability of runoff responses. This is a fundamental part of best practices as it requires a practitioner to obtain and review available data, make and justify selections and record judgement decisions which require understanding and is expected to result in better design flood estimates.

(c) Empirical methods are associated with much higher levels of uncertainty in the design flood estimates in urban areas, as few have been derived using data from urban catchments or calibrated for municipal catchments. Furthermore, empirical methods in South Africa are generally regarded as being more suitable to larger, rural catchments (Van der Spuy and Rademeyer, 2014). As a result, there is only one empirical method recommended in this guideline, i.e. the Regional Maximum Flood (RMF) method, with its application used as an indication of the maximum expected floods, and not necessarily for any design purposes. For further information, please refer to Chapter 7, Section 7.5.

Most of the DFE methods in South Africa (see Figure 4-2) were developed for rural catchments, however, some methods have been modified to be applied in an urban context (Van Vuuren *et al.*, 2013). In contrast, some of the CSM systems in use have been developed specifically for urban applications and provide stormwater design support. However, most CSM systems run at sub-hourly time steps, which is best suited to urban flood estimation, but there are relatively few sub-hourly, or even hourly, rainfall stations in South Africa. Without sub-daily rainfall data it is not possible to run a continuous simulation so event-based DFE methods will need to be applied. In summary, the selection of the preferred DFE methods for municipal applications is based upon the following:

(a) Probabilistic methods are the first-choice methods where suitable streamflow records are available. As highlighted above, streamflow data for municipal watercourses are rare, but records may be available for major rivers that pass through municipal areas.

(b) Deterministic methods are well suited to provide the output needed for municipal DFE (peak flow, volume, and duration) and can be applied in an event-based and/or CSM mode. Furthermore, input data requirements encourage the interrogation of factors affecting runoff variability, leading to better judgement decisions by practitioners.

(c) Only one empirical method (RMF) is considered to be useful for DFE in municipal catchments.

4.6 Selection of Design Flood Estimation Methods for Urban Applications

Typically, the choice or selection of any method is driven by suitability of method, input data requirements, data availability and the required output, i.e. peak discharge, runoff volume or both. Model structures generally vary from the simplest empirical methods to more complex, physically-based deterministic methods with an associated increase in data, catchment attribute and calibration requirements (Singh, 1995; Johnson, 2003; Devi, 2015). In addition to the above, Table 4-2 contains a summary of the DFE/CSM selection criteria applicable to urban catchments.

It is recommended that the municipality takes the lead and identifies methods best suited to their local conditions, development control requirements and wider catchment management objectives. However, the choice/selection of method(s) used in a specific application is the decision of the practitioner in consultation with the Municipal Engineer. The methods chosen are not restricted to those recommended in this best practice guideline, but the rationale and justification for the selection of method(s) should always be presented in the DFE report.

Table 4-2: Selection criteria applicable to DFE and CSM in urban catchments

Municipal function & catchment size	Primary objective	Requirement and Suggested Method
Development control Small (< 5 ha)	• Conveyance (channel/culvert design).	Estimation of peak discharges: • Event-based, e.g. RM and SCS-SA.
Development control Small (< 5 ha)	• Stormwater drainage design. • SuDS. • Detention and retention storage.	• Estimation of peak discharge and runoff volume. • CSM.
Municipal flood lines Small (< 5 ha) to medium (up to 5 km^2)	• Development control.	Estimation of peak discharges: • Event-based approach using discretised computer models which are incrementally adjusted at main tributaries (channel attenuation deemed to be minimal).
Municipal flood lines Large (> 5 km^2)	• Development control.	Estimation of peak discharge and runoff volume: • Event-based approach using discretised computer models which are incrementally adjusted at main tributaries and routed along the main channel (channel attenuation deemed to be significant).
Disaster management	• Hazard management. • Disaster anticipation. • Response and recovery programmes.	Extreme event analyses (e.g. T=1 000 yrs.): • Peak discharge and runoff volume. • Event-based approach using discretised computer models which are incrementally adjusted at main tributaries and routed along the main channel.
Spatial Development Plans (SDP) & Integrated Development Plans (IDP) Large (> 5 km^2)	• Catchment management and planning.	• Peak discharges, volumes, and frequency (especially smaller return periods) for assessment of flood scenarios. • CSM (catchment and channel storage deemed to be significant).

The DFE methods recommended for consideration in urban areas are the following:

(a) Probabilistic methods (see Chapter 6).

(b) Event-based deterministic rainfall-runoff methods:

- Rational Method (RM; see Section 7.2).
- SCS-SA Method (SCS-SA; (see Section 7.3).
- Discretised computer models, e.g. SWMM (see 7.4)

(c) Empirical methods (RMF; see 7.5).

(d) CSM, e.g. SWMM (see Chapter 8).

Further details on other methods not included in this best practice guideline can be obtained from SANRAL (2013) and Van der Spuy and Rademeyer (2010). In order to assess the uncertainty in using event-based DFE and CSM methods, all appropriate approaches should be included in any specific design situation and not be limited to using only the more simplified methods, e.g. the +160-year-old Rational Method (Alexander, 2002b). Given the lack of guidelines, the application of more than one appropriate method is recommended as best practice. The selection of the best flood estimate from the range of values obtained requires professional judgement and experience and provides an indication of the uncertainty associated with the selected value.

The following should be noted:

(a) Averaging, or taking a weighted average of the output of different DFE methods **is NOT good practice**, unless an indicative design flood estimate is required for planning purposes. Averaging, or taking a weighted average from different methods emphasises the uncertainty of the DFE practitioner and indicates that the practitioner believes methods applied and/or the catchment parameters selected for the method(s) are incorrect.

(b) Secondary methods, i.e. methods which provide similar/comparable estimates as the primary/preferred method, may be used to review the parameters used in the selected primary/preferred deterministic method.

(c) If a probabilistic method is the preferred method, secondary methods **should** be used to review the design peak flows where the return period is more than **1.5 times** the length of the period of record used in the analysis.

(d) Generally, probabilistic methods should not be used to estimate return periods longer than double the record length of the available reliable data, since there is increased uncertainty of estimates for longer return periods.

4.7 Summary of Data Required for Methods

A summary of the input data required for DFE methods recommended for use in this guideline, is contained in Table 4-3.

Table 4-3: Summary of data requirements for event-based DFE methods

Method	Input data/parameters*	Cross Reference	Source**
Probabilistic	Daily rainfall AMS *n*-Duration rainfall AMS	Section 5.3 and Chapter 6	• Local data • SAWS: https://www.weathersa.co.za/home/equiries_climatedata • ARC:https://www.arc.agric.za/arc-iscw/Pages/Climate-Monitoring-Services.aspx • SASRI: https://sasri.org.za/weather-services/ • Lynch database: https://cwrr.ukzn.ac.za/resources/acru/
	Peak flow AMS	Section **5.4** and Chapter 6	• Local data • DWS: https://www.dws.gov.za/Hydrology/Default.aspx
Rational Method	A, α, β, ARF, $D_{\%}$, H, L_{CH}, L_O, n, P_T, S, S_{CH}, T_C and γ.	Sections 5.3,, 5.6, 5.7, 5.8, 5.10 and 7.2	• Local Digital Terrain Model (DTM) data • Geographical Information System (GIS) data. • Digital Elevation Model (DEM) data.
SCS-SA	A, H, LAT, LONG, L_{CH}, L_O, MAP, n, P_T, S, S_{CH}, T_C and T_L. Number of sub-catchments and/or Hydrological Response Units (HRUs). SCS hydrological soils groups: • Infiltration/drainage rates (mm/h). • Binomial or taxonomic soil classification. Land cover, treatment, and stormflow potential. Curve Numbers (CN). Soil depth category. Rainfall distribution Types (1-4).	Sections 5.3, 5.6, 5.7, 5.8, 5.10 and 7.3	• Local Digital Terrain Model (DTM) data • Geographical Information System (GIS) data. • Digital Elevation Model (DEM) data. • Quinary catchments: https://www.waterresearchobservatory.org • Hydrological soils: https://www.waterresearchobservatory.org.
SMMM	• *Discretise subcatchments and drainage system* • *Soil infiltration characteristics (Horton or Green and Ampt)* • *Imperviousness ratios derived from land cover information* • *Subcatchment slope roughness, and width or overland flow length* • *Conduit characteristics (geometry and roughness)* • *Drainage system slopes based on junction invert elevations* • *Stage / surface area relationship for storage objects* • *Rainfall, either continuous record or event hyetographs*	Sections	• Practitioner from terrain models or mapping • SWMM Reference Manuals: Three Volumes: • https://cfpub.epa.gov/si/si_public_record_report.cfm?Lab=NRMRL&dirEntryId=327450 • https://cfpub.epa.gov/si/si_public_record_report.cfm?Lab=NRMRL&dirEntryId=309346, • https://cfpub.epa.gov/si/si_public_record_report.cfm?Lab=NRMRL&dirEntryId=337162 • Open SWMM: https://www.openswmm.org/ReferenceDocumentation • Aerial photography or site inspection • Local authority GIS, practitioner site inspection or engineering judgement • Practitioner from GIS catchment and drainage system delineation • Soils mapping or derived from underlying geology
RMF	A and K.	Sections 5.6 and 7.5	• Local Digital Terrain Model (DTM) data • Geographical Information System (GIS) data. • Digital Elevation Model (DEM) data. • SANRAL (2013).

* Parameter	Description (units)
AMS	Annual maximum series (mm) or (m^3/s).
A	Catchment area (km^2).
α	Rural area distribution factor (%).

ARF	Areal Reduction Factor (%).
β	Urban area distribution factor (%).
$D_{\%}$	Dolomitic area (%)
H	Height difference along overland flow path (m).
K	Kovács regional constant
LAT	Latitude (decimal degrees)
LONG	Longitude (decimal degrees)
L_C	Distance to catchment s centroid distance (km).
L_{CH}	Length of longest watercourse (km).
L_O,	Hydraulic length of overland flow path (km).
MAP	Mean annual precipitation (mm).
n	Roughness coefficient for overland flow.
P_T	Design rainfall depth (mm).
S	Average catchment slope (%).
S_{CH}	Average main watercourse slope (m/m).
T_C	Time of concentration (minutes/hours).
T_L	Lag time (hours).
γ	Lake area distribution factor (%).

** See Chapter 5 for more details.

4.8 Chapter References

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5. DATA SOURCES AND DERIVATION OF CATCHMENT CHARACTERISTICS

5.1 Chapter Overview

This chapter contains a summary of currently available data sources for rainfall, flow, climate, catchment geomorphology, land cover, and soils, which can be used in Flood Frequency Analyses (FFA) and DFE. Typical quality control and consistency checks which should be performed when using observed data, are outlined in Section 5.9.

It is recommended that Regional Linear Moment Algorithm and Scale Invariance (RLMA&SI) method should be used to estimate design rainfall (see Section 5.3.3). This estimates design rainfall at each 1’ x 1’ gridded point within the catchment and these values should be area weighted to estimate the point design rainfall for the catchment (see Section 5.3.3). The conversion of point design rainfall to catchment design rainfall using Areal Reduction Factors (ARFs) is necessary to account for the spatial variability of rainfall and the recommended approach is detailed in Section 5.3.4. Despite shortcomings, Alexander’s geographically-centred method (2001) listed as Eq. 5-1 and shown in Figure 5-6 is recommended for general use until new ARF methods are available to be deployed at a national scale. In addition, it is recommended that the Upper 90% design rainfall value estimated by the RLMA&SI method be considered as an interim design value to account for climate change (see Section 5.3.3).

Catchment response time parameters, e.g. time of concentration (T_C), lag time (T_L) and/or time to peak (T_P), which are fundamental input to all event-based DFE methods in ungauged catchments, are described in Section 5.10. The use of both Eq. 5-8 and Eq. 5-13 is recommended, while the NRCS velocity and/or segmental methods (Eqs. 5-10 to 5-12) should be used in the case of man-made (constructed) flow paths where the surface roughness is easier to define.

5.2 Choice of Data Sources

5.2.1. Benefits of using local data

When performing FFA, the best choice is to use observed flow data from at or near the site where the design needs to be performed, if this is available. For FFA and hydrological modelling, long records of consistent and good quality data are required. Generally, the length of record should not be less than 20 years for FFA, and a rule of thumb is that the return period estimated should not exceed double the record length, i.e. a record with 20 years of record, should not be used to estimate design values for return periods exceeding 40 years. The relevant FFA data sources are summarised in Sections 5.3. to 5.8. The basic consistency and quality control checks recommended are provided in Section 5.9.

5.2.2. Benefits of using regional data

Generally, if long records of good quality observed flow data are not available at the site of interest, then one or more of the appropriate methods detailed in Chapter 6 should be applied. Relatively few long (e.g. 50 to 100 years or more) observed rainfall and flow records are available in South Africa. Hence, the use of regional approaches, which supplement the time limited observed records with information from surrounding stations/sites, are widely recommended in the literature and generally result in more reliable and consistent design estimates. An example of a regional approach is the estimation of design rainfall, as detailed in Section 5.3.3.

5.3 Sources of Rainfall Data and Information

Rainfall data are required as input for the event-based DFE methods and CSM models, detailed in Chapters 7 and 8, respectively. This includes both time series of observed data and design rainfall values for different return periods and durations.

In South Africa, daily rainfall data are recorded manually by an observer at 08:00 every day and represent the rainfall for the previous 24-hour period. For smaller catchments, where the catchment response time may be less than 1-day, data from continuously recorded rainfall are required. Historically, continuously recorded rainfall measured by siphon-type rain gauges was recorded autographically and digitised into an electronic form. Details on the identification and quantification of errors and inconsistencies in the digitised rainfall data can be obtained from Smithers and Schulze (2000a). The autographic recording mechanism has been replaced by the use of tipping bucket rain gauges with data recorded by data loggers.

5.3.1. Rainfall time series data

Both daily and sub-daily rainfall data can be obtained from a number of different organisations. The sources of observed rainfall data include the following:

(a) **South African Weather Services (SAWS)**: SAWS is the primary custodian of climate data in South Africa and raw observed rainfall data can be purchase from SAWS. These data will require error checking and infilling of missing data where necessary. The data can be requested from the following link: https://www.weathersa.co.za/home/equiries_climatedata.

(b) **Agricultural Research Council (ARC):** The ARC has a network of climate stations in South Africa, and they can be contacted using the following link: https://www.arc.agric.za/arc-iscw/Pages/Climate-Monitoring-Services.aspx.

(c) **South African Sugar Research Institute (SASRI):** SASRI operates a network of climate stations in the sugarcane growing regions of South Africa and data from these stations can be obtained after registering using the following link: https://sasri.org.za/weather-services/.

(d) **Municipalities**: Many metropolitan councils and municipalities in South Africa have their own climate or rain gauge monitoring networks and can be contacted for further information. If these organisations do not currently monitor and record rainfall data in their jurisdiction, then they should be encouraged to do so.

(e) **Private observers**: Much of the daily rainfall data in the SAWS database has been supplied by private individuals. Hence, individuals who have private records of rainfall should be identified using local knowledge and records from these people should be solicited.

All the data obtained from the above sources require extensive checking and the identification, flagging and infilling of missing and suspect data values should be performed (see Section 5.9).

In a study commissioned by the WRC and undertaken by Lynch (2004), daily rainfall data from 12 153 stations in South Africa were collated from a number of sources. These data sets were extensively error-checked and suspect or missing data were flagged and infilled where possible by Smithers and Schulze (2000b). A Graphical User Input (GUI), i.e. Daily Rainfall Extraction Utility (DREU), was developed by Kunz (2004) to facilitate the identification and extraction of the daily rainfall data and calculation of statistics from the database, which contains daily rainfall data up to the year 2000. The database and extraction utility can be downloaded from the Centre for Water Resources Research (CWRR) software download section using the following link: https://cwrr.ukzn.ac.za/resources/acru/, which facilitates access to the following site from which the files can be downloaded at: https://www.dropbox.com/s/6n935g34kwgbrki/ daily_rainfall_utility.rar?dl=0.

The extraction utility and GUI enables the following to be performed:

(a) Select or identify rainfall station(s) by descriptive name or SAWS identification number.

(b) Identify a user selectable number of rainfall stations closest to a user input geographical coordinate.

(c) Identify stations located within an area or region defined by geographical coordinates for the NW and SE corners.

(d) Select the "best" or most representative rainfall station for a particular point of interest (i.e. the "driver" station concept).

(e) Extract observed daily rainfall values from the comprehensive database for one or more selected stations.

(f) Calculate monthly and annual rainfall totals from the daily values.

(g) Calculate an accumulative rainfall total for a particular period (i.e. growing season).

(h) Adjust daily rainfall values using a monthly factor in order to correct for some systematic errors.

(i) Output the extracted rainfall data to various formats (e.g. comma separated values).

The input options for the GUI are shown in Figure 5-1.

5.3.2. Infilling of missing data

Rainfall records characterised by missing data need to be carefully interrogated when estimating design values and they limit the use of daily CSMs, since they are reliant on a continuous rainfall data input series (Pitman, 2011).

Lynch (2004) highlighted the importance of rainfall data infilling and emphasised that a missing day implies an incomplete month and consequently an incomplete year. Hence, the DREU as introduced above, can be used to extract infilled/patched quality-controlled rainfall data. A summary of the infilling techniques developed by Smithers and Schulze (2000b) and results used by Lynch (2004), is contained in Appendix 4 (Chapter 14).

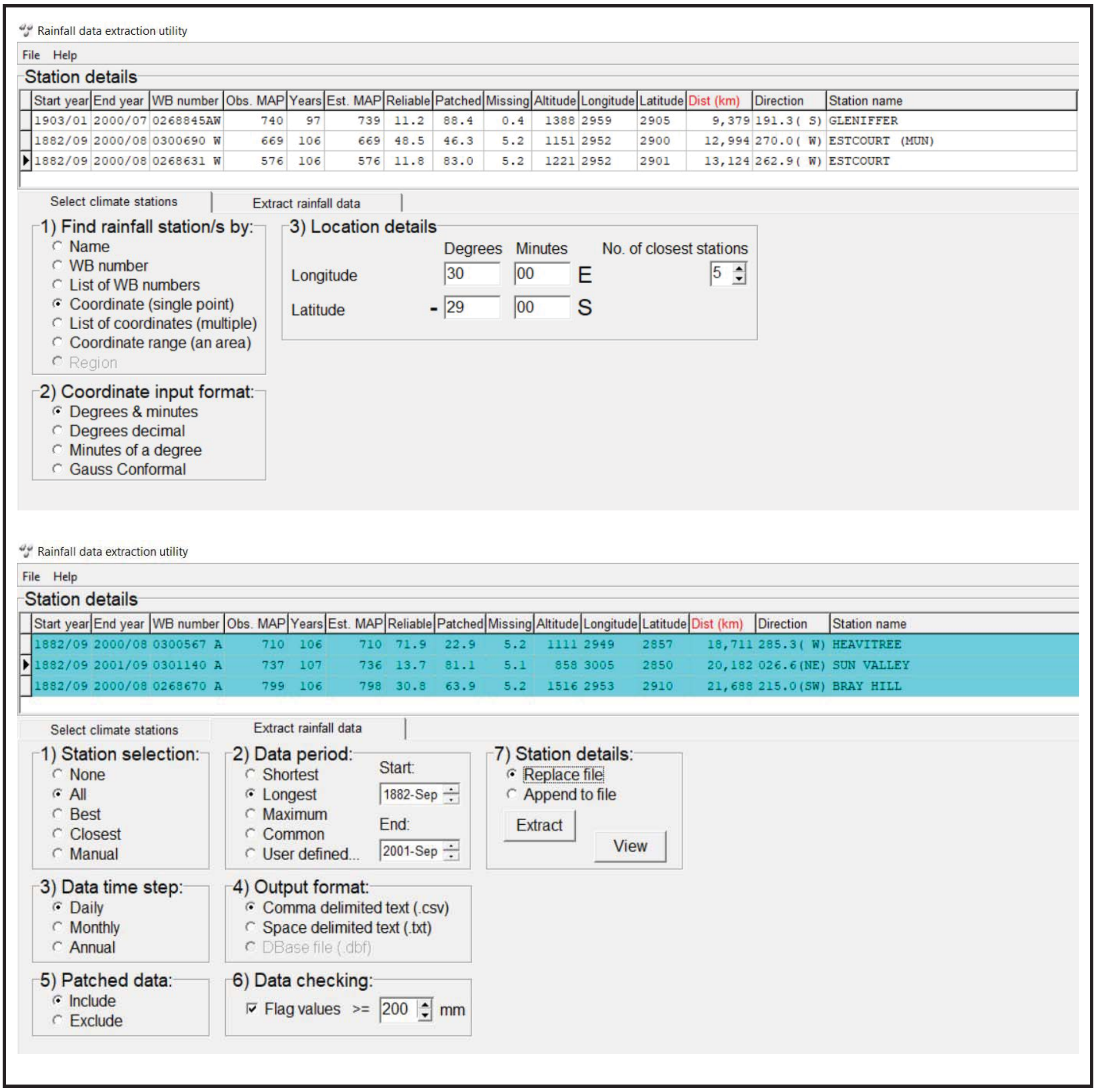


Figure 5-1: Daily Rainfall Extraction Utility (DREU)

5.3.3. Design rainfall information

Event-based DFE methods (e.g. RM and SCS-SA methods) detailed in Chapter 7 (Sections 7.2 and 7.3), respectively, require design rainfall for a selected duration and return period as input or for disaggregation into a design hyetograph. A number of approaches have been developed over the years to estimate short and long duration design rainfalls for South Africa as summarised by Smithers and Schulze (2000a) and Smithers and Schulze (2000b), respectively. The RLMA&SI approach developed by Smithers and Schulze (2003) is currently the recommended approach for Design Rainfall Estimation (DRE) in South Africa (Gericke and du Plessis, 2011; SANRAL, 2013) and enables DREs to be estimated at any 1' x 1' grid point in South Africa and for durations ranging from 5-minutes to 7-days and for 2 to 200 year return periods. The GUI for the software is shown in Figure 5-2. The software for the implementation of the RLMA&SI approach can be downloaded from the following link:

https://ukzn-iis-02.ukzn.ac.za/unp/beeh/hydrorisk/background%20rainfall.htm.

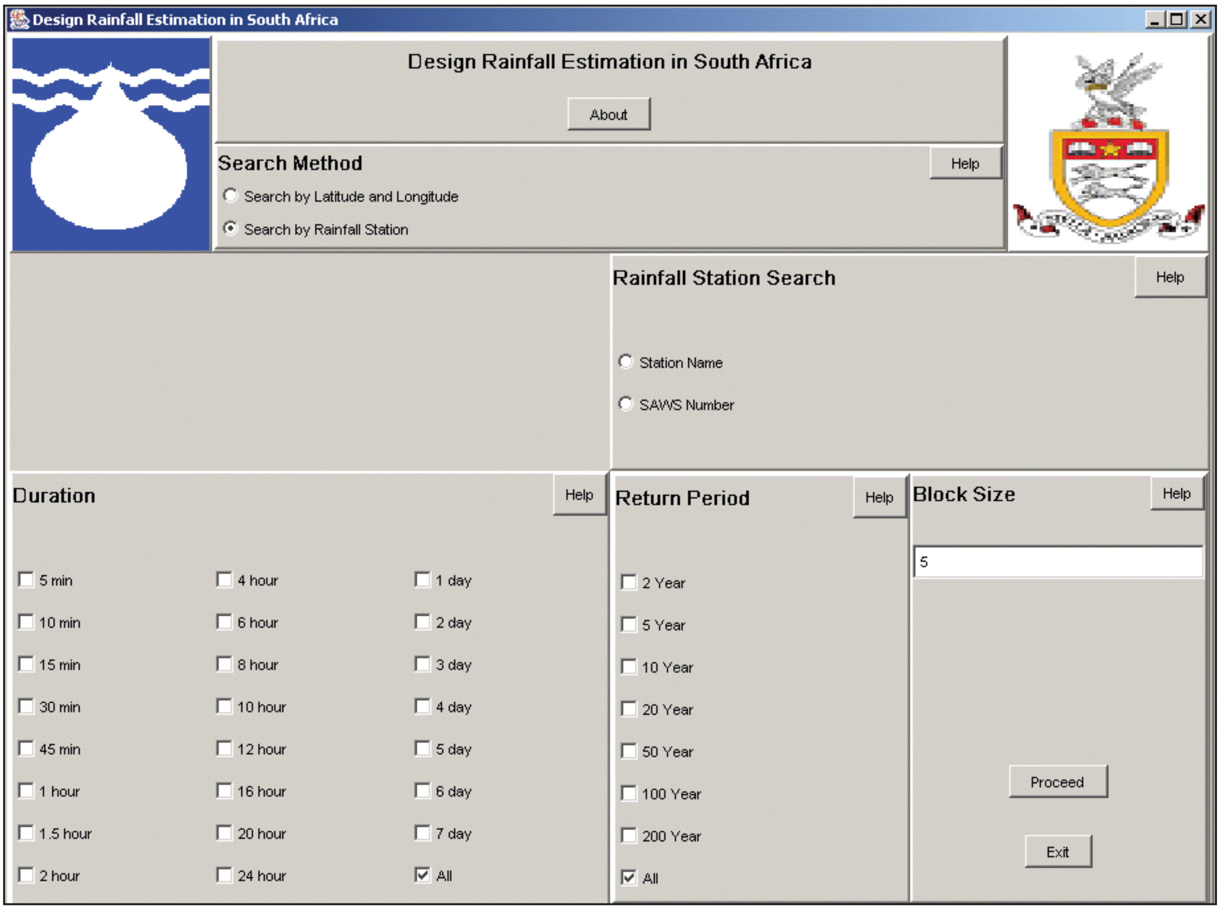


Figure 5-2: RLMA&SI GUI for design rainfall estimation in South Africa

Typical output from the RLMA&SI software is shown in Figure 5-3 and Figure 5-4, respectively. The 1 to 7-day design rainfalls for the 5 closest rainfall stations to the selected site are shown in Figure 5-3. Figure 5-4 shows design rainfall for the selected durations and return periods computed using the RLMA&SI approach at the selected site and at all grid points falling within the user selected 1' block size. In Figure 5-3 and Figure 5-4, three design values are presented for each return period, namely:

(a) 2 – Median 2-year return period design value for the location. This is the value that is recommended for the design.

(b) 2L – Lower 90% confidence interval for 2-year return period design value.

(c) 2U – Upper 90% confidence interval for 2-year return period design value.

The median, lower and upper 90% confidence levels are computed for all the selected return periods. It is recommended that the median design values be used for design in practice. The upper and lower confidence intervals values can be used to assess the confidence in the median design rainfall. If the range of the confidence level is large, then the sensitivity on the design should be assessed using the upper 90% confidence value.

The RLMA&SI method assumes that the rainfall data are stationary, i.e. the statistics of the rainfall data do not change with time. Given the changes in climate as a consequence of global increases in temperature with consequent increases in the water holding capacity of the atmosphere, changes in both the magnitude and frequency of extreme rainfall events are expected. The projected impacts of climate change on design rainfall have been reported in some studies undertaken in South Africa with recommended preliminary adjustment factors. For example, Schulze *et al.* (2010b) recommend that current design rainfalls in the Cape Town Metro area should be increased by 15% to account for the future impacts climate change.

Similarly, Schulze *et al.* (2010a) also recommended a 15% increase for the eThekwini Metro area. In a more recent study for the City of Tshwane, Davis and Schulze (2021) report that design rainfall values are expected to increase slightly in some areas while in other areas future design rainfalls are expected to decrease for return periods greater than 50 years. On a national scale, Schütte *et al.* (2022) report 1-day design rainfall events are expected to increase over most of southern Africa from the present (1961-1990) to the near future (2015-2044), with larger increases for longer return periods, as shown in Figure 5-5.

Given the above brief review of the potential impacts of climate change on design rainfalls, and with suggested preliminary increases ranging from 15% to 60%, it is recommended that the Upper 90% design rainfall value estimated by the RLMA&SI method be considered as an interim design value to account for climate change. However, practitioners need to be aware of current ongoing research into how to accommodate the impacts of climate change on design rainfalls (e.g. by Johnson *et al.*, 2021a), and it is recommended that updated and more definitive results should be used once these studies are completed and published.

The station selected and the five closest stations are listed

Station Name	SAWS Number	Distance (km)	Record (Years)	Latitude (°)	(')	Longitude (°)	(')	MAP (mm)	Altitude (m)	Duration (m/h/d)	Return Period (years) 2	2L	2U	5	5L	5U	10	10L	10U
CEDARA COLLEGE	0239482_W	0.0	40	29	32	30	17	876	1134	1 d	55.6	55.2	55.8	77.8	77.3	78.2	95.2	94.0	96.1
										2 d	70.0	69.4	70.6	98.5	97.6	98.9	122.1	120.5	123.7
										3 d	79.8	79.1	80.6	112.7	111.7	113.1	140.2	138.2	142.2
										4 d	87.4	86.5	88.2	123.1	122.0	123.7	152.6	150.0	154.9
										5 d	92.0	91.1	92.8	128.8	127.6	129.4	158.6	155.7	160.9
										6 d	99.0	98.1	100.0	138.1	136.7	139.0	168.9	165.8	171.3
										7 d	102.5	101.6	103.5	142.0	140.7	143.0	172.8	169.7	175.3
CEDARA AGR RES STN,	0239482_A	0.0	78	29	32	30	17	876	1134	1 d	54.6	54.3	54.9	76.5	76.0	76.9	93.6	92.4	94.5
										2 d	69.4	68.8	70.0	97.7	96.8	98.1	121.1	119.5	122.6
										3 d	77.8	77.0	78.5	109.7	108.8	110.2	136.6	134.6	138.5
										4 d	84.6	83.8	85.4	119.2	118.2	119.8	147.8	145.3	150.0
										5 d	90.5	89.6	91.3	126.6	125.5	127.3	155.9	153.2	158.2
										6 d	96.8	95.9	97.7	134.9	133.6	135.8	165.0	162.0	167.4
										7 d	101.2	100.3	102.2	140.2	138.9	141.2	170.6	167.5	173.1
BENEAN	0239514_W	4.0	37	29	34	30	18	1445	1181	1 d	66.1	65.7	66.4	92.6	91.9	93.1	113.3	111.9	114.4
										2 d	85.9	85.1	86.5	120.8	119.7	121.3	149.8	147.8	151.7
										3 d	96.9	96.0	97.8	136.8	135.5	137.3	170.2	167.7	172.6
										4 d	103.9	102.9	104.9	146.5	145.2	147.2	181.5	178.4	184.3
										5 d	113.7	112.5	114.7	159.1	157.7	159.9	195.9	192.5	198.8
										6 d	121.1	120.0	122.3	168.8	167.2	169.9	206.5	202.7	209.5
										7 d	130.7	129.5	132.0	181.1	179.5	182.4	220.4	216.4	223.6
MERRIVALE	0239421_W	6.5	65	29	30	30	14	849	1060	1 d	51.1	50.8	51.3	71.6	71.1	71.9	87.6	86.5	88.4
										2 d	62.6	62.0	63.1	88.1	87.3	88.4	109.2	107.7	110.5
										3 d	69.6	69.0	70.3	98.3	97.4	98.7	122.3	120.5	124.1
										4 d	76.1	75.3	76.7	107.2	106.2	107.7	132.8	130.5	134.8
										5 d	82.0	81.1	82.7	114.7	113.7	115.3	141.3	138.8	143.3
										6 d	87.7	86.9	88.6	122.3	121.1	123.1	149.6	146.8	151.7
										7 d	92.3	91.5	93.2	127.9	126.8	128.8	155.6	152.9	157.9
ALLERTON (VET)	0239604_W	8.0	87	29	34	30	21	1072	882	1 d	58.6	58.2	58.9	82.1	81.5	82.5	100.4	99.2	101.4
										2 d	73.5	72.8	74.1	103.5	102.5	103.9	128.3	126.6	129.9
										3 d	83.5	82.8	84.3	117.9	116.9	118.4	146.7	144.6	148.8
										4 d	91.5	90.6	92.4	129.0	127.8	129.6	159.9	157.1	162.3
										5 d	98.3	97.3	99.2	137.7	136.4	138.3	169.5	166.5	172.0
										6 d	104.8	103.9	105.9	146.1	144.7	147.1	178.8	175.5	181.4
										7 d	112.2	111.2	113.3	155.4	154.0	156.5	189.1	185.7	191.9
BOTANIC GARDENS - PMB	0239605_P	9.0	83	29	35	30	21	1001	882	1 d	57.0	56.1	57.5	83.0	82.0	83.5	104.7	102.8	106.7
										2 d	73.0	71.7	74.3	106.0	104.6	106.8	133.7	130.3	136.8
										3 d	84.0	82.2	85.7	122.3	120.5	123.4	154.4	150.0	158.0
										4 d	91.5	89.7	93.4	132.0	130.1	133.2	165.5	160.9	170.1
										5 d	98.4	96.4	100.3	140.2	138.3	141.7	174.4	169.6	178.6
										6 d	104.8	103.0	106.6	148.1	146.0	149.5	182.8	178.1	187.1
										7 d	110.7	109.0	112.6	154.9	152.9	156.4	190.3	185.6	194.6

Figure 5-3: Two to 10 year return period design rainfall for 1-7-day durations for the 5 closest rainfall stations to selected site (Smithers and Schulze (2000b)

Gridded values of all points within the specified block

Latitude		Longitude		MAP	Altitude	Duration	Return Period (years)								
(°)	(')	(°)	(')	(mm)	(m)	(m/h/d)	2	2L	2U	5	5L	5U	10	10L	10U
29	31	30	16	849	1055	5 m	11.6	7.5	15.7	16.2	10.5	22.1	19.9	12.7	27.1
						10 m	15.6	10.9	20.3	21.9	15.2	28.5	26.7	18.5	35.0
						15 m	18.6	13.5	23.6	26.0	18.9	33.1	31.8	23.1	40.7
						30 m	23.4	17.5	29.3	32.8	24.5	41.1	40.1	29.8	50.5
						45 m	26.8	20.4	33.3	37.6	28.5	46.6	46.0	34.7	57.3
						1 h	29.6	22.7	36.4	41.4	31.7	51.0	50.6	38.6	62.8
						1.5 h	33.9	26.4	41.4	47.4	36.9	57.9	58.0	44.9	71.2
						2 h	37.3	29.3	45.3	52.2	41.1	63.4	63.9	50.0	77.9
						4 h	43.4	34.3	52.4	60.7	48.0	73.4	74.3	58.4	90.3
						6 h	47.3	37.6	57.1	66.3	52.6	80.0	81.1	64.0	98.4
						8 h	50.4	40.1	60.7	70.6	56.1	85.1	86.3	68.2	104.6
						10 h	52.9	42.1	63.7	74.1	59.0	89.2	90.6	71.7	109.6
						12 h	55.0	43.9	66.2	77.1	61.4	92.7	94.3	74.7	114.0
						16 h	58.6	46.8	70.3	82.0	65.5	98.5	100.3	79.7	121.1
						20 h	61.5	49.2	73.7	86.1	68.9	103.3	105.3	83.8	127.0
						24 h	64.0	51.3	76.7	89.6	71.8	107.4	109.6	87.3	132.0
						1 d	54.3	43.5	65.0	76.0	60.9	91.1	92.9	74.1	112.0
						2 d	67.0	58.0	75.8	93.8	81.2	106.1	114.8	98.8	130.5
						3 d	75.8	68.7	82.8	106.1	96.1	116.0	129.8	116.9	142.6
						4 d	82.4	73.4	91.4	115.4	102.7	128.0	141.2	125.0	157.3
						5 d	88.0	77.2	98.6	123.2	108.1	138.1	150.7	131.6	169.8
						6 d	92.8	80.6	104.9	129.9	112.7	146.9	158.9	137.2	180.6
						7 d	97.0	83.5	110.5	135.9	116.8	154.8	166.2	142.2	190.4
29	31	30	17	880	1118	5 m	11.7	7.5	15.8	16.3	10.6	22.1	20.0	12.9	27.2
						10 m	15.7	10.9	20.4	21.9	15.3	28.6	26.8	18.6	35.1
						15 m	18.6	13.6	23.7	26.1	19.0	33.1	31.9	23.1	40.7
						30 m	23.5	17.6	29.4	32.9	24.6	41.2	40.3	30.0	50.6
						45 m	26.9	20.5	33.4	37.7	28.7	46.8	46.2	34.9	57.5
						1 h	29.7	22.8	36.6	41.6	31.9	51.2	50.8	38.8	62.9
						1.5 h	34.0	26.5	41.5	47.6	37.1	58.1	58.3	45.2	71.5
						2 h	37.5	29.5	45.4	52.5	41.3	63.6	64.2	50.3	78.2
						4 h	43.6	34.5	52.6	61.0	48.3	73.7	74.7	58.8	90.6
						6 h	47.6	37.9	57.4	66.7	53.0	80.4	81.6	64.5	98.8
						8 h	50.7	40.4	61.0	71.0	56.6	85.4	86.8	68.8	105.0
						10 h	53.2	42.5	63.9	74.5	59.5	89.6	91.2	72.4	110.1
						12 h	55.4	44.3	66.5	77.5	62.0	93.1	94.9	75.4	114.5
						16 h	59.0	47.3	70.7	82.6	66.2	99.0	101.0	80.5	121.7
						20 h	61.9	49.7	74.1	86.7	69.6	103.8	106.0	84.7	127.6
						24 h	64.4	51.8	77.0	90.2	72.5	107.9	110.3	88.3	132.6
						1 d	54.6	44.0	65.3	76.5	61.5	91.5	93.6	74.9	112.5
						2 d	67.5	58.6	76.2	94.5	82.0	106.7	115.6	99.8	131.2
						3 d	76.4	69.3	83.4	107.0	97.0	116.8	130.9	118.0	143.6
						4 d	83.1	74.1	92.0	116.4	103.7	128.8	142.3	126.2	158.4
						5 d	88.7	78.0	99.2	124.2	109.2	139.0	151.9	132.9	170.9
						6 d	93.6	81.4	105.6	131.0	113.9	147.9	160.3	138.6	181.9
						7 d	97.9	84.4	111.3	137.0	118.1	155.9	167.7	143.7	191.6

Figure 5-4: Two to 10 year return period design rainfalls estimated using the RLMA&SI at the user selected site and at grid points falling within the selected block size

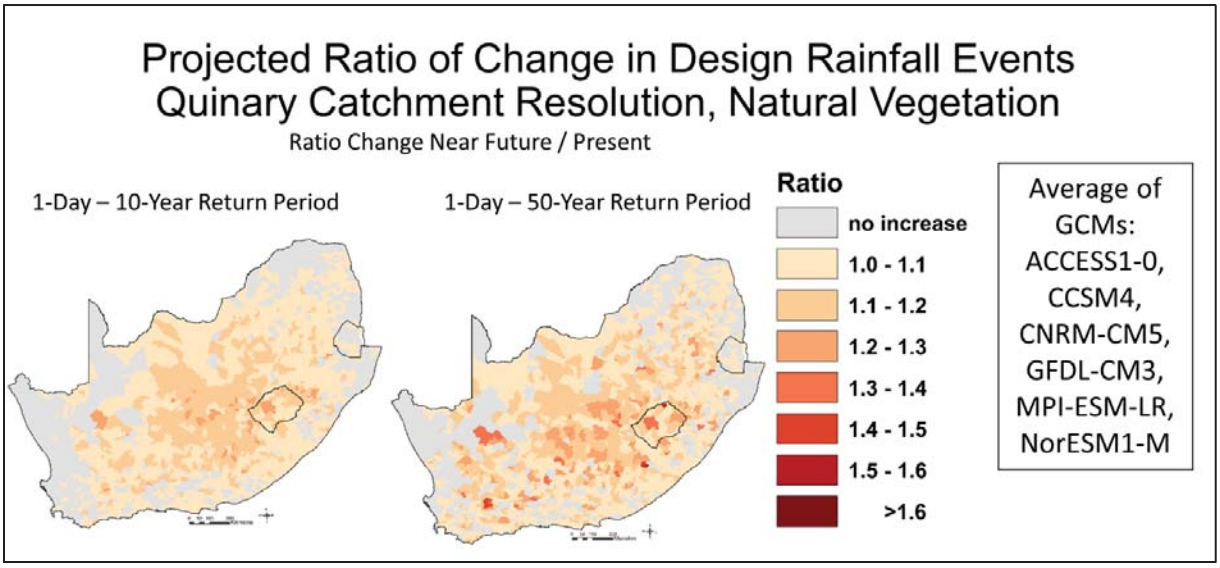


Figure 5-5: Projected changes from the present to the near future in design rainfalls

For: the 1:10-year return period 1-day rains (left) and the 1:50-year 1-day (right), derived from outputs from multiple Global Circulation Models (GCMs) (Schütte *et al.*, 2022)

It is recommended that design rainfalls be computed for all grid points which fall within the catchment. By overlaying these on the catchment, the variation of design rainfall in the catchment can be assessed. It is recommended that the area weighted (or average) value of the gridded values within the catchment be used to calculate the catchment design point rainfall. The averaged catchment design rainfall value is still a point estimate of design rainfall and ARFs should still be applied, where appropriate, to convert the design point rainfall into a catchment design rainfall value. As an alternative to the above, the design rainfall from a representative point in the catchment can be used to represent the catchment point design rainfall.

In summary, the gridded design point and average design rainfall values applicable to a user-specified block size in the RLMA&SI software, can be estimated using the following steps:

(a) Identify a single rainfall station located approximately at the geographical centre of the catchment as the base station to estimate the RLMA&SI gridded design point rainfall values.

(b) With the single rainfall station as selected in Step (a), specify the block size in such a way that the whole extent of the catchment under consideration is covered with grid points.

(c) After running the RLMA&SI software, the output (gridded design point rainfall values for all return periods) associated with the two standard durations respectively larger and smaller than the catchment-specific storm duration (T_C), needs to be averaged using the arithmetic mean. Thereafter, the average catchment design rainfall associated with T_C needs to be determined by means of linear interpolation.

5.3.4. Areal reduction factors

Design point rainfall estimates are only applicable to a limited spatial area, and for larger areas, the average areal design rainfall depth is likely to be less than the maximum design point rainfall depth. Areal Reduction Factors (ARFs) are used to describe this relationship between point and areal rainfall, *i.e.* design point rainfall depths are converted to an average areal design rainfall depth for a catchment-specific critical storm duration and catchment area.

In South Africa, the estimation of ARFs is limited to the storm-centred approaches of Van Wyk (1965) and Wiederhold (1969), and the geographically-centred approach of Alexander (2001). These methods are only applicable to specific temporal and spatial scales and do not account for any regional differences. Only the method proposed by Van Wyk (1965) is regarded as being probabilistically correct, *i.e.* ARFs vary with return period. However, both the methods of Van Wyk (1965) and Wiederhold (1969) are storm-centred approaches, which are currently incorrectly applied by practitioners in a geographically-centred manner. Alexander's geographically-centred method (2001) was transposed from methods developed in the United Kingdom (UK) with little local verification and it is also regarded as being probabilistically incorrect, *i.e.* ARFs remain constant irrespective of the return period under consideration.

Despite, the above shortcomings, Alexander's geographically-centred method (2001), listed as Eq. 5-1 and shown in Figure 5-6, is recommended for general use until new ARF methods are available to be deployed at a national scale.

$$ARF = [\mathbf{90000 - 12800}\boldsymbol{Ln}(\boldsymbol{A}) + \mathbf{9830}\boldsymbol{Ln}(\mathbf{60}\boldsymbol{T_C})]^{\mathbf{0.4}} \qquad \text{Eq. 5-1}$$

Where: ARF = areal reduction factor (%),

A = catchment area (km²), and

T_C = time of concentration/critical storm duration (hours).

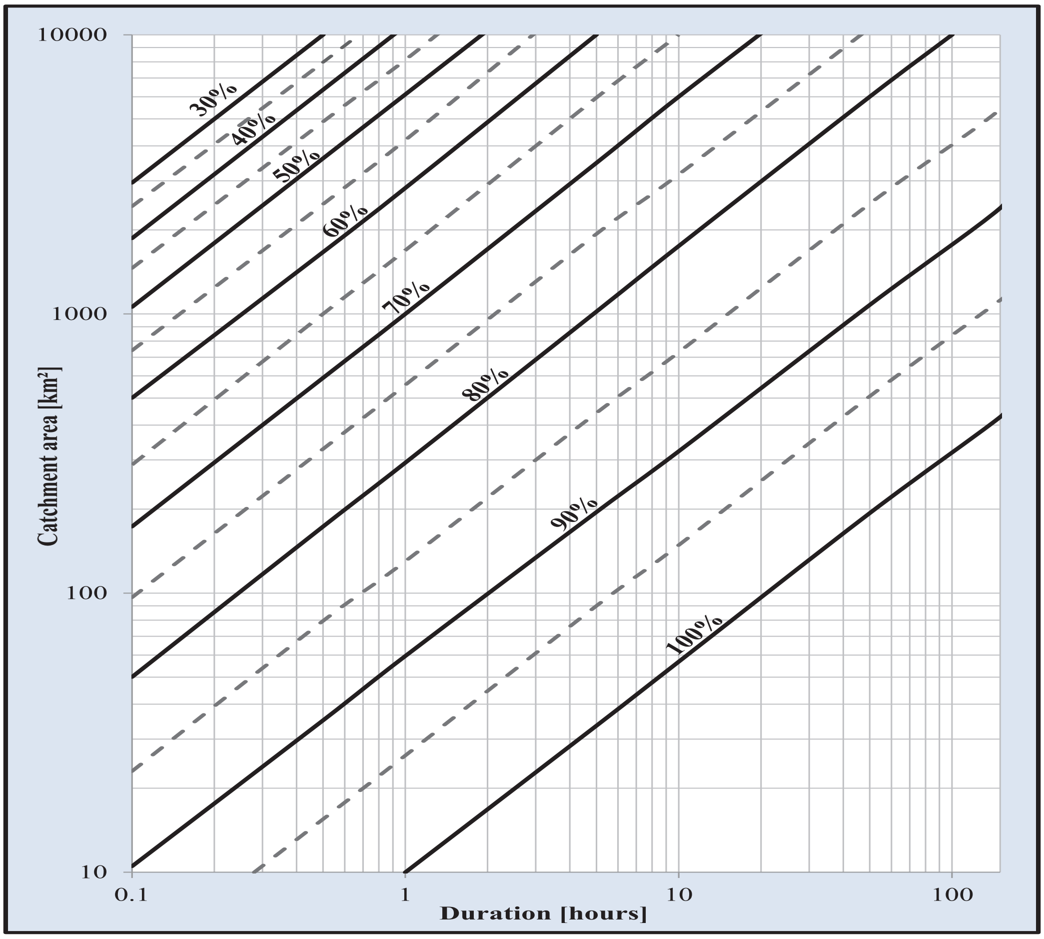


Figure 5-6: Revised ARF diagram for South Africa (Alexander, 2001)

5.3.5. Use of synthetic design hyetographs

Some of the methods used for DFE (e.g. SCS_SA, SWMM) require input hyetographs with time steps shorter than 1-day. If observed rainfall data recorded at intervals of less than 1-day are not available at or near the site of interest, then typically synthetic temporal distributions of rainfall are used to a daily rainfall value into a hyetograph. Three general approaches to synthetic design storm generation have been identified by Mouton *et al.* (2022) as shown in Figure **5-7**, with the Intensity-Duration-Frequency (IDF) based curves being widely used in South Africa.

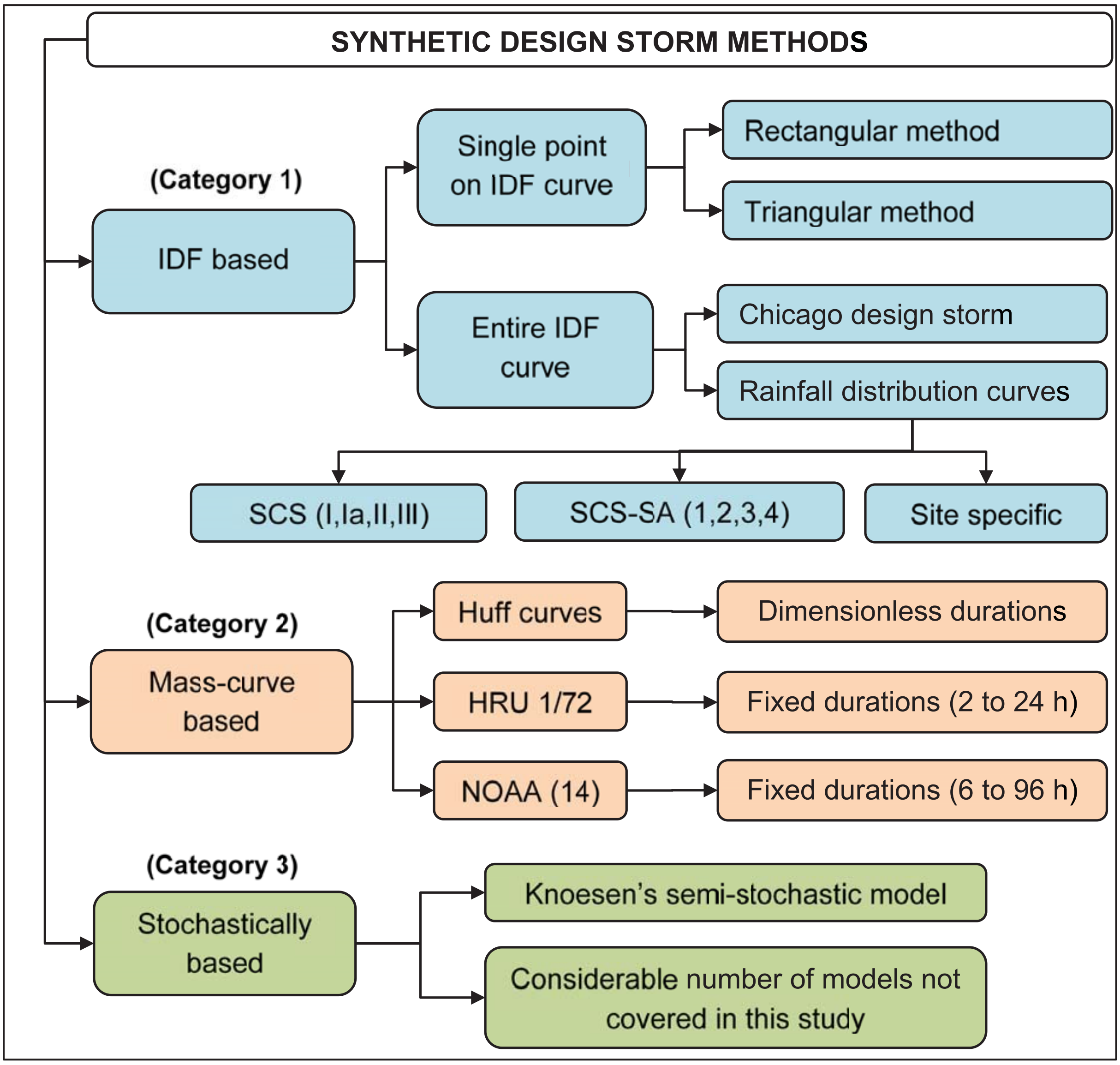


Figure 5-7: General categorisation of approaches used to methods used to estimate design storms (Mouton *et al.*, 2022)

The regionalised SCS-SA curves derived for South Africa by Weddepohl (1988) shown in Figure 5-8 and regions in Figure 5-9. Further details on the distributions are documented in the SCS-SA manuals (Schulze and Schmidt, 1987a)

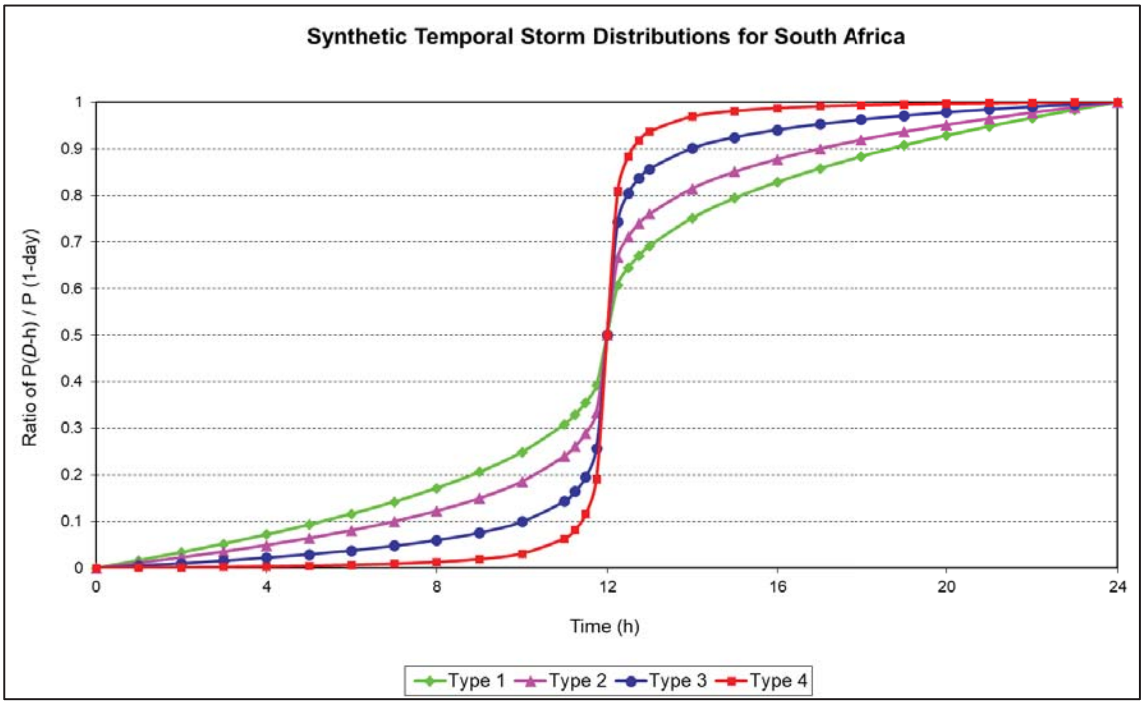


Figure 5-8: SCS-SA synthetic rainfall distributions

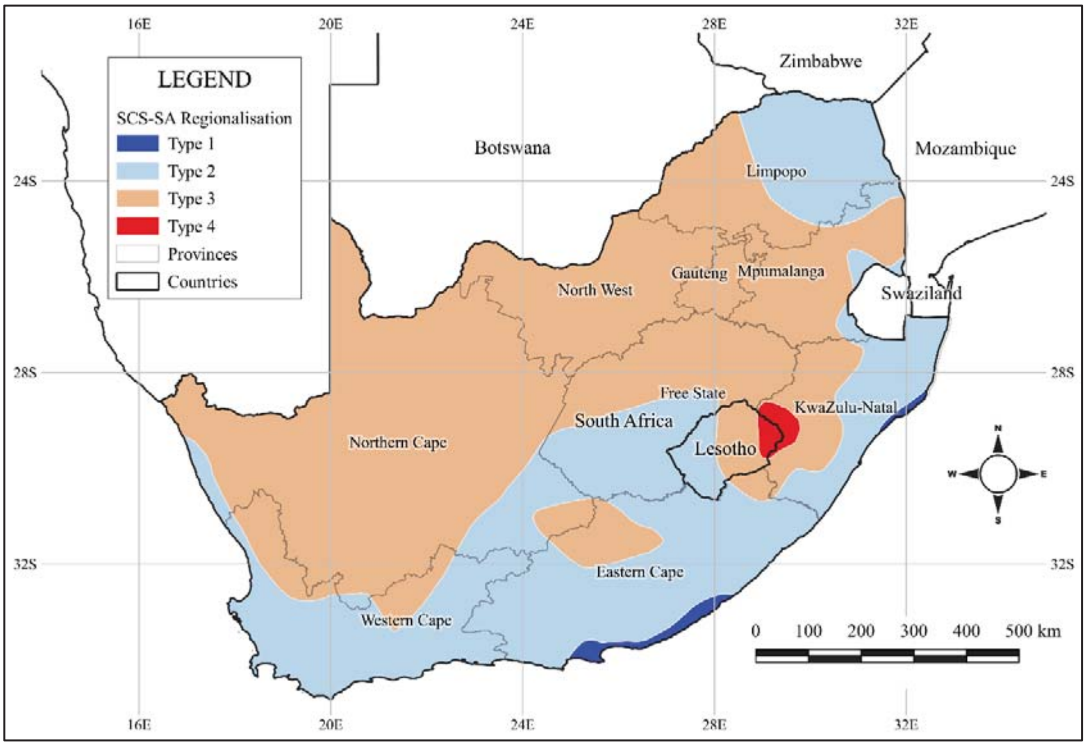


Figure 5-9: SCS-SA regions (Weddepohl, 1988; cited by Mouton *et al.*, 2022)

In a recent pilot study conducted in Gauteng, Mouton *et al.* (2022) recommend that intermediate SCS-SA distributions provide a better fit to at-site design storms and that the performance of the Chicago Design Storm (CDS) was good and has potential for application in South Africa.

These design storm distributions should be applied with circumspection when used with discretised computer models since they are very “peaky” which can result in extremely high rainfall intensities being applied for a short duration at the centre of the hyetograph. It is recommended that hyetographs extracted directly from the continuous rainfall record or stochastically derived from the daily rainfall data will provide a more valid hyetograph.

5.4 Sources of Flow Data and Information

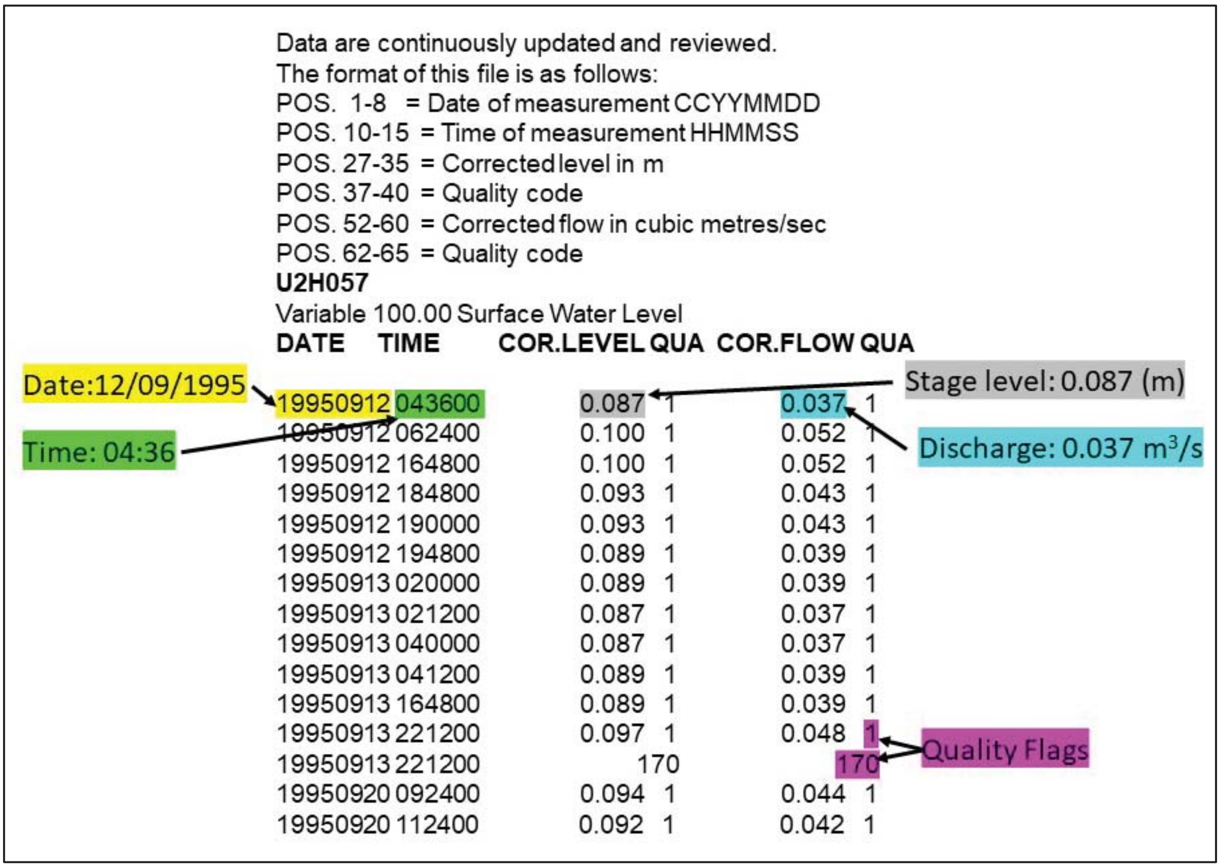
Some Metropolitan Municipalities and local authorities may have detailed historical and up-to-date flow data for some of the catchments under their jurisdiction. Where this type of local flow data is available, this information should be quality controlled and be used in FFAs.

The Department of Water and Sanitation (DWS) is mandated to monitor a network of flow-gauging stations in South Africa. As shown in the red outlined boxes in Figure 5-10, primary, average daily flow, monthly runoff volumes and monthly and annual maximum flow data can be accessed from the following link: https://www.dws.gov.za/Hydrology/Default.aspx. The site also contains the upstream catchment area and latitude and longitude of the site. It is recommended that location of gauges and catchments areas obtained from DWS should be checked against other sources. Downloaded data are in text format and the site is only able to download a limited numbers of records at a time. Hence, if the entire requested record is not available after downloading, the data should be downloaded in batches by altering the start date and combined into a complete record.

Figure 5-10: Data that can extracted from the DWS website

5.4.1. Primary flow data

The primary flow data consist of the time series of flow data as historically digitised from autographic stage recorded data or currently from stage depths recorded by data loggers at flow-gauging sites. Typically, points on the chart are digitised at changes of slope and hence the data points are at irregular intervals. An extract of primary flow data is shown in Figure 5-11 and the quality codes included in the data set need to be understood and carefully interrogated. A list of quality codes and their explanation is contained in Appendix 5 (Chapter 15).


Data are continuously updated and reviewed.
The format of this file is as follows:
POS. 1-8 = Date of measurement CCYYMMDD
POS. 10-15 = Time of measurement HHMMSS
POS. 27-35 = Corrected level in m
POS. 37-40 = Quality code
POS. 52-60 = Corrected flow in cubic metres/sec
POS. 62-65 = Quality code
U2H057
Variable 100.00 Surface Water Level

DATE	TIME	COR.LEVEL	QUA	COR.FLOW	QUA
19950912	043600	0.087	1	0.037	1
19950912	062400	0.100	1	0.052	1
19950912	164800	0.100	1	0.052	1
19950912	184800	0.093	1	0.043	1
19950912	190000	0.093	1	0.043	1
19950912	194800	0.089	1	0.039	1
19950913	020000	0.089	1	0.039	1
19950913	021200	0.087	1	0.037	1
19950913	040000	0.087	1	0.037	1
19950913	041200	0.089	1	0.039	1
19950913	164800	0.089	1	0.039	1
19950913	221200	0.097	1	0.048	1
19950913	221200		170		170
19950920	092400	0.094	1	0.044	1
19950920	112400	0.092	1	0.042	1

Figure 5-11: Example of primary flow data extracted from DWS

5.4.2. Limitations of DWS flow data and stage-discharge rating tables

The flow-gauging network used by DWS was designed primarily to monitor average flow conditions; hence, the structural limit of flow-gauging weirs and the associated maximum rated stage are frequently exceeded under high flow conditions. A summary of missing data and recorded river stages which exceeded the Discharge rating Tables (DTs) in the annual maximum series (AMS) from 806 DWS flow-gauging weirs, which each have 20 or more years of record, is shown in Figure 5-12.

Typically, DTs are used to convert the observed depth of flow or river stage above the weir crest into a discharge (m^3/s) and can be accessed for each flow-gauging site under consideration. A typical example of a DT is shown in Figure 5-13. In this example, the DT has only been generated up to a stage of 3.36 m and all higher stages have the same flow rate (225.3 m^3/s). Hence, all observed data points which have a depth exceeding 3.36 m will have erroneous peak discharges as shown in Figure 5-14.

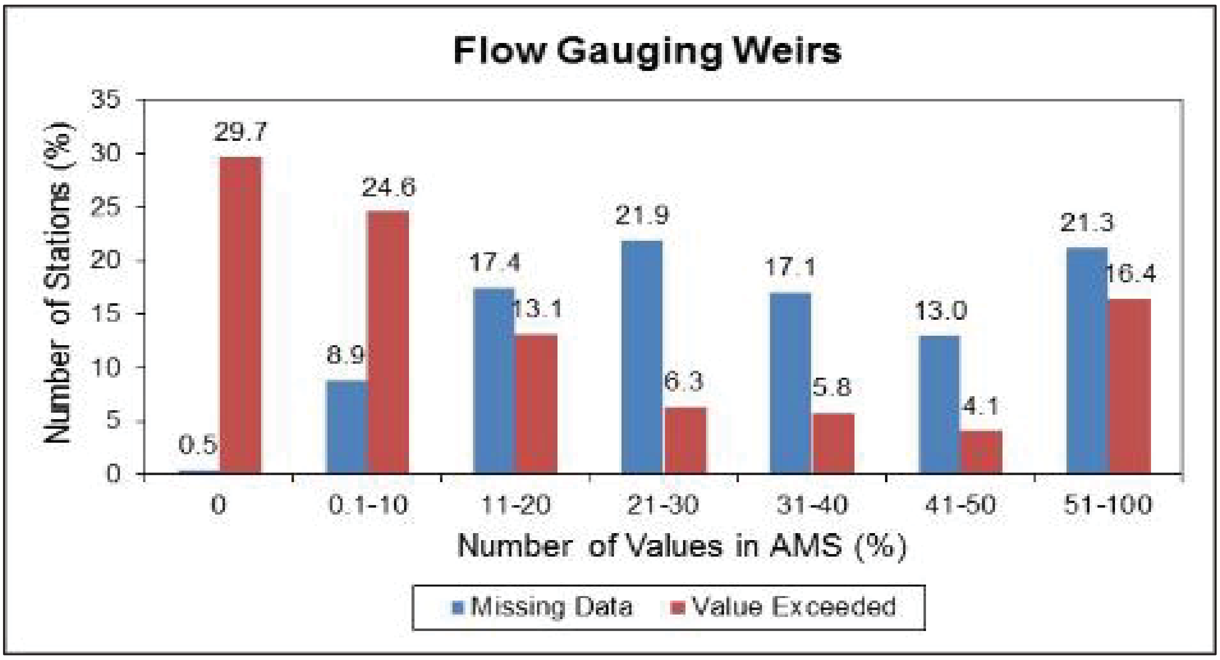


Figure 5-12: Missing data and rating table exceedances at 806 DWS flow gauges which have 20 or more years of record

STATION NO U2H057
DATE OF APPLICATION 1992-06-15
DT NO 2
DISCHARGE IN CUMEC FOR 1CM RISE IN WATER LEVEL

METRE	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.00	0.000000	0.0002	0.0009	0.0026	0.0052	0.0092	0.0144	0.0212	0.0296	0.0398
0.10	0.0518	0.0657	0.0817	0.0998	0.1202	0.1429	0.1679	0.1955	0.2256	0.2584
0.20	0.2939	0.3321	0.3733	0.4174	0.4645	0.5147	0.5681	0.6247	0.6847	0.7480
0.30	0.8147	0.8850	0.9589	1.037	1.118	1.203	1.292	1.385	1.482	1.583
0.40	1.688	1.798	1.911	2.030	2.152	2.280	2.411	2.548	2.690	2.843
0.50	3.010	3.189	3.377	3.574	3.779	3.992	4.212	4.440	4.674	4.915
0.60	5.162	5.416	5.676	5.943	6.215	6.493	6.778	7.068	7.364	7.665
0.70	7.973	8.286	8.604	8.928	9.258	9.593	9.933	10.28	10.63	10.99
0.80	11.35	11.71	12.09	12.46	12.84	13.23	13.62	14.02	14.42	14.83
0.90	15.24	15.66	16.08	16.51	16.94	17.37	17.81	18.26	18.71	19.16
1.00	19.62	20.09	20.56	21.03	21.51	21.99	22.48	22.97	23.47	23.97
1.10	24.47	24.98	25.50	26.02	26.54	27.07	27.60	28.14	28.68	29.22
1.20	29.77	30.32	30.88	31.45	32.01	32.58	33.16	33.74	34.32	34.91
1.30	35.50	36.09	36.69	37.30	37.91	38.52	39.13	39.75	40.38	41.01
1.40	41.64	42.28	42.92	43.56	44.21	44.86	45.52	46.18	46.84	47.51
1.50	48.18	48.86	49.53	50.22	50.90	51.59	52.29	52.99	53.69	54.39
1.60	55.10	55.82	56.53	57.25	57.98	58.71	59.44	60.17	60.91	61.65
1.70	62.40	63.15	63.90	64.66	65.42	66.18	66.95	67.72	68.49	69.27
1.80	70.05	70.83	71.62	72.41	73.21	74.00	74.81	75.61	76.42	77.23
1.90	78.04	78.86	79.68	80.51	81.33	82.16	83.00	83.83	84.67	85.52
2.00	86.36	87.21	88.07	88.92	89.78	90.65	91.51	92.38	93.25	94.12
2.10	95.00	95.88	96.77	97.65	98.54	99.44	100.3	101.2	102.1	103.0
2.20	103.9	104.9	105.8	106.7	107.6	108.5	109.4	110.4	111.3	112.2
2.30	113.2	114.1	115.1	116.0	116.9	117.9	118.8	119.8	120.8	121.7
2.40	122.7	123.6	124.6	125.6	126.5	127.5	128.5	129.5	130.5	131.5
2.50	132.4	133.4	134.4	135.4	136.4	137.4	138.4	139.4	140.4	141.4
2.60	142.4	143.5	144.5	145.5	146.5	147.5	148.6	149.6	150.6	151.7
2.70	152.7	153.7	154.8	155.8	156.9	157.9	159.0	160.0	161.1	162.1
2.80	163.2	164.2	165.3	166.3	167.4	168.5	169.5	170.6	171.7	172.8
2.90	173.8	174.9	176.0	177.1	178.2	179.3	180.3	181.4	182.5	183.6
3.00	184.7	185.8	186.9	188.0	189.1	190.2	191.3	192.4	193.6	194.7
3.10	195.8	196.9	198.0	199.1	200.3	201.4	202.5	203.6	204.8	205.9
3.20	207.0	208.2	209.3	210.4	211.6	212.7	213.8	215.0	216.1	217.3
3.30	218.4	219.6	220.7	221.9	223.0	224.2	225.3	225.3	225.3	225.3
3.40	225.3	225.3	225.3	225.3	225.3	225.3	225.3	225.3	225.3	225.3
3.50	225.3	225.3	225.3	225.3	225.3	225.3	225.3	225.3	225.3	225.3

Figure 5-13: Example of a stage-discharge rating table

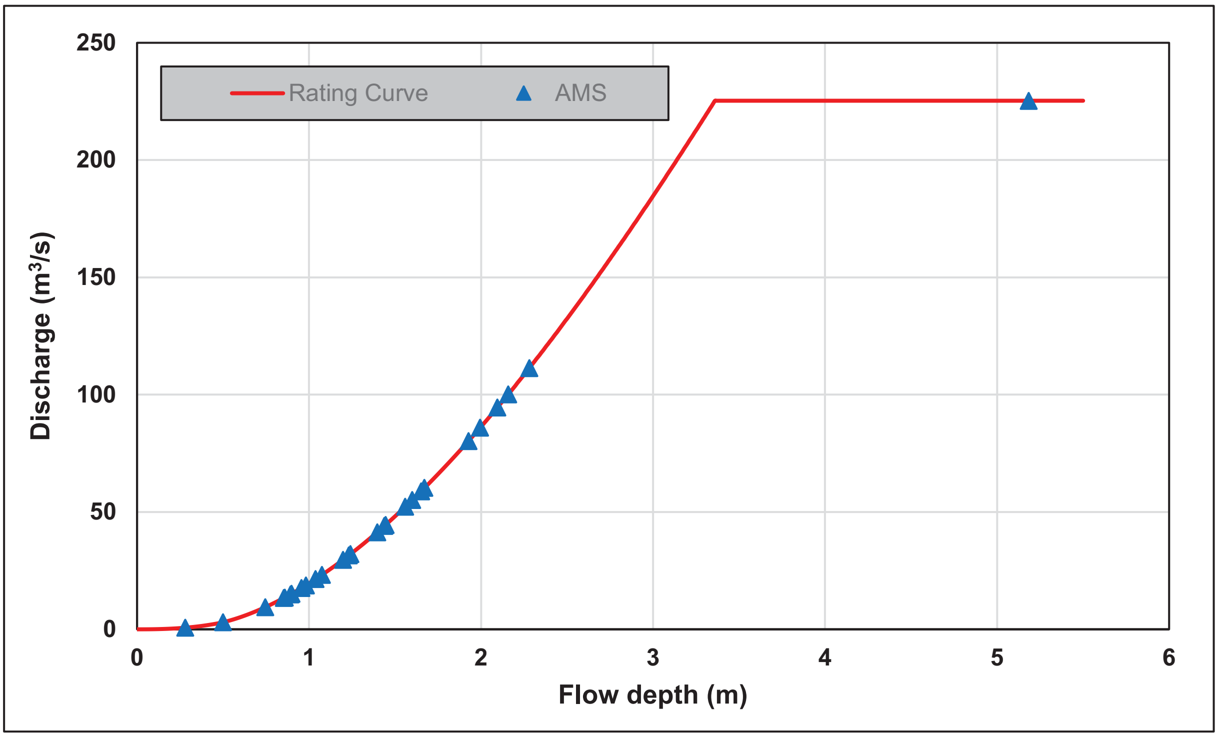


Figure 5-14: Example of a stage-discharge rating curve exceedance with erroneous peak discharge

As shown in Figure 5-14, observed stage levels exceeding the maximum rated stage at gauging weirs is a common problem in South Africa. Hence, in cases where the observed flood levels exceed the maximum rated flood level (H) of a standard DWS DT, the DT may be extended up to bankfull flow conditions using appropriate regression analyses. High flow extensions above bankfull flow conditions should only be considered in cases where the existing DT include floodplain flow on the full width of the floodplain. In summary, individual stage extensions (H_E), whether for bankfull or above bankfull flow conditions, should be limited to a maximum of 20%, i.e. $H_E \leq 1.2\ H.$ In the case of above bankfull flow conditions, the relevance of the general extension procedure described above, should be tested and compared to other relevant hydraulic extension methods, e.g. slope-area method and/or stepped backwater analysis if surveyed cross-section data are available. In addition to the above-mentioned 20% limit, the hydrograph shape, especially the peakedness due to a steep rising limb in relation to the hydrograph base length, and the relationship between observed peak discharge (Q_{Pxi}) and associated direct runoff (Q_{Dxi}) value, should be used as additional criteria to justify the H_E extensions up to 20%. Typically, in such cases, the additional volume of direct runoff (Q_{DE}) due to the extrapolation should be limited to 5%, i.e. $Q_{DE} \leq 0.05\ Q_{Dxi}$ (Gericke and Smithers, 2017).

Given that there is no one-size-fits-all approach/method available for the extension of stage-discharge rating curves in South Africa, and that it is not possible to apply all the different indirect extension methods (e.g. hydraulic, and one-dimensional modelling methods) and/or direct extension methods (e.g. at-site conventional current gauging) at each site, it is recommended that DTs should be extended to include all the recorded levels and no data in the AMS should be excluded when estimating the design flood. However, the individual stage extensions (H_E) should be limited to a maximum of 20%, i.e. $H_E \leq 1.2\ H,$ while adhering to the $Q_{DE} \leq 0.05\ Q_{Dxi}$ limit.

Overall, the extension of DTs should be carefully considered and a careful analysis of the quality flags in the flow data should be done. Typically, the quality control procedures outlined in Section 5.9 should be undertaken.

5.4.3. Flow data for flood frequency analysis

As shown in Figure 5-10, the AMS data can be downloaded from the DWS website for use in probabilistic FFA (see Chapter 6). An example of the extracted AMS is shown in Figure 5-11. The data quality flags ($, A, M) included in the extracted AMS data must be investigated and the impacts of these on the estimated design peak discharges need to be investigated and understood.

5.4.4. Anecdotal records

If local information is not available, the local Municipality should be encouraged to establish a database of flood observations and encourage municipal officials and members of public to report flood events. Information that will be useful to flood studies, include the following:

(a) The time and date of the observed flood event.

(b) The water level (high water mark, photos, etc.).

(c) The condition of the stream during and after flood has subsided (e.g. photos).

Newspaper reports can also be a useful source of the above information.

Department of Water and Sanitation			SAFMAXLEVELFLOW Output 2022/08/15 12:02		
Yearly Maximum Values for Hydrological Years ordered by DATE					
U2H057 Slang Spruit @ Pietermaritzburg					
Hydro		**MAX**	**MAX**	**MAX**	
Year	**Date**	**Time**	**Level (m)**	**low (cumec)**	
1996	19951019	21:00	0.501	3.027	M
1997	19970906	19:00	0.281	0.691	M
1998	19980221	20:24	1.438	44.078	M
1999	19990202	19:24	2.281	111.393	M
2000	19991223	15:12	1.199	29.716	M
2001	20001108	13:24	1.995	85.940	M
2002	20011201	22:35	5.184	225.331	A
2003	20030511	18:22	0.855	13.426	
2004	20040223	16:34	1.397	41.449	
2005	20050327	16:41	1.601	55.175	M
2006	20060319	16:36	0.746	9.458	M
2007	20061221	16:00	0.893	14.951	
2008	20071101	19:48	1.672	60.319	
2009	20081227	20:24	1.232	31.558	
2010	20091209	22:12	1.560	52.289	
2011	20110506	18:12	0.899	15.198	
2012	20111127	23:24	0.859	13.583	M
2013	20130106	17:48	0.957	17.680	M
2014	20140224	18:12	1.446	44.599	$
2015	20150301	16:24	2.096	94.650	
2016	20160316	20:00	1.242	32.125	
2017	20170411	21:48	1.076	23.268	M
2018	20180404	19:12	1.653	58.924	M
2019	20190423	04:00	1.038	21.413	M
2020	20191113	17:36	1.928	80.340	M
2021	20201118	19:12	0.982	18.799	M
2022	20211230	18:36	2.158	100.151	M
2023	20221001	00:00			M
Explanation of codes:					
$ ... Gauge Plate Readings					
A ... Above Rating					
M ... Missing Data					

Figure 5-15: Example of AMS data set extracted from DWS website

5.5 Climate Data

Some of the CSM models used for DFE as listed in Chapter 8 require additional climatological data sets as input. The institutions listed as sources of rainfall data in Section 5.3 generally also monitor other climate variables (e.g. temperature, evaporation, etc.) and can be approached for access to climate data.

The South African Atlas of Climatology and Agrohydrology produced by Schulze (2007) is an extremely useful source of agrohydrological information and statistics (e.g. MAP). The atlas is accessible via SAEON's data portal (https://sarva.saeon.ac.za/). There are more than 3 300 data sets available via this portal (https://catalogue.saeon.ac.za/), of which 457 form part of the Climatological and Agrohydrological Atlas. The website can be searched for information.

For example, the MAP grid developed by Lynch (2004) can also be found at this link: https://catalogue.saeon.ac.za/records/10.15493/SARVA.BEEH.10000054. Another source of climate data assembled at Quaternary Catchment level for the WR2012 study (Bailey and Pitman, 2016) which can be accessed after registration using the following link: https://waterresourceswr2012.co.za/.

5.6 Catchment Geomorphology and Topography

Currently, with the availability of Geographical Information Systems (GIS), which has encompassed every field in engineering and natural sciences, accurate, efficient and consistent methods are available to estimate geomorphological catchment characteristics. Comprehensive sets of spatial and hydrological tools are available in both commercial, e.g. ArcGIS™ (ESRI, 2016) and open-source, e.g. GRASS (2017) and QGIS (2017) software packages.

In this best practice guideline, reference to specific software applications in the GIS environment, is neither an endorsement of the software nor regarded as compulsory. Subsequently, practitioners are free to use any software application meeting their requirements, while any reference being made to any software packages in any section is merely done to highlight the typical processes and/or steps involved.

5.6.1. Open-source GIS-based software

Given the obvious financial benefit/implication of using open-source software, the following packages are available for downloading in the public domain:

Quantum GIS: The market leader in open-source GIS software. It has a large user base and support, with on-going developments by a devoted volunteer community. It has 900+ tools available in 25 toolboxes which are comparable to the functionalities available in ArcGIS.

However, some of the highly specialised tools available in ArcGIS are not currently available from QGIS. In addition, QGIS has known stability issues associated with 3-D and LiDAR data processing. QGIS Version 3.22 LTR or newer can be downloaded at: https://qgis.org/en/site/forusers/download.html

Google Earth Pro: Free desktop tool with advanced GIS and mapping features to create maps, import and export GIS data, and access historical images. It can be downloaded at: https://www.google.com/earth/download/gep/agree.html

LAStools: Desktop tool that can be used to process LiDAR data files by using a collection of highly efficient, scriptable tools with multi-core batching that process LAS, compressed LAZ, Terrasolid BIN, ESRI shapefiles, ASCII and others. It is also compatible with QGIS to view a point clouds in 2-D or 3-D. It can be downloaded at: https://downloads.rapidlasso.de/LAStools.zip

Hydrologic Modelling System (HEC-HMS): The software is designed to simulate all the hydrological processes within a dendritic catchment system. In terms of catchment geomorphology, it is very useful for catchment delineations and the estimation of catchment characteristics, e.g. longest flow path length and slope, centroidal flow path and slope, 10-85 slope, average catchment slope, and various catchment shape parameters. HEC-HMS 4.10 Portable Version or newer can be downloaded at:
https://www.hec.usace.army.mil/software/hec-hms/downloads.aspx

PCSWMM: This software has sophisticated built in GIS functionality that enables automatic catchment subdivision and drainage system discretisation along with the display of background images and web based maps that significantly enhances the capabilities of EPS-SWMM. A trial version can be obtained from: https://www.pcswmm.com/Trial

5.6.2. Geomorphological data sources

All the data related to catchment geomorphology and topography, as outlined in this and the subsequent sections, should be verified using one or more approach when no meta data details are available. For example, relying only upon site development surveys for flood line determination would result in inconsistent flood line estimates along a watercourse. Hence, alternative, or additional data sources should be consulted, given that survey details may vary, and the coverage is usually limited to the property boundary rather than the floodplain area and a proper hydraulic analysis would be affected. Thus, it is incumbent on the DFE practitioner to instruct the surveyor to extend the survey sufficiently to ensure accurate computation of the water surface profile. In the absence of a well-defined control, the survey should extend at least 5 times the expected width of the floodwater spread downstream of the reach of interest.

Publicly available sources of terrain data are usually too coarse for adequate flood mapping (e.g. 5 m contours). In such cases, it is recommended that a municipal survey of sufficient detail (e.g. 0.25 m contours) is used for flood mapping. However, obtaining baseline survey coverage data for the whole municipal area may not be affordable in the short-term, but it should be an objective in the medium-term. Modern aerial survey methods, e.g. LiDAR, provide high levels of detail and are suitable for a wide range of planning and design requirements related to flood and hazard mapping. In addition, these detailed surveys are also useful for catchment delineations required for DFE.

The Chief Directorate of National Geo-spatial Information (CDNGI), formerly the Chief Directorate of Surveys and Mapping (CDSM), is regarded as the main source of GIS-based geomorphological data in South Africa. Typically, raster data sets, aerial photography, various map series (e.g. 1: 10 000 Orthophotos, 1: 50 000 Topographical maps, and 1: 250 000 Regional maps), geodetic information, and ancillary data sets are available. The CDNGI Geospatial Portal can be accessed at: http://www.cdngiportal.co.za/CDNGIPortal/.

Despite having all the above information available in a digital format, experience has shown that many novice practitioners struggle to identify the appropriate map series in question. Hence, the subsequent paragraphs serve as a summarised explanation:

Typically, the 1: 250 000 map series sheets cover an area of 1° latitude and 2° longitude . The maps are designated with a four-digit code derived from the top left-hand (north-west) corner of the map; the first two depicting the degree latitude and the second two depicting the degree longitude. The 2923 map will therefore cover the area from 29° to 30° latitude, and between 23° and 25° longitudes (Haarhoff and Cassa, 2009).

The next map series is the 1: 50 000 Topographical map series, each map covering a square of one quarter of a degree in both directions. Figure 5-16 shows how the 2824 square degree is split into sixteen 1: 50 000 maps. The 1: 10 000 Orthophoto series is the most detailed and readily available. Each map covers an area of three minutes by three minutes. There are 25 such maps for each of the 1: 50 000 maps, numbered from 1 to 25 in the same sequence as you would read a book (*i.e.* five rows of five numbers). Therefore, map 2926AB13 would be an Orthophoto exactly in the centre of the 2926AB 1:50 000 map.

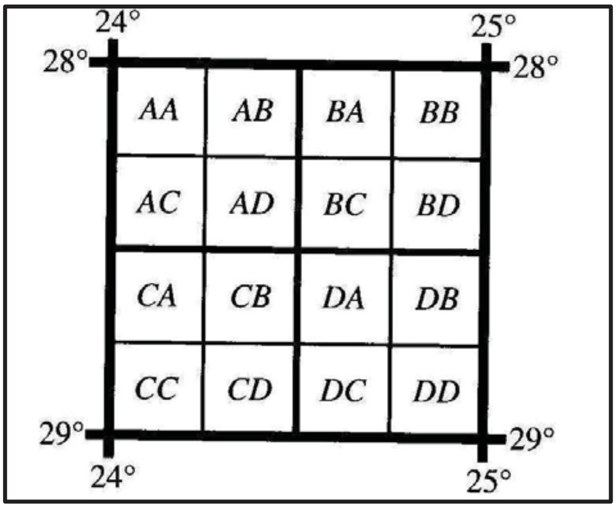


Figure 5-16: Splitting the 2824 square degree into 1: 50 000 maps (after Haarhoff and Cassa, 2009)

With Global Positioning System (GPS) technology, it is easy to find an exact coordinate in the field. It is also useful to be able to rapidly work out which map covers a selected location. Consider the following GPS coordinates: (i) Latitude, 25° 35' 22" S, and (ii) Longitude, 27° 04' 53" E. The first four digits of the map number are 2527, taken directly from the degrees' latitude and longitude. The minutes of latitude show that the coordinate falls between 30' and 45' (thus the third quarter of the degree). The minutes of longitude show that the coordinate falls between 0' and 15' (thus the first quarter of the degree). Remembering the notation shown in Figure 5-16, the next two digits are thus CA. For the 1:50 000 map, the map number is thus 2527CA. To get the map number for the 1:10 000 map, take this process further within map 2527CA. The minutes of latitude show that the coordinate falls between 33' and 36' (thus the second 'column' of the map). The minutes of longitude show that the coordinate falls between 3' and 6' (thus the second 'row' of the map). The map number is thus 2527CA07.

Apart from the CDNGI Geo-spatial data portal, the following websites/data sets are regarded as useful sources of geo-spatial information related to DFE:

(a) **Digital elevation models:** The Shuttle Radar Topography Mission (SRTM) Digital Elevation Model (DEM) for Southern Africa at either a 90-m or 30-m resolution are available at: https://www.usgs.gov/tools/earthexplorer. Google Earth most often utilises the SRTM 1-arc second data. The SRTM 1-arc second DEM is quite suitable for catchment boundary delineation, but it generally does not result in a good representation of drainage patterns, except possibly in rugged terrain.

As an alternative, the ASTER 30-metre DEM could also be used; however, the SRTM 30-m DEM is claimed to be more accurate than the ASTER 30 DEM, given that SRTM uses radar observations to construct the DEM, while ASTER uses stereo imagery and photogrammetric techniques to extract the DEM. The ASTER 30 DEM are available at: https://www.earthdata.nasa.gov/news/new-aster-gdem

The Alaska Satellite Facility (ASF) created radiometrically terrain corrected (RTC) products by correcting synthetic aperture radar (SAR) geometry and radiometry, to result in the Alos Palsar RTC DEM (GeoTIFF file) at either a 12.5-metre and 30-metre resolution, respectively. The respective DEMs are available at: https://asf.alaska.edu/data-sets/derived-data-sets/alos-palsar-rtc/alos-palsar-radiometric-terrain-correction/

Some Metros in South Africa, for example the City of Tshwane (email GeoInfoService@TSHWANE.GOV.ZA), the City of Johannesburg, (start here https://eservices.joburg.org.za/onlinemaps or here https://eservices.joburg.org.za/onlinemaps/Pages/Log-a-Query.aspx) the City of Ekurhuleni (email onlinemaps@ekurhuleni.gov.za) and eThekwini, (start here https://gis-ethekwini.opendata.arcgis.com/ or here http://gis.durban.gov.za/gis_Website/internetsite/) have high resolution digital elevation date that they supply on request. These data are available either in DEM (geotiff) format with a 1 m cell size (CoJ) or in LiDAR DTM (ASCII or .las) format with a point density capable of yielding a DEM raster surface with a resolution oof 1 m or better.

Vieira (2018) evaluated the suitability of different DEMS to derive geomorphological catchment characteristics. DEMs were obtained/generated using five different sources, e.g. open-source DEMs (SRTM 90, SRTM 30, Aster 30, and Alos Palsar), and field data (Unmanned Aerial Vehicle (UAV), and Terrestrial Laser scanner). All data sets were compared against a total of 18 control points obtained from the Global Navigation Satellite System-GNSS in Brazil, South America. The field-based techniques outperformed the open-source DEMs, but given the financial implications involved, the Alos Palsar DEM was recommended.
In this best practice guideline, the use of the Alos Palsar DEM is thus recommended, while in all cases where detailed hydraulic analyses need to be conducted, LiDAR data for the extraction of contours at a 0.25 m interval is recommended.

Irrespective of which DEM source is used, a hydrologically corrected and depressionless DEM is required for the determination of geomorphological catchment characteristics. For example, the *Hydrology* toolset contained in the *Spatial Analyst Tools* toolbox of ArcGIS™ can be applied to provide a hydrologically corrected and depressionless DEM. In other words, all 'sinks', i.e. cells with a lower elevation compared to the surrounding cells, are filled to generate continuous flow direction and flow accumulation rasters for the identification of catchment areas for specified pour points located at the catchment outlet. The hydrologically corrected DEM also need to be projected and transformed to enable the estimation of geomorphological catchment characteristics, e.g. area (A), perimeter (P), hydraulic length (L_H), centroid distance (L_C), average slope (S), etc.

Care should be taken when hydrologically correcting DEMs to remove sinks in areas with known or suspected endorheic drainage, for example deflation pans in the Kalahari or the eastern Highveld.

(b) **SCALGO** Live: Innovative, large-scale terrain data-processing technology and digital tools. Incorporates a national flood risk platform for working with climate adaptation, urban planning, emergency management, and administration of watercourses with specific applications in catchment delineation and flow accumulation: https://scalgo.com/en-US/live-flood-risk

(c) **Sentinel Hub**: Satellite data, e.g. Sentinels, Landsat and other providers easily accessible for browsing and/or analyses: https://www.sentinel-hub.com/

(d) **Meshroom:** 3-D Reconstruction Software based on the AliceVision framework, which is a Photogrammetric Computer Vision Framework providing 3-D Reconstruction and Camera Tracking algorithms to produce high-resolution DEMs and enable photogrammetric applications in DFE: https://alicevision.org/#meshroom

(e) **PlanetGIS**: Focuses on decision support and asset management systems. The complete relational database platform combined with the power of spatial modelling, make it ideal for the manipulation, re-classification, and linking various relations between data sets, e.g. land cover and contours: https://planetgis.co.za/download.php

(f) **DWS water quality data exploration tool layers:** Primary, secondary, and quaternary drainage regions, rivers, river orders (Strahler order), dams, lakes and lagoons in South Africa and Google Earth (*.kmz) file format: https://www.dws.gov.za/iwqs/wms/data/000key2data.asp

5.6.3. Catchment area

Catchment areas can be reasonably accurately determined by using appropriate software, provided that a hydrologically corrected and depressionless DEM is prepared. The following process is suggested when catchment areas need to be estimated:

(a) Use Google Earth to identify and confirm the exact location of the catchment outlet or flow-gauging weir.

(b) Use the applicable map series and appropriate Computer Aided Design (CAD) or GIS software (e.g. ArcGIS, QGIS, and/or HEC-HMS) to delineate and determine the catchment areas.

(c) The calculated catchment areas can/should be compared to the area obtained from other sources (if available).

(d) If the two areas for the catchment are markedly different, then further investigations are required. Where necessary, corrections should be made to the areas provided from the above sources.

5.6.4. Catchment hydraulic length and centroid

The hydraulic length (L_H) is the distance measured along the longest river from the catchment outlet to the catchment boundary upstream of the fingertip tributary. For example, in ArcGIS, it can be estimated using the *Longest Flow Path* tool in the *Hydrology* toolset. Similarly, the *Mean Center* tool in the *Measuring Geographic Distributions* toolset contained in the *Spatial Statistics Tools* toolbox can be used to estimate the centroid of a catchment. The centroid distance (L_C), i.e. the distance along the main river/watercourse between the outlet and the point on the main river closest to the centroid of the catchment, can then be established by using the *Measure* tool in ArcMap.

However, as highlighted above, the use of a specific software applications in the GIS environment, is neither recommended nor regarded as compulsory, for example both L_H and L_C could be determined in QGIS as well.

5.6.5. Average catchment slope

Currently, given the general availability of high resolution DEM data sets, it is assumed that most practitioners would rely on using these data sets and GIS software (e.g. ArcGIS, QGIS, and/or HEC-HMS) to estimate the average catchment slope. Although, it still remains important to have the relevant background about the fundamentals involved. Thus, as an alternative, the average catchment slope (S) can also be determined using the following manual/semi-automated methods:

(a) **Grid method:** A grid of at **least** 50 squares is superimposed over the catchment area. At each grid intersection point, the horizontal (shortest) distance between the contour intervals which straddle the grid point along a line that passes through the grid point, is measured. The average catchment slope (Eq. 5-2) is consequently defined as the average slope perpendicular to the nearest contour line at each grid point as illustrated in Figure 5-17 (Alexander, 2001).

$$S = \frac{\Delta H}{\sum_{i=1}^{N} \frac{L_i}{N}} \qquad \text{Eq. 5-2}$$

(b) **Empirical method:** Schulze *et al.* (1992) suggested the use of the following empirical method (Eq. 5-3) to determine the average catchment slope**:**

$$S = \frac{M \Delta H * 10^{-2}}{A} \qquad \text{Eq. 5-3}$$

(c) **Neighbourhood or Average Maximum Technique:** This method (Eq. 5-4) is included as the standard slope algorithm in the ArcGIS™ environment to generate slope rasters from a raw DEM and/or point elevation GIS data sets to enable the determination of average catchment slopes and steepness frequency distributions. The slope raster generation is based on a cell matrix approach which represents the maximum change in elevation over the distance between the cell and its eight neighbouring cells. Typically, in a 3 x 3 search window (grid network with nine cells, C_1 to C_9), eight grid points from the surrounding cells are used to calculate the average slope of the central cell (C_5) using unequal weighting coefficients, which are proportional to the reciprocal of the square of the distance from the kernel centre (ESRI, 2006).

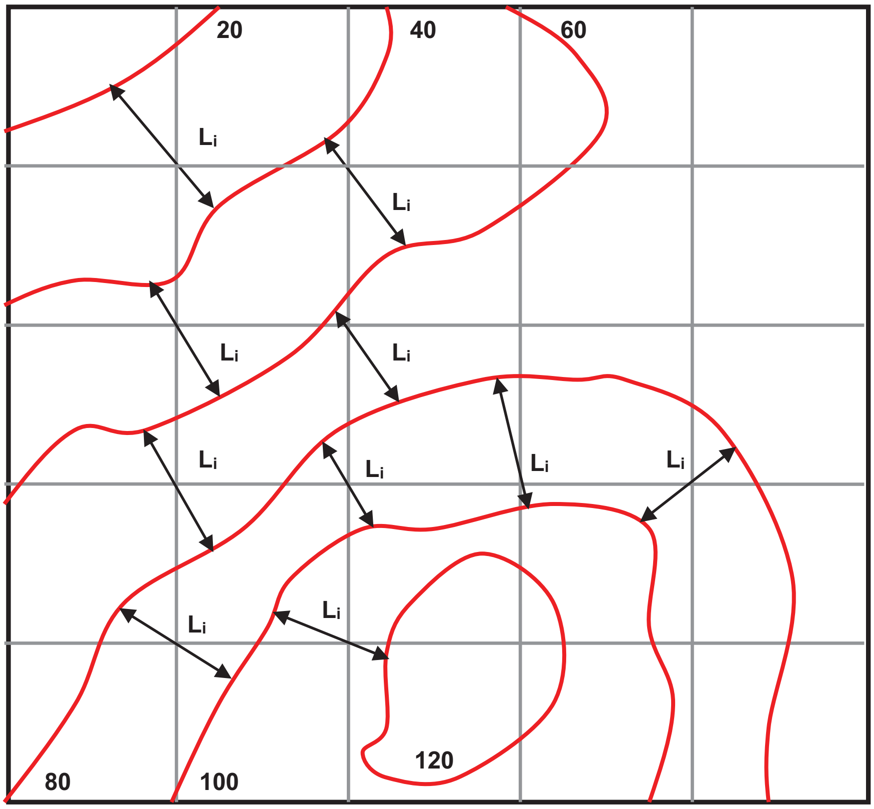


Figure 5-17: Average catchment slope using the Grid method (Alexander, 2001)

$$S = \sqrt{\left(\frac{\Delta z}{\Delta x}\right)^2 + \left(\frac{\Delta z}{\Delta y}\right)^2} \qquad \text{Eq. 5-4}$$

where:
- S = average catchment slope (m/m),
- A = catchment area (km²),
- $C_{1\text{-}4/\,6\text{-}9}$ = surrounding cells,
- C_5 = centre cell,
- ΔH = contour interval (m),
- L_i = horizontal distance between consecutive contours (m),
- M = total length of all contour lines within the catchment (m),
- N = number of grid points or cells,
- x_C = east-west cell size,
- y_C = north-south cell size,
- $\frac{\Delta z}{\Delta x}$ = rate of change of the slope surface n an east-west direction

from the centre cell (C_5) = $\left[\frac{(C_3 + 2C_6 + C_9) - (C_1 + 2C_4 + C_7)}{(N\,x_C)}\right]$, and

$$\frac{\Delta z}{\Delta y}$$ = rate of change of the slope surface in a north-south direction

from the centre cell (C_5) = $\left[\frac{(C_7 + 2C_8 + C_9) - (C_1 + 2C_2 + C_3)}{(N\, y_C)}\right]$.

5.6.6. Length of longest watercourse and average slope

The main watercourse length (L_{CH}) is defined as the distance measured along the main channel from the catchment outlet to the end of the channel (fingertip tributary) near the catchment boundary. This distance can be measured quite accurately on topographical maps (1: 50 000) by using a divider set at 0.25 km (5 mm). In the ArcGIS™ environment, L_{CH} can be estimated using the *Longest Flow Path* tool in the *Hydrology* toolset, while the longitudinal profiles can be obtained from the DEM using the *Stack Profile* tool in the *Functional Surface* toolset contained in the *3D Analyst* toolbox (Gericke, 2019). The average main watercourse slope can be determined by using the following methods (Van der Spuy and Rademeyer, 2021):

(a) **Equal-area method:** An average slope line is drawn or positioned in relation to the longitudinal profile of the main watercourse in such a way that the area above (A_1) this line equals the area below (A_2) the line. This relationship is shown in Eq. 5-5.

$$S_{CH} = \frac{(H_T - H_B)}{L_{CH}} \qquad \text{Eq. 5-5}$$

(b) **10-85 method:** The United States Geological Survey (USGS) developed this method, with the relationship shown in Eq. 5-6.

$$S_{CH} = \frac{\left(H_{0.85L_{CH}} - H_{0.10L_{CH}}\right)}{(0.75L_{CH})} \qquad \text{Eq. 5-6}$$

(c) **Taylor-Schwarz method:** This method is preferred by the Department of Water and Sanitation (DWS) and the Natural Environment Research Council (NERC) also proposed the use thereof in the United Kingdom Flood Studies Report (UK FSR) (NERC, 1975; Van Der Spuy and Rademeyer, 2021). The longitudinal profile of the main watercourse is subdivided into sub-reaches of which the velocities are related to the square root of the slope. The index is equivalent to the slope of a uniform channel with the same length as the longest watercourse and an equal travel time. This relationship is shown in Eq. 5-7.

$$S_{CH} = \left(\frac{L_{CH}}{\sum_{i=1}^{N} \frac{L_i}{\sqrt{S_i}}} \right)^2 \qquad \text{Eq. 5-7}$$

where:

$A_i = \left(\frac{H_i + H_{i+1}}{2} - H_B \right) L_i$ (m²),

$H_T = \frac{\left(\sum_{i=1}^{N} A_i * 2 \right)}{L_{CH}} + H_B$ (m),

H_B = height at catchment outlet (m),

H_i = specific contour interval height (m),

L_{CH} = length of main watercourse (m),

L_i = distance between two consecutive contours (m),

$H_{0.85L}$ = height of main watercourse at length $0.85L_{CH}$ (m),

$H_{0.10L}$ = height of main watercourse at length $0.10L_{CH}$ (m),

S_{CH} = average main watercourse slope (m/m), and

S_i = slope between two consecutive contours (m/m).

Given the ease of application and possible inclusion of Eqs. 5-5 to 5-7 in an automated spreadsheet application, the use of all three equations to highlight any possible uncertainties involved, is recommended.

5.7 Land Cover

A number of sources of Land Cover (LC) information are available for use in DFE. Generally, the use of locally derived information should be prioritised. However, in the absence of local detailed LC information, generalised information derived at a national scale may be used.

The date of the data source of the LC cover information should be checked. This is to ensure that information from old data sources, which may not represent current LC conditions, is not inadvertently used. For DFE, it is recommended that current LC information should be used. It is also recommended that projections of possible future LC conditions in the catchment should also be investigated and used to assess the potential on future flood conditions in the catchment as a consequence of changing LC dynamics in the catchment.

5.7.1. Use of local land cover information

Some Metropolitan Municipalities and local authorities may have detailed historical and current information on the LC for catchments under their jurisdiction. Where available, this information should be used in preference to national approaches to derive LC information.

Municipalities should generate and retain LC information for catchments within their municipal boundaries. This information already forms part of the Integrated Development Plan (IDP) and Spatial Development Framework (SDF) that are reviewed and updated on a regular basis. Importantly, these will also contain planned future development in the catchment which is an important factor in DFE.

Municipal offices should make such information available for practitioners undertaking DFE. In doing so, they will be responsible for ensuring the information is up-to-date and used in a consistent manner.

5.7.2. Use of national land cover information

A number of National Land Cover (SANLC) data sets have been derived for South Africa and these data sets may be used to determine LC for given historical time. These data sets are summarised below:

(a) **Acocks (1988):** Natural vegetation classes represented by Acocks' Veld Types (Acocks, 1988): http://daffarcgis.nda.agric.za/portal/home/item.html?id=b6313f58bc8349ddb56808ff6e14ead4

(b) **NASA-SEDAC:** Global Man-made Impervious Surface (GMIS) Dataset for 2010 derived from Landsat, v1 (2010): https://sedac.ciesin.columbia.edu/data/set/ulandsat-gmis-v1

(c) **SANBI National Vegetation maps:** https://www.sanbi.org/biodiversity/foundations/national-vegetation-map/

(d) **SANLC 1990:** Actual land cover/use classes in the 1990 National Land Cover data set (DEA and GTI, 2016): https://egis.environment.gov.za/sa_national_land_cover_datasets

(e) **SANLC 2000:** Actual land cover/use classes in the 2000 National Land Cover data set (ARC and CSIR, 2005): https://egis.environment.gov.za/sa_national_land_cover_datasets

(f) **SANLC 2013/2014:** Actual land cover/use classes in the 2013/2014 National Land Cover data set (DEA and GTI, 2015): https://egis.environment.gov.za/sa_national_land_cover_datasets

(g) **SANLC 2018:** Actual land cover/use classes in the 2018 National Land Cover data set (DEA and GTI, 2019): https://egis.environment.gov.za/sa_national_land_cover_datasets

(h) **SANLC 2020:** Actual land cover/use classes in the 2020 National Land Cover data set (DEA and GTI, 2021): https://egis.environment.gov.za/sa_national_land_cover_datasets

(i) **Tsinghua University 2018:** Annual change information of global impervious surface areas from 1985 to 2018 at a 30 m resolution: https://developers.google.com/earth-engine/datasets/catalog/Tsinghua_FROM-GLC_GAIA_v10

The SANLC 2013/2014 data set contains 72-LC classes. The SANLC 1990 data set was subsequently generated as a complementary data set with the same 72-LC classes used in the SANLC 2013/2014 data set. These 72-LC classes are aligned with the South African National Standards (SANS) 1877, which is the SA Bureau of Standards designated NLC classification standard for South Africa (DEA and GTI, 2016).

The SANLC 2018 consists of 73-LC classes which includes natural rivers, lakes, estuaries, lagoons, and artificial dams with individual class definitions that are more hydrologically focused than the SANLC 2013/2014 data set. The SANLC 2018 data set also contains more detailed fallow land, coastal land, road and rail and mine classes that have no direct equivalent classes in the SANLC 1900 and 2013/2014 (DEA and GTI, 2019). The LC classes in the SANLC 2018 adhere to new gazetted LC classification standard (SANS 19144-2). However, modifications were made to ensure compatibility and comparability to the previous SANLC 1900 and 2013/2014 data sets.

An example of an image from the SANLC 2018 data set is shown in Figure 5-18.

The NLC information should always (or as far as possible), be validated using reliable local information. For example, Google Earth imagery for corresponding time periods can be used to validate the NLC and hydrological condition for the period of simulation, and to check if there have been any significant LC or land use changes in the catchment.

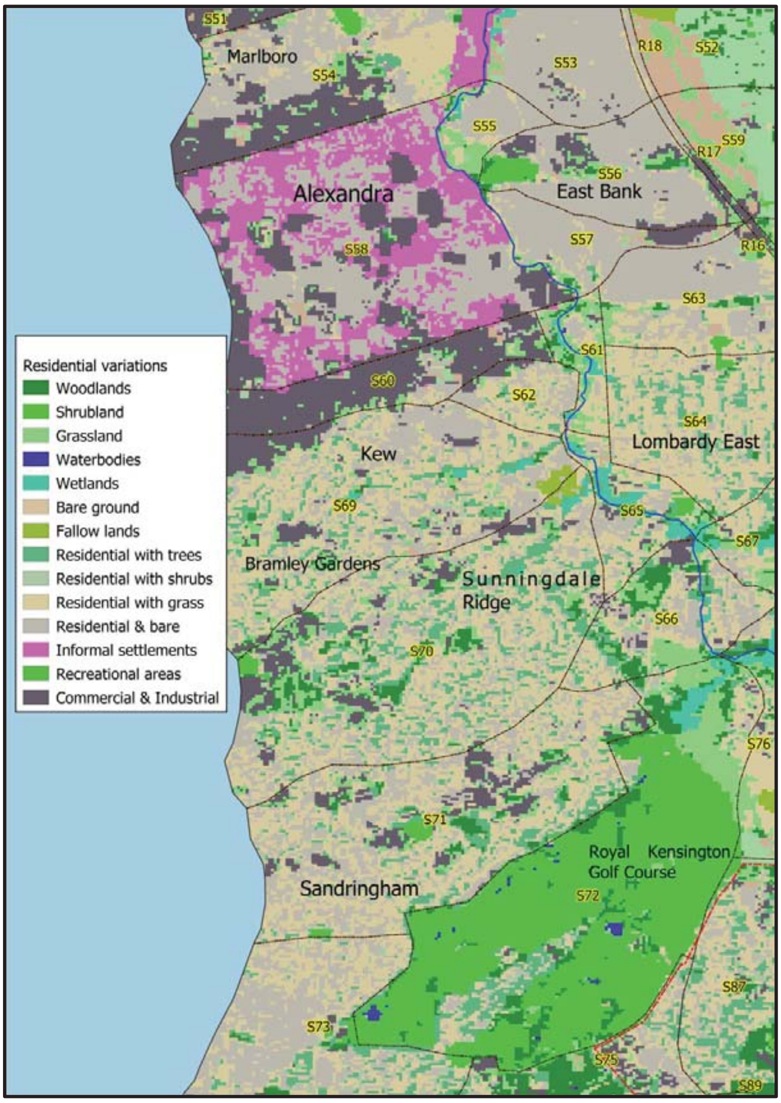


Figure 5-18: Spatial land cover information of the upper Jukskei River (Gauteng) (extracted from DEA and GTI, 2019)

NASA's Socioeconomic and Data Applications Centre (SEDAC) have produced a Global Man-made Impervious Surface (GMIS) data set using Landsat Version 1 (2010), as detailed in Brown de Colstoun *et al.* (2017). The data are an outcome of NASA's space shuttle programme, and it provides good spatial coverage for South Africa for the rapid determination of impermeable areas, especially for larger urban catchments. An example for the upper Jukskei River is shown in Figure 5-19. The data are based on the 2010 imagery and the user will need to check for new developments since that time (e.g. Google Earth or other sources).

The data can be accessed via the following link:

https://sedac.ciesin.columbia.edu/data/set/ulandsat-gmis-v1

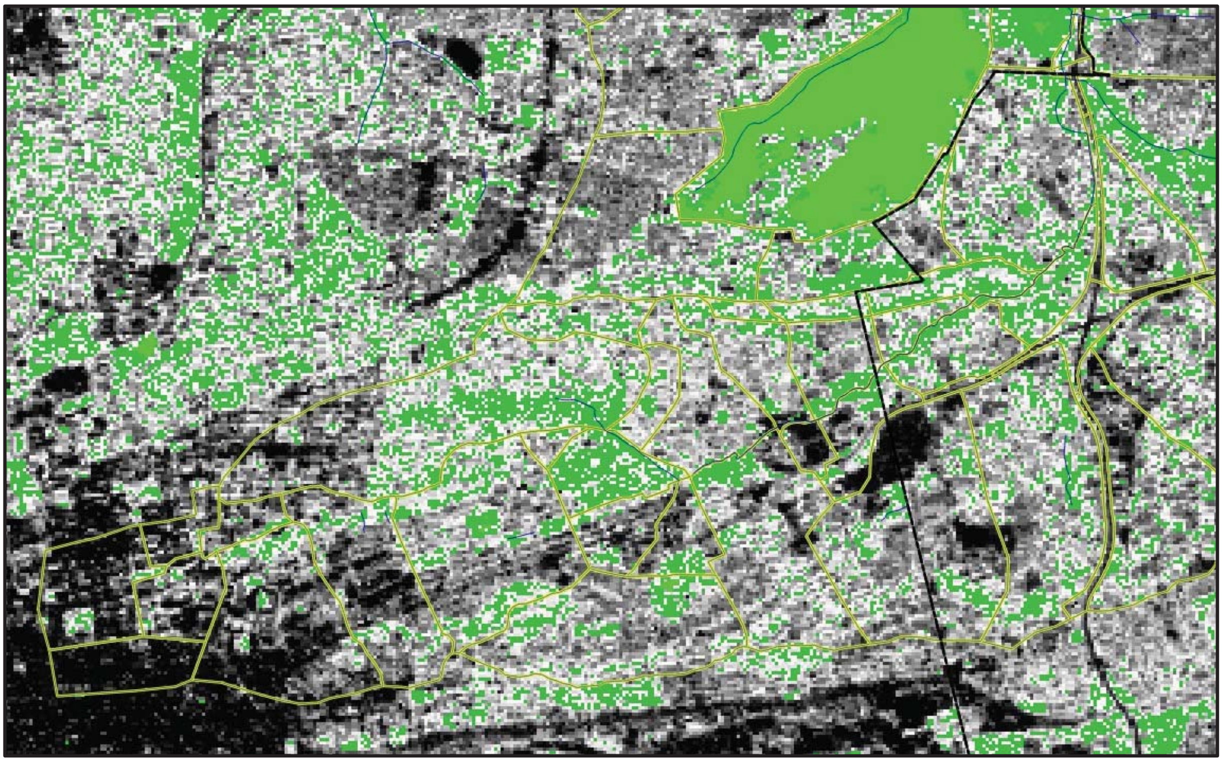


Figure 5-19: Spatial representation of impervious areas in the upper Jukskei River (NASA-SEDAC, 2010; (Brown de Colstoun *et al.*, 2017)

5.8 Soils

A number of sources of soils information may be used to derive soil related model parameters. Generally, the use of locally derived information should be prioritised. However, in the absence of detailed local soils information, generalised information derived at a national scale may be used.

5.8.1. Information derived from local soil surveys

Municipalities may retain relevant local soil information and should be approached accordingly. To ensure consistency in DFE baseline information, municipalities should consider adopting relevant data sets and developing these further with local information. Local information should include information from geotechnical reports for site developments, permeability studies for SuDS designs and specialist studies for Environmental Impact Assessments (EIAs). If collected over time, these data sets will provide useful references for DFE and CSM.

Two soil classification systems are used in South Africa. A binomial soil classification system, consisting of soil forms and series, was derived for South Africa by Macvicar *et al.* (1977) and is commonly referred to as the “red book” in industry. A taxonomic soil classification consisting of soil forms, families and series was derived by SCWG (1991) and is commonly referred to as the "blue book" in industry.

Both SCS soil groups (A, A/B, B, B/C, C, C/D and D) and the 11 typical soil textural classes have been linked to the binomial and taxonomic soil classification systems (see Chaper 5 in Schulze, 1995a; Section 6.12 in Smithers and Schulze, 1995a). The hydrological attributes (PWP, DUL and PO) associated with each of the 11 soil textural classes may be determined by Table 6.12.1 in Smithers and Schulze (1995a). The ACRU theory (Schulze, 1995a) and user manuals (Smithers *et al.*, 1996) can be downloaded at:

https://cwrr.ukzn.ac.za/resources/acru/

5.8.2. Information from underlying geology

Likely soil characteristics can also be determined from the underlying geology. For example, soils overlying the Halfway House granites are quite uniform, varying only in relation to their topographical position, hill crest, hill slope, and valley bottom. In general:

- Physical weathering predominates in the dry western part of the country where Weinert's N value exceeds 5 so most soils here can be expected to be relatively sandy and permeable
- In the eastern, wetter, parts of the country where N is less than 5, chemical weathering predominates:
    - Acid igneous rocks such as granite, and coarse grained sedimentary rocks such as sandstone weather to sandy permeable soils
    - Basic igneous rocks such as basalt and fine grained sedimentary rocks such as shales and mudrocks, weather to clayey impermeable soils

More detail can be obtained from:

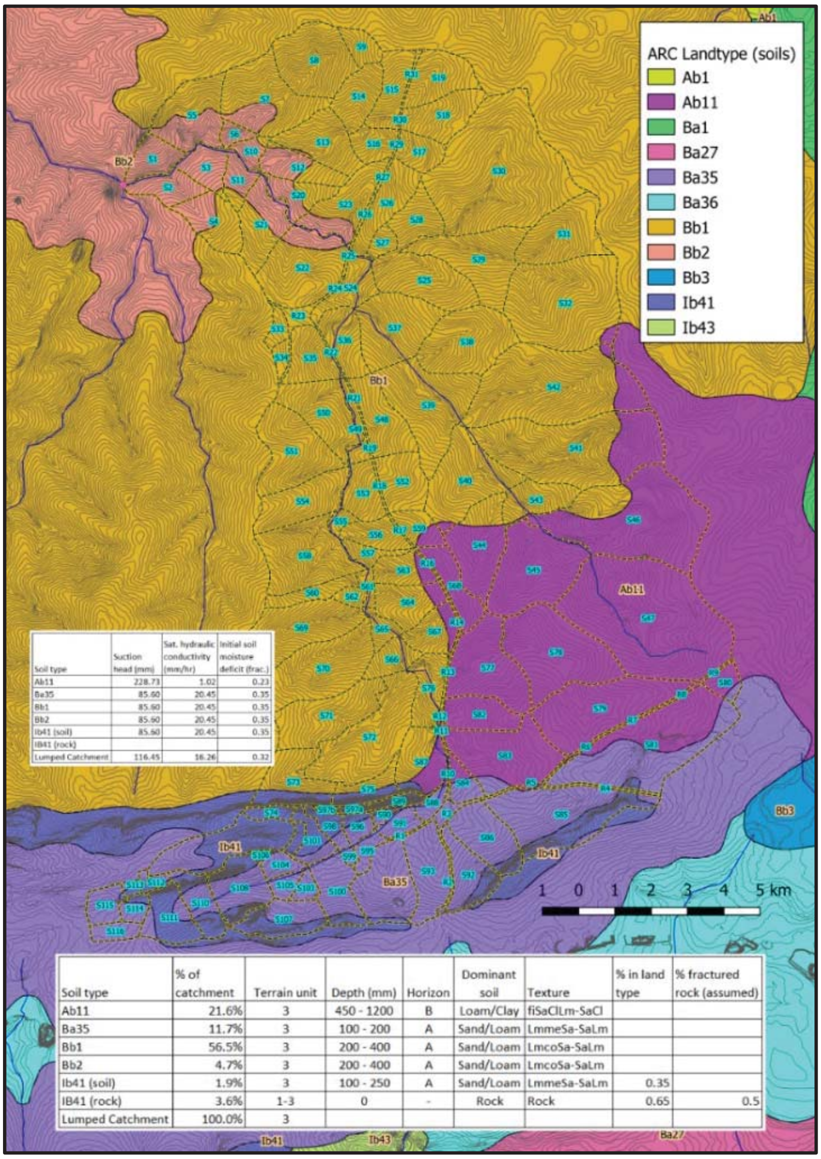
https://repository.up.ac.za/bitstream/handle/2263/30093/02chapters3-4.pdf?sequence=3&isAllowed=y

A 1:1 000 000 geological map in GIS compatible format is available free of charge from the Council for Geoscience at:

https://www.geoscience.org.za/index.php/publication/downloadable-material.

5.8.3. Hydrological attributes of soils derived from national Land Type maps

Land Type maps have been produced for South Africa (SIRI, 1987). A Land Type is a LC class with a defined macroclimate, terrain form and soil pattern, each display a marked degree of uniformity, and, at a national scale, there would be little advantage into further delineation into smaller and more uniform landscape entities for the purpose of agricultural potential determination. Land Types were mapped by superimposing climate maps for the region over a pedo-system map and identifying unique Land Type units. The data collected during the terrain, soil and climate survey phases were then compiled into an inventory. An example of a Land Type map is shown in Figure 5-20. They are described in terms of 27 broad soil forms and include general soil-water characteristics such as depth classes and percentage clay classes, as well as categories based on soil structure characteristics that can influence rainfall-runoff responses.



Soil type	Suction head (mm)	Sat. hydraulic conductivity (mm/hr)	Initial soil moisture deficit (frac.)
Ab11	228.73	1.02	0.23
Ba35	85.60	20.45	0.35
Bb1	85.60	20.45	0.35
Bb2	85.60	20.45	0.35
Ib41 (soil)	85.60	20.45	0.35
IB41 (rock)			
Lumped Catchment	116.45	16.26	0.32

Soil type	% of catchment	Terrain unit	Depth (mm)	Horizon	Dominant soil	Texture	% in land type	% fractured rock (assumed)
Ab11	21.6%	3	450 - 1200	B	Loam/Clay	fiSaClLm-SaCl		
Ba35	11.7%	3	100 - 200	A	Sand/Loam	LmmeSa-SaLm		
Bb1	56.5%	3	200 - 400	A	Sand/Loam	LmcoSa-SaLm		
Bb2	4.7%	3	200 - 400	A	Sand/Loam	LmcoSa-SaLm		
Ib41 (soil)	1.9%	3	100 - 250	A	Sand/Loam	LmmeSa-SaLm	0.35	
IB41 (rock)	3.6%	1-3	0	-	Rock	Rock	0.65	0.5
Lumped Catchment	100.0%	3						

Figure 5-20: Example of the spatial distribution of Land Types

Each land type includes up to five terrain units, for example, as depicted Figure 5-21 for Land Type Ba35. This allows more detailed analysis of soil series at a local scale.

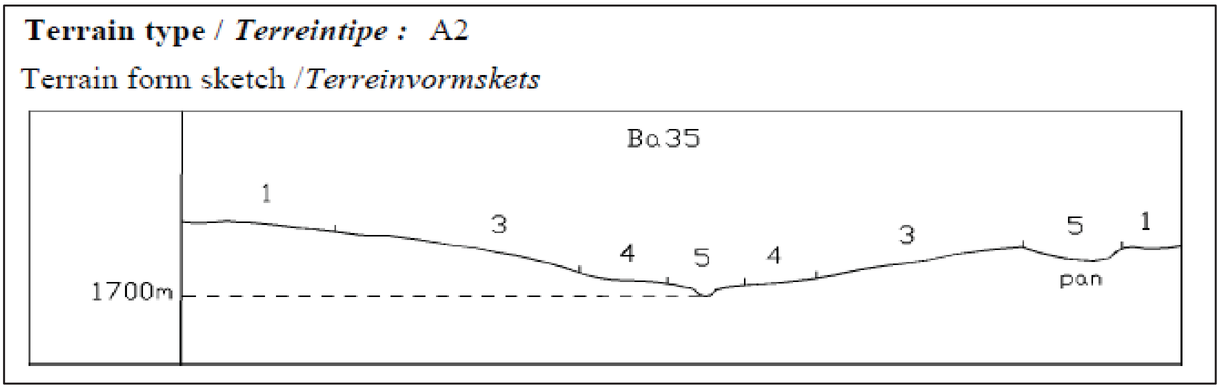


Figure 5-21:Example of the different terrain units used in the description of soils in the ARC Land Type Survey (SIRI, 1987)

Hydrological important attributes such as the thicknesses of the various soil horizons, soil water holding capacity at critical retention levels and the erodibility of the soil have been derived for all soil series identified in South Africa via the AUTOSOILS decision support tool (Pike and Schulze, 1995) and their area weighted attributes have been mapped at the level of Land Types. This information is available at:
https://catalogue.saeon.ac.za/records/10.15493/SARVA.BEEH.10000451

An alternative approach to determine hydrological important attributes of soils is to use the Soil Water Characteristics, which is a program included with the Soil Plant Air Water (SPAW: http://irrigationtoolbox.com/NEH/UserGuides/SPAW%20User%20Guide.pdf) water budgeting tool for farm fields, ponds, and inundated wetlands. The SPAW model performs daily hydrological water budgeting using the SCS Runoff Curve Number (CN) method. It is used to simulate soil water tension, conductivity and water holding capability based on the soil texture, with adjustments to account for gravel content, compaction, salinity, and organic matter. This software can be used to derive soil water characteristics for soil types identified from the ARC Land Type maps. The software provides rapid calculation of soil water characteristics such as porosity, field capacity, wilting point, and saturated hydraulic conductivity for either broad soil texture categories or for specific percentages of sand and clay in soils. It also provides the data necessary to calculate the rate of depletion of hydraulic conductivity with decreasing soil moisture. Importantly, the software allows for the application of soil density factors, related to bulk density, that will allow practitioners to test sensitivity of flood estimates to potential soil compaction in urban environments. The software is available at from the following link:

https://www.ars.usda.gov/research/software/download/?softwareid=492&modecode=80-42-05-10%20

5.9 Quality Control and Consistency Checks

All observed data must carefully be checked, and quality controlled for errors and inconsistencies. As indicated in Section 5.2, the use of observed data should be the first choice for undertaking a FFA when appropriate, consistent, good quality and long records of observations are available.

This section outlines quality and consistency checks for rainfall and runoff data which should always be performed when using observed data.

5.9.1. Check data quality flags in the observed data

Observed data which have been subjected to a quality control process should have data quality codes/flags/meta-data associated with the event in the data. An example of data flags from the AMS flow data downloaded from the DWS site (see Section 5.4) is shown in Table 5-1. The full range of data quality codes used by DWS is summarised in Table 15-1 in Appendix 5.

Table 5-1: Examples of data flags in AMS flow data from DWS

Hydro Year	Date	Time	Level (m)	Flow (m^3/s)	Codes
1955	19550302	22:00	0.61	13.854	
1956	19551224	23:54	0.43	7.034	E
……..					
2015	20150406	02:24	0.473	8.491	
2016	20160316	12:36	0.366	5.065	
2017	20170222	13:24	0.568	12.093	M
2018	20180323	13:48	0.598	13.342	M
2019	20190423	16:00	0.537	10.852	
2020	20200426	23:00	0.432	7.099	Q
2021	20210131	10:24	0.502	9.53	M
Explanation of codes:					
> – Greater than					
A – Above rating					
E – Estimated data					
M – Missing data					
Q – Data not audited					

5.9.2. Check for flow-gauging weir rating table exceedances.

An example of where the rating table has been exceeded by a measured river stage (depth) is shown in Figure 5-14. This is a frequent occurrence at many of the DWS flow-gauging weirs, as shown in Figure 5-14 as well. If two or more values in the AMS have the same value, then this is frequently the result of the rating table being exceeded and further evaluation of the data should be performed.

Hence, where possible, it is recommended that the flow-gauging weir is re-rated, or as highlighted in Section 5.4.2, rating tables/curves should be extended to include all the recorded levels and no data in the AMS should be excluded when estimating the design flood. However, the individual stage extensions (H_E) should be limited to a maximum of 20%, i.e. $H_E \leq 1.2\ H$, while adhering to the $Q_{DE} \leq 0.05\ Q_{Dxi}$ limit.

5.9.3. Visual inspection of peak discharge AMS

The graphical plotting positions outlined in Chapter 6, Section 6.3, should be used to plot the AMS data as shown, for example, in Figure 5-22.

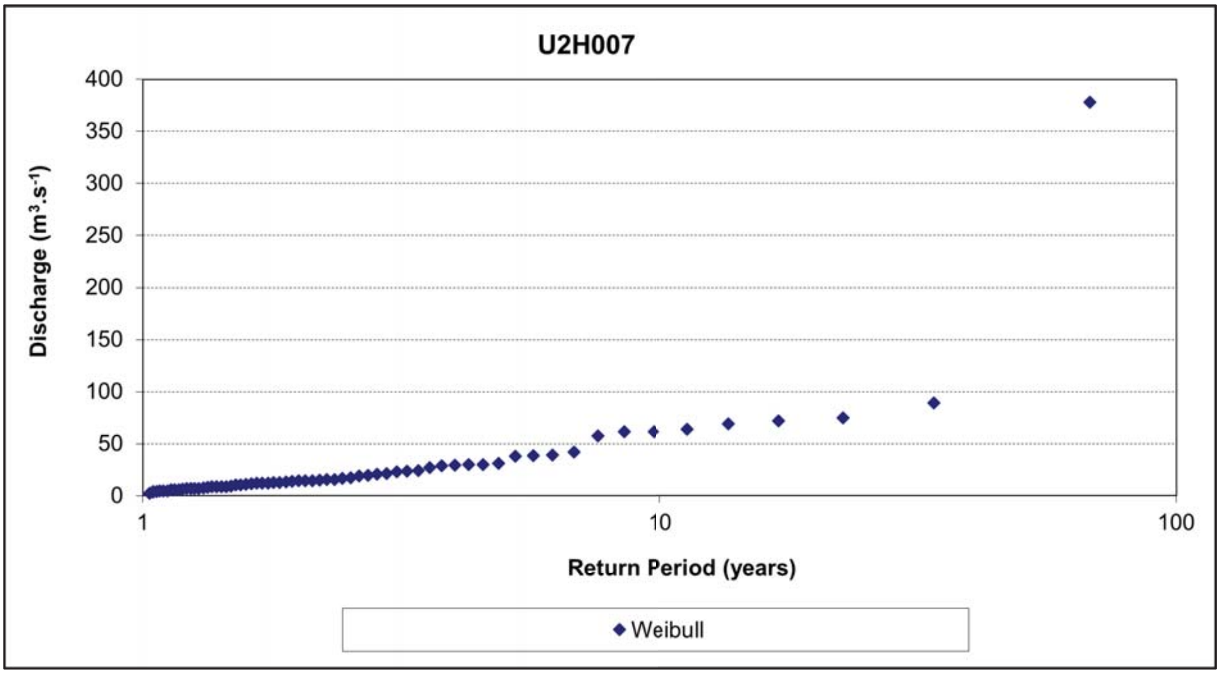


Figure 5-22: Visual inspection of AMS for Gauging Weir U2H007 plotted using the Weibull plotting position

From this example, it is evident that there is a peak discharge of 378 m^3/s in the AMS which is not consistent with the other values in the AMS. This value should then be validated by checking the flow conditions at nearby flow-gauging stations for the same day and with rainfall recorded at rain gauges located in the upstream catchment. If surrounding flow and rain gauges recorded abnormally high values for the same day, then the outlier is deemed to be valid and should be included in FFA.

As outlined in Section 5.4.4, anecdotal evidence of flood conditions can also be used to corroborate outliers.

5.9.4. Check for data consistency

A time series plot of rainfall and runoff volumes or depths, for example as shown in Figure 5-23, should be performed.

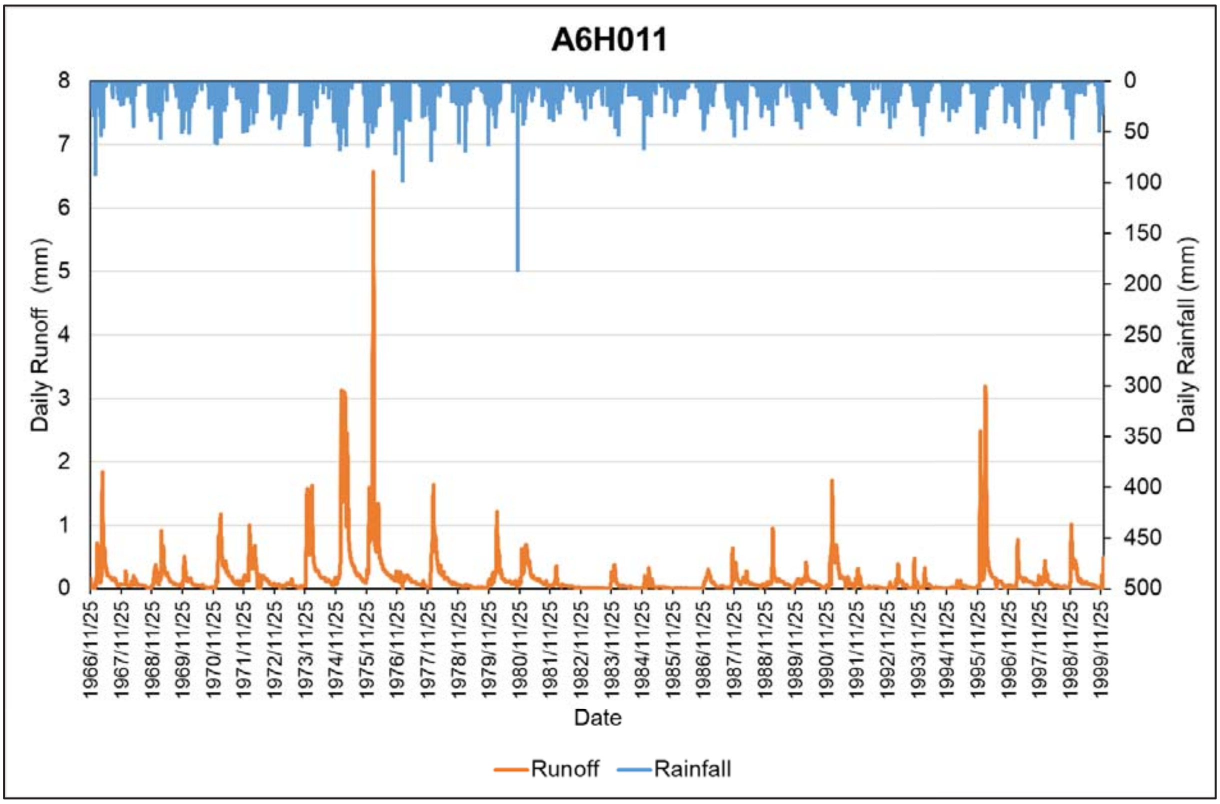


Figure 5-23: Rainfall and runoff time series data for Catchment A6H011

From Figure 5-23 it is clear that there are inconsistencies between the rainfall and runoff on some days with large observed rainfall events, which result in relatively small runoff events. Similarly, some days with large runoff events do not correspond with a large rainfall event. The representativeness of the selected rainfall station(s) for the catchment may be the cause of the apparent inconsistencies in the data and this and other possible causes of the inconsistency in observed rainfall and runoff data from the catchment should be investigated.

Double mass plots of accumulated data can be used to check for and identify errors and changes in the consistency in observed data. An example of this is shown in Figure 5-24.

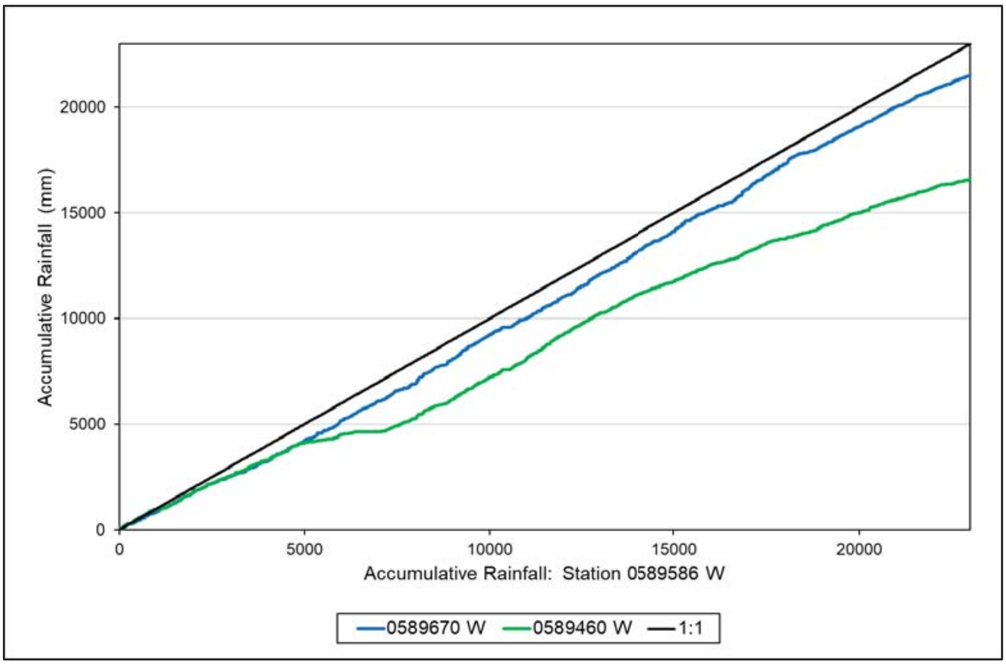


Figure 5-24: Accumulative rainfall recorded at rain gauges 0589670W, 0589460W and 0589586W

From Figure 5-24, it is clear that the rainfall recorded at Rain Gauges 0589586W and 0589670W are relatively consistent, but less rainfall was recorded at Rain Gauge 0589670W. Similarly, it is evident that the rainfall recorded at Rain Gauges 0589670W and 0589660W are similar and consistent for a period, after which, there is a change in the trend of the rainfall recorded at the two sites. In this example, the changes in the rainfall recorded at Rain Gauge 0589460W need careful investigation and assessment before it can be confidently used in any hydrological analyses or flood study.

5.10 Catchment Response Time

In the application of event-based deterministic DFE methods, it is acknowledged that both the spatial and temporal distribution of runoff, as well as the critical duration of rainfall, are influenced by the catchment response time. The catchment response time is normally expressed as a single time parameter, e.g. time of concentration (T_C), lag time (T_L) and/or time to peak (T_P). Therefore, apart from the average areal (catchment) design rainfall, catchment response time parameters are also regarded as fundamental input to all event-based DFE methods in ungauged catchments. Thus, errors in estimated catchment response time will directly impact on estimated peak discharges.

Note: In many of the T_C and/or T_L methods/equations reported in the literature/used in practice, migration between dimensional systems and what seems to be a standard Manning's roughness coefficient (n) value, which is in fact a special-case Manning's roughness coefficient, are encountered. Hence, please pay special attention to the dimensional units and type of roughness coefficients recommended when using a specific method.

5.11 Time of concentration

Multiple definitions are used in the literature to define Time of Concentration (T_C). The most commonly used conceptual, physically-based definition of T_C is defined as the time required for runoff, as a result of effective rainfall with a uniform spatial and temporal distribution over a catchment, to contribute to the peak discharge at the catchment outlet or, in other words, the time required for a 'water particle' to travel from the catchment boundary along the longest watercourse to the catchment outlet (Kirpich, 1940; McCuen, 1984; 2005; USDA-NRCS, 2010; SANRAL, 2013).

The time of concentration (T_C) is the most commonly used time parameter in urban hydrology. A clear distinction needs to be made between overland and channel flow conditions. Flow length criteria, i.e. overland flow distances (L_O) associated with specific slopes (S_O), are normally used as a limiting variable to quantify overland flow conditions, but flow retardance factors (i_p), Manning's overland roughness parameters (n) and overland conveyance factors (ϕ) are also used (Viessman *et al.*, 1989; Seybert, 2006; USDA-NRCS, 2010). In terms of overland flow conditions, the South African Drainage Manual (SANRAL, 2013) recommends the use of Eq. 5-8 (Kerby), while Eq. 5-9 (NRCS kinematic wave) is also frequently used in practice. In terms of channel flow in natural water courses, SANRAL (2013) recommends Eq. 5-13 (USBR), which is essentially a modified version of the Kirpich method (Kirpich, 1940). Thus in this best practice guideline, the use of both Eq. 5-8 and Eq. 5-13 is recommended, while the NRCS velocity and/or segmental methods (Eqs. 5-10 to 5-12) should be used in the case of man-made (constructed) flow paths where the surface roughness is easier to define.

The hydraulic and/or empirical methods commonly used in South Africa to estimate the T_C are discussed in the following paragraphs:

(a) **Kerby's method:** This empirical method (Eq. 5-8) is commonly used to estimate the T_C both as mixed sheet and/or shallow concentrated overland flow in the upper reaches of small, flat catchments. It was developed by Kerby (1959; cited by Seybert, 2006) and is based on the drainage design charts developed by (Hathaway, 1945; cited by Seybert, 2006). Therefore, it is sometimes referred to as the Kerby-Hathaway method. As highlighted above, SANRAL (2013) also recommends the use of Eq. 5-8 for overland flow in South Africa. McCuen (1984) highlighted that this method was developed and calibrated for catchments in the USA with areas less than 4 ha, average slopes of less than 1% and Manning's overland roughness parameters (n) varying between 0.02 and 0.8. In addition, the length of the flow path is a straight-line distance from the most distant point on the catchment boundary to the start of a fingertip tributary (well-defined watercourse) and is measured parallel to the slope. The flow path length must also be limited to ± 100 m.

$$T_{C1} = 1.4394\left(\frac{nL_O}{\sqrt{S_O}}\right)^{0.467} \qquad \text{Eq. 5-8}$$

(b) **NRCS kinematic wave method:** This hydraulic method (Eq. 5-9) is limited to sheet overland flow. In general, overland flow can be classified as Hortonian and/or saturated overland flow. Hortonian overland flow is produced when the rainfall intensity exceeds the infiltration capacity of soil until saturation is reached, while saturated overland flow is produced by rainfall on already saturated areas near main watercourses and valleys. In both cases, the effective rainfall becomes available as direct runoff that flows over the land surface towards a watercourse/channel in either a quasi-laminar (sheet flow) or anastomosing (shallow concentrated flow) state. In the case of sheet flow, which applies to Eq. 5-9, the flow depths are of the same order of magnitude as the surface resistance (roughness parameters). At some point in the upper reaches of a catchment, sheet flow will transition to shallow concentrated flow characterised by well-defined gullies and flow depths exceeding the flow resistance heights. The transition point between sheet and concentrated flow is characterised by the presence of continuous surface depression stores collecting sheet flow from radial directions. These depression stores only contribute to runoff when their storage capacities are exceeded. The spilled water flows then downstream in a shallow, concentrated fashion towards the main watercourses (Seybert, 2006). Equation 5-9 was originally developed to avoid the iteratively use of the original Kinematic wave method. It is based on a power-law relationship between design rainfall intensity and duration (Welle and Woodward, 1986; cited by Gericke and Smithers, 2014). Gericke and Smithers (2016b) compared Eq. 5-9 and five other overland flow T_C equations to the Kerby equation (Eq. 5-8). In considering the overall average consistency measures compared to the Kerby equation (Eq. 5-8), the NRCS kinematic wave equation (Eq. 5-9) provided relatively the smallest bias (< 10%), with a mean error ≤ 1 minute.

$$T_{C2} = \frac{5.476}{P_2^{\,0.5}}\left(\frac{nL_O}{\sqrt{S_O}}\right)^{0.8} \qquad \text{Eq. 5-9}$$

(c) **NRCS velocity method:** This hydraulic method is commonly used to estimate T_C both as shallow concentrated overland and/or channel flow (Seybert, 2006). Either Eq. 5-10 or Eq. 5-11 can be used to express the T_C for concentrated overland or channel flow. In the case of main watercourse/channel flow, this method is referred to as the NRCS segmental method, which divides the flow path into segments of reasonably uniform hydraulic characteristics. Separate travel time calculations are performed for each segment based on either Eq. 5-10 or Eq. 5-11, while the total T_C is computed using Eq. 5-12 (USDA-NRCS, 2010):

$$T_{C3i} = 0.0167\left(\frac{nL_{O,CH}}{R^{0.667}\sqrt{S_{O,CH}}}\right) \quad \text{Eq. 5-10}$$

$$T_{C3i} = 0.0167\left(\frac{L_{O,CH}}{18\log\left(\frac{12R}{k_s}\right)\sqrt{RS_{O,CH}}}\right) \quad \text{Eq. 5-11}$$

$$T_{C3} = \sum_{i=1}^{N} T_{Ci} \quad \text{Eq. 5-12}$$

(d) **USBR method:** Equation 5-13 was proposed by the (USBR, 1973) to be used as a standard empirical method to estimate the T_C in hydrological designs, especially culvert designs based on the California Culvert Practice (CCP, 1955; cited by Li and Chibber, 2008). As highlighted above, Eq. 5-13 is recommended by SANRAL (2013) for use in South Africa for defined, natural watercourses/channels. It is also used in conjunction with Eq. 5-8 which estimates overland flow time, to estimate the total travel time (overland plus channel flow). Van Der Spuy and Rademeyer (2021) highlighted that Eq. 5-13 tends to result in estimates that are either too high or too low and recommend the use of a correction factor (τ) as shown in Eq. 5-14 and listed in Table 5-2

$$T_{C4} = \left(\frac{0.87L_{CH}^{2}}{1000S_{CH}}\right)^{0.385} \quad \text{Eq. 5-13}$$

$$T_{C4a} = \tau\left(\frac{0.87L_{CH}^{2}}{1000S_{CH}}\right)^{0.385} \quad \text{Eq. 5-14}$$

where:
- $T_{C1,\,2}$ = overland time of concentration (minutes),
- T_{C3} = overland/channel flow time of concentration computed using the NRCS method (minutes),
- T_{C3i} = overland/channel flow time of concentration of segment i (minutes),
- $T_{C4,4a}$ = channel flow time of concentration (hours),
- i = critical rainfall intensity of duration T_C (mm/h),
- k_s = Calibrated surface (Nikuradze) roughness parameter (m),

$L_{O,\,CH}$	= length of flow path, either overland or channel flow (m),
n	= Manning's roughness parameter for overland flow,
P_2	= two-year return period 24-hour design rainfall depth (mm),
R	= hydraulic radius which equals the flow depth (m),
$S_{O,\,CH}$	= average overland or channel slope (m/m), and
τ	= correction factor.

Table 5-2: Correction factors (τ) for T_C (Van Der Spuy and Rademeyer, 2021)

Area (A, km²)	Correction factor (τ)
< 1	2
1-00	2-0.5log$A$
100-5 000	1
5 000-100 000	2.42-0.385log$A$
> 100 000	0.5

5.11.1. Lag time

Conceptually, Lag Time (T_L) is generally defined as the time between the centroid of effective rainfall and the peak discharge of the resultant direct runoff hydrograph. Computationally, T_L can be estimated as a weighted T_C value when, for a given storm, the catchment is divided into sub-areas and the travel times from the centroid of each sub-area to the catchment outlet are established by the relationship expressed in Eq. 5-15 and shown in Figure 5-25.

$$T_L = \frac{\sum\left(A_i Q_i T_{Ti}\right)}{\sum\left(A_i Q_i\right)} \qquad \text{Eq. 5-15}$$

where

T_L	= lag time (hours),
A_i	= incremental catchment area/sub-area (km²),
Q_i	= incremental runoff from A_i (mm), and
T_{Ti}	= travel time from the centroid of A_i to catchment outlet (hours).

Given that only DFE methods suitable for application in urban hydrology are listed in Chapters 7 and 8, and that T_L is only applicable to the SCS method, only the two methods listed below are recommended for possible use.

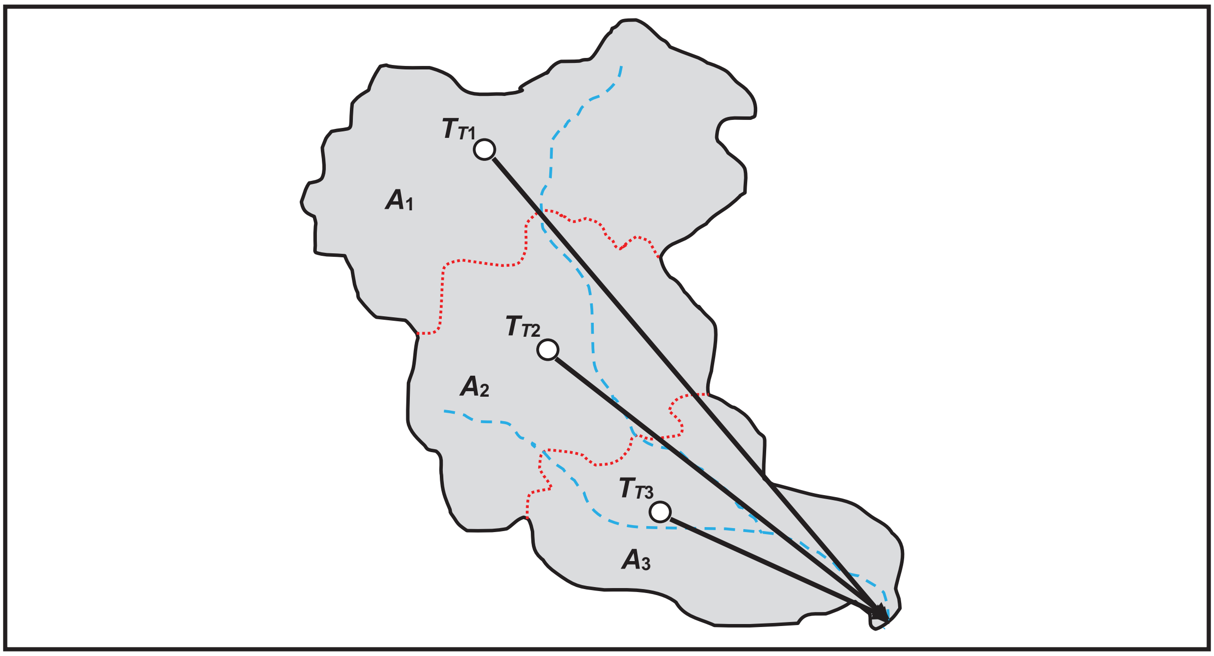


Figure 5-25: Conceptual travel time from the centroid of each sub-area to the catchment outlet (USDA-NRCS, 2010)

(a) **SCS lag method:** This method was developed by the USDA SCS in 1962 (Reich, 1962) to estimate T_C where mixed overland flow conditions in catchment areas up to 8 km² exists. However, using the relationship of $T_L = 0.6T_C$, Eq. 5-16 can also be used to estimate T_L in catchment areas up to 16 km² (McCuen, 2005).

$$T_{L1} = \frac{L_H^{\ 0.8}\left[\dfrac{25\,400}{CN} - 228.6\right]^{0.7}}{168.862\,S^{0.5}} \qquad \text{Eq. 5-16}$$

(b) **Schmidt-Schulze (SCS-SA) method:** Schmidt and Schulze (1984a) estimated T_L from observed rainfall and streamflow data in 12 agricultural catchments in South Africa and the USA with catchment areas smaller than 3.5 km² by using three different methods to develop Eq. 5-17. This equation is used in preference to the original SCS lag method Eq. 5-16 in South Africa, especially when stormflow response includes both surface and subsurface runoff as frequently encountered in areas of high MAP or on natural catchments with good land cover (Schulze *et al.*, 1992).

$$T_{L2} = \frac{A^{0.35} MAP^{1.10}}{41.67\,S^{0.3} i_{30}^{\ 0.87}} \qquad \text{Eq. 5-17}$$

where $T_{L1\text{-}2}$ = lag time (hours),

A = catchment area (km²),

CN = runoff curve number,

i_{30} = 2-year return period 30-minute rainfall intensity (mm/h),

L_H = hydraulic length of catchment (km),

MAP = Mean Annual Precipitation (mm), and

S = average catchment slope (m/m).

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6. PROBABILISTIC RAINFALL AND FLOOD FREQUENCY ANALYSIS

6.1 Chapter Overview

Where long, good quality records of observed rainfall or flow records are available, the data should be used to conduct either a rainfall frequency analysis (RFA) or FFA to estimate the design events. In the case of design rainfall estimation, the General Extreme Value (GEV) distribution is recommended, while both the Log-Pearson Type 3 (LP3) and GEV distributions are generally suitable for DFE in South Africa. However, it should be noted that in a recent study, Calitz and Smithers (2020) established that the Generalised Pareto distribution (GPA) fitted to the annual maximum series (AMS) by Linear Moments (L-Moments), is the best general distribution for FFA in South Africa, but this recommendation, has not yet been adopted in practice. It is recommended that available software be used to fit the selected probability distribution(s) to the data (see Section 6.8) or the simple frequency factor method be used to manually estimate the design events.

It is also recommended that the AMS extracted for each hydrological year be used in frequency analyses, as this basic requirement ensures that the events in the series are independent and, in general, there is little difference in design events between frequency analyses using the AMS or Partial Duration Series (PDS) for return periods greater than about 10 years.

In addition to fitting an appropriate theoretical probability distributions to the data, it is highly recommended that the Weibull Plotting Position (PP) (see Section 6.3) be used to plot the AMS, as it provides a visual overview of the data and enable the identification of any anomalies in the data, while it is also regarded as more conservative and provide unbiased estimates of return periods of all distributions (Stedinger *et al.*, 1993). It is also recommended that the design events computed using the selected probability distribution(s), be included in the plot as this provides a visual assessment of the suitability of the selected distribution. Given the general use of the Cunnane PP in South Africa, and the promising results demonstrated by the newly developed Z-set PP (Van der Spuy and Du Plessis, 2022a), these PPs can be used as a valuable addition to the Weibull PP.

6.2 Introduction

By conducting a RFA or FFA, the results of the analyses are expressed as the relationship between the rainfall or peak discharge at a site and the probability that this rainfall or discharge value will be exceeded in any one year (Alexander, 2001). This is called the Annual Exceedance Probability (AEP) and is expressed as the 1: *N*-year event, e.g. 1: 20 years. From Eq. 6-1 this means that the estimated rainfall/discharge has a 5% probability of being exceeded in any given year. The recurrence interval or return period (T), which is the reciprocal of the AEP, is thus the long-term average interval between events that equal or exceed a given magnitude or severity in any given year (Eq. 6-1). Equation 6-2 can be used to estimate the AEP for a specified number of years or design life. If the AEP is defined by legislation ruling, e.g. government policy, the AEP is pre-determined, and the expected design life (N) could be found by making use of Eq. 6-3.

$$T = \frac{1}{P} \qquad \text{Eq. 6-1}$$

$$P_N = 1 - \left(1 - \frac{1}{T}\right)^N \qquad \text{Eq. 6-2}$$

$$N = \frac{\log\left[1 - P(x > or = x)N\right]}{\log\left[\frac{(T-1)}{T}\right]} \qquad \text{Eq. 6-3}$$

where:
- T = recurrence interval or return period (years),
- P = probability that a rainfall/flood event of magnitude equal or larger than the rainfall/flood with return period T will occur in any given year,
- P_N = probability that a rainfall/flood event of magnitude equal or larger than the rainfall/flood with return period T will occur in a specific period (N), and
- N = expected design life (years).

Worked examples applicable to the use of Eqs. 6-1 to 6-3 are included in Appendix 6B (Section 16.2).

The procedures for direct probabilistic RFA of observed rainfall or FFA of runoff events involve the: (i) identification and summarisation of the observed rainfall or flood peak data, (ii) estimation of parameters to enable the probabilistic fitting of data, and (iii) selection and fitting of appropriate theoretical probability distribution(s) to the data.

The assumption is made that the distribution of the relatively short sample of available observed data is the best estimate of the distribution of the population of all events that have occurred at the site. The selected distribution fitted to the sample of observed data is then assumed to approximate the distribution of the entire population of events. A critical assumption made in probabilistic analyses is that the data are stationary, i.e. there are no changes to the statistics of the data over time. Hence, both the impact of changes in the catchment and potential changes in climate which impact on runoff need to be considered.

The procedures for performing probabilistic analysis to estimate both design rainfall and design floods are detailed in standard hydrology texts (e.g. Chow *et al.*, 1988; Stedinger *et al.*, 1993), and a brief overview of the methodology is given below, while the detailed calculation procedures are included in Appendix 6A (Section 16.1).

6.3 Summarisation of Data

In most hydrological analyses, only the largest rainfall for the selected event duration or daily peak discharge value in each hydrological year is identified, i.e. the AMS. As an alternative, the Partial Duration Series (PDS) is also sometimes used; however, the use of the PDS is very limited in DFE/RFA/FFA and the probabilistic curve fitting is limited to the one-tailed exponential distribution (Alexander, 2001). In considering the AMS approach, the other rainfall/flood peak values occurring during the hydrological year are thus ignored even if they are larger than the maximum values in other years. For example, if records have been kept for 30 years, then 30 annual maximum are identified and used in the analyses. In the PDS, the selection procedure entails that some of the monthly/annual maximum peaks may be excluded in the series using a threshold exceedance value (Kite, 1988). In cases where the number of ranked peak events is equal to the number of data years, then the PDS is referred to as an Annual Exceedance Series (AES), e.g. 30 years of record will result in the 30 highest peak events being used, irrespective of the year of occurrence.

It is recommended that the AMS extracted for each hydrological year be used in frequency analyses, as this basic requirement ensures that the events in the series are independent and, in general, there is little difference in design events between frequency analyses using AMS or PDS for return periods greater than about 10 years.

The summarisation of observed rainfall or flood peak data includes the ranking of either the AMS or PDS in a descending order of magnitude, after which, an exceedance probability is assigned to the plotted values (Schulze *et al.*, 1995; Chadwick and Morfett, 2004). A plotting position (PP) equation (Eq. 6-4) is commonly used in South Africa to assign an exceedance probability to values in an AMS. It assumes that if (n) values are distributed uniformly between 0% and 100% probability, then there must be (n + 1) intervals, (n - 1) intervals between the data points and two intervals at the end (Chow *et al.*, 1988; SANRAL, 2013). The Cunnane PP is generally used in South Africa, while it is also being recommended by DWS (Van der Spuy and Rademeyer, 2021). However, the Weibull PP is regarded as more conservative and provide unbiased estimates of return periods of all distributions (Stedinger *et al.*, 1993). Hence, the preferred use of the Weibull PP is recommended.

$$T = \frac{n+a}{m-b} \qquad \text{Eq. 6-4}$$

where: T = return period (years),

a = constant Table 6-1,

b = constant Table 6-1,

m = number, in descending order, of the ranked events (peak flows), and

n = number of observations/record length (years).

Table 6-1: Common plotting position input parameters (SANRAL, 2013)

Method	Plotting position	Theoretical probability distribution
Beard (1962)	a = 0.40 and b = 0.30	Pearson Type 3
Blom (1958)	a = 0.25 and b = 0.375	Normal
Cunnane (1978)	a = 0.20 and b = 0.40	General purpose
Greenwood (1979)	a = 0.00 and b = 0.35	GEV and Wakeby
Gringorten (1963)	a = 0.12 and b = 0.44	Extreme Value Type 1, GEV and Exponential
Weibull (1939)	a = 1.00 and b = 0.00	Normal and Pearson Type 3

In a recent study, Van der Spuy and Du Plessis (2022a) developed a new *Z*-set PP approach, which provided a much improved/realistic plot of the theoretical probability distributions and portrayal of outliers, while it is also more effective in eliminating the assignment of noticeably different probabilities to design events of similar magnitude. However, the *Z*-set PP has not yet been tested and adopted in practice; hence, it is recommended that the Z-set PP be used as a valuable addition to the Weibull and Cunnane PPs, respectively. The calculation of the Z-set plotting position is described in Appendix 6C (Section 16.3).

The PP technique, as typically applied at a single rainfall station or flow-gauging site, is be summarised as follows:

(a) Direct probabilistic analyses must be conducted at each site in order to summarise the data, estimate parameters, and select and fit appropriate theoretical probability distributions.

(b) The AMS data can be summarised by ranking the data in a descending order of magnitude, i.e. extract the AMS values at each station, i.e. maximum daily rainfall or peak discharge in each hydrological year.

(c) Assign an order number (ranking) to each of the data points, starting at the largest rainfall/flood peak value with m = 1 to m = n for the smallest value in the data series.

(d) This ranking order approach is preferred, since it relates directly to an AEP or return period, which directly relates to risk. If the data are sorted in ascending order (noted in some literature), the probability value assigned to a data point, indicates the probability of non-exceedance.

(e) Apply Eq. 6-4 to assign a return period (*T*) to each data point. Given that *T* is the reciprocal of the AEP, the probability of exceedance is reflected.

(f) Plot the data on logarithmic scales.

(g) Repeat the process for different rainfall durations (e.g. daily or sub-daily in the case of rainfall).

(h) Estimate the design rainfall or floods (based on a specific probability distribution) at various return periods, e.g. 2, 5, 10, 20, 50, 100 and 200 years.

If the fitted distributions are added to the plot following above steps, then the goodness-of-fit (GOF) to the plotted data can be assessed.

6.4 Parameter Estimation Methods

Parameter estimation methods available for fitting theoretical probability distributions to observed rainfall or flood peak values include Linear Moments (LM), Maximum Likelihood (ML), Method of Moments (MM), Probable Weighted Moments (PWM) and Method of Least-squares (MLS) (Yevjevich, 1972; Chow *et al.*, 1988; Kite, 1988; Stedinger *et al.*, 1993). All these methods will, within limits, estimate the parameters of a theoretical probability distribution from a particular data sample (Kite, 1988). LM estimators are used extensively internationally as a standard procedure for RFA/FFA, screening of discordant data and testing clusters for homogeneity (Smithers and Schulze, 2000a)

Some caution and criticism of the use of LM is also evident in the literature. Alexander (2001) cautions that LMs are not very sensitive to outliers and emphasised that both low and high outliers are important characteristics of the flood peak maxima. The suppression of the effect of outliers could result in unrealistic estimates of longer return period values. Therefore, further investigation of LM for possible general use in South Africa is necessary (Smithers, 2012). Alexander (2001) recommends either MM or PWM estimators for probability distribution fitting in South Africa either at a single site or when a regional approach is adopted. In this best practice guideline, the use of MM is recommended, while the use of either PWM and/or LM should also be considered in cases, e.g. presence of outliers, where deemed as required.

6.5 Theoretical Probability Distributions

The fitting of an appropriate theoretical probability distribution to a data set provides a compact and smoothed representation of the frequency distribution revealed by the limited information available and enables the systematic extrapolation to frequencies beyond the data set range (Smithers and Schulze, 2000a).

The different theoretical probability distributions and their associated calculation procedures are included in Appendix 6A (Section 16.1).

6.6 Recommended Probability Distributions

The GEV fitted by LM (Hosking, 1990) is recommended as the best distribution to estimate both short and long duration design rainfall in South Africa (Smithers, 1996; Smithers and Schulze, 2000b; 2000a). Alexander (2001) recommended the use of the LP3 probability distribution for DFE in South Africa. Van der Spuy and Rademeyer (2021) recommended the Log-Normal (LN), LP3 and GEV probability distributions for RFA and FFA at a single site in South Africa, while Görgens (2007) and Van der Spuy and Du Plessis (2022b) both regard the LP3 and GEV probability distributions as the most appropriate to be used locally.

Calitz and Smithers (2020) demonstrated that the Generalised Pareto distribution (GPA) fitted to the AMS by LM is the best general distribution for FFA in South Africa. According to Van der Spuy and Du Plessis (2022b), the GEV seems to be more stable than the LP3 or LN in predicting lower AEPs, irrespective of the record length under consideration, while the GEV is also regarded as less sensitive to outliers than the LP3. Therefore, until the GPA/LM has been adopted in practice, the following approach is recommended:

(a) The GEV distribution should be used for RFA in South Africa.

(b) The LP3 and GEV distributions should both be considered for DFE/FFA in South Africa. The GEV is the preferred choice at the lower probabilities given that it performs poorly above an AEP of 50%. Hence, the LP3 should then be used; however, in the lower AEP ranges characterised by log-transformed data with a positive skewness, the LP3 should not be considered.

(c) Investigate and promote the general application of the GPA/LM as a more suitable or alternative probability distribution.

(d) In all cases, visually compare the GOF between the various probability distributions and the observed AMS data points. Carefully consider all contributing factors before choosing a distribution, especially if a single probability distribution does not fit all the data points well.

6.7 Single Site and Regional Approaches

A single site approach requires that each rainfall station or flow-gauging weir within the relevant catchment or located at the relevant flow-gauging site at the catchment outlet be investigated to determine the record length, data quality (consistency, errors, missing data, and outliers), and topographical position (Smithers and Schulze, 2000a; 2000b). To develop either a rainfall depth-duration-frequency (DDF) or flood-magnitude-frequency relationship at every single site, the steps as explained above in Sections 6.3 to 6.5, need to be followed.

Regional flood approaches combine information for a region and are more applicable to large, rural catchments. However, the empirical RMF method is included in this best practice guideline (see Section 7.5) to provide an indication of the maximum observed floods to date in the region. In terms of design rainfall, the RLMA&SI method (see Section 5.3.3), is a typical example of a regional approach for design rainfall estimation in South Africa. In principal, the rationale of a regional approach is to supplement the relatively short observed records by the incorporation of spatial randomness using data from different sites in a region (Schaefer, 1990; Nandakumar et al., 1995). This allows for the estimation of design events where no information exists (ungauged) at a site (Pilon and Adamowski, 1992). However, care must be exercised to ensure that a regional approach is not applied outside of the region where the method was developed, nor outside of the range of observations used to develop the method (Cordery and Pilgrim, 2000). There is a lot of evidence that regional approaches are generally more accurate and consistent than the application of an at-site analysis (e.g. Lettenmaier, 1985; Potter, 1987; cited by Cunnane, 1989; Alexander, 1990; Hosking and Wallis, 1997; Cordery and Pilgrim, 2000).

6.8 Software to Fit Probability Distributions to Data

The following software can be used to fit theoretical probability distributions to data:

(a) Design Flood Estimation Tool developed by Gericke and du Plessis (2013). Available at: https://data.waterresearchobservatory.org/

(b) RMC-BestFit developed by the Unites States Army Corps of Engineers. Available at: https://www.rmc.usace.army.mil/Software/RMC-BestFit/.

(c) Software packages to implement L-Moments in Fortran (https://github.com/brenonf/L-Moments), R (https://cran.r-project.org/web/packages/lmom/index.html), and Python (https://pypi.org/project/lmoments/0.1.0/) code.

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7. EVENT-BASED DESIGN FLOOD ESTIMATION

7.1 Chapter Overview

As highlighted in Chapter 4 (see Section 4.4), rainfall-runoff modelling can be broadly classified as event-based DFE methods and CSM. In applying event-based DFE methods, the initial conditions and losses are estimated at the beginning of a storm event and are regarded as being constant, while in CSM rainfall-runoff modelling, the water balance is simulated on a continuous basis; thereby eliminating the need for initial conditions and loss assumptions.

In South Africa, three event-based approaches to at-site DFE are available: (i) probabilistic, (ii) deterministic, and (iii) empirical methods. The event-based DFE methods recommended for urban areas in South Africa are discussed in this chapter and include the deterministic Rational Method (RM), Soil Conservation Services – South Africa (SCS-SA) and the empirical Regional Maximum Flood (RMF) methods.

The RM is detailed in Section 7.2. Despite being criticised as a subjective and inaccurate method, the RM still widely used today, and various studies (as highlighted in Section 7.7) have confirmed the suitability thereof in both small and larger catchments exceeding 15 km². Hence, the use of the RM in combination with other DFE methods is recommended. The gridded RLMA&SI design rainfall (see Chapter 5, Section 5.3.3) should be as the rainfall input, while the weighted runoff coefficients should be based on the most recent data sources applicable to catchment geomorphology, land cover, land use, and soils (see Chapter 5, Sections 5.6 to 5.8).

The SCS-SA method is included in Section 7.3. The SCS-SA method can be manually applied; however, it is recommended that the SCS-SA software (Schulze *et al.*, 2004) is used. The SCS-SA software can be requested from: CWRR@ukzn.ac.za. When using the SCS-SA method, it is recommended that the Joint Association Method (JAM) be limited to return periods (T) ≤ 20-year, while the Median Condition Method (MCM) of Curve Number (CN) approach is recommended for all return periods. In addition, it is also recommended that the 1-day gridded RLMA&SI design rainfall (see Chapter 5, Section 5.3.3) be used as rainfall input.

PC-SWMM, as outlined in Section 7.4, and discussed in detail in Chapter 8 can also be applied for event-based analysis or continuous simulation.

The RMF methods are included in Section 7.5 and are empirically derived upper limit flood peaks that could be expected at a given site; hence, in applying these methods, a return period is not associated with the estimated maximum flood peak. As a result, typical return period factors have been proposed in the literature to downscale and associate the RMF flood peaks with a specific return period. The RMF methods should be regarded as a conservative approach to estimate upper limit floods which can be used when conducting sensitivity analyses and users are cautioned that some observed floods have exceeded the values estimated in some regions by the RMF, which was published in 1988.

7.2 Rational Method

7.2.1. Introduction

The Rational Method (RM) was developed in Ireland by Mulvaney in 1855 and it is still the most commonly used method in the world. The RM is applicable to both small (urban) and rural (natural) catchments but is criticised as being subjective and inaccurate (Alexander, 2002a; Smithers, 2012). However, the method is still widely used today due to the simple model structure, which is both easy to use and to understand (Lee and Heaney, 2003; Parak and Pegram, 2006; Coombes *et al.*, 2015).

7.2.2. Limitations and assumptions

The RM is based on the law of the conservation of mass and assumes that the peak discharge of a catchment is directly proportional to the size of the contributing area and the rainfall intensity. The RM estimates the T-year flood peak resulting from T-year rainfall intensity for durations equal to the critical storm duration or time of concentration (T_C). Storm losses are represented by a dimensionless runoff coefficient, which are published as tables for a range of land covers or can be estimated as a function of MAP, slope, permeability, land use, vegetation, and urbanisation within a catchment. Return period adjustment factors are used to decrease the runoff coefficients for events with return periods less than 50 years (Alexander, 2001; SANRAL, 2013).

It is generally recommended that the RM should only be applied to catchments smaller than 15 km². However, experienced users believe and have demonstrated that the RM can be successfully applied in larger catchments (see Section 7.7). The RM can only estimate the flood peaks and typically triangular shaped hydrographs are used to estimate the hydrograph associated with the peak discharge. The following assumptions are relevant when applying the RM (SANRAL, 2013):

(a) Rainfall has a uniform spatial and temporal distribution over the catchment.

(b) Peak runoff occurs at the end of the critical storm duration, i.e. T_C.

(c) Runoff coefficients remain constant throughout the duration of the storm.

(d) The return period of the peak runoff is the same as the input rainfall intensity.

According to SANRAL (2013) the RM provides good DFE results when compared to other methods, but caution must be taken when applying the method. Naidoo (2020) identified the RM as one of the better performing methods over a range of catchment sizes in South Africa. The method is an approximate, deterministic method; hence the skills and experience of the practitioner, as well as the careful selection of runoff coefficients are crucial in obtaining reliable estimates (Pilgrim and Cordery, 1993; Smithers, 2012; SANRAL, 2013).

In addition, Pilgrim and Cordery (1993) identified the following weaknesses associated with the RM:

(a) The level of judgement required to determine the most realistic runoff coefficient is largely subjective.
(b) The variability of the coefficients between different hydrological regimes in the same catchment is not accommodated.
(c) The estimation of catchment response time is subjected to regional differences in the time of concentration and cannot be based only on measured catchment characteristics.
(d) The assumption of uniform rainfall intensity and the exclusion of temporary storage limit the use in urban and small rural catchments.

Hence, the use of a probabilistic as opposed to a deterministic approach to determine the runoff coefficients is recommended in the literature inaccurate (Pilgrim and Cordery, 1993; Alexander, 2001; Parak and Pegram, 2006; Smithers, 2012). In Australia, the probabilistic approach of the RM in catchments up to 250 km² resulted in more acceptable results without any variation in the probabilistic runoff coefficients with catchment characteristics (Pilgrim and Cordery, 1993). Hence, in using a regional probabilistic approach to the RM, the direct conversion from rainfall to a design flood of the same return period is achieved and eliminates the subjective selection of the runoff coefficient. The Standard Design Flood (SDF) method (Alexander, 2002c) is effectively a probabilistic regionally calibrated version of the RM, but the SDF method was only calibrated for larger rural catchments. According to Alexander the method can be applied to catchments ranging from 10 km^2 to 40 000 km^2 and hence is not recommended for use in urban areas or rural catchments smaller than 10 km^2.

7.2.3. Input data requirements

The input data requirements of the RM are summarised in Table 7-1, which is a summary of the detailed information as included in Chapter 4 (see Section 4.7).

Table 7-1: RM input data requirements

Method	Input data/parameters	Cross Reference	Source
RM	A, α, β, ARF, $D_{\%}$, H, L_{CH}, L_O, n, P_T, S, S_{CH}, T_C and γ.	Sections 5.3, 5.6, 5.7, 5.8, 5.10 and 7.2	• Local Digital Terrain Model (DTM) data • Geographical Information System (GIS) data. • Digital Elevation Model (DEM) data.

Note: Catchment area (A, km^2), rural area distribution factor (α, %), Areal Reduction Factor (ARF, %), urban area distribution factor (β, %), dolomitic area ($D_{\%}$, %), height difference along overland flow path (H, m), length of longest watercourse (L_{CH}, km), hydraulic length of overland flow path (L_O, km), mean annual precipitation (MAP), roughness coefficient for overland flow (n), design rainfall depth (P_T, mm), average catchment slope (S, %), average main watercourse slope (S_{CH}, m/m), time of concentration (T_C, minutes/hours), and lake area distribution factor (γ, %).

7.2.4. Calculation procedures

<u>Time of concentration (T_C):</u>

Please refer to Chapter 5, Section 5.11 for all the details related to the T_C.

$$T_{C1} = 0.604\left(\frac{nL_1}{\sqrt{\frac{H}{1000L_1}}}\right)^{0.467} \qquad \text{Eq. 7-1}$$

$$T_{C2} = \left(\frac{0.87L_2^{\ 2}}{1000S_{CH}}\right)^{0.385} \qquad \text{Eq. 7-2}$$

$$T_{C3} = \left(\frac{L_3}{3.6\bar{v}}\right) \qquad \text{Eq. 7-3}$$

$$T_C = T_{C1} + T_{C2} + T_{C3} \qquad \text{Eq. 7-4}$$

where:
- T_{C1-3} = time of concentration (hours),
- H = height difference along overland flow path (m),
- L_1 = hydraulic length of overland flow path (km),
- L_2 = length of longest watercourse (km; see Section 5.6.6),
- L_3 = length of artificial flow path (km),
- n = roughness coefficient for overland flow (Table 7-2),
- S_{CH} = average main watercourse slope (m/m; see Section 5.6.6), and
- $\bar{v}$ = average/design velocity (m/s; Table 7-3).

Table 7-2: Manning's *n* values applicable to overland flow conditions (SANRAL, 2013)

Surface description	*n*
Paved areas	0.02
Clean, compacted soil, no stones	0.10
Bare ground	0.10
Eroded Karoo	0.15
Densely vegetated Karoo	0.24
Sparse grass, rough surface	0.30
Poor grass cover	0.30
Medium grass cover	0.40
Thick grass cover	0.80
Ploughed fields	0.40

Table 7-3: Maximum permissible average velocity (Mathee *et al.*, 1986)

Artificial canal: Type of material/cover	*v* (m/s)
Light sand. very erodible	0.2-0.3
Very loosely sand	0.3-0.4
Coarse sand or lightly sandy	0.4-0.6
Sand	0.6-0.75
Sandy loam	0.75-0.8
Loamy soil. very erodible	0.8-1.0
Clay-loam	1.0-1.2
Clay	1.2-1.5
Gravel	1.5-2.0
Conglomerates. soft shale/stones	2.0-2.5
Hard rock	3.0-4.5
Concrete	4.5-6.0
Poor grass cover on highly erodible soil	1.0 max
Average grass cover on erodible soil	1.2 max
Good grass cover on non-erodible soil	1.5 max
Very dense grass cover on non-erodible soil	2.5 max

Weighted runoff coefficients:

In South Africa, the runoff coefficients (*C*) for rural and urban catchment areas are commonly estimated using the values proposed by SANRAL (2013), as listed in Table 7-4 to Table 7-6. These values have been adapted from Horner and Flynt (1936), Vorster (1940) and Chow (1964) by the (then) Directorate of Water Affairs and first published in the Drainage Manual in 1983 (Rooseboom *et al.*, 1983).

$$C_1 = C_p + C_s + C_v \qquad \text{Eq. 7-5}$$

$$C_{1D} = C_1(1 - D_{\%}) + C_1 D_{\%}\left(\sum\left(D_{factor} C_{S\%}\right)\right) \qquad \text{Eq. 7-6}$$

$C_{1T} = F_T C_{1D}$ Eq. 7-7

$C_T = \alpha C_{1T} + \beta C_2 + \gamma C_3$ Eq. 7-8

where:

α = rural area distribution factor (%),

β = urban area distribution factor (%),

γ = lake area distribution factor (%),

C_1 = rural runoff coefficient between zero and one,

C_{1D} = rural runoff coefficient incorporating the effect of dolomite areas,

C_{1T} = rural runoff coefficient incorporating the effect of initial saturation,

C_2 = urban runoff coefficient between zero and one,

C_3 = lake runoff coefficient,

C_p = runoff coefficient according to average soil permeability (Table 7-4),

C_s = runoff coefficient according to average catchment slope (Table 7-5),

C_T = weighted runoff coefficient for T-year return period,

C_v =runoff coefficient according to average land use/vegetation (Table 7-6),

$D\%$ = dolomitic area (%; Table 7-7), and

F_T = return period adjustment factor (Table 7-8).

Table 7-4: Soil permeability (C_p) runoff coefficients (SANRAL, 2013)

Soil permeability/ class		Mean Annual Precipitation (MAP, mm)		
		< 600	600-900	> 900
Very permeable	A	0.03	0.04	0.05
	A/B	0.04	0.06	0.07
Permeable	B	0.05	0.08	0.10
	B/C	0.08	0.12	0.15
Semi-permeable	C	0.12	0.16	0.20
	C/D	0.16	0.21	0.25
Impermeable	D	0.21	0.26	0.30

Table 7-5: Surface slope (C_s) runoff coefficients (SANRAL, 2013)

Slope (%)	Mean Annual Precipitation (MAP, mm)		
	< 600	600-900	> 900
0-3	0.01	0.03	0.05
3-10	0.06	0.08	0.11
10-30	0.12	0.16	0.20
> 30	0.22	0.26	0.30

Table 7-6: Land use and vegetation (C_v) runoff coefficients (SANRAL, 2013)

Land use and vegetation	Mean Annual Precipitation (MAP, mm)		
	< 600	600-900	> 900
Rural			
Thick bush & plantations	0.03	0.04	0.05
Light bush& farmlands	0.07	0.11	0.15
No vegetation	0.26	0.28	0.30
Grass land	0.17	0.21	0.25
Cultivated land, contoured	0.07	0.11	0.15
Cultivated land	0.17	0.21	0.25
Urban	**< 600**	**600-900**	**> 900**
Lawns: Sandy & flat (< 2%)	0.05-0.1	0.05-0.1	0.05-0.1
Lawns: Steep (> 7%)	0.15-0.2	0.15-0.2	0.15-0.2
Lawns: Heavy soil, flat (< 2%)	0.13-0.17	0.13-0.17	0.13-0.17
Lawns: Heavy soil, steep (< 7%)	0.25-0.35	0.25-0.35	0.25-0.35
Houses	0.3-0.5	0.3-0.5	0.3-0.5
Flats	0.5-0.7	0.5-0.7	0.5-0.7
Light industry	0.5-0.8	0.5-0.8	0.5-0.8
Heavy industry	0.6-0.9	0.6-0.9	0.6-0.9
City centre	0.7-0.95	0.7-0.95	0.7-0.95
Suburban	0.5-0.7	0.5-0.7	0.5-0.7
Streets	0.7-0.95	0.7-0.95	0.7-0.95
Maximum flood	1	1	1

Table 7-7: Dolomite adjustment factors for different slope classes (SANRAL, 2013)

Slope (%)	D_{factor}
0-3	0.10
3-10	0.20
10-30	0.35
> 30	0.50

Table 7-8: Return period adjustment factor (F_T) (SANRAL, 2013)

Return period [years]	2	5	10	20	50	100
Steep and impermeable catchments	0.75	0.80	0.85	0.90	0.95	1
Flat and permeable catchments	0.50	0.55	0.60	0.67	0.83	1
DWS (Van der Spuy & Rademeyer, 2021)	0.32	0.50	0.61	0.71	0.83	0.92
O'Loughlin & Robinson (1987)	0.85	0.95	1	1.05	1.15	1.20

Design rainfall:

In the past, the design rainfall estimation in the RM was based on using either DDF relationship as proposed by Midgley and Pitman (1978) and/or the modified Hershfield equation (Alexander, 2001; SANRAL, 2013). For the purpose of this best practice guideline, it is recommended that all design rainfall estimates are based on the gridded RLMA&SI approach, as detailed in Chapter 5, Section 5.3.3.

As highlighted in Section 5.3.3, it is recommended that that the average value of the RLMA&SI-based design rainfall values should be used to calculate the catchment point design rainfall. Given that the latter design rainfall is still a point estimate, an ARF should be applied. Currently, the ARF method proposed by Alexander (2001) should be used as the preferred method (see Chapter 5, Section 5.3.4). For the ease of reference, Eq. 5-1, is listed again as Eq. 7-9.

$$ARF = [\mathbf{90000} - \mathbf{12800}\boldsymbol{Ln}(\boldsymbol{A}) + \mathbf{9830}\boldsymbol{Ln}(\mathbf{60}\boldsymbol{T_C})]^{\mathbf{0.4}} \qquad \text{Eq. 7-9}$$

$$I_T = \frac{P_T}{T_C} \qquad \text{Eq. 7-10}$$

$$I_{TAvg} = I_T\left(\frac{ARF}{100}\right) \qquad \text{Eq. 7-11}$$

Peak discharge:

$$Q_T = 0.278 C_T I_{TAvg} A \qquad \text{Eq. 7-12}$$

where: Q_T = peak flow for T-year return period (m^3/s),

A = catchment area (km²),

C_T = weighted runoff coefficient for T-year return period,

I_T = design point rainfall intensity (mm/h), and

I_{TAvg} = average design rainfall intensity (mm/h).

7.3 SCS-SA Method

7.3.1. Introduction

The United States Department of Agriculture (USDA) SCS method has been widely used internationally for the estimation of peak discharges, runoff volume and hydrograph shape in rural and urban catchment areas up to 30 km² (USDA, 1972). Internationally, it is one of the most widely applied DFE methods and forms the basis for infiltration calculations in various CSM systems (Harbor, 1994; Boughton and Droop, 2003; Aichele and Andresen, 2013).

In South Africa, it was introduced by Reich (1962), but it became more popular in 1979 after being adapted for general use in southern Africa by Schulze and Arnold (1979). Based on extensive research by, *inter alia*, Schulze (1982), Schmidt and Schulze (1984b) and Dunsmore *et al.* (1986) and the development of extended databases, an updated version of the 1979 SCS design manual was produced in 1987 in the form of three reports published by the WRC, namely:

(a) An extended theory-based *Flood volume and peak discharge from small catchments in southern Africa, based on the SCS technique* (Schmidt and Schulze, 1987b).

(b) A *User manual for SCS-based design runoff estimation in southern Africa* (Schmidt and Schulze, 1987a).

(c) Appendices to the above reports (Schmidt *et al.*, 1987).

The above manually-based SCS method has been computerised (Schulze *et al.*, 1992) and the method is now widely used for the estimation of design floods from small catchments in South Africa. The adaptations to the original SCS method for southern Africa, termed SCS-SA, include the following:

(a) Refinements to the soils classification to cater for soils in southern Africa and the linking of these to the local (Binomial and Taxonomic) soil classification systems.

(b) The development of methods to account for regional differences in median antecedent soil moisture conditions prior to large rainfall events and for the joint association between rainfall and runoff.

(c) The estimation of design rainfall and typical storm distributions for southern Africa.

(d) The development of an empirical equation to estimate lag time from small catchments in southern Africa.

7.3.2. Limitations and assumptions

The SCS-SA method is not as sensitive as the RM to user inputs and is recommended for DFE on a considerable range of land use and catchment size categories. The SCS-SA method considers most factors that affect runoff, including the catchment size, catchment geomorphology, land use, soil types, antecedent soil moisture conditions, and the quantity and temporal distribution of rainfall (storm duration). It computes the *T*-year flood hydrograph based on the *T*-year 1-day rainfall, uses a typical unit volume runoff hydrograph of triangular shape, and storm losses estimated as a function of a Curve Number (*CN*). However, the procedures used to estimate the lag time and the most representative *CN* values are highly subjective and can result in inconsistencies (Schulze *et al.*, 1992)

The SCS-SA method was originally developed for small catchments (A < 30 km²) and the software enables up to nine (9) sub-catchments or Hydrological Response Units (HRUs) to be modelled. Typically, a catchment is discretised into HRUs, the runoff is simulated and summed from the HRUs and the total is used to estimate the peak discharge. This approach of computing the runoff from each HRU accounts for the non-linear runoff response from a catchment, and the use of weighted CNs assumes that the runoff response is linear and is not recommended. Given the small catchment size, the use of ARFs (Chapter 5, Section 5.3.4) with the SCS-SA method is generally not necessary.

In a recent assessment, Gericke (2021) applied the standard SCS method using area weighted parameters in 48 gauged catchments located in four climatologically different regions of South Africa. The SCS method proved to be one of the best performing methods in the catchment areas ranging from 22 to 31 283 km^2. Hence, this serves as confirmation that the SCS-SA method, which is an improved version of the standard SCS method, could potentially be applied to larger catchments, given appropriate determination of input parameters.

7.3.3. Input data requirements

The input data requirements of the SCS-SA method are summarised in Table 7-9, which is an extract of the information as included in Chapter 4 (see Section 4.7).

Table 7-9: SCS-SA input data requirements

Method	Input data/parameters	Cross Reference	Source
SCS-SA	A, H, LAT, LONG, L_{CH}, L_O, MAP, n, P_T, S, S_{CH}, T_C and T_L. Number of sub-catchments and/or Hydrological Response Units (HRUs). SCS hydrological soils groups: • Infiltration/drainage rates (mm/h). • Binomial or taxonomic soil classification. Land cover, treatment, and stormflow potential.	Sections 5.3, 5.6, 5.7, 5.8, 5.10 and 7.3.	• Local Digital Terrain Model (DTM) data • Geographical Information System (GIS) data. • Digital Elevation Model (DEM) data. • Quinary catchments: • Hydrological soils: https://www.waterresearchobservatory.org/

Method	Input data/parameters	Cross Reference	Source
	Curve Numbers (CN). Soil depth category. Rainfall distribution Types (1-4).		

Note: Catchment area (A, km^2), height difference along overland flow path (H, m), latitude (LAT), longitude (LONG), length of longest watercourse (L_{CH}, km), hydraulic length of overland flow path (L_O, km), mean annual precipitation (MAP), roughness coefficient for overland flow (n), design rainfall depth (P_T, mm), average catchment slope (S, %), average main watercourse slope (S_{CH}, m/m), time of concentration (T_C, hours), and lag time (T_L, hours).

7.3.4. Estimation of runoff volume

Stormflow is defined as the direct runoff response to a given rainfall event, and consists of both surface runoff and subsurface flows, but excludes baseflow (i.e. the delayed subsurface response). Stormflow depth is calculated in the SCS-SA model using Eq. 7-13.

$$Q_V = \frac{(P_T - I_A)^2}{P_T - I_A + S_R} \qquad \text{Eq. 7-13}$$

$$S_R = \frac{25\,400}{CN} - 254 \qquad \text{Eq. 7-14}$$

$$I_A = cS_R \qquad \text{Eq. 7-15}$$

where:
- c = seasonal soil moisture status coefficient, normally 0.1 for South Africa,
- CN = runoff Curve Number,
- I_A = initial losses/abstractions (e.g. depression storage, interception, and initial infiltration, normally $0.1S_R$ (mm),
- P_T = 1-day design rainfall depth for T-year return period using either the RLMA&SI design rainfall information (mm) or user estimated design value,
- Q_V = stormflow depth (mm), and
- S_R = potential maximum soil water retention (mm).

The stormflow depth (Q_V) represents a uniform depth over the catchment and may be converted to volume by multiplying with the depth with catchment area. The potential maximum soil water retention (S_R) is related to hydrological soil properties, land cover and land management conditions, as well as to the soil moisture status of the catchment prior to a rainfall event and finds expression through a dimensionless response index termed the Curve Number (*CN*). The *CN* and S_R are related as shown in Eq. 7-14. The determination of inputs to Eqs. 7-13 and 7-14, namely the *CN* value and 1-day design rainfall depth, are discussed in the subsequent sections.

7.3.5. Initial Curve Numbers

The *CN* value is an index expressing a catchment's stormflow response to a rainfall event. The characteristics considered in estimating a representative *CN* value for a catchment are:

(a) Hydrological properties of the soil.

(b) Land cover properties (including land use, its treatment, and hydrological conditions).

(c) Catchment antecedent soil moisture status, i.e. the soil's relative wetness or dryness just prior to the rainfall event.

CN values are initially estimated and based on only the soil and land cover properties; hence, antecedent soil moisture conditions are ignored. These initial (also erroneously termed "average") *CN* values are designated *CN-II*, and values are given for a wide range of land cover and treatment classes, stormflow potentials and hydrological soil groups in Table 7-10 The *CN-II* values may be adjusted up (i.e. increased, for a relatively "wet" catchment) or down (i.e. decreased to a lower final *CN*, for a relatively "dry" catchment) according to the catchment's soil moisture status typically prevailing before the design events occur, following the procedures given in the subsequent sections.

Table 7-10: Initial *CN-II* values for different land-use categories and hydrological soil groupings (after Schmidt and Schulze, 1987a; NEH, 2004; Schulze *et al.*, 2004)

Land Cover Class	Land Treatment/ Practice/Description	Stormflow Potential*	Hydrological Soil Group						
			A	A/B	B	B/C	C	C/D	D
Fallow	1 = Straight row		77	82	86	89	91	93	94
	2 = Straight row + conservation tillage	High	75	80	84	87	89	91	92
	3 = Straight row + conservation tillage	Low	74	79	83	85	87	89	90
Row Crops	1 = Straight row	High	72	77	81	85	88	90	91
	2 = Straight row	Low	67	73	78	82	85	87	89
	3 = Straight row + conservation tillage	High	71	75	79	83	86	88	89
	4 = Straight row + conservation tillage	Low	64	70	75	79	82	84	85
	5 = Planted on contour	High	70	75	79	82	84	86	88
	6 = Planted on contour	Low	65	69	75	79	82	84	86
	7 = Planted on contour + conservation tillage	High	69	74	78	81	83	85	87
	8 = Planted on contour + conservation tillage	Low	64	70	74	78	80	82	84
	9 = Conservation structures	High	66	70	74	77	80	82	82
	10 = Conservation structures	Low	62	67	71	75	78	80	81

Land Cover Class	Land Treatment/ Practice/Description	Stormflow Potential*	Hydrological Soil Group						
			A	A/B	B	B/C	C	C/D	D
	11 = Conservation structures + conservation tillage	High	65	70	73	76	79	80	81
	12 = Conservation structures + conservation tillage	Low	61	66	70	73	76	78	79
Garden Crops	1 = Straight row	Low	45	56	66	72	77	80	83
	2 = Straight row	High	68	71	75	79	81	83	84
Small Grain	1 = Straight row	High	65	71	76	80	84	86	88
	2 = Straight row	Low	63	69	75	79	83	85	87
	3 = Straight row + conservation tillage	High	64	70	74	78	82	84	86
	4 = Straight row + conservation tillage	Low	60	67	72	76	80	82	84
	5 = Planted on contour	High	63	69	74	79	82	84	85
	6 = Planted on contour	Low	61	67	73	78	81	83	84
	7 = Planted on contour + conservation tillage	High	62	68	73	77	81	83	84
	8 = Planted on contour + conservation tillage	Low	60	66	72	76	79	81	82
	9 = Planted on contour - winter rainfall region	Low	63	66	70	75	78	80	81
	10 = Conservation structures	High	61	67	72	76	79	81	82
	11 = Conservation structures	Low	59	65	70	75	78	80	81
	12 = Conservation structures + conservation tillage	High	60	67	71	75	78	80	81
	13 = Conservation structures + conservation tillage	Low	58	64	69	73	76	78	79
Close Seeded Legumes or Rotational Meadow	1 = Straight Row	High	66	72	77	81	85	87	89
	2 = Straight Row	Low	58	65	72	75	81	84	85
	3 = Planted on contour	High	64	70	75	80	83	84	85
	4 = Planted on contour	Low	55	63	69	74	78	81	83
	5 = Conservation structures	High	63	68	73	77	80	82	83
	6 = Conservation structures	Low	51	60	67	72	76	78	80
Sugarcane	1 = Straight row: trash burnt		43	55	65	72	77	80	82
	2 = Straight row: trash mulch		45	56	66	72	77	80	83
	3 = Straight row: limited cover		67	73	78	82	85	87	89
	4 = Straight row: partial cover		49	60	69	73	79	82	84
	5 = Straight row: complete cover		39	50	61	68	74	78	80
	6 = Conservation structures: limited cover		65	70	75	79	82	84	86
	7 = Conservation structures: partial cover		25	46	59	67	75	80	83
	8 = Conservation structures: complete cover		6	14	35	59	70	75	79
Veld (range) and Pasture	1 = Veld/pasture in poor condition	High	68	74	79	83	86	88	89
	2 = Veld/pasture in fair condition	Moderate	49	61	69	75	79	82	84
	3 = Veld/pasture in good condition	Low	39	51	61	68	74	78	80
	4 = Pasture planted on contour	High	47	57	67	75	81	85	88
	5 = Pasture planted on contour	Moderate	25	46	59	67	75	80	83
	6 = Pasture planted on contour	Low	6	14	35	59	70	75	79
Irrigated Pasture		Low	35	41	48	57	65	68	70
Meadow		Low	30	45	58	65	71	75	81
Woods and Scrub	1 = Woods	High	45	56	66	72	77	80	83
	2 = Woods	Moderate	36	49	60	68	73	77	79
	3 = Woods	Low	25	47	55	64	70	74	77
	4 = Brush - Winter rainfall region	Low	28	36	44	53	60	64	66
Orchards	1 = Winter rainfall region, understory of crop cover		39	44	53	61	66	69	71
Orchards Forests & Plantations	1 = Humus depth 25 mm; Compactness: compact		52	62	72	77	82	85	87
	2 = " " " moderate		48	58	68	73	78	82	85
	3 = " " " loose/friable		37	49	60	66	71	74	77
	4 = Humus depth 50 mm; Compactness: compact		48	58	68	73	78	82	85
	5 = " " " moderate		42	54	65	70	75	78	81
	6 = " " " loose/friable		32	45	57	62	67	71	74
	7 = Humus depth 100 mm; Compactness: compact		41	53	64	69	74	77	80
	8 = " " " moderate		34	47	59	64	69	72	75
	9 = " " " loose/friable		23	37	50	56	61	64	67
	10 = Humus depth 150 mm; Compactness: compact		37	49	60	66	71	74	77
	11 = " " " moderate		30	43	56	61	66	69	72

Land Cover Class	Land Treatment/ Practice/Description	Stormflow Potential*	Hydrological Soil Group						
			A	A/B	B	B/C	C	C/D	D
	12 = " " " loose/friable		18	33	47	52	57	61	65
Urban/Sub-urban Land Uses	1 = Open spaces, parks, cemeteries >75% grass cover	Low	39	51	61	68	74	78	80
	2 = Open spaces, parks, cemeteries 75% grass cover	Medium	49	61	69	75	79	82	84
	3 = Open spaces, parks, cemeteries < 50% grass cover	High	68	73	79	83	86	88	89
	4 = Commercial/business areas 85% grass cover		89	91	92	93	94	95	95
	5 = Industrial districts 72% impervious+		81	85	88	90	91	92	93
	6 = Residential: lot size 50 m^2 65% impervious+		77	81	85	88	90	91	92
	7 = " " 1000 m^2 38% impervious+		61	69	75	80	83	85	87
	8 = " " 1350 m^2 30% impervious+		57	65	72	77	81	84	86
	9 = " " 2000 m^2 25% impervious+		54	63	70	76	80	83	85
	10 = " " 4000 m^2 20% impervious+		51	61	68	75	78	82	84
	11 = Paved parking lots, roofs, etc.		98	98	98	98	98	98	98
	12 = Streets/roads: tarred, with storm sewers, curbs		98	98	98	98	98	98	98
	13 = " gravel		76	81	85	88	89	90	91
	14 = " dirt		72	77	82	85	87	88	89
	15 = " dirt-hard surface		74	79	84	88	90	91	92
Generalized *CN* numbers	Agriculture		63	66	69	74	77	79	82
	Open space		36	49	60	67	73	76	78
	Forest		25	47	55	64	70	74	77
	Disturbed land		72	77	82	85	88	89	90
	Residential		61	69	76	81	84	86	88
	Paved		98	98	98	98	98	98	98
	Commercial: Industrial		84	86	88	89	90	91	93

* High stormflow potential = poor hydrological condition.
Medium stormflow potential = fair hydrological condition.
Low stormflow potential = good hydrological condition.

\+ % Impervious = the percentage of the area which is impervious and directly connected to a drainage system.

If 100% of the impervious area are directly connected to the drainage system, and the impervious area percentages in Table 7-10 or the pervious land use assumptions are not applicable, Eq. 7-16 should be used to compute a composite *CN* (NEH, 2004).

$$CN_c = \boldsymbol{CN_p + (\frac{P_{imp}}{100})(98 - CN_p)} \qquad \text{Eq. 7-16}$$

where: CN_c = composite runoff curve number,
CN_p = pervious runoff curve number, and
P_{imp} = percent imperviousness (%).

7.3.6. Hydrological soil groupings

<u>SCS hydrological soils groups:</u>

It is important to have a good understanding of the soils present in any catchment, as well as their association with the different Hydrological Soil Groups (HSGs), since it has a direct influence on the catchment response time and associated runoff. The following four hydrological soil groups are of importance in the SCS method (Schulze *et al.*, 1992):

(a) **Group A (Low runoff potential, very permeable):** Infiltration rate is high, soils are deep (well drained) and texture is coarse (gravel and coarse sand). Final infiltration rate ± 25 mm/h and permeability rate > 7.6 mm/h. The knowledge of dolomite areas is of great importance for the deterministic and empirical flood estimation methods. It is important to note that dolomite areas may absorb as much as 90% of the runoff for underground storage. While dolomite areas should always be classified as very permeable, where urban development takes place, it is compulsory to minimise infiltration to reduce the risk of sinkhole formation. Thus, areas with dolomitic geology subject to urban development should be classified as impermeable.

(b) **Group B (Moderately low runoff potential, permeable):** Infiltration rate is medium and medium effective soil depth (sandy, sandy loam). Final infiltration rate ± 13 mm/h and permeability rate between 3.8 and 7.6 mm/h.

(c) **Group C (Moderately high runoff potential, semi-permeable):** Infiltration rate is low, depths are shallow, and texture is fine (silt, loam, and clayey sand). Final infiltration rate ± 6 mm/h and permeability rate between 1.3 and 3.8 mm/h.

(d) **Group D (High runoff potential, impermeable):** Very low infiltration rates, very shallow and/or expansive soils (clay, peat, and rock). Final infiltration rate ± 3 mm/h and permeability rate < 1.3 mm/h.

In addition to the above hydrological soil groupings, the following should be noted:

(a) For southern Africa, intermediate soil groups (A/B, B/C, C/D) have been identified and should be used in the classification of soil groups.

(b) The typical final infiltration and permeability rates provided above both refer to a saturated soil.

(c) Final infiltration rates refer to soils with a short grass cover.

(d) Infiltration rate is controlled by surface conditions, whereas permeability rates, are controlled by properties of the soil profile.

(e) Infiltration or percolation tests, following standard procedures (e.g. Bouwer, 1986), may be conducted at a number of sites in the catchment to assist in soil group selection.

<u>Southern African hydrological soils groupings:</u>

Soils in southern African have been divided *taxonomically* by the Soil Classification Working Group (SCWG, 1991) into 73 *soil forms*, each identified by a sequence of diagnostic soil horizons. Each of these soil forms in turn is sub-divided into *soil families* on the basis of soil physical and chemical properties. Also still in use in southern Africa, and important in that the national soils maps have been produced on this basis, is the *binomial* soil classification system in which 41 soil forms are sub-divided (again on the basis of physical and chemical characteristics) into *soil series*, of which 501 have been identified and described by Macvicar *et al.* (1977).

Hydrological interrelationships between soil forms, families and series have been described by Schulze *et al.* (1991). Each soil form/family (Table 5.2 in *Visual SCS-SA* User Manual) and also for the soil form/series (Table 5.3 in *Visual SCS-SA* User Manual) can be selected by means of a drop-down list in the *Visual SCS-SA* software. Each soil family has been assigned a corresponding soil series and the 501 soil series identified have been designated to an SCS HSG, i.e. A, B, C or D, or to intermediate groups, namely A/B, B/C and C/D. Details of the complete taxonomic and binomial classification of soils in southern Africa into hydrological soil groups are available in the *Visual SCS-SA* User Manual (Schulze *et al.*, 2004).

The major source of mapped soils information in South Africa consists of detailed soils Land Type images produced by the Agricultural Research Council's Institute for Soil, Climate and Water (ARC-ISCW) and which may, together with accompanying documentation, be purchased from the Institute. A Land Type is a mapping unit, each of which has, in turn, been sub-divided into terrain units consisting of the scarp, crest, mid-slope, foot-slope and valley bottom. However, the spatially dominant terrain unit is usually taken to represent the Land Type. Further information of Land Type maps and contact details are available from the ARC at:

https://www.arc.agric.za/arc-iscw/News%20Articles%20Library/Soil%20Information%20at%20the%20ARC.pdf

Procedures for determining hydrological soil groups:

The following procedures may be used to determine the HSG of the SCS-SA method:

Field work and laboratory analyses can be used to determine the final infiltration and permeability rates. The hydrological soil group categories A to D can be determined as described above. Where information on soil form and soil family (with its associated texture class), or alternatively soil form and series is known, the SCS soil groupings associated with each soil series can be extracted from the *Visual SCS-SA* User Manual (taxonomic and binomial classification of soils and associated hydrological soil groups). Additional details regarding the procedure used to classify the soil forms and series into HSGs are given in Schulze and Arnold (1979) and Schulze and Schmidt (1987b).The national generalised soil group map in which the soil groups derived from Land Type maps have been mapped for South Africa at the resolution of the 5 838 Quinary catchments (Schulze, 2012), is shown in Figure 7-1. It is recommended that the GIS-based shapefiles and information available at:

https://www.waterresearchobservatory.org/ be used to extract catchment specific HSG information.

The more detailed national generalised soil group map in which the soil groups are mapped at a spatial resolution 27 491 Terrain Units (TUs) in South Africa, is shown in Figure 7-2

The map of the SCS-SA HSGs produced by Schulze and Schütte (2018) is at a more detailed spatial resolution than the map produced by Schulze (2012) and is recommended for use in practice. A GIS-based shapefile is available at: https://www.waterresearchobservatory.org.

Computed stormflow is highly sensitive to the HSG selected and every effort should therefore be made to accurately determine the appropriate group and to become familiar with the current prevailing soil classification procedures for southern Africa. Available soil maps and Land Type maps should be consulted in this regard and a field inspection of soils is strongly recommended.

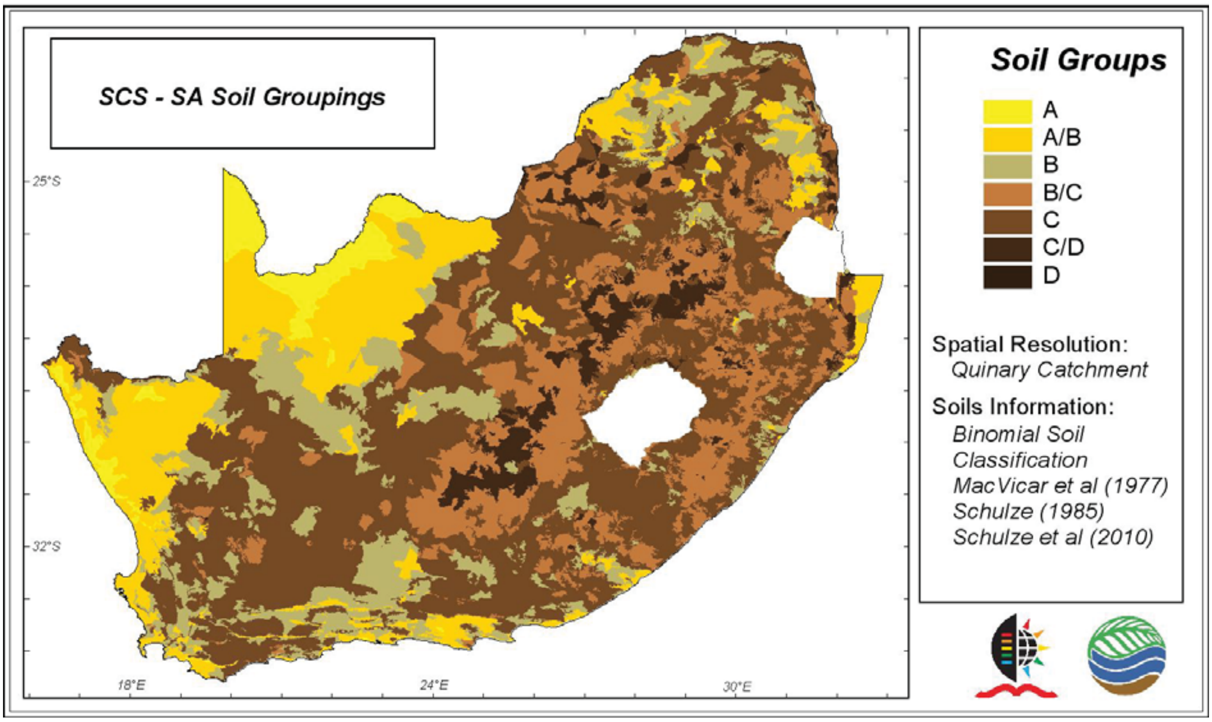

Figure 7-1: Generalised SCS soil grouping classification for South Africa derived from Land Type maps (Schulze, 2012)

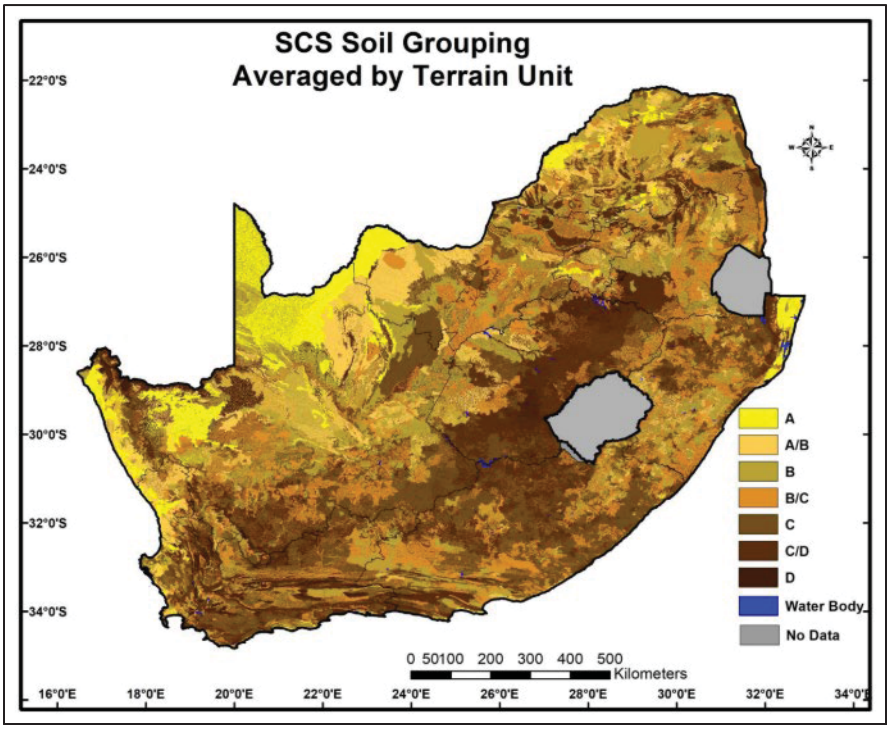

Figure 7-2: SCS-SA soil groupings across SA at a spatial resolution of Terrain Units (Schulze and Schütte, 2018)

<u>Adjustment of soil groups in the field:</u>

Due to the variable nature of soil properties even within a specific series, some further guidelines for adjustment of soil groups in the field are summarised below:

(a) **Soil depth:** Where typically deep soils are in the shallow phase, for example on steep slopes, they should be downgraded one group (e.g. B becomes B/C).

(b) **Surface sealing:** Where surface sealing is evident in loco, soils should be downgraded one group.

(c) **Topographical position:** Generally, series in bottomlands may be downgraded (e.g. B to B/C) and series formed on uplands upgraded one group (e.g. B to A/B).

(d) **Parent material:** Identical series derived from different parent materials may require re-grouping, e.g. series derived from Table Mountain sandstones would be upgraded relative to the same soil series derived from more clayey Dwyka tillites.

Further details are provided by Schulze (1984) and Schmidt and Schulze (1987b).

7.3.7. Land cover grouping

In the SCS-SA method, the effects of surface conditions of a catchment are evaluated by means of assessing the land cover, land treatment and stormflow potential:

(a) **Land cover:** Defines the primary catchment cover which can comprise a range of annual and perennial crops, veld, and forest, as well as non-agricultural cover such as water surfaces and urban or suburban conditions.

(b) **Land treatment:** Applies mainly to agricultural land uses and includes primarily mechanical practices such as conservation structures (e.g. contours, terraces, etc.) and management practices such as grazing control and rotation of crops.

(c) **Stormflow potential:** Stormflow potential is influenced by management practices. Three categories of stormflow potential are available, e.g. high, moderate, and low. In the context of agricultural crops, the use of conservation practices (e.g. minimum tillage) results in crop residue being left on the soil surface, which will result in a low stormflow potential. In the context of pasture or veld, a high stormflow potential would occur as a result of heavy grazing or recent burning (cover less than 50% of the area), while a low stormflow potential is associated with light grazing or plant cover on > 75% of the area. Under forest conditions, a high stormflow potential exists when undergrowth is sparse and there is a compact, shallow organic and litter layer (humus < 50 mm deep), while a low stormflow potential exists when undergrowth is dense and there is a loose, deep organic and litter layer (humus > 100 mm deep).

The following points should be noted when determining the *CN* values:

(a) The *CN* values listed in Table 7-10 are based on work conducted in the USA and do not cover all land use characteristics typically found in southern Africa. Hence, interpolation from similar land cover classes must be used in such cases.

(b) The *CN* values represent soil/land cover combinations for initial conditions of so-called "average" catchment wetness just prior to an event.

(c) The user should attempt to establish the land cover/treatment conditions likely to prevail in the catchment during the design life of the structure. Thus, the felling/cutting cycle of forest plantations or sugarcane would determine the percentage of the catchment area represented by a chosen *CN* value. Similarly, urban growth or projected changes in land cover and use should be accounted for in deriving the catchment *CN* value.

(d) Owing to future changes in land cover and use, which may result in an increase in the design stormflow, caution should be exercised when using *CN* values < 50.

(e) Due to the heterogeneous nature of the soils and land cover in a catchment, there could be a marked variation in *CN* values between sub-areas of a catchment. Such variations in *CN* must be accounted for by delineating a catchment into sub-catchments or HRUs, each with relatively homogeneous soil/land cover response characteristics. Curve Numbers from the sub-catchments should not be averaged and the runoff volume from the individual sub-catchments must be summed to obtain the total catchment runoff.

(f) The user is referred to Schmidt and Schulze (1987b) for more detail on the classification of land cover.

7.3.8. Adjustment of initial Curve Numbers

The *CN* value assigned to a particular soil and land cover condition is an index of stormflow response prior to consideration of the catchment soil moisture conditions. Since stormflow response is highly sensitive to the catchment's wetness, adjustments to *CN* values should be made for soil moisture status. Actual soil moisture conditions just prior to a rainfall event will depend upon the interrelationship of rainfall, drainage and evaporation amounts for a selected antecedent period preceding the rainfall event. Soil moisture conditions will thus be influenced by those properties of the soil which affect absorption of rainfall, the retention and redistribution of soil water, as well as those characteristics of vegetation which affect the drying out of the soil due to evaporation of water from the soil and plant transpiration.

A method to adjust the initial *CN-II* values to be more representative of conditions prior to design rainfall events in southern Africa, has been developed to account for regional climatic characteristics in conjunction with combinations of soil and vegetation properties. Runoff was simulated using for 712 relatively homogeneous climatic and hydrological response zones delineated in southern Africa by Dent *et al.* (1988) using the *ACRU* daily soil water budgeting model (Schulze, 1989) for a combination of soil and vegetation conditions, to determine typical soil moisture statuses prior to design storms. Three soil depth categories, (namely deep, intermediate, and shallow), three soil textural classes (namely sand, loam, and clay) and three vegetation cover conditions (namely dense, intermediate, and sparse) were used to categorise 27 soil/land cover combinations. Table 7-11 contains a summarised description of each category. More details regarding the methods used to adjust *CN* for prevailing soil moisture prior to rainfall events are provided by Schmidt and Schulze (1987b).

Table 7-11: Description of soil depth/texture classes and land cover conditions used in soil moisture status analyses (simplified after Schmidt and Schulze, 1987b)

<table>
<tr><th colspan="4">Soil depth categories</th></tr>
<tr><th>Category</th><th>Topsoil horizon depth (m)</th><th>Subsoil horizon depth (m)</th><th>Total depth (m)</th></tr>
<tr><td>Deep</td><td>0.30</td><td>0.80</td><td>1.10</td></tr>
<tr><td>Intermediate</td><td>0.25</td><td>0.50</td><td>0.75</td></tr>
<tr><td>Shallow</td><td>0.15</td><td>0.15</td><td>0.30</td></tr>
<tr><th colspan="4">Soil texture categories</th></tr>
<tr><td colspan="4">Sand (coarse textured): Well-drained, and low stormflow response.</td></tr>
<tr><td colspan="4">Loam (medium textured): Intermediately drained, and intermediate stormflow response.</td></tr>
<tr><td colspan="4">Clayey (fine textured): Poorly drained, and high stormflow response.</td></tr>
<tr><th colspan="4">Land cover categories</th></tr>
<tr><td colspan="4">Dense: Canopy cover is high, with high potential transpiration rates, and deep rooted.</td></tr>
<tr><td colspan="4">Intermediate: Canopy cover is medium, with medium potential transpiration rates, and intermediate rooting depth.</td></tr>
<tr><td colspan="4">Sparse cover: Canopy cover is low, with low potential transpiration rates, and shallow roots.</td></tr>
</table>

Based on the climatological database for the 712 zones and the 27 soil/land cover combinations, typical soil moisture related adjustments to the *CN* values can be made as follow:

(a) **Median Condition Method (MCM):** Estimates the soil moisture status that can be expected (statistically) to occur the most frequently (i.e. the median condition) prior to a design rainfall event in southern Africa.

(b) **Joint Association Method (JAM):** Accounts for simulated soil moisture conditions preceding those individual rainfall events which are considered for a design series of simulated stormflows in southern Africa.

<u>Median Condition Method (MCM):</u>

The median soil moisture status (i.e. that soil moisture content just prior to a selected storm event, the value of which is equalled or exceeded for 50% of the selected storm events), was determined using the five largest independent daily rainfall totals in each year of record. From this median soil moisture storage, a change from an initially assumed soil moisture storage was computed (ΔS).

The initial Curve Number *CN-II* from Table 7-10 was then adjusted to a *final* Curve Number, CN_f, based on the respective ΔS, using Eq. 7-17:

$$CN_f = \frac{1100}{\frac{1100}{CN\text{-}II} - \frac{\Delta S}{25.4}}$$ Eq. 7-17

When using the SCS-SA software (Schulze *et al.*, 2004) a user can select to adjust the *CN-II* into a CN_f which is then automatically done after this option is selected. Although not recommended, ΔS for a selected climate zone can be extracted from Schmidt and Schulze (1987c) and CN_f can be manually calculated.

<u>Joint Association Method (JAM)</u>:

The adjustment for a median antecedent soil moisture condition based on a large number of storms does not account for the effect which the event-by-event variation of soil moisture status can have when estimating design runoff responses. The "joint association" between the rainfall amount and the catchment moisture status may result in the second, third or even the fourth largest rainfall event of a year producing the largest flood, owing to specific soil moisture conditions prevailing just prior to the rainfall event. To account for this "joint association" between rainfall and catchment moisture status, the *CN* adjustment procedures discussed above were applied to the five highest independent daily rainfall events for each year of record obtained from the rainfall stations representing each of the 712 climate/hydrological response zones of southern Africa.

The stormflow response to each selected rainfall event for a range of initial *CN-II* values was computed after adjusting *CN-II* for prevailing soil moisture conditions prior to the specific event for each of the 27 soil/land cover combinations. A frequency analysis was performed on the resulting series of annual maximum daily stormflow depths to indicate the 50, 80, 90 and 95 percentile values of non-exceedance. These percentile values approximate the 2, 5, 10 and 20 year return period daily stormflow depths, respectively.

The results provide a direct estimate of stormflow depth, and hence volume, having already accounted for the joint association of rainfall depth and catchment moisture status. The stormflow series derived thus used the daily rainfall records of the rainfall station representing each climate response zone. Although most zones had a representative rainfall station with more than 50 years of daily rainfall data, a number of zones had stations with only a 30-year record. The frequency analyses were therefore presented only to the 95-percentile value (i.e. approximating the 20-year return period) and extrapolation beyond this return period is not recommended.

From the above, it is therefore recommended that the JAM be used only for $T \leq$ 20-year, while the MCM applies to all return periods.

7.3.9. Adjusted Curve Numbers for near-saturated catchment conditions

While the SCS-SA method was not developed to compute the Probable Maximum Flood (PMF), it may, with due caution, be used to estimate the PMF by deriving a probable maximum precipitation amount (values of which must be obtained from other sources) falling on a saturated or near-saturated catchment. Sobhani (1976) derived Eq. 7-18 to adjust an initial Curve Number *CN-II* to one for wet conditions, CN_w, namely:

$$CN_w = \frac{CN\text{-}II}{0.4036 + 0.0059\,CN\text{-}II} \qquad \text{Eq. 7-18}$$

CN_w is then used in Eq. 7-14 to compute Q_V from Eq. 7-13.

7.3.10. Estimation of one-day design rainfall depth

In the SCS-SA method, the 1-day design rainfall depth is used to compute the daily stormflow depth. This makes the SCS-SA method particularly attractive to users since daily rainfall data are widely available in southern Africa. Four options are included for input of 1 day design rainfall in the *Visual SCS-SA* software:

(a) Option 1 – By Rainfall Station Search: Uses at-site design rainfall estimated by Adamson (1981) for over 2 200 rainfall stations in southern Africa.

(b) Option 2 – From the Hydrological Response Zones' Representative Station: Use design rainfall computed for up to the 20-year return period estimated from the rainfall station assigned to the zone.

(c) Option 3 – User Entered Values for the Selected Return Periods: The user has performed probabilistic analyses (see Chapter 6) to estimate 1-day design rainfall using at-site rainfall data.

(d) Option 4 – Using the RLMA&SI approach (see Section 5.3.3).

The one-day design rainfall for a chosen frequency of recurrence is substituted as P_T into Eq. 7-13 to compute the resulting design stormflow depth. Given the limited length of rainfall records used in the analyses, Options 2 should not be used. Although outdated, Option 2 can be considered and if long and reliable rainfall records are available then Option 3 can be used. However, it is generally recommended that the RLMA&SI method (see Section 5.3.3), as incorporated in the *Visual SCS-SA* software, be used to estimate the design rainfall.

7.3.11. Time distribution of design rainfall intensity

Rainfall intensity has a marked effect on the timing and magnitude of the peak discharge in small catchments. Catchment response time (see Chapter 5, Section 5.10) will determine what storm duration, and hence rainfall intensity, is likely to produce the critical flood peak. A small catchment with a short response time will have critical flood peaks usually produced by short, high intensity storms, while a large catchment with a long response time, will have critical flood peaks produced by longer storm durations which are generally of lower intensity.

As highlighted in the previous section, the SCS-SA method makes use of 1-day design rainfall depth to compute the total stormflow depth (Eq. 7-13). This design rainfall depth is distributed over time in the course of a day as represented by a hyetograph. This time distribution depends on the synoptic conditions and rainfall mechanisms typically producing design storms in southern Africa. Four general types of time distribution curves of rainfall intensity have been determined for southern Africa from recorded rainfall data (Weddepohl, 1988). These synthetic time distribution curves are termed Types 1, 2, 3 and 4 and Figure 7-3 illustrates the spatial distribution of the four synthetic time distribution curves over southern Africa.

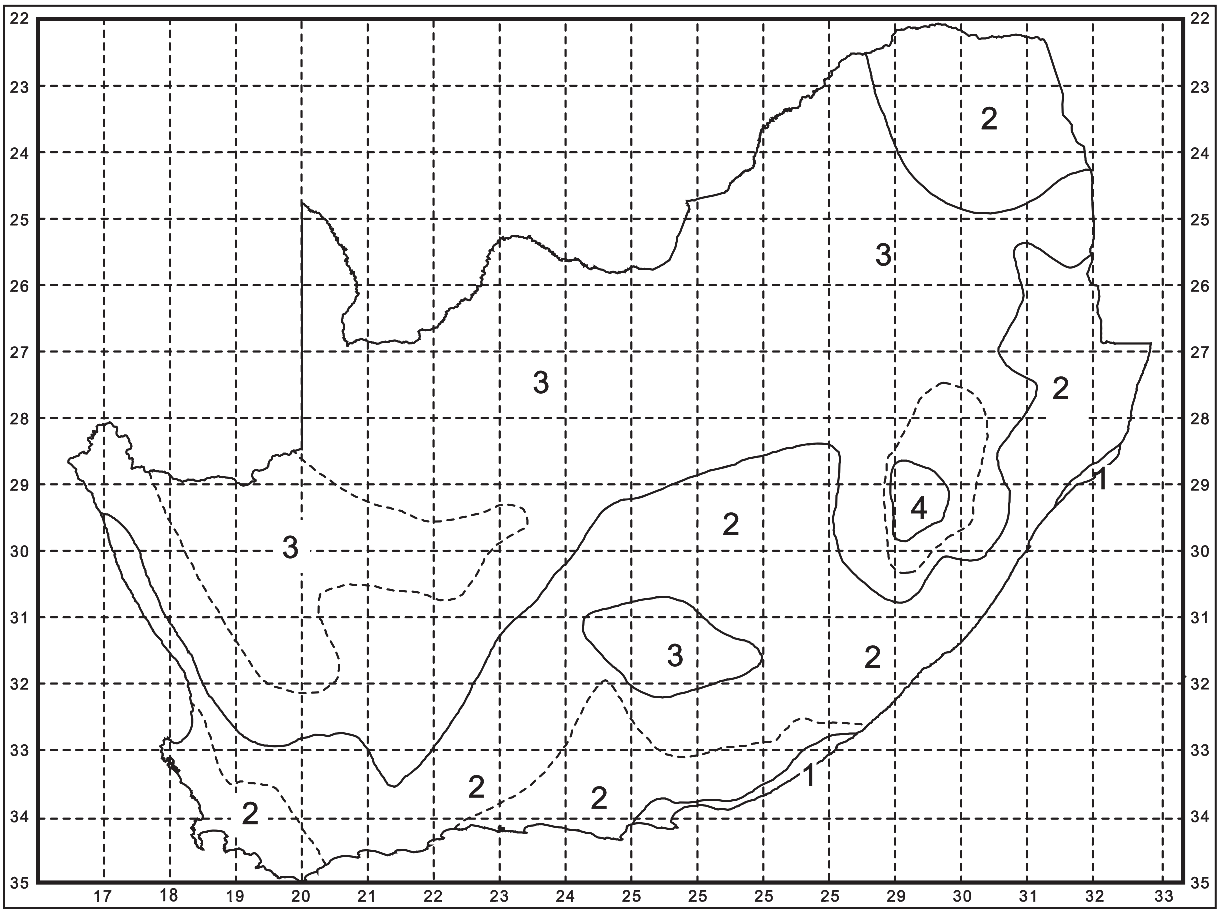

Figure 7-3: Design rainfall intensity distribution types over southern Africa (after Weddepohl, 1988)

The use of the synthetic time distributions of rainfall intensity assumes the total depth of 1-day rainfall for any duration to conform to the intensity-duration relationship for the region regardless of that duration. This allows for the use of the appropriate synthetic distribution for all catchments, regardless of the response time. An element of conservatism (i.e. a tendency to rather overestimate the peak discharge slightly) has been built into the procedures used to derive these time distribution curves (Schmidt and Schulze, 1987b), since it is unlikely that, for different durations, that individual rainfall intensities will correspond to the design rainfall intensities.

In the regionalisation of the four synthetic time distributions of rainfall intensity Figure 7-3 the Type 1 distribution contains the lowest rainfall intensities, representing rainfall produced often by a frontal or general rain situation, while the Type 4 distribution contains the highest rainfall intensities, typifying convective thunderstorms in which virtually all the day's design rain falls in a short duration. The synthetic time distributions are shown in Figure 7-4.

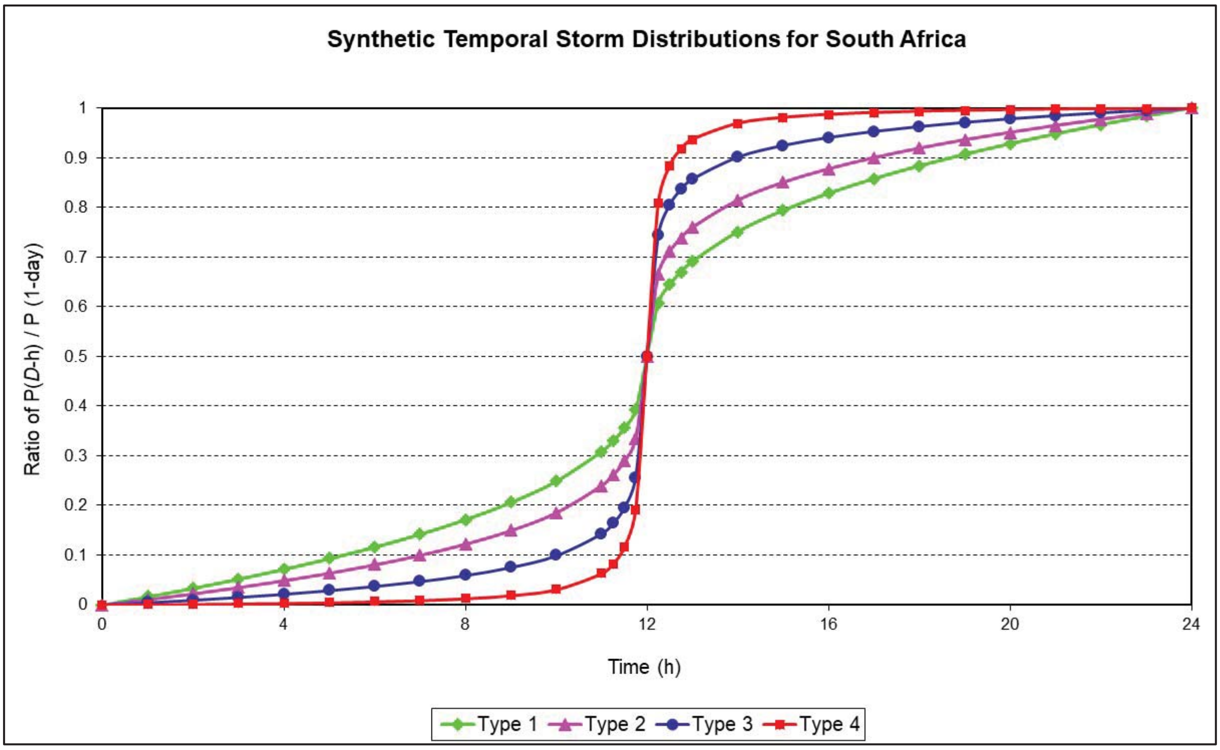

Figure 7-4: Time distribution of accumulated rainfall depths used in the SCS-SA method

7.3.12. Catchment response time

As highlighted in Chapter 5, Section 5.10, catchment response time, which is an index of the rate at which the generated stormflow moves through a catchment, is an important factor in determining the timing and magnitude of the peak discharge, and hence the hydrograph shape. In the SCS-SA method, lag time (T_L) (see Section 5.10.2) is used as an index of the catchment's response time and expressed as a proportional ratio of the time concentration (T_C). Four options are available in the *Visual SCS-SA* software to estimate lag time:

(a) Time of concentration (see Section 5.10.1 for the detailed discussion).

$$T_{L1} = 0.6T_C \qquad \text{Eq. 7-19}$$

(b) Summation of travel times along flow path reaches (see Section 5.10.1, with specific reference to the NRCS methods based on Eqs. 5-10 to 5-12). Average flow velocities can also be estimated from Figure 7-5 using the so-called Uplands nomograph. The travel times (T_C) can then be converted to T_L using Eq. 7-19.

(c) SCS lag equation (see Eq. 5-16) listed as Eq. 7-20:

$$T_{L2} = \frac{L_H^{\ 0.8}\left[\frac{25\,400}{CN} - 228.6\right]^{0.7}}{168.862\,S^{0.5}} \qquad \text{Eq. 7-20}$$

(d) Schmidt-Schulze SCS-SA lag equation (see Eq. 5-17) listed as Eq. 7-21:

$$T_{L3} = \frac{A^{0.35}MAP^{1.10}}{41.67\,S^{0.3}i_{30}^{\ 0.87}} \qquad \text{Eq. 7-21}$$

where: $T_{L1\text{-}3}$ = lag time (minutes/hours),

A = catchment area (km²),

CN = runoff curve number,

i_{30} = 2-year return period 30-minute rainfall intensity (mm/h),

L_H = hydraulic length of catchment (km),

MAP = mean annual precipitation (mm), and

S = average catchment slope (m/m; see Section 5.6.5).

It is recommended that the i_{30} and *MAP* values be obtained from the RLMA&SI software (see Section 5.3.3). Alternatively, details on estimating the i_{30} rainfall intensity using the 2-year return period 1-day design rainfall depth and regional multiplication factors are contained in Schulze *et al.* (2004) and the *MAP* can be determined from Lynch (2004) or Kunz (2004). When using Eq. 7-21 in the *Visual SCS-SA* software, the values of *A*, *S* and *MAP* are input parameters/variables and $i_{30,2}$ is computed automatically.

The use of empirical equations should be restricted to catchments where hydraulic calculations of flow velocity for various reaches cannot be made. Equation 7-21 should be used in preference to Eq. 7-19 when the stormflow response comprises of both surface runoff and a subsurface component, which occurs frequently in areas of relatively high MAP, or on natural catchments with a good surface cover or in some urban catchments. Equation 7-21 is recommended to be more suited for use in semi-arid and arid areas with limited vegetative cover and shallow soils.

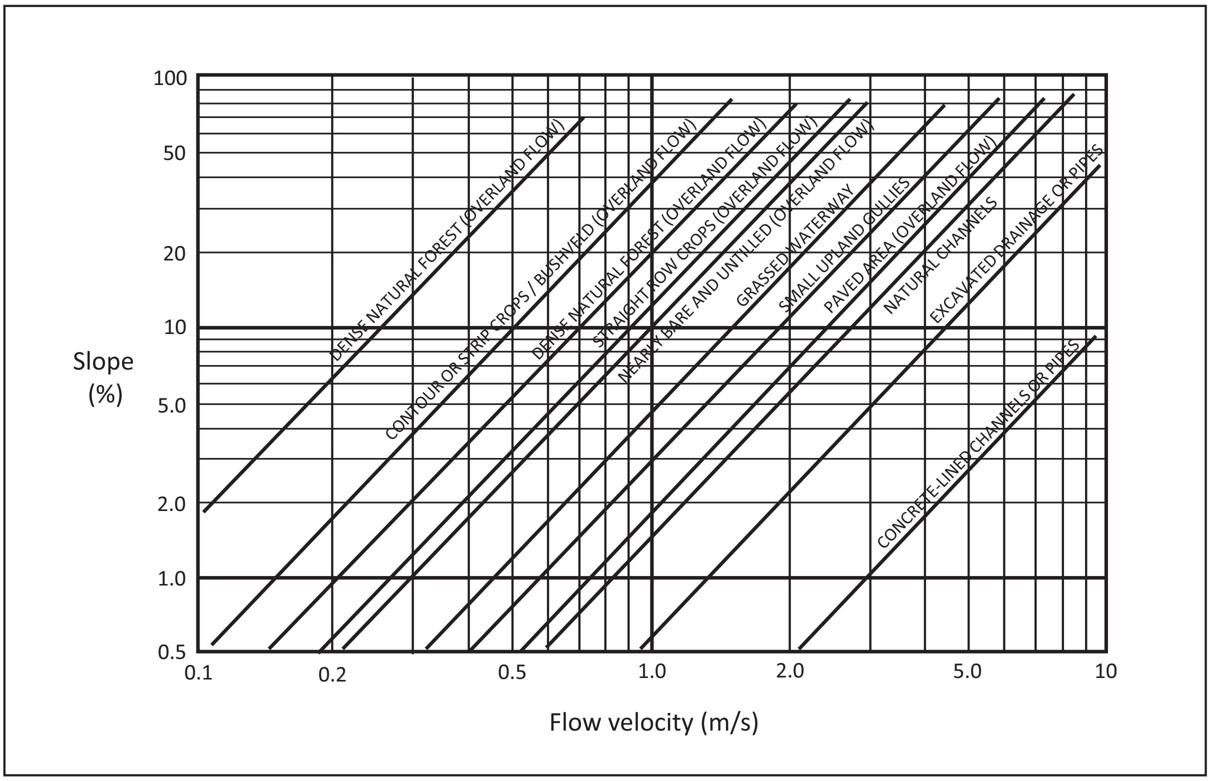

Figure 7-5: Uplands nomograph for estimating flow velocities (Schulze and Arnold, 1979)

7.3.13. Estimation of peak discharge

The estimation of peak discharge by the SCS-SA method is based on the triangular unit hydrograph concept. This unit hydrograph represents the temporal distribution of stormflow for an incremental unit depth of stormflow, ΔQ_V, occurring in a unit duration of time, ΔD, where D is normally assumed to be equal to T_C. Assuming a triangular shaped hydrograph with the time to peak being 3/8 of the total hydrograph base length, the peak discharge for a storm with a uniform rainfall distribution with respect to time may be derived to be:

$$Q_T = \frac{0.2083 A Q_v}{\frac{T_C}{2} + T_L} \qquad \text{Eq. 7-22}$$

where: A = catchment area (km²),

Q_T = peak discharge for T-year return period (m³/s),

Q_V = stormflow depth (mm),

T_C = time of concentration; assumed as equal to the storm critical storm duration (D; hours), and

T_L = lag time based on either Eqs. 7-19, 7-20, or 7.21.

In order to account for the non-uniformity of rainfall intensity during a storm event, it is necessary to divide the storm into increments of shorter duration (ΔD *or* ΔT_C) and to compute the corresponding increment of runoff, as shown in Eq. 7-23:

$$\Delta q_P = \frac{0.2083 A \Delta Q_v}{\frac{\Delta D}{2} + T_L} \qquad \text{Eq. 7-23}$$

where: Δq_P = peak discharge of incremental unit hydrograph (m^3/s),

ΔQ_V = incremental stormflow depth (mm), and

ΔD = unit incremental duration of time (h), used with the distribution of daily rainfall to account for rainfall intensity variations (see Section 5.3.5).

As illustrated in Figure 7-6, the runoff hydrograph response to a given rainfall total is determined by superimposing incremental hydrographs based on the time distribution of rainfall intensity and the stormflow response characteristics of the catchment. This approach can only be realistically applied using detailed spreadsheets or by using the *Visual SCS-SA* software. The determination of the time distribution of design rainfall intensity and catchment response time in Eq. 7-22 is detailed in Sections 7.3.11 and 7.3.12, respectively.

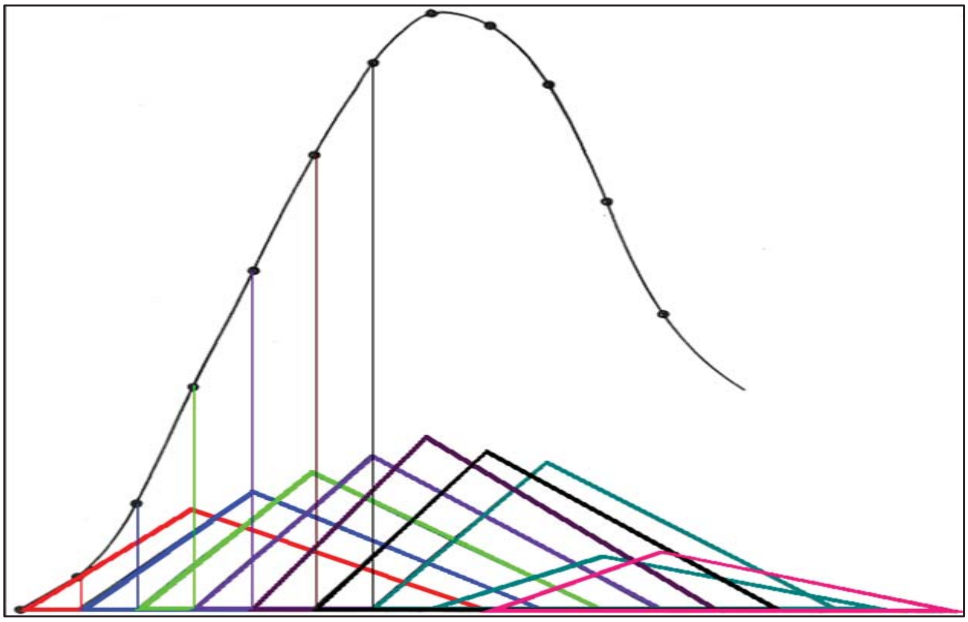

Figure 7-6: Schematic representation of hydrograph generation using incremental unit hydrographs in the SCS-SA method

7.4 PC-SWMM

Since SWMM, and by extension PC-SWMM, are used for both event based computations and continuous simulation modelling (CSM) and because almost all of the data requirements and model procedures are identical, the reader is referred to Chapter 8 for further discussion.

The only significant difference between CSM and event modelling using SWMM is the duration of the input rainfall and hence the model run time. When considering a single event the hyetograph duration is typically 24 hours or less and the model is run for a similar length of time. Continuous simulation uses hyetographs with durations of weeks to decades and the run durations are similarly long.

7.5 RMF Methods

7.5.1. Introduction

The Regional Maximum Flood (RMF) methods are empirically derived upper limit envelopes derived from observed maximum flood peaks in a region. The RMF is only a function of the catchment area and location within a defined hydrologically homogeneous region (SANRAL, 2013).

The RMF can be based on two different approaches, which provide comparable or similar estimates:

(a) **Franco-Rodier (1967):** A total of 1 200 maximum flood peaks representative of most regions in the world were plotted against catchment areas to develop a family of flood envelope curves as illustrated in Figure 7-7. The regional flood envelope curves become straight lines for catchment areas exceeding 100 km² and converge to a single point where the runoff and area respectively represents the total world mean annual runoff and total world catchment areas.

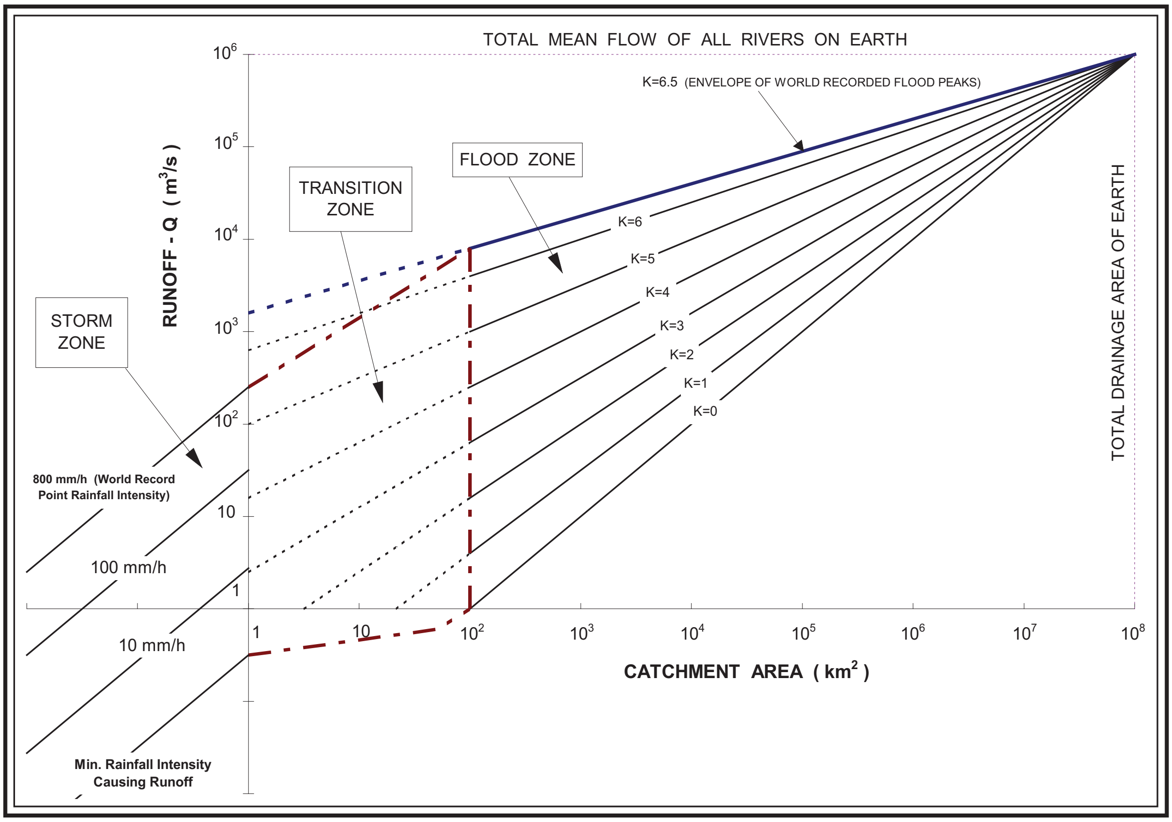

Figure 7-7: Franco-Rodier upper limit flood envelope curves (Van Der Spuy and Rademeyer, 2021)

Three flood envelope zones are evident in Figure 7-7: (i) Storm zone (A < 1 km² and $T_C \leq$ 15 minutes), (ii) a transition zone 1 km^2 < A < 100 km^2 between the storm zone and the flood zone, and (iii) the flood zone (A > 100 km² and the flood peak depend on the spatial and temporal rainfall distribution and catchment characteristics.

The Franco-Rodier equation (Eq. 7-24) is applicable to the flood zone (ii), and (iii) transitional zone (1 ≤ A ≤ 100 km²: between storm and flood zones).

$$Q_{RMF1} = 10^6\left(\frac{A}{10^8}\right)^{1-0.1K} \quad \text{Eq. 7-24}$$

(b) **Kovács (1988):** Eight hydrologically similar regions (Kovács regions; Figure 7-8) were delimited and associated regional envelope curves with flood, transitional and storm zones were developed based on a joint consideration of the regional **K**-values, maximum observed 3-day rainfall, catchment characteristics and 519 observed flood peaks. Table 7-12 presents all the Q_{RMF2} equations as proposed by Kovács (1988) for the different Kovács regions in Southern Africa.

Table 7-12: RMF regional classification in Southern Africa (SANRAL, 2013)

Regional constant (K)	Transition zone		Flood zone	
	Q_{RMF2} (m³/s)	Areal range (km²)	Q_{RMF2} (m³/s)	Areal range (km²)
2.8	$30A^{0.262}$	1-500	$1.74A^{0.720}$	500-500 000
3.4	$50A^{0.265}$	1-300	$5.25A^{0.660}$	300-500 000
4	$70A^{0.340}$	1-300	$15.9A^{0.600}$	300-300 000
4.6	$100A^{0.380}$	1-100	$47.9A^{0.540}$	100-100 000
5	$100A^{0.500}$	1-100	$100A^{0.500}$	100-100 000
5.2	$100A^{0.560}$	1-100	$145A^{0.480}$	100-30 000
5.4	$100A^{0.620}$	1-100	$209A^{0.460}$	100-20 000
5.6	$100A^{0.680}$	1-100	$302A^{0.440}$	100-10 000

where:
A = catchment area (km²),
K = Kovács regional constant,
Q_{RMF1} = Franco-Rodier RMF (m³/s), and
Q_{RMF2} = Kovács RMF (Equations in Table 7-12).

Table 7-13: Q_T/Q_{RMF} (K_T) ratios for different regions and catchment areas in South Africa, Lesotho and Swaziland (SANRAL, 2013)

Region	T (years)	K_T	Effective catchment area (A_e, km²)									
			10	30	100	300	1 000	3 000	10 000	30 000	100 000	300 000
K8	50	5.060	0.537	0.508	0.474	0.503	0.537	0.570	0.607			
	100	5.250	0.668	0.645	0.617	0.640	0.668	0.695	0.724			
	200	5.410	0.803	0.788	0.769	0.784	0.803	0.821	0.838			
K7	50	4.700	0.447	0.416	0.380	0.411	0.447	0.482	0.523			
	100	4.890	0.556	0.525	0.492	0.523	0.556	0.588	0.623			
	200	5.040	0.661	0.635	0.607	0.633	0.661	0.687	0.716			
K6	50	4.500	0.447	0.416	0.380	0.411	0.447	0.482	0.526	0.566		
	100	4.690	0.556	0.528	0.494	0.524	0.556	0.588	0.626	0.660		
	200	4.860	0.676	0.650	0.624	0.650	0.676	0.701	0.733	0.758		
K5	50	4.300	0.447	0.416	0.380	0.411	0.447	0.482	0.525	0.567	0.617	
	100	4.480	0.550	0.521	0.488	0.517	0.550	0.582	0.619	0.657	0.699	
	200	4.640	0.661	0.636	0.608	0.633	0.661	0.687	0.718	0.748	0.780	
K4	50	3.840	0.416	0.385	0.350	0.381	0.416	0.453	0.496	0.541	0.591	
	100	4.040	0.524	0.495	0.462	0.491	0.524	0.558	0.597	0.636	0.679	
	200	4.200	0.629	0.603	0.576	0.602	0.629	0.660	0.692	0.724	0.758	
K3	50	3.260	0.426	0.426	0.426	0.390	0.426	0.463	0.506	0.548	0.602	0.651
	100	3.500	0.562	0.562	0.562	0.529	0.562	0.595	0.631	0.666	0.710	0.749
	200	3.680	0.692	0.692	0.692	0.665	0.692	0.718	0.745	0.771	0.804	0.831
K2	50	2.400	0.317	0.317	0.317	0.281	0.317	0.353	0.398	0.444	0.500	0.560
	100	2.660	0.428	0.428	0.428	0.391	0.428	0.463	0.506	0.549	0.598	0.651
	200	2.910	0.570	0.570	0.570	0.536	0.570	0.600	0.638	0.672	0.710	0.753
K1	50	2.400	0.317	0.317	0.317	0.281	0.317	0.353	0.398	0.444	0.500	0.560
	100	2.660	0.428	0.428	0.428	0.391	0.428	0.463	0.506	0.549	0.598	0.651
	200	2.910	0.570	0.570	0.570	0.536	0.570	0.600	0.638	0.672	0.710	0.753

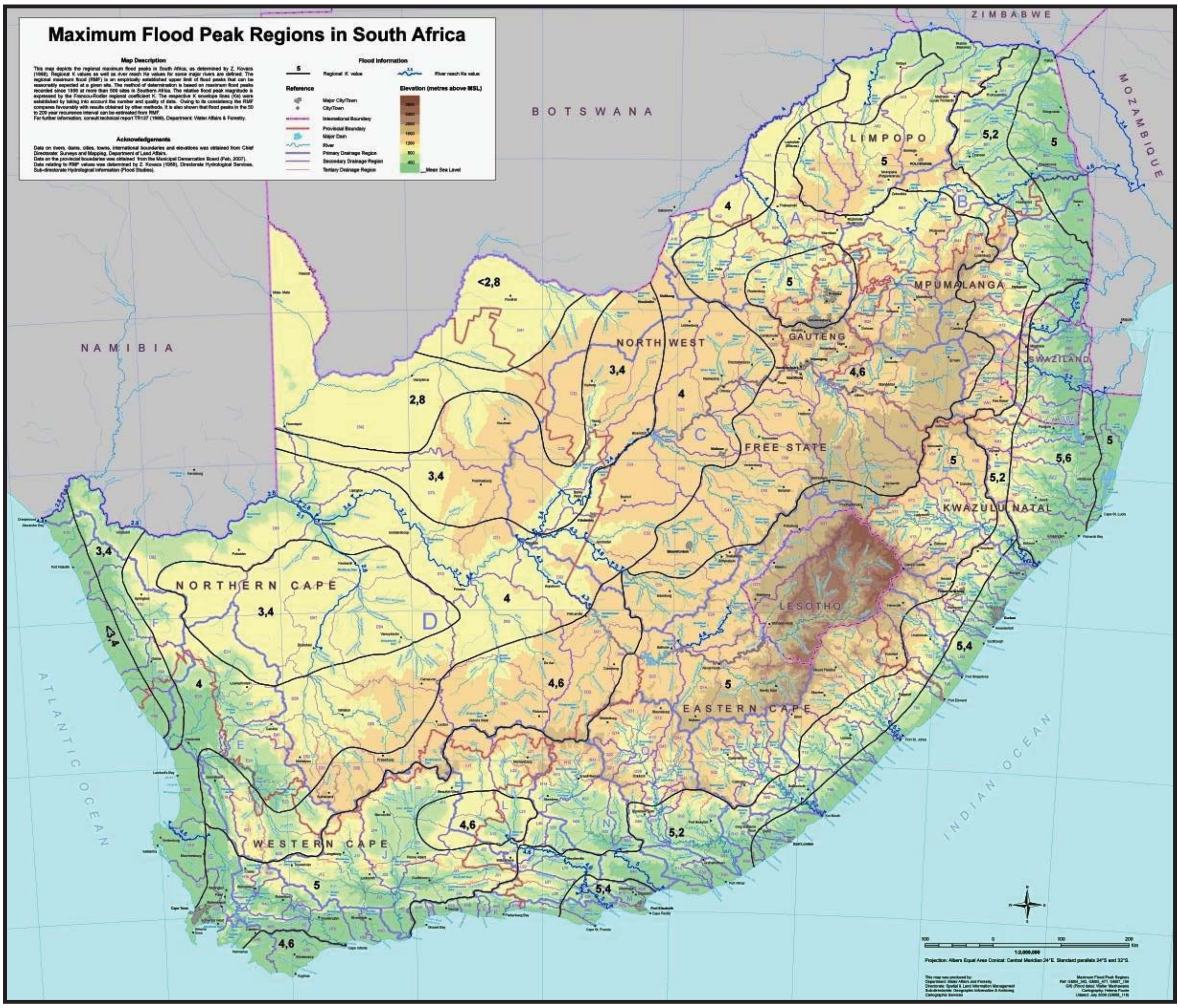

Figure 7-8: Maximum flood peak (Kovács) regions in Southern Africa (Van Der Spuy and Rademeyer, 2021)

Given that the RMF only presents upper limit flood peaks that have been observed in a region, Kovacs (1988) proposed an unorthodox analysis of the K-values and representative, independent flood peaks to provide Q_T/Q_{RMF} ratios to express the 50 to 200-year flood peaks as a fraction of the RMF values estimated with the equations listed in Table 7-12. The Q_T/Q_{RMF} (K_T) ratios are listed in Table 7-13 and are dependent on both the region and effective catchment area.

7.5.2. Limitations and assumptions

The primary assumption of the Franco-Rodier method is that the magnitude of a flood is not only dependent on the rainfall retention characteristics of a catchment (area and slope), but also on factors such as possible limits on extreme rainfall. These extreme rainfall limits can be expected to be controlled by regionally dominant weather systems, while average catchment slope would also be regionally coherent. It was thus established that regional upper envelopes/limits to extreme flood peaks, as well as regional relationships between area and extreme flood peaks, are plausible.

A disadvantage of the RMF method is that it is not linked to a return period or probability of exceedance as it is estimated directly from the upper envelope of observed flood peaks in a region. Kovács (1988) estimated the return period of the RMF to be greater than 200 years and developed a methodology to estimate design peak discharges as a ratios (fractions) of the RMF. An analysis by Pegram and Parak (2004) estimated that the return period of the RMF was closest to the 200 year return period value. However, Görgens (2002) indicated that Kovács' method of estimating the design peak discharges was too simplistic and recommended that the 50-, 100- and 200-year ratios must be factored down by 0.7, 0.8 and 0.9, respectively. This implies that the RMF peaks have return periods that are actually much larger than 200 years, as opposed to the original estimation of Kovács and hence the RMF approach to DFE can be seen as a conservative approach to the upper limit flood estimates.

7.5.3. Input data requirements

The input data requirements of the RMF method are summarised in Table 7-14 which is a shortened version of the information as included in Chapter 4 (*see* Section 4.7).

Table 7-14: RMF method input data requirements

Method	Input data/parameters	Cross Reference	Source
RMF	*A* and *K*.	Sections 5.6 and 7.5	• Local Digital Terrain Model (DTM) data • Geographical Information System (GIS) data. • Digital Elevation Model (DEM) data. • SANRAL (2013).

Note: Catchment area (*A*, km^2), and Kovács regions (*K*, Figure 7-6).

7.6 Event-based DFE Software

In considering the current availability data and information and of technological advances in improved computing power and various software developments in hydrology, the use of software in the application of DFE methods is encouraged to improve the design of hydraulic structures. With the aid of Hydro-informatics, several design flood estimation software packages, in addition to the commercially available event modelling and CSM software packages, have been developed internationally to apply the understanding of the fundamental hydrological processes involved and to complement various user manuals and guidelines (e.g. CEH, 2008; Stewart *et al.*, 2009; Seibert and Vis, 2012; Piscopia *et al.*, 2015; ARR, 2016)

In South Africa, the development of event-based DFE software is limited to the Visual SCS-SA (Schulze *et al.*, 2004), the Utility Programs for (UPD, Van Dijk, 2005) and the Design Flood Estimation Tool (DFET, Gericke, 2010; Gericke and du Plessis, 2013; Gericke, 2021) software packages. For the most part, these packages provide computer based assistance to the implementation of the manual "paper based" methods discussed elsewhere and do not fall into the same category as sophisticated, discretised models such as Mike Urban or SWMM that can be used for either single event or continuous simulation computation.

The PC-based SCS software utilises the well-known *CN* approach developed by USDA (1985) to estimate both the design peak discharge and runoff volume in catchment areas < 30 km². The Visual SCS-SA software is an update of the latter software and is driven by a Graphical User Interface (GUI) within the Windows operating environment.

The UPD software (http://www.sinotechcc.co.za/Software/UPD/upd.html) complements the Drainage Manual (SANRAL, 2013), which is regarded as an authoritative text on DFE in South Africa and consists of a number of modules for event-based DFE and the hydraulic design of drainage structures. However, no catchment parameter estimation functionalities are available in the UPD software, and the design rainfall information is limited to the TR102 daily design rainfall database (Adamson, 1981). The software developer has however indicated that an updated version is due for release in the near future (van Dijk, 2023, pers. com)

The DFET Version 1.4 (see Figure 7-9) (https://data.waterresearchobservatory.org/)

is a spreadsheet-based utility which includes of all the DFE methods currently used in South Africa. The powerful data management framework enables the organisation and estimation of both catchment parameters and design rainfall using the latest information and technology.

Most of the South African DFE software packages are periodically updated in an attempt to overcome some of the inherent limitations associated with the DFE methods currently used in South Africa, as well as to remain somehow relevant in an international context to ultimately enhance the practitioners' decision-making process. Practitioners also play a pivotal role in the validation of these software packages by means of comparisons using either hand-calculations or other relevant software. Regular feedback is also given at Continuous Professional Development (CPD) courses administrated and accredited by the Engineering Council of South Africa (ECSA). Furthermore, the initiation and implementation of the National Flood Studies Programme (NFSP) in South Africa (Smithers *et al.*, 2014), also support the development and maintenance of such DFE software tools.

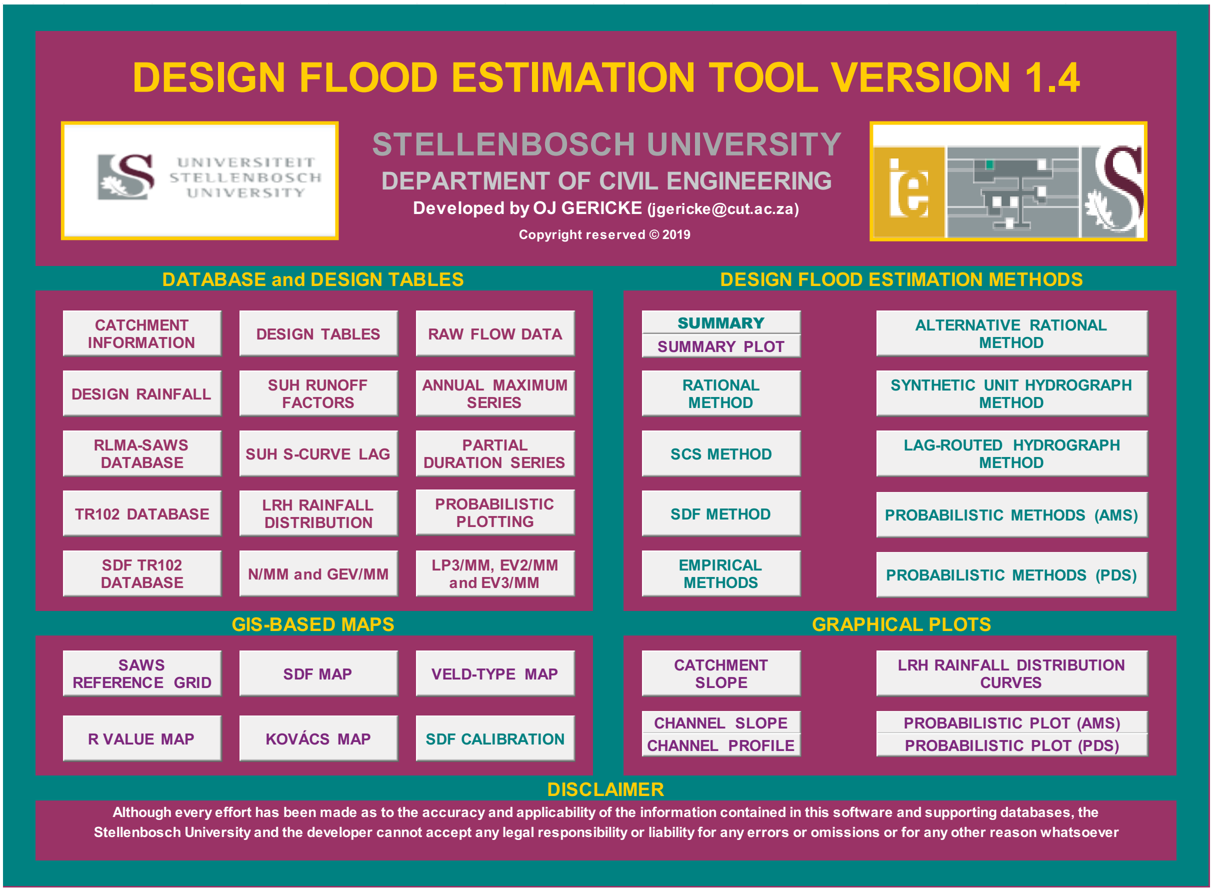

Figure 7-9: DFET Version 1.4 Home page (after Gericke, 2021)

7.7 Performance of Methods

7.7.1. Single site

Gericke and du Plessis (2013) assessed the performance of various DFE methods in gauged catchments ranging from 100 km² to 10 000 km² in the C5 secondary drainage region, South Africa. The results showed that the simplified "small catchment" ($A \leq 15$ km²) deterministic DFE methods, e.g. RM and SCS method, provided acceptable results when compared to the probabilistic analyses applicable to all the catchment sizes and return periods under consideration, except for the 2-year return period. Less acceptable results were demonstrated by the "medium catchment" (15 km² $< A \leq$ 5 000 km²) deterministic, e.g. Synthetic Unit Hydrograph (SUH) and Lag-routed Hydrograph (LRH), and "large catchment" (> 5 000 km²) empirical DFE methods.

Naidoo (2020) assessed the performance of DFE methods applied at DWS by using synthesised dam inflow data from 157 dam sites in South Africa with catchment areas ranging from 10 to 108 360 km^2. In terms of the mean absolute relative error (MARE) values, the RM, SUH and the empirical Catchment Parameter (CAPA) method ranked amongst the top three best performing methods, with the average MARE values ranging between 56% and 82%.

The poorest performance was demonstrated by the Standard Design Flood (SDF) method, (average MARE = 294%). However, these "acceptable" estimates using the top three methods were only evident at approximately 40% of the sites under consideration, whilst spatial mapping of methods performance resulted in no identifiable regional trends in the performance.

In applying the DFET Version 1.4, Gericke (2021) used a ranking-based selection procedure to assess the performance of all the event-based DFE methods used in South Africa by considering 48 gauged catchments (22 km² ≤ A ≤ 31 283 km²) located in four climatological regions. The SCS, RM and CAPA methods provided the best estimates of the at-site probabilistic flood peaks, while the SDF method proved to be the least appropriate.

More recently, the performance of the SCS-SA methods has been assessed using the *CN*s as published in the literature (Dlamini, 2019; Maharaj, 2020; Smithers *et al.*, 2021). The results highlighted that the published *CN*s are not representative and should be updated accordingly.

7.7.2. Regional

A number of Regional Flood Frequency Analysis (RFFA) methods have been developed, which cover all or parts of South Africa. These include methods developed by Van Bladeren (1993), Meigh *et al.* (1997), Mkhandi *et al.* (2000), the Joint Peak-Volume (JPV) method (Görgens, 2007), and Haile (2011). The performance of these methods has been assessed at 41 selected flow-gauging sites in the province of KwaZulu-Natal (KZN) in South Africa (Smithers *et al.*, 2015) and at 318 flow-gauging stations and 89 synthesised dam inflow records located throughout South Africa (Nathanael *et al.*, 2018). Smithers *et al.* (2015) found that for KZN, the JPV method associated with a regionalised GEV distribution and SUH veld zone regionalisation, generally demonstrated the best performance. Nathanael *et al.* (2018) reported that the Haile method generally performs better than the other RFFA methods; however, it also consistently underestimates the design floods. Due to the poor overall performance of the RFFA methods assessed, both studies recommended that a new RFFA method should be developed for application in design flood practice in South Africa.

7.8 Emerging New Developments

As part of the NFSP, a number of new event-based approaches to DFE have been developed and users should be aware of these and consider their application when they are ready. These include the following:

(a) **Extreme design rainfall:** As part of a revision to estimate extreme rainfalls in South Africa, a cluster analysis has been used to identify 17 relatively homogeneous rainfall regions in South Africa using data from 1 641 daily rain gauges (Johnson, 2021). A generalisation of L-moments, commonly referred to as LH-moments, were found to have better fit at 68% of the sites compared to the L-moments. Empirical Bayesian Kriging was used to estimate the design values at ungauged sites. It should be noted that 60% of the updated design rainfall values exceeded the RLMA&SI values for the T = 200-year, 1-day event, with an average difference of 13%.

(b) **One-day Probable Maximum Precipitation (PMP):** PMP values have been updated using 380 representative rainfall stations in South Africa (Johnson and Smithers, 2020). The updated PMPs exceeded the currently used PMPs (HRU, 1972) at 80% of the sites under consideration. This result is ascribed both to the longer period of records used in the study with 70% of the extreme events used in the study occurring after the HRU (1972) study, and that an updated World Meteorological Organisation (WMO) approach was used to estimate PMPs.

(c) **Catchment response time:** The development of regionalised methods to estimate catchment response times has been undertaken in four different climatological regions in South Africa (Gericke, 2015; Gericke and Smithers, 2016a; 2017; Gericke and Smithers, 2018).

(d) **ARFs:** A national-scale study has recently been completed to estimate geographically-centred and probabilistically correct ARFs representative of the different rainfall regions associated with the RLMA&SI regionalisation scheme in South Africa (Gericke *et al.*, 2022). The 78 homogeneous RLMA&SI rainfall clusters were merged into 46 clusters to increase the size of the clusters and the number of rainfall stations within a particular cluster. Long duration geographically-centred and probabilistically correct ARFs were estimated using a total of 2 053 artificial circular catchments and 1 779 daily rainfall stations located within the 46 clusters. Subsequently, five (5) ARF regions were deduced from the 46 clusters and all clusters in a particular ARF region were used for the final derivation of a non-linear (second-order polynomial) log-transformed empirical ARF equation. The new regional ARF equation performed similarly, and as expected, when compared to a selection of geographically-centred ARF estimation methods currently used in local and/or international practice in a range of catchment sizes. A web-based software application (https://data.waterresearchobservatory.org/) was developed to enable the consistent estimation of ARFs within the five (5) ARF regions (Gericke *et al.*, 2022).

(e) **RMF method:** An update to the estimation of the RMF method is being completed (Msasule, 2022).

(f) **Hydrological response in urban catchments:** A study is currently underway to improve the understanding of hydrological responses and flood estimation for different levels and types of urban development in South Africa (Loots, 2017). From the pilot study in two urban catchments in Tshwane, it was concluded that the parameters currently used for runoff simulation using SWMM in South African urban areas do not provide accurate results in gauged catchments when compared to the observed flow data. In addition, the impact of urban development on the hydrological responses from eight South African urban catchments has shown that both total runoff and baseflow volumes increase with increased developmental levels, while statistically insignificant trends in flood peaks were evident in most catchments. However, there is an increasing trend in flood peaks where catchments are subjected to progressive urban growth linked to informal settlements (Loots *et al.*, 2022). Calibrated *CN* values from the above-mentioned catchments were used to extrapolate preliminary SWMM parameter values and *CN*s for urban DFE in ungauged catchments in South Africa.

(g) **Ensemble SCS-SA:** An ensemble approach to the SCS-SA model has been developed in a pilot study with improved model performance, while confidence intervals for the design values were explicitly estimated (Dlamini, 2019; Smithers *et al.*, 2021). In general, the ensemble SCS-SA model demonstrated the ability to reproduce observed design flood estimates with reasonable accuracy over a wide range of return periods and for larger than anticipated catchment areas.

(h) **Best practices for *CN* derivation:** In a pilot study, the best methods to derive *CN*s to replicate values from published *CN* tables, or for best model performance, have been established. The *CN*s linked to South African land cover and soil classifications can be derived from simulated data. The data derived *CN*s resulted in the best SCS-SA model performances (Maharaj, 2020; Smithers *et al.*, 2021).

(i) **Outliers:** The best methods to detect outliers and the sensitivity of estimated design events to the presence of outliers, selection of probability distributions, record length and network density, have been investigated (Johnson *et al.*, 2021b; Singh, 2021).

(j) **Best probability distributions for South Africa:** In using observed flow data from 296 river flow-gauges and 87 dam inflow sequences, it was found that the best performing probability distributions in South Africa for the estimation of design peak discharges are the Generalised Pareto (GPA), Kappa-3 (KAP3), and LP3. As a result, the GPA fitted by L-moments was recommended for general use for DFE in South Africa (Calitz, 2020; Calitz and Smithers 2020).

(k) **Homogeneous flood regions:** In using a cluster analysis and flow data from 296 river gauges and 87 dam inflow sequences, 42 relatively homogeneous flood regions have been identified in South Africa (Calitz, 2020; Calitz and Smithers 2020).

(l) **Regional DFE techniques:** In using the 42 relatively homogeneous flood regions, three regionalised DFE models have been developed: (i) Quantile Regressions (QRT), (ii) Regional Index Flood (RIF), and (iii) Probabilistic Rational (PRM). The RIF and QRT were the best performing methods in the homogenous clusters. However, the application of the QRT method is limited to the return periods used in the model development. Hence, the RIF method was recommended for general application South Africa, with some caution exercised on the east coast of KZN (Calitz, 2020; Calitz and Smithers 2020).

(m) **Local information from donor sites:** In a pilot study using flow data from 48 sites in the north-eastern part of South Africa, the use of local information from one or more gauged donor sites and a simple approach has been shown to generally improve design flood estimates at ungauged sites (Khoosal, 2021).

7.9 Chapter References

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8. COMPUTER MODELS AND CONTINUOUS SIMULATION MODELLING

8.1 Background

Computing capacity has increased rapidly in recent years and is now accessible to almost all municipal engineers and practitioners. As a result, many of the complex processes inherent in catchment flood responses, but substantially simplified in conventional DFE methods, can now be considered in the design flood analysis.

Stormwater modelling software, designed for simulation and design of the hydraulic performance of drainage networks, represents many of the processes at a higher level. Design flood estimation is one of the outputs from these models which have largely changed the approach to DFE in municipal catchments. The four processes used in network analysis are the same required for design flood analysis:

(a) Rainfall input.

(b) Surface runoff estimation (“loss model”).

(c) Overland flow.

(d) Network conveyance.

Of these processes, (c) and (d) are usually combined in conventional DFE methods unless a discretised (distributed) catchment approach is considered. In stormwater models there are still simplifications in each of the processes, but computational capacity allows for substantial improvements in representing the natural complexity of hydrological responses. Among the most important are:

- It is possible to represent the variable temporal and spatial characteristics of rainfall.
- Similarly, the spatial variation of the physical characteristics of a catchment can be represented. This improves both the loss model calculations for generating surface runoff, and the overland flow calculations as runoff passes over different surfaces (and different slopes).
- Stormwater models are designed to analyse sections of drainage networks. Model discretisation to sub-catchments is easily done. This also requires the hydraulic routing of runoff through the network, and it provides a more accurate representation of the timing of the flood wave at a point in the catchment. Most stormwater models now run unsteady (time variant) hydraulic simulation that will reflect the natural attenuating effect of the drainage network on the flood peak. This aspect is absent from lumped catchment analyses using event-based DFE methods.

- As a result, model simulations are run at very short time-steps (minutes or seconds), allowing time variant processes to be reflected in detail.

Concerns that there is too much data preparation required to run stormwater models for DFE appear to be diminishing if the increase in licences for stormwater software is a measure. Currently there are over 200 licenses for PCSWMM in South Africa (CHI 2023 pers. comm.). Most are expected to have converted from the free SWMM software of which there may be many more users in the country. Furthermore, the availability of detailed spatial data is increasing rapidly and GIS techniques for integration at catchment and sub-catchment levels enable rapid incorporation into models.

Stormwater models have the potential for multiple uses. They can be used for single event analysis of an event of a given probability or used for continuous simulation of a combination of events or a full rainfall time series. The objectives and benefits of continuous simulation are outlined in sections below. They are also useful for "before and after" scenarios as are often required under local stormwater bylaws. The analysis not only reflects the effect of land cover and land use changes, but also the response times with changes of conveyance in the drainage network. The detail in these comparisons can have important implications on design solutions.

Another consideration is the benefit of design flood *simulation* over design flood *estimation*. Deterministic models will always give the same result for the same set of input parameters and ignores the effect of natural variability (this is inherent in the assumptions for the methods). This applies equally to modern stormwater models and conventional DFE methods. However, as indicated above the modern stormwater models reflect a higher level of hydrological complexity combined with hydraulic functions that also influence flood peak flow. They are designed for simulation and therefore enable analysis and testing of the performance of a drainage network under different conditions that can include the effects of the natural variability of any part of the process. This capability provides important support to the practitioner looking to prepare a defendable case for the recommendation of a flood peak or flood hydrograph for flood risk or design applications.

Hence, the decision whether to use a stormwater model, such as the SWMM model described below, or one of the conventional DFE methods remains with the professional DFE practitioner. The method(s) selected must be fit-for-purpose.

At the same time, it is incumbent on the Municipal Engineer to ultimately recommend, or endorse, the practitioner's selection.

8.2 Stormwater Management Model

The **S**torm**w**ater **M**anagement **M**odel (SWMM) has a long history of development and a very active user community well documented in the literature, recently by Loots & Smithers (2020) and on the internet (for example https://www.openswmm.org). The software is constantly being updated by its formal custodians with the most recent official version, namely Version 5.2.2 released by the US EPA in January 2022 and available here:

https://www.epa.gov/water-research/storm-water-management-model-swmm

8.2.1. Introduction

SWMM is one of the most widely used stormwater models with applications in CSM or event-based modelling, while losses are accounted for by using either Hortonian flow, Green-Ampt infiltration or SCS *CN* values. SWMM is generally used to design stormwater drainage, but it can also be used to track non-point source pollutant loadings, to evaluate LID infrastructure, to model combined sanitation and stormwater conditions, and to model flood control in urban areas and natural systems (Elliot & Trowsdale, 2007; Fletcher *et al.*, 2013; Texas A&M University, 2007).

SWMM is a discretised CSM model that uses the principle of the conservation of mass to estimate runoff from a sub-catchment. The sub-catchment is assumed to be a rectangular, non-linear reservoir with a uniform slope and a width that drains to a single outlet. Inflow is generated by precipitation and losses by evaporation and infiltration. The net excess water is assumed to form a pond of depth (*d*) on the sub-catchment surface. Depression storage (*ds*) is included to account for the surface ponding on flat areas and vegetation. The Manning equation is used to express the runoff volumetric flow rate (*Q*) (Rossman & Huber, 2016). According to Texas A&M University (2007), the percentage of impervious areas and infiltration parameters have the largest influence on runoff volumes. The peak discharge is influenced by the length and slope of the flow paths, while the routing accuracy is dependent on the time step used.

Loots and Smithers (2020) investigated methods and modelling software in use for urban design flood estimation in South Africa. SWMM compared favourably to other modelling software due to a combination of cost (it is free) and technical capabilities that include:

(a) It is designed for urban applications. Most of the hydrological and hydraulic parameters are measured from the physical attributes of conditions in the catchment (soils, slopes, impermeability, etc.), though parameters like Curve Numbers (CN) for urban conditions are not yet developed for South African conditions. Therefore, SWMM models can be calibrated of suitable storm and flood records are available.

(b) Project catchments can be modelled as lumped models or can be discretised in great detail and networked with hydraulic links (conduits) to address spatial differences, rainfall distributions and network attenuation.

(c) The software can use sub-daily and/or daily rainfall input.

(d) Data preparation and model setup time requirements have substantially reduced with the increasing availability of spatial data in South Africa. Practitioners are increasingly likely to set up a SWMM model as prepare a Rational Method or SCS assessment.

(e) Suitable for routing applications based on the steady, kinematic, or dynamic wave theory.

(f) SWMM has capacity for 1D pipe and open-channel flow hydraulic modelling, and 2D floodplain modelling, though for floodline determination it is not yet as practical as the likes of HEC-RAS.

(g) It is regarded as relatively accurate, with reliability levels of approximately 10% (runoff volumes) and 20% (peak discharges).

(h) Cost effective; software is available in the public domain.

Two of the largest Metropolitan Municipalities in South Africa recommend the use of SWMM modelling for stormwater infrastructure design and runoff modelling for all new developments. The City of Cape Town has recommended it since 2002 (City of Cape Town Development Service, 2002) and The City of Johannesburg is currently in the process of developing a stormwater design manual with recommendation to use SWMM modelling for the analysis of stormwater management systems (Barnard *et al.*, 2019). The City of eThekwini uses SWMM for operational planning and disaster management.

8.2.2. History of SWMM

The Stanford Watershed Model (SWM) is regarded as being the first software-based hydrological model and was developed in the 1960s (James, 1965, cited by Boughton & Droop, 2003). Numerous models have since been developed to simulate water quantity and/or quality in both rural and urban catchments. Many of them that achieved mainstream use were developed by the United States Environmental Protection Agency (USEPA) and United States Army Corps of Engineers (USACE). These included the HSPF model (Hydrologic Simulation Program – Fortran) first released in 1980 for basin scale hydrological and water quality analysis, which is an updated version of the SWM that uses the Green-Ampt model for infiltration. Another was the Stormwater Management Model (SWMM) designed specifically for urban catchments and can be used for CSM or event-based modelling using either Hortonian flow or Green-Ampt infiltration. In South Africa the ACRU model, a conceptual agrohydrological CSM software first developed in the late 1970s and early 1980s as a surface water resources model for agricultural catchments uses the SCS equations for storm runoff estimation. It has been adapted to use daily rainfall data most commonly available in South Africa for continuous simulation and has been used successfully in urbanised areas in South Africa (Schulze, 1995b; Schmitz and De Villiers, 1997; Smithers *et al.*, 2007; Smithers *et al.*, 2013; Smithers *et al.*, 2021). Therefore, it allows for wider application for continuous simulation across the country though it is currently limited to applying SCS_SA standardised storm profiles for flood analysis. In addition, the hydraulic functionality in the software is limited placing the ACRU model at a disadvantage in analysing urban stormwater networks.

Internationally, the most frequently used models for CSM in urban catchments are SWMM and MIKE URBAN (Zoppou, 2001; Elliot & Trowsdale, 2007; Jacobson, 2011; Yao *et al.*, 2015; Zhang *et al.*, 2015; Bisht *et al.*, 2016; Faust & Dulcy, 2016). However, with growing climate change and water security concerns, stormwater is considered as an important part of urban water resources and software is fast adapting to meet the demands. InfoWorks ICM (Integrated Catchment Modelling), produced by Innovyze® is an example of the continued development of urban software that seeks to integrate all aspects of urban water systems. Aiming at aspects such as infrastructure management, resilience and disaster management, design flood estimation would only be one part of what the software can provide.

Because the software has exceptionally comprehensive capabilities and because the code is freely available numerous organisations have used the basic SWMM engine to develop proprietary versions of the software. Many of these products use modified versions of the SWMM engine and many require specially formatted input files and generate proprietary results and output files that can only be utilised by that particular version of the software.

Part of the success of the SWMM software in mainstream stormwater analysis is that it is freely available without license costs. It has also been adapted with enhanced GIS and post processing capabilities with versions such as XPSWMM and PCSWMM that are available under licence, though at substantially lower costs than other commercial software packages. SWMM is probably the most widely used urban catchment modelling software in South Africa. Therefore, only the SWMM model will be discussed in more detail in the subsequent sections.

The SWMM home page on the EPA website describes the program as:

> *"EPA's Storm Water Management Model (SWMM) is used throughout the world for planning, analysis, and design related to stormwater runoff, combined and sanitary sewers, and other drainage systems. It can be used to evaluate grey infrastructure stormwater control strategies, such as pipes and storm drains, and is a useful tool for creating cost-effective green/grey hybrid stormwater control solutions. SWMM was developed to help support local, state, and national stormwater management*

objectives to reduce runoff through infiltration and retention, and help to reduce discharges that cause impairment of waterbodies.

The software is a Window-based desktop program with open source code the is free for use worldwide.

For general guidance on the use of SWMM user should refer initially to the User's Manual (Rossman, 2008).

8.2.3. PC-SWMM

Although the native SWMM5 program available from the US EPA has a powerful graphical user interface (GUI) this interface does lack certain capabilities that simplify both model construction and the processing of results. For example, the GUI can only accept a single background image and is incapable of interrogating a background digital elevation model. Similarly, while it is capable of displaying hydrographs it does not have powerful data processing power so cannot generate flow-duration curves or extract statistical properties from generated data series. Much of the processing of output from SWMM relies on third party software such as Microsoft Excel.

This guideline recommends the use of PCSWMM developed by Computational Hydraulics International in Guelph, Canada, (https://www.chiwater.com/Home) for the following reasons:

a) It already has a strong user base in South Africa and is well supported with annual courses. This user support is driven by a strong South African connection amongst its developers.

b) It uses a recent version of the native SWMM engine and is updated within a few months of release of any upgrade to the SWMM engine.

c) Its input and output files are fully compatible with EPA's free version of the software, allowing models developed in PCSWMM to be shared with organisations that do not have the commercial software. (This is particularly advantageous for local authorities who may not have the budget to purchase and update commercial software.)

d) Its more recent versions use the GIS content of the SWMM input file itself and do not make use of external GIS files.

e) It is, however, capable of exporting all of the components of the model as native ESRI shape files that are compatible with QGIS, ArcGIS and most other GIS software.

f) It has powerful GIS processing capability that simplifies model development, for example it is able to automatically discretise a catchment, using a background DEM as the basis for defining the drainage pattern and subdividing the catchment into subcatchments of a target area.

g) For flood modelling it can extract river and floodplain geometry from a background DEM, either automatically at a user defined spacing or at user drawn locations. These transects can also be imported from or exported to HEC-RAS

h) It has powerful output processing capability that allows direct extraction of statistical information from results of continuous modelling.

i) Its scenario computation capability allows similar models with small parameter changes to be processed in parallel and graphical output from the results of these computations to be compared directly in the same window without further processing. An example of the usefulness of this facility is to allow comparison of before and after scenarios to test the effectiveness of proposed SuDS or to ascertain the impact that catchment changes may have on flood discharges.

j) It has reasonably good map generation capability allowing the direct export of report quality annotated maps.

8.2.4. Discretised Model

The SWMM model is a discretised representation of the catchment. It represents the catchment as a drainage network with conduits linking junctions (otherwise known as a link-node model) This is different from the concept of a distributed model where the hydrological and hydraulic processes are represented in a raster form with the hydrological processes taking place in each raster cell and the hydraulic processes represented as flow cascading from cell to cell.

The SWMM model therefore comprises the following major components:

<u>Subcatchments</u>

The catchment area to be modelled is discretised into one or more subcatchments where the majority of hydrological processes take place. Each subcatchment is conceptualised as a rectangular area of constant slope and defined width. Runoff is computed per unit width, treating the subcatchment as a nonlinear reservoir with the flow at the catchment outlet calculated using a variation of Manning's equation.

Each catchment is assigned a proportion of imperviousness and the runoff hydrographs from the pervious and impervious parts computed separately. The runoff hydrographs can be directed straight to the subcatchment outlet or first routed to the other component of the subcatchment, for example for SuDS analysis all or part of the runoff from the impervious area can be routed first to the pervious area of the subcatchment. The catchment imperviousness is the most important parameter in determining the total volume of runoff as well as having a significant effect on the response time. This parameter must therefore be chosen with care and attention must be paid to the distinction between total imperviousness and directly connected imperviousness.

Hydrological losses are determined by the infiltration characteristics of the soil as described in Section 8.2.6 .

Each subcatchment also requires the following parameters for pervious and impervious areas:

Manning's Roughness: While guidance is given in Table 7-2 this response time of the system is quite sensitive to this parameter, so the modeller is advised to refer to the literature for example Rossman and Huber (2016)

Depression storage: Also referred to as initial abstraction this parameter represents the depth of rain required to wet the vegetation or trapped in small depressions on the catchment surface.

Rainfall or Run-on: Subcatchments receive inflow from three possible sources, most commonly from rainfall as determined by the hyetograph associated with the raingage assigned to the subcatchment, but also potentially as run-on from either an adjacent upslope subcatchment or from an Outfall. This last source facilitates return flow from the hydraulic system back to the hydrological system.

Outflow from a subcatchment can be directed to another subcatchment or to a Junction in the hydraulic system.

It is important to understand that each subcatchment is conceptualised in the computational scheme as a homogeneous object with uniform characteristics and the definition of subcatchment boundaries should be driven by this concept. Areas with significantly different topographical characteristics, for example a hillslope draining to a valley bottom, or significant differences in soil permeability, imperviousness or vegetation should be modelled as separate subcatchments

Conduits

Conduits are the hydraulic links that define the drainage pattern of the system. In most natural applications these objects would form a dendritic drainage system, but SWMM's use of the full St Venant dynamic equations allows for looped drainage and reverse flows. Although SWMM does have options for steady flow and kinematic routing in conduits there are significant limitations to these mathematical schemes and the modeller is strongly advised to make use of the full dynamic routing capability of the software.

The slope of each conduit is determined by the levels of the Junctions at its end points and the defined length of flow between these junctions. PCSWMM's GIS allows the conduit to be drawn with a convoluted path from its upstream to downstream junction and determines the length of the conduit from the geographic length of this path. The computational concept is, however, unaware of any plan geometry.

Each conduit has constant cross sectional geometry over its entire length. Irregular cross sections defined by transects are either averaged to a single representative geometry or assigned to different conduits linked in series.

In addition to its ability to accept irregular sections representing natural rivers or floodplains SWMM has numerous built in cross sections that can be selected from the GUI.

The hydraulic roughness of conduits is defined by the assigned a Manning’s n value.

The mathematical scheme requires that the conduit length satisfy the Courant condition, i.e. that the computational time step is less than the time that it takes a small gravity wave to pass down the length of the conduit, the celerity of the gravity wave being the square root of (g.y). Although the in-built intelligence of the mathematical scheme does allow the program to make small adjustments to the computational time step to satisfy this condition the modeller should be aware of this limitation and the possible numerical instability associated with short conduits.

<u>Junctions</u>

Junctions are the links between the hydrological and hydraulic blocks as well as between hydraulic objects. Hence inflow from subcatchments or from user defined hydrographs can only be directed into the drainage system at junctions. Similarly overflow from the hydraulic system or inflow or efflux from the groundwater aquifers can only take place at junctions.

Junctions are assigned invert and rim elevations from the digital elevation model. When linked to open channels the depth of the junction should relate to the depth of the channel. When linked to pipes the depth of the junction can be greater than the diameter of the conduit to allow for surcharging (pressurisation) of the conduit.

As a legacy from its time as a limited urban drainage program, SWMM requires that each junction is given a diameter as a manhole and uses this diameter to determine the storage in the junction at each time step. The continuity equation is solved at each junction for each time step using the storage in the junction as well as the storage in half the length of each conduit linking to that junction.

If the computed hydraulic grade line of the drainage system rises above the rim level at a junction water is lost from the system. This water can be either lost as “flooding” which is flagged as such in the program output or can be directed into undefined storage that then flows back into the drainage system when downstream capacity allows. Both of these are error conditions that should be eliminated by correcting the geometry of the drainage system.

Storages

Storages are a special type of junction that allow a defined stage / storage relationship that is used in the solution of the continuity equation to compute the outflow hydrograph. Outflow from a storage is generally controlled by a weir or orifice. SWMM permits multiple outlets to be assigned to storages, for example compound weirs can be simulated by defining several weirs of different widths and crest elevations.

Control Devices

SWMM has two types of control devices namely weirs and orifices. The mathematical schemes used to compute flows through these devices are very similar. Unsubmerged orifices are treated as weirs and weirs which have been defined with limited heigh (e.g. kerb inlets) become orifices.

Orifices can be either rectangular or circular and defined as side or bottom outlets.

Several different types of weirs, i.e. broad crested or sharp with rectangular, triangular or trapezoidal shape can be defined.

The computational scheme allows both types of control devices to have submerged outlets.

The capacity of control devices can be varied during the duration of the model run using rules that can be written into the input file. For example, an orifice can be controlled to open from zero to its ultimate size over some period of time, so emulating a piping failure of a dam, or the crest level of a weir can be controlled to drop over a period of time emulating the erosion associated with the overflow of a dam crest.

Outfalls

Every model must contain at least one Outfall where water leaves the system. Outfalls can be defined as free flowing with no influence on the upstream system or defined with some form of control as sophisticated as a sinusoidal representation of tidal water levels.
An Outfall can receive flow from only a single Conduit or control device but may receive flow from multiple subcatchments.
More recent versions of SWMM allow flow from an Outfall to be directed onto a subcatchment, a function that is useful when modelling SuDS.

8.2.5. Limitations and assumptions

The most important limitation of computer modelling is to believe that the model accurately represents the complexity of the physical system.

Smooth overland flow seldom, if ever, exists in the physical world, even sheeted steel roofs are corrugated with flow in rills in the valleys of the corrugations.

Soils are heterogeneous with highly variable infiltration capacity determined by localised variations in vegetation, residual macro and microstructure, subsurface movement of shallow groundwater that can influence infiltration rate, etc.

The modeller must therefore be very aware that even the most sophisticated model remains a gross simplification of natural catchment.

During long duration continuous simulation, the only variables that change are the rainfall and the soil moisture content that determines the infiltration capacity of the soil. All other catchment parameters, most importantly imperviousness and roughness associated with changes in vegetation are assumed to remain constant.

8.2.6. Input data requirements Hydrology

Infiltration

Three different methods of calculating infiltration are in common us in South Africa and are available for selection in SWMM. It should be noted that the values of the parameters given in the tables that follow are guidelines only and should be calibrated wherever possible. Considerable guidance and detailed discussion can be found in the *Stormwater Management Model Reference Manual Volume 1 – Hydrology* (Rossman & Huber, 2016)

- **Horton**

Horton's method is an empirical equation that describes the decrease of infiltration rate from water on the surface of the soil. The decrease is an exponential decay function from an initial infiltration rate to some equilibrium rate over a period of time. The integrated version of this equation developed by Green (1984; 1986) takes account of conditions where the rainfall intensity is less than the infiltration rate.

$$f = f_c + (f_0 - f_c)e^{-kt} \qquad \text{Eq. 8.1}$$

where: f = infiltration rate

f_c = minimum or equilibrium infiltration rate

f_0 = initial or maximum infiltration rate

k = constant that reflects how rapidly the infiltration rate decays

t = time elapsed since infiltration began

Typical values of the parameters as suggested by Green are given in Table 8-2 and Table 8-2

Table 8-1: Typical Values for the Parameters in Horton's Equation

Soil Type	f_o (dry)	f_c (equilibrium)
	mm/h	mm/h
Sandy Soil	125	15
Loam Soil	50-75	5-10
Clay Soil	5-25	0-5

Table 8-2: Rate of Decay of Infiltration for Different values of k

k			Percent of decline of infiltration capacity towards equilibrium value f_c after 1 hour
(sec^{-1})	($hour^{-1}$)	(day^{-1})	
0.00056	2.02	48.4	76
0.00083	2.99	71.7	95
0.00111	4.00	95.9	98
0.00139	5.00	120.1	99

- **Green and Ampt**

The Green-Ampt equation (Green & Ampt, 1911) explains the infiltration of water through soil using Darcy's law. Mein and Larsen (1973) adapted the equation for steady rainfall and Chu (1978) showed how the equation could be applied to unsteady rainfall.

The mechanism is simplified because infiltrated water is assumed to move downward through the soil as an abrupt wetting front that separates the wetted and unwetted soils. In reality the wetted front may not be abrupt and the soil above the front may not be fully saturated. But the approach is preferable to that of Horton because the equation represents a realistic physical process and can be adjusted as better information or explanations become available (Richards 1931), or to take account of a driving head of water standing above the soil surface that is used by SWMM in the analysis of LIDs.

The form of the Green-Ampt equation is:

$$f = K_{sat}\left(1 + \frac{(\phi - \theta)(d + \psi)}{F}\right) \qquad \text{Eq. 8.2}$$

where

f	=	infiltration rate
K_{sat}	=	Hydraulic conductivity of the saturated soil
Φ	=	porosity of the soil
θ	=	initial volumetric water content of the soil
d	=	depth of driving head above the soil surface (usually ignored)
ψ	=	capillary suction head at the wetting front
F	=	cumulative infiltration

The calculation is sensitive to the term ($\Phi - \theta$), i.e. the difference between the porosity of the soil, which is effectively equal to the total moisture capacity of the soil, and the initial moisture content of the soil, so care should be taken in the selection of the value of θ. The value of this parameter is related to the field capacity of the soil, i.e. the moisture content when all available water has drained out under gravity, and the wilting point, which is the point at which moisture is so tightly bound by capillary tension that it is no longer available to plants.

Table 8-3: Suggested Green-Ampt Parameters

USDA Soil-Texture Class	Hydraulic Conductivity K_1	Wetting Front Suction Head Yf	Porosity	Water Retained at Field Capacity	Water Retained at Wilting Point
	mm/h	mm	m^3/m^3	m^3/m^3	m^3/m^3
Sand	120.40	49.02	0.437	0.062	0.024
Loamy Sand	29.97	60.96	0.437	0.150	0.047
Sandy Loam	10.92	109.22	0.453	0.190	0.085
Loamy Sand	3.30	88.90	0.463	0.232	0.116
Silt Loam	6.60	169.93	0.501	0.284	0.135
Sandy Clay Loam	1.52	219.96	0.398	0.244	0.136
Clay Loam	1.02	210.06	0.464	0.310	0.187
Silty Clay Loam	1.02	270.00	0.471	0.342	0.210
Sandy Clay	0.51	240.03	0.430	0.321	0.211
Silty Clay	0.51	290.07	0.479	0.371	0.251
Clay	0.25	320.04	0.475	0.378	0.265

After Rawls et al. (1983), Torno (1993), and Rawls & Saxton (2006).

- **Soil Conservation Service (SCS) Equation**

The SCS equation is often incorrectly formulated as an infiltration equation because differentiation of the equation yields an infiltration rate that is proportional to rainfall (Torno, 1992). The procedure is, however, in common use in South Africa and available as a modelling methodology in SWMM. Readers wishing to use this method are referred to the literature, for example Schmidt and Schulze (1987) or Rossman and Huber (2016).

If the curve numbers given in Table 8-4 are used to define the loss function, then the imperviousness ratio of the subcatchments must be set to zero in order to avoid double accounting of the runoff from the impermeable areas.

Loots and Smithers (2020) used the 2013/2014 SANLC classes, defined by the Department of Environmental Affairs (DEA), to recommend CNs for SWMM as listed in Table 8-4, these values are, however, based on the results of a single study and should be considered as a guideline only.

Table 8-4: Preliminary Curve Numbers recommend for use in SWMM (Loots and Smithers, 2020)

* DEA Land Use No.	Land Use Type	*CN*						
		A	A/B	B	B/C	C	C/D	D
5	Thicket/Dense bush	25	47	55	64	70	74	77
6	Woodland/Open bush	45	56	66	75	77	80	83
7	Grassland	49	61	69	75	79	82	84
9	Low shrubland	28	36	44	53	60	64	66
32	Plantations/Woodlots mature	34	47	59	64	69	72	75
33	Plantation/Woodlots young	37	49	60	66	71	74	77
35	Mines 1 bare	89	91	92	93	94	95	95
36	Mines 2 semi-bare	89	91	92	93	94	95	95
42	Urban commercial	89	91	92	93	94	95	95
43	Urban industrial	89	91	92	93	94	95	95
44	Urban informal (dense trees/bush)	60	68	74	78	82	85	86
45	Urban informal (open trees/bush)	64	69	77	81	84	86	87
46	Urban informal (low veg/grass)	66	71	79	83	86	88	89
47	Urban informal (bare)	78	82	86	88	90	91	92
48	Urban residential (dense trees/bush)	51	59	65	69	73	76	77
49	Urban residential (open trees/bush)	55	60	68	72	75	77	78
50	Urban residential (low veg/grass)	57	62	70	74	77	79	80
51	Urban residential (bare)	69	73	77	79	81	82	83
52	Urban school and sports ground	49	61	69	75	79	82	84
53	Urban smallholding (dense trees/bush)	27	49	57	66	72	76	79
54	Urban smallholding (open trees/bush)	47	58	68	77	79	82	85
55	Urban smallholding (low veg/grass)	50	62	70	76	80	83	85
56	Urban smallholding (bare)	55	67	75	81	85	88	90
57	Urban sports and golf (dense trees/bush)	39	51	61	68	74	78	80
58	Urban sports and golf (open trees/bush)	45	56	66	75	77	80	83
59	Urban sports and golf (low veg/grass)	25	47	55	64	70	74	77
60	Urban sports and golf (bare)	68	74	79	83	86	88	89
61	Urban township (dense trees/bush)	73	77	81	84	76	87	88
62	Urban township (open trees/bush)	75	79	83	86	78	89	90
63	Urban township (low veg/grass)	76	80	84	87	79	90	91
64	Urban township (bare)	77	81	85	88	80	91	92
65	Urban village (dense trees/bush)	37	59	67	76	82	86	89
66	Urban village (open trees/bush)	57	68	78	87	89	92	95
67	Urban village (low veg/grass)	60	72	80	86	90	93	95

* DEA Land Use No.	Land Use Type	*CN*						
		A	A/B	B	B/C	C	C/D	D
68	Urban village (bare)	65	77	85	91	95	98	95
69	Urban built-up (dense trees/bush)	89	91	92	93	94	95	95
70	Urban built-up (open trees/bush)	89	91	92	93	94	95	95
71	Urban built-up (low veg/grass)	89	91	92	93	94	95	95
72	Urban built-up (bare)	89	91	92	93	94	95	95

* Only land uses associated with urban areas are listed

Since the curve number (CN) is fixed for the duration and so does not allow for variation of infiltration rate with antecedent wetting or drying of the soil of the computation this method is not suited for CSM.

<u>Rainfall</u>

Each subcatchment must receive rainfall via its assigned Raingage (sic model terminology)

For event modelling it is usual practice to assign the same Raingage to all subcatchments and to assign some form of design storm hyetograph to that Raingage. PCSWMM has a built-in function to generate numerous design storm hyetographs for example, SCS, SCS-SA, Chicago and many others of different recurrence intervals based on the 24 hour rain depth.

It is possible to simulate the effect of storm movement over a catchment by assigning different Raingages to different subcatchments while using the same hyetograph lagged by some incremental amount.

Continuous simulation is achieved by assigning a single Raingage with a long duration (weeks to decades) rainfall record to all subcatchments.

If given a sample of hyetograph with short time steps PCSWMM is capable of stochastically generating a statistically similar rainfall distribution from a daily time series. This capability should, however, be used with caution since each hyetograph is generated using a scheme seeded with some random number so each hyetograph is likely to be different from the last and multiple applications of the process will yield slightly different results.

Continuous rainfall records are representative of rain at a single point and so may not accurately represent conditions over the entire catchment, even if the rain gauge does happen to fall within the catchment.

8.2.7. Input Data Requirements: Hydraulics

Input data for the hydraulics block of SWMM are derived from the digital elevation model, high resolution aerial images or site visits, and potentially the records of the local authority.

PCSWMM can be used to extract river cross section details directly from the DEM, or cross sections can be input from survey data. The same data set as used for the HEC-RAS computation of the floodlines can be used to supplement the catchment wide river geometry.

Modelling rivers as trapezoidal sections should be avoided if possible because these sections are frequently hydraulically more efficient than natural compound sections comprising a floodplain and incised channel. The use of overly efficient sections will result in an underestimate of the hydrograph routing and hence an underestimate of catchment response time and an overestimate of the flood peak discharge.

Stream channel and floodplain roughness can be obtained from the literature, for example Chow (1973) or Henderson (1966).

Some local authorities in South Africa have good GIS based representations of the drainage systems within their areas of jurisdiction, but in most cases this information is sparse and generally inaccurate. GIS based information can be imported directly into the model using the tools available in PCSWMM but in almost all cases these data will have to be checked. The design standard recurrence interval for underground municipal drainage systems ranges from 2 years to 10, so the modeller can use engineering judgement to include conduits of this capacity in the model.

Runoff from events with higher recurrence interval is most often on surface so the flow paths can be determined directly from the terrain model.

SWMM is most frequently configured as a dual drainage model with lower flows contained in the underground conduits and high flows running overland. These dual drainage elements joint at the model Junctions. The links between the below and above ground conduits can be as simple as assigning inlet and outlet elevation offsets to the above ground conduits at each junction, or as complex as defining stage / discharge curves to the linkage. Most frequently the link is made using a weir or orifice control object.

8.2.8. Calculation procedures

A simplified calculation procedure follows the steps outlined below:

(a) Using the digital elevation model or background contour plan discretise the drainage system:

  i. Define all subcatchment boundaries ensuring there are no gaps or overlaps. Measure catchment areas and overland flow lengths from which the catchment width can be calculated.

  ii. Locate all junctions and assign invert and rim elevations.

iii. Define all conduits running from the upslope to the downslope Junction.

iv. Identify all significant storage areas and assign stage / surface area curves to these storages.

v. Ensure that the model has at least one Outfall and which is fed by a single (possibly dummy) conduit.

(b) Assign soils, imperviousness, surface roughness and depression storage characteristics to each subcatchment.

(c) Define the hydraulic characteristics of each Conduit and control device.

(d) Assign the rainfall hyetographs to as many Raingages as necessary and allocate the Raingages to the appropriate subcatchments.

i. Even when doing CSM with a long duration rainfall record create one event hyetograph with a rainfall intensity and 24 hour depth greater than the largest event in the continuous record.

(e) Set the computational parameters:

i. Chose the hydraulic computational scheme – generally dynamic routing unless there is a good reason to use steady state or kinematic routing.

ii. Set the time steps.

. The hydraulic computation time step should be of the order of a few seconds.

. The hydrological time step must be an integral multiple or fraction of the hyetograph time step.

iii. Ensure that the computational dates are the same as the dates of the rainfall record.

(f) Run the model using the largest event hyetograph defined in (d)

i. Check the results to ensure there is anomalies such as flooding at Junctions or Storages or unintended surcharging of Conduits.

ii. Make adjustments to eliminate these errors.

iii. Repeat (f) until the model runs without errors.

(g) For event modelling

i. Run the model with the design event hyetographs and record the objective function results such as peak discharges, maximum water depths in storage basins, etc.

ii. Make changes to the calibration parameters and rerun the model to test the sensitivity of the results to variations in these parameters.

iii. Record the variations and understand the uncertainty in the results.

(h) For CSM:

i. Run the model with the long duration rainfall record. Be aware that computation can take many hours, so do not start the computation until you are sure that all parameters have been set correctly.

ii. Extract the computed events from the long duration hydrograph and compare these event hydrographs to the driving event hyetographs. Note that the highest peak discharge may not be associated with the greatest rainfall intensity and that the largest event runoff volume may not be associated with the greatest event rainfall depth.

iii. Record and understand these uncertainties.

iv. If possible, adjust the calibration parameters and rerun the model.

8.3 Continuous Simulation Modelling (CSM)

The estimation of future floods with an acceptable risk, quantified by the AEP, can be performed by using CSM to simulate streamflow time series with an adequate record length to ultimately enable the extraction of the required flood statistics. Some of the main benefits of CSM in DFE include:

(a) Temporal rainfall patterns within events are real, avoiding uncertainties associated with synthetic rainfall profiles.

(b) Natural wet and dry sequences are represented, negating the uncertainty associated with selection of initial conditions.

(c) The effect of multiple storages within a drainage network can be analysed.

(d) Uncertainty in using the assumption of the T-year storm generating the T-year flood (peak or volume) is largely avoided. This may only apply to the lower order return periods if the rainfall time series is short.

(e) The streamflow time series is more representative of the natural time series for flood frequency analysis (again this will be limited by the length of the rainfall time series).

8.4 Recommended approach to using Stormwater Models and CSM in Design Flood Estimation

Stormwater models as described above have become more accessible and easier to use, to the extent that it is possible to use PC-SWMM with a Google Maps satellite image background to estimate runoff as rapidly as can be done using the Rational or SCS methods. Importantly, they allow the practitioner to provide a higher level of analysis and improve the confidence in the design flood estimate. This is largely because there is less simplification of the hydrological processes and more opportunity to test the simplifications and the effects of natural variability in the hydrological processes.

Another important factor that improves the confidence in design flood estimates from stormwater models lies in the hydraulic capabilities that better reflect the translation and attenuation of flood waves through a catchment. This applies to both overland flow and network conveyance processes that have important bearing on the shape of flood hydrographs.

For these reasons the use of stormwater software such as the SWMM software is recommended as best practice for application in all municipalities in South Africa for single event based design flood estimation.

CSM takes a step further in mitigating some of the assumptions inherent in single event analysis. These include:

- Avoiding the need to assume initial conditions at the start of an event.
- Avoiding the assumption that the T-year storm causes the T-year flood peak.
- Avoids reliance on empirical estimates of critical storm durations.
- Avoids uncertainties in the use of standardised design storm profiles (where sub-hourly rainfall records are available).
- Mitigates the uncertainty of the effect of multiple storage systems in an urban catchment on flood peak and volume.

The list is substantial, though it also highlights an important limitation. Rainfall records at sub-hourly intervals are not widely available in South Africa other than in most of the metropolitan cities. In addition, even daily rainfall record lengths may be short and using CSM to simulate the rare flood events (typically >10 years) will be limited.

Automatic weather stations (5min recording intervals) are increasingly affordable, and it is expected the number of stations installed will grow in municipalities across South Africa in the near future. In the meantime, the recommended approach to DFE is as follows:

(a) Where sub-hourly rainfall records are available locally (or regionally where appropriate), CSM is used to estimate design flood hydrographs for return periods up to not more than twice the length of the rainfall record length.

(b) Single event analysis, based on the Smithers and Schulze (2002) design rainfall generator or a probability analysis of more recent (post 2000) rainfall records, combined with the latest standardised storm profiles, are used to determine the design flood hydrographs for all events greater than the length of rainfall records.

(c) The overlap between the CSM and single event estimates shall be a transition zone evaluated by the DFE practitioner for final recommendation.

(d) Where inadequate, or no sub-hourly rainfall records are available, then the approach in (ii) shall apply to the entire range of design flood estimates.

8.5 Performance of CSM

In a pilot study using the SWMM in Tshwane, Loots and Smithers (2020) showed that the parameters currently used in the Modified Green-Ampt infiltration method for runoff modelling in South African urban need to be updated and developed updated preliminary SCS Curve Numbers for urban land use types in South Africa. However, generic values for these parameters will still be vulnerable to local, onsite variations.

The approach to modelling is therefore also a key factor in the performance in the models. The effort put into DFE will be fit-for-purpose, but where higher confidence is required more effort should be put into the modelling. For the reasons given in the previous section, if sufficient attention is given to rainfall distribution and intensity, to the heterogeneity of the catchment (soils, land cover, etc.) and the attenuating effect of the conveyance network on the shape and peak of the hydrograph, the level of confidence in the performance of the models will improve substantially.

Calibration is a key part of performance and must remain part of DFE good practice (see Section 10.4). Ball (2020) showed that with calibration, SWMM model estimation of design floods could achieve errors of less than 10% in peak flows. He proposes that this should be a target for municipal design flood estimation.

8.6 Emerging New Developments

SCS-SA CSM: A new SCS-SA based Continuous Simulation Modelling (CSM) approach has been developed (Smithers *et al.*, 2021). Using a spreadsheet based GUI, the SCS-SA method uses design runoff volumes estimated from the daily runoff estimated using the *ACRU* model (Schulze, 1995b; Smithers and Schulze, 1995b), with design peak charge computed as outlined in Section 7.3. Users have the choice of selecting their own land cover and soils categories for up to nine sub-catchments/HRUs, or can use default parameters for a user selected Quinary Catchment (QC) in the database developed by Schulze (2013) comprising of parameters for 5 838 QCs in South Africa. The performance of the SCS-SA CSM on 19 small catchments in South Africa was generally better than the SCS-SA method, and further refinement of the transformation of the design volumes into design peak discharges is recommended (Smithers *et al.*, 2021).

Distributed Models. An emerging field in hydrological modelling for runoff computation is the development of distributed models based on a raster digital elevation model of the catchment. Hydrological and hydraulic parameters such as soil infiltration characteristics, impermeability and surface roughness are assigned to each raster cell with the software then computing the direction and magnitude of runoff. The application of this method is restricted in South Africa by the limited availability of high resolution digital elevation data, and in urban areas generally because it is difficult to define features such as kerbs which may have significant influence on the runoff pattern.

8.7 Chapter References

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9. FLOOD HYDRAULICS

9.1 Overview

The design flood discharge provides the basis for determining the important practical manifestations of a flood: the magnitudes and the spatial and temporal variations of water levels and inundated areas. These require hydraulic calculations.

The water levels in a river channel for a specified discharge depend on the physical characteristics of the channel, including any artificial structures such as weirs and bridges and significant local natural features such as sudden changes in slope or cross section. The complexity of natural channels makes hand calculation of the hydraulics impractical in most cases, and various model software products are available for general use. Different models are based on different assumptions regarding flow conditions, and selection of an appropriate model requires an appreciation of the implications of the assumptions in relation to the requirements for the study site. The data required for running a model depend on the model type and how water surface elevations are calculated for the assumed flow type.

This chapter provides preliminary guidance for model selection and identification of the input information required for model application. The necessary details should be obtained from the manuals for the particular software selected. Estimation of flow resistance coefficients is an important and often neglected consideration; details of prediction methods applicable for different channel characteristics and conditions are appended in Chapter 17.

9.2 Flow Types

The basic flow conditions recognized in model formulation, and about which assumptions are made, are shown in Figure 9-1

The first distinction between flow types is their classification as steady or unsteady according to variations with time. If flow conditions (including discharge, velocity and flow depth) do not change with time, then the flow is steady. If they do change with time, then the flow is unsteady.

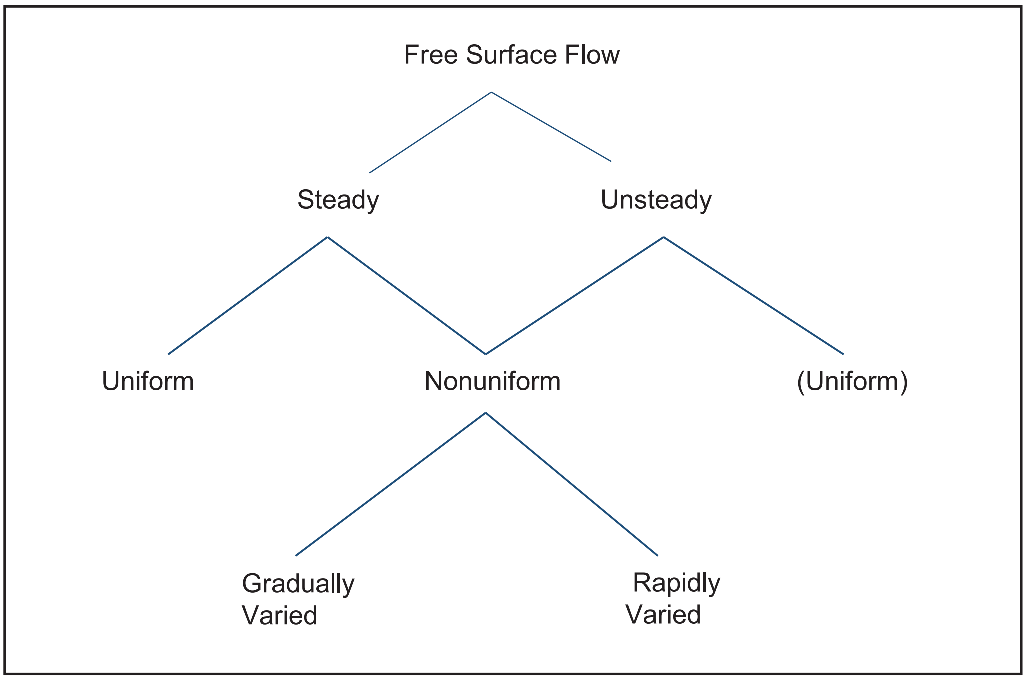

Figure 9-1: Basic types of free surface flow (James, 2020)

Flow is also classified as uniform or nonuniform according to its spatial variation. If the flow conditions do not vary spatially, then the flow is uniform; if the flow conditions do vary spatially, then the flow is nonuniform. Uniform flow would occur in a channel with unvarying cross-section shape and surface roughness on a constant slope. Any disruption of channel uniformity results in nonuniform flow, the nature of the response depending on whether the flow is subcritical (the Froude number, Fr, < 1) or supercritical (Fr >1). Nonuniformity may occur in one (1D), two (2D) or three (3D) dimensions depending on the variations of velocity in the longitudinal, transverse and vertical directions.

Unsteady uniform flow is not possible, but nonuniform flow may be steady or unsteady. Both cases of nonuniform flow are further classified as gradually varied (GV) or rapidly varied (RV). In gradually varied flow the changes in flow condition take place over long distances, such as the backing up which occurs upstream of a dam or weir. In rapidly varied flow the changes take place over comparatively short distances, such as over a structure, through a constriction, or in a hydraulic jump. Rapidly and gradually varied changes are fundamentally different in nature, the former being associated primarily with local changes in boundary geometry with no significant influence of surface resistance, and the latter being determined primarily by flow resistance.

Examples of flow profiles resulting from the disruption of uniform flow by a small structural feature are shown in Figure 9-2. Which profile would actually occur depends on whether the channel is mild (a. and b.) or steep (c. and d.) and whether the feature is hydraulically small

(a. and c.) or large (b. and d.) "Mild" and "steep" are indicated by the uniform flow depth being subcritical or supercritical, and so a channel being mild or steep depends on the discharge as well as the channel slope. A feature being hydraulically small or large also depends on the discharge as well as its size. The same feature in a particular channel could therefore result in different profile types as the discharge varies.

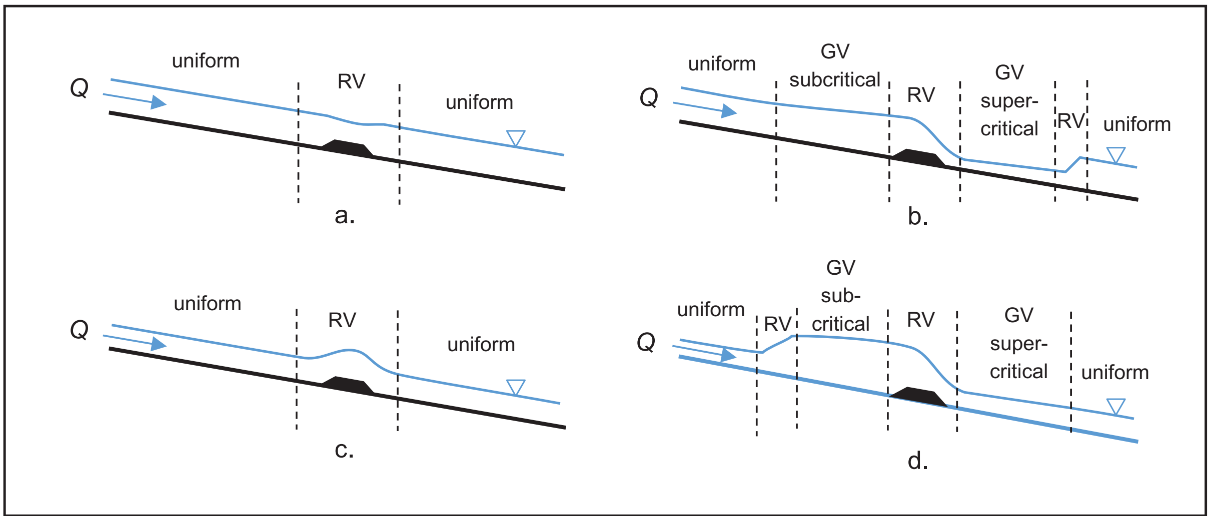

Figure 9-2: Steady water surface profiles induced by a simple structure in a uniform channel

9.3 Model Types

Although natural flood flows are invariably unsteady and nonuniform, it is not always necessary to describe the flow fully in order to make useful predictions of water levels. The simplifying assumptions underlying the different model types make for more efficient computation and easier data compilation and input.

The simplest assumption for water level prediction would be of one-dimensional, steady, uniform flow. This would give a useful first indication of flow conditions in a long confined channel where the peak flood discharge has a relatively long duration and the channel conditions do not vary significantly in the flow direction.

For these conditions, the flow depth can be related to the discharge through the continuity equation.

$$Q = AV \qquad \text{Eq. 9-1}$$

where: Q = discharge (m^3/s)

A = cross-sectional area (m^2)

V = cross-section average velocity (m/s)

The velocity is related to the flow depth and channel characteristics by a resistance equation. The following three equations are commonly used.

Chézy: $$V = C\sqrt{RS} \qquad \text{Eq. 9-2}$$

Darcy-Weisbach: $$V = \sqrt{\frac{8g}{f}}\sqrt{RS} \qquad \text{Eq. 9-3}$$

Manning: $$V = \frac{1}{n}R^{2/3}S^{1/2} \qquad \text{Eq. 9-4}$$

where: R = the hydraulic radius (= A/P) (m)

A = the cross-sectional area (m^2)

P = the wetted perimeter (m)

S = the friction gradient (which is equal to the channel gradient, S_o, for steady, uniform flow).

C = Chézy resistance coefficient (m$^{1/2}$/s)

f = Darcy-Weisbach friction factor

n = Manning resistance coefficient (s/m$^{1/3}$)

Equations 9-2 to 9-4 are essentially equivalent and coefficient values can be easily converted between them using the following relationships.

$$C = \sqrt{\frac{8g}{f}} = \frac{R^{1/6}}{n} \qquad \text{Eq. 9-5}$$

If the disruptions to uniform flow are considerable, then nonuniform computations are required. The water surface elevation at any location is the flow depth plus the elevation of the bed above an arbitrarily defined datum (i.e. $z + y$ in Figure 9-3).

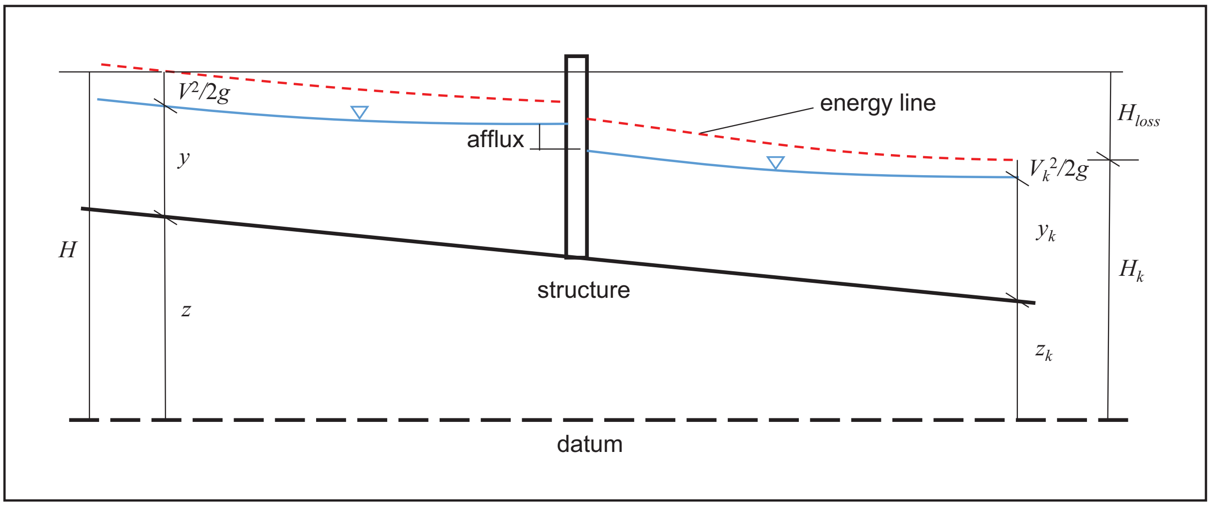

Figure 9-3: Steady 1D subcritical water surface profile

For steady, one-dimensional flow, the flow depth, discharge and total energy are related by

$$H = z + y + \alpha \frac{V^2}{2g} \qquad \text{Eq. 9-6}$$

with

$$V = \frac{Q}{A(y)} \qquad \text{Eq. 9-7}$$

in which: H = total energy (m)

z = bed elevation above datum (m)

y = flow depth (m)

V = cross-section average velocity (m/s)

α = kinetic energy correction factor to account for nonuniform velocity distribution

g = gravitational acceleration (m/s^2)

Q = discharge (m^3/s)

$A(y)$ = cross-sectional area, dependent on y (m^2)

The flow depth and hence the water level can therefore be calculated for a given discharge if H is known. The local value of H is calculated as the value at a known location downstream (for subcritical flow) or upstream (for supercritical flow) plus all energy losses between the two locations. So, for the situation in Figure 9-3

$$H = H_k + H_{loss} \quad \text{Eq. 9-8}$$

in which: k = subscript indicating the known values of H and its constituents

H_{loss} = all losses between known and calculated locations

If the assumptions of steady conditions and/or one-dimensional flow are not made, computations require the solution of the differential forms of the continuity and momentum equations. The reader should refer to the relevant software manuals for descriptions of the equations and methods for their solution. Basic underlying theory is also presented in standard texts, such as Henderson (1966), French (1985), Chanson (2004).

Readily available computer models are available based on the different assumptions of flow conditions, including 1D steady, 1D unsteady, 2D steady and 2D unsteady. The Conveyance Estimation System (CES) (Knight et al., 2010) predicts 1D profiles, but includes computation of transverse velocity distributions.

9.4 Model Selection

The selection of an appropriate model for defining flood water surface levels involves matching the needs of the analysis and the study site characteristics to the attributes of available models, in particular their dimensionality and their capability for steady or unsteady analysis. It is important to appreciate that higher dimensionality of modelling does not necessarily lead to better accuracy of the results, and that the opposite may be true (Knight et al., 2010); the simplest model capable of producing the required results should be chosen.

In most practical cases the variation of channel characteristics is too great to allow the assumption of uniform flow, and nonuniform computations are required. If the velocity and water surface elevation vary only in the streamwise direction 1D modelling is usually sufficient. This includes cases of channels with compound sections, where flood plains are not extensive and can be treated by subdivision of cross sections. Where flow velocities occur transverse to as well along the main streamwise direction, then 2D modelling is required. Such cases include extensive overbank flooding (especially between buildings in urban areas), flow division around islands, and other laterally unbounded flow situations. (However, note that recent versions of HEC-RAS can compute divided and converging flows, allowing 1D modelling of simple divided flow systems, such as around islands in rivers.)

Both one- and two-dimensional flows may be treated as steady or unsteady. An assumption of steady flow, i.e. a single constant discharge, makes for a much simpler analysis and is often sufficient. Flow can be assumed to be steady if the peak stage (water level) corresponds closely with the peak discharge and occurs almost simultaneously over consecutive computational cross sections. (For strongly unsteady flows the relationship between stage and discharge is different for rising and falling discharges and the maximum stage will occur after the peak discharge, at a correspondingly lower value.) The American Society of Civil Engineers (1996) suggests that an unsteady flow analysis is necessary.

(a) if the discharge changes rapidly, such as in the case of a dam break.

(b) for channel networks where the flow divides and recombines; and

(c) if the channel slope is milder than about 1:2500 (0.00040). A steady flow analysis is usually adequate for slopes greater than about 1:1000 (0.0010). For slopes between these limits an unsteady analysis may be necessary if there are significant tributary inflows or backwater effects from receiving streams.

Unsteady modelling is also usually required for tidally controlled situations, where the downstream boundary condition varies with time.

9.5 Input Data Requirements

The calculations outlined in section 9.3 require the bed elevation and flow depth to be known at the downstream location, and all energy losses to be quantified. The losses include friction, expansion and contraction losses and losses through structures.

The data requirements depend on the type of model and are specified in the particular user manuals. Typical requirements and some general guidelines, mainly for steady 1D modelling of subcritical flows, are described below. For supercritical flows the direction of computations and relative locations of boundary conditions are reversed. Requirements for unsteady and 2D modelling are more detailed, but similar in kind.

9.5.1. Channel configuration

This defines the river reaches to be modelled and how they are connected. The system may be a single channel, a dendritic system, or a looped network. The length of river over the study area is usually divided into a number of separate reaches to improve the accuracy of the computations, to enable reliable description of the variation of channel characteristics along its course, and to enable water levels to be output at the required locations. The calculations are then carried out from reach to reach, with the water surface at the beginning section of one reach becoming the known section at the end of the next one upstream. The configuration enables the computations for different reaches to be linked and managed.

9.5.2. Cross section geometry

Cross sections are located at intervals along reaches. They extend over the entire width of possible inundation, including flood plains, and are oriented perpendicular to the flow direction. The geometry is defined by ground surface point elevations and horizontal distances between them.

9.5.3. Cross section location and spacing

Cross sections are required wherever reach characteristics change, including changes of discharge, shape, roughness and at locations of in-channel structures. Consideration should be given to the possibility of future modifications, such as new structures, river stabilization works or morphological changes. Care should be taken to include locations where control effects could be different at different discharges. For example, in Figure 9-2 Figure 9-2: Steady water surface profiles induced by a simple structure in a uniform channel the structural feature is identical in (a) and (b), but acts as a control in (b) but not in (a); cross sections before and after the structure may not be required for computing the profile in (a), but definitely would be for (b). The accuracy of predictions is influenced strongly by the distances between adjacent surveyed cross sections, which should be kept as short as practically possible. Knight et al. (2010) list the following general rules for locating cross sections.

- At model limits (especially the downstream control location).
- Either side of all structures.
- At all flow and level measuring locations (for calibration).
- At all sites of flooding concern.
- Representing the channel geometry.
- About 20 times the bankfull width apart, as a first estimate.
- A maximum of $0.2D/S_o$, where D is the local water depth and S_o is the channel slope.
- A maximum of $L/30$ apart, where L is the length scale of the physically important wave (flood or tide).
- The area of successive sections is between 2/3 and 3/2 of the previous sections.
- The conveyance (K) for successive sections is between 4/5 and 5/4 of the previous sections, where.

$$Q = K S^{1/2} \qquad \text{Eq. 9-9}$$

So, using the Manning equation (Eq. 9-4),

$$K = \frac{A^{5/3}}{n P^{2/3}} \qquad \text{Eq. 9-10}$$

9.5.4. Channel resistance and energy loss coefficients

The hydraulic calculations require estimation of energy losses between adjacent cross sections. These include friction losses, expansion or contraction losses associated with changes in cross-sectional shape and area, and losses associated with flow through any intervening structures.

The friction losses between adjacent specified cross sections are determined as the product of the friction gradient and the distance between the sections. The friction gradient (S) is calculated using one of the standard flow resistance equations that relate the flow velocity to the flow depth and channel characteristics (Eqs 9-2, 9-3 and 9-4). Application of these equations requires an input value of the appropriate resistance coefficient. Approaches and methods for estimating resistance coefficients are presented in Appendix 8 (Chapter 17). Knight et al. (2010) maintain that channel roughness (and its representation) is arguably the single most important issue to resolve prior to successful modelling.

Contraction and expansion losses can be accounted for by increasing the resistance coefficient value, or related to the change in cross-section averaged velocity heads between adjacent cross sections, typically as

$$h_{loss} = C\left|\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right| \qquad \text{Eq. 9-11}$$

where: h_{loss} = the expansion or contraction loss

C = the contraction or expansion loss coefficient

V_1 and V_2 = velocities at adjacent cross sections 1 and 2

Values of C for contractions range from 0.1 for gradual changes to 0.6 for abrupt changes, and for expansions range from 0.3 for gradual to 0.8 for abrupt changes (Brunner, 2010).

9.5.5. Boundary conditions

Nonuniform computations proceed upstream for subcritical flows and downstream for supercritical flows, so a downstream starting condition is required for subcritical flows (the usual case) and an upstream starting condition for supercritical flows. This may be specified in different ways: as a known water surface elevation, the critical depth, or the uniform flow depth.

If a known water surface elevation or the uniform depth is specified, it is important that it is within the range of water levels to be expected, or the solution may diverge from the correct values. The possible water surface variations for flow on mild and steep slopes are shown in Figure 9-4. These show the ranges of flow depths relative to the uniform and critical depths in response to disruptions from uniform flow by back-ups or drawdowns.

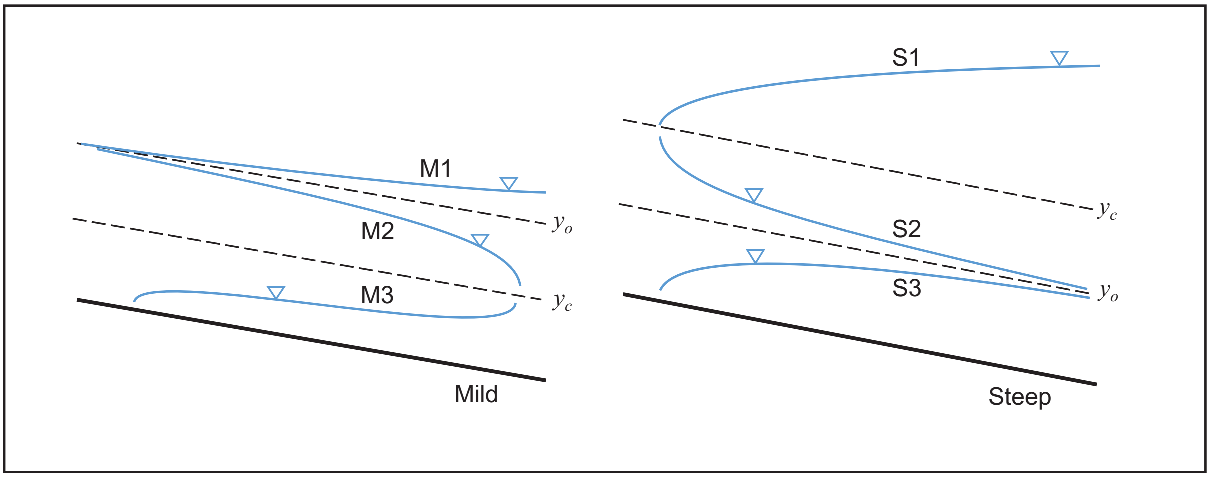

Figure 9-4: Gradually varied flow profiles on mild and steep slopes

Ideally, the starting flow depth should correspond to a true control, i.e. a location where the flow depth can be determined directly from the discharge without reference to conditions elsewhere. This requires the flow downstream of the location to be supercritical, and the flow upstream to be subcritical. Structures such as weirs, culverts and bridges commonly act as controls, some software, for example HEC-RAS will, however, not allow the water surface profile computation to commence at a bridge or culvert. In cases where no distinct control feature exists, the downstream boundary condition may be assumed to be the local uniform flow depth, provided it is set reasonably far downstream from the study site and that a good representation of the channel slope can be estimated. It is good practice then to repeat the analysis with slightly different flow depths to confirm that the effect at the study site of the specified value is not significant.

For unsteady flow modelling the input discharge or stage hydrograph needs to be provided at the upstream boundary. The downstream boundary condition should be specified as a stage-discharge relationship, rather than a single water surface elevation to allow for the temporal variations. Starting values of depth and discharge (or velocity) may also need to be specified at all cross sections. 2D and 3D models have additional boundary condition requirements, as specified in the relevant software manuals.

For coastal rivers the boundary condition should be specified in relation to tidal variations and storm surge levels, including their temporal variations for unsteady modelling, many local authorities specify the starting water surface elevation as high spring tide plus an allowance for storm surge. The possibility of estuary bar formation or breaching should be considered.

9.5.6. In-channel structures

While structures such as weirs, culverts and bridges may act as controls, they may also be submerged, with the actual control being further downstream. In such cases they would still raise the water level upstream by introducing an energy loss requirement. The raised water level then becomes the starting condition for computing the water surface profile further upstream. The afflux, or amount by which the structure raises the water level, requires estimation through hydraulic analysis. Methods for the analysis of in-channel structures are described in various textbooks and guidelines, including Chow (1959), Henderson (1966), United States Bureau of Reclamation (1973), CSRA (1994) Chanson (2004), Novak et al. (2001), Knight et al. (2010), James (2020), all of which cite other accessible primary sources. Computer software packages, such as HEC-RAS, include basic hydraulic structure analyses.

9.6 Uncertainty and Calibration

There is always a degree of uncertainty associated with predictions of flood water surface levels, and this should be quantified if possible, or at least appreciated in the interpretation of results. Uncertainties arise from inaccuracies in measured input data or estimated input parameters. The geometry of channel cross sections can change with time through erosion and deposition of sediment, or by river control measures or other developments; these should be anticipated in the analysis. Channel resistance and energy loss coefficients are particularly difficult to estimate reliably because of their variability with discharge and long-term or seasonal changes of vegetation.

Wherever possible, input parameters should be calibrated against observations. Values should be adjusted to match predicted and measured water levels or velocities for measured discharges at selected cross sections. The process becomes more complicated for 2D modelling because of the additional components of velocity in magnitude and direction.

If direct calibration is not feasible, then at least some sensitivity analyses should be carried out. The prediction analysis should be repeated with a range of values of the uncertain parameters, taken one at a time, to gain an appreciation of the effect of the uncertainty on the final results.

9.7 Conclusion

Selection of an appropriate model and the specification of input is largely subjective, but requires an appreciation of the underlying phenomena and how these are accounted for in the hydraulic models considered. The guidance given here for model selection and for specification of input information should be supplemented by thorough perusal of the relevant software manuals to ensure prediction of accurate and defendable flood lines.

9.8 References

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10. UNCERTAINTY, ACCURACY, CONSISTENCY AND CALIBRATION

Uncertainty is one of the fundamental issues in design flood estimation (Ball et al., 2019). This applies to both the hydraulic and hydrological components of DFE. Greater attention is given here to the hydrological functions with its wider range of parameters and variable rainfall inputs. However, consideration of accuracy, uncertainty and calibration in the hydraulic conditions is equally important, with the hydrological response itself a potentially uncertain variable input.

Developing a consistent approach DFE, supported by regular tests for accuracy through calibration forms the basis of managing uncertainty recommended in this guideline. Local authorities have a key role to play in directing the approach and developing local databases and undertaking calibration exercises across the municipal area to help improve flood prediction accuracy. This will include the installation of a network of rainfall and flow monitoring stations that may have previously been seen to be the responsibility of the DWS and SAWS.

10.1 Uncertainty

Two types of uncertainty are identified for design hydrology (Ball et al., 2019):

(a) Inherent (aleatory) uncertainty due to randomness or natural variability of hydrological systems, and

(b) Knowledge-based (epistemic) uncertainty where the methods used in predicting the outcomes of the hydrological processes are limited by the state of knowledge of the processes. This will include the problem of limited data.

Pappenberger and Beven (2006) consider uncertainty analysis part of good scientific practice in applying hydrological prediction methods. Methods for uncertainty estimation include the likes of Monte Carlo Simulations that have achieved common practice in water balance analyses in South Africa, but much less so in design flood estimation. The objectives of uncertainty analysis is closely associated with that of calibration; the latter will provide a measure of accuracy while understanding uncertainty will improve the understanding of the safe range of application of a DFE method. Sources of uncertainty in DFE include (Ball et al., 2019):

(a) Data uncertainty,

(b) Parametric uncertainty,

(c) Structural uncertainty (relating to the mathematical model being used and the inherent assumptions),

(d) Regionalisation uncertainty (relating to the application of regional parameters to a local site),

(e) Predictive uncertainty (including combinations of those above), and

(f) Deep uncertainty (relating to the robustness of the DFE methods where there may be unknown weaknesses in the methods that may affect, for example, predictions in future conditions under climate change.

The reasons for the slow uptake of uncertainty analysis in DFE appear to be like those linked to the resistance to calibration; it is difficult to perform, and too subjective, the results are not easily understood by policy makers, it's not useful in understanding hydrological and hydraulic processes, and there are even those that feel uncertainty analysis is not relevant to physically based (deterministic) models (Pappenberger and Beven, 2006).

For the current level DFE practice in South Africa it is proposed that uncertainty analysis is part of best practice rather than good practice. It may be developed for application in large metropolitan catchments where integrated flood management plans are prepared. It may be applied to smaller municipalities once calibration becomes more mainstream, but actioned more infrequently, perhaps once every few years (see recommendations below). It is less likely to be adopted for on-site and single point risk analyses where instead municipalities will seek to develop regional parameters (across the municipality) such that accuracy is improved, and uncertainty is reduced. Hence uncertainty analysis for DFE is not explored in detail in this version of the guideline, though this can be reviewed in future updates. However, this overview should alert practitioners and municipal engineers on the significance of uncertainty and the potential support to decision making that uncertainty analysis may offer.

10.2 Accuracy

The accuracy of the DFE is the degree to which the estimated design flood accurately reflects the magnitude and timing of the flood event that is likely to occur. Chong-yu Xu (2021) emphasises it is also important to consider the performance of a DFE method over the range of return periods and catchment conditions it is to be applied.

The accuracy required for DFE is seldom, if ever, stated. Due to the uncertainties of hydrological science, accuracy can only be confirmed on a site-by-site basis. The accuracy of different methods is sometimes expressed in relative terms that imply that certain methods are likely to be more accurate than others (Table 10-1). Accuracy is typically linked to the level of detail to which the hydrological processes are represented in the methods.

Alexander (2002) states that all DFE methods in use in South Africa at the time "have a wide, unquantifiable band of uncertainty" around their estimates of the flood magnitude-frequency relationship at any location. This is because the associated hydrological and meteorological processes have numerically unquantifiable upper limits. Accuracy appears to have improved substantially with the introduction of catchment modelling methods. Applying continuous simulation to calibrate an urban catchment Ball (2021) proposes that a target accuracy of 10% across a range of flood magnitudes is achievable.

Table 10-1: Summary of accuracy of urban flood models mentioned in this guideline (after Loots & Smithers, 2020)

Model	Accuracy
Rational Method	Coefficient selection is subjective, so accuracy can be poor
SCS Method	Less subjective than Rational method, but CN-value is approach-dependent.
ACRU	Relatively accurate, but further verification was recommended.
Mike Urban & SWMM	Seen as relatively accurate but most accurate where 2D overland flow is used.

The accuracy of the estimate is only measured by comparing the estimated design flood to recent events or historical flood data. In the absence of such data, both the municipal engineer and DFE practitioner need defendable approach.

10.3 Accuracy vs Consistency

Practitioners who are aware of the importance of accuracy may argue that adequate calibration data is rarely available, and that consistency is therefore more important. Rowley and Wilkinson examined the relative benefits and disadvantages of the two objectives in urban stormwater management (Table 10-2). The potential implications are perhaps better represented in Figure 10-1.

There may be an inclination to adopt consistency over accuracy as possibly a practical or pragmatic approach, but the inherent bias should not be ignored. Without ever testing the extent of the bias, the magnitude of the error will never be known.

Table 10-2: Accuracy vs Consistency (after Rowley & Wilkinson, 2018)

	Accuracy	Consistency
The not so good	• Moving target. • Expensive and time consuming. • Hard to verify. • Numerous variables. • Site specific, not regional.	• Not representative of actual condition. • Relies on assumptions in lieu of latest data. • Based on management decisions. • Standards may be outdated.
The good	• Reflects actual flows. • Facilities sized appropriately. • Defendable. • Model fidelity can be very good. • Useful for actual events or flood response.	• Cost effective. • Globally manage assumptions/models. • Builds upon previous plan. • Continuity over long term. • Works at a regional level.

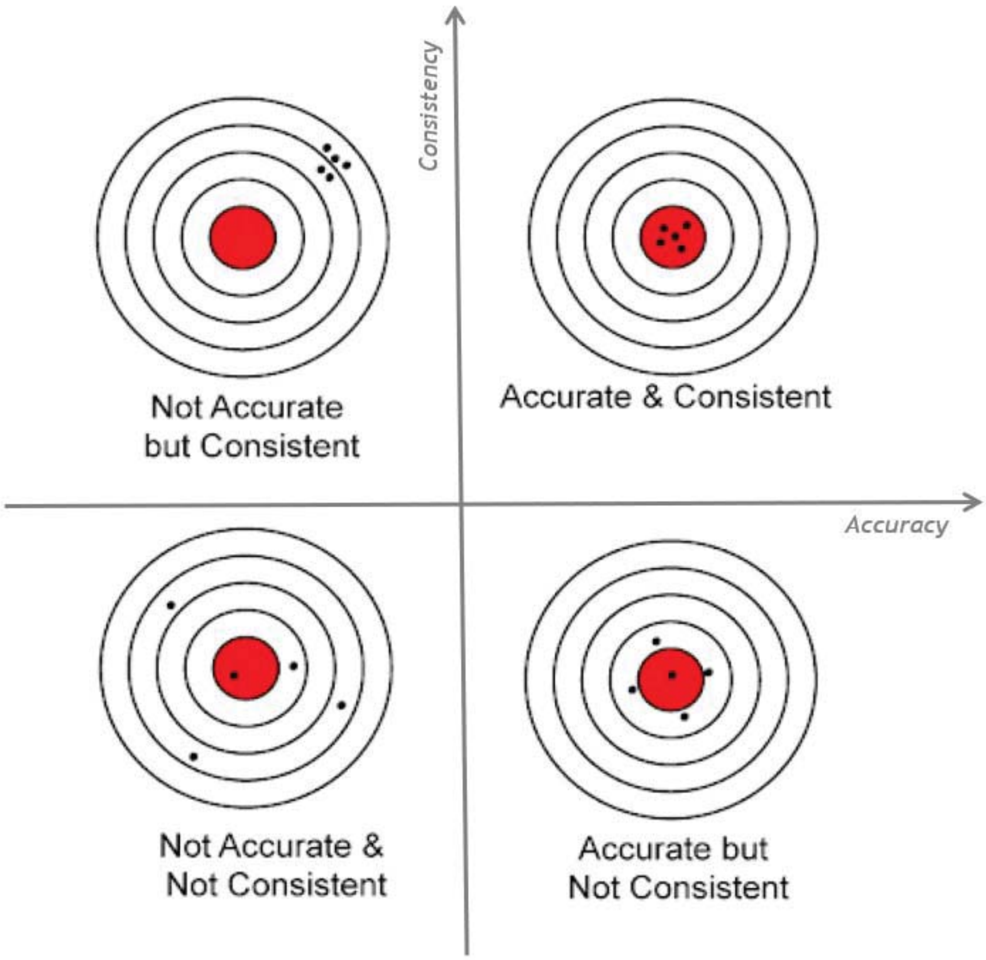

Figure 10-1: Consistency and accuracy

10.4 Calibration

Calibration is another foundation of hydrological science. All prediction methods that include naturally variable catchment parameters need calibration to confirm accuracy, including the Rational Method. There are a range of calibration methods in practice, and they continue to be developed (Chong-yu Xu, 2021).

Calibration requires measured or observed historical data, typically measured rainfall in the catchment and measured flow rates at or near the point of interest. Other useful data would include measures of the physical condition of catchment characteristics that are used to define the parameters in the DFE method (soils, soil saturation, condition of land cover, bridge blockages, etc.). All too often practitioners claim that very little, if any useful data is available and calibration is therefore impossible. Experience suggests that instead it is rare that a practitioner will investigate whether any data is available.

Calibration is part of good practice to be applied on a regular basis. In each DFE the practitioner should address calibration. In applications where accuracy is of limited concern calibration will not be necessary. These may include culvert and bridge sizing where oversizing is not a financial concern, or where there are low hazard conditions or where land sterilisation is not a concern. This would be explained in the section on calibration in the DFE report. Otherwise, there should be a description of the calibration undertaken (e.g. high level or detailed), or whether there has been agreement with the municipal engineer that calibration is not required.

Like the reasons given for uncertainty analysis, it is not intended to address calibration methods in detail in this version of the guideline. As calibration data availability improves over time, municipalities and practitioners may explore the different calibration methods available. Instead, attention is given to the development of hydrological databases for calibration, and sources of data already within communities that can provide a measure of model calibration immediately.

10.5 Recommendations

It is recommended that municipalities take charge of addressing the issues of uncertainty and accuracy of DFE in their area of jurisdiction. At the centre of the effort will be the development of data for calibration and ensuring that there are regular checks of accuracy of the DFE methods being applied. Actions include:

(a) Develop a consistent approach to DFE in the municipality, including calibration as a standard part of good practice. This may include different methods being applied to different situations, hazards and types of land development.

(b) Test the performance (accuracy) of the DFE methods by calibration on a regular basis. This may be done in-house or through the practitioners appointed by developers.

(c) Establish a network of weather and flow (or water level) stations across the catchments affecting flows in the municipal boundary.

(d) Encourage communities to monitor and share data. Domestic rain gauge data, river and culvert observations, high water marks, etc. are all useful for compiling flood reports and can be valuable for model calibration.

(e) Collect and collate geotechnical data received as part of land development applications. This can be checked against soils and land cover databases (see Chapter 5) and help develop a site-specific soils and land cover data set for the municipality.

(f) Establish a managed repository for data storage, ideally with a team to continually analyse and review the data for quality and consistency.

Initially the frequency of calibration may be fairly low (e.g. perhaps one or two calibrations across a small municipality per year), but will become more frequent as data becomes more available (e.g. one or two per urban catchment per year). Calibration frequency may also be influenced by uncertainty analyses undertaken every perhaps every few years. This process of developing DFE accuracy through calibration and uncertainty analysis as part of municipal DFE requires further research.

Practitioners will be key participants in the calibration of DFE methods. Skills will need to be developed in calibration methods, especially where multi-parameter catchment models (e.g. SWMM) are being applied and where calibration event data is limited. An important source of calibration data will already be available within the local communities who are likely to be very happy to share information. Even a partial calibration can be of value:

- Two or more daily rainfall values for the same event in the vicinity of the study catchment will give an indication of rainfall distribution.
- If the event is recent, discussions with residents will give an indication of event timing and duration of the event.
- The same discussions can provide observed water levels in the river or drain that can be used to calculate peak flow rates and duration of flow.

The above detail can be used to obtain a sense of the catchment response and can be considered a very high level of calibration. It also achieves potentially important consultation with the local community. Clearly, recent events offer better recall than older events. As a general guide, rain gauge data and observed water levels (if observed in daylight) provided by local members of a community are found to be fairly reliable. This data can also offer substantial support to any formal records for the event and will improve calibration outcomes.

10.6 References

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11. APPENDIX 1: GLOSSARY OF TERMS

The following definitions are relevant to the planning and management of floods and flood lines. They will assist Municipalities in their responsibilities in development control and disaster management, and in their oversight role of practitioners undertaking design flood estimation.

Annual Exceedance Probability (AEP)	The probability of an event (rainfall or flood) being exceeded in any one year. (also see Return Period)
Antecedent conditions	The hydrological state of the catchment, storages (soil, attenuation facilities) and drainage network prior to the occurrence of a design event. These conditions are generally assumed or ignored in DFE. Average conditions may be adopted to preserve design probability, but ‘average’ may not necessarily be ‘dry’ or mid-range values of parameters. Sensitivity of flood responses to antecedent conditions will increase DFE uncertainty.
Consequence	The outcome or impact from a risk that may be social, economic or environmental.
Cost-benefit assessment	An evaluation of both the cost and the benefits of a planned flood management intervention. Ideally the value of the benefits should be greater than the cost of the intervention.
Continuous simulation	Computation of a time series of any objective function, e.g. discharge in a watercourse using a representative input time series, generally rainfall, to drive the response of a catchment model
Design life (structures)	The desired life or the amortisation period (loan repayment period) of a flood management measure. Setting the design flood standard for a structure or flood management intervention is important in the context of design life, risk and flood tolerance. The probability that an undesirable event (of return period T) will occur within the design life (N) of a structure is given by Eqn. 6-2.

Design life (flood lines)	This is not defined in the South African context but is a particular problem in expanding towns and cities where factors influencing hydrological flood responses are not constant (see stationarity below). It is also a problem where different methods of DFE and/or hydraulic analysis are applied in determining flood lines in sections along the length of a watercourse. With authority over all development, a Municipality will be responsible for overseeing the consistency and suitability of flood lines on all watercourses that will assure the safety and viability (design life) of riparian properties It is recommended that any floodline determination older than 5 years should trigger an automatic review and updated calculation required unless it is clear that changes to the catchment and to the watercourse geometry have been negligible. Any floodlines older than 10 years should be recalculated.
Event-based analysis (Single event analysis)	
Flood duration	The period of time during which the discharge does not drop below a given limit or water level, thus defining the hydrograph shape.
Flood hydrograph (flood wave)	The discharge in the river from the time of commencement of the flood until the discharge returns to normal
Flood level	This typically refers to the maximum water level reached during a flood event at a location on a watercourse or stormwater system. Note that the maximum flood level does not necessarily coincide with the flood peak due to the natural attenuating effect of hydraulic resistance (natural or artificial) in the watercourse.
Flood line	The ground elevation along a watercourse at which the maximum water surface design flood shall reach. It results from the combined effect of the flow rate and hydraulic conditions in the watercourse during the estimated design flood event.
Flood peak (peak discharge)	The maximum flow rate during a flood. Peak discharge is the most useful parameter and does not remain constant while a flood wave moves along a watercourse, changes are fairly gradual where there are no tributaries and they are independent of local changes in the watercourse.
Flood volume	The volume of water in flood wave above a given discharge.
Hazard	A situation with the potential to result in harm.

Heterogeneity	A measure of the variation of a data set, essentially the possibility that a selected part of a data set will not have the same statistical properties as the overall data set.
Probability	The likelihood of a certain event occurring in any year. This can be expressed as a fraction (e.g. the 100-year event = 0.01, see below), a percentage chance (e.g. a 1% flood) or in gaming terminology (e.g. a 100 to 1 flood). If all other factors remain the same, the likelihood of a certain flood event remains the same every year, given by the inverse of Eqn. 6-1, i.e.. $Probability, \quad p = \frac{1}{T}$
Residual risk	The risk that remains after management and mitigation measures have been applied.
Return Period (Return Interval or Recurrence Interval)	The average elapsed time between occurrences of an event with a certain magnitude or greater. It converts probability to a relative frequency as given in Eqn. 6-1.
Risk	The combination of probability and consequence.
Risk assessment	The evaluation of the potential hazards and the risks associated with them.
Stationarity	Where all potential external factors are presumed constant. This is a key assumption in DFE. External factors include, for example, climate and catchment development. These are often not constant and can have significant bearing on the uncertainty of DFE.
Tolerability	The willingness to live with a risk to secure certain benefits and in the confidence that it is being properly controlled. Tolerability requires an understanding of the risks and the regular review of them.
Uncertainty	Both baseline (e.g. catchment) conditions and storm event conditions are dynamic and lead to variations and uncertainty in hydrological responses (Ref 2). Limited local data, event observations (monitoring) and on-site measurement of hydrological and hydraulic parameters will increase uncertainty in DFE. This is in turn transferred to the design decisions for flood management interventions.
Vulnerability	This refers to the resilience of a community, sector or environment to respond to a hazardous condition.

12. APPENDIX 2: LEGISLATION

12.1 Reporting Standards

The local authority should prepare a template for the preparation of floodline determination reports, an example is the City of Tshwane's requirement for report contents given below.

12.1.1. City of Tshwane Floodline Report Contents

FLOOD LINE DETERMINATION REPORT GUIDELINE

(i) INTRODUCTION

(a) Site location and property description

(b) Objective purpose and scope of the report

(ii) HYDROLOGY

(a) Catchment overview

- Provide a description of study area
- Topographical detail of the catchment
- General geology of the catchment

(b) Regional climate and storm rainfall information

(c) Catchment delineation and detail

(d) Hydrological parameters and choice of hydrological methods used (min 3 must be used)

(a) Rational method (<15 km^2)

(b) Alternative rational methods (<15 km^2)

(c) SWMM model (no limitations)

(d) Standard design flood (SDF) method (no limitations)

(e) Adjusted SDF (no limitations)

(f) Unit hydrograph method (no limitations)

(g) Empirical method (no limitations)

(h) Soil conservation services SA (SCS-SA) method (no limitations)

(i) Adjusted Brandsby-Williams (<15 km^2)

(j) Synthetic hydrograph method (no limitations)

(k) Illinois urban drainage area simulator (Illudas)

(l) Regional maximum flood (RMF) (Kovac's empirical method)

(e) Summary table with 1:50 and 1:100-year peak flow values

(f) Choice and motivation for proposed peak flow values

(g) Obtained peak run-off data from Tshwane RFMIS

(iii) HYDRAULIC INFORMATION

(a) Contours/survey

(b) Control sections

(c) Boundary conditions (upstream and downstream)

(d) Cross section design

- Flow changes
- Slope changes
- Possible obstructions
- Dams/weirs
- Samuel's equation or similar for cross section spacing

(e) Watercourse details (roughness)

(f) Hydraulic model used (HEC-RAS or similar)

(g) Detail of model parameters and reasons therefor

(iv) ANNEXURES

(a) Location map

(b) Catchment layout plan

(c) Layout plan

(a) Cadastral boundaries

(b) Watercourse centre line

(c) Contours

(d) Position of cross sections

(e) Position of any control points

(d) Flood line plan

 (a) Property description

 (b) Cadastral boundaries, contours, watercourse centre line

 (c) 1:50 and 1:100-year flood lines

 (d) Flood lines certification (signature/Reg. no./date)

 (e) Table with the 1:50 and 1:100-year peak run off values

(a) Long section with flow depths

(b) Cross sections with flow depths

(c) Hydrology calculation sheets

(d) Summary table of hydraulic model outputs

12.1.2. eThekwini Municipality Floodline Strategy

ANNEXURE 1 (To the Flood line and Stormwater Management Policy)

ETHEKWINI MUNICIPALITY FLOODLINE STRATEGY ANNEXURE

BACKGROUND

The eThekwini Municipal area is crossed by many valley lines, all of which carry flood waters downstream and ultimately to the sea. These floods are unpredictable and if unmanaged can cause destruction of property and loss of life. Added to this, is the predicted climate change impacts which talk of an increase in rainfall intensity and an associated increase in flood levels.

The flood related damage, in addition to the loss of property, infrastructure and life, ultimately has a negative effect on the economy of the city in that clean up and repair costs whether they be private or public funds, are being used to restore the present situation rather than improve or grow the city.

Furthermore, it is imperative for the eThekwini Municipality to play its required role in ensuring that new development is not only protected from the risk of flooding but that the new development does not have a negative effect on existing development.

In light of the above it is imperative that the flood line policy cover all flood line aspects regarding development in both the public and private sector, within the eThekwini Municipal area.

This Annexure relates to any site development works on any parcel of land within the eThekwini Municipal area.

No person unless he has obtained prior written approval from the authorised officer will be permitted to carry out, cause or permit to be carried out, any site development works including sub-divisional proposals, situated in any area, subject to inundation by floodwaters resulting from any watercourse with a known and defined channel and with a catchment area exceeding one square kilometre from a storm with a frequency of more than 1 in 100 years

<u>DEFINITIONS</u>

- **Site development works**
  - ➢ Shall mean: Any earthworks upon premises the result of which would permanently change the level of any portion of the surface of the ground upon the premises.

- **M.S.L. (Mean Sea Level)**
  - ➢ A vertical datum provides the reference surface from which all heights are measured. Classically, vertical datums are defined in terms of the geoid, or some approximation to it. As the geoid is, in turn, defined in terms of some average of mean sea level (MSL), it has become customary to define the vertical datum with respect to tide gauge measurements of MSL. The vertical datum in South Africa is referred to as the Land Levelling Datum (LLD), and is based upon tide gauge measurements made more than a century ago. A network of precise levelling benchmarks is tied to this datum, as are the heighted town survey marks in the urban areas. Less directly, the heights of trigonometrical beacons also refer to LLD.

- **1 in 100 year flood**
  - ➢ The term “1 in 100 year flood” is misleading because it leads people to believe that it happens only once every 100 years. The truth is that an uncommonly big flood can happen any year. The term “100-year flood” is really a statistical designation, and there is a 1-in-100 chance that a flood this size will happen during any year. Perhaps a better term would be the “1-in-100 year chance flood”.

    The actual number of years between floods of any given size varies a lot. Big floods happen irregularly because the climate naturally varies over many years. We sometimes get big floods in successive or nearly successive years with several very wet years in a row.

ANNEXURE REQUIREMENTS

Before proceeding with future development on the property:

Professional advice should be sought from a registered professional engineer/technologist specialising in hydrological and hydraulic design work where upon the professional engineer/technologist will be required to provide the following information in relation to the development of the property, as listed below.

Additional information may be requested in respect of any particular development.

1. **REQUIREMENTS FOR ALL DEVELOPMENT**

   1.1 The level in metres M.S.L. of the 1 in 50 year and 1 in 100 year flood line on the property.

   1.2 Calculations by the Engineer indicating method of determining the flood line employed.

   1.3 The position of the flood line on the property, whether the proposed development is inside or outside the flood plain and the floor level of the proposed development in metres M.S.L. where appropriate.

   1.4 The information from section 1.3 is to be shown on all plans relating to the development.

   1.5 If all or part of the development including the earthworks is inside the existing 1 in 50 year flood plain, then the Engineer must certify:

      a. that the foundations are capable of withstanding any forces or flood effects on the development and be prepared and able to provide design calculation to that effect if called upon to do so;

      b. that the development in the flood plain will not affect the flooding or flood levels on adjacent properties, neither upstream/ downstream nor opposite properties, as the case may be, and such certification is to be backed up by calculations.

2. **REQUIREMENTS FOR ALL RESIDENTIAL DEVELOPMENTS (NEW OR EXTENSIONS)**

In addition to all requirements as indicated in section 1. (1.1 to 1.5), the following information is required;

2.1 The floor level of the proposed new development given in 1.4 above is below the 1 in 50 year flood level the development will not be approved. Extensions to existing development may be allowed within the 1 in 50 year flood plain provided that such extensions are not used for habitable purposes e.g. en-suite bathrooms may be permitted.

2.2 If the floor level of the proposed development given in 1.4 above is above the 1 in 50 year flood level, then the Engineer must certify that there is a continuous means of access (whether private or public) from the residential building to a public road all at a level above the 1 in 50 year flood level.

3. <u>REQUIREMENTS FOR SUB-DIVISIONAL SCHEME APPROVAL</u>

3.1 In terms of Section.14 of the National Water Act.1998, all plans relating to the subdivision of land shall indicate the maximum level likely to be reached on an average of every <u>hundred years</u> by floodwaters, on the land in question, emanating from any watercourse with a known and defined channel and with a catchment area exceeding one square kilometre.

3.2 All plans relating to the subdivision of land shall indicate the maximum flood level under a flood of recurrence period <u>1 in 50 years</u> from any such watercourse.

3.3 When necessary to show a flood line on a plan, an Engineer shall calculate the position of that flood line and the following minimum information should be provided with the sub-divisional application:-

a. The principle method of calculation and formula utilised for flood level prediction;

b. The assumed 1 in 50 year or a 1 in 100 year rainfall intensity in the catchment;

c. The catchment area taken in hectares;

d. The assumed catchment runoff factor where appropriate.

Deviation from this policy will only be considered in exceptional circumstances, provided that a <u>formal written application</u> has been forwarded <u>to the Deputy Head: Coastal, Stormwater and Catchment Management</u>.

Approved by Head: Engineering

12.2 Bylaws

It is strongly recommended that local authorities prepare bylaws on stormwater management, catchment management and land use in floodplains. Extracts from the bylaws of the City of Cape Town and the City of Johannesburg are given below.

12.2.1. City of Cape Town Stormwater Management Bylaws

Cape Town South Africa

Stormwater Management By-law, 2005

Published in Western Cape Provincial Gazette no. 6300 on 23 September 2005

Assented to on 30 August 2005

Commenced on 23 September 2005

[Up to date as at 8 December 2021]

1. Definitions

In this by-law, unless inconsistent with the Context: -

"Council" means the municipal council of the City of Cape Town, or any political structure, political office-bearer, committee, councillor, or official of the council, delegated to exercise powers or perform duties in terms of this by-law;

"floodplain" means the land adjoining a watercourse which, in the opinion of the Council, is susceptible to inundation by floods up to the one hundred year recurrence interval;

"private stormwater system" means a stormwater system owned, operated or maintained by a person other than the Council;

"stormwater" means water resulting front natural precipitation and/or the end accumulation thereof and includes groundwater and spring water ordinarily conveyed by the stormwater system, as well as sea water within estuaries, but excludes water in a drinking water or waste water reticulation system;

"stormwater system" means both the constructed and natural facilities, including pipes, culverts, watercourses and their associated floodplains, whether over or under public or privately owned land, used or required for the management, collection, conveyance, temporary storage, control, monitoring, treatment, use and disposal of stormwater;

"water pollution incident" means an incident or occurrence which has a detrimental impact on a potential detrimental impact on the quality of the water in the stormwater system to such an extent that public health or the health of natural ecosystems may be threatened, and "watercourse" means: -

(a) a river, spring, stream, channel or canal in which water flows regularly or intermittently, and

(b) a vlei, wetland, dam or lake into which or from which water flows, and includes, where relevant, the bed and the banks of such watercourses.

2. Application

(1) This by-law binds any organ of state.

(2) Any provision in any other by-law dealing specifically with stormwater, is subject to the provisions of this by-law.

3. Prohibited discharges

No person may, except with the written consent of the Council and subject to any conditions it may impose, discharge, permit to enter or place anything other than stormwater into the stormwater system.

4. Protection of stormwater system

No person may, except with the written consent of the Council and subject to any conditions it may impose-

(a) damage, endanger, destroy or undertake any action likely to damage, endanger or destroy, the stormwater system or the operation thereof;

(b) discharge from any place, or place onto any surface, any substance other than stormwater, where that substance could reasonably be expected to find its way into the stormwater system;

(c) discharge, permit to enter or place anything likely to damage the stormwater system or interfere with the operation thereof or contaminate or pollute the water therein;

(d) construct or erect any structure or thing over or in such a position or in such a manner so as to interfere with or endanger the stormwater system or the operation thereof; or

(e) make an opening into a stormwater pipe, canal or culvert; or

(f) drain, abstract or divert any water directly from the stormwater system, or

(g) fill, excavate, shape, landscape, open up or remove the ground above, within, under or immediately next to any part of the stormwater system.

5. Prevention of flood risk

No person may, except with the written consent of the Council and subject to any conditions it may impose-

(a) obstruct or reduce the capacity of the stormwater system;

(b) change the design or the use of, or otherwise modify any aspect of the stormwater system which, alone or in combination with other existing or potential land uses, may cause an increase in flood levels or create a potential flood risk; or

(c) undertake any activity which, alone or in combination with other existing or future activities, may cause an increase in flood levels or create a potential flood risk.

6. Studies and assessments

(1) The conditions which the Council may impose in terms of Sections 3, 4, and 5, may include, but are not limited to -

(a) the establishment of flood lines;

(b) the undertaking of impact assessments, and

(c) environmental impact studies or investigations which may be required by any applicable environmental legislation.

(2) The costs of any study undertaken in terms of the provisions of subsection (1), will be for the account of the applicant.

7. Water pollution incidents

In the event of an incident contemplated in Section 3 or Section 4(b) and (c) -

(a) the owner of the property on which the incident took place, or is still in the process of taking place, or

(b) the person responsible for the incident, if the incident is not the result of natural causes,

shall immediately report the incident to the council, and at own cost, take all reasonable measures which, in the opinion of the Council, will contain and minimise the effects of the pollution, by undertaking cleaning up procedures, including the rehabilitation of the environment, as required by the Council.

8. Stormwater systems on private land

(1) Every owner of property on which private stormwater systems are located, shall-

(a) not carry out any activity which will or which, in the opinion of the Council, could reasonably be expected to impair the effective functioning of the stormwater system, and

(b) at own cost, keep such stormwater systems functioning effectively, including undertaking the refurbishment and reconstruction thereof if, in the opinion of the Council, it should be reconstructed or refurbished.

(2) The provisions of subsection (1) do not apply to the extent that the Council has accepted responsibility for any of the duties contained therein, either in a formal maintenance agreement or in terms of a condition of a servitude.

9. Provision of infrastructure

(1) The Council may-

(a) construct, expand, alter, maintain or lay any drains, pipes or other structures related to the stormwater system on or under any immovable property, and ownership of these drains, pipes or structures shall vest in the municipality;

(b) drain stormwater or discharge water from any municipal service works into any natural watercourse, and

(c) do any other thing necessary or desirable for or incidental, supplementary or ancillary to any matter contemplated by subsection (a).

(2) When the Council exercises its powers in terms of subsection (1)(a) in regard to immovable property not owned by the municipality, it shall comply with the provisions of the By-law Relating to the Management and Administration of the City of Cape Town's immovable Property.

10. Powers of the council

(1) The Council may-

(a) demolish, alter or otherwise deal with any building, structure or other thing constructed, erected or laid in contravention of the provisions of this by-law;

(b) fill in, remove and make good any ground excavated, removed or placed in contravention with the provisions of this by-law;

(c) repair and make good any damage done in contravention of the provisions of this by-law or resulting from a contravention;

(d) remove anything discharged, permitted to enter into the stormwater system or natural watercourse in contravention of the provisions of this by-law;

(e) remove anything damaging, obstructing or endangering or likely to obstruct, endanger or destroy any part of the stormwater system;

(f) seal off or block any point of discharge from any premises if such discharge point is in contravention of the provisions of this by-law, irrespective of whether the point is used for lawful purposes;

(g) cancel any permission granted in terms of this by-law if the conditions under which the permission was granted are not complied with;

(h) by written notice, direct any owner of property to allow the owner of a higher lying property to lay a stormwater drain pipe or gutter over his or her property for the draining of concentrated stormwater,

(i) by written notice, direct any owner of property to retain stormwater on such property or, at the cost of such owner, to lay a drain pipe or gutter to a suitable place indicated by the Council, irrespective of whether the course of the pipe or gutter will run over private property or not, and

(j) discharge stormwater into an watercourse, whether on private land or not.

(2) The Council may, in any case where it seems that any action or neglect by any person or owner of property may lead to a contravention of the provisions of this by-law, give notice in writing to such person or owner of property to comply to such requirements as the Council may deem necessary to prevent the occurrence of such contravention.

(3) The Council may recover all reasonable costs incurred as a result of action taken in terms of subsection (1) from a person who was responsible for a contravention of the provisions of this by-law or the owner of the property on which a contravention occurred.

11. Offences and penalties

(1) Any person who-

(a) contravenes any provision of this by-law;

(b) fails to comply with the terms of any notice issued in terms of this by-law;

(c) threatens, resists, hinders or obstructs or uses foul, abusive or insulting language towards or at a councillor or an employee or contractor of the Council in the exercise of any powers or performance of any duties or function in terms of this by-law, or falsely holds himself or herself to be a councillor or an employee or a contractor of the Council,

shall be guilty of an offence and be liable, on conviction, to the payment of a fine.

12.2.2. Extract from the City of Johannesburg Stormwater Management Bylaws

Johannesburg South Africa

Stormwater Management By-law, 2010

Published in Gauteng Provincial Gazette no. 181 on 25 October 2010

Commenced on 25 October 2010

[Up to date as at 4 January 2022]

The Municipal Manager of the City of Johannesburg Metropolitan Municipality hereby, in terms of Section 13(a) of the Local Government: Municipal Systems Act, 2000 (Act No 32 of 2000), publishes the Storm water By-laws for the City of Johannesburg Metropolitan Municipality as approved by its Council set out here-under.

Chapter 1

Interpretation, purpose and application and responsibility for complying with by-laws

1. Definitions

In these By-laws, unless the context otherwise indicates -

"Agency" means the Johannesburg Roads Agency (Pty) Ltd., established by the Council as a service provider fulfilling a responsibility under these By-laws which responsibility has been assigned to It in terms of section 81(2) of the Local Government: Municipal Systems Act, 2000 (Act No 32 of 2000), and in relation to a situation where there is no Agency or other service provider contemplated in that section, means the Council;

"Buffer" means an area or strip of land on a development site or property, which is to be, or is utilised for the management of stormwater or conservation of the riparian habitat as defined in section 1 of the National Water Act, 1998 (Act No 36 of 1998);

"catchment area" means an area of land in its natural state, from which stormwater runoff originates;

"Completion certificate" means the written acknowledgement by the Agency of the satisfactory completion of all work on a construction site approved by the Council, including any work shown on the approved building plans or approved plans of a township concerning the provision of municipal infrastructure, and any revision of such plans and field change approved by the Council;

"design storm event," means a theoretical storm event which generates stormwater, of a given frequency interval and duration, used in the analysis and design of a stormwater facility;

"developer" means any person undertaking or proposing to undertake a development and includes a developer of a township;

"diversion" means the routing of stormwater in a direction other than its natural discharge direction and "divert" has a corresponding meaning;

"floodplain" means an area of land adjacent to a watercourse, or water body, with a catchment area exceeding 30 ha that will be inundated by floodwater on average once in a 100 years as determined by a professional engineer, on the basis that the minimum width of a floodplain is 32 m on each side of the centre line of the water course or water body;

"hydrograph" means a graph indicating stormwater runoff rate, inflow rate and discharge rate of stormwater, past a specific point over a time period;

"hydrograph method" means a method of estimating a hydrograph, using a mathematical simulation;

"post-development condition" means the condition of any property after the conclusion of development thereon and "post development" has a corresponding meaning;

"Pre-development condition" means the condition of any property or portion of a property as it existed in its unaltered natural state prior to any development on that property and "pre-development" has a corresponding meaning;

"professional engineer-means any person who is registered with the Engineering Council of South Africa as a professional engineer or a professional engineering technologist, who is qualified in the engineering field concerned and who is considered competent by the Agency and who has been approved by it;

"stormwater" means the surface stormwater runoff that results from-any natural form of precipitation of water or moisture in any form;

"stormwater drainage system" means every stormwater drainage facility and stormwater drainage feature forming part of a system that combines to lead stormwater from a higher lying area;

"stormwater system" means any natural or man-made system which functions independently or together with another such system to collect, convey, store, purify,

infiltrate and discharge stormwater, including any stormwater facility and water course;
'

"township" means a township approved in terms of the Town-planning and Townships Ordinance, 1986 (Ordinance No 15 of 1966) or any other law;

"watercourse" means -

(a) a river or spring;

(b) a natural channel in which water flows regularly or intermittently;

(c) a wetland, lake or dam into which, or from which, water flows; and

(d) any collection of water which the Minister concerned may, in terms of the National Water Act, 1998 by notice in the Government Gazette, declare to be a watercourse, and a reference to a watercourse includes, where relevant, its bed and banks;

2. Purpose of By-laws ……………

3. Application of By-laws and manual ……………

4. Responsibility for complying with By-laws

(1) A developer who proposes to undertake or undertakes any work or action contemplated in these By-laws, is responsible for compliance, and for ensuring compliance, with any provision of these By-laws relating to such work or action.

(2) A contractor or agent appointed by a developer to carry out any work or action contemplated in these By-laws is jointly and severally responsible with that developer for compliance, and for ensuring compliance, with any provision of these By-laws relating to such work or action.

(3) An owner of property which has been developed, is responsible for compliance, and for ensuring compliance, with any provision of these By-laws which is applicable in respect of that property after conclusion of that development.

Chapter 2

Site development activity permits

5. Permits required

A permit Is required for any of the following site development activities: …………..

6. Exceptions to permit requirements ………………….

7. Applications for permits ………………….

8. Expiry of permits ……………………

9. Professional engineer required

Any document contemplated in section 7(3)(b), (c) and (d) must be prepared by a professional engineer, if any of the following conditions exists:

(a) The proposed development on the property concerned constitutes a major development;

(b) if the site development activity incorporates any stormwater facility or other improvement relating to stormwater in a public road for which facility or improvement the Agency will assume responsibility for maintenance;

(c) if a site development activity is to take place within a floodplain or within 100 m from the centre line of any water course;

(d) If a site develop activity is to take place on a property shown by a 15 000 scale geological map to be underlain by, or within 500 m of, dolomitic geology; or

(e) in respect of any other site development activity, if the Agency considers it to be in the public interest to require that the documents concerned be prepared by a professional engineer.

10. Site development activity plan

A developer must in respect of any development site in respect of which a permit is required in terms of section 7, prepare a site development activity plan.

11. Stormwater drainage plan

(1) A developer must in respect of any development site for which a permit is required in terms of section 7, prepare a stormwater drainage plan.

(2) A plan contemplated in subsection (1), must relate to the collection, transportation, treatment and discharge of stormwater from the development site concerned and must include a profile view of the property concerned and construction details and notes relevant to such plan.

(3)

(a) A plan contemplated in subsection (1) must contain an analysis of the impact of stormwater quantity up to 500 m or a greater distance required by a notice in writing by an authorised official served on the developer concerned, downstream from the property on which the development site concerned is situated, which may result from the proposed development on that site and must contain features to mitigate such impact.

(b) For the purposes of paragraph (a), any existing and potential impact of stormwater, including -

(i) increased sedimentation and streambank erosion and discharge;

(ii) flooding;

(iii) overcharging of any existing closed stormwater conveyance facility;

(iv) discharge to a closed depression;

(v) discharge to an existing off-site stormwater runoff control facility;

(vi) infiltration to groundwater or any area of land with the ability to contribute to, or recharge, ground water;

(vii) deterioration of stormwater quality; and

(viii) any spill and discharge of a pollutant into stormwater, must be evaluated and mitigated.

12. Off-site stormwater drainage analyses ………………………

13. Geotechnical reports ……………………….

14. Soils Investigations reports …………………………..

15. Permit modifications …………………………….

Chapter 3

Erosion and sediment control

Part 1 – Minor developments

16. Control for minor developments ………………

17. Requirements for minor developments ………………..

Part 2 – Major developments

18. Provisions applicable to major developments ……………

19. Stormwater control measures for major developments ……………

20. Control of off-site erosion ……………

21. Stabilisation of temporary conveyance channels and outlets ………………………

22. Stormwater drain inlet protection ………………………….

23. Trenches for municipal services ……………………………

24. Constructed access routes …………………………….

25. Removal of temporary facilities ………………………….

26. Dewatering of development sites …………………

27. Control of pollution other than sediment ………………..

28. Maintenance of erosion and sediment control facilities …………………..

29. Erosion control design storm event

Any stormwater facility designed for the control of erosion and sediment, must be designed for a 2-year recurrence interval design storm event of any duration from 0.25 hours to seven days.

30. Installation of rain meter

For the purpose of ensuring compliance with the relevant provisions of this Part, an authorised official may, by notice in writing, require a developer to install and maintain a rain meter, of a kind required by that official, on a development site and to furnish the Agency with a written statement within two days after the conclusion of every week, specifying the quantity of rainfall that fell on that site during the previous week.

Chapter 4

Grading

31. Grading plans ……………………….

32. Drainage ……………………….

33. Change in topography of development site ……………………

34. Maintenance ……………………

Chapter 5

Stormwater management

Part 1 – Major developments

35. Application

If a proposed development on any property constitutes a major development, the requirements of this Part apply in respect of the development site concerned.

36. Development activities

(1) If one or more of the following conditions exist on a development site, the requirements of this Part apply to the maximum extent practically possible in respect of that site, and in respect of any adjacent property which is part of the development:

(a) a development site greater than 4000 m^2 in area in size with 40 per cent or more impervious surface, prior to commencement of the development;

(b) A development site from which stormwater is discharged to a watercourse or water body which has a water quality problem documented in the records of the Council or the Agency, and includes, but is not limited to, a watercourse and water body -

(i) listed in a report required under the National Water Act, 1996, and designated as not being beneficially used; or

(ii) listed under the National Water Act, 1998, as not expected to meet water quality standards or water quality goals contemplated in that Act; and

(c) a development site in respect of which the need for stormwater control measures additional to those applicable to that site has been identified by an authorised official.

37. Approved hydrological methods for design

(1) For the purposes of any estimation of peak stormwater runoff rate used in the design of any stormwater quantity control facility, a hydrograph method of analysis approved by the Agency must be utilised.

(2) Any storage facility that forms a part of a storm water quality control facility must be designed by using a method approved by the Agency.

(3) Any calculation method used for a design contemplated in subsection (1), must be described, the value of any parameter and variable must be stated, and the reason for selecting a specific range of values must be set out in a design report for a proposed storm water management strategy for the property concerned, prepared on behalf of a developer and such report must be submitted to the Agency for approval.

38. Stormwater quantity control

(1) Subject to the provisions of subsection (2), the following requirements for stormwater quantity control apply:

(a)

(i) All stormwater entering a development site in its predevelopment state from a depression or conduit must be received on that site at a naturally occurring or otherwise legally existing location;

(ii) all stormwater leaving a development site must at all times during and after development, be discharged at a naturally occurring or otherwise legally existing discharge location so as not to be diverted onto or away from any adjacent downstream property: Provided that a diversion which will correct an existing downstream stormwater problem, may, on written application by a developer on a form prescribed by the Agency, be permitted in writing by an authorised official;

(iii) for the purpose of this paragraph "naturally occurring location" means the location of any watercourse, channel, depression or marshy area existing as an established system, identifiable on a topographic representation of the property in the records of the Council, either from a map, photograph, site inspection, decision of a court of law or other means approved in writing by the Agency;

(b) the post-development peak stormwater discharge rate from a development site for a 5- to 25-year recurrence interval design storm event of any duration from 0.25 to 24 hours, or any other design storm event stipulated by the Agency up to and including a

50-year design storm event, may not at any time exceed the pre-development peak stormwater runoff rate from that site for the same design storm event;

(c) any closed depression which receives stormwater discharge from a development site must be analysed using a hydrograph method for routing stormwater, and infiltration relating to such depression must be addressed, if relevant;

(d) If a proposed development will result in a discharge of stormwater to a closed natural depression that has a water surface area greater than 500 ma at overflow elevation, the following requirements must be complied with for the purpose of an analysis contemplated in paragraph (c):

(i) the stormwater runoff hydrograph from a 100-year design storm event, of any duration from 24 hours to seven days from the pre-development catchment area draining to a closed depression contemplated in paragraph (c), must be routed into that depression using only infiltration as outflow from the depression;

(ii) if a portion of such closed depression is located off the development site concerned, the impact of stormwater on any adjacent property must be taken into account;

(iii) if overflow of such closed depression occurs, the closed depression must be analysed as a detention or infiltration pond, to determine whether the depression can safely cope with the expected quantity of stormwater;

(iv) no discharge from a closed depression may exceed the discharge rate from that depression immediately prior to the development, resulting from a 2, 10, 25 and 100-year design storm event of any duration from 0.25 hours to seven days and a control structure to regulate outflow from such depression, an emergency overflow spillway and an access road must be provided and other design criteria required in writing by the Agency must be complied with;

(v) If a closed depression will be maintained by the Agency, a servitude in respect thereof must, subject to the provisions of section 43(1), be registered in favour of the Council to protect the Council's rights; and

(vi) if a development will create a stormwater runoff from the property concerned to a closed depression located off the development site, the volume of runoff discharged may not be increased beyond the effect of a 2,10,25 and 100-year design storm event of any duration from 0,25 hours to seven days;

(e) any stormwater quantity control facility to be provided, must be designed to meet, as a minimum performance standard, the requirements of this section, unless -

(i) stormwater from a development site will discharge to a Council stormwater system approved by an authorised official to receive stormwater from that site; or

(ii) stormwater from a development site discharges to a receiving body of water and ft can be demonstrated by the developer, to the satisfaction of an authorised official, that stormwater quantity control is not warranted;

(f) if the conditions downstream from a development site are determined by an authorised official to be exceptionally sensitive to potential stormwater discharges from that site compared to the situation immediately prior to the development, that official may, by notice in writing, require a factor of safety to be applied in respect of the total storage volume of any attenuation and detention facility and a reduction of the stormwater released from the site concerned;

(g) no attenuation facility or open stormwater quantity control facility may be located -

(i) in a public road;

(ii) on any land zoned as public open space under an applicable town planning scheme, without written approval of the Council;

(iii) in any floodplain below the 50-year floodline; or

(iv) in any wetland without approval of the Department of Water Affairs and Forestry;

(h) reasonable access to any stormwater facility to enable ease of maintenance, as determined by an authorised official, must be provided;

(i) if conditions on a development site are appropriate for infiltration of stormwater and ground water quality on that site is protected, streambank erosion control must be implemented, utilising infiltration to the fullest extent practicable; and

(j) any quantity control facility contemplated in paragraph (g), must be selected, designed and maintained according to the manual, and may not be built within a vegetated buffer, except for a stormwater conveyance system approved in writing by an authorised official and subject to the provisions of the National Environmental Management Act, 1998 (Act No 107 of 1998).

(2) The Agency may, if it considers that circumstances relating to stormwater management in respect of any development so requires, by notice in writing, require the developer concerned to comply with any additional requirement relating to control of the peak discharge or quantity of stormwater, specified in the notice.

(3) No person may do anything, which may interfere with the proper functioning and the ease of maintenance of any structure or facility contemplated in this section.

39. Combination of quality and quantity control facilities …………………………

40. Quality control requirements

(1) Subject to the provisions of subsection (2), the following requirements for storm water quality control apply:

(a) A best management practice concerning stormwater quality control must be utilised in respect of any stormwater facility relating to stormwater quality in respect of a development site, to the maximum extent practically possible;

(b) a stormwater facility for any treatment relating to the quality of stormwater must be of a size sufficient to hold and treat stormwater runoff from a 2-year recurrence interval design storm event of any duration;

(c) no structure relating to stormwater quality control may be built within a vegetated buffer, other than a conveyance system approved in writing by an authorised official;

(d)

(i) Treatment of stormwater discharge must be provided by utilising a wetpond or biofiltration or both, based on a best management practice: Provided that another best management practice may be utilised subject to the granting of a deviation or exemption from the provisions of the manual in terms of section 61;

(ii) a wetpond is required for a development site on which an impervious surface greater than 2 ha for use by motor vehicles, will be created by the development from which stormwater will discharge -

(aa) directly to a municipal or private regional stormwater facility or closed depression without providing stormwater quantity control on the development site concerned; or

(bb) directly or indirectly to a water course or wetland within 1 km downstream of the development site concerned;

(e) all stormwater must, prior to its discharge to a stormwater facility based on an appropriate best management practice and designed to utilise infiltration, pass through a stormwater treatment facility designed to remove suspended solids; and

(f) All stormwater from a development site on which heavy construction equipment is used, maintained or stored or on which any petroleum product is stored or transferred to such equipment or any vehicle, and from any vehicle washing bay, must be treated by an oil/ water separator of a size effectively to prevent pollution of such stormwater.

(2) The Agency may, if it considers that circumstances relating to stormwater quality control in respect of any development so require, or that the requirements of subsection (1) do not afford adequate protection for any water quality sensitive area on site or within the catchment area where the property concerned is situated, by notice in writing, require the developer concerned to comply with any additional requirement, relevant to such control, specified in that notice.

Part 2 – Major and minor developments

41. Application

If a proposed development on any property constitutes a major or minor development, the requirements of this part apply in respect of the development site concerned.

42. Stormwater drainage facilities

(1) An on-site stormwater drainage facility must be provided on every development site and must be of sufficient capacity to convey -

(a) stormwater without flooding or otherwise damaging any existing or proposed structure;

(b) any post-development peak stormwater runoff from a development site resulting from a 5-year recurrence interval design storm event, of any duration from 0.25 to 24 hours; and

(c) any existing stormwater runoff upstream from a development site that will be conveyed through that site, taking potential development upstream from the site into account.

(2)

(a) in estimating a peak stormwater runoff rate used in the design of a stormwater drainage facility contemplated in subsection (1), either the rational method as described in the manual, or a hydrograph method of analysis approved by the Agency in writing, must be used.

(b) The selection method, and all parameters or variables used in estimation in terms of paragraph (a), must be stated and explained in a design report contemplated in section 37(3).

(3)

(a) Any existing storm water facility and any other conveyance facility up to 500 m downstream from a development site that falls within the downstream portion of an off-site stormwater drainage analysis, contemplated in section 12 must have sufficient capacity to convey, without flooding or otherwise damaging any existing or proposed structure, on or off site, a post-development peak stormwater discharge contemplated in subsection (1)(b).

(b) Any pipe stormwater drainage system must have capacity to convey stormwater runoff from a 5-year recurrence interval design storm event of any duration and any such system that conveys stormwater on the surface of land must be capable of conveying the runoff from a 25-year recurrence interval design storm event of any duration.

(4) No stormwater drainage facility utilising a closed conveyance structure such as pipes, may discharge directly onto the surface of a public road.

43. Servitudes ………………….

44. Wetlands …………………

45. Regional storm water facilities

(1) If the Agency considers that the public would benefit by the establishment of a regional stormwater facility which would serve as an alternative to the construction of separate on-site stormwater drainage facilities on various properties, it may construct such facility to provide stormwater quantity and quality control for more than one development.

(2) A regional stormwater facility must be located outside the 50-year floodline of any watercourse. ……………..

46. Planning of catchment areas

(1) A policy, adopted by the Council concerning the management of stormwater In any catchment area, must be used by the Agency to develop requirements for a catchment area for the control at source of stormwater, stormwater treatment and erosion control at any water course and requirements relating to any wetland or other water quality sensitive area.

(2) The Agency may for the purposes of subsection (1), on the basis of a policy contemplated in that subsection, by written notice served on a developer or owner of property, require him or her to comply with any requirement stipulated in that notice, in addition to any requirement of these By-laws.

(3) Any requirement of a policy contemplated in subsection (1), may by notice in terms of subsection (2), be made applicable to the owner of a property on which a development was completed prior to date of commencement of these By-laws, if any stormwater facility or other measure to manage stormwater or to prevent pollution at the time of that development, did not comply with Part D – Urban Stormwater Management of the Guidelines for the Provision of Engineering Services in Residential Townships issued by tine national Department of Community Development in 1983, commonly referred to as the Blue Book.

Chapter 6

Operation and maintenance

Chapter 7

Critical drainage areas

53. Additional requirements

(1) In order to mitigate or eliminate any potential stormwater-related impact on any critical drainage area, the Agency may by notice in writing served on a developer or owner of property, require any stormwater drainage facility in excess of those required in terms of these By-laws, to be provided by that developer or owner.

(2) For the purposes of subsection (1), "critical drainage area" means -

(a) any area underlain by, or shown on a 1:50 000 scale geological map to be within 500 m of any dolomitic geology;

(b) land with a slope of 3 m horizontal to 1 m vertical or greater, as determined -

(i) from a topographic survey of the site prepared by a qualified land surveyor;

(ii) from a topographic map maintained by the Council, if other topographic survey information is not available;

(iii) from a contour map generated from a 25 m grid digital elevation data obtainable from the Chief Director: Surveys and Mapping appointed in terms of section 2A of the Land Survey Act, 1997 (Act No 8 of 1997); or

(iv) by an authorised official based on a field investigation of the area concerned;

(c) any geologic area hazardous to life or property, historically documented as an unstable slope in the records of the Council or the Agency;

(d) Land within 50 m of the high water mark of any body of water where fish spawn and which contains a rearing habitat for fish, reflected in the records of the Council or the Agency;

(e) land designated a critical drainage area in a comprehensive stormwater drainage plan adopted by the Agency;

(f) any refuse disposal site or land fill site of the Council;

(g) land which is a wetland for the purpose of any National or Provincial legislation or policy;

(h) land in respect of which requirements for the management of ground water or any aquifer as defined in the National Water Act, 1998, or sole source ground water aquifer exist under the National Environmental Management Act, 1998, or any other law;

(i) land which drains to a natural closed depression;

(j) land used for the protection of wildlife habitat and designated by the Council as a critical drainage area;

(k) land designated by or under any law as a conservation or protected area, a nature reserve or a protected environment; and

(l) land determined by the Agency to have a high potential for stormwater drainage, and stormwater quality, problems, or to be sensitive to the effects of stormwater runoff.

(3)

(a) If, for the purpose of considering the applicability of subsection (2) to any land, a conflict is found to exist between a map and any other available source of information contemplated in that subsection, the decision as to whether or not land is a critical drainage area must be made by the Agency.

(b) For the purposes of paragraph (a), an authorised official may by written notice require a developer or owner of property to furnish him or her with a site inspection survey and any other topographic data specified in that notice.

Chapter 8

Stormwater pollution

54. Prohibition of pollution ……………

55. Maintenance of pollution control device ……………

56. Exemptions ……………

57. Test procedures ……………..

56. Storm water quality: not addressed ……………

Chapter 9

Miscellaneous

59. Experimental best management practices

(1) If no appropriate best management practice which must be utilised for the purpose of complying with any relevant provision of these By-laws is contained in the manual, an experimental best management practice may be prepared by a developer and submitted to the Agency for approval.

(2) An experimental best management practice approved in terms of subsection (1) may be utilised by the developer concerned.

(3) The Agency may, by notice in writing addressed to a developer contemplated in subsection (1), require the operation of an approved experimental best management practice to be monitored by that developer for a period specified in that notice, in order to ascertain the effectiveness of its operation with a view to the future use of such practice.

(4) A developer must during the period specified in a notice in terms of subsection (3) submit a written report to the Agency on the effectiveness of the operation of the experimental best management practice concerned at the intervals specified in that notice.

60. Deviations and exemptions from By-laws ……………..

These By-laws are referred to as the Stormwater Management By-laws.

13. APPENDIX 3: SOIL CHARACTERISTICS

13.1 Hydrological Characteristics of Soils

13.1.1. Infiltration

Three different methods of calculating infiltration are in common use in South Africa and are available for selection in SWMM. It should be noted that the values of the parameters given in the tables that follow are guidelines only and should be calibrated wherever possible. Considerable guidance and detailed discussion can be found in the Stormwater Management Model Reference Manual Volume 1 – Hydrology (Rossman & Huber 2016)

<u>Horton</u>

Horton's method is an empirical equation that describes the decrease of infiltration rate from water on the surface of the soil. The decrease is an exponential decay function from an initial infiltration rate to some equilibrium rate over a period of time. The integrated version of this equation developed by Green (1984; 1986) takes account of conditions where the rainfall intensity is less than the infiltration rate.

$$f = f_c + (f_0 - f_c)e^{-kt} \qquad \text{Eq. 13-1}$$

where: f = infiltration rate

f_c = minimum or equilibrium infiltration rate

f_0 = initial or maximum infiltration rate

k = constant that reflects how rapidly the infiltration rate decays

t = time elapsed since infiltration began

Typical values of the parameters as suggested by Green are given in Table 13-1 and Table 13-2

Table 13-1: Typical Values for the Parameters in Horton's Equation

Soil Type	f_o (dry)	f_c (equilibrium)
	mm/h	mm/h
Sandy Soil	125	15
Loam Soil	50-75	5-10
Clay Soil	5-25	0-5

Table 13-2: Rate of Decay of Infiltration for Different Values of k

k			Percent of decline of infiltration capacity towards equilibrium value f_c after 1 hour
(sec^{-1})	($hour^{-1}$)	(day^{-1})	
0.00056	2.02	48.4	76
0.00083	2.99	71.7	95
0.00111	4.00	95.9	98
0.00139	5.00	120.1	99

Green and Ampt

The Green-Ampt equation (Green & Ampt, 1911) explains the infiltration of water through soil using Darcy's law. Mein and Larsen (1973) adapted the equation for steady rainfall and Chu (1978) showed how the equation could be applied to unsteady rainfall.

The mechanism is simplified because infiltrated water is assumed to move downward through the soil as an abrupt wetting front that separates the wetted and unwetted soils. In reality the wetted front may not be abrupt and the soil above the front may not be fully saturated. But the approach is preferable to that of Horton because the equation represents a realistic physical process and can be adjusted as better information or explanations become available (Richards 1931), or to take account of a driving head of water standing above the soil surface that is used by SWMM in the analysis of LIDs.

The form of the Green-Ampt equation is:

$$f = K_{sat}\left(1+\frac{(\phi-\theta)(d+\psi)}{F}\right) \qquad \text{Eq. 13-2}$$

where f = infiltration rate

K_{sat} = Hydraulic conductivity of the saturated soil

Φ = porosity of the soil

Θ = initial volumetric water content of the soil

d = depth of driving head above the soil surface (usually ignored)

ψ = capillary suction head at the wetting front

F = cumulative infiltration

The calculation is sensitive to the term ($\Phi - \Theta$), i.e. the difference between the porosity of the soil, which is effectively equal to the total moisture capacity of the soil, and the initial moisture content of the soil, so care should be taken in the selection of the value of *Θ*. The value of this parameter is related to the field capacity of the soil, i.e. the moisture content when all available water has drained out under gravity, and the wilting point, which is the point at which moisture is so tightly bound by capillary tension that it is no longer available to plants.

<u>Soil Conservation Service (SCS) Equation</u>

The SCS equation is often incorrectly formulated as an infiltration equation because differentiation of the equation yields an infiltration rate that is proportional to rainfall (Torno, 1992). The procedure is, however, in common use in South Africa and available as a modelling methodology in SWMM. Readers wishing to use this method are referred to the literature, for example, Schmidt and Schulze (1987) or Rossman and Huber (2016).

Table 13-3: Suggested Green-Ampt Parameters

USDA Soil-Texture Class	Hydraulic Conductivity K_1	Wetting Front Suction Head ψ	Porosity	Water Retained at	
				Field Capacity	Wilting Point
	mm/h	mm	m^3/m^3	m^3/m^3	m^3/m^3
Sand	120.40	49.02	0.437	0.062	0.024
Loamy Sand	29.97	60.96	0.437	0.150	0.047
Sandy Loam	10.92	109.22	0.453	0.190	0.085
Loamy Sand	3.30	88.90	0.463	0.232	0.116
Silt Loam	6.60	169.93	0.501	0.284	0.135
Sandy Clay Loam	1.52	219.96	0.398	0.244	0.136
Clay Loam	1.02	210.06	0.464	0.310	0.187
Silty Clay Loam	1.02	270.00	0.471	0.342	0.210
Sandy Clay	0.51	240.03	0.430	0.321	0.211
Silty Clay	0.51	290.07	0.479	0.371	0.251
Clay	0.25	320.04	0.475	0.378	0.265

After Rawls *et al*. (1983), Torno (1993), and Rawls & Saxton (2006)

13.2 Problem Soils

Care should be taken to ensure that stormwater management facilities are not adversely impacted by problem soils. For example, excessive erosion can occur where soils that are very erodible or dispersive are not properly protected, embankments constructed of expansive clay could be vulnerable to internal erosion if the clay dries out and cracks, or storage basins could leak excessively and fail by piping of the subsoil if constructed on collapsing soils. The reader is referred to the considerable body of literature on this topic, for example, Diop *et al*. (2011) and Department of Public Works (2007).

Dispersive and erodible soils are potentially a significant problem on the Halfway House granite geology of the northern part of the Johannesburg Metropolitan Area. Diop *et al*. (2011), note that soils originating from all granites and granodiorites of the Swazian Complex are potentially dispersive.

14. APPENDIX 4: SUMMARY OF DAILY RAINFALL INFILLING TECHNIQUES

The infilling procedure algorithms developed and implemented by Smithers and Schulze (2000b) and the results used by Lynch (2004) were based on one or a combination of the following techniques:

(a) **Inverse Distance Weighting (IDW):** The IDW technique inversely weights the rainfall records from rainfall stations surrounding the rainfall station under consideration, depending on the distance of those rainfall stations from the rainfall station under consideration. A procedure for selecting neighbouring rainfall stations was established from each quadrant around the rainfall station under consideration. This approach ensured that a certain number of rainfall stations are selected from each of the four quadrants surrounding the station to minimise the uncertainty introduced when the closest few rainfall stations are all in the same direction from the rainfall station under consideration.

(b) **Expectation Maximisation Algorithm (EMA):** The EMA technique was adopted and refined by Makhuvha *et al.* (1997) and Pegram (1997) to infill missing rainfall data monthly. The EMA technique revolves around a recursive action of substituting missing data in a multiple linear regression relationship to re-estimate the values between the data at the rainfall station under consideration and the data from the nearby control rainfall stations. Smithers and Schulze (2000b) modified the monthly time step EMA-based procedures developed by Pegram (1997), which selects suitable control stations and performs simultaneous infilling of missing data, to operate on a daily time step to infill missing daily rainfall data.

(c) **Monthly Infilling (MI) technique:** A regression approach was used to infill the non-existing missing monthly rainfall data by using the surrounding control rainfall stations as described by Zucchini (1984; cited by Lynch, 2004). The monthly database (observed and infilled) by Dent (1989; cited by Lynch, 2004) was interrogated and the monthly infilled values of zero and/or ≤ 2 mm were extracted.

(d) **Median Ratio (MR) technique:** The MR technique depends on the median values between the rainfall station under consideration and the nearest control rainfall station to estimate a proportionality ratio. The latter proportionality ratio is used to correct the data from the rainfall station under consideration and to infill the missing daily data series. The advantage of the MR technique is that the closest control rainfall station with non-existing data will be replaced by the second closest control rainfall station (Lynch, 2004).

The EMA and MR techniques are the most effective infilling techniques in the DREU (Lynch, 2004). Any missing observed rainfall values not infilled by using the EMA and MR techniques, were infilled using the IDW technique. Subsequently, zero and less than 2 mm rainfall values, as derived by Dent (1989; cited by Lynch, 2004), are then used to infill any remaining missing values that have not been infilled. The South African daily rainfall database has more than doubled in size with the infilling techniques described above. The rainfall database consists of 105 753 218 daily observed values with 236 154 934 infilled values (Lynch, 2004). The observed and infilled rainfall database therefore contains 341 908 152 values (Lynch, 2004).

15. APPENDIX 5: DATA QUALITY CODES IN DWS FLOW DATA

The data quality codes used by DWS are summarised in Table 15-1

Table 15-1: Data quality codes used in DWS flow data

Quality code numbering	Quality code listing	Description
1		Good, continuous data
2		Good, edited data
3		Preserved historical data
4	Q	Unaudited
5		Height derived from flow
6	D	Drops
7	Q	Good, edited data (unaudited)
8	@	Good weekly reading
25	Q	Unaudited Gauge Plate (GP) readings/dip level readings
26	$	Audited GP readings/dip level readings
27	&	Good monthly reading
50	S	Gap filled data
58	H	Downstream stage (H_b) not available/044 assume no submergence
59	V	Static or reverse flow due to backwater submergence conditions
60	A	Above Rating
64	E	Audited estimate
65	E	Unaudited estimate
66	*	Program estimate
70	?	Unknown
78	U	Not accumulated (unreliable)
79	%	Accumulated (unreliable)
80	+	Accumulated (reliable)
81	#	Wet day within accumulated rainfall period
90	<	Water level below instrument
91	>	Minimum value
92	<	Maximum value
93	<	Dry borehole
94	>	Artesian borehole level
95	<	Borehole seepage
100	?	Flag: Dam under construction. New FSL not yet implemented.
130	E	Used previous week's level as an estimate for this week
140	!	Data not yet checked
150	^	Rating table extrapolated – flows estimated
151	M	Data missing
152	~	Negative
153	F	No height data\044 Flow data only
154	[	Reversal start
155	[	Reversal end
160	Z	No info for stage/discharge determination (zero DT loaded)
161	T	Rating missing

Quality code numbering	Quality code listing	Description
162	R	Rating unreliable
163	G	Gate(s) in operation – no spillway discharge
164	B	Continuously variable submergence flow derivation; DT in operation
165	P	Estuarine water level recording only – no flow calculated
170	M	Period of No Record (PNR)
171	M	Data exists, but unreliable
172	M	Temporary gap
173	?	Data unreliable
201	[	Data not recorded or incomplete
245	V	Undefined submergence flow calc program exception
246	M	No cross-sectional area upstream of notch/structure
247	V	Upstream stage (H_a) > rating table limit – no calculation performed
248	V	No H_b data (submergence)
249	V	No H_a data (upstream stage)
250	V	Structural submergence > 97.7%
251	V	Static or reverse flow possibilities
252	V	Froude number > 0.8 at inlet section
253	V	Flow not converging to constant value after max # iterations
254	A	Rating Table exceeded
255	M	Data missing

16. APPENDIX 6: PROBABILITY

16.1 Appendix 6A: Calculation procedures for probability distributions

Conservation statistics (Missing data excluded): Applicable to the period of record with continuous observations/measurements.

$$\bar{x} = \frac{\sum x}{N} \quad \text{Eq. 16-1}$$

$$\overline{\log x} = \frac{\sum \log(x)}{N} \quad \text{Eq. 16-2}$$

$$s = \left[\frac{\sum\left(x-\bar{x}\right)^2}{N-1}\right]^{0.5} \quad \text{Eq. 16-3}$$

$$s_{log} = \left[\frac{\sum\left(\log(x)-\overline{\log(x)}\right)^2}{N-1}\right]^{0.5} \quad \text{Eq. 16-4}$$

$$c_v = \frac{s}{\bar{\bar{x}}} \quad \text{Eq. 16-5}$$

$$c_v = \frac{s_{\log}}{\overline{\log(x)}} \quad \text{Eq. 16-6}$$

$$g = \left(\frac{N}{(N-1)(N-2)}\right)\left(\frac{\sum\left(x-\bar{x}\right)^3}{s^3}\right) \quad \text{Eq. 16-7}$$

$$g_{log} = \left(\frac{N}{(N-1)(N-2)}\right)\left(\frac{\sum\left(\log(x)-\overline{\log(x)}\right)^3}{s_{\log}{}^3}\right) \quad \text{Eq. 16-8}$$

where:

c_v = coefficient of variation,

c_{vlog} = coefficient of variation of the observed value logarithms,

g = skewness coefficient,

g_{log} = skewness coefficient of the observed value logarithms,

N = total number of observations (sample size),

s = standard deviation of observed values (m^3/s or mm),

s_{log} = standard deviation of the observed value logarithms (m^3/s or mm),

x = observed values (m^3/s or mm),

$\bar{x}$ = mean of observed values (m^3/s or mm), and

$\overline{\log x}$ = mean of observed value logarithms (m^3/s or mm).

Conservation statistics (Missing data included): Historically weighted variables are used to incorporate the influence of historical observations prior to period of continuous observations/measurements. Missing data and low/high outliers are also incorporated.

$$\overline{x_h} = \frac{\left((W_T)\sum x_b + \sum x_a\right)}{\left(Y_T - (W_T)(L_W)\right)} \quad \text{Eq. 16-9}$$

$$s_h = \left[\frac{\left((W_T)\sum {d_b}^2 + \sum {d_a}^2\right)}{\left(Y_T - (W_T)(L_W) - 1\right)}\right]^{0.5} \quad \text{Eq. 16-10}$$

$$g_h = \left[\frac{\dfrac{\left(Y_T - (W_T)(L_W)\right)\left((W_T)\sum {d_b}^3 + \sum {d_a}^3\right)}{s^3}}{\left(Y_T - (W_T)(L_W) - 1\right)\left(Y_T - (W_T)(L_W) - 2\right)}\right] \quad \text{Eq. 16-11}$$

where: d_a, d_b = deviations of x_a+ x_b from $\overline{x_h}$,

g_h = historically weighted skewness coefficient,

L_W = low outliers including zero flows,

N_A = floods equal to or above high threshold,

N_B = floods between high and low thresholds,

N_C = missing data,

s_h = historically weighted standard deviation (m^3/s),

W_T = weight applied to data, $\dfrac{(Y_T - N_A)}{N_B}$

x_a = peak flows equal to or above the high threshold (m^3/s),

x_b = peak flows below the high threshold (m^3/s),

$\overline{x_h}$ = historically weighted mean (m^3/s), and

Y_T = total time span, N_A+N_B+N_C (years).

<u>Theoretical probability distributions:</u> The design flood values (peak flows at *T*-year) is depicted by Q_T. Similarly, the Q_T values (m^3/s) can be replaced with design rainfall (P_T) values (mm) when design rainfall estimation is considered.

Normal (N/MM) distribution:

$$Q_T = \bar{x} + sy \quad \text{Eq. 16-12}$$

Extreme Value Type I (EV1/MM) distribution:

$$Q_T = \overline{x} + s(0.781\,W_T - 0.451) \quad \text{Eq. 16-13}$$

Extreme Value Type II (EV2/MM) distribution:

$$Q_T = \overline{x} + \sqrt{\frac{s^2}{\mathrm{var}(y)}}\left(1 - E(y) - kW_T\right) \quad \text{Eq. 16-14}$$

Extreme Value Type III (EV3/MM) distribution:

$$Q_T = \overline{x} + \sqrt{\frac{s^2}{\mathrm{var}(y)}}\left(-1 + E(y) + kW_T\right) \quad \text{Eq. 16-15}$$

Log-Normal (LN/MM) distribution:

$$Q_T = anti\log\left[\overline{\log(x)} + s_{\log} W_T\right] \qquad \text{Eq. 16-16}$$

Log-Extreme Value Type I (LEV1/MM) distribution:

$$Q_T = anti\log\left[\overline{\log(x)} + s_{\log}\left(0.781\, W_T - 0.451\right)\right] \qquad \text{Eq. 16-17}$$

Log-Pearson Type III (LP3/MM) distribution:

$$Q_T = anti\log\left[\overline{\log(x)} + s_{\log} W_T\right] \qquad \text{Eq. 16-18}$$

Generalised Logistic (GLO/LM) distribution:

$$Q_T = Q_{Med}\left[1 + \frac{\beta}{k}\left(1 - (T-1)^{-k}\right)\right] \qquad \text{Eq. 16-19}$$

with:

$$\beta = \frac{t_2 k \sin(\pi k)}{k\pi(k + t_2) - t_2 \sin(\pi k)}$$

$$k = -t_3$$

$$t_2 = \frac{2\left[\frac{1}{N}\sum_{m=2}^{N}\frac{(m-1)}{(N-1)}Q_m\right] - \bar{x}}{\bar{x}}$$

$$t_3 = \frac{6\left[\frac{1}{N}\sum_{m=3}^{N}\frac{(m-1)(m-2)}{(N-1)(N-2)}Q_m\right] - 6\left[\frac{1}{N}\sum_{m=2}^{N}\frac{(m-1)}{(N-1)}Q_m\right] + \bar{x}}{2\left[\frac{1}{N}\sum_{m=2}^{N}\frac{(m-1)}{(N-1)}Q_m\right] - \bar{x}}$$

where:

- *E (y)* = mean of the standardised variate (Table 16-1),
- *k* = shape parameter (Table 16-1),
- *m* = number, in ascending order, of the ranked peak events,
- *N* = number of observations/record length [years],
- Q_m = ranked annual maximum flood peak (m^3/s),
- Q_{Med} = median annual maximum flood peak (m^3/s),
- Q_T = peak flow for *T*-year return period (m^3/s),
- *s* = standard deviation of observed values (m^3/s),
- s_{log} = standard deviation of the observed value logarithms (m^3/s),
- *T* = return period (years),
- *var (y)* = variance of the standardised variate (Table 16-1) ,
- W_T = frequency factor for *T*-year return period or LN standard variate (Table 16-1 and Table 16-2),
- $\bar{x}$ = mean of observed values (m^3/s), and
- *y* = standardised variate.

Table 16-1: General Extreme Value distribution parameters

Distribution	Skew (g)	*T*-year return period (years)												k	$E(y)$	$var(y)$
		2	5	10	20	50	100	200	500	1 000	2 000	5 000	10 000			
		Standardised variate W_T														
GEV3	-2.0	0.3062	0.7713	0.886	0.939	0.969	0.978	0.983	0.986	0.987	0.987	0.987	0.988	1.0000	1.0000	1.0000
	-1.8	0.3102	0.8057	0.938	1.002	1.040	1.053	1.060	1.065	1.066	1.067	1.067	1.068	0.9319	0.9731	0.8235
	-1.6	0.3142	0.8412	0.993	1.070	1.119	1.137	1.147	1.153	1.156	1.157	1.158	1.158	0.8617	0.9492	0.6729
	-1.4	0.3183	0.8791	1.052	1.145	1.209	1.233	1.248	1.258	1.261	1.264	1.265	1.266	0.7892	0.9286	0.5446
	-1.2	0.3224	0.9201	1.119	1.232	1.313	1.347	1.367	1.382	1.389	1.393	1.396	1.397	0.7149	0.9115	0.4360
	-1.0	0.3267	0.9646	1.193	1.330	1.435	1.481	1.511	1.534	1.545	1.552	1.557	1.560	0.6394	0.8986	0.3447
	-0.8	0.3311	1.012	1.275	1.442	1.577	1.641	1.685	1.721	1.738	1.750	1.760	1.764	0.5635	0.8899	0.2686
	-0.6	0.3356	1.063	1.365	1.568	1.743	1.831	1.893	1.949	1.977	1.998	2.016	2.025	0.4883	0.8859	0.2055
	-0.4	0.3400	1.117	1.463	1.707	1.933	2.053	2.143	2.227	2.273	2.307	2.340	2.358	0.4149	0.8866	0.1536
	-0.2	0.3443	1.172	1.566	1.860	2.147	2.309	2.436	2.563	2.635	2.693	2.750	2.783	0.3443	0.8917	0.1113
	0.0	0.3485	1.227	1.674	2.023	2.383	2.598	2.774	2.960	3.073	3.166	3.264	3.323	0.2777	0.9011	0.0772
	0.2	0.3524	1.281	1.783	2.193	2.637	2.917	3.156	3.422	3.590	3.735	3.896	3.999	0.2158	0.9141	0.0504
	0.4	0.3560	1.334	1.891	2.366	2.905	3.260	3.576	3.943	4.187	4.405	4.658	4.827	0.1594	0.9300	0.0299
	0.6	0.3593	1.384	1.996	2.538	3.179	3.619	4.025	4.515	4.855	5.170	5.551	5.816	0.1088	0.9479	0.0152
	0.8	0.3622	1.430	2.096	2.705	3.453	3.985	4.492	5.127	5.583	6.019	6.566	6.959	0.0640	0.9669	0.00579
	1.00	0.3649	1.473	2.189	2.864	3.721	4.350	4.966	5.763	6.353	6.934	7.686	8.243	0.0246	0.9864	0.000936
	1.10	0.3661	1.492	2.233	2.941	3.851	4.530	5.203	6.086	6.749	7.410	8.278	8.931	0.0067	0.9962	0.0000729
GEV1	1.136	0.3665	1.499	2.249	2.968	3.897	4.594	5.287	6.202	6.893	7.583	8.496	9.185	0.0006	0.99965	0.0000006
	1.1396	0.3665	1.500	2.250	2.970	3.902	4.600	5.296	6.214	6.907	7.601	8.517	9.210	0.0000	1.00000	0.0000000
	1.144	0.3666	1.501	2.252	2.973	3.908	4.608	5.306	6.228	6.925	7.622	8.544	9.242	-0.0007	1.00043	0.0000009
GEV2	1.15	0.3666	1.502	2.255	2.978	3.915	4.619	5.320	6.247	6.949	7.651	8.581	9.285	-0.0017	1.0010	0.0000050
	1.18	0.3670	1.507	2.267	3.000	3.953	4.672	5.391	6.345	7.069	7.797	8.765	9.500	-0.0067	1.0039	0.0000747
	1.28	0.3680	1.525	2.308	3.071	4.078	4.846	5.623	6.668	7.472	8.288	9.386	10.23	-0.0224	1.0135	0.000878
	1.4	0.3692	1.546	2.355	3.154	4.222	5.050	5.898	7.053	7.954	8.881	10.15	11.13	-0.0399	1.0247	0.00293
	1.6	0.3710	1.577	2.426	3.281	4.451	5.375	6.340	7.682	8.752	9.872	11.43	12.68	-0.0660	1.0427	0.00866
	1.8	0.3725	1.604	2.491	3.398	4.663	5.681	6.760	8.290	9.532	10.85	12.73	14.25	-0.0887	1.0597	0.0169
	2.0	0.3739	1.629	2.549	3.505	4.858	5.966	7.157	8.872	10.29	11.81	14.01	15.82	-0.1086	1.0756	0.0270
	2.5	0.3766	1.679	2.670	3.730	5.281	6.591	8.039	10.19	12.02	14.05	17.08	19.65	-0.1480	1.1106	0.0584
	3.0	0.3787	1.718	2.764	3.907	5.620	7.103	8.773	11.32	13.53	16.04	19.85	23.18	-0.1769	1.1392	0.0941
	3.5	0.3802	1.747	2.838	4.048	5.894	7.520	9.381	12.26	14.82	17.75	22.30	26.34	-0.1986	1.1627	0.1311
	4.0	0.3814	1.771	2.896	4.161	6.118	7.864	9.887	13.06	15.92	19.23	24.44	29.13	-0.2155	1.1821	0.1675
	4.5	0.3823	1.789	2.943	4.253	6.302	8.150	10.31	13.74	16.85	20.50	26.31	31.58	-0.2288	1.1983	0.2022
	5.0	0.3831	1.805	2.982	4.329	6.454	8.389	10.67	14.31	17.66	21.60	27.92	33.72	-0.2395	1.2117	0.2345
	5.5	0.3837	1.817	3.014	4.392	6.583	8.591	10.97	14.81	18.35	22.55	29.34	35.60	-0.2483	1.2232	0.2647
	6.0	0.3842	1.828	3.042	4.447	6.694	8.767	11.23	15.24	18.95	23.39	30.59	37.28	-0.2564	1.2340	0.2956
	6.5	0.3847	1.838	3.066	4.495	6.795	8.926	11.47	15.63	19.51	24.16	31.76	38.85	-0.2656	1.2468	0.3349

Table 16-2: Normal and Pearson III probability distributions (Standardised variate W_T)

Skew (g)	*T*-year return period [years]											
	2	**5**	**10**	**20**	**50**	**100**	**200**	**500**	**1 000**	**2 000**	**5 000**	**10 000**
	Standardised variate W_T											
-1.4	0.225	0.832	1.041	1.168	1.270	1.319	1.352	1.380	1.394	1.404	1.412	1.416
-1.2	0.195	0.844	1.086	1.243	1.380	1.450	1.502	1.551	1.578	1.598	1.618	1.628
-1.0	0.164	0.852	1.128	1.317	1.492	1.589	1.664	1.741	1.787	1.824	1.862	1.885
-0.8	0.132	0.856	1.166	1.389	1.606	1.733	1.837	1.948	2.018	2.077	2.143	2.186
-0.7	0.116	0.857	1.183	1.423	1.663	1.806	1.926	2.057	2.140	2.213	2.296	2.351
-0.6	0.099	0.857	1.200	1.458	1.720	1.880	2.016	2.168	2.267	2.355	2.457	2.525
-0.5	0.083	0.857	1.216	1.491	1.777	1.954	2.108	2.282	2.398	2.502	2.625	2.708
-0.4	0.067	0.855	1.231	1.524	1.833	2.029	2.200	2.399	2.532	2.653	2.798	2.899
-0.3	0.050	0.853	1.245	1.555	1.889	2.104	2.294	2.517	2.668	2.808	2.977	3.096
-0.2	0.033	0.850	1.258	1.586	1.945	2.178	2.388	2.636	2.807	2.966	3.161	3.299
-0.1	0.017	0.846	1.270	1.616	2.000	2.252	2.482	2.757	2.948	3.127	3.349	3.507
0.0	0.000	0.842	1.282	1.645	2.054	2.326	2.576	2.878	3.090	3.291	3.540	3.719
0.1	-0.017	0.836	1.292	1.673	2.107	2.400	2.670	3.000	3.234	3.456	3.734	3.935
0.2	-0.033	0.830	1.301	1.700	2.159	2.472	2.763	3.122	3.378	3.622	3.930	4.154
0.3	-0.050	0.824	1.309	1.726	2.211	2.544	2.857	3.244	3.522	3.789	4.128	4.375
0.4	-0.067	0.816	1.317	1.750	2.261	2.616	2.949	3.366	3.667	3.957	4.327	4.598
0.5	-0.083	0.808	1.323	1.774	2.311	2.686	3.041	3.488	3.812	4.125	4.527	4.822
0.6	-0.099	0.799	1.328	1.797	2.359	2.755	3.132	3.609	3.956	4.294	4.728	5.048
0.7	-0.116	0.790	1.333	1.819	2.407	2.824	3.223	3.730	4.100	4.462	4.929	5.274
0.8	-0.132	0.780	1.336	1.839	2.453	2.891	3.312	3.850	4.244	4.631	5.130	5.501
1.0	-0.164	0.758	1.340	1.877	2.542	3.022	3.488	4.087	4.530	4.966	5.533	5.955
1.2	-0.195	0.733	1.341	1.910	2.626	3.149	3.660	4.322	4.814	5.300	5.935	6.410
1.4	-0.225	0.705	1.337	1.938	2.706	3.271	3.828	4.553	5.095	5.632	6.336	6.864

16.2 Appendix 6B: Worked examples

Example 1: A rainfall/flood event has a recurrence interval of 1: 20 years. Determine the probability for the following cases:

(a) Event occurs at least once in a 3-year period:

$$P_N = 1-\left(1-\frac{1}{T}\right)^N$$

$= \underline{14.26\%}$

(b) Event occurs three times in a 3-year period:

$$P_N = \left(\frac{1}{T}\right)^N$$

$= \underline{0.013\%}$

Example 2: A rainfall/flood event has a recurrence interval of 1: 50-year. Determine the probability if the flood occurs or being exceeded in the next 10 years.

$P_N \quad = 1-\left(1-\frac{1}{T}\right)^N$

= <u>18.293%</u>

Example 3: A rainfall/flood has a recurrence interval of 1:10-year. Determine the probability if the flood occurs or being exceeded in the next 50 years.

$P_N \quad = 1-\left(1-\frac{1}{T}\right)^N$

= <u>99.485%</u>

Example 4: The overflow crest of a coffer dam is to be secured against the 1: 50-year flood. If the probability of overtopping is to be limited to 10%, determine the following:

(a) How long the construction of the dam should take to statistically prevent flooding of the coffer dam:

$$N = \frac{\log[1-P(x > or = x)N]}{\log\left[\frac{(T-1)}{T}\right]}$$

$$N = \frac{\log[1-0.1]}{\log\left[\frac{(50-1)}{50}\right]}$$

= <u>5.215 years</u>

(b) How long the construction of the dam should may take if the design capacity of its temporary river diversion works can handle a 1: 5-year flood, with a 20% chance of being overtopped?

$$N = \frac{\log[1-P(x > or = x)N]}{\log\left[\frac{(T-1)}{T}\right]}$$

= <u>1 year</u>

(c) How long may the construction of the dam should take if the 1: 20-year flood will completely destroy all construction work if it has not yet been completed? The estimations should be based on a 95% degree of certainty:

$$N = \frac{\log\left[1 - P(x > or = x)N\right]}{\log\left[\frac{(T-1)}{T}\right]}$$

= <u>1 year</u>

Example 5: A FFA (2 to 200-year return periods) at flow-gauging weir U2H011 (Msunduzi River at Henley Dam) need to be conducted by using the most appropriate theoretical probability distribution(s). The structural limit of the weir is 513 m^3/s and the catchment area covers 176 km².
The AMS of U2H011 is listed in Table 16-3.

Table 16-3: AMS at U2H011 (Msunduzi River at Henley Dam)

Hydrological year	AMS (m^3/s)	Hydrological year	AMS (m^3/s)
1957	-1.0	1987	465.9
1958	19.4	1988	35.0
1959	19.7	1989	56.9
1960	29.0	1990	178.9
1961	42.4	1991	114.7
1962	68.9	1992	35.0
1963	133.9	1993	19.7
1964	35.1	1994	74.2
1965	29.9	1995	10.9
1966	20.7	1996	97.3
1967	245.9	1997	170.8
1968	22.5	1998	57.3
1969	53.8	1999	134.2
1970	35.1	2000	32.9
1971	73.0	2001	39.1
1972	82.3	2002	29.8
1973	209.7	2003	10.2
1974	150.7	2004	12.2
1975	344.7	2005	149.7
1976	229.4	2006	86.2
1977	45.9	2007	49.0
1978	99.5	2008	104.2
1979	73.0	2009	46.1
1980	10.5	2010	50.8
1981	17.8	2011	24.7
1982	-1.0	2012	29.1
1983	5.8	2013	28.6
1984	33.1	2014	20.5
1985	66.2	2015	21.4
1986	28.8	2016	9.0

Note that “-1” represents missing data

Solution:

Table 16-4: Statistical properties at U2H011

Normal data				Log$_{10}$-transformed data			
$\bar{x}$	***s***	***g***	***c$_v$***	$\bar{x}$	***s***	***g***	***c$_v$***
76.22	85.81	2.53	1.13	1.68	0.42	0.19	0.25

Table 16-5: Theoretical probability distribution results at U2H011

Return period	Probability distributions (m^3/s)			
	LN/MM	**GEV/MM**	**LP3/MM**	**GLO/LM**
2	48	57	**46**	44
5	108	125	**107**	91
10	165	177	**168**	137
20	234	233	**246**	197
50	347	315	**383**	311
100	451	384	**517**	434
200	574	460	**683**	603

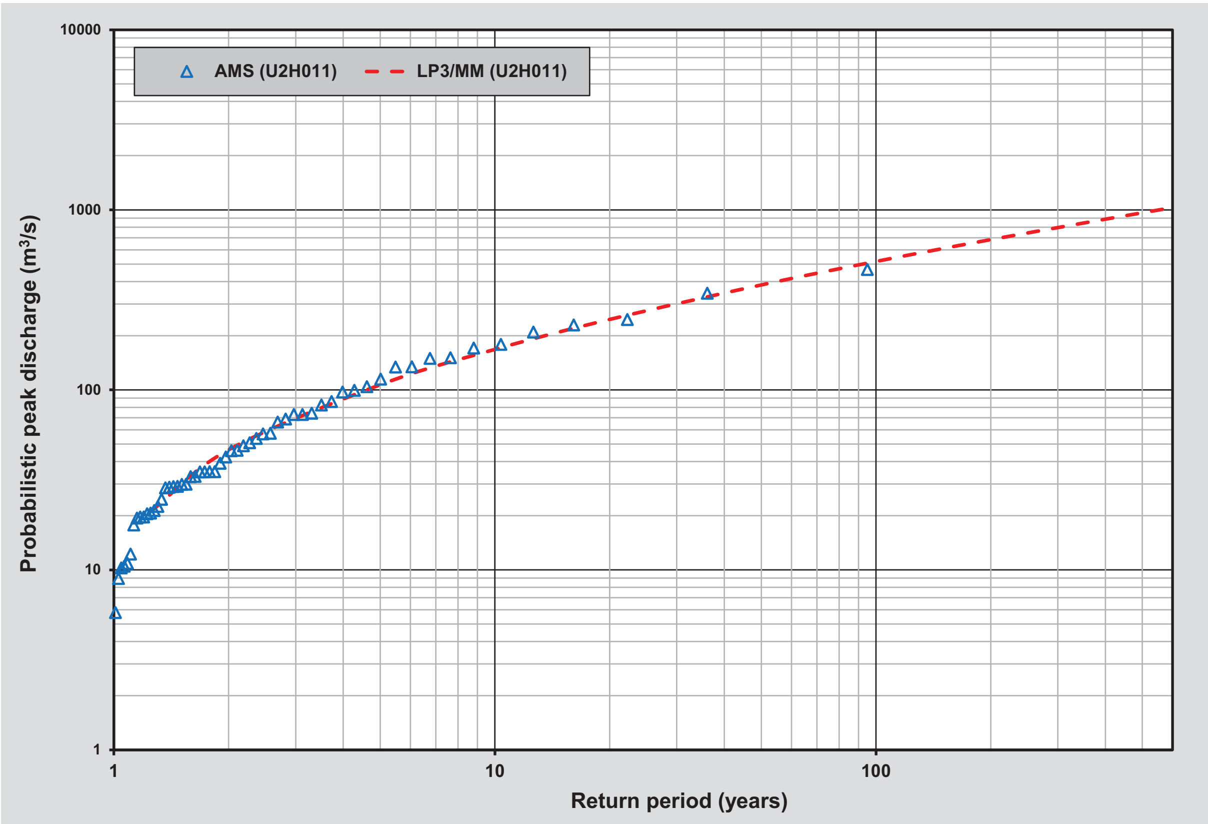


Figure 16-1:Probabilistic plot based on the ranked AMS and Weibull plotting position at U2H011

16.3 Appendix 6C: Z-Set Plotting Position

Plotting position formulae (Table 16-6) are commonly used in South Africa to assign an exceedance probability to flood peaks. It assumes that if (*n*) values are distributed uniformly between 0% and 100% probability, then there must be (*n* + 1) intervals, (*n* - 1) intervals between the data points and two intervals at the ends (Chow *et al*., 1988; SANRAL, 2013). The Cunnane plotting position is generally used in South Africa, while it is also being recommended by DWS (Van der Spuy and Rademeyer, 2021).

$$T = \frac{n+a}{m-b} \qquad \text{Eq.16.20}$$

where: T = return period (years),

a = constant (Table 16-6),

b = constant (Table 16-6),

m = number, in descending order, of the ranked events (peak flows), and

n = number of observations/record length (years).

Table 16-6: Common plotting position formulae (SANRAL, 2013)

Method	Plotting position	Theoretical probability distribution
Beard (1962)	a = 0.40 and b = 0.30	Pearson Type 3
Blom (1958)	a = 0.25 and b = 0.375	Normal
Cunnane (1978)	a = 0.20 and b = 0.40	General purpose
Greenwood (1979)	a = 0.00 and b = 0.35	GEV and Wakeby
Gringorten (1963)	a = 0.12 and b = 0.44	Extreme Value Type 1, GEV and Exponential
Weibull (1939)	a = 1.00 and b = 0.00	Normal and Pearson Type 3

The PP technique is summarised as follows:

- Arrange the given data-series – e.g. the Annual Maximum Series (AMS) of flood peaks – in descending order.
- Assign an order number to each of the data (termed as ranking (m) of the data), starting at the highest flood peak with m = 1 to m = n for the lowest flood peak.

  This ranking order is preferred, since it relates directly to an Annual Exceedance Probability (AEP), which directly relates to risk. If the flood peak data are sorted in ascending order (noted in some references) the probability value, assigned to a flood peak data point, indicates probability of non-exceedance.

- Apply Equation 1 to assign a return period T to every flood peak. T indicating the return period, being the invers of the probability that the corresponding flood peak, $\boldsymbol{Q_m}$, will be exceeded.

Van der Spuy and Du Plessis (2022a) highlight a number of key shortfalls of all the existing plotting position in use in South Africa. The two most obvious problems are:

- The 'ranked' PP (i.e. probability) assigned to an outlier, is most probably always incorrect and can influence the analyst to choose the least appropriate theoretical probability distribution.
- Due to the ranking process, values in the dataset having the same, or very similar, magnitudes will be assigned different PPs (probabilities). This may distort the visual appearance of the PP to such an extent that it may complicate the choice of the most applicable probability.

Van der Spuy and Du Plessis (2022b) subsequently developed a new plotting position which aimed to address these shortfalls. The new plotting position is named the Z-set plotting position and uses the following Z-scores to determine its PP:

$Z_{m.Weibull}$ - Z-score of the Weibull PP

To determine the Z-score of the Weibull PP:

- Calculate the Weibull PP (as an exceedance probability)
- Convert the exceedance probability to a Z-score value using one of the following methods:
  - The Z-score charts of the Normal distribution – see Table 16-8.
  - Table 3A.1a in RDM, where *G(y)* = 100 – Weibull PP as a % and *y* = the z-score of the Weibull PP.
  - In Excel: $\boldsymbol{Z_{m.Weibull}}$ = NORM.S.INV(P_m)

$$Z_{Q_m} = \frac{Q_m - \overline{Q}}{S_Q}$$ - Z-score of the AMS flood peak data Eq.16.21

$$Z_{logQ_m} = \frac{\log Q_m - \overline{logQ}}{S_{logQ}}$$ - Z-score of the log-transformed AMS flood peak data Eq.16.22

Where *m* indicates the *m*th order statistic, in the ranked dataset, Q_m, $\overline{Q}$, and S_Q the ranked flood peak value, the average of the flood peak values and the standard deviation of the flood peak values respectively and $\log Q_m$, $\overline{logQ}$, and S_Q the log of the flood peak value, the average of the log flood peak values and the standard deviation of the log flood peak values respectively.

The ranked Z-score is calculated using Equation 16.23.

$$Z_{m.Z\text{-}set} = 0.0902 * Z_{m.Weibull} + 0.1564 * Z_{Q_m} + 0.8083 * Z_{logQ_m} \quad \text{Eq.16.23}$$

To determine the probability of exceedance ($P_{m.Z\text{-}set}$) of the associated Q_m value, which represents the plotting position of the $Z_{m.Z\text{-}set}$ value, use one of the following methods:

- Use Normal distribution Z-score charts – see Table 16-9.
- Table 3A.1a in RDM, where y = $\boldsymbol{Z_{m.Z\text{-}set}}$ and $\boldsymbol{P_{m.Z\text{-}set}}$ = 100 – G(y)%.
- In Excel: $\boldsymbol{P_{m.Z\text{-}set} = 1 - NORM.S.DIST(Z_{m.Z\text{-}set}, TRUE)}$ Eq.16.24

Table 16-7: Woodstock Dam (V1R003), as an illustration to calculate the Z-set PP

Statistics		$Q_{ave} = 498\ m^3/s$		$S = 436\ m^3/s$		$logQ_{ave} = 2.5605$		$S_{log} = 0.3542$	
rank	**AMS flood peak data**		**Weibull PP**		**AMS flood peaks**		**Z-set PP**		
$\boldsymbol{m}$	**Q**	**logQ**	$\boldsymbol{P_{Weibull}}$ (1/Table 16.20)	$\boldsymbol{Z_{Weibull}}$	$\boldsymbol{Z_Q}$ (Eq. 16.21)	$\boldsymbol{Z_{logQ}}$ (Eq. 16.22)	$\boldsymbol{Z_{Z\text{-}set}}$ (Eq. 16.23)	$\boldsymbol{P_{Z\text{-}set}}$ (Eq. 16.24)	
1	2915	3.4646	0.01333	2.21636	5.54256	2.55231	3.13002	0.0009	
2	1400	3.1461	0.02667	1.93221	2.06825	1.65318	1.83422	0.0333	
3	1380	3.1399	0.04000	1.75069	2.02239	1.63554	1.79640	0.0362	
4	1275	3.1055	0.05333	1.61336	1.78159	1.53852	1.66792	0.0477	
5	1020	3.0086	0.06667	1.50109	1.19681	1.26496	1.34519	0.0893	
...	...	...	...	...	...	...	...	...	
70	100	2.0000	0.93333	-1.50109	-0.91300	-1.58223	-1.55725	0.9403	
71	91	1.9590	0.94667	-1.61336	-0.93364	-1.69785	-1.66408	0.9520	
72	79	1.8976	0.96000	-1.75069	-0.96116	-1.87121	-1.82092	0.9657	
73	76	1.8808	0.97333	-1.93221	-0.96804	-1.91868	-1.87675	0.9697	
74	72	1.8573	0.98667	-2.21636	-0.97721	-1.98496	-1.95742	0.9749	

The effect of the new Z-set PP is illustrated in Figure 16-2

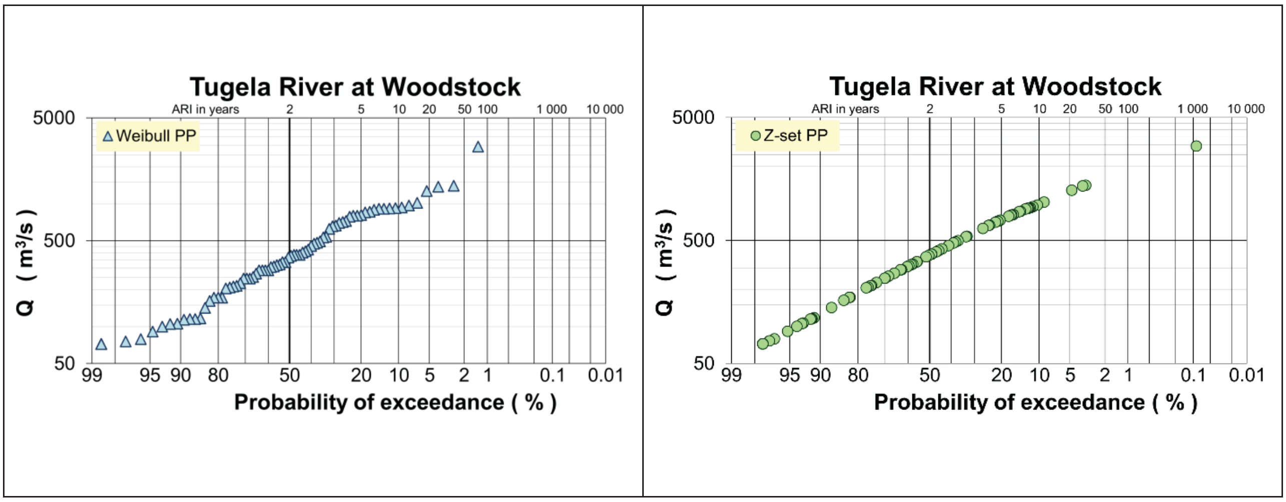


Figure 16-2: Compare Z-set PP with existing Weibull PP

Table 16-8: Z-Score from exceedance probability (EP)

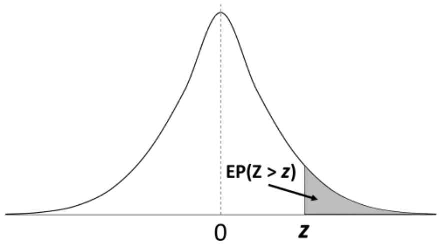
Standard Normal Distribution: z-score from exceedance probability (EP)

The table provides the standardised normal z-score of its EP (%)

The horizontal heading indicates increments of an extra digit (e.g. for row 0.007, column 6 indicates EP = 0.0076%, and so forth)

For EP > 50%: z(EP) = -z(100 – EP)

e.g.: z(60) = -z(100-60) = -z(40) = -0.25335



EP (%)	0	1	2	3	4	5	6	7	8	9
0.001	4.26489	4.24356	4.22400	4.20594	4.18915	4.17347	4.15875	4.14487	4.13176	4.11932
0.002	4.10748	4.09619	4.08541	4.07507	4.06516	4.05563	4.04645	4.03760	4.02906	4.02080
0.003	4.01281	4.00507	3.99756	3.99026	3.98318	3.97629	3.96958	3.96304	3.95668	3.95046
0.004	3.94440	3.93848	3.93269	3.92703	3.92150	3.91608	3.91078	3.90558	3.90049	3.89549
0.005	3.89059	3.88578	3.88107	3.87643	3.87188	3.86740	3.86301	3.85868	3.85443	3.85024
0.006	3.84613	3.84207	3.83808	3.83415	3.83028	3.82646	3.82270	3.81899	3.81533	3.81173
0.007	3.80817	3.80466	3.80119	3.79778	3.79440	3.79107	3.78778	3.78453	3.78132	3.77815
0.008	3.77501	3.77191	3.76885	3.76583	3.76283	3.75987	3.75695	3.75405	3.75119	3.74835
0.009	3.74555	3.74277	3.74003	3.73731	3.73462	3.73195	3.72932	3.72670	3.72412	3.72155
0.01	3.71902	3.69487	3.67270	3.65220	3.63313	3.61530	3.59855	3.58275	3.56779	3.55360
0.02	3.54008	3.52719	3.51485	3.50303	3.49168	3.48076	3.47024	3.46009	3.45028	3.44080
0.03	3.43161	3.42271	3.41407	3.40568	3.39752	3.38958	3.38185	3.37431	3.36697	3.35980
0.04	3.35279	3.34595	3.33927	3.33272	3.32632	3.32005	3.31391	3.30790	3.30199	3.29621
0.05	3.29053	3.28495	3.27948	3.27410	3.26881	3.26362	3.25851	3.25348	3.24854	3.24367
0.06	3.23888	3.23416	3.22952	3.22494	3.22043	3.21598	3.21160	3.20727	3.20301	3.19880
0.07	3.19465	3.19055	3.18651	3.18252	3.17858	3.17468	3.17084	3.16704	3.16328	3.15957
0.08	3.15591	3.15228	3.14870	3.14515	3.14165	3.13818	3.13475	3.13136	3.12800	3.12468
0.09	3.12139	3.11813	3.11491	3.11172	3.10856	3.10543	3.10234	3.09927	3.09623	3.09322
0.1	3.09023	3.06181	3.03567	3.01145	2.98888	2.96774	2.94784	2.92905	2.91124	2.89430
0.2	2.87816	2.86274	2.84796	2.83379	2.82016	2.80703	2.79438	2.78215	2.77033	2.75888
0.3	2.74778	2.73701	2.72655	2.71638	2.70648	2.69684	2.68745	2.67829	2.66934	2.66061
0.4	2.65207	2.64372	2.63555	2.62756	2.61973	2.61205	2.60453	2.59715	2.58991	2.58281
0.5	2.57583	2.56897	2.56224	2.55562	2.54910	2.54270	2.53640	2.53019	2.52408	2.51807
0.6	2.51214	2.50631	2.50055	2.49488	2.48929	2.48377	2.47833	2.47296	2.46766	2.46243
0.7	2.45726	2.45216	2.44713	2.44215	2.43724	2.43238	2.42758	2.42283	2.41814	2.41350
0.8	2.40892	2.40438	2.39989	2.39545	2.39106	2.38671	2.38240	2.37814	2.37393	2.36975
0.9	2.36562	2.36152	2.35747	2.35345	2.34947	2.34553	2.34162	2.33775	2.33392	2.33012
1	2.32635	2.29037	2.25713	2.22621	2.19729	2.17009	2.14441	2.12007	2.09693	2.07485
2	2.05375	2.03352	2.01409	1.99539	1.97737	1.95996	1.94313	1.92684	1.91104	1.89570
3	1.88079	1.86630	1.85218	1.83842	1.82501	1.81191	1.79912	1.78661	1.77438	1.76241
4	1.75069	1.73920	1.72793	1.71689	1.70604	1.69540	1.68494	1.67466	1.66456	1.65463
5	1.64485	1.63523	1.62576	1.61644	1.60725	1.59819	1.58927	1.58047	1.57179	1.56322
6	1.55477	1.54643	1.53820	1.53007	1.52204	1.51410	1.50626	1.49851	1.49085	1.48328
7	1.47579	1.46838	1.46106	1.45381	1.44663	1.43953	1.43250	1.42554	1.41865	1.41183
8	1.40507	1.39838	1.39174	1.38517	1.37866	1.37220	1.36581	1.35946	1.35317	1.34694
9	1.34076	1.33462	1.32854	1.32251	1.31652	1.31058	1.30469	1.29884	1.29303	1.28727
10	1.28155	1.22653	1.17499	1.12639	1.08032	1.03643	0.99446	0.95417	0.91537	0.87790
20	0.84162	0.80642	0.77219	0.73885	0.70630	0.67449	0.64335	0.61281	0.58284	0.55338
30	0.52440	0.49585	0.46770	0.43991	0.41246	0.38532	0.35846	0.33185	0.30548	0.27932
40	0.25335	0.22754	0.20189	0.17637	0.15097	0.12566	0.10043	0.07527	0.05015	0.02507
50	0.00000	-0.02507	-0.05015	-0.07527	-0.10043	-0.12566	-0.15097	-0.17637	-0.20189	-0.22754

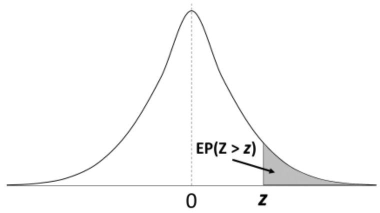
Table 16-9: Exceedance Probability from Z-Score

Standard Normal Distribution: Exceedance probability (EP) from z-score

The table provides the EP (%) of the standardised normal *z*-score.

For z-scores < 0: EP(-z) = 1 – EP(z)

e.g.: EP(-1) = 1 – EP(1) = 1 – 15.866% = 84.134%



Z	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	50.000	49.601	49.202	48.803	48.405	48.006	47.608	47.210	46.812	46.414
0.1	46.017	45.620	45.224	44.828	44.433	44.038	43.644	43.251	42.858	42.465
0.2	42.074	41.683	41.294	40.905	40.517	40.129	39.743	39.358	38.974	38.591
0.3	38.209	37.828	37.448	37.070	36.693	36.317	35.942	35.569	35.197	34.827
0.4	34.458	34.090	33.724	33.360	32.997	32.636	32.276	31.918	31.561	31.207
0.5	30.854	30.503	30.153	29.806	29.460	29.116	28.774	28.434	28.096	27.760
0.6	27.425	27.093	26.763	26.435	26.109	25.785	25.463	25.143	24.825	24.510
0.7	24.196	23.885	23.576	23.270	22.965	22.663	22.363	22.065	21.770	21.476
0.8	21.186	20.897	20.611	20.327	20.045	19.766	19.489	19.215	18.943	18.673
0.9	18.406	18.141	17.879	17.619	17.361	17.106	16.853	16.602	16.354	16.109
1.0	15.866	15.625	15.386	15.151	14.917	14.686	14.457	14.231	14.007	13.786
1.1	13.567	13.350	13.136	12.924	12.714	12.507	12.302	12.100	11.900	11.702
1.2	11.507	11.314	11.123	10.935	10.749	10.565	10.383	10.204	10.027	9.8525
1.3	9.6800	9.5098	9.3418	9.1759	9.0123	8.8508	8.6915	8.5343	8.3793	8.2264
1.4	8.0757	7.9270	7.7804	7.6359	7.4934	7.3529	7.2145	7.0781	6.9437	6.8112
1.5	6.6807	6.5522	6.4255	6.3008	6.1780	6.0571	5.9380	5.8208	5.7053	5.5917
1.6	5.4799	5.3699	5.2616	5.1551	5.0503	4.9471	4.8457	4.7460	4.6479	4.5514
1.7	4.4565	4.3633	4.2716	4.1815	4.0930	4.0059	3.9204	3.8364	3.7538	3.6727
1.8	3.5930	3.5148	3.4380	3.3625	3.2884	3.2157	3.1443	3.0742	3.0054	2.9379
1.9	2.8717	2.8067	2.7429	2.6803	2.6190	2.5588	2.4998	2.4419	2.3852	2.3295
2.0	2.2750	2.2216	2.1692	2.1178	2.0675	2.0182	1.9699	1.9226	1.8763	1.8309
2.1	1.7864	1.7429	1.7003	1.6586	1.6177	1.5778	1.5386	1.5003	1.4629	1.4262
2.2	1.3903	1.3553	1.3209	1.2874	1.2545	1.2224	1.1911	1.1604	1.1304	1.1011
2.3	1.0724	1.0444	1.0170	0.99031	0.96419	0.93867	0.91375	0.88940	0.86563	0.84242
2.4	0.81975	0.79763	0.77603	0.75494	0.73436	0.71428	0.69469	0.67557	0.65691	0.63872
2.5	0.62097	0.60366	0.58677	0.57031	0.55426	0.53861	0.52336	0.50849	0.49400	0.47988
2.6	0.46612	0.45271	0.43965	0.42692	0.41453	0.40246	0.39070	0.37926	0.36811	0.35726
2.7	0.34670	0.33642	0.32641	0.31667	0.30720	0.29798	0.28901	0.28028	0.27179	0.26354
2.8	0.25551	0.24771	0.24012	0.23274	0.22557	0.21860	0.21182	0.20524	0.19884	0.19262
2.9	0.18658	0.18071	0.17502	0.16948	0.16411	0.15889	0.15382	0.14890	0.14412	0.13949
3.0	0.13499	0.13062	0.12639	0.12228	0.11829	0.11442	0.11067	0.10703	0.10350	0.10008
3.1	0.09676	0.09354	0.09043	0.08740	0.08447	0.08164	0.07888	0.07622	0.07364	0.07114
3.2	0.06871	0.06637	0.06410	0.06190	0.05976	0.05770	0.05571	0.05377	0.05190	0.05009
3.3	0.04834	0.04665	0.04501	0.04342	0.04189	0.04041	0.03897	0.03758	0.03624	0.03495
3.4	0.03369	0.03248	0.03131	0.03018	0.02909	0.02803	0.02701	0.02602	0.02507	0.02415
3.5	0.02326	0.02241	0.02158	0.02078	0.02001	0.01926	0.01854	0.01785	0.01718	0.01653
3.6	0.01591	0.01531	0.01473	0.01417	0.01363	0.01311	0.01261	0.01213	0.01166	0.01121
3.7	0.01078	0.01036	0.00996	0.00957	0.00920	0.00884	0.00850	0.00816	0.00784	0.00753
3.8	0.00723	0.00695	0.00667	0.00641	0.00615	0.00591	0.00567	0.00544	0.00522	0.00501
3.9	0.00481	0.00461	0.00443	0.00425	0.00407	0.00391	0.00375	0.00359	0.00345	0.00330
4.0	0.00317	0.00304	0.00291	0.00279	0.00267	0.00256	0.00245	0.00235	0.00225	0.00216

17. APPENDIX 8: FLOOD HYDRAULICS

17.1 FLOW RESISTANCE COEFFICIENT ESTIMATION

Estimation of the resistance coefficient (either the Chézy C, Darcy-Weisbach f or Manning n) is possibly the greatest source of uncertainty in determining flood levels. The Manning n is the most widely used, and general guidance for selecting values for different conditions is included in the various software manuals, and in guideline documents, such as that by Arcement and Schneider (1989). A common approach is to select a value by matching channel characteristics to previously calibrated values as presented in tables (e.g. Chow, 1959) or photographic guides (e.g. Barnes, 1967; Hicks and Mason, 1991). Table 17-1 lists selected values as recommended by Chow (1959) for a wide range of conditions and by Julien (2002) for sand-bed channels.

The use of tables is unreliable because of the largely subjective description of channel characteristics, the high variability of natural characteristics, and the significant variation of n with flow conditions. These all contribute to the wide ranges of values in Table 17-1. Photographic guides improve the ability to select a representative value, but it remains difficult to predict the direction and rate of trend of n with flow depth or discharge. Various procedures and equations have also been proposed for relating resistance coefficients to quantifiable channel characteristics and flow conditions; some of these are presented here.

Flow resistance in rivers arises mainly from two sources: surface resistance resulting from the shear stress at the boundary in contact with the flow, and form resistance resulting from the unsymmetrical distribution of pressure and the dissipation of turbulent energy produced by flow separation around submerged or partially submerged boundary irregularities. The latter includes resistance associated with emergent vegetation and boulders, and also with flow patterns induced by the channel form, such as secondary circulation around bends. The origins of the Chézy, Darcy-Weisbach and Manning equations show that they were developed for, and are really only appropriate for, describing surface resistance. The Darcy-Weisbach equation is favoured where surface resistance dominates, because f is dimensionless. The Manning equation has been widely used in natural channels where resistance contributions other than surface shear are significant. The coefficient n is then used as a lumped parameter to include the effects of surface irregularities, cross section variations, obstructions, vegetation and channel planform in addition to surface roughness. The Chézy equation is not widely used in practice.

Table 17-1: Manning n values for natural rivers (adapted from Chow (1959) and Julien (2002))

Channel Description	Range of n
Minor streams (< 30 m wide)	
Main channel	
Clean and straight	0.025-0.033
Straight with stones and weeds	0.030-0.040
Clean, winding	0.033-0.045
Winding with stones and weeds	0.035-0.060
Very weedy with pools, timber and underbrush	0.075-0.150
Mountain streams, tree-lined, gravel or cobble beds	0.030-0.050
Mountain streams, tree-lined, cobbles with large boulders	0.040-0.070
Flood plains	
Grass, short	0.025-0.035
Grass, high	0.030-0.050
Field crops, mature	0.030-0.050
Brush, scattered, heavy weeds	0.035-0.070
Brush, light with trees in foliage	0.040-0.080
Brush, medium to dense with trees in foliage	0.070-0.160
Heavy stand of trees, little undergrowth, flood stage below branches	0.080-0.120
Heavy stand of trees, little undergrowth, flood stage within branches	0.100-0.160
Major streams (> 30 m wide)	
Regular, no boulders or brush	0.025-0.060
Irregular, rough section	0.035-0.10
Sand-bed channels	
Lower regime (plane, ripples, dunes)	0.010-0.040
Upper regime (plane, antidunes)	0.010-0.018

For small artificial channels where resistance is due only to shear resistance, f can be reliably estimated using the Moody diagram or the corresponding Blasius and Colebrook-White equations. These apply to both laminar and all the regimes of turbulent flow. For flood flows in rivers conditions will invariably be hydraulically rough-turbulent and more appropriate methods are available. It is noteworthy that, for shear resistance, f varies significantly with flow depth while n (due to the higher exponent on R in the Manning equation) is relatively constant with flow depth. Both vary significantly where form roughness dominates, to account for the inappropriateness of the equations.

The effects of the two sources of resistance on stage-discharge relationships are significantly different. The depth-averaged flow velocity increases significantly with flow depth where shear resistance dominates but stays relatively constant where form resistance dominates. This means that an increase in discharge is accommodated in shear-dominated flow largely by an increase in velocity, with a relatively small increase in flow depth, and in form-dominated flow largely by an increase in depth with little change in velocity.

Although emphasis here is on resistance coefficients for one-dimensional modelling, it is important to note that values may be different for two- or three-dimensional modelling. A resistance coefficient accounts for the effects on the flow of all processes that are not explicitly modelled. For example, flow around a bend involves complex three-dimensional secondary circulation. The resistance coefficient used in a three-dimensional model would only need to account for surface shear resistance, but in a one-dimensional model would need to account also for the energy loss associated with the secondary circulation. The value of resistance coefficient must therefore be chosen to account for the processes that are not described explicitly by the model being used, i.e. the value to be specified depends on the model used as well as on the characteristics of the river. The specification of resistance coefficient values for different resolution models is discussed further by McGahey et al. (2009) and Morvan et al. (2008).

17.1.1. Channel bed resistance

The effect of the channel bed is a primary consideration when evaluating the resistance in a river. It is important first to distinguish between immobile bed and mobile bed conditions. All alluvial riverbeds move during sufficiently high flows, but those consisting of gravels, cobbles or boulders may often be considered to be immobile. Under both conditions the bed presents both surface and form resistance, but different treatments are required because immobile roughness elements are fixed and can be measured, while mobile beds change form with flow condition and predicting the form must be part of the analysis.

<u>Immobile beds</u>

For an immobile bed, the resistance coefficient depends strongly on the size of the substrate material (k) relative to the flow depth (D). For river flood flows the roughness will be primarily in the small-scale ($D/k > \sim 10$) to intermediate-scale ($\sim 1 < D/k < 10$) ranges. Various methods for predicting f across these ranges have been proposed (e.g. Hey, 1979; Ferguson, 2007; Rickenmann and Recking, 2011; Namaee et al., 2017). The simplest reliable equation is the logarithmic relationship for f, as calibrated by Hey (1979), using $k = k_s = 3.5d_{84}$, where d_{84} is the size of bed particles for which 84% are smaller, i.e.

$$\frac{1}{\sqrt{f}} = 2.03 \log\left(\frac{12.4\,R}{3.5\,d_{84}}\right) \qquad \text{Eq. 17-1}$$

This has been shown to agree well with more complicated equations for $\sim 0.3 < D/d_{84} < 100$.

<u>Mobile beds</u>

A granular bed becomes mobile when the surface shear stress exceeds a critical value; the bed particles are then set in motion and bed forms develop. These forms follow a recognized sequence with distinctly different geometries as the flow intensity increases (Figure 17-1).

Lower regime bed forms are associated with subcritical flow, have profiles out of phase with the water surface, and present significant form resistance to the flow. Upper regime bed forms are associated with supercritical flow, have profiles in phase with the water surface and present relatively little form resistance to the flow. Because the nature and size of bed forms change with the flow condition, the overall resistance of a riverbed can vary considerably with discharge. It is common for the stage-discharge relationship to have dislocated variations for the lower and upper regimes, with a range of discharges occurring for approximately the same flow depth through the transition between them. For the case shown in Figure 17-2 the velocity (and hence the discharge) increases by a factor of two without the flow depth changing, as the dunes wash out in the transition. (This is a wide, shallow channel; variations are typically more gradual for deep channels.) It is clear that a single resistance coefficient cannot be assigned to such a river.

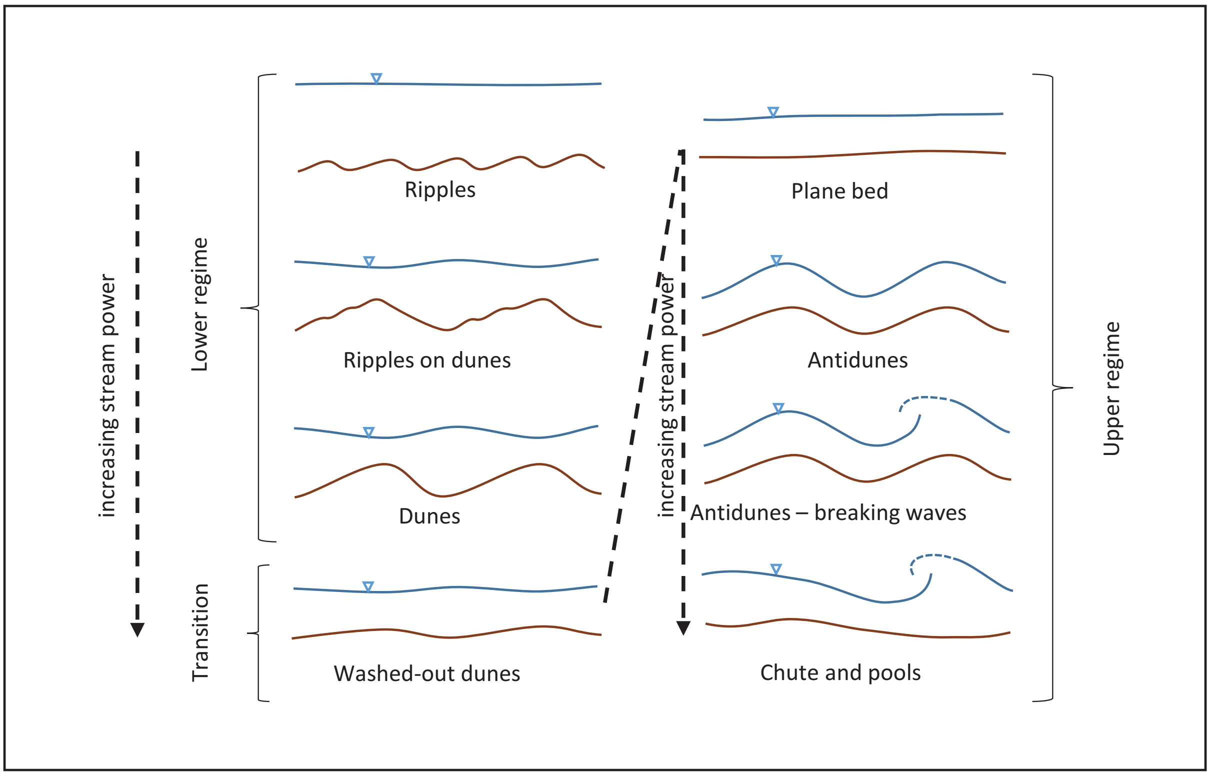


Figure 17-1: Classification of bed forms (adapted from Simons and Richardson (1961))

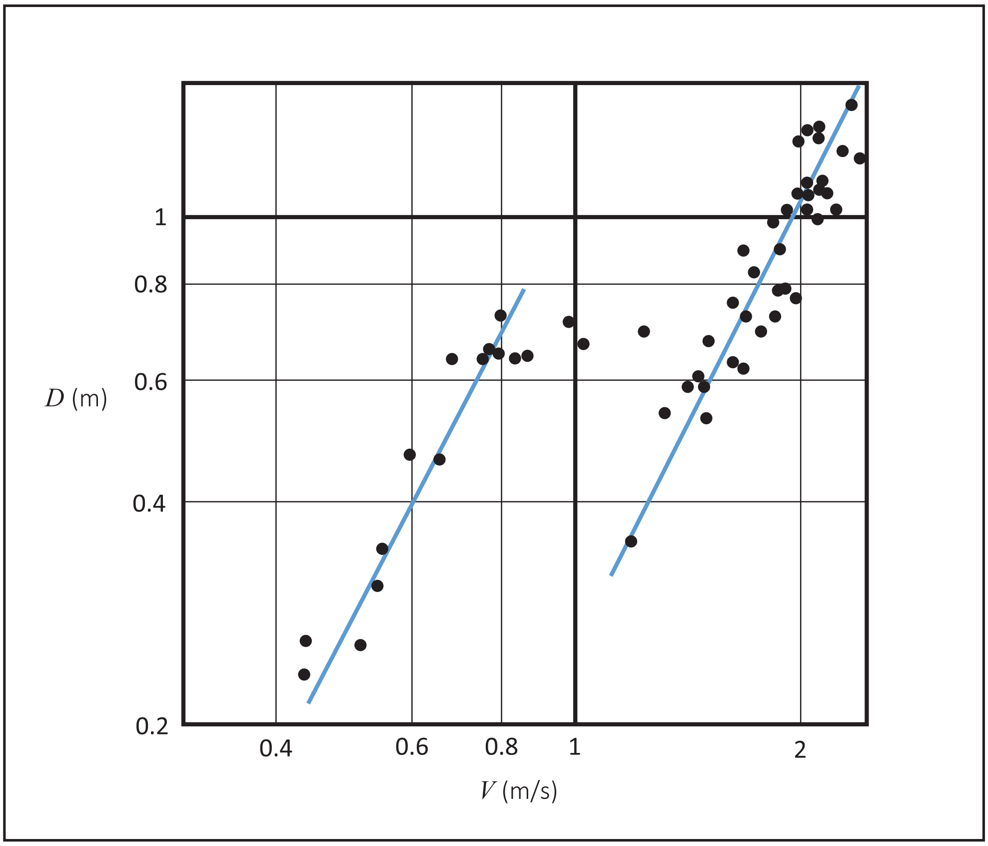


Figure 17-2: Stage-velocity relationship for the Rio Grande near Bernalillo, New Mexico (adapted from Nordin (1964), as presented by Graf (1971))

Many methods have been proposed for estimating alluvial resistance. Among the best known and most widely accepted are those of Einstein & Barbarossa (1952), Engelund (1966, 1967), Brownlie (1983), van Rijn (1984), White et al. (1987), Karim (1995) and Wu and Wang (1999, 2001). All involve the calculation of a depth-discharge pair by nominating one and calculating the other. The method of Brownlie (1983) is the only one that enables solution of a particular depth-discharge pair without iteration for both lower and upper regime conditions. It can be applied through the following steps to calculate the flow depth (assumed to be equal to the hydraulic radius for a wide channel) corresponding to a specified unit-width discharge.

Data input required: S, v, S_s, d_{16}, d_{50}, d_{84}

1. Specify the unit-width discharge (q) for which the flow depth is required.
2. Calculate q_* from

$$q_* = \frac{q}{\sqrt{g\,d_{50}^3}} \qquad \text{Eq. 17-2}$$

   and σ_g from

$$\sigma_g = \sqrt{\frac{d_{84}}{d_{16}}} \qquad \text{Eq. 17-3}$$

3. Calculate R_L (the hydraulic radius if flow is in the lower regime) from

$$\frac{R_L}{d_{50}} = 0.3724\,q_*^{0.6539}\,S^{-0.2542}\,\sigma_g^{0.1050} \qquad \text{Eq. 17-4}$$

   and R_U (the hydraulic radius if flow is in the upper regime) from

$$\frac{R_U}{d_{50}} = 0.2836\,q_*^{0.6248}\,S^{-0.2877}\,\sigma_g^{0.08013} \qquad \text{Eq. 17-5}$$

4. Determine the actual regime:

   - Calculate $V = q/R$, and hence Fr_g from

$$\mathrm{Fr}_g = \frac{V}{\sqrt{(S_s - 1)\,g\,d_{50}}} \qquad \text{Eq. 17-6}$$

     for both regimes, i.e. Fr_{gU} and Fr_{gL}.

- Calculate Fr_g^* from

$$Fr_g^* = 1.74\, S^{-1/3} \qquad \text{Eq. 17-7}$$

and δ from

$$\delta = \frac{11.6\nu}{u_*^{/}} \qquad \text{Eq. 17-8}$$

with $u_*^{/} = (gR_U S)^{0.5}$

- Calculate the lower limit of the upper regime, Fr^*_{gU}, from

$$Fr^*_{gU} = 1.25\, Fr_g^* \qquad \text{Eq. 17-9}$$

if $d_{50}/\delta \geq 2$, or

$$\log\left(\frac{Fr^*_{gU}}{Fr_g^*}\right) = -0.02469 + 0.1517 \log\left(\frac{d_{50}}{\delta}\right) + 0.8381\left(\log\left(\frac{d_{50}}{\delta}\right)\right)^2 \qquad \text{Eq. 17-10}$$

if $d_{50}/\delta < 2$

- Calculate the upper limit of the lower regime, Fr^*_{gL}, from

$$Fr^*_{gL} = 0.8\, Fr_g^* \qquad \text{Eq. 17-11}$$

if $d_{50}/\delta \geq 2$, or

$$\log\left(\frac{Fr^*_{gL}}{Fr_g^*}\right) = -0.2026 + 0.07026 \log\left(\frac{d_{50}}{\delta}\right) + 0.9330\left(\log\left(\frac{d_{50}}{\delta}\right)\right)^2 \qquad \text{Eq. 17-12}$$

if $d_{50}/\delta < 2$

$Fr_{gU} > Fr^*_{gU}$ indicates upper regime, $Fr_{gL} < Fr^*_{gL}$ indicates lower regime, and values between the limits indicate the transitional state. Select R_L or R_U accordingly, as the correct hydraulic radius for the specified unit-width discharge.

Example 17.1: Compute the stage-discharge relationship, and hence the variation with depth of Manning's n for an alluvial channel with the following characteristics.

Channel gradient = 0.00095 m/m

Sediment $S_s = 2.65$

$d_{16} = 0.19$ mm

$d_{50} = 0.30$ mm

$d_{84} = 0.55$ mm

Solution: Calculations are presented for one condition in each flow regime; results are given for the full range.

Lower Regime

1. Specify $q = 0.40\ \mathrm{m^3/s/m}$

2. Then

$$q_* = \frac{q}{\sqrt{g\,d_{50}^3}} \qquad \text{Eq. 17-2}$$

$$= \frac{0.40}{\sqrt{9.8 \cdot 0.00030^3}} = 24600$$

$$\sigma_g = \sqrt{\frac{d_{84}}{d_{16}}} \qquad \text{Eq. 17-3}$$

$$= \sqrt{\frac{0.00055}{0.00019}} = 1.60$$

3.

$$R_L = d_{50} \cdot 0.3724\, q_*^{0.6539}\, S^{-0.2542}\, \sigma_g^{0.1050} \qquad \text{Eq. 17-4}$$

$$= 0.00030 \cdot 0.3724 \cdot 24600^{0.6539} \cdot 0.00095^{-0.2542} \cdot 1.60^{0.1050}$$

$$= 0.512\ \mathrm{m}$$

$$R_U = d_{50} \cdot 0.2836\, q_*^{0.6248}\, S^{-0.2877}\, \sigma_g^{0.08013} \qquad \text{Eq. 17-5}$$

$$= 0.00030 \cdot 0.2836 \cdot 24600^{0.6248} \cdot 0.00095^{-0.2877}\, 1.60^{0.08013}$$

$$= 0.362\ \mathrm{m}$$

4. Regime:

$$V_L = \frac{q}{R_L} = \frac{0.40}{0.512} = 0.781 \text{ m/s}$$

$$\text{Fr}_{gL} = \frac{V_L}{\sqrt{(S_s - 1)\, g\, d_{50}}} \qquad \text{Eq. 17-6}$$

$$= \frac{0.781}{\sqrt{(2.65-1)\cdot 9.8 \cdot 0.00030}} = 11.2$$

$$V_U = \frac{q}{R_U} = \frac{0.40}{0.362} = 1.11 \text{ m/s}$$

$$\text{Fr}_{gU} = \frac{V_U}{\sqrt{(S_s - 1)\, g\, d_{50}}} \qquad \text{Eq. 17-6}$$

$$= \frac{1.11}{\sqrt{(2.65-1)\cdot 9.8 \cdot 0.00030}} = 15.9$$

$$\text{Fr}_g^* = 1.74\, S^{-1/3} \qquad \text{Eq. 17-7}$$

$$= 1.74 \cdot 0.00095^{-1/3} = 17.7$$

$$u_*^{/} = \sqrt{g\, R_U\, S}$$

$$= \sqrt{9.8 \cdot 0.362 \cdot 0.00095} = 0.0581 \text{ m/s}$$

$$\delta = \frac{11.6\nu}{u_*^{/}} \qquad \text{Eq. 17-8}$$

$$= \frac{11.6 \cdot 00000010}{0.058} = 0.00020 \text{ m}$$

so $\dfrac{d_{50}}{\delta} = \dfrac{0.00030}{0.00020} = 1.50 < 2$

and then

$$\log\left(\frac{\text{Fr}^*_{gL}}{\text{Fr}_g^*}\right) = -0.2026 + 0.07026\log\left(\frac{d_{50}}{\delta}\right) + 0.9330\left(\log\left(\frac{d_{50}}{\delta}\right)\right)^2 \qquad \text{Eq. 17-12}$$

$$= -0.2026 + 0.07026\log(1.50) + 0.9330\left(\log(1.50)\right)^2$$

$$= -0.161$$

so $\dfrac{\mathrm{Fr}^*_{gl}}{\mathrm{Fr}^*_g} = 10^{-0.161} = 0.689$ and $\mathrm{Fr}^*_{gL} = 0.689 \cdot 17.7 = 12.2$

$$\log\left(\frac{\mathrm{Fr}^*{}_{gU}}{\mathrm{Fr}^*_g}\right) = -0.02469 + 0.1517\log\left(\frac{d_{50}}{\delta}\right) + 0.8381\left(\log\left(\frac{d_{50}}{\delta}\right)\right)^2 \quad \text{Eq. 17-10}$$

$$= -0.02469 + 0.1517\log(1.50) + 0.8381(\log(1.50))^2$$

$$= 0.028$$

so $\dfrac{\mathrm{Fr}^*_{gU}}{\mathrm{Fr}^*_g} = 10^{0.028} = 1.07$ and $\mathrm{Fr}^*_{gU} = 1.07 \cdot 17.7 = 18.9$

$Fr_{gL} < Fr^*_{gL}$ $(11.2 < 18.9)$ so regime is Lower, and $R = R_L = 0.512 \text{ m}$

and Manning $n = \dfrac{R_L^{2/3} S^{1/2}}{V_L} = \dfrac{0.512^{2/3} \cdot 0.00095^{1/2}}{0.781} = 0.025$

Transitional Regime

1. Specify $q = 0.80 \text{ m}^3/\text{s/m}$

2. Then $q_* = \dfrac{q}{\sqrt{g\, d_{50}^3}}$ Eq. 17-2

$$= \frac{0.80}{\sqrt{9.8 \cdot 0.00030^3}} = 49181$$

$$\sigma_g = \sqrt{\frac{d_{84}}{d_{16}}} \quad \text{Eq. 17-3}$$

$$= \sqrt{\frac{0.00055}{0.00019}} = 1.60$$

3. $R_L = d_{50} \cdot 0.3724\, q_*^{0.6539}\, S^{-0.2542}\, \sigma_g^{0.1050}$ Eq. 17-4

$$= 0.00030 \cdot 0.3724 \cdot 49181^{0.6539} \cdot 0.00095^{-0.2542} \cdot 1.60^{0.1050}$$

$$= 0.805 \text{ m}$$

$$\begin{aligned} R_U &= d_{50} \cdot 0.2836\, q_*^{0.6248}\, S^{-0.2877}\, \sigma_g^{0.08013} \\ &= 0.00030 \cdot 0.2836 \cdot 49181^{0.6248} \cdot 0.00095^{-0.2877}\, 1.60^{0.08013} \qquad \text{Eq. 17-5} \\ &= 0.559 \text{ m} \end{aligned}$$

4. Regime:

$$V_L = \frac{q}{R_L} = \frac{0.80}{0.805} = 0.994 \text{ m/s}$$

$$\begin{aligned} \mathrm{Fr}_{gL} &= \frac{V_L}{\sqrt{(S_s - 1)\, g\, d_{50}}} \qquad \text{Eq. 17-6} \\ &= \frac{0.994}{\sqrt{(2.65-1) \cdot 9.8 \cdot 0.00030}} = 14.2 \end{aligned}$$

$$V_U = \frac{q}{R_U} = \frac{0.80}{0.559} = 1.43 \text{ m/s}$$

$$\begin{aligned} \mathrm{Fr}_{gU} &= \frac{V_U}{\sqrt{(S_s - 1)\, g\, d_{50}}} \qquad \text{Eq. 17-6} \\ &= \frac{1.43}{\sqrt{(2.65-1) \cdot 9.8 \cdot 0.00030}} = 20.5 \end{aligned}$$

$$\begin{aligned} \mathrm{Fr}_g^* &= 1.74\, S^{-1/3} \qquad \text{Eq. 17-7} \\ &= 1.74 \cdot 0.00095^{-1/3} = 17.7 \end{aligned}$$

$$\begin{aligned} u_*' &= \sqrt{g\, R_U\, S} \\ &= \sqrt{9.8 \cdot 0.559 \cdot 0.00095} = 0.0721 \text{ m/s} \end{aligned}$$

$$\begin{aligned} \delta &= \frac{11.6\nu}{u_*'} \qquad \text{Eq. 17-8} \\ &= \frac{11.6 \cdot 00000010}{0.0721} = 0.00016 \text{ m} \end{aligned}$$

so $\qquad \dfrac{d_{50}}{\delta} = \dfrac{0.00030}{0.00016} = 1.88 < 2$

and then

$$\log\left(\frac{\mathrm{Fr}^*_{gL}}{\mathrm{Fr}^*_g}\right) = -0.2026 + 0.07026\log\left(\frac{d_{50}}{\delta}\right) + 0.9330\left(\log\left(\frac{d_{50}}{\delta}\right)\right)^2 \quad \text{Eq. 17-12}$$

$$= -0.2026 + 0.07026\log(1.88) + 0.9330(\log(1.88))^2$$

$$= -0.113$$

so $\dfrac{\mathrm{Fr}^*_{gl}}{\mathrm{Fr}^*_g} = 10^{-0.113} = 0.771$ and $\mathrm{Fr}^*_{gL} = 0.771 \cdot 17.7 = 13.6$

$$\log\left(\frac{\mathrm{Fr}^*_{gU}}{\mathrm{Fr}^*_g}\right) = -0.02469 + 0.1517\log\left(\frac{d_{50}}{\delta}\right) + 0.8381\left(\log\left(\frac{d_{50}}{\delta}\right)\right)^2 \quad \text{Eq. 17-10}$$

$$= -0.02469 + 0.1517\log(1.88) + 0.8381(\log(1.88))^2$$

$$= 0.080$$

so $\dfrac{\mathrm{Fr}^*_{gU}}{\mathrm{Fr}^*_g} = 10^{0.080} = 1.20$ and $\mathrm{Fr}^*_{gU} = 1.20 \cdot 17.7 = 21.2$

$Fr_{gL} > Fr^*_{gL}$ $(14.2 > 13.6)$ and $Fr_{gU} < Fr^*_{gU}$ $(20.5 < 21.2)$ so regime is Transitional. R is therefore undetermined, and Manning's n cannot be calculated.

<u>Upper Regime</u>

1. Specify $q = 2.40\ \mathrm{m^3/s/m}$

2. Then $q_* = \dfrac{q}{\sqrt{g\, d_{50}^3}}$ Eq. 17-2

$$= \frac{2.40}{\sqrt{9.8 \cdot 0.00030^3}} = 147500$$

$$\sigma_g = \sqrt{\frac{d_{84}}{d_{16}}} \quad \text{Eq. 17-3}$$

$$= \sqrt{\frac{0.00055}{0.00019}} = 1.60$$

3. $$R_L = d_{50} \cdot 0.3724 q_*^{0.6539} S^{-0.2542} \sigma_g^{0.1050}$$ Eq. 17-4

$$= 0.00030 \cdot 0.3724 \cdot 147500^{0.6539} \cdot 0.00095^{-0.2542} \cdot 1.60^{0.1050}$$

$$= 1.65 \text{ m}$$

$$R_U = d_{50} \cdot 0.2836 q_*^{0.6248} S^{-0.2877} \sigma_g^{0.08013}$$ Eq. 17-5

$$= 0.00030 \cdot 0.2836 \cdot 147500^{0.6248} \cdot 0.00095^{-0.2877} \, 1.60^{0.08013}$$

$$= 1.11 \text{ m}$$

4. Regime:

$$V_L = \frac{q}{R_L} = \frac{2.40}{1.65} = 1.45 \text{ m/s}$$

$$\mathrm{Fr}_{gL} = \frac{V_L}{\sqrt{(S_s - 1) g \, d_{50}}}$$ Eq. 17-6

$$= \frac{1.45}{\sqrt{(2.65 - 1) \cdot 9.8 \cdot 0.00030}} = 20.8$$

$$V_U = \frac{q}{R_U} = \frac{2.40}{1.11} = 2.16 \text{ m/s}$$

$$\mathrm{Fr}_{gU} = \frac{V_U}{\sqrt{(S_s - 1) g \, d_{50}}}$$ Eq. 17-6

$$= \frac{2.16}{\sqrt{(2.65 - 1) \cdot 9.8 \cdot 0.00030}} = 30.9$$

$$\mathrm{Fr}_g^* = 1.74 S^{-1/3}$$ Eq. 17-7

$$= 1.74 \cdot 0.00095^{-1/3} = 17.7$$

$$u_*^{/} = \sqrt{g R_U S}$$

$$= \sqrt{9.8 \cdot 1.11 \cdot 0.00095} = 0.102 \text{ m/s}$$

$$\delta = \frac{11.6 \nu}{u_*^{/}}$$ Eq. 17-8

$$= \frac{11.6 \cdot 00000010}{0.102} = 0.000114 \text{ m}$$

so $\dfrac{d_{50}}{\delta} = \dfrac{0.00030}{0.000114} = 2.63 > 2$

and then

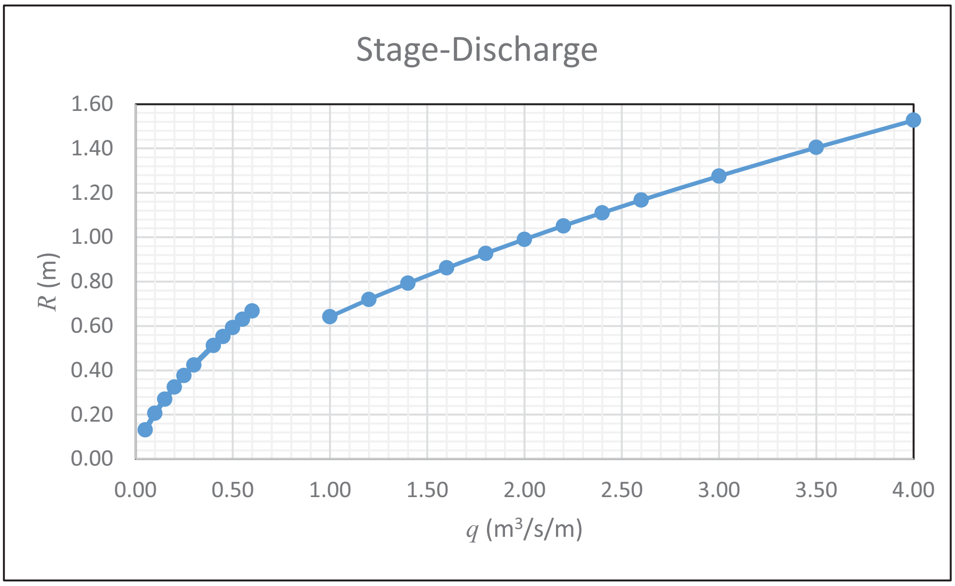
$$\mathrm{Fr}^*_{gL} = 0.8\,\mathrm{Fr}^*_g = 0.8\cdot 17.7 = 14.2 \qquad \text{Eq. 17-11}$$

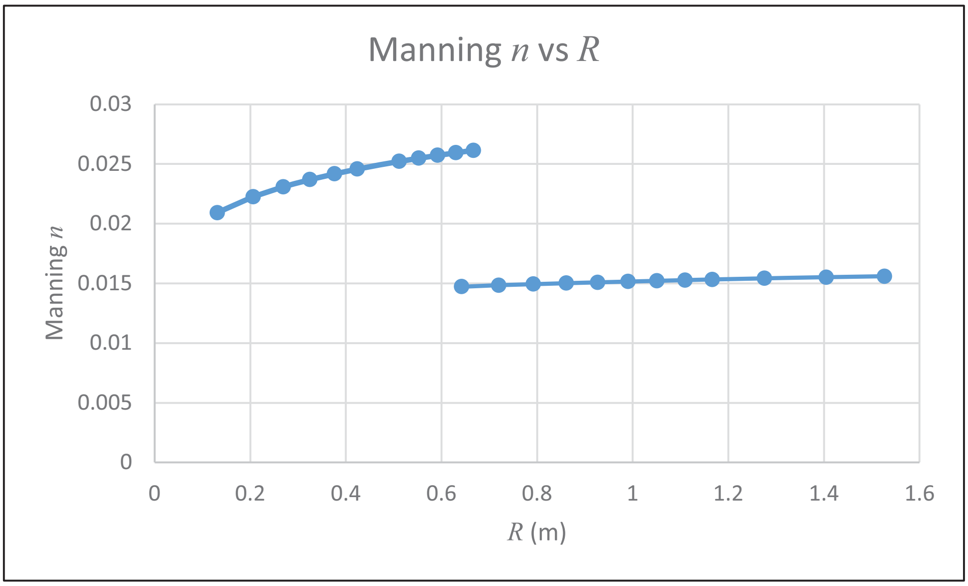
$$\mathrm{Fr}^*_{gU} = 1.25\,\mathrm{Fr}^*_g = 1.25\cdot 17.7 = 22.1 \qquad \text{Eq. 17-9}$$

$Fr_{gU} > Fr^*_{gU}$ $(30.9 > 22.1)$ so regime is Upper, and $R = R_U = 1.11\ \mathrm{m}$

and Manning $n = \dfrac{R_U^{2/3}\,S^{1/2}}{V_U} = \dfrac{1.11^{2/3}\cdot 0.00095^{1/2}}{2.16} = 0.015$

Repeating the calculations for a range of q values gives the stage-discharge relationship and the variation of Manning's n with depth shown below. Notice how the discharge almost doubles as shown in the graph below with no increase in flow depth around 0.6 m, and that Manning's n increases through the lower regime and then reduces significantly to remain almost constant in the upper regime.





17.1.2. Vegetation resistance

The effect of vegetation on flow resistance is difficult to describe because of the variability of plant characteristics, their spatial distribution, seasonal changes and reconfiguration of foliage under the influence of the flow. Fully submerged and emergent vegetation require different treatment – resistance under submerged conditions can be treated as being mainly surface type, while emergent stems impose significant form resistance. The resistance changes significantly as submergence takes place, as illustrated by the relationship between Manning's n and flow depth for medium-length turf grass compiled by Ree (1949) (Figure 17-3). The following review is taken mostly from James (2010).

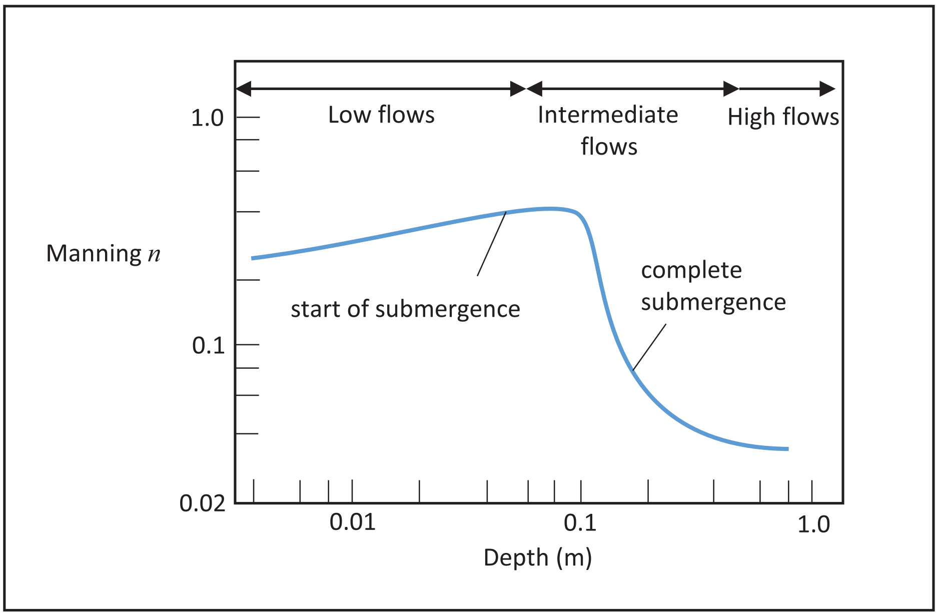


Figure 17-3: Variation of flow resistance of grass with flow depth (adapted from Ree (1949))

Submerged vegetation

Most methods proposed for estimating the resistance of submerged vegetation have been developed for grasses used for lining artificial channels, but may also be used for grass-type vegetation in rivers and on flood plains. Kouwen and his co-workers have developed a method for estimating the resistance coefficient that explicitly accounts for the effect of bending (Kouwen and Unny, 1973; Kouwen and Li, 1980; Kouwen *et al.*, 1981; Kouwen, 1988; Kouwen, 1992). The Darcy-Weisbach equation (Eq. 9-3) is used, with the resistance coefficient given by

$$\frac{1}{\sqrt{f}} = a + b \log\left(\frac{D}{k}\right) \qquad \text{Eq. 17-13}$$

in which a and b are coefficients that depend on the bent state of the vegetation (Table 17-2), and k is the roughness height (m) given by

$$k = 0.14\, h \left(\frac{\left(\frac{MEI}{\tau_o}\right)^{0.25}}{h}\right)^{1.59} \qquad \text{Eq. 17-14}$$

in which : h = the vegetation height (m)

M = a nondimensional representation of stem density

E = the stem material's modulus of elasticity (N/m^2)

I = the stem's second moment of area (m^4)

τ_o = the total boundary shear (N/m^2) as given by

$$\tau_o = \rho g D S \qquad \text{Eq. 17-15}$$

The vegetation lining characteristics represented by M, E, and I are lumped together and treated as one variable MEI (Nm^2). The coefficients a and b depend on whether the stems are erect or prone, which is determined by the relationship of the boundary shear velocity ($=(\tau_o/\rho)^{0.5}$) to a critical value given by the lesser of

$$u_{*\,crit} = 0.028 + 6.33\,MEI^2 \qquad \text{Eq. 17-16}$$

and

$$u_{*\,crit} = 0.23\,MEI^{0.106} \qquad \text{Eq. 17-17}$$

The values of a and b for different conditions defined by the shear velocity are listed in Table 17-2.

Table 17-2: Values of coefficients a and b for submerged vegetation

Condition	Criterion	a	b
erect	$u_*/u_{*crit} \le 1.0$	0.15	1.85
prone	$1.0 < u_*/u_{*crit} \le 1.5$	0.20	2.70
prone	$1.5 < u_*/u_{*crit} \le 2.5$	0.28	3.08
prone	$2.5 < u_*/u_{*crit}$	0.29	3.50

The value of MEI (Nm^2) can be determined from the following relationships with the vegetation stem length h (m), obtained from measurements on natural grass linings.

All data: $$MEI = 223\,h^{3.125} \qquad \text{Eq. 17-18}$$

Green grasses: $$MEI = 319\,h^{3.3}$$ Eq. 17-19

Dead or dormant grasses: $$MEI = 25.4\,h^{2.26}$$ Eq. 17-20

Example 17.2: A flood plain on a slope of 0.0010 is covered by grass with a blade length of 10 cm. Determine the variation of Manning's n for flow depths up to 1.0 m.

Solution: Calculations are shown for one depth, and results for the full range.

Choose $D = 0.50$ m

Manning's n can be determined from the Darcy-Weisbach f through the equivalence of the equations. The method of Kouwen (1992) allows calculation of f from

$$\frac{1}{\sqrt{f}} = a + b\log\left(\frac{D}{k}\right)$$ Eq. 17-13

$$k = 0.14\,h\left(\frac{\left(\dfrac{MEI}{\tau_o}\right)^{0.25}}{h}\right)^{1.59}$$ Eq. 17-14

$h = 0.10$ m

$$\begin{aligned}\tau_o &= \rho\,g\,D\,S \\ &= 1000\cdot 9.8\cdot 0.50\cdot 0.0010 \\ &= 4.90\ \text{N/m}^2\end{aligned}$$ Eq. 17-15

For all grass data: $$\begin{aligned}MEI &= 223\,h^{3.125} \\ &= 223\cdot 0.10^{3.125} \\ &= 0.167\ \text{Nm}^2\end{aligned}$$ Eq. 17-18

So $$k = 0.14\cdot 0.10\left(\frac{\left(\dfrac{0.167}{4.90}\right)^{0.25}}{0.10}\right)^{1.59} = 0.142\ \text{m}$$

a: $\quad u_* = \sqrt{\dfrac{\tau_o}{\rho}} = \sqrt{\dfrac{4.90}{1000}} = 0.070 \text{ m/s}$

$u_{*\,crit}$ is the lesser of

$$u_{*\,crit} = 0.028 + 6.33\, MEI^2 \qquad \text{Eq. 17-16}$$
$$= 0.028 + 6.33 \cdot 0.167^2$$
$$= 0.205 \text{ m/s}$$

or

$$u_{*\,crit} = 0.23\, MEI^{0.106} \qquad \text{Eq. 17-17}$$
$$= 0.23 \cdot 0.167^{0.106}$$
$$= 0.190 \text{ m/s}$$

so $\quad u_{*\,crit} = 0.190 \text{ m/s}$, the lesser value

and $\quad \dfrac{u_*}{u_{*\,crit}} = \dfrac{0.070}{0.190} = 0.37$

Hence from Table 17-2, $\quad a = 0.15$

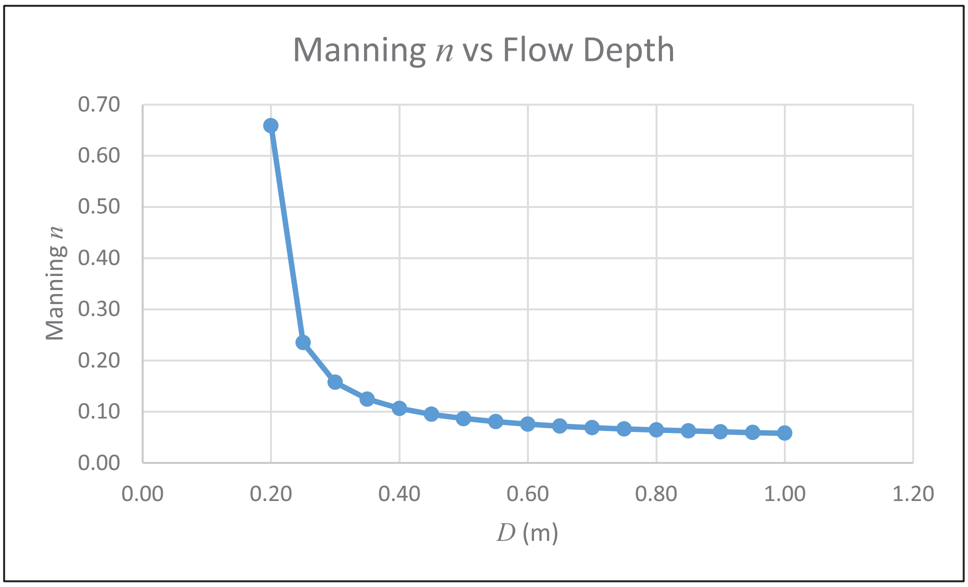
And from Table 17-2 $\quad b = 1.85$

Therefore $\quad \dfrac{1}{\sqrt{f}} = 0.15 + 1.85 \log\left(\dfrac{0.50}{0.142}\right) = 1.16$

(Note that this equation applies only for positive values of $1/(f^{0.5})$, i.e. for $\dfrac{D}{k} > 10^{-(a/b)}$, which is for D > about 0.18 m in this case)

Then $\quad n = \dfrac{D^{1/6}}{\sqrt{8g}\,\dfrac{1}{\sqrt{f}}} = \dfrac{0.50^{1/6}}{\sqrt{8 \cdot 9.8} \cdot 1.16} = 0.087$

Repeating the calculations for a range of depths gives the variation of Manning's n with depth as shown in the graph below. Note the similar trend with Figure 17-3 in the submerged zone.



Extensive emergent vegetation

Emergent vegetation (e.g. reeds and bulrushes) is common in rivers, flood plains and palustrine wetlands, where it imposes significant resistance on the flow (similar to the low flow region in Figure 17-3. It may occur extensively or in fragmented distributions, especially as strips along river banks or in patches within river channels, each situation requiring different treatment.

For extensive emergent vegetation formulations have been developed that include the surface resistance of the bed and the form resistance of the stems (Petryk and Bosmajian, 1975; James et al., 2004 and others). As shown by James (2021), these can be expressed as either the Darcy-Weisbach (Eq. 9-3) or Manning (Eq. 9.4) equation with the resistance coefficient being a combination of surface and form contributions. The Darcy-Weisbach friction factor is then given by

$$f = f^{/} + f^{//} \qquad \text{Eq. 17-21}$$

in which $f^{/}$ is the usual friction factor for the bed, and $f^{//}$ is an effective friction factor for the form resistance, given by

$$f^{//} = 4\,C_D \frac{A_p}{A_{bf}} \qquad \text{Eq. 17-22}$$

in which C_D = the stem drag coefficient

A_p = the total stem area projected in the flow direction, which can be approximated as NDd

N = the number of stems per unit area

D = the flow depth

d = the stem diameter

A_{bf} = the bed area not exposed to shear stress, which can be approximated as $(1-N\pi d^2/4)$

Using the Manning equation, the effective resistance coefficient is given by

$$n = \sqrt{n'^2 + n''^2} \qquad \text{Eq. 17-23}$$

with n' being the value for the bed and n'' the value accounting for form resistance, given by

$$n'' = \sqrt{\frac{R^{1/3}}{8g} 4 C_D \frac{A_p}{A_{bf}}} \qquad \text{Eq. 17-24}$$

The hydraulic radius, R, can be approximated by the flow depth.

James et al. (2008) derived relationships for C_D from experimental values for bulrushes, reeds and willow stems. Average values are represented by

$$C_D = 221\mathrm{Re}^{-0.57} \qquad \text{Eq. 17-25}$$

with upper limits represented by

$$C_D = 701\mathrm{Re}^{-0.66} \qquad \text{Eq. 17-26}$$

and lower limits by

$$C_D = 51\mathrm{Re}^{-0.43} \qquad \text{Eq. 17-27}$$

In these equations Re is the stem Reynolds number, given by Vd/ν.

Note that an iterative solution is required for the solution of the resistance equation because the required V is needed to calculate Re. This can be done by assuming a value for V and then iterating the calculations to satisfactory convergence.

If stage-discharge data are available for calibration, it would be most convenient and reliable to lump together the resistance terms and calibrate a single friction factor or Manning n, using the flow depth to represent the hydraulic radius.

Example 17.3: An extensive reedbed has a slope of 0.0020. The reeds have an average diameter of 5 mm and there are about 150 stems per square metre. The substrate has a Manning n value of 0.015. Compute the stage-discharge and stage-velocity relationships, and the variation with depth of the total Manning's n for flow depths up to 1.0 m.

Solution: Calculations are shown for one depth, and results for the full range. Because the discharge coefficient varies with the Reynolds number, the solution is iterative, beginning with an assumed velocity; only the calculations for the correct velocity are shown.

Choose $D = 0.50\ \text{m}$

Try $V = 0.0865\ \text{m/s}$

$$V = \frac{1}{n} D^{2/3} S^{1/2} \qquad \text{Eq. 9.4}$$

$$n = \sqrt{n'^2 + n''^2} \qquad \text{Eq. 17-23}$$

$$n' = 0.015$$

$$n'' = \sqrt{\frac{D^{1/3}}{8g} 4 C_D \frac{A_p}{A_{bf}}} \qquad \text{Eq. 17-24}$$

$$A_p = N\, d\, D$$
$$= 150 \cdot 0.005 \cdot 0.50 = 0.375\ \text{m}^2$$

$$A_{bf} = 1 - N \frac{\pi d^2}{4}$$
$$= 1 - 150 \frac{\pi\, 0.005^2}{4} = 0.997\ \text{m}^2$$

$$C_D = 221 \text{Re}^{-0.57} \qquad \text{Eq. 17-25}$$

$$\text{Re} = \frac{V\, d}{\nu}$$
$$= \frac{0.0865 \cdot 0.005}{0.000010} = 433$$

So $$C_D = 221 \cdot 433^{-0.57} = 6.94$$

and $$n^{//} = \sqrt{\frac{0.50^{1/3}}{8\,g} 4 \cdot 6.94 \frac{0.375}{0.997}} = 0.325$$

Then $$n = \sqrt{n^{/2} + n^{//2}} = \sqrt{0.015^2 + 0.325^2} = 0.325$$

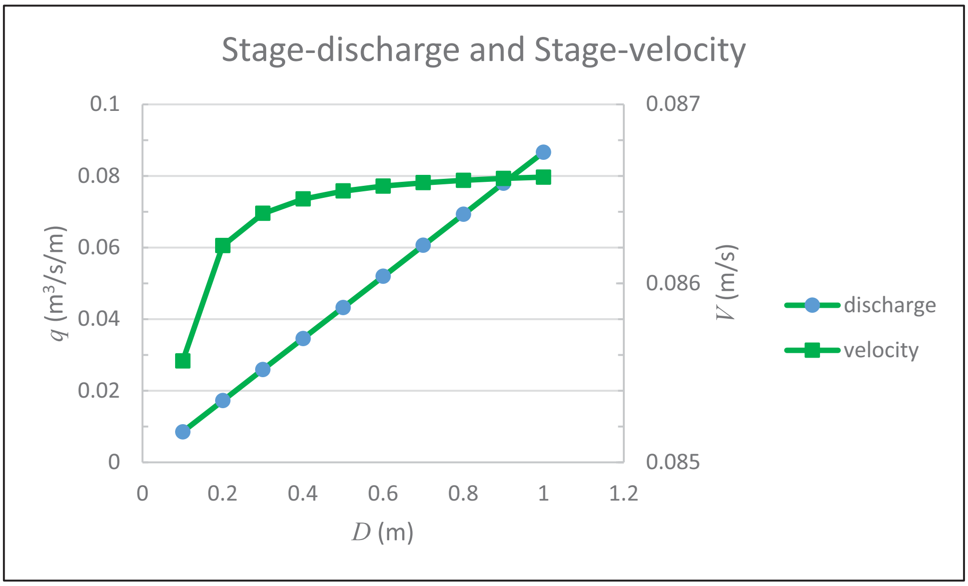
and $$V = \frac{1}{0.325} 0.50^{2/3}\, 0.0020^{1/2} = 0.0866 \text{ m/s}$$

which is close enough to the assumed value to accept.

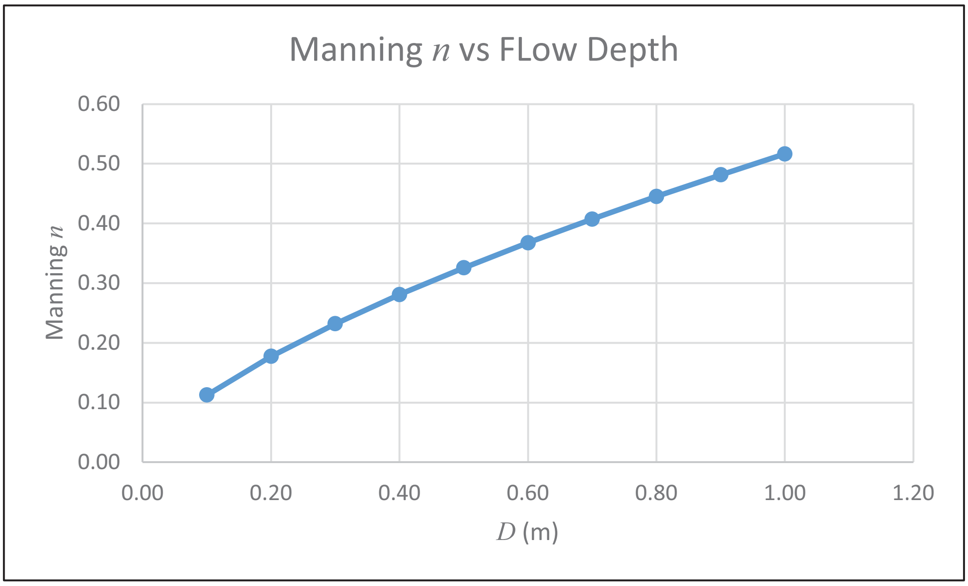
The unit-width discharge is then the product of the velocity and the flow depth, i.e.

$$q = V \cdot D = 0.0866 \cdot 0.50 = 0.0433 \text{ m}^3\text{/s/m}$$

Repeating the calculations for other flow depths leads to the stage-discharge and stage-velocity relationships shown below.



Note that the velocity changes rapidly with depth at shallow depths, and then becomes relatively constant at greater depths. This confirms that when form roughness dominates over boundary shear an increase in discharge is accommodated more by an increase in depth than velocity. The strong variation in Manning's n with depth (shown below) arises because the equation form is based on resistance by boundary shear and is inconsistent with dominant form roughness; the inconsistency must be accounted for by the n value.



Emergent bank vegetation

Emergent vegetation frequently occurs in discontinuous patterns in rivers, a particularly common occurrence being as strips along river banks. In such cases, the total channel conveyance can be estimated by subdividing the cross-section into vegetated and clear zones (Figure 17-4) calculating the discharge separately for the different zones and then adding the zonal discharges (James and Makoa, 2006), i.e.

$$Q_{total} = Q_{veg} + Q_{clear} \qquad \text{Eq. 17-28}$$

where Q_{total} is the total discharge and Q_{veg} and Q_{clear} are the discharges within the vegetated and clear zones respectively. If the bank strips are narrow Q_{veg} may be insignificant in comparison to Q_{clear}, and could be ignored.

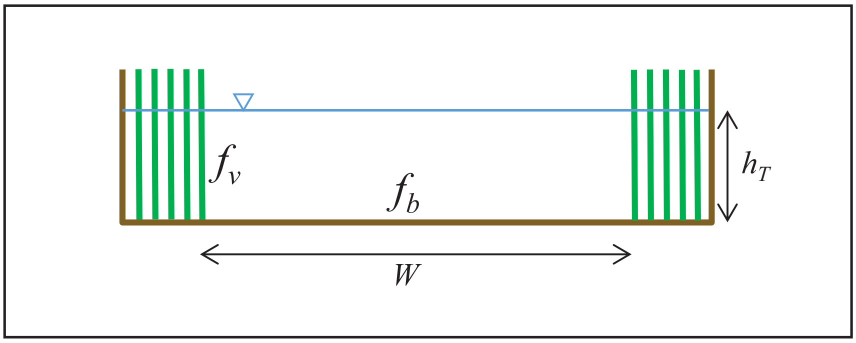


Figure 17-4: Subdivision of cross-section into clear and vegetated zones

The velocity within the vegetation strips can be calculated as described for extensive vegetation above. The average velocity within the clear channel section between the vegetation boundaries can be calculated using a conventional resistance equation with a composite resistance coefficient accounting for the bed surface and vegetation boundaries. Hirschowitz (2007) showed that the overall Darcy-Weisbach resistance coefficient can be calculated as

$$f = \frac{f_b W + 2 f_v h_T}{W + 2 h_T} \qquad \text{Eq. 17-29}$$

in which

f_b and f_v = the resistance coefficients for the bed and vegetation interface surfaces respectively

W = the bed width (m)

h_T = the water depth at the vegetation interface (m)

Eq. 17-29 allows for vegetation on both banks but is easily modified if it is on one bank only.

The resistance coefficient for the bed (f_b) can be estimated by the methods described in Section 17.1.1. For the vegetation interface, Kaiser (1984) proposed that

$$f_v = f_{To} + f_I \qquad \text{Eq. 17-30}$$

in which f_{To} is due to the vegetation structure. Kaiser (1984) suggested $0.06 < f_{To} < 0.10$, but Hirschowitz and James (2009) suggest that this term is probably negligible for width-depth ratios greater than about 5. The term f_I is due to the flow interaction, and is given by

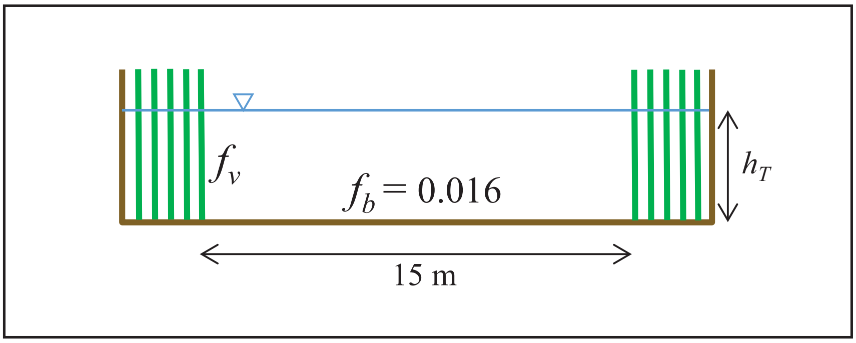
$$f_I = 0.18 \log\left(0.0135 \frac{V_{\text{inf}}^{\ 2}}{h_T V_v^{\ 2}} \right) \qquad \text{Eq. 17-31}$$

In Eq. 17-31 V_{inf} is the depth-averaged velocity that would occur as a result of bed resistance only without the influence of vegetation and can be estimated by the methods presented in Section 17.1.1. V_v is the unaffected velocity within the vegetation, which can be calculated as for extensive emergent vegetation, above. The height h_T is measured in metres (the number 0.0135 is also a length in metres).

These calculations should be carried out in terms of the Darcy-Weisbach resistance coefficients, and only converted to Manning's n for the total composite value if it is required in this form. (The bed and vegetation interface f values cannot be converted to Manning's n values without knowing or assuming the associated values of R.)

Example 17.4: Reeds with the characteristics given in example 18.3 grow along the banks of a river with a slope of 0.0020 and a clear width of 15 m. Assume f for the channel bed to be 0.016. Assuming the discharge contribution of the bank zones to be negligible, compute the stage-discharge relationship and the variation with depth of the total Manning's n for flow depths up to 1.0 m for the clear channel.

Solution: Calculations are shown for one depth, and results for the full range. The relevant equations are in terms of the Darcy-Weisbach f, so this equation is used and Manning's n calculated from the results.



Choose $D = h_T = 0.50$ m

$$Q = AV = A\sqrt{\frac{8g}{f}}\sqrt{RS}$$

$$R = \frac{A}{P}$$

$$A = W h_T = 15 \cdot 0.50 = 7.5 \text{ m}^2$$

$$P = W + 2h_T = 15 + 2 \cdot 0.5 = 16 \text{ m}$$

So $$R = \frac{7.5}{16} = 0.47 \text{ m}$$

$$f = \frac{f_b W + 2 f_v h_T}{W + 2 h_T} \qquad \text{Eq. 17-29}$$

$$f_b = 0.016$$

$$f_v = f_{To} + f_I \qquad \text{Eq. 17-30}$$

$$f_{To} = 0 \quad \text{for a wide channel}$$

$$f_I = 0.18 \log\left(0.0135\,\frac{V_{\inf}^{\,2}}{h_T\,V_v^{\,2}}\right) \qquad \text{Eq. 17-31}$$

$$V_{\inf} = \sqrt{\frac{8g}{f}}\sqrt{RS}$$
$$= \sqrt{\frac{8\cdot 9.8}{0.016}}\sqrt{0.5\cdot 0.0020} = 2.21 \text{ m/s}$$

$$V_v = 0.0866 \text{ m/s} \quad \text{from Example (17.3)}$$

So

$$f_I = 0.18 \log\left(0.0135\,\frac{V_{\inf}^{\,2}}{h_T\,V_v^{\,2}}\right)$$
$$= 0.18 \log\left(0.0135\,\frac{2.21^2}{0.50\cdot 0.0866^2}\right) = 0.224$$

and $f_v = 0 + 0.224 = 0.224$

Then

$$f = \frac{f_b\,W + 2\,f_v\,h_T}{W + 2\,h_T} = \frac{0.016\cdot 15 + 2\cdot 0.224\cdot 0.50}{15 + 2\cdot 0.50} = 0.029$$

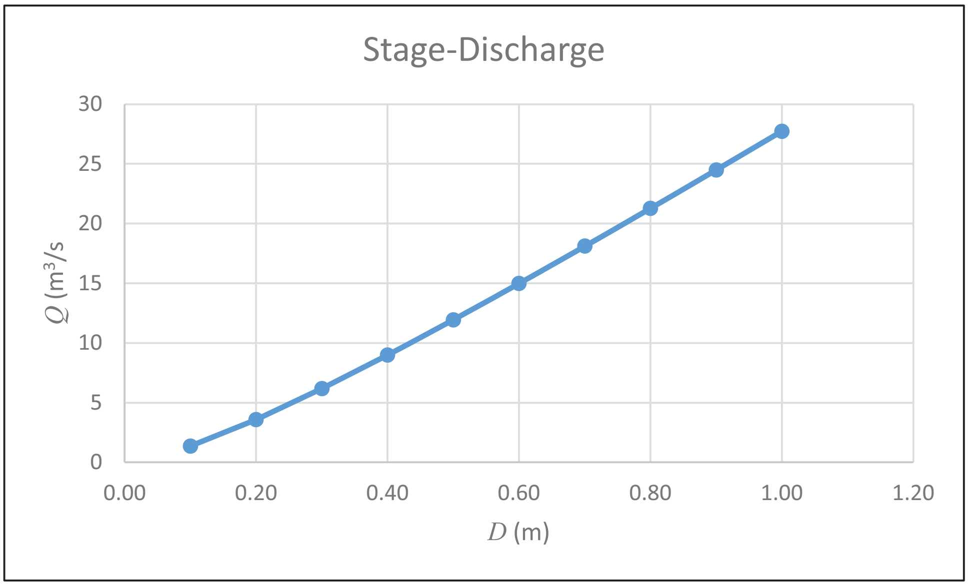
So

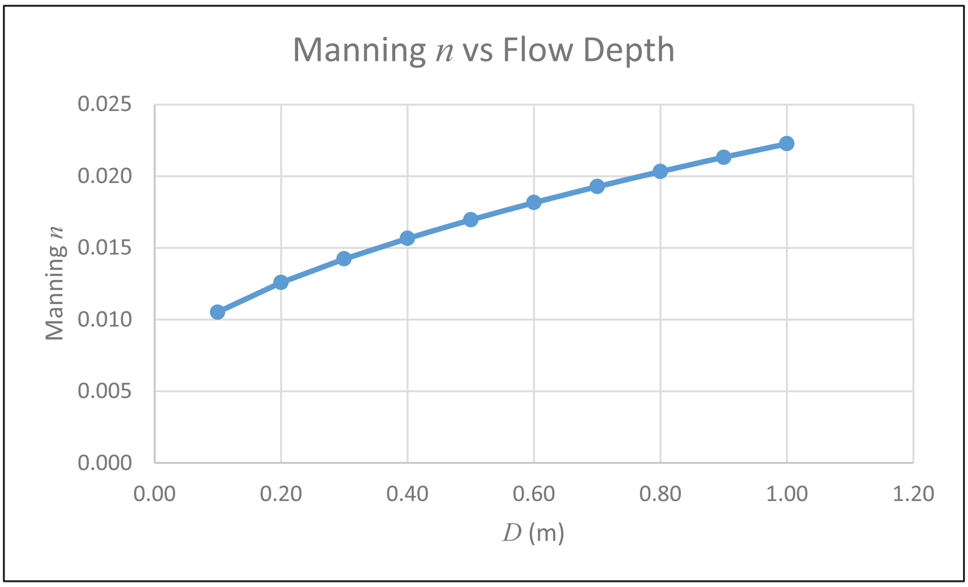
$$Q = 7.5\cdot\sqrt{\frac{8\cdot 9.8}{0.029}}\sqrt{0.47\cdot 0.0020} = 12.0 \text{ m}^3/\text{s}$$

Manning's *n* can be found from *f*, through the equivalence of the Darcy-Weisbach and Manning equations, i.e.

$$n = \frac{R^{1/6}\sqrt{f}}{\sqrt{8g}} = \frac{0.47^{1/6}\sqrt{0.029}}{\sqrt{8\cdot 9.8}} = 0.017$$

Repeating the calculations for other flow depths leads to the stage-discharge relationship and the variation of Manning's *n* with depth shown below.





17.1.3. Bend resistance

The resistance to flow of a channel is significantly increased by the presence of bends. The additional resistance is the result of the development and decay of secondary circulation as flow progresses through a bend.

The most widely used method for accounting for bend losses in meandering channels is the SCS method, proposed by the United States Soil Conservation Service (1963), which provides an adjustment to the basic value of Manning's n in terms of the channel sinuosity (s) (which is defined as the distance along the channel between two points divided by the straight line distance between the points). The adjustment, as linearized by James (1994), is expressed as

$$\left.\begin{aligned} \frac{n^{/}}{n} &= \left(\frac{f^{/}}{f}\right)^{1/2} = 0.43\,s + 0.57 \qquad \text{for } s < 1.7 \\ \frac{n^{/}}{n} &= \left(\frac{f^{/}}{f}\right)^{1/2} = 1.30 \qquad \text{for } s \geq 1.7 \end{aligned}\right\} \qquad \text{Eq. 17-32}$$

in which $n^{/}$ is the adjusted value. The energy loss is actually associated with the bend characteristics, rather than the sinuosity per se, and the SCS adjustment implicitly assumes a particular form of bend to occur commonly in natural channels. James (1995a) applied a secondary circulation model developed by Chang (1983, 1984) to develop a more general, rationally based relationship, i.e.

$$\frac{n^{/}}{n} = \left(\frac{f^{/}}{f}\right)^{1/2} = 0.992\,e^{2.03\,D/r_c} \qquad \text{Eq. 17-33}$$

in which

r_c = the radius of curvature of the bends (m)

Liu (1997) proposed a purely empirical equation based on laboratory data, i.e.

$$\frac{n^{/}}{n} = \left(\frac{f^{/}}{f}\right)^{1/2} = 0.941\,e^{0.764\,b/r_c} \qquad \text{Eq. 17-34}$$

in which

b = the channel width (m)

Very tight inner bends sometimes induce the occurrence of local critical flow with subsequent expansion, causing local acceleration or "spill" resistance (Leopold et al., 1960). James and Myers (2002) proposed an empirical equation for bend resistance where this phenomenon is known to occur, i.e.

$$\frac{n^{/}}{n} = \left(\frac{f^{/}}{f}\right)^{1/2} = 12.052\left(\frac{b}{r_c}\right)^{1.152}\left(\frac{D}{r_c}\right)^{0.605} \qquad \text{Eq. 17-35}$$

Spill resistance is probably uncommon in natural and most designed bend geometries, but its effect is significant. James and Myers (2002) recommend Eq. 17-35 if spill resistance is known to occur, Eq. 17-33 if it is expected not to occur, and Eq. 17-34 if its occurrence is uncertain.

17.1.4. Spatially-varied roughness

At a channel-reach scale the bed surface roughness may vary across the width and/or along the length of the channel. This may result from the occurrence of different bed particle sizes through hydraulic sorting, varying bed form sizes and shapes in sand-bed channels, or growth patterns of in-channel vegetation. The Conveyance Estimation System (CES) (Knight et al., 2010) proposes combining up to three components, to allow for surface (n_{sur}), vegetation (n_{veg}) and irregularity (n_{irr}) contributions to a local "unit roughness", n_l. These components may be recognised independently and combined as

$$n_l = \left(n_{sur}^2 + n_{veg}^2 + n_{irr}^2\right)^{1/2} \qquad \text{Eq. 17-36}$$

Flow interactions between regions with different roughnesses can enhance the resistance on a reach scale (Garbrecht and Brown, 1991) and accurate conveyance prediction requires 2D turbulence modelling. (The CES accounts for the transverse interaction in terms of the depth-averaged velocity.) Unless the interactions are extreme (such as with compound channels, as described in Section 17.1.5), quite reliable estimates are possible for longitudinally consistent variations across a section by subdividing the cross-section and summing the constituent conveyances. Garbrecht and Brown (1991) demonstrate that the error incurred by ignoring flow interactions in following this approach is within 5% for channels with width-to-depth ratios exceeding 20, but can be significant in relatively narrow channels. Some 1D models (e.g. HEC-RAS (Brunner, 2016)) follow this approach and allow specification of limited variation of n across sections. An alternative approach is to specify an effective value of Manning's n to represent the resistance of the entire cross-section, n_e.

A number of formulations have been proposed for estimating an effective Manning n value for situations where only surface roughness contributes to resistance, and the roughness varies across the flow section (but not longitudinally) (James and Jordanova, 2010). These can be expressed as weighted averages of local Manning n values, i.e.

$$n_e = \left(\frac{\sum_{i=1}^{N}\left(K_i\, n_i^a\right)}{K}\right)^{1/a} \qquad \text{Eq. 17-37}$$

in which: n_e = the effective value

n_i = the local value

i = subscript referring to the subsection associated with n_i,

N = the number of subsections considered

K = the weighting variable

a = an exponent that depends on the nature of the relationship assumed between subsection flow conditions

The most commonly used expressions of Eq. 17-37 are those of Pavlovski (1931) who proposed $a = 2$, and Horton (1933) who proposed $a = 3/2$, both with K equal to the wetted perimeter, P.

James and Jordanova (2010) showed that Eq. 17-37 can be used for longitudinal as well as transverse variations, using $K = A$ (the surface area covered) and $a = 1$. This means that an effective value of n can be reasonably estimated as an average of constituent values weighted simply by areal coverage, i.e.

$$n_e = \frac{\sum_{i=1}^{N}\left(A_i\, n_i\right)}{A} \qquad \text{Eq. 17-38}$$

The effective Manning n approach does not account for the interaction between subsection flows through transverse momentum exchange, which is considerable for overbank flows (see section 17.1.5), but may also be significant for inbank flows where large differences in subsection roughnesses occur. Such interaction effects can be accounted for by lateral distribution computational models, such as that incorporated in the Conveyance Estimation System (CES) produced by HR Wallingford (Knight et al., 2010). This can be applied only to transverse roughness variations, however. The CES includes a "Roughness Advisor" to assist in estimating channel roughness, a "Conveyance Generator" that uses this estimation as well as the channel morphology to predict the channel conveyance, and an "Uncertainty Estimator" for indicating the uncertainty associated with the conveyance calculation.

17.1.5. Compound channels

A compound, or two-stage, channel comprises a main channel with overbank sections or floodplains on one or both sides (Figure 17-5). Compound cross sections are common in natural rivers, and are also frequently engineered to increase conveyance for flood flows while preserving natural conditions in the central portion at lower flows to meet environmental objectives (James, 1995b). At overbank stages the hydraulics is complicated by interaction between the main channel and floodplain flow regions. With a straight main channel the velocity difference between the regions causes intense shear and associated turbulence along their interface, dissipating energy and effectively increasing resistance. The transfer of momentum across the interface implies an effective shear on the interface, which has a resisting effect on the main channel flow and a propelling effect on the flood plain flow. With a meandering main channel there is a significant exchange of volumes of water between the main channel and floodplains, causing energy dissipation through flow expansion and contraction and induced secondary circulation.

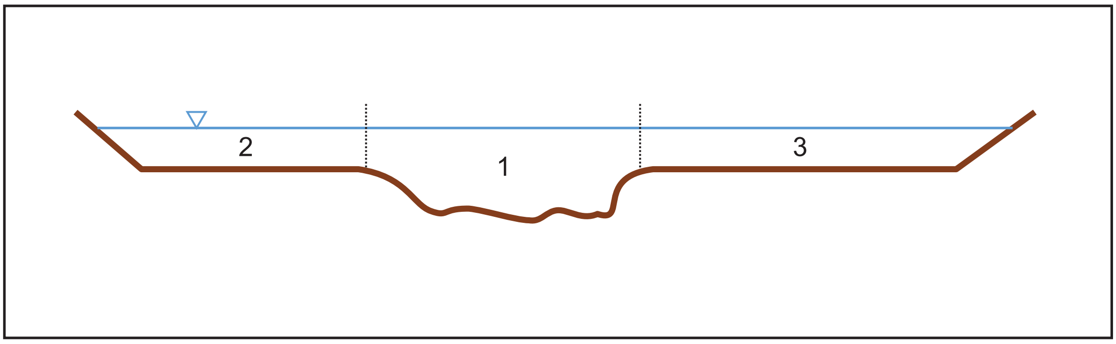


Figure 17-5: Compound channel cross section

In both the straight and meandering cases, the interaction between the flow zones is complex and its effect on conveyance can only be realistically assessed through high resolution computational modelling. The lateral distribution computations of the Conveyance Estimation System (Knight et al., 2010) does account for the interaction in channels with irregular shapes. In most other 1D and 2D models the interaction is not explicitly described and its effect on conveyance must be otherwise accounted for. Chen et al. (2019) provide a comprehensive review of computational and non-computational approaches for predicting stage-discharge relationships in compound channel flows. It is noteworthy that Werner and Lambert (2007) found that 2D codes that do not account for the zonal interaction are not necessarily superior to simpler 1D approaches.

Straight compound channels

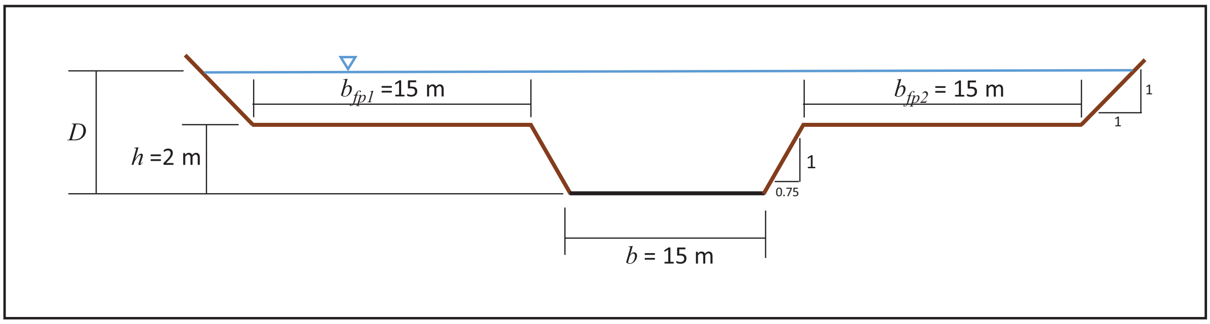
For a straight compound channel, applying a resistance equation, such as Manning's, to the whole compound cross section (the single channel method, SCM) results in an underestimate of the discharge for a specified water level. This is caused by the inadequacy of the equation in dealing with the sudden increase in the wetted perimeter as the water level rises above the flood plain level. An alternative is to divide the cross section into separate main channel and flood plain zones with more regular shapes, applying the resistance equation to obtain the discharge in each zone separately, and then adding them to get the total discharge (the divided channel method, DCM). This approach still does not account for the effective resistance arising from the zonal interaction, and the total discharge will be underestimated.

Treating the whole compound section as a single unit (SCM) will therefore invariably *underestimate* the discharge, while adding zonal discharges with the division surface excluded from the wetted perimeter (DCM) will invariably *overestimate* the discharge. The two calculations will therefore give upper and lower bounds, with the true discharge lying somewhere between them. Depending on the accuracy required, and bearing in mind the sensitivity of the predictions to basic resistance coefficient estimates, a mean of these values may be sufficient for some practical applications. Where greater accuracy is required, adjustment of the DCM value is necessary, for which various methods have been proposed.

The most widely accepted method for computing stage-discharge relationships in straight compound channels is the FCFA method of Ackers (1992). The calculation procedure is complicated and laborious, following the DCM approach with subdivisions as shown in Figure 17-5, and correcting the zonal discharges before addition. The method is also described, with example calculations, by Wark et al. (1994).

Other acceptable methods for straight compound channels have been proposed by Wormleaton and Merrett (1990), Moreta and Martin-Vide (2010), Yang et al. (2012), Christodoulou (1992), Lambert and Myers (1998), and Huthoff et al. (2008). The last three methods each depend on a single empirical parameter, and so would be the easiest to calibrate if field information is available.

Example 17.5: Compare stage-discharge relationships for the compound channel shown in cross section below using the Single Channel Method and the unadjusted Divided Channel Method. The longitudinal slope is 0.00050. Manning's n is 0.025 for the main channel and 0.040 for the floodplains.



Solution: Calculations are shown for one depth, and results for the full range.

Choose $D = 3.0$ m

Single Channel Method

$$Q = AV = A\frac{1}{n}R^{2/3}S^{1/2}$$

$$A = A_{mc} + A_{fp1} + A_{fp2}$$

$$A_{mc} = (b + 2\cdot 0.75\cdot h)D - 2\cdot\frac{1}{2}\cdot 0.75\,h^2$$

$$= (15 + 2\cdot 0.75\cdot 2.0)3.0 - 2\cdot\frac{1}{2}\cdot 0.75\cdot 2.0^2$$

$$= 51.0 \text{ m}^2$$

$$A_{fp1} = A_{fp2} = (D-h)b_{fp1} + \frac{1}{2}(D-h)^2$$

$$= (3.0-2.0)\cdot 15 + \frac{1}{2}(3.0-2.0)^2$$

$$= 15.5 \text{ m}^2$$

$$A = 51.0 + 15.5 + 15.5 = 82.0 \text{ m}^2$$

$$R = \frac{A}{P}$$

$$P = P_{mc} + P_{fp1} + P_{fp2}$$

$$P_{mc} = b + 2\left(h^2 + (0.75h)^2\right)^{0.5}$$

$$= 15 + 2\left(2.0^2 + (0.75 \cdot 2.0)^2\right)^{0.5} = 20.0 \text{ m}^2$$

$$P_{fp1} = P_{fp2} = b_{fp1} + \left(2(D-h)^2\right)^{0.5}$$

$$= 15.0 + \left(2(3.0-2.0)^2\right)^{0.5}$$

$$= 16.4 \text{ m}$$

$$P = 20.0 + 16.4 + 16.4 = 52.8 \text{ m}$$

$$R = \frac{A}{P} = \frac{82.0}{52.8} = 1.55 \text{ m}$$

The composite Manning n can be found from Horton's equation, i.e. Eq. 17-37 with $K = P$ and $a = 3/2$,

$$n_e = \left(\frac{\sum_{i=1}^{N}\left(P_i n_i^{3/2}\right)}{P}\right)^{2/3}$$

So $$n_e = \left(\frac{P_{fp1}\, n_{fp1}^{3/2} + P_{fp2}\, n_{fp2}^{3/2} + P_{mc}\, n_{mc}^{3/2}}{P}\right)^{2/3}$$

$$= \left(\frac{16.4 \cdot 0.040^{3/2} + 16.4 \cdot 0.040^{3/2} + 20.0 \cdot 0.025^{3/2}}{52.8}\right)^{2/3}$$

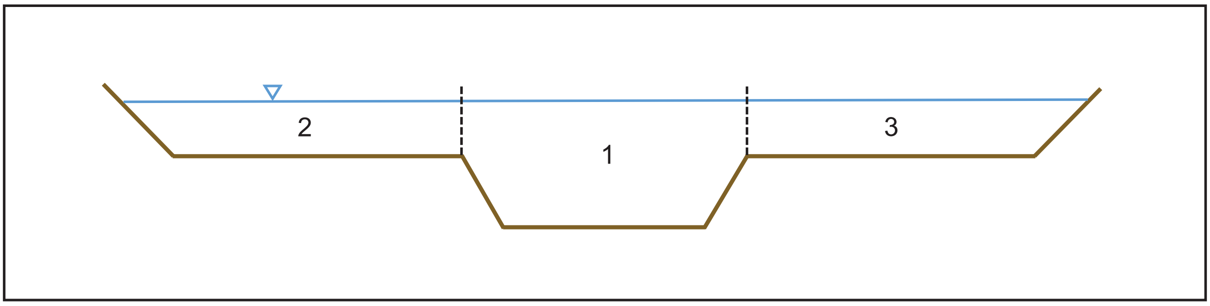
$$= 0.035$$

Therefore

$$Q = 82.0 \frac{1}{0.035} 1.55^{2/3}\, 0.00050^{1/2} = 70.2 \text{ m}^3/\text{s}$$

Divided Channel Method

Divide channel cross section into 3 zones, with vertical divisions at the main channel banks.



$$Q = Q_1 + Q_2 + Q_3$$

$$Q_1 = A_1 V_1 = A_1 \frac{1}{n_1} R_1^{2/3} S^{1/2}$$

$$A_1 = (b + 2 \cdot 0.75 \cdot h) D - 2 \cdot \frac{1}{2} \cdot 0.75 h^2$$

$$= (15 + 2 \cdot 0.75 \cdot 2.0) 3.0 - 2 \cdot \frac{1}{2} \cdot 0.75 \cdot 2.0^2$$

$$= 51.0 \text{ m}^2$$

$$R_1 = \frac{A_1}{P_1}$$

$$P_1 = b + 2\left(h^2 + (0.75h)^2\right)^{0.5}$$

$$= 15 + 2\left(2.0^2 + (0.75 \cdot 2.0)^2\right)^{0.5} = 20.0 \text{ m}^2$$

$$R_1 = \frac{A_1}{P_1} = \frac{51.0}{20.0} = 2.55 \text{ m}$$

So $$Q_1 = 51.0 \cdot \frac{1}{0.025} \cdot 2.55^{2/3} \cdot 0.00050^{1/2} = 85.1 \text{ m}^3/\text{s}$$

$$Q_2 = Q_3 = A_3 V_3 = A_3 \frac{1}{n_3} R_3^{2/3} S^{1/2}$$

$$A_3 = (D - h) b_{fp2} + \frac{1}{2}(D - h)^2$$

$$= (3.0 - 2.0) \cdot 15 + \frac{1}{2}(3.0 - 2.0)^2$$

$$= 15.5 \text{ m}^2$$

$$R_3 = \frac{A_3}{P_3}$$

$$P_3 = b_{fp2} + \left(2(D-h)^2\right)^{0.5}$$

$$= 15.0 + \left(2(3.0-2.0)^2\right)^{0.5}$$

$$= 16.4 \text{ m}$$

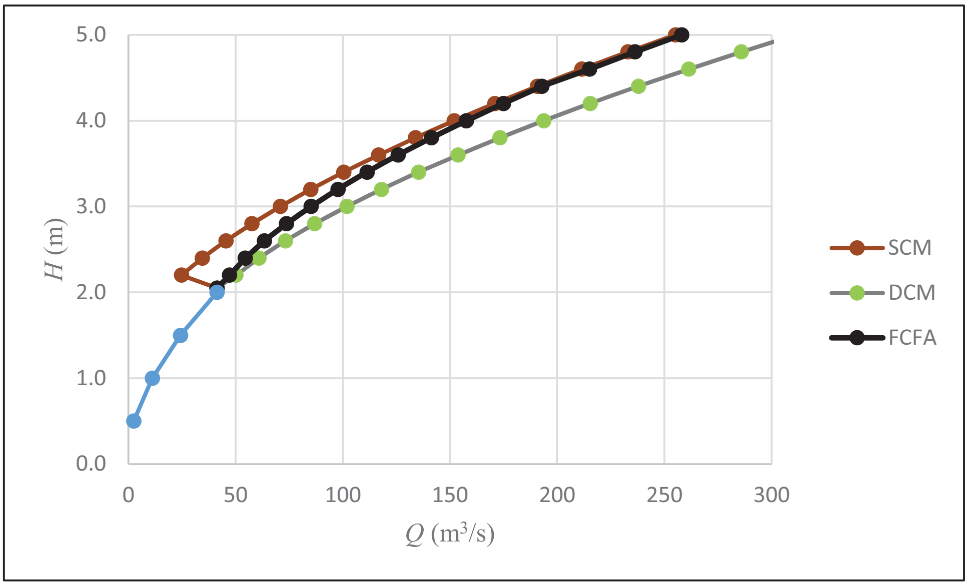
$$R_3 = \frac{A_3}{P_3} = \frac{15.5}{16.4} = 0.95 \text{ m}$$

So $$Q_2 = Q_3 = 15.5 \cdot \frac{1}{0.040} \cdot 0.95^{2/3} \cdot 0.00050^{1/2} = 8.4 \text{ m}^3/\text{s}$$

And

$$Q = Q_1 + Q_2 + Q_3 = 85.1 + 8.4 + 8.4 = 101.9 \text{ m}^3/\text{s}$$

Repeating the calculations for other flow depths leads to the stage-discharge relationships shown below.



Note the significant reduction in discharge predicted by the SCM as the water level rises above the floodplain level. This occurs because P increases substantially with a very small increase in A, resulting in a significant under-representation of R.

The two methods predict respectively an underestimation (SCM) and an overestimation (DCM) of discharge, and vice versa of water level. So, for example, the stage for a discharge of 100 m^3/s lies somewhere between 3.00 and 3.45 m.

For more accurate predictions one of the more complete methods would need to be applied. The stage-discharge relationship as predicted by the FCFA method of Ackers (1992) is also shown. This suggests that the SCM method becomes increasingly accurate as the stage increases.

Meandering compound channels

For meandering compound channels with straight flood plain boundaries, the most widely accepted method is that of James and Wark (1992), also presented with example calculations by Wark et al. (1994). This is also a DCM method, with the cross-section divided into 4 zones: the main channel below the flood plain level, the flood plain over the meander width, and two outer flood plain zones. The zonal discharges are calculated separately, accounting for the relevant energy loss mechanisms, and added to get the total. As for the FCFA method, the calculations are complicated and laborious. Other DCM methods have been proposed by Greenhill and Selllin (1993) and Mohanty (2019), and SCM methods by Shiono et al. (1999) and Rameshwaran and Willetts (1999).

Methods have also been proposed for meandering compound channels with sinuous flood plains by Lambert and Sellin (2000) and James and Myers (2002). The latter is particularly simple, with the total discharge obtained as the sum of the main channel bankfull discharge and the discharge of the floodplain calculated separately. The horizontal division plane is included in the wetted perimeter for the upper channel and excluded for the lower one, and the Manning n values for both adjusted for sinuosity as described in section 17.1.3. This method has not been confirmed for straight flood plains.

17.1.6. Conclusion

Quantification of flow resistance is crucial for reliable flood level prediction. Selection of an appropriate equation and estimation of a representative resistance coefficient is largely subjective, but requires an appreciation of the underlying phenomena and how these are accounted for in the hydraulic model to be used. The resistance coefficient depends on the physical characteristics of the river and also on the flow conditions. The methods presented here are based on limited data and, considering the high variability of natural river characteristics, cannot be expected to be universally accurate. Site specific data should be used wherever possible to confirm results, calibrate equations or develop reliable coefficient formulations. It is sound practice to carry out sensitivity analyses on uncertain parameters, to assess their influences on computed water surfaces.

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