Waikato Regional Council Technical Report 2020/06

Waikato stormwater runoff modelling guideline

Updated version May 2020

www.waikatoregion.govt.nz
ISSN 2230-4355 (Print)
ISSN 2230-4363 (Online)

Waikato
REGIONAL COUNCIL
Te Kaunihera ā Rohe o Waikato

Prepared by:
Earl Shaver (Aqua Terra International Limited)

Edited by:
Megan Wood (Wainui Consulting Limited)

For:
Waikato Regional Council
Private Bag 3038
Waikato Mail Centre
HAMILTON 3240

May 2020

Document #: 16213155

Peer reviewed by: Megan Wood (Wainui Consulting Limited) Duncan Grant (Waikato Regional Council)	Date	August 2019

Approved for release by: Brent Sinclair (Waikato Regional Council)	Date	May 2020

Preface

Waikato Regional Council is pleased to release two new guidelines to address stormwater management: *Waikato stormwater management guideline* (TR2020/07) and *Waikato stormwater runoff modelling guideline* (TR2020/06).

Hamilton is the fourth largest city in New Zealand and while not equivalent in size to the large metropolitans, the population is forecast to increase by 32 per cent between 2006 and 2031[1]. The population in the Waikato Region grew faster than the national average between 2006 and 2013, with fastest growth experienced in the Waikato District (10.1%), Waipa District (9.8%) and Hamilton City (9.3%)[2].

The region supports over 35,000 km of streams and rivers, many of which are impacted by both rural and urban land use. The level of forecasted population growth in the region makes it imperative to ensure appropriate management of urban stormwater to help to protect our region's waterways from further degradation and to restore and enhance them.

The Waikato Regional Council has a number of statutory plans and policies that provide the framework to manage the region's natural resources and that support the formation of the Waikato Stormwater Management Guideline. The Te Ture Whaimana o Te Awa o Waikato – the Vision and Strategy is the prevailing document and is embedded within the Waikato Regional Policy Statement. Te Ture Whaimana o Te Awa o Waikato covers the Waikato and Waipa Rivers and their catchments. The entire Waikato Region and the remainder of the catchments not captured under the Te Ture Whaimana o Te Awa o Waikato are covered by the Waikato Regional Plan which must give effect to the Waikato Regional Policy Statement, which in turn must give effect to the National Policy Statement for Freshwater Management.

In producing these documents, the Waikato Regional Council would like to acknowledge the history of stormwater management and the many individuals who have contributed to progressing stormwater practice in New Zealand and overseas.

History of development and effort

Stormwater management best practice guidelines in use within the country largely have their origins with the Auckland Regional Council's (ARC's) documents: *Guidelines for stormwater runoff modelling in the Auckland Region* ("TP108" in 1999), *Stormwater treatment devices design guideline manual* ("TP10" in 1992 and an update in 2003) and *Low impact design* manual for the Auckland Region ("TP124" manual" in 2000). Research and promulgation emanated from monitoring of streams, estuaries and harbours that revealed issues around sediment and chemical contaminants, and the alteration of the timing and quantity of rainfall-runoff that comes with development. ARC's guidelines reflected overseas knowledge and practice, and research and characteristics of the Auckland Region undertaken in the 1990s. In the 2000s Wellington Regional Council, Auckland City/Metrowater, North Shore City, Waitakere City, Christchurch City Council, Kapiti District Council and other cities and councils also progressed some of its own targeted research, while developing guidance or rules for stormwater management in its jurisdiction.

ARC continued to investigate stormwater under the Stormwater Action Plan that commenced in 2004, while Crown Research Institutes Landcare Research (e.g., Low Impact Design and Development research programme) and the National Institute for Water and Atmospheric Research (several stormwater and estuarine research programmes) progressed understanding and new tools. The University of Auckland and other universities undertook additional stormwater research. Guidelines produced in the 2010s incorporated the findings of the New

[1] Waikato Regional Land Transport Programme 2012/13 – 2014-15

[2] https://www.waikatoregion.govt.nz/Environment/Environmental-information/Environmental-indicators/Community-and-economy/p1a-report/p1a-data/ Viewed June 2018.

Zealand and overseas research, including the Hawkes Bay and Bay of Plenty Regional Councils' stormwater guidelines and the New Zealand Transport Agency stormwater guideline.

Overseas work drawn upon by the Waikato Regional Council and other agencies in New Zealand include research and practice that are embodied in technical reports and best practice guidelines from the United States Department of Agriculture National Resource Conservation Service (NRCS), United States Environmental Protection Agency, United Stated Federal Highway Agency, United States Corps of Engineers and proactive stormwater management localities including but not limited to the states of Delaware, Florida, Maryland, Washington and local or regional agencies such as the Denver Urban Drainage and Flood Control District, the city of Washington DC, the city of Portland Oregon and the city of Austin Texas. Research progressed by the American Society of Civil Engineers Environment and Water Resource Institute (ASCE EWRI) also has been utilised. Information was gleaned from several Australian Crown Research Centres (CRC) and university research, including the latest incarnation that addresses stormwater in a more holistic manner – the CRC for Water Sensitive Cities.

Content and differences

The *Waikato Stormwater Runoff Modelling Guideline* is based on Auckland Council's Technical Publication 108 (TP108) *Guidelines for stormwater runoff modelling in the Auckland Region* (Auckland Regional Council, 1999) and replaces use of this guideline in the Waikato Region.

The two new guidelines importantly reflect the local characteristics of the Waikato Region. Previously stormwater design was based on ARC's documents, which incorporated a number of assumptions relevant to Auckland and which facilitated its implementation at the time. The Waikato guidelines return to the roots of TR-55 for its runoff calculations, while incorporating recommendations from the ASCE EWRI's recommendations to adjust the storage computation. The Waikato guideline further returned to the basis of the TR-55 method and adopts different soil groups than the assumptions in TP108, while requiring use of site specific soil information. As a result, the application is more scientifically valid for use in the Waikato than the Auckland guidance that they replace.

The consequence is that generally more runoff volume must be addressed to manage stormwater from what has been historically occurring in the region. Due to differences between catchments and soils in Auckland and the Waikato, the Auckland runoff modelling method results in devices that often are under-sized for Waikato conditions, and hence are not meeting expected performance, which leads to potential adverse effects. The Waikato guidelines will provide for devices and stormwater management that are designed for the Waikato Region conditions.

Another important aspect of the guidelines is the ongoing effort to address stormwater as part of urban development (i.e. low impact design, water sensitive cities) and at source rather than incorporating stormwater after the urban landscape has been designed or something appended at the bottom of the cliff. A low impact design scoring matrix is included in the guideline that enables quantification of how much low impact design has been incorporated into an urban development.

A new volume control criteria is included (in addition to existing peak flow control and water quality treatment criteria); developments will need to be designed to retain (reuse or soak) the initial abstraction volume of runoff. This criteria is to help offset the effects of impervious areas. Also, sections have been included on managing stormwater runoff from industrial areas, rural residential areas, and on managing the effects of urban stormwater runoff on Waikato Regional Council administered drainage districts. A specific section has been included on retrofitting stormwater management devices into existing built up areas.

Consultation

Consultation was undertaken in the development of this guideline, including:

- Internal consultation with Waikato Regional Council staff.
- Targeted workshops with territorial authorities.

- External workshops with key stakeholders including Iwi, territorial authorities, New Zealand Transport Agency, consultants, major industry representatives, surveyors, developers and Engineering New Zealand (Waikato Branch).

Feedback and companion guidance

Waikato Regional Council welcomes feedback on these guidelines as they are used in practice. As any guideline, amendments will be made to designs as new research and practice observation emerges that merit revisiting aspects within these guidelines.

Hard copies will not be sold or officially issued. It is the responsibility of the user of this guideline to ensure they download the most up-to-date version of the *Waikato Stormwater Runoff Modelling Guideline*.

The two new guidelines are among a series of best practice that Waikato Regional Council has published:

Principal Waikato Regional Council stormwater and related companion guidelines and documents:

- Waikato stormwater management guideline (TR2020/07).
- Waikato stormwater runoff modelling guideline (TR2020/06).
- Erosion and sediment control guidelines for soil disturbing activities (TR2009/02).
- Managing land use change and Council's administered drainage areas (TR2014/13).
- Environment Waikato best practice guidelines for waterway crossings (TR06/25R).

Acknowledgements

Most of the information contained in this guideline has been obtained from the following three sources:

- Auckland Regional Council, 1999, Guidelines for Stormwater Runoff Modelling in the Auckland Region, Technical Publication No. 108.
- Natural Resources Conservation Service (previously the Soil Conservation Service), 1986, Urban Hydrology for Small Watersheds, United States Department of Agriculture, Technical Release 55.
- Natural Resources Conservation Service, 2009, Part 630 Hydrology National Engineering Handbook, United States Department of Agriculture.

Waikato Regional Council would like to thank Graham Levy from Beca for review comments that were received and that have been incorporated into this guideline.

Useful comments have also been received from Matthew Lillis (Hamilton City Council), Scott King (AECOM), Britta Jensen (Opus) and Iain Smith (Beca).

Waikato Regional Council would like to thank all of those who attended the first stakeholder workshop held on 28 February 2017 and who provided input to both guidelines. Representatives attended from Hamilton City Council, Hauraki District Council, Matamata-Piako District Council, South Waikato District Council, Taupo District Council, New Zealand Transport Agency, Fonterra, AECOM, Beca, BBO, Civil Plan, CKL, Opus, Stantec, Tonkin & Taylor, Wainui Environmental and Stormwater 360.

Waikato Regional Council would like to thank the following for their attendance at a targeted territorial authority workshop on 22 February 2018 to discuss the key changes in the approach from previous Auckland Council guidance that has been used in the Waikato. The following people attended this workshop Andrea Phillips (Hamilton City Council), Matthew Lillis (Hamilton City Council), Mark Marr (Hamilton City Council), Sarah Pitches (Waipa District Council), Richard Pullar (Waikato District Council) and Scott Wilson (Opus – on behalf of Waikato District Council).

Waikato Regional Council would like to thank those who attended the two further workshops held on 2 May and 10 May 2018 and for the feedback provided at both workshops. These two workshops were attended by representatives from Hamilton City Council, Hauraki District Council, Matamata Piako District Council, South Waikato District Council, Taupo District Council, Thames Coromandel District Council, Waikato District Council, Waipa District Council, New Zealand Transport Agency, Engineering New Zealand Waikato Branch, Fonterra, AECOM, Beca, Bloxam Burnett & Olliver, Blue Wallace, BTW Company, Cheal Consultants, CKL, Gray Consulting, Harrison Grierson, Hartland Environmental, Key Solutions Ltd, Lewis Consulting Ltd, Opus, PF Olsen NZ, Stantec, Stormwater 360, Tonkin and Taylor, Civil Plan, Wainui Environmental and Waikato Regional Council.

May 2020 Edition

Waikato Regional Council would like to thank all those that provided feedback on the first version of this guideline, and those that attended the three Territorial Authority workshops held in March and May 2019.

Table of Contents

Figures

Tables

1 Background

Implementation of good stormwater management depends on having good design guidelines for the individual stormwater management devices in place. However, it is also dependent on having a robust hydrological design approach that is reasonable to use and generally accepted by the engineering community. There is also value in having a common hydrologic design approach used by the design community ensuring consistency throughout a catchment and the region.

Waikato Regional Council historically referred parties to the Auckland Regional Council's Technical Publication No.108 "*Guidelines for Stormwater Runoff Modelling in the Auckland Region*" (TP108) for a methodology for stormwater runoff modelling in the Waikato Region. However, Waikato Regional Council has now determined that it would be preferable to have its own stormwater runoff modelling guideline for use in the Waikato.

The Waikato Stormwater Runoff Modelling Guideline provides a standalone hydrological design guideline for the region. It is based, in large part, upon the historic TP108 guideline, which was in turn based on a hydrologic design approach developed in the United States. This guideline includes updates in the methodology from the United States and also specific changes that ensure the methodology is applicable to the Waikato Region.

If the methodology outlined in this guideline is used then practitioners will be able to demonstrate compliance with regulatory requirements rather than using a hydrologic design approach that the council either does not accept or is not familiar with.

If a practitioner wants to use an alternative methodology, there is scope for Waikato Regional Council to consider the proposed method. However, the consultant must be able to demonstrate that it is robust and provides comparable outputs to the approach outlined in this guideline.

The Waikato Stormwater Runoff Modelling Guideline is a companion guideline for the Waikato Stormwater Management Guideline[3].

1.1 Basis for the design approach

This guideline presents a recommended use of the U.S. Soil Conservation Service rainfall-runoff model for stormwater management purposes in the Waikato Region. It is based on Technical Release No. 55 (TR55)[4] prepared by the U.S. Soil Conservation Service, which is now known as the Natural Resources Conservation Service (NRCS) and is referred to as NRCS in this guideline.

The rainfall-runoff model outlined in the Waikato Stormwater Runoff Modelling Guideline is intended for use in the Waikato Region for stormwater management design to provide a standard tool that will ensure consistent results from different practitioners. The model is suitable for:

- Assessing the effects of land use change
- Modelling both frequent and extreme events
- Applying to distributed (a network of sub-catchments) or lumped catchments, and
- Simulating natural systems as well as engineered systems (such as pipe networks).

The model can be applied using a number of available software packages to predict runoff volumes, flow rates and the timing of peak flows. Peak flow rates can also be estimated using an alternative compatible graphical method included in this guideline.

[3] Waikato Regional Council, 2020
[4] National Resource Conservation Service, 1986

It is emphasised that the model has not been verified in Waikato catchments as per what was undertaken in the Auckland Region to inform the development of TP108.

This modelling approach is considered to provide reasonable estimates of the difference between pre and post-development peak flows and runoff volumes, to enable the design of stormwater management systems.

1.2 Hydrological modelling software

There are various computer software packages that can be used to assist with assessing hydrological conditions to inform the design of stormwater management systems for proposed developments.

HEC-HMS is a hydrologic modelling system produced by the US Army Corps of Engineers that is designed to simulate the complete hydrologic processes of watershed systems. This software package is compatible for use with the modelling approach outlined in this guideline.

There are other hydrologic modelling packages that are also suitable. If you are unsure if what you are proposing to use is suitable, contact Waikato Regional Council.

2 Analytical overview

The model begins with a rainfall amount uniformly imposed on the catchment over a specified time distribution. Mass rainfall is converted to mass runoff by using a runoff curve number (CN), which is based on soils, ground cover, the amount of impervious area, interception and surface storage. Runoff is then transformed into a hydrograph by using unit hydrograph theory and routing procedures that depend on runoff travel time through segments of the catchment.

Key features of the model are as follows:

- Design 24-hour rainfall depths are derived using the National Institute of Water and Atmospheric Research (NIWA) High Intensity Rainfall Design System (HIRDS) using the most recent version that is available[5].
- Climate change must be accounted for in the post-development calculations to determine storage requirements for stormwater management devices.
- A standard 24-hour temporal rainfall pattern, having peak rainfall intensity at mid-duration. Shorter duration rainfall bursts with a range of durations from 10 minutes to 24 hours are nested within the 24-hour temporal pattern as shown in Figure 2-1 below.

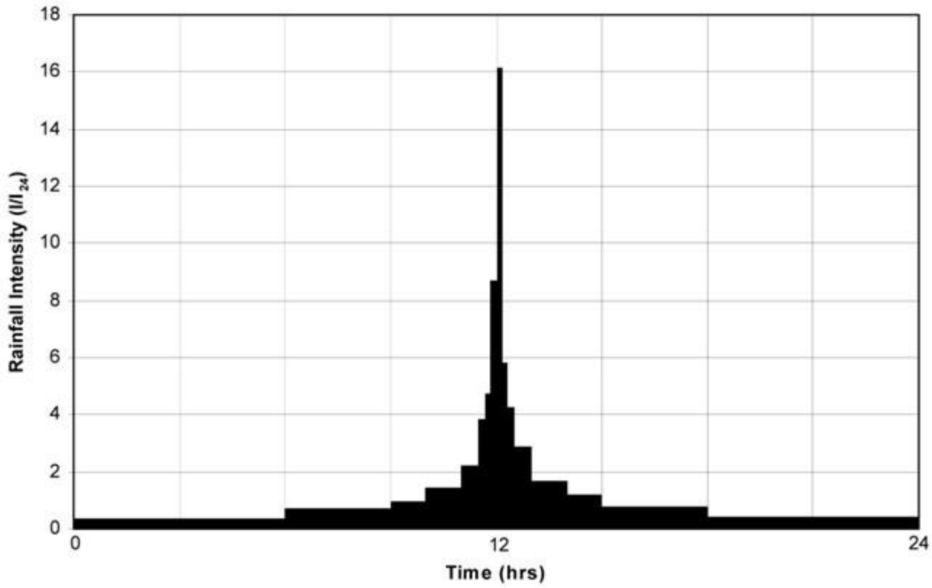


Figure 2-1: Temporal rainfall pattern[6]

- Rainfall runoff depth is calculated using NRCS rainfall-runoff curves, with curve number determined from NRCS guidelines.
- A runoff hydrograph is calculated using the standard NRCS synthetic unit hydrograph as shown in Figure 2-2 below.

[5] https://hirds.niwa.co.nz/
[6] Auckland Regional Council, 1999

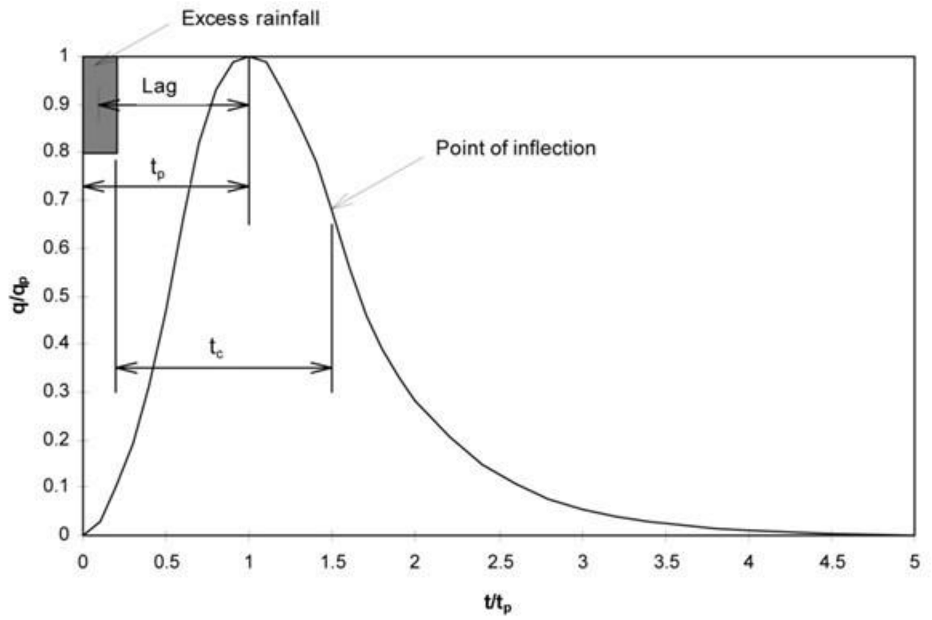


Figure 2-2: NRCS unit hydrograph[7]

- Time of concentration is estimated using equations and nomographs presented in New Zealand specific guidelines[8].
- Separate analysis of pervious and impervious components of urban catchments is undertaken for calculation of runoff volumes.
- Effects of development on runoff depth are predicted using the standard NRCS guidelines. Specific flow rates are still determined using Auckland Council recommendations (shown as Figure 8-1 in these guidelines), which may change once locally developed criteria are developed.

[7] National Resource Conservation Service, 1986
[8] Building Industry Authority, 2002

3 Limitations of the model

The procedures of the model have been simplified by assumptions about some parameters. These simplifications, however, limit the use of the procedures and can provide results that are less accurate than more detailed methods. The user should examine the sensitivity of the analysis to a variation of the peak discharge or hydrograph to ensure that any errors are tolerable.

Other constraints are the following:

- The methods of the model are based on open and unconfined flow overland or in channels. For large events during which flow is divided between reticulated and overland flow, more information about hydraulics is needed to determine time of concentration (t_c). After flow enters a closed system, the discharge can be assumed to be constant until another flow is encountered at a junction or another inlet.
- Rainfall data is attained from NIWA's High Intensity Rainfall Design System (HIRDS) to approximate local rainfall using the most current version of HIRDS that is available at the time.
- Soil types should be based on site soils testing. Section 5.3 of this guideline describes four hydrologic soil classifications and has applicable hydraulic conductivity limits for each soil classification. On-site soil testing will identify which soil classification is appropriate for each site based on the parameters discussed in Section 5.3.
- The model is applicable to both rural and urban catchments. Parameters for land cover types have been provided based on the standard NRCS guidelines.
- The model has been prepared as a standard tool for converting a design rainfall depth into a design runoff event of the same exceedance frequency. The model has not been validated for the Waikato Region and the absolute accuracy of the method is unknown (this is similar to other hydrologic models for which outputs have not been locally verified).
- The model accuracy for historical flood events simulated from historical storms will be dependent on the antecedent ground conditions and spatial rainfall variation. Antecedent ground conditions are variable, depending on the season and the timing of the storm with the sequence of storms. If this type of information is required, it is recommended to re-calibrate the model parameters (i.e. curve numbers) from nearby gauged catchments for the particular storm to estimate the spatial rainfall distribution from nearby rain gauges.

4 Rainfall data

The design rainfall is calculated from a standard 24-hour temporal pattern and an estimate of the design 24-hour rainfall depth.

4.1 Temporal pattern

The temporal pattern (shown in Figure 2-1) was derived from an analysis of depth-duration-frequency data from long-term rainfall records representative of the Auckland Region. Design rainfall bursts with a range of durations up to 24 hours were nested within a 24-hour storm, which was then normalised by the 24-hour rainfall depth.

The design storm indices, presented in terms of normalised rainfall intensity (I/I_{24}) for the Auckland Region are presented in Table 4-1 below.

Table 4-1: Normalised 24 hour design storm[9]

Time (hours:mins)	Time interval (mins)	Normalised rainfall intensity (I/I_{24})
0:00 -	360	0.34
6:00 -	180	0.74
9:00 -	60	0.96
10:00 -	60	1.4
11:00 -	30	2.2
11:30 -	10	3.8
11:40 -	10	4.8
11:50 -	10	8.7
12:00 -	10	16.2
12:10 -	10	5.9
12:20 -	10	4.2
12:30 -	30	2.9
13:00 -	60	1.7
14:00 -	60	1.2
15:00 -	180	0.75
18:00 – 24:00	360	0.40

It is recognised that there will be errors associated with using this normalised design storm however developing normalised storms for a variety of locations within the Waikato Region has not yet been undertaken. It can be developed for specific sites once 24-hour rainfall depths have been defined using HIRDS. These values have to be converted into a form that allows a balanced storm to be developed in Graphical HEC or HEC-HMS. For both of these software packages a normalised depth-duration-frequency (DDF) relationship can be multiplied by the 24-hour total depths to provide duration-depth data.

Developing site specific DDF tables or curves does require a higher level of understanding of the nested storm approach and should only be undertaken by experienced practitioners.

[9] Auckland Regional Council, 1999

4.2 Design rainfall depth

Design 24-hour rainfall depths are derived using NIWA's HIRDS web-based system using the current version of HIRDS that is available.[10] The case studies in this guideline have used HIRDS Version 3 rainfall data.

4.3 Climate change

The climate is changing. While climate change is a natural process, increased greenhouse gas concentrations are projected to exacerbate the drivers of our climate in ways that may be irreversible. Even if significant global action is taken now to reduce greenhouse gas concentrations, a degree of climate change is inevitable in our lifetime.

The Resource Management (Energy and Climate Change) Amendment Act 2004 requires councils to have particular regard to the effects of climate change.

WRC's Regional Policy Statement provides a basis for planning for and undertaking climate change adaptation actions. The Regional Policy Statement acknowledges the need to manage natural hazards such as flooding landslides and large scale rock/soil mass movements, severe weather events, drought and fire. Climate change will increase the risk from these hazards and make their management even more important. The key policies within the Regional Policy Statement relevant to climate change adaptation are contained below:

> *The effects of climate change (including climate variability) may impact our ability to provide for our wellbeing, including health and safety. While addressing this issue generally, specific focus should be directed to the following matters:*
>
> *a) Increased potential for storm damage and weather-related natural hazards; and*
> *b) Long term risks of sea level rise to settlements and infrastructure such as through increased coastal flooding and erosion.*
>
> *3.6 Adapting to climate change land use is managed to avoid the potential adverse effects of climate change induced weather variability and sea level rise on:*
>
> *a) Amenity;*
> *b) The built environment, including infrastructure;*
> *c) Indigenous biodiversity;*
> *d) Natural character;*
> *e) Public health and safety; and*
> *f) Public access.*
>
> *4.1.13 Incorporating effects of climate change*
> *Local authorities should, and regional and district plans shall, recognise and provide for the projected effects of climate change, having particular regard to:*
>
> *a) Historic long-term local climate data;*
> *b) Projected increase in rainfall intensity, taking account of the most recent national guidance and assuming a minimum increase in temperature of 2.1°C by 2090 (relative to 1990 levels); and*
> *c) Projected increase in sea level, taking into account the most recent national guidance and assuming a minimum increase in sea level of 0.8m by 2090 (relative to 1990 levels).*

Note that 4.1.13 b) and c) are minimum values and the most current guidance on projected temperature and sea level rise shall be used.

The nature of climate change data is that it is being regularly updated and hence climate change guidance is being regularly updated. For the purposes of stormwater design, practitioners are

[10] https://hirds.niwa.co.nz/

directed to use the most up to date Ministry for the Environment climate change guidance, which can be found at the following website:

http://www.mfe.govt.nz/climate-change/climate-change-resources/guidance-local-government

The following lists the MfE guidance that can be found at the above listed website:

Climate change effects and impacts assessment

- Climate change effects and impacts assessment: A guidance manual for local government in New Zealand
- Climate change projections for New Zealand snapshot
- Climate Change projections for New Zealand

Coastal hazards and climate change

- Coastal hazards and climate change: A guidance manual for local government in New Zealand
- Preparing for coastal change: A guide for local government in New Zealand (summary publication)

Tools for estimating the effects of climate change on flood flow

- Tools for estimating the effects of climate change on flood flow: A guidance manual for local government in New Zealand
- Preparing for future flooding: A guide for local government in New Zealand (summary publication)

Waikato Regional Council has prepared a Climate Change Guideline[11] to assist internal regional council staff in planning for climate change in relation to the many and varied operational activities delivered by the regional council. This guideline also caters to a wider audience, including those external to the regional council. Refer to this guideline for advice on how to incorporate climate change into your development proposal. This guideline is available on the Waikato Regional Council website.

Incorporating climate change predictions into stormwater design is important if infrastructure is to maintain the same level of service throughout its lifetime, and to ensure that development occurs in areas that will not be subject to future flood risk. Climate change is occurring now but predicted temperature increases are what is expected to occur sometime in the future. As a result for stormwater design, **pre-development rainfall data should not be adjusted for climate change while post-development rainfall data should be adjusted for climate change.** In the current version of HIRDS Version 4, the 'historic' data represents the existing/pre-development rainfall data.

[11] https://www.waikatoregion.govt.nz/assets/WRC/Services/publications/other-publications/Climate-Change-Guideline-ICM-FINAL-Sept-2017.pdf

4.4 Areal reduction factors

Areal reduction factors are used to apply point estimates of rainfall to large catchments for event durations of 24 hours or less and areas greater than 20 km^2. No reduction factor should be applied for catchments that are smaller than 20 km^2.

When both the above criteria relating to durations and area are met, it is recommended that the following equation and associated parameters be used to derive areal reduction factors.[12]

$$ARF = 1 - 0.023\, A^{0.43} D^{-0.52} (\, lnT)^{0.23}$$ **Equation 4-1**

Where ARF = areal reduction factor
A = area in km^2
D = duration in hours
T = return period in years

[12] Carey-Smith et al, August 2018

5 Estimating runoff

The NRCS runoff curve number (CN) method is[13]:

$$\boldsymbol{Q = \frac{(P - I_a)^2}{(P - I_a) + S}}$$ **Equation 5-1**

Where:
Q = runoff depth (mm)
P = rainfall depth (mm)
S = potential maximum retention after runoff begins (mm)
I_a = initial abstraction (mm)

Rainfall losses relate to two parameters:

- Initial abstraction, and
- Runoff curve numbers.

5.1 Retention parameters

Several retention parameters are used in developing a relationship of rainfall to runoff. The initial abstraction (I_a) is considered as the boundary between the storm size that produces runoff and the storm size that does not produce runoff. The soil storage parameter (S) is dependent upon the soil-cover complex and, in principle, should not vary from storm to storm. It is in excess of the initial abstraction so that the maximum possible loss is given by $I_a + S$[14].

The soil storage parameter relates to soil and land use conditions of the catchment through the curve number, CN, and are detailed below.

Soil storage parameter (S)

$$\boldsymbol{S = \left(\frac{1000}{CN} - 10\right) 25.4 \quad (mm)}$$ **Equation 5-2**

Where S = potential maximum retention after runoff begins (mm)

CN ranges from 0 for zero runoff to 100 for total runoff.

Initial abstraction (I_a)

The relationship between S and I_a is described using the following equation:

$$\boldsymbol{I_a = 0.05S}$$ **Equation 5-3**

This equation, in conjunction with solving for *S,* provides the initial abstraction for use in calculating runoff. It should be noted that Equation 5-3 has been modified from the equation in the original NRCS version[15].

Table 5-1 below provides an estimation of I_a for various curve numbers that clearly demonstrates the retention capability of highly permeable soils and the impacts that increased imperviousness and less permeable soils can have on downstream flows.

[13] National Resources Conservation Service, 1986
[14] National Resources Conservation Service, 1985
[15] ASCE/EWRI – ASABE Curve Number Task Force 2017

Table 5-1: Initial abstraction values for runoff curve numbers

Curve number	I_a (mm)	Curve number	I_a (mm)	Curve number	I_a (mm)
40	19.0	60	8.5	80	3.2
41	18.3	61	8.1	81	3.0
42	17.5	62	7.8	82	2.8
43	16.8	63	7.5	83	2.6
44	16.2	64	7.1	84	2.4
45	15.5	65	6.8	85	2.2
46	14.9	66	6.5	86	2.1
47	14.3	67	6.3	87	1.9
48	13.8	68	6.0	88	1.7
49	13.2	69	5.7	89	1.6
50	12.7	70	5.4	90	1.4
51	12.2	71	5.2	91	1.3
52	11.7	72	4.9	92	1.1
53	11.3	73	4.7	93	1.0
54	10.8	74	4.5	94	0.8
55	10.4	75	4.2	95	0.7
56	10.0	76	4.0	96	0.5
57	9.6	77	3.8	97	0.4
58	9.2	78	3.6	98	0.3
59	8.8	79	3.4		

This Waikato Stormwater Runoff Modelling Guideline is to be used in conjunction with the Waikato Stormwater Management Guideline with particular reference to Section 7.2.7 of the latter guideline.

This section states that retention is to be provided for the depth of runoff equivalent to the initial abstraction for the site's impervious areas to offset the loss of the initial abstraction that un-compacted pre-development pervious areas had. If soil remediation is not provided for pervious areas that have been earthworked, then the initial abstraction of runoff from the entire site should be retained. In those situations, the pervious areas shall assume a reduction in ground permeability from the pre-development permeability. The soil group classification should be reduced by one classification (for example Group A to B, Group B to C and Group C to D).

5.2 Curve numbers

The major factors that determine CN are the hydrologic soil group (HSG), ground cover type, treatment, hydrologic condition and antecedent runoff condition. Table 5-2 below presents the most commonly used curve numbers for urban and rural catchments. Curve numbers for catchments in the Waikato Region should be selected using Table 5-2 according to the values presented below.

Appendix A provides an expanded list of curve numbers for land uses beyond those provided in Table 5-2.

Table 5-2: Runoff curve numbers for most urban and rural lands[16]

Cover description					
Cover type and hydrologic condition	**Hydrologic condition**	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, reserves, etc.)					
Condition (grass cover < 50%)	Poor	68	79	86	89
Fair condition (grass cover 50%-75%)	Fair	49	69	79	84
Good condition (grass cover >75%)	Good	39	61	74	80
Impervious areas					
Paved parking lots, roofs, driveways, etc.		98	98	98	98
Streets and roads*					
Paved; kerbing and catchpits (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Pasture, grassland, or range – continuous forage for grazing	Poor Fair Good	68 49 39	79 69 61	86 79 74	89 84 80
Straight row crops	Poor Good	72 67	81 78	88 85	91 89+
Bush – bush-weed-grass mixture with bush being the major element	Poor Fair Good	48 35 30	67 56 48	77 70 65	83 77 73
Bush – grass combination (orchard or tree farm)	Poor Fair Good	57 43 32	73 65 58	82 76 72	86 82 79
Bush**	Poor Fair Good	45 36 30	66 60 55	77 73 70	83 79 77
Farmsteads – buildings, lanes, driveways, and surrounding lots		59	74	82	86

* As calculations for runoff volumes are undertaken separately for pervious and impervious areas, the generalised curve numbers incorporating pervious and impervious surfaces provided by NRCS are not included in the table.

Some of the cultivated agricultural land categories are not included and Table 2-2b of Technical Release No. 55 should be referred to which is located in Appendix A.

** Bush condition:

Poor: forest litter, small trees, and bush are destroyed by heavy grazing or regular burning
Fair: woods are grazed but not burned, and some forest litter covers the soil
Good: woods are protected from grazing, and litter and bush adequately cover the soil

[16] Natural Resources Conservation Service, 1986

Runoff from catchments with a fair mix of soil or land use types can be modelled using an area-weighted curve number if the catchment is homogeneous. A homogeneous catchment is defined as a catchment where all areas drain through common flow paths.

Where a catchment contains a significant impervious component connected to a pipe network, the catchment should be considered heterogeneous. Heterogeneous catchments should be modelled by division into separate homogeneous sub-catchments connected by hydraulic elements. The weighted curve number for a homogeneous catchment should be calculated as:

$$\boldsymbol{CN = \frac{\sum CN_i A_i}{A_{tot}}}$$

Equation 5-4

There are limitations to using the above general equation for an entire catchment using a general curve number.

- Curve numbers describe average conditions that are useful for design purposes. If the rainfall event used is a historical storm, the modelling accuracy decreases.
- Use the runoff curve number equation with caution when recreating specific features of an actual storm. The equation does not contain an expression for time and, does not account for rainfall duration or intensity.
- The NRCS runoff procedures apply only to direct surface runoff. They do not consider large sources of subsurface flow or high ground water levels that contribute significant runoff. These conditions generally apply to HSG A soils and forest areas that have low CNs. Good judgement and experience based on stream gauge records are needed to adjust CNs as conditions warrant.

5.3 Hydrological soil groups

As discussed above, the hydrological soil group is used to determine the curve number. The hydrological soil groups (HSGs) are described as follows[17]:

<u>Group A soils</u>

Soils in this group have low runoff potential when thoroughly wet. Water is transmitted freely through the soil. Group A soils typically have less than 10% clay and more than 90% sand or gravel and have gravel or sand textures. Some soils having loamy sand, sandy loam, loam or silt loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35% rock fragments.

The limits on the diagnostic physical characteristics of Group A are as follows:

- The saturated infiltration rate of all soil layers exceeds 40.0 micrometres/second (µm/s).
- The depth to any water impermeable layer is greater than 0.5 m.
- The depth to the water table is greater than 0.5 m
- Soils that are deeper than 1 m to a water impermeable layer and a water table are in Group A if the saturated infiltration rate of all soil layers within 1 m of the surface exceeds 10 µm/s.

<u>Group B soils</u>

Soils in this group have moderately low runoff potential when thoroughly wet. Water transmission through the soil is unimpeded. Group B soils typically have between 10% and 20% clay and 50% to 90% sand and have loamy sand or sandy loam textures. Some soils having loam, silt loam, silt, or sandy clay loam textures may be placed in this group if they are well aggregated, of low bulk density or contain greater than 35% rock fragments.

[17] Natural Resources Conservation Service, 2009

The limits on the diagnostic physical characteristics of Group B are as follows:

- The saturated infiltration rate in the least transmissive layer between the surface and 0.5 m ranges from 10 to 40.0 µm/s.
- The depth to any water impermeable layer is greater than 0.5 m.
- The depth to the water table is greater than 0.5 m
- Soils that are deeper than 1 m to a water impermeable layer and a water table are in Group B if the saturated infiltration rate of all soil layers within 1 m of the surface exceeds 4.0 µm/s but is less than 10 µm/s.

<u>Group C soils</u>

Soils in this group have moderately high runoff potential when thoroughly wet. Water transmission through the soil is somewhat restricted. Group C soils typically have between 20% and 40% clay and less than 50% sand and have loam, silt loam, sandy clay loam, clay loam and silty clay loam textures. Some soils having clay, silty clay or sandy clay textures may be placed in this group if they are well aggregated, of low bulk density or contain greater than 35% rock fragments.

The limits on the diagnostic physical characteristics of Group C are as follows:

- The saturated infiltration rate in the least transmissive layer between the surface and 0.5 m is between 1.0 and 10.0 µm/s.
- The depth to any water impermeable layer is greater than 0.5 m
- The depth to the water table is greater than 0.5 m
- Soils that are deeper than 1 m to a restriction and a water table are in Group C if the saturated infiltration rate of all soil layers within 1 m of the surface exceeds 0.40 µm/s but is less than 4.0 µm/s.

<u>Group D soils</u>

Soils in this group have high runoff potential when thoroughly wet. Water movement through the soil is restricted or very restricted. Group D soils typically have greater than 40% clay, less than 50% sand and have clayey textures. In some areas, they also have high shrink-swell potential. All soils with a depth to a water impermeable layer less than 0.5 m and all soils with a water table also within 0.5 m of the surface are in this group, although some may have a dual classification if they can be adequately drained.

The limits on the physical diagnostic characteristics of Group D are as follows:

- For soils with a water impermeable layer at a depth between 0.5 m and 1 m, the saturated infiltration rate in the least transmissive soil layer is less than or equal to 1.0 µm/s.
- For soils that are deeper than 1 m to a restriction or water table, the saturated infiltration rate of all soil layers within 1 m of the surface is less than or equal to 0.40 µm/s.

As an aside, the saturated vertical soil coefficient of permeability may be assumed to equal the soil infiltration rate[18].

Soils having water tables within 0.5 m of the surface can be classified as Type D soils without soil testing, assuming that these soils are not drained. Initial abstraction is then identified with the appropriate cover type.

[18] Infiltration Standards Review Committee, 2009

5.3.1 Dual classification

This classification is based on a method from NRCS[19]. Certain wet soils are placed in Group D based solely on the presence of a water table within 0.5 m of the surface even though the saturated hydraulic conductivity may be favourable for water transmission. If these soils can be adequately drained, then they are assigned to dual hydrologic soil groups (A/D, B/D, C/D) based on their saturated hydraulic conductivity and the water table depth when drained. The first letter applies to the drained condition and the second to the undrained condition. For the purpose of selecting the HSG, adequately drained means that the seasonal high-water table is kept at least 0.5 m below the surface in a soil where it would be higher in a natural state.

There are significant areas in the Waikato Region that are very flat and getting positive drainage to an outfall is difficult. As a result, rural drainage networks have been constructed to provide land drainage for agricultural purposes, to lower localised ground water levels and to reduce flooding potential. Many of these drainage networks have been formalised to constitute regional or district council administered drainage areas.

Drainage areas are where the use of dual classifications are most appropriate. In an undrained condition, the soils ability to transmit water is limited due to elevated groundwater levels. When drainage networks are constructed to lower the local groundwater levels and to facilitate drainage, the ability of the soil to transmit water may be enhanced (depending on the soils unsaturated ability to transmit water) and the dual soil classification can be used.

The dual classification of soils is also appropriate where a private drainage system has been, or is proposed to be, installed to lower groundwater levels.

5.3.2 Hydrologic soil group assignment

The decision matrix is provided in Table 5-3 can be used to determine a soil's HSG. If saturated hydraulic conductivity data is available and deemed reliable, then this data, as well as water table depth information, can be used to place the soil into the appropriate HSG. If this data is not available, the HSG is determined by observing the properties of the soil in the field. Factors such as texture, compaction (bulk density), strength of soil structure, clay mineralogy and organic matter are considered in estimating the hydraulic conductivity of any water impermeable layer and the depth to any high-water table are used to determine correct HSG for the soil. The property that is most limiting to water movement generally determines the soil's HSG.

[19] National Resources Conservation Services, 2009

Table 5-3: Assignment of hydrologic soil group[19]

Depth to water impermeable layer[1]	Depth to high water table[2]	K_{sat} of least transmissive layer in depth range	K_{sat} depth range	HSG[3]
< 0.5 m	-	-	-	D
0.5 m to 1 m	< 0.5 m	> 40.0 µm/s	0 to 0.5 m	A/D
		> 10.0 to ≤ 40.0 µm/s	0 to 0.5 m	B/D
		. 1.0 to ≤ 10.0 µm/s	0 to 0.6 m	C/D
		≤ 1.0 µm/s	0 to 0.6 m	D
	≥ 0.5 m	> 40 µm/s	0 to 0.5 m	A
		> 10.0 to ≤ 40 µm/s	0 to 0.5 m	B
		> 1.0 to ≤ 10 µm/s	0 to 0.5 m	C
		≤ 1.0 µm/s	0 to 0.5 m	D
> 1 m	< 0.5 m	> 10 µm/s	0 to 1 m	A/D
		> 4.0 to ≤ 10.0 µm/s	0 to 1 m	B/D
		> 0.40 to ≤ 4.0 µm/s	0 to 1 m	C/D
		≤ 0.4 µm/s	0 to 1 m	D
	0.5 m to 1 m	> 40.0 µm/s	0 to 0.5 m	A
		> 10.0 to ≤ 40.0 µm/s	0 to 0.5 m	B
		> 1.0 to ≤ 10.0 µm/s	0 to 0.5 m	C
		≤ 1.0 µm/s	0 to 0.5 m	D
	> 1 m	> 10.0 µm/s	0 to 1 m	A
		> 4.0 to ≤ 10.0 µm/s	0 to 1 m	B
		> 0.40 to ≤ 4.0 µm/s	0 to 1 m	C
		≤ 0.40 µm/s	0 to 1 m	D

Notes:

1 An impermeable layer has a K_{sat} less than 0.01 µm/s or a component restriction of fragipan; duripan; petrocalcic; orstein; petrogypsic; cemented horizon; densic material; placic; bedrock, paralithic; bedrock, lithic; bedrock, densic; or permafrost.

2 High water table during any month during the year.

3 Dual HSG classes are applied only for wet soils (water table less than 0.5 m. If these soils can be drained, a less restrictive HSG can be assigned, depending on the K_{sat}.

The HSG can be determined through site soils testing. In that situation, the HSG should reflect the dominant scenario of soil properties and land use. The dataset developed is a snapshot in time and reflects the use and management of that site.

A resource for determining HSG is Landcare Research's ongoing project to map New Zealand's soil resources. Soil mapping in the Waikato Region is extensive with significant information on soil texture, permeability, depth class and drainage. This information can be viewed on the Landcare Research website[20].

It is expected that soil testing will be undertaken at every development site to inform classification of soil group, this will then inform selection of the curve numbers relevant to the required analysis.

[20] https://smap.landcareresearch.co.nz/

5.3.3 Site soil testing to determining hydrologic soil group

HSGs are essential in determining stormwater runoff rates and volumes. As such, there is a need for a methodology to associate site areas with an applicable HSG. The determination of an HSG is based on the following HSG testing procedures. The procedure outlined below can be used, or a testing method determined appropriate by a qualified geotechnical professional.

- Sites less than 2,000 m^2: require as a minimum: one soil profile pit and one soil boring. The purpose of each soil profile pit is to establish detailed information on groundwater conditions and soil morphology. Data recorded in each soil boring is then compared to the reference soil profile pit to confirm consistency between the profile pit and the boring. Where soil and/or groundwater properties vary significantly between soil boring and profile pit, additional soil profile pits shall be conducted as necessary to resolve such differences and accurately characterise the soil mapping.
- Sites from 2,000 m^2 to 1 hectare: require one soil profile pit and four soil borings.
- Where the HSG is to be determined for a mapping unit larger than one hectare within the limits of the overall site, a minimum of one additional soil profile pit and two additional soil borings shall be conducted for each additional hectare.
- All soil explorations shall be located generally equidistant from each other and the boundaries of the mapping to maximise the ability to interpolate between test locations so as to provide adequate characterisation of the soils. Figure 5-1 below shows conceptually where pits and boring should be located. In all cases, a soil profile pit may be conducted in place of a required soil boring; however, a soil boring cannot be used as a substitute for a soil profile pit except as stated below.
- In terms of obtaining the detailed information, the information can be developed as site analysis moves from conceptual to detailed design. This can result in a staged approach, where the use of available soils maps would suffice for outlining the concept design for the stormwater system or consideration of a site suitability report. Site testing would then follow as part of detailed design to refine concept design sizes.
- Where catchment areas extend beyond the developer's boundary, soil maps may be used to provide for runoff calculations if more detailed information does not exist.
- For large scale catchment management planning projects, soil maps may be used to provide high level runoff calculations.
- In areas where a soil profile pit would substantially disturb the existing area and create an undesirable condition or where significant environmental disturbance will occur in an area that is not intended for future development, two soil borings may be conducted in place of a required soil profile pit. A soil profile pit shall then be located at the closest available location representative of the soil boring locations[21].

[21] State of New Jersey, 2004 (revised 2016)

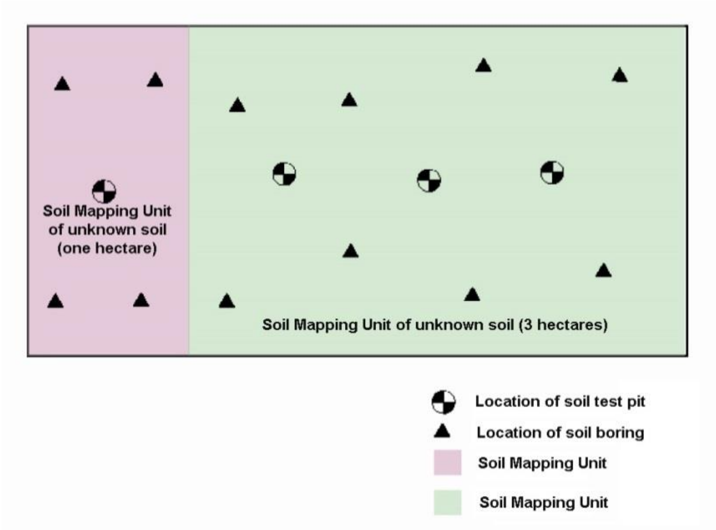


Figure 5-1: Indicative soil testing site arrangement

Where soil and/or groundwater properties vary significantly between soil explorations, additional soil profile pits shall be conducted as necessary to resolve such difference and accurately characterise the soils.

Soil profile pits and soil borings shall extend to the depth of the seasonal high-water table or the deeper of two metres below existing grade or one metre below proposed grade. The determination of the soils HSG is based upon the depth to restrictions (where soil morphological characteristics which restrict the vertical movement of water including but not limited to abrupt textural boundaries, fragipan, bedrock, dense or cemented soils).

The depth to the seasonally high-water table and the permeability rate of the most restrictive soil horizon above either the restriction or the seasonally high-water table shall also be determined.

5.3.3.1 Permeability testing procedures

There are a number of permeability testing methods that can be undertaken to determine HSGs. Any testing should be undertaken under the guidance of a qualified geotechnical professional to ensure that accurate results are obtained. Examples of tests include the following:

- Falling head permeability test or
- Constant head permeability test.

Falling head permeability testing is usually undertaken on fine grained soils while constant head permeability testing is undertaken on coarse grained soils.

5.3.3.2 Evaluation of testing results

In the event that all site soil characteristics are the same then a single HSG can be used for pre-development soils. If the results are markedly different, the area of the site for each different HSG shall be determined and the HSG in addition to the cover description shall be recorded to calculate pre-development peak discharges.

In a post-development condition, the analysis shall incorporate whether the soils are rehabilitated and whether fill material has been imported to the site and HSGs assigned to those soils if the permeability rates are less than the original soils on which the fill is placed.

5.4 Land use cover parameters

Curve numbers should be selected based on soil type and land use based on Table 5-2. If the appropriate land use is not listed in Table 5-2 then the NRCS publication TR 55[22] should be read and Table 2-2 of that publication reviewed for an appropriate CN (Table 2-2 from this NRCS guidance is provided in Appendix A).

Land use type should be assessed by field reconnaissance, aerial photographs or land use maps. Land use factors incorporated into these and the NRCS guidelines are:

- Cover type (type of vegetation or use),
- Soil treatment (management of cultivated lands), and
- Hydrologic condition (density of vegetation, surface roughness, etc.).

Impervious areas should be modelled with a curve number of 98 and zero initial abstraction. Impervious areas within homogeneous catchments can be allowed for by using area-weighted values for CN and I_a.

Impervious area should be measured from aerial photographs or by other methods. Note that the percent impervious values in this guideline and NRCS CNs shown in Appendix A were not developed for Waikato conditions and should not be relied upon.

For homogeneous catchments:

$$\boldsymbol{CN = \frac{98A_{imperv} + CN_{perv}A_{perv}}{A_{total}}}$$ **Equation 5-5**

Initial abstraction is calculated using the following equation:

$$\boldsymbol{I_a = 0.05\ x\ S\ (mm)}$$ **Equation 5-6**

Catchments containing significant impervious areas connected directly to a reticulated stormwater system should not be modelled as homogeneous because the impervious connected component will have a more rapid response time than the pervious component of the catchment.

This effect will be more marked in an urbanised catchment with volcanic soils. In such cases, a more realistic representation of the catchment may be obtained by modelling the connected impervious areas and pervious areas as separate sub-catchments. Time response for the respective sub-catchments will be different and should be calculated according to the procedure in the following section. Any unconnected impervious areas (i.e. those impervious areas draining onto pervious areas) should be included in the pervious sub-catchment.

[22] National Resources Conservation Service, 1986

6 Runoff calculations

The runoff depth (CN and rainfall from previous section) is converted to a catchment hydrograph using the dimensionless NRCS unit hydrograph.

6.1 Unit hydrograph

The NRCS unit hydrograph was developed by averaging dimensionless unit hydrographs from a number of natural catchments with little or no storage. Individual hydrographs were made dimensionless by dividing by peak flow rate, q_p and time to peak, t_p. The resulting dimensionless unit hydrograph was previously shown in Figure 2-2, which is repeated below for context with this discussion. The ordinates of this curve are shown in Table 6-1 below.

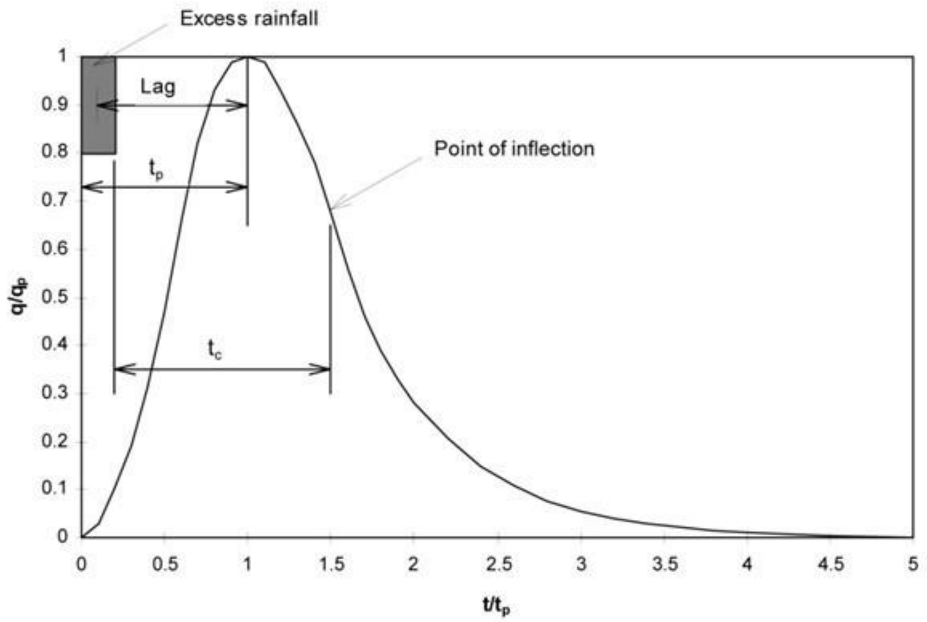


Figure 6-1:Figure 2-2: NRCS unit hydrograph (previously shown as Figure 2-2)[23]

The time to peak, t_p, of the NRCS unit hydrograph is shorter than the catchment time of concentration, t_c. The NRCS hydrograph is defined such that t_c is the time to the inflection point of the hydrograph recession limb. This leads to the following relationship[24]:

$$t_p = \frac{2}{3} t_c \qquad \textbf{Equation 6-1}$$

Various software packages require the user to enter either t_c or t_p in applying the NRCS unit hydrograph.

[23] National Resources Conservation Service, 1986
[24] McCuen, 1998

Table 6-1: NRCS unit hydrograph ordinates

t/t_p	q/q_p	t/t_p	q/q_p	t/t_p	q/q_p
0	0	1.1	0.99	2.4	0.147
0.1	0.03	1.2	0.93	2.6	0.107
0.2	0.10	1.3	0.86	2.8	0.077
0.3	0.19	1.4	0.78	3.0	0.055
0.4	0.31	1.5	0.68	3.2	0.040
0.5	0.47	1.6	0.56	3.4	0.029
0.6	0.66	1.7	0.46	3.6	0.021
0.7	0.82	1.8	0.39	3.8	0.015
0.8	0.93	1.9	0.33	4.0	0.011
0.9	0.99	2.0	0.28	4.5	0.005
1.0	1.00	2.2	0.207	5.0	0

The unit hydrograph is applied to a specific catchment by factoring it by the time to peak t_p, and the peak flow rate. The peak flow rate q_{ip}, from a short duration rainfall burst is related to the runoff depth of the burst Q_i by:

$$q_{ip} = k\frac{Q_i A}{t_p}$$ **Equation 6-2**

The standard NRCS unit hydrograph predicts 3/8 of the runoff depth under the rising limb. This corresponds to a coefficient in the above equation of k = 2(3/8) = ¾ if consistent units are used.

A hydrograph number of ¾ is recommended for the Waikato Region.

7 Time of concentration

Time of concentration (t_c) is the time required for runoff to travel from the hydraulically most distant point in a catchment to the outlet. The hydraulically most distant point is the point with the longest travel time to the catchment outlet and not necessarily the point with the longest flow path to the outlet. Time of concentration is generally applied only to surface runoff and may be calculated using many different methods.

The method suggested here relates to water moving through a catchment first as sheet and shallow concentrated flow, network flow and finally as open channel flow. In effect, the calculations for time of concentration are a summation of individual travel times by the various flows.

$$T_c = T_{t1} + T_{t2} + \cdots ..T_{tm}$$ **Equation 7-1**

T_c = Time of concentration (hours)
M = number of individual flow segments

To determine the time of concentration for a site it is necessary to assess the individual flow segments present at the site, i.e. the portion of the site that has sheet flow and the portion of the site that has network flow, be it road flow, piped flow or open channel flow.

Individual travel times need to be determined for each flow segment using the information provided in the following subsections. The travel times for each flow segment are then added together in accordance with Equation 7-1 above to determine the site time of concentration.

This assessment needs to be undertaken for pre- and post-development conditions at the site. When time of concentration is less than 10 minutes, the minimum value of 10 minutes should be used.

Variation to this requirement requires concurrence with council review staff.

7.1 Sheet and shallow concentrated flow

Sheet and shallow flow is usually found at the top of catchments. The travel time for sheet flow incorporates Manning's roughness coefficient (n) and an equation for sheet and shallow channel flow is provided below.

$$T_t = {100nL^{0.33}}/{S^{0.2}}$$ **Equation 7-2**

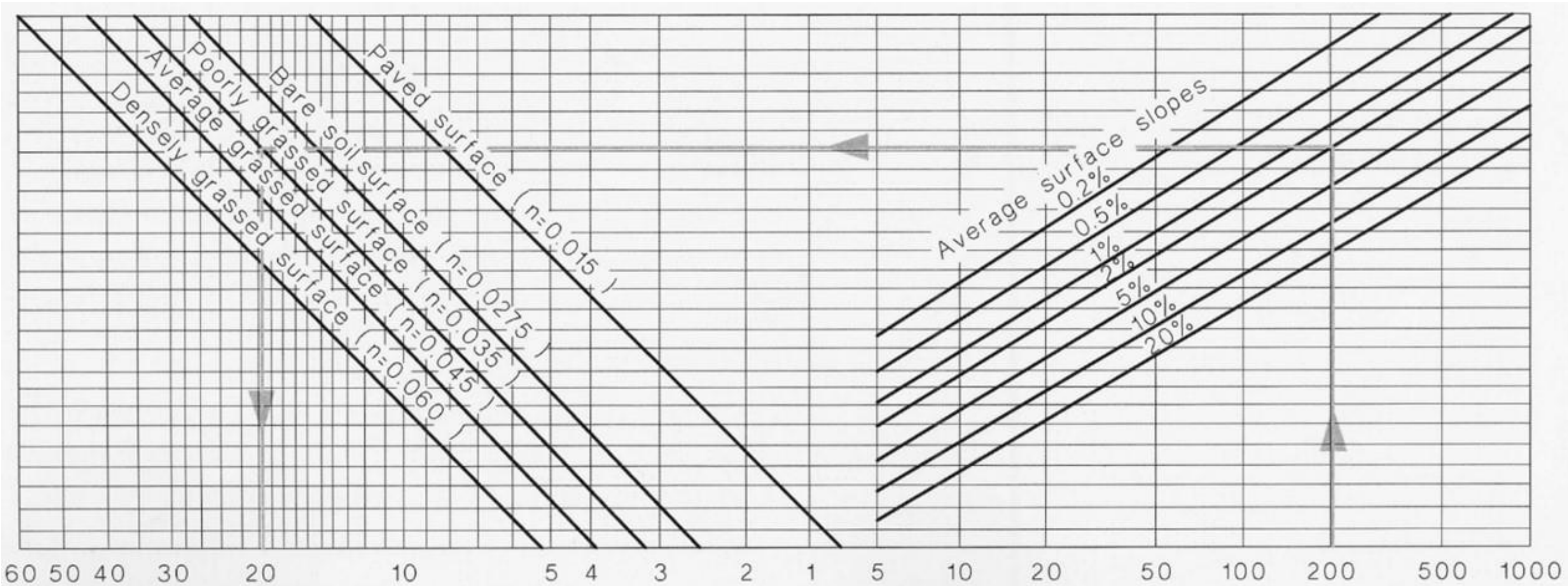
Where:
T_t = time in minutes
L = length of overland flow in metres
S = slope in %
n = Mannings value for surface roughness coefficient (typical values given in Table 7-1)

Table 7-1: Manning's *n* roughness values for overland flow[25]

Overland Surface	*n*
Paved surface	0.015
Bare soil	0.0275
Poorly grassed	0.035
Average grassed surface	0.045
Pasture	0.045
Mature bush	0.06
Dense grass	0.06
Shrubs and bushes	0.08

Figure 7-1 below provides a graphical means to calculate time of travel for overland flow.

[25] Yen and Chow, 2016



TIME OF TRAVEL OVER SURFACE (in minutes) **LENGTH OF OVERLAND FLOW (metres)**

Example: **For surface water flowing 200 m over an average grassed surface at a slope of 2% the time of travel is 20 minutes**

Figure 7-1: Nomograph for estimating overland sheet flow times[26]

[26] Building Industry Authority, 2002

For rural catchments or where input values vary from those shown in the nomograph, use the equation rather than the nomograph to calculate the travel time.

The designer must ensure that the flow travel time is calculated from the very top of the catchment.

7.2 Concentrated network flow

The time of network flow as another component of t_c is comprised of the time of road channel flow, pipe network flow and open channel flow.

7.2.1 Time of road channel flow

The time of road channel flow is the time taken for water to flow from the point of entry to the road channel to the point of discharge into a drain or other outlet. Flow times can be estimated using Figure 7-2 below.

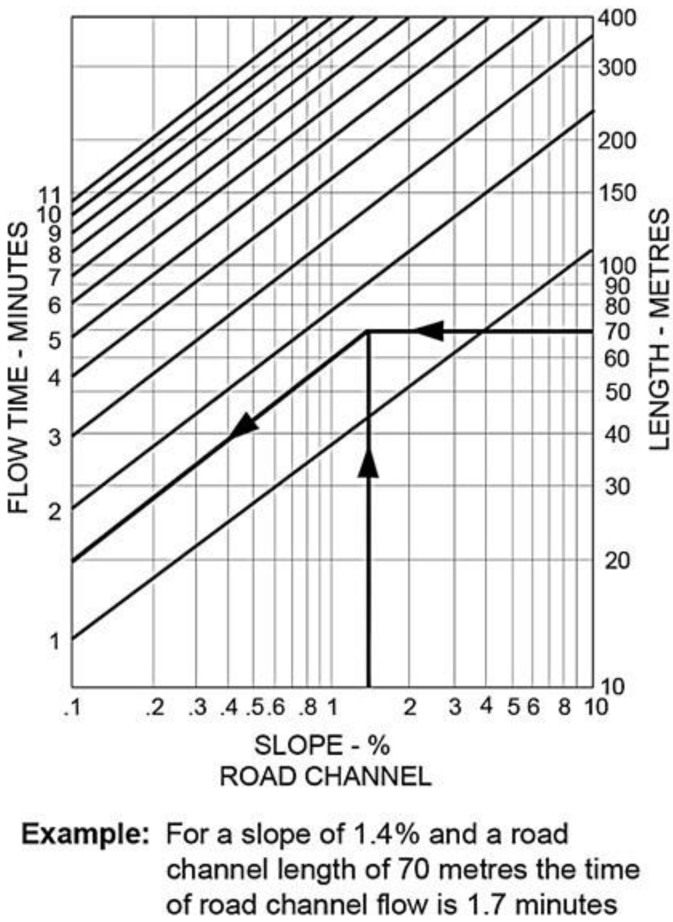


Figure 7-2: Road channel flow time[27]

---

[27] Building Industry Authority, 2002

7.2.2 Time of pipe network flow

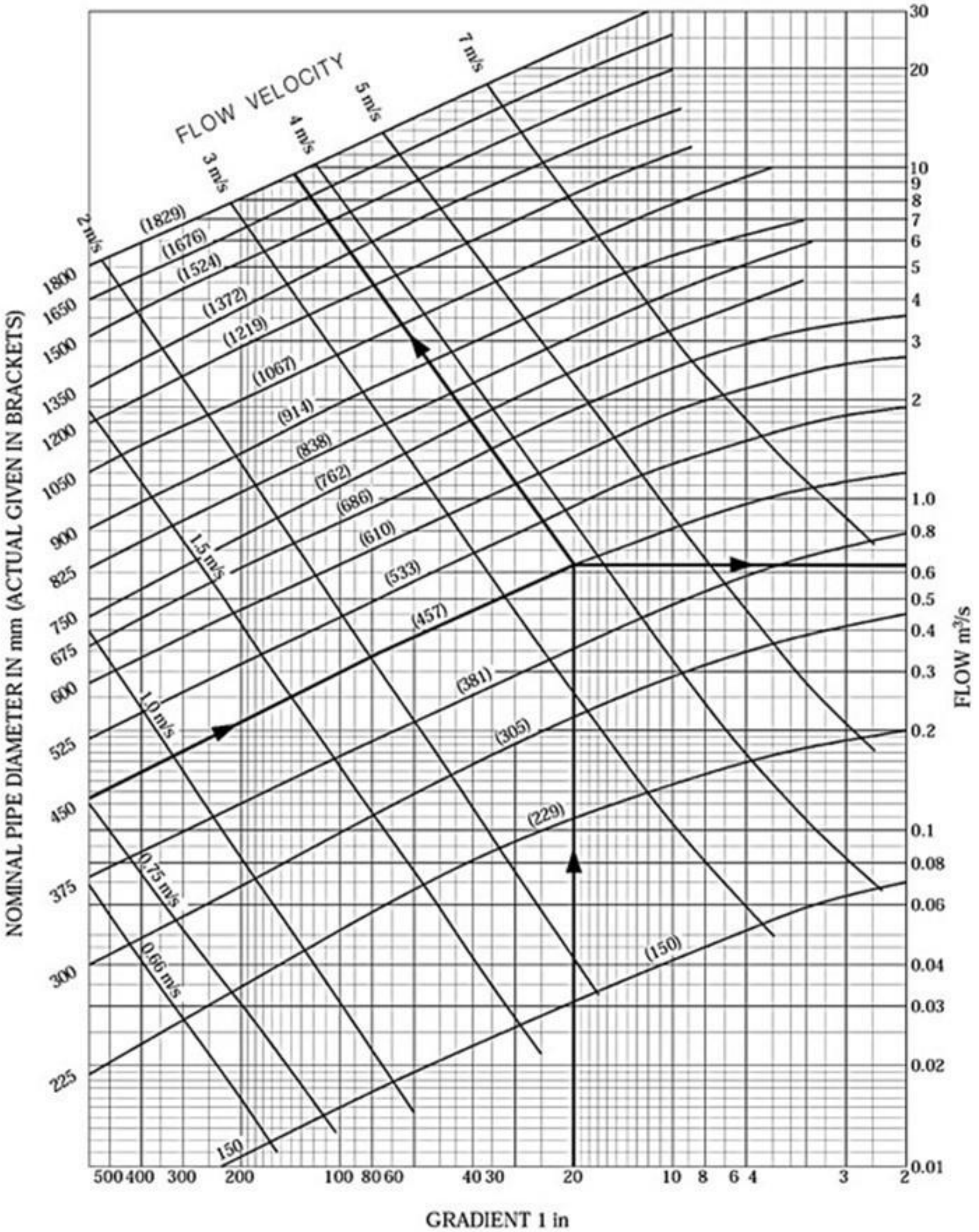
As longitudinal sections may not be available during site preliminary design, typical pipe flow velocities for different gradients are provided in Table 7-2 below. The time of network flow can be determined using this information.

Table 7-2: Typical pipe flow velocities for different gradients[28]

Gradient	Typical velocity (m/s)
Flat gradient	0.6
Moderate gradient	1.5
Steep gradient	3.0

A nomograph is provided in Figure 7-3 below to determine time of pipe flow for flow in a concrete pipe.

[28] Christchurch City Council, 2003



Example: a 450 mm internal diameter pipe with a gradient of 1 in 20 will have a flow of 0.63 m³/s at a velocity of 3.75 m/s

Figure 7-3: Pipe flow velocity determination[29]

Where the pipe changes material, diameter or gradient the time taken in each section of the pipe can be calculated and the component times combined.

For pipes with Manning's 'n' other than 0.013 (concrete) the velocity determined from Figure 7-3 can be multiplied by the ratio of 0.013/n. Other values of Manning's 'n' for different pipe materials should be based on manufacturer's recommendations.

[29] Building Industry Authority, 2002

7.2.3 Open channel flow

The time of flow in open channels can be determined by using the Manning equation to determine the velocity of flow. The Manning's equation is shown in the following equation:

$$V = R^{2/3}S^{1/2}/_{n}$$ **Equation 7-3**

Where:
V = mean velocity of flow (m/s)
S = the slope of the hydraulic energy gradient - normally can be considered as the channel slope (m/m)
R = hydraulic radius (m)
n = Manning's roughness coefficient

The time of flow in the open channel, can then be calculated based on the total length of the channel and the mean velocity of flow along the channel.

7.3 Catchment flow

There are a number of equations that can be used for calculating the catchment time of concentration. The one mentioned below is from the Ministry of Business, Innovation and Employment Department of Building and Housing guidance on E1 Surface Water.[30]

$$t_c = 0.0195(L^3/_{H})^{0.385}$$ **Equation 7-4**

Where:
t_c = time of concentration (minutes)
L = Length of catchment (m) measured along the flow path
H = rise from bottom to top of catchment (m)

This equation can be used in catchments where there are significant changes in gradient along the channel slope or where the open channel is in a rural area, which would apply to most situations in the Waikato Region.

7.4 Alternative equations

Other equations for calculating time of concentration include:

- NRCS lag formula, where 1.67 times the lag equals the time of concentration
- The Carter lag equation for catchments that are partially natural channels and partially reticulated
- The Eagleson lag equation that includes a factor for converting lag to time of concentration, and
- Kerby-Hathaway formula for calculating the time of concentration for very small catchments in which surface flow dominates.

There are numerous other equations that may be adequate depending on the situation that they are used. When calculating the time of concentration, justification should be provided for the equation used.

[30] https://www.building.govt.nz/assets/Uploads/building-code-compliance/e-moisture/e1-surface-water/asvm/e1-surface-water-amendment-9.pdf

8 Analysis

The information presented in the previous sections outlines how to obtain most inputs needed to calculate flow rates and storm volumes. Several other items, including site hydraulic conductivity testing to inform soil group classification and ultimately the land use curve number (discussed in this section) allow the designer to complete the hydrologic analysis.

Appendix B provides blank worksheets that provide assistance in completing the analysis. It is understood that many designers will have spreadsheets that accomplish the same goal, however the worksheets are provided for those who want to understand the process and variables that are needed to complete the analysis.

One additional item that is needed for calculating the specific peak flow rate is Figure 8-1 from ARC's TP108 that is provided below. It is recognised that this figure was based on analysis of Auckland catchments and ideally the same assessment would be undertaken for the Waikato Region to derive a figure specific to the Waikato. It is recommended this assessment be undertaken, but that in the interim the figure provided below be utilised in the Waikato.

To then complete the analysis, follow the steps shown in the worksheet to calculate peak flow rates, runoff depth and runoff volumes.

In order to analyse pre and post development hydrology it is necessary to accomplish four different development scenarios. They include:

1. Pre-development site analysis where the pre-development land use is assessed to obtain pre-development peak discharges, generally for the 2, 10 and potentially for the 100 year ARI events, however this will depend on the downstream receiving environment.[31]
2. Post-development site analysis where the impervious and pervious areas are grouped to determine post-development peak flows, generally for the 2, 10 and potentially 100 year ARI events, as per above.
3. Post-development site analysis of the impervious surfaces only to obtain:
    - Post-development water quality volume.
    - Storm volumes for impervious surfaces generally for the 2, 10 and potentially 100 year ARI events.
4. Post-development site analysis of the site pervious areas to obtain:
    - Post-development water quality volume.
    - Storm volumes for pervious surfaces generally for the 2, 10 and potentially 100 year ARI events.

Calculations must be undertaken separately for pervious and impervious surfaces to calculate the total volumes associated with water quality and extended detention. This approach provides a more accurate and more consistent calculation for volume. Grouping pervious and impervious surfaces together for the analysis tends to under predict volumes associated with those storms.

Calculations can be grouped for pervious and impervious surfaces to assess peak discharges for the 2, 10 and 100 year ARI events and for consideration of timing.

If designing a stormwater detention device, the device should be designed for the total volume of stormwater that will drain to the device, not the difference between the post-development volume and the pre-development volume. If the pre-development volume is subtracted from the post-development volume, then the detention device will <u>not</u> be sized appropriately, as the actual ponding volumes in the device would be based on the change in volume reaching the

[31] Refer to WRC's Waikato Stormwater Management Guideline for further details of required design criteria.

device, instead of the full post-development volume reaching the device. Hydraulic modelling packages such as HEC-HMS help to refine the assessment of the required detention volumes.

Climate change adjusted rainfall must be used to determine post-development peak flows. Rainfall that hasn't been adjusted for climate change must be used to determine pre-development peak flows.

8.1 Incorporation of initial abstraction retention

The initial abstraction is a rainfall depth and converting it to a volume of runoff for impervious surfaces is straightforward as all of the rainfall that falls on the impervious surface is assumed to be directly converted to runoff.

For pervious areas that are not rehabilitated, additional analysis is necessary. In those situations:

1. The pre-development runoff curve number (CN) is used to calculate the runoff volume for the area that will be pervious but not rehabilitated post-development.
2. The post-development CN should be determined assuming that the soil group is altered to account for soil disruption and compaction that occurs during site development. As discussed in Section 5.1, the soil group classification should be reduced by one classification to account for this. Hence a soil that is Group A for the pre-development condition, should be considered Group B for the post-development condition (similarly, a Group B soil becomes Group C and Group C becomes Group D).
3. The required runoff retention due to soil compaction of pervious areas is determined using the following equation:

$$\boldsymbol{V_r = (I_{a1} - I_{a2})\ x\ A_{pervious}} \qquad \textbf{Equation 8-1}$$

   Where:
   V_r = Stormwater runoff volume for pervious areas that is to be retained (m^3)
   I_{a1} = Initial abstraction for pervious area for pre-development soil condition (m)
   I_{a2} = Initial abstraction for pervious area for post-development soil condition where soil rehabilitation has not been undertaken (m)
   A = Surface area of compacted pervious surface (m^2)

4. This volume is then combined with the impervious area retention volume to arrive at a total site retention volume.

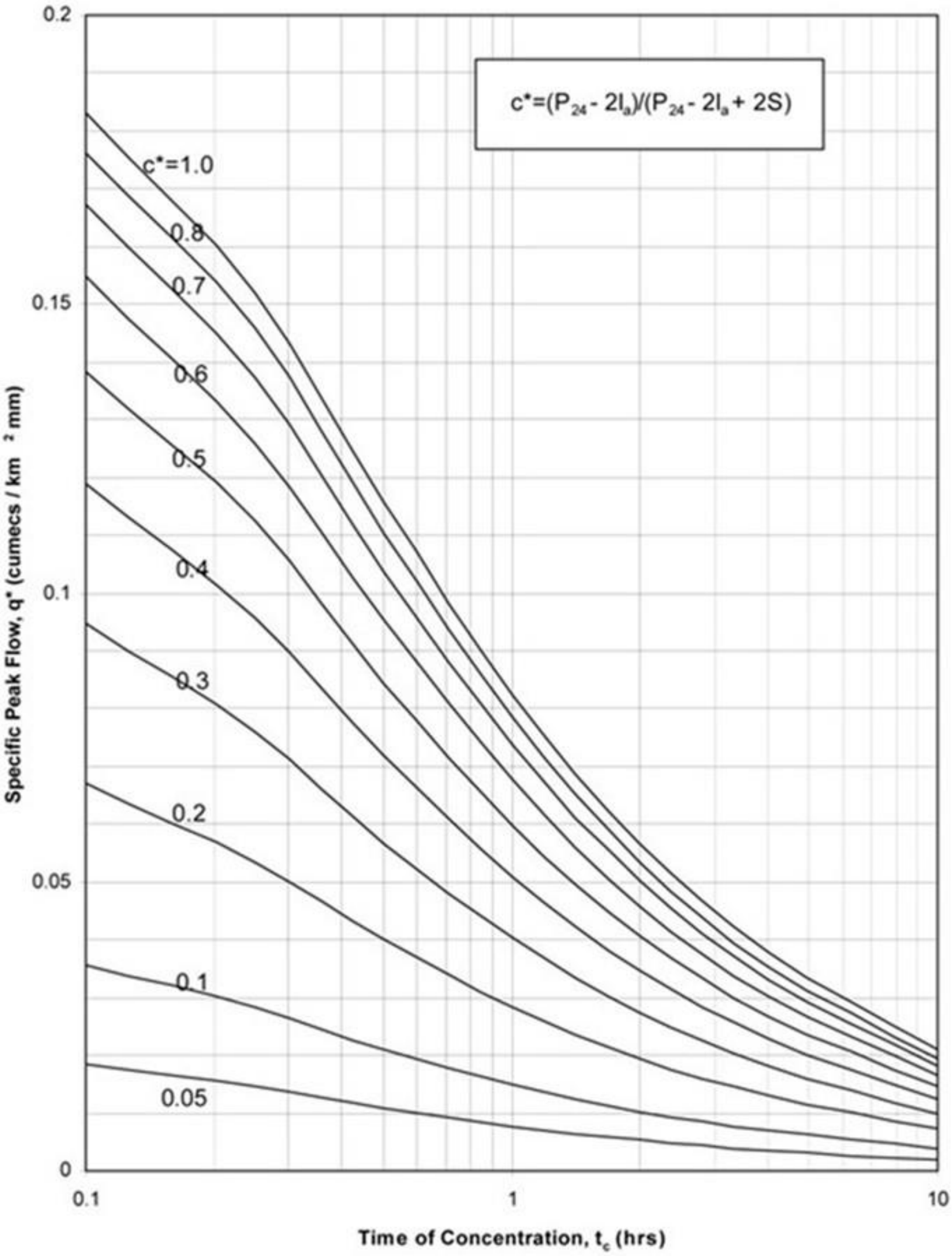


Figure 8-1: Determining the specific flow rate[32]

[32] Auckland Regional Council, 1999

9 Examples

9.1 Case study 1

A residential subdivision is proposed on a 20 hectare site just north of Hamilton City. The following details describe the site and the proposal:

- There are no up-catchment flows draining through the site.
- Site slope is 2%
- Pre-development condition is pasture
- Post-development land use is residential, comprising 150 lots, with average lot size being 400m^2.
- Site drains into the upper reaches of a stream, which requires
  - Attenuation of the 2 and 10 year ARI peak flows to pre-development flow rates
  - Extended detention, and
  - Water quality treatment.
- Downstream flooding is not a concern.
- 2 year rainfall = 65.6mm
- 10 year rainfall = 98.3mm
- The downstream channel is considered to be unstable hence requires the extended detention volume to be assessed based on 1.2 x Water Quality Volume for the post-development condition.

Pre-development

Site soils are orthic brown soils that have a high permeability in pasture conditions but are impacted negatively in earthworked areas[33].

Based on soil testing the soil in pre-developed condition is considered to be a Group B soil while in developed condition without rehabilitation is a Group C soil.

Worksheet 1 and Worksheet 2 (provided in Appendix B of this guideline) have been filled out for this site. The worksheets are provided on the next pages.

Worksheet 1 and Worksheet 2 show:

- Pre-development calculations of the 2 and 10 year ARI peak discharges

As assessed using Worksheet 1 and 2, for the pre-development site:
Pre-development CN = 69
I_a = 5.71 mm
Slope = 2%
Time of concentration = 3.2 hours

2 year ARI event:

- Peak flow rate, q_p= 0.42 m^3/s
- Runoff depth, Q_{24} = 20.6 mm
- Runoff volume, V_{24} = 4,118 m^3

10-year ARI event:

- Peak flow rate, q_p = 0.82 m^3/s
- Runoff depth, Q_{24} = 41.4 mm
- Runoff volume, V_{24} = 8,287 m^3

[33] Zanders, 2001

Project: Case study 1 **By:** ______ **Date:** ______

Location: North of Hamilton **Checked:** ______ **Date:** ______

Scenario: Pre-developed **(Pre-developed or post-developed)**

1. **Runoff Curve Number (CN) and Initial Abstraction (I_a)**

Soil name and classification	Cover description (cover type, treatment and hydrologic condition	Curve Number (CN)	Area (km^2)	Product of CN x Area
Orthic brown soil	Pasture	69	0.2	13.8
		TOTALS	0.2	13.8

CN (weighted) = $\frac{Total\ Product\ of\ CN\ x\ Area}{Total\ Area}$ = 13.8 / 0.2 = 69

Initial abstraction

$S = \left(\frac{1000}{CN} - 10\right)25.4\ \ (mm)$ = (1000 / 69 - 10) x 25.4 = 114.3 mm

$I_a = 0.05\,S$ = 0.05 x 114.3 = 5.71 mm

2. **Time of Concentration (T_c)**

(a) Sheet and shallow concentrated flow

From Equation 7-2 or from Figure 7-1: From Equation 7-2 as it is a rural catchment

$T_t = {100nL^{0.33}}/{S^{0.2}}$ = 100 x 0.045 x 300$^{0.33}$ /2$^{0.2}$ = 25.7 minutes

n = 0.045 (Manning's n roughness for pasture from Table 7-1)

L = 300m (length of overland flow)

S = 2%

(b) Concentrated network flow

i. Road channel flow from Figure 7-2: Nil for pre-developed

ii. Pipe network flow from Table 7-2 and Figure 7-3: Nil for pre-developed

iii. Open channel flow from Equation 7-3:

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

The flow goes through a transition at approximately 300m to open channel flow for a length of 340m. The channel is relatively small with the depth approximately 0.5 m, the width approximately 0.4 m and with near vertical side slopes. The slope of the channel is 2% (0.02m/m). The Manning's roughness coefficient for the channel is 0.12 as it is densely vegetated and not maintained.

R = hydraulic radius = Area / wetted perimeter
Area = 0.4 x 0.5 = 0.2 m^2 (width of channel x depth of channel)
Wetted perimeter = 0.5 + 0.4 + 0.5 = 1.4 m (perimeter of the cross sectional area that is wet)
R = 0.2 / 1.4 = 0.14 m
V = $0.14^{2/3}$ x $0.02^{1/2}$ / 0.12
= 0.31 m/s
At this velocity it takes 340 m / 0.31m/s = 1097 seconds to travel the distance, or T_t = 18.3 minutes

(c) Time of concentration
$T_c = T_{t1} + T_{t2} + \cdots..T_{tm}$ = 25.7 mins + 18.3 mins = 41.2 mins = 0.73 hours

SCS Lag for HEC-HMS = $t_p = \frac{2}{3} t_c$ = 2/3 x 3.2 = 0.49 hours

Worksheet 2: Graphical Peak Flow Rate

Project: Case study 1 **By:** ______ **Date:** ______

Location: North of Hamilton **Checked:** ______ **Date:** ______

Scenario: Pre-developed **(Pre-developed or post-developed)**

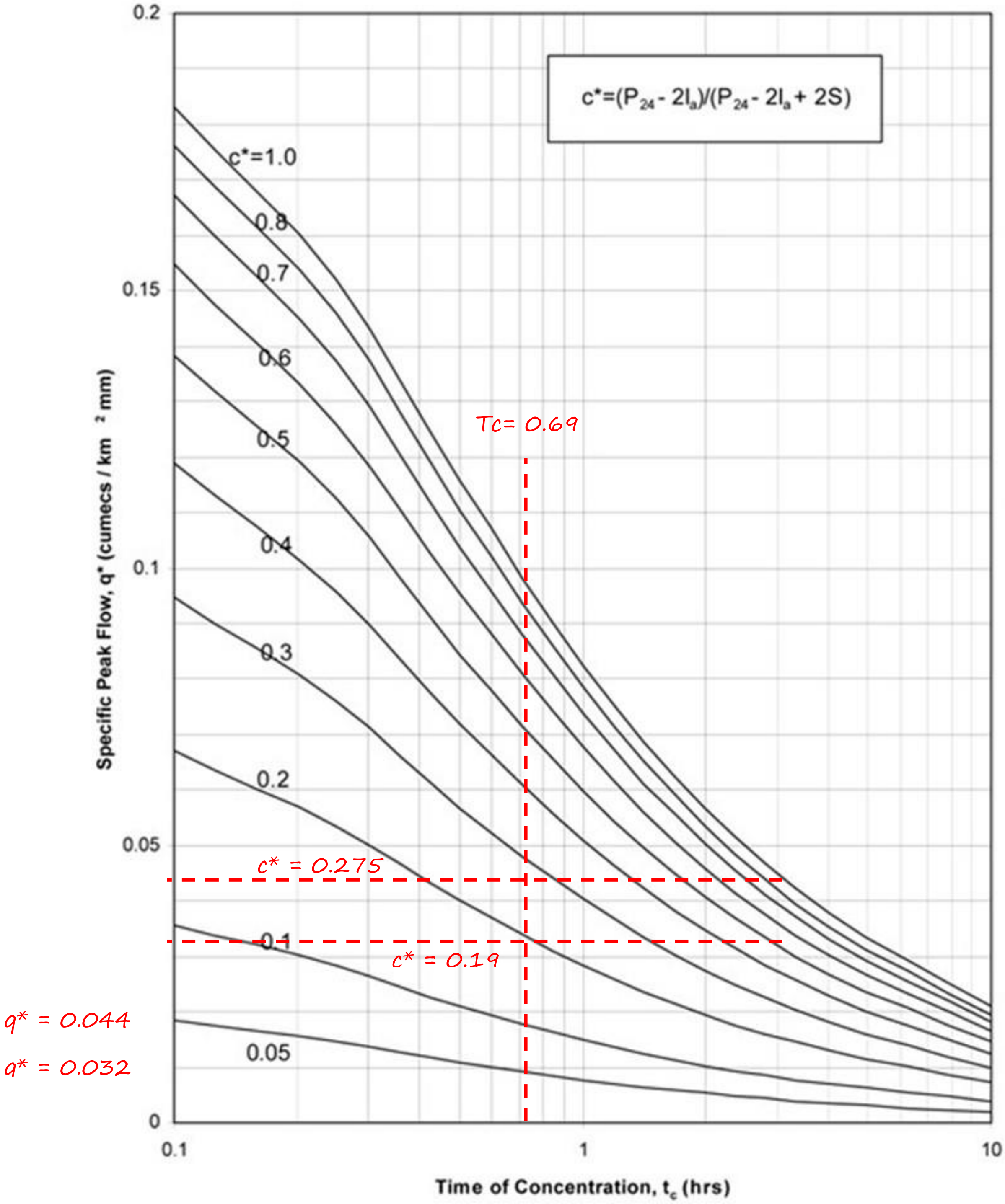
1. Data

Catchment area (A) =	0.2	km^2
Runoff curve number (CN) =	69	(from Worksheet 1)
Initial abstraction (Ia) =	5.71	(from Worksheet 1)
Time of concentration (Tc) =	0.73	hours (from Worksheet 1)

2. Storage

Storage (S) = 114.3 mm (from Worksheet 1)

	Storm #1	Storm #2	Storm #3
Average Recurrence Interval (ARI)	2	10	
24-hour rainfall depth P24 (mm)	65.6	93.3	
Compute c*: $c^* = \frac{P_{24} - 2I_a}{P_{24} - 2I_a + 2S}$	0.19	0.275	
Specific peak flow rate q* (From Figure 8-1)	0.032	0.044	
Peak flow rate $q_p = q^* A P_{24}$ (m^3/s)	0.42	0.82	
Runoff depth $Q_{24} = \frac{(P_{24} - I_a)^2}{(P_{24} - I_a) + S}$	20.6	41.4	
Runoff volume $V_{24} = 1000 x Q_{24} A$ (m^3)	4,118	8,287	


c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Tc= 0.69
c* = 0.275
c* = 0.19
q* = 0.044
q* = 0.032
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)
0.2
0.15
0.1
0.05
0
0.1
1
10

Post-development

Climate change adjusted rainfall must be determined for the post-development scenario.

2 year ARI existing rainfall = 65.6 mm
Using the predicted increase in temperature of 2.1°C, from Table 4-3, for a 2 year ARI event with a 24 hour duration the percentage adjustment to apply to the existing rainfall is 9.03%.
2 year ARI CC rainfall = 71.5 mm

10 year existing rainfall = 98.3 mm
From Table 4-3, for a 10 year ARI event with a 24 hour duration the percentage adjustment to apply to the existing rainfall is 13.23%
10 year ARI CC rainfall = 111.3 mm

CN of pervious areas = 79 – The CN of 79 was chosen as the pervious areas soils were not rehabilitated so the original CN of 69 was not used and the site soil permeability was reduced one classification resulting in the pre-development B soil being reduced to a C soil..
CN of impervious areas = 98
Percentage impervious cover = 65%
Average CN = 90
Piped stormwater system

Calculations must be undertaken separately for pervious and impervious surfaces to calculate the total volumes associated with water quality and extended detention. This approach provides a more accurate and more consistent calculation for volume. Grouping them together for the analysis tends to under-predict volumes associated with those storms. To assess peak discharges for the 2, 10 and 100 year ARI events, pervious and impervious surfaces can be grouped for consideration of timing and peak discharges.

Using Worksheets 1 and 2, the following assessments for the post-development scenario have been undertaken (the worksheets are provided below):

- Post development pervious area only
- Post development impervious area only
- Post development for the whole site

The following summarises the results of the calculations:

2-year storm peak flow rate = 1.36 m^3/s (from 'Post-developed whole site' worksheets)
> Runoff depth – pervious areas = 34.2 mm, runoff volume = 2,394 m^3 (from 'Post-developed – pervious' worksheets)
> Runoff depth – impervious areas = 66.4 mm, runoff volume = 8,626 m^3 (from Post-developed – impervious' worksheets)
> Total runoff volume = 11,020 m^3

10-year storm peak flow rate = 2.4 m^3/s
> Runoff depth - pervious areas = 66.4 mm, runoff volume = 4,646 m^3
> Runoff depth – impervious areas = 106 mm, runoff volume = 13,780 m^3
> Total runoff volume = 18,426 m^3

Water quality rainfall is 1/3 of 2-year storm or 23.8 mm of rainfall over a 24-hour period.
> Runoff depth - pervious surfaces = 4.73 mm, runoff volume = 331 m^3
> Runoff depth – impervious surfaces = 19.2 mm, runoff volume = 2,502 m^3
> Total runoff volume = 2,833 m^3.

As retention of the initial abstraction should be provided, there are two steps for impervious and pervious surfaces:

- For impervious surfaces, the retention can be determined by taking the pre-development site initial abstraction (5.71 mm) and multiply it by the surface area of impervious surfaces (in this case 130,000 m^2) giving 742 m^3 of retention storage for the impervious surface.

  For pervious surfaces calculate the storage required for pervious surfaces as follows:

  - The pre-development runoff curve number (CN) is used to calculate the initial abstraction (I_{a1}) for the pervious areas.
  - The post-development CN should be determined assuming that the soil group is altered to account for soil disruption and compaction where rehabilitation is not going to be done. Thus, a group A soil should be considered as a B soil in the post-development scenario (similar to a B becomes C, C becomes D). The initial abstraction (I_{a2}) is based on the altered soil grouping due to compaction.
  - The required runoff retention due to soil compaction of the pervious areas is the following:

    I_{a1} = pervious area initial abstraction in (m) for pre-development soils = 0.00571m

    I_{a2} = pervious area initial abstraction for post-development soils where soil rehabilitation has not been done in (m). The initial abstraction = 0.0034 m

    A = post-development compacted pervious surface area in (m^2) = 70,000 m^2.

    V_r = stormwater runoff volume (V_r) for pervious areas that should be retained (m^3).

    $V_r = (I_{a1} – I_{a2})x(A) = 161.7\ m^3$

The site water quality volume can be reduced by the total reduction in runoff volume provided by retention of runoff from both pervious and impervious surfaces. In this case the volume reduction is 903.7 m^3. The water quality volume for the site is now 1,929.3 m^3

Extended detention storage is 1.2 times water quality volume as there is downstream erosion of the stream channel. Storage for extended detention = 2,315.16 m^3 that must be released over a 24-hour period.

The water quality volume does not alter the volumes associated with larger storms as the water quality volume is dead storage in ponds.

Worksheet 1: Runoff Parameters and Time of Concentration

Project: Case study 1 **By:** ______ **Date:** ______

Location: North of Hamilton **Checked:** ______ **Date:** ______

Scenario: Post-developed - pervious **(Pre-developed or post-developed)**

1. Runoff Curve Number (CN) and Initial Abstraction (I_a)

Soil name and classification	Cover description (cover type, treatment and hydrologic condition	Curve Number (CN)	Area (km^2)	Product of CN x Area
Orthic brown soil	Grass	79	0.07	5.53
		TOTALS	0.07	5.53

CN (weighted) = $\frac{Total\ Product\ of\ CN\ x\ Area}{Total\ Area}$ = 5.53 / 0.07 = 79

Initial abstraction

$S = \left(\frac{1000}{CN} - 10\right) 25.4 \quad (mm)$ = ((1000 / 79) − 10) x 25.4 = 67.5

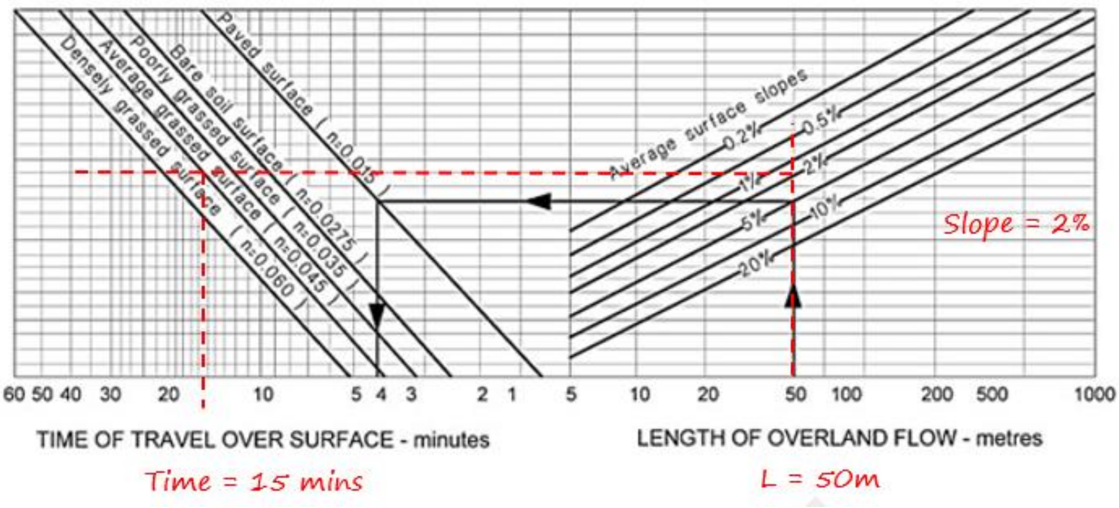
$I_a = 0.05\, S$ = 0.05 * 67.5 = 3.4

2. Time of Concentration (T_c)

(a) Sheet and shallow concentrated flow

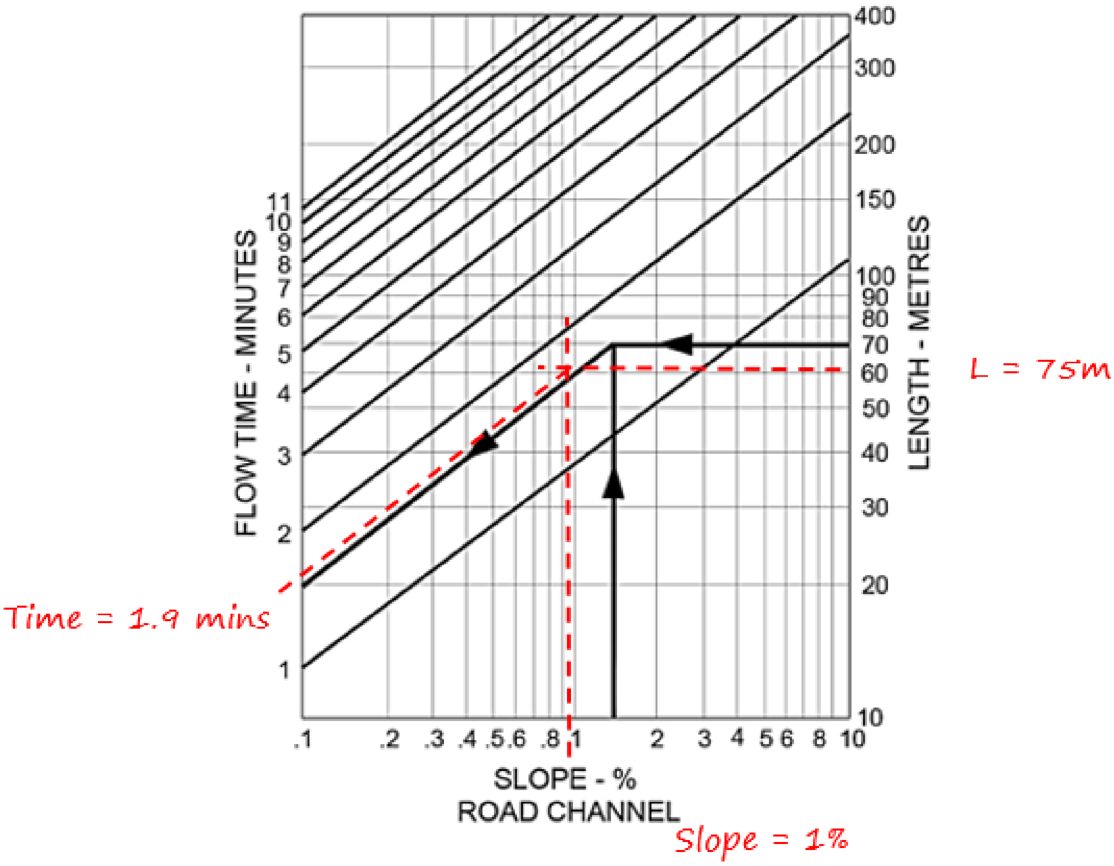
From Equation 7-2 or from Figure 7-1: 50m sheet flow, 2% slope, average grass. From Figure 7-1 Tc = 15 minutes

$T_t = 100nL^{0.33} / S^{0.2}$



(b) Concentrated network flow

i. Road channel flow from Figure 7-2: *Tc (road channel flow) = 1.9 minutes*



ii. Pipe network flow from Table 7-2 and Figure 7-3: *Flat gradient, typical velocity = 0.6 m/s from Table 7-2. Pipe network is 450m long, hence Tc = 450 m / 0.06 m/s = 750 s = 12.5 minutes*

iii. Open channel flow from Equation 7-3: *No open channel flow*

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

(c) Time of concentration

$T_c = T_{t1} + T_{t2} + \cdots..T_{tm}$ = 15 mins + 1.9 mins + 12.5 mins = 29.4 mins = 0.49 hours

SCS Lag for HEC-HMS = $t_p = \frac{2}{3} t_c$ = 0.49 * 2/3 = 0.33 hours

Worksheet 2: Graphical Peak Flow Rate

Project: Case study 1 **By:** ______ **Date:** ______

Location: North of Hamilton **Checked:** ______ **Date:** ______

Scenario: Post-developed - pervious **(Pre-developed or post-developed)**

1. Data

Catchment area (A) = 0.07 km^2

Runoff curve number (CN) = 79 (from Worksheet 1)

Initial abstraction (Ia) = 3.4 (from Worksheet 1)

Time of concentration (Tc) = 0.49 hours (from Worksheet 1)

2. Storage

Storage (S) = 67.5 mm (from Worksheet 1)

	Storm #1	**Storm #2**	**Storm #3**
Average Recurrence Interval ARI (year)	Water quality	2 year	10 year
24-hour rainfall depth P24 (mm)	23.8	71.5	111.3
Compute c*: $c^* = \frac{P_{24} - 2I_a}{P_{24} - 2I_a + 2S}$	Unnecessary for volume calculations		
Specific peak flow rate q* (From Figure 8-1)	Unnecessary for volume calculations		
Peak flow rate q_p: $q_p = q^* A P_{24}$ (m^3/s)	Unnecessary for volume calculations		
Runoff depth Q_{24}: $Q_{24} = \frac{(P_{24} - I_a)^2}{(P_{24} - I_a) + S}$	4.73	34.2	66.4
Runoff volume V_{24}: $V_{24} = 1000 x Q_{24} A$ (m^3)	331	2,394	4,646

Worksheet 1: Runoff Parameters and Time of Concentration

Project: Case study 1 **By:** ________ **Date:** ________

Location: North of Hamilton **Checked:** ________ **Date:** ________

Scenario: Post-developed - impervious **(Pre-developed or post-developed)**

1. Runoff Curve Number (CN) and Initial Abstraction (I_a)

Soil name and classification	Cover description (cover type, treatment and hydrologic condition	Curve Number (CN)	Area (km^2)	Product of CN x Area
-	Impervious surfaces	98	0.13	12.74
		TOTALS	0.13	12.74

CN (weighted) = $\frac{Total\ Product\ of\ CN\ x\ Area}{Total\ Area}$ = 12.74 / 0.13 = 98

Initial abstraction

$S = \left(\frac{1000}{CN} - 10\right) 25.4 \ \ (mm)$ = ((1000 / 98) – 10) x 25.4 = 5.2 mm

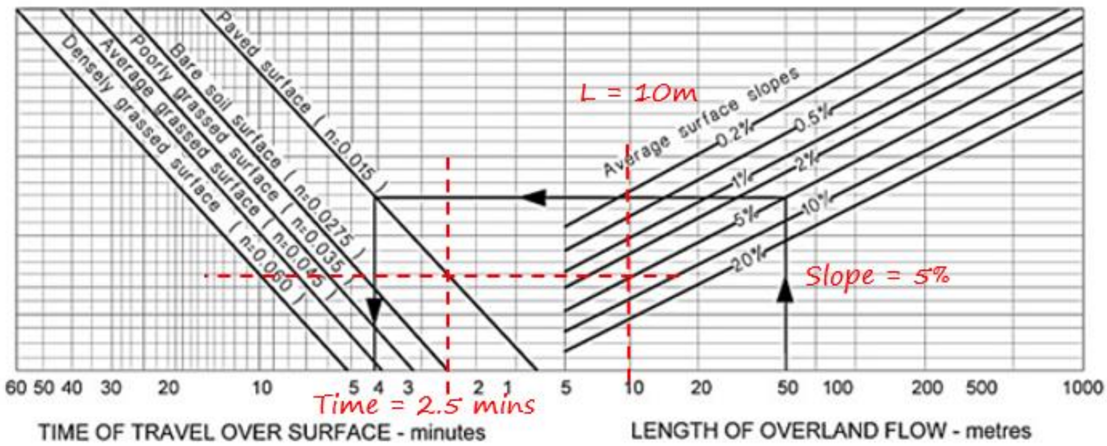
$I_a = 0.05\, S$ = 0.05 * 5.2 = 0.3 mm

2. Time of Concentration (T_c)

(a) Sheet and shallow concentrated flow

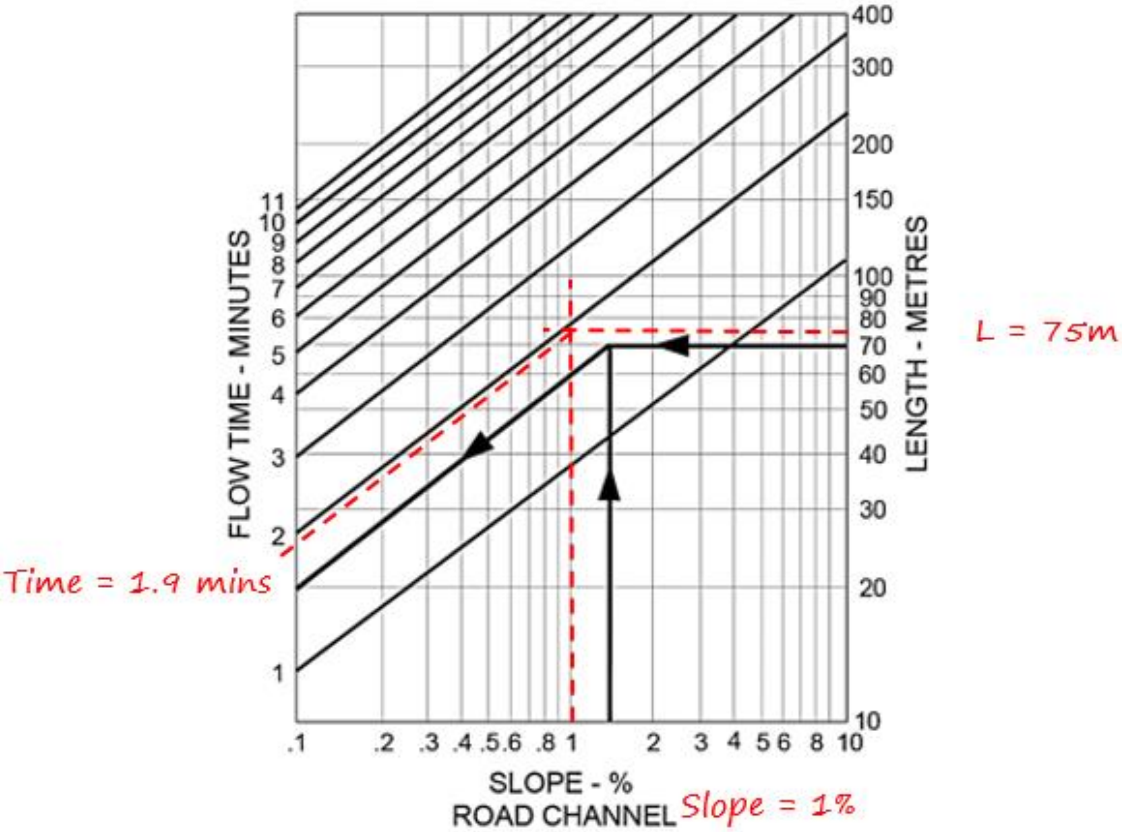
From Equation 7-2 or from Figure 7-1: Length = 10m at slope of 5%, with paved surface. From Figure 7-1 = 2.5 minutes

$T_t = {100nL^{0.33}}/{S^{0.2}}$



(b) Concentrated network flow

i. Road channel flow from Figure 7-2: *1% road grade, 75 m long. From Figure 7-2 Tc = 1.9 minutes*



ii. Pipe network flow from Table 7-2 and Figure 7-3: *Flat gradient, typical velocity = 0.6 m/s from Table 7-2. Pipe network is 450m long, hence Tc = 450 m / 0.06 m/s = 750 s = 12.5 minutes*

iii. Open channel flow from Equation 7-3: *No open channel flow*

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

(c) Time of concentration

$T_c = T_{t1} + T_{t2} + \cdots..T_{tm}$ = 2.5 *mins + 1.9 mins + 12.5 mins = 16.9 mins = 0.28 hours*

SCS Lag for HEC-HMS = $t_p = \frac{2}{3}t_c$ = *0.28 * 2/3 = 0.19* hours

Worksheet 2: Graphical Peak Flow Rate

Project:	Case study 1	**By:**		**Date:**	
Location:	North of Hamilton	**Checked:**		**Date:**	
Scenario:	Post-developed - impervious	**(Pre-developed or post-developed)**			

1. Data

Catchment area (A) = 0.13 km^2

Runoff curve number (CN) = 98 (from Worksheet 1)

Initial abstraction (Ia) = 0.3 (from Worksheet 1)

Time of concentration (Tc) = 0.28 hours (from Worksheet 1)

2. Storage

Storage (S) = 5.2 mm (from Worksheet 1)

	Storm #1	Storm #2	Storm #3
Average Recurrence Interval ARI (year)	Water quality	2 year	10 year
24-hour rainfall depth P24 (mm)	23.8	71.5	111.3
Compute c*: $c^* = \frac{P_{24} - 2I_a}{P_{24} - 2I_a + 2S}$	Unnecessary for volume calculations		
Specific peak flow rate q* (From Figure 8-1)	Unnecessary for volume calculations		
Peak flow rate q_p: $q_p = q^* A P_{24}$ (m^3/s)	Unnecessary for volume calculations		
Runoff depth Q_{24}: $Q_{24} = \frac{(P_{24} - I_a)^2}{(P_{24} - I_a) + S}$	19.2	66.4	106
Runoff volume V_{24}: $V_{24} = 1000 x Q_{24} A$ (m^3)	2,502	8,626	13,780

Worksheet 1: Runoff Parameters and Time of Concentration

Project: Case study 1 **By:** ______ **Date:** ______

Location: North of Hamilton **Checked:** ______ **Date:** ______

Scenario: Post-developed – whole site (to calculate peak flows) **(Pre-developed or post-developed)**

1. Runoff Curve Number (CN) and Initial Abstraction (I_a)

Soil name and classification	Cover description (cover type, treatment and hydrologic condition	Curve Number (CN)	Area (km^2)	Product of CN x Area
Orthic brown, disturbed	Grass	79	0.07	5.53
-	Impervious surfaces	98	0.13	12.74
		TOTALS	0.20	18.27

CN (weighted) = $\frac{Total\ Product\ of\ CN\ x\ Area}{Total\ Area}$ = 18.27 / 0.20 = 91.4

Initial abstraction

$S = \left(\frac{1000}{CN} - 10\right) 25.4 \quad (mm)$ = ((1000 / 91.4) – 10) x 25.4 = 23.9 mm

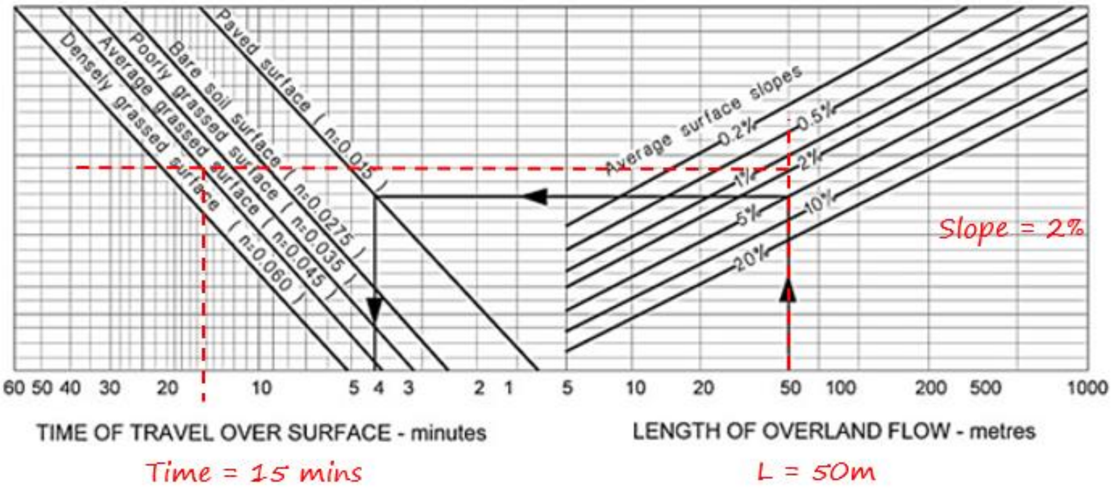
$I_a = 0.05\,S$ = 0.05 * 23.9 = 1.2 mm

2. Time of Concentration (T_c)

(a) Sheet and shallow concentrated flow

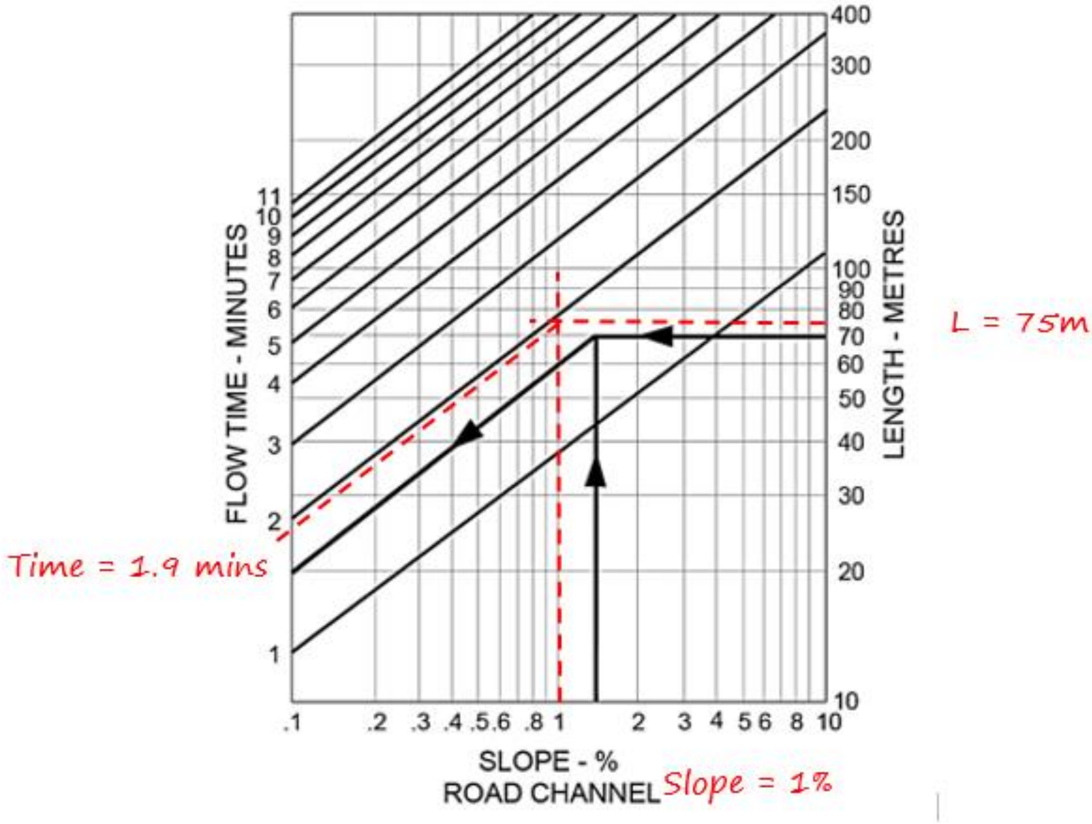
From Equation 7-2 or from Figure 7-1: 50m sheet flow, 2% slope and average grass. From Figure 7-1 Tc = 15 minutes

$T_t = 100nL^{0.33}/S^{0.2}$



(b) Concentrated network flow

i. Road channel flow from Figure 7-2: *1% road grade, 75 m long. From Figure 7-2 Tc = 1.9 minutes*



ii. Pipe network flow from Table 7-2 and Figure 7-3: *Flat gradient, typical velocity = 0.6 m/s from Table 7-2. Pipe network is 450m long, hence Tc = 450 m / 0.06 m/s = 750 s = 12.5 minutes*

iii. Open channel flow from Equation 7-3: *No open channel flow*

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

(c) Time of concentration

$T_c = T_{t1} + T_{t2} + \cdots..T_{tm}$ = 15 mins + 1.9 mins + 12.5 mins = 29.4 mins = 0.49 hours

SCS Lag for HEC-HMS = $t_p = \frac{2}{3}t_c$ = 0.49 * 2/3 = 0.33 hours

Worksheet 2: Graphical Peak Flow Rate

Project:	Case study 1	**By:**	**Date:**
Location:	North of Hamilton	**Checked:**	**Date:**
Scenario:	Post-developed – whole site (to calculate peak flows)	**(Pre-developed or post-developed)**	

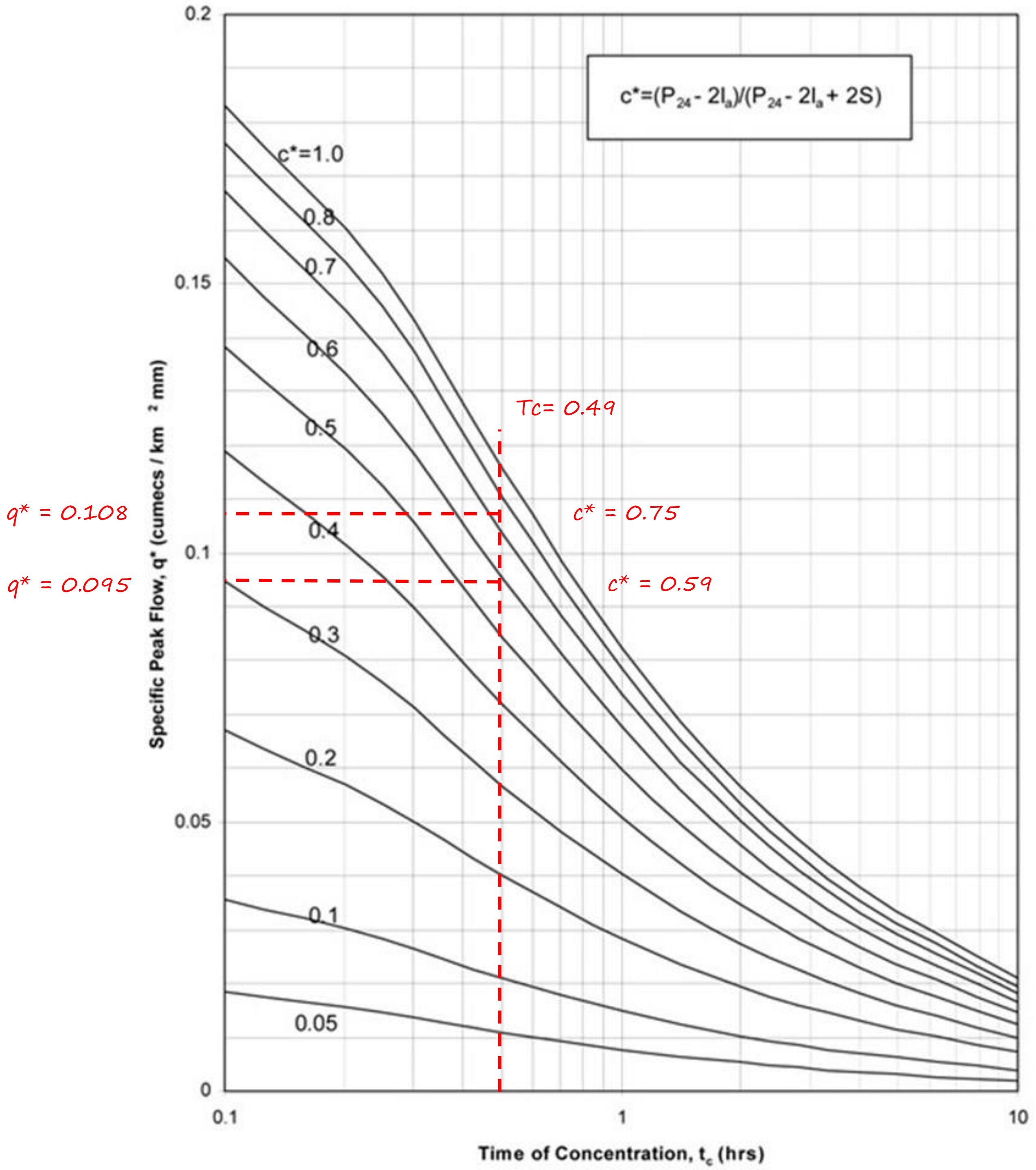
1. Data

Catchment area (A) =	0.2	km^2
Runoff curve number (CN) =	91.4	(from Worksheet 1)
Initial abstraction (Ia) =	1.2	(from Worksheet 1)
Time of concentration (Tc) =	0.49	hours (from Worksheet 1)

2. Storage

Storage (S) =	23.9	mm (from Worksheet 1)

	Storm #1	Storm #2	Storm #3
Average Recurrence Interval ARI (year)		2 year	10 year
24-hour rainfall depth P24 (mm)		71.5	111.3
Compute c*: $c^* = \frac{P_{24} - 2I_a}{P_{24} - 2I_a + 2S}$		0.59	0.75
Specific peak flow rate q* (From Figure 8-1)		0.095	0.108
Peak flow rate q_p: $q_p = q^* A P_{24}$ (m^3/s)		1.36	2.4
Runoff depth Q_{24}: $Q_{24} = \frac{(P_{24} - I_a)^2}{(P_{24} - I_a) + S}$			
Runoff volume V_{24}: $V_{24} = 1000xQ_{24}A$ (m^3)			



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Tc= 0.49
q* = 0.108
q* = 0.095
c* = 0.75
c* = 0.59
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, tc (hrs)
0.2
0.15
0.1
0.05
0
0.1
1
10

9.2 Case study 2

A commercial development is proposed in a catchment that is a tributary of Lake Taupo. The following details describe the site and the proposal:

- The site is two hectares in size with no external runoff crossing the property.
- Slope is 4%
- Catchment length is 0.223 km
- Pre-development land use is pasture
- Ultimate development of the site will result in 80% impervious surfaces
- The site drains into the upper reaches of a stream, which requires
  - Attenuation of the 2 and 10 year ARI peak flows to pre-development flow rates
  - Extended detention, and
  - Water quality treatment.
- Downstream flooding is not a concern.
- 2 year rainfall = 73.2mm
- 10 year rainfall = 105.3mm
- The downstream channel is considered to be stable hence the extended detention volume is to be assessed based on 1 x Water Quality Volume for the post-development condition.

Pre-development

Site soils are orthic pumice soils, specifically perch-gley pumice soils. These soils, when drained can have very high hydraulic conductivity rates, but in this situation, there is a perched water table that provides periodic wetness. As such the soil fits into the definition of having a dual classification. Testing during September (hypothetical) shows that the site soil has a hydraulic conductivity of 4 µm/s, which places the site soil in the hydrologic soil Group C category.

Worksheet 1 and Worksheet 2 (provided in Appendix B of this guideline) have been filled out for this site. The worksheets are provided on the next pages.

Worksheet 1 and Worksheet 2 show:

- Pre-development calculations of the 2 and 10 year ARI peak discharges

As assessed using Worksheet 1 and 2, for the pre-development site:
Pre-development CN = 79
I_a = 3.4 mm
Slope = 4%
Time of concentration = 2.3 hours

2 year ARI event:

- Peak flow rate, q_p= 0.072 m^3/s
- Runoff depth, Q_{24} = 35.5 mm
- Runoff volume, V_{24} = 710 m^3

10-year ARI event:

- Peak flow rate, q_p = 0.09 m^3/s
- Runoff depth, Q_{24} = 61.3 mm
- Runoff volume, V_{24} = 1,226 m^3

Worksheet 1: Runoff Parameters and Time of Concentration

Project: Case study 2 **By:** ______ **Date:** ______

Location: Taupo **Checked:** ______ **Date:** ______

Scenario: Pre-developed **(Pre-developed or post-developed)**

1. Runoff Curve Number (CN) and Initial Abstraction (I_a)

Soil name and classification	Cover description (cover type, treatment and hydrologic condition	Curve Number (CN)	Area (km^2)	Product of CN x Area
Orthic pumice	Pasture – high water table	79	0.02	1.58
		TOTALS	0.02	1.58

CN (weighted) = $\frac{Total\ Product\ of\ CN\ x\ Area}{Total\ Area}$ = 1.58 / 0.02 = 79

Initial abstraction

$S = \left(\frac{1000}{CN} - 10\right) 25.4 \quad (mm)$ = (1000 / 79 – 10) x 25.4 = 67.5 mm

$I_a = 0.05\,S$ = 0.05 x 67.5 = 3.4 mm

2. Time of Concentration (T_c)

(a) Sheet and shallow concentrated flow

From Equation 7-2 or from Figure 7-1: From Equation 7-2 as it is a rural catchment

$T_t = 100nL^{0.33}/S^{0.2}$ = 100 x 0.045 x $223^{0.33}$ / $4^{0.2}$ = 20.6 minutes

n = 0.045 (mannings n roughness for pasture from Table 7-1)

L = 223m (length of overland flow)

S = 4%

(b) Concentrated network flow

i. Road channel flow from Figure 7-2: Nil for pre-developed

ii. Pipe network flow from Table 7-2 and Figure 7-3: Nil for pre-developed

iii. Open channel flow from Equation 7-3: Nil for pre-developed

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

(c) Time of concentration

$T_c = T_{t1} + T_{t2} + \cdots .. T_{tm}$ = 20.6 mins = 0.34 hours

SCS Lag for HEC-HMS = $t_p = \frac{2}{3} t_c$ = 2/3 x 2.3 = 0.23 hours

Worksheet 2: Graphical Peak Flow Rate

Project: Case study 2 **By:** ______ **Date:** ______

Location: Taupo **Checked:** ______ **Date:** ______

Scenario: Pre-developed **(Pre-developed or post-developed)**

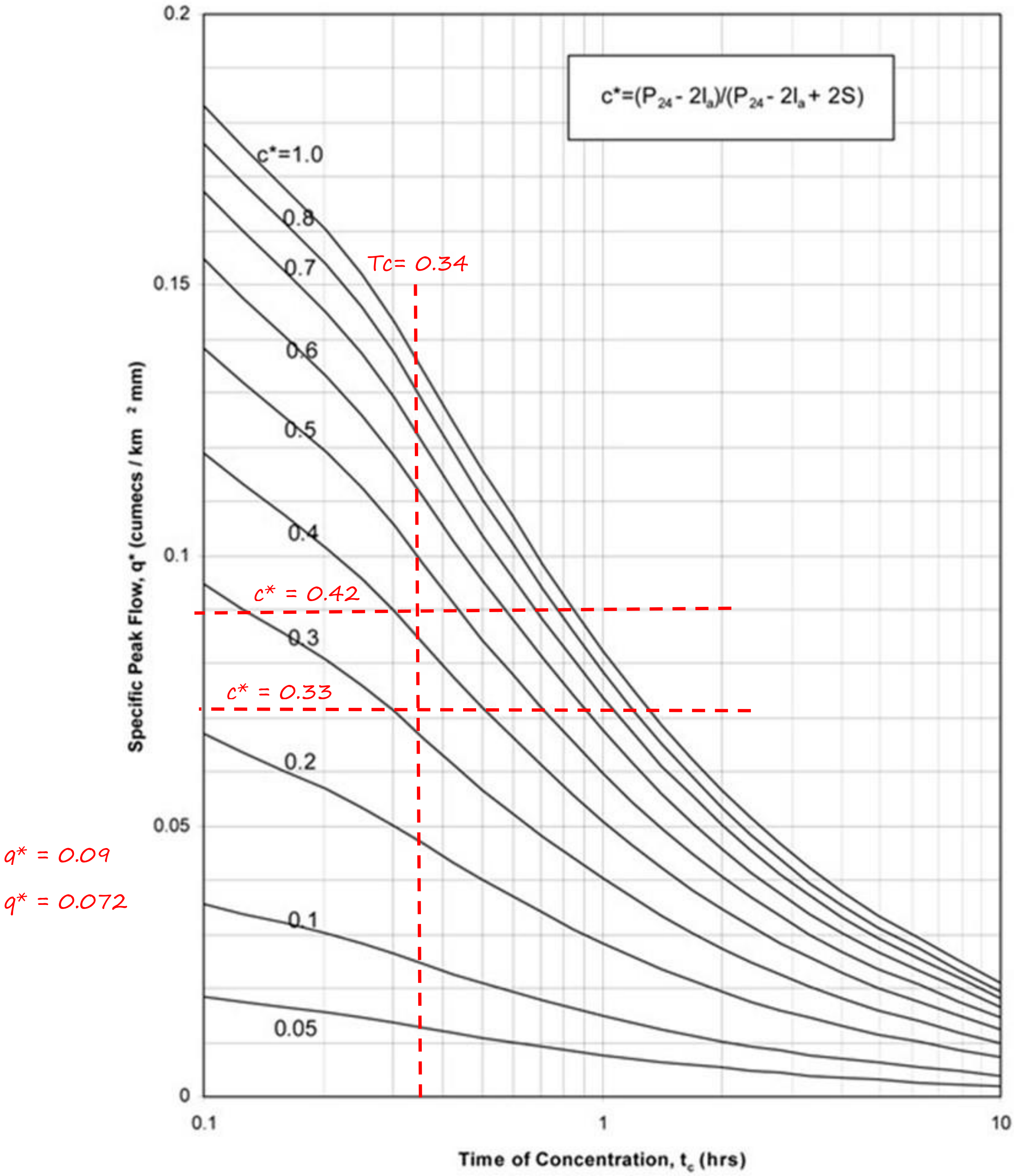
1. Data

Catchment area (A) =	0.02	km^2
Runoff curve number (CN) =	79	(from Worksheet 1)
Initial abstraction (Ia) =	3.4	(from Worksheet 1)
Time of concentration (Tc) =	0.34	hours (from Worksheet 1)

2. Storage

Storage (S) = 67.5 mm (from Worksheet 1)

	Storm #1	Storm #2	Storm #3
Average Recurrence Interval (ARI)	2	10	
24-hour rainfall depth P24 (mm)	73.2	105.3	
Compute c*: $c^* = \frac{P_{24} - 2I_a}{P_{24} - 2I_a + 2S}$	0.33	0.42	
Specific peak flow rate q* (From Figure 8-1)	0.029	0.033	
Peak flow rate $q_p = q^* A P_{24}$ (m^3/s)	0.072	0.09	
Runoff depth $Q_{24} = \frac{(P_{24} - I_a)^2}{(P_{24} - I_a) + S}$	35.5	61.3	
Runoff volume $V_{24} = 1000xQ_{24}A$ (m^3)	710	1,226	



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Tc= 0.34
c* = 0.42
c* = 0.33
q* = 0.09
q* = 0.072
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, tc (hrs)
0.2
0.15
0.1
0.05
0
0.1
1
10

Post-development

Climate change adjusted rainfall must be determined for the post-development scenario.

2 year ARI existing rainfall = 73.2 mm.
Using the predicted increase in temperature of 2.1°C, from Table 4-3 for a 2 year ARI event with a 24 hour duration, the percentage adjustment to apply to the existing rainfall is 9.03%.
2 year ARI CC rainfall = 79.8 mm

10 year ARI existing rainfall = 105.3 mm
From Table 4-3, for a 10 year ARI event with a 24 hour duration the percentage adjustment to apply to the existing rainfall is 13.23%.
10 year ARI CC rainfall = 119.2 mm

CN of pervious areas = 79 as soil rehabilitation has been done
CN of impervious areas = 98
Percentage impervious cover = 80%
Average CN = 94.2
Piped stormwater system
Length is 0.25 km
Slope is 1% due to significant site re-grading.

Calculations must be undertaken separately for pervious and impervious surfaces to calculate the total volumes associated with water quality and extended detention. This approach provides a more accurate and more consistent calculation for volume. Grouping them together for the analysis tends to under predict volumes associated with those storms. On the other hand, peak discharges for the 2, 10 and 100-year events can be grouped for consideration of timing and peak discharges.

Using Worksheets 1 and 2, the following assessments for the post-development scenario have been undertaken (the worksheets are provided below):

- Post development pervious area only
- Post development impervious area only
- Post development for the whole site

The following summarises the results of the calculations:

2-year storm peak flow rate = 0.185 m^3/s (from 'Post-developed whole site' worksheets)
    Runoff depth – pervious areas = 40.6 mm, runoff volume = 162.4 m^3 (from 'Post-developed – pervious' worksheets)
    Runoff depth – impervious areas = 77.7 mm, runoff volume = 1,195 m^3 (from Post-developed – impervious' worksheets)
    Total runoff volume = 1,327.4 m^3

10-year storm peak flow rate = 0.286 m^3/s
    Runoff depth - pervious areas = 73.2 mm, runoff volume = 292.6 m^3
    Runoff depth – impervious areas = 114 mm, runoff volume = 1,824 m^3
    Total runoff volume = 2,116.6 m^3

Water quality rainfall is 1/3 of 2-year storm or 23.8 mm of rainfall over a 24-hour period.
    Runoff depth - pervious surfaces = 5.9 mm, runoff volume = 23.6 m^3
    Runoff depth – impervious surfaces = 22 mm, runoff volume = 352 m^3
    Total runoff volume = 375.6 m^3

Extended detention storage is equivalent to the water quality volume as there is no downstream erosion of the stream channel. Storage for extended detention = 375.6 m^3 that must be released over a 24-hour period.

Worksheet 1: Runoff Parameters and Time of Concentration

Project: Case study 2 **By:** ________ **Date:** ________

Location: Taupo **Checked:** ________ **Date:** ________

Scenario: Post-developed – pervious **(Pre-developed or post-developed)**

1. Runoff Curve Number (CN) and Initial Abstraction (I_a)

Soil name and classification	Cover description (cover type, treatment and hydrologic condition	Curve Number (CN)	Area (km²)	Product of CN x Area
Orthic pumice	Grass	79	0.004	0.316
		TOTALS	0.004	0.316

CN (weighted) = $\frac{Total\ Product\ of\ CN\ x\ Area}{Total\ Area}$ = 0.316 / 0.004 = 79

Initial abstraction

$S = \left(\frac{1000}{CN} - 10\right) 25.4\ \ (mm)$ = ((1000 / 79) – 10) x 25.4 = 67.5

$I_a = 0.05\, S$ = 0.05 * 67.5 = 3.4

2. Time of Concentration (T_c)

From total site area analysis, the pervious areas are the farthest point from the outlet so time of concentration for the 'post-developed pervious areas only' is the same as the time of concentration for the 'post-developed whole site'. The runoff from pervious surfaces must travel overland and then along the kerbed area and into the pipe system before reaching the outlet. Calculations below repeated from 'post-developed whole site' worksheet.

(a) Sheet and shallow concentrated flow

From Equation 7-2 or from Figure 7-1:

25m flow distance at 1% slope (significant site levelling) on paved surfaces (from Worksheet for Post-development – imperv), Tc = 4 minutes

50m flow distance for grassed areas at 1% slope (from Worksheet for Post-development – pervious) = 13 minutes

Total travel time = 17 minutes

$T_t = {100nL^{0.33}}/{S^{0.2}}$

(b) Concentrated network flow:

i. Road channel flow from Figure 7-2: Flow along kerbing, approximately 50m long at 1% road grade (From Worksheet Post-development – Imperv), Tc = 1.4 minutes

ii. Pipe network flow from Table 7-2 and Figure 7-3: 200m of pipe at low gradient, typical velocity = 0.6 m/s from Table 7-2. Pipe network is 200m long, hence Tc = 200 m / 0.06 m/s = 333.3 s = 5.5 minutes

iii. Open channel flow from Equation 7-3: No open channel flow

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

(c) Time of concentration

$T_c = T_{t1} + T_{t2} + \cdots..T_{tm} =$ 17 mins + 1.4 mins + 5.5 mins = 23.9 mins = 0.4 hours

SCS Lag for HEC-HMS = $t_p = \frac{2}{3}t_c$ = 0.4 * 2/3 = 0.27 hours

Worksheet 2: Graphical Peak Flow Rate

Project: Case study 2 **By:** ______ **Date:** ______

Location: Taupo **Checked:** ______ **Date:** ______

Scenario: Post-developed - pervious **(Pre-developed or post-developed)**

1. Data

Catchment area (A) = 0.004 km^2

Runoff curve number (CN) = 79 (from Worksheet 1)

Initial abstraction (Ia) = 3.4 (from Worksheet 1)

Time of concentration (Tc) = 0.4 hours (from Worksheet 1)

2. Storage

Storage (S) = 67.5 mm (from Worksheet 1)

	Storm #1	**Storm #2**	**Storm #3**
Average Recurrence Interval ARI (year)	Water quality	2 year	10 year
24-hour rainfall depth P24 (mm)	26.6	79.8	119.2
Compute c*: $c^* = \frac{P_{24} - 2I_a}{P_{24} - 2I_a + 2S}$	Unnecessary for volume calculations		
Specific peak flow rate q* (From Figure 8-1)	Unnecessary for volume calculations		
Peak flow rate q_p: $q_p = q^* A P_{24}$ (m^3/s)	Unnecessary for volume calculations		
Runoff depth Q_{24}: $Q_{24} = \frac{(P_{24} - I_a)^2}{(P_{24} - I_a) + S}$	5.9	40.6	73.2
Runoff volume V_{24}: $V_{24} = 1000 x Q_{24} A$ (m^3)	23.6	162.4	292.6

Worksheet 1: Runoff Parameters and Time of Concentration

Project: Case study 2 **By:** ____ **Date:** ____

Location: Taupo **Checked:** ____ **Date:** ____

Scenario: Post-developed - imperv **(Pre-developed or post-developed)**

1. Runoff Curve Number (CN) and Initial Abstraction (I_a)

Soil name and classification	Cover description (cover type, treatment and hydrologic condition	Curve Number (CN)	Area (km^2)	Product of CN x Area
-	Impervious surfaces	98	0.016	1.568
		TOTALS	0.016	1.568

CN (weighted) = $\frac{Total\ Product\ of\ CN\ x\ Area}{Total\ Area}$ = 1.568 / 0. 16 = 98

Initial abstraction

$S = \left(\frac{1000}{CN} - 10\right) 25.4 \quad (mm)$ = ((1000 / 98) – 10) x 25.4 = 5.2 mm

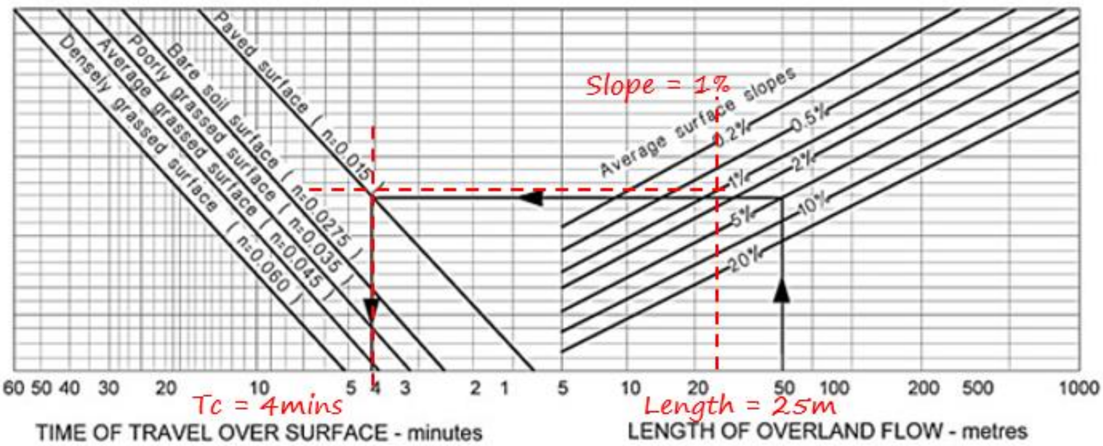
$I_a = 0.05\, S$ = 0.05 * 5.2 = 0.26 mm

2. Time of Concentration (T_c)

(a) Sheet and shallow concentrated flow

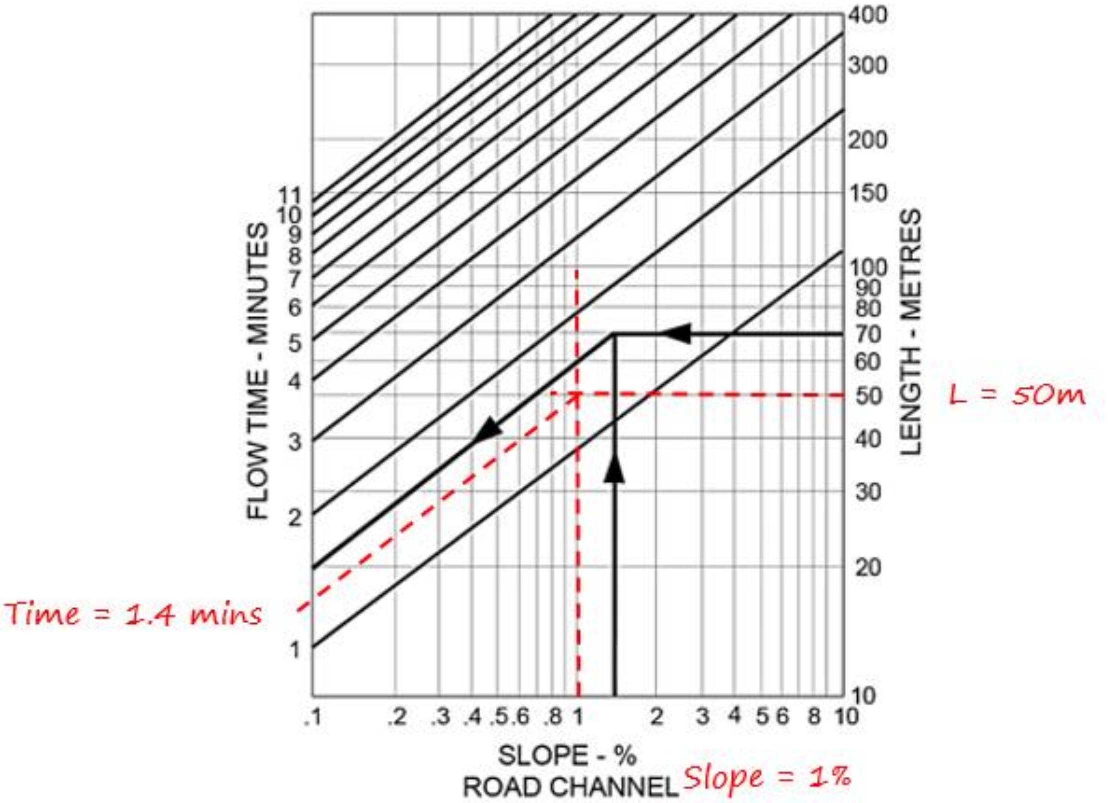
From Equation 7-2 or from Figure 7-1: Length = 25m, at slope of 1%, with paved surface. From Figure 7-1, Tc = 4 minutes

$T_t = \,^{100nL^{0.33}}/_{S^{0.2}}$



(b) Concentrated network flow

i. Road channel flow from Figure 7-2: *Flow along kerbing: approximately 50m length, slope 1%. From Figure 7-2, Tc = 1.4 minutes*



ii. Pipe network flow from Table 7-2 and Figure 7-3: *Flat gradient, typical velocity = 0.6 m/s from Table 7-2. Pipe network is 200m long, hence Tc = 200 m / 0.6 m/s = 333.3 s = 5.5 minutes*

iii. Open channel flow from Equation 7-3: *No open channel flow*

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

(c) Time of concentration

$T_c = T_{t1} + T_{t2} + \cdots..T_{tm} =$ *4 mins + 1.4 mins + 5.5 mins = 10.9 mins = 0.18 hours*

SCS Lag for HEC-HMS = $t_p = \frac{2}{3}t_c$ = *0.18 * 2/3 = 0.12* hours

Worksheet 2: Graphical Peak Flow Rate

Project: Case study 2 **By:** ______ **Date:** ______

Location: Taupo **Checked:** ______ **Date:** ______

Scenario: Post-developed – imperv **(Pre-developed or post-developed)**

3. Data

Catchment area (A) = 0.016 km^2

Runoff curve number (CN) = 98 (from Worksheet 1)

Initial abstraction (Ia) = 0.26 (from Worksheet 1)

Time of concentration (Tc) = 0.18 hours (from Worksheet 1)

4. Storage

Storage (S) = 5.2 mm (from Worksheet 1)

	Storm #1	Storm #2	Storm #3
Average Recurrence Interval ARI (year)	Water quality	2 year	10 year
24-hour rainfall depth P24 (mm)	26.6	79.8	119.2
Compute c*: $c^* = \frac{P_{24} - 2I_a}{P_{24} - 2I_a + 2S}$	Unnecessary for volume calculations		
Specific peak flow rate q* (From Figure 8-1)	Unnecessary for volume calculations		
Peak flow rate q_p: $q_p = q^* A P_{24}$ (m^3/s)	Unnecessary for volume calculations		
Runoff depth Q_{24}: $Q_{24} = \frac{(P_{24} - I_a)^2}{(P_{24} - I_a) + S}$	22	74.7	114
Runoff volume V_{24}: $V_{24} = 1000 x Q_{24} A$ (m^3)	352	1,195	1,824

Worksheet 1: Runoff Parameters and Time of Concentration

Project: Case study 2 **By:** ______ **Date:** ______

Location: Taupo **Checked:** ______ **Date:** ______

Scenario: Post-developed – whole site **(Pre-developed or post-developed)**

1. Runoff Curve Number (CN) and Initial Abstraction (I_a)

Soil name and classification	Cover description (cover type, treatment and hydrologic condition	Curve Number (CN)	Area (km^2)	Product of CN x Area
Orthic pumice	Grass	79	0.004	0.316
-	Impervious surfaces	98	0.016	1.568
		TOTALS	0.02	1.884

CN (weighted) = $\frac{Total\ Product\ of\ CN\ x\ Area}{Total\ Area}$ = 1.884 / 0.02 = 94.2

Initial abstraction

$S = \left(\frac{1000}{CN} - 10\right) 25.4 \quad (mm)$ = ((1000 / 94.2) – 10) x 25.4 = 15.64 mm

$I_a = 0.05\,S$ = 0.05 * 15.64 = 0.78 mm

2. Time of Concentration (T_c)

(a) Sheet and shallow concentrated flow

From Equation 7-2 or from Figure 7-1:

25m flow distance at 1% slope (significant site levelling) on paved surfaces (from Worksheet for Post-development – imperv), Tc = 4 minutes

50m flow distance for grassed areas at 1% slope (from Worksheet for Post-development – pervious) = 13 minutes

Total travel time = 17 minutes

$$T_t = {100nL^{0.33}}/{S^{0.2}}$$

(b) Concentrated network flow:

i. Road channel flow from Figure 7-2: Flow along kerbing, approximately 50m long at 1% road grade (From Worksheet Post-development – Imperv), Tc = 1.4 minutes

ii. Pipe network flow from Table 7-2 and Figure 7-3: *200m of pipe at low gradient, typical velocity = 0.6 m/s from Table 7-2. Pipe network is 200m long, hence Tc = 200 m / 0.06 m/s = 333.3 s = 5.5 minutes*

iii. Open channel flow from Equation 7-3: *No open channel flow*

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

(c) Time of concentration

$T_c = T_{t1} + T_{t2} + \cdots..T_{tm} =$ *17 mins + 1.4 mins + 5.5 mins = 23.9 mins = 0.4 hours*

SCS Lag for HEC-HMS = $t_p = \frac{2}{3}t_c$ = *0.4 * 2/3 = 0.27* hours

Worksheet 2: Graphical Peak Flow Rate

Project: Case study 2 **By:** ______ **Date:** ______

Location: Taupo **Checked:** ______ **Date:** ______

Scenario: Post-developed – whole site **(Pre-developed or post-developed)**

1. Data

Catchment area (A) = 0.02 km^2

Runoff curve number (CN) = 94.2 (from Worksheet 1)

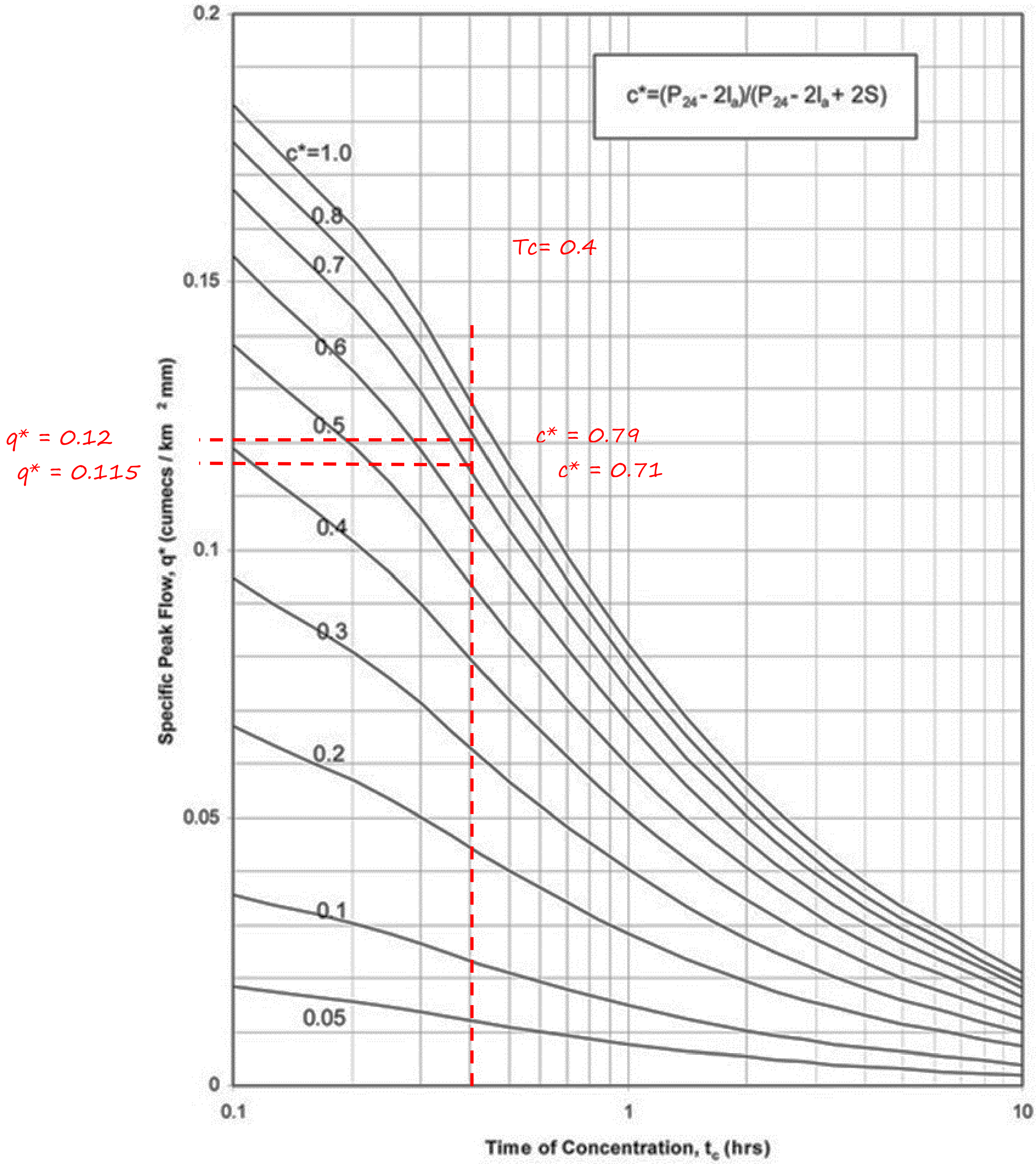
Initial abstraction (Ia) = 0.78 (from Worksheet 1)

Time of concentration (Tc) = 0.4 hours (from Worksheet 1)

2. Storage

Storage (S) = 15.64 mm (from Worksheet 1)

	Storm #1	Storm #2	Storm #3
Average Recurrence Interval ARI (year)		2 year	10 year
24-hour rainfall depth P24 (mm)		79.8	119.2
Compute c*: $c^* = \frac{P_{24} - 2I_a}{P_{24} - 2I_a + 2S}$		0.71	0.79
Specific peak flow rate q* (From Figure 8-1)		0.116	0.12
Peak flow rate q_p: $q_p = q^* A P_{24}$ (m^3/s)		0.185	0.286
Runoff depth Q_{24}: $Q_{24} = \frac{(P_{24} - I_a)^2}{(P_{24} - I_a) + S}$			
Runoff volume V_{24}: $V_{24} = 1000 x Q_{24} A$ (m^3)			



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Tc= 0.4
c* = 0.79
c* = 0.71
q* = 0.12
q* = 0.115
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)
0.2
0.15
0.1
0.05
0
0.1
1
10

References

American Society of Civil Engineers 2009. A review of infiltration standards and practices in Clark County. Prepared by the Infiltration Standards Review Committee, Southwest Washington Branch, Oregon Section. Reston, Virginia, American Society of Civil Engineers.

American Society of Civil Engineers/Environmental Water Resources Institute, American Society of Agricultural and Biological Engineers, 2017. Revision of Chapter 10 of the NRCS National Engineering Handbook.

Auckland Regional Council 1999. Guidelines for stormwater runoff modelling in the Auckland Region. Technical Publication No. 108. Auckland, Auckland Regional Council.

Beca Carter Hollings & Ferner 1999. Methods of analysis for stormwater management design guidelines preparation stage. Prepared for Auckland Regional Council by Beca Carter Hollings & Ferner Ltd.

Carey-Smith T, Henderson R, Singh S, 2018. High Intensity Rainfall Design System, Version 4. Prepared for Envirolink, National Institute of Water & Atmospheric Research Ltd.

Christchurch City Council 2003. Waterways, wetlands and drainage guide, Part B: Design. Christchurch, Christchurch City Council.

Department of Environmental Protection 2004. (Revised 2014 and 2016). NJ Stormwater Best Management Practices Manual, Appendix E, State of New Jersey. Trenton, NJ, Department of Environmental Protection.

McCuen RH 1998. Hydrological analysis and design, 2nd ed. Upper Saddle River, NJ, Prentice Hall.

Ministry for the Environment 2008. Climate change effects and impacts assessment: A guidance manual for local government in New Zealand. Prepared for the Ministry for the Environment by NIWA, MWH NZ Ltd., Earthwise Consulting Ltd, and the Ministry for the Environment, Wellington.

Ministry for the Environment 2016. Climate change projections for New Zealand: Atmosphere projections based on simulations from the IPCC Fifth Assessment. Wellington, Ministry for the Environment.

Natural Resources Conservation Service (previously the Soil Conservation Service) 1986. Urban hydrology for small watersheds. Technical release no. 55. Washington, DC, US Department of Agriculture.

Natural Resources Conservation Service 1985. National engineering handbook, Section 4, Hydrology. Washington, DC, Natural Resources Conservation Service.

Natural Resources Conservation Service 2009. Part 630 Hydrology, national engineering handbook. Washington, DC, Natural Resources Conservation Service.

Waikato Regional Council 2020. Waikato stormwater management guideline. Waikato Regional Council Technical Report 2020/07. Hamilton, Waikato Regional Council.

Yen and Chow, 2016, Table 3-5 of the EPA SWMM Reference Manual Volume 1 – Hydrology (revised).

Zanders JM, McLeod M, Thornburrow D 2001. Pasture to housing: implications of urban development for stormwater infiltration. Proceedings of New Zealand Water and Wastes Association Conference, New Zealand.

Appendix A: Runoff curve numbers

Table 2-2a.-Runoff curve numbers for urban areas[1] (SCS, 1986)

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3] :					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

[1] Average runoff condition, and Ia = 0.2S.
[2] The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
[4] Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 24, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 2-2b.-Runoff curve numbers for cultivated agricultural lands[1] (SCS, 1986)

Cover description			Curve numbers for hydrologic soil group-			
Cover type	**Treatment[2]**	**Hydrologic condition[3]**	**A**	**B**	**C**	**D**
Fallow	Bare soil		77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

[1] Average runoff condition, and Ia = 0.2S.

[2] *Crop residue cover* applies only if residue is on at least 5% of the surface throughout the year.

[3] Hydrologic condition is based on combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes in rotations, (d) percent of residue cover on the land surface (good >_ 20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Table 2-2c.-Runoff curve numbers for other agricultural lands[1] (SCS, 1986)

Cover description		Curve numbers for hydrologic soil group-			
Cover type	**Hydrologic condition**	**A**	**B**	**C**	**D**
Pasture, grassland, or range-continuous forage for grazing.[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow-continuous grass, protected from grazing and generally mowed for hay.		30	58	71	78
Brush-brush-weed-grass mixture with brush the major element.[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[4]30	48	65	73
Woods-grass combination (orchard or tree farm).[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods.[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	[4]30	55	70	77
Farmsteads-buildings, lanes, driveways, and surrounding lots.		59	74	82	86

[1] Average runoff condition, and Ia = 0.2S.

[2] *Poor*: <50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

[3] *Poor*: <50% ground cover.
Fair: 50 to 75% ground cover.
Good: > 75% ground cover.

[4] Actual curve number is less than 30; use CN = 30 for runoff computations.

[5] CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

[6] *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair:. Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Table 2-2d.-Runoff curve numbers for arid and semi-arid rangelands[1] (SCS, 1986)

Cover description		Curve numbers for hydrologic soil group-			
Cover type	Hydrologic condition[2]	A[3]	B	C	D
Herbaceous-mixture of grass, weeds, and	Poor		80	87	93
low-growing brush, with brush the	Fair		71	81	89
minor element.	Good		62	74	85
Oak-aspen-mountain brush mixture of oak brush,	Poor		66	74	79
aspen, mountain mahogany, bitter brush, maple,	Fair		48	57	63
and other brush.	Good		30	41	48
Pinyon-juniper-pinyon, juniper, or both;	Poor		75	85	89
grass understory.	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory.	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub-major plants include saltbush,	Poor	63	77	85	88
greasewood, creosote bush, black brush, bursage,	Fair	55	72	81	86
palo verde, mesquite, and cactus.	Good	49	68	79	84

[1] Average runoff condition, and Ia = 0.2S. For range in humid regions, use table 2-2c.
[2] Poor: < 30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70% ground cover.
Good: >70% ground cover.
[3] Curve numbers for group A have been developed only for desert shrub.

Appendix B: Worksheets

Worksheet 1: Runoff Parameters and Time of Concentration

Project: ______________________ **By:** __________ **Date:** __________

Location: ______________________ **Checked:** __________ **Date:** __________

Scenario: ______________________ **(Pre-developed or post-developed)**

1. Runoff Curve Number (CN) and Initial Abstraction (I_a)

Soil name and classification	Cover description (cover type, treatment and hydrologic condition	Curve Number (CN)	Area (km^2)	Product of CN x Area
		TOTALS		

CN (weighted) = $\frac{Total\ Product\ of\ CN\ x\ Area}{Total\ Area}$ =

Initial abstraction

$S = \left(\frac{1000}{CN} - 10\right) 25.4 \quad (mm)$ =

$I_a = 0.05\, S$ =

2. Time of Concentration (T_c)

(a) Sheet and shallow concentrated flow

From Equation 7-2 or from Figure 7-1:

$$T_t = 100nL^{0.33} / S^{0.2}$$

(b) Concentrated network flow

i. Road channel flow from Figure 7-2:

ii. Pipe network flow from Table 7-2 and Figure 7-3:

iii. Open channel flow from Equation 7-3:

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

(c) Time of concentration

$T_c = T_{t1} + T_{t2} + \cdots .. T_{tm}$ = hours

SCS Lag for HEC-HMS = $t_p = \frac{2}{3} t_c$ = hours

Worksheet 2: Graphical Peak Flow Rate

Project: ______________________ **By:** __________ **Date:** __________

Location: ______________________ **Checked:** __________ **Date:** __________

Scenario: ______________________ **(Pre-developed or post-developed)**

1. Data

Catchment area (A) = ______________ km^2

Runoff curve number (CN) = ______________ (from Worksheet 1)

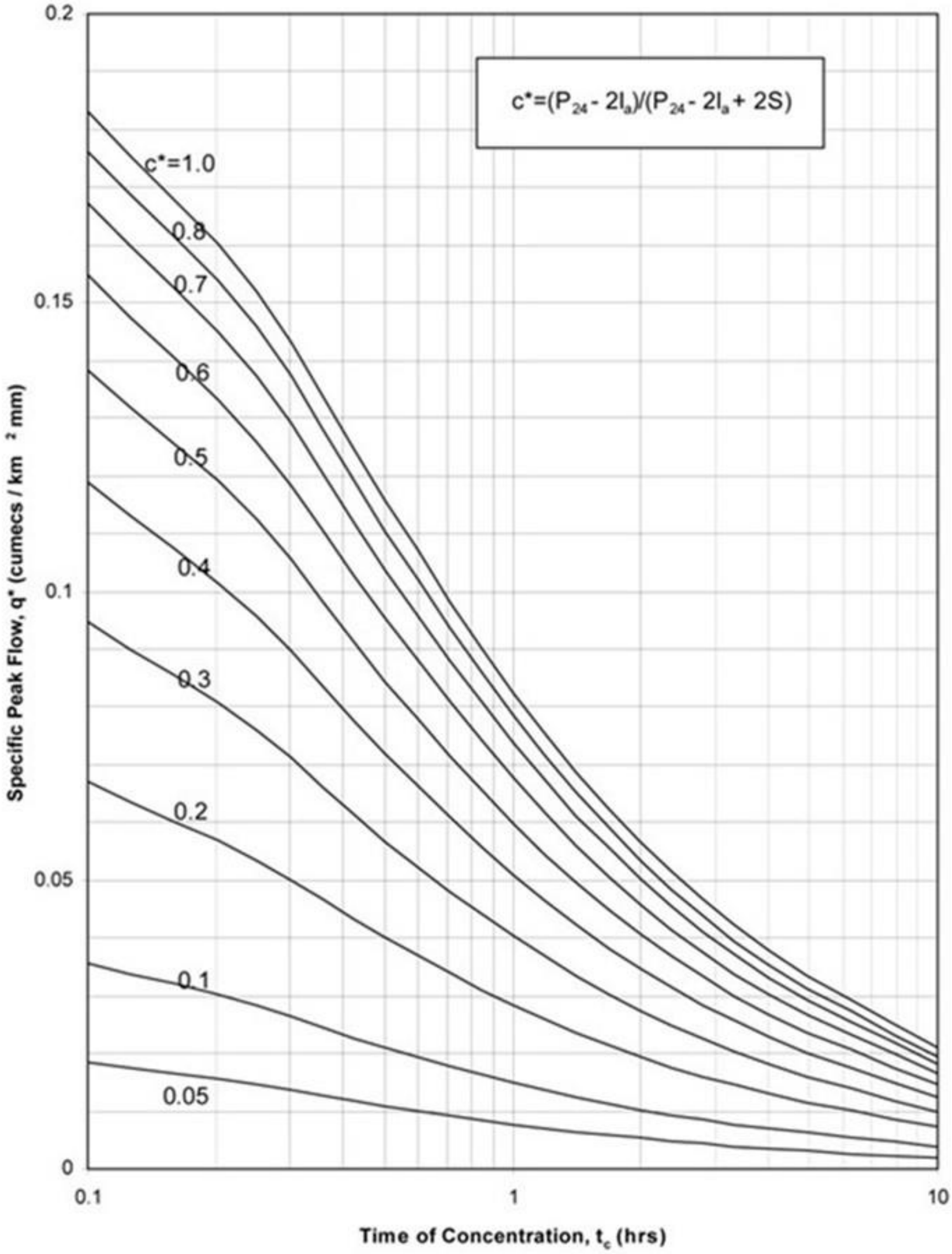
Initial abstraction (Ia) = ______________ (from Worksheet 1)

Time of concentration (Tc) = ______________ hours (from Worksheet 1)

2. Storage

Storage (S) = ______________ mm (from Worksheet 1)

	Storm #1	Storm #2	Storm #3
Average Recurrence Interval ARI (year)			
24-hour rainfall depth P24 (mm)			
Compute c*: $c^* = \frac{P_{24} - 2I_a}{P_{24} - 2I_a + 2S}$			
Specific peak flow rate q* (From Figure 8-1)			
Peak flow rate q_p: $q_p = q^* A P_{24}$ (m^3/s)			
Runoff depth Q_{24}: $Q_{24} = \frac{(P_{24} - I_a)^2}{(P_{24} - I_a) + S}$			
Runoff volume V_{24}: $V_{24} = 1000xQ_{24}A$ (m^3)			


c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km 2 mm)
0.2
0.15
0.1
0.05
0
0.1
1
10
Time of Concentration, tc (hrs)