Wellington Water

Reference Guide for Design Storm Hydrology

Standardised Parameters for Hydrological Modelling

9 April 2019

Our water, our future.

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1 Introduction

1.1 Objective

This guide has been developed for stormwater modelling professionals and land developers to outline the steps involved for estimating flood hydrology for small ungauged urban catchments in the region managed by Wellington Water Limited (WWL). It is intended that the methodology detailed in this guide will be used to assist in understanding the flood flows entering a site, and appropriately plan for and manage the resulting runoff associated with any change in land use. For more detailed explanation of how the hydrological parameters were developed and model calibration, please consult the full report '*Standardised Parameters for Stormwater Modelling*' (Cardno, 2017b).

1.2 Introduction

Urban growth and development typically result in changes to the stormwater runoff from a site. These changes can be divided into four key impacts:

1. Changes to the primary stormwater flowpath. The primary stormwater flowpath can either be natural through stream channels; or artificial through street gutters, stormwater pipes and/or open drainage channels.
2. Changes to the overland flowpath. This is the flowpath of stormwater over land either from rainfall or from waterways that have been blocked or broken their banks.
3. Loss of storage where floodwaters naturally ponded.
4. An increase in the impervious (sealed) area, resulting in an increase in the volume flow rate and timing of stormwater runoff.

Without appropriate design and mitigation, these changes can have an adverse effect on neighbours and downstream users increasing the flood risk to people and property. This guide provides a standard method for calculating the runoff from catchments in the Wellington region. It should be used to quantify the stormwater runoff from a site pre- and post-development, in order to assess the environmental effects associated with a development.

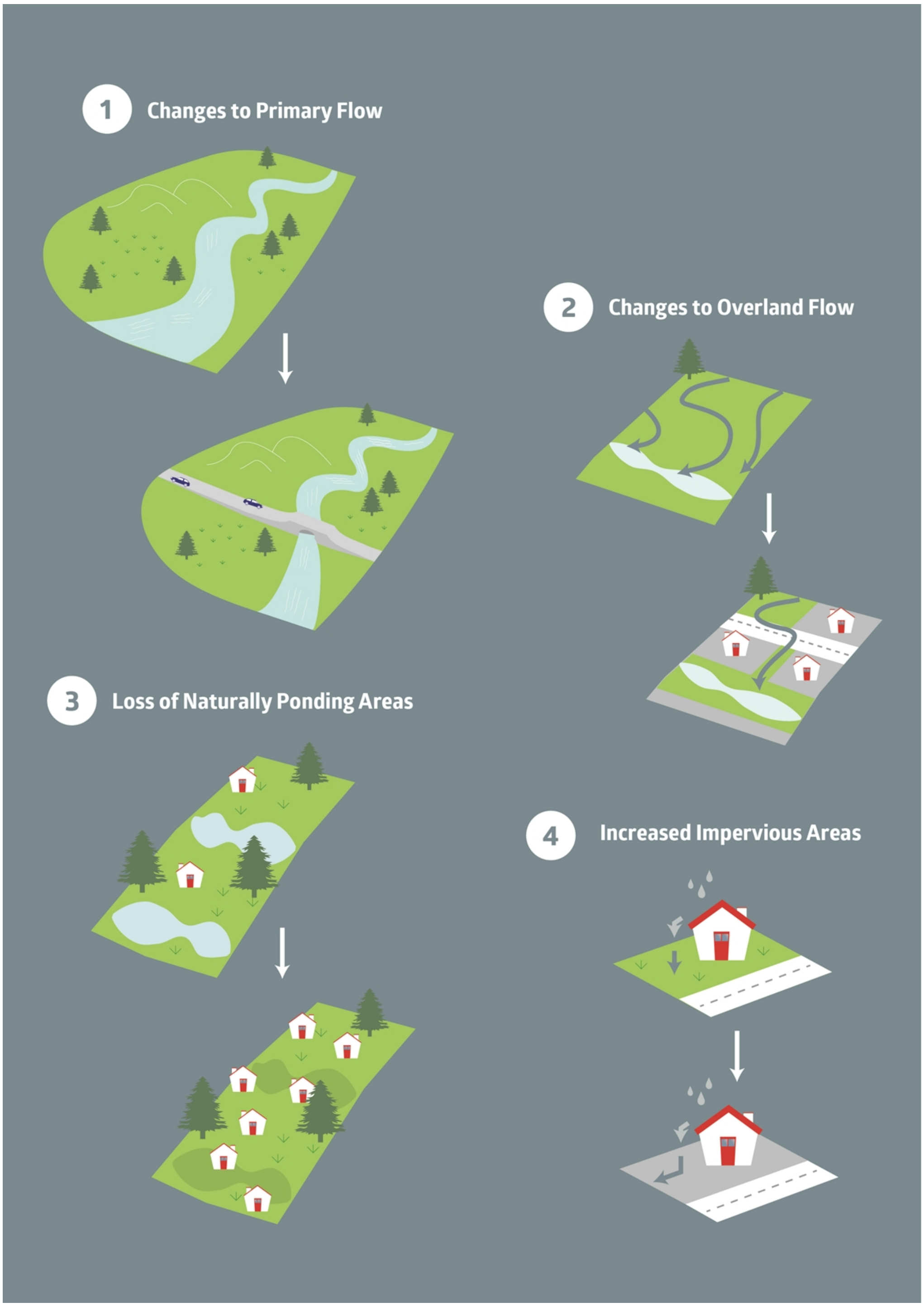

Figure 1-1 Concept diagram of the four key changes caused by urban growth and development which result in changes to stormwater runoff from a site.

1.3 Managing the impact of changes to the imperviousness of a site

In order to limit the adverse effects on neighbouring and downstream users from an increase in stormwater runoff, hydraulic neutrality should be achieved by the development. This means limiting post-development runoff to pre-development levels. In terms of mitigating the flood risk, this is measured at the 10% annual exceedance probability (AEP) (1 in 10-year likelihood of occurrence) through to the 1% AEP (1 in 100-year likelihood of occurrence that includes the predicted impacts of climate change).

The diagram in Figure 1-2 depicts the required steps to estimate the stormwater runoff of significant rainfall events. Section 2 of this report explains how to calculate the model parameters, Section 3 explains how to estimate the rainfall hyetograph, and Section 4 provides a worked example using the hydrological software HEC-HMS (freeware available at http://www.hec.usace.army.mil/software/hec-hms/)

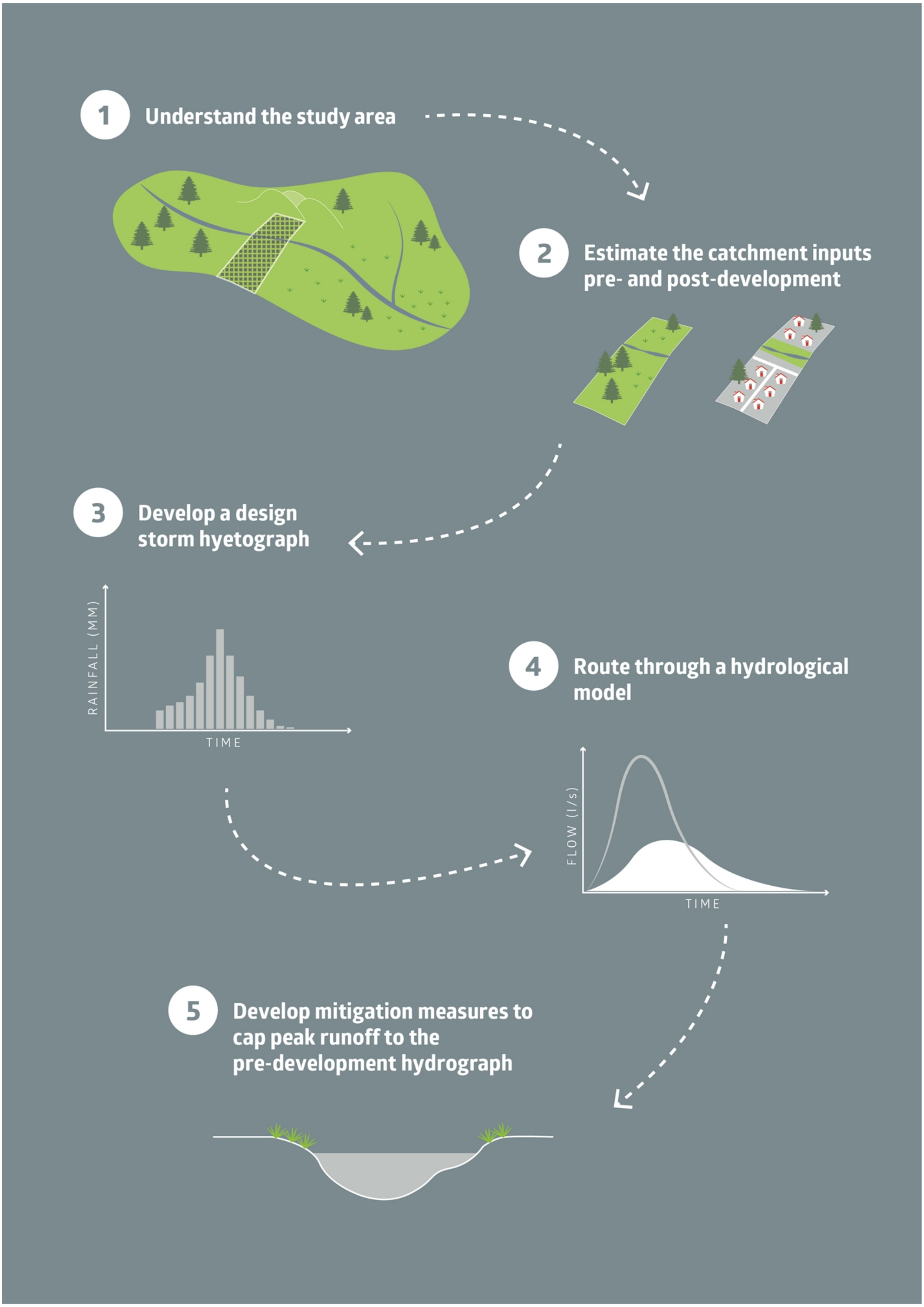

Figure 1-2 Process diagram showing the required steps to estimate and manage catchment runoff from large flood events

The steps are briefly explained below:

1) Understand the study area

   Determine the catchment area of the proposed development and the upstream catchment. Consider the impact any upstream or downstream boundaries may have on runoff leaving the site, and make allowances in your model setup for these controls. For example, if the downstream boundary was an open channel or harbour, runoff from the site may be 'tail-water' controlled (i.e. if the receiving channel was in flood or at high tide, runoff from your site may be restricted).

2) Estimate the catchment inputs pre- and post-development

   The required catchment inputs are:

   - catchment area;
   - curve number;
   - connected impervious area;
   - initial abstraction; and
   - time of concentration.

   See Section 2 and Appendix B for details.

3) Develop design storm hyetographs

   Develop design storm hyetographs for significant rainfall events of magnitudes between a 10% and 1% AEP including the predicted impacts of climate change. See Section 2 and Appendix C for details.

4) Route through a hydrological model

5) Develop mitigation measures to cap peak runoff post-development at pre-development levels

   Mitigation measures may include designing a storage detention pond on site to attenuate flood flows; reducing the impervious area of the development (consider using permeable paving) or reducing peak runoff through the installation of rainwater tanks etc. The design of mitigation measures is not covered by this guideline, though they should be appropriately designed in order to reduce post-development peak runoff to pre-development levels.

These steps should be followed where:

- There is no existing hydraulic model of the catchment. WWL is in the process of developing hydraulic models for urban catchments across the region. These models incorporate more detail than the methodology outlined in this guideline and have been validated to real flood events. They are therefore considered to be more accurate and should be drawn upon where available. Contact the Land Development Manager or the Flood Modelling Manager at WWL for further information.
- The catchment is defined as being a small ungauged urban catchment. To meet this definition, it should have a calculated time of concentration from the catchment divide to the catchment outlet of no more than two hours. (If the catchment is gauged then a real flood event should also be considered to validate the model parameters against. The selected event should be a moderate to large flood resulting from high intensity rainfall.)

The hydrological model being used to estimate catchment runoff is the Soil Conservation Service (SCS) runoff curve number method. This widely used approach was the preferred choice for the region as calibration of the model parameters from three gauged catchments across the region resulted in a good match between modelled and measured flows (Cardno, 2017b); and this method is supported by both InfoWorks and DHI hydraulic modelling software, as well as the freeware HEC-HMS. This methodology may be combined with more detailed hydraulic models to route catchment hydrographs through the drainage network. For complex situations or where more detail is required it is recommended that a full hydraulic model is developed as this should give a better representation of storage and channel routing within the catchment.

This Reference Guide has been developed to address the hydrological component of stormwater modelling. For hydraulic modelling please consult WWL's *Hydraulic Specifications for Stormwater Modelling* document (Capacity Infrastructure Services, 2013).

1.4 SCS runoff curve number method

The hydrological model setup differs slightly depending on the software choice of the analyst, however the results are comparable. Table 1-1 summaries the model setup parameters for three common programmes: InfoWorks ICM, DHI and HEC-HMS.

Table 1-1 Hydrological model setup for difference software programmes

Software	Loss Method	Transformation Method	Initial Abstraction	Curve Number	Antecedent Moisture Conditions (AMC)	Time of Concentration
InfoWorks	CN	SCS Unit	Either SCS factor or absolute value (in m)	Composite curve number	Change between normal and wet AMC by changing SCS Index	SCS-User-Tc (minutes)
DHI	SCS Generalised	SCS Dimensionless	Absolute value (in mm)	Composite curve number	Change between normal and wet AMC by adjusting the curve number	Lag = 0.6 x Tc (hours)
HEC-HMS	SCS Curve Number	SCS Unit Hydrograph	Absolute value (in mm)	Curve number and connected impervious area	Change between normal and wet AMC by adjusting the curve number	Lag = 0.6 x Tc (minutes)

2 SCS Model Parameters

This method uses a number of catchment parameters to estimate the peak and volume of runoff during design storm events. The required parameters are:

a) Catchment area (and areal reduction factors);

b) Curve number (CN) and connected impervious area (CIA);

c) Initial abstraction (I_a);

d) Antecedent moisture condition (AMC); and

e) Time of concentration (Tc).

For import into hydraulic modelling, catchment parameters should be calculated on a subcatchment basis where subcatchments are defined based on topography and the drainage network. Where part of the runoff is routed through a piped network, the analyst should consider the size of this network and location of inlets in defining the catchment boundaries.

It is worth highlighting that often in extreme flood events the piped stormwater network may not be able to convey all floodwaters. In these situations, surface flooding will follow topography. Where is it acknowledged that the stormwater network has limited capacity, care should be taken to accurately represent the catchment boundaries based on the likely flowpath of surface flooding.

2.1 Catchment area

This should be calculated for each subcatchment, noting the required units for input into the software. The minimum catchment area should be determined by the analyst based on the required level of detail necessary for the purpose of the model.

When modelling large rural catchments an aerial reduction factor may be applied. This is to represent the non-uniform spatial distribution of rainfall that can occur within a catchment. It is based on the acknowledgement that for a specified annual exceedance probability event and storm duration, the average rainfall depth over an area is less than the point rainfall depth. Areal reduction factors are specified in Wellington Water’s *Regional Stormwater Hydraulic Modelling Specifications*, Table 5.2. This table has been replicated in Appendix A for ease of reference.

Where the purpose of hydrological modelling is to size infrastructure or develop flood hazard maps based on design storms, areal reduction factors will not typically be applied. This is because the heaviest point of rainfall intensity (for a given magnitude event) may occur anywhere within the catchment, and infrastructure (or planning measures) should be developed for this intensity. For example, a 1% AEP storm at a 30-minute duration may occur in subcatchment A today, through it is equally plausible that this event may occur in subcatchment B. Though it is unlikely that this storm will occur in subcatchment A and B at the same time, infrastructure and planning measures should be developed for the plausible situation of this storm occurring in either subcatchment A or B.

If mitigation measures are being developed to manage the runoff from larger areas, the analyst may be justified in treating the upstream catchment area as one and applying an areal reduction factor to rainfall.

2.2 Curve number

A curve number map has been developed based on the underlying soil drainage characteristics and land cover of the catchment (curve number table and map can be seen in Appendix B).

The soil drainage component was derived from the Land Environments of New Zealand (LENZ) drainage layer and Fundamental Soils data layer (FSL) permeability layer and refined based on local knowledge of the Wellington soils. The land cover component was derived from the Land Cover Database (LCDB v4.1).

The curve number map should be intersected with the subcatchment boundaries and a weighted curve number derived for the pervious component of each subcatchment. Where the analyst is using InfoWorks or DHI software, a composite curve number should be calculated to account for the connected impervious area. In HEC-HMS the connected impervious area should be determined and entered directly into the specified field, alongside the weighted curve number for the pervious component of the catchment.

The weighted curve number can be calculated using [Equation 1].

$$Weighted\ CN = \frac{\sum CN_x A_x}{A_{tot}}$$

[Equation 1]

Impervious areas have a curve number of 98 (Table B.1 in Appendix B). These areas may be either directly connected to the drainage network (connected impervious) or unconnected to the drainage network (unconnected impervious). Lakes, rivers and any other areas that typically have a permanently wetted surface are also considered impervious as runoff cannot infiltrate through the surface. (Any significant storage ponds within a catchment should be specifically accounted for within a hydrological or hydraulic model to determine how runoff may be attenuated within the feature).

Using Figure 2-1, the composite curve number for a connected impervious subcatchment can be determined based on the pervious curve number and the connected impervious area (as a percentage). For example, if 30% of an urban area is impervious (and connected to the drainage network), and the pervious area has a curve number of 60, the resulting composite curve number will be 72.

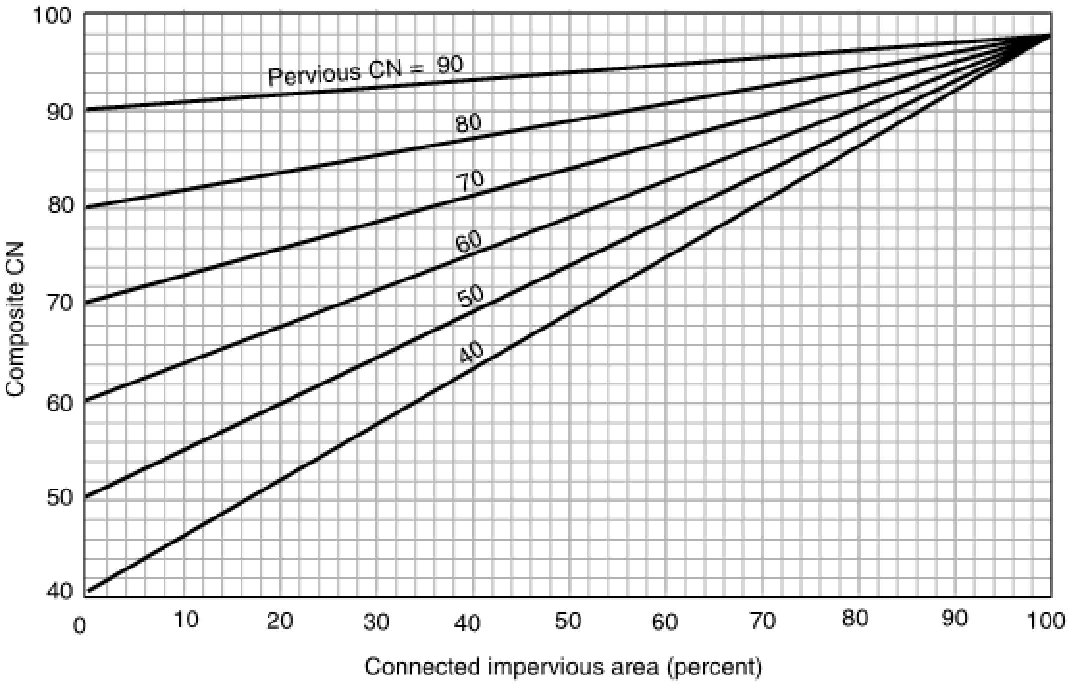

Figure 2-1 Composite curve numbers

Source: United States Department of Agriculture (USDA), 1986

Runoff from impervious areas unconnected to the drainage network is dispersed over the surrounding pervious area as sheetflow. For the purposes of simplicity and noting that in most ungauged urban areas throughout the Wellington region there is very little unconnected impervious areas, these areas can be considered as pervious.

2.3 Initial abstraction

A weighted catchment initial abstraction value should be estimated based on the catchment landuse.

For undeveloped catchments (i.e. rural, pasture, forestry), an initial abstraction coefficient of 0.1 should be used. This is 0.1 of the storage depth [Equation 2], where the storage depth is a function of the curve number [Equation 3]. This value is higher than commonly used in New Zealand, though it has been found to calibrate well with data from the Wellington region. This I_a value accounts for water retained in surface depressions, interception by vegetation, evaporation and infiltration.

$$I_a = 0.1S_t$$

[Equation 2]

The units of I_a is mm

$$S_t = \left(\frac{1000}{CN} - 10\right)25.4$$

[Equation 3]

The units of S_t is mm

For the pervious component of developed catchments, the initial abstraction is likely to be less than $0.1S_t$ as often the pervious component of the catchment has been developed compacting the soils to form grassed road berms, back yards or parks. Based on studies across New Zealand (ARC, 1999; Watts and Hawke, 2003) a value of 5 mm is recommended.

For the impervious component of developed catchments, an initial abstraction value of 0 mm should be used. This may be conservative when modelling flat catchments as water may be retained in surface depressions (more so in areas without a piped stormwater network), though the impact of this on the overall catchment hydrology is likely to be minimal.

The initial abstraction values are summarised in Table 2-1.

Table 2-1 Initial abstraction values

Landuse	Initial Abstraction
Undeveloped (i.e. rural, pasture, forestry)	$0.1 S_t$
Developed pervious	5 mm
Developed impervious	0 mm

In mixed landuse catchments where a portion of the landuse is undeveloped, the I_a for the undeveloped area should be calculated as a function of S to determine the initial abstraction depth in millimetres [Equation 2]. This value can then be weighted based on area with values from the urban portion of the catchment. Where the catchment is entirely rural, InfoWorks software allows the user to directly enter the factor rather than the abstraction depth.

The weighted I_a can be calculated using Equation 4.

$$Weighted\ I_a = \frac{0.1S_{tr}A_r + 0_iA_i + 5_pA_P}{Total\ area}$$

[Equation 4]

Where I_a is in mm;
r is the rural portion of the catchment;

i is the impervious portion of the developed catchment; and
p is the pervious portion of the developed catchment

2.4 Antecedent moisture conditions

For design storms the antecedent moisture conditions should be modelled as 'normal' or AMC II (2).

2.5 Time of concentration

For undeveloped catchments where the length of the main channel exceeds 1000m, the time of concentration should be calculated using the empirical equations of Ramser Kirpich and Bransby Williams, and the answers averaged. This approach most closely aligned with the observed time of concentration of nine events from the three modelled catchments. Ramser Kirpich equation is detailed below in Equation 5, and Bransby Williams in Equation 6.

$$Ramser\ Kirpich\ Tc = \ 0.0195\ L^{0.77} {S_a}^{-0.385}$$

[Equation 5]

Where Tc is in minutes;

L is flow length from the farthest point on the catchment divide to the outlet in metres (m); and

S_a is the average channel slope in metres per metre (m/m)

$$Bransby\ Williams\ Tc = \frac{FL}{A^{0.1}S^{0.2}}$$

[Equation 6]

Where Tc is in minutes;

F in 92.7;

L is length in kilometres (km);

A is the area in hectares (ha); and

S is the slope in metres per kilometre (m/km)

For developed/mixed landuse catchments and undeveloped catchments where the length of the main channel is less than 1000m, the time of concentration should be calculated from the component parts of Tc [Equation 8].

$$Tc\ (component\ parts) = overland\ flow + shallow\ concentrated\ flow + open\ channel\ flow + pipe\ flow$$

[Equation 7]

If the time of concentration is being calculated for release into an urban stormwater network which incorporates detailed modelling of sumps and pipes, a minimum time of concentration of 5 minutes should be set (minimum time of entry into the stormwater network). Where sumps and pipes are not modelled a minimum time of concentration of 10 minutes should be used (minimum time of entry + minimum pipe and channel flow). This is tabulated in Table 3.

Table 2-2 Minimum time of concentration

	Minimum time of concentration
Minimum time of entry Overland and gutter flow. For catchments discharging directly into a hydraulic model where the piped stormwater network and open channel are explicitly modelled.	5 minutes
Minimum time of entry + time of pipe and channel flow Overland, gutter, pipe and channel flow. For catchments where the piped stormwater network and open channels are not explicitly modelled.	10 minutes

Consideration should be given to how runoff from the furthest point in the catchment drains to the outlet in flood conditions (the outlet being the point of discharge from the catchment, or the point of entry into a hydraulic model). In flood conditions runoff may exceed the capacity of the piped stormwater network and travel as gutter flow or channel flow. If the piped stormwater network is incorporated into the hydrological model (rather than explicitly modelled in a hydraulic model), it may be necessary to use a different time of concentration for large storm events if the piped stormwater network forms a significant part of the time of concentration, and the capacity of the piped stormwater network is less than the design event.

To estimate the time of concentration of the component parts, time of overland flow (also often referred to as sheet flow) can be determined using Friend's equation [Equation 8], time of shallow concentrated flow for pervious areas can be estimated from Manning's derived equation for unpaved areas [Equation 9], and for impervious areas from Manning's derived equation for gutter flow [Equation 10]. Time of pipe flow can be estimated as a function of pipe velocity [Equation 11] and time of open channel flow can be estimated from Manning's equation [Equation 11, 12 and 13].

$$Time\ of\ overland\ flow = \frac{107nL^{0.333}}{s^{0.2}}$$

[Equation 8]

Where overland flow is in minutes;

n is Horton's roughness value for the surface;

L is length in metres (m); and

s is slope in percentage (integer i.e. 3.0 for 3%)

Horton's roughness values are similar, though not identical, to Manning's *n*. Horton's roughness values are detailed in Table 2-3.

Table 2-3 Horton's roughness values

Surface Type	Horton's roughness values
Paved	0.015
Bare Soil	0.0275
Poorly Grassed	0.035
Average Grass	0.045
Dense Grass	0.06

Overland flow in urban areas is typically short in the order of 20 to 50 m. In rural residential and rural areas, the length of overland flow may be up to 200 m, thereafter the flow forms small rills, channel and tracks and is referred to as shallow concentrated flow. Table 2-4 provides the maximum recommended length of overland flow.

Table 2-4 Recommended maximum length of overland sheet flow

Surface condition	Assumed maximum flow length (m)
Urban	50
Steep (i.e. >10%) grassland (Horton's n = 0.045)	20
Steep (i.e. > 10%) bushland (Horton's n = 0.035)	50
Medium gradient (approx.. 5%) bushland or grassland	100
Flat (0-1%) bushland or grassland	200

Source: Queensland Urban Drainage Manual, 2013

$$Time\ of\ shallow\ concentrated\ flow = \frac{L}{295\ S^{0.5}}$$

[Equation 9]

Where shallow concentrated flow is in minutes;
L is length in metres (m); and
S is slope in metres per metre (m/m)

Using manning's derived formula to estimate velocity [Equation 12] R_h is assumed to be equal to the depth of flow (wide rectangular channel flow theory). The constant of 295 has been derived assuming a grassed waterway of 0.12 m deep and a Manning's roughness value of 0.05 (multiplied by 60 to convert to minutes).

$$Time\ of\ gutter\ flow = 0.025\,\frac{L}{S^{0.5}}$$

[Equation 10]

Where gutter flow is in minutes
L is length in metres (m); and
s is slope in percentage (i.e. 3 for 3%)

$$Time\ of\ pipe\ flow = \frac{L}{V \times 60}$$

[Equation 11]

Where pipe flow is in minutes based on the velocity of flow;
L is length in metres (m); and
V is 3 m/s for low gradients less than 5%, and 5 m/s for moderate to steep gradients

$$Velocity\ of\ open\ channel\ flow\ (V) = \frac{1}{n}\left(R_h^{2/3}\right)\left(S^{1/2}\right)$$

[Equation 12]

Where the velocity of open channel flow is in metres per second (m/s);
n is Manning's *n*;
R_h is hydraulic radius; and
S is the bottom slope of the channel in metres per metre (m/m)

Where R_h can be calculated from

$$R_h = \frac{A}{P}$$

[Equation 13]

where A is the cross-sectional area in metres squared (m^2); and
P is the wetted perimeter of the cross-sectional area of flow in metres (m)

Time of open channel flow can therefore be estimated from channel length over velocity.

$$Time\ of\ open\ channel\ flow = \frac{L}{V \times 60}$$

[Equation 14]

Where time of open channel flow is in minutes;
L is length in metres (m); and
V is in metres per second (m/s)

3 Nested storm rainfall profile

Based on analysis of the time of concentration of typical ungauged urban catchments and tested in various model validations (Cardno, 2017b), a 12-hour storm duration is recommended for model runs, in most instances. The exception to this is when the total flood volume is of interest. For example, where floodwaters cannot freely drain from the system and are being stored on the floodplain or where the primary stormwater network at the downstream end of the model is still running at capacity (the volume of runoff into the system exceeding the rate at which the system can drain), so the flood extent may still be increasing. In circumstances where flood volume is paramount, or where smaller magnitude events (i.e. less than a 10% AEP) are being considered as part of the design, a 24-hour nested design storm should also be tested.

Where flood storage or detention infrastructure is part of the catchment, appropriate consideration should be given to the impact this may have on catchment runoff. Where significant flood storage or detention infrastructure is being considered (i.e. attenuating runoff from over 500 properties or similar) an alternative rainfall runoff methodology appropriate for the routing and detention of runoff should also be used to verify results from this method.

Upon review of a number of storm profiles from across the region, it was determined that the short duration storm rainfall intensity peaks (on average using the average variability methodology) at approximately 67% of the total storm duration (or 8 hours through a 12-hour storm) (Cardno, 2017a). For design storm events, the HIRDS rainfall depths for a given magnitude event should be nested around this point. This will give a temporal distribution rainfall profile similar to that in Figure 3-1 and a cumulative plot similar to Figure 3-2. Appendix C provides further detail on how to disaggregate the HIRDS rainfall intensity into a nested storm profile.

The rainfall depth estimates need to be adjusted by 20% to allow for the estimated effects of climate change. This is consistent with the Ministry for the Environment September 2018 publication (*Climate Change Projections for New Zealand: Atmosphere Projections Based on Simulations from the IPCC Fifth Assessment, 2nd edition)* which suggests that the average annual temperature increase will be between 0.7 and 3.0 degrees by 2100.

HIRDS rainfall depth estimates are accessible through the internet (https://hirds.niwa.co.nz/).

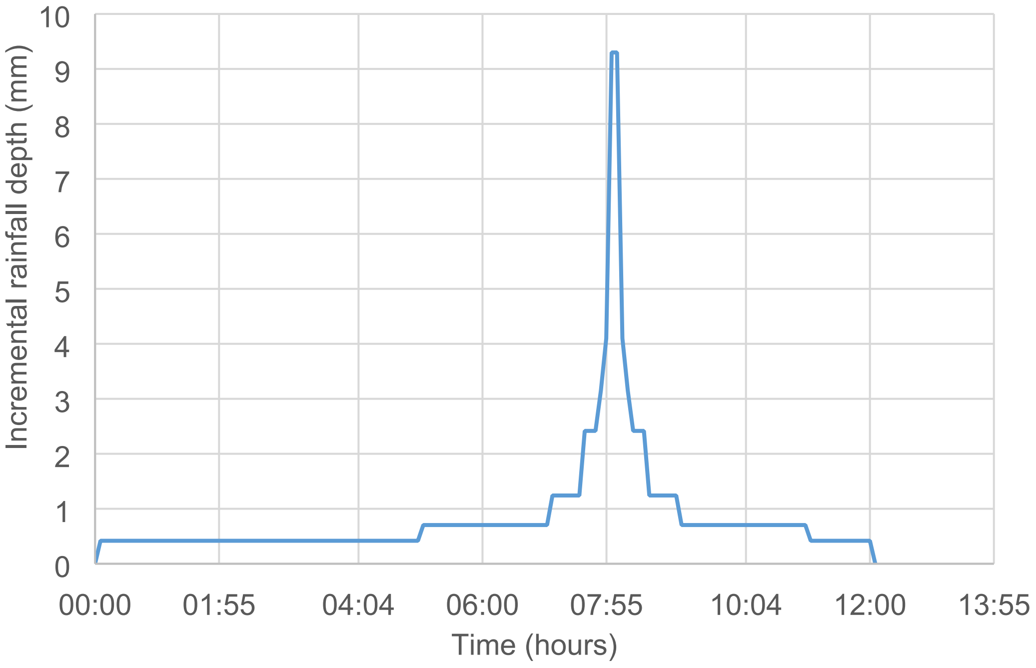

Figure 3-1 Nested storm rainfall distribution for a 12-hour duration storm and total rainfall depth of 126 mm

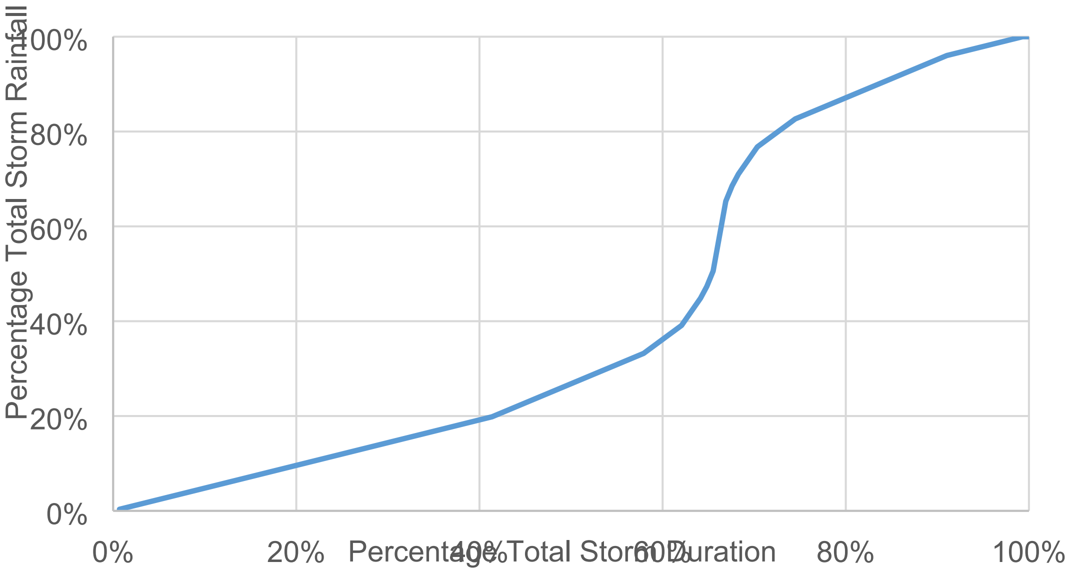

Figure 3-2 Nested storm cumulative rainfall distribution

4 Worked example

This section provides a worked example of how to determine the SCS model parameters and flood hydrograph. This example assumes an 11.9 ha area of land is to be developed from the existing pasture/forestry landuse into low-density residential housing. The proposed development will have a connected impervious area of 35%. For the development to be hydraulically neutral, the hydrology of the catchment pre- and post-development is determined, and then mitigation measures should be developed to cap post-development runoff to the pre-development hydrograph. In this example, flood hydrographs are developed for the 10% AEP and 1% AEP including the predicted impacts of climate change events. Mitigation measures are not developed.

Following the steps from Figure 1-2 and Section 1.3, the pre- and post-development flood hydrology is determined in Sections 4.1 to Figure 4-13 below.

4.1 1) Understand the study area

The study area is defined by the catchment boundary displayed in Figure 4-1. In terms of understanding the study area, a number of questions should be answered, including:

- Are there any upstream or downstream controls that should be taken into account when modelling runoff from the proposed development site?
- Are there significant features within the catchment that may influence the flood hydrograph that should be explicitly accounted for (e.g. retention dams, bridges or culverts that may throttle flood flows etc.)?
- Does the catchment need to be divided into subcatchments? (Developing subcatchments may be necessary to reflect the natural or proposed stormwater drainage paths within the catchment, and to size and mitigate the effects of changes to flowpaths and storage areas.)
- How are changes to the rainfall intensity and tidal boundary, attributed to the possible effects of climate change, being managed?

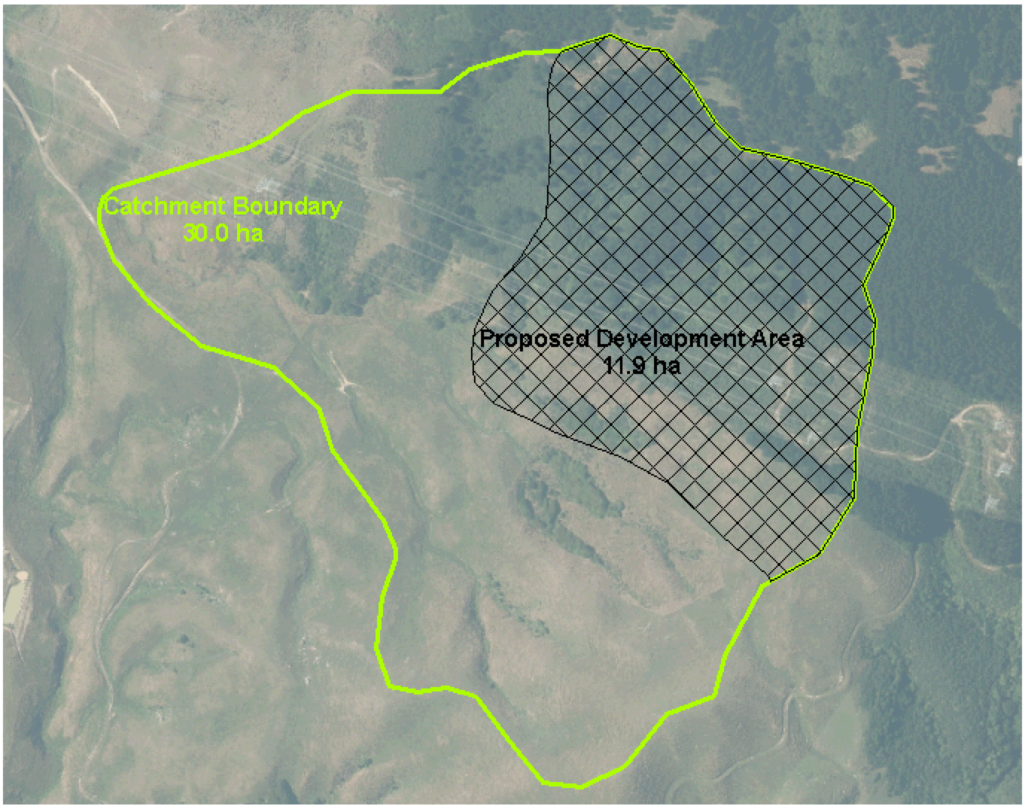

Figure 4-1 Catchment boundary

The following assessment has been made with respect to understanding the study area (Table 4-1)

Table 4-1 Understanding the study area

Assessment
The proposed development site is in the upper reaches of the catchment. No upstream or downstream controls are likely to influence runoff from this area. Rainfall depth estimates will be increased by 20% to allow for the effects of climate change. As mitigation measures are not part of this guideline, the catchment has not being divided into subcatchments.

4.2 2) Estimate the catchment inputs

The five catchment input parameters are detailed below.

4.2.1 Catchment area

The catchment area remains the same between both the pre- and post-development scenarios (Table 4-2).

Table 4-2 Catchment area pre- and post-development

Pre-Development		Post-Development	
Calculation	Value	Calculation	Value
Area (Figure 4-1)	11.9 ha, or 0.119 km^2	Same as pre-development	11.9 ha, or 0.119 km^2

4.2.2 Curve number and connected impervious area

The curve number has been estimated through intersecting the catchment area with the curve number layer (Figure 4-4). A weighted curve number value has been determined.

Depending on the modelling software being used by the analyst, the connected impervious area of 35% post-development may need to be incorporated within the curve number value or entered directly into a separate field in the software. Within the DHI and InfoWorks software programmes, the connected impervious area should be incorporated into the curve number value using the composite curve number relationship in Figure 2-1 (and shown in the worked example in Figure 4-5). Within HEC-HMS, either a composite curve number can be used (and a connected impervious area of 0) or the actual curve number and actual connected impervious area.

Table 4-3 Curve number value pre- and post-development

Pre-Development		Post-Development	
Calculation	Value	Calculation	Value
$Weighted\ CN = \frac{\sum CN_x A_x}{A_{tot}}$ [Equation 1]	Based on the values in Figure 4-4: $CN = \frac{(63x1.7) + (68x6.9) + (72x3.3)}{11.9}$ $= 68.4$ (with a CIA of 0)	DHI or InfoWorks software: Composite CN based on Figure 2-1	Composite CN based on Figure 4-4 and Figure 4-5 = 78
		HEC-HMS software: CN of 68.4 and CIA of 35%	CN = 68.4 CIA = 35%

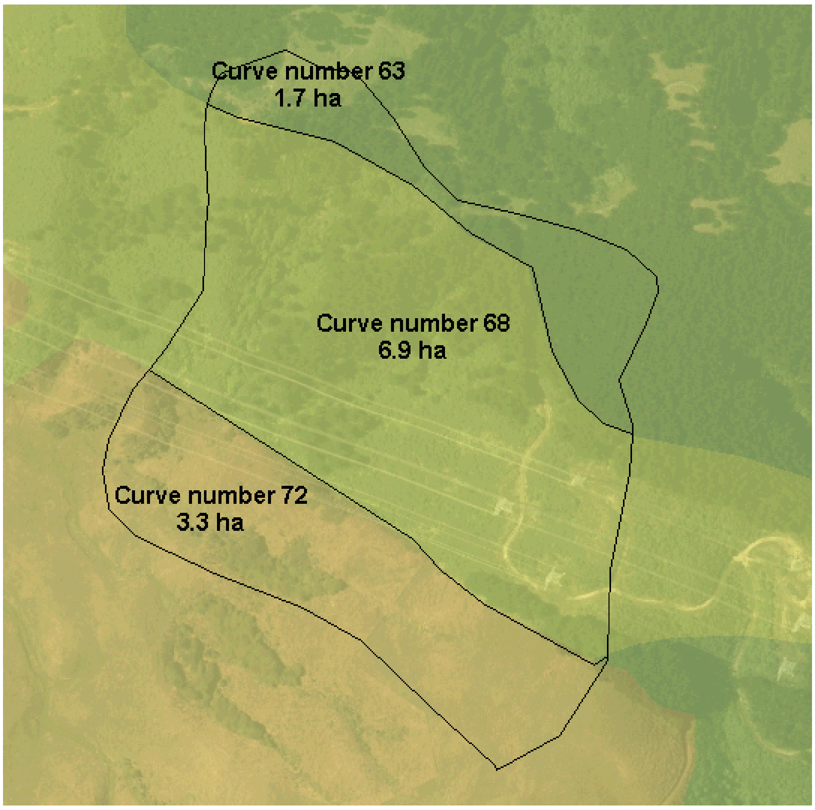

Figure 4-4 Intersection of the proposed development site with the curve number layer

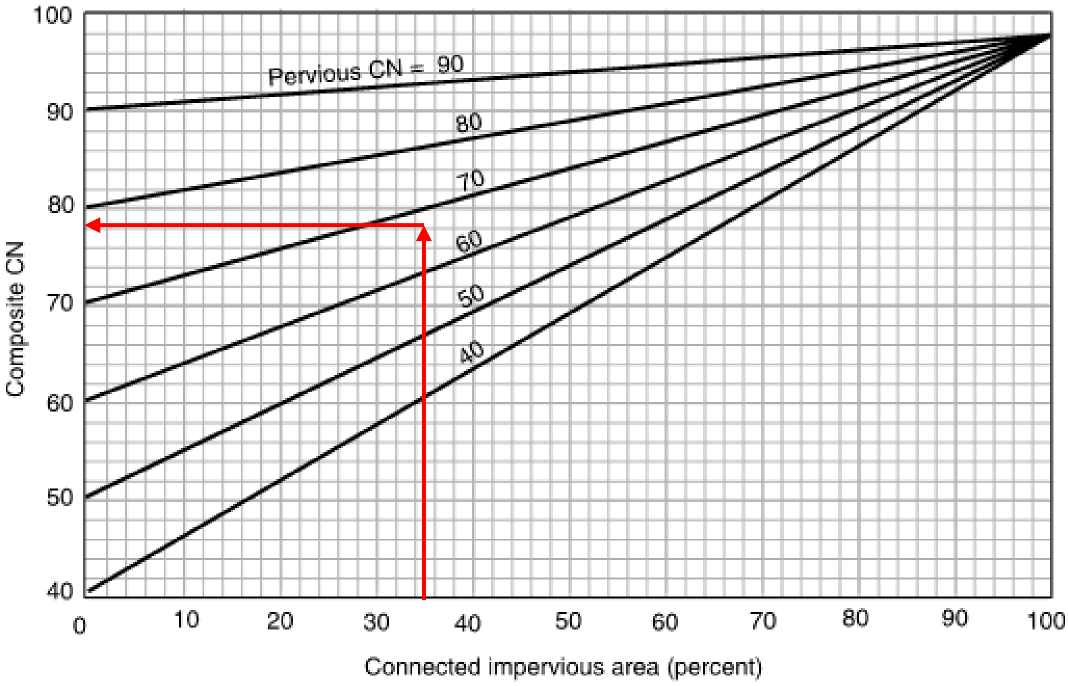

Figure 4-5 Composite curve number based on a CN of 68 and CIA of 35%

4.2.3 Initial abstraction

Pre-development the site is considered to be undeveloped, therefore initial abstraction is based on Equation 2. Post-development the site is developed with 35% of the area impervious, and 65% of the area pervious. Initial abstraction is a weighted value based on Equation 4. Table 4-4 shows the calculations.

Table 4-4 Initial abstraction pre- and post-development

Pre-Development		Post-Development	
Calculation	**Value**	**Calculation**	**Value**
In a rural catchment: $I_a = 0.1S_t$ [Equation 2] Where: $S_t = \left(\frac{1000}{CN} - 10\right)25.4$ [Equation 3]	Based on a weighted catchment CN of 68.4: $I_a = 0.1\left(\frac{1000}{68.4} - 10\right)25.4$ $= 11.7\ mm$	$Weighted\ I_a = \frac{0.1S_{tr}A_r + 0_i}{Total\ a}$ [Equation 4]	$Weighted\ I_a$ $= 0x0.35 + 5x0.65$ $= 3.25\ mm$ (Calculation slightly different to Equation 4 by using the percentage impervious/ percentage pervious proportions. Therefore calculation not divided by the total area.).

4.2.4 Antecedent moisture conditions

Always ‘normal’ or AMCII, unless an observed (gauged) flood event is being used for validation. If this is the case, estimate the antecedent moisture conditions based on the preceding rainfall in the catchment prior to the event and the stream baseflow.

Table 4-5 Antecedent moisture conditions pre- and post-development

Pre- and Post-Development
Normal or AMCII

4.2.5 Time of concentration

As the catchment length from divide to outlet is less than 1000 m, the time of concentration is calculated based on the component parts, with the equations worked through in 0.

$$Tc\ (component\ parts) = overland\ flow + shallow\ concentrated\ flow + open\ channel\ flow + pipe\ flow$$

[Equations 7-14]

Table 4-6 Time of concentration pre- and post-development

Pre-Development		Post-Development	
Calculation	**Value**	**Calculation**	**Value**
$Time\ of\ overland\ flow$ $= \frac{107nL^{0.333}}{s^{0.2}}$ [Equation 8]	Where: n = dense grass 0.06 (Table 2-3) L = maximum length for overland flow through steep bushland is 50 m (Table 2-4). s = 24. Based on: - upstream elevation 240 m - downstream elevation 228 m - length 50 m $slope\ (\%) = \frac{(240 - 228)}{50}x100$ $= 24$ So: $Time\ of\ overland\ flow$ $= \frac{107x0.06x50^{0.333}}{24^{0.2}}$ $= 12.5\ minutes$	The time of concentration will be dependent on how the stormwater drainage paths are altered (based on the proposed layout/design of the development). The resulting time of concentration will likely be between the pre-development value of 16.1 minutes and the minimum value of 10 minutes. (For the purpose of this worked example, 10 minutes is assumed).	10 minutes
$Time\ of\ shallow\ concentrated\ flow$ $= \frac{L}{295\ S^{0.5}}$ [Equation 9]	Where: L = 95 m (Figure 4-9) S = 0.295. Based on: - upstream elevation 228 m - downstream elevation 200 m - length 95 m $slope\ = \frac{(228 - 200)}{95}$ $= 0.295\ m/m$ So: $Time\ of\ shallow\ concentrated\ flow$ $= \frac{95}{295x0.295^{0.5}}$ $= 0.6\ minutes\ (36\ seconds)$		
$Velocity\ of\ open\ channel\ flow\ (V)$ $= \frac{1}{n}(R_h^{2/3})(S^{1/2})$ [Equation 12] Where R_h can be calculated from	Where: n = 0.12 R_h = 0.27. Based on: $R_h = \frac{1}{3.66}$ S = 0.248. Based on: - upstream elevation 200 m - downstream elevation 123 m		

Pre-Development		Post-Development	
Calculation	**Value**	**Calculation**	**Value**
$R_h = \frac{A}{P}$ [Equation 13] So $Time\ of\ open\ channel\ flow$ $= \frac{L}{V \times 60}$ [Equation 14]	- length 95 m $slope = \frac{(200 - 123)}{310}$ $= 0.248\ m/m$ So: $Velocity\ of\ open\ channel\ flow\ ($ $= \frac{1}{0.12}(0.27^{2/3})(0.248^{1/2})$ $= 1.7\ m/s$ Therefore: $Time\ of\ open\ channel\ flow$ $= \frac{310}{1.7 \times 60}$ $= 3\ minutes$		
$Tc\ (component\ parts) = overland\ flo$ [Equation 7]	$Tc\ (component\ parts)$ $= 12.5 + 0.6 + 3$ $= 16.1\ minutes$		

Figure 4-9 Time of concentration component parts

4.3 3) Develop design storm hyetograph

The design storm hyetographs for the 10% and 1% AEP events have been developed from the HIRDS depth-duration-frequency rainfall estimates to the centroid of the catchment, and the nested storm profile in Appendix C. The 100-year storm event values have been adjusted by 20%. This figure is based on current guidance from the Ministry for the Environment (2018). The rainfall depth estimates are displayed in Table 4-7, and the nested storm rainfall distributions in Figure 4-5.

Table 4-7 HIRDS rainfall depth estimates

	10% AEP	1% AEP climate change
Duration (minutes)	**Depth (mm)**	**Depth (mm)**
10	10.1	20.0
20	14.7	29.2
30	18.3	36.2
60	26.5	52.6
120	35.8	70.2
360	57.8	111.2
720	78.1	148.6

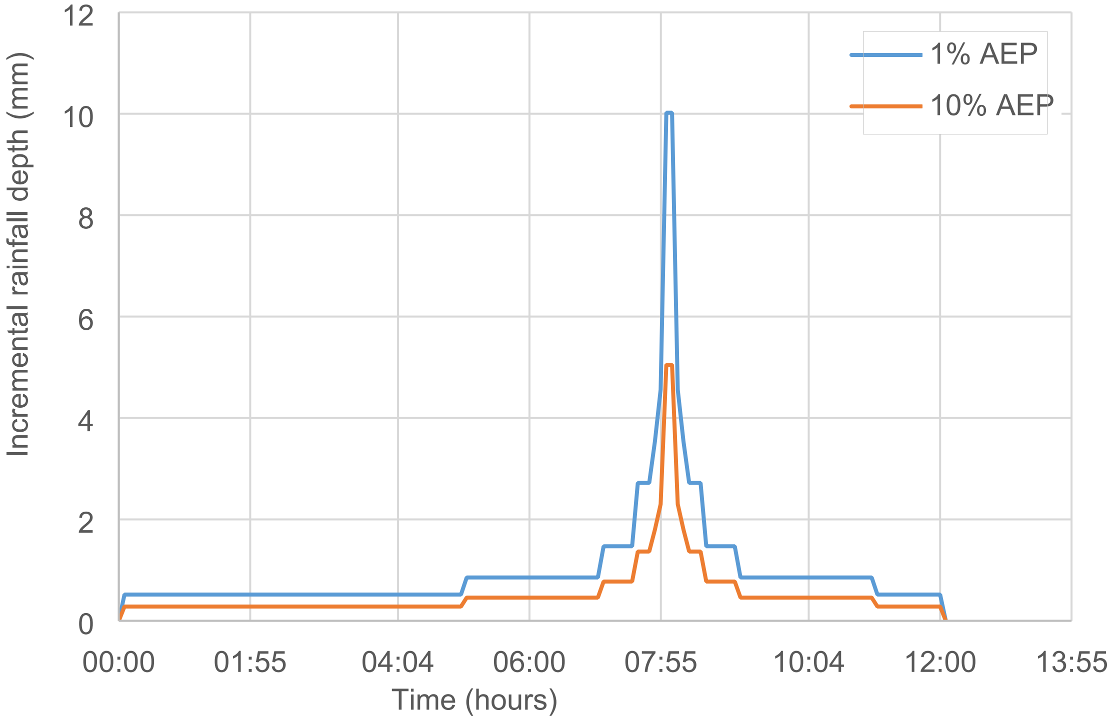

Figure 4-11 Nested storm rainfall distribution for the 10% AEP and the 1% AEP (including an allowance for climate change) design storms

4.4 4) Route though a hydrological model

Using the catchment input parameters from Section 4.2, and the rainfall hyetographs from Section 4.3, the following results are produced when routed through HEC-HMS: Table 4-8 provides a summary of the change in peak and volume, while Figure 4-13 and Figure 4-14 shows the flood hydrographs. (The setup of HEC-HMS was as per Table 1-1. When using HEC-HMS to estimate catchment runoff, the software requires a lag time rather than time of concentration. This is equivalent to 0.6 of the time of concentration.)

Table 4-8 Summary results

	10% AEP		1% AEP plus climate change	
	Pre-Development	Post-Development	Pre-Development	Post-Development
Peak (m^3/s)	0.58	1.06	1.84	2.52
Volume (m^3)	2857	5509	8802	12443

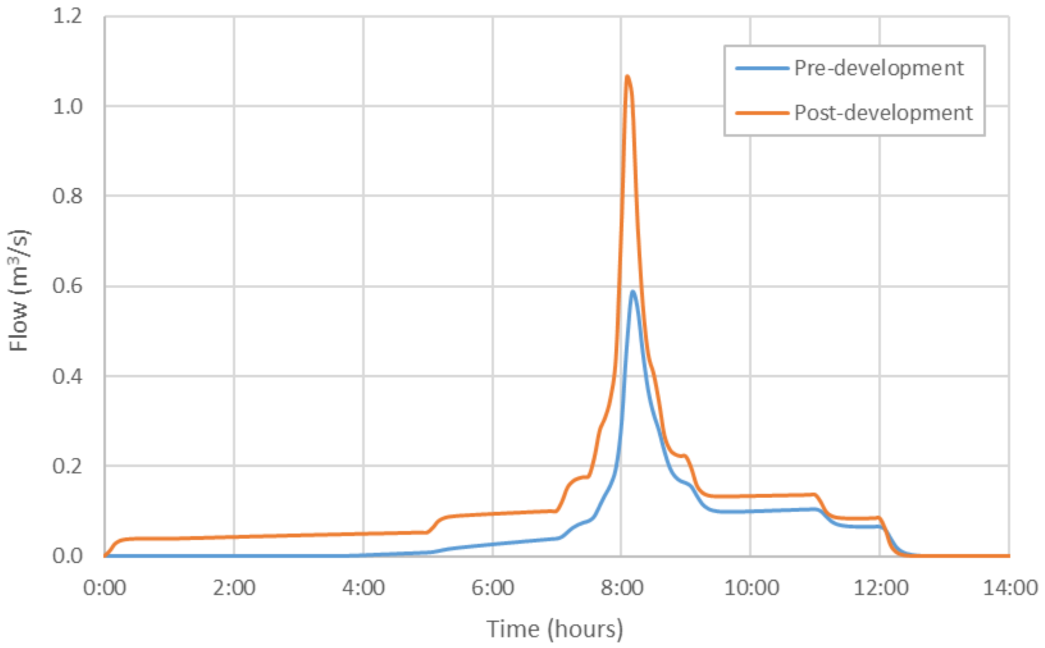

Figure 4-13 Pre- and post-development flood hydrographs in 10% AEP nested storm flood event

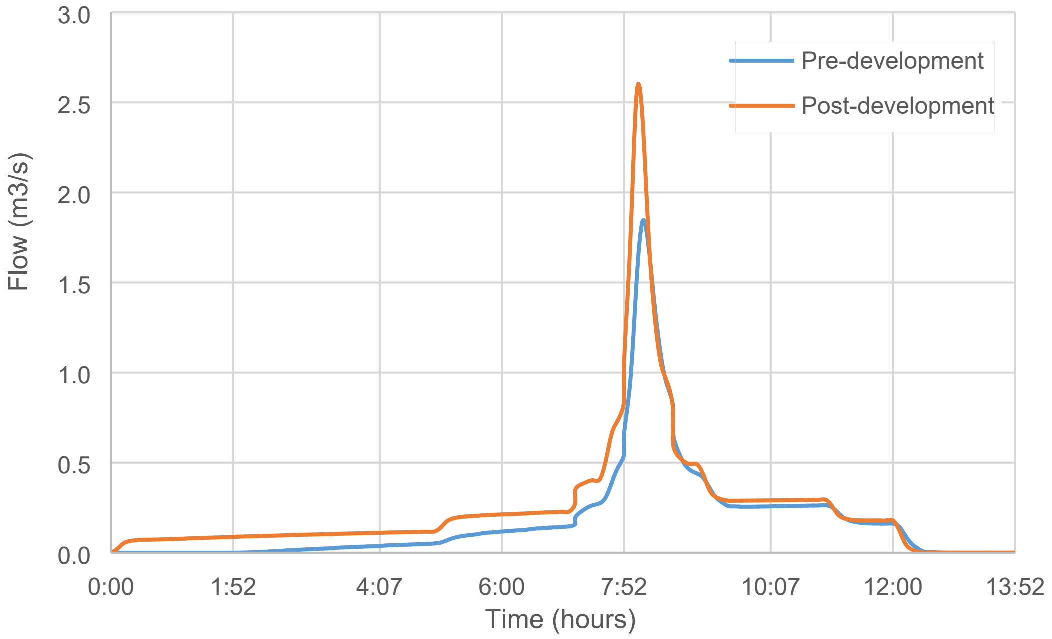

Figure 4-14 Pre- and post-development flood hydrographs in 1% AEP (including an allowance for climate change) nested storm flood event

4.5 5) Develop mitigation measures

The analyst is required to develop mitigation measures to cap peak runoff and volume post-development at the pre-development levels.

5 References

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United States Department of Agriculture (USDA), 1986. Urban hydrology for small watersheds: TR55. USA.

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Appendix A: Areal Reduction Factors

Table A.1 Areal Reduction Factors

Area (km^2)	Duration (hours)								
	0.5	**1**	**2**	**4**	**6**	**12**	**24**	**48**	**96**
1	0.95	0.96	0.97	0.98	0.98	0.98	0.99	0.99	0.99
2	0.94	0.95	0.96	0.97	0.97	0.98	0.98	0.99	0.99
5	0.91	0.93	0.95	0.96	0.96	0.97	0.98	0.98	0.99
10	0.88	0.91	0.93	0.95	0.95	0.97	0.97	0.98	0.98
20	0.85	0.89	0.91	0.93	0.94	0.96	0.97	0.97	0.98
50	0.79	0.84	0.88	0.91	0.92	0.94	0.96	0.97	0.97
100	0.74	0.80	0.85	0.89	0.90	0.93	0.94	0.96	0.97
200	0.68	0.76	0.82	0.86	0.88	0.91	0.93	0.95	0.96
500	0.60	0.69	0.76	0.82	0.85	0.88	0.91	0.93	0.95

Source: Regional Stormwater Hydraulic Modelling Specifications v4, December 2013. Table 5.2 (after WRC, 1995)

Appendix B: Curve Number Tables and Map

Table B.1 Curve number values used to formulate the SCS curve number map

	SOIL GROUP			
LAND COVER	A Sand, loamy sand, or sandy loam (low runoff potential)	B Silt loam or loam	C Sandy clay loam	D Clay loam, silty clay loam, sandy clay, silty clay, or clay (high runoff potential)
Alpine tussock/grass	66	77	84	87
Bare	66	77	84	87
Forest	28	46	63	71
Impervious	98	98	98	98
Pasture-Crop	37	59	72	78
Scrub/Flax	33	54	68	75
Urban Open Space	37	59	72	78

Appendix C: Nested storm profile

Assuming the HIRDS depth duration frequency data for a 10% AEP event is as per Table C.1, then the nested storm profile can be calculated as per Table C.2.

Table C.1 Example data of HIRDS rainfall intensity for a 10% AEP magnitude event

Duration (minutes)	Depth (mm)
10	10.7
20	15.9
30	20.1
60	30.0
120	43.8
360	79.7
720	116.3

Table C.2 Example data showing the distribution of HIRDS rainfall intensity data nested around a midpoint of 8 hours

Start Time	End Time	Percentage of rainfall intensity		Example rainfall depth (mm)
0:00	0:05	0.014 (1/72th)	of 12 hour intensity less 6 hour intensity	0.51
0:05	0:10	0.014	of 12 hour intensity less 6 hour intensity	0.51
0:10	0:15	0.014	of 12 hour intensity less 6 hour intensity	0.51
0:15	0:20	0.014	of 12 hour intensity less 6 hour intensity	0.51
0:20	0:25	0.014	of 12 hour intensity less 6 hour intensity	0.51
0:25	0:30	0.014	of 12 hour intensity less 6 hour intensity	0.51
0:30	0:35	0.014	of 12 hour intensity less 6 hour intensity	0.51
0:35	0:40	0.014	of 12 hour intensity less 6 hour intensity	0.51
0:40	0:45	0.014	of 12 hour intensity less 6 hour intensity	0.51
0:45	0:50	0.014	of 12 hour intensity less 6 hour intensity	0.51
0:50	0:55	0.014	of 12 hour intensity less 6 hour intensity	0.51
0:55	1:00	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:00	1:05	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:05	1:10	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:10	1:15	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:15	1:20	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:20	1:25	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:25	1:30	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:30	1:35	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:35	1:40	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:40	1:45	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:45	1:50	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:50	1:55	0.014	of 12 hour intensity less 6 hour intensity	0.51
1:55	2:00	0.014	of 12 hour intensity less 6 hour intensity	0.51
2:00	2:05	0.014	of 12 hour intensity less 6 hour intensity	0.51
2:05	2:10	0.014	of 12 hour intensity less 6 hour intensity	0.51
2:10	2:15	0.014	of 12 hour intensity less 6 hour intensity	0.51
2:15	2:20	0.014	of 12 hour intensity less 6 hour intensity	0.51
2:20	2:25	0.014	of 12 hour intensity less 6 hour intensity	0.51
2:25	2:30	0.014	of 12 hour intensity less 6 hour intensity	0.51
2:30	2:35	0.014	of 12 hour intensity less 6 hour intensity	0.51
2:35	2:40	0.014	of 12 hour intensity less 6 hour intensity	0.51
2:40	2:45	0.014	of 12 hour intensity less 6 hour intensity	0.51

2:45	2:50	0.014	of 12 hour intensity less 6 hour intensity	0.51
2:50	2:55	0.014	of 12 hour intensity less 6 hour intensity	0.51
2:55	3:00	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:00	3:05	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:05	3:10	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:10	3:15	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:15	3:20	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:20	3:25	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:25	3:30	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:30	3:35	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:35	3:40	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:40	3:45	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:45	3:50	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:50	3:55	0.014	of 12 hour intensity less 6 hour intensity	0.51
3:55	4:00	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:00	4:05	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:05	4:10	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:10	4:15	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:15	4:20	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:20	4:25	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:25	4:30	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:30	4:35	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:35	4:40	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:40	4:45	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:45	4:50	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:50	4:55	0.014	of 12 hour intensity less 6 hour intensity	0.51
4:55	5:00	0.014	of 12 hour intensity less 6 hour intensity	0.51
5:00	5:05	0.021 (1/48th)	of 6 hour intensity less 2 hour intensity	0.75
5:05	5:10	0.021	of 6 hour intensity less 2 hour intensity	0.75
5:10	5:15	0.021	of 6 hour intensity less 2 hour intensity	0.75
5:15	5:20	0.021	of 6 hour intensity less 2 hour intensity	0.75
5:20	5:25	0.021	of 6 hour intensity less 2 hour intensity	0.75
5:25	5:30	0.021	of 6 hour intensity less 2 hour intensity	0.75
5:30	5:35	0.021	of 6 hour intensity less 2 hour intensity	0.75
5:35	5:40	0.021	of 6 hour intensity less 2 hour intensity	0.75
5:40	5:45	0.021	of 6 hour intensity less 2 hour intensity	0.75
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6:00	6:05	0.021	of 6 hour intensity less 2 hour intensity	0.75
6:05	6:10	0.021	of 6 hour intensity less 2 hour intensity	0.75
6:10	6:15	0.021	of 6 hour intensity less 2 hour intensity	0.75
6:15	6:20	0.021	of 6 hour intensity less 2 hour intensity	0.75
6:20	6:25	0.021	of 6 hour intensity less 2 hour intensity	0.75
6:25	6:30	0.021	of 6 hour intensity less 2 hour intensity	0.75
6:30	6:35	0.021	of 6 hour intensity less 2 hour intensity	0.75
6:35	6:40	0.021	of 6 hour intensity less 2 hour intensity	0.75
6:40	6:45	0.021	of 6 hour intensity less 2 hour intensity	0.75
6:45	6:50	0.021	of 6 hour intensity less 2 hour intensity	0.75
6:50	6:55	0.021	of 6 hour intensity less 2 hour intensity	0.75
6:55	7:00	0.021	of 6 hour intensity less 2 hour intensity	0.75
7:00	7:05	0.083 (1/12th)	of 2 hour intensity less 1 hour intensity	1.15
7:05	7:10	0.083	of 2 hour intensity less 1 hour intensity	1.15
7:10	7:15	0.083	of 2 hour intensity less 1 hour intensity	1.15
7:15	7:20	0.083	of 2 hour intensity less 1 hour intensity	1.15
7:20	7:25	0.083	of 2 hour intensity less 1 hour intensity	1.15
7:25	7:30	0.083	of 2 hour intensity less 1 hour intensity	1.15

7:30	7:35	0.167 (1/6th)	of 60 minute intensity less 30 minute intensity	1.65
7:35	7:40	0.167	of 60 minute intensity less 30 minute intensity	1.65
7:40	7:45	0.167	of 60 minute intensity less 30 minute intensity	1.65
7:45	7:50	0.5 (1/2)	of 30 minute intensity less 20 minute intensity	2.1
7:50	7:55	0.5 (1/2)	of 20 minute intensity less 10 minute intensity	2.6
7:55	8:00	0.5 (1/2)	of 10 minute intensity	5.35
8:00	8:05	0.5 (1/2)	of 10 minute intensity	5.35
8:05	8:10	0.5 (1/2)	of 20 minute intensity less 10 minute intensity	2.6
8:10	8:15	0.5 (1/2)	of 30 minute intensity less 20 minute intensity	2.1
8:15	8:20	0.167 (1/6th)	of 60 minute intensity less 30 minute intensity	1.65
8:20	8:25	0.167	of 60 minute intensity less 30 minute intensity	1.65
8:25	8:30	0.167	of 60 minute intensity less 30 minute intensity	1.65
8:30	8:35	0.083 (1/12th)	of 2 hour intensity less 1 hour intensity	1.15
8:35	8:40	0.083	of 2 hour intensity less 1 hour intensity	1.15
8:40	8:45	0.083	of 2 hour intensity less 1 hour intensity	1.15
8:45	8:50	0.083	of 2 hour intensity less 1 hour intensity	1.15
8:50	8:55	0.083	of 2 hour intensity less 1 hour intensity	1.15
8:55	9:00	0.083	of 2 hour intensity less 1 hour intensity	1.15
9:00	9:05	0.021 (1/48th)	of 6 hour intensity less 2 hour intensity	0.75
9:05	9:10	0.021	of 6 hour intensity less 2 hour intensity	0.75
9:10	9:15	0.021	of 6 hour intensity less 2 hour intensity	0.75
9:15	9:20	0.021	of 6 hour intensity less 2 hour intensity	0.75
9:20	9:25	0.021	of 6 hour intensity less 2 hour intensity	0.75
9:25	9:30	0.021	of 6 hour intensity less 2 hour intensity	0.75
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9:35	9:40	0.021	of 6 hour intensity less 2 hour intensity	0.75
9:40	9:45	0.021	of 6 hour intensity less 2 hour intensity	0.75
9:45	9:50	0.021	of 6 hour intensity less 2 hour intensity	0.75
9:50	9:55	0.021	of 6 hour intensity less 2 hour intensity	0.75
9:55	10:00	0.021	of 6 hour intensity less 2 hour intensity	0.75
10:00	10:05	0.021	of 6 hour intensity less 2 hour intensity	0.75
10:05	10:10	0.021	of 6 hour intensity less 2 hour intensity	0.75
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10:15	10:20	0.021	of 6 hour intensity less 2 hour intensity	0.75
10:20	10:25	0.021	of 6 hour intensity less 2 hour intensity	0.75
10:25	10:30	0.021	of 6 hour intensity less 2 hour intensity	0.75
10:30	10:35	0.021	of 6 hour intensity less 2 hour intensity	0.75
10:35	10:40	0.021	of 6 hour intensity less 2 hour intensity	0.75
10:40	10:45	0.021	of 6 hour intensity less 2 hour intensity	0.75
10:45	10:50	0.021	of 6 hour intensity less 2 hour intensity	0.75
10:50	10:55	0.021	of 6 hour intensity less 2 hour intensity	0.75
10:55	11:00	0.021	of 6 hour intensity less 2 hour intensity	0.75
11:00	11:05	0.014 (1/72th)	of 12 hour intensity less 6 hour intensity	0.51
11:05	11:10	0.014	of 12 hour intensity less 6 hour intensity	0.51
11:10	11:15	0.014	of 12 hour intensity less 6 hour intensity	0.51
11:15	11:20	0.014	of 12 hour intensity less 6 hour intensity	0.51
11:20	11:25	0.014	of 12 hour intensity less 6 hour intensity	0.51
11:25	11:30	0.014	of 12 hour intensity less 6 hour intensity	0.51
11:30	11:35	0.014	of 12 hour intensity less 6 hour intensity	0.51
11:35	11:40	0.014	of 12 hour intensity less 6 hour intensity	0.51
11:40	11:45	0.014	of 12 hour intensity less 6 hour intensity	0.51
11:45	11:50	0.014	of 12 hour intensity less 6 hour intensity	0.51
11:50	11:55	0.014	of 12 hour intensity less 6 hour intensity	0.51
11:55	12:00	0.014	of 12 hour intensity less 6 hour intensity	0.51
12:00	12:05	0		0