Sunshine Coast COUNCIL

A Review of Simple Peak Flow Estimation Methods

for use on the Sunshine Coast
following the release of ARR 2016

Document Information

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Introduction

ARR Book 3 Peak Flow Estimation (2016) does not recommend use of the rational method. It considers it an arbitrary design method and states "*the continued use of arbitrary design methods and information cannot be justified*". It recommends "*that an at-site Flood Frequency Analysis (FFA) be used for estimation of the design peak flood discharges quantiles*". In situations where there is no observed data of a suitable quality for at-site Flood Frequency Analysis it recommends "*Regional Flood Frequency Estimation (RFFE) techniques be applied*" and provides a draft (RFFE) model for this purpose. Where estimation of the full flood hydrograph is required, flood modelling techniques outlined in ARR Book 5 Flood Hydrograph Estimation (2016) are recommended.

Sunshine Coast Council (SCC) is concerned for the accuracy of the draft ARR RFFE and the practicality of requiring hydrologic modelling for basic urban stormwater calculations (sizing pits, pipes and cross drainage).

This report considers the suitability of draft ARR RFFE tool and the continued use of the rational method on the Sunshine Coast. The consideration is based upon comparison with reliable 10% and 1% AEP peak flow estimates at a number of gauging stations located within the Sunshine Coast region.

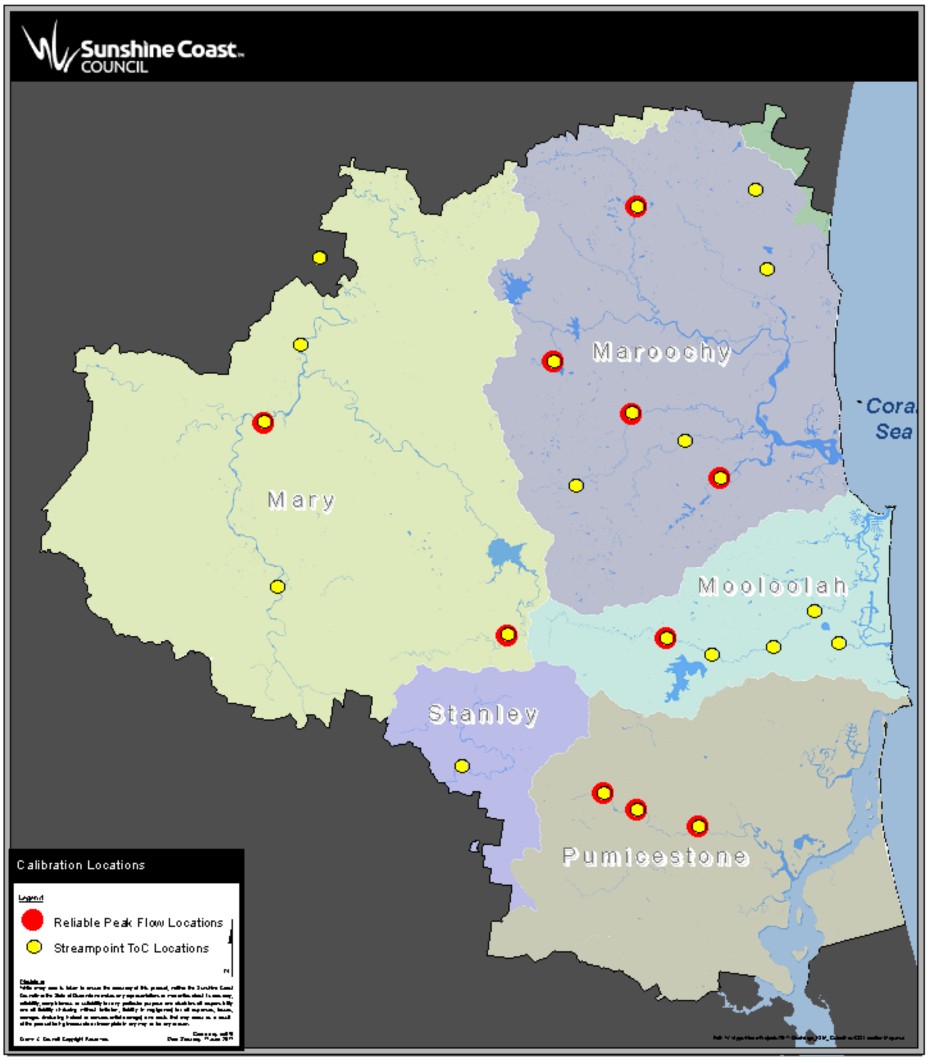


Figure 1 Locations used to inform this Study

1 Regional Flood Frequency Estimation (RFFE)

Sunshine Coast Council collated peak flow estimates for 10% and 1% AEPs from 12 gauged locations on the Sunshine Coast where reasonable data was available.

The catchment areas of these 12 locations ranged from 18.6km^2 to 485km^2. Of these 12 gauged locations it was observed that 1 was influenced from the backwater of a downstream structure and the other was influenced by an upstream water supply dam.

10% AEP peak flow estimates were adopted from Flood Frequency Analysis (FFA) and modelled information. 1% AEP peak flow estimates were adopted from modelling only. All modelled estimates were from recent modelling in the Pumicestone (WBM, 2017), Maroochy (WBM, 2017), Mooloolah (Cardno, 2015) and Mary (DHI, 2012) catchments. Importantly, flow and flood frequency estimates benefited from the ratings derived from hydraulic models and modelled results have been calibrated to FFA at all 12 locations, with fitting bias applied based upon length of record.

ARR RFFE estimates were derived for each of the 10 locations using the draft RFFE tool in advanced mode (available at http://rffe.arr-software.org/)

The 10% AEP and 1% AEP estimates of peak flow were plotted against area. A power curve was fit through the FFA or modelled data in each case. These fits are shown in the following Figures.

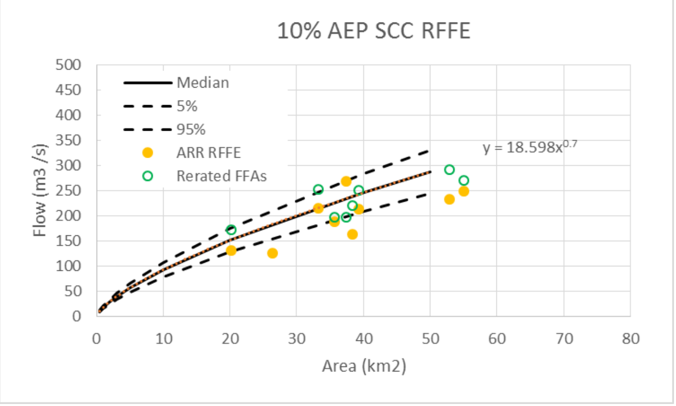


Figure 2 10% AEP FFA and ARR RFFE

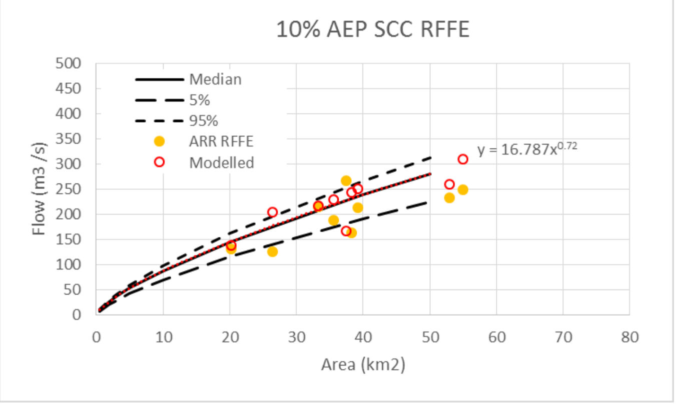


Figure 3 10% AEP Modelled and ARR RFFE

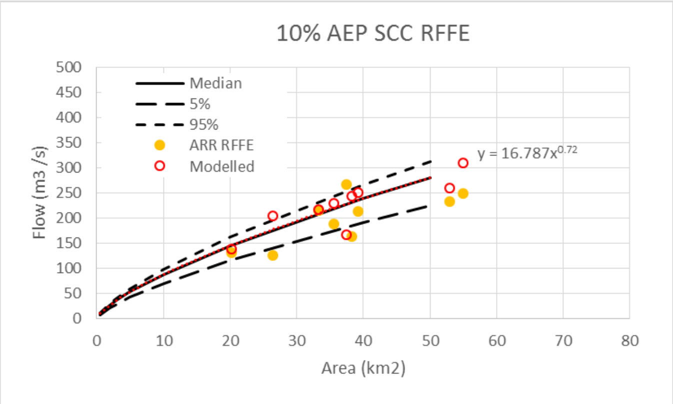


Figure 4 1% AEP Modelled and ARR RFFE

1.1 SCC RFFE

The fitted median curves were considered for a Sunshine Coast Regional Flood Frequency Estimation formula. It was decided that the 10% AEP would be based upon FFA estimates and the 1% AEP would be based upon modelled values.

The 39% AEP Peak flow was not considered in the determination of a SCC RFFE as the uncertainty associated with the FFA estimates, which when based upon partial series estimation methods would ordinarily be the preferred estimate. In addition 39% AEP peak flows will have greater sensitivity to interception losses within catchments, it is expected that this would make fitting an RFFE more difficult.

The formula of the SCC RFFE (2017) is therefore:

$$Q_{10\%\ AEP} = 18.598.Area^{0.70}$$

$$Q_{1\%\ AEP} = 22.951.Area^{0.80}$$

This method is not recommended for catchments less than 15km^2, given the analysis which has derived this method has not considered catchments areas smaller than 18.6km^2.

In developing the SCC RFFE (2017) a relationship with design 24hr rainfall intensity was also considered, as Gold Coast City Council had found this to be a dependent variable when developing their RFFE. It was observed that predictive capability of the SCC RFFE was only marginally improved when rainfall intensity was included and it was decided to exclude it to keep the method as simple as possible.

1.2 Comparative Performance of SCC and ARR RFFEs

Peak flow estimates of the ARR and SCC RFFEs were plotted against the adopted peak flows for the 12 gauged locations. This is shown in Figure 5 and Figure 6 for the 10% and 1% AEPs respectively.

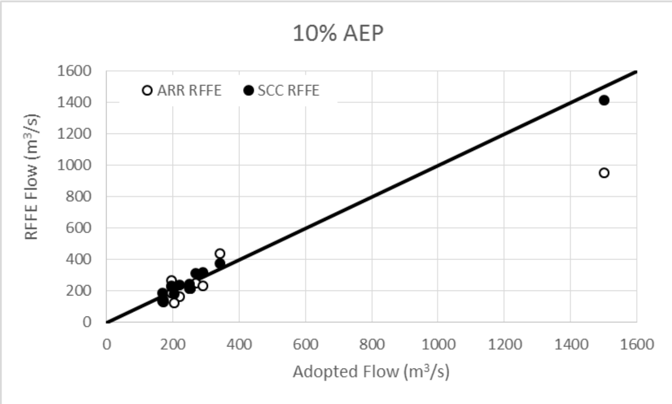


Figure 5 Comparison of SCC and ARR RFFE at 10% AEP

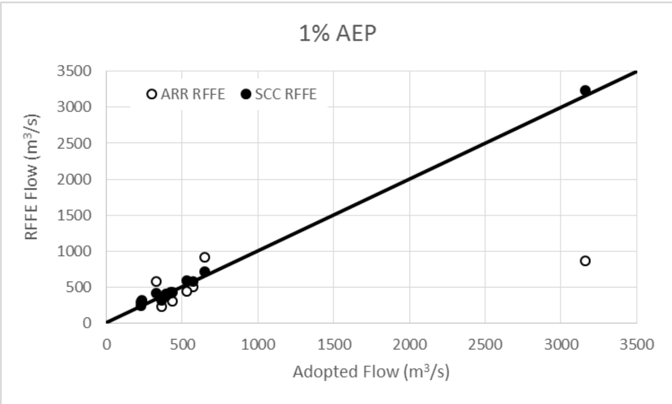


Figure 6 Comparison of SCC and ARR RFFE at 1% AEP

It is observed that the ARR (2016) RFFE is not a particularly good estimator of peak flows for Sunshine Coast Catchments. The coefficient of determination is 0.927 for the 10% AEP and 0.424 for the 1% AEP. (0.672 for the 10% AEP and 0.48 for the 1% AEP, with the outlier removed).

It is suggested that this is due to the ARR RFFE database of peak flows being undermined by poor ratings. SCC has observed that most flood studies undertaken on the Sunshine Coast require ratings to be re-derived from hydraulic models for gauged locations.

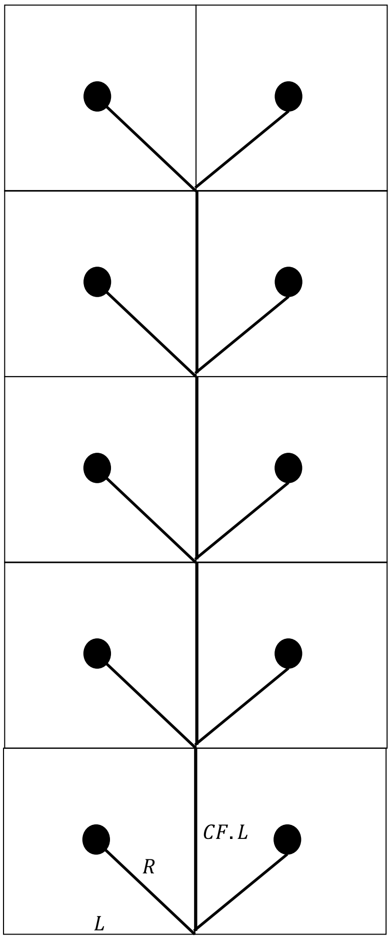
Comparatively the SCC RFFE shows better correlation with adopted peak flow data. The coefficient of determination is 0.993 for the 10% AEP and 0.997 for the 1% AEP. (0.841 for the 10% AEP and 0.898 for the 1% AEP, with the outlier removed).

2 Time of Concentration and the Rational Method

SCC recognised the value of the simplicity of the rational method in supporting urban stormwater calculations, however SCC also recognises the weakness of the method in relying on a Time of Concentration (ToC) methods of questionable veracity for the region. Accepting the recommendation of ARR (2016) for peak flow estimates to be based on modelling, SCC has explored the idea of deriving regional ToC estimates from a comprehensive multivariate modelling investigation. At completion this involved a total of 18,810 discrete model runs.

2.1 Multivariate Model Analysis (Concentrated/Channelised Flow)

A notional URBS model that included channel and catchment routing was developed to analyse model combinations of area (0.001 to 40km^2), slope (0.5-10%) and fraction impervious (0 to 90%). The model adopts Nambour IFD rainfall applied as a Duration Independent Storm (DIS) temporal pattern over 24 hours and over 10 equal area sub-catchments with reach lengths determined from area in accordance with Figure 7.



$$L = \sqrt{\left(\frac{Area}{10}\right)/CF}$$

$$R = 0.5\sqrt{L^2 + (CF.L)^2}$$

$$Dav = R + 2.CF.L$$

Figure 7 Notional Catchment Layout

The model adopted the α, β and m parameters of the Pumicestone Catchment model (BMT WBM, 2017b).

Table 1 Notional URBS Model Parameters

α	m	β
0.25	0.78	2.8

The model was calibrated with a fraction impervious (0%) to the 1% AEP SCC RFFE, with an emphasis on matching the Mooloolah River Gauge (39km^2) 1% AEP Flood Frequency Estimate of 423m^3/s as this gauge's flood frequency has benefited from significant rating review. It was observed that a CF value of 2.6 needed to be applied to the notional model to match the D_{av} of the URBS model at this location. When this was applied with a 30% proportional loss applied to the 24hr DIS temporal pattern the peak flow estimated was 420m^3/s. 0% fraction impervious was specified as the 1% AEP SCC RFFE has been derived from predominantly natural catchments.

The method was cross-checked for agreement with the 10% AEP SCC RFFE. The agreement was good with the only adjustments being 10% AEP rainfall and a 33% proportional loss.

The high proportional loss is an outcome of applying areal reduction factors to the longer durations of the IFD prior to calculating the DIS pattern. This was done to increase sensitivity to fraction impervious.

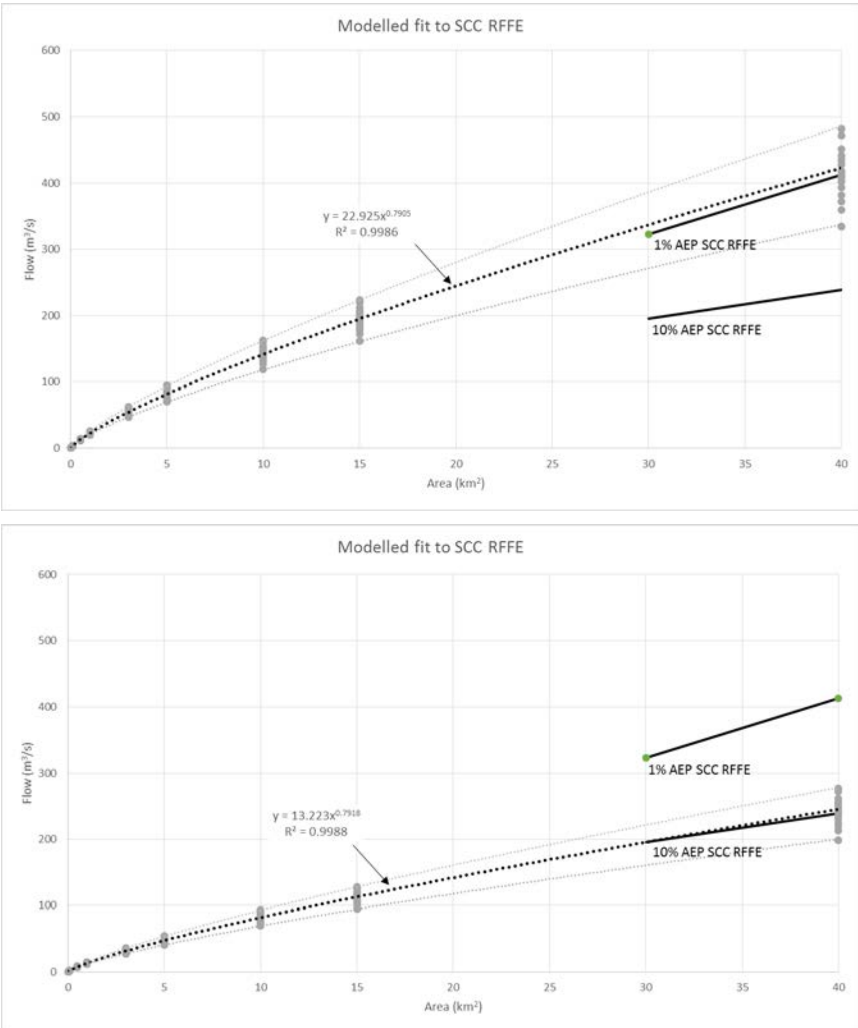


Figure 8 Model Fits to SCC RFFE

The model returned peak flow. Peak flow was converted to intensity using the rational method formula, with C assumed equal to 1.

$$I = \frac{3.6\,Q}{C.Area}$$

Using a linear regression using logarithms of the Nambour IFD (corrected with ARR2016 Areal Reduction Factors) and duration. ToC was estimated as the duration of the intensity which yielded the peak flow.

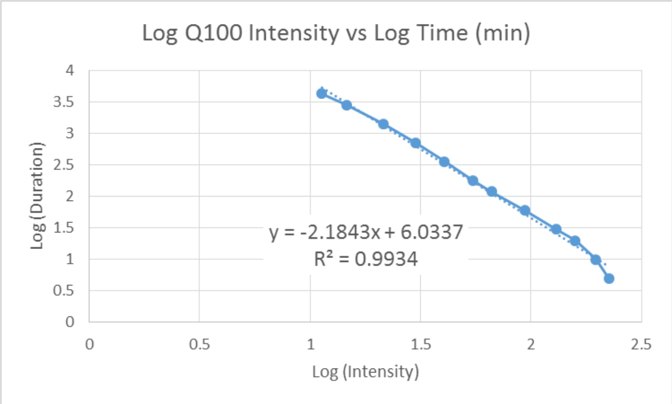


Figure 9 Log Intensity (Nambour) vs Log Duration

For each % impervious area considered, 165 peak flows were calculated for the combination of Area (0.001 – 40 km^2) and slope (0.5 – 10%). Each of these peak flows were transformed to an intensity.

It was determined that there was a power-law relationship between ToC and Area:

$$ToC = a.Area^b \qquad \text{SCC (2017)}$$

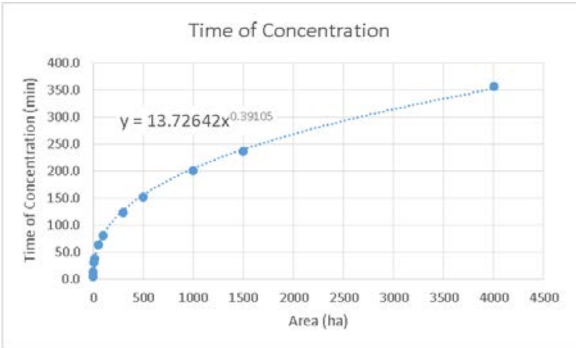


Figure 10 Time of Concentration derived from Area (CF 2.6, Slope 2.5, Imp 0%)

It was also determined that the variable a and b of this equation also had a power-law relationship with slope. Details are provided in Appendix A.

$$a = c.Slope^d$$

$$b = e.Slope^f$$

Further it was determined that the variables c, d, e and f were correlated with fraction impervious (f_i). Details of this are provided in Appendix B.

$$c = 12.814.e^{-0.801\,f_i}$$

$$d = -0.00372.f_i - 0.0835$$

$$e = -0.00784.f_i + 0.38324$$

$$f = -0.00337.f_i - 0.02783$$

Results Validation

Estimates of ToC from SCC (2017) were compared to original ToC estimates derived from peak flow estimates of the URBS model converted to intensity through the rational method formula and duration from IFD interpolation. The comparison showed excellent correlation (R^2=0.996) for durations up to approximately 360 minutes (6 hours). For durations exceeding 6 hours it is observed that the ToC values calculated using SCC (2017) are under estimated.

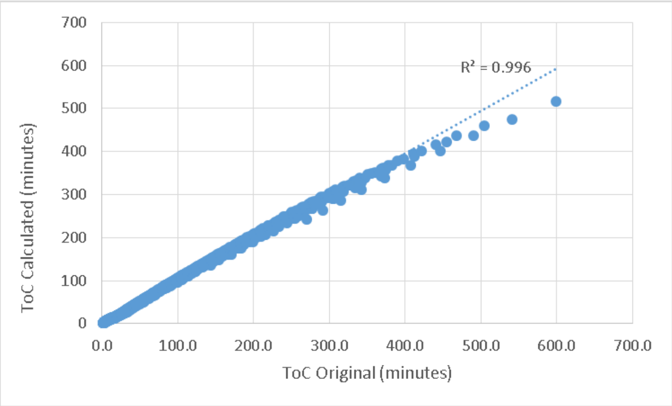


Figure 11 ToC estimates: Formula Estimates vs Model Derived

Catchment Shape

Figure 7 indicates a parameter labelled CF. This parameter controls the shape of the catchment. The analysis was repeated for a number of CF values to understand the impact of catchment shape on ToC. Figure 12 demonstrates the influence that the CF value has on catchment shape, noting each shape has the same area.

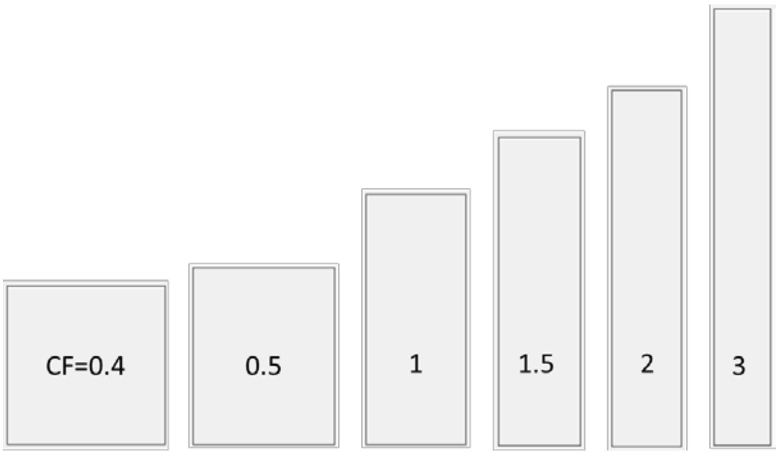


Figure 12 CF value influence on Catchment Shape

It was also observed that a relationship existed between a function of centroid distance and area, and the value of CF. This relationship allows CF to be estimated from the centroid length (km) and area (km^2). The range of CF is (0.4<CF<4). Values outside of this range indicate an unusual catchment shape outside the range of shapes considered in this analysis.

$$CF = -3.688.\,LN\left(\frac{0.5\sqrt{Area}}{Centroid}\right) - 0.9195$$

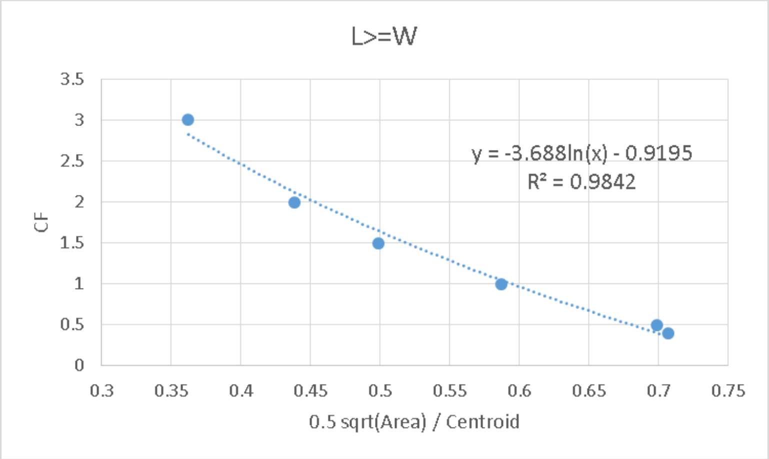


Figure 13 CF equation for catchments where Length ≥ Width

It was observed that the constant factors, offsets and exponents in the previous equations for the parameters c, d, e and f varied with the catchment shape factor (CF). This is shown in Appendix C.

The SCC ToC equation for channelised/concentrated flow utilises c, d, e and f calculated as:

$$c = (0.8998\ CF + 12.574).\, e^{(0.0046\ CF - 0.8057)\ f_i}$$

$$d = (-0.0029\ CF - 0.0011).\, f_i - (-0.0111\ CF - 0.0845)$$

$$e = -0.0074.\, f_i + (0.01\ CF + 0.3814)$$

$$f = (0.0013\ CF - 0.0041).\, f_i - (-0.0054\ CF - 0.0245)$$

Comparison with FFA Derived ToC Estimates

Estimates of ToC derived from the SCC (2017) formulas were compared to ToC estimates derived from FFA estimates (estimated probability of historic floods was derived from the flood frequency curves and for the same events ToC was estimated from the rainfall IFD duration that produced the equivalent event probability when losses were accounted for). This comparison was limited to 400 minutes (based on the observations from Figure 11).

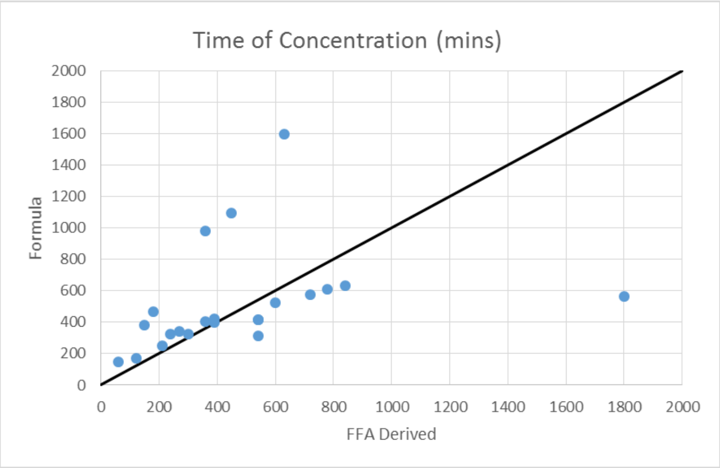


Figure 14 SCC (2017) ToC estimates compared with FFA derived ToC estimates

The comparison indicates that the estimates derived from SCC ToC formulas slightly over estimate ToC relative to FFA derived values, however, SCC ToC values are determined from coarse estimates of centroid length and fraction impervious was assumed to be 0. It should also be noted that the FFA derived ToC values only have 30 minute precision.

2.2 Large Natural Catchments

It was determined that predictive performance observed in Section 2.1.3 could be improved by using surface geology. Surface geology was used as determinant for the estimation of a Z parameter, where Z replaces fraction impervious in the SCC ToC formula but is not bound by 0 and 1. This parameter is a mechanism of adjustment for largely undeveloped catchments that not only accounts for fraction impervious and surface geology but also implicitly accounts for vegetation and the impacts of spatial variations in loss and model routing parameters.

Surface geology data was sourced from the Queensland Wetlands Program website (wetlandinfo.ehp.qld.gov.au) and the proportion of geology types was determined for the catchment areas of 22 streampoints (Figure 1) that had ToC values estimated from FFA methodology. Values of Z for each geology type, as shown in Table 2 were calibrated such that ToC estimates from the SCC ToC formula provided good agreement with ToC values estimated from FFA methods. The agreement can be observed in Figure 15. Further details relating to this fitting are provided in Appendix D.

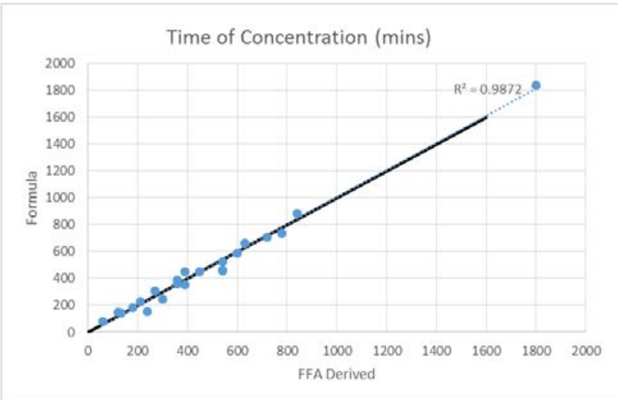


Figure 15 Time of Concentration, Z values as calibrated with Surface Geology

Table 2 Geology Types of Sunshine Coast Catchments

Geology	Z Value	Description
AL	-0.8	Alluvium
AR	-0.7	Arenite
AR-MU	-1.5	Arenite-Mudrock
BA	1.2	Basalt
CO	2.5	Colluvium
FE	0.5	Felsite
MU	-3	Mudrock
GR	2.5	Granitoid
MA	-1.7	Mafites
SR	0.15	Sedimentary Rock
MSM	2.5	Mixed Sedimentary Rock and Mafites
MMFE	3	Mixed Mafites and Felsites
PE	-3	Pelsites
WB	-1	Water Bodies
Mis	-8.5	Miscellaneous Unconsolidated Sediments

Thus for any given location of interest, the geology of the upstream catchment can be used to estimate a Z value for use in SCC (2017) ToC. Z is estimated in accordance with:

$$Z_{Catchment} = \sum_{x=1}^{n} \frac{A_x Z_x}{A_{Catchment}}$$

Where Z_x values are provided in Table 2.

It is recognised that the determination of Z value with geology could be conjectured to be a curve fitting exercise. Until such time that a blind validation is completed at other locations with data extracted from regional models or from neighbouring regional councils, the calculation of a Z parameter for a given catchment should be limited to larger natural catchments that have non-homogenous surface geology.

2.3 Error in Estimation of Peak Flow (Large Natural Catchments)

Flow estimates were derived using the ARR RFFE, SCC RFFE and rational method for the 10 catchments adopted for the RFFE review documented in Section 1. The catchment areas of the 10 catchments ranged from 18.6km^2 to 485km^2 and all catchments were assumed largely natural ($f_i \leq 0.2$).

Adopted 10% AEP estimates for the comparison were derived from FFA, whereas adopted 1% AEP estimates were derived from recent modelling in the Pumicestone (BMT WBM, 2017b), Maroochy (BMT WBM, 2017a), Mooloolah (Cardno, 2015) and Mary (DHI, 2012) catchments. Importantly, flow and flood frequency estimates have benefited from the ratings derived from hydraulic models.

Flows were estimated using the RFFE tools and formula discussed in Section 1. Summary statistics of the error between estimated flows and adopted flows is shown in Table 4.

Table 3 Error in Peak Flow Estimation between RFFE methods

	ARR RFFE		SCC RFFE	
	10% AEP	1% AEP	10% AEP	1% AEP
Average RMS Error	22%	28%	12%	7%
Median RMS Error	22%	24%	12%	4%
Max RMS Error	39%	76%	19%	26%
Average Error	-15%	-7%	2%	3%
Median Error	-18%	-8%	2%	2%

Flows were then derived using the rational method with various ToC estimates. Bureau of Meteorology 2016 IFD has been the basis of design rainfall used in all rational method calculations.

Table 4 shows error statistics between adopted flows and flows estimated from the original ToC values derived from a FFA method. Original ToC values are shown in Appendix D.

Table 4 Error in Peak Flow Estimation between methods (FFA derived ToC)

	Rational Method					
	QUDM ‘C’ values		‘C’ = 1.0		Calibrated ‘C’	
	C_{10}=0.74	C_{100}=0.89	C_{10}=1.0	C_{100}=1.0	C_{10}=0.8	C_{100}=0.88
	10% AEP	1% AEP	10% AEP	1% AEP	10% AEP	1% AEP
Average RMS Error	10%	9%	27%	15%	**9%**	**9%**
Median RMS Error	8%	6%	26%	10%	**8%**	**6%**
Max RMS Error	22%	29%	53%	46%	**22%**	**28%**
Average Error	-6%	1%	27%	14%	**1%**	**0%**
Median Error	-7%	-3%	26%	10%	**1%**	**-3%**

The analysis was then repeated with ToC estimates derived from the SCC (2015) formula. Results are shown in Table 5.

Table 5 Error in Peak Flow Estimation between methods (SCC 2015 Formula derived ToC)

	Rational Method					
	QUDM 'C' values		'C' = 1.0		Calibrated 'C'	
	C_{10}=0.74	C_{100}=0.89	C_{10}=1.0	C_{100}=1.0	C_{10}=0.8	C_{100}=0.88
	10% AEP	1% AEP	10% AEP	1% AEP	10% AEP	1% AEP
Average RMS Error	14%	13%	25%	15%	**11%**	**13%**
Median RMS Error	12%	12%	23%	12%	**8%**	**13%**
Max RMS Error	26%	32%	66%	48%	**33%**	**30%**
Average Error	-7%	0%	25%	12%	**0%**	**-1%**
Median Error	-9%	-3%	23%	9%	**-2%**	**-4%**

Table 6 shows error statistics based on rational method with ToC estimates derived from the SCC 2017 formula using geology as the Z parameter determinant.

Table 6 Error in Peak Flow Estimation between methods (SCC 2017 Formula derived ToC)

	Rational Method					
	QUDM 'C' values		'C' = 1.0		Calibrated 'C'	
	C_{10}=0.74	C_{100}=0.89	C_{10}=1.0	C_{100}=1.0	C_{10}=0.8	C_{100}=0.88
	10% AEP	1% AEP	10% AEP	1% AEP	10% AEP	1% AEP
Average RMS Error	9%	10%	27%	16%	**6%**	**9%**
Median RMS Error	8%	8%	26%	13%	**5%**	**8%**
Max RMS Error	17%	18%	48%	33%	**18%**	**18%**
Average Error	-6%	2%	27%	14%	**2%**	**1%**
Median Error	-7%	0%	26%	13%	**0%**	**-1%**

Table 7 shows error statistics based on rational method with ToC estimates derived from the Bransby-Williams formula.

Table 7 Error in Peak Flow Estimation between methods (Bransby-Williams ToC)

	Rational Method					
	QUDM 'C' values		'C' = 1.0		Calibrated 'C'	
	C_{10}=0.74	C_{100}=0.89	C_{10}=1.0	C_{100}=1.0	C_{10}=0.9	C_{100}=1.0
	10% AEP	1% AEP	10% AEP	1% AEP	10% AEP	1% AEP
Average RMS Error	20%	16%	14%	14%	**11%**	**14%**
Median RMS Error	21%	17%	8%	9%	**9%**	**9%**
Max RMS Error	32%	29%	49%	41%	**34%**	**41%**
Average Error	-18%	-10%	11%	2%	**0%**	**2%**
Median Error	-21%	-15%	6%	-5%	**-4%**	**-5%**

(9 Samples)

Table 8 shows error statistics based on rational method with ToC estimates derived from the Queensland Department of Main Roads formula.

Table 8 Error in Peak Flow Estimation between methods (QLD Department of Main Roads ToC)

	Rational Method					
	QUDM 'C' values		'C' = 1.0		**Calibrated 'C'**	
	C_{10}=0.74	C_{100}=0.89	C_{10}=1.0	C_{100}=1.0	**C_{10}=0.74**	**C_{100}=0.88**
	10% AEP	1% AEP	10% AEP	1% AEP	10% AEP	1% AEP
Average RMS Error	24%	24%	37%	27%	**24%**	**24%**
Median RMS Error	29%	16%	22%	12%	**29%**	**16%**
Max RMS Error	55%	87%	109%	111%	**55%**	**85%**
Average Error	-1%	7%	34%	21%	**-1%**	**6%**
Median Error	-10%	-4%	22%	8%	**-10%**	**-5%**

(9 Samples)

It is observed that:

1. The ARR RFFE provides the poorest estimation of 10% and 1% AEP peak flows. It should not be used on the Sunshine Coast.
2. The rational method using SCC ToC (2017) values is the best method for estimating 10% and 1% AEP peak flows.
3. The SCC RFFE is the second best method for estimating 10% and 1% AEP peak flows. Error statistics indicate only a marginal loss of predictive performance. It is considered a good method for rapid peak flow estimation for catchments greater than 15km^2.
4. The QUDM C_{10} parameter natural catchments appears to be an underestimate on the Sunshine Coast but the C_{100} parameter for 1%AEP estimation appears to provide good estimates. A better C_{10} value for natural catchments was observed to be 0.8.
5. Given calibrated C_{10} value of 0.8 and C_{100} value of 0.88. A rational method frequency factor of 1.1 should be adopted for the 1% AEP event.
6. There is a slight reduction in error statistics using SCC ToC (2017) estimates compared to the FFA derived ToC values. This is a likely outcome of FFA derived ToC values only having half hourly precision.
7. The rational method using Bransby-Williams ToC values provides better predictive performance of 10% and 1% AEP peak flows on the Sunshine Coast than the rational method using the Queensland Department of Main Roads ToC estimates.
8. The rational method with Queensland Department of Main Roads ToC produces estimates that as just as poor as the ARR RFFE. It is not recommended for use on the Sunshine Coast.
9. The Bransby-Williams ToC formulas does not perform as well as the SCC ToC (2017) method on the Sunshine Coast.
10. SCC ToC (2015) estimates should not be used in preference to SCC ToC (2017) estimates for peak flow estimation on the Sunshine Coast.
11. Bureau of Meteorology 2016 IFD is appropriate for use in rational method calculations.

2.4 Multivariate Model Analysis (Sheet Flow)

The structure of the multivariate URBS model described in Section 2 is suited to larger catchments that channelise and concentrate flow. This form of flow is represented by the channel routing parameter (α) in the notional model. For very small catchments (<2 ha), where flows do not concentrate in channels and occur as sheet flow, a longer ToC is expected.

The notional model was revisited, modifying the model to remove the influence of the channel routing parameter (α) and rely solely on catchment routing parameter (β). This resulted in a single subcatchment. The catchment shape factor (CF) was no longer relevant as, in the previous model, this parameter represented the arrangement of the 10 subcatchments and the channel routing that occurs between them.

The analysis was repeated, considering slope and fraction impervious. A further 3135 model runs were completed to derived parameters for a new variant of the SCC ToC equation representing catchments dominated by sheet flow runoff.

Appendix D provides details of the parameter fits for the catchment routing/sheet flow variant of the SCC ToC equation. The adopted parameter formulas for c, d, e and f for this form of the ToC equation are:

$$c = 22.3784. e^{-0.8185\, f_i}$$

$$d = -0.00748. f_i - 0.07817$$

$$e = -0.00699. f_i + 0.38949$$

$$f = 0$$

Figure 16 shows the 1% AEP peak flows from the revised model with a 0% fraction impervious. When compared to results of the previous model which incorporated channel and catchment routing, as previously shown in Figure 8, it can be observed that peak flows of the new model are reduced. The variance in these figures represents the range of slopes that have been considered (0.5 - 10%). It can be observed that the variance in peak flow from the model which represents catchment routing only is less than the prior model which also included channel routing.

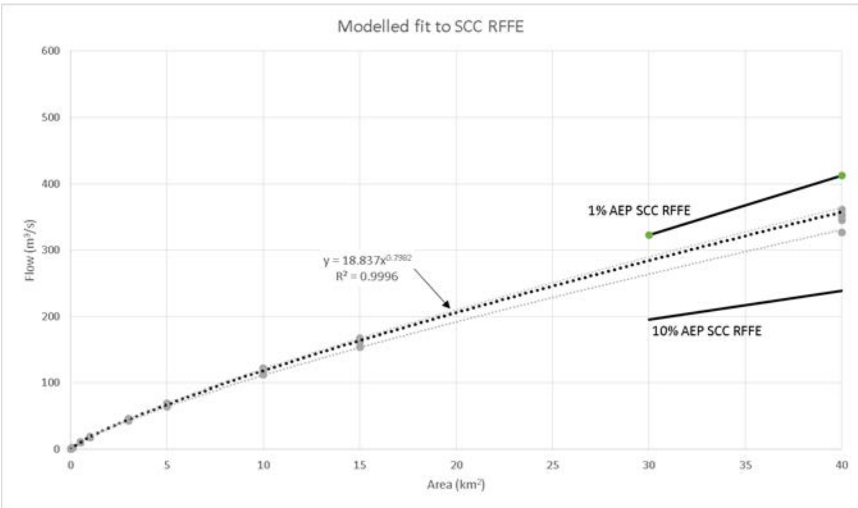


Figure 16 Notional (Sheet Flow) URBS model 1% AEP flows, 0% fraction impervious

2.5 Travel Time

The catchment routing model is sensitive to slope in other ways. It was observed that travel time of the peak was sensitive to slope. The travel time of the peak was calculated as the difference between the time of peak flow and the time of peak rainfall (TTPP). This was compared with ToC estimates derived from the suite of peak flow estimates derived from the new model. The comparison between travel time of the peak and the ToC is shown in Figure 17.

It can be observed that TTPP does not appear to be a good surrogate for ToC as it is generally always shorter than ToC. A similar observation is also made from the channel and catchment routing modelling. Figure 18 provides this comparison and again indicates that the modelled TTPP is shorter than ToC. It is also apparent from Figure 18 that the relationship between TTPP and ToC derived from the catchment and channel routing model is not sensitive to slope. This is not to say that TTPP is not sensitive to slope, rather it is equally sensitive to slope as ToC.

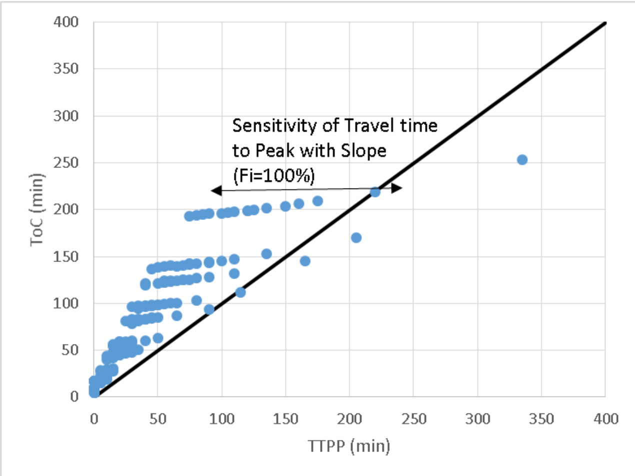


Figure 17 ToC vs TTPP, (Catchment Routing/Sheet Flow)

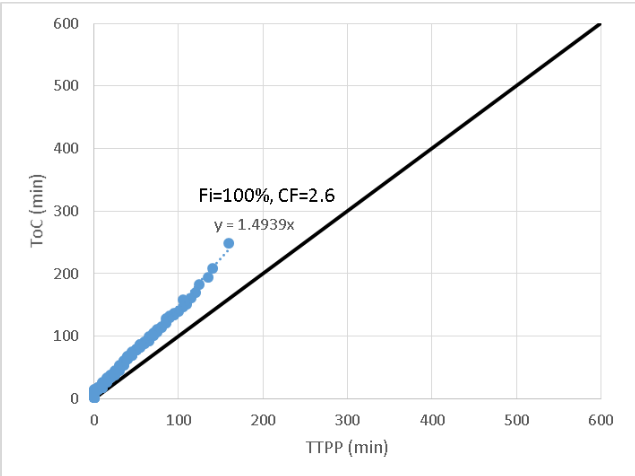


Figure 18 ToC vs TTPP, (Channel and Catchment Routing/Concentrated Flow)

The linear relationship observed in Figure 18 indicated that TTPP should be able to be estimated from ToC. A similar modelling analysis to the original ToC formula derivation for channelised and concentrated flow was undertaken, with the 18,810 discrete model runs repeated, this time also reporting TTPP to determine how the relationship with ToC varied with Z parameter and catchment shape (CF).

The following equations were derived (for channelised\concentrated flow):

$$TTPP = g.ToC/h$$

Where:

$$g = 1.4$$

$$h = (0.2062\ CF + 2.7913) * e^{(0.0028\ CF - 0.8333)*Z})$$

Appendix F provides further details on the model fitting of the h parameter using the CF and Z parameters. The parameter g is a correction factor to adjust TTPP to match observations from real events at a number of stream point locations. This data is also provided in Appendix F and shows that TTPP is not suitable for use at locations that are tidally affected. It is assumed that this constraint would also apply to locations that are affected by backwater.

Table 9 SCC ToC Formula Parameters for pre development inflow calculations

	Catchment Shape (CF)							
Z	**0.4**	**0.5**	**0.8**	**1**	**1.5**	**2**	**2.5**	**3**
-0.5	4.36	4.39	4.48	4.54	4.69	4.85	5.00	5.15
-0.2	3.39	3.42	3.49	3.54	3.66	3.78	3.90	4.02
0	2.87	2.89	2.96	3.00	3.10	3.20	3.31	3.41
0.1	2.64	2.66	2.72	2.76	2.85	2.95	3.04	3.14
0.4	2.06	2.08	2.12	2.15	2.23	2.30	2.38	2.45
0.6	1.74	1.76	1.80	1.82	1.89	1.95	2.01	2.08
0.8	1.48	1.49	1.52	1.54	1.60	1.65	1.71	1.76
0.9	1.36	1.37	1.40	1.42	1.47	1.52	1.57	1.62
1	1.25	1.26	1.29	1.31	1.35	1.40	1.45	1.49

It is observed from Table 9 that the parameter h approaches the value for the parameter g (1.4) as the parameter Z approaches 1.0 (i.e. paved catchments) and when the catchment shape parameter approaches a square (CF 0.4). In situations when h approximates g, TTPP approximates ToC.

2.6 Determining Pre and Post Development Inflows

Rational method in combination with SCC ToC (2017) may be used for pre and post development inflow estimation.

Pre Development ToC

The most appropriate ToC methodology for pre-development conditions is dependant on several factors such as catchment area, catchment slope, vegetation and topography. The decision flowchart in Figure 19 should be used to determine which pre-development ToC methodology is suitable for use. This decision flowchart has been derived based on comparisons with QUDM Friend's Equation for overland flow and some sample hydrology models. It will ensure that the longest ToC is applied to each catchment in the pre-development scenario, and will ensure a proper transition between Sheet Flow and Concentrated/Channelised Flow methodologies.

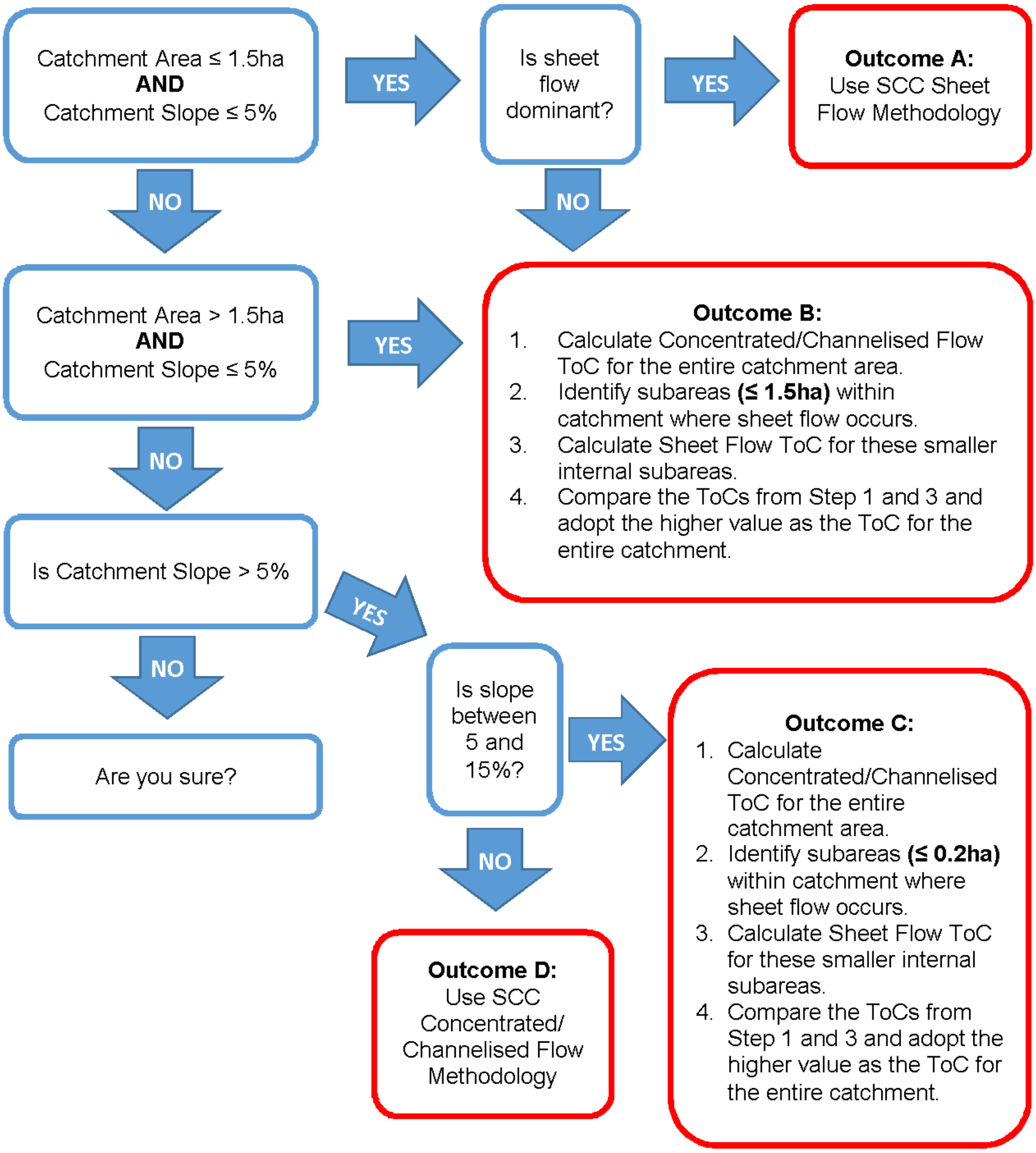


Figure 19 Pre-development ToC Methodology Decision Flowchart

When determining the ToC (minutes) for pre-development conditions, the parameters outlined in Table 10 should be used with Area (hectares) and Bed Slope (%).

$$ToC = a.Area^{b} \qquad \text{SCC (2017)}$$

Table 10 SCC ToC Formula Parameters for pre-development inflow calculations

Parameter	Concentrated/Channelised	Sheet Flow
a	$c.Slope^{d}$	
b	$e.Slope^{f}$	
c	$(0.8998\ CF + 12.574).exp^{(0.0046\ CF - 0.8057)\ Z}$	$22.3784.exp^{(-0.8185\ Z)}$
d	$(-0.0029\ CF - 0.0011).Z + (-0.0111\ CF - 0.0845)$	$-0.00748.Z - 0.07817$
e	$-0.0074.Z + (0.01\ CF + 0.3814)$	$-0.00699.Z + 0.38949$
f	$(0.0013\ CF - 0.0041).Z_i + (-0.0054\ CF - 0.0245)$	0

Where the pre-development ToC formula estimates ToC values less than 5 minutes, the ToC value shall be set as 5 minutes.

Z Parameter

For pre-development conditions the Z parameter is used in place of fraction impervious. Appendix E provides the calibrated Z values for the 22 natural catchment stream points considered in Section 2.2. The variation in these values in indicative of how the geology, soil types, surface roughness and different model loss and routing parameters may influence the estimation of peak flow on an undeveloped catchment. The following Z parameter values are recommended for adoption under pre development conditions.

Table 11 Simplified Z Parameters for pre development

Surface	Z
Pavement	1.0
Medium Density	0.6
Low Density	0.4
Bare Soil	0.1
Poorly Grassed	0
Average Grassed	-0.2
Densely Grassed	-0.5

Post Development ToC

$$ToC = a.Area^b \qquad \text{SCC (2017)}$$

Table 12 SCC ToC Formula Parameters for post-development

Parameter	Concentrated/Channelised
a	$c.Slope^d$
b	$e.Slope^f$
c	$(0.8998\,CF + 12.574).e^{(0.0046\,CF - 0.8057)\,f_i}$
d	$(-0.0029\,CF - 0.0011).f_i - (-0.0111\,CF - 0.0845)$
e	$-0.0074.f_i + (0.01\,CF + 0.3814)$
f	$(0.0013\,CF - 0.0041).f_i - (-0.0054\,CF - 0.0245)$

Where the post-development ToC formula estimates ToC values less than 5 minutes, the ToC value shall be set as 5 minutes.

Fraction Impervious

For post-development conditions the fraction impervious is used instead of the Z parameter. Residential/Industrial-and Commercial development should adopt a value of 0.9 and 1.0 for the fraction impervious parameter value respectively. These values are consistent with the values prescribed in the Planning Scheme Policy for Development Works (Stormwater Management) SC6.14.3.6.

It is important that the highest fraction impervious is adopted for any zoned use as the stormwater infrastructure delivered with development must be able to cater for future upzoning.

2.7 C_{10} Factor

When applying the rational method on the Sunshine Coast, the value of C_{10} should vary linearly with fraction impervious between 0.8 (f_i =0) and 0.9 (f_i=1.0), as shown in Table 13.

Table 13 C_{10} Factor

F_i	C_{10}
≤ 0	0.80
0.1	0.81
0.2	0.82
0.3	0.83
0.4	0.84
0.5	0.85
0.6	0.86
0.7	0.87
0.8	0.88
0.9	0.89
≥ 1.0	0.9

C_{10} should not be varied with Z parameters derived from the surface geology method described in Section 2.2.

2.8 Frequency Factors

It is recommended that the frequency factors presented in Table 4.5.2 of QUDM be replaced with those shown in Table 14, for use with the rational method on the Sunshine Coast.

Table 14 Frequency Factor

		Frequency factor (Fy)		
AEP (%)	**ARI (years)**	**QUDM**	**Calibrated**	**Adopted**
63%	1	0.8		**0.9**
39%	2	0.85		**0.925**
18%	5	0.95		**0.975**
10%	10	1	0.8	**1**
5%	20	1.05		**1.025**
2%	50	1.15		**1.075**
1%	100	1.2	0.88	**1.1**

$$F_y = 0.9 + 0.1\, log10(ARI)$$

As noted previously in Section 1.1, 39% AEP estimates have not featured in this investigation, however rational method estimates were checked against the available modelled and FFA data to confirm that the Frequency Factor of Table 14 was appropriate. Results provided in Appendix G indicate that the rational method estimates appear reasonable given the uncertainty in the modelled and FFA estimates used for comparison. Results also appear to indicate that the geology based determination of the Z parameter no longer is not beneficial for 39% AEP peak inflow estimation and the $Z = 0$ appears to be a better assumption for the natural catchments used for comparison.

3 Conclusions and Recommendations

It has been concluded that the rational method does remain appropriate for continued (but specific) use on the Sunshine Coast, with some adjustment to C values. Specific use is limited to the estimation of peak inflow on simple catchments, where hydrographs are not required and that do not involve such complexities as:

- Partial area affects
- Storage systems
- Flow bypass
- Drainage network conveyance

It is considered that the rational method can continue to be used to assist with basic urban stormwater calculations, through the provision of sub catchment peak inflows and, in circumstances that do not involve the above complexities, it can be used to compliment uncalibrated models as a method of validating peak inflow estimates. The SCC RFFE can also be used as an estimation and verification tool on catchments that are predominantly natural and larger than 15km^2.

It is recommended that the model verification with SCC methods, be based upon the model representing the catchment in a natural condition.

It is recommended that the SCC ToC formula (2017) documented in this review, be the basis for rational method calculation on the Sunshine Coast.

Where the rational method is used for drainage calculations informing new infrastructure in small urban catchments, fraction impervious should be applied in the SCC ToC formula. Where the ToC formula estimates ToC values less than 5 minutes, the ToC value shall be set as 5 minutes.

For cross drainage calculations in larger catchments that are predominantly undeveloped, the SCC RFFE can be used. The recommended minimum catchment area is 15km^2. For catchments areas less than this, the rational method can be applied with SCC ToC formula parameters for Concentrated/Channelised flow. In these circumstances, the Z parameter is used in the SCC ToC formula, which can be determined using surface geology as described in Section 2.2 or using the simplified values presented in Table 11.

For the assessment of pre and post development inflows the rational method should be used in combination with the SCC ToC formula, as described in Section 2.6. Figure 19 should be used to identify which ToC methodology is to be applied in the pre-development scenario.

When applying the rational method on the Sunshine Coast, it is recommended that the values of C_{10} be adopted as described in Section 2.7

When using the rational method to estimate inflows for other AEPs, frequency factors should be adopted as described in Section 2.8.

Bureau of Meteorology 2016 IFD are recommended for application with the rational method on the Sunshine Coast.

4 References

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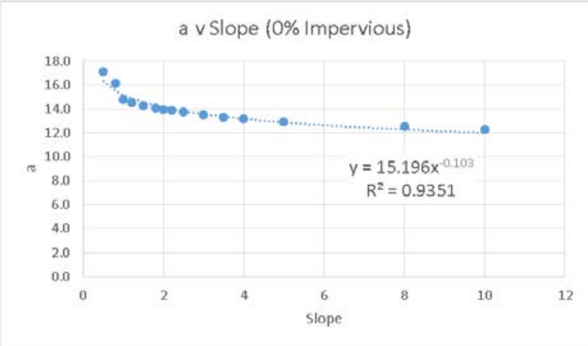
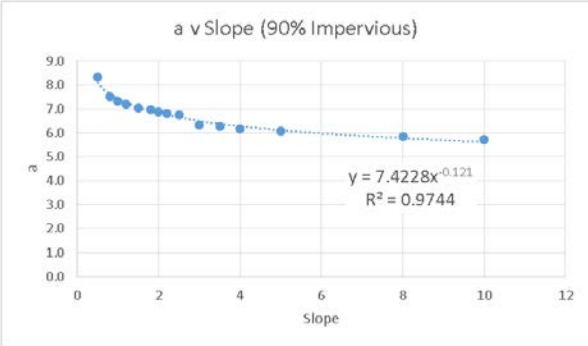
5 Glossary

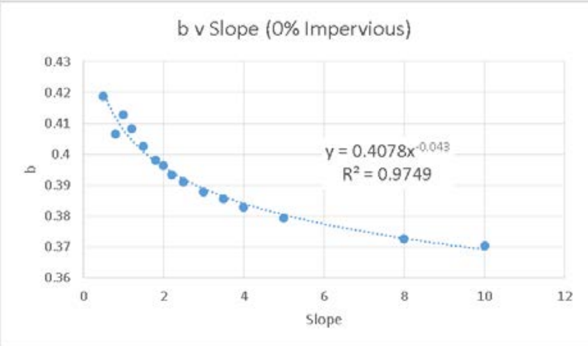
AEP	Annual Exceedance Probability
ARF	Areal Reduction Factor
ARR	Australian Rainfall and Runoff
DIS	Duration Independent Storm, a temporal pattern derived from the IFD.
IFD	Intensity Frequency Duration design rainfall
FFA	Flood Frequency Analysis
QUDM	Queensland Urban Drainage Manual
SCC	Sunshine Coast Council
ToC	Time of Concentration, the duration of rainfall associated with peak discharge
RFFE	Regional Flood Frequency Estimation

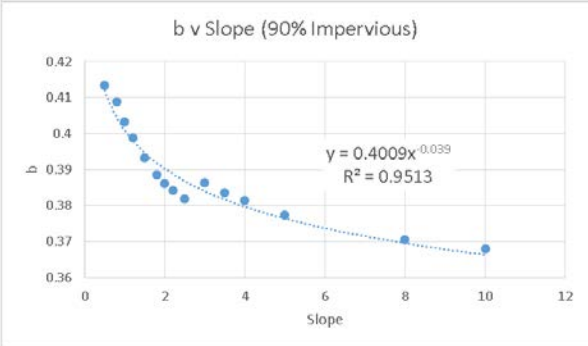
Appendix A ToC Parameters with Slope (CF = 2.6)

Catchment and Channel Routing



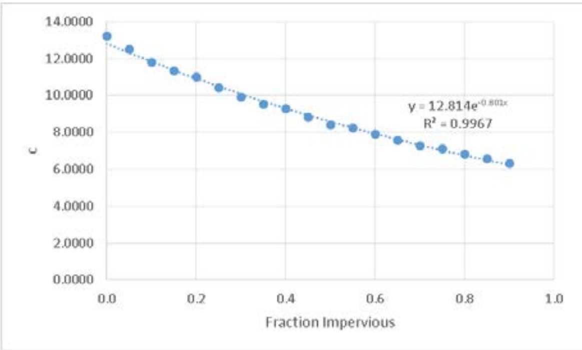


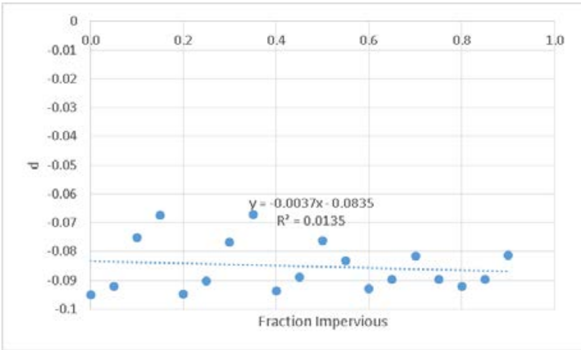


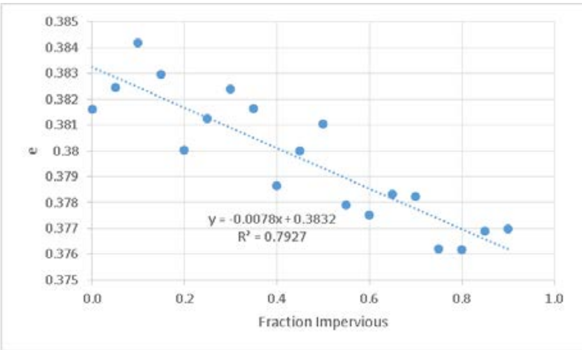


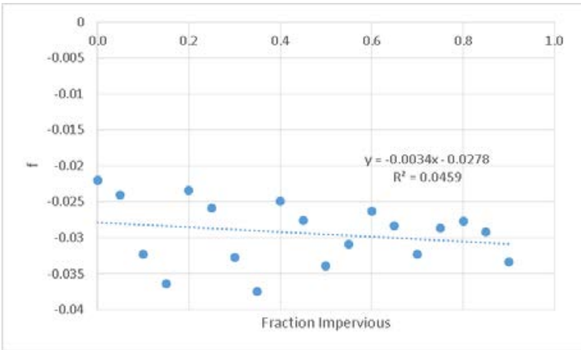
Appendix B ToC Parameters with Fraction Impervious

Catchment and Channel Routing



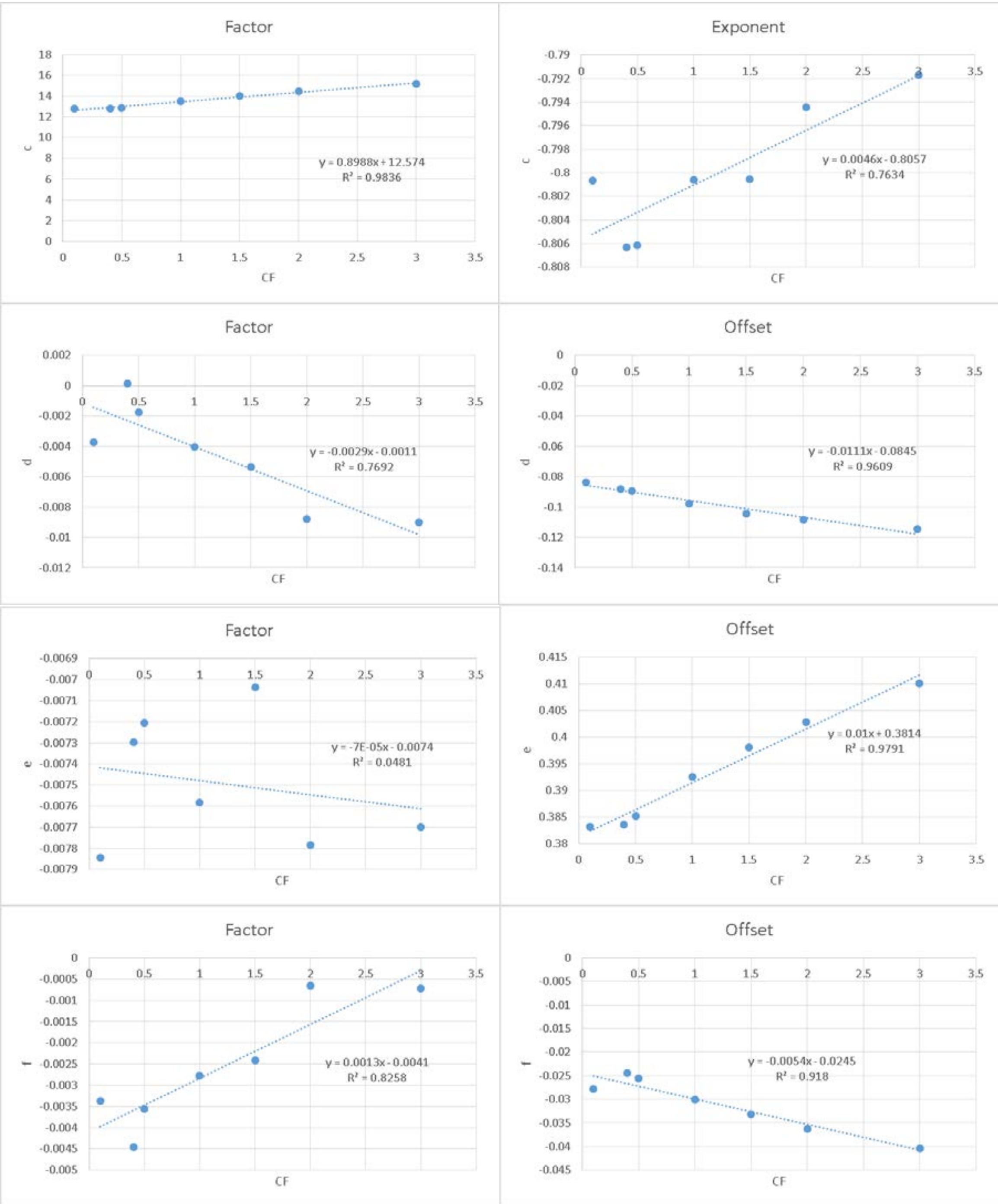






Appendix C ToC Parameters with CF

Catchment and Channel Routing



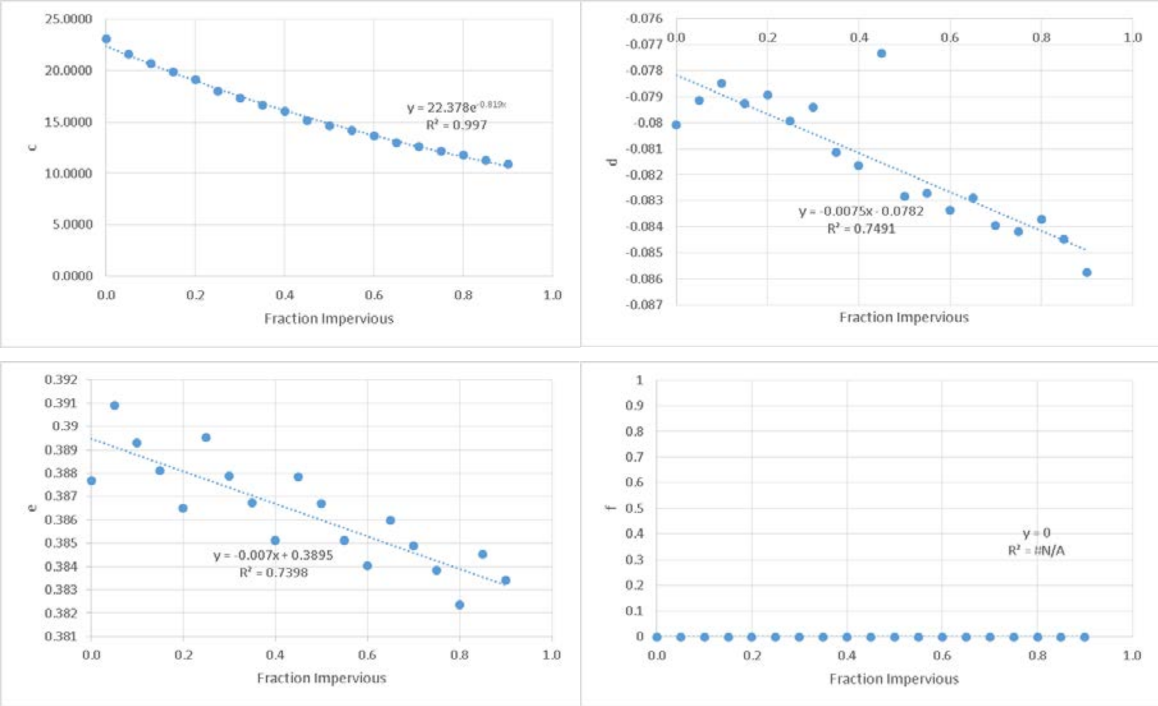
Appendix D ToC Calculations at Stream Point Locations

Location	CA	Centroid L	Slope	CF	imp fr	f	e	d	c	b	a	ToC	FFA Derived ToC	
RNS91	37.4	8.0	1.28	2.63	**0.12**	-0.04	0.41	-0.11	13.53	0.40	13.15	361.84	6.0	360.0
RNS92	33.3	6.8	2.17	2.24	**0.85**	-0.04	0.40	-0.12	7.44	0.39	6.80	155.80	4.0	240.0
RPE91	11.9	2.7	6.75	0.74	**0.72**	-0.03	0.38	-0.10	7.43	0.36	6.19	80.10	1.0	60.0
RPE92	38.3	8.2	2.61	2.68	**0.14**	-0.04	0.41	-0.12	13.43	0.39	12.02	305.58	4.5	270.0
REC92	52.9	9.5	1.91	2.62	**-0.12**	-0.04	0.41	-0.11	16.38	0.40	15.23	463.50	9.0	540.0
RPA92	42.8	10.5	1.98	3.38	**-0.10**	-0.04	0.42	-0.12	16.94	0.40	15.60	457.35	9.0	540.0
RCO91	18.6	3.2	6.81	0.54	**0.19**	-0.03	0.39	-0.09	11.18	0.37	9.39	146.67	2.0	120.0
RCO92	35.4	5.1	4.38	1.07	**0.11**	-0.03	0.39	-0.10	12.44	0.37	10.78	229.14	3.5	210.0
RCO93	55.7	9.9	2.57	2.68	**-0.14**	-0.04	0.41	-0.11	16.71	0.39	15.02	451.03	6.5	390.0
RDC91	31.9	5.3	1.40	1.40	**-0.59**	-0.03	0.40	-0.10	22.19	0.40	21.48	522.72	9.0	540.0
RDC92	57.9	10.3	0.73	2.75	**-1.38**	-0.04	0.42	-0.10	45.07	0.42	46.54	1837.80	30.0	1800.0
RML91	39.3	7.3	2.76	2.19	**0.32**	-0.04	0.40	-0.11	11.25	0.39	10.05	245.48	5.0	300.0
MML01	46.4	10.8	2.16	3.34	**0.20**	-0.04	0.41	-0.12	13.31	0.40	12.10	354.40	6.5	390.0
RML92	79.1	12.3	1.65	2.83	**-0.14**	-0.04	0.41	-0.11	16.84	0.40	15.90	590.24	10.0	600.0
RML93	101.1	14.5	1.45	2.99	**-0.22**	-0.04	0.41	-0.12	18.10	0.41	17.34	737.34	13.0	780.0
RML94	104.4	14.8	1.36	3.00	**-0.39**	-0.04	0.41	-0.11	20.75	0.41	20.04	883.19	14.0	840.0
RSR91	103.7	11.9	1.26	2.21	**-0.24**	-0.04	0.41	-0.11	17.68	0.40	17.25	709.37	12.0	720.0
ROB91	26.4	7.3	1.06	2.93	**1.14**	-0.04	0.40	-0.13	6.16	0.40	6.11	144.23	2.5	150.0
RMY91	110.1	10.6	2.40	1.67	**1.06**	-0.04	0.39	-0.11	6.03	0.38	5.48	185.35	3.0	180.0
RMY92	303.6	25.4	1.34	3.03	**1.06**	-0.04	0.40	-0.13	6.60	0.40	6.35	390.30	6.0	360.0
RMY93	735.5	24.1	1.10	1.20	**1.01**	-0.03	0.39	-0.10	6.10	0.38	6.04	450.38	7.5	450.0
RMY94	816.8	36.8	0.78	2.57	**1.00**	-0.04	0.40	-0.12	6.71	0.40	6.92	664.08	10.5	630.0

	Geology Derived "Z" value															
	Calibrated	-0.8	-0.7	-1.5	1.2	2.5	0.5	-3	2.5	-1.7	0.15	2.5	3	-3	-1	-8.5
Location	Z	**AL**	**AR**	**AR-MU**	**BA**	**CO**	**FE**	**MU**	**GR**	**MA**	**SR**	**MSM**	**MMFE**	**PE**	**WB**	**Mis**
RNS91	0.1245	0.21	0.01	0.22		0.14	0.13		0.16	0.11	0.01					
RNS92	0.847	0.04			0.57		0.39									
RPE91	0.7205	0.01			0.34	0.03	0.44				0.17					
RPE92	0.1375	0.10			0.14						0.33					
REC92	-0.1165	0.22	0.12		0.02	0.01					0.63					
RPA92	-0.1035	0.22	0.05		0.04	0.04	0.06	0.05			0.53					
RCO91	0.1935	0.01	0.33		0.02	0.13	0.02				0.49					
RCO92	0.1055	0.04	0.23		0.01	0.07	0.04				0.61					
RCO93	-0.1375	0.01	0.49		0.01	0.05	0.03				0.41					
RDC91	-0.591		0.55	0.07		0.05	0.10		0.08	0.13						0.03
RDC92	-1.383	0.29	0.27	0.08			0.09		0.06	0.06						0.11
RML91	0.3225	0.14	0.25		0.18	0.14					0.29					
MML01	0.1985	0.17	0.27		0.15	0.12					0.29					
RML92	-0.136	0.19	0.39		0.09	0.07					0.24				0.03	
RML93	-0.2155	0.25	0.39		0.08	0.06					0.21				0.02	
RML94	-0.387	0.26	0.39		0.07						0.20				0.02	
RSR91	-0.244	0.09	0.39		0.11	0.06			0.01	0.22	0.12	0.06				
ROB91	1.142	0.02			0.91					0.02		0.04				
RMY91	1.062	0.07			0.07	0.02			0.55	0.23						
RMY92	1.168	0.06			0.03	0.02			0.36	0.13	0.14	0.22		0.04		
RMY93	1.0065	0.07		0.04	0.12	0.02	0.06		0.24	0.09	0.11	0.19	0.02	0.03	0.01	
RMY94	1.0035	0.07		0.07	0.11	0.02	0.10		0.23	0.08	0.09	0.18	0.03	0.02		

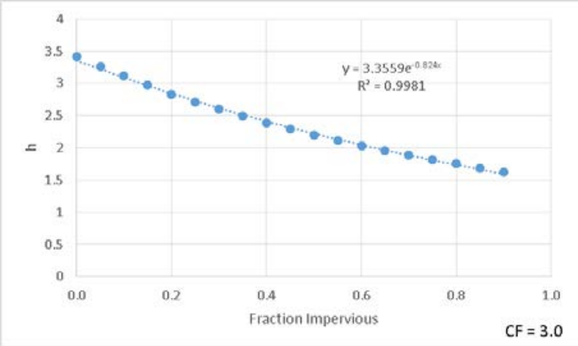
Appendix E Sheet Flow ToC Parameters

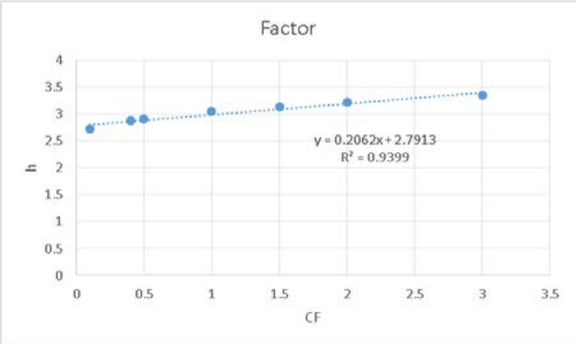
Catchment Routing

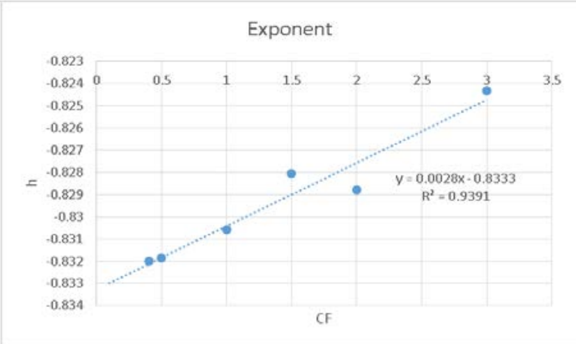


Appendix F Time between Peak Flow and Peak Rainfall

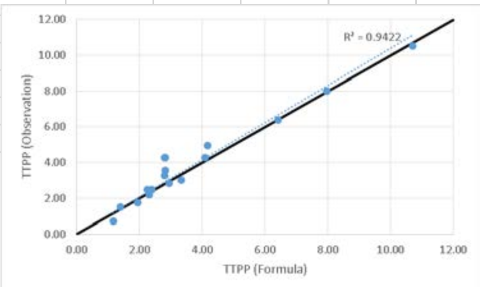
Catchment and Channel Routing







								g	1.4		1%	10%	
Location	CA	Centroid L	CF	Z	ToC (min)	ToC (hrs)	h	TTPP (hrs)	TTPP (min)		Tp (Observed)		
RNS91	37.4	8.0	2.63	0.12	362	6.03	3.01	2.81	168		*4.26*	6	
RNS92	33.3	6.8	2.24	0.85	156	2.60	1.62	2.25	135		*2.48*	3.5	
RPE91	11.9	2.7	0.74	0.72	80	1.33	1.62	1.16	69		*0.71*	1	
RPE92	38.3	8.2	2.68	0.14	306	5.09	2.98	2.39	143		*2.48*	3.5	
REC92	52.9	9.5	2.62	-0.12	463	7.72	3.67	2.95	177		*2.84*	4	
RPA92	42.8	10.5	3.38	-0.10	457	7.62	3.80	2.81	169		*4.26*	6	
RCO91	18.6	3.2	0.54	0.19	147	2.44	2.47	1.39	83		1.5		
RCO92	35.4	5.1	1.07	0.11	229	3.82	2.76	1.94	116		*1.77*	2.5	
RCO93	55.7	9.9	2.68	-0.14	451	7.52	3.75	2.81	169		*3.55*	5	
RDC91	31.9	5.3	1.40	-0.59	523	8.71	5.03	2.43	146				
RDC92	57.9	10.3	2.75	-1.38	1838	30.63	10.52	4.08	245		*4.26*	6	
RML91	39.3	7.3	2.19	0.32	245	4.09	2.48	2.31	138		2.2	3.1	
MML01	46.4	10.8	3.34	0.20	354	5.91	2.95	2.80	168		*3.27*	4.6	
RML92	79.1	12.3	2.83	-0.14	590	9.84	3.78	3.65	219	Tidal	*5.75*	8.1	
RML93	101.1	14.5	2.99	-0.22	737	12.29	4.07	4.23	254	Tidal			
RML94	104.4	14.8	3.00	-0.39	883	14.72	4.69	4.39	263	Tidal	*10.01*	14.1	
RSR91	103.7	11.9	2.21	-0.24	709	11.82	3.97	4.17	250		*4.97*	7	
ROB91	26.4	7.3	2.93	1.14	144	2.40	1.32	2.54	153				
RMY91	110.1	10.6	1.67	1.06	185	3.09	1.30	3.32	199		3		
RMY92	303.6	25.4	3.03	1.06	390	6.51	1.42	6.40	384		6.4		
RMY93	735.5	24.1	1.20	1.01	450	7.51	1.32	7.97	478		8		
RMY94	816.8	36.8	2.57	1.00	664	11.07	1.45	10.69	641		10.5		
											AEP Correction		
												RML91	
											39%	4	0.55
											10%	3.1	0.71
											1%	2.2	



Appendix G 39% AEP Peak Flow Estimate Comparison

	39% AEP Peak Flow (m^3/s)				
Catchment	**Modelled**	**FFA**	**FFA Comment**	**Rational Method (SCC)[1]**	**Rational Method (SCC)[2]**
RCO91	79	*37*	Likely no partial series or partial series correction	87	78
RCO92	127	*85*		129	122
RCN91	68	NA		NA	
RCO93	149	*91*		135	142
RNS92	106	110		145	101
RNS91	70	*91*		105	102
REC92	103	131		135	141
RNS93	164	*131*		NA	166
RPE92	125	96		119	111
RML91		*132*		136	117
RMY92		647**		903	594
ROB91		*73*	No partial series or partial series correction	112	68

[1] Adopts SCC ToC, SCC C and F_y factors and geology based Z parameters

[2] Adopts SCC ToC, SCC C and F_y factors and Z =0

** Gensen (2014) estimated 546m^3/s

RCN91 not calculated due to backwater

RNS93 not calculated due to upstream dam

Appendix H Case Study Comparisons with QUDM

Prepared by

The purpose of the case studies is to demonstrate the application of the Sunshine Coast Council Rational Method (SCC Rational Method) to different situations and provide a comparison to the peak flow estimates yielded by the QUDM Rational Method to those same situations.

Two Case Study situations have been considered:

Case Study 1 – Rural Residential

Case Study 2 – Urban Development

Case Study 1 – Rural Residential

Background

An estimate of the peak flow for the 1%AEP design event is required for the design of an upgrade of a culvert crossing in a rural residential neighbourhood, which is required as a condition of a development approval.

The location of the culvert upgrade (and therefore location where the peak flow estimate is required) is shown in Figure H1. This figure also shows the catchment and location of concentrated flowpaths which were determined from the available contours and site inspection.

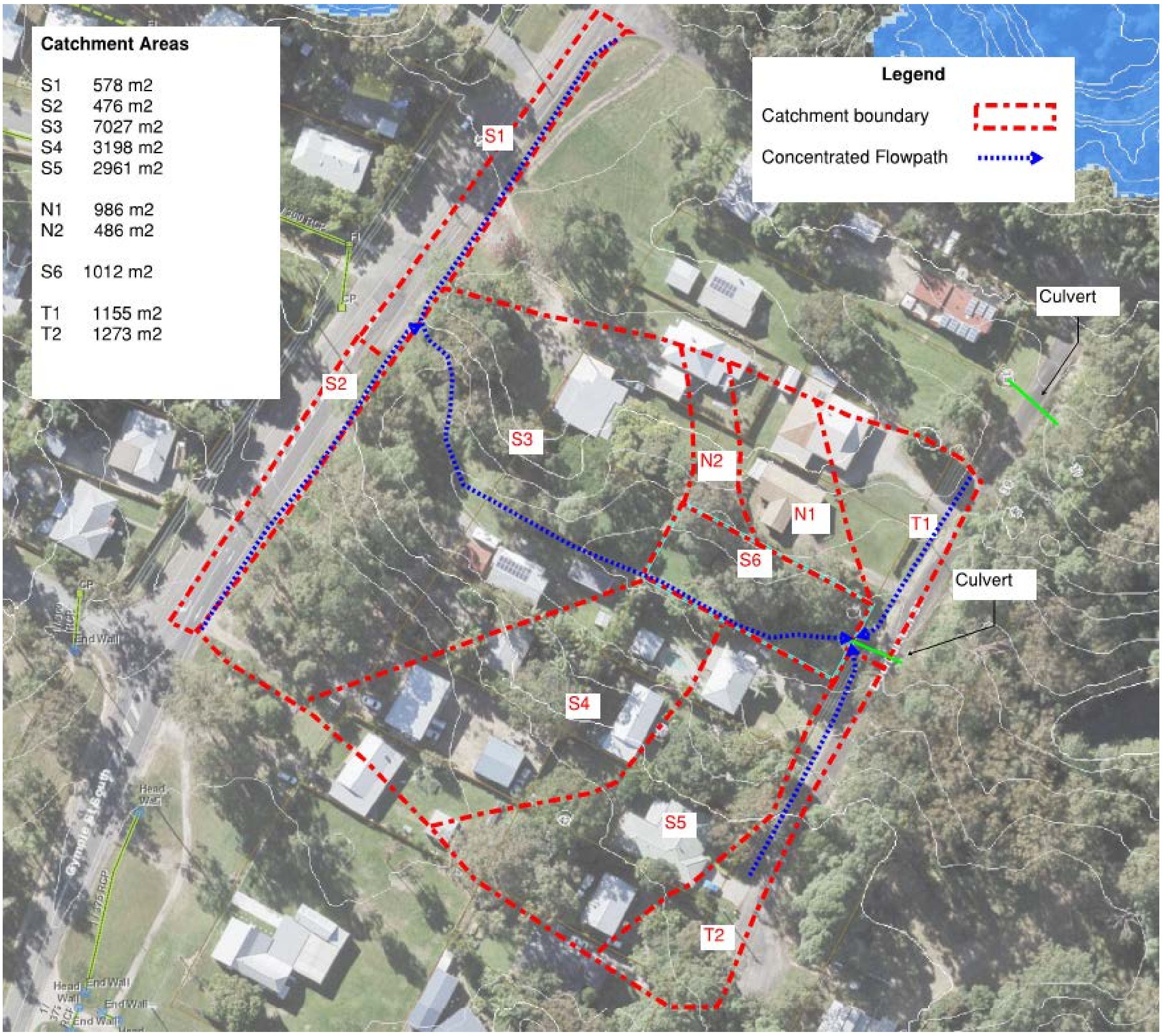


Figure H1 – Case Study 1 Catchment Plan

QUDM Rational Method

An estimate of the peak 1%AEP design flow was firstly obtained using the QUDM Rational Method from the procedures documented in Chapter 4 of QUDM V4 (IPWEA, 2017).

The general form of the QUDM Rational Method is:

Q = CIA/360

Where:

- Q (m3/s) = peak flow for the annual exceedance probability (AEP) under consideration
- C = coefficient of discharge and is related to fraction imperviousness and the AEP being considered
- I (mm/hr) = average rainfall intensity for the time of concentration and AEP being considered
- A (ha) = catchment area

Time of Concentration

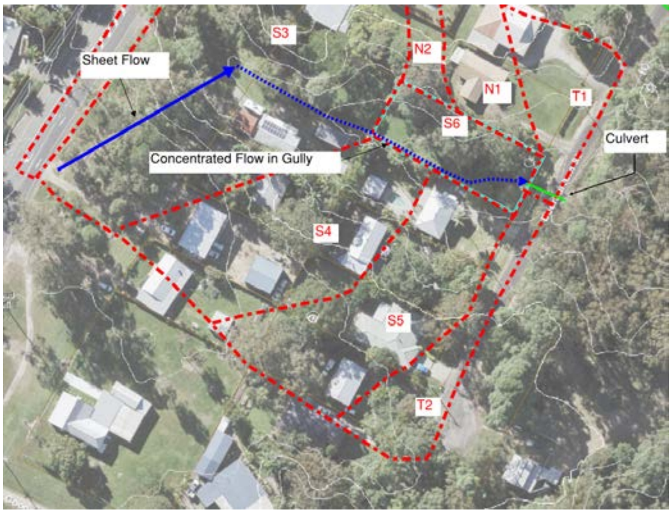
The time of concentration is the time required for flow from the most remote part of the catchment to reach the outlet.

The study catchment was discretised into sub-catchments as shown in Figure H1 to better understand the different components of flow travel time and whether the partial area effect was likely to be a consideration.

From Figure H1 it is not immediately clear which of the possible flow paths will take the longest time to reach the catchment outlet. The time of concentration was therefore estimated for 3 possible flowpaths in order to determine which was the longest travel time. The options investigated are described below and shown in Figure H2:

- Flow path 1 – Concentrated flow in table drain in sub-catchment S1 followed by concentrated flow in gully in sub-catchments S3 and S6
- Flow path 2 – Sheet flow in the upper portion of sub-catchment S3 followed by concentrated flow in the gully in sub-catchments S3 and S6
- Flow path 3 - Sheet flow in sub-catchment S5 followed by concentrated flow in the gully in the lower portion of sub-catchment S6

Flow path 1



Flow path 2

Flow path 3

Figure H2 – Flow Path Options Investigated

Flow Path 1

The components of travel time for flow path 1 are:

- Travel time in table drain in sub-catchment S1. Estimate obtained using QUDM Figure 4.8 based on length of 86m and fall of 2m. Yields travel time of 1.2 minutes
- Travel time in gully in sub-catchments S3 and S6. Estimate obtained using QUDM Figure 4.8 based on length of 140m and fall of 4m. Yields travel time of 1.6 minutes
- Total estimated travel time is 1.2 minutes + 1.6 minutes = 2.8 minutes ~ 3 minutes

Flow Path 2

The components of travel time for flow path 2 are:

- Sheet flow in the upper portion of sub-catchment S3. Estimate obtained using QUDM Figure 4.6 based on length of 77m, slope of 6.5% and surface type ‘average grassed’. Yields travel time of 14 minutes.

- Travel time in gully in sub-catchments S3 and S6. Estimate obtained using QUDM Figure 4.8 based on length of 105m and fall of 2m. Yields travel time of 1.6 minutes
- Total estimated travel time is 14minutes + 1.6minutes = 15.6 minutes ~ 16minutes

Flow Path 3

The components of travel time for flow path 3 are:

- Sheet flow in sub-catchment S5. Estimate obtained using QUDM Figure 4.6 based on length of 95m, slope of 5% and surface type 'average grassed'. Yields travel time of 16 minutes.
- Travel time in gully in sub-catchment S6. Estimate obtained using QUDM Figure 4.8 based on length of 20m and fall of 0.4m. Yields travel time of <1 minute
- Total estimated travel time is ~ 16minutes

From the above analysis it can be seen that Flow paths 2 and 3 yield the same estimate of travel time, so **16 minutes** was adopted as the time of concentration.

The average rainfall intensity (I) corresponding to a duration of 16 minutes for the 1%AEP design event was determined by accessing the BoM IFD generator for the study location:

http://www.bom.gov.au/water/designRainfalls/revised-ifd/?year=2016

From this an average rainfall intensity for the 1% AEP event was determined as **I = 213 mm/hr**

Coefficient of Discharge

The *Sunshine Coast Planning Scheme 2014* requires that drainage design account for a fully developed upstream catchment (*Planning Scheme Policy for Development Works*). This means that the drainage design should consider all landuse within the catchment to be developed in accordance with the zone designations of the planning scheme, where such designations result in an intensification over the existing uses.

The zone maps for the study catchment were therefore reviewed and it was determined that all of the study catchment is designated as being within the Rural Residential Zone. The Sunshine Coast Planning Scheme provides minimum fraction impervious (fi) and coefficient of discharge values which override QUDM, and for this zoning are *(Table SC6.14.6 Planning Scheme Policy for Development Works*):

- Fraction impervious = fi =0.60
- The coefficient of discharge for the 10%AEP (C_{10})= 0.82

It is clear from the aerial imagery that the fi stipulated by the planning scheme is greater than currently exists within the catchment, so the above values were adopted for the analysis.

The frequency factor F for the 1%AEP event is F = 1.20

Therefore, the coefficient of discharge for the 1%AEP design event is $C = F \times C_{10} = 1.2 \times 0.82 = 0.98$

The Peak flow estimate is then estimated from the QUDM Rational method formula:

$Q_{1\%AEP}$ = CIA/360 = 0.98 * 213 (mm/hr) * 1.92 (ha) / 360 = **1.12 m^3/s**

The *Sunshine Coast Planning Scheme 2014* also requires that allowance is made in drainage design for the future effects of climate change. Such allowance is to be provided by increasing design rainfall intensities by 20%.

So, the design discharge estimate, including allowance for climate change is:

$Q_{1\%AEP\ CC}$ = CIA/360 = 0.98 * 1.2 * 213 (mm/hr) * 1.92 (ha) / 360 = **1.34 m^3/s**

SCC Rational Method

Time of Concentration

SCC (2017) provides two different methods for determining time of concentration (TOC) depending on whether the situation being analysed is 'pre-development' or 'post development'. Where drainage infrastructure is being designed (such as in this case study) then the 'post development' TOC methods should be used.

The 'pre-development' TOC methods are mainly applicable when an analysis is required of the likely change in flowrate following development within a catchment.

The SCC (2017) time of concentration formula has the following form:

$$ToC = a.Area^b \qquad \text{SCC (2017)}$$

Where:

$$a = c.Slope^d$$

$$b = e.Slope^f$$

And,

Table 12 SCC ToC Formula Parameters for post development

Parameter	Concentrated/Channelised
a	$c.Slope^d$
b	$e.Slope^f$
c	$(0.8998\ CF + 12.574).e^{(0.0046\ CF - 0.8057)\ f_i}$
d	$(-0.0029\ CF - 0.0011).f_i - (-0.0111\ CF - 0.0845)$
e	$-0.0074.f_i + (0.01\ CF + 0.3814)$
f	$(0.0013\ CF - 0.0041).f_i - (-0.0054\ CF - 0.0245)$

The key parameters are therefore slope (m/m) and fraction imperviousness (decimal) as well as catchment shape factor (CF).

Catchment slope is determined over the entire flow length involving the longest section of sheet flow to the catchment outlet. The flow path used to determine catchment slope is shown in Figure A3. This flow path had a length of 115m and a fall of 5.5m, yielding a slope of 0.05 (m/m).

Fraction imperviousness is determined in the same method as the QUDM example above and is fi=0.60 based on planning scheme requirements consistent with the zoning of the catchment.

The catchment shape factor (CF) is derived from the following equation:

$$CF = -3.688. LN\left(\frac{0.5\sqrt{Area}}{Centroid}\right) - 0.9195$$

The location of the catchment centroid was estimated by defining the catchment in a GIS package and using the GIS tool for estimating centroid location. The "Centroid" parameter in the above equation is the distance from the catchment outlet to the catchment centroid in metres. The distance was estimated as 100m.

Substituting these values into the above equations yields an estimated time of concentration of **9 minutes**.

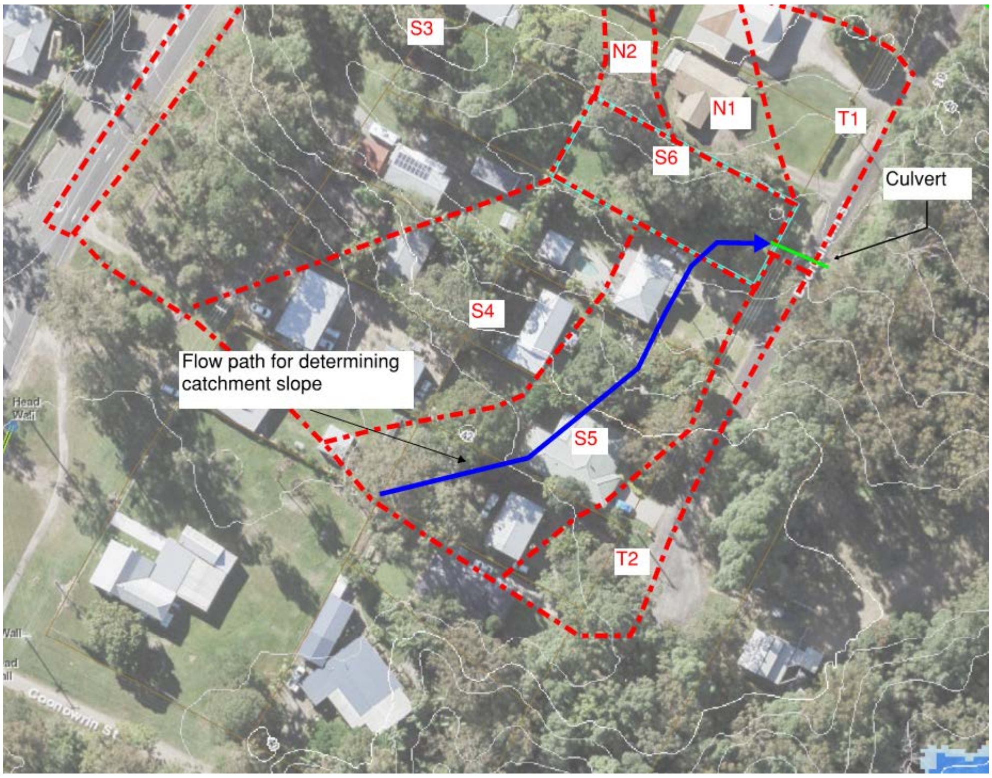


Figure H3 – Flow Path for Determining Catchment Slope

The average rainfall intensity (I) corresponding to a duration of 9 minutes for the 1%AEP design event was determined by accessing the BoM IFD generator for the study location:

http://www.bom.gov.au/water/designRainfalls/revised-ifd/?year=2016

From this an average rainfall intensity was determined as **I = 270 mm/hr**

Coefficient of Discharge

SCC (2017) provides C_{10} values by fraction impervious in Table 13 which is reproduced below. Based on the adopted fraction impervious the estimated C_{10} value is 0.88.

Table 13 C_{10} Factor

F_i	C_{10}
≤ 0	0.80
0.1	0.81
0.2	0.82
0.3	0.83
0.4	0.84
0.5	0.85
0.6	0.86
0.7	0.87
0.8	0.88
0.9	0.89
≥ 1.0	0.9

SCC (2017) adopts a frequency factor of 1.1 for the 1%AEP event. Therefore, the coefficient of discharge for the 1%AEP design event is $C = F \times C_{10} = 1.1 \times 0.88 = 0.97$

The Peak flow estimate is then estimated from the QUDM Rational method formula:

$Q_{1\%AEP}$ = CIA/360 = 0.97 * 270 (mm/hr) * 1.92 (ha) / 360 = **1.39 m^3/s**

So, the design discharge estimate, including allowance for climate change is:

$Q_{1\%AEP\ CC}$ = CIA/360 = 0.97 * 1.2 * 270 (mm/hr) * 1.92 (ha) / 360 = **1.67 m^3/s**

Comparison and Conclusions

The methods documented in SCC (2017) yield a significantly shorter time of concentration than QUDM methods for the study catchment. Accordingly, the SCC (2017) rational method yields a peak flow estimate which is 25% higher than the QUDM Rational method estimate for the 1%AEP design event.

Part of the reason for this difference is that the QUDM TOC method is based on existing conditions/flowpaths, where-as the SCC (2017) method directly takes account of the development potential (zoning) in the estimate of TOC. The QUDM TOC method can only account for development potential if many assumptions and a conceptual design is undertaken for the possible future drainage infrastructure in the catchment, which in this example would be impractical.

The SCC (2017) method therefore provides a solution which is more robust in terms of catering for future development potential in the catchment which can affect TOC as well as fi and C_{10} parameters.

Case Study 2 – Urban In-Fill Development

Background

An urban infill development to create 4 residential lots is proposed on a lot of area 3,022m^2. The lot is currently undeveloped and contains grass and scattered trees. The surrounding areas are low density residential. The lot drains towards a downstream private lot that contains a stormwater inlet (headwall). The catchment characteristics are shown in Figure H4 below and shows catchment areas contributing to the existing headwall, which is the location of interest.

A hydrologic model is required in order to assess the magnitude of the changes in stormwater characteristics resulting from the development of the site, and to design mitigation strategies such as detention facilities.

Rational method calculations are required in order to verify the accuracy of the hydrologic model at predicting peak flows at the subject location.

The below analysis provides a comparison of peak flow estimates derived using the QUDM Rational Method and the SCC Rational method. Estimates were obtained for the following scenarios:

1. Natural (i.e. no development in the catchment);
2. Pre-development (prior to development of the study site – i.e. current conditions); and
3. Post development (following development of the site)

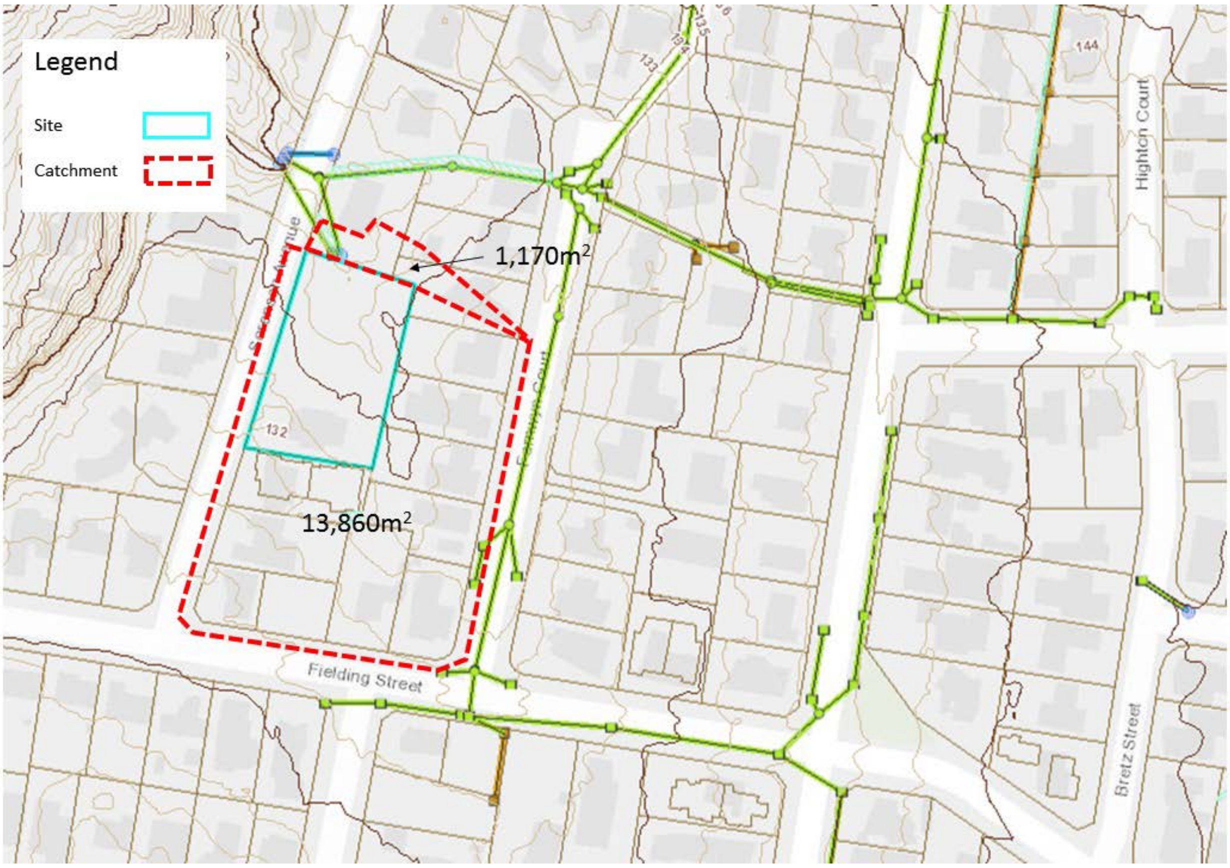


Figure H4 – Case Study 2 Catchment Plan

Natural Catchment Conditions

Although natural catchment conditions are not reflective of current or future conditions, the data on which the SCC 2017 Rational Method is based were largely obtained from natural catchments and hence it is these types of catchment which the method can be expected to be most reliable for.

Derivation of peak flows for this scenario therefore so provides a useful point for comparison to the hydrologic model before alterations to the model landuse are made to reflect either the pre-development or post-development scenarios.

QUDM Rational Method

Time of Concentration

The time of concentration is the time required for flow from the most remote part of the catchment to reach the outlet. For the natural scenario it is assumed that there would no formal stormwater drainage. Stormwater is assumed to sheet flow from the surface areas of the existing residential lots and then concentrate into a swale as it flows through the development site.

The components of travel time are therefore:

- Sheet flow in existing residential lots. Estimate obtained using QUDM Figure 4.6 based on length of 69m, slope of 5% and surface type ‘average grassed’. Yields travel time of 15 minutes.
- Travel time in swale through the development site. Estimate obtained using QUDM Figure 4.8 based on length of 67m and fall of 2m. Yields travel time of <1 minute
- Total estimated travel time is ~ 15minutes

The average rainfall intensity (I) corresponding to a duration of 15 minutes for a range of recurrence interval design events was determined by accessing the BoM IFD generator for the study location:

http://www.bom.gov.au/water/designRainfalls/revised-ifd/?year=2016

Coefficient of Discharge

An impervious fraction (fi) of zero is assumed for the natural scenario and the corresponding C_{10} value obtained from Table SC6.14.6 (Planning Scheme Policy for Development Works) is:

$C_{10} = 0.70$

The standard QUDM Frequency Factors were applied for each recurrence interval along with the corresponding IFD data in order to derive the pre-development peak flow estimates provided in Table A1.

Table A1 – QUDM Rational Method – Natural Peak Flow Estimates

EY/AEP	F	C_{10}	I (mm/hr)	A (ha)	Q (m^3/s)
0.5EY	0.85	0.70	111	1.503	0.28
0.2EY	0.95	0.70	135	1.503	0.37
10%	1.00	0.70	154	1.503	0.45
2%	1.15	0.70	202	1.503	0.68
1%	1.20	0.70	223	1.503	0.78

SCC Rational Method

Time of Concentration

SCC (2017) provides two different methods for determining time of concentration (TOC) depending on whether the situation being analysed is 'pre-development' or 'post development'.

Within the pre-development method, there are a further two sets of formulas for determining time of concentration depending on whether flows are predominantly sheet flow or channelized.

Due to the presence of the swale through the majority of the development site it was determined that the concentrated flow form of the SCC (2017) equations for estimating time of concentration were most appropriate.

The SCC (2017) time of concentration formula has the following form:

$$ToC = a.Area^{b} \qquad \text{SCC (2017)}$$

Where:

$a = c.Slope^{d}$

$b = e.Slope^{f}$

and,

Table 10 SCC ToC Formula Parameters for pre development inflow calculations

Parameter	Concentrated/Channelised	Sheet Flow
Catchment size	*All*	*<2 ha*
a	$c.Slope^{d}$	
b	$e.Slope^{f}$	
c	$(0.8998\ CF + 12.574).e^{(0.0046\ CF - 0.8057)\ Z}$	$22.3784.e^{-0.8185\ Z}$
d	$(-0.0029\ CF - 0.0011).Z - (-0.0111\ CF - 0.0845)$	$-0.00748.Z - 0.07817$
e	$-0.0074.Z + (0.01\ CF + 0.3814)$	$-0.00699.Z + 0.38949$
f	$(0.0013\ CF - 0.0041).Z_{i} - (-0.0054\ CF - 0.0245)$	0

The key parameters are therefore slope (m/m), catchment shape (CF) and Z parameter.

Catchment slope is determined over the entire flow length, with the flow path having a length of 136m and a fall of 6.5m, yielding a slope of 0.05 (m/m).

The Z parameter is determined based on the values in Table 11 which is reproduced below, with a value of -0.2 being selected as appropriate.

Table 11 Simplified Z Parameters for pre development

Surface	Z
Pavement	1.0
Medium Density	0.6
Low Density	0.4
Bare Soil	0.1
Poorly Grassed	0
Average Grassed	-0.2
Densely Grassed	-0.5

The catchment shape factor (CF) is derived from the following equation:

$$CF = -3.688.LN\left(\frac{0.5\sqrt{Area}}{Centroid}\right) - 0.9195$$

The location of the catchment centroid was estimated by defining the catchment in a GIS package and using the GIS tool for estimating centroid location. The “Centroid” parameter in the above equation is the distance from the catchment outlet to the catchment centroid in metres. The distance was estimated as 90m.

Substituting these values into the above equations yields an estimated time of concentration of **15 minutes**.

Coefficient of Discharge

SCC (2017) provides C_{10} values by fraction impervious in Table 12 which is reproduced below. A natural catchment has fi=0, yielding an estimated C_{10} value of 0.80.

Table 13 C_{10} Factor

F_i	C_{10}
≤ 0	0.80
0.1	0.81
0.2	0.82
0.3	0.83
0.4	0.84
0.5	0.85
0.6	0.86
0.7	0.87
0.8	0.88
0.9	0.89
≥ 1.0	0.9

SCC (2017) adopts frequency factors for each recurrence event based on the values reproduced in Table 14 below.

Table 14 Frequency Factor

		Frequency factor (Fy)		
AEP (%)	ARI (years)	QUDM	Calibrated	Adopted
63%	1	0.8		**0.9**
39%	2	0.85		**0.925**
18%	5	0.95		**0.975**
10%	10	1	0.8	**1**
5%	20	1.05		**1.025**
2%	50	1.15		**1.075**
1%	100	1.2	0.88	**1.1**

The peak flow estimates based on the above parameters are summarised in Table H2.

Table H2 – SCC (2017) Rational Method – Natural Peak Flow Estimates

EY/AEP	F	C_{10}	I (mm/hr)	A (ha)	Q (m^3/s)
0.5EY	0.925	0.80	111	1.503	0.34
0.2EY	0.975	0.80	135	1.503	0.44
10%	1	0.80	154	1.503	0.51
2%	1.075	0.80	202	1.503	0.73
1%	1.1	0.80	223	1.503	0.82

It can be seen from Tables H1 and H2 that the SCC 2017 and QUDM estimates agree closely for the 1% AEP event. The more frequent events the estimates derived using SCC 2017 are up to 24% greater due to a combination of the higher frequency factors and higher coefficient of runoff used in SCC 2017 compared to QUDM.

2. Pre-Development

QUDM Rational Method

Time of Concentration

For the pre-development situation (i.e. current situation), there is no formal stormwater drainage upstream of the location of interest (the headwall inlet). Stormwater sheet flows from the surface areas of the existing residential lots and then concentrates into a swale as it flows through the development site.

The components of travel time are therefore:

- Sheet flow in existing residential lots. Given the degree of uncertainty surrounding overland flow within an urban catchment, adopting standard inlet times is most appropriate. This is the time for flows to reach the point where the first inlet to the drainage system should occur (i.e. the point where an inter-allotment drainage inlet should, but doesn't, exist). The applicable standard inlet time for the catchment slope is 13 minutes.
- Travel time in swale through the development site. Estimate obtained using QUDM Figure 4.8 based on length of 67m and fall of 2m. Yields travel time of <1 minute
- Total estimated travel time is ~ 13minutes

The average rainfall intensity (I) corresponding to a duration of 13 minutes for a range of recurrence interval design events was determined by accessing the BoM IFD generator for the study location:

http://www.bom.gov.au/water/designRainfalls/revised-ifd/?year=2016

Coefficient of Discharge

The pre-development catchment is not homogeneous and contains both undeveloped and existing residential landuse. Care must be taking in applying the QUDM rational method to such non-homogenous catchment situations. In this situation the runoff from pervious and impervious surfaces are intermixed and an average of the impervious fraction (fi) and coefficient of runoff is appropriate.

An area-weighted approach to estimating C_{10} was adopted as follows:

$$C_{10} = \frac{(C_{10} \text{ of site}) \times (\text{site area}) + (C_{10} \text{ of existing residential}) \times (\text{area of existing residential})}{\text{Total catchment area}}$$

Using the C_{10} values from Table SC6.14.6 (Planning Scheme Policy for Development Works):

$C_{10} = (0.70 \times 3022m^2 + 0.86 \times 12008m^2) / 15030m^2 = 0.83$

The standard QUDM Frequency Factors were applied for each recurrence interval along with the corresponding IFD data in order to derive the pre-development peak flow estimates provided in Table H3.

Table A3 – QUDM Rational Method – Pre-Development Peak Flow Estimates

EY/AEP	F	C_{10}	I (mm/hr)	A (ha)	Q (m^3/s)
0.5EY	0.85	0.83	119	1.503	0.35
0.2EY	0.95	0.83	145	1.503	0.48
10%	1.00	0.83	165	1.503	0.57
2%	1.15	0.83	217	1.503	0.86
1%	1.20	0.83	240	1.503	1.00

SCC Rational Method

Time of Concentration

Due to the urban context and presence of the swale within the development site, it was determined that the concentrated flow form of the SCC (2017) equations for estimating time of concentration were most appropriate for the pre-development scenario.

The SCC (2017) time of concentration formula has the following form:

$$ToC = a.Area^{b} \qquad \text{SCC (2017)}$$

Where:

$a = c.Slope^{d}$

$b = e.Slope^{f}$

And,

Table 10 SCC ToC Formula Parameters for pre development inflow calculations

Parameter	Concentrated/Channelised	Sheet Flow
Catchment size	*All*	*<2 ha*
a	$c.Slope^{d}$	
b	$e.Slope^{f}$	
c	$(0.8998\ CF + 12.574).e^{(0.0046\ CF - 0.8057)\ Z}$	$22.3784.e^{-0.8185\ Z}$
d	$(-0.0029\ CF - 0.0011).Z - (-0.0111\ CF - 0.0845)$	$-0.00748.Z - 0.07817$
e	$-0.0074.Z + (0.01\ CF + 0.3814)$	$-0.00699.Z + 0.38949$
f	$(0.0013\ CF - 0.0041).Z_i - (-0.0054\ CF - 0.0245)$	0

The key parameters are therefore slope (m/m), catchment shape (CF) and Z parameter.

Catchment slope is determined over the entire flow length, with the flow path having a length of 136m and a fall of 6.5m, yielding a slope of 0.05 (m/m).

The Z parameter is determined based on the values in Table 11 which is reproduced below.

Table 11 Simplified Z Parameters for pre development

Surface	Z
Pavement	1.0
Medium Density	0.6
Low Density	0.4
Bare Soil	0.1
Poorly Grassed	0
Average Grassed	-0.2
Densely Grassed	-0.5

An overall area-weighted "Z" parameter was therefore adopted as follows:

$$Z = \frac{(\text{"Z" of site}) \times (\text{site area}) + (\text{"Z" of existing residential}) \times (\text{area of existing residential})}{\text{Total catchment area}}$$

$Z = (-0.2 \times 3022\text{m}^2 + 0.6 \times 12008\text{m}^2) / 15030\text{m}^2 = 0.44$

The catchment shape factor is unchanged from the "natural" scenario.

Substituting these values into the above equations yields an estimated time of concentration of **9 minutes**.

Coefficient of Discharge

SCC (2017) provides C_{10} values by fraction impervious in Table 12 which is reproduced below.

The overall fraction impervious can be estimated based on an area-weighted calculation as follows:

$$fi = \frac{(\text{fi of site}) \times (\text{site area}) + (\text{fi of existing residential}) \times (\text{area of existing residential})}{\text{Total catchment area}}$$

$fi = (0.0 \times 3022\text{m}^2 + 0.80 \times 12008\text{m}^2) / 15030\text{m}^2 = 0.64$

Based on the adopted fraction impervious the estimated C_{10} value is 0.86.

Table 13 C_{10} Factor

F_i	C_{10}
≤ 0	0.80
0.1	0.81
0.2	0.82
0.3	0.83
0.4	0.84
0.5	0.85
0.6	0.86
0.7	0.87
0.8	0.88
0.9	0.89
≥ 1.0	0.9

SCC (2017) adopts frequency factors for each recurrence event based on the values reproduced in Table 14 below.

Table 14 Frequency Factor

		Frequency factor (Fy)		
AEP (%)	ARI (years)	QUDM	Calibrated	Adopted
63%	1	0.8		**0.9**
39%	2	0.85		**0.925**
18%	5	0.95		**0.975**
10%	10	1	0.8	**1**
5%	20	1.05		**1.025**
2%	50	1.15		**1.075**
1%	100	1.2	0.88	**1.1**

The peak flow estimates based on the above parameters are summarised in Table H4.

Table H4 – SCC (2017) Rational Method – Pre-Development Peak Flow Estimates

EY/AEP	F	C_{10}	I (mm/hr)	A (ha)	Q (m^3/s)
0.5EY	0.925	0.86	137	1.503	0.46
0.2EY	0.975	0.86	168	1.503	0.59
10%	1.00	0.86	191	1.503	0.69
2%	1.075	0.86	253	1.503	0.98
1%	1.1	0.86	280	1.503	1.11

It can be seen from Tables H3 and H4 that the 1%AEP peak flow estimates derived using SCC 2017 are 10% greater than those derived following the QUDM methodology due to both a higher coefficient of runoff and shorter TOC.

Post-Development

A concept drainage design was developed for the site and incorporates a rear-of-allotment drainage system along the eastern and northern site boundaries. This system will service the proposed new lots within the site, as well as providing a point of connection for the existing residential areas which discharge flows towards the site.

The development has the potential to increase peak flows discharged downstream due to the increased imperviousness of the site and the faster conveyance of flows through the rear-of-allotment drainage system.

QUDM Rational Method

Time of Concentration

The components of travel time for the developed site are:

- Standard inlet time of the existing residential areas to the proposed rear-of-allotment system. Based on catchment slope this is 13 minutes; and
- Conveyance time within the proposed rear-of-allotment system. Estimate obtained using QUDM Figure 4.8 based on length of 67m and fall of 2m. Yields travel time of <1 minute

The time of concentration is therefore estimated to be ~**13 minutes**. It is unsurprising that this value is unchanged from the pre-development scenario, as most of the catchment is already urbanised and flow through the study site is already concentrated/channelized in the current situation.

Coefficient of Discharge

The catchment will be a homogeneous low density residential landuse post development. The applicable C_{10} value from Table SC6.14.6 (Planning Scheme Policy for Development Works) is 0.86.

The resulting peak flows based on the revised C_{10} and TOC estimates are provided in Table H5. These flows are approximately **4%** greater than the pre-development peak flows based on the same (QUDM) methodology.

Table H5 – QUDM Rational Method – Post-Development Peak Flow Estimates

EY/AEP	F	C_{10}	I (mm/hr)	A (ha)	Q (m^3/s)
0.5EY	0.85	0.86	119	1.503	0.36
0.2EY	0.95	0.86	145	1.503	0.49
10%	1.00	0.86	165	1.503	0.59
2%	1.15	0.86	217	1.503	0.89
1%	1.20	0.86	240	1.503	1.03

From Table H3 and Table H5 it can be seen that the development of the site is predicted to result in an increase in peak flows of around 4% using the QUDM methodology.

SCC Rational Method

Time of Concentration

The post-development form of the SCC (2017) rational method equations is reproduced in Table 12 below.

Table 12 SCC ToC Formula Parameters for post development

Parameter	Concentrated/Channelised
a	$c.Slope^{d}$
b	$e.Slope^{f}$
c	$(0.8998\ CF + 12.574).e^{(0.0046\ CF - 0.8057)\ f_i}$
d	$(-0.0029\ CF - 0.0011).f_i - (-0.0111\ CF - 0.0845)$
e	$-0.0074.f_i + (0.01\ CF + 0.3814)$
f	$(0.0013\ CF - 0.0041).f_i - (-0.0054\ CF - 0.0245)$

The key parameters are therefore slope (m/m), catchment shape factor and fraction imperviousness (decimal).

Catchment slope is determined over the entire flow length, with the flow path having a length of 136m and a fall of 6.5m, yielding a slope of 0.05 (m/m).

Fraction imperviousness is determined in the same method as the QUDM example above and is fi=0.80 based on planning scheme requirements consistent with the zoning of the catchment.

The location of the catchment centroid is unchanged from earlier scenarios.

Substituting these values into the above equations yields an estimated time of concentration of **7 minutes**.

Coefficient of Discharge

SCC (2017) provides C_{10} values by fraction impervious in Table 13. Based on the adopted fraction impervious the estimated C_{10} value is 0.88.

The peak flow estimates based on the above parameters are summarised in Table H6. These flows are approximately **11-14%** greater than the pre-development peak flows based on the same (SCC 2017) methodology.

Table H6 – SCC (2017) Rational Method – Post-Development Peak Flow Estimates

EY/AEP	F	C_{10}	I (mm/hr)	A (ha)	Q (m^3/s)
0.5EY	0.925	0.88	149	1.503	0.51
0.2EY	0.975	0.88	183	1.503	0.66
10%	1	0.88	210	1.503	0.77
2%	1.075	0.88	280	1.503	1.11
1%	1.1	0.88	311	1.503	1.26

From Tables H4 and H6 it can be seen that peak flows are predicted to increase by 11-13% using the SCC 2017 methodology. This is a significantly greater increase than predicted using QUDM.

Comparison and Conclusions

A summary of results is provided in Table H7 for the 1%AEP event. It can be seen that the SCC (2017) methodology predicts greater peak flows compared to QUDM for all scenarios. The difference between the methodologies becomes more pronounced as the imperviousness of the scenarios increases.

Table H7 – Comparison of Peak Flow Estimates for 1%AEP Event (m^3/s)

Methodology	Natural	Pre	Post
SCC 2017	0.82	1.11	1.26
QUDM	0.78	1.00	1.03

Next Steps

A hydrologic model will be developed as the next step. The model should initially be based on a fully naturalised catchment in order to verify the model predictions against the SCC 2017 Rational method estimates. The landuse within the model will then be altered to reflect pre and post development scenarios.

How the model is applied and what the design discharge characteristics are for the site, will depend on further investigation and consideration of the following questions:

1. Is detention already provided to the existing residential parts of the catchment?
2. What is the capacity of the existing downstream drainage network?
3. Are there existing flooding or drainage issues downstream of the site?

Where there is an opportunity to resolve existing drainage issues downstream of the site then discussion should be had with Council to determine if part of the works could qualify as Trunk Infrastructure

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