Brisbane River Catchment Flood Study: Comprehensive Hydrologic Assessment

Draft Final Hydrology Report

Prepared for the State of Queensland (acting through): Department of State Development, Infrastructure and Planning/Department of Natural Resources and Mines

15 May 2015
Revision: 2
Reference: 238021

Document control record

Document prepared by:

Aurecon Australasia Pty Ltd
ABN 54 005 139 873
Level 14, 32 Turbot Street
Brisbane QLD 4000
Locked Bag 331
Brisbane QLD 4001
Australia
T +61 7 3173 8000
F +61 7 3173 8001
E brisbane@aurecongroup.com
W aurecongroup.com

Document control						aurecon
Report title		Draft Final Hydrology Report				
Document ID		238021-0000-REP-WW-0005	**Project number**	238021		
File path		**238021-0000-REP-WW-0005_Draft Final Hydrology Report.docx**				
Client		Prepared for the State of Queensland (acting through): Department of State Development, Infrastructure and Planning/Department of Natural Resources and Mines			**Client contact**	Pushpa Onta
Rev	**Date**	**Revision details/status**	**Prepared by**	**Author**	**Verifier**	**Approver**
A	31 October 2014	Internal Review	C Smyth	R Ayre F Diermanse L Toombes P Hart	T Graham	C Berry
B	1 December 2014	Client Review	C Smyth	R Ayre F Diermanse L Toombes P Hart	T Graham	C Berry
0	15 December 2014	Final Draft	C Smyth	R Ayre F Diermanse L Toombes P Hart	T Graham	C Berry
1	14 April 2015	Final	C Smyth	R Ayre F Diermanse L Toombes P Hart	T Graham	C Berry
2	6 May 2015	Final	C Smyth	R Ayre F Diermanse L Toombes P Hart	T Graham	C Berry
Current revision		**2**				

Approval			
Author signature		Approver signature	
Name	Rob Ayre (RPEQ 4887)	Name	Craig Berry (RPEQ 8153)
Title	Project Leader	Title	Project Director

Brisbane River Catchment Flood Study: Comprehensive Hydrologic Assessment

Date 15 May 2015
Reference 238021
Revision 2

Aurecon Australasia Pty Ltd
ABN 54 005 139 873
Level 14, 32 Turbot Street
Brisbane QLD 4000
Locked Bag 331
Brisbane QLD 4001
Australia
T +61 7 3173 8000
F +61 7 3173 8001
E brisbane@aurecongroup.com
W aurecongroup.com

Important things you should know about this report

Report subject to change

This report is subject to change as the assessments undertaken have been based solely upon hydrological modelling and are subject to continuous improvement. Aspects of these assessments that are affected by hydraulics will need to be verified during the hydraulic modelling phase. Therefore the estimates presented in this report should be regarded as interim and possibly subject to change as further iteration occurs in conjunction with the hydraulic modelling phase of the Brisbane River Catchment Flood Study.

Exclusive use

This report and hydrologic model data has been prepared by Aurecon at the request of the State of Queensland acting through the Department of State Development, Infrastructure and Planning ("Client").

The basis of Aurecon's engagement by the Client is that Aurecon's liability, whether under the law of contract, tort, statute, equity or otherwise, is limited as set out in the Conditions of Contract schedules: DSDIP-2077-13 and agreed variations to the scope of the contract (terms of the engagement).

Third parties

It is not possible to make a proper assessment of this report without a clear understanding of the terms of engagement under which the report has been prepared, including the scope of the instructions and directions given to and the assumptions made by the consultant who has prepared the report.

The report is scoped in accordance with instructions given by or on behalf of the Client. The report may not address issues which would need to be addressed by a third party if that party's particular circumstances, requirements and experience with such reports were known; and the report may make assumptions about matters of which a third party is not aware.

Aurecon therefore does not assume responsibility for the use of, or reliance on, the report by any third party and the use of, or reliance on, the report by any third party is at the risk of that party.

Limits on scope and information

Where the report is based on information provided to Aurecon by other parties including state agencies, local governments authorised to act on behalf of the client, and the Independent Panel of Experts appointed by the client, the report is provided strictly on the basis that such information that has been provided is accurate, complete and adequate. Aurecon takes no responsibility and disclaims all liability whatsoever for any loss or damage that the Client or any other party may suffer resulting from any conclusions based on information provided to Aurecon, except to the extent that Aurecon expressly indicates in the report or related and supporting documentation, including the hydrologic models, analytical tools and associated datasets and metadata, that it has accepted or verified the information to its satisfaction.

Legal documents

The report may contain various remarks about and observations on legal documents and arrangements such as contracts, supply arrangements, leases, licences, permits and authorities. A consulting engineer can make remarks and observations about the technical aspects and implications of those documents and general remarks and observations of a non-legal nature about the contents of those documents. However, as a Consulting Engineer, Aurecon is not qualified, cannot express and should not be taken as in any way expressing any opinion or conclusion about the legal status, validity, enforceability, effect, completeness or effectiveness of those arrangements or documents or whether what is provided for is effectively provided for. They are matters for legal advice.

Aurecon team

The Aurecon Team consists of Aurecon as lead consultant, supported by Deltares, Royal HaskoningDHV, and Don Carroll Project Management and Hydrobiology.

Acknowledgement

Team Aurecon would like to acknowledge the contribution of the Independent Panel of Experts in providing direction and insightful assistance during the development of the Monte Carlo Simulation Framework which was produced as part of this study. Team Aurecon would also like to express our appreciation of the assistance from members of the Technical Working Group in providing data and models as well as useful insights into the behaviour of the catchment response. Lastly Team Aurecon would like to thank the Steering Committee and the Project Director and Project Manager for providing the opportunity to be involved in such an important project for South-East Queensland.

Executive summary

Introduction

The Queensland Floods Commission of Inquiry Final Report (QFCOI, 2014), which was issued in March 2012, contains a recommendation that required a flood study be conducted of the Brisbane River catchment. In accordance with this recommendation, the State of Queensland is managing the conduct of this study, known as the Brisbane River Catchment Flood Study (BRCFS), in a number of separate phases, namely:

Phase 1: Data Collection, Collation, Review and Storage of Existing Data (complete)

Phase 2: Comprehensive Hydrologic Assessment (current)

Phase 3: Comprehensive Hydraulic Assessment

Phase 4: Brisbane River Floodplain Management Study (BRCFMS) and Brisbane River Floodplain Management Plan (BRCFMP)

The Aurecon Team was commissioned to undertake Phase 1 and Phase 2 of the study: the Comprehensive Hydrologic Assessment (CHA). This assessment has a requirement for various methodologies to be utilised and for the various methods to corroborate each other so that the most realistic estimates of flood flow and flood volume can be made for nominated locations throughout the Brisbane River catchment.

Overview

The purpose of the CHA is to develop and apply state of the art methods that produce consistent and robust hydrologic models and analytical techniques that will enable the CHA to provide best estimates of a range of flood flows and flood volumes for annual exceedance probabilities (AEP) across the entire Brisbane River system.

It is anticipated that the BRCFS would provide much of the technical data/information/knowledge about flood flow behaviour and characteristics across various sections of the entire Brisbane River catchment. The outcomes from the CHA will form part of the inputs to the Comprehensive Hydraulics Assessment which will allow flood levels and associated characteristics in lower Brisbane River and tributaries to be estimated. Together these studies will provide the information upon which the associated Brisbane River Floodplain Management Study (BRFMS) and the Brisbane River Floodplain Management Plan (BRFMP) will be subsequently prepared.

The study has produced a number of tools including consolidated stream flow rating curves, a comprehensive set of calibrated hydrologic models of the entire Brisbane River Catchment, a dam operations model of the flood mitigation dams and a Monte-Carlo Simulation framework for estimating stochastic design flood estimates. Some elements of these tools, such as the stream flow rating curves and calibrated hydrologic models, may require further refinement to ensure consistency between the routing characteristics of the hydrologic and subsequent hydraulic modelling phase.

A reconciliation process has been adopted to assimilate the estimates of peak flows and flood volumes for the different assessment methods for the various locations nominated for investigation. This process has endeavoured to make use of the strengths of each of the assessment techniques and the best use of the available data/information to produce consistent and robust estimates across the entire flood frequency range.

The resultant Monte-Carlo Simulation framework is considered to have advantages over more "traditional" approaches in flood risk analysis in that it explicitly considers all relevant physical processes that contribute to flood events. A practical disadvantage is that it is generally more complex to implement, but this has been addressed by constructing the MCS framework in the Delft-FEWS environment that enables efficient data management, manipulation and visualisation.

For the rainfall sampling scheme within the MCS Framework, three methods were tested: TPT, CRC-CH and CSS. Eventually, the TPT method was chosen as the preferred method for the *current study*, because this method provided the best match between the rainfall IFD curves on one hand and the available synthetic spatio-temporal rainfall patterns on the other hand. The other two methods (CSS and CRC-CH) are nevertheless considered very promising for future applications of Monte Carlo applications, especially if more synthetic spatio-temporal rainfall patterns become available.

A periodic review of the methods and data employed in this study is desirable, so that revision of the study can be performed at a time when and if more relevant information or alternate methodologies become available.

Study methodology, findings and outcomes

Methodology

In the CHA, three approaches are used to estimate peak discharges and flow volumes for a range of Annual Exceedance Probabilities (AEP):

1. Flood Frequency Analysis (FFA)
2. Design Event Approach (DEA)
3. Monte Carlo Simulation (MCS)

The DEA and MCS methods are both referred to as '*rainfall based methods*', as they both rely on rainfall statistics in combination with a rainfall-runoff model to compute peak flows and flow volumes at locations of interest. With the FFA method, peak flows and flow volumes for given AEPs are derived directly from observed flows.

The assessment accounts for two scenarios: the conditions referred to as *'no-dams condition'* and the *'with-dams condition'.* The dams referred to in the scenarios are the major water storages that exist within the catchment; these are Somerset Dam and Wivenhoe Dam, both of which have flood mitigation capability in addition to water supply functions. Other reservoirs considered are Cressbrook Creek, Lake Manchester, Moogerah and Perseverance dams. These are dams that are primarily water supply storages with fixed crest spillways which do not actively provide flood mitigation. Other minor dams exist in the catchment but these reservoirs were considered not to have significant impacts on regional flooding and so therefore they have not been considered.

Findings

For ‘no-dams conditions’, DEA and MCS results were available for all 23 locations of interest. However, this number was reduced to 22 as the estimates for Rifle Range Road on Lockyer Creek were only reliable for within bank flows. FFA results were available for 11 locations as limited or no reliable data was available for the other 12 locations. For the ‘no-dams conditions’, the reconciled design flows for the majority of the locations are based on a combination of:

- Empirical estimates from rated flow data for high values of the Annual Exceedance Probability (AEP)
- Flood frequency analysis results for intermediate values of the AEP
- Monte Carlo Simulations results for low values of AEP

The choice of bounds between the ‘high’, ‘intermediate’ and ‘low’ range of AEP’s differed per location. For locations for which no or limited reliable data on peak discharges were available and, hence, no FFA results as well, the reconciled design flows for the high and intermediate range of AEP values were based on data and FFA results of nearby stations. The reconciled estimates were successfully validated for spatial consistency.

Figure 0-1-1 presents the reconciled flood frequency curve for the Brisbane River at Brisbane City Gauge for the no-dams conditions.

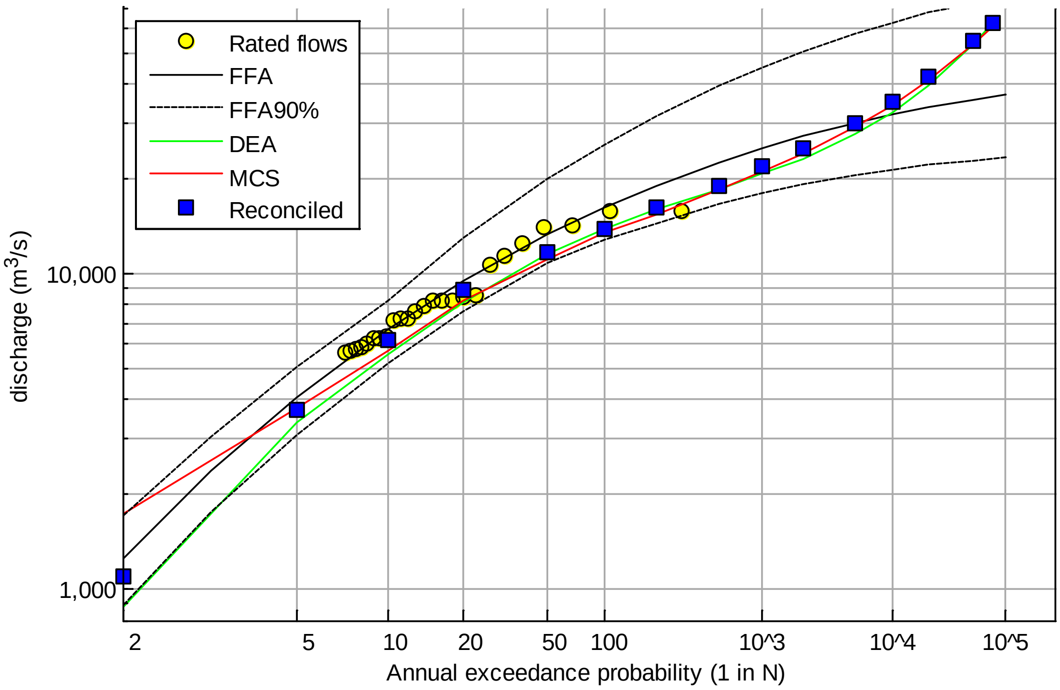


Figure 0-1-1 Reconciled flood frequency curve for Brisbane River at Brisbane City Gauge: no-dams conditions

For ‘with-dams conditions’, there are limited data series with sufficient length to justify conducting a FFA. Standard distributions such as LPIII and GEV are also not necessarily appropriate for the sites influenced by the flood mitigation dams. Therefore only DEA and MCS results were available for ‘with-dams conditions’, but these estimates have been compared against the available rated flow record where possible. Reconciled ‘with-dams conditions’ estimates were derived for the Stanley River at Somerset Dam, six locations along the Lower Brisbane River: Wivenhoe Dam, Savages Crossing, Mt

Crosby Weir, Moggill, Centenary Bridge and Brisbane City and for the Bremer River at Ipswich. For 'with-dams conditions', the reconciled design flows for these locations are based on a combination of:

- Empirical estimates from rated flows for (very) frequent events
- Monte Carlo Simulations results for frequent events to extreme events

These reconciled estimates were also successfully validated for spatial consistency.

Figure 0-1-2 presents the reconciled flood frequency curve for the Brisbane River at Moggill for the with-dams conditions.

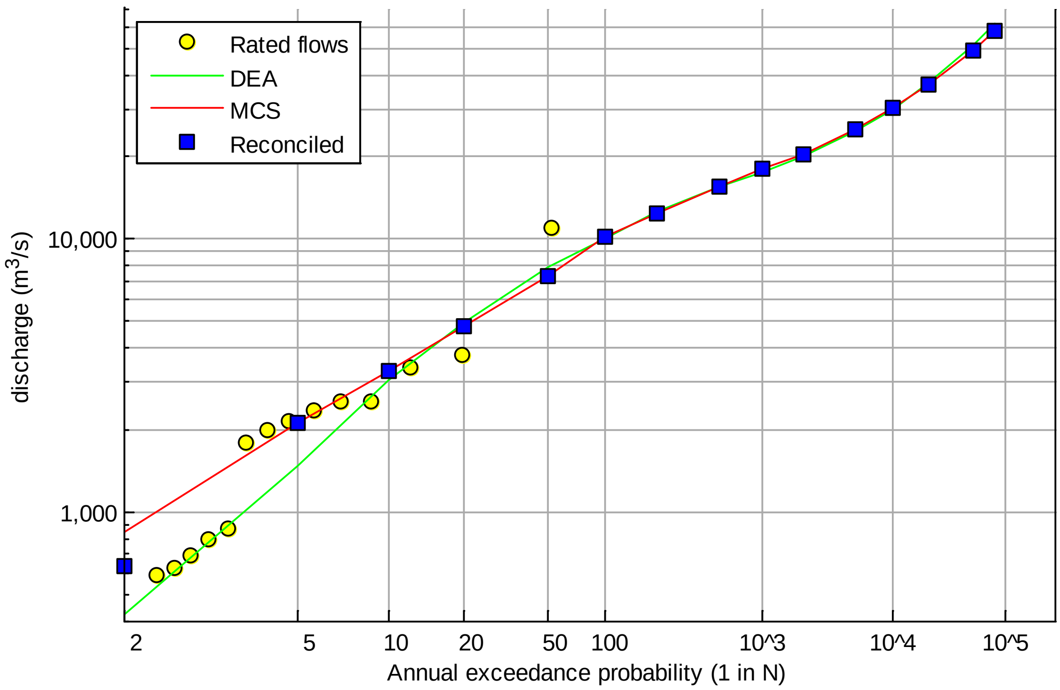


Figure 0-1-2 Reconciled flood frequency curve for Brisbane River at Moggill: with-dams conditions

Outcomes

The study has produced the following major outcomes in the form of:

- Rating Curves for Primary Stream Gauge Locations
- Calibrated Hydrologic Models of the entire Brisbane River Catchment
- A Joint Probability/Monte Carlo Simulation Framework implemented in a Delft-FEWS environment

These tools and methodologies have enabled the following estimates to be generated as part of the study:

- Estimates of probabilistic design flood outputs at nominated locations for a range of design floods ranging between 1 in 2 AEP to the Probable Maximum Flood for both the 'no-dams conditions' and the 'with-dams conditions'
- Estimates of peak flow and flood volume flood frequency curves at the nominated locations
- Ensembles of stochastic design flood hydrographs suitable for input into a range of detailed hydraulic models

- Ensembles of stochastic design ocean water level hydrographs, suitable for input into a range of detailed hydraulic models
- Design flood estimates at nominated locations for a range of AEPs based upon the standard design event approach
- An evaluation of the overall performance of the three methods: Flood Frequency Analysis; Design Event Approach and Monte-Carlo Simulation over the range of events investigated
- Reconciled and recommended design flood estimates for the nominated locations
- Estimates of the AEP of significant historic (actual) flood events

The reconciled and recommended estimates for the nominated locations within the Brisbane River Catchment for the design peak flow estimates are summarised in Table 0-1-1 and Table 0-1-2 for the no-dams conditions and with-dams conditions respectively. These estimates have been rounded to avoid providing the perception of unwarranted precision.

The effect of the presence of the flood mitigation dams and the four other water supply dams in the catchment has been assessed as part of the study by comparing the estimates of the two scenarios. The dams have been represented in their current configuration and the operation of the flood mitigation dams of Somerset Dam and Wivenhoe Dam is based upon the emergency response procedure specified in the 2013 version of the Flood Manual (Seqwater, 2013).

Figure 0-1-3 shows the comparison between no-dams conditions reconciled estimates and with-dams conditions reconciled estimates for the Brisbane River at Savages Crossing.

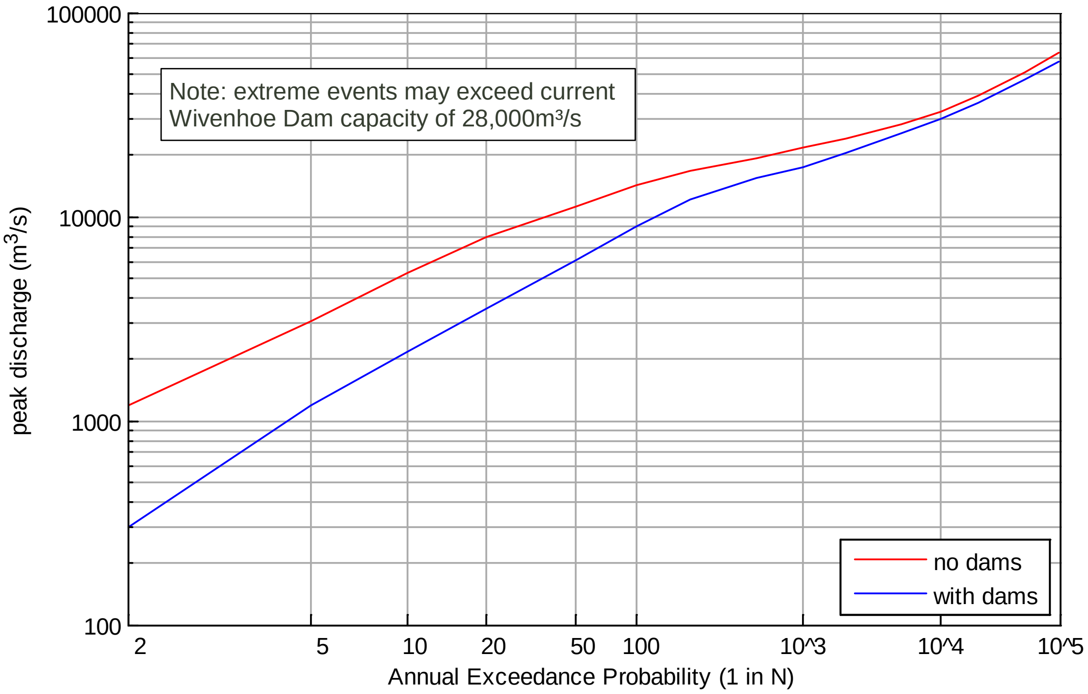


Figure 0-1-3 Reconciled flood frequency curve for Brisbane River at Savages Crossing: no-dams versus with-dams conditions

The comparison of the combined effect of the dams shows that the dams provide an impact of between 29 to 42% reduction in peak flow rates for the 1 in 100 AEP event for locations situated downstream of Wivenhoe Dam. Typically the reduction in peak flow is around 30%. The mitigation of peak flow extends over the full flood frequency range, although it diminishes for larger events. It should be recognised that for releases from Wivenhoe Dam that are in excess of 28,000 m^3/s (the maximum capacity of the main and auxiliary spillways), it is assumed that Wivenhoe Dam will not fail even though the dam will be overtopped by these larger events. This is a non-conservative assumption and so the estimates in excess of this value downstream of Wivenhoe Dam should be treated with caution.

The estimates for Ipswich are based only upon flows emanating from the Bremer River catchment, and therefore the estimates do not necessarily reflect the effects of the Brisbane River. This effect can only be fully considered during the Comprehensive Hydraulic Assessment Phase of the overall BRCFS study due to the complex interaction of hydraulic effects at this location.

An assessment of the uncertainty of the estimates for the locations considered suggest that quite wide confidence limits are expected for most locations for the more frequently occurring events (AEP 1 in 2 to 1 in 20). At best, the 90% confidence interval is around 55%, whilst some sites have confidence limits of nearly 85%. The uncertainty tends to increase with increasing flood magnitude and with the lack of direct evidence for assessing the accuracy of the estimates, it needs to be recognised that a significant degree of uncertainty remains with the derived flood frequency curves, particularly for the range of rare to extreme events. However, some greater reliance can be placed on estimates for locations where there is consistency between the three independent methods.

Locations that are considered the most unreliable include the Bremer River catchments of Walloon, Amberley and Loamside. This is because of the inconsistency between FFA and the rainfall based approaches due to a possible underestimation of the underlying design rainfall Intensity-Frequency-Duration (IFD) data. Less confidence can also be placed in the Lockyer Creek catchment estimates due to higher uncertainty in the high stage rating of these sites which occurs when flow exceeds the main channel capacity when it spills into the extensive floodplain.

The estimates derived from the MCS exhibit wide variability, which is a reflection of the variation in contributing factors such as rainfall depth, rainfall spatial distribution, rainfall temporal distribution, and antecedent catchment conditions which includes catchment wetness (rainfall loss rates) and initial reservoir levels. For a given combination of rainfall burst duration and AEP, the variability of peak discharges are caused by the combined influence of the variability of initial losses, spatio-temporal rainfall patterns and, for the ‘with-dams conditions’, initial reservoir volumes.

The range of rainfall depth estimates of different durations is illustrated in the figure below, which shows the relationship between AEP of the input rainfall depth and the resultant peak flow for the Brisbane River at Moggill. This Figure shows that for the 1 in 100 AEP rainfall event, the range of estimates varies from about 7,000 m^3/s to 20,000 m^3/s.

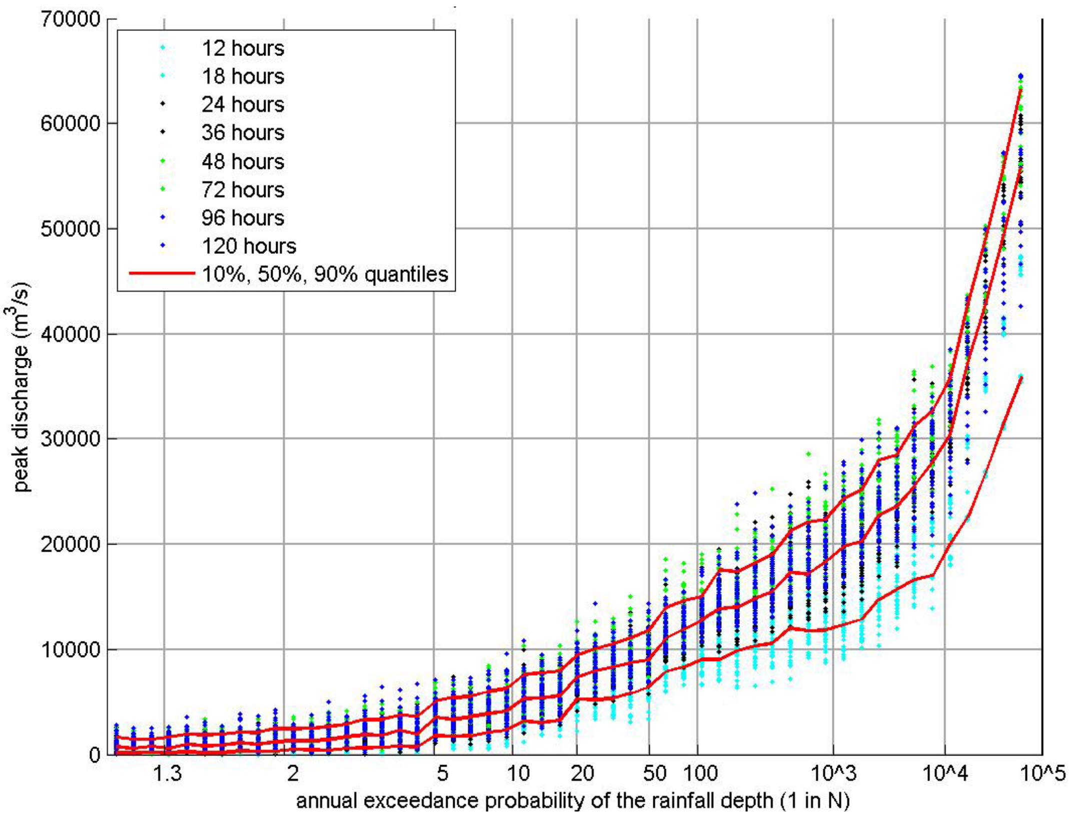


Figure 0-1-4 'No-dams conditions' peak flow estimates; Brisbane River at Moggill

The following observations are made from the results:

- For frequent to large events (rainfall AEP values > 1 in 100) the distance between the 10% quantile and the 50% quantile is generally equal to the distance between the 50% quantile and the 90% quantile, indicating a fairly uniform distribution of peak discharges for a given rainfall AEP. For some locations, for example Wivenhoe, this is not the case. This is mainly due to the fact that fairly short duration rainfall bursts of 3 hours and 6 hours were considered as well for location Wivenhoe Dam. These events result in low peak discharges at Wivenhoe compared to the longer duration bursts and cause a negative skew in the distribution of peak discharges for a given rainfall AEP. If these short duration bursts were omitted, the distribution functions would be fairly symmetric
- For extreme events (rainfall AEP < 1 in 2,000), peak discharges are often 'clustered'. For these events, variations in losses are small which means the variation in peak discharges for a given rainfall AEP is mainly related to the spatio-temporal pattern. For events with rainfall AEP<1 in 2,000, GTSMR based patterns are used to model the spatio-temporal rainfall distribution (Deltares, 2014a). This means a single spatial distribution is applied, which corresponds to the IFD rainfall depth of a 72-hour, 1 in 50 AEP event. For the temporal distribution, 10 temporal patterns are adopted from the GTSMR approach for a range of durations (24, 36, 48, 72, 96 and 120 hours). The number of spatio-temporal rainfall patterns used in the MCS simulations for events with rainfall AEP<1 in 2,000 is therefore relatively low, which reduces the variability in peak discharges for a given rainfall AEP. For some locations this has a clear effect on especially the 10%-quantile lines. For example for location Amberley, the 10% quantile is equal to the lower bound, which means not much value should be placed on the actual meaning of the 10% quantiles for these extreme events

- For a given rainfall AEP, there is a larger variation in the 'with-dams conditions' compared to the 'no-dams conditions'. The dams clearly add more variability to the flood scenarios and, hence, more variability in peak flows. The increased variability has several causes:
  - Starting levels of the dams are modelled as a stochastic variable. This means the available storage volumes of the reservoirs at the start of an event differ per simulated event, which increases the variability of resulting peak discharges for a single AEP
  - The percentage of rain which falls upstream of the main dams is different for different simulated events due to the application of stochastic spatio-temporal rainfall patterns. This means the percentage of rain that can be "controlled" by the dams vary, which also leads to an increase in the variability of resulting peak discharges for a single AEP
  - Different burst durations are considered, which means a single AEP corresponds to a variety of different catchment average rainfall depths. The total rainfall depth for long burst durations is higher than the total rainfall depth for short burst durations. The dampening effect of the dams will therefore be different for different burst durations, which also leads to an increase in the variability of resulting peak discharges for a single AEP

Reconciled No-dams Conditions peak discharges

Table 0-1-1 Peak discharges (m^3/s) versus AEP. Note: the 1 in 100,000 AEP peak discharge is only provided for locations for which the AEP of the PMP is below 1 in 100,000

Location	AEP (1 in N)													
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	PMP DF	AEP of PMP
Linville	150	710	1,300	1,900	2,900	3,700	4,200	4,800	5,400	6,000	7,900	13,700	21,400	500,000
Gregors Creek	330	1,400	2,500	3,700	5,300	6,500	7,400	8,800	9,700	10,800	14,500	26,300	36,300	260,000
Fulham Vale	370	1,700	2,700	4,000	5,400	6,400	7,400	8,800	9,700	11,100	14,700	26,600	34,500	250,000
Peachester	120	300	420	540	680	780	870	980	1,100	1,200	1,400	1,900	2,600	9,710,000
Woodford	210	510	750	1,000	1,300	1,500	1,700	2,000	2,200	2,500	3,200	4,300	6,000	4,070,000
Somerset Dam	540	1,400	2,200	3,000	3,900	4,600	5,200	6,100	6,700	7,400	9,500	13,400	18,300	750,000
Tinton	37	210	390	590	840	1,100	1,200	1,400	1,600	1,800	2,400	3,400	6,000	2,360,000
Middle Ck	670	2,400	4,500	6,600	9,000	10,900	12,400	14,600	16,500	18,800	25,200	57,800	65,700	150,000
Wivenhoe	670	2,400	4,600	6,800	9,300	11,200	12,800	15,100	16,800	19,000	25,000	49,200	54,800	140,000
Helidon	73	230	400	590	800	960	1,100	1,300	1,500	1,700	2,200	3,400	6,700	2,840,000
Gatton	89	410	830	1,300	2,300	3,100	3,700	4,400	5,000	5,600	7,900	13,600	24,000	650,000
Glenore Grove	99	570	1,200	2,000	3,200	4,000	4,900	5,800	6,500	7,400	10,400	18,300	27,700	460,000
Savages Crossing	670	3,100	5,200	8,100	11,600	14,300	16,600	19,100	21,500	23,900	32,600	63,800	63,800	100,000
Mount Crosby	830	3,100	5,400	8,100	11,400	13,800	16,100	18,800	21,300	23,400	32,400	N/A	62,600	90,000
Walloon	260	680	1,100	1,300	1,600	1,900	2,200	2,500	2,800	3,100	4,000	5,500	8,700	1,570,000
Kalbar Weir	200	590	950	1,200	1,600	1,700	1,800	2,100	2,300	2,600	3,400	4,600	7,600	2,180,000
Amberley	230	630	1,000	1,400	2,000	2,200	2,400	2,800	3,000	3,400	4,500	6,400	9,980	1,110,000
Loamside	65	210	310	390	490	580	670	780	870	980	1,200	1,700	2,800	4,770,000
Ipswich	440	1,400	2,100	2,700	3,500	3,900	4,400	5,200	5,800	6,500	8,800	13,200	18,400	540,000
Moggill	1,100	3,800	6,400	9,300	12,300	14,600	17,000	19,900	23,000	25,900	35,800	N/A	64,400	80,000

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	PMP DF	AEP of PMP
Centenary Bridge	1,100	3,700	6,200	9,000	11,800	14,000	16,400	19,300	22,300	25,300	35,500	N/A	64,900	80,000
Brisbane	1,100	3,700	6,200	8,900	11,800	13,900	16,300	19,100	22,000	25,000	34,600	N/A	62,800	80,000

Reconciled With-dams Conditions peak discharges

Table 0-1-2 Peak discharges (m^3/s) versus AEP; Reconciled results. Note: the 1 in 100,000 AEP peak discharge is only provided for locations for which the AEP of the PMP is below 1 in 100,000

Location	AEP (1 in N)													
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	PMP DF	AEP of PMP
Somerset Dam	0	800	1,300	1,800	2,200	2,500	3,000	3,300	3,600	4,000	5,400	10,700	20,900	750,000
Wivenhoe Dam	0	470	930	1,700	3,300	6,300	8,800	10,300	12,500	12,900	21,200	35,800	43,700	140,000
Savages Crossing	190	1,300	2,100	3,500	5,800	8,500	11,800	15,000	17,500	19,500	29,000	56,900	56,900	100,000
Mount Crosby	200	1,300	2,200	3,600	6,000	8,600	11,700	14,800	17,100	19,700	27,200	N/A	55,500	90,000
Ipswich	390	1,300	2,000	2,500	3,300	3,600	4,000	5,000	5,600	6,000	8,300	12,000	16,700	540,000
Moggill	630	2,100	3,300	4,800	7,300	10,200	12,400	15,700	18,000	20,400	29,300	N/A	57,600	80,000
Centenary Bridge	640	2,100	3,300	4,800	7,100	9,900	11,900	15,000	17,700	19,900	28,500	N/A	55,900	80,000
Brisbane	700	2,200	3,300	4,800	7,100	9,900	12,000	14,900	17,500	19,700	27,600	N/A	53,800	80,000

Note *: Estimates shown in red are above 28,000m^3/s which exceed the maximum release capacity of Wivenhoe Dam. These estimates should be treated with caution. The estimates shown in bluefor Somerset Dam should also be regarded with caution as they represent the flows associated with the design floods specific to Wivenhoe Dam.

Contents

Appendices

Appendix H

Assessment of the implications of climate change on flood estimation – discussion paper

Appendix I

Reconciled and recommended flood frequency estimates report

Figures

Tables

1 Introduction

1.1 Study background

A large to rare flood event occurred over southeast Queensland during the second week of January 2011 that included episodes of flash flooding in Toowoomba and the Upper Lockyer Valley and extensive riverine flooding in the Brisbane and Bremer Rivers.

Tragically, 35 people lost their lives as a result of flooding throughout Queensland during December 2010 and January 2011. An estimated 200,000 people were affected throughout Queensland during this period. In south-east Queensland the flooding resulted in approximately $3.5 to $4.5 billion worth of damage alone (DEWS, 2014). A major recovery program was established throughout the region and an inquiry into the flood event was announced by the State Government on Monday 17 January 2011.

The Queensland Floods Commission of Inquiry Final Report (QFCOI, 2012), which was issued in March 2012, contains a recommendation, (Recommendation 2.2) that required a flood study be conducted of the Brisbane River catchment. In accordance with this recommendation, the State of Queensland is managing the conduct of this study, known as the Brisbane River Catchment Flood Study (BRCFS), in a number of separate phases, namely:

Phase 1: Data Collection, Collation, Review and Storage of Existing Data (complete)

Phase 2: Comprehensive Hydrologic Assessment (current)

Phase 3: Comprehensive Hydraulic Assessment (current)

Phase 4: Brisbane River Floodplain Management Study (BRCFMS) and Brisbane River Floodplain Management Plan (BRCFMP)

The Aurecon Team was commissioned to undertake Phase 1 and Phase 2 of the study: the Data Collection, Collation, Review and Storage of existing Data and the Comprehensive Hydrologic Assessment (CHA). This assessment has a requirement for various methodologies to be utilised and for the various methods to corroborate each other so that the most realistic estimates of flood flow and flood volume can be made for nominated locations throughout the Brisbane River catchment.

The Aurecon Team consists of Aurecon as lead consultant, supported by Deltares, Royal HaskoningDHV, and Don Carroll Project Management and Hydrobiology.

1.2 Study purpose

The purpose of the CHA is to develop and apply state of the art methods that produce consistent and robust hydrologic models and analytical techniques that will enable the CHA to provide best estimates of a range of flood flows and flood volumes for annual exceedance probabilities (AEP) across the entire Brisbane River system.

The assessment accounts for two scenarios: the conditions referred to as *'no-dams condition'* and the *'with-dams condition'*.

The dams referred to in the scenarios are the major water storages that exist within the catchment; these are Somerset Dam and Wivenhoe Dam, both of which have flood mitigation capability. Other reservoirs considered are Cressbrook Creek, Lake Manchester, Moogerah and Perseverance dams. These are dams that are primarily water supply storages with fixed crest spillways which do not actively provide flood mitigation.

In the CHA, three approaches are used to estimate peak discharges and flow volumes for a range of Annual Exceedance Probabilities (AEP):

1. Flood Frequency Analysis (FFA)
2. Design Event Approach (DEA)
3. Monte Carlo Simulation (MCS)

The DEA and MCS methods are both referred to as referred to as '*rainfall based methods*', as they both rely on rainfall statistics in combination with a rainfall-runoff model to compute peaks flows and flow volumes at locations of interest. With the FFA method, peak flows and flow volumes for given AEPs are derived directly from observed flows by statistical analysis.

It is anticipated that the BRCFS would provide much of the technical data/information/knowledge about flood flow behaviour and characteristics across various sections of the entire Brisbane River catchment. The outcomes from the CHA will form part of the inputs to the Comprehensive Hydraulics Assessment which will allow flood levels and associated characteristics in lower Brisbane River and tributaries to be estimated.

Together these studies will provide the information upon which the associated Brisbane River Floodplain Management Study (BRFMS) and the Brisbane River Floodplain Management Plan (BRFMP) will be subsequently prepared.

1.2.1 QFCOI recommendation 2.2

An extract from the QFCOI Final Report (QFCOI, 2012) is provided below that describes Recommendation 2.2. Recommendation 2.3 refers to a similar exercise to be conducted for the Bremer River.

2.2 Brisbane City Council, Ipswich City Council and Somerset Regional Council and the Queensland Government should ensure that, as soon as practicable, a flood study of the Brisbane River catchment is completed in accordance with the process determined by them under recommendation 2.5 and 2.6.

The study should:

- Be comprehensive in terms of the methodologies applied and use different methodologies to corroborate results
- Involve the collation, and creation where appropriate, of the following data:
  - Rainfall data including historical and design data and radar
  - Stream flow data
  - Tide levels
  - Inundation levels and extents
  - Data on the operation of Wivenhoe and Somerset dams
  - River channel and floodplain characteristics including topography, bathymetry, development and survey data

- Involve determining the correlation between any of the data sets above
- Produce suitable hydrologic models run in a Monte Carlo framework, taking account of variability over the following factors:
  - Spatial and temporal rainfall patterns
  - Saturation of the catchment
  - Initial water level in dams
  - Effect of operating procedures
  - Physical limitations on the operation of the dams
  - Tidal conditions
  - Closely occurring rainfall events
- Validate hydrologic models to ensure they reproduce:
  - Observed hydrograph attenuation
  - Probability distributions of observed values for total flood volume and peak flow
  - Timing of major tributary flows
  - Observed flood behaviour under no dams conditions and current conditions
- Produce a suitable hydraulic model or models that:
  - Are able to determine flood heights, extents of inundation, velocities, rate of rise and duration of inundation for floods of different probabilities
  - Are able to deal with movement of sediment and changes in river beds during floods
  - Are able to assess historical changes to river bathymetry
  - Are able to be run in a short time to allow detailed calibration and assessment work
  - Characterise the backwater effect at the confluence of the Brisbane and Bremer rivers and other confluences as appropriate
- Involve analysis of the joint probability of floods occurring in the Brisbane and Bremer rivers (and any other pair of rivers if considered appropriate)
- Be iterative, and obtain a short-term estimate of the characteristics of floods of different probabilities in all significant locations in the catchment (at least Brisbane City, Ipswich City and at Wivenhoe Dam) in order to determine the priorities for the rest of the study

1.2.2 Study area overview

The study area includes the entire Brisbane River Basin located in south-east Queensland, which covers a catchment area of approximately 13,570 km^2 to the centre of Brisbane City and includes the main stream and its major tributaries. A number of key locations within various sub-catchments of the Brisbane River basin were identified for consideration in the study. These locations are shown in Figure 2-1.

1.2.3 Study framework, governance and stakeholder consultation

In June 2012, the State Government issued a response (Qld, 2012) to the QFCOI Final Report outlining how it would address the recommendations made by the Commission. Five implementation groups were formed to address the specific issues. The implementation groups relevant to this study included the Planning Implementation Group (PIG) and Dams Implementation Groups (DIG). The PIG had the stewardship of the BRCFS and BRCFMS under the chair of the Department of State Development and Infrastructure Planning (DSDIP). As a result of closure of most of the QFCOI recommendations by mid-2013, the governance structure was changed to establish a Brisbane River

Catchment Flood Studies Implementation Committee of departmental Directors-General and council CEOs to oversee the Flood Study. The governance structure as at September 2014 is shown in Figure 1-1.

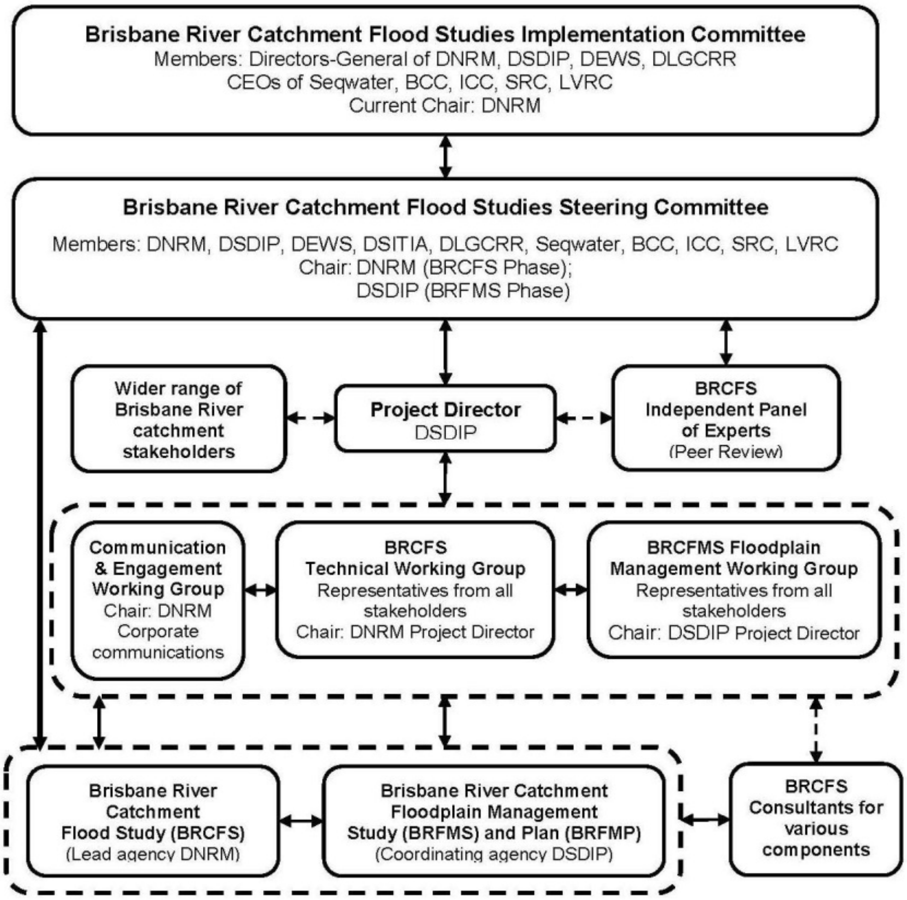


Figure 1-1 BRCFS Governance structure (September 2014)

The DIG was led by the Department of Energy and Water Supply (DEWS) with responsibility for the investigation of operational procedures for Wivenhoe and Somerset Dan Optimisation Study (WSDOS) – QFCOI Recommendation 2.12.1 and North Pine Dam Optimisation Study (NPDOS). The DIG was disbanded in August 2014 on completion of the WSDOS and NPDOS reports.

BRCFS and WSDOS are closely related because both studies rely on estimates of catchment hydrology to achieve their respective purposes. Seqwater conducted the hydrologic assessment of the Brisbane River catchment for WSDOS and this was reported in a Final Report (Seqwater, 2013). This study has been used as a basis for the BRCFS Comprehensive Hydrologic Assessment, although it needs to be recognised that the objectives of both studies are different and hence the associated hydrologic assessments whilst similar, have different requirements.

The BRCFS Project Director (Mr Con de Groot) of DSDIP is responsible for coordinating the BRCFS coordinating working group.

A Project Steering Committee was established in 2012 to provide overall guidance and oversight on the BRCFS and provides advice to the PIG. Project Steering Committee chaired by Mr Lyall Hinrichsen of DNRM and its membership includes all identified stakeholder organisations.

The Comprehensive Hydrologic Assessment is managed by Dr Pushpa Onta of DNRM on behalf of the State Government.

The project stakeholders include representatives from the following organisations:

- Australian Bureau of Meteorology (BoM)
- Department of Energy and Water Supply (DEWS)
- Former Department of Local Government, Community Recovery and Resiliency (DLGCRR)
- Department of Natural Resources and Mines (DNRM)
- Former Department of Science Information Technology, Innovation and the Arts (DSITIA)
- Former Department of State Development and Infrastructure Planning (DSDIP)
- Since February 2015, Department of Infrastructure, Local Government and Planning (DILGP)
- Brisbane City Council (BCC)
- Ipswich City Council (ICC)
- Lockyer Valley Regional Council (LVRC)
- Somerset Regional Council (SRC)
- Seqwater

The Project Steering Committee is supported by a Technical Working Group (TWG) which is coordinated by DNRM with representatives from the relevant agencies. The TWG was formed in 2013 to coordinate technical advice and resolve technical issues in the various studies.

An Independent Panel of Experts (IPE) comprising eminent people with a high level of expertise in related disciplines was also formed in 2013 to provide expert peer review, advice and technical/scientific guidance for various work packages to be investigated as part of the Brisbane River Catchment Floodplain Studies Project.

The Independent Panel of Experts for the CHA comprises of:

- Mr Erwin Weinmann (Chair)
- Mr Mark Babister
- Emeritus Professor Colin Apelt

1.3 Study objective

1.3.1 Data/information

A review of the available flood related data and information is the first stage of works in the CHA process. This review focussed on three primary data sources, as identified in the study brief:

1. Seqwater/SKM ongoing work on hydrology undertaken as part of the Wivenhoe Somerset Dam Optimisation Study (WSDOS), (Seqwater, 2013 and DEWS, 2014) project
2. Aurecon work on existing data compilation and review undertaken as part of the BRCFS Phase 1 (Aurecon, 2013)
3. Brisbane City Council (BCC) work on updating the Digital Terrain Model (DTM) (BCC, 2014) for the Disaster Management Tool (DMT), (BCC, Advanced Draft 2013, Draft 2014 and Final 2014), bed sensitivity analysis and the review of rating curves

The review process has aimed to assess the available data, identify any data gaps and understand the impacts that each available dataset may have on the outcome of the CHA, including a detailed understanding and review of the adopted rating curves throughout the catchment. The review process is iterative and will not be completed until later in the project. The review of the rating curves is considered interim as it presents the outcomes of the first-pass review process. The second pass can only occur once the detailed hydraulic model has been developed and fully calibrated as part of the Comprehensive Hydraulic Assessment.

1.3.2 Methodologies

The study has considered a number of techniques for estimating design floods, including flood frequency analysis (FFA), design event approach (DEA) and Monte Carlo Simulation (MCS) techniques. The main objective of this project is to develop and apply up-to-date, consistent, and robust hydrologic models and analytical techniques for comprehensive hydrologic assessment within the study area.

1.3.3 Flood estimation at various locations

A reconciliation process was devised to assimilate estimates from the various methods to provide the best estimates of the flood flows corresponding to a range of Annual Exceedance Probabilities (AEPs), from 1 in 2 AEP to the Probable Maximum Flood (PMF) at each of the nominated key locations within the catchment. The AEP of the PMP for each location varies in accordance with catchment area as defined in Figure 6 of Book VI of AR&R (EA, 2003).

The outputs of the project including estimated frequency curves for flood flows and volumes, discharge hydrographs for a range of flood event realisations, and associated uncertainties will be used in the subsequent Comprehensive Hydraulic Assessment to determine flood levels, extent, velocity data and associated flood maps for the purpose of floodplain planning and risk management purposes.

Key reporting locations selected by the Study Team in consultation with the TWG and IPE are shown in Table 1-1. These locations are shown in Figure 2-1.

Table 1-1 Key reporting locations within Brisbane River Catchment

Location Number	Reporting Location	Sub-catchment	Catchment Area (km^2)	AEP of PMP (1 in N)
1	Linville	Upper Brisbane River	2,005	500,000
2	Gregors Creek	Upper Brisbane River	3,885	260,000
3	Watts Bridge/**Fulhamvale**	Upper Brisbane River	3,975	250,000
4	Peachester	Stanley River	104	9,710,000
5	Woodford	Stanley River	250	4,070,000
6	Somerset Dam	Stanley River	1,335	750,000
7	Tinton	Cressbrook Creek	420	2,360,000
8	Middle Creek	Upper Brisbane River	6,710	150,000
9	Wivenhoe Dam	Upper Brisbane River	7,020	140,000
10	Russell Siding/**Helidon**	Lockyer Creek	270	2,840,000
11	Gatton	Lockyer Creek	1,550	650,000
12	Glenore Grove	Lockyer Creek	2,230	460,000

Location Number	Reporting Location	Sub-catchment	Catchment Area (km^2)	AEP of PMP (1 in N)
*	Lyons Bridge/**Rifle Range Road**	Lockyer Creek	2,540	400,000
13	Lowood/Vernor/**Savages Crossing**	Lower Brisbane River	10,180	100,000
14	Mt Crosby Weir	Lower Brisbane River	10,600	90,000
15	Walloon	Bremer River	620	1,570,000
16	Kalbar Weir	Warrill Creek	470	2,180,000
17	Amberley	Warrill Creek	920	1,110,000
18	Loamside	Purga Creek	215	4,770,000
19	Ipswich	Bremer River	1,850	540,000
20	Moggill	Lower Brisbane River	12,600	80,000
21	Jindalee/**Centenary Bridge**	Lower Brisbane River	12,915	80,000
22	Port Office Gauge/**Brisbane City**	Lower Brisbane River	13,570	80,000

Note: * The results of the hydrologic modelling at Lyons Bridge/Rifle Range Road were not included in the final assessment due to the fact that the high flow rating at this location cannot be sufficiently defined and the hydrologic model has multiple flow paths representing this location, so no definitive estimate can be obtained.

1.4 Report structure

This report provides an overview of the methodologies and data that have been considered in producing estimates of design floods for various locations within the Brisbane River Catchment. More detailed information on various aspects can be found in the Technical Reports that are referenced as Appendices to this main volume.

The report provides an outline of the data that has been considered for the study. The report also describes the three methods that have been used to derive a range of estimates; Flood Frequency Analysis (FFA); Design Event Approach (DEA) and Monte Carlo Simulation (MCS). The report then provides a summary of the reconciled and recommended estimates for the nominated locations using a combination of the methodologies outlined above.

The Appendices contain copies of the supporting Technical Reports that have been produced as part of this phase of the study.

2 Background/history

2.1 Catchment description

2.1.1 Catchment

The Brisbane River catchment has a total catchment area of 13,570 km^2 to the Port Office Gauge which is located in the heart of Brisbane City. The catchment is bounded by the Great Dividing Range to the west and a number of smaller coastal ranges including the Brisbane, Jimna, D'Aguilar and Conondale Ranges to the north and east. Most of the Brisbane River catchment lies to the west of the coastal ranges. The catchment is complex in nature, combining urban and rural land, flood mitigation dams, tidal influences and numerous tributaries with the potential for individual or joint flooding.

The river system itself consists of the Brisbane River and six major tributaries, two of which are situated downstream of Wivenhoe Dam. Refer to Figure 2-1.

Cooyar Creek, Emu Creek and Cressbrook Creek are all major tributaries of the Upper Brisbane River situated upstream of Wivenhoe Dam. Cooyar Creek and Emu Creek each have a catchment area of just over 1,000 km^2, whilst Cressbrook Creek has a catchment area of only 620 km^2 and is situated about the mid-point of the Brisbane River catchment. These catchments tend to flow in an easterly direction away from the Great Dividing Range and as a consequence experience less rainfall on average than other parts of the catchment. The mean annual rainfall of the Upper Brisbane River catchment is around 880 mm.

The predominant land uses in the Upper Brisbane River catchment are grazing and forestry. The higher, more steeply sloped regions of the catchment are forested with both natural and plantation forest. Soils associated with this area are mainly leached and hard setting loamy soils. In the grazing areas situated on the lower floodplains the vegetation cover consists of pasture, grasslands and scattered open forest.

The Stanley River catchment is the only major tributary that flows from the Conondale and D'Aguilar Ranges located on the eastern edge of the catchment. This catchment is closest to the coast and as such it tends to be the wettest part of the overall Brisbane River catchment with a mean annual rainfall of nearly 1,200 mm. Somerset Dam is located on the lower reach of the Stanley River just upstream from its confluence with the Brisbane River.

The major land use of the Stanley River catchment is grazing, although the lower undulating areas of the catchment are cultivated. The higher, rugged regions of the coastal ranges are forested with both natural and plantation forest. There is a diverse range of soil types in the Stanley River catchment, although most can be classified as friable loamy soils or hard setting loamy soils.

Lockyer Creek, incorporating Laidley Creek, flows from the escarpment of the Great Dividing Range and joins the Brisbane River just downstream of Wivenhoe Dam. Lockyer Creek has a total catchment area of nearly 3,000 km^2, making it the largest of the tributary streams. The Lockyer Valley floodplains are intensively farmed with significant irrigated agricultural production. The higher steeper regions of the catchment situated in the south and west are forested. There are also substantial alluvial deposits in the wide and flat floodplains of the Lower Lockyer Valley that are used for groundwater water supply for agriculture.

The remaining tributary is the Bremer River which rises in the Little Liverpool Range and Macpherson Range and joins with the Brisbane River at Moggill. The Bremer River catchment has a mean annual rainfall of about 900 mm. The catchment is generally hilly and lightly forested. The main land use of the catchment is grazing and agriculture. The Bremer River catchment includes the Warrill Creek and Purga Creek tributaries that meet upstream of the city of Ipswich. The Bremer River catchment covers an area of approximately 1,850 km^2.

Various topographic characteristics of different regions of the Brisbane River are summarised in Table 2-1. This information was sourced from the *Brisbane River and Pine River Flood Study Report Number 18 on Regional Loss Models* (DNRM, 1994). The regions shown in the table were defined as sub-catchments of the various sub-basins that had a stream gauge as an outlet.

Table 2-1 Brisbane River catchment characteristics

Region	Stream	Area (km^2)	Stream Length (km)	Elevation (m AHD)	Slope (S1085) (m/km)	Forest (%)
Upper Brisbane River						
COO	Cooyar	976	87.5	450	4.57	34
LIN	Brisbane	1,044	56.3	300	4.97	53
EMU	Emu	913	90.6	420	2.80	29
GRE	Brisbane	968	38.8	140	0.69	18
CRE	Cressbrook	321	30.2	540	5.30	50
WDI	Brisbane	1,470	95.8	80	0.70	42
Stanley River						
SDI	Stanley	1,331	83.4	140	0.56	37
Lockyer Creek						
HEL	Lockyer	377	25.8	260	8.26	55
TEN	Tenthill	465	46.6	350	9.73	5
GAT	Lockyer	711	44.0	440	10.90	41
LAI	Laidley	286	41.9	160	5.72	54
LYO	Lockyer	597	42.8	90	0.93	13
Bremer River						
WAL	Bremer	625	51.8	80	2.49	27
IPS	Bremer	238	27.8	80	4.31	32
Warrill Creek						
KAL	Warrill	468	34.1	210	0.20	15
AMB	Warrill	448	41.1	50	1.62	10

Region	Stream	Area (km^2)	Stream Length (km)	Elevation (m AHD)	Slope (S1085) (m/km)	Forest (%)
Purga Creek						
PUR	Purga	222	40.1	50	2.66	23
Lower Brisbane River						
SAV	Brisbane	725	74.9	70	2.49	43
MTC	Brisbane	364	40.2	40	1.00	48
JIN	Brisbane	378	40.0	60	0.17	55
POG	Brisbane	332	21.0	35	5.08	41

The Brisbane River is tidal to just below Mt Crosby Weir, which is located some 90 km from the mouth of the river. The Bremer River is also tidal in its lower reaches, extending to The Basin.

Numerous local creeks flow into the Brisbane and Bremer River systems in the urbanised downstream reaches. Some of the more significant creeks include, Bundamba Creek, Moggill Creek, and Oxley Creek. These local creeks tend be substantially modified due to drainage improvement works associated with development that has occurred over time within their watersheds.

The Brisbane River system passes through numerous towns and two major cities which are located in the lower reaches of the basin. It also passes through rural and agricultural land. As such, flooding in the river has the potential to affect large numbers of residents and businesses.

2.1.2 Dams

Two multi-purpose dams are located in its upper reaches of the Brisbane River catchment, both of which were built to supplement Brisbane's water supply and to provide flood mitigation. Wivenhoe Dam was completed in 1986 and has a catchment area of approximately 7,000 km^2, (inclusive of the Stanley River catchment). Somerset Dam completed in 1953 is located upstream of Lake Wivenhoe on the Stanley River near Kilcoy. It has a catchment area of about 1,320 km^2. Therefore only around half the overall Brisbane River catchment to Brisbane City is regulated by these dams.

The current functional divide between flood mitigation and water supply capacity of Wivenhoe Dam and Somerset Dam is summarised in Table 2-2.

Table 2-2 Capacities of Wivenhoe Dam and Somerset Dam

Dam	Function	Capacity (ML)	Proportion (%)
Wivenhoe	Water Supply	1,165,200	37.2
	Flood Mitigation	1,970,000	62.8
	Total	3,135,200	100.0
Somerset	Water Supply	379,800	34.5
	Flood Mitigation	721,000	65.5
	Total	1,100,800	100.0

There are also a number of smaller dams located within the catchment on the tributaries to the Brisbane River that are used predominately for water supply for industrial, irrigation or town water supply.

Four of these smaller water supply dams have been considered in this assessment. These dams include Perseverance Dam built in 1965, and Cressbrook Creek Dam constructed in 1983 which are situated upstream of Wivenhoe Dam in the Cressbrook Creek catchment. These dams are used as water supply dams for Toowoomba and surrounding areas.

Lake Manchester is located on Cabbage Tree Creek just upstream from Mt Crosby Weir on the Lower Brisbane River. It is one of the original water supply dams for Brisbane and was constructed in 1916. Moogerah Dam which is situated on Reynolds Creek, a tributary of Warrill Creek was constructed in 1961 and is used for town water, industrial and irrigation supply.

The presence of the dams in the catchment results in many of the available stream gauging stations having non-stationary data series, affecting the homogeneity of the data required for use in the hydrologic assessments.

Table 2-3 provides a summary of the dams considered in this assessment. The location of the dams is shown in Figure 2-1.

Table 2-3 Key dams in Brisbane River Catchment

Name	Stream	Completion date	Catchment area (km^2)	Full supply capacity (ML)
Cressbrook Creek Dam	Cressbrook Creek	1983	318	81,800
Lake Manchester	Cabbage Tree Creek	1916	73	26,200
Moogerah Dam	Reynolds Creek	1961	222	83,800
Perseverance Dam	Perseverance Creek	1965	114	30,100
Somerset Dam	Stanley River	1953	1,324	379,800
Wivenhoe Dam	Brisbane River	1986	6,981	1,165,200

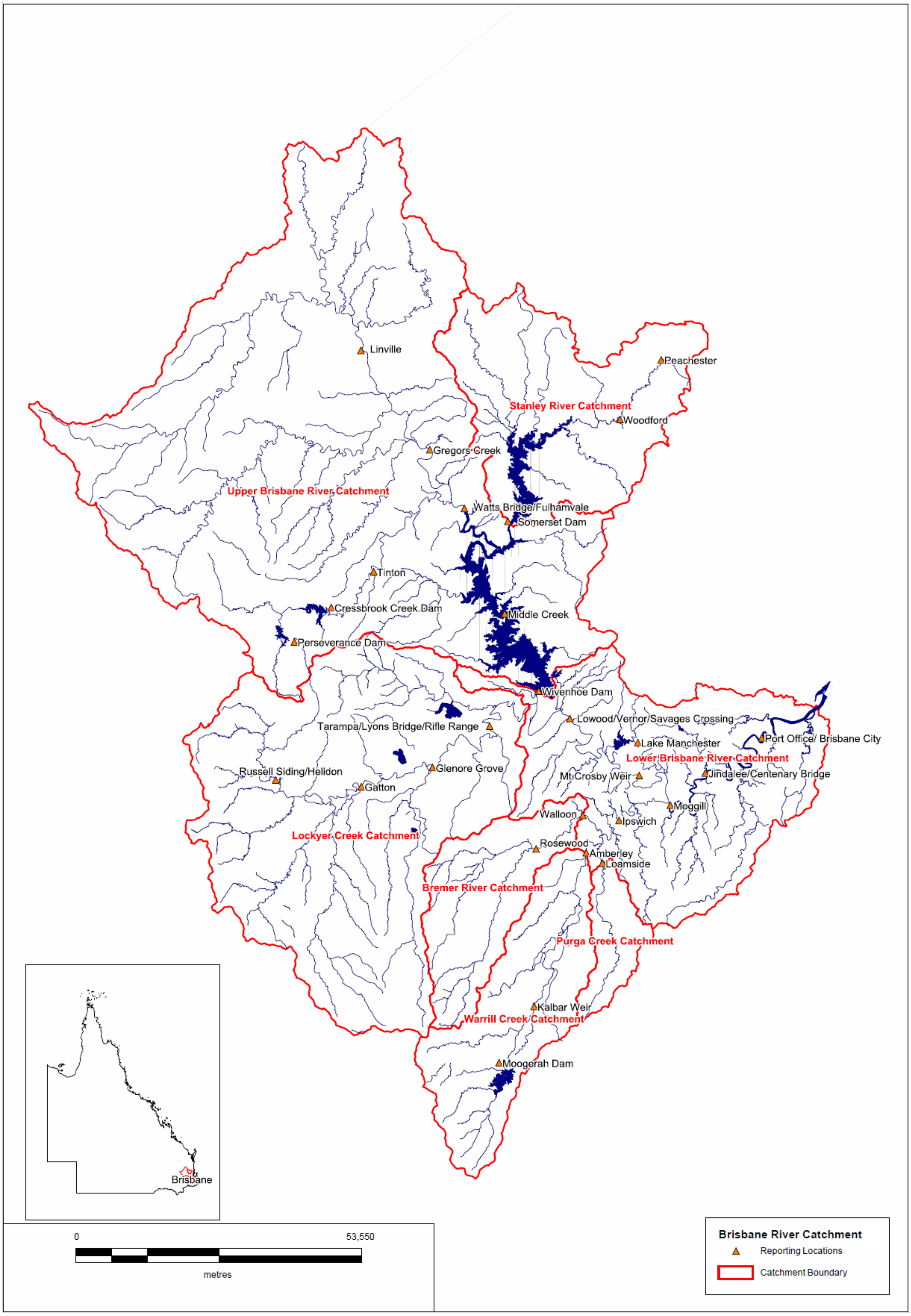


Figure 2-1 Brisbane River catchment

2.1.3 Flood behaviour

Flooding is a natural part of the flow regime of the Brisbane River and many floods have been observed along the river and its tributaries. Its major cause is heavy rainfall over all or part of the catchment. The nature and magnitude of flooding resulting from heavy rainfall depends upon the catchment conditions, with the worst floods occurring when a heavy storm occurs over an already saturated catchment, (examples include February 1893, January 1974 and January 2011).

Major land use changes within the catchment have the potential to modify the flood response as do the operation of mitigation storages of Somerset and Wivenhoe Dams.

The complex interaction of the spatial and temporal characteristics of the catchment allied to the influence of the mitigation dam operations means that routine techniques for assessing design flood estimates do not represent the potential variability that can occur within the system and that joint probability analyses are considered more appropriate.

Figure 2-2 provides a schematic representation of the Brisbane River catchment obtained from the Seqwater Flood Operations Centre (FOC). This schematic provides an indication of the approximate travel time along the main Brisbane River and the associated river distance along the Lower Brisbane River from Wivenhoe Dam to Brisbane City.

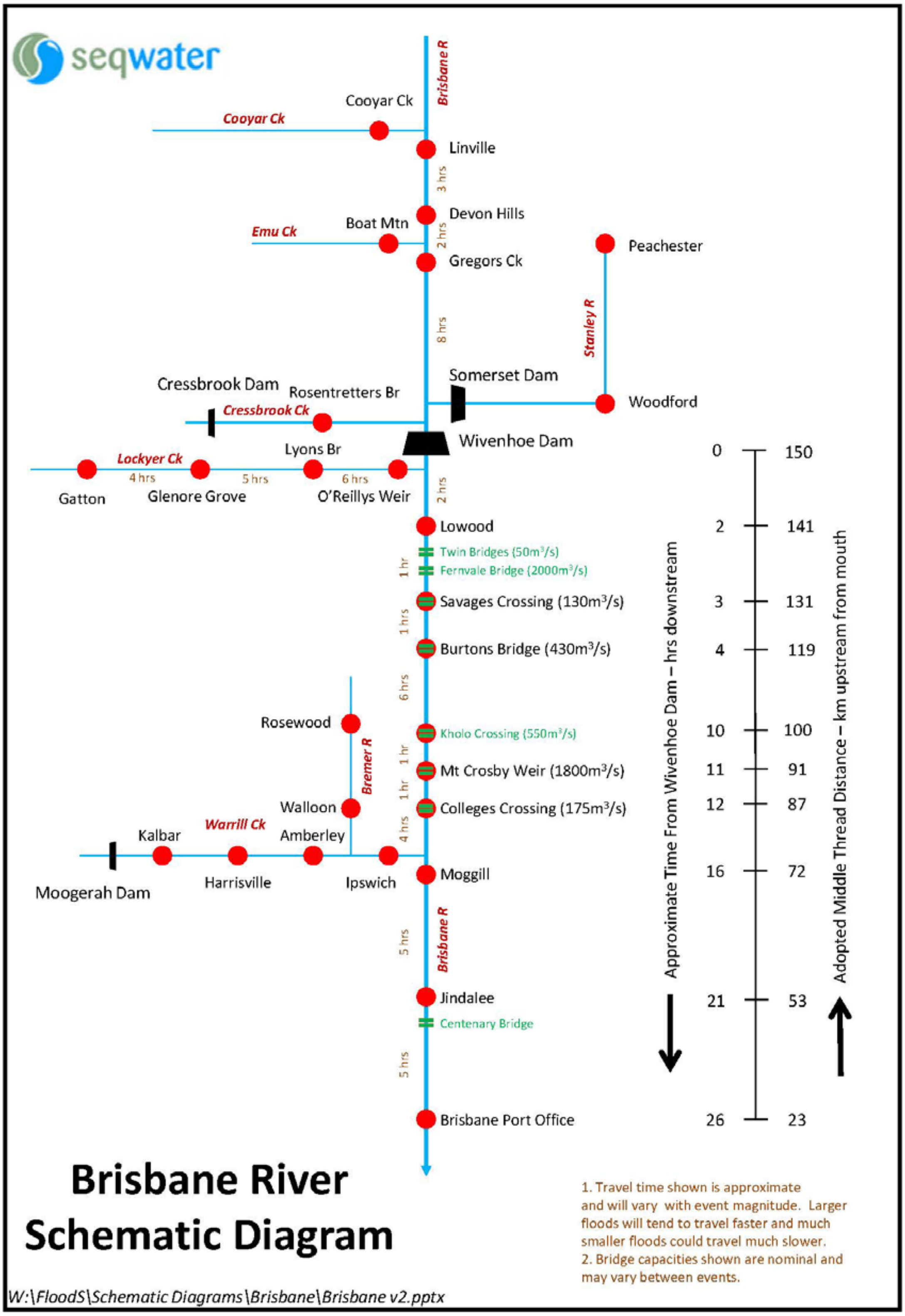


Figure 2-2 Brisbane River schematic (Source: Seqwater Flood Operations Centre)

2.2 Flood history

2.2.1 Significant floods and impacts

The Brisbane River catchment has a history of flooding with evidence dating back to 1824 during the exploration of the Brisbane River by John Oxley. Official records at the Brisbane City Gauge date back to 1841, whilst flood records for Ipswich extend back as far as 1893 and indicate that the both cities have a long history of flooding. An extensive summary of the flood history of the Brisbane River can be found on the Bureau of Meteorology website (http://www.bom.gov.au/index.php).

2.2.1.1 Upper Brisbane and Stanley Rivers

Flood records for most river height recording stations in the upper reaches of the Brisbane and Stanley Rivers are quite extensive with records for Woodford dating back to the 1890s. Two major flood events occurred in Brisbane within a month in February 1893. The first event was a result of extremely heavy rainfall falling in the upper reaches of the Stanley River around Peachester. This event occurred well before the completion of the Somerset Dam in 1953 and Wivenhoe Dam in 1986 and it is still regarded as the flood of record for many parts of the catchment.

A number of large floods have occurred in the Upper Brisbane River in the last twenty years. The February 1999 flood was the largest observed in the Upper Brisbane River catchment up until the January 2011 which produced large scale inundation of residential and agricultural areas throughout the Brisbane Valley.

2.2.1.2 Bremer River

The largest flood in the 20th century occurred in January 1974, rising to a height of 20.7 m on the Ipswich flood gauge located at David Trumpy Bridge. The flood caused widespread damage in the Ipswich area with newspapers at the time reporting that 2,000 properties were inundated.

The flood of record at Ipswich was the first flood in February 1893 when the river reached 24.5 m, however this was largely due to backwater from the Brisbane River as rainfall over the Bremer catchment was relatively minor for this event. The flood event in January 2011 (19.25 m), although smaller than the January 1974 event, caused significant residential and commercial damage with 8,600 properties impacted, with some 1,200 completely inundated.

Ipswich is also vulnerable to flooding that emanates solely from the Bremer River catchment as evidenced by the January 1947 flood which reached 15.19 m on the gauge. This is an example of why the joint probability approach needs to be considered within the Brisbane River Catchment and the benefit of applying Monte-Carlo methods to account for this situation.

A summary of the flood history of the Bremer River at Ipswich (David Trumpy Bridge) is presented in Figure 2-3 (Refer BoM website).

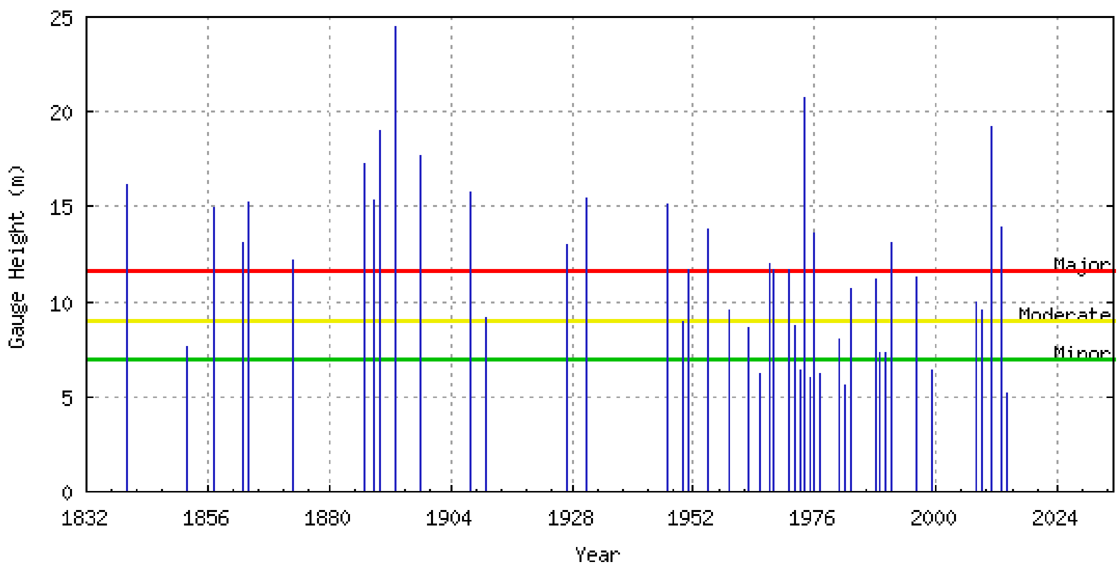


Figure 2-3 Flood history of Bremer River at Ipswich (BoM, 2014)

2.2.1.3 Lower Brisbane River

Flood records for Brisbane extend back as far as the 1840s and indicate that the city has had at least a dozen major floods during this period. The largest flood of the 20th century occurred in January 1974, rising to a height of 5.45 m on the Brisbane City Gauge. This flood caused widespread damage in Brisbane, affecting at least 8,000 properties (BoM, 2014). The most recent major flood occurred in January 2011, when the river peaked at 4.46 m. Although this event was 1 m lower than the January 1974 event, this flood also caused widespread property damage. Approximately 11,900 homes and 2,500 businesses suffered full inundation during the January 2011 flood event with another 14,700 homes and 2,500 businesses partially affected (Aurecon, 2014). It should be noted that no floor level survey exists for some local authority areas, so the numbers of properties impacted by different flood events quoted are approximate.

The largest flood level recorded at the Port Office Gauge is 8.43 m which was associated with the January 1841 flood event. This level was noted by J.B. Henderson, the government hydraulics engineer, in an address to Parliament in 1896 (BCC, 1976). However the flood event of January 1893 has more extensive records and it peaked slightly lower at 8.35 m.

A summary of the flood history of the Brisbane River at the Brisbane City Gauge taken from the BoM website is shown in Figure 2-4.

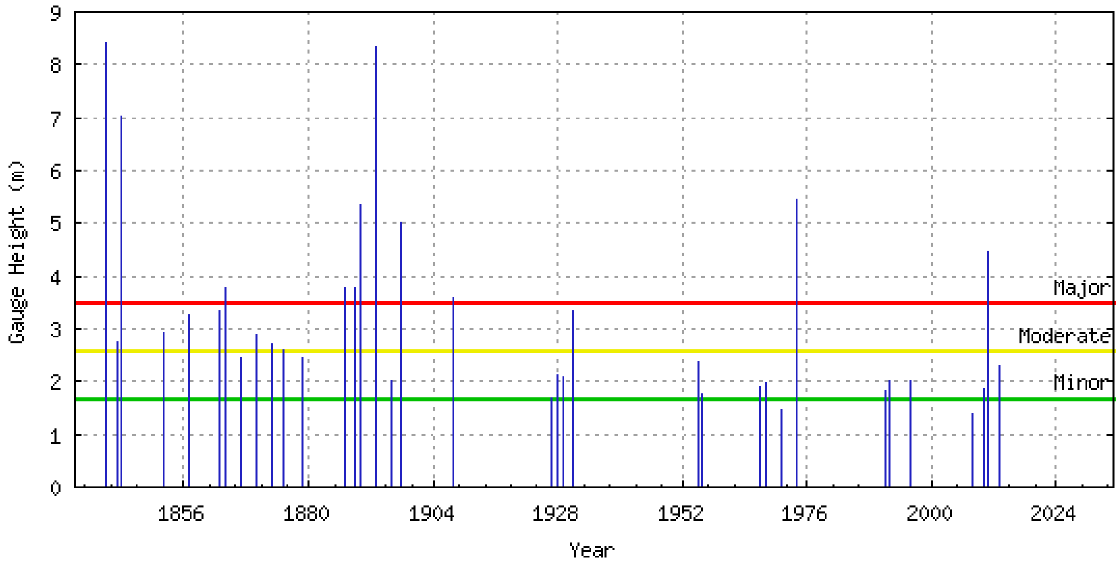


Figure 2-4 Flood history at Brisbane River at Brisbane City (BoM, 2013)

2.3 Previous flood studies

2.3.1 Upper Brisbane and Stanley River

Flood studies of the Upper Brisbane River and Stanley River have largely been associated with Wivenhoe Dam and Somerset Dam. The most recent flood studies of the dams have been associated with the Brisbane River Flood Model study conducted by Seqwater (Seqwater, 2013). This study focused on producing event flood hydrology models of the Brisbane River Basin which included Somerset and Wivenhoe Dam. The models developed were runoff-routing models that have been used as the basis for this current investigation. The resulting hydrographs were used to stress test different dam operation procedures as part of the Wivenhoe and Somerset Dam Optimisation Study (WSDOS) (Seqwater, 2014). This assessment included simulating a range of historical and stochastic flood events including extreme floods up to the Probable Maximum Flood (PMF) for identifying possible improvement in operational procedures for flood mitigation and dam safety objectives. However, annual exceedence probabilities were not assigned to the various flood hydrographs as part of this study.

2.3.2 Bremer River

The most recent investigations involving the Bremer River at Ipswich include assessments undertaken by SKM (2000), KBR (2002), Sargent Consulting (2006) and DHI (2006). These reports include the application of a 'Monte Carlo' analysis of design flows by Sargent Consulting. This application was limited to the exploration of the variation in the 1 in 100 AEP flood magnitude. The study was based on the existing XP-RAFTS model of the Brisbane River catchment and it involved the simulation of 100 trials, which covered storm durations ranging from 24 hours to 72 hours. Variables considered in the assessment included storm duration, storm spatial distribution, storm temporal pattern, initial loss, and the initial levels in each of Somerset and Wivenhoe Dams. Other techniques in the study included flood frequency analyses, runoff-routing modelling and one-dimensional hydraulic modelling.

The Monte Carlo assessment resulted in a best estimate for the 1 in 100 AEP event for the Bremer River at Ipswich of 2,600 m^3/s within a range of 2,000 m^3/s to 3,100 m^3/s. For the Bremer River at Walloon the associated best estimate peak flow for the 1 in 100 AEP event was 1,200 m^3/s within a range of 900 m^3/s to 1,500 m ^{3}/s, whilst for Warrill Creek at Amberley the best estimate for the 1 in 100

AEP event was 1,800 m^3/s within a range of 1,300 m^3/s to 2,200 m^3/s. These estimates represent the current catchment conditions including the presence of Moogerah Dam in the Warrill Creek catchment.

A summary of recent flood study estimates is provided in Table 2-4.

Table 2-4 Comparison of design flood estimates: Bremer River at Ipswich 1 in 100 AEP

Date	Reference	No-dams Peak Flow (m^3/s)	With-dams Peak Flow (m^3/s)
1993	DNRM –DEA/FFA	-	3,010
1998	SKM – DEA/FFA	-	3,200
2006	Sargent Consulting – MCS	-	2,600
2006	Sargent Consulting – DEA	-	2,910
2006	Sargent Consulting – Hydraulics	-	3,200*
2013	Seqwater/Jacobs – FFA	-	3,240
2015	***Aurecon (Current Study)***	***3,900***	***3,600***

Note: * Only estimates derived from hydraulic modelling consider the influence of the Brisbane River.

2.3.3 Lower Brisbane River

The most recent investigations for the Lower Brisbane River and concentrating on Brisbane City were the studies conducted by SKM dating from 1999 to 2003. This range of studies resulted in an Independent Review Panel (IRP) consisting of Prof Russell Mein, Prof Colin Apelt, Dr John Macintosh and Mr Erwin Weinmann recommending estimates for the 1 in 100 AEP at Brisbane City Gauge (Port Office) of 6,000 m^3/s and a corresponding level of 3.3 m AHD. The Review Panel also recommended undertaking further studies including Monte Carlo analyses.

In the aftermath of the January 2011 flood event, Seqwater commissioned SKM to undertake a joint calibration of a hydrologic and hydrodynamic model of the Lower Brisbane River (2011). The Queensland Floods Commission of Inquiry appointed Mr Mark Babister of WMAWater to review this report and also provide a report on the best estimate of the 1 in 100 AEP at certain points along the Brisbane and Bremer Rivers. A number of experts critiqued Mr Babister's estimate and highlighted the limitations of the approach he adopted and the limited time that had been made available to undertake these estimates. This in turn led to the QFCOI recommendation 2.2 to undertake a comprehensive flood study of the Brisbane River catchment.

Table 2-3 provides a summary of the comparison between previous design flood studies for the Brisbane River at Brisbane City Gauge.

Table 2-5 Comparison of design flood estimates: Brisbane River at Brisbane City Gauge 1 in 100 AEP

Date	Reference	No-dams Peak Flow (m^3/s)	With-dams Peak Flow (m^3/s)
1984	QWRC/BCC	11,500	5,510
1993	DNRM – DEA/FFA	14,910	9,120
1998	SKM – DEA	-	9,560
1999	SKM – DEA/FFA	-	8,000
2003	SKM – FFA	10,100	5,040
2003	SKM – DEA	-	6,500

Date	Reference	No-dams Peak Flow (m^3/s)	With-dams Peak Flow (m^3/s)
2003	IRP	12,000	6,000
2004	SKM – Hydraulics	-	5,970
2011	WMAwater	13,000	9,500
2013	Seqwater/Jacobs – FFA	15,190	-
2015	***Aurecon (Current Study)***	***13,900***	***9,900***

The difference in estimates can be attributed to the difference in techniques utilised and data available at the time of the respective analysis.

BCC have also completed two studies (BCC, 2009 and 2014) that has investigated the use a two-dimensional hydraulic model of the Lower Brisbane River to underpin a disaster management tool to examine the extent of flooding in rare to extreme flood events. These studies resulted in the production of a series of inundation profiles for the Lower Brisbane River for a range of possible flood events. No AEPs were assigned to the various flood profiles as the purpose of the mapping is for emergency response, not floodplain planning.

3 Data

3.1 Introduction

The CHA comprises of many key input datasets and tasks. The CHA process includes both tasks that are reliant on completion of previous tasks and tasks that are iterative. Figure 3-1 shows the key datasets and tasks, and the relationships between these and completion of the study. It can be inferred from this figure that there are three critical tasks which are required for conduct of the study:

- Collection of all required input datasets
- Understanding of historic events and datasets
- Adoption of first-pass rating curves.

Figure 3-1 shows that there is an iterative process that involves the adoption of rating curves, the hydrologic modelling and the generation of model outputs. Therefore, first-pass curves are required to commence the study but will need to be reviewed in later stages of the study. Reference is made to the Aurecon report Data, Rating Curve and Historical Flood Review Report (Aurecon, 2014 Refer Appendix A).

Data forms the basis of all of the three methods considered for use in the production of design flood estimates. Stream flow data is used directly in the Flood Frequency Analysis. The recorded water level data is converted to rated flows by the stream gauge rating. Some of this information then needs to be modified in order to produce a stationary data set, such as removing the influence of the dams.

The other two methods are rainfall based methods that use a wide range of data inputs including rainfall depth, rainfall temporal patterns and rainfall loss rates, as well as dam and stream flow data for calibration purposes. All this information is integrated through the hydrologic models to produce estimates of stream flow hydrographs and hence peak flows and flood volumes required for analysis.

The process of producing design flood estimates is necessarily iterative, because consistency of estimates needs to be achieved for all locations and for the full range of flood magnitudes under consideration. Regional comparisons and the reconciliation process are used to incorporate consistency into the estimates. Modifications to fundamental inputs such as rating curves and model parameters are required to achieve a desired level of consistency between estimates at different locations.

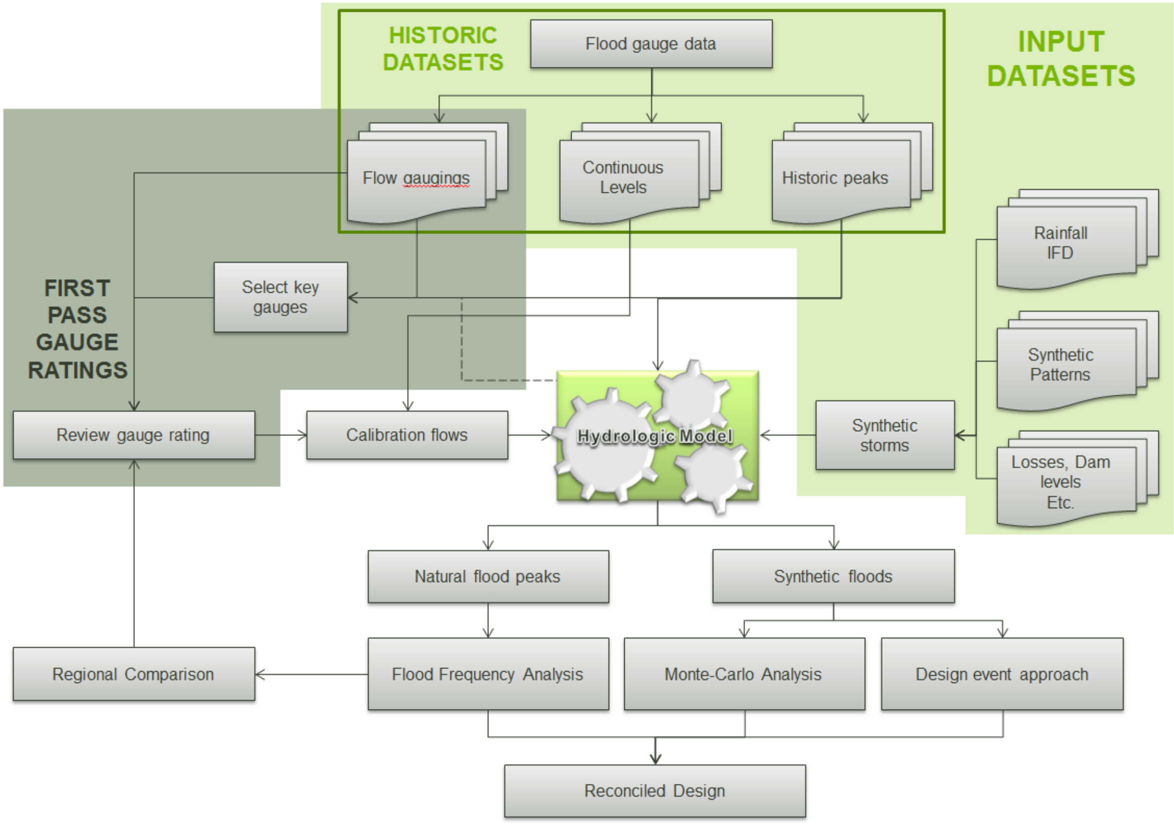


Figure 3-1 Interaction of key study inputs and tasks

3.2 Rainfall

3.2.1 Historical rainfall

Historical rainfall data in the form of daily rainfall and pluviograph records is required for the calibration of the URBS hydrologic model and as input into the Monte Carlo analysis. This information was sourced from the Bureau of Meteorology (BoM) and also from Seqwater (Seqwater 2013). Data provided by BoM is in a raw format which has not necessarily been quality controlled. The Seqwater data was processed as event data suitable for the calibration of the URBS (Carroll, 2012) hydrologic model. This data set was reviewed for obvious errors and suspect data and so proved to be the most useful information to utilise during the assessment. Data was utilised for 48 flood events from the period dating from 1887 to 2013.

Figure 3-2 shows a map of the historical rainfall stations available within the Brisbane River Catchment.

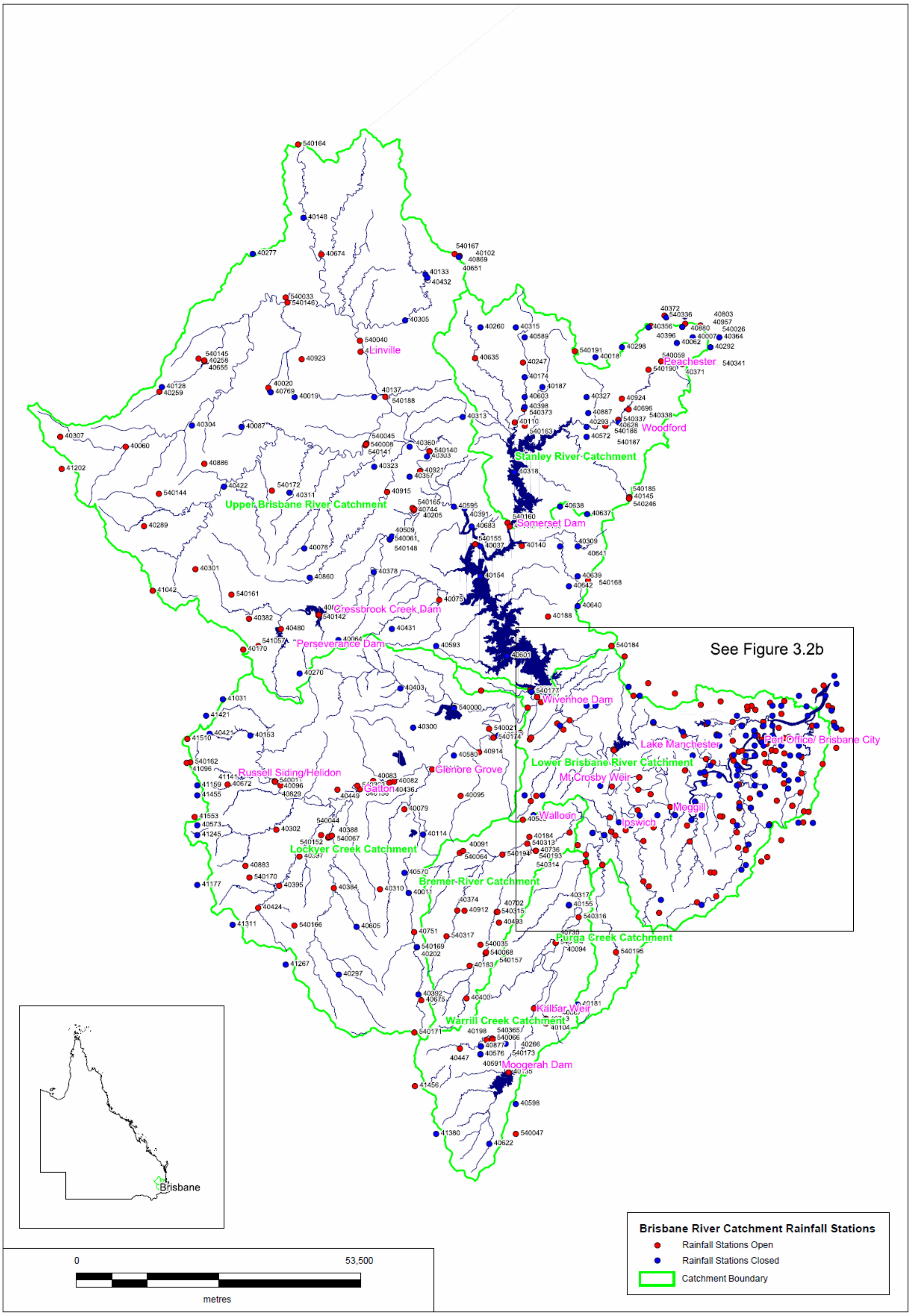


Figure 3-2a Rainfall stations within Brisbane River Catchment

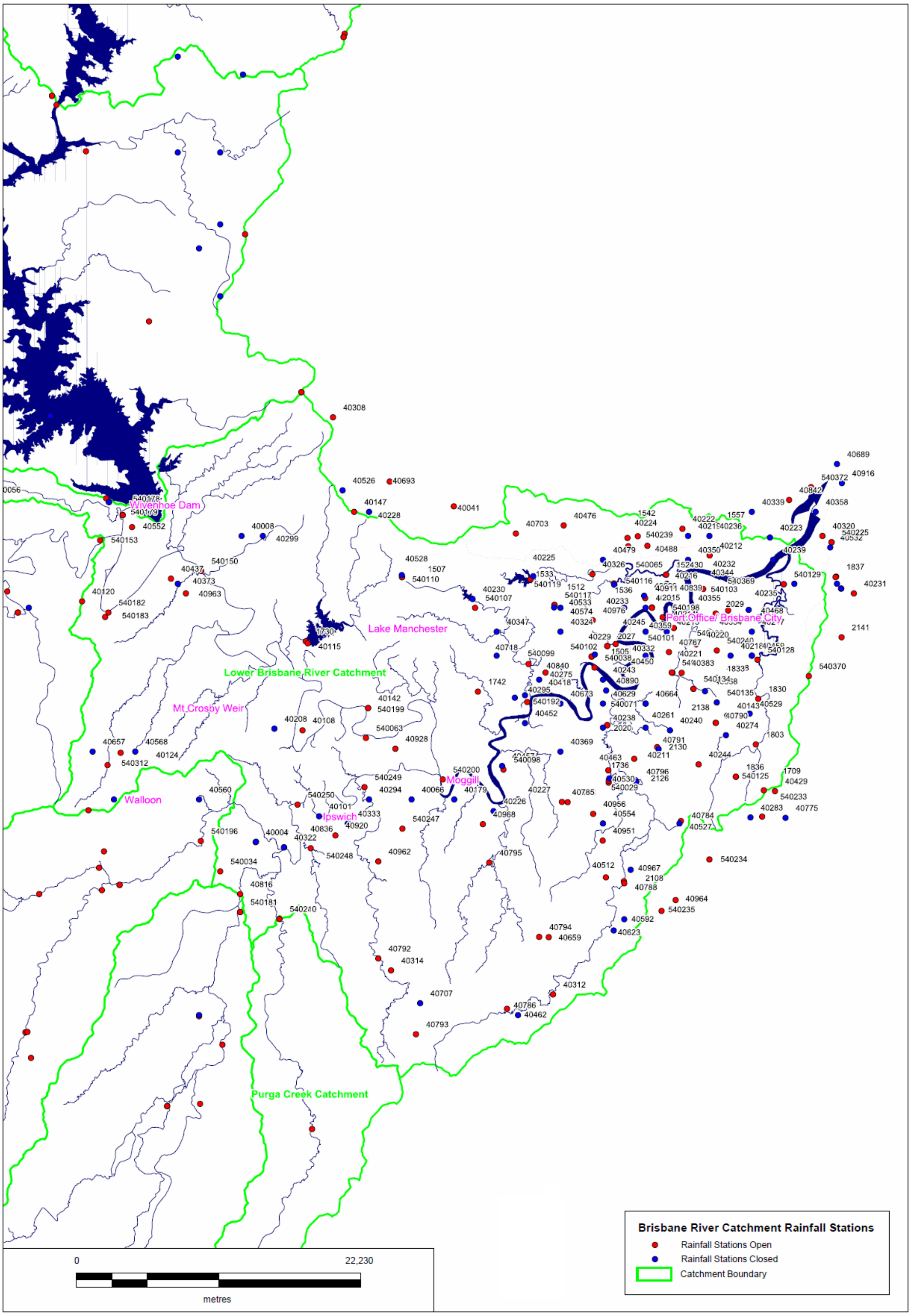


Figure 3-2b Rainfall stations within Brisbane River Catchment

Continuous rainfall records are generally required for hydrologic model calibration. However, as the event based data is already included in the URBS model files (Seqwater, 2013), no additional continuous rainfall record was required. Details of available rainfall stations within the Brisbane River catchment can be found in the *Collection, Collation, Review and Storage of Existing Data – FINAL report* (Aurecon, 2013) and in the Seqwater *Brisbane River Flood Hydrology Models* report (Seqwater, 2013).

A select range of sites that had sufficient record length (> 30 years) of continuous rainfall data was used for the MCS Pilot Study. This information was used to examine burst statistics which are required in one of the sampling techniques associated with the application of the MCS. The sites used are listed below:

- Boat Mountain gauge on Emu Creek with records from 1993 to present
- Ravensbourne gauge in the upper Perseverance Creek catchment with records from 1956 to 1997
- Kirkleagh gauge on Somerset Dam with records from 1959 to 1991

Peak annual catchment rainfall estimates for durations ranging from one day to ten days were also obtained from WMAwater (WMAwater, 2014) for use in the MCS analysis. This data was based on gridded historical records obtained from the BoM database.

3.2.2 Design rainfall

Intensity-frequency-duration (IFD) data is required for input to the design event approach and Monte Carlo analyses. This data is available for two separate IFD datasets: that produced for the 1987 version of Australian Rainfall and Runoff (AR&R), (EA, 1987); and that produced in 2013 for the current update of AR&R (BoM, 2013). The 2013 IFD data has been used for the derivation of the reconciled and recommended design flood estimates. This data covers the range of design rainfalls from 1 in 2 AEP to 1 in 100 AEP.

Engineers Australia conducted comparisons between the IFD estimates derived from the 1987 and 2013 data sets for a range of locations around Australia including Brisbane. The comparison was performed by considering the six standard duration- frequency ranges shown in Volume 2 of Australian Rainfall and Runoff, (EA, 1987), and presenting the percentage change as a map. The outcome of this comparison indicates the following:

- 1 hour, 1 in 2 AEP – A reduction of between 10 to 20%, (except on the coastal fringe)
- 12 hour, 1 in2 AEP – A reduction of between 20 to 30%, (mainly on the coastal ranges)
- 72 hour, 1 in 2 AEP – Mixture of reductions and increases of between 20 to 30%
- 1 hour, 1 in 50 AEP – A reduction of between 10 to 30%
- 12 hour, 1 in 50 AEP – Mixture of reductions and increases of between 10 to 30%
- 72 hour, 1 in 50 AEP – Increases of between 10 to 30%

On the recommendation of the IPE, the 2013 IFD data set has been adopted for the purpose of this study, although a sensitivity analysis using the 1987 IFD has been conducted.

For the large to rare flood magnitude range, CRC-Forge (Hargraves, 2004) design rainfall estimates were used. This covers the range up to the limit of credible extrapolation, 1 in 2,000 AEP. For extreme rainfall estimates (Probable Maximum Precipitation), the generalised techniques described by the GSDM and GTSMR (BoM, 2003) were adopted. The techniques specified in Book VI of AR&R (EA, 2003), have been used to interpolate design rainfall estimates between 1 in 2,000 AEP and the PMP.

WMAWater have these two gridded datasets available from information that they have extracted from the BoM website. These two gridded datasets were provided by WMAWater (WMAWater, 2013) for the Brisbane River catchment extents.

3.3 Streamflow

Figure 3-3 shows the historical stream gauge stations available within the Brisbane River catchment.

3.3.1 Peak height record

Peak height records have been obtained from the Bureau of Meteorology for the available relevant flood warning sites. This information is required for use in developing a series of annual peak flood flows for input into the flood frequency analysis (FFA).

3.3.2 Continuous record

Up-to-date continuous gauge recordings for DNRM gauges have been collected from the DNRM website. Limited continuous gauge recordings have been collected from BoM. This information is required for determining peak flow and volumetric flood frequency analyses and is used in the re-calibration of the hydrologic model. Details of these gauges can be found in the supporting technical report in the *Data, Rating Curve and Historical Flood Review Report* (Aurecon, 2014 Refer Appendix A).

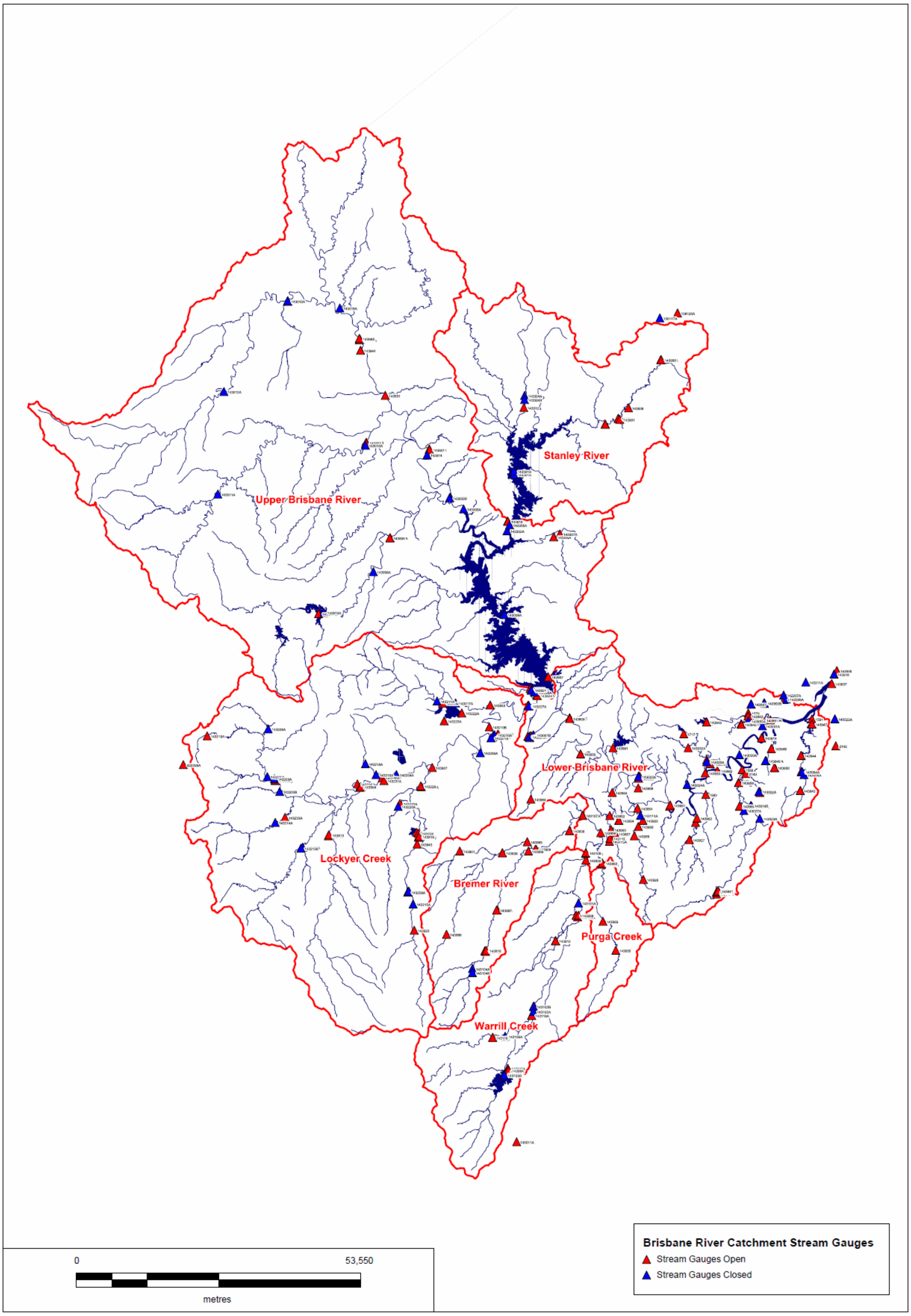


Figure 3-3 Stream gauge stations within Brisbane River Catchment

3.3.3 Rating curves

A detailed review of the existing rating curves generated by Seqwater, DNRM, BoM and other sources was undertaken and recommendations were made as to the rating curves to be adopted for the BCRFS Hydrology. The recommendations, as presented in the *Data, Rating Curve and Historical Flood Review Report* (Aurecon, 2014, refer Appendix A), are summarised in Table 3-2 and Table 3-3.

Available gauge data has been reviewed to classify the flood gauges according to their proposed priority of usage for the Brisbane River Catchment Flood Study – Hydrology Phase. Gauge review priority classifications are described in Table 3-1. Selection of gauge classification has been based on multiple criteria.

Note that this classification of 'primary' and 'secondary' refers only to the importance and level of detail of the rating review, not necessarily to the overall importance of the gauge (although this is one of the selection criteria). A number of gauges that may otherwise be considered to be of high priority for purposes such as dam operations or flood warning, have nevertheless been identified as secondary for the rating review due to inconsistent or inadequate data, or other factors precluding the usefulness of detailed independent hydraulic analysis.

Gauge ratings have been developed using best available information, including stream gauge flow measurements, independent hydraulic modelling and hydrologic model correlation, and have been iteratively reviewed and updated throughout the hydrologic model recalibration and flood frequency analysis processes to ensure consistency throughout the hydrologic assessment. Nevertheless, the ratings should be subject to ongoing review and update as additional reliable data becomes available (flow measurements, revised modelling etc). In particular, the Lower Brisbane River ratings, which are currently based on DMT TUFLOW modelling results, should be reviewed as part of the Comprehensive Hydraulic Assessment phase of the BRCFS.

Table 3-1 Gauge review priority classifications

Classification	Description
Primary	Gauge is considered to be of high importance to the hydrologic modelling and has data of sufficient quality and quantity to allow independent assessment of the rating to be undertaken
Secondary	Gauge is generally considered to be of either moderate importance to the hydrologic modelling, or of high importance but with some factor limiting the benefit of a detailed assessment (eg the gauge may already have a high level of confidence, or may lack data). Review of existing data and ratings has been undertaken to identify level and range of confidence in the existing rating
Other	Gauge is in a location of low priority or may have limited or conflicting data preventing reliable determination of a rating curve

Table 3-2 Summary of primary gauge rating recommendations

Gauge description	Recommended rating description
Catchment: Stanley River to Somerset **Stream:** Stanley River **Site:** Woodford **Gauge No:** 143901A **Owner:** DNRM	Power-law fit of data up to 2.5m then hydraulic model results Flows below 20m³/s weir controlled and dependent on local Stanley River flows only. Flows above 50m³/s dependent on combined flows from Stanley River (at gauge) and downstream tributary. Rating provides a good fit of revised hydrologic model results, however could be unreliable if flow distribution varies significantly from the ratio assumed by the hydraulic modelling

Gauge description	Recommended rating description
Catchment: Brisbane River to Wivenhoe **Stream:** Brisbane River **Site:** Linville **Gauge No:** 143007A **Owner:** DNRM	Power-law fit of data up to 2.7m then hydraulic model results Site is considered a good gauge location. Flow gauging and hydrologic model data are consistent and high flows are well contained. Rating provides a good fit of flow gaugings and hydrologic model data
Catchment: Lockyer Creek to O'Reilly's Weir **Stream:** Lockyer Creek **Site:** Glenore Grove **Gauge No:** 143807 **Owner:** BoM	Power-law fit of data up to 2.5m then hydraulic model results Rating is considered to be good up to around 13m (900m³/s) with generally good fit of flows (translated from Lyons Bridge) and hydrologic model data. Generally good agreement above this level and rating is considered reasonable, but becomes very sensitive to changes in level
Catchment: Bremer River to Walloon **Stream:** Bremer River **Site:** Walloon **Gauge No:** 143107A **Owner:** DNRM	DNRM rating up to 5m then hydraulic model results Generally good fit of flow gaugings and hydrologic model data up to about 9m. Rating becomes fairly sensitive at high flows and potentially affected by backwater from major Brisbane River/Warrill Creek floods due to 'choke point' that forms in the reach downstream of the Warrill Creek confluence.
Catchment: Warrill Creek to Amberley **Stream:** Warrill Creek **Site:** Amberley **Gauge No:** 143108A **Owner:** DNRM	Power-law fit of data up to 5m then hydraulic model results Good fit of flow gaugings. Deviates significantly from Seqwater rating above 8m due to breakout of flows upstream of gauge location. Rating is considered to be good, but becomes very sensitive to changes in level above 10m (1200m³/s)
Catchment: Purga Creek to Loamside **Stream:** Purga Creek **Site:** Loamside **Gauge No:** 143113A **Owner:** DNRM	DNRM rating up to 6m then hydraulic model results Generally good fit of flow gaugings and hydrologic model data. Rating is considered to be reasonable, but becomes very sensitive to changes in level above 7.5m (170m³/s)
Catchment: Lower Brisbane River **Stream:** Brisbane River **Site:** Mt Crosby Weir **Gauge No:** 430003A **Owner:** Seqwater	Rating updated based on review of gaugings, steady-state release flows and DMT TUFLOW model results Gauge location is considered to be reasonable with well-defined weir crest and relatively confined channel. Rating provides generally good fit of flow gauging, steady flow release and most hydrologic data, although it is noted that a number of the hydrologic model results deviate significantly from the rating Importantly, the rating is considered relatively unreliable between around 1,200 and 2,000m³/s. Interference of the bridge is considered a likely cause
Catchment: Lower Brisbane River **Stream:** Brisbane River **Site:** Centenary Bridge **Gauge No:** 43982 **Owner:** BoM	Rating updated based on review of gaugings, steady-state release flows and DMT TUFLOW model results Rating provides generally good fit of flow gauging, steady flow release and hydrologic data. Rating is considered to be reasonable, with a fairly well contained site and flow gauging up to high flows (10,000m³/s). However, site is subject to significant dynamic effects, meaning that there is not a direct relationship between flow and level

Table 3-3 Summary of secondary gauge rating recommendations

Gauge description	Recommended rating description
Catchment: Stanley River to Somerset **Stream:** Stanley River **Site:** Peachester **Gauge No:** 143303A **Owner:** DNRM	3-stage best-fit of flow gauging and hydrologic model data Reasonable match of flow gauging and hydrologic model data below 6m. Gauge becomes sensitive to changes in level above 7m with limited reliable data available for calibration. Hydrologic model data shows noticeable scatter but consistent general trend.
Catchment: Stanley River to Somerset **Stream:** Kilcoy Creek **Site:** Mt Kilcoy **Gauge No:** 143312A **Owner:** DNRM	Seqwater Rating based on two-stage best-fit of flow gauging and hydrologic model data Reasonable match of flow gauging data up to 5.5m. Upper rating is based solely on hydrologic model data and significant scatter is observed in the results above 5m. Upper rating is also very sensitive to changes in level. Upper rating is therefore considered to be unreliable but of fairly low importance overall
Catchment: Brisbane River to Wivenhoe **Stream:** Brisbane River **Site:** Gregors Creek **Gauge No:** 143009A **Owner:** DNRM	3-stage best-fit of flow gauging and hydrologic model data Good agreement with DNRM HEC-RAS model although details of model are not confirmed. Reasonable match of flow gauging data up to 9m and hydrologic model data above that level, however noticeable scatter is evident in the low level flow gauging data. Site is well confined but known to have issues with changes to section and sand extraction downstream. The rating is considered to be reasonable, but not necessarily consistent
Catchment: Brisbane River to Wivenhoe **Stream:** Brisbane River **Site:** Caboonbah **Gauge No:** 143900 **Owner:** BoM	3-stage best-fit of flow gauging and hydrologic model data Reasonable agreement with Seqwater rating. Flow measurements translated from Middle Creek gauge site to improve shape of low-flow rating. Gauge site has been closed since construction of Wivenhoe Dam.
Catchment: Brisbane River to Wivenhoe **Stream:** Brisbane River **Site:** Middle Creek **Gauge No:** 143008A **Owner:** DNRM	3-stage best-fit of flow gauging and hydrologic model data Rating based on flow gauging up to 2,600 m^3/s and is considered to be good within this range. Little data available for validation of rating above this range but few recorded higher levels and gauge site has been closed since construction of Wivenhoe Dam.
Catchment: Lockyer Creek to O'Reilly's Weir **Stream:** Lockyer Creek **Site:** Gatton **Gauge No:** 143904 **Owner:** BoM	3-stage best-fit of flow gauging and hydrologic model data Rating shows similar trend to Seqwater rating based on hydrologic model data and independent SKM hydraulic model, however consistency of this model to BRCFS hydrology not confirmed. Rating was adjusted to improve better match of hydrologic model results and improve consistency with downstream flows at Glenore Grove and Rifle Range Road. No flow gauging data is available for comparison. Gauge location is well confined and should provide reasonable rating conditions up to bank-full condition, but is not rated above 16m (2700m^3/s)
Catchment: Lockyer Creek to O'Reilly's Weir **Stream:** Lockyer Creek **Site:** Gatton Weir **Gauge No:** 143236A **Owner:** Seqwater	Seqwater rating Relatively close proximity to Gatton gauge. Very limited low-level flow gauging and limited hydrologic model data due to short gauge record. Gauge location is well confined and should provide reasonable rating conditions up to bank-full condition, but is not rated above 17m (2700m^3/s)

Gauge description		Recommended rating description
Catchment: **Stream:** **Site:** **Gauge No:** **Owner:**	Lockyer Creek to O'Reilly's Weir Lockyer Creek Helidon 143203C BoM	3-stage best-fit of flow gauging and hydrologic model data Reasonable agreement with DNRM and Seqwater ratings. Stream flow gauging only available for low flows. Significant scatter in hydrologic model data as model calibration weighted heavily towards the Glenore Grove site which has six times the catchment area. Rating is considered to have limited reliability.
Catchment: **Stream:** **Site:** **Gauge No:** **Owner:**	Lockyer Creek to O'Reilly's Weir Laidley Creek Warrego Hwy 143904 BoM	DNRM Rating Good agreement with flow gauging up to 7.6m so considered to be a relatively reliable rating, however rating becomes sensitive to changes in level above 5m
Catchment: **Stream:** **Site:** **Gauge No:** **Owner:**	Lockyer Creek to O'Reilly's Weir Lockyer Creek Rifle Range Rd 143229A DNRM	Power law best-fit of flow gauging and hydrologic model data Reasonable fit of flow gauging data up to 15.85m (830m^3/s). Perched channel in wide floodplain with unreliable and potentially inconsistent response above bank-full capacity. Rating should not be used above bank-full (15.5m approx)
Catchment: **Stream:** **Site:** **Gauge No:** **Owner:**	Bremer River to Walloon Bremer River Adams Bridge 143110A DNRM	DNRM Rating (Seqwater rating very similar) up to 4.4m gauge height then power law best-fit of hydrologic model data Good fit of flow gauging up to 4.3m. Basis of projection above this level unknown and appears to predict higher levels/lower flows than Seqwater hydrologic model results. Rating becomes sensitive to changes in level above 4m
Catchment: **Stream:** **Site:** **Gauge No:** **Owner:**	Bremer River to Walloon Bremer River Rosewood 143909 BoM	Best-fit of hydrologic model data Reasonable agreement with hydrologic model data but no independent confirmation data available. Poor detail below 4.5m. Higher emphasis should be placed on Walloon gauge rating
Catchment: **Stream:** **Site:** **Gauge No:** **Owner:**	Warrill Creek to Amberley Warrill Creek Junction Weir 143118 Seqwater	Seqwater rating Based on hydraulic model up to around 200m^3/s. Reportedly very low reliability above this but shows reasonable match of the limited hydrologic model data available
Catchment: **Stream:** **Site:** **Gauge No:** **Owner:**	Purga Creek to Loamside Purga Creek Peak Crossing 143869 Seqwater	Best fit of hydrologic model data only Limited record length and no independent data. Generally low confidence in gauge rating magnitude

Gauge description	Recommended rating description
Catchment: Lower Brisbane River **Stream:** Brisbane River **Site:** Savages Crossing **Gauge No:** 143001C **Owner:** DNRM	Rating updated based on review of gaugings, steady-state release flows and DMT TUFLOW model results Rating provides reasonable fit of flow gauging, steady flow release and hydrologic model data. Well contained site but believed to be subject to changes in rating. Available data displays some historical variation, most notably an abrupt change during/after the 2011 flood event. Gauge is considered to be reasonably rated but not particularly consistent
Catchment: Lower Brisbane River **Stream:** Brisbane River **Site:** Moggill **Gauge No:** 143951 **Owner:** BoM/Seqwater	Rating updated based on review of gaugings, steady-state release flows and DMT TUFLOW model results Rating provides generally good fit of steady flow release and hydrologic data, but no flow gauging available for comparison. Rating is considered to be reasonable, with a fairly well contained site. Revised rating tends to predict higher flows than previously estimated due to dynamic effects and attenuation evident in the TUFLOW model but not properly represented in the hydrologic model
Catchment: Lower Brisbane River **Stream:** Brisbane River **Site:** Brisbane City **Gauge No:** 143838 **Owner:** Seqwater	Rating updated based on review of gaugings, steady-state release flows and DMT TUFLOW model results Rating is highly tide dependent even up to high flow rates (>10,000m³/s). Site has also been subjected to dredging and other changes, the effects of which are unquantified Overall, the current rating appears to give a reasonable estimate of the flow order-of-magnitude and match of historical flood events for flows in the range 6,000 to 16,000 m³/s. The site/rating is complex and improving the rating would require significant work (hydraulic modelling) that is outside the scope the current study

The adopted rating curves are provided in tabular format in the report which is contained in *Data, Rating Curve and Historical Flood Review Report* (Aurecon, 2014 Refer Appendix A).

The effect of revising the rating curves at various locations has resulted in changes to the rated flows adopted in the calibration of the sub-catchment models. Table 3-4 provides a summary of the changed characteristics for the calibration events.

Table 3-4 Relative impact on rated flows

Sub-catchment model	Key gauging station	Relative impact on rated peak flow
Stanley River	Woodford	+30 to +75%
	Somerset Dam	Nil
Upper Brisbane River	Linville	+1 to -6%
	Gregors Creek	-1 to -6%
	Wivenhoe Dam	Nil
Lockyer Creek	Gatton	
	Glenore Grove	-10 to +30%
	Lyons Bridge/Rifle Range Road	
Bremer River	Walloon	+4 to +21%
Warrill Creek	Amberley	0 to +35%
Purga Creek	Loamside	-1 to +25%

Sub-catchment model	Key gauging station	Relative impact on rated peak flow
Lower Brisbane River	Savages Crossing	-2 to +7%
	Mt Crosby Weir	-1 to +4%
	Moggill	-3 to +8%

Figure 3-4 below provides an example of the range of rating curves for the Brisbane River at Gregors Creek that have been considered. This figure is a reproduction of Figure 16(b) from the *Data, Rating Curve and Historical Flood Review Report* (Aurecon 2015, Refer Appendix A).

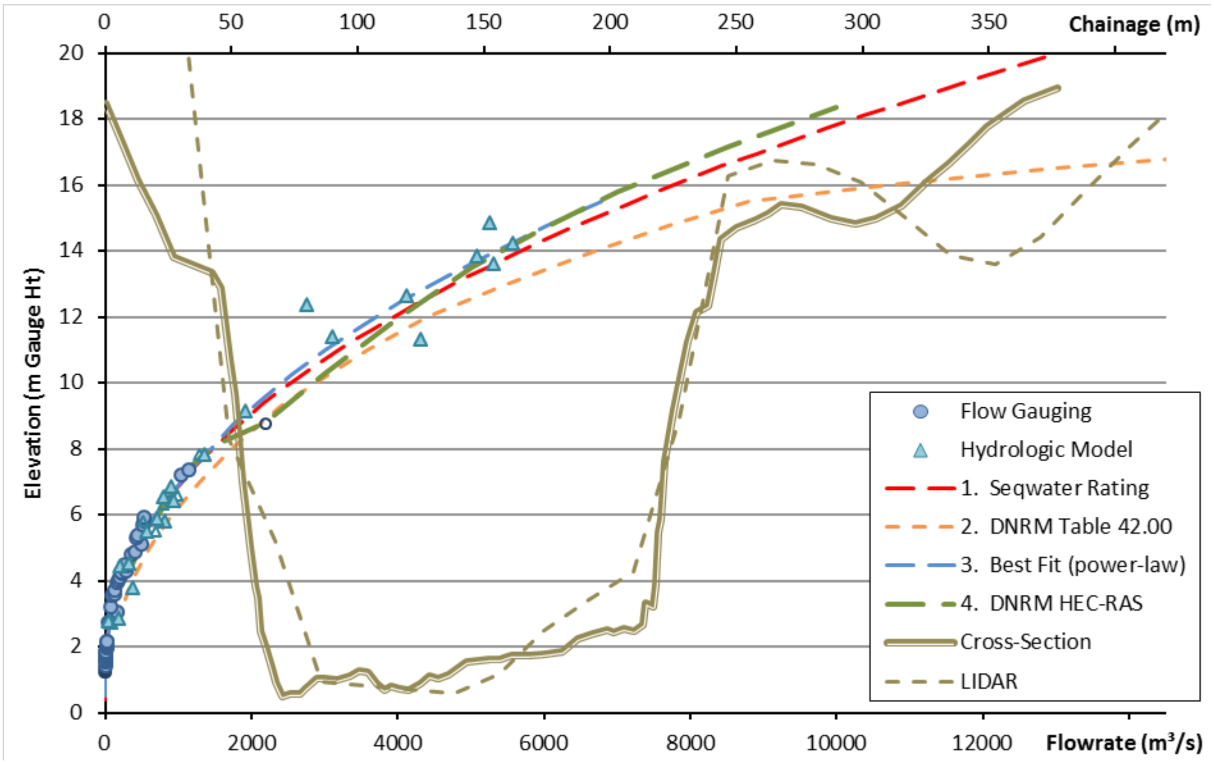


Figure 3-4 Rating comparison – Brisbane River at Gregors Creek (full range)

Figure 3-5 below has been reproduced from the *Data Rating Curve and Historical Flood Review Report* (Aurecon 2015, Refer Appendix A). This figure shows that review of the flood gauge rating curves is a complex and iterative process that is tied into other aspects of the hydrologic assessment, including calibration of hydrologic models and flood frequency analysis of gauges across the catchment. These processes are dependent upon the gauge ratings, but achieving catchment-wide consistency may require ongoing review and adjustment of the ratings.

The recalibration of the hydrologic model feeds into the Design Event Approach and Flood Frequency Analysis. These techniques are being applied concurrently and the results are being fed back into the recalibration. This process forms a feedback loop in terms of the overall flood estimation methodology. The iteration is continued until a satisfactory consistency between outcomes is achieved.

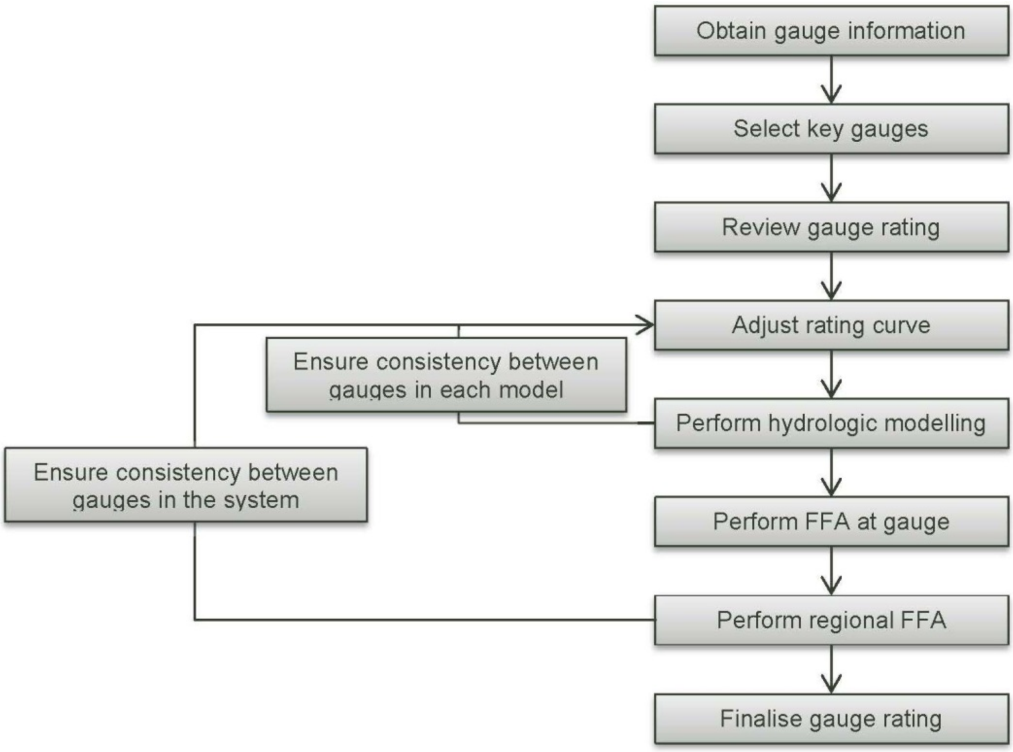


Figure 3-5 Overview of rating curve review methodology

3.4 Tides

Predicted series of tide levels for the mouth of the Brisbane River have been obtained from the BoM publication of standard tide data, which is available from the BoM website. This data was used to develop a probabilistic model for the derivation of storm surge combined with tide levels for the mouth of the Brisbane River. This was developed as part of the MCS framework, refer to Section 7 of this report.

Estimates of the storm surge were obtained from the draft GHD report that was commissioned for the BCC to investigate the implementation of the coastal plan, (GHD, 2014).

3.5 Dam operations

3.5.1 With-dams conditions storage levels

Daily flow volumes from the historical simulations representing the current water supply infrastructure were obtained from the Integrated Quantity Quality Model (IQQM) provided by DSITIA (DSITIA, 2013a). This includes data for the period 1889 to 30 June 2011 (120 years) from the following dams:

- Wivenhoe (full supply capacity = 1,165,200 ML)
- Somerset (full supply capacity = 379,800 ML)
- Moogerah (full supply capacity = 83,800 ML)
- Lake Manchester (full supply capacity = 26,200 ML)
- Cressbrook (full supply capacity = 81,800 ML)
- Perseverance (full supply capacity = 30,100 ML)

It is noted that the models assume current infrastructure, operations and full entitlement demands for the full period of simulation. Operations are as per the Moreton Resource Operation Plan (ROP), which are assumed to be close to current operations. DSITIA indicated that the results are “fairly different” to the current level of use for all systems, for the following reasons:

- Lake Manchester has an extractive demand of 5,800 ML/a in the model. In practice, Lake Manchester was used in the recent drought to supplement supplies from Wivenhoe. To DSITIA’s knowledge, Lake Manchester has not been used since the drought
- Full entitlement demand for the Wivenhoe system is about 273,000 ML/a. Some recent work that DSITIA has completed for DEWS has indicated that 'current' use is about half of this value

Whilst it is acknowledged that the ROP simulation may not necessarily reflect the actual current operation of the Seqwater Grid supply system, it does reflect the current entitlements available to users. Therefore it is considered that this data provides a reasonable representation of the long term performance of the system. This information was used to generate correlations between the various dam storage levels in the MCS analysis.

3.5.2 No-dams conditions flood volumes

Daily flow volumes from the historical simulations representing pre-development conditions were obtained from the Integrated Quantity Quality Model (IQQM) provided by DSITIA (DSITIA, 2013b). This includes for the period 1889 to 30 June 2013 (123 years) for the following locations:

- Brisbane River at Linville
- Brisbane River at Gregors Creek
- Brisbane River at Fulham Vale
- Stanley River at Woodford
- Stanley River at Somerset Damsite
- Cressbrook Creek at Rosentretters
- Brisbane River at Watts Bridge
- Brisbane River at Wivenhoe Damsite
- Lockyer Creek at Helidon
- Lockyer Creek at Gatton
- Lockyer Creek at Glenore Grove
- Lockyer Creek at Rifle Range Road
- Bremer River at Walloon
- Warrill Creek at Amberley
- Purga Creek at Loamside
- Brisbane River at Moggill
- Brisbane River at Port Office Gauge

The simulation scenario adopted is the Water Resource Plan (WRP) pre-development scenario (1889-June 2000), which has been extended to June 2013.

The model extension was done using existing rainfall-runoff calibrations and adjusting to recorded flow data. It should be noted that these daily flows were derived using a Sacramento Model (Rainfall-runoff) of the catchment which was calibrated to rated flows derived from available rating curves for the selected gauges. These ratings are different to those adopted in this study and so therefore it is likely that there will be some differences in the estimates obtained from this data. The difference s in adopted rating is not considered to be significant for the low flow range in most instances.

However, this data provides a long record length which is homogenous and stationary. It is considered suitable for use in providing some relative comparison with the estimates of flood volume derived from this study.

3.5.3 Dam releases

Gated dams

Dam release records for Somerset Dam and Wivenhoe Dam were used as inputs into the determination of the inflows for the various historical flood events. Reference is made to Section 5.2 of the Seqwater report (2013). The estimated inflows and recorded release were then used to calibrate the URBS hydrologic models.

Fixed crest dams

Dam release records of Perservance, Cressbrook Creek, Lake Manchester and Moogerah Dam were also used to estimate inflows for various historical events.

3.6 Stochastic space time patterns

The synthetic storm data prepared by the BoM and SKM (SKM, 2013) for Seqwater's WSDOS study have been used for the Monte Carlo Simulation (MCS) analyses. The BoM and Jacobs (Jacobs, 2014) were also commissioned as part of the BRCFS to prepare additional stochastic replicates based upon the January 2013 flood event for inclusion in the MCS. These patterns provide a realistic representation of the variability of the spacial and temporal distribution of rainfall across the Brisbane River catchment. Refer to Section 4.4 for further details.

4 Hydrologic models

4.1 Introduction

In the design event approach (DEA) and the Monte Carlo simulations approach (MCS), synthetic events are simulated with a hydrological model to derive a range of peak flows and flow volumes at the locations of interest. The Brisbane River hydrological model was developed by Seqwater and implemented in the URBS hydrological model suite (Carroll, 2012). The model was calibrated by Seqwater (2013) and subsequently recalibrated by Aurecon. Refer to *Hydrologic Model Recalibration Report* (Aurecon, 2014, Appendix C).

In the adopted model configuration, the Brisbane River catchment is divided into seven distinct sub-catchment models based on a review of topography and drainage patterns, major dam locations, key locations of interest for real time flood operations, and consideration of the best use of available data including water level gauges. Dams and reservoirs are modelled within URBS as well, with the exception of Wivenhoe Dam and Somerset Dam. The latter two are modelled in RTC tools, an open source, modular toolbox dedicated to real-time control (RTC) of hydraulic structures like weirs, pumps, hydro turbines, water intakes, etc. Figure 4-1 and Figure 4-2 show the computational workflow for the 'no-dams' and 'with-dams' conditions. More details on the hydrological and dam operation models are provided in subsequent sections.

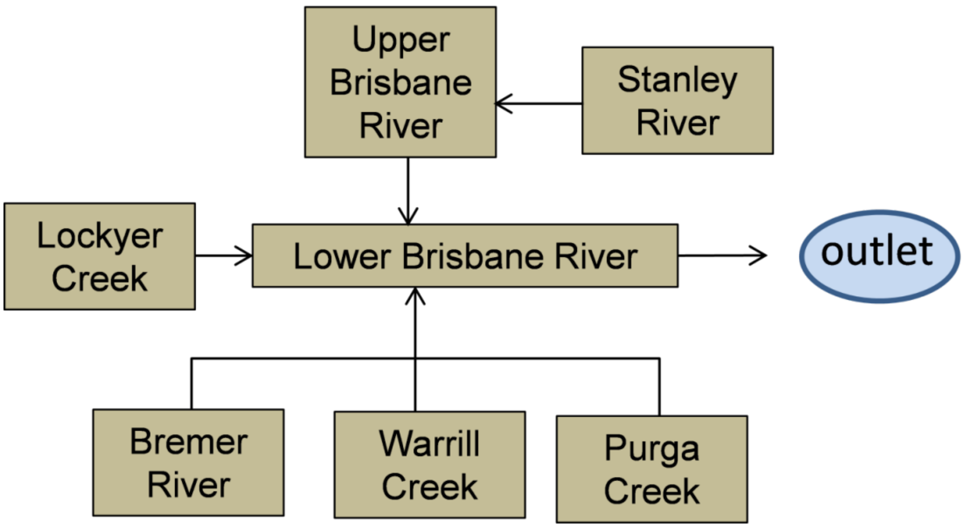


Figure 4-1 Computational workflow for the 'no-dams' conditions

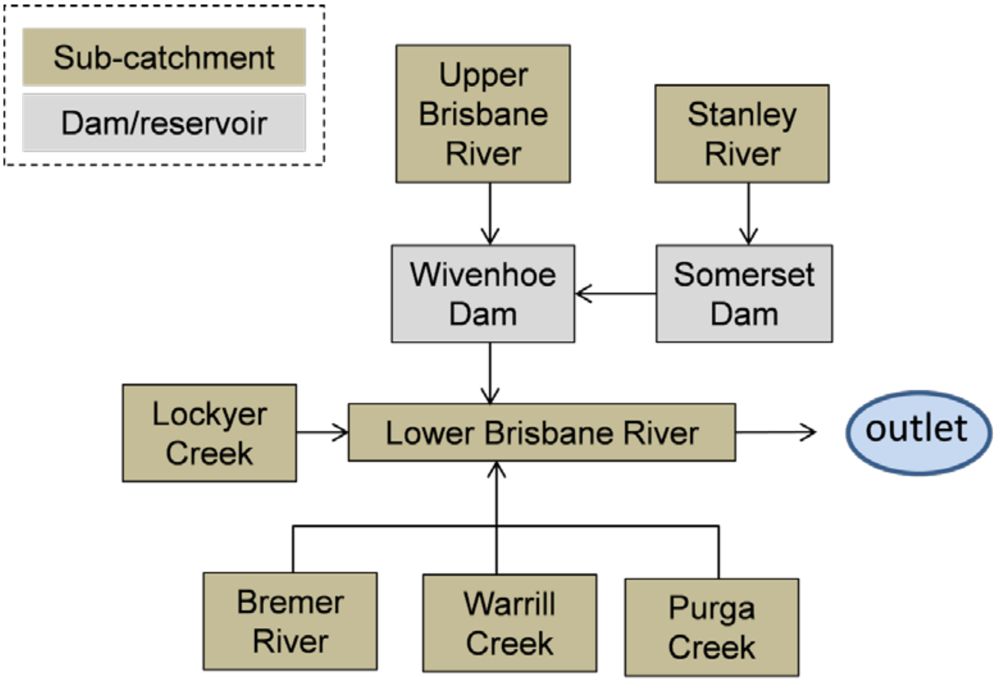


Figure 4-2 Computational workflow for the 'with-dams' conditions

4.2 Runoff-routing model

Hydrologic modelling techniques are being utilised to estimate design flood flows and volumes using both the standard design event approach as outlined by Australian Rainfall and Runoff (AR&R) (EA, 2003) and also as part of a Monte Carlo simulation framework.

As part of the Comprehensive Hydrology Assessment for the Brisbane River Catchment Flood Study (BRCFS), a review of the URBS model developed by Seqwater for the Wivenhoe and Somerset Dams Optimisation Study (WSDOS) (DEWS, 2014) was undertaken. A review of the rating curves generated by Seqwater, DNRM, BoM and other sources was also undertaken.

These reviews were required as the purpose for the hydrologic modelling in the WSDOS study and the current investigation are different. The following differences in the objectives for the BRCFS Hydrology Study are present:

- Greater emphasis on the range of design floods beyond the observed and 'measured' range, thus it is important to include the extrapolation of the revised rating curves
- Improved representation of the key flood production characteristics of the different sub-catchments for estimation of design floods over the broader range of flood magnitudes
- Ensuring consistency of modelling assumptions with AR&R recommendations for modelling of extreme events

A combination of the outcomes from these reviews defined the modifications required to the Seqwater URBS model for use in the BRCFS. These modifications call for a recalibration of the URBS model to ensure that its application covers the range of events required by the BRCFS study (1 in 2 AEP up to the Probable Maximum Flood).

The analysis has been conducted using the URBS model version 5.70 Beta which included amendments to the base flow model that enabled the base flow volume factor to be limited by reference to the rainfall magnitude. The Monte-Carlo Simulation framework was modified to enable the amendments to the representation of base flow to be included. These modifications are described in *Hydrologic model recalibration report* (Aurecon, 2015, Refer Appendix C), and *Monte-Carlo simulation framework and enhanced MCS methodology report* (Aurecon, 2015, Refer Appendix D).

4.2.1 Model layout

The URBS model review assessed the modifications required to the URBS model to ensure that it is fit for purpose in deriving design flood estimates for the Brisbane River Catchment Flood Study (BRCFS). Figure 4-3 shows the sub-catchment division of the runoff-routing model adopted by Seqwater (Seqwater, 2013).

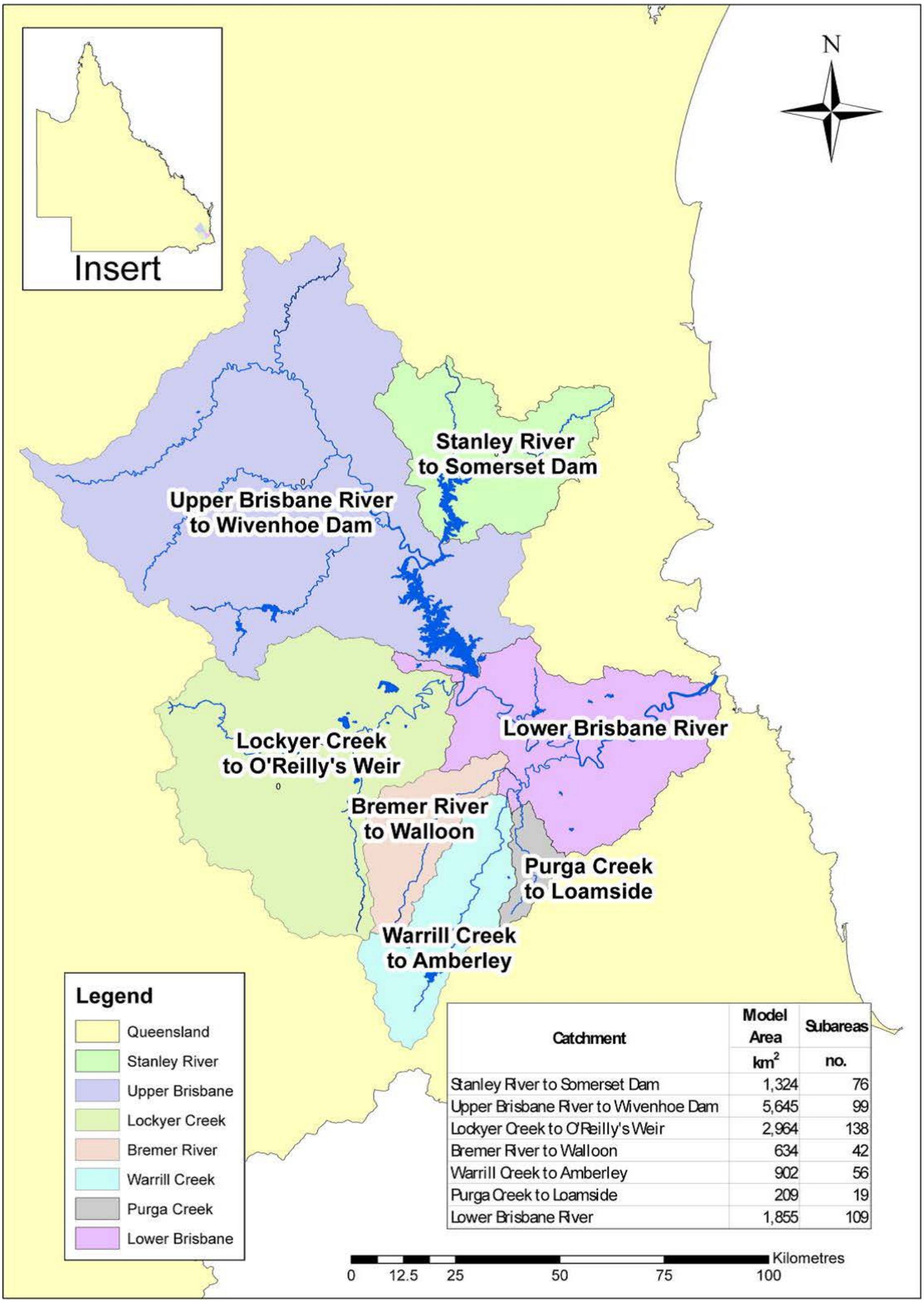
A summary of the outcomes of the URBS model review, as presented in the report titled *Hydrologic Model Calibration and Validation Review Report* (Aurecon, 2015 refer to Appendix B) is as follows:

1. Remove the Kedron Brook catchment from the Brisbane River catchment area in the Lower Brisbane model. Kedron Brook does not drain into the Brisbane River and it was inadvertently included by Seqwater. However this modification does not change the calibration performance of the Lower Brisbane River model as Kedron Brook entered downstream of the lowest reference gauge which is the Brisbane City Gauge
2. Adopt the inclusion of:
   i) Impervious fractions to represent increased runoff volume in urban areas
   ii) Urbanised areas to represent reduced response times
   iii) Reduced reach length factors for heavily modified reaches in the Lower Brisbane model
3. Adopt changes to the channel routing parameters for the following sub-catchment models:

   Lockyer Creek to O'Reillys Weir – n = 0.85

   Purga Creek to Loamside – n = 0.85

   Bremer River to Walloon – n = 0.85
4. Reject amendments to conceptual storages based upon DMT hydraulic model, but modify the adopted relationships by reducing the storage for flows above 10,000 m^3/s by 20%. Do not change the representation of the online conceptual storages as doing so introduces greater complexity that is not warranted. (Note that this review recommendation was based on interim DMT TUFLOW hydraulic model results. Subsequent refinement/recalibration of the upstream hydrologic models identified that the adopted relationships were no longer appropriate, while development of level-storage relationships and level-discharge rating curves based on improved DMT TUFLOW model results allowed revised storage relationships to be calculated. This development is discussed in the report titled *Hydrologic Model Recalibration Report*; refer to Appendix B)
5. Reject the suggested change of including a diminishing CL rate by introducing a maximum soil storage infiltration capacity. This adds further complexity without necessarily producing a better model calibration
6. Maintain the linear base flow model as the introduction of a non-linear base flow model does not change the model calibration performance significantly. Introduce a Base flow Volume Factor to cap the base flow based upon the findings of the AR&R Project 7 Stage 2 Final Report



Catchment	Model Area km^2	Subareas no.
Stanley River to Somerset Dam	1,324	76
Upper Brisbane River to Wivenhoe Dam	5,645	99
Lockyer Creek to O'Reilly's Weir	2,964	138
Bremer River to Walloon	634	42
Warrill Creek to Amberley	902	56
Purga Creek to Loamside	209	19
Lower Brisbane River	1,855	109

Figure 4-3 Brisbane River sub-catchment layout (from Seqwater 2013)

Channel routing in the URBS model is based upon the non-linear Muskinghum model which includes Alpha as a channel routing lag parameter and ‘n’ as an exponent as the Muskinghum non-linearity parameter. Setting Muskingum ‘n’ to a value other than 1, assumes the non-linear Muskingum model. This allows the model to vary lag with flow; a value less than 1 implies a decrease in lag with increasing flow, whereas a value greater than 1 implies vice versa.

What was also clearly identified in the review was the need for channel routing and conceptual storage parameters to be reassessed during the hydraulic modelling phase of the BRCFS. This means that a further iteration of calibration may need to occur once a fully calibrated hydraulic model of the Lower Brisbane River becomes available which shows that the adopted rating curves and routing characteristics are inconsistent. This exercise is considered part of a continuous improvement approach which may also include reviews after every major flood event.

4.2.2 Model calibration methodology

The recalibration process has been carried out following a similar methodology to the Seqwater calibration process wherever possible. Due to data limitations and project constraints, the recalibration process has focussed on five key events: January 1974, May 1996, February 1999, January 2011 and January 2013. These events were selected as they represent moderate to major floods and they also contain the best recent records in terms of spatial and temporal rainfall and stream flow information. Events such as the January 1893 event were not used for calibration due to a lack of temporal rainfall data and the sparse amount of stream flow records. Events prior to 1955 have limited pluivograph data and so the temporal representation of these events is generally poor. Seqwater treated the ten historic events prior to 1955 as verification events, (Seqwater, 2013), and this approach was adopted in the current study.

A revised set of recommended parameters has been calculated from the results for these five events and has been applied to all 38 of Seqwater’s calibration events. A comparison of the model results was performed using the recommended parameters.

A number of changes to the models were recommended as part of the *Hydrologic Model Calibration and Validation Review*, as shown in Table 4-1.

Table 4-1 Adopted URBS model changes

	Stanley	Upper Brisbane	Lockyer	Bremer	Warrill	Purga	Lower Brisbane
Include revised rating curves	✓	✓	✓	✓	✓	✓	✓
Include channel routing non-linearity (n = 0.85)			✓	✓		✓	
Remove Kedron Brook catchment (Seqwater subareas 111, 113, 97, 99 and105)							✓
Include impervious fractions, urbanised areas and reduced reach length factors							✓
Modification of conceptual storage volumes (based on physical storage characteristics)		✓					✓

The recalibration process has seen further modifications to a number of the models as follows:

- Stanley River model: the reporting location for the Woodford gauge was moved to the downstream junction to represent total flows through this area, as the hydraulic model of this area indicated that this was a more appropriate location
- Lockyer Creek model: the schematisation of the lower Lockyer Creek was modified to include the main channel and three separate bypass locations. This was felt to be an appropriate schematisation for this reach where the main channel is perched and the breakout flows travel slowly through the floodplain
- Lower Brisbane model: Calibration parameters alpha and beta were set to typical values for the local tributaries. Main channel routing time was reduced by applying a reach length scaling factor. Storage-discharge relationships used in the conceptual storages have been related directly to physical properties of the river and floodplain by combining level-volume relationships taken from DTM with level-flow relationships estimated from the main gauge rating curves

The URBS model calibration process has been implemented to establish a single set of model parameters that achieve a reasonable calibration across a wide range of flood event types and magnitudes ranging from 1 in 2 AEP up to the Probable Maximum Flood. It is acknowledged that slightly improved performance statistics for any individual event could be achieved by adjusting the parameters for each event.

In general the recalibration process has seen either an improved or equivalent quality of calibration for all catchments when compared to the Seqwater results:

- Calibration results were similar in the Stanley River subcatchment and were slightly improved in the Upper Brisbane subcatchment, especially for the 1999, 2011 and 2013 events
- Calibration in the Lockyer Creek has focussed on Glenore Grove due to the unsuitability of the stream gauges further downstream for recording out-of-channel flows. The calibration at Glenore Grove is improved across all events
- Overall, calibration in the Bremer River and Warrill Creek subcatchments was similar and a slight improvement in calibration was achieved in the Purga Creek subcatchment
- Calibration in the Lower Brisbane model was notably improved for most events, particularly with respect to the timing of flow routing along the river

4.2.3 Sub-catchment model parameters

The recommended alpha and beta parameters remain similar for the Upper Brisbane River sub-catchments where the only changes to the models were to rating curves, and conceptual storages in the lower reaches (for the pre-Wivenhoe conditions). In the Stanley River where the model was modified around the Woodford gauge, the alpha value was reduced and the beta value was increased. In the models where channel routing non-linearity was introduced (Lockyer, Bremer and Purga), alpha values were increased due to the different routing exponent to obtain a reasonable calibration and beta values were modified as required. In the Warrill Creek model, where only the rating curves were modified, the alpha value was slightly increased and the beta value was decreased. In the Lower Brisbane, alpha and beta were increased slightly to provide a typical representation of local tributaries but do not necessarily represent any individual tributary. A reach length factor was applied to main channel to match routing times observed between stream gauges along the Brisbane River.

The recommended model parameters for each sub-catchment model are shown in Table 4-2.

Table 4-2 Recommended model parameters

Sub-catchment	Alpha	Beta	m	n
Stanley River	0.11	5.7	0.8	1.0
Upper Brisbane River	0.12	2.8	0.8	1.0
Lockyer Creek	0.49	3.1	0.8	0.85
Bremer River	0.79	2.8	0.8	0.85
Warrill Creek	0.79	2.5	0.8	0.85
Purga Creek	0.93	3.8	0.8	0.85
Lower Brisbane River	0.30[a]	4.0	0.8	1.0

Notes: (a) Reach length factor of 0.2 applied to main channel reach lengths
Alpha = channel routing lag parameter
Beta = catchment lag parameter
m = catchment non-linearity parameter
n = channel routing - Muskingum non-linearity routing parameter

When comparing model results from the recommended parameters runs across the full range of verification events, all of the examined flow gauges generally show a good correlation between calculated and rated peak flow rates and event volumes with no obvious flow rate related bias.

4.2.4 Limitations of the hydrologic model and calibration

URBS uses non-linear Muskingham routing to perform runoff-routing program with a primary focus towards flood forecasting and design flood hydrology. Several areas of the river exhibit characteristics that are difficult or impossible to represent in a hydrologic model. These areas include (but are not necessarily limited to):

- The lower Lockyer Creek and Bremer River floodplains, which are affected by backwater from the Brisbane River. Depending on coincident timing of tributary flows, river flows can backflow some distance up the tributary. Water stored within the tributary floodplain remains until combined river flows recede. Similarly, the areas upstream of the confluence of Bremer River and Warrill Creek (notably the gauge site at Walloon) are potentially affected by backwater during large events
- Several areas of the Brisbane River catchment, particularly in the lower Brisbane River, display inconsistent channel characteristics where the river breaks out into certain areas at high flows while remaining channelized in others. These areas are often offline from the main river channel and the models have attempted to account for these flow patterns by including storage nodes. These relate a defined additional storage volume to a known flow rate in the river, however this representation does not allow the dynamic response of the river whereby the water level (and hence storage) may lag several hours behind the river flow
- Each of the sub-catchment URBS model is characterised by a single set of parameters (alpha, beta and m) listed in Table 4-2. In certain areas reach length factors have been used to modify the effective channel routing parameter applied to that area, however in general the parameters are assumed to be consistent across the catchment. The model therefore represents the general catchment characteristics, as measured at the main calibration locations, but individual sub-areas within the model have not been explicitly represented or calibrated. Minor tributary flows should be treated as indicative only and used with caution

The recalibration process has been conducted using best available information, however many aspects of this data are acknowledged to be unverified and/or potentially subject to future change. Known limitations include:

- Where possible, the gauge ratings have been developed independently from the hydrologic model, but often reliable validation data (eg flow measurements) are only available for low to moderate flows, with the ratings having to be extrapolated using alternate methods (eg hydraulic modelling)
- A single consistent rating has been used at each site. Many factors (eg changes to channel shape, vegetation) may cause the rating to exhibit gradual or rapid change. Consistency of rating was one of the criteria used to select primary calibration sites, however accuracy to any one specific event is not guaranteed
- Several of the ratings, particularly in the Lockyer and Bremer catchments, become very sensitive at high flows, with small variations in level, whether due to measurement inaccuracy or variation in the rating, potentially leading to large changes in rated flow. High flow measurements at these gauges (identified in Table 3-2 and Table 3-3) should be treated with caution
- Lower Brisbane ratings were developed based on results from the DMT TUFLOW model, adjusted where possible/necessary to match independent flow measurements. However, the DMT TUFLOW model was developed prior to finalisation of the current BRCFS study and was therefore calibrated using flows that are not necessarily consistent with the current BRCFS hydrology, nor will the DMT TUFLOW model (and subsequently derived ratings) necessarily be consistent with the future BRCFS hydraulics phase hydraulic modelling

Calibration of the hydrologic model is therefore dependent on data that is subject to continuous improvement. In particular, updated hydraulic modelling of the lower Brisbane River areas is to be conducted as part of the hydraulics phase of the BRCFS. The recalibration process highlights the need for review of the performance of the hydrology models, particularly the lower Lockyer Creek floodplain area and Lower Brisbane, to be carried out once a calibrated hydraulic model is available to ensure that the routing characteristics of the two approaches are consistent. However it must be acknowledged that due to limitations of the hydrologic routing implicit in the URBS models it may not be possible to fully replicate complex dynamic or hydraulic phenomena (eg backwater effects).

4.3 Dam operations model

4.3.1 Introduction

Wivenhoe Dam and Somerset dam are simulated with the real-time control software RTC tools. RTC-Tools is an open source, modular toolbox dedicated to real-time control (RTC) of hydraulic structures like weirs, pumps, hydro turbines, water intakes, etc. It can be used in standalone mode or in combination with hydraulic models for general modelling studies as decision support component in operational forecasting and decision-support systems, for example for drought management and water allocation, flood mitigation or the dispatch of hydropower assets. RTC tools are used as a real-time forecasting model for the operational management of the Wivenhoe and Somerset reservoirs by Seqwater.

Somerset Dam and Wivenhoe Dam are operated in accordance with procedures outlined in the *Manual of Operation Procedures for Flood Mitigation at Wivenhoe Dam and Somerset Dam* Revision 11 (Seqwater, 2013). The capacity of the urban water supply compartment that relates to Wivenhoe Dam's Full Supply Level (FSL) is 1,165,000 ML. The Dam can also store up to an additional 1,967,000 ML as temporary flood storage up to EL 80.0 m AHD. Flood releases are made through the main gated spillway (which contains five radial gates), and also an auxiliary spillway that consists of a three bay fuse plug embankment. The radial gates should be fully open prior to the initiation of the first fuse plug embankment. Refer to Figure 4-4 for the schematic representation of the main spillway of Wivenhoe Dam.

For Somerset Dam, the capacity of the urban water supply compartment related to its FSL is 380,000 ML with 721,000 ML volume available for use for temporary flood storage up to EL 109.7 m AHD. Somerset Dam is equipped with four regulator cone dispersion valves, eight sluice gates and eight sector gates. During flood operations the eight sector gates are fully opened to allow free overflow over the spillway prior to the onset of the flood. The regulator valves are generally not used for flood releases as elevated tailwater levels tend to impair the performance of the valves. Therefore the eight sluice gates and the spillway flows are the main flood release mechanisms for Somerset Dam during a flood event. Refer to Figure 4-5 for a schematic representation of Somerset Dam.

The Dam Operations Module as implemented in RTC tools is based upon the Loss of Communications (LOC) emergency flood operation procedure described in the Flood Manual (Seqwater, 2013). The reason to implement the LOC scenario instead of the regular dam operation strategy is the fact that the latter is relatively complex due to the iterative nature of the decision process and therefore difficult to implement especially in a Monte Carlo Simulation framework. Bearing in mind project constraints and that the purpose of this study is for floodplain management (ie not operational management), the implementation of the LOC was preferred.

The Loss of Communications (LOC) emergency flood operation procedure was successfully implemented in the RTC tools model. The model performance of the RTC tools dam operations model was compared to Seqwater's GoldSim model. Model results were compared for 24 synthetic events, ranging from moderate to extreme flood events. The comparison showed that predicted Wivenhoe Dam outflow hydrographs of RTC tools closely matched the predicted hydrographs of the GoldSim model. As a follow-up activity, the drain-down process incorporated into the LOC was modified to reflect the normal operation procedure and mimic the seven day drainage requirement.

The LOC approach does not take into consideration the downstream tributary flows as the releases from Wivenhoe Dam are based solely on the lake level. This approach is therefore limited because it does not take into consideration the delay in releases to ensure that the releases are not coincident with the flows emanating from the downstream tributaries. As a consequence, the LOC scenario on average results in slightly 'conservative' estimates of peak discharges and flow volumes in the Lower Brisbane River. For floods within the range of 2,000 m^3/s to 16,000 m^3/s, the peak flow in the mid-Brisbane River and Lower Brisbane River according to the LOC scenario are on average in the order of 5 to 10% higher than the peak discharges that result from the Dam operations using the Flood Manual procedures (Seqwater, 2013). This means the derived frequency curves for the 'with-dams conditions' are conservative as well. An investigation of the application of an adjustment of the resultant peak flow estimates was conducted as a result. Refer to Section 9.2.1 for further discussion of this issue.

4.3.2 Wivenhoe dam

The target release of Wivenhoe Dam is based on Wivenhoe Dam headwater levels only. Headwater levels are determined by inflow and release rates. Inflow into and outflow from the Wivenhoe Dam reservoir will result in level changes of Wivenhoe Dam. The Level-Volume relation for Wivenhoe Dam is taken from the Wivenhoe Technical Data, as described in Appendix E of the *Manual of Operational Procedures for Flood Mitigation at Wivenhoe Dam and Somerset Dam* (Seqwater, 2013) Wivenhoe Dam has two relevant inflows:

1. The unregulated inflow from the Upper Brisbane River, as simulated with the URBS hydrological model
2. The releases from Somerset Dam, as determined from the RTC model of Somerset Dam

As release rates influence the lake level and the lake level influences target outflow rates, the control actions are determined at each time step, based on the situation in the previous time step and taking into account any constraints that may apply. The current implementation of rating curves (level versus total outflow) for the main gated spillway Wivenhoe Dam flow, as well as for the situation of fuse plug breaches is based on the available tables in the Flood Manual (Table 7.3.1 and Appendix F of Seqwater, 2013). For practical purposes, the individual (radial) gates of Wivenhoe Dam are not modelled in the RTC model. However, constraints related to the successive gate operations (opening and closing) are taken into account in the form of lookup tables.

The discharge increment value (per control time step) is used as a rate of change constraint for the combination of Wivenhoe Dam radial gates. For lake levels below EL74.0 m AHD, a limit of 6 increments per hour, or 3 m/hour, (1 increment = 0.5 m) is taken as the constraint in case the water level is rising and a limit of 3 increments per hour, or 1.5 m/hour, is taken as the constraint in case the water level is falling. For lake levels above EL74.0 m AHD, a limit of 20 increments per hour (10 m/hour) was implemented.

Crest overtopping can also occur, which is modelled as a sharp crested weir for the main Dam (dimensions: 2,000 m effective weir length, crest level EL80.1 m AHD, weir coefficient 1.7) and a broad crested weir for the saddle Dams (dimensions: 580 m combined effective weir length, crest level 80.0 m AHD, weir coefficient 1.4). It is assumed that Wivenhoe Dam will not fail if it is overtopped and therefore dam failure will not be modelled. In reality, as stated earlier, overtopping is considered a major threat to the security of Wivenhoe Dam. Wivenhoe Dam is overtopped by an event with a 1 in 100,000 AEP, when the Lake Level reaches EL 80.0 m AHD. However, the process of dam breaching and subsequent flooding downstream is out of the scope of the BRCFS project and therefore the dam is assumed not breach under any circumstance.

4.3.3 Somerset Dam

The decision to determine which control action to take at Somerset Dam is dependent on the headwater levels of both Wivenhoe Dam and Somerset Dam. Headwater levels are determined by inflow and release rates. The Level-Volume relation for Somerset Dam is taken from the Somerset Technical Data, as described in Appendix B of the Manual of Operational Procedures for Flood Mitigation at Wivenhoe Dam and Somerset Dam (Seqwater, 2013). Somerset Dam has one relevant inflow: the Stanley River as simulated with the URBS hydrologic model. The target outflow from Somerset Dam is directly routed to Wivenhoe Dam reservoir without any delay. That is the travel time between the two reservoirs is assumed to be instantaneous.

A lookup table is implemented in RTC tools to describe the relation between outflow releases on one hand and the Somerset HW level and the state of the sluice gates on the other hand. Besides releases through the sluice gates, Somerset Dam can also make releases through the radial gates over an ogee crest spillway. The ogee spillway crest level is EL100.45 m AHD. At EL107.45 m AHD, flood waters commence to flow over the Dam crest and flow occurs through the ‘breeze way’. To account for this discharge, the Dam crest is assumed to operate as a broad crested weir. As with Wivenhoe Dam, Somerset Dam is assumed not to fail if it is overtopped and so therefore failure is not modelled.

Only sluice gates are used to adjust the release from Somerset Dam. For this purpose, the target line of Figure 4-6 is used. This target line is followed as closely as possible to maintain a balance in the volumes of Wivenhoe and Somerset reservoirs.

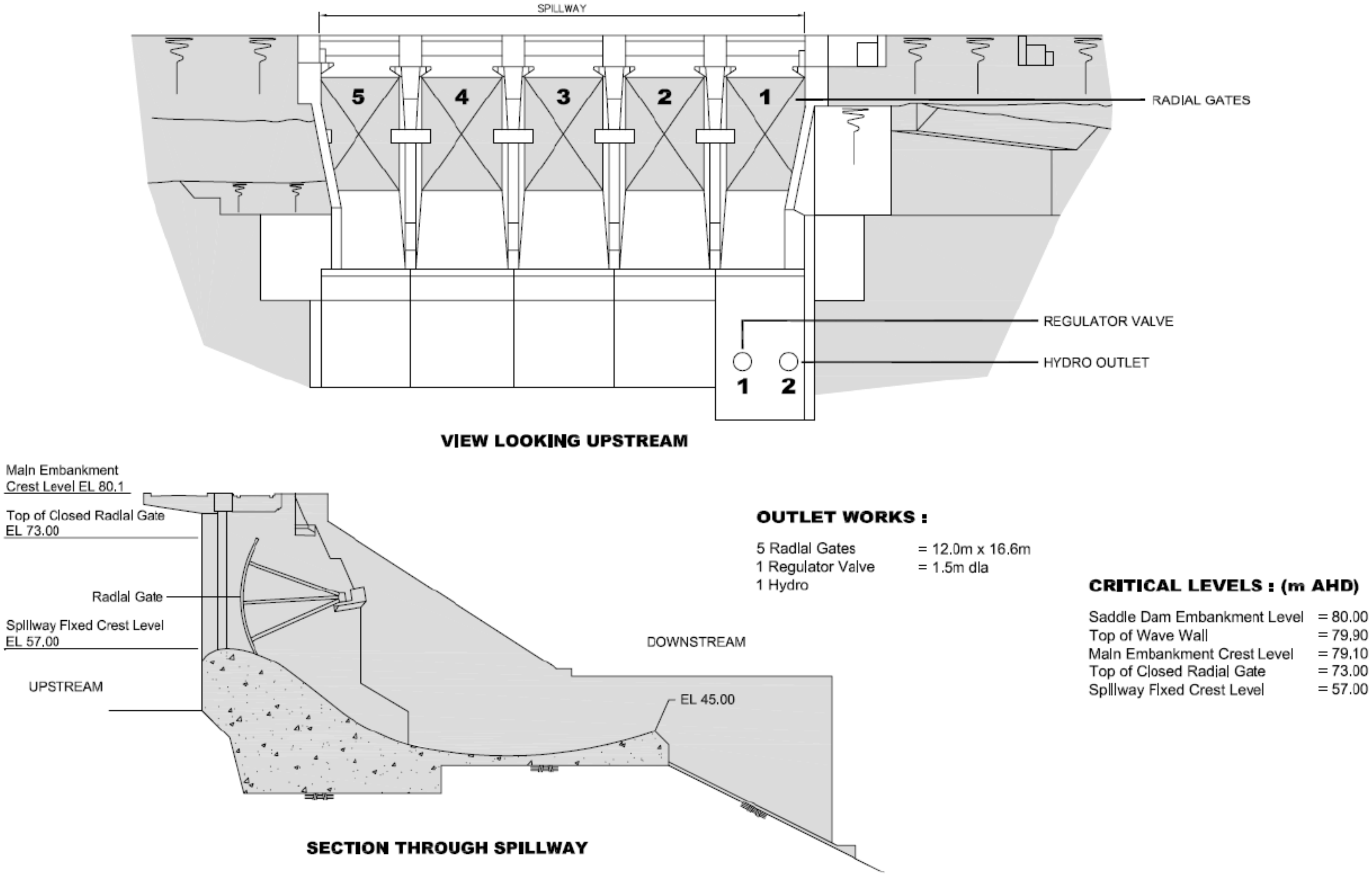


Figure 4-4 Schematic view of Wivenhoe Dam (from Seqwater Drawing No. A3-00392)

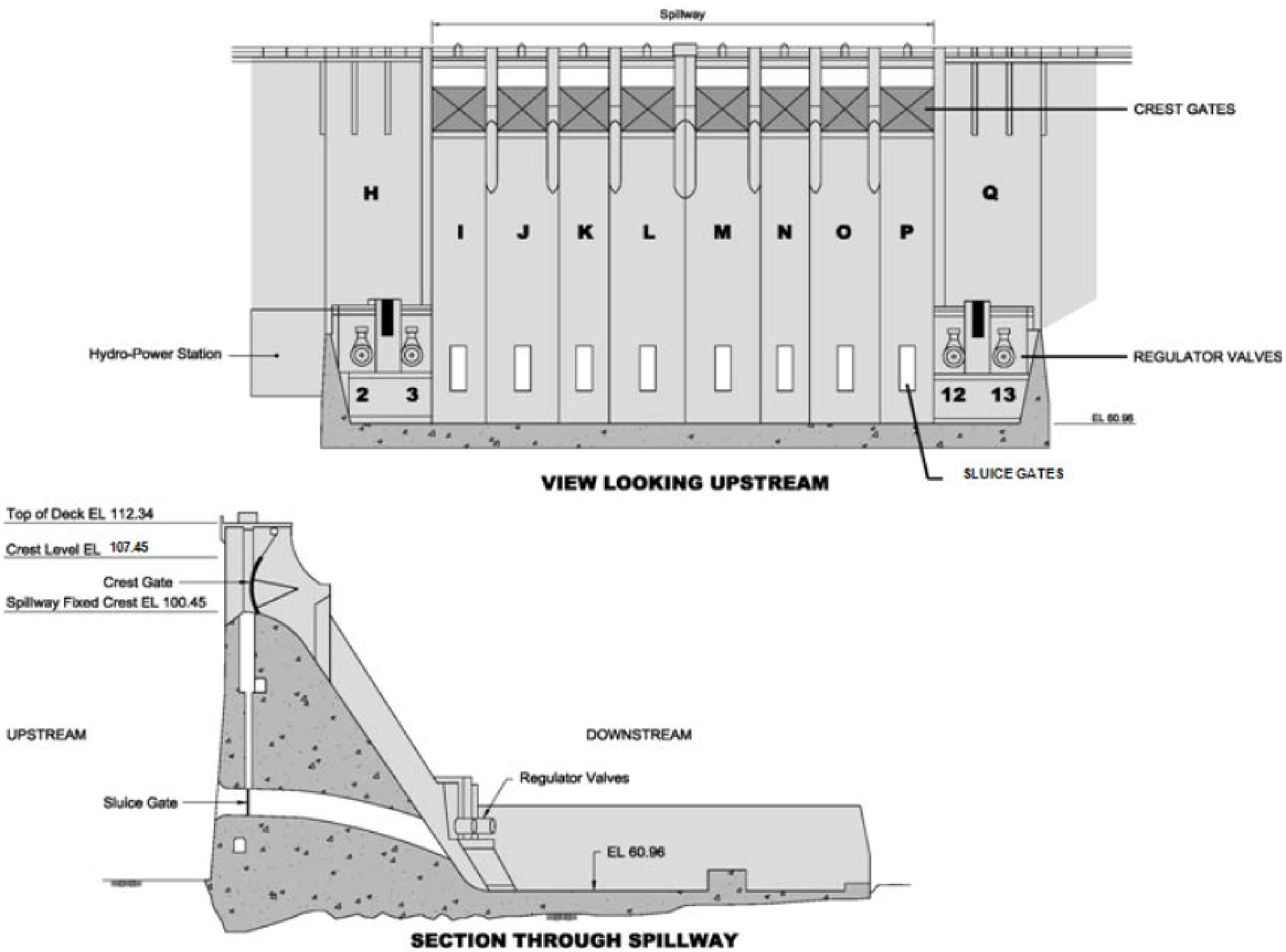


Figure 4-5 Schematic view of Somerset Dam (from Seqwater SEQWC Flood Operations)

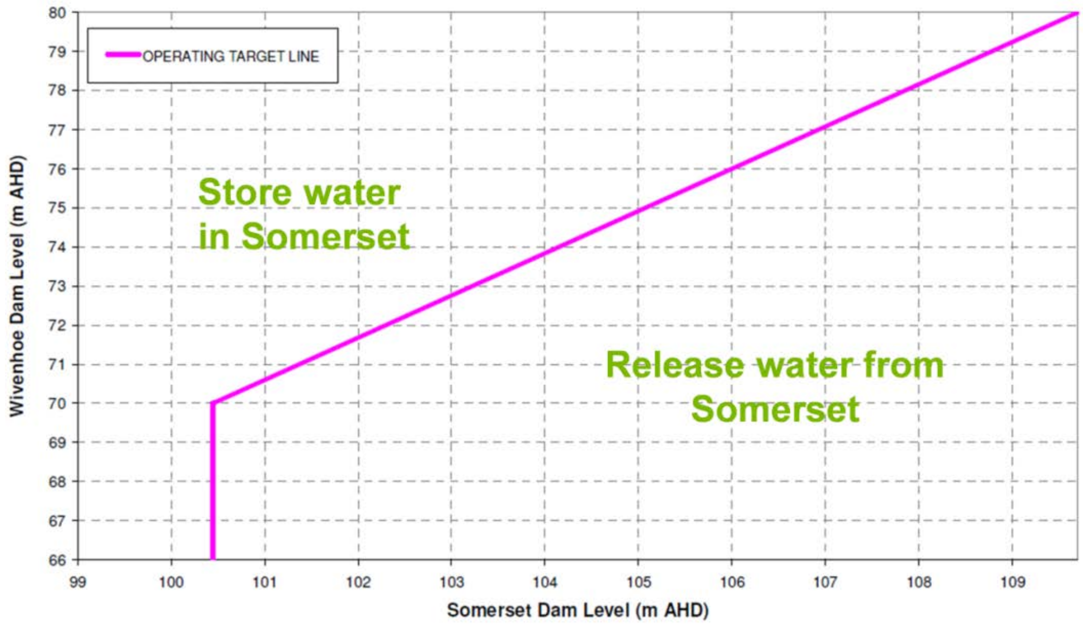


Figure 4-6 Somerset Dam loss of communications procedure

4.4 Stochastic generation of storm patterns

Data of stochastically generated space-time rainfall patterns for the Brisbane River catchment was provided to the BRCFS project by Seqwater. This data was developed as part of the WSDOS project. The method of generating the synthetic events is described in *Brisbane River Catchment Dams and Operational Alternatives Study* (SKM, 2013):

> *These synthetic flood events were produced using a world-leading technique for stochastic generation of space-time rainfall fields, which were generated from radar data A multiplicative-random cascade approach was used to generate 90 replicates of stochastic space-time rainfall patterns across the Brisbane River catchment. The position of the catchment was moved around within the generated spatial domain of the stochastic space-time data to six different possible positions and different segments of time were selected from 10 of the longer replicates. This resulted in 600 space time patterns that were adopted for the stochastic simulation. The generated space time patterns were verified against spatial patterns observed in historical rainfall events that have occurred in the Brisbane River catchment between 1954 and 2012.*

Note: * Jacobs was commissioned as part of the BRCFS to conduct an assessment of the January 2013 flood event using the same techniques. Refer *Additional Stochastic Space-Time Rainfall Replicates for Brisbane river Catchment*, (Jacobs, 2014).

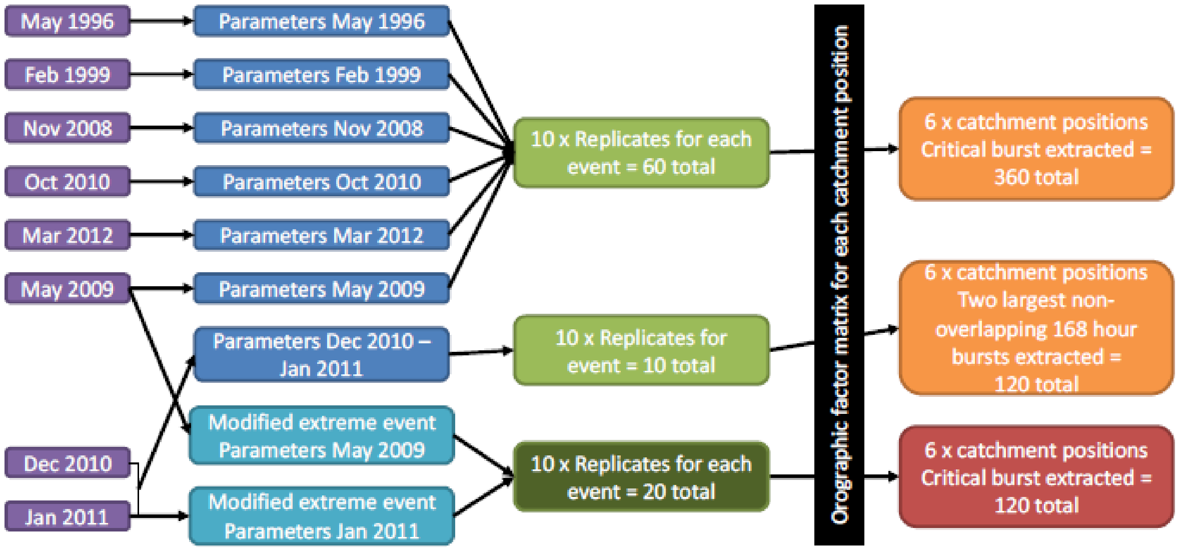


Figure 4-7 Flow chart for production of space-time rainfall patterns (copied from SKM 2013)

Data in SKM (2013) is provided for nine events. Per event there are nine ‘replicates’. The replicates for each single event cover the same period, however the length of the period (number of days) is different for each event. Based on the nine ‘replicates’ of the nine events, the Brisbane River catchment is placed at six locations (see Figure 6-1 of SKM, 2013) in the 256 km by 256 km square (the model domain of the BoM simulation model). In 2014, additional storm patterns were derived based on the January 2013 event as part of the Brisbane River Catchment Flood Study (Jacobs, 2014).

Table 4-3 gives an overview of the ten events on which the 660 synthetic rainfall patterns are based, the applicable minimum and maximum duration (hours), minimum and maximum ARI and the number of spatial patterns. For each event, a list is provided with the maximum burst rainfall depth in mm for different time frames (eg 24 hours; 36 hours; 48 hours; 72 hours; 120 hours; 168 hours) as well as the burst offset.

Furthermore, for each storm pattern a detailed spreadsheet was provided, listing:

- A time series of rainfall for each of the 534 URBS model subareas
- Averaged rainfall in 12 sub-catchments (Table 4-4)

Table 4-3 Overview of space-time rainfall patterns

Event	Min-max applicable duration (h)	Min-max applicable ARI (years)	Number of patterns
1996	24-72	1-1000000	60
1999	24-72	1-1000000	60
2008	24-112	1-1000000	60
2009 normal	24-72	1-1000000	60
2009 slow	24-167	1-1000000	60
2010	24-24	1-1000000	60
2010-2011	24-168	1-1000000	120

Event	Min-max applicable duration (h)	Min-max applicable ARI (years)	Number of patterns
2011	24-35	1-1000000	6
	24-168	1-1000000	54
2012	24-167	1-1000000	60
2013	24-142	1-1000000	60

Table 4-4 The 12 ‘sub-catchments’

Sub-catchments	
Somerset Dam	Lower Lockyer
Upstream of Linville	All Lockyer
Linville to Gregors Creek	Bremer, Warrill and Purga
Gregors Creek to Wivenhoe	Lower Brisbane Only
All Upstream of Wivenhoe Dam	Moggill
Upper Lockyer	Outlet

5 Flood frequency analysis

5.1 Methodology

Flood frequency analysis uses statistical analysis of recorded floods to estimate the magnitude of floods of a selected probability of exceedance. The procedures are typically applied to peak discharges. They may sometimes be applied to flood volumes or even maximum flows over some time period such as a month, although relatively little evidence is available on appropriate types of probability distributions in these cases. Flood frequency analysis is dependent upon the assumption that the variable being examined can be considered to be drawn randomly from a well-behaved statistical distribution.

General guidance on flood frequency analysis is provided in AR&R (1987) and its subsequent updates), however it must be noted that this document is not intended as a strict code of practice. A number of advancements in FFA techniques are addressed in the draft flood frequency chapter of the new version of AR&R Book IV (Kuczera and Franks 2006), although the status of this document is still identified as for review purposes. In 2011, Engineers Australia released a policy statement retracting a number of the specific recommendations in AR&R (1987) and advising that designers should be aware of current best practice standards and adopt the appropriate approach for the set of circumstances.

Flood frequency analysis may be a useful method at a site where streamflow records of at least moderate length are available. It is desirable to have at least 10 to 15 years of data, although situations may occur where short records may have to be used as there is no better alternative. Criteria for deciding if flood frequency analysis should be used are given in the guidelines in AR&R (2003) Book III Section 2.6. The accuracy of flood frequency estimates is indicated by the confidence limits, however factors other than length of record affect the accuracy of the estimate, and methods and formulae leading to the criteria in Book III Section 2.6 are also useful as indicating the likely accuracy of flood frequency estimates.

5.1.1 Data series and distribution fitting

5.1.1.1 Data series

Flood frequency analysis is based on the assumption that the data provides a representative sample of a randomly distributed homogeneous data set. Book IV Section 2.2.1 of AR&R (2003) and Book IV Section 2.1.3 of AR&R (EA Draft, 2006) identify a range of issues that may affect homogeneity of the data population that are addressed in Table 5-1. The historical record at each gauge location was carefully examined to identify issues of homogeneity. Where such issues are encountered, they will be rectified as discussed in the sections below, highly qualified and given reduced priority in the flood frequency analysis, or removed from the record entirely.

Table 5-1 Factors affecting homogeneity of gauge data

Issue	Response
Daily readings, possibly with some intermediate readings during some floods for part of the record, and continuous recording through the remainder	Recent gauge records usually supply continuous level records, but data becomes more sparse (and less reliable) for older gauges. Review of historical gauge records will need to examine quantity and quality of available data
Change of gauging station site	With a few exceptions, gauge locations have been relatively constant. Minor changes to the gauge location should have minimal impact on the gauge rating, however the gauge history will need to be carefully reviewed to identify changes in location and/or datum and impacts
Inability to allow for change of station rating curve, for example resulting from insufficient high-stage gaugings	The Rating Curve Review, particularly for primary gauges, has assessed ratings up to and beyond largest recorded floods at the gauge. Extrapolation of the rating must be consistent with channel shape and properties
Construction of large storages, levees and channel improvements	The six major dams, in particular Wivenhoe and Somerset, represent potentially significant changes to the catchment characteristics. The historical records will need to be adjusted to account for influence of the dams
Growth in the number of farm dams on the catchment Changes in land use such as clearing, different farming practices, soil conservation works, reforestation, and urbanisation	AR&R states that the available evidence indicates that unless changes to the catchment involve large proportions of the total area or large changes in the storage on the catchment, the effects on flood magnitudes are likely to be low and effects are likely to be larger for small floods than for the large floods that are of interest in design Aurecon has investigated the inclusion of urbanisation into the Seqwater URBS model and found that increases on peak flow rates of up to 2.5% changes to flood volumes of up to 4% were observed. The area of catchment impacted by urbanisation is only 2.5% of the total Brisbane River
Changes to rainfall and flood mechanisms including long-term climate change and pseudo-periodic shifts that persist over periods lasting from several years to several decades	Issues relating to periodic shifts in weather patterns have been related to climate indexes such as the Interdecadal Pacific Oscillation (IPO) have been identified and discussed in papers such as Micevski (2006), however the effects not well understood and there is little guidance on how to address the issues in a flood frequency analysis Period of gauge data record will be compared with IPO records to identify gauges that may be at risk of significant bias

Issues considered in the derivation of homogeneous data series for various locations within the catchment include:

- Calculation of flows at rated gauges
- Calculation of flows at isolated unrated gauges
- Translation of levels from unrated gauges
- Extension of records using rainfall-runoff
- Extension of records using historical flood
- Elimination of dam influence

The annual peak series at each site has been compiled from numerous data sources of varying period and quality including:

- Continuous stream gauge records (of levels then converted to flows using a site rating curve), typically recorded by automatic gauges for modern records and by manual staff recordings for older historical records
- Flood peak records, listing peak levels for noted individual flood events
- URBS modelling of selected historical flood events with parameters calibrated to match available gauge records at reliable sites. Model conditions (ie dams) represent those present at the time of the flood event
- URBS modelling of the same events with the same parameters but with the model modified to represent 'no-dams conditions'

Figure 5-1 shows the annual peak series for the Brisbane River at Savages Crossing. This is a reproduction of Figure 4-22 of the *Flood frequencies analysis report* (Aurecon 2015, Refer Appendix F). This gauging station is situated downstream of Cressbrook Creek Dam, Perseverence Dam, Somerset Dam and Wivenhoe Dam and so its record has been influenced by the presence of all of these dams during its history.

Figure 5-1 highlights several of the issues associated with compiling a consistent and homogeneous annual flood record from the available data sources. Several distinct historical periods are evident affecting the quantity and quality of available data:

1887-1906: Limited data of major floods from gauge records and URBS model simulations, but missing low to moderate flood events. Inclusion in analysis only possible using Bayesian fitting methods

1907-1952: Continuous gauge record (typically manually read) provides consistent annual maximum flow data

1953-1982: Continuous instrument gauge record but with some influence from Somerset Dam. URBS modelling used to identify and remove dam influence where possible

1983-2013: Continuous gauge record but with significant influence from Somerset and Wivenhoe Dam. Many minor events almost completely mitigated. URBS modelling used to identify and remove dam influence where possible

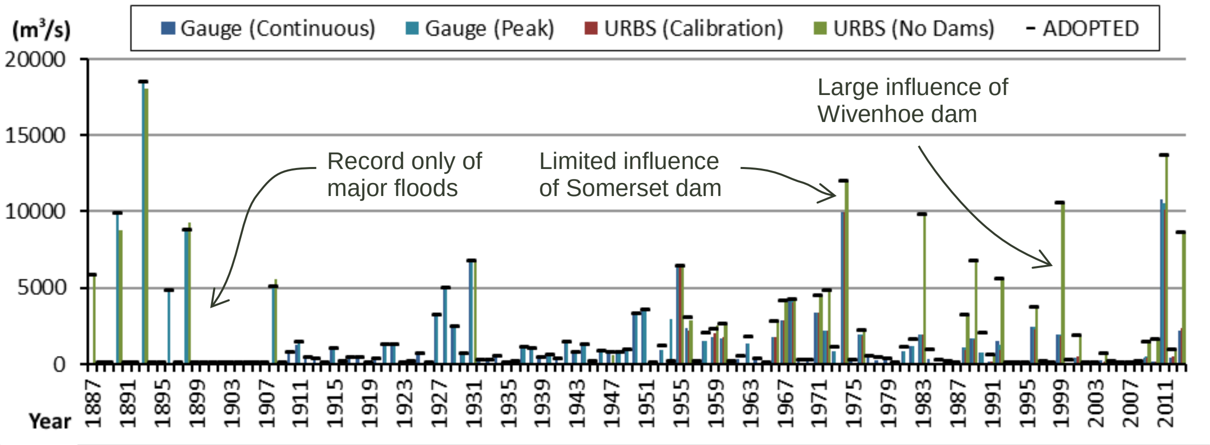


Figure 5-1 Annual peak flow record for the mid Brisbane River at Savages Crossing

Volume frequency analysis was conducted for three sites where reliable flow ratings and continuous stream gauge record unaffected by dams were available. The volume analysis assessed volumes over a fixed duration rather than complete event volumes, which is more consistent with Design Event and Monte-Carlo simulation assessments based on simulation of rainfall bursts, but means that the analysed volumes include baseflow and potentially include flows from separate rainfall events.

5.1.1.2 Distribution fitting

Flood frequency studies require the selection of a probability distribution for fitting to the data. There is no universally accepted probability distribution and historically many different types have been used and/or recommended. AR&R (1987) discusses two general approaches for design procedures, the first being to fit several different types of distribution to each catchment and adopt the distribution which gives the best fit, while the second (recommended) approach is to adopt a single distribution for all catchments in a region or for all Australia based on the best overall fit in many catchments. The Log-Pearson Type III (LPIII) distribution was adopted as the standard distribution in AR&R (1987) because it was found to consistently fit flood data as well, if not better than other probability families for Australian catchments, although the Generalised Extreme Value (GEV) distribution has been gaining popularity as a statistical distribution for flood frequency analysis. Engineers Australia currently does not specifically recommend either distribution as there is no conclusive reason that one distribution should or does consistently provide a better fit of the data.

The GEV and LPIII probability functions are both highly adaptive three-parameter distributions and it is the selection of these parameters that has the greatest influence. Given the current debate regarding LPIII versus GEV, both distributions were fitted to the data for each catchment. The performance of these distributions across the gauge locations was assessed and a single distribution type (found to be the LPIII as discussed in Section 5.1.3.3) was adopted for the final assessment.

There are numerous procedures for fitting the statistical distributions to data. The AR&R (1987) standard methodology adopted the method of moments based on preserving the logarithms of flows, but acknowledged that other methods had been found by some studies to give better results. The method of L-moments has been espoused as giving better parameter estimates for data containing outlying values, while Bayesian methods are generally more flexible. Regardless of the method used, it must be demonstrated to provide a good fit of the data. Methods for improving the fit include:

- Use of regional skew characteristics to minimise overall influence of high and low-end outliers
- Identification of outliers through both statistical assessment and visual inspection
- Checking the fitted distribution against the plotted data and using engineering judgement to identify inconsistencies or other issues with the data and fit

Figure 5-2 which is a reproduction of Figure 4-23 of the *Flood frequencies analysis report* (Aurecon 2015, Refer Appendix F), presents the fitted flood frequency curves to the derived annual series for the Brisbane River at Savages Crossing for the ‘no-dams conditions’. Both the GEV and LPIII distributions are displayed along with the 90% confidence interval.

The confidence interval defines the probability that a range contains the true population function. The 5% and 95% confidence limits enclose the 90% confidence interval. Confidence intervals for a standard probability function can be estimated mathematically, however commonly available methods are not compatible with the advanced sampling techniques and Bayesian fitting methods implemented by FLIKE. Whilst the brief requested that 80%, 90% and 95% confidence limits be derived for each site, FLIKE currently only provides for the determination of 90% confidence limits. This is considered satisfactory for providing an indication of the uncertainty in the flood frequency estimates.

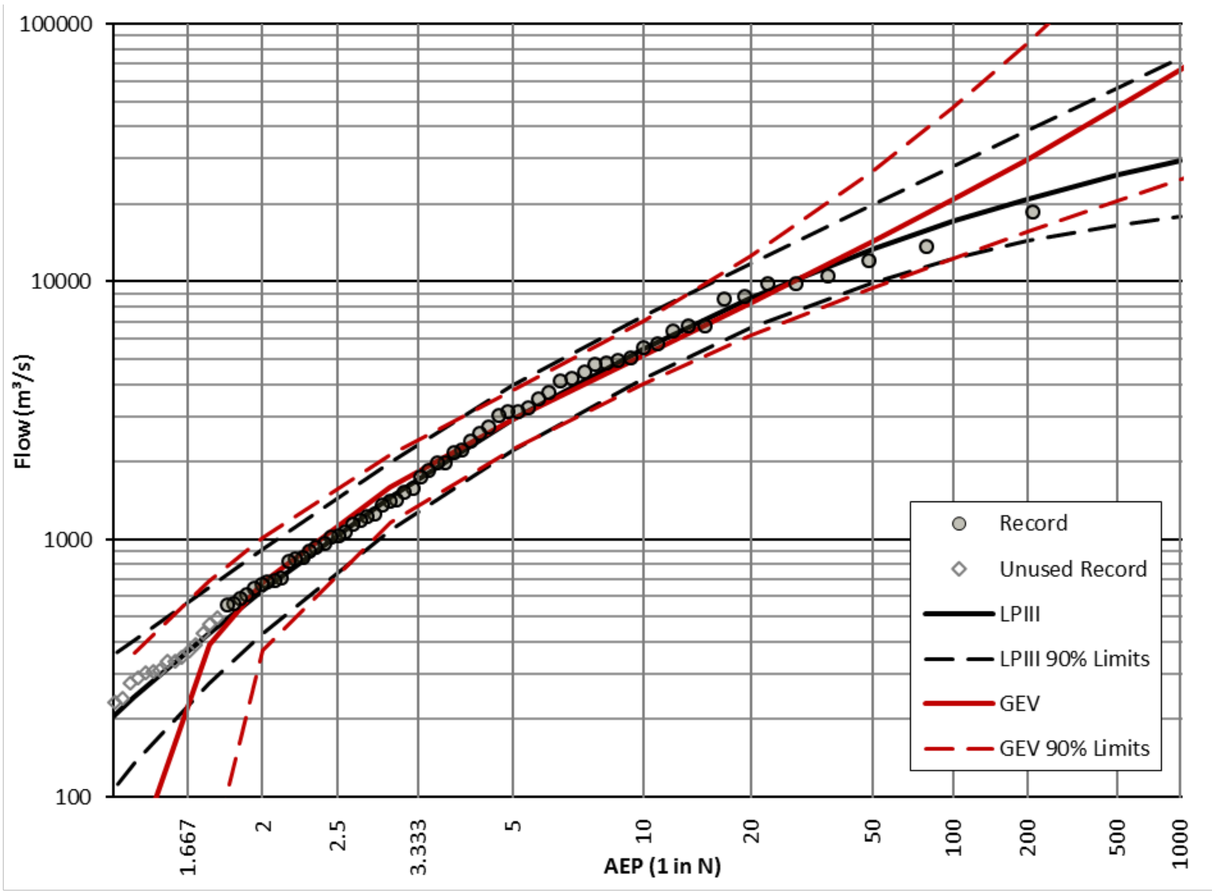


Figure 5-2 Flood frequency analysis at Savages Crossing 'No-dams conditions'

Independent at-site flood frequency assessment was undertaken for ten primary gauge locations considered to have reliable gauge and rating information. Sites included in this assessment for the peak flow analysis are:

- Stanley River at Woodford
- Brisbane River at Linville
- Brisbane River at Gregors Creek
- Lockyer Creek at Glenore Grove
- Bremer River at Walloon (combined with Rosewood)
- Warrill Creek at Amberley
- Purga Creek at Loamside
- Brisbane River at Savages Crossing (combined with Vernor and Lowood)
- Brisbane River at Mt Crosby Weir
- Brisbane River at Moggill

Sites included in the assessment of flood volume analysis are:

- Brisbane River at Linville
- Brisbane River at Gregors Creek

- Bremer River at Walloon (combined with Rosewood)

5.1.2 Limits of extrapolation

Large extrapolations of flood frequency analyses are not recommended. AR&R Book VI Section 1.2 recommends that the 1 in 100 AEP flood is the largest event that should be estimated by direct frequency analysis for important work, and the maximum flood that should be estimated by this means under any circumstances is the 1 in 500 AEP event.

Consistent with these recommendations, the preferred methodology is to use the results of the flood frequency analysis for assessment of moderate to large flood events (from 1 in 2 AEP up to 1 in 100 AEP) and comparison with stochastic rainfall assessment within this range. The stochastic rainfall and other methods such as PMP/PMF calculations should be used for extrapolation to rare and extreme flood frequencies.

5.1.3 Regional flood frequency analysis

A commonly encountered problem associated with estimating flood flows is estimating the flood flow of a given AEP at a location where the historical monitored information is inadequate for frequency analysis. Regional analysis techniques which draw upon (or transfer) better gauge records from nearby and/or hydrological similar sites can help improve or benchmark results derived by other methods. The application of regional frequency techniques may also result in improvements in terms of consistency (between the locations), robustness and reliability.

There are a number of regional flood frequency analysis (RFFA) techniques available for application. The recent AR&R Project 5 Stage 2 Report, (Rahman et al 2012), provides a summary of approaches that are available for application. Project 5 considered a number of RFFA methods which were then selected for detailed investigation. All RFFA methods use the results of at-site FFA as basic data.

A RFFA method then essentially consists of two principal steps:

1. Formation of regions: This involves formation of regions from the available streamflow gauging stations
2. Development of regional estimation models: This involves development of prediction equations to estimate flood quantiles, based on the results of at-site FFA within the region

In RFFA, formation of regions can be based on proximity in geographic or catchment attributes space. A region can be fixed, having a definite boundary or it can be formed in geographic or catchment attributes space with respect to the ungauged catchment of interest. AR&R Project 5 examined the applications of the following RFFA methods:

1. Probabilistic Rational Method (PRM)
2. Quantile Regression Technique (QRT)
3. Parameter Regression Technique (PRT)
4. Index Flood Method
5. Probabilistic Model (PM)/ Large Flood Regionalisation Model (LFRM)

The AR&R Project 5 report provided a summary of each of these techniques. The original intention was to apply the widely applied index flood method of Hosking and Wallis (1997), as it has proven to be suitable for a wide variety of applications, and then use the new ARR Project 5 Regional Analysis Tool which incorporates the Parameter Regression Technique (PRT) to validate the regional characteristics derived from the at-site frequency analysis.

A brief explanation of the Index Flood Method and Parameter Regression Technique is provided below, along with a description of what was finally adopted:

5.1.3.1 Index flood method

AR&R Project 5 summarised the Index Flood Method and reiterated that the key assumption in the method is that the distribution of floods at different sites within a homogeneous region is the same except for a site-specific scale, or index flood factor. Homogeneity with regard to the index flood relies on the concept that the standardised flood peaks from individual sites in the region follow a common probability distribution with identical parameter values. From all the methods examined in the AR&R Project 5, the Index Flood Method involves the strongest assumptions on homogeneity.

The limitation with this approach is the need to define so many characteristics to define the homogeneous regions. If a site is not classified appropriately, the estimation of flood quantiles can be affected significantly. Therefore it was decided not to proceed with the application of this approach.

5.1.3.2 ARR Project 5 regional analysis tool

Stage 1 and Stage 2 of AR&R Revision Project 5 have now been completed which has resulted in a comprehensive review of gauging stations across Australia (up to 676 gauged catchment have been used), and a comprehensive review of a number of regional flood frequency estimation (RFFE) approaches to determine the best approach for the determination of peak discharge estimates at ungauged or poorly gauged sites, or to be used as a comparison to at site Flood Frequency approaches, where only low quality site specific data is available.

The ARR Project 5 team developed a software application tool which automates the preferred ARR RFFE 2012 method, with the user required to input just the latitude and longitude (to derive design rainfall intensities, and to determine the Region of Influence) and the catchment area to the point where a flood quantile estimate is required.

Unfortunately Aurecon were unable to apply the Project 5 tool as the Beta version of the tool was withdrawn due to some problems being identified in its implementation. Therefore it was not possible to apply the tool to the current study, but this could be considered once it becomes available in the future.

5.1.3.3 Catchment weighted analysis

The flood frequency estimates of the primary sites (listed in Section 0) were reassessed using FLIKE's Bayesian inference method with Gaussian prior distributions to include weighted catchment skew and standard deviation parameters. The weighted catchment skew and standard deviation were also applied to secondary gauges to improve the overall consistency of the estimates.

These secondary sites included:

- Stanley River at Peachester
- Stanley River at Somerset Dam (combined with Silverton)
- Cooyar Creek at Damsite
- Brisbane River at Fulhamvale (combined with Plainlands and Watts Bridge)
- Brisbane River at Wivenhoe Dam (combined with Caboonbah and Middle Creek)
- Lockyer Creek at Helidon (combined with Russell Siding)
- Lockyer Creek at Gatton
- Lockyer Creek at Lyons Bridge/Rifle Range Road

- Brisbane River at Brisbane City Gauge

Overall 19 sites were assessed using flood frequency analysis techniques. Analysis of the Brisbane River catchment sites identified that the GEV distribution could usually provide a reasonable representation of the upper or lower tails of the gauge data, but in many cases when fitted to the full available range of data produced an upper tail that did not appear consistent with the expected frequency distribution, as typified in Figure 5-2 above where the GEV distribution appears to diverge significantly above 1 in 50 AEP. In most situations the Log-Pearson III distribution provided a good overall representation of the full data set, as well as being relatively consistent with the design event and Monte-Carlo simulation methodologies. Since the primary objective of the FFA is to provide a consistent assessment flows across the range of 1 in 2 to 1 in 100 AEP and to reconcile with other methods at and above this range, the Log-Pearson III distribution was adopted as the standard probability function for all gauges.

Regional analysis techniques that draw upon better gauge records from nearby and/or hydrological similar sites can help improve results derived at a location where the historical monitored information is inadequate for frequency analysis, or may result in improvements in terms of consistency (between the locations), robustness and reliability. An alternate approach loosely based on the Index Flood Method was adopted for the BRCFS analysis. The procedure adopted was as follows:

- An unbiased flood frequency assessment of a range of primary gauges (labelled in Figure 5-3) considered to have reliable record length and flow estimates was undertaken
- The frequency distribution parameters (skew and standard deviation) were analysed to determine if consistent catchment-wide values or trends could be identified
- These catchment values were then returned back into the site analyses for all sites as Gaussian prior distribution parameters used with the Bayesian inference method adopted by the FLIKE flood frequency analysis software

Skew estimates based on single 'at-site' analysis, especially those with short gauge records, can be sensitive to the presence of outliers in the upper or lower tail of the data, and it is well recognised that the accuracy can generally be improved by weighting the station skew with generalised values obtained from pooled information from other sites in the region. Review of the preliminary at-site analysis shown in Figure 5-3 identified a typical skew of around -0.8 with no discernible relationship to catchment area or other obvious catchment property. A catchment weighted skew with a mean of -0.8 and standard deviation of 0.1 was used for subsequent analysis at all gauge sites.

The catchment weighted skew had a strong influence on the curvature of the frequency curve at a number of gauges, however in all cases this influence promoted greater consistency with both other gauges and with alternative flow estimation techniques based on rainfall data. Based on this evidence, use of a catchment weighted skew parameter is strongly endorsed.

Use of a regional standard deviation is a recognised technique, but the relationship is potentially more complicated and a standardised methodology is not appropriate or available. After application of catchment weighted skew, a similar investigation of standard deviation identified a relatively weak correlation with catchment area. This correlation was slightly improved by applying a rainfall intensity weighting factor, as shown in Figure 5-4. The resulting relationship was applied to the frequency analysis using a relatively unrestrictive standard deviation of 0.12 to avoid unduly suppressing natural site characteristics. Introduction of catchment weighted standard deviation generally had a minor influence on the frequency curve. In cases where a strong influence was observed, comparison with other data suggests it has a positive benefit in promoting greater consistency with both other gauges and with alternative flow estimation techniques based on rainfall data. Catchment weighted standard deviation has therefore been used for this study, but it is recommended that further investigation is undertaken, such as comparison with AR&R Project 5 when this becomes available.

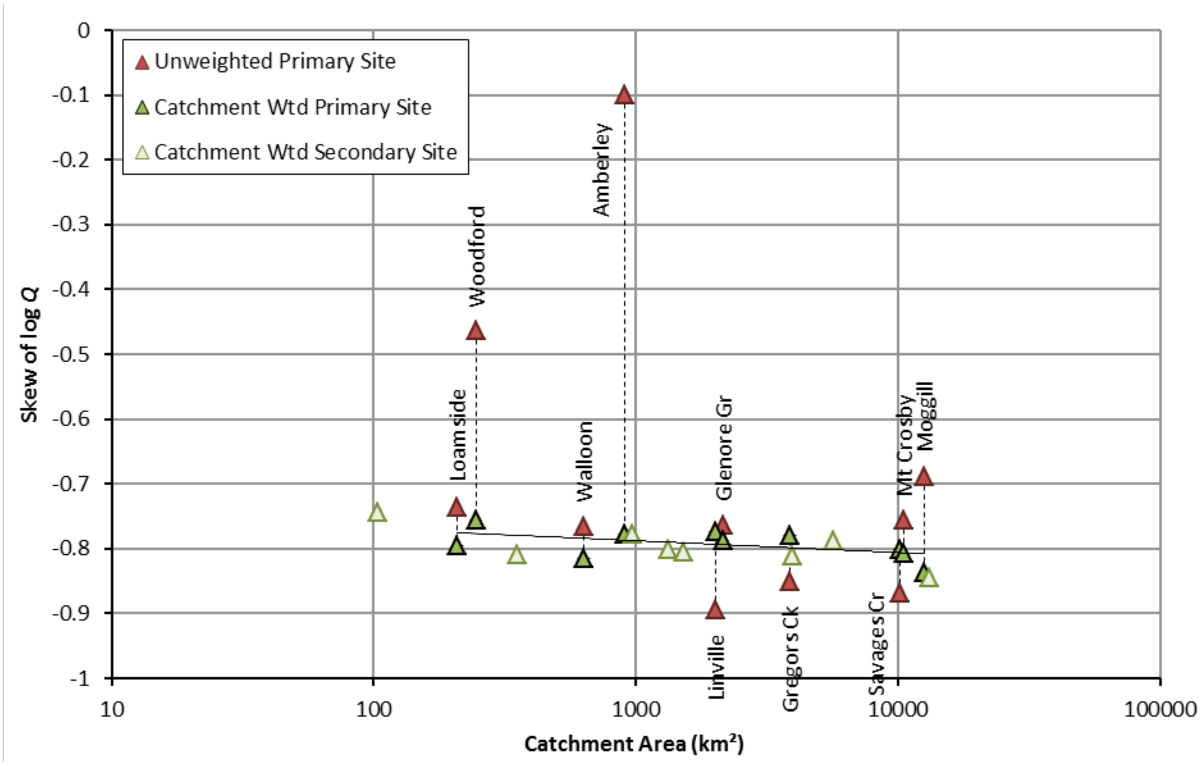


Figure 5-3 Skew of log Q before and after application of catchment weighted parameters

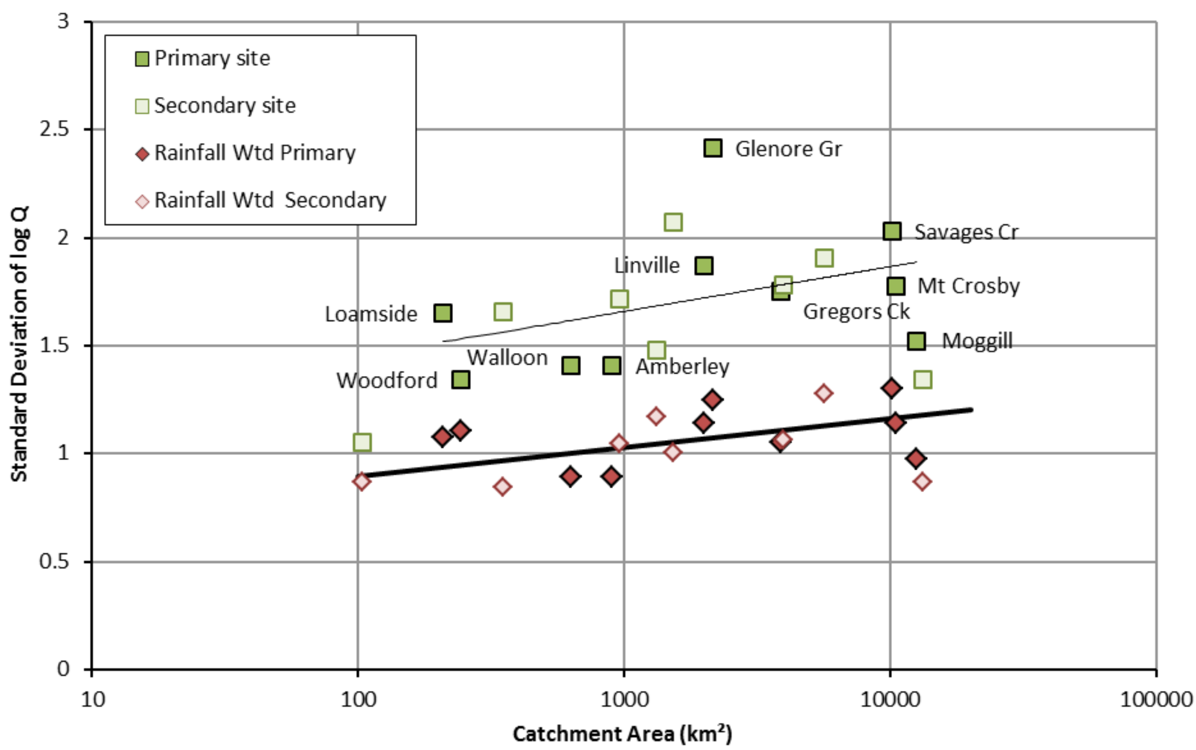


Figure 5-4 Skew of log Q before and after application of catchment weighted parameters

5.1.4 Volume frequency analysis

Annual flood volumes were also assessed using a similar approach to the peak flow frequency analysis. Due to the more stringent requirements for conducting a volume frequency analysis (continuous stream gauge unaffected by dam influence) the volume frequency analysis could only be conducted at three sites, being Linville, Gregors Creek and Walloon. Due to the difficulty of separating the flow volume associated with an individual independent rainfall event, the volume analysis assessed flow volume recorded over a fixed duration rather than complete event volumes. This approach is more consistent with Design Event and Monte-Carlo simulation assessments, which are based on simulation of rainfall bursts, but means that the analysed volumes include baseflow and potentially include flows from separate rainfall events.

Raw and catchment weighted volume frequency curves were produced for durations of 24, 48 and 72 hour. Catchment weighting was performed using the same skew and standard deviation parameters that were developed for the flood frequency analysis. An example of the fitted frequency curves to the derived annual series of 24 hour flood volume at Walloon is provided in Figure 5-5.

The volume data visually appears to follow similar trends to the flows and application of the catchment parameters appeared to produce a reasonable fit of the data that is more consistent with volume frequency relationship calculated using the Design Event approach. However it should be cautioned that there is insufficient data to fully confirm the validity of these parameters or to derive independent volume parameters.

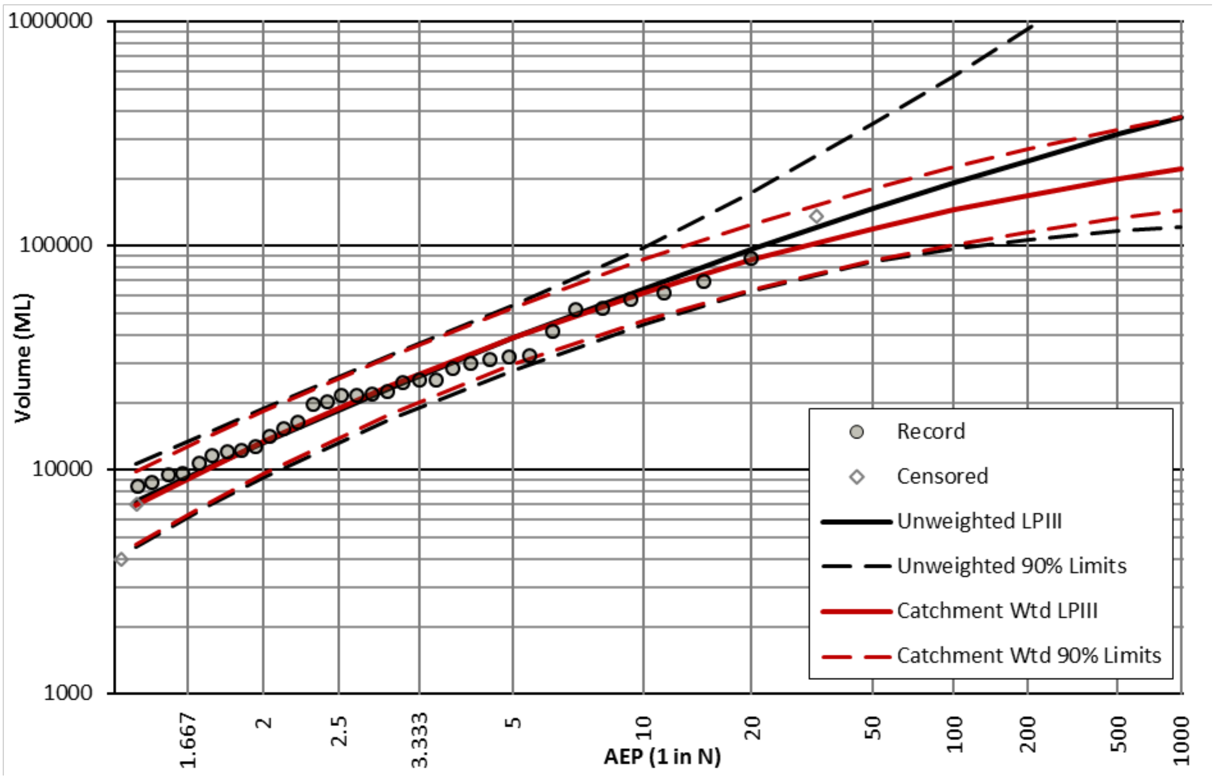


Figure 5-5 24 hour volume frequency analysis at Walloon

5.2 Assumptions and limitations

Flood frequency analysis is based on statistical analysis of historical flow data. This historical data was compiled from numerous sources with varying degrees of accuracy and uncertainty. Even at its best, flood frequency analysis is associated with significant uncertainty in the frequency estimates. Although the integrated assessment has been conducted based on best currently available data using current best-practice techniques a number of limitations have been identified. Specific limitations include but are not restricted to:

- **Length of record** – analysis is based on historical record that is assumed to represent a random, but statistically balanced sample. The longer the record, the greater the statistical reliability of the sample, however the reliability of the data may decrease for older records. Where appropriate, the data record has been extended using additional data
- **Accuracy of gauge readings** – the primary data source for estimating flows has been stream gauge records. Factors affecting the accuracy of these records include:
  - **Automatic vs. manual recording** – recent gauge records usually supply instrument recorded continuous level records. Older gauges were typically manually recorded at 24 hour intervals, sometimes with more frequent recordings during flood events. The recordings may therefore miss flood peaks, particularly for short, minor events or at night
  - **Reliability of records** – Automatic instrument recordings are subject to jamming or malfunction and may not correctly record flood peaks. Where possible records have been correlated to other data sources, however if no correlating data exists it is often impossible to identify where ‘missing’ floods should have occurred
  - **Flood peak records** – records of peak flood heights have been used at locations or in periods where continuous record is not available. Unless otherwise noted, it has been assumed that these records identify all significant floods above a certain magnitude within the period of record. This may not be correct as there is typically no record of what criteria were used to select specific events, nor whether these criteria are consistent across the period of record
  - **Conversion of flows to levels** – rating curves are used to covert recorded stream gauge levels into flows. The Rating Curve Review worked to improve confidence in the gauge ratings however there are limitations on this assessment
  - **Rating accuracy** – primary gauges have been assessed using independent hydraulic modelling to generate rating curves. These models were calibrated to available flow measurements and other data, but extrapolation of the rating is still dependent upon the accuracy of the model. Ratings at secondary gauges were developed using flow measurements where available and extrapolation using URBS model results. These ratings are considered to have limited accuracy and results should be used with caution. Lower Brisbane gauge ratings were developed using results from the DMT TUFLOW model. This model was calibrated using flows based on Seqwater URBS modelling that has been superseded by the BRCFS hydrology
  - **Rating consistency** – rating curves assume a consistent relationship between flow and level. A number of sites exhibit variability in the relationship, which may be due to short or long-term changes to the channel bed, vegetation or other factors. The rating curves have been developed to represent typical or average conditions. Flow estimates for specific events may therefore have a certain margin of error. Several sites also displayed shifts in the gauge datum. Where possible these have been corrected, however it is often not readily apparent whether the change is simply a translation of the gauge zero or represents a more significant change in the channel properties
  - **Rating sensitivity** – flowing across wide floodplains can exhibit significant sensitivity in the rating whereby small changes in level represent a large change in flow. These ratings may be particularly susceptible to changes in floodplain vegetation

- **URBS modelling** – flow records have in a number of cases been extended or modified using results of URBS hydrologic modelling. These models have been calibrated against available data, but flow estimates are dependent on the availability and accuracy of the calibration data

- **Homogeneity of data record** – analysis assumes that the available record represents a random sample taken from a homogeneous data set. Many factors can result in long-term changes to catchment characteristics including:
  - **Influence of Dams** – several gauge sites, primarily in the lower Brisbane River and Warrill Creek catchments are affected by dams. Where data records are available, URBS modelling has been undertaken to adjust the record to account for the dam influence. The accuracy of this adjustment is dependent upon the availability and accuracy of input data (eg rainfall, losses) and the ability of the model to represent with-dams and no-dams conditions. Where data records are not available for a specific event, a generic relationship between no-dams and with-dams conditions has been used to estimate no-dams conditions. This provides an estimate of typical dam impacts, but does not necessarily represent the exact impact
  - **Catchment and stream properties** – changes in land use such as urbanisation, changes in land use and construction of farm dams affect the catchment runoff characteristics. The area of catchment impacted by urbanisation is only 2.5% of the total Brisbane River but inclusion of urbanisation into the Seqwater URBS model and was found to increase peak flow rates by up to 2.5%, which is minor but not negligible. The assessment has assumed that the catchment and stream conditions are consistent throughout the period of record
  - **Climatic changes** – periodic shifts in weather patterns are noted to produce periods of drought or flood. Short records may be susceptible to significant bias if they span periods dominated by one extreme of the climactic cycle. Long-term climate change has also not been considered
  - **Catchment weighting parameters** – regional/catchment analysis has been used to improve consistency and confidence of the flood frequency analysis. Generally the results of the catchment weighting are consistent with expectations and other data, however parts of the methodology used are unique to the BRCFS assessment and so are not considered as a standard application of flood frequency analyses

6 Design event approach

6.1 Introduction

The Design Event Approach is a rainfall based assessment of flood hydrographs using runoff-routing models. It is a well-accepted procedure that is described in AR&R (EA, 2003). The primary input into the runoff-routing model is the design rainfall depth and associated temporal distribution of the rainfall. Loss rates, representing the antecedent condition of the catchment can also have a significant influence on the resultant design flood estimates.

Section 3.6.6.4 of the Brisbane River Catchment Flood Study (BRCFS) brief (DSDIP, 2013) provides details of the design event hydrologic modelling requirements. The brief indicates that hydrologic modelling is required for:

"design flood events with Annual Exceedance Probabilities (AEP) of 50%,20%, 10%, 5%, 2%, 1%, 0.5%, 0.2%, 0.1%, 0.05%, 0.01%, 0.001% as well as the Probable Maximum Flood (PMF). Hydrographs are to be produced for the full range of standard AR&R storm durations"

The brief also requires that design flood estimates be derived for two conditions at each nominated location, 'no-dams' and 'with-dams conditions'. The 'no-dam conditions' represents the condition of the catchment without the presence of the six major dams:

- Wivenhoe
- Somerset
- Perseverence
- Cressbrook Creek
- Lake Manchester
- Moogerah Dam

The level of urban development in the 'no-dams' conditions was not changed to reflect a 'pre-development' scenario, so it should be recognised that this scenario represents the behaviour of the catchment response simply without the presence of the dams.

6.2 Methodology

The process used to estimate design floods for both nominated conditions is based upon the calibrated hydrologic models in conjunction with inputs of the following:

- Design Rainfall Depths
  - Frequent to Large range – AR&R (EA, 2003): 1 in 2 AEP to 1 in 100 AEP
  - Large to Rare range – CRC-Forge (Hargraves, 2005): 1 in 100 AEP to 1in 2000 AEP
  - Transition from 1 in 2000 AEP to Probable Maximum Precipitation: AR&R Book VI (EA, 2003)
  - Probable Maximum Precipitation: GSDM, (BoM, 2003) and GTSMR (BoM, 2003)
- Areal Reduction Factors (Jordan et al, 2013)
- Design Rainfall Temporal Patterns (EA, 1987, and BoM, 2003)
- Rainfall Loss Rates (Seqwater, 2013)
- Base-flow (Murphy et al, 2011)

Figure 6-1 provides an indication of the event classifications for which the various sources of design rainfall are applicable.

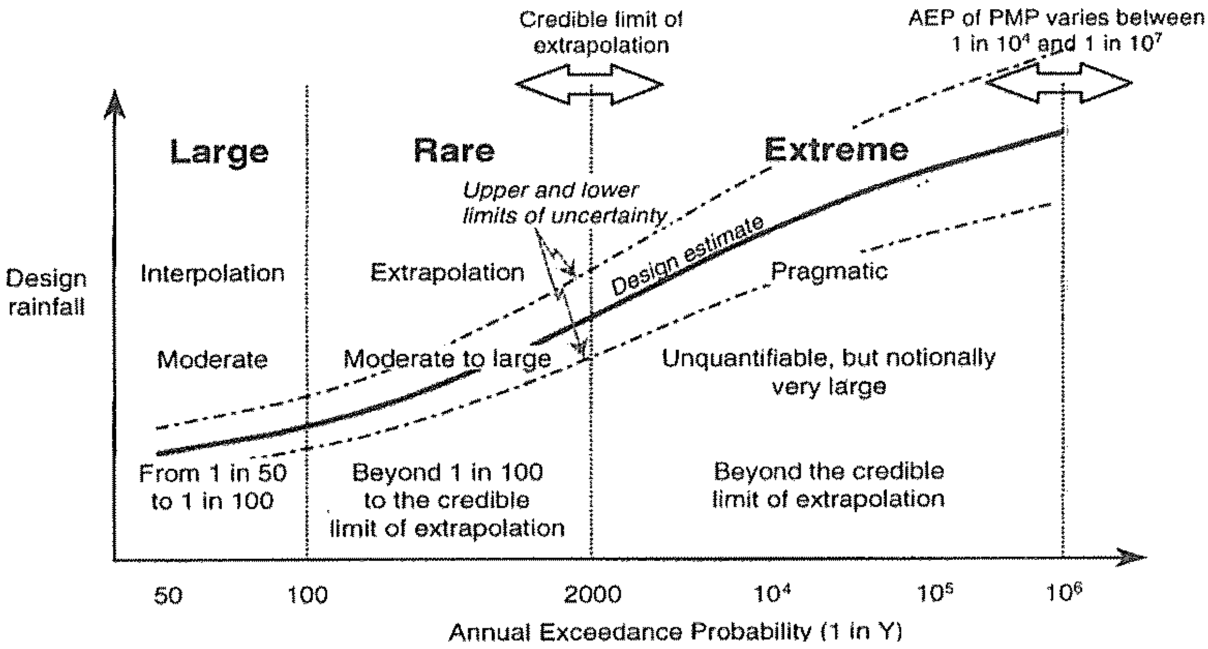


Figure 6-1 Flood event classifications from AR&R (EA, 2003)

6.2.1 Hydrologic models

6.2.1.1 No-dams condition

The models developed during the recalibration process and discussed in the Aurecon Team's *Hydrologic Model Recalibration Report* (Aurecon, 2014 Appendix C) have been used as the basis for the design event modelling. These models have been modified to remove all reference to the dams, including storage details and reduced reach length factors for drowned reaches. The catchment data has also been adjusted to remove the effect of impervious area associated with the reservoirs.

The design event modelling for the no-dams conditions is not considered a pre-development scenario; it represents the current catchment conditions with the dams removed. Similarly, it does not represent a future development scenario.

The analysis has been conducted using the URBS model version 5.70 Beta. This analysis has been undertaken using the URBS model in a standalone manner, outside of the Delft-FEWS platform.

6.2.1.2 With-dams conditions

The models developed during the recalibration process and discussed in the Aurecon Team's *Hydrologic Model Recalibration Report* (dated 10 July 2014) have been used as the basis for the design event modelling. These models have been used in conjunction with the dam operations module to produce design flood estimates for the with-dams conditions.

As with the 'no-dams condition' modelling, this analysis has been undertaken using the URBS model in a standalone manner, outside of the Delft-FEWS platform. Somerset and Wivenhoe inflows were taken from the Stanley and Upper Brisbane URBS models, processed through the Dam Operations Module, and Wivenhoe outflows were input back into the Lower Brisbane model.

6.2.2 Design rainfall estimates

The process adopted in developing design event IFD values is presented in the following sections. The key steps in this process are identified as follows in Figure 6-2:

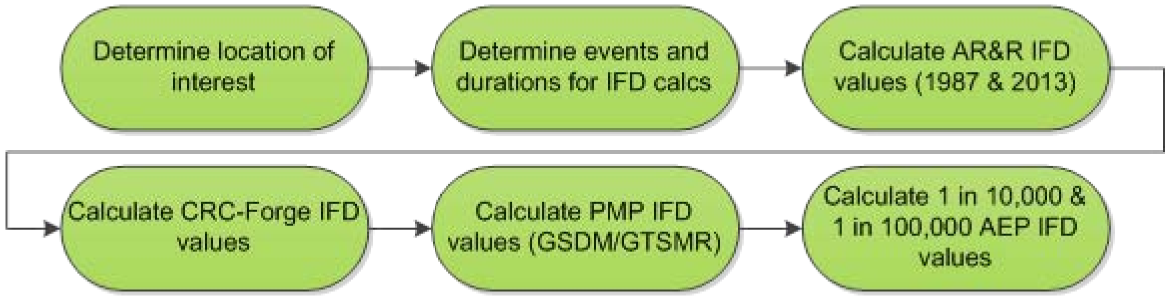


Figure 6-2 Process for deriving design IFD estimates

Design rainfall estimates have been derived using procedures outlined in AR&R (EA, 2003) for the frequent to large range (AEP up to 1 in 100). CRC-Forge techniques have been used to derive design rainfall estimates in the large to rare range (1 in 100 < AEP < 1 in 2000). Due to observed inconsistencies between the 1 in 100 AEP intensities derived using the AR&R and CRC-Forge methods, rainfall intensities in the range 1 in 100 < AEP < 1 in 2000 were derived by applying the CRC-Forge growth factors to the BoM IFD data to ensure a monotonically increasing rainfall frequency curve. Extreme design rainfalls have been estimated using the procedures outlined in the generalised methods, Generalised Short Duration Method (GSDM) and Generalised Tropical Storm Method Revised (GTSMR), (BoM, 2003).

Spatially varying IFD values have been calculated for each of the 534 URBS subareas. A summary of the adopted methods is presented in Table 6-1. This methodology has been applied twice at each location, once for 1987 AR&R IFD values and once for 2013 AR&R IFD values. The 2013 AR&R IFD values have been used as the basis for the final design flood estimates.

Location specific PMP and ARF values have been calculated. For catchments with an area of less than 1000 km^2 both the GSDM and GTSMR PMP methods were applied.

Table 6-1 Adopted IFD calculation methods

Event duration (hrs)	1987 AR&R (50% to 1% AEP)	2013 AR&R (50% to 1% AEP)	CRC-Forge* (0.5% to 0.05% AEP)	Extreme events (0.01% and 0.001% AEP)	PMP
1	Y	Y	Factored	AR&R Book VI Section 3.6.3	GSDM
2	Y	Y	Factored	AR&R Book VI Section 3.6.3	GSDM
3	Y	Y	Factored	AR&R Book VI Section 3.6.3	GSDM
6	Y	Y	Factored	AR&R Book VI Section 3.6.3	GSDM
12	Y	Y	Factored	AR&R Book VI Section 3.6.3	Interpolated
18	Interpolated	Interpolated	Interpolated	AR&R Book VI Section 3.6.3	Interpolated
24	Y	Y	Y	AR&R Book VI Section 3.6.3	GTSMR
36	Interpolated	Interpolated	Interpolated	AR&R Book VI Section 3.6.3	GTSMR
48	Y	Y	Y	AR&R Book VI Section 3.6.3	GTSMR
72	Y	Y	Y	AR&R Book VI Section 3.6.3	GTSMR
96	Extrapolated	Y	Y	AR&R Book VI Section 3.6.3	GTSMR
120	Extrapolated	Y	Y	AR&R Book VI Section 3.6.3	GTSMR
144	Extrapolated	Y	Extrapolated	AR&R Book VI Section 3.6.3	Extrapolated
168	Extrapolated	Y	Extrapolated	AR&R Book VI Section 3.6.3	Extrapolated

Note: * The CRC-Forge growth factors were applied to the BoM IFD data to ensure a monotonically increasing rainfall frequency curve.

6.2.3 Areal reduction factors

Areal Reduction Factors (ARF) are calculated as per AR&R Project 2 (Jordan et al, 2013) for each of the nominated locations of interest. The areal reduction factors were applied to IFD and CRC-Forge point design rainfall estimates only, as the PMP estimates are already areal estimates.

6.2.4 Temporal patterns

Design temporal patterns have been sourced from AR&R Volume 2, (EA, 1987) and the GSDM and GTSMR (BoM, 2003). For GSDM events (1, 2, 3 and 6 hour durations) the temporal pattern has been applied as per Section 5 of the GTSMR Guidebook (BoM, 2003). This has only been applied in catchments smaller than 1000 km^2. For GTSMR events the Coastal AVM temporal pattern for the relevant standard area at each location of interest has been applied as per Section 4 of the GTSMR Guidebook (BoM, 2003).

For the 12 hour event, both the GSDM temporal pattern and the 24 hour GTSMR temporal pattern, with the time increments halved, have been applied. The worst case results have been adopted. This process has only been applied in catchments smaller than 1000 km^2.

A summary of the temporal patterns applied at each location is provided in Table 6-2.

Table 6-2 PMP temporal patterns

Location	GSDM temporal pattern (1, 2, 3 & 6hr)	12hr temporal pattern	GTSMR temporal pattern (24, 36, 48 & 72hr)
Peachester	Y	GSDM + Standard Area 100	Standard Area 100
Somerset Dam	N/A	N/A	Standard Area 1,000
Woodford	Y	GSDM + Standard Area 100	Standard Area 100
Linville	N/A	N/A	Standard Area 2,500
Gregors Creek	N/A	N/A	Standard Area 5,000
Fulham Vale	N/A	N/A	Standard Area 5,000
Tinton	Y	GSDM + Standard Area 500	Standard Area 500
Middle Creek	N/A	N/A	Standard Area 5,000
Wivenhoe	N/A	N/A	Standard Area 5,000
Helidon	Y	GSDM + Standard Area 500	Standard Area 500
Gatton	N/A	N/A	Standard Area 1,000
Glenore Grove	N/A	N/A	Standard Area 2,500
Walloon	Y	GSDM + Standard Area 500	Standard Area 500
Kalbar Weir	Y	GSDM + Standard Area 500	Standard Area 500
Amberley	Y	GSDM + Standard Area 1000	Standard Area 1,000
Loamside	Y	GSDM + Standard Area 100	Standard Area 100
Savages Crossing	N/A	N/A	Standard Area 10,000
Mt Crosby Weir	N/A	N/A	Standard Area 10,000
Ipswich	N/A	N/A	Standard Area 2,500
Moggill	N/A	N/A	Standard Area 10,000
Centenary Bridge	N/A	N/A	Standard Area 10,000
Brisbane	N/A	N/A	Standard Area 10,000

Temporal patterns for the intermediate range of flood magnitudes between the 1 in 100 AEP and the PMP event have been interpolated. The interpolation has been conducted using normalised curves of the cumulative temporal patterns. Linear interpolation has been used to estimate the incremental values for each of the respective flood magnitudes. This approach has the advantage of avoiding anomalies between flood magnitudes in the large to rare range (especially the 1 in 100, 1 in 200 and 1 in 500 events).

6.2.5 Rainfall loss rates

6.2.5.1 Initial loss

Adopted initial losses have been applied as presented in Table 6-3. These values were based on extensive sensitivity analyses, comparisons with FFA results and the constraint that values need to be physically realistic. The losses were applied to different sub-catchments as indicated.

Table 6-3 Adopted initial loss values

Event (AEP 1 in N)	Stanley & Bremer Initial loss (mm)	Upper & Lower Brisbane Initial loss (mm)	Lockyer Creek Initial loss (mm)
2	40	50	60
5	32	40	48
10	24	30	36
20	16	20	24
50	8	10	12
≥ 100	0	0	0

6.2.5.2 Continuing loss

Continuing loss values were obtained during the recalibration process (refer to the Aurecon Team's *Hydrologic Model Recalibration Report* (Aurecon, 2014, refer Appendix C). The median continuing loss values are as follows:

- Stanley River: 2.5 mm/hr
- Upper Brisbane River: 2.4 mm/hr
- Lockyer Creek: 2.8 mm/hr
- Bremer River: 1.3 mm/hr
- Warrill Creek: 2.0 mm/hr
- Purga Creek: 2.0 mm/hr
- Lower Brisbane River: 2.2 mm/hr

The value for the Lower Brisbane River represents the average value applied to the upstream sub-catchments. It is similar to the typical continuing loss of 2.1 mm/hr calculated for the Lower Brisbane River during the recalibration process. These values were used as the starting point for the reconciliation process. The adopted continuing loss rates were then derived from the reconciliation process with the FFA estimates at various key locations. These values provided the best overall agreement between the two sets of results. Table 6-4 provides a summary of the adopted continuing loss values for the range of flood magnitudes.

Table 6-4 Adopted continuing loss values

Event (AEP 1in N)	Stanley & Bremer Continuing loss (mm/hour)	Lower & Upper Brisbane Continuing loss (mm/hour)	Lockyer Creek Continuing loss (mm/hour)
2	2.0	2.5	3.0
5	1.6	2.0	2.4
10	1.2	1.5	1.8
20	0.8	1.0	1.2
50	0.4	0.5	0.6
≥ 100	0.4	0.5	0.6

The adopted continuing loss rates therefore should be considered more of a calibration factor that is used in conjunction with the model routing parameters to achieve an acceptable consistency, rather than a physically based rainfall loss rate.

6.2.6 Base-flow

Baseflow is the portion of stream flow that comes from the sum of deep subsurface flow and delayed shallow subsurface flow. Generally it represents between 5 to 15% of the peak surface runoff in a hydrograph in the Brisbane River catchment. Its consideration and inclusion is important to adequately define the flood volume, especially in respect to the operation of the flood mitigation dams.

In accordance with the URBS model review presented in Aurecon's *Hydrologic Model Calibration and Validation Review Report* (Aurecon 2015, refer Appendix B) a Baseflow Volume Factor is applied according to the magnitude of the design rainfall event. The adopted Baseflow Volume Factors in Table 6-5 have been sourced from AR&R Revision Project 7.

Table 6-5 Adopted baseflow volume factors

Rainfall AEP (1 in N)	Baseflow Volume Factor from AR&R Project 7
2	1.6
5	1.2
10	1.0
20	0.8
50	0.7
100	0.6
>100	Extrapolated

The base flow volume factor is applied to design events to limit the base flow contribution, especially for the rare to extreme flood magnitude range (beyond 1 in 100 AEP).

The base flow volume factor (BFVF) as per AR&R Project 7 was included in the AR&R Design Event Approach (DEA). During the calibration phase baseflow parameters were calibrated for each location under investigation. These parameters are based on the URBS baseflow model:

$$BF(i) = BR \times BF(i-1) + BC \times QR^{BM}$$

Where BF(i) = Baseflow at current time step

BF(i-1) = Baseflow at previous time step

BR = Baseflow recession constant (daily value)

BC = Baseflow constant (daily value)

QR = Quick flow component of hydrograph

BM = Baseflow exponent

The BR and BC are daily time-step parameters and URBS makes internal adjustments to account for the model time step. The BM exponent determines whether linear or non-linear baseflow routing is to be adopted. For the Brisbane River catchment, BM was assumed to be 1, ie a linear model.

It can be shown that BFVF (the ratio of baseflow to quick runoff) = BC/(1 – BR), when BM = 1.

The URBS' RAINURBS module was modified to include the BFVF parameter for the 1 in 10 AEP event as provided in Table 1 of the AR&R Project 7 report. For the Brisbane catchment this was set to 0.15. Adjustment was made to this value based on the design ARI under investigation using the factors in Table 6-5 as provided in the AR&R Project 7 report. A power curve was fitted to these adjustment factors to extrapolate these factors for AEPs beyond 1 in 100.

Sensitivity testing to baseflow factors was conducted and is reported in the *Hydrologic Model Calibration and Validation Review Report* (Aurecon 2015, refer Appendix B), with the base flow factors found to have comparatively little impact on the overall peak flow, flood volume and hydrograph shape. Changes to the peak flow were less than 2.5%.

6.2.7 'No-Dams Conditions' Design Event Approach estimates

An example of the hydrographs derived from the application of the design event approach using the 2013 IFD is shown in Figure 6-3 for the Brisbane River at Linville. Peak flow estimates for a range of flood magnitudes have been derived for each location. Figure 6-4 and Figure 6-5 show the derived flood frequency curves based upon the DEA for sites located on the Brisbane River and tributaries.

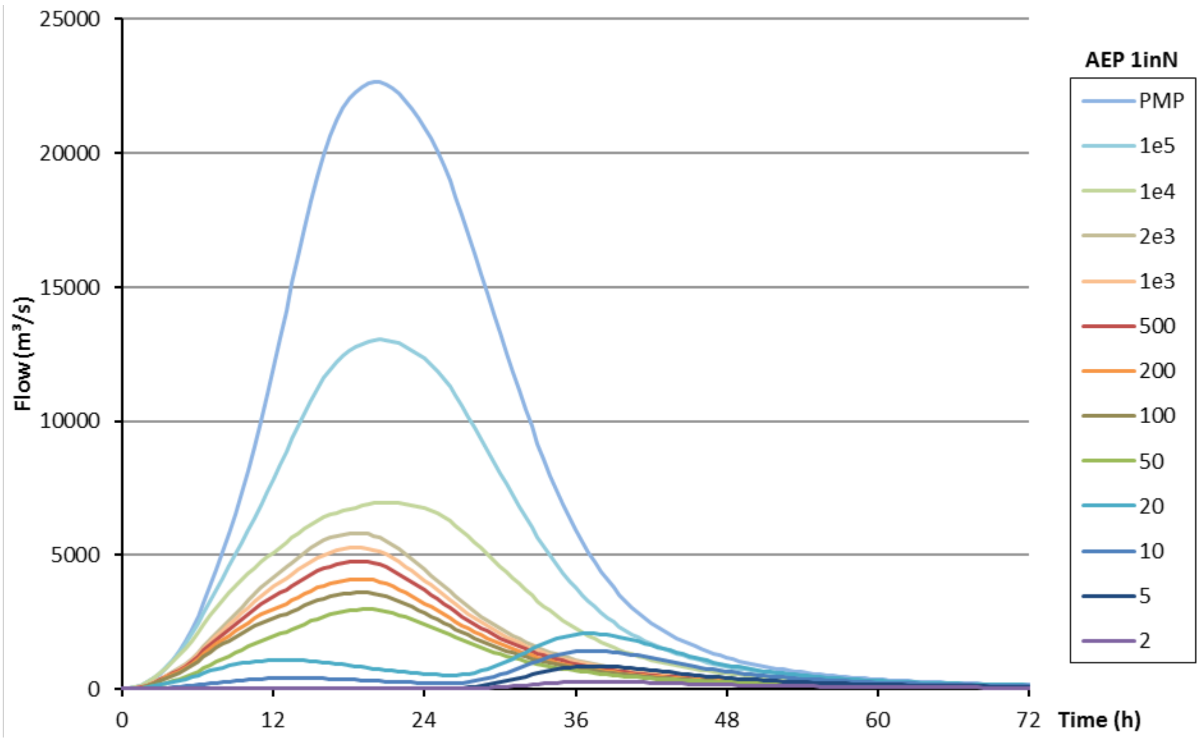


Figure 6-3 Brisbane River at Linville 2013 IFD Zone 3 design event hydrographs

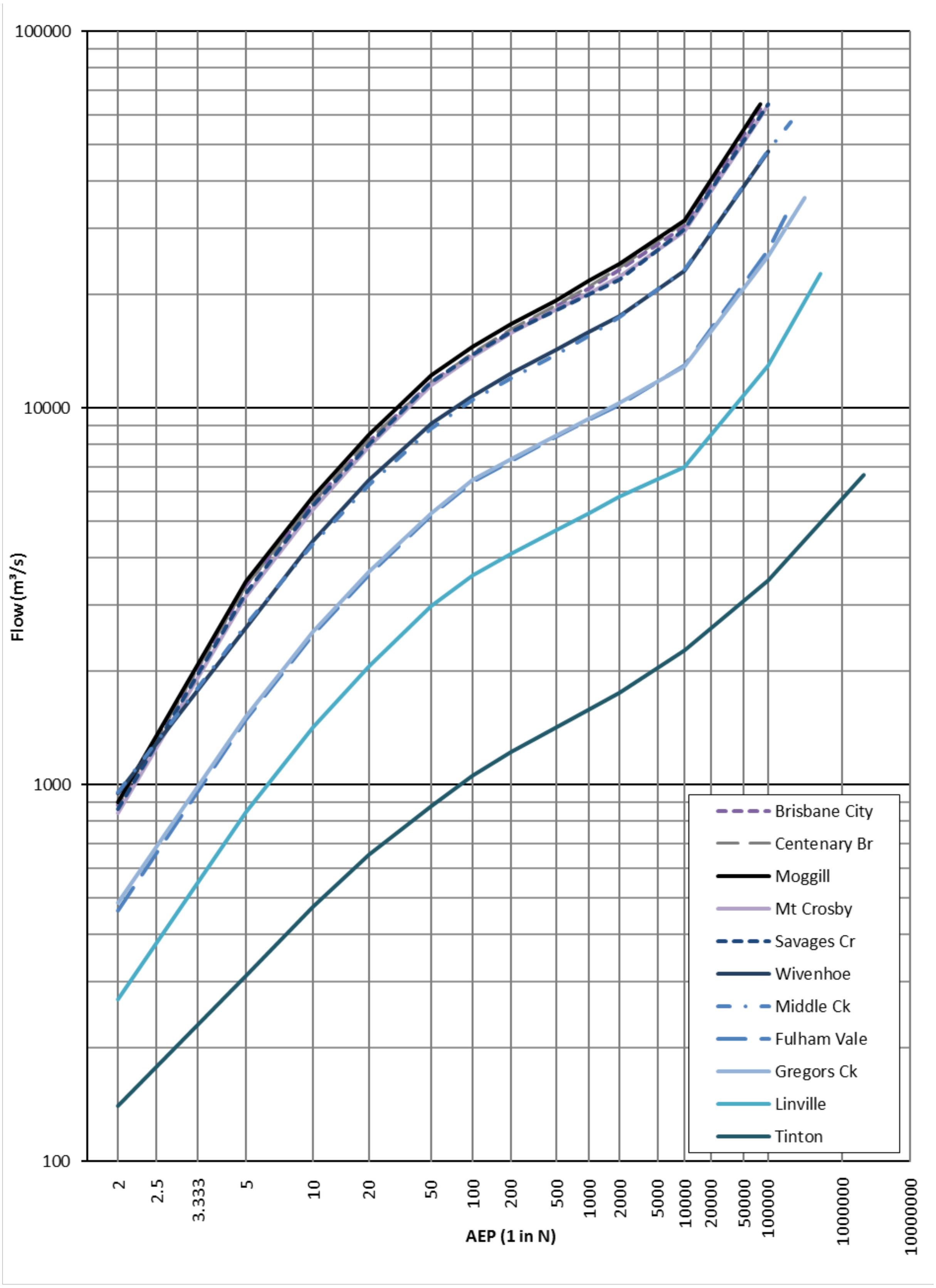


Figure 6-4 Peak flow rates 2013 IFD design events – Brisbane River catchment sites

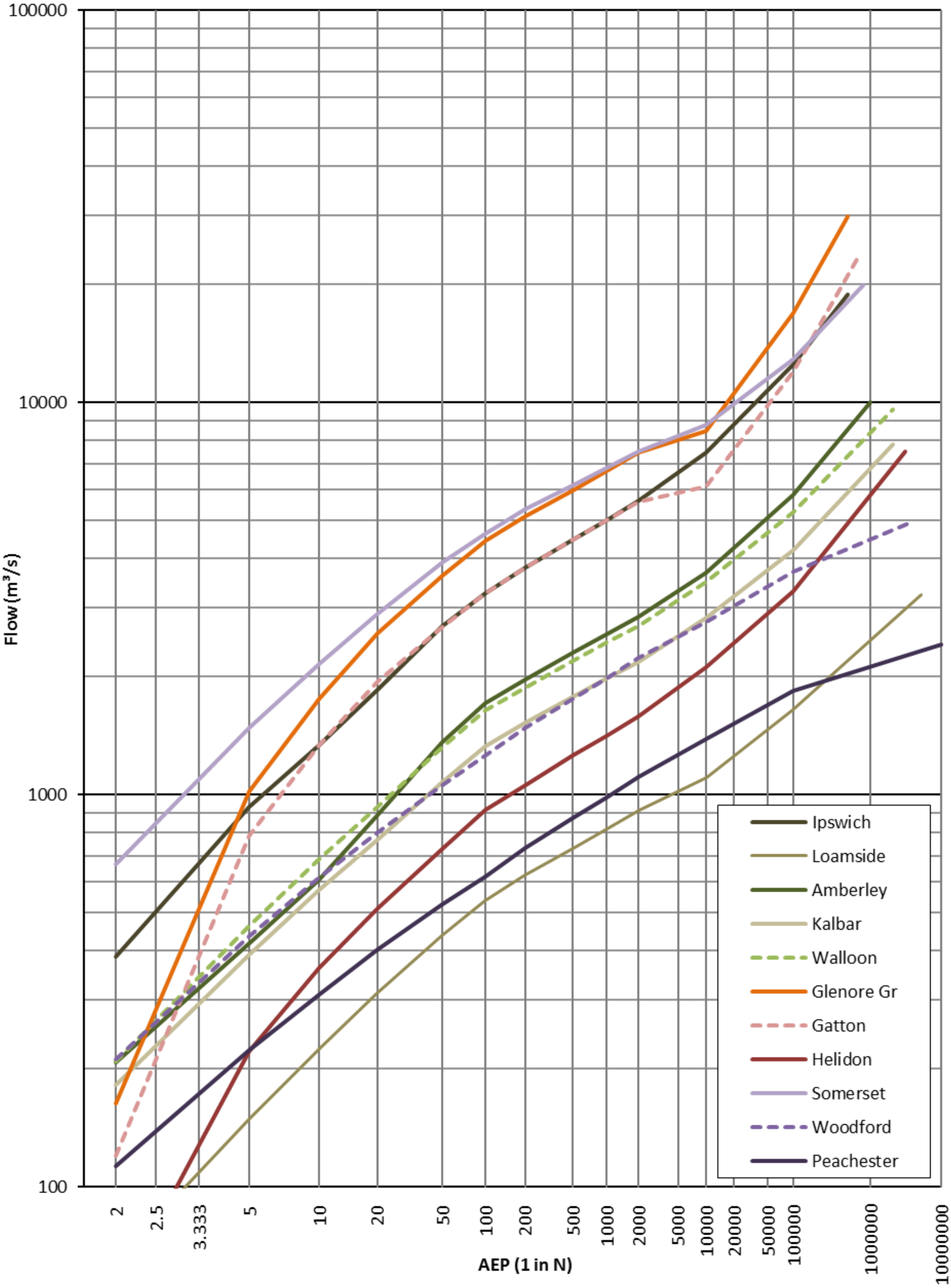


Figure 6-5 Peak flow rates 2013 IFD design events – tributary catchment sites

In addition to the estimation of flow hydrographs and peak flows, flood volumes were also estimated using this approach and these volumes have been compared to the results of the FFA and MCS methods.

Design flood estimates for each location were determined by running the complete range of storm durations and flood magnitudes. The peak instantaneous flow for each flood magnitude, its associated storm duration and flood volume were included in the summary tables and associated hydrograph plots.

Design event flood frequency curves for the Brisbane River catchments generally show similar a trend with flow magnitude generally increasing with distance downstream. The tributary design frequency curves tend to show different trends for frequencies greater than 1 in 100 AEP with the Lockyer Creek catchments in particular exhibiting a more rapid increase in flow magnitude with event rarity. This characteristic can be related to rainfall intensities and adopted catchment losses. Floodplain storage attenuation effects are evident in the Brisbane River downstream of Wivenhoe with the flood peak of larger floods tending to decrease slightly between Savages Crossing and Mt Crosby and from Moggill to Brisbane City. These characteristics are generally consistent with known behaviour of the Brisbane River catchment.

For minor events (≤ 1 in 5 AEP) the critical duration is at least 36 hours at all sites as rainfall losses removing a significant proportion of the volume of short-duration rainfall events. For large to extreme events (≥ 1 in 50 AEP) the critical duration typically increases with catchment area, consistent with longer time for runoff to concentrate from the catchment.

Design Event analysis is dependent on parameters that are based on normalised and/or regionalised characteristics, such as IFD and temporal patterns, or on values that must be estimated using knowledge of the catchment behaviour or else assumed from typical values. The currency of and appropriateness of a number of these parameters are currently being assessed as part of the review of AR&R. At the time of the BRCFS assessment some of the updates had been released (eg IFD) however other components were not available (eg temporal patterns).

Testing was performed to investigate the sensitivity of the flow and volume predictions to several of the main parameters, and identified that:

- Adopting 1987 or 2013 IFD data results in a difference in peak flow of around ±10% across the full range of frequencies covered by the AR&R IFD tables (1 in 2 to 1 in 100 AEP). The 2013 IFD data typically increases flows in the Stanley and Bremer River systems, but decreases flows in Upper Brisbane River and Lockyer Creek. Mid- and Lower Brisbane River flows are relatively unchanged
- Rainfall losses have significant influence on high AEP flows due to the lower intensity/depth. There is relatively little influence on low AEP flows due to the higher intensity rainfall and typically lower losses adopted
- Temporal patterns have significant impact across the full range of flood frequencies. Using Zone 2 temporal patterns instead of Zone 3 results in flow increases of 20% to 80%

High AEP flows are strongly affected by assumed losses and temporal patterns. Flow estimates therefore have low reliability unless confirmed against other sources. Low AEP flows are not as sensitive to losses but are still affected by IFD and temporal pattern. Due to this uncertainty, confirmation using independent assessment methods (flood frequency analysis and Monte-Carlo simulation) is recommended.

6.2.8 'With-Dams Conditions' Design Event Approach estimates

Design Event Approach modelling of 'with-dams conditions' was undertaken to assess the influence of Somerset and Wivenhoe, as well as the four other major dams considered by the URBS models (Cressbrook Creek, Perseverance, Lake Manchester and Moogerah) on flows downstream of those dams.

The 'with-dams conditions' modelling assumed starting dam water levels are at Full Supply Level (FSL), which is a common assumption for DEA flood studies and generally conservative in terms of downstream flows produced. Sensitivity testing of the extreme limit of the mitigation of Wivenhoe Dam releases on downstream Brisbane River flows, being no release from Wivenhoe, was also undertaken. It is acknowledged that the latter scenario is unlikely to be realistic, particularly for larger events, but represents an absolute maximum influence independent of any future changes to dam capacity or operating procedure. It should be recognised that given the current configuration of the dams, there is a finite capacity to limit the release of floodwaters downstream. For example, the current capacity of Wivenhoe Dam is insufficient to capture all of the 1 in 100 AEP flood volume. Therefore, whilst this scenario testing provides an indication of the minimum bound scenario, it is not feasible to operate the dams in this manner.

No additional sensitivity testing of starting water level was undertaken using the DEA due to the significance of other factors such as temporal and spatial variability that are not considered by Design Event type modelling and better represented by the Monte-Carlo approach.

The 'with-dams conditions' design event frequency curves are provided in the *Design event approach* report (Aurecon, 2014, refer to Appendix G). Figure 6-6 shows the reduction in peak discharge attributable to the dams starting at Full Supply Volume (FSV). The main Brisbane River dams provide significant attenuation in the mid-Brisbane River (Wivenhoe to Mt Crosby) for minor events, but become less effective as the magnitude of the flood increases. The attenuation relationship at Wivenhoe release tends to be somewhat erratic, particularly above 1 in 500 AEP when breaching of the fuse plug spillways and then overtopping of the dam crest cause abrupt change in the storage-discharge relationship. Attenuation in the lower Brisbane River (Moggill to Brisbane) follows a similar trend to the mid-river sites, but with a smaller attenuation amount due to the additional inflows from the Bremer River. Moogerah Dam tends reduce peak flows at Amberley by around 20% across the full range of events, while at Ipswich the dam influence on Bremer River flows is minor at high AEP increasing to around 10% at low AEP.

The dam influence on peak flow volumes tends to show similar trends to the peak flows. The reduction in 24 hour volume is generally similar order of magnitude as the peak flow since the peak flows in the lower Brisbane catchments are caused by long-duration events with a prolonged peak. The reduction in 72 hour volume is slightly lower, since the dams retard but do not permanently store flow.

Peak flows in the Brisbane River downstream of Wivenhoe with no release from Wivenhoe Dam are shown in Figure 6-6. Due to rainfall IFD and loss characteristics, the Lockyer Creek catchment tends to have significantly lower flows for frequent events than the upper Brisbane and Stanley Rivers, so removing the contribution of these catchments from the lower Brisbane River results in a significant reduction (up to 90% at Savages Crossing for the 1 in 2 AEP event) that gradually decreases to around 60% at PMF. Downstream of the Bremer River confluence the exclusion of Wivenhoe Dam release reduces peak flows by 50% to 60%.

It is cautioned that these reductions are based on a uniform catchment-wide Design Event temporal pattern, and in reality the actual reduction for any particular event will be dependent on the spatial and temporal distribution of rainfall across the catchment. The DEA results should be indicative but do not necessarily represent median or typical values.

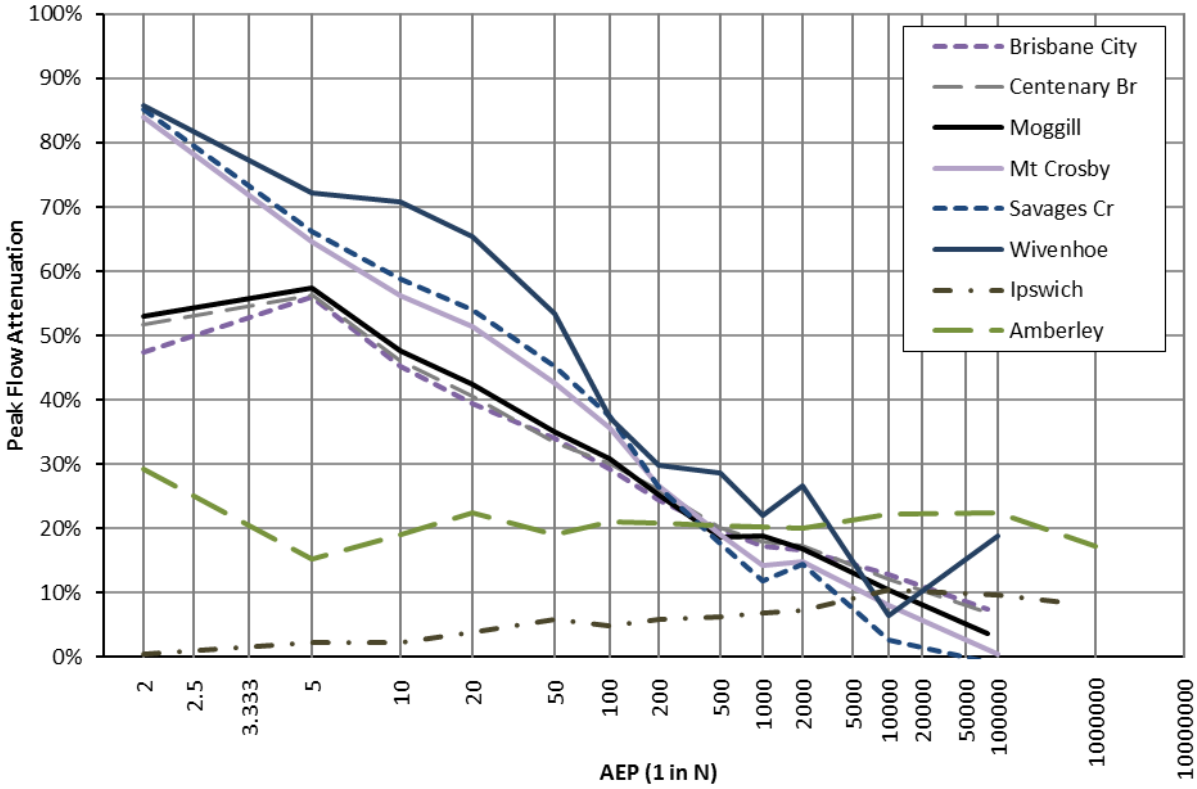


Figure 6-6 Peak flow attenuation assuming all dams at FSV

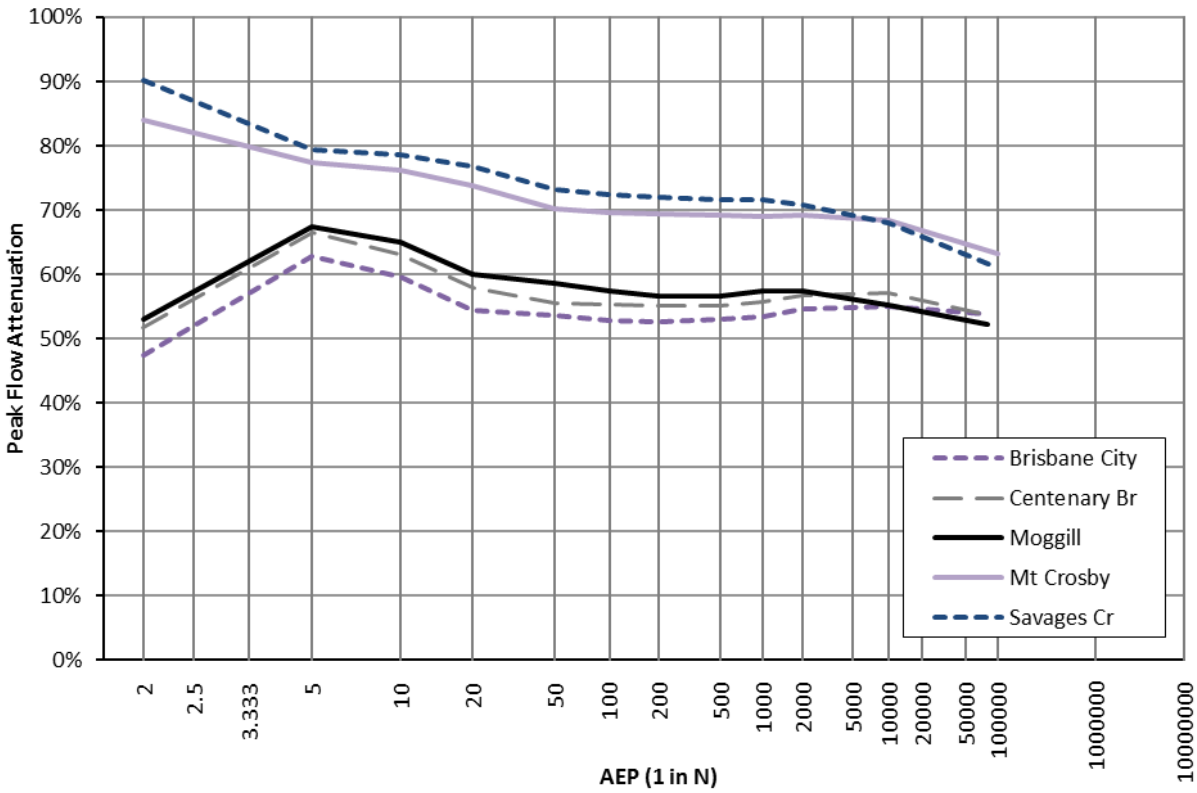


Figure 6-7 Peak flow attenuation assuming no release from Wivenhoe

6.3 Assumptions

6.3.1 AEP neutrality

A limitation with the application of the Design Event Approach is the assumption that the process for transforming design rainfall to design flood estimates is AEP neutral, that is, no bias is introduced which results in the design flood estimates having a different AEP to that of the original design rainfall. Factors which can contribute to this include the antecedent conditions such as rainfall loss rates and initial reservoir levels; adopted routing parameters; and assumed spatial and temporal variation of rainfall over the catchment.

Loss rates applied in the DEA have been adjusted to achieve reconciliation between the DEA estimates and the FFA and MCS estimates. This has resulted in the adoption of loss rates that vary with flood magnitude. This trend is not necessarily supported by observations of the calibration event adopted loss rates.

6.3.2 Uniformly spaced AEP

The DEA assumes that all locations within the catchment are subject to a uniform AEP rainfall event. This does not necessarily occur and realistic variations in spatial distributions of rainfall AEP suggest that this assumption may tend to result in an overestimation of the resultant flood magnitude, particularly for more frequently occurring events.

6.3.3 Uniform temporal distribution

Under the DEA, all locations within the catchment are assigned a uniform temporal distribution. This does not account for possible storm movement and tends to limit the timing of the contribution of runoff response for different parts of the catchment. This can limit the representativeness of the DEA, especially for locations downstream of Wivenhoe Dam, which are influenced by downstream tributaries such as Lockyer Creek and the Bremer River. The natural variation of timing of runoff from these catchments due to storm movement is not necessarily captured by the use of a uniform temporal pattern over the entire catchment.

6.3.4 Initial reservoir levels

Under the DEA for the with-dams conditions, the initial reservoir levels were assumed to be Full Supply Level (FSL) of all dams. This is a common assumption that is adopted for flood studies, but it should be recognised that this assumption may distort estimates of flood levels downstream of the reservoirs, because this assumption is generally conservative, especially for frequently occurring flood events.

6.4 Limitations

6.4.1 Small scale catchments

The Design Event Approach was found to be suited to small scale catchments, as evidenced by the comparison of the estimates derived from the DEA and the other techniques such as flood frequency analysis (FFA). In small scale catchments the influence of spatial and temporal variability of rainfall is limited and therefore does not tend to distort the outcome of the assessment. The nominated locations situated in the headwaters of the catchment and which are not influenced by the presence of the mitigation dams indicate that the DEA methodology performs in a commensurate way to the other techniques.

However, whilst the DEA is considered appropriate for the following locations within the Brisbane River catchment, it was not used in preference to the MCS estimates as the MCS provided greater consistency with the other locations downstream of the dams:

- Brisbane River at Linville
- Brisbane River at Gregors Creek
- Brisbane River at Fulhamvale
- Stanley River at Peachester
- Stanley River at Woodford
- Creesbrook Creek at Tinton
- Lockyer Creek at Helidon
- Lockyer Creek at Gatton
- Lockyer Creek at Glenore Grove
- Bremer River at Walloon
- Warrill Creek at Kalbar
- Warrill Creek at Amberley
- Purga Creek at Loamside

The DEA is therefore not considered appropriate for application to the locations situated adjacent to or downstream of the major tributary junctions. These sites include:

- Stanley River at Somerset Dam
- Brisbane River at Middle Creek
- Brisbane River at Wivenhoe Dam
- Brisbane River at Savages Crossing
- Brisbane River at Mt Crosby Weir
- Bremer River at Ipswich
- Brisbane River at Moggill
- Brisbane River at Centenary Bridge
- Brisbane River at Brisbane City Gauge

6.4.2 With-dams conditions

The 'with-dams conditions' also creates an issue with respect to the application of the spatial and temporal distribution of the design rainfall. In previous studies, an assumption in respect to the proportion of rainfall and relative timing of the downstream tributary runoff has had to be made. This assumption is quite arbitrary as there is no definitive method for estimating concurrent flooding.

In addition, the initial reservoir level assumption of FSL influences the more frequently occurring flood events, which can lead to an over estimation of the 1 in 2 to 1 in 20 AEP events.

For locations downstream of the Wivenhoe Dam in particular, the assumption of uniform temporal patterns tends to limit the effect of the timing of the downstream tributary contributions. This is compounded by the operation of the dam, which tends to delay and attenuate the upstream flood hydrographs. It is suspected that this may result in an underestimation of the true magnitude flood frequency of locations situated below the dam for this scenario.

It is acknowledged that the dam operations based upon the Loss of Communications gate strategy, means that the releases from Wivenhoe Dam do not consider the downstream flows implicitly, thereby reducing the significance of the timing issue to some extent. However, the application of uniform temporal patterns still represents a major limitation of this approach.

Most of the dams represented in the URBS model do not have PMF capacity. In the simulations, flows in excess of the dam capacity are assumed to overtop the dam wall using a weir flow formula. Consequences of overtopping (eg dam failure) are not considered. Further discussion is provided in the '*Dam Operations Module Implementation Report*' (Aurecon, 2015).

7 Monte-Carlo simulation framework

7.1 Methodology

7.1.1 Concept

In the Monte Carlo Simulations (MCS) approach, a large number of synthetic events is simulated with the combination of a hydrological model and a reservoir simulation model. Flood flow exceedance probabilities at key locations are derived from the number of exceedances in the model simulations. So, if a threshold discharge value at a specific location is exceeded in the majority of simulated events, the AEP of this threshold is estimated to be relatively high, whereas if a threshold is exceeded in only a few events, the AEP is estimated to be relatively low.

The MCS method has the advantage over more “traditional” approaches in flood risk analysis in that it explicitly considers all relevant physical processes and associated variability’s that contribute to flood events. A practical disadvantage if the method is that it is generally more complex to implement. The main challenge in the MCS approach is to generate realistic and representative synthetic flood events. This means the synthetic events should correctly account for probabilities of occurrence of all factors contributing to flood flows such as rainfall (depth, duration, spatial and temporal patterns), antecedent moisture conditions, initial reservoir volumes and ocean water levels. The likelihood of combined occurrences (correlations) of these factors needs to be taken into account as well. And, finally, the relevant physical processes in the catchment during flood events need to be correctly simulated.

Reference is made to the *Monte Carlo Simulation Report* (Aurecon, 2014, refer to Appendix D) for greater detail.

7.1.2 Framework

The Monte Carlo Framework consists of three major components:

1. Pre-processing: A combination of advanced statistical techniques to generate a large set of realistic and representative synthetic flood events. These events are characterised by rainfall, antecedent moisture conditions, initial reservoir volumes and ocean water levels
2. Processing: Simulation of the synthetic events with a combination of a hydrological model (URBS) and a reservoir simulation model (RTC tools) to obtain peak discharges and flow volumes at each location of interest
3. Post-processing: Statistical techniques to combine the results of 1 and 2 to derive annual exceedance probabilities for a range of flood flows and volumes across the entire Brisbane River system

The Monte Carlo Simulation model for the BRCFS was implemented in the Delft-FEWS framework. Delft-FEWS is a component-based modelling framework that incorporates a wide range of general data handling utilities and open interfaces to many hydrological and hydraulic models that are commonly used around the world, including the URBS hydrological model and RTC tools for reservoir modelling. Delft-FEWS can be used for data storage and retrieval tasks, simple forecasting systems and in highly complex operational forecasting systems. The advantage of using Delft-FEWS for all communication between components is that intermediate results (time series data) can be inspected for checking and debugging. Moreover, the modular setup of Delft-FEWS enables to replace components without much effort. A further advantage is that organisations like Seqwater and the Bureau of Meteorology are familiar with Delft-FEWS. This means the framework can easily be transferred among these and other organisations, which provides great opportunities to develop similar tools for other catchments.

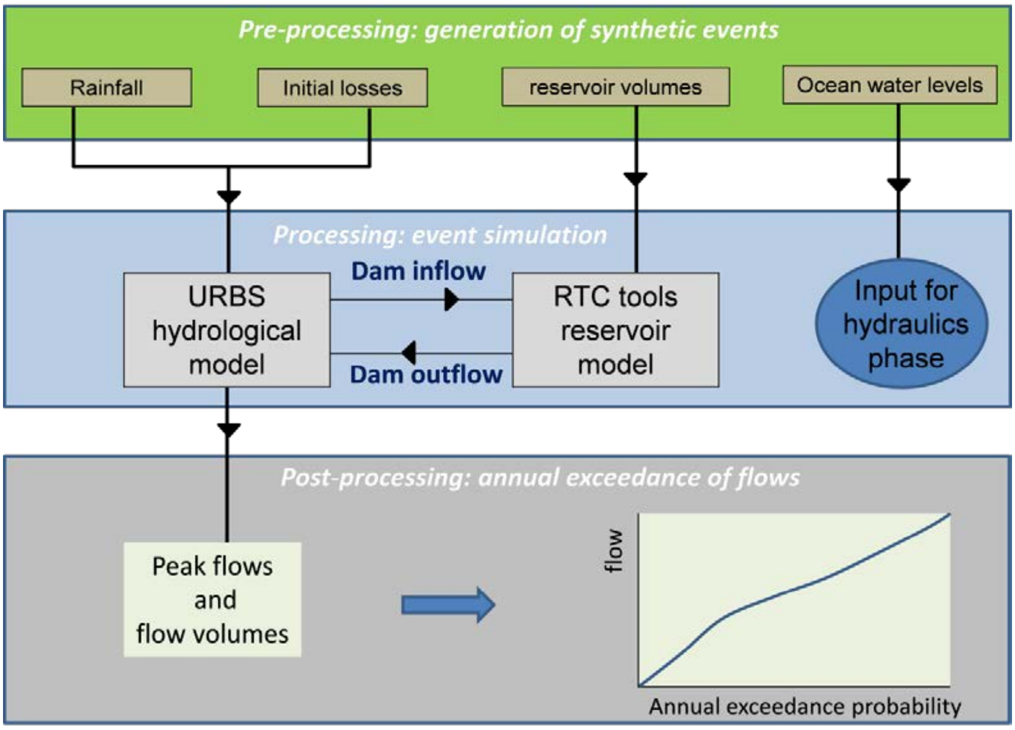


Figure 7-1 Schematic view of the Monte Carlo framework

7.1.3 Generation of synthetic flood events

Generation of synthetic flood events through statistical sampling is the core of the MCS approach. It is a method to simulate the variability of the relevant flood processes. This variability, inevitably present in natural processes and human interventions, is the main reason why design flows are quantifies in terms of probabilities (AEP's). For example, the natural variability of rainfall is such that it is impossible to predict the maximum 24-hour rainfall that will occur at a specific location next year. However, based on observations from the past, it *is* possible to estimate the *probability* that the maximum 24-hour rainfall exceeds certain threshold levels. In the Monte Carlo framework, these exceedance probabilities are mathematically quantified with statistical distribution functions. By taking repeated statistical samples from these statistical distribution functions, a (large) number of synthetic flood scenarios is generated that correctly represent the (statistical) characteristics of the flood processes under consideration. In essence, statistical sampling from distribution functions is a sophisticated version of 'throwing the dice'.

Since rainfall is the main driving force for flood events the implemented procedure is to first sample the rainfall characteristics of each event and to subsequently sample the other stochastic variables, conditional on the sampled value of the rainfall. This means, for example, if an extremely high rainfall depth is sampled, (ie a rainfall depth with a low AEP), the probability increases that the sampled ocean water level is extreme (ie low AEP) as well (since rainfall and ocean water levels are correlated).

Realistic sampling of rainfall is complex, because it needs to account for spatial and temporal correlations. For this purpose, an innovative method from the BoM for generating stochastic spatio-temporal rainfall patterns was adopted (see Section 4.4). For the rainfall sampling scheme within the MCS Framework, three methods were tested: TPT, CRC-CH and CSS. The TPT method was chosen as the preferred method for the *current study*, because this method provided the best match between the rainfall statistics (intensity-duration-frequency curves, or IFD curves) on one hand and the available BoM synthetic spatio-temporal rainfall patterns on the other hand. The other two methods (CSS and CRC-CH) are nevertheless considered very promising for future applications of Monte Carlo simulations, especially if more synthetic spatio-temporal rainfall patterns become available.

In the TPT sampling method, a number of burst durations are considered. For each burst duration, the rainfall depth is randomly sampled from the intensity-duration-frequency (IFD) curve, which capture exceedance probabilities of threshold rainfall depths. The steps in the TPT approach can be summarized as follows:

1. Choose a range of durations around the critical storm duration. For each duration carry out the following steps (i-v):
   i) Divide the range of relevant values rainfall depth into 60 intervals (bins)
   ii) Generate 21 rainfall events per bin, by selecting spatio-temporal rainfall events for which the rainfall depth and duration correspond to the duration and depth of the bin (see section 7.1.5 for more details)
   iii) For each of the 60*21 events, take a sample of the remaining random variables (initial losses, continuing losses, ocean water levels, initial dam water levels) and run the combined hydrological/reservoir model to obtain the peak discharge and flow volumes at the location of interest for each simulated event
   iv) Derive the conditional exceedance probability of peak discharge and flow volume, given the rainfall depth
   v) Derive the exceedance probabilities of a range of discharges or flow volumes through application of the total probability theorem
2. Step 1 results in a set of frequency curves, one for each burst duration. These frequency curves are subsequently combined to a single frequency curve, using an ‘envelope approach’. This means for each AEP of interest, the maximum over the corresponding design flows of the frequency curves of step 1 is selected

7.1.4 Statistics and correlations

The procedure as described in the previous section requires statistical distribution functions and correlation models. Statistical distribution functions are derived from observed or simulated time series. Statistics are usually derived by counting the annual number of exceedances of a range of threshold levels. For some variables, statistics were derived as part of the BRCFS; for other variables the statistics could be obtained from other sources such as ARR. ARR also proved to be a valuable resource for the quantification of correlations between stochastic variables, for example the correlation between ocean water levels and rainfall. For other pairs of variables, series of combined observations/simulations were gathered and analysed as part of the BRCFS, to derive correlations. If correlations were found to be irrelevant (ie correlation coefficients close to zero), they were not included in the MCS model to reduce the model complexity. The following statistical dependencies (correlations) between random variables were identified as relevant and have been incorporated in the Monte Carlo simulations:

- Spatial and temporal correlation of rainfall. This dependence is taken into account in the BoM synthetic rainfall patterns, which are incorporated in the Monte Carlo Framework
- Mutual correlations between antecedent moisture conditions (initial losses) of the various sub-catchments. These correlations are taken into account in the Monte Carlo simulations using a Gaussian copula model
- Correlation between rainfall and ocean water levels. This is modelled with a threshold-excess logistic model, as provided by ARR
- Correlation between rainfall and reservoir volumes. Reservoir volumes at the beginning of high rainfall events are on average significantly higher than reservoir volumes at any given day. For this reason, marginal distribution functions of reservoir volumes are based on observed reservoir volumes at the beginning of high rainfall events. The 'remaining' correlation is weak, ie the correlation between the total rainfall depth of a high rainfall event and the reservoir volume at the beginning of such an event. The latter is therefore not included in the MCS framework
- Mutual correlations of initial dam water levels. These are simulated with the skewed student-t copula model

7.1.5 Incorporation of stochastic rainfall patterns

Step 1b of section 7.1.3 mentions the use of stochastic spatio-temporal patterns in the sampling procedure. A storm pattern is sampled that 'matches' the selected burst duration and sampled rainfall depth. This means a filter criterion is required to make a selection of the spatio-temporal rainfall patterns for which depth and duration match within acceptable limits with selected burst duration and sampled rainfall depth. Since an exact match usually cannot be found, the rainfall depth needs to be scaled in such a way that the catchment rainfall depth is equal to the rainfall depth of step 1a. This means the rainfall of the spatio-temporal pattern is multiplied with a scaling factor. To explain the concept, the following parameters are defined:

R_1 = catchment average (burst) rainfall depth as sampled in step 1a

R_2 = catchment average (burst) rainfall depth from the sampled synthetic rainfall pattern

C = scaling factor: C = R1/R2

The rainfall intensities of the sampled synthetic rainfall pattern are multiplied with the scaling factor, C, to ensure that the catchment rainfall depth is equal to the rainfall depth of step 1a. To prevent that unrealistic storm patterns are created in the up-scaling process, the scaling factor should be limited to a certain range. According to Alan Seed of the BoM (personal communication) the scaling factor should not be outside the range [0.5, 2]. The allowed scaling factor of 2 in rainfall depth is therefore the first filter criterion that is applied in the selection process of the synthetic rainfall patterns.

The selection procedure only considers the burst period of interest. For example, if a 24-hour burst period is considered, a period of 24 hours of rainfall is sampled from the stochastic spatio-temporal rainfall pattern. This means no pre- or post-burst rainfall is sampled.

7.1.6 Event simulation and post processing

A combination of a hydrological model and a reservoir simulation model is used to derive peak discharges and flow volumes at the catchment outlet for each synthetic event. Details of the models were presented in Section 4.

The final step in the Monte Carlo Simulation procedure consists of the derivation of flood flows and associated Annual Exceedance Probabilities (AEPs), based on the simulations of the generated synthetic events. The estimated AEP of a threshold discharge at the location of interest is derived from the number of synthetic events in which this threshold is exceeded. In other words: if a threshold value is exceeded in the majority of simulated events, the AEP of this threshold is relatively high, whereas if a threshold is exceeded in only a few events, the AEP is relatively low.

In a *traditional* Monte Carlo application ("crude" Monte Carlo), a threshold peak discharge which is exceeded in 10% of the simulated events has an AEP is equal to 1 in 10 and a threshold peak discharge which is exceeded in 2% of the simulated events has an AEP is equal to 1 in 50. However, in the TPT method this is not the case. The TPT is a more sophisticated Monte Carlo method, which is significantly more efficient than a crude Monte Carlo method in terms of computation times. One of the consequences of the increased sophistication is that the post-processing is less straightforward. In the TPT method, synthetic events have different weighting factors, which mean not all events contribute equally to the exceedance probability. The mathematical details of the computation procedure for computing annual exceedance probabilities is described in Section 8 of the *Monte-Carlo Simulation Framework and Enhanced MCS Methodology Report*, (Aurecon, 2015, refer to Appendix D).

7.1.7 Computational steps

Figure 7-2 shows the computational scheme of the MCS framework. The procedure in this Figure is carried out separately for each river location/gauge of interest. The scheme is applied for a TPT based sampling method, but also for potential alternative sampling strategies.

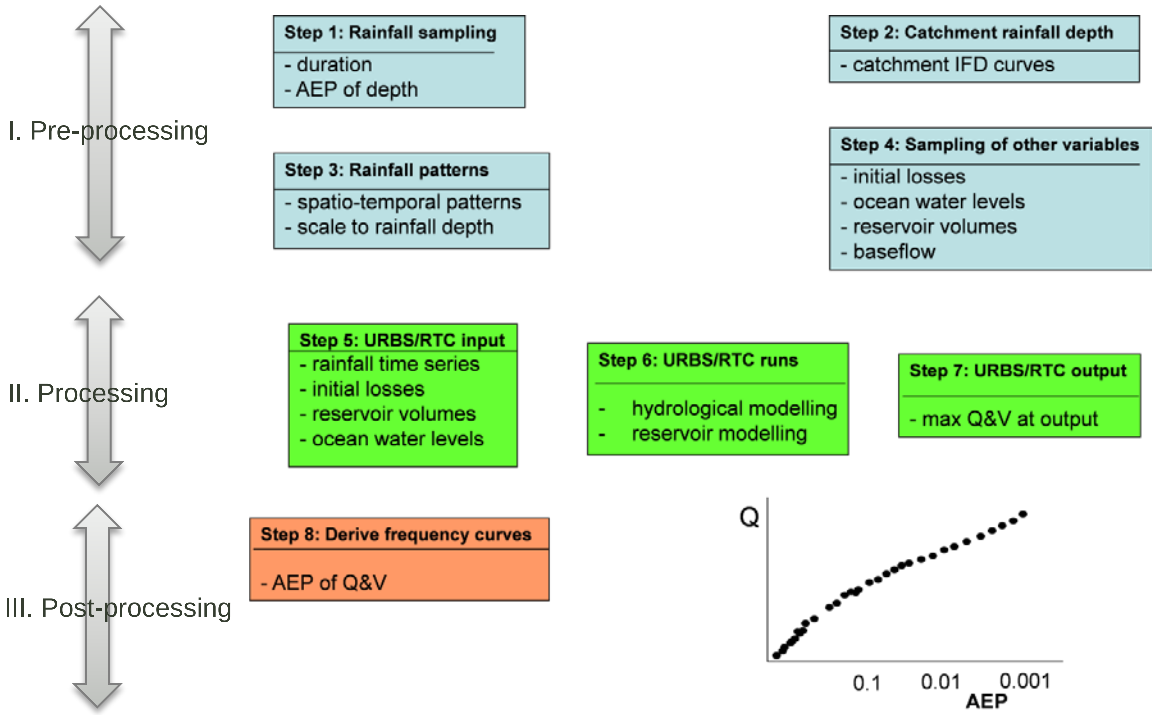


Figure 7-2 Computational steps of the Monte Carlo simulation framework

The three stages of Figure 7-2 and the steps of the computation can be summarised as follows.

I. *Pre-processing (steps 1-4)*: Generate *N* synthetic events, characterized by rainfall depth, rainfall duration, spatial and temporal distribution of rainfall, initial losses, reservoir volumes and ocean water levels. In the sampling process, mutual correlations between these variables are taken into account

   1. For each event, select the rainfall duration and sample the AEP of the rainfall depth
   2. For each event, derive the catchment-average rainfall depth, based on the duration and AEP of step 1. Here, the catchment refers to the upstream catchment of the river gauge/location under consideration. The rainfall depth is sampled form catchment IFD curves. These curves were derived from the available IFD curves from the Bureau of Meteorology (Green et al, 2012) and CRC-FORGE correction factors for more extreme events to derive point rainfall intensities for each of the sub-areas of the URBS model. Subsequently, an Areal Reduction Factor (ARF) was applied to the rainfall intensities (ARR) to account for the fact that rainfall is generally not equally extreme all over the catchment
   3. For events with rainfall AEP>1 in 2,000, sample one of the synthetic rainfall patterns as generated by the BoM stochastic space-time simulation model and reported in SKM (2013) and Jacobs (2014) and scale the rainfall intensities of these patterns in such a way that the catchment average total burst rainfall depth of each event is in accordance with step 2. For events with rainfall AEP<1 in 2,000, GTSMR based patterns (BoM, 2003) are used to model the spatio-temporal rainfall distribution
   4. For each event, sample losses, reservoir volumes, ocean water levels and baseflow. In the sampling process, mutual/spatial correlations and correlations with rainfall intensities are taken into account were relevant. Losses are assumed to be a function of the AEP of the rainfall. This function was derived in the reconciliation process, where MCS results are compared with results of other methods

II. Processing (steps 5-7): Simulate the N synthetic events with the URBS hydrological model and RTC dam operations module and derive the (N) peak discharges and flow volumes at the catchment outlet

   5. Prepare input files for the URBS/RTC models, based on the sampled values of steps 1-4
   6. Simulate the N synthetic events with the URBS hydrological model and RTC dam operations model
   7. Derive the N peak discharges and flow volumes at the catchment outlet

III. Post-processing (step 8)

   8. Apply MCS post-processing to peak discharges and flow volumes for a set of pre-defined Annual Exceedance Probabilities (AEP)

7.2 Output of the Monte Carlo framework

7.2.1 Flow series and flow statistics

The main output of the Monte Carlo Simulations framework consists of design peak flows, design flood volumes and associated annual exceedance probabilities. Furthermore, the framework produces hydrographs of all simulated events. The TPT method considers eight (burst) durations. For each of the eight considered rainfall durations, the following output files are created:

- A CSV-file containing all realisations of the random variables (rainfall depth, reservoir volume, initial loss, peak ocean water level) and the main URBS model output (peak flow, flood volume for different durations)
- A netcdf file that contains the flow hydrographs which are 240 hours in length

An additional csv-file is provided that contains the designs flow estimates for peak discharge and 24-hour, 48-hour and 72-hour flow volumes. Design flows are provided for AEP's of 1 in 2, 5, 10, 20, 50, 100, 200, 500, 1,000, 2,000, 10,000 and 100,000, as well as the AEP of the PMP.

The following durations have been considered in the current study:

- 12, 18, 24, 36, 48, 72, 96 and 120 hours for:
  - Brisbane River locations, downstream of Wivenhoe Dam for both 'with dams' and 'no dams' conditions
  - Wivenhoe Dam and Somerset Dam for 'with dams' conditions
- 3, 6, 12, 18, 24, 36, 48 and 72 hours for
  - Wivenhoe Dam and Somerset Dam for 'no dams' conditions
  - Tributary locations and Brisbane River locations upstream of Wivenhoe dam, for both 'with dams' and 'no dams' conditions

Future users of the Delft-FEWS framework have the opportunity to select other sets of burst durations if desired.

7.2.2 Ocean water levels

The MCS framework also generates ocean water levels (time series and peak values) even though they are of no influence on derived design flows and flow hydrographs. Generated series of ocean water levels will be used in the follow-up phase of the BRCFS: the hydraulics phase. Therefore, ocean water level series are an additional output of the MCS framework. The format of the ocean water level series is the same as the format of the discharge hydrographs: a netcdf-file for each considered duration, containing hydrographs which are 240 hours in length.

In the Monte Carlo Simulation framework, frequency distributions for ocean water levels in Moreton Bay were adopted from (GHD, 2014). It is taken into account that high surges and high rainfall often have the same meteorological cause (Cyclones, East Coast Lows and tropical storms). The resulting correlation of peak ocean levels and rainfall depth is modelled with the correlation model of AR&R Revision Project 18 (Zheng et al. 2013a, 2013b). This correlation model provides (correlated) samples of rainfall and peak ocean water level.

Ocean water level time series are composed with a standardised time series for astronomical tide and a standardised dimensionless storm surge hydrograph. The storm surge hydrograph has a total duration of 7 days, during which the surge is approximately two days above 50% of the peak surge (and, hence, five days below 50% of the peak surge). The adopted storm surge tide model is based upon the latest available information derived for the mouth of the Brisbane River (GHD, 2014). However, whilst the peak surge level relationship has been able to be adopted, the shape of the associated storm surge hydrograph has not been able to be derived directly from this study. The adopted storm surge hydrograph is based upon a study conducted for the Sunshine Coast (Aurecon, 2013) and therefore may not be truly representative of the mouth of the Brisbane River or indeed the corresponding synoptic driver for the peak storm surge levels.

7.3 Assumptions

The Monte Carlo Simulations concept in principle does not rely on any assumptions. In practice, however, assumptions are often required in the formulation of the various model components, for example due to data limitations. Extensive comparison between the output of each component with available measurements is essential to improve the reliability of the overall framework. However, for extreme events, ie events with a magnitude that is significantly higher than observed events this is not possible. Therefore, the Monte Carlo simulations framework relies on the assumption that the observed statistics and physical processes are correctly extrapolated to describe and quantify extreme events.

7.4 Limitations

Monte-Carlo Simulation removes many of the limitations common to design event approach methodology. This method is particularly advantageous in capturing the joint probability of flooding from the Brisbane River and its major tributaries (eg Bremer River and Lockyer Creek), and from catchment and oceanic flooding. MCS is therefore considered to be especially advantageous for locations along the Lower Brisbane.

The main limitations are related to "practical issues". First of all, a Monte Carlo Simulation method is generally complex to implement. As a consequence, there is a relatively small (but growing) group of experts capable of implementing such a method.

The definition of correlations and distributions for relationships between input parameters needs to be determined for the specific data set that is available to the investigation. This can be somewhat limiting if insufficient local data is available. In the BRCFS, a novel approach has been adopted to assist in the definition of space and time patterns (BoM and Jacobs, 2014). This is seen as being one of the differentiating characteristics of the application of this MCS application. However, even this approach is limited to a range of observed events, and is not yet proven for the extreme range of the flood frequency spectrum.

Another challenge is that this method usually requires a large number of model simulations, which may limit the practical application of the method. Fortunately, the hydrological model and dam operations model that were used in this study require relatively little computation time. Computation times for a single output location near the catchment outlet, simulation approximately 10,000 events, are in the order of five hours on a 64 bit machine, Windows 7, Solid State Drive (SSD) with 16Gb and 4 cores (duplicated, so actually 8 cores). For upstream locations with smaller catchment areas the runtime is in the order of two to three hours.

8 Design flood estimates

8.1 Introduction

The reconciled peak flows and flow volumes are based on the main results of the Flood Frequency Analysis (FFA), the Design Event Approach (DEA) and the Monte Carlo Simulations (MCS) and empirical estimates derived from the data. Refer to Appendix D, F and G for details of the techniques. When reconciling the results obtained from the application of the three methods it is important to recognise the strengths and limitations of each method.

8.1.1 Sources of flow estimate

Flood frequency analysis is an assessment of flows measured directly at the site. It is dependent on the physical and statistical reliability of the available data, including the accuracy of the flow rating curve, the length of the data record, and the statistical representativeness of the flows observed in that period of record. Flood frequency analysis is most reliable for frequent flood events. Extrapolation to large and rare events can be strongly influenced by the presence (or lack of) extreme events in the data record.

The Design Event Approach has numerous limitations. It is dependent on hydrologic modelling to convert rainfall to runoff, which infers assumptions of adopted temporal pattern and spatially uniform rainfall distribution (uniform with respect to AEP) across the catchment. A fundamental assumption is that flood AEP is equal to the AEP of the causal rainfall, which is not necessarily correct. It is necessary to adopt 'AEP neutral' losses that are typically higher for frequent events and decrease with flood magnitude. Monte-Carlo Simulation removes many of the limitations common to Design Event methodologies. MCS is particularly advantageous in capturing the joint probability of flooding from the Brisbane River and its major tributaries (eg Bremer River and Lockyer Creek), and from catchment and oceanic flooding. MCS is therefore considered to be especially advantageous for locations along the Lower Brisbane. The MCS also has the advantage for the with-dams conditions of implicitly capturing the influence of the varying initial reservoir levels.

Both MCS and DEA approaches are dependent on hydrologic modelling and the factors that influence the relationships between rainfall, runoff and flow. These factors, such as initial losses[1] and continuing losses, must be related to the rainfall event, using relationships that are often semi-empirical and/or difficult to reliably quantify. The primary advantage of MCS and DEA is that the typically longer record and spatial consistency of rainfall records makes extrapolation to extreme events more reliable than site-specific stream gauge records. Another advantage is that the application of hydrologic model enables these approaches to capture effects of physical limits, such as flow capacity, in the system on

[1] In MCS, initial losses are represented as random variables, independent of the magnitude of the rainfall event, so for MCS only continuing losses will be related to event magnitude

flood frequencies. These models are also readily adaptable to include changes in land use or channelization or the inclusion of new reservoirs.

8.1.2 Consistency of data

FFA flow estimates are derived from site-based stream gauge records. These records are independent from the DEA and MCS flows derived using rainfall/hydrologic modelling, and are also independent from each other. The first step of the reconciliation process is to ensure that the various flow estimates are consistent locally and regionally. This was achieved by making sure that:

- Hydrologic model flows are consistent with rated flows. Calibration of the URBS models (Aurecon, 2014a) should ensure that the models are reasonably consistent with the stream gauge ratings
- Rated flows for in-line gauges are consistent with each other, with the URBS model results and with other available verification sources (eg stream gaugings, 2D hydrodynamic model simulation results). This can be achieved by ensuring that rated flows for historical events at consecutive gauges are consistent (within known accuracy limits/uncertainty of gauge level, rainfall etc)
- Flood frequency predictions are consistent throughout the catchment. Comparison of mean, standard deviation and skewness as a function of catchment area and location can be used to identify gauges with suspect flow ratings or stream gauge records. Regional analysis and use of regional skewness can be used to improve confidence in the FFA predictions

8.1.3 Event magnitude classes

Four design flow classes can be distinguished with respect to event magnitude:

1. Frequent events: AEP values ranging from 1 in 2 to 1 in 50
2. Large events: AEP values ranging from 1in 50 to 1 in 100
3. Rare events: AEP values ranging from 1 in 100 to 1 in 2,000
4. Extreme events: AEP values ranging from 1 in 2,000 to the AEP of the Probable Maximum Precipitation (PMP)

This is consistent with AR&R notional design event classes and is a convenient way to distinguish the uncertainty inherent within flood events of various magnitudes.

8.2 Reconciliation process

8.2.1 No-dams condition

In order to reconcile design flow estimates, initial and continuing loss parameters in the DEA and MCS models were chosen in such a way that DEA and MCS results are as much as possible in accordance with FFA results for frequent events, (refer to Appendix I, *Reconciled and recommended flood frequency estimates report, Aurecon 2015).* This reconciliation procedure is constrained by the requirements that:

- Loss values need to be consistent with those generally adopted in practice
- Loss values should be relatively consistent (within rational explanation) across sub-catchments

Sensitivity runs were carried out for the DEA and MCS models to analyse which loss parameters would provide a good match with FFA results. Resulting loss parameters are presented in the *Reconciled and recommended flood frequency estimates report* (Aurecon, 2015, Refer Appendix I). Subsequently, DEA and MCS runs were carried out for all locations with the selected loss values and results will be compared with FFA. For this purpose, Figures are produced for each location in the catchment, containing:

- Plotting positions of rated flows
- Derived frequency curves of FFA, MCS and DEA

The produced Figures were analysed extensively to verify whether the frequency curves of MCS and DEA are in accordance with FFA (and rated flows). For locations where this is not the case, a probable cause was identified and a decision was made on whether the following needs to be reconsidered:

- The selected loss values for DEA and MCS
- The applied FFA probability distribution function and/or fit method
- The reliability of the series of rated peak discharges
- The reliability of IFD curves as used in the DEA and MCS methods

In the end, the approach that produces design flows that are considered most 'realistic' was adopted. In cases were DEA and MCS methods provide similar results, the MCS method is the preferred choice. The main reasons are that the MCS method is expected to provide more reliable design flow estimates for the 'with-dams conditions' and also more realistic design flow hydrographs.

These are a few locations for which no (reliable) rated flows are available and, hence, no FFA results as well. Reconciled estimates therefore in principle should be based on either DEA or MCS results only. However, this may lead to inconsistencies with reconciled results of nearby locations for which reconciled design flow estimates were based on FFA results or probability estimates from rated flows. To improve consistency in peak flows of nearby locations, the rated flows of the nearby locations are included in the reconciliation process for locations for which no (reliable) rated flows are available.

As a final verification, the flood frequency curves are verified for internal consistency between locations. For any given AEP, the following Figures are made:

1. Peak flow (Q) versus catchment area (A) for all locations
2. Q/A versus A for various (all) locations

The first Figure should reveal an increasing trend; the second should reveal a decreasing trend. If this is not the case for some locations, it will be verified if this can be explained from physical characteristics of the specific catchments under consideration. If no such explanation can be given, the reconciliation process needs to be re-iterated.

8.2.2 With-dams conditions

Flood frequency analysis of stream gauge records for 'with-dams' conditions is considered to be of limited benefit, particularly for the locations on the Brisbane River downstream of Wivenhoe as:

- Consistent post-dam data record is limited (approximately 30 years)
- The data will not fit a known statistical distribution
- Data is influenced by dam operations and therefore not fully homogeneous

Because of these issues, traditional FFA methods, including calculation of a probability distribution and the subsequent fitting of confidence limits cannot be conducted. However, rated flows can be assigned a probability estimate ('plotting position') to allow a general comparison with flow probability estimates from the MCS and DEA approaches. This comparison can only be made for AEP values above 1 in N, where N is the length of the length of the series of rated flows in years.

Results of the DEA approach can potentially be reconciled by adjusting starting dam water levels used in the design events to produce an 'AEP neutral' level, similar to the concept of 'AEP neutral' losses. However, for the purpose of this study this approach has not been adopted and the initial reservoir level is assumed to be FSL. For MCS this is in principle no option, as the starting dam water level is randomly generated from the derived distribution functions as described in Monte-Carlo simulation framework and enhanced MCS methodology Report (Aurecon, 2015, Refer Appendix D).

8.3 Reconciliation estimates – No-dams condition

For 'no-dams conditions', DEA and MCS results were available for all 22 locations of interest. FFA results were available for 17 locations as limited or no (reliable) data was available for the other 5 locations. DEA and MCS results for location Rifle Range Road were not used in the reconciliation procedure because the URBS hydrological model simulation results are only main channel flows and not total flows for this location.

No uniform approach could be applied for all locations due to differences in data availability and differences in the mutual consistency between FFA, DEA and MCS results. The 22 locations were divided into six 'clusters'; for each cluster a different approach was used. The subdivision in clusters is based on two criteria:

1. Data availability
2. Differences in design peak discharges between the various methods

For 'no-dams conditions', the reconciled design flows for the majority of the locations are based on a combination of:

- Empirical estimates from rated flows for frequent events
- Flood frequency analysis results for frequent to large events
- Monte Carlo Simulations results for large to extreme events

Table 8-1 presents a summary of the peak flows for the no-dams condition, whilst Figure 8-1 to Figure 8-5 shows the reconciled flood frequency curve for various sites in the Brisbane River catchment for the no-dams condition, including:

- Brisbane River at Gregors Creek
- Brisbane River at Savages Crossing
- Lockyer Creek at Glenore Grove
- Bremer River at Ipswich
- Brisbane River at Brisbane City

The results indicate that for the Bremer River at Ipswich, the DEA and MCS results do not correspond with the reconciled estimates which have been derived with consideration of data from other adjacent locations. This is because the rainfall based approaches are dependent on the ARR design rainfalls, which for the Bremer River catchment appear to be underestimated, refer WMAWater, 2014.

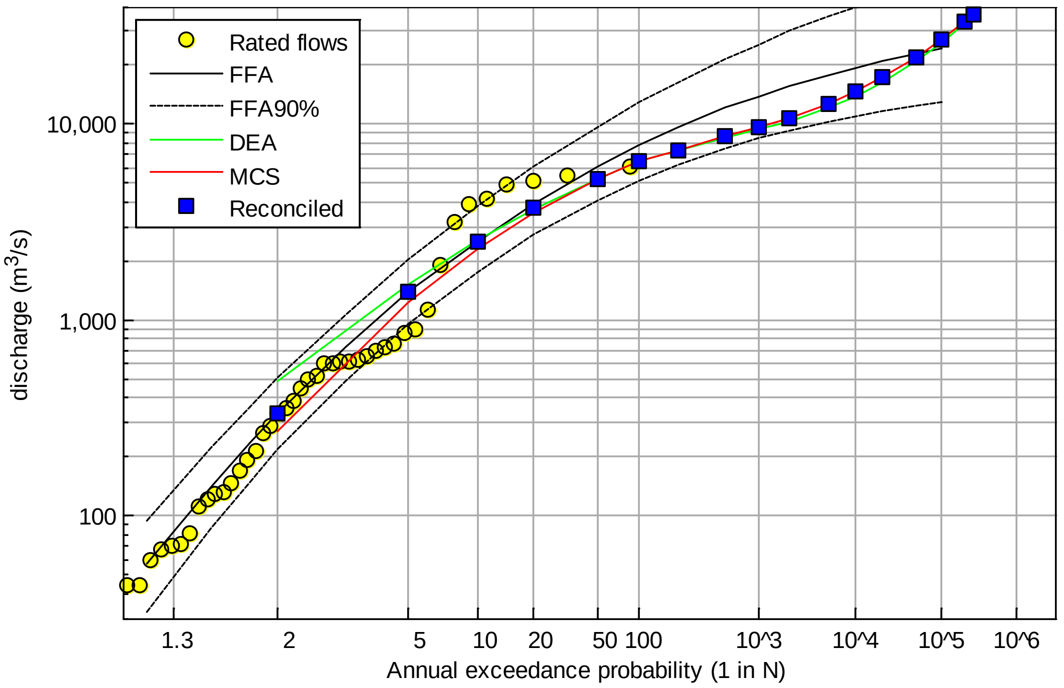


Figure 8-1 Reconciled flood frequency curve for Brisbane River at Gregors Creek; no-dams condition

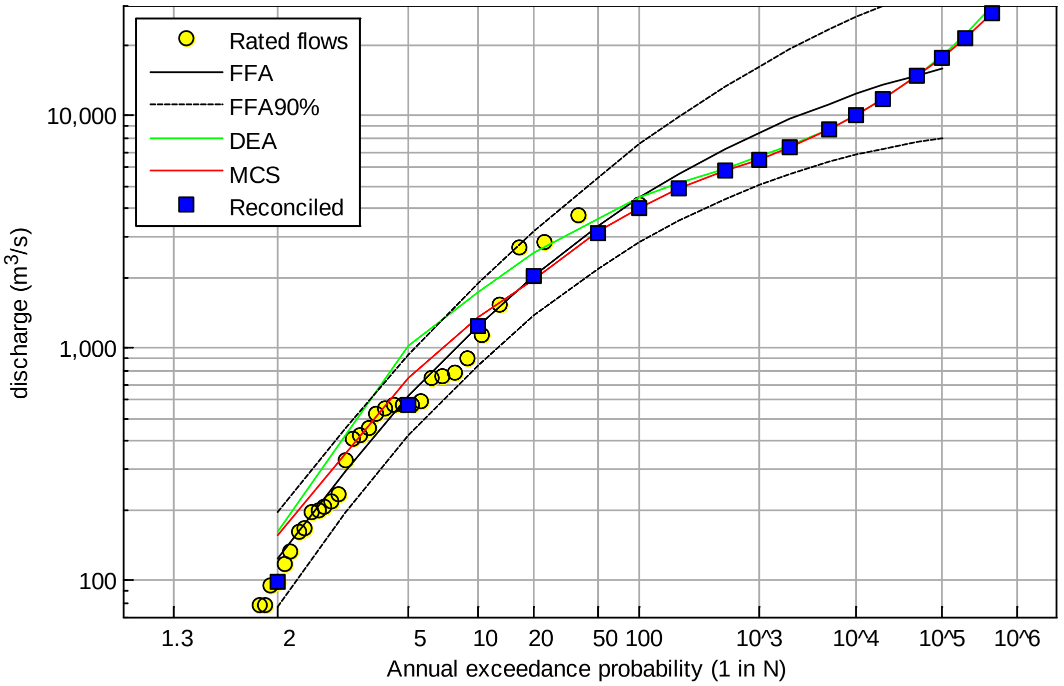


Figure 8-2 Reconciled flood frequency curve for Lockyer Creek at Glenore Grove; no-dams condition

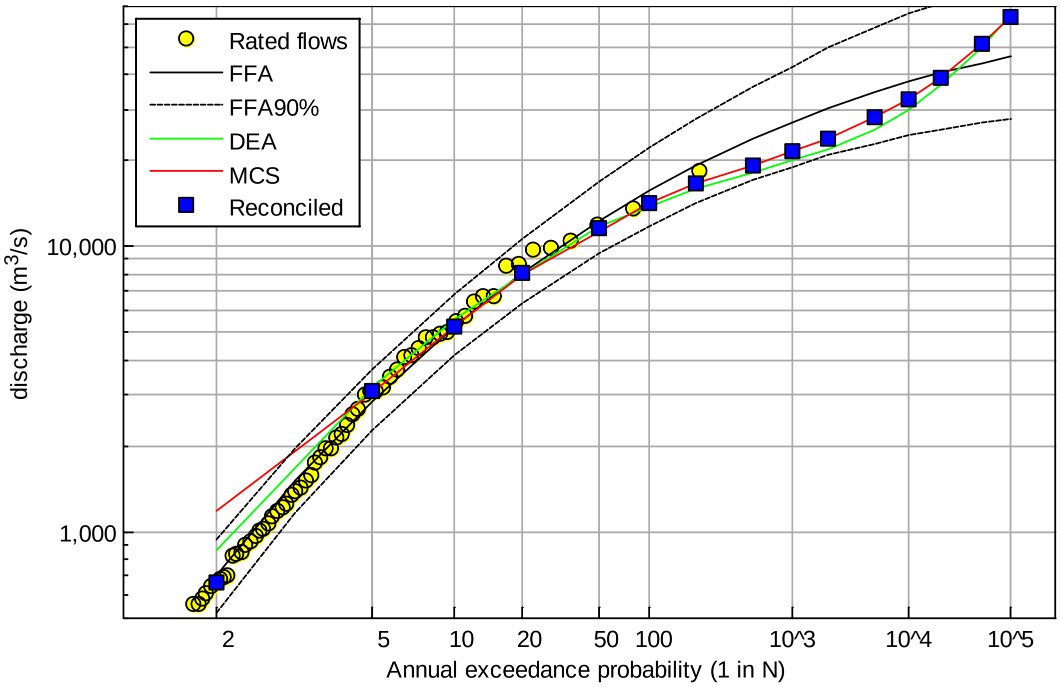


Figure 8-3 Reconciled flood frequency curve for Brisbane River at Savages Crossing; no-dams condition

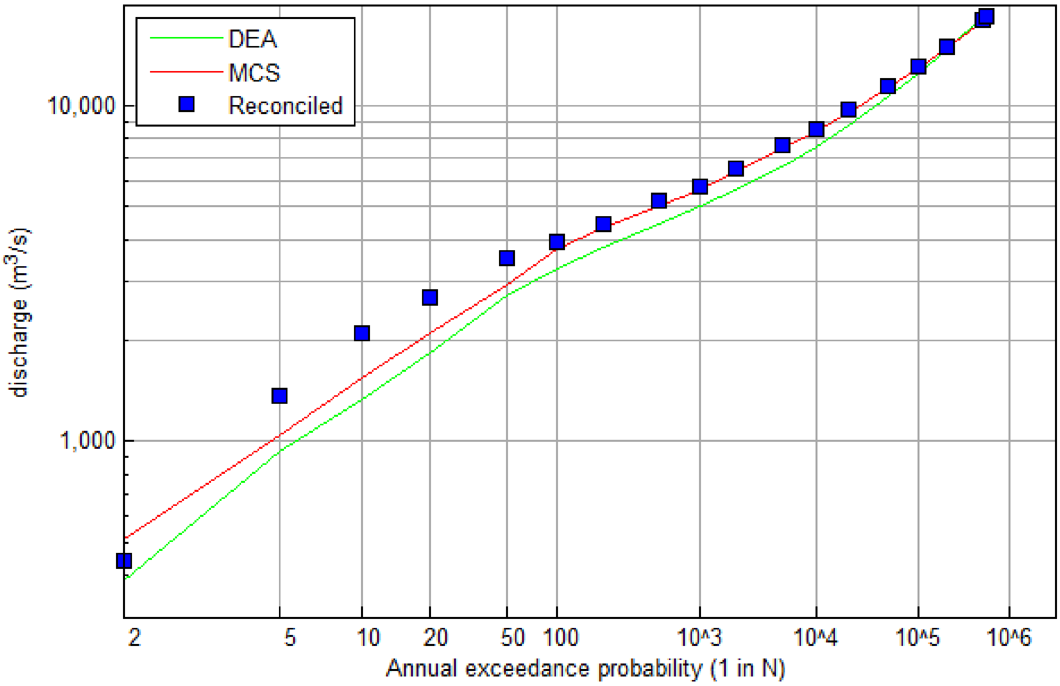


Figure 8-4 Reconciled flood frequency curve for Bremer River at Ipswich; no- dams condition

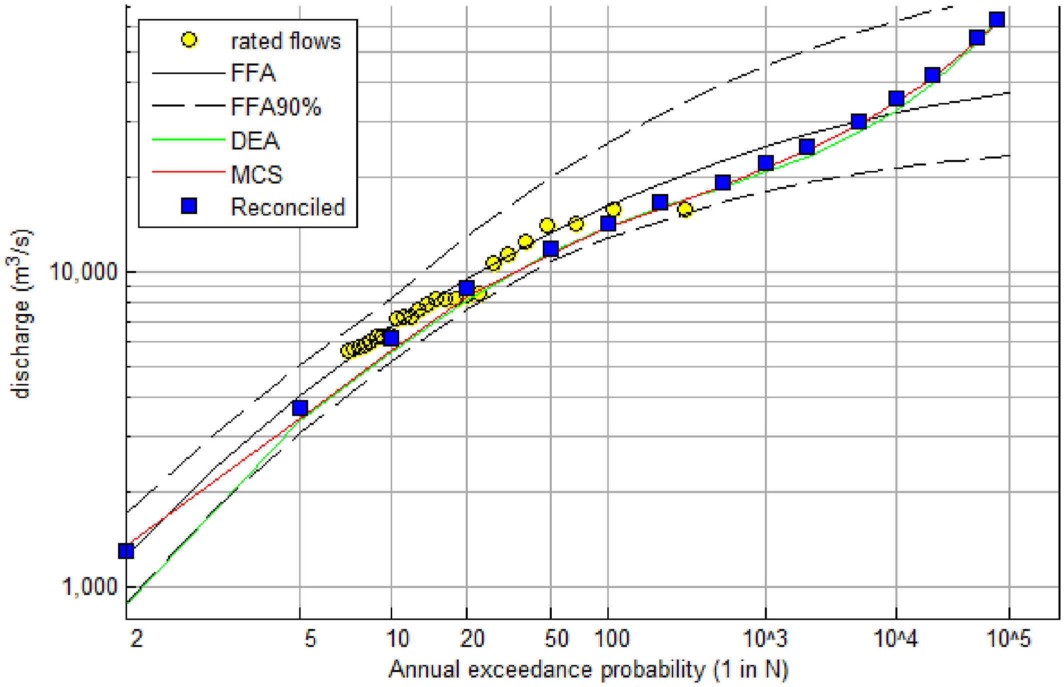


Figure 8-5 Reconciled flood frequency curve for Brisbane River at Brisbane City; no-dams conditions

The choice of bounds between the 'high', 'intermediate' and 'low' range of AEP's differed per location. For locations for which limited or no (reliable) data on peak discharges were available and, hence, no FFA results, the reconciled design flows for the high and intermediate range of AEP values were based on data and FFA results of nearby stations.

8.4 Reconciliation estimates – With-dams conditions

For 'with-dams conditions', generally limited data series are available. For the Lower Brisbane locations a series of approximately 30 years of rated flows are available, starting in the year of completion of Wivenhoe Dam. This period starts and ends with several major flood events (1983, 2011, 2013) but also overlaps the longest drought in Brisbane's recorded history. The record may therefore not be statistically representative. Reconciliation of design flows based on these rated flows should therefore be done with care. Furthermore it is not possible to derive a statistical distribution function that matches this 'unbalanced' series of rated flows. The FFA analysis has therefore not been carried out for 'with-dams conditions'.

The eight 'with-dams conditions' locations were divided into four 'clusters'; for each cluster a different approach was used. The subdivision in clusters is based on two criteria:

1. Data availability
2. Differences in design peak discharges between the various methods

For 'with-dams conditions', the reconciled design flows for these locations are based on a combination of:

- Empirical estimates from rated flows for (very) frequent events
- Monte Carlo Simulations results for frequent events to extreme events

Table 8-2 presents a summary of the peak flow estimates for the with-dams conditions, whilst Figure 8-6 to Figure 8-8 shows the reconciled flood frequency curve for various locations in the Brisbane River catchment located downstream of Wivenhoe Dam for the with-dams conditions. The results indicate that for the Bremer River at Ipswich, the DEA and MCS results do not correspond with the reconciled estimates which have been derived with consideration of data from other adjacent locations. This is because the rainfall based approaches are dependent on the ARR design rainfalls, which for the Bremer River catchment appear to be underestimated, refer WMAWater, 2014.

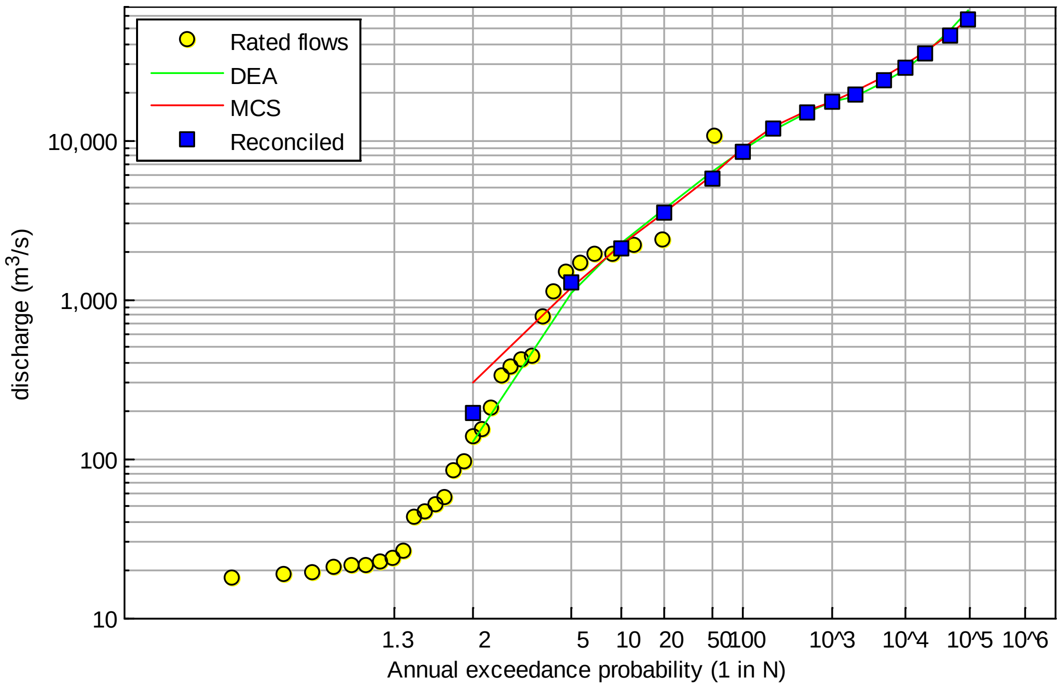


Figure 8-6 Reconciled flood frequency curve for Brisbane River at Savages Crossing; With-dams conditions

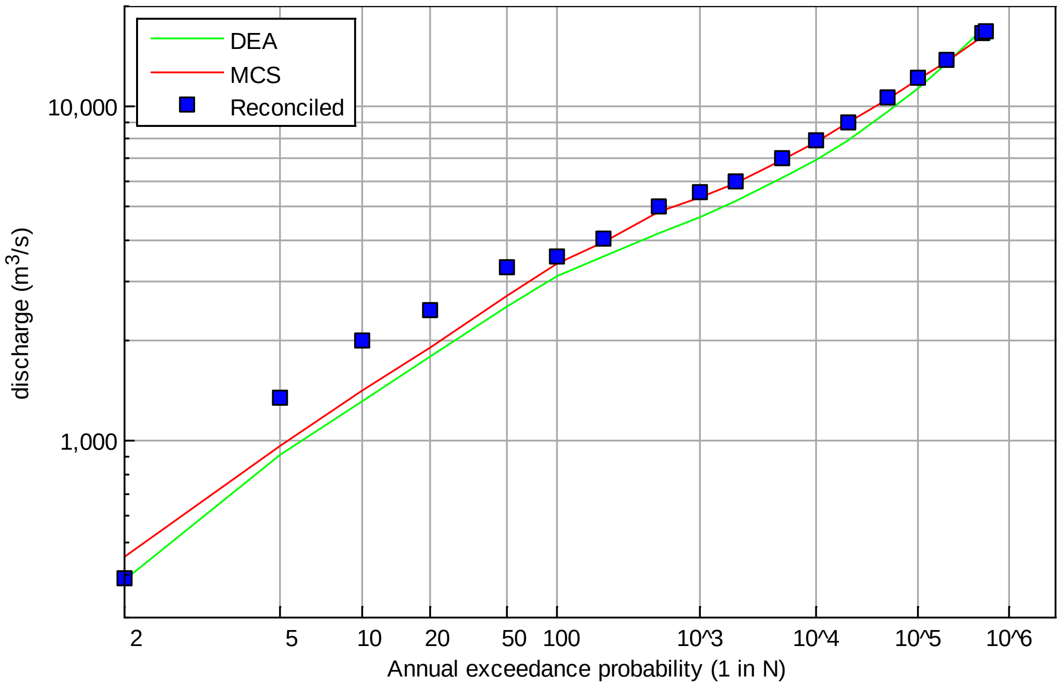


Figure 8-7 Reconciled flood frequency curve for Bremer River at Ipswich; With-dams conditions

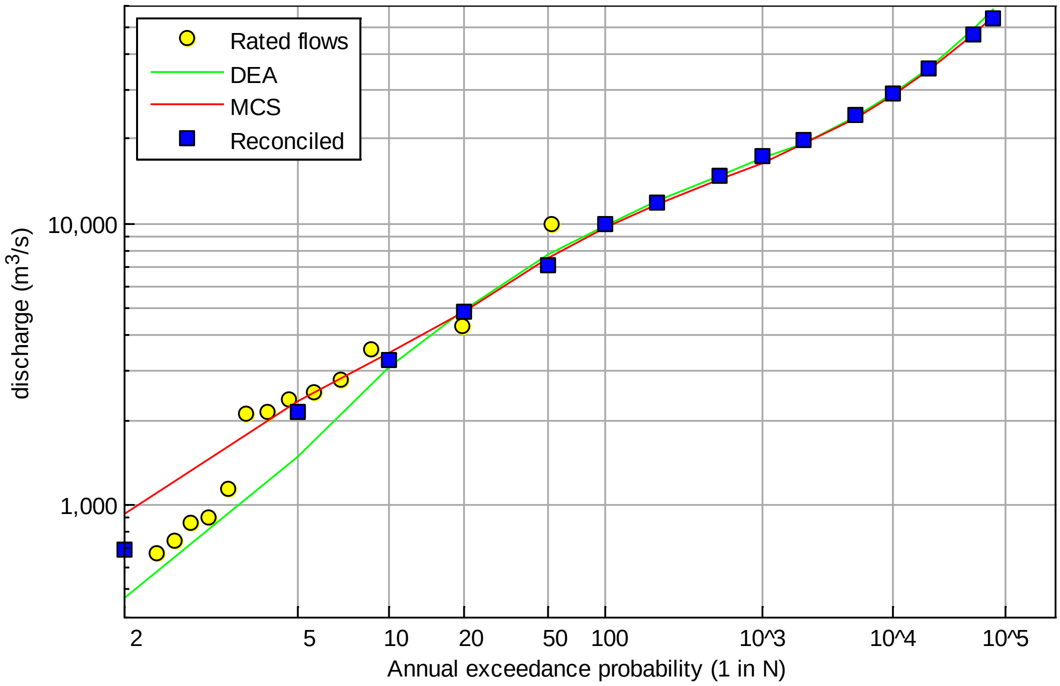


Figure 8-8 Reconciled flood frequency curve for Brisbane River at Brisbane City; With-dams conditions

Table 8-1 Peak discharges (m^3/s) reconciled results: no-dams conditions. Note: the 1 in 100,000 AEP peak discharge is only provided for locations for which the AEP of the PMP is below 1 in 100,000

Location	AEP (1 in N)													
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	PMP DF	AEP of PMP
Linville	150	710	1,300	1,900	2,900	3,700	4,200	4,800	5,400	6,000	7,900	13,700	21,400	500,000
Gregors Creek	330	1,400	2,500	3,700	5,300	6,500	7,400	8,800	9,700	10,800	14,500	26,300	36,300	260,000
Fulham Vale	370	1,700	2,700	4,000	5,400	6,400	7,400	8,800	9,700	11,100	14,700	26,600	34,500	250,000
Peachester	120	300	420	540	680	780	870	980	1,100	1,200	1,400	1,900	2,600	9,710,000
Woodford	210	510	750	1,000	1,300	1,500	1,700	2,000	2,200	2,500	3,200	4,300	6,000	4,070,000
Somerset Dam	540	1,400	2,200	3,000	3,900	4,600	5,200	6,100	6,700	7,400	9,500	13,400	18,300	750,000
Tinton	37	210	390	590	840	1,100	1,200	1,400	1,600	1,800	2,400	3,400	6,000	2,360,000
Middle Ck	670	2,400	4,500	6,600	9,000	10,900	12,400	14,600	16,500	18,800	25,200	57,800	65,700	150,000
Wivenhoe	670	2,400	4,600	6,800	9,300	11,200	12,800	15,100	16,800	19,000	25,000	49,200	54,800	140,000
Helidon	73	230	400	590	800	960	1,100	1,300	1,500	1,700	2,200	3,400	6,700	2,840,000
Gatton	89	410	830	1,300	2,300	3,100	3,700	4,400	5,000	5,600	7,900	13,600	24,000	650,000
Glenore Grove	99	570	1,200	2,000	3,200	4,000	4,900	5,800	6,500	7,400	10,400	18,300	27,700	460,000
Savages Crossing	670	3,100	5,200	8,100	11,600	14,300	16,600	19,100	21,500	23,900	32,600	63,800	63,800	100,000
Mount Crosby	830	3,100	5,400	8,100	11,400	13,800	16,100	18,800	21,300	23,400	32,400	N/A	62,600	90,000
Walloon	260	680	1,100	1,300	1,600	1,900	2,200	2,500	2,800	3,100	4,000	5,500	8,700	1,570,000
Kalbar Weir	200	590	950	1,200	1,600	1,700	1,800	2,100	2,300	2,600	3,400	4,600	7,600	2,180,000
Amberley	230	630	1,000	1,400	2,000	2,200	2,400	2,800	3,000	3,400	4,500	6,400	9,980	1,110,000
Loamside	65	210	310	390	490	580	670	780	870	980	1,200	1,700	2,800	4,770,000
Ipswich	440	1,400	2,100	2,700	3,500	3,900	4,400	5,200	5,800	6,500	8,800	13,200	18,400	540,000
Moggill	1,100	3,800	6,400	9,300	12,300	14,600	17,000	19,900	23,000	25,900	35,800	N/A	64,400	80,000
Centenary Bridge	1,100	3,700	6,200	9,000	11,800	14,000	16,400	19,300	22,300	25,300	35,500	N/A	64,900	80,000
Brisbane	1,100	3,700	6,200	8,900	11,800	13,900	16,300	19,100	22,000	25,000	34,600	N/A	62,800	80,000

Table 8-2 Peak discharges (m^3/s) reconciled results: with-dams conditions. Note: the 1 in 100,000 AEP peak discharge is only provided for locations for which the AEP of the PMP is below 1 in 100,000

Location	AEP (1 in N)													
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	PMP DF	AEP of PMP
Somerset Dam	0	800	1,300	1,800	2,200	2,500	3,000	3,300	3,600	4,000	5,400	10,700	20,900	750,000
Wivenhoe	0	470	930	1,700	3,300	6,300	8,800	10,300	12,500	12,900	21,200	35,800	43,700	140,000
Savages Crossing	190	1,300	2,100	3,500	5,800	8,500	11,800	15,000	17,500	19,500	29,000	56,900	56,900	100,000
Mount Crosby	200	1,300	2,200	3,600	6,000	8,600	11,700	14,800	17,100	19,700	27,200	N/A	55,500	90,000
Ipswich	390	1,300	2,000	2,500	3,300	3,600	4,000	5,000	5,600	6,000	8,300	12,000	16,700	540,000
Moggill	630	2,100	3,300	4,800	7,300	10,200	12,400	15,700	18,000	20,400	29,300	N/A	57,600	80,000
Centenary Bridge	640	2,100	3,300	4,800	7,100	9,900	11,900	15,000	17,700	19,900	28,500	N/A	55,900	80,000
Brisbane	700	2,200	3,300	4,800	7,100	9,900	12,000	14,900	17,500	19,700	27,600	N/A	53,800	80,000

Note *: Estimates shown in red are above 28,000m^3/s which exceed the maximum release capacity of Wivenhoe Dam. These estimates should be treated with caution. Estimates shown in blue for Somerset dam should be treated with caution as well as they are the flows associated with the design flood estimates specific to Wivenhoe Dam.

8.5 Key findings and discussion

8.5.1 No-dams conditions – Peak flows

The following observations were made for no-dams conditions:

- For AEP<1 in 50, differences between MCS and DEA design flows are generally relatively small compared to differences in design flows for these methods on one hand and FFA design flows on the other hand. This shows that in this range of AEP-values, the peak discharges of the two rainfall based methods are mainly determined by the catchment average rainfall depth, which is the same for both methods. Losses are near zero in this range in both methods
- For the majority of locations there is reasonably good agreement between MCS and DEA results on one hand and FFA results (and plotting positions of rated flows) on the other hand for the range of floods from 1 in 2 AEP to 1 in 100 AEP
- Design peak discharges for the Bremer sub-catchment locations Amberley, Walloon and Loamside are underestimated by both the DEA and MCS methods. This is most likely caused by the fact that rainfall depths of the IFD curves that are used as input for MCS and DEA methods are suspected to be an underestimation of the 'actual' rainfall depths. This conclusion was independently confirmed by WMAwater (WMAwater 2014). For this reason, a correction was applied on MCS design peaks for these locations to obtain reconciled design peak flows
- For location Peachester, both DEA and MCS methods underestimated design peak flows. For this reason, a correction was applied on MCS design peaks for these locations to obtain reconciled design peak flows

8.5.2 With-dams conditions – Peak flows

- For Brisbane River locations downstream of Wivenhoe Dam, differences in MCS and DEA design flows for AEP<1 in 10 are small
- For Wivenhoe Dam, DEA design flows are higher than MCS design flows for AEP≥1 in 100. This is mainly caused by the fact that the reservoirs are assumed to be at full supply level at the beginning of each simulated event in the DEA approach. In the MCS simulations, starting dam levels are modelled as stochastic variables
- The differences in MCS and DEA design flows at Wivenhoe Dam are not reflected at locations along the Lower Brisbane River. For the Lower Brisbane River locations, MCS design flows are generally higher than DEA design flows
- For the majority of locations there is a reasonable agreement between MCS and DEA design flows on one hand and plotting positions of rated flows on the other hand, except for AEP values greater than 1 in 10

8.5.3 Comparison of No-dams and With-dams conditions – Peak flows

- .For all locations except Somerset Dam, 'With dams' design flows (peaks and volumes) are consistently lower than 'no dams' design flows
- For all locations except Somerset Dam, 'With dams' peak flows of individually simulated events are, with a few exceptions, lower than corresponding 'no dams' peak discharges
- For location Somerset Dam, 'With dams' peak flows of individually simulated events are often higher than corresponding 'no dams' peak discharges, especially in the range of extreme events. This seemingly inconsistency may be partly caused by the fact that MCS results for Somerset Dam

were abstracted from the Wivenhoe Dam simulation run. The reason to use this special approach instead of carrying out an individual run for the Somerset Dam catchment, is that the operation of the Somerset Dam heavily depends on Wivenhoe Dam levels, which means an individual run for the Somerset Dam catchment is not meaningful for 'with dams' conditions. However, results show the validity of the currently applied approach is doubtful as well. It is therefore recommended not to adopt the derived 'with dams' results for location Somerset Dam

- The reducing effects of the dams on peak discharges are lowest for location Ipswich, which is explained from the fact that peak flows at Ipswich are only influenced by Moogerah Dam, not by Somerset Dam and Wivenhoe Dam
- The dams reduce the1 in 100 AEP peak discharge at Moggill from 14,600 m^3/s to 10,200 m^3/s

Figure 8-9 shows the comparison between the no-dams conditions and with-dams conditions reconciled flood frequency curves for the Brisbane River at Wivenhoe Dam.

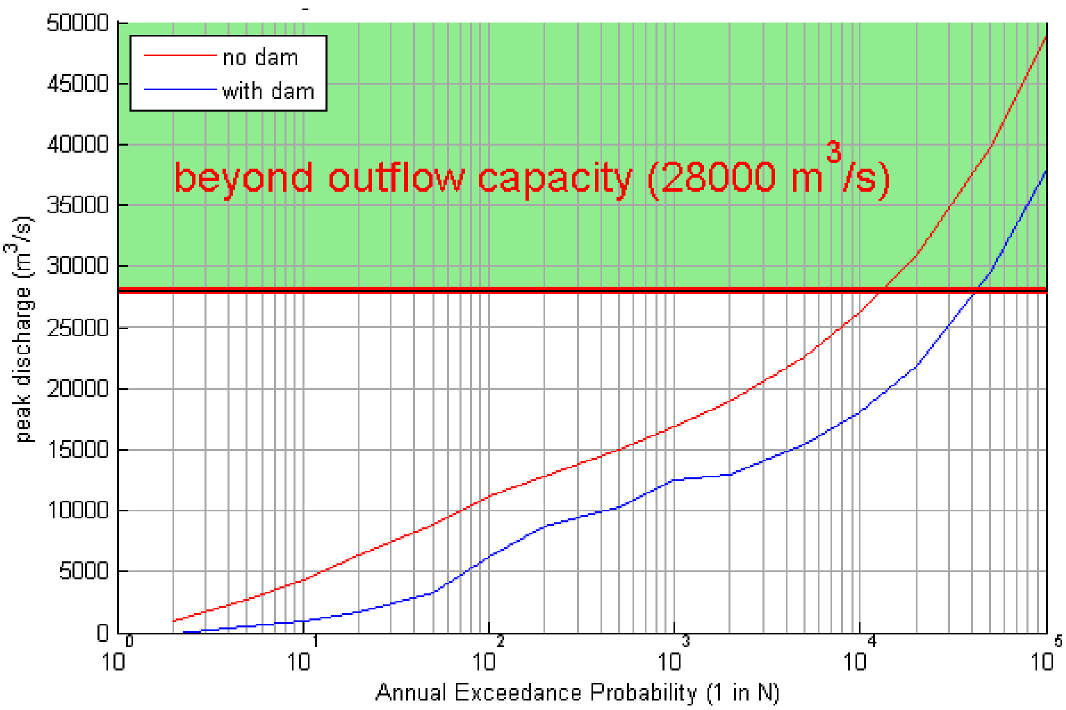


Figure 8-9 Reconciled flood frequency curve for Brisbane River at Wivenhoe Dam: no-dams condition versus with-dams condition

Table 8-3 compares design peak discharges for 'no dams' and 'with dams' conditions. It can be observed that the existence of the dams result in the following reduction in 1 in 100 AEP design peak flows:

- Nearly 50% at Somerset Dam and Wivenhoe Dam
- Between 29% and 41% at locations along the Brisbane River downstream of Wivenhoe Dam
- 8% at Ipswich

This is all in accordance with what is expected. The observation of the last bullet is explained from the fact that peak flows at Ipswich are only influenced by Moogerah Dam, and not directly by the operation of Somerset Dam and Wivenhoe Dam.

Table 8-3 Comparison between reconciled ‘no dams’ and ‘with dams’ design flows (m^3/s) at key sites

	AEP 1 in 10			AEP 1 in 100			AEP 1 in 1,000		
location	no dams m^3/s	with dams m^3/s	reduction %	no dams m^3/s	with dams m^3/s	reduction %	no dams m^3/s	with dams m^3/s	reduction %
Somerset	2,200	1,300	41%	4,600	2,500	46%	6,700	3,600	46%
Wivenhoe	4,600	930	80%	11,200	6,300	44%	16,800	12,500	26%
Savages Cr.	5,200	2,100	60%	14,300	8,500	41%	21,500	17,500	19%
Mt. Crosby	5,400	2,200	59%	13,800	8,600	38%	21,300	17,100	20%
Ipswich	2,100	2,000	5%	3,900	3,600	8%	5,800	5,600	3%
Moggill	6,400	3,300	48%	14,600	10,200	30%	23,000	18,000	22%
Cent. Bridge	6,200	3,300	47%	14,000	9,900	29%	22,300	17,700	21%
Brisbane	6,200	3,300	47%	13,900	9,900	29%	22,000	17,500	20%

The ability of Somerset Dam and Wivenhoe Dam to mitigate peak flow rates varies the further downstream of Wivenhoe Dam the point of interest is located. Runoff in the downstream tributary streams of Lockyer Creek and the Bremer River can potentially generate large flood events in downstream locations which the mitigation dams will have limited effect on. Reference is made to the WSDOS (Seqwater,2014) results and the sensitivity assessment conducted on the minimum bound scenario conducted using the DEA approach. Refer to the *Design event approach report,* (Aurecon,2015, Refer Appendix G).

8.5.4 Flood volumes

Flow volumes for different durations can be derived directly from the hydrographs that are produced with the MCS and DEA approach for both conditions. Subsequently, frequency curves can be derived in the same manner as frequency curves are derived for peak discharges. Durations of 1, 2 and 3 days have been investigated.

Figure 8-10 presents a comparison between the DEA and MCS flood volumes for the Brisbane River at Moggill for the no-dams condition.

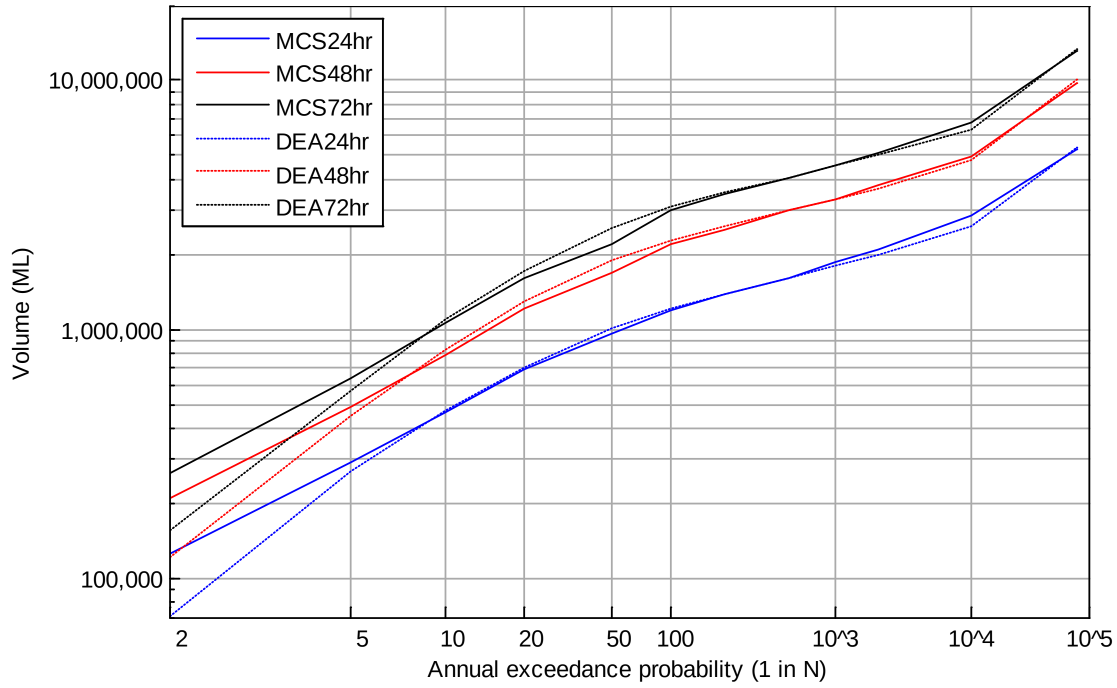


Figure 8-10 Comparison of flood volume frequency curves for Brisbane River at Moggill; MCS versus DEA; no dams conditions

DEA and MCS flow volumes are generally in good agreement. This is no surprise as these volumes are mainly determined by rainfall depth and losses. The DEA and MCS models use the same statistics for (catchment averaged) rainfall depth, and more or less the same loss parameters. For some locations, there are still differences:

- Gatton and Glenore Grove: higher loss parameters were adopted in the MCS framework than in the DEA framework, which results in lower flow volumes for AEP-values in the range 1 in 10 – 1 in 1,000. However, or AEP=1 in 2, the MCS volumes are *higher* than DEA flow volumes
- Woodford: MCS flow volumes are significantly higher than DEA flow volumes for this location
- Location Tinton: MCS flow volumes are significantly lower than DEA flow volumes, especially for AEP=1 in 2
- Locations Kalbar Weir, Amberley, Loamside, Walloon, Ipswich, Savages Crossing, Mount Crosby, Moggill, Centenary Bridge and Brisbane: MCS flow volumes are significantly higher than DEA flow volumes for AEP=1 in 2

Rated flows volumes for 'no-dams conditions' were derived for a small selection of locations (Gregors Creek, Linville and Wallloon) for the purpose of validation of MCS and DEA volume frequency curves. Flows at these three locations can be considered as not being influenced by any of the dams.

Rated daily flow volumes for these locations were derived from historical simulations with the Integrated Quantity Quality Model (IQQM), provided by DSITIA, (DSITIA, 2013). The IQQM simulations were carried out for rainfall conditions for the period 1889 to 30 June 2013 (123 years) for the various locations within the Brisbane River catchment. The simulation scenario adopted is the Water Resource Plan (WRP) pre-development scenario (1889-June 2000) extended to June 2013. Model extension was done using existing rainfall-runoff calibrations and adjusting to recorded flow data. It should be noted that these daily flows were derived using a Sacramento Model (Rainfall-runoff) of the catchment which was calibrated to rated flows derived from available rating curves for the selected gauges. These ratings are different to those adopted in the current study and therefore a potential cause for differences in the estimates obtained from this data.

Mutual differences between design flow volumes of the various methods are consistent with the mutual differences observed for design peak flows. This means DEA/MCS flow volumes are reasonably consistent with FFA results for locations Linville and Gregors Creek, whereas design flow volumes for location Walloon are underestimated by the DEA/MCS simulations. Figure 8-11 shows the comparison between the MCS, DEA and rated flow volumes for the Brisbane River at Gregors Creek for the 24 hour flood volume.

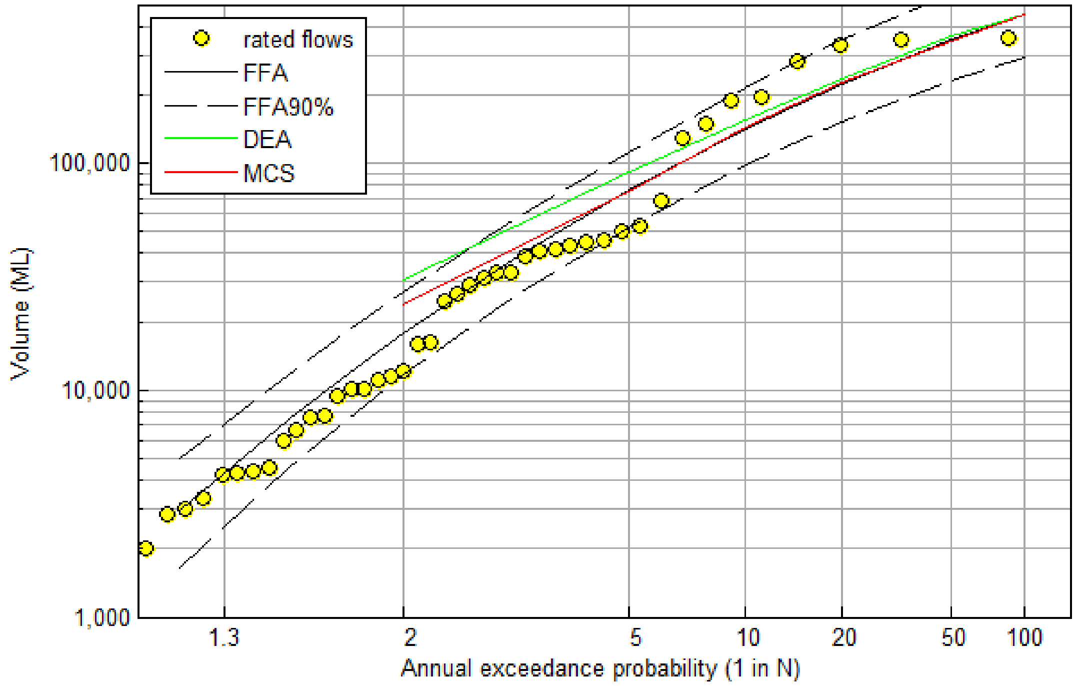


Figure 8-11 Comparison of 24 hour flood volumes Brisbane River at Gregors Creek; no-dams condition

Figure 8-12 presents a comparison between the DEA and MCS flood volumes for the Brisbane River at Moggill for the with-dams condition. Similar to no-dams conditions, DEA and MCS flow volumes are generally in good agreement. This is no surprise as these volumes are mainly determined by rainfall depth and losses. The main difference between both approaches is the fact that MCS uses a stochastic model of initial dam levels, whereas DEA assumes the dams are at full supply level at the start of the event. However, the influence of this difference in approach is relatively minor. For Brisbane River locations downstream of Wivenhoe Dam (and Ipswich as well), the main noticeable difference in results is the fact that MCS flow volumes are significantly higher than DEA flow volumes for AEP=1 in 2. This was also the case for ‘no dams’ conditions.

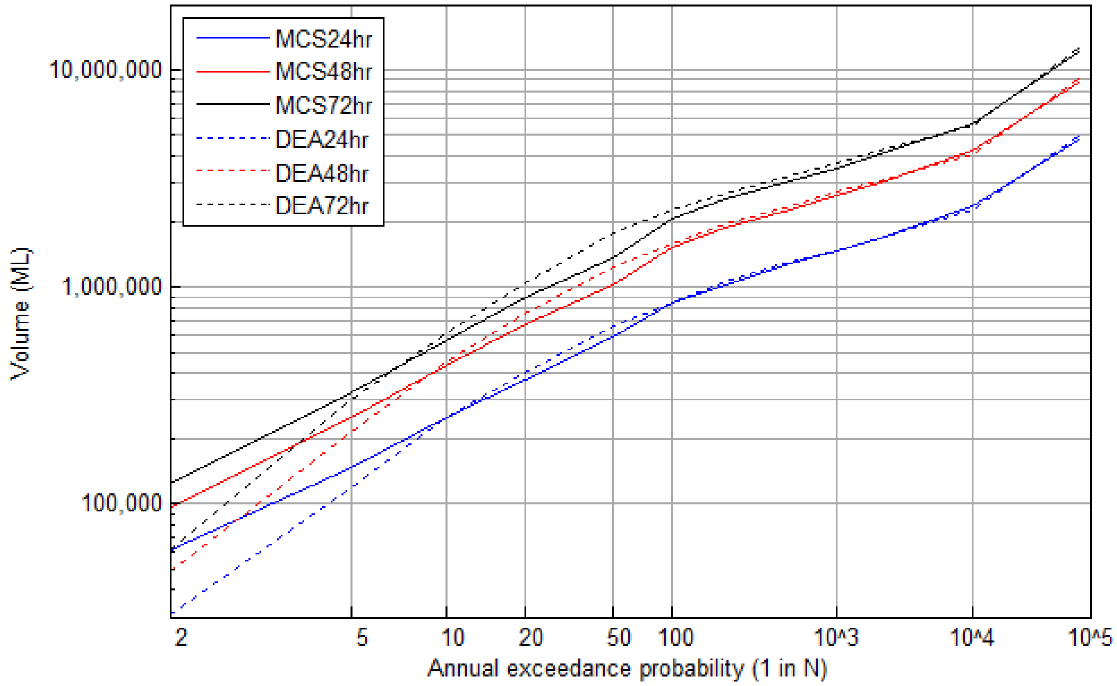


Figure 8-12 Comparison of flood volume frequency curves for Brisbane River at Moggill; MCS versus DEA; with dams conditions

8.5.5 Critical durations

In the Monte Carlo approach, several burst durations have been considered. For a given location and AEP, the duration that results in the highest design flow is referred to as the 'critical duration'. Figure 8-13 shows frequency curves for location Fulham Vale (no dams conditions) for eight durations. It shows that for an AEP of 1 in 2, the longest considered duration of 72 hours is critical. For lower AEP-values, other durations are critical. Durations of 3 hours and 6 hours are not critical for this location and could, in hindsight, have been omitted from the simulations.

Similar Figures have been produced for other locations and also for 'with dams' conditions.

For 'no dams' conditions, the following was observed:

- Burst durations of 3 hours and 6 hours are never critical, except in some cases for PMP conditions
- For AEP's of 1 in 2 and 1 in 5, longer durations are generally more critical. This is due to the fact that short duration / high AEP events have a relatively low rainfall depth. A large proportion of the rainfall does not reach the river system for these events due to initial losses
- For catchments < 1,000 km^2, critical durations are in the range of 12-24 hours
- For catchments between 1,000 km^2 and 5,000 km^2, critical durations are in the range of 18-48 hours
- For catchments between 5,000 km^2 and 10,000 km^2, critical durations are in the range of 36-72 hours
- For catchments >10,000 km^2, critical durations are in the range of 48-96 hours

For 'with dams' conditions, the following was observed:

- Critical durations are generally higher than critical durations for 'no dams' conditions'
- Burst durations of 120 hours are in a substantial number of cases critical

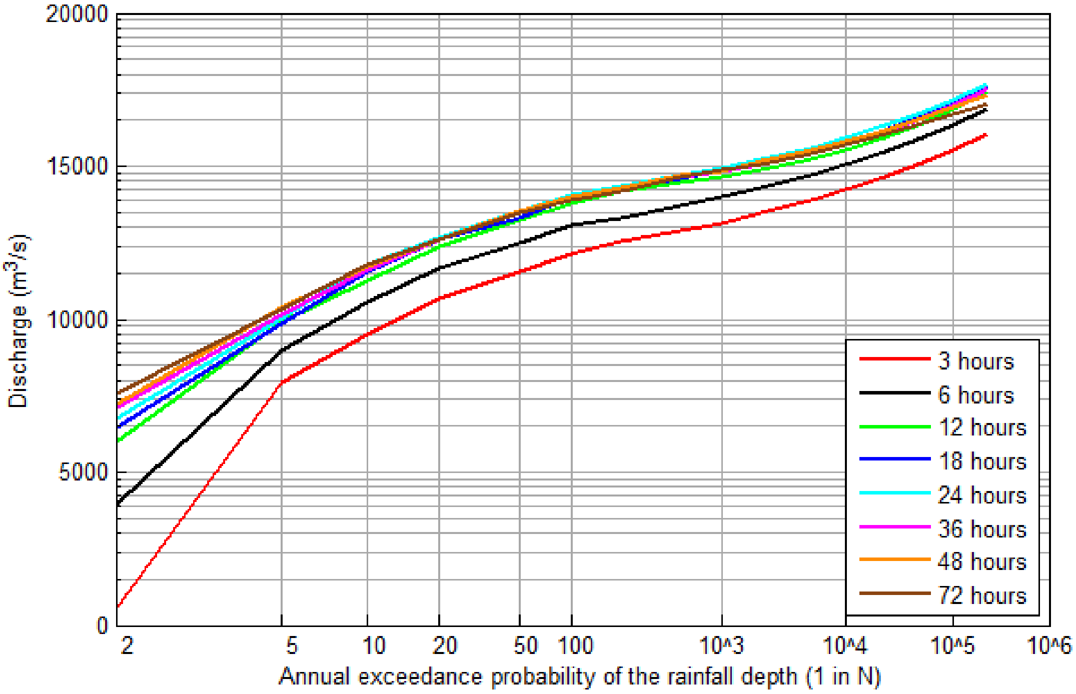


Figure 8-13 Frequency curves for location Fulham Vale for different burst durations, no-dams conditions

The last observation raises the question if durations above 120 hours should have been considered in the MCS simulation runs. Therefore, a sensitivity analysis was carried out in which results were compared of two TPT runs: a run in which burst durations up to 120 hours were considered and run in which burst durations up to 168 hours were considered. Figure 8-14 – Figure 8-17 show resulting frequency curves for peak discharges and 72-hour flow volumes at locations Savages Crossing and Moggill. The Figures show that the influence of the larger durations on the frequency curves is negligible. The applied upper limit in the current study for considered burst durations of 120 hours is therefore considered acceptable.

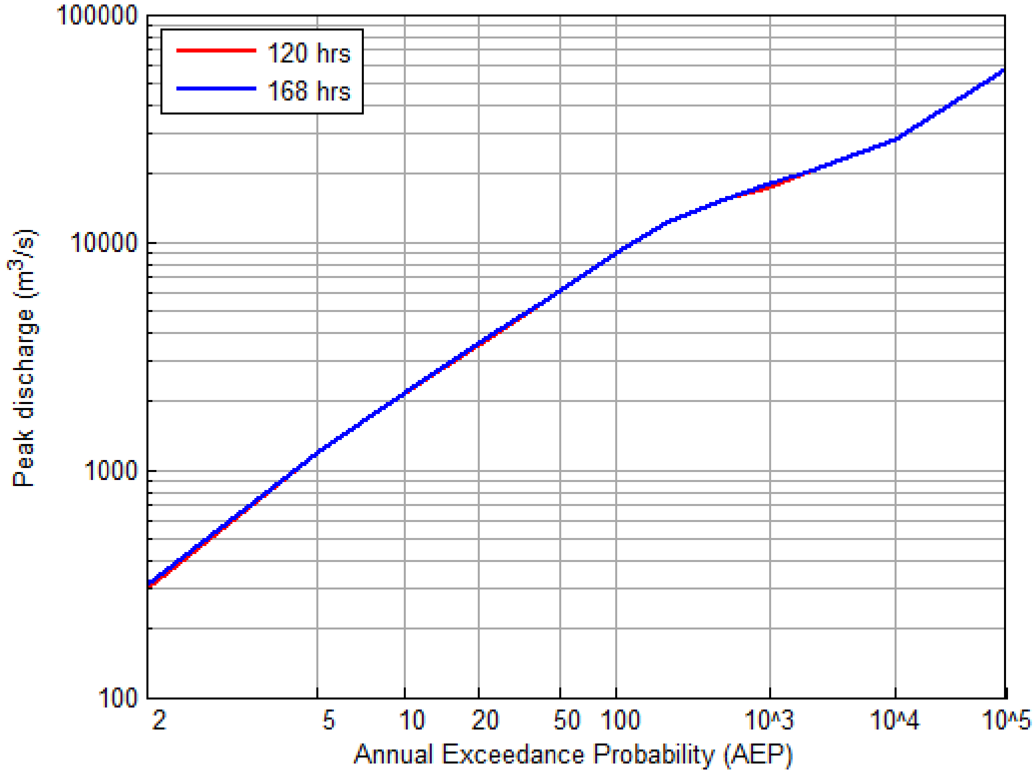


Figure 8-14 Frequency curves of peak discharges at location Savages Crossing; comparison of a TPT run in which burst durations up to 120 hours were considered with a TPT run in which burst durations up to 168 hours were considered

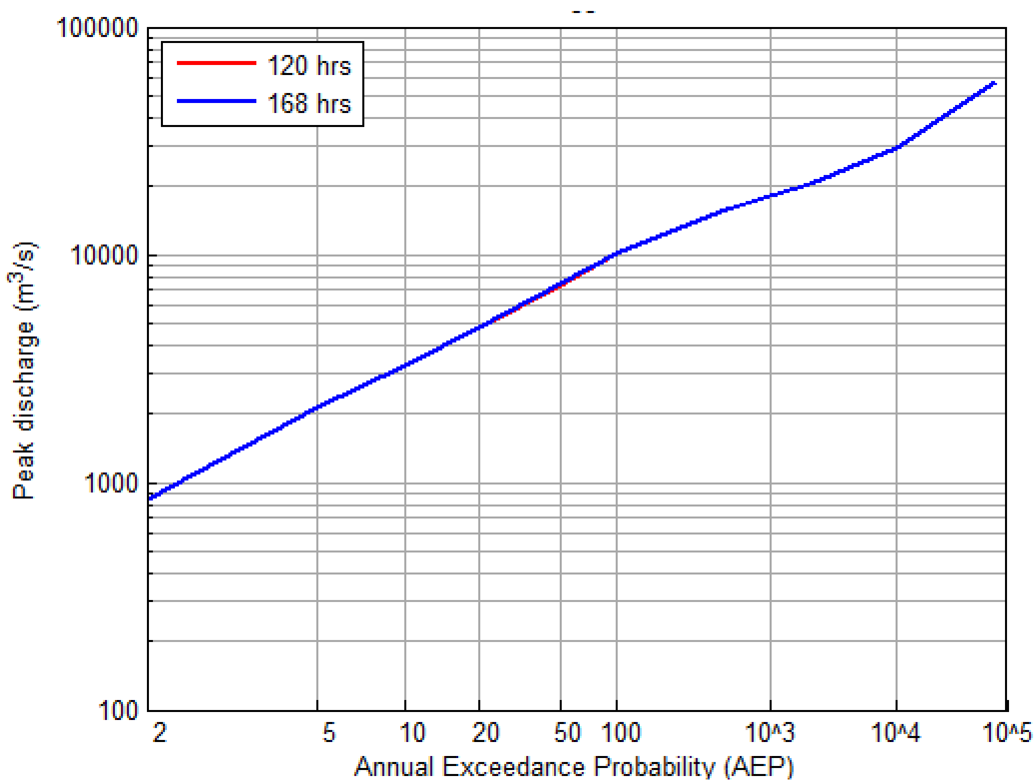


Figure 8-15 Frequency curves of peak discharges at location Moggill; comparison of a TPT run in which burst durations up to 120 hours were considered with a TPT run in which burst durations up to 168 hours were considered

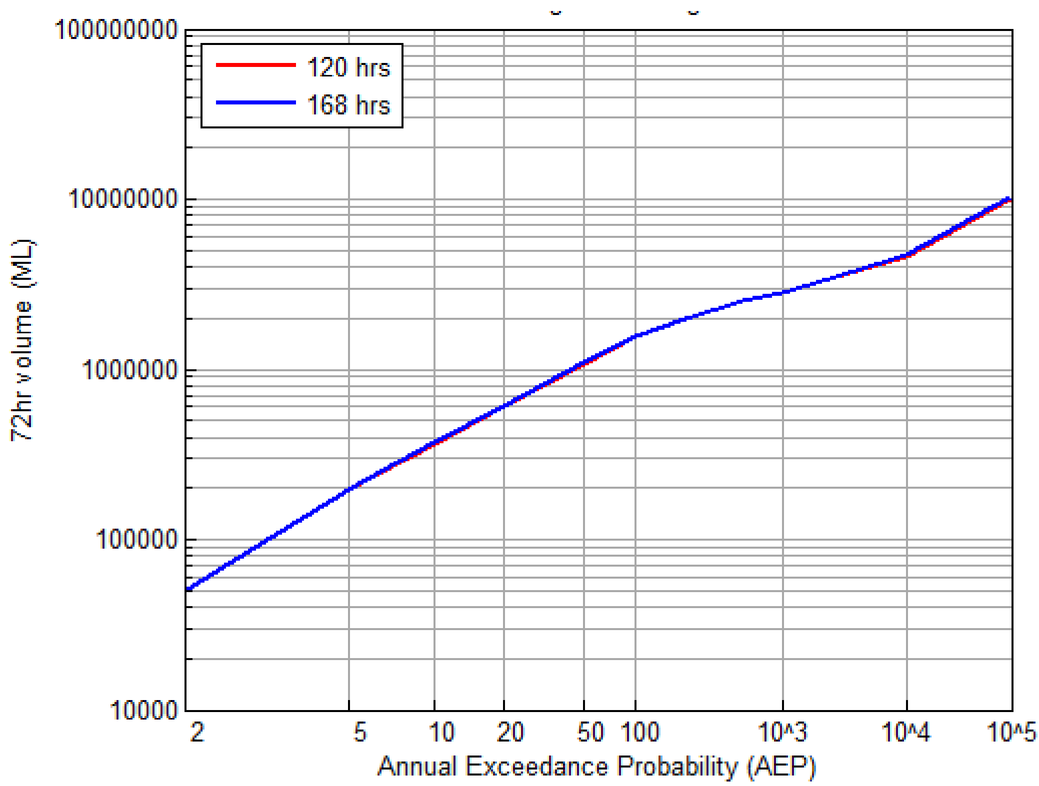


Figure 8-16 Frequency curves of 72 hour flow volumes at location Savages Crossing; comparison of a TPT run in which burst durations up to 120 hours were considered with a TPT run in which burst durations up to 168 hours were considered

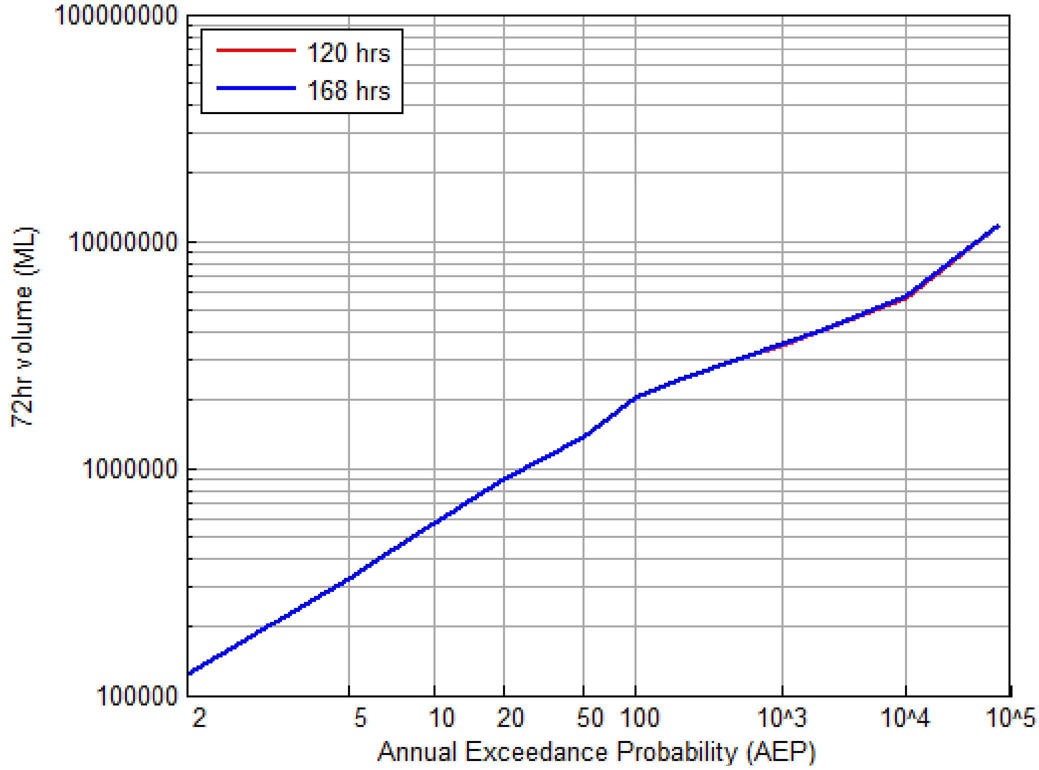


Figure 8-17 Frequency curves of 72 hour flow volumes at location Moggill; comparison of a TPT run in which burst durations up to 120 hours were considered with a TPT run in which burst durations up to 168 hours were considered

8.6 Historical flood assessment

This section describes the methodology used for assigning AEP's to historic events. AEP's have been assigned to historic events at all gauge stations at which both historic flow event analysis has been undertaken and reconciled design flow frequency curves have been derived. The analysis has been undertaken for both 'no-dams' and 'with-dams' conditions.

Table 8-4 provides the best estimate historic event flow (m^3/s) for the 1893, 1974, 1999, 2011 and 2013 flood events, under both 'no-dams' and 'with-dams' conditions. Historic event flows were derived for each gauge station using a range of methods including from several gauged rating series (continuous record, peak record and the rating derived during this study) and URBS modelling, both URBS 'no-dams conditions' and URBS calibration modelling. Table 8-4 below presents the most appropriate estimate historic event flow at each gauge station picked from the range of results obtained.

The 1893 event was prior to any dam construction/influence in the catchment and effectively represents a 'no-dams conditions'. Subsequently no 'with-dams conditions' results are presented for the 1893 event.

For those gauge stations that are located upstream of any dam influence, historic flow estimates under 'no- dams' and 'with-dams conditions' scenarios are the same.

Table 8-4 Peak flow estimate (m^3/s) for major flood events

Location	Jan 1893		Jan 1974		Feb 1999		Jan 2011		Jan 2013	
	No Dams	With Dams	No Dams	With Only SD	No Dams	With Dams	No Dams	With Dams	No Dams	With Dams
Linville	3,420	-	2,500	2,500	2,670	2,670	3,950	3,950	1,780	1,780
Gregors Creek	6,660	-	4,960	4,960	5,510	5,510	6,120	6,120	3,180	3,180
Woodford	2,100	-	990	990	1,130	1,130	1,410	1,410	860	860
Glenore Grove	2,410	-	2,740	2,740	420	420	4,130	4,130	2,870	2,870
Walloon	710	-	2,810	2,810	480	480	2,470	2,470	1,380	1,380
Amberley	1,070	-	2,790	2,280	200	190	960	780	1,960	1,210
Loamside	290	-	760	760	70	70	210	210	320	320
Savages Crossing	18,010	-	11,950	9,870	10,510	1,900	13,630	10,500	8,530	2,220
Mt Crosby Weir	17,740	-	11,880	10,290	9,870	1,840	13,170	9,900	8,000	2,340
Moggill	17,940	-	13,400	12,450	9,290	1,800	13,630	10,880	9,490	3,780
Centenary Bridge	17,440	-	13,090	12,270	8,720	2,130	12,820	10,210	8,820	3,540
Brisbane City	15,830	-	12,950	11,750	8,510	2,140	12,490	9,910	8,630	3,580

Note: SD means Somerset Dam.

Table 8-4 demonstrates that during the February 1999 and January 2013 events the dams had a significant mitigating effect on flood magnitude, with peak flows being reduced by between 60% and 80% downstream of Wivenhoe Dam. This large reduction in peak flows results in higher AEPs under the ‘with-dams conditions’. The actual recorded January 2013 flood event at Moggill had an AEP of around 1 in 15. Under the ‘no-dams conditions’, this flood event would have been much more severe, corresponding to an AEP of 1 in 20. Refer to Figure 8-18.

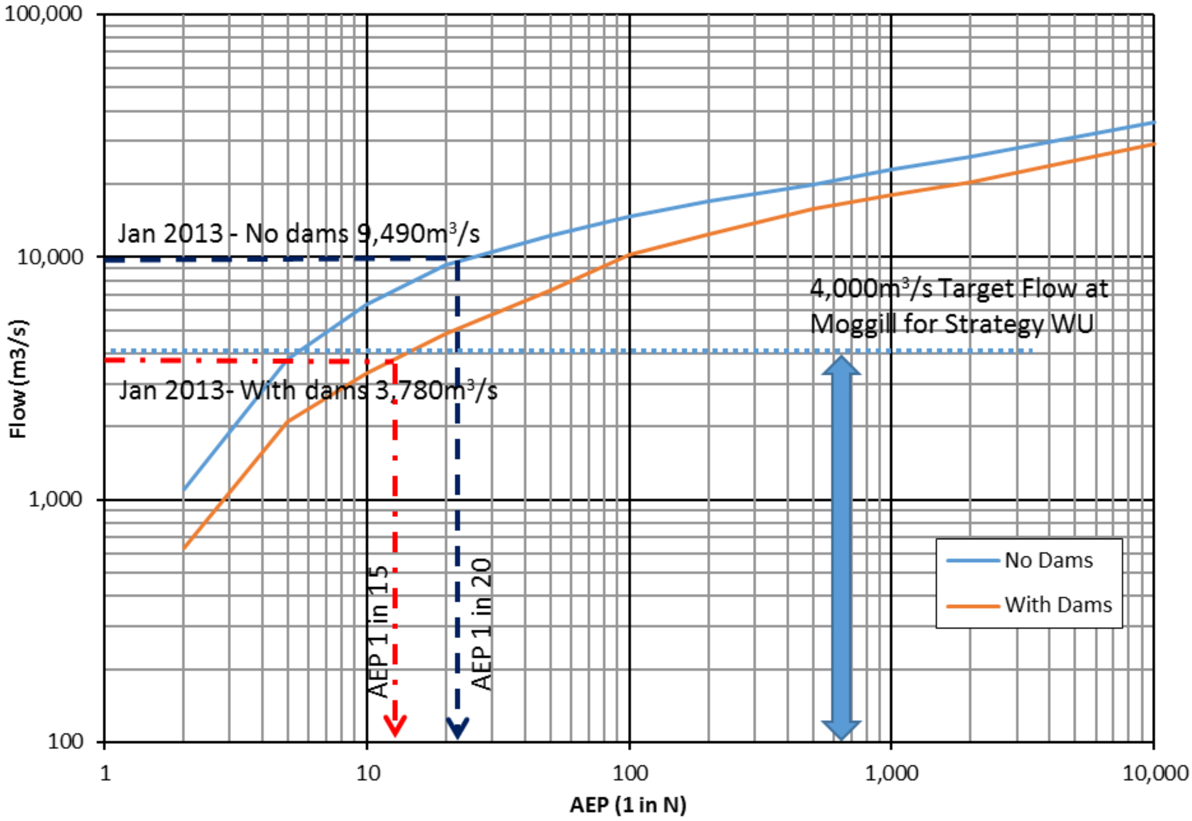


Figure 8-18 Brisbane River at Moggill – AEP of historic flood January 2013

Table 8-4 also shows that for the January 1974 and January 2011 flood events, that at gauge locations downstream of Wivenhoe Dam typically the reduction in peak flows for these events was in the order of 20% to 30%. The estimated AEP’s for these events decrease under the ‘with-dams conditions’. This may initially seem counter intuitive, but it is the result of the flood magnitude and the dam release strategy employed during these events; noting that in January 1974, only Somerset dam was available whilst in January 2011 both Somerset and Wivenhoe dams were operated in conjunction. The estimate for the January 1974 flood AEP is therefore an overestimate for the ‘with-dams conditions’ as the actual flows relate to the operation of Somerset dam only.

For the Brisbane River at Moggill in the January 2011 flood event, the AEP for the ‘no-dams conditions’ is estimated to be around 1 in 85, whereas for the ‘with-dams conditions’ the actual recorded flow has an AEP of about 1 in 125. Refer to Figure 8-19.

This apparent anomaly is due to the shape of the respective flood frequency curves (no-dams conditions versus with-dams conditions) and the relative degree of mitigation achieved by the dams during the 2011 event compared to other possible events of similar magnitude as identified by the Monte-Carlo simulation. The degree of mitigation that is able to be achieved by the dams is a function of the inflow flood magnitude (both peak flow and volume), the initial reservoir level, and the relative contribution of downstream tributaries, and can therefore vary significantly. For example, during the February 1999, Wivenhoe Dam was at only around 65% of its FSL capacity prior to the onset of the flood event, which meant that there was a substantial storage deficit that had to be satisfied before releases could occur. So despite the inflow for this event being larger than the January 1974 and January 2013 flood events, the outcome was that a smaller release was made resulting in substantial mitigation being achieved downstream of Wivenhoe Dam. The impact of initial reservoir levels on resultant peak flows is captured in the MCS framework as evidenced by the variability of estimates. Generally speaking, less mitigation is likely for larger inflow flood events.

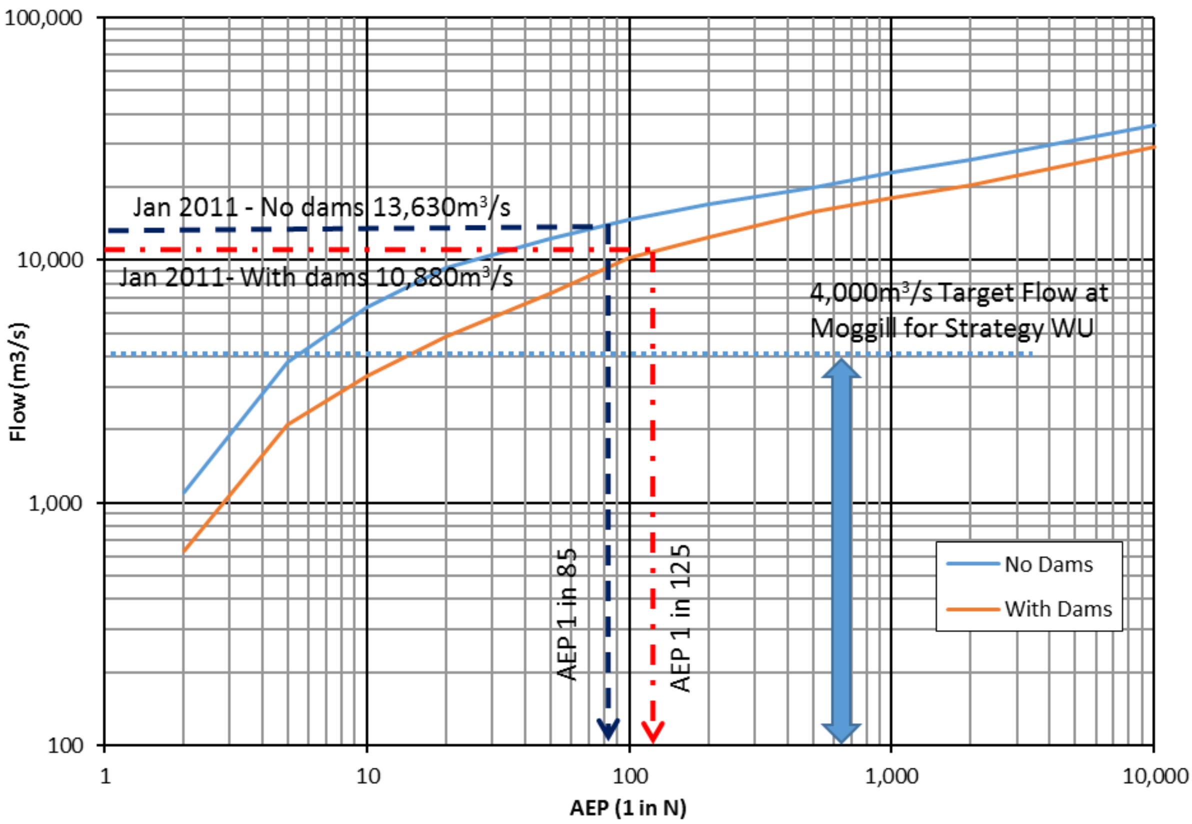


Figure 8-19 Brisbane River at Moggill – AEP of historic flood January 2011

In addition to the 5 major flood events highlighted in Table 8-4 above, there were additional notable historic events in other years at certain gauge stations, often only in single sub-catchments rather than across the whole Brisbane River Catchment. Table 8-5 below provides the best estimate historic event flow (m^3/s) for the other notable flood events. As all locations are not directly influenced by the presence of the dams, the flows represent the no- dams conditions.

Table 8-5 Peak flow estimates (m^3/s) for other notable historic events

Event	Gregors Creek	Glenore Grove	Walloon
1887			1,530
1890		2,755	
1959		3,710	
1971	3,950		

Event	Gregors Creek	Glenore Grove	Walloon
1976			920
1983	5,190		
1989	4,170		
1996			1,060

Flood frequency curves were then derived for each gauge station using the reconciled design peak flows (m^3/s) presented in Table A4 (no-dams conditions) and Table B3 (with-dams conditions) in Appendices A and B respectively of the *Reconciled and recommended flood frequency estimates report* (Aurecon 2015 Refer Appendix I). Historic event AEP's were calculated for given flows. Using the regression equation approach allowed for interpolation of event AEP's between a range of design flow AEP's presented (2, 5, 10, 20, 50, 100, 200, 500, 1,000, 2,000, 10,000).

Table 8-6 below presents the estimated AEP's for the 1893, 1974, 1999, 2011 and 2013 flood events, at key gauge stations under both the 'no- dams' and 'with-dams conditions'.

Table 8-6 Estimated AEP's for the actual 1893, 1974, 1999, 2011 and 2013 event peak flows

Location	AEP (1 in N)				
	Jan 1893	Jan 1974	Feb 1999	Jan 2011	Jan 2013
	No Dams	No Wivenhoe[a]	With dams	With dams	With dams
Linville	80	35	40	140	20
Gregors Creek	115	40	55	80	15
Glenore Grove	30	35	4	110	40
Walloon	5	1000 [b]	3	450 [b]	25
Amberley	10	490	<3	9	45
Loamside	9	410	2	5	10
Savages Crossing	340	55 – 135 [a]	9	155	10
Mt Crosby Weir	350	60 – 150 [a]	8	135	10
Moggill	270	70 – 200 [a]	4	125	15
Centenary Bridge	280	75 – 220 [a]	5	110	10
Brisbane City	175	75 – 185 [a]	5	100	10

Notes: a Wivenhoe Dam was not present at the time of the January 1974 flood event. Therefore the assessment of the no-dams conditions AEP is considered an underestimate and with-dams conditions AEP is considered an overestimate for those locations situated on the Brisbane River downstream of Wivenhoe Dam.

b Gauge rating is considered unreliable during major Bremer and Brisbane River due to likely backwater influence

Table 8-7 below presents the calculated AEP for the other notable historic flood events, at key gauge stations under both the 'no- dams' and 'with-dams conditions'.

Table 8-7 Estimated AEPs for other notable actual flood event peak flows

Location	AEP (1 in N)							
	1887	1890	1959	1971	1976	1983	1989	1996
Gregors Creek				25		50	25	
Glenore Grove		35	80					
Walloon	40				10			10

9 Sensitivity analysis

9.1 Climate change

The Flood Study is based on current and past climate variability. However, given the onset of climate change, it is important that the potential implications of climate change for the flood modelling are understood and that users of the model outputs are aware of the limitations of the model for understanding flooding effects into the future and what key variables might be used if models were to be used to test various climate change affected scenarios. This need was recognised in section 3.6.7.10 of the Invitation to Offer (July 2013) which requested a discussion paper on an ‘Assessment of Implications of Climate Change on Flood Estimation’ be prepared.

A discussion paper, Assessment of Implications of *Assessment of the implications of climate change on flood estimation- discussion paper* (Aurecon, 2015, refer Appendix H) was produced and provides discussion on what the relevant climatic variables are, projected climatic changes for South-East Queensland, and a practical means of incorporating the predicted change into the BRCFS. Importantly, the paper includes a review of what constitutes best practice in accounting for climate change in flood modelling for a range of interstate and international organisations.

This section of the report summarises the discussion paper. The full discussion paper can be found in Appendix H.

9.1.1 Review of climate change science

This section reviewed the current understanding of changes in the hydrological cycle. It did not exhaustively review the observed changes in climate variables, as the overall purpose of the paper was to provide guidance and recommendations for including projected future climate change into flood risk studies. However, in some cases these observed changes are relevant for understanding projected future changes in relevant climatic variables and therefore these are discussed.

9.1.2 Review of best practice incorporation of climate change

The review included all climate parameters known to affect flooding in the Brisbane River catchment: high rainfall, high sea-levels and/or storm surge. Climatic variables that potentially affect rainfall driven flood occurrence were also reviewed and include: rainfall, the antecedent condition of the catchment prior to rainfall, and the preceding river water-levels. Storm surge or high sea-levels can also have a direct influence on flooding at the tail end of the basin.

A review of existing guidance and policy from select interstate (QLD, NSW) and international government bodies (England and Wales, Scotland, The Netherlands and USA) on how they incorporate climate change into flood risk management projects was undertaken.

Whilst the guidance provided by each government body relates to their geographical jurisdiction and is not necessarily applicable elsewhere ie in South-East Queensland, it was useful in identifying the physical parameters that have been amended and the mechanism by which hydrologic and/or hydraulic models have been changed to take account of climate change projections.

9.1.3 Discussion of practical implementation on Brisbane River Catchment Flood Study

Most organisations focus on rainfall depth and sea levels as the two physical parameters with the clearest climate change signals and those most practically altered to take account of future projections. It is important to stress however, that there is considerable uncertainty involved in estimating potential changes in parameters related to rainfall, antecedent conditions, sea-level, and storm surges. Changes in sea-level have a relatively straightforward relationship with global temperature change, as a result of expansion of sea-water.

Changes in rainfall and antecedent conditions are more difficult to estimate. The related processes of rainfall and their rates are poorly captured by low-resolution global circulation models. Although regional climate models have a higher resolution and can simulate rainfall patterns in greater detail, there is no reason to assume that these can provide accurate predictions of possible changes in rainfall either. Also choices between different downscaling techniques are rather a matter of assumption.

However, assuming ranges for changes in rainfall patterns, based on the best currently available knowledge, part of this uncertainty can be captured. When new recommendations become available (eg from the ARR programme), the parameters suggested in this paper can be updated.

There is little information on the potential impacts of climate change on antecedent conditions in Australia and none that could be found for South-East Queensland. Given the potential importance of antecedent conditions on catchment flood conditions it may be sensible for the flood study to undertake a sensitivity analysis, through varying initial loss parameters in hydrological modelling, for example, initial loss parameters could be reduced to simulate a wetter catchment (wetter antecedent conditions) at the start of a rainfall/flood event.

Climate change would also potentially influence initial reservoir levels as the long term runoff from the catchment would be impacted in terms of the quantity of runoff and also the temporal distribution of the runoff. This in turn would impact the probability distribution used to describe the initial reservoir relationships for the various dams. However, the impact of possible climate change would most likely be a secondary effect compared to the assumed reservoir operation rules and change in usage and therefore its impact would be difficult to quantify.

Future patterns of precipitation change from the latest CMIP5 experiments using the RCP8.5 (high climate change) scenario indicate that South-East Queensland may see a slight decline in total annual rainfall of -4.8% (+/-22.1% variation between different climate models), while precipitation during December-February is expected to increase by around 6.8% (+/-22.2%). The Brisbane River catchment flood season correlates strongly with the December to February period, with the majority of the major historical events (including 2013, 2011, 1974, and 1893) occurring in this period, and therefore it might be a reasonable conclusion that rainfall and wetter antecedent conditions during the historically catchment flood prone period will increase.

Continuing sea-level rise will lead to increasing water levels at the tail end of the river basin, especially during storm surge. As the range of projections for sea-level rise is very wide, it is advisable to test the effects of several estimated projections. While offshore sea-level rise may be some 10% higher along the Australian coast, we assume that IPCC (2013) global sea-level rise estimates are valid for the coast of Eastern Australia.

9.1.4 Recommendation

Table 9-1 below provides recommendations for the Brisbane River catchment flood study on the parameters that should be considered for adoption to investigate the potential influence of climate change.

In line with the original brief we have provided clear and practical advice for a range of time horizons.

It is important to keep in mind that considerable uncertainties are involved in these estimates, and in the future these parameters may be (considerably) different from those indicated. However, as these recommendations for climate change parameters should be as practical and clear as possible, four future periods were selected: 2030, 2050, 2070 and 2100. The range of uncertainties is included in the different estimates. For changes in sea-level and storm tide levels, these are depending on the projected rate of global average warming, and response of glaciers and ice caps.

Table 9-1 Proposed climate parameters for inclusion in flood risk studies

Parameter	2030	2050	2070	2100
Design rainfall depth *	+5%	+10%	+15%	+20%
Average sea-level **	+0.09 – 0.21m	+0.17 – 0.38m	+0.19 – 0.60m	+0.26 – 0.82m
Storm tide level ***	2.59 – 2.71m	2.67 – 2.88m	2.69 – 3.10m	2.76 – 3.32m

* Design rainfall depth for flood risk studies, such as described in the AR&R method (see also “Increasing Queensland’s resilience to inland flooding in a changing climate: Final report on the Inland Flooding Study” 2010).Proposed percentage increases are relative to the benchmark year of 2014.

** Estimates for 2050 and 2100 based on the “likely range” of sea-level rise for the periods 2046-2065 and 2081-2100, as reported in the Summary for Policymakers of IPCC 2013. Estimates for 2030 and 2070 were linearly interpolated between the baseline and 2100, as no scenario information is readily available for these time periods from the IPCC AR5 WG1 report. Changes are relative to the benchmark period of 1986-2005.

*** The 100-year storm surge level (currently 2.5m) at Moreton Bay, superimposed on the projected sea-level rise from IPCC.

Further to the advice above, it is recommended initial loss parameters in the hydrological model are altered by +/- 10% to investigate the influence of changes in antecedent conditions of catchment flood risk.

9.2 Alternate dam operations strategy

As part of the WSDOS investigation (DEWS, 2014), alternate dam operations strategies were investigated. The discussion paper published in March 2014 indicated that:

The Government is particularly interested in feedback on the most promising new option, labelled in the report as “Alternative Urban 3”.

The findings of the WSDOS Report suggest this option achieves the best balance of the three key objectives for the dams:

1. *Securing our water supply*
2. *Flood mitigation and*
3. *Safeguarding the dams for very rare, but extremely large, potential floods*

The WSDOS Report evaluates 32 ways of balancing these three objectives and also discusses at length eight different ways of operating the dam during a flood. This discussion paper focuses on the most promising options and also how the dams operate now, detailing what each means, the difference between them, and the impacts if a new option is implemented.

In summary, under Alternative Urban 3, the three strategies are:

- *Water Supply: remains unchanged at 37 percent*
- *Urban Flood Mitigation: is increased from 24 percent to 35 percent (the rural flood mitigation strategy is removed)*
- *Dam Safety: is marginally decreased to 28 percent*

The advantage of Alternative Urban 3 is that more space is available to protect houses and buildings from damage during large floods. This is achieved by increasing the urban flood mitigation space by removing the rural flood mitigation strategy and also taking a small share of the space currently set aside to protect the dam.

Alternative Urban 3 would also introduce, for that small share of the current dam safety space, a new upper limit at Moggill of 6,000 cubic metres of floodwater per second. This new upper limit may delay or avoid much higher and more damaging releases for dam safety. The dam safety strategy would still be used if a larger flood continues to develop.

The dam operations module that has been adopted in this study is currently based on the Loss of Communications (LOC) strategy described in the Revision 11 Flood Manual (Seqwater, 2013).

A new Revision 12 of the Manual was adopted in November 2014. While the LOC emergency procedure in Revision 12 is very similar to that in Revision 11, it is not exactly the same. The instructions in Revision 12 for Wivenhoe dam releases are based solely on dam water level and gate settings and are identical to those in Revision 11. So there are only (minor) differences in LOC procedures between revisions 11 and 12 for Somerset dam:

- Only the sluice gates are used to adjust releases from Somerset Dam into Wivenhoe Dam
- Somerset Dam Headwater level and Wivenhoe Dam Headwater level are used as the basis for decision whether to store or release flood water from Somerset Dam
- This decision making is guided by the Wivenhoe Somerset Interaction diagram in Figure 6.3.1 of Revision 11 and by the Somerset Dam Guide Curve in Figure 6.2.1 of Revision 12
- The Somerset Dam Guide Curve of Revision 12 is broadly similar to the Wivenhoe Somerset Interaction diagram of Revision 11 but it is different in shape

It is conceivable that the history of water level in Wivenhoe Dam could be affected by the changes in Somerset Dam LOC procedure, resulting in some change in the history of releases from Wivenhoe Dam.

The changes to the LOC in Revision 12 are unlikely to have a significant effect, although this has not been quantified. It is recommended to update the LOC dam operations model in RTC tools if/once the dam safety assessment of Somerset Dam and Wivenhoe Dam is completed.

9.2.1 Bias adjustment assessment

The *Dam Operations Module Implementation Report* (Aurecon, 2014 refer Appendix E) describes the representation of the dam operations strategy for flood mitigation dams within the BRCFS. The Dam Operations Module is applied in both the Monte Carlo Simulation (MCS) framework and the Design Event Approach (DEA) to derive 'with-dams conditions' flood frequency curves.

The Dam Operations Module is based upon the Loss of Communications (LOC) emergency flood operation procedure described in the Flood Manual (Seqwater, 2013). This LOC procedure results in peak flows downstream that deviate from the situation where the Full Operation Strategy Model (FOSM) would have been applied. The use of LOC therefore can introduce a bias in the 'with-dams conditions' flood frequency curves which could be adjusted for to account for the systematic bias introduced by the use of a conservative representation. This adjustment is required for the results of both the Monte Carlo Simulation framework and the Design Event Approach.

As a consequence, an investigation was conducted using data developed by Seqwater as part of the WSDOS investigation. The objective was to derive the bias adjustment from GoldSim model simulation results of 3840 synthetic events as provided by Seqwater. However, analysis of these 3,840 simulations and considerations showed that the Seqwater Goldsim modelling that is suitable for many other tasks contains assumptions that make it unsuitable for reliably estimating bias adjustment. Therefore, there is no sound justification for the degree of detail in the bias adjustment. As a result, it was determined that no bias adjustment was applied to the subsequent peak flow frequency curves.

9.3 Minimum Bound Release Scenario

The study brief (DSDIP, 2013) included a requirement for the absolute minimum bound flood frequency curves to be derived. Refer to section 3.6.7.4, which is reproduced below:

> *A check should be undertaken of the absolute minimum bound of the flood frequency curve, by generating modelled flood estimates assuming no-outflow from Wivenhoe Dam. This minimum bound scenario is the magnitude of design floods that would occur in Brisbane and Ipswich with contribution limited to only the catchments of tributaries and the Brisbane River downstream of Wivenhoe Dam. It should be noted that this does not represent a realistic case because in many floods there will be necessary flood releases in Wivenhoe Dam. However, this analysis will be important to define the absolute lower limit of the flood frequency curve and to check that the derived 'with-dams' flood frequency curve that includes simulation of the dam operations should be above the lower limit.*

Peak flows in the Brisbane River downstream of Wivenhoe with no release from Wivenhoe Dam were investigated using the DEA and are discussed in Section 6.2.8. In the mid-Brisbane River between the Lockyer Creek and Bremer River confluences, removal of the Wivenhoe Dam release reduces flows by up to 90% at Savages Crossing for the 1 in 2 AEP event, decreasing to around 60% at PMF. Downstream of the Bremer River confluence the exclusion of Wivenhoe Dam release typically reduces peak flows by 50% to 60%.

It should be recognised that given the current configuration of the dams, there is a finite capacity to limit the release of floodwaters downstream. For example, the current capacity of Wivenhoe Dam is insufficient to capture all of the 1 in 100 AEP flood volume. Therefore, whilst this scenario testing provides an indication of the minimum bound scenario, it is not feasible to operate the dams in this manner.

9.4 Initial Reservoir Level at FSL

In the DEA approach, starting water levels of the dams are assumed to be at full supply level (FSL). This is an arbitrary choice, likely to be on the conservative side, i.e. resulting in increased design flow estimates. The MCS approach method has the advantage over the DEA approach that the variability of starting water levels of the dams are explicitly taken into account by modelling these water levels as stochastic variables.

The flexibility of the MCS method is such that starting dam water levels can also be assumed equal to full supply level. The MCS framework was therefore used to assess the influence of starting dam water levels (stochastic versus full supply level) on design flows in a sensitivity analysis. Design discharges of the following three alternatives were compared:

1. Stochastic starting levels
2. Starting levels at FSL

3. Starting levels at 50% of FSL – volume

Note: the starting levels are implemented equally for all dams. So, in case of option 2, starting water levels of all dams are at FSL.

Figure 9-1 – Figure 9-7 show the resulting frequency curves for locations Wivenhoe Dam, Savages Crossing, Mount Crosby, Ipswich, Moggill, Centenary Bridge and Brisbane. Frequency curves of the 'no dams' case were added to put results in perspective. The following is observed:

- For all locations and AEP's, design flows of the stochastic starting levels are lower than design flows of the FSL starting levels and higher than design flows of the 50%FSL starting levels
- Relative differences are smallest for location Ipswich, because Ipswich design flows are only influenced by Moogerah dam, not by Wivenhoe Dam and Somerset Dam
- Relative differences in design flows are maximum at Wivenhoe Dam, and decrease further downstream along the Brisbane River
- For Brisbane River locations downstream of Wivenhoe Dam, mutual differences between the three 'with dams' options are relatively small compared to differences with the 'no dams' option
- The influence of the choice of water levels is still visible for extreme events (AEP ~1 in 10,000)

The last observation may come somewhat as a surprise, as initial water levels are generally considered to be mainly relevant for more frequent events. However, the scatter plots of Figure 9-8 – Figure 9-13 confirm that this is not the case. These Figures compare derived peak discharges for the case in which starting water levels are assumed to be at FSL (horizontal axis) with the case in which starting water levels are assumed to be at 50% of FSL-volume (vertical axis). It shows that differences in peak flows can be substantial, even for extreme events. For example, events in which the FSL starting water level results in peak discharges of ~60,000 m^3/s at Moggill, may result in peak discharges of ~50,000 m^3/s at Moggill if the starting water level is equal to 50% of FSL volume, which is a substantial difference. Figure 9-8 clearly demonstrates that an increase in starting dam levels may contribute to the breach of a Wivenhoe Dam fuse plug, which automatically results in a significant increase in peak discharges.

Figure 9-14 shows an example of an event for which differences in peak discharge at location Savages Crossing between 50%FSL and 100%FSL starting levels are relatively large. The difference in flow volumes of the two hydrographs is approximately equal to 50% of the FSL volume of Wivenhoe dam, which shows the additional 50%FSL storage indeed causes the strong reduction in peak flows downstream.

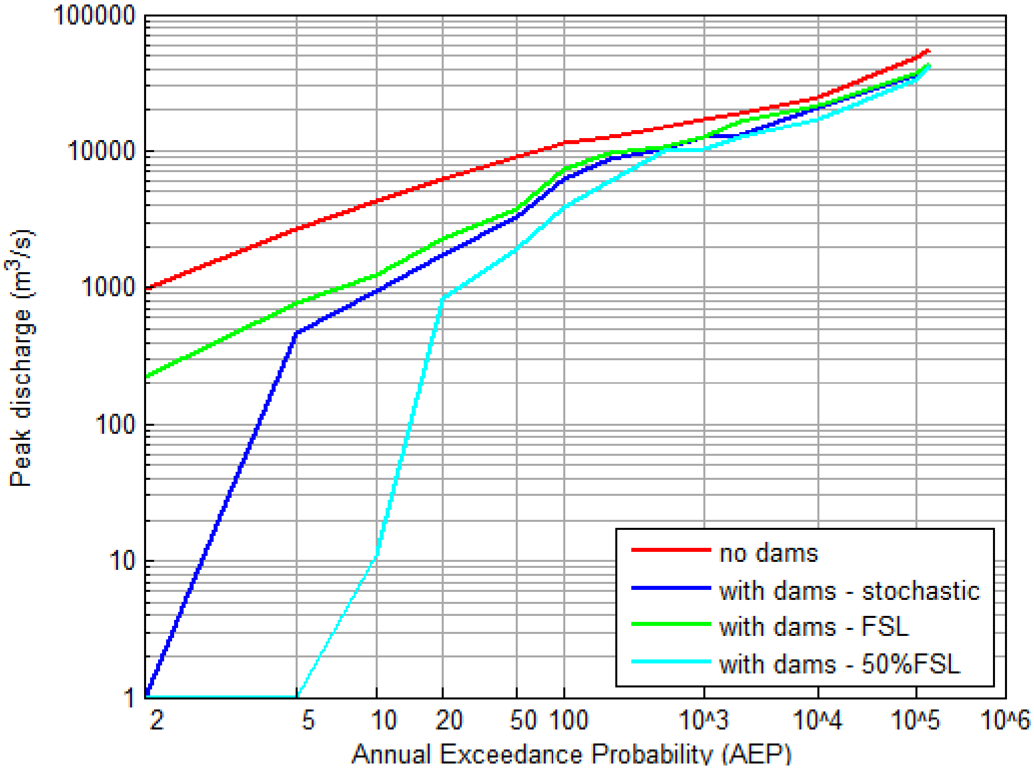


Figure 9-1 Frequency curves for location Wivenhoe dam; 'no dams' case versus three different options for the starting level of the 'with dams' case

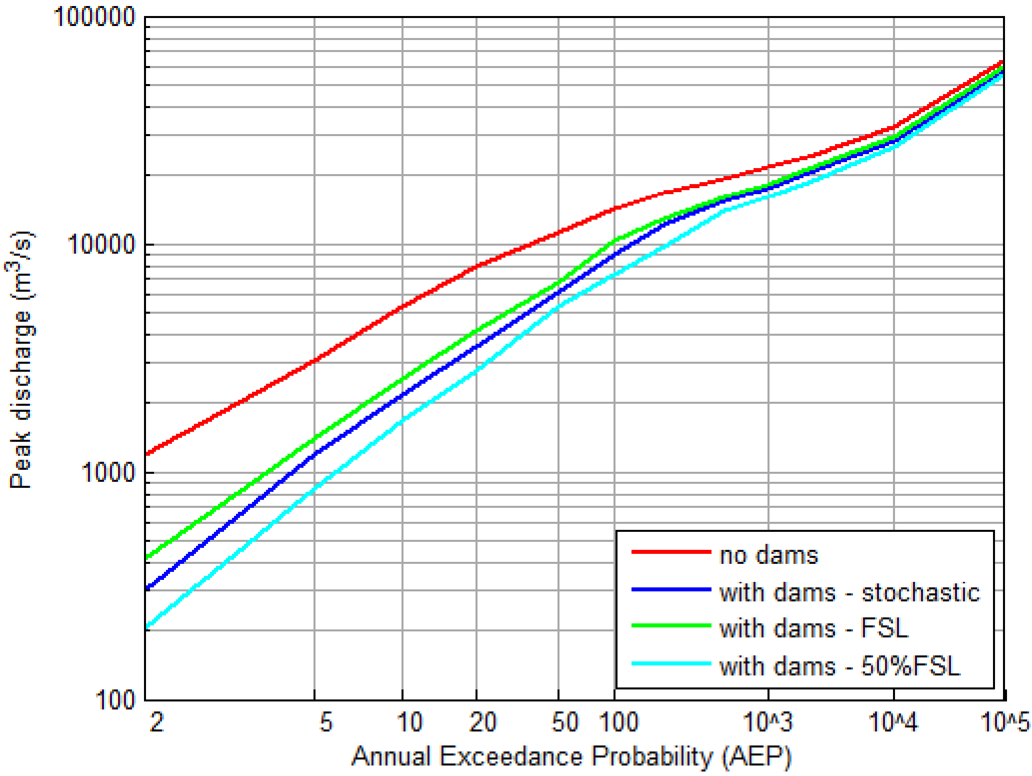


Figure 9-2 Frequency curves for location Savages Crossing; 'no dams' case versus three different options for the starting level of the 'with dams' case

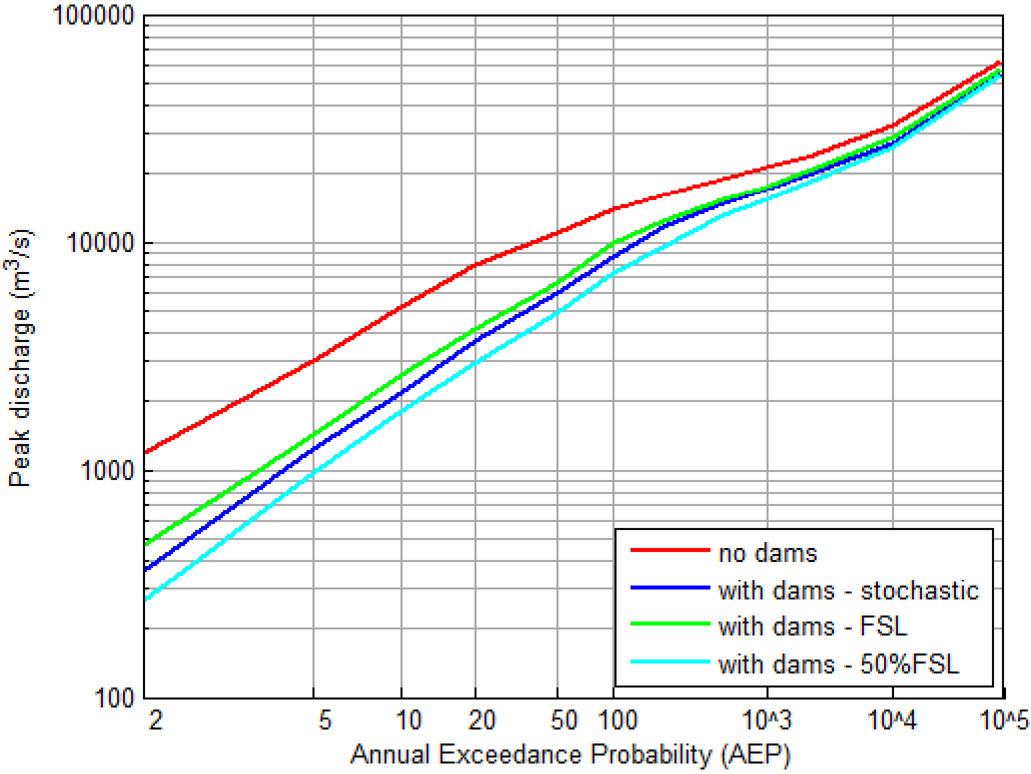


Figure 9-3 Frequency curves for location Mount Crosby; 'no dams' case versus three different options for the starting level of the 'with dams' case

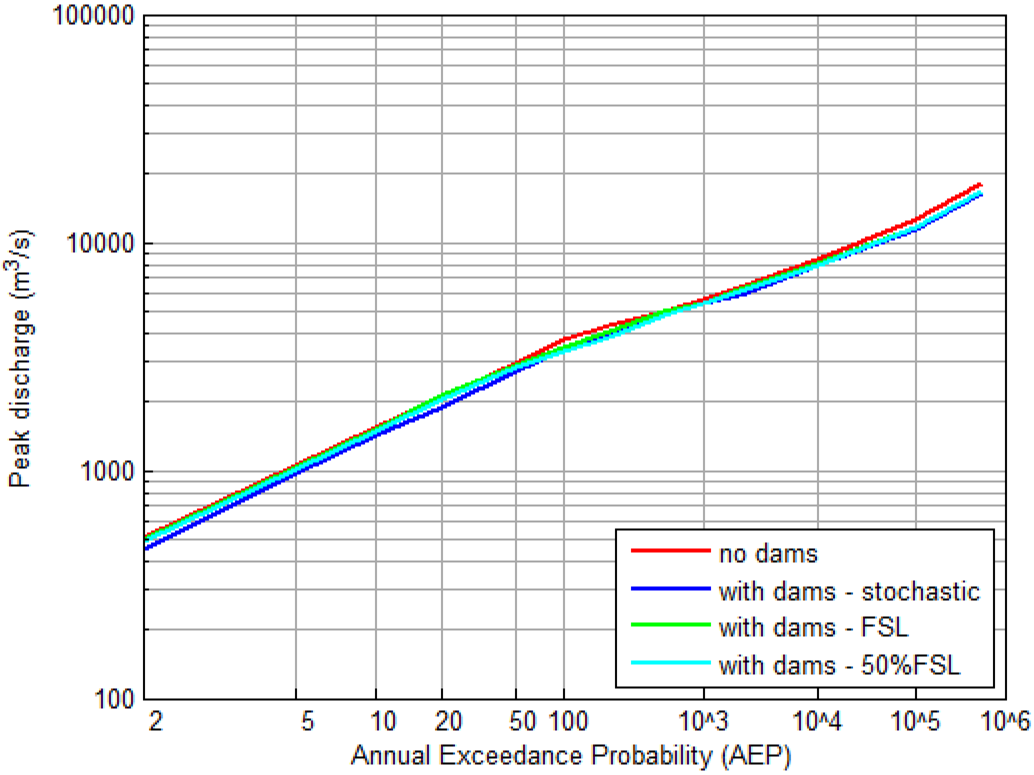


Figure 9-4 Frequency curves for location Ipswich; 'no dams' case versus three different options for the starting level of the 'with dams' case

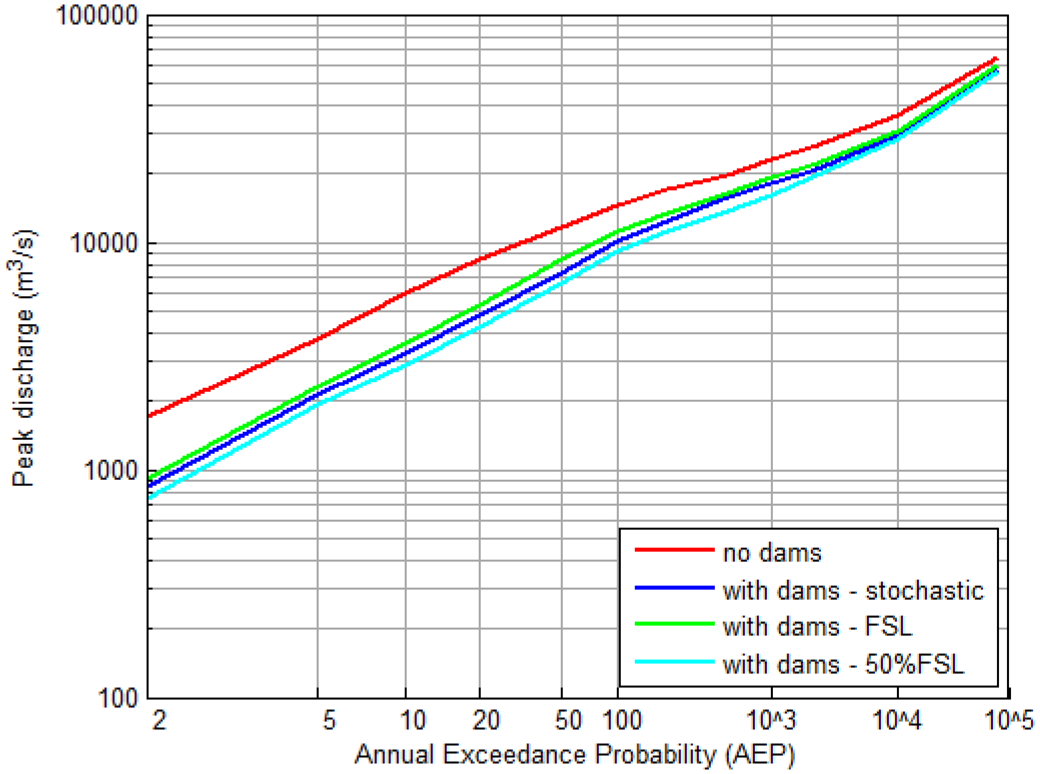


Figure 9-5 Frequency curves for location Moggill; 'no dams' case versus three different options for the starting level of the 'with dams' case

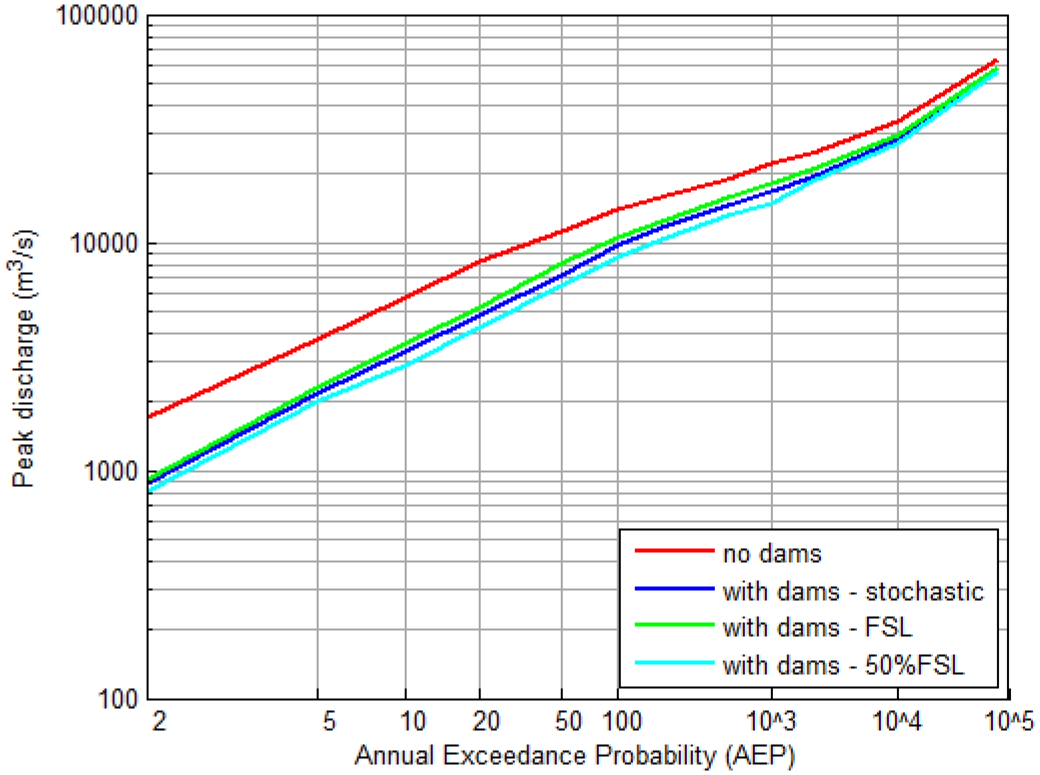


Figure 9-6 Frequency curves for location Centenary Bridge; 'no dams' case versus three different options for the starting level of the 'with dams' case

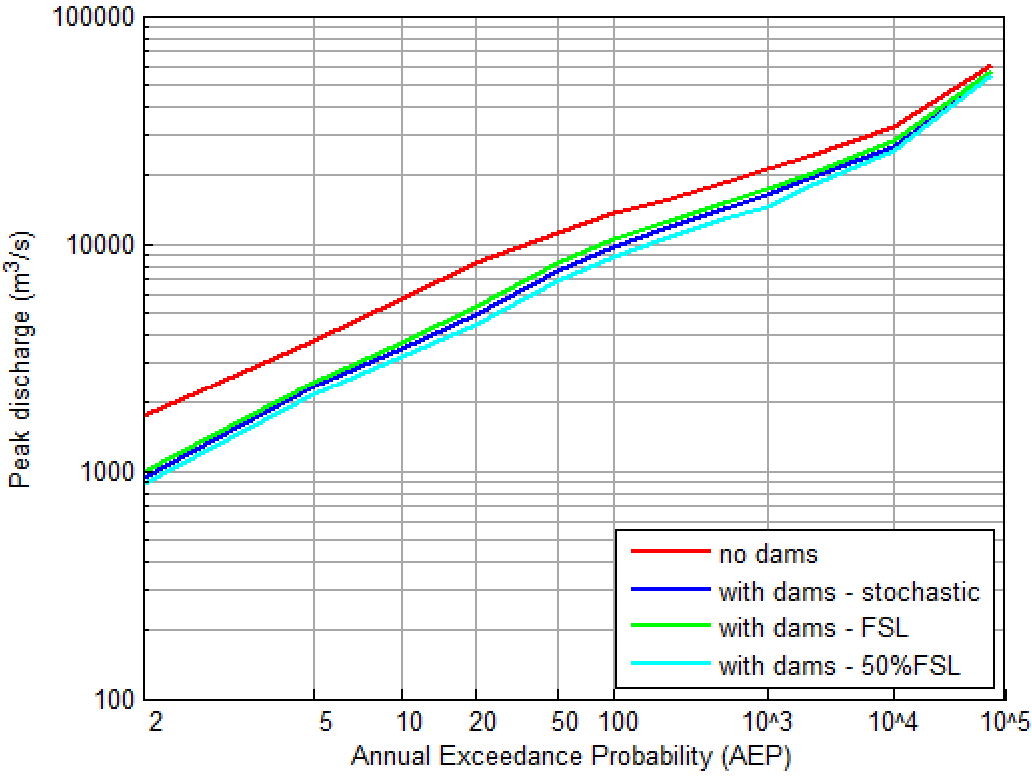


Figure 9-7 Frequency curves for location Brisbane; 'no dams' case versus three different options for the starting level of the 'with dams' case

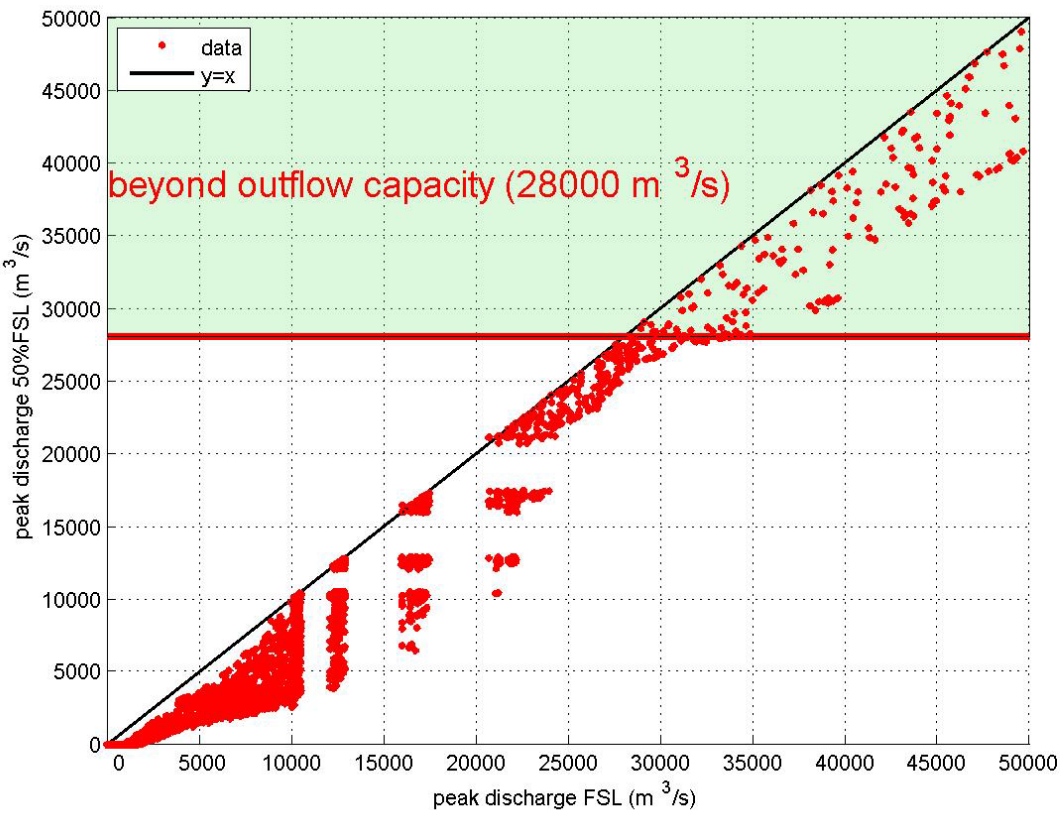


Figure 9-8 Comparison of peak flows at Wivenhoe Dam for two options for the starting levels: FSL versus 50% FSL

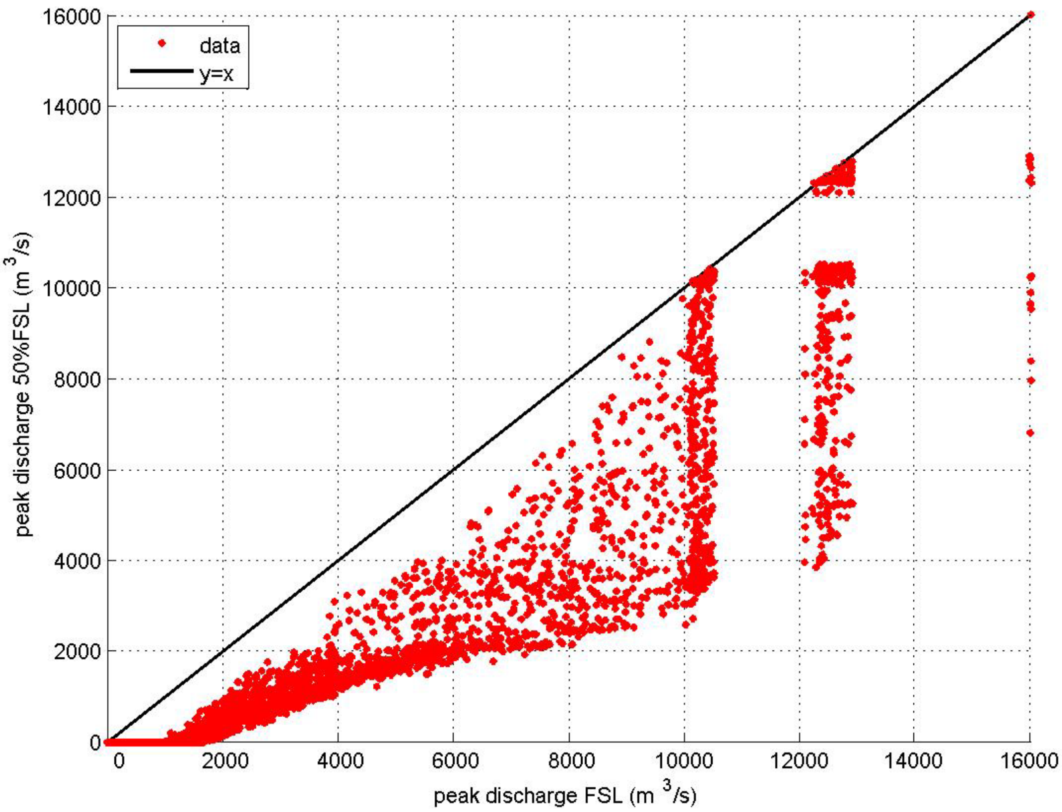


Figure 9-9 Zoomed version of Figure 9-8

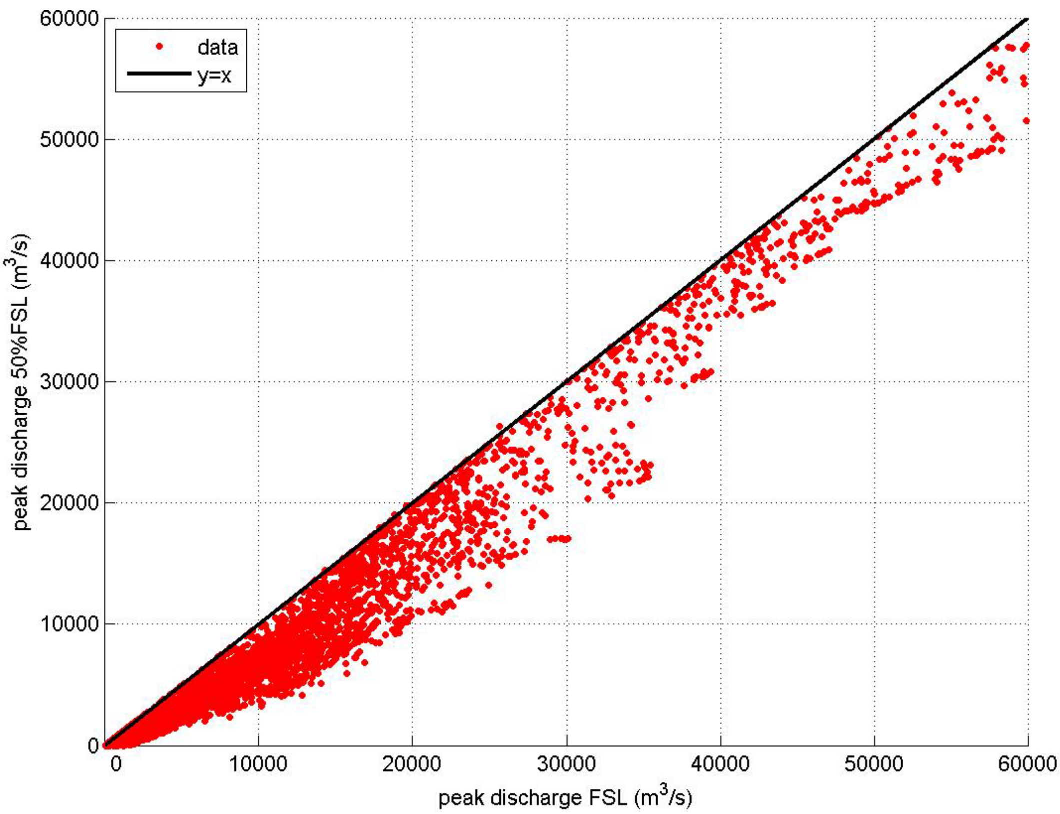


Figure 9-10 Comparison of peak flows at Savages Crossing for two options for the starting levels: FSL versus 50% FSL

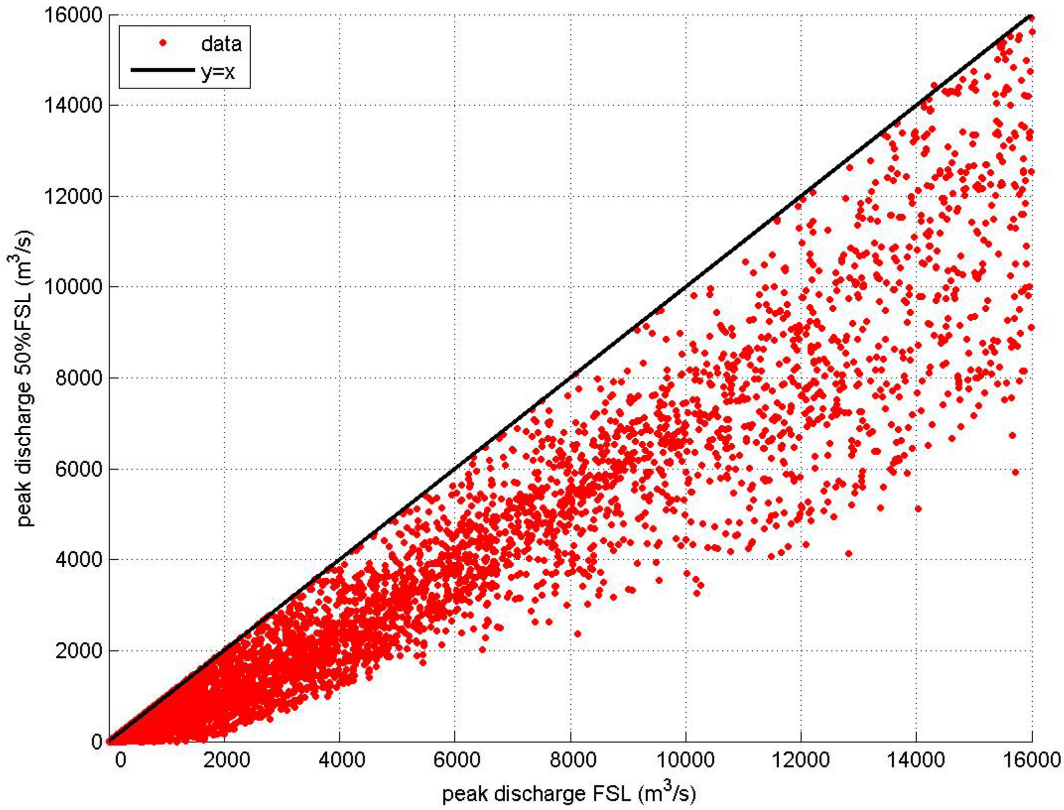


Figure 9-11 Zoomed version of Figure 9-10

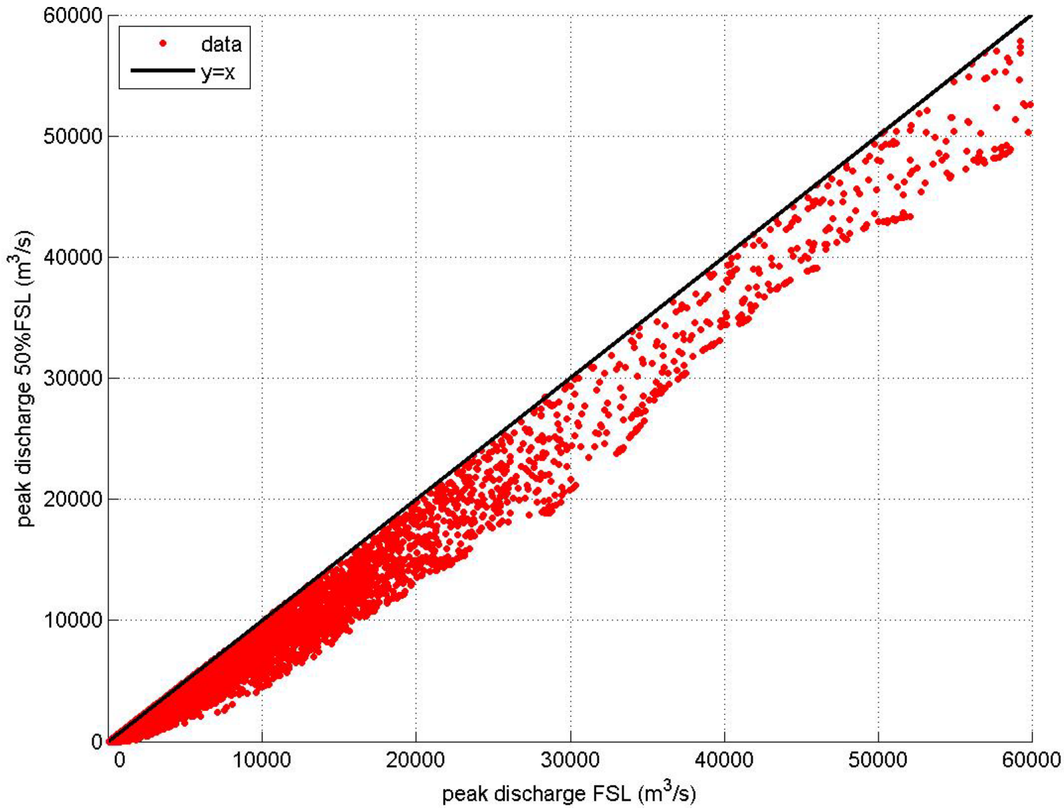


Figure 9-12 Comparison of peak flows at Moggill for two options for the starting levels: FSL versus 50% FSL

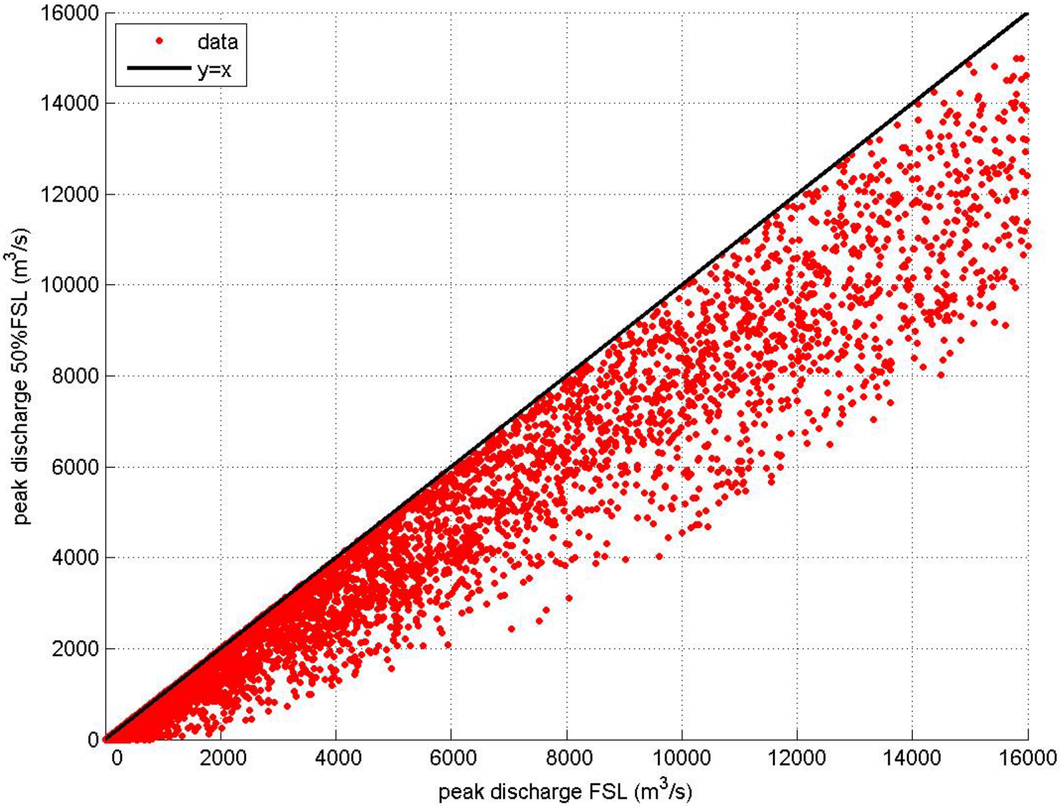


Figure 9-13 Zoomed version of Figure 9-12

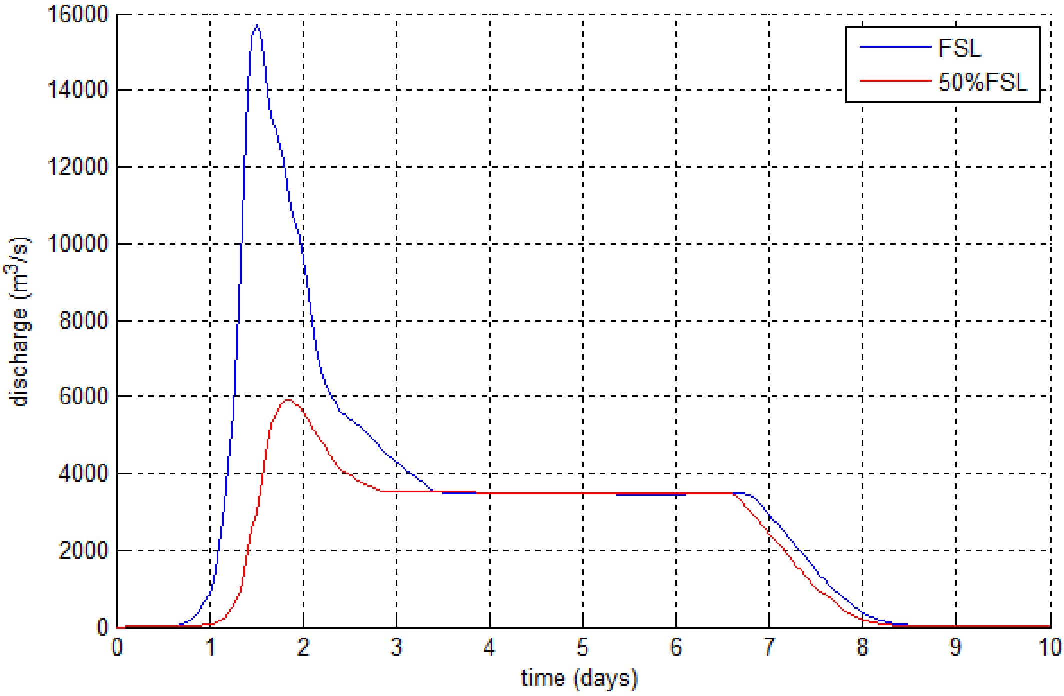


Figure 9-14 Flow hydrographs of an event for which differences in peak discharge between 50%FSL and 100% FSL starting levels are relatively large; location Savages Crossing

.

10 Conclusions

10.1 Key findings

10.1.1 Data and rating curve review

The rating curve review noted that it is considered important to achieve consistency between all stages of the BRCFS. The rating curve development, including the hydraulic modelling conducted for the primary gauges, will need to be continually reviewed during the ongoing phases of the overall study. The results and ratings presented are currently only the interim stage of the rating review. They are based on the best available data but must continue to be updated as the quantity and quality of the available data improves, including:

- During calibration of the hydraulic models (including future stages of the BRCFS) to ensure consistency between the hydrologic model results and the hydraulic modelling used for the primary rating curves, and to update the model outputs used for development of the secondary rating curves based solely on hydrologic model results
- During flood frequency analysis to provide (unless otherwise justified) a consistency of response between gauges across the Brisbane River catchment

Realistically, the ratings (and other components of the hydrologic analysis) should also continue to be reviewed throughout the hydraulic modelling and subsequent stages of the BRCFS, and ongoing gauging and refinement of the ratings should be continued as opportunity arises to continue improving the definition and confidence in the ratings for use in flood operations.

10.1.2 Hydrologic model review and recalibration

The recalibration process has been carried out following a similar methodology to the Seqwater calibration process wherever possible, (refer Seqwater, 2013). The primary recalibration of the model parameters focussed on five key events of varying sizes for which the most reliable data was available: January 1974, May 1996, February 1999, January 2011 and January 2013. Although a number of larger floods were recorded in the late 1800's, such as January 1893 and January 1898, the lack of temporal definition of the pre-1955 events mean they are not suitable for use as calibration events.

A revised set of recommended parameters was calculated from the results for these five events, and were then been applied to all 38 of the post-1955 calibration events and 10 pre-1955 validation events used by Seqwater. A comparison of the model results was performed using the recommended parameters.

A number of changes to the models were recommended as part of the *Hydrologic Model Calibration and Validation Review* (Aurecon 2015, refer Appendix B), listed in Table 10-1. The recalibration process has seen further modifications to a number of the models as follows:

- Stanley River model: the reporting location for the Woodford gauge was moved to the downstream junction to represent total flows through this area, as the hydraulic model of this area indicated that this was a more appropriate location
- Lockyer Creek model: the schematisation of the lower Lockyer Creek was modified to include the main channel and three separate bypass locations. This was felt to be an appropriate schematisation for this reach where the main channel is perched and the breakout flows travel slowly through the floodplain
- Lower Brisbane model: Calibration parameters alpha and beta were set to typical values for the local tributaries. Main channel routing time was reduced by applying a reach length scaling factor. Storage-discharge relationships used in the conceptual storages have been related directly to physical properties of the river and floodplain by combining level-volume relationships taken from DTM with level-flow relationships estimated from the main gauge rating curves

Table 10-1 Adopted URBS model changes

	Stanley	Upper Brisbane	Lockyer	Bremer	Warrill	Purga	Lower Brisbane
Include revised rating curves	✓	✓	✓	✓	✓	✓	✓
Include channel routing non-linearity (n = 0.85)			✓	✓		✓	
Remove Kedron Brook catchment (Seqwater subareas 111, 113, 97, 99 and105)							✓
Include impervious fractions, urbanised areas and reduced reach length factors							✓
Modification of conceptual storage volumes (based on physical storage characteristics)		✓					✓

The subareas representing Kedron Brook were removed from the Lower Brisbane River subcatchment model as Kedron Brook does not join the Brisbane River. The removal of Kedron Brook has minimal impact on the estimated flows as it was connected downstream of the last point of interest, the Brisbane River at Brisbane City Gauge.

The recalibration process has generally seen an either an improved or equivalent quality of calibration for all catchments when compared to the Seqwater results. Calibration results were similar in the Stanley River subcatchment and were slightly improved in the Upper Brisbane subcatchment, especially for the 1999, 2011 and 2013 events. Calibration in the Lockyer Creek subcatchment is difficult to assess given the uncertainty associated with the rating curves, especially in the higher flow range for the stream gauges situated in the lower reaches of this catchment, however the calibration at Glenore Grove is improved across all events. Overall, calibration in the Bremer River and Warrill Creek subcatchments was similar and a slight improvement in calibration was achieved in the Purga Creek subcatchment. Calibration in the Lower Brisbane model was notably improved for most events, particularly with respect to the timing of flow routing along the river.

What was also clearly identified in the review was the need for channel routing and conceptual storage parameters to be reassessed during the hydraulic modelling phase of the BRCFS. This means that a further iteration of calibration may need to occur once a fully calibrated hydraulic model of the Lower Brisbane River becomes available should it demonstrate that the adopted rating curves and routing characteristics are inconsistent. This exercise is considered part of a continuous improvement approach which may also include reviews after every major flood event.

The recommended alpha and beta parameters remain similar for the Upper and Lower Brisbane river subcatchments where the only changes to the models were to rating curves, conceptual storages and schematisation of the lower reaches in the Lower Brisbane model. In the Stanley River where the model was modified around the Woodford gauge, the alpha value was reduced and the beta value was increased. In the models where channel routing non-linearity was introduced (Lockyer, Bremer and Purga), alpha values were increased to obtain a reasonable calibration and beta values were modified as required. In the Warrill Creek model, where only the rating curves were modified, the alpha value was slightly increased and the beta value was decreased.

The recommended model parameters for each sub-catchment model are shown in Table 10-2.

Table 10-2 Recommended model parameters

Sub-catchment	Alpha	Beta	m	n
Stanley River	0.11	5.7	0.8	1.0
Upper Brisbane River	0.12	2.8	0.8	1.0
Lockyer Creek	0.49	3.1	0.8	0.85
Bremer River	0.79	2.8	0.8	0.85
Warrill Creek	0.79	2.5	0.8	0.85
Purga Creek	0.93	3.8	0.8	0.85
Lower Brisbane River	0.30[a]	4.0	0.8	1.0

Notes: (a) Reach length factor of 0.2 applied to main channel reach lengths
Alpha = channel routing lag parameter
Beta = catchment lag parameter
m = catchment non-linearity parameter
n = channel routing - Muskingum non-linearity routing parameter

When comparing model results from the recommended parameters runs across the full range of verification events, all of the examined flow gauges generally show a good correlation between calculated and rated peak flow rates and event volumes with no obvious flow rate related bias.

10.1.3 Flood frequency analysis

In accordance with Section 3.6.6.1 of the Brisbane River Catchment Flood Study (BRCFS) brief, flood frequency analyses were undertaken to predict design flows, and volumes for design events in the Brisbane River Catchment. The analyses were conducted on sites that were considered to have sufficient quality record that could be used to construct stationary series suitable for inclusion in the assessment.

A preliminary assessment of peak flows of ten primary gauges was undertaken. The resulting flood frequency curves are shown in Figure 10-1 and display significant variation in magnitude, slope and curvature. Regional analysis was conducted to determine catchment weighting parameters. The catchment weighted frequency curves are shown in Figure 10-2 and display much greater consistency of curvature and slope while still maintaining individuality that is consistent with known behaviour of the catchments.

Figure 10-3 shows peak flow estimates for 1 in 2, 10 and 100 AEP from the catchment weighted frequency curves are shown as function of catchment area. These peak flows generally show an increasing trend with catchment area as expected, and although there is significant variation, consistent trends can be observed within and between catchments. Bremer River gauges consistently exhibit the highest flows and the Lockyer Creek catchments the lowest, which is consistent with rainfall intensity and loss characteristics of the catchments.

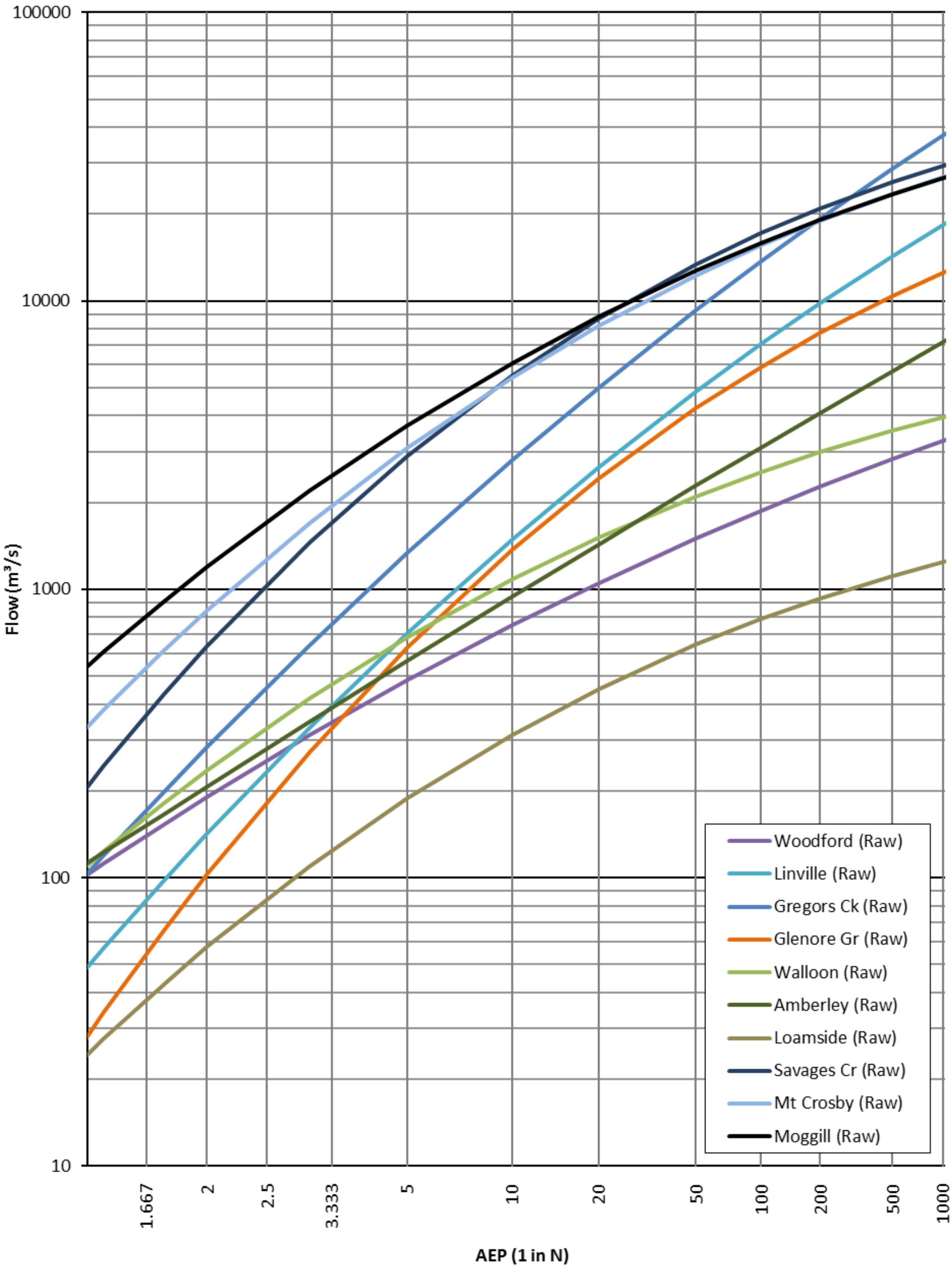


Figure 10-1 Raw (unweighted) flood frequency curves at primary gauges

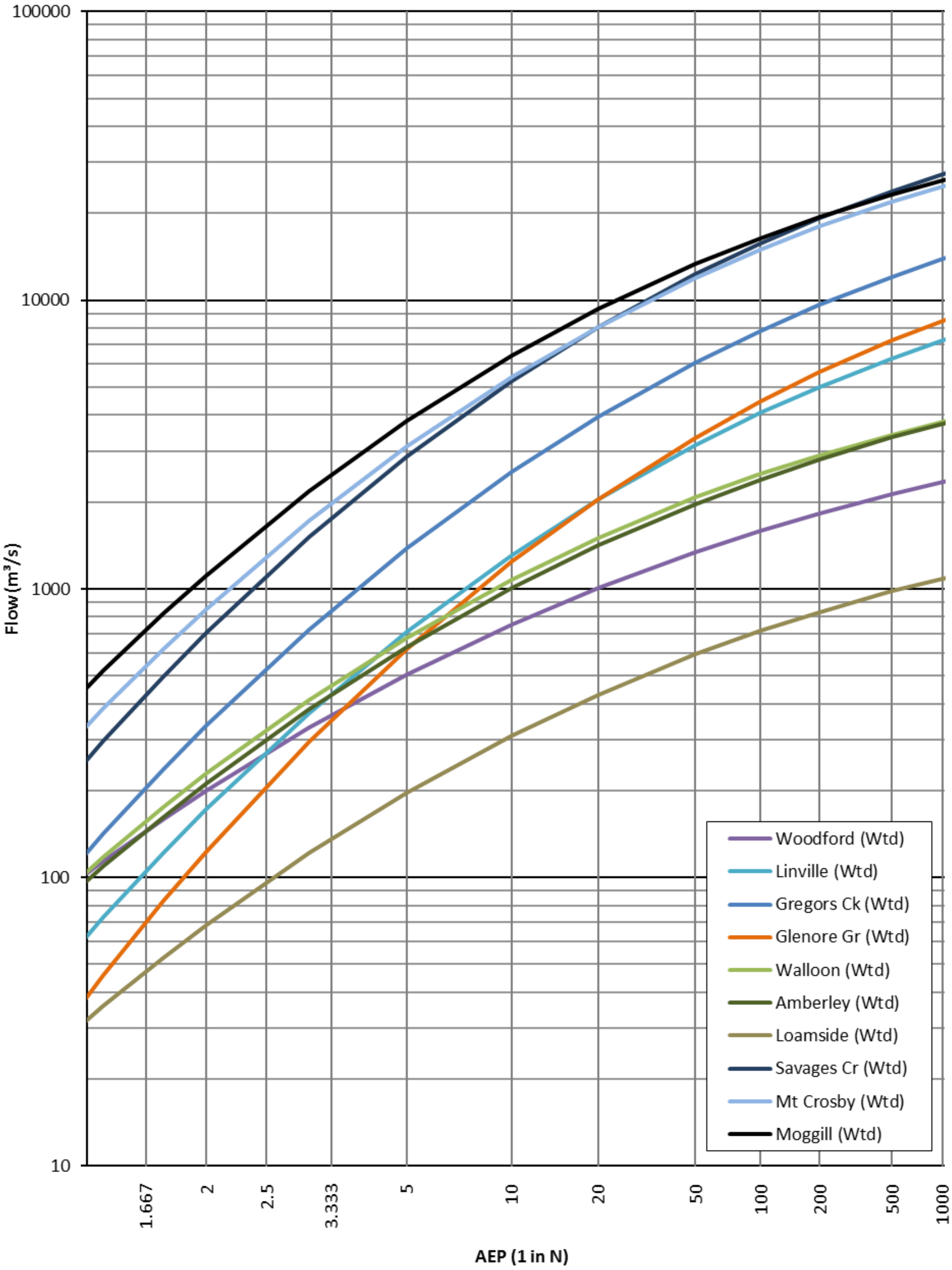


Figure 10-2 Catchment weighted flood frequency curves at primary gauges

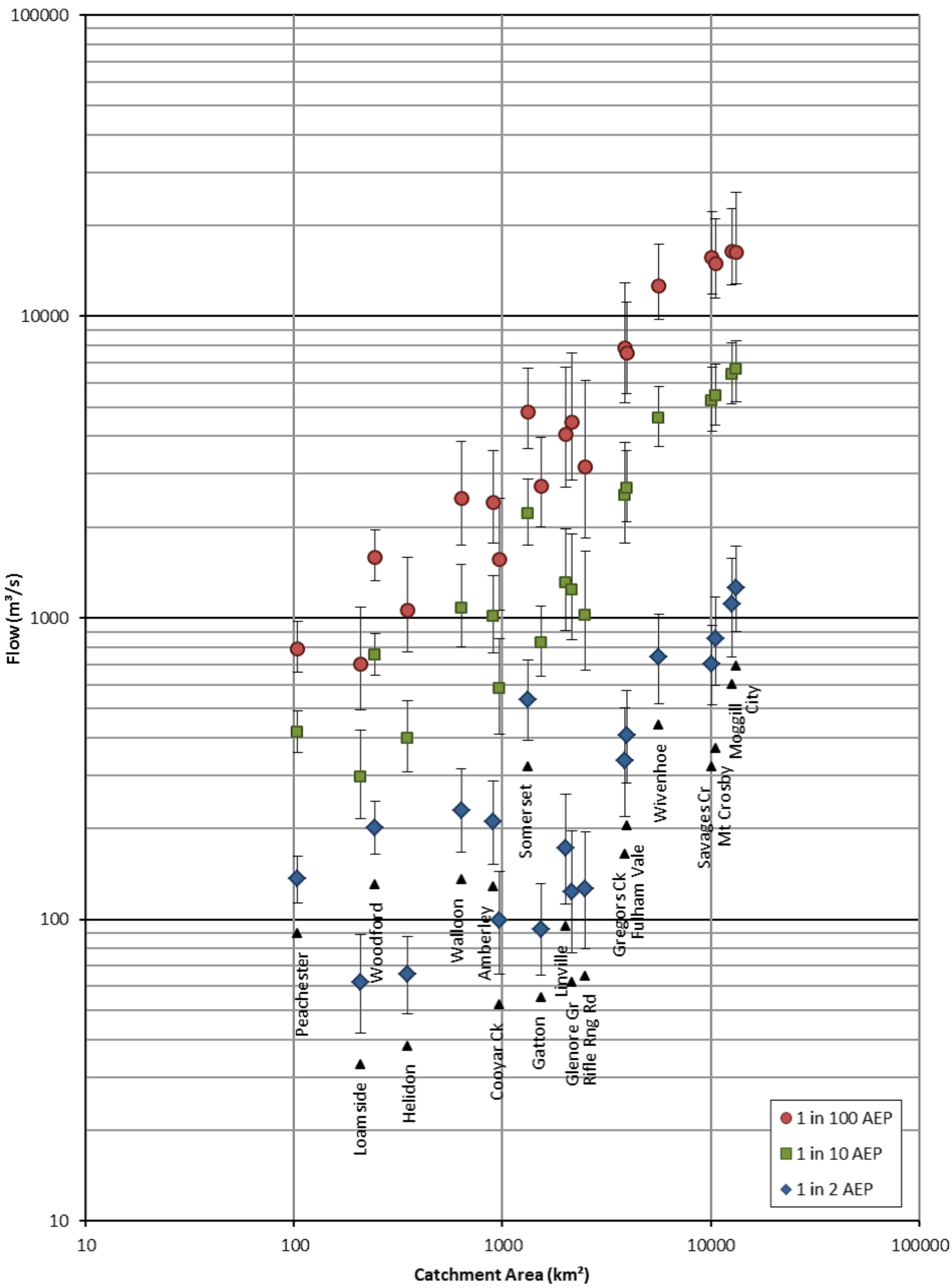


Figure 10-3 Key flood frequency flow estimates for all catchments

Linville, Gregors and Walloon are realistically the only three sites where the available gauge data is suitable for volume frequency analysis. The flood frequency analysis at these sites benefited from catchment weighted adjustment. Application of catchment weighting to the volume analysis is problematic as there is no reliable data to determine a regional relationship. Visually, the volume data appears to follow a similar trend to the flows so the same catchment weighted skew used for the flow frequency analysis (mean = -0.8, std deviation = 0.1) was applied to the volumes. Flood volumes for 1 day, 2 day and 3 day durations were estimated for each of these sites.

The volume frequency assessment was therefore conducted using the same catchment weighted standard deviation as was used for the flow assessment. Due to the uncertainty associated with their application, the catchment weighted frequency curves have been calculated for interest but should be treated with caution.

Estimates of flood volume have been used to assist in deriving appropriate loss model values in the rainfall based techniques.

10.1.4 Design event approach

In accordance with Section 3.6.6.4 of the Brisbane River Catchment Flood Study (BRCFS) brief, hydrologic modelling was undertaken to predict design flows, volumes and critical storm durations for design events in the Brisbane River Catchment.

The models developed during the recalibration process and discussed in the Aurecon Team's *Hydrologic Model Recalibration Report* (Aurecon 2015, refer Appendix C) have been used as the basis for the design event modelling. The 'no-dams conditions' model representation included removing the influence of the dams by modifying reach length factors and impervious areas. The 'with-dams conditions' model representation included the incorporation of the RTC Dam Operations module in the analysis process to simulation the impact of the flood mitigation dams.

The *Design event approach report* (Aurecon, 2014, refer Appendix G) presents how the calibration models were modified with design specific parameters and assumptions regarding the IFD input, the temporal patterns, the rainfall losses and the base-flow, all depending on the exceedance probability of the design events.

The modelling results show trends of growth and attenuation of the peak flood flows and flood volumes along the catchment, which reveals the significant influence of the assumed rainfall temporal patterns, and hence the limitation of this overall approach. The frequency curves show that the model predicts limited variation in peak flow rates within the Lower Brisbane River. In fact the curves demonstrate some attenuation in peak flow between Savages Crossing and Brisbane City despite the contribution of the Bremer River catchment within this reach of the river. This characteristic is attributed to the relative timing of the flows due to the assumed temporal patterns and the effects of storage at the confluence of the Brisbane and Bremer Rivers.

Flood volumes are predicted by the design model to increase only marginally by 4% to 9% between Moggill and Brisbane City for the full range of events from the 1 in 2 AEP to the PMF despite the contribution of an additional of nearly 1,000 km^2 of local catchment area.

As expected the critical storm duration increases with the downstream locations, ranging from 18 hours at the upper catchments up to 72 hours in the Lower Brisbane River in the case of the 1 in 20 AEP flood. However, it is also noted that the critical storm duration generally decreases with the frequency of the flood events, with the 1 in 5 AEP duration being 72 hours in Brisbane City, whilst it is only 24 hours for the rarer events including the PMF.

Sensitivity testing on the initial loss rates adopted for the production of design estimates was performed. Initial loss rates equal to 0 mm (ie no loss) were applied for the 1 in 2 AEP to 1 in 50 AEP events as part of this sensitivity assessment. As expected the 1 in 2 AEP events are very sensitive to the adopted initial loss rate, with increases of between 60% and 90% for peak flows. The critical duration is also influenced by the selection of initial loss rate, with the most noticeable change being the reduction in critical duration from 36 hours to between 6 hours and 24 hours for the 1 in 2 AEP event.

The application of Zone 2 temporal patterns instead of Zone 3 temporal patterns has a significant impact on peak flow rates. For the 1 in 100 AEP event the flows generally increase between 40% and 60% by adopting the Zone 2 temporal patterns, which demonstrates that the estimates are very sensitive to the assumed temporal patterns. Given the proximity of the Brisbane River catchment to the boundary between Zone 2 and Zone 3 patterns, the application of the Zone 3 temporal patterns is considered somewhat arbitrary. Given the inconsistency of estimates with the FFA results for the Lockyer Creek and Bremer River catchments in particular, the assumption of temporal pattern is regarded as being a major drawback of this method.

10.1.5 Monte Carlo simulation framework

The Monte Carlo Simulation (MCS) framework for the comprehensive hydrologic assessment of the Brisbane River Catchment Study is described below. The framework quantifies statistics, correlations and physical interactions of the most relevant flood forcing factors in the Brisbane river catchment: rainfall depth, event duration, spatial-temporal distribution of rainfall, antecedent soil moisture conditions (initial losses), reservoir volumes and ocean water levels. The MCS framework generates a long series of realistic synthetic events for rainfall, ocean water level, initial reservoir levels and antecedent moisture condition. The events are then simulated with the URBS hydrological model in combination with the RTC model for dam operations. Computed peak discharges and flow volumes at the 22 locations of interest are derived from the URBS model output. Frequency curves are subsequently derived with a statistical post-processing procedure. The Monte Carlo simulation framework has been implemented in the Delft-FEWS system.

The following conclusions are drawn:

- The proposed computation scheme of Figure 7-2 provides what is required for the BRCFS-hydrology phase: a joint probability approach for the derivation of design flows and volumes, taking into account spatial and temporal variation of rainfall over the Brisbane River catchment
- The method has the advantage over more "traditional" approaches in flood risk analysis in that it explicitly considers all relevant physical processes that contribute to flood events. A practical disadvantage is that it is generally more complex to implement
- The following statistical dependencies (correlations) between random variables were identified as relevant and have been incorporated in the Monte Carlo simulations:
  - Spatial and temporal correlation of rainfall. This dependence is taken into account in the BoM synthetic rainfall patterns, which are incorporated in the Monte Carlo Framework
  - Mutual correlations between antecedent moisture conditions (initial losses) of the various sub-catchments. These correlations are taken into account in the Monte Carlo simulations using a Gaussian copula model
  - Correlation between rainfall and ocean water levels. This is modelled with a threshold-excess logistic model
  - Correlation between rainfall and reservoir volumes. Reservoir volumes at the beginning of high rainfall events are on average significantly higher than reservoir volumes at any given day. For this reason, marginal distribution functions of reservoir volumes are based on observed reservoir volumes at the beginning of high rainfall events. The 'remaining' correlation is weak, ie the correlation between the total rainfall depth of a high rainfall event and the reservoir volume at the beginning of such an event. The latter is therefore not included in the MCS framework
  - Mutual correlations of reservoir volumes at the beginning of a high rainfall event. These are simulated with the skewed student-t copula model

- For the rainfall sampling scheme, three methods were tested: TPT, CRC-CH and CSS. Eventually, the TPT method was chosen as the preferred method for the current study, because this method provided the best match between the rainfall IFD curves on one hand and the available synthetic spatio-temporal rainfall patterns on the other hand. The other two methods (CSS and CRC-CH) are nevertheless considered very promising for future applications of Monte Carlo applications, especially if more synthetic spatio-temporal rainfall patterns become available
- Computation times for a single output location near the catchment outlet are in the order of five hours on a 64 bit machine, Windows 7, Solid State Drive (SSD) with 16Gb and 4 cores (duplicated, so actually 8 cores). For upstream locations with smaller catchment areas the runtime is in the order of two to three hours

10.1.6 Dam operation

In the Brisbane River Catchment Flood Studies, frequency curves are derived for two conditions: 'no-dams conditions' and 'with-dams conditions'.

For 'with-dams conditions', the following dams are considered:

- Wivenhoe
- Somerset
- Moogerah
- Lake Manchester
- Perseverance
- Cressbrook Creek

Moogerah, Lake Manchester, Perseverance and Cressbrook Dams are modelled in the URBS hydrological model as level pool storages with fixed crest spillway relationships. The storage representation and associated relationships are consistent with the description contained in the Brisbane River Flood Models, Seqwater (2013). No alterations have been made to the URBS model with respect to these four dams within the context of the BRCFS. The modelling of these four Dams was therefore not further discussed in this report, the focus was on Somerset Dam and Wivenhoe Dam.

The Somerset and Wivenhoe Dam Operations Module was implemented within the real-time control software RTC tools as a component of the Delft-FEWS framework for use in assessing the 'with-dams conditions' design flood estimates associated with the Monte Carlo Simulation techniques of flood estimation. The Dam Operations Module was based upon the Loss of Communications (LOC) emergency flood operation procedure described in the Flood Manual. The reason to implement the LOC scenario instead of the regular dam operation strategy is the fact that the latter is relatively complex to implement especially in a Monte Carlo Simulation framework. Noting that the purpose of this study is for floodplain management (ie not operational management), the implementation of the LOC was preferred for use in the Monte Carlo Simulation framework. A more detailed representation of the dam operations would also have resulted in excessive computational times that could not be accommodated in the study timeframe.

The Loss of Communications (LOC) emergency flood operation procedure was successfully implemented in the RTC tools model. This model will be used within the Delft-FEWS framework for use in assessing the 'with-dams conditions' design flood estimates associated with the Monte Carlo Simulation techniques of flood estimation. The model performance of the RTC tools dam operations model was compared to Seqwater's GoldSim model. Model results were compared for 24 synthetic events, ranging from moderate to extreme flood events. The comparison showed that predicted

Wivenhoe Dam outflow hydrographs of RTC tools closely matched the predicted hydrographs of the GoldSim model. As a follow-up activity, the drain-down process incorporated into the LOC was modified to reflect the normal operation procedure and mimic the seven day drainage requirement.

The LOC scenario on average results in slightly 'conservative' estimates of peak discharges and flow volumes in the Lower Brisbane River. For floods within the range of 2,000 m^3/s to 16,000 m^3/s, the peak flow in the mid-Brisbane River and Lower Brisbane River according to the LOC scenario are on average in the order of 5 to 10% higher than the peak discharges that result from the Dam operations using the Flood Manual procedures (2013 flood Manual). This means the derived frequency curves for the 'with-dams conditions' will be conservative as well.

Designs that will be based on these frequency curves will therefore be more 'robust'. If this additional robustness is undesired (ie too costly) the derived 'with-dams conditions' frequency curves could be adjusted to account for this effect by applying a bias adjustment based on the differences in peak flow between the LOC and FOSM.

The application of a bias adjustment was investigated as part of the current study however it was found that the adjustment may lead to a distortion of the frequency curve due to the highly variable nature of the simulations. The bias introduced by the LOC operation is small compared to the degree of natural variability. Therefore, a bias adjustment was not applied as it was considered it could not be justified and so the reconciled frequency curves have been adopted without modification.

10.1.7 Climate change

A climate change discussion paper (Aurecon, 2014, refer to Appendix H) has been prepared that provides clear advice on how various key input parameters could be amended to take account of climate change over a range of time horizons. It should be noted that the report was provided as advice to this study only and has not been implemented as yet.

10.1.8 Reconciliation

This report describes the main results of the Flood Frequency Analysis (FFA), the Design Event Approach (DEA) and the Monte Carlo Simulations (MCS) and proposes reconciled design flows for a range of AEP's for 'no-dams conditions' and 'with-dams conditions' based on these results. For 'no-dams conditions' DEA and MCS results were available for all 22 locations of interest. FFA results were available for 17 locations as no (reliable) data was available for the other 5 locations. DEA and MCS results for location Rifle Range Road were not used in the reconciliation procedure because the URBS hydrological model simulation results were unreliable for this location. For 'no-dams conditions', the reconciled design flows for the majority of the locations are based on a combination of:

- Empirical estimates from data for high values of the Annual Exceedance Probability (AEP)
- Flood frequency analysis results for intermediate values of the AEP
- Monte Carlo Simulations results for low values of AEP

The choice of bounds between the 'high', 'intermediate' and 'low' range of AEP's differed per location. For locations for which no (reliable) data on peak discharges were available and, hence, no FFA results as well, the reconciled design flows for the high and intermediate range of AEP values were based on data and FFA results of nearby stations. The reconciled estimates were successfully validated for spatial consistency.

For 'with-dams conditions', generally limited data series are available. For the Lower Brisbane locations a series of approximately 30 years of rated flows are available, starting in the year of completion of Wivenhoe Dam. This period starts and ends with several major flood events (1983, 2011, 2013) but also overlaps the longest drought in Brisbane's recorded history. The record may therefore not be statistically representative. Reconciliation of design flows based on these rated flows should therefore be done with care. Furthermore it is not possible to derive a statistical distribution function that matches this 'unbalanced' series of rated flows. The FFA analysis has therefore not been carried out for 'with-dams conditions'.

The eight 'with-dams conditions' locations were divided into four 'clusters'; for each cluster a different approach was used. The subdivision in clusters is mainly based on data availability.

For 'with-dams conditions', the reconciled design flows for these locations are based on a combination of:

- Empirical estimates from rated flows for (very) frequent events
- Monte Carlo Simulations results for frequent events to extreme events

These reconciled estimates were also successfully validated for spatial consistency.

10.2 Effect of the dams

The effect of the presence of the flood mitigation dams and the four other water supply dams in the catchment has been assessed as part of the study. The dams have been represented in their current configuration and the operation of the flood mitigation dams of Somerset Dam and Wivenhoe Dam is based upon the 2013 version of the Flood Manual (Seqwater, 2013). The emergency operation procedure known as the 'Loss of Communications' procedure has been adopted to represent the operation of the dams.

The comparison of the combined effect of the dams shows that the dams provide an impact of between 29 to 41% reduction in peak flow rates for the 1 in 100 AEP event for locations situated downstream of Wivenhoe Dam. The mitigation effect of the dams is greatest immediately downstream of the dams, but the effect diminishes further downstream from the dam as the influence of the downstream tributaries becomes more dominant.

The effect on peak flow extends over the full flood frequency range, although it should be recognised that for releases from Wivenhoe Dam that are in excess of 28,000 m^3/s, the results are based upon the assumption that Wivenhoe Dam will not fail. This is a non-conservative assumption and so the results should be treated with caution above this flow rate.

Figure 10-4 shows the comparison between the reconciled flood frequency curves no-dams and with-dams conditions for the Brisbane River at Brisbane City Gauge.

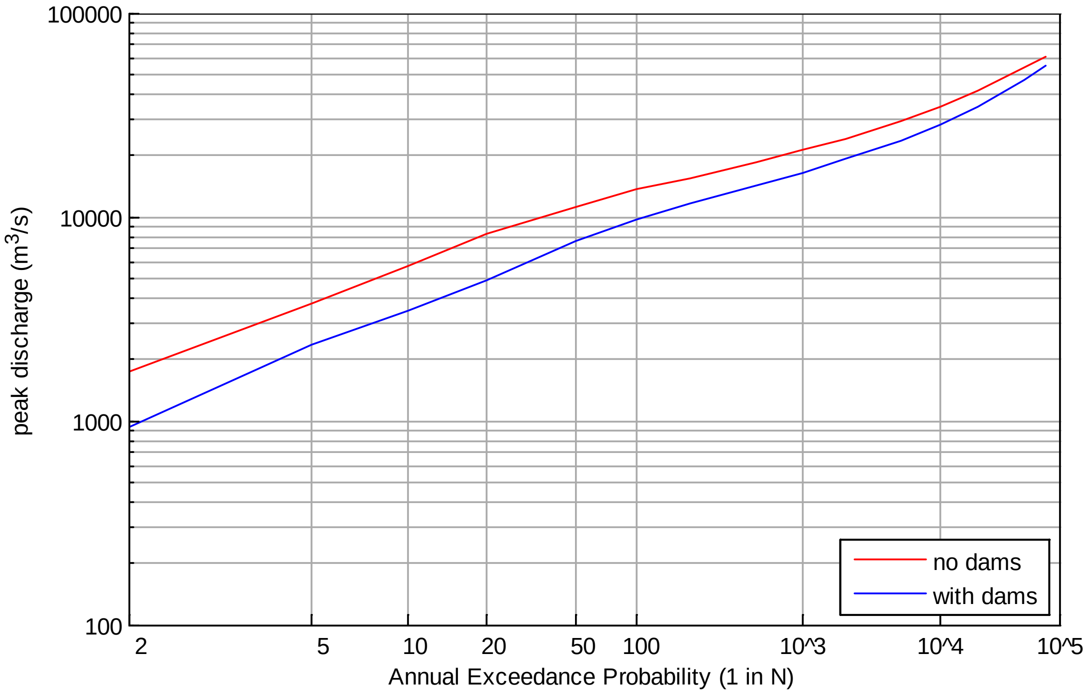


Figure 10-4 Comparison between no-dams conditions and with-dams conditions peak flow rates: Brisbane River at Brisbane City Gauge

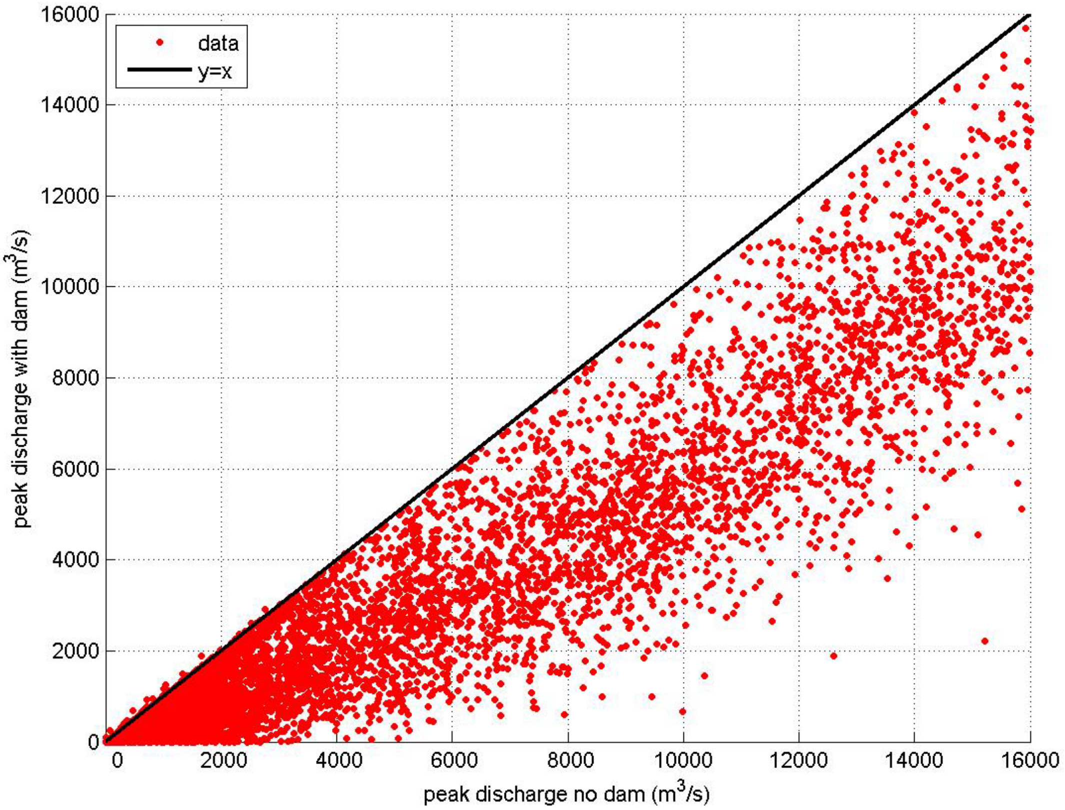


Figure 10-5 Comparison of peak flow at Brisbane River at Brisbane City Gauge; no-dams condition versus with-dams condition

Figure 10-5 shows the mitigation effect of the operation of the dams for the Brisbane River at Brisbane City Gauge for all of the simulated events in the MCS. The diagonal line shows the line of no mitigation. Events plotted below the diagonal line indicate that the dams provide mitigation to varying degrees, whereas for those events plotted above the line the effect of the dams is to slightly increase these peak flows. The further away from the line the points are plotted the greater the impact of the operation of the dams.

Some events result in minor increase in flood peaks. These events are likely to be associated with floods that result in the initiation of the fuse plugs at Wivenhoe Dam, resulting in a coincidence of flooding with the downstream tributaries.

Upstream of Somerset Dam, the town of Kilcoy and the D'Aguilar Highway are impacted by elevated storage levels in the dam. Figure 10-6 shows the frequency curve derived for storage levels at Somerset Dam. Flood levels in excess of EL102.45 m AHD impact the town and Mary Smokes Bridge.

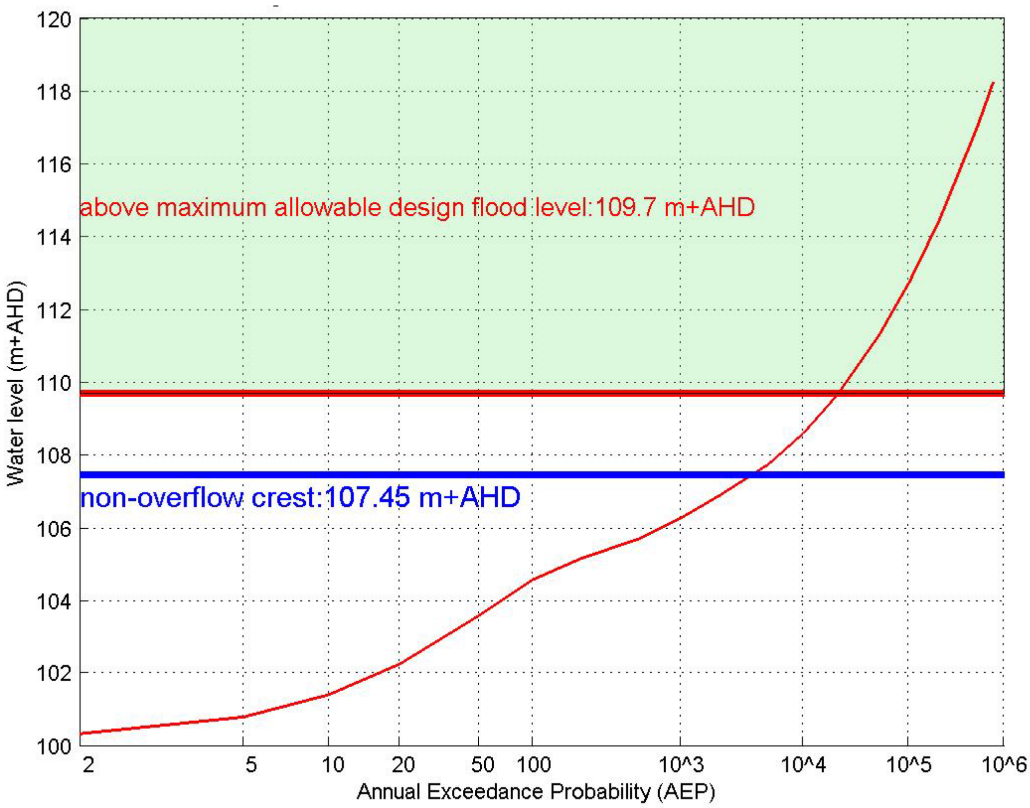


Figure 10-6 MCS flood frequency curve for peak flood levels at Somerset Dam

10.3 Estimate uncertainty

Uncertainty in this context is the estimated amount or percentage by which an observed or calculated value may differ from the true value. This study has required the 'best estimate' of flood frequencies at a number of locations within the Brisbane River Catchment. It must be recognised however, that by necessity design flood estimates involve a considerable degree of uncertainty which may need to be allowed for in the subsequent decision making processes associated with floodplain management.

A substantial degree of uncertainty in flood frequency estimates is inherent from the high degree of variability of hydrologic factors that produce floods and the limited sample available from the total population of such factors. Additional uncertainty may arise from the following sources of error in the basic data and in the methods adopted for the design flood estimation:

- Systematic errors and inconsistencies in the basic rainfall and water level observations at gauging sites (for example the Bremer River and Warrill Creek design rainfall IFD and FFA comparison)
- Uncertainty in rating curves used to convert water level observations into flow estimates. This especially affects the high range of the rating curve which corresponds to the larger floods of interest and dynamic effects including tidal influences (for example Brisbane River at Port Office Gauge)
- Errors introduced by the adjustment of flood data for the effects of changes in the hydrologic and hydraulic conditions (for example the inclusion of the dams and changes due to dredging or main channel erosion between major flood events)
- Uncertainty in the choice of model parameters or statistical distributions for flood frequency analyses or runoff-routing models (for example LPIII or GEV distributions in the FFA)
- Uncertainty introduced by simplified representation of catchment characteristics or operational decisions in reservoir simulation models (for example the adoption of the LOC procedure to simulate the operation of Somerset and Wivenhoe Dam, or storm tide relationships)

Confidence limits determined as part of the FFA process provide an indication of how some of the uncertainties affect the resulting design flood estimates, although they do not generally reflect all of the uncertainty factors involved in the flood estimation process.

The 90% Confidence Interval for a range of locations is present in Table 10-3 for the no-dam conditions. These are estimates derived after applying catchment-weighted parameters at the individual stations.

Table 10-3 90% confidence intervals: no-dams conditions

Location	AEP (1 in 2)			AEP (1 in 5)			AEP (1 in 10)			AEP (1 in 20)		
	5% Lower CL	Estimate	95% Upper CL	5% Lower CL	Estimate	95% Upper CL	5% Lower CL	Estimate	95% Upper CL	5% Lower CL	Estimate	95% Upper CL
Linville	112	172	260	492	711	1,050	909	1,310	1,980	1,410	2,040	3,160
Gregors Creek	219	335	503	954	1,380	2,040	1,760	2,540	3,800	2,710	3,940	6,060
Woodford	164	201	246	431	505	595	644	753	889	858	1,010	1,200
Glenore Grove	77	124	197	422	621	936	844	1,240	1,900	1,380	2,050	3,200
Savages Crossing	515	704	943	2,260	2,870	3,680	4,150	5,230	6,760	6,350	8,050	10,700
Mt Crosby	597	852	1,170	2,460	3,120	3,980	4,360	5,430	6,930	6,460	8,090	10,600
Walloon	167	229	317	511	678	923	803	1,080	1,500	1,100	1,500	2,150
Amberley	153	211	287	478	630	845	765	1,010	1,380	1,070	1,420	1,990
Loamside	42	62	89	134	186	261	216	297	424	302	416	608

Location	AEP (1 in 2)			AEP (1 in 5)			AEP (1 in 10)			AEP (1 in 20)		
	5% Lower CL	Estimate	95% Upper CL	5% Lower CL	Estimate	95% Upper CL	5% Lower CL	Estimate	95% Upper CL	5% Lower CL	Estimate	95% Upper CL
Moggill	744	1,110	1,580	2,940	3,820	4,910	5,110	6,420	8,180	7,460	9,310	12,000
Brisbane	899	1,260	1,720	3,090	4,070	5,070	5,220	6,680	8,300	7,580	9,500	13,000

The comparison of estimates indicates that there is a spread of results across the various sites. Some locations such as Savages Crossing have quite tight confidence intervals over the range of flood magnitudes, with around a 55% error bound at the 1 in 20 AEP event. On the other end of the scale locations such as Gregors Creek and Linville have confidence intervals that are nearly 85% for the 1 in 20 AEP event.

However, the confidence interval is not the only measure of the reliability of the estimate, with consideration of the rating ratio of the respective gauges also being a consideration. Locations such as Centenary Bridge which has some high flow measurements at around the 1 in 100 AEP range would provide greater confidence than other locations such as Glenore Grove in Lockyer Creek, which has a poorly rated high stage rating.

It will be important for users of the design flood hydrology estimates to recognise the inherent uncertainty in the estimates. Table 10-4 provides a summary of the 1 in 100 AEP 90% confidence intervals obtained from the FFA for the various sub-catchments in the Brisbane River catchment.

Table 10-4 1 in 100 AEP 90% Confidence Intervals: No-dams conditions

Location	5% Confidence Limit (m^3/s)	FFA estimate (m^3/s)	95% Confidence Limit (m^3/s)
Peachester	660	790	970
Woodford	1,330	1,580	1,960
Somerset Dam	3,640	4,810	6,740
Linville	2,700	4,060	6,750
Gregors Creek	5,180	7,820	12,900
Wivenhoe Dam	8,430	13,400	24,000
Helidon	773	1,060	1,580
Gatton	2,000	2,730	3,960
Glenore Grove	2,870	4,450	7,560
Walloon	1,730	2,490	3,830
Amberley	1,760	2,410	3,580
Loamside	495	701	1,080
Savages Crossing	11,800	15,700	22,200
Mt Crosby	11,500	14,900	21,000
Moggill	12,700	16,400	22,700
Brisbane City	12,800	16,300	25,800

10.4 Variability

Variability in the context of this project refers to how "spread out" a group of data is. Typically this is taken to be the range, or dispersion of the data. Examples of the variability in the data include the spatial and temporal distribution of rainfall over the catchment. Comparisons of the design rainfall CRC-Forge data over the catchment highlight that for example the 24 hour, 1 in 100 AEP rainfall depths vary from 587 mm in the Stanley River to 170 mm in the headwaters of the Lockyer Creek catchment.

Likewise the difference in initial and continuing loss rates provides an indication of the variability over the catchment.

The resultant effect of the variability in input parameters is evidenced in the *Reconciled and recommended flood frequency estimates report* (Aurecon, 2015, refer Appendix I), which provides examples of estimated hydrographs at Brisbane River at Brisbane City Gauge for the 1 in 100 AEP event. Figure 10-7 provides an example of the variation in hydrographs for a range of rainfall durations. These hydrographs are for the with-dams conditions. It is evident that the peak flow for each of these hydrographs is quite variable, with a range of peak flows of between nearly 6,000 m^3/s to nearly 10,000 m^3/s. This is the variability in peak flows for eight events, in general about 160 events are considered for a single rainfall AEP, which further increases variability

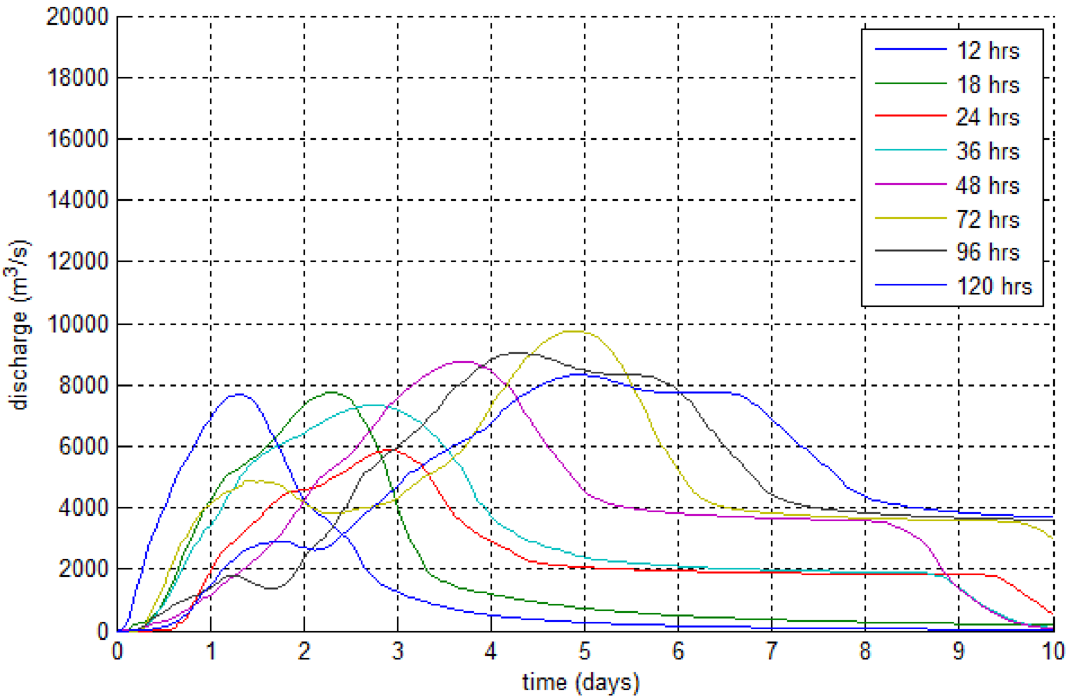


Figure 10-7 'With-dams conditions' hydrographs at location Brisbane River at Brisbane City Gauge for a range of rainfall burst durations, rainfall AEP ≈ 1 in 100; first set of samples

The range of rainfall depth estimates of different durations for a particular location is illustrated in Figure 10-8. This Figure shows the relationship between AEP of the input rainfall depth and the resultant peak flow for the Brisbane River at Moggill. This Figure shows that for the 1 in 100 AEP rainfall event, the range of estimates varies from about 6,000 m^3/s to 20,000 m^3/s. There appears to be a greater range for more extreme events although the differences between the 10%, 50% and 90% quantiles remain relatively uniform.

Similar Figures have been produced for all locations of interest (Appendix I). The following is observed from these Figures:

- For rainfall AEP values > 1 in100) the distance between the 10% quantile and the 50% quantile is generally equal to the distance between the 50% quantile and the 90% quantile, indicating a fairly uniform distribution of peak discharges for a given rainfall AEP. For some locations, for example Wivenhoe, this is not the case. This is mainly due to the fact that fairly short duration rainfall bursts of 3 hours and 6 hours were considered as well for location Wivenhoe Dam. These events result in low peak discharges at Wivenhoe compared to the longer duration bursts and cause a negative skew in the distribution of peak discharges for a given rainfall AEP. If these short duration bursts would have been omitted, the distribution functions would be fairly symmetric
- For extreme events (rainfall AEP<1 in 2,000), peak discharges are often 'clustered'. For these events, variations in losses are small which means the variation in peak discharges for a given rainfall AEP is mainly related to the spatio-temporal pattern. For events with rainfall AEP<1 in 2,000, GTSMR based patterns are used to model the spatio-temporal rainfall distribution (Deltares, 2015). This means a single spatial distribution is applied, which corresponds to the IFD rainfall depth of a 72-hour, 50-year ARI event. For the temporal distribution, 10 temporal patterns are adopted from the GTSMR approach for a range of durations (24, 36, 48, 72, 96 and 120 hours). The number of spatio-temporal rainfall patterns used in the MCS simulations for events with rainfall AEP<1 in 2,000 is therefore relatively low, which reduces the variability in peak discharges for a given rainfall AEP. For some locations this has a clear effect on especially the 10%-quantile lines. For example for location Amberley, the 10% quantile is equal to the lower bound, which means not much value should be placed on the actual meaning of the 10% quantiles for these extreme events
- For a given rainfall AEP, there is more variation in the 'with dams' case compared to the 'no dams case'. The dams clearly add more variability to the flood scenarios and, hence, more variability in peak flows. The increased variability has several causes:
  - Starting levels of the dams are modelled as a stochastic variable. This means the available storage volumes of the reservoirs at the start of an event differ per simulated event, which increases the variability of resulting peak discharges for a single AEP
  - The percentage of rain which falls upstream of the main dams is different for different simulated events due to the application of stochastic spatio-temporal rainfall patterns. This means the percentage of rain that can be "controlled" by the dams vary, which also leads to an increase in the variability of resulting peak discharges for a single AEP
  - Different burst durations are considered, which means a single AEP corresponds to a variety of different catchment average rainfall depths. The total rainfall depth for long burst durations is higher than the total rainfall depth for short burst durations. The dampening effect of the dams will therefore be different for different burst durations, which also leads to an increase in the variability of resulting peak discharges for a single AEP

The variability will need to be considered by users of the design flood hydrology estimates especially as the BRCFS moves further into the hydraulics and flood plain management phases of the overall study. It will be important for the hydraulics phase of the Brisbane River Catchment Flood Study which will convert the design flood hydrographs into design flood levels at different locations. It will be important that sufficient representative hydrographs are selected for testing to establish the critical events for determining the peak flood levels for various design events at different locations within the Lower Brisbane River. The range of events to be tested will need to ensure that a combination of hydrographs adequately represents; short duration, high peak hydrographs and long duration low peak hydrographs. This will cover the channel control dominated conditions and flood storage dominated conditions for various locations that may result in peak flood levels.

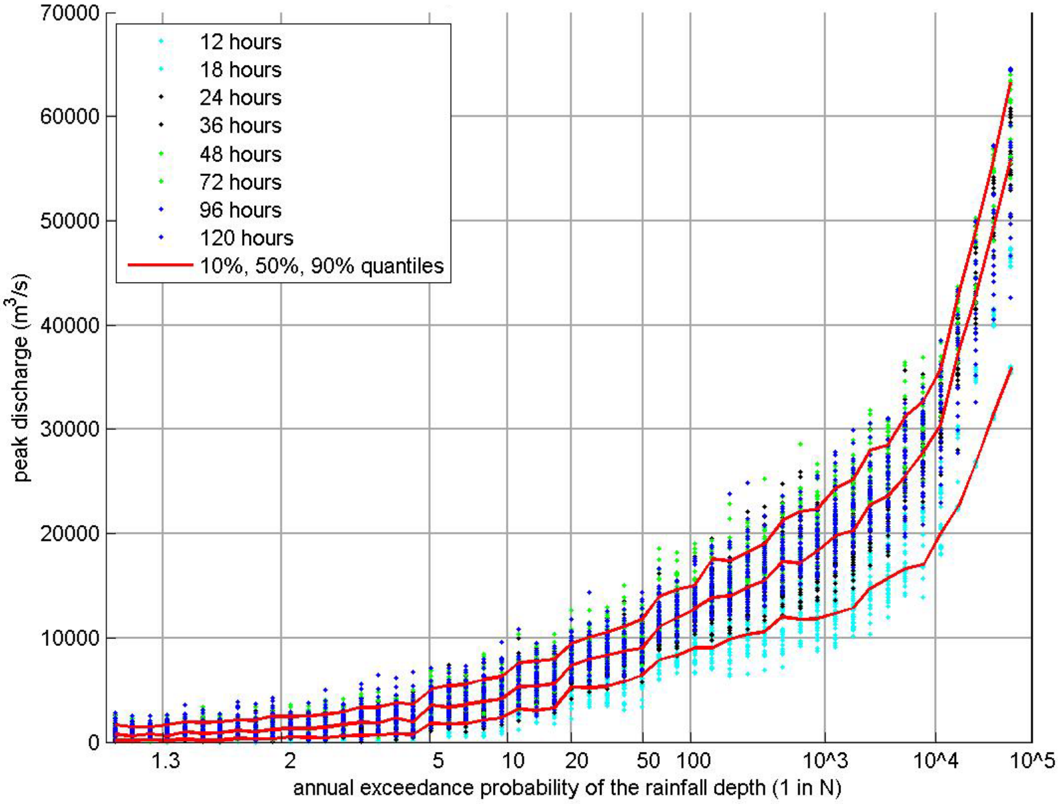


Figure 10-8 'No-dams conditions' peak flow estimates; Brisbane River at Moggill

11 Related ongoing and recommended future work

11.1 Hydrology and Hydraulic Interface

The Comprehensive Hydrologic Assessment Phase of the Brisbane River Catchment Food Study provides inputs into the Comprehensive Hydraulic Assessment Phase of the overall study in the form of an ensemble of hydrographs and downstream tidal levels. This work is briefly described below, as it is additional to the estimation of the reconciled and recommended design flood estimates that forms the outcomes of the CHA.

The study brief required that:

> *Probabilistic design flood outputs at key locations along the Brisbane River and major tributaries for a range of Annual Exceedance Probability (AEP) of 50%, 20%, 10%, 5%, 2%, 1%, 0.5%, 0.2%, 0.1%, 0.05%, 0.01%, and 0.001% as well as the Probable Maximum Flood.*
>
> *The MCS based outputs should include*
>
> *(a) probabilistic estimates of design floods (peak flows and flood volumes) for both 'no-dam' and 'with-dam' (current) conditions and the associated range of uncertainty about the estimates, and*
>
> *(b) ensembles of stochastic design flood hydrographs suitable for input into a range of detailed hydraulic modelling packages. The hydrographs should be 'tagged' against the storm characteristics that produced them to allow any significant correlations with the forcing factors for ocean level anomalies to be taken account of in the hydraulic modelling*.

The ensemble of stochastic design flood hydrographs consists of local and total hydrographs at around 150 input locations which will be ingested into the 'fast' hydraulic model. This data set is derived from the MCS event sets based on the Brisbane City key location (ie for the whole of catchment) representing around 7,560 events in total covering the entire flood frequency range.

The hydraulic model has the following boundary sources:

a. Catchment inflows from the URBS models as a mixture of local and total (upstream boundaries) hydrographs
b. Wivenhoe Dam Release
c. Moreton Bay water levels (Brisbane Bar)

The hydrologic modelling consists of five URBS vector files to represent the sub-catchment models downstream of Wivenhoe Dam (ie an URBS .vec file for each of the Lockyer, Bremer, Warrill, Purga and Lower Brisbane URBS models). The revised .vec files do not change the hydrologic routing, but simply produce around 100 hydrographs for input to the hydraulic models. Each output hydrograph location is assigned a unique ID and has a set of meta data associated with it.

11.2 Data

The main data quality issues affecting the outcome of the Comprehensive Hydrologic Assessment include the definition of rating curves, design rainfall inconsistencies and the possible representativeness of the spatio-temporal patterns utilised in the Monte Carlo Simulation Framework.

11.2.1 Rating curves

The high stage rating curves at a number of sites are subject to quite large uncertainty, which can be improved by either obtaining some measurements during the next large flood event or by some further detailed hydraulic modelling. The ability to obtain measurements is obviously dependent upon the occurrence of a suitable flood event and the availability of hydrographic teams to be able to undertake the measurements. The priority for these sites include the Lower Brisbane River (in particular, Savages Crossing, Mount Crosby Weir and Centenary Bridge), and the Bremer River (specifically Walloon and Amberley). Improving the high stage rating at these sites would be also useful in strengthening the calibration of the Hydraulic Model.

Secondary sites could be further validated to improve the overall consistency within the hydrologic model.

11.2.2 Rainfall

Inconsistencies with the design rainfall and the flood frequency estimates for the Bremer River catchment, especially in Warrill Creek were observed during the reconciliation process. Comparisons between the observed rainfalls for a number of historic events and the IFD estimates generated from the BoM IFD(2013) and CRC-Forge(2005) values suggest that four large to rare flood events have been observed in this catchment in the last 125 years. This has resulted in inconsistencies between estimates obtained from the rainfall based approaches and the direct flood frequency approach. The review of the IFD by the BoM which is scheduled for the end of 2015 may improve this issue and so a check of the revised IFD estimates for this catchment is recommended in the future.

The transition between IFD and CRC-Forge estimates has also been noted as an issue for a number of locations. This issue is related to the fact that the IFD estimates and the CRC-Forge estimates for the 1 in 100 AEP events are different. The current BoM revision project should address this issue when the revised IFD estimates become available at the end of 2015. It is recommended that a review of the IFD estimates for all locations should be undertaken when the revised estimates become available in late 2015.

Storm spatio-temporal patterns utilised in this project are based upon nine different historic events. Whilst the range of events is considered appropriate, it is considered that there may be a lack of long duration events in this available sample (> 7 days). If a long duration rainfall event occurs in the future and suitable radar imagery is captured it is recommended that the storm pattern catalogue should be updated to test the effect of incorporating a long duration event in the framework.

11.2.3 Streamflow

If a subsequent large flood event occurs in part or all of the Brisbane River Catchment, it is recommended that the relevant data (rainfall, stream levels and dam operations data) be collected and used to validate or update the calibration of the hydrologic models developed in this study.

11.2.4 Storm surge and tide levels

It is recommended that some further analysis be undertaken to derive a storm surge hydrograph that is commensurate with the latest GHD study and specific for the mouth of the Brisbane River. The hydrograph adopted for the MCS assessment is considered appropriate, but it is acknowledged that it is not necessarily specific to the mouth of the Brisbane River as it is based on observations made on the Sunshine Coast. This work will not unduly affect the generation of the peak flow estimates that are the main focus of the Comprehensive Hydrologic Assessment. Further investigation of the storm surge and tide levels is recommended.

11.3 Models

11.3.1 Runoff-routing model

The re-calibrated hydrologic model of the Lower Brisbane River may need to be re-visited in the future once a fully calibrated detailed hydraulic model has been developed as part of the Comprehensive Hydraulic Assessment. The configuration of the routing parameters and reach lengths, and adopted conceptual storage characteristics could be further modified to ensure consistency between the hydrologic and hydraulic models.

11.3.2 Dam operations model

The adoption of a simplified dam operations representation is recognised and the impact on the estimates of peak flow downstream of Wivenhoe Dam has been assessed by utilising simulations conducted by Seqwater as part of the WSDOS investigation. The use of the simplified approach results in the peak flow estimates being over-estimated by between 5 to 10% on average when compared to the results of the normal operating procedure (based on a comparison at Moggill).

The current dam operations module is based on the Loss of Communications (LOC) procedure as outlined in the Flood Manual (Seqwater, 2013). As such this approach cannot distinguish with other proposed operating procedures such as Alternate Urban 3, unless the LOC is modified accordingly.

Revision 12 of the Flood Manual was gazetted in November 2014. As previously noted, it was expected that alternate operating strategies would retain the same LOC operation as specified in Revision11 of the Manual. Unfortunately, while the LOC emergency procedure in Revision 12 is very similar to that in Revision 11, it is not exactly the same.

- The instructions in Revision 12 for Wivenhoe Dam releases are based solely on dam water level and gate settings and are identical to those in Revision 11
- The instructions for management of Somerset Dam levels in Revision 12 are very similar to those in Revision 11
  - Only the sluice gates are used to adjust releases from Somerset Dam into Wivenhoe Dam
  - Somerset Dam Headwater level and Wivenhoe Dam Headwater level are used as the basis for decision whether to store or release flood water from Somerset Dam
  - This decision making is guided by the Wivenhoe Somerset Interaction diagram in Figure 6.3.1 of Revision 11 and by the Somerset Dam Guide Curve in Figure 6.2.1 of Revision 12
  - The Somerset Dam Guide Curve of Revision 12 is broadly similar to the Wivenhoe Somerset Interaction diagram of Revision 11 but it is different in shape
  - It is conceivable that the history of water level in Wivenhoe Dam could be affected by this, resulting in some change in the history of releases from Wivenhoe Dam

Differences in the use of the modified Somerset Dam guide curve of Revision 12 are not expected to be substantial, but nevertheless it is recommended the RTC Tools module be updated to incorporate the new relationships.

It is recommended that a watching brief be kept on the dam safety assessment of Somerset Dam and Wivenhoe Dam and a decision made as to the likely need to modify the dam operations module, when the situation of any changed configuration of the dams or their operating procedures becomes evident.

11.4 Iteration of hydrologic model calibration

As discussed previously, the future works could involve the possible re-iteration of the hydrologic model calibration based upon the outcome of the detailed hydraulic model calibration which will occur in the Comprehensive Hydraulic Assessment phase of the overall study. This may involve the revision of rating curves for locations situated in the Lower Brisbane River, the changing routing parameters of the Lower Brisbane River sub-catchment model, or modification of the conceptual storage relationships adopted in the Lower Brisbane River model. This is considered to be a continuous improvement approach which will ensure the models evolve and improve over time.

12 References

Australian Government Bureau of Meteorology, (2014), (http://www.bom.gov.au)

Australian Government Bureau of Meteorology, (2003a). Guidebook to the Estimation of Probable Maximum Precipitation: Generalised Tropical Storm Method. HAS, HRS8, Melbourne, August 2003

Australian Government Bureau of Meteorology, (2003b). The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method. HAS, Melbourne, June 2003

Aurecon, (2014), WSDOS/NPDOS Integrated Assessment WSDOS Results, Final Report for DEWS, Aurecon 237065 Rev 3 March 2014

Aurecon, (2013a), Data Collection, Collation and Storage of Existing Data – Final Report, Aurecon 233550 Rev 3 April 2013

Aurecon, (2013b), Sunshine Coast Storm Tide Study, Aurecon 215317 Rev 1, December 2013

BCC, (2014a), Brisbane River Catchment Disaster Management Tool Model Development and Calibration Outcomes Final Report, Prepared for DSDIP, Brisbane City Council, Brisbane, November 2014

BCC, (2014b), Brisbane River Catchment Flood Study Digital Terrain Model Development and Bed Level Sensitivity Analysis Final Report, Prepared for DSDIP, Brisbane City Council, Brisbane, November 2014

BCC, (2014c), Brisbane River Catchment Disaster Management Tool Model Development and Calibration Outcomes Draft Final Report, Prepared for DSDIP, Brisbane City Council, Brisbane, June 2014

BCC, (2013), Brisbane River Catchment Flood Study Disaster Management Tool (DMT) Interim Calibration Report, Prepared for DSDIP, Brisbane City Council, Brisbane, October 2013

BCC, (2009), Brisbane River Hydraulic Model to Probable Maximum Flood (PMF) Final Report, Brisbane City Council, Brisbane, June 2009

BCC, (1976), The Early Floods of the Brisbane-Bremer River System, 1823-1867, Murdoch Wales (In association with Geoffrey Cossins and Robert Broughton), Brisbane City Council, 1976

Carroll, D. G., (2012), URBS Unified River Basin Simulator: A Rainfall Runoff Routing Model for Flood Forecasting and Design, Version 5.0 User Manual. DCPM, Brisbane, 2012

CSIRO, (2000), Floodplain Management in Australia, Best Practice Principles and Guidelines. SCARM Report No 73, CSIRO Publishing, Victoria

DEWS, (2014a), Wivenhoe and Somerset Dam Optimisation Study Report, DEWS, Brisbane, March 2014

DEWS, (2014b), Wivenhoe and Somerset Dam Optimisation Study – Discussion Paper, DEWS, Brisbane, April 2014

DHI Water & Environment, (2006), Ipswich River MIKE-11 Model Upgrade – Final Report, Prepared for Ipswich City Council, May 2006

DNRM, (1994), Regional Loss Models for Brisbane and Pine River Catchments, Report 18. Department of Natural Resources and Mines, Brisbane, 1994

DSDIP, (2013), Invitation to Offer No. DSDIP-2077-13, for the provision of a comprehensive hydrologic assessment as part of the Brisbane River Catchment Flood Study, Version 005, DSDIP, Brisbane, July 2013

DSITIA, (2013a), IQQM Text Files for Model 202 – ROP Full Utilisation, DSITIA, November 2013

DSITIA, (2013b), IQQM Text Files for Model 233 – WRP Pre-Development, DSITIA, November 2013

Engineers Australia, (2003), Australian Rainfall and Runoff, A Guide to Flood Estimation, Volume 1, Engineers Australia, Barton ACT, 2003

Engineers Australia, (1987), Australian Rainfall and Runoff, A Guide to Flood Estimation, Volume 2, Engineers Australia, Barton ACT, 1987

Engineers Australia, (2006), AR&R Book IV (2006): Estimation of Peak Discharge. Kuczera and Franks – Draft only

GHD, (2014), Coastal Plan Implementation Study Draft Report, Prepared for Brisbane City Council, GHD, Brisbane, September 2014

Hargraves, G, (2005), Final Report Extreme Rainfall Estimation Project, Resource Sciences Centre, Brisbane, 2005

Hill, P., Graszkiewicz, Z., Melanie, T., and Nathan, R., (2014), Project 6: Loss Models for Catchment Simulation – Rural Catchments Stage 4 Report, AR&R Revision Project P6/S4, Engineers Australia, February 2014

Hill, P., Graszkiewicz, Z., Sih, K., and Rahman, A., (2013), Project 6: Loss Models for Catchment Simulation – Rural Catchments Stage 2 Report, AR&R Revision Project P6/S2/016A, Engineers Australia, March 2013

Independent Review Panel, (2003), Review of the Brisbane River Flood Study, Brisbane City Council, Brisbane, September 2003

IPCC (2013). Climate Change 2013: The Physical Science Basis. Contribution of Working Group I to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change (in press)

Jacobs, (2014), Additional Stochastic Space-Time Rainfall Replicates for Brisbane River Catchment, Prepared for Aurecon, SKM Melbourne, May 2014

Joint Flood Taskforce, (2011), Joint Flood Taskforce Report, Brisbane City Council, Brisbane, March 2011

Jordon, P., Weinmann, W., Hill, P., and Wiesenfeld, C., (2013), Project 2: Collection and Review of Areal Reduction Factors from Applications of the CRC-Forge Method in Australia Final Report, AR&R Revision Project P2/S2/012, Engineers Australia, April 2013

KBR, (2002), Ipswich Rivers Flood Study Phase 3 Final Report. Ipswich City Council, Ipswich, 2002

Murphy, R., Graszkiewicz, Z., Hill, P., Neal, B., and Nathan, R. (2011), Project 7: Baseflow for Catchment Simulation Stage 2 Report, AR&R Revision Project P7/S2/017, Engineers Australia, August 2011

Nathan, R. and Weinmann, E. (2013), Monte Carlo Simulation Techniques, AR&R Discussion Paper D2, Engineers Australia, May 2013

Queensland Government, (2012), Queensland Government Response to Queensland Floods Commission of Inquiry Final Report, Qld Government, Brisbane, July 2012

QFCOI, (2012), Queensland Floods Commission of Inquiry, Final Report, QFCOI, Brisbane, March 2012

QFCOI, (2011), Queensland Floods Commission of Inquiry, Interim Report, QFCOI, Brisbane, August 2011

Rahman, A., Haddad, K., Zamen, M., Ishal, E., Kuczera, G., and Weinmann, E., (2012), Project 5: Regional Flood Methods Stage 2 Report, AR&R Revision Project P5/S2/015, Engineers Australia, June 2012

Sargent Consulting, (2006), Ipswich Rivers Flood Study Rationalisation Project Phase 3 Re-estimation of Design Flows Final Report, Prepared for Ipswich Rivers Improvement Trust and Ipswich City Council, Ipswich, September 2006

Sargent Consulting, (2006), Ipswich Rivers Flood Study Rationalisation Project, Phase 3 Monte Carlo Analysis of Design Flows, Prepared for Ipswich Rivers Improvement Trust and Ipswich City Council, Ipswich, January 2006

Seqwater, (2014), 'Wivenhoe-Somerset Dam Optimisation Study – Simulation of Alternative Flood Operations Options' Seqwater, Brisbane, March 2014

Seqwater, (2013), Brisbane River Flood Hydrology Models, Volume 1 and 2, Seqwater, Brisbane December 2013

Seqwater, 2013, Manual of Operation Procedures for Flood Mitigation at Wivenhoe Dam and Somerset Dam, Revision 11, November 2013

SKM (2013), Brisbane River Catchment Dams and Operational Alternatives Study, Prepared for Seqwater, SKM, Melbourne, October 2013

SKM, (2012), Lockyer Creek Flood Risk Management Study, Volume 1, Prepared for LVRC, SKM, Brisbane, June 2012

SKM, (2004), Recalibration of the MIKE11 Hydraulic Model and Determination of the 1 in 100 AEP Flood Levels, Prepared for BCC, SKM, Brisbane, February 2004

SKM, (2003a), Brisbane River Flood Study – Further Investigation of Flood Frequency Analysis Incorporating Dam Operations and CRC-Forge Rainfall Estimates – Brisbane River, Prepared for BCC, SKM, Brisbane, December 2003

SKM, (2003b), Flood Frequency Analysis for Brisbane River Catchment Summary Report – DRAFT, Prepared for BCC, SKM, Melbourne, August 2003

SKM, (2000), Ipswich Rivers Flood Study Phase 1 & Phase 2 Final Report, Prepared for Ipswich Rivers Improvement Trust and Ipswich City Council, SKM, 2000

SKM, (1998), Brisbane River Flood Study – Final Report, Prepared for BCC, SKM, Toowoomba, June 1998

WMAwater, (2013), Gridded IFD Data from BoM Database, WMAwater, GIS Mapinfo coverages, October 2013

WMAwater, (2014), Individual Storm Totals and Key Catchments Data Base, WMAwater, Excel Spreadsheets, 2014

WMAwater, (2011a), Brisbane River 2011 Flood Event – Flood Frequency Analysis – Final Report, Prepared for QFCOI, Sydney, September 2011

WMAwater, (2011b), Supplementary Report – Ipswich Flood Frequency Analysis – Final Report, Prepared for QFCOI, Sydney, October 2011

Zheng, F., Westra, S., and Leonard, M., (2013), Project 18: Interaction of Coastal Processes and Severe Weather Events Stage 3 Draft Report, AR&R Revision Project P18/S3, Engineers Australia, December 2013

Zheng, F., Westra, S., Sisson, S., and Leonard, M., (2013), Flood risk estimation in Australia's coastal zone: modelling the dependence between extreme rainfall and storm surge, draft paper for the Hydrology and Water Resources Symposium in Perth

Zheng, F., S. Westra, and S. A. Sisson, (2013), Quantifying the dependence between extreme rainfall and storm surge in the coastal zone, Journal of Hydrology, 505, 172-187

13 Glossary

13.1 Hydrologic terms

AEP: Annual Exceedance Probability – is a measure of the likelihood (expressed as a probability) of a flood event reaching or exceeding a particular magnitude in any one year. A 1% (AEP) flood has a 1% (or 1 in 100) chance of occurring or being exceeded at a location in any year

AHD: Australian Height Datum (m), the standard reference level in Australia

AR&R: Australian Rainfall and Runoff (AR&R) is a national guideline document for the estimation of design flood characteristics in Australia. It is published by Engineers Australia. The current 2003 edition is now being revised. The revision process includes 21 research projects, which have been designed to fill knowledge gaps that have arisen since the 1987 edition

CHA: Comprehensive Hydrologic Assessment

CL: Continuing Loss (mm/hour). The amount of rainfall during the later stages of the event that infiltrates into the soil and is not converted to surface runoff in the hydrologic model

CRC-CH: Cooperative Research Centre – Catchment Hydrology. In this report, CRCH-CH usually refers to a Monte Carlo sampling method that was developed by the CRC-CH

CSS: Complete Storm Simulation. This is one of the proposed Monte Carlo sampling methods

Cumulative probability: The probability of an event occurring over a period of time, any time in that period. This probability increases over time

DEA: Design Event Approach. A semi-probabilistic approach to establish flood levels, which only accounts for the variability of the rainfall intensity

Design flood event: Hypothetical flood events based on a design rainfall event of a given probability of occurrence (ie AEP). The probability of occurrence for a design flood event is assumed to be the same as the probability of rainfall event upon which it is based (EA, 2003)

DMT: Disaster Management Tool. Work completed by BCC in 2014 for Queensland Government as part of the development of an interim disaster management tool until the completion of the BRCFS

DTM: Digital Terrain Model

EL (m AHD): Elevation (in metres) above the Australian Height Datum

FFA: Flood Frequency Analysis – a direct statistical assessment of flood characteristics

Flood mitigation manual (Flood Manual): A flood mitigation manual approved under section 371E(1)(a) or 372(3) of the Water Supply (Safety and Reliability) Act 2008 (QLD)

FOSM: Flood Operations Simulation Model (refer Seqwater 2014)

Floodplain: Area of land adjacent to a creek, river, estuary, lake, dam or artificial channel, which is subject to inundation by the PMF (CSIRO, 2000)

FSL: Full Supply Level – maximum normal water supply storage level of a reservoir behind a dam

FSV: Full Supply Volume – volume of the reservoir at FSL

GEV: Generalised Extreme Value statistical distribution

GIS: Geographic Information System

GL: Gigalitres This is a unit of volume used in reservoir studies. A Gigalitre = 1,000,000,000 litres or equivalently 1,000,000 m^3

GSDM: Generalised Short Duration Method of extreme precipitation estimation for storms of less than 6 hour duration and catchments of less than 1,000 km^2. Refer BoM, 2003

GTSMR: Revised Generalised Tropical Storm Method of extreme precipitation estimation for storms of tropical origin. Applicable to storm durations of up to 168 hours and catchments up to 150,000km^2. Refer BoM, 2003

IFD-curves: Intensity-Frequency-Duration curves, describing the point- or area-rainfall statistics. In the current report rainfall depth is generally used as an alternative to rainfall intensity. Rainfall depth is the product of duration and intensity. It was decided to maintain the term "IFD" as this is the terminology that the reader is most likely to be familiar with

IL: Initial Loss (mm). The amount of rainfall that is intercepted by vegetation or absorbed by the ground and is therefore not converted to runoff during the initial stages of the rainfall event

LOC: Loss of Communications dam operating procedure, refer Flood Manual (Seqwater 2013)

LPIII: Log-Pearson Type III statistical distribution

IQQM: Integrated Quantity and Quality Model for water resources planning

JPA: Joint Probability Approach. A general term for probabilistic methods to establish design flood levels

MCS: Monte Carlo Simulation

MHWS: Mean High Water Spring Tide level

ML: Megalitre. This is a unit of volume used in reservoir studies. A megalitre is equal to 1,000,000 litres or, equivalently, 1,000 m^3

m^3/s: Cubic metre per second – unit of measurement for instantaneous flow or discharge

PMF: Probable Maximum Flood – the largest flood that could conceivably occur at a particular location, resulting from the PMP (CSIRO, 2000) and Australia Rainfall and Runoff, 2003 (EA, 2003)

PMP: Probable Maximum Precipitation – the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of year, with no allowance made for long-term climatic trends (CSIRO, 2000; EA 2003)

PMP DF: Probable Maximum Precipitation Design Flood – the flood event that results from the PMP event

Quantiles: Values taken at regular intervals from the inverse of the cumulative distribution function (CDF) of a random variable.

Stochastic flood event: Statistically generated synthetic flood event. Stochastic flood events include variability in flood input parameters (eg temporal and spatial rainfall patterns) compared to design flood events. Stochastic flood events by their method of generation exhibit a greater degree of variability and randomness compared to design flood events (See also Design flood event)

Synthetic flood event: See Stochastic flood event

TPT: Total Probability Theorem. This is one of the fundamental theorems in statistics. In this report, TPT refers to a Monte Carlo sampling method that is based on stratified sampling and, hence, makes use of the total probability theorem

URBS: Unified River Basin Simulator. A rainfall runoff routing hydrologic model (Carroll, 2012)

13.2 Study related terms

BCC: Brisbane City Council

BoM: Australian Bureau of Meteorology

BRCFS: Brisbane River Catchment Flood Study

BRCFM: Brisbane River Catchment Floodplain Management Study

BRCFMP: Brisbane River Catchment Floodplain Management Plan

Delft-FEWS: Flood Early Warning Systems, a software package developed by Deltares, initially for the purpose of real-time flood forecasting. Delft-FEWS is used all over the world, including by the Environment Agency (UK) and the National Weather Service (US). Currently, it is also being implemented by Deltares and BoM for flood forecasting in Australia. The Monte Carlo framework for the BRCFS-Hydrology Phase will be implemented in Delft-FEWS

DEWS: Department of Energy and Water Supply

DIG: Dams Implementation Group

DNRM: Department of Natural Resources and Mines

DSITIA: Department of Science Information Technology, Innovation and the Arts

DSDIP: Department of State Development and Infrastructure Planning

EA: Engineers Australia formally known as The Institute of Engineers, Australia

GA: General Adapter, an interface between the Delft-FEWS environment and an external module

IC: Implementation Committee of the BRCFS

ICC: Ipswich City Council

IPE: Independent panel of experts to the BRCFS

LVRC: Lockyer Valley Regional Council

ND: No-dams condition. This scenario represents the catchment condition without the influence of the dams and reservoirs. The reservoir reaches have effectively been returned to their natural condition

NPDOS: North Pine Dam Optimisation Study conducted in response to the QFCOI Final Report

PIG: Planning Implementation Group

QFCOI: Queensland Floods Commission of Inquiry

RTC: Real-Time Control. A software package for simulations of reservoir operation. RTC tools is used for the simulation of Wivenhoe and Somerset reservoirs

SC: Steering Committee of the BRCFS

SRC: Somerset Regional Council

TWG: Technical Working Group

WD: With-dams condition. This scenario represents the catchment condition with the influence of the dams and reservoirs represented in their current (2013) configuration

WSDOS: Wivenhoe and Somerset Dam Optimisation Study conducted in response to the QFCOI Final report

Appendices

Appendix A
Data, rating curve and historical flood review report

Appendix B
Hydrologic model calibration and verification review report

Appendix C Hydrologic model recalibration report

Appendix D
Monte-Carlo simulation report

Appendix E
Dam operations module implementation report

Appendix F
Flood frequency analysis report

Appendix G
Design event approach report

Appendix H
Assessment of the implications of climate change on flood estimation – discussion paper

Appendix I Reconciled and recommended flood frequency estimates report

Aurecon Australasia Pty Ltd
ABN 54 005 139 873
Level 14, 32 Turbot Street
Brisbane QLD 4000
Locked Bag 331
Brisbane QLD 4001
Australia

T +61 7 3173 8000
F +61 7 3173 8001
E brisbane@aurecongroup.com
W aurecongroup.com