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Brisbane River Catchment Flood Study: Comprehensive Hydrologic Assessment

Reconciled and Recommended Flood Frequency Estimates Report

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

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Aurecon team

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

Executive summary

Brisbane River catchment flood study

The State of Queensland, Australia, initiated a comprehensive hydrologic assessment as part of the Brisbane River Catchment Flood Study (BRCFS) in response to the devastating floods in January 2011 and subsequent recommendations of the Queensland Floods Commission of Inquiry. In accordance with this recommendation, the State of Queensland is managing the conduct of this study in a number of separate phases. Aurecon Team, consisting of Aurecon, Deltares, Don Carroll project management and Royal HaskongDHV, was commissioned to undertake the second phase of the study: a Comprehensive Hydrologic Assessment (CHA). The main objective of the CHA is to produce a set of competing methods for estimating design floods in the Brisbane River catchment, followed by an extensive reconciliation process to identify the most reliable design flood estimates of a range of flood flows for annual exceedance probabilities across the entire Brisbane River system. The CHA needs to consider two scenarios, referred to as 'no-dams' and 'with-dams'. The dams referred to are the major water storages that exist within the catchment; these are Somerset Dam and Wivenhoe Dam, both of which have flood mitigation capacity. Other reservoirs considered are the Cressbrook, Manchester, Moogerah and Perseverance dams. In the model simulations, the level of urban development in the 'no-dams' condition 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.

The results of the comprehensive hydrologic assessment will serve as input for a hydraulic assessment, which is also part of the Brisbane River Catchment Flood Study. The main purpose of the hydraulic assessment is to derive flood levels, whereas the hydrologic assessment serves to derive flood flows (peak flows and volumes).

Objective of this report

This report describes the estimated design peak flows and design flood volumes for different methodologies. Reconciled and recommended design flows for 'no-dams' and 'with-dams' conditions are proposed on the basis of these results.

Considered methodologies

Three approaches have been applied to estimate AEP peak flows and flow volumes:

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

Flood frequency analysis is an assessment of flows measured directly at the site. A subset of flows (peaks and/or volumes) of flood events is composed either by selecting the highest flow for each year in the available series of recording or by selecting all flows above a user-defined threshold. A statistical distribution function is derived from the set of high flows in such a way that exceedance probabilities of the distribution function are as much as possible in accordance with selected rated flows. The fitted distribution function describes the relation between AEP and associated design flows.

The Design Event Approach and Monte Carlo Simulations are both rainfall based methods, which means flow statistics are based on rainfall statistics in combination with rainfall-runoff and reservoir modelling. In the Design Event Approach, a spatially uniform rainfall distribution (uniform with respect to AEP) across the catchment is assumed in the simulations. A fundamental assumption is that the flood AEP is equal to the AEP of the causal rainfall, which is not necessarily the case. For this reason it is necessary to adopt 'AEP neutral' losses and 'AEP neutral' initial reservoir volumes. However, there does not appear to be a broadly accepted approach to define 'AEP neutral' initial reservoir volumes. In the current study, the reservoirs are assumed to be at full supply level at the beginning of each simulated event in the DEA approach. Monte-Carlo Simulation removes many of the limitations common to Design Event methodologies. In the Monte Carlo Simulations approach, a variety of randomly generated synthetic events is simulated. The method has the advantage over more "traditional" approaches in flood risk analysis in that it explicitly considers all relevant factors that contribute to flood events, including (variability in) rainfall depth, spatial and temporal distribution of rainfall, antecedent soil moisture conditions, initial reservoir volumes and ocean water levels. The likelihood of *combined* occurrences of these factors is also taken into account. The AEP of the simulated flows are based on the ordering of all simulated flows which means flood AEP is generally not equal to the AEP of the causal rainfall.

Reconciliation process

For 'no-dams' conditions, DEA and MCS results were available for all 22 nominated locations of interest. FFA results were available for 17 locations as no (reliable) series of rated flows was available for the other 5 locations. Loss parameters in the DEA and MCS methods were chosen in such a way that resulting flood frequency curves were in accordance with FFA flood frequency curves and probability estimates from rated flows. This was accomplished by adopting three sets of loss parameters:

1. Loss parameters for Stanley and Bremer locations (relatively low losses)
2. Loss parameters for Lockyer locations (relatively high losses)
3. Loss parameters for Upper and Lower Brisbane locations ('intermediate losses)

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

- Probability estimates from rated flows for 'frequent' events – range of 'high' AEP values
- Flood frequency analysis results for frequent to 'large' events – range of 'intermediate' AEP values
- Monte Carlo Simulations results for 'large' to 'extreme' events – range of 'low' AEP values

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

For 'with-dams' conditions, rated flows were available for seven out of eight nominated locations. The series contained approximately 30 years of rated flows for most locations, which was considered too short to derive a flood frequency distribution with the FFA approach. For 'with-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 – range of 'high' AEP values
- Monte Carlo Simulations results for frequent events to extreme events – range of 'intermediate' and 'low' AEP values

For locations for which no (reliable) rated flows on peak flows were available and, hence, no FFA results as well, the reconciled design flows for all values of the AEP values were entirely based on MCS simulation results.

Results

The main output of the reconciliation process consists of:

1. Reconciled 'no-dams' design peak flows for 22 locations
2. Reconciled 'no-dams' 24 hour-, 48 hour- and 72 hour design flow volumes for 22 locations
3. Reconciled 'with-dams' design peak flows for eight locations
4. Reconciled 'with-dams' 24 hour-, 48 hour- and 72 hour design flow volumes for eight locations

These results are presented in various Tables and Figures. Separate Tables are provided for the design estimates of the three individual methods (FFA, DEA and MCS) and the reconciled design flows.

Conclusions

No-dams

- 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 flows 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
- Design peak flows 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. 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

With-dams

- 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
- .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 flows
- For location Somerset Dam, 'With-dams' peak flows of individually simulated events are often higher than corresponding 'no-dams' peak flows, 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 (see section 6.1). This shows the validity of this approach is doubtful. It is therefore recommended not to adopt the derived 'with-dams' results for location Somerset Dam
- The reducing effects of the dams on peak flows 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 the 1 in 100 AEP peak discharge at Moggill from 14,600 m^3/s to 10,200 m^3/s
- 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

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1 Introduction

1.1 Brisbane River catchment flood study

The Queensland Floods Commission of Inquiry Final Report, which was issued in March 2012, contains a recommendation (Recommendation 2.2) that requires 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 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 and Brisbane River Floodplain Management Plan

The Aurecon Team, consisting of Aurecon, Deltares, Don Carroll project management and Royal HaskongDHV was commissioned to undertake the Comprehensive Hydrologic Assessment (CHA). This assessment needs to be comprehensive with a requirement for various methodologies to be utilised and for them to corroborate each other. The main objective of the CHA is to develop and apply a state-of-the-art method that produces consistent and robust hydrologic models and analytical techniques that will enable the CHA to provide best estimates of a range of flood flows for annual exceedance probabilities across the entire Brisbane River system.

The method needs to be able to account for two scenarios: the conditions referred to as ‘no-dams’ and ‘with-dams’. The dams referred to are the major water storages that exist within the catchment; these are Somerset Dam and Wivenhoe Dam, both of which have flood mitigation capacity. Other reservoirs considered are the Cressbrook, Lake Manchester, Moogerah and Perseverance dams. The level of urban development in the ‘no-dams’ scenario 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. For each scenario, a separate hydrological model was implemented; the difference being the inclusion of the dams, including the operating rules, and the impervious areas that represent the reservoir areas.

In the Comprehensive Hydrologic Assessment, three approaches are used to estimate peak flows 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 rated flows.

1.2 Scope of report

The current report describes the reconciliation process in which the rainfall based flood estimates are reconciled with results from flood frequency analyses and probability estimates from rated flows. Any significant differences between the results of different methods are documented and explained. The assessment accounts for the two scenarios: ‘no-dams’ and ‘with-dams’.

2 Reconciliation procedure

2.1 Sources of flow estimate

In the BRCFS, three approaches have been used to estimate AEP peak flows and flow volumes:

1. Flood Frequency Analysis (FFA); see (Aurecon 2015a)
2. Design Event Approach (DEA)); see (Aurecon 2015b)
3. Monte Carlo Simulation (MCS)[1]; see (Deltares 2015)

When reconciling the available data it is important to recognise the strengths and limitations of each method.

Flood frequency analysis is an assessment of flows measured directly at the site. The first step in the approach is the composition of a set of rated flows (peaks and/or volumes) of flood events. This is done by either selecting the highest flow for each year in the available series of recording or by selecting all flows above a user-defined threshold. Subsequently, a statistical distribution function is derived in such a way that exceedance probabilities of the distribution function are as much as possible in accordance with the selected rated flows. The fitted distribution function describes the relation between AEP and associated design flows. The reliability of the design flow estimates 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 rated flows 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 and Monte Carlo Simulations are both rainfall based methods, which means flow statistics are based on rainfall statistics in combination with simulations of hydrological processes and reservoir operations. These approaches have the advantage of allowing changes in catchment conditions, such as the presence of dams, to be explicitly modelled.

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 in this method is that flood AEP is equal to the AEP of the causal rainfall, which is not necessarily the case. It is necessary to adopt ‘AEP neutral’ losses and initial reservoir volumes that are typically higher for frequent events and decrease with flood magnitude.

[1] MCS computations were carried out with the ‘TPT-method’ for sampling rainfall depth.

Monte-Carlo Simulation removes many of the limitations common to Design Event methodologies. In the Monte Carlo Simulations approach, a variety of synthetic events is simulated. The method has the advantage over the other two approaches in that it explicitly considers all relevant factors that contribute to flood events, including rainfall depth, spatial and temporal distribution of rainfall, antecedent soil moisture conditions, initial reservoir volumes and ocean water levels. Furthermore, the likelihood of combined occurrences of these factors is taken into account. The AEP of the simulated flows are based on the ordering of all simulated flows which means that flood AEP is generally not equal to the AEP of the causal rainfall. Monte Carlo Simulations 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. The method is therefore considered to be especially advantageous for locations along the Lower Brisbane. The MCS approach has also the advantage, for the with-dams scenario, of capturing the influence of varying initial water levels in storages.

Both Monte Carlo Simulations and Design Event Approaches are dependent on hydrologic modelling and the factors that influence the relationships between rainfall, runoff and flow. These factors, such as initial losses 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 Monte Carlo Simulations and Design Event Approaches over flood frequency analysis 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 in the system, such as flow capacities, on flood frequencies.

2.2 Consistency of data source predictions

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:

- Rated flows are consistent with hydrologic model flows. Calibration of the URBS models (Aurecon, 2014) 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

2.3 Event magnitude classes

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

[i] Frequent events: AEP values ranging from 1 in 2 to 1 in 50

[ii] Large events: AEP values ranging from 1in 50 to 1 in 100

[iii] Rare events: AEP values ranging from 1 in 100 to 1 in 2,000

[iv] Extreme events: AEP values ranging from 1 in 2,000 to the AEP of the Probable Maximum Precipitation (PMP)

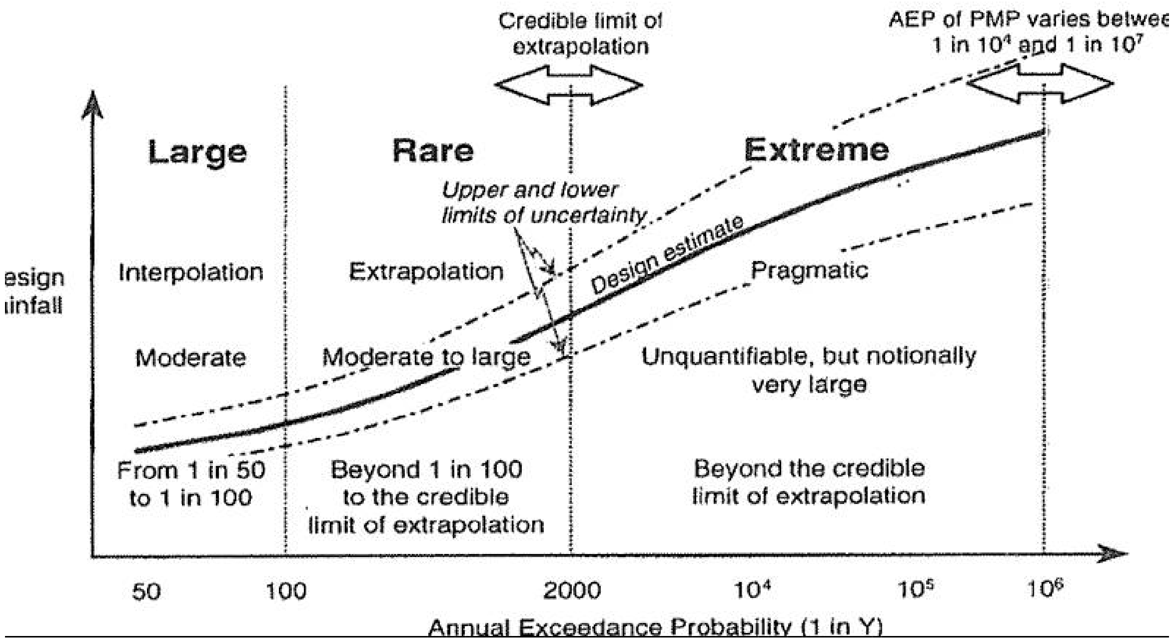


Figure 2-1 Design characteristics of notional design event classes (Figure adopted from ARR book VI)

Class [i] – frequent events. In this range, the results of the FFA and empirical estimates from rated flows would generally be afforded highest priority, unless rating curves at the specific location are considered unreliable. For these events, rainfall statistics of DEA and MCS are based on Intensity Duration Frequency (IFD) tables from the BoM (Green et al., 2012).

Class [ii] – large events. In this range, FFA and rated flows are expected to provide the most reliable results if a long series of reliable rated flows is available. Otherwise, the DEA/MCS methods are preffered. For these events, the rainfall statistics of DEA and MCS methods are also based on Intensity Duration Frequency (IFD) tables from the BoM.

Class [iii] – rare events. In this range, extrapolation techniques are required. FFA estimates are generally considered unreliable for this range, which is why DEA and MCS estimates are preferred. Rainfall statistics of DEA and MCS are mostly based on the “CRC-FORGE” method (Nandakumar et al, 1997).

Class [iv] – extreme events. In this range, IFD curves are mostly based on the GDSM (Bureau of Meteorology, 2003a) and GTSMR (Bureau of Meteorology, 2003b) methods. For this class, again, DEA and MCS estimates are preferred over FFA estimates. The estimated design flows in this range strongly depend on the estimated value of the AEP of the PMP (Probable Maximum Precipitation). This AEP is estimated with the following formula (ARR, book VI):

$$AEP_{PMP} = 10^{{}^{10}\log(A)-9} \quad ; \mathrm{A} \geq 100 \qquad (1)$$

In this formula, *A* is the catchment size in km^2. The AEP values that follow from this formula are *best estimates*, but there is a high degree of uncertainty to these estimates. According to (ARR book VI), the uncertainties in the estimate are quantified as follows:

- The recommended AEP values plus or minus one order of magnitude of AEP should be regarded as the confidence limits with about 75% subjective probability that the true AEP lies within these limits

- The recommended AEP values plus or minus two orders of magnitude of AEP should be regarded as the notional upper and lower limits for the true AEPs

For example, location Savages Crossing has a best estimate AEP of the PMP equal to 1 in 100,000. According to the first bullet above, it can be stated with 75% certainty that the actual value of this AEP is somewhere between 1 in 1,000,000 and 1 in 10,000. According to the second bullet above, the absolute lower and upper limits of this AEP are equal to 1 in 10,000,000 and 1 in 1,000. This shows the large degree of uncertainty involved in the estimate of the AEP of the PMP. Estimated design flows for AEP- values below 1 in 2,000 (the range of extreme events, see Figure 2-1) depend strongly on the estimated AEP of the PMP and are therefore highly uncertain.

2.4 Reconciliation process for the 'no-dams' scenario

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. 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 chapter 4. 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 flows
- 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' will be 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' scenario and also more realistic design flow hydrographs.

There 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.

2.5 Reconciliation of 'with dam' scenario results

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. The validity and use of this concept may need to be discussed further, as there does not appear to be a broadly accepted approach to define 'AEP neutral' starting dam levels. In the current study, the reservoirs are assumed to be at full supply level at the beginning of each simulated event in the DEA approach. For MCS this is in not the case, as the starting dam water level is randomly generated from the derived distribution functions as described in chapter 6 of (Deltares, 2015). The flexibility of the MCS method is such that starting dam water levels can also be assumed equal to full supply level. The influence of starting dam water levels (random versus full supply level) on design water levels was assessed in a sensitivity analysis. For a discussion on this sensitivity analysis, the interested reader is referred to the main report of the *Brisbane River Catchment Flood Study – Final Hydrology Report*, (Aurecon, 2015).

3 Locations of interest

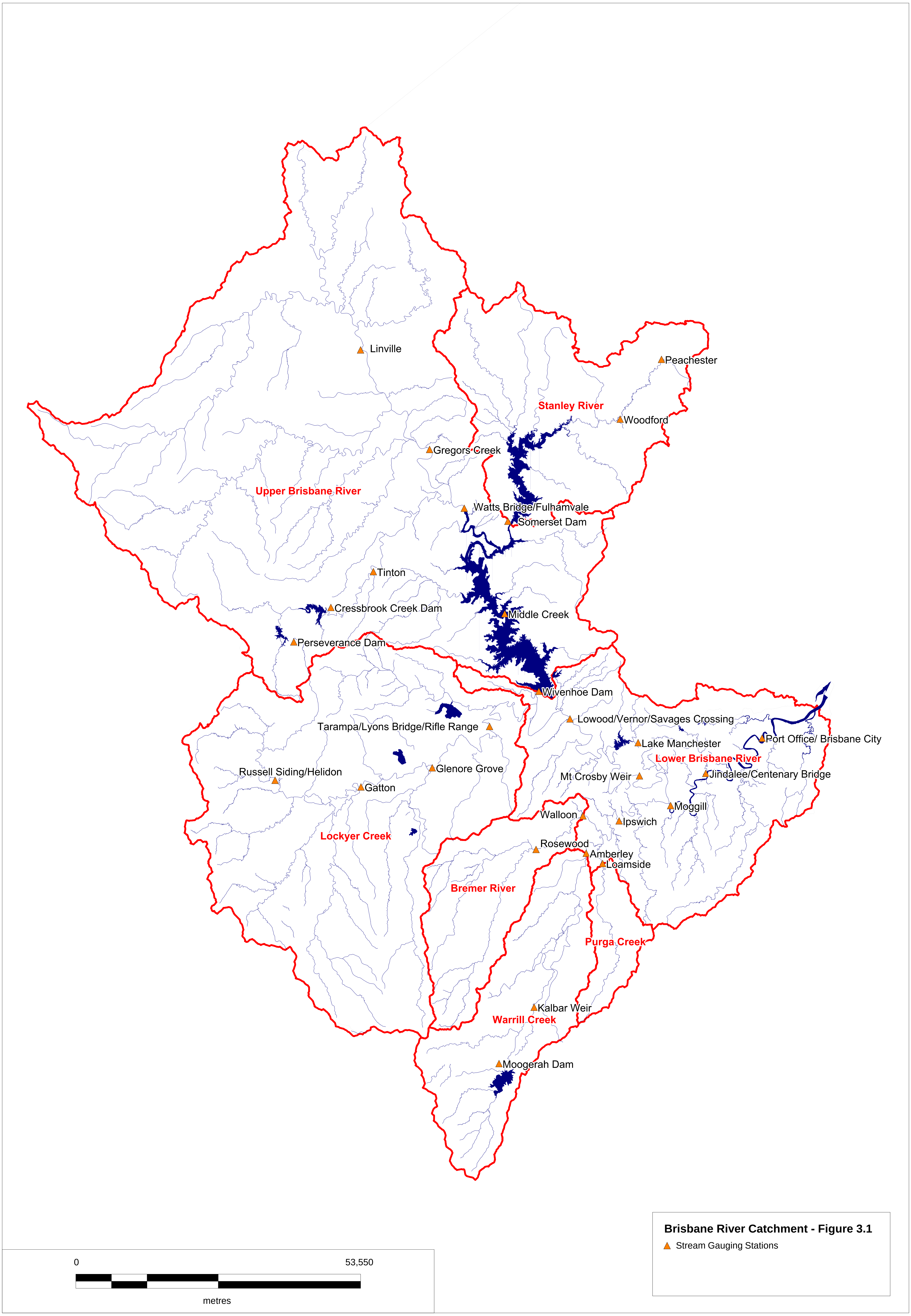
Table 3-1 shows 22 locations for which frequency curves have been derived. These locations are also shown in Figure 3-1. For an extensive explanation of the selection of these locations, the reader is referred to the report of (Aurecon, 2015a); a summary is presented below.

The Project Brief specified a minimum list of 19 sites throughout the catchment to be included in the flood frequency analysis for the BRCFS. During the course of the project, six sites were added:

- Somerset Dam (inflow)
- Wivenhoe Dam (inflow)
- Lockyer Creek at Gatton
- Stanley River at Woodford
- Lockyer Creek at Glenore Grove
- Warrill Creek at Kalbar Weir

At the same time, three sites were considered of little added value to the project and, hence, removed from the list:

- Rosewood: Comparison of the pre-1955 URBS model flows with the post-1955 gauge record suggests that the modelled storm events (including 1893) are not particularly significant events in the Bremer River catchment. Inclusion as a historical data set was therefore not recommended
- Silverton: This gauge is closed although data is available between 1919 and 1968, records since 1942 are affected by Somerset Dam in varying degrees of construction. The reliability of any rating and flood frequency analysis at this site are therefore limited value. The records were used, however, to extend the rated flow series for location Somerset Dam
- Rifle Range Road: At this location, Lockyer Creek is a perched channel in a wide floodplain with very limited/unreliable response above bank-full capacity



Brisbane River Catchment - Figure 3.1

Stream Gauging Stations

Table 3-1 Overview of locations of interest

No.	Location	area (km^2)	River	AEP of PMP (1 in X)	Period of rated flows used in the analysis
1	Linville	1,996	Upper Brisbane	500,000	1965-2013
2	Gregors Creek	3,849	Upper Brisbane	260,000	1963-2013
3	Fulham Vale	4,553	Upper Brisbane	220,000	1920-2011
4	Peachester	103	Stanley	9,710,000	1928-2013
5	Woodford	245	Stanley	4,070,000	1887-2013
6	Somerset Dam	1,324	Stanley	750,000	1955-2013
7	Tinton	423	Cressbrook	2,360,000	-
8	Middle Ck	6,665	Upper Brisbane	150,000	-
9	Wivenhoe	6,980	Upper Brisbane	140,000	1893-2013
10	Helidon	351	Lockyer	2,840,000	1927-2013
11	Gatton	1,527	Lockyer	650,000	1887-2013
12	Glenore Grove	2,149	Lockyer	460,000	1955-2013
13	Savages Crossing	10,126	Lower Brisbane	100,000	1887-2013
14	Mount Crosby	10,507	Lower Brisbane	90,000	1887-2013
15	Walloon	634	Bremer	1,570,000	1962-2013
16	Kalbar Weir	458	Warrill	2,180,000	-
17	Amberley	902	Warrill	1,110,000	1962-2013
18	Loamside	209	Purga	4,770,000	1887-2013
19	Ipswich	1,850	Bremer	540,000	-
20	Moggill	12,578	Lower Brisbane	80,000	1887-2013
21	Centenary Bridge	12,877	Lower Brisbane	80,000	-
22	Brisbane	13,198	Lower Brisbane	80,000	1841-2013

The last column of Table 3-1 shows the period in which rated flows were gathered that were used in the analyses. Locations for which no (reliable) rated flow estimates are available are indicated with a '-' in the last column of Table 3-1. The years shown in this column are the first and last year of the series of rated flows but rated flows are not necessarily available for all the years in between. Rated flows in some cases have not been measured directly at site. For example, the Wivenhoe record is only available from 1983 to 2013, but the 'no-dams' series was augmented with rated flows from Middle Creek and Caboonbah, which has records back to 1890. Further details of the composition of these series of rated flows are described in (Aurecon, 2015a). The fifth column of Table 3-1 shows the AEP (annual exceedance probability) of the PMP (probable maximum precipitation). These were derived through application of equation (1).

4 Computation settings

4.1 Reconciled losses

In order to reconcile the 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. At the same time, spatial variation of loss parameters was reduced as much as possible. This reconciliation procedure, described in (Deltares, 2014) resulted in three different sets of loss parameters:

1. Loss parameters for Stanley and Bremer sub-catchments (relatively low losses)
2. Loss parameters for the Lockyer sub-catchment (relatively high losses)
3. Loss parameters for Upper and Lower Brisbane locations ('intermediate' losses)

The loss parameters increase with increasing rainfall AEP. For a given rainfall AEP, each loss parameter is a constant, except the MCS initial loss parameter. In the MCS approach, initial loss parameters are median values of a beta-distribution (see Deltares, 2015).

Table 4-1 to Table 4-6 show adopted initial and continuing loss parameters for all sub-catchments. The proposed continuing losses for the MCS framework are higher than the DEA continuing losses, accounting for the fact that the application of the same set of continuing loss parameters would result in higher total losses in the DEA runs because DEA rainfall patterns are more uniformly distributed in space and time (see Deltares, 2014).

Table 4-1 Initial loss parameters for Bremer and Stanley sub-catchments

Rainfall AEP (1 in N)	DEA	MCS
2	40mm	40mm
5	32mm	32mm
10	24mm	24mm
20	16mm	16mm
50	8mm	8mm
100	0mm	8mm

Table 4-2 Continuing loss parameters for Bremer and Stanley sub-catchments

Rainfall AEP (1 in N)	DEA	MCS
2	2.0mm/hr	2.5mm/hr
5	1.6mm/hr	2.0mm/hr
10	1.2mm/hr	1.5mm/hr
20	0.8mm/hr	1.0mm/hr
50	0.4mm/hr	0.5mm/hr
100	0.4mm/hr	0.5mm/hr

Table 4-3 Initial loss parameters for the Lockyer sub-catchment

Rainfall AEP (1 in N)	DEA	MCS
2	60mm	70mm
5	48mm	60mm
10	36mm	50mm
20	24mm	40mm
50	12mm	30mm
100	0mm	20mm

Table 4-4 Continuing loss parameters for the Lockyer sub-catchment

Rainfall AEP (1 in N)	DEA	MCS
2	3.0mm/hr	6.0mm/hr
5	2.4mm/hr	4.0mm/hr
10	1.8mm/hr	3.0mm/hr
20	1.2 mm/hr	2.5mm/hr
50	0.6mm/hr	2.0mm/hr
100	0.6mm/hr	1.0mm/hr

Table 4-5 Initial loss parameters for Upper and Lower Brisbane sub-catchments

Rainfall AEP (1 in N)	DEA	MCS
2	50mm	50mm
5	40mm	40mm
10	30mm	30mm

Rainfall AEP (1 in N)	DEA	MCS
20	20mm	20mm
50	10mm	10mm
100	0mm	10mm

Table 4-6 Continuing loss parameters for Upper and Lower Brisbane sub-catchments

Rainfall AEP (1 in N)	DEA	MCS
2	2.5mm/hr	6.0mm/hr
5	2.0mm/hr	3.0mm/hr
10	1.5mm/hr	2.5mm/hr
20	1.0mm/hr	1.5mm/hr
50	0.5mm/hr	0.5mm/hr
100	0.5mm/hr	0.5mm/hr

4.2 Considered burst durations

Frequency curves resulting from MCS and DEA approaches were derived for all locations of Table 3-1. In both approaches, frequency curves were derived by taking the envelope of the frequency curves for individual burst durations. In the DEA approach, frequency curves were derived for the following 14 durations: 1, 2, 3, 6, 12, 18, 24, 36, 48, 72, 96, 120, 144 and 168 hours. In the MCS approach, only a subset was considered; an upper limit was defined for the durations. The upper limit depends on the size of the catchment and on the considered scenario (‘no-dams’ or ‘with-dams’). The motivation for introducing such an upper limit is to prevent the undesired effect that ‘within bursts’ influence design peak flows (see section 5.4.3. of the *Monte Carlo Simulation Framework and Methodology Report,* Aurecon, 2015, for a more extensive discussion on ‘within bursts’ and the motivation for the upper limit). The selected upper limit increases with increasing catchment size for the ‘no-dams’ situation, as the critical duration tends to increase with increasing catchment size. For ‘with-dams’ conditions, the focus is on the larger sub-catchments and effects of within-bursts can be attenuated significantly by the dams. Therefore, the upper limit of the considered bursts durations is higher for the ‘with-dams’ situation. The impact of the different durations was examined at Woodford, were longer durations were included in the ‘no-dams conditions’ simulations.

Table 4-7 Maximum burst durations considered

catchment area (km^2)	maximum burst duration (hrs)	
	No-dams	With-dams
<1,000	24	-
<5,000	48	120
<10,000	72	120
>10,000	96	120

4.3 Selection of Monte Carlo simulation events

In the MCS approach, simulations were carried out for combinations of 8 burst durations and 60 values of the rainfall AEP. For each combination of duration and rainfall AEP, 21 events are generated 'through' random sampling of the spatio-temporal rainfall patterns, initial reservoir volumes, initial losses and ocean water levels. This means for each location $8 \times 60 \times 21 = 10{,}080$ synthetic events are simulated. The catchment average rainfall depth in each simulation is obtained from IFD Tables and areal reduction factors. Each location of interest has its own unique catchment IFD table and, as a consequence, each location has its unique set of 10,080 synthetic rainfall events that are generated and simulated in the MCS framework.

For the downstream locations along the Lower Brisbane River (Moggill, Centenary Bridge, Brisbane), the catchment IFD-curves are very similar. Therefore, it was decided to carry out the same set of 10,080 event simulations for these three locations, using the IFD curves for location Brisbane. This increases the mutual consistency of the MCS simulation results for these three locations and this benefit was considered to outweigh the slight change in rainfall statistics for locations Moggill and Centenary Bridge. Similarly, it was decided to carry out the same set of 10,080 event simulations for locations Savages Crossing and Mount Crosby, using the IFD curves for location Mount Crosby.

5 Reconciled ‘no-dams’ estimates

5.1 Introduction

The approach to the reconciliation process was described in section 2. The current chapter describes the results and choices in more detail.

Appendix A.1 contains tables with reconciled design flows for a range of AEP values for the 22 locations of interest, as well as estimated design flows for the three applied methods (FFA, DEA and MCS. Appendix A.2 contains figures with frequency curves for the 22 locations of interest. Each Figure contains the following graphs:

- Plotting positions of rated peak flows (if available)
- FFA results (if available)
- FFA 90 percent uncertainty quantiles (if available)
- DEA results
- MCS results
- Proposed reconciled design peak flows

For the plotting positions of rated peak flows, the following formula was used:

$$P = \frac{r - c}{N + 1 - 2c} \qquad (2)$$

In which:

P = probability of exceedance (plotting position)

r = ranking number (1 = highest peak discharge, 2 = second highest peak discharge etc)

c = plotting constant, taken equal to 0.4

For each location, two figures are provided:

1. Results for the AEP range 1 in 2 – AEP of PMP
2. Results for the AEP range 1 in 2 – 1 in 100

The first Figure shows all results available for a single location, the second Figure zooms in on the range of AEP values where a meaningful comparison between FFA and the rainfall based methods can be made. Section 5.3 describes how the reconciled design flows were derived. First, section 5.2 discusses the results of FFA, DEA and MCS approaches.

5.2 Comparison of estimated design flows

The following general observations are made from the Figures of Appendix A.2:

1. For AEP<1in 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 flows 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
2. 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
3. Design peak discharge for the Bremer sub-catchment locations Amberley, Walloon and Loamside are underestimated by both the DEA and MCS methods
4. There are some other "individual" mismatches between DEA/MCS on one hand and the rated flows/FFA on the other hand:
   i) Linville and Gregors Creek: slight mismatch between DEA and FFA for AEP=1 in 2
   ii) Peachester: DEA and MCS design flows are significantly lower than FFA design flows
   iii) Helidon: MCS design flows appear to be too low for AEP =1 in 2 and 1 in 5
   iv) Gatton and Glenore Grove: DEA design flows appear to be too high for a range of AEP values
   v) Savages Crossing: MCS design flows appear to be too high for AEP =1 in 2

As stated above, the MCS and DEA design peak flows are low compared to FFA peak flows for locations Amberley, Walloon and Loamside. These three locations are all situated in the Bremer sub-catchment (including Warrill and Purga sub-catchments). A better match might be obtained by reducing the loss parameters, but adopted loss parameters for the Bremer catchment are already lower than adopted loss parameters of other catchments (see section 4.1). For location Amberley, two observed events have a peak discharge that, according to the DEA and MCS results, have an AEP<1 in100. The highest of these two flows even has an AEP<1 in 1,000 according to the DEA and MCS results. Taking into account that the length of record is approximately 50 years, this is suspicious (though not impossible).

The critical duration for Amberley according to the DEA method is 24 hours. The BoM IFD table for location Amberley that was used as input for the DEA and MCS methods shows that the 24 hour rainfall depth with AEP=1 in 500 is equal to 273 mm for the Amberley catchment. Since 1887, there have been three events with a 24-hour rainfall depth approximately equal to this rainfall depth (see Table 5-1). So, according to the BoM IFD tables, there have been three 24-hour rainfall bursts in a period of 130 years, all with an AEP of approximately 1in 500. Again, this is not impossible, but very suspicious.

Table 5-1 24 hour rainfall and corresponding peak flows for location Amberley

Event	Peak discharge (m^3/s)	24hr rainfall (mm)
19760209	1394	198
1 in 100 AEP	1690	213
20130123	1955	274
1 in 500 AEP	2282	273
18870119	2738	279
19740124	2787	263

This is an indication that rainfall depths of IFD curves for this location may significantly underestimate the 'actual' rainfall depths, which would largely explain why DEA and MCS peak flows are significantly lower than FFA peak flows. The results for nearby locations Loamside and Walloon are additional evidence in favour of this hypothesis. Further support for this hypothesis is provided by the memo of (WMAWater, 2014), in which rainfall depths from BoM IFD curves are compared with observed rainfall depths for several stations in the Bremer sub-catchment. The comparison showed that BoM IFD curves generally underestimate rainfall depth in the Bremer sub-catchment for the high rainfall events.

5.3 Reconciled peak flows

5.3.1 Introduction

This section discusses the procedure that was applied to derive reconciled design peak flows for 'no-dams' conditions. 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 to derive reconciled design peak flows. The subdivision in clusters is based on two criteria:

a. Data availability
b. Differences in design peak flows between the various methods

The following sections describe the reconciliation process for each of the six clusters.

5.3.2 Cluster 1

Table 5-2 shows the twelve locations of cluster 1. These are all locations for which rated flows and FFA results are available. The following approach was used in the reconciliation process for these locations:

- Range of 'high' AEP values ($AEP \geq p_1$): design peak flows estimated directly from plotting positions of rated flows
- Range of 'Intermediate' AEP values ($p_2 < AEP < p_1$): design peak flows estimated from FFA results
- Range of 'low' AEP values ($AEP \leq p_2$): design peak flows estimated from MCS results

Parameters p_1 and p_2, as shown in Table 5-2 define the bounds between the three ranges. The choice of parameters p_1 and p_2 was made separately for each location to guarantee as much as possible a smooth transition from one range to the other. Further smoothing of reconciled flow estimates was established through application of (log-) linear interpolation between reconciled design flows of AEP values p_2^- and p_2^+, where:

p_2^- = highest AEP of interest that is smaller than p_2

p_2^+ = lowest AEP of that is higher than p_2

In other words, p_2^- and p_2^+ are the direct 'neighbours' of p_2 in the list of AEP values of interest.

Table 5-2 Locations of cluster 1 and corresponding parameters p_1 and p_2 (see text for explanation of p_1 and p_2)

Location	P_1	P_2
Linville	1\2	1\10
Gregors Creek	1\2	1\20
Fulham Vale	1\5	1\50
Woodford	1\2	1\100
Somerset Dam	1\2	1\50
Wivenhoe	1\5	1\50
Helidon	1\2	1\50
Gatton	1\5	1\50
Glenore Grove	1\5	1\50
Savages Crossing	1\5	1\50
Mount Crosby	1\5	1\50
Moggill	1\2	1\50

For locations Wivenhoe, Gatton, Mount Crosby and Moggill, the reconciled estimate for AEP=1 in 2 could not be derived directly from the plotting positions of the rated flows. This is due to the fact that the number of rated flows is relatively small compared to the number of years of the observation series for these locations. The reconciled estimate for AEP=1 in 2, was therefore derived from the estimate for AEP=1 in 5, using the FFA results as a scaling factor:

$$Q_{rec}\left(1/2\right) = \frac{Q_{FFA}\left(1/2\right)}{Q_{FFA}\left(1/5\right)} Q_{rec}\left(1/5\right) \qquad (3)$$

In which:

$Q_{rec}(p)$ = reconciled peak discharge for AEP=*p*

$Q_{FFA}(p)$ = FFA design peak discharge for AEP=*p*

5.3.3 Cluster 2

Table 5-3 shows the three locations of cluster 2. These are all locations in the Bremer/Warrill/Purga sub-catchments for which rated flows and FFA results are available. The approach for these locations is similar to the approach for the locations of cluster 1. The only difference is that for the range of 'low' AEP's the reconciled estimates are based on corrected MCS results. The correction was applied to account for the fact that rainfall depths of the IFD curves that are used as input for MCS (and DEA) are suspected to be an underestimation of the 'actual' rainfall depths (see the discussion of section 5.2). The applied correction is a constant discharge that is added to the MCS design flows for all AEP values lower than or equal to p_2 (see Table 5-3 for values of p_2). The correction is equal to:

$$Q_{corr} = 0.5 * \left[Q_{FFA}\left(p_2\right) - Q_{MCS}\left(p_2\right) \right] \qquad (4)$$

In which:

Q_{corr} = correction that is applied on derived MCS peak flows

$Q_{FFA}(p)$ = FFA design peak discharge for AEP=*p*

$Q_{MCS}(p)$ = MCS design peak discharge for AEP=*p*

Table 5-3 Locations of cluster 2 and corresponding parameters p_1 and p_2

Location	P_1	P_2
Walloon	1\2	1\20
Amberley	1\2	1\100
Loamside	1\5	1\20

With this approach, it is assumed that the FFA design flows for AEP <= p_2 are expected to be an overestimation, whereas MCS design flows for AEP <= p_2 are expected to be an underestimation. The reconciled peak flows are therefore a compromise between the two approaches. The chosen correction is somewhat arbitrary, but that is inevitable as no hard evidence is available to prefer one method over the other for these locations.

5.3.4 Cluster 3

Cluster 3 consists of a single location: Peachester. For this location, DEA and MCS appear to significantly underestimate design flows (see Figure A.2.4a and Figure A.2.4b) for most AEP values for which rated flow estimates are available. For AEP-values above 1 in100, design flow estimates are therefore based on plotting positions of rated flows and FFA results. For AEP values below 1 in 100 no rated flow estimates are available, which means no further information on potential underestimation of MCS/DEA design flow estimates is available. However, the underestimation of design flows for AEP values above 1 in 100 is an indication that MCS/DEA can be expected to underestimate design flows for AEP values below 1 in 100 as well. To account for this inconsistency, it was decided to apply a correction on MCS design flows to obtain reconciled peak flows. Similar to the cluster 2 locations, a correction is applied to the reconciled estimates for all AEP values below or equal to p_2 (p_2 = 1 in 100, see Table 5-4). The correction is equal to:

$$\mathrm{Q}_{\mathrm{corr}} = \left[Q_{FFA}\left(p_2\right) - Q_{MCS}\left(p_2\right) \right] \qquad (5)$$

In which:

Q_{corr} = correction that is applied on derived MCS peak flows

$Q_{\mathrm{FFA}}(\mathrm{p})$ = FFA design peak discharge for AEP=p

$Q_{\mathrm{MCS}}(\mathrm{p})$ = MCS design peak discharge for AEP=p

The only difference between equations (4) and (5) is the factor 0.5 in equation (4). This means the correction for location Peachester is, in relative terms, twice as high as for the cluster 2 locations. The motivation for this difference in approach is that for location Peachester the MCS and DEA design flows are (significantly) lower than estimates based on rated flows for a wide range of AEP values (1 in 5 – 1 in 100), whereas for the cluster 2 locations the significant differences are observed for a relatively small range of AEP values. Again, the choice of the applied correction is somewhat arbitrary, but that is inevitable as no hard evidence is available to prefer one method over the other for this location.

Table 5-4 Locations of cluster 3 and corresponding parameters p_1 and p_2

Location	P_1	P_2
Peachester	1 in 5	1 in 100

5.3.5 Cluster 4

Cluster 4 consists of a single location: Brisbane. For this location, rated flows and FFA results are available. However, the number of peak flows used in the FFA analysis (25) is small compared to the length of the observation series (173 years). As a result, design flow estimates for high AEP's (1 in 2 and 1 in 5 years) cannot be estimated directly from the plotting positions of the rated flows. For this reason, reconciled design flows for these high AEP's are derived from reconciled design flows at Moggill, using the following formula:

$$Q_{rec;Bri}\left(p\right)=\frac{Q_{DEA;Bris}\left(p\right)}{Q_{DEA;Mogg}\left(p\right)}Q_{rec;Mogg}\left(p\right) \qquad (6)$$

In which:

$Q_{rec;Bri}(p)$ = reconciled peak discharge for AEP=p; location Brisbane

$Q_{rec;Mogg}(p)$ = reconciled peak discharge for AEP=p; location Moggill

$Q_{DEA;Bri}(p)$ = DEA design peak discharge for AEP=p; location Brisbane

$Q_{DEA;Mogg}(p)$ = DEA design peak discharge for AEP=p; location Moggill

This means the ratio of the DEA peak discharge at Moggill versus the DEA peak discharge Brisbane is maintained in the reconciled estimates at Brisbane for AEP values of 1 in 2 and 1 in 5. After comparison with design flow estimates at location Moggill, it was decided to apply equation (6) for the whole range of AEP-values. The main motivation for this choice is to reduce the peak wave attenuation that is sometimes observed in the MCS design flow estimates on the river stretch between Moggill and Brisbane. Even though the peak wave attenuations as derived with the MCS simulations may be realistic it was decided to apply an approach that 'errs on the safe side'. This could be accomplished with the approach as described with equation (6), because DEA design flows show less peak wave attenuation than MCS design flows, due to the relatively wide hydrographs that are generated in the DEA approach.

5.3.6 Cluster 5

Table 5-5 shows the four locations of cluster 5. These are all locations for which only a little or 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. However, this may lead to inconsistencies with reconciled results of nearby locations for which reconciled design flow estimates were based on FFA results or empirical estimates from rated flows.

Table 5-5 Locations of cluster 5 and corresponding locations (see text for an explanation)

Location	Related locations
Middle Creek	Wivenhoe
Kalbar Weir	Amberley
Ipswich	Walloon, Amberley, Loamside
Centenary Bridge	Moggill

To increase consistency in peak flows of nearby locations, the locations in the first column of Table 5-5, for which no FFA results are available, are coupled with a nearby location for which FFA results are available (second column of Table 5-5). The reconciled estimates are based on the reconciled estimates of the selected nearby location, using the following formula:

$$Q_{rec;loc1}(p) = \frac{Q_{DEA;\mathrm{loc1}}(p)}{Q_{DEA;\mathrm{loc2}}(p)} Q_{rec;\mathrm{loc2}}(p) \qquad (7)$$

In which:

$Q_{\mathrm{rec;loc1}}(\mathrm{p})$ = reconciled peak discharge for AEP=*p*; location first column of Table 5-5

$Q_{\mathrm{rec;loc2}}(\mathrm{p})$ = reconciled peak discharge for AEP=*p*; location second column of Table 5-5

$Q_{\mathrm{DEA;loc1}}(\mathrm{p})$ = DEA peak discharge for AEP=*p*; location first column of Table 5-5

$Q_{\mathrm{DEA;loc2}}(\mathrm{p})$ = DEA peak discharge for AEP=*p*; location second column of Table 5-5

Note that location Ipswich is coupled to three (upstream) locations: Walloon, Amberley and Loamside. In this specific case, $Q_{\mathrm{DEA;loc2}}(\mathrm{p})$ and $Q_{\mathrm{rec;loc2}}(\mathrm{p})$ are the sum of the reconciled and DEA design peak flows of Walloon, Amberley and Loamside.

5.3.7 Cluster 6

Cluster 6 consists of a single location: Tinton. For this location, no rated flows are available and also no nearby location with rated flows is available that can be used to provide better estimates of design flows for the high range of AEP's. For this location, reconciled design flows are taken equal to the MCS design flows.

5.3.8 Tables

Appendix A.1 contains the Tables with FFA, DEA, MCS and reconciled design peak flows for 'no-dams' conditions. Table 5-6 summarizes the reconciled peak flows for a number of key locations.

Table 5-6 Reconciled 'No-dams' design flows (m^3/s) at a number of key locations

Location	AEP 1 in 10	AEP 1 in 100	AEP 1 in 1,000
Fulham Vale	2,700	6,400	9,700
Somerset Dam	2,200	4,600	6,700
Wivenhoe Dam	4,600	11,200	16,800
Glenore Grove	1,200	4,000	6,500
Savages Crossing	5,200	14,300	21,500
Ipswich	2,100	3,900	5,800
Moggill	6,400	14,600	23,000
Brisbane	6,200	13,900	22,000

5.4 Catchment area versus peak discharge

Appendix A.3 provides figures in which the catchment area is compared to design peak flows. For a range of AEP values, the following four figures are provided:

[1] Design peak flow (Q) versus catchment area (A) for FFA, DEA and MCS

[2] Reconciled design peak flow (Q) versus catchment area (A)

[3] Q/A versus A for FFA, DEA and MCS

[4] Reconciled Q/A versus A

Figures [1] and [2] should reveal an increasing trend, Figures [3] and [4] second should reveal a decreasing trend. The Figures in Appendix A.3 show that this is indeed the case. Most noteworthy in Figures [3] and [4] are the three locations in the Stanley River sub-catchment (Peachester, Woodford and Somerset dam). These locations are highlighted in the Figures, and it can be observed that these locations appear as “outliers” in the plotted relation between Q/A versus A. This can be explained by the fact that the Stanley catchment generally receives significantly higher rainfall than other sub-catchments in the Brisbane River catchment. This is also shown in Figure 5-1, where 1 in 100 AEP rainfall depths according to the ARR2013 IFD curves are provided for three durations. The Stanley locations are clear outliers, with rainfall depths that are approximately twice as high as for other sub-catchments of similar sizes.

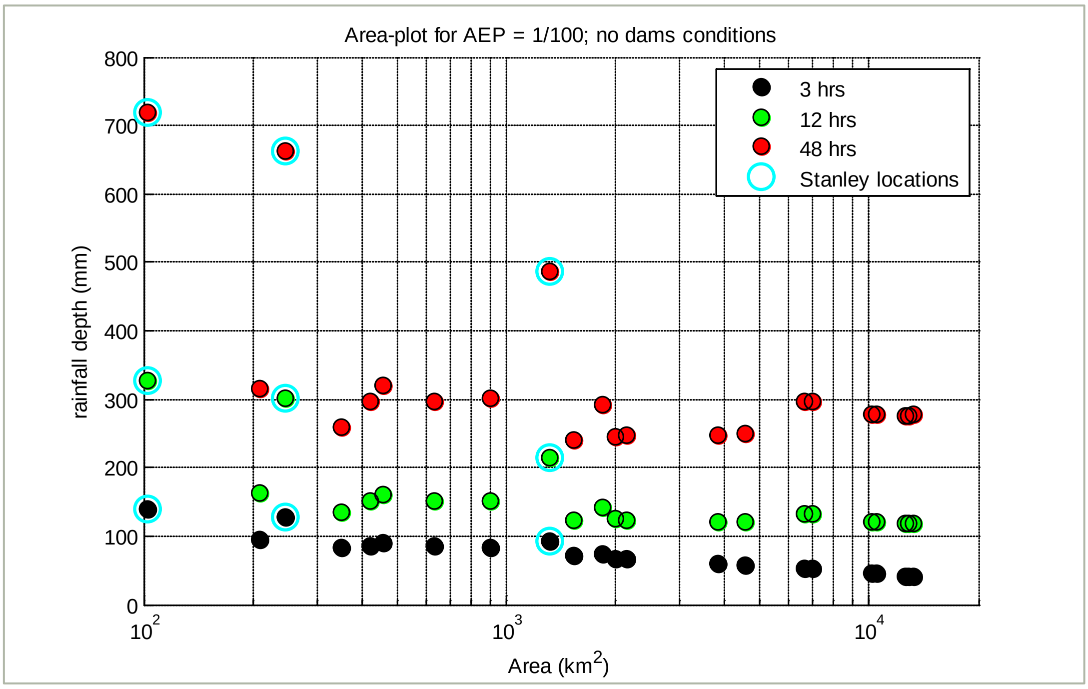


Figure 5-1 Rainfall depth with AEP=1 in 100 for three durations for the catchments of the locations of interest according to the ARR2013 IFD curves

Figure 5-2 shows the spatial distribution over the Brisbane River catchment of the 24-hour rainfall depth with AEP=1 in 100 according to the ARR2013 IFD curves, again demonstrating that rainfall depths for the Stanley sub-catchment are significantly higher than for the rest of the Brisbane River catchment.

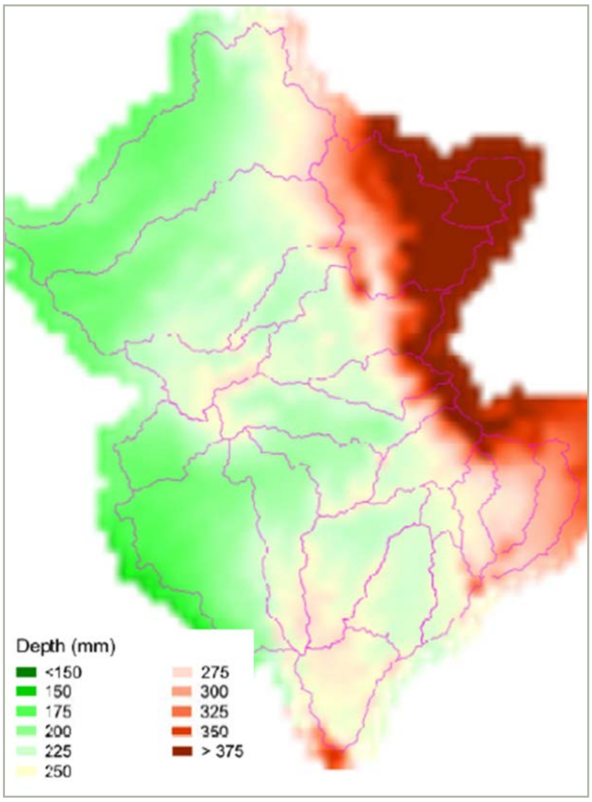


Figure 5-2 Spatial distribution over the Brisbane River catchment of the 24-hour point rainfall depth with AEP=1 in 100 according to the ARR2013 IFD curves

Another noteworthy aspect is the ratio of design peak discharge versus catchment area for locations Peachester and Woodford, as shown in Figures A.3.11 – A.3.15 (the left two of the encircled locations). According to the MCS results, this ratio is higher for location Woodford, whereas according to the DEA results this ratio is lower for location Woodford. The DEA design peak flows are more in line with what could be expected since Woodford (245 km^2) has a larger catchment than Peachester (103 km^2). In other words: the MCS design peak flows at Peachester and Woodford seem mutually inconsistent. MCS peak flows for Woodford are (significantly) higher than DEA peak flows, for location Peachester this is exactly the other way around. Comparison with FFA results shows that the main cause of the inconsistency lies in the underestimation of MCS peak flows at Peachester (see also sections 5.2 and 5.3.4).

5.5 Flow volumes

5.5.1 Comparison of DEA and MCS results

Flow volumes for different durations can be derived directly from the hydrographs that are produced with the MCS and DEA approach. Subsequently, frequency curves were derived, similar to frequency curves for peak flows. Appendix C contains ‘no-dams’ design flow volumes for 24-, 48- and 72-hour durations for each location as derived with the DEA and MCS approaches. Similar to appendix A, Appendix C contains Tables and frequency plots of design flows as well as Figures in which the design flow volumes are plotted against the size of the catchment area. The main difference with Appendix A is that the frequency plots in appendix C.2 only contain results of DEA and MCS volume frequency analyse, ie no volumes from rated flows and/or FFA. The reason is that series of rated flow volumes are not available for most locations.

The Figures of Appendix C.2 show that 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. This is in line with the observations in section 5.4, where it was found that MCS peak flows are higher than DEA peak flows 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

5.5.2 Comparison with rated flow volumes

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 uninfluenced by the dams. Rated daily flow volumes for these locations were derived from historical simulations with the Integrated Quantity Quality Model (IQQM), provided by DSITIA. 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.

Appendix C.4 contains frequency plots for locations Gregors Creek, Linville and Wallloon with IQQM-based rated flow volumes and FFA, DEA and MCS design flows. 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.

Especially for location Walloon it is worth considering applying a correction to the reconciled flow volumes that is in line with the proposed correction for design peak values as described in section 5.3.3. For the purpose of consistency this correction would have to be applied to all Bremer sub-catchment locations. The problem is that for these locations (Kalbar Weir, Amberley, Loamside, Ipswich) no rated flow volumes are available to base such a correction on. The alternative is to derive the correction from the ratio of the reconciled and MCS *peak flows*:

$$V_{rec} = \frac{Q_{rec}}{Q_{MCS}} V_{MCS} \qquad (8)$$

In which:

V_{rec} = reconciled flow volume

Q_{rec} = reconciled peak discharge

Q_{MCS} = MCS design peak discharge

V_{MCS} = MCS design flow volume

Figure 5-3 to Figure 5-5 show reconciled flow volumes for location Walloon, based on this correction factor. The correction generally improves the results (compare the reconciled results with MCS results), if we can assume the rated volumes and FFA estimates to be correct. On the other hand, for AEP = 1 in 2 the correction results in an increase in differences reconciled design flows on one hand and FFA results and rated flows on the other hand. This shows there is no guarantee the correction based on peak flows will improve reconciled estimates on flow volumes. The correction is therefore omitted.

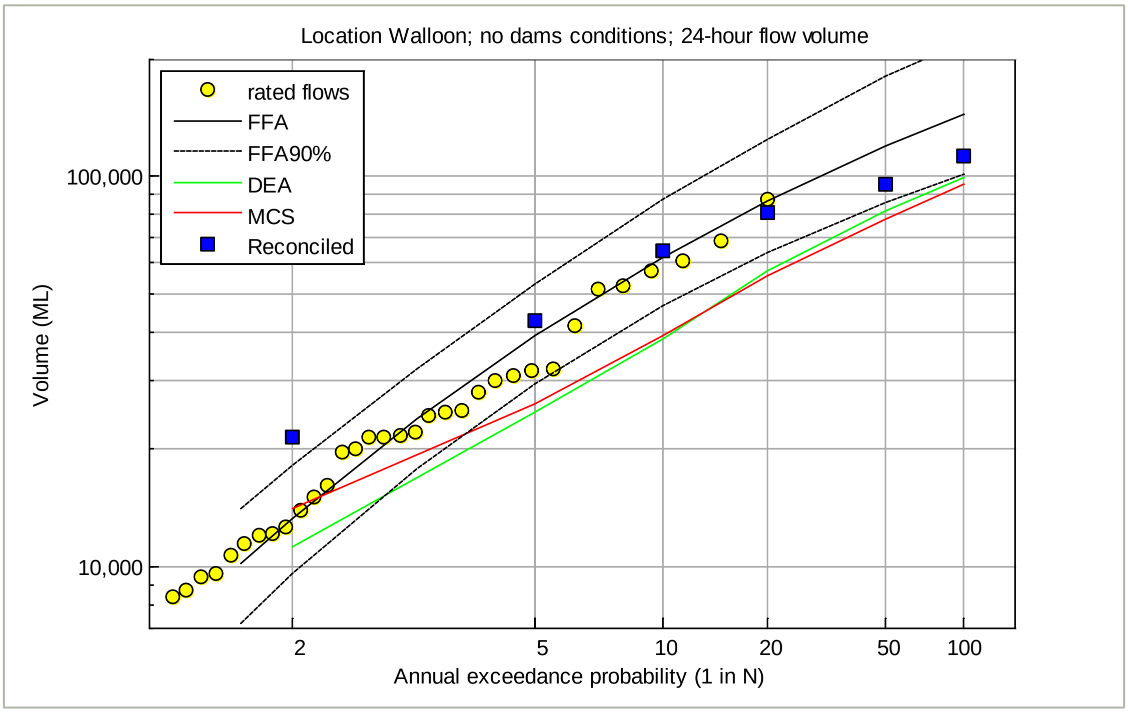


Figure 5-3 Derived frequency curves with FFA, DEA and MCS for 24-hour flow volumes at location Walloon, in combination with reconciled flow volumes

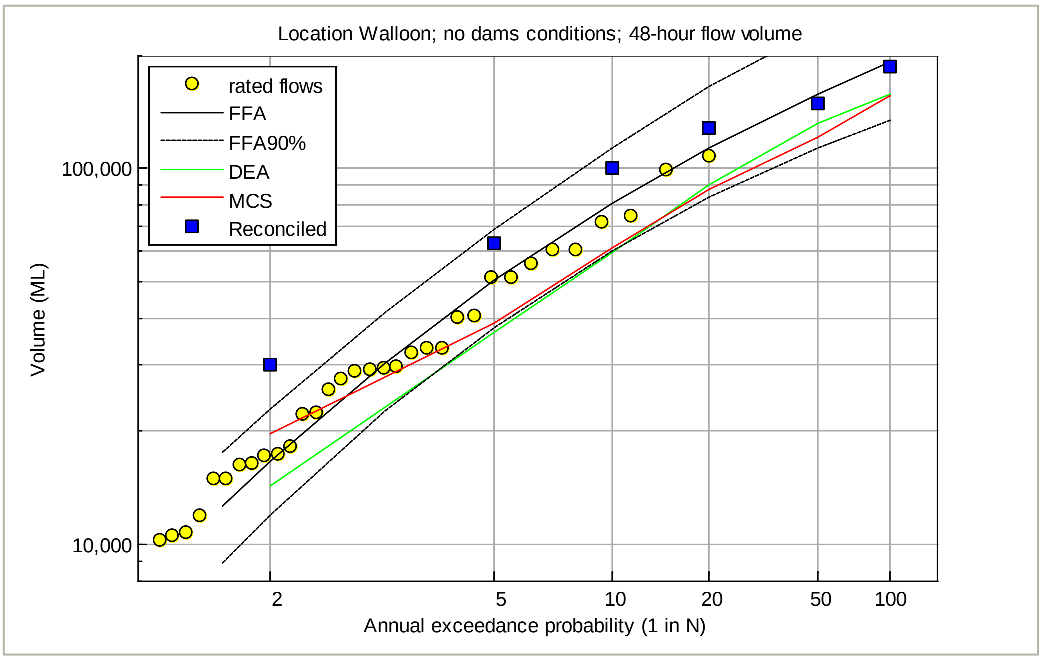


Figure 5-4 Derived frequency curves with FFA, DEA and MCS for 48-hour flow volumes at location Walloon, in combination with reconciled flow volumes

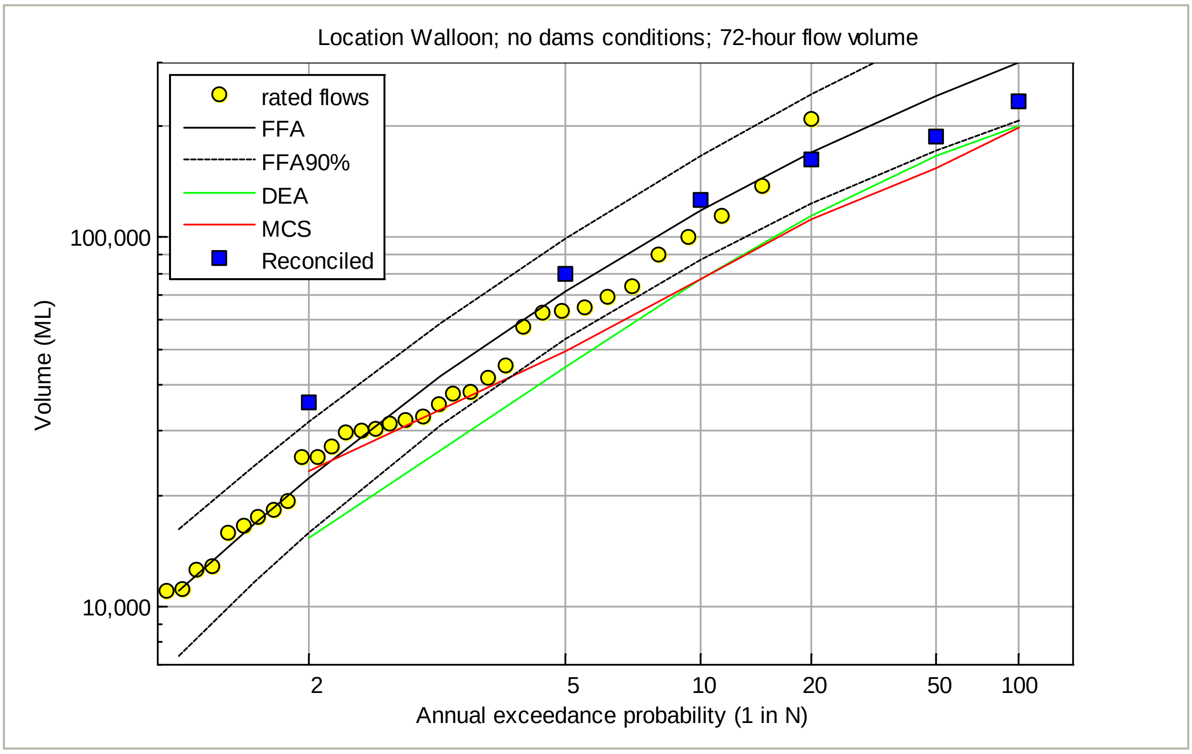


Figure 5-5 Derived frequency curves with FFA, DEA and MCS for 72-hour flow volumes at location Walloon, in combination with reconciled flow volumes

6 Reconciled ‘with-dams’ estimates

6.1 Introduction

The approach to the reconciliation process was described in section 2. The current chapter describes the results and choices in more detail. MCS and DEA simulations for ‘with-dams’ conditions were carried out for eight locations:

1. Somerset Dam
2. Wivenhoe Dam
3. Savages Crossing
4. Mount Crosby
5. Ipswich
6. Moggill
7. Centenary Bridge
8. Brisbane

MCS results for Somerset Dam were abstracted from the Wivenhoe Dam simulation run. The reason to use these results 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 as no coincident flooding has been considered in the Upper Brisbane River catchment. This is a difference with the ‘no-dams’ conditions, for which frequency curves for Somerset Dam were based on an individual run for the Somerset Dam catchment as in the no-dams conditions . This means the simulated synthetic events for ‘no-dams’ and ‘with-dams’ conditions are different for location Somerset. For the other seven locations, the simulated synthetic events for ‘no-dams’ and ‘with-dams’ conditions are the same.

6.2 Reconciled peak flows

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 locations were divided into four ‘clusters’; for each cluster a different approach was used to derive reconciled design peak flows. The subdivision in clusters is based on two criteria:

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

The following sections describe the reconciliation process for each of the four clusters.

6.2.1 Cluster 7

Cluster 7 Consists of three locations: Somerset Dam, Mount Crosby and Moggill. The reconciled design flows for these locations are based on a combination of:

- Empirical estimates from rated flows for (very) frequent events (AEP≥ p_1)
- Monte Carlo Simulations results for frequent events to extreme events (AEP< p_1)

The values of parameter p_1 are shown in Table 6-1.

Table 6-1 Locations of cluster 7 and corresponding parameter p_1

Location	P_1
Somerset Dam	1 in 10
Mount Crosby	1 in 5
Moggill	1 in 5

6.2.2 Cluster 8

Cluster 8 consists of three locations: Savages Crossing, Centenary Bridge and Brisbane. The reconciled peak flows of these locations are partly based on reconciled results of the locations in the second column of Table 6-2 to obtain better spatial consistency in ‘with-dams’ peak flows. The following formula was used to reach this objective:

$$Q_{rec;loc1}(p) = \frac{Q_{DEA;\mathrm{loc}1}(p)}{Q_{DEA;\mathrm{loc}2}(p)} Q_{rec;\mathrm{loc}2}(p) \qquad (9)$$

In which:

$Q_{rec;loc1}(p)$ = reconciled peak discharge for AEP=p; location first column of Table 6-2

$Q_{rec;loc2}(p)$ = reconciled peak discharge for AEP=p; location second column of Table 6-2

$Q_{DEA;loc1}(p)$ = DEA peak discharge for AEP=p; location first column of Table 6-2

$Q_{DEA;loc2}(p)$ = DEA peak discharge for AEP=p; location second column of Table 6-2

Table 6-2 Locations of cluster 8 and corresponding locations (see text for an explanation)

Location	Related locations
Savages Crossing	Mount Crosby
Centenary Bridge	Moggill
Brisbane	Moggill

The main motivation to apply this approach for the three locations in the first column of Table 6-2 is to reduce the peak wave attenuation that is sometimes observed in the MCS design flow estimates on the river stretch between Savages Crossings and Mount Crosby and on the stretch between Moggill and Brisbane. Even though the peak wave attenuation as derived with the MCS simulations may be realistic it was decided to apply an approach that ‘errs on the safe side’.

6.2.3 Cluster 9

Cluster 9 consists of one location: Wivenhoe Dam. For this location the reconciled results are entirely based on MCS simulation results, because the available set of observed ‘with-dams’ peak flows was considered to be too limited to be used as reconciled estimate for frequent events.

6.2.4 Cluster 10

Cluster 10 consists of one location: Ipswich. For this location, a correction was applied on reconciled ‘*no-dams*’ peak flows to account for the fact that BoM rainfall statistics in the Bremer catchment are suspected to be an underestimation of the ‘actual’ rainfall depths (see the discussion of section 5.2). To improve consistency between ‘no-dams’ and ‘with-dams’ reconciled peak flows for Ipswich, the following approach was used to derive reconciled ‘with-dams’ estimates:

$$Q_{rec;\mathrm{WD}}\left(p\right)=\frac{Q_{MCS;\mathrm{WD}}\left(p\right)}{Q_{MCS;\mathrm{ND}}\left(p\right)}Q_{rec;\mathrm{ND}}\left(p\right) \qquad (10)$$

$Q_{\mathrm{rec;WD}}(\mathrm{p})$ = reconciled ‘with-dams’ peak discharge at Ipswich for AEP=*p*

$Q_{\mathrm{rec;ND}}(\mathrm{p})$ = reconciled ‘no-dams’ peak discharge at Ipswich for AEP=*p*

$Q_{\mathrm{MCS;WD}}(\mathrm{p})$ = ‘with-dams’ MCS peak discharge at Ipswich for AEP=*p*

$Q_{\mathrm{MCS;ND}}(\mathrm{p})$ = ‘no-dams’ MCS peak discharge at Ipswich for AEP=*p*

6.2.5 Results

Appendix B.1 contains Tables with DEA, MCS and reconciled design peak flows for ‘with-dams’ conditions. Appendix B.2 contains the corresponding Figures. The following information is provided in these Figures:

- Plotting positions of rated peak flows (if available)
- DEA results
- MCS results
- Proposed reconciled design peak flows

The following is observed from the figures:

1. For Brisbane River locations downstream of Wivenhoe Dam, differences in MCS and DEA design flows for AEP<1 in 10 are small
2. 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
3. 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

4. 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

6.3 Catchment area versus peak discharge

Appendix B.3 contains Figures in which catchment area is compared with peak discharge. For a range of AEP values, the design peak flows according to DEA and MCS are plotted against catchment area. The most noticeable results identify that for AEP=1 in 10, the MCS design flow at Wivenhoe Dam (catchment size 6980 km^2) is lower than the design flows at locations Ipswich (catchment size 1850 km^2) and Somerset Dam (catchment size 1324 km^2). This clearly shows the reducing effect of Wivenhoe Dam on these 'frequent' flood events. For AEP $\leq$ 1 in 100, the design peak flow at Wivenhoe Dam is significantly higher than design peak flows at Ipswich and Somerset Dam.

6.4 No-dams versus with-dams flows

Appendix E contains Figures in which MCS 'with-dams' results are compared to MCS 'no-dams' results. Similar figures are provided for the DEA results. The following Figures are provided for each location:

1. Frequency curves for 'no dam' and 'with dam' situations
2. Scatter plots of simulated 'no dam' peak flows versus 'with dam' peak flows
3. Frequency plots for 'no dam' flow volumes and 'with dam' flow volumes

No scatter plot is provided for location Somerset Dam because the simulated synthetic events for 'no-dams' and 'with-dams' conditions are different for this location. For the other seven locations, the simulated synthetic events for 'no-dams' and 'with-dams' conditions are the same, which means a meaningful scatter plot can be produced for these locations.

It can be observed from the Figures in Appendix E that:

- 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 flows
- For location Somerset Dam, 'With-dams' peak flows of individually simulated events are often higher than corresponding 'no-dams' peak flows, 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 (see section 6.1). This shows the validity of this approach is doubtful. It is therefore recommended not to adopt the derived 'with-dams' results for location Somerset Dam
- The reducing effects of the dams on peak flows 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

Table 6-3 compares design peak flows 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 6-3 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%

6.5 Flow volumes

Appendix D contains 'with-dams' design flow volumes for 24-, 48- and 72-hour durations for each location as derived with the DEA and MCS approaches. Similar to appendices A-C, Appendix D contains Tables and frequency plots of design flows as well as Figures in which the design flow volumes are plotted against the size of the catchment area.

The Figures of Appendix D.2 show that 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. 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, as was already noted in section 6.2.5 (bullet items 2 and 3).

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.

7 Additional Monte Carlo simulation results

7.1 Introduction

The current section describes a selection of additional outputs provided by the MCS framework. Some of these outputs, ie ocean water levels and flow hydrographs, are used directly as input for the hydraulics phase of the BRCFS. Others are useful for interpreting the derived Monte Carlo design flows and contribute to a further understanding of the system.

7.2 Rainfall AEP versus peak discharge

Appendix F contains figures in which the peak discharge of each simulated event is plotted against the annual exceedance probability of the rainfall depth. For ‘no-dams’ conditions, these Figures are provided for 22 locations, for ‘with-dams’ conditions the Figures are shown for eight locations: Somerset Dam, Wivenhoe Dam, Ipswich and the five Lower Brisbane River locations. Different colours are used to distinguish for different burst durations. Furthermore, three lines are added that represent the 10%, 50% and 90% quantiles. These lines can be interpreted as follows: for a given rainfall AEP, the p-percentile is the peak discharge which has a p% probability of being exceeded, given the occurrence of a rainfall event with a rainfall depth that corresponds to the AEP shown on the horizontal axis. For a given combination of rainfall burst duration and AEP, the variability of peak flows in the Figures are caused by the combined influence of the variability of initial losses, spatio-temporal rainfall patterns and, for the ‘with-dams’ case, initial reservoir volumes.

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 flows 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 flows at Wivenhoe compared to the longer duration bursts and cause a negative skew in the distribution of peak flows 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 flows are often ‘clustered’. For these events, variations in losses are small (see section 4) which means the variation in peak flows 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 flows 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 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 ate the start of an events differ per simulated events, which increases the variability of resulting peak flows 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 flows 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 flows for a single AEP

7.3 Discharge hydrographs

One of the relevant inputs for the hydraulics phase of the BRCFS is a set of discharge hydrographs at the locations of interest. As an example, Figure 7-1 shows a set of simulated hydrographs at location Brisbane for a range of rainfall burst durations for 'no-dams' conditions. The AEP of the rainfall depth of the simulated bursts that resulted in these hydrographs is approximately equal to 1 in 100. Figure 7-2 shows a second batch of discharge hydrographs at Brisbane for the exact same rainfall durations and rainfall AEP. Differences between the two Figures are caused by differences in the selected storm patterns and initial losses. The comparison of the two Figures, and the comparison of the hydrographs for different rainfall durations, gives an impression of the variation of hydrographs produced by the MCS framework.

Figure 7-3 and Figure 7-4 show similar hydrographs for 'with-dams' conditions. The comparison of Figure 7-1 and Figure 7-2 on the one hand and Figure 7-3 and Figure 7-4 on the other hand gives insight in the differences in hydrographs between 'no-dams' and 'with-dams' conditions. As expected, the 'with-dams' hydrographs have lower peaks and a more extended drawdown phase.

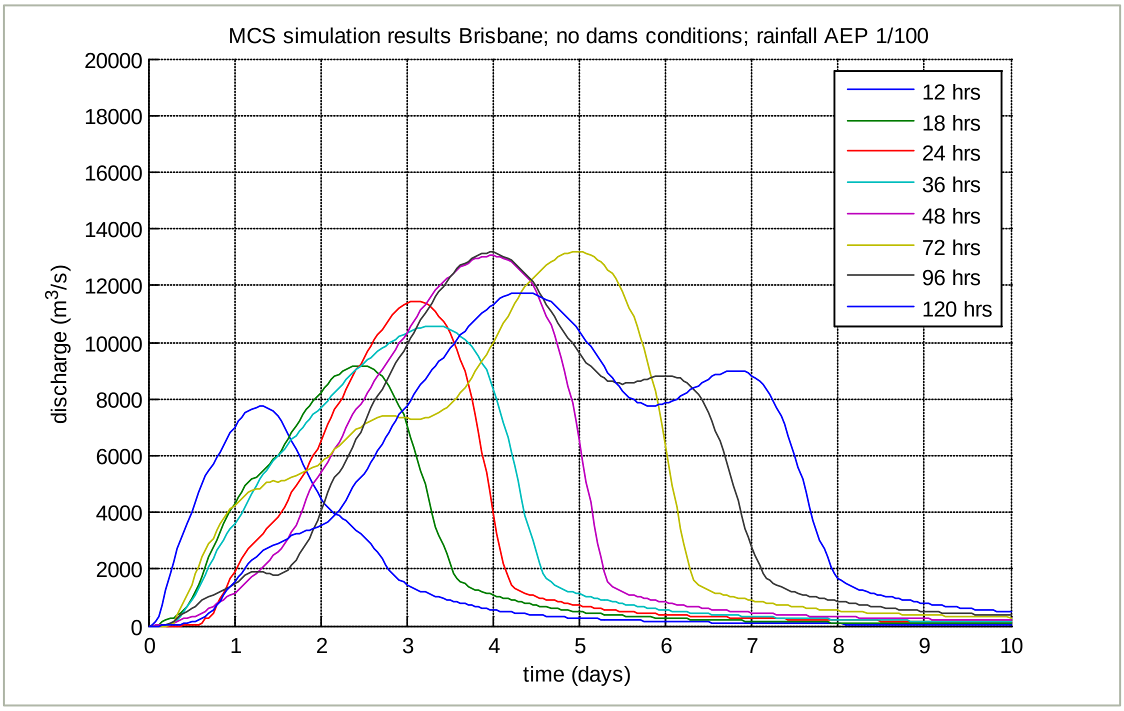


Figure 7-1 'No-dams' hydrographs at location Brisbane for a range of rainfall burst durations, rainfall AEP ≈ 1 in 100; first set of samples

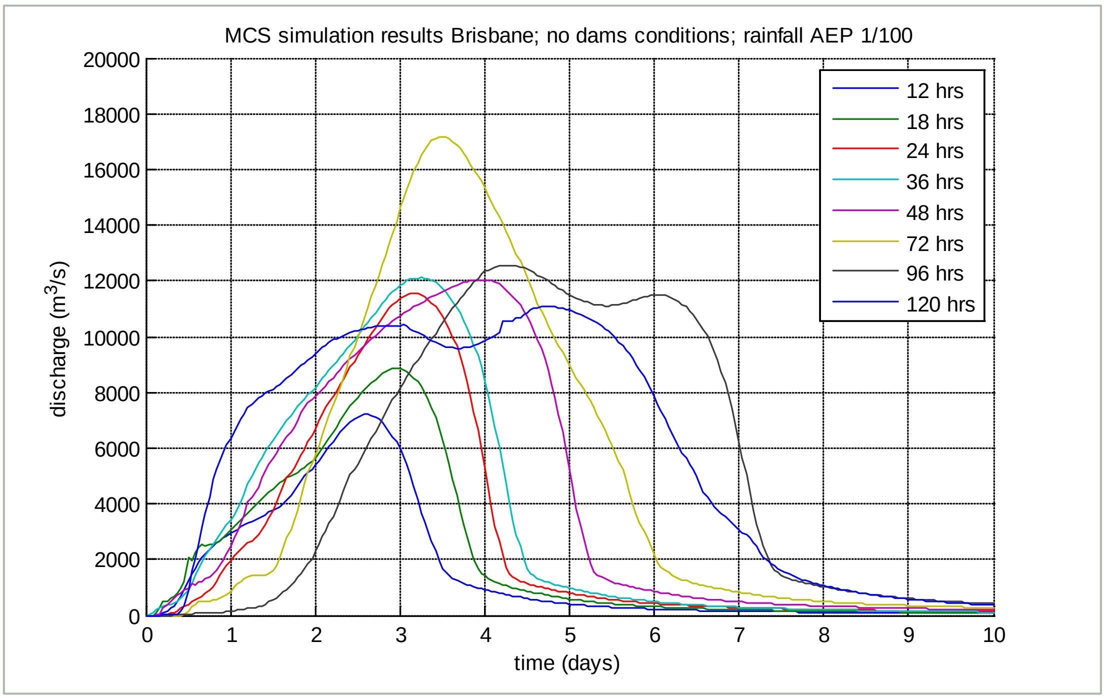


Figure 7-2 'No-dams' hydrographs at location Brisbane for a range of rainfall burst durations, rainfall AEP ≈ 1 in 100; second set of samples

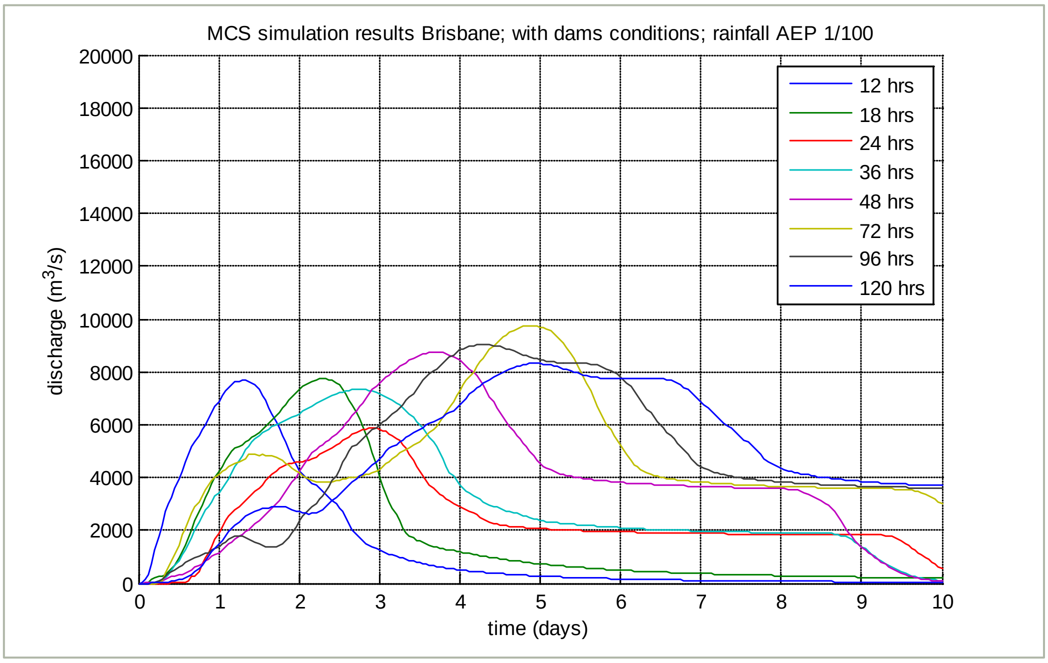


Figure 7-3 'With-dams' hydrographs at location Brisbane for a range of rainfall burst durations, rainfall AEP ≈ 1 in 100; first set of samples

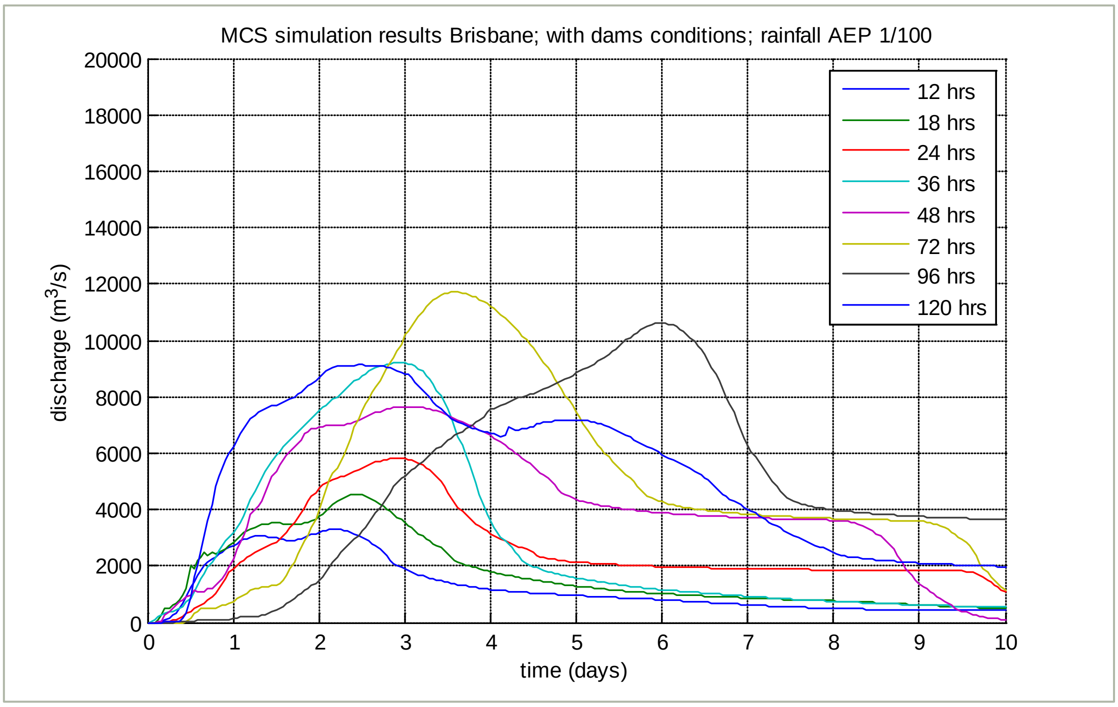


Figure 7-4 'With-dams' hydrographs at location Brisbane for a range of rainfall burst durations, rainfall AEP ≈ 1 in 100; second set of samples

7.4 Timing of peak flows and peak ocean water levels

If the peak of the ocean water level occurs two days before the start of the rainfall event, the surge will have no effect on flood levels in the Lower Brisbane and Bremer Rivers, whereas if it occurs near the end of the rainfall event the effect may be substantial. Figure 7-5 shows a histogram of differences in timing between peak flows at Brisbane City and peak ocean water levels at the Brisbane River outlet, for simulated 72 hour bursts, for 'no-dams' conditions. Figure 7-6 shows a similar plot for 'with-dams' conditions. Positive values refer to peak flows occurring later than peak ocean water levels. It shows that the simulated peak flow at Brisbane City generally occurs later than the simulated peak ocean water level, which is in accordance with what is expected. The histograms for 'with-dams' and 'no-dams' conditions are similar. The main difference is that the histograms for 'with-dams' conditions has a wider spread of simulated time differences.

For the with-dams conditions, the peak of the ocean water level may occur up to two days before the peak of the river discharge at Brisbane, although such events are clearly exceptions to the rule. Figure 7-7 shows resulting 'no-dams' and 'with-dams' hydrographs at location Brisbane for a simulated synthetic event for which the peak of the 'with-dams' hydrograph occurs several days before the peak of the ocean water level and also more than two days before the peak of the 'no-dams' hydrograph. The 'no-dams' hydrograph has a peak discharge of approximately 1800 m^3/s, which occurs around four days after the beginning of the simulation of the event. In the 'with-dams' case this peak is absent, which means the dams retained the majority of the abundant flow. Because this peak is absent, the secondary peak that occurs after one day is the highest peak in the 'with-dams'. This clearly demonstrates how the dams can influence the hydrographs in such a way that peaks occur earlier (or later) in the event. This is the reason why peaks in the 'with-dams' case can occur several days before the peak of the ocean water level, whereas this does not occur in the 'no-dams' case.

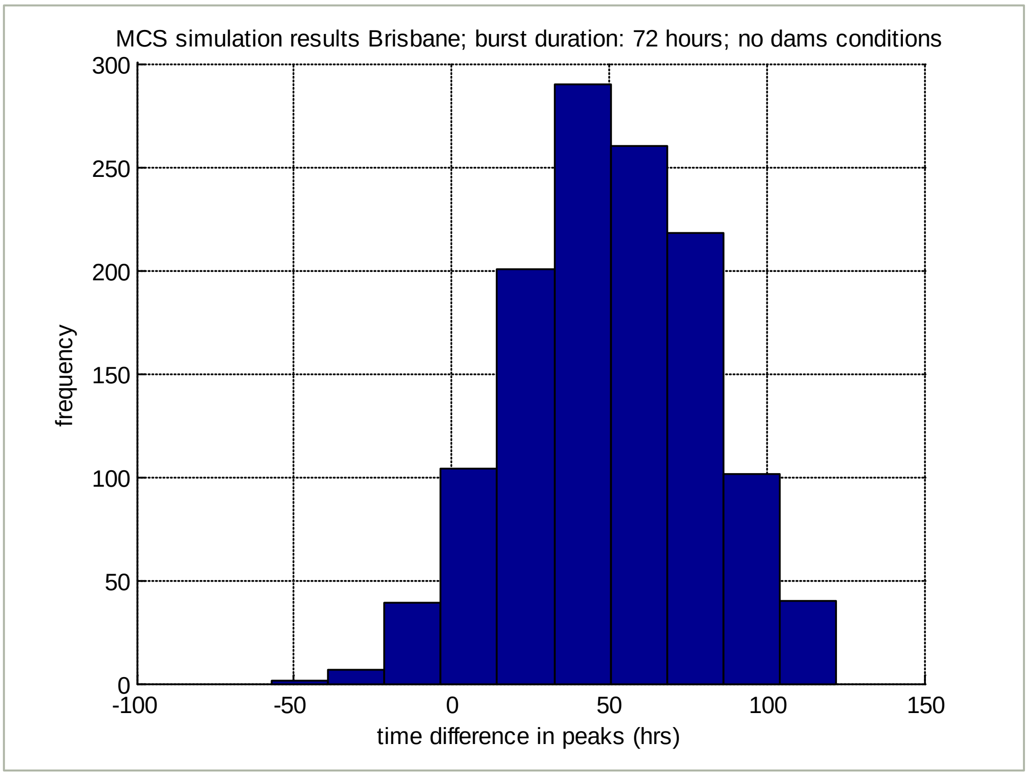


Figure 7-5 Histogram of difference in timing between peak flows at Brisbane City and peak ocean water levels at the Brisbane River outlet, for simulated 72 hour bursts. Positive values refer to peak flows occurring later than peak ocean water levels; 'no-dams' results

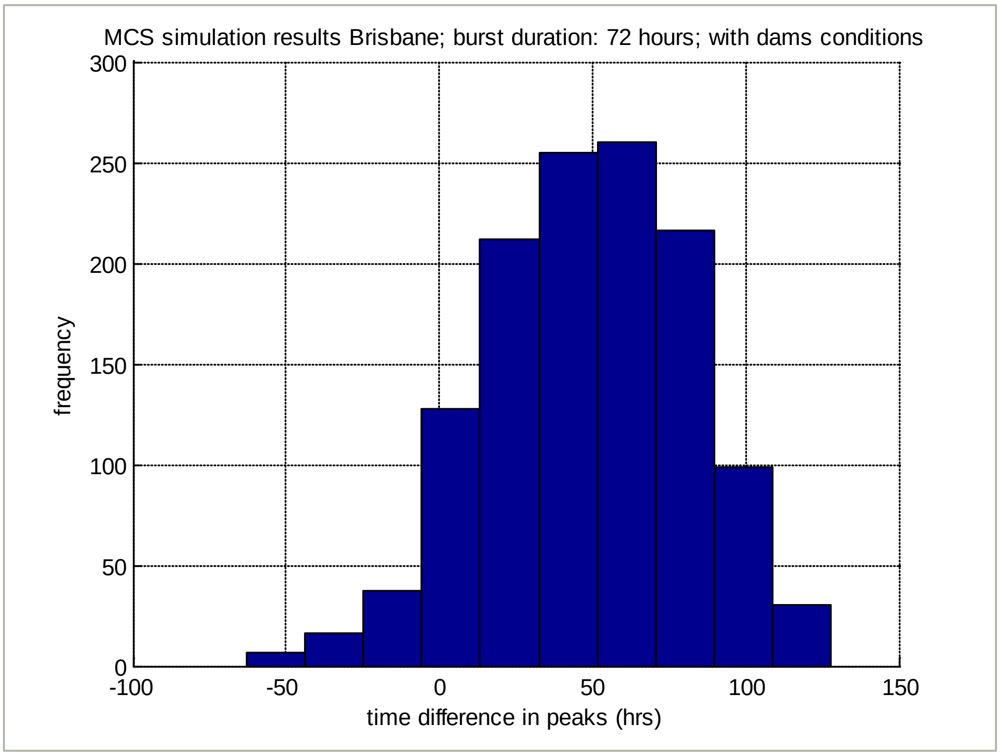


Figure 7-6 Histogram of difference in timing between peak flows at Brisbane City and peak ocean water levels at the Brisbane River outlet, for simulated 72 hour bursts. Positive values refer to peak flows occurring later than peak ocean water levels; 'with-dams' results

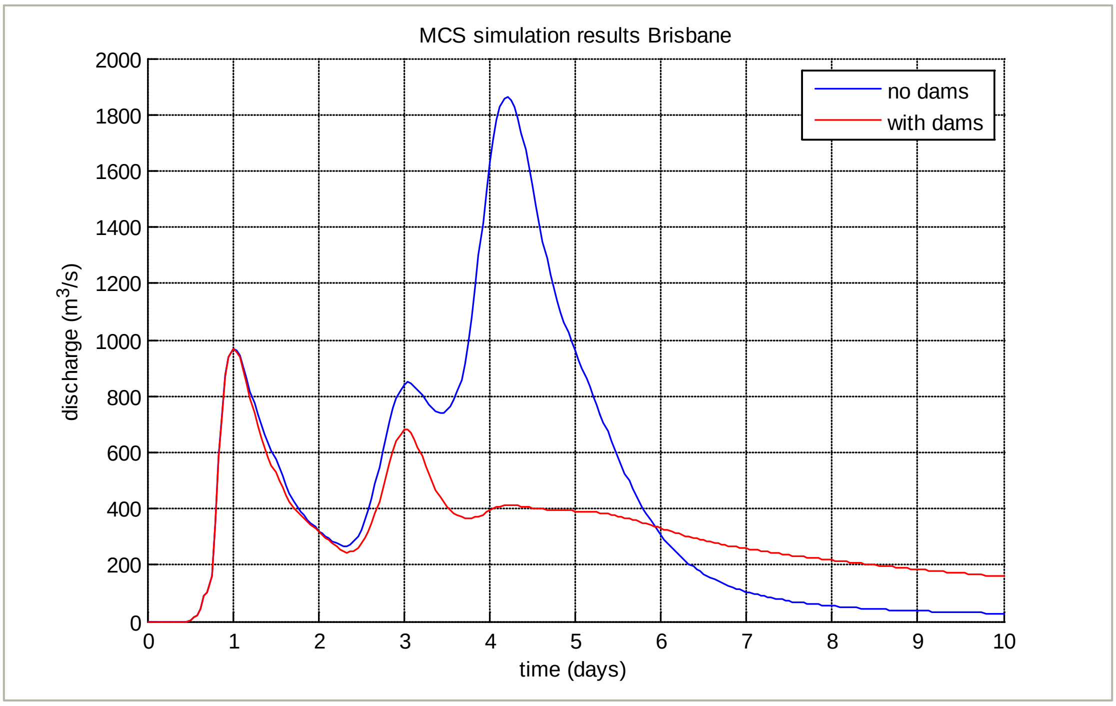


Figure 7-7 Simulated hydrographs for 'no-dams' and 'with-dams' conditions for the same synthetic event; Example case in which the 'with-dams' peak discharge occurs several days before the 'no-dams' peak discharge

7.5 Critical durations

Appendix G shows Figures of frequency curves for various burst durations. For 'no-dams' conditions, the following is 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 therefore does not end up in 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 is 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.

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 7-8 – Figure 7-11 show resulting frequency curves for peak flows 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.

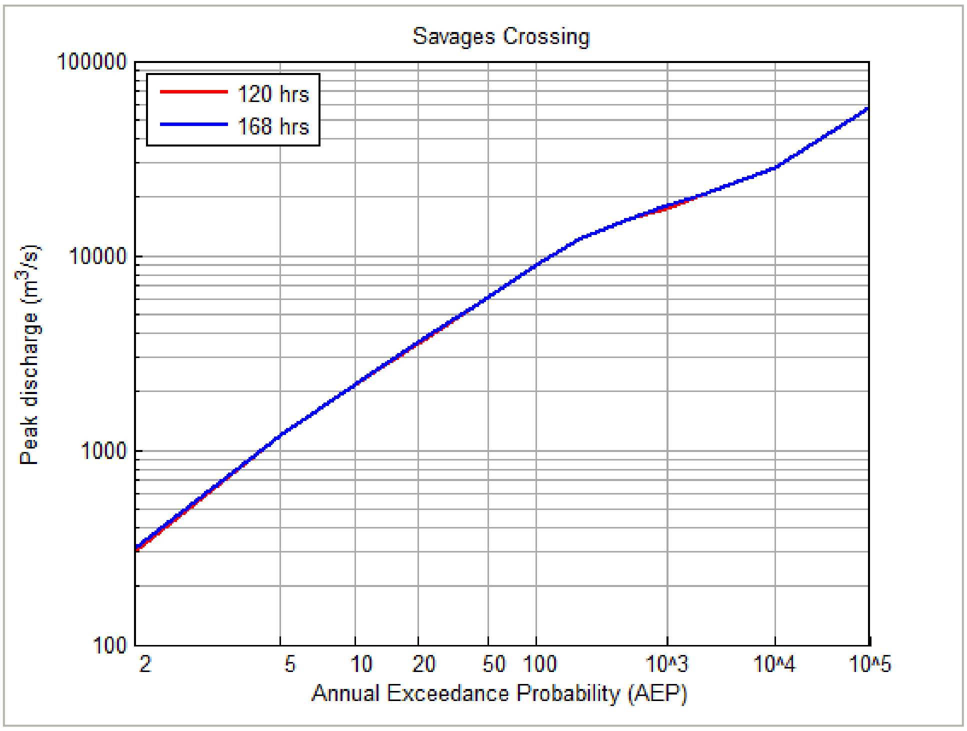


Figure 7-8 Frequency curves of peak flows at location Savages Crossing; comparison of a TPT run in which burst durations up to 120 hours were considered with a TPT run in which burst durations up to 168 hours were considered

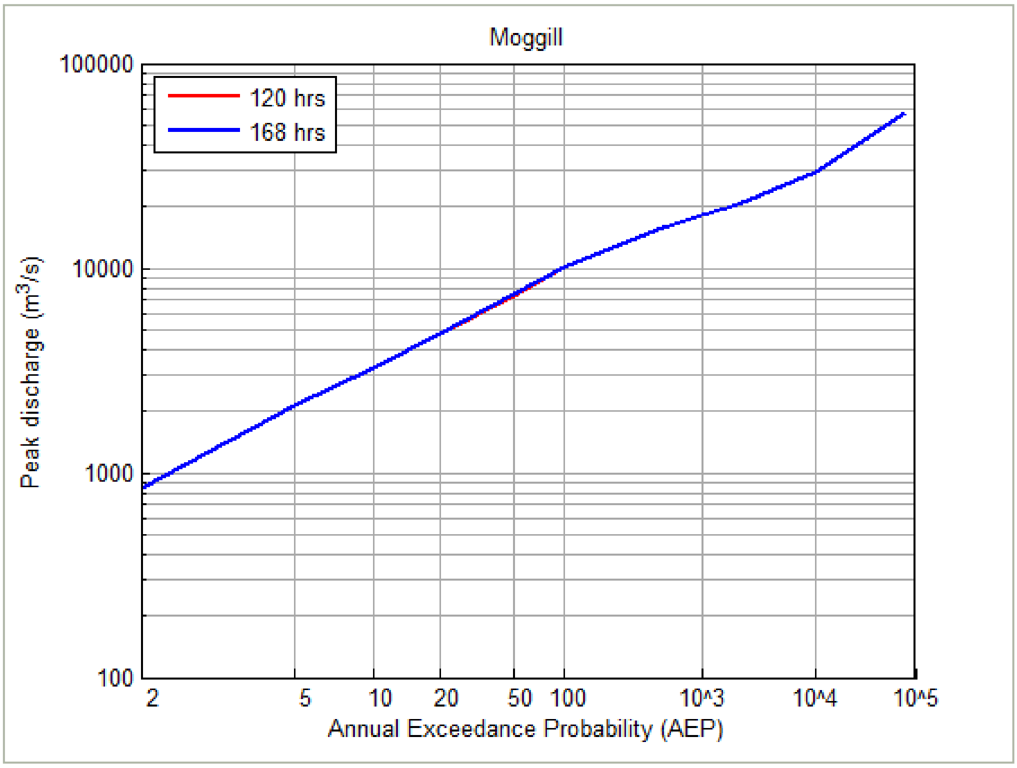


Figure 7-9 Frequency curves of peak flows at location Moggill; comparison of a TPT run in which burst durations up to 120 hours were considered with a TPT run in which burst durations up to 168 hours were considered

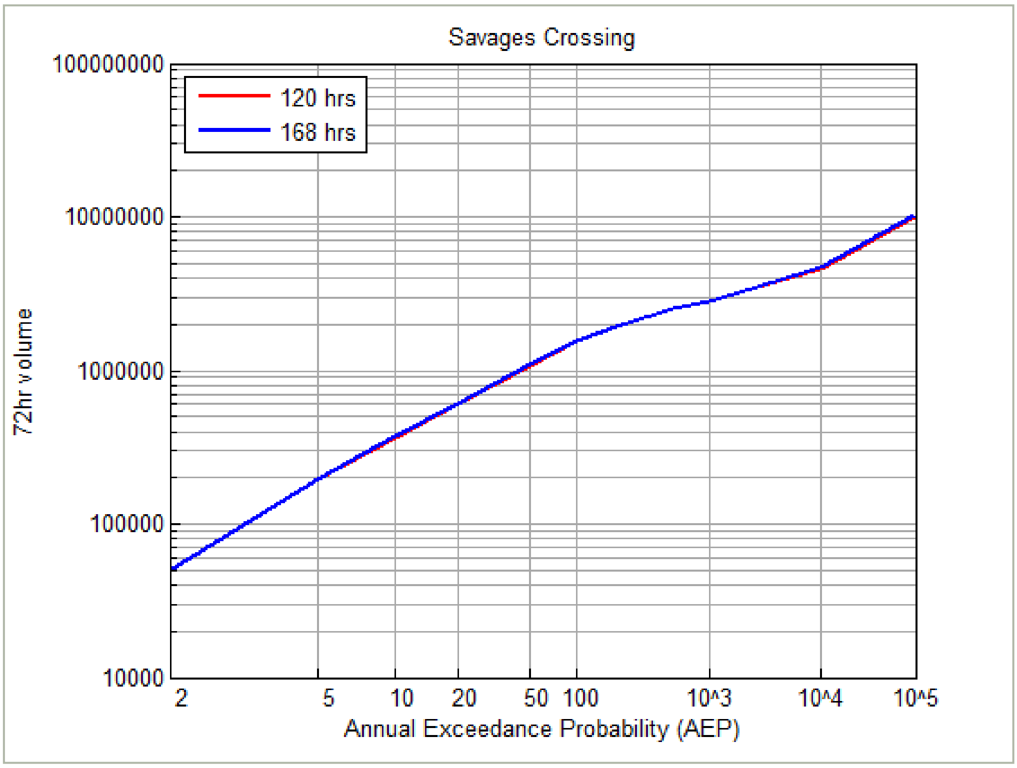


Figure 7-10 Frequency curves of 72 hour flow volumes at location Savages Crossing; comparison of a TPT run in which burst durations up to 120 hours were considered with a TPT run in which burst durations up to 168 hours were considered

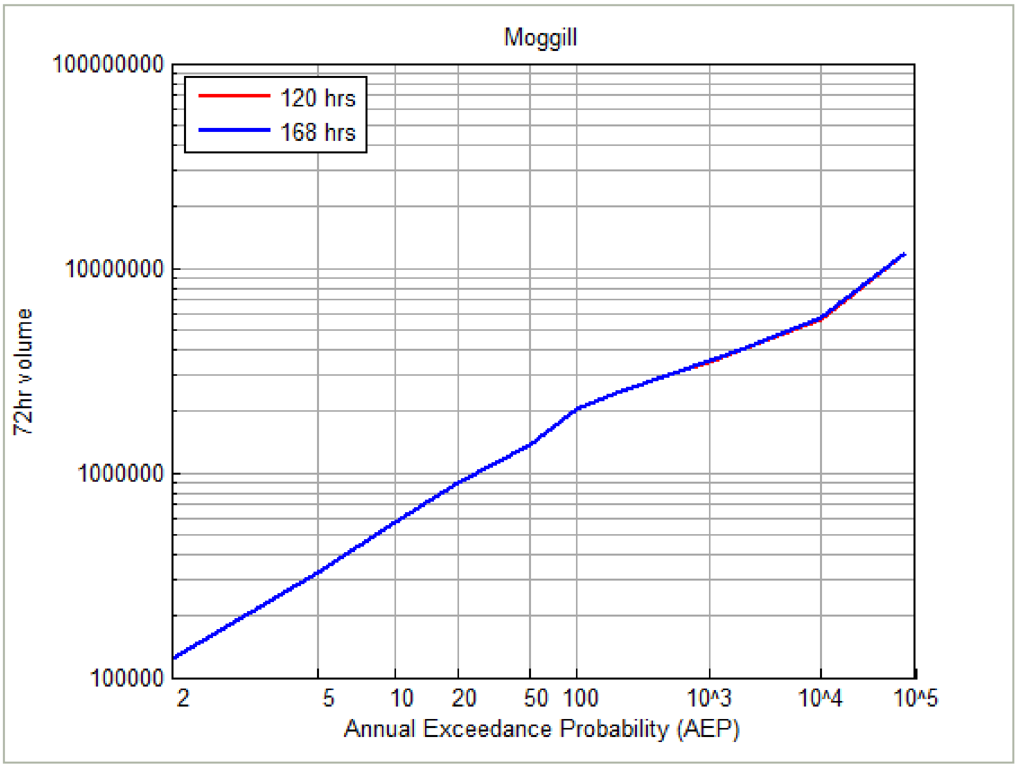


Figure 7-11 Frequency curves of 72 hour flow volumes at location Moggill; comparison of a TPT run in which burst durations up to 120 hours were considered with a TPT run in which burst durations up to 168 hours were considered

8 Conclusions

8.1 Adopted methodology

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' 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) series of rated flows was available for the other 5 locations. 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

For locations for which no (reliable) data on peak flows 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 rated flows and FFA results of nearby stations. The reconciled estimates were successfully validated for spatial consistency.

For 'with-dams' conditions, 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.

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 'with-dams' estimates were also successfully validated for spatial consistency.

8.2 Results

8.2.1 No-dams

- 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 flows 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
- Design peak flows 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. 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

8.2.2 With-dams

- 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
- .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 flows
- For location Somerset Dam, 'With-dams' peak flows of individually simulated events are often higher than corresponding 'no-dams' peak flows, 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 (see section 6.1). This shows the validity of this approach is doubtful. It is therefore recommended not to adopt the derived 'with-dams' results for location Somerset Dam
- The reducing effects of the dams on peak flows 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
- 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

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10 Glossary

10.1 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)

10.2 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
‘No-dams’ design peak flows

A.1 Tables

This appendix contains tables with peak flows for a range of AEP values for all locations. 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.

Table A1 Peak flows (m^3/s) versus AEP; FFA results

Location	AEP (1 in N)											
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000
Linville	170	710	1,300	2,000	3,100	4,100	5,000	6,300	7,200	8,200	10,200	12,800
Gregors Creek	340	1,400	2,500	3,900	6,100	7,800	9,600	12,100	13,900	15,600	19,500	24,300
Fulham Vale	410	1,500	2,700	4,000	6,000	7,600	9,100	11,200	12,700	14,200	17,300	20,900
Peachester	140	300	420	540	680	790	890	1,000	1,100	1,200	1,300	1,500
Woodford	200	510	750	1,000	1,300	1,600	1,800	2,100	2,300	2,500	2,900	3,400
Somerset Dam	540	1,400	2,200	3,000	4,000	4,800	5,500	6,500	7,100	7,700	8,900	10,300
Tinton												
Middle Ck												
Wivenhoe	740	2,700	4,600	6,800	10,000	12,600	15,100	18,500	21,000	23,300	28,300	34,300
Helidon	66	230	400	590	850	1,100	1,300	1,500	1,700	1,900	2,300	2,800
Gatton	93	430	830	1,300	2,100	2,700	3,400	4,300	5,000	5,600	7,000	8,700
Glenore Grove	120	620	1,200	2,000	3,300	4,400	5,600	7,200	8,500	9,700	12,400	15,900
Savages Crossing	700	2,900	5,200	8,100	12,300	15,700	19,200	23,800	27,300	30,600	37,700	46,200
Mount Crosby	850	3,100	5,400	8,100	11,900	14,900	18,000	21,900	24,800	27,500	33,300	40,000
Walloon	230	680	1,100	1,500	2,100	2,500	2,900	3,400	3,800	4,100	4,800	5,600
Kalbar Weir												
Amberley	210	630	1,000	1,400	2,000	2,400	2,800	3,400	3,800	4,100	4,900	5,800
Loamside	66	190	310	430	590	720	840	990	1,100	1,200	1,400	1,700

Location	AEP (1 in N)											
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000
Ipswich												
Moggill	1,100	3,800	6,400	9,300	13,300	16,400	19,400	23,200	26,000	28,500	33,700	39,600
Centenary Bridge												
Brisbane	1,300	4,100	6,700	9,500	13,400	16,300	19,100	22,600	25,100	27,400	32,000	37,200

Table A2 Peak flows (m^3/s) versus AEP; DEA results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Linville	270	840	1,400	2,100	3,000	3,600	4,100	4,800	5,300	5,800	7,000	13,000	22,700
Gregors Creek	490	1,500	2,500	3,700	5,300	6,400	7,300	8,500	9,400	10,300	12,900	25,400	36,300
Fulham Vale	460	1,500	2,500	3,600	5,200	6,400	7,200	8,400	9,300	10,200	13,000	26,300	34,500
Peachester	110	220	310	400	530	620	730	870	980	1,100	1,400	1,800	2,400
Woodford	210	430	610	800	1,100	1,300	1,500	1,800	2,000	2,200	2,800	3,700	5,000
Somerset Dam	670	1,500	2,200	2,900	3,900	4,600	5,300	6,200	6,800	7,500	8,800	12,900	19,900
Tinton	140	310	480	650	880	1,100	1,200	1,400	1,600	1,800	2,300	3,500	6,700
Middle Ck	960	2,700	4,400	6,300	8,800	10,500	12,000	13,900	15,600	17,500	23,300	48,400	57,400
Wivenhoe	950	2,600	4,400	6,400	9,100	10,800	12,400	14,300	15,900	17,600	23,100		48,100
Helidon	51	220	360	510	730	910	1,100	1,300	1,400	1,600	2,100	3,300	7,500
Gatton	120	790	1,300	1,900	2,700	3,300	3,800	4,500	5,000	5,600	6,100	12,000	23,700
Glenore Grove	160	1,000	1,800	2,600	3,600	4,400	5,100	6,000	6,700	7,400	8,500	16,900	29,900
Savages Crossing	860	3,200	5,500	8,000	11,700	13,900	15,900	18,200	20,100	22,000	29,800		64,200
Mount Crosby	840	3,200	5,400	7,900	11,500	13,700	15,800	18,300	20,200	22,300	29,500		63,600

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Walloon	210	460	690	930	1,300	1,600	1,900	2,200	2,400	2,700	3,500	5,300	9,600
Kalbar Weir	180	390	570	770	1,100	1,300	1,500	1,800	2,000	2,200	2,800	4,200	7,800
Amberley	210	420	610	890	1,400	1,700	2,000	2,300	2,600	2,900	3,700	5,800	10,000
Loamside	65	150	220	310	440	540	620	730	820	910	1,100	1,600	3,200
Ipswich	390	930	1,300	1,800	2,700	3,300	3,800	4,500	5,000	5,600	7,500	12,500	18,900
Moggill	900	3,500	5,800	8,500	12,200	14,600	16,700	19,300	21,800	24,200	31,500		64,100
Centenary Bridge	890	3,400	5,600	8,200	11,600	14,000	16,200	18,800	21,100	23,600	31,200		64,400
Brisbane	890	3,400	5,600	8,200	11,700	13,900	16,000	18,600	20,800	23,300	30,400		62,400

Table A3 Peak flows (m^3/s) versus AEP; MCS results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Linville	160	670	1,300	1,900	2,900	3,700	4,200	4,800	5,400	6,000	7,900	13,700	21,400
Gregors Creek	270	1,200	2,300	3,500	5,300	6,500	7,400	8,800	9,700	10,800	14,500	26,300	36,300
Fulham Vale	280	1,200	2,300	3,400	5,100	6,400	7,400	8,800	9,700	11,100	14,700	26,600	34,500
Peachester	97	200	280	370	480	610	690	800	900	1,000	1,300	1,800	2,400
Woodford	230	470	700	920	1,200	1,500	1,700	2,000	2,200	2,500	3,200	4,300	6,000
Somerset Dam	730	1,500	2,200	2,800	3,900	4,600	5,200	6,100	6,700	7,400	9,500	13,400	18,300
Tinton	37	210	390	590	840	1,100	1,200	1,400	1,600	1,800	2,400	3,400	6,000
Middle Ck	1,100	2,700	4,200	6,200	9,000	11,500	12,900	15,000	16,900	18,900	24,400	47,400	57,100
Wivenhoe	980	2,700	4,300	6,300	8,900	11,200	12,800	15,100	16,800	19,000	25,000	49,200	54,800
Helidon	28	160	320	500	740	960	1,100	1,300	1,500	1,700	2,200	3,400	6,700

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Gatton	110	550	1,000	1,600	2,300	3,100	3,700	4,400	5,000	5,600	7,900	13,600	24,000
Glenore Grove	160	750	1,400	2,000	3,200	4,000	4,900	5,800	6,500	7,400	10,400	18,300	27,700
Savages Crossing	1,200	3,100	5,300	7,900	11,200	14,300	16,600	19,100	21,500	23,900	32,600		63,800
Mount Crosby	1,200	3,000	5,200	7,900	10,800	13,800	16,100	18,800	21,300	23,400	32,400		62,600
Walloon	170	420	670	890	1,300	1,600	1,900	2,200	2,500	2,800	3,700	5,200	8,400
Kalbar Weir	160	370	540	770	1,100	1,300	1,500	1,800	1,900	2,200	2,800	4,100	6,800
Amberley	230	450	630	970	1,400	1,800	2,100	2,500	2,700	3,100	4,200	6,100	9,700
Loamside	60	140	220	310	430	520	610	720	810	920	1,100	1,600	2,800
Ipswich	510	1,000	1,500	2,100	2,900	3,700	4,300	5,000	5,600	6,400	8,400	12,600	18,100
Moggill	1,700	3,800	5,900	8,500	11,700	14,600	17,000	19,900	23,000	25,900	35,800		64,400
Centenary Bridge	1,700	3,700	5,800	8,200	11,100	13,900	16,000	18,800	22,000	24,700	34,100		63,900
Brisbane	1,700	3,800	5,700	8,200	11,100	13,600	15,500	18,600	21,300	24,200	32,600		61,100

Table A4 Peak flows (m^3/s) versus AEP; Reconciled results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	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
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
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
Peachester	120	300	420	540	680	780	870	980	1,100	1,200	1,400	1,900	2,600
Woodford	210	510	750	1,000	1,300	1,500	1,700	2,000	2,200	2,500	3,200	4,300	6,000
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

Location	AEP (1 in N)											
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Tinton	37	210	390	590	840	1,100	1,200	1,400	1,600	1,800	2,400	3,400	6,000
Middle Ck	670	2,400	4,500	6,600	9,000	10,900	12,400	14,600	16,500	18,800	25,200		65,700
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
Helidon	73	230	400	590	800	960	1,100	1,300	1,500	1,700	2,200	3,400	6,700
Gatton	89	410	830	1,300	2,300	3,100	3,700	4,400	5,000	5,600	7,900	13,600	24,000
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
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
Mount Crosby	830	3,100	5,400	8,100	11,400	13,800	16,100	18,800	21,300	23,400	32,400		62,600
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
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
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
Loamside	65	210	310	390	490	580	670	780	870	980	1,200	1,700	2,800
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
Moggill	1,100	3,800	6,400	9,300	12,300	14,600	17,000	19,900	23,000	25,900	35,800		64,400
Centenary Bridge	1,100	3,700	6,200	9,000	11,800	14,000	16,400	19,300	22,300	25,300	35,500		64,900
Brisbane	1,100	3,700	6,200	8,900	11,800	13,900	16,300	19,100	22,000	25,000	34,600		62,800

A.2 Figures of frequency curves

This section contains figures with frequency curves for the 22 locations of interest. Each Figure contains the following graphs:

- Empirical frequency estimates from rated flows (if available)
- FFA results (if available)
- FFA 90 uncertainty quantiles (if available)
- DEA results
- MCS results
- Proposed reconciled design peak flows

For each location, 2 figures are provided:

1. Results for the AEP range 1 in 2 – AEP of PMP
2. Results for the AEP range 1 in 2 – 1 in 100

This means the first plot displays results for all AEP-values of interest, whereas the second plot zooms in on the results for the higher range of AEP-values.

Figure A.2.1a | Location Linville; no dams conditions

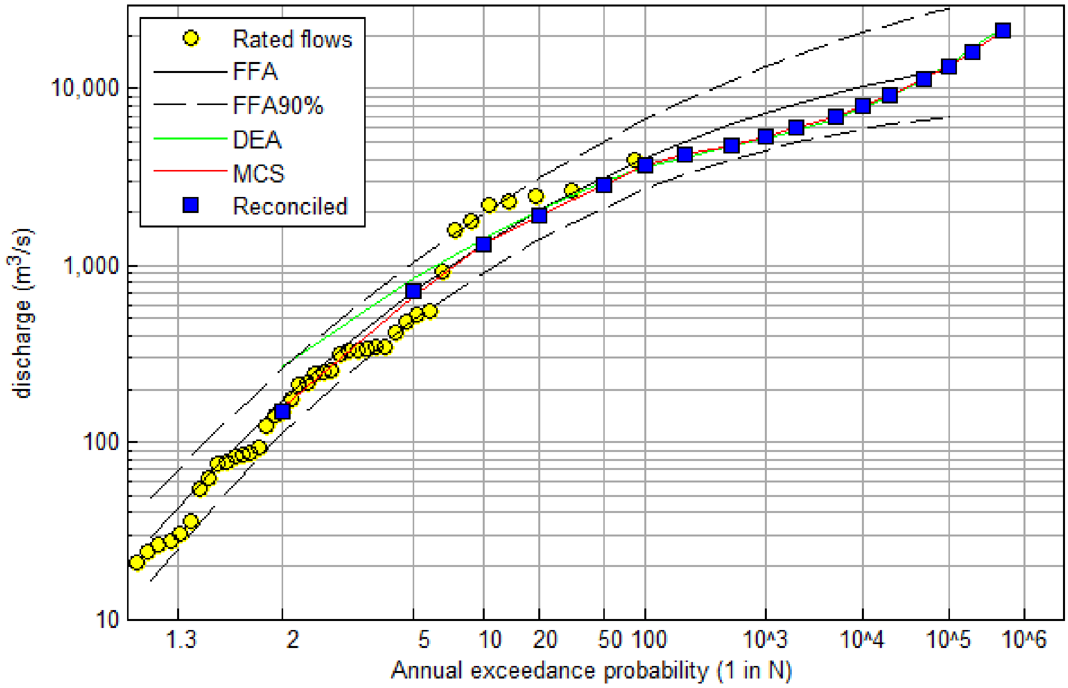


Figure A.2.1b | Location Linville; no dams conditions

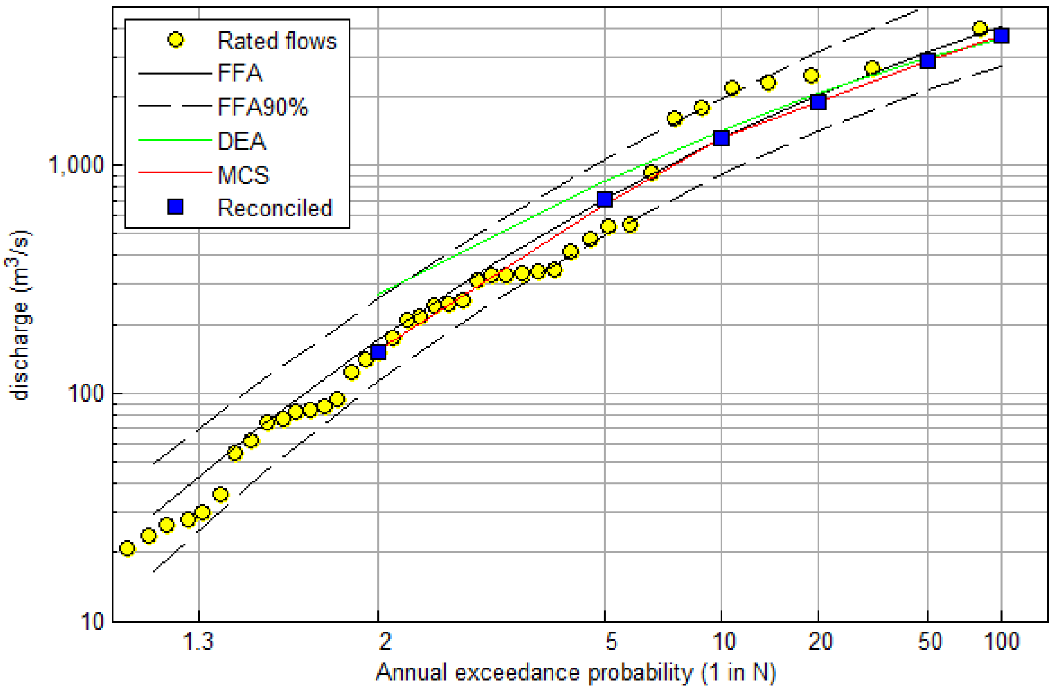


Figure A.2.2a | Location Gregors Creek; no dams conditions

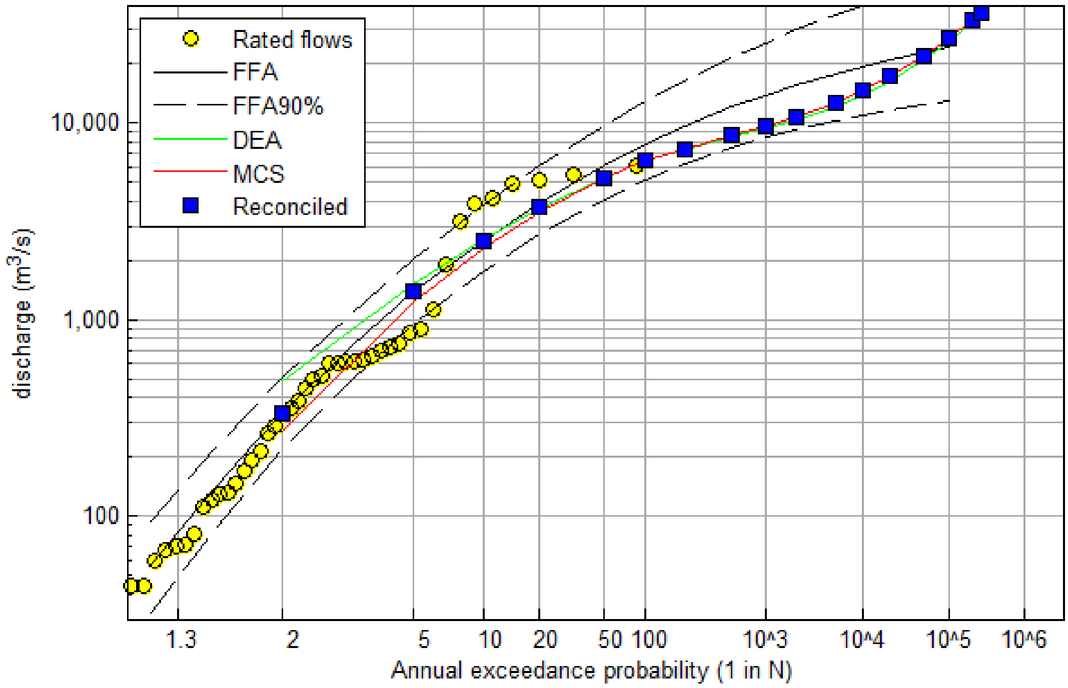


Figure A.2.2b | Location Gregors Creek; no dams conditions

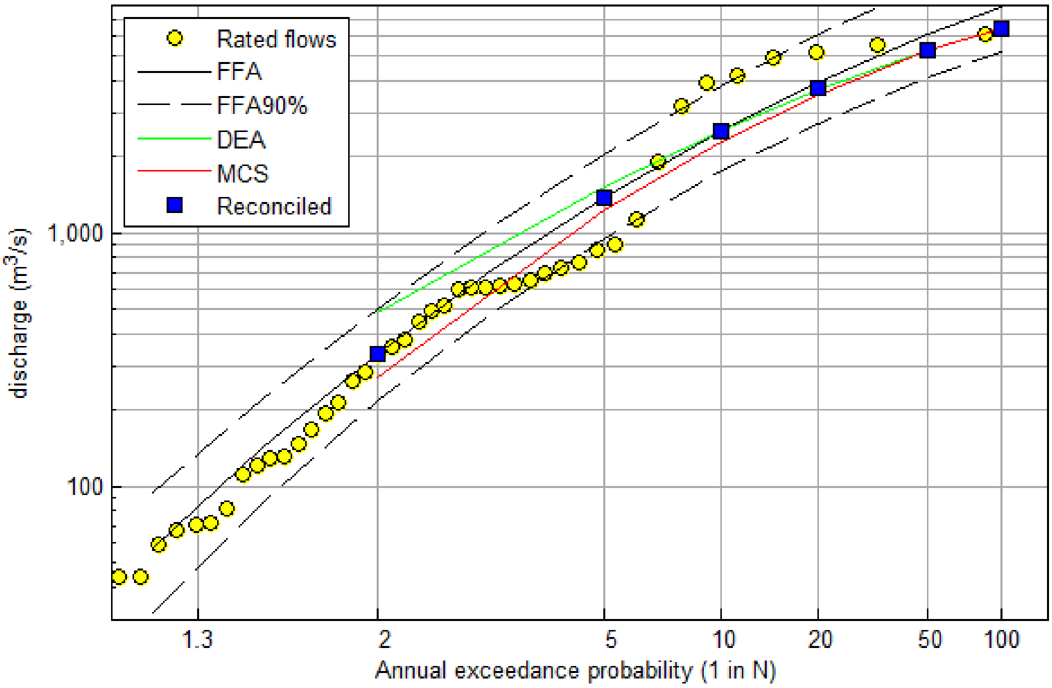


Figure A.2.3a | Location Fulham Vale; no dams conditions

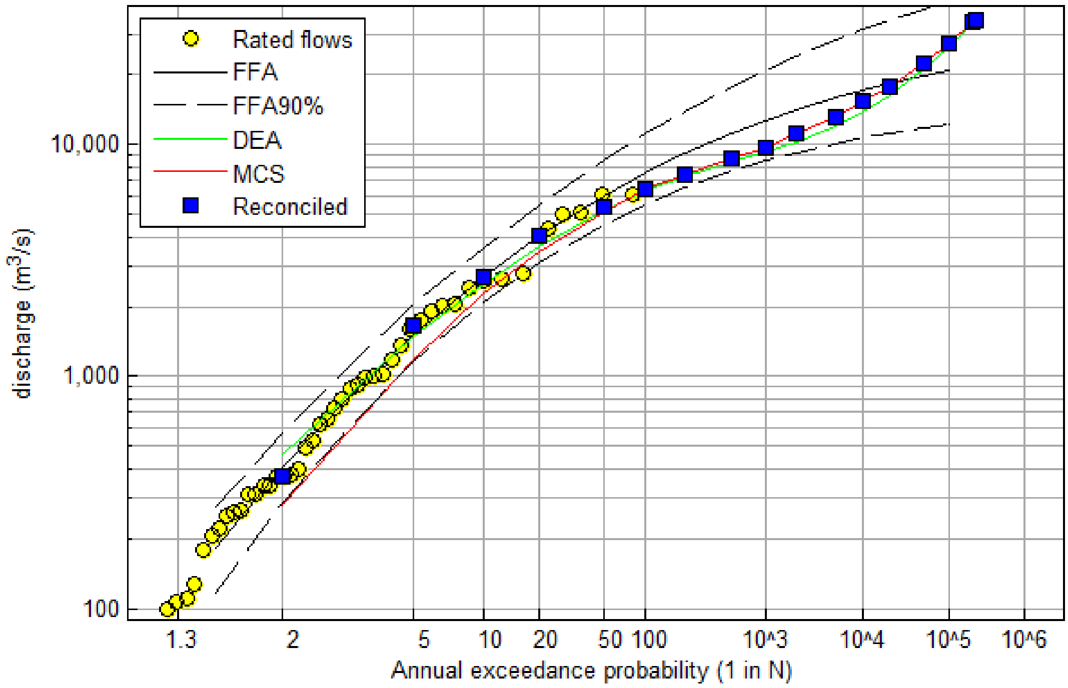


Figure A.2.3b | Location Fulham Vale; no dams conditions

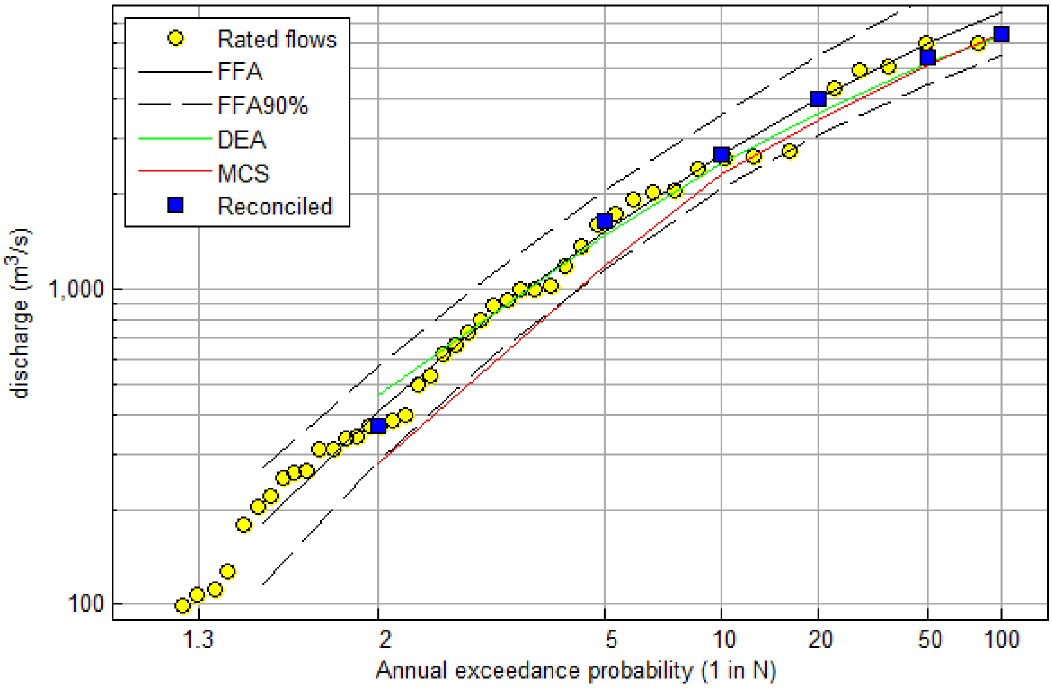


Figure A.2.4a | Location Peachester; no dams conditions

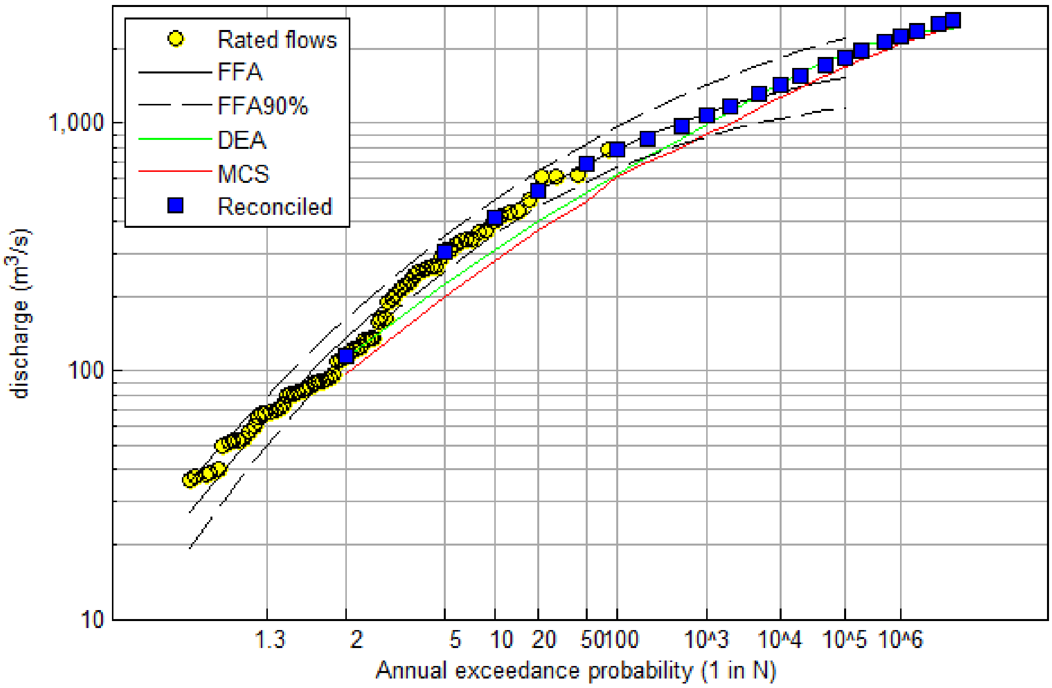


Figure A.2.4b | Location Peachester; no dams conditions

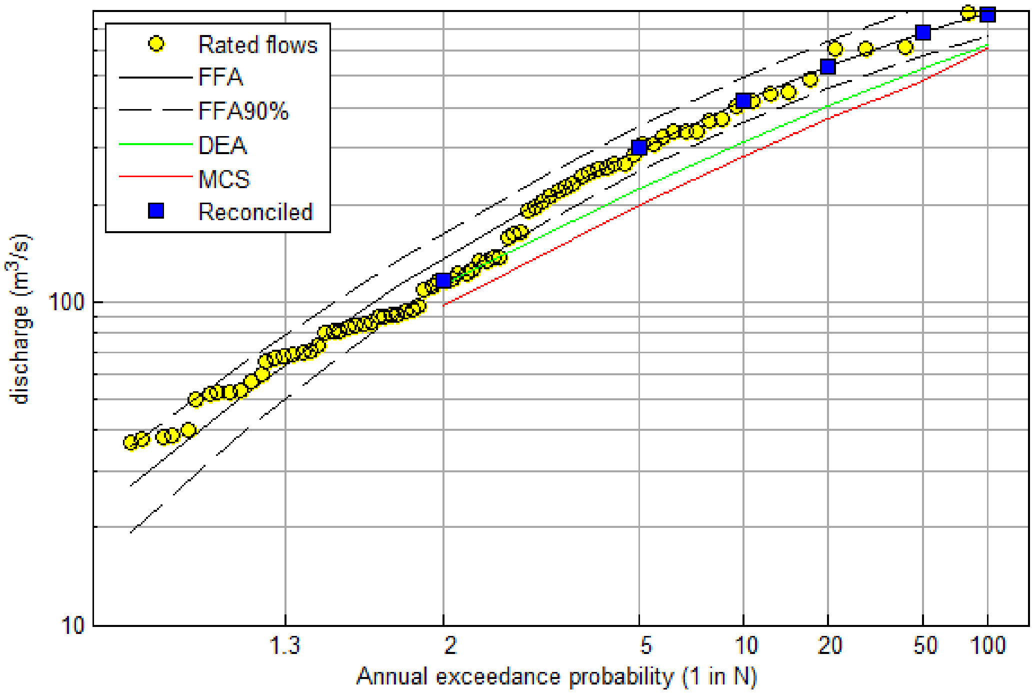


Figure A.2.5a | Location Woodford; no dams conditions

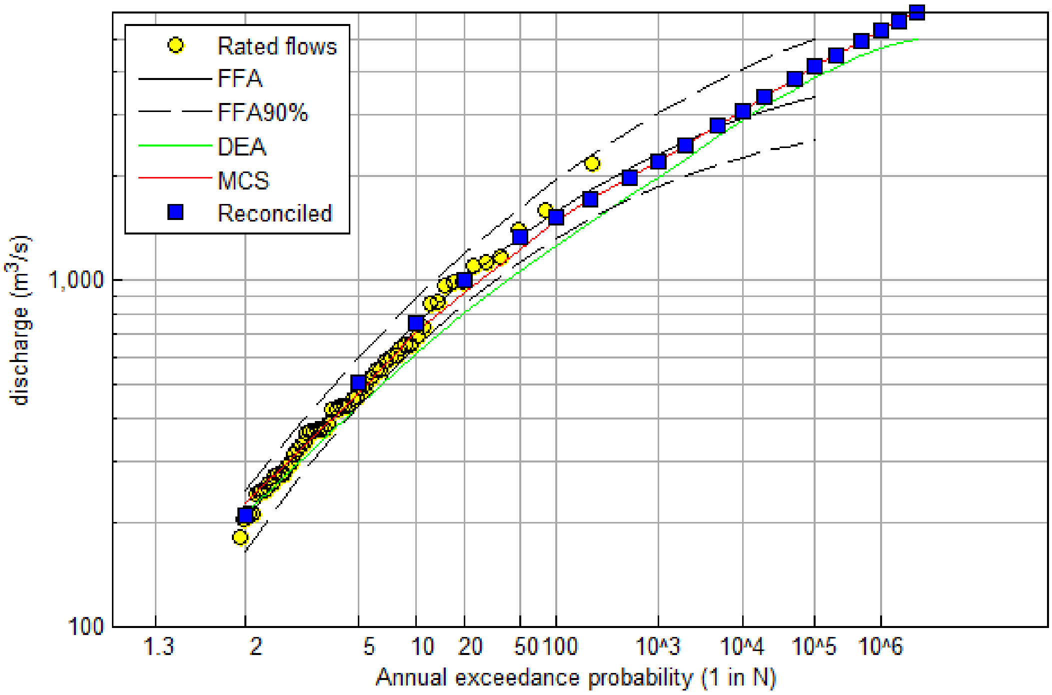


Figure A.2.5b | Location Woodford; no dams conditions

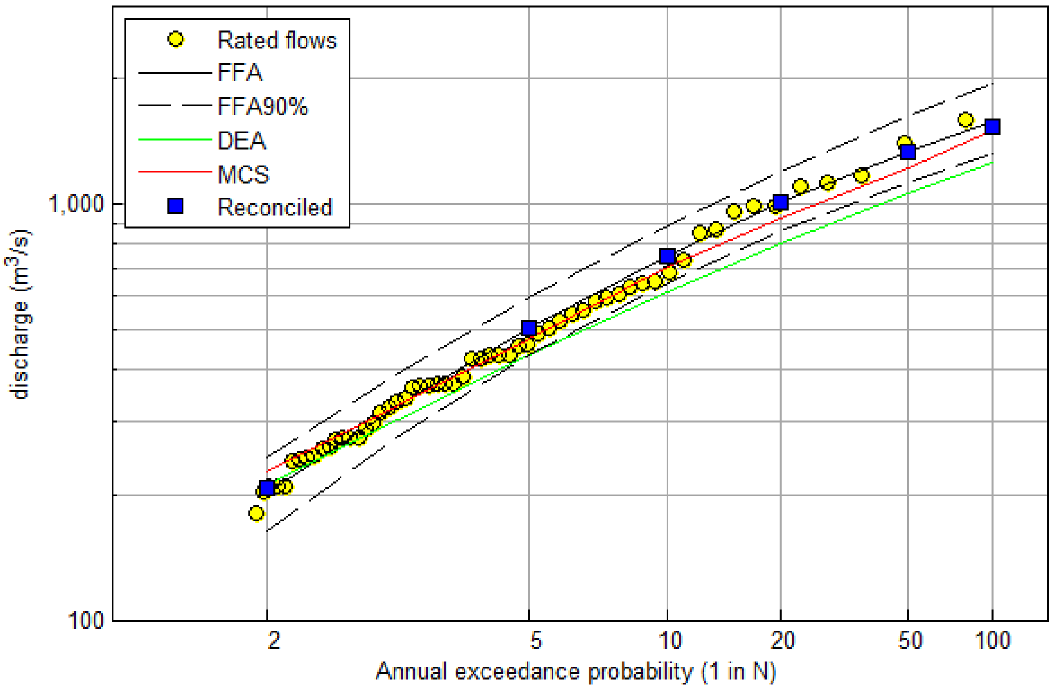


Figure A.2.6a | Location Somerset Dam; no dams conditions

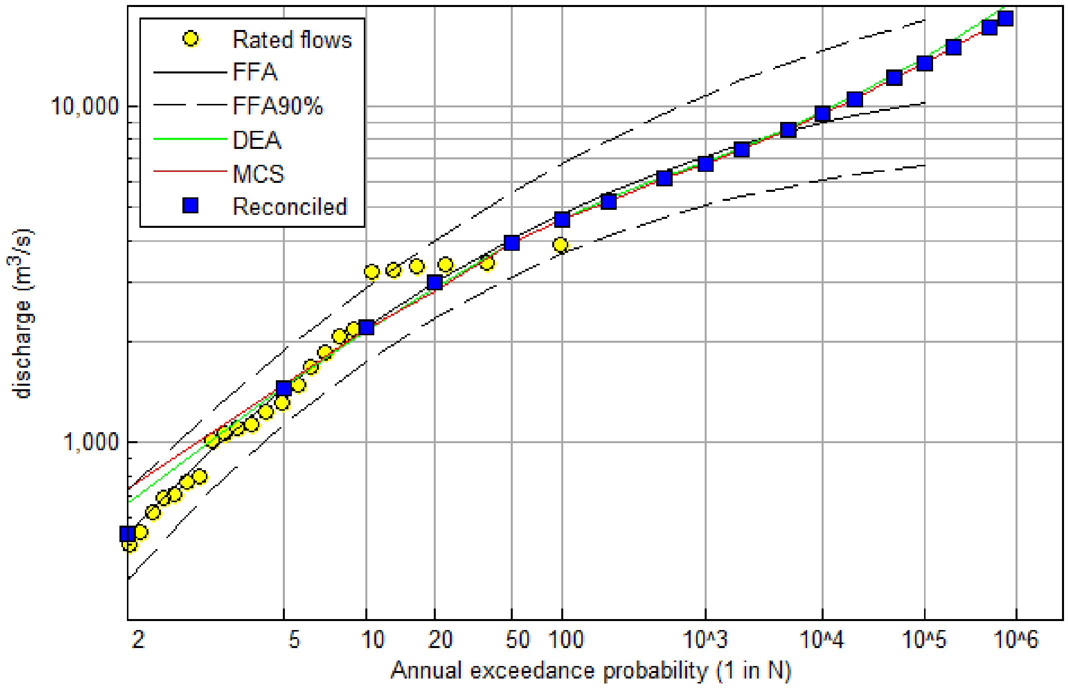


Figure A.2.6b | Location Somerset Dam; no dams conditions

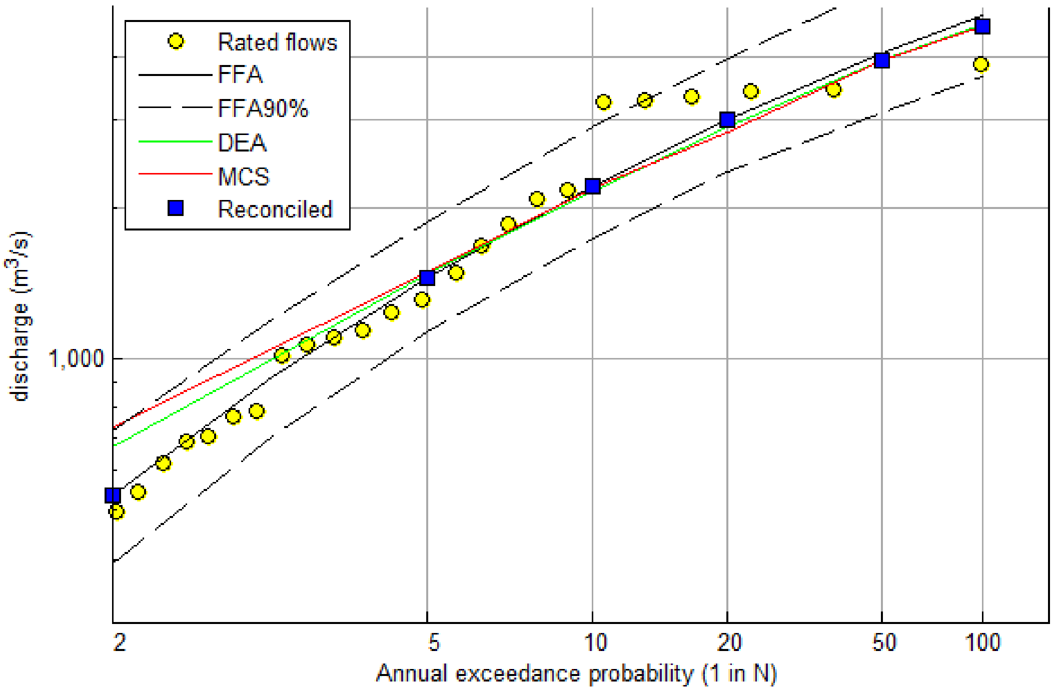


Figure A.2.7a | Location Tinton; no dams conditions

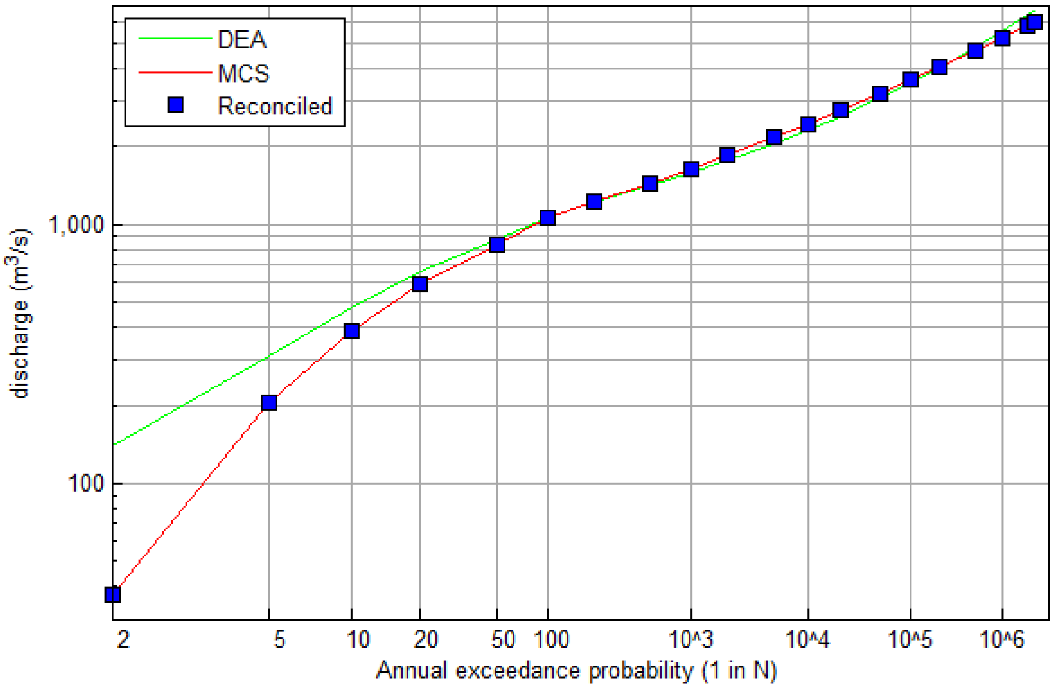


Figure A.2.7b | Location Tinton; no dams conditions

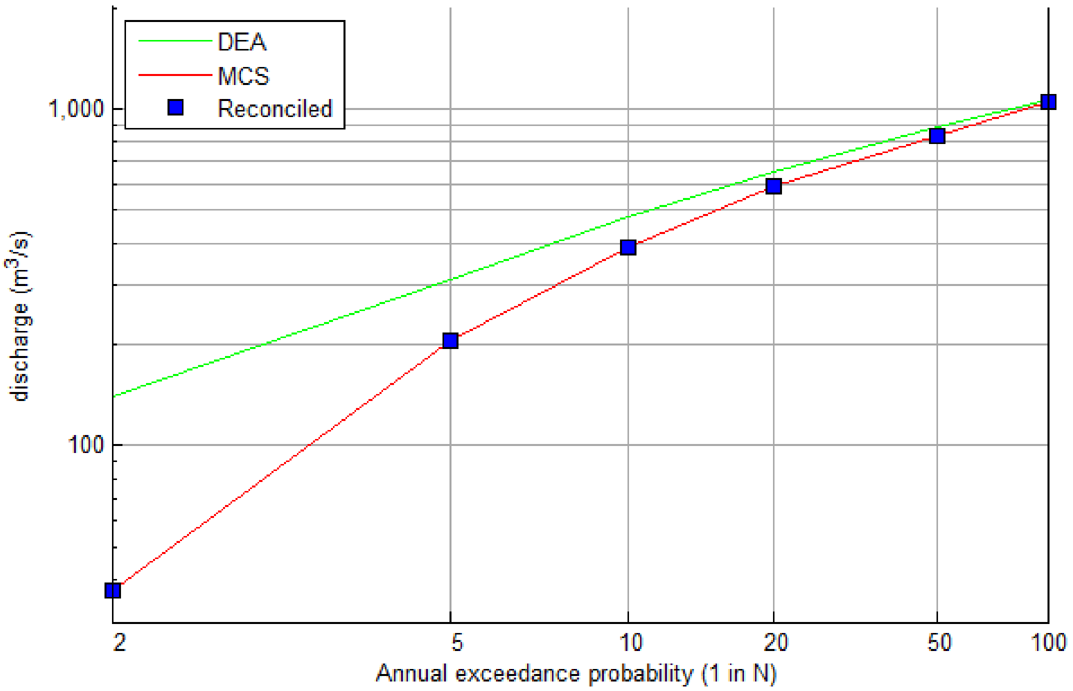


Figure A.2.8a | Location Middle Ck; no dams conditions

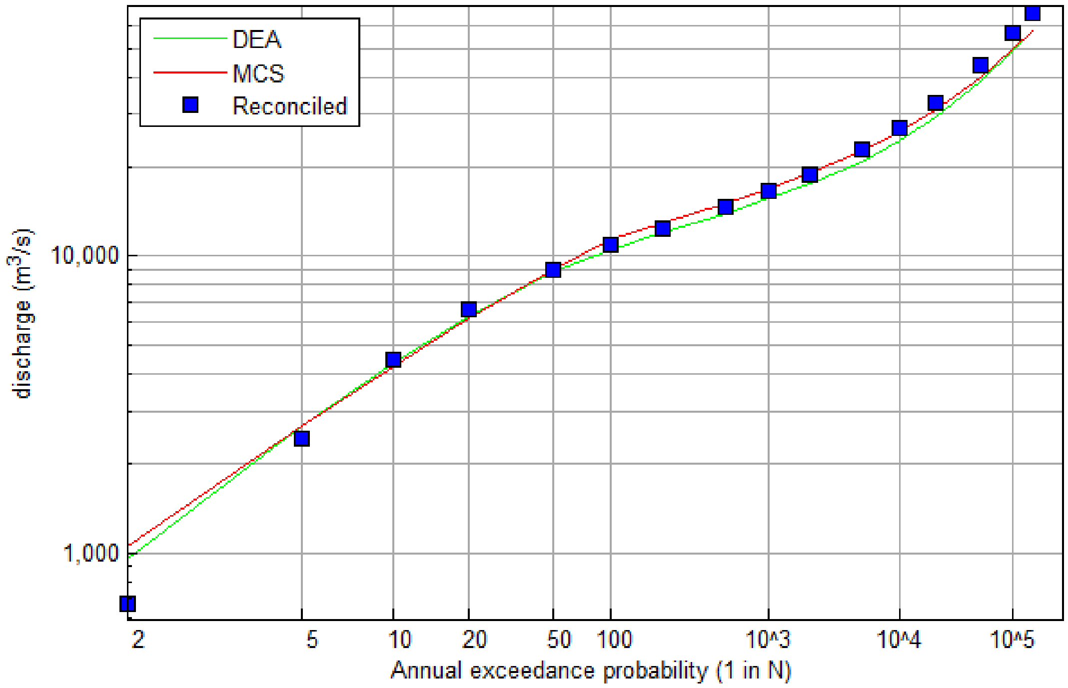


Figure A.2.8b | Location Middle Ck; no dams conditions

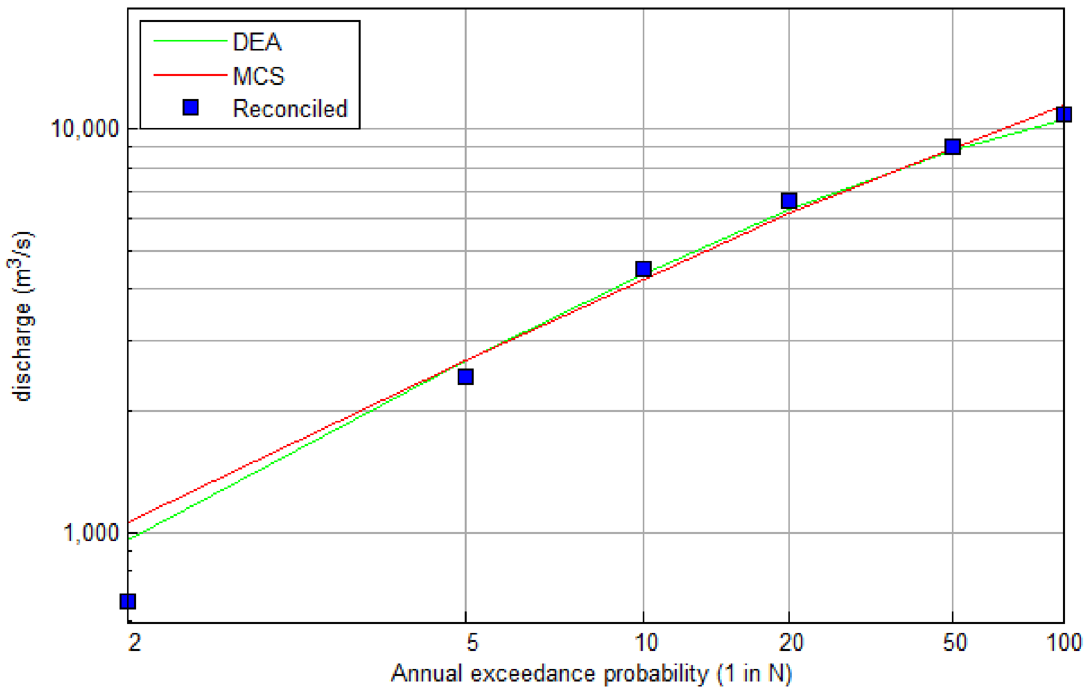


Figure A.2.9a | Location Wivenhoe; no dams conditions

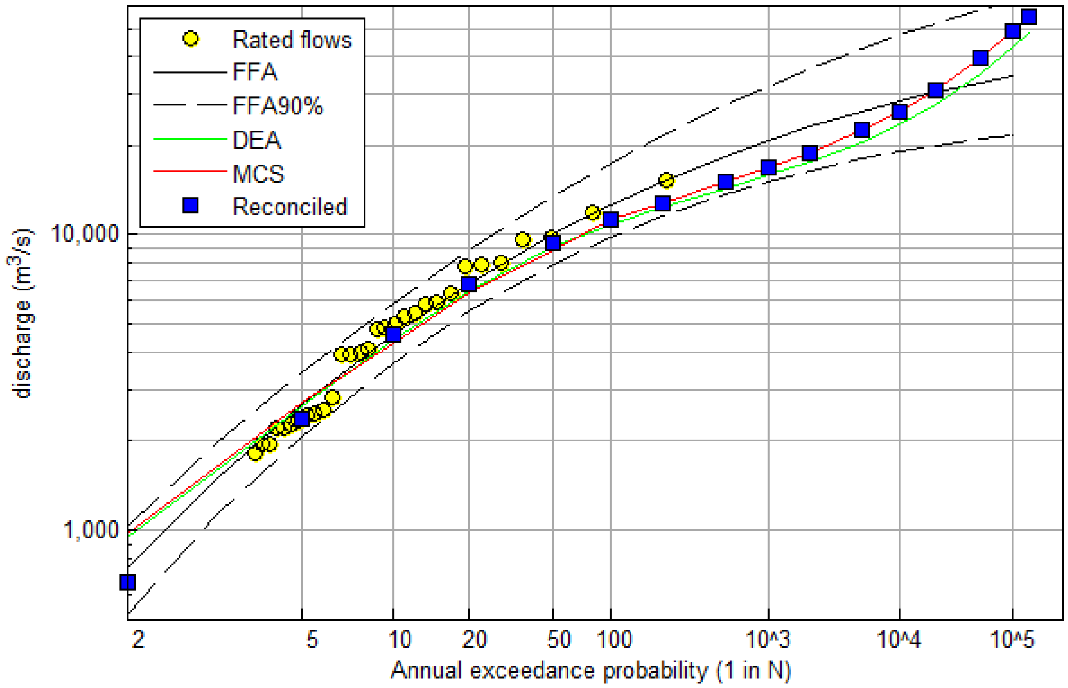


Figure A.2.9b | Location Wivenhoe; no dams conditions

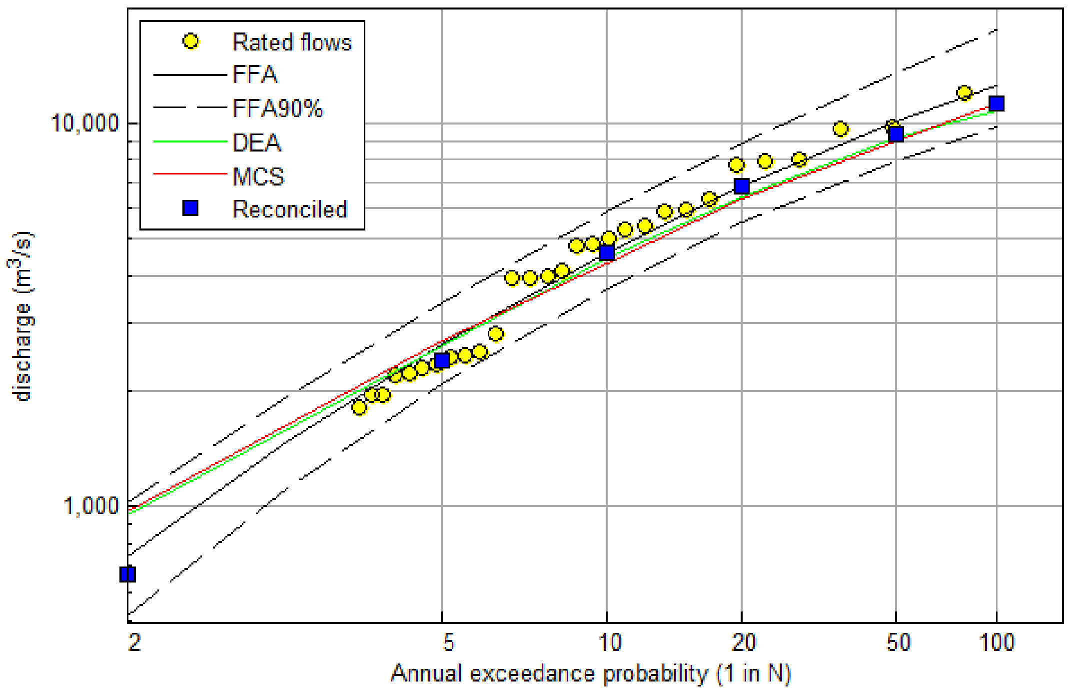


Figure A.2.10a | Location Helidon; no dams conditions

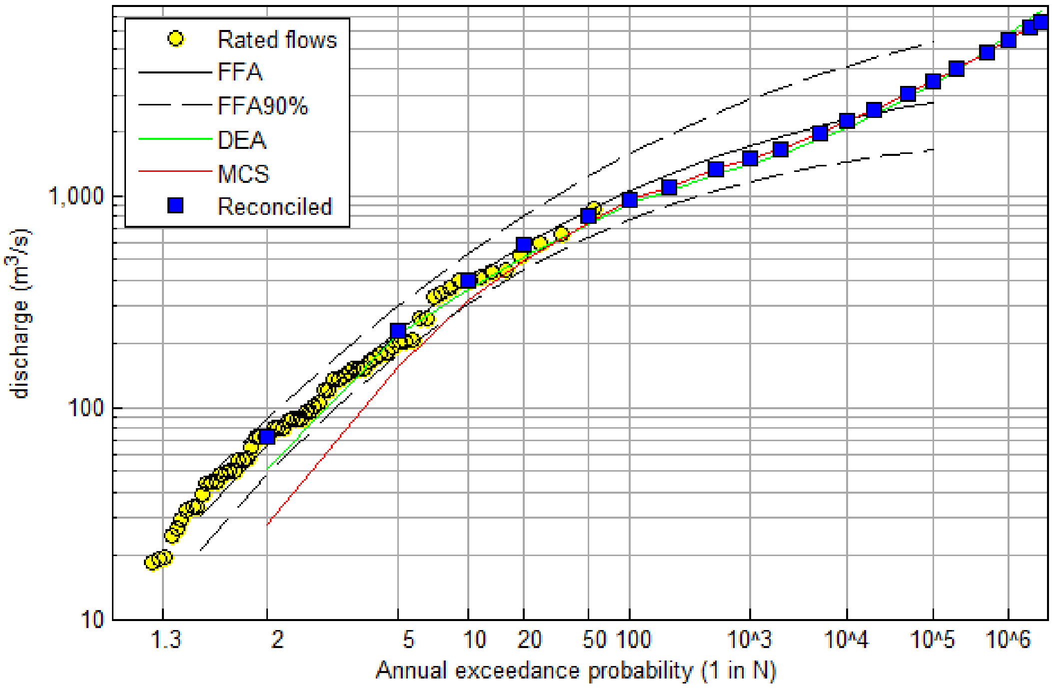


Figure A.2.10b | Location Helidon; no dams conditions

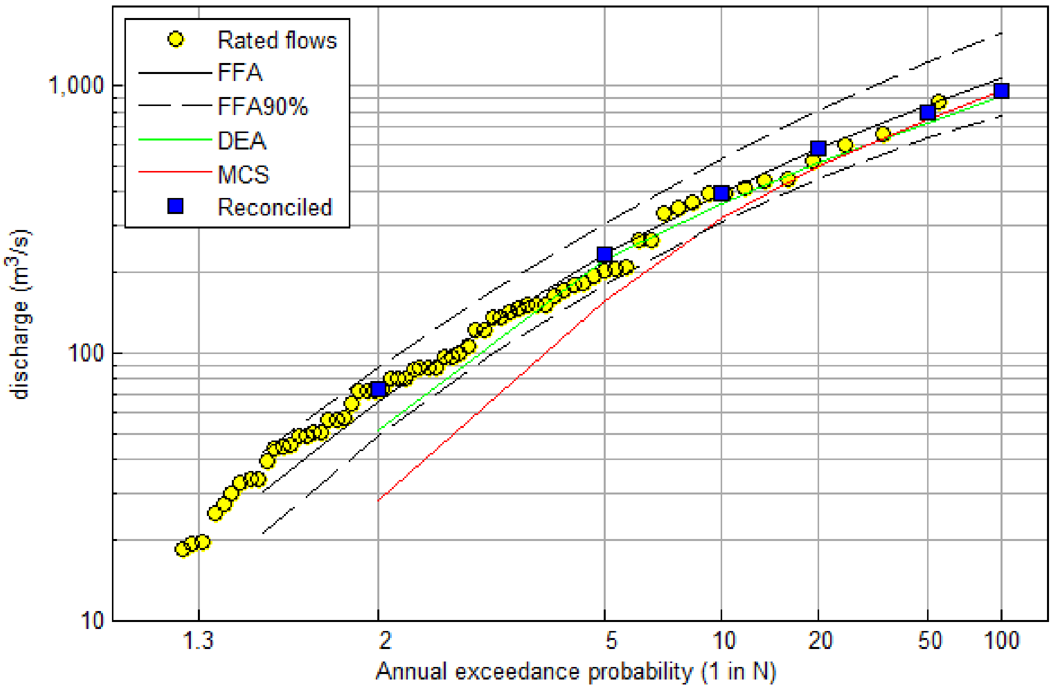


Figure A.2.11a | Location Gatton; no dams conditions

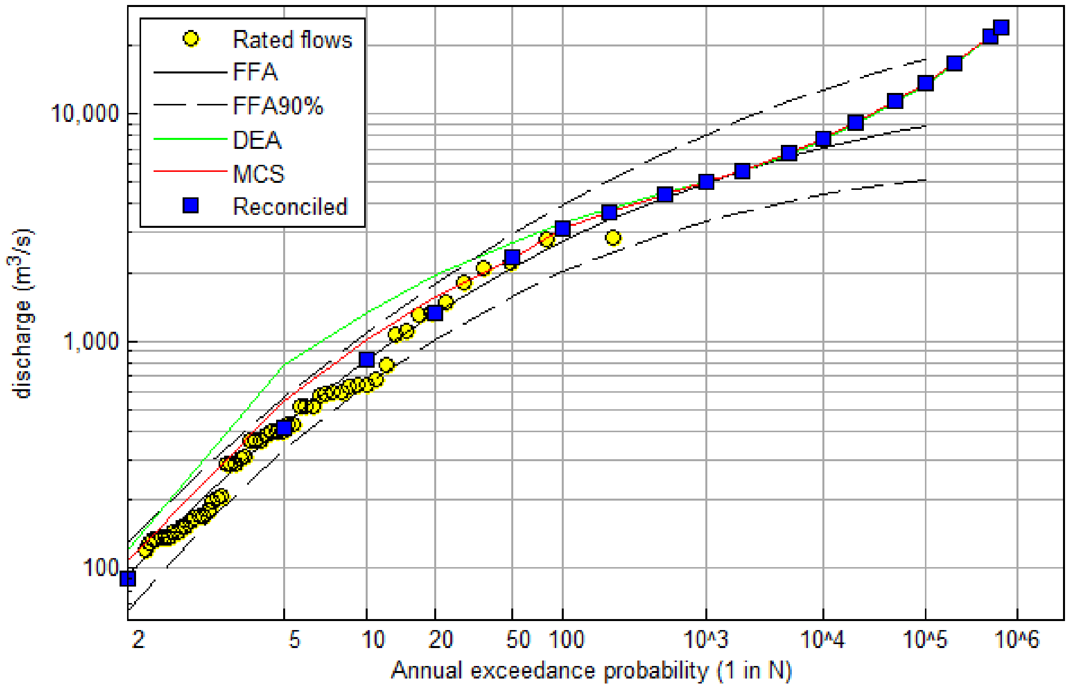


Figure A.2.11b | Location Gatton; no dams conditions

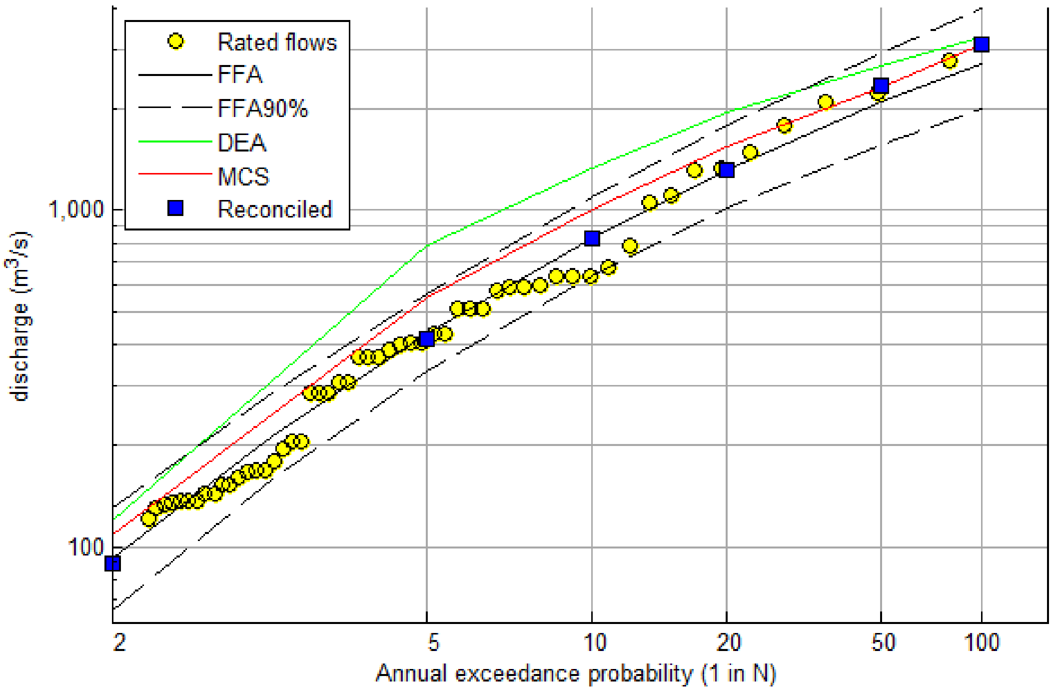


Figure A.2.12a | Location Glenore Grove; no dams conditions

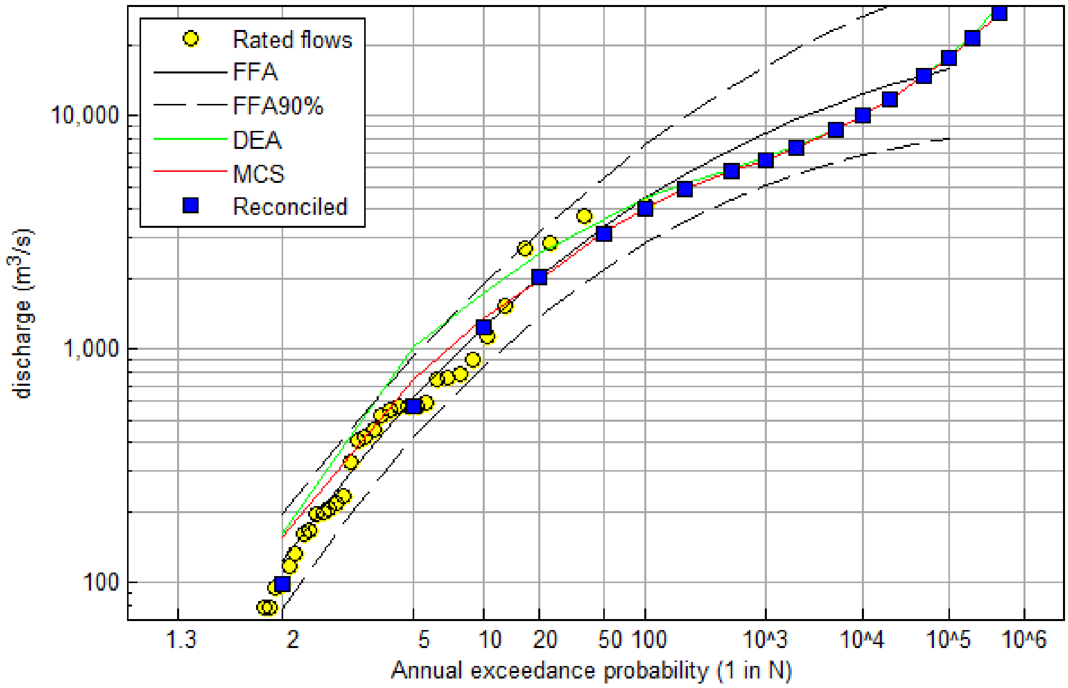


Figure A.2.12b | Location Glenore Grove; no dams conditions

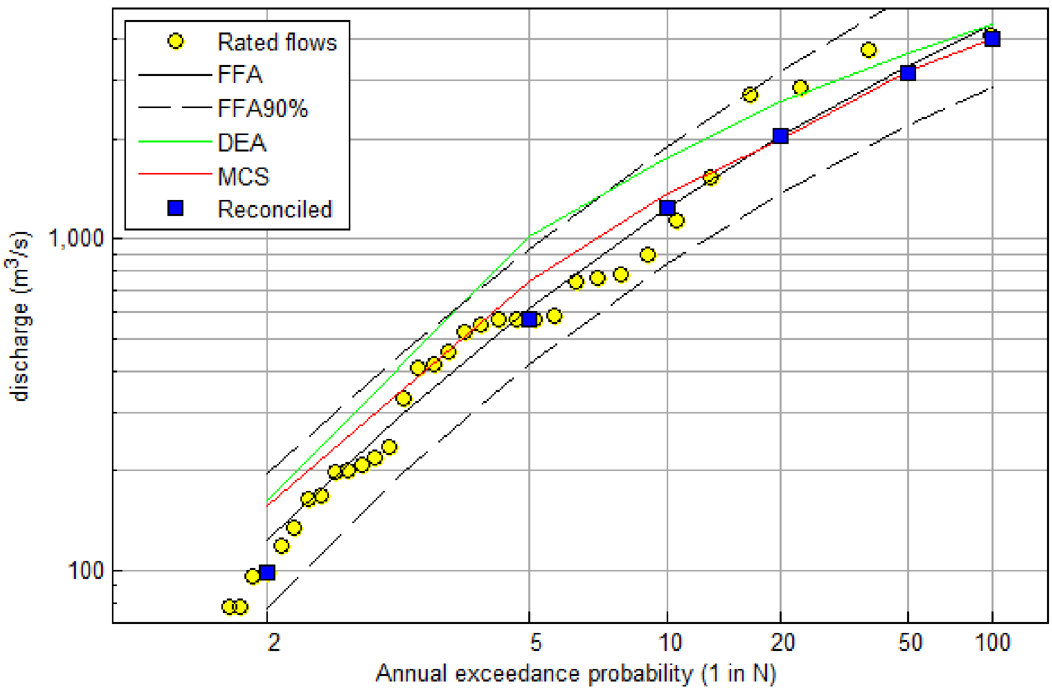


Figure A.2.13a | Location Savages Crossing; no dams conditions

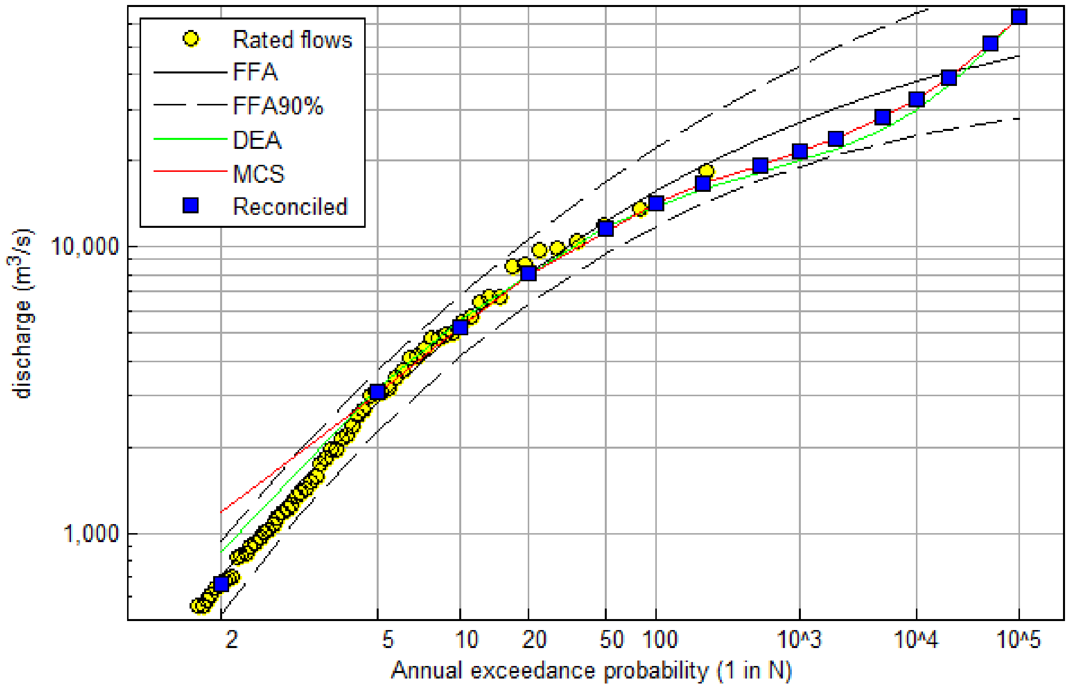


Figure A.2.13b | Location Savages Crossing; no dams conditions

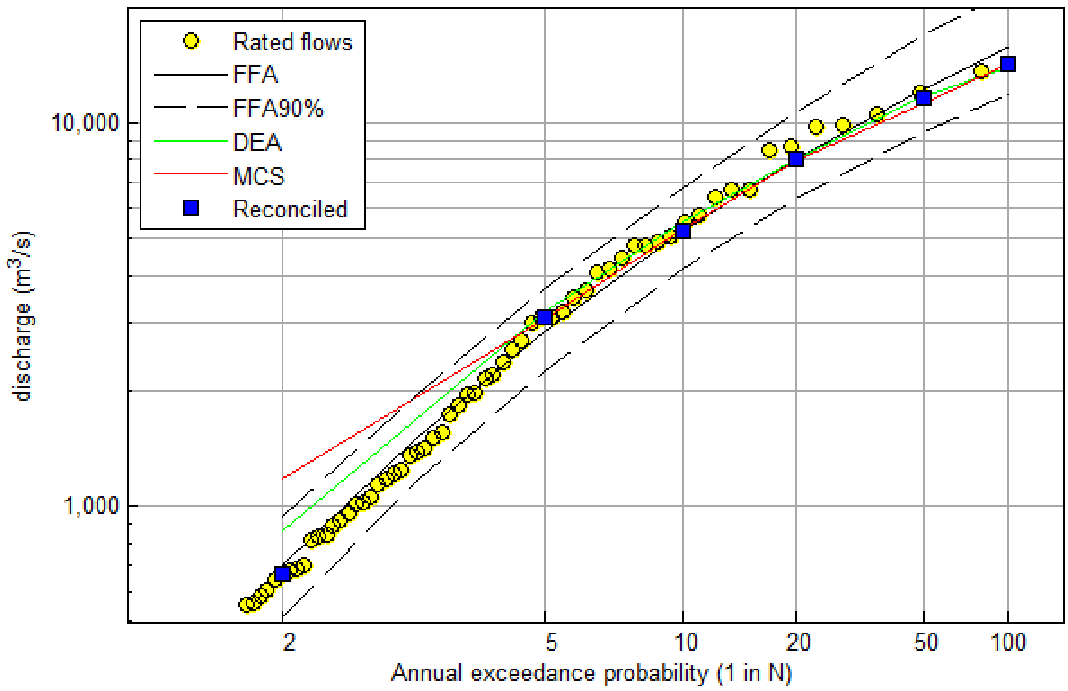


Figure A.2.14a | Location Mount Crosby; no dams conditions

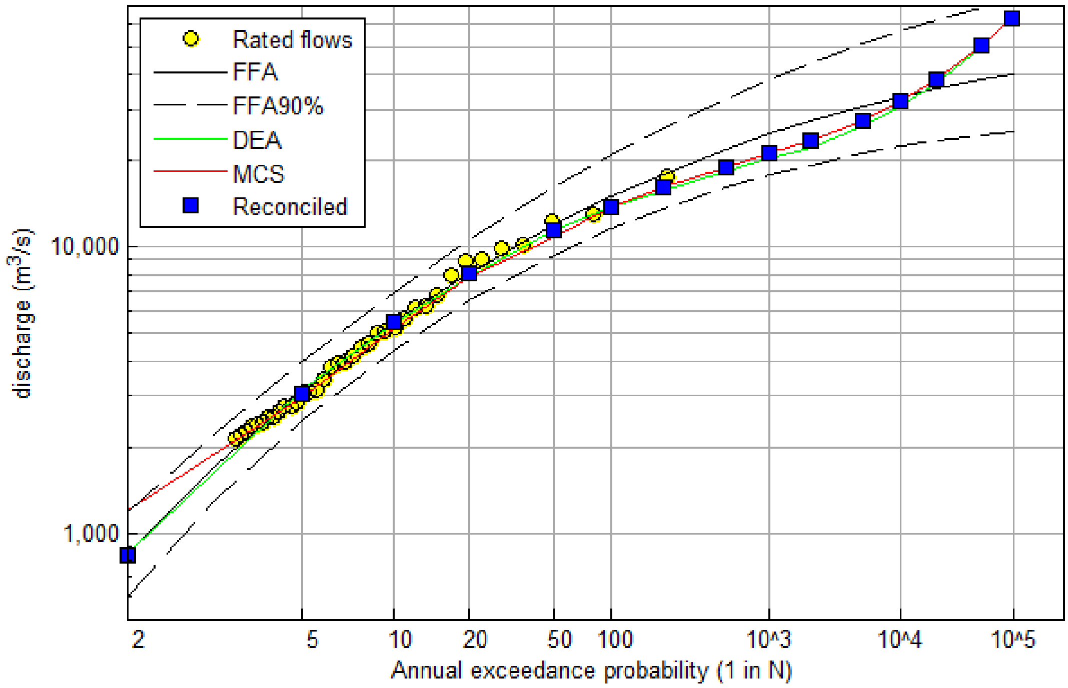


Figure A.2.14b | Location Mount Crosby; no dams conditions

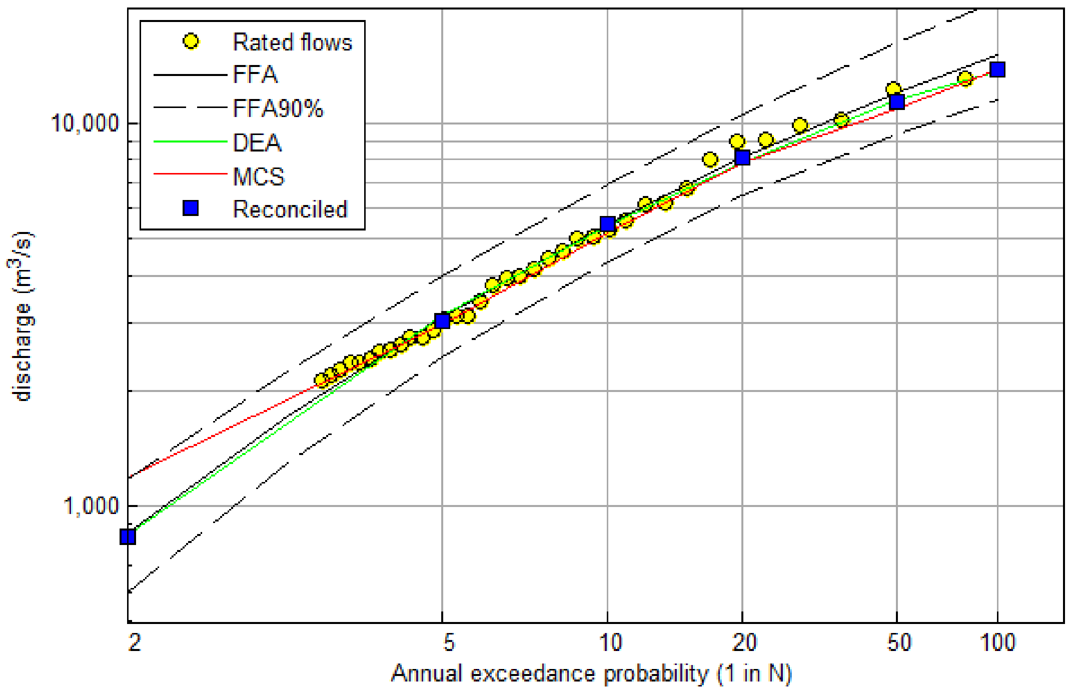


Figure A.2.15a | Location Walloon; no dams conditions

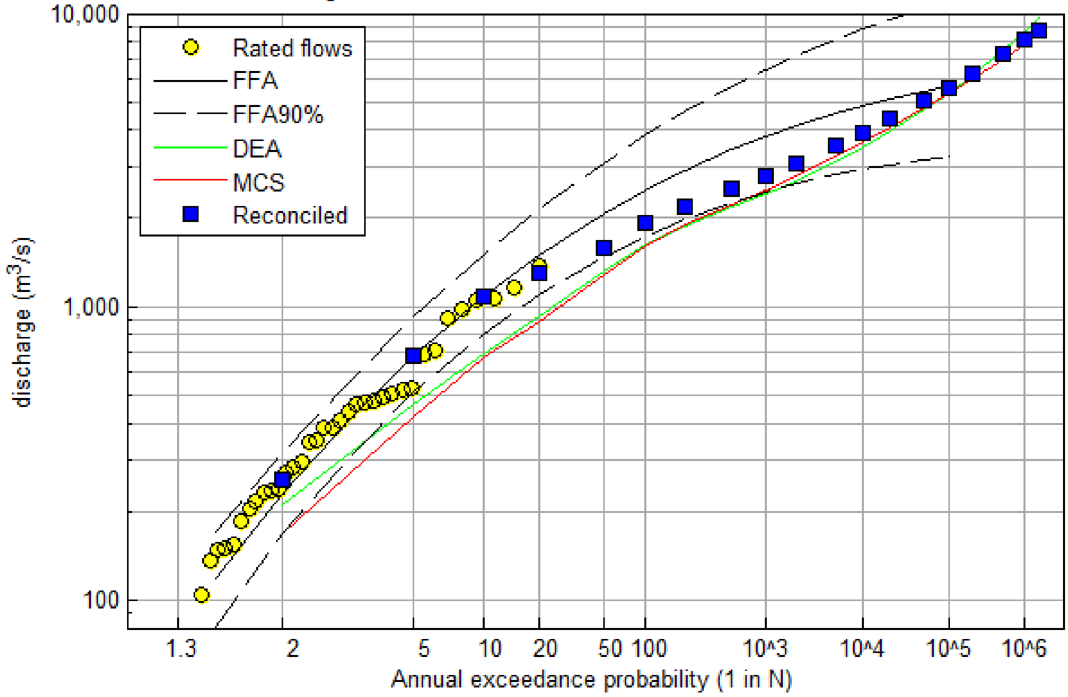


Figure A.2.15b | Location Walloon; no dams conditions

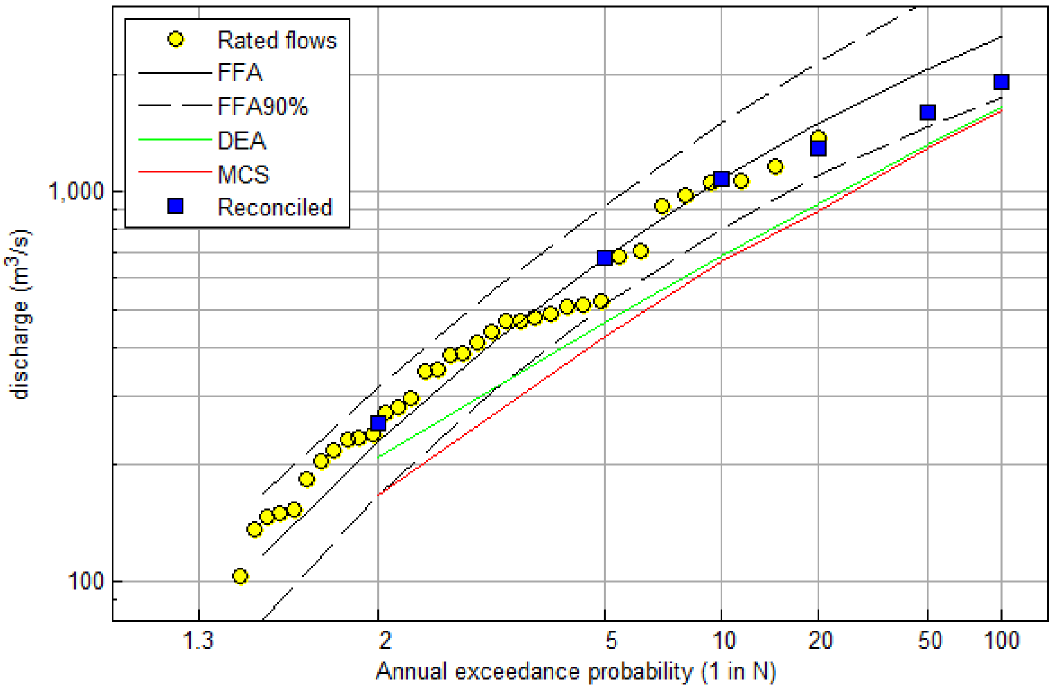


Figure A.2.16a | Location Kalbar Weir; no dams conditions

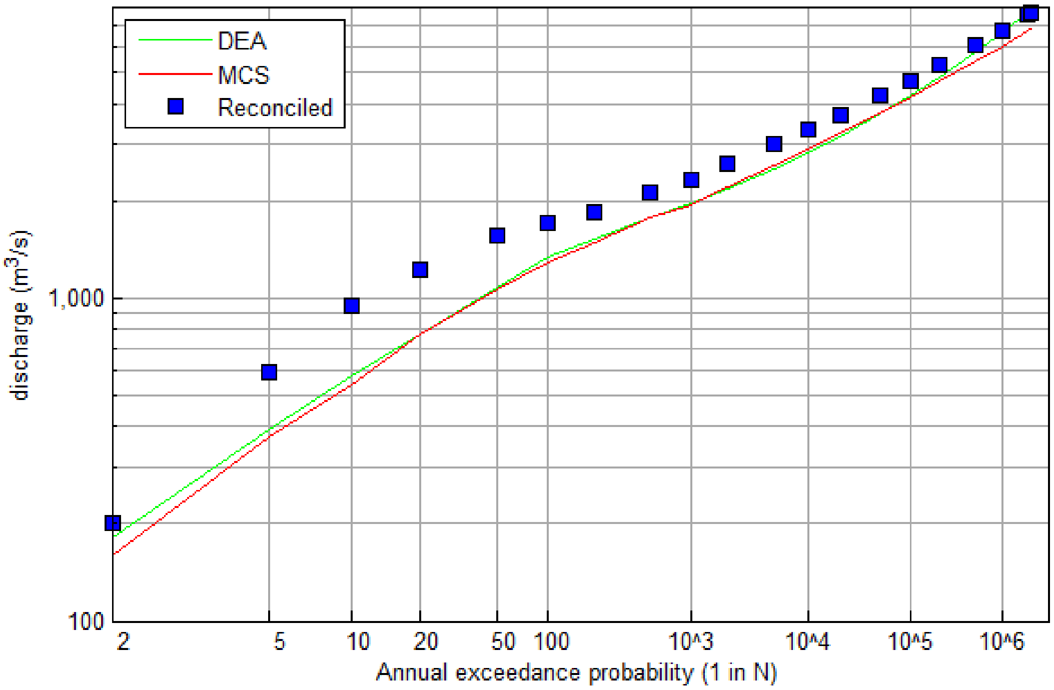


Figure A.2.16b | Location Kalbar Weir; no dams conditions

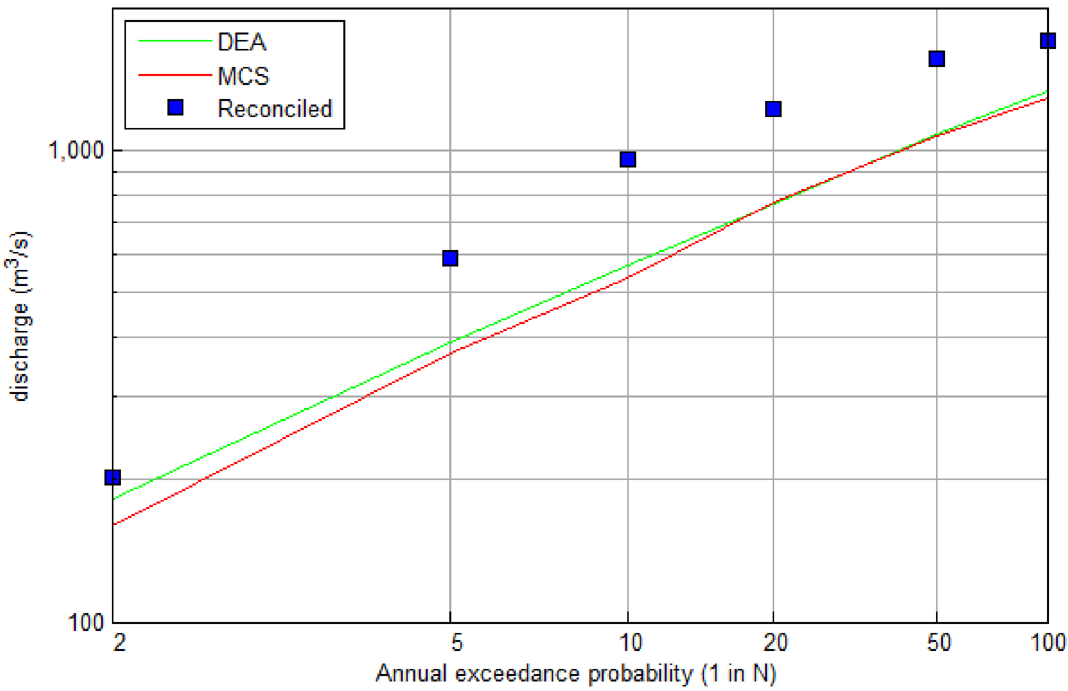


Figure A.2.17a | Location Amberley; no dams conditions

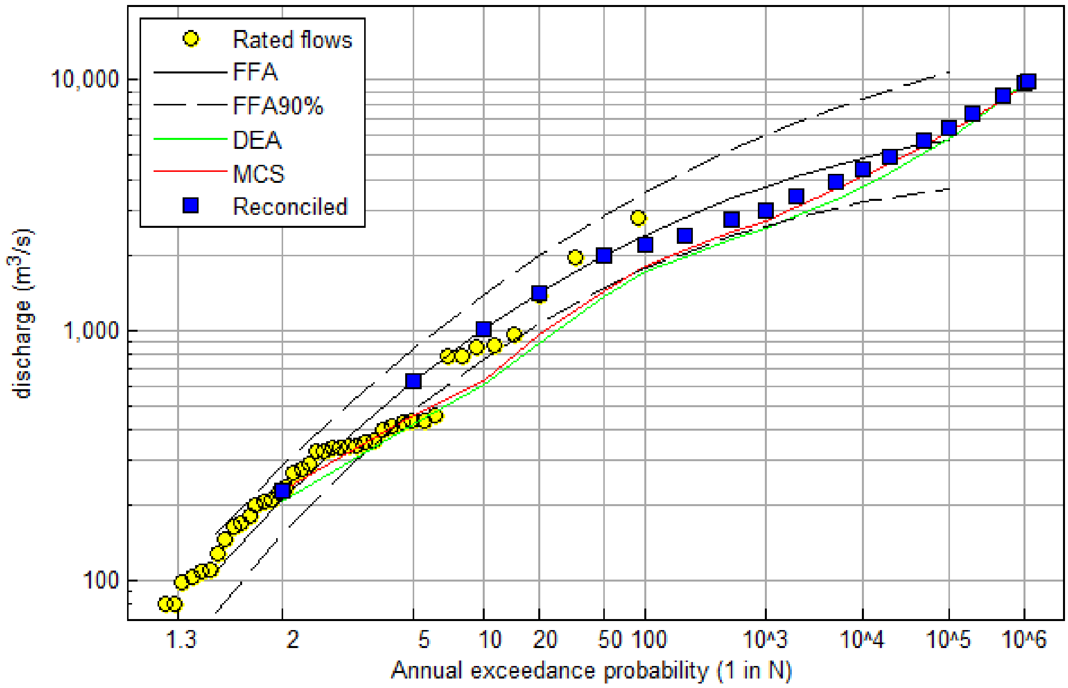


Figure A.2.17b | Location Amberley; no dams conditions

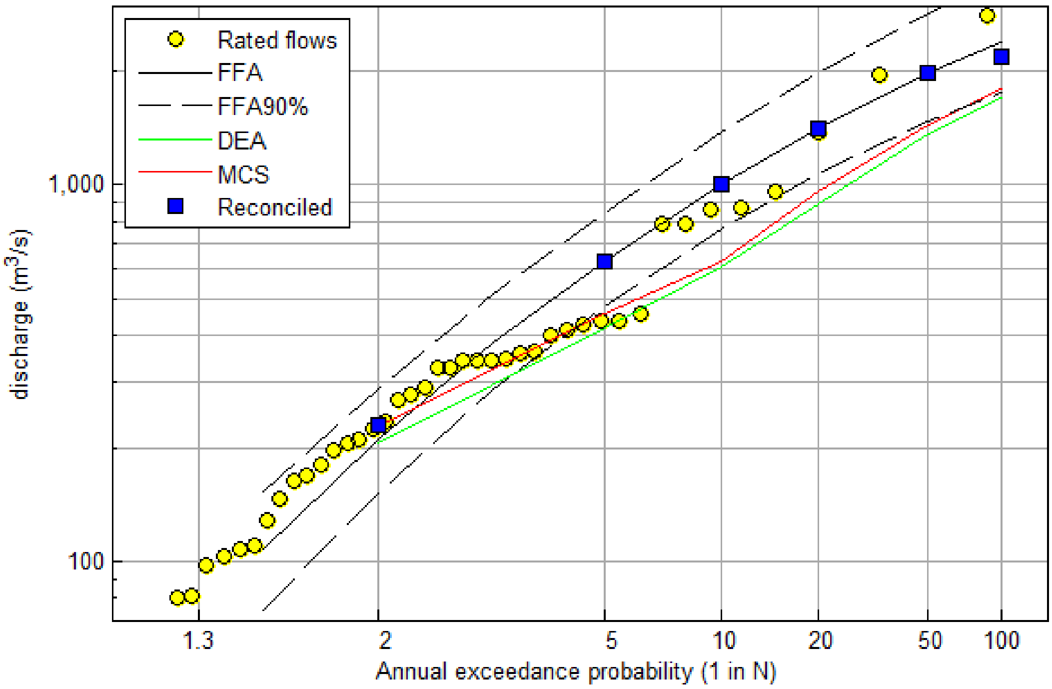


Figure A.2.18a | Location Loamside; no dams conditions

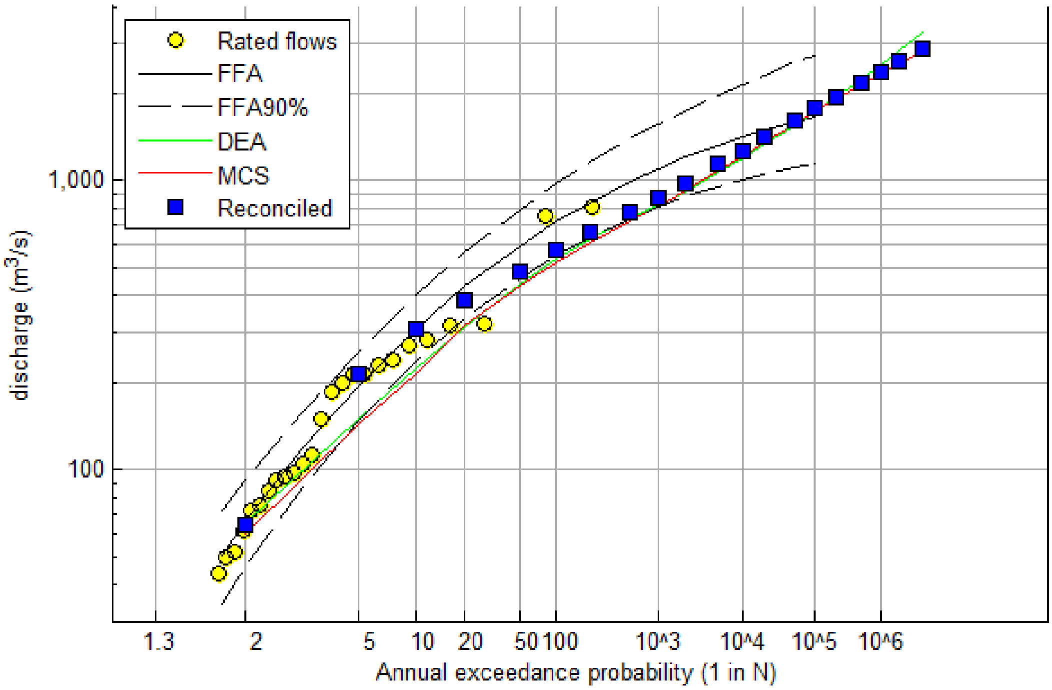


Figure A.2.18b | Location Loamside; no dams conditions

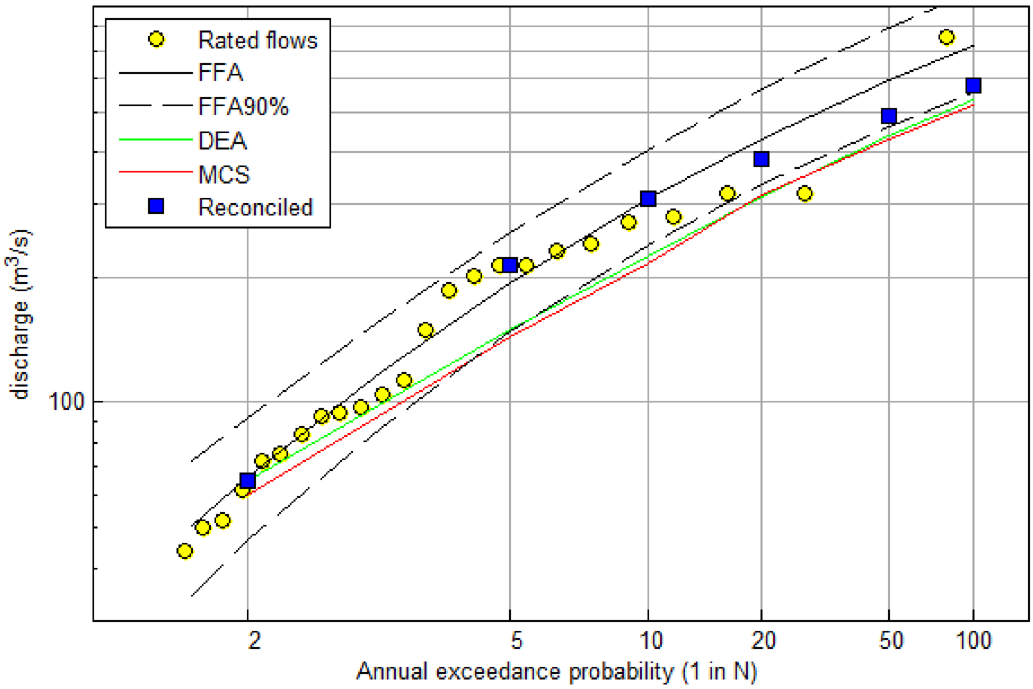


Figure A.2.19a | Location Ipswich; no dams conditions

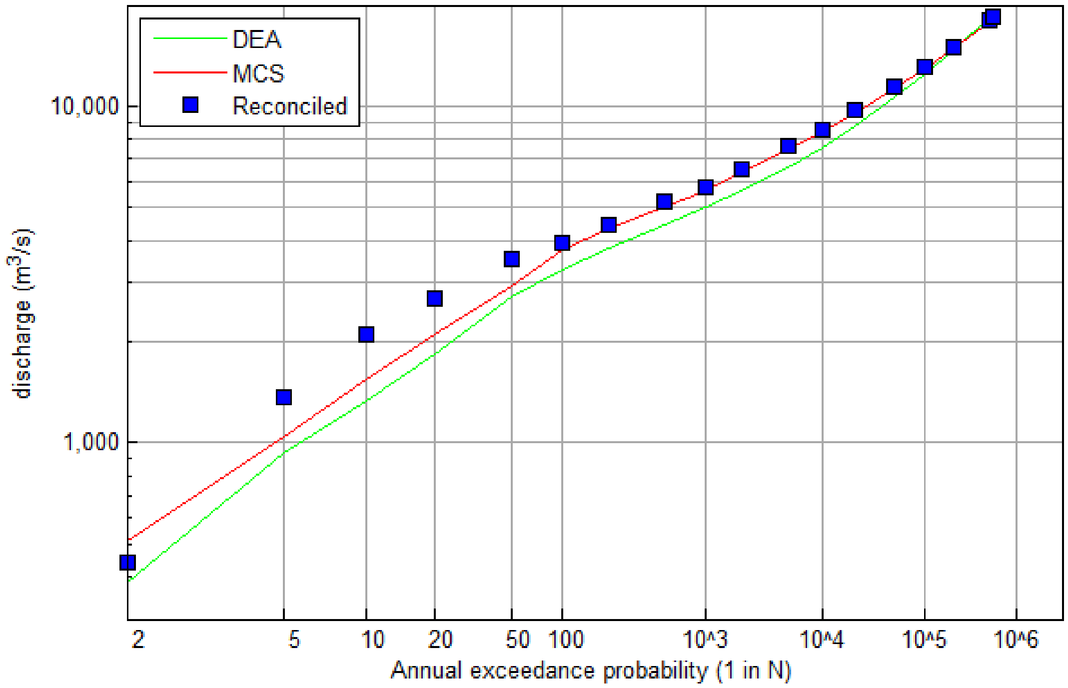
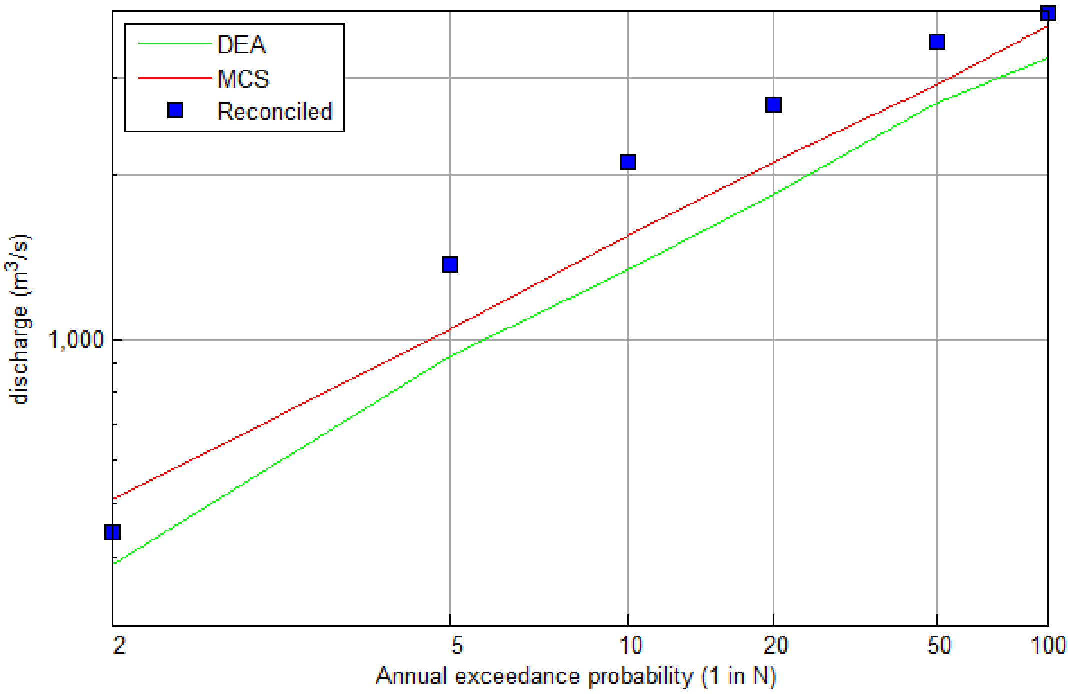


Figure A.2.19b | Location Ipswich; no dams conditions



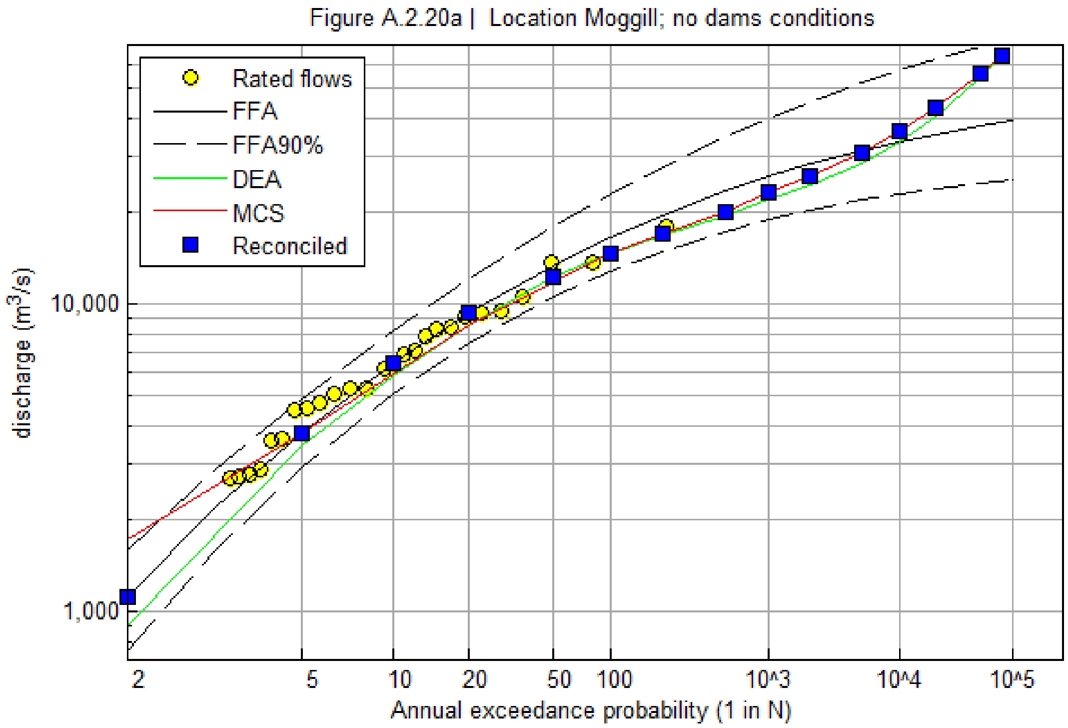


Figure A.2.20a | Location Moggill; no dams conditions
Rated flows
FFA
FFA90%
DEA
MCS
Reconciled
discharge (m3/s)
10,000
1,000
2
5
10
20
50
100
10^3
10^4
10^5
Annual exceedance probability (1 in N)

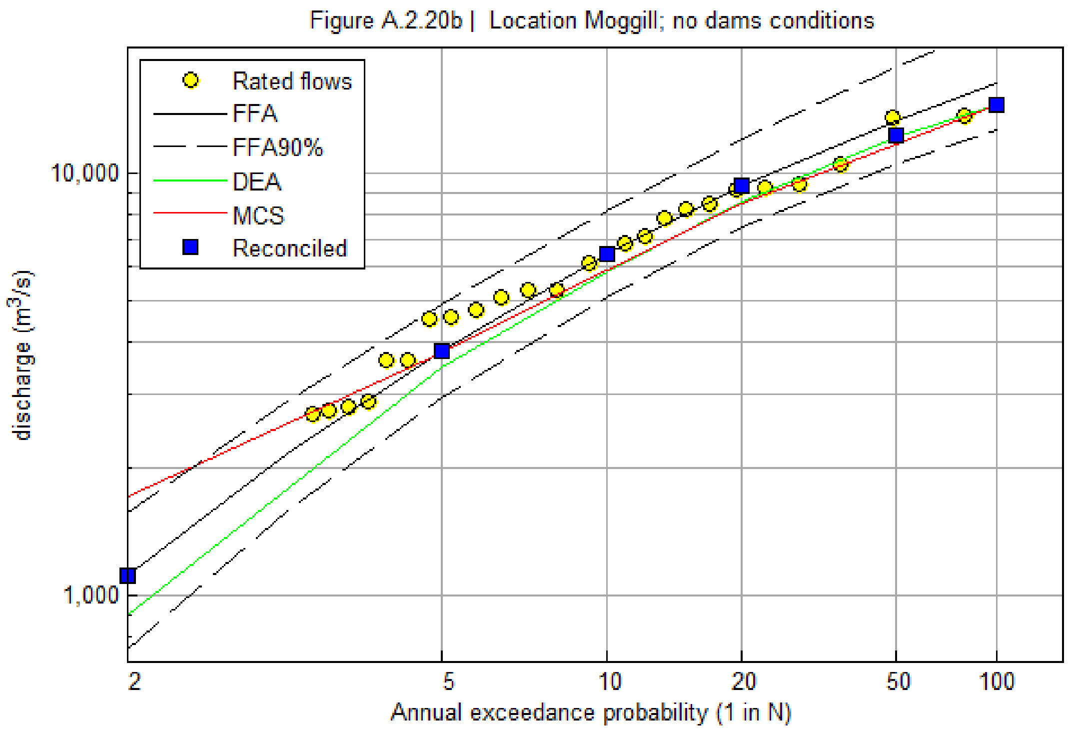


Figure A.2.20b | Location Moggill; no dams conditions
Rated flows
FFA
FFA90%
DEA
MCS
Reconciled
discharge (m3/s)
10,000
1,000
2
5
10
20
50
100
Annual exceedance probability (1 in N)

Figure A.2.21a | Location Centenary Bridge; no dams conditions

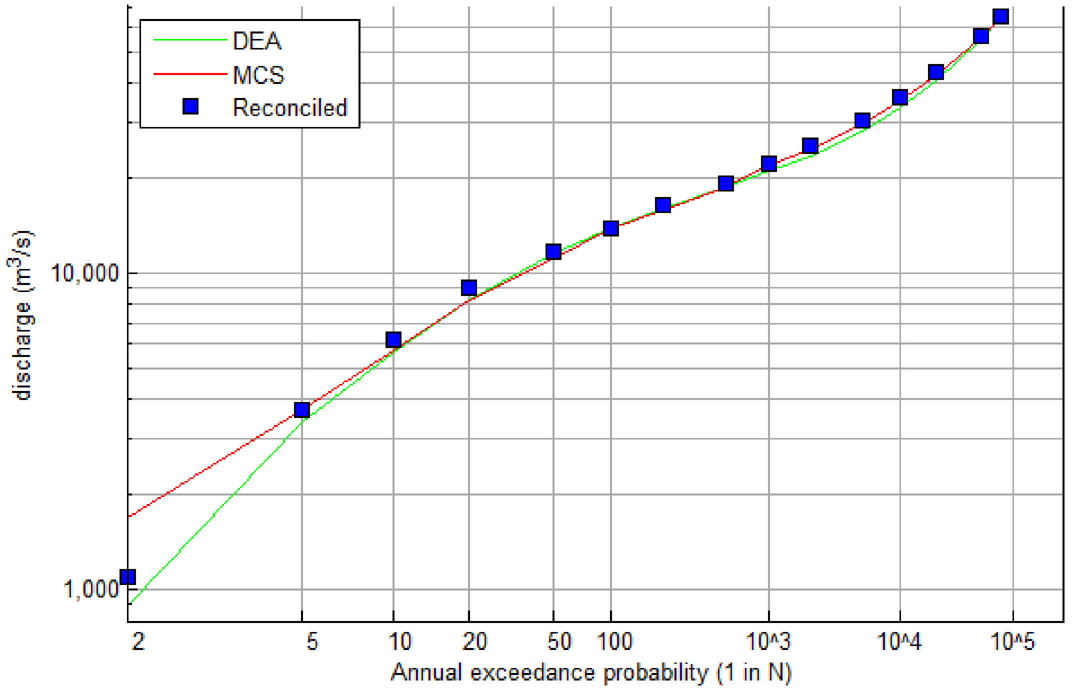


Figure A.2.21b | Location Centenary Bridge; no dams conditions

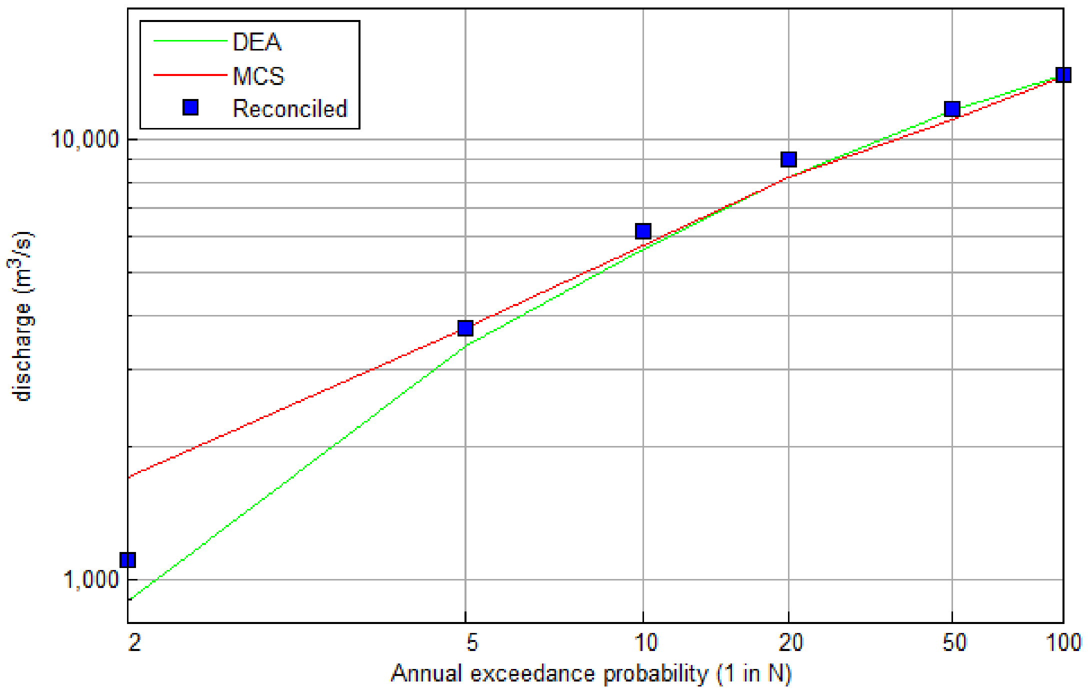


Figure A.2.22a | Location Brisbane; no dams conditions

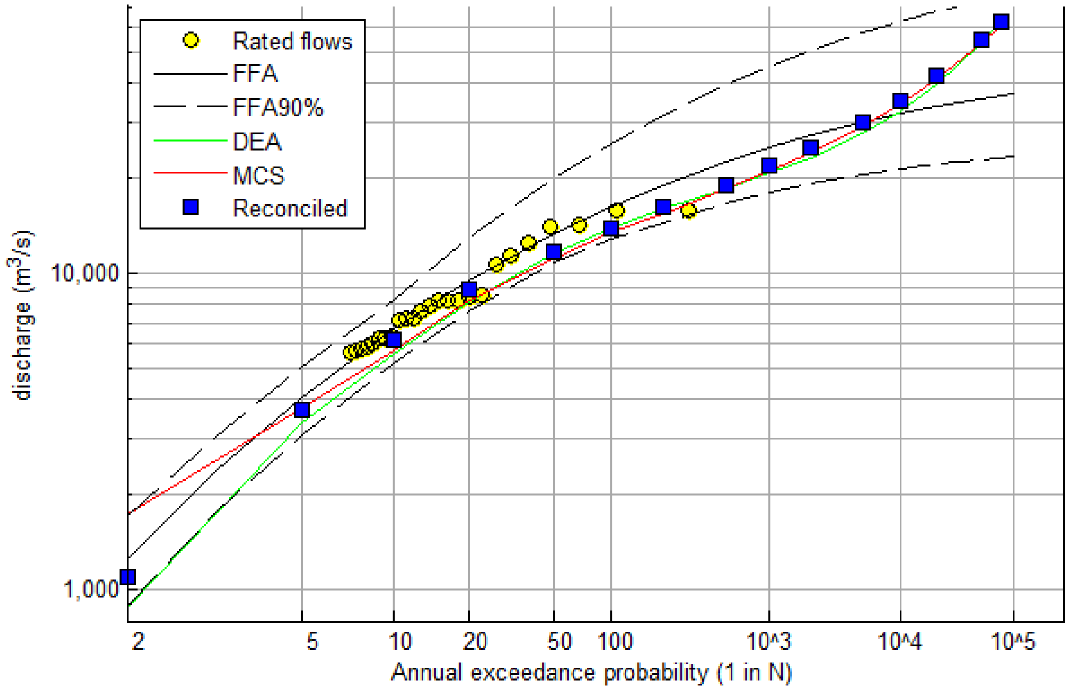
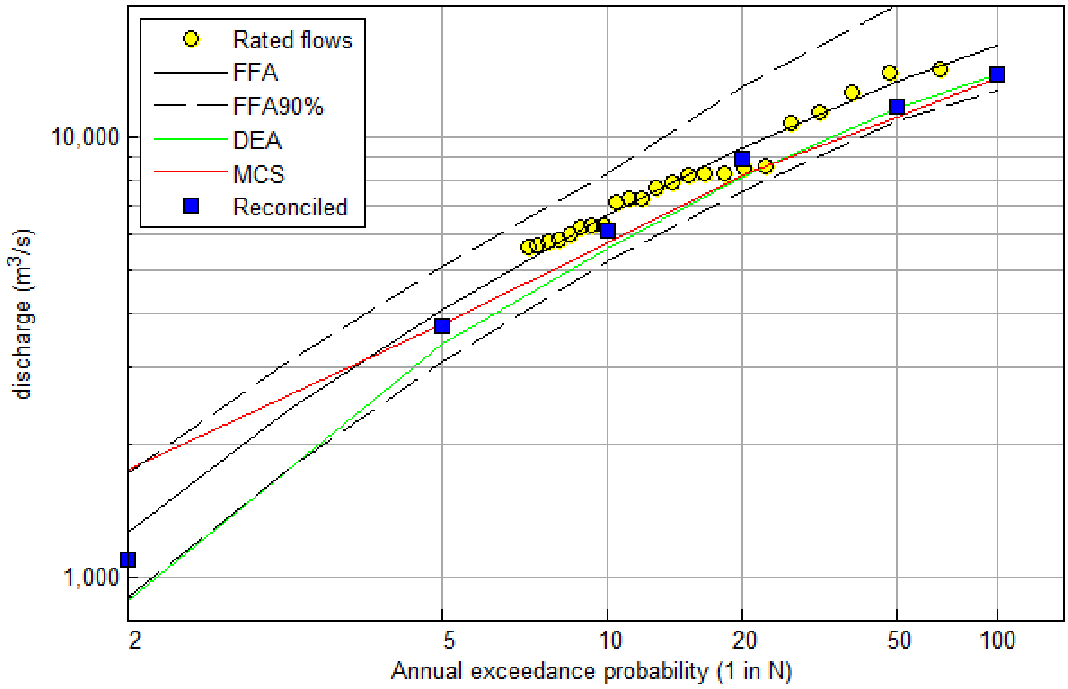


Figure A.2.22b | Location Brisbane; no dams conditions



A.3 Plots of catchment area versus peak discharge

This section provides Figures in which catchment area is compared with peak discharge. For a range of AEP's, the following four Figures are provided:

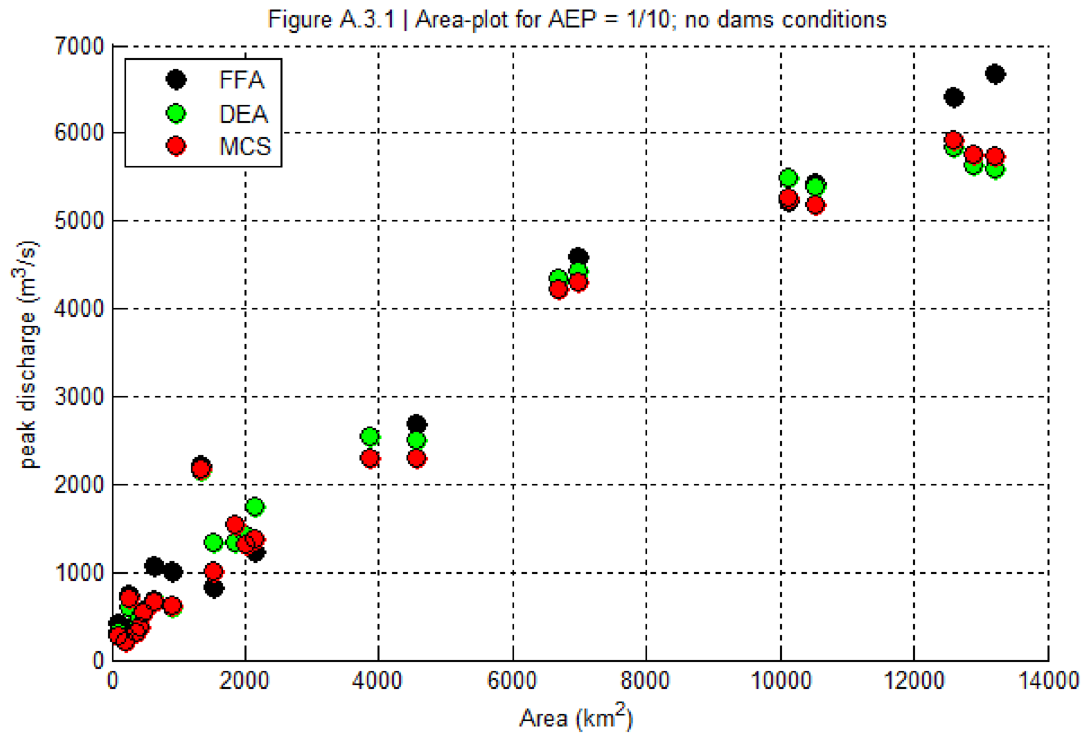
[1] Design peak flow (Q) versus catchment area (A) for FFA, DEA and MCS

[2] Reconciled design peak flow (Q) versus catchment area (A)

[3] Q/A versus A for FFA, DEA and MCS

[4] Reconciled Q/A versus A

Peak flow (Q) versus catchment area (A)



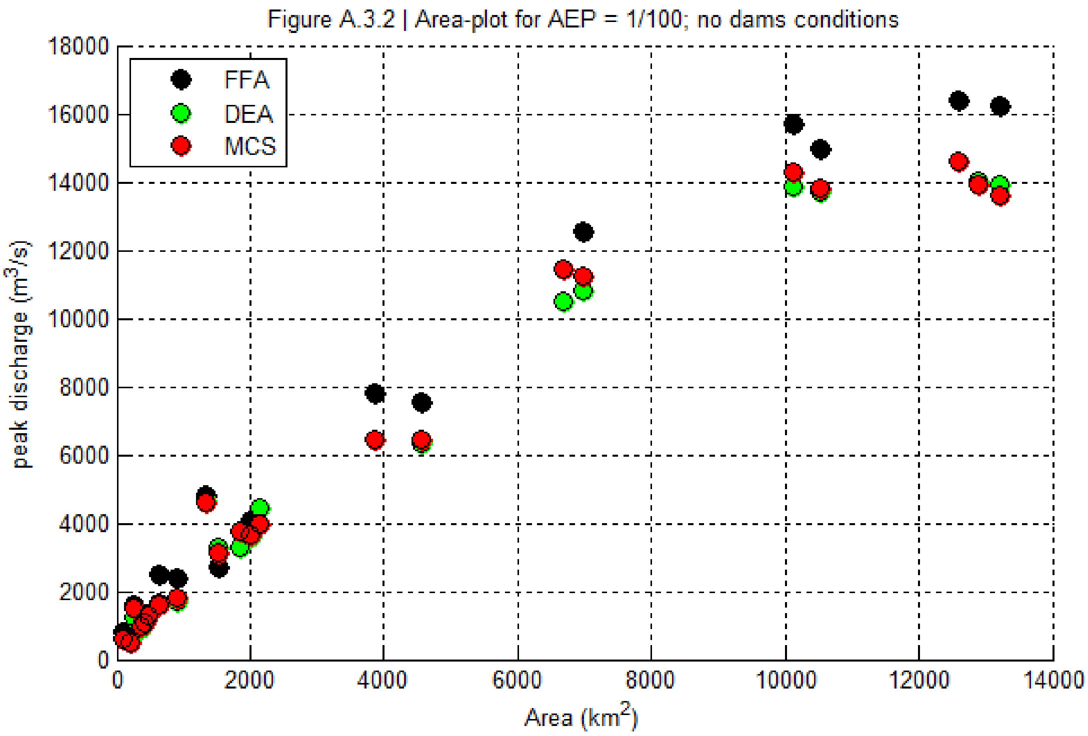


Figure A.3.3 | Area-plot for AEP = 1/1,000; no dams conditions

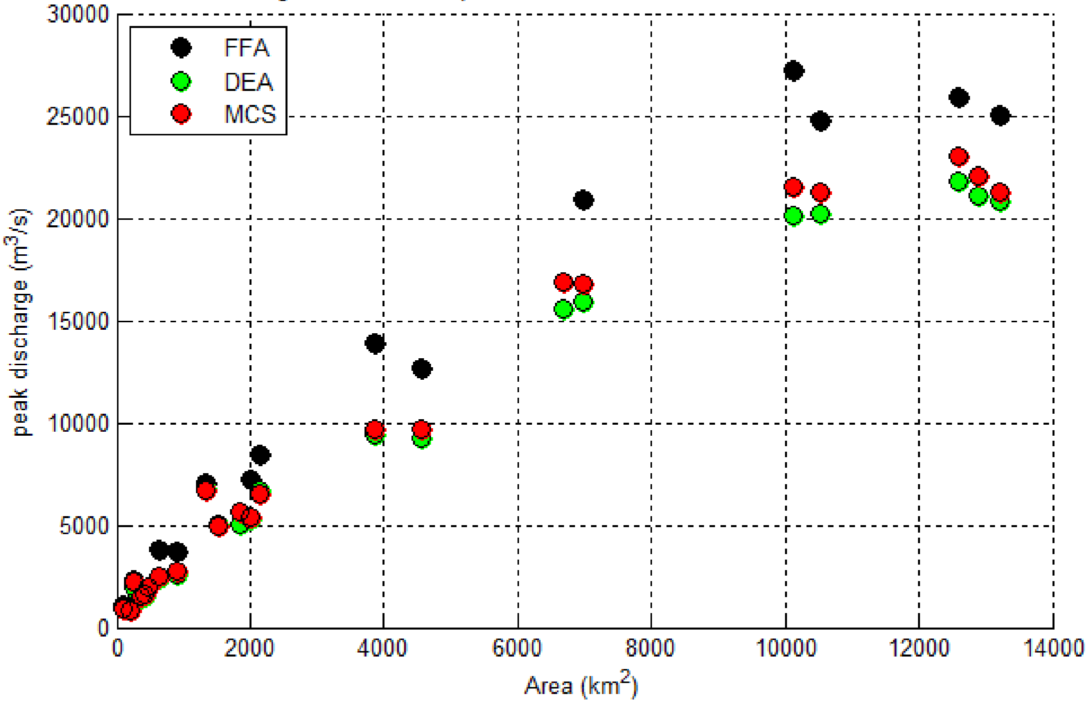


Figure A.3.4 | Area-plot for AEP = 1/10,000; no dams conditions

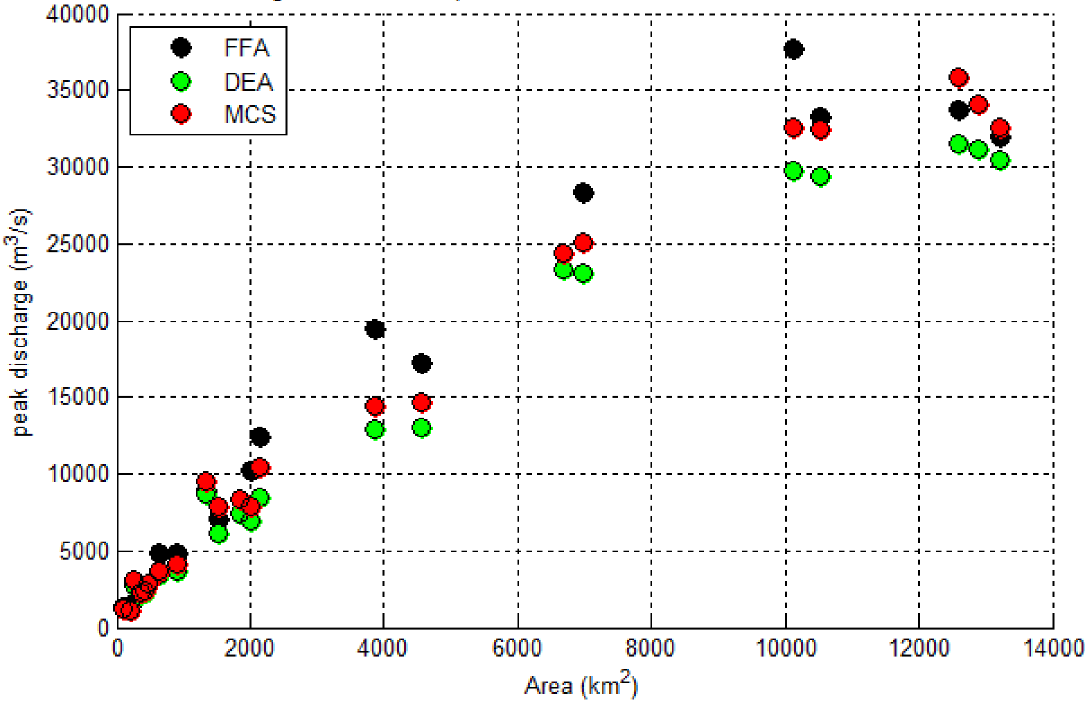


Figure A.3.5 | Area-plot for AEP = 1/100,000; no dams conditions

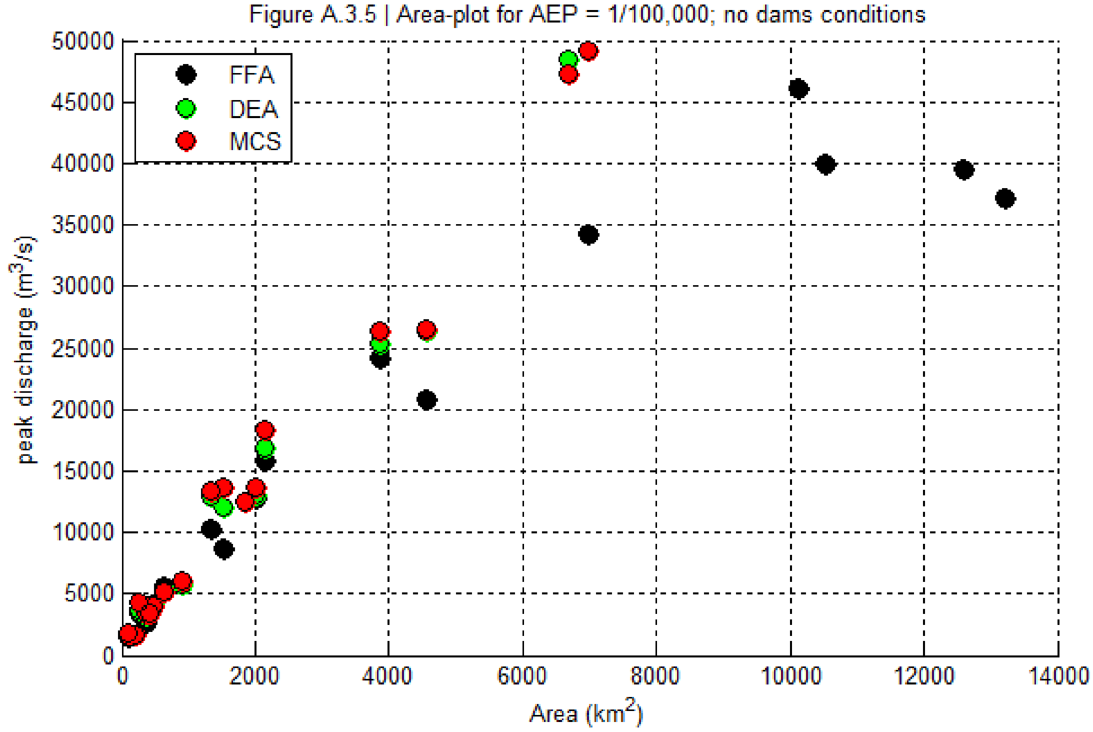


Figure A.3.6 | Area-plot for AEP = 1/10; no dams conditions

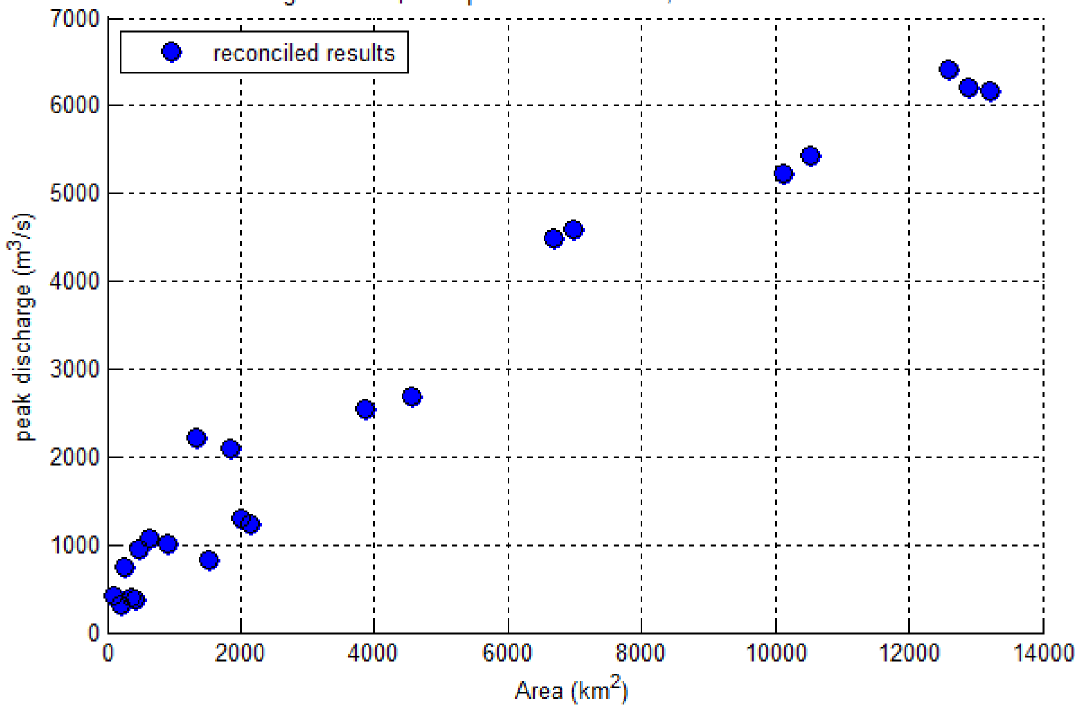


Figure A.3.7 | Area-plot for AEP = 1/100; no dams conditions

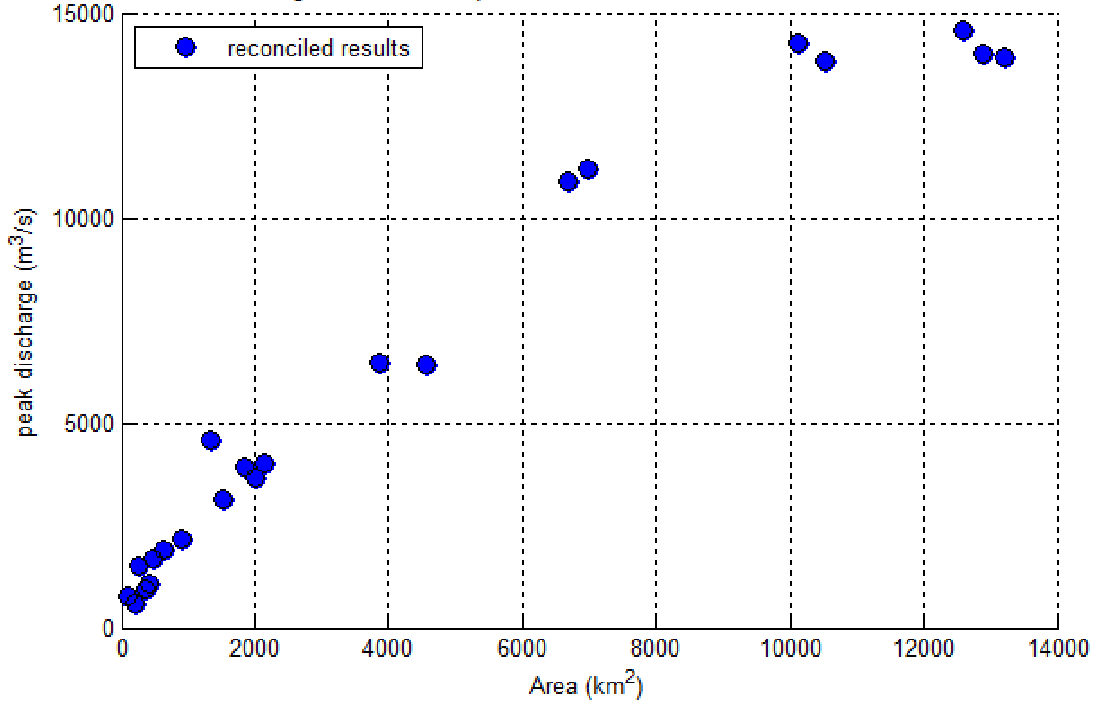


Figure A.3.8 | Area-plot for AEP = 1/1,000; no dams conditions

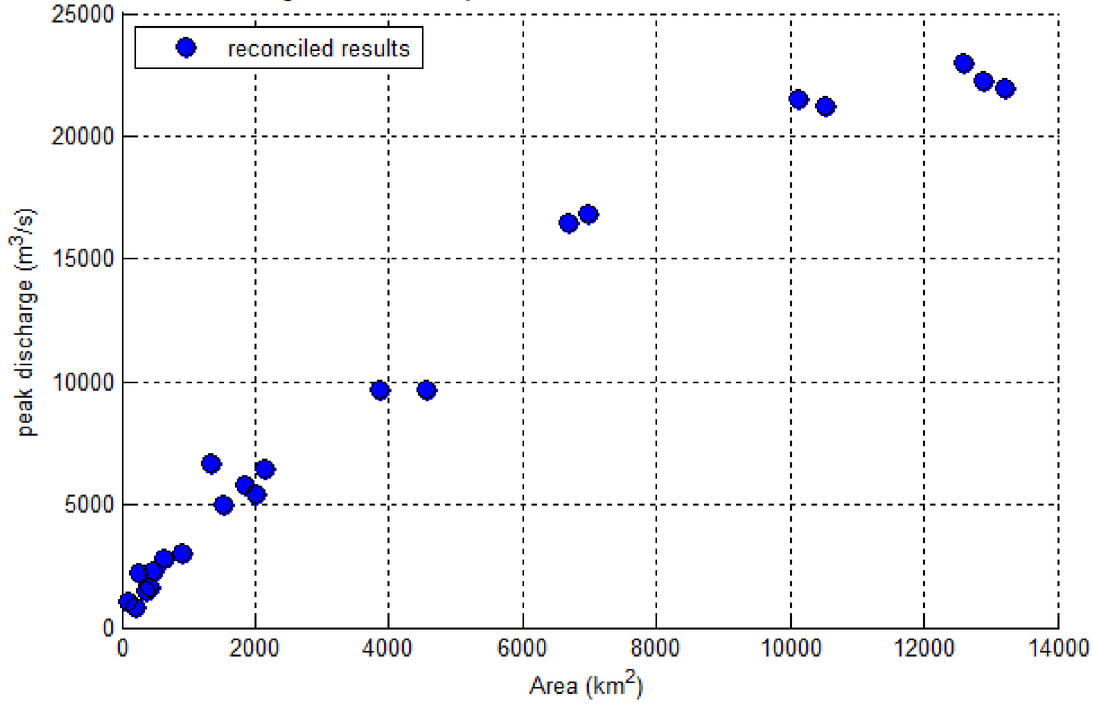


Figure A.3.9 | Area-plot for AEP = 1/10,000; no dams conditions

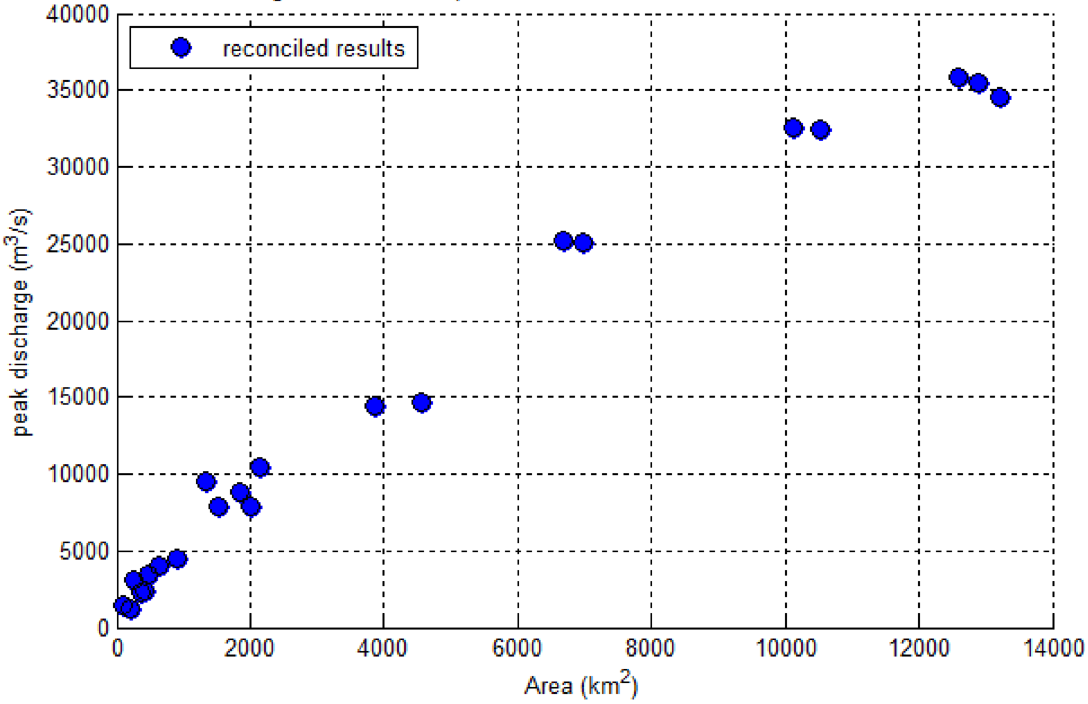
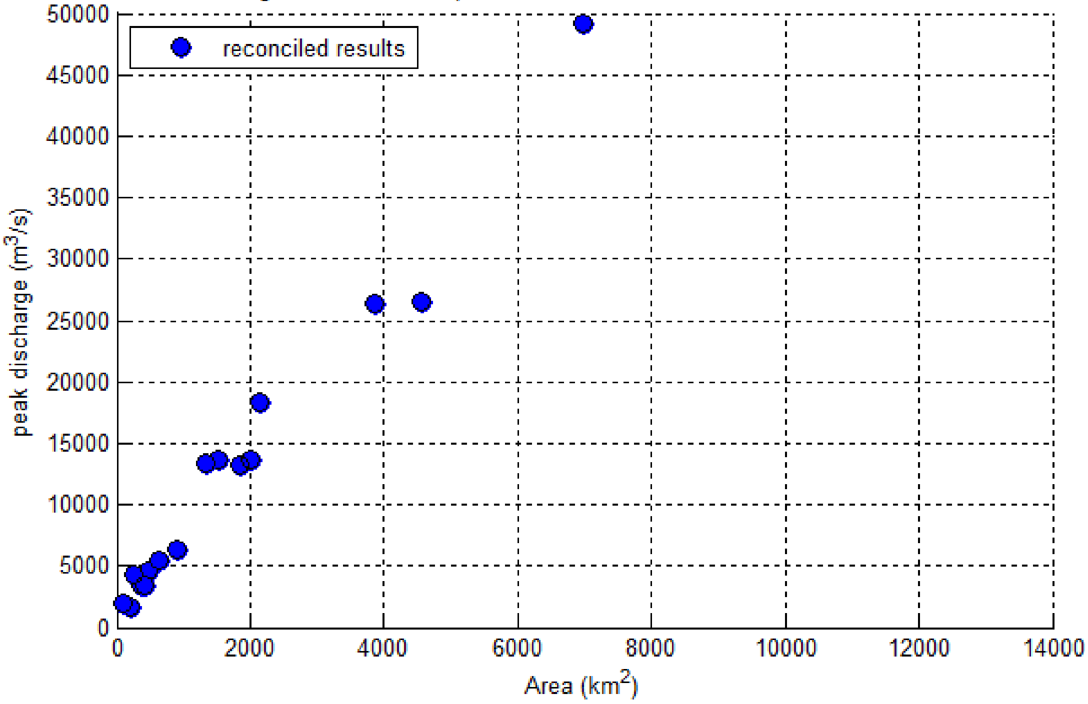


Figure A.3.10 | Area-plot for AEP = 1/100,000; no dams conditions



Q/A versus catchment area

Figure A.3.11 | Area-plot for AEP = 1/10; no dams conditions

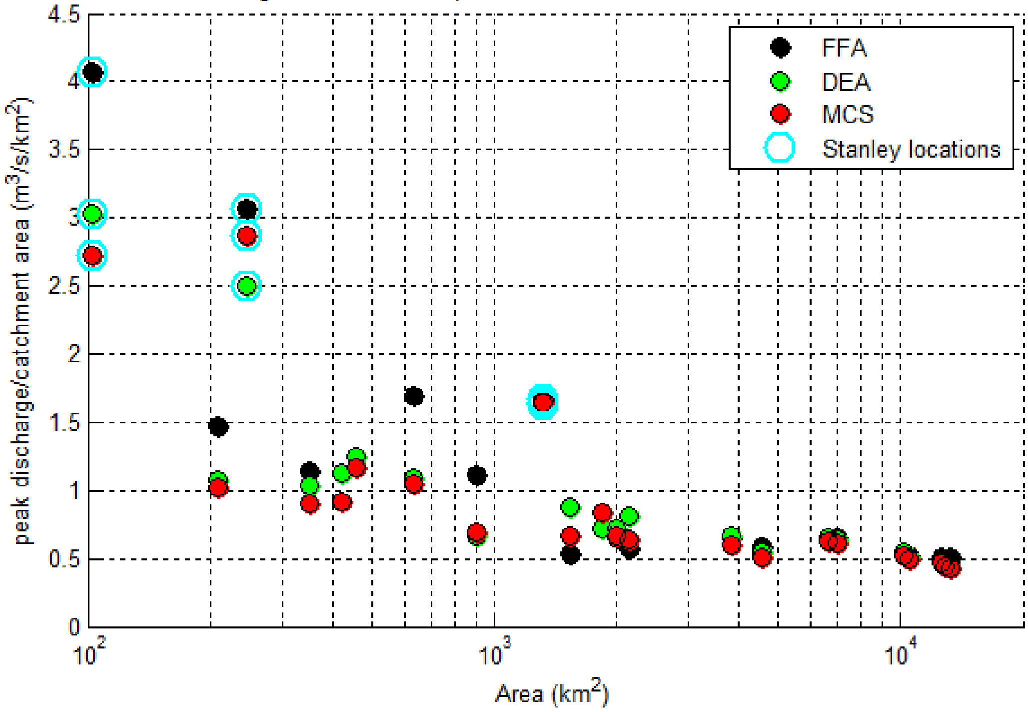
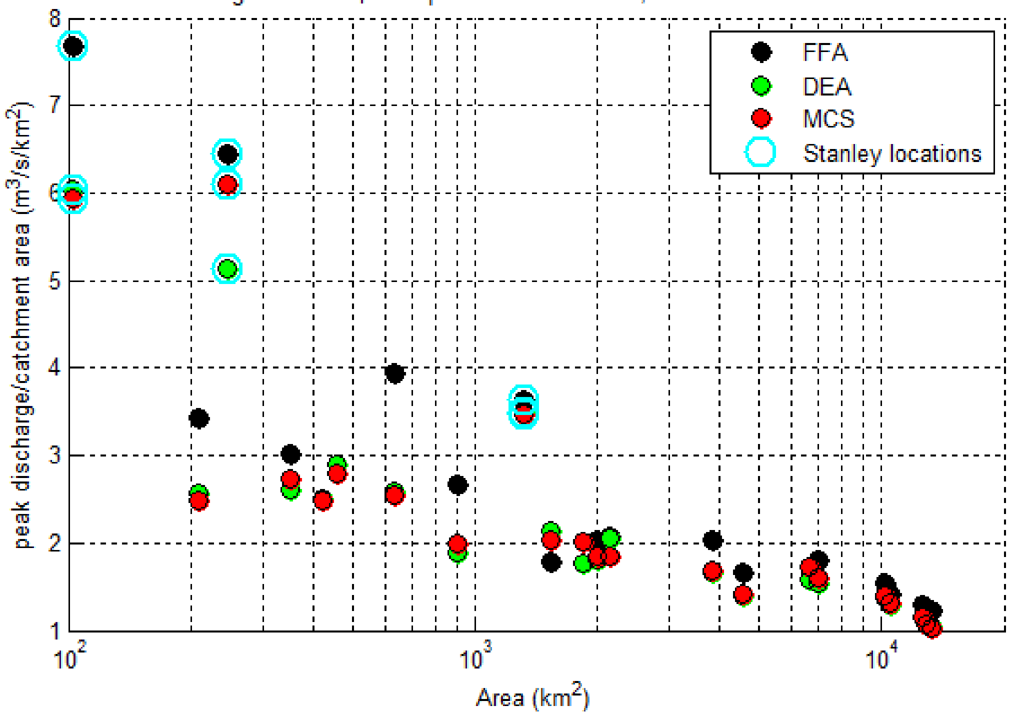


Figure A.3.12 | Area-plot for AEP = 1/100; no dams conditions



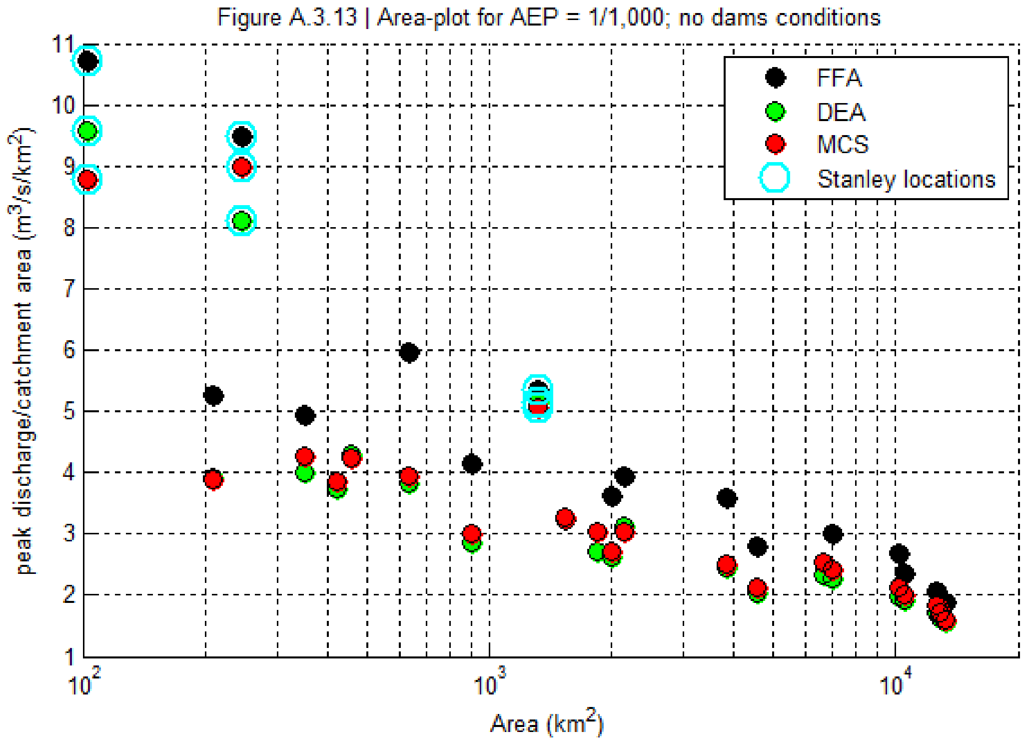


Figure A.3.13 | Area-plot for AEP = 1/1,000; no dams conditions
peak discharge/catchment area (m3/s/km2)
Area (km2)
FFA
DEA
MCS
Stanley locations

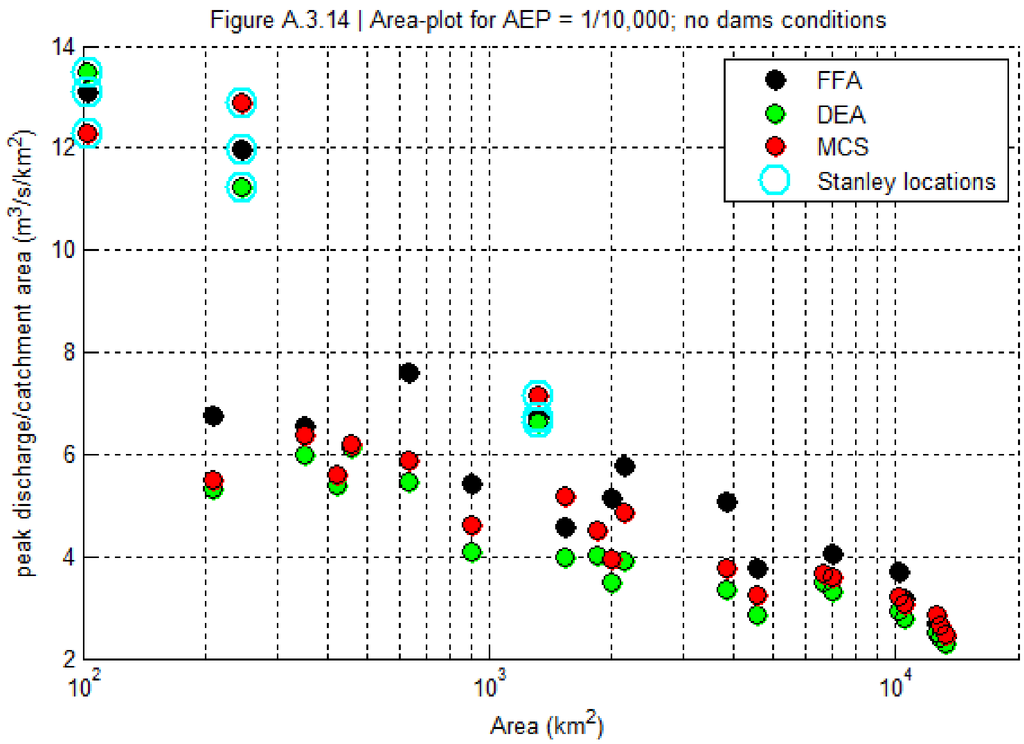


Figure A.3.14 | Area-plot for AEP = 1/10,000; no dams conditions
peak discharge/catchment area (m3/s/km2)
Area (km2)
FFA
DEA
MCS
Stanley locations

Figure A.3.15 | Area-plot for AEP = 1/100,000; no dams conditions

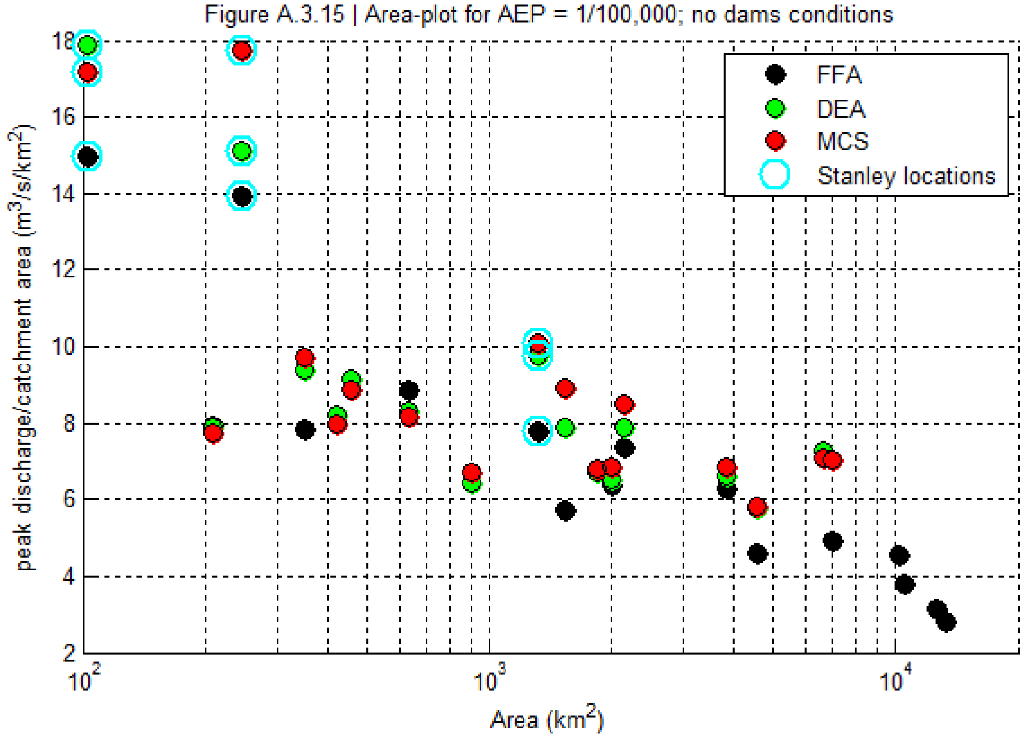


Figure A.3.16 | Area-plot for AEP = 1/10; no dams conditions

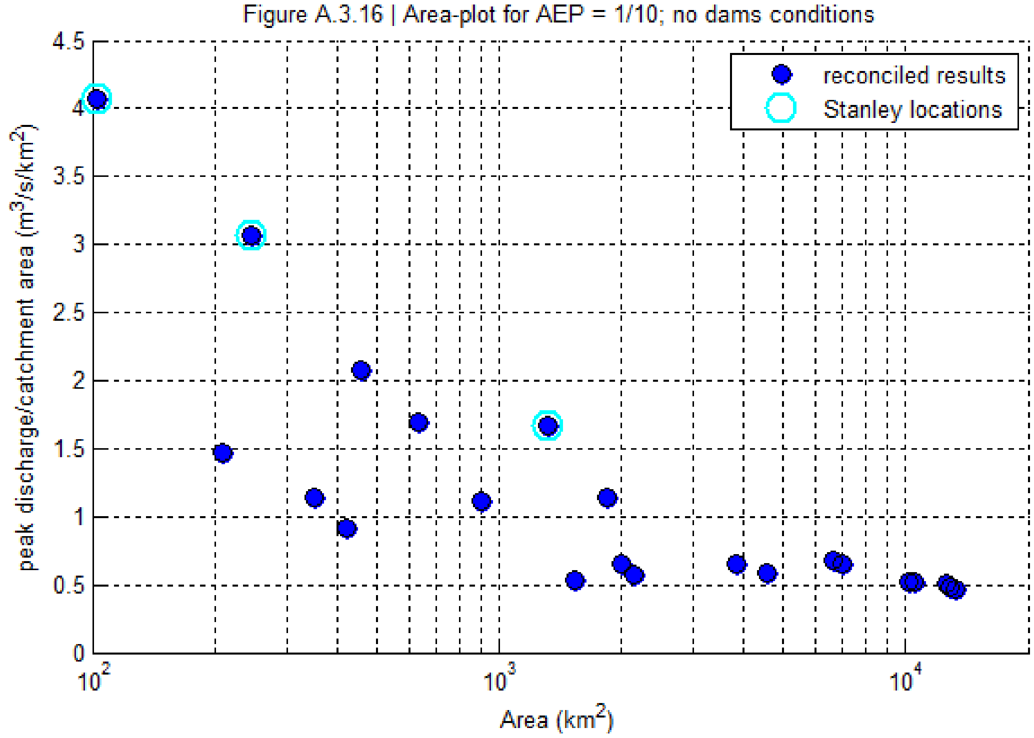


Figure A.3.17 | Area-plot for AEP = 1/100; no dams conditions

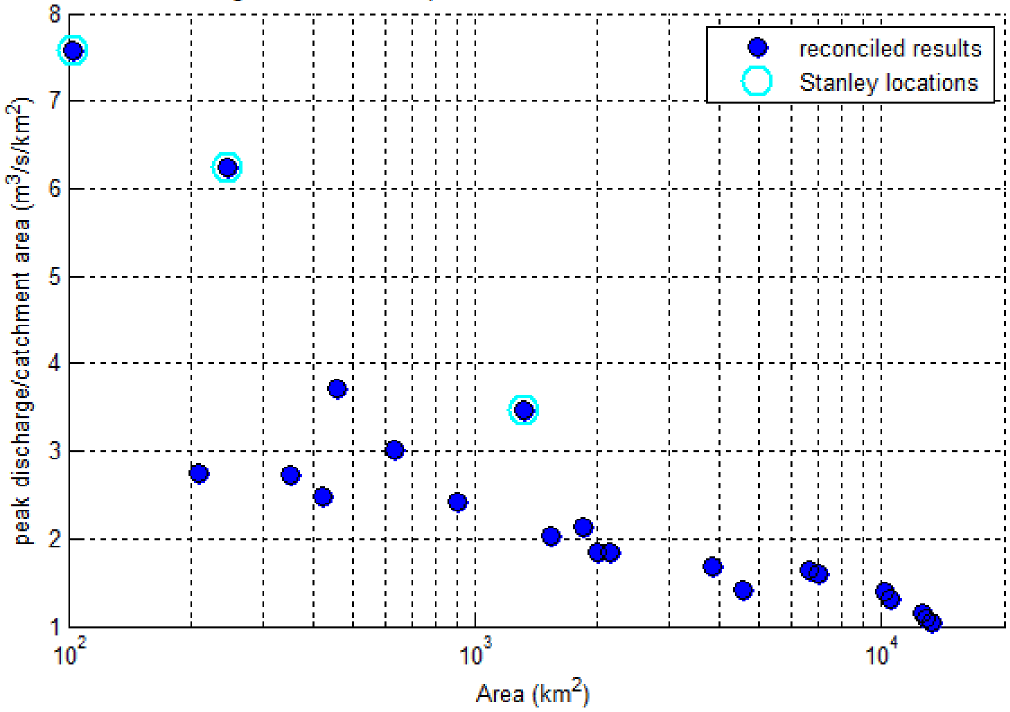


Figure A.3.18 | Area-plot for AEP = 1/1,000; no dams conditions

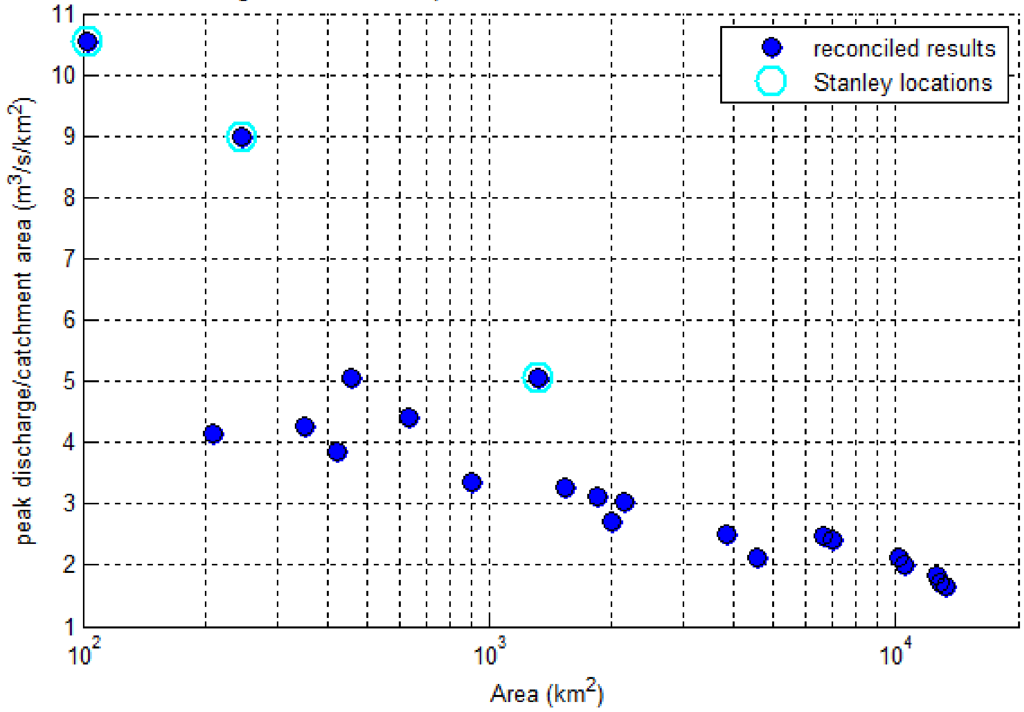


Figure A.3.19 | Area-plot for AEP = 1/10,000; no dams conditions

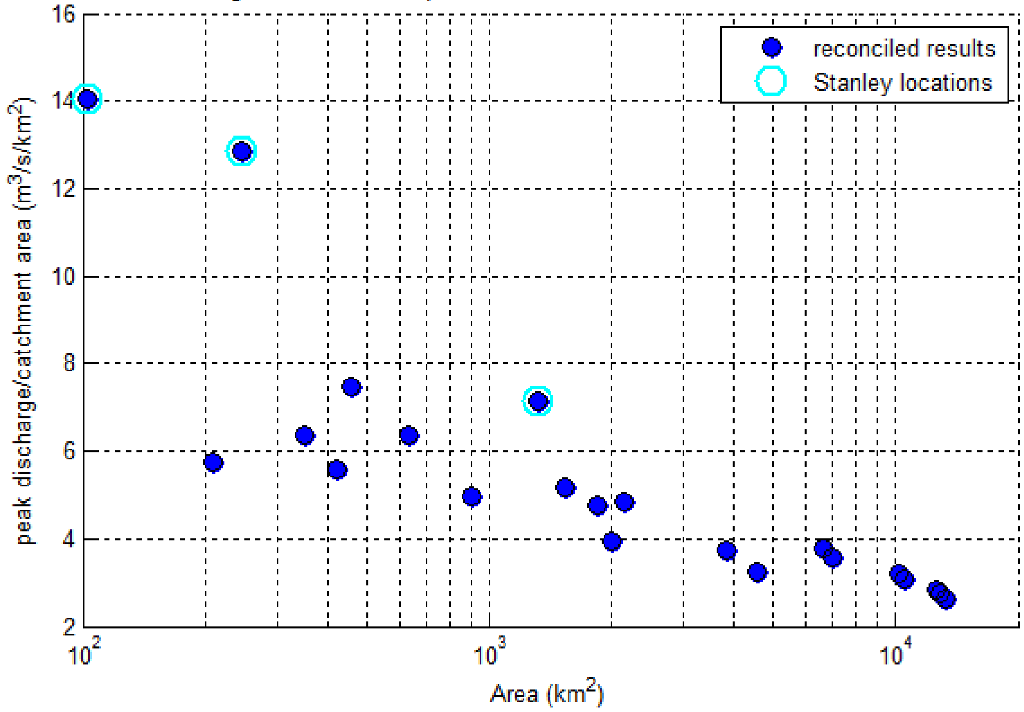
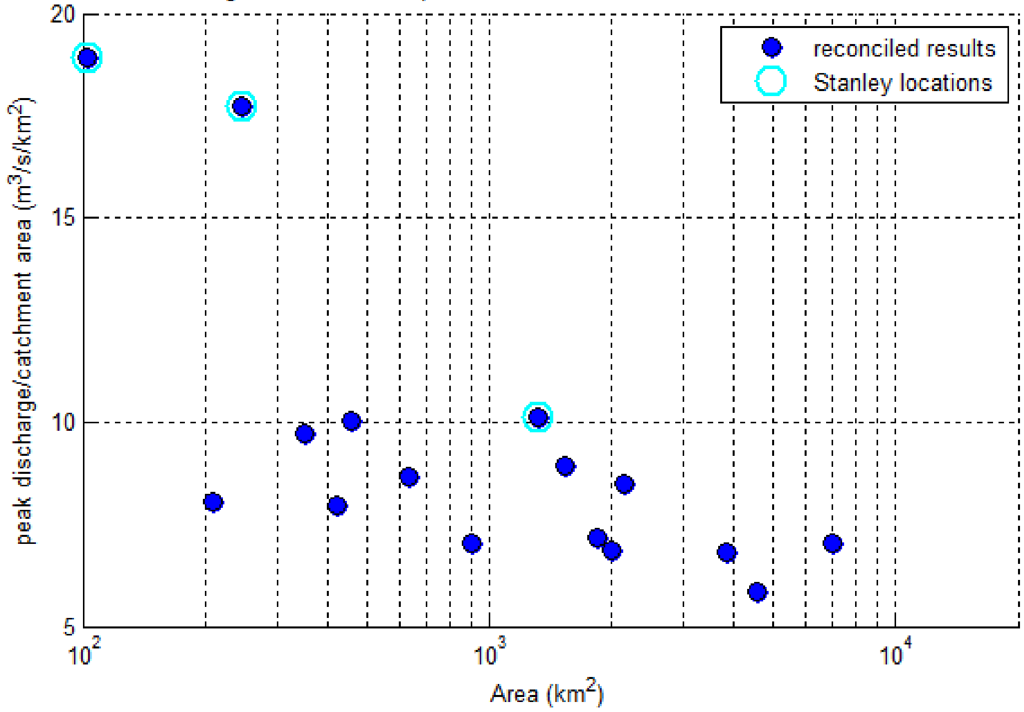


Figure A.3.20 | Area-plot for AEP = 1/100,000; no dams conditions



Appendix B
‘With-dams’ frequency tables and figures for peak flows

B.1 Tables

Wivenhoe peak outflows above 28,000 m^3/s are displayed in red, because the maximum outflow capacity of Wivenhoe dam is equal to 28,000 m^3/s. Estimates in excess of the maximum outlet capacity should be treated with caution, as the dam is likely to fail under such circumstances. 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.

Table B1 Peak flows (m^3/s) versus AEP; DEA results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Somerset Dam													
Wivenhoe	130	730	1,300	2,200	4,200	6,800	8,700	10,200	12,400	12,900	21,600		39,000
Savages Crossing	130	1,100	2,300	3,700	6,400	8,700	11,700	15,000	17,700	18,800	29,000		64,900
Mount Crosby	130	1,100	2,400	3,800	6,600	8,800	11,600	14,800	17,300	19,000	27,100		63,300
Ipswich	380	910	1,300	1,800	2,500	3,100	3,600	4,200	4,700	5,200	6,700	11,300	17,300
Moggill	420	1,500	3,100	4,900	7,900	10,100	12,500	15,700	17,700	20,100	28,200		61,800
Centenary Bridge	430	1,500	3,000	4,900	7,700	9,800	12,000	15,000	17,300	19,500	27,400		59,900
Brisbane	470	1,500	3,100	4,900	7,700	9,800	12,100	14,900	17,200	19,400	26,500		57,700

Table B2 Peak flows (m^3/s) versus AEP; MCS results

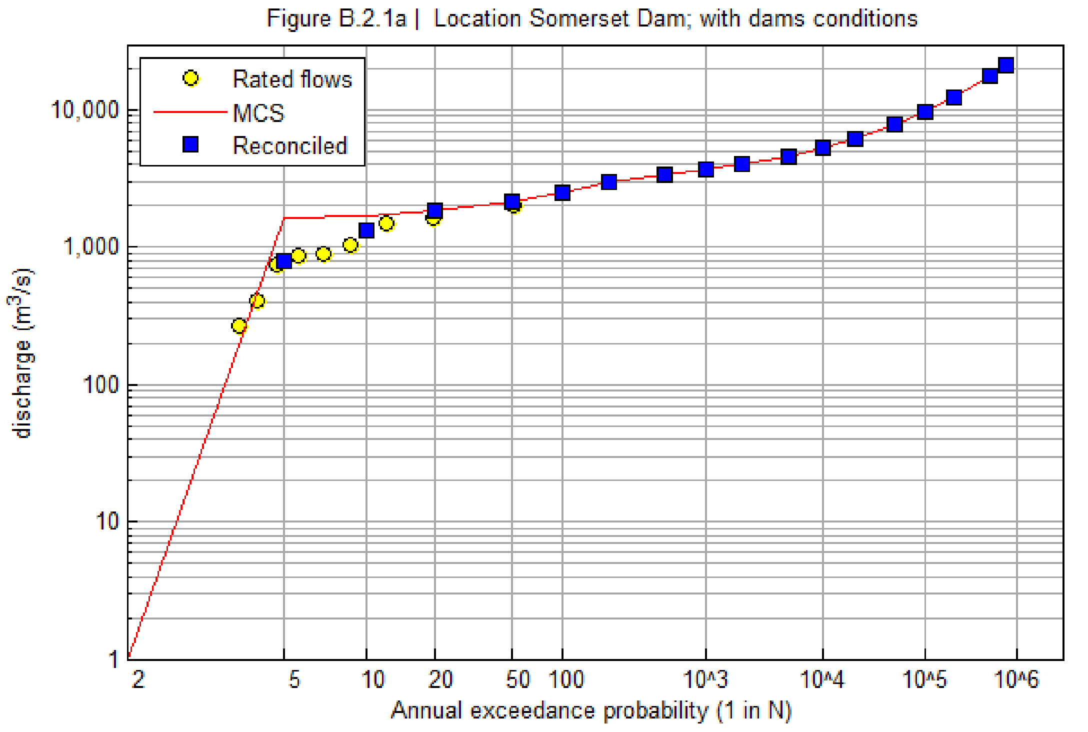
Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Somerset Dam	0	1,700	1,700	1,800	2,200	2,500	3,000	3,300	3,600	4,000	5,400	10,700	20,900
Wivenhoe	0	470	930	1,700	3,300	6,300	8,800	10,300	12,500	12,900	21,200	35,800	43,700
Savages Crossing	300	1,200	2,200	3,500	6,100	9,000	12,000	15,500	17,500	20,400	28,000		57,200
Mount Crosby	360	1,200	2,200	3,600	6,000	8,600	11,700	14,800	17,100	19,700	27,200		55,500
Ipswich	450	960	1,400	1,900	2,700	3,400	3,900	4,800	5,400	5,900	7,900	11,500	16,400

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Moggill	850	2,100	3,300	4,800	7,300	10,200	12,400	15,700	18,000	20,400	29,300		57,600
Centenary Bridge	880	2,200	3,300	4,800	7,200	9,700	11,900	14,600	16,800	19,500	28,100		56,700
Brisbane	930	2,300	3,500	4,900	7,600	9,700	11,700	14,400	16,300	19,100	26,900		54,800

Table B3 Peak flows (m^3/s) versus AEP; Reconciled results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	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
Wivenhoe	0	470	930	1,700	3,300	6,300	8,800	10,300	12,500	12,900	21,200	35,800	43,700
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
Mount Crosby	200	1,300	2,200	3,600	6,000	8,600	11,700	14,800	17,100	19,700	27,200		55,500
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
Moggill	630	2,100	3,300	4,800	7,300	10,200	12,400	15,700	18,000	20,400	29,300		57,600
Centenary Bridge	640	2,100	3,300	4,800	7,100	9,900	11,900	15,000	17,700	19,900	28,500		55,900
Brisbane	700	2,200	3,300	4,800	7,100	9,900	12,000	14,900	17,500	19,700	27,600		53,800

B.2 Frequency plots



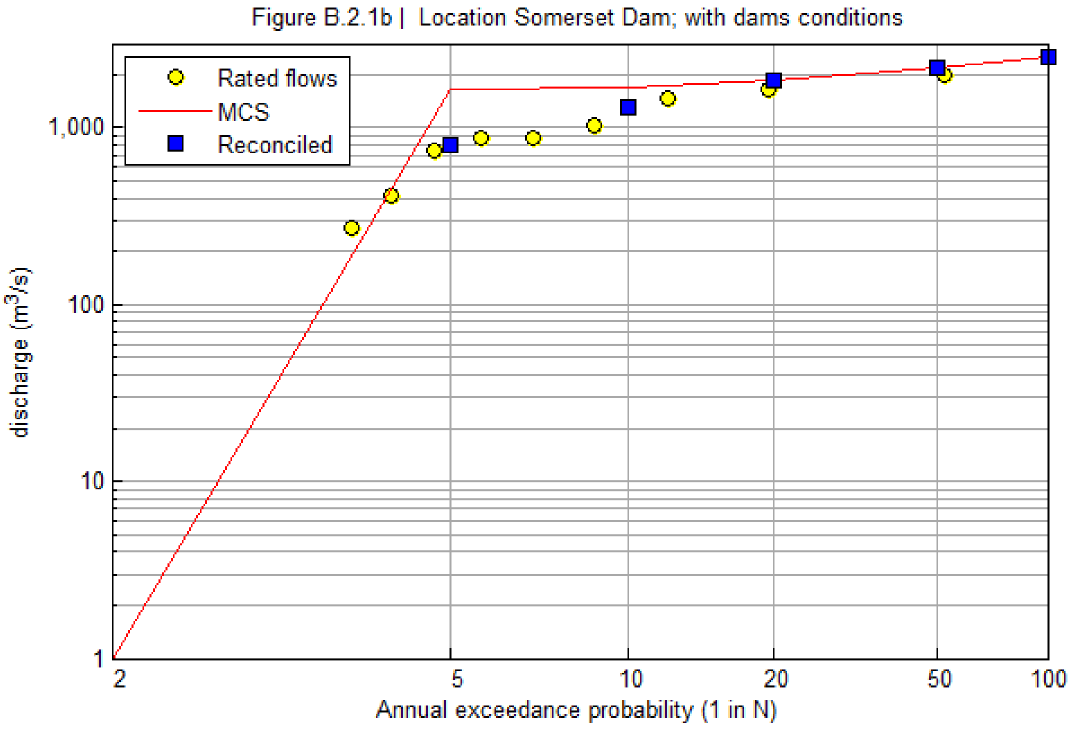


Figure B.2.2a | Location Wivenhoe; with dams conditions

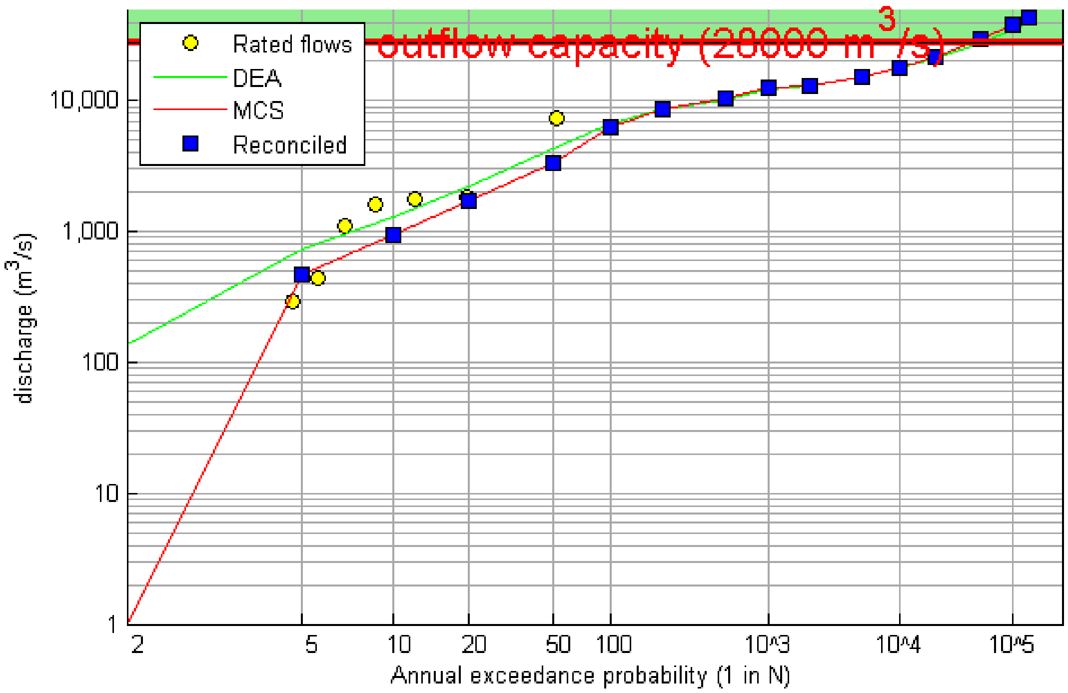


Figure B.2.2b | Location Wivenhoe; with dams conditions

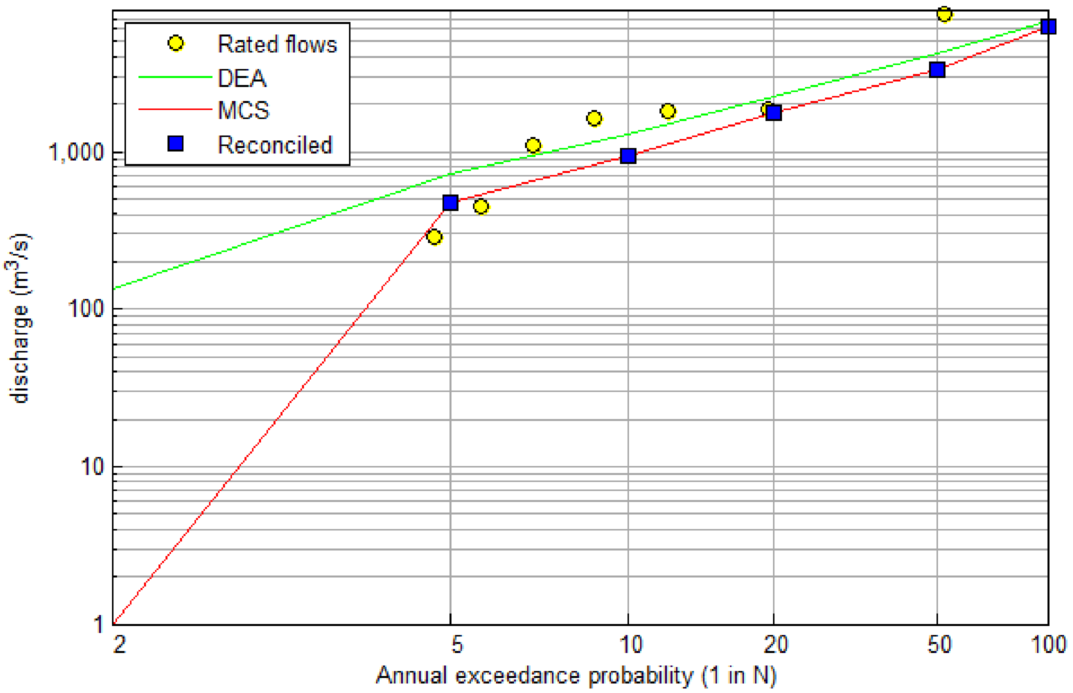


Figure B.2.3a | Location Savages Crossing; with dams conditions

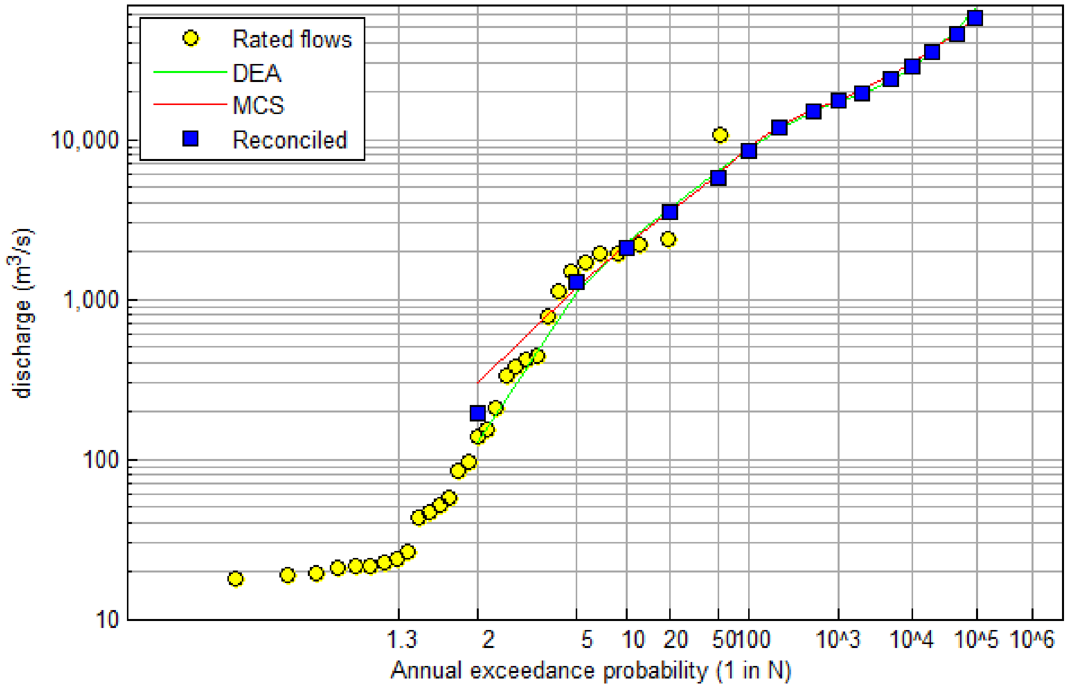


Figure B.2.3b | Location Savages Crossing; with dams conditions

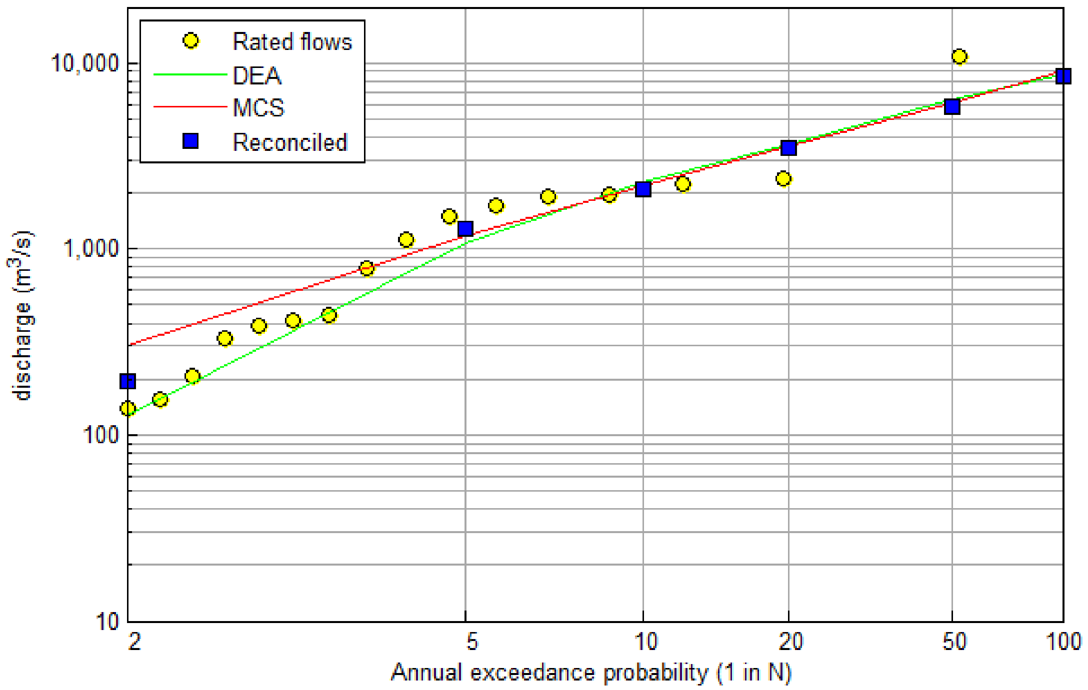


Figure B.2.4a | Location Mount Crosby; with dams conditions

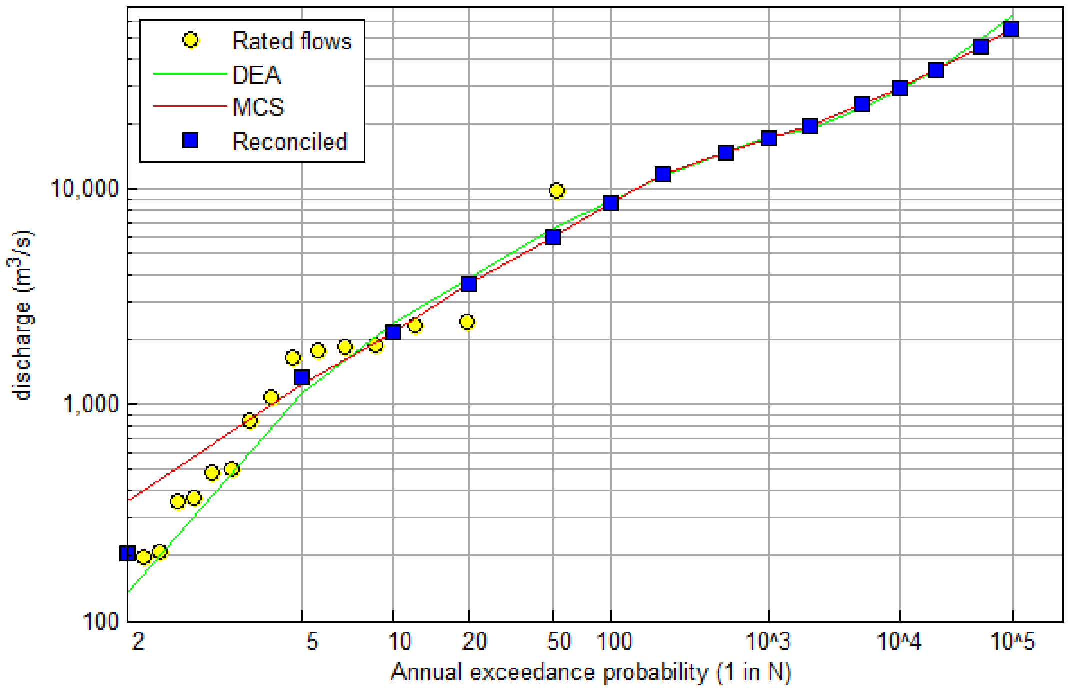


Figure B.2.4b | Location Mount Crosby; with dams conditions

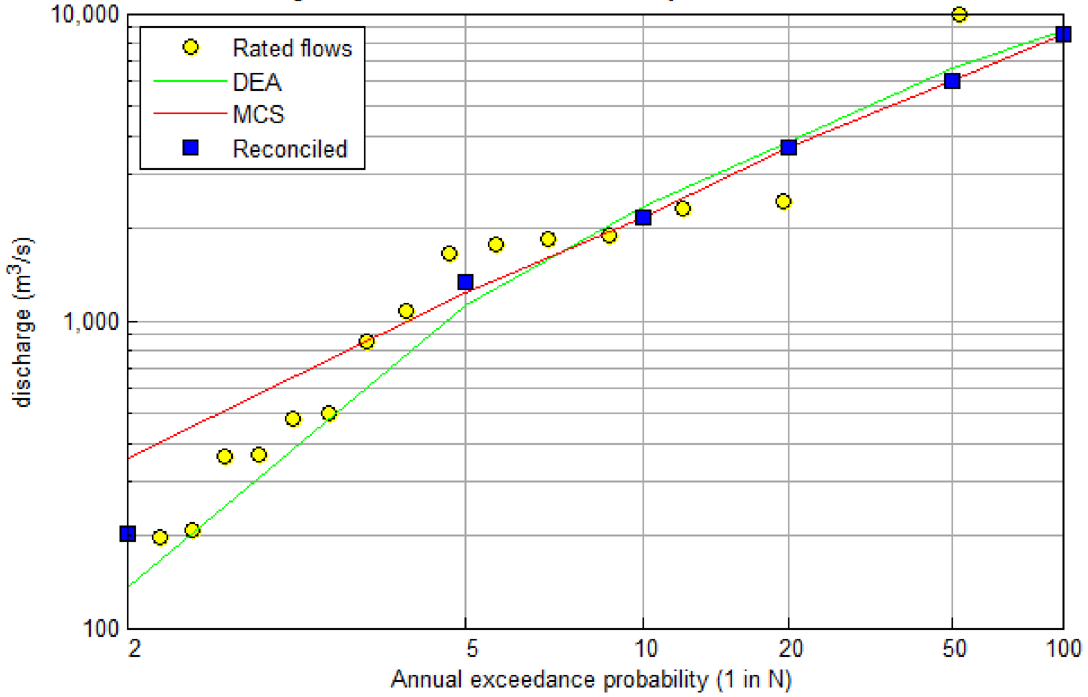


Figure B.2.5a | Location Ipswich; with dams conditions

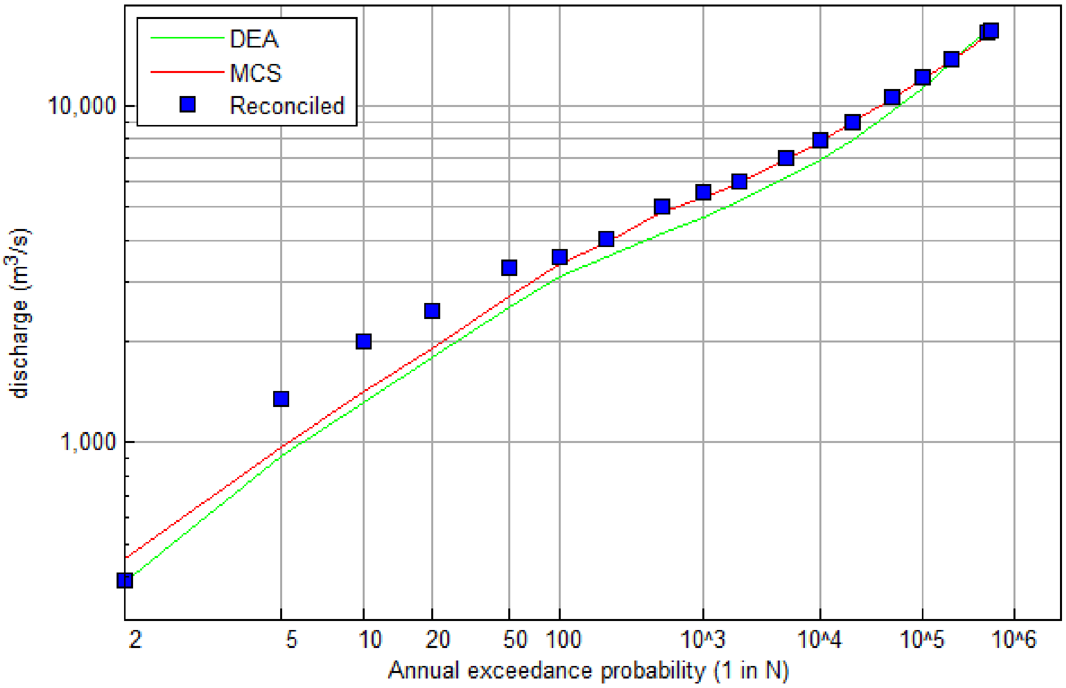


Figure B.2.5b | Location Ipswich; with dams conditions

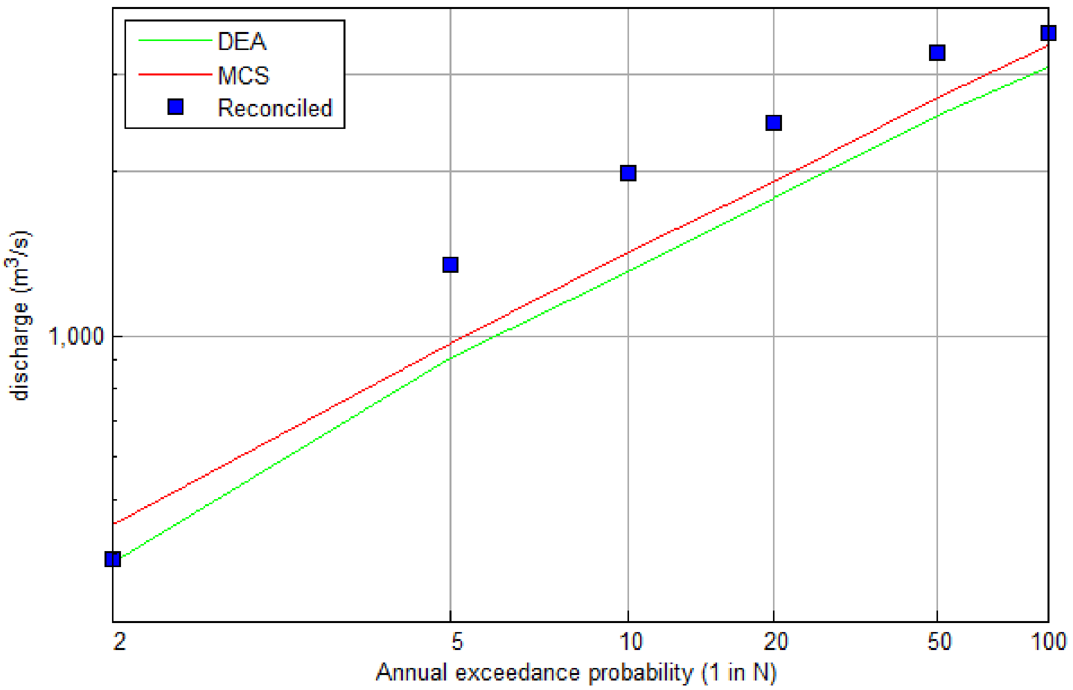


Figure B.2.6a | Location Moggill; with dams conditions

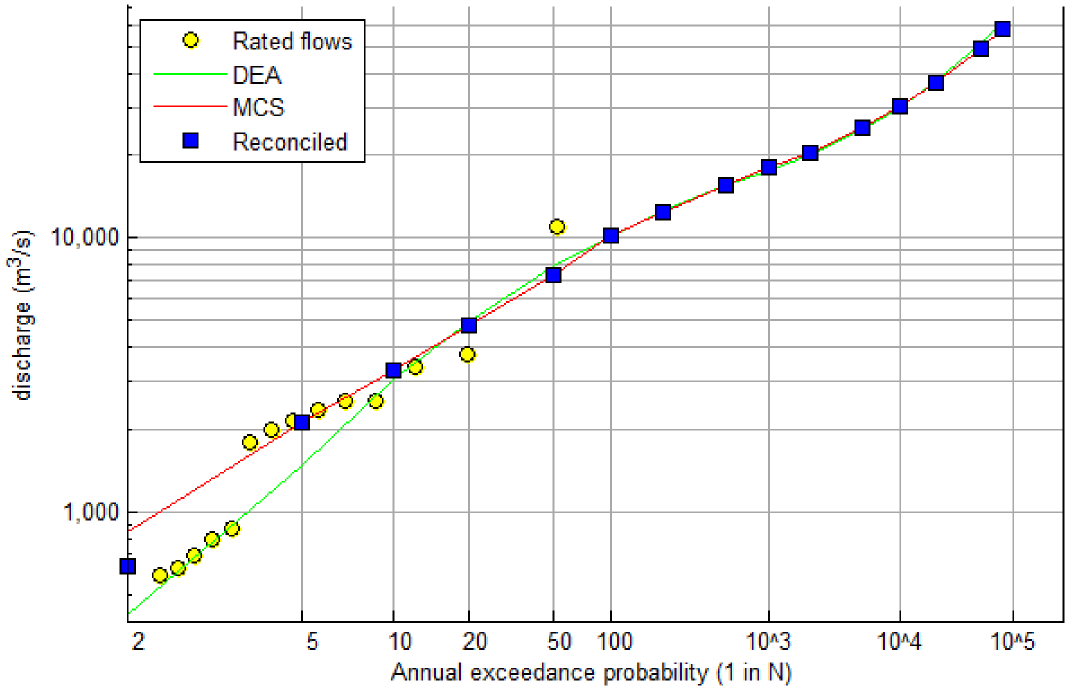


Figure B.2.6b | Location Moggill; with dams conditions

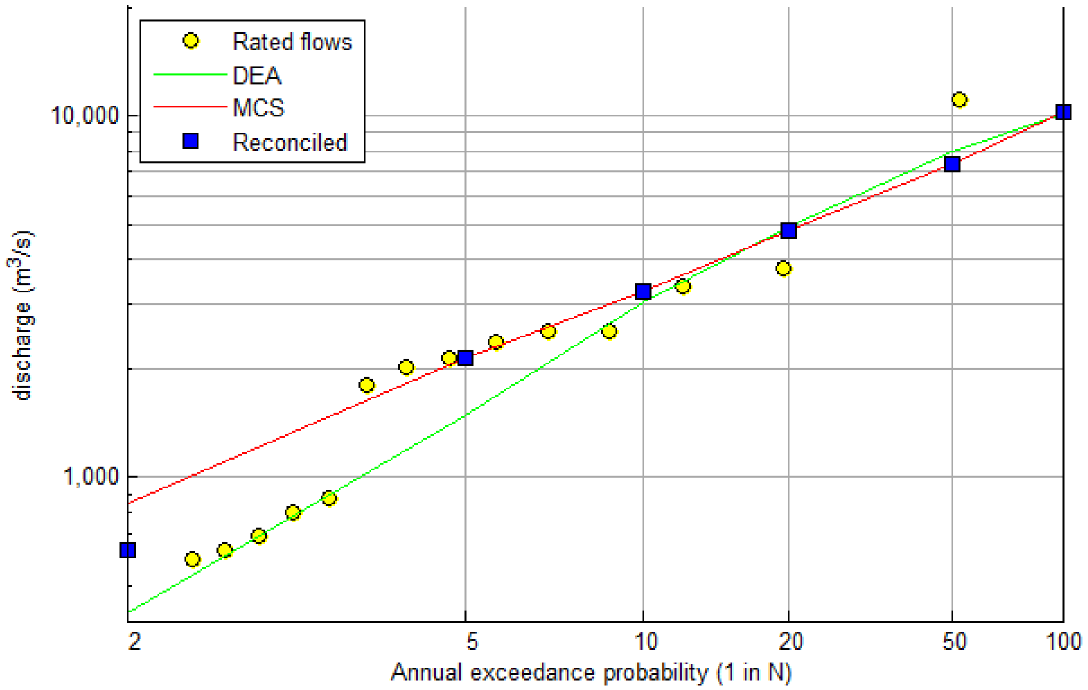


Figure B.2.7a | Location Centenary Bridge; with dams conditions

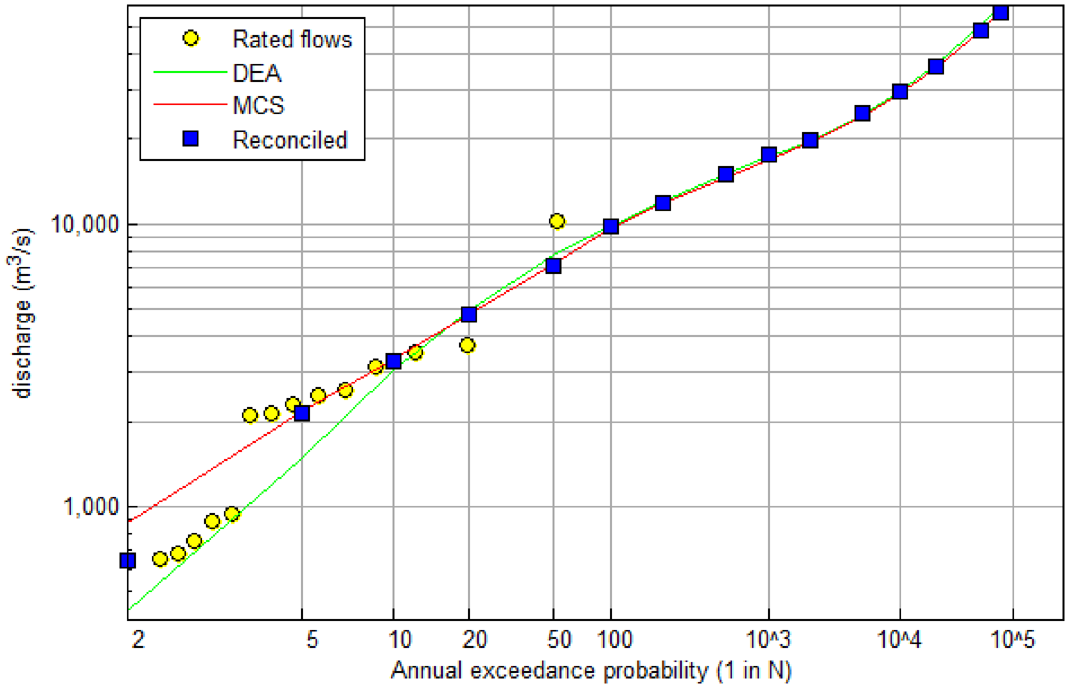


Figure B.2.7b | Location Centenary Bridge; with dams conditions

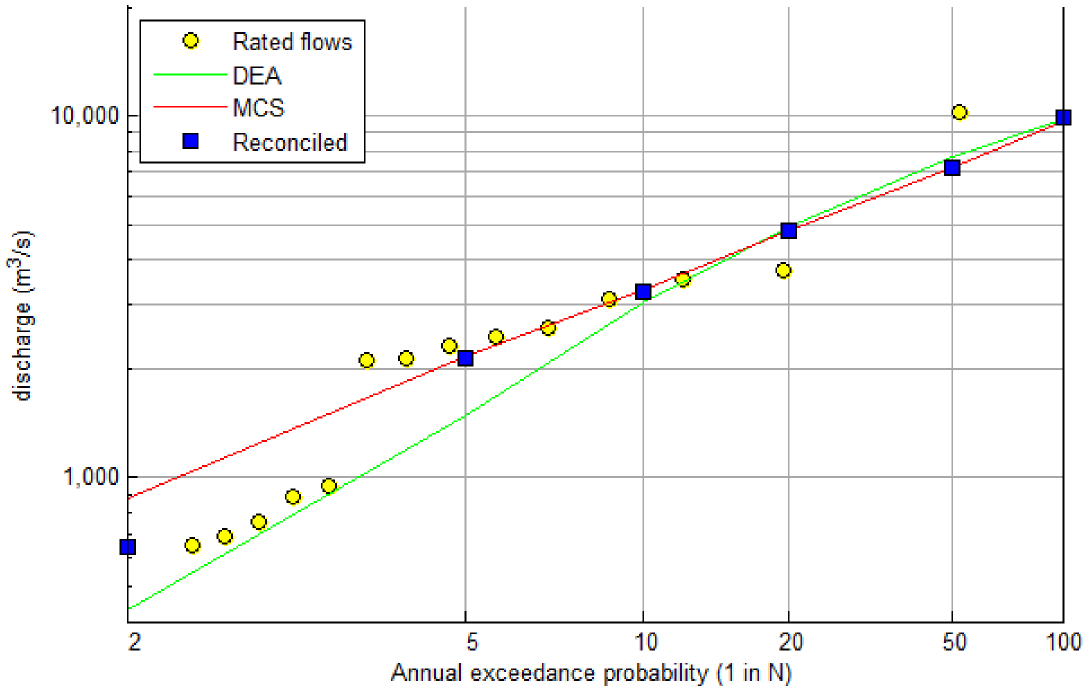


Figure B.2.8a | Location Brisbane; with dams conditions

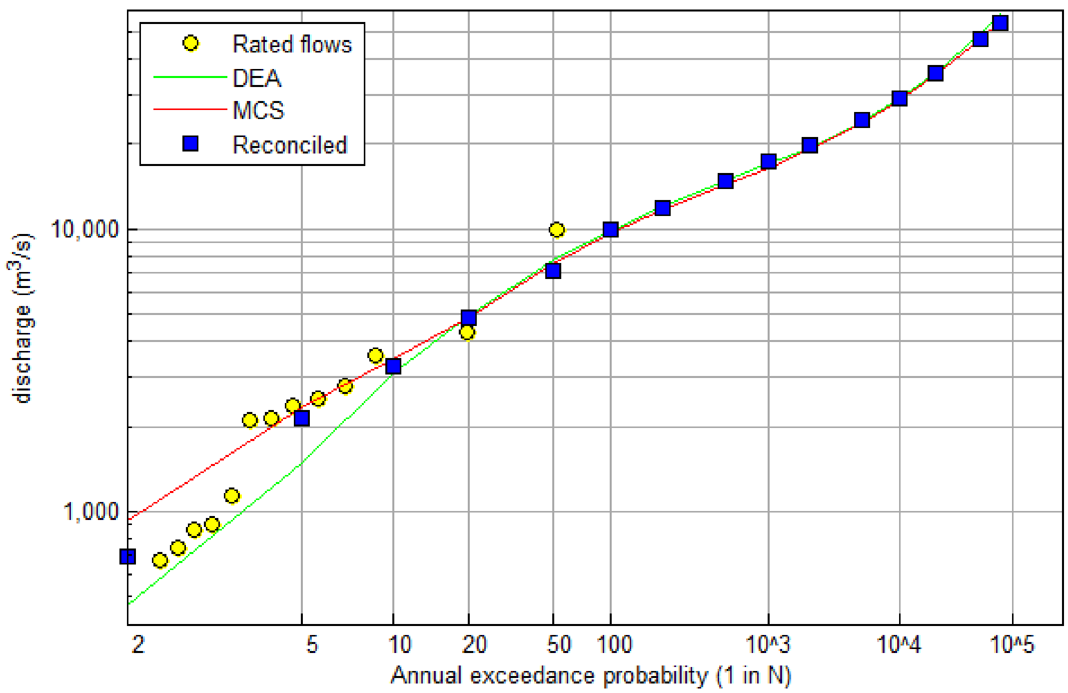
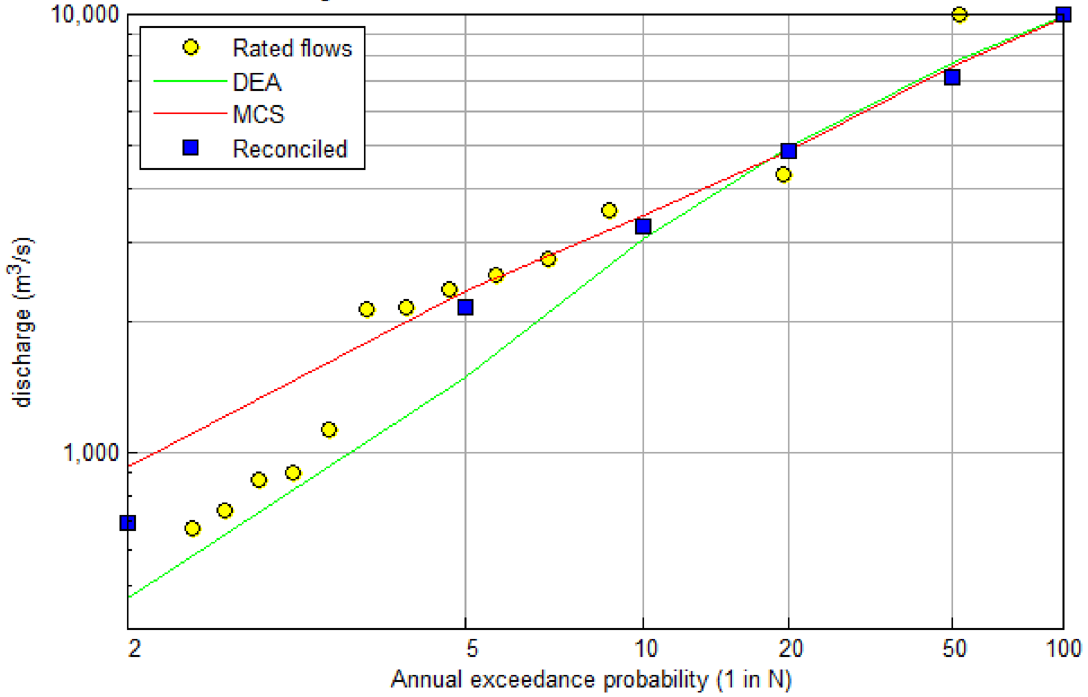
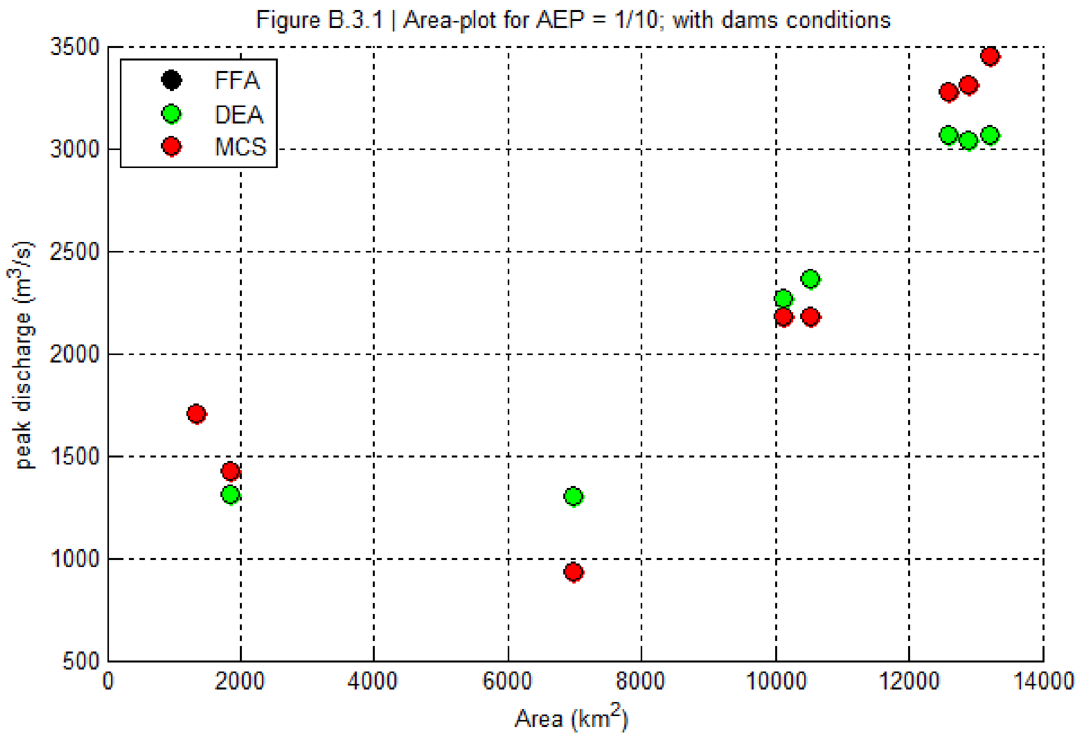


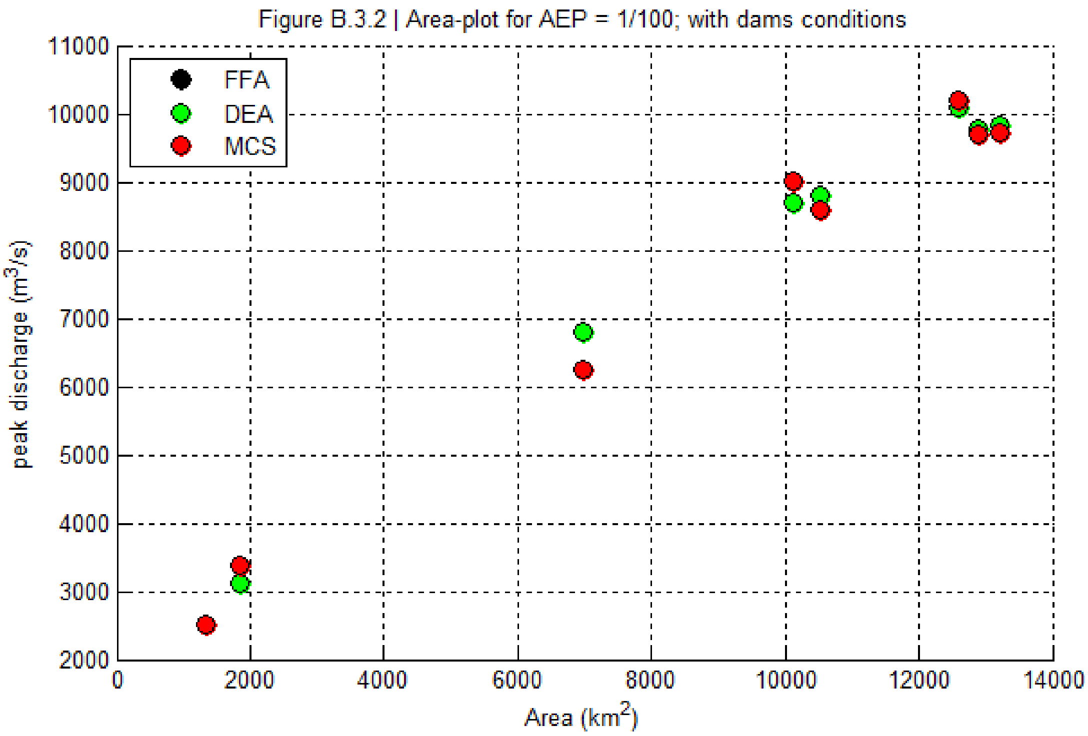
Figure B.2.8b | Location Brisbane; with dams conditions



B.3 Plots of catchment area versus peak discharge

This section contains figures in which catchment area is compared with peak discharge. For a range of AEP's, the design peak flows according to DEA and MCS are plotted against catchment area.





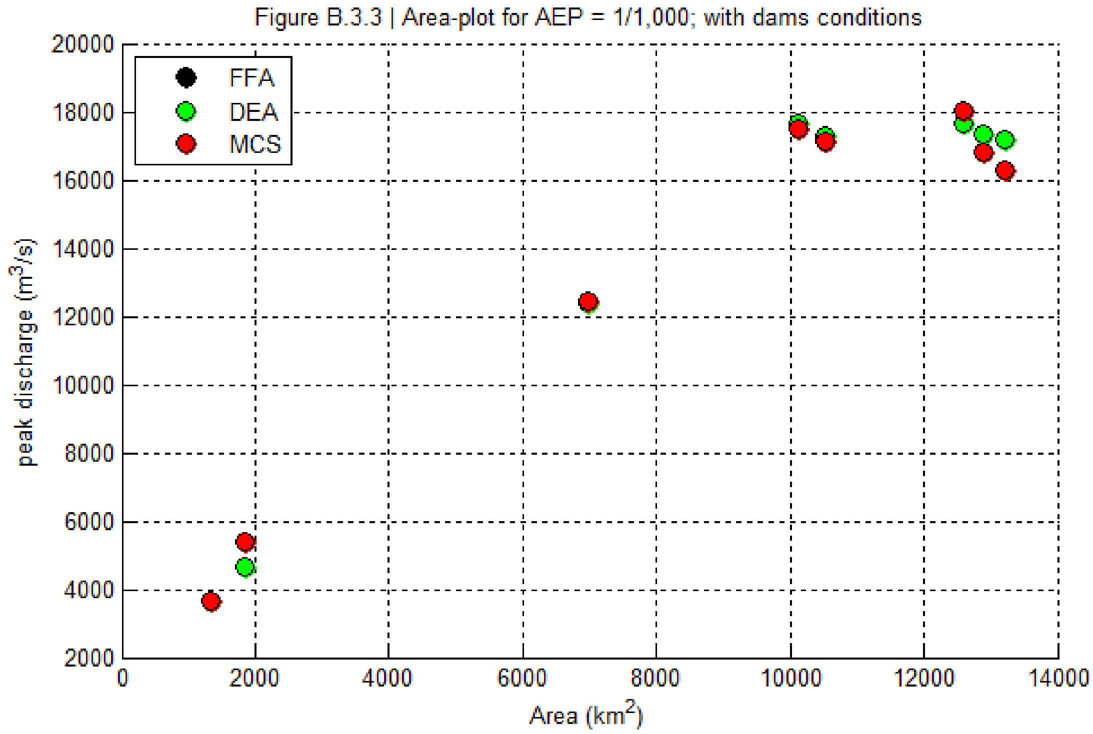


Figure B.3.3 | Area-plot for AEP = 1/1,000; with dams conditions
FFA
DEA
MCS
peak discharge (m3/s)
Area (km2)
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
2000
4000
6000
8000
10000
12000
14000

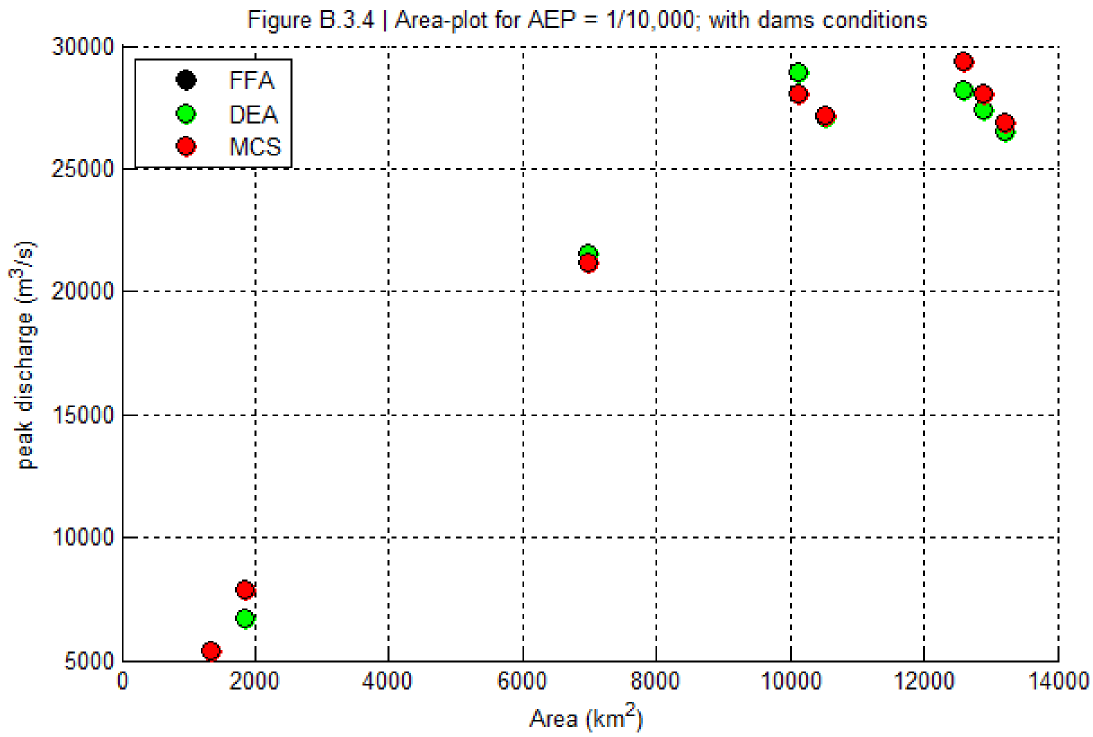


Figure B.3.4 | Area-plot for AEP = 1/10,000; with dams conditions
FFA
DEA
MCS
peak discharge (m3/s)
Area (km2)
30000
25000
20000
15000
10000
5000
0
2000
4000
6000
8000
10000
12000
14000

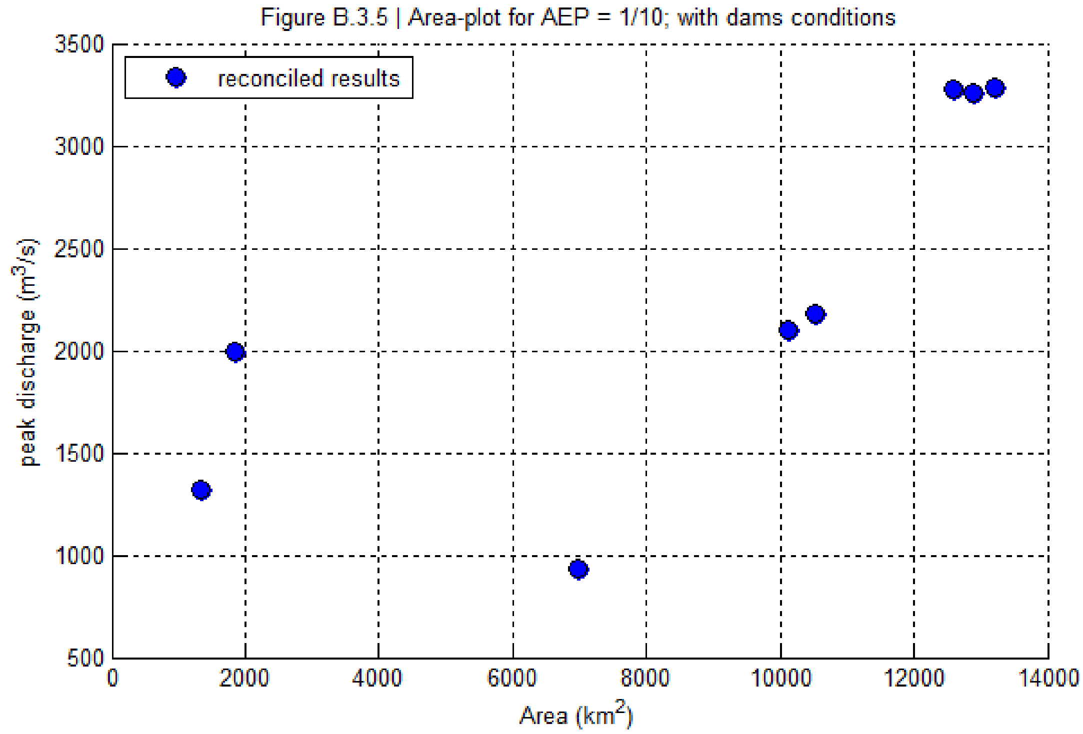


Figure B.3.5 | Area-plot for AEP = 1/10; with dams conditions
reconciled results
peak discharge (m3/s)
Area (km2)
3500
3000
2500
2000
1500
1000
500
0
2000
4000
6000
8000
10000
12000
14000

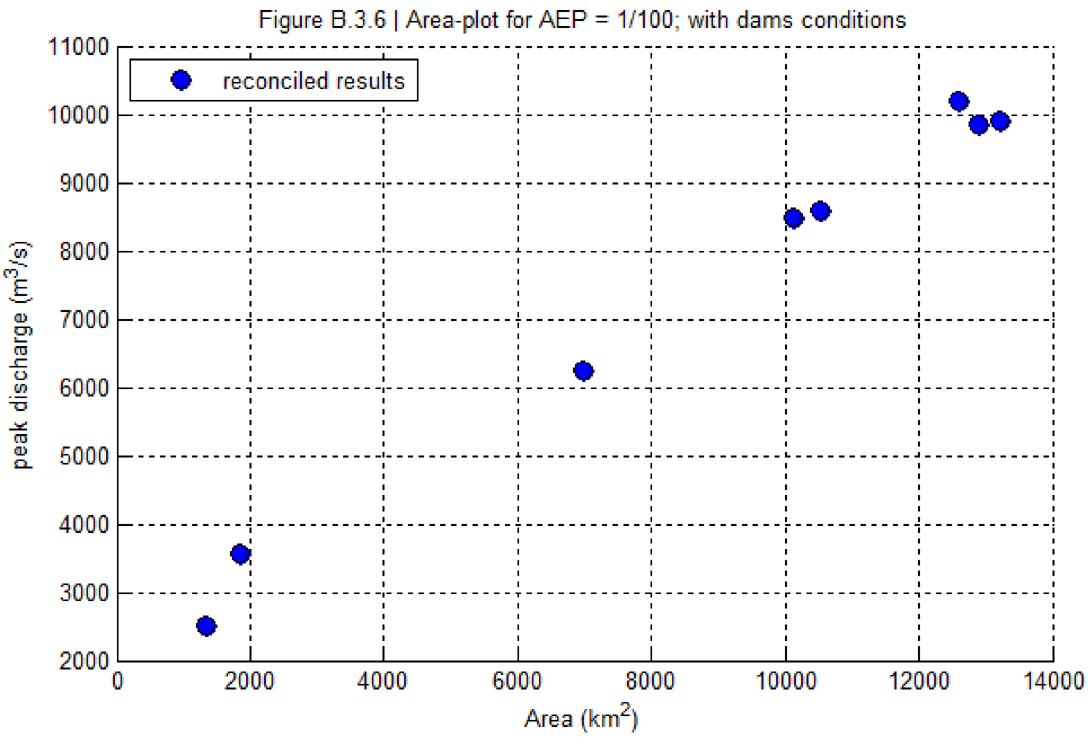


Figure B.3.6 | Area-plot for AEP = 1/100; with dams conditions
reconciled results
peak discharge (m3/s)
Area (km2)
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
0
2000
4000
6000
8000
10000
12000
14000

Figure B.3.7 | Area-plot for AEP = 1/1,000; with dams conditions

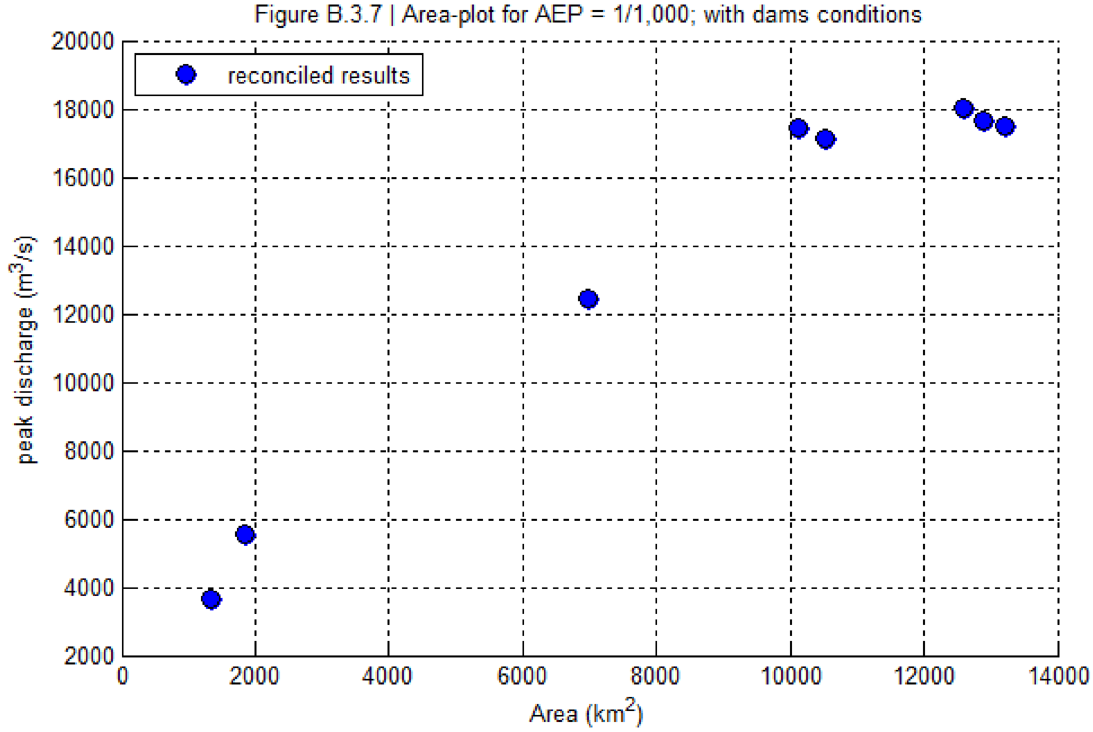
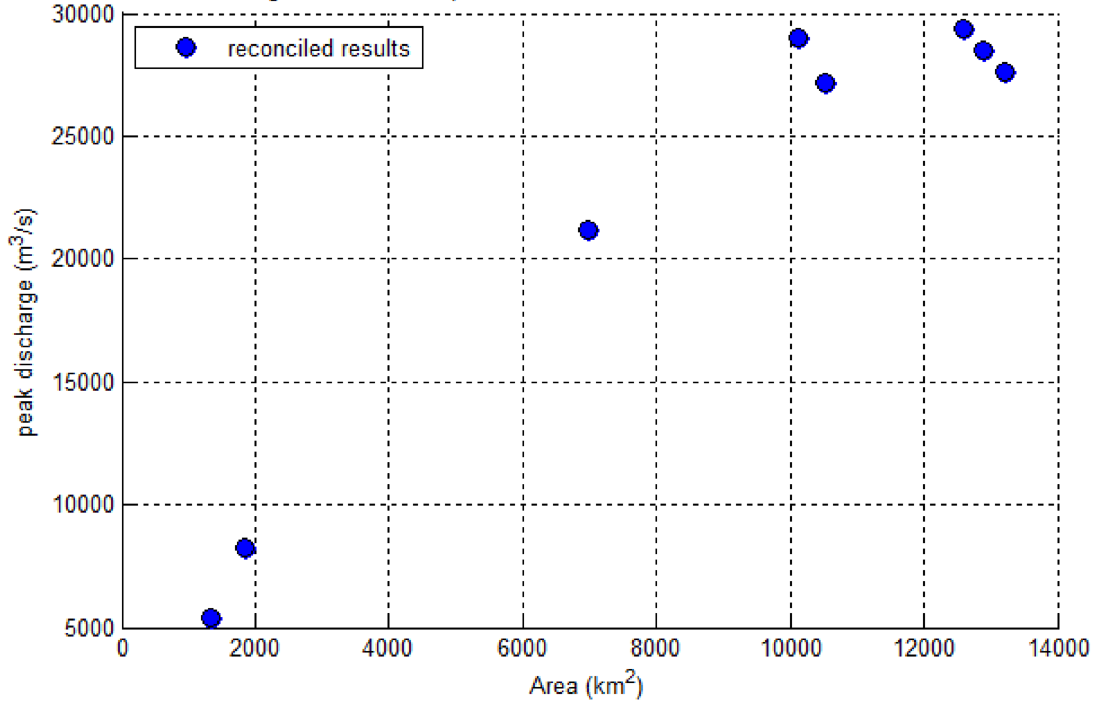


Figure B.3.8 | Area-plot for AEP = 1/10,000; with dams conditions



B.4 Frequency plots of reservoir water levels

The Figures below show MCS frequency curves for water levels at Somerset Dam and Wivenhoe Dam.

Figure B.4.1 Location Somerset Dam; with dams conditions

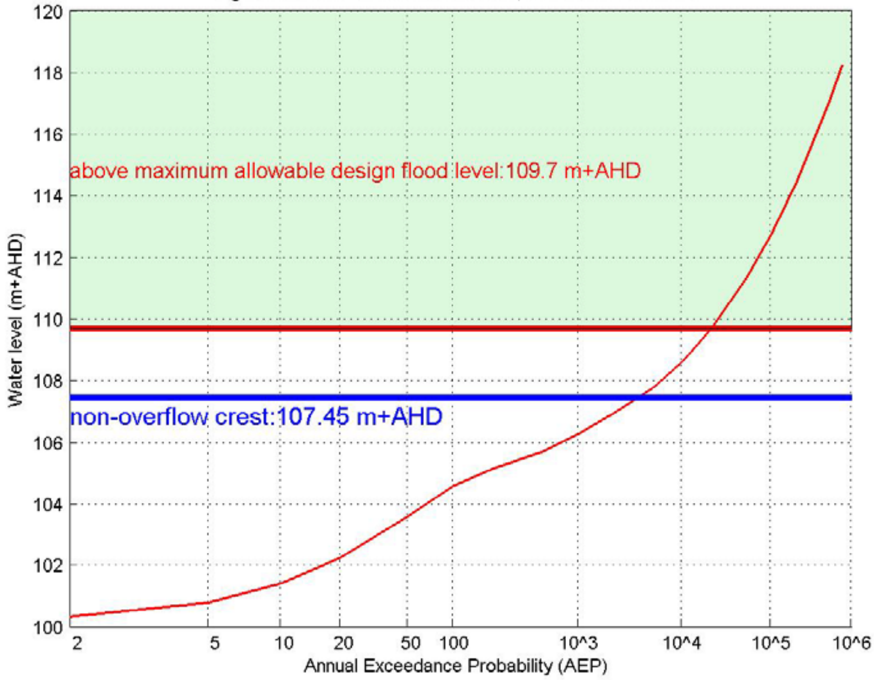
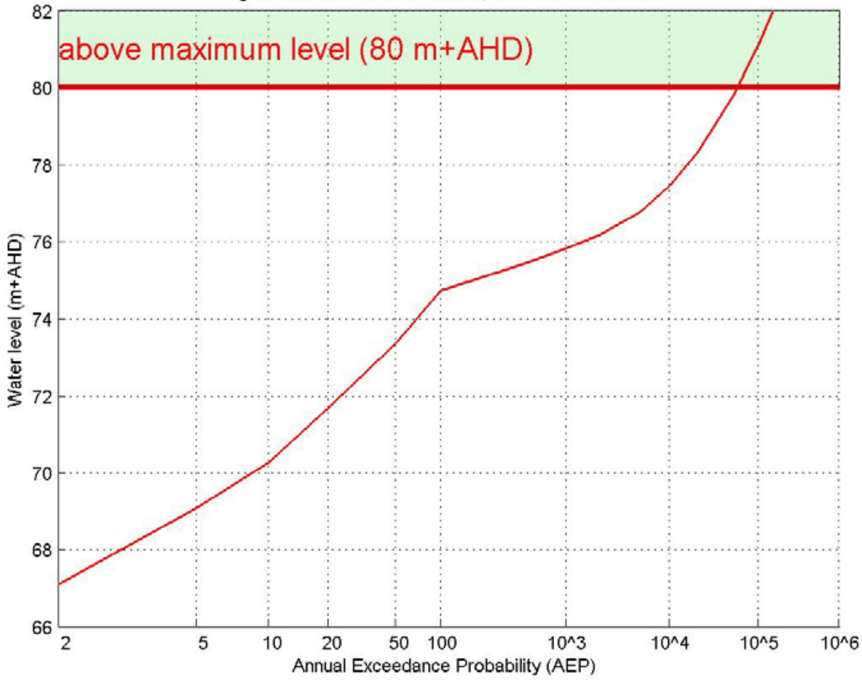


Figure B.4.2 Location Wivenhoe; with dams conditions



Appendix C
'No-dams' design flow volumes

C.1 Tables

Note: the 1 in 100,000 AEP flow volume is only provided for locations for which the AEP of the PMP is below 1 in 100,000.

Table C1 Maximum 24-hour flow volumes (1000 ML) versus AEP; DEA results

Location	AEP (1 in N)											
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Linville	16	48	81	130	200	240	280	320	350	380	500	870	1,500
Gregors Creek	30	92	150	230	360	450	510	580	640	710	930	1,800	2,500
Fulham Vale	29	90	150	230	360	450	510	580	640	700	940	1,800	2,400
Peachester	6	12	18	24	32	39	45	52	60	68	86	120	150
Woodford	12	25	37	51	68	82	95	110	130	140	180	250	330
Somerset Dam	39	86	130	180	250	300	360	420	470	530	650	920	1,300
Tinton	7	15	24	36	54	66	75	87	97	110	140	200	370
Middle Ck	70	190	320	470	690	810	930	1,100	1,200	1,300	1,800	3,700	4,300
Wivenhoe	70	190	330	480	700	830	960	1,100	1,200	1,400	1,800		3,700
Helidon	2	9	15	25	39	49	56	66	73	81	110	160	360
Gatton	6	36	60	99	160	190	220	260	290	320	430	730	1,300
Glenore Grove	9	50	85	140	210	270	310	360	400	440	600	1,100	1,700
Savages Crossing	66	240	410	620	910	1,100	1,300	1,500	1,600	1,800	2,300		5,100
Mount Crosby	65	240	420	620	910	1,100	1,300	1,500	1,600	1,800	2,400		5,100
Walloon	11	25	38	57	82	99	110	130	150	160	210	300	520
Kalbar Weir	10	21	33	47	66	80	91	100	120	130	160	230	410
Amberley	14	32	47	68	100	120	140	160	180	200	250	400	680
Loamside	4	9	14	20	28	34	40	46	51	57	69	99	160

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Ipswich	27	64	95	140	200	240	280	320	360	400	530	870	1,300
Moggill	71	270	470	700	1,000	1,200	1,400	1,600	1,800	2,000	2,600		5,400
Centenary Bridge	71	270	460	690	980	1,200	1,400	1,600	1,800	2,000	2,600		5,400
Brisbane	71	270	460	690	990	1,200	1,400	1,600	1,800	2,000	2,600		5,300

Table C2 Maximum 48-hour flow volumes (1000 ML) versus AEP; DEA results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Linville	21	62	120	200	310	390	430	500	540	600	730	1,300	2,100
Gregors Creek	40	120	240	380	600	740	830	940	1,000	1,100	1,400	2,600	3,700
Fulham Vale	38	120	230	370	600	730	820	940	1,000	1,100	1,400	2,700	3,500
Peachester	10	20	30	40	54	64	76	89	100	110	140	190	220
Woodford	20	43	63	85	120	140	160	190	220	240	300	400	490
Somerset Dam	60	140	220	310	430	520	610	710	790	890	1,100	1,500	1,900
Tinton	8	21	36	56	85	100	120	130	150	160	200	300	540
Middle Ck	100	310	540	810	1,200	1,500	1,700	1,900	2,100	2,400	3,000	5,700	6,700
Wivenhoe	100	310	550	840	1,200	1,500	1,700	2,000	2,200	2,500	3,100		5,900
Helidon	3	11	22	37	60	74	85	99	110	120	150	240	520
Gatton	8	43	86	150	240	290	340	390	430	480	620	1,000	1,800
Glenore Grove	12	63	130	210	340	420	480	560	620	690	880	1,500	2,400
Savages Crossing	100	380	700	1,100	1,600	2,000	2,300	2,600	2,900	3,300	4,100		8,800
Mount Crosby	100	390	710	1,100	1,700	2,000	2,300	2,700	3,000	3,300	4,200		8,900

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Walloon	14	37	60	90	130	160	180	210	230	250	310	460	740
Kalbar Weir	12	31	50	74	110	130	140	160	180	200	240	350	590
Amberley	20	53	85	120	170	210	240	270	300	330	420	620	990
Loamside	5	13	21	32	46	55	64	74	83	92	110	150	240
Ipswich	39	100	170	240	350	420	480	560	620	690	860	1,300	1,800
Moggill	120	450	830	1,300	1,900	2,300	2,600	3,000	3,300	3,700	4,800		10,000
Centenary Bridge	120	450	840	1,300	1,900	2,200	2,600	3,000	3,300	3,700	4,800		10,000
Brisbane	130	460	850	1,300	1,900	2,200	2,600	3,000	3,300	3,700	4,700		9,800

Table C3 Maximum 72-hour flow volumes (1000 ML) versus AEP; DEA results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Linville	22	69	150	250	390	480	540	620	680	740	890	1,600	2,700
Gregors Creek	43	140	290	490	770	940	1,100	1,200	1,300	1,400	1,700	3,300	4,600
Fulham Vale	40	130	290	490	760	940	1,100	1,200	1,300	1,400	1,700	3,400	4,400
Peachester	12	26	38	51	68	81	96	110	130	140	180	240	290
Woodford	24	55	81	110	150	180	210	250	280	310	380	500	620
Somerset Dam	69	180	280	400	560	680	790	920	1,000	1,200	1,400	1,900	2,400
Tinton	8	24	46	72	110	130	150	170	190	200	250	370	680
Middle Ck	110	370	690	1,100	1,600	2,000	2,200	2,600	2,800	3,100	3,800	7,400	8,700
Wivenhoe	120	380	710	1,100	1,700	2,000	2,300	2,700	2,900	3,200	4,000		7,700
Helidon	3	12	26	47	75	93	110	120	140	150	180	290	650

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Gatton	8	46	100	190	310	370	420	490	540	600	750	1,300	2,200
Glenore Grove	12	67	150	270	430	540	610	700	780	860	1,100	1,900	3,000
Savages Crossing	120	460	900	1,400	2,200	2,700	3,000	3,500	3,900	4,300	5,400		12,000
Mount Crosby	120	470	920	1,500	2,200	2,700	3,100	3,600	4,000	4,400	5,500		12,000
Walloon	15	45	77	120	170	200	230	260	290	320	380	560	930
Kalbar Weir	14	39	65	94	130	160	180	210	220	250	300	430	750
Amberley	22	65	110	160	230	280	310	360	390	430	510	770	1,300
Loamside	6	16	27	40	58	70	81	93	100	110	130	190	310
Ipswich	43	120	210	320	460	550	630	720	800	880	1,100	1,600	2,300
Moggill	160	570	1,100	1,700	2,600	3,100	3,500	4,100	4,500	5,000	6,400		13,000
Centenary Bridge	160	580	1,100	1,700	2,600	3,100	3,500	4,100	4,500	5,000	6,400		13,000
Brisbane	170	590	1,100	1,800	2,600	3,100	3,600	4,100	4,500	5,000	6,400		13,000

Table C4 Maximum 24-hour flow volumes (1000 ML) versus AEP; MCS results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Linville	11	40	81	120	180	240	270	320	360	390	520	890	1,400
Gregors Creek	24	75	140	230	340	450	520	600	670	730	980	1,800	2,400
Fulham Vale	19	72	140	230	350	440	510	600	670	740	1,000	1,800	2,400
Peachester	7	12	18	24	31	38	45	53	60	65	90	120	150
Woodford	17	33	47	62	84	100	120	140	160	180	230	310	390
Somerset Dam	45	88	130	180	250	310	350	410	450	500	650	930	1,200

Location	AEP (1 in N)												
	2	**5**	**10**	**20**	**50**	**100**	**200**	**500**	**1,000**	**2,000**	**10,000**	**100,000**	**AEP of PMP**
Tinton	4	13	22	32	46	61	71	82	93	100	140	200	330
Middle Ck	76	190	310	480	670	880	1,000	1,200	1,300	1,400	2,000	3,600	4,100
Wivenhoe	73	190	320	480	690	880	990	1,200	1,300	1,500	1,900	3,600	4,100
Helidon	2	8	16	24	35	46	53	62	69	78	110	160	310
Gatton	8	29	51	82	110	160	190	230	260	290	420	720	1,200
Glenore Grove	11	43	75	110	160	220	270	330	360	410	590	1,000	1,600
Savages Crossing	88	230	390	600	860	1,100	1,300	1,500	1,700	1,900	2,600		5,000
Mount Crosby	90	230	390	600	860	1,100	1,300	1,500	1,700	1,800	2,600		5,000
Walloon	14	26	39	55	78	95	110	130	140	160	220	310	480
Kalbar Weir	13	24	34	47	65	79	91	110	120	130	170	250	390
Amberley	20	38	53	74	100	130	150	180	200	220	280	420	650
Loamside	4	10	14	20	27	33	38	46	52	56	75	100	150
Ipswich	35	75	110	150	200	250	290	340	380	430	580	900	1,200
Moggill	130	290	470	690	960	1,200	1,400	1,600	1,900	2,100	2,900		5,300
Centenary Bridge	130	290	460	680	930	1,200	1,300	1,600	1,800	2,100	2,800		5,200
Brisbane	130	290	460	700	940	1,100	1,300	1,600	1,800	2,000	2,700		5,100

Table C5 Maximum 48-hour flow volumes (1000 ML) versus AEP; MCS results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Linville	15	53	120	190	270	380	420	480	530	590	760	1,300	2,000
Gregors Creek	32	110	220	350	520	710	810	930	1,000	1,100	1,500	2,700	3,600
Fulham Vale	27	98	220	350	520	710	810	920	1,000	1,100	1,500	2,700	3,500
Peachester	10	19	28	40	51	62	75	89	99	110	140	190	220
Woodford	26	51	73	100	140	170	200	240	280	310	380	500	610
Somerset Dam	69	140	210	300	410	510	590	690	770	860	1,100	1,500	1,800
Tinton	5	17	33	52	71	98	110	130	140	160	200	300	510
Middle Ck	120	290	510	810	1,100	1,400	1,700	1,900	2,100	2,400	3,100	5,700	6,600
Wivenhoe	120	300	520	790	1,200	1,500	1,700	2,000	2,100	2,400	3,200	6,000	6,900
Helidon	3	11	22	38	52	72	83	96	110	120	150	240	460
Gatton	10	37	72	120	180	250	300	360	390	450	610	1,000	1,700
Glenore Grove	15	56	100	170	250	370	440	510	570	660	880	1,500	2,200
Savages Crossing	150	370	630	980	1,400	1,900	2,200	2,600	2,800	3,200	4,400		8,600
Mount Crosby	150	380	650	1,000	1,500	1,900	2,200	2,600	2,900	3,300	4,400		8,900
Walloon	20	39	61	88	120	160	180	210	230	250	330	470	720
Kalbar Weir	18	34	52	75	100	130	150	170	190	210	260	380	590
Amberley	30	60	88	130	170	220	250	280	320	350	450	670	1,000
Loamside	6	14	21	32	42	54	63	75	82	93	110	160	230
Ipswich	52	110	170	250	330	420	490	560	630	710	920	1,400	1,800
Moggill	210	490	790	1,200	1,700	2,200	2,500	3,000	3,300	3,800	5,000		9,800
Centenary Bridge	210	500	790	1,200	1,700	2,200	2,500	2,900	3,300	3,700	4,900		9,800

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Brisbane	220	510	810	1,200	1,700	2,200	2,500	2,900	3,200	3,700	4,900		9,700

Table C6 Maximum 72-hour flow volumes (1000 ML) versus AEP; MCS results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Linville	17	65	140	220	340	470	520	610	660	730	920	1,600	2,600
Gregors Creek	38	120	260	440	660	920	1,000	1,200	1,300	1,400	1,800	3,300	4,500
Fulham Vale	32	120	260	420	650	890	1,000	1,200	1,300	1,400	1,800	3,300	4,400
Peachester	13	25	35	47	64	77	91	110	130	150	190	250	290
Woodford	32	63	92	130	170	210	250	310	360	400	510	660	790
Somerset Dam	87	170	260	380	490	630	750	860	980	1,100	1,400	1,900	2,300
Tinton	6	20	40	62	92	120	140	160	180	200	250	360	650
Middle Ck	150	370	630	1,000	1,400	1,900	2,200	2,500	2,800	3,100	3,900	7,400	8,600
Wivenhoe	150	390	660	1,000	1,500	2,000	2,300	2,600	2,900	3,200	4,100	7,700	8,900
Helidon	3	13	28	45	66	92	100	120	130	150	180	290	590
Gatton	11	44	90	150	210	310	380	450	490	560	730	1,200	2,100
Glenore Grove	19	66	130	210	320	460	550	650	730	800	1,000	1,800	2,700
Savages Crossing	190	460	820	1,200	1,900	2,600	3,000	3,500	3,800	4,400	5,700		11,000
Mount Crosby	190	480	850	1,300	2,000	2,600	3,000	3,600	3,900	4,400	5,800		12,000
Walloon	23	49	77	110	150	200	220	260	290	320	400	580	910
Kalbar Weir	22	44	66	95	130	170	190	220	240	270	330	470	790
Amberley	38	75	110	170	240	290	330	380	420	460	570	850	1,300

Location	AEP (1 in N)											
	2	**5**	**10**	**20**	**50**	**100**	**200**	**500**	**1,000**	**2,000**	**10,000**	**100,000**	**AEP of PMP**
Loamside	8	17	26	39	53	68	79	93	100	120	140	200	290
Ipswich	63	140	210	320	440	570	640	740	830	900	1,100	1,700	2,400
Moggill	270	630	1,100	1,600	2,200	3,000	3,500	4,000	4,600	5,100	6,700		13,000
Centenary Bridge	270	650	1,100	1,600	2,200	3,000	3,400	4,000	4,500	5,100	6,700		13,000
Brisbane	280	660	1,100	1,700	2,300	3,000	3,400	4,000	4,500	5,000	6,800		13,000

C.2 Frequency plots

Figure C.2.1 | Location Linville; no dams conditions

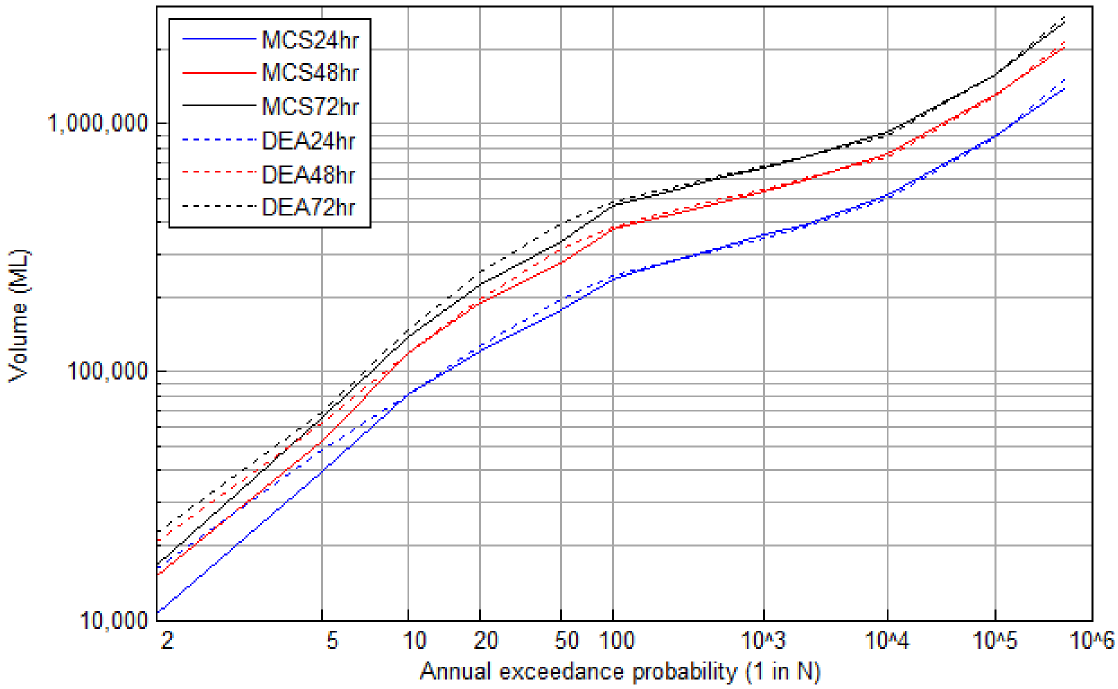


Figure C.2.2 | Location Gregors Creek; no dams conditions

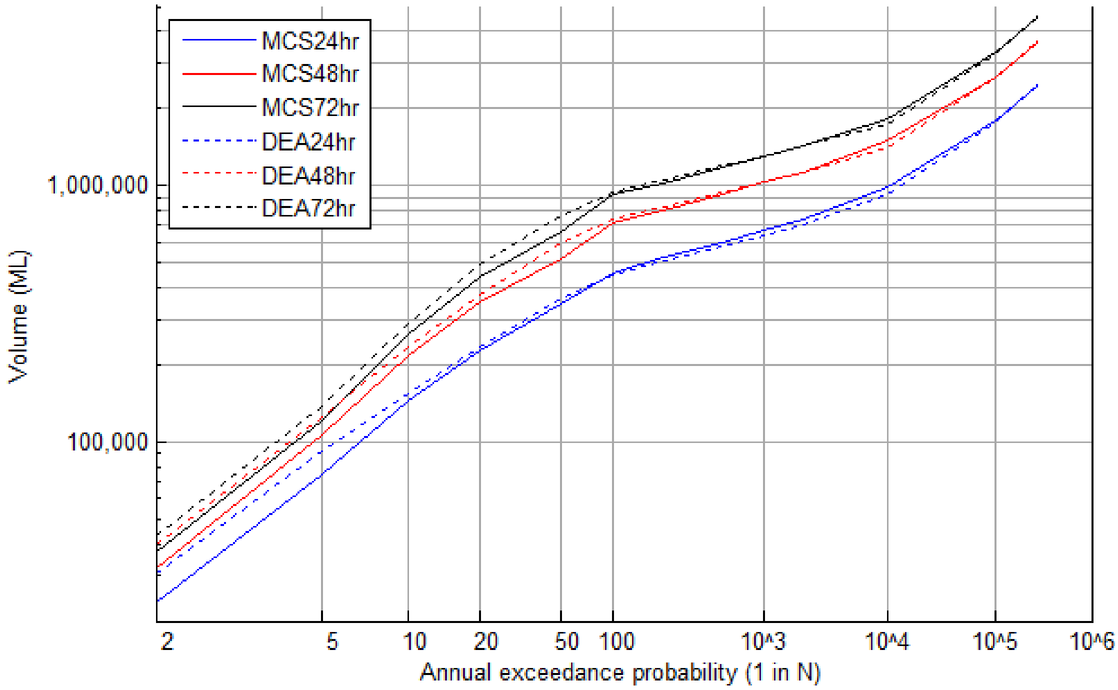


Figure C.2.3 | Location Fulham Vale; no dams conditions

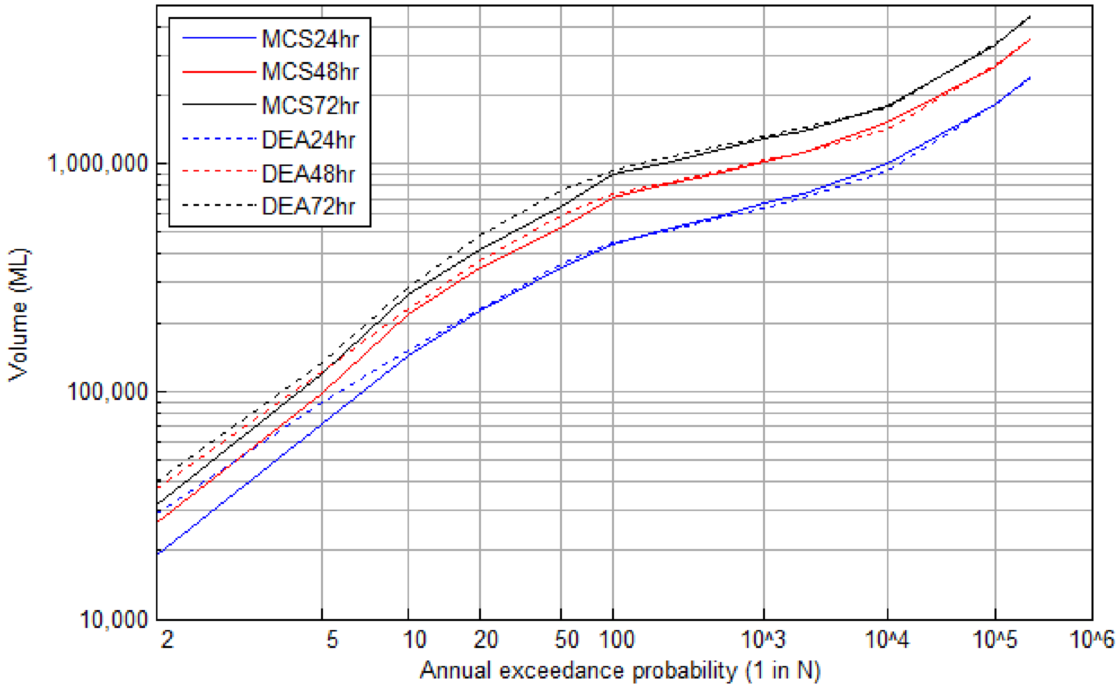


Figure C.2.4 | Location Peachester; no dams conditions

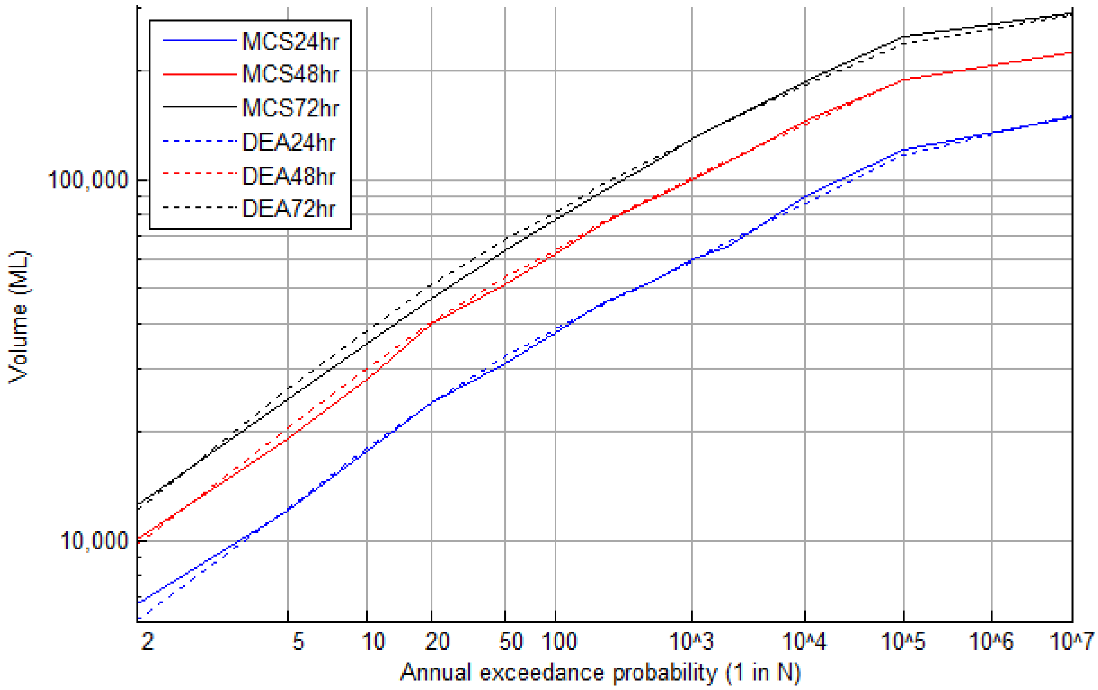


Figure C.2.5 | Location Woodford; no dams conditions

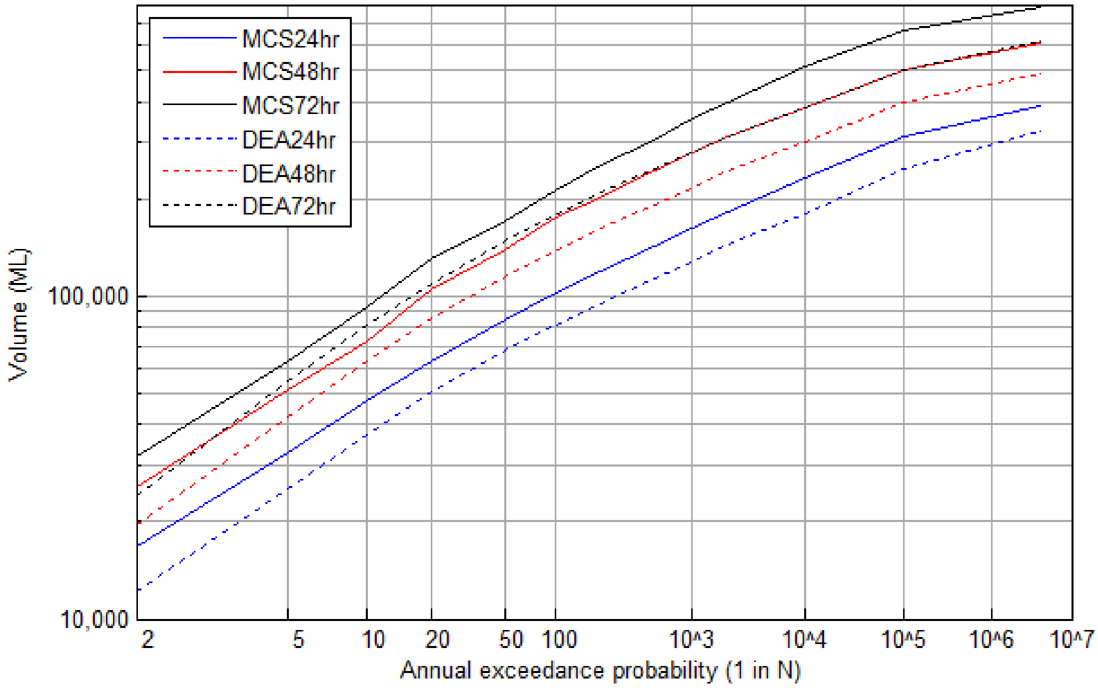


Figure C.2.6 | Location Somerset Dam; no dams conditions

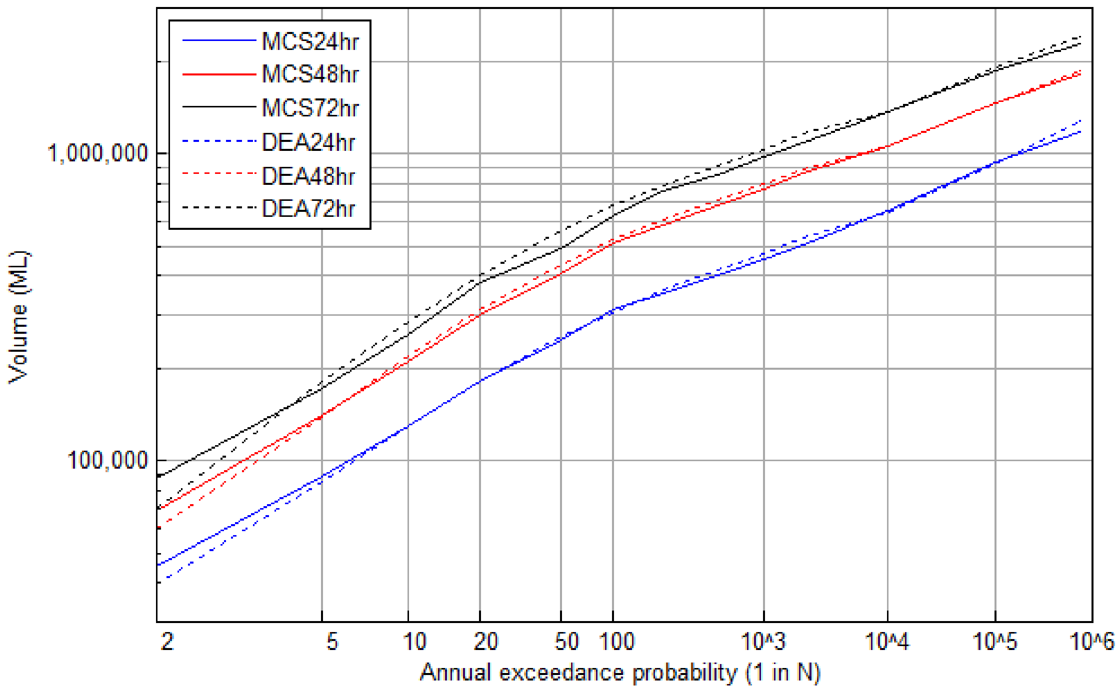


Figure C.2.7 | Location Tinton; no dams conditions

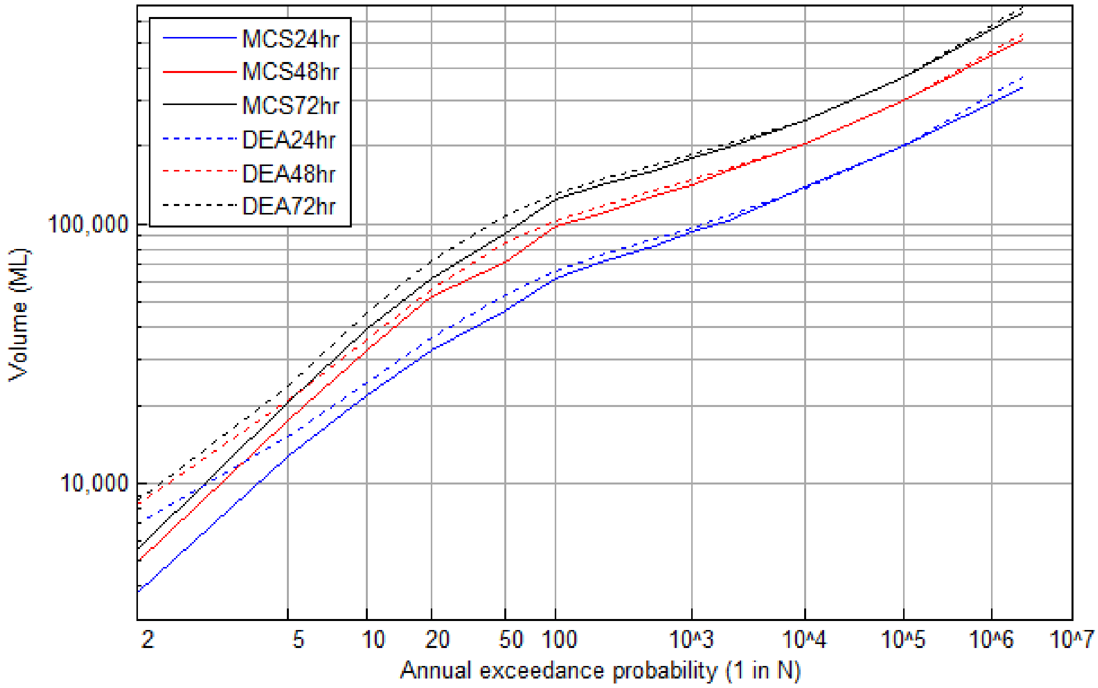


Figure C.2.8 | Location Middle Ck; no dams conditions

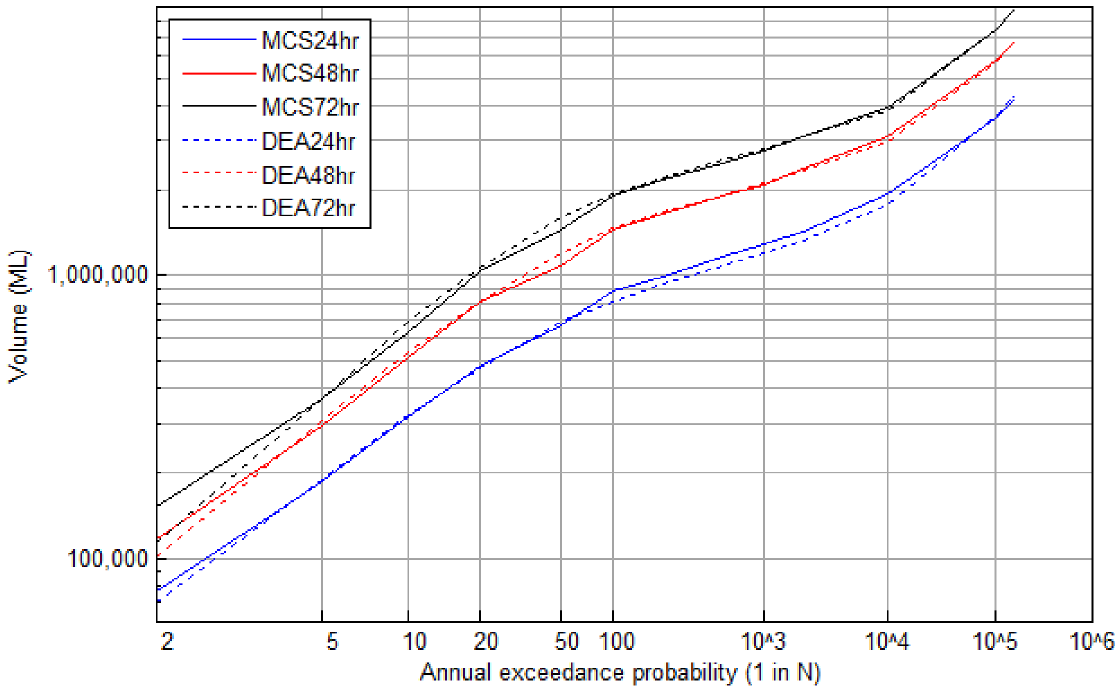


Figure C.2.9 | Location Wivenhoe; no dams conditions

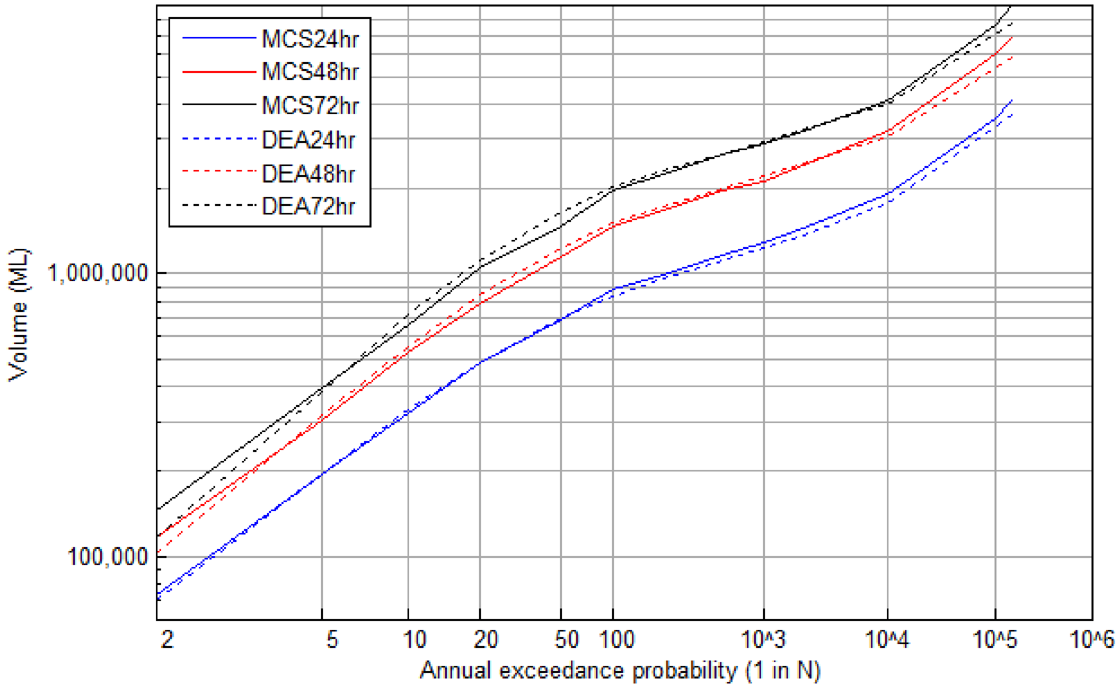


Figure C.2.10 | Location Helidon; no dams conditions

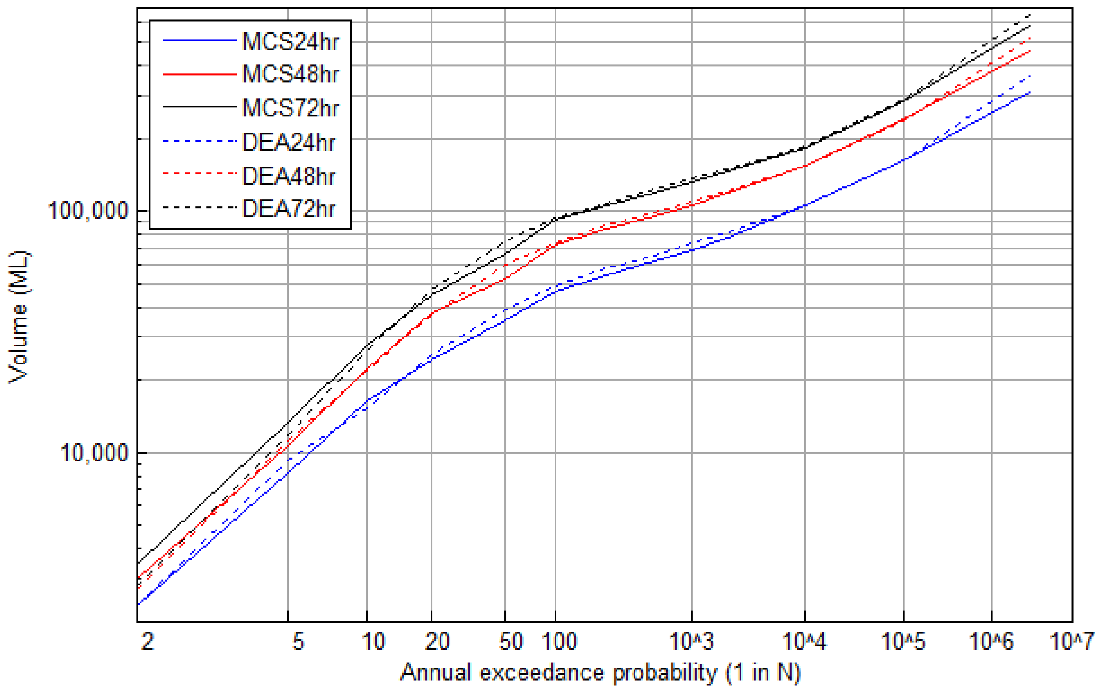


Figure C.2.11 | Location Gatton; no dams conditions

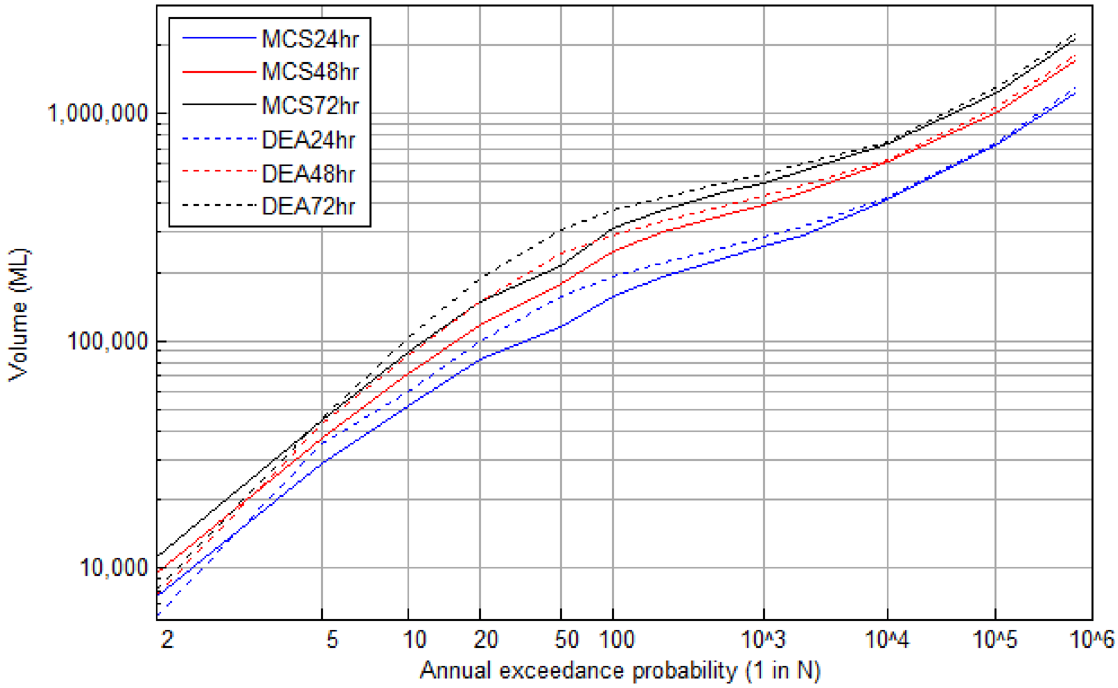


Figure C.2.12 | Location Glenore Grove; no dams conditions

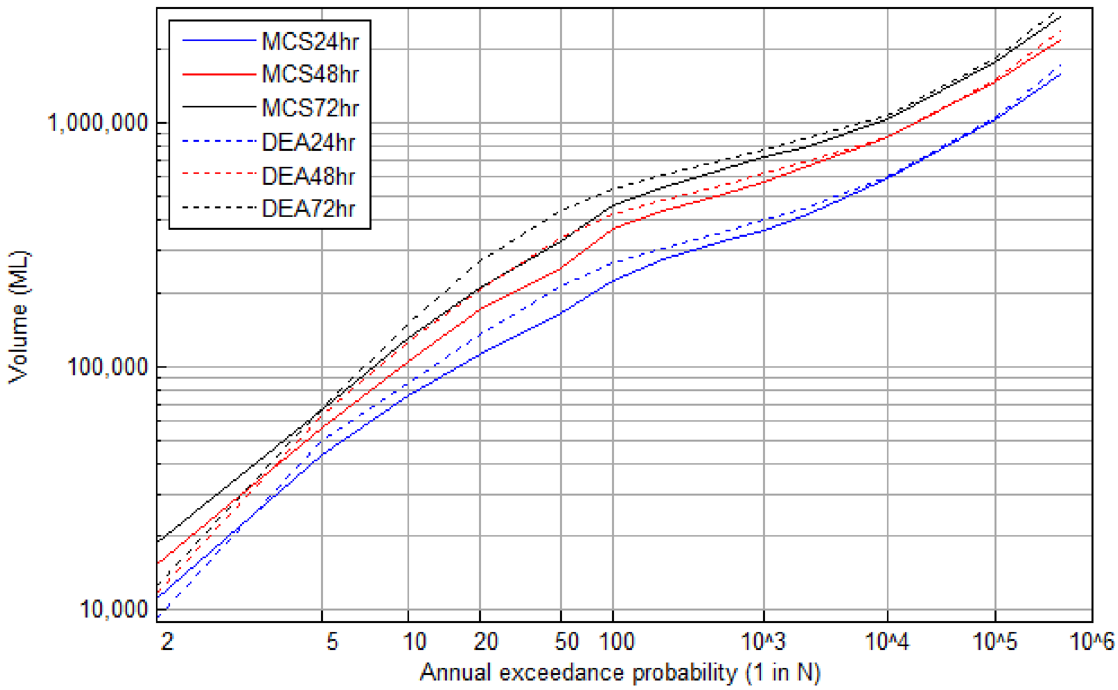


Figure C.2.13 | Location Savages Crossing; no dams conditions

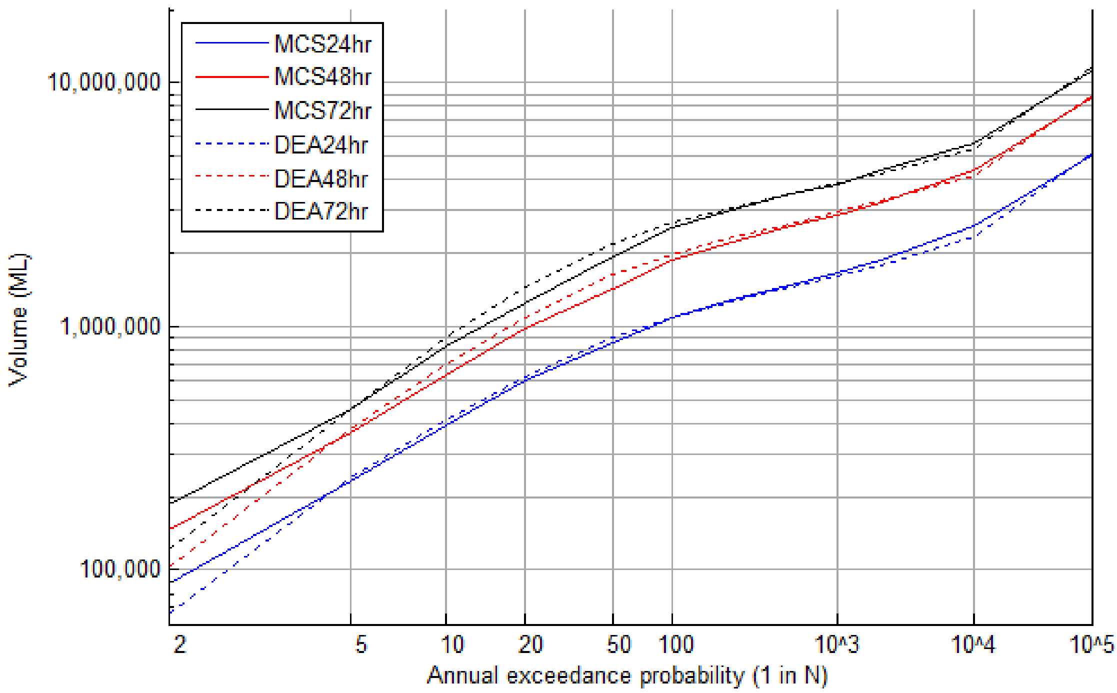


Figure C.2.14 | Location Mount Crosby; no dams conditions

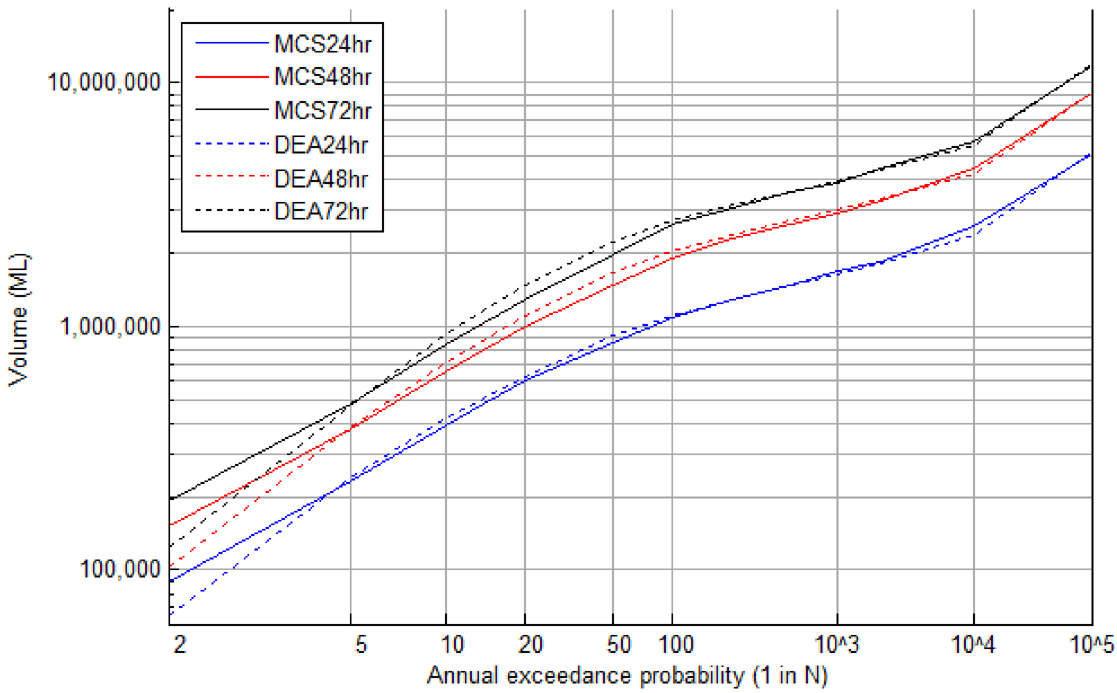


Figure C.2.15 | Location Walloon; no dams conditions

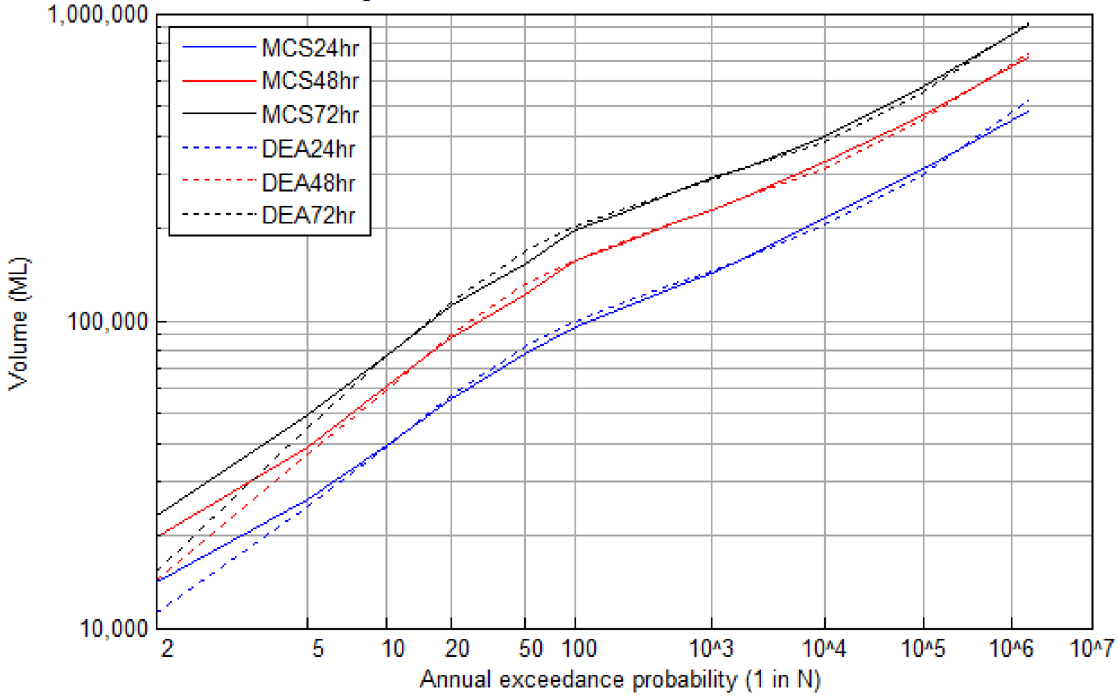


Figure C.2.16 | Location Kalbar Weir; no dams conditions

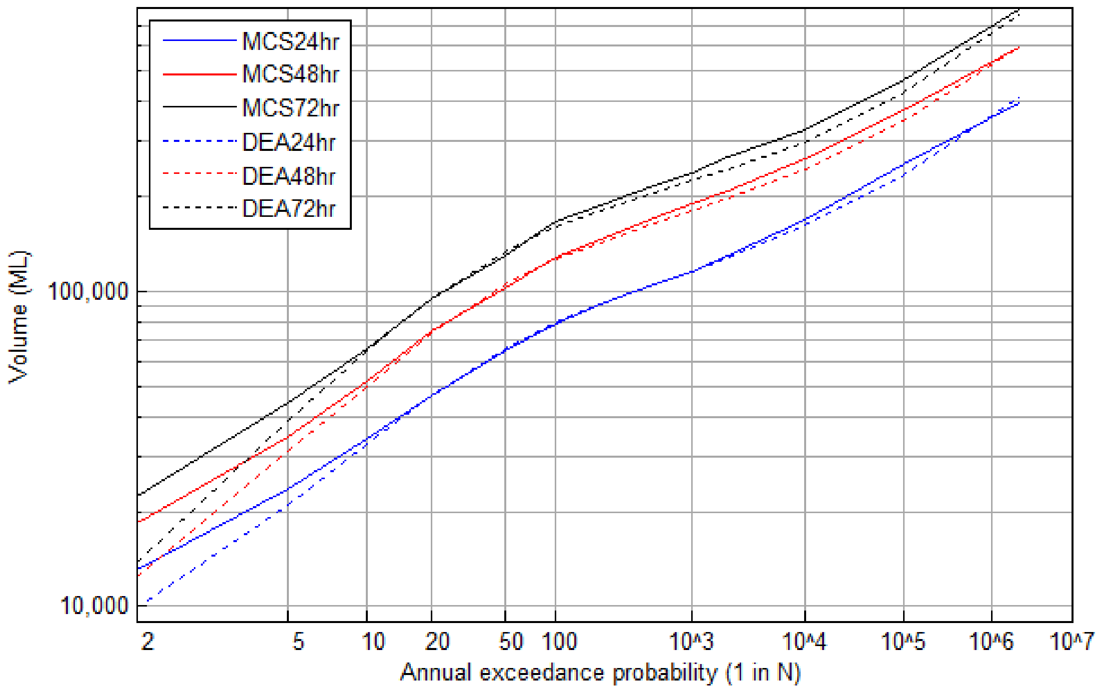


Figure C.2.17 | Location Amberley; no dams conditions

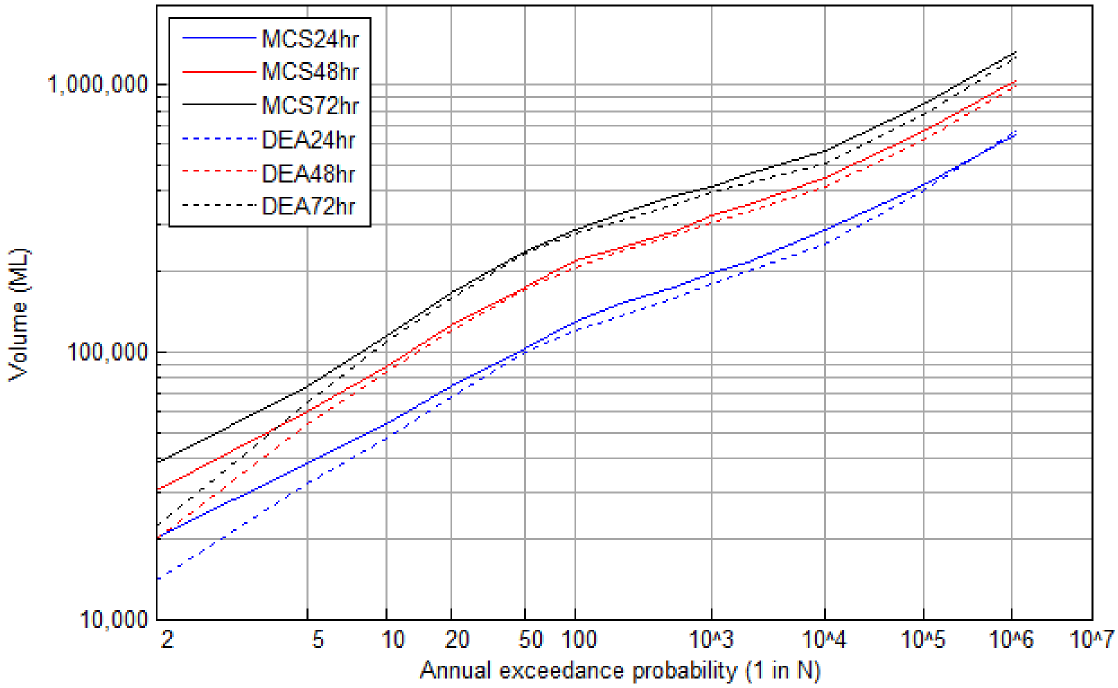


Figure C.2.18 | Location Loamside; no dams conditions

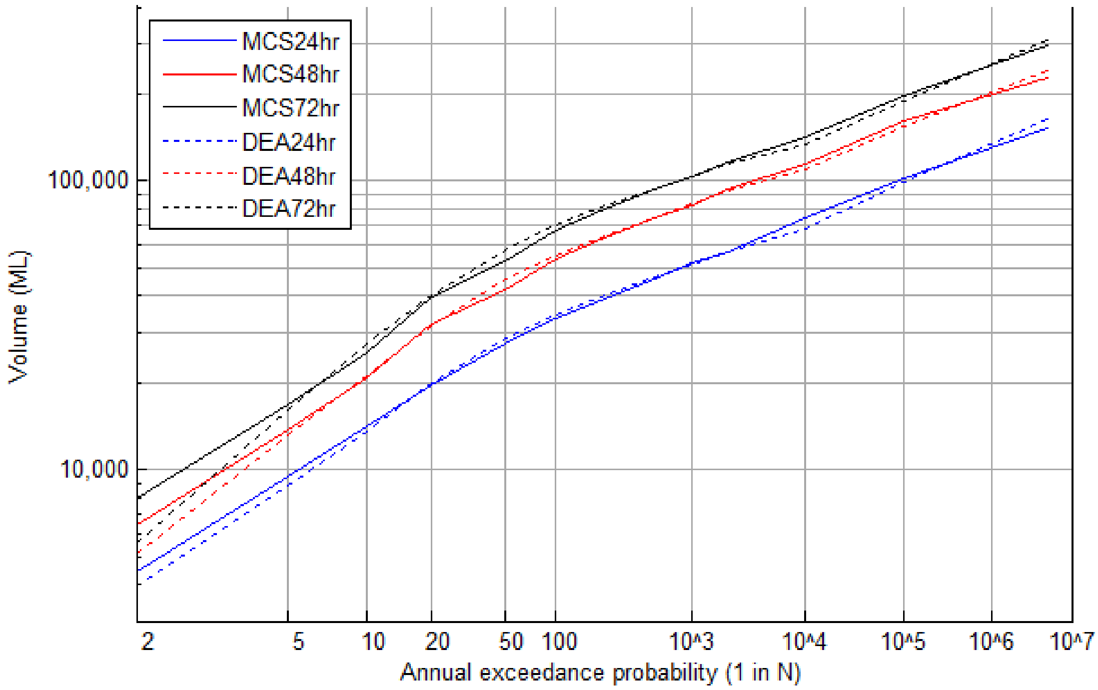


Figure C.2.19 | Location Ipswich; no dams conditions

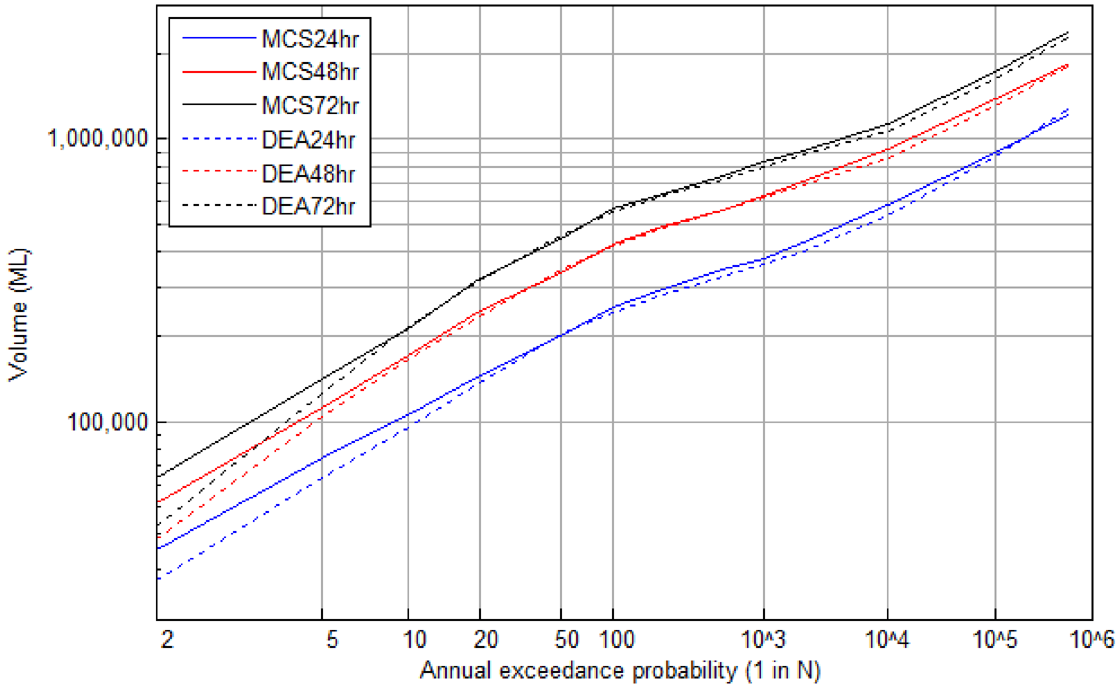


Figure C.2.20 | Location Moggill; no dams conditions

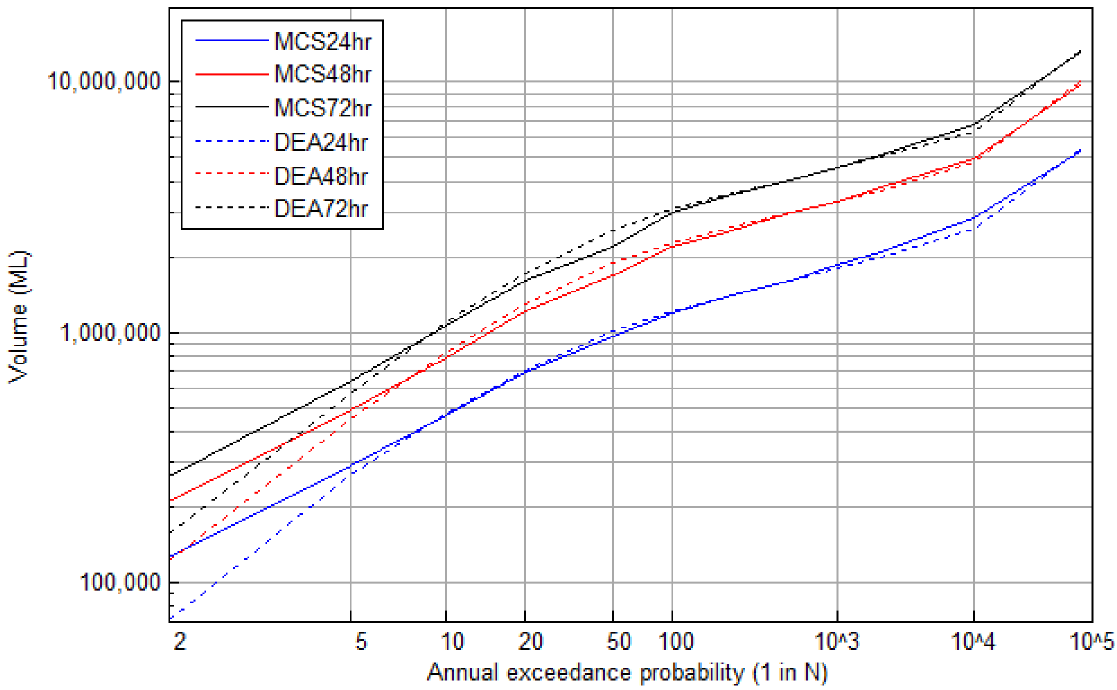


Figure C.2.21 | Location Centenary Bridge; no dams conditions

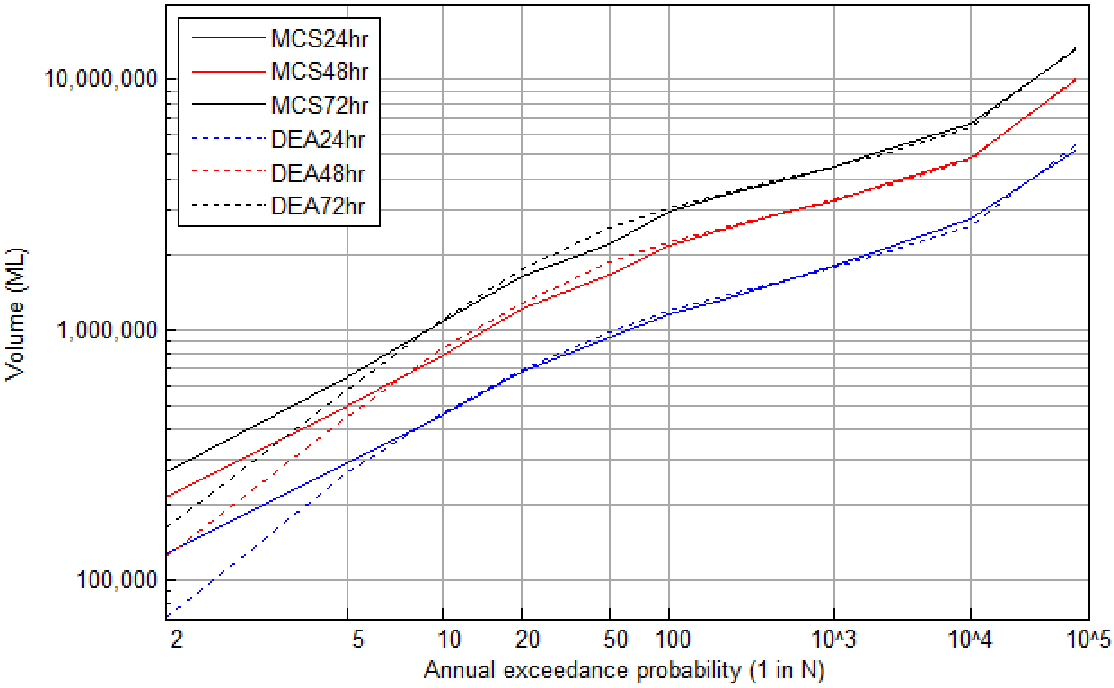
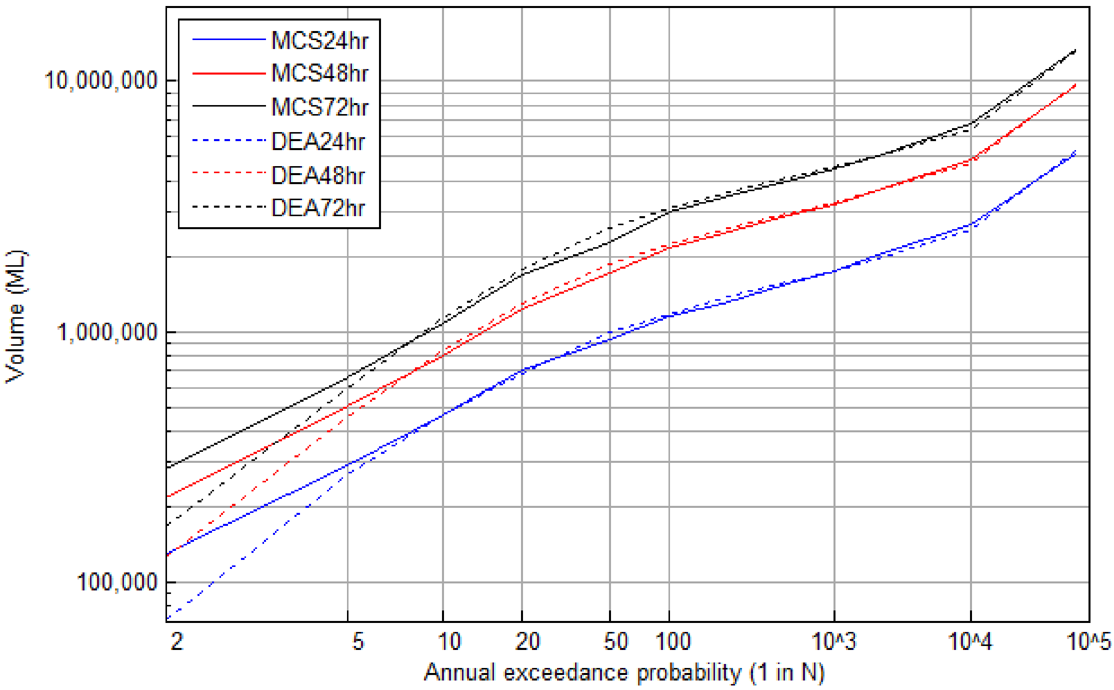


Figure C.2.22 | Location Brisbane; no dams conditions



C.3 Plots of catchment area versus flow volumes

This section contains ten figures in which flows 24-, 48- and 72 hour flow volumes are plotted against catchment area: five for DEA results (one figure per AEP) and five for MCS results.

Figure C.3.1 | Area-plot for AEP = 1/10; no dams conditions

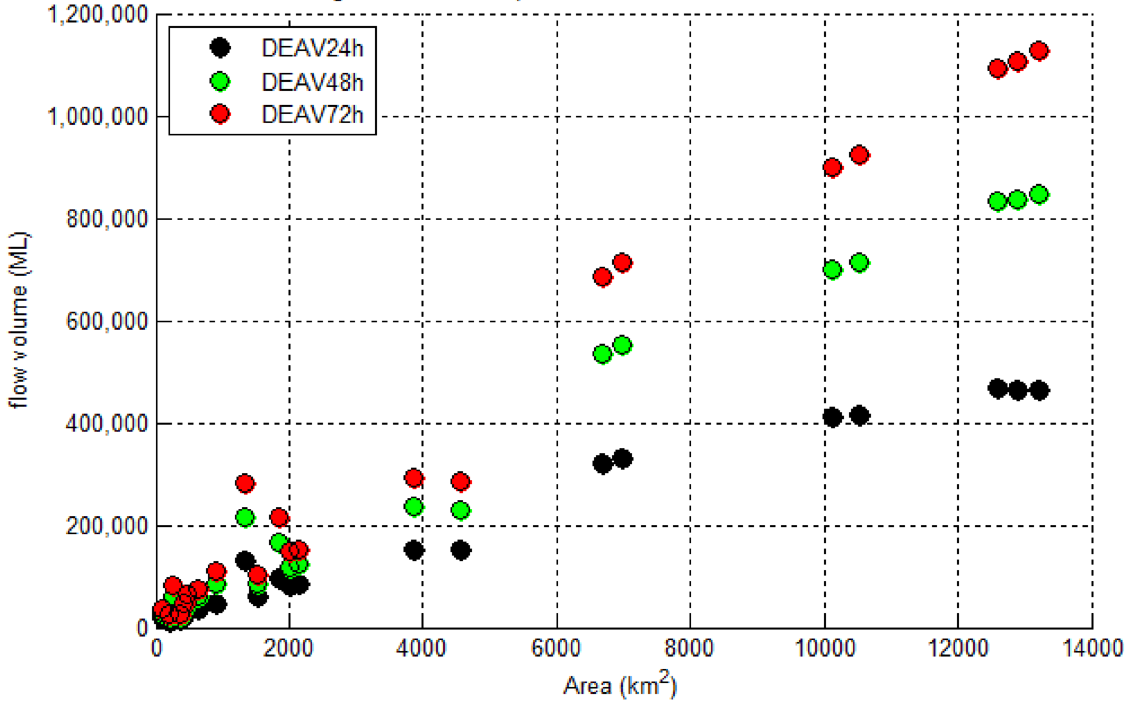


Figure C.3.2 | Area-plot for AEP = 1/100; no dams conditions

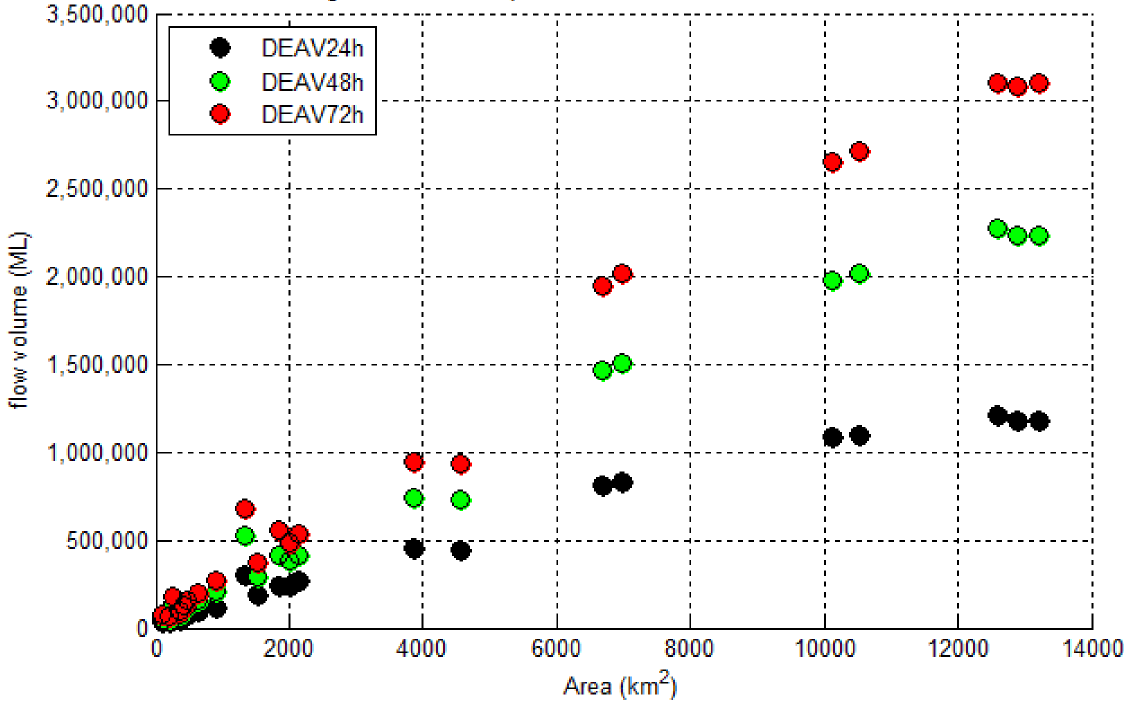


Figure C.3.3 | Area-plot for AEP = 1/1,000; no dams conditions

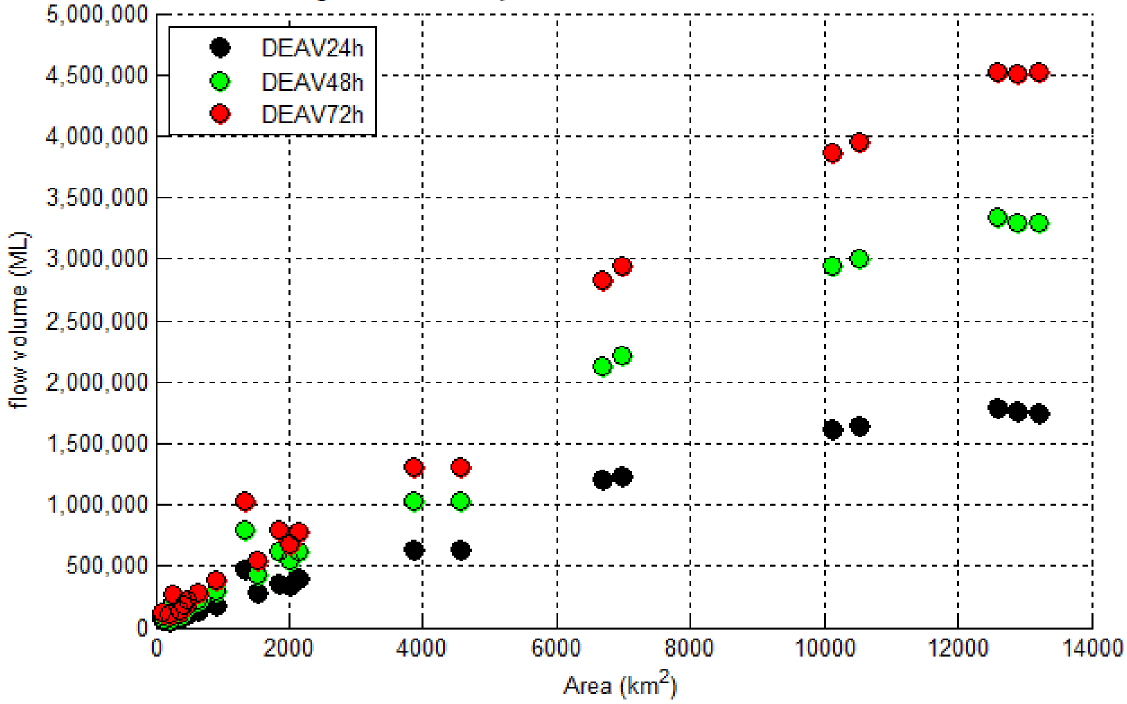
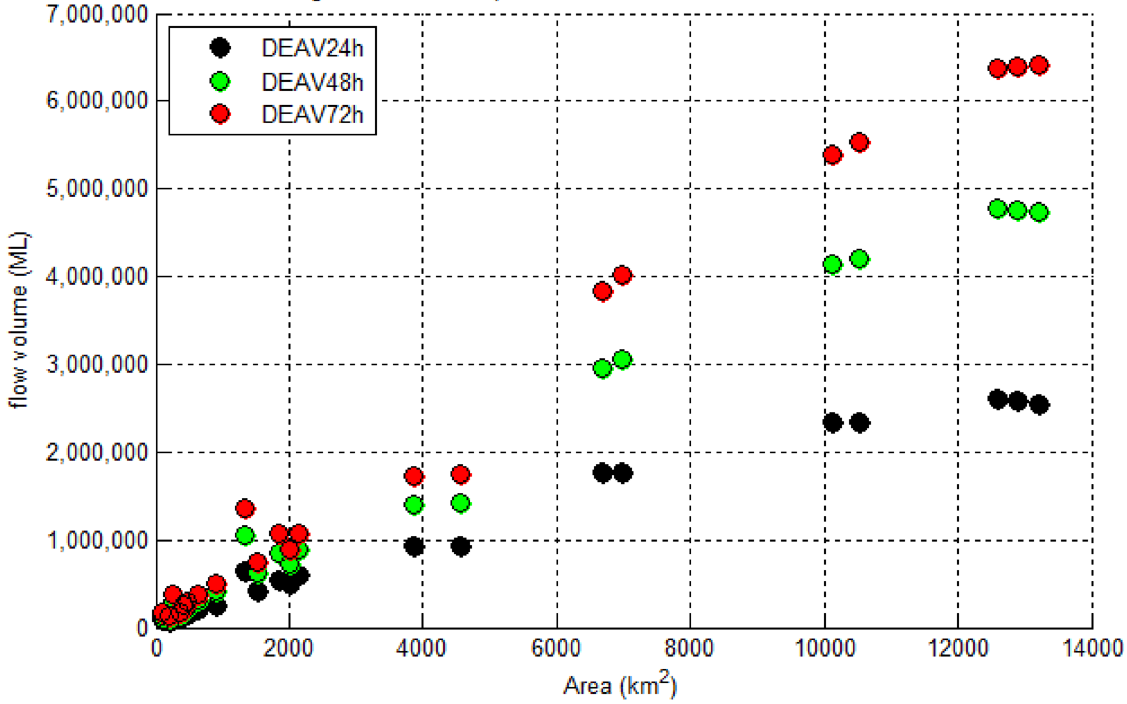


Figure C.3.4 | Area-plot for AEP = 1/10,000; no dams conditions



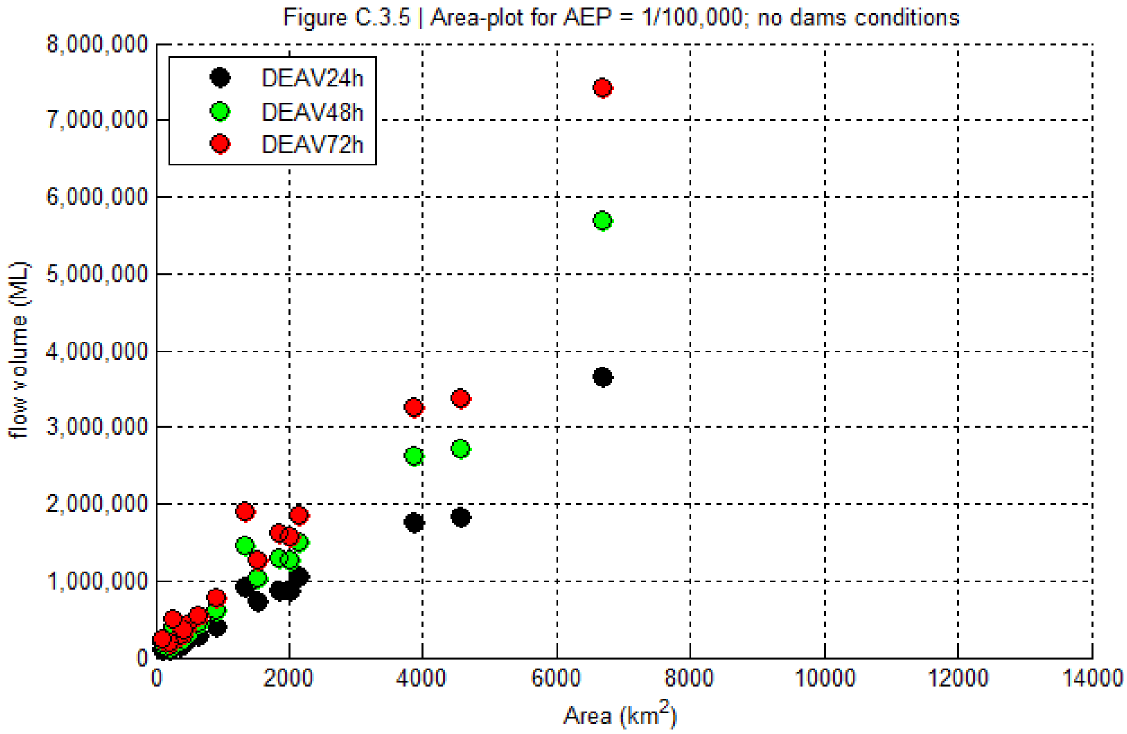


Figure C.3.5 | Area-plot for AEP = 1/100,000; no dams conditions
DEAV24h
DEAV48h
DEAV72h
flow volume (ML)
Area (km2)

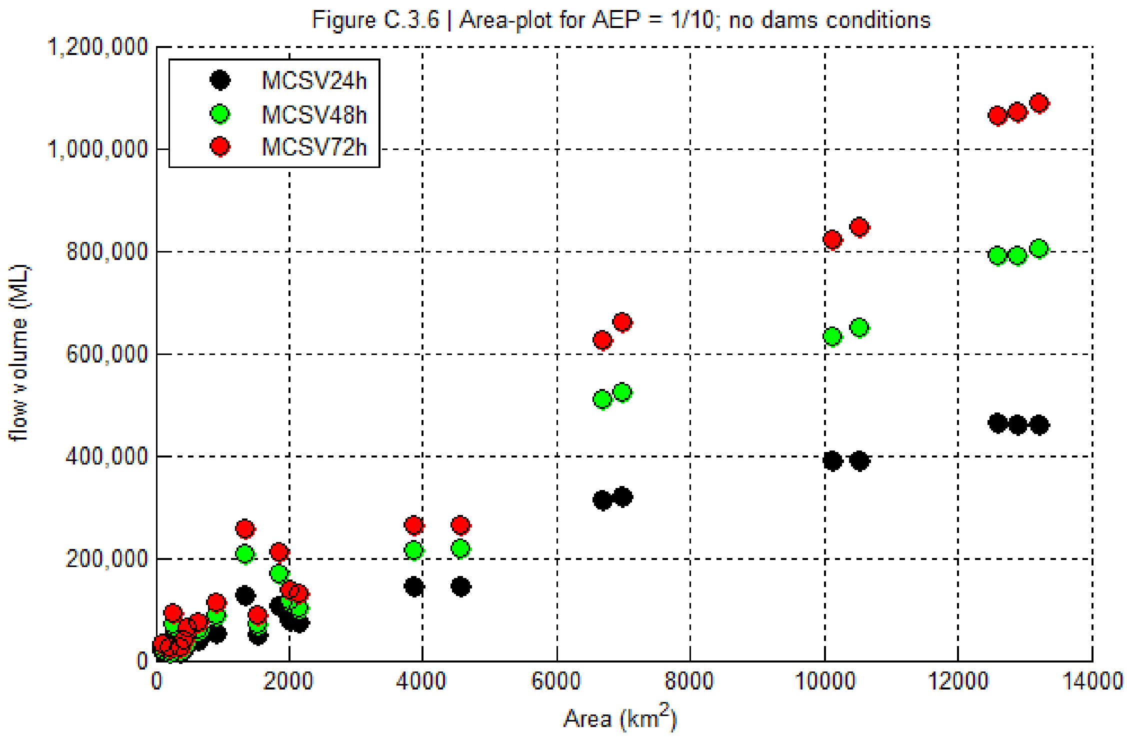


Figure C.3.6 | Area-plot for AEP = 1/10; no dams conditions
MCSV24h
MCSV48h
MCSV72h
flow volume (ML)
Area (km2)

Figure C.3.7 | Area-plot for AEP = 1/100; no dams conditions

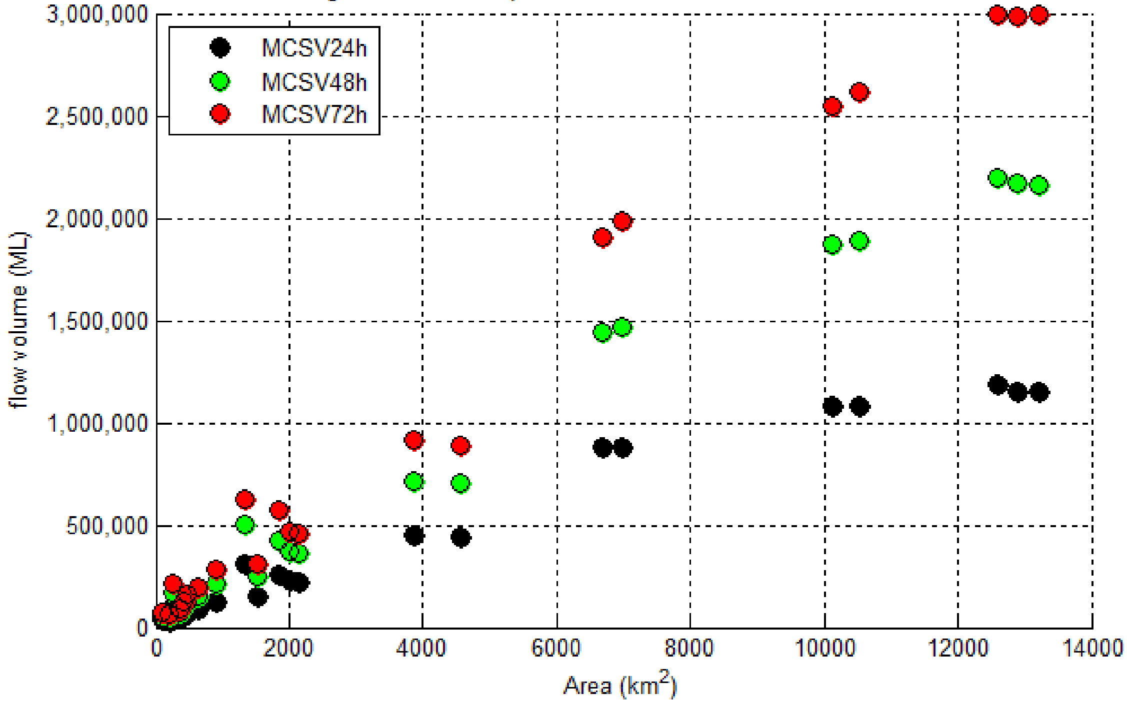
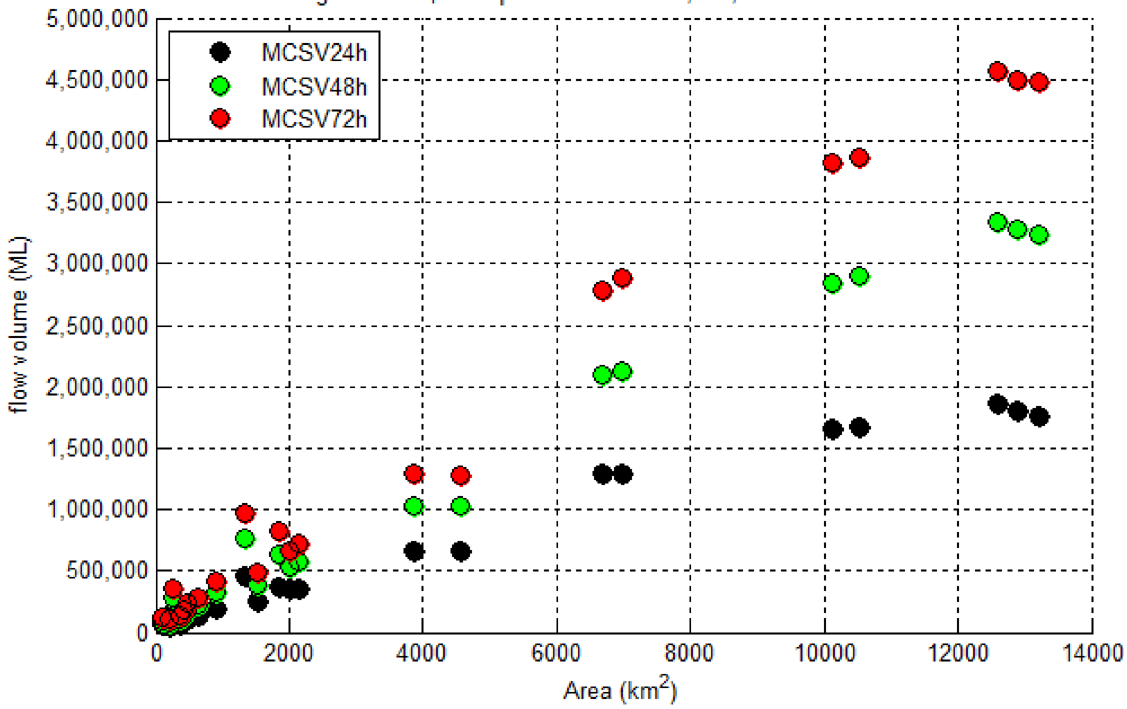


Figure C.3.8 | Area-plot for AEP = 1/1,000; no dams conditions



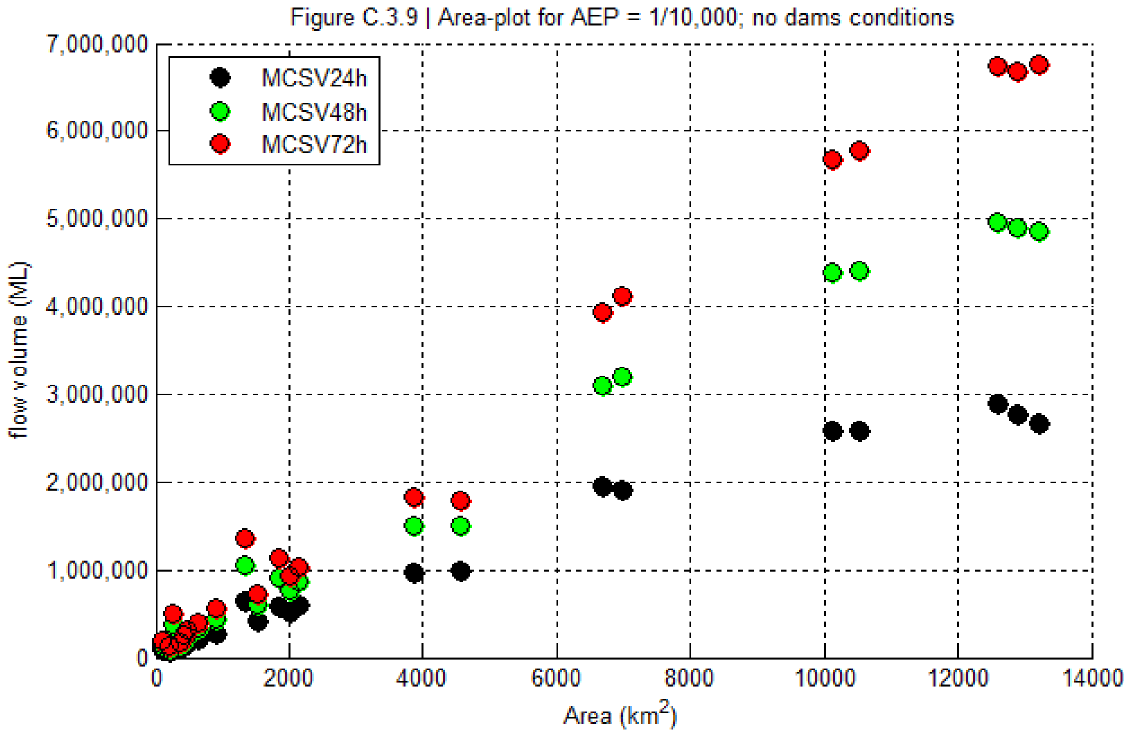


Figure C.3.9 | Area-plot for AEP = 1/10,000; no dams conditions
MCSV24h
MCSV48h
MCSV72h
flow volume (ML)
Area (km^2)
0
1,000,000
2,000,000
3,000,000
4,000,000
5,000,000
6,000,000
7,000,000
2000
4000
6000
8000
10000
12000
14000

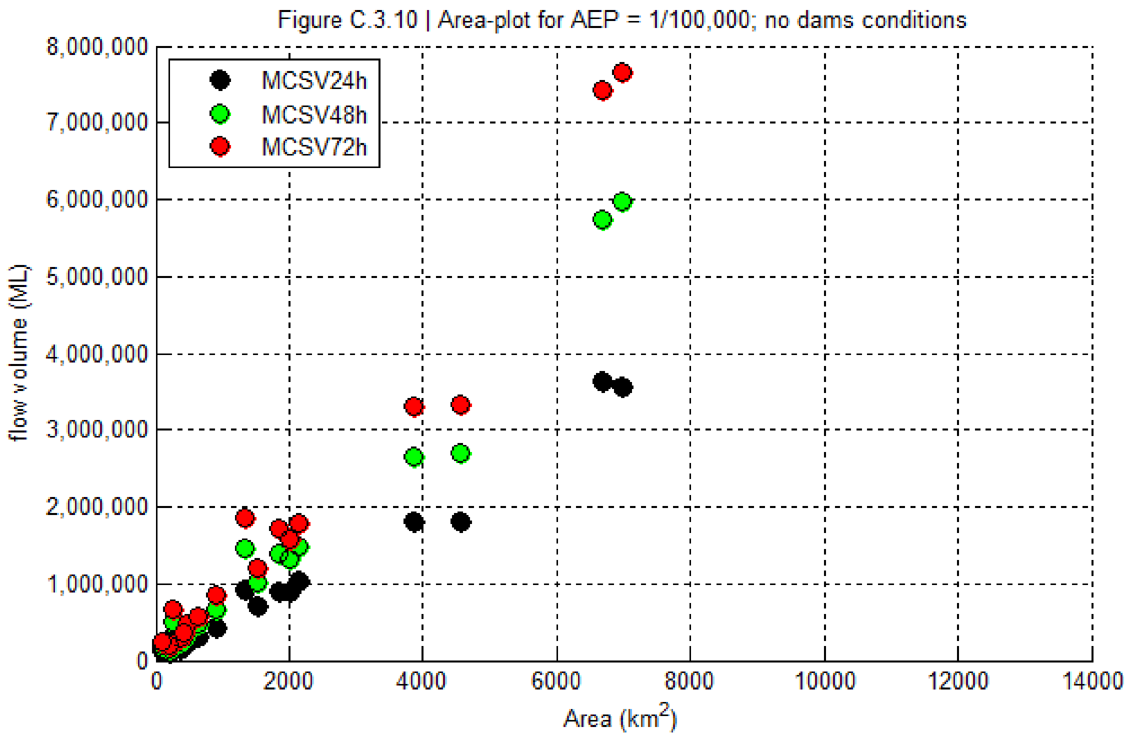


Figure C.3.10 | Area-plot for AEP = 1/100,000; no dams conditions
MCSV24h
MCSV48h
MCSV72h
flow volume (ML)
Area (km^2)
0
1,000,000
2,000,000
3,000,000
4,000,000
5,000,000
6,000,000
7,000,000
8,000,000
2000
4000
6000
8000
10000
12000
14000

C.4 Comparison with observed volumes for selected locations

Figure C.4.1 | Location Gregors Creek; no dams conditions; 24-hour flow volume

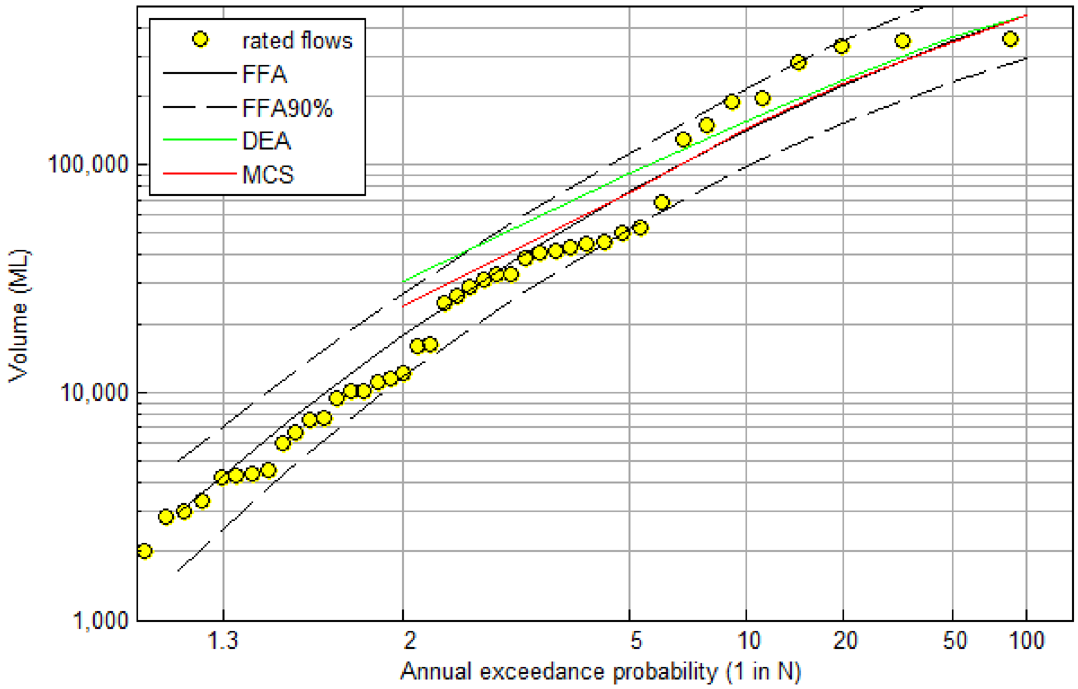


Figure C.4.2 | Location Gregors Creek; no dams conditions; 48-hour flow volume

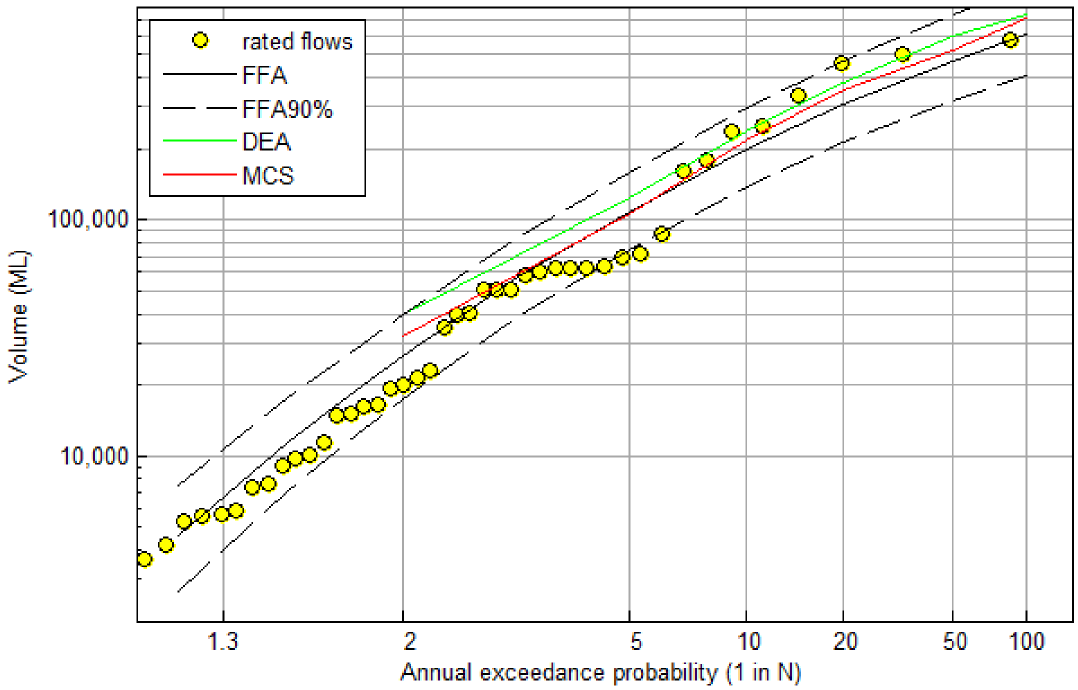


Figure C.4.3 | Location Gregors Creek; no dams conditions; 72-hour flow volume

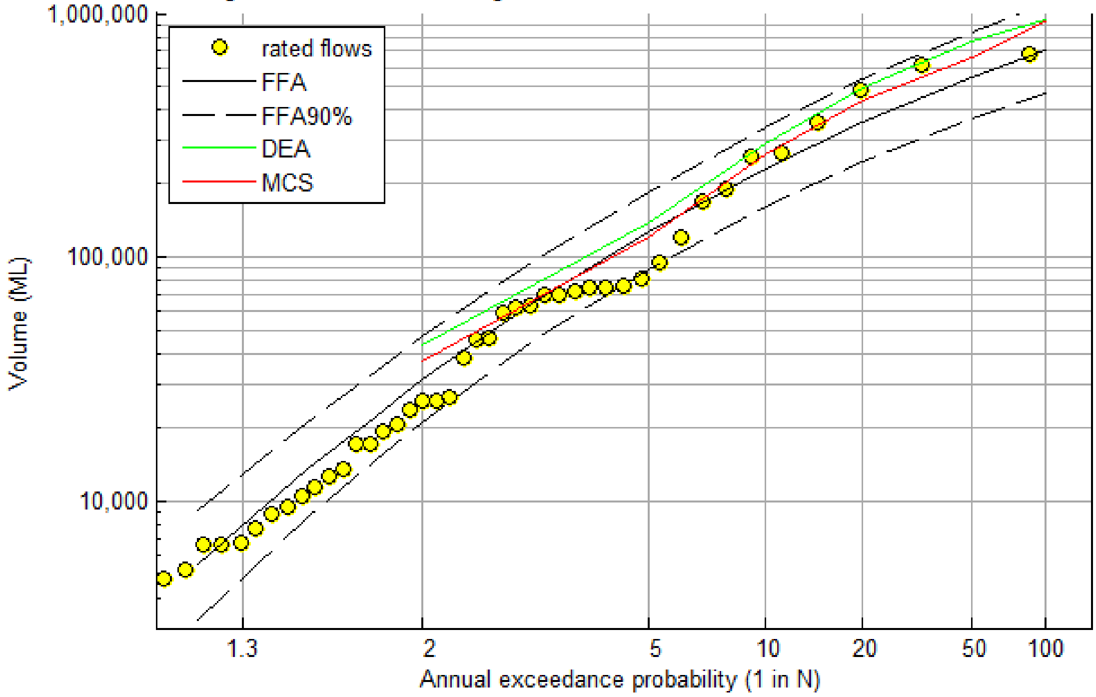


Figure C.4.4 | Location Linville; no dams conditions; 24-hour flow volume

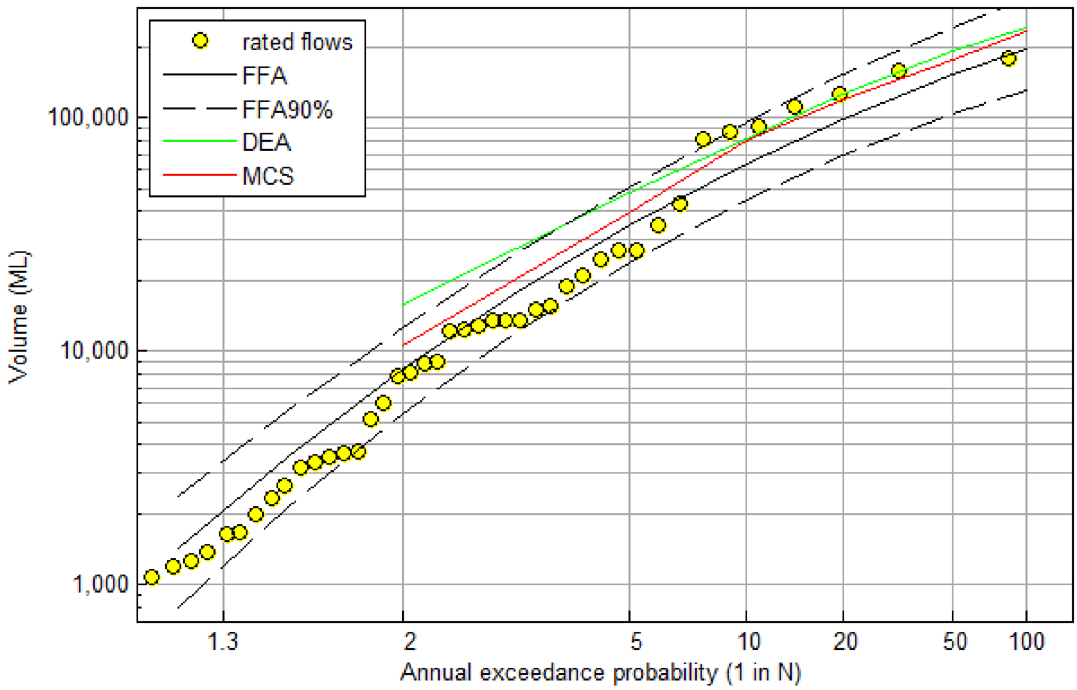


Figure C.4.5 | Location Linville; no dams conditions; 48-hour flow volume

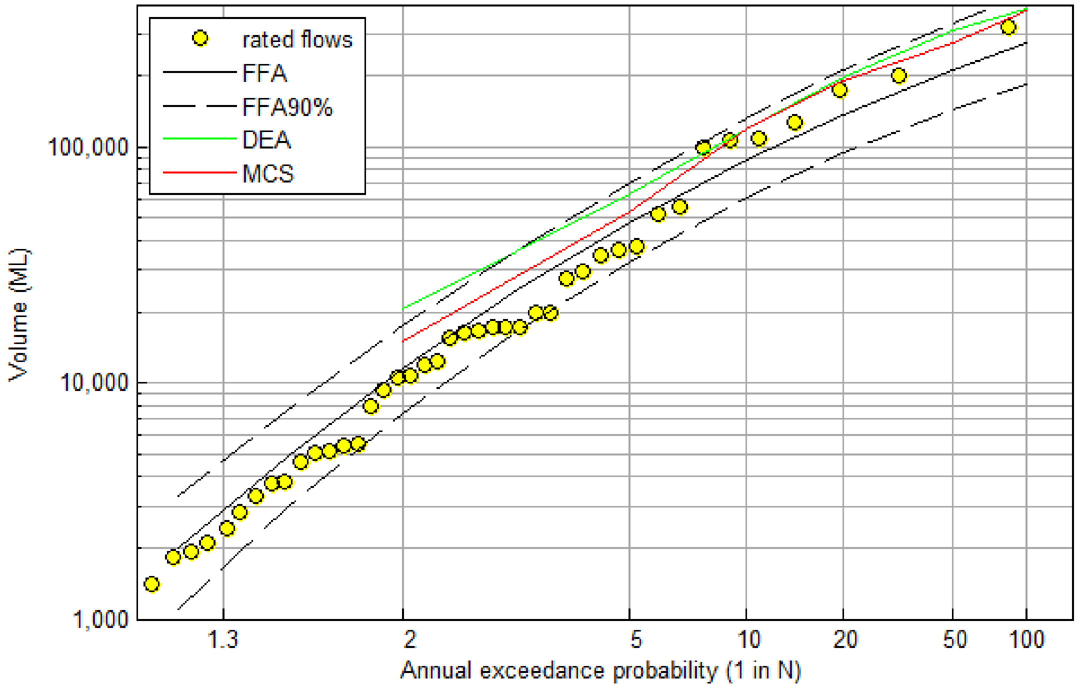


Figure C.4.6 | Location Linville; no dams conditions; 72-hour flow volume

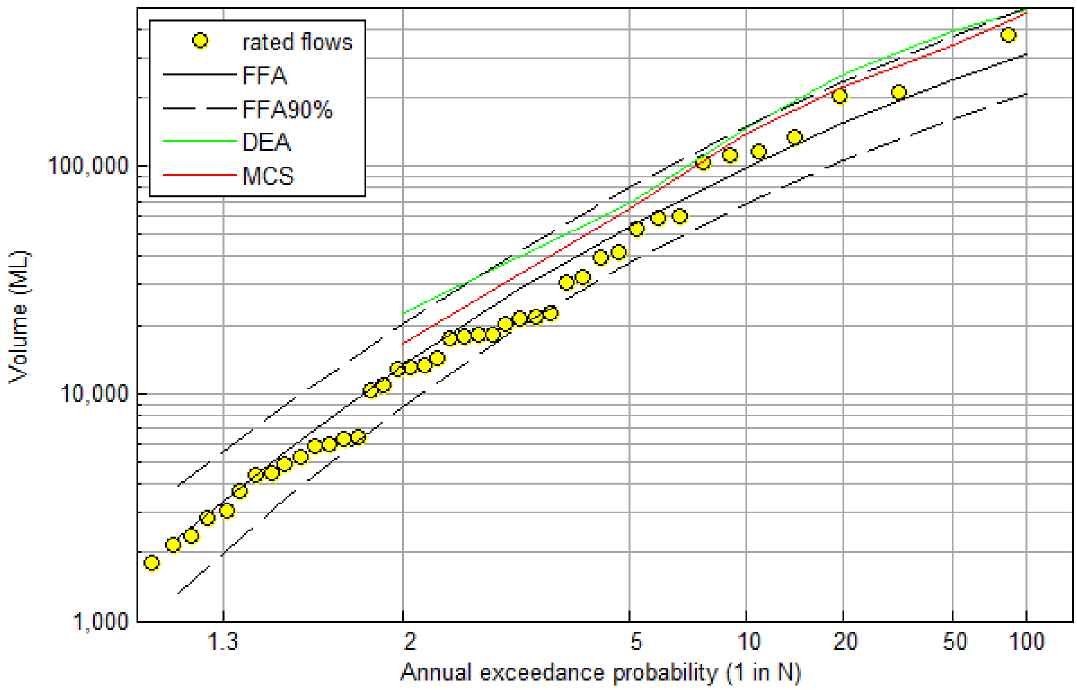


Figure C.4.7 | Location Walloon; no dams conditions; 24-hour flow volume

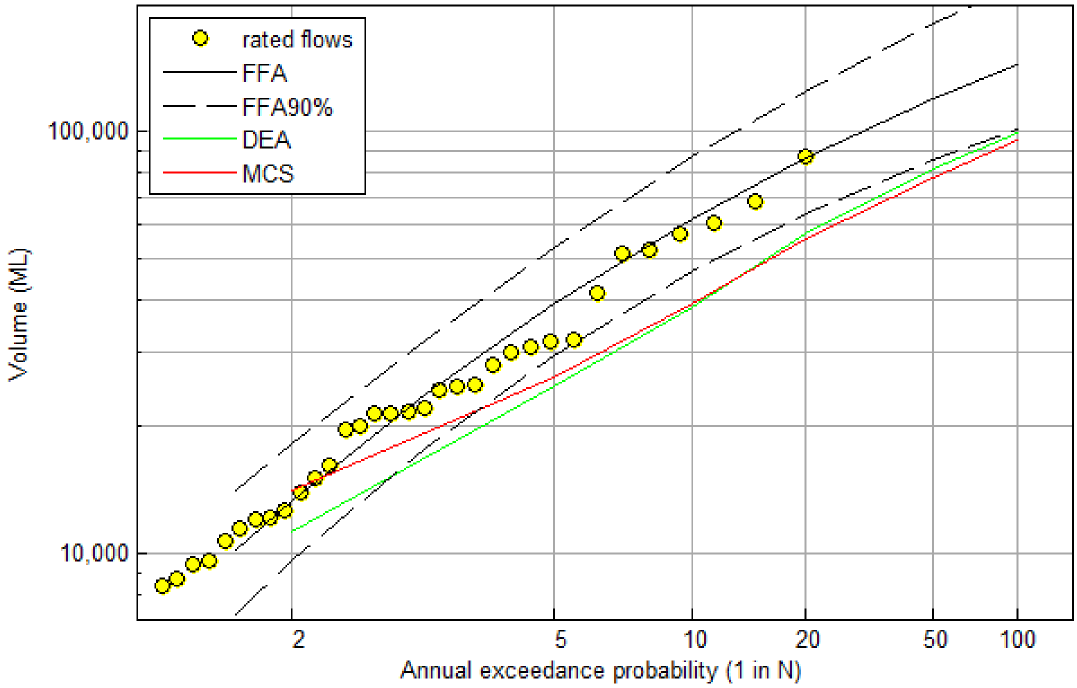


Figure C.4.8 | Location Walloon; no dams conditions; 48-hour flow volume

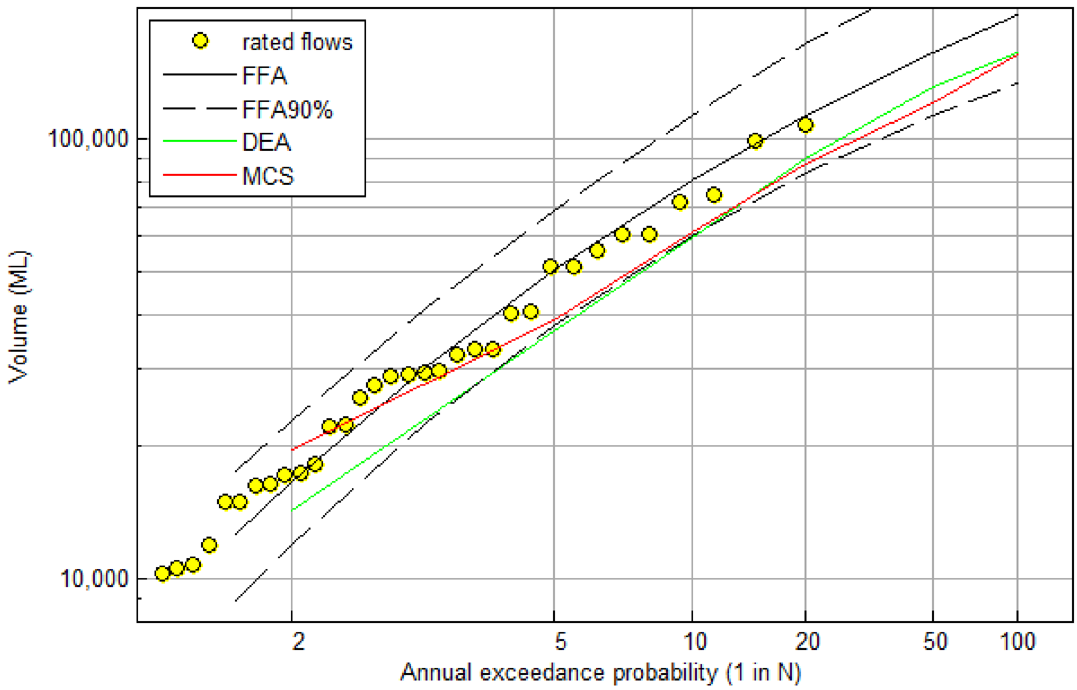
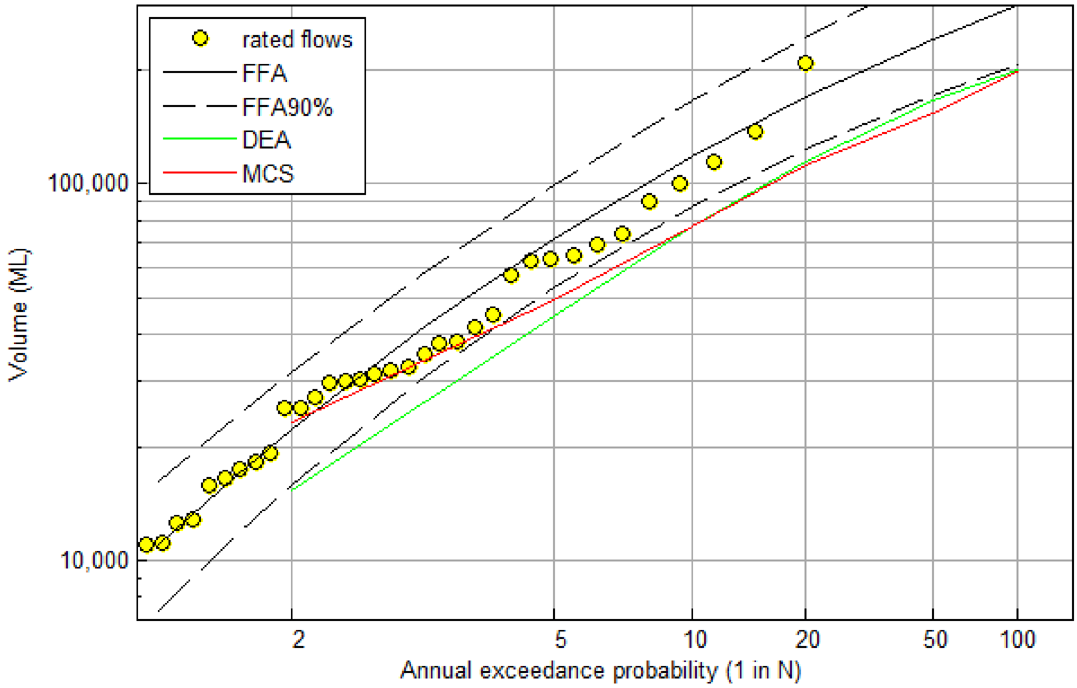


Figure C.4.9 | Location Walloon; no dams conditions; 72-hour flow volume



Appendix D
'With-dams' design flow volumes

D.1 Tables

Note: the 1 in 100,000 AEP flow volume is only provided for locations for which the AEP of the PMP is below 1 in 100,000.

Table D1 Maximum 24-hour flow volumes (1000ML) versus AEP; DEA results

Location	AEP (1 in N)											
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Somerset Dam													
Wivenhoe	11	62	110	190	350	520	690	860	970	1,100	1,500		2,900
Savages Crossing	11	89	190	310	530	740	950	1,200	1,400	1,500	2,100		4,700
Mount Crosby	11	91	190	320	540	750	960	1,200	1,400	1,500	2,100		4,700
Ipswich	25	58	90	130	190	220	260	300	340	380	490	810	1,200
Moggill	31	120	250	400	660	850	1,100	1,300	1,500	1,700	2,300		5,000
Centenary Bridge	33	120	250	410	650	830	1,000	1,300	1,500	1,600	2,300		5,000
Brisbane	37	120	260	420	660	840	1,000	1,300	1,400	1,600	2,200		4,800

Table D2 Maximum 48-hour flow volumes (1000ML) versus AEP; DEA results

Location	AEP (1 in N)											
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Somerset Dam													
Wivenhoe	22	120	210	360	670	920	1,200	1,500	1,700	2,000	2,500		5,000
Savages Crossing	21	170	340	580	990	1,300	1,600	2,000	2,400	2,700	3,500		7,900
Mount Crosby	21	170	360	600	1,000	1,400	1,700	2,100	2,400	2,700	3,600		8,000
Ipswich	35	95	150	220	320	390	450	520	580	650	810	1,200	1,700
Moggill	49	220	450	760	1,200	1,600	1,900	2,400	2,700	3,100	4,100		9,200

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Centenary Bridge	54	220	460	770	1,200	1,600	1,900	2,400	2,700	3,100	4,100		9,200
Brisbane	61	230	470	790	1,300	1,600	2,000	2,400	2,700	3,100	4,100		9,000

Table D3 Maximum 72-hour flow volumes (1000ML) versus AEP; DEA results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Somerset Dam													
Wivenhoe	32	170	300	520	970	1,300	1,600	2,000	2,300	2,600	3,400		7,100
Savages Crossing	31	230	460	790	1,400	1,800	2,200	2,700	3,100	3,500	4,600		11,000
Mount Crosby	30	230	470	820	1,400	1,900	2,300	2,800	3,200	3,700	4,800		11,000
Ipswich	39	120	200	300	430	530	600	690	760	840	1,000	1,600	2,200
Moggill	62	300	610	1,000	1,800	2,300	2,700	3,300	3,700	4,200	5,600		12,000
Centenary Bridge	67	310	630	1,100	1,800	2,300	2,700	3,300	3,700	4,200	5,600		13,000
Brisbane	77	330	650	1,100	1,800	2,300	2,800	3,300	3,800	4,300	5,600		12,000

Table D4 Maximum 24-hour flow volumes (1000ML) versus AEP; MCS results

Location	AEP (1 in N)												
	2	5	10	20	50	100	200	500	1,000	2,000	10,000	100,000	AEP of PMP
Somerset Dam	0	94	120	150	170	180	220	270	300	330	430	830	1,600
Wivenhoe	0	40	78	150	270	480	640	830	950	1,100	1,500	2,800	3,200
Savages Crossing	22	91	170	260	450	670	920	1,200	1,300	1,500	2,100		4,300
Mount Crosby	26	94	170	280	460	670	910	1,200	1,300	1,500	2,100		4,300

Location	AEP (1 in N)												
	2	**5**	**10**	**20**	**50**	**100**	**200**	**500**	**1,000**	**2,000**	**10,000**	**100,000**	**AEP of PMP**
Ipswich	30	67	95	130	180	220	260	300	330	380	520	800	1,100
Moggill	61	150	250	370	580	840	1,000	1,300	1,500	1,700	2,400		4,800
Centenary Bridge	64	160	260	380	590	820	990	1,200	1,400	1,600	2,300		4,700
Brisbane	67	170	260	390	620	820	980	1,200	1,400	1,600	2,200		4,600

Table D5 Maximum 48-hour flow volumes (1000ML) versus AEP; MCS results

Location	AEP (1 in N)												
	2	**5**	**10**	**20**	**50**	**100**	**200**	**500**	**1,000**	**2,000**	**10,000**	**100,000**	**AEP of PMP**
Somerset Dam	0	130	180	210	270	300	350	450	490	550	730	1,400	2,700
Wivenhoe	0	75	150	280	500	830	1,100	1,300	1,600	1,800	2,500	5,000	5,700
Savages Crossing	39	150	290	450	770	1,100	1,500	1,900	2,200	2,600	3,600		7,600
Mount Crosby	44	160	290	460	800	1,200	1,500	2,000	2,200	2,600	3,600		7,600
Ipswich	44	98	150	210	300	390	440	510	580	640	810	1,200	1,700
Moggill	96	250	430	670	1,000	1,500	1,900	2,300	2,600	3,000	4,200		8,800
Centenary Bridge	100	260	450	690	1,100	1,500	1,800	2,300	2,600	3,000	4,200		8,700
Brisbane	110	270	470	710	1,100	1,500	1,800	2,300	2,500	3,000	4,200		8,700

Table D6 Maximum 72-hour flow volumes (1000ML) versus AEP; MCS results

Location	AEP (1 in N)											
	2	**5**	**10**	**20**	**50**	**100**	**200**	**500**	**1,000**	**2,000**	**10,000**	**100,000**	**AEP of PMP**
Somerset Dam	0	150	200	260	320	380	460	590	600	680	980	1,900	3,600
Wivenhoe	0	110	210	390	670	1,100	1,400	1,700	2,000	2,300	3,300	6,800	7,700
Savages Crossing	52	200	360	610	1,100	1,500	2,000	2,500	2,800	3,300	4,600		10,000
Mount Crosby	59	200	390	640	1,100	1,600	2,000	2,600	3,000	3,400	4,700		10,000
Ipswich	53	120	190	280	400	490	560	650	740	820	1,000	1,600	2,200
Moggill	120	330	570	900	1,400	2,100	2,500	3,100	3,500	4,100	5,700		12,000
Centenary Bridge	130	340	590	950	1,400	2,100	2,500	3,100	3,500	4,100	5,700		12,000
Brisbane	140	360	610	970	1,500	2,100	2,600	3,100	3,500	4,100	5,700		12,000

D.2 Frequency plots

Figure D.2.1 | Location Somerset Dam; with dams conditions

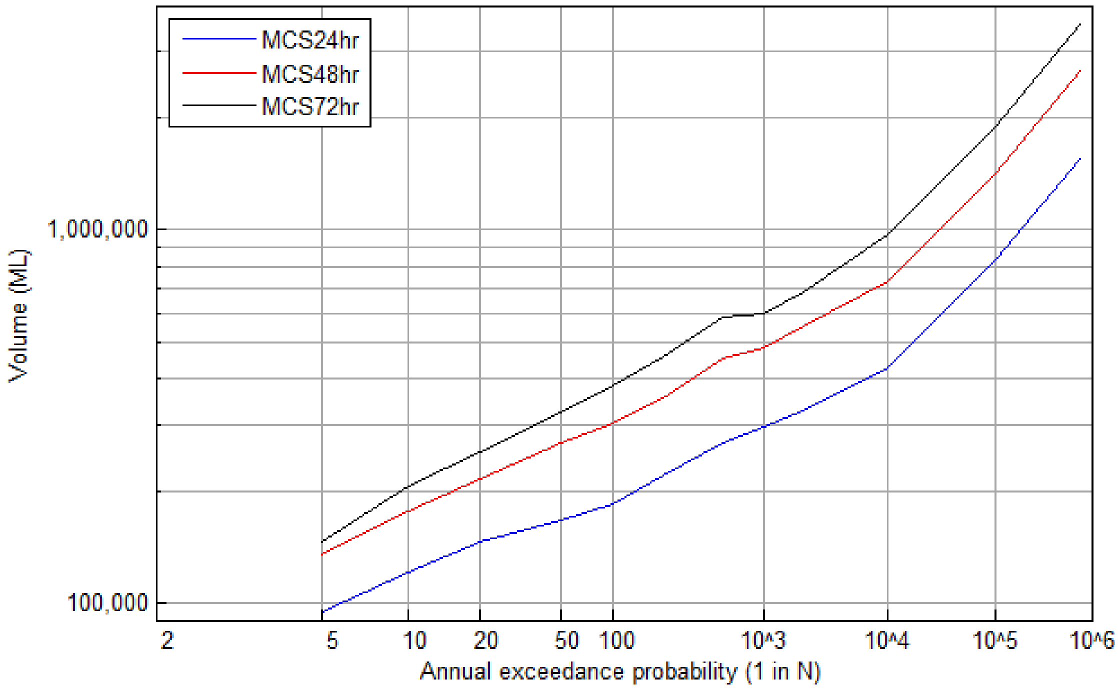


Figure D.2.2 | Location Wivenhoe; with dams conditions

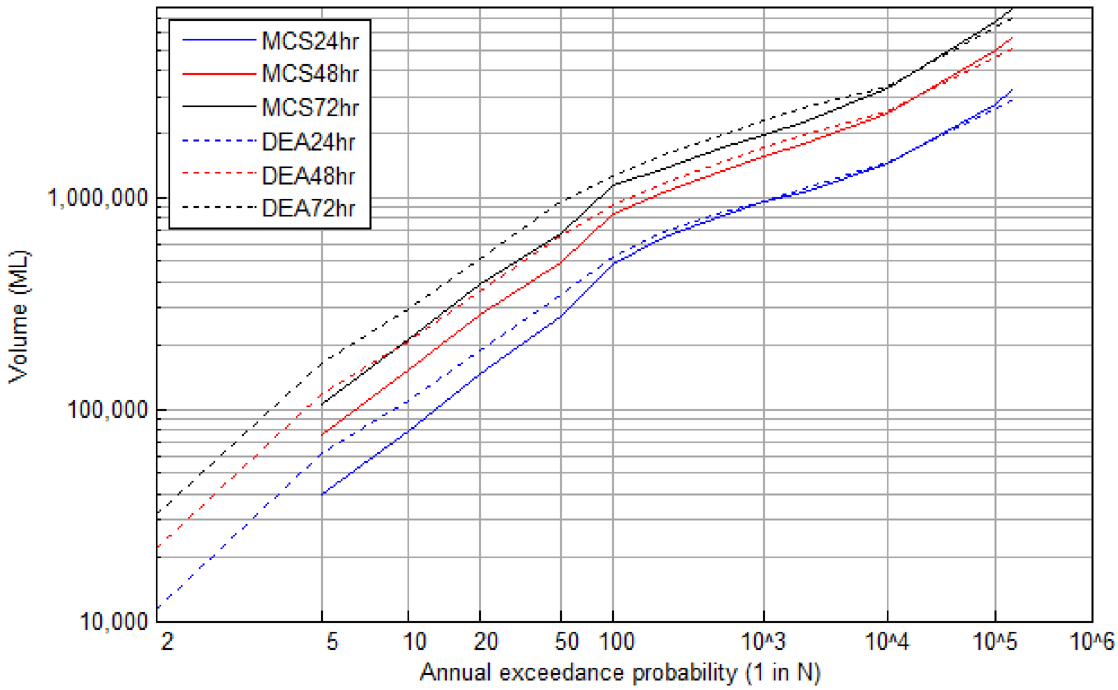


Figure D.2.3 | Location Savages Crossing; with dams conditions

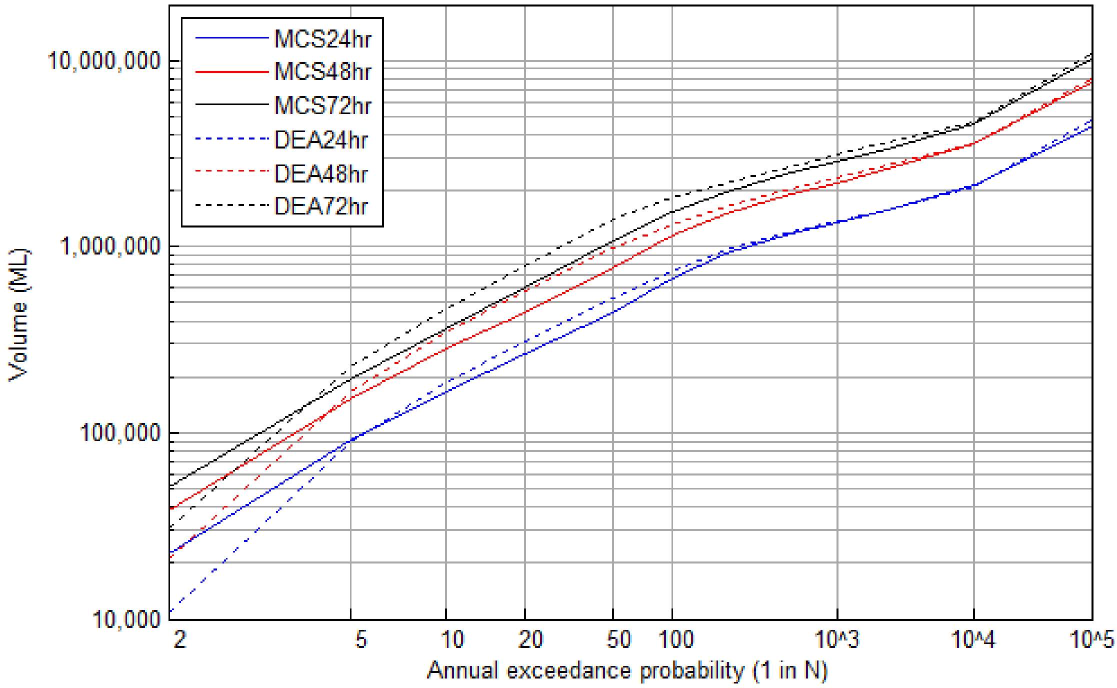


Figure D.2.4 | Location Mount Crosby; with dams conditions

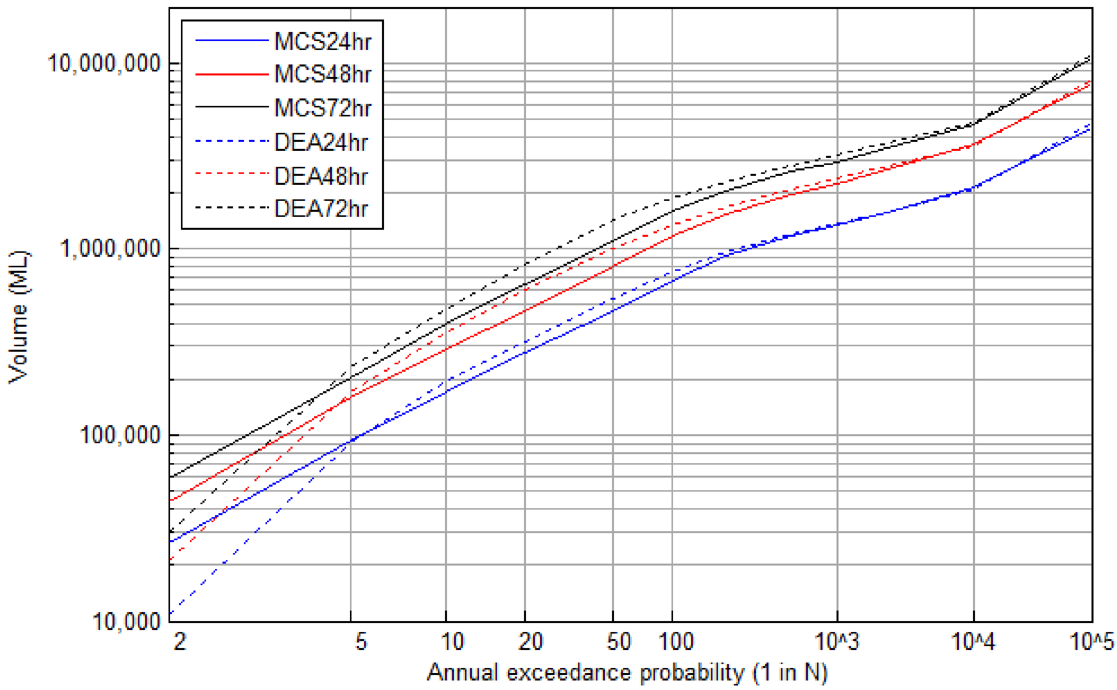


Figure D.2.5 | Location Ipswich; with dams conditions

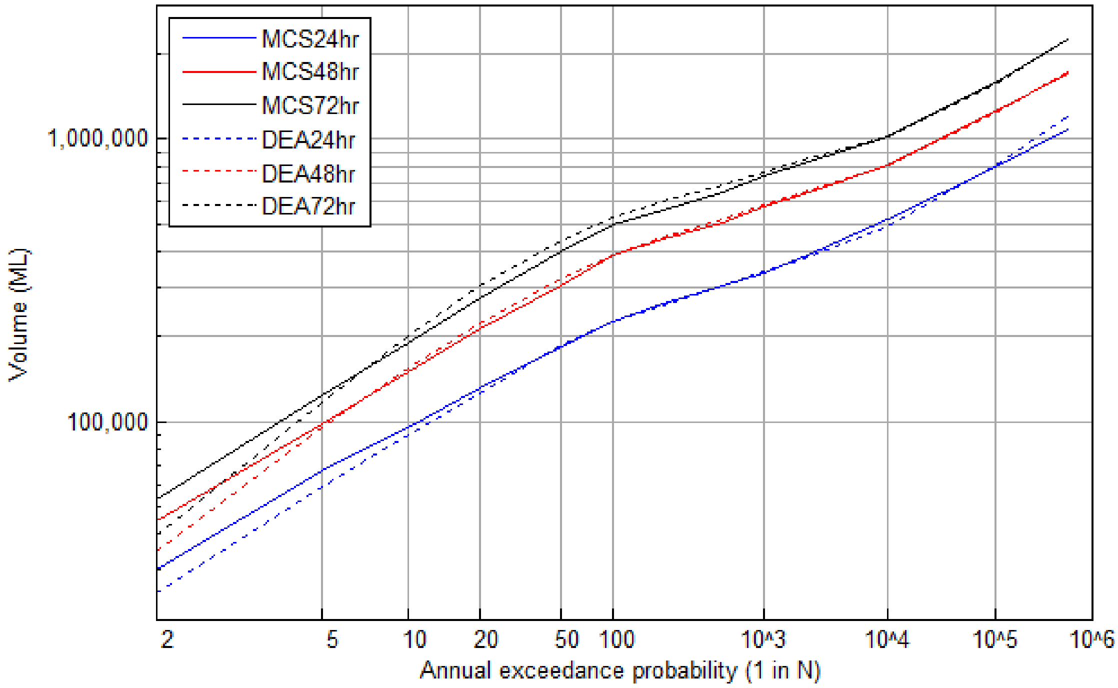


Figure D.2.6 | Location Moggill; with dams conditions

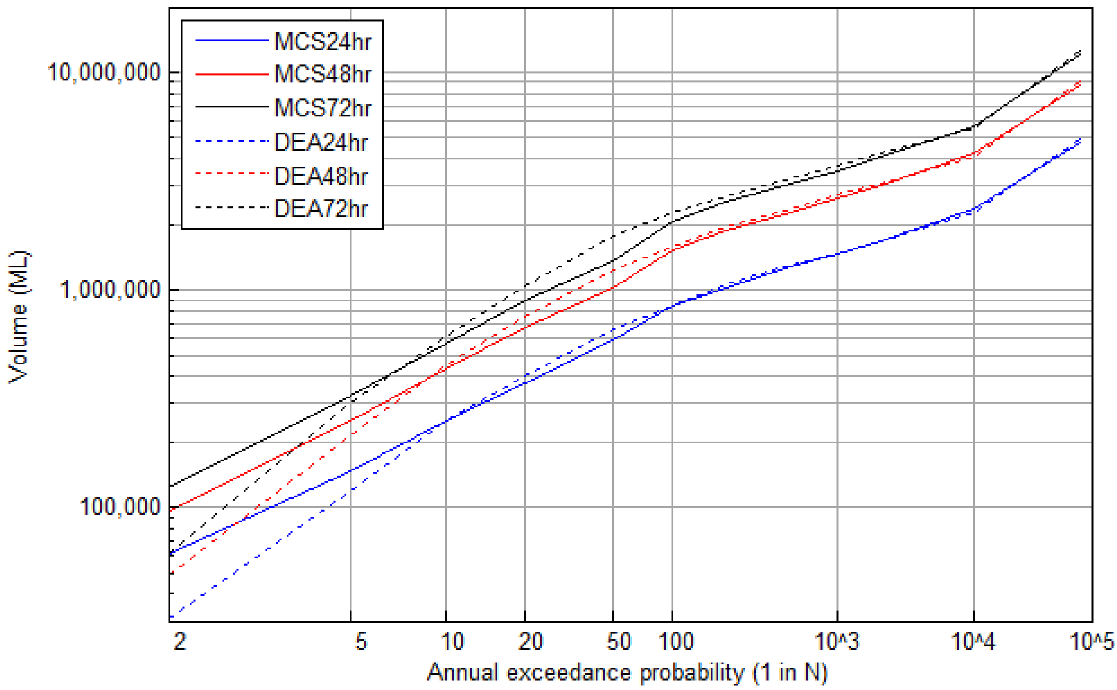


Figure D.2.7 | Location Centenary Bridge; with dams conditions

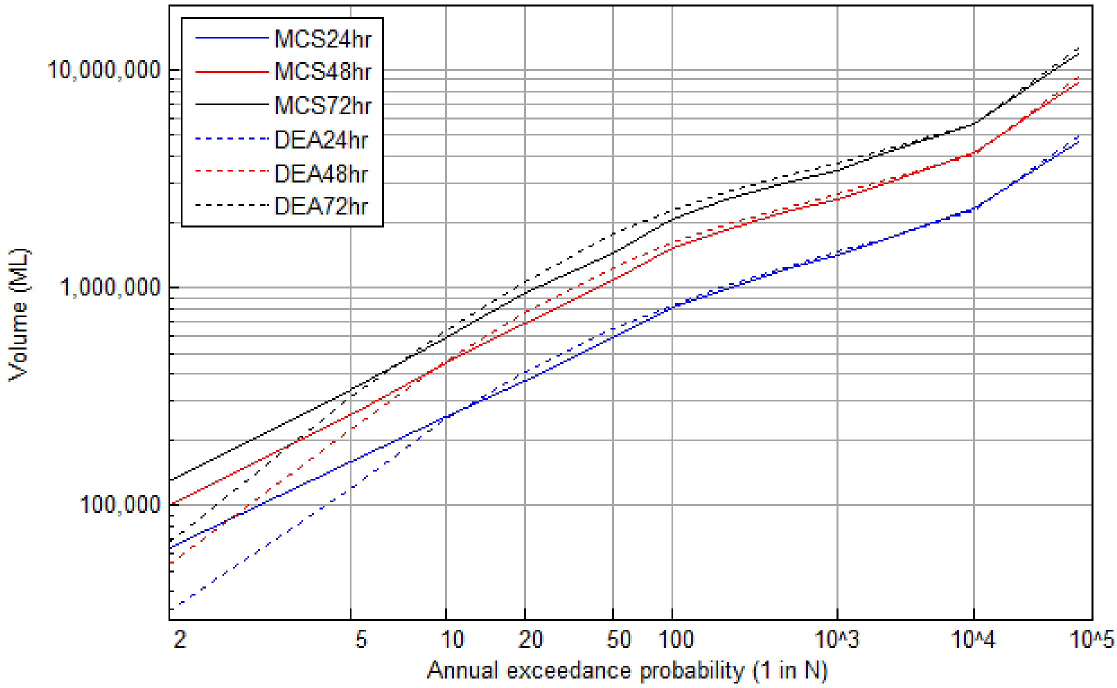
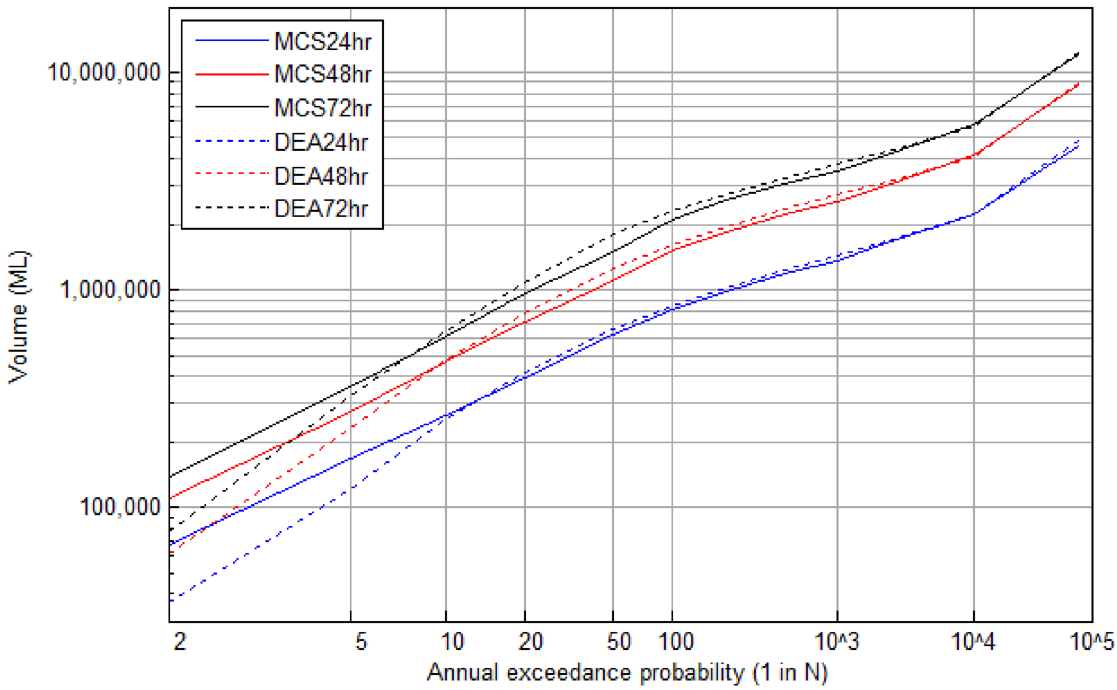


Figure D.2.8 | Location Brisbane; with dams conditions



D.3 Plots of catchment area versus flow volumes

This section contains nine figures in which flows 24-, 48- and 72 hour flow volumes are plotted against catchment area: four for DEA results (one figure per AEP) and five for MCS results.

Figure D.3.1 | Area-plot for AEP = 1/10; with dams conditions

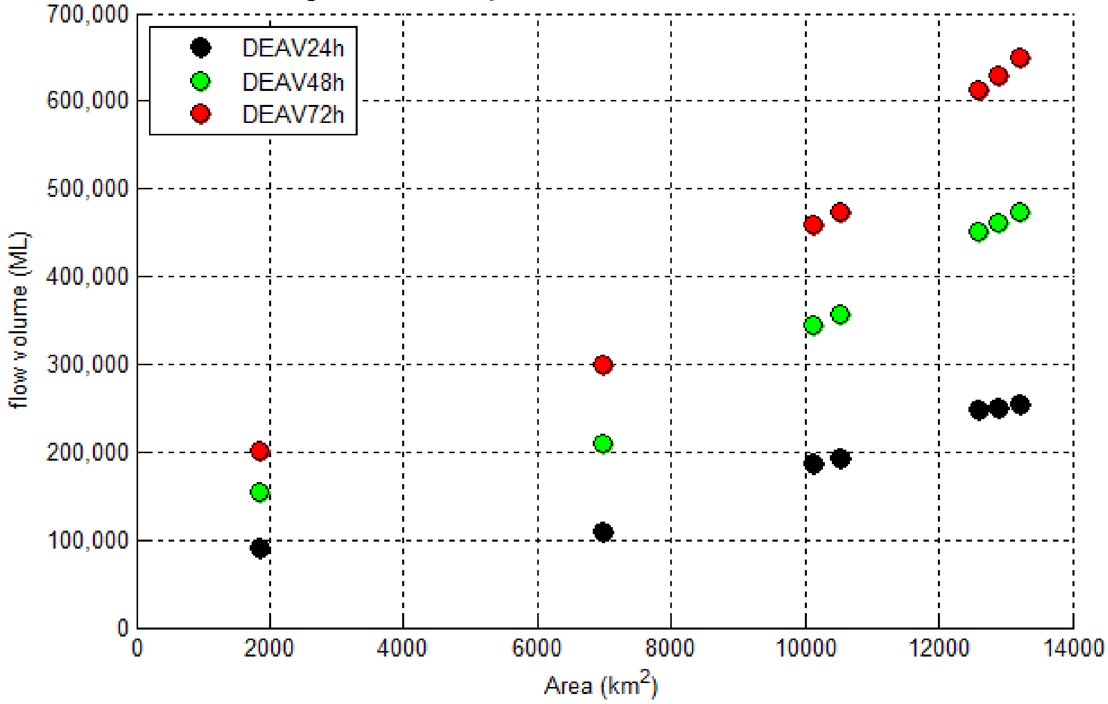


Figure D.3.2 | Area-plot for AEP = 1/100; with dams conditions

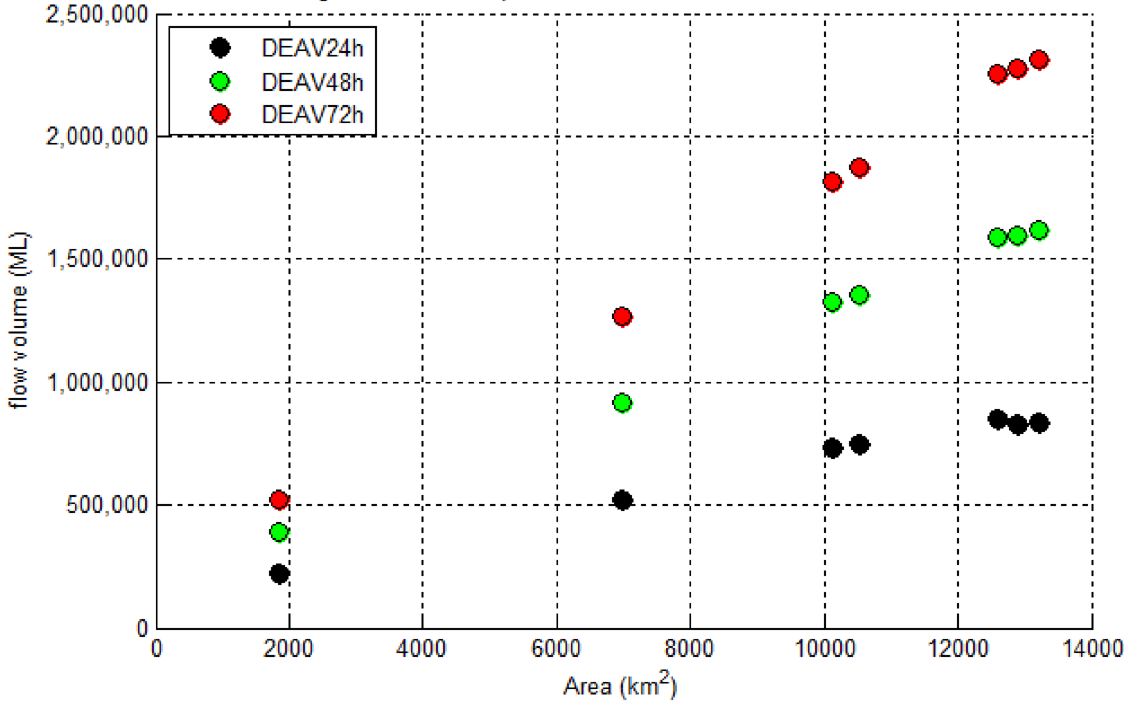


Figure D.3.3 | Area-plot for AEP = 1/1,000; with dams conditions

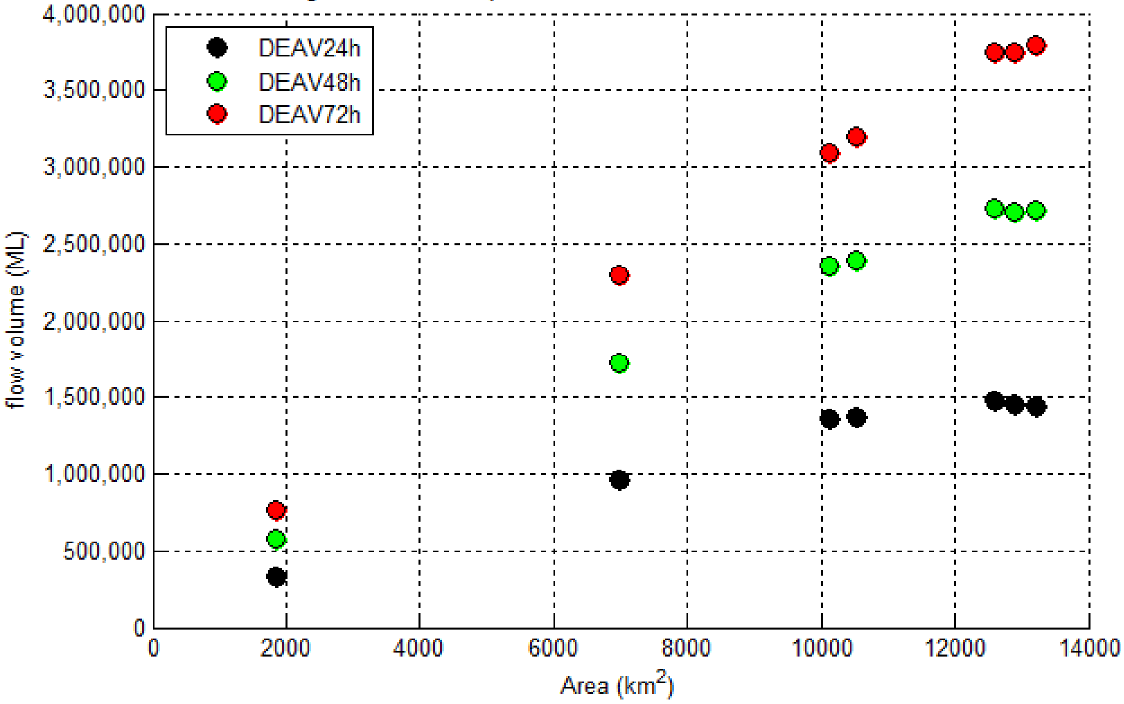
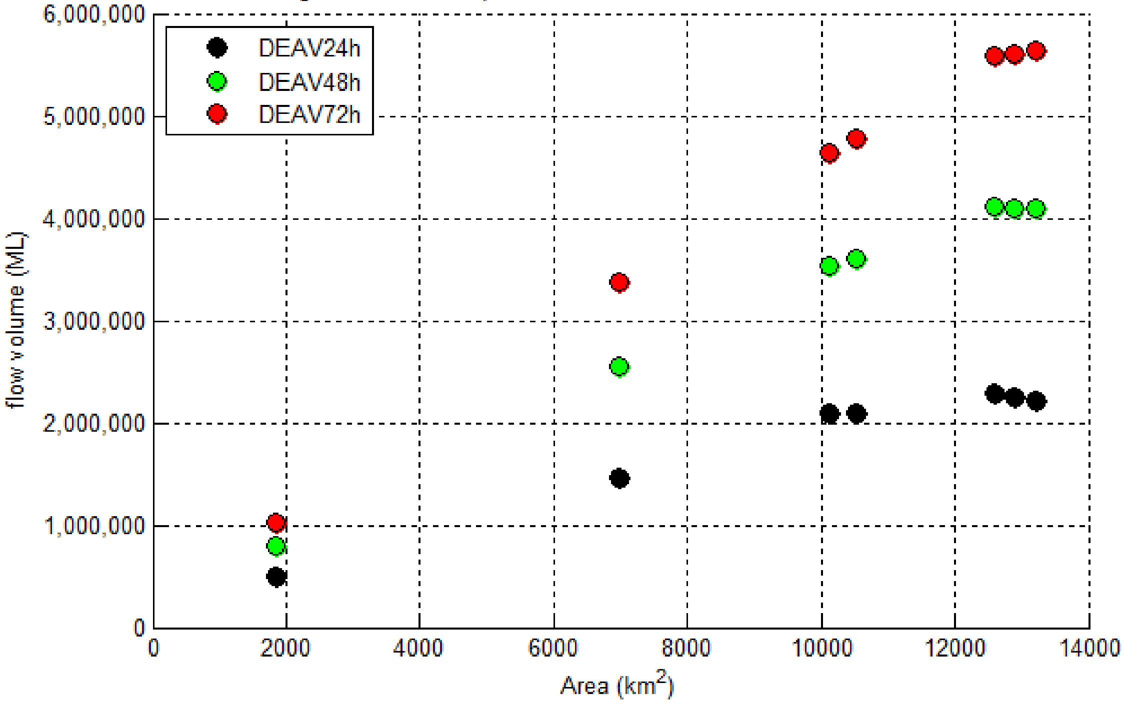


Figure D.3.4 | Area-plot for AEP = 1/10,000; with dams conditions



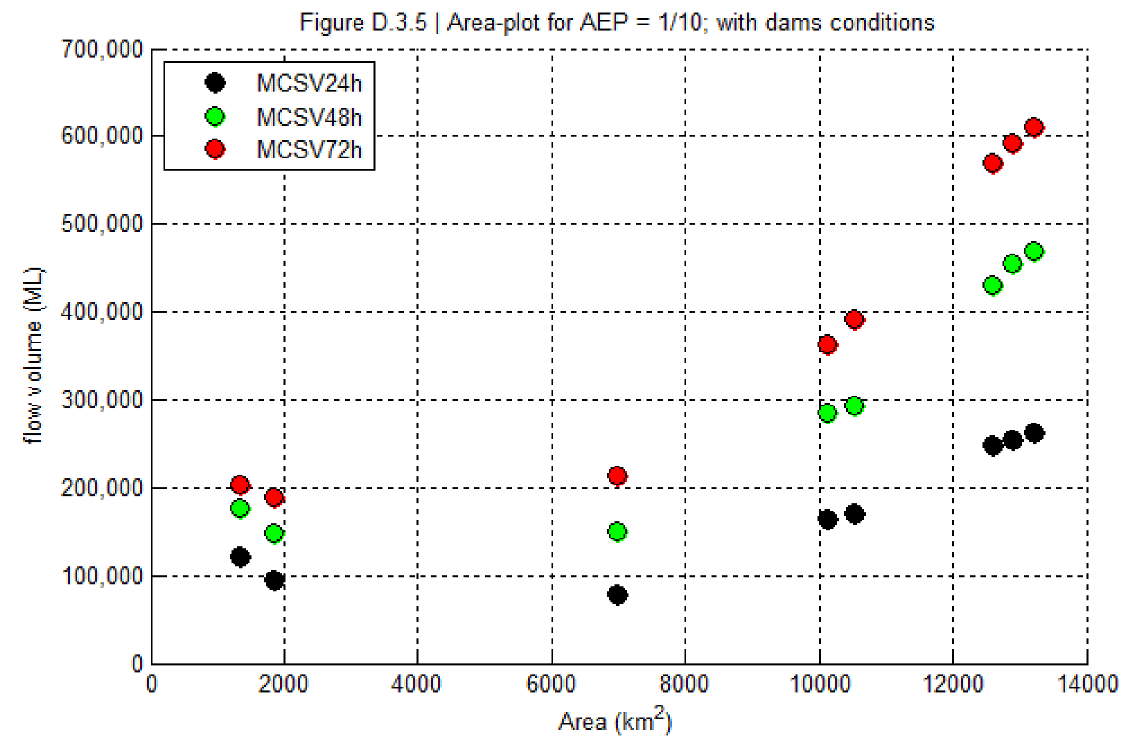


Figure D.3.5 | Area-plot for AEP = 1/10; with dams conditions
MCSV24h
MCSV48h
MCSV72h
flow volume (ML)
700,000
600,000
500,000
400,000
300,000
200,000
100,000
0
0
2000
4000
6000
8000
10000
12000
14000
Area (km2)

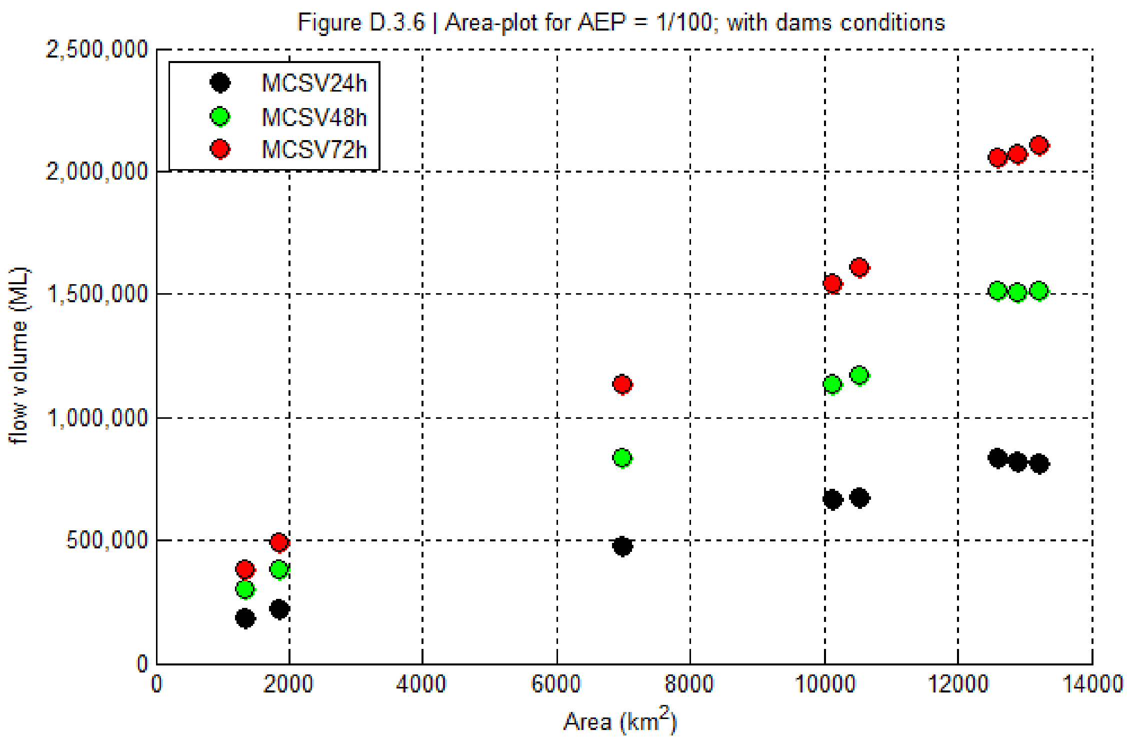


Figure D.3.6 | Area-plot for AEP = 1/100; with dams conditions
MCSV24h
MCSV48h
MCSV72h
flow volume (ML)
2,500,000
2,000,000
1,500,000
1,000,000
500,000
0
0
2000
4000
6000
8000
10000
12000
14000
Area (km2)

Figure D.3.7 | Area-plot for AEP = 1/1,000; with dams conditions

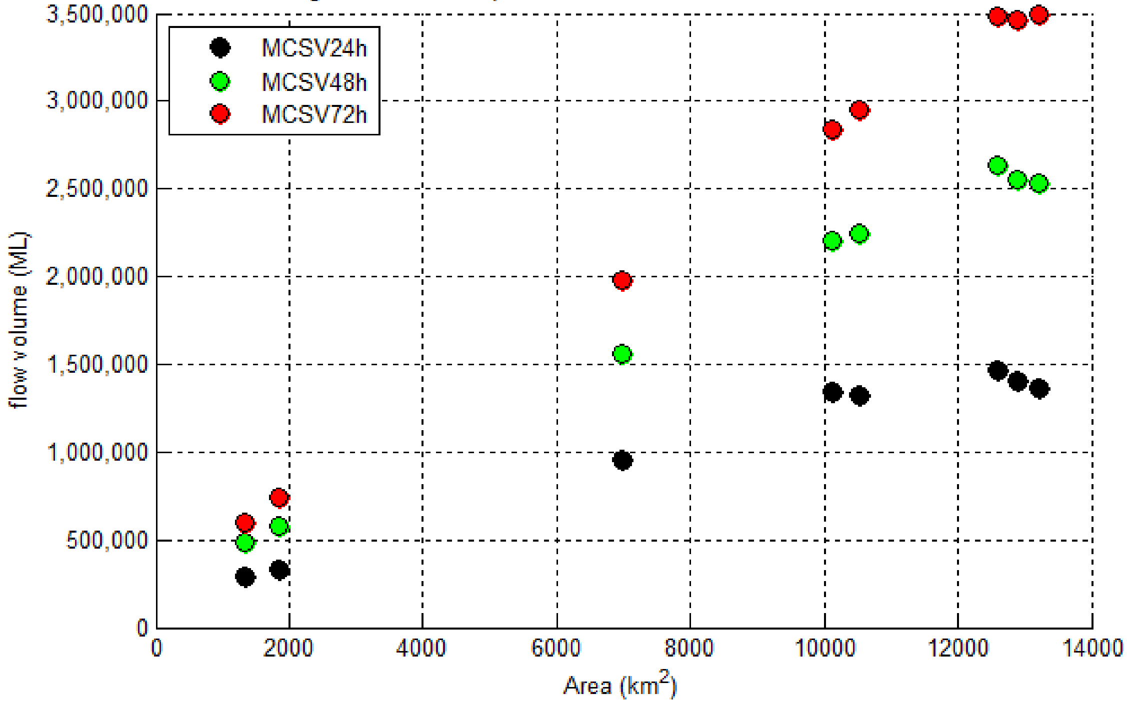
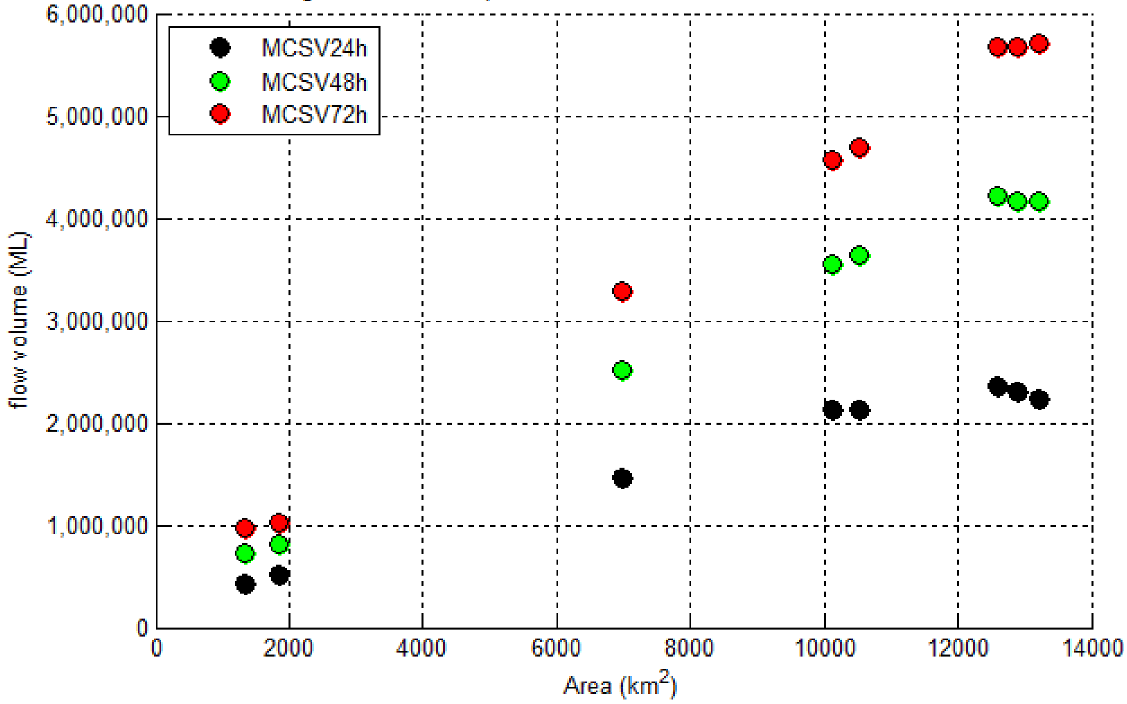


Figure D.3.8 | Area-plot for AEP = 1/10,000; with dams conditions



Appendix E
Comparison of 'with-dams' and 'no-dams' results

This appendix contains Figures in which ‘with-dams’ results are compared to ‘no-dams’ results. These Figures are provided for locations Ipswich, Somerset Dam and Wivenhoe Dam and the five Lower Brisbane locations. The following Figures are provided for each location:

1. Frequency plots for ‘no dam’ peak flows and ‘with dam’ peak flows
2. Scatter plots of simulated ‘no dam’ peak flows versus ‘with dam’ flows
3. Frequency plots for ‘no dam’ flow volumes and ‘with dam’ flow volumes

No scatter plot is provided for location Somerset Dam, for reasons explained in section 6.4. This appendix contains the following sections:

E1. MCS frequency plots of peak flows
E2. MCS scatter plots for peak flows
E3. MCS frequency plots of flow volumes
E4. DEA Frequency plots of peak flows
E5. DEA scatter plots for peak flows
E6. DEA frequency plots of flow volumes

E.1 MCS frequency plots of peak flows

Figure E.1.1 | Location Somerset Dam; MCS simulation results

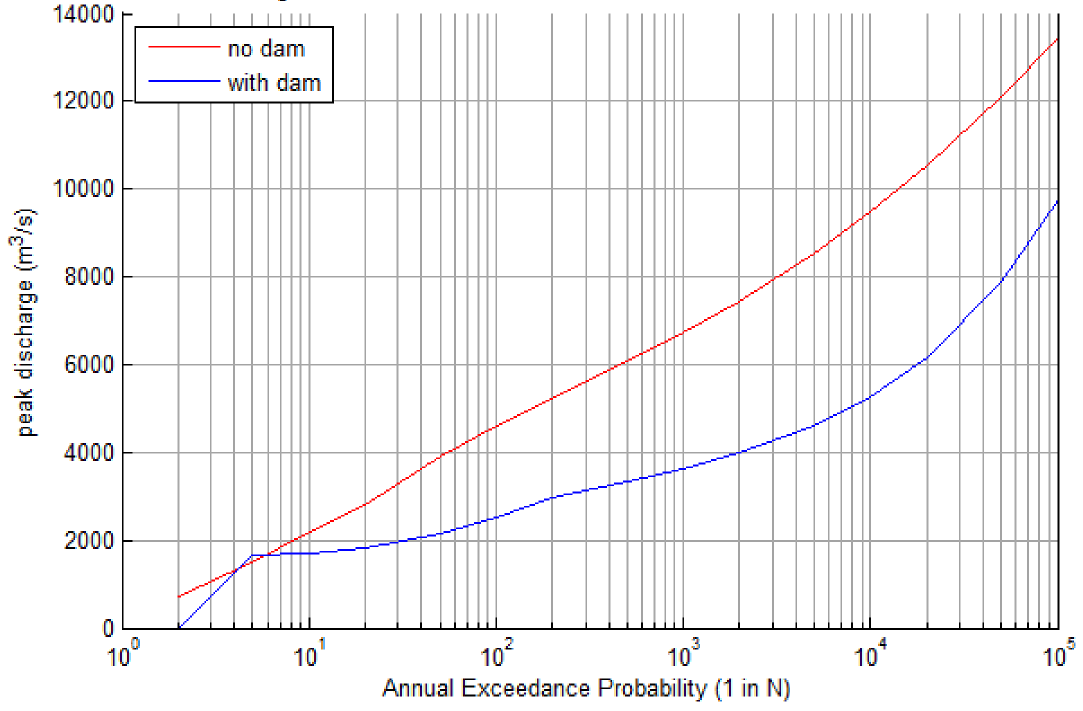


Figure E.1.2 | Location Wivenhoe; MCS simulation results

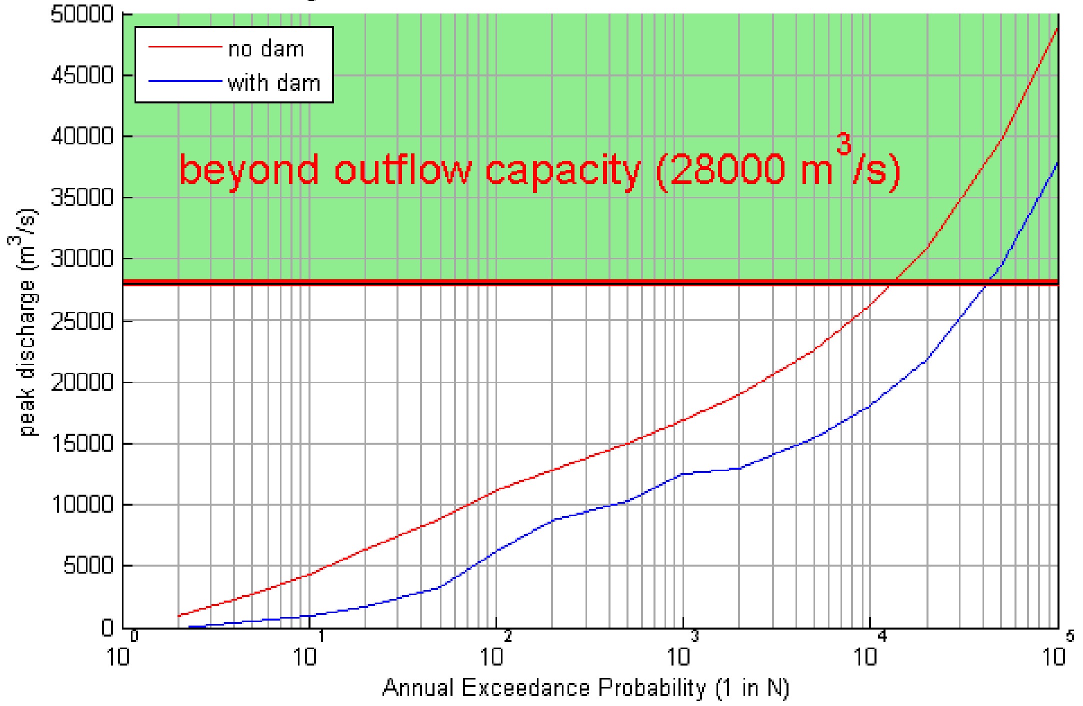


Figure E.1.3 | Location Savages Crossing; MCS simulation results

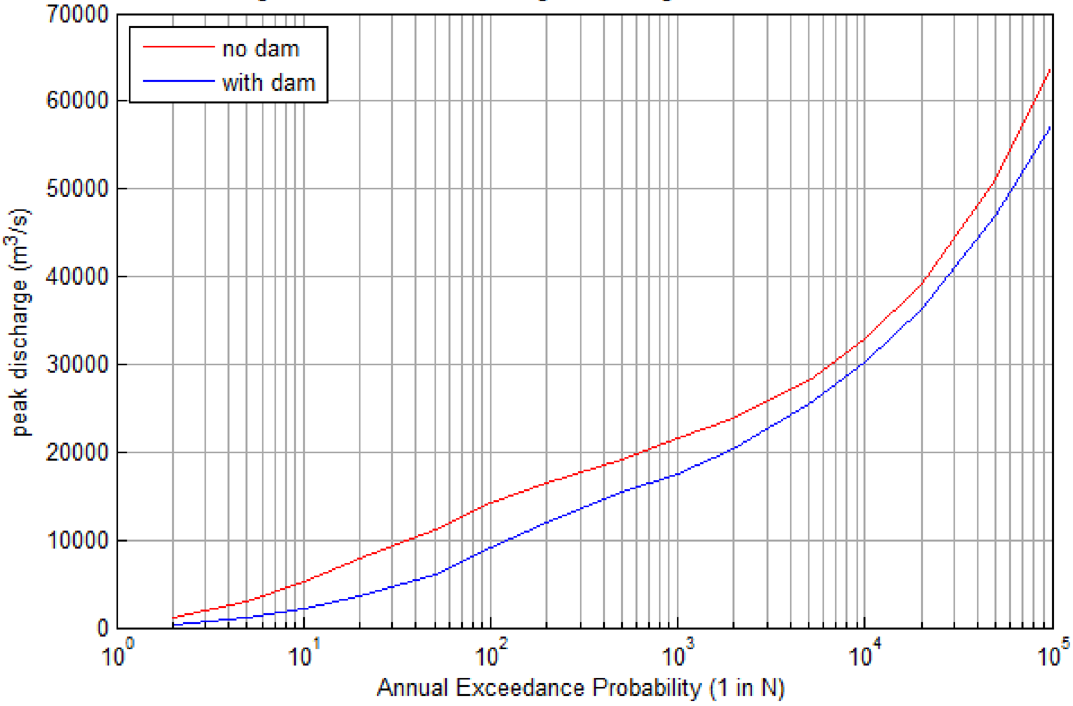


Figure E.1.4 | Location Mount Crosby; MCS simulation results

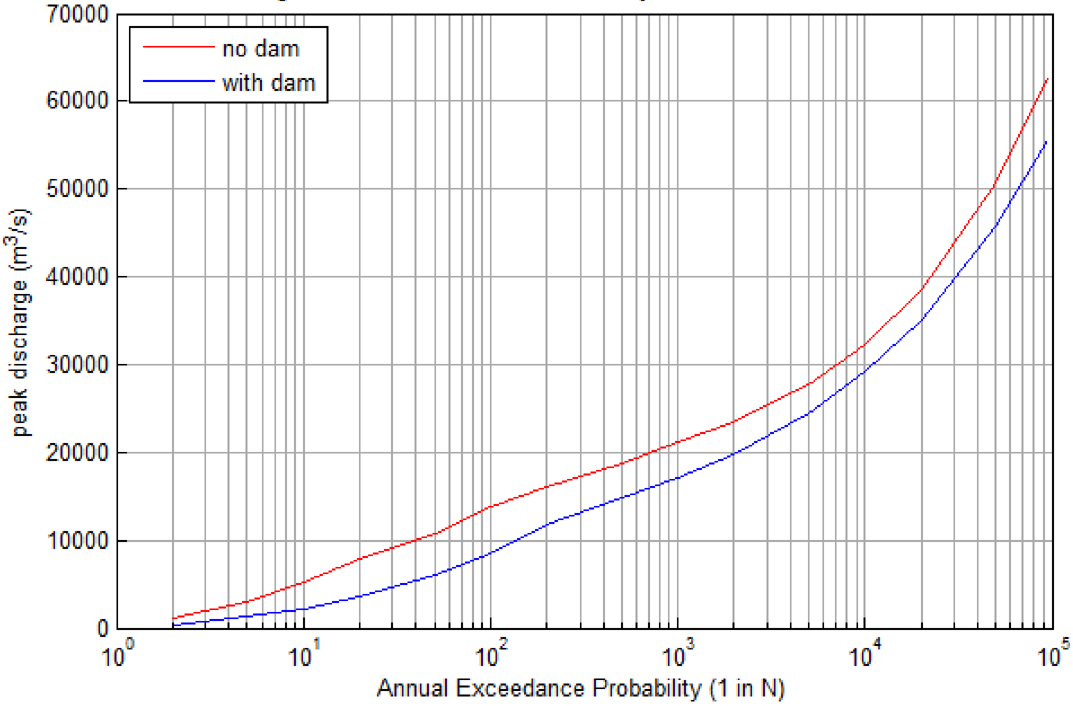


Figure E.1.5 | Location Ipswich; MCS simulation results

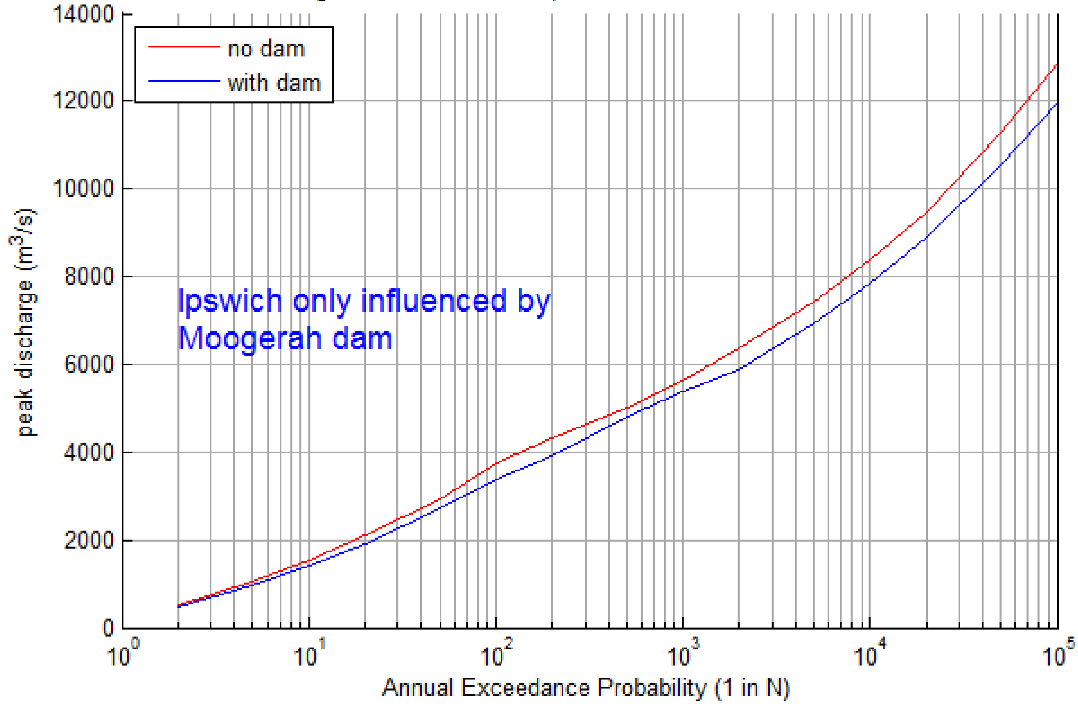
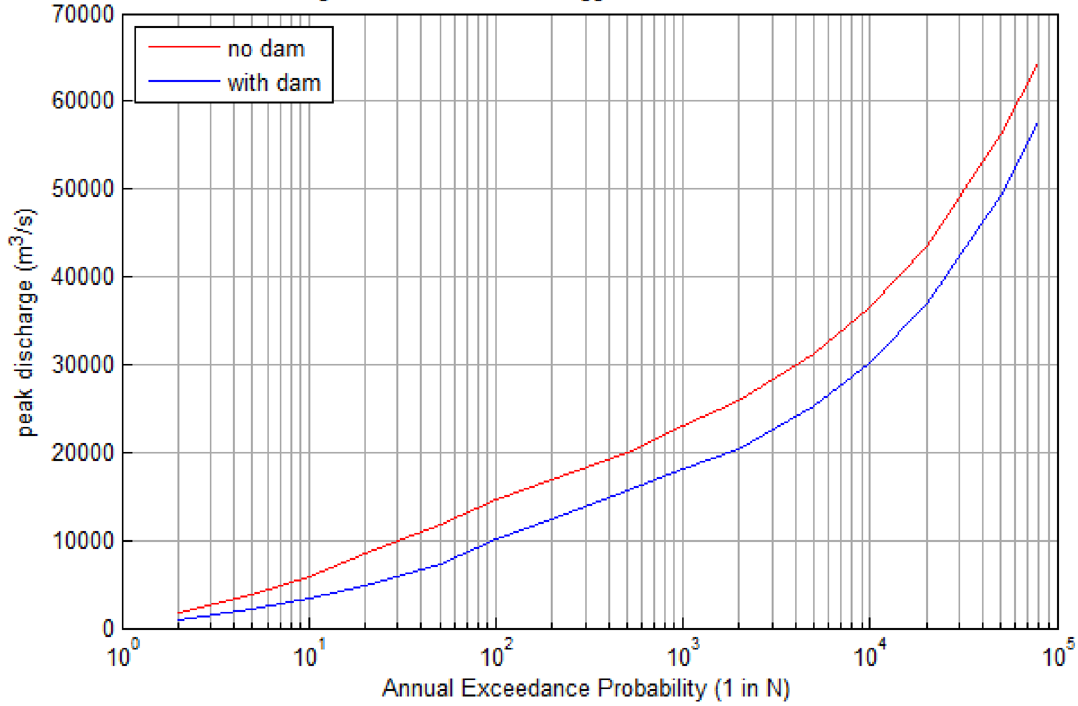


Figure E.1.6 | Location Moggill; MCS simulation results



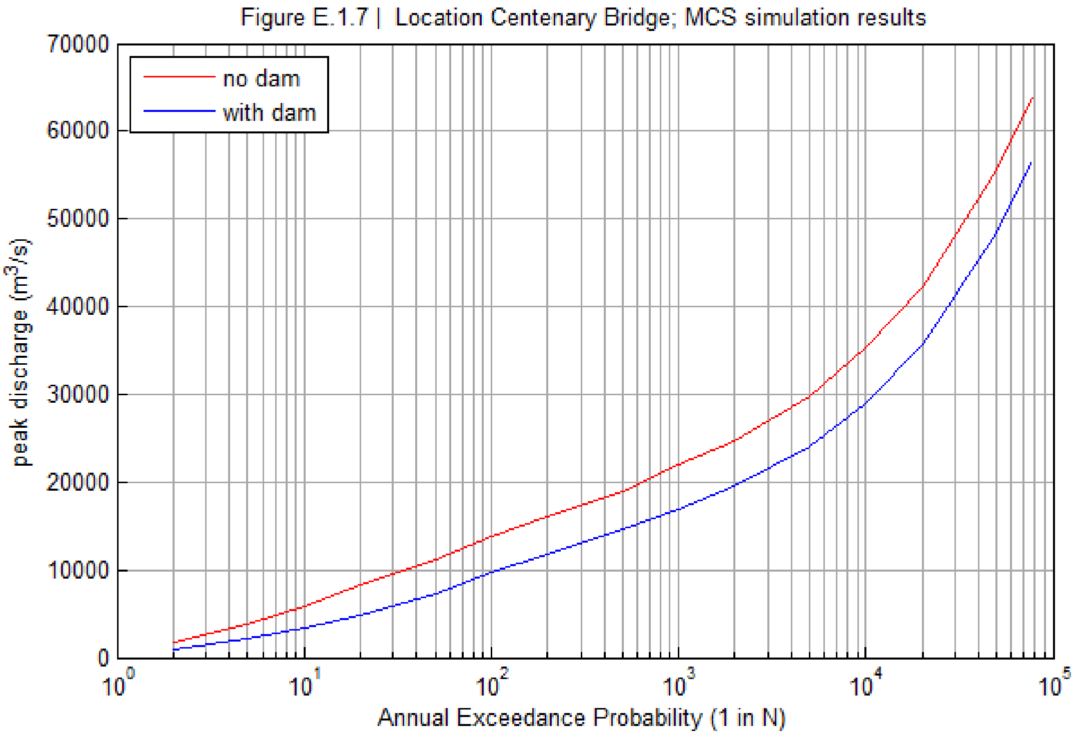


Figure E.1.7 | Location Centenary Bridge; MCS simulation results
no dam
with dam
peak discharge (m^3/s)
70000
60000
50000
40000
30000
20000
10000
0
10^0
10^1
10^2
10^3
10^4
10^5
Annual Exceedance Probability (1 in N)

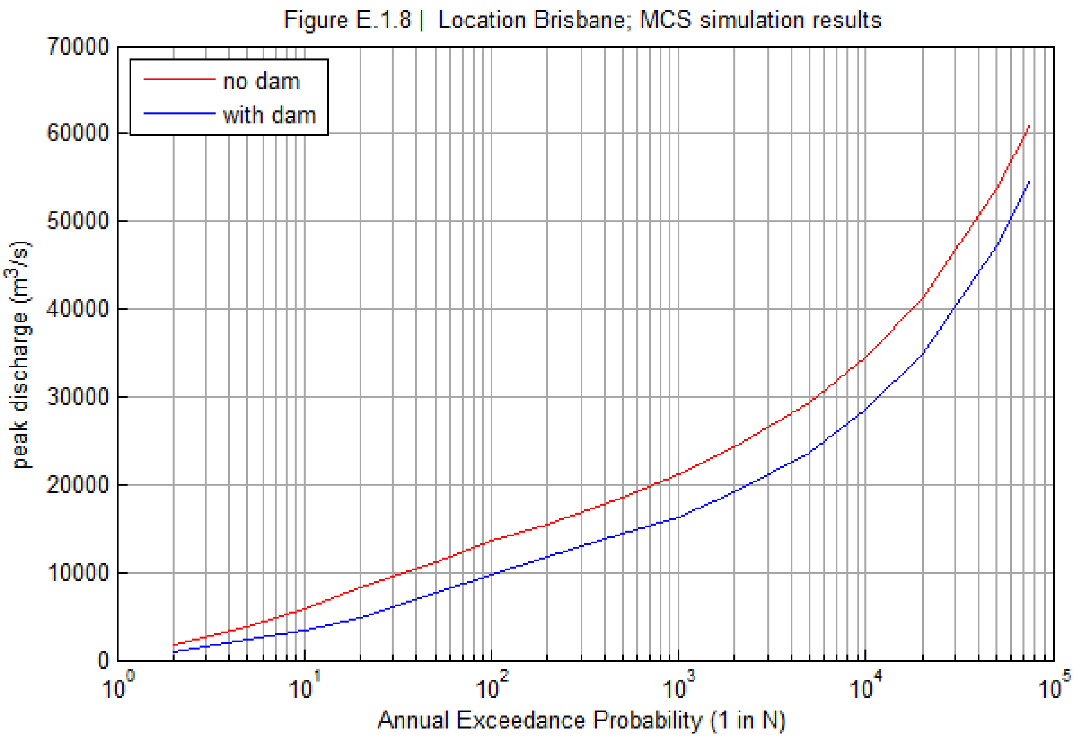


Figure E.1.8 | Location Brisbane; MCS simulation results
no dam
with dam
peak discharge (m^3/s)
70000
60000
50000
40000
30000
20000
10000
0
10^0
10^1
10^2
10^3
10^4
10^5
Annual Exceedance Probability (1 in N)

E.2 MCS scatter plots for peak flows

Figure E.2.1 | Location Somerset Dam; MCS simulation results

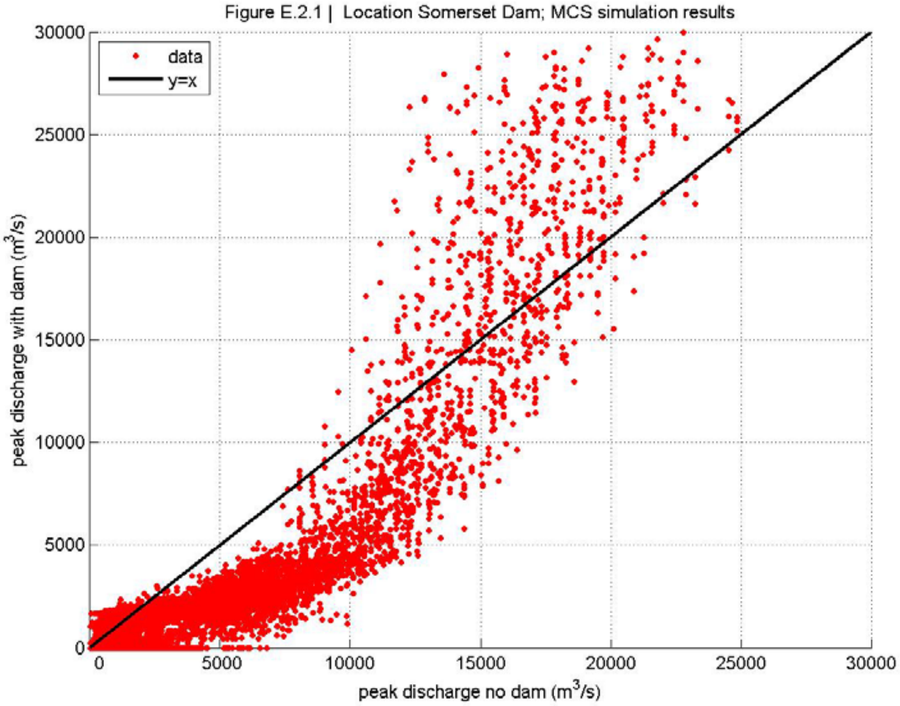


Figure E.2.2 | Location Wivenhoe; MCS simulation results

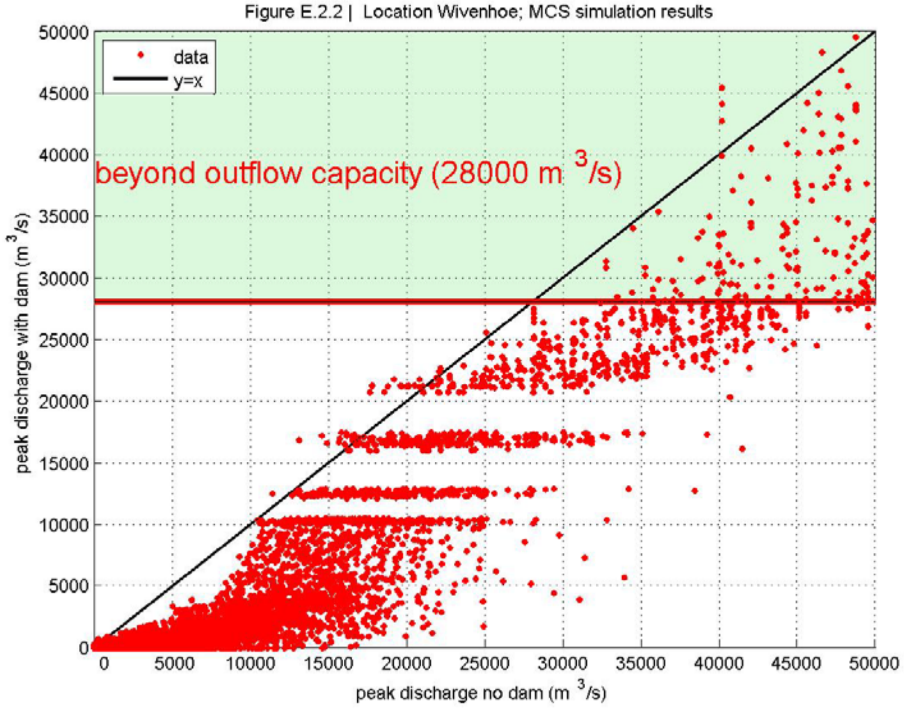


Figure E.2.3 | Location Savages Crossing; MCS simulation results

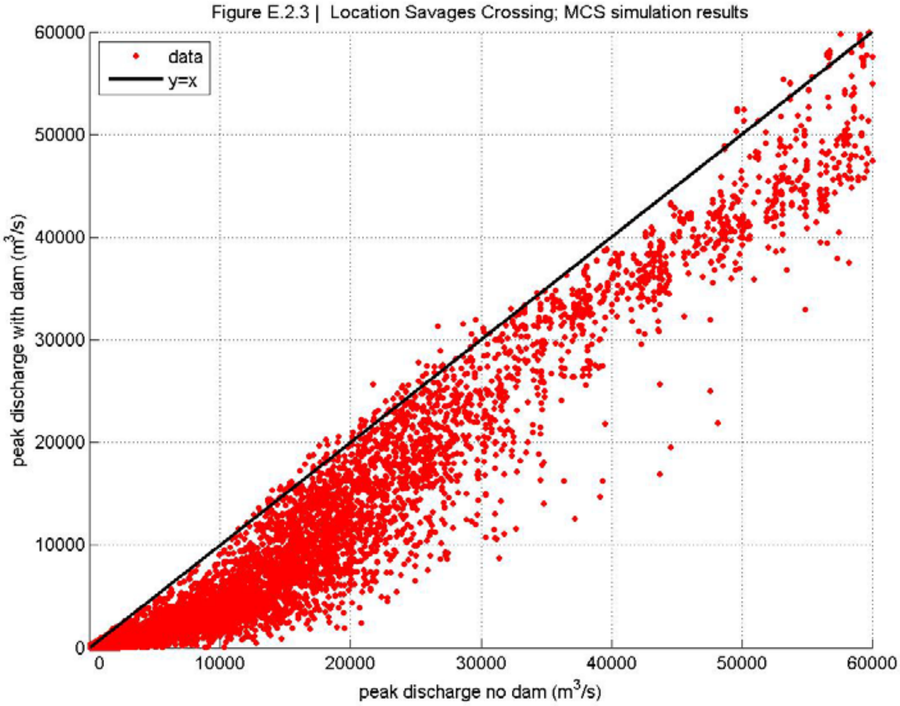


Figure E.2.4 | Location Mount Crosby; MCS simulation results

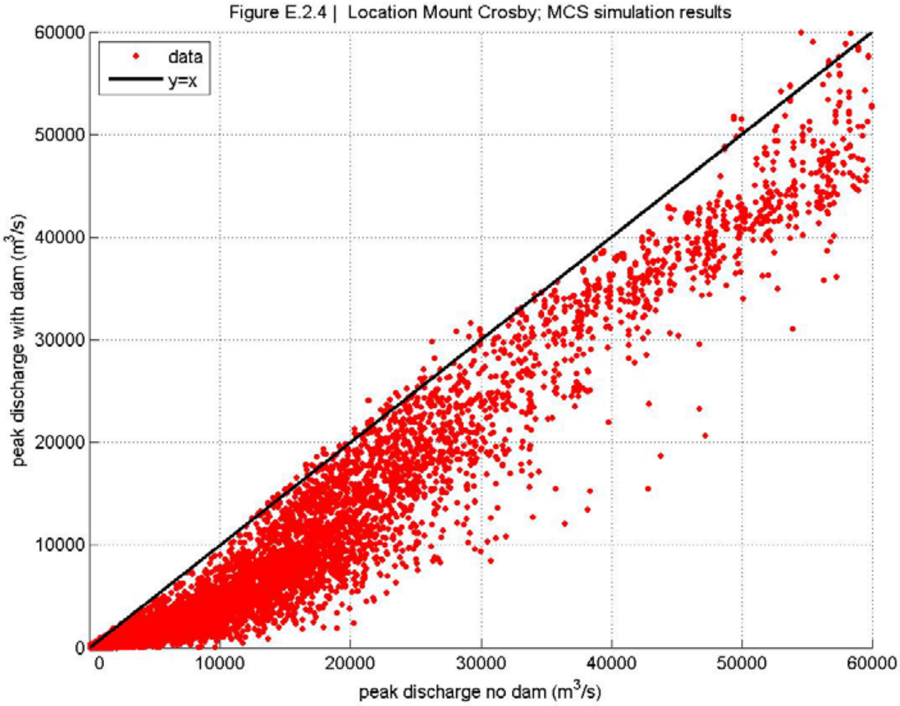


Figure E.2.5 | Location Ipswich; MCS simulation results

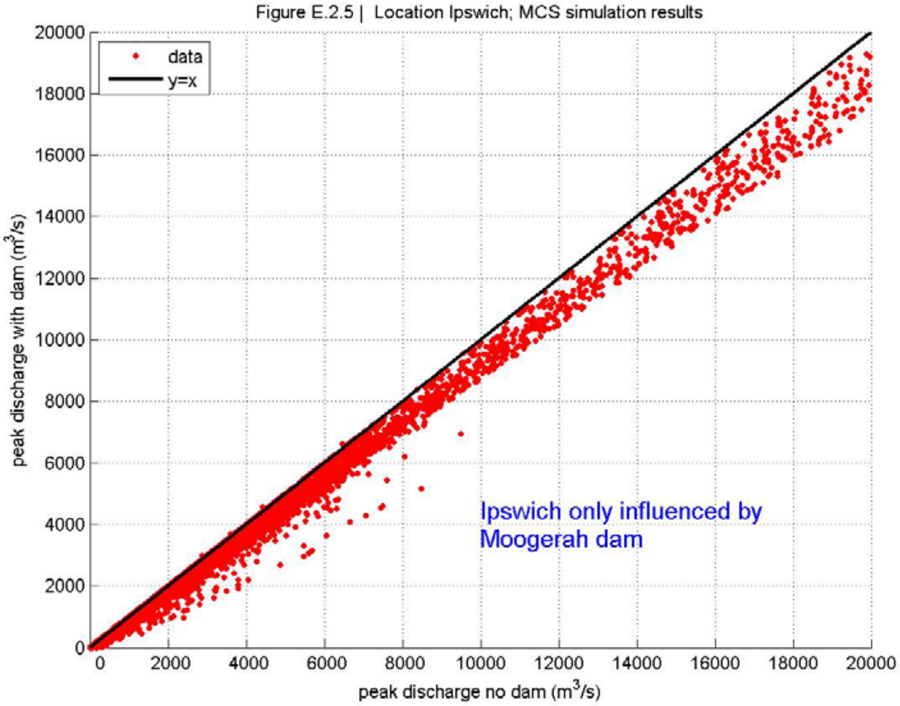


Figure E.2.6 | Location Moggill; MCS simulation results

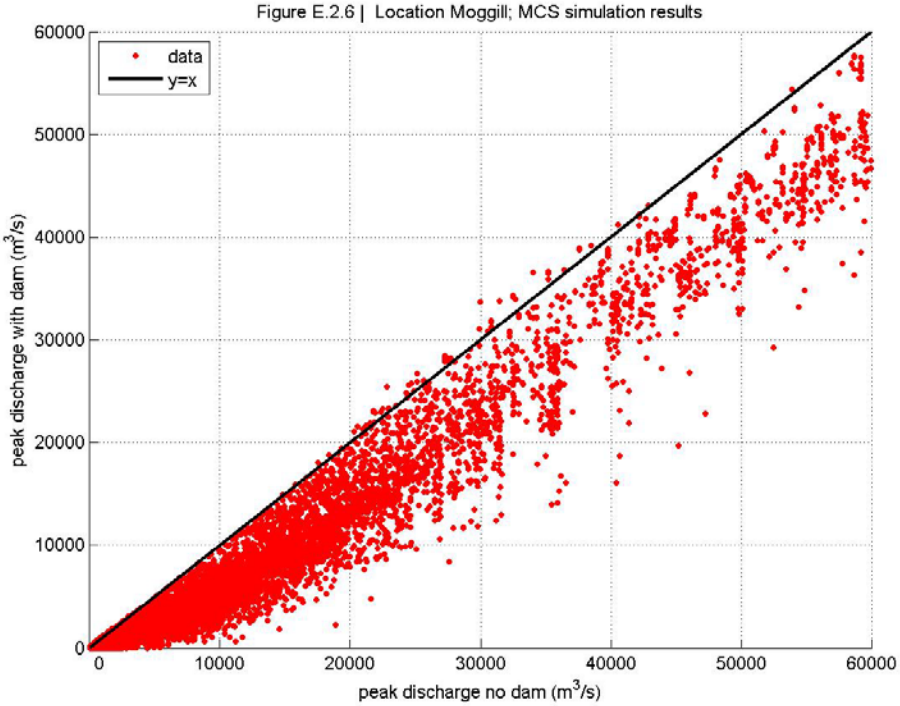


Figure E.2.7 | Location Centenary Bridge; MCS simulation results

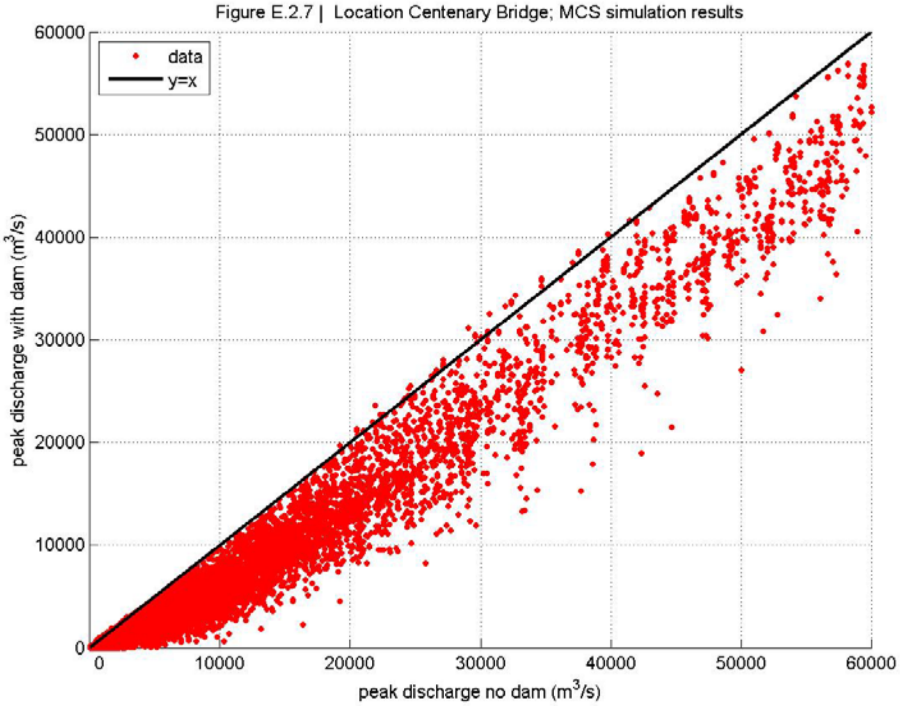
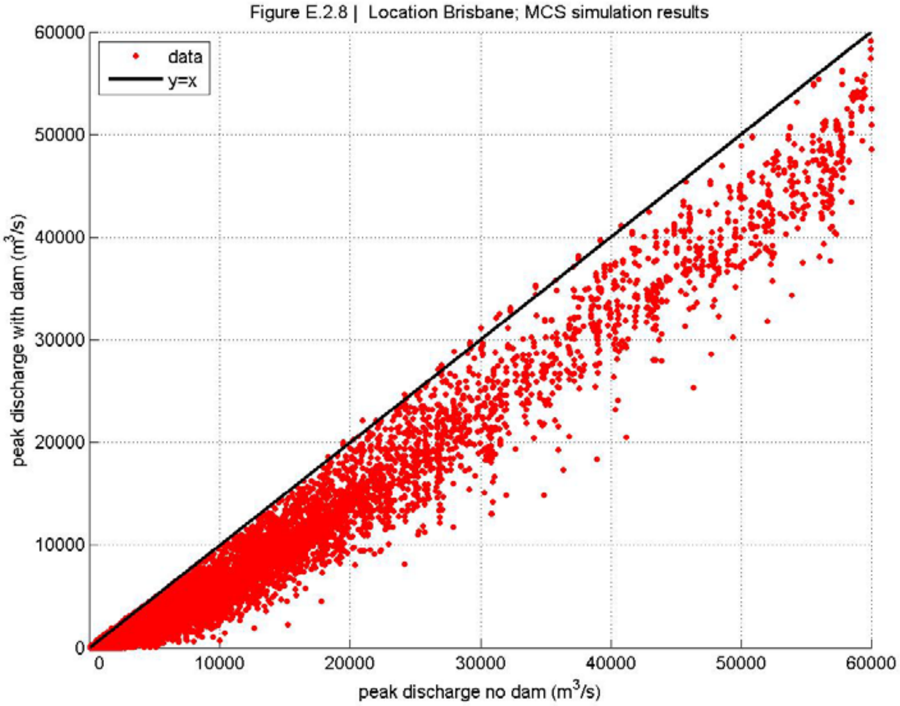


Figure E.2.8 | Location Brisbane; MCS simulation results



E3. MCS frequency plots of flow volumes

Figure E.3.1 | Location Somerset Dam

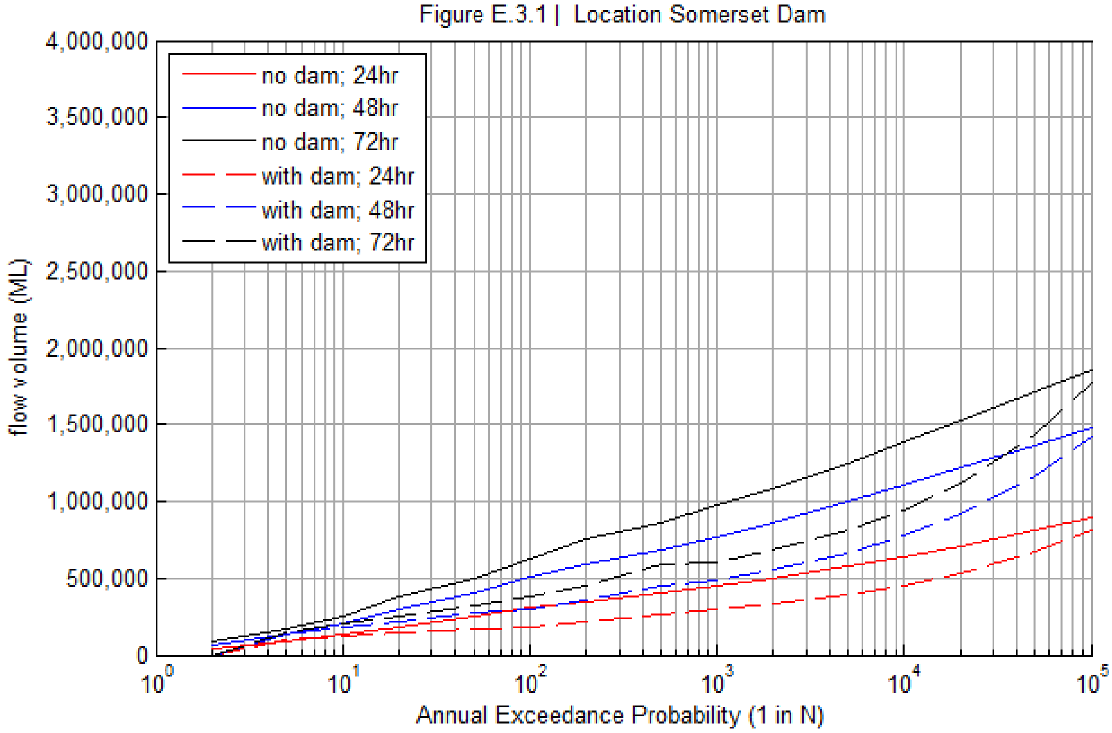


Figure E.3.2 | Location Wivenhoe

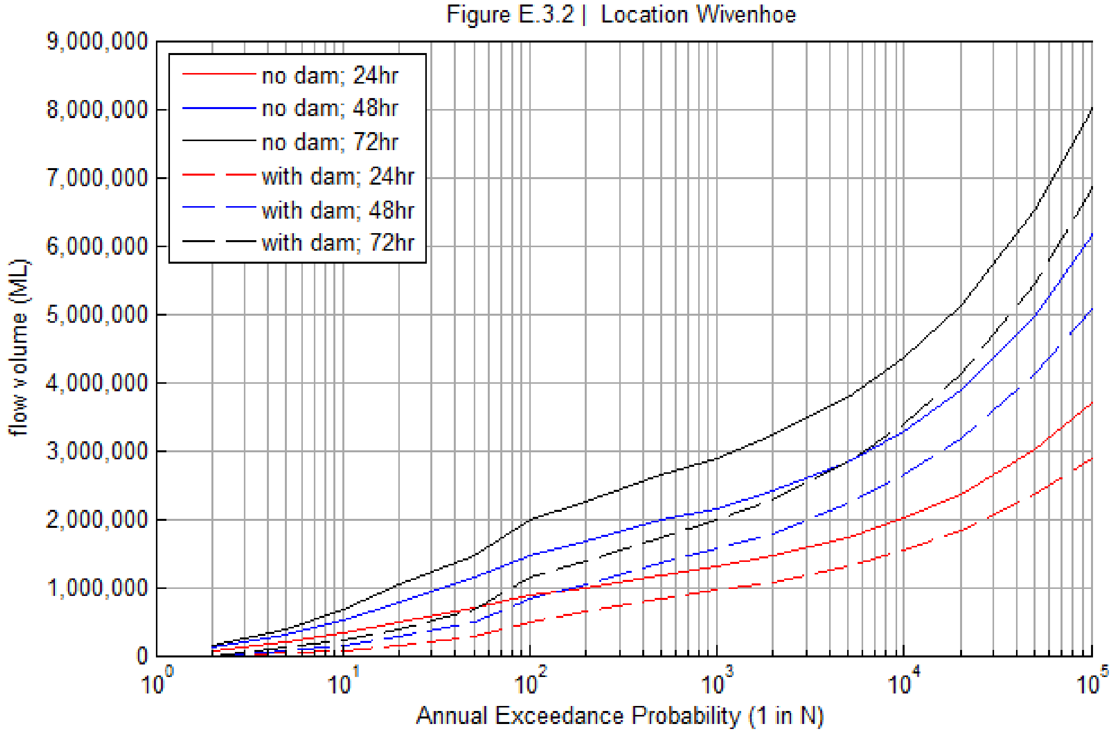


Figure E.3.3 | Location Savages Crossing

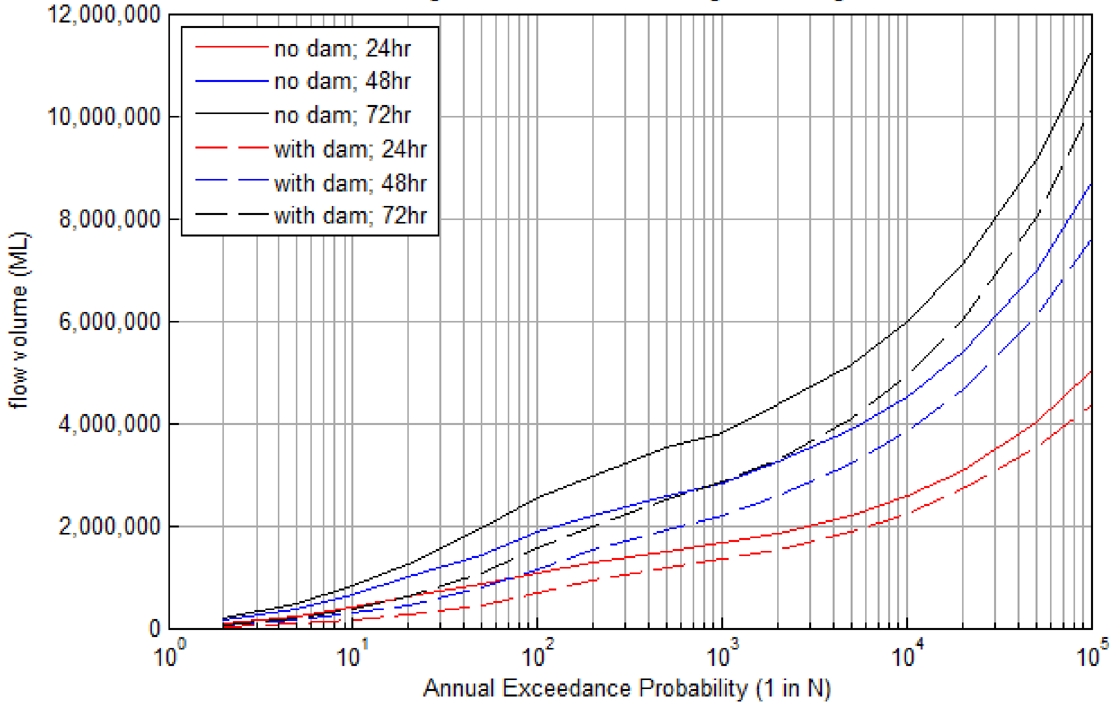
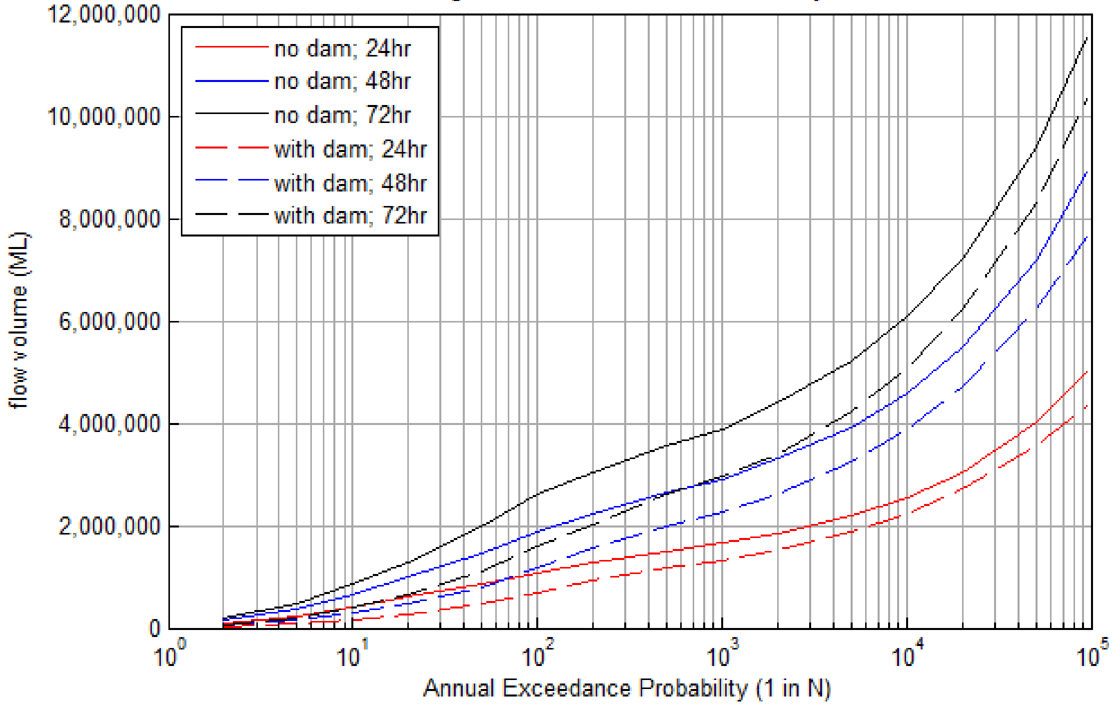


Figure E.3.4 | Location Mount Crosby



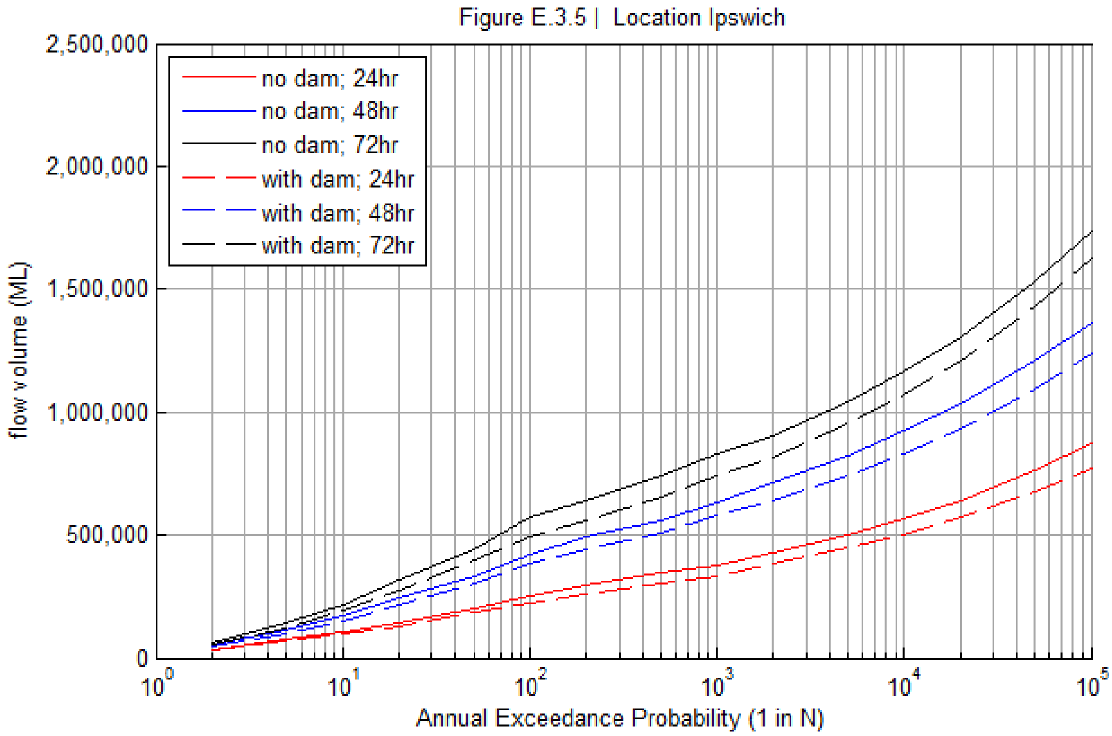


Figure E.3.5 | Location Ipswich
no dam; 24hr
no dam; 48hr
no dam; 72hr
with dam; 24hr
with dam; 48hr
with dam; 72hr
flow volume (ML)
Annual Exceedance Probability (1 in N)

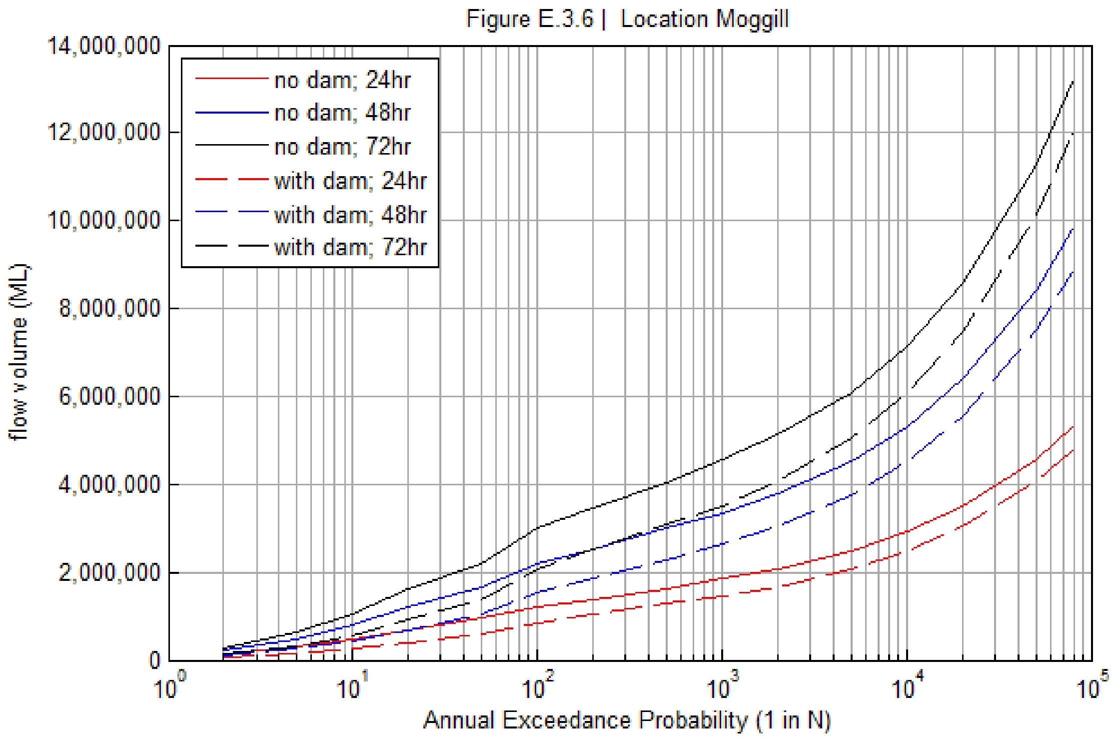


Figure E.3.6 | Location Moggill
no dam; 24hr
no dam; 48hr
no dam; 72hr
with dam; 24hr
with dam; 48hr
with dam; 72hr
flow volume (ML)
Annual Exceedance Probability (1 in N)

Figure E.3.7 | Location Centenary Bridge

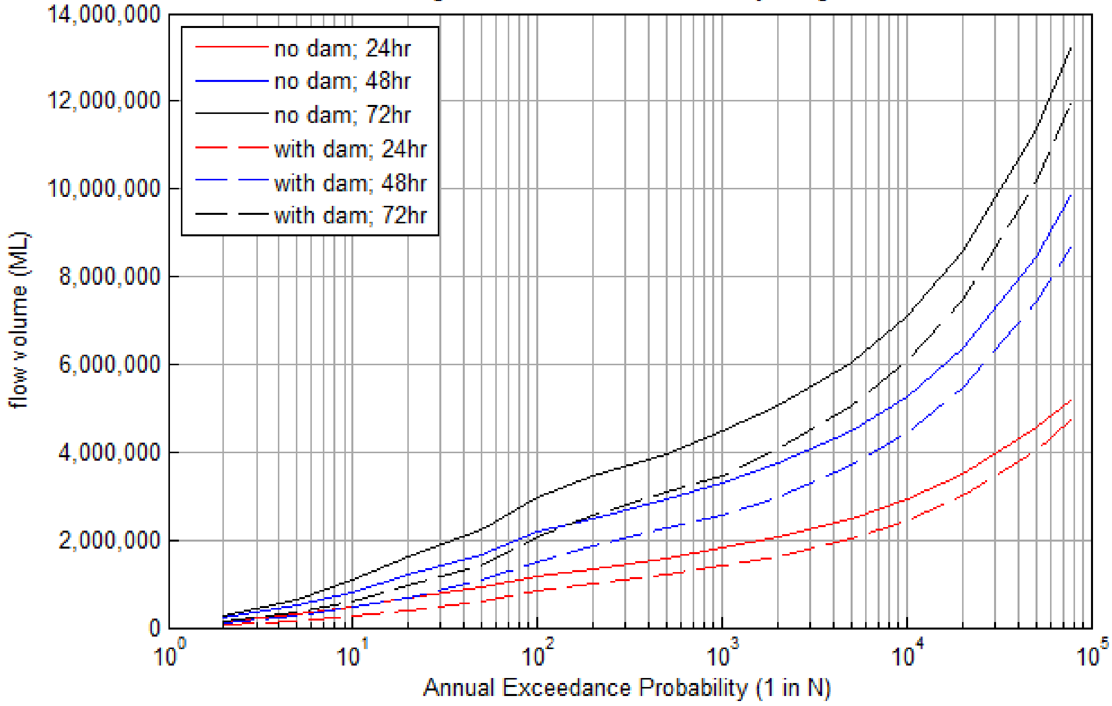
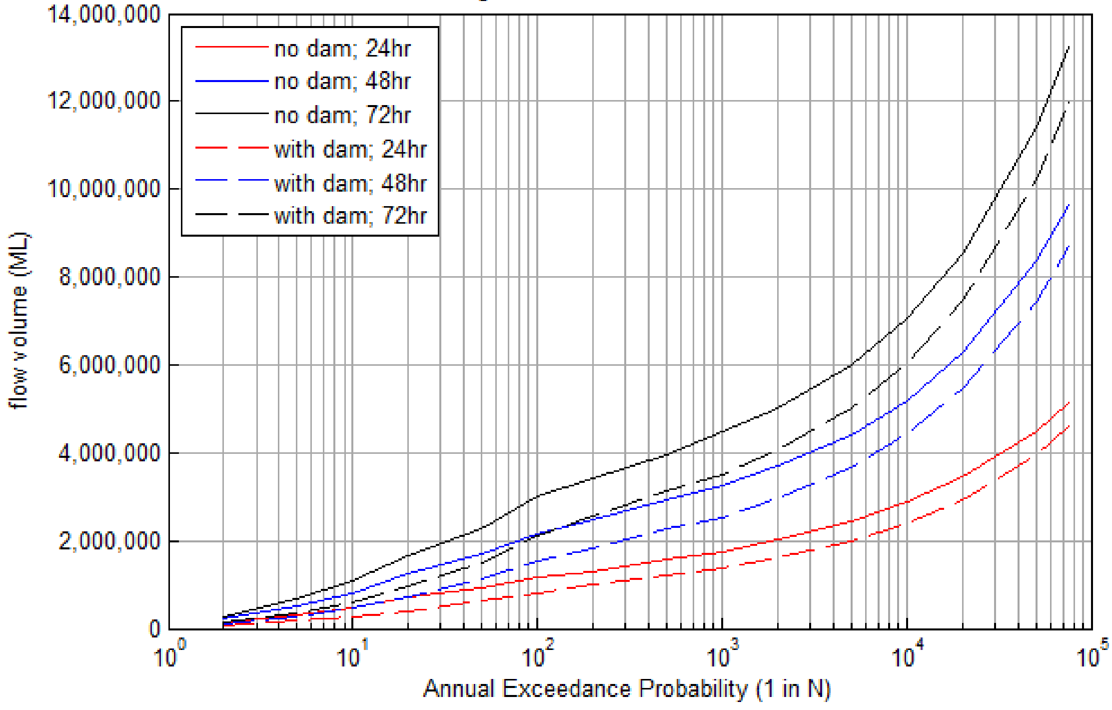
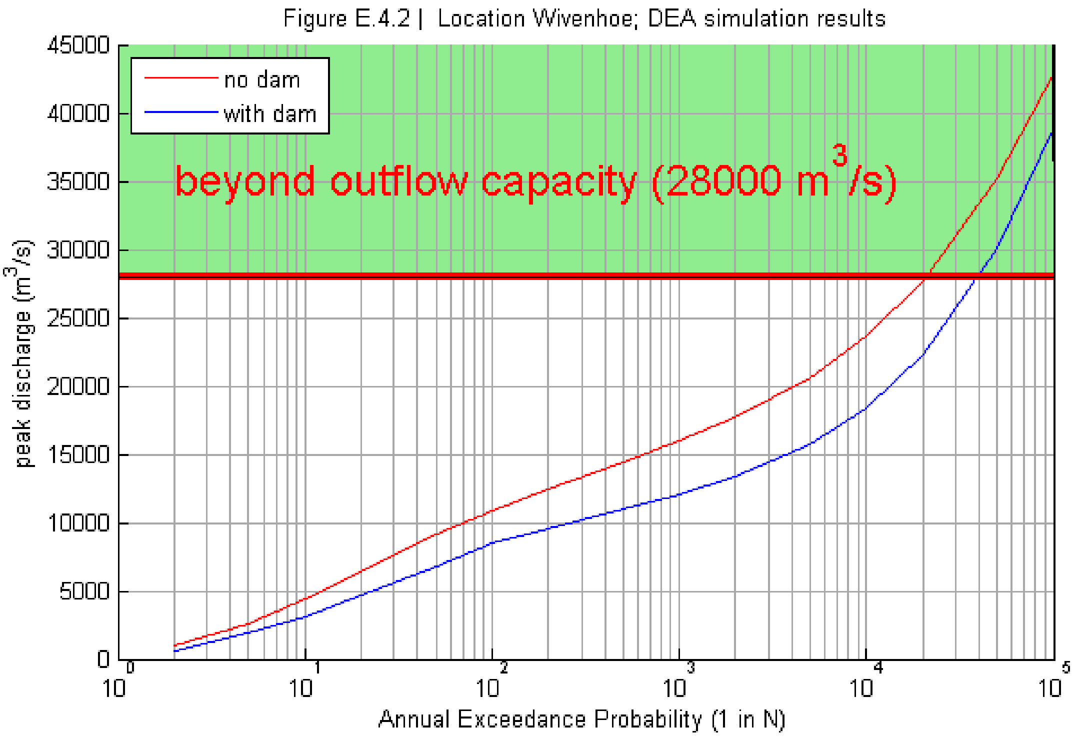


Figure E.3.8 | Location Brisbane



E.4 DEA frequency plots of peak flows



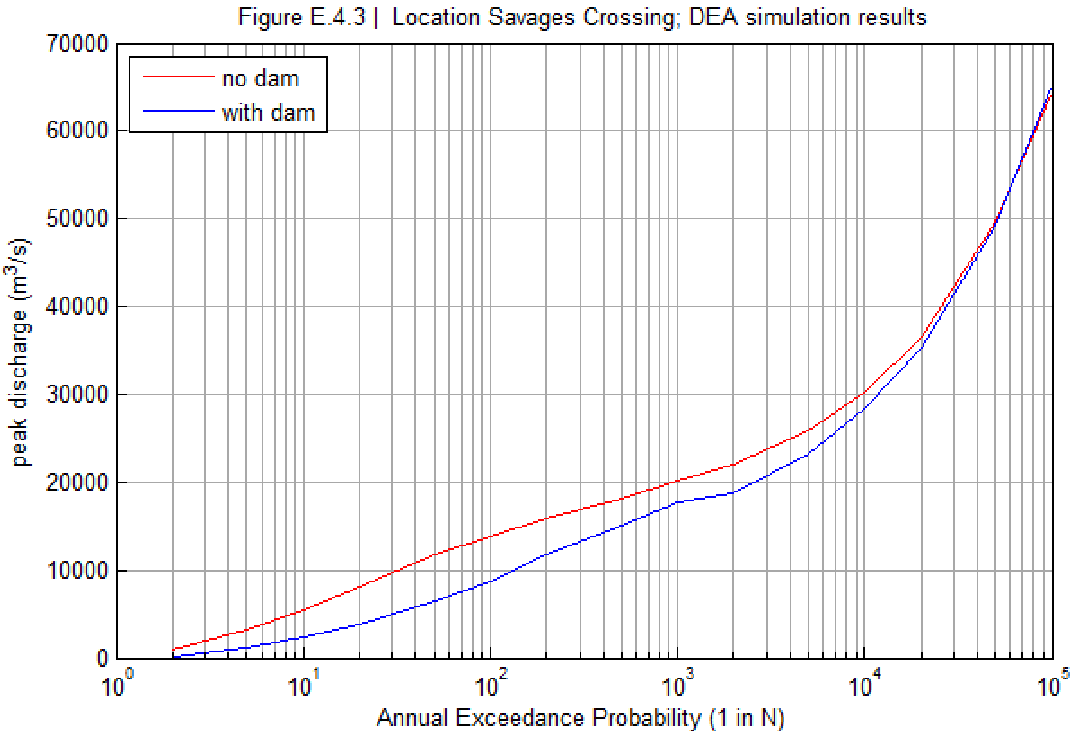


Figure E.4.4 | Location Mount Crosby; DEA simulation results

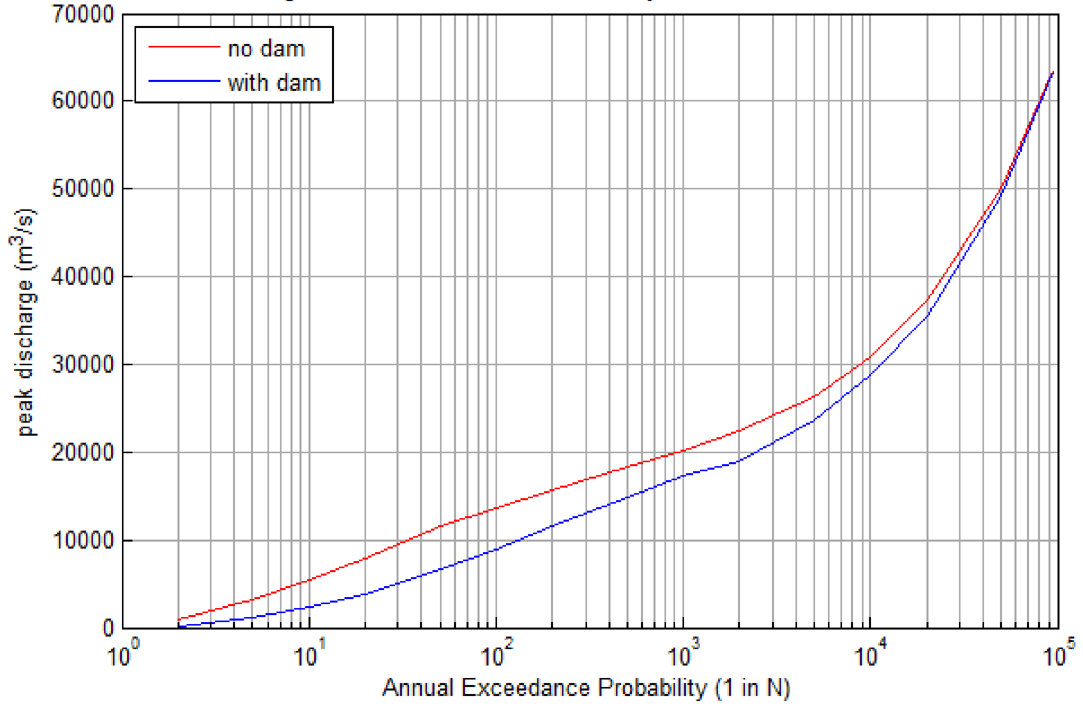


Figure E.4.5 | Location Ipswich; DEA simulation results

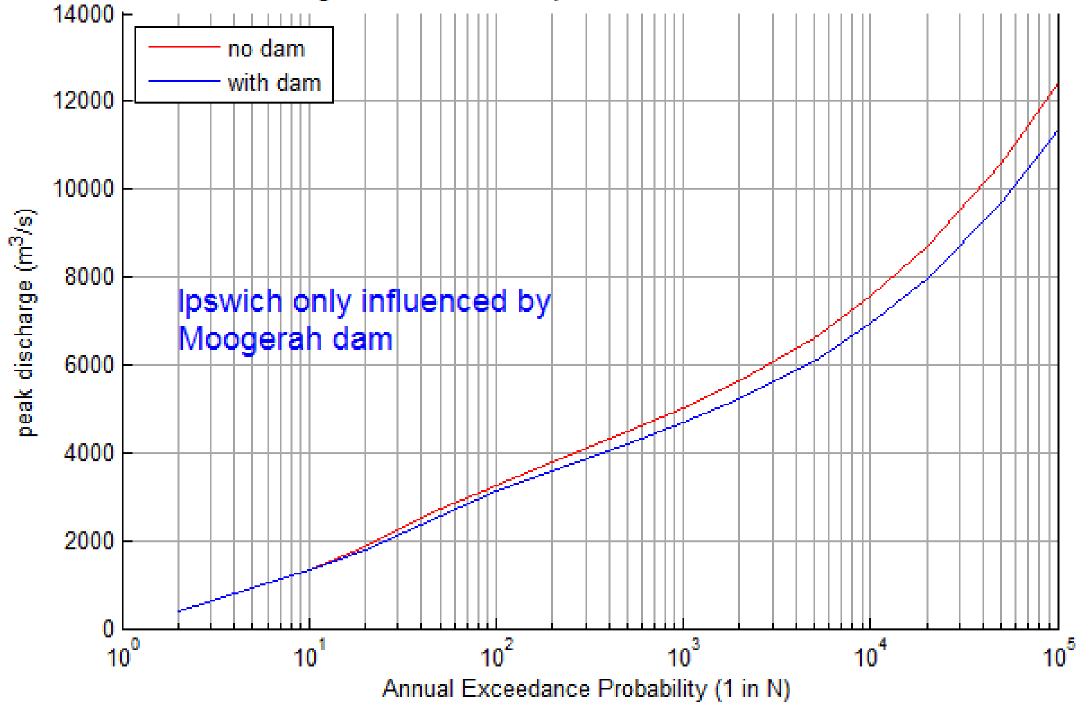


Figure E.4.6 | Location Moggill; DEA simulation results

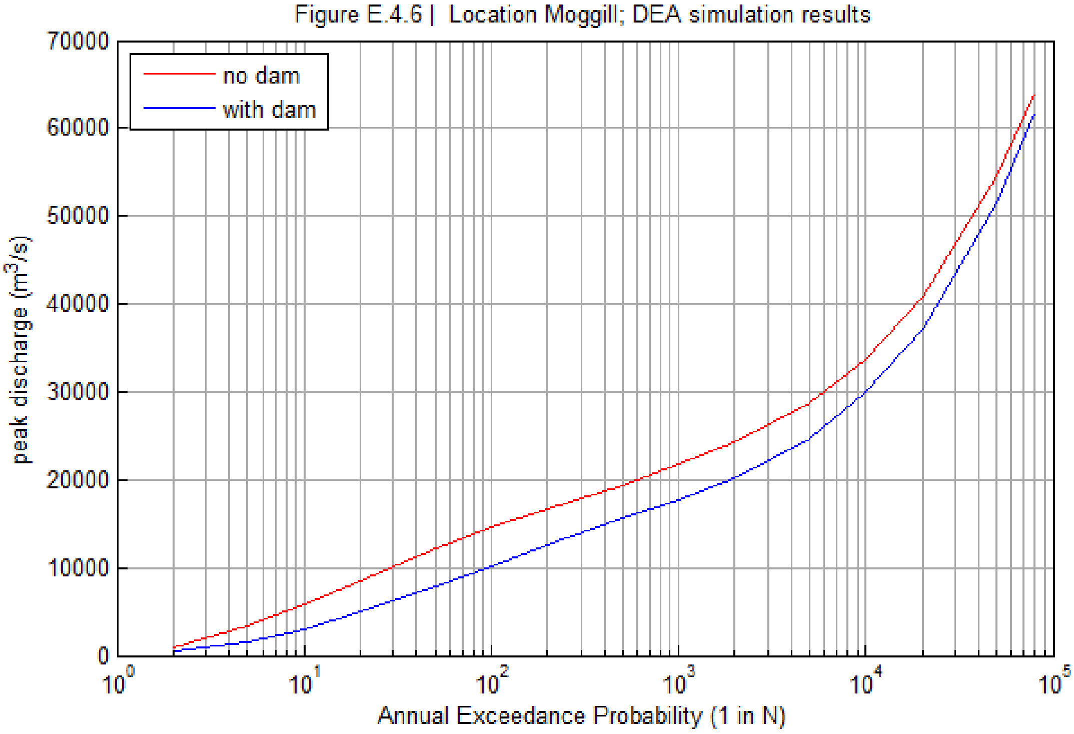
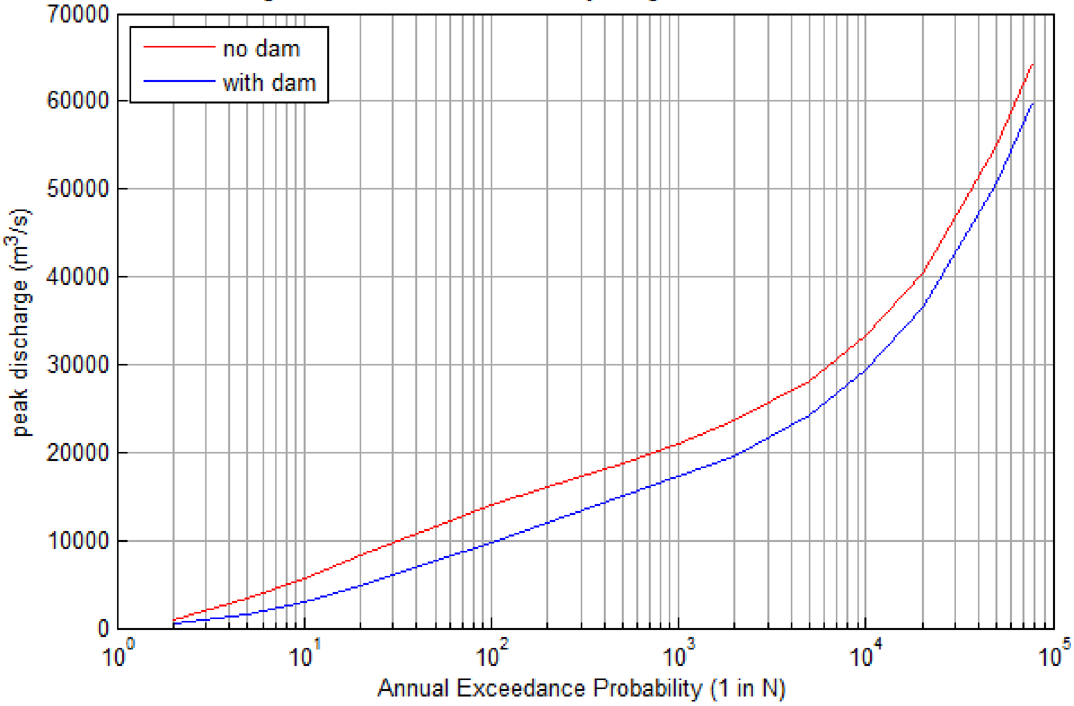


Figure E.4.7 | Location Centenary Bridge; DEA simulation results



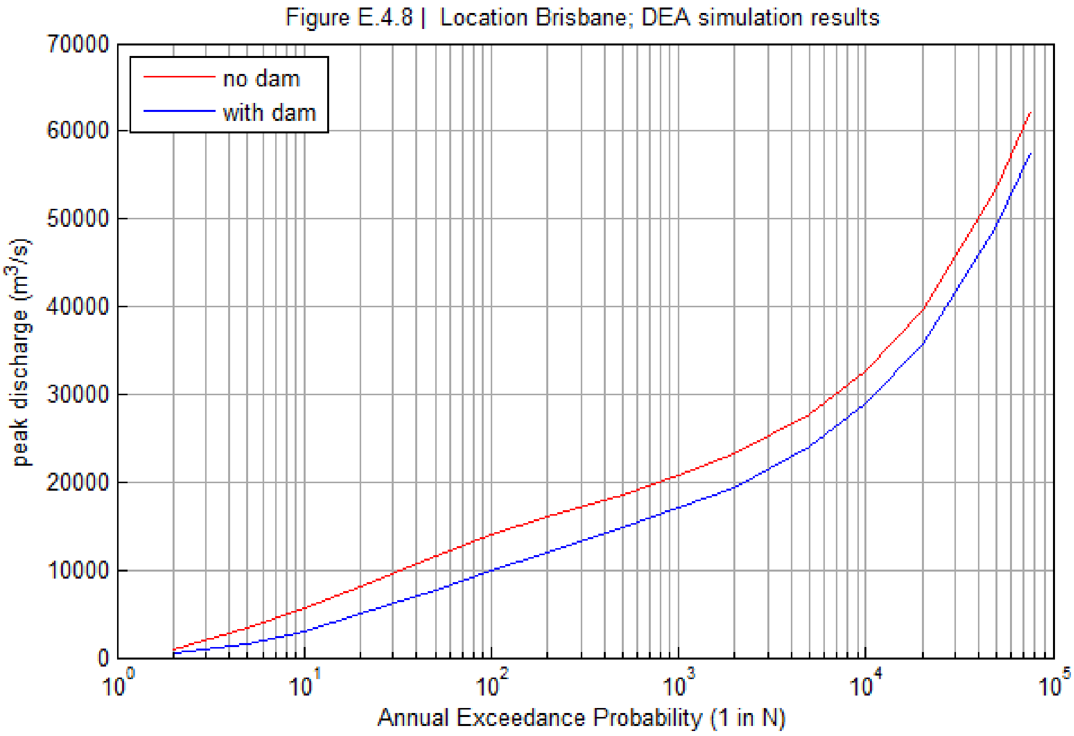


Figure E.4.8 | Location Brisbane; DEA simulation results
no dam
with dam
peak discharge (m3/s)
Annual Exceedance Probability (1 in N)
70000
60000
50000
40000
30000
20000
10000
0
10^0
10^1
10^2
10^3
10^4
10^5

E.5 DEA scatter plots for peak flows

Note: Figure E.5.1 is not available, as no DEA run was carried out for location Somerset for 'with-dams' conditions.

Figure E.5.2 | Location Wivenhoe; DEA simulation results

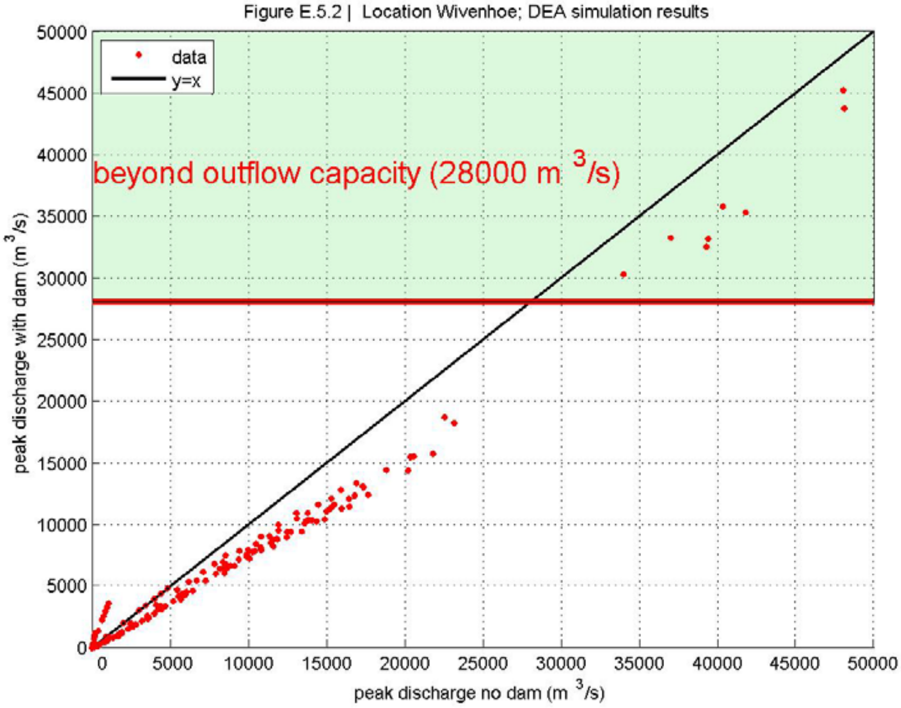


Figure E.5.3 | Location Savages Crossing; DEA simulation results

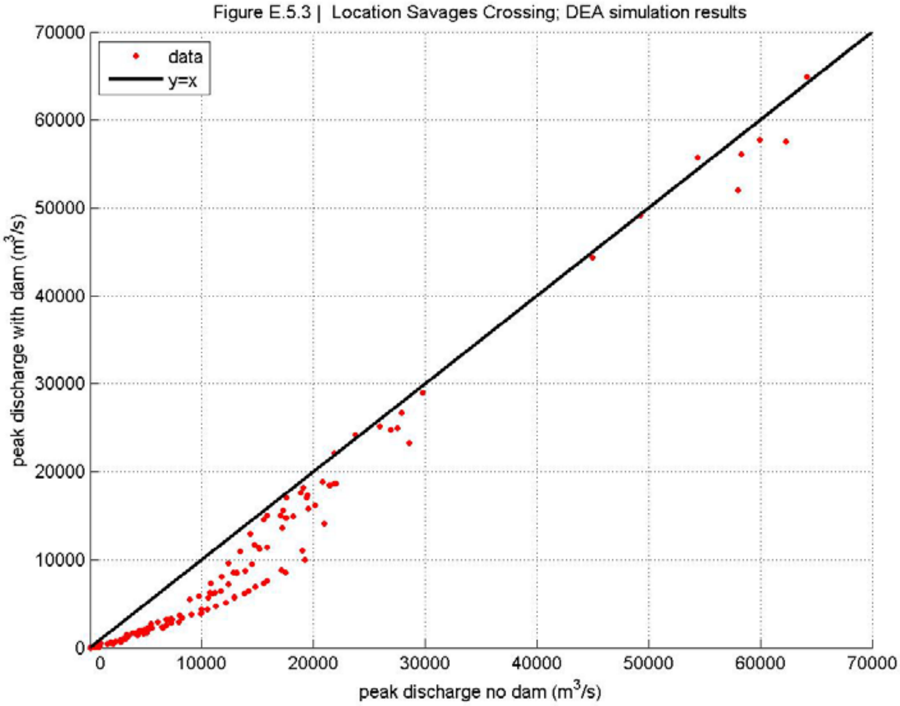


Figure E.5.4 | Location Mount Crosby; DEA simulation results

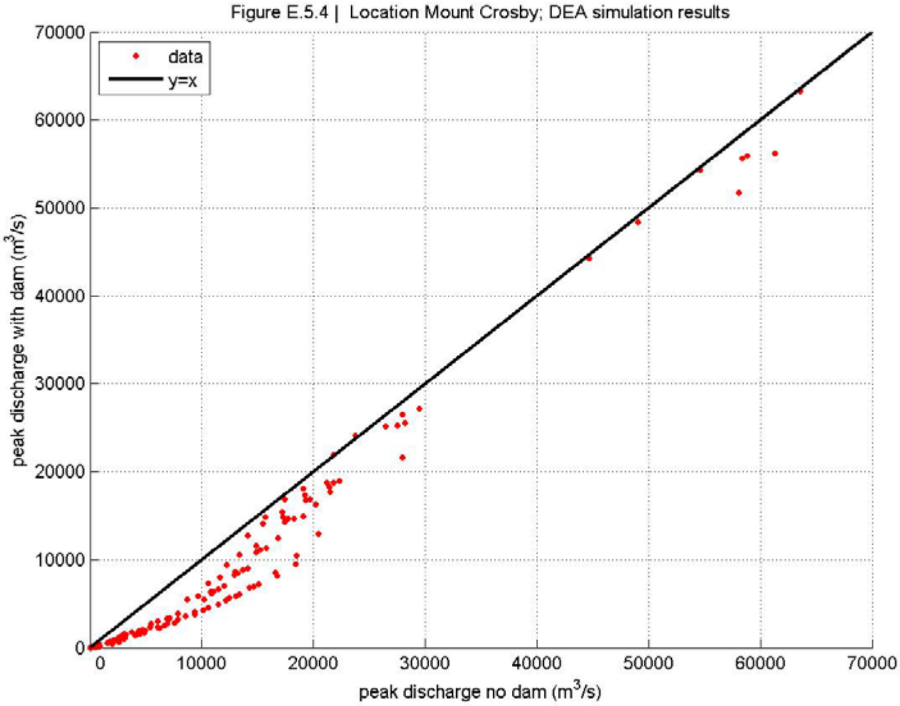


Figure E.5.5 | Location Ipswich; DEA simulation results

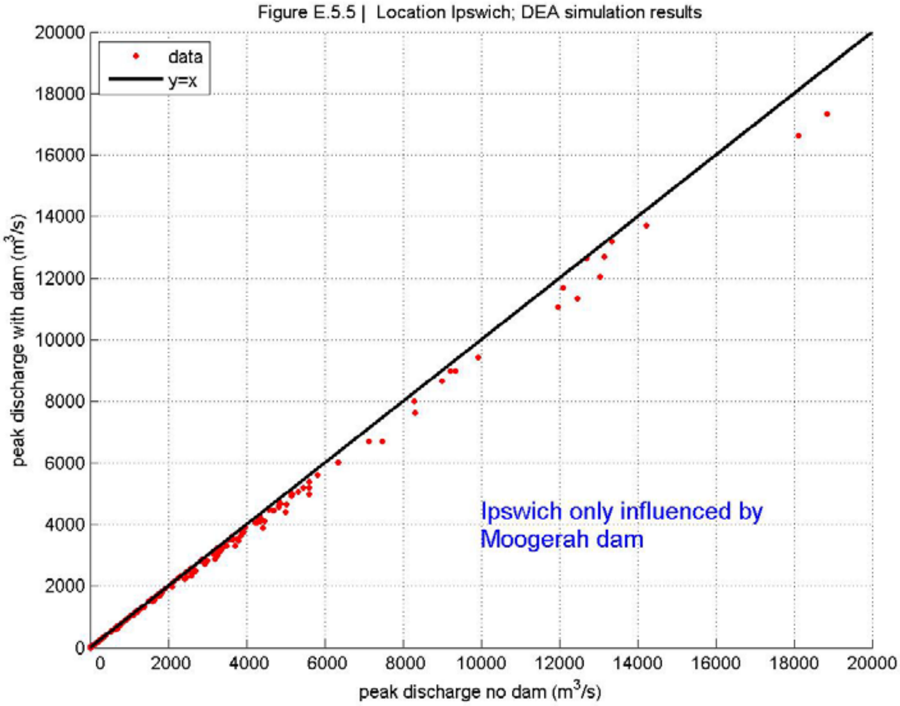


Figure E.5.6 | Location Moggill; DEA simulation results

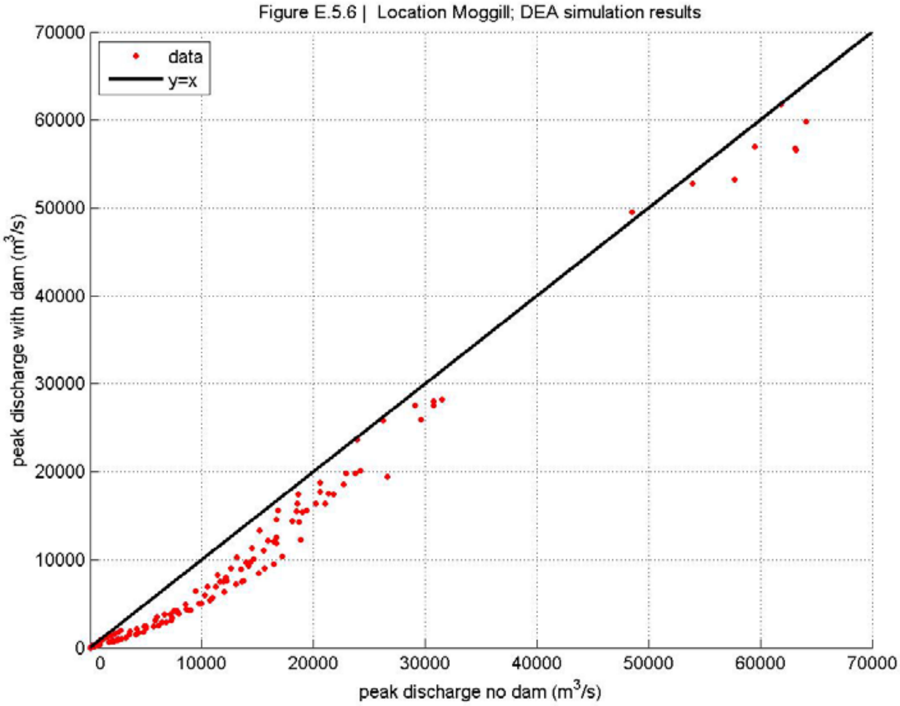


Figure E.5.7 | Location Centenary Bridge; DEA simulation results

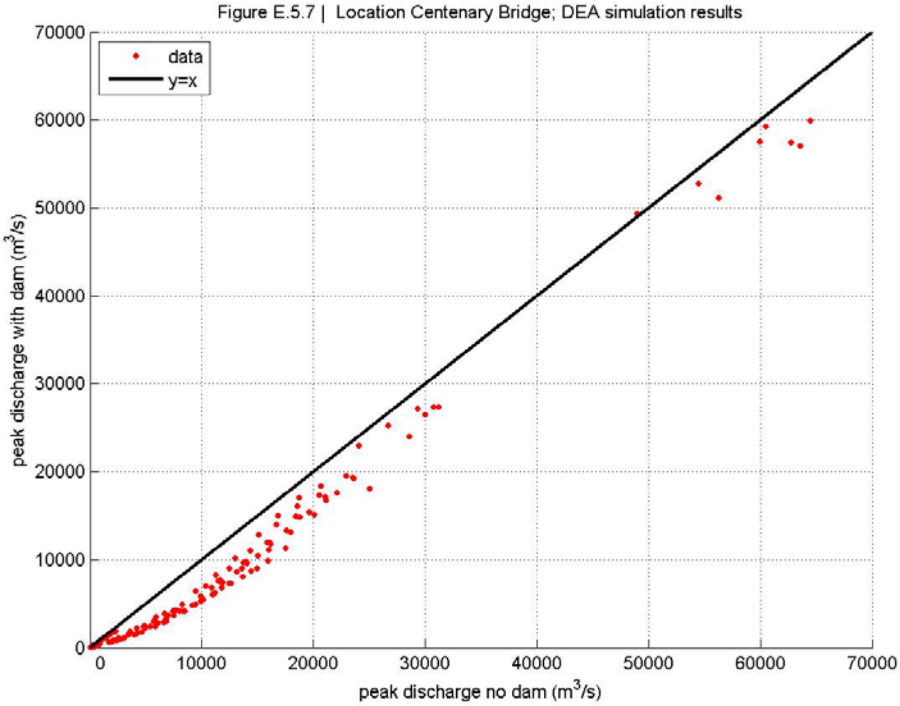
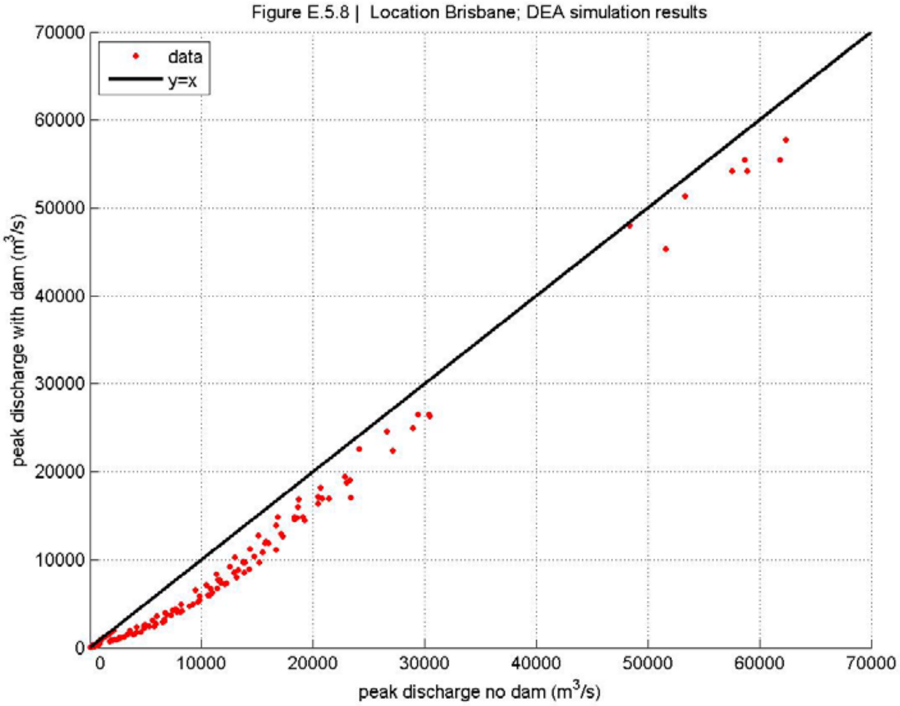


Figure E.5.8 | Location Brisbane; DEA simulation results



E6 DEA frequency plots of flow volumes

Note: Figure E.6.1 is not available, as no DEA run was carried out for location Somerset for 'with-dams' conditions.

Figure E.6.2 | Location Wivenhoe; DEA simulation results

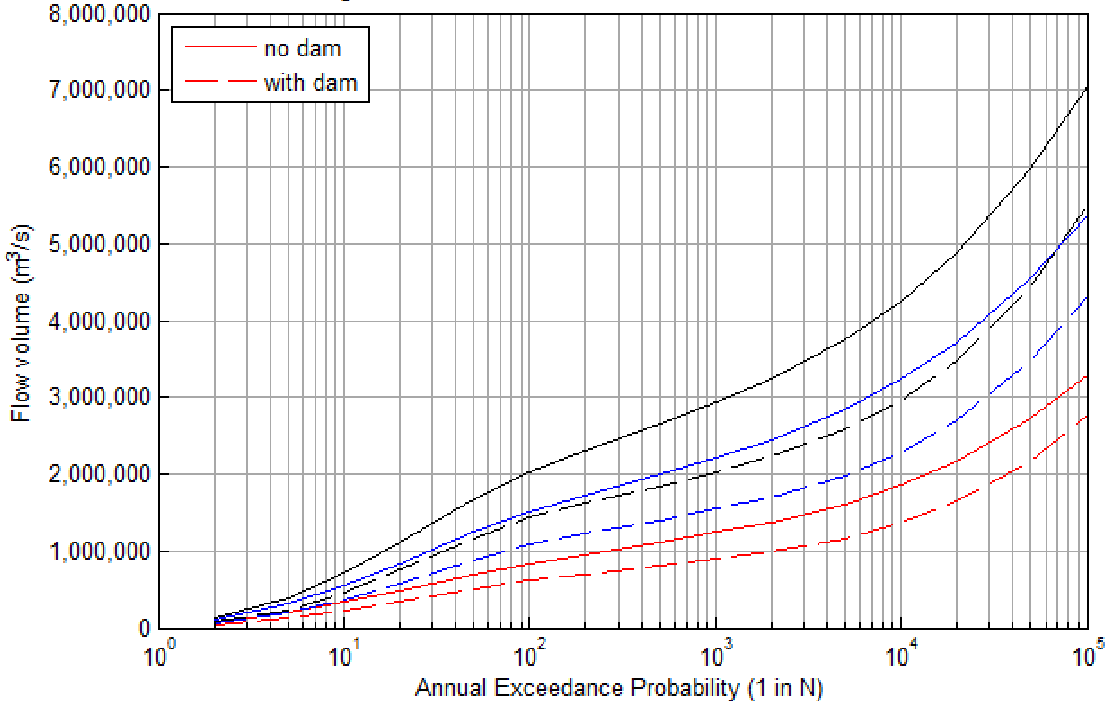


Figure E.6.3 | Location Savages Crossing; DEA simulation results

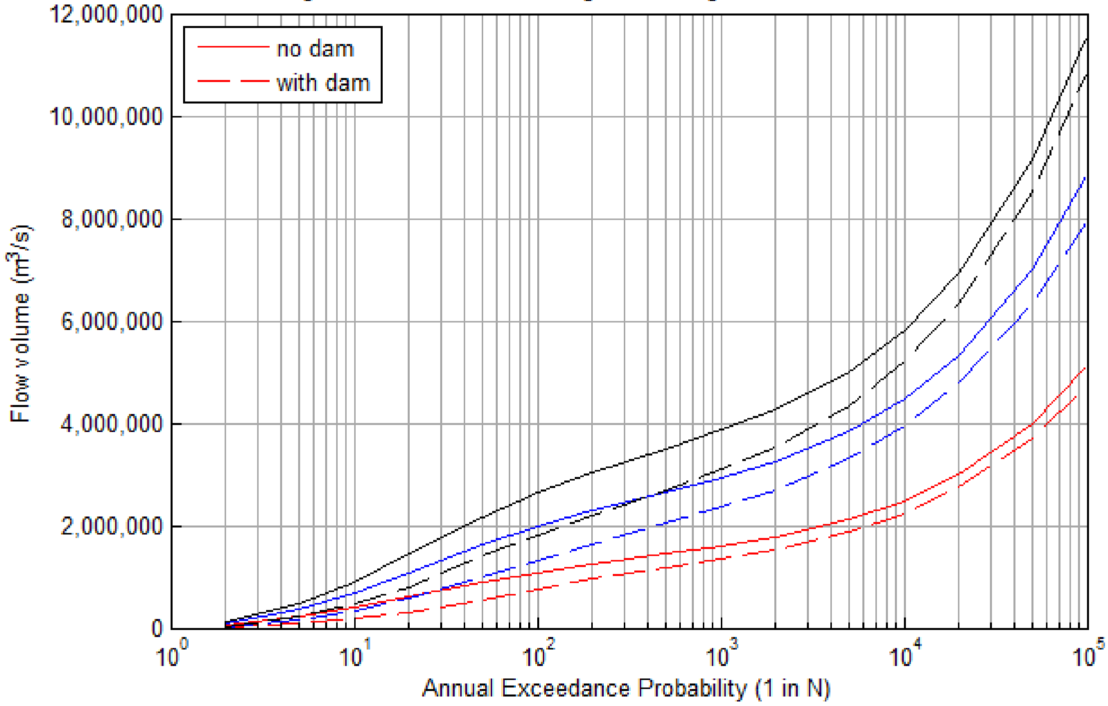


Figure E.6.4 | Location Mount Crosby; DEA simulation results

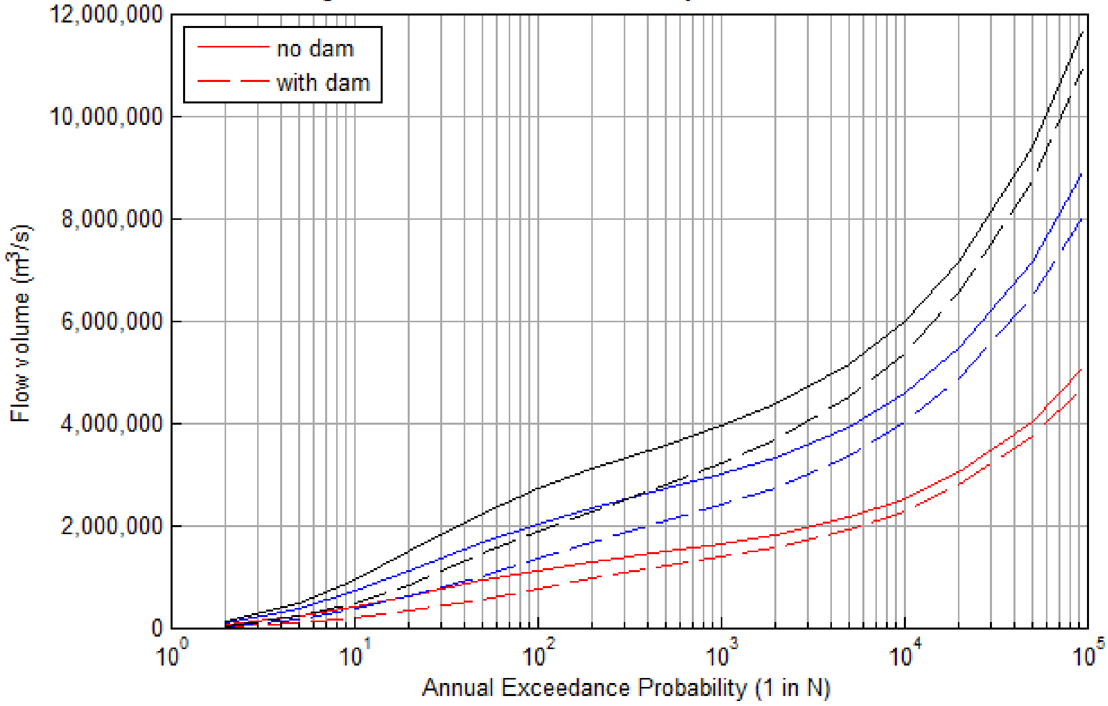


Figure E.6.5 | Location Ipswich; DEA simulation results

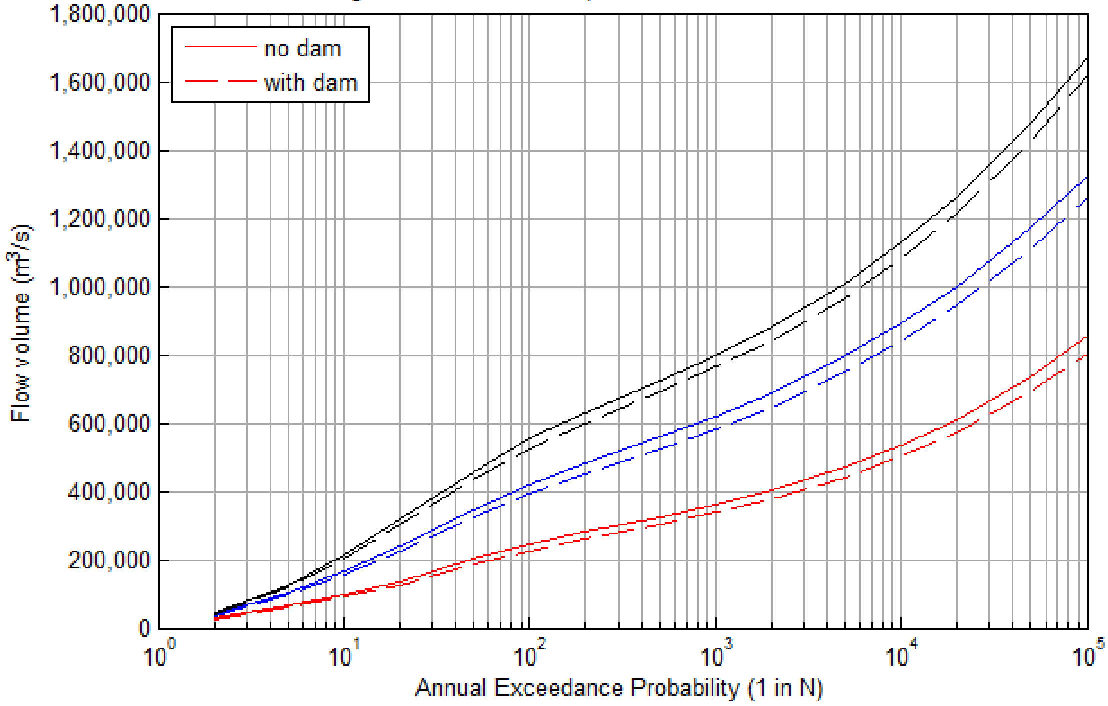


Figure E.6.6 | Location Moggill; DEA simulation results

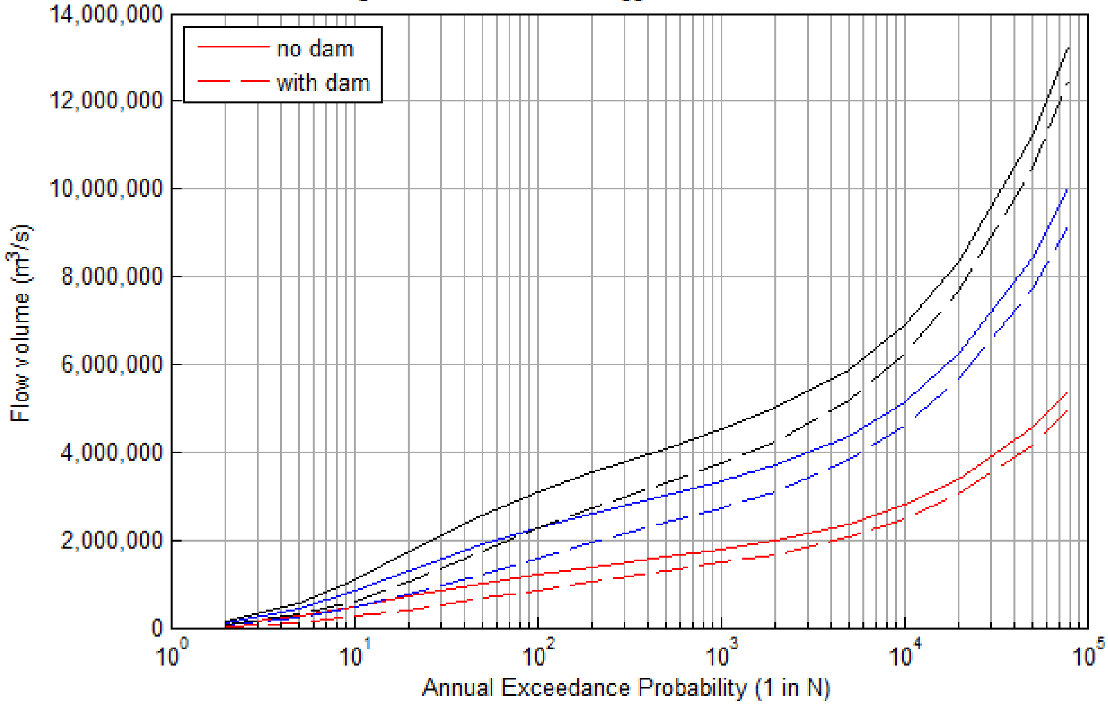
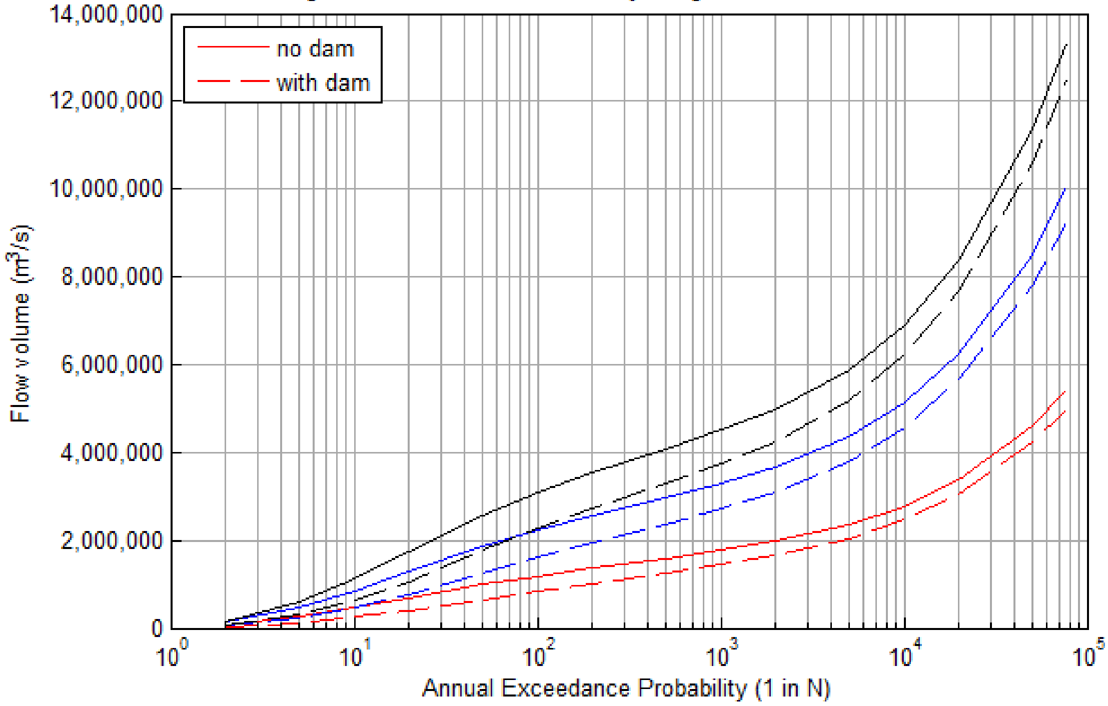


Figure E.6.7 | Location Centenary Bridge; DEA simulation results



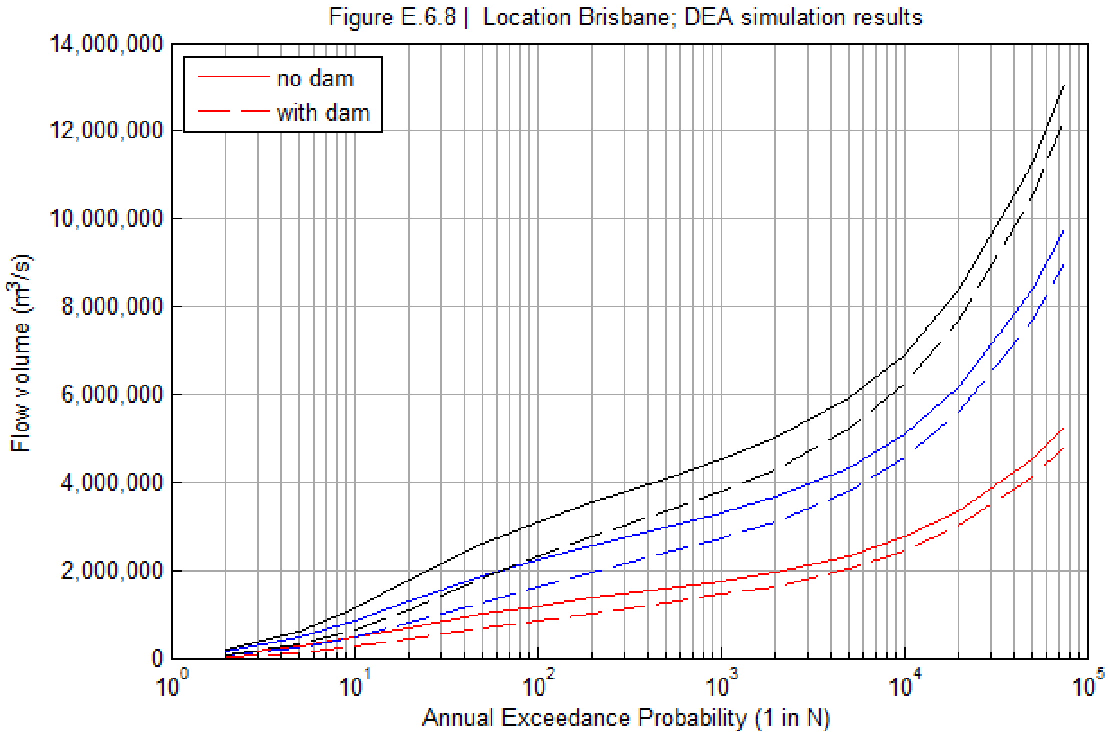


Figure E.6.8 | Location Brisbane; DEA simulation results
no dam
with dam
Flow volume (m3/s)
14,000,000
12,000,000
10,000,000
8,000,000
6,000,000
4,000,000
2,000,000
0
10^0
10^1
10^2
10^3
10^4
10^5
Annual Exceedance Probability (1 in N)

Appendix F
Peak discharge versus AEP of the rainfall depth

This appendix contains figures in which the peak discharge of each simulated event is plotted against the annual exceedance probability of the rainfall depth. Different colours are used to distinguish for different burst durations. Furthermore, three lines are added that represent the 10%, 50% and 90% quantiles. These lines can be interpreted as follows: for a given rainfall AEP, the p-percentile is the peak discharge which has a p% probability of being exceeded, given the occurrence of a rainfall event with a rainfall depth that corresponds to the AEP shown on the horizontal axis. For a given combination of rainfall burst duration and AEP, the variability of peak flows in the Figures are caused by the combined influence of the variability of initial losses, spatio-temporal rainfall patterns and, for the ‘with-dams’ case, initial reservoir volumes.

F.1 No-dams conditions

Figure F.1.1 | Location Linville; no dams conditions

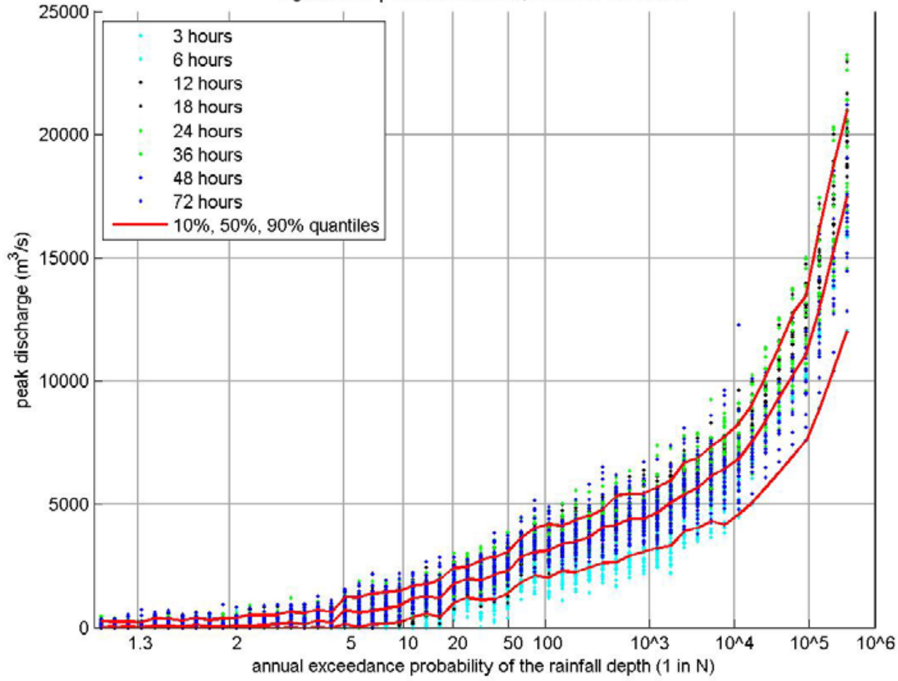


Figure F.1.2 | Location Gregors Creek; no dams conditions

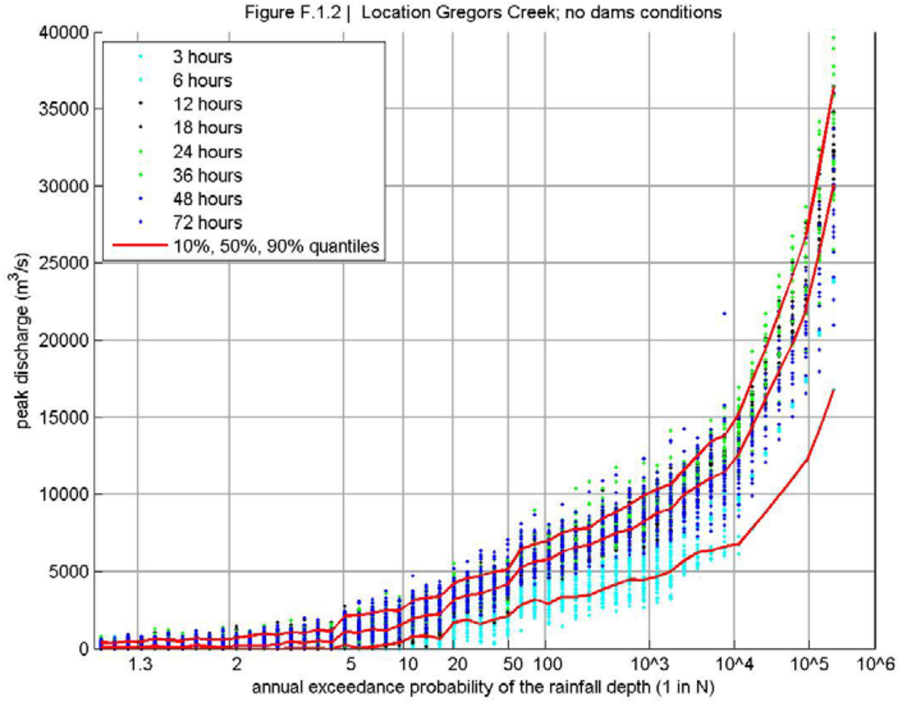


Figure F.1.3 | Location Fulham Vale; no dams conditions

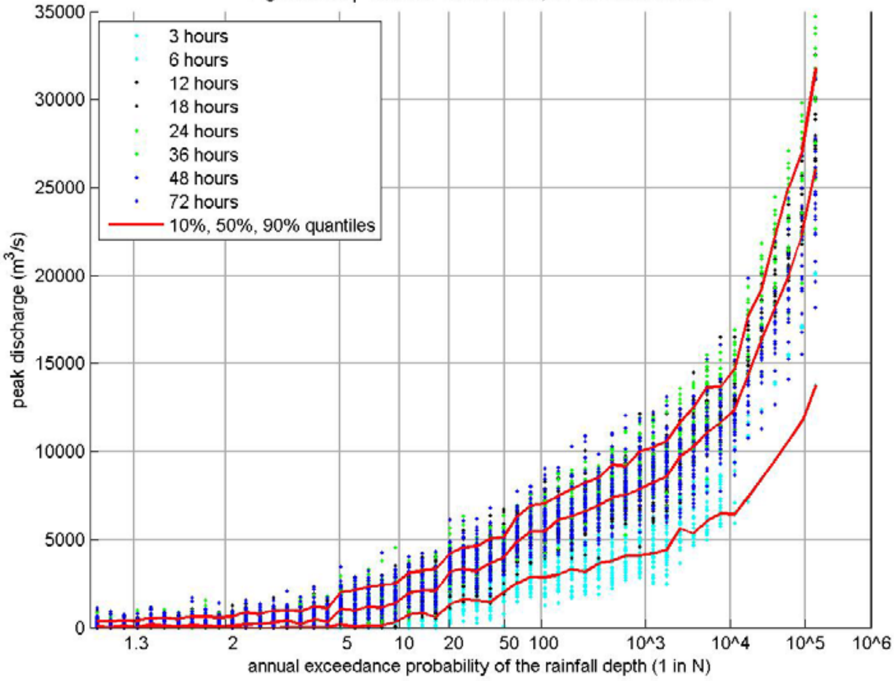
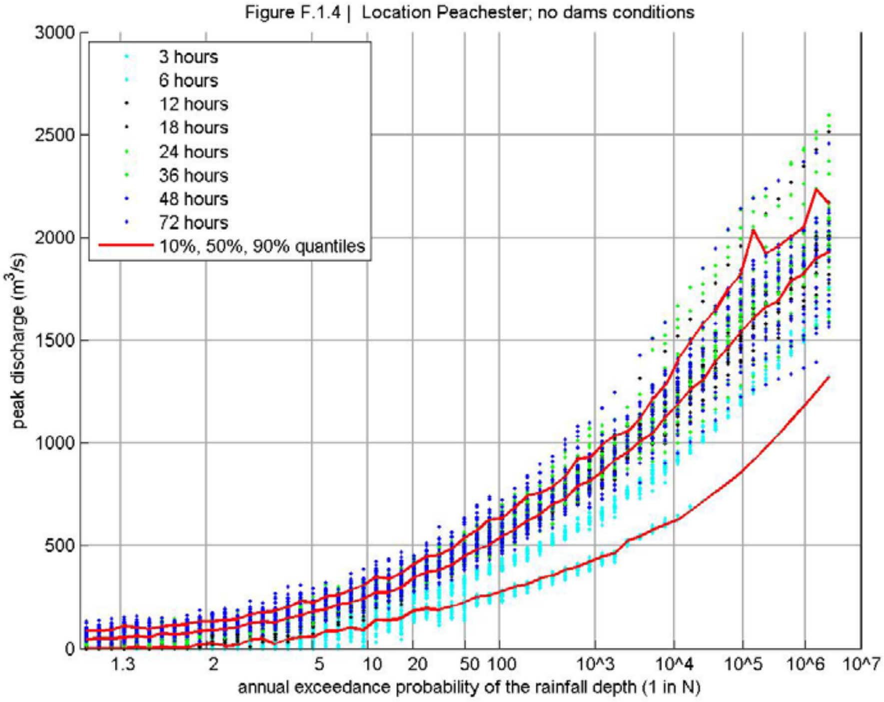


Figure F.1.4 | Location Peachester; no dams conditions



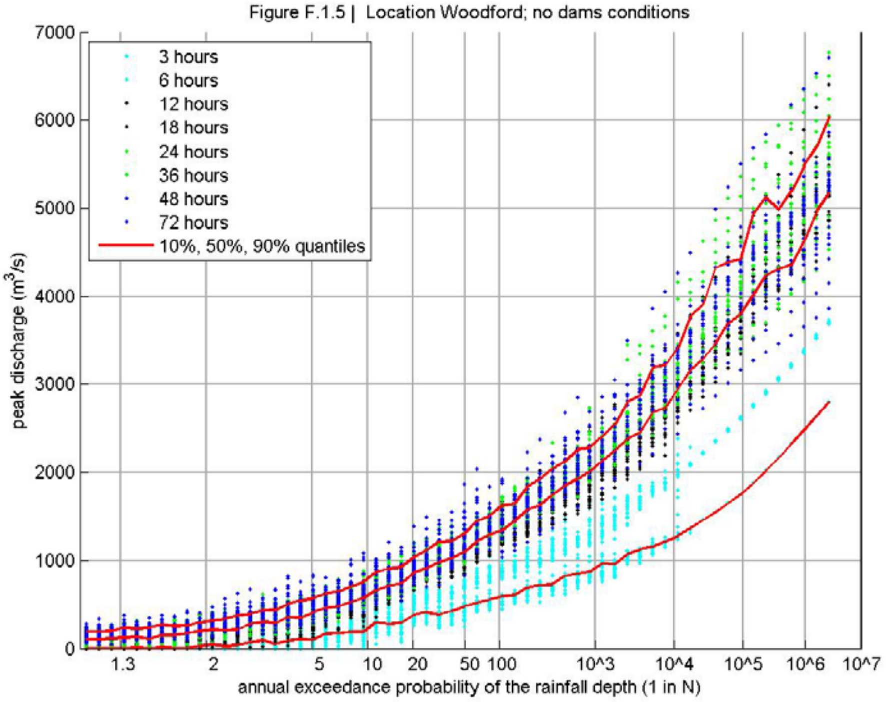


Figure F.1.5 | Location Woodford; no dams conditions

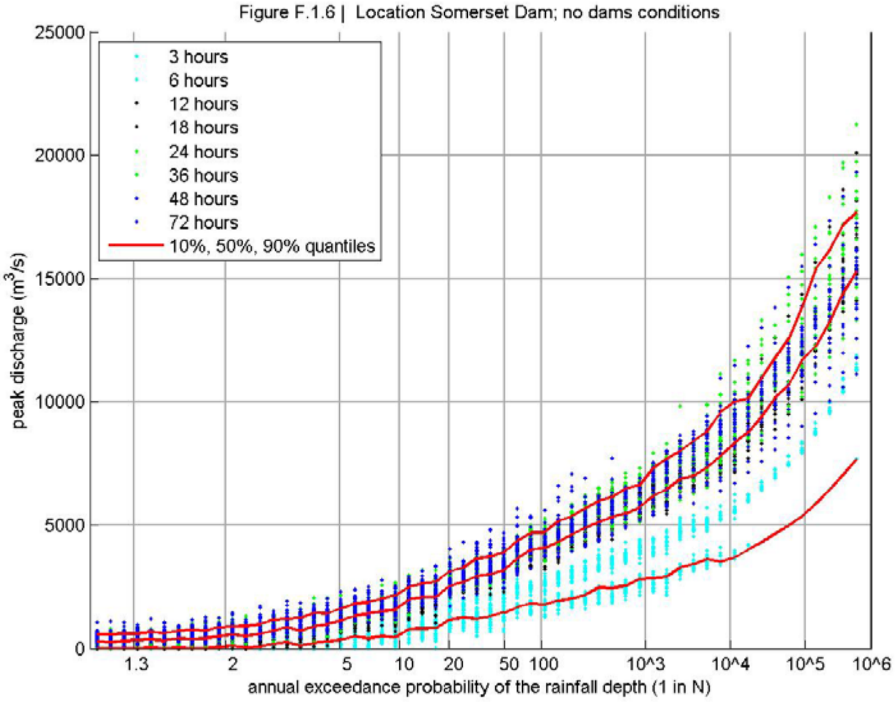


Figure F.1.6 | Location Somerset Dam; no dams conditions

Figure F.1.7 | Location Tinton; no dams conditions

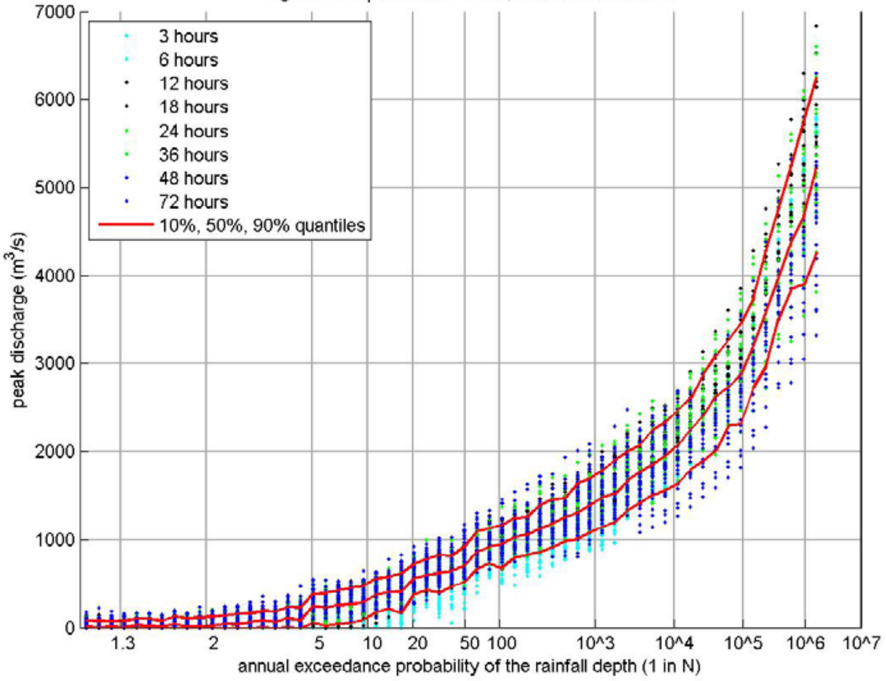


Figure F.1.8 | Location Middle Ck; no dams conditions

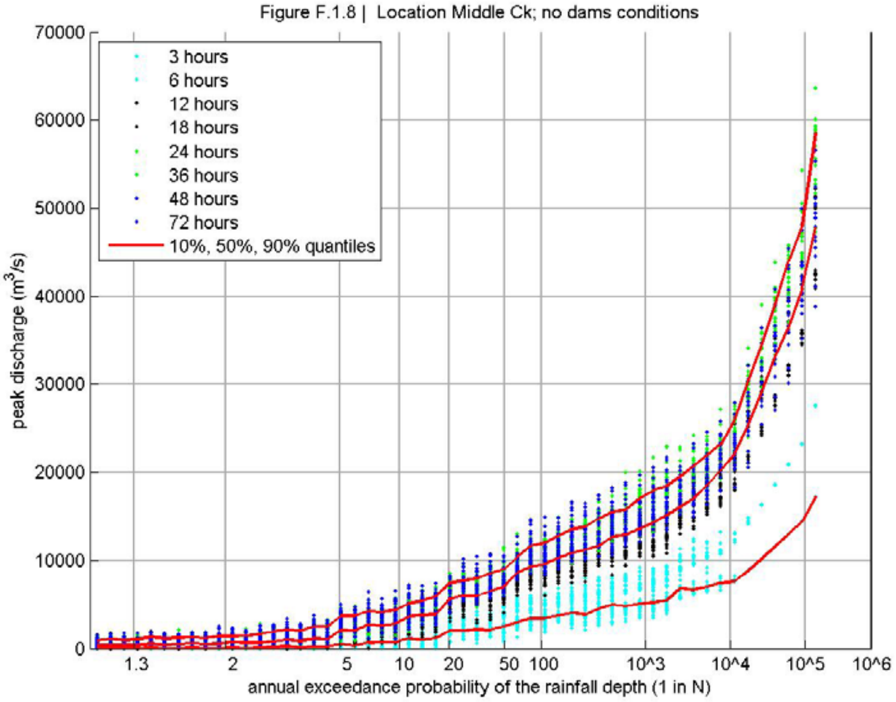


Figure F.1.9 | Location Wivenhoe; no dams conditions

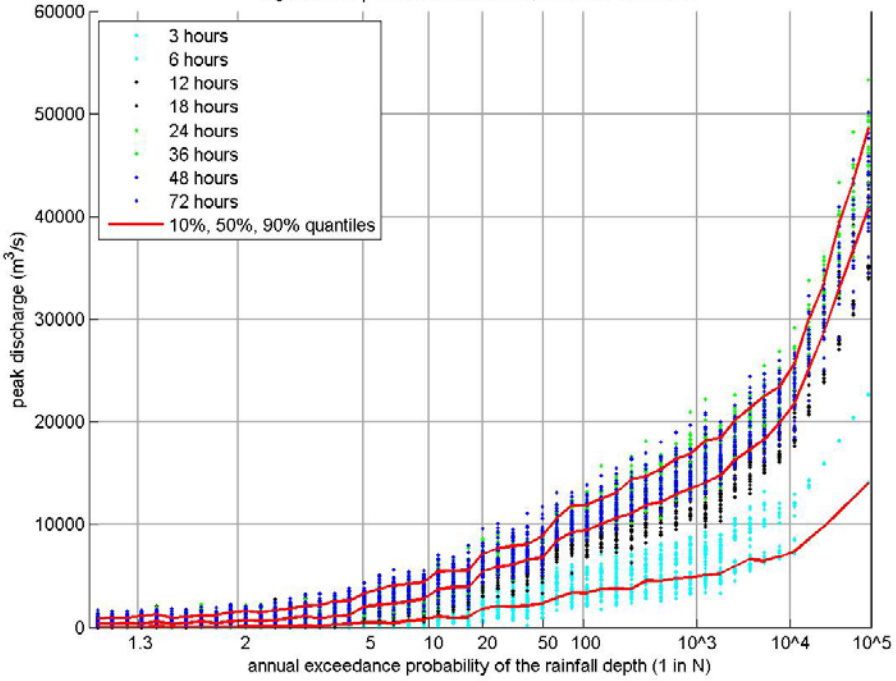


Figure F.1.10 | Location Helidon; no dams conditions

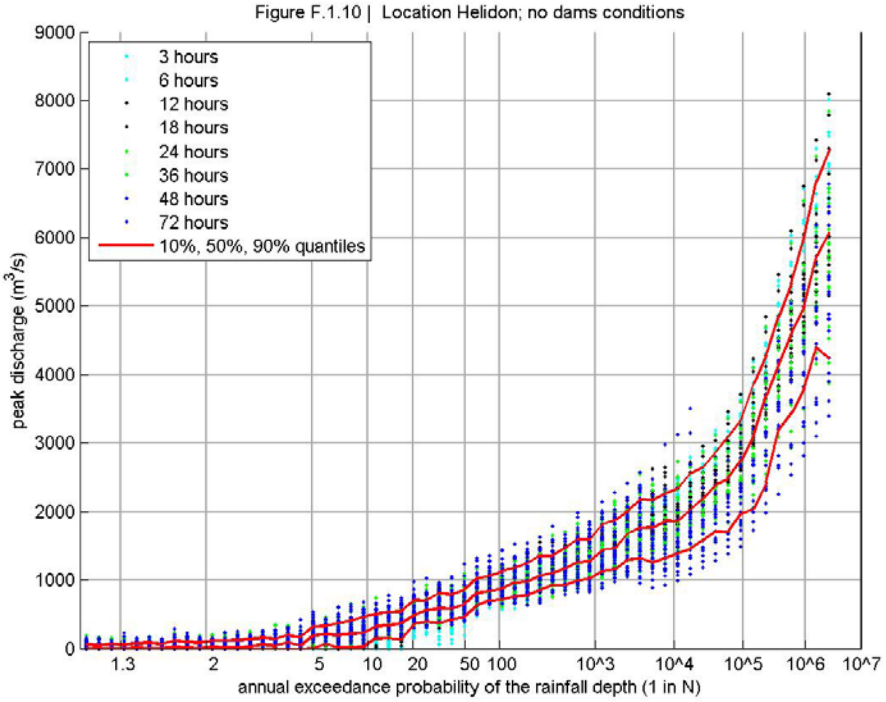


Figure F.1.11 | Location Gatton; no dams conditions

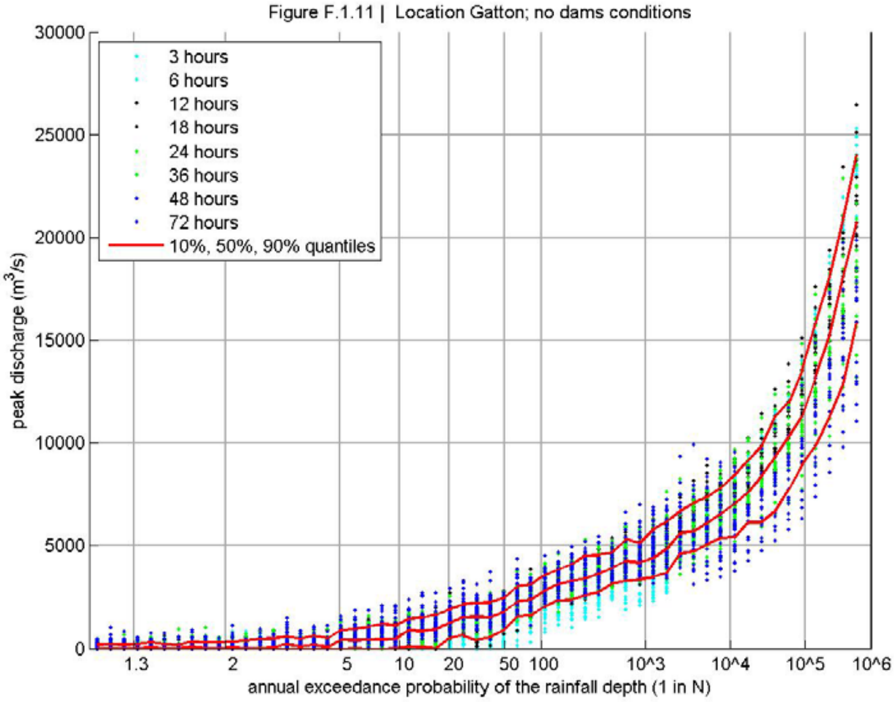


Figure F.1.12 | Location Glenore Grove; no dams conditions

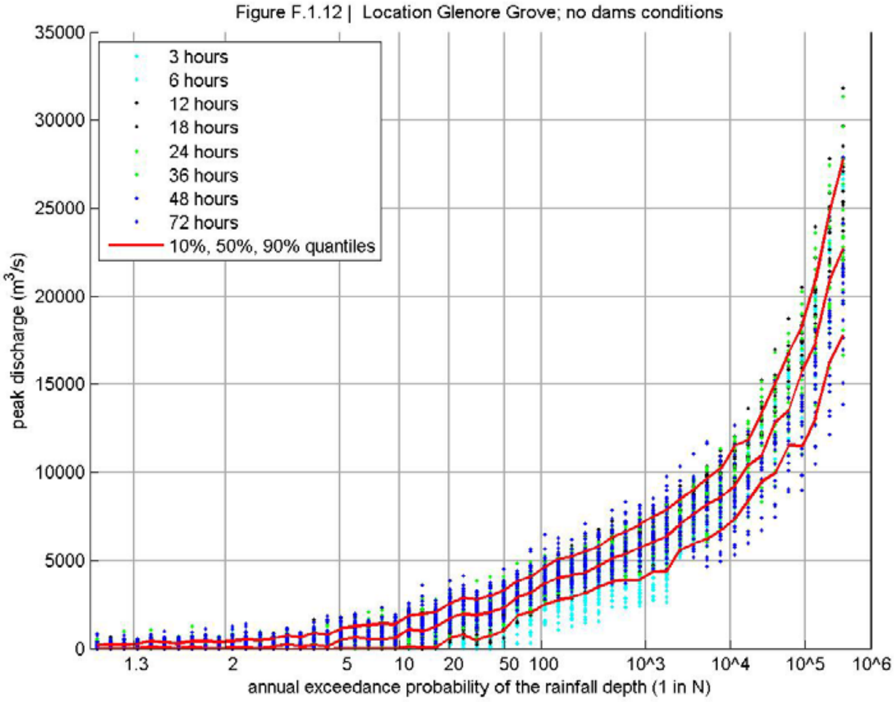


Figure F.1.13 | Location Savages Crossing; no dams conditions

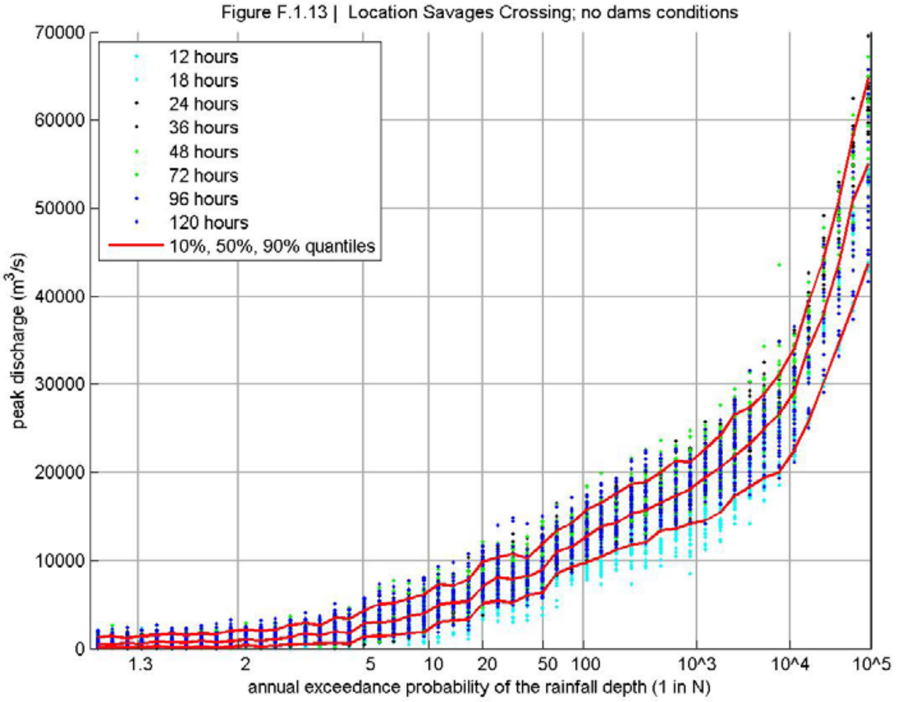


Figure F.1.14 | Location Mount Crosby; no dams conditions

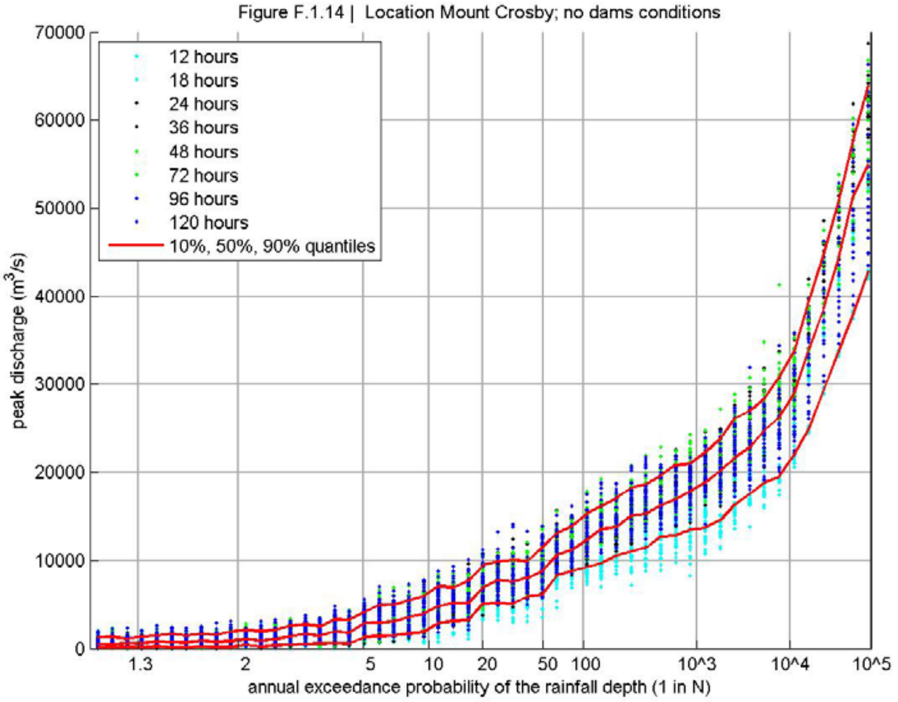


Figure F.1.15 | Location Walloon; no dams conditions

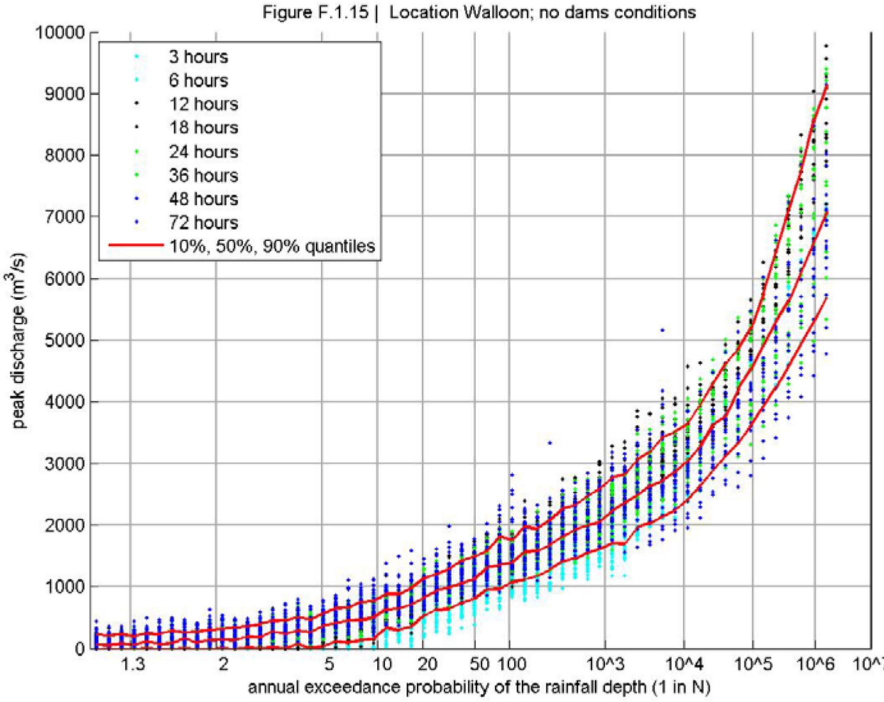


Figure F.1.16 | Location Kalbar Weir; no dams conditions

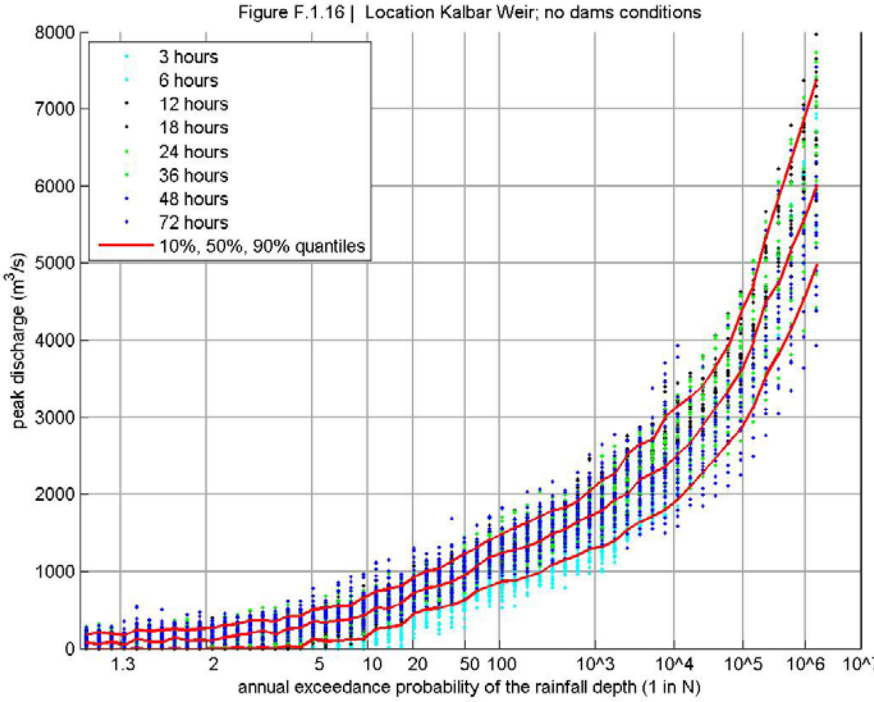


Figure F.1.17 | Location Amberley; no dams conditions

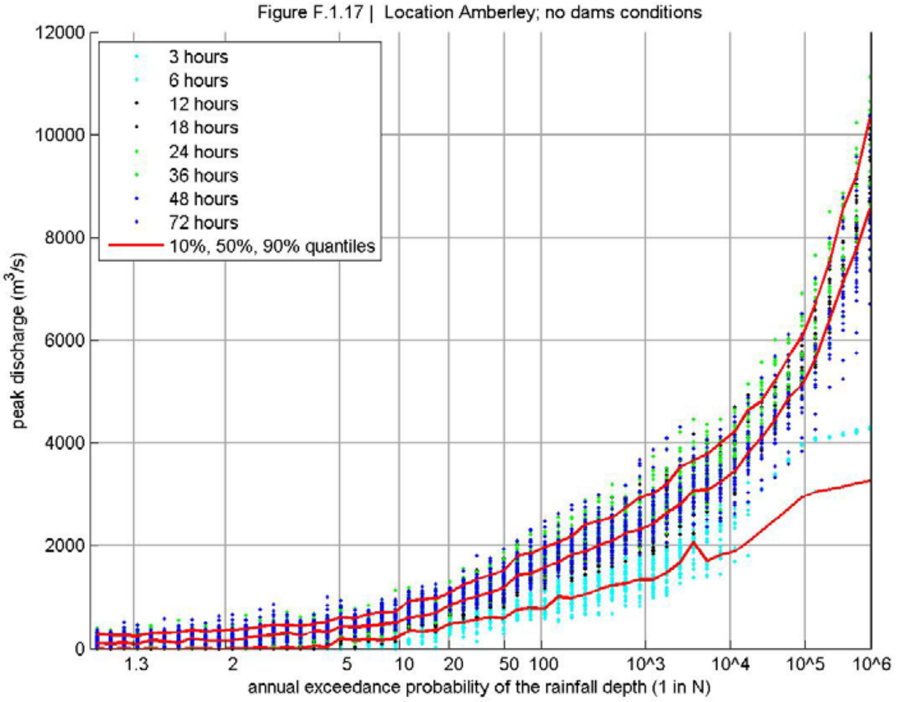


Figure F.1.18 | Location Loamside; no dams conditions

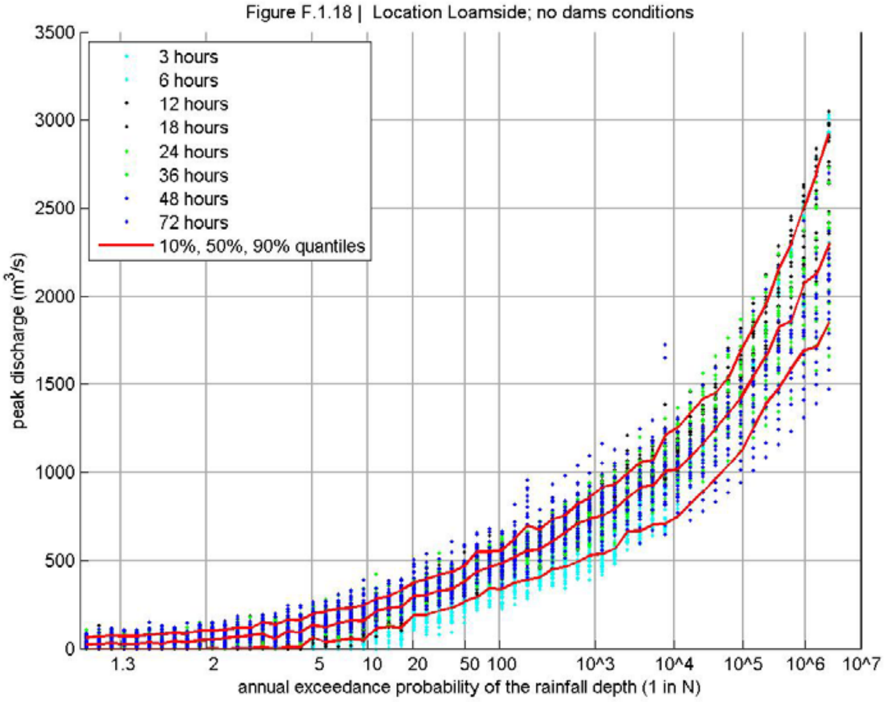


Figure F.1.19 | Location Ipswich; no dams conditions

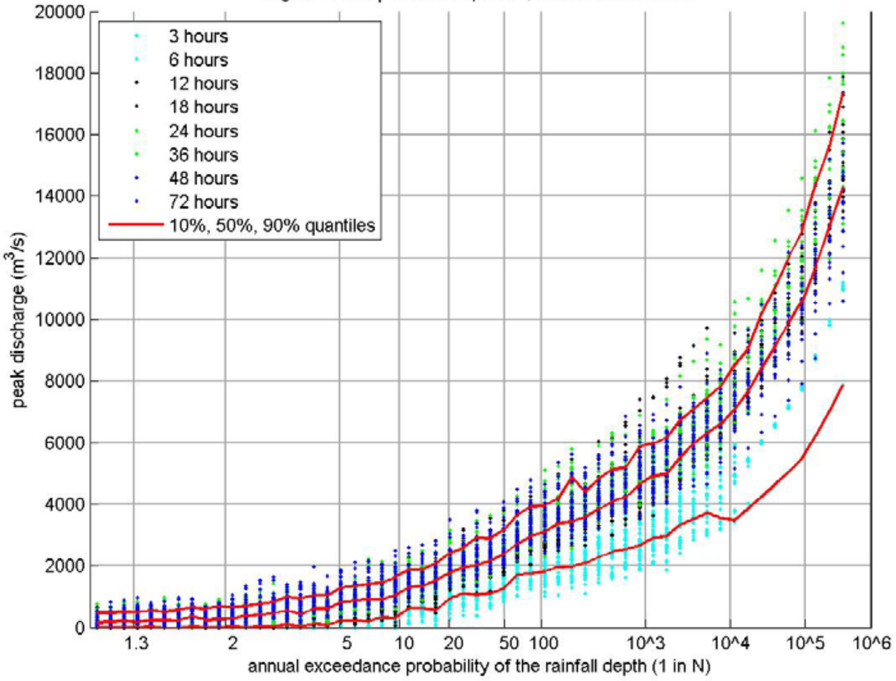


Figure F.1.20 | Location Moggill; no dams conditions

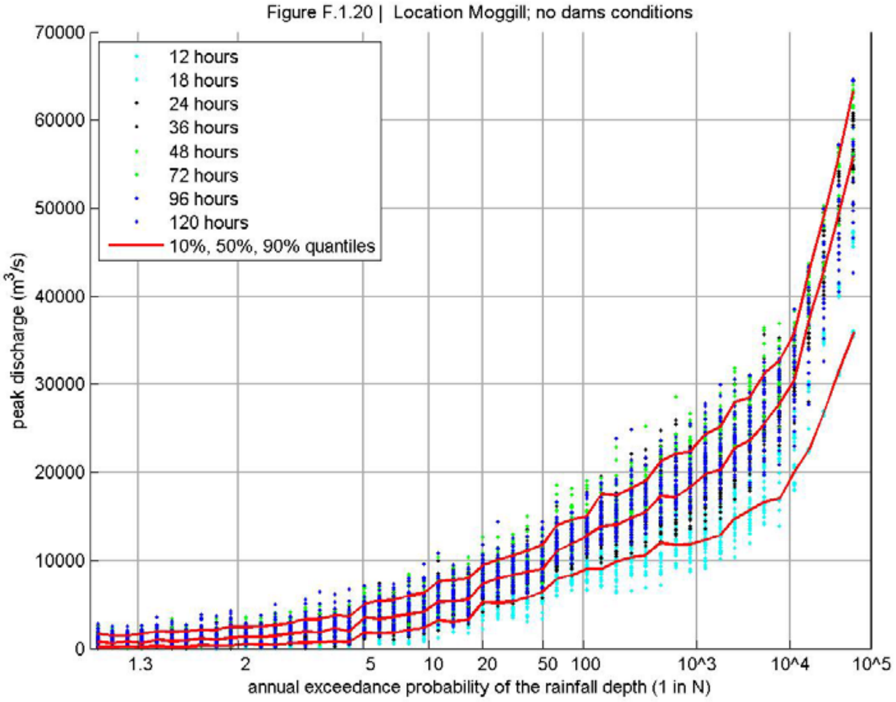


Figure F.1.21 | Location Centenary Bridge; no dams conditions

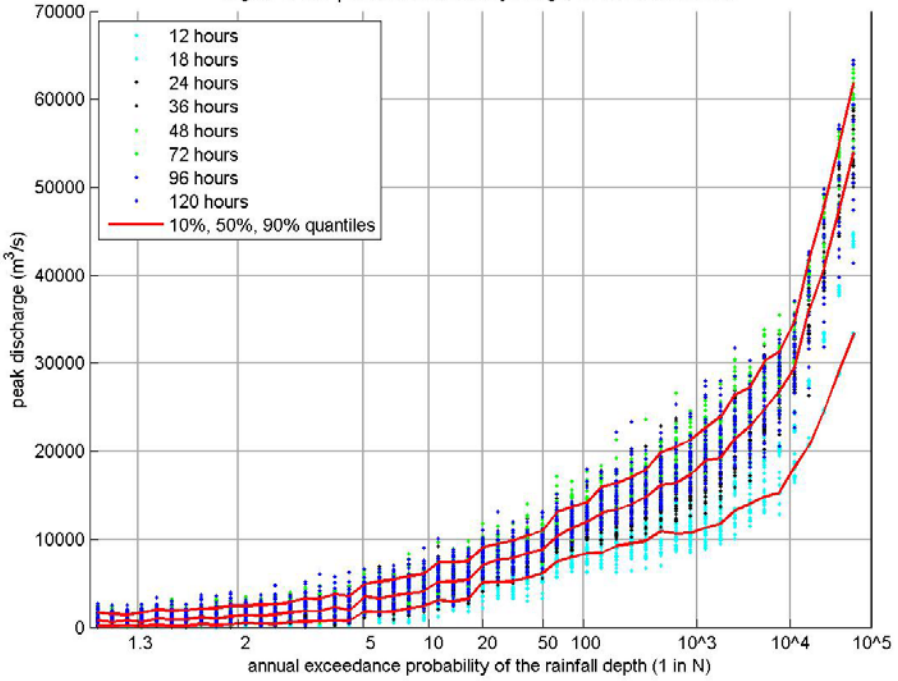
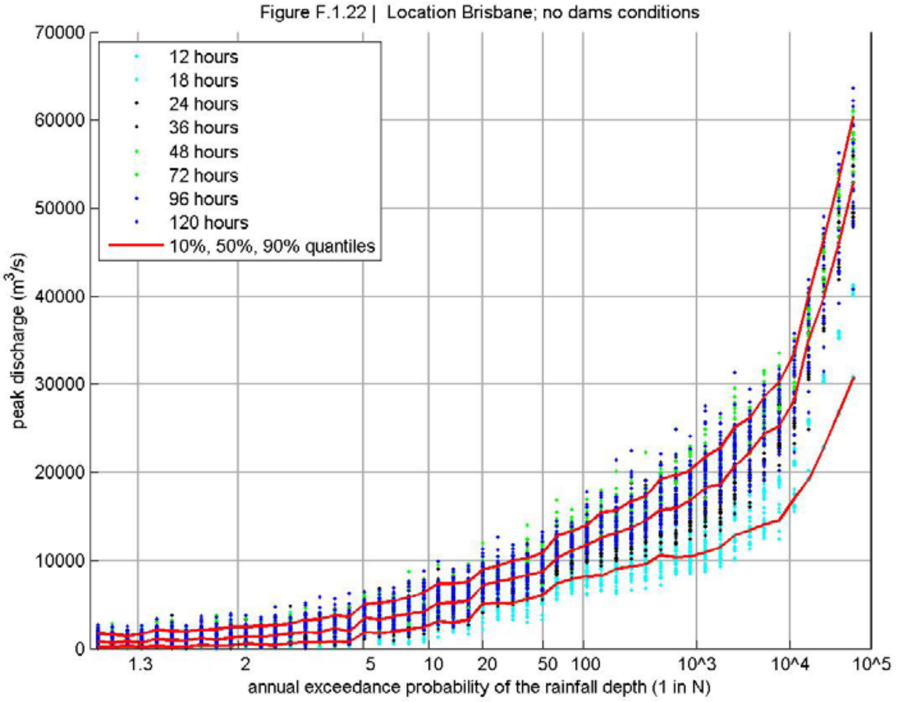
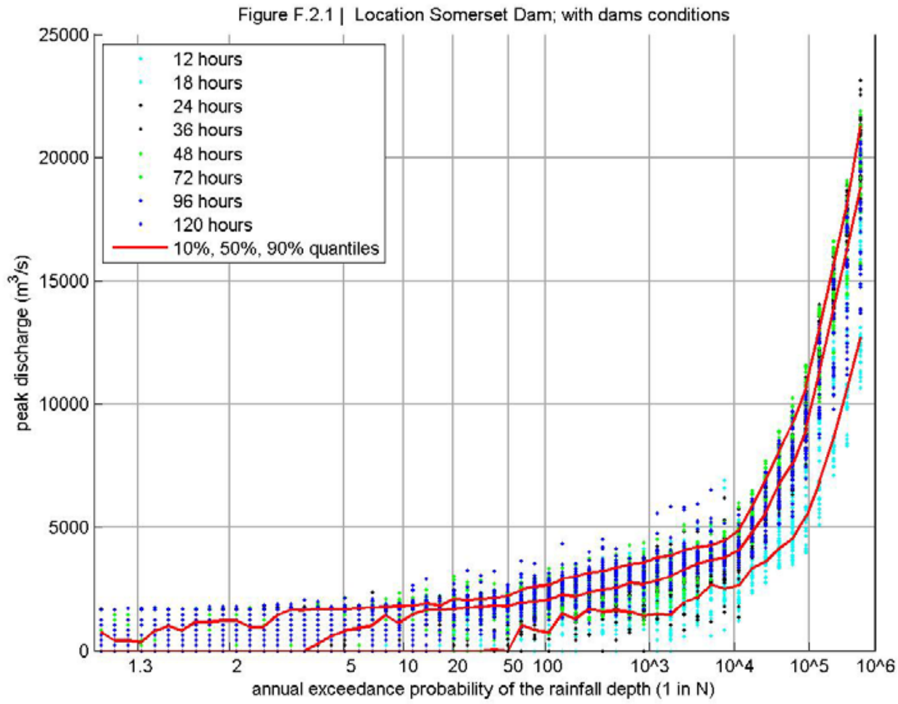


Figure F.1.22 | Location Brisbane; no dams conditions



F.2 With-dams conditions



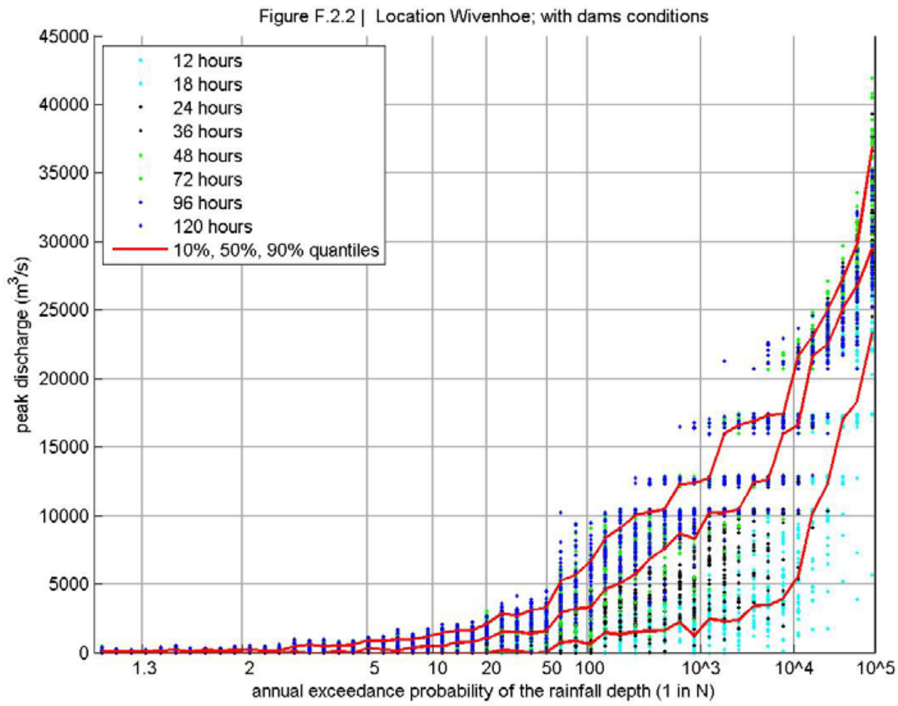


Figure F.2.3 | Location Savages Crossing; with dams conditions

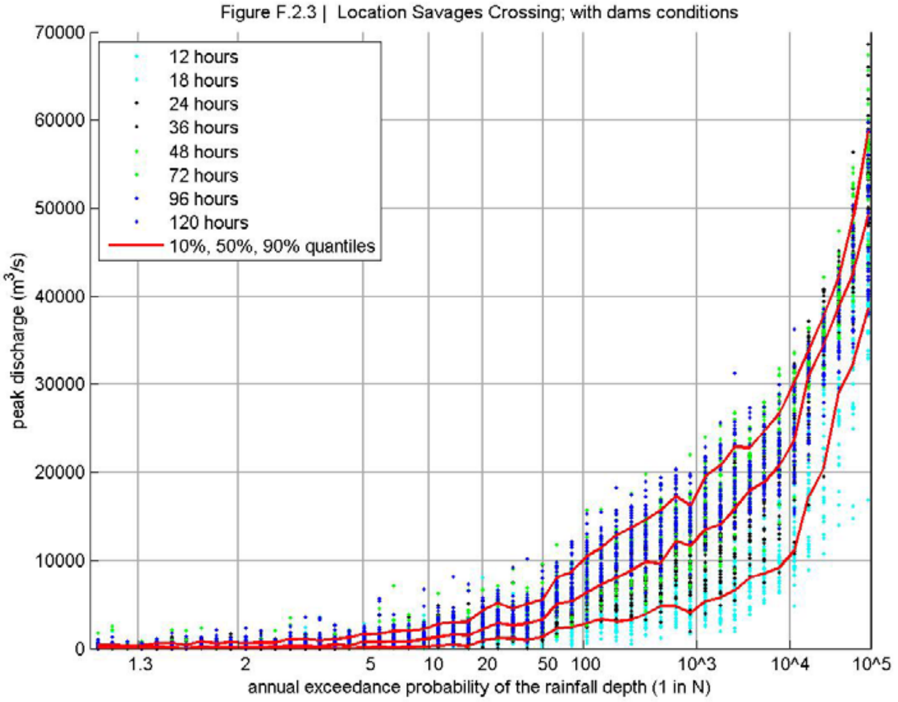


Figure F.2.4 | Location Mount Crosby; with dams conditions

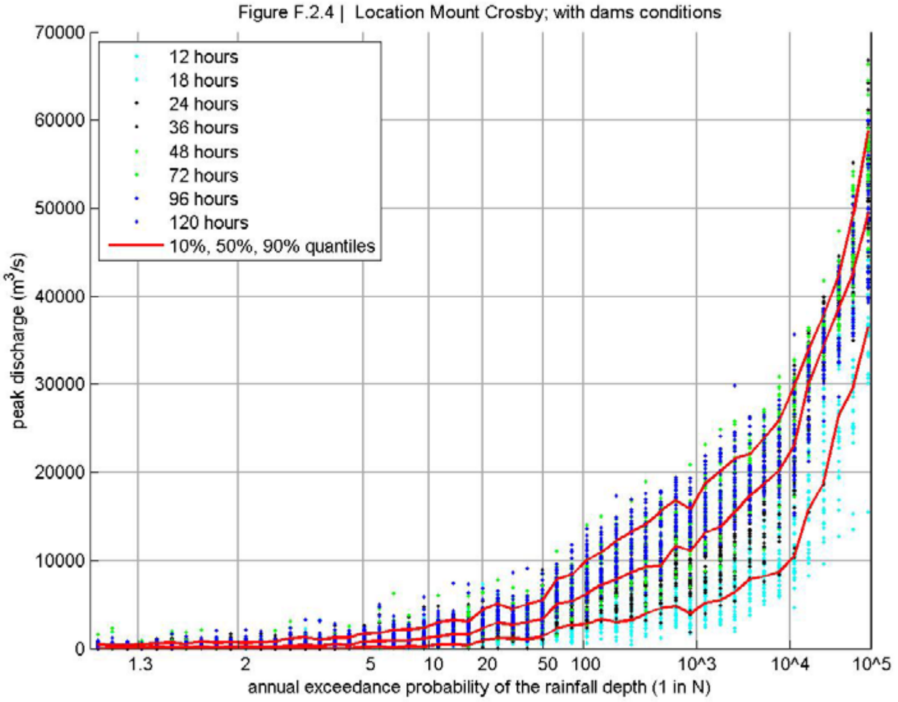


Figure F.2.5 | Location Ipswich; with dams conditions

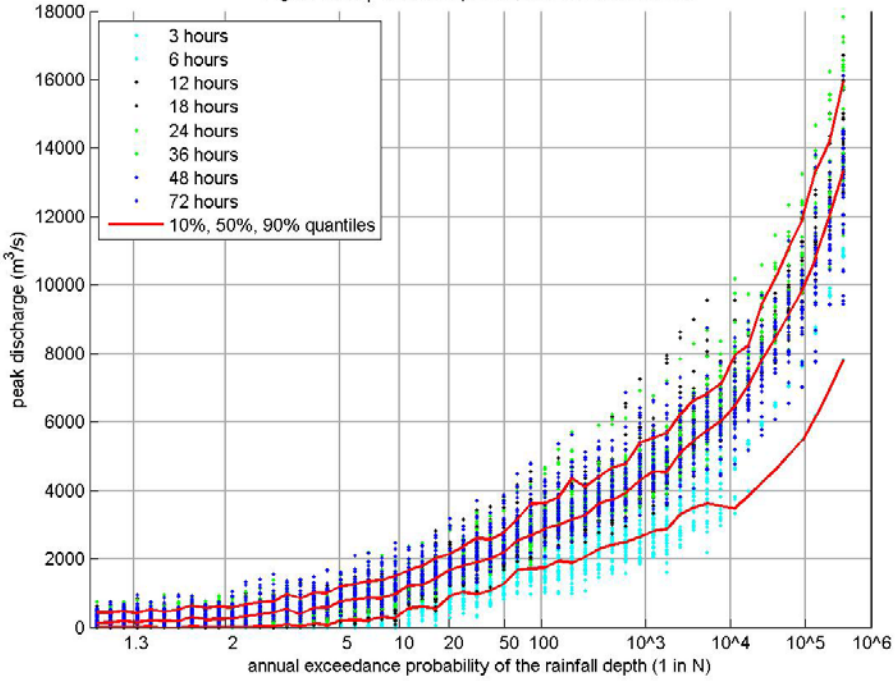


Figure F.2.6 | Location Moggill; with dams conditions

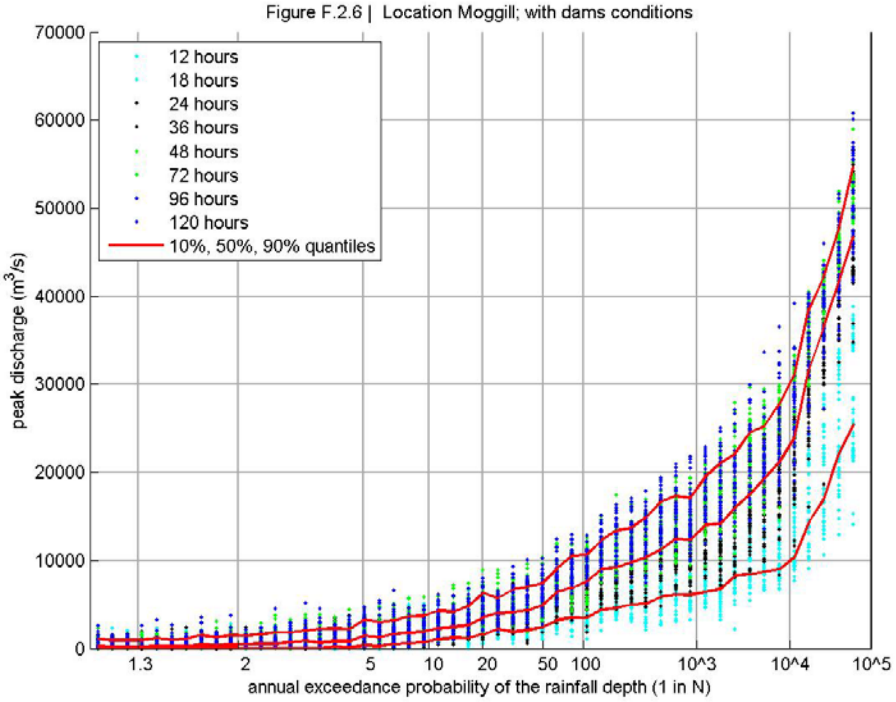


Figure F.2.7 | Location Centenary Bridge; with dams conditions

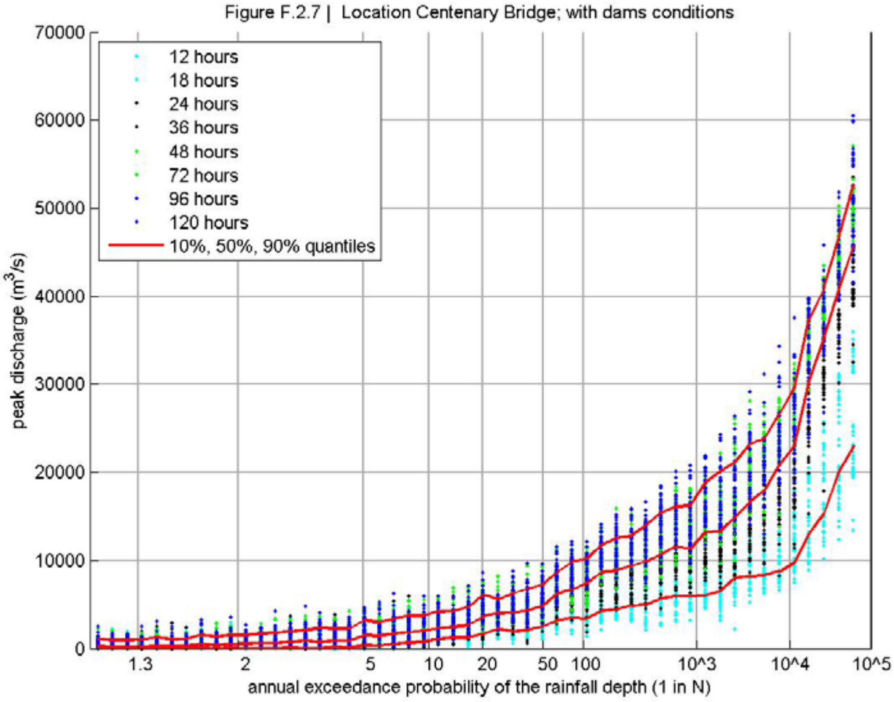
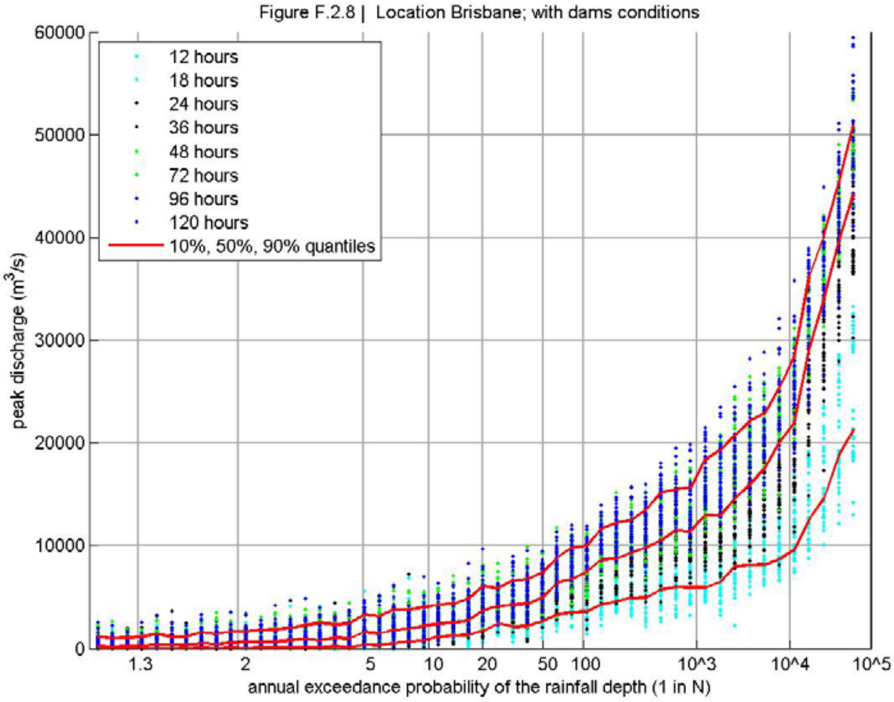


Figure F.2.8 | Location Brisbane; with dams conditions



Appendix G
Frequency curves per burst duration

This appendix contains Figures that show frequency curves for various burst durations.

G.1 No-dams conditions

Figure G.1.1 | Location Linville; no dams conditions; catchment size: 1,996 km^2

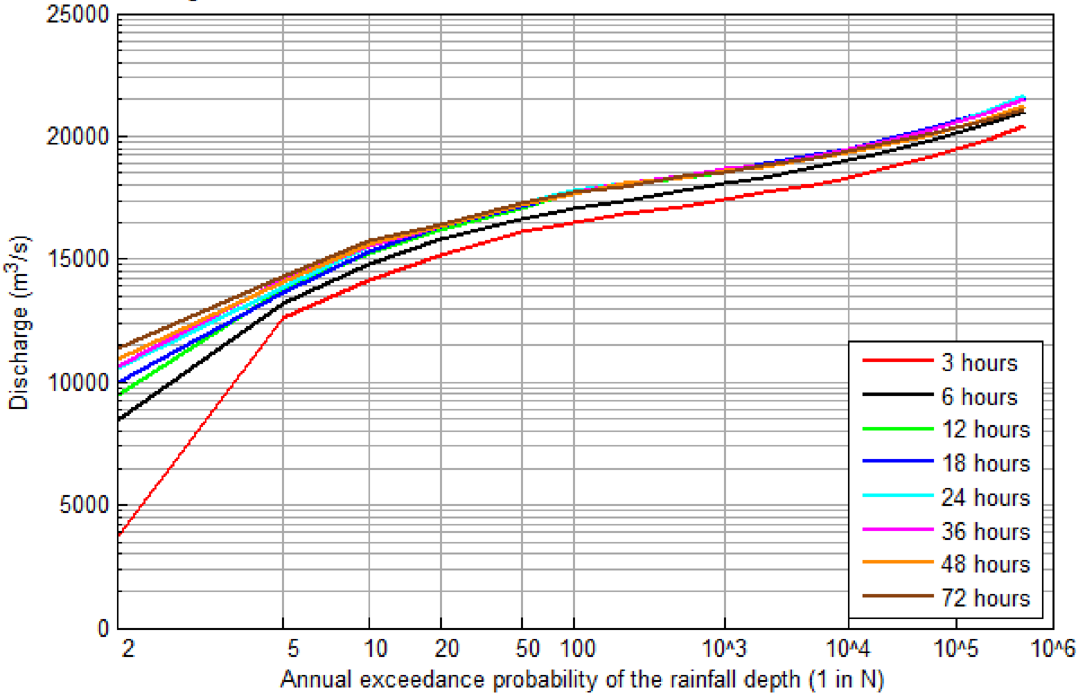


Figure G.1.2 | Location Gregors Creek; no dams conditions; catchment size: 3,849 km^2

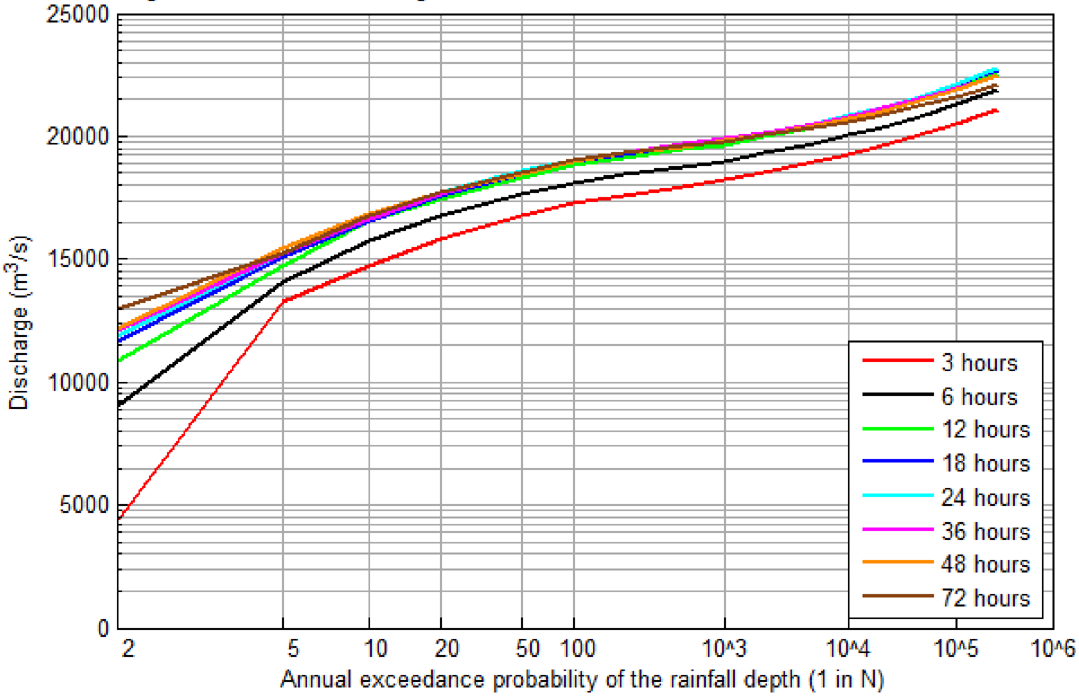


Figure G.1.3 | Location Fulham Vale; no dams conditions; catchment size: 4,553 km^2

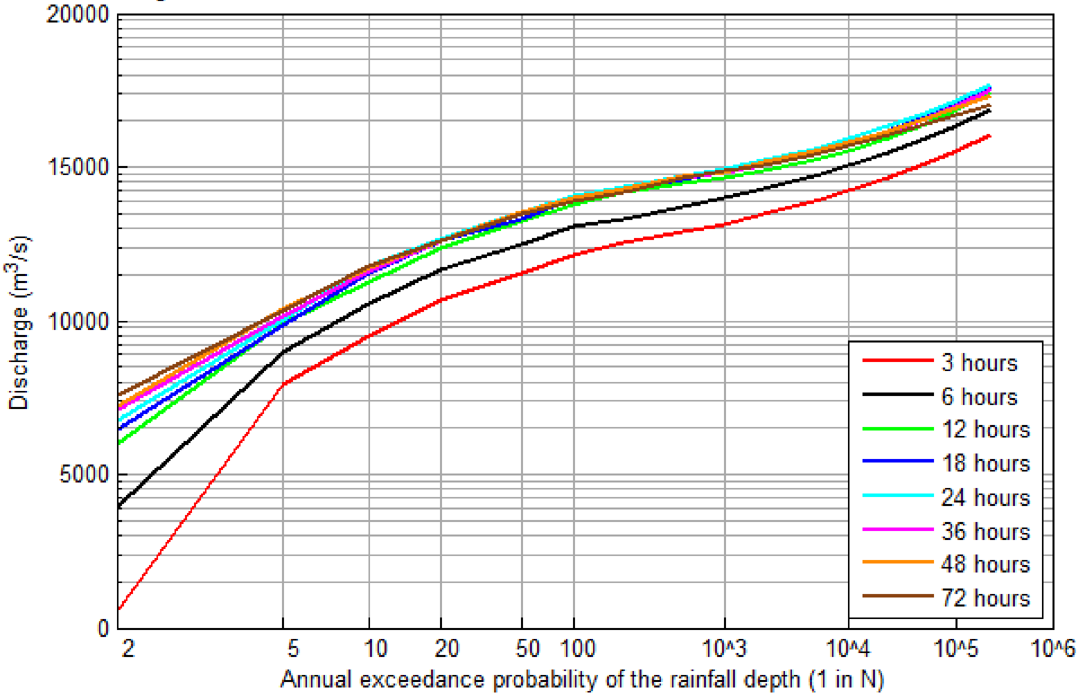


Figure G.1.4 | Location Peachester; no dams conditions; catchment size: 103 km^2

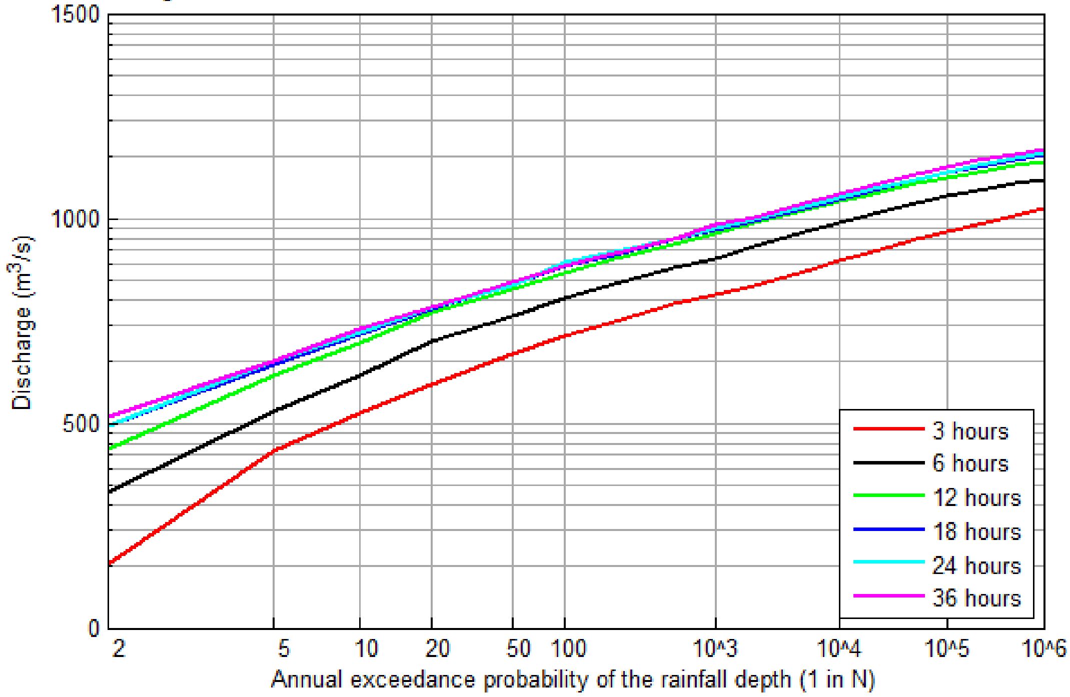


Figure G.1.5 | Location Woodford; no dams conditions; catchment size: 245 km^2

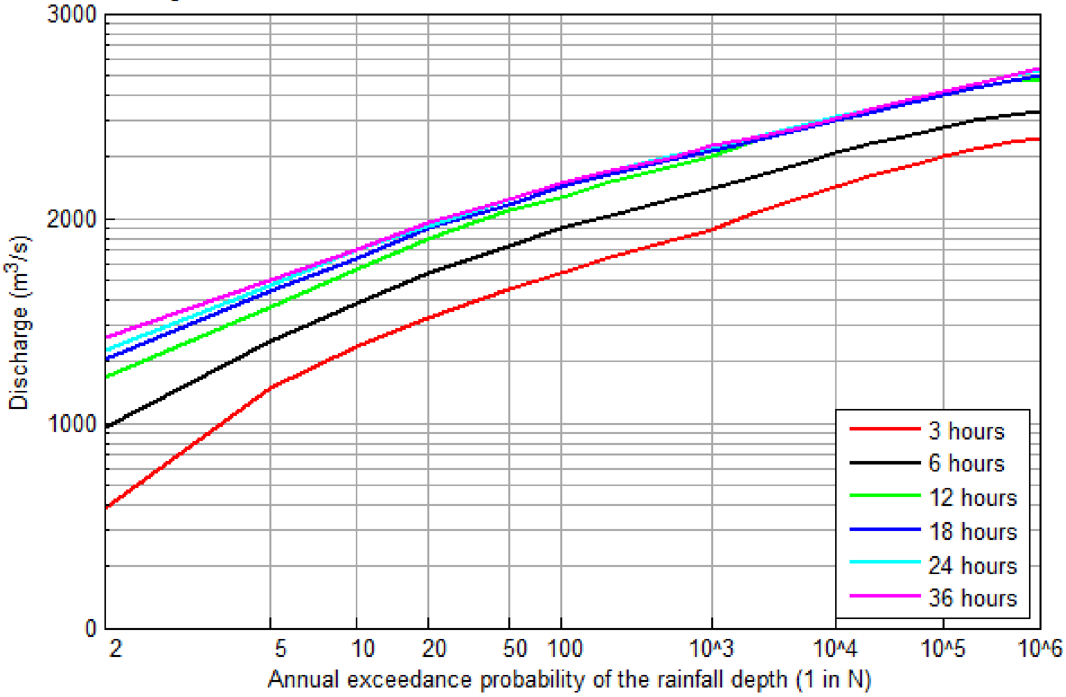


Figure G.1.6 | Location Somerset Dam; no dams conditions; catchment size: 1,324 km^2

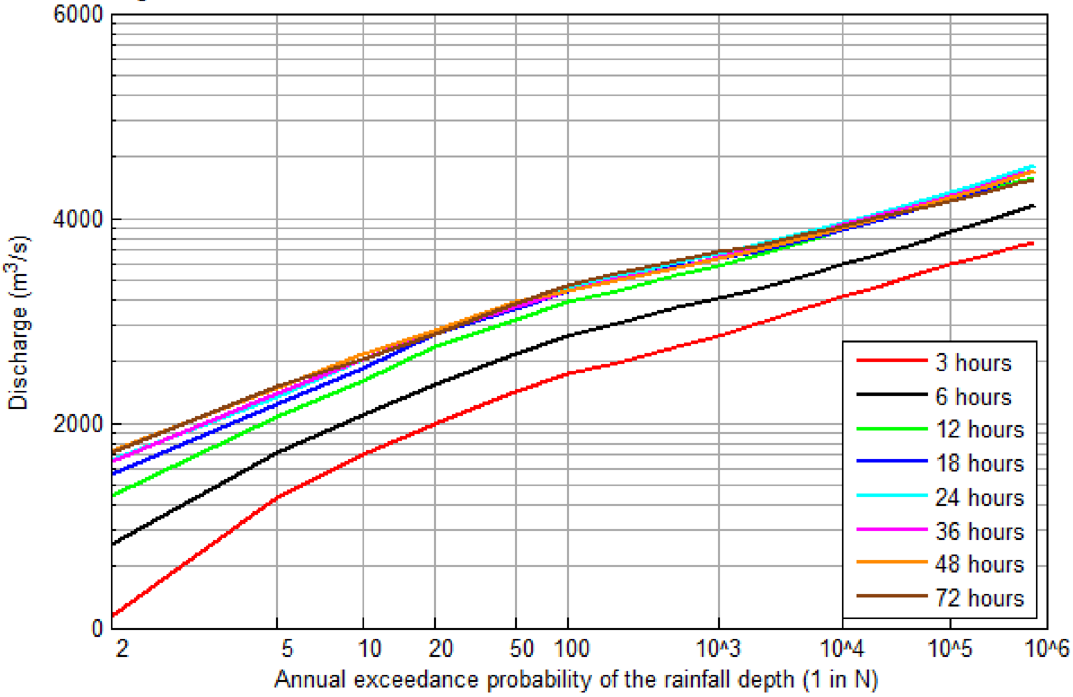


Figure G.1.7 | Location Tinton; no dams conditions; catchment size: 423 km^2

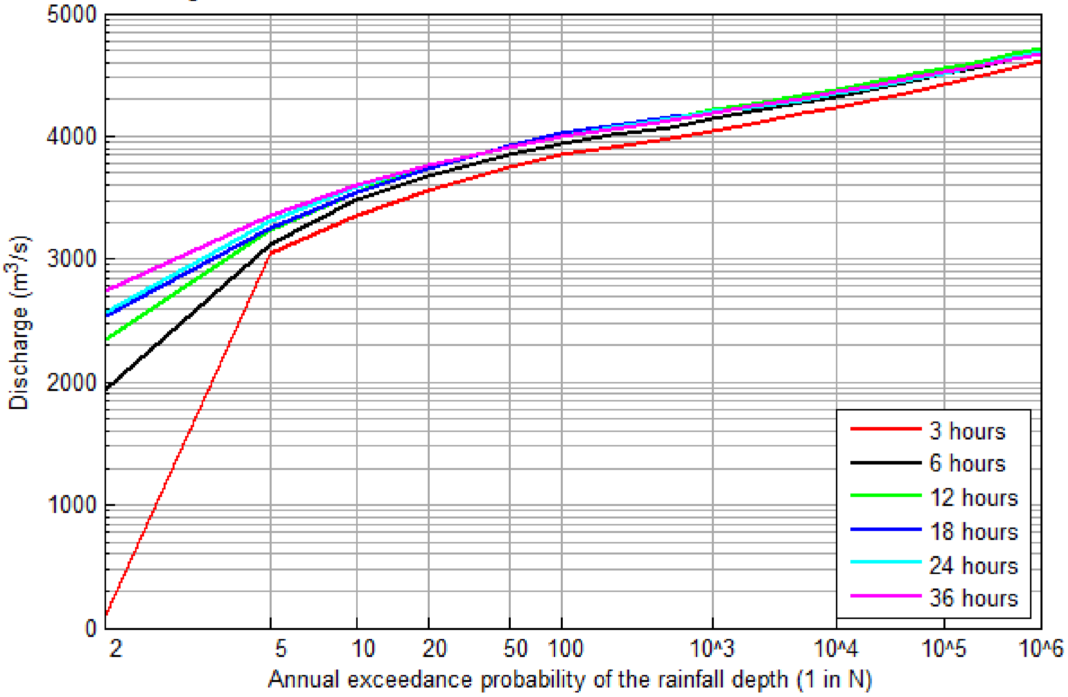


Figure G.1.8 | Location Middle Ck; no dams conditions; catchment size: 6,665 km^2

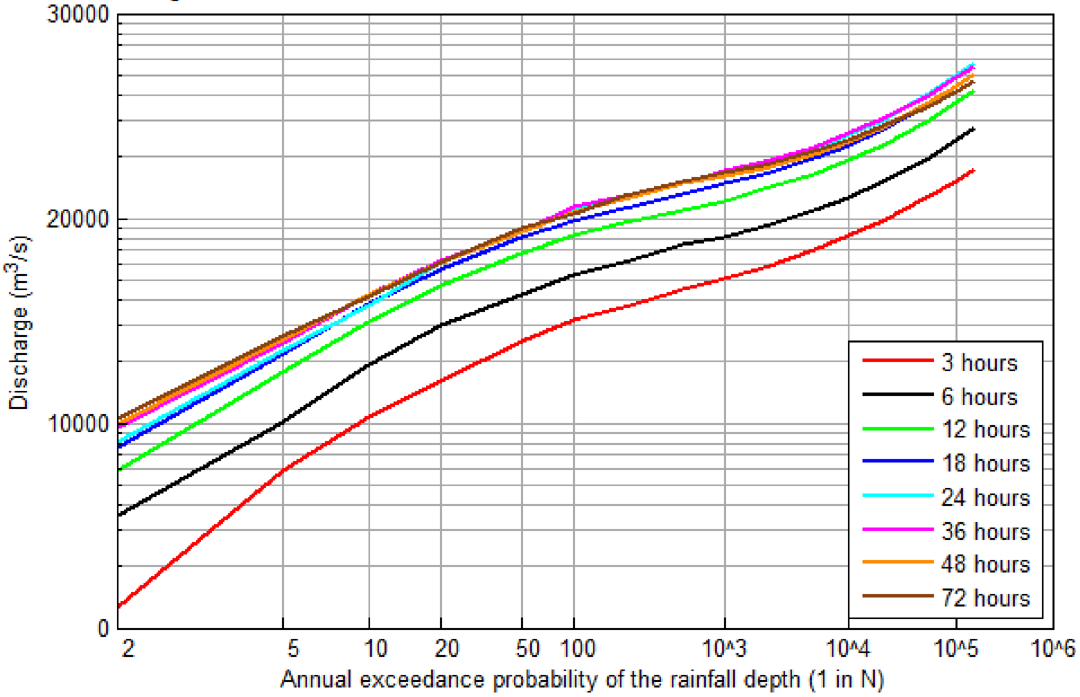


Figure G.1.9 | Location Wivenhoe; no dams conditions; catchment size: 6,980 km^2

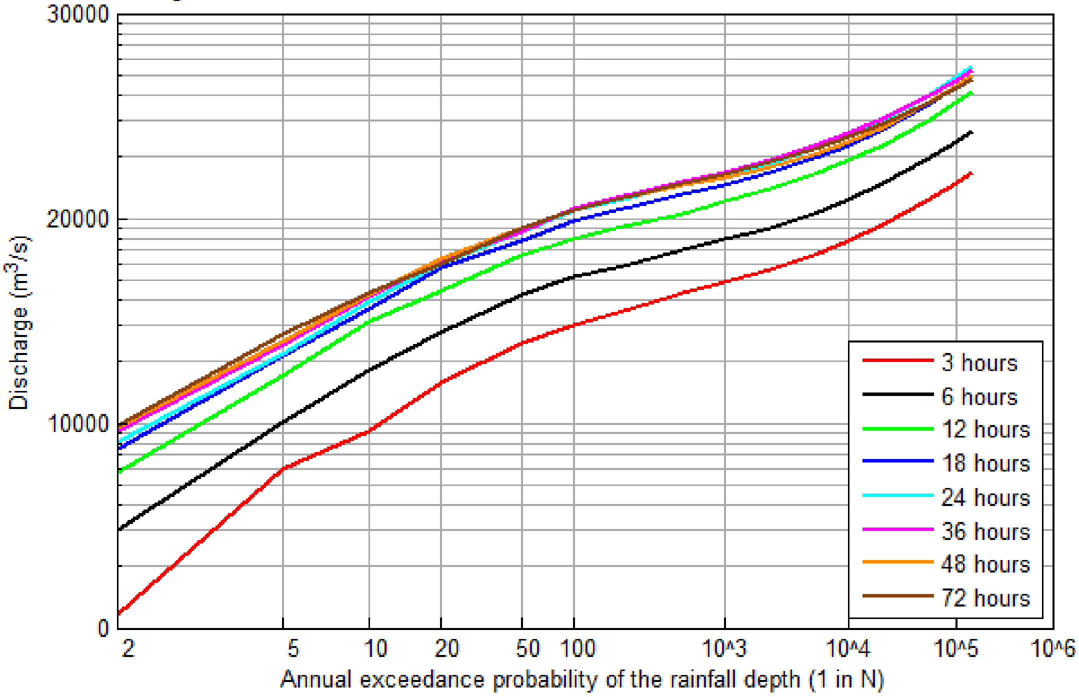


Figure G.1.10 | Location Helidon; no dams conditions; catchment size: 351 km^2

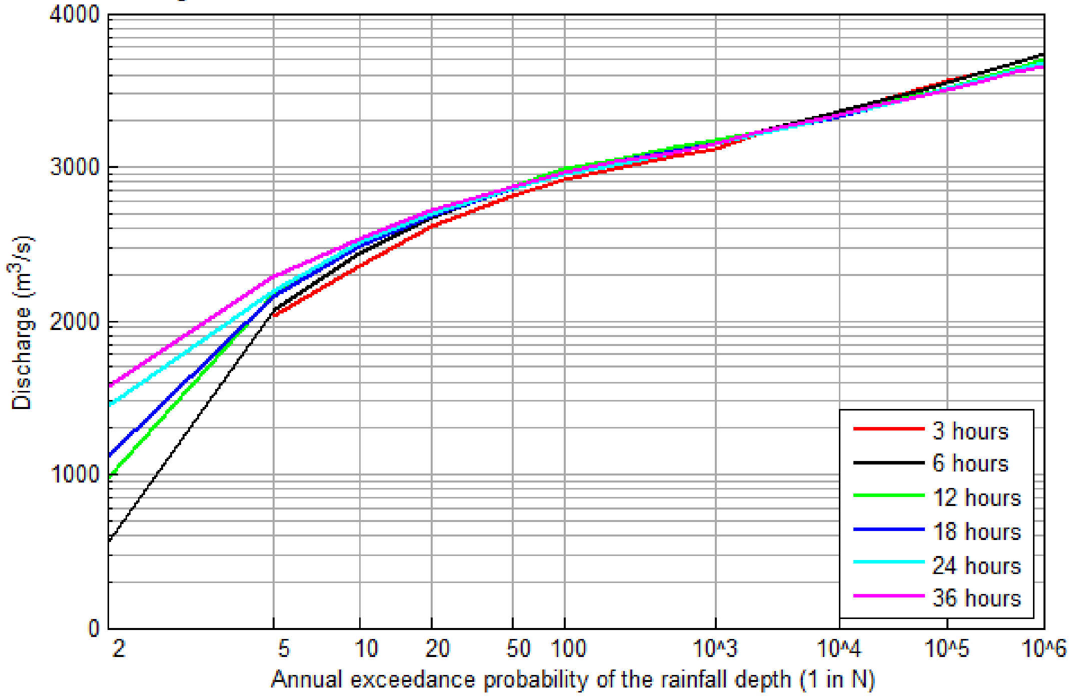


Figure G.1.11 | Location Gatton; no dams conditions; catchment size: 1,527 km^2

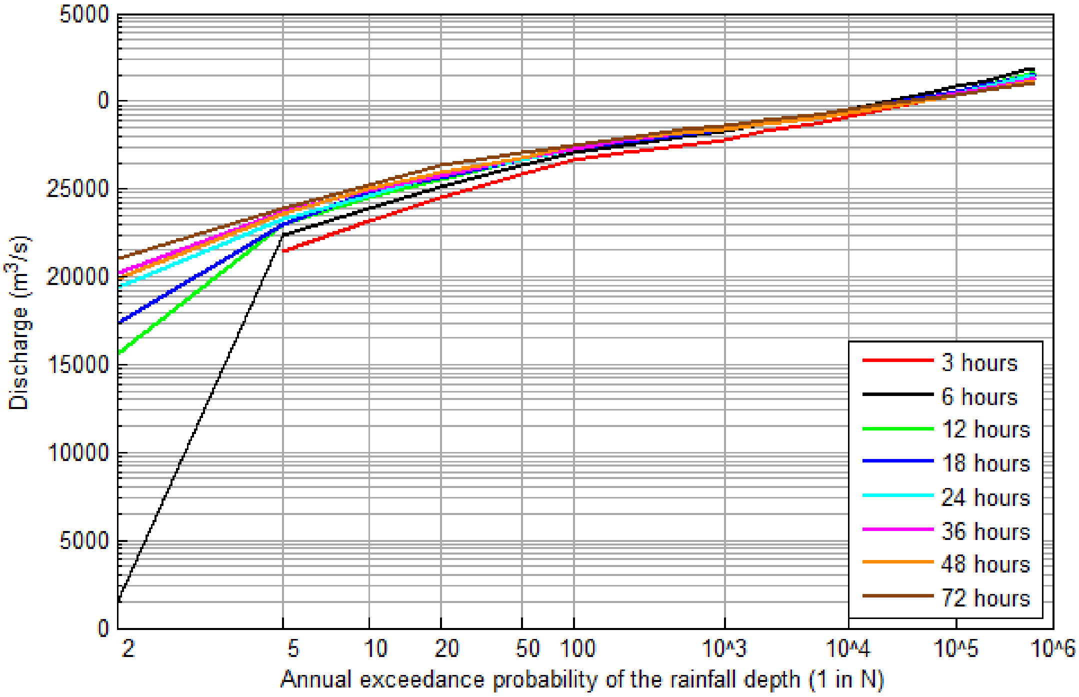
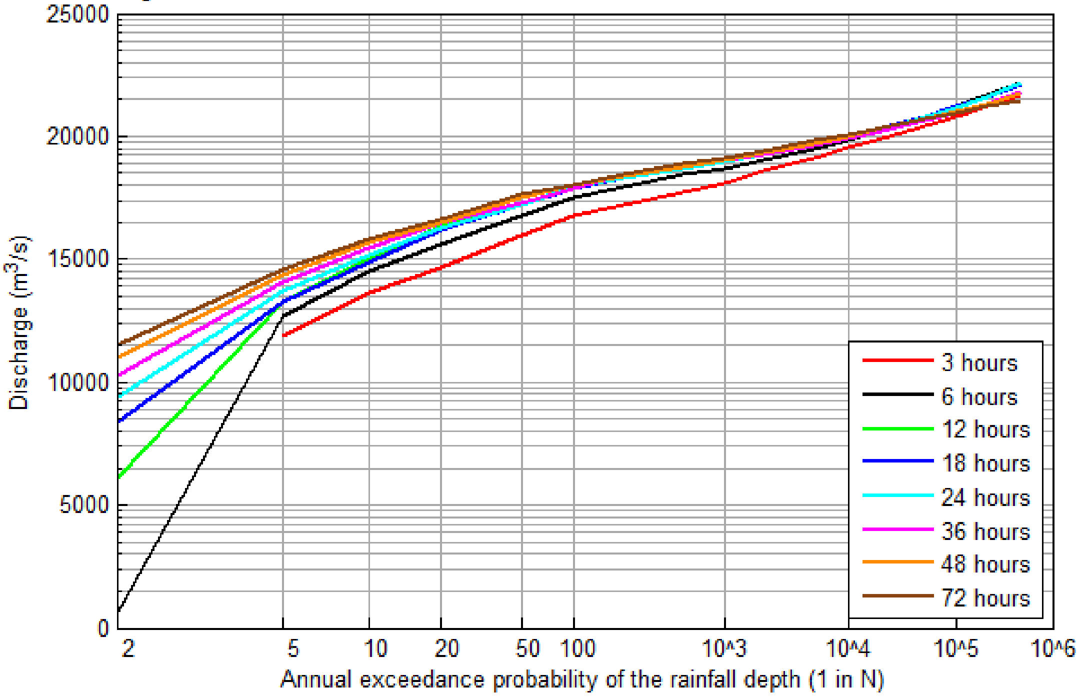


Figure G.1.12 | Location Glenore Grove; no dams conditions; catchment size: 2,149 km^2



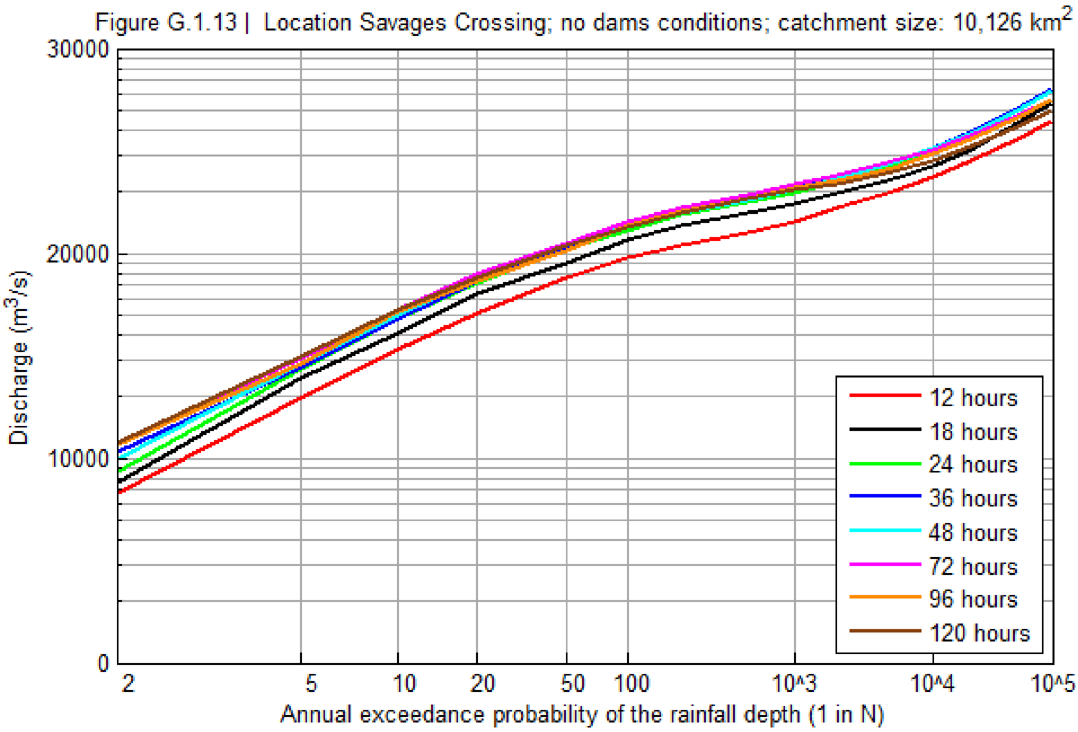


Figure G.1.13 | Location Savages Crossing; no dams conditions; catchment size: 10,126 km2
30000
20000
10000
0
Discharge (m3/s)
2
5
10
20
50
100
10^3
10^4
10^5
Annual exceedance probability of the rainfall depth (1 in N)
12 hours
18 hours
24 hours
36 hours
48 hours
72 hours
96 hours
120 hours

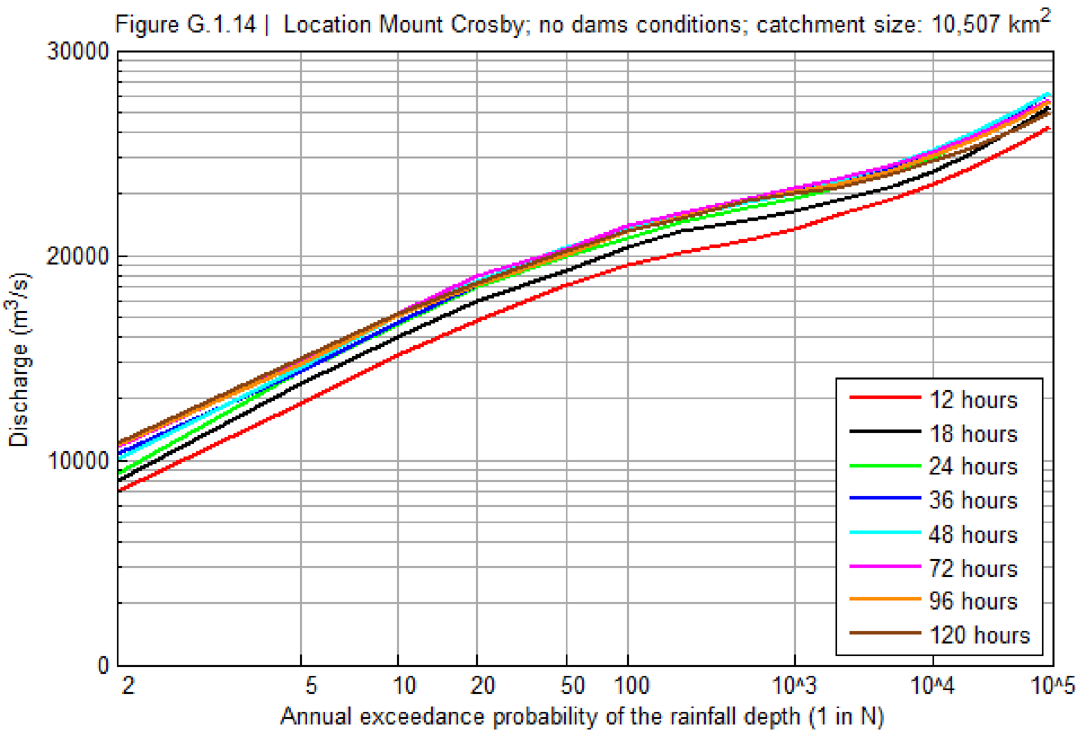


Figure G.1.14 | Location Mount Crosby; no dams conditions; catchment size: 10,507 km2
30000
20000
10000
0
Discharge (m3/s)
2
5
10
20
50
100
10^3
10^4
10^5
Annual exceedance probability of the rainfall depth (1 in N)
12 hours
18 hours
24 hours
36 hours
48 hours
72 hours
96 hours
120 hours

Figure G.1.15 | Location Walloon; no dams conditions; catchment size: 634 km^2

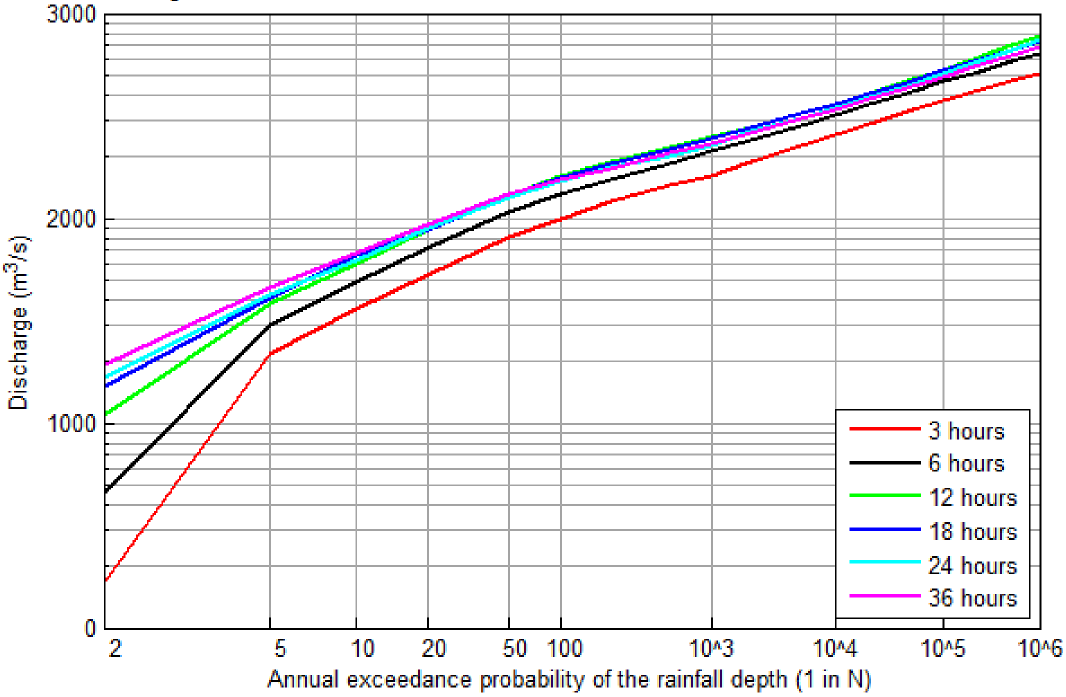
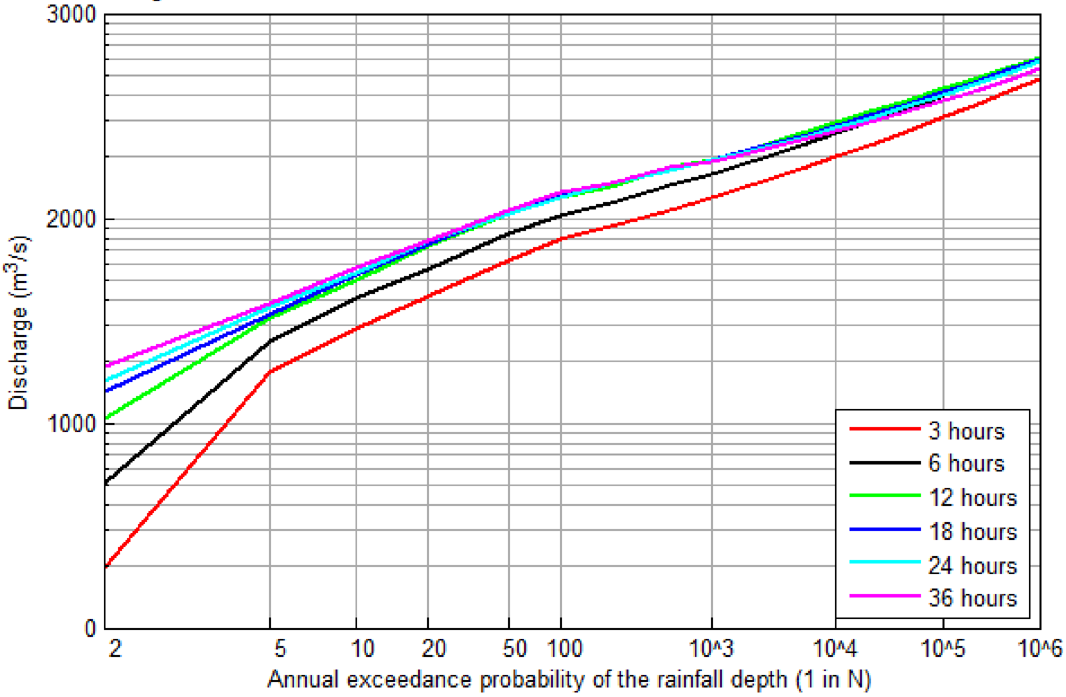


Figure G.1.16 | Location Kalbar Weir; no dams conditions; catchment size: 458 km^2



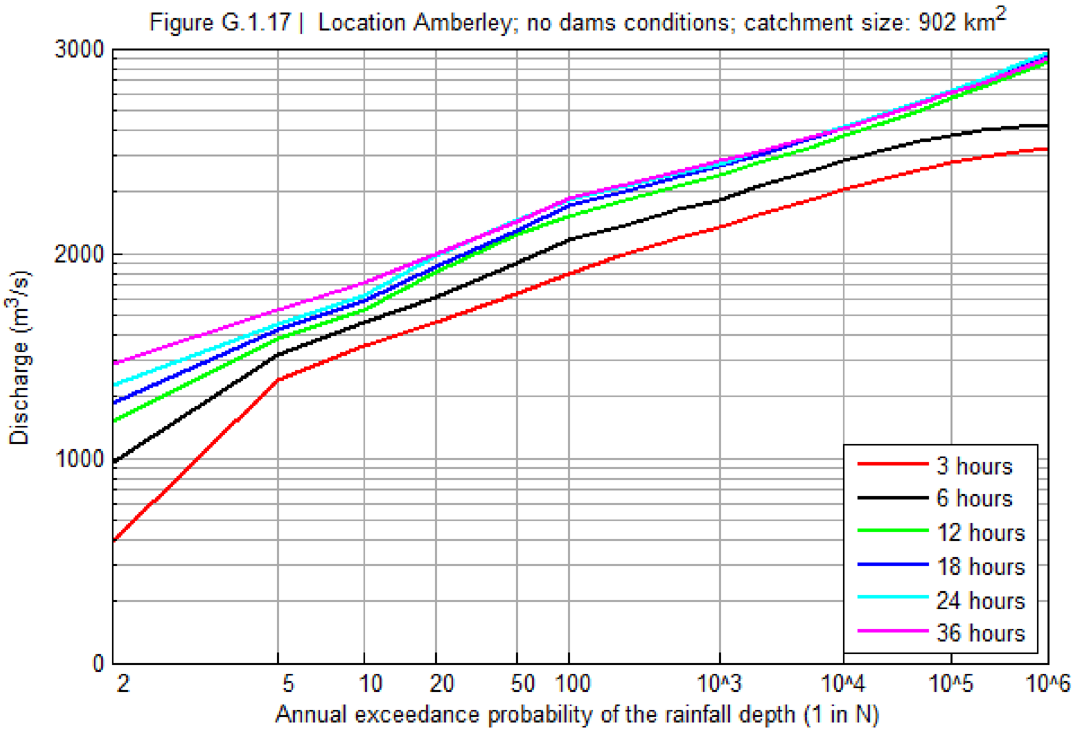


Figure G.1.17 | Location Amberley; no dams conditions; catchment size: 902 km2
Discharge (m3/s)
3000
2000
1000
0
2
5
10
20
50
100
10^3
10^4
10^5
10^6
Annual exceedance probability of the rainfall depth (1 in N)
3 hours
6 hours
12 hours
18 hours
24 hours
36 hours

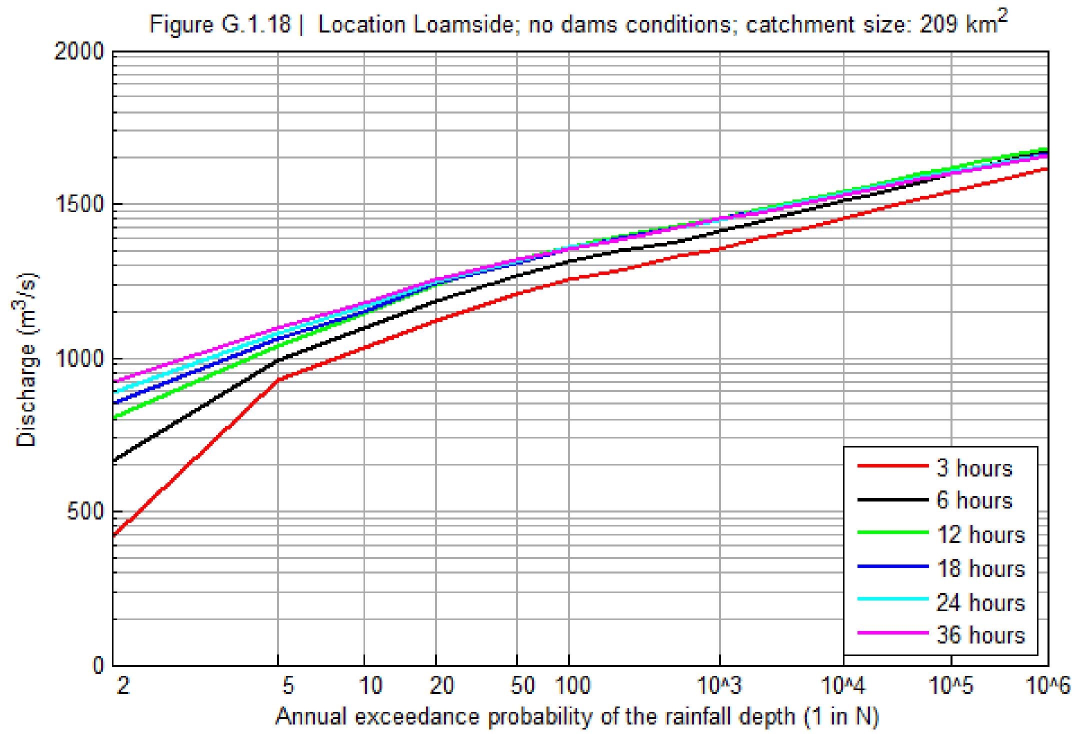


Figure G.1.18 | Location Loamside; no dams conditions; catchment size: 209 km2
Discharge (m3/s)
2000
1500
1000
500
0
2
5
10
20
50
100
10^3
10^4
10^5
10^6
Annual exceedance probability of the rainfall depth (1 in N)
3 hours
6 hours
12 hours
18 hours
24 hours
36 hours

Figure G.1.19 | Location Ipswich; no dams conditions; catchment size: 1,850 km^2

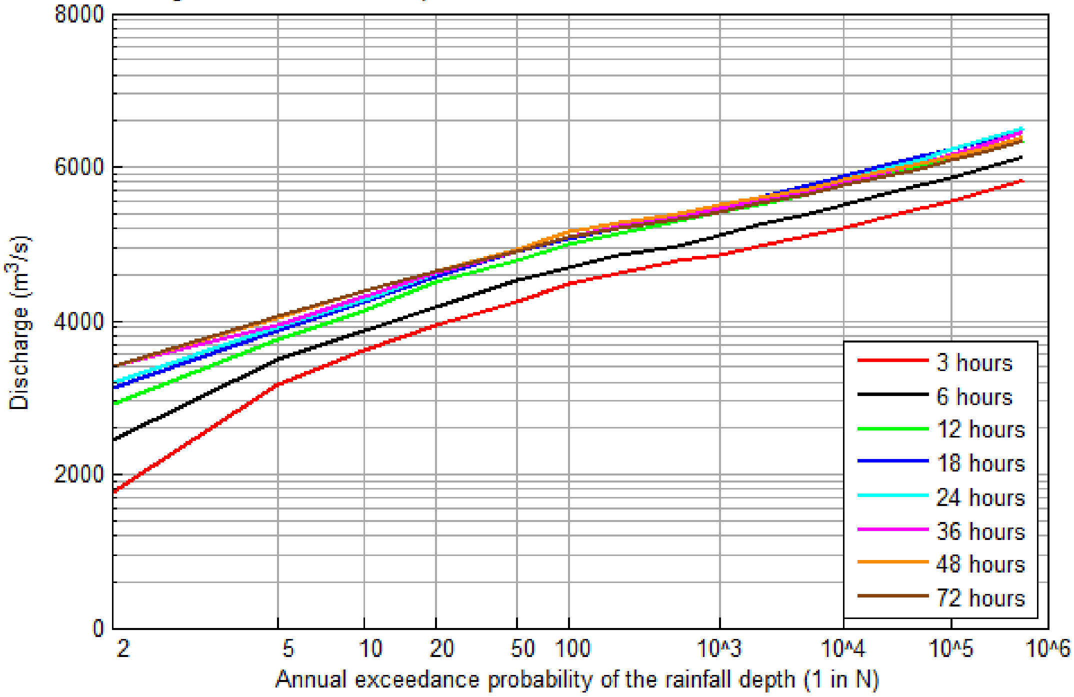


Figure G.1.20 | Location Moggill; no dams conditions; catchment size: 12,578 km^2

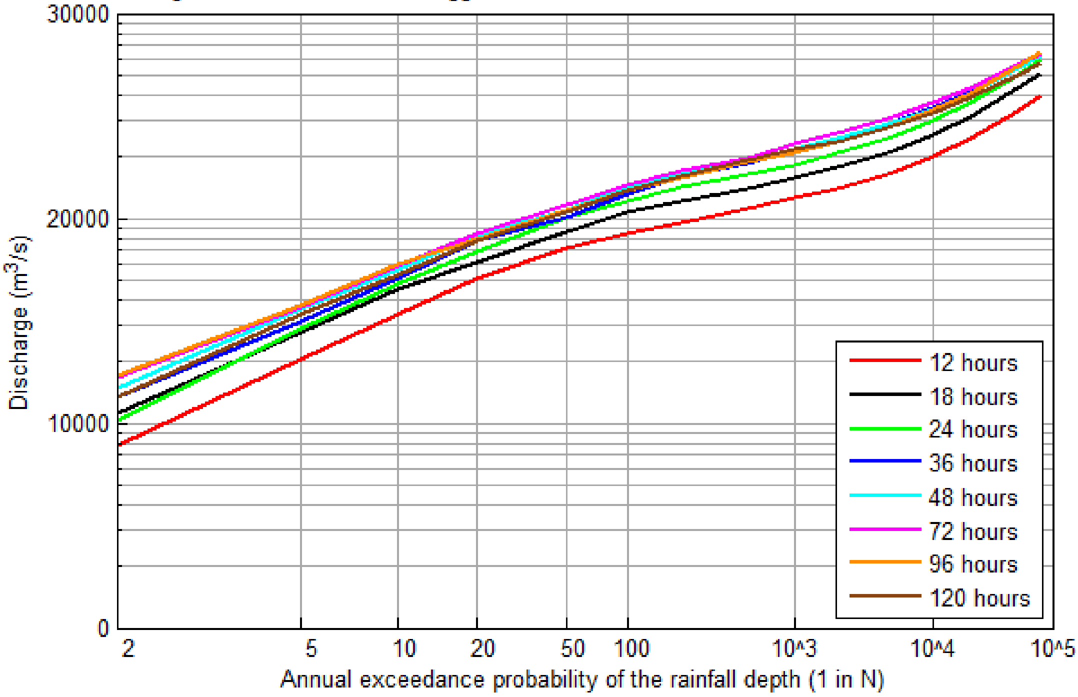


Figure G.1.21 | Location Centenary Bridge; no dams conditions; catchment size: 12,877 km^2

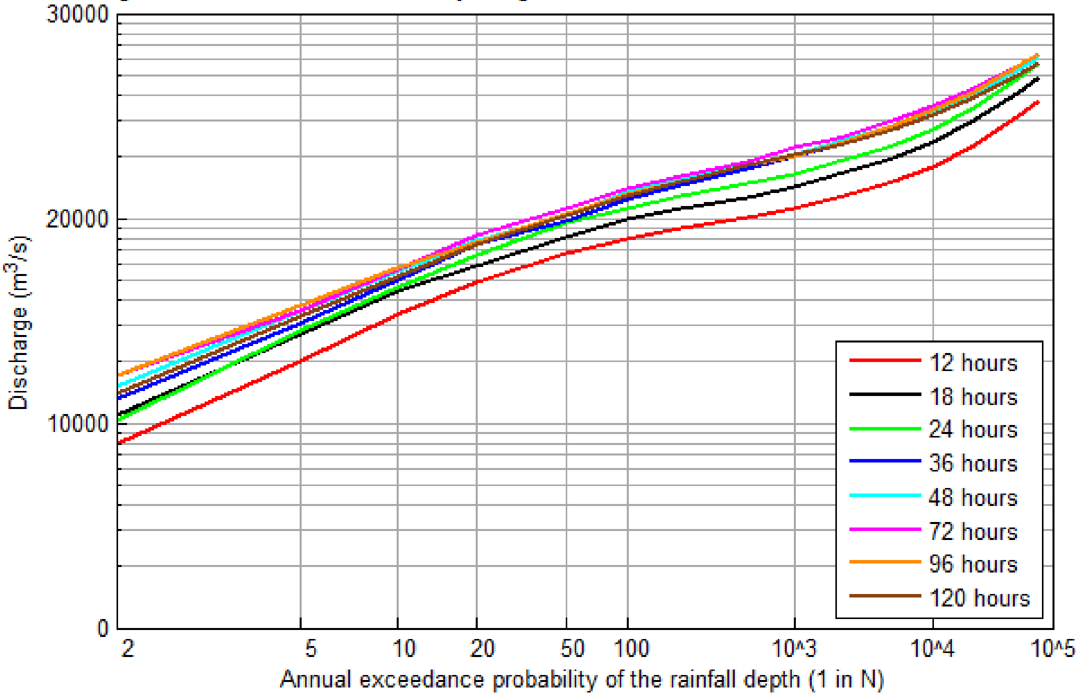
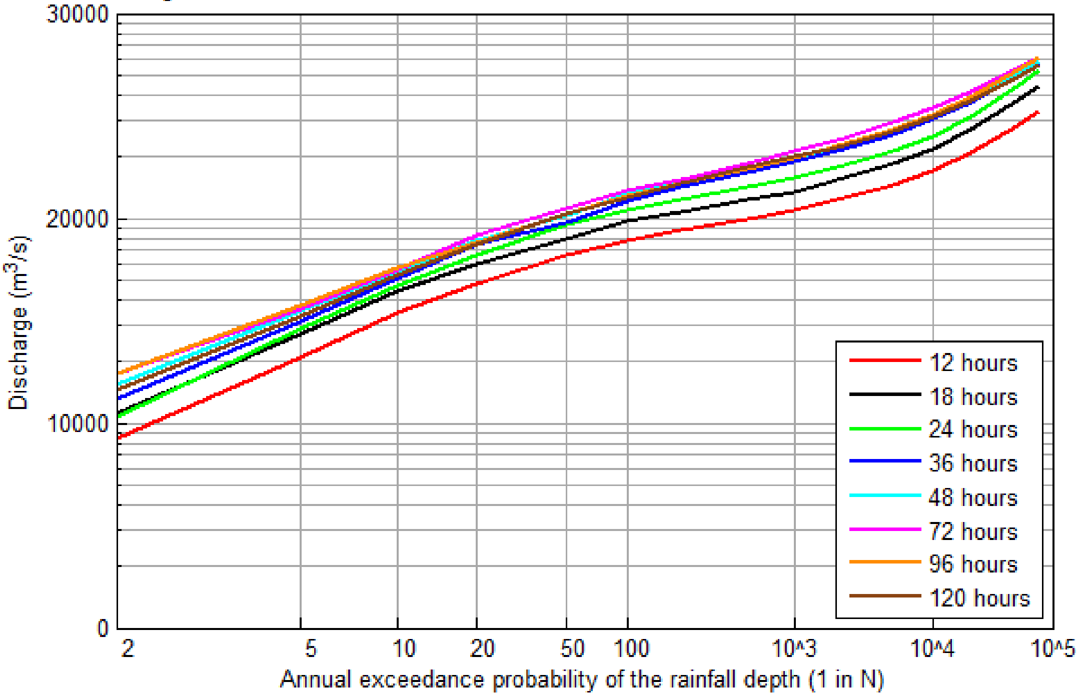


Figure G.1.22 | Location Brisbane; no dams conditions; catchment size: 13,198 km^2



G.2 With-dams conditions

Figure G.2.1 | Location Somerset Dam; with dams conditions; catchment size: 1,324 km^2

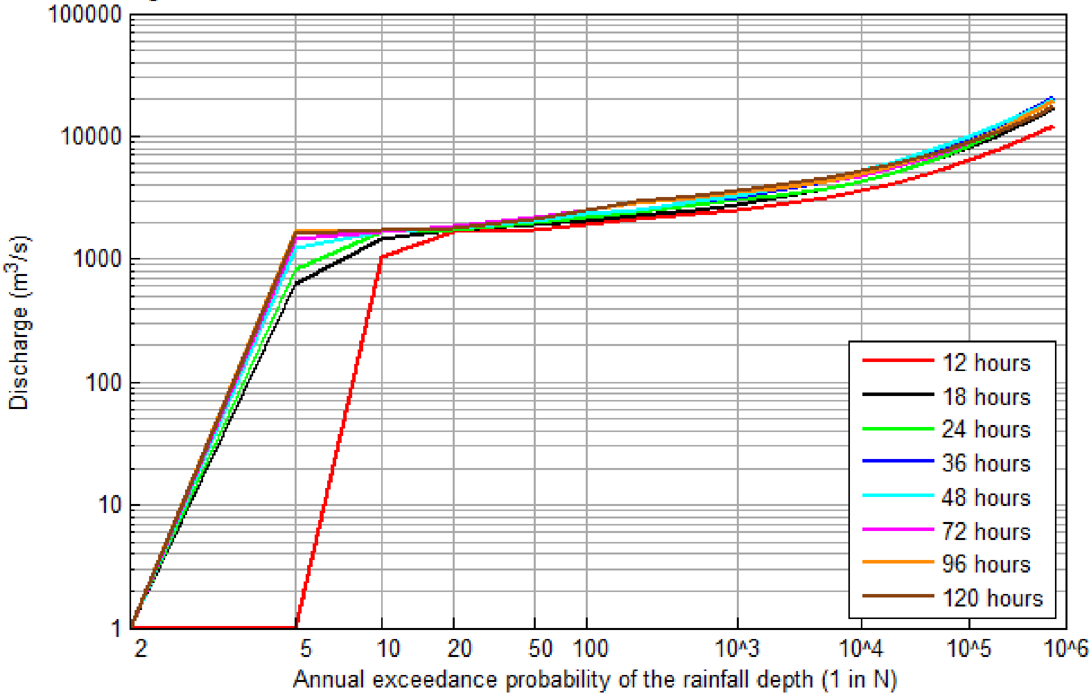


Figure G.2.2 | Location Wivenhoe; with dams conditions; catchment size: 6,980 km^2

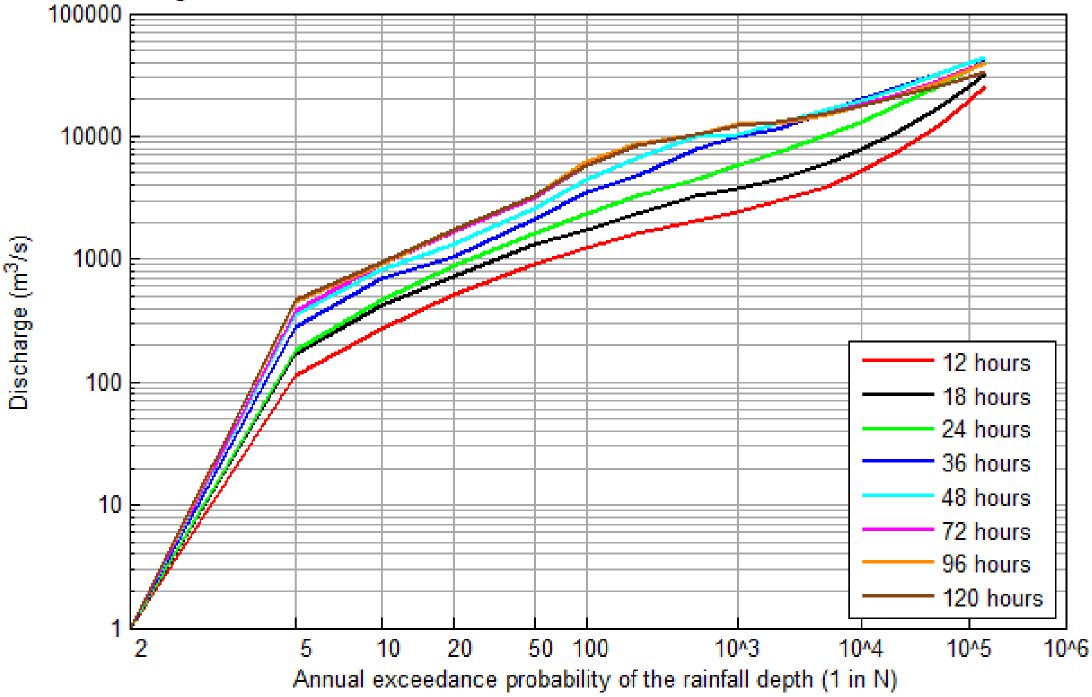


Figure G.2.3 | Location Savages Crossing; with dams conditions; catchment size: 10,126 km^2

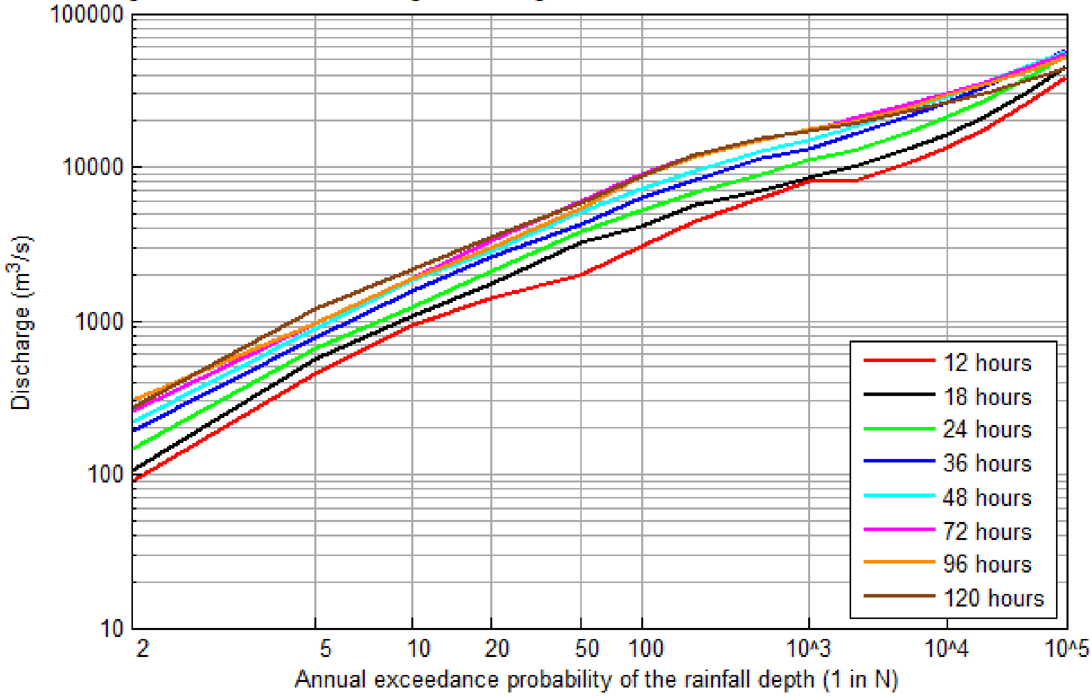


Figure G.2.4 | Location Mount Crosby; with dams conditions; catchment size: 10,507 km^2

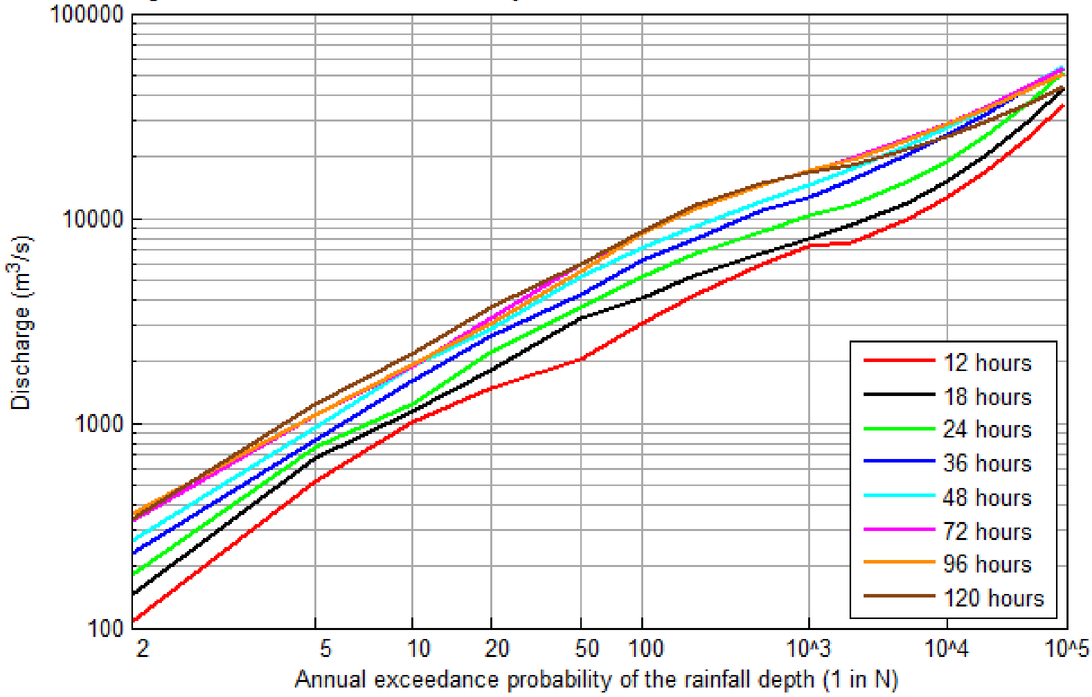


Figure G.2.5 | Location Ipswich; with dams conditions; catchment size: 1,850 km^2

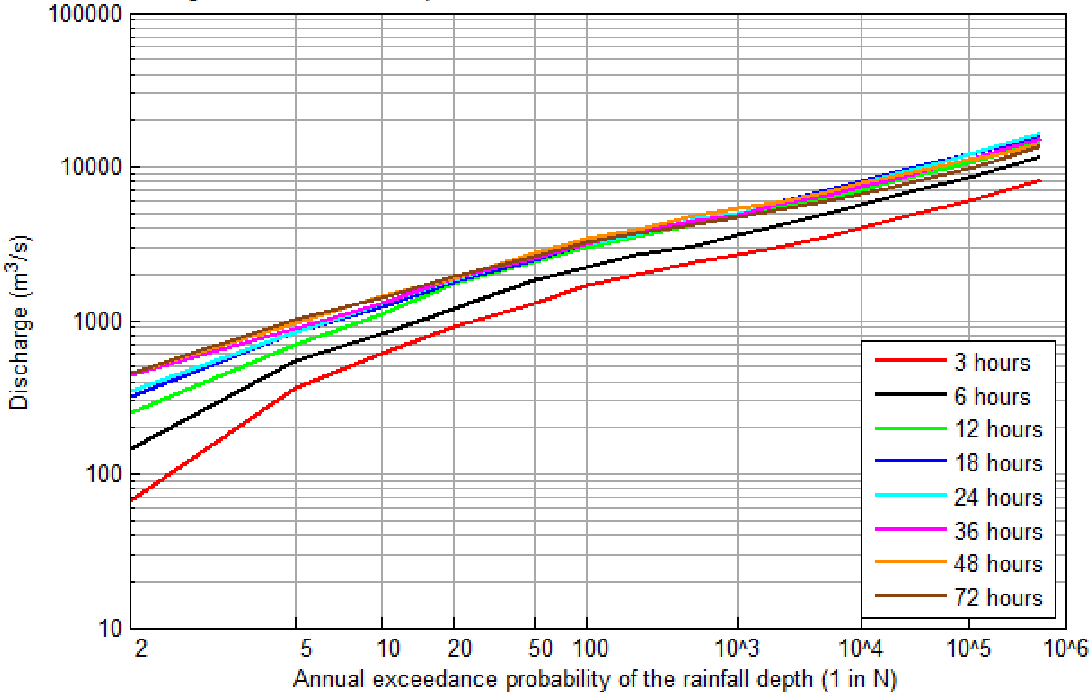


Figure G.2.6 | Location Moggill; with dams conditions; catchment size: 12,578 km^2

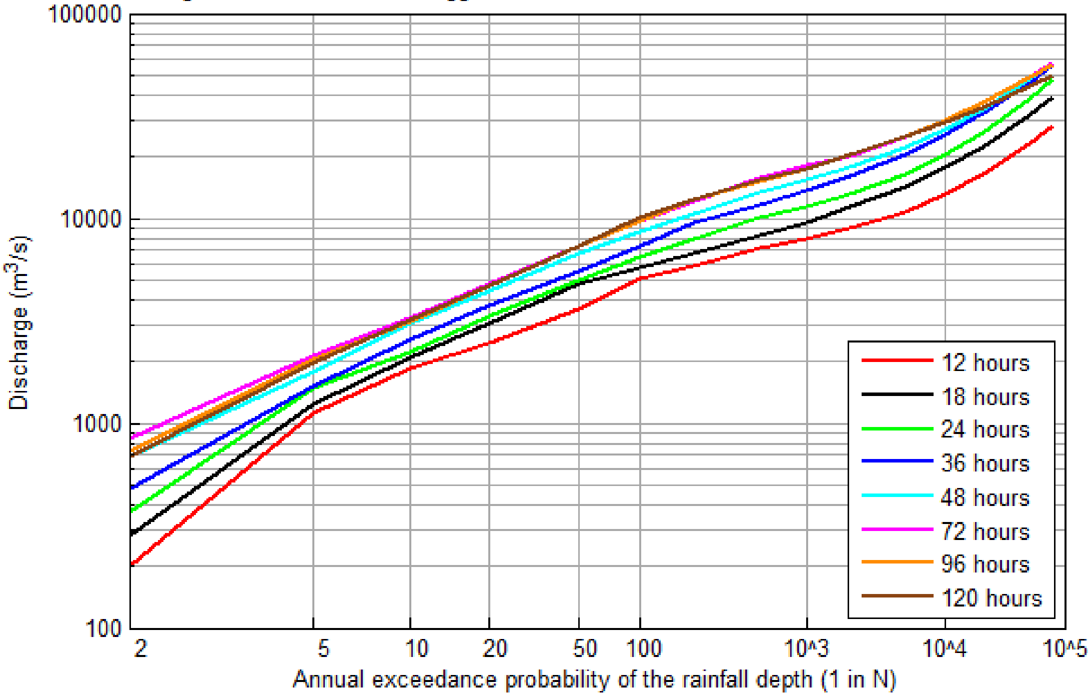


Figure G.2.7 | Location Centenary Bridge; with dams conditions; catchment size: 12,877 km^2

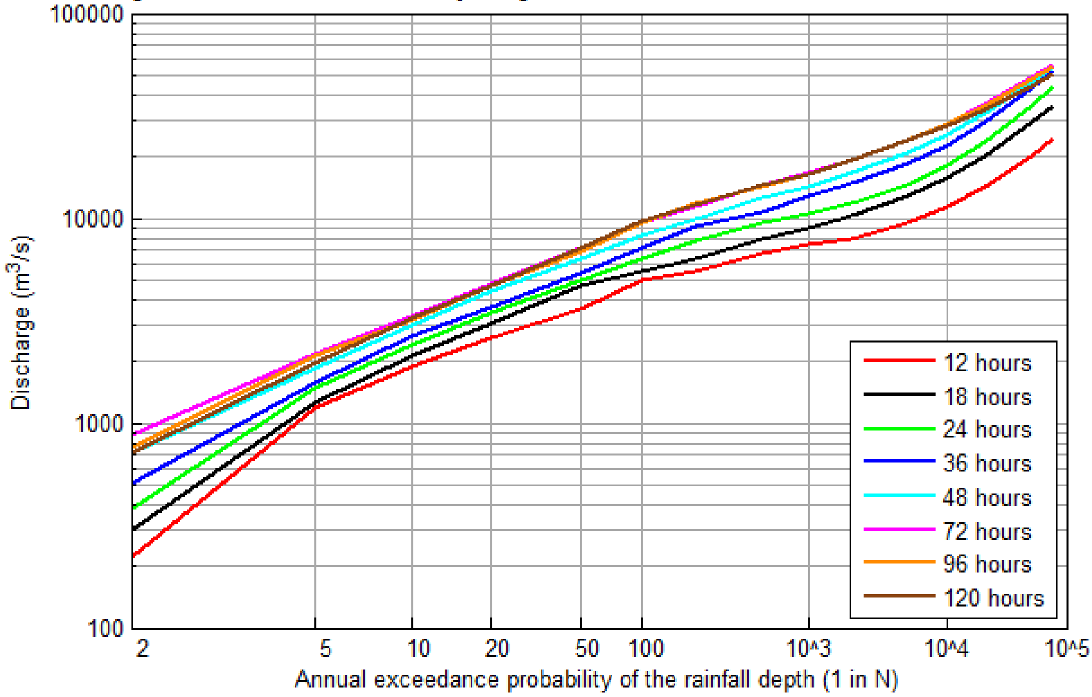
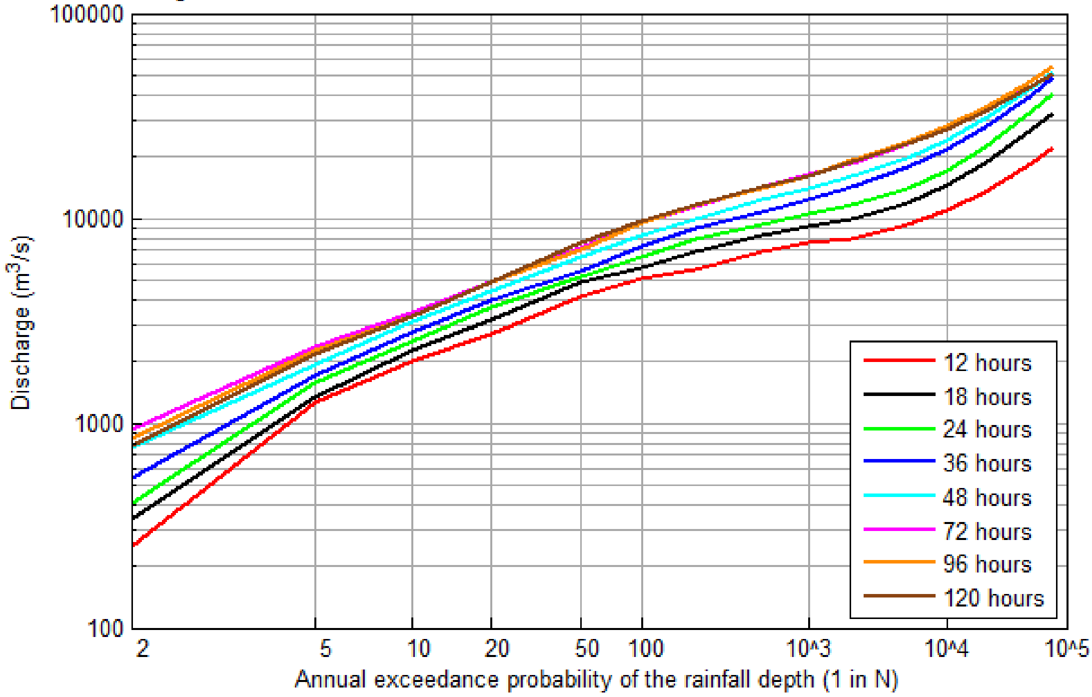


Figure G.2.8 | Location Brisbane; with dams conditions; catchment size: 13,198 km^2



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