QUEENSLAND FLOOD COMMISSION OF INQUIRY

BRISBANE RIVER 2011 FLOOD EVENT – FLOOD FREQUENCY ANALYSIS

FINAL REPORT

SEPTEMBER 2011

wma water

Level 2, 160 Clarence Street
Sydney, NSW, 2000

Tel: 9299 2855
Fax: 9262 6208
Email: wma@wmawater.com.au
Web: www.wmawater.com.au

BRISBANE RIVER 2011 FLOOD EVENT – FLOOD FREQUENCY ANALYSIS

FINAL REPORT

SEPTEMBER 2011

Project Brisbane River 2011 Flood Event – Flood Frequency Analysis	**Project Number** 111024
Client Queensland Floods Commission of Inquiry	**Client's Representative**
Authors Mark Babister Monique Retallick	**Prepared by** [redacted]
Date 18 September 2011	**Verified by** [redacted]

Revision	Description	Date
1	FINAL REPORT	18 SEP 11

BRISBANE RIVER 2011 FLOOD EVENT – FLOOD FREQUENCY ANALYSIS

TABLE OF CONTENTS

PAGE

LIST OF APPENDICES

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

1 This report was prepared at the request of the Queensland Floods Commission of Inquiry (The Commission). It investigates the 1% AEP (Q100) flood level on the Brisbane River from Moggill to the ocean and assesses the probability of the 2011 flood event. An addendum report will address similar issues on the Bremer River.

2 While every attempt has been made to conduct a thorough, rigorous and scientific flood frequency analysis, there have been a number of difficulties in developing a dataset suitable for this purpose. In particular, the analysis relies on adjustments to recorded levels at Brisbane River Port Office to account for changes to river morphology, and is dependent on the use of a rating curve to convert recorded levels into flows. WMAwater consider that the flood frequency assessment could be improved by undertaking further steps to improve the rating curve and the adjustments to the historical record. Measures by which such improvements can be made are identified in this report, but require a much longer timeframe than that available for this assessment. It is recommended that further investigations be undertaken including a thorough review of the rating curves, assessment of astronomical tide influence in the historical record, and development of a suitable two-dimensional model of the Lower Brisbane River to assess the effects of geomorphological changes.

3 Section 2 of this report outlines the scope of the investigation. Section 3 provides background on determining design flood levels, the use of the 1% AEP level for planning purposes, freeboard and outlines best practice in conducting a flood frequency analysis. Section 4 details the history of flooding on the Brisbane River. Section 5 provides a brief summary of previous estimates of the 1% AEP flood level and flow. The rating curve and associated data used in this investigation and how it was derived is detailed in Section 6, while Section 7 details the analysis. Section 8 presents the main conclusions and outlines a process for developing robust flood frequency estimates.

4 This report interchangeably uses the terms 1% AEP, 100 year ARI and Q100. In Queensland the term “Q100” is regularly used to denote the level or flow that has a 1% chance of occurring in any one year. “Q” is normally used in water engineering to denote flow, so application of the term “Q100” to indicate flood level can create confusion. The term is not widely used in practice outside Queensland. The distinction can be particularly important in coastal areas, as the 100 year ARI flood level in the lower reaches of rivers is caused by a combination of ocean levels and flow (and other contributing factors) and is not necessarily a result of 100 year ARI flow alone.

2. SCOPE

5 The Commission has requested that Mark Babister of WMAwater undertake the following:

1. Conduct a flood frequency analysis and determine the 1% AEP flood level for key locations on the Brisbane River below its junction with the Bremer River and on the Bremer River in the vicinity of Ipswich using information available prior to the January 2011 event. This work should be used to determine 1% AEP flood levels at up to 8 key locations in the Brisbane and Bremer Rivers and to produce 1% AEP flood profiles. This work should include a review of the SKM 1% AEP flood profile.

2. Repeat task 1 with the 2011 event included in the historical dataset.

3. Using results of task 1 and 2 determine the ARI and AEP of the January 2011 floods at particular points along the Brisbane River and Bremer River.

6 The following locations were identified as being of interest between Moggill and Brisbane:

- 13 Bridge St., Redbank (off-bank),
- Cnr. Ryan St. and Woogaroo St., Goodna,
- Corner Moggill Rd, Birkin Rd, Bellbowrie (Coles),
- Corner Thiesfield St, Sandringham Pl, Fig Tree Pocket,
- 312 Long St East, Graceville,
- Brisbane Markets, Rocklea,
- Softstone St, Tennyson (Tennyson Reach apartments),
- 15 Cansdale St, Yeronga,
- 42 Ferry Rd, West End (Aura apartments),
- 81 Baroona Rd, Paddington (Epic Cycles), and
- Brisbane City Gauge.

3. BACKGROUND ON DETERMINING DESIGN FLOOD LEVELS

3.1. Use of 1% AEP Flood Level for Planning Purposes

7 For planning purposes it is necessary to decide what level of flood risk is acceptable for individuals and the community. Ideally planning levels should be decided on the basis of risk, where both probability and consequences are considered, but in most locations in Australia the 1% AEP (100 year ARI) flood is designated as having an acceptable risk for residential planning purposes regardless of the consequences. This approach often leads us to a planning level, line or map which defines whether flood-related controls on prospective development are applicable or not.

8 The 1% AEP (100 year) flood level has not always been utilised for flood planning purposes. Prior to the use of the 1% AEP (100 year) as a design flood standard it was common for communities to simply use the largest historical flood on record for planning purposes. The 1% AEP (100 year) planning level was first adopted for residential housing in the ACT in the 1970s and has subsequently been adopted in most locations throughout Australia. The 100 year standard is also used extensively in the USA.

9 While the use of a standard event such as the 1% AEP (100 year ARI) flood event for planning purposes may provide a level of consistency, ideally flood planning levels would be determined on the basis of a flood risk assessment. While such flood risk assessments have been carried out in many locations throughout Australia there has been a reluctance to move away from the 1% AEP flood standard. Floodplain risk management studies often show that there are strong social and economic reasons for considering a higher standard in some locations, such as:

- Where rare flood levels are significantly higher and likely to cause significant devastation (an example would include locations where the 200 year event is over 2m above the 100 year ARI event); and
- Where inundation of the location will have significant economic and social consequences for a much wider region (an example would be the inundation of the CBD and regional service section of a major city or town, which may disrupt/prevent the provision of essential services for a much larger regional population).

10 For these reasons the city of London is moving to a planning level above the 0.2% AEP (500 year ARI) level for the Thames estuary. Many parts of the Netherlands use planning levels above the 0.1% AEP (1000 year ARI) level as in many places inundation would have catastrophic consequences (including loss of life) and take many months to pump out.

11 It is very rarely possible to eliminate flood risk as this would require placing development above the Probable Maximum Flood (PMF) level which cannot generally be justified on economic grounds and in other cases may simply not be physically possible.

12 The 1% AEP (100 year) flood is a theoretical flood with a specified probability of being exceeded. An actual flood event is whatever happens to occur, and may be larger than, or less than, the 1% AEP (100 year) event and may vary in probability along the reaches of a long river.

13 Any actual flood event will vary in some manner from the 100 year event. Such variations are primarily due to differences in rainfall, as the rainfall that occurs in an actual event is different in duration, intensity and spatial and temporal pattern to that which is used to derive the 100 year flood. Variation in other flood producing factors, for example how wet the catchment is before the event, or the location of the storm centre within the catchment can also have an impact on the size of flood, and also contribute to differences.

3.2. Freeboard

14 Freeboard is used to account for several factors including uncertainty in the flood estimate, differences in water level across the floodplain due to local factors, wind waves, waves caused by passing vehicles and the cumulative effect of future development. Freeboard is in effect a factor of safety that allows for uncertainty in underlying data and is commonly used in both Australian and international practice. The NSW Flood Development Manual describes the purpose of freeboard as being "to provide reasonable certainty that the reduced risk exposure provided by selection of a particular flood as the basis of a FPL is actually provided." (Reference 24, Appendix K7).

15 The additional buffer that freeboard includes an allowance for any minor increases in flood level due to the building of key infrastructure projects (such as roads and rail lines), which may have a cumulative impact on flood levels. In practical terms this ensures that minor changes in the 1% AEP flood do not result in houses falling into a high flood risk category.

16 Freeboard traditionally varies between 300mm (0.3m, or 1 foot) and 500mm (0.5m) but can be up to 1m in places. It is considered best practice in Australia to use 500 mm. In the coastal zone a separate allowance is often made for sea level rise resulting from climate change, as the impact of sea level rise decreases with distance from coast. Different freeboard amounts can often be applied to different types of development such as critical infrastructure or commercial/industrial development.

3.3. Flood Frequency Analysis Theory

17 The two basic methods for determining the probability of different flood levels are Flood Frequency Analysis (FFA) and the rainfall based Design Flood Method (DFM). FFA is the process of fitting a probability distribution to a series of flood peaks at a particular location. The DFM fits a probability distribution to observed rainfall and uses hydrologic and hydraulic modelling techniques to convert catchment rainfall of a certain probability to a flood level or flow, which is assumed to be of the same probability.

18 FFA provides a direct measure of flood probability and does not require assumptions about the different catchment wide processes and variables that contribute to peak flood levels or flow at a particular location. This allows for all the historically observed variability in rainfall intensity, storm volume, storm duration, storm type and antecedent conditions to be included. Unlike the DFM, FFA also provides a measure of the uncertainty of flood estimates. FFA is the method that most other flood estimation techniques used in Australian Practice are checked against.

19 While flood frequency has a number of advantages, as it can only be used when:

- A long flood record exists,
- The flood record is homogenous or can be adjusted to a near homogenous state,
- A reliable rating curve exists, and
- The probability of the event to be derived does not require extrapolation too far beyond the observed record length.

20 FFA should not be used to extrapolate far beyond the extent permitted given the period of record, as estimates are very dependent upon the assumed distribution. For example, a dataset with a 20 year length of record will not give a good estimate of the 1% AEP (100 year ARI) flow. When estimating rare events well beyond the period of record, rainfall based methods are recommend by Australian Rainfall and Runoff 1987 (Reference 7). In this situation rainfall based methods have advantages over FFA, as the probability of rare rainfalls can be estimated by regional techniques and extreme events can be approximated by methods that consider the limits of storm efficiency and the moisture holding capacity of the atmosphere.

21 The historical flood record can be analysed using either an annual series (where the largest event in a calendar or water year is extracted) or a Peak Over Threshold (POT) series (where the largest independent peaks are extracted). In South East Queensland a water year needs to start in winter as nearly all flood events occur in the wet season (over late spring and summer). For the analysis presented in this report, a water year is defined from July to June. While several floods occurred in 2010-2011 water year only the January flood (the largest) would be considered in an annual series. A POT series can be difficult to extract as there is no definitive way to determine if events are independent, which is a requirement of FFA. For this reason it is often not used. For example in February 1893, three floods occurred within 3 weeks, and while these events were probably caused by 3 separate meteorological events, the wet catchment and swollen rivers produced by the first flood influenced the magnitude of the subsequent flood peaks.

22 The current best practice advice on conducting FFA is contained in the ARR Draft chapter of the current revision of ARR (Reference 28). The major changes in the application of FFA (from those described in ARR 1987 (Reference 7) in Australia are:

- The removal of the recommendation to use the Log Pearson 3 (LP3) distribution, and
- The replacement of log space moment based fitting techniques.

3.3.1. Probability Distribution

23 Many probability distributions have been applied to FFA and this has been a very active field of research. However, it is not possible to determine the "correct" form of the distribution and no rigorous proof exists that any particular distribution is more appropriate than another. ARR (Reference 28) provides further discussion on this issue. Two broad approaches are possible. The first is to use a range of distributions and adopt the one which provides the "best fit". The other is to use a single distribution for a region. While no distribution is recommended the Generalised Extreme Value (GEV) and Log Pearson 3 (LP3) are suggested as a starting point (Reference 28) as they have been shown to fit Australian data well.

3.3.2. Fitting Method

24 Recent research has suggested that the fitting method is as important as the adopted distribution. The traditional fitting method has generally been based on moments and this makes the fit very sensitive to the highest and lowest observed flow values. Recent research has shown that L-moment and Bayesian likelihood approaches are much more robust than traditional moment fitting and hence these are the current recommended methods.

25 For this analysis a Bayesian maximum likelihood approach has been adopted in preference to L-moments because the method allows the inclusion of large historical flood information outside the period of continuous record. While not necessary at the Port Office it would be required at other locations.

26 This study used the Flike flood frequency analysis software developed by Kuczera (Reference 29).

3.3.3. Historical Flood Information

27 In many locations in Australia data detailing the early flood record (from early settlement and hence prior to the establishment of continuous gauging stations) is incomplete and only the large events tend to be well documented. Where major floods are known to occur it is possible to include this information in modern flood frequency analysis via Bayesian methods. This is particularly important where these early floods are known to be larger than those contained in the continuous record even if little is known about the exact height or flow.

28 It is very important to include historical information when the continuous historic record does not contain many of the top ranking historic events. To not do so will probably result in underestimation of the probability of flooding.

3.3.4. Rating Curve

29 Flood frequency analysis is typically undertaken on flows, not flood levels, as flows lend themselves to the FFA methods and assumed distributions. As flow is not generally measured directly, a rating curve is required to convert observed peak flood levels into flow. The rating curve (height-discharge relationship) adopted for the estimation of stream flows from the recorded gauge heights is critical to the success of flood frequency analysis. A poor quality rating curve results in a poor estimate of flow. Where there has been a significant change in river cross section or where flow is affected by tidal effects or tributary inflows a family of rating curves is often produced.

3.3.5. Long Term Climate Variability

30 The flood record on the east coast of Australia exhibits periods of a decade or longer timescale that are flood or drought dominated. This was first recognised by Erskine and Warner in 1988 (Reference 30).

31 Short term climate variability on the east coast of Australia is characterised by the interannual El Nino/Southern Oscillation (ENSO). There is a marked increase in flood risk in eastern Australia during the La Nina phase. The El Nino phase typically contains few major floods (Reference 31).

32 There is also considerable evidence that longer term processes have a major impact on flood risk. The Inter-decadal Pacific Oscillation (IPO) is a pattern of Pacific Ocean temperature variation that shifts phase at a timescale typically lasting 15-30 years. On the east coast of Australia nearly all large events occur during an IPO negative period.

33 Figure 1 shows the IPO index from 1880 to 2000. Note that the two large flood events in that period in Brisbane occur close to IPO low points.

34 It is important that flood frequency analysis is carried out over a long period so that the results are not biased to either of these climate cycles.

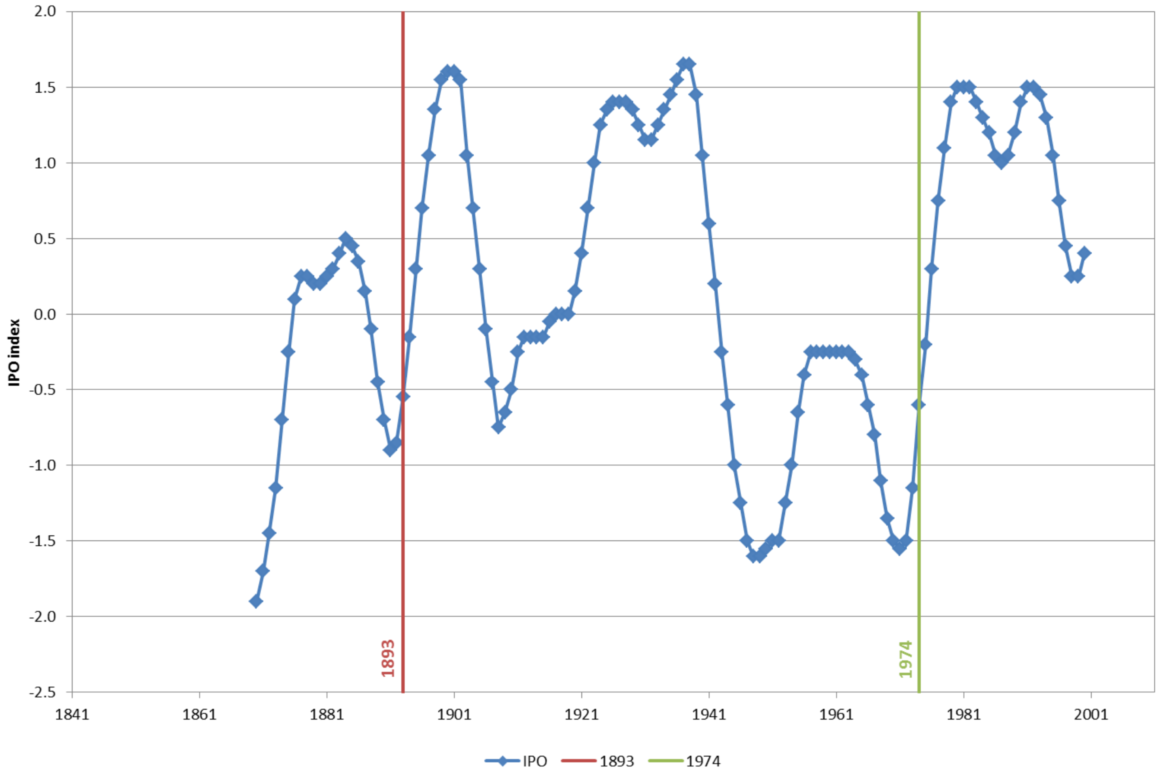


Figure 1: IPO Index 1880-2000

3.3.6. The Need for a Homogeneous Data Set

35 When conducting a flood frequency analysis a homogeneous data set, which is consistent and based on the same catchment conditions is required. The homogeneous data set needs to be free of the impacts of dams, levees, and significant changes in urbanisation and the river morphology. Creating a homogeneous data set is often not a simple process.

4. BRISBANE RIVER FLOOD HISTORY

4.1. Flood History

36 Little information is often available for floods that occurred early in the settlement of a region. However, historical flood information can be found from official correspondence, newspapers and even old parliament session documents. When examining historical flood records there is a desire to find accurately measured records. FFA techniques used in the past meant that where only vague information (such as "it was larger than the 1893 flood", or "it was the largest observed flood") was available it could not be incorporated into the analysis. However, the use of modern Bayesian techniques has meant this prior information can be incorporated. It is important if implementing this approach to determine whether the floods were actually significant. For example when people first settle a region they are still trying to establish what is the "norm" with regard to flooding, so there is often several floods called "significant" which are not that significant.

37 Historically studies of the Brisbane River have been reluctant to place too much weight on earlier events (Reference 16, Section 5). Investigation of historical records suggests there is plenty of evidence to prove the early events are credible and significant. The 1841 event was recorded in a number of locations and was discussed in the QLD Parliament after the 1893 event. Credible evidence also exists for the magnitude of the 1844 event. There are also references to earlier floods (1824, 1825, 1836, 1839 (Reference 32)) however, besides the 1825 event, they appear not to be significant, and for all these early events the evidence is not detailed enough for their inclusion.

38 Looking at historical events and comparing or ranking them with more modern events is complicated by the changes in the River that have occurred over the years since European settlement. These changes are discussed in Section 4.2 and include dredging, river widening and also the construction of Somerset and Wivenhoe Dams.

39 While the 1974 flood was very significant, when looking at the full flood record (1841-2011) with appropriate adjustments made for the impact of dams and dredging of the river, under current conditions the 1841, 1893 (2 events) and 2011 are larger than the 1974 under pre and post dam conditions.

4.1.1. 14th January 1841

40 The 1841 flood is the highest recorded in Brisbane's history at 8.41m AHD at the Port Office gauge. According to Reference 32:

> *"In 1896, JB Henderson, the Government Hydraulics Engineer in an address to Parliament reported that he found by examination of earlier plans that the 1841 flood was [7 centimetres] higher than the flood of 5th February 1893."*

41 While the 1841 flood produced the highest recorded flood level at the Port Office when proper adjustments are made for changes in the river it is no longer the largest known flood on the Brisbane River. In 1841 a sandbar was present at the mouth of the Brisbane river which exacerbated the flood levels upstream and no works had been carried out to dredge and straighten the river. Accounting for these factors would reduce the flood level by an estimated 1.92m to 6.48 mAHD at the Port Office gauge.

4.1.2. 10th January 1844

42 The 1844 event peaked at approximately 4 feet below the level of the 1841 event at 7.03 mAHD. The adjusted 1844 level is 5.11 mAHD.

4.1.3. February 1893

43 1893 was a wet year on the Brisbane River with several flood peaks occurring including 3 floods within February (peaking on the 5th, 12th and 19th). The first and largest event resulted in a peak of 8.35 mAHD recorded in Brisbane. The second event peaked at 2.15m AHD, with the third event peaking close to the level of the first at 8.09m AHD. Houses were washed away at Ipswich and Brisbane. During the first event the “Elamang” and the gunboat “Paluma” were carried into the Botanical Gardens area, and the "Natone" onto the Eagle Farm flats. In the third event these boats were refloated.

44 It is noteworthy that the 1893 flood peak of 8.35 mAHD occurred following the removal of the downstream bar in 1864 and in a period when the Brisbane River was being dredged on an ongoing basis to improve River navigability.

4.1.4. 25- 29th January 1974

45 The 1974 event was the highest flood recorded on the Brisbane River during the 1900’s with the river peaking at 5.45 mAHD. During the event 8,000 households were affected. The flood peak at the Port Office would have been marginally higher had Somerset Dam not been constructed in the 1940’s (completed in the 1950’s). The substantial river works carried out since 1893 are estimated to have lowered this flood level by approximately 1.5m (Reference 18).

4.1.5. January 2011

46 The January 2011 event was the largest experienced on the Brisbane River since the construction of Wivenhoe dam. The river peaked at a height of 4.27 mAHD at the Port Office gauge and 4.46 mAHD at the City gauge. Despite the gauges being located directly opposite each other on either bank of the river a discrepancy between the two was recorded. This issue was raised in Reference 39.

47 The Port Office gauge is located at a dock on the left bank of the River at the corner of Edward and Alice streets. The City gauge lies at approximately the same river chainage

but on the right bank. The Port Office gauge is operated by Marine Safety Queensland (MSQ) which is a State agency lying under the Department of Transport and Main Roads. Conversations with MSQ staff in the Tides Office indicate that the discrepancy is known and attributed to mechanical failure of the City gauge. The City gauge is operated by Seqwater and is the gauge which the BOM Brisbane Flood Alert System currently refers to.

48 Whether or not these discrepancies are attributed to the failure of the City Gauge or transverse slope on the river due to the river meander upstream of the gauges is not known. Of interest would be whether Seqwater believe they experienced any mechanical failure with respect to gauged water level at the City gauge during the event.

4.1.6. Flood Events on the Brisbane River

49 The largest events on the Brisbane River are summarised in Table 1.

Table 1: Brisbane River Flood History

Event	Recorded Flood Level (mAHD)
1841	8.43
1844	7.02
1890	5.33
1893 (5th February, a)	8.35
1893 (19th February, b)	8.09
1898	5.02
1974	5.45
2011	4.27 (4.46)

Note: Levels as recorded at Port Office (unadjusted for comparability)

50 Historical peak flood heights are available at Brisbane for the Port Office/City gauge, Lowood, Moggill and Savages Crossing. Table 2 summarises the period for which data is available. Several other gauges with shorter record lengths have not been included.

Table 2: Period of Record

Location	Start	End	Period of Record (Years)
Moggill	1966	2011	46
Lowood	1910	2011	102
Port Office/City Gauge	1841	2011	171

4.2. History of River Changes

51 Changes to the Brisbane River have been driven by three distinct priorities:

1. *Navigation* – The Brisbane River was an important transport link to agriculturally valuable lands in the Darling Downs. Dredging and works carried out to aid

navigation started in the 1860's and continued through until the 1940's (Reference 3);

2. *Flood Mitigation* – from the 1930's river widening works were carried out and training walls were installed. These works were aimed at mitigating floods and reducing problems with sedimentation of the river, as had occurred following the 1893 event (Reference 3); and
3. *Development* – In recent times there has been a large amount of infrastructure built on the banks of the river or within the river. These structures can restrict flow during large flood events and can have localised impacts on flood level. This study has not attempted to quantify the effects of development on flood behaviour. These changes are assumed to be small in comparison with the other changes.

52 With the construction of Wivenhoe dam cutting off sediment supply from a large part of the catchment, it is unclear if the river has reached a new equilibrium or if accretion is continuing to occur. This needs to be determined using up to date bathymetric survey and an appropriate assessment.

4.2.1. Details of River Works

53 The Brisbane River has a long history of dredging beginning in 1864. The following text and table have been largely adapted from References 5 and 19. Dredging locations and dates are summarised in Table 3.

54 Originally, a shallow bar covered the entrance to the river to a depth of 5 ft (approx. 1.5m) at low tide and 12 ft at high tide. Upstream of the bar, the river deepened to 24 ft (approx. 7.3m) until a rocky area at Lytton. In 1864 a channel was cut across the bar to allow larger ships to access Brisbane port. From 1866 until 1891 numerous smaller dredging projects were undertaken, including a channel through the Fisherman's Islands to Pelican Bay, a channel through Redbank Flats and Cockatoo Shoal in the upper reaches of the river and the deepening of the river near the Eagle Farm and Pinkenba Flats.

55 The flood of 1893 undid a lot of the dredging work, silting up the river and reducing its depth to only 6ft (approximately 1.8m). The bar was also reduced to 8ft 6inches (approximately 2.5m) in depth. The restoration work to restore the river to pre-flood conditions was completed in 1895. In the early 1900s, the curves of several river bends were adjusted to straighten the river and a series of training walls were built to improve scouring action. Deep dredging was undertaken from Brisbane City to the river mouth. The deposit from these works was used to create Bishop Island, which also has an influence on the river flow behaviour. These works were finished in 1912.

56 Dredging of the river reached its peak in 1940 (Reference 3). The abandonment of the city for port purposes has lead to the discontinuation of dredging in this region.

57 The 1999, Brisbane River Flood Study (Reference 17) accounted for the effects of dredging by adjusting flood heights for the initial bar dredging in 1864 (reduced flood levels

by 0.4m) and the major dredging works completed in 1912 (reduced flood levels by 1.52m). The adjustment of 1.52m came from an estimate by Henderson (as quoted in Reference 17) that dredging works would achieve a reduction in flow levels of 5 feet (approx. 1.5m). Analysis of Moggil and Port Office gauges in the December 1999 report (Reference 18) suggests that whilst dredging would impact flood levels in line with Henderson's earlier prediction the works would not significantly impact larger floods.

Table 3: Key Dredging Dates, Depths and Locations on the Brisbane River (Adapted from McLeod, 1977 and Thompson, 2002 (References 5 and 19)

Key Dredging Dates				
Date	Depth			Comments
	Bar	Draught	River	
1824	5-12'		24'	24ft until rocky area at Lytton was reached
1864		11'9"		Francis' channel through bar completed
1866			12'	Fitzroy dredge- created a channel through Redbank flats
1867		17'		Brisbane river mouth dredge- Fisherman Islands and across Pelican Bank
1871		17'	10'6"	Eagle Farm flats dredged, river depth is to town
1874				Cockatoo Shoal dredge
1877	17'		15'	Pinkenba Flats dredge
1879				Heath's Channel Dredged
1891b	20'			Maintenance dredge
1893	8'6"		6'	The 1893 flood silts up the river
1895	15'		15'-16'	restoration work from flood damages completed
1898			20'	Lytton Rocks removed
1900's				Tips of bends straightened: Kangaroo point, Garden point, Bulimba point, Kinellan Point
1900's				A series of Training walls was built to improve scouring action including the 8,600 Hamilton Wall
1908-12	24'		24'	A new straight bar cutting was made, spoil created bishop island. Major dredging undertaken
1965				Removal of Seventeen Mile rocks
N/A				Outer River Bar
				Inner River Bar

4.3. Impacts of Dams on Flood Levels

58 Wivenhoe and Somerset Dams and their operation have a significant effect on downstream flood levels. Table 4 summarises their characteristics.

Table 4: Dam Characteristics (Source: Reference 27)

	Wivenhoe Dam	Somerset Dam
Completed	1985	1959
Water Supply Storage (GL)	1150	370
Temporary Flood Storage	1450	524
Location	Brisbane River Upstream of Lockyer and Bremer	Stanley River Upstream of Brisbane River
Catchment (km^2)	7000 including Somerset Dam	1330
Reservoir Surface Area (km^2)	107.5	42.1

59 Both dams have a dedicated flood storage component which is used to mitigate floods. The mitigation benefits of the dam are larger effect on small events. The mitigation benefits of the dam are greater if the dam is below full supply level.

60 The 1999 flood was essentially captured by Wivenhoe dam. Modelling by WMAwater of the 2011 event showed that drawdowns between 0 and 17% had no impact on the peak flow and downstream flood levels. While a 25% drawdown resulted in a decrease in flood levels of 300 mm at the Port Office (Reference 38).

61 A 96 year simulation of Wivenhoe dam carried out by Reference 18 based on daily rainfall showed the dam was above 90% storage capacity 80% of the time and above 75% storage capacity 95% of the time. Reference 18 investigated the impact of different dam levels on the Q100 discharge hydrographs at Port Office using a MIKE 11 model. It was estimated that at full supply level the peak of the flood was approximately 8500m^3/s and at 50% supply level the peak will be reduced by 1800 m^3/s.

62 Because flood events tend to occur in wet periods it can be misleading to use the probability of different storage levels as the likely level before a flood. There is a much higher probability that the dam will be near full supply level at the beginning of the event. This is also confirmed by examining at the historic record. Reference 18 found that prior to 7 out of 9 historic events the dam level would be at or above full supply level. Prior to the 1974 event there would have been enough rainfall to fill the dam to spillway level (Reference 18). The 2011 event was preceded by 2 events (October and December) and was full before the event. While we are unsure whether the first 1893 event was preceded by a large amount of rainfall the subsequent large event (2 weeks later) was.

63 Figure 2 compares peak inflow and outflow for Wivenhoe dam from a number of sources for occasions when the dam is full. Also plotted on the graph is the 1:1 line (or 50% reduction in flow by the dam as recommended by the 2003 Review Panel (Reference 20)). While there is considerable scatter amongst the data points the graph shows below an inflow of 6000 - 8000 m^3/s the attenuation is quite high while around 12000 m^3/s the attenuation is quite low. It is not unexpected that there would be some scatter as two floods could have a similar peak inflow and very different volumes and hydrograph shapes. There is however a reasonable correlation of volume and peakflow. For peak outflows up to 4000 m^3/s (max discharge allowed at Moggill under W3) the discharge is very dependent on the flow occurring in the Lockyer and Bremer Systems.

64 The 2011 event has a peak inflow of 11000 m^3/s (WMA) and 11500 m^3/s (Seqwater) and a peak discharge of 7500 m^3/s (giving a reduction of 32-35%). If you average the peak inflow and outflow to remove some of the oscillations the numbers become much closer.

65 Figure 3 to 5 depict pre and post dam flow at Port Office, Moggill and Savages Crossing. The 50% reduction line, as adopted by the 2003 Review Panel is also shown.

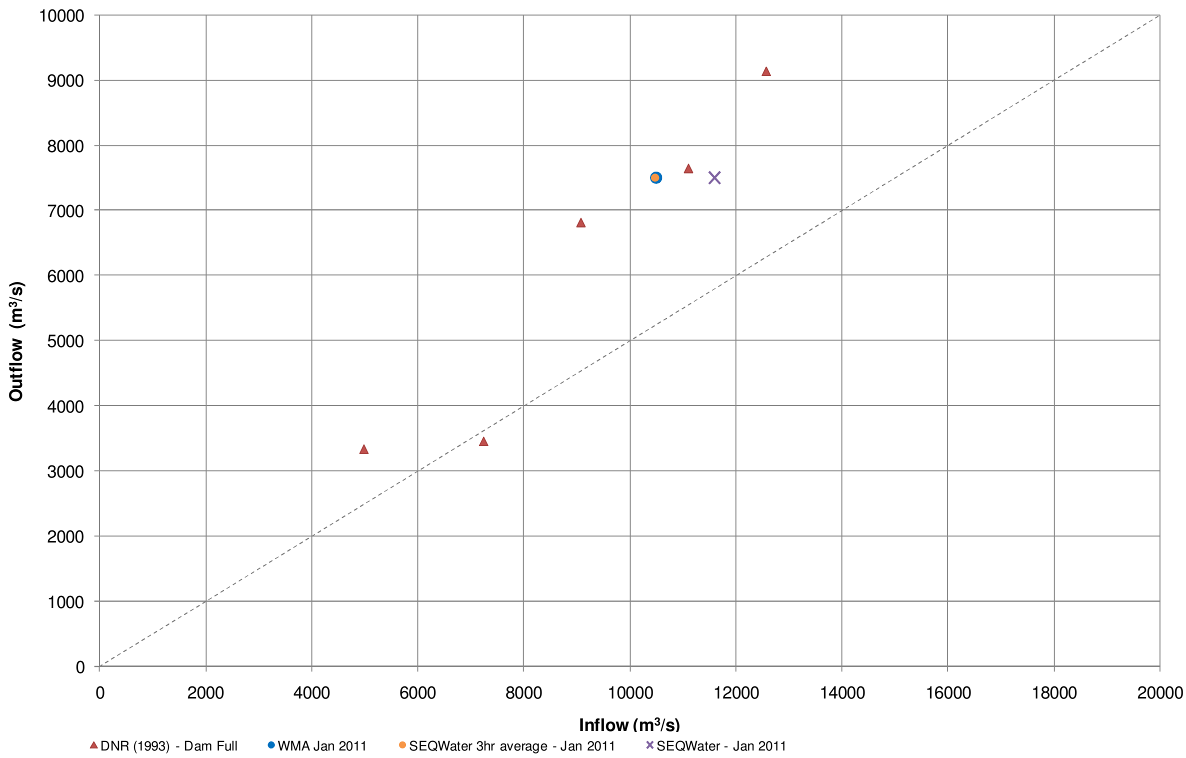


Figure 2: Inflow vs Outflow Wivenhoe Dam

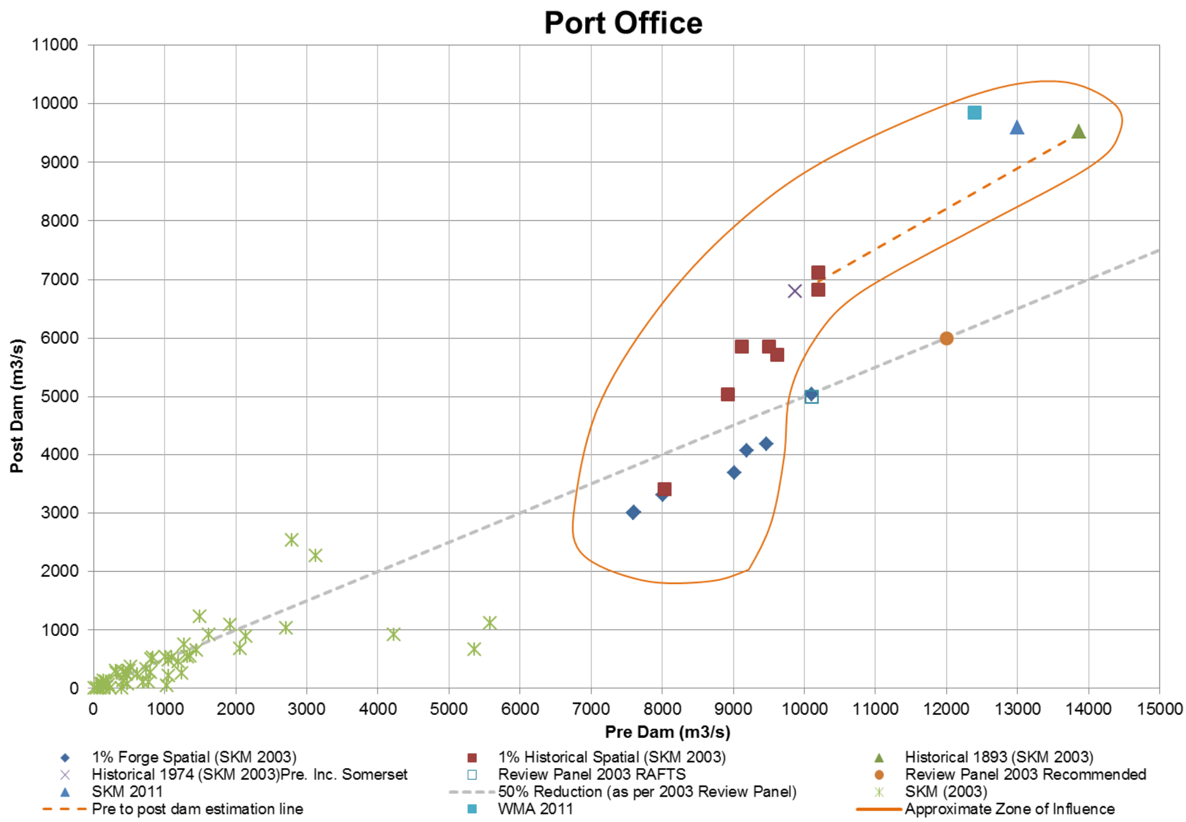


Figure 3: Port Office- Pre and Post dam flow

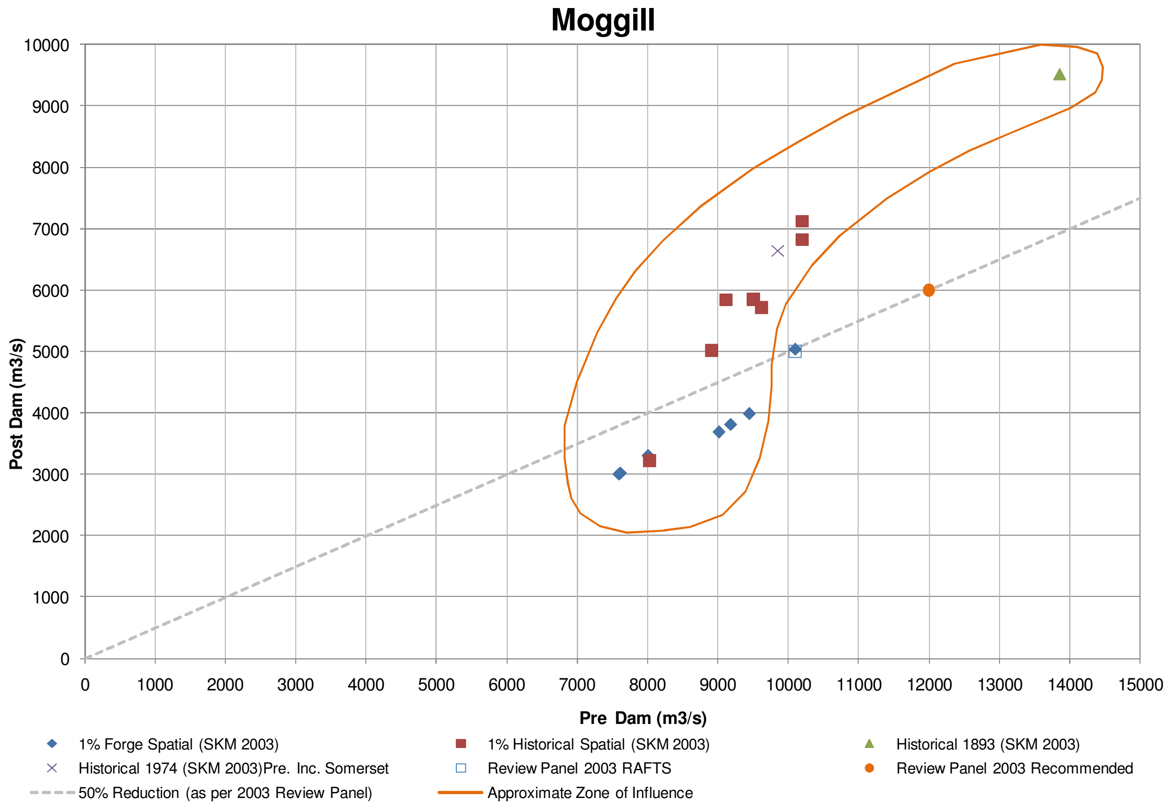


Figure 4: Moggill – Pre and Post Dam flow

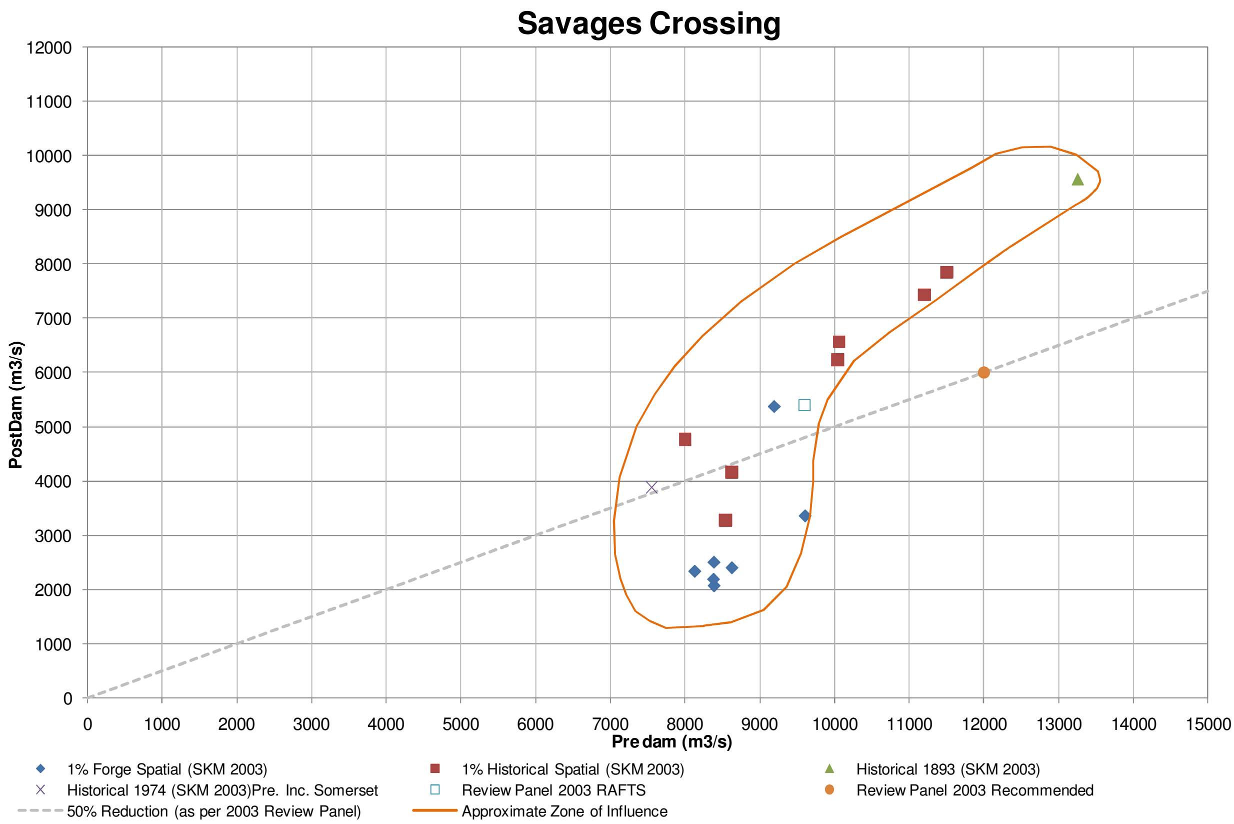


Figure 5: Savages Crossing – Pre and Post Dam Flow

4.4. Other Effects on Historical Flood Records

66 Other factors which influence historical flood record (either effecting the volume of runoff or effecting flow behaviour) and result in a non-homogeneous record include:

- Urbanisation,
- Extraction for water supply,
- Changes in catchment vegetation, and
- Obstructions in the river eg. bridges, walkways which restrict flow and reduce flood storage.

67 Urbanisation increases the amount of impervious surfaces, resulting in an increase in the runoff volume. The effects of urbanisation are most pronounced on small floods and on small catchments. Significant urbanisation has occurred in the Lower Brisbane River Catchment since the 1970's, but the total amount of urbanisation in the catchment is relatively small. Urbanisation can be neglected when assessing flood risk on large rivers like the Brisbane river, however it is important when looking at flooding on small creeks and tributaries.

68 Extraction of water for water supply (eg Wivenhoe dam) lowers dam levels and potentially increases the amount of mitigation that can be achieved by the dam. This is discussed in Section 4.3.

69 Deforestation or the removal of vegetation can also increase the runoff during an event. This effect is much more pronounced in small floods.

70 Obstructions that affect the flow behaviour or restrict storage will result in localised increases in flood levels. In the Brisbane CBD for example there are a lot of obstructions to the flow in the form of bridges and walkways. Most bridges only have a relatively minor and localised impact when the water level is below the underside of the deck.

5. PREVIOUS STUDIES OF Q100 AT PORT OFFICE

71 A significant number of flood studies and subsequent reviews have been undertaken on the Brisbane River. Since the 1970's there have been several revisions to the Q100 flow and flood level although WMAWater understand that the Flood Planning Level (FPL), established in 1984, has remained constant. Lower Brisbane River studies have generally referred to the Brisbane City and/or Port Office gauge. Although these gauges are at the similar chainage along the Brisbane River they are on opposite banks. Further discussion on the discrepancies between the two is discussed in Section 4.1.5 and 6.

72 Following the flood event of January 1974 the Cities Commission engaged Snowy Mountains Engineering Corporation (SMEC) to determine flood damage along the Brisbane River for floods of various magnitudes (Reference 3). This study also produced flood maps that show areas inundated for a range of flood heights between 2 mAHD and 10 mAHD at the City gauge. Included in the study was a flood frequency analysis carried out by Brisbane City Council using an annual series from 1887 to 1974.

73 Grigg (Reference 4) undertook a 'comprehensive evaluation of the proposed Wivenhoe Dam on the Brisbane River' in 1977. Although no Q100 design event flows were estimated for the Brisbane City gauge it is noted that the probable frequency of the Brisbane City gauge reaching 8 mAHD is 1 in 110 years (before the addition of the Wivenhoe dam) based on a flood frequency analysis. Hausler and Porter completed a report on the 'Hydrology of Wivenhoe Dam' in September 1977 (Reference 33) which, although completed before the full design of the dam was completed, includes the dams predicted effects. It was this study which provided the original design estimates of Q100 for Wivenhoe dam although it does not include flood estimates at the Brisbane City gauge.

74 The first study to establish design flows for the area downstream of the Wivenhoe dam was Weeks (1984, Reference 6). This report built upon the findings of his 1983 report on design floods at the dam itself. Design floods were calculated by using the design rainfalls as input into a calibrated runoff-routing model. Weeks (Reference 6) estimated a Q100 flow of 5510 m^3/s at the City gauge when Wivenhoe dam was in operation. This allowed for a peak outflow from the dam of 3500 m^3/s. By January 1985, for the purpose of the Wivenhoe Dam Operations Manual, Hegerty and Weeks (Reference 34) undertook a flood frequency analysis of flooding in the lower Brisbane River catchment taking into account operation of the Somerset and Wivenhoe dams. Flood frequency plots suggest a Q100 peak flow of up to 6800 m^3/s was derived for the Brisbane City gauge (Reference 34). A number of subsequent reports quote a Q100 peak flow of 6800 m^3/s derived in a 1984 study which was apparently used to set the flood planning levels. This 1984 report is referred to in the June 1999 City Design report (Reference 17) as being the "most recent study completed by Council's Water Supply and Sewerage Department". While WMAwater have not been able to obtain a copy of this report, it is thought this may be an earlier version of Hegerty and Weeks, January 1985 (Reference 34).

75 Following the completion of Wivenhoe dam 1985 a number of hydrology reports and design flood estimate studies were undertaken to establish peak flows both at the Wivenhoe dam and downstream in Brisbane City. Greer's 1992 study includes a summary of information from analyses completed prior to 1991 for Q100 and PMF pre and post dam scenarios (Reference 8). This study only references a pre dam peak flow of 11 500 m^3/s for the City gauge to Brisbane City Council 1984. Ayre, Culter and Ruffini undertook calibration of runoff-routing models in 1992 (Reference 9, which it is believed is also known as the DNR report Brisbane River Flood Study 7a). In March 1993 a DNR report (Reference 10) revised the peak flow estimate up to 8580 m^3/s. This report was apparently considered a 'draft' and revised in August 1993 (Reference 14). Reference 14 adopted the higher value of 9120 m^3/s for the Q100 peak flow at the Port Office based on a storm over the whole Brisbane catchment, as this spatial pattern was critical for the PMF. However, a value of 9380 m^3/s was also given for a storm in only the Upper Brisbane catchment and it seems it is this value which has been referenced in subsequent documents (Reference 17 and 23).

76 The next major study is the June 1998 SKM Brisbane River Flood Study - Final Report (Reference 15). This study used hydrologic and hydraulic models that were calibrated to four events and verified against another four events to establish a post dam peak Q100 flow of 9560 m^3/s at the Port Office. A design flood level for this event was estimated as 5.34 mAHD at Port Office. In December 1998, Brisbane City Council commissioned a review of this report by Mein (Reference 16). This review suggested that, although the approach used in the report was appropriate, the magnitude of the Q100 peak flow was an over estimate. The review considered that:

- Too much emphasis was given to historic events in the 1800's suggesting a higher emphasis should be placed on historic events from the 1900s, and
- Questioning the assumption that the dams were full prior to an event was questionable.

77 Revision the analysis based on the above considerations would have the effect of reduing the peak flow estimate. The review was also concerned about the misclosure between flood frequency analysis and the rainfall runoff approach. The concern was focused on the use of zero losses and the absence of an areal reduction factor to reduce the misclosure.

78 A year on from SKM's original Brisbane River Flood Study, City Design Brisbane City Council completed the Brisbane River Flood Study June 1999 (Reference 17). This report suggested that the most recent Flood Study prior to this was in 1984 and that SKM's 1998 study was only completed to draft status. This study addressed some of the recommendation from Mein (Reference 16). The study found that the 1% AEP (100 year) design flood levels in the river were significantly higher than the current development control levels (set by the 1984 study) by 1m up to almost 3m (Reference 17, Section 6). It estimated a Q100 peak flow of 8600 m^3/s and level of 5 mAHD at the Port Office gauge. It concluded that if current development control levels remained that these would have a return period of 1 in 55 years. One of the most important opinions expressed in this report

is *“The simple option of saying that the current development control level represents the 1 in 100 flood level is not valid.”* (Reference 17, Section 8).

79 These new flows and level were not adopted and instead City Design prepared a *Further Investigations into the Brisbane River Flood Study* in December of the same year (Reference 18). In this report the peak flow for the 1893 event was reduced and thus reduced the estimate of Q100 peak flow by 600m^3/s to a value 8000m^3/s at the Port Office Gauge and the design flood level from 5 mAHD to 4.7 mAHD.

80 It would appear that this level was still not adopted as a planning level and SKM were commissioned by Council to prepare two further reports which were issued on 8th and 28th August 2003 to an Independent Review Panel (Reference 20). These reports have not been made available to WMAwater, although we have been assured that it is essentially the same as the final report issued December 2003 (Reference 21), it is not known what changes were made between the August and December SKM reports. Based on the two SKM August 2003 reports, the independent review panel concluded that a Q100 peak flow of 6000 m^3/s with dam with an estimated flood level of 3.3 mAHD at the Port Office gauge was a more likely estimate than previous estimates of over 8000 m^3/s. The panel also recommended that a plausible range of ±1000 m^3/s and ±0.5m for peak flow and level respectively was appropriate. The review proposed a pre dam flow of 12 000 m^3/s and that the dams reduced the flow by 50%. It is noted that the 2003 Review Panel terms of reference includes the following statement “*Even if the Q100 changes from 6800 m^3/s, it is likely that the Development Control Level will remain the same as is currently used in the Brisbane City Plan.”*

81 In December 2003 the final report was issued by SKM (Reference 21). This report *“used the rainfall-runoff model developed as part of SKM’s 1998 study with additional information and statistical techniques to reassess the plausible range of the Q100 flood”.* The report gives an estimated Q100 peak flow at Port Office Gauge of 6500 m^3/s with a range from 5000 m^3/s to 8000 m^3/s corresponding to a flood level of 3.51 mAHD with a range of 2.76 mAHD to 4.41 mAHD. SKM suggest that the peaks are lower than in their previous 1998 study as areal reduction factors were used, there was more consideration of variation in temporal and spatial characteristics of rainfall, better knowledge of dam operating procedures and inclusion of regional streamflow information in the statistical flood frequency analysis.

82 SKM re-calibrated the 1998 Mike 11 hydraulic model to determine 1% AEP (100 year ARI) flood levels in a report from February 2004 (Reference 22). Although the December 2003 report had later found Q100 to be 6500 m^3/s, the 2003 Review Panel recommended Q100 flow of 6000 m^3/s at the Port Office gauge be used to giving a Q100 flood level at the Brisbane Port Office of 3.16 mAHD.

83 A number of other studies were undertaken between 2004 and the January 2011 flood event although none revised the Q100 estimate. Following the January 2011 event, the Joint Flood Taskforce released a report in March (Reference 27) which states that the

current Q100 peak flow was last estimated in 2003 to be 6000 m^3/s with a corresponding flood level of 3.3 mAHD including the uncertainty bounds as recommended by the 2003 Independent Panel Review. The Taskforce report (Reference 27) states that at the time of the 2011 flood, Brisbane City Council had defined the Defined Flood Event (DFE) to be 6800 m^3/s and the Defined Flood Level (DFL) to be 3.7 mAHD. This was first set in 1978 and reconfirmed in 2003 (Reference 27, page 17). WMAwater were not provided with any 1978 reports to confirm this, and as this is also prior to the construction of Wivenhoe dam, it is assumed this is may actually a reference to the works undertaken in the 1984 report (a copy of which has not been provided). As an interim approach to apply until the conclusion of the Commission of Inquiry and the conduction of a comprehensive flood study recommended by the Taskforce, the Taskforce recommended that the peak flood level from the January 2011 event now be used as the level on which Brisbane City Council bases its considerations for setting habitable floor levels and decisions concerning new development and redevelopment.

84 Table 5 below summarises the change in estimates of Q100 and flood levels at the Port Office gauge over time based on the reports reviewed. This information is also presented in Figure 6 and Figure 7. It should be noted that these estimates were not adopted by Council for flood planning levels and WMAwater believe that the flood planning levels have stayed constant since either 1978 or 1984.

Table 5: Estimates of Q100 peak flow (including effects of Wivenhoe dam) and flood level at Brisbane City / Port Office gauge

Report/Study Date	**Q100 Peak Flow (m^3/s)**	**Q100 Peak Level (mAHD)**
November 1984	5510	-
1984 (unknown report)	6800	3.3
January 1985	6800	-
March 1993	8580	-
August 1993	9120 / 9380	-
June 1998	9560	5.34
June 1999	8600	5.00
December 1999	8000	4.70
September 2003	6000 (±1000)	3.3 (±0.5)
December 2003	6500 (±1500)	3.51 (range 2.76 to 4.41)
February 2004	6000	3.16
March 2011	-	4.46 / 4.27*

* January 2011 Flood Level at City Gauge / Port Office Gauge

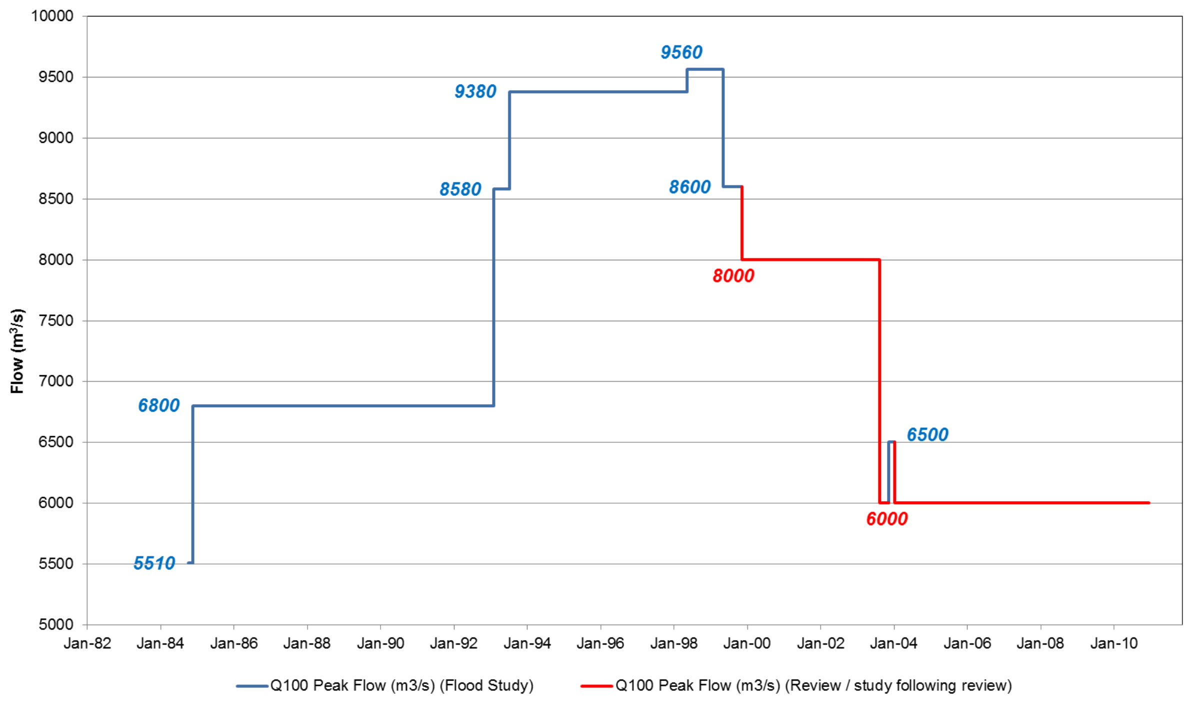


Figure 6: Changes in Q100 Peak Flow Estimates at Port Office/Brisbane City Gauges

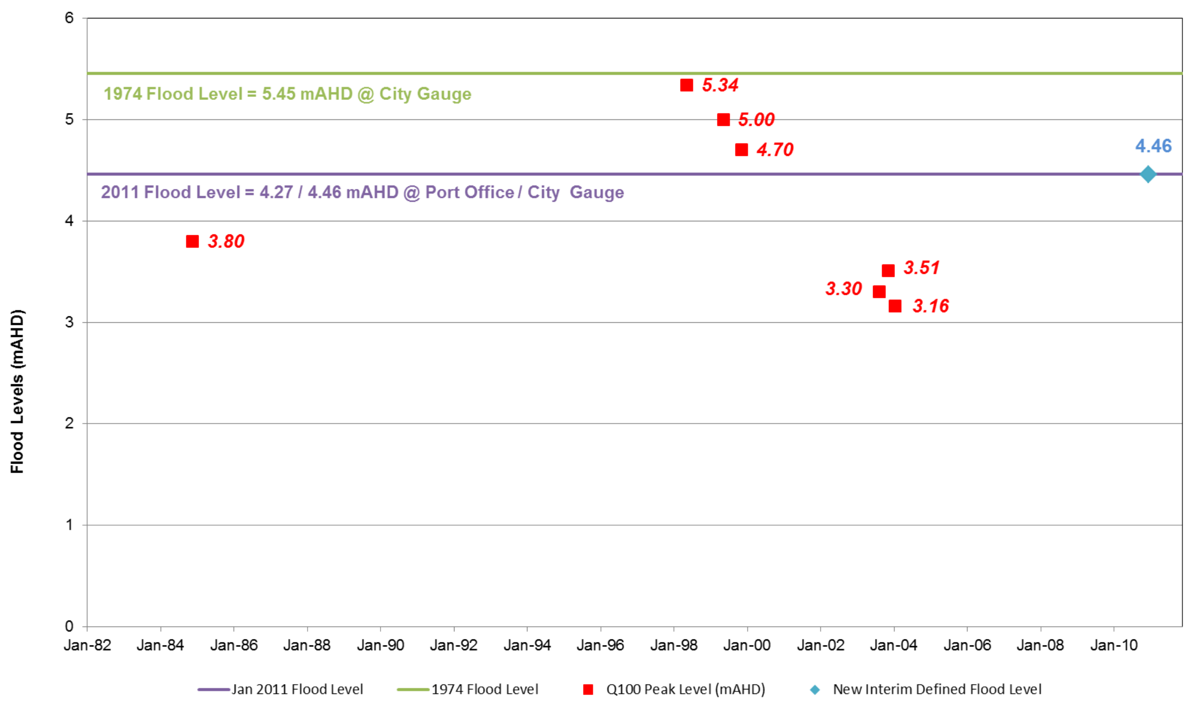


Figure 7: Changes in Q100 Level Estimates at Port Office/Brisbane City Gauges

6. RATING CURVES

6.1. Introduction

85 Rating also known as height-discharge curves, are developed for a specific location, usually a gauging station, in order to convert height (stage) into a value of discharge (flow). Ideally a rating curve will be developed for a location with stable cross section geometry. Rating curve accuracy can also be compromised when the location is prone to backwatering or tidal influence. The rating curve is best developed using a series of gaugings (height and discharge observations) from a number of different sized events. The 2011 flood event highlighted the need to revise key rating curves within the Brisbane River catchment.

86 A series of rating curves were developed at the Port Office gauge using the current SKM/Seqwater Mike 11 model (Reference 35), earlier Mike 11 modelling by Brisbane City Council and SKM (Reference 18), estimates of flow from previous studies and information about tides and dredging.

6.2. Deriving a Rating Curve

87 A rating curve is often developed based on a series of data sources and techniques. This may include observations of height and discharge, which are typically limited to smaller events due to the rarity of and access issues associated with large flood events. In order to extend the rating curve beyond observed events, extrapolation techniques are often employed. Extrapolation methods include hydrodynamic models, simple hydraulic equations such as Manning's equation or curve fitting techniques.

88 While a rating curve is best derived from a series of observed height and discharge gaugings, if data is not available or are unreliable a rating curve may be developed using a calibrated hydrodynamic model which should be informed by accurate and relevant bathymetric and topographical data. To ensure the robustness of a rating curve derived from a hydrodynamic model sensitivity to downstream levels, tides and changes in cross section topography should be tested.

6.2.1. Bathymetric Data

89 The previous studies appear to have used three sets of bathymetric data. Detailed survey was conducted in 1873 from Victoria Bridge to Moreton Bay (Reference 18). Following the 1974 flood event the Department of Harbours and Marine carried out a detailed survey (Reference 3, Part 3 Section 3). The 1998 - 1999 SKM and BCC studies (Reference 15, 17, and 18) appear to have accessed newer survey however no date is given. We have not had access to this data in its original form. Data such as this is critical to the development of a rating curve and is important for understanding how the river changes with time. Reference 18 incorporated the 1873 survey into a Mike 11 model to estimate conditions for the 1893 flood event.

90 Collection and review of bathymetric data and its use in a two dimensional (2D) hydrodynamic modelling would substantially aid current efforts to construct rating curves for the Port Office which are representative of height-discharge conditions at the time of large flood events (eg. 1893 and 1974).

6.2.2. Topographic Data

91 Depending on the size of a particular event, rating curves can be sensitive to overbank conditions and topography. Often a change in slope is seen in the rating curve as flow enters the overbank. Future work utilising 2D hydrodynamic modelling to develop rating curves specific to a point in time needs to ensure that overbank topography is representative of the time of the event in question.

92 If a 2D model is developed it should use high resolution survey data (LiDAR) for current conditions. To model earlier events it would be necessary to draw upon a range of data sets including aerial photography, ortho-photo maps, 1873 and 1974 survey details.

6.3. Port Office Rating

93 The Port Office gauge is located approximately 1km upstream of the Storey Bridge on the left bank of the Brisbane River. It is operated by Marine Safety Queensland, part of the Department of Transport and Main Roads. A second gauge, the Brisbane City gauge, is located directly opposite on the right bank of the river. This gauge is operated by Seqwater and is central to the Brisbane flood warning system. In most instances the Port Office and City gauges will record identical information and as such the names of each are used interchangeably.

94 While previous studies have developed a rating relationship there appears to be no official rating curve for the Port Office/City gauge. In the many reports reviewed few details are given in regard to the rating curve used to convert historical stage observations (at Port Office) into associated peak discharges. A consistent feature of the reviewed reports is that the existence of a rating curve can only be implied, other than Reference 15. Although sufficient details of the rating curves are not provided in previous reports, various reports suggest that they have been provided by BoM, Brisbane City Council, or developed based on modelling.

6.3.1. Observed Height-Discharge Data

95 A key objective of reviewing previous reports was to establish observed height-discharge observations for the Port Office, or other locations, that could be used in establishing a reliable rating curve. There were very few documented instances where height and discharge were measured, many of which did not present the actual measurements. For most gauges, gaugings tend to be in the lower range of floods however because the Port Office is effected by tides most tended to be in the upper ranges.

96 During the review of previous reports, information was found that indicated that many gaugings had previously been undertaken. Reference 3 indicated that significant gauging of discharge had been carried out for events in 1931, 1951, 1955 and 1968 although none of the original information relating to these gaugings has been found in any of the available reports. Further investigation could likely unearth these and such work would be of some help in confirming the rating curve for Port Office.

97 Reference 18 shows that in large floods the peak flow at Moggill, Jindalee and Port Office tends to remain approximately constant. This is supported by model results in Reference 15 and Reference 21 as well as results from the model used in Reference 35. Modelling carried out by SKM (Reference 35) of the January 2011 event where ~ 9,600 m^3/s was measured at Jindalee and hence it can be assumed that ~ 9,600 m^3/s also occurred at Port Office.

6.3.2. Deriving a High Flow Rating at the Port Office

98 The 2011 flood event provides extra information about large floods that was not available to previous studies. By combining the limited information about height and discharge available for the 2011, 1974 and 1893 floods it is possible to estimate a high flow rating curve. This is not an easy task as there are conflicts between the data available.

99 The recorded stage for the 1893 flood event was 8.35 mAHD, however it is necessary to adjust this height for current conditions. Reference 17 adjusted all the recorded stages from 1864 to 1917 by 1.52m (5 feet) except for the 1893 event and assumed the discharge of this event was 14 600 m^3/s. Table 10.2 of Reference 17 presents 5 estimates of flow ranging from 11 300 to 16 990 m^3/s for the 1893 event. Two of these estimates are based on velocity measurements taken during the event at Indooroopilly and Victoria Bridges (16 990 m^3/s and 14 600 m^3/s respectively). Reference 3 (Part 3, Section 2) details problems associated with reverse flow on the inside bend making measuring flow at Indooroopilly Bridge difficult during the 1930's and 1950's. Reference 18 modelled the 1893 event using cross sections from 1873 and estimated the peak flow at 11 600 m^3/s.

100 WMAwater tested the impact of dredging using SKM's Mike 11 model. This relatively simplistic testing indicated that the dredging and river straightening works would effect the 1893 flood level and the 1.52m estimate used by Reference 17 for smaller floods was probably appropriate for the 1893 event. This gives a plausible range of levels for the 1893 event under current conditions of between 6.83 mAHD and 8.35 mAHD and a peak flow of between 11 300 m^3/s to 16 990 m^3/s.

101 Reference 3 discusses issues with gaugings from the Centenary Bridge during the 1974 event. There are several flow estimates for 1974 including 9800 m^3/s (Reference 15), 9873 m^3/s (Reference 17). Information learnt from ratings of the 2011 event at Jindalee suggests that these earlier estimates were slightly low, with a revised Jindalee estimate of

10 900 m^3/s (Reference 35). This gives a plausible range of peak flows of 9800 m^3/s to 10 900 m^3/s and a peak height of 5.45 mAHD.

102 For the 2011 event a flow of 9600 m^3/s has been used with a height range of 4.27 to 4.46 mAHD.

103 A best estimate high flow rating curve has been developed using the above plausible height-flow dataset. The lower end of the adopted curve is based on an average of the rating curves established in the 1998 and 1999 Flood Studies.

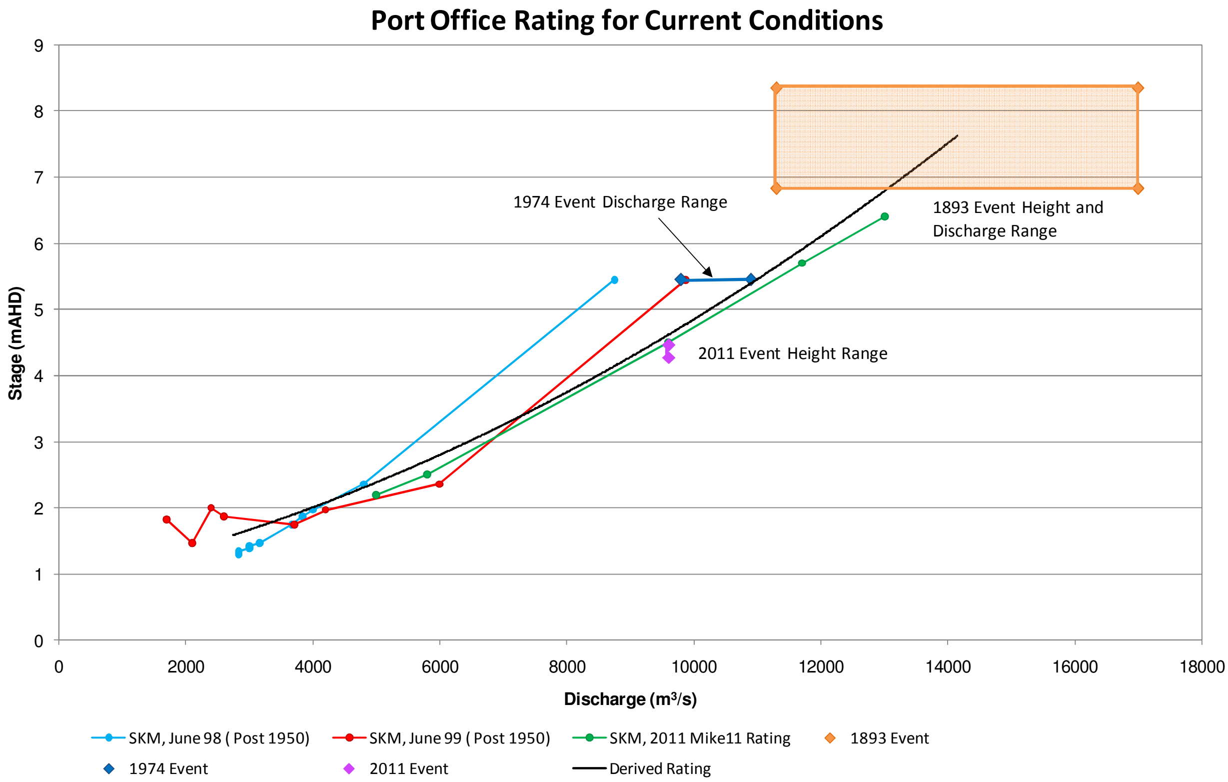


Figure 8: Port Office Rating for Current Conditions

7. ANALYSIS

7.1. Flood Frequency Analysis

7.1.1. Previous Flood Frequency Analysis

104 Flood Frequency Analysis (FFA) has been undertaken previously as discussed in Section 5 (Previous Studies). In particular, extensive FFA was undertaken in 1985 (Weeks), 1993 (DNR), 1998/1999 (SKM), and 2003 (SKM).

7.1.2. Choice of Location for At-Site Analysis

105 As discussed (refer to Section 3), important factors for reliable estimation of larger flood magnitudes include:

- long record length,
- homogeneous conditions over the length of record, which can be affected by:
  - dam construction,
  - changes to the gauge location and/or river cross-section at the gauge,
  - land-use changes, and
  - climate change or long-term climatic cycles, and
- preferably the inclusion of the largest known historical floods within the continuous period of record.

106 While there are several gauges on the Brisbane River, the majority of these gauges have relatively short record lengths (less than 35 years), making them of limited value in estimating larger events such as the 1% AEP (100 year ARI) flood magnitude. For short records FFA becomes an extrapolation method and estimates are heavily biased by what information is contained within the short record. The Port Office gauge at Brisbane is the notable exception with a record of large events from 1841 (over 170 years).

107 With regards to record length, other gauges which are potentially suitable for the estimation of the 1% AEP (100 year) Brisbane River flow include:

- gauge 143001 at Savages Crossing (established in 1909 at Lowood and relocated twice, to Vernor in 1950 and then to the current location in 1958, for a composite record length of 112 years),
- gauge 143003 at Mt Crosby Weir (record length of 76 years from 1900-1975 inclusive), and
- gauge at Moggill (record length of 46 years since 1965).

108 The relative merits of using the gauges identified above for FFA on the Brisbane River are discussed below.

7.1.2.1. Savages Crossing and Mt Crosby Weir

109 The gauges at Savages Crossing and Mt Crosby Weir both record flow from a similar area of the Brisbane River catchment as there are no major tributaries between them. Both records have the problem that they are affected by the construction of Somerset dam (and also Wivenhoe dam for Savages Crossing). Reference 12 (DNR, 1993) accounts for this change by undertaking a separate FFA for each of the two portions of the record, before and after Somerset dam construction. However the analysis gave the illogical result that the 1% AEP (100 year) flow for the later period (with Somerset Dam) was found to be substantially higher than the earlier period.

110 Reference 15 (SKM, 1998) accounted for the construction of Somerset dam by adjusting the flows recorded after the construction of Somerset dam to pre dam conditions and undertaking FFA on the entire record. This approach gave results that were reasonably consistent with results from the Moggill and Port Office gauges.

7.1.2.2. Moggill Gauge

111 The Moggill gauge was established in 1965, after the construction of Somerset dam. The current record length is therefore 47 years (inclusive of 2011), just over a quarter of the length of the Port Office record. The main source of heterogeneity in the Moggill record is the construction of Wivenhoe dam in 1985. Additionally, the gauge has been operational during a relatively dry period compared to other gauge records, with only been two major floods occurring in this period, including the January 2011 flood. The results for larger events from FFA at this site are therefore heavily influenced by whether the January 2011 flood is included or not.

112 Due to uncertainty about the effects of Wivenhoe dam on observed floods, Reference 15 only considered the period from 1965 to 1983 (19 years inclusive). During this period only one major flood event occurred (1974).

7.1.2.3. Port Office Gauge

113 This gauge is subject to heterogeneity from multiple sources. The most notable of which are:

- changes to the river bathymetry, both from natural sources and engineering works. Notable changes that have been identified include channel modification, construction of training walls and artificial islands to alter the natural tidal flow patterns, removal of sand bars, and dredging,
- construction of dams (most notably Wivenhoe and Somerset dams, introducing two step changes into the record), and
- changes to catchment land-use.

114 The major advantage of using the Port Office gauge is the significantly longer record length. This earlier period also captures much more information about large events on the Brisbane River and is summarised in Table 6.

Table 6: Recorded Floods by Period (Adjusted for Pre Dam Conditions)

Period	1841 to 1907 (67 years inclusive)	1908 to 2011 (104 years inclusive)
Number of recorded annual floods greater than 2,000 m^3/s	17	14
Number of recorded annual floods greater than 8,000 m^3/s	5	2

115 The Port Office gauge is therefore considered significantly more likely than the Moggill gauge to adequately capture long term climatic variation for the Brisbane River catchment. Despite the uncertainty introduced by changes to river channel conditions and the uncertainty over whether early flood measurements can be adjusted to current datum. It is therefore considered the best location for estimating the 1% AEP (100 year ARI) flow between Moggill and Brisbane City.

116 For these reasons previous flood frequency analyses have generally been focussed on the Port Office gauge, and it is also considered the most suitable site for conducting FFA for the purposes of addressing The Commission's questions within the scope of this report.

7.1.3. Creating a Homogeneous Data Set

117 Flood frequency analysis needs to be carried out on a homogeneous dataset. In order to do this at the Port Office gauge the flow record needs to be adjusted to consistent conditions. The gate operations used at Wivenhoe dam target specific flows at Moggill. This produces a stepped flow curve for smaller discharges. Flood frequency analysis assumes the flow curve will be smooth, and therefore it must be carried out on pre dam flows.

118 In order to construct a homogeneous data set the flow record was adjusted to represent the peak flow that each flood would produce under current catchment conditions without the presence of Wivenhoe and Somerset dams. The early floods had to be adjusted for dredging, river straightening and the bar that was removed in 1864, while the later floods required adjustment for Somerset and Wivenhoe dams.

119 For floods prior to 1917 the 1.52m dredging adjustment was used and for those prior to 1864 the 0.4m bar adjustment was also used. This is the same approach as used in Reference 17 other than the dredging adjustment has been applied to all events (including 1841 and 1893). For smaller events the flow adjustment used in Reference 17 was also used. For larger events the high flow rating derived as part of the current study was used (refer to Section 6.3.2).

120 Adjustments were made to the 1974 event to account for Somerset dam and to the 2011 event to account for both dams. Every attempt was made to make adjustments in a consistent and non contradictory manner. The adopted high flow estimates are presented

in Table 7 below. It is noteworthy that the 1841, the second 1893 event and the 2011 event are essentially the same size.

Table 7: Homogeneous Data Set of Flood Levels for the Brisbane River

Event	Recorded Level (ie. As measured during the event) (mAHD)	Adjusted Level (mAHD)*	Pre Dam Current Conditions	
			Height (mAHD)	Flow (m^3/s)
1893 (a)	8.35	6.83	6.83	13700
1893 (b)	8.09	6.57	6.57	12600
1841	8.43	6.51	6.51	12500
2011	4.27	4.27	6.40	12400
1974	5.45	5.45	5.50	11300
1844	7.03	5.11	5.11	10400
1890	5.33	3.81	3.81	8100
1898	5.02	3.50	3.50	7500

*Includes 1.52m prior to 1917 and an additional 0.4m adjustment for prior to 1864

7.1.4. Ranking of Events

121 Historic events need to be ranked from largest to smallest in order determine their plotting position. Ranking of events was carried out on the homogeneous dataset (ie. Pre dam levels described in Section 7.1.3. Only the larger events are included in Table 8 though all were included in the subsequent flood frequency analysis. The second 1893 event is not included in the ranked series as an annual series was used in the flood frequency analysis.

Table 8: Ranking of Historic Events (Annual Series)

Event	Pre Dam Flood Level (mAHD)	Rank
1893 (a)	6.83	1
1841	6.51	2
2011	6.40	3
1974	5.50	4
1844	5.11	5
1890	3.81	6
1898	3.50	7

122 Table 8 demonstrates that if the flood frequency analysis focus is only on events in the 20th century then it will result in a very different answer to one which includes the 19th and 21st century flood information.

7.1.5. Plotting Position

123 By considering the rank and period of record it is possible to estimate the most likely probability (AEP or ARI) of each event. The plotting position is used for plotting an observed event on a flood frequency diagram. The plotting position generated by

considering the rank is the most likely probability based on sampling theory and not the actual probability of an actual event. The Cunnane formula is used to determine the plotting position:

$$PP(m) = \frac{m - 0.4}{N + 0.2}$$

Where PP = plotting position
m= rank of the flood in the series.
N= number of years in the record (171 years for Port Office gauge)

Table 9: Plotting Position and Most Likely Probability of Historic Events

Event	Pre Dam Flood Level (mAHD)	Rank	Plotting position (AEP) %	ARI (1/PP)
1893 (a)	6.83	1	0.35	285
1841	6.51	2	0.94	107
2011	6.40	3	1.52	66
1974	5.50	4	2.10	48
1844	5.11	5	2.69	37
1890	3.81	6	3.27	31
1898	3.50	7	3.86	26

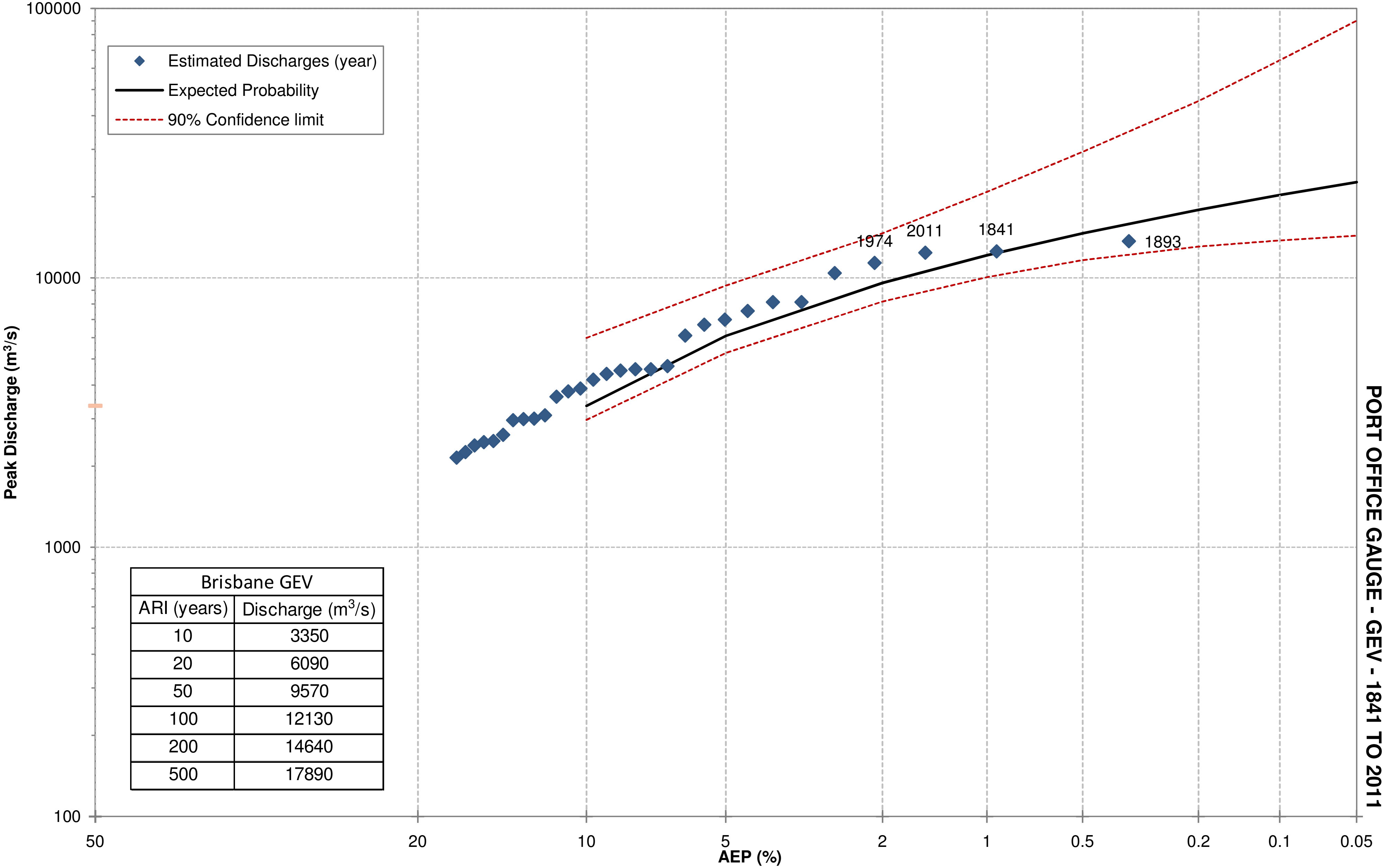
7.1.6. Flood Frequency Analysis – Port Office Gauge

124 Flow data series at Port Office (refer to Appendix B) were analysed using the Generalised Extreme Value (GEV) and Log Pearson 3 (LP3) distributions. Frequency analysis of this location presented a range of complications. While it is a very long record by Australian standards it is very hard to produce a consistent rating curve and properly account for the effects of the astronomical tide and storm surge. This causes the rating curve to be less reliable at low flows and causes a focus on high flows. Figure 9 and Figure 10 show the pre dam fit of the GEV and LP3 distributions. In both cases the fitting algorithm was challenged by the top few floods which have very similar flow values. The fits are sensitive to minor changes to the top few flows.

125 Analysis at the Port Office gauge was undertaken with and without the January 2011 flood data point. Additionally, the analysis was performed on two sections of the flood record:

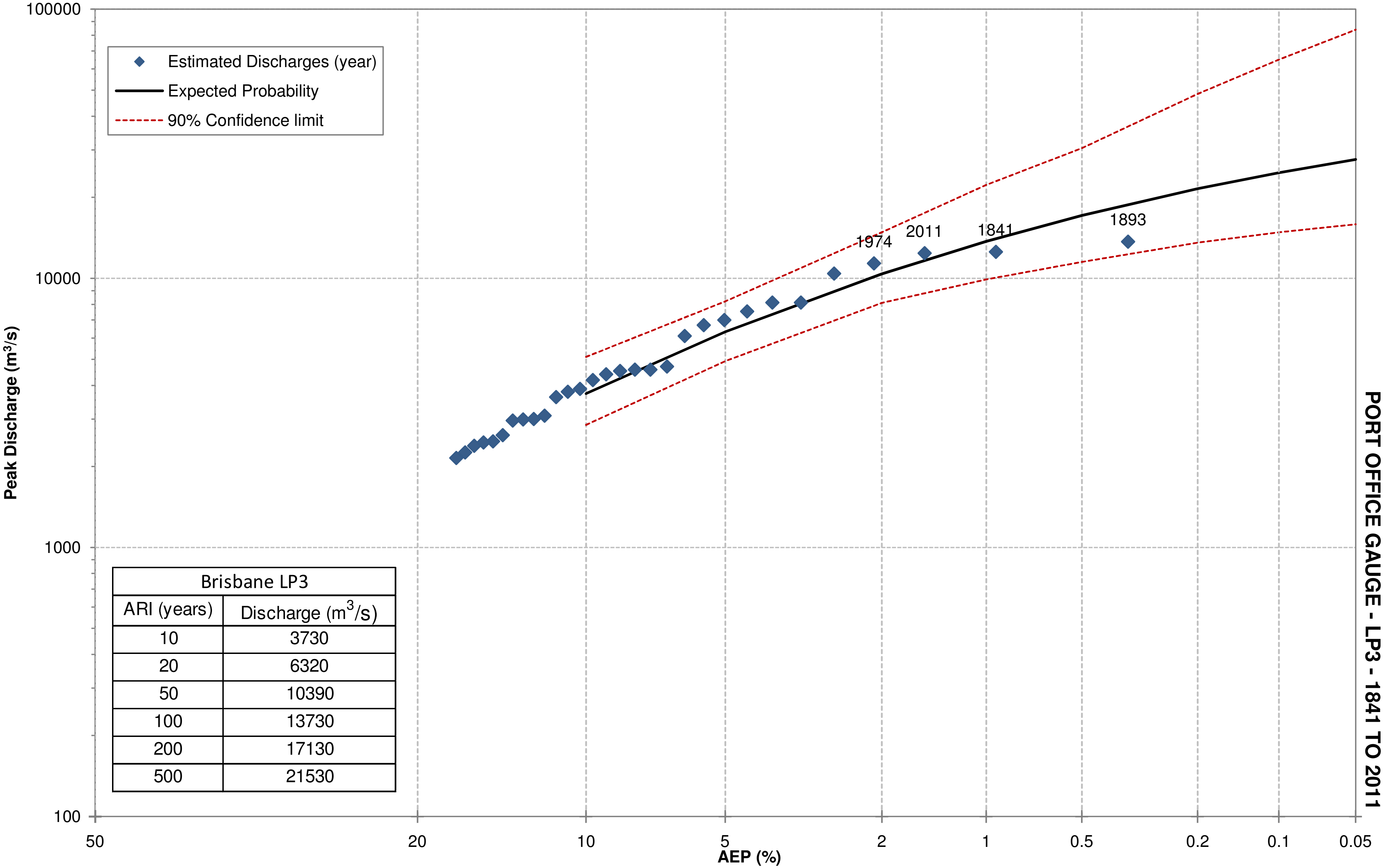
- The full record (1841 to 2010/2011), and
- A partial record, consistent with the period of record from the Lowood/Savages Crossing composite gauge (1908 to 2010/2011).

126 The main purpose of conducting the analysis on the partial record from the 20th century was to ascertain the influence of the “wetter” 19th century period of record on the results, bearing mind that the 19th century period is also more heavily affected by uncertainty from changes to river bathymetry from channel works. This comparison can therefore provide some understanding as to how flood frequency estimates from the gauges with 20th century records such as Moggill and Lowood might change with longer records.



Brisbane GEV	
ARI (years)	Discharge (m^3/s)
10	3350
20	6090
50	9570
100	12130
200	14640
500	17890

FIGURE 9
FLOOD FREQUENCY ANALYSIS
PORT OFFICE GAUGE - GEV - 1841 TO 2011



Brisbane LP3	
ARI (years)	Discharge (m^3/s)
10	3730
20	6320
50	10390
100	13730
200	17130
500	21530

FIGURE 10
FLOOD FREQUENCY ANALYSIS
PORT OFFICE GAUGE - LP3 - 1841 TO 2011

127 Results from the 4 flood frequency analyses undertaken at the Port Office gauge are shown in Table 10 (for both GEV and LP3 distributions).

Table 10: Comparison of Q100 Estimates for Considered Approaches

Data set/ Case	Q100 (m^3/s)	
	GEV	LP3
1841-2011[a]	12 130	13 730
1841-2010[a]	11 740	13 900
1908-2011[b]	10 740	16 610
1908-2010[b]	9 510	13 900

[a] 141 censored flows lower than 2,000 m^3/s
[b] 90 censored flows lower than 2,000 m^3/s

128 Conducting the flood frequency analysis without the 2011 event changes the average of the GEV and LP3 estimates by only 95 m^3/s.

129 The partial record results are influenced by the relative lack of data points during this period. Only 14 floods in the 104 year period are above the 2000 m^3/s threshold. Below this threshold tidal effects at the Port Office gauge have a far greater influence on recorded level than Brisbane River runoff, and the flows determined from a rating table below this level are therefore subject to significant uncertainty. The longer record has 30 gauged floods, resulting in an improved fit with less variability resulting from the distribution assumed for the analysis.

130 The distribution fits and confidence limits for the 1841 to 2011 period are illustrated in Figure 9 and Figure 10. The quantile estimates are shown in Table 11.

Table 11: Flood Frequency Analysis Results (1841-2011)

AEP (%)	ARI	Design Flows (m^3/s) at the Brisbane Port Office	
		GEV	LP3
20	5	440	1740
10	10	3350	3730
5	20	6090	6320
2	50	9570	10 390
1	100	12 130	13 730
0.5	200	14 640	17 130

Note: based on annual series 1841-2011, 141 censored flows lower than 2,000 m^3/s

131 In this case the 1% AEP estimates by the GEV and LP3 are relatively similar, with the LP3 providing a slightly better fit. On this basis a 1% AEP estimate of 13 000 m^3/s was adopted for the pre dam case. This estimate is similar to those of the more recent flood frequency estimates of 13 700 m^3/s (Reference 15) and 12 300 m^3/s (Reference 17).

132 Using Figure 3 without applying any weight to the 2011 event a value of 9000 m^3/s is obtained as the post dam (Wivenhoe and Somerset dams) flow. The 2011 data provides

the only real data point on the performance of the dam and suggests a post dam flow of 10 000 m^3/s using WMAwater's estimate and 9500 m^3/s using SKM's estimate. On the basis of these 3 datasets a post dam flow of 9500 m^3/s was adopted.

133 Based on these conclusions the 2011 flood event has a probability of 0.83% AEP (120 year ARI) under current conditions and under pre dam conditions would have a probability of 1% AEP (100 year ARI).

7.1.7. Uncertainty of Peak Flood Estimates

134 Design flow estimates by their very nature have a considerable level of uncertainty associated with them. The work presented herein trade off the benefits of a very long flow record with the uncertainty associated with the Port Office rating curve. The uncertainty limits shown on Figure 9 and 10, have a larger confidence limit above the flow estimate (expected probability line) than below. The information gained from the 2011 flood event shows that the dams mitigation potential can be considerably less than what was previously assumed by other studies (Figure 3). Weighing up these factors suggests that the uncertainty bound below the best estimate post dam flow (9500 m^3/s) is smaller than the uncertainty bound above the estimate.

7.2. Rainfall Comparisons

135 A major concern of the 2003 Review Panel was the misclose between flood frequency and rainfall runoff estimates. This section examines some of the causes.

136 The flood record is dominated by events between the 1840's and 1890's for which there is very little corresponding rainfall data. Figure 11 shows the number of long term rain gauges (with records longer than 30 years) in the Brisbane River Catchment. Only gauges listed in the BoM Water Resources Station Catalogue (Reference 37) were included in the analysis and obvious duplicates were removed or amalgamated. The 1840's events are potentially captured by only one gauge. The number of gauges increased towards 22 by the 1890. This means that any catchment average rainfalls developed for these events are likely to contain a high degree of uncertainty. This figure also demonstrates that rainfall based methods will be dominated by information from the 20th Century.

137 While the probability of rainfall is not usually the same as the probability of the flood, it can give a general indication of the likelihood of the event. The 2011 rainfall totals for 3 days were compared to the 1987 ARR design rainfalls and a separate series of catchment design rainfalls based on data up to 2009. This more recent analysis also makes use of spatial surface fitting techniques that were not available in 1987. Both design rainfall estimates were only based on official BoM gauges, which do not include alert gauges. Table 12 compares catchment average rainfalls for the catchment to Wivenhoe, the Lockyer, the Bremmer and total catchment to the Port Office gauge.

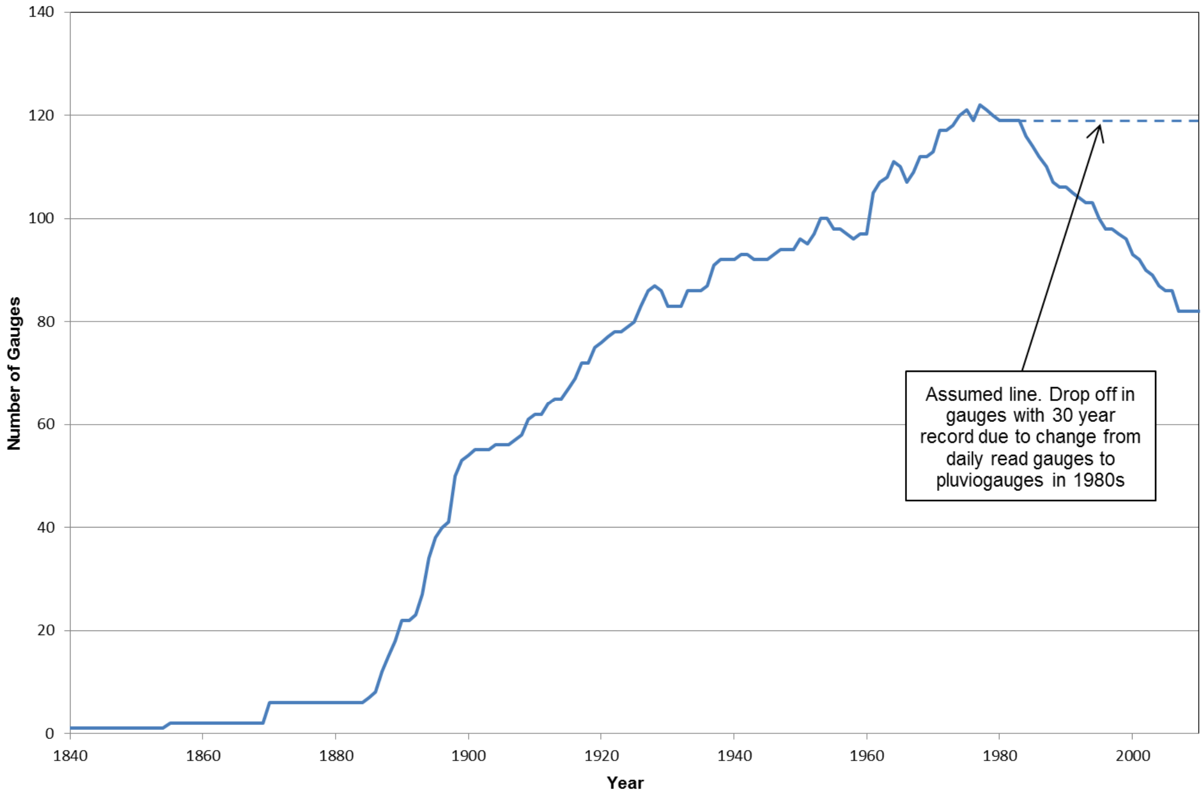


Figure 11: Number of Long Term Rainfall Gauges Available Compared to the Length of Historical Flood Record.

Table 12: Catchment Average Rainfall (mm)

Location	2011 3 Day Peak Rainfall (Seqwater Analysis) (mm)	72 hr 100 Yr ARR 87 (mm)			72 hr 100 Yr based on updated rainfall (mm)		
		No Areal Reduction	0.9 Areal Reduction	0.8 Areal Reduction	No Areal Reduction	0.9 Areal Reduction	0.8 Areal Reduction
To Wivenhoe	326	386	347	309	421	379	337
Lockyer	252	329	296	263	332	299	266
Bremmer	204	350	315	280	390	351	312
All Catchments	280	372	335	298	402	362	321

138 The design rainfall depths have been adjusted using an areal reduction factor. This factor adjusts point rainfall estimates to be used as catchment wide estimates. The use of areal reduction factors have been discussed quite considerably in the 1998 review (Reference 16). While the authors have no theoretical problem with the application of areal reduction factors we have the concern that the BoM rain gauges used in both design assessments (ARR 87 and revised design assessment) have a bias in their location that leads to an underestimation of catchment average rainfall. This is because rain gauges tend to be located on relatively flat land that is suitable for farming or a town. This problem was also found in the 2011 event by SKM (Reference 36) and Seqwater (Reference 26). This is also probably a source of some of the misclosure between rainfall based methods and

flood frequency based methods in some of the earlier studies. This misclosure has been found by the author on other catchments on the east coast of Australia with relatively rugged terrain.

139 Table 12 suggests that on a 72 hour rainfall basis the 2011 event upstream of the dam was slightly larger than a 1% AEP event and slightly smaller than a 1% AEP event downstream of the dam.

7.3. Determining the 1% AEP Line

140 The following locations were identified by The Commission as being of interest:

- 13 Bridge St., Redbank (off-bank),
- Cnr. Ryan St. and Woogaroo St., Goodna,
- Corner Moggill Rd, Birkin Rd, Bellbowrie (Coles),
- Corner Thiesfield St, Sandringham Pl, Fig Tree Pocket,
- 312 Long St East, Graceville,
- Brisbane Markets, Rocklea,
- Softstone St, Tennyson (Tennyson Reach apartments),
- 15 Cansdale St, Yeronga,
- 42 Ferry Rd, West End (Aura apartments),
- 81 Baroona Rd, Paddington (Epic Cycles), and
- Brisbane City Gauge.

7.3.1. Mike 11 Model

141 The Mike 11 Model (Version 2) developed by SKM for Seqwater as described in Reference 38 (and Version 1 described in Reference 35) was calibrated by SKM to Moggill, Jindalee and the Port Office. It was intended to use this model to fit a flood surface between Moggill and the Port Office for a peak post dam design flow of 9500 m^3/s. When this model was compared to observed flood height data in the 2011 Joint Taskforce report (Reference 27, Table 3) problems were found with the fit (Figure 12). While the model fitted well at Moggill, Jindalee and the Port Office the fit was slightly low between Moggill and Jindalee and up to 1.8m low between Jindalee and the Port Office. This problem demonstrates the need for organisations to consider all agencies data when calibrating models. The Mike 11 model was therefore unsuitable to be used in profile generation.

7.3.2. Profiles

142 As part of the prescribed work scope The Commission required profile information on peak flood levels between Moggill and the Brisbane River mouth for the 2011 event and 1% AEP. January 2011 levels at each location were estimated by adjusting the Mike 11 model results to match the observed data from 2011 Joint Taskforce (Reference 27). From this, approximate flood levels at the points of interest identified by The Commission were

determined. The same process was adopted for the 1% AEP flood levels using a peak post dam flow of 9500 m^3/s. These profiles are presented on Figure 13 and summarised in Table 13.

Table 13: Estimated 1% AEP Peak Flood Level and 2011 Peak Flood Level for Locations on the Brisbane River

Location	Estimated 1% AEP Peak Flood Level (mAHD)	Approximate January 2011 Peak Flood Level (mAHD)
13 Bridge St., Redbank (off-bank)	16.81	17.21
Cnr. Ryan St. and Woogaroo St., Goodna	15.96	16.37
Cnr. Moggill Rd. and Birkin Rd., Bellbowrie (off-bank)	14.63	15.04
Cnr. Thiesfield St. and Sandringham Pl., Fig Tree Pocket	10.86	11.22
312 Long St. East, Graceville	9.76	10.10
Brisbane Markets, Rocklea	9.51	9.84
Softstone St., Tennyson (Tennyson Reach Apartments)	9.58	9.90
15 Cansdale St., Yeronga (off-bank)	8.58	8.85
42 Ferry Rd., West End	6.55	6.75
81 Baroona Rd., Paddington (off-bank)	5.77	5.95
Brisbane City Gauge	4.32	4.46

143 Sensitivity testing using the flow estimate from the 1841 to 2010 data set found that the 1% AEP (Q100) height estimate at Moggill and the Port Office would reduce by approximately 0.5m and 0.2m respectively. While there are only minor differences in pre dam estimates (between the 1841-2010 and 1841-2011 datasets) the conversion to post dam, without knowledge gained from the 2011 event regarding dam mitigation ability, results in a post dam estimate of 500m^3/s less.

7.3.3. Review of the 1% AEP Flood Line

144 The 1% AEP (Q100) event as currently defined achieves has a peak level of 3.3 mAHD at Port Office/City gauge. Figure 13 shows Brisbane City Council's current "Q100" flood level profile (adapted from Reference 27). When this is compared to the revised 1% AEP flood profile based on 9500 m^3/s there is up to 3 metres discrepancy between the two near Moggill and a 1m difference at Port Office gauge.

FIGURE 12

SKM MIKE 11 JANUARY 2011 PEAK LEVEL PROFILE VS OBSERVED 2011 FLOOD LEVELS (2011 REVIEW PANEL)

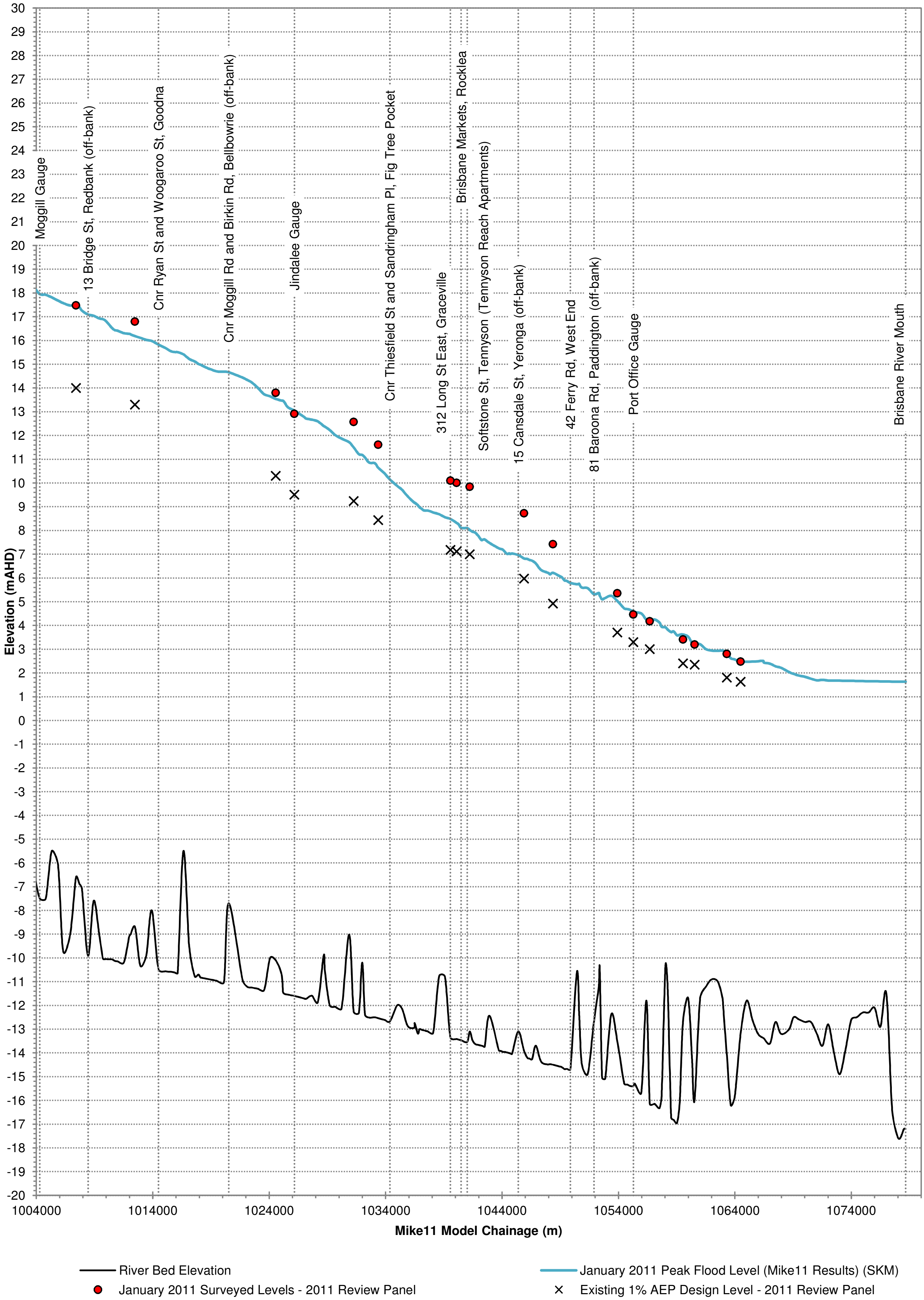
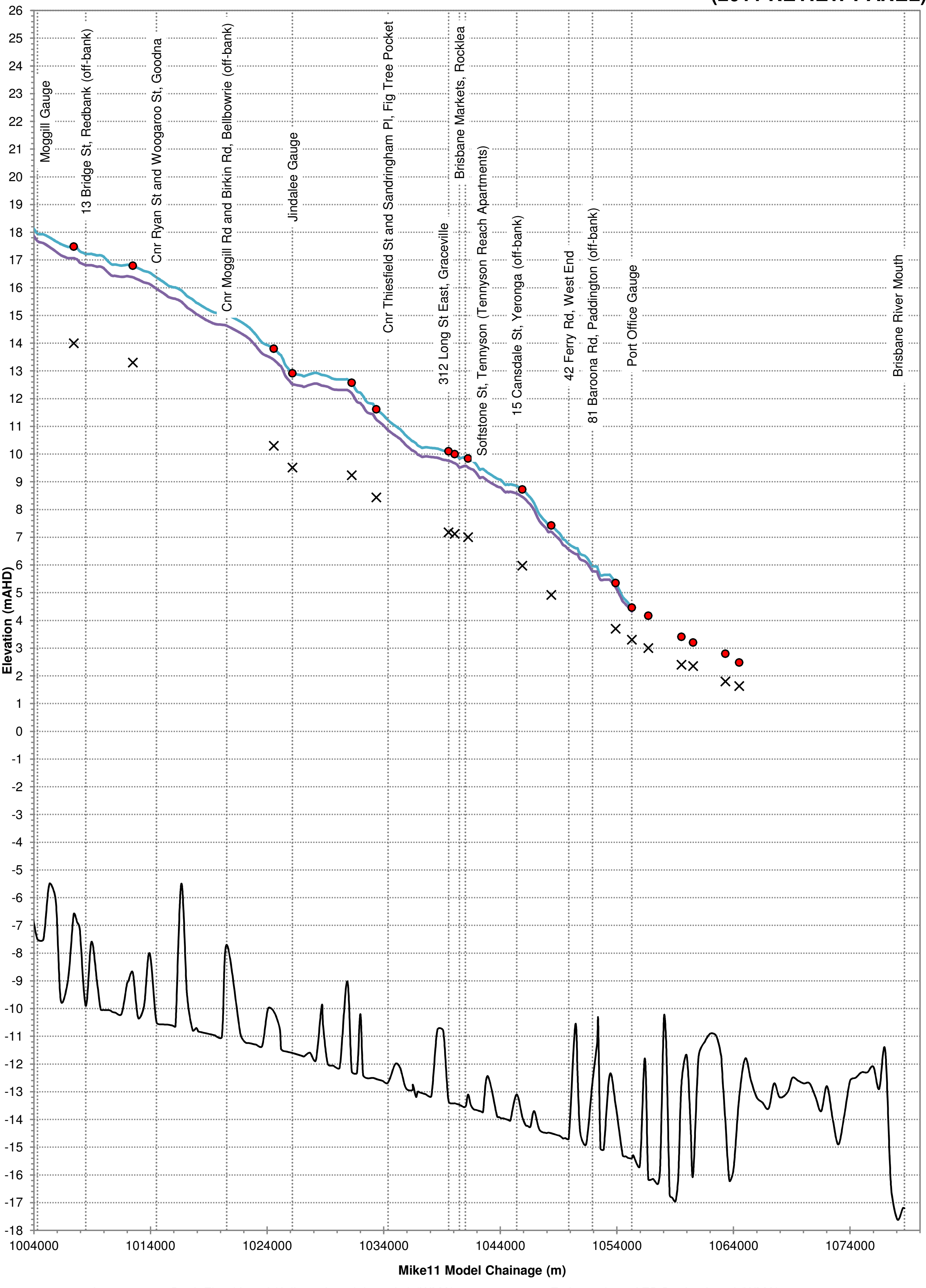


FIGURE 13

Q100 LINE & JANUARY 2011 PEAK LEVEL PROFILE VS OBSERVED 2011 FLOOD LEVELS (2011 REVIEW PANEL)



January 2011 Peak Flood Level (Adjusted model results) (WMA)
Estimated 1% AEP Design Level (WMA)
January 2011 Surveyed Levels - 2011 Review Panel
Existing 1% AEP Design Level - 2011 Review Panel
River Bed Elevation

8. CONCLUSIONS AND RECOMMENDATIONS

145 A flood frequency analysis has been carried out on the Port Office gauge record for the period from 1841 to 2011. The 2011 flood shows the credibility of the large floods that occurred early in the settlement of Brisbane. This analysis gives a pre dam flow of 13 000 m^3/s which is consistent with many earlier estimates. This estimate is not sensitive to the inclusion of the 2011 flood. This pre dam estimate translates to a post dam estimate of 9500 m^3/s. This estimate is slightly sensitive to the new information gained from the January 2011 event on how Wivenhoe dam mitigates large floods. Without this new information the post dam flow would 9000 m^3/s (500m^3/s less).

146 The current Q100 flood line used by Brisbane City Council is significantly below the revised 1% AEP (Q100) flood line calculated by this study with a difference ranging from approximately 3m at Moggill to approximately 1m at the Port Office. The new line is slightly below observed levels of the 2011 flood event. The frequency analysis found that the 2011 flood has a return period of approximately 120 year ARI with Wivenhoe and Somerset dams in place (post dam) and a return period of approximately 100 year ARI under pre dam conditions.

147 The major source of uncertainty in estimating flood risk for Brisbane comes from the uncertainty of the rating relationship at the Port Office gauge. While this is not an easy location to generate rating curves it is necessary if the benefit of the long term gauge record is to be properly utilised.

8.1. Improving the Rating Relationship at Port Office Gauge

148 A detailed study needs to be undertaken to improve the rating relationship at the Port Office gauge. This study needs to draw upon all the information held by Council and State Government. The rating information held by different organisations also needs to be consolidated and objectively reviewed.

149 The study needs to contain the following components:

- Development of a suitable industry standard 2D hydrodynamic model of the lower reaches of the Brisbane River. This model needs to be suitable for assessing historical changes to the river bathymetry and needs to have a run time that is practical for detailed calibration and assessment of changes,
- A detailed search of all data sources on the bathymetry of Brisbane River needs to be undertaken. This study needs to produce best estimate maps of the bathymetry at different times during Brisbane's development. A current survey of the bathymetry also needs to be undertaken and the current morphological behaviour of the river needs to be understood,
- Astronomical tide need to be calculated for the flood events that occurred prior to the regular recording of tides,
- Where sufficient tidal and meteorological information is available the storm surge component at the river mouth needs to be estimated for each historical event,

- The methodology that has been developed under Research Project 18 of Australian Rainfall and Runoff for the calculation of the joint probability of river flooding and elevated ocean levels, should be applied to the lower reaches of Brisbane River so that flood risk can be properly quantified, and
- The sensitivity of flood levels to elevated ocean levels from climate change needs to determined.

Following the completion of the above tasks a revised flood frequency analysis should be carried out using the current best practice. This analysis should explore the use of a regional flood frequency approach.

9. REFERENCES

1. Chow, V. T.

   Open Channel Hydraulics
   McGraw Hill Book Company Inc., 1959.

2. Bureau of Meteorology

   Brisbane Floods of January 1974
   1974.

3. Snowy Mountains Engineering Corporation

   Brisbane River Flood Investigations
   Brisbane City Council, November 1975.

4. Grigg, T,G..

   A Comprehensive Evaluation of the Proposed Wivenhoe Dam on the Brisbane River

   Co-ordinator General's Department , June 1977.

5. McLeod, G. R. C.

   A Short History of the Dredging of the Brisbane River, 1860-1910

   University of Queensland, 1977.

6. Weeks, W.D.

   Wivenhoe Dam – Report on Downstream Flooding

   Queensland Water Resources Commissions, 143005.PR/4, 1984

7. Pilgrim DH (Editor in Chief)

   Australian Rainfall and Runoff – A Guide to Flood Estimation

   Institution of Engineers, Australia, 1987.

8. Greer, M.

   Report on Flood Data for Queensland Catchments – including Design Flood Estimates

   Water Resources Commission, 1992

9. Department of Natural Resources

Brisbane River and Pine River Flood Study - Report 7a: Brisbane River Flood Hydrology Report on Runoff-Routing Model Calibration

September 1992.

10. Department of Natural Resources

Brisbane River and Pine River Flood Study - Report 8a: Design Flood Estimation for Somerset and Wivenhoe Dam – Main Report

March 1993.

11. Department of Natural Resources

Brisbane River and Pine River Flood Study - Report 8b: Probable Maximum Precipitation Estimates

March 1993.

12. Department of Natural Resources

Brisbane River and Pine River Flood Study - Report 8c: Flood Frequency Analysis Results

March 1993.

13. Department of Natural Resources

Brisbane River and Pine River Flood Study - Report 8d: Runoff-Routing Model Layouts

March 1993.

14. Department of Natural Resources

Brisbane River and Pine River Flood Study - Report 13: Brisbane River Flood Hydrology Report on Downstream Flooding

August 1993.

15. SKM

Brisbane River Flood Study

Brisbane City Council, June 1998.

16. Mein, R.

Brisbane River Flood Study Review of Hydrological Aspects

Monash University December 1998.

17. City Design

Brisbane River Flood Study

Brisbane City Council, June 1999.

18. City Design

Further investigations into the Brisbane River Flood Study

Brisbane City Council, December 1999.

19. Thompson Computing Pty. Ltd

Brief History of the Brisbane City in the 19th Century

2002.

20. Mein, R., Apelt, C., Macintosh, J., Weinmann, E.

Review of Brisbane River Flood Study

Independent Review Panel, 3 September 2003.

21. SKM

Brisbane River Flood Study - Further Investigation of Flood Frequency Analysis Incorporating Dam Operations and CRC-Forge Rainfall Estimates - Brisbane River

December 2003.

22. SKM

Recalibration of the MIKE11 Hydraulic Model and Determination of the 1 in 100 AEP Flood Levels

City Design Modelling Services, February 2004.

23. Crime and Misconduct Commission

Investigation - Brisbane River Flood Levels

March 2004.

24. New South Wales Government

Floodplain Development Manual

April 2005.

25. Bureau of Meteorology

Known Floods in the Brisbane and Bremer River Basin

9th December 2009

26. SEQ Water

Report on the Operation of Somerset Dam and Wivenhoe Dam

March 2011.

27. Joint Flood Taskforce

Independent Review: Joint Flood Taskforce Report

8 March 2011.

28 Kuczera. G. and Franks, S.

Australian Rainfall and Runoff – Book IV – Estimation of Peak Discharge (Draft)

Institution of Engineers, Australia, January 2006.

29 Kuczera, G,

Bayesian Flood Frequency Analysis Software (Version 4.50)

Department of Civil Surveying and Environmental Engineering, University of Newcastle, NSW, 2001

30 Erskine, W.D., and Warner, R.F.

Geomorphic Effects of Alternating Flood and Drought Dominated Regimes on a NSW Coastal River

Fluvial Geomorphology Australia, Academic Press, Sydney, p223-244, 1988

31 Kiem, A.S., Franks, S.W. and Kuczera, G.

Multi-decadal Variability of Flood Risk

Geophysical Research Letters, 30(2):1035, DOI:10.1029/2002GL015992

32 Wales, M., Cossins, G., Broughton, R.

The Early Floods of the Brisbane-Bremer River System, 1823-1867

Brisbane City Council, 1976

33 Hausler, G. and Porter, N.

Report on the Hydrology of Wivenhoe Dam

Queensland Irrigation and Water Supply Commission, September 1977

34 Hegerty, K.L. and Weeks, W.D.

Hydrology Report for Manual of Operational Procedures for Flood Mitigation for Wivenhoe and Somerset Dam

Brisbane City Council and Queensland Water Resources Commission, 1985

35. SKM

Joint Calibration of a Hydrologic and Hydrodynamic Model of the Lower Brisbane River –Technical Report

Seqwater, 24 June 2011

36. SKM

January 2011 Flood Event: Report on the Operation of Somerset and Wivenhoe Dam

SKM, 11 March 2011

37. BoM

Water Resoucres Station Catalogue

http://www.bom.gov.au/hydro/wrsc, Accessed: September 2011

38. WMAwater

Review of Hydraulic Modelling – Queensland Floods Commission of Inquiry

July 2011

39. M.J O'Brien

Brisbane Flooding January 2011: An Avoidable Disaster – Submission in Response to Hydraulic Modelling Reports

31 August 2011

Appendix A

APPENDIX A: GLOSSARY

Taken from the Floodplain Development Manual (April 2005 edition)

acid sulfate soils	Are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.
Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
Average Recurrence Interval (ARI)	The long term average number of years between the occurrence of a flood as big as, or larger than, the selected event. For example, floods with a discharge as great as, or greater than, the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
caravan and moveable home parks	Caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the LG Act.
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
consent authority	The Council, government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of DIPNR, as having the function to determine an application.
development	Is defined in Part 4 of the Environmental Planning and Assessment Act (EP&A Act). **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development. **new development:** refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power.

	redevelopment: refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.
disaster plan (DISPLAN)	A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
ecologically sustainable development (ESD)	Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.
effective warning time	The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
emergency management	A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.
flash flooding	Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.
flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood awareness	Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.
flood education	Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.
flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.
flood liable land	Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).

flood mitigation standard	The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.
floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammetic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.
flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the "flood liable land" concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPL's are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the "standard flood event" in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain. **continuing flood risk:** the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.
flood storage areas	Those parts of the floodplain that are important for the temporary storage of

	floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.
freeboard	Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
habitable room	**in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom. **in an industrial or commercial situation:** an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.
hydraulics	Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.
hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
local drainage	Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
major drainage	Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves: • the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or • water depths generally in excess of 0.3 m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These

conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or

- major overland flow paths through developed areas outside of defined drainage reserves; and/or
- the potential to affect a number of buildings along the major flow path.

mathematical/computer models — The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.

merit approach — The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State's rivers and floodplains.

The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.

minor, moderate and major flooding — Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood:

minor flooding: causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded.

moderate flooding: low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered.

major flooding: appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.

modification measures — Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.

peak discharge — The maximum discharge occurring during a flood event.

Probable Maximum Flood (PMF) — The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.

Probable Maximum Precipitation (PMP)	The PMP is 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 the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to “water level”. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.

Appendix B

APPENDIX B: Port Office Adopted Annual Series

Table B 1: Port Office Adopted Annual Series

Year	Adopted Values in Current Flood Frequency Analysis (m^3/s)	Source
1841	12534	Section 6
1843	1940	SKM June 1999 report
1844	10410	Section 6
1845	8120	SKM June 1999 report
1852	2252	SKM June 1999 report
1857	2963	SKM June 1999 report
1863	3789	SKM June 1999 report
1864	4574	SKM June 1999 report
1870	3001	SKM June 1999 report
1873	2614	SKM June 1999 report
1875	2455	SKM June 1999 report
1879	2149	SKM June 1999 report
1887	4574	SKM June 1999 report
1889	4525	SKM June 1999 report
1890	8132	Section 6
1893	13690	Average of 5 estimates from SKM June 1999 report
1898	7528	Section 6
1908	6100	SKM June 1999 report
1927	3618	SKM June 1999 report
1928	4398	SKM June 1999 report
1929	3884	SKM June 1999 report
1931	7000	SKM June 1999 report
1955	6704	SKM June 1999 report
1956	4189	SKM June 1999 report
1967	2990	SKM June 1999 report
1968	4704	SKM June 1999 report
1971	2478	SKM June 1999 report
1974	11300	Section 6
1991	2387	SKM June 1999 report
1996	3087	SKM June 1999 report
2011	12400	Section 6