5.5 Model Calibration – 3 April 2013

5.5.1 Rainfall and Harbour Water Level Data

Figure 5-4 shows the recorded Harbour water levels at Fort Denison and rainfall depths recorded at Observatory Hill. The rainfall event commencing on 3 April 2013 was characterised by two distinct rainfall bursts with less intense intermediate rainfall summarised as follows:

- The first major burst commenced at 06:55 on 3 April with a total depth of 23.4mm falling in 1 hour;
- 30mm of rain fell over the ensuing 19 hours;
- The second major burst commenced at 03:00 on 4 April with a total depth of 22mm falling in 1 hour.

The total rainfall event occurred over 24 hours with the downstream tide levels varying from a low tide of -0.27m AHD to a high tide level of 0.93m AHD, as shown in Figure 5-4.

The recorded rainfall depths at the Observatory Hill rainfall gauge have been compared with the design IFD data, as shown in Figure 5-5. This indicates that the rainfall event was of a magnitude comparable with a 1 year ARI design rainfall event, corresponding to a short 30 minute burst period within the overall event.

5.5.2 Observed and Simulated Flood Behaviour

A single flooding incident was reported for this flood event, being the flooding that occurred at a car park on Mary Ann Street, Ultimo, shown in Figure 5-6. Also shown in this figure are the peak depths resulting from the modelling undertaken.

As with the 12 August 1983 calibration event, blockage assumptions were required in order to replicate observed flood behaviour. The precise entrance level of the car park is not known, though based on modelling undertaken the ponded depths at the low points adjacent to the car park appears to be high enough to cause flooding within the car park.

In lieu of more detailed observations for this event, the model is considered to be adequately representing the observed flooding behaviour.

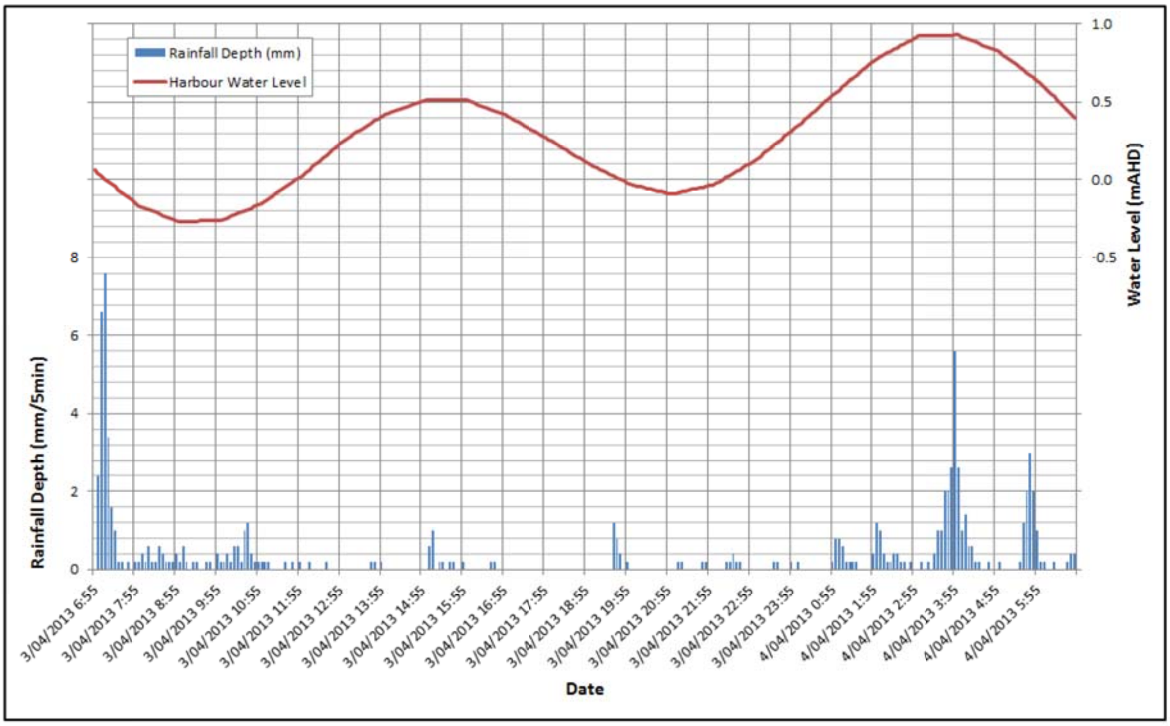


Figure 5-4 Recorded rainfall and harbour water level – 3 April 2013

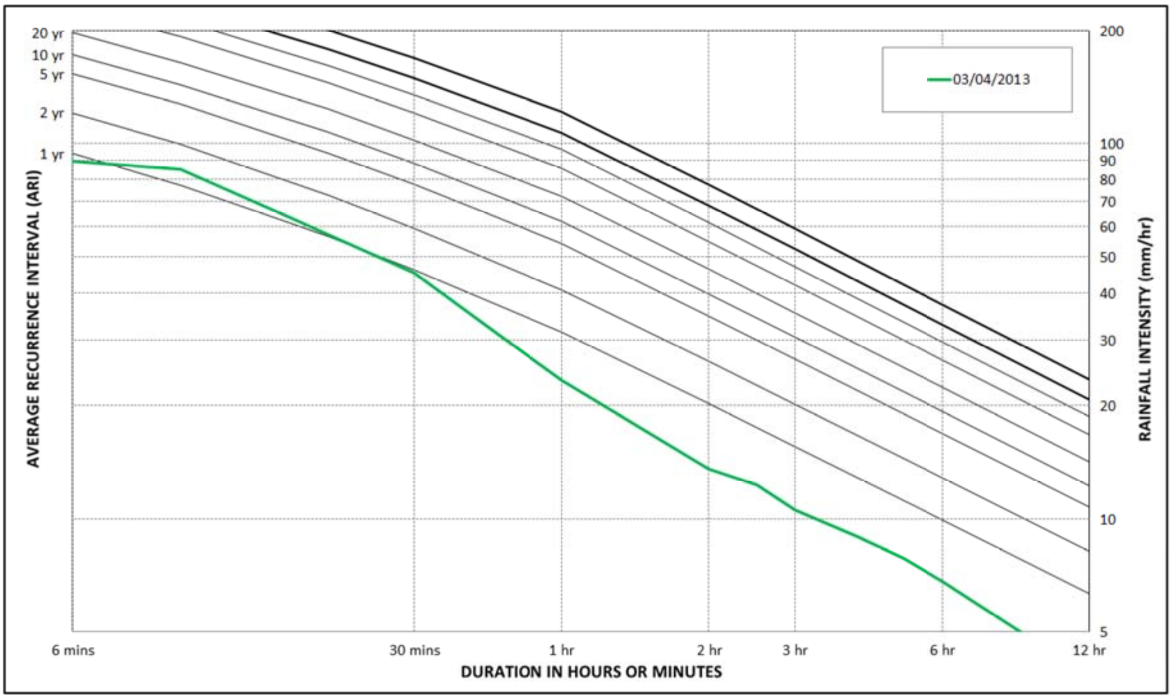
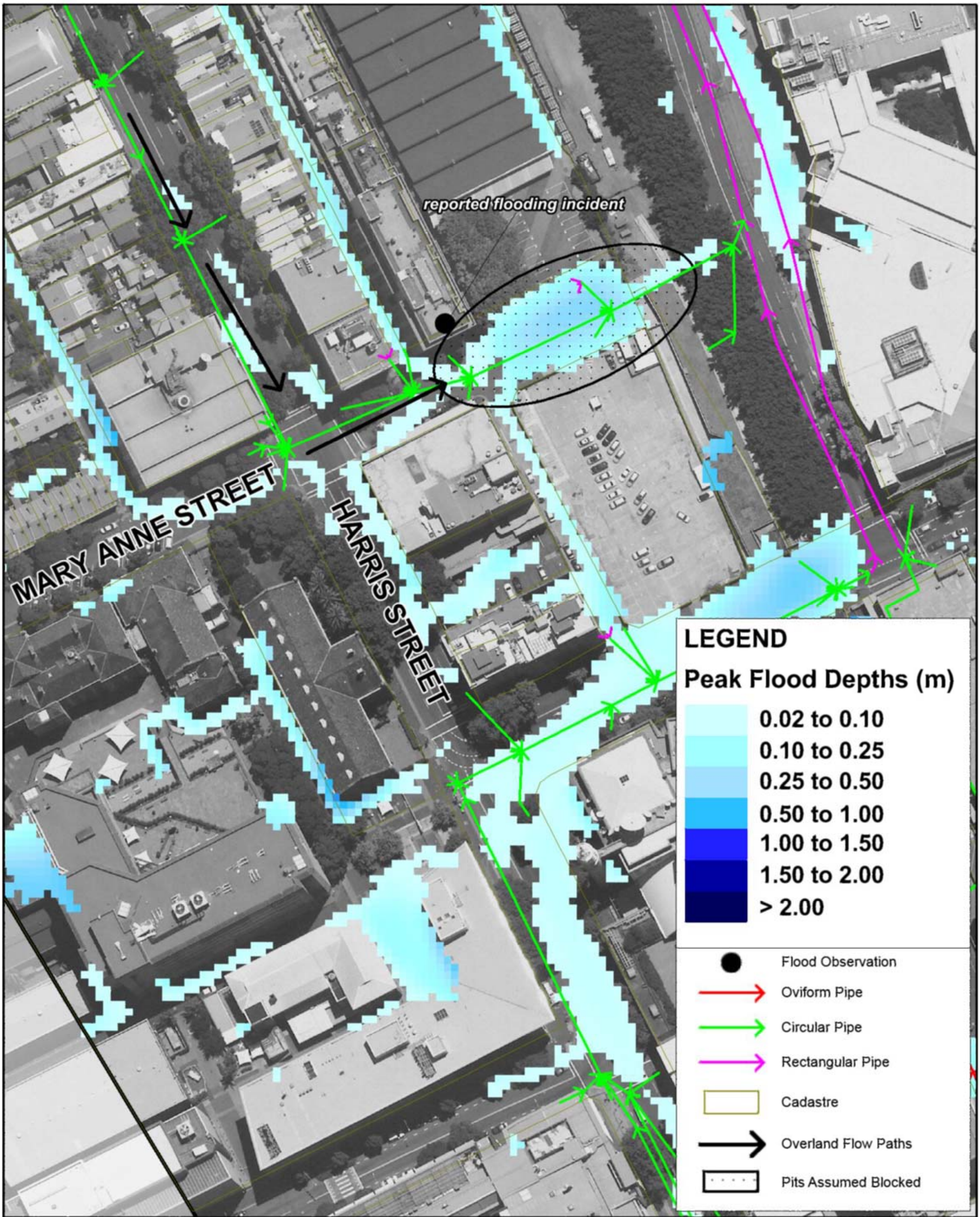


Figure 5-5 Comparison of 3 April 2013 rainfall with IFD relationships



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Model Calibration Results
3 April 2013

Figure: 5-6

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5.6 Model Calibration – 8 March 2012

5.6.1 Rainfall and Harbour Water Level Data

Figure 5-7 shows the recorded Harbour water levels at Fort Denison and rainfall depths recorded at Observatory Hill. A total rainfall depth of approximately 74mm fell over an 8 hour period with the rainfall event generally coinciding with a high tide level of 1.11m AHD.

The recorded rainfall depths at the Observatory Hill rainfall gauge have been compared with the design IFD data, as shown in Figure 5-8. This indicates that the rainfall event was of a magnitude comparable with a 2 year ARI design rainfall event for durations between 30 minutes and 6 hours.

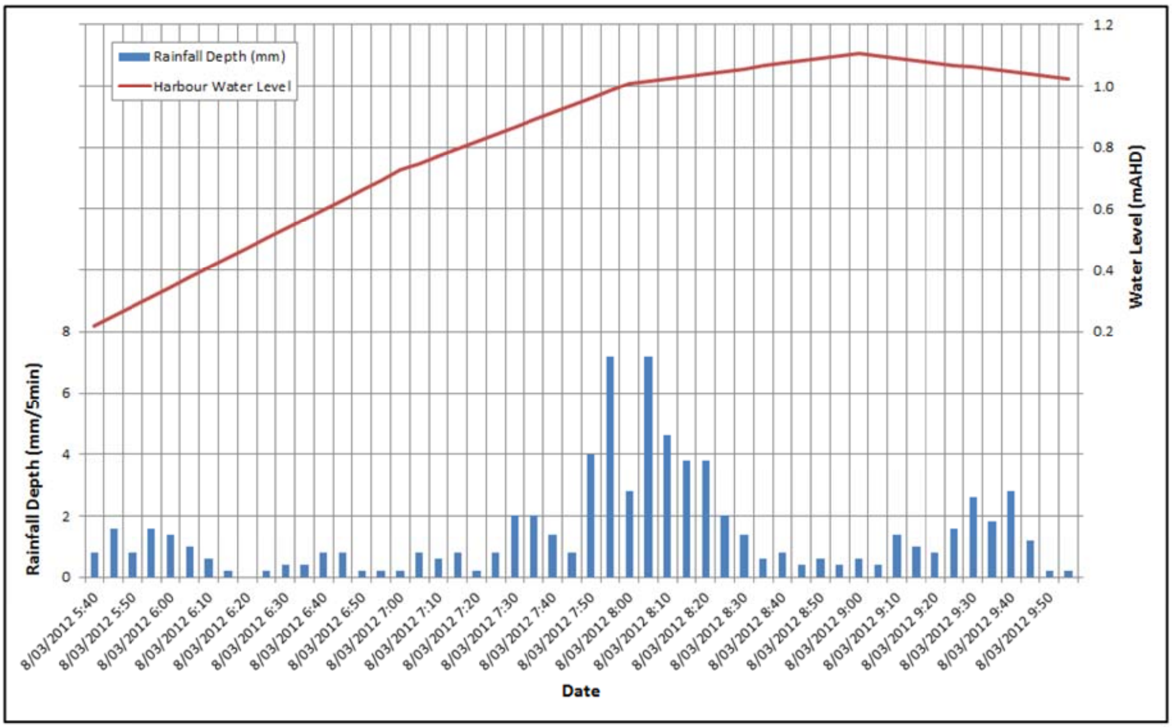


Figure 5-7 Recorded rainfall and harbour water level – 8 March 2012

5.6.2 Observed and Simulated Flood Behaviour

Results of modelling at the key locations reported by the community consultation respondents are discussed in the following sections.

5.6.2.1 Pitt Street (near Wilmot Street), Sydney

Resident reported repeated flooding of car park and expressed concerns about safety.

Figure 5-9 shows that Pitt Street is an overland flow path at this location and, depending on entrance level to the car park, may result in flooding from street level.

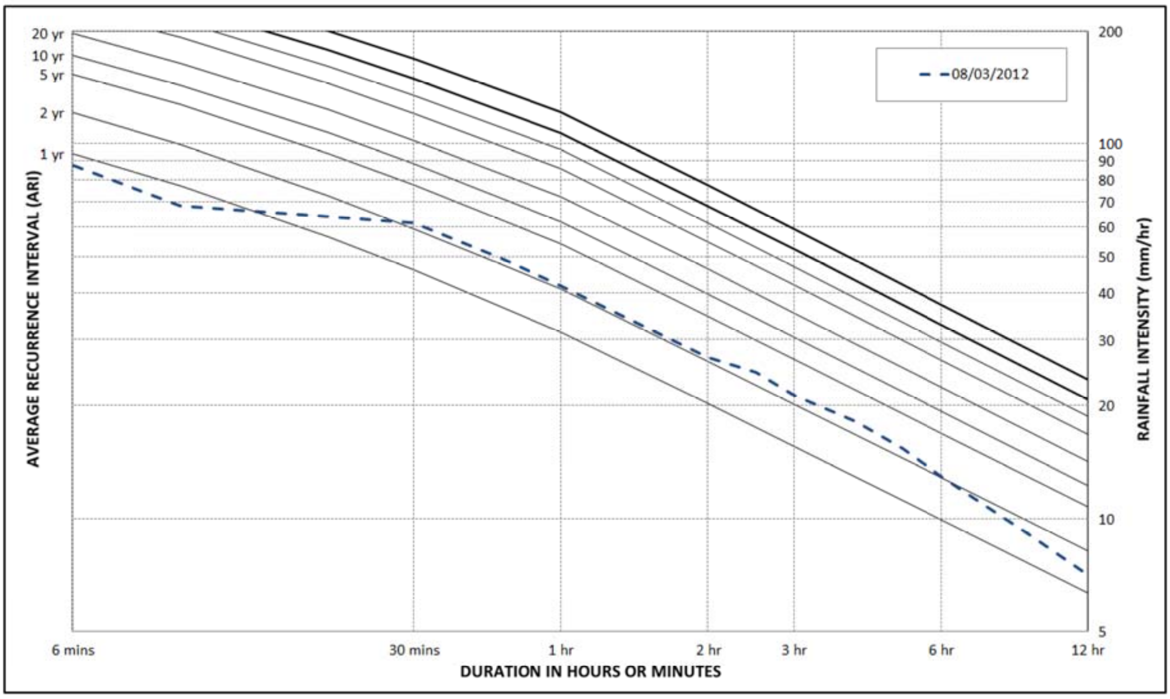


Figure 5-8 Comparison of 8 March 2012 rainfall with IFD relationships

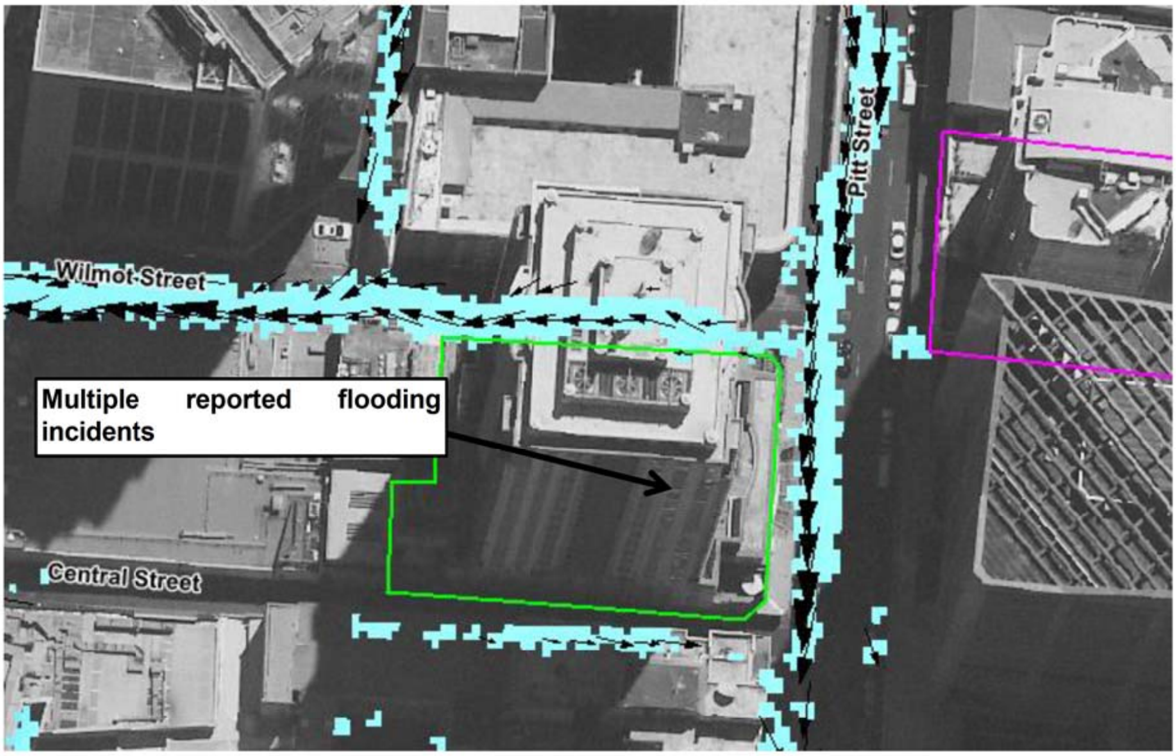


Figure 5-9 Flooding reports at Century Tower

5.6.2.2 Corner of Goulburn Street and Wentworth Street, Sydney

At the corner of Goulburn Street and Wentworth Street, it was reported that flood water exceeded the height of the gutter, flowed onto footpath and passed through front entry doors of a commercial building. The respondent did not remember the date on which this event occurred.

Figure 5-10 shows that this location is at the confluence of overland flow paths from Alberta Street, Commonwealth Street and Wentworth Street. The reported flow behaviour is supported by modelling which demonstrates this location is part of an overland flow path.

Figure 5-10 Flooding reports at corner of Goulburn and Wentworth Streets

5.6.2.3 Commonwealth Street, Surry Hills (flooding from Batman Lane)

A long term resident (10 years) reported flooding of their garage at the rear of the property which possibly occurred in 2005. The respondent also reported blockage of the drainage system inlet from rubbish left in Batman Lane. This reiterates the requirement for some degree of blockage to be incorporated into the design modelling.

Figure 5-11 shows the location of reported flooding. The figure does not show enough water to contribute to flooding of the garage, though a flow path (albeit shallow) along Batman Lane is shown. Flooding from a larger event may cause minor flooding at this location. Since the location is so high in the catchment, extensive flooding is unlikely without compounding influences such as blockage of drainage paths (lane way gutters etc.).

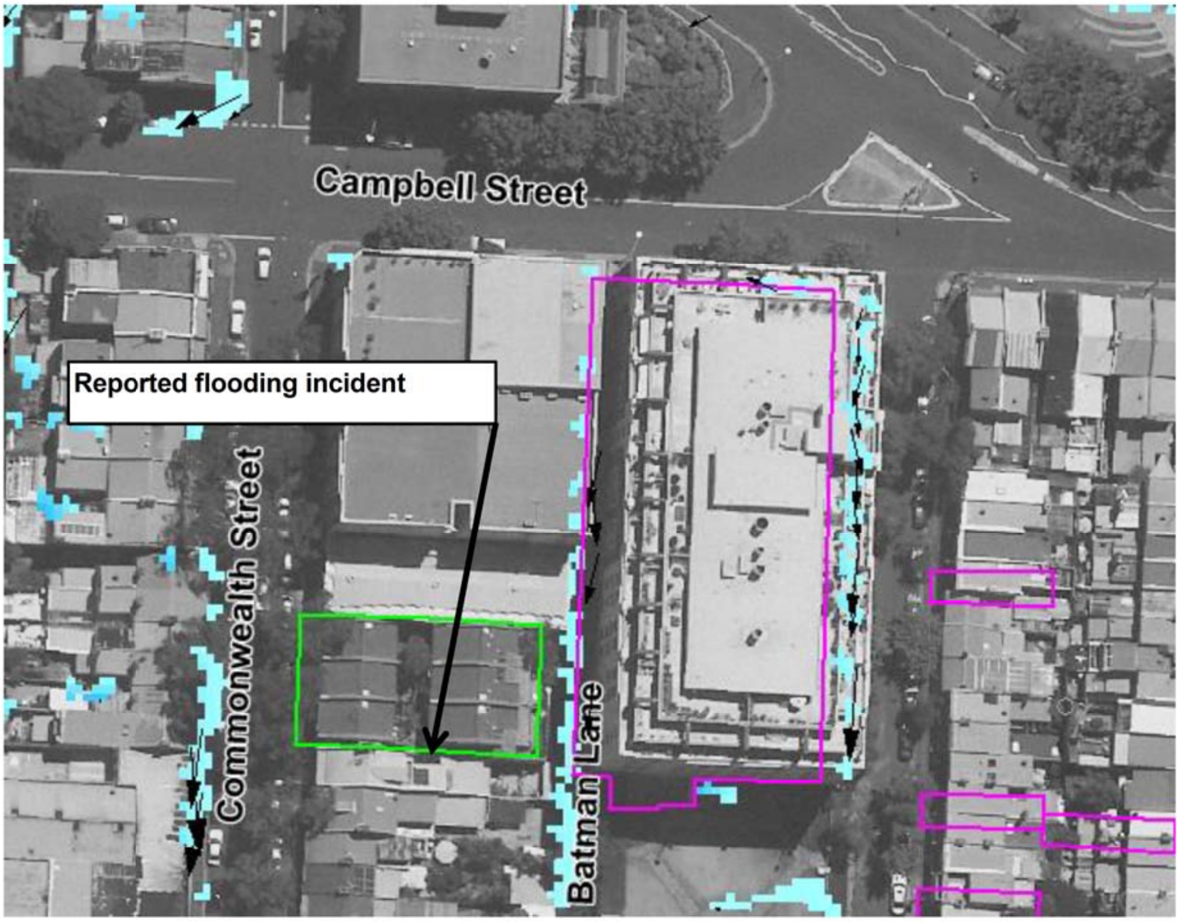


Figure 5-11 Flooding reports at Commonwealth Street from Batman Lane

5.6.2.4 Crown Street Public School, Surry Hills

Two separate reports indicate water flows onto Crown Street via Crown Street Public School in Surry Hills. A resident reported that flood waters have drained into their cellar in 2009, 2010, 2011 and 2012.

Figure 5-12 shows the modelling results indicating that Crown Street is functioning as an overland flow path and a flow path draining from the school is further observed, thus replicating the reported flood behaviour.

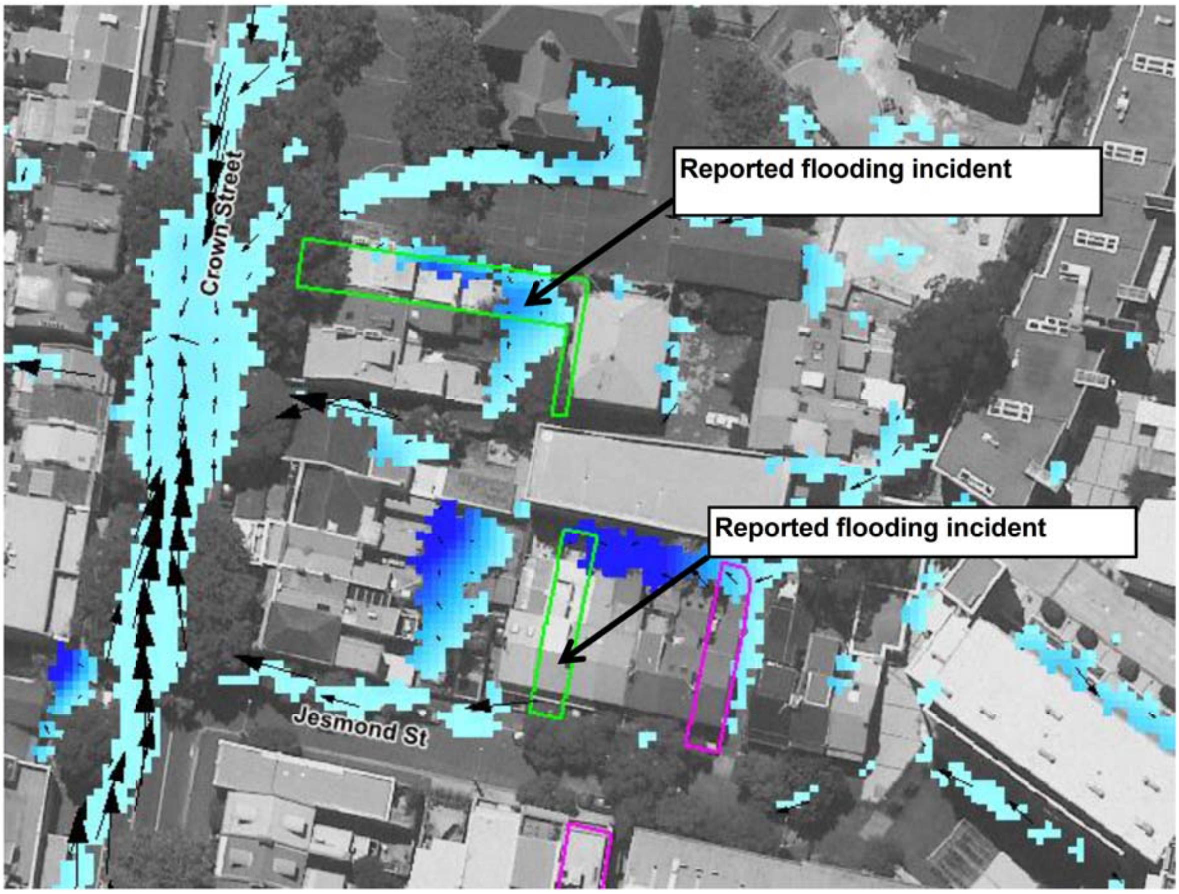


Figure 5-12 Flooding reports near Crown street school

5.7 Historical Accounts of Flooding from TROVE database review

Section 2.4 presents results of a review of newspaper articles for further insight into key historic flood events and flood behaviour within the Darling Harbour catchment. The database details were restricted to flooding events prior to approximately 1950. Catchment conditions, including stormwater drainage infrastructure and extent of development, are likely to be significantly different now compared with conditions at the time of these historical records which limits the validity of using these details for model calibration. However, these historic details can be useful to verify that flooding occurs in the reported locations, thus validating the modelling tool developed for this study. A comparison of the reported flood mechanisms has been made with modelled conditions of the 8 March 2012 event.

In the Darling Harbour catchment the key accounts of flooding are as follows:

- Main flow path identified near Central Railway Station from Surry Hills (June 1949);
- Haymarket – 4 feet deep (March 1912);
- Ultimo Road – 5 feet deep (June 1949); and
- Darling Harbour flooding has been reported to be exacerbated by a high tide coinciding with a local rainfall event (April 1905).

Figure 5-13 and Figure 5-14 show the modelled peak flood depths near Central Railway station and the Haymarket region, respectively.

Eddy Avenue is shown to function as an overland flow path conveying upstream flows from Foveaux Street and Elizabeth Street. The modelled flow is shallow with a peak depth of only 0.15 m. The flow however is relatively quick moving having a peak velocity of 1.3 m/s. This flow type supports the fast flowing characteristics described in the historical accounts; however, the depth of water predicted in the model is less than reported.

The June 1949 flood event was an approximately 5 year ARI (20% AEP) event compared to the modelled event which was a 2 year ARI event providing some explanation for the difference in flow magnitude. Upstream catchments conditions would also have been different. It is not known if the dedicated bus lane which is conveying the water in Figure 5-13 existed in 1949.

Modelled flooding in the Haymarket region and at Ultimo Road is shown to generally agree with the behaviour observed. In 1912 4 feet (1.2 m) of water was reported in Haymarket and 5 feet (1.5 m) of water was reported on Ultimo road. Albeit at a different scale, these two flood indicators are both replicated in the TUFLOW verification modelling. There are numerous reasons why the modelled flood depths are different to observed conditions including the fact the verification event was a 2 year ARI event instead of a 5 year ARI event for 1949. Event pit blockages and the pit/pipe configuration in 1949 are unknown and would also influence water depths.

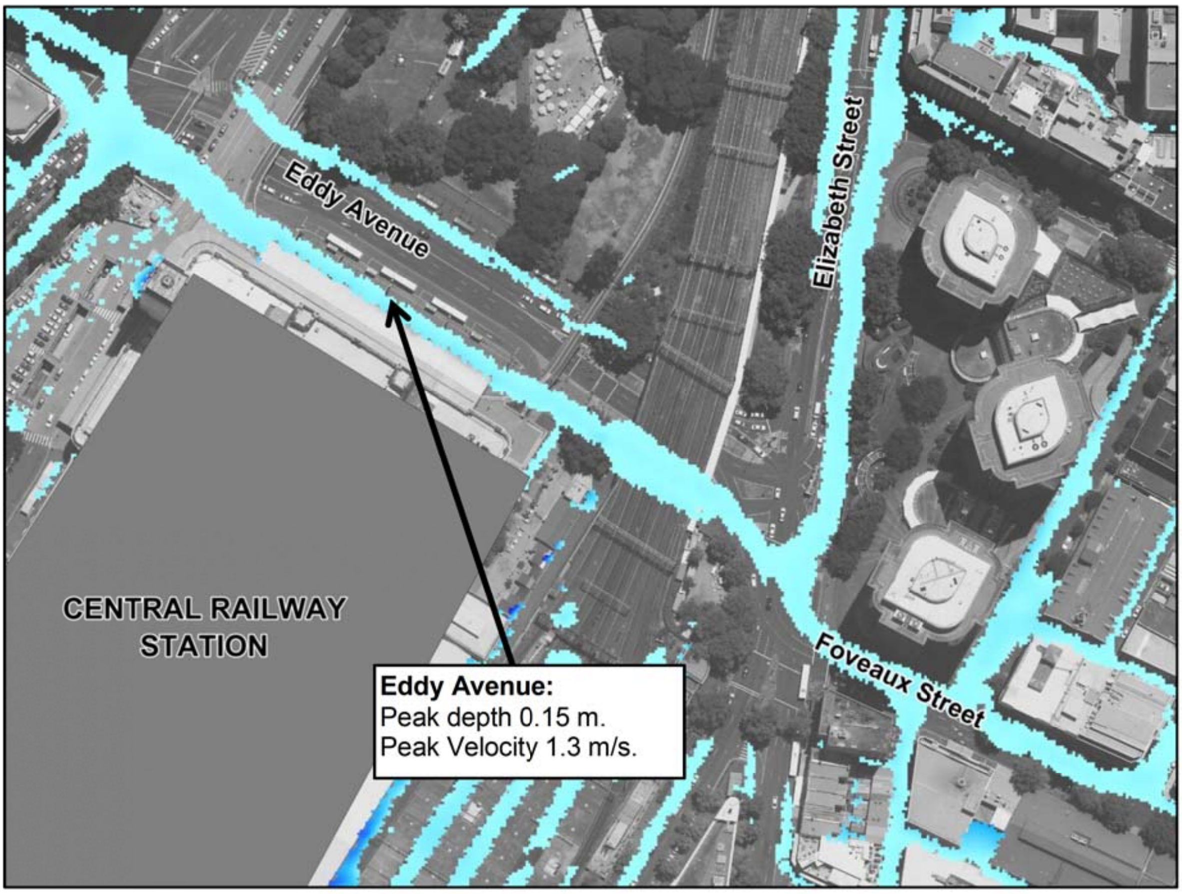


Figure 5-13 Peak flood depths at Eddy Avenue near Central Railway Station

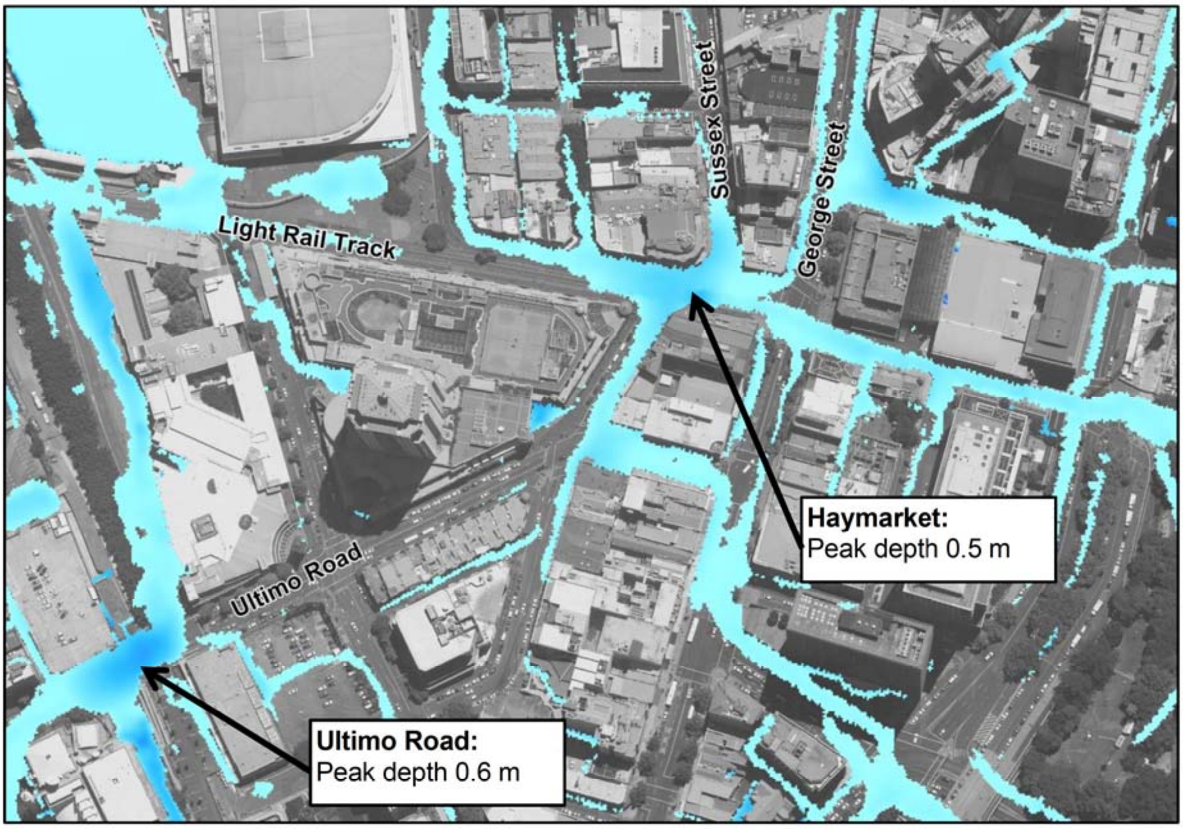


Figure 5-14 Peak flood depths at Haymarket and Ultimo Road

Due to the anecdotal nature of the newspaper flood reports and the fact that the reported flood events occurred over 60 years ago, these flood observations could not be strictly used as a calibration data set. Replication of the general flow behaviour however has proven valuable in validating the model schematisation.

5.8 Historical Accounts of Flooding from SWC Records

As presented in Section 2.3, SWC has an extensive database of historic flood reports. Reports of flooding prior to 1983 were not considered as calibration events since the catchment conditions which resulted in the flooding are unknown. However, these historic details can be useful to verify that flooding occurs in the reported locations, thus validating the modelling tool developed for this study. A comparison of the reported flood mechanisms has been made against those modelled by the 8 March 2012 event.

Figure 5-15 and Figure 5-16 shows the SWC flooding reports which weren't included as part of the model calibration. It is noted that the location of these flooding reports are approximate, since the address references often refer to buildings which no longer exist.

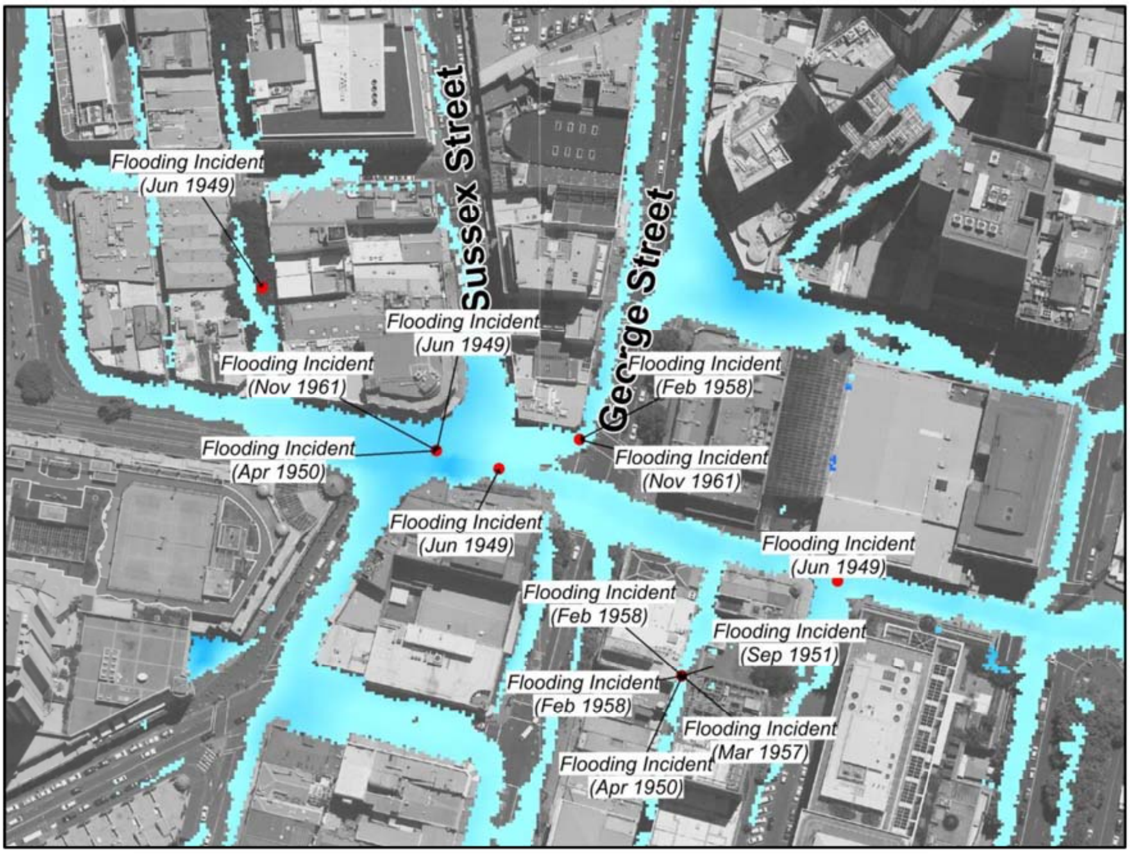


Figure 5-15 SWC historic flooding reports in Haymarket area

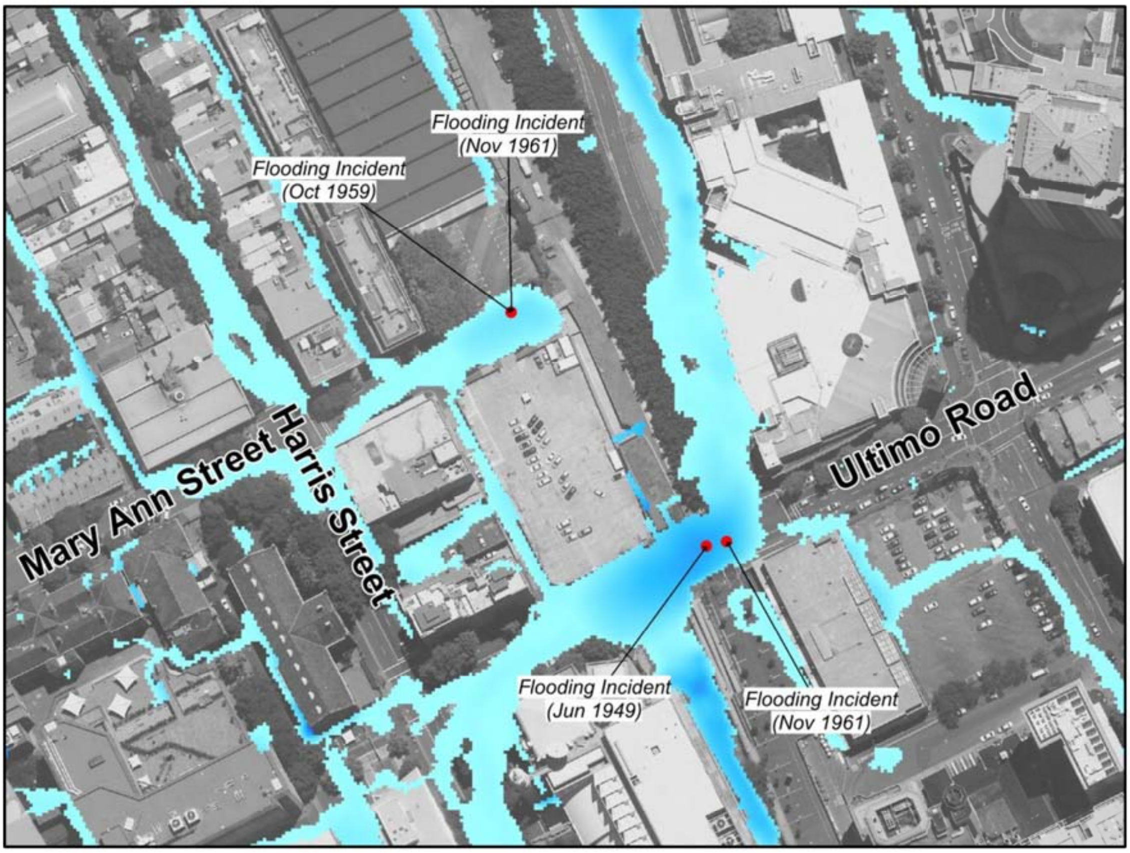


Figure 5-16 SWC historic flooding reports in Ultimo area

As presented in Figure 5-15 and Figure 5-16, the historic reports of flooding consistently affect the same locations. Further, current catchment flood modelling shows flooding still occurs at the historic locations.

Parker Lane had reported flooding in April 1950, September 1951, March 1957 and February 1958. Flooding at this location resulted in garage and basement flooding, flooding of a public convenience and footpath flooding.

The intersection of Sussex, George, Hay and Thomas Streets has reported flooding in June 1949, April 1950, February 1958 and November 1961. Flooding at this location resulted in hotel cellar flooding, ground floor shop flooding and road flooding.

Flooding on Dixon Street resulted in hotel cellar flooding and on Campbell Street flood waters rose to above the level of the building line.

Ultimo Road had reported flooding in June 1949 and November 1961 with road flooding threatening buildings. The tram depot near Omnibus Lane and Mary Ann Street reported flooding in October 1959 and November 1961.

At all locations, modelled flooding is shown to provide a reasonable representation of the observed behaviour.

5.9 Catchment Flow Verification

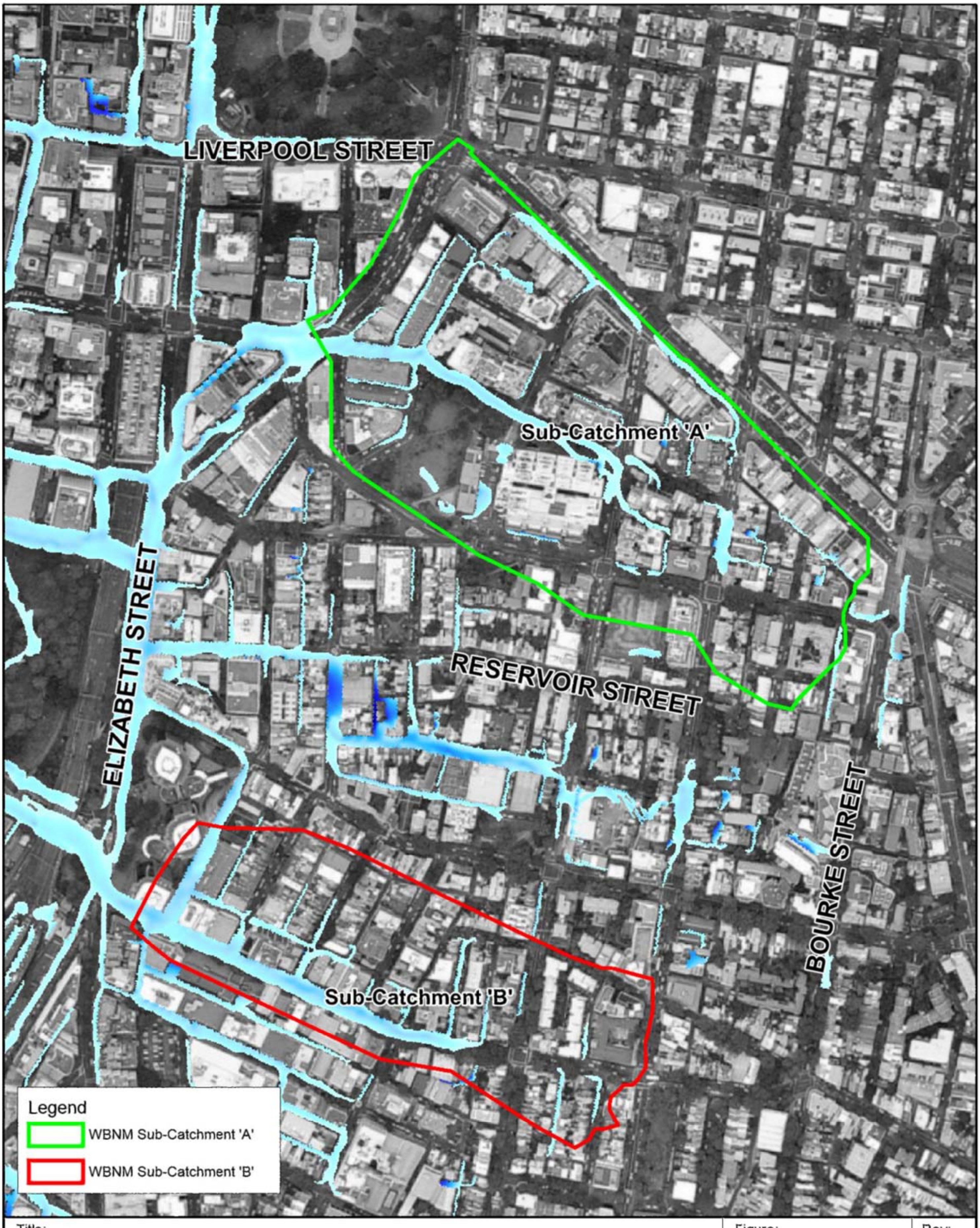
Verification of the adopted “direct-rainfall” approach for modelling the catchment hydrology has been achieved by undertaking additional hydrological modelling of selected sub-catchments within the overall study area using alternate modelling methods.

The verification approach involved setting up a WBNM model for two separate sub-catchments, as shown in Figure 5-17.

5.9.1 Watershed Bound Network Model (WBNM)

WBNM is a runoff-routing hydrological model used to represent catchment rainfall-runoff relationships. WBNM has been developed and tested using Australian catchments in the states of NSW, Queensland, Victoria and South Australia. WBNM models are developed on the basis of a catchment divided into a number of sub-areas based on the stream network. This allows hydrographs to be calculated at various points within the catchment, and the spatial variability of rainfall and rainfall losses to be modelled. WBNM separates overland flow routing from channel routing, allowing changes to either or both of these processes, for example in urbanising catchments.

WBNM uses a Lag Parameter (also referred to as the C value) to calculate the catchment response time for runoff. The Lag Parameter is important in determining the timing of runoff from a catchment, and therefore the shape of the hydrograph. The general relationship is that a decrease in lag time results in an increase in flood peak discharges (Boyd et al., 2007).



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5.9.2 Flow Verification Results

The WBNM model has been schematised using recommended parameters to represent the subject sub-catchments.

Modelling using both WBNM and the TUFLOW model developed for this study has been undertaken for the following design rainfall events:

- 10% AEP, 90 minute duration storm; and
- 1% AEP, 90 minute duration storm.

Comparisons between the calculated catchment discharge and the cumulative volume are given in Figure 5-18 for sub-catchment 'A' and Figure 5-19 for sub-catchment 'B'. The figures show that for both catchments and for both design storms modelled, the flow generated by TUFLOW correlates well with the WBNM estimates. The following observations can be made:

- The timing of the rising limbs of the hydrographs compare favourably;
- The timing of the peaks and troughs in the hydrographs shape compare favourably;
- TUFLOW produces a slightly more 'peaky' catchment response with marginally higher peak flows; and
- WBNM produces a higher cumulative volume of runoff.

WBNM has been verified against empirical data and can therefore be relied upon to provide a reasonable estimate of the expected runoff for these sub-catchments. However, WBNM is a lumped catchment approach and does not represent all the physical features within the catchment which are being modelled in the TUFLOW model (e.g. steep, paved overland flow paths), which may explain some of the differences in the calculated hydrograph shapes.

The differences in cumulative volume can be attributed to the residual volume of water in the TUFLOW model (water trapped in storage) throughout the simulation. Once this residual volume has been taken into account, the difference between the total volumes calculated between the two methods is less than 2%.

The good correlation demonstrated between the two modelling methods indicates that the modelling methodology adopted for the Darling Harbour Flood Study provides a reasonable basis to assess overall flood behaviour.

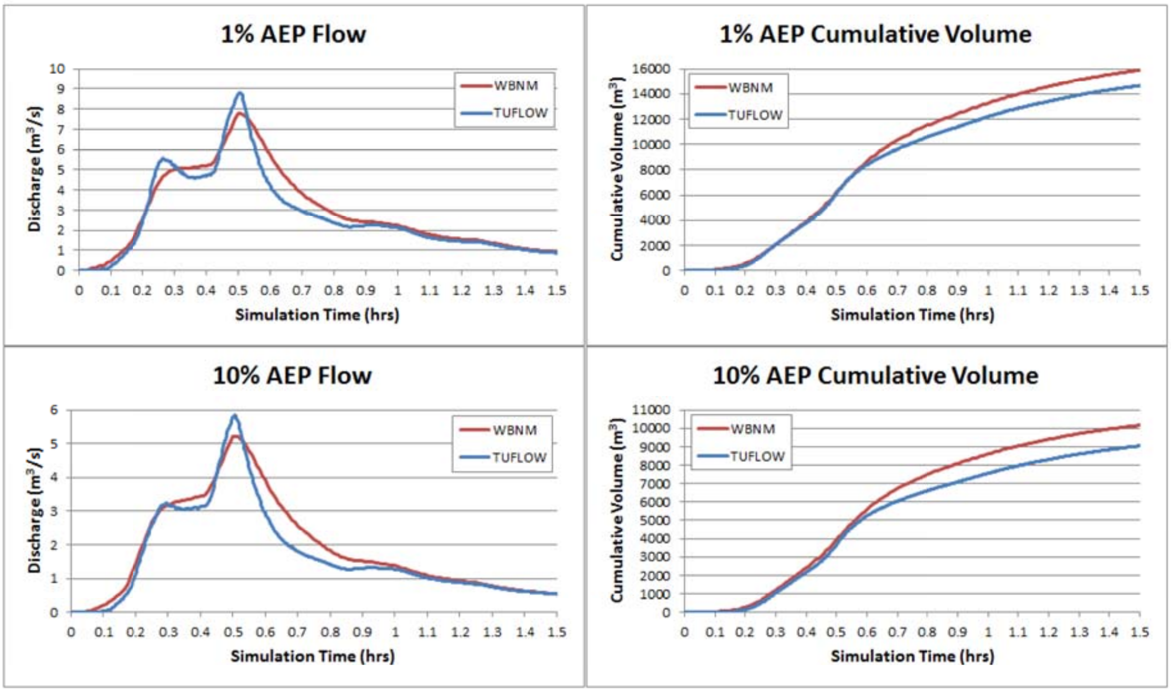


Figure 5-18 Catchment Flow Verification for Sub-Catchment 'A' (15ha area)

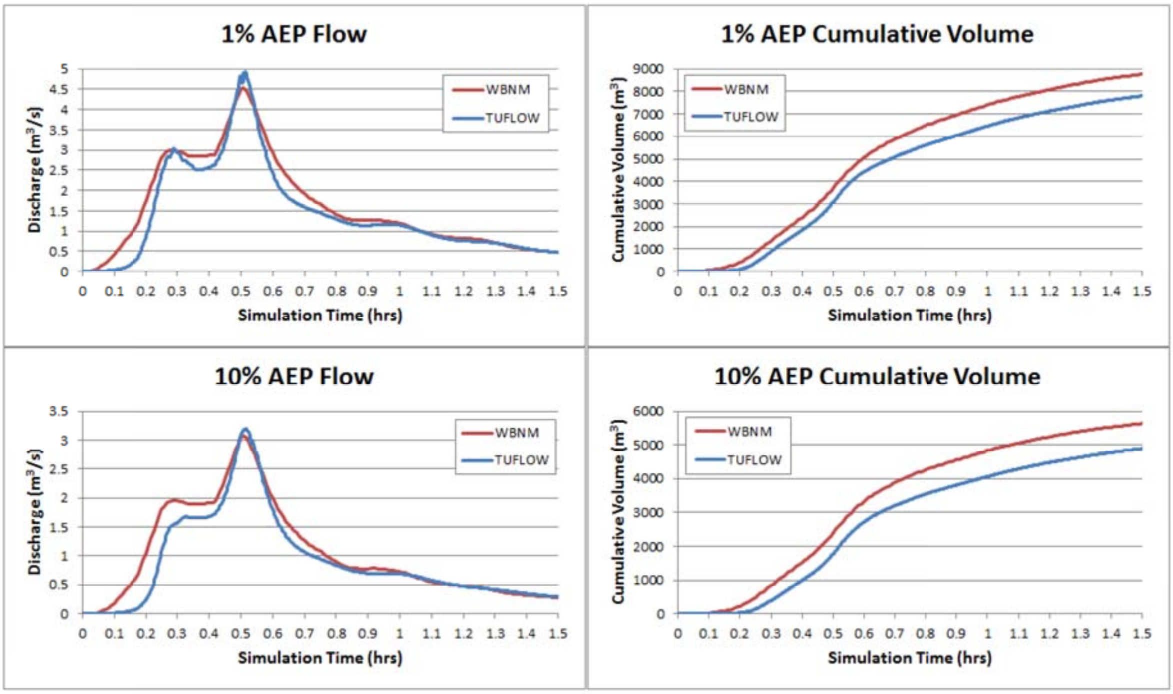


Figure 5-19 Catchment flow verification for sub-catchment 'B' (8.2ha area)

5.10 Model Parameters Adopted for Design Event Modelling

The values for the Manning's '*n*' roughness and rainfall infiltration losses developed for the defined land use categories (refer to Figure 2-4) determined through the model calibration and validation process and adopted for design event modelling are shown in Table 5-2.

Table 5-2 Adopted TUFLOW model parameters

Land Use Category	Manning's '*n*'	Fraction Impervious	Initial Loss (mm)	Pervious Area Infiltration Loss (mm/h)
Roads	0.02	100%	1.0	0.0
Buildings	N/A	100%	1.0	0.0
Public Recreation	0.05	10%	10.0	2.5
Metro Centre	0.04	90%	1.0	2.5
Rail Corridor	0.04	10%	1.0	2.5
General Residential	0.04	90%	1.0	2.5
Mixed Use	0.04	90%	1.0	2.5
Commercial Core	0.04	90%	1.0	2.5
Darling Harbour	0.03	90%	1.0	2.5

5.11 Summary of Model Calibration

Every effort has been made to fully utilise the limited historic accounts of flooding. In the absence of surveyed flood level records, anecdotal accounts of flood behaviour have been sourced from Sydney Water records and community consultation undertaken for this study. For all verification events, the model has demonstrated an ability to reasonably simulate observed flood behaviour as described by anecdotal reports.

To strengthen the verification process, historical accounts of flooding (some of which occurred over 60 years ago) have also been obtained. The general flood mechanisms described are well represented by the model.

Flows from TUFLOW have been compared to flows generated by WBNM. WBNM is a hydrological model which uses empirical relationships determined from Australian catchments. The peak flows and volume match well with the WBNM estimates.

Fully utilising the available information available, the developed model is demonstrated to be a suitable tool for design flood estimation.

6 DESIGN FLOOD CONDITIONS

Design floods are estimated floods used for planning and floodplain management investigations. They are based on having a probability of occurrence specified as either:

- Annual Exceedance Probability (AEP) expressed as a percentage; or
- Average Recurrence Interval (ARI) expressed in years.

Refer to Table 6-1 for a definition of AEP and the ARI equivalent.

Table 6-1 Design flood terminology

ARI[1]	AEP[2]	Comments
500 years	0.2%	An estimated flood or combination of floods which represent the worst case scenario with a 0.2% probability of occurring in any given year.
100 years	1%	As for the 0.2% AEP flood but with a 1% probability.
50 years	2%	As for the 0.2% AEP flood but with a 2% probability.
20 years	5%	As for the 0.2% AEP flood but with a 5% probability.
10 years	10%	As for the 0.2% AEP flood but with a 10% probability.
5 years	18%	As for the 0.2% AEP flood but with a 18% probability.
2 years	39%	As for the 0.2% AEP flood but with a 39% probability.
PMF[3]		An estimated flood or combination of floods which represents the Probable Maximum Flood event possible.

1 Average Recurrence Interval (years)

2 Annual Exceedance Probability (%)

3 Probable Maximum Flood

The design events simulated include the PMF event, 0.2%, 1%, 2%, 5%, 10%, 18% and 39% AEP events for catchment derived flooding and the 1 year ARI Sydney Harbour water level for ocean/tidal derived flooding. The 1% AEP flood is generally used as a reference flood for land use planning and control.

In determining the design floods it is necessary to take into account the critical storm duration of the catchment. Small catchments are more prone to flooding during short duration storms while for large catchments longer durations will be critical. For example, considering the relatively small size of the study area catchments, they are potentially prone to higher flooding from intense storms extending over a few hours rather than a couple of days.

6.1 Design Rainfall

Design rainfall parameters have been derived using standard procedures defined in *Australian Rainfall and Runoff – A Guide to Flood Estimation* (AR&R) (Pilgrim, DH, 2001) which are based on statistical analysis of recorded rainfall data across Australia. The derivation of location specific design rainfall parameters (e.g. rainfall depth and temporal pattern) for the Darling Harbour catchment is presented herein.

6.1.1 Rainfall Depths

Design rainfall depth is based on the generation of intensity-frequency-duration (IFD) design rainfall curves utilising the procedures outlined in AR&R (Pilgrim, DH, 2001). These curves provide rainfall depths for various design magnitudes for durations from 5 minutes to 72 hours.

The Probable Maximum Precipitation (PMP) is used in deriving the Probable Maximum Flood (PMF) event. The theoretical definition of the PMP is "the greatest depth of precipitation for a given duration that is physically possible over a given storm area at a particular geographical location at a certain time of year" (Pilgrim, DH, 2001). The ARI of a PMP/PMF event ranges between 10^4 and 10^7 years. The PMP has been estimated using the Generalised Short Duration Method (GSDM) derived by the Bureau of Meteorology. The method is appropriate for durations up to 6 hours and considered suitable for small catchments in the Sydney region.

A range of storm durations from 15 minutes to 9 hours were modelled in order to identify the critical storm duration for design event flooding in the catchment. Table 6-2 shows the average design rainfall intensities based on AR&R adopted for the modelled events.

Table 6-2 Rainfall intensities for design events (mm/h)

Duration	2 YR ARI	5 YR ARI	10% AEP	5% AEP	2% AEP	1% AEP	0.2% AEP	PMP
15 min	83	108	122	140	164	182	222	640
25 min	66	85	97	112	132	148	180	n/a
30 min	60	78	89	103	122	136	166	460
45 min	48	63	72	84	99	111	136	400
1.00 h	41	53	61	71	84	95	116	340
1.50 h	32	42	48	56	66	74	91	293
2.0 h	26	35	40	46	55	62	76	260
2.5 h	23	30	35	40	48	53	n/a	228
3.0 h	20	27	31	36	42	47	58	210
4.0 h	n/a	n/a	n/a	n/a	n/a	n/a	n/a	180
4.5 h	16	20	23	27	32	36	44	n/a
5.0 h	n/a	n/a	n/a	n/a	n/a	n/a	n/a	158
6.0 h	13	17	19	22	26	30	36	138
9.0 h	10	13	15	17	20	23	28	n/a

The areal reduction factor takes into account the unlikelihood that larger catchments will experience rainfall of the same design intensity over the entire area. Due to the relatively small size of the catchment and adopting a conservative approach, an aerial reduction factor was not applied in this study.

6.1.2 Temporal Patterns

The IFD data presented in Table 6-2 provides for the average intensity that occurs over a given storm duration. Temporal patterns are required to define what percentage of the total rainfall depth occurs over a given time interval throughout the storm duration.

For frequent, large and rare design flood events including the 20% to 0.5% AEP events, design temporal rainfall patterns from AR&R (Pilgrim, DH, 2001) for temporal zone 1 have been adopted. For the PMF event, the temporal pattern as provided in BOM (2003) was used.

The same temporal pattern has been applied across the whole catchment. This assumes that the design rainfall occurs simultaneously across each of the modelled sub-catchments. The direction of a storm and relative timing of rainfall across the catchment may be determined for historical events if sufficient data exists, however, from a design perspective the same pattern across the catchment is generally adopted.

6.1.3 Rainfall Losses

The rainfall losses utilised in calibration modelling (refer to Section 5.10) have been adopted for all design event modelling, excluding the PMF event, with the adopted values shown in Table 5-2. The PMF event modelling has adopted losses are as per AR&R recommendations (Pilgrim, DH, 2001) with an initial loss of 0mm and a continuing loss of 1mm/h.

The applied losses are varied across the hydraulic model extent based on the land use surface type as illustrated in Figure 4-3. As outlined in Section 4, the land use surface types were identified based on aerial photography and GIS data supplied by Council.

6.1.4 Critical Storm Duration

A series of model runs were carried out in order to identify the critical storm duration for the Darling Harbour catchment. Standard durations from the 15-minute to the 9-hour events were simulated utilising the design temporal patterns from AR&R (Pilgrim, DH, 2001).

No single critical storm duration was found for the study area, but rather, the critical duration varies across the catchment. Some regions of the catchment are affected more by the total volume produced in a given rainfall event, particularly in trapped low points. The variation in critical storm duration is discussed further in Section 7.1.2.

6.2 Design Ocean Boundary

The 2010 NSW Government document entitled “Flood Risk Management Guide – incorporating sea level rise benchmarks in flood risk assessments” recommends that the local catchment 1% AEP flood should be run in conjunction with a 5% ARI tailwater condition. It further recommends that the inverse scenario be run to confirm that the dominant flooding mechanism is not from downstream water levels. If the flooding from the downstream water is demonstrated to produce peak flood conditions in parts of the catchment, an envelope of both scenarios must be used to define the extent of the 1% AEP flood.

Modelling undertaken has confirmed that in all Darling Harbour catchment locations the 1% AEP local catchment flood with a 5% AEP tailwater generates higher flood levels than the 5% AEP flood with a 1% AEP tailwater. Because the local catchment flood dominates the tailwater flood, an envelope does not need to be developed when producing design flood results.

The 2008 NSW Government document entitled “Fort Denison: Sea Level Rise Vulnerability Study” presents the Sydney Harbour design still water levels, which are shown in Table 2-3. There is little variation in harbour water levels for different frequencies, specifically, the 1% AEP harbour water level is only 0.06 m higher than the 5% AEP flood level. This also explains why the 1% AEP local catchment flood with a 5% AEP tailwater is always dominant for the subject catchment.

The 2010 NSW Government document entitled “Flood Risk Management Guide – incorporating sea level rise benchmarks in flood risk assessments” does not give guidance for the combination of annual exceedance probabilities of the local catchment flood and tailwater conditions for design events other than the 1% AEP flood.

Based on other NSW flood studies, the proposed combination of local catchment floods with tailwater scenarios is presented in Table 6-3.

Due to the small variations in Sydney Harbour water levels for differing frequencies, the inverse combinations are not required to be simulated. The small variation in Sydney Harbour water levels for differing frequencies also means that design flood levels are not sensitive to the local flood and tailwater combinations chosen.

Table 6-3 Local catchment flood/tailwater combinations

Design Event	Local Catchment Flood	Tailwater#
2 year ARI	2 year ARI	1 year ARI
5 year ARI	5 year ARI	2 year ARI
10% AEP (10 year ARI)	10% AEP (10 year ARI)	2 year ARI
5% AEP (20 year ARI)	5% AEP (20 year ARI)	5 year ARI
2% AEP (50 year ARI)	2% AEP (50 year ARI)	10% AEP (10 year ARI)
1% AEP (100 year ARI)	1% AEP (100 year ARI)	5% AEP (20 year ARI)
0.2% AEP (500 year ARI)	0.2% AEP (500 year ARI)	1% AEP (100 year ARI)
PMF	PMF	1% AEP (100 year ARI)

modelled as static/constant peak water level.

6.3 Pit Inlet Blockages

Based on community consultation feedback for frequent events and the Sydney Development Control Plan (DCP), different pit blockages were adopted based on the magnitude of the storm. The following pit blockages were used for design event modelling:

5 year ARI and more frequent:

- Kerb inlets (on-grade) pits are assumed to be 20% blocked; and
- Sag pits are assumed to be 50% blocked.

Rarer than the 5 year ARI:

- Kerb inlets (on-grade) pits are assumed to be 50% blocked; and
- Sag pits are assumed to be 100% blocked.

6.4 Modelled Design Events

6.4.1 Catchment Derived Flood Events

A range of design events were defined to model the behaviour of catchment derived flooding within the Darling Harbour catchment including the 2 year ARI, 5 year ARI, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.2% AEP and PMF events. The catchment derived flood events were based on the following:

- Design rainfall parameters derived from standard procedures defined in AR&R (Pilgrim, DH, 2001);
- Static Harbour water boundary as presented in Table 6-3; and
- Blockage of drainage infrastructure as detailed in Section 6.3.

6.4.2 Tidal Inundation

Limited tidal inundation has been investigated based on the 1 year ARI Sydney Harbour water level (1.24 m AHD) (see Appendix A, Figure A- 36).

6.5 Conclusion

Design flood conditions have been simulated by generating design rainfall and tidal conditions for the Darling Harbour catchment. Design rainfall depth is based on the generation of intensity-frequency-duration (IFD) design rainfall curves utilising the procedures outlined in ARR (Pilgrim, DH, 2001). A range of storm durations were modelled using standard temporal patterns in order to capture the worst-case flooding in the catchment.

7 DESIGN FLOOD RESULTS

A range of design flood events were modelled, the results of which are presented and discussed below. The simulated design events included the 2 year ARI, 5 year ARI, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.2% AEP and PMF events for catchment derived flooding and the 1 year ARI Harbour level for the tidal inundation mapping.

A range of design event storm durations have been simulated for each event. The design results presented in the remainder of the report represent the maximum values across all durations (peak envelope) for each design event simulated.

A series of design flood maps are provided in Appendix A. Supplementary to mapped results output, tabular results of peak flood behaviour have been provided for all design events in Table 7-1 and Table 7-2. The locations of flooding behaviour reported in Table 7-1 and Table 7-2 are shown in Figure 7-1 and Figure 7-2, respectively.

Table 7-1 Peak design flood levels

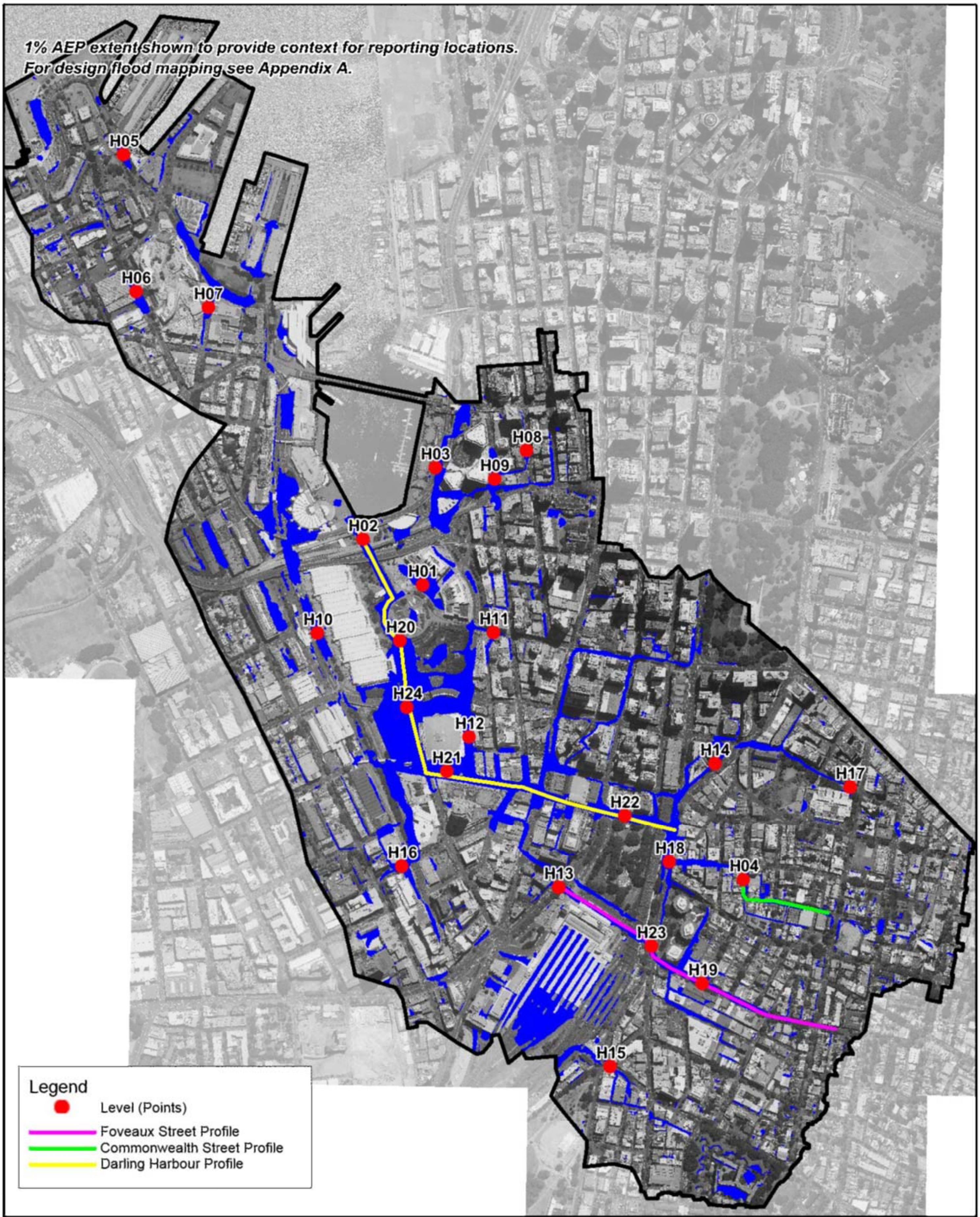
Location#	2yr ARI	5yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	0.2% AEP	PMF
H01	3.38	3.40	3.42	3.42	3.43	3.44	3.45	4.29
H02	2.43	2.44	2.45	2.46	2.60	2.69	2.83	3.50
H03	2.76	2.76	2.76	2.77	2.82	2.87	2.95	3.34
H04	16.54	16.60	17.23	17.32	17.39	17.45	17.57	18.09
H05	2.60	2.63	2.68	2.73	2.76	2.79	2.85	3.00
H06	6.47	6.55	7.23	7.32	7.42	7.53	7.77	10.81
H07	2.54	2.60	2.75	2.79	2.82	2.85	2.90	3.16
H08	11.34	11.36	11.37	11.38	11.39	11.40	11.42	11.57
H09	5.40	5.51	5.62	5.69	5.73	5.77	5.87	6.24
H10	2.77	2.85	2.89	2.95	3.02	3.09	3.18	4.47
H11	6.82	6.83	6.85	6.88	6.89	6.90	6.92	6.99
H12	2.88	3.01	3.08	3.14	3.18	3.23	3.43	4.62
H13	11.49	11.52	11.53	11.54	11.55	11.56	11.58	11.72
H14	17.06	17.09	17.10	17.11	17.12	17.13	17.14	17.31
H15	24.37	24.39	24.40	24.42	24.42	24.43	24.46	24.66
H16	4.45	4.52	4.57	4.60	4.63	4.67	4.74	5.22
H17	35.06	35.07	35.07	35.09	35.09	35.10	35.11	35.25
H18	11.24	11.28	11.35	11.41	11.45	11.49	11.59	12.33
H19	19.50	19.53	19.55	19.57	19.58	19.61	19.65	19.90
H20	2.67	2.67	2.68	2.85	3.03	3.16	3.40	4.54
H21	3.15	3.21	3.28	3.34	3.38	3.43	3.53	4.68
H22	7.61	7.64	7.66	7.69	7.72	7.75	7.83	8.28
H23	16.25	16.27	16.29	16.30	16.30	16.31	16.33	16.54
H24	2.48	2.48	2.74	2.91	3.06	3.19	3.43	4.63

Refer to Figure 7-1 for the reporting locations

Table 7-2 Peak design flood flows – pipe (P) and overland (Q)

Location[#]	2yr ARI	5yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	0.2% AEP	PMF
Q01	0.0	0.1	0.1	0.7	2.9	5.3	10.6	91.6
Q02	0.5	0.7	1.0	1.2	1.3	1.6	2.1	7.7
Q03	0.4	1.5	2.9	3.9	4.9	6.1	8.8	34.1
Q04	0.7	1.6	2.4	3.4	4.0	5.0	6.9	21.7
Q05	1.9	4.0	5.5	8.1	10.5	13.9	21.3	92.2
Q06	0.7	1.1	1.4	1.8	2.0	2.3	3.0	9.3
Q07	1.6	2.7	3.2	4.0	4.4	5.2	6.9	25.2
Q08	2.5	5.7	9.1	14.0	18.9	24.3	36.1	154.6
Q09	0.2	0.2	0.3	0.4	0.4	0.4	0.6	2.0
Q10	0.0	0.0	0.0	0.2	0.3	0.3	0.6	27.4
Q11	0.3	2.4	3.9	5.5	6.8	8.2	10.5	20.7
Q12	0.2	0.8	3.4	6.5	9.8	13.4	21.5	99.6
P01	0.5	0.6	0.5	0.6	0.6	0.6	0.6	0.7
P02	1.7	2.2	1.6	1.5	1.7	2.0	1.9	2.6
P03	0.2	0.3	0.0	0.1	0.1	0.1	0.1	0.1
P04	2.9	3.8	4.1	4.8	5.2	5.6	6.3	9.1
P05	1.8	2.7	3.1	3.4	3.7	3.9	4.4	7.4
P06	4.8	5.6	6.1	6.5	6.8	7.1	7.6	10.1
P07	7.3	8.7	8.6	9.0	9.3	9.5	9.9	11.4
P08	12.4	14.8	14.7	15.5	15.9	16.3	16.9	19.5
P09	2.2	2.8	3.0	3.3	3.5	3.7	4.0	5.5
P10	3.9	4.6	4.9	5.3	5.5	5.7	6.1	8.2
P11	2.9	3.4	2.1	2.3	2.5	2.7	2.9	3.9

[#] Refer to Figure 7-2 for the reporting locations



Title:

Darling Harbour Result Locations
Level Recording - Points and Profiles

Figure: 7-1

Rev: -

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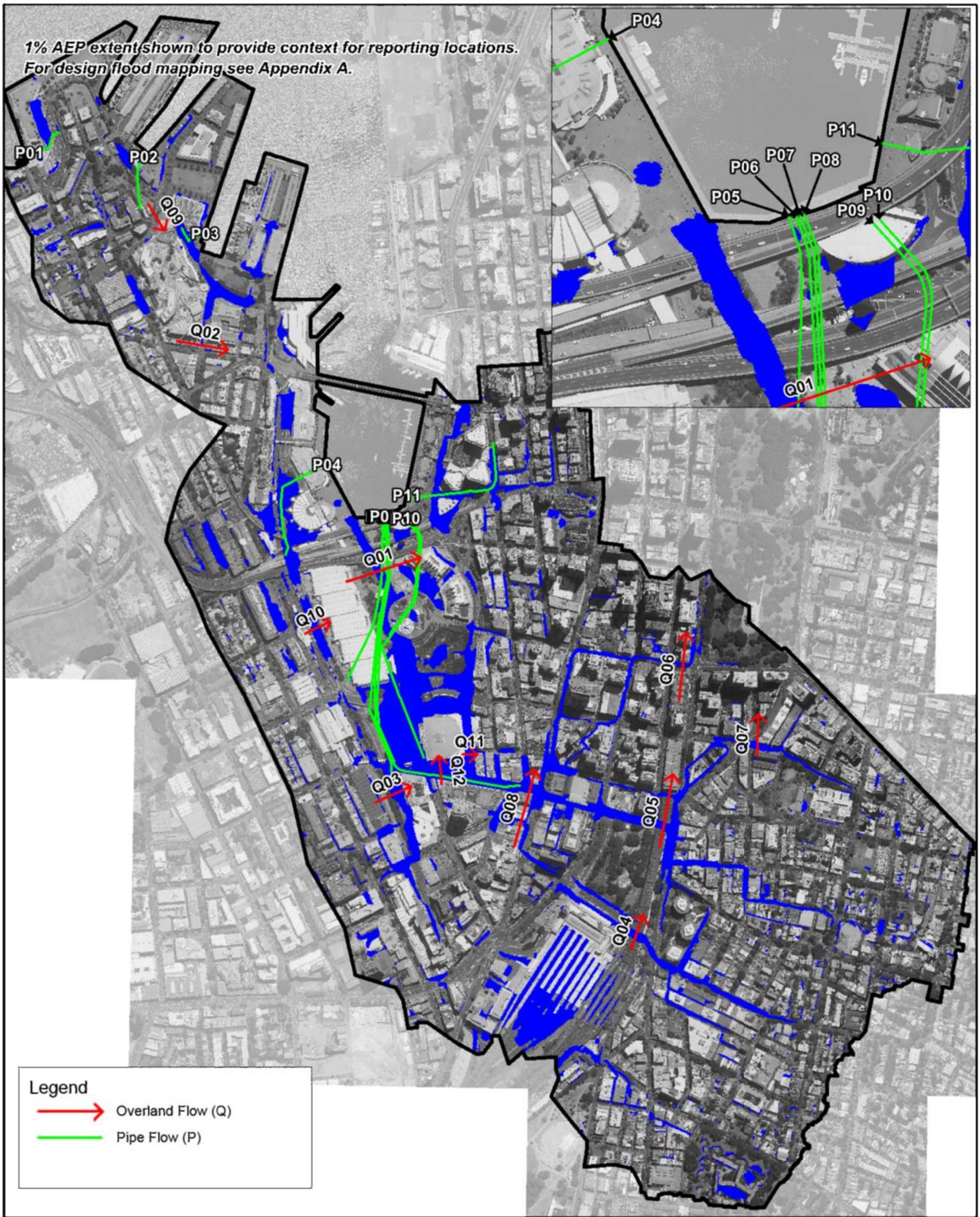
Filepath : S:\WATER\PROJECTS\S20012 - Water - Darling Harbour Catchment Flood Study\Drafting\WOR\Figures\FIGURE07-1_ResultLocations_FLOW.wor



Title:

Darling Harbour Result Locations
Flow Recording - Overland and Pipe Flow

Figure: 7-2

Rev: -

0 375 750m

Approx. Scale

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7.1 Peak Flood Conditions

7.1.1 Flooding Behaviour

7.1.1.1 Overview

Section 2.1 provides a general overview of the layout of the drainage network infrastructure and major flow paths. The trunk drainage network across the study area is comprised of predominantly pipe reaches. Overland flow routes are generally confined to the road network which is typical of urban environments, but even more pronounced in the Darling Harbour catchment.

The Darling Harbour catchment has two distinct catchments areas with the Western Distributor being the divide. Flows underneath the Western Distributor arrive from the Surry Hills area to the south-east. North of the Western Distributor, flood waters have very small catchment areas and flow quickly to Cockle Bay/Sydney Harbour by the shortest distance.

High in the catchment upstream of the Western Distributor (south east Surry Hills area), steep streets quickly convey flows downstream to the Darling Harbour area. Figure 7-4 shows the peak flood level profile at Foveaux Street for all modelled design events and demonstrates the limited flood level sensitivity to the event exceedance probability. A noted exception to the limited sensitivity in upper catchment reaches is at Commonwealth Street. Commonwealth Street has a trapped low point which was identified in the community consultation stage of the study. Figure 7-5 shows the peak flood level profile for the Commonwealth Street trapped low point for all modelled design events. Figure 7-5 shows the limited flood level sensitivity to event recurrence interval in Ann Street which feeds the trapped low point and Reservoir Street which drains overflow. The flood level in the trapped low point however increases by 0.9m from the 2 year ARI event to the 1% AEP event and is further sensitive to pit blockage assumptions.

Downstream of Elizabeth Street (and the railway line), the catchment slope starts to reduce. Sub-surface conduits become very important in relieving flood waters. Overland flow at Q01 (see Figure 7-2) first initiates in the 5% AEP (20 year ARI) event. For events more frequent than the 2% AEP event, pipes P05, P06, P07, P08, P09 and P10 drained catchment flows. For the 5% AEP event, these 6 pipes convey a peak flow of 43m^3/s. In the 1% AEP event, overland flow at Q01 is 5.3m^3/s, representing 10% of the flow from the catchment at this location.

North of the Western Distributor, flooding in the catchment is from localised catchments with small upstream catchment areas. These catchments may drain to trapped low points such as Pyrmont Road (point H06) where piped infrastructure is critical in relieving flooding.

Peak flood behaviour for design modelling is best interpreted by reviewing the extensive series of design flood mapping figures presented in Appendix A.

FIGURE 7-3
DARLING HARBOUR PROFILE - DESIGN EENT RESULTS

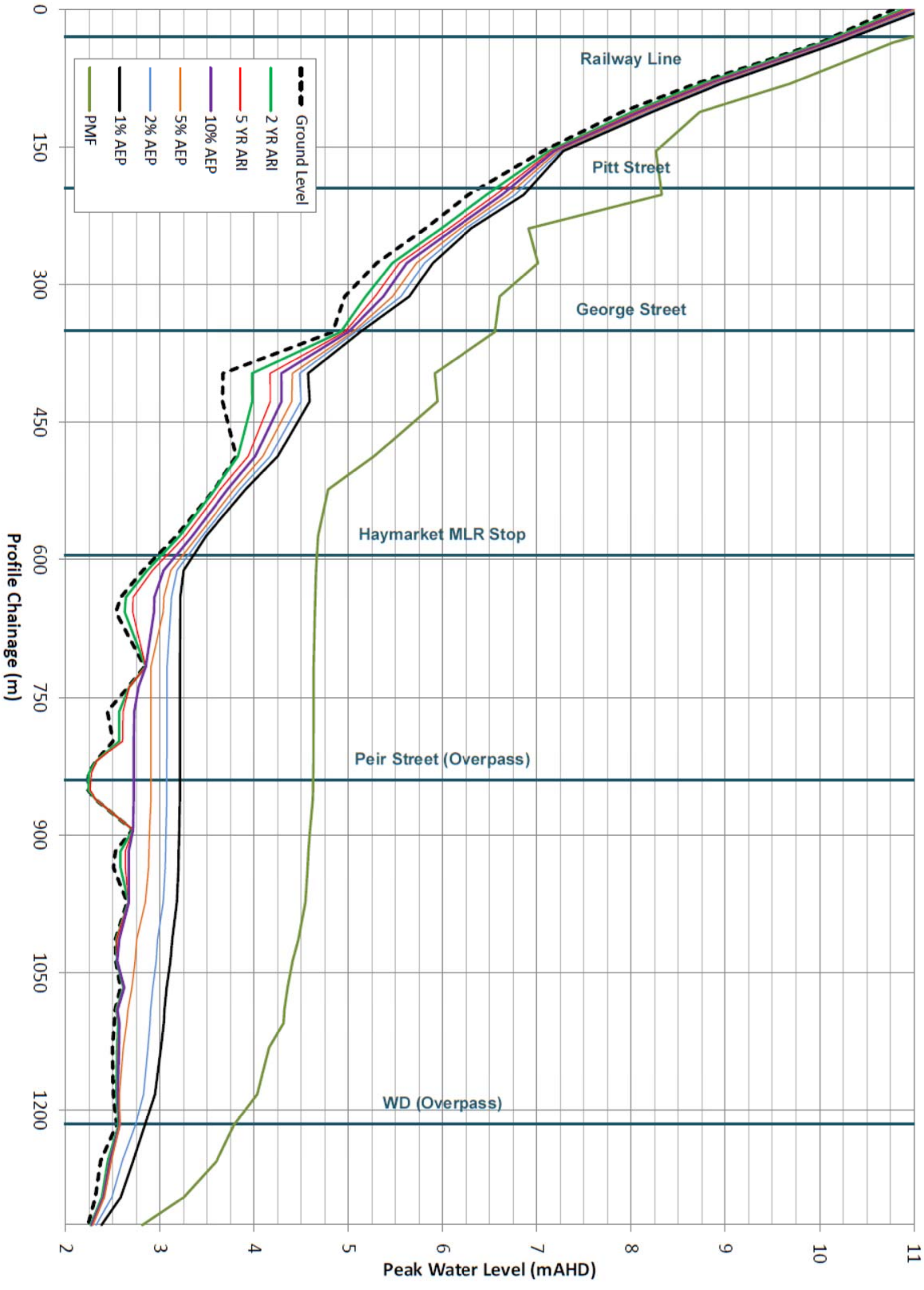

FIGURE 7-4
FOVEAUX STREET PROFILE - DESIGN EENT RESULTS

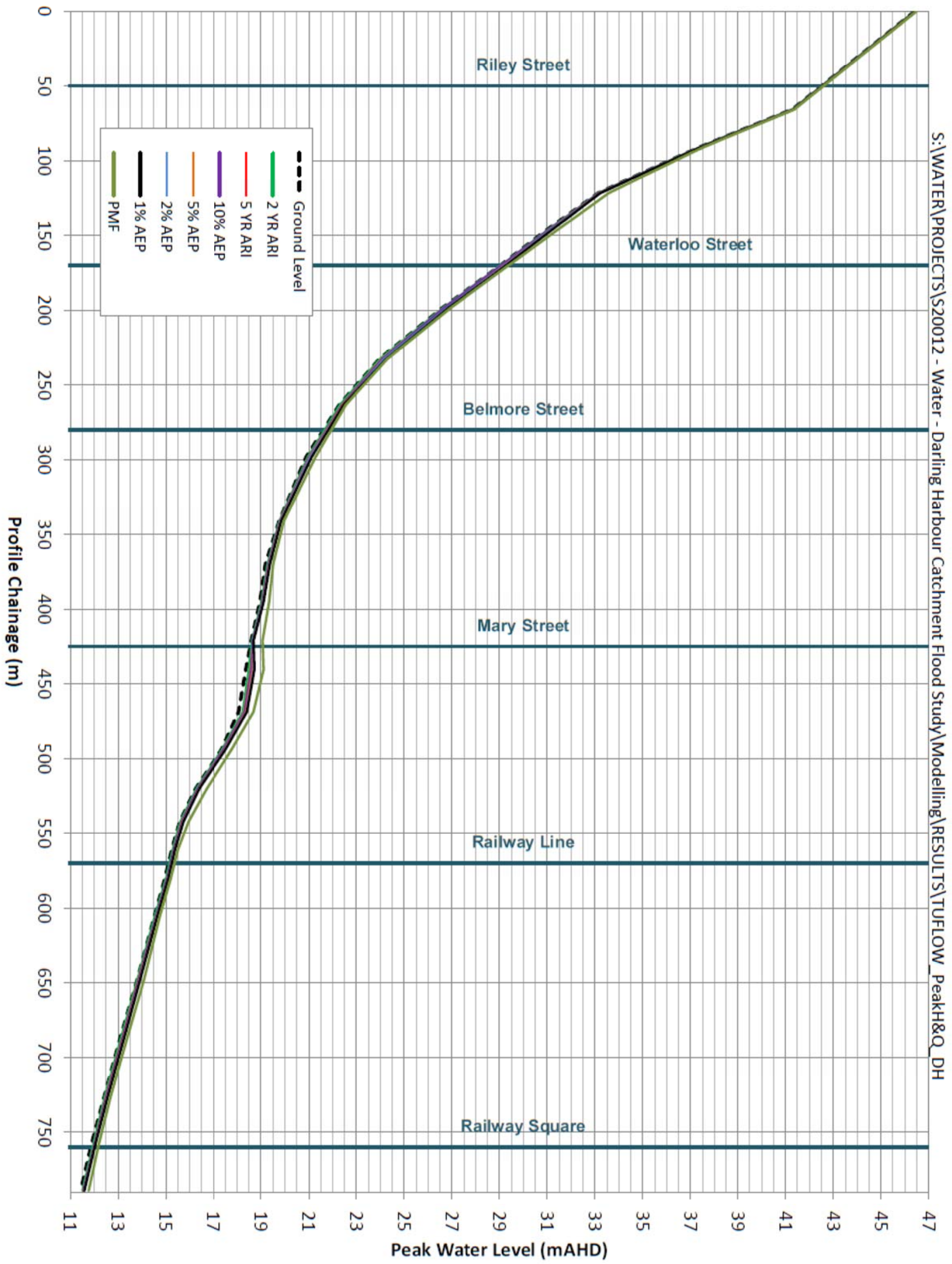

FIGURE 7-5
COMMONWEALTH STREET PROFILE - DESIGN EENT RESULTS