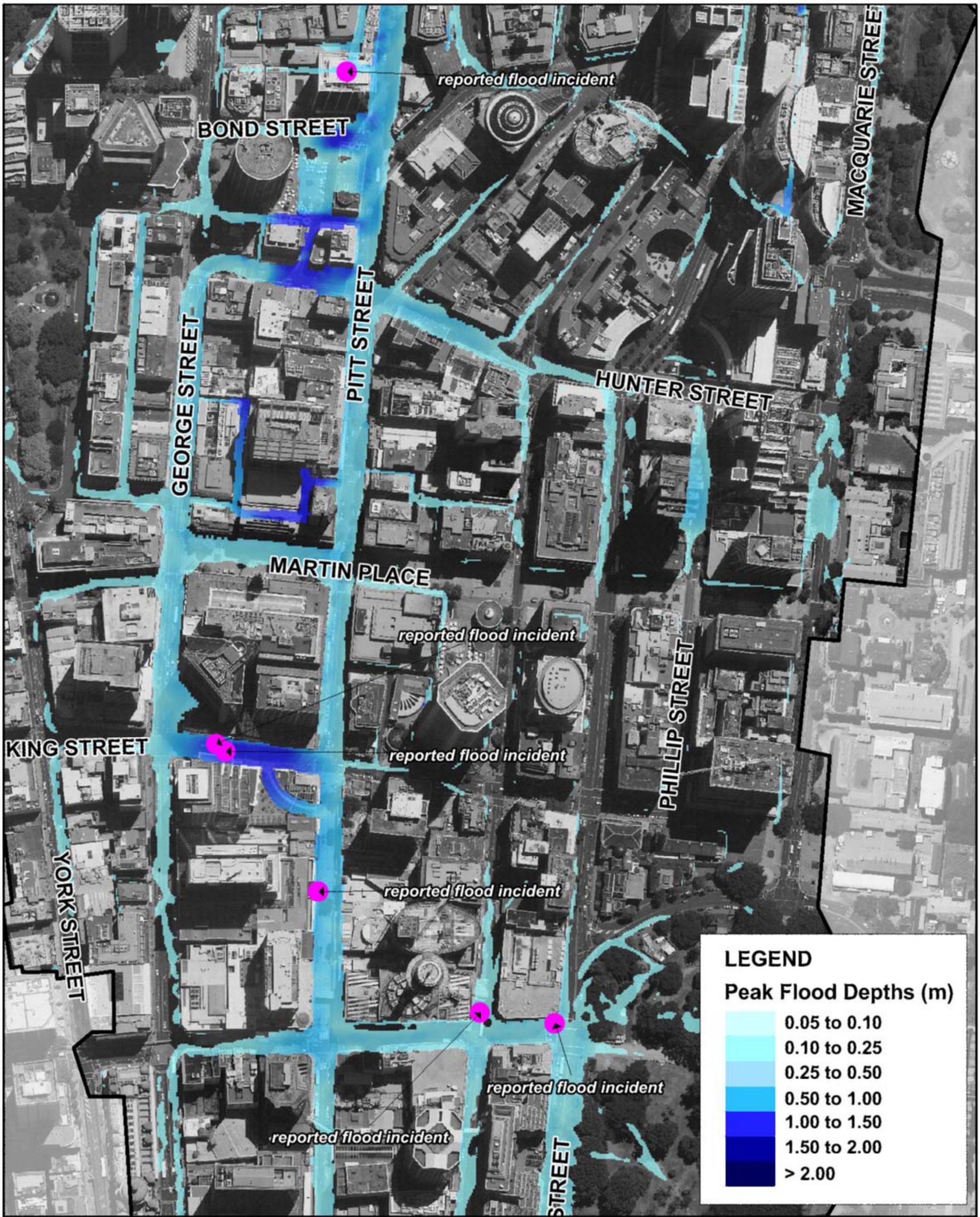

Title:

Peak Flood Depth - Verification Event 8 November 1984

Figure: 5-3

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Approx. Scale

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5.5 Model Calibration – 26 January 1991

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. A total rainfall depth of approximately 65 mm fell over a 1 hour period with the peak of the rainfall occurring at 2:55 PM which coincided with a low tide level of -0.1m AHD.

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 50 year ARI design rainfall event for a 25 minute duration.

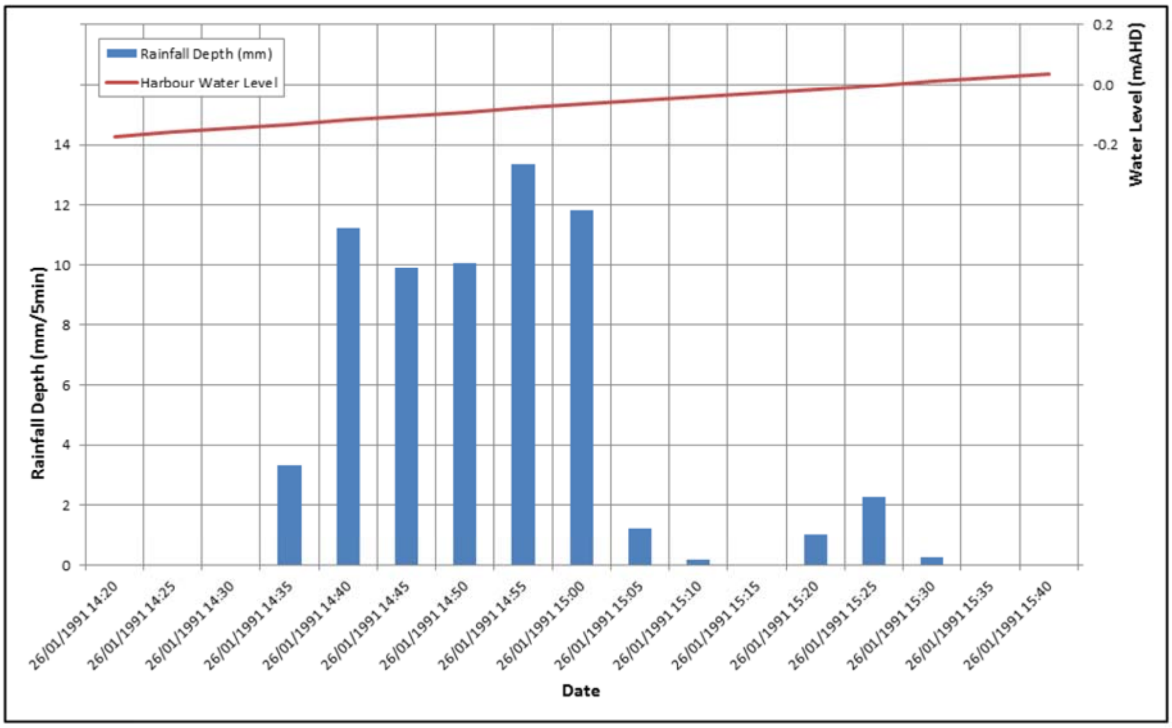


Figure 5-4 Recorded rainfall and harbour water level – 26 January 1991

5.5.2 Observed and Simulated Flood Behaviour

Three reports of flood behaviour for the 26 January 1991 event are available in the City Area catchment. These flooding reports are sourced from the Sydney Water Corporation Historic Flood Database and are presented below. Figure 5-6 shows the peak depth results from this verification event and shows locations of each of these reports of flooding.

- King Street (between George and Pitt Streets), Sydney: Build-up of water in King Street sag. Flood level exceeded entry level of car park for property associated to flooding incident.
- Bond Street, Sydney: Water flowed down the car park ramp in Bond Street.
- Macquarie Place (corner of Alfred and Loftus Streets), Sydney: Water ponded above the kerb level to approximately ankle depth.

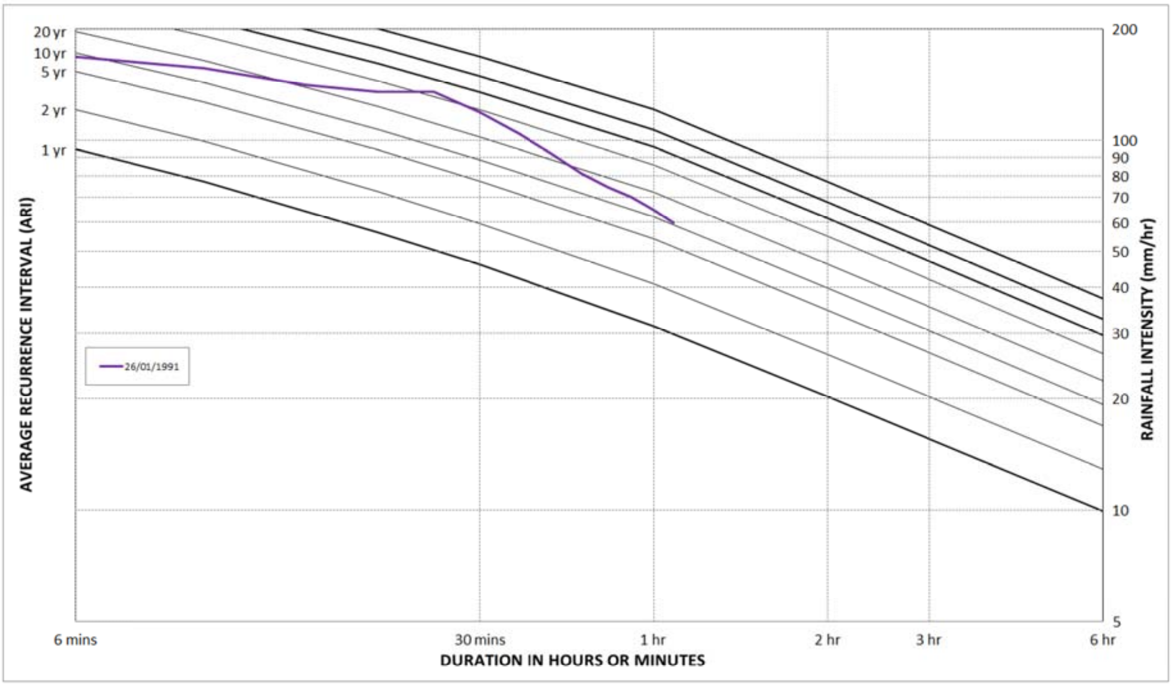


Figure 5-5 Comparison of 26 January 1991 rainfall with IFD relationships

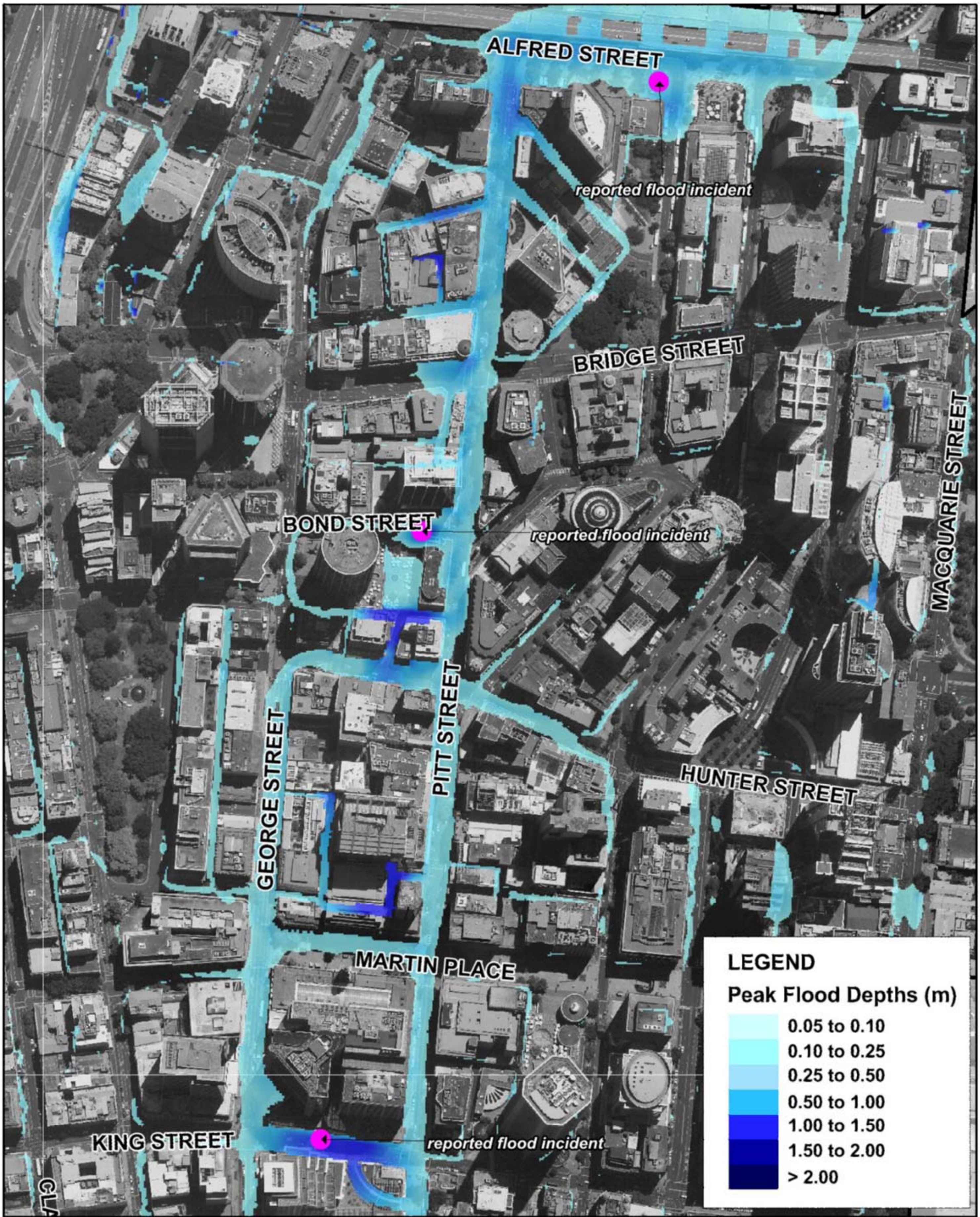
As shown in Figure 5-6, the TUFLOW model results of the 26 January 1991 calibration compares well with observed flooding behaviour, summarised as follows:

- The sag on King Street between Pitt Street and George Street was flooded in the January 1991 event as well as the November 1984 event. Calibration modelling for the January 1991 modelling shows that a peak depth of 1.1 m was is being predicted at this low point which is only 0.1 m less than occurred in the November 1984 event. The predicted peak flood depth at this location is likely to be sufficient to cause flooding of the car park.

- The water in Bond Street reached a maximum depth of 0.9 m in front of the entrance to the car park. This level is considered sufficient to trigger the reported filling of the car park

- At the corner of Alfred Street and Loftus Street, the model is predicting a depth of 300 mm which is slightly higher than the ankle deep estimate provided at this location.

Based on the available data, the model is considered to be adequately representing the observed flooding behaviour for the 26 January 1991 event.



Title:

Peak Flood Depth - Verification Event 26 January 1991

Figure: 5-6

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Approx. Scale

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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.

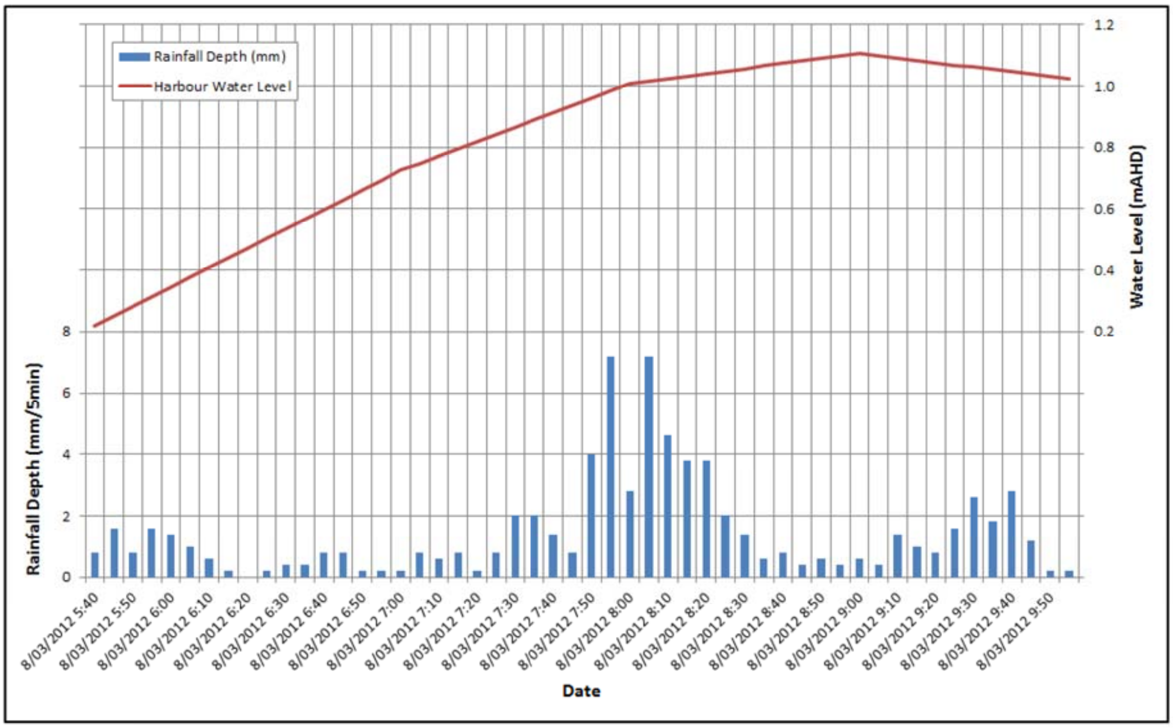


Figure 5-7 Recorded rainfall and harbour water level – 8 March 2012

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 Hickson Road, Walsh Bay

A longer term resident (9 years) reported flooding in a basement garage on Hickson Road. The road is further reported to be flooded after intense storms. Figure 5-9 shows the flooding in the area. As shown, the street is flooded above kerb level which would provide the opportunity for flows to enter basement levels through driveways or other road level entrances. This modelled level of road inundation provides support to the observations from the resident.

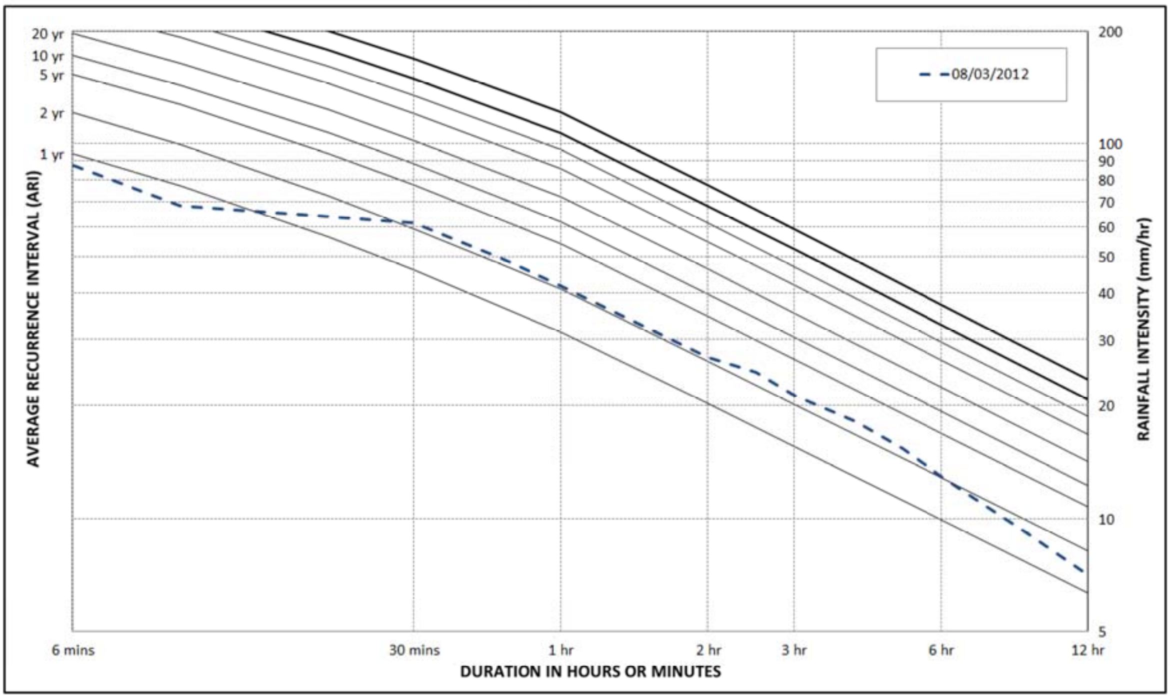


Figure 5-8 Comparison of 8 March 2012 rainfall with IFD relationships

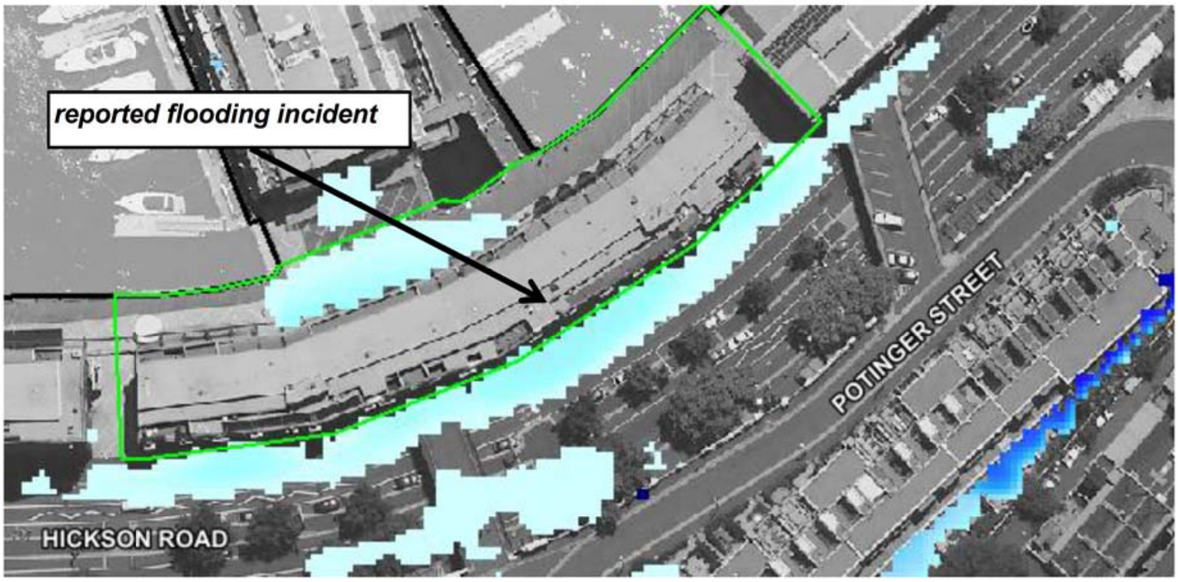


Figure 5-9 Flooding reports on Hickson Road, Walsh Bay

5.6.2.2 Gloucester Street, The Rocks

A resident on Harrington Street reported that regular flooding occurs at the low point on Gloucester Street. The low point on Gloucester Street is not effectively drained and the standing water which remains after rainfall events seeps into the basement storage below. The same resident reported that the footpath on the western side of Cumberland Street (near the intersection with Essex Street) has standing water after every rainfall event. Figure 5-10 shows results of flooding in the area near Gloucester Street.

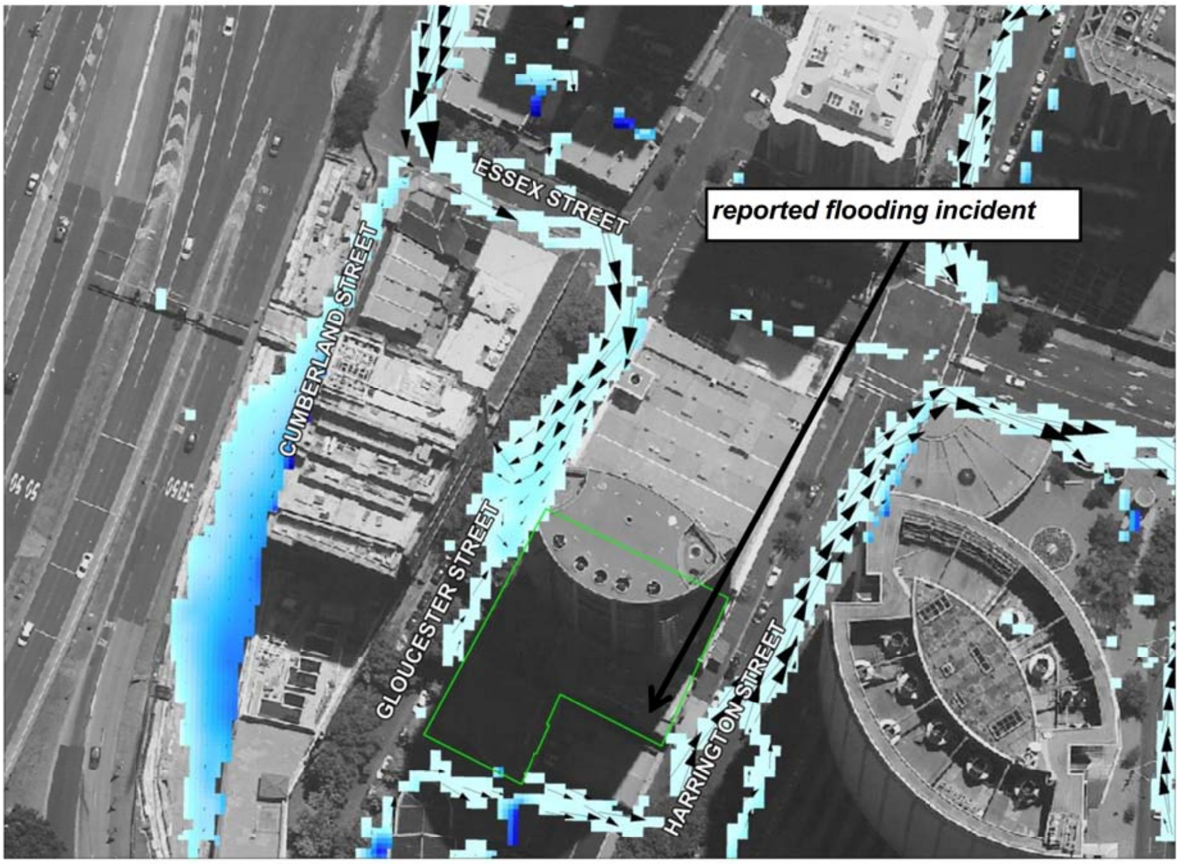


Figure 5-10 Flooding reports on Gloucester Street, The Rocks

The results presented in Figure 5-10 demonstrate that the model is simulating the observed flooding behaviour on Gloucester Street and at the corner of Essex Street and Cumberland Street. Overall the model is produces flood behaviour generally consistent with the reports obtained via the community consultation.

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 City Area 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 City Area catchment the key accounts of flooding are as follows:

- Circular Quay flooded to depth of 1-4 feet (April 1949, March 1914)
- Pitt Street Flooded (1877, 1878, 1912, 1913, 1938 and 1949)

- Main flow path has been identified as Market Street, Elizabeth Street and Park Street from Hyde Park (October 1877, August 1878, June 1949)

Flooding behaviour predicted by the model in the vicinity of Circular Quay is shown in Figure 5-11. Historic accounts of flooding report that flooding has previously occurred to depths of 1- 4 ft in this general area. The modelling shows that this location is susceptible to flooding. Upstream overland flows, mainly from Pitt Street, flow rapidly in a northern direction towards Circular Quay and Sydney Harbour. Alfred Street is relatively flat and upstream flows spread out at this location before flowing under the Cahill Expressway to Sydney Harbour. For the March 2012, January 1991 and November 1984 calibration events, the modelled depths at Alfred Street range from very shallow depths to 0.9m. This supports the historic reports of this area being flood prone.

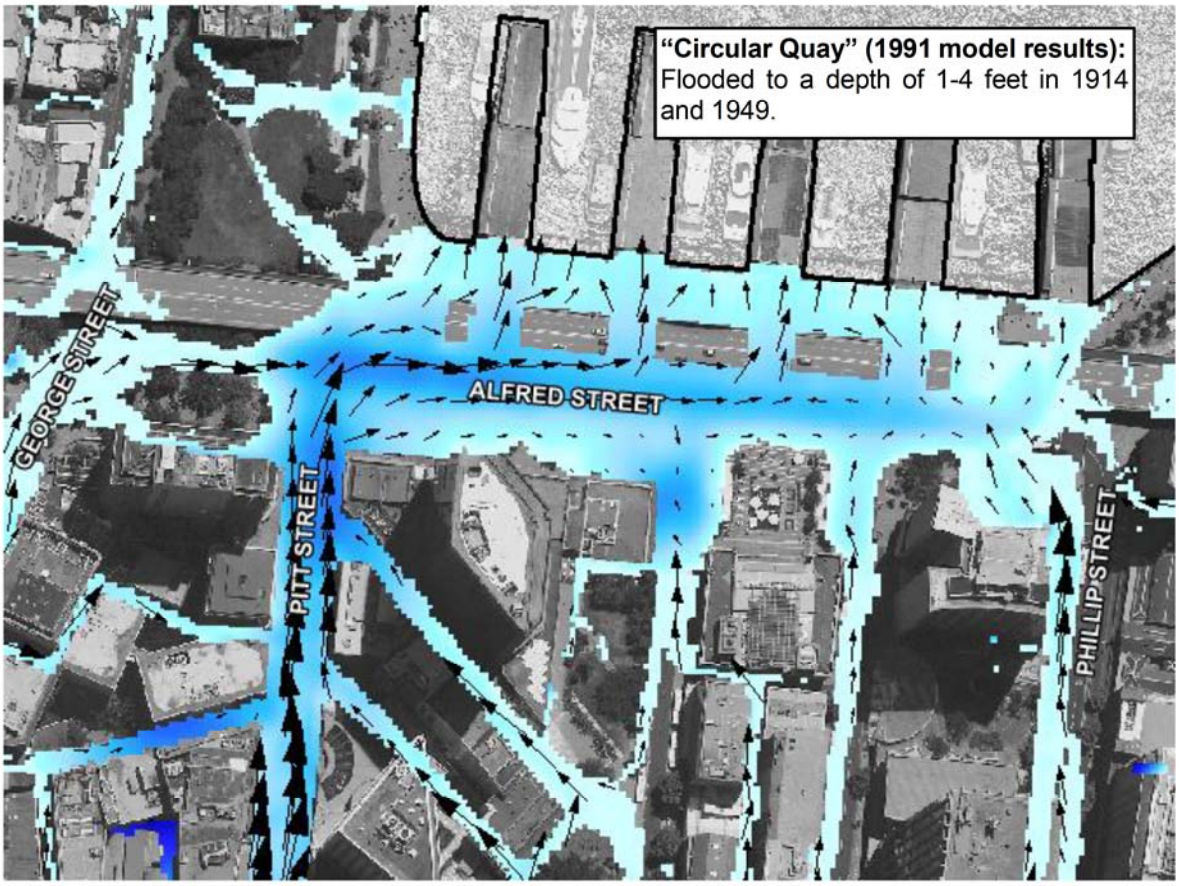


Figure 5-11 Historic flooding reports at Circular Quay.

Pitt Street has a long history of flooding incidents. A descriptive passage from an evening news broadcast on 1 August 1878 sets the standard for the flood potential:

"In less than a quarter of an hour from the commencement of what we call nothing less else than a deluge, a boat might have been pulled along Pitt Street from Market Street to King Street, and in King Street from Pitt Street to George Street"

Historic accounts of flooding refer to storms in 1877, 1878, 1912, 1913, 1938 and 1949. Further reports of flooding were noted for the November 1984 event where water reached a depth of at least 300 mm which exceeded the kerb level and flooded adjacent shops.

Figure 5-12 supports the historic accounts of Pitt Street functioning as an overland flow path. Figure 5-12 further shows that Pitt Street flows are generated from flows along Market Street and Park Street which convey Hyde Park run-off.

Figure 5-12 Historic flooding reports on Pitt Street and from Hyde Park

Due to the anecdotal nature of the newspaper flood reports and the fact that the reported flood events occurred over 130 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 5-13 and Figure 5-14 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.

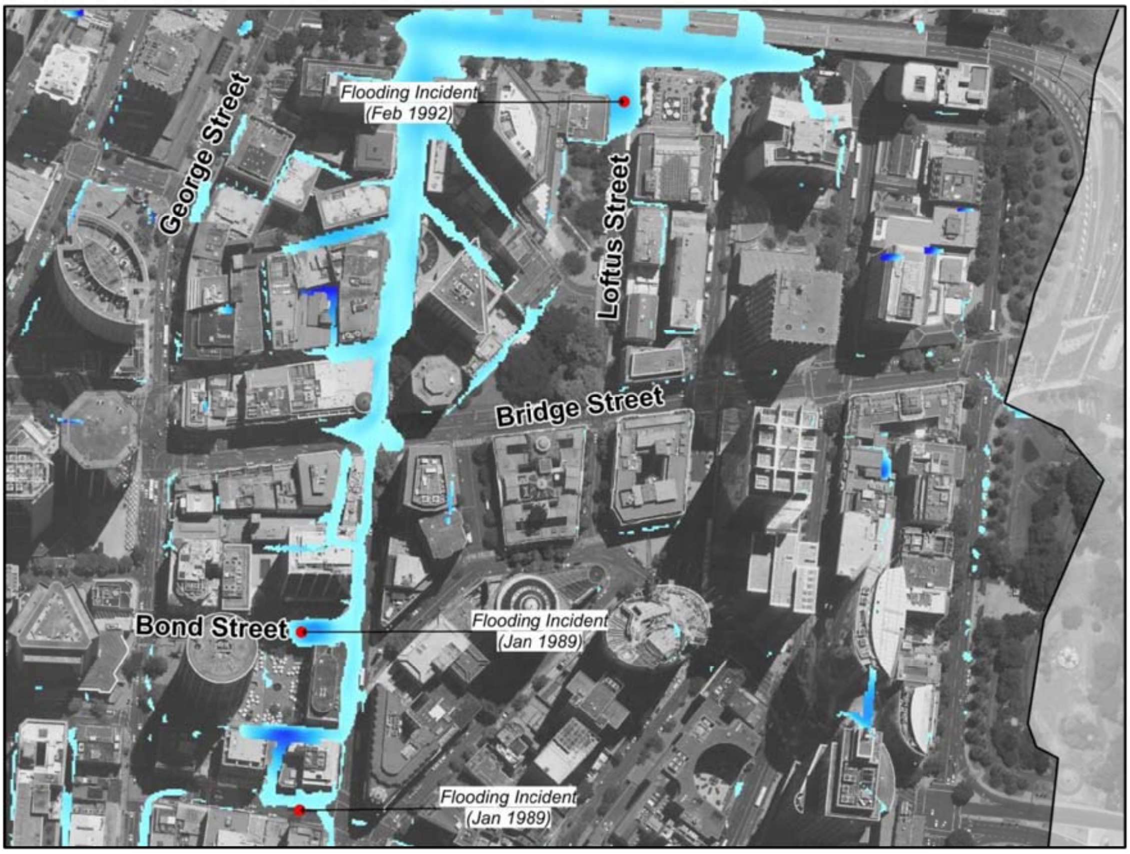


Figure 5-13 SWC Historic flooding reports in Haymarket area

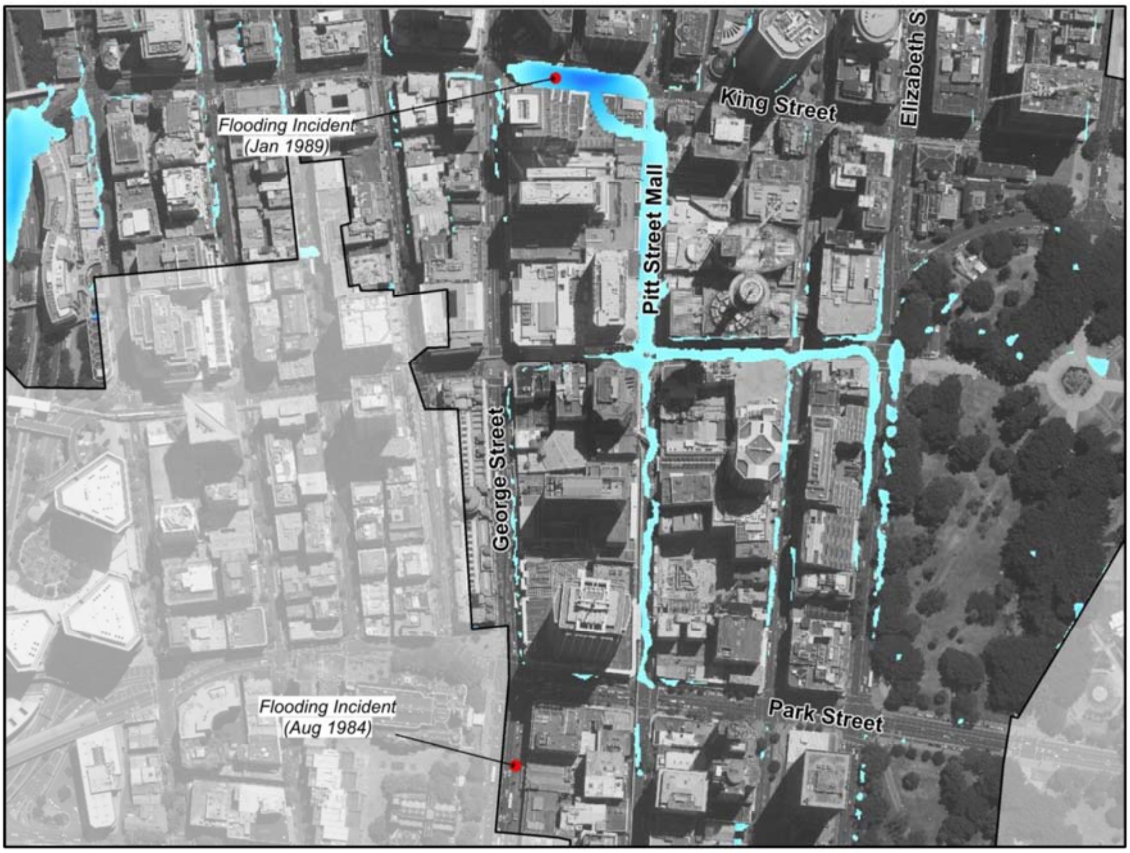


Figure 5-14 SWC Historic Flooding reports in Ultimo Area

As presented in Figure 5-13 and Figure 5-14, the historic reports of flooding consistently affect the same locations. Further, current catchment flood modelling shows flooding still occurs at the historic locations.

Loftus Lane flooding was previously presented in the January 1991 calibration event. Flooding at this location was additionally reported to have occurred in January 1992 which resulted in street flooding.

The car park entrance in Bond Street was overtopped for the January 1989 event. This flooding mechanism had previously been reported for the January 1991 calibration event.

Hunter Street reported flooding in January 1989 resulting in a car park entrance being overtopped.

The King Street low point between George and Pitt Streets has numerous reports of flooding. January 1989 recorded another event which resulted in flooding at this location. For the 1989 event, the car park entrance in the low point was not overtopped.

In August 1984 it was reported that Town Hall Station and Sydney Square Arcade was inundated. Town Hall Station is on the divide between the Darling Harbour and City Area catchments and therefore flooding (without contributing factors such as a blockage event) is unlikely. Sydney Square Arcade couldn't be identified, therefore this report of flooding wasn't able to be used for verification.

At all locations, with the exception of the Town Hall Station flooding report, modelled flooding is shown to typically represent the observed flood behaviour.

5.9 Catchment Flow Verification

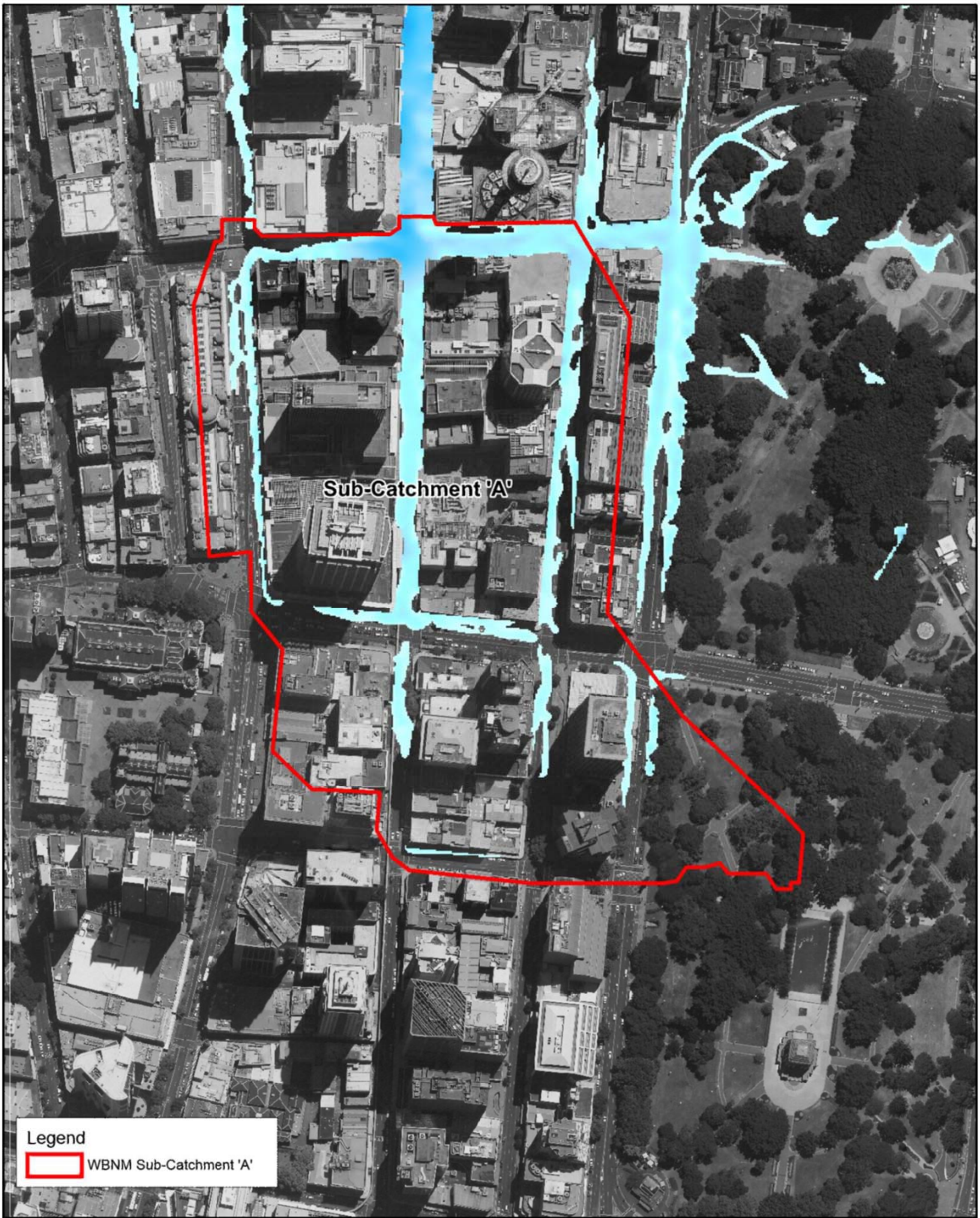
Verification of the adopted "direct-rainfall" approach for modelling the catchment hydrology has been achieved by undertaking additional hydrological modelling of selected sub-catchments within the overall study area using alternate modelling methods.

The verification approach involved setting up a WBNM model for a sub-catchment, as shown in Figure 5-15.

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).



Title:

City Area
Sub-Catchment for Catchment Flow Verification

Figure: 5-15

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BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

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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 5-16. The figure show that 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.

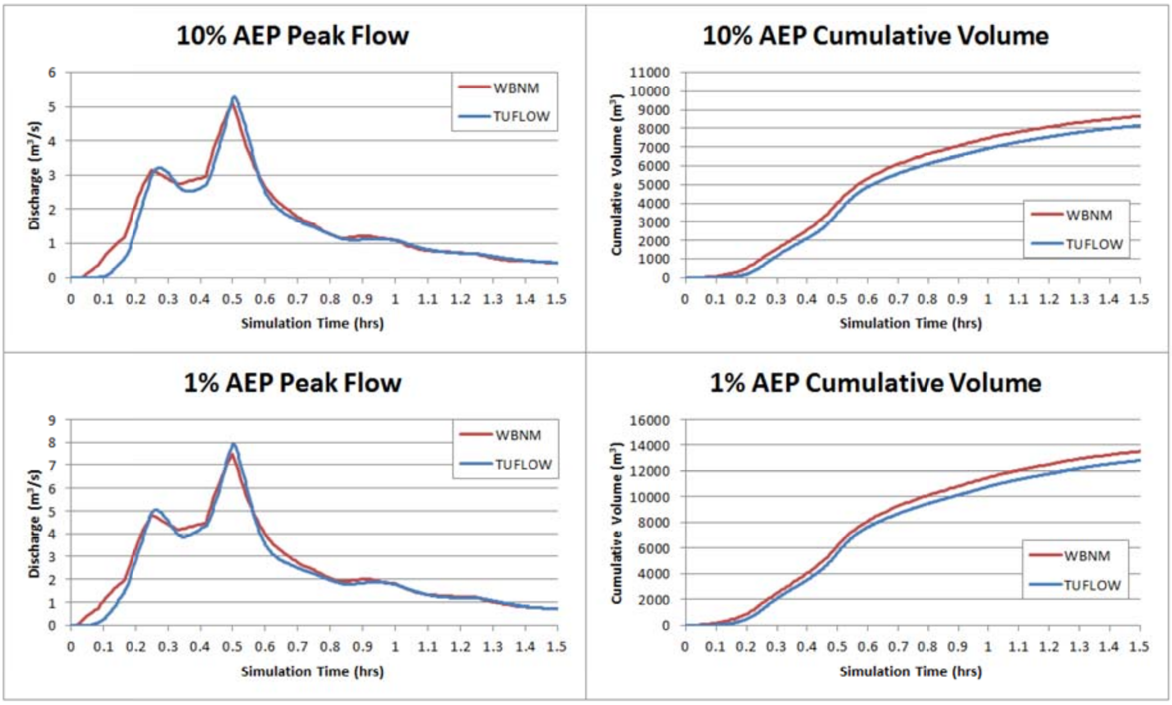


Figure 5-16 Catchment flow verification for sub-catchment 'A' (12.5ha area)

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 volume 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 City Area Flood Study provides a reasonable basis to assess overall flood behaviour.

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

5.11 Summary of Model Verification

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 City Area 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	480
45 min	48	63	72	84	99	111	136	400
1.00 h	41	53	61	71	84	95	116	350
1.50 h	32	42	48	56	66	74	91	300
2.0 h	26	35	40	46	55	62	76	265
2.5 h	23	30	35	40	48	53	n/a	232
3.0 h	20	27	31	36	42	47	58	213
4.0 h	n/a	n/a	n/a	n/a	n/a	n/a	n/a	183
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	160
6.0 h	13	17	19	22	26	30	36	142
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 City Area 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 City Area 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 City Area 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).

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

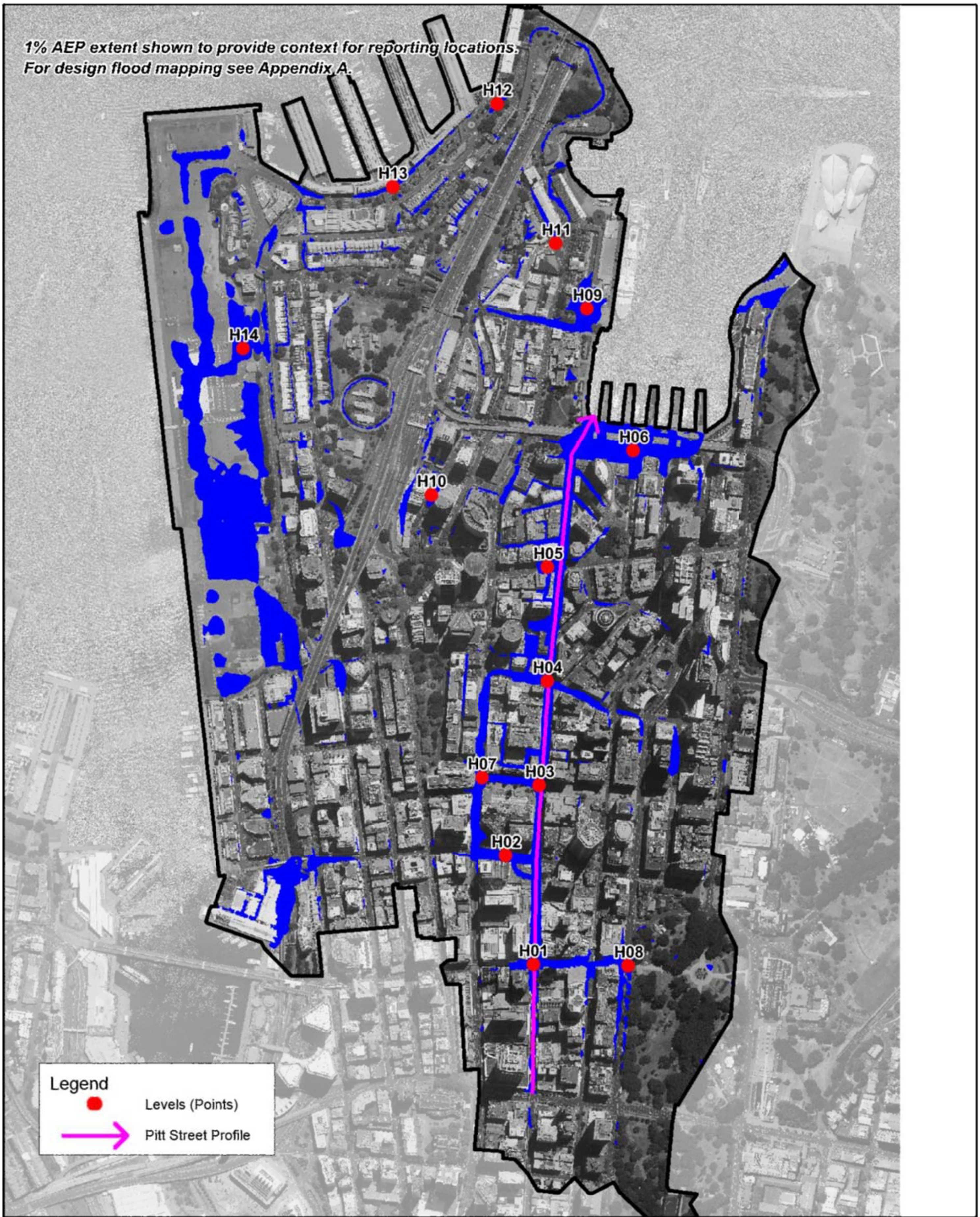
Location#	2yr ARI	5yr ARI	10% AEP	5% AEP	2% AEP	1% AEP	0.2% AEP	PMF
H01	18.59	18.68	18.71	18.77	18.82	18.86	18.94	19.49
H02	16.95	17.18	17.28	17.33	17.37	17.41	17.47	17.87
H03	14.81	14.86	14.94	15.00	15.05	15.10	15.19	15.75
H04	9.48	9.53	9.58	9.64	9.70	9.73	9.82	10.48
H05	5.09	5.24	5.41	5.48	5.54	5.59	5.71	6.47
H06	2.50	2.62	2.71	2.76	2.81	2.85	2.91	3.20
H07	15.49	15.54	15.60	15.63	15.65	15.68	15.72	16.02
H08	23.23	23.26	23.27	23.28	23.30	23.31	23.34	23.59
H09	2.41	2.50	2.59	2.65	2.69	2.72	2.77	2.94
H10	21.04	21.08	21.10	21.12	21.13	21.16	21.23	21.51
H11	11.02	11.04	11.06	11.08	11.09	11.10	11.11	11.24
H12	2.61	2.63	2.65	2.68	2.71	2.74	2.77	2.81
H13	2.51	2.55	2.56	2.58	2.59	2.61	2.64	2.89
H14	2.47	2.49	2.54	2.57	2.59	2.60	2.63	2.78

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	0.9	1.3	1.6	2.0	2.1	2.4	3.0	8.0
Q02	1.6	2.8	3.5	4.4	5.2	6.2	8.6	28.7
Q03	1.7	3.9	4.8	6.7	8.2	9.8	13.4	47.0
Q04	1.0	3.2	9.2	13.0	16.7	20.7	29.1	101.7
Q05	2.9	6.5	11.2	15.8	20.4	25.1	35.1	120.5
Q06	0.6	4.4	10.0	14.9	20.1	25.4	36.7	134.0
Q07	0.8	1.3	2.0	2.8	3.1	3.5	5.2	11.2
P01	3.6	3.9	4.1	4.0	4.1	4.2	4.2	4.4
P02	1.0	1.3	1.4	1.4	1.5	1.5	1.5	1.7
P03	1.4	1.8	2.0	2.1	2.2	2.2	2.5	2.7
P04	0.6	0.9	0.7	0.7	0.8	0.8	0.9	1.5
P05	0.7	0.9	0.9	1.0	1.1	1.2	1.4	2.0
P06	4.9	5.0	4.9	5.0	5.0	4.9	5.0	5.1
P07	5.1	5.5	5.6	5.7	5.8	5.9	6.1	6.8
P08	3.8	4.1	3.3	3.4	3.5	3.6	3.9	4.7

Refer to Figure 7-2 for the reporting locations



Title:

City Area Result Locations
Level Recording - Points and Profile

Figure: 7-1

Rev: -

0 250 500m

Approx. Scale

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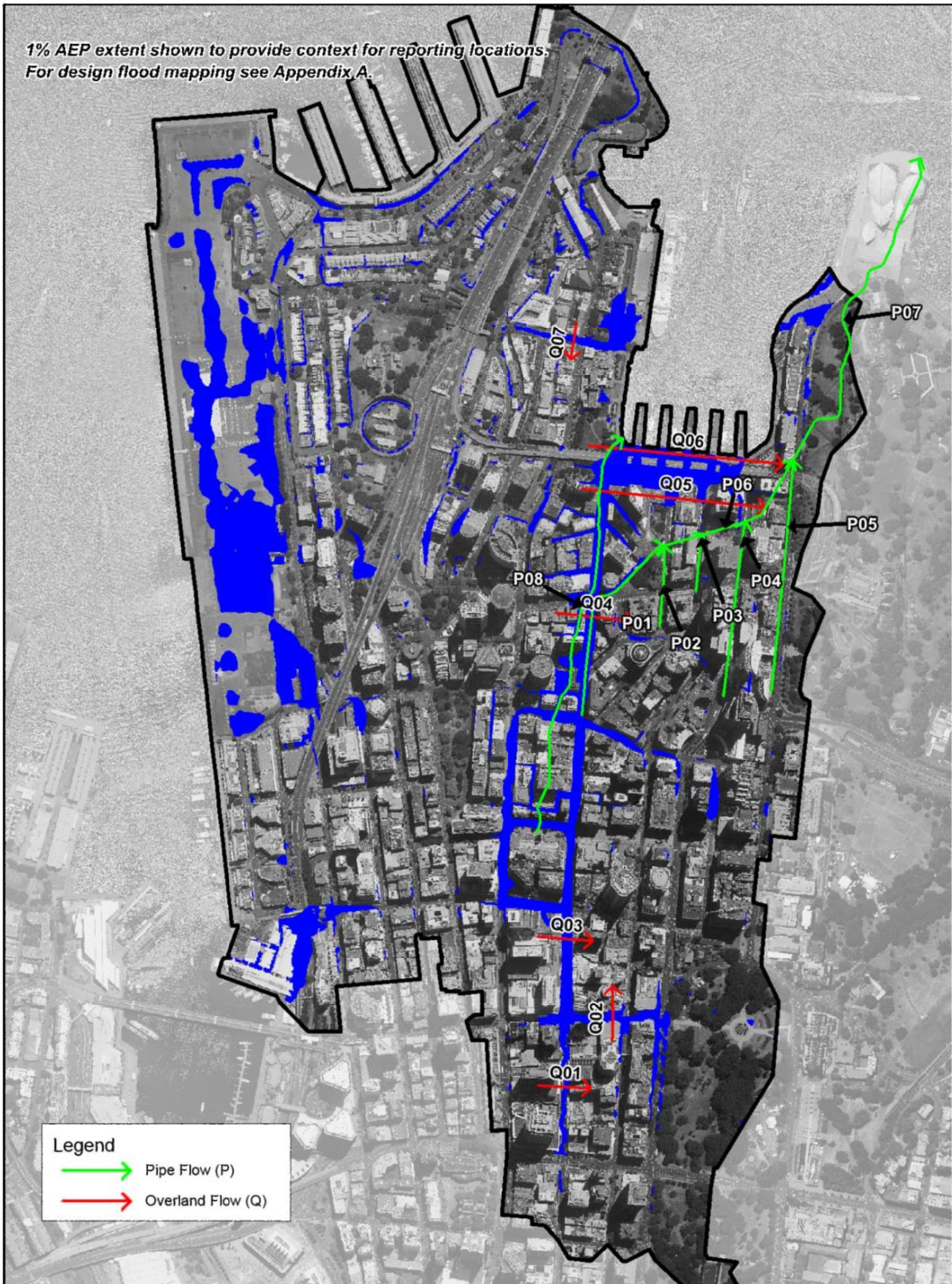
Filepath : S:\WATER\PROJECTS\S20022 - Water - Sydney CBD Catchment Flood Study\Drafting\WOR\Figures\FIGURE07-1_ResultLocations_LEVEL.wor



Title:

City Area Result Locations
Flow Recording - Overland and Pipe Flow

Figure: 7-2

Rev: -

BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

0 250 500m
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 Sydney City Area catchment.

Pitt Street forms the primary overland flow path that drains the majority of the City Area catchment. The top of the Pitt Street catchment is bounded by Hyde Park to the east, Liverpool Street to the south and York Street to the west. Runoff from the catchment extremities drains quickly to the primary overland flow path along Pitt Street downstream to Circular Quay (i.e. in a northerly direction).

Figure 7-3 shows the peak flood level profile along Pitt Street (for the location of profile see Figure 7-1). These flood levels show that the upstream portion of Pitt Street (chainage 600m; southern end) is not as sensitive to the event exceedance probability as the lower parts of the catchment (chainage 1200m; northern end). From the 2 year ARI to the 1% AEP, flood levels at Martin Place only increase by 0.2 m compared with 0.4 m lower downstream at Alfred Street where the catchment has a lower slope and is conveying greater flow. The modelling shows that water ponds in the Alfred Street area immediately adjacent to Circular Quay.

As presented in Section 4.3, significant drainage infrastructure exists along the alignment of this flow path, specifically:

- The Tank Stream which is aligned between George and Pitt Streets and is represented as Pipe P08 in Figure 7-2: and
- Pipe P07 (refer to Figure 7-2) which runs in a direction towards Sydney Opera House before discharging into Sydney Harbour.

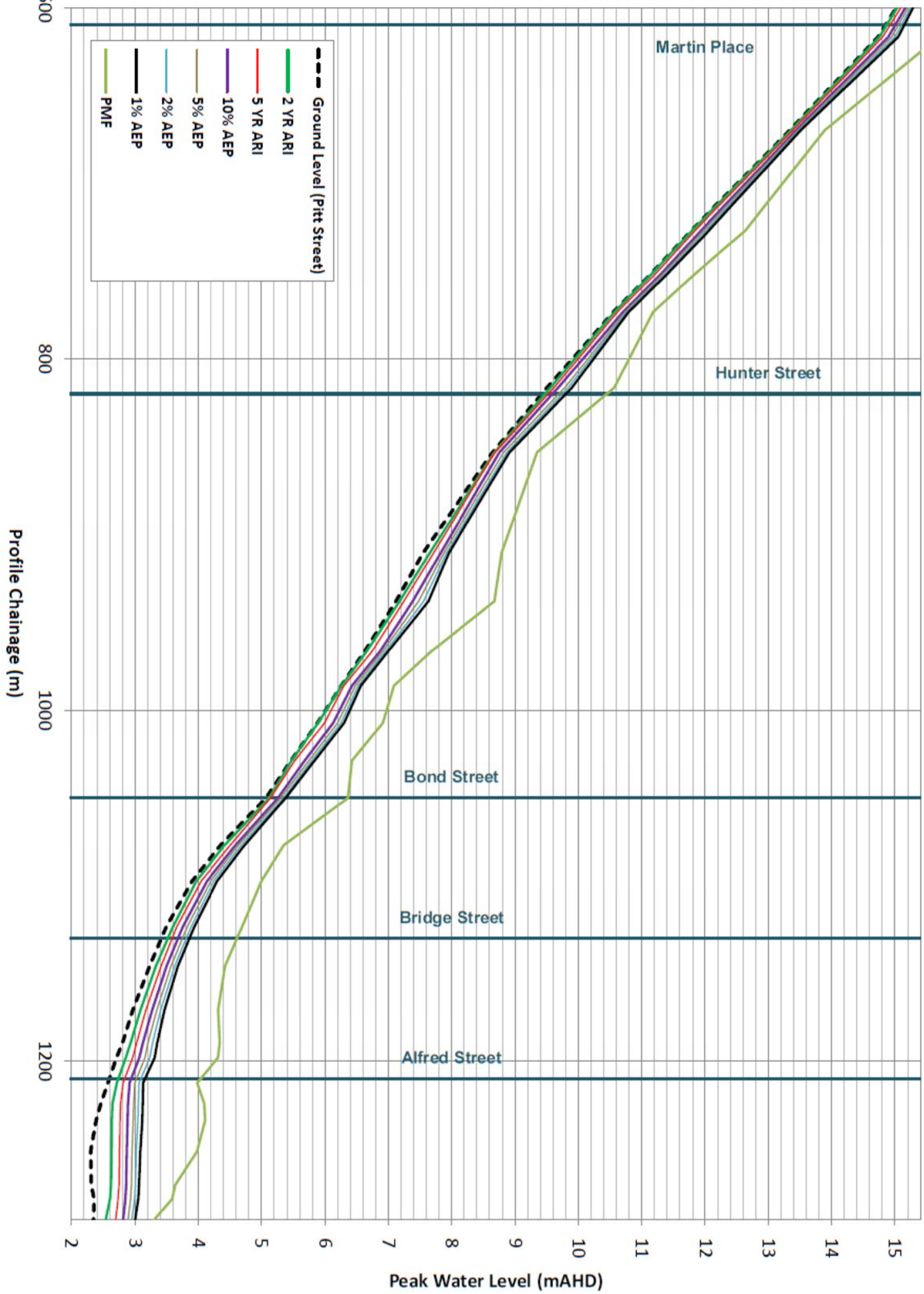
Modelling results show that both of these drainage systems are flowing approximately at capacity for the 2 year ARI event (see Table 7-2).

Overland flows in the upper reaches of the Pitt Street overland flow path range from 1.7 m^3/s in the 2 year ARI event to 9.8 m^3/s in the 1% AEP event at Pitt Street Mall (location Q03 in Table 7-2). Lower in the catchment near Bridge Street (location Q04), peak overland flows range from 1.0 m^3/s in the 2 year ARI event to 20.7 m^3/s in the 1% AEP event.

Flooding in the rest of the Sydney City Area catchment is generally a result of concentration of overland flow from localised catchments in trapped low points where limited drainage capacity currently exists.

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
PITT STREET PROFILE - DESIGN EVENT RESULTS



7.1.2 Catchment-Derived Flood Events

As presented in Section 6, a range of durations has been modelled and enveloped for each annual exceedance probability modelled. For complete catchment modelling, it is common for different durations to produce critical flood levels at different locations. Upper catchment reaches or isolated areas with small catchments will likely respond to a shorter duration event. Lower catchment reaches, catchment areas with large upstream detention volumes or large upstream areas will likely respond to longer storms with greater volume. Given a single duration is not appropriate to define critical conditions throughout the catchment, all durations are modelled and the results of each combined to form an envelope grid. This ensures that critical design flood conditions are presented in the mapping across the entire catchment.

Figure 7-4 shows the 1% AEP critical duration assessment for the City Area catchment. As shown, the majority of the catchment is critical for the 90 minutes and 120 minute duration, with localised upper catchment areas and the Walsh Bay area critical for the 25 minute storm duration.

Table 7-3 shows the differences in flood level for individual storm durations compared with the maximum flood level envelope which combines all durations. The single storm duration which most represents the maximum flood levels across the study area is the 90 minute storm. This duration has therefore been selected as the critical duration for the sensitivity analysis and climate change modelling. For all design event modelling however, all storm durations have been modelled to most accurately produce a peak flood envelope.

Table 7-3 Critical duration assessment (peak flood level difference (m) from maximum envelope)

Location#	015min	025min	030min	045min	060min	090min	120min	180min	270min	360min	540min
H01	-0.08	-0.01	-0.02	-0.04	-0.01	+0.00	-0.02	-0.13	-0.18	-0.24	-0.26
H02	-0.07	-0.02	-0.03	-0.04	-0.01	+0.00	+0.00	-0.09	-0.14	-0.20	-0.23
H03	-0.10	-0.03	-0.04	-0.05	-0.01	+0.00	+0.00	-0.11	-0.16	-0.21	-0.25
H04	-0.14	-0.05	-0.05	-0.07	-0.02	+0.00	-0.01	-0.09	-0.14	-0.22	-0.24
H05	-0.17	-0.06	-0.07	-0.06	-0.01	+0.00	+0.00	-0.11	-0.17	-0.24	-0.30
H06	-0.13	-0.05	-0.05	-0.04	-0.01	+0.00	+0.00	-0.07	-0.11	-0.15	-0.19
H07	-0.05	-0.02	-0.02	-0.02	-0.01	+0.00	+0.00	-0.06	-0.09	-0.12	-0.13
H08	-0.02	+0.00	+0.00	-0.01	-0.01	+0.00	-0.01	-0.04	-0.05	-0.07	-0.08
H09	-0.08	-0.04	-0.05	-0.05	-0.01	+0.00	-0.02	-0.06	-0.08	-0.14	-0.20
H10	-0.03	+0.00	-0.01	-0.03	-0.01	-0.01	-0.01	-0.07	-0.09	-0.12	-0.14
H11	-0.01	+0.00	+0.00	-0.01	+0.00	+0.00	-0.01	-0.04	-0.06	-0.07	-0.08
H12	-0.05	-0.01	-0.02	-0.01	+0.00	+0.00	+0.00	-0.08	-0.11	-0.14	-0.15
H13	-0.01	+0.00	-0.01	-0.02	-0.01	+0.00	-0.01	-0.06	-0.07	-0.10	-0.11
H14	-0.08	-0.04	-0.03	-0.02	-0.01	-0.01	+0.00	-0.03	-0.03	-0.04	-0.05

Refer to Figure 7-1 for the reporting locations

The design flood results, as presented in a flood mapping series in Appendix A, are the maximum condition for all of the modelled durations. For each of the simulated design events, a map of peak flood level, depth and velocity is presented covering the study area.