7 Model Calibration and Validation

Model calibration, where data supports this, is achieved through carrying out simulations of recorded flood events and then making adjustments to the model parameters through the comparison of observed and modelled results. Often the variables are interdependent, but are also not necessarily constant between events. Therefore, model validation is undertaken to test the appropriateness of the adopted calibration parameters for different historical events and provide an indication of parameter variability.

Model calibration and validation depends on several factors, such as:

- The availability, completeness and quality of rainfall and flood level event data;
- The amount of reliable data collected during the historical flood information survey;
- The variability of events – preferably events would cover a range of flood sizes
- The geographic coverage of data available for each event; and
- The underlying data used in the development of the models.

7.1 Selection of Calibration and Validation Events

The selection of suitable historical events for calibration and validation is largely dependent on the availability of relevant historical flood information. Ideally the calibration and validation process should cover a range of flood magnitudes to demonstrate the suitability of a model for the range of design events to be considered.

Table 7-1 lists the three events identified for model calibration and validation which vary in both magnitude and availability of historical data.

Table 7-1 Calibration and Validation Events

Date of Event	Calibration or Validation
17 August 1998	Calibration
23-24 February 2013	Validation
30 April 1988	Validation

The August 1998 calibration event generally equates to a 100 year ARI, with the February 2013 event generally equating to a 1 year ARI and the April 1988 event generally equating to a 10 year ARI.

Historic rainfall and tidal data have been used to derive the boundary conditions applied in the hydraulic model. Observed flood information in the form of flood levels and flood mechanisms have been used to check the performance of the hydraulic model for the calibration and validation events. The historic data, observed flood information, modelling approach and model results for each of these calibration and validation events are discussed in further detail in Sections 7.2, 7.3 and 7.4.

The August 1998 flood has been used as the principal calibration event, given the availability and completeness of rainfall and tidal data. An extensive record of historical flood levels and anecdotal evidence on observed flood mechanisms are also available across an extensive coverage of the study area. The August 1998 event is the largest of the three flood events with significant out of bank flooding across the study area. The use of this larger flood event allows calibration of the model to both in-bank and out of bank flows and provides confidence in the model results for the less frequent flood event range.

As model calibration and validation depends on several factors, including the reliability of the historical data sets and underlying data used in the development of the models, it is important to acknowledge the limitation of the calibration process. All of the model parameters have been kept within normal bounds generally considered for a catchment flood study of this nature. Further consideration has been given to sensitivity testing of key model parameters on design flood conditions as presented in Section 10.

7.2 August 1998 Model Calibration

The August 1998 flood has been used as the principal calibration event, given the scale of the event and availability of rainfall, tidal and historical flood levels.

As evident from historic records of the August 1998 flood event, the total depth and pattern of rainfall together with the degree of blockages at bridges and culverts were key influences on peak flood levels in the study area during this event. The blockage of culverts is therefore one of the principal calibration parameters within the hydraulic model for this events as it has a major influence on flow routing and flood levels.

7.2.1 Rainfall Data

The 17 August 1998 storm was characterised by a strip of very severe rainfall along the Illawarra Escarpment particularly over the central and northern suburbs of Wollongong. The most significant recorded rainfall intensities occurred over a 3 to 6 hour period with some rainfall stations recording intensities with an event magnitude in the order of 100 years ARI. The highest 24 hour rainfall total was recorded at the station at Mt Ousley (442.6 mm up to 09:00 on Tuesday 18 August 1998). The event had a significant rainfall intensity gradient increasing from the coastline toward the escarpment.

Rainfall data are available from a number of daily and pluviograph stations in the Wollongong region for the 1998 event. Historically, flooding within the Hewitts Creek catchment is caused by intense rainfall bursts over durations of less than 1 day. The pluviograph data have therefore been used as the primary rainfall dataset within the WBNM model for generating model flows as it captures the rainfall patterns throughout the flood event. The daily rainfall totals have been used to inform the development of rainfall isohyets for the August 1998 flood event (see Figure 7-3).

Data from two pluviograph stations within the Hewitts Creek catchment were used for calibration of the model. The Bulli Pass station is located in the Illawarra Escarpment in the west of the study area while the Thirroul Bowling Club station is located in the northeast of the study area in the Thomas Gibson Creek sub-catchment (see Figure 3-3 for the location of these stations within the study area). Hyetographs of the 1998 storm recorded by these stations are shown in Figure 7-1 for a 5 hour period from 15:00 on the 17 August 1998. This period represented the peak of the rainfall during the 1998 flood event.

It is evident from the hyetograph that there are differences in both the timing and amount of rainfall across the study area particularly towards the peak of the rainfall event in the late afternoon. The timing of the rainfall will vary as the storm passes over the study area. The spatial distribution of the rain gauges in the study area captures this variability in the timing of the rainfall. The main rainfall burst for the event occurred in the upper catchment during a one-hour period from approximately 18:20 on the 17 August 1998. A rainfall depth of 102.5 mm was recorded at the Bulli Pass station during this one-hour period. The peak 5 minute rainfall depth was recorded at Thirroul Bowling Club with a depth of 17mm at 19:55. The 24 hour totals (from 9am on the 17 of August) were 411.5mm and 330mm for Bulli Pass and Thirroul Bowling Club stations respectively.

To gain an appreciation of the relative intensity of the August 1998 event, the recorded rainfall depths for various storm durations were compared with the design IFD data for the Hewitts Creek study area as shown in Figure 7-2.

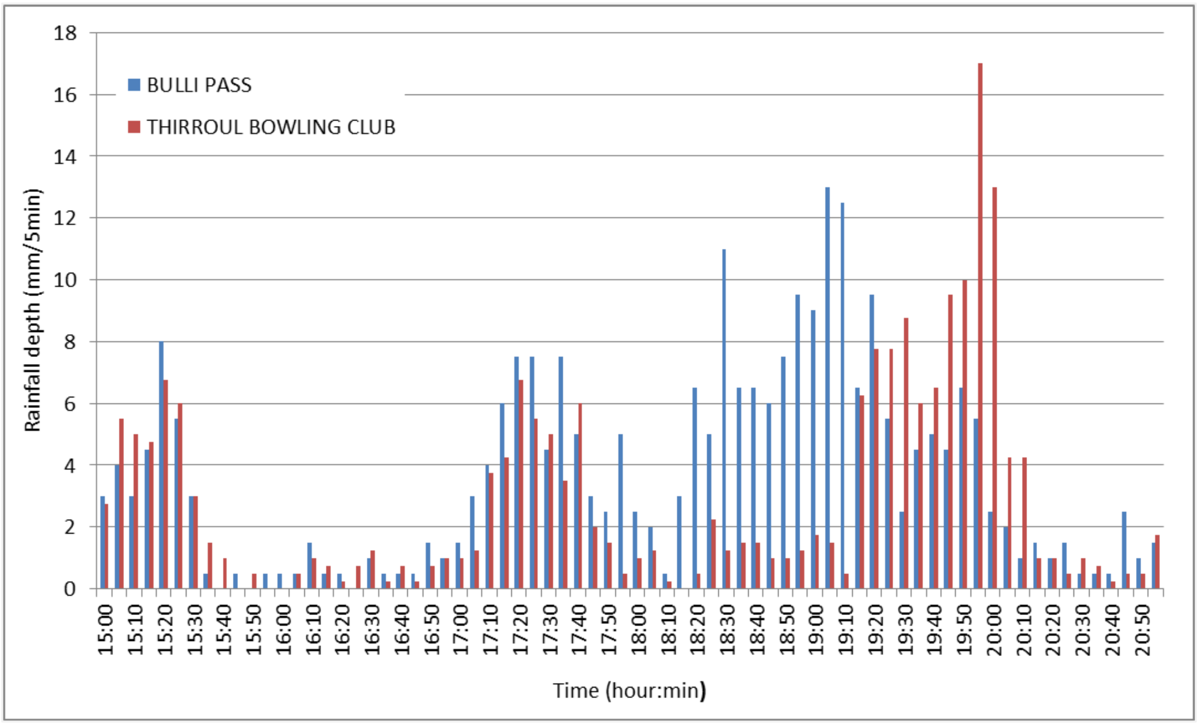


Figure 7-1 Hyetographs for Pluviograph Stations within the Hewitts Creek Catchment – 17 August 1998 Event

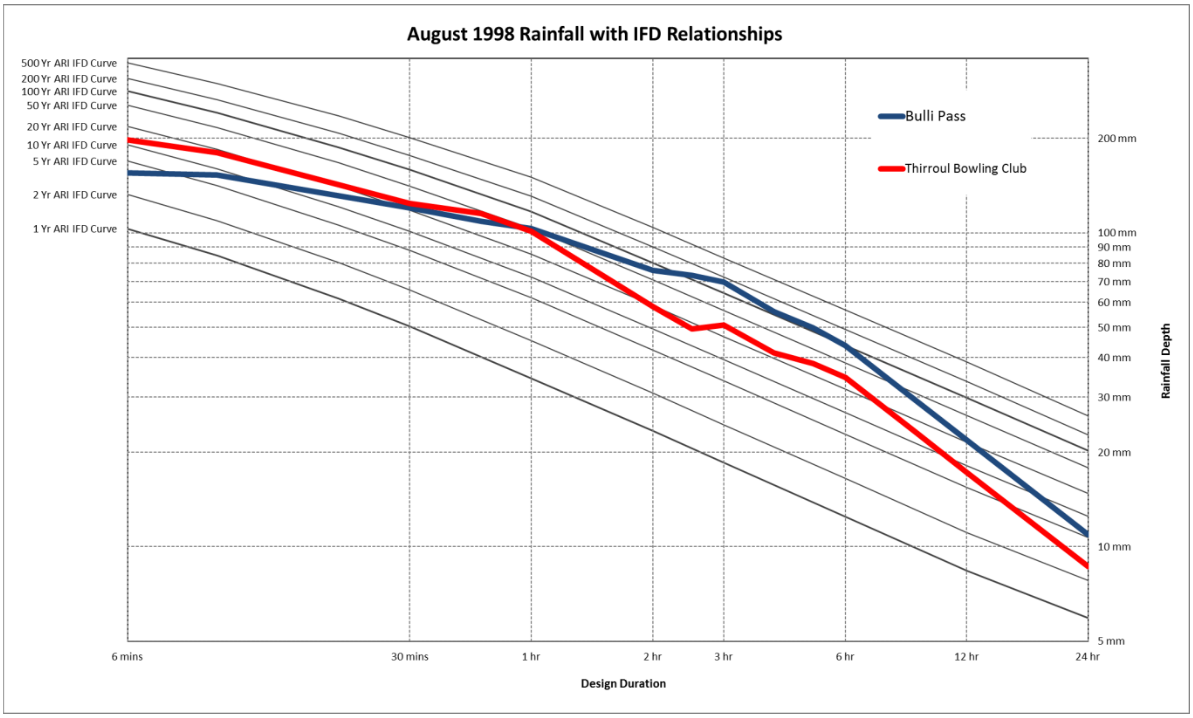


Figure 7-2 Comparison of August 1998 Rainfall with IFD Relationships

The August 1998 event generally tracks between a 100 year ARI and a 200 year ARI design rainfall depth in the upper reaches of the study area at Bulli Pass between the 2 hour and 6 hour design durations. At the Thirroul Bowling Club gauge, in the lower reaches of the study area, the August 1998 event generally tracks between a 20 year ARI and a 50 year ARI design rainfall depth. For the upper reaches of the study area, the following comparisons to design rainfall depths can be made for the Bulli Pass Station:

- 6-hour duration – 262mm recorded compared with 262mm design 100 year ARI;
- 12-hour duration – 261.9mm recorded compared with 360mm design 100 year ARI; and
- 24-hour duration – 262.1mm recorded compared with 480mm design 100 year ARI.

Rainfall isohyets were estimated based on recorded 24 hour rainfall totals for the 1998 event as shown in Figure 7-3. The spatial rainfall distribution within the WBNM model has been developed using these isohyets to factor the pluviograph rainfall patterns from these two stations.

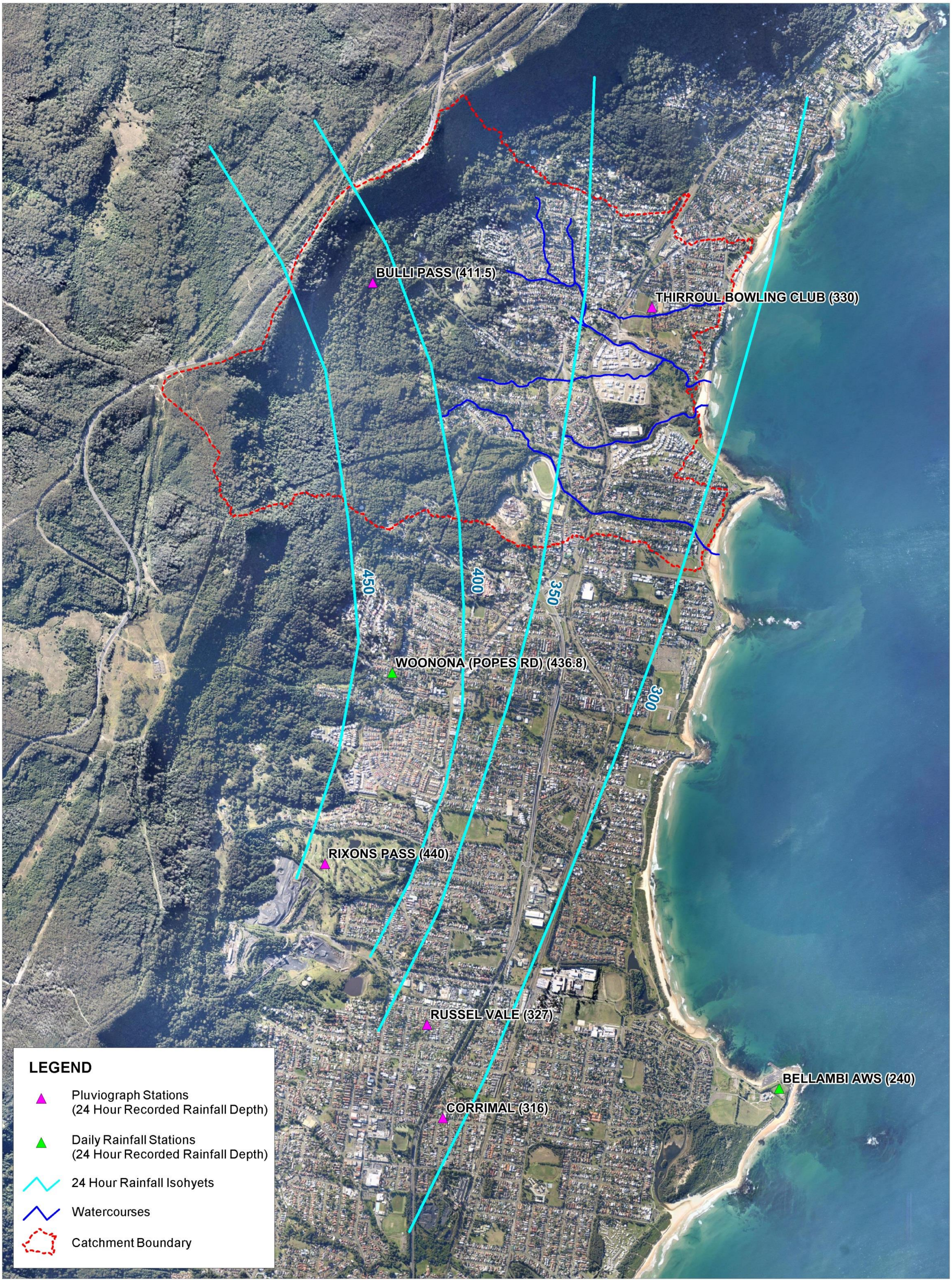
7.2.2 Antecedent Conditions

The antecedent catchment condition reflecting the degree of wetness of the catchment prior to a major rainfall event directly influences the magnitude and rate of runoff. The initial loss-continuing loss model has been adopted in the WBNM hydrologic model developed for the Hewitts Creek catchment.

The event of 17 August 1998 was the second of two significant rain events affecting the Illawarra coast in August 1998. The first occurred during the period 6 – 8 August. The Flood Data Report on 17 August 1998 Storms (DLWC & Wollongong City Council, 2002) notes that the rainfall totals were already well above average for the month, in some cases exceeding over three times the average before the start of the of the flood event leaving the area in a highly saturated state. Table 7-2 provides details of the daily rainfall totals for stations in the vicinity of the Hewitts Creek Catchment in the days leading up to the flood event.

Table 7-2 Rainfall Totals in Days Leading up to the 17 August 1998 Event

Station	Total Rainfall Depth (mm)	
	24hr to 09:00 on the 16 August 1998	24hr to 09:00 on the 17 August 1998
Bulli Pass	103.5	61.5
Rixons Pass	102.5	67
Bellambi	66.5	41



Title:

August 1998 Rainfall Isohyets

Figure: 7-3

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7.2.3 Downstream Boundary Condition

The time-varying downstream water level boundary applied in the TUFLOW model for the August 1998 event is based on tidal data from the Port Kembla tidal gauge. The relationship between observed tide levels at Port Kembla and recorded rainfall at Thirroul Bowling Club and Bulli Pass for a 5 hour period from 15:00 on 17 August 1998 is shown in Figure 7-4.

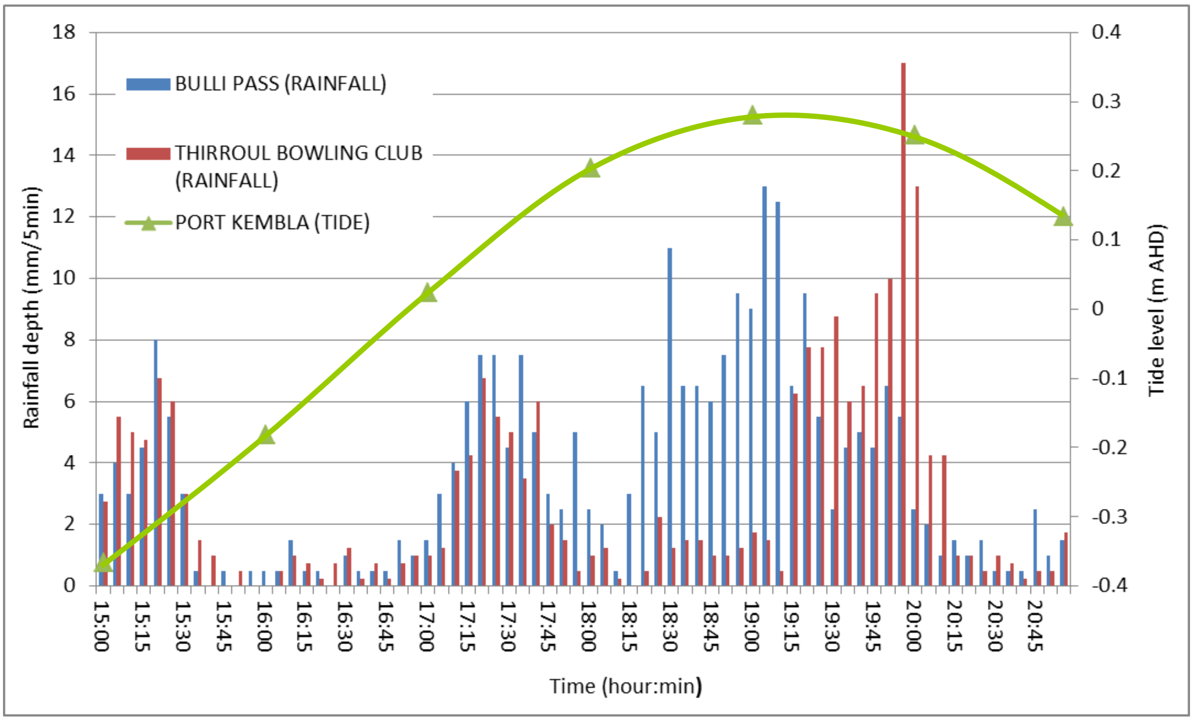


Figure 7-4 Comparison of Observed Rainfall and Tidal Data - 17 August 1998 Event

The chart indicates that the peak tide occurred during the main rainfall burst at Bulli Pass. The peak of the flood at the entrance to the creeks is likely to have coincided with the ebbing tide and the entrance conditions would have been controlled by peak flood flows in the creek channels. The flows and levels would in turn have been affected by the degree of scour of the channel and beach berm at the entrance to the creeks, which is discussed further in Section 7.2.4.

7.2.4 Adopted Model Parameters

7.2.4.1 Lag Parameter

A Lag Parameter value of 1.29 has been adopted for this study which is consistent with the calibrated and validated parameter used as part of the Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a) and the Fairy and Cabbage Tree Creeks Flood Studies (BMT WBM Pty Ltd, 2009). For the upper reaches of the study area, the WBNM model has been used to provide the total flows from the sub-areas representing the Illawarra Escarpment. Downstream of the Illawarra Escarpment, the WBNM model has been used to provide local flow inputs into the TUFLOW hydraulic model at the various sub-area outlets. The TUFLOW hydraulic model simulates the behaviour of the runoff from the hydrological model by routing the flow hydrographs through the two dimensional grid of the study area. As the routing of the sub-area flows is being undertaken within the hydraulic model and not internally routed through the WBNM model, changing the Lag Parameter within the WBNM model will have a negligible impact on the timing and shape of the flow hydrograph from sub-catchments downstream of the Illawarra Escarpment.

7.2.4.2 Blockages of Culverts and Bridges

Culvert and bridge blockages were a key influence on peak flood levels in the study area during the August 1998 event and have therefore been used as a key calibration parameter within the hydraulic model. It was readily apparent from inspections carried out on the days after 17 August 1998 flood, from residents' recollections and from both residents' and Council's photographs, that flood levels had been elevated due to the very large volumes of flood-borne debris that were swept through the catchment during the event. The vast volumes of debris had partially blocked and, in many cases, fully blocked culverts and bridges. The adopted calibration approach involved modelling a number of blockage scenarios with reference to observed blockage information where this was available until an acceptable level of agreement between observed and modelled levels was reached. This approach has some uncertainties, particularly as little was known of the degree of blockages at many of the structures within the study area.

Table 7-3 lists the degree of blockages applied to each of the structures in the TUFLOW model based on the final model calibration results.

Table 7-3 Modelled Structure Blockage - August 1998 Event

Watercourse	Street	Structure Type	Structure ID (Refer to Figure 6-2)	% Blockage Applied to Structure (Current Flood Study, 2013)
Hewitts Creek	51 George Street, Thirroul	Bridge	19	85
Hewitts Creek	47 George Street, Thirroul	Bridge	20	90
Hewitts Creek	Kelton Lane, Thirroul	Bridge	21	50
Hewitts Creek	Lachlan Street, Thirroul	Culvert	22	75
Hewitts Creek	Lawrence Hargrave Drive, Thirroul	Culvert	23	100
Hewitts Creek	Illawarra Railway	Bridge	26	0
Hewitts Creek	Downstream of Illawarra Railway near Lawrence Hargrave Drive, Thirroul	Bridge	Calibration model only. ID not included in Figure 6-2.	0
Hewitts Creek	Downstream of Illawarra Railway near Lawrence Hargrave Drive, Thirroul	Bridge	Calibration model only. ID not included in Figure 6-2.	0
Hewitts Creek (eastern tributary)	Palm Grove, Thirroul	Culvert	29	100

Watercourse	Street	Structure Type	Structure ID (Refer to Figure 6-2)	% Blockage Applied to Structure (Current Flood Study, 2013)
Hewitts Creek (eastern tributary)	Virginia Terrace, Thirroul	Culvert	30	100
Hewitts Creek (eastern tributary)	George Street, Thirroul	Culvert	31	100
Hewitts Creek (western tributary)	Deborah Avenue, Thirroul	Culvert	32	100
Hewitts Creek (western tributary)	Virginia Terrace, Thirroul	Culvert	33	100
Hewitts Creek (western tributary)	George Street (West), Thirroul	Culvert	34	100
Hewitts Creek	Hamilton Road, Thirroul	Foot Bridge	28	0
Hewitts Creek	Lawrence Hargrave Drive, Thirroul	Culvert	24	100
Hewitts Creek	High Street, Thirroul	Culvert	25	100
Thomas Gibson Creek	Lawrence Hargrave Drive, Thirroul	Culvert	35	0
Thomas Gibson Creek	Illawarra Railway, Thirroul	Culvert	36	20
Thomas Gibson Creek	Thomas Gibson Park, Thirroul	Culvert	37	100
Thomas Gibson Creek	McCauley Street, Thirroul	Culvert	38	60
Thomas Gibson Creek	Cliff Parade, Thirroul	Culvert	39	0
Woodlands Creek	Princes Highway, Bulli	Culvert	15	0
Woodlands Creek	Disused heavy vehicle safety ramp at Princes Highway, Bulli	Culvert	16	0
Woodlands Creek	Illawarra Railway, Bulli	Culvert	17	0

Watercourse	Street	Structure Type	Structure ID (Refer to Figure 6-2)	% Blockage Applied to Structure (Current Flood Study, 2013)
Woodlands Creek	McCauley Beach Estate, Bulli	Culvert	Calibration model only. ID not included in Figure 6-2.	0
Tramway Creek	Illawarra Railway, Bulli	Culvert	14	60
Slacky Creek	William Street, Bulli	Culvert	1	90
Slacky Creek	Hobart Street, Bulli	Culvert	2	0
Slacky Creek, (western tributary)	Hobart Street, Bulli	Culvert	3	0
Slacky Creek (western tributary)	Hobart Street, Bulli	Culvert	4	0
Slacky Creek	Hobart Street, Bulli	Culvert	5	0
Slacky Creek	Hobart Street (coal haulage embankment), Bulli	Culvert	6	0
Slacky Creek	Adjacent to Bulli Showground and Racing Complex, Bulli	Culvert	7	100
Slacky Creek	Princes Highway, Bulli	Culvert	8	100
Slacky Creek	Park at Black Diamond Place, upstream of Illawarra Railway, Bulli	Culvert	9	0
Slacky Creek	Illawarra Railway (Creek opening and Beacon Avenue underpass), Bulli	Culvert	10	0
Slacky Creek	Just south of Beach Street, Bulli	Bridge	11	0
Slacky Creek	Blackall Street. Bulli	Bridge	12	50

Structure ID's 13, 18 and 27 as listed in Table 6-6 were not included in the hydraulic model for the 17 August 1998 calibration event as these were not in place at the time of this flood event.

7.2.4.3 Rainfall Losses

Based on a review of the antecedent conditions and the loss rate adopted as part of the Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a), the loss rates detailed in Table 7-4 have been adopted for the August 1998 flood event as part of the current flood study.

Table 7-4 Rainfall Loss Rates - August 1998 Event

Rainfall Loss Type	Surface Type	Values (Current Flood Study, 2013)	Value (Hewitts Creek Flood Study, Forbes Rigby Pty Ltd., 2002a)
Initial Loss	Pervious	0 mm	0 mm
Initial Loss	Impervious	0 mm	0 mm
Continuing Loss	Pervious	2.5 mm/h	2.5 mm/h
Continuing Loss	Impervious	0 mm/h	0 mm/h

Based on the model results, the rainfall loss rates adopted from the Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a), as listed in Table 7-4, provided satisfactory results when comparing oberved and modelled flood levels for the calibration and validation events. For the 2013 validation event, a comparison of the modelled and observed flood levels at the OEH gauge upstream of Lawrence Hargrave Drive, Thirroul, on Hewitts Creek indicates that the model provides a relatively good rate of rise, shape, peak and volume when compared to the recorded data (refer to Section 7.3.5.1) indicating the loss rates adopted are appropriate.

7.2.4.4 Bathymetry of the Creek Entrances

The geometry of the channels at the entrances of the creeks affects the levels and flows along the downstream reaches of the study area. The entrances to the creeks naturally open and close to the ocean predominantly as a result of sediment transport, tidal, wave, wind and fluvial process. The entrances will naturally close as waves and wind deposit sand in the entrances forming a berm. Runoff from the catchment and/or saline water from waves then builds up on the catchment side until the ponding level is higher than the berm. Naturally when the water level gets higher than the berm, the sand is scoured and the creek is connected to the ocean. This can also occur at lower levels if citizens illegally dig a channel through the berm. The entrance will then close over time.

The highly dynamic nature of the entrances of the creeks with respect to beach berm patterns presents challenges in defining appropriate initial conditions of the entrance channel geometry for hydraulic modelling. The entrances of the creeks have been modelled as fixed with the geometry of the entrances defined at the start of the flood event.

Observed flood level data along the downstream reaches of Slacky Creek, Hewitts Creek and Flanagans Creek was used to inform the most appropriate geometry of the entrance to the creeks. A number of model simulations were undertaken with various fixed entrance dimensions until an acceptable level of agreement between observed and modelled water levels along the downstream reaches was reached.

The lateral width at the entrances to the creeks was limited by the extent of the coastal dunes. The minimum elevation of the entrance to the creeks was limited to 0m AHD where a 'control' on the maximum depth of erosion is inferred to occur by the presence of a rock shelf (Cardno Lawson Treloar Pty Ltd., 2007). Based on a comparison of the modelled and observed levels, the following minimum reduced levels have been adopted at the entrance to Slacky Creek, Tramway Creek, Hewitts Creek and Flanagans Creek:

- Slacky Creek – 1 m AHD;

- Tramway Creek – 0.8m AHD;
- Hewitts Creek – 0.1m AHD; and
- Flanagans Creek – 0.6m AHD.

The entrance to Thomas Gibson Creek appears less affected by the build-up of beach berm due to the location of the entrance where it discharges at Thirroul Beach. Therefore, the reduced levels at the entrance of this creek have not been adjusted as part of the model calibration.

Woodlands Creek forms a tributary of Hewitts Creek approximately 0.3km upstream of McCauley's Beach and is therefore not affected by the coastal processes which affect the entrances to the other creeks in the study area.

7.2.4.5 Hydraulic Roughness

The various land surface types which define the hydraulic roughness zones have been determined using aerial photography, cadastral data and site visit notes. Google Earth Aerial imagery has been used to identify significant land use changes in the study area and adjustments to the roughness zones have been made accordingly for the 1998 calibration event and 1988 validation event. For these events, the land use at McCauley Beach Estate, Thirroul, has been changed from residential urban blocks to parkland, to reflect the land use type prior to the development of this site.

The values of the roughness coefficients have been based on industry standards (e.g. Chow, 1959 and Arcement and Schneider, 1989) and values adopted in previous TUFLOW models developed by BMT WBM. Given the confidence in the values applied, limited adjustments have been made to the roughness co-efficient as part of the model calibration process.

The adopted Manning's 'n' roughness coefficients for the land uses within the study area are listed in Table 7-5. For verification of the roughness coefficients a comparison was undertaken with values used in the HEC RAS model from the Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a) and 2D hydraulic models for flood studies in the Wollongong region. The hydraulic roughness coefficients applied in previous studies are summarised in Table 7-6. Comparing roughness coefficients for the various land use types in Table 7-5 and Table 7-6 shows a close similarity in adopted roughness coefficients.

Table 7-5 Adopted Manning's 'n' Roughness Coefficients in the TUFLOW Model

Land Use	Manning's 'n'
Grass (maintained)	0.030
Parkland	0.040
Dense vegetation	0.090
Riparian corridor	0.090
Creek channel	0.060 – 0.120
Tidal inundation zone	0.035
Roads, car parks, open concrete	0.020
Railway corridor	0.080
Buildings	1.000
Urban blocks (brick walls and fences)	0.070

Table 7-6 Manning's 'n' Roughness Coefficients for other Wollongong LGA Flood Studies

Land Use	Manning's 'n'
Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a) – Modelled using HEC RAS	
Creek channels	0.02 – 0.10
Floodplain	0.02 – 0.150
Combined Catchments of Whartons, Collins and Farrahars Creeks, Bellambi Gully and Bellambi Lake Flood Study (Lyall & Associates, 2011) – Modelled using TUFLOW	
Roads (concrete)	0.02
Well Maintained Grass (e.g. golf course)	0.03
Grass (lawns)	0.045
Thick Vegetation	0.100
Very dense vegetation	0.2
Fenced properties	1
Buildings	1-10

7.2.5 August 1998 Model Calibration Results

Following the August 1998 flood event, a significant amount of data on observed flood levels were captured in the study area from flood marks left by the flood. Council undertook land surveys shortly after the 17 August 1998 storm event to capture the level of the flood marks. At that time, Council also issued questionnaires to residents in flood affected areas to gain further knowledge on the flood heights, flood mechanisms and damages to private and commercial property. This information was subsequently used to conduct further surveys of historical flood levels. The information related to the survey of flood levels has been entered into a Geographical Information System (GIS) database which provided details on the spatial location and observed levels for this flood event. Detailed descriptions of the flooding, provided through the community questionnaires, has been spatially referenced and used to validate the modelled flood mechanisms.

Longitudinal profiles showing the peak water surface for the 17 August 1998 flood along the modelled creek reaches for the current Flood Study and the 2002 Flood Study are presented in Appendix C. These profiles include observed flood levels located either directly along (i.e. at structures) or adjacent to (i.e. creek banks) the modelled creeks. The observed flood mark ID is provided on these long section profiles for cross reference against the observed flood mark ID in Figure 7-5.1 to Figure 7-5.10.

7.2.5.1 Characteristics of Flooding for the August 1998 Flood Event

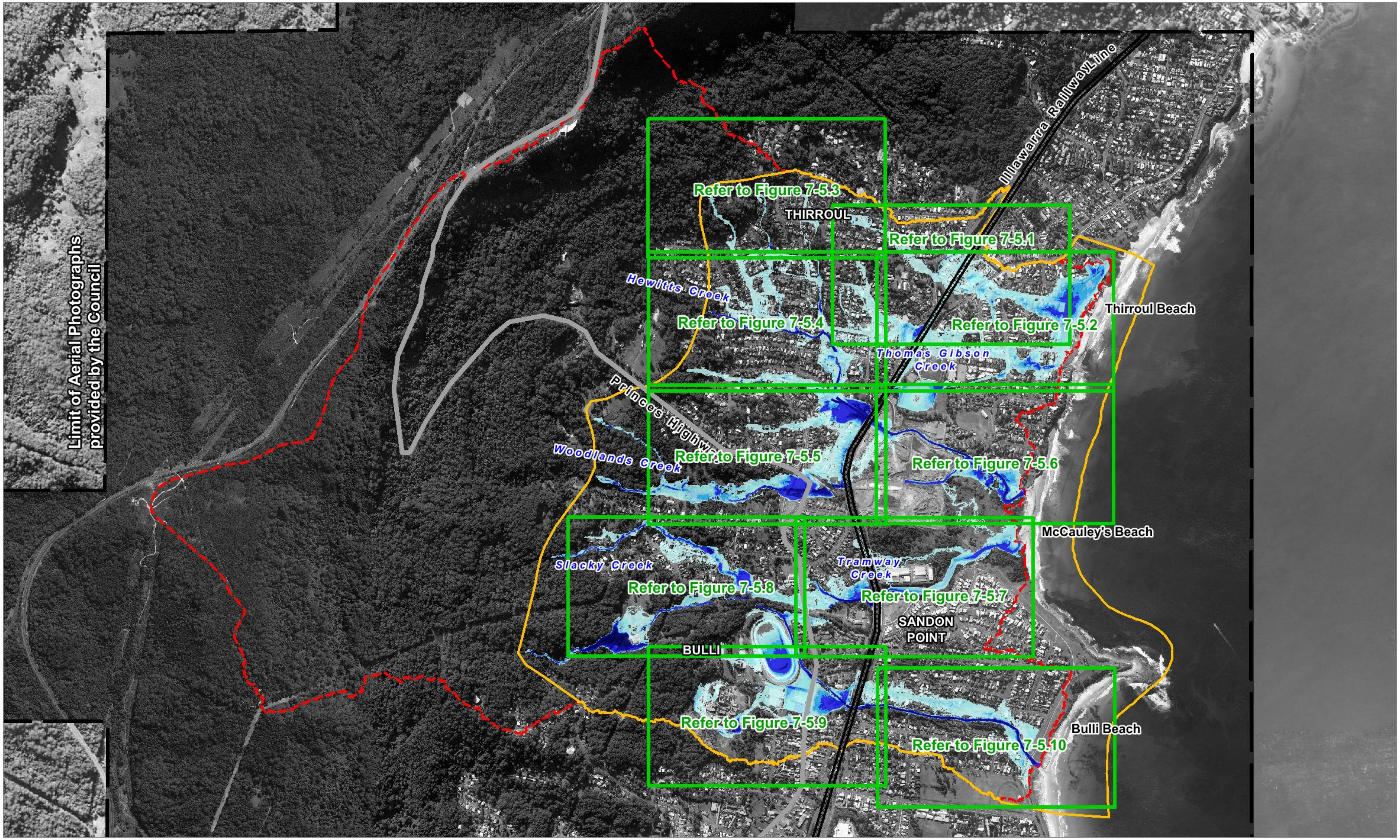
Figure 7-5 is a key map showing the layout of ten plans covering the modelled creek reaches. These plans show:

- The depth and extent of the modelled flood envelope together with the direction of the modelled flows, as indicated by flow velocity vectors;

- The locations of the observed flood levels with a flood level ID linked to a map table showing the observed flood level, the corresponding peak modelled level and the difference between observed and modelled levels; and

- Information on the modelled flood mechanisms through the use of flow velocity vectors. A comparison of these modelled flood mechanisms with observed flood mechanisms is provided in Appendix B.

Further discussion on the comparison between the modelled peak flood levels and observed flood levels and details of the flood mechanisms for each of the ten plan areas is detailed in the remainder of this section.



LEGEND

- TUFLOW Model Boundary
- Catchment Boundary
- Map Windows

Peak Flood Depths (m)

- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

Title:
Key Map for August 1998 Flood Event

Figure: 7-5

Rev: 1

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Approx. Scale

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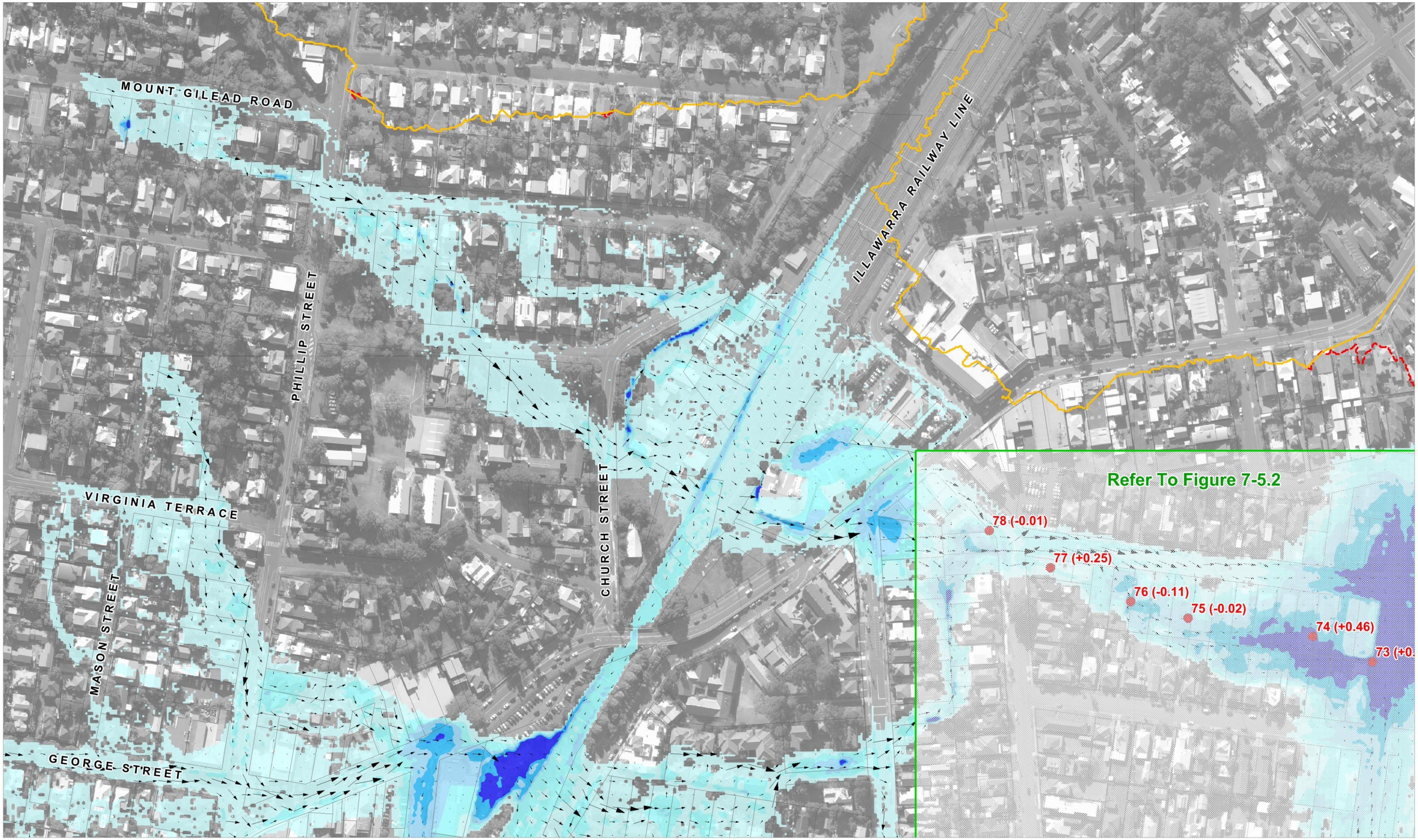

LEGEND

Flow Velocity Vector

Cadastre

TUFLOW Model Boundary

Catchment Boundary

Peak Flood Depths (m)

- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

Title:

Characteristics of Flooding in the Upper Reaches of Thomas Gibson Creek for the August 1998 Flood Event

Figure: 7-5.1

Rev: 0

N

0 50 100m

Approx. Scale

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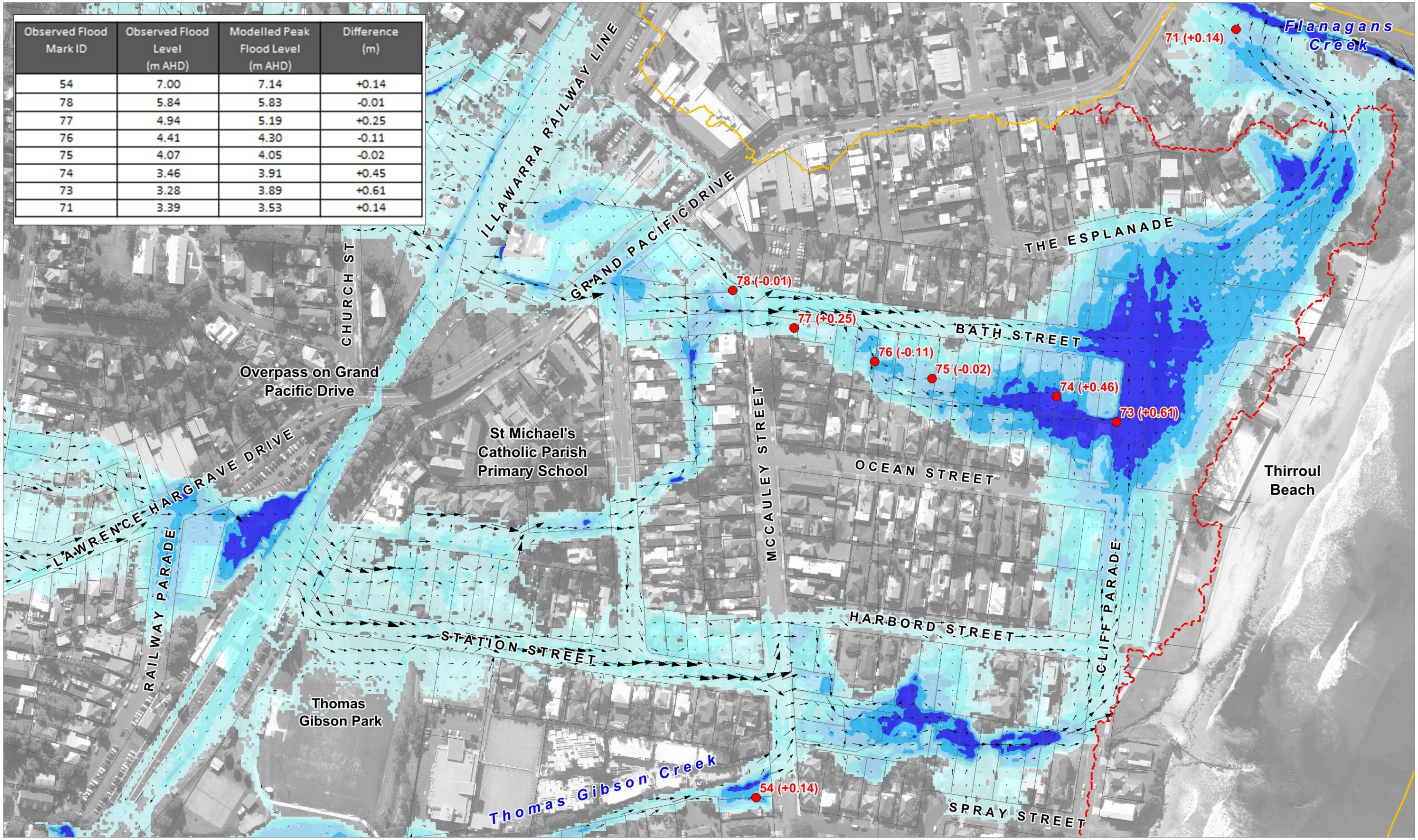
Characteristics of Flooding in Upper Reaches of Thomas Gibson Creek (Refer to Figure 7-5.1)

Upstream of the Illawarra Railway, Thirroul, there are limited reaches of defined creek networks, with flooding in these upper reaches primarily resulting from overland flows. The model results indicate that this overland flow follows two routes upstream of the railway:

- To the northwest of this catchment, flows pass from Mount Gilead Road, Thirroul, across Philip Street, Thirroul, and Church Street, Thirroul, before crossing the Illawarra Railway north of the overpass on the Grand Pacific Drive, Thirroul.
- To the southwest of this sub- catchment, the model results indicate that the overland flows pass from Mason Street, Thirroul, across Virginia Terrace, Thirroul, and along Philip Street to Lawrence Hargrave Drive, Thirroul. The flows at Lawrence Hargrave Drive combine with flood flows from the eastern tributary of Hewitts Creek. These flows from the eastern tributary of Hewitts Creek spill onto George Street, Thirroul, at the culvert inlet and join with flows from the Thomas Gibson Creek at the intersection of George Street and Lawrence Hargrave Drive. The flood flows pass onto Railway Parade before crossing the Illawarra Railway at Thirroul Railway Station.

The modelled overland flood mechanisms correlate well with the observed flood mechanism in this area. Observed flood information indicates that there was flooding at a number of properties in Mason Street. Flood water moved in a south easterly direction from the front of 9 Mason Street and exited at the back of 15 Mason Street. Flooding caused damage to garden fences and garden sheds with reported flood depths of between 0.10m and 0.25m. Flood flows were observed to cross Virginia Terrace, Thirroul and through the front yard of the property at 126 Phillip Street, Thirroul. At 382-384 Lawrence Hargrave Drive, Thirroul, flooding was observed in the car park of the property. Flood water also reported to have entered through the back door of this property and partially inundated the ground floor.

Observed Flood Mark ID	Observed Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (m)
54	7.00	7.14	+0.14
78	5.84	5.83	-0.01
77	4.94	5.19	+0.25
76	4.41	4.30	-0.11
75	4.07	4.05	-0.02
74	3.46	3.91	+0.45
73	3.28	3.89	+0.61
71	3.39	3.53	+0.14



LEGEND

- Observed Flood Mark
- 74 (0.46) — ID Number (Difference between Observed and Modelled Level)
- Flow Velocity Vector
- Cadastre
- TUFLOW Model Boundary
- Catchment Boundary

Peak Flood Depths (m)

- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

Title:
Characteristics of Flooding in the Lower Reaches of Thomas Gibson Creek for the August 1998 Flood Event

Figure: 7-5.2

Rev: 2

N

0 50 100m
Approx. Scale

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Characteristics of Flooding in the Lower Reaches of Thomas Gibson Creek (Refer to Figure 7-5.2)

The TUFLOW model was found to correlate well with the majority of the observed flood marks in this area with the modelled peak flood levels generally within a ± 0.3m tolerance of the observed flood levels. Although there are some localised variations between the modelled and observed levels, the overall fit in this area suggests that the flows and hydraulic model parameters are of the correct order.

To the south of the sub-catchment, east of the Illawarra Railway, Thomas Gibson Creek has a more defined channel than elsewhere within this sub-catchment. This channel runs from Lawrence Hargrave Drive, Thirroul, through Thomas Gibson Park, Thirroul, and discharges to Thirroul Beach at Cliff Parade, Thirroul. Flooding was observed at a number of properties along this reach in the 1998 flood event.

- At Newbold Close, Thirroul, the creek is formed as an overland floodway through the rear gardens of these properties where flood water rose to an observed depth of approximately 1.0 meter along the floodway;
- At 42 McCauley Street, Thirroul, the resident reported flood water inundation of 0.1m in their garage with water running through the grounds of their property passing from upstream at McCauley Street, Thirroul and exiting at the northeast corner of their property. Towards the downstream end of this reach, flooding was observed at two properties on Spray Street, Thirroul;
- At 3 Spray Street, Thirroul, the residents reported that water flowed from the street down the eastern side of their property; and
- The occupier at 5 Spray Street, Thirroul, reported overland flows originating from Spray Street, Thirroul, passing through their property and exiting to Thomas Gibson Creek at the rear of their house.

The modelled flood mechanisms and flood levels correlate well with the observed flood information along this reach. The culverts at the upstream extent of this creek at Lawrence Hargrave Drive, Thirroul, the Illawarra Railway, Thirroul and Thomas Gibson Park, Thirroul have been modelled with 0%, 20% and 100% blockages respectively. In order to replicate the observed flood levels at McCauley Street, Thirroul, a 60% blockage has been applied to this structure.

North of this open channel reach, the model results indicate that the overland flows east of the Illawarra Railway primarily follow two overland flow paths along Station Street, Thirroul, and Bath Street, Thirroul. Flows along Station Street combine with the open channel creek flows at McCauley Street. These flows pass east along Harbord Street, Thirroul and the open creek channel before combining again at Cliff Parade, Thirroul. The flows then follow a northwards path along Cliff Parade, Thirroul and join with the overland flows from Bath Street, Thirroul, with diverted flows continuing northwards to Flanagan's Creek at The Esplanade, Thirroul. Flooding along Cliff Parade, Thirroul and surrounding streets is affected by the inadequate capacity of the drainage system. There are no observed flood mechanisms for this event along this reach to correlate the modelled flood mechanisms; however the majority of the observed flood levels correlate well with the modelled flood levels. At the eastern end of Bath Street, Thirroul, the model overestimates flooding when compared to the observed flood levels at two neighbouring properties with a maximum difference in levels of

0.62m at Map Reference ID 73 (Figure 7-5.1). It was not possible to replicate these observed levels at this location through changes to the Manning's n values, blockages applied to structures and the dimensions of the entrance to Flanagan's Creek. A 0% blockage has been applied to the structure at Cliff Parade, Thirroul, on the main channel of Thomas Gibson Creek to maximise the discharge to the Tasman Sea at the entrance to this creek and reduce spilling of flood water northwards along Cliff Parade, Thirroul. Given the good correlation between observed and modelled flood levels and mechanisms at remaining locations within this sub-catchment, the flows and hydraulic model parameters are considered to be of the correct order.

LEGEND

Flow Velocity Vector

Cadastre

TUFLOW Model Boundary

Catchment Boundary

Peak Flood Depths (m)

0.00 to 0.25
0.25 to 0.50
0.50 to 0.75
0.75 to 1.00
1.00 to 2.00
2.00 to 4.00
> 4.00

Title:

Characteristics of Flooding in the Upper Reaches of Hewitts Creek for the August 1998 Flood Event

Figure: 7-5.3

Rev: 0

N

0 50 100m

Approx. Scale

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Characteristics of Flooding in the Upper Reaches of Hewitts Creek (Refer to Figure 7-5.3)

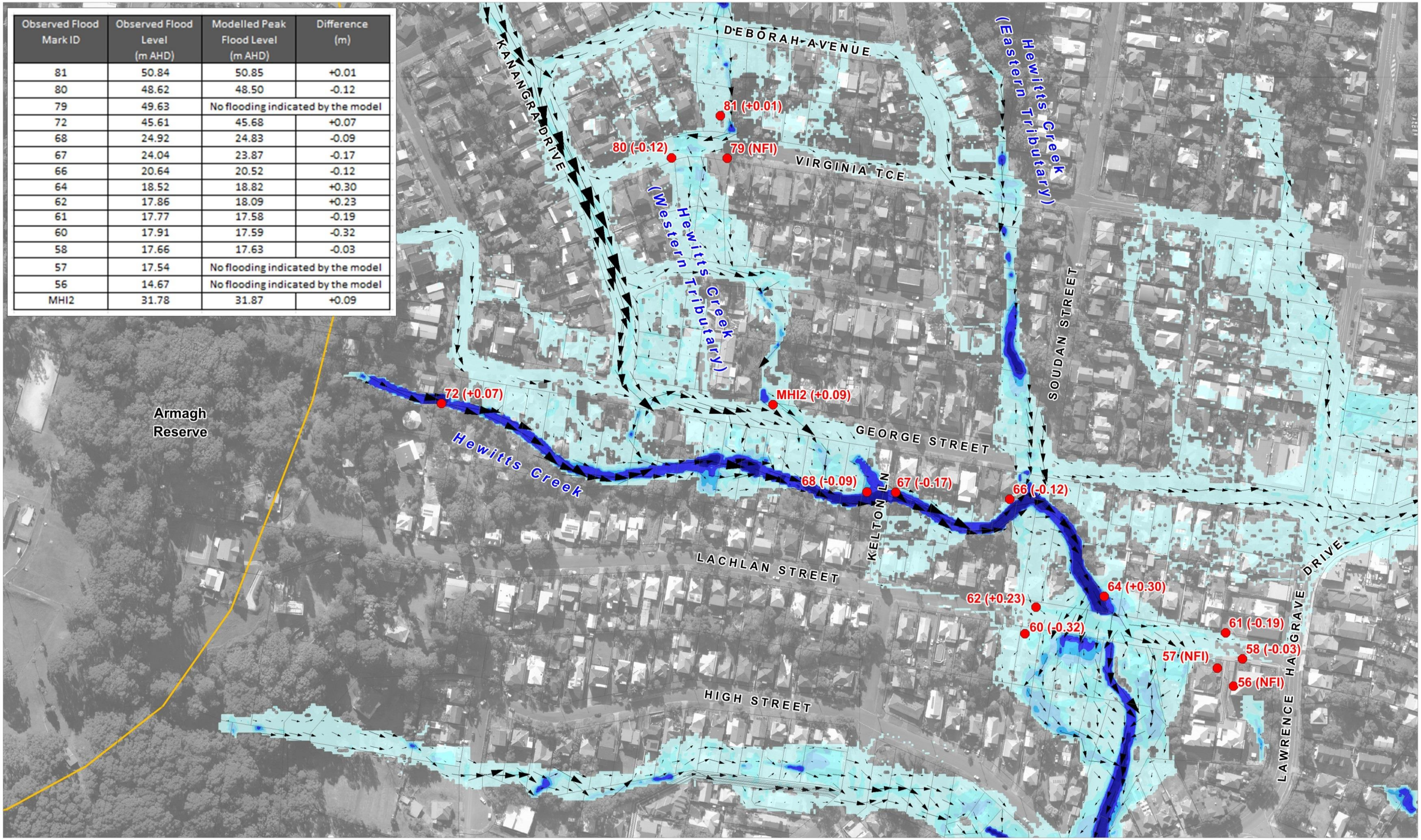
The modelled creek reaches in this area includes the upper reaches of two tributaries of Hewitts Creek which drain the northwest of the study area. The western tributary runs from north of Cornock Avenue, Thirroul, and connects with Hewitts Creek south of George Street, Thirroul upstream of Kelton Lane, Thirroul. The eastern tributary runs from Nardoo Crescent, Thirroul, to south of George Street, Thirroul, connecting with the main channel of Hewitts Creek south of the intersection of George Street and Soudan Street, Thirroul.

No observed flood mechanisms are available for the western tributary for comparison with modelled flood mechanisms. On the eastern tributary of Hewitts Creek flooding was reported at a number of locations:

- On Hicks Road, Thirroul, flood water was reported to have inundated the back gardens of properties. Water rose to a reported depth of 0.15m in the garden of 2 Hicks Road, Thirroul.
- The residents of 4 Nardoo Crescent, Thirroul, reported significant erosion of their back garden with overland flows passing from north to south across their property. The residents reported a 0.2m depth of flooding in the grounds of their property.
- Flooding was observed at 31 Arunta Drive, Thirroul, from both surface water flows off Arunta Drive, Thirroul and from out of bank flooding from the creek at the rear of the property.

The modelled flood mechanisms correlate well with the observed flood mechanisms along this tributary. There is no observed flood level information to compare with modelled levels along this tributary.

Observed Flood Mark ID	Observed Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (m)
81	50.84	50.85	+0.01
80	48.62	48.50	-0.12
79	49.63	No flooding indicated by the model	
72	45.61	45.68	+0.07
68	24.92	24.83	-0.09
67	24.04	23.87	-0.17
66	20.64	20.52	-0.12
64	18.52	18.82	+0.30
62	17.86	18.09	+0.23
61	17.77	17.58	-0.19
60	17.91	17.59	-0.32
58	17.66	17.63	-0.03
57	17.54	No flooding indicated by the model	
56	14.67	No flooding indicated by the model	
MHI2	31.78	31.87	+0.09



LEGEND

67 (-0.26) ID Number (Difference between Observed and Modelled Level)

MHI2 (0.09) Maximum Height Indicator (Difference between Observed and Modelled Level)

57 (NFI) ID Number (No Flood Indicated)

Observed Flood Mark

Flow Velocity Vector

Cadastre

TUFLOW Model Boundary

Catchment Boundary

Peak Flood Depths (m)
- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

Title:
Characteristics of Flooding in the Middle Reaches of Hewitts Creek for the August 1998 Flood Event

Figure: 7-5.4

Rev: 1

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0 50 100m
Approx. Scale

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Characteristics of Flooding in the Middle Reaches of Hewitts Creek (Refer to Figure 7-5.4)

The TUFLOW model was found to correlate well with the majority of the observed flood marks in this area with the modelled peak flood levels generally within a ± 0.3m tolerance of the observed flood levels. Although there are some localised variations between the modelled and observed levels, the overall fit in this area suggests that the flows and hydraulic model parameters are of the correct order.

The modelled creek reaches in this area include the main channel of Hewitts Creek from George Street, Thirroul, to south of Lachlan Street, Thirroul, and two tributaries of Hewitts Creek which drain the northwest of the study area. The model results also indicate two significant overland flow paths in this area along Kanangra Drive, Thirroul and to the rear of properties along High Street, Thirroul.

No observed flood mechanisms are available for the western tributary for comparison with modelled flood mechanisms. At Deborah Avenue, Thirroul, the culvert has been assumed to be fully blocked based on the size of the culvert (1.5m in diameter) and the likely debris load for this flood event along the vegetated upper reaches of this creek. With a 100% blockage of the culvert, the modelled flood mechanisms indicate that flood water spills from the culvert inlet at Deborah Avenue, Thirroul, and flows west and east along Deborah Avenue, Thirroul. These flows combine with flows from the eastern tributary at Virginia Terrace, Thirroul, and with the overland flows along Kanangra Drive, Thirroul. At Virginia Terrace, Thirroul, the model results indicate a significant volume of flows overtopping Virginia Terrace, Thirroul and passing overland through properties to Jennifer Crescent, Thirroul, where they combine with overland flow from Kanangra Drive, Thirroul. The observed flood levels at Virginia Terrace, Thirroul, correlate well with the modelled flood levels at two locations. In order to replicate these levels in the model, the culvert at Virginia Terrace, Thirroul, has been assumed to be fully blocked for this flood event. The model does not indicate flooding at one observed flood mark at this location (map Reference ID 79, Figure 7-5.2) which may be as a result of local factors that have not been included in the model (i.e. debris on the road causing water to change direction). Downstream of Jennifer Crescent, Thirroul, the modelled flows primarily remain within the creek banks before crossing George Street, Thirroul, at the culvert inlet and spilling into Hewitts Creek. One observed flood level is available upstream of the culvert inlet on George Street, Thirroul. In order to replicate the observed levels in the model at this location, this culvert has been fully blocked for this flood event.

On the eastern tributary of Hewitts Creek flooding was reported at a number of locations:

- At Virginia Terrace, Thirroul, residents reported that flooding was increased by a blockage that occurred in the stormwater pipe running under the road resulting in flood water spilling over the road.
- At 34 Soudan Street, Thirroul, the residents observed creek flows inundating the garden to the rear of their property.
- At George Street,Thirroul, the culvert was observed to be blocked and significant flows were reported to be flowing along George Street, Thirroul.

The culverts at Palm Grove, Virginia Terrace and George Street, Thirroul, have modelled with 100% blockages based on the size of the culverts (all culverts are less than 1.2m in diameter), the likely debris load for this flood event along the vegetated upper reaches of this creek and the observed blockages. The modelled flood mechanisms correlate well with the observed flood mechanisms along this tributary based on these culvert blockages. There is no observed flood level information to compare with modelled levels along this tributary.

Numerous properties were affected by flooding along George Street, Thirroul, from a combination of out of bank flows from the main channel of Hewitts Creek and overland flows along George Street, Thirroul. Numerous images following the August 1998 flood event show significant deposits of material along the creek bed and floodplain (see Figure 7-6 and Figure 7-8) which affected the flooding behaviour and flood levels during this flood event. Details from the 1998 Storm Data Report (Wollongong City Council, 2002) also indicate that blockages of structures along this reach had a significant influence on water levels.

Figure 7-6 Deposited Debris on Hewitts Creek at George Street, Thirroul - August 1998 Event

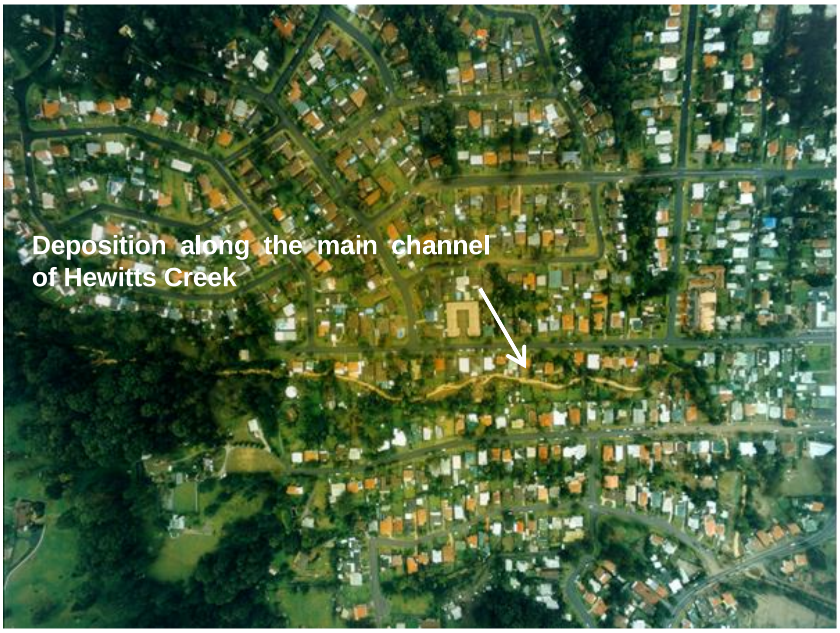


Figure 7-7 Aerial photograph of Deposited Debris on Hewitts Creek - August 1998 Event

The responses received as part of the community survey following the August 1998 flood event reported the following:

- At 75 George Street, Thirroul, the resident reported that at the height of the storm, the water level in his back yard was in excess of 3m.
- At 69 George Street, Thirroul, the water was reported to be 0.35m deep in the garage.
- At 51 George Street, Thirroul, the water was reported to be 1m deep in the garage.
- Numbers 37-39 George Street, Thirroul, sit on a raised platform. The residents reported that their yard was covered by approximately 0.1m of water.

The modelled flood levels and mechanisms correlate well with the observed flood levels along this reach of Hewitts Creek. In order to replicate the observed flood levels, the bridge at Kelton Lane, Thirroul, has been modelled with a 50% blockage while the two access bridges at 47 George Street, Thirroul and 51 George Street, Thirroul, have been modelled with 85% and 90% blockages respectively. These blockages are supported by photographs of flooding along Hewitts Creek following this flood event (refer to Figure 7-6 as an example of flooding at 51 George Street, Thirroul). In order to correlate the modelled and observed levels at 75 George Street, Thirroul, the channel bed elevations have been raised to represent the deposition of material on the bed of the creek at this location during this flood event.

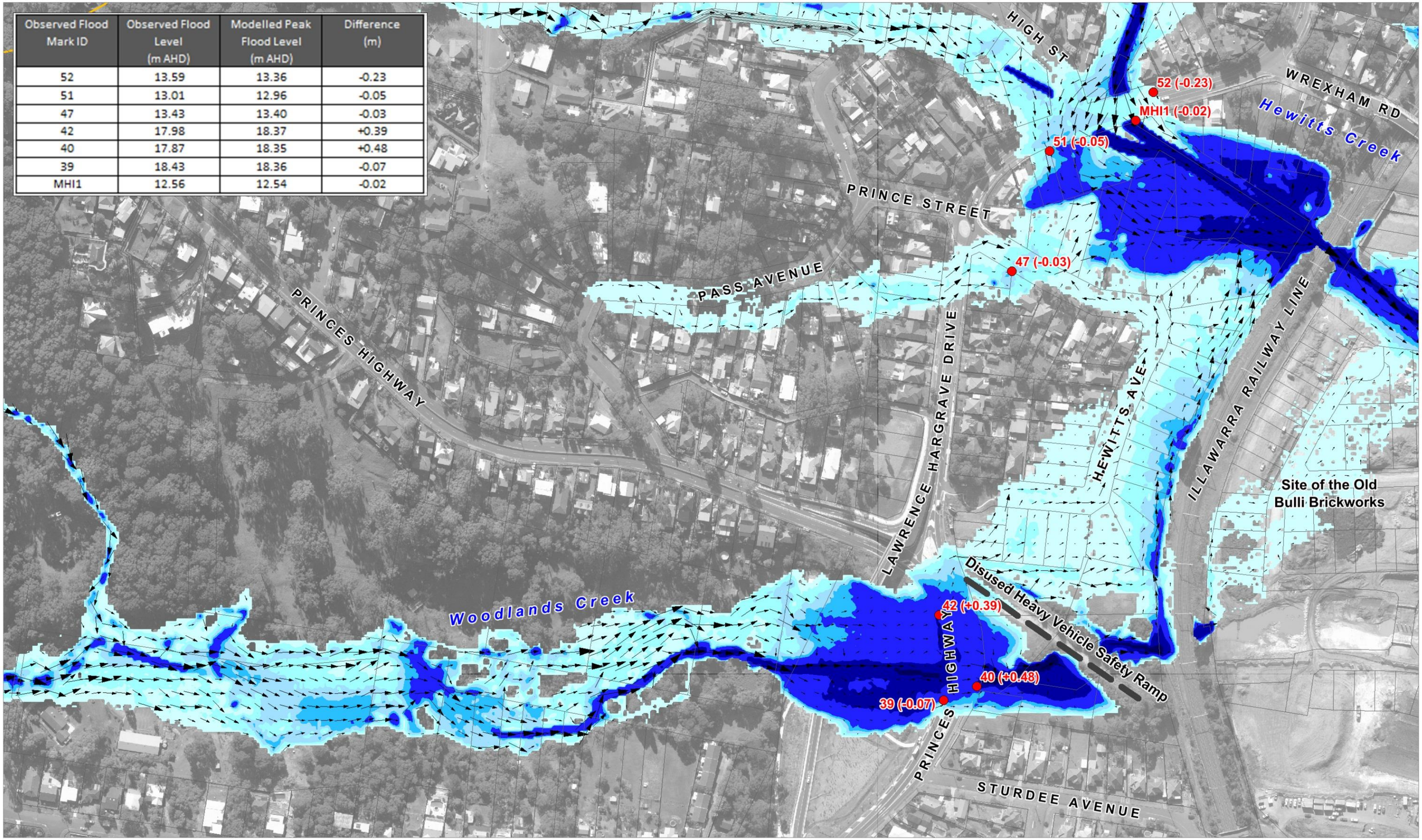
At Lachlan Street, Thirroul, residents reported that numerous properties experienced flooding. On the northern side of Lachlan Street, Thirroul, Hewitts Creek runs to the rear of the gardens of a number of properties. During the August 1998 flood event, the creek overflowed its banks and flooded the back yards of 23, 19, 17 and 15 Lachlan Street, Thirroul:

- At 17 Lachlan Street, Thirroul, the resident reported that flood water flowed through their back yard at a depth of approximately 0.5m.
- The brick garage of 15 Lachlan Street, Thirroul, was knocked down by the force of the flood water and also resulted in damage to the property at 11a Lachlan Street, Thirroul (see Figure 7-8).
- Further downstream, the residents of 416 Lawrence Hargrave Drive, Thirroul, reported flooding at the rear of their property with damage to a boundary fence.

Figure 7-8 Flood Damage at Lachlan St, Thirroul - August 1998 Event

On the southern side of Lachlan Street, Thirroul, flooding was observed to enter properties from flows spilling along Lachlan Street. Flood water exited these properties through the southern boundaries before re-joining Hewitts Creek downstream of Lachlan Street, Thirroul. The resident at 6 Lachlan Street, Thirroul, reported that 0.3m – 0.5m of water covered the back yard, whilst the resident at 12 Lachlan Street, Thirroul, reported between 1m and 1.5m of water at their property. The modelled flood mechanism correlate well with the observed flood mechanisms at this location. The modelled flood levels were also found to correlate well with the majority of the observed flood marks in this area. In order to replicate these levels in the model, the culvert under Lachlan Street, Thirroul, has been modelled as 75% blocked with 100% blockage applied to the handrails. At map reference ID 60 (Figure 7-5.2) the model is under predicting water levels by 0.32m when compared to the observed flood level. The modelled flood level at a neighbouring property (Map Reference ID 62, Figure 7-5.2), correlates well and indicate that the flood levels at map reference ID 60 may have been influenced by unreported local factors, such as a localised blockage between neighbouring properties, which are not replicated in the model. The calibrated model does not show flooding at two locations at the eastern end of Lachlan Street, Thirroul (Map Reference ID 56 and 57, Figure 7-5.2). Based on a comparison of the observed flood levels and the modelled ground levels, the flooding at these observed locations was likely shallow (i.e. less than 0.2m deep) and may be as a result of unreported local factors that have not been included in the model (i.e. debris on the road causing a rise and/or diversion in flood water). Although there are some localised variations between the modelled and observed levels in this area, the overall fit between observed and modelled levels suggests that the flows and hydraulic model parameters are of the correct order.

Observed Flood Mark ID	Observed Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (m)
52	13.59	13.36	-0.23
51	13.01	12.96	-0.05
47	13.43	13.40	-0.03
42	17.98	18.37	+0.39
40	17.87	18.35	+0.48
39	18.43	18.36	-0.07
MHI1	12.56	12.54	-0.02



LEGEND

- Observed Flood Mark
- 40 (+0.48) — ID Number (Difference between Observed and Modelled Level)
- MHI1 (-0.02) — Maximum Height Indicator (Difference between Observed and Modelled Level)
- Flow Velocity Vector
- Cadastre
- TUFLOW Model Boundary
- Catchment Boundary

Peak Flood Depths (m)
- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

Title:
Characteristics of Flooding in Woodlands Creek and the Middle Reaches of Hewitts Creek for the August 1998 Flood Event

Figure: 7-5.5

Rev: 2

N

0 50 100m
Approx. Scale

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Characteristics of Flooding in Woodlands Creek and the Middle Reaches of Hewitts Creek (Refer to Figure 7-5.5)

The TUFLOW model was found to correlate well with the majority of the observed flood marks in this area with the modelled peak flood levels generally within a ± 0.3m tolerance of the observed flood levels. Although there are some localised variations between the modelled and observed levels, the overall fit in this area suggests that the flows and hydraulic model parameters are of the correct order.

On Hewitts Creek, flooding was observed at multiple properties at the intersection of High Street, Thirroul, and Lawrence Hargrave Drive, Thirroul. On High Street, Thirroul, it was reported that flood water entered property numbers 2 and 4 from the west and ran through their properties before exiting onto High Street, Thirroul. Flooding was observed to overtop Lawrence Hargrave Drive, Thirroul and affected the grounds of properties downstream of this road. The resident at 10 Wrexham Road, Thirroul, indicated that flood water ran through the back corner of their yard at a depth of 1-2m and deposited a large amount of silt and debris. The modelled flood levels correlate well with the observed flood levels at Lawrence Hargrave Drive, Thirroul. In order to replicate the observed flood levels, the culverts under Lawrence Hargrave Drive, Thirroul and on High Street, Thirroul, have been assumed to be fully blocked. Photographic evidence following the August 1998 flood event indicate a significant build-up of debris at the entrance to the culvert on Hewitts Creek at Lawrence Hargrave Drive, Thirroul (refer to Figure 7-9).

Figure 7-9 Debris at Culvert Inlet on Lawrence Hargrave Drive, Thirroul - August 1998 Event

Between Lawrence Hargrave Drive, Thirroul and the Illawarra Railway, the model results indicate that flood flows from Hewitts Creek combined with overland flows originating from Pass Avenue, Thirroul and spill into Hewitts Creek across Hewitts Avenue, Thirroul. The observed flood mechanisms and levels correlate well with the model results at this location. The resident at 19 Pass Avenue, Thirroul, reported flooding through the grounds of their property and numerous neighbouring properties. At 25

Hewitts Avenue, Thirroul, flood water was observed to enter the property from the south west boundary and proceeded through the yard and house before exiting to the north onto Hewitts Avenue, Thirroul. The resident reported flood water up to 1m deep. At 28 Hewitts Avenue, Thirroul, it was reported that flood water moved north through their property with approximately 2 inches (50mm) of water covering the ground floor of their property. The modelled flood level and mechanisms at Hewitts Avenue, Thirroul, correlates well with the observed flood levels and mechanisms.

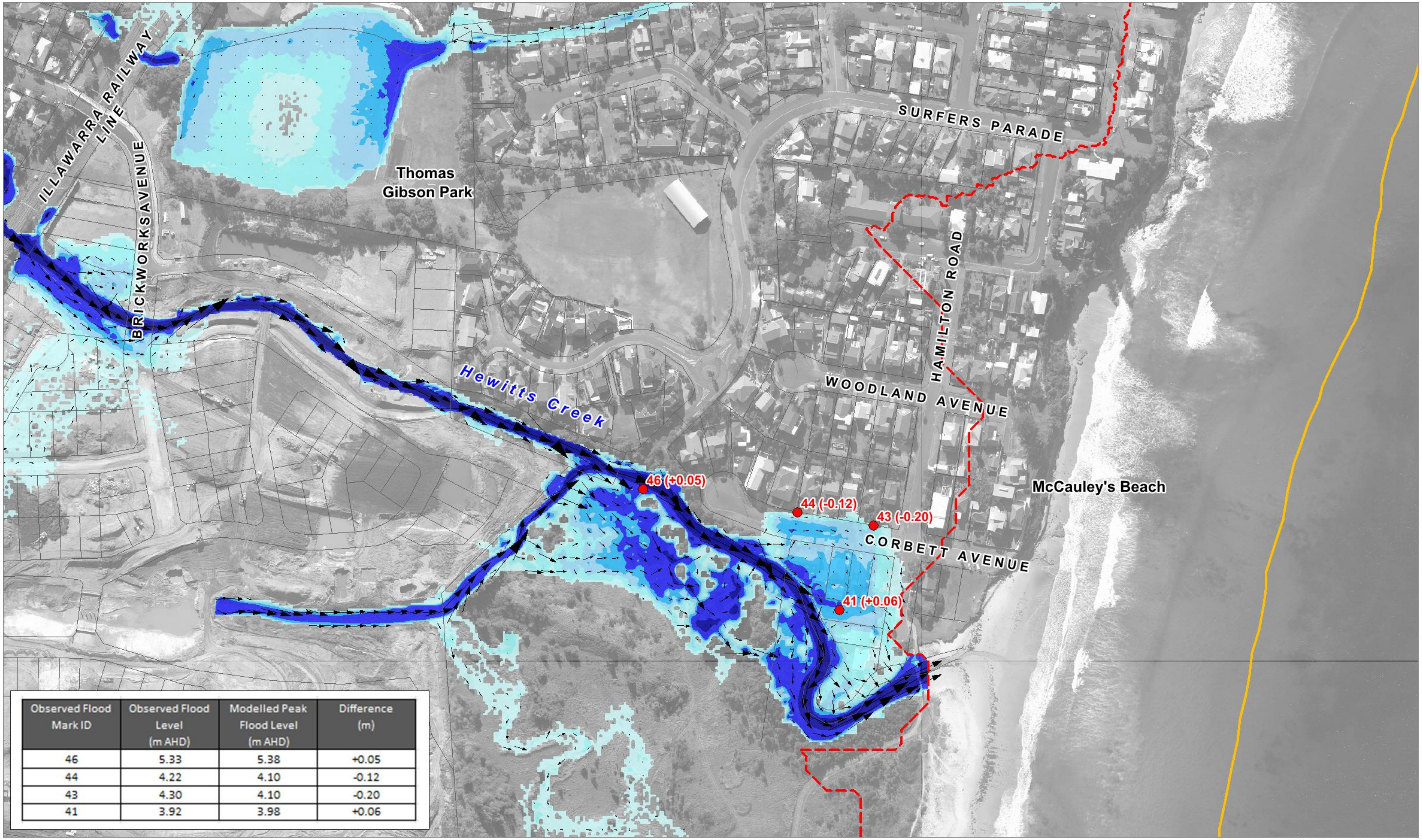
Between Lawrence Hargrave Drive and the Illawarra Railway, the level of flooding is also affected by flood flows from Woodlands Creek to the south. The model results for this event indicate that flow transfers to the Hewitts Creek catchment by overtopping the disused heavy vehicle safety ramp at Princes Highway, Thirroul. Flows also transfer to Hewitts Creek along the base of the railway embankment from surcharged flows at the culvert inlet on Woodlands Creek at the Illawarra Railway, Bulli. These flood flows primarily affected properties along Hewitts Avenue, Thirroul, bounded by the disused heavy vehicle safety ramp at Princes Highway to the south and the Illawarra Railway to the east. The modelled flood mechanisms correlate well with the observed flood mechanisms. Observed flood information indicates that numerous houses in this location reported flooding. The resident at 4 Hewitts Avenue, Thirroul, indicated that flood water moved through their property from the south and exited to the north onto Hewitts Avenue, Thirroul. Flood water up to 1m deep was reported at 12 Hewitts Avenue, Thirroul, with flows cutting across the back of the property and heading north. The resident at 14 Hewitts Avenue, Thirroul, reported the same pattern of flooding with water entering their property from 12 Hewitts Avenue, Thirroul, to the south and moving through to 14 Hewitts Avenue, Thirroul, to the north before exiting into 16 Hewitts Avenue, Thirroul. The resident at 14 Hewitts Avenue, Thirroul indicated that water adjacent to the railway line was approximately 1.5m deep, with depths of 0.8m and 2.3m water near their house and garage. A 0% blockage has been applied to the culvert at the Illawarra Railway and the two structures immediately downstream of the railway on Hewitts Creek for this event. The model was found to correlate well with the observed flood marks at Lawrence Hargrave Drive, Thirroul, indicating that a 0% blockage of this structure is appropriate for this flood event. No observed flood information is available to calibrate the blockages at the two structures immediately downstream of the railway. As a 0% blockage has been applied to the Illawarra Railway culvert, the same blockage 0% blockage has been applied to these structures immediately downstream.

On Woodlands Creek, upstream of the disused heavy vehicle safety ramp at Princes Highway, Bulli, there is limited information on observed flood mechanisms for comparison with modelled flood mechanisms. An aerial photograph of flooding along this reach indicates a significant extent of flooding upstream of Princes Highway, Bulli (see Figure 7-10) which correlates well with the modelled flood extent at this location (see Figure 7-5.3).

Figure 7-10 Aerial Photograph of Flooding along Woodlands Creek - August 1998 Event

A number of observed flood levels are available at Lawrence Hargrave Drive, Bulli. The model correlates well with the highest of these observed flood levels (Map Reference ID 39, Figure 7-5.3) and over-estimates the water levels at the two other locations (Map Reference ID 42 and 40, Figure 7-5.3). The model results indicate a relatively flat water surface profile at the peak of the event extending for approximately 250m upstream of the disused heavy vehicle safety ramp at Princes Highway, Bulli, along Woodlands Creek. This flat water surface profile results from the backwater effect of the disused heavy vehicle safety ramp at Princes Highway, Bulli, which has a controlling influence on the peak water levels upstream of this ramp embankment. Given the flat surface water profile predicted by the model, it was not possible to replicate the varying observed flood levels at this location. Given the model correlates well to the highest of these observed flood levels, it suggests that the flows and hydraulic model parameters are of the correct order. The best correlation between modelled and observed flood levels was achieved with 0% blockages applied to the culverts at Princes Highway, the disused heavy vehicle safety ramp and the Illawarra Railway for this event.

Immediately downstream of the Illawarra Railway, Bulli, the model results show flow transferring northwards from Woodlands Creek to Hewitts Creek..



Observed Flood Mark ID	Observed Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (m)
46	5.33	5.38	+0.05
44	4.22	4.10	-0.12
43	4.30	4.10	-0.20
41	3.92	3.98	+0.06

LEGEND

- Observed Flood Mark
- 41 (0.06) ID Number (Difference between Observed and Modelled Level)
- Flow Velocity Vector
- Cadastre
- TUFLOW Model Boundary
- Catchment Boundary

Peak Flood Depths (m)
- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

Title:
Characteristics of Flooding in the Lower Reaches of Hewitts Creek and Woodlands Creek for the August 1998 Flood Event

Figure: 7-5.6

Rev: 1

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0 50 100m
Approx. Scale

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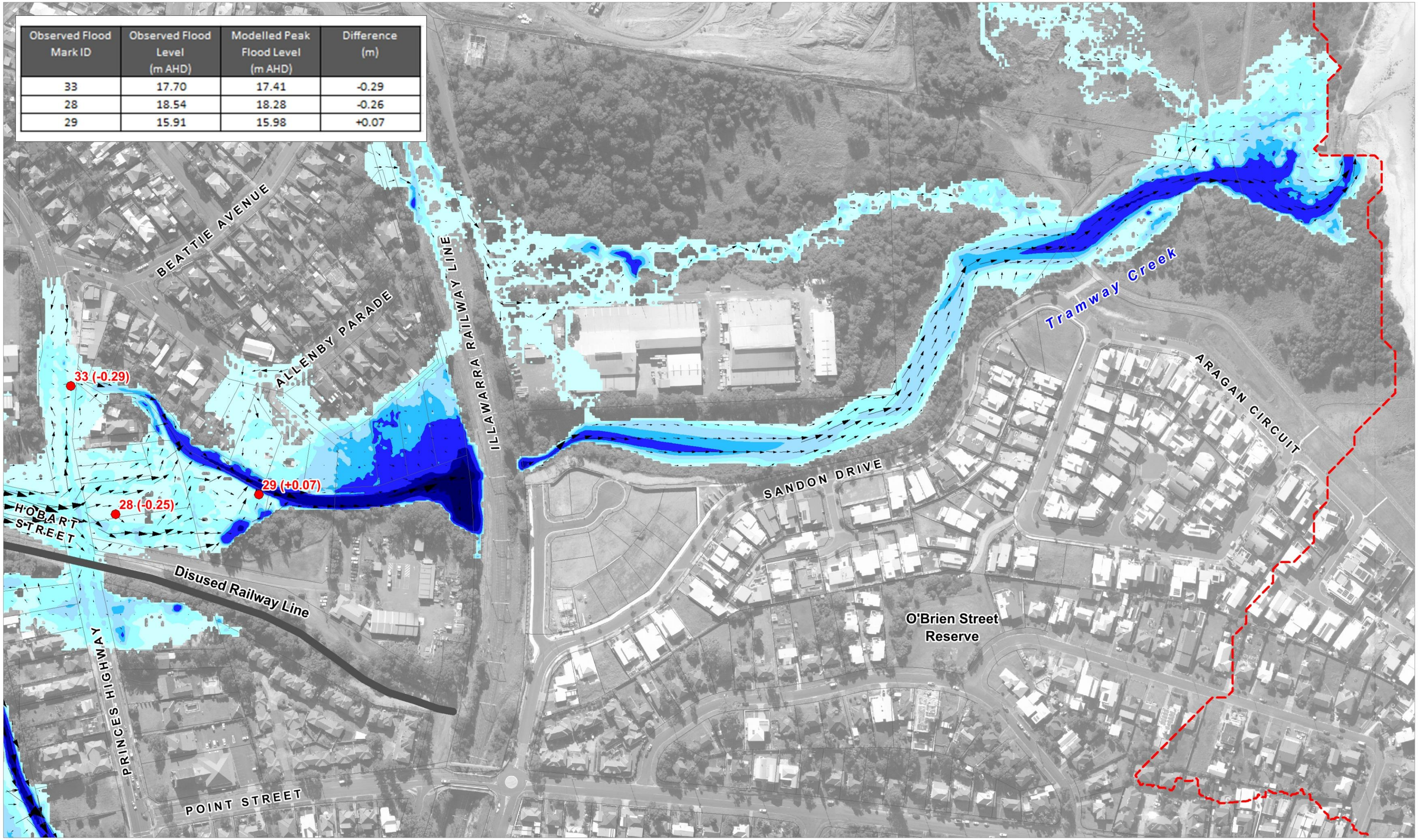
Characteristics of Flooding in the Lower Reaches of Hewitts Creek and Woodlands Creek (Refer to Figure 7-5.6)

The TUFLOW model was found to correlate well with the observed flood marks in this area with the modelled peak flood levels all within a ± 0.3m tolerance of the observed flood levels. The overall fit in this area suggests that the flows and hydraulic model parameters are of the correct order.

Observed flood information in this area is confined to the lower reaches of Hewitts Creek. Flooding was reported by the residents of 13 Corbett Avenue, Thirroul. They reported flood water entering the front and rear of their property via burst water pipes on the street and flows spilling from the creek. The entire back yard of their property was under approximately 0.4m of water. Numbers 6 and 8 Hamilton Road, Thirroul, also reported flooding where both houses had water enter their property from Hewitts Creek. The resident at 8 Hamilton Road, Thirroul, reported there was approximately 0.1m of water at the back steps of their house and that the flood water cut across the creek bend and through the corner of their property. The resident at 6 Hamilton Road, Thirroul, reported approximately 1m of water in the garage underneath their home, approximately 0.3m of water on the front lawn and approximately 0.6m of water in their back yard. The modelled flood mechanisms and flood levels correlate well with the observed flood mechanisms and levels in this location. The best correlation between modelled and observed flood levels was achieved with a 0% blockage applied to the footbridge at Hamilton Road, Thirroul.

The modelled flood mechanisms indicate flows transferring from Woodlands Creek to Tramway Creek across the open parkland in the south of this area. Flood water spills from the right bank of Woodlands Creek upstream of its confluences with Hewitts creek and crosses into Tramway Creek upstream of the entrance to the creek.



Observed Flood Mark ID	Observed Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (m)
33	17.70	17.41	-0.29
28	18.54	18.28	-0.26
29	15.91	15.98	+0.07

LEGEND

- Observed Flood Mark
- 29 (0.07) ID Number (Difference between Observed and Modelled Level)
- Flow Velocity Vector
- Cadastre
- TUFLOW Model Boundary
- Catchment Boundary

Peak Flood Depths (m)

- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

Title:
Characteristics of Flooding in Tramway Creek for the August 1998 Flood Event

Figure: 7-5.7

Rev: 1

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Approx. Scale

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Characteristics of Flooding in Tramway Creek (Refer to Figure 7-5.7)

The TUFLOW model was found to correlate well with the observed flood marks in this area with the modelled peak flood levels all within a ± 0.3m tolerance of the observed flood levels. The overall fit in this area suggests that the flows and hydraulic model parameters are of the correct order.

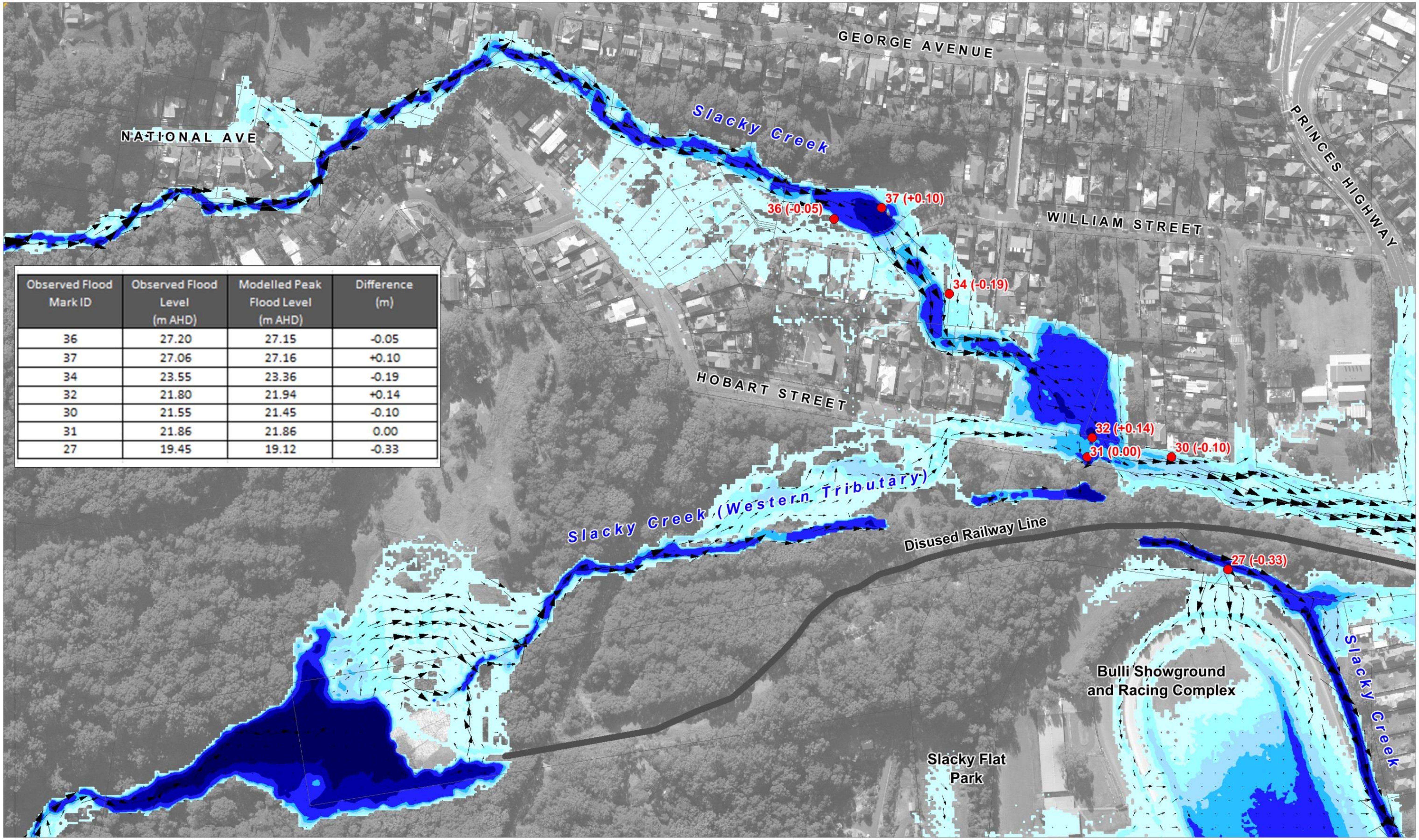
At Hobart Street, Bulli, the coal haulage embankment of the now disused railway line within the Slacky Creek catchment is known to divert flows eastwards along Hobart Street, Bulli, to the Tramway Creek catchment due to the capacity of the culverts being exceeded (refer to the next Section on the Characteristics of Flooding in the Upper Reaches of Slacky Creek for further details). Flooding was reported at a number of properties as follows:

- The residents of 163 and 165 Princes Highway, Bulli, observed water flowing north from Hobart Street, Bulli, towards Tramway Creek.
- At 163 Princes Highway, Bulli, the residents reported that floodwaters covered the driveway to a depth of approximately 0.1m and entered the workshop to the rear of their property.
- The residents of 169 Princes Highway, Bulli, reported that floodwater water entered their property from the south, near Princes Highway, Bulli and ran down the side of the house and exited to the rear of their property. Floodwaters reached a depth of approximately 0.9m at the side of the house.
- The residents of 17a Allenby Parade, Bulli, reported surface water from a blocked drain in Allenby Parade was flowiong down both sides of their house, and exiting into Tramway Creek at the south-eastern corner of their property. At no stage did their home flood.
- Surface water runoff was reported to have affected the property at 1A Allenby Parade, Bulli. The residents of this property reported flows from the northwest front side of their house which existed their property towards Tramway Creek to the south.

The modelled flood mechnanisms and levels generally correlate well with the observed flood mechanisms and levels at this location. In order to correlate the levels upstream of the Illawarra Railway, the culvert has been modelled with a 60% blockage.

There are no observed flood levels or flood mechainsms downstream of the Illawarra Railway. The model results indicate that flows generally remain in-bank with some out of bank flows near the entrance to the creek where flows in Tramway Creek are joined by flows from Woodlands Creek to the north.

The results indicate that the flows and hydraulic model parameters are of the correct order.



Observed Flood Mark ID	Observed Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (m)
36	27.20	27.15	-0.05
37	27.06	27.16	+0.10
34	23.55	23.36	-0.19
32	21.80	21.94	+0.14
30	21.55	21.45	-0.10
31	21.86	21.86	0.00
27	19.45	19.12	-0.33

LEGEND

- Observed Flood Mark
- 34 (-0.19) ID Number (Difference between Observed and Modelled Level)
- Flow Velocity Vector
- Cadastre
- TUFLOW Model Boundary
- Catchment Boundary

Peak Flood Depths (m)
- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

Title:
Characteristics of Flooding in the Upper Reaches of Slacky Creek for the August 1998 Flood Event

Figure: 7-5.8

Rev: 2

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0 50 100m
Approx. Scale

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Characteristics of Flooding in the Upper Reaches of Slacky Creek (Refer to Figure 7-5.8)

The TUFLOW model was found to correlate well with the majority of the observed flood marks in this area with the modelled peak flood levels generally within a ± 0.3m tolerance of the observed flood levels. Although there are some localised variations between the modelled and observed levels, the overall fit in this area suggests that the flows and hydraulic model parameters are of the correct order.

Several properties at 7 National Avenue, Bulli, and 9 National Avenue, Bulli, experienced flooding during the 1998 event. The residents of 9 National Avenue, Bulli, indicated that water entered from National Avenue, Bulli and ran through property numbers 9 and 11 to join flows in Slacky Creek to the rear of these properties with the lower garage at 9 National Avenue, Bulli, flooded to a depth of approximately 1m. The water running through the grounds of this property was 0.1m – 0.2m deep and very fast moving. The units in 7 National Avenue, Bulli, reported a similar pattern of flooding, with very fast flowing water through their property, sweeping full garbage bins downstream. Flooding in the garages of this property was approximately 0.75m deep with flood water entering some of the lower ground floor units.

Further downstream, a number of properties on George Avenue, Bulli, reported flooding:

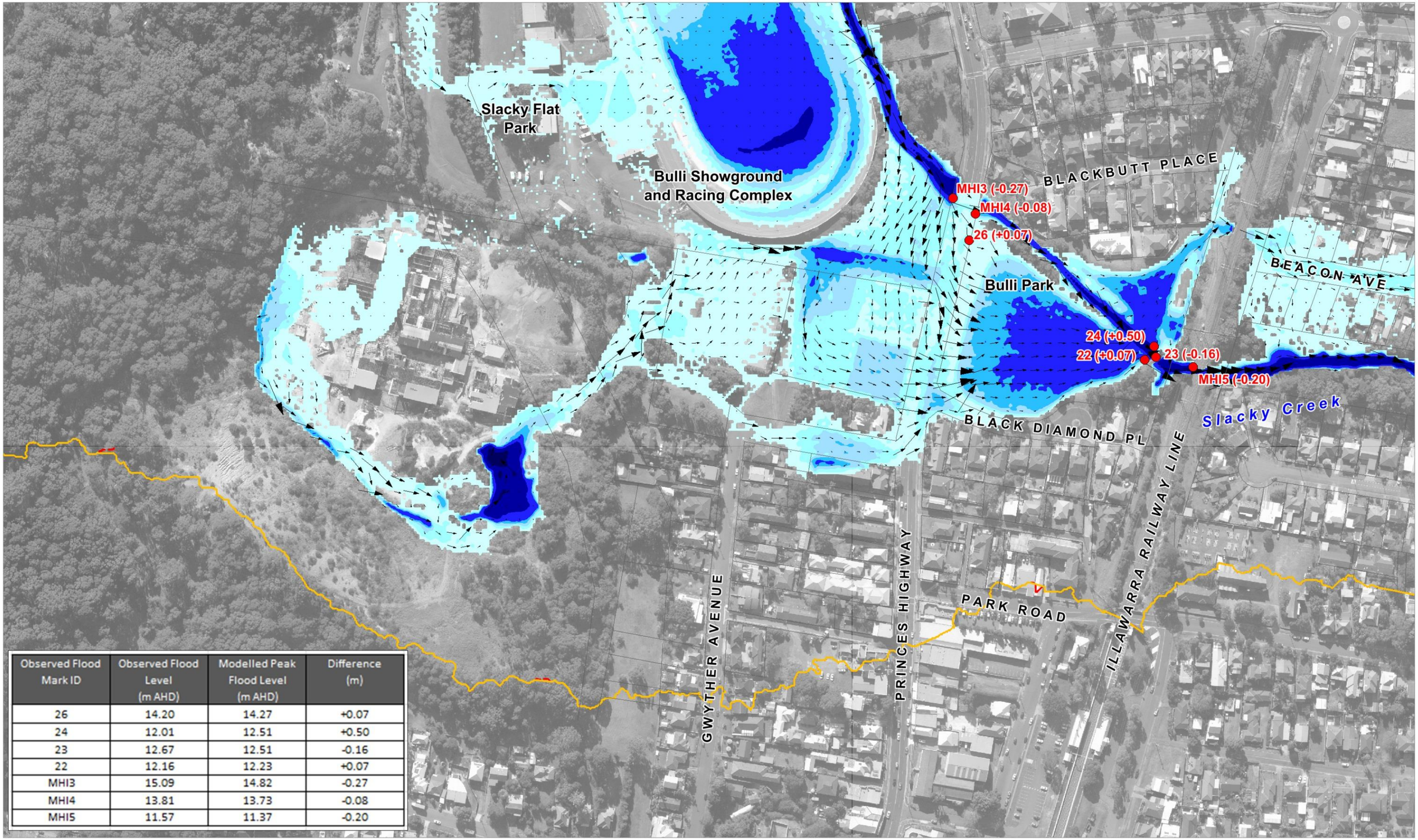
- The resident at 67 George Avenue, Bulli, observed severe erosion all along the creek, with their own property losing approximately 4m of land (see Figure 7-11).
- The resident at 65 George Avenue, Bulli, reported water flowing rapidly through their back yard from the creek as well as surface water runoff flowing from George Avenue, Bulli, through their property to the Creek.
- The resident at 63 George Avenue, Bulli, observed the creek breaking its banks which resulted in the destruction of a boundary fence, flood damages to their garage and erosion of their land.
- The resident at 61 George Avenue, Bulli, reported similar flooding issues as reported at 63 George Avenue, Bulli, with approximately 4m of their property including two sheds and a sewerage line eroded and washed away by the flood water.
- Across the creek from George Avenue Bulli, surface water flooding was observed at 42 Hobart Street, Bulli, where the residents observed water flowing from Hobart Street, Bulli, through their property and exiting through their back yard to Slacky Creek.
- The residents of 54 Hobart Street, Bulli, reported water flowing through their back yard from out of bank flooding from Slacky Creek.

The modelled flood mechanisms correlate well with the observed flood mechanisms along this reach of Slacky Creek from National Avenue, Bulli, to George Avenue, Bulli. The observed erosion and subsidence of the creek banks reported in this location was not assessed as part of this study.

Figure 7-11 Erosion downstream of National Avenue, Bulli – August 1998 Event

Further downstream, at William Street Bulli, the modelled flood levels correlate well with three observed flood levels upstream and downstream of the William Street road culvert. A 90% blockage of the William Street culvert resulted in the best correlation between observed and modelled flood levels at this location.

At Hobart Street, Bulli, the coal haulage embankment of the now disused railway line within the Slacky Creek catchment is known to divert flows eastwards along Hobart Street, Bulli, to the Tramway Creek catchment due to the capacity of the culverts being exceeded. The model results correlate well with the observed flood mechanisms and levels at this location. The best correlation between observed and modelled levels was achieved by applying 0% blockages to the culverts on Hobart Street Bulli and the coal haulage embankment of the now disused railway line. The model results indicate that flows from both Slacky Creek main channel and the tributary of Slacky Creek, (which drains the south of this sub-catchment) both contribute to flood flows along Hobart Street, Bulli. These flows divert westwards along Hobart Street, Bulli, where they combine with flows from Tramway Creek. Downstream of the coal haulage embankment, at the Bulli Showground and Racing Complex, Bulli, there is no information on observed flood mechanisms. The model results indicate flooding of these grounds for this event through a low ground point to the north of the Showground. There is one observed flood level at this location which indicates that the model is under-predicting water levels by 0.33m. A 100% blockage has been applied to the culvert on Slacky Creek downstream of this observed flood level to achieve the best fit between modelled and observed flood levels. Given the good correlation between modelled and observed flood levels at Hobart Street, Bulli, and further downstream on Slacky Creek at Princes Highway, Bulli, the overall fit along the upper reaches of Slacky Creek indicates that the flows and hydraulic model parameters are of the correct order.



Observed Flood Mark ID	Observed Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (m)
26	14.20	14.27	+0.07
24	12.01	12.51	+0.50
23	12.67	12.51	-0.16
22	12.16	12.23	+0.07
MHI3	15.09	14.82	-0.27
MHI4	13.81	13.73	-0.08
MHI5	11.57	11.37	-0.20

LEGEND

- Observed Flood Mark
- 24 (0.50) — ID Number (Difference between Observed and Modelled Level)
- MHI3(-0.27) — Maximum Height Indicator (Difference between Observed and Modelled Level)
- Flow Velocity Vector
- Cadastre
- TUFLOW Model Boundary
- Catchment Boundary

Peak Flood Depths (m)
- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

Title:
Characteristics of Flooding in the Middle Reaches of Slacky Creek for the August 1998 Flood Event

Figure: 7-5.9

Rev: 2

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0 50 100m
Approx. Scale

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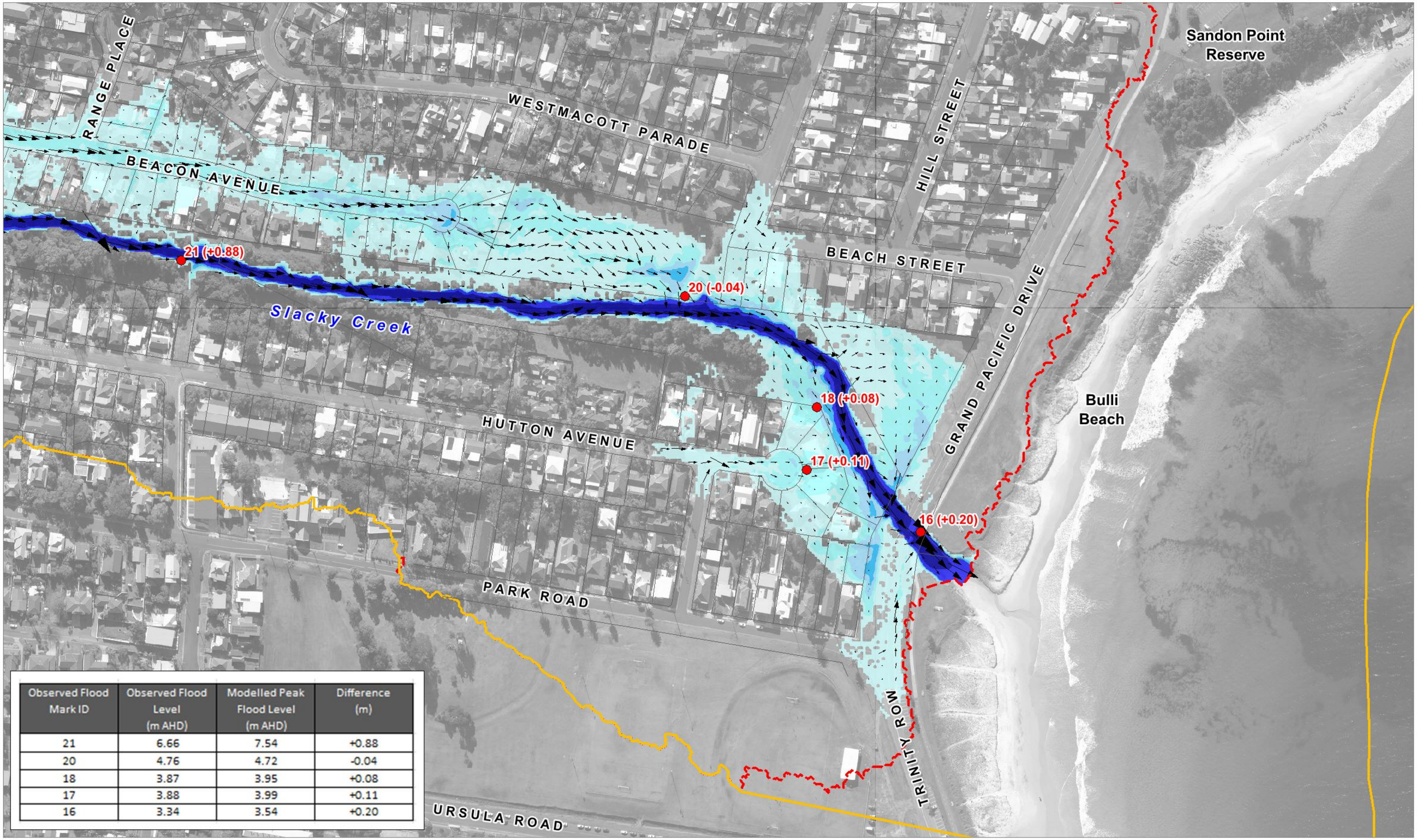
Characteristics of Flooding in Middle Reaches of Slacky Creek (Refer to Figure 7-5.9)

The TUFLOW model was found to correlate well with the majority of the observed flood marks in this area with the modelled peak flood levels generally within a ± 0.3m tolerance of the observed flood levels. Although there are some localised variations between the modelled and observed levels, the overall fit in this area suggests that the flows and hydraulic model parameters are of the correct order.

No details are available on the observed flood mechanisms in this location for this event. The modelled flood mechanisms indicate that out of bank flows from Slacky Creek combine with overland flows from south of the Bulli Showground and Racing Complex, Bulli, upstream of Princes Highway, Bulli. The combined flows spill over the Princes Highway, Bulli and into the storage reservoir at Black Diamond Place, Bulli. Flows exit the storage reservoir along Slacky Creek at the culvert within park at Black Diamond Place, Bulli and further north through the pedestrian underpass on the Illawarra Railway at Beacon Avenue, Bulli. The modelled flood levels generally correlate well with the observed flood levels along this reach. In order to achieve this correlation in levels, a 90% blockage has been applied to the culvert on Princes Highway, Bulli, with the remaining culverts at park at Black Diamond Place, Bulli and the Illawarra Railway, Bulli, modelled with 0% blockages.

There are three observed flood levels located within 10m of each other at the culvert in park at Black Diamond Place, Bulli, upstream of Illawarra Railway, Bulli (Map Reference ID 22, 23 and 24, Figure 7-5.6). There is a 0.66m difference between the highest and lowest observed level at this location which is unexpected given the close proximity of the observed marks. The model correlates well with two of the observed levels at this location. The model also correlates well with the observed level at the Illawarra Railway culvert, 25m downstream of the culvert in park at Black Diamond Place, Bulli. Given the good correlation with the observed flood levels at the majority of points in this area, the model is considered to perform satisfactorily at this location.



Observed Flood Mark ID	Observed Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (m)
21	6.66	7.54	+0.88
20	4.76	4.72	-0.04
18	3.87	3.95	+0.08
17	3.88	3.99	+0.11
16	3.34	3.54	+0.20

LEGEND

- Observed Flood Mark
- 21 (0.88) ID Number (Difference between Observed and Modelled Level)
- Flow Velocity Vector
- Cadastre
- TUFLOW Model Boundary
- Catchment Boundary

Peak Flood Depths (m)

- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

Title:
Characteristics of Flooding in the Lower Reaches of Slacky Creek for the August 1998 Flood Event

Figure: 7-5.10

Rev: 1

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0 50 100m
Approx. Scale

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Characteristics of Flooding in the Lower reaches of Slacky Creek (Refer to Figure 7-5.10)

The TUFLOW model was found to correlate well with the majority of the observed flood marks in this area with the modelled peak flood levels generally within a ± 0.3m tolerance of the observed flood levels. Although there are some localised variations between the modelled and observed levels, the overall fit in this area suggests that the flows and hydraulic model parameters are of the correct order.

Observed flood mechanisms along this reach are limited to the downstream reaches at Hutton Avenue, Bulli, and Beach Street, Bulli, with flooding reported at 17 and 19 Beach Street, Bulli and 2 and 4 Hutton Avenue, Bulli. The residents at 17 and 19 Beach Street, Bulli, both reported stormwater flowing from Beach Street, Bulli, through their properties and exiting the rear of their properties to join Slacky Creek. The residents reported that the drains on Beach Street, Bulli, were unable to cope with the volume of surface water flows. The residents of 17 Beach Street, Bulli, reported 0.1m of water inside their house, 0.05m of water in their garage 0.4m, water in the grounds of their property and against the walls of the house. The residents of 2 and 4 Hutton Avenue, Bulli, both reported surface water flowing through their properties from Hutton Avenue, Bulli, to Slacky Creek at the rear of their properties. The residents of 4 Hutton Avenue, Bulli, reported 0.4m of water in the grounds of their property while the residents of 2 Hutton Avenue, Bulli, reported 0.2m of water in their yard.

The modelled flood mechanisms and levels generally correlate well with the observed flood levels and mechanisms in this area. The best correlation between observed and modelled levels at Hutton Avenue, Bulli and Blackhall Street, Bulli, was achieved by applying a 50% blockage to the culvert at Blackhall Street, Bulli.

At Map Reference ID 21 (Figure 7-5.6), the model is over-predicting levels when compared to the observed flood levels. The observed flood mark at this location indicates that the flood level was approximately 0.6m below the top of bank for this event. Given the size of this flood event, it is unlikely that the in-bank water levels would have been so low and it is likely that this value is erroneous. The model also correlates well with observed values upstream and downstream of this location.

7.2.5.2 Longitudinal Profiles

Longitudinal profiles showing the simulated peak flood levels, water surface profile and observed flood levels for the August 1998 event are shown in Appendix B.

The profiles show the significant influence of the structures and structure blockages on the peak water levels, particularly at the larger hydraulic structures in the study area which have been modelled with structure blockages (refer to Table 7-7). Based on the longitudinal profiles and a review of the model results, Table 7-7 provides a list of the key structures which resulted in a significant impact on flood levels for the August 1998 event. For a full list of blockages applied to all structures for the August 1998 event, refer to Table 7-3.

Table 7-7 Structures with a Significant Impact on Flood Levels - August 1998 Event

Watercourse	Street	Structure Type	Structure ID (refer to Figure 6-2)	% Blockage Applied to Structure (current Flood Study)
Hewitts Creek	51 George Street, Thirroul	Bridge	19	85
Hewitts Creek	47 George Street, Thirroul	Bridge	20	90
Hewitts Creek	Kelton Lane, Thirroul	Bridge	21	50
Hewitts Creek	Lachlan Street, Thirroul	Culvert	22	75
Hewitts Creek	Lawrence Hargrave Drive, Thirroul	Culvert	23	100
Hewitts Creek	Illawarra Railway	Bridge	26	0
Hewitts Creek	Lawrence Hargrave Drive, Thirroul	Culvert	24	100
Thomas Gibson Creek	Illawarra Railway, Thirroul	Culvert	36	20
Thomas Gibson Creek	Cliff Parade, Thirroul	Culvert	39	0
Woodlands Creek	Illawarra Railway, Bulli	Culvert	17	0
Tramway Creek	Illawarra Railway, Bulli	Culvert	14	60
Slacky Creek	Hobart Street, ,Bulli	Culvert	2	0
Slacky Creek, western tributary	Hobart Street, Bulli	Culvert	3	0
Slacky Creek	Hobart Street, Bulli	Culvert	5	0
Slacky Creek	Hobart Street (Coal haulage embankment), Bulli	Culvert	6	0
Slacky Creek	Princes Highway, Bulli	Culvert	8	100
Slacky Creek	Park at Black Diamond Place, upstream of Illawarra Railway, Bulli	Culvert	9	0
Slacky Creek	Illawarra Railway (Creek opening and Beacon Avenue underpass), Bulli	Culvert	10	0

7.3 February 2013 Model Validation

The 24 February 2013 flood event was a relatively minor flood event when compared to previous historic flood events in Wollongong with the event generally tracking less than a 1 year ARI. This flood event was chosen to allow a validation of the hydraulic model to be used for the design model runs, incorporating any physical changes in the study area within the hydraulic model since the 1998 calibration event. In addition, using a 1 year ARI event, allows an assessment of the performance of the model in replicating observed flooding behaviour for a range of flood events. The February 2013 event has been identified following community consultation undertaken as part of this study.

7.3.1 Rainfall Data

Rainfall data are available from a number of pluviograph stations for the February 2013 event including Rixons Pass, Russel Vale and Bulli Bowling Club (this station was previously named Thirroul Bowling Club). Bulli Bowling Club is located within the Hewitts Creek Catchment and data from this station have been used to derive flows for the 2013 validation event. A hyetograph of the 2013 storm recorded by this station is shown in Figure 7-12 for a 5 hour period from 22:00 on the 23 of February 2013. This period represented the peak of the rainfall during this flood event.

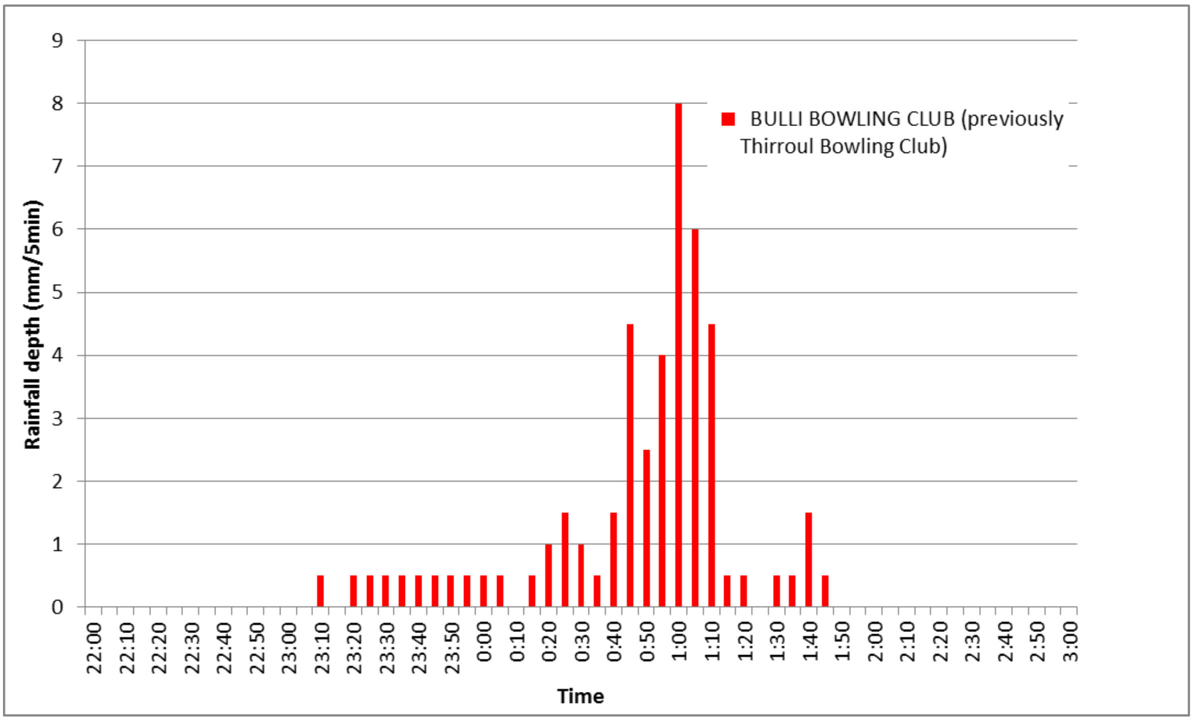


Figure 7-12 Hyetograph for Bulli Bowling Club Pluviograph Station – 23-24 February 2013 Event

The main rainfall burst for the event occurred during a one-hour period from approximately 00:30 on the 24 February 2013 with a rainfall depth of 33.5 mm recorded for this one hour period at the Bulli Bowling Club station. The 24 hour total for this station, from 9am on the 23 of February, was 70mm.

To gain an appreciation of the relative intensity of the February 2013 event, the recorded rainfall depths for various storm durations were compared with the design IFD data for the Hewitts Creek study area as shown in Figure 7-13.

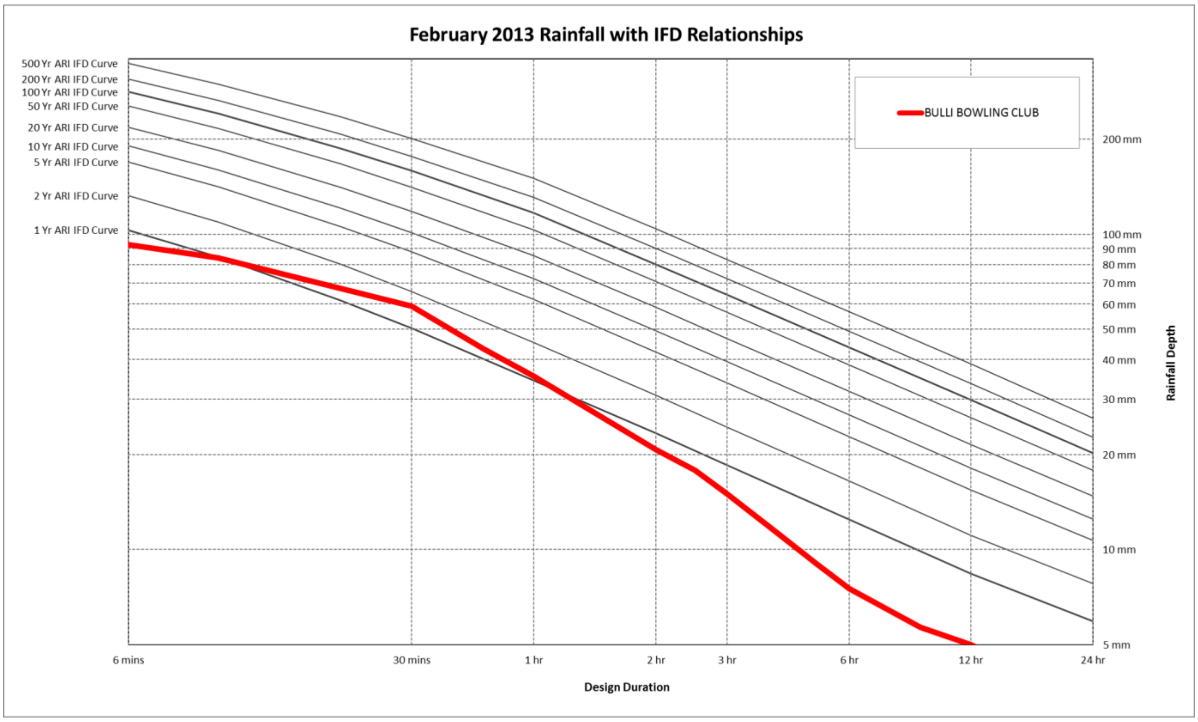


Figure 7-13 Comparison of February 2013 Rainfall with IFD Relationships

The February 2013 event generally tracks less than a 1 year ARI for the longer design durations and between a 1 year ARI and 2 year ARI for the shorter design durations. The following comparisons to design rainfall depths can be made:

- 3-hour duration – 45mm recorded compared with 55.6mm design 1 year ARI;
- 6-hour duration – 45mm recorded compared with 74.7mm design 1 year ARI; and
- 12-hour duration – 60mm recorded compared with 100.6mm design 1 year ARI.

There were insufficient data from gauges within the vicinity of the study area to generate rainfall isohyets. Spatial weighting has not been applied to the rainfall for this event.

7.3.2 Antecedent Conditions

The month of February 2013 was a relatively dry month in Wollongong. A review of daily rainfall totals for gauges in the vicinity of the study area indicates that zero rainfall was recorded for the majority of the days in February 2013 leading up to the event.

7.3.3 Downstream Boundary Condition

A dynamic downstream water level boundary for the February 2013 has been derived from tidal data at the Port Kembla tidal gauge. The relationship between observed tide levels at Port Kembla and recorded rainfall at Bulli Bowling Club for a 5 hour period from 22:00 on the 23 of February 2013 is shown in Figure 7-14 (this period represented the peak of the rainfall during this flood event).

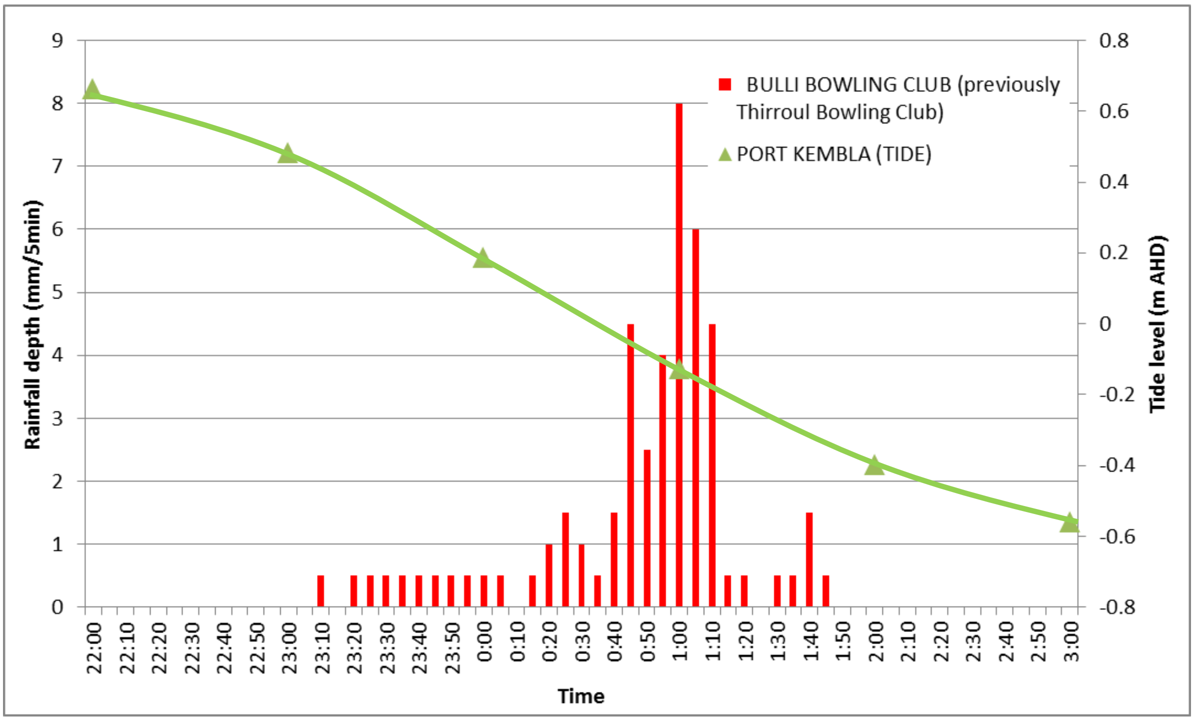


Figure 7-14 Comparison of Observed Rainfall and Tidal Data – 23-24 February 2013 Event

The chart indicates that the peak tide occurred before the main rainfall burst at Bulli Bowling Club. The peak of the flood at the entrance to the creeks coincided with a falling tide and the entrance conditions would have been controlled by peak flood flows in the creek channels.

7.3.4 Model parameters

As discussed earlier, model validation is undertaken to test the appropriateness of the adopted calibration parameters for different historical events and provides an indication of parameter variability. During the August 1998 event, culvert and bridge blockages were a key influence on peak flood levels and these blockages were therefore used as a key calibration parameter within the hydraulic model for the 1998 event.

The culvert blockages adopted for the 1998 calibration event are unique to this flood event and have not been adopted for the February 2013 model validation simulation. Given the size of the flood event, i.e. the rainfall for this event is less than a 1 year ARI, all of the culverts and bridges have been modelled as unblocked for this event.

As per the 17 August 1998 flood event, the entrances of the creeks have been modelled as fixed with the geometry of the entrance defined at the start of the flood event. The geometry of the entrances defined for the 17 August 1998 flood event has been used for the 2013 validation event.

Based on a review of the antecedent conditions the loss rates adopted for the 2013 event are detailed in Table 7-8.

Table 7-8 Rainfall Loss Rates – February 2013 Event

Rainfall Loss Type	Values
Initial Loss – Pervious	10 mm
Initial Loss – Impervious	1 mm
Continuing Loss – Pervious	2.5mm/hour
Continuing Loss – Impervious	0mm/hour

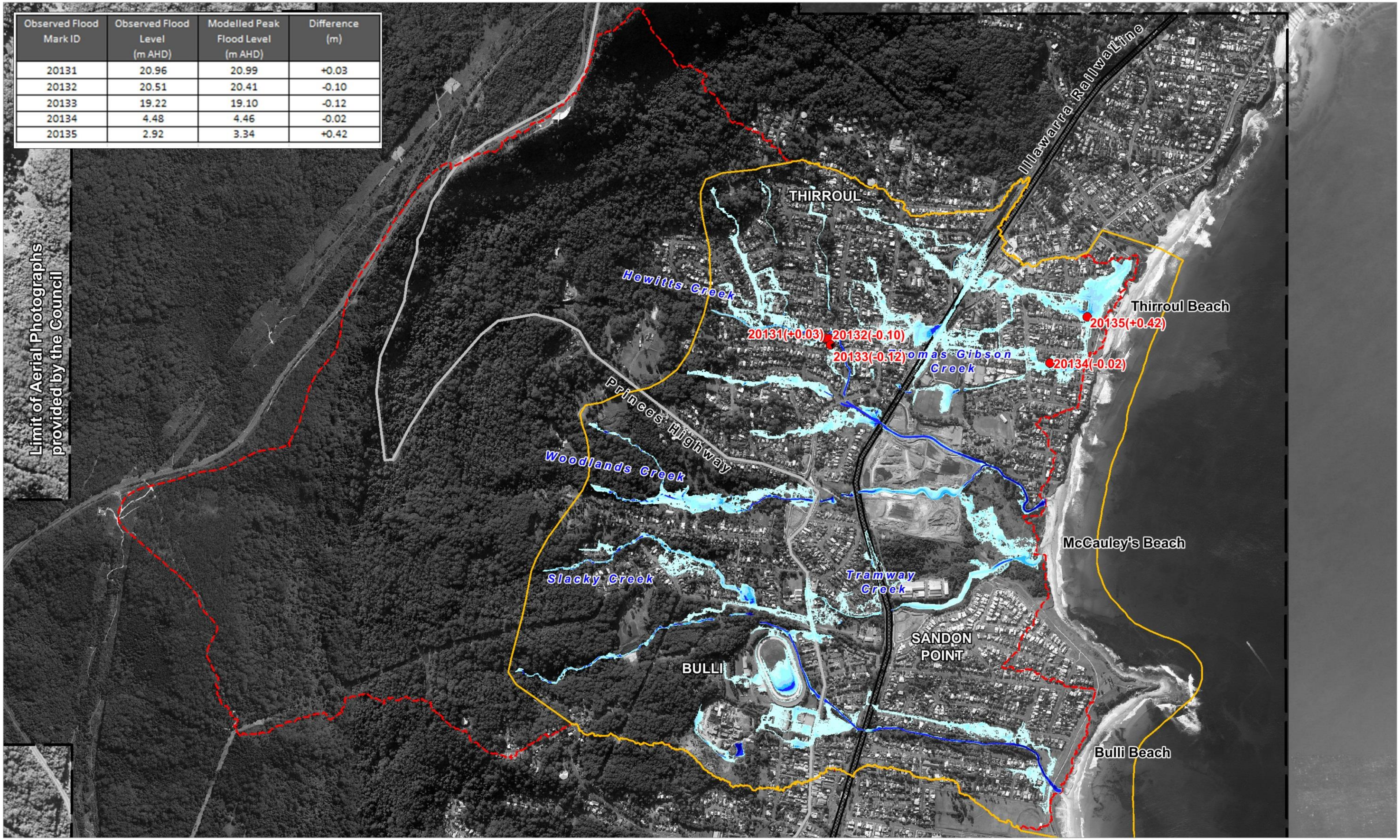
The remaining model parameters adopted for the August 1998 calibration event are unchanged for this model simulation.

7.3.5 February 2013 Model Validation Results

As part of the community consultation in 2013, residents were asked to identify where flooding was an issue at their property and/or on their street. Where residents provided flood mark information for the flood event on 24 February 2013, the reduced level of these flood marks were captured as part of the additional survey works (refer to Section 5.4). In addition to these flood mark levels, water level data were provided from two water level gauges operated by NSW Public Works MHL on behalf of OEH upstream and downstream of the road bridge on Hewitts Creek at Lawrence Hargrave Drive, Thirroul. This OEH data have been used as the main validation dataset as it is a gauged, quality controlled dataset. Information provided by NSW Public Works MHL indicates that the datum for the gauge at the mouth of Hewitts Creek needs to be reviewed and that the data for this gauge are unreliable. Data from this gauge have therefore not been used as part of this model validation.

Figure 7-15 is a map showing the depth and extent of the modelled flood envelope for the 24 February 2013 flood event. The locations of the observed flood levels captured as part of the additional survey works are shown with the flood level ID linked to a table providing details on the observed flood level, the corresponding peak modelled level and the difference in observed and modelled levels.

Observed Flood Mark ID	Observed Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (m)
20131	20.96	20.99	+0.03
20132	20.51	20.41	-0.10
20133	19.22	19.10	-0.12
20134	4.48	4.46	-0.02
20135	2.92	3.34	+0.42



LEGEND

- Observed Flood Mark
- 20131 (-0.25) ID Number (Difference between Observed and Modelled Level)
- 20132 (NFI) ID Number (No Flood Indicated)
- TUFLOW Model Boundary
- Catchment Boundary

Peak Flood Depths (m)
- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

Title:
Characteristics of Flooding in the Study Area for February 2013 Flood Event

Figure: 7-15

Rev: 1

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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7.3.5.1 Modelled and Observed Flood Levels

The TUFLOW model was found to correlate well with the majority of the observed flood marks within the study area for this validation event with the modelled peak flood levels generally within a ± 0.3m tolerance of the observed flood levels. The model also achieved a good fit to the recorded levels at the OEH gauges indicating that the hydraulic model parameters are of the correct order (refer to the discussion below on OEH Gauges). It is also important to note that only limited observed data are available for the Hewitts Creek and Thomas Gibson Creek catchments and no observed flood levels are available for this event in the catchments of Slacky Creek, Tramway Creek and Woodlands Creek.

The resident at 15 Lachlan Street, Thirroul, reported flood flows from Hewitts Creek crossed through the rear of their property and into Lachlan Street, Thirroul, which resulted in damage to their sheds. The model results correlate well with the observed flood levels and mechanisms at this property. In order to replicate the flood mechanisms and levels at this property, it was necessary to raise the bed levels in the creek adjacent to this property by an average of 0.5m Photographs taken after this flood events show rocks and debris along the channel bed in the vicinity of this property (refer to Figure 7-16)

Figure 7-16 Rocks and debris along Hewitts Creek (November 2013)

On Thomas Gibson Creek, the residents of 31 McCauley Street, Thirroul, reported that the creek running through their property broke its banks turned their house into an island, with water levels up to 0.5m deep. The model results correlate well with the observed flooding described at this location with the modelled water level within 0.02m of the observed flood level. At 2/27 Ocean Street, Thirroul, the residents reported that flooding from the stormwater drain rose to 0.5m at the road gutter, inundating the front garden of unit 3 front extending about 2 metres up the common driveway at this property. The model indicates more extensive flooding at this location than what was reported with a modelled flood level 0.41m higher than the observed flood level. As noted in Section 7.2.5, it is possible that the observed levels at this location are affected by the volume of water lost to the urban drainage system which has not been modelled as part of this study.

<u>OEH Gauges</u>

Figure 7-17 and Figure 7-18 shows a comparison of the modelled and observed flood levels at the OEH gauges upstream and downstream of Lawrence Hargrave Drive, Thirroul, on Hewitts Creek.

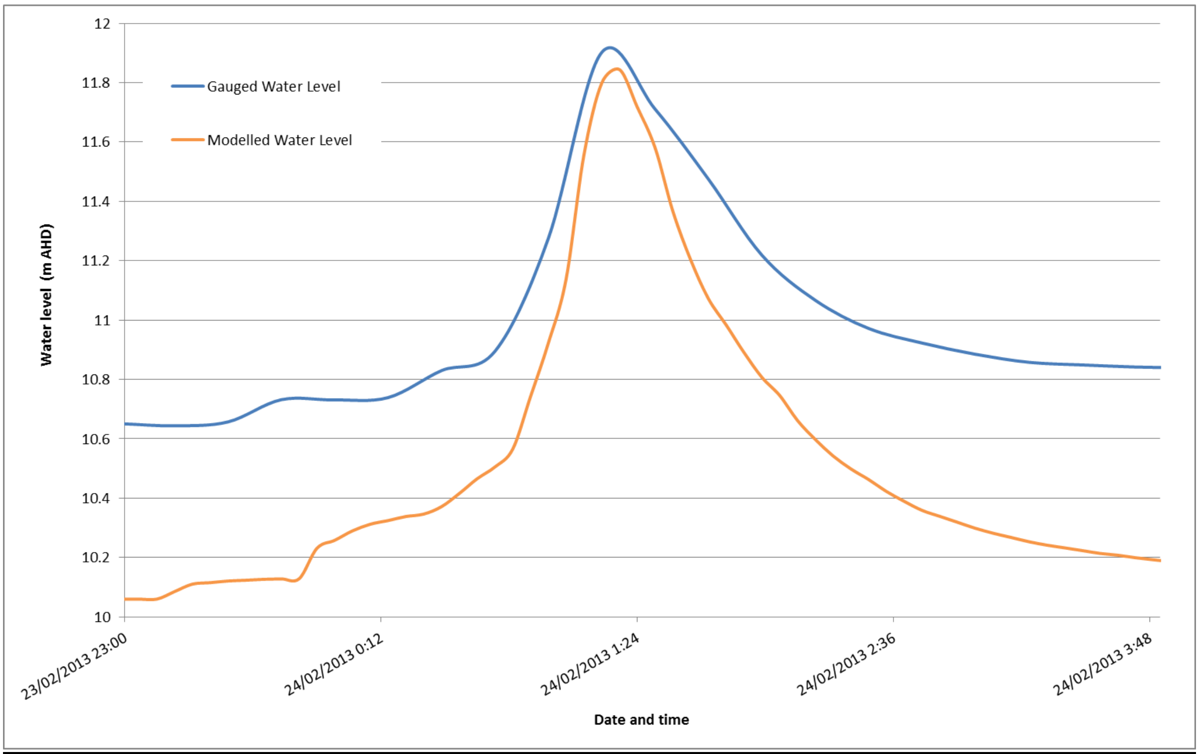


Figure 7-17 Comparison of modelled and observed flood levels at the OEH gauge upstream of Lawrence Hargrave Drive, Thirroul, Hewitts Creek – 23-24 February 2013 Event

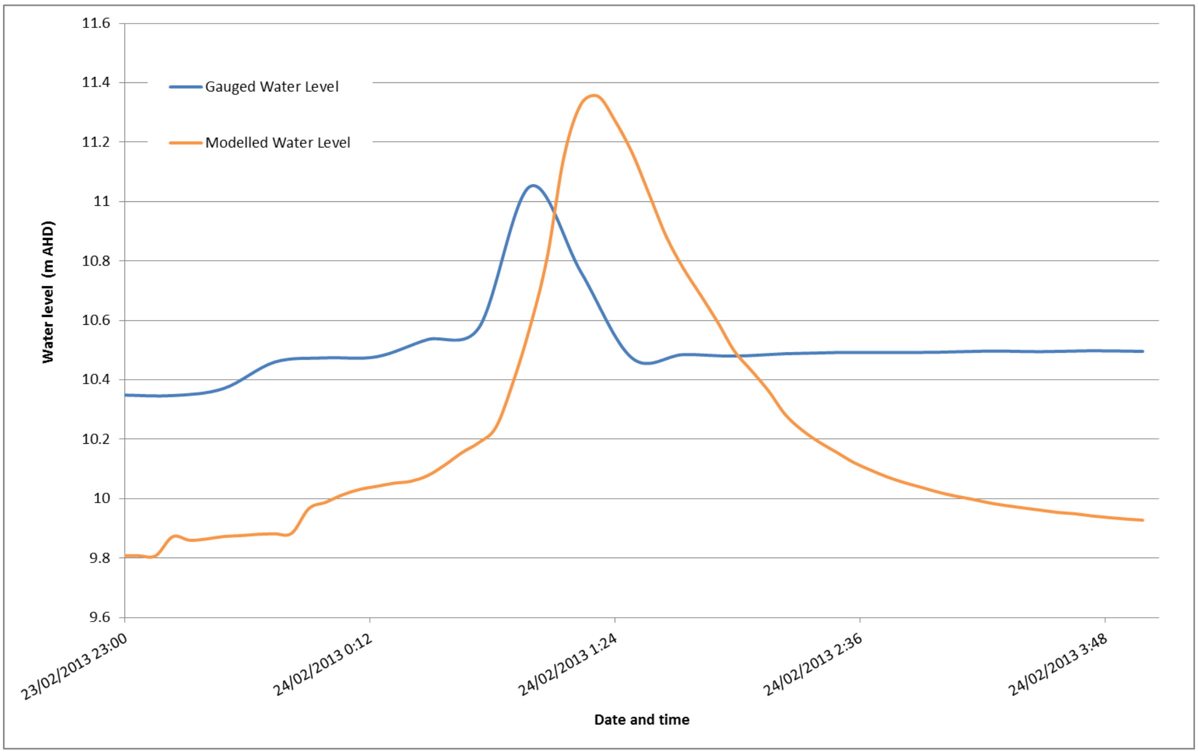


Figure 7-18 Comparison of modelled and observed flood levels at the OEH gauge downstream of Lawrence Hargrave Drive, Thirroul, Hewitts Creek – 23-24 February 2013 Event

At the upstream gauge, the results show a good correlation with the shape and timing of the peak of the flood event. The modelled peak level is within 0.1m of the recorded flood level while the timing of the peak coincides with the recorded flood peak. The model provides a poorer fit to the observed gauge data downstream of Lawrence Hargrave Drive, Thirroul. The results show that the peak of the modelled hydrograph peak is 0.3m higher than the recorded water level and the timing of the modelled peak is approximately 20 minutes later than the observed peak. The recorded data indicates that the hydrograph downstream of Lawrence Hargrave Drive, Thirroul, peaks approximately 15 minutes earlier than the hydrograph upstream of Lawrence Hargrave Drive, Thirroul. At this location, it is not possible for the gauge downstream of Lawrence Hargrave Drive, Thirroul, to peak earlier than the upstream gauge indicating that there are issues with the quality of the data from the gauge downstream of Lawrence Hargrave Drive, Thirroul.

The obvert of this structure is 12.55m AHD and is not overtopped for this flood event. The gauged flood level information indicates that this structure has a larger afflux effect on water levels compared to the results from the hydraulic model. Given the issues regarding the quality of the data from the downstream gauge and good correlation between the recorded and modelled levels at the OEH gauge upstream of Lawrence Hargrave Drive, Thirroul and the observed flood level locations elsewhere in the study area, the model is considered to perform well for this event and no changes were made to the key model parameters adopted for the August 1998 flood event.

7.4 April 1988 Model Validation

Widespread flooding occurred in Wollongong in late April 1988, particularly in the northern suburbs where the rainfall was most intense. Heavy rain developed in the area at the end of the month with approximately 326 mm of rain being recorded over three days at the Bulli Pass station, from 28 to 30 April.

7.4.1 Rainfall Data

Rainfall data are available from a number of daily and pluviograph stations for the April 1988 event. Pluviograph data is available from the Bulli Pass station and has been used to generate flows within the WBNM model for this event. Both the pluviograph data and daily rainfall data have been used to inform the development of rainfall isohyets for the April 1988 flood event (refer to Figure 7-20).

A hyetograph of the April 1988 storm recorded by the Bulli Pass station is shown in Figure 7-19 for a 6 hour period from 03:00 on the 30 April 1988 which represents the peak of the rainfall during the event.

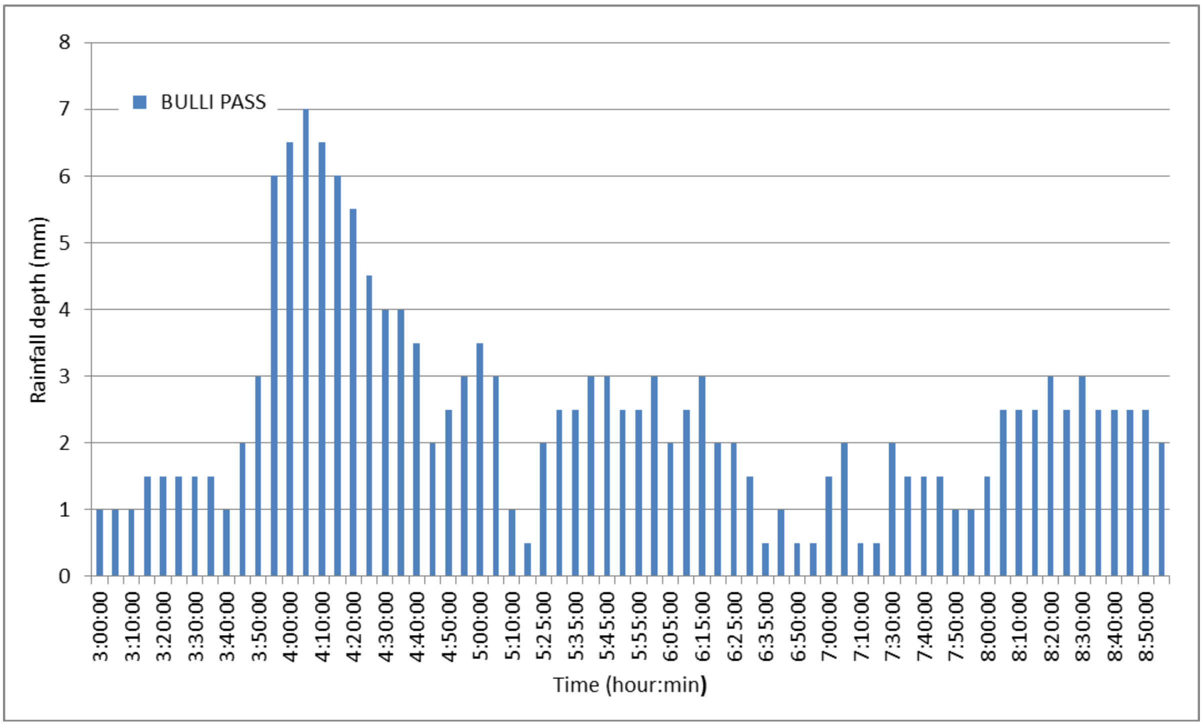


Figure 7-19 Event Hyetographs for Bulli Pass Station – 30 April 1988 Event

The main rainfall burst for the event occurred during a one-hour period from approximately 03:30 on 30 April 1988. A rainfall depth of 56 mm was recorded at the Bulli Pass station during this one-hour period. The 24 hour total for this station (from 9am on 30 April) was 233mm.

To gain an appreciation of the relative intensity of the April 1988 event, the recorded rainfall depths for various storm durations were compared with the design IFD data for the Hewitts Creek study area as shown in Figure 7-20.

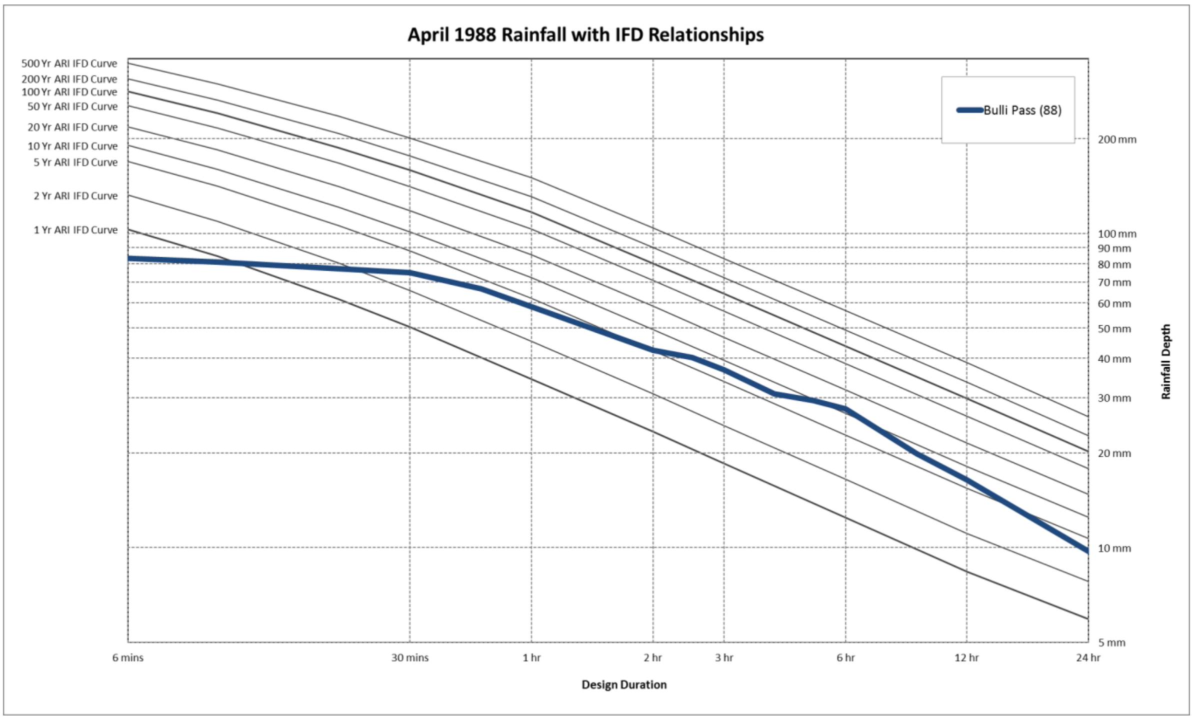
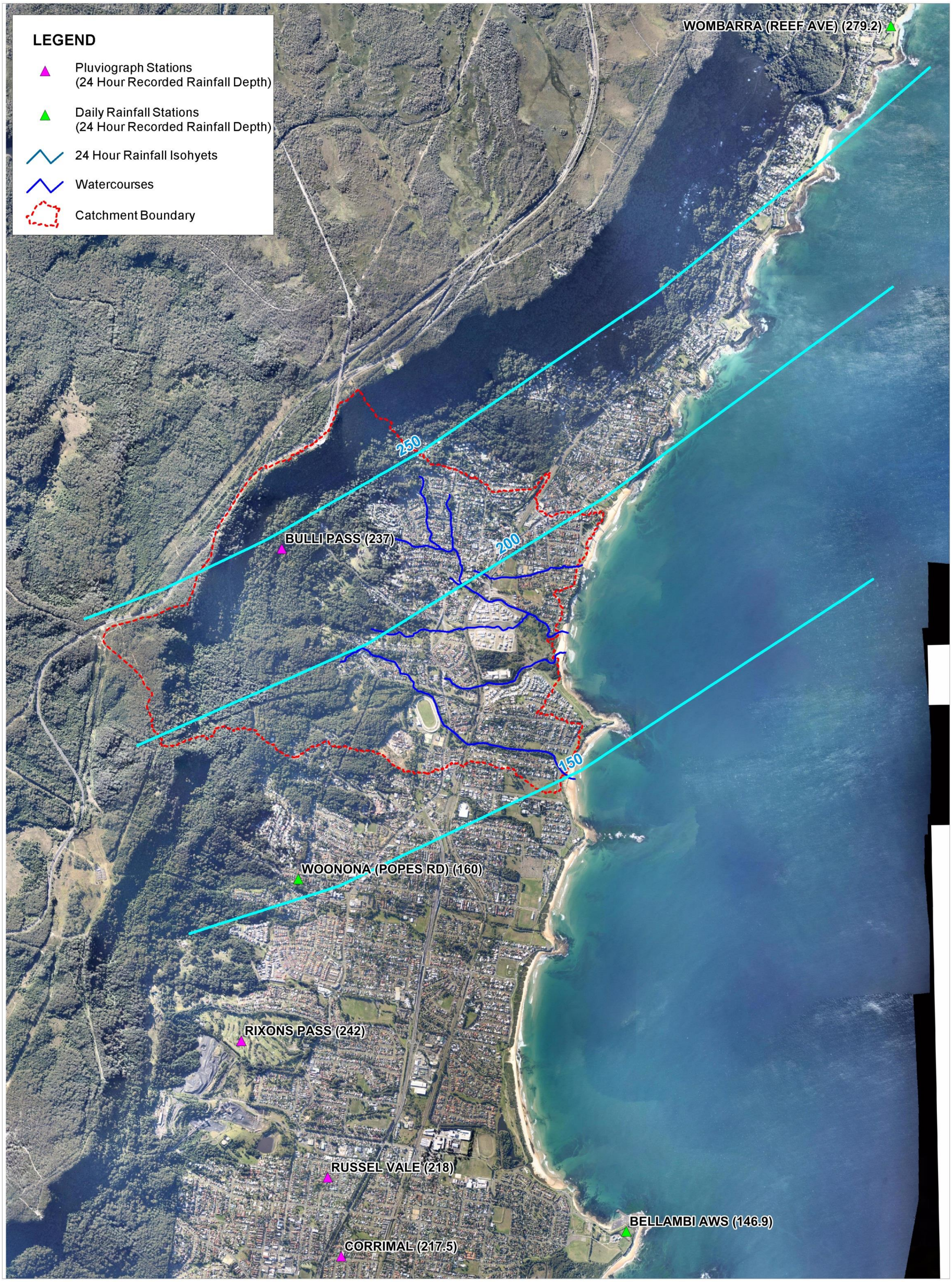


Figure 7-20 Comparison of April 1988 Rainfall with IFD Relationships

The April 1988 event generally tracks between a 5 year ARI and 10 year ARI for durations of greater than 2 hours. The following comparisons to design rainfall depths can be made:

- 6-hour duration – 165.5mm recorded compared with 160.2mm design 10 year ARI;
- 12-hour duration – 197.04mm recorded compared with 217.4mm design 10 year ARI; and
- 24-hour duration – 233.5mm recorded compared with 299.0mm design 10 year ARI.

Rainfall isohyets were estimated based on recorded 24-hr rainfall totals for the 1998 event as shown in Figure 7-21. This isohyet information was used to inform the spatial weighting of rainfall for this event within the WBNM model.



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April 1988 Rainfall Isohyets

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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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7.4.2 Antecedent Conditions

Heavy continuous rain was recorded throughout the City of Wollongong from 3 to 10 April 1988 (total depth of approximately 234 mm at Bulli Pass station) leaving the area in a highly saturated state. Fourteen days later, after some light intermittent rain, heavy rain again developed with approximately 326 mm of rain being recorded over the three days at Bulli Pass station from 28 to 30 April.

7.4.3 Downstream Boundary Condition

Based on details provided in the Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a), a dynamic downstream water level boundary for the April 1988 event has been developed. The peak of the overland flows coincides with a rising tide, with the peak of the tide occurring after the peak catchment runoff as shown in Figure 7-22. Therefore the entrance conditions would likely have been controlled by peak flood flows in the creek channels.

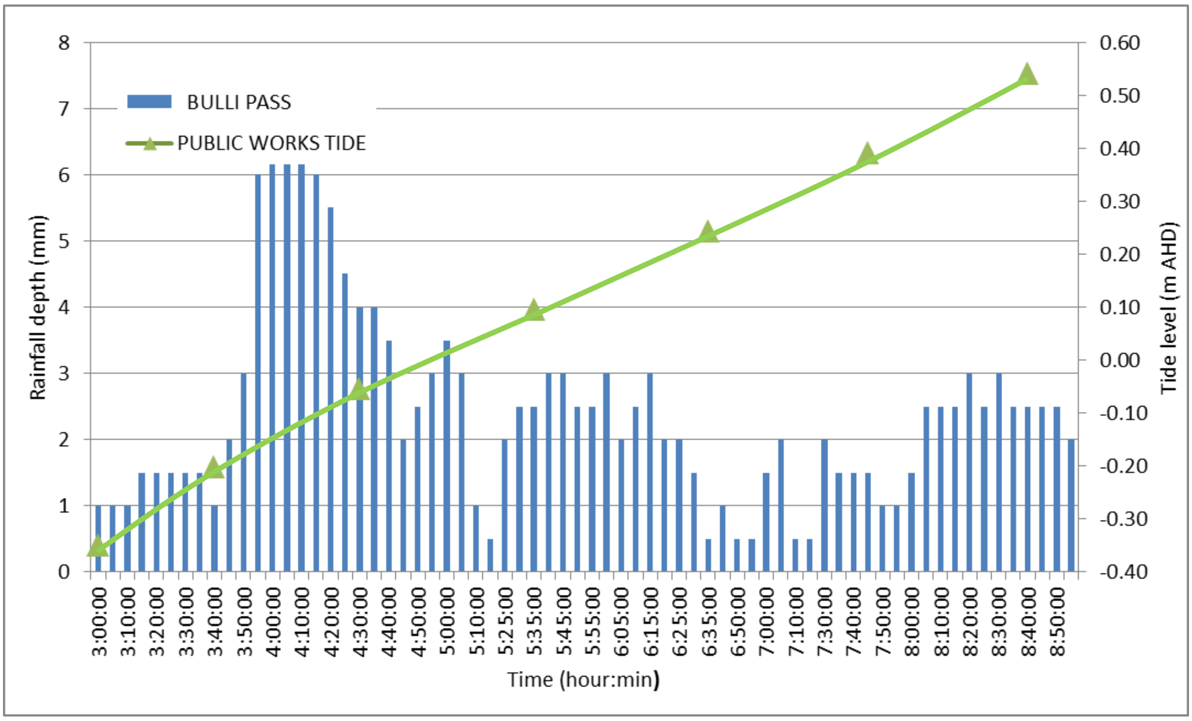


Figure 7-22 Comparison of Observed Rainfall and Tidal Data – 30 April 1988 Event

7.4.4 Model Parameters

Model validation is undertaken to test the appropriateness of the adopted calibration parameters for different historical events and provide an indication of parameter variability. During the August 1998 event, culvert and bridge blockages were a key influence on peak flood levels and these blockages were therefore used as a key calibration parameter within the hydraulic model for the 1998 event.

The culvert blockages adopted for the 1998 calibration event are unique to this flood event and have not been adopted for this model validation simulation. A unique set of culvert blockages have been adopted for this validation event based on a comparison of modelled and observed flood levels. Table 7-9 lists the culverts at which culvert blockages have been applied. All other culverts in the model have been modelled as unblocked.

Table 7-9 Modelled Structure Blockages - April 1988 Event

Structure ID (refer to Figure 6-2)	Location	Watercourse	% Blockage Applied to Structure
23	Lawrence Hargrave Drive, Bulli	Hewitts Creek	55
24	Lawrence Hargrave Drive, Bulli	Hewitts Creek	50

Based on a review of the antecedent conditions the loss rates adopted for the 1988 event are detailed in Table 7-10.

Table 7-10 Rainfall Loss Rates – April 1988 Event

Rainfall Loss Type	Values
Initial Loss – Pervious	0 mm
Initial Loss – Impervious	1 mm
Continuing Loss – Pervious	2.5mm/hour
Continuing Loss – Impervious	0mm/hour

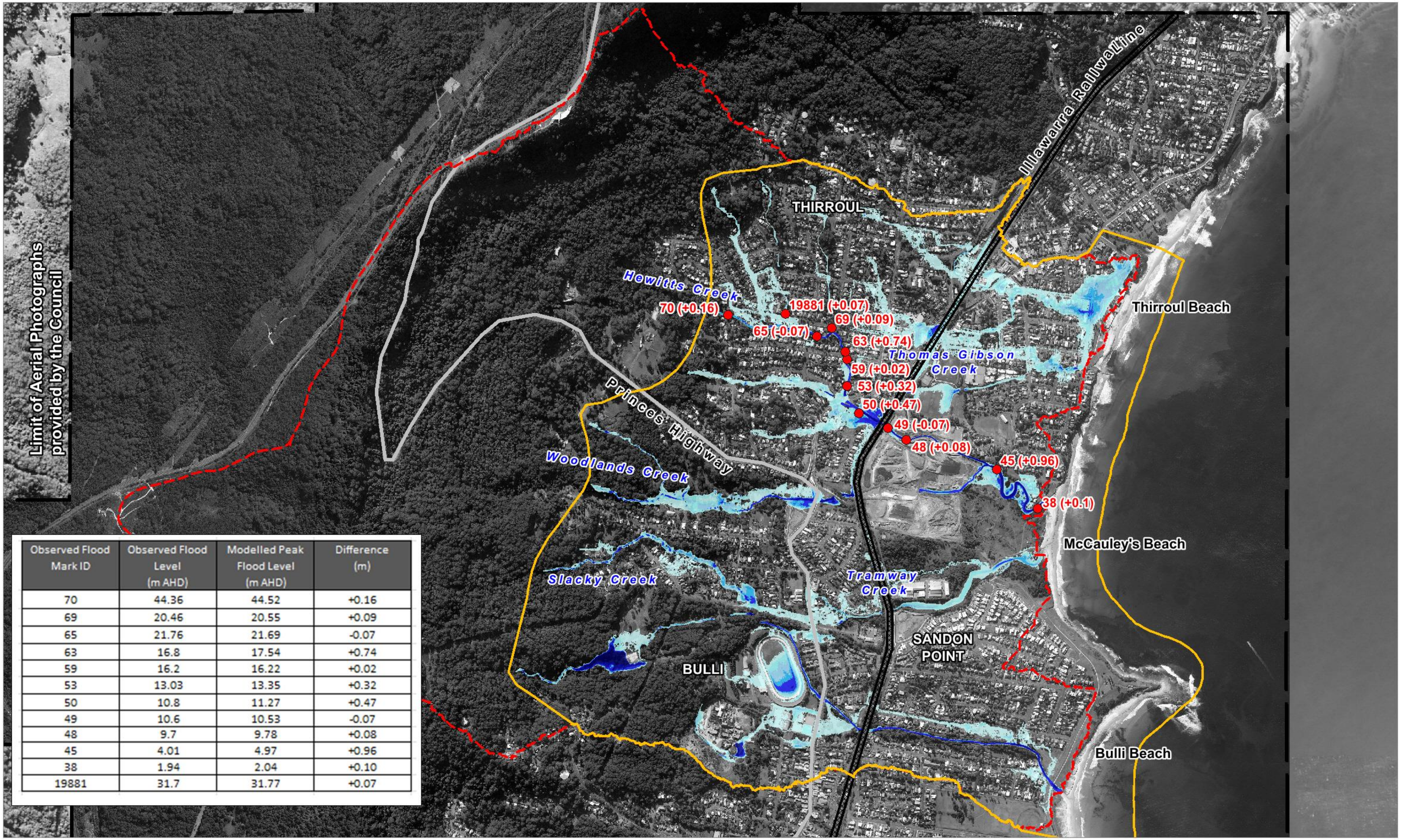
As per the 17 August 1998 flood event, the entrances of the creeks have been modelled as fixed with the geometry of the entrance defined at the start of the flood event. The geometry of the entrances defined for the 17 August 1998 flood event has been used for the 2013 validation event.

The remaining model parameters adopted for the 1998 calibration event are unchanged for this model simulation.

7.4.5 April 1988 Model Validation Results

Observed flood level information is available at a number of locations for the April 1988 event from the database of historic levels provided by Wollongong City Council (see Figure 4-3). This information was used to compare the modelled water levels to the observed levels for this flood event.

Figure 7-23 shows the results of modelling the 30 April 1988 flood event. The map shows the depth and extent of the modelled flood envelope. The locations of the observed flood levels shown with the flood level ID linked to a table providing information on the observed flood level, the corresponding peak modelled level and the difference between observed and modelled levels.



Observed Flood Mark ID	Observed Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (m)
70	44.36	44.52	+0.16
69	20.46	20.55	+0.09
65	21.76	21.69	-0.07
63	16.8	17.54	+0.74
59	16.2	16.22	+0.02
53	13.03	13.35	+0.32
50	10.8	11.27	+0.47
49	10.6	10.53	-0.07
48	9.7	9.78	+0.08
45	4.01	4.97	+0.96
38	1.94	2.04	+0.10
19881	31.7	31.77	+0.07

LEGEND

- Observed Flood Mark
- 65 (-0.07) ID Number (Difference between Observed and Modelled Level)
- TUFLOW Model Boundary
- Catchment Boundary

Peak Flood Depths (m)

- 0.00 to 0.25
- 0.25 to 0.50
- 0.50 to 0.75
- 0.75 to 1.00
- 1.00 to 2.00
- 2.00 to 4.00
- > 4.00

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Characteristics of Flooding in the Study Area for April 1988 Flood Event

Figure: 7-23

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7.4.5.1 Modelled and Observed Flood Levels

The TUFLOW model was found to correlate well with the majority of the observed flood marks along Hewitts Creek for this validation event with the modelled peak flood levels generally within a ± 0.3m tolerance of the observed flood levels. The overall fit between modelled and observed levels suggests that the hydraulic model parameters are of the correct order. Limited information is available on the location or description of the observed flood marks to undertake a check on these data. It is also important to note that no observed flood levels are available for this event in Slacky Creek, Tramway Creek, Woodlands Creek and Thomas Gibson Creek.

The calibrated TUFLOW model was found to give a good fit with the observed flood levels in the upstream reaches of Hewitts Creek at George Street. At Lachlan Street, the modelled flood level is approximately 0.7m higher than the observed flood levels upstream of this culvert, with the model predicting a 1.3m afflux across this structure for this event against an observed afflux of 0.6m. Given the dimensions of this structure and the flows for this event, the modelled afflux at the peak of the flood is within the expected range suggesting that observed level upstream of culvert may be in error.

At Lawrence Hargrave Drive, the modelled flood levels correlate reasonably well with the observed levels upstream of the road culverts. Downstream of the road culverts, the model is over-predicting water levels when compared to the observed flood levels. The levels along this reach are controlled by the afflux at the downstream Illawarra Railway underpass. Given the good correlation in levels on Hewitts Creek directly downstream of this railway underpass and upstream of Lawrence Hargrave Drive, the recorded level at this location may be erroneous and the model is considered to be performing well.

The alignment of Hewitts Creek has been modified south of Corbett Avenue for this flood event based on river centreline data provided as part of historical survey plans. As limited information is available on this old channel alignment, the dimensions and elevations of this channel have been estimated based on existing channel details. The model results indicate that the model is over-predicting water levels at the location of this old channel alignment by approximately 1m. This over estimation in water levels is likely to be as a result of the approximation of the creek channel and floodplain elevations along the old channel alignment. Given the good correlation in levels upstream and downstream of this location, no further changes were made to the model schematisation to improve the model results at one isolated flood mark.

Given that the model generally correlates well with the majority of the observed flood marks along Hewitts Creek for this event, no changes were made to the key model parameters adopted for the 1998 calibration event.

7.5 Determination of Design Model Parameters

Throughout the model calibration process, emphasis has been placed on reaching agreement between recorded and simulated flood conditions with respect to peak water levels and relative timing of occurrence (where data supports this) in addition to replicating the various observed flooding mechanisms and direction of floodplain flow.

The model calibration achieved a good agreement in regards to observed conditions within the majority of the Hewitts Creek study area for the principal calibration event of August 1998. Based on limited observed flood data, the model validation results indicate that the model parameters adopted for the calibration event are appropriate, supported by the results from the model validation against the February 2013 and April 1988 events. The final hydraulic model parameter values adopted from the calibration and validation exercise are shown in Table 7-11. Further details of the final hydraulic model parameter values adopted from the calibration and validation exercise for the design flood conditions are discussed in Section 8.

Table 7-11 Adopted Model Parameters

Model	Parameter	Value	Comment
WBNM	Initial Loss – Pervious	0 mm	An initial loss of 0mm is based on the likelihood that the short duration rainfall events that result in the critical durations for this catchment will not occur in isolation but part of a longer rainfall event with preceding rainfall to wet the catchment. Research has shown that the relatively short bursts of intense rainfall which are critical for maximizing flood flows in the coastal creek systems of the Wollongong area (Rigby et al, 2003) are also commonly associated with more general lead rainfall.
	Initial Loss – Impervious	0 mm	
	Continuing Loss – Pervious	2.5mm/hr	Model calibration and validation results indicate that the adopted continuing loss parameters gave a reasonable fit to historic flood data for the calibration and validation events. These values are similar to adopted design continuing loss rate as recommended in AR&R (Pilgrim, 2001) and adopted for previous studies in the Wollongong region.
	Continuing Loss – Impervious	0mm/hr	
	WBNM Lag Parameter	1.29	Value based on results of calibration and validation and values adopted as part of previous calibration and validation work undertaken for the Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a). The adopted value was applied globally for the entire catchment as recommended in WBNM (WBNM, 2007).
TUFLOW	Blockage of culverts and bridges	Variable	Culvert and bridge blockages were a key calibration parameter within the hydraulic model for the August 1998 flood event. A unique set of culvert blockages has been applied for the design flood events based on Council's Conduit Blockage Policy (Wollongong City Council, 2009) (Refer to Section 8)
	Downstream Boundary Condition	Variable	The values adopted for the calibration and validation events are based on recorded tide levels from the Port Kembla tide gauge (August 1998 and February 2013 events) and the OEH Waverider buoy off Port Kembla (April 1998 event). Further details on the design flood levels and coincident catchment and ocean flooding conditions are discussed in Section 8.
	Minimum elevation at the entrance of the Creeks	Slacky Creek – 1 m AHD Tramway Creek – 0.8m AHD	Observed flood level data along the downstream reaches of Slacky Creek, Hewitts Creek and Flanagans Creek for the August 1998 flood event was used to inform the most appropriate geometry of the entrance to the creeks. A number of model simulations were undertaken with various fixed entrance dimensions until an acceptable level of agreement between observed

Model	Parameter	Value	Comment
		Hewitts Creek – 0.1m AHD Flanagans Creek – 0.6m AHD	and modelled water levels was reached. The lateral width at the entrances to the creeks was limited by the extent of the coastal dunes. The minimum elevation of the entrance to the creeks was limited to 0m AHD where a 'control' on the maximum depth of erosion is inferred to occur by the presence of a rock shelf. The entrance to Thomas Gibson Creek appears less affected by the build-up of beach berm and the reduced levels at the entrance of this creek have not been adjusted as part of model calibration and validation.
	Manning's n (channel)	0.035 - 0.10	Considered representative of the creeks in the catchment.
	Manning's n (floodplain)	0.03 – 1.00	Variability largely reflects land use on the floodplain (forested, roads, urban lots, parklands, buildings, etc.).

8 DESIGN FLOOD CONDITIONS

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

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

Table 8-1 lists the design events simulated and includes a definition of AEP and the ARI equivalent.

Table 8-1 Design Flood Terminology

ARI[1]	AEP[2]	Comments
500 years	0.2%	A flood or combination of floods which represent the worst case scenario likely to occur on average once every 500 years
200 years	0.5%	As for the 0.2% AEP flood but with a 0.5% probability or 200 year return period
100 years	1%	As for the 0.5% AEP flood but with a 1% probability or 100 year return period
50 years	2%	As for the 0.5% AEP flood but with a 2% probability or 50 year return period
20 years	5%	As for the 0.5% AEP flood but with a 5% probability or 20 year return period
10 years	10%	As for the 0.5% AEP flood but with a 10% probability or 10 year return period
5 years	20%	As for the 0.5% AEP flood but with a 20% probability or 5 year return period
Extreme Flood/PMF[3]		A flood or combination of floods which represent an extreme scenario

1 Average Recurrence Interval (years)
2 Annual Exceedance Probability (%)
3 PMF (Probable Maximum Flood) is not necessarily the same as an Extreme Flood

The design events simulated include the PMF event, 0.2%, 0.5%, 1%, 2%, 5%, 10%, and 20% AEP events for catchment derived flooding and a “Normal Tide” (0.6m AHD) and “Storm Tide” (2.3m AHD and 2.6m AHD) for ocean/tidal derived flooding. The 1% AEP flood is generally used as a reference flood for land use planning and control and as a baseline event for evaluation of model sensitivity.

In accordance with current engineering practice and documentation provided by the Department of Environment and Climate Change and Water (DECCW), a *“Flood Envelope”* approach was adopted for defining design water surface levels and flow velocities. The flood envelope approach incorporated the following variables:

- Design rainfall;
- Ocean boundary condition;
- Structure blockages in line with Wollongong City Council’s Conduit Blockage Policy (Wollongong City Council, 2009);

- Initial water levels in the ICOLLs; and
- The entrance conditions at the ICOLLs.

Table 8-2 lists the design flood envelopes and the variables modelled for each design flood envelope. Further discussion on each of these variables is in the following report sections.

Table 8-2 Design Flood Combinations

Design Flood Envelope	Variables				
	Design Rainfall	Ocean Boundary Condition	Structure Blockage Scenarios[1)]	Initial Water Level in ICOLLs	Entrance Conditions at the ICOLLs
20% AEP (5 year ARI)	20% AEP (5 year ARI) 2hr and 9hr durations	Normal Tide (0.63 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
10% AEP (10 year ARI)	10% AEP (10 year ARI) 2hr and 9hr durations	Normal Tide (0.63 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
5% AEP (20 year ARI)	5% AEP (20 year ARI) 2hr and 9hr durations	Normal Tide (0.63 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
	20% AEP (5 year ARI) 2hr and 9hr durations	5% AEP Storm Tide (2.3 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
2% AEP (50 year ARI)	2% AEP (50 year ARI) 2hr and 9hr durations	Normal Tide (0.63 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
	20% AEP (5 year ARI) 2hr and 9hr durations	5% AEP Storm Tide (2.3 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
1% AEP (100 year ARI)	1% AEP (100 year ARI) 2hr and 9hr durations	Normal Tide (0.63 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
	1% AEP (100 year ARI) 2hr and 9hr durations	5% AEP Storm Tide (2.3 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open

Design Flood Envelope	Variables				
	Design Rainfall	Ocean Boundary Condition	Structure Blockage Scenarios 1)	Initial Water Level in ICOLLs	Entrance Conditions at the ICOLLs
	5% AEP (20 year ARI) 2hr and 9hr durations	1% AEP Storm Tide (2.6 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
0.5% AEP (200 year ARI)	0.5% AEP (200 year ARI) 2hr and 9hr durations	Normal Tide (0.63 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
	5% AEP (20 year ARI) 2hr and 9hr durations	1% AEP Storm Tide (2.6 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
0.2% AEP (500 year ARI)	0.2% AEP (500 year ARI) 2hr and 9hr durations	Normal Tide (0.63 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
	5% AEP (20 year ARI) 2hr and 9hr durations	1% AEP Storm Tide (2.6 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
PMF	PMF	Normal Tide (0.63 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
	5% AEP (20 year ARI) 2hr and 9hr durations	1% AEP Storm Tide (2.6 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open

1) Refer to Table 8-5 for further information

8.1 Design Rainfall

Design rainfall parameters are derived from standard procedures defined in AR&R (Pilgrim, 2001) which are based on statistical analysis of recorded rainfall data across Australia. The derivation of location specific design rainfall parameters (e.g. WBNM Lag Parameter, rainfall depth and temporal patterns) for the Hewitts Creek study area is presented below.

8.1.1 WBNM Lag Parameter

A Lag parameter value of 1.29 has been adopted based on the results from the model calibration and validation and values adopted for other flood studies within the Wollongong region. The adopted value is applied globally for the entire catchment as recommended in WBNM (Boyd et al, 2007).

8.1.2 Rainfall Depths

Design rainfall depth is based on the generation IFD design rainfall curves utilising the procedures outlined in Pilgrim (2001). These curves provide rainfall depths for various design magnitudes (up to the 1% AEP) and 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, 2001). The ARI of a PMP/PMF event ranges between 10^4 and 10^7 years and is beyond the “credible limit of extrapolation”. That is, it is not possible to use rainfall depths determined for the more frequent events (e.g. 1% AEP event and less) to extrapolate the PMP. 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 Wollongong region.

Due to the relatively small size of the catchment and adopting a conservative approach, no areal reduction factor was applied in this study. The areal reduction factor takes into account the unlikelihood that larger catchments will experience rainfall of the same design intensity (e.g. 1% AEP event) over the entire area.

8.1.3 Temporal Patterns

Temporal patterns are required to define what percentage of the total rainfall depth occurs over a given time interval throughout the storm duration. Zone 1 temporal patterns from AR&R (Pilgrim, 2001) are built into the WBNM model and have been used for this study.

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-areas. 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.

8.1.4 Rainfall Losses

Table 8-3 provides details of the initial and continuing rainfall losses applied to pervious and impervious areas of the catchment.

Table 8-3 Initial and Continuing Rainfall Losses

Rainfall Loss Type	Surface Type	Value
Initial Loss	Pervious	0 mm
Initial Loss	Impervious	0 mm
Continuing Loss	Pervious	2.5 mm/h
Continuing Loss	Impervious	0 mm/h

An initial loss of 0mm is based on the likelihood that the short duration rainfall events that result in the critical durations for this catchment will not occur in isolation but part of a longer rainfall event with preceding rainfall to wet the catchment. Research has shown that the relatively short bursts of intense rainfall which are critical for maximising flood flows in the coastal creek systems of the Wollongong region (Rigby et al, 2003) are also commonly associated with more general lead rainfall.

These values of initial and continuing losses are consistent with the recommended ranges for design event losses in AR&R (Pilgrim, 2001) and are similar to those used in the hydrologic model calibration and verification events. These values are also consistent with other flood studies in the Wollongong region as indicated in Table 8-4.

The applied losses are linearly varied across the study area based on the impervious percentage (i.e. 100% impervious – 0mm initial and continuing loss applied) of the land use surface type. As outlined in Section 6.4.1.2, the percentages of pervious and impervious areas have been estimated based on aerial photography and cadastral data supplied by Council.

Table 8-4 Losses Adopted for Other Flood Studies in the Wollongong Region

Flood Study	Initial Loss	Continuing Loss
Towradgi Creek Flood Study (Bewsher Consulting Pty Ltd., 2003a)	10mm*	2.5mm/h*
Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002)	0mm	2.5mm/hr
Duck Creek Flood Study (BMT WBM Pty Ltd, 2012)	20mm	2.5mm/h
Fairy and Cabbage Tree Creeks Flood Studies (BMT WBM Pty Ltd, 2009)	0mm	2.0mm/h
Combined Catchments of Whartons, Collins and Farrahars Creeks, Bellambi Gully and Bellambi Lake Flood Study (Lyall and Associates, 2011)	10mm*	2.5mm/h*

*Design Storm of between 5 and 500 years

8.1.5 Critical Storm Duration

A range of storm durations were modelled in order to identify the critical storm duration for design event flooding in the catchment. The 10% AEP and 1% AEP events have been selected to allow a comparison of the critical storm duration between a frequent and a rarer flood event. The event durations simulated were the 15min, 25min, 30min, 45min, 1hr, 1.5hr, 2hr, 3hr, 4.5hr, 6hr and 9hr and 11hr. Both the 10% AEP and 1% AEP events have been run assuming all the culverts are unblocked. An additional simulation has been undertaken for the 1% AEP event to check the impact of a scenario whereby all of the culverts are fully blocked based on Council's Conduit Blockage Policy (Wollongong City Council, 2009). This blockage scenario assessment has been undertaken to evaluate whether there are any changes in the critical storm duration as there is potential for longer duration events with a greater rainfall volume to produce higher flood levels due to blockages compared with the shorter duration events (i.e. the critical storm duration for all culverts unblocked).

The 2hr and 9hr storm durations have been identified as critical for the Hewitts Creek study area. For those locations that do not have a critical storm duration of 2hr and 9hr, the peak flood level from these durations are within 0.1m of the peak flood levels generated by the critical storm durations.

8.2 Ocean Boundary Conditions

DECCW's guideline entitled *Flood Risk Management Guide: Incorporating Sea Level Rise Benchmarks in Flood Risk Assessments* (DECCW, 2010) contains interim advice in relation to the coincident catchment and ocean flooding conditions which should be adopted when preparing flood studies in coastal areas. The interim advice is an update of the Department of Environment and Climate Change (DECC) draft *Floodplain Management Guideline No. 5 Ocean Boundary Conditions, 2004* and will be subject to review following the release of the update of Australian Rainfall and Runoff (IEAust, 1998).

The interim advice recommends that peak "*Storm Tide"* levels of 2.3 m AHD and 2.6 m AHD be adopted for deriving design flood envelopes for events of 5% AEP and 1% AEP respectively.

When modelling *"Storm Tide"* conditions, a dynamic boundary condition has been applied as shown in Figure 8-1. The timing of the peak tide level was adjusted to coincide with the peak catchment inflow for the critical rainfall event durations. When modelling the "Normal Tide" condition, a fixed tide level of 0.63m AHD was applied in the model.

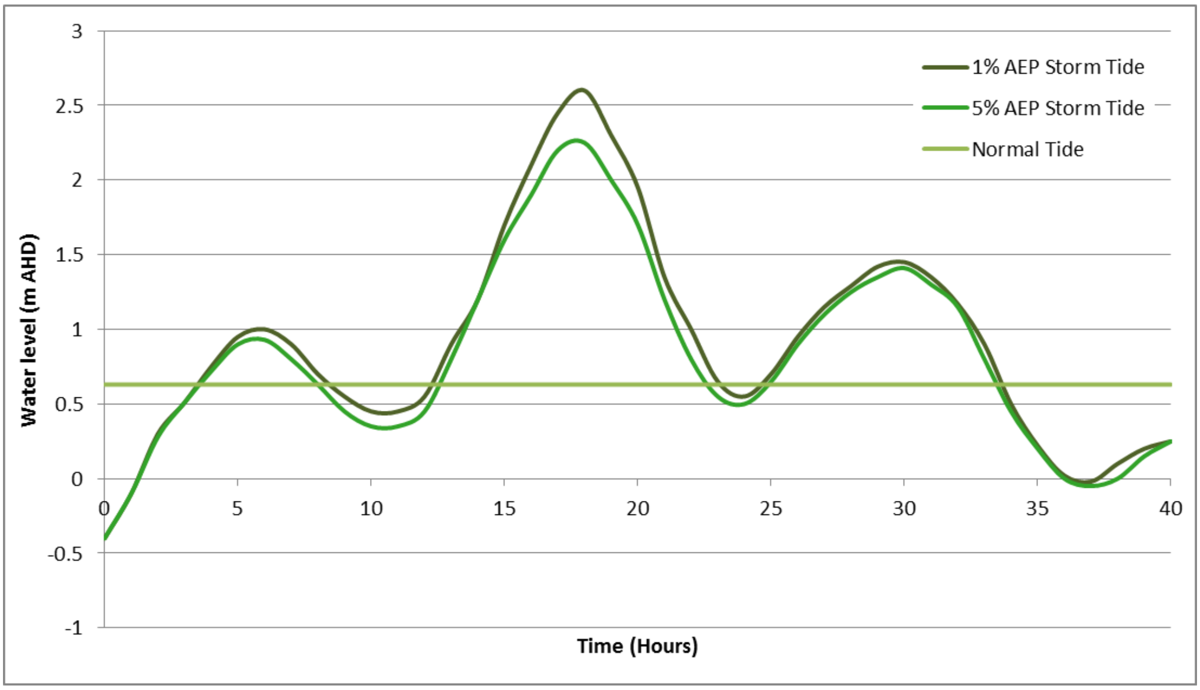


Figure 8-1 Design Ocean Boundary Conditions

8.3 Structure Blockage Scenarios

A range of blockage scenarios were modelled in order to identify the critical blockage scenario for design event flooding in the catchment. In developing the blockage scenarios for the Hewitts Creek study area, consideration has been given to the blockages applied in the previous Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a). Knowledge on flow patterns from the 2D modelling undertaken as part of the model calibration and validation for the current flood study has also been used in identifying the culvert blockage scenarios. Table 8-5 provides details of the culvert blockage scenarios modelled in order to identify the critical blockage scenario for the study area.

Table 8-5 Culvert Blockage Scenarios

Blockage Scenario	Details
B01	All culverts and bridges unblocked
B02	All culverts and bridges blocked as per the Councils Conduit Blockage Policy (Wollongong City Council, 2009).
B03	All culverts unblocked except for the culvert on the tributary of Slacky Creek off Hobart Street, Bulli and the culverts on Slacky Creek at Hobart Street, Bulli and the coal haulage embankment of the now disused railway, Bulli. The blockages applied to the culverts have been based on the Councils Conduit Blockage Policy (Wollongong City Council, 2009). This scenario maximises the diversion in flows eastwards along Hobart Street, Bulli, to Tramway Creek.
B04	All culverts unblocked except for the culverts on Slacky Creek within the detention basin at Black Diamond Place, Bulli and at the Illawarra Railway, Bulli. The blockages applied to the culverts have been based on the Councils Conduit Blockage Policy (Wollongong City Council, 2009). This scenario maximises the diversion in flows eastwards along Beacon Avenue, Bulli.
B05	All culverts unblocked except for the Illawarra Railway underpass at Beacon Avenue, Bulli. The blockage applied to the underpass has been based on the Councils Conduit Blockage Policy (Wollongong City Council, 2009). This scenario maximises the diversion in flows eastwards along Slacky Creek.
B06	All culverts unblocked except for the culvert on Woodlands Creek at the Illawarra Railway, Thirroul. The blockage applied to the culvert has been based on the Councils Conduit Blockage Policy (Wollongong City Council, 2009). This scenario maximises the diversion in flows northwards to Hewitts Creek.
B07	All culverts unblocked except for the culvert on the western tributary of Hewitts Creek at Deborah Avenue, Thirroul. The blockage applied to this culvert has been based on the Councils Conduit Blockage Policy (Wollongong City Council, 2009). This scenario maximises the diversion in flows eastwards to the eastern tributary of Hewitts Creek.
B08	All culverts unblocked except for the culvert on the eastern tributary of Hewitts Creek at Georges Street, Thirroul. The blockage applied to the culvert has been based on the Councils Conduit Blockage Policy (Wollongong City Council, 2009). This scenario maximises the diversion in flows eastwards to Thomas Gibson Creek.
B09	All culverts unblocked except for the culvert on Hewitts Creek at Lachlan Street, Thirroul. The blockage applied to the culvert has been based on the Councils Conduit Blockage Policy (Wollongong City Council, 2009). This scenario maximises the diversion in flows eastwards to Thomas Gibson Creek.
B10	All culverts unblocked except for the culvert on Hewitts Creek at the Illawarra Railway. The blockage applied to the culvert has been based on the Councils Conduit Blockage Policy (Wollongong City Council, 2009). This scenario maximises the diversion in flows southwards to Woodlands Creek.

The critical blockage scenario for the study area has been assessed for the 1% AEP design rainfall event (2 hour duration) with normal tide conditions. Figure 8-2 shows the critical blockage scenarios across the study area. The figure contains a table comparing the differences in peak water levels for blockage scenario B02 (i.e. the blockage scenario that is most critical across the majority of the study area), to the remaining blockage scenarios at selected reporting locations. The results indicate that:

- Blockage scenarios B01, B02 and B04 result in the highest water levels across the study area;
- Blockage scenarios B03, B04, B05, B07, B08 and B10 do not produce water levels which are higher than blockage scenarios B01, B02 and B04 at the selected reporting locations;
- Blockage scenario B06 and B09 results in marginally higher water levels at reporting locations 12, 13 and 18 respectively when compared to results from B01, B02 and B04. These localised differences in levels are less than or equal to 0.05m.

8.4 ICOLLs

8.4.1 Initial Water Level in the ICOLLs

During periods of little or no rainfall, water levels upstream of the entrance to ICOLLs are governed primarily by the rate of groundwater outflows through the sand berm which separates the freshwater from the Tasman Sea. Coastal processes result in a continuing cycle of a beach berm developing at the creek entrances with dry weather catchment flow ponding behind the berm. Naturally when the water level gets higher than the berm, the sand is scoured and the ICOLL is connected to the ocean.

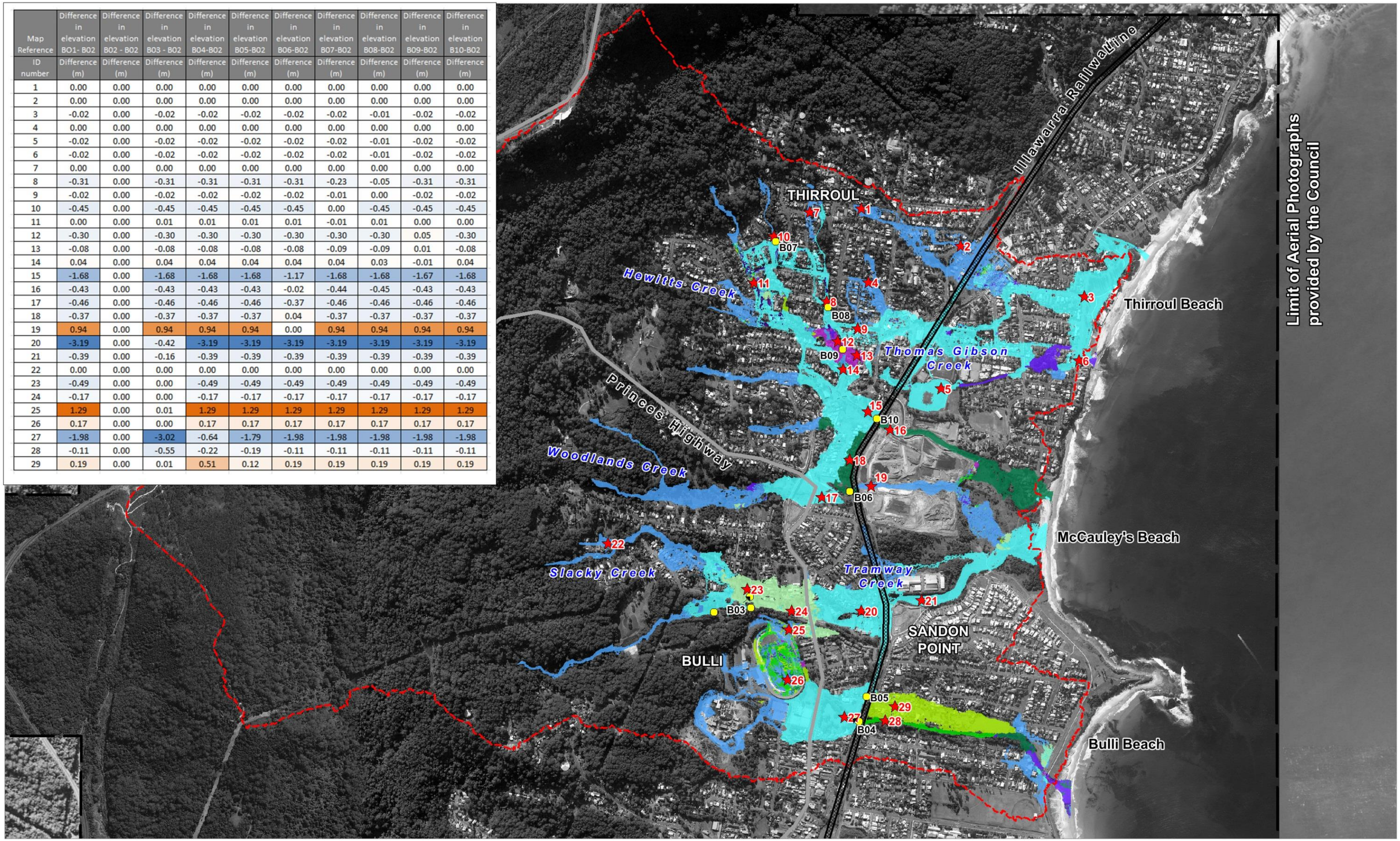
Within the modelled study area, Flanagans Creek, Slacky Creek and Hewitts Creek are defined as ICOLLs. For the design model runs, the entrance to the ICOLLs have been modelled as open based on the assumption that the berm has been breached under natural conditions. Under these conditions, the initial water level in ICOLLs is governed by the design ocean boundary conditions and initial design model inflows.

8.4.2 Entrance Conditions at the ICOLLs

The highly dynamic nature of the entrances of the creeks with respect to beach berm patterns presents challenges in defining appropriate initial conditions of the entrance channel geometry for hydraulic modelling. The entrances of the creeks have been modelled as fixed with the geometry of the entrances defined at the start of the flood event. The geometry of the entrances have been defined based on the results of the calibration and validation events. Based on the calibration and validation model results, the following minimum reduced levels have been adopted at the entrances of the ICOLLs within the study area:

- Slacky Creek – 1 m AHD;
- Tramway Creek – 0.8m AHD;
- Hewitts Creek – 0.1m AHD; and
- Flanagans Creek – 0.6m AHD.

Map Reference ID number	Difference in elevation B01- B02 Difference (m)	Difference in elevation B02 - B02 Difference (m)	Difference in elevation B03 - B02 Difference (m)	Difference in elevation B04-B02 Difference (m)	Difference in elevation B05-B02 Difference (m)	Difference in elevation B06-B02 Difference (m)	Difference in elevation B07-B02 Difference (m)	Difference in elevation B08-B02 Difference (m)	Difference in elevation B09-B02 Difference (m)	Difference in elevation B10-B02 Difference (m)
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3	-0.02	0.00	-0.02	-0.02	-0.02	-0.02	-0.02	-0.01	-0.02	-0.02
4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	-0.02	0.00	-0.02	-0.02	-0.02	-0.02	-0.02	-0.01	-0.02	-0.02
6	-0.02	0.00	-0.02	-0.02	-0.02	-0.02	-0.02	-0.01	-0.02	-0.02
7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8	-0.31	0.00	-0.31	-0.31	-0.31	-0.31	-0.23	-0.05	-0.31	-0.31
9	-0.02	0.00	-0.02	-0.02	-0.02	-0.02	-0.01	0.00	-0.02	-0.02
10	-0.45	0.00	-0.45	-0.45	-0.45	-0.45	0.00	-0.45	-0.45	-0.45
11	0.00	0.00	0.01	0.01	0.01	0.01	-0.01	0.01	0.00	0.00
12	-0.30	0.00	-0.30	-0.30	-0.30	-0.30	-0.30	-0.30	0.05	-0.30
13	-0.08	0.00	-0.08	-0.08	-0.08	-0.08	-0.09	-0.09	0.01	-0.08
14	0.04	0.00	0.04	0.04	0.04	0.04	0.04	0.03	-0.01	0.04
15	-1.68	0.00	-1.68	-1.68	-1.68	-1.17	-1.68	-1.68	-1.67	-1.68
16	-0.43	0.00	-0.43	-0.43	-0.43	-0.02	-0.44	-0.45	-0.43	-0.43
17	-0.46	0.00	-0.46	-0.46	-0.46	-0.37	-0.46	-0.46	-0.46	-0.46
18	-0.37	0.00	-0.37	-0.37	-0.37	0.04	-0.37	-0.37	-0.37	-0.37
19	0.94	0.00	0.94	0.94	0.94	0.00	0.94	0.94	0.94	0.94
20	-3.19	0.00	-0.42	-3.19	-3.19	-3.19	-3.19	-3.19	-3.19	-3.19
21	-0.39	0.00	-0.16	-0.39	-0.39	-0.39	-0.39	-0.39	-0.39	-0.39
22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
23	-0.49	0.00	0.00	-0.49	-0.49	-0.49	-0.49	-0.49	-0.49	-0.49
24	-0.17	0.00	0.00	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17
25	1.29	0.00	0.01	1.29	1.29	1.29	1.29	1.29	1.29	1.29
26	0.17	0.00	0.00	0.17	0.17	0.17	0.17	0.17	0.17	0.17
27	-1.98	0.00	-3.02	-0.64	-1.79	-1.98	-1.98	-1.98	-1.98	-1.98
28	-0.11	0.00	-0.55	-0.22	-0.19	-0.11	-0.11	-0.11	-0.11	-0.11
29	0.19	0.00	0.01	0.51	0.12	0.19	0.19	0.19	0.19	0.19



LEGEND

TUFLOW Model Boundary

Catchment Boundary

Reporting Locations
10 ID number (refer to map table)

Blockage Locations
B01 ID number

Critical Culvert Blockage Scenarios

B01, B02, B03, B04, B05, B06, B07, B08, B09, B10

Title:
Critical Culvert Blockage Scenarios

Figure:
8-2

Rev:
-

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Approx. Scale

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8.5 Design Flood Results

A range of design flood events were modelled (refer to Table 8-2), the results of which are presented and discussed below. The design results presented in the remainder of the report represent the maximum values across all scenarios for each design event simulated.

8.5.1 Interpretation of Results

The interpretation of the maps and other data presented in this report should include an appreciation of the limitations of the modelling and general accuracy. While the points below highlight these limitations, it is important to note that results presented provide an up-to-date prediction of design flood behaviour using the best modelling techniques currently available. Points to remember include:

- Recognition that no two floods behave in exactly the same manner;
- Design floods are a best estimate of an “average” flood for their probability of occurrence; and
- The topography datasets (ALS, ground survey and “Works as Executed” drawing) used to generate the model DEM all have varying uncertainties associated.

Flood depths and flood extents, which are determined using this DEM, should be interpreted accordingly.

8.5.2 Peak Design Flood Levels

Design flood results are presented for the simulated design events including the 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5%, 0.2% AEP and PMF events. Predicted peak flood levels at selected reporting locations are provided in Table 8-6 and the reporting locations shown in Figure 8-7. Longitudinal profiles along the alignment of the watercourses are provided in Appendix C for the full range of design event magnitudes considered.

Table 8-6 Predicted Flood Levels at Selected Reporting Locations

Location (refer to Figure 8-7)	Design Peak Flood Levels (m AHD)							
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
Upstream (US) Illawarra Railway, Thirroul	14.24	14.38	14.41	14.45	14.46	14.48	14.49	14.95
US McCauley Street, Thirroul	7.09	7.11	7.13	7.15	7.17	7.19	7.21	7.52
US Cliff Parade, Thirroul	4.51	4.57	4.63	4.70	4.75	4.80	4.86	5.15
US Deborah Ave., Thirroul	63.09	63.13	63.17	63.21	63.25	63.28	63.33	63.54
US Virginia Terrace, Thirroul	49.87	49.93	50.00	50.09	50.16	50.23	50.28	50.59

Location (refer to Figure 8-7)	Design Peak Flood Levels (m AHD)							
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
US George Street, Thirroul	32.20	32.23	32.29	32.34	32.38	32.43	32.48	32.70
US Palm Crescent, Thirroul	75.03	75.05	75.08	75.12	75.14	75.18	75.20	75.32
US Virginia Terrace, Thirroul	40.82	40.87	40.91	40.95	40.99	41.02	41.07	41.23
US George Street, Thirroul	29.22	29.29	29.36	29.41	29.48	29.55	29.62	29.88
US Kelton lane, Thirroul	24.02	24.24	24.46	24.94	25.35	25.61	25.91	26.61
US Lachlan Street, Thirroul	18.86	18.93	19.02	19.11	19.19	19.27	19.37	19.81
US Lawrence Hargrave, Thirroul	14.30	14.36	14.44	14.50	14.57	14.63	14.96	16.21
US Illawarra Railway, Thirroul	12.28	12.73	13.19	13.66	14.06	14.48	14.94	16.17
US Brickworks Avenue, Thirroul	10.82	11.05	11.25	11.46	11.58	11.61	11.83	13.24
US Hamilton Road, Thirroul	2.03	2.08	2.31	2.56	2.60	2.71	2.70	4.00
US Princes Highway, Thirroul	18.90	18.95	18.99	19.08	19.15	19.22	19.31	19.68
US Illawarra Railway, Thirroul	16.21	16.30	16.39	16.49	16.59	16.68	16.80	17.22
US Air Avenue, Thirroul	11.13	11.15	11.17	11.18	11.19	11.20	11.21	12.65
US Illawarra Railway, Bulli	17.07	17.10	17.17	17.24	17.29	17.34	17.40	17.69
US William Street, Bulli	27.27	27.33	27.42	27.50	27.57	27.66	27.74	28.11
US Hobart Street, Bulli	22.22	22.35	22.51	22.62	22.72	22.80	22.89	23.42
US coal haulage embankment, Bulli	21.79	22.47	22.81	22.90	22.95	23.01	23.07	23.43
US Bulli Showground, Bulli	17.89	17.98	18.10	18.20	18.32	18.38	18.42	18.55
US Princes Highway, Bulli	14.47	14.56	14.61	14.64	14.65	14.61	14.62	14.85
Park at Black Diamond Place, Bulli	12.99	13.02	13.07	13.12	13.22	13.37	13.55	14.68
US Illawarra Railway, Bulli	12.36	12.52	12.71	12.91	13.12	13.29	13.50	14.63

Location (refer to Figure 8-7)	Design Peak Flood Levels (m AHD)							
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
US Illawarra Railway (Beacon Avenue), Bulli	12.66	12.69	12.77	12.84	12.90	13.12	13.36	14.55
US timber footbridge (Beach Street), Bulli	4.08	4.25	4.43	4.60	4.75	4.87	5.01	5.58
US Blackhall Street, Bulli	2.62	2.68	2.78	2.89	2.98	3.05	3.16	4.36

8.5.3 Design Flood Hydrographs

The simulated design hydrographs for the critical storm duration are presented at the entrance to the four creeks:

- Thomas Gibson Creek at Thirroul Beach, Thirroul (refer to Figure 8-3);
- Hewitts Creek (including Woodlands Creek tributary), upstream of Hamilton Road, Thirroul (refer to Figure 8-4);
- Tramway Creek, upstream of the creek entrance at McCauley Beach, Bulli (refer to Figure 8-5); and
- Slacky Creek, upstream of Blackhall Street, Bulli (refer to Figure 8-6).

The simulated hydrographs shown in Figure 8-3 to Figure 8-6 have a relatively rapid rise. This has consequences in terms of flood warning and response which should be considered in future floodplain management investigations. Predicted peak flows at selected reporting locations are shown in Table 8-7 for the full range of design event magnitudes considered.

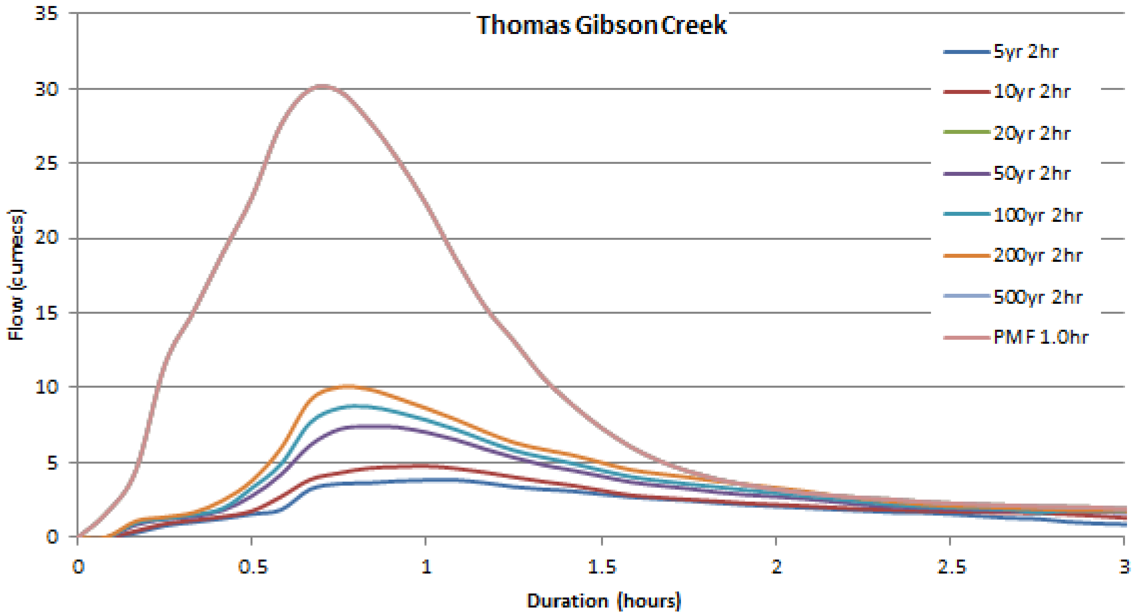

Figure 8-3 Design Flood Hydrographs for Thomas Gibson Creek at Thirroul Beach, Thirroul

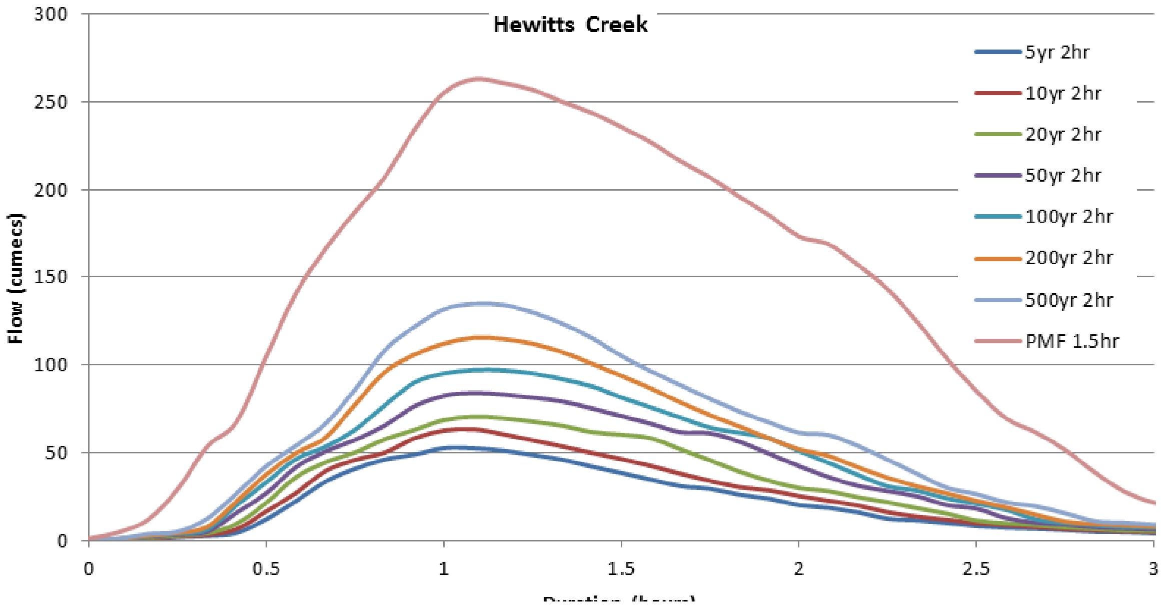

Figure 8-4 Design Flood Hydrographs for Hewitts Creek (Including Woodlands Creek Tributary) Upstream of Hamilton Road, Thirroul

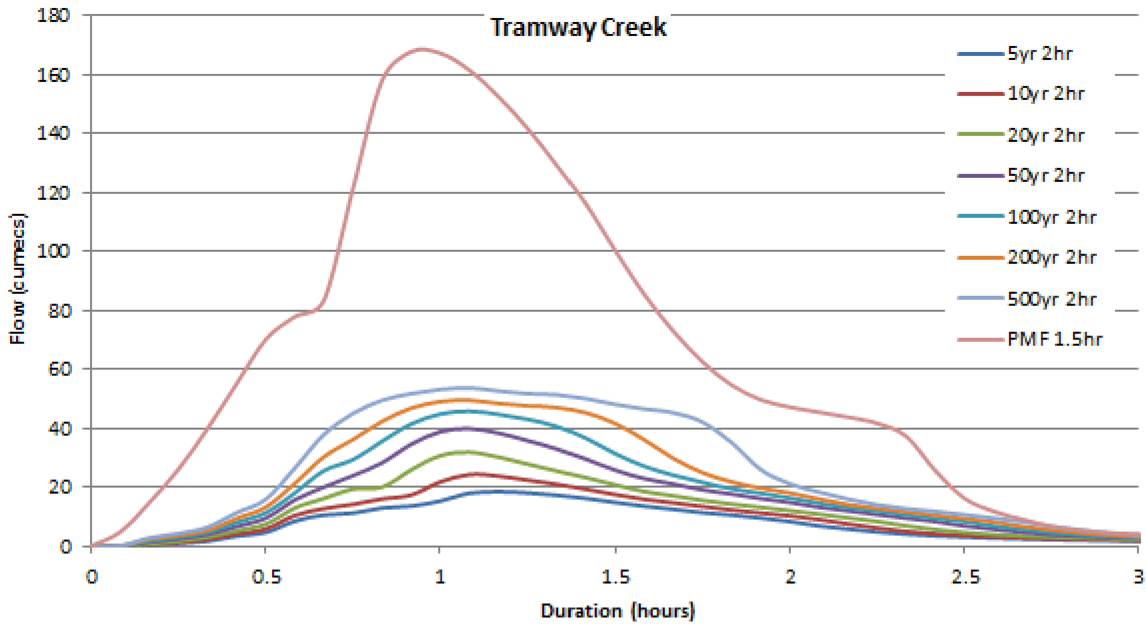

Figure 8-5 Design Flood Hydrographs for Tramway Creek Upstream of the Creek Entrance at McCauley Beach, Bulli

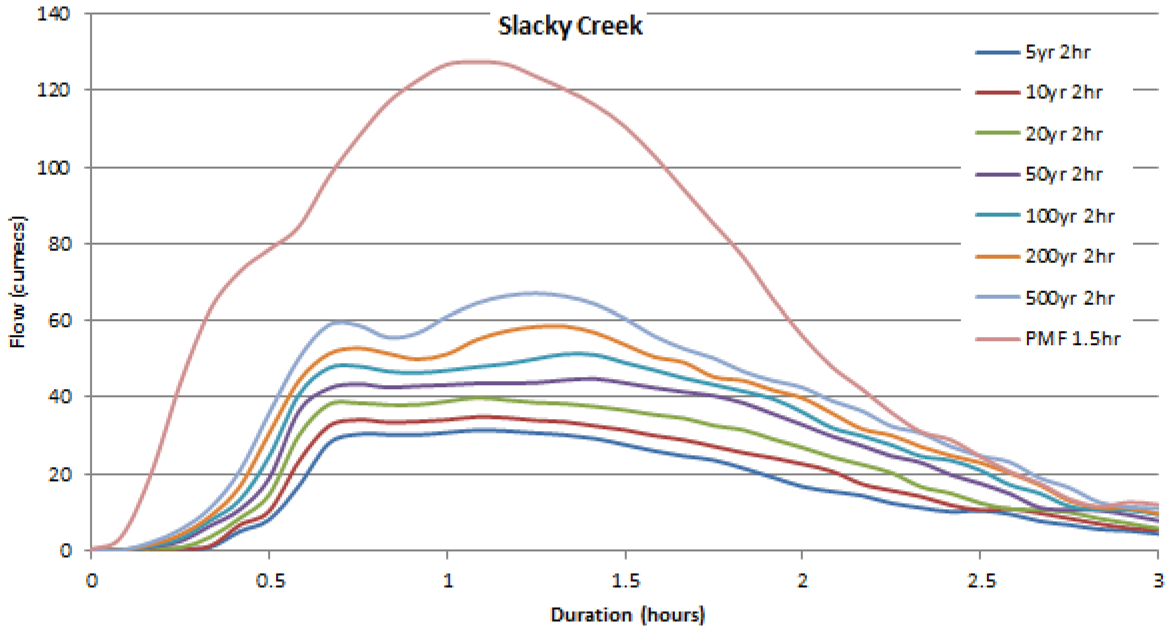

Figure 8-6 Design Flood Hydrographs for Slacky Creek Upstream of Blackhall Street, Bulli

Table 8-7 Design Peak Flows at Selected Reporting Locations

Location (refer to Figure 8-7)	Design Peak Flood Flows (m^3/s)							
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
US Illawarra Railway, Thirroul	0.8	1.0	1.1	1.2	1.4	1.5	1.7	5.0
US McCauley Street, Thirroul	2.1	2.6	3.1	3.9	4.4	4.9	5.7	20.6
US Cliff Parade, Thirroul	5.3	6.6	8.3	10.2	11.8	13.6	15.9	30.5
US Deborah Ave., Thirroul	4.7	5.7	6.9	8.2	9.4	10.7	12.6	22.6
US Virginia Terrace, Thirroul	4.9	6.0	7.3	8.6	9.7	10.6	11.7	18.9
US George Street, Thirroul	6.5	7.6	9.0	10.2	11.5	12.3	14.0	22.7
US Palm Crescent, Thirroul	0.7	0.8	1.0	1.2	1.3	1.5	1.8	3.0
US Virginia Terrace, Thirroul	2.6	3.1	3.8	4.3	4.9	5.5	6.4	10.0
US George Street, Thirroul	4.7	5.6	6.9	7.7	9.1	10.3	11.8	18.3
US Kelton lane, Thirroul	30.8	37.1	45.3	53.5	61.5	70.5	82.9	152.3
US Lachlan Street, Thirroul	35.0	41.9	51.0	59.8	68.6	78.4	92.2	167.3
US Lawrence Hargrave, Thirroul	34.3	41.7	51.5	60.7	70.2	79.4	93.3	165.1
US Illawarra Railway, Thirroul	42.4	53.2	66.9	81.1	93.9	103.8	120.3	243.1
US Brickworks Avenue, Thirroul	42.8	53.3	66.9	81.3	94.3	103.1	121.4	291.2
US Hamilton Road, Thirroul	52.8	63.3	76.4	107.7	102.0	129.2	135.4	301.0
US Princes Highway, Thirroul	23.6	30.2	40.3	48.7	57.1	62.7	72.4	144.8
US Illawarra Railway, Thirroul	22.3	25.4	28.6	31.2	32.9	34.2	35.5	47.2

Location (refer to Figure 8-7)	Design Peak Flood Flows (m^3/s)							
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
US Air Avenue, Thirroul	21.4	22.7	23.8	24.4	24.8	25.2	25.7	37.3
US Illawarra Railway, Bulli	18.4	22.7	30.6	40.0	49.1	58.7	73.4	164.9
US William Street, Bulli	19.0	22.5	28.1	33.3	38.8	44.9	52.6	104.5
US Hobart Street, Bulli	19.1	23.0	28.2	33.8	39.2	44.9	51.9	68.2
US coal haulage embankment, Bulli	16.0	16.7	19.7	22.3	24.8	27.4	29.7	37.2
US Bulli Showground, Bulli	21.4	22.3	25.3	28.2	30.5	32.9	35.1	41.8
US Princes Highway, Bulli	25.4	27.9	31.1	35.2	39.6	44.3	50.2	79.3
Park at Black Diamond Place, Bulli	16.1	17.8	20.9	22.9	24.6	26.3	28.5	67.7
US Illawarra Railway, Bulli	24.3	26.8	30.8	34.2	35.9	37.4	41.8	68.9
US Illawarra Railway (Beacon Avenue), Bulli	10.4	11.8	13.5	15.4	17.9	21.2	24.7	45.1
US timber footbridge (Beach Street), Bulli	28.1	31.7	36.5	42.6	48.1	54.0	63.4	123.9
US Blackhall Street, Bulli	30.1	33.8	39.1	45.7	51.4	57.1	65.8	125.0
US Blackhall Street (footbridge), Bulli	30.9	34.8	39.7	46.5	52.3	58.0	66.6	134.0

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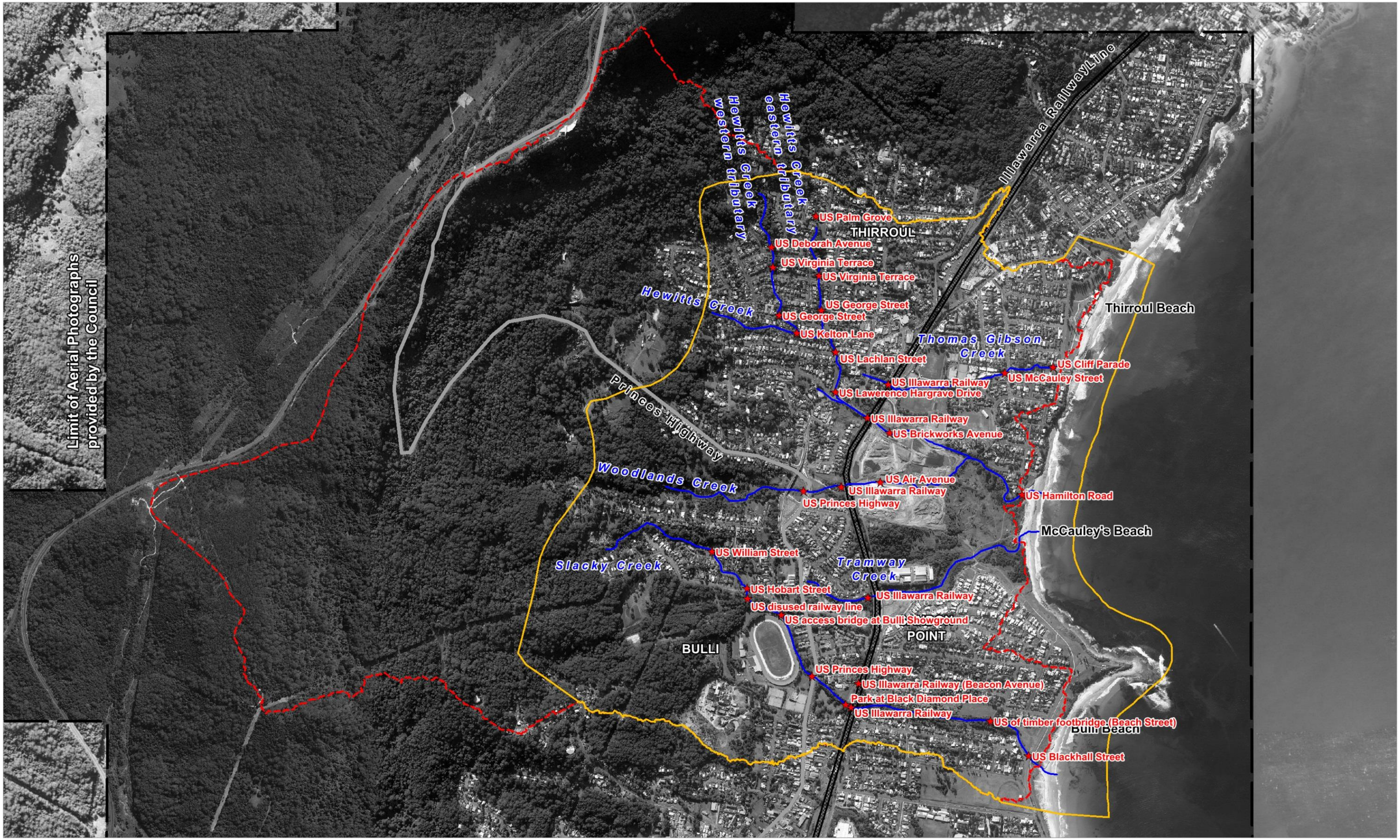

LEGEND

TUFLOW Model Boundary

Catchment Boundary

Watercourses

★ Reporting Locations

Title:

Locations of Reported Predicted Flood Levels and Flows

Figure: 8-7

Rev: 0

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Approx. Scale

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8.5.4 Mapping of Flood Behaviour

Design flood mapping is undertaken using outputs from the TUFLOW hydraulic model. The hydraulic model results grids have been filtered to remove the shallow water depths on 2D model cells which aren't part of significant overland flow paths based on a depth clipping threshold of 0.15m. In order to ensure the continuity of mapping along the overland flow paths and considering the steep nature of the topography in the modelled upper catchment areas, a hazard criterion has also been used when filtering the model results. This hazard criterion re-instates high-velocity shallow-depth flow paths which would otherwise have been removed. The final stage of the filtering process removes isolated water bodies less than 100 m^2 in area which form when ground depressions, which aren't part of the main flows paths, may be filled to depths greater than 0.15m.

Maps have been produced showing water level, water depth and velocity. The maps present the peak value across all scenarios for each design event simulated. Provisional flood hazard categories, hydraulic categories, flood emergency response classification and preliminary residential flood planning area and levels are derived from the hydrodynamic model results and are also mapped. The mapping outputs are presented in Appendix D

8.5.5 Comparison with Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a)

A comparison between the TUFLOW model results from the current Flood Study and the HEC RAS model results from the 2002 Flood Study has been undertaken for two separate scenarios:

- Pre-existing conditions, i.e. the conditions that existed at the time of the 2002 Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a); and
- Existing conditions, i.e. current catchment conditions.

When reviewing the comparison of results in this section, it is important to note that the schematisation of a 2D model (TUFLOW) and the model computations are fundamentally different when compared to a 1D model (HEC RAS). The defining assumption for 1D modelling is that only the forces, velocities, and variations in the stream direction (upstream and downstream) are significant, and that those in the transverse or lateral direction are negligible. 2D modelling computes and accounts for the transverse components. These differences between 1D and 2D models need to be considered when reviewing the results throughout this section of the report and the long section profiles in Appendix C.

8.5.5.1 Pre-existing conditions

8.5.5.1.1 Model schematisation

The TUFLOW model has been schematised and parameterised to provide for similar representation where appropriate, to the 1% AEP design event for pre-existing conditions, i.e. the conditions that existed at the time of the 2002 Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a). The 1% AEP design flood event has been simulated in the TUFLOW model for comparison with the results from the HEC RAS hydraulic model developed as part of the Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a). The HEC RAS model calibrated to the August 1998 flood event formed the basis for the design flood modelling as part of the 2002 Flood Study. Details of the key HEC RAS model parameters adopted for the design flood modelling are provided in Section 6.2.2.

The TUFLOW model, calibrated to the August 1998 flood event, was used as the basis for simulating the 1% AEP design flood event for pre-existing conditions for comparison with the HEC RAS model results. The key model parameters and simulated scenarios were as per the HEC RAS model. The main differences in the schematisation and parameters between the two models are as follows:

- The inflow hydrographs to the TUFLOW model were derived using an updated WBNM hydrological model. The primary differences between the WBNM model developed as part of the current Flood Study and the model developed as part of the 2002 Flood Study include:
    - The catchment and sub-area delineation has been updated as part of the current Flood Study (refer to Section 6.4.1.1); and
    - The proportion of impervious area (also referred to as Fraction Impervious) has been updated as part of the current Flood Study (refer to Section 6.4.1.2).
- The TUFLOW model uses unsteady state simulations to route the inflow hydrographs. The HEC RAS model has been run in steady state mode
- The Manning's 'n' values applied to the channel and floodplain differs slightly across the study area between the calibrated TUFLOW model and the HEC RAS model;
- Additional structures have been included in the TUFLOW model which were not included in the HEC RAS model. These structures were identified during a review of the HEC RAS model data; and
- The dimensions of the Illawarra Railway culvert on Hewitts Creek have been updated as part of the current flood study. New survey of this culvert was completed by Council in December 2013 which indicates that the culvert has approximately 35% greater cross sectional area when compared to the survey data used in the HEC RAS model.

8.5.5.1.2 Model results

Table 8-8 and Table 8-9 provide a comparison of the peak water levels and flows between the HEC RAS model and the TUFLOW model at a number of reporting locations in the study area for the 1% AEP design flood event for pre-existing conditions. Appendix C2 contains water surface profiles showing a comparison between the HEC RAS model peak water level results and the TUFLOW model peak water level results for the 1% AEP design flood event for pre-existing conditions.

Table 8-8 Comparison of Peak Water Levels for the 1% AEP Event for Pre-existing Conditions between the HEC RAS Model and the TUFLOW Model

Reporting location	Watercourse	1% Design Peak Levels for Pre-existing Conditions (m AHD)		Difference in Peak Water Level (m)
		TUFLOW	HEC RAS	
US Illawarra Railway, Thirroul	Thomas Gibson Creek	14.38	15.01	-0.63
US McCauley Street, Thirroul	Thomas Gibson Creek	7.15	7.18	-0.03
US Cliff Parade, Thirroul	Thomas Gibson Creek	4.74	4.77	-0.03
20m US Deborah Ave., Thirroul	Hewitts Creek (western tributary)	63.25	62.91	+0.34
US Virginia Terrace, Thirroul	Hewitts Creek (western tributary)	50.15	50.43	-0.28
US George Street, Thirroul	Hewitts Creek (western tributary)	32.34	32.48	-0.14
15m US Palm Crescent, Thirroul	Hewitts Creek (eastern tributary)	75.12	75.11	+0.01
US Virginia Terrace, Thirroul	Hewitts Creek (eastern tributary)	40.98	40.76	+0.22
60m US George Street, Thirroul	Hewitts Creek (eastern tributary)	29.41	28.46	+0.95
US Kelton Lane, Thirroul	Hewitts Creek (Main Channel)	24.80	25.43	-0.63
US Lachlan Street, Thirroul	Hewitts Creek (Main Channel)	19.29	18.65	+0.64
US Lawrence Hargrave, Thirroul	Hewitts Creek (Main Channel)	14.78	15.98	-1.20
US Illawarra Railway, Thirroul	Hewitts Creek (Main Channel)	14.06	15.99	-1.93
US of footbridge near creek entrance, Thirroul	Hewitts Creek (Main Channel)	2.42	2.67	-0.25
US Princes Highway, Thirroul	Woodlands Creek	18.97	18.52	+0.45
US heavy vehicle safety ramp, Thirroul	Woodlands Creek	18.96	18.46	+0.50
US Illawarra Railway, Thirroul	Woodlands Creek	16.57	17.24	-0.67
US Illawarra Railway, Bulli	Tramway Creek	17.17	17.70	-0.53
US William Street, Bulli	Slacky Creek	27.56	27.12	+0.44
US Hobart Street, Bulli	Slacky Creek	22.48	22.09	+0.39

Reporting location	Watercourse	1% Design Peak Levels for Pre-existing Conditions (m AHD)		Difference in Peak Water
US Princes Highway, Bulli	Slacky Creek	14.69	14.63	+0.06
Park at Black Diamond Place, Bulli	Slacky Creek	12.92	13.03	-0.11
US Illawarra Railway, Bulli	Slacky Creek	11.88	12.81	-0.93
US of timber footbridge (Beach Street), Bulli	Slacky Creek	4.45	4.91	-0.46
US Blackhall Street, Bulli	Slacky Creek	4.25	2.88	+1.37

Table 8-9 Comparison of Peak Flows for the 1% AEP Event for Pre-existing Conditions between the HEC RAS Model and the TUFLOW Model at a Sample of Reporting Locations

Reporting location	Watercourse	1% Design Peak Flows for Pre-existing Conditions (m^3/s)		Difference (m^3/s)	Relative Percentage Difference
		TUFLOW	HEC RAS		
US Cliff Parade, Thirroul	Thomas Gibson Creek	11.4	10.7	-0.7	6%
US George Street, Thirroul	Hewitts Creek (western tributary)	11.2	11.6	0.4	-4%
US George Street, Thirroul	Hewitts Creek (eastern tributary)	9.1	7.5	-1.6	19%
US Kelton lane, Thirroul	Hewitts Creek	61.7	57.5	-4.2	7%
US Lachlan Street, Thirroul	Hewitts Creek	68.8	63.9	-5.0	7%
US Lawrence Hargrave, Thirroul	Hewitts Creek	70.4	59.8	-10.6	-14%
US Illawarra Railway, Thirroul	Hewitts Creek	93.6	117.7	24.1	26%
US Brickworks Avenue, Thirroul	Hewitts Creek	150.3	117.7	-32.7	24%
US Princes Highway, Thirroul	Woodlands Creek	50.4	48.2	-2.2	4%
US Illawarra Railway, Thirroul	Woodlands Creek	32.2	49.4	17.1	-42%
US Illawarra Railway, Bulli	Tramway Creek	34.8	47.9	13.1	-32%
US William Street, Bulli	Slacky Creek	39.0	20.0	-19.0	64%
US Hobart Street, Bulli	Slacky Creek	39.1	23.2	-15.9	51%
US coal haulage embankment, Bulli	Slacky Creek	27.0	23.2	-3.8	15%
US Princes Highway, Bulli	Slacky Creek	31.1	43.6	12.5	-33%
US Illawarra Railway,	Slacky Creek	42.4	58.0	15.6	-31%

Reporting location	Watercourse	1% Design Peak Flows for Pre-existing Conditions (m^3/s)		Difference (m^3/s)	Relative Percentage Difference
Bulli					
US Blackhall Street, Bulli	Slacky Creek	44.5	59.3	14.8	-29%

There is generally a good correlation between the TUFLOW and HEC RAS model results presented in the modelled water surface profiles (Appendix C2), the peak water levels presented in Table 8-8 and the peak flows presented in Table 8-9. There are some locations with noticeable differences between results and further discussion on these locations is provided in the following sections and in Appendix C2.

Thomas Gibson Creek

Table 8-8 indicates that the peak 1% AEP design flood levels for pre-existing conditions from the TUFLOW model are generally within ± 0.3m of the peak flood levels from the HEC RAS model at the reporting locations along Thomas Gibson Creek except upstream of the Illawarra Railway, Thirroul (-0.63m difference).

The difference in peak water levels upstream of Illawarra Railway, Thirroul, is a function of differences in the peak flows between the TUFLOW model and the HEC RAS model. The Thomas Gibson Creek catchment extends upstream of the Illawarra Railway, Thirroul, to Lachlan Street, Thirroul. Upstream of the culvert inlet on Lawrence Hargrave Drive, Thirroul, the peak 1% AEP flow in the TUFLOW model (0.3m^3/s) is considerably less than the peak flow in the HEC RAS model (11.1 m^3/s). The catchment area contributing to this culvert inlet is comparable between the TUFLOW model (0.0051km^2) and the HEC RAS model (0.0069 km^2). The variance in flows is a function of differences in the schematisation of the HEC RAS model and the TUFLOW model. The inflow hydrograph to the HEC RAS model upstream of Lawrence Hargrave Drive, Thirroul, includes diverted flows from the Hewitts Creek catchment. For the 2002 Flood Study, the diversion in flows is represented in the WBNM hydrological model through the use of informal detention storage and secondary flow paths linking the Hewitts Creek catchment at Lachlan Street, Thirroul to the Thomas Gibson Creek catchment upstream of Lawrence Hargrave Drive, Thirroul. As part of the current study, this diversion in flows is not represented in the WBNM model as the inflow hydrographs have been routed through a two dimensional grid of the study area (refer to Section 6.4.4). Therefore, the interaction in flows between the Hewitts Creek catchment and the Thomas Gibson Creek catchment has been dynamically modelled within the 2D domain of the TUFLOW model. The TUFLOW model results indicate that there is a limited transfer of flow (peak flow is less than 0.01 m^3/s) from the Hewitts Creek catchment to the Thomas Gibson Creek catchment along Lachlan Street, Thirroul for the 1% AEP event for pre-existing conditions.

The comparison of long section profiles indicates that the most significant differences in the water surface profiles occurs upstream of the Illawarra Railway, Thirroul (as discussed above), within Thomas Gibson Park, Thirroul and downstream of McCauley Street, Thirroul. Further discussion on these differences in the water surface profiles is provided in Appendix C2.

The results in Table 8-9 indicate that the peak flood flows from the TUFLOW model are within 10% of the peak flood flows from the HEC RAS model at the one reporting location along this creek.

Hewitts Creek western tributary

Table 8-8 indicates that the peak 1% AEP design flood levels for pre-existing conditions from the TUFLOW model are generally within ± 0.3m of the peak flood levels from the HEC RAS model at the majority of reporting locations along the western tributary of Hewitts Creek. There is a slightly greater difference in the water levels upstream of Deborah Avenue, Thirroul (+0.34m) which is considered to be a function of differences in the schematisation between the 1D and 2D models, with the TUFLOW model better capturing the overland flow mechanisms at this structure once it is overtopped.

The comparison of long section profiles indicates that there is generally a good correlation between the TUFLOW model results and the HEC RAS model results. The most significant difference in the water surface profiles occurs at the culverts on Palm Crescent, Thirroul and Virginia Terrace, Thirroul. Further discussion on the differences in water surface profiles at these locations is provided in Appendix C2.

The results in Table 8-9 indicate that the peak flood flows from the TUFLOW model are within 10% of the peak flood flows from the HEC RAS model at the one reporting location along this creek.

Hewitts Creek eastern tributary

Table 8-8 indicates that the peak 1% AEP design flood levels for pre-existing conditions from the TUFLOW model correlate well with the HEC RAS model results at two of the three reporting locations along this modelled creek reach. The peak flood levels from the TUFLOW model are within ± 0.3m of the peak flood levels from the HEC RAS model upstream of Palm Crescent, Thirroul, and Virginia Terrace, Thirroul. Further investigation has been carried out at the culvert on George Street, Thirroul, where the water levels upstream of this culvert are +0.95m higher in the TUFLOW model when compared to the HEC RAS model.

The HEC RAS model has been schematised with a culvert length of 15.6m at George Street, Thirroul (i.e. the distance between the culvert inlet and outlet). Based on a review of aerial photos and following a site inspection, the length of this culvert is approximately 75m which is significantly longer than the HEC RAS modelled culvert length. The culvert within the TUFLOW model has been modelled with a length of 75m and the losses associated with this additional culvert length result in higher water levels upstream of the culvert inlet in the TUFLOW model when compared to the HEC RAS model. In addition, the peak modelled flows in the TUFLOW model are 19% higher than the peak flows from the HEC RAS model upstream of George Street, Thirroul (refer to Table 8-9). This difference in peak flows (1.6m^3/s) also contributes to the differences in peak flood levels at George Street, Thirroul. The TUFLOW model results indicate that a small proportion of flows from the western tributary of Hewitts Creek diverts to the eastern tributary of Hewitts Creek along Deborah Avenue, Thirroul. This transfer in flows in not replicated in the HEC RAS model.

The comparison of long section profiles indicates that there is generally a good correlation between the TUFLOW model results and the HEC RAS model results. The most significant differences in the water surface profiles occurs upstream of Deborah Avenue, Thirroul and George Street, Thirroul (as discussed above). Further discussion on the differences in water surface profiles at this location is provided in Appendix C2.

Hewitts Creek main channel

Table 8-8 indicates that the peak 1% AEP design flood levels for pre-existing conditions from the TUFLOW model correlate well with the HEC RAS model results at a limited number of reporting locations along this modelled creek reach.

Upstream of the Illawarra Railway, Thirroul, the HEC RAS model results are 1.93m higher than the TUFLOW model results. The variance in levels is primarily as a function of differences in the dimensions of the culvert on Hewitts Creek at the Illawarra Railway, Thirroul, between the TUFLOW and HEC RAS models. The increased cross sectional area of the culvert used in the TUFLOW model results in greater flows through this culvert reducing the water levels upstream of the Illawarra Railway, Thirroul. In addition to the main differences in model schematisation and parameters set out in Section 8.5.5.1, the following location specific differences between models were identified upstream of the Illawarra Railway, Thirroul:

- The channel slope in the TUFLOW model immediately upstream of the Illawarra Railway has a reduced gradient when compared to the HEC RAS model. It was found that the survey chainages did not tie in with the HEC RAS model chainages which has resulted in an inappropriately steep rise in the channel bed levels immediately upstream of the Illawarra Railway culvert;

The following items were noted in relation to the schematisation of the HEC RAS model:

- The ineffective flow areas at the Illawarra Railway could be improved to better define the transition between main channel and floodplain flows; and
- Default expansion and contraction losses have been applied to cross sections immediately upstream and downstream of the Illawarra Railway culvert. Adjustments to these values are recommended to better capture the energy loss resulting from flow contraction and expansion at the culvert inlet and outlet.

At Lawrence Hargrave Drive, Thirroul, the difference in water levels (-1.20m) is as a result of the changes to the dimensions of the culvert at the Illawarra Railway, Thirroul, as discussed above. The backwater effect resulting from conditions at the Illawarra Railway culvert extends upstream to Lawrence Hargrave Drive, Thirroul and the difference in water levels is reflected in the results at Lawrence Hargrave Drive, Thirroul.

Upstream of Lachlan Street, Thirroul, the TUFLOW model results are +0.64m higher than the HEC RAS model results. The modelled peak flows upstream of this culvert are marginally higher in the TUFLOW model (68.8m^3/s) when compared to the HEC RAS model (63.9m^3/s). The model calibration and validation results indicate that the TUFLOW model was found to correlate well with the majority of the observed flood marks in the vicinity of this structure with the modelled peak flood levels generally within a ± 0.3m tolerance of the observed flood levels (refer to Section 7.2.5). The overall fit between observed and modelled levels in this area suggests that the TUFLOW model schematisation and parameters are of the correct order.

Upstream of Kelton Lane, Thirroul, the TUFLOW model results are -0.63m lower than the HEC RAS model results. The modelled peak flows upstream of this bridge are marginally higher in the TUFLOW model (61.7m^3/s) when compared to the HEC RAS model (57.5m^3/s). The TUFLOW model was found to correlate well with the observed flood marks in this area for the August 1998 flood event with the modelled peak flood levels within a ± 0.3m tolerance of the observed flood levels upstream and downstream of this bridge (refer to Section 7.2.5). The good fit between observed and modelled levels at this bridge suggests that the TUFLOW model schematisation and parameters are of the correct order.

The comparison of long section profiles indicates that there is generally a good correlation between the TUFLOW model results and the HEC RAS model results along the upper reaches of Hewitts Creek. From Lachlan Street, Thirroul to downstream of the Illawarra Railway, Thirroul, there are more significant differences in the water surface profiles as discussed above. Further discussion on these differences in water surface profiles is provided in Appendix C2.

The results in Table 8-9 indicate that the peak flood flows from the TUFLOW model are within 10% of the peak flood flows from the HEC RAS model at the majority of the reporting locations along Hewitts Creek. Upstream of the Illawarra Railway, Thirroul, the peak flows in the TUFLOW model are 26% lower than the peak flows in the HEC RAS model. This is primarily as a result of increased conveyance through the Illawarra Railway culvert as a function of the larger culvert dimensions in the TUFLOW model. Upstream of Brickworks Avenue, Thirroul, the peak flows in the TUFLOW model are 24% higher than the peak flows in the HEC RAS model. This is primarily as a result of increased flows in the channel downstream of the Illawarra Railway, Thirroul, through a combination of increased conveyance through the Illawarra Railway culvert and overland flows which transfer from Woodlands Creek to Hewitts Creek as shown in Figure 8-8. This overland flow transfer between Woodlands Creek and Hewitts Creek, downstream of the Illawarra Railway, is not replicated in the HEC RAS model.

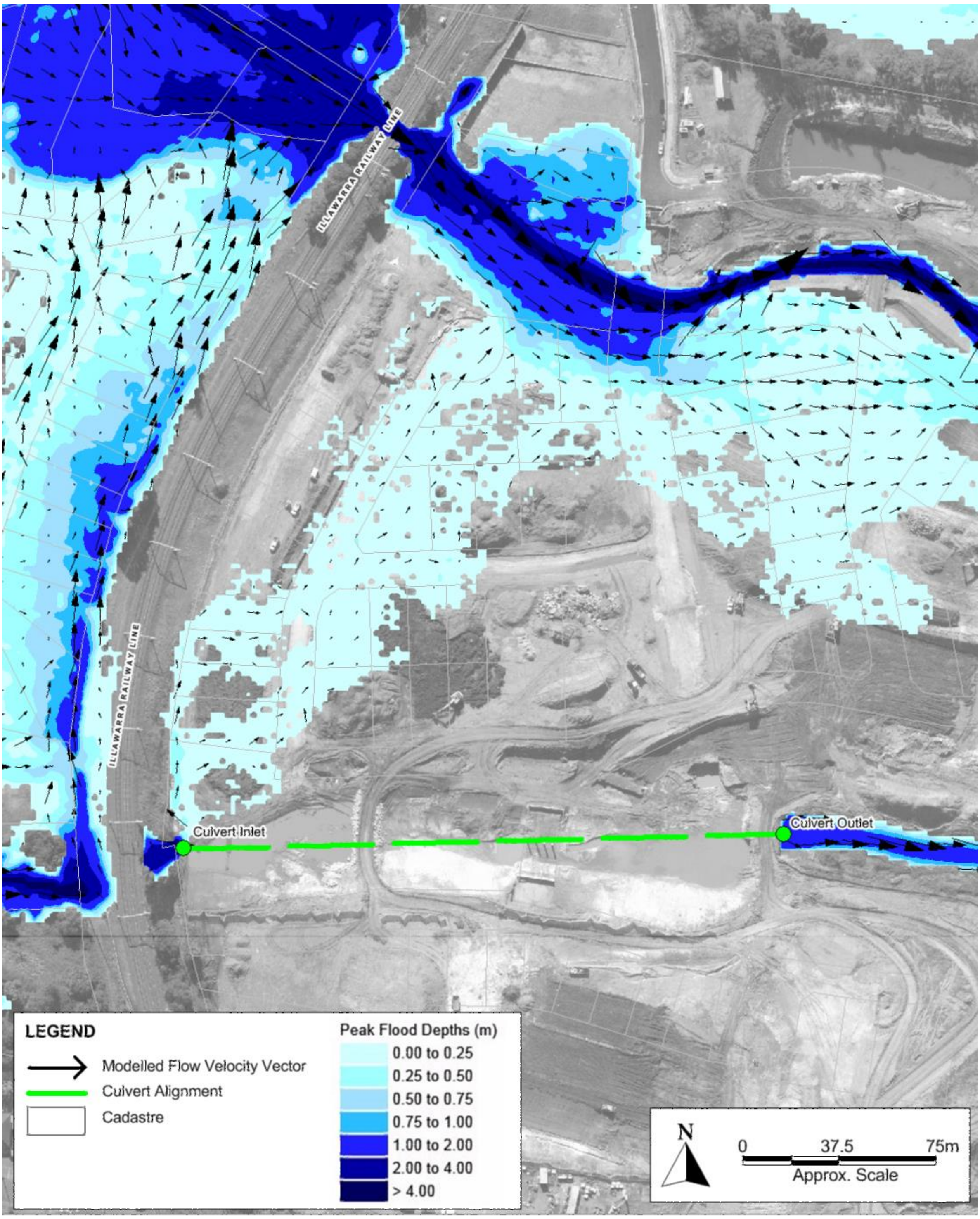


Figure 8-8 TUFLOW Modelled Flood Depths and Mechanism for the 1% AEP Event for Pre-Existing Conditions on Woodlands Creek and Hewitts Creek Downstream of the Illawarra Railway, Thirroul

Woodlands Creek

Table 8-8 indicates that the peak 1% AEP design flood levels for pre-existing conditions from the TUFLOW model are greater than ± 0.3m of the peak flood levels from the HEC RAS model at all of the reporting locations along this reach. This is primarily a function of differences in the schematisation between the TUFLOW and HEC RAS models.

Upstream of the Princes Highway, Thirroul and the disused heavy vehicle safety ramp, Thirroul, the TUFLOW model results are +0.45m and +0.5m higher than the HEC RAS model results. The peak modelled flow within the TUFLOW model for the 1% AEP event ($50.4m^3/s$) is comparable with the HEC RAS peak modelled 1% AEP flow ($48.2m^3/s$) at the Princes Highway, Thirroul. The model calibration results for the August 1998 flood event indicates that the TUFLOW model results correlated well with the highest of the observed flood levels upstream of the Princes Highway, Thirroul (refer to Section 7.2.5) for this event. This indicates that the TUFLOW model schematisation and parameters are of the correct order. For larger flood events, such as the 1% AEP event, the water levels at this location are mainly influenced by the degree of overtopping of the disused heavy vehicle safety ramp, Thirroul. A comparison of the profiles used to represent the disused heavy vehicle safety ramp shows that the HEC RAS model has a more simplified representation of this structure when compared to the representation of this structure within the ground surface of the TUFLOW model. The ground surface of the TUFLOW model at this location has been generated from LiDAR data and better represents the geometry of this structure and the flood mechanisms at this location once this structure is overtopped.

Upstream of the Illawarra Railway, Thirroul, the TUFLOW model results are -0.67m lower than the HEC RAS model results. The modelled peak flows upstream of this railway culvert are lower in the TUFLOW model ($32.2m^3/s$) when compared to the HEC RAS model ($49.4m^3/s$). Figure 8-9 shows the TUFLOW modelled flood depths and mechanisms in the vicinity of the disused heavy vehicle safety ramp, Thirroul. The TUFLOW model results indicate that the majority of the flow overtopping the disused heavy vehicle safety ramp, Thirroul, crosses to the main channel of Hewitts Creek upstream of the Illawarra Railway, Thirroul, through overland flows running along the base of the Illawarra Railway embankment and through properties along Hewitts Avenue, Thirroul. A small portion of the flow overtopping the safety ramp re-enters the Woodlands Creek channel upstream of the Illawarra Railway, Thirroul. The schematisation of the HEC RAS model results in all flows overtopping the safety ramp re-entering the Woodlands Creek channel upstream of the Illawarra Railway, Thirroul. This primarily results in the higher water levels and flows upstream of the railway culvert when compared to the TUFLOW model results.

The comparison of long section profiles indicates that there is generally a good correlation between the TUFLOW model results and the HEC RAS model results. The main differences in the water surface profile occur at the culvert on the Illawarra Railway, Thirroul (as discussed above) and along the channel downstream of the Illawarra Railway, Thirroul. Further discussion on these differences in water surface profiles is provided in Appendix C2.

The results in Table 8-9 indicate that the peak flood flows from the TUFLOW model are within 10% of the peak flood flows from the HEC RAS model upstream of Princes Highway, Thirroul. The differences in flows at the Illawarra Railway, Thirroul, are a function of differences in schematisation between the TUFLOW and HEC RAS models as discussed above.

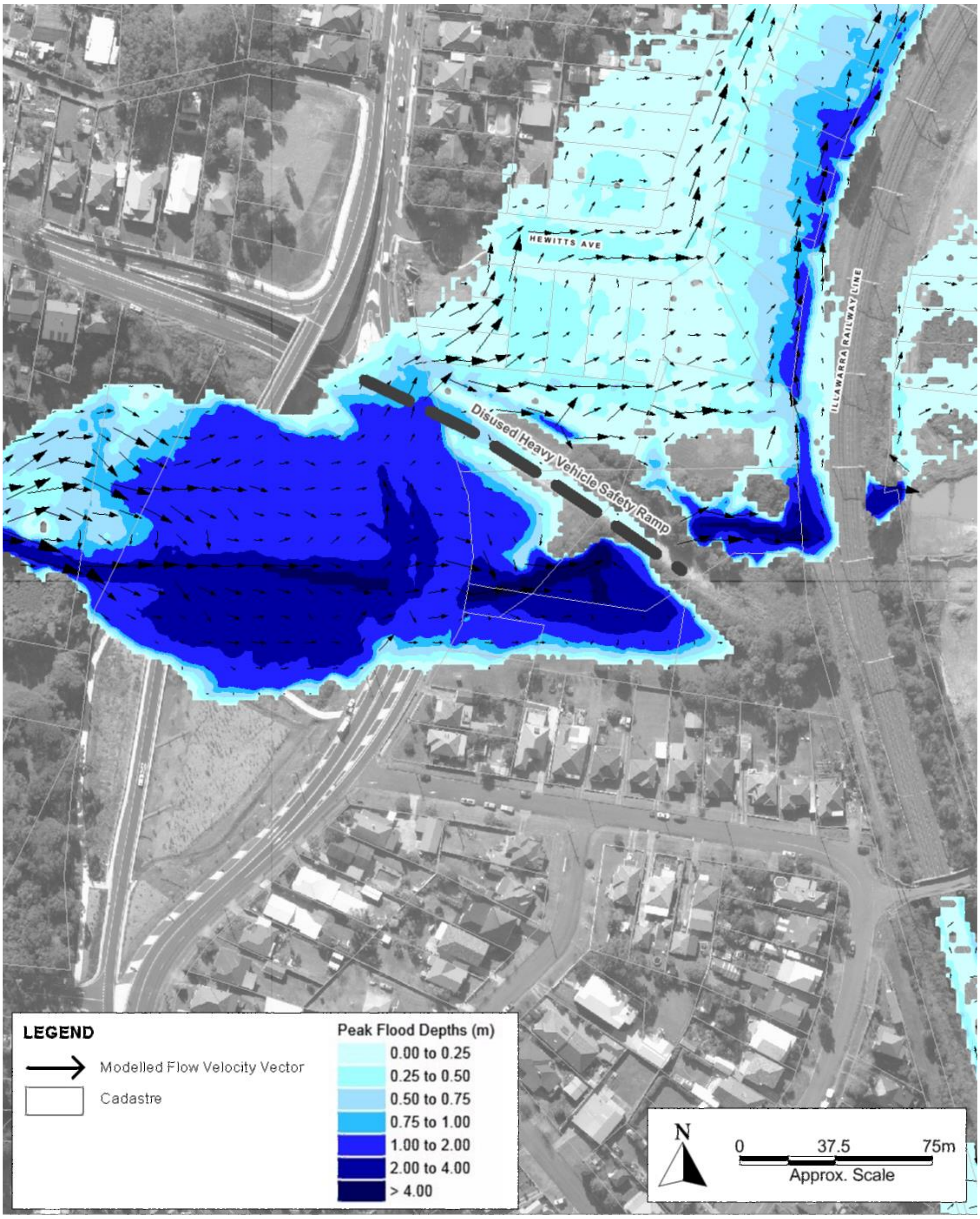


Figure 8-9 TUFLOW Modelled Flood Depths and Mechanisms for the 1% AEP Event for Pre-Existing Conditions on Woodlands Creek in the Vicinity of the Disused Heavy Vehicle Safety Ramp, Thirroul

<u>Tramway Creek</u>

Table 8-8 indicates that the peak 1% AEP design flood levels for pre-existing conditions from the TUFLOW model are greater than ± 0.3m of the peak flood levels from the HEC RAS model at the one reporting location along this reach, upstream of the culvert on the Illawarra Railway, Bulli (-0.67m difference).

A comparison of the peak flows in Tramway Creek upstream of the Illawarra Railway, Bulli, indicates that the flows are lower in the TUFLOW model (34.8m^3/s) when compared to the HEC RAS model (47.9m^3/s). These differences in flows are a result of differences in schematisation between the two models. For the 2002 Flood Study, the transfer of flows between the Slacky Creek catchment and the Tramway Creek catchment is represented in the WBNM hydrological model through the use of informal detention storage and secondary flow paths linking the Slacky Creek catchment at Hobart Street, Bulli to the Tramway Creek catchment. As part of the current study, the diversion in flows is not represented in the WBNM model and the inflow hydrographs have been routed through a two dimensional grid of the entire study area (refer to Section 6.4.4). Therefore, the interaction in flows between Slacky Creek and Tramway Creek has been dynamically modelled within the 2D domain of the TUFLOW model. The TUFLOW model results indicate that the flow peak flow along Hobart Street, Bulli, directly east of the Hobart Street culverts is approximately 45m^3/s for the 1% AEP event for pre-existing conditions. Figure 8-10 shows the TUFLOW modelled flood depths and mechanisms in the vicinity of the coal haulage embankment of the now disused railway line, Bulli. The TUFLOW model results indicate that some of the flows spilling along Hobart Street, Bulli, passes through the underpass on Princes Highway, Bulli and does not transfer to the Tramway Creek catchment. Some of this flow re-enters the Slacky Creek channel south of the coal haulage embankment. The peak flow through this underpass in the TUFLOW model for the 1% AEP event for pre-existing conditions is approximately 10.5m^3/s. The flow through this structure reduces the volume of flow transferred to the Tramway Creek channel and is not represented in the HEC RAS model.

The TUFLOW model was found to correlate well with the observed flood marks along Tramway Creek upstream of the Illawarra Railway, Bulli for the August 1998 calibration event (refer to Section 7.2.5). The modelled peak flood levels were all within a ± 0.3m tolerance of the observed flood levels. The overall fit in this area suggests that the TUFLOW hydraulic model parameters and model schematisation are of the correct order.

The comparison of long section profiles indicates that there is generally a good correlation between the TUFLOW model results and the HEC RAS model results. The main differences in the water surface profile occurs upstream and downstream of the culvert on the Illawarra Railway, Bulli (as discussed above) and at Princes Highway, Bulli. Further discussion on these differences in water surface profiles is provided in Appendix C2.

The results in Table 8-9 indicate that the peak flood flows from the TUFLOW model are 32% lower in the TUFLOW model when compared to the HEC RAS model upstream of the Illawarra Railway, Bulli. These differences in flows are a function of differences between the schematisation of the TUFLOW model and the HEC RAS model discussed above.

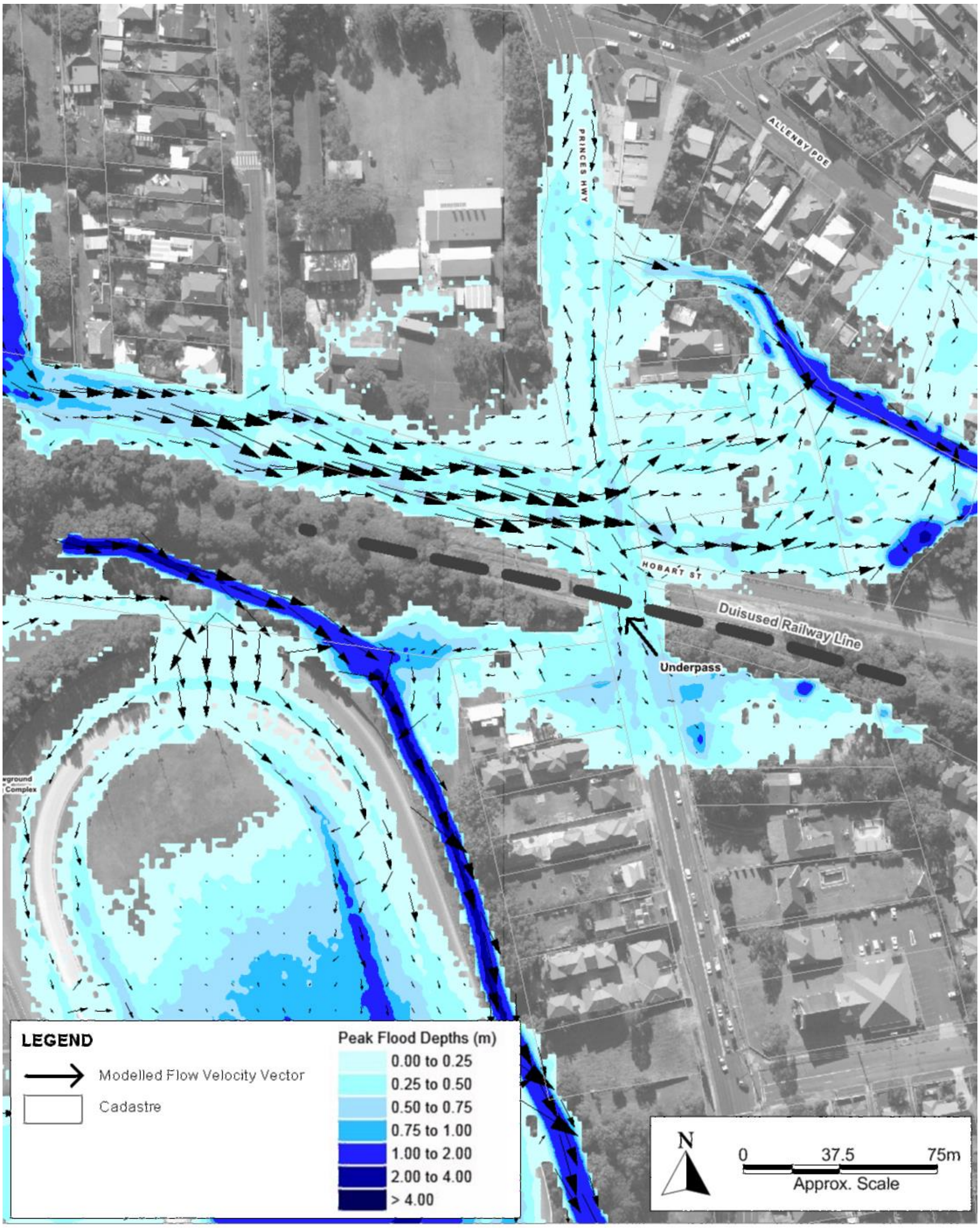


Figure 8-10 TUFLOW Modelled Flood Depths and Mechanisms for the 1% AEP Event for Pre-Existing Conditions on Slacky Creek and Tramway Creek in the vicinity of the coal haulage embankment of the now disused railway line, Bulli

<u>Slacky Creek</u>

Table 8-8 indicates that the peak 1% AEP design flood levels for pre-existing conditions from the TUFLOW model are generally within ± 0.3m of the peak flood levels from the HEC RAS model at the majority of reporting locations along Slacky Creek. There are greater differences in the water levels upstream of William Street, Bulli (+0.44m), Hobart Street, Bulli (+0.39m) and Beach Street, Bulli (-0.46m) which are considered to be a function of differences in the schematisation between the 1D and 2D models resulting in differences between modelled flows as discussed later in this section. Further discussion on the large differences in levels at the Illawarra Railway, Bulli, and Blackhall Street, Bulli, is provided below.

Upstream of the Illawarra Railway, Bulli, the peak TUFLOW modelled water level is -0.93m lower than the HEC RAS modelled water level. The water levels at this culvert are maximised for the “critical blockage pattern” which maximises flow at the Illawarra Railway culvert. The TUFLOW model results indicate that the flows upstream of the Illawarra Railway, Bulli, are controlled by the culverts and the embankment which form the detention basin adjacent to Slacky Creek at Black Diamond Place, Bulli. The model results indicate that the detention basin culverts in the park at Black Diamond Place, Bulli directly upstream of the Illawarra Railway culvert are not overtopped for the 1% AEP event “critical blockage pattern”. The embankment of the detention basin adjacent to the Illawarra Railway is overtopped. Flows overtopping this embankment spill northwards towards the Beacon Avenue underpass, Bulli and southwards into the Slacky Creek upstream of the Illawarra Railway, Bulli (refer to Figure 8-11). The TUFLOW model better replicates these overland flood mechanisms once the embankment is overtopped. In addition, the peak flows along Slacky Creek in the vicinity of the Illawarra Railway are less than the peak modelled HEC RAS flows (refer to Table 8-9) which also contributes to the difference in flood levels at this location. The TUFLOW model was found to correlate well with the observed flood marks along Slacky Creek at Black Diamond Place, Bulli, for the August 1998 calibration event (refer to Section 7.2.5). The modelled peak flood levels were all within a ± 0.3m tolerance of the observed flood levels. The calibration fit in this area suggests that the TUFLOW hydraulic model parameters and model schematisation are of the correct order.

Upstream of Blackhall Street, Bulli, the peak TUFLOW modelled water level is +1.37m higher than the HEC RAS modelled water surface profile. The water levels upstream of the culvert on Blackhall Street, Bulli are maximised when a 100% blockage has been applied to the culvert. The road elevation above this culvert is approximately 3.7m AHD. The TUFLOW model results for the 1% AEP event for pre-existing conditions indicate that flows overtop this culvert and spill across the roadway into Bulli Beach, Bulli. The peak water surface elevations in the TUFLOW model results upstream of this culvert are consistent with flows overtopping this structure and roadway. In addition, the TUFLOW model was found to correlate well with the observed flood marks along Slacky Creek at Blackhall Street, Bulli, for the August 1998 calibration event (refer to Section 7.2.5). The modelled peak flood levels were all within a ± 0.3m tolerance of the observed flood levels. The overall fit in this area suggests that the TUFLOW hydraulic model parameters and model schematisation are of the correct order.

The comparison of long section profiles indicates that there is generally a good correlation between the TUFLOW model results and the HEC RAS model results. The main differences in the water surface profiles occur along the upstream reaches of Slacky Creek and at Blackhall Street, Bulli (as discussed above). Further discussion on the differences between water surface profiles is provided in Appendix C2.

The results in Table 8-9 indicate that there are large differences in the peak flood flows between the TUFLOW model and the HEC RAS model at a number of reporting locations along Slacky Creek. At the upstream extent of the modelled reach at William Street, Bulli and Hobart Street, Bulli, the TUFLOW modelled peak flows are 64% and 51% higher than the peak flows in the HEC RAS model at these locations. A review of the WBNM hydrological sub – areas and hydrological outputs from both the current Flood Study and the 2002 Flood Study indicates that the flows in the TUFLOW model are of the correct order at William Street, Bulli and Hobart Street, Bulli.

Downstream of the coal haulage embankment of the now disused railway line, Bulli, the flows in TUFLOW model are consistently lower than the flows in the HEC RAS model. The flows along this reach are maximised when the culverts at Hobart Street, Bulli and the coal haulage embankment, Bulli, are unblocked. The results indicate that less flow passes downstream of this embankment in the TUFLOW model when compared to the HEC RAS model resulting in reduced flows along this reach in the TUFLOW model when compared to the HEC RAS model.

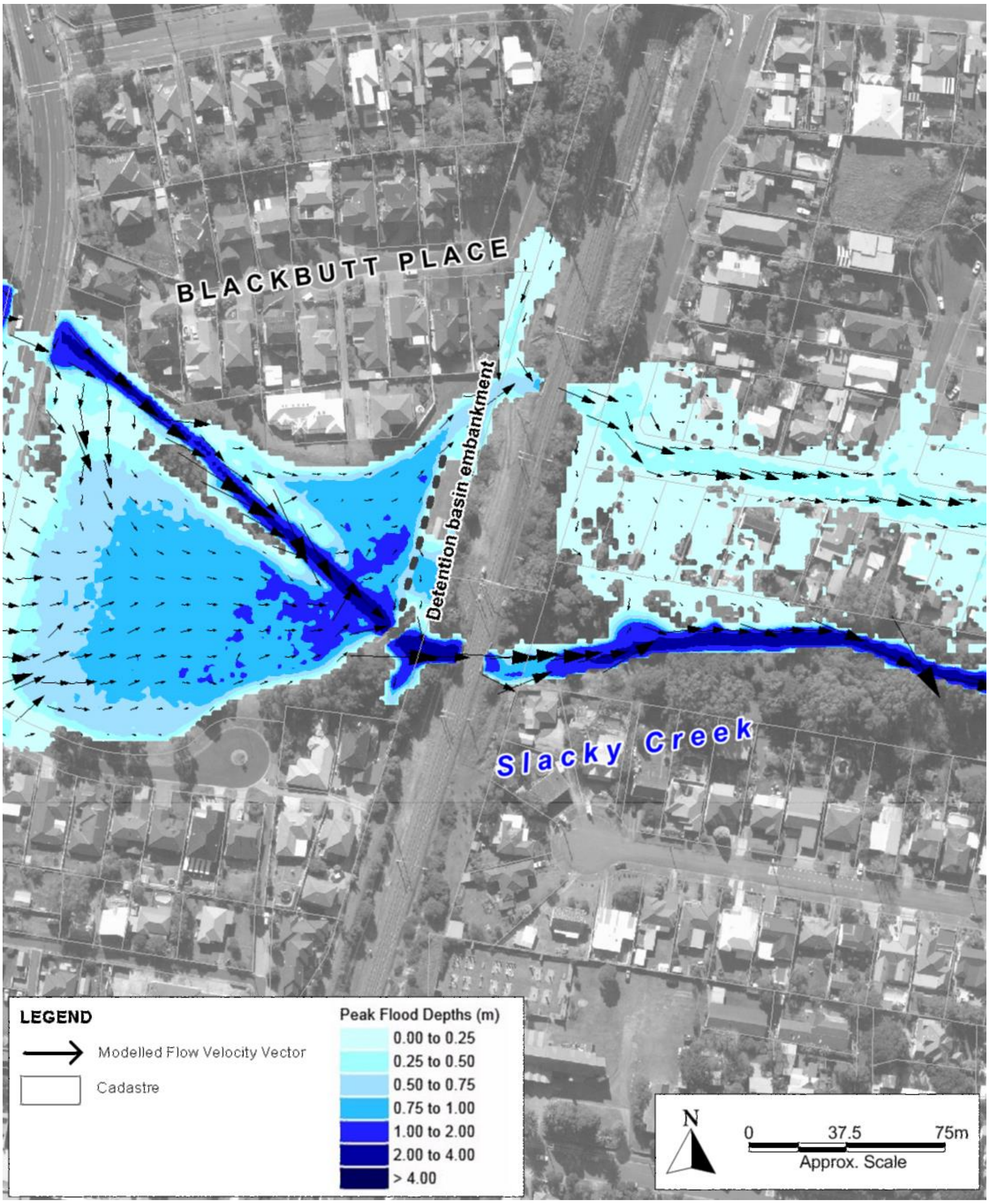


Figure 8-11 TUFLOW Modelled Flood Depths and Mechanisms for the 1% AEP Event for Pre-Existing Conditions on Slacky Creek in the vicinity of the Illawarra Railway, Bulli

8.5.5.2 Existing conditions

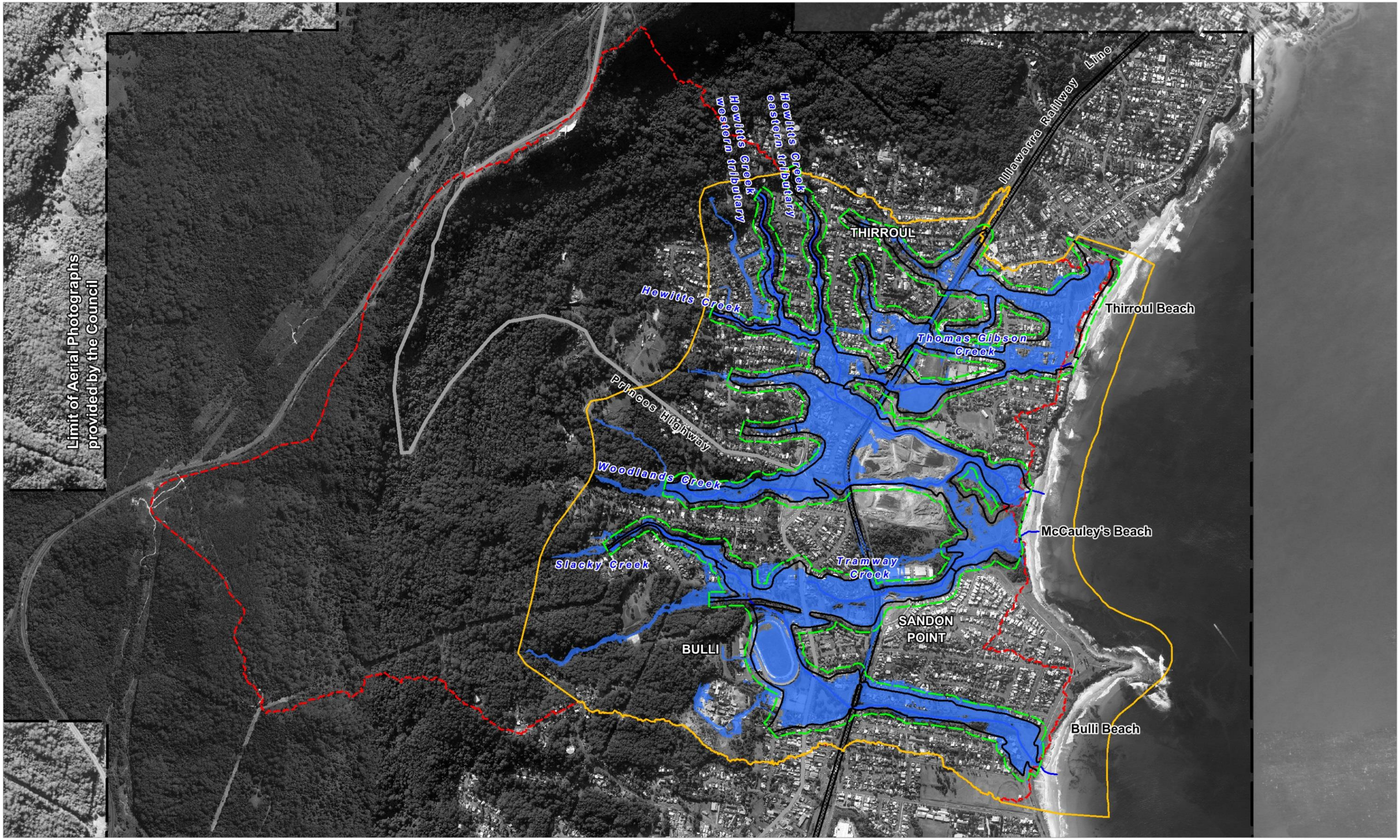
The TUFLOW design flood results from the current Flood Study have been compared with the design flood results from the HEC RAS hydraulic model developed as part of the Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a). The purpose of the comparison has been to determine the differences in areas identified as flood prone land and properties affected by flooding between the current Flood Study and the 2002 Flood Study.

Figure 8-12 shows a comparison between the PMF extents from the current Food Study and the 2002 Flood Study. The map indicates that there are variances in the flood extents throughout the study area. These variances in extents are a result of the following model differences:

- Model computations between 2D and 1D hydraulic models;
- The schematisation and extent of the hydraulic models. The TUFLOW model has been schematised to route the flow hydrographs through the two dimensional grid of the study area while the HEC RAS model has been schematised to primarily route flows along the creek channel alignment. As a result, the TUFLOW model results includes additional overland flow routes and provides a more extensive coverage of the study area;
- Available data. The current flood study includes additional detailed ground survey data captured through a LiDAR survey (refer to Section 1.4.2); and
- Catchment features which influence flooding. A number of flood mitigation measures have been implemented since the completion of the 2002 flood study which influence flooding in the catchment (refer to Section 1.4.4).

The number of properties affected by flooding is determined from the extent of flood prone land and the property lot boundaries. A direct comparison of the number of properties affected by flooding between the current flood study and the 2002 flood study has not been undertaken for the following reasons:

- Differences in the flood extents between the current flood study and the 2002 flood study as detailed above; and
- Differences in the number of property lots as a result of developments and property lot subdivisions since the completion of the 2002 flood study.



LEGEND

- Catchment Boundary
- TUFLOW Model Boundary
- HEC RAS Model Boundary
- TUFLOW Model PMF Extent (Current Flood Study)
- HEC RAS Model PMF Extent (2002 Flood Study)

Title:
PMF Extent from Current Flood Study and 2002 Flood Study

Figure: 8-12

Rev: 0

N

0 250 500m
Approx. Scale

BMT WBM
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Filepath : S:\WATER\PROJECTS\S1290 - Water - Hewitts Ck Flood Study\Drafting\WOR\Figures\Figure8-12_V11.5.wor

8.5.6 Hydraulic Categorisation

There are no prescriptive methods for determining what parts of the floodplain constitute floodways, flood storages and flood fringes. Descriptions of these terms within the Floodplain Development Manual (NSW Government, 2005) are essentially qualitative in nature. Of particular difficulty is the fact that a definition of flood behaviour and associated impacts is likely to vary from one floodplain to another depending on the circumstances and nature of flooding within the catchment.

The hydraulic categories as defined in the Floodplain Development Manual are:

- **Floodway -** Areas that convey a significant portion of the flow. These are areas that, even if partially blocked, would cause a significant increase in flood levels or a significant redistribution of flood flows, which may adversely affect other areas.

- **Flood Storage** - Areas that are important in the temporary storage of the floodwater during the passage of the flood. If the area is substantially removed by levees or fill it will result in elevated water levels and/or elevated discharges. Flood Storage areas, if completely blocked would cause peak flood levels to increase by 0.1m and/or would cause the peak discharge to increase by more than 10%.

- **Flood Fringe** - Remaining area of flood prone land, after Floodway and Flood Storage areas have been defined. Blockage or filling of this area will not have any significant effect on the flood pattern or flood levels.

The provisional hydraulic categorisation in other Wollongong LGA flood studies is generally based on the findings of Howells et al, 2003. The approach to defining provisional hydraulic categories as part of this study has therefore been defined by the criteria proposed by Howells et al, 2003:

Floodway is defined as areas where:

- Velocity x depth greater than 0.25 m^2/s and velocity greater than 0.25 m/s; or

- Velocity greater than 1 m/s.

Flood storage areas were identified as those areas which do not operate as floodways but where the depth of inundation exceeded 1 m.

Flood fringe is the remaining area of land affected by flooding, after floodway and flood storage areas have been defined.

The results of applying the above criteria were reviewed and minor adjustments made to ensure continuity of the floodway was maintained and remove small pockets of floodway and flood storage areas.

Preliminary hydraulic category mapping for the 1% AEP and PMF design events is included in Appendix D (Figure D-20 and Figure D-21). Some manual editing of the floodway has been required to ensure continuity of the floodway was maintained at appropriate locations.

8.5.7 Provisional Hazard Categories

The NSW Government's Floodplain Development Manual (NSW Government, 2005) defines flood hazard categories as follows:

- **High hazard** – possible danger to personal safety; evacuation by trucks is difficult; able-bodied adults would have difficulty in wading to safety; potential for significant structural damage to buildings; and
- **Low hazard** – should it be necessary, trucks could evacuate people and their possessions; able-bodied adults would have little difficulty in wading to safety.

The key factors influencing flood hazard or risk are:

- Size of the flood;
- Rate of rise - effective warning time;
- Community awareness;
- Flood depth and velocity;
- Duration of inundation;
- Obstructions to flow; and
- Access and evacuation;

The provisional flood hazard level is often determined on the basis of the predicted flood depth and velocity. This is conveniently done through the analysis of flood model results. A high flood depth will cause a hazardous situation while a low depth may only cause an inconvenience. High flood velocities are dangerous and may cause structural damage while low velocities have no major threat.

Figures L1 and L2 in the Floodplain Development Manual (NSW Government, 2005) are used to determine provisional hazard categorisations within flood liable land. These figures are reproduced in Figure 8-13. The provisional hydraulic hazard is included in the mapping series provided in Appendix D for the 1% AEP and PMF events (Figure D-22 and Figure D-23).

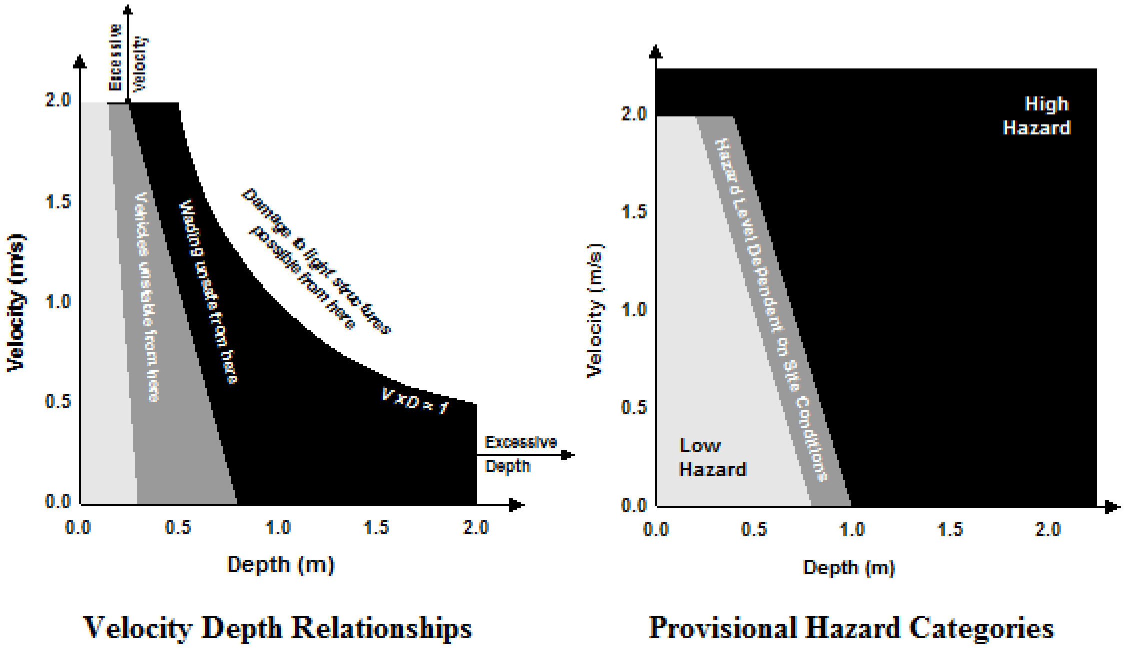


Figure 8-13 Provisional Flood Hazard Categorisation

8.5.8 Preliminary True Hazard

The provisional hazard categorisations (Section 8.5.8) were reviewed to consider other factors such as the rate of rise of floodwaters, duration of flooding, evacuation problems and the effective flood access. The full list of factors reviewed and the associated comments relating to the preliminary true hazard assessment are provided in Table 8-10.

Table 8-10 Assessment of Preliminary True Hazard

Factor	Comment
Size of flood	Preliminary true hazard has been determined by the 1% AEP and PMF provisional flood hazard categorisation which captures the size of the flood.
Effective warning time	The critical storm duration across the majority of the study area is 2 hours and there is limited effective warning time. No particular areas would be subject to a higher hazard category on the basis of the limited flood warning time.
Flood Readiness	Generally flood readiness in the Hewitts Creek study area is relatively high due to the history of flooding in the catchment and community awareness programmes undertaken by Council. No particular parts of the study area could be defined as being more or less flood ready than another. Therefore, the provisional flood hazard has not been altered due to flood readiness.
Rate of rise of floodwaters	Through workshop proceedings as part of a case study of two floodplains in the Illawarra region (Maratea et al, n.d.), it was determined that areas with a high rate of rise should be assessed for inclusion in the preliminary true high hazard extent. A combined rate of rise and depth criterion was adopted to define high hazard areas. High hazard areas were defined as those with a rate of rise of greater than 1m per hour and a flood depth of greater than 500mm. Rate of rise of floodwaters is generally high in the Hewitts Creek study area which impacts on the time residents have to prepare for the onset of flooding. Areas which have a rate of rise greater than 1m/hour and reach a flood depth of greater than 500mm have been compared to the provisional hazard

Factor	Comment
	categories for the 1% AEP event. No additional areas have been defined as high hazard based on the rate of rise of flood waters.
Depth and velocity of floodwaters	Preliminary true hazard has been determined by 1% AEP and PMF provisional flood hazard categories which captures the depth and velocity of floodwaters.
Duration of flooding	Through workshop proceedings as part of a case study of two floodplains in the Illawarra region (Maratea et al, n.d.), 24 hours was determined as the threshold duration of inundation which would identify a property to be classified as high hazard. The duration of local catchment flooding is generally less than 6 hours across the Hewitts Creek study area. This will result in inconvenience to residents; however no additional areas have been defined as high hazard due to duration of flooding.
Evacuation problems	Flood warning and the development of Local Flood Plans by the State Emergency Service (SES), are widely used throughout NSW to reduce flood damages and the risk to life in locations of existing development. Given the quick response of the catchment and the SES policy of encouraging people to stay in their homes rather than evacuate, no additional areas have been defined as high hazard due to evacuation problems.
Effective flood access	Through workshop proceedings as part of a case study of two floodplains in the Illawarra region (Maratea et al, n.d.), effective flood access was determined to be a road which is flooded to a depth of less than 300mm of water. A property was considered to have an access hazard issue if it had no effective access for at least 24 hours. There are no areas within the study area which are inundated to 300mm for a period greater than 24 hours; therefore no additional areas have been defined as high hazard due to effective flood access.
Type of development.	The degree of hazard to be managed is also a function of the type of development and resident mobility. This may alter the type of development considered appropriate in new development areas and modify management strategies in existing development areas. Wollongong City Council currently has development control policies (DCPs) which consider and manage development within floodplains. The DCPs provide guidelines for potential development within for high, medium and low flood risk precincts. Through workshop proceedings as part of a case study of two floodplains in the Illawarra region (Maratea et al, n.d.), it was determined that the hazard category of individual properties would not be altered due to current land use type. However, it was considered useful to identify properties which are currently located within the floodplain which may require special consideration in terms of flood impacts such as schools, aged care facilities and community buildings. No additional areas have been defined as high hazard due to the type of development.

Based on the above assessment, no additional areas have been defined as high hazard and the provisional hazard categorisations remain unchanged (refer to the provisional hydraulic hazard category maps in Appendix D, Figure D-22 and Figure D-23).

Detailed floor level survey may warrant a review of the preliminary true hazard categories as part of the Floodplain Risk Management Study.

8.5.9 Flood Emergency Response Classification (FERC)

The NSW Government's Floodplain Development Manual (NSW Government, 2005) requires flood studies and subsequent floodplain risk management studies to address the management of continuing flood risk to both existing and future development areas. Continuing flood risk may vary across a floodplain and as such the type and scale of emergency response does also. To assist the NSW SES with emergency response planning, floodplain communities may be classified into the following categories:

- **High Flood Island** – high ground within a floodplain. Road access may be cut by floodwater creating an island. The flood island includes enough land higher than the limit of flooding to provide refuge.
- **Low Flood Island** – high ground within a floodplain. Road access may be cut by floodwater creating an island. The flood island is lower than the limit of flooding.
- **High Trapped Perimeter** – fringe of the floodplain. Road access may be cut by floodwater. The area includes enough land higher than the limit of flooding to provide refuge.
- **Low Trapped Perimeter** – fringe of the floodplain. Road access may be cut by floodwater. The flood island is lower than the limit of flooding.
- **Areas with Overland Escape** Routes – areas available for continuous evacuation. Access roads may cross low lying flood prone land but evacuation can take place by walking overland to higher ground.
- **Areas with Rising Road Access** – areas available for continuous evacuation. Access roads may rise steadily uphill away from rising floodwaters. Evacuation can take place vehicle and communities cannot be completely isolated before inundation reaches its maximum; and
- **Indirectly Affected Areas** – areas outside the limit of flooding and therefore will not be inundated or lose road access. They may be indirectly affected as a result of flood damaged infrastructure or due to loss of services.

The DECC recommends that the classification of the floodplain be undertaken for the PMF, 5% AEP event and 1% AEP event (DECC, 2007). The FERC is included in the mapping series provided in Appendix E for the design events recommended by the DECC (Figure D-17 to Figure D-19).

A review of the FERC for the Hewitts Creek study area for the 1% AEP design flood event indicates that the majority of the study area is classified as "High Trapped Perimeter" and "Not Flood Affected". The remaining areas are classified as "High Flood Island", "Low Flood Island", "Rising Road Access Areas" and "Indirectly Affected Areas". The change in classifications varies slightly between AEP flood events and the PMF.

When preparing the FERC, it is important to note that consideration has only been given to the flood risk within the Hewitts Creek study area and does not consider the flood risk within adjoining catchments. Consideration of the flood risk within adjoining catchments may alter the flood emergency response classifications, particularly for areas around the perimeter of the Hewitts Creek study area.

It is recommended that FERC are reviewed as part of the Floodplain Risk Management Study stage to include consideration of flood risk in adjoining catchments, potential changes to the classification arising from detailed floor level survey data and to include input from the SES.

8.5.10 Preliminary Residential Flood Planning Level

Preliminary flood planning levels and extents were developed for the 1% AEP flood event under current conditions, and incorporating projected sea level rise increases of 0.4m and 0.9m respectively (refer to Section 9 for further information on these sea level rise increases).

The flood planning levels and extents have been derived based on the following methodology:

- 0.5 m has been added to the filtered (refer to Section 8.5.4) hydraulic model water level grids;
- The extent has been increased to reflect additional areas within the 1% AEP event peak flood level plus 0.5m, based on the TUFLOW ground surface grid. This step was undertaken using WaterRIDE software. In some areas of the catchment, particularly the steeper overland flow areas, this step results in significant increases in the flood extent; and
- The preliminary residential flood planning level may exceed the PMF peak flood level in some locations and therefore extend beyond the PMF flood extent. Therefore areas of the preliminary flood planning area outside the PMF extent have been removed.

Preliminary residential flood planning maps has been provided in Appendix D as Figure D-24, Figure D-25 and Figure D-26.

8.6 Summary of Design Flood Events

The developed models have been applied to derive design flood conditions within the Hewitts Creek study area using the design rainfall and tidal conditions described earlier in this Section. The design events considered in this study include the 20% AEP (5-year ARI), 10% AEP (10-year ARI), 5% AEP (20-year ARI), 2% AEP (50-year ARI), 1% AEP (100-year ARI) 0.5% AEP (200-year ARI), 0.2% AEP (500-year ARI) and Probable Maximum Flood (PMF) events. The model results for the design events considered have been presented in a detailed flood mapping series for the catchment in Appendix D. The flood data presented includes peak flood water levels and flood depths, peak flood velocities, provisional flood hazard categories, hydraulic categories, flood emergency response classification and preliminary residential flood planning area and levels.

9 SEA LEVEL RISE ANALYSIS

In 2009 the NSW State Government announced the NSW Sea Level Rise Policy Statement (DECC, 2009) that adopted sea level rise planning benchmarks to ensure consistent consideration of sea level rise in coastal areas of NSW. These planning benchmarks provided increases (above 1990 mean sea level) of 40 cm by 2050 and 90 cm by 2100. However, on 8 September 2012 the NSW Government announced its Stage One Coastal Management Reforms which no longer recommends state-wide sea level rise benchmarks for use by local councils. Instead councils have the flexibility to consider local conditions when determining future hazards of potential sea level rise. Council's adopted sea level rise projections are 0.4m for 2050 and 0.9m for 2100 which are in line with the NSW Chief Scientist and Engineer's Report (NSW Government, 2012).

The model results for the sea level rise projections have been translated into 0.4m and 0.9m flood planning levels and areas (refer to Section 8.5.10). These areas have been mapped to meet the needs of Council in addressing the NSW Coastal Planning Guideline: Adapting to Sea Level Rise, (DoP, 2010). The guideline outlines an approach to assist councils, State agencies, planners and development proponents when addressing sea level rise in land-use planning and development assessment. Mapping of the flood planning levels and areas for sea level rise projections of the 0.4m and 0.9m has been provided in Appendix D as Figure D-25 and Figure D-26.

9.1 The Potential Impact of Sea Level Rise

Worsening coastal flooding as a consequence of sea level rise is likely to impact lowland areas along the coastal extent of the study area, e.g. along Lake Parade, Thirroul and Blackhall Street, Bulli. Future planning will need to take due consideration of this increased flood risk.

The model configuration and assumptions adopted for these potential sea level rise impacts are discussed in the following sections.

9.1.1 Ocean Water Level

Based on the Councils adopted sea level rise projections, design ocean boundary conditions were raised by 0.4 m and 0.9 m to assess the potential impact of sea level rise on flood behaviour in the Hewitts Creek study area.

The ocean water level boundary conditions for current flood conditions are discussed in Section 8.2. The sea level rise allowances provide for direct increases in these ocean water levels. Table 9-1 presents a summary of adopted peak ocean water levels when applying the sea level rise benchmarks to the 1% AEP design flood event.

9.1.2 Modelled Scenarios

The same approach as set out in Section 8 for developing the current1% AEP event design flood envelope was adopted for developing flood envelopes representative of a 0.4m and 0.9m rise in sea levels. The scenarios modelled are detailed in Table 9-1.

Table 9-1 Sea Level Rise Flood Combinations

Flood Envelope	Design Rainfall Event	Sea Level Rise Ocean Boundary Condition	Structure Blockage Scenarios[1)]	Initial Water Level in the ICOLLs	Entrance Condition at the ICOLLs
1% AEP (100 year ARI) +0.4 m sea level rise	1% AEP (100 year ARI) 2hr and 9hr durations	Normal Tide (1.03 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
	1% AEP (100 year ARI) 2hr and 9hr durations	5% AEP Storm Tide (2.7 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
	5% AEP (20 year ARI) 2hr and 9hr durations	1% AEP Storm Tide (3.0 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
1% AEP (100 year ARI) +0.9 m sea level rise	1% AEP (100 year ARI) 2hr and 9hr durations	Normal Tide (1.53 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
	1% AEP (100 year ARI) 2hr and 9hr durations	5% AEP Storm Tide (3.2 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open
	5% AEP (20 year ARI) 2hr and 9hr durations	1% AEP Storm Tide (3.5 m AHD)	B01, B02 and B04	Ocean Boundary Condition	Open

1) Refer to Table 8-5 for further information

9.1.3 Impacts of Sea Level Rise

Table 9-2 indicates the changes in water levels resulting from a 0.4m and 0.9m rise in sea levels and the differences in levels compared to the 1% AEP critical flood envelope for the reporting locations indicated in Figure 8-7.

Table 9-2 Peak Flood Levels for Sea Level Rise Impacts

Location (refer to Figure 8-7)	1% AEP Peak Water Level (m AHD)			Difference (m)	
	Design Flood Event	+ 0.4m sea level rise	+ 0.9m sea level rise	+ 0.4m sea level rise	+ 0.9m sea level rise
US Illawarra Railway, Thirroul	14.46	14.46	14.46	0.00	0.00
US McCauley Street, Thirroul	7.17	7.17	7.17	0.00	0.00
US Cliff Parade, Thirroul	4.75	4.75	4.75	0.00	0.00
US Deborah Ave., Thirroul	63.25	63.25	63.25	0.00	0.00
US Virginia Terrace, Thirroul	50.16	50.16	50.16	0.00	0.00
US George Street, Thirroul	32.38	32.38	32.38	0.00	0.00
US Palm Crescent, Thirroul	75.14	75.14	75.14	0.00	0.00
US Virginia Terrace, Thirroul	40.99	40.99	40.99	0.00	0.00
US George Street, Thirroul	29.48	29.48	29.48	0.00	0.00
US Kelton lane, Thirroul	25.35	25.35	25.35	0.00	0.00
US Lachlan Street, Thirroul	19.19	19.19	19.19	0.00	0.00
US Lawrence Hargrave, Thirroul	14.57	14.57	14.57	0.00	0.00
US Illawarra Railway, Thirroul	14.06	14.06	14.06	0.00	0.00
US Brickworks Avenue, Thirroul	11.58	11.58	11.59	0.00	+0.01
US Hamilton Road, Thirroul	2.60	3.00	3.50	+0.40	+0.90
US Princes Highway, Thirroul	19.15	19.15	19.15	0.00	0.00
US Illawarra Railway, Thirroul	16.59	16.59	16.59	0.00	0.00
US Air Avenue, Thirroul	11.19	11.19	11.19	0.00	0.00
US Illawarra Railway, Bulli	17.29	17.29	17.29	0.00	0.00
US William Street, Bulli	27.57	27.57	27.57	0.00	0.00
US Hobart Street, Bulli	22.72	22.72	22.72	0.00	0.00
US coal haulage embankment, Bulli	22.95	22.95	22.95	0.00	0.00
US Bulli Showground, Bulli	18.32	18.32	18.32	0.00	0.00
US Princes Highway, Bulli	14.65	14.65	14.65	0.00	0.00
US Bulli Park, Bulli	13.22	13.22	13.22	0.00	0.00
US Illawarra Railway, Bulli	13.12	13.12	13.12	0.00	0.00
US Illawarra Railway (Beacon	12.90	12.90	12.90	0.00	0.00

Location (refer to Figure 8-7)	1% AEP Peak Water Level (m AHD)			Difference (m)	
	Design Flood Event	+ 0.4m sea level rise	+ 0.9m sea level rise	+ 0.4m sea level rise	+ 0.9m sea level rise
Avenue), Bulli					
US timber footbridge (Beach Street), Bulli	4.75	4.75	4.75	0.00	0.00
US Blackhall Street, Bulli	2.98	3.08	3.54	+0.10	+0.56

The impacts on flood levels and extents resulting from projected sea level rise are relatively low with impacts confined to the downstream reaches of the Study Area. The results presented in Table 9-2 show that a 0.4m and 0.9m rise in sea levels results in localised impacts on the peak water levels for the 1% AEP event along both Hewitts and Slacky Creeks. The rise in water levels along these creeks is equivalent to the corresponding sea level rise and is localised to within 200m of the creek entrances.

9.2 Summary of Sea Level Rise Analysis

The potential impacts of future sea level rise have been considered for the design event scenarios defined in Table 9-1. The model results for the sea level rise projections have been translated into 0.4m and 0.9m flood planning levels and areas, provided as flood planning maps in Appendix D. Impacts on flood levels and areas resulting from projected sea level rise is confined to the downstream reaches of the study area. These impacts are relatively low for both a 0.4m sea level rise and a 0.9m sea level rise. Future planning and floodplain risk management in the catchment will need to take due consideration of the increasing flood risk under possible sea level rise conditions.

10 SENSITIVITY TESTS

A number of sensitivity tests have been undertaken on the modelled flood behaviour in the Hewitts Creek study area. In defining sensitivity tests, consideration has been given to the most appropriate tests taking into account catchment properties and simulated design flood behaviour. The tests undertaken have included:

- WBNM Lag Parameter;
- Intensity of flood producing rainfall events;
- Hydraulic roughness; and
- Channel sedimentation.

The rationalisation for each of these sensitivity tests along with adopted model configuration/parameters and results are summarised in the following sections.

The 1% AEP critical flood envelope (refer to Table 8-2), forms the baseline design flood condition against which the sensitivity model results are compared.

10.1 WBNM Lag Parameter

Sensitivity tests on the WBNM Lag Parameter have been undertaken by assessing a Lag Parameter value of 1.70. As discussed in Section 6.4.4, experimental derivation of the Lag Parameter found that a value of 1.68 gave a good fit to all the data and a Lag Parameter Value of 1.70 is considered to be a good 'average' value, particularly for catchments where calibration data are not available to verify this parameter. The results of the sensitivity tests on WBNM Lag Parameter are summarised in Table 10-1 for the reporting locations displayed in Figure 8-7.

Table 10-1 Peak Flood Levels for Sensitivity Test to WBNM Lag Parameter

Location (refer to Figure 8-7)	1% AEP Peak Water Level (m AHD)		Difference (m)
	Design Flood Event	Lag Parameter C 1.7	
US Illawarra Railway, Thirroul	14.46	14.46	+0.00
US McCauley Street, Thirroul	7.17	7.16	-0.01
US Cliff Parade, Thirroul	4.75	4.74	-0.01
US Deborah Ave., Thirroul	63.25	63.20	-0.04
US Virginia Terrace, Thirroul	50.16	50.08	-0.07
US George Street, Thirroul	32.38	32.35	-0.02
US Palm Crescent, Thirroul	75.14	75.13	-0.01
US Virginia Terrace, Thirroul	40.99	40.97	-0.02
US George Street, Thirroul	29.48	29.44	-0.04
US Kelton lane, Thirroul	25.35	24.76	-0.58

Location (refer to Figure 8-7)	1% AEP Peak Water Level (m AHD)		Difference (m)
US Lachlan Street, Thirroul	19.19	19.09	-0.09
US Lawrence Hargrave, Thirroul	14.57	14.49	-0.08
US Illawarra Railway, Thirroul	14.06	13.83	-0.23
US Brickworks Avenue, Thirroul	11.58	11.54	-0.04
US Hamilton Road, Thirroul	2.60	2.67	+0.07
US Princes Highway, Thirroul	19.15	19.08	-0.07
US Illawarra Railway, Thirroul	16.59	16.50	-0.09
US Air Avenue, Thirroul	11.19	11.18	-0.01
US Illawarra Railway, Bulli	17.29	17.25	-0.04
US William Street, Bulli	27.57	27.47	-0.11
US Hobart Street, Bulli	22.72	22.60	-0.12
US coal haulage embankment, Bulli	22.95	22.92	-0.03
US Bulli Showground, Bulli	18.32	18.18	-0.14
US Princes Highway, Bulli	14.65	14.64	-0.01
US Bulli Park, Bulli	13.22	13.17	-0.05
US Illawarra Railway, Bulli	13.12	13.03	-0.09
US Illawarra Railway (Beacon Avenue), Bulli	12.90	12.88	-0.03
US timber footbridge (Beach Street), Bulli	4.75	4.68	-0.07
US Blackhall Street, Bulli	2.98	2.94	-0.04

As discussed in Section 6.4.4, the WBNM model has been used to provide local flow inputs into the TUFLOW hydraulic model at the various sub-area outlets downstream of the Illawarra Escarpment. The TUFLOW hydraulic model simulates the behaviour of the runoff from the hydrological model by routing the flow hydrographs through the two dimensional grid of the study area. As the routing of the sub-area flows is being undertaken within the hydraulic model and not internally routed through the WBNM model, changing the Lag Parameter within the WBNM model has a relatively minor impact on model results.

Changes to the Lag Parameter C generally results in water levels that are within 0.1m of the peak flood levels from the 1% AEP design event model results at the reporting locations. There are some localised reaches where the differences exceed 0.1m, i.e. upstream of the Illawarra Railway, Thirroul, however at the majority of locations the differences are less than 0.1m and the model is considered insensitive to changes in the Lag Parameter C.

10.2 Intensity of Flood Producing Rainfall Events

The sensitivity of the model to changes in the intensity of flood producing rainfall events, due to the potential impacts of climate change, has been assessed by comparing the 0.5% AEP and 0.2% AEP peak flood levels to the baseline design flood condition. The 0.5% and 0.2% AEP represents an average rainfall intensity increase above the design 1% AEP condition of the order of 15% and 30% respectively. The results of the sensitivity analysis are summarised in Table 10-2 for the reporting locations displayed in Figure 8-7.

Table 10-2 Peak Flood Levels for Rainfall Intensity Sensitivity Test

Location (refer to Figure 8-7)	1% AEP Peak Water Level (m AHD)			Difference (m)	
	Design Flood Event	0.5% AEP Event	0.2% AEP Event	0.5% AEP Event	0.2% AEP Event
US Illawarra Railway, Thirroul	14.46	14.48	14.49	+0.01	+0.03
US McCauley Street, Thirroul	7.17	7.19	7.21	+0.02	+0.04
US Cliff Parade, Thirroul	4.75	4.80	4.86	+0.05	+0.11
US Deborah Ave., Thirroul	63.25	63.28	63.33	+0.03	+0.08
US Virginia Terrace, Thirroul	50.16	50.23	50.28	+0.07	+0.13
US George Street, Thirroul	32.38	32.43	32.48	+0.05	+0.10
US Palm Crescent, Thirroul	75.14	75.18	75.20	+0.03	+0.06
US Virginia Terrace, Thirroul	40.99	41.02	41.07	+0.03	+0.08
US George Street, Thirroul	29.48	29.55	29.62	+0.07	+0.15
US Kelton lane, Thirroul	25.35	25.61	25.91	+0.26	+0.56
US Lachlan Street, Thirroul	19.19	19.27	19.37	+0.08	+0.18
US Lawrence Hargrave, Thirroul	14.57	14.63	14.96	+0.06	+0.39
US Illawarra Railway, Thirroul	14.06	14.48	14.94	+0.42	+0.88
US Brickworks Avenue, Thirroul	11.58	11.61	11.83	+0.03	+0.25
US Hamilton Road, Thirroul	2.60	2.71	2.70	+0.11	+0.10
US Princes Highway, Thirroul	19.15	19.22	19.31	+0.07	+0.16
US Illawarra Railway, Thirroul	16.59	16.68	16.80	+0.09	+0.21
US Air Avenue, Thirroul	11.19	11.20	11.21	+0.01	+0.02
US Illawarra Railway, Bulli	17.29	17.34	17.40	+0.05	+0.12
US William Street, Bulli	27.57	27.66	27.74	+0.08	+0.17
US Hobart Street, Bulli	22.72	22.80	22.89	+0.08	+0.18
US coal haulage embankment, Bulli	22.95	23.01	23.07	+0.06	+0.12
US Bulli Showground, Bulli	18.32	18.38	18.42	+0.06	+0.10
US Princes Highway, Bulli	14.65	14.61	14.62	-0.05	-0.03
US Bulli Park, Bulli	13.22	13.37	13.55	+0.14	+0.33
US Illawarra Railway, Bulli	13.12	13.29	13.50	+0.17	+0.38
US Illawarra Railway (Beacon Avenue), Bulli	12.90	13.12	13.36	+0.22	+0.45
US timber footbridge (Beach Street), Bulli	4.75	4.87	5.01	+0.12	+0.26
US Blackhall Street, Bulli	2.98	3.05	3.16	+0.07	+0.18

The analysis shows relatively small increases in peak flood levels along Thomas Gibson Creek and along the eastern and western tributaries of Hewitts Creek. Generally the differences are < 0.1m for both the 0.5% AEP and 0.2% AEP events at the reporting locations.

Along the main channel of Hewitts Creek, Woodlands Creek, Tramway Creek and Slacky Creek, the results show that there are more significant differences in peak levels between the 1% AEP baseline design event and the 0.5% AEP event and 0.2% AEP event. The average difference in peak levels along these creeks, at the reporting locations, is +0.09m for the 0.5% AEP event and the peak difference in levels is +0.42m upstream of the Illawarra Railway on Hewitts Creek. For the 0.2% AEP event, the average difference in peak levels is +0.21m and the peak difference in levels is +0.88m upstream of the Illawarra Railway on Hewitts Creek.

10.3 Hydraulic Roughness

Sensitivity tests on the hydraulic roughness (Manning's 'n') have been undertaken by applying a 20% decrease and a 20% increase in the adopted values for the baseline design flood conditions. Whilst adopted design parameters are within typical ranges, the inherent variability/uncertainty in hydraulic roughness warrants consideration of the relative impact on adopted design flood conditions. The results of the sensitivity tests on hydraulic roughness are summarised in Table 10-3 for the reporting locations displayed in Figure 8-7.

Table 10-3 Peak Flood Levels for Hydraulic Roughness Sensitivity Test

Location (refer to Figure 8-7)	1% AEP Peak Water Level (m AHD)			Difference (m)	
	Design Flood Event	Manning's 'n' + 20%	Manning's 'n' - 20%	Manning's 'n' + 20%	Manning's 'n' - 20%
US Illawarra Railway, Thirroul	14.46	14.47	14.45	+0.01	-0.01
US McCauley Street, Thirroul	7.17	7.18	7.16	+0.01	-0.01
US Cliff Parade, Thirroul	4.75	4.76	4.75	+0.01	+0.00
US Deborah Ave., Thirroul	63.25	63.27	63.24	+0.03	0.00
US Virginia Terrace, Thirroul	50.16	50.16	50.15	-0.00	-0.01
US George Street, Thirroul	32.38	32.41	32.39	+0.03	+0.01
US Palm Crescent, Thirroul	75.14	75.17	75.14	+0.03	+0.00
US Virginia Terrace, Thirroul	40.99	41.02	40.96	+0.03	-0.03
US George Street, Thirroul	29.48	29.54	29.43	+0.06	-0.04
US Kelton lane, Thirroul	25.35	25.41	25.36	+0.07	+0.01
US Lachlan Street, Thirroul	19.19	19.20	19.18	+0.01	0.00
US Lawrence Hargrave, Thirroul	14.57	14.58	14.56	+0.01	-0.01
US Illawarra Railway, Thirroul	14.06	14.04	14.07	-0.02	+0.01
US Brickworks Avenue, Thirroul	11.58	11.51	11.58	-0.07	0.00
US Hamilton Road, Thirroul	2.60	2.61	2.67	+0.01	+0.07

Location (refer to Figure 8-7)	1% AEP Peak Water Level (m AHD)			Difference (m)	
	Design Flood Event	Manning's 'n' + 20%	Manning's 'n' - 20%	Manning's 'n' + 20%	Manning's 'n' - 20%
US Princes Highway, Thirroul	19.15	19.17	19.13	+0.02	-0.02
US Illawarra Railway, Thirroul	16.59	16.65	16.59	+0.06	0.00
US Air Avenue, Thirroul	11.19	11.29	11.12	+0.10	-0.07
US Illawarra Railway, Bulli	17.29	17.32	17.27	+0.03	-0.02
US William Street, Bulli	27.57	27.61	27.57	+0.04	0.00
US Hobart Street, Bulli	22.72	22.76	22.68	+0.04	-0.03
US coal haulage embankment, Bulli	22.95	22.97	22.97	+0.01	+0.01
US Bulli Showground, Bulli	18.32	18.37	18.29	+0.05	-0.04
US Princes Highway, Bulli	14.65	14.73	14.56	+0.08	-0.09
US Bulli Park, Bulli	13.22	13.23	13.23	+0.01	+0.01
US Illawarra Railway, Bulli	13.12	13.10	13.15	-0.02	+0.03
US Illawarra Railway (Beacon Avenue), Bulli	12.90	12.88	12.95	-0.02	+0.05
US timber footbridge (Beach Street), Bulli	4.75	4.86	4.67	+0.11	-0.08
US Blackhall Street, Bulli	2.98	3.09	2.95	+0.11	-0.03

The model simulation results generally show minor increases in peak flood level (typically < 0.1m) for a 20% increase in Manning's 'n' values. Similarly for a 20% reduction in Manning's 'n' values, the model results generally show minor decreases in peak flood level (typically < 0.1m)and the model is considered insensitive to changes in Manning's 'n' values.

10.4 Channel Sedimentation

Sensitivity model runs have been undertaken to determine the impact of channel sedimentation along zones of potential deposition. The material deposited along the channel reaches may include sediment, vegetation and rubbish. To reflect the loss of conveyance which can occur when material is deposited in the bed of the creeks, the cross sectional area of the mildly steep reaches of creeks which lie to the west of the Illawarra Railway been reduced by raising bed elevations by a nominal 0.5m. Figure 10-1 shows the reaches where adjustments to the cross sectional area have been made to reflect channel sedimentation. The reaches identified correlate with the reaches where deposition has previously occurred, particularly after the August 1998 flood event. This flood event highlighted the significant issue of sediment, vegetation and rubbish being deposited in the upper and middle reaches of the creeks and adjacent flow areas. The approach adopted is consistent with the methodology undertaken for other Council flood studies in the Wollongong region (e.g. Combined Catchments of Whartons, Collins and Farrahars Creeks, Bellambi Gully and Bellambi Lake Flood Study, 2011).

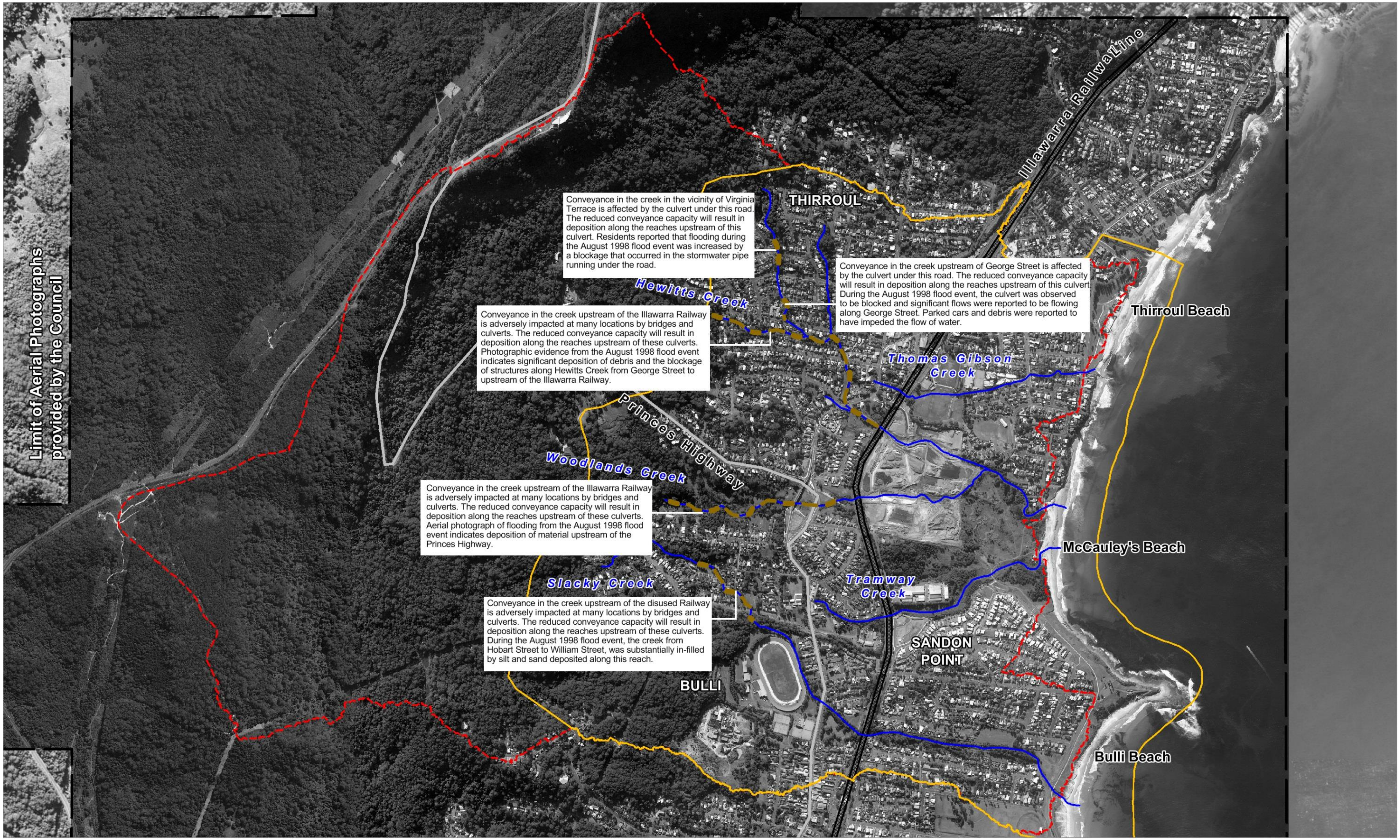
The results of the sensitivity tests on channel sedimentation are summarised in Table 10-4 for the reporting locations displayed in Figure 8-7.

Table 10-4 Peak Flood Levels for Channel Sedimentation Sensitivity Test

Location (refer to Figure 8-7)	1% AEP Peak Water Level (m AHD)		Difference (m)
	Design Flood Event	Creek bed levels raised by 0.5m	
US Illawarra Railway, Thirroul	14.46	14.46	0.00
US McCauley Street, Thirroul	7.17	7.17	0.00
US Cliff Parade, Thirroul	4.75	4.75	0.00
US Deborah Ave., Thirroul	63.25	63.25	0.00
US Virginia Terrace, Thirroul	50.16	50.13	-0.02
US George Street, Thirroul	32.38	32.37	0.00
US Palm Crescent, Thirroul	75.14	75.14	0.00
US Virginia Terrace, Thirroul	40.99	40.99	0.00
US George Street, Thirroul	29.48	29.48	0.00
US Kelton lane, Thirroul	25.35	25.58	+0.23
US Lachlan Street, Thirroul	19.19	19.21	+0.02
US Lawrence Hargrave, Thirroul	14.57	14.56	-0.01
US Illawarra Railway, Thirroul	14.06	14.06	0.00
US Brickworks Avenue, Thirroul	11.58	11.59	+0.01
US Hamilton Road, Thirroul	2.60	2.60	0.00
US Princes Highway, Thirroul	19.15	19.15	-0.01
US Illawarra Railway, Thirroul	16.59	16.59	0.00
US Air Avenue, Thirroul	11.19	11.19	0.00
US Illawarra Railway, Bulli	17.29	17.29	0.00
US William Street, Bulli	27.57	27.57	0.00
US Hobart Street, Bulli	22.72	22.72	0.00
US coal haulage embankment, Bulli	22.95	22.95	0.00
US Bulli Showground, Bulli	18.32	18.32	0.00
US Princes Highway, Bulli	14.65	14.65	0.00
US Bulli Park, Bulli	13.22	13.22	0.00
US Illawarra Railway, Bulli	13.12	13.12	0.00
US Illawarra Railway (Beacon Avenue), Bulli	12.90	12.90	0.00
US timber footbridge (Beach Street), Bulli	4.75	4.75	0.00
US Blackhall Street, Bulli	2.98	2.98	0.00

For the majority of locations, the model simulation results show negligible differences in peak flood levels for increases in the creek bed levels. The flood levels along the selected zones of potential deposition are primarily controlled by the blockages at the structures.

Consideration will need to be given during the next stage of the study to flood risk management measures that reduce the risk of blockages at these structures.



LEGEND

- TUFLOW Model Boundary
- Catchment Boundary
- Watercourses
- Channel Sedimentation

Title:

Sensitivity Analysis - Channel Sedimentation
Channel Reaches where Creek Bed Levels have been Raised

Figure: 10-1

Rev: 0

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Approx. Scale

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10.5 Summary of Sensitivity Analysis

A series of sensitivity tests have been undertaken on the modelled flood behaviour of the Hewitts Creek study area. The tests provide a basis for determining the relative sensitivity of modelling results to adopted parameter values. The parameters assessed are detailed in Table 10-5.

Table 10-5 Summary of Sensitivity Tests

Parameter	Baseline Parameter Value	Sensitivity Parameter Value
WBNM Lag Parameter	1.29	1.7
Rainfall Intensity	1% AEP event 2hr and 9hr durations	0.5% AEP event, 2hr and 9hr durations (an increase in the order of 15% from the baseline)
		0.2% AEP event, 2hr and 9hr durations (an increase in the order of 30% from the baseline)
Hydraulic Roughness	Refer to Table 7-5	+20% in Manning's 'n' values
		-20% in Manning's 'n' values
Channel Sedimentation	Refer to Section 6.5.2	0.5m increase in creek bed levels at selected locations

The results of the sensitivity analysis indicate that the model is generally insensitive to the majority of the parameters assessed with the exception of the rainfall intensity.

11 CONCLUSIONS

The objective of the study was to undertake a detailed review of the Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a) and establish models as necessary for accurate flood level prediction. Central to this was the development of a two-dimensional hydraulic model of the study area.

In completing the flood study, the following activities were undertaken:

- Collation of data relevant to a review and update of the Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002 a);
- Acquisition of topographical data for the catchment including cross sections and hydraulic structure survey;
- Consultation with the community to acquire historical flood information and liaison in regard to flooding concerns/perceptions;
- Development of a hydrological model (using WBNM software) and hydraulic model (using TUFLOW software) to simulate flood behaviour in the catchment;
- Calibration and validation of the models using available data for the April 1988 event, August 1998 event and February 2013 event;
- Prediction of design flood conditions in the catchment using the calibrated and validated models;
- Production of design flood mapping series;
- Comparison of design flood results between the current study and the Hewitts Creek Flood Study (Forbes Rigby Pty Ltd., 2002a);
- Prediction of sea level rise impacts using the using the calibrated and validated models; and
- Establishment of the model sensitivity to changes in model parameters.

The principal outcome of the flood study is the understanding of current and future flood behaviour in the catchment and in particular flood level information that will be used to set appropriate flood planning levels for the study area. The outputs from the flood study will also form the basis for the subsequent floodplain risk management activities, being the next stage of the floodplain management process. The hydraulic model developed for this study also provides a tool for assessment of the potential flood impact of future development in the catchment.

It is important to note that results presented in this report provide an up-to-date prediction of flood behaviour using the best modelling techniques currently available. However, the interpretation of the maps and other data presented in this report should include an appreciation of the limitations of their accuracy.

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