

IDENTIFYING FLOOD RISK FOR RAPID CREEK

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Introduction

Rapid Creek experienced flooding caused by heavy rainfall over Darwin in February 2011, during the formation of Cyclone Carlos. A rainfall of 340 mm was recorded at Darwin Airport (Refer Figure 1) on 15 February 2011 and the stream gauge on Rapid Creek downstream of McMillans Road recorded a peak height of 3.74 m (gauge datum value). A number of houses in the suburb of Millner were affected by flooding.

The highest peak attained at this gauge before this event was 3.67 m in March 1977.

Sinclair Knight Merz was subsequently commissioned by the NT Government Department of Natural Resources, Environment, the Arts and Sports (NRETAS)ⁱ to examine flooding of Rapid Creek.

The study brief required:

- A hydrology study to produce design hydrographs for input into a hydraulic model
- A hydraulic model to determine flood levels across the creek and its flood plain downstream of a flood control weir 500 m upstream of Henry Wrigley Drive
- Flood plain maps showing the extent of inundation, the highest hazard areas inundated and the calculated depths of inundation for a range of flood scenarios

Rapid Creek rises in the Marrara Swamp at the eastern end of Darwin Airport, and flows for 9.8 km discharging into Beagle Gulf at the southern end of Casuarina Beach (Refer Figure 1). The Rapid Creek catchment covers an area of 28 sq km and includes parts of the suburbs of Karama, Malak, North Lakes, Anula, Moil, Jingili, Wulagi, Alawa, Casuarina, Wanguri, Nakara, Brinkin, Millner and Rapid Creek.

In built up areas of the catchment runoff enters the creek via underground piped drainage systems as well as unlined and lined open drains. Parts of the catchment to the south of McMillans Road remain undeveloped. The Marrara Swamp is drained by two separate drainage lines, one on the north western and the other on the south western side of the swamp. Where the two drainage lines join to form Rapid Creek, a flood control weir was constructed in 1985.

Hydrology study

The hydrology study comprised:

- A flood frequency analysis. Collection of continuous streamflow records commenced in the 1960s at gauging station G8150127 on Rapid Creek
- Establishment of an URBS rainfall-runoff model

- Calibration of the URBS model using available recorded data for floods on Rapid Creek. Suitable records of floods were available from 1969 up to and including the flood of February 2011
- Review the calibrations and adopt parameters to use in URBS model design runs
- Run the URBS model in design mode using the adopted parameter set; together with appropriate loss values to simulate peak discharges consistent with the flood frequency analysis
- Run the URBS model in design mode using estimated probable maximum precipitation to produce hydrographs for probable maximum floods.

Figure 1. Rapid Creek catchment



Flood frequency analysis

SKM's in-house software package GETDAT was used for flood frequency analysis. The Generalised Extreme Value distribution was fitted to an annual series of peak flows from 1963/64 to 2010/2011 in order to find peak discharges for the probabilities of interest. Values of LH moment shift from zero to four were tried.

Table 1 shows peak discharges with LH shift = 0 and for a range of Annual Exceedance Probabilities (AEPs).

The flood control weir was constructed upstream of the gauging station in 1985. This makes a discontinuity in the recorded annual series because the flood control weir attenuates flood peaks. The flood control weir commands approximately two thirds of

the catchment area above the gauging station, so this attenuation can be significant for a particular largest annual flood, depending on its size and nature.

To examine this effect a second annual series was constructed for post-flood control weir conditions. For the largest flood in each year before 1985, the calibrated URBS model was used to derive a flood peak at G8150127 as if the weir had existed from the start of the series. Flood frequency analysis was carried out for the resulting series and Table 2 show the results for LH shift = 0.

Table 1. Calculated raw data flood quantiles for LH shift = 0 using Generalised Extreme Value distribution

Annual Exceedance Probability (AEP)	Average Recurrence Interval (ARI) (yrs)	Peak discharge (Q_{peak}) (m^3/sec)	Lower Confidence Interval (m^3/sec)	Upper Confidence Interval (m^3/sec)
2	0.500	43	35	51
5	0.200	74	62	86
10	0.100	96	79	113
20	0.050	116	94	142
50	0.020	144	110	189
100	0.010	165	120	230
200	0.005	186	130	279
500	0.002	214	141	347

A comparison of the raw data and post-weir series flood frequencies is shown in Figure 2, plotted against a logarithmic scale of ARI. As expected, the impact of the flood control weir is to shift the flood frequency curve downward as a result of attenuation of flood peaks.

Table 2. Calculated flood quantiles for LH shift = 0 using Generalised Extreme Value Distribution on series of annual flows adjusted to post-weir conditions

Annual Exceedance Probability (AEP)	Average Recurrence Interval (ARI) (yrs)	Peak discharge (Q_{peak}) (m^3/sec)	Lower Confidence Interval (m^3/sec)	Upper Confidence Interval (m^3/sec)
0.500	2	38	31	46
0.200	5	68	56	78
0.100	10	88	72	103
0.050	20	108	86	131
0.020	50	134	102	174
0.010	100	155	114	217
0.005	200	176	124	268
0.002	500	204	134	341

It can be argued that the post-weir series should be adopted because these data more closely reflect current conditions. The adopted flood frequencies are those for the post-weir series with LH shift zero. LH = 0 gives the largest (most conservative) values of the flood peaks for the post weir series.

The adopted flood frequency curve is shown in Figure 3 and listed in Table 2.

Figure 2. Comparison of raw data series and post-weir data series flood frequencies with moment shift LH = 0

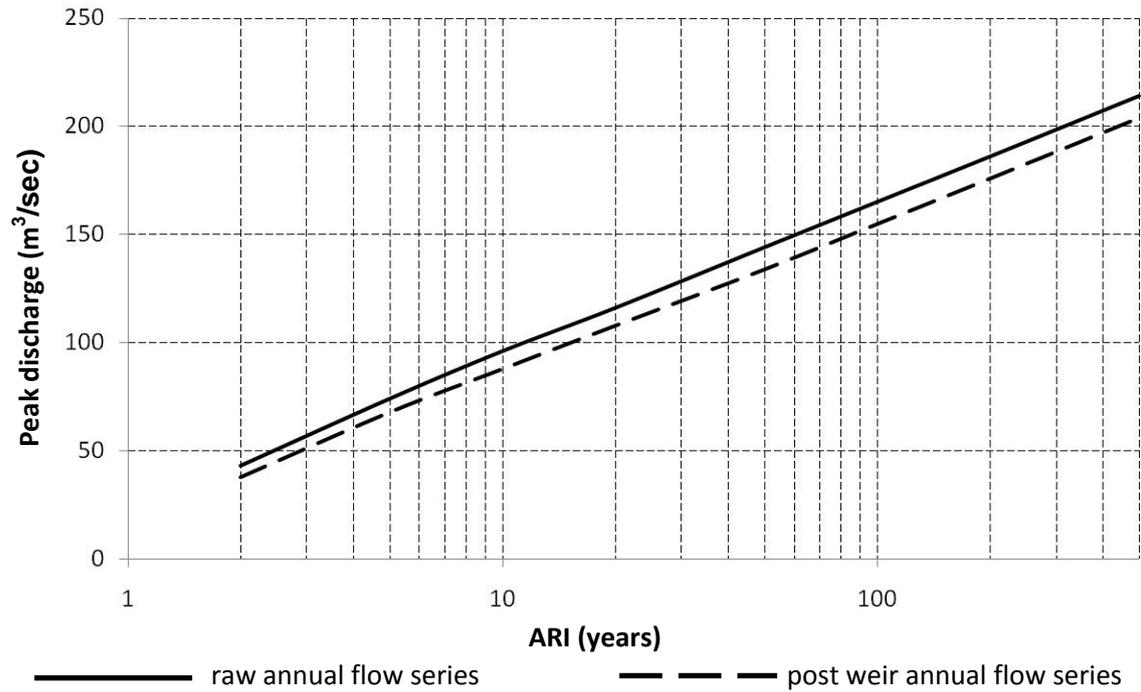
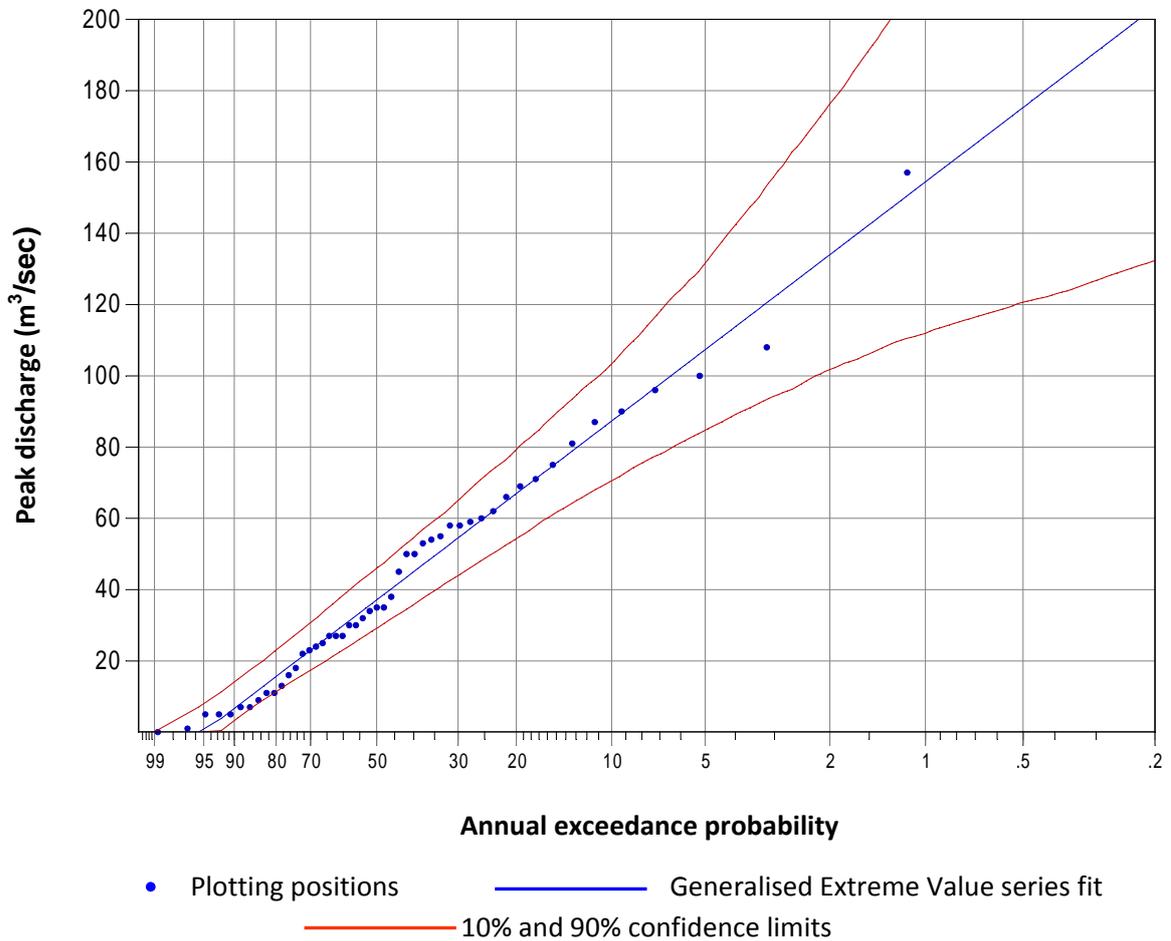


Figure 3. Adopted flood frequency curve



URBS modelling study

The URBS model is a hydrologic network model that calculates discharges at locations of interest from rainfall inputs. URBS is described in the URBS Manual (Carroll, 2009.)

For Rapid Creek:

- The catchment area to the outlet of Rapid Creek is 27.8 sq km.
- The catchment area to stream gauging station (G8150127) is 18.9 sq km.
- The catchment area to the flood control weir that was constructed in 1985 is 13.7 sq km.
- There is additional storage in the Marrara swamp which has a catchment area of approximately 2.4 sq km.
- The area has been divided into 26 sub-areas for the URBS model

The Marrara swamp has been modelled as a special storage. An elevation-storage relationship was derived from topographic data. The swamp was assumed to start empty for each storm.

In the absence of more specific data, it has been assumed that all overflows from the Marrara Swamp split 50-50 between the two flow paths. The outflow from the swamp was estimated as weir flow with a weir coefficient of 1.6 and a weir length of 100 m (as the sum of the two broad flow paths.)

The flood control weir has also been modelled as a separate feature. For the flood control weir, data was input as a storage-discharge relationship. The discharge data were based on flows over the design weir profile calculated using the weir formula $Q=C \times L \times H^{3/2}$. with a weir coefficient C of 1.6. In the absence of detailed survey, the volume of storage above the weir was inferred from topographic mapping. The invert of the weir is close to the level of the creek bed such that there is no 'dead storage'.

Calibration of the URBS model involves running the model with observed rainfall data to produce calculated hydrographs at a location where they can be compared to recorded hydrographs. For Rapid Creek there is a stream gauging station (G8150127) downstream of the bridge over McMillans Road (Figure 1.) The catchment area to this location is 18.9 sq km, so that it commands approximately two thirds of the catchment.

In order to calculate hydrographs from observed rainfall for comparison with recorded hydrographs the data required for each storm to be modelled is:

- Flows at G8150127
- Sufficient rainfall data to describe how rain varied over the catchment area during the storm
- Sufficient rainfall data to describe how rain varied in time over the catchment area

G8150127 commenced operation in 1963 and records extend to the present time. The gauging station measures water level and this can be converted to flow using a rating curve. There are 10 rating curves that cover the period 1963 to the present. The recorded water levels associated with each storm modelled were entered into URBS, together with the appropriate rating curve in force at the time the storm occurred.

The record of G8150127 was examined and 26 storms selected for modelling. The storms have varying sizes, durations, months of occurrence during the wet season and cover the time both before and after construction of the flood control weir.

The URBS model has been run for these storms and the best match between the calculated hydrograph and the observed hydrograph at G8150127 has been found by adjusting parameters as described in the URBS manual.

Table 3 summarises the results of the first set of runs using the URBS BASIC model and shows that:

- only the main channel routing parameter is used (α)
- The 3rd column is the Nash-Sutcliffe (NS) goodness of fit value calculated by the URBS program. (NS values an range from – infinity to 1 and values closest to 1 indicate a good match between the observed and calculated hydrographs)
- Shaded lines represent storms where insufficient goodness of fit was obtained to use in the averaging calculations. (NS value of 0.85 was adopted as the cut-off)
- The rank in the 5th column is used in the averaging calculations to weight the average toward larger events. (It is not the same as ranks assigned in flood frequency analysis.) Events with NS less than 0.85 were not assigned a rank. The weighted average values were:

$$\alpha = \frac{\sum(\alpha_i \times NS_i \times (23 - m_i))}{\sum NS_i \times (23 - m_i)}$$

where NS_i = Goodness of fit measure for i^{th} flood, m_i is the rank of the i^{th} flood and the summation is over the 22 floods with NS greater than 0.85

Table 3. Summary of calibration runs for URBS BASIC model

Event No.	Date of event	Goodness of fit at G8150127 (Nash Sutcliffe)	Size of event (Q_{peak} m ³ /sec) at G8150127	Rank of event (m_i)	Initial loss (IL (mm))	Continuing loss rate (CL mm/hr)	URBS parameter α
1	5-02-69	0.896	42.6	20	160	8.0	1.6
2	1-03-72	0.891	35.5	22	40.0	6.0	2.6
3	25-12-74	0.910	120	2	15.0	8.0	3.3
4	16-03-77	0.941	104	4	20.0	8.0	2.8
5	21-01-80	0.931	71.5	9	20.0	6.0	2.5
6	22-01-81	0.931	89.5	6	20.0	9.0	2.3
7	22-01-82	0.835	82.7		40.0	2.0	2.8
8	10-03-83	0.921	87.5	7	50.0	6.0	2.0
9	18-02-84	0.897	45.2	18	12.5	6.5	2.1
10	13-04-85	0.577	43.4		50	12	2.6
11	10-12-90	0.895	44.5	19	20	8	1.9
12	5-01-91	0.877	107	3	45	6	1.6
13	28-12-93	0.893	62.7	13	10	8.5	1.7
14	1-03-95	0.843	54.1		20	1	1.4
15	23-12-96	0.893	65.5	12	10	8.5	1.7
16	28-12-96	0.983	56.3	13	10	12	1.4
17	3-01-97	0.980	100.4	5	45	6	1.6
18	1-03-97	0.885	76.5	8	0	3	1.5
19	21-01-98	0.884	68.3	11	0	20	1.6

Event No.	Date of event	Goodness of fit at G8150127 (Nash Sutcliffe)	Size of event (Q_{peak} m ³ /sec) at G8150127	Rank of event (m_i)	Initial loss (IL (mm))	Continuing loss rate (CL mm/hr)	URBS parameter α
20	12-03-98	0.982	45.4	17	0	6	1.6
21	8-04-99	0.949	49.1	16	15	1	1.8
22	16-11-02	0.860	41.2	21	10	45	1.7
23	24-01-06	0.905	57.7	14	10	6.5	1.4
24	6-04-10	0.700	56.2		35	5	1.2
25	15-02-11	0.904	157	1	0	0.0	1.4
26	13-01-03	0.941	70.6	10	10	25	1.4

The resulting weighed average α value was 1.95 and the average NS value over the twenty two runs used to calculate it was 0.945.

These twenty two storms were then re-run with α fixed at 1.95 and the same losses as tabulated above. The average NS value was 0.819. The average error in peak discharge was -0.7% and there were nine overestimates and 13 underestimates of peak. For the top ten events the average peak error was 5.3% and ranged from -9.6% for the flood of February 2011 to +41.5% for 25 December 1974.

The weighting procedure is taken to give reasonable results.

Similar URBS model runs were carried out for the URBS SPLIT model with routing as a function of stream length L, as modified by urbanisation index U where appropriate. A third set of runs were carried out with the URBS SPLIT model and routing as a function of stream length (L) main channel slope (S_c) and contributing area slope (CS) and modified by the index of urbanisation (U) where appropriate. From the comparison in Table 4, the URBS BASIC model was adopted for design runs.

■ Table 4. Comparison of results of three URBS model types used.

Model type	Average NS for individual calibrated storms	Overall NS for calibrated storms using weighted average parameters	Average peak error for top ten storms using weighted average parameters
URBS BASIC	0.945	0.819	+5.3%
URBS SPLIT routing as a function of stream length	0.964	0.784	-5.6%
URBS SPLIT routing as function of stream length and slopes	0.929	0.808	-7.9%

A further advantage of adopting the URBS BASIC model (with routing as a function of stream length only) is that a relationship exists between URBS α and the routing parameter K_C of the well-known RORB runoff routing model, which had previously been

applied to Rapid Creek (Connell Wagner, 2004, Sinclair Knight Merz, 2011) and used to develop regional relationships (Pearse, Jordan and Collins, 2002).

The relationship between URBS α and RORB K_C (Carroll, 2009) is:

$$K_C = \alpha \times d_{av}$$

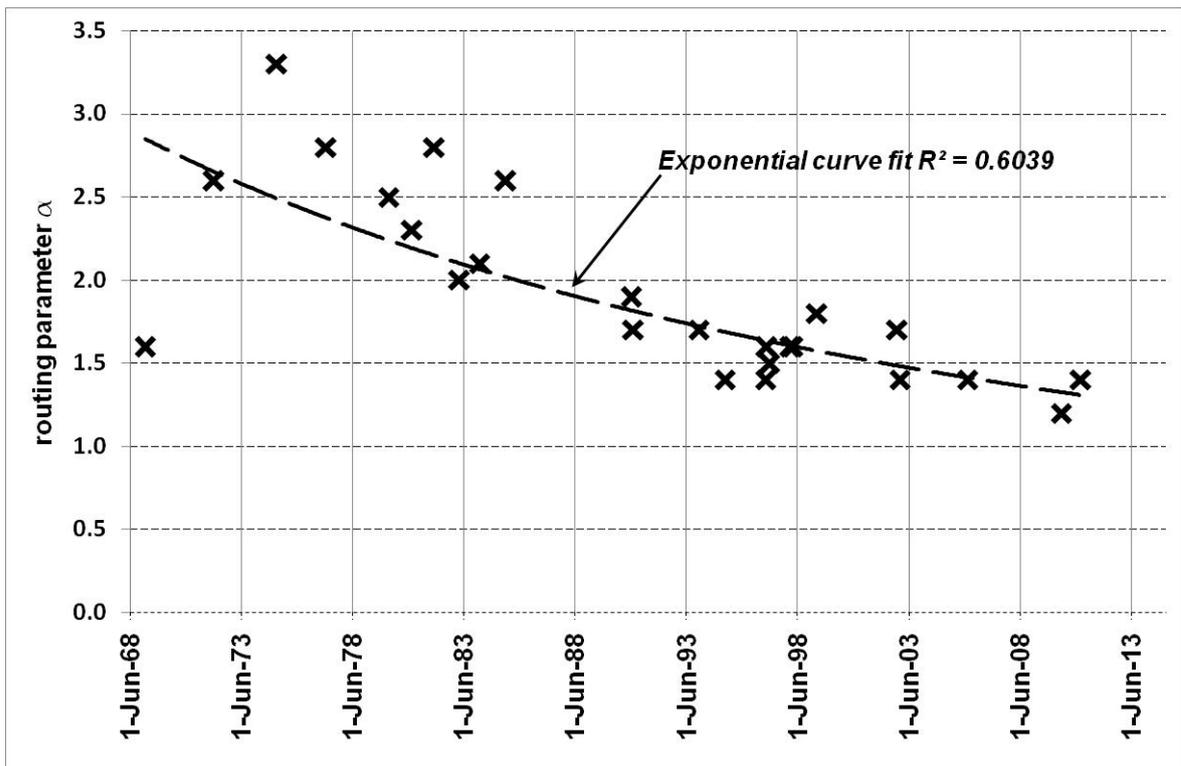
where d_{av} is the stream length from the centroid of the catchment to its outlet.

For Rapid Creek, the K_C value equivalent to an α value of 1.95 as above is $K_C=13.1$.

Figure 4 shows how the 'best fit' routing parameter α , for the URBS BASIC runs and the selected storms, varies over time for each of the storms modelled. Although there is some scatter as expected, there is a clear trend for α to decrease in time.

This is taken to be as a result of the impact of urbanisation of parts of the catchment area. As more development occurs, runoff occurs more quickly from the increased extent of paved surfaces and hydrographs are 'peakier'. The volume of temporary storage seen by flood waves moving through these urbanised areas also reduces. Therefore a smaller routing parameter is required to produce a match.

Figure 4. Variation of URBS BASIC routing parameter α with time



The URBS BASIC model with routing as a function of stream length and an α value of 1.12 (corresponding to $K_C=7.5$) was adopted, consistent with the increasing level of development in the catchment.

Design runs were carried out using the URBS BASIC model with $\alpha = 1.12$ to calculate hydrographs at G8150127, the outfall to the sea and at the outflow from each model sub-area downstream of the flood control weir for input to an hydraulic model.

The calculated design flow peaks at G8150127 are shown in Table 5. The losses from rainfall were adjusted to produce design peak discharges similar to those indicated by flood frequency analysis.

■ **Table 5. Comparison of peak discharges from design runs and flood frequency analysis**

AEP (Annual exceedance probability)	ARI (years)	Initial loss (mm)	Continuing loss rate (mm/hr)	Peak discharge calculated from URBS design runs (m3/sec)	Peak discharge adopted from flood frequency analysis (m3/sec)
0.200	5	27.5	3.0	67.5	68
0.100	10	27.5	2.0	88.4	88
0.050	20	21.0	2.0	108	108
0.020	50	19.0	2.0	134	134
0.010	100	17.5	2.0	156	155
0.005	200	16.0	1.5	177	176
0.002	500	15.0	1.5	205	204

The resulting hydrographs were used as inputs to hydraulic model runs for Q20, Q50, Q100, Q200 and Q500.

Probable Maximum Flood

Probable maximum floods (PMFs) were derived by running the calibrated URBS BASIC model with calculated probable maximum precipitation (PMP).

The losses adopted were the same as for the Q500 design flood viz; initial loss = 15 mm and continuing loss rate = 1.5 mm per hour.

The critical durations for the Rapid Creek catchment are generally less than 6 hours and the PMPs for storm durations of 15 minutes to 6 hours were calculated using the Generalised Short Duration Method – GSDM – (Commonwealth Bureau of Meteorology, 2003 (A)).

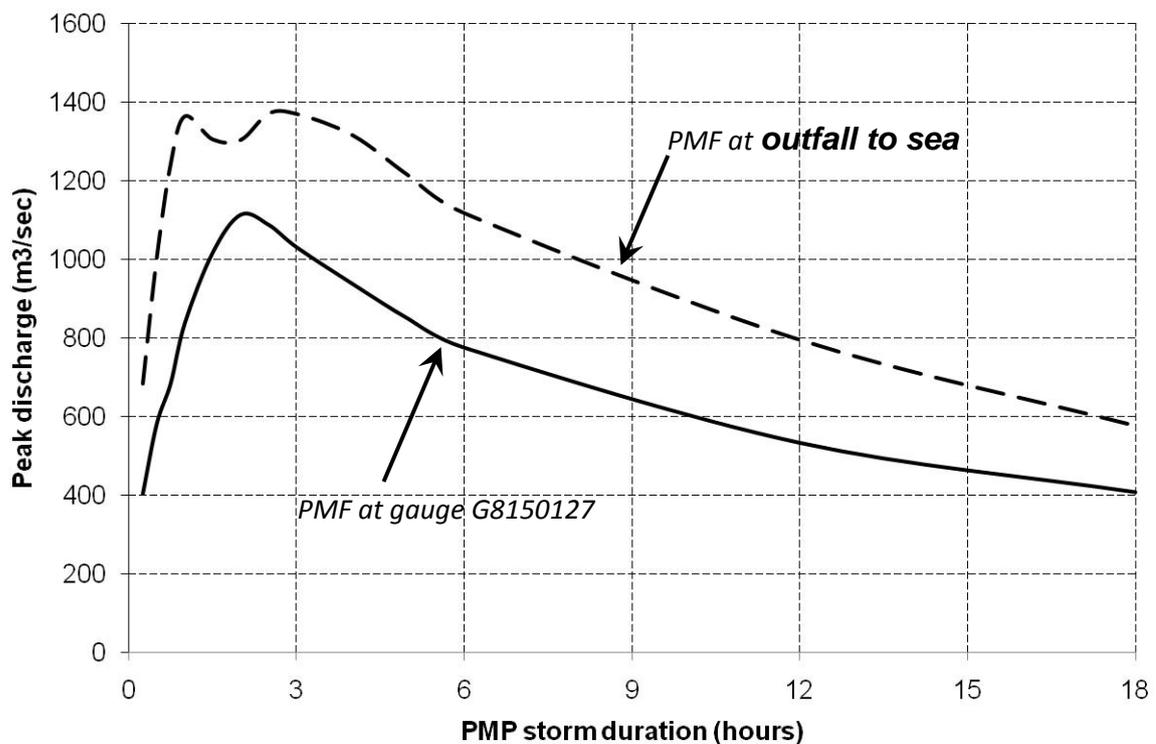
Peak discharges for the probable maximum floods calculated by URBS runs using the calculated PMPs are shown in Table 6 and Figure 5. These show that the critical duration for the PMF is 2 hours for G8150127 and 2.5 hours for the sea outfall. (PMF for longer durations in Figure 5 were interpolated between GSDM and the Revised Generalised Tropical Storm Method – GTSMr – (Commonwealth Bureau of Meteorology 2003 (B)) which can be used to calculate PMP for durations of 24 to 120 hours.)

■ **Table 6. Calculated PMFs**

Duration	Probable maximum precipitation over 28.7 sq km Rapid Creek catchment (mm)	Calculated peak discharge (m3/sec) at	
		G8150127	Sea outfall
15 min	190	400	680
30 min	280	580	1,000
45 min	360	690	1,240

Duration	Probable maximum precipitation over 28.7 sq km Rapid Creek catchment (mm)	Calculated peak discharge (m ³ /sec) at	
		G8150127	Sea outfall
1.0 hour	430	840	1,360
1.5 hour	490	1,020	1,300
2.0 hour	550	1,110	1,300
2.5 hour	580	1,090	1,370
3.0 hour	610	1,030	1,370
4.0 hour	680	940	1,320
5.0 hour	740	850	1,210
6.0 hour	780	780	1,120

■ Figure 5. Calculated PMF peak discharges



Hydraulic Study

A hydraulic model of Rapid Creek was developed in the hydrodynamic modelling package TUFLOW. The TUFLOW model is a DOS-based program with a GIS based interface and is useful for simulating depth-averaged 2D (two dimensional) and 1D (one dimensional) free surface flows. It has capability of dynamically linking 1D networks with 2D model domains and has the ability to model 1D culvert and bridge structures within the 2D domains. The Rapid Creek model was set up and run using TUFLOW version 2011-09-AF-w32.

Model development

Model extent

The extent of the TUFLOW model was determined in order to simulate flood behaviour of the Rapid Creek main channel and floodplain from immediately downstream of the flood control weir to the outlet of the creek into Beagle Gulf. The extent of the model covers a 5 km reach of Rapid Creek that includes the main channel crossings at Henry Wrigley Drive, McMillans Road, and Trower Road. The model also includes the constructed open channel that enters Rapid Creek from the Charles Darwin University campus. The extent of the Rapid Creek TUFLOW model is shown in Figure 6.

Model structure

A 2.8 kilometre length of the creek from the flood control weir to Trower Road was represented in the model as a 1D channel. The 1D channel was defined from each top of bank using 22 available surveyed cross sections. The 1D channel and hydraulic structures were modelled as 1D elements nested within the 2D model domain. The 2D domain was defined at a 5 metre grid spacing.

The extents of the TUFLOW models 1D and 2D domains are shown in Figure 6.

The benefit of a 1D/2D model of Rapid Creek is that it enables the creek channel to be more accurately represented in the model. Modelling the creek channel in 2D can result in a poor representation of the channel. A finer 2D cell size can be used to improve the channel's representation in 2D; however this would result in far more computing requirements and model run times. The main channel was represented in 1D and the floodplain in 2D.

Model terrain

Cross sections of the creek to define the 1D channel were sourced from the following:

- Field survey of the creek channel between the flood control weir and Henry Wrigley Drive
- Field survey of the creek channel between McMillans Road and the gauging station (G8150127)

Ground surface elevations of the TUFLOW model's 2D domain were defined using the following data sets:

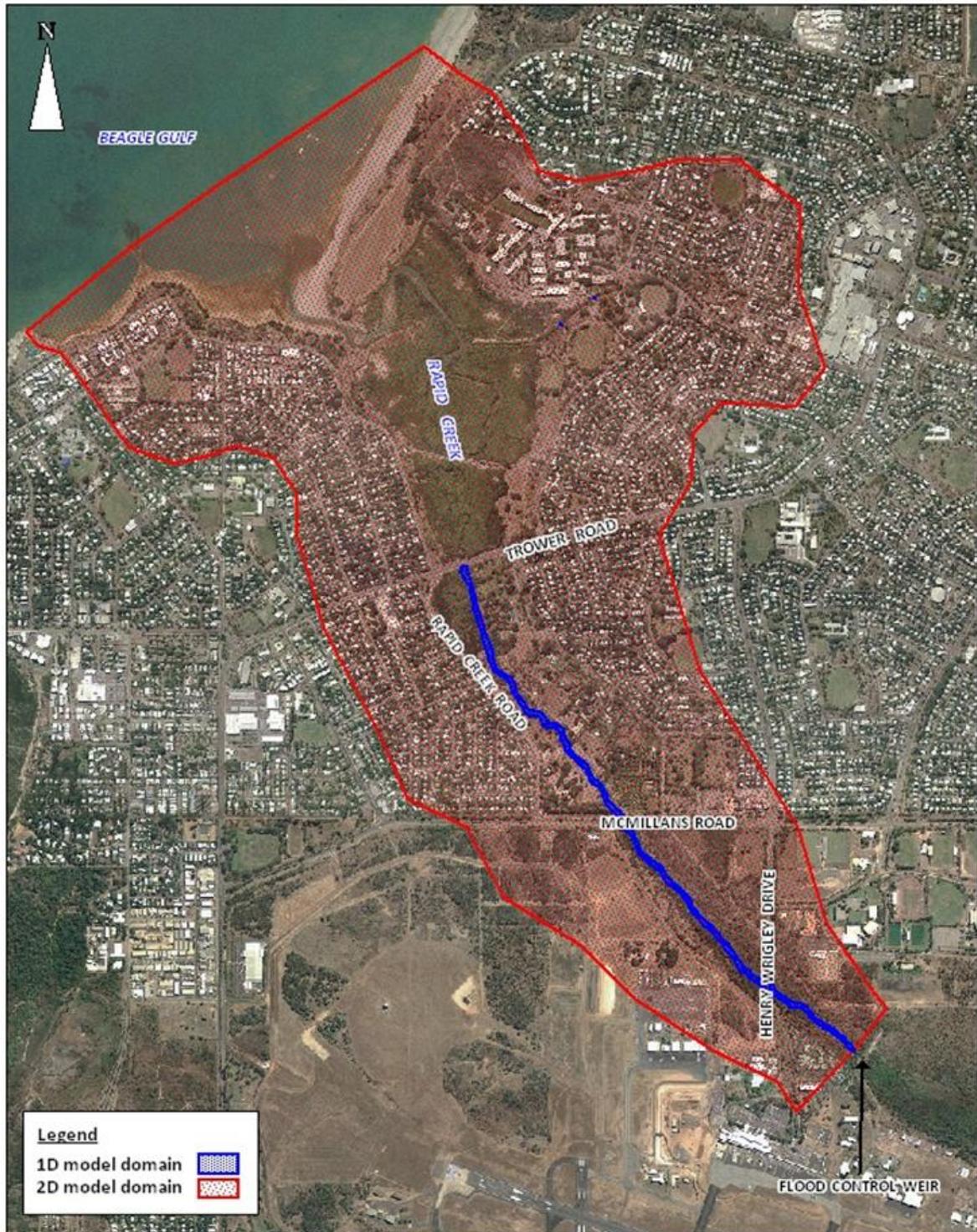
- Digital Elevation Model (DEM) developed from photogrammetry (2011) of the study area. The DEM was provided by the Department of Lands and Planning
- Bathymetric survey of the estuary downstream of Trower Road collected in November 2012

Model calibration

The 1D/2D TUFLOW model was calibrated revisited to water levels recorded at the gauging station (G8150127) and the surveyed flood marks from the February 2011

flood event. Using the calibrated URBS model hydrographs as inflows to the TUFLOW model, the model was calibrated by adjusting the Manning's 'n' roughness values of the 1D channel and 2D domain until a satisfactory match to the recorded peak flood levels was achieved.

■ Figure 6. Rapid Creek TUFLOW model extent



Selected hydraulic roughness values

Manning's 'n' roughness values of 0.10 to 0.12 were applied to the 1D channel for the model calibration. The land use categories within the model's 2D domain and their Manning's 'n' roughness values adopted for the model calibration are shown in Table 7.

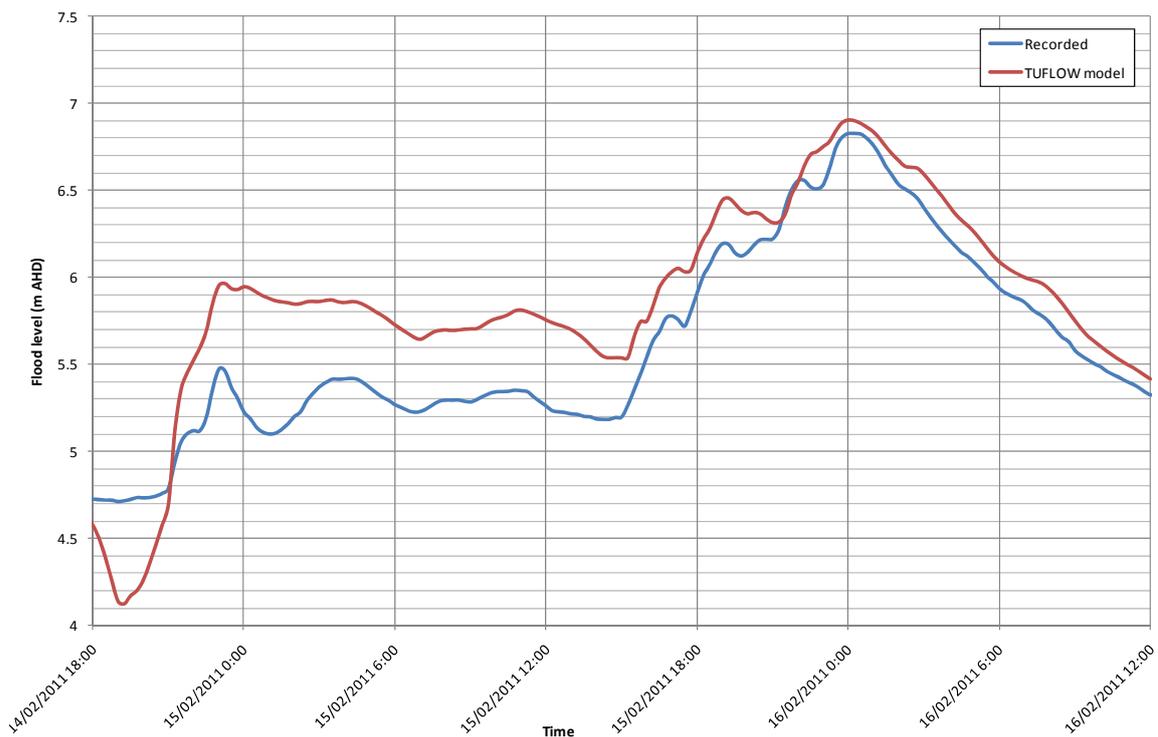
Table 7. Land use categories and adopted Manning's 'n' roughness values in 2D model domain

Land use category	Manning's 'n'
Road corridors	0.035
Residential lots	0.500
Open space with scattered vegetation	0.045
Rural lots	0.070
Creek channel through mangroves	0.100
Estuary channel / open water	0.030
Mangroves	0.300
Riparian bank vegetation	0.150
Mango plantation	0.090
University campus	0.100

Calibration to gauge G8150127 records

The flood event of 15th February 2011 was used for calibration as it is the rank 1 event and also has the most detailed information on flood levels. The gauge recorded a peak flood level of 6.83 m AHD at midnight on 16th February. A graph showing the recorded flood levels over the 15th and 16th of February compared with flood levels from the calibrated 1D/2D TUFLOW model is shown in Figure 7.

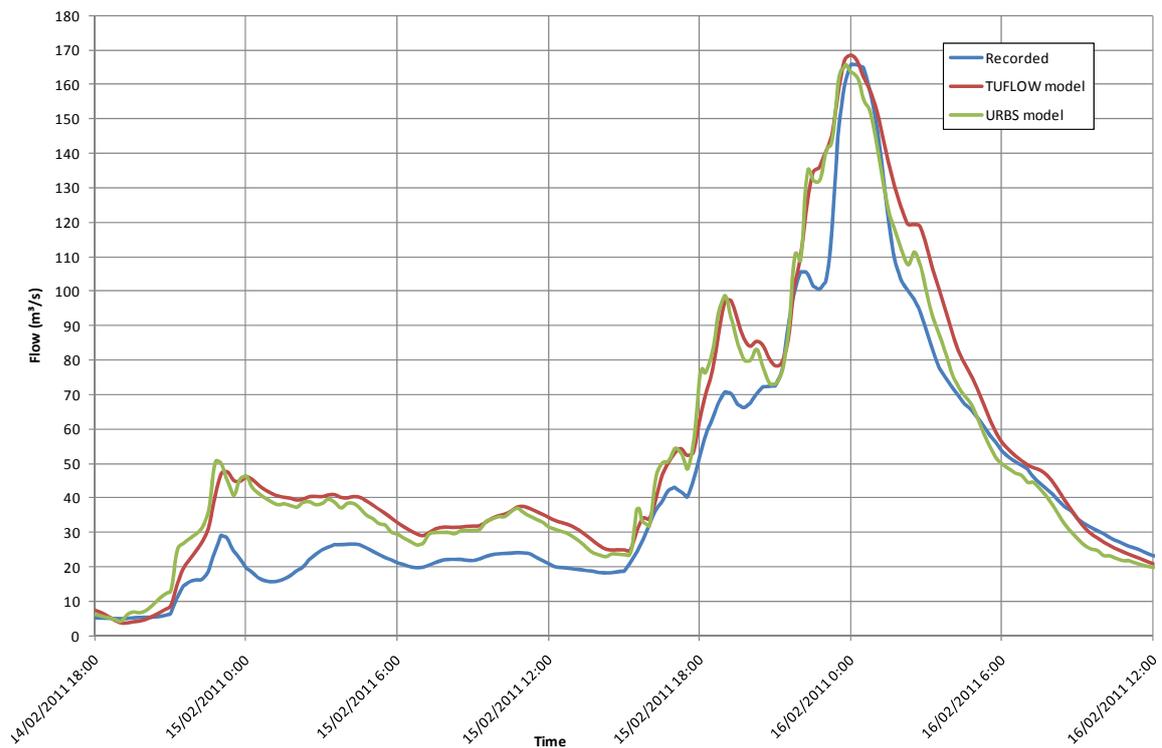
Figure 7. Recorded and modelled flood levels at gauge G8150127



The 1D/2D TUFLOW model was able to reproduce the recorded peak flood level at the gauge to within 70 mm, producing a peak flood level of 6.90 m AHD at midnight on 16 February 2011. There is a poorer fit to recorded levels over the 24 hours prior to the peak of the flood. This is considered to be the result of the URBS hydrologic model flows compared to the actual gauged flows.

Flows at the gauge extracted from the TUFLOW model compared with the recorded gauged flows and the URBS hydrologic model flows are shown in Figure 8. The peak flow at the gauge from the TUFLOW model is 169 m³/s compared to the recorded and URBS-modelled peak flow of 166 m³/s.

Figure 8. Recorded and modelled flows at gauge G8150127



Calibration to recorded flood marks

Various flood marks were noted during the flood of February 2011 and were surveyed after the flood by NRETAS. Nine flood marks were between Trower Road and McMillans Road, the majority adjacent to residential areas on the left overbank area of the creek. There is very good agreement between the modelled and surveyed levels with 8 of the 9 modelled peak flood levels being within 0.03 m (30 mm) of the surveyed flood marks. The modelled flood level at the ninth location is within 60 mm of the surveyed level.

Another nine flood marks were between McMillans Road and the flood control weir. Of these, six modelled levels show good agreement and were within 0.1 m of the recorded levels. Three modelled levels show a poorer fit and are lower than the recorded level by between 0.13 m and 0.27 m. The poorest fit is to the recorded level upstream of Henry Wrigley Drive. The poor fit could be the result of either:

- Blockage of the Henry Wrigley Drive culverts during the February 2011 event causing an increase in the recorded upstream flood level

- The URBS model underestimating the peak flow from the flood control weir, or
- Local turbulence

Model limitations

The 1D/2D TUFLOW model was calibrated and able to reproduce the majority of surveyed peak flood levels from the February 2011 event to within 0.10 m. The model was considered satisfactorily calibrated to this event and appropriate for modelling the design storm event scenarios of interest to this study.

The model was not validated against more than one historical event due to the lack of historical flood data. Validation of the model to another historical event could further improve confidence in the model's ability to simulate flood behaviour of Rapid Creek.

Design flood modelling

Design events

The calibrated 1D/2D TUFLOW model was used to simulate the following design flood scenarios:

- Q100 with Highest Astronomical Tide (HAT) + 0.8 m sea level rise (4.16 m)
- Probable Maximum Flood (PMF) with HAT + 0.8 m sea level rise (4.16 m)

A number of other scenarios were also run using an earlier version of the calibrated TUFLOW model. In this earlier version of the model, the entire creek was represented in 2D and the bathymetric data of the estuary collected in October/November 2012 was not taken into account.

Design inflow hydrographs for the TUFLOW model were extracted from the URBS hydrologic model and a static downstream water level boundary was applied. The model was run for multiple duration storm events so that critical flood heights, depths and velocities were obtained. Design durations modelled typically ranged from the 45 minute storm up to the 6 hour storm.

Flood modelling results

The results of maximum flood height, depth, and velocity-depth product were determined for each scenario and used as inputs to floodplain mapping.

Flow and stage hydrographs at selected locations along the creek were also output from the model in order to confirm the critical duration was captured at each location.

The Q100 hydrographs were indicative of the design recurrence intervals modelled and showed the following key characteristics of design flood behaviour in Rapid Creek:

- At Henry Wrigley Drive the critical design flood levels result from the 4.5 hour duration storm. This is due to the flood control weir's impact of attenuating peak flows from the upstream catchment
- At McMillans Road and G8150127, critical flooding is from the 1 hour duration storm due to inflows from the fast responding sub catchments between McMillans

Road and the flood control weir. This is typically followed by a second flood peak of a smaller magnitude

- Critical flooding between Trower Road and the ocean outlet is from the 2 hour to 4.5 hour duration storm events. Flood levels over this length of the creek are controlled by a constriction at the outlet and the amount of floodplain storage downstream of Trower Road

Impact of sea level rises

Sea levels that formed the downstream boundary conditions for TUFLOW model runs were either:

- Current mean sea level
- Current Highest Astronomical Tide (HAT)
- Current mean sea level plus 1% AEP (1 in 100 year) storm surge
- Year 2100 HAT (as current HAT plus 0.8 m)

The water surface profiles and the flood plain maps showed that the influence of downstream sea level on the extent of flooding is largely in the area downstream of Trower Road.

In some cases there are also minor differences in flood levels immediately upstream of Trower Road but in all cases the flood profiles are identical above chainage 3,500 m, which roughly corresponds to the location of the gauging station G8150127.

Floodplain mapping

The base data for the flood plain maps was the digital elevation model (DEM) supplied by the NT Government from 2011 photogrammetry of the study area.

The steps involved in producing the maps were:

- All ASCII datasets from the TUFLOW hydraulic model were converted to raster using ArcGIS
- To produce flood extents, the ArcGIS raster data were converted to polygons
- ArcGIS was used to produce flood water surface AHD contours at 0.25 m intervals
- To derive flood depths ArcGIS was used to classify depth values into the 0.0 to 0.5, 0.5 to 1.2, 1.2 to 2.0 and greater than 2.0 ranges. ArcGIS was then used to create regions corresponding to these depth ranges and then used to create a single dataset
- The floodway (the most hazardous portion of the flow) was defined by selecting for values of velocity \times depth larger than 1.0 and missing values converted to 'no data'. Similarly depths greater than 2.0 were selected and missing values set to 'no data'. These two sets were combined then raster data converted into regions meeting the hazardous criteria as a single dataset

The mapping of Q100 flood levels which were calculated using 1D channel representation, with actual bathymetry downstream of Trower Road, and 2D representation of the flood plain, can be taken as giving the best currently available estimates of the extent of flooding for land use planning.

Similarly mapping of the Probable Maximum Flood can be taken as the best currently available for emergency planning.

Conclusion

Investigations to determine the flood risk for Rapid Creek have been updated using:

- available recent photogrammetric and field survey data,
- a hydrologic study using the URBS model and which takes into account the construction of a flood control weir in 1985 and non-stationarity of the flood series as a result of development
- the most recent available hydrometric data including a new rank 1 flood which occurred on 15th and 16th of February 2011 during the formation of Cyclone Carlos
- flood frequency estimates based on the Generalised Extreme Value distribution
- new estimates of the probable maximum precipitation and probable maximum flood
- a 1D/2D TUFLOW hydraulic model of the creek downstream of gauging station G8150127 calibrated using the flood of February 2011
- a range of downstream sea levels including taking account of sea level rise resulting from climate change
- GIS-based digital flood mapping suitable for land use planning and emergency planning

The flood plain maps indicate the frequencies of flooding of properties in the suburbs of Millner and Rapid Creek and indicate more extensive flooding of properties on both sides of the creek during a probable maximum flood.

The results of this investigation can be used as a basis for estimating flood damages, for assessment of flood mitigation options and for land use planning and emergency planning.

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Since late 2012, the Water Resources Group of the former Department of Natural Resources, Environment, The Arts and Sports has been located in the Department of Land Resources Management.