Connell Wagner

Rapid Creek Flood Study
Revised Final Report

May 1999
Reference A798

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1. Introduction

1.1 Purpose of this report

Connell Wagner was commissioned in June 1998 by the Department of Lands, Planning and Environment to undertake a study into flooding of Rapid Creek, which drains the Darwin Airport and part of Darwin's northern suburbs.

The study aims to examine the flood impact that could result from current, proposed and likely future developments in the Rapid Creek catchment area. These could come from two sources:

> development within the airport grounds. A number of sites on the airport periphery have been cleared and built on in recent years. Further, management of the airport passed into the private sector in 1998 and more developments are likely as facilities are improved. It is understood that there are proposals for commercial development within the site.

> development within the "green" strip that includes Rapid Creek between McMillans Road and Trower Road. Between McMillans Road and Trower Road, Rapid Creek flows through a largely undeveloped corridor some 300 to 400 m wide and we understand there is pressure to re-zone this land to allow development.

This report describes the hydrologic and hydraulic investigation undertaken as part of this study. Results are presented for hydrologic modeling of the catchment above McMillans Road, design hydrographs are derived and water surface profile analysis is carried out to determine flood levels along the creek in the area of interest. Maps showing the likely extent of inundation for design floods are presented and maps showing likely flood hazard. An examination is also made of the likely impact of development.

1.2 Catchment description

Rapid Creek drains the Marrara Swamp which is located at the eastern end of the Darwin Airport. It flows generally north westerly to the sea and forms the boundary between the suburbs of Brinkin and Rapid Creek near where it discharges to the "Beagle Gulf" at the southern end of Casuarina Beach.

The catchment of Rapid Creek is shown in Figure 1. The catchment area is 19.4 sq km up to the stream gauging station downstream of McMillans Road (G8150127).

The following features influence the hydrology of Rapid Creek:

> The catchment area south of McMillans Road is not fully developed. However, large parts of the airport grounds have been cleared and constructed open drains serve the runways and other facilities, resulting in the speeding up of runoff to Rapid Creek from this area. Parts of the suburbs of Moil, Anula, Malak and Karama drain to Rapid Creek north of McMillans Road. These areas are fully built-up and are served by underground pipe drainage systems, which also result in quick runoff to the Creek. The suburb known as North Lakes, which is built around the Marrara Golf Course also drains to Rapid Creek, as does the Marrara Sporting Complex.

> The Marrara Swamp retains flows generated east of Amy Johnson Drive. The swamp is a large shallow depression which fills during prolonged wet season rain. The swamp covers some 15ha and is large enough to attenuate and delay the peak of flood waters generated in the upper catchment area before they enter Rapid Creek. The swamp is drained by two separate drainage lines, one on the north western and one on the south western side of the swamp. (See reaches 2 and 5 in Figure 3).

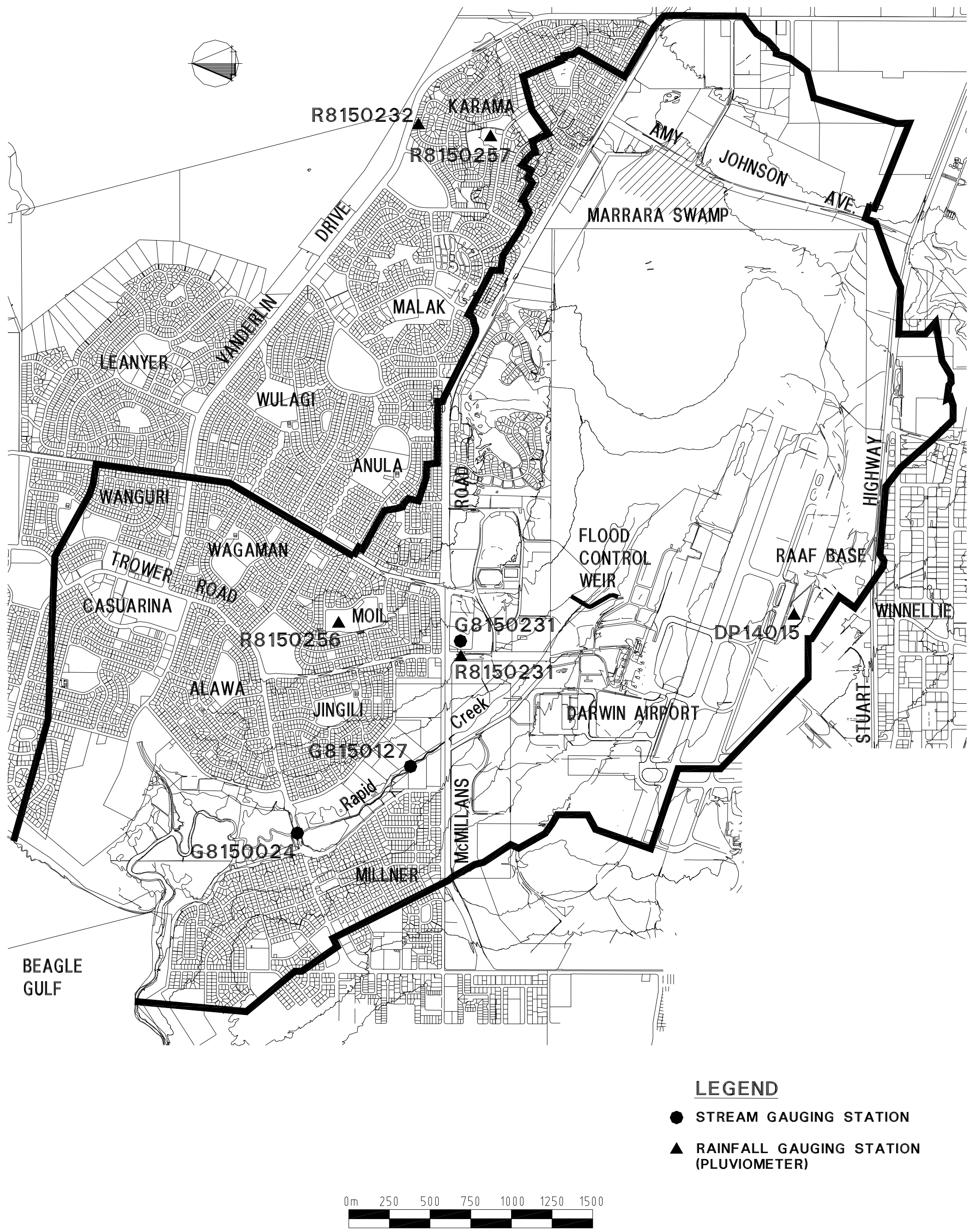
> A flood control weir was constructed in 1987 where these two drainage lines re-join to form Rapid Creek. The weir was modified in 1990. This also results in floodwaters being delayed and peak discharges diminished.

> North of McMillans Road the Creek drains the suburbs of Millner, Jingili, Rapid Creek and Alawa. These are also fully developed, but the Creek is contained within a "green" corridor which includes Freshwater Farm, the Darwin Water Gardens and parts of the Casuarina Coastal Reserve.

> North of Trower Road the Creek is tidally influenced and flood events will be exacerbated if their peaks coincide with high tides.

Further descriptions of Rapid Creek and its catchment can be found in the reports listed in Section 8.

Figure 1, RAPID CREEK CATCHMENT



2. Availability of hydrologic data

Hydrologic data was made available to this study by the Water Resources Division of the Department of Lands Planning and Environment and the Bureau of Meteorology as described in the following.

2.1 Water level and streamflow data

Water levels are measured at stream gauging stations. Where a relationship between water level and flow exists the water level records can be converted to flow records. (This requires discharge measurements during flood events or a theoretical relationship based on weir geometry.) This relationship is known as a "rating curve".

Water level is measured continuously in the Rapid Creek catchment at the locations shown in Table 1.

TABLE 1, STREAM GAUGING STATION LOCATIONS IN THE RAPID CREEK CATCHMENT

Gauging Station No.	Stream	Location	Start of Records	End of Records	Rating Curve (i.e. flow records available)
G8150024	Rapid Creek	Trower Road	17-02-1981	ONGOING	NO
G8150127	Rapid Creek	downstream of McMillans Road	05-11-1963	ONGOING	YES
G8150231	Moil main drain	Henry Wrigley Drive	15-10-1984	ONGOING	YES

No rating curve has been derived for the Trower Road gauging station because it is in the tidally influenced zone of Rapid Creek.

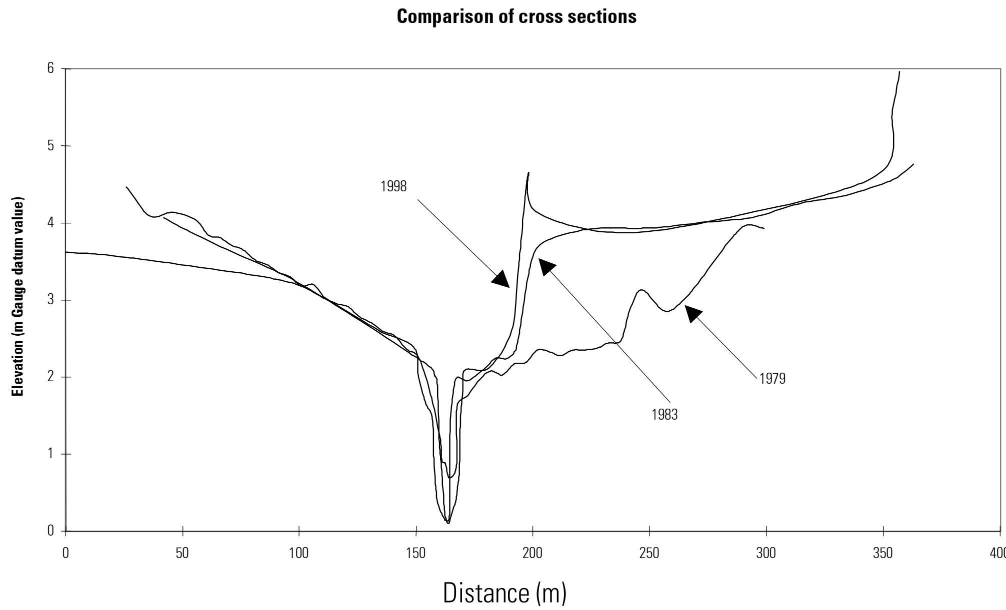
Water Resources Division advises that a number of rating curves have been used at gauging station G8150127 over time. The station was relocated in 1968 and the early rating curves apply before that time. The current rating has been applied from 1 September 1978 to the present. During this time there have been some changes to the creek. Figure 2 shows cross sections through the line of the flow recorder as surveyed in 1979, 1983 and in June 1998 for this study. Note that in examining Figure 2 it should be borne in mind that a reinforced concrete weir just downstream of this cross section is controlling low and medium flows

It appears that the creek is being encroached upon by earthworks on the right bank, and some leveling on the left bank has occurred some time after 1983. It is understood that filling of lot 4589 took place in 1980. It is difficult to say what the effect of the changes since 1983 has been. It is also uncertain what the effect of any changes downstream would have had on the high flow part of this rating curve.

The relationship between elevation and the cross sectional areas of flow are similar in 1983 and 1998 and it is not unreasonable to assume that the rating applicable in 1983 can be applied up to 1998. However, Water Resources Division has indicated that they will attempt to check the high flow rating next wet season depending on flows and availability of personnel to carry out flood gauging.

The flow data provided by Water Resources Division is based on the adoption of the current rating for all flows after 1 September 1979.

Figure 2, COMPARISON OF CROSS SECTION AREAS AT FLOW RECORDER SECTION OF G8150127 FOR 1979, 1983 AND PRESENT CONDITIONS



2.2 Rainfall data

For flood studies in small catchments, pluviometric rainfall data is required.

Pluviometers measure rainfall continuously and allow rainfalls for flood-producing storms to be input to hydrologic models in a series of small time steps. A time increment of 0.5 hour was adopted for this study.

Pluviometers are located as shown in Table 2.

TABLE 2, PLUVIOMETRIC RAINFALL RECORDERS IN THE RAPID CREEK CATCHMENT

Station Number	Location	Operated by	Start of Records	Record processed up to
DP14015	Airport	Bureau of Meteorology	Records obtained from 01/01/91	01/07/97
R8150231	Henry Wrigley Drive	Water Resources Division	15/10/84	06/05/98
R8150256	Moil School	Water Resources Division	17/01/85	06/05/98
R8150257	Karama School	Water Resources Division	05/03/84	06/05/98
R8150232	Karama lot 4395 Vanderlin Drive	Water Resources Division	03/02/84	06/05/98

Not all of these records were available for all of the flood events chosen for hydrologic modeling (see Section 3) for various reasons including:

> - instrument failure
> - record not yet processed (in the case of recent flood events).

The records for R8150232 were not used because they are no closer, nor are likely to be any more representative of catchment rainfall than, R8150257.

3. Method of hydrologic analysis

3.1 RORB model

The RORB model was chosen for hydrologic analysis.

RORB is a general purpose runoff routing computer model which simulates floods by calculating runoff in a network of user-designated sub-areas and summing from the top of the catchment downwards in sequence. A full description of RORB and its capabilities can be found in the user manual (ACADS, 1983.)

A model was established for application to the Rapid Creek catchment above gauging station G8150127. The catchment was divided into 11 sub-areas as shown in Figure 3.

For each of these sub-areas, the computer model calculates runoff from rainfall and "losses" and this forms an input to the stream network at the centroid of each sub-area.

Other features of the model established for this study included the following:

> A special storage was placed to represent Marrara Swamp at the outlet from sub-area A. (Refer Figure 3). Spill beyond the swamp during large floods was assumed to occur over a 50 m wide path and the "broad-crested weir formula" was applied.

> The flow spilling out of Marrara Swamp was assumed to split 50-50 between the north western and south western drainage lines which lead away from the swamp.

> A special storage was used to represent the flood control weir. Discharge from this weir was calculated for the 1990 as-constructed profile (refer PAWA, 1991) using the broad crested weir formula. A relationship between discharge and the volume of floodwaters temporarily stored behind the weir during floods, was estimated from available topographic mapping.

The central stepped portion of this weir is some 25 m long and has a cease-to-flow level of 12.5 m AHD. The side walls total more than 450 m in length and are uneven because of erosion, settlement etc, but have an average elevation of about 16 m. The flow capacity of the central portion is some 100 m^3/s. For the long side walls a large flow increase can occur for only a small increase in depth. For example, at 16.5 m, the discharge is approximately 350 m^3/s. For the purposes of calculating a probable maximum flood (see Section 4.5), the rating curve was extended above 17 m by extrapolating the weir flow relationship.

> RORB allows for channel reaches to be natural channels, unlined constructed channels or lined constructed channels. An appropriate mix of these was used to reflect the network of unlined open drains in the airport, underground pipe drains in the built-up areas and the "natural " condition of Rapid Creek.

> Sub-area "I" is the catchment above gauging station G8150231 (Moil main drain at Henry Wrigley Drive) and is the southwestern corner of the surburb of Moil. It is fully built up residential land and is served by underground piped drains that drain south towards McMillans Road. It is possible for model-calculated hydrographs to be printed out for this location for comparison with gauged flows. It is also possible to vary model parameters to obtain a match between calculated and observed hydrographs here and these can be varied independently of the parameters employed in the remainder of the model.

However, the objective of the present study is to determine flood impacts in Rapid Creek downstream of the airport. Sub-area "I" represents only 1 sq km or 5 percent of the total catchment area. Therefore, there is unlikely to be much value in varying model parameters within the sub-area. It is shown in Section 4 that adequate comparisons have been derived by comparing recorded and calculated hydrographs at G8150127 and adjusting parameters that apply to the whole model.

> At McMillans Road, the drainage systems serving sub-area "I" cross beneath the road in large diameter pipes. However, during major storms, peak flows may exceed the capacity of these pipes, but the excess flows may not accumulate to a level where they will cross McMillans Road. Therefore, a model "reach" was inserted to receive flows that are in excess of the capacity of the main underground pipe drains crossing McMillans Road., This reach takes the flows westward to Rapid Creek at the Kimmorley Bridge, rather than southward across McMillans Road to Rapid Creek within the airport grounds. The sum of the capacities of pipes under McMillans Road in this area is such that no excess flow would follow this route for the storms modeled in calibration and verification (see Sections 3.2 and 3.3). This would only occur for the 1 in 100 year or larger floods.

3.2 Model calibration

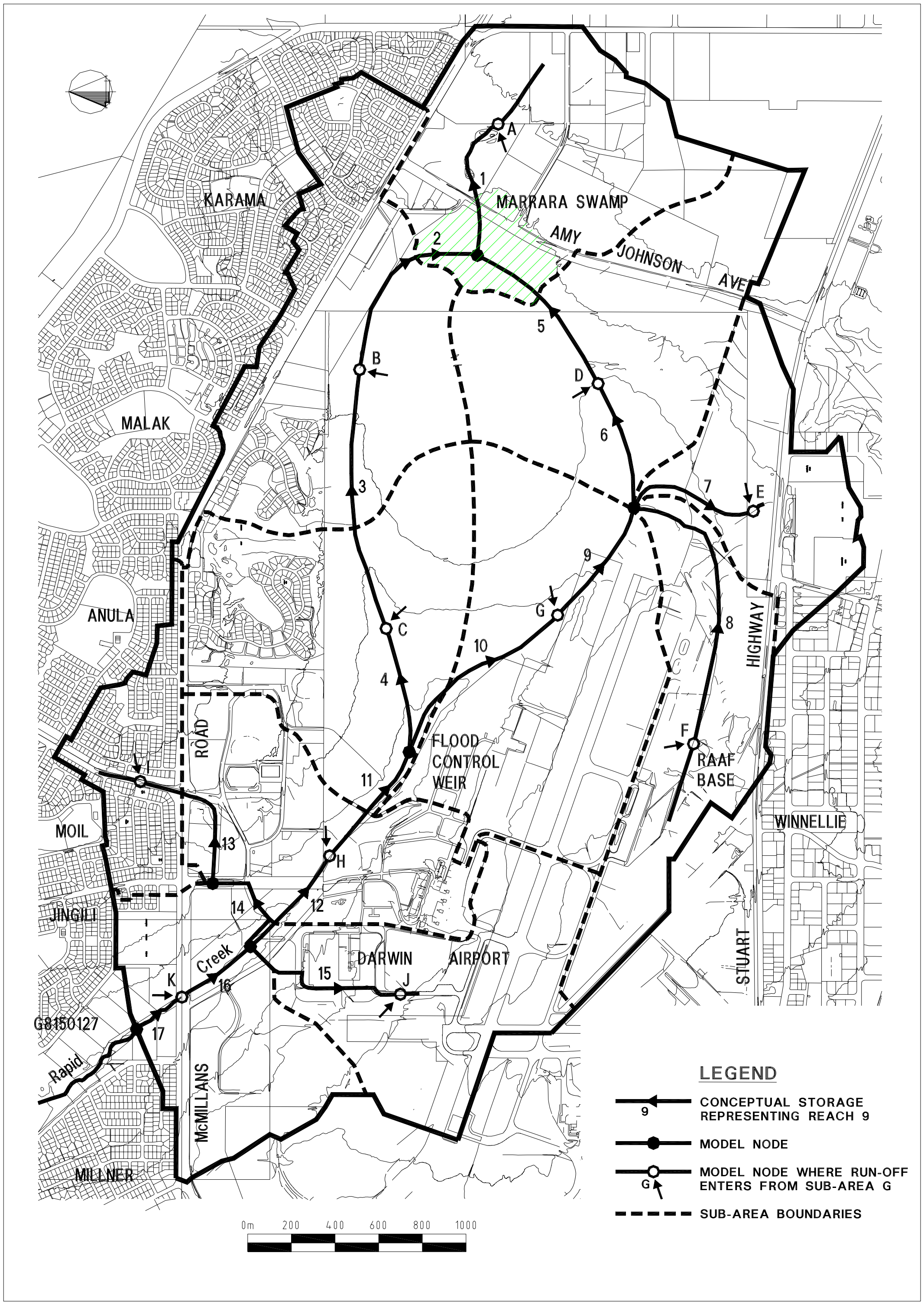
The RORB User Manual recommends that the established model be calibrated by adjusting the model parameters such that a good comparison is obtained between calculated and observed discharge hydrographs for selected storm events. The calibrated model should then be independently tested on further flood events. The streamflow record for G8150127 (Rapid Creek downstream of McMillans Road) for November 1963 to May 1998 was obtained from Water Resources Division of the Department of Lands, Planning and Environment. The record was examined and a list was compiled of 27 occurrences of peak discharges in excess of 40 m^3/sec. The maximum peak discharge was 115 m^3/sec on 25 December 1974 (during Cyclone Tracy). That is, 27 events had peak discharges larger than one-third of the largest event on record.

The construction of the flood control weir in the late 1980s introduces a "discontinuity" in the streamflow record and calibration of the model to earlier floods might give erroneous results for later events. Therefore a second list was prepared for floods after 1990. On eight occasions after September 1990 the peak discharge exceeded 60 m^3/sec (or 50% of the maximum flood on record). This was considered a good basis for calibrating the model and five of these were selected (arbitrarily) for "fitting" of the model parameters, with the remaining three being used for independent testing as shown in Table 3.

TABLE 3, FLOOD EVENTS USED FOR CALIBRATION OF THE HYDROLOGIC MODEL

Date of flood	Peak discharge at G8150127 m^3/sec	Used for "fit" or "test" model runs
5 January 1991	112	FIT
6 January 1991	71	TEST
23 December 1993	60	FIT
23 December 1996	63	TEST
3 January 1997	101	FIT
1 March 1997	75	FIT
8 January 1998	61	FIT
20 January 1998	66	TEST

Figure 3, RORB MODEL LAYOUT FOR THE RAPID CREEK CATCHMENT



The model parameters which can be adjusted are the storage delay time from the centroid of the catchment to the outlet (Kc) and the exponent in the storage-discharge relationship for temporary storage in model reaches (m). In addition, the losses from rainfall can be varied to help produce a good comparison between observed and calculated hydrographs. Two loss models are available in RORB version 3, the initial loss-continuing loss model and the runoff coefficient model. Generally the best results were found using the runoff coefficient model and zero or a small value of initial loss (see Section 4).

3.3 RORB data files

For the first storm modeled (5 January 1991), base flow was separated from the hydrograph prior to analysis. This was done by the composite recession curve method of standard hydrologic texts. It was apparent that the resulting base flow was small compared to the hydrograph peak and for subsequent events, complete hydrographs were used. The storm of 5 January 1991 was also remodeled using the whole hydrograph.

In theory base flow separation should be used for all events because the RORB program adjusts the losses (the runoff coefficient or continuing loss rate) to force the hydrograph volumes to match during fit and test runs. However, this is probably only a problem where rainfall on the receding limb of the hydrograph produces stream rises that significantly increase the volume of runoff that is represented by the recorded hydrograph.

It is a requirement of the RORB model that total rainfalls for the duration of the storm which produces each flood have to be specified for each sub-area in the model. Sub-area rainfalls were calculated by summing the pluviometric rainfalls for the duration of the storm. For those events where all records were available, these were assigned to sub-areas as shown in Table 4, which also shows which pluviometer's temporal pattern was used for each sub-area.

TABLE 4, ASSIGNING OF TEMPORAL PATTERNS & CALCULATION OF SUB-AREA RAINFALLS

Sub-area	Temporal Pattern as for Pluviometer Number	WEIGHTING USED FOR SUMMING OF PLUVIOMETER RAINFALL TO DERIVE SUB-AREA RAINFALL FOR DURATION OF STORM			
		DP14015	R8150257	R8150231	R8150256
A	R8150257		1		
B	R8150257		1		
C	R8150231		0.5	0.5	
D	R8150257	0.5	0.5		
E	DP14015	1			
F	DP14015	1			
G	DP14015	1			
H	R8150231			1	
I	R8150256				1
J	R8150231	0.5			0.5
K	R8150231			1	

Where all data was not available minor adjustments were made to the values in Table 4.

An example data file is in Appendix A.

3.4 Flood frequency analysis

3.4.1 Observed annual floods

Whilst the RORB model gives the shape of the design hydrographs, Section 4.4 indicates that the flood frequency analysis should be used to determine the magnitude of design floods with average recurrence intervals of up to about 1 in 500 years. Therefore, an analysis of flood frequencies has been carried out based on the recorded streamflow at G8150127. This was done in accordance with the procedures in Australian Rainfall and Runoff (AR&R - IE Aust., 1987) Chapter 10 for an annual series of observed flood peaks.

The "water year "is defined as 1 September to 31 August and the maximum peak discharge at G1850127 for each of 35 years from 1963/64 to 1997/98 was extracted from the gauging station records.

It was found that in the 1967/68 water year no flow was recorded and in 1966/67 the peak discharge was only 1.09 m^3/s. This is not to say that no rain fell in these wet seasons but that the swamps upstream would have absorbed any runoff prior to it reaching the gauging station in these drought years. (During these years the El Nino effect produced prolonged drought over southern Australia.) These very low flows can distort the flood frequency analysis. These two years, and four other years of low flow were omitted in accordance with procedures in AR&R section 10.7.2.

A flood frequency analysis was then carried out and a Log Pearson type III statistical distribution was fitted, 5% and 95% confidence limits were calculated and at the upper end of the flood frequency curve an adjustment was made for "expected probability" to correct for small sample bias in accordance with procedures in Australian Rainfall and Runoff. The results are given in Section 4.3.

3.4.2 Adjusted annual floods

The annual series of peak discharges recorded at G8150127 is non-homogeneous in as much as the peaks of floods will have been attenuated after 1987 by the flood control weir. Flood frequency analysis requires a homogeneous series and therefore it is desirable to adjust peak discharges to form a homogeneous series.

Two optional methods for calculating design flood hydrographs have been considered:

> Adjust the peak discharges observed <u>before</u> 1987 to reflect conditions prevailing <u>after</u> the flood control weir was constructed. Fit a flood frequency curve to the series of 1963/64 to 1986/87 adjusted peak flows plus 1987/88 to 1997/88 observed peak flows. Extract design peak flows for the 1 in 20 year, 1 in 50 year and 1 in 100 year floods. Adjust RORB model parameters to simulate these peak flows and adopt the resulting design flood hydrographs.

> Adjust the peak discharges observed <u>after</u> 1987 to reflect conditions prevailing <u>before</u> the flood control weir was constructed. Fit a flood frequency curve to the series of 1963/64 to 1986/87 observed peak flows plus the 1987/88 to 1997/98 adjusted peak flows. Extract design peak flows for the 1 in 20 year, 1 in 50 year and 1 in 100 year floods. Adjust the parameters in the RORB model (that does not include the flood control weir) to simulate these peak flows. Use the same model parameters (in the RORB model that includes the flood control weir) to generate design flood hydrographs.

TABLE 5, ANNUAL SERIES OF PEAK DISCHARGES RECORDED AT G8150127

Rank	Date of flood	Peak discharge (m3/s)		
		As recorded	If weir existed pre-1987	If weir not constructed in 1987
col 1	Col 2	col 3	col 4	col 5
1	25-12-74	116	82.5	116
2	05-01-91	112	112	164
3	16-03-77	103	75.7	103
4	03-01-97	101	101	103
5	22-01-81	88.9	53.6	88.9
6	10-03-83	87.1	47.5	87.1
7	22-01-82	80.3	44.5	80.3
8	21-01-80	71.2	39.1	71.2
9	21-01-98	66.2	66.2	(20/01/98) 104
10	28-12-93	60.4	60.4	105
11	25-01-93	57.0	57.0	85.5
12	01-03-95	49.3	49.3	73.9
13	18-02-84	45.0	24.5	45.0
14	13-04-85	43.1	20.2	43.1
15	24-02-74	42.6	21.3	42.6
16	05-02-69	41.9	21.6	41.9
17	09-12-95	41.4	41.4	62.1
18	01-03-72	35.0	18.3	35.0
19	10-02-87	34.9	26.5	34.9
20	02-01-79	34.4	16.0	34.4
21	02-12-88	33.0	33.0	49.5
22	11-01-86	32.8	15.1	32.8
23	31-12-87	23.2	23.2	34.7
24	11-03-76	20.0	10.1	20.0
25	06-03-71	16.3	8.00	16.30
26	27-01-78	16.0	8.30	16.00
27	28-01-73	12.6	5.90	12.60
28	06-01-92	12.4	12.4	18.6
29	30-12-65	9.79	4.80	9.79
30	26-01-64	7.76	3.60	7.76
31	26-03-65	7.43	3.72	7.43
32	11-02-70	6.70	5.09	6.70
33	15-01-90	5.98	5.98	8.97
34	15-12-66	1.09	0.83	1.09
35	01-09-67	0.00	0.00	0.00

Option 1: flood frequency analysis for post-flood-control-weir conditions
In order to understand the effect of the flood control weir, simulation was carried out for the 8 floods which were used in the calibration and verification of the RORB model as if the weir had not been constructed. The results are shown in Table 6.

TABLE 6, PEAK DISCHARGES CALCULATED BY RORB MODEL FOR SELECTED FLOODS WITH AND WITHOUT THE FLOOD CONTROL WEIR

Date of flood	Peak q with weir	Peak q without weir	Reduction attributable to weir (%)
05 January 1991	106	164	35
08 January 1998	64.3	104	38
28 December 1993	60.8	105	42
3 January 1997	97.3	103	6
1 March 1997	73.2	75.4	3
6 January 1991	78.3	100	22
23 December 1996	41.4	49.9	16
20 January 1998	61.3	87.8	30

An attempt was made to explain the variation in the percentage reduction. It was not able to be correlated against either the size of flood or simple indices of the distribution of rainfall. After discussions with the Water Resources Group of the Department of Lands, Planning and Environment, a definition of the time base for the hydrographs was agreed and percentage reduction of the peak was correlated against (time base * peak discharge). A weak relationship was found with a correlation coefficient (R-squared) of 0.34.

A revised annual series was derived using the relationship as shown in the fourth column of Table 5.

The rank one flood reduces by 29 percent from 115 to 82.5 m^3/sec. Reductions for other floods vary up to about 50%. This is consistent with the expectation that large floods are going to be less affected by the weir than are small floods.

Option 2: flood frequency analysis for pre-flood-control-weir conditions

The second option is preferred in theory because:

> It requires adjustment of only 11 peak discharges compared to 24 for the first option.

> Of these 11, four had already been modeled in the fit and test runs and therefore the adjustment could be made using RORB. This gives more confidence than an arbitrary adjustment given the lack of any clear reason why the flood control weir attenuates some floods more than others. (Refer to the discussion concerning option 1.)

> The effect of the flood control weir may distort the flood frequency analysis such that a better fit might be expected for the series based on pre-weir conditions.

The revised series is presented in column 5 of Table 5 and the results are presented in Section 4. The peak flows calculated using this approach were adopted for design.

Table 6 shows that with the adjustment to remove the effects of the flood control weir, the flood peak occurring as a result of the storm of 08 January 1998 becomes more critical (larger) than that for the storm of 20 January 1998 and should therefore be used in the annual series. It is not known if this effect occurs in any other wet seasons on record without undertaking a detailed examination of all large storms in each wet season. The flood frequency derived from the annual series in the last column of table 5 may under-estimate peak discharges if this effect occurs in other years.

4. Results of hydrologic analysis

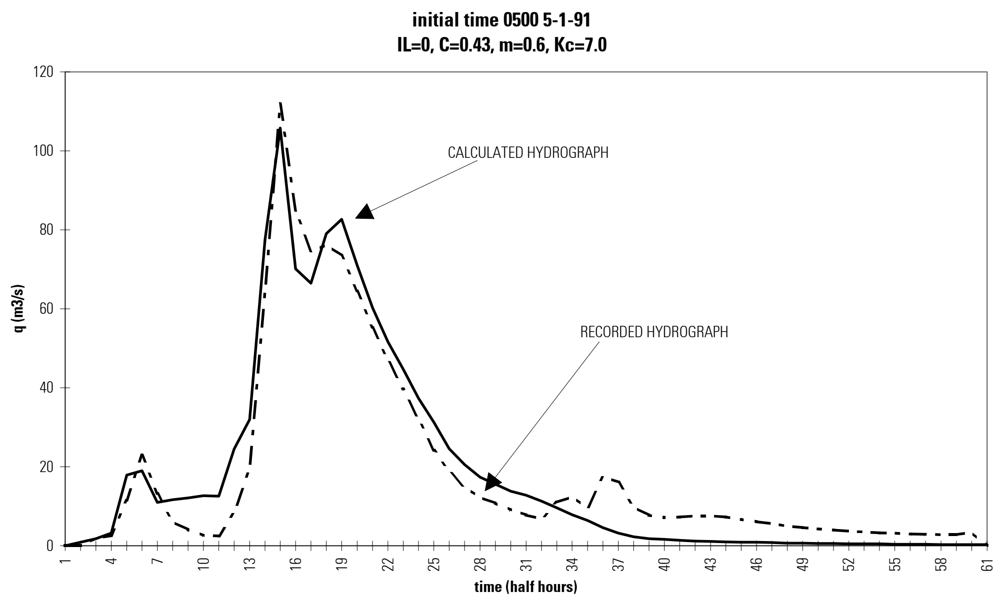
4.1 Model calibration (fit) runs

4.1.1 Flood of 5 January 1991

For the flood of 5 January 1991 pluviometric rainfall data were not available at G8150231 (Moil main drain and G8150256 (Moil School.) Nevertheless, a good fit (that is, a good comparison between observed and calculated hydrographs) was obtained, although in comparison to other floods the value of the storage-delay time parameter (Kc) was low.

A number of parameter combinations were tried and the results for the best run are shown in Figure 4.

Figure 4, BEST FIT RUN FOR FLOOD OF 5 JANUARY 1991



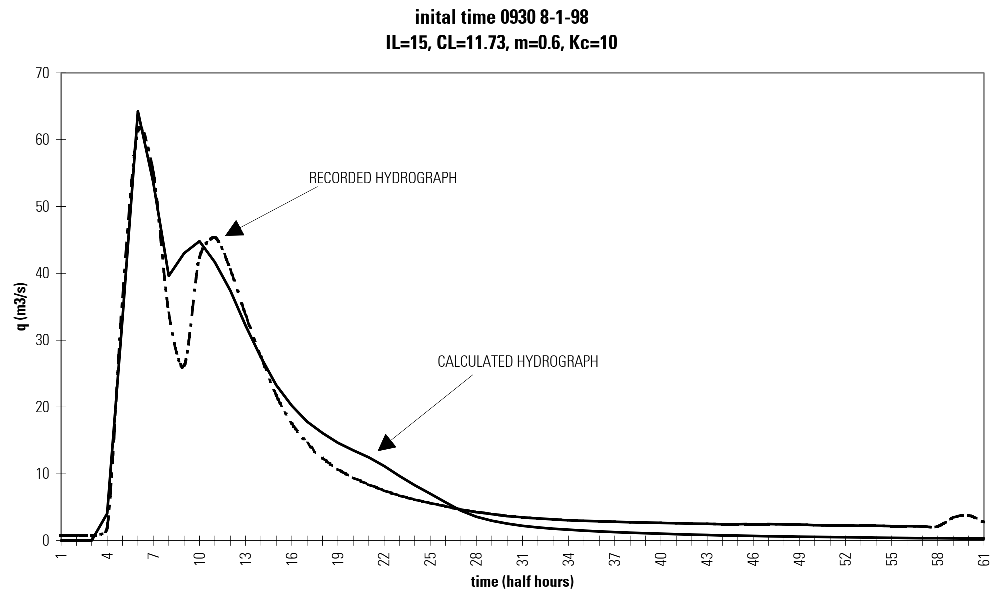
4.1.2 Flood of 8 January 1998

For the flood of 8 January 1998 data from all sources listed in Section 3 were available except that the pluviometer data recorded for the Bureau of Meteorology at Darwin Airport) station DP14015) have not yet been processed for 1998.

However, consistency was observed in the records collected at the other pluviometers operated by Water Resources Division and a good fit was obtained. In this case the best fit was found using the initial loss-continuing loss rate model but an adequate fit was still obtained using the initial loss-runoff coefficient model.

Figure 5 shows the results of the best fit run for this flood.

Figure 5, BEST FIT RUN FOR FLOOD OF 8 JANUARY 1998



4.1.3 Flood of 28 December 1993

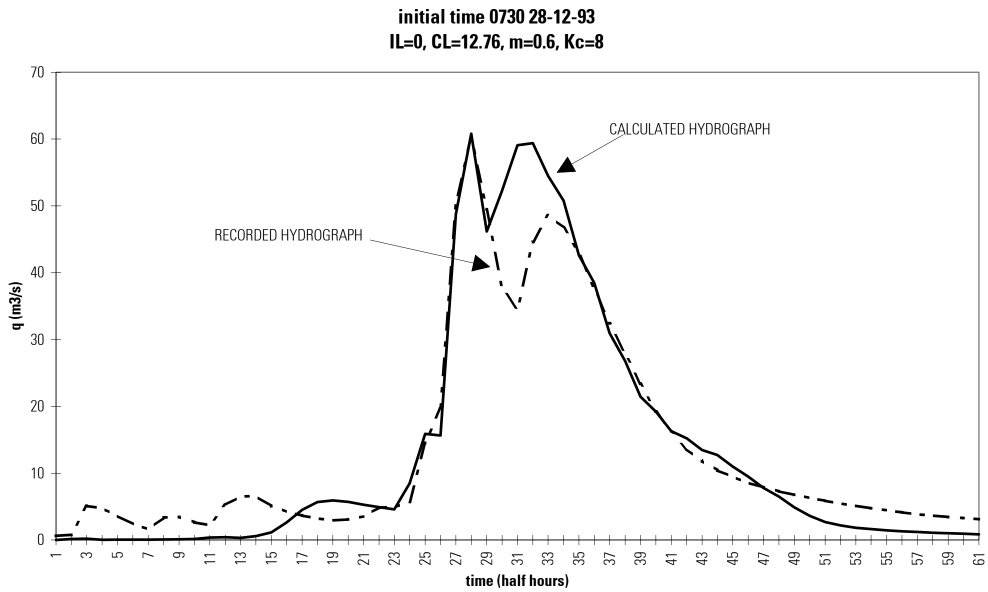
Better results were also found when the flood of 28 December 1993 was modeled using the initial loss-continuing loss rate model than the initial loss-runoff coefficient model. However, for this storm the second peak was consistently over-estimated.

The hydrographs at G8150127 are typically two-peaked and the second peak is a result of the delay of runoff from the upper catchment by the Marrara Swamp and the flood control weir.

For this storm, the most intense rainfall and the largest total storm rainfall were recorded at G8150257 at Karama School near the upper part of the catchment. It is possible that the averaging used to determine the sub-area rainfalls over-estimates the rainfall on the upper catchment giving rise to the higher peak.

The best fit hydrograph is shown in Figure 6.

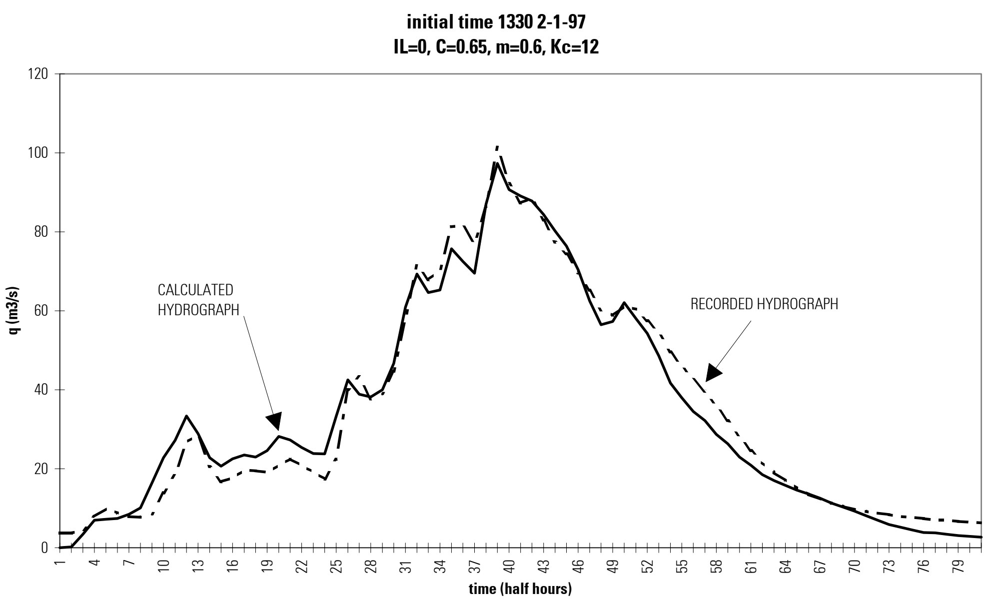
Figure 6, BEST FIT RUN FOR FLOOD OF 28 DECEMBER 1993



4.1.4 *Flood of 3 January 1997*

This flood is the fourth largest since 1974 with a peak discharge of 101 m^3/s. The storm producing this flood was prolonged and multiple peaks of rainfall intensity occurred. However, a good modeling result was achieved as shown in Figure 7.

Figure 7, BEST FIT RUN FOR FLOOD OF 3 JANUARY 1997

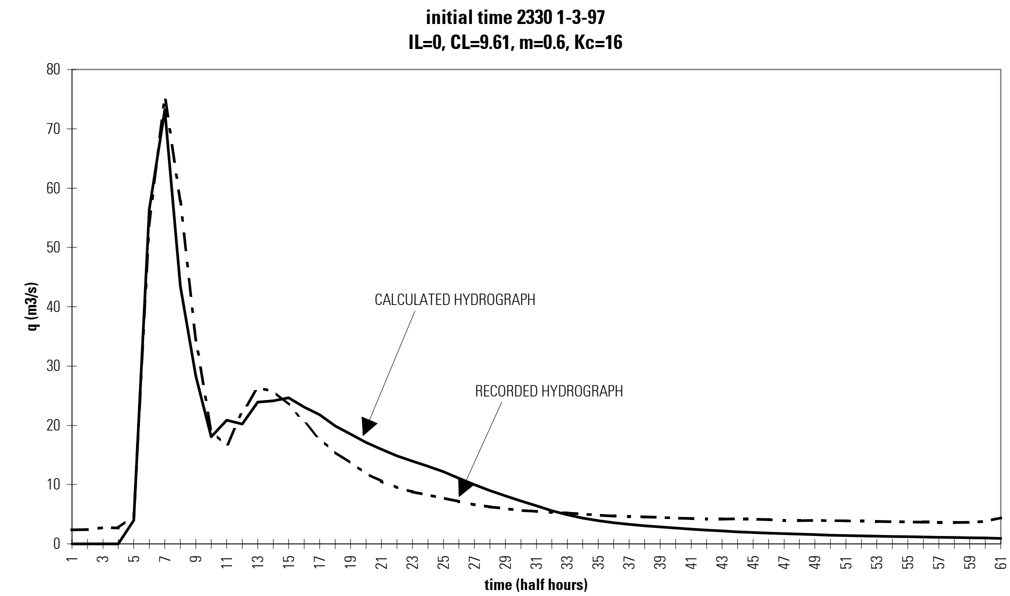


4.1.5 Flood of 1 March 1997

A very sharp flood peak of 75 m^3/s was recorded on 1 March 1997 at G8150127.

A good modeling result was also able to be achieved for this flood, but this time the storage-delay time parameter was higher than the average for other floods for which fit runs were carried out. The best fit hydrograph is shown in Figure 8.

Figure 8, BEST FIT RUN FOR FLOOD OF 1 MARCH 1997



4.1.6 Summary of fit calibration runs

Table 7 summarises the computer model runs used to match calculated and recorded hydrographs for the selected floods.

TABLE 7, SUMMARY OF BEST FIT RUNS

Flood date	Initial Loss (mm)	Continuing loss rate (CL) or Runoff coefficient (C)	Exponent in storage-discharge relationship (m)	Storage-delay time for catchment centroid (Kc)	Average absolute ordinate error (indicates goodness of fit)	Comment
05/01/1991	20	C= 0.40	0.6	5.0	22.6	Good
08/01/1998	15	CL=11.7	0.6	10.0	21.8	Fair to good
28/12/1993	0	CL=12.8	0.6	8.0	23.1	Fair to good
03/01/1997	0	C= 0.65	0.6	12.0	10.3	Very good
01/03/1997	0	CL= 9.61	0.6	16.0	20.9	Good

Table 7 shows that a range of Kc values was found but better results were observed consistently with m=0.6 rather than m=0.8, which is the default value adopted by the RORB program. It is encouraging that good to very good results were found for the two largest floods modeled.

It was necessary to adopt a Kc value for test runs in order to verify the calibrated model using different floods to those used in the calibration runs. An average value of 10.0 was adopted although it could be argued that a lesser value should be adopted in order to produce design hydrographs which are conservative (that is, with higher peaks).

4.2 Model verification (test) runs

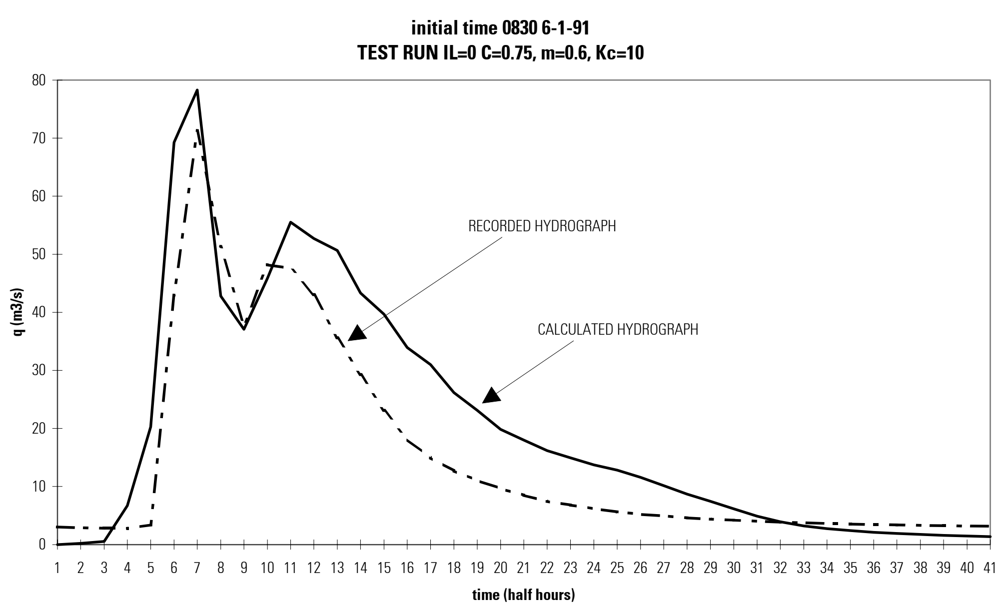
The floods selected for test runs were those of 6 January 1991, 23 December 1996 and 20 January 1998 as shown in Table 3.

For each of these the calibrated RORB model was run using the adopted parameters m=0.6 and Kc=10.0 and the only parameter which could be varied in the model input was the initial loss (the program calculates runoff coefficient or continuing loss rate such that volumes are equal for the recorded and calculated hydrographs). The results are described in the following.

4.2.1 Flood of 6 January 1991

For this flood a comparison of recorded and calculated hydrographs is shown in Figure 9.

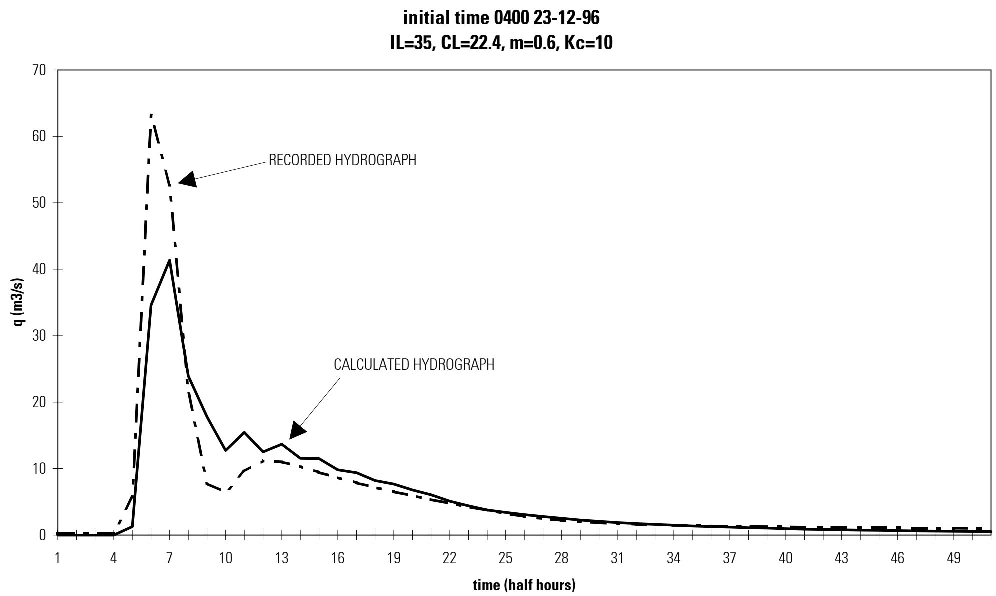
Figure 9, TEST RUN FOR FLOOD OF 6 JANUARY 1991



4.2.2 *Flood of 23 December 1996*

The storm of 23 December 1996 was brief but intense , resulting in a sharp flood peak. This peak was under-estimated in the test runs. The best result was found using the initial loss continuing loss rate model with a high initial loss (35mm) as shown in Figure 10.

Figure 10, TEST RUN FOR FLOOD OF 23 DECEMBER 1996



This flood is notable for the low second peak relative to the first flood peak. Table 8 shows this.

TABLE 8, RATIOS OF SECOND FLOOD PEAK TO FIRST FLOOD PEAK RECORDED AT GAUGING STATION G8150127

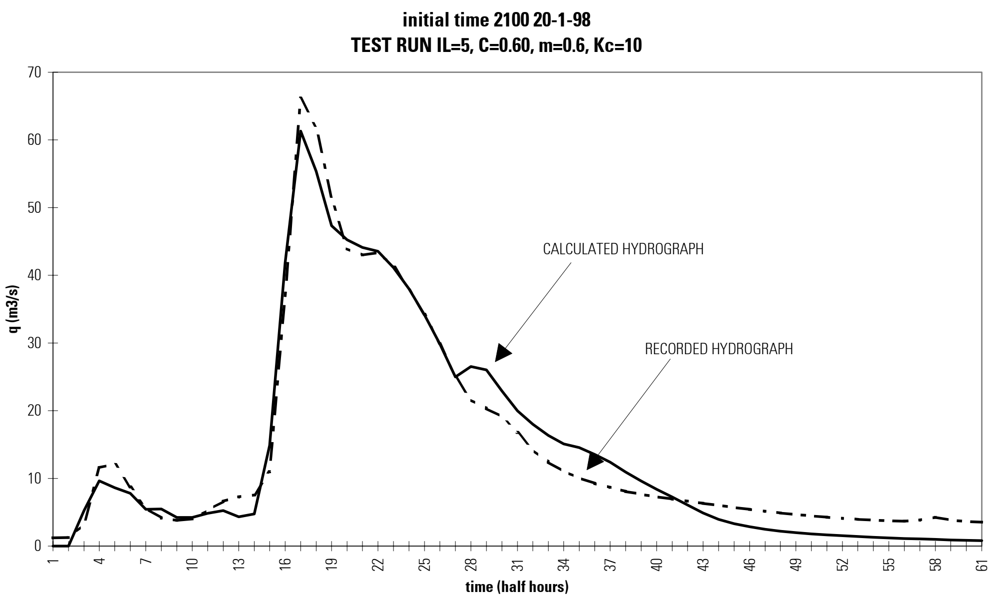
FLOOD	FIRST FLOODPEAK (m^3/s)	SECOND FLOOD PEAK (m^3/s)	RATIO OF SECOND TO FIRST
05/01/91	112	76	0.67
06/01/91	71	48	0.67
28/12/93	60	49	0.81
23/12/96	63	11	0.18
03/01/97	rainstorm and resulting hydrograph too complex		
01/03/97	75	26	0.35
08/01/98	61	45	0.74
20/01/98	66	43	0.66

The two peaked nature of recorded hydrographs reflects the action of the flood control weir. In addition to the above, a model run was made with a uniform rainfall of 100mm over 1 hour. Whilst the peak discharges vary with the value of Kc chosen the ratio of second to first flood peak varied only over the limited range of 0.51 to 0.55. Therefore, it would appear that the only way that the ratio can be as low as 0.18 (as for the flood of 23 December 1996), is if the rainfall in the lower part of the catchment (that is, below the flood control weir) is significantly higher than the rainfall in the upper part of the catchment. This is not reflected in the recorded rainfalls for the four sources used in this study and it is concluded that either one (or more) of the gauges under-registered the rainfall or that an intense storm cell developed over an area of the lower catchment not near the rain gauges.

4.2.3 Flood of 20 January 1998

A very good test result was found for the flood of 20 January 1998 using the adopted parameters as shown in Figure 11.

Figure 11, TEST RUN FOR FLOOD OF 20 JANUARY 1998



4.2.4 Summary of test runs

A summary of the test run results is given in Table 9.

TABLE 9, SUMMARY OF TEST RUN RESULTS

Flood date	Initial Loss (mm)	Continuing loss rate (CL) or Runoff coefficient (C)	Exponent in storage-discharge relationship (m)	Storage-delay time for catchment Centroid (Kc)	Average absolute ordinate error (indicates goodness of fit)	Comment
06/01/91	0	C=0.75	0.6	10	45.9	fair to good
23/12/96	40	CL=18.2	0.6	10	30.8	Peak 33% low
20/01/98	5	C=0.60	0.6	10	16.7	very good

The good results for two out of three floods suggests adoption of the parameters Kc=10 and m=0.6 in design runs. The result is not as good for the flood of 23 December 1996 but there is strong evidence that this result must be a deficiency in the rainfall information (or a water level recorder malfunction).

It can also be shown that good results can be obtained by going back over the five storms used in the calibration runs and fixing the parameters at Kc and m =10 and 0.6 respectively. To achieve this volumes were allowed to vary such that calculated and observed hydrographs were not necessarily equal, but runoff coefficient varied over a narrow band between 0.55 and 0.65 as shown in Table 10.

TABLE 10, RESULTS FOR CALIBRATION FLOODS RUN WITH KC=10, M=0.6, INITIAL LOSS-RUNOFF COEFFICIENT MODEL & RUNOFF VOLUMES ALLOWED TO BE UNEQUAL

Flood date	Initial Loss (mm)	Runoff coefficient (C)	Average absolute ordinate error (indicates goodness of fit)	Calculated hydrograph peak (m^3/s)	Recorded hydrograph peak (m^3/s)	Percentage error in calculated hydrograph volume
05/01/91	20	0.55	31.8	110	112	+31%
08/01/98	5	0.65	23.4	59	61	+1%
28/12/93	0	0.60	44.3	56	60	+39%
03/01/97	0	0.65	12.9	102	102	-1%
01/03/97	0	0.65	22.5	75	75	+12%

Since this reflects the way in which the model will be used in design and predicts peak discharges very well, it is considered to add further support for the adoption of these parameters.

Therefore Kc=10, m=0.6 and the use of the initial loss-runoff coefficient model with initial loss zero is proposed for use in design runs.

4.3 Results of flood frequency analysis

As discussed in Section 3.4, flood frequency analysis was carried out for:

> the annual series of peak discharges recorded at gauging station G8150127,

> an annual series with pre-1987 peak flows adjusted to reflect post-1987 conditions,

> an annual series with post-1987 peak flows adjusted to reflect pre-1987 conditions.

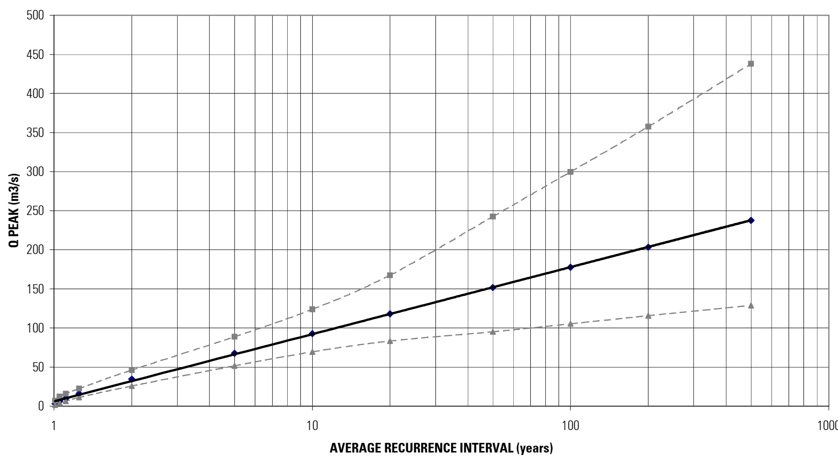
4.3.1 Observed annual floods

Figure 12(a) shows the flood frequency curve derived from the annual series of peak discharges recorded at gauging station G8150127. The derived parameters of the flood frequency distribution are:

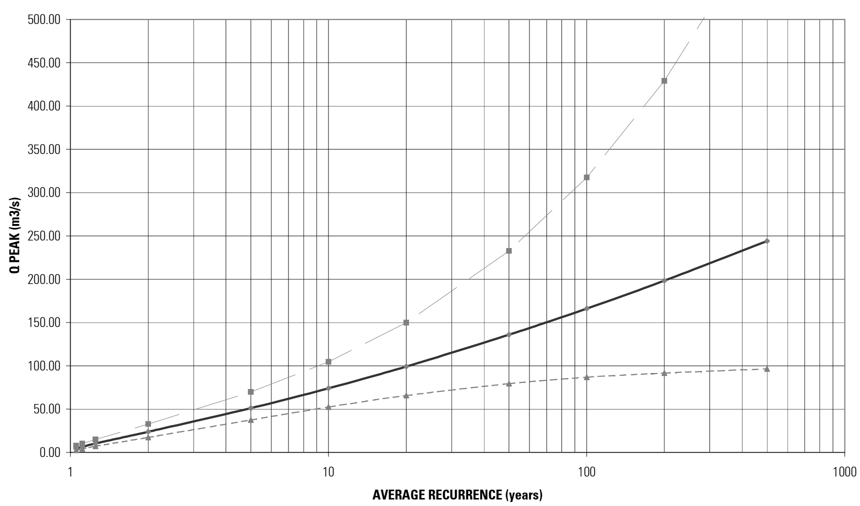
Figure 12. FLOOD FREQUENCY CURVES

NOTE:
1. PLOTS ON LOG-NORMAL PROBABILITY PAPER ARE AVAILABLE IN THE CALCULATIONS
2. EXPECTED PROBABILITY ADJUSTMENT IS NOT SHOWN
3. DASHED LINES ARE 5% AND 95% CONFIDENCE LIMITS

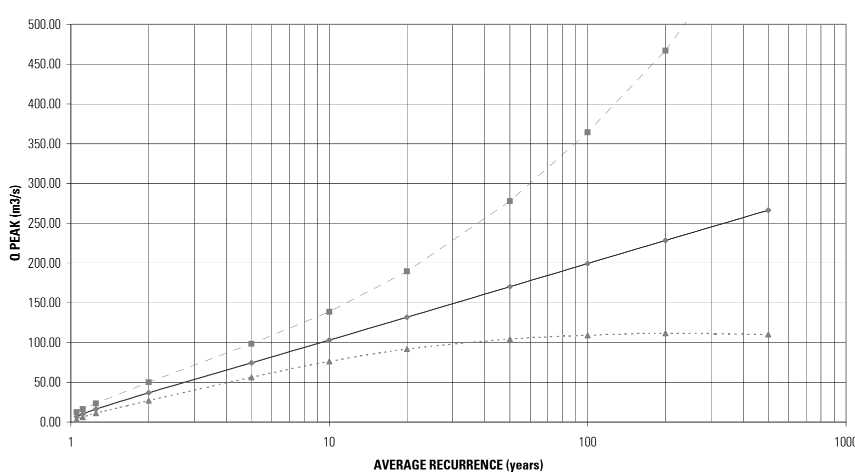
12(a), FLOOD FREQUENCY CURVE - OBSERVED SERIES



12(b) FLOOD FREQUENCY CURVE - OPTION 1 ADJUSTMENT



12(c) FLOOD FREQUENCY CURVE - OPTION 2 ADJUSTMENT



- Skewness of the logarithms of peak flows Gs -0.4885
- Standard deviation of the logarithms of peak flows Ss 0.3789
- Mean of the logarithms of peak flows Ms 1.5055.

The design flood frequencies are shown in Table 11.

4.3.2 Annual flood series for post-flood-control-weir conditions (Option 1)

Figure 12(b) shows the results of the flood frequency analysis using the annual peak flow series formed by reducing the pre-1987 peaks to allow for the effect of the flood control weir.

The parameters of this distribution are:

- Skewness of the logarithms of peak flows Gs -0.2807
- Standard deviation of the logarithms of peak flows Ss 0.4144
- Mean of the logarithms of peak flows Ms 1.3424.

The peak flow values are less on average than for the original series.

The resulting design flood frequencies are shown in table 11.

4.3.3 Annual series for pre-flood-control-weir conditions (Option 2)

Figure 12(c) shows the results of the flood frequency analysis using the annual peak flow series formed by increasing the post-1987 peaks to remove the effect of the flood control weir.

The parameters of the LP3 fitted flood frequency distribution for this case are

- Skewness of the logarithms of peak flows Gs -0.5325
- Standard deviation of the logarithms of peak flows Ss 0.3988
- Mean of the logarithms of peak flows Ms 1.5299.

As expected, the mean of the logarithms of peak discharges are higher, reflecting the absence of the flood control weir. The standard deviation and the skewness similar to previous values for the observed series. Hydrographs with peak flows that match these peak flows were calculated by adjusting the parameters in the RORB model. Then, the flood control weir was put back into the model and design hydrographs calculated by RORB using the same parameters. The resulting peak flows are also shown in table 11.

TABLE 11, PEAK DISCHARGES FOR DESIGN FLOODS DERIVED FROM FLOOD FREQUENCY ANALYSIS

AVERAGE RECURRENCE INTERVAL (ARI) (YEARS)	PEAK DISCHARGE (M3/S)		
	OBSERVED FLOODS	ACCOUNT FOR FLOOD CONTROL WEIR BY OPTION 1	ACCOUNT FOR FLOOD CONTROL WEIR BY OPTION 2 (bracketted figures are peak flows before the flood control weir is put back into the model)
20	129	107	107 (140)
50	170	154	139 (188)
100	201	197	167 (232)

Whilst the option 2 method of adjustment may be preferred from the theoretical view point it produces results that are considerably lower than those from the option 1 method and those from flood frequency analysis directly on the observed series.

There are some uncertainties in performing analysis on series that have been adjusted by either method. The biggest problem would appear to be that the calculated effect of the flood control weir depends on the hydrograph shape and the distribution of rainfall over the catchment. When using the option 2 method, uniform rainfall is used to calculate the hydrograph and this produces one particular hydrograph shape, for which the peak is attenuated by some 25 to 30 percent. However uniform rainfall is unlikely. Real storms always produce some variation over the catchment. Therefore the attenuation of peak hydrographs should vary over a larger range(see Table 10.)

Given the uncertainty in the methods used to adjust the series for inhomogeneity, it is considered reasonable to adopt the results from flood frequency analysis directly on the observed series that is, Q100=201 m^3/s, Q50=170 m^3/s and Q20=129 m^3/s.

4.4 Design flood hydrographs

Australian Rainfall and Runoff Chapter 12 gives a guide to the choice between rainfall-runoff modeling and flood frequency analysis as the method of flood estimation. Using the procedures described therein, the predicted flood peaks should match those indicated by the flood frequency analysis for average recurrence intervals up to:

Y (years) = $1.5 \times F \times N^{0.75} \times e^{0.06N}$,

where F is given in Tables in Australian Rainfall and Runoff and N is the number of years of stream flow records. Tables 12.1 and 12.2 of Australin Rainfall and Runoff with:

- 35 years of records,
- a standard deviation of the logarithms of peak flows of 0.38,
- a skewness of the logarithms of peak flows of –0.49,
- zone 2 in AR&R Figure 12.1, and
- Class 1 flood estimation method,

give a value of Y = 260 years, which suggests the peak discharge of the design hydrographs should match the discharges indicated by the flood frequency analysis at least up to and including a 1 in 260 year flood.

Design flood hydrographs were calculated using the calibrated and verified RORB model with runoff coefficients adjusted to produce peaks of the same order as those predicted from the flood frequency analysis using option 2 to account for the effect of the flood control weir (see Table 11).

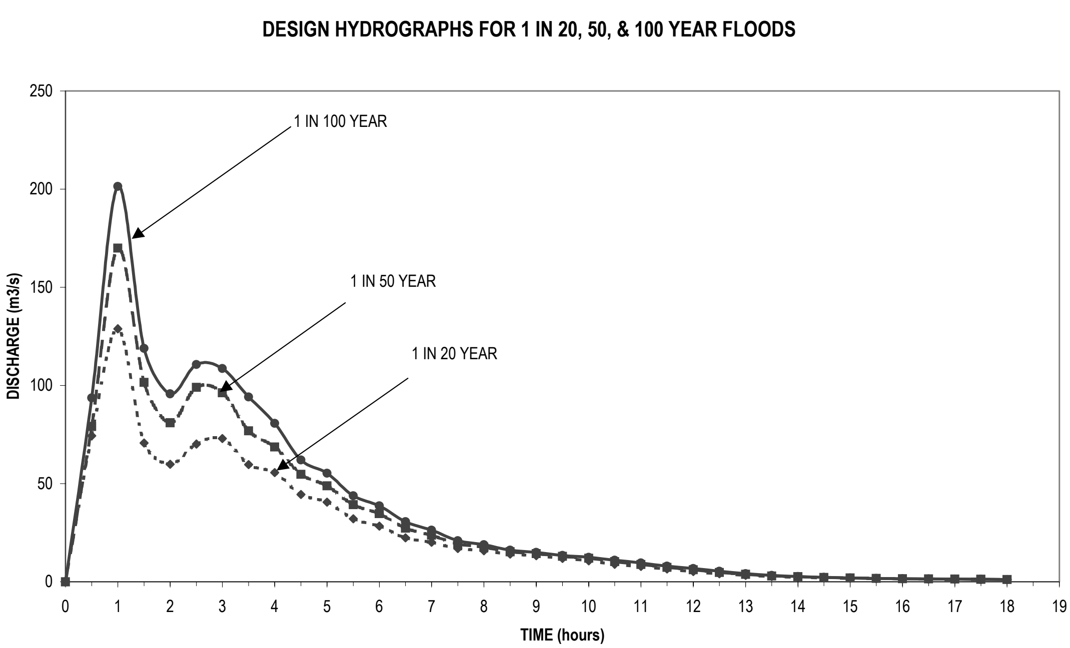
Storms of 1 hour duration were found to be critical in all cases. The runoff coefficients were 1.00, 1.09 and 1.12 for the 1 in 20 year, 1 in 50 year and 1 in 100 year design storms respectively. Runoff coefficients are greater than 1 because the shape of the design hydrographs is determined by the calibrated and verified RORB model but the flood frequency analysis is used to determine the peak magnitude.

It may be possible to further refine the flood frequency analysis. This could involve extensive modeling of flood events before 1987 to prepare a more accurate 35 year annual series that includes the effect of the weir, or modeling the remaining seven events after 1987 to increase confidence in the series that does not include the effect of the weir. In the latter case, there remains the problem of converting design flood hydrographs (based on uniform rainfall) to post-weir conditions.

However, the refinement may be within the limits of pre 1987 data accuracy and it is considered that further investigation is not warranted at this stage.

Design hydrographs calculated from 1 in 20, 50 and 100 year 1 hour storms are shown in Figure 13.

Figure 13, 1 IN 20, 50 AND 100 YEAR DESIGN FLOOD HYDROGRAPHS



These hydrographs are significantly different from those predicted by Cameron McNamara in the previous Rapid Creek hydrologic studies (Cameron McNamara,1982), both in shape and in peak magnitude. There are several possible reasons for this, including:

> Rainfall intensities used in this study are from the 1987 Australian Rainfall and Runoff which was not available at the time of the Cameron McNamara study.

> The derivation of flood peak magnitudes in this study is dependent on the flood frequency analysis. However for the present study:
   - 15 years more data is now available,
   - adjustments have been made to omit years of zero or low flow in order to eliminate a bias that produces high negative skew in the flood frequency distribution, and
   - adjustments have been made to the expected probability of rare events to eliminate small sample bias in accordance with Australian Rainfall and Runoff procedures.

   These adjustments have pushed the flood frequency curve upwards in this study. Similar adjustments do not appear to have been made in the previous study.

> The adoption by Water Resources Division of 1979/80 as the period of application of the current rating table has significantly changed the peak discharges for flood events used in the Cameron McNamara report.

> The hydrograph shape in this study reflects the influence of the flood control weir constructed in 1987 (and modified in 1990). The results of calibration and verification of the model indicate that hydrograph shape is reasonably well predicted

  The flood control weir significantly reduces the peak discharges of the design floods from upstream. A 1 in 20 year flow passes through the central portion of the weir (peaking at 64 m^3/s and an elevation of 15.5 m). A 1 in 50 year a height of 15.8 m corresponding to a discharge of 91 m^3/s. A 1 in 100 year flood fills the central portion and just overtops the side walls, reaching 16.1 m corresponding to a discharge of 136 m^3/s.

> Eight events were used in the fit and test runs for this study compared to 4 in the Cameron McNamara study.

> Since 1982 there have been developments in the airport site that have increased the amount of paved area and the likely speed of runoff reaching Rapid Creek.

4.5 Probable maximum floods

A Probable Maximum Flood (PMF) constitutes a limiting value of floods that could reasonably be expected to occur. For the purposes of this report, probable maximum floods have been derived by input of the Probable Maximum Precipitation (PMP) into the calibrated rainfall-runoff model.

In turn, the Probable Maximum Precipitation has been derived using procedures of Bulletin 53 (Bureau of Meteorology, 1994 as amended in December 1996).

For small catchments and storm durations less than 6 hours, the Generalised Short Duration Method (GSDM) is applicable and the procedures of Chapter 4 of Bulletin 53 were followed in order to estimate the spatial and temporal distribution of rainfall resulting from extreme storms of durations between 1 and 6 hours centred over the Rapid Creek catchment. The rainfall is most intense in a central ellipse and the estimated peak rainfall depth is given in Table12.

TABLE 12, CALCULATED RAINFALL DEPTH (MM) IN THE CENTRAL ELLIPSE OF A PROBABLE MAXIMUM STORM CENTRED OVER RAPID CREEK

STORM DURATION (HOURS)	1	2	3	4	5	6
RAINFALL DEPTH (mm)	463	590	663	725	782	826

The calculated rainfall depths were apportioned among the RORB model sub-areas and used as input to the model. In accordance with Chapter 12 of Australian Rainfall and Runoff Section 13.4.3, it was not considered appropriate to use the value of 0.6 as the exponent of the storage-discharge relationship (m) in the RORB model. Australian Rainfall and Runoff argues that non-linearity of flood behaviour is less during extreme floods than would be predicted from modeling of lesser floods and a value of m=0.8 was adopted.

The storage delay time is closely related to the value of m and a change in one usually requires a change in the other in order to produce similar modeling results. From an examination of the model runs made during calibration of the RORB model for this catchment, a value of Kc = 6.0 was adopted for use in predicting PMF.

Section 13.4.2 of Australian Rainfall and Runoff discusses the values of loss from rainfall to be used in calculating PMF. These should be at the lower end of values derived from model calibration runs. In these runs (see Tables 7 and 9), initial losses ranged from zero to 40 mm but were zero for four of the 8 flood events. In accordance with the discussion in AR&R it is reasonable to adopt zero initial loss for the calculation of PMF.

The continuing loss rates varied from 9.6 to 18.2 mm per hour (4 events) and runoff coefficients from 0.4 to 0.75 (4 events). AR&R advises that "a value equal to our lower than the minimum derived value should be adopted". For this study a runoff coefficient model was better in 50% of the calibration runs and has been used for design runs. Therefore a coefficient equal to or greater than the maximum derived value is appropriate and 0.8 has been adopted.

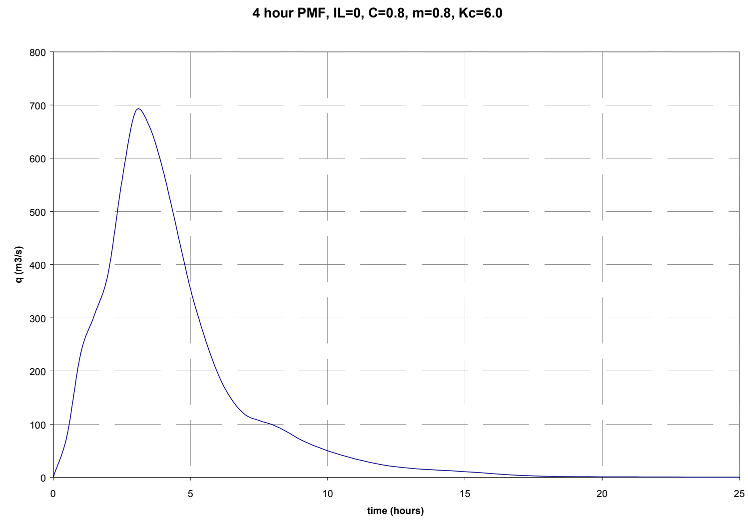
The resulting peak discharges are in Table 13 and it can be seen that the maximum peak is 689 m^3/s for a 4 hour probable maximum storm.

TABLE 13, PEAK DISCHARGES CALCULATED BY THE RORB MODEL FOR PROBABLE MAXIMUM FLOODS OF DURATIONS 1 - 6 HOURS

DURATION (HOURS)	1	2	3	4	5	6
PMF PEAK DISCHARGE (m^3/s)	497	620	676	689	649	609

The hydrograph of the 4 hour duration PMF is given in Figure 14.

Figure 14, 4 HOUR PROBABLE MAXIMUM FLOOD HYDROGRAPH



An approximate recurrence interval for the peak discharge of the calculated PMF can be found from extending the flood frequency curve. Extension as a straight line on a discharge vs log-average-recurrence-interval plot was used. (This relationship has a correlation coefficient of 0.9999 and is considered a reasonable first approximation.) An average recurrence in the order of 1 in 200,000 years was indicated and is within the range of ARIs discussed in Section 13.5.3 of Australian Rainfall Runoff .

5. Water surface profile analysis

5.1 Data

Table 14 summarises the study data used for the hydraulic analysis.

TABLE 14, DATA USED IN THE HYDRAULIC ANALYSIS

Description	Format
1 m contour information over length of Rapid Creek	Digital
Cadastral information over the length of Rapid Creek	Digital
Dept of Transport & Works drawings of bridge upgrade works at McMillans Road	Hardcopy
Dept of Transport & Works drawings of bridge upgrade works at Trower Road	Hardcopy
Subdivision drawings of local drainage works upstream of Trower Road	Hardcopy
Colour photographs of the creek and surrounds taken in July 1998	Hardcopy
Flow hydrographs at G8150127 for the flood event that occurred in January 1991	Digital
Cross sections of Rapid Creek at G8150127 generated in 1979, 1983 and 1998	Digital
Flood inundation map derived from debris marks surveyed after the January 1991 flood (from PAWA Water Resources Group file M120)	Hardcopy
Design flood hydrographs and Probable Maximum Flood hydrograph at G8150127 derived as described in Sections 4.4 and 4.5 of this Report	Digital
Design tide levels	Hardcopy
1 in 100 year storm surge levels	Hardcopy

5.2 Methodology

5.2.1 MIKE 11 Model setup

A. The MIKE 11 model

The MIKE 11 modelling system, version 4.01(a) was adopted for the Rapid Creek Flood Study.

MIKE 11 was developed by the Danish Hydraulic Institute for the simulation of unsteady flow in open channel flow systems. This is an industry accepted modelling package and has been used in many Engineering studies world wide.

As described in its Technical Reference Manual, MIKE 11 uses an implicit finite difference scheme for the computation of unsteady flows in rivers and estuaries. It solves the vertically integrated equations of conservation of continuity and momentum (the Saint Venant Equations) throughout the model, based on the following assumptions:

> The water is incompressible and homogeneous

> The bottom-slope is small

> The wave lengths are large compared to the water depth

> The flow is sub-critical. (If super-critical flow is encountered, MIKE 11 automatically uses a refined version of the Saint Venant Equations).

The implicit finite difference solution scheme used by MIKE 11 was developed by Abbott and Ionescu (1967) and calls for a "staggered grid" to be set up over the system. In this staggered grid, MIKE 11 applies the continuity equation at "H-Points" (where the model cross sections are located) and applies the momentum equation at "Q-Points" (located midway between the

H-Points). This staggered grid is the major difference between MIKE 11 and other equivalent packages, including RUBICON.

The package can describe sub-critical and super-critical flow conditions. When super-critical flow is encountered, a reduced momentum equation is applied which neglects the convective momentum term in the Saint Venant Equations.

A feature of the MIKE 11 modeling system is its ability to represent a wide range of hydraulic structures under all flow regimes.

The flow regimes used by MIKE 11 at culverts include:

> Zero flow

> Inflow critical

> Inflow partially full and outflow critical

> Inflow submerged and outflow critical

> Orifice flow

> Full culvert flow with free outflow

> Inflow and outflow partially full

> Inflow submerged and outflow partially full

> Inflow partially full and outflow submerged

> Fully submerged

The flow regimes used by MIKE 11 at weirs and other overflow structures include:

> Zero flow

> Drowned flow

> Free overflow

A strong feature of MIKE 11 is the stability and robustness of its solution scheme. As a result, MIKE 11 is particularly adept at computing low flow situations. During low flow, some model sections may "dry out" (the water level falls below the bed). This leads to numerical difficulties which, if no special measures are taken, will prohibit the continuation of the computations. MIKE 11 introduces a narrow slot below the invert of the cross sections so that calculations can continue in such situations. If the water level falls below the invert of the slot, MIKE 11 automatically introduces a small amount of water to prevent a complete drying out. The amount of water that is being added to the system can be checked using the mass error check facility.

The later releases of MIKE 11 utilise a new flow description. The "high order fully dynamic" flow description includes a high order and upstream centred description of the friction terms in the momentum equation. Using this description allows longer timesteps to be used. The MIKE 11 Technical Reference Manual recommends that this flow description is used for all cases where dynamic flow descriptions are applicable. The previous “fully dynamic” flow description is still included in the package to allow reproduction of previous results. The “high order fully dynamic” flow description has been used in the Rapid Creek MIKE 11 model.

B. Application to Rapid Creek

From inspection of the contour information, a model layout was selected and cross sections were derived. Where additional information was available (bridge and subdivision drainage drawings), further detail was added to the main channel. Refer to Figure 15.

It was noted from inspection of the supplied cross sections at the McMillans Road stream gauge that significant fill had been placed some time between 1979 and 1983. Between 1983 and 1998, it seems as though a levee has been added (refer Figure 2, page 8). This fill is reflected in the contour information.

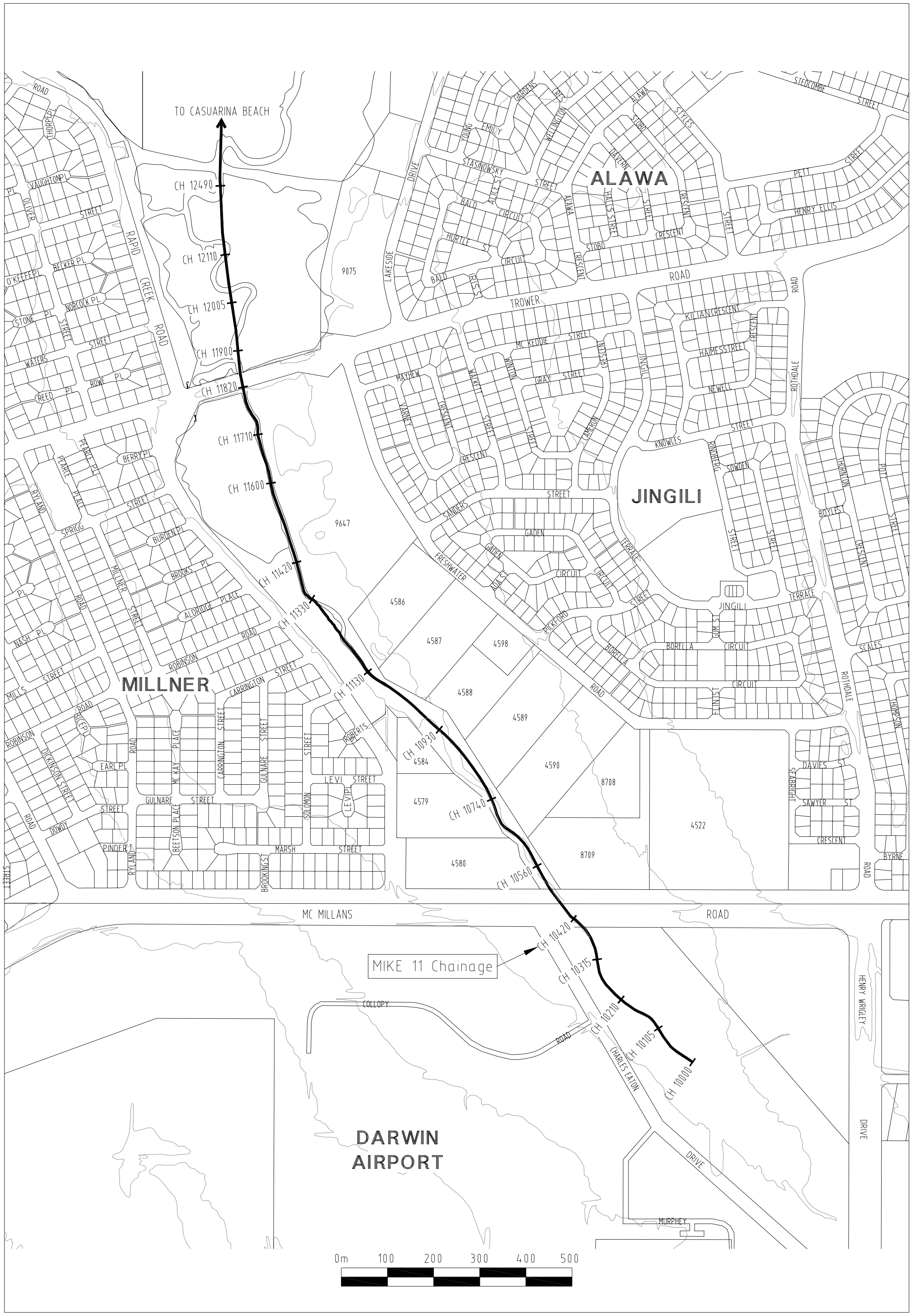
Using the colour photographs and knowledge of the site, Manning’s roughness values were assigned to represent the existing state of development. The adopted roughnesses are:

- Mangroves 0.13
- Residential development 0.12
- Mango Plantation 0.05-0.07
- Farm land 0.05
- Main channel 0.05

The inflow hydrographs were applied to the upstream model boundary as shown in Figure 13. At the downstream model limit, a tidal boundary was applied.

The McMillans Road and Trower Road crossings were rigorously modeled using standard MIKE 11 weirs and culverts and the geometry shown on the bridge drawings obtained from the Dept. of Transport and Works.

Figure 15, MIKE 11 MODEL LAYOUT



5.2.2 Model calibration

Flow hydrograph data was available for the January 1991 flood event at the stream gauging station downstream of McMillans Road (G8150127). These flows were applied at the upstream model boundary (upstream of McMillans Road). The model predicted an 8 per cent reduction in peak discharge resulting from attenuation in the reach between the upstream end of the model and G8150127. Therefore, the hydrographs at the upstream end of the model were factored by 1.08 to allow the model predictions of flood level to be directly comparable with the recorded peak flood levels.

5.2.3 Design flood analysis

Similarly, the 1 in 20 year, 50 year and 100 year design floods and the probable maximum flood were run through the model. Unfactored design flood discharges were run through the model and the predicted discharges at the stream gauging station were compared with the unfactored discharges at the upstream end of the model to assess the amount of attenuation. The discharges at the upstream end were then factored up until the discharges at the stream gauging station matched those calculated in the hydrologic analysis.

The following flood plain scenarios were investigated:

> With conditions existing before filling on the right bank in the vicinity of lot 4589 (see figure 15 for allotment numbers),

> With the filling identified in the 1983 and 1998 survey at the cross section through G8150127,

> With additional filling to similar levels on allotments 4586, 4587, 4588 and 4590.

In the absence of detailed survey, the cross section through the stream gauging station (see Figure 2) and the contours on the topographic plan were used as a guide to the extent of fill. It appears that lot 4589 in its "natural condition" falls from an elevation of about 10.5 m at Freshwater Road to 6 m near Rapid Creek, a distance of about 275 m. The survey shown in Figure 2 suggests that the right bank of the creek was filled to near level for a distance of about 100 m eastward in 1983. This corresponds to the 7 m contour on the topographic map deviating a similar distance toward the river in lot 4589.

Therefore the following adjustments were made to cross section levels in the model, in order to simulate the effects of filling and to gain an indication of how fill might affect relative flood levels consistent with the accuracy of the hydraulic and hydrologic modeling:

> For the "natural condition":
  – cross sections at chainage 11,330 and 11,130 have overbank areas as extracted from the topographic map with a channel profile and bed level inferred from the design plans of Trower Road bridge, subdivision works and the 1979 cross section at the gauging station (refer Table 14).
  – The cross section at chainage 10,930 was the the 1979 surveyed profile at the gauging station extended by contour data from the topographic map.
  – The cross section at chainage 10,740 was derived similarly to those at 11,330 and 11,130 including design data on the McMillans Road bridge.

> For the “existing conditions” the sections are as above except that at chainage 10,930 the 1983 surveyed profile was used and extended by contour data from the topographic map.

  It should be noted that the only substantive difference between the 1983 and 1998 profiles is the levee shown in the 1998 survey. This levee is of variable height and is not continuous. It is likely that floodwaters will find a way beyond the limits of the levee and that for major floods some parts of it would be overtopped and therefore it is unlikely to have any significant effect on major flood levels.

> For the condition with “filling of lots 4586 to 4590” the sections at 11,330, 11,130, and 10,740 had an area blocked out on the right bank, to a height of 7.0 m and a similar shape to that on the 1983 surveyed profile, to represent fill.

5.3 Results

5.3.1 Model calibration

Recorded water levels were available at the stream gauging station downstream of McMillans Road (G8150127) and included a recorded hydrograph and a peak flood level. The recorded peak flood level was 6.58 m AHD at 12:03 pm on 5 January 1991. The model predicted a flood peak level of 6.54 m AHD at 12:20 pm on 5 January 1991.

Further, the lateral extent of flooding was inferred from the calculated water surface levels at cross sections. Good agreement was found with the observed water levels marked on a topographic plan in 1991 following this flood.

The calibration was accepted.

5.3.2 Design flood modeling

Table 15 summarises the peak flood levels predicted for the 1 in 20 year, 50 year and 100 year design floods. In order to assess the sensitivity of the results to the sea level prevailing at the time of the flood peak, two different starting tail water levels have been examined:

> Highest Astronomical Tide (HAT),

> Mean High Water Neaps (MHWN).

The current profiles on lots 4586 to 4590 were used for the model runs that gave the results in Table 15.

Table 16 contains the results for the case where lots 4586, 4587, 4588, 4589 and 4590 are all filled to a similar extent.

Note that the results of the “natural condition” model runs are not tabulated because they are very similar to those in Table 15 except for a small and localised fall at chainages 10,930 and 10,740. (See also B. in the following discussion.)

TABLE 15, 1 IN 20 YEAR, 1 IN 50 YEAR AND 1 IN 100 YEAR DESIGN PEAK FLOOD LEVELS WITH FILLING OF LOT 4589 (CURRENT CONDITION)

MIKE 11 CHAINAGE (km)	PEAK FLOOD LEVEL (m AHD)					
	100 yr HAT	50 yr HAT	20 yr HAT	100 yr MHWN	50 yr MHWN	20 yr MHWN
10.000	10.25	10.09	9.91	10.25	10.09	9.91
10.210	9.29	9.12	8.93	9.29	9.12	8.93
10.420	8.55	8.37	8.12	8.56	8.37	8.13
10.560	7.82	7.63	7.38	7.82	7.63	7.38
10.740	7.37	7.20	6.99	7.37	7.20	6.99
10.930	6.60	6.43	6.24	6.59	6.42	6.23
11.130	5.91	5.72	5.53	5.89	5.69	5.49
11.330	5.58	5.39	5.14	5.56	5.34	5.04
11.420	5.51	5.33	5.08	5.49	5.29	4.96
11.600	5.45	5.28	5.04	5.43	5.24	4.89
11.820	5.41	5.25	5.02	5.39	5.21	4.87
11.900	4.41	4.27	4.16	4.25	3.99	3.65
12.110	4.26	4.17	4.10	3.54	3.37	3.19
12.490	4.18	4.12	4.08	3.34	3.19	3.02
13.500	4.03	4.03	4.03	1.09	1.09	1.09

TABLE 16, 1 IN 20 YEAR, 1 IN 50 YEAR AND 1 IN 100 YEAR DESIGN PEAK FLOOD LEVELS WITH FILLING OF LOTS 4586 TO 4589

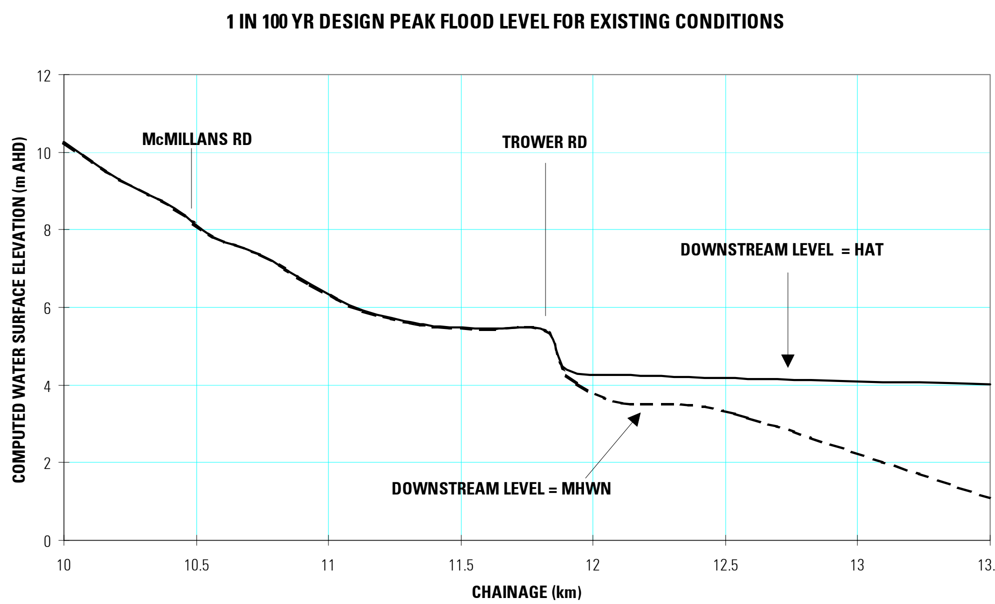
MIKE 11 CHAINAGE (km)	PEAK FLOOD LEVEL (m AHD)					
	100 yr HAT	50 yr HAT	20 yr HAT	100 yr MHWN	50 yr MHWN	20 yr MHWN
10.000	10.25	10.09	9.91	10.25	10.09	9.91
10.210	9.30	9.13	8.93	9.30	9.13	8.94
10.420	8.60	8.41	8.15	8.60	8.41	8.16
10.560	7.98	7.78	7.58	7.98	7.78	7.59
10.740	7.64	7.48	7.32	7.64	7.48	7.32
10.930	6.71	6.53	6.33	6.71	6.52	6.32
11.130	6.14	5.97	5.76	6.13	5.94	5.74
11.330	5.61	5.40	5.13	5.58	5.35	5.02
11.420	5.52	5.33	5.07	5.50	5.29	4.93
11.600	5.45	5.28	5.02	5.43	5.24	4.85
11.820	5.41	5.25	5.00	5.39	5.21	4.83
11.900	4.41	4.27	4.15	4.23	3.99	3.64
12.110	4.26	4.17	4.10	3.54	3.35	3.18
12.490	4.18	4.12	4.07	3.33	3.15	3.03
13.500	4.03	4.03	4.03	1.09	1.09	1.09

Two effects are demonstrated in these tables.

A. Effect of tailwater level

The effect of adopting different tail water levels is shown in Figure 16. The choice of tail water level makes little difference upstream of the Trower Road bridge.

Figure 16, EFFECT OF TAIL WATER LEVEL ON 1 IN 100 YEAR FLOOD LEVEL



The tailwater levels considered in Figure 17 do not include the effect of storm surge. Storm surge is a rise in sea level that occurs during cyclones when reduced barometric pressures over a large area cause a general rise and strong winds push the water toward the land.

The storm surge during Cyclone Tracy in 1974, a category 4 cyclone, was about 4 m. Cyclone Thelma, which passed Darwin early in 1999 was a category 5 cyclone and storm surges larger than 4 m should now be considered likely although they have less chance of occurring. Figure 17 suggests that a 4m storm surge coinciding with MHWN is unlikely to influence 1 in 100 year flood levels beyond Trower Road. That is, only extremely rare storm surges or storm surges that occur during the peak of spring tides or rarer events such as the HAT will have an effect on flood levels in the area of interest to this study.

The joint probability of a major flood occurring together with a major high tide and a large storm surge is very small. For example, a 1 in 100 year flood has a 1% chance of occurring in any one year and the HAT has approximately a 5% chance. The joint probability of occurrence is in the order of 0.05 per cent (0.01*0.05 =0.0005).

The occurrence of flood producing rains and storm surges are not totally independent events, because cyclones are associated with both. However, they can not be considered as dependent events either because many intense storms in the Rapid Creek catchment are associated with short duration convective cells that do not result from cyclones. Further, the intensity of prolonged rainfall over land that results from the passage of a cyclone is not necessarily linked to the magnitude of the storm surge. Factors such as the location of the cyclone and the path it takes, the time of year and the amount of moisture available in the air, will result in variations in the intensity of rainfall over land, even though considerable storm surge may arise in the sea.

Therefore the occurrence of a major flood together with a high tide and a storm surge has a probability even less than 0.0005, say somewhere in the vicinity of 10^{-5} or even smaller.

Model runs with storm surge added to tide level have not been carried out because the probability is low. For the purposes of this study, it is assumed that a 1 in 100 year storm produces a 1 in 100 year flood for design catchment conditions. It is possible to modify the calculated probabilities using a rigorous joint probability analysis and there may be a very small component of coincident frequency associated with storm surge and tide combinations high enough to affect levels beyond Trower Road. However, such an analysis is beyond the scope of the present study.

B. Effect of filling of lots 4586 to 4590

The effect of filling of the flood plain on a 1 in 100 year flood is as follows:

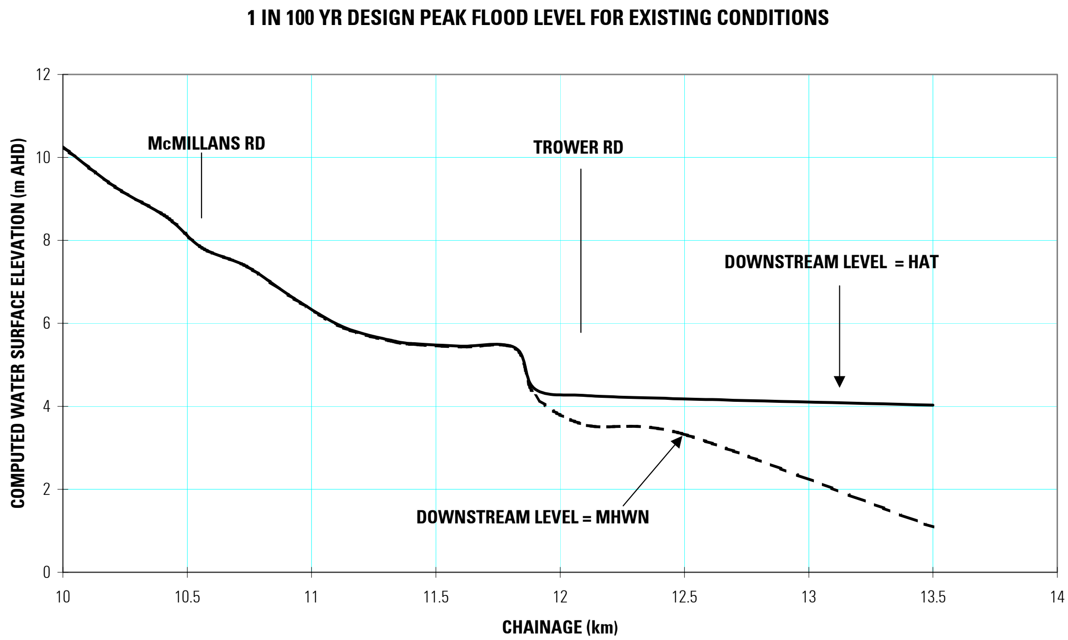
- The effect of filling of lot 4589 has been a localised increase in water level of up to 180 mm which diminishes to small values within a short distance upstream and downstream

  The impact would be an increase of this order in the depth of inundation within the few properties flooded along Rapid Creek Road in the vicinity of the intersection with Solomon Street.

- The effect of filling on allotments 4586-4590, to a similar level as exists on lot 4589, is to increase levels by 240 mm to 370 mm over a distance of some 800 m and diminishing within a short distance upstream and downstream (refer Figure 17). This would be reflected in a corresponding increase in the depth of flooding of properties along Rapid Creek Road in the vicinity of the intersection with Solomon Street. In addition, there would be a localised extension of the area expected to be inundated by up to about 40 m to the west, such that additional properties would be affected.

The results of this study indicate the sort of increase in flood level that would occur as a result of placing fill similar to that existing on lot 4589 (up to 7 m AHD). However, lower or higher fill levels could be proposed and individual development proposals should be considered on merit.

Figure 17, EFFECT OF FILLING ALLOTMENTS 4586 TO 4590 ON 1 IN 100 YEAR FLOOD



LEVELS

5.3.3 Modeling of Probable Maximum Flood

Table 17 presents the peak flood levels predicted by the model for a 4 hour duration Probable Maximum Flood.

TABLE 17, PROBABLE MAXIMUM FLOOD LEVELS

MIKE 11 CHAINAGE (km)	PEAK FLOOD LEVEL (m AHD)
10.000	11.16
10.210	10.23
10.420	9.44
10.560	8.90
10.740	8.40
10.930	7.62
11.130	7.00
11.330	6.71
11.420	6.58
11.600	6.40
11.820	6.21
11.900	5.95
12.110	5.34
12.490	5.03
13.500	4.03

The Probable Maximum Flood (PMF) would peak in the order of 690 m^3/sec (compared to a 1 in 100 year flood of 201 m^3/sec). A map showing inundation by a Probable Maximum Flood (refer Figure 21) shows the following:

> Much of the area between Freshwater Road and Rapid Creek to the north of Sanders Street is liable to inundation, and some properties along Freshwater Road in the vicinity of Mayhew Street are liable to inundation.

> To the south of Sanders Street, land within about 200 m of Rapid Creek is subject to inundation.

> West of Rapid Creek, a large number of properties would be affected in Levi Street, Solomon Street, Gulnare Street, Carrington Street, Robinson Road, Aldridge Place, Brooks Place, Burden Place, Sprigg Street, Berry Place and Trower Road.

5.4 Conclusions

A MIKE 11 hydraulic model has been set up and calibrated to Rapid Creek. The model area of interest was between the McMillans Road and Trower Road crossings.

The model was satisfactorily calibrated to the January 1991 flood event. Information available at the McMillans Road stream gauge included a discharge hydrograph and a peak flood level.

Design floods for 1 in 20 year, 1 in 50 year and 1 in 100 year Average Recurrence Interval were run through the model. Two tail water levels were considered (Mean High Water Neaps - MHWN and Highest Astronomical Tide - HAT). The HAT level is the highest still water level that would occur as a result of the influence of gravitational forces in the absence of wind set up, storm surge etc. This is a relatively high level and the joint probability of it occurring together with a major storm is almost negligible. Upstream of the Trower Road bridge, the calculated flood level is insensitive to the chosen tide level (see Figure 16).

The sensitivity of the peak flood levels to the extent of filling of allotments on the right bank downstream of McMillans Road has been assessed.

It is concluded that filling of lot 4589 has had a localised effect which it could be argued will make flooding slightly worse during a 1 in 100 year flood. Filling to similar levels in the other allotments on the eastern side of the creek between the Water Gardens and McMillans Road would increase flood levels by a significant amount and inundation could be expected to occur on additional properties on the western side of the creek.

Floods larger than a 1 in 100 year flood are extremely rare with the chances of their occurrence in any one year being less than 1 percent. However consideration of the probable maximum flood shows that much of the corridor of non residential land between McMillans road and Trower Road is on a flood plain, as are a large number of residences in the lower-lying parts of the suburb of Millner. Land use planning and occupant behavior should take into account that any artificial obstructions to flood flows can have an impact on the depth and extent of inundation when rare floods do occur. It is not uncommon for those affected by major floods to commence litigation against any party that has undertaken works which may have changed the depth and extent of inundation, and this can be expected to include actions against approving authorities.

Irrespective of flooding of Rapid Creek, a large storm surge coinciding with a high tide can be expected to cause sea-water flooding in these areas. A description of the extent and depth is beyond the scope of this report.

Figures 18 to 21 show the predicted extent of inundation for the design floods considered assuming current conditions and tide level at Mean High Water Neaps.

Figure 18, EXTENT OF INUNDATION FOR A 1 IN 20 YEAR FLOOD

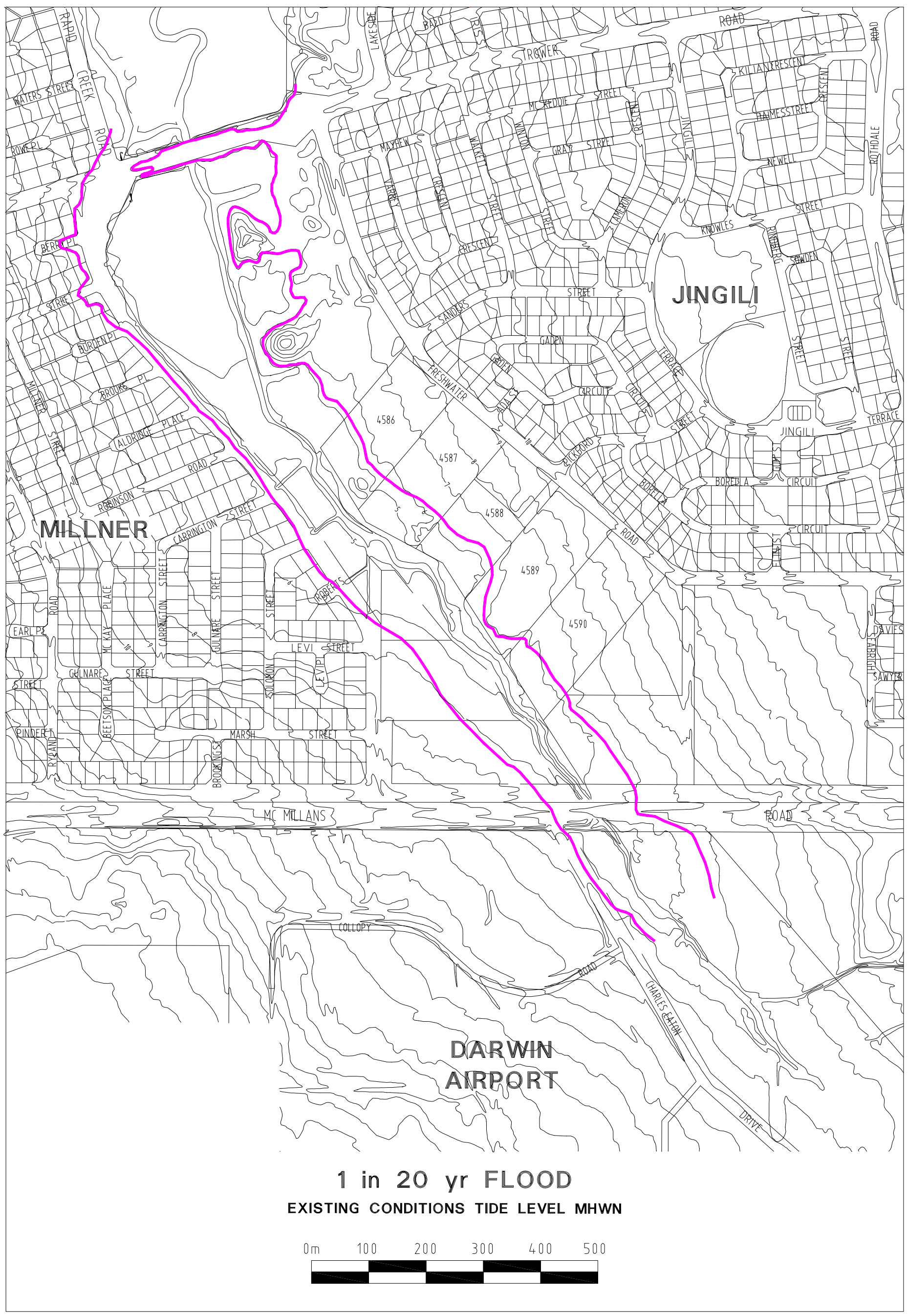


Figure 19, EXTENT OF INUNDATION FOR A 1 IN 50 YEAR FLOOD

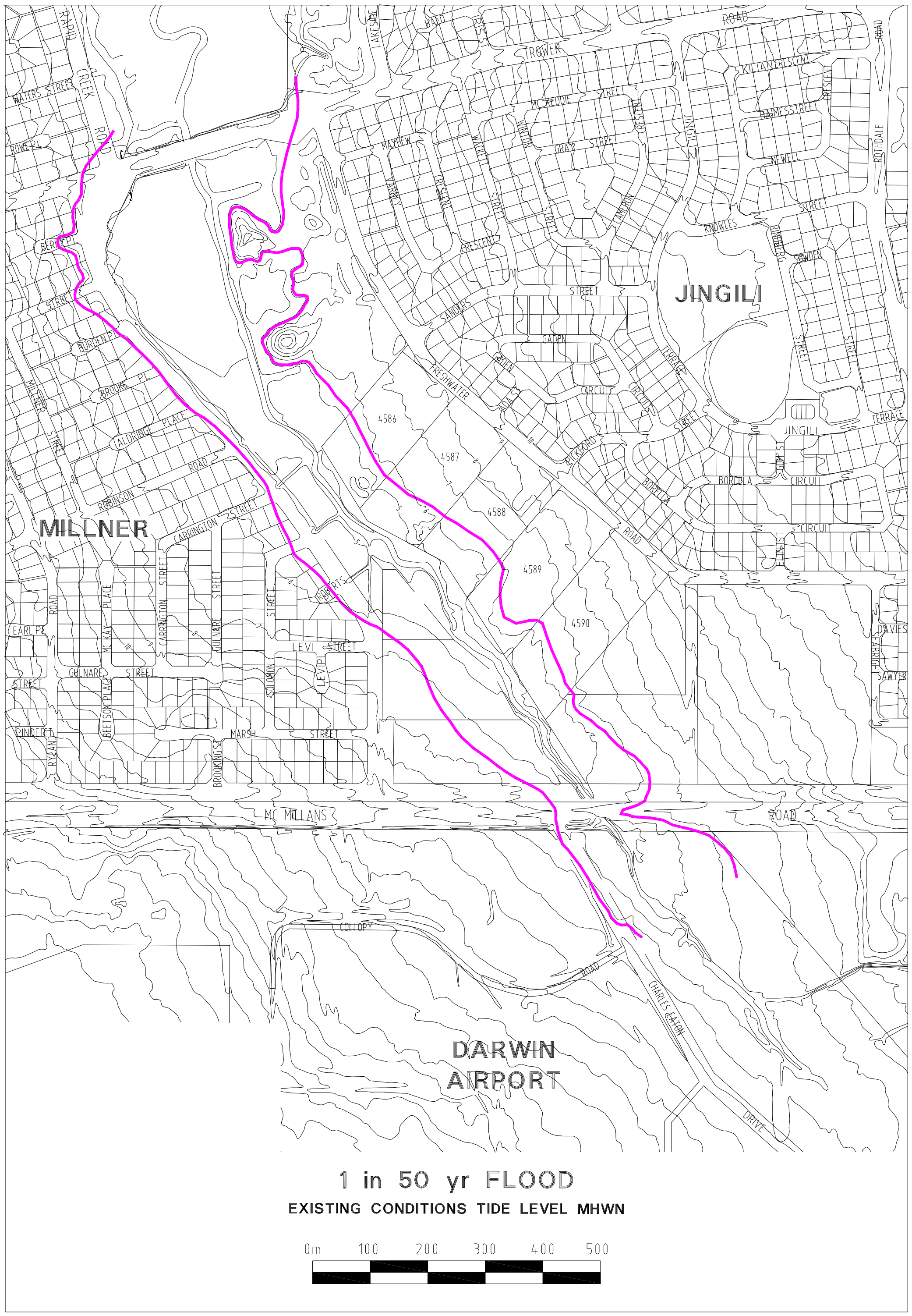


Figure 20, EXTENT OF INUNDATION FOR A 1 IN 100 YEAR FLOOD

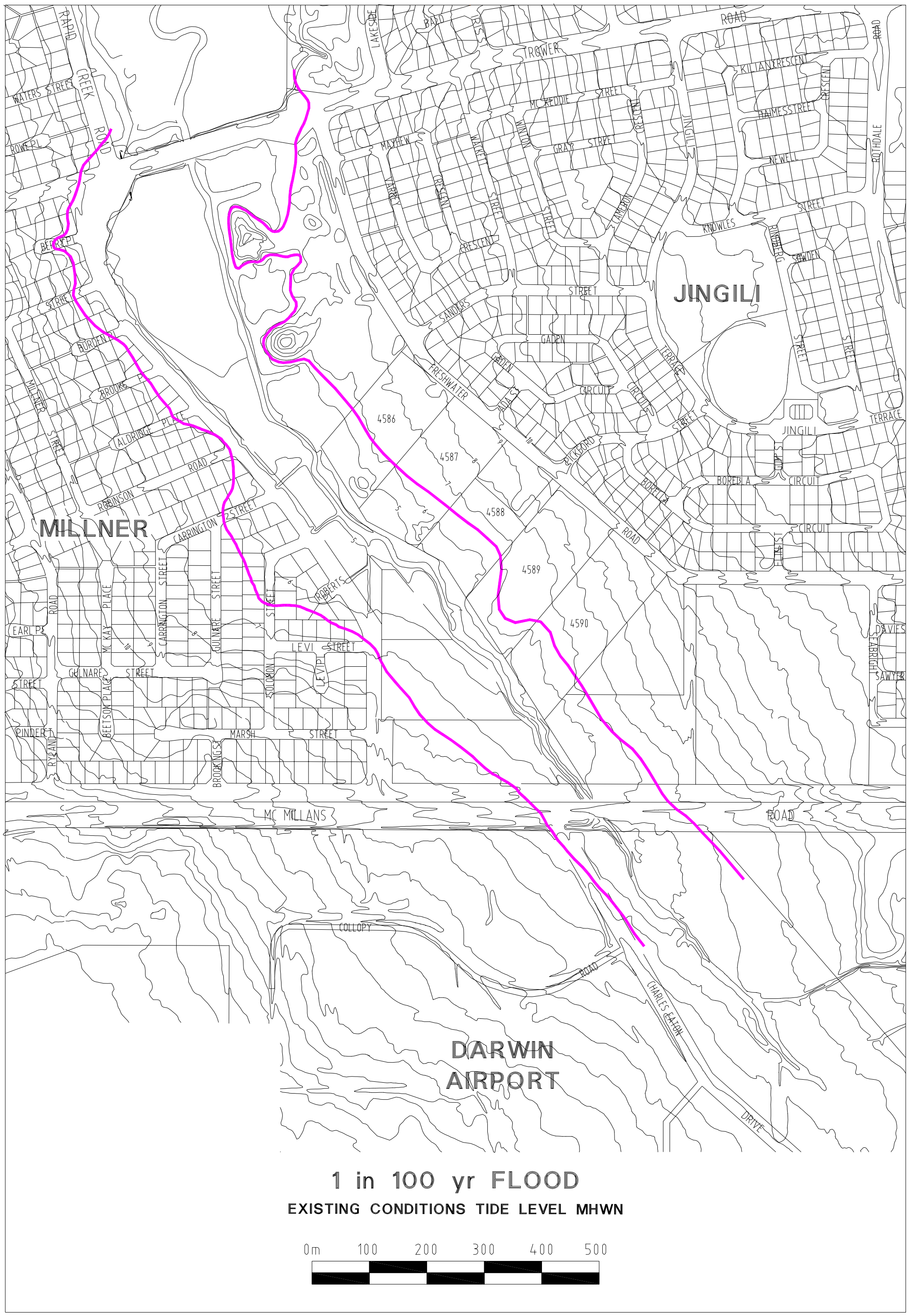
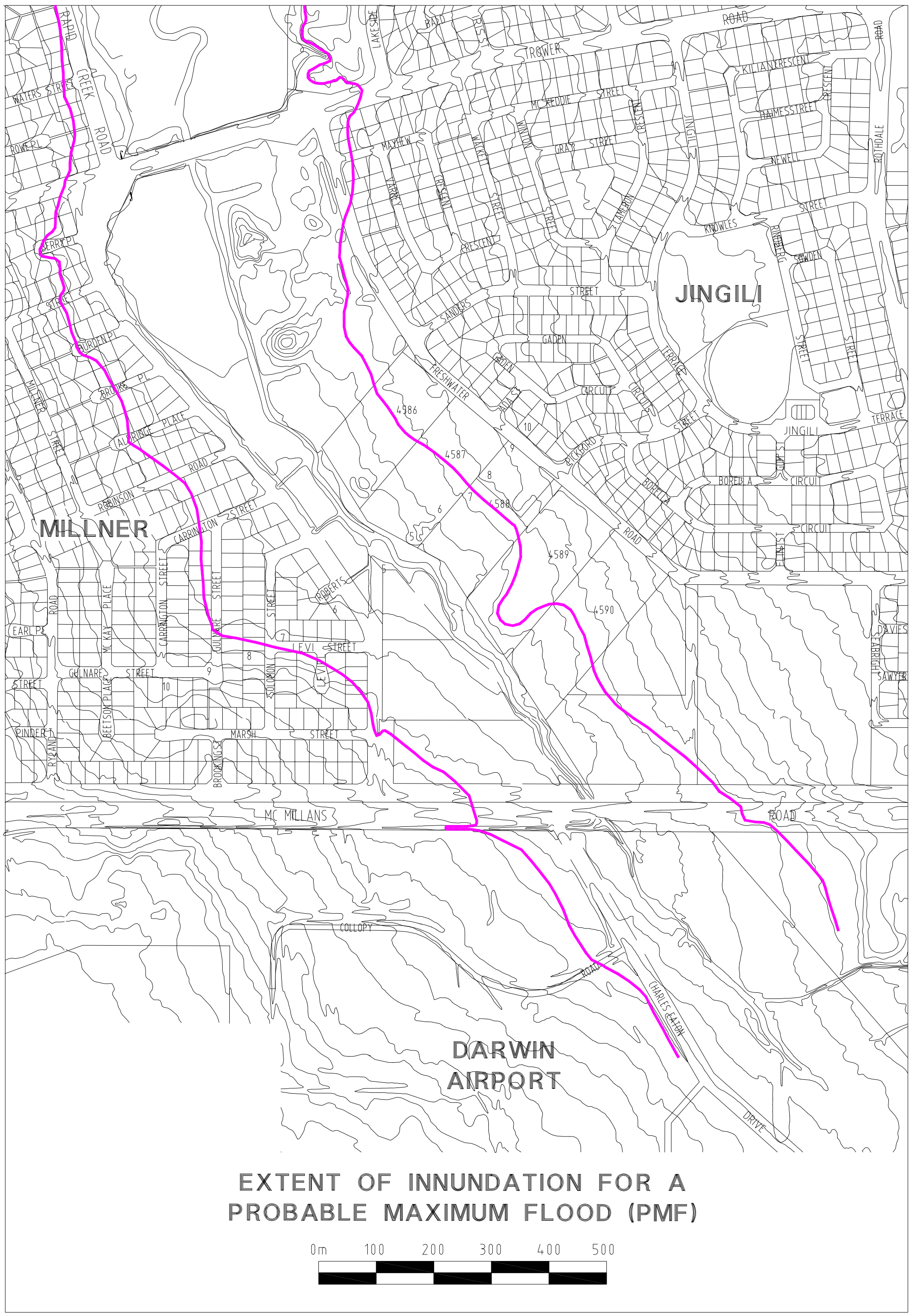


Figure 21, EXTENT OF INUNDATION FOR A PROBABLE MAXIMUM FLOOD



6. Flood hazard zones

The extent to which floodwaters represent a hazard is a function of both their depth and velocity.

The following criteria have been used to assess the degree of hazard as described in Figure 7 of the NSW Floodplain Development Manual (1977) :

- For zero flow velocity (eg. a backwater):
  - if the depth is between 0.8 m and 1.0 m then the hazard category is medium, and
  - If the depth exceeds 1.0 m then the hazard category is high.

- For a velocity of 2.0m/sec:
  - if the depth is between 0.2 m and 0.4 m the hazard category is medium, and
  - if the depth is greater than 0.4 m the hazard category is high.

- For velocities between zero and 2.0 m per second, linear interpolation is used.

- For velocities in excess of 2.0 m/sec the floodwaters are considered hazardous no matter what the depth.

In assessing the flood hazard it should also be borne in mind that:

- light vehicles can be unstable as a result of buoyancy forces for depths of 0.3 m or more,

- the velocity of floodwaters passing between buildings can produce a hazard, which may not be apparent if only average velocity is considered.

Therefore the flood hazard maps should be considered a guide only to the likely danger posed by flooding in the area of interest.

Hazard zones for the 1 in 20 year, 1 in 50 year, 1 in 100 year and probable maximum floods with existing conditions are shown in Figures 22 to 25 respectively.

Figure 22, 1 IN 20 YEAR FLOOD HAZARD ZONES

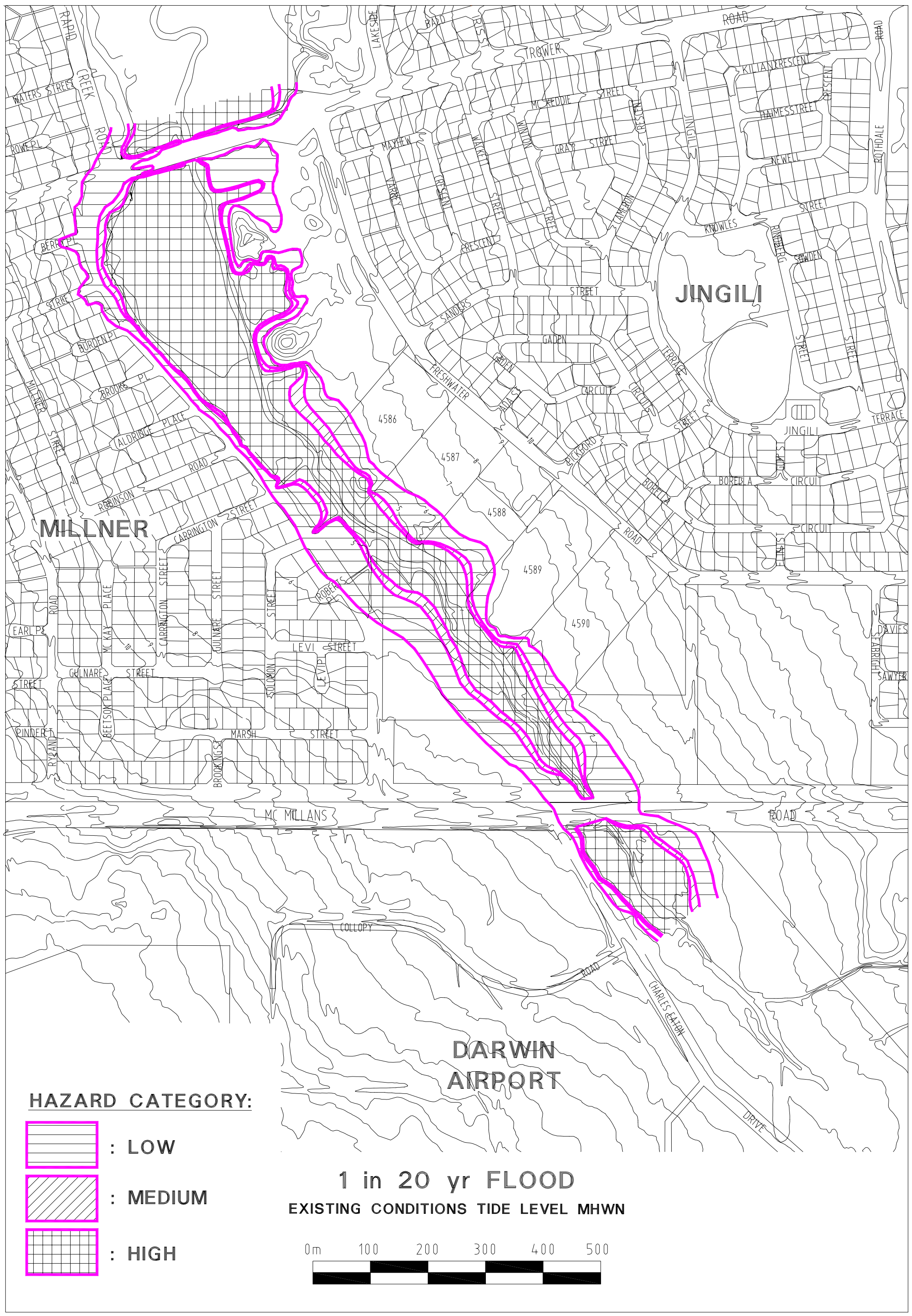


Figure 23, 1 IN 50 YEAR FLOOD HAZARD ZONES

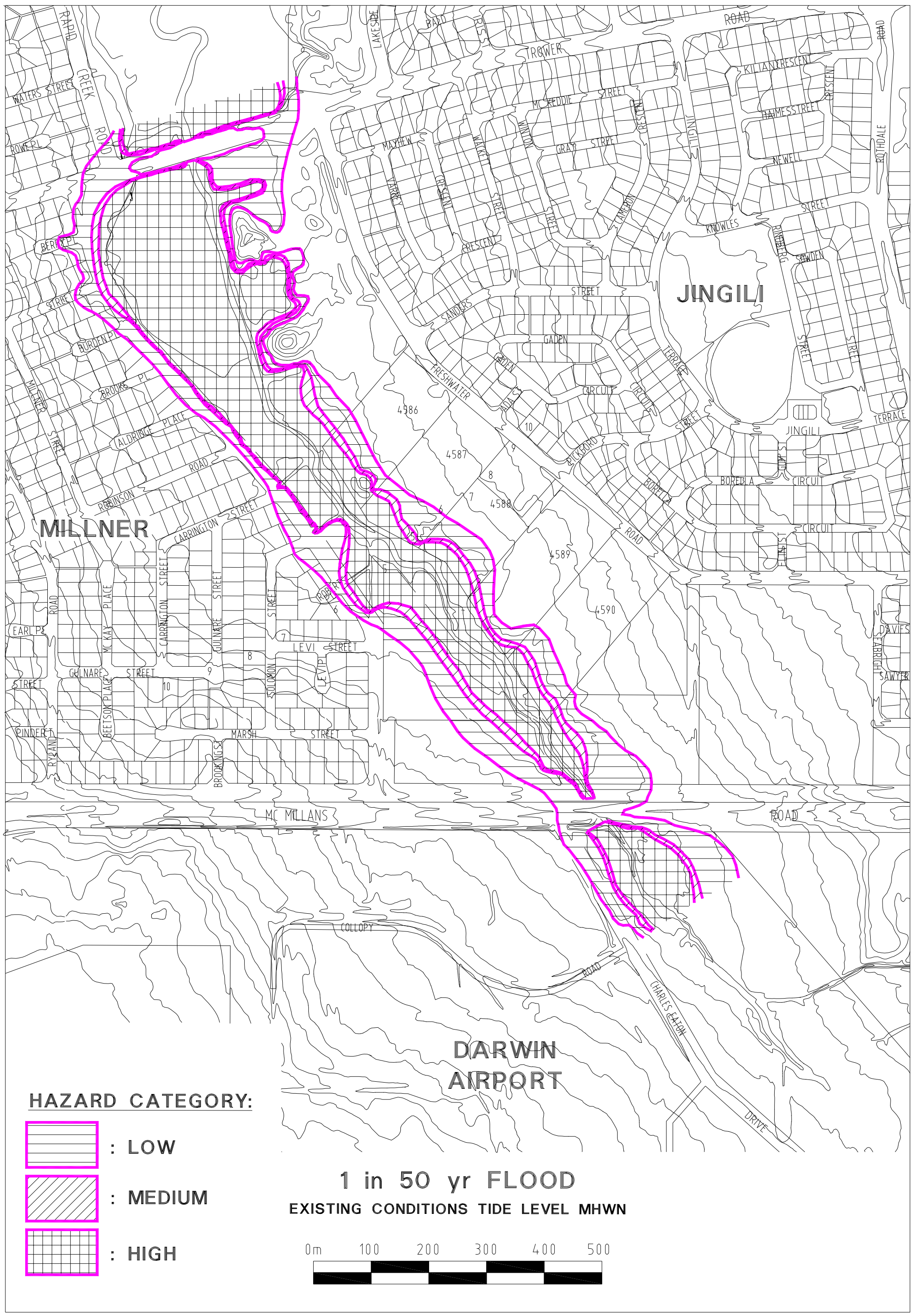


Figure 24, 1 IN 100 YEAR FLOOD HAZARD ZONES

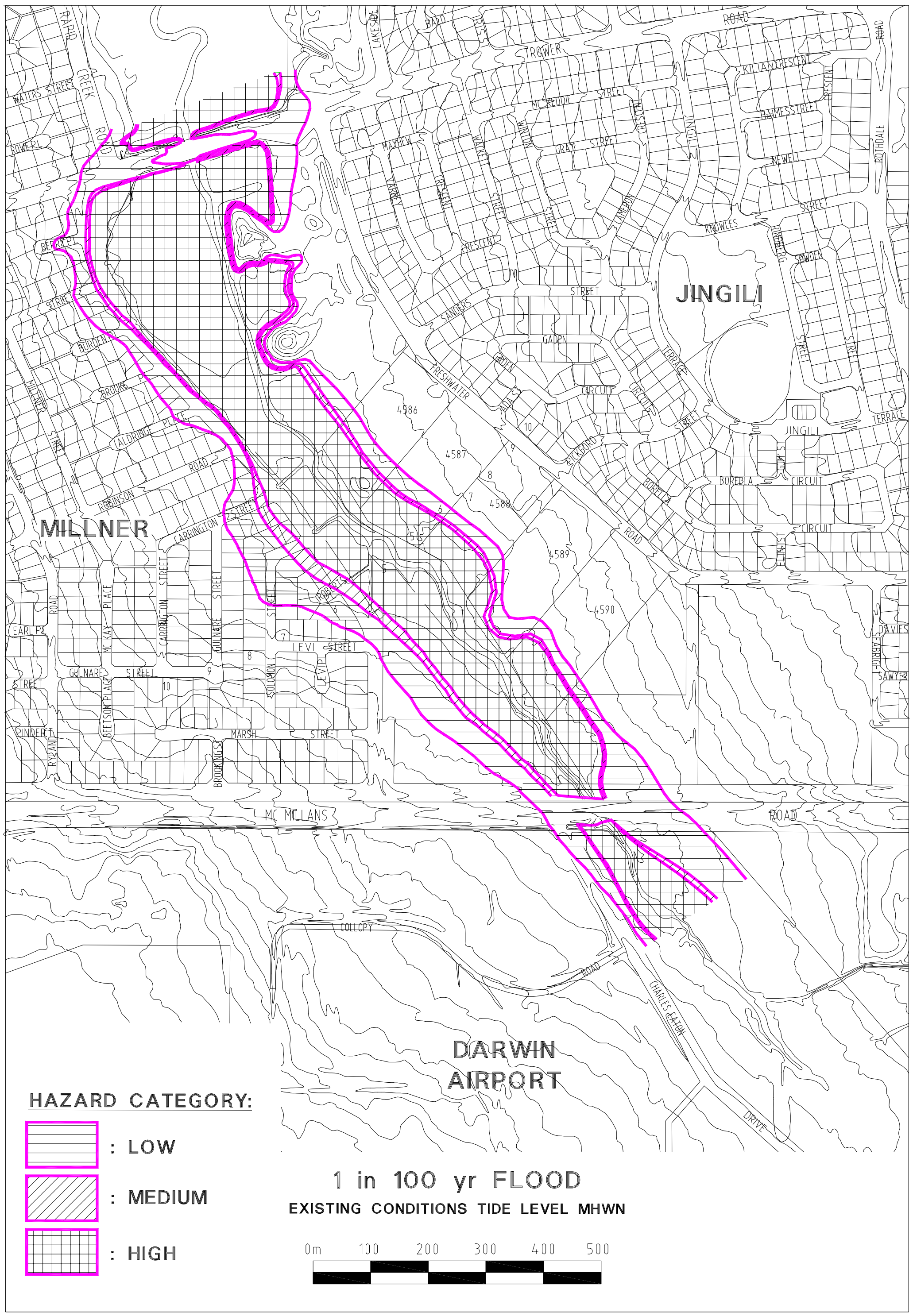
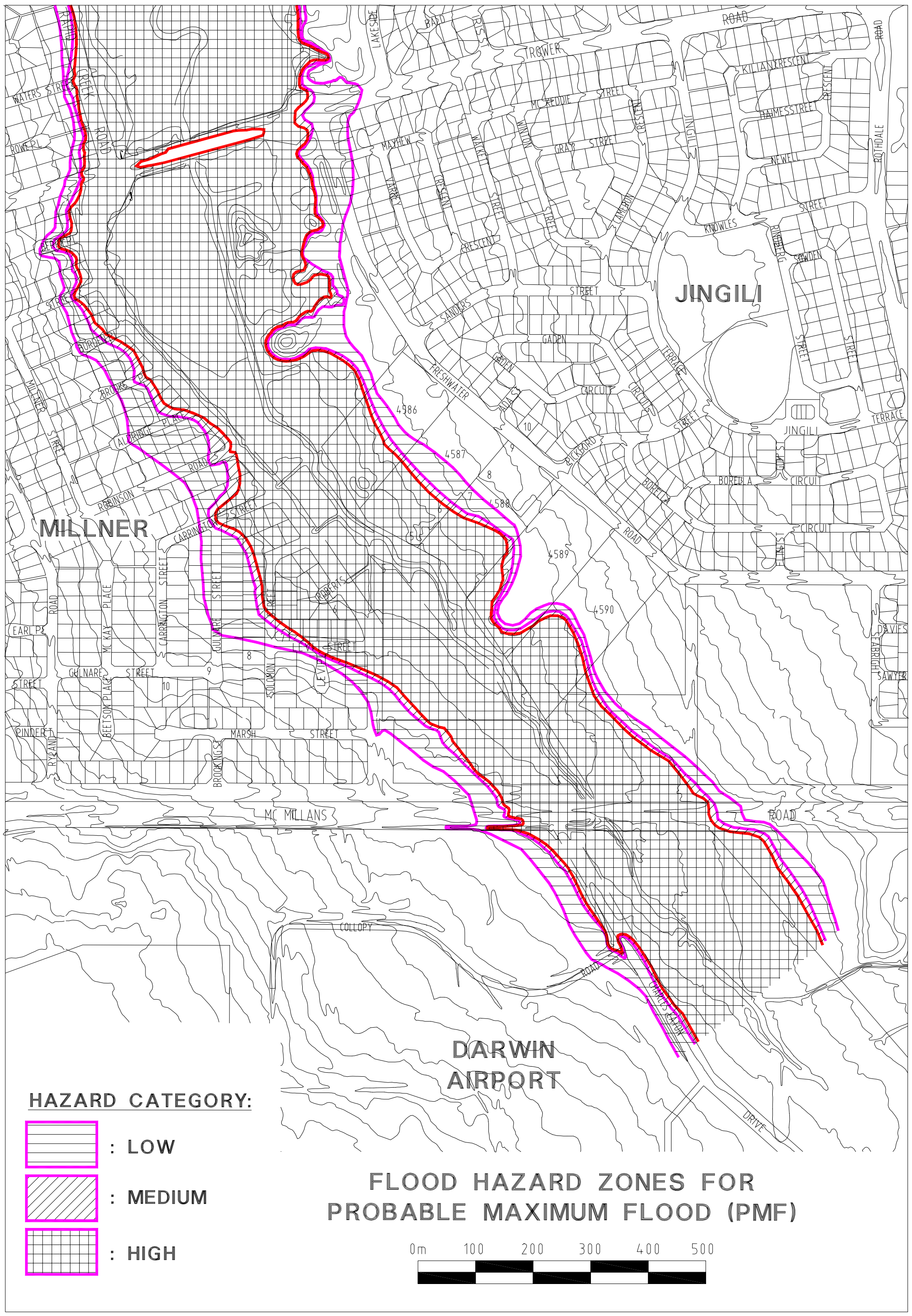


Figure 25, PROBABLE MAXIMUM FLOOD HAZARD ZONES



7. Impact of further development

As discussed in the previous Section 6, development in the Rapid Creek flood plain would impact on the distribution of floodwaters during major floods. It is likely that the flows would be deeper on the Millner side of the creek and the extent of flooding would be greater, affecting more residential allotments.

It is also likely that development will proceed within the airport grounds. In the last 10 to 20 years the extent of cleared, hardstanding and paved areas has increased within the airport with the construction of a new terminal building, development of general aviation facilities, car parks, etc. Speeding up of runoff has also occurred with previously slow flowing drainage lines being re-constructed as trapezoidal open drains and in some cases concreted.

The airport has recently passed to private ownership and it is possible that there will be applications for further development of peripheral airport land as commercial or light industrial sites. Much of the airport is within the Rapid Creek catchment and therefore such development will have a further impact on the speed and magnitude of runoff coming from the catchment. However, the soil types and the intensity of storms in Darwin are such that a high percentage of the rain that falls on undeveloped land can become runoff and therefore the changes that occur do not affect runoff from major floods as much as minor storm events.

An indication of the effect of such development can be found using the RORB hydrologic model and changing the parameters that govern the magnitude and timing of runoff generation. In RORB, areal variability of runoff-producing characteristics is introduced by a factor, called the runoff capacity index, that is used in the calculation of rainfall-excess (that fraction of the rainfall which becomes runoff) on each sub-area. The runoff capacity indices used in the model are given in table 18, together with increased values used in considering the following two development scenarios (see Figure 26):

1. future general increases in development in the airport grounds, including development in parts of sub-areas H, J and K which are fronting McMillans Road and along the main entry roads to the airport, to the extent where these areas are 12 to 15% paved, and

2. a further increase in development in parts of sub-areas H, J and K which are fronting McMillans Road and along the main entry roads to the airport to the extent where these areas become 35% paved, typical of fully built up and landscaped urban areas.

The lower part of the table shows the calculated change in peak discharges calculated by RORB for the 1 in 20 yr, 50 yr, and 100 yr floods for each of the two development scenarios.

The results in Table 18 indicate that with development around McMillans Road / Charles Eaton Drive up to the equivalent of 35% of the area being rendered impervious peak discharges can be expected to increase by up to 12 percent.

Figure 27 shows the way peak discharges could be expected to increase with development.

Figure 26, AREAS WHERE EXTENSIVE DEVELOPMENT IS LIKELY

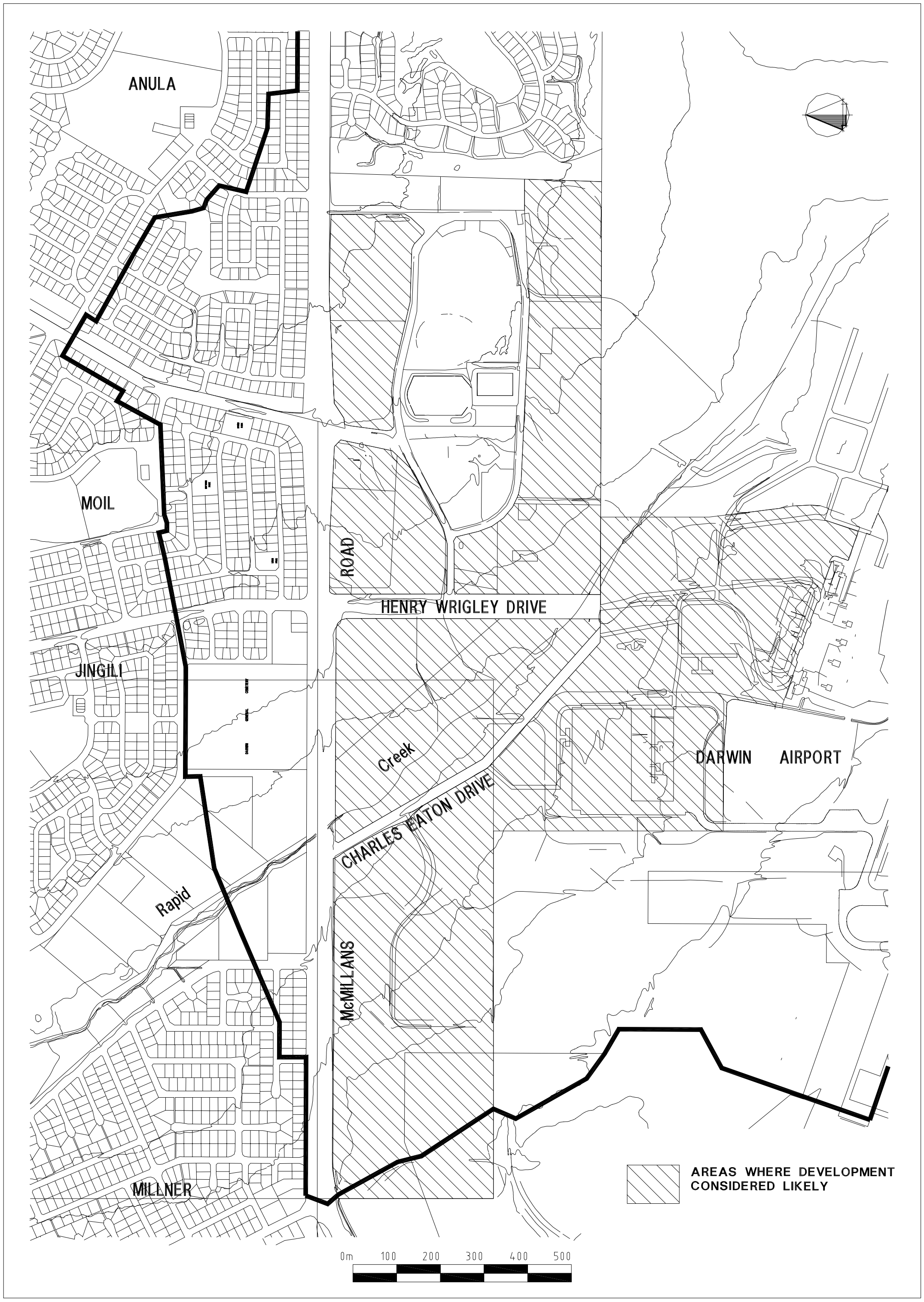
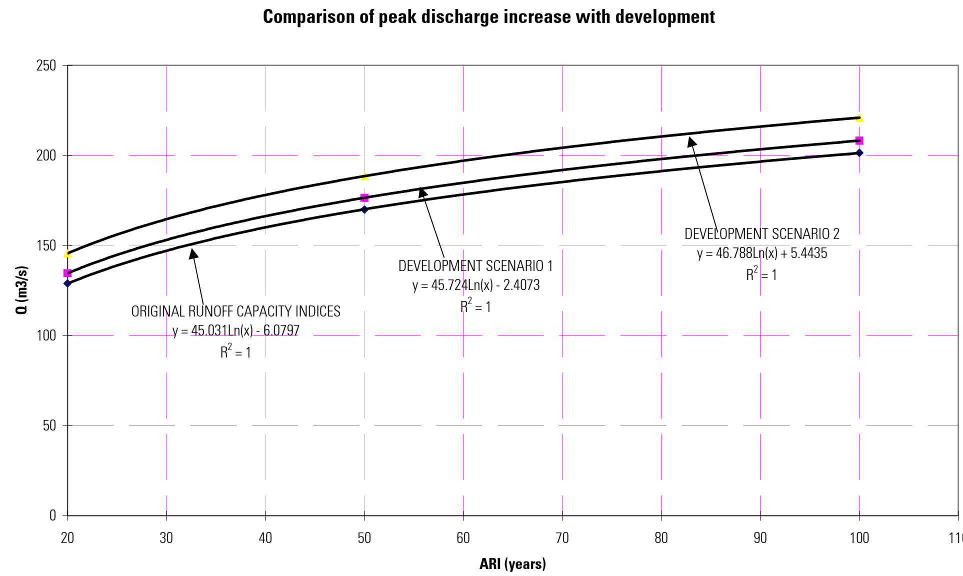


Table 18, RUNOFF CAPACITY INDICES AND THEIR EFFECT ON PEAK DISCHARGE AT G8150027 FOR A 1 IN 100 YEAR FLOOD

SUB-AREA	RUNOFF CAPACITY INDEX		
	ORIGINAL MODEL AS CALIBRATED	WITH FUTURE DEVELOPMENT	
		1. GENERAL INCREASE IN AIRPORT DEVELOPMENT	2. EXTENSIVE DEVELOPMENT IN MCMILLANS ROAD/CHARLES EATON DRIVE AREA
A	0.05	0.10	0.10
B	0.12	0.12	0.12
C	0.15	0.15	0.15
D	0.02	0.02	0.02
E	0.12	0.12	0.12
F	0.05	0.10	0.10
G	0.02	0.10	0.10
H	0.10	0.12	0.25
I	0.35	0.35	0.35
J	0.05	0.12	0.25
K	0.05	0.15	0.25
PEAK Q20 (m^3/s)	129	135	146
PEAK Q50 (m^3/s)	170	176	189
PEAK Q100 (m^3/s)	201	208	221

Figure 27, COMPARISON OF PEAK DISCHARGE INCREASES WITH DEVELOPMENT



Under the first development scenario, the increase in peak discharge ranges from 5.4% for the 1 in 100 year flood to 3.9% for the 1 in 20 year flood. The change in calculated flood levels would be small.

Under the second development scenario the peak discharges increase by up to 15 percent. The estimated increase in peak flood levels is shown in Table 19.

Table, 19, ESTIMATED INCREASE IN PEAK FLOOD LEVEL FOR DEVELOPMENT SCENARIO 2.

ARI (yrs)	Q (m^3/s)	Interpolated peak flood level at Section										
		10.000	10.210	10.420	10.560	10.740	10.930	11.130	11.330	11.425	11.600	11.820
20	146	+0.10	+0.11	+0.15	+0.15	+0.12	+0.11	+0.12	+0.18	+0.19	+0.21	+0.21
50	189	+0.10	+0.14	+0.11	+0.14	+0.14	+0.15	+0.17	+0.15	+0.13	+0.10	+0.08
100	221	+0.10	+0.13	+0.08	+0.15	+0.16	+0.16	+0.17	+0.15	+0.14	+0.14	+0.07

These rises have the potential to increase the lateral extent of flooding by some 30 to 40 m on the western or Millner side of Rapid Creek and therefore to inundate additional allotments and to increase the depth of flooding on allotments already indicated as liable to flooding in Figures 18, 19 and 20.

We also understand that the Department of Transport and Works has plans to raise one or both of the bridges that convey McMillans Road across Rapid Creek. Currently these bridges and their supporting structures impede the Rapid Creek flow and force floodwaters to accelerate through the bridge opening. These bridges are also frequently overtopped, when the flood flow exceeds the capacity of the bridge opening, perhaps once every 2 years or more. These factors result in an increase in water level upstream, known as bridge afflux. This rise in water level results in additional (temporary) storage of water upstream of the bridge constriction during the passage of a flood and this tends to attenuate the flood peaks

If the bridges are raised to reduce the frequency of overtopping the afflux will be reduced and the attenuation effect will be less. That is, flood peaks downstream of the bridge could be slightly higher.

However, the amount of temporary storage engaged by the bridge afflux is small compared to the volume of water in a major flood wave coming down Rapid Creek. Any increase in downstream flood peak is therefore likely to be small for intermediate floods and insignificant for rare floods such as those considered in this report. However, such effects should be checked during the design development for raising of the bridges.

8. References

1. Australian Rainfall and Runoff. A guide to Flood Estimation. Volume 1. The Institution of Engineers, Australia, Canberra, 1987

2. RORB-Version 3 Runoff Routing Program, User Manual. E. M. Laurenson and R. G. Mein Publ. In conjunction with ACADS, August, 1983

3. Rapid Creek Hydrology Study. Cameron McNamara for NT Government, 1982

4. Rapid Creek Management Plan. Draft, Clouston, 1994

5. The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method. Bureau of Meteorology, Bulletin 53, Australian Government Publishing Service, Canberra, 1994.

Appendix A

Example RORB data file.

RAPID CREEK FLOOD STUDY

REVISED FINAL REPORT

CORRIGENDUM 07 OCTOBER 1999

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1. Replace Table 5, page 15, with Table 5 attached.
2. Replace Sections 4.3 and 4.4, pages 25 to 29 inclusive, with Sections 4.3 and 4.4 attached.
3. In Section 5.3.3, page 43 replace "(compared to a 1 in 100 year flood of 220 m3/sec)" with "(compared to a 1 in 100 year flood of 201 m3/sec)"
4. Replace Table 18, page 55, with Table 18 attached.
5. Replace Figure 27 and Table 19, page 56, with Figure 27 and Table 19 attached.

Peter Saunders

Senior Engineer

07/10/99

TABLE 5, ANNUAL SERIES OF PEAK DISCHARGES RECORDED AT G8150127

Rank	Date of flood	Peak discharge (m3/s)		
		As recorded	If weir existed pre-1987	If weir not constructed in 1987
col 1	Col 2	col 3	col 4	col 5
1	25-12-74	116	82.5	116
2	05-01-91	112	112	164
3	16-03-77	103	75.7	103
4	03-01-97	101	101	103
5	22-01-81	88.9	53.6	88.9
6	10-03-83	87.1	47.5	87.1
7	22-01-82	80.3	44.5	80.3
8	21-01-80	71.2	39.1	71.2
9	21-01-98	66.2	66.2	(20/01/98) 104
10	28-12-93	60.4	60.4	105
11	25-01-93	57.0	57.0	85.5
12	01-03-95	49.3	49.3	73.9
13	18-02-84	45.0	24.5	45.0
14	13-04-85	43.1	20.2	43.1
15	24-02-74	42.6	21.3	42.6
16	05-02-69	41.9	21.6	41.9
17	09-12-95	41.4	41.4	62.1
18	01-03-72	35.0	18.3	35.0
19	10-02-87	34.9	26.5	34.9
20	02-01-79	34.4	16.0	34.4
21	02-12-88	33.0	33.0	49.5
22	11-01-86	32.8	15.1	32.8
23	31-12-87	23.2	23.2	34.7
24	11-03-76	20.0	10.1	20.0
25	06-03-71	16.3	8.00	16.30
26	27-01-78	16.0	8.30	16.00
27	28-01-73	12.6	5.90	12.60
28	06-01-92	12.4	12.4	18.6
29	30-12-65	9.79	4.80	9.79
30	26-01-64	7.76	3.60	7.76
31	26-03-65	7.43	3.72	7.43
32	11-02-70	6.70	5.09	6.70
33	15-01-90	5.98	5.98	8.97
34	15-12-66	1.09	0.83	1.09
35	01-09-67	0.00	0.00	0.00

The good results for two out of three floods suggests adoption of the parameters Kc=10 and m=0.6 in design runs. The result is not as good for the flood of 23 December 1996 but there is strong evidence that this result must be a deficiency in the rainfall information (or a water level recorder malfunction).

It can also be shown that good results can be obtained by going back over the five storms used in the calibration runs and fixing the parameters at Kc and m =10 and 0.6 respectively. To achieve this volumes were allowed to vary such that calculated and observed hydrographs were not necessarily equal, but runoff coefficient varied over a narrow band between 0.55 and 0.65 as shown in Table 10.

TABLE 10, RESULTS FOR CALIBRATION FLOODS RUN WITH KC=10, M=0.6, INITIAL LOSS-RUNOFF COEFFICIENT MODEL & RUNOFF VOLUMES ALLOWED TO BE UNEQUAL

Flood date	Initial Loss (mm)	Runoff coefficient (C)	Average absolute ordinate error (indicates goodness of fit)	Calculated hydrograph peak (m^3/s)	Recorded hydrograph peak (m^3/s)	Percentage error in calculated hydrograph volume
05/01/91	20	0.55	31.8	110	112	+31%
08/01/98	5	0.65	23.4	59	61	+1%
28/12/93	0	0.60	44.3	56	60	+39%
03/01/97	0	0.65	12.9	102	102	-1%
01/03/97	0	0.65	22.5	75	75	+12%

Since this reflects the way in which the model will be used in design and predicts peak discharges very well, it is considered to add further support for the adoption of these parameters.

Therefore Kc=10, m=0.6 and the use of the initial loss-runoff coefficient model with initial loss zero is proposed for use in design runs.

4.3 Results of flood frequency analysis

As discussed in Section 3.4, flood frequency analysis was carried out for:

> the annual series of peak discharges recorded at gauging station G8150127,

> an annual series with pre-1987 peak flows adjusted to reflect post-1987 conditions,

> an annual series with post-1987 peak flows adjusted to reflect pre-1987 conditions.

4.3.1 Observed annual floods

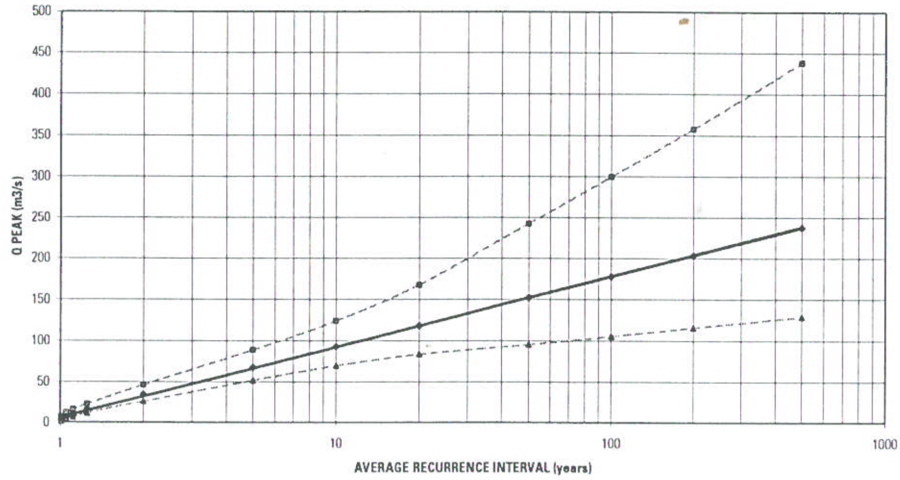
Figure 12(a) shows the flood frequency curve derived from the annual series of peak discharges recorded at gauging station G8150127. The derived parameters of the flood frequency distribution are:

Figure 12. FLOOD FREQUENCY CURVES

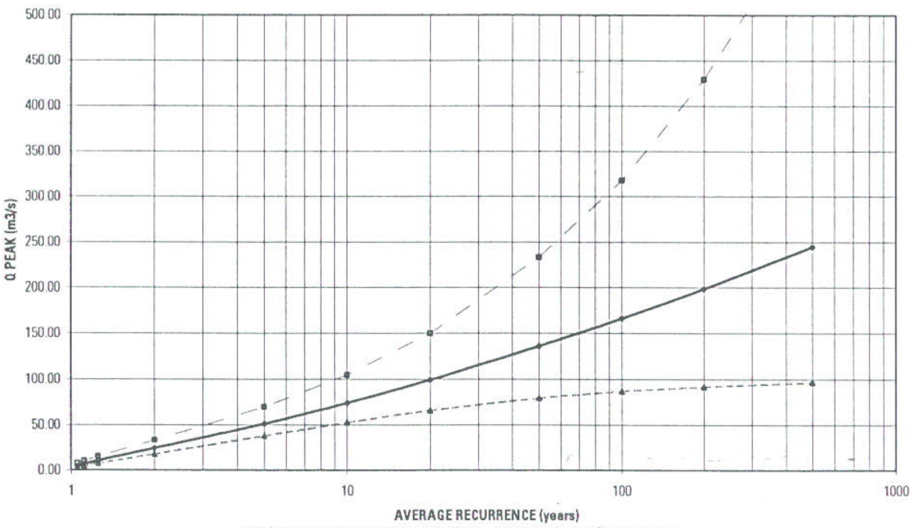
NOTE:

1. PLOTS ON LOG-NORMAL PROBABILITY PAPER ARE AVAILABLE IN THE CALCULATIONS
2. EXPECTED PROBABILITY ADJUSTMENT IS NOT SHOWN
3. DASHED LINES ARE 5% AND 95% CONFIDENCE LIMITS

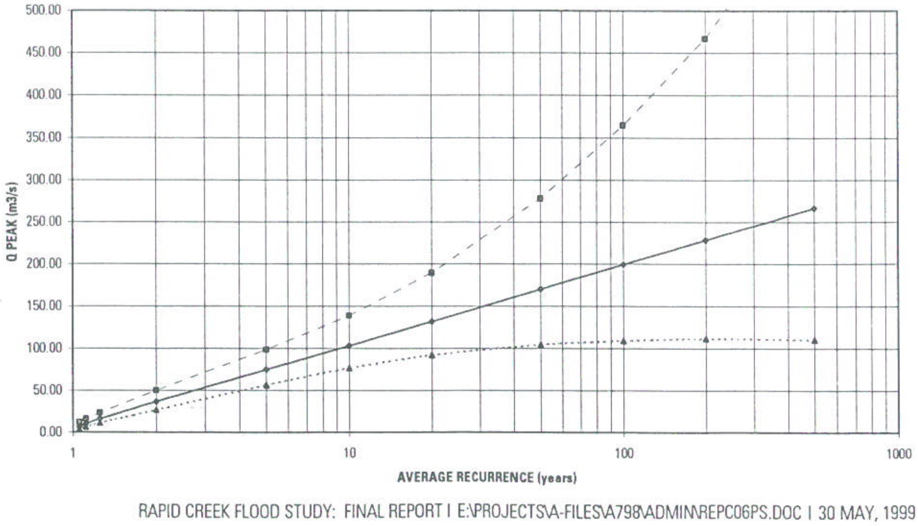
12(a). FLOOD FREQUENCY CURVE - OBSERVED SERIES



12(b) FLOOD FREQUENCY CURVE - OPTION 1 ADJUSTMENT



12(c) FLOOD FREQUENCY CURVE - OPTION 2 ADJUSTMENT



- > Skewness of the logarithms of peak flows Gs -0.4885
- > Standard deviation of the logarithms of peak flows Ss 0.3789
- > Mean of the logarithms of peak flows Ms 1.5055.

The design flood frequencies are shown in Table 11.

4.3.2 *Annual flood series for post-flood-control-weir conditions (Option 1)*

Figure 12(b) shows the results of the flood frequency analysis using the annual peak flow series formed by reducing the pre-1987 peaks to allow for the effect of the flood control weir.

The parameters of this distribution are:

- > Skewness of the logarithms of peak flows Gs -0.2807
- > Standard deviation of the logarithms of peak flows Ss 0.4144
- > Mean of the logarithms of peak flows Ms 1.3424.

The peak flow values are less on average than for the original series.

The resulting design flood frequencies are shown in table 11.

4.3.3 *Annual series for pre-flood-control-weir conditions (Option 2)*

Figure 12(c) shows the results of the flood frequency analysis using the annual peak flow series formed by increasing the post-1987 peaks to remove the effect of the flood control weir.

The parameters of the LP3 fitted flood frequency distribution for this case are

- > Skewness of the logarithms of peak flows Gs -0.5325
- > Standard deviation of the logarithms of peak flows Ss 0.3988
- > Mean of the logarithms of peak flows Ms 1.5299.

As expected, the mean of the logarithms of peak discharges are higher, reflecting the absence of the flood control weir. The standard deviation and the skewness similar to previous values for the observed series. Hydrographs with peak flows that match these peak flows were calculated by adjusting the parameters in the RORB model. Then, the flood control weir was put back into the model and design hydrographs calculated by RORB using the same parameters. The resulting peak flows are also shown in table 11.

TABLE 11, PEAK DISCHARGES FOR DESIGN FLOODS DERIVED FROM FLOOD FREQUENCY ANALYSIS

AVERAGE RECURRENCE INTERVAL (ARI) (YEARS)	PEAK DISCHARGE (M3/S)		
	OBSERVED FLOODS	ACCOUNT FOR FLOOD CONTROL WEIR BY OPTION 1	ACCOUNT FOR FLOOD CONTROL WEIR BY OPTION 2 (bracketted figures are peak flows before the flood control weir is put back into the model)
20	129	107	107 (140)
50	170	154	139 (188)
100	201	197	167 (232)

Whilst the option 2 method of adjustment may be preferred from the theoretical view point it produces results that are considerably lower than those from the option 1 method and those from flood frequency analysis directly on the observed series.

There are some uncertainties in performing analysis on series that have been adjusted by either method. The biggest problem would appear to be that the calculated effect of the flood control weir depends on the hydrograph shape and the distribution of rainfall over the catchment. When using the option 2 method, uniform rainfall is used to calculate the hydrograph and this produces one particular hydrograph shape, for which the peak is attenuated by some 25 to 30 percent. However uniform rainfall is unlikely. Real storms always produce some variation over the catchment. Therefore the attenuation of peak hydrographs should vary over a larger range(see Table 10.)

Given the uncertainty in the methods used to adjust the series for inhomogeneity, it is considered reasonable to adopt the results from flood frequency analysis directly on the observed series that is, Q100=201 m^3/s, Q50=170 m^3/s and Q20=129 m^3/s.

4.4 Design flood hydrographs

Australian Rainfall and Runoff Chapter 12 gives a guide to the choice between rainfall-runoff modeling and flood frequency analysis as the method of flood estimation. Using the procedures described therein, the predicted flood peaks should match those indicated by the flood frequency analysis for average recurrence intervals up to:

$Y \text{ (years)} = 1.5 \times F \times N^{0.75} \times e^{0.06N}$,

where F is given in Tables in Australian Rainfall and Runoff and N is the number of years of stream flow records. Tables 12.1 and 12.2 of Australin Rainfall and Runoff with:

- 35 years of records,
- a standard deviation of the logarithms of peak flows of 0.38,
- a skewness of the logarithms of peak flows of –0.49,
- zone 2 in AR&R Figure 12.1, and
- Class 1 flood estimation method,

give a value of Y = 260 years, which suggests the peak discharge of the design hydrographs should match the discharges indicated by the flood frequency analysis at least up to and including a 1 in 260 year flood.

Design flood hydrographs were calculated using the calibrated and verified RORB model with runoff coefficients adjusted to produce peaks of the same order as those predicted from the flood frequency analysis using option 2 to account for the effect of the flood control weir (see Table 11).

Storms of 1 hour duration were found to be critical in all cases. The runoff coefficients were 1.00, 1.09 and 1.12 for the 1 in 20 year, 1 in 50 year and 1 in 100 year design storms respectively. Runoff coefficients are greater than 1 because the shape of the design hydrographs is determined by the calibrated and verified RORB model but the flood frequency analysis is used to determine the peak magnitude.

It may be possible to further refine the flood frequency analysis. This could involve extensive modeling of flood events before 1987 to prepare a more accurate 35 year annual series that includes the effect of the weir, or modeling the remaining seven events after 1987 to increase confidence in the series that does not include the effect of the weir. In the latter case, there remains the problem of converting design flood hydrographs (based on uniform rainfall) to post-weir conditions.

However, the refinement may be within the limits of pre 1987 data accuracy and it is considered that further investigation is not warranted at this stage.

Design hydrographs calculated from 1 in 20, 50 and 100 year 1 hour storms are shown in Figure 13.

Figure 13, 1 IN 20, 50 AND 100 YEAR DESIGN FLOOD HYDROGRAPHS

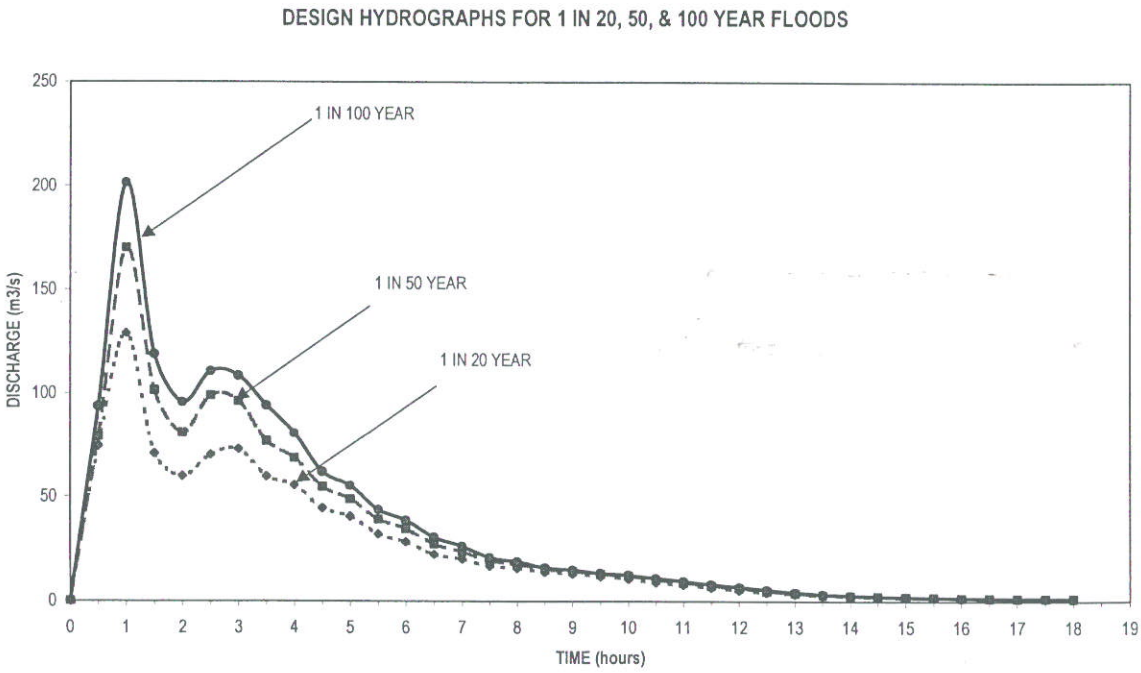
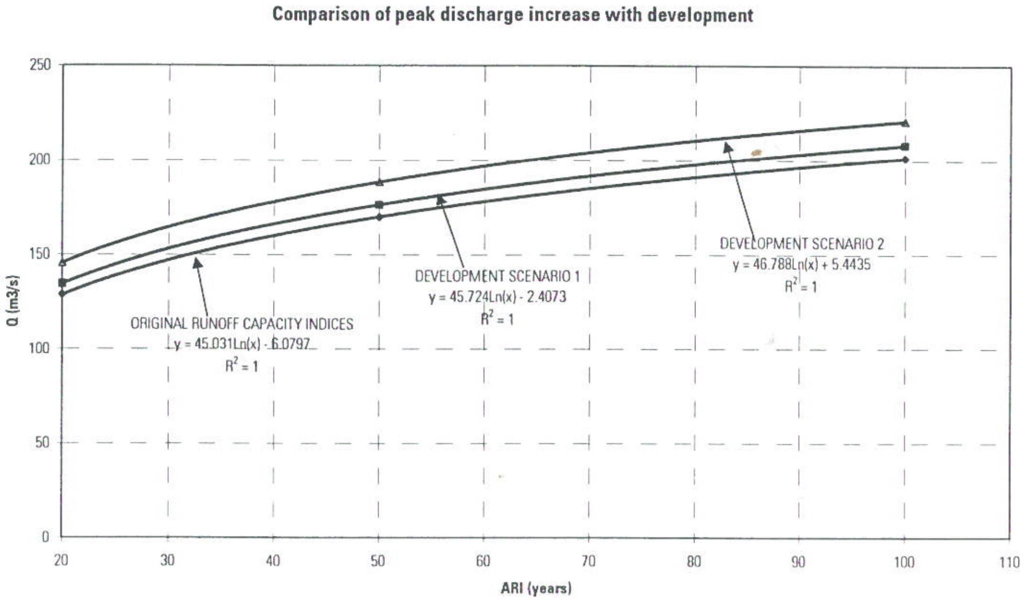


Table 18, RUNOFF CAPACITY INDICES AND THEIR EFFECT ON PEAK DISCHARGE AT G8150027 FOR A 1 IN 100 YEAR FLOOD

SUB-AREA	RUNOFF CAPACITY INDEX		
	ORIGINAL MODEL AS CALIBRATED	WITH FUTURE DEVELOPMENT	
		1. GENERAL INCREASE IN AIRPORT DEVELOPMENT	2. EXTENSIVE DEVELOPMENT IN MCMILLANS ROAD/CHARLES EATON DRIVE AREA
A	0.05	0.10	0.10
B	0.12	0.12	0.12
C	0.15	0.15	0.15
D	0.02	0.02	0.02
E	0.12	0.12	0.12
F	0.05	0.10	0.10
G	0.02	0.10	0.10
H	0.10	0.12	0.25
I	0.35	0.35	0.35
J	0.05	0.12	0.25
K	0.05	0.15	0.25
PEAK Q20 (m^3/s)	129	135	146
PEAK Q50 (m^3/s)	170	176	189
PEAK Q100 (m^3/s)	201	208	221

Impact of further development

Figure 27, COMPARISON OF PEAK DISCHARGE INCREASES WITH DEVELOPMENT



Under the first development scenario, the increase in peak discharge ranges from 5.4% for the 1 in 100 year flood to 3.9% for the 1 in 20 year flood. The change in calculated flood levels would be small.

Under the second development scenario the peak discharges increase by up to 15 percent. The estimated increase in peak flood levels is shown in Table 19.

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ARI (yrs)	Q (m^3/s)	Interpolated peak flood level at Section										
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These rises have the potential to increase the lateral extent of flooding by some 30 to 40 m on the western or Millner side of Rapid Creek and therefore to inundate additional allotments and to increase the depth of flooding on allotments already indicated as liable to flooding in Figures 18, 19 and 20.

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