SINCLAIR KNIGHT MERZ
SKM

Northern Territory Government

Rapid Creek Flood Study

STUDY REPORT

- E
- 6 December 2013

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Executive Summary

Rapid Creek experienced unprecedented flooding caused by heavy rainfall over Darwin in February 2011, during the formation of Cyclone Carlos. A rainfall of 340 mm was recorded at Darwin Airport on 15 February, 2011 and the gauge on Rapid Creek downstream of McMillan's Road recorded a peak height of 3.74 m on the same day. The highest peak attained at this gauge before this event was in March 1977 when a gauge height of 3.67 m was recorded. A number of houses in the suburb of Rapid Creek were affected and some were seriously damaged by flooding during this event.

This study examines flooding for Rapid Creek using a hydrology study to define peak flows likely to occur in Rapid Creek for a range of design flow scenarios, a hydraulic study to calculate peak water levels associated with those peak flows and preparing flood plain maps that depict the extent and depth of inundation associated with those flows.

The design flow scenarios included a number of floods that have a low probability of occurrence and the probable maximum flood. These were examined for a range of conditions at the Rapid Creek sea outfall at Casuarina Beach, including scenarios that adopt a 0.8 m sea level rise to year 2100.

Hydrologic analysis consisted of a flood frequency analysis and rainfall-runoff modelling using the URBS model. The parameters derived from URBS model calibration runs were assessed against regional prediction equations and the URBS Basic model was used for design runs to calculate flows for input into the hydraulic modelling.

Hydraulic analysis was carried out using the TUFLOW model. The model was calibrated against observations of flood height during the flood event of 16 February 2011 (associated with the formation of Cyclone Carlos). After calibration, the TUFLOW model was run for design scenarios with inputs from the URBS design runs and an appropriate sea outfall level.

The flood plain maps are presented as an Appendix.

The flood plain maps show that during a Q100 flood event:

- Henry Wrigley Drive is overtopped on the northern side of the culverts and there is significant overtopping at McMillans Road (flooding over McMillans Road occurs for floods as low as Q2 to Q5)
- Between the Rapid Creek gauging station and Trower Road, there is expansion of floodwaters into the low-lying areas on the left and right overbanks. A breakout of flow on the right overbank near the gauging station causes flooding of an existing 'rural' property and a number of existing residential properties on the western side of Rapid Creek Road are affected

- Trower Road is overtopped at the Rapid Creek Road intersection and at a second location approximately 275m to the east (near the Freshwater Road intersection)
- Floodwaters downstream of Trower Road are confined to the creek and mangrove overbank areas and to the constriction at the outlet
- Floodwaters from the university open channel catchment threaten a number of existing structures on the university campus

The flood plain maps show that during the Probable Maximum Flood event:

- The extent of flooding between the flood control weir and the Rapid Creek gauging station is generally 400m to 450m in width and affects a number of existing developments and all of the road crossings
- The extent of flooding increases to approximately 700m in width in the area between the gauging station and Trower Road and affects a widespread number of existing properties on the western side of Rapid Creek Road (in Millner) and the eastern side of Freshwater Road (in Jingili)
- Downstream of Trower Rd there is inundation of properties adjacent Rapid Creek Road and Lakeside Drive and increased inundation of the university campus
- At the outlet to the sea there is a breakout of flow to the north of the outlet constriction

Rapid Creek is a unique community asset that provides passive recreation and water play opportunities within a built up urban environment. As such any flood mitigation works should be unobtrusive and not involve extensive channel works.

Raising of the existing flood control weir has been examined and is unlikely to have a significant impact on flooding downstream in the suburbs of Millner and Rapid Creek.

It is likely that flood mitigation solutions will comprise of a combination of removing obstructions to the flow, including enlarging bridge openings and construction of minor levees or flood walls. Measures to mitigate the impact of flooding should be investigated.

1. Introduction

1.1. General

Sinclair Knight Merz has been commissioned by the former NT Government Department of Natural Resources, Environment, the Arts and Sport, now the Department of Land Resource Management (DLRM) to examine flooding of Rapid Creek.

The study brief (see Appendix B) requires:

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

The scenarios required are shown in Table 1.

Table 1. Flood scenarios considered

Sea level condition	Annual Exceedance Probability (AEP) of flood (Average Recurrence Interval, years)					
	5% (20)	2% (50)	1% (100)	0.5% (200)	0.2% (500)	Probable Maximum Flood
Current (2012) mean sea level	✓	✓	✓	✓	✓	✓
Current (2012) HAT[1]			✓		✓	✓
Current (2012) mean sea level plus 100 yr ARI storm surge[2]	✓		✓			
Future[3] (2100) HAT			✓		✓	✓

This report discusses the hydrologic study and hydraulic modelling that has calculated the flood water surface profiles for these scenarios. It presents the resulting flood plain maps and briefly discusses the findings and implications for flood plain management.

[1] HAT is Highest Astronomical Tide
[2] Storm surge is taken from Darwin Area Storm Surge Inundation Maps at the NT Government website
[3] The sea level rise to 2100 is specified by the brief as 0.8 m

1.2. Reliance Statement

The sole purpose of this report and the associated services performed by Sinclair Knight Merz Pty Ltd (SKM) is to in accordance with the scope of services set out in the contract between SKM and the former Department of Natural Resources, Environment, The Arts and Sport (NRETAS). That scope of services, as described in this report, was developed with NRETAS.

In preparing this report, SKM has relied upon, and presumed accurate, certain information (or absence thereof) provided by the Client and other sources. Except as otherwise stated in the report, SKM has not attempted to verify the accuracy or completeness of any such information. If the information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions as expressed in this report may change.

SKM derived the data in this report from a variety of sources. The sources are identified at the time or times outlined in this report. The passage of time, manifestation of latent conditions or impacts of future events may require further examination of the project and subsequent data analysis, and re-evaluation of the data, findings, observations and conclusions expressed in this report. SKM has prepared this report in accordance with the usual care and thoroughness of the consulting profession, for the sole purpose of the project and by reference to applicable standards, procedures and practices at the date of issue of this report. For the reasons outlined above, however, no other warranty or guarantee, whether expressed or implied, is made as to the data, observations and findings expressed in this report.

This report should be read in full and no excerpts are to be taken as representative of the findings. No responsibility is accepted by SKM for use of any part of this report in any other context.

Assumptions and limitations are further discussed in this report together with recommendations for further data collection and analyses that may improve confidence in the results.

This report has been prepared on behalf of, and for the exclusive use of the former NRETAS, now DLRM, and is subject to, and issued in connection with, the provisions of the agreement between SKM and NRETAS. SKM accepts no liability or responsibility whatsoever for, or in respect of, any use of, or reliance upon, this report by any third party.

2. Hydrology Study

2.1. Rapid Creek catchment area

Rapid Creek rises in the Marrara Swamp at the eastern end of Darwin Airport, and flows for 9.8 km discharging into the sea (Beagle Gulf) at the southern end of Casuarina Beach (Refer Figure 1). The Rapid Creek catchment covers an area of 28 sq km and includes parts of suburbs of Karama, Malak, Anula, Moil, Jingili, Wulagi, Alawa, Casuarina, Wanguri, Nakara and Brinkin, Millner and Rapid Creek. In these built up areas of the catchment runoff enters the creek via underground piped drainage systems as well as unlined and lined open drains. Large parts of the catchment to the south of McMillans Road are still undeveloped. The Marrara Swamp is drained by two separate drainage lines, one on the north western and the other on the south western side of the swamp. Where the two drainage lines re-join to form Rapid Creek, a flood control weir exists which attenuates the peak discharge and delays the floodwaters. The flood control weir was constructed in 1985.

2.2. Methodology

The approach taken for the hydrology study involved the following steps:

Flood frequency analysis

1) Derive a series of annual maximum peak discharges recorded at the Rapid Creek gauging station
2) Fit a flood frequency distribution to the series and estimate Q20, Q50, Q100, Q200 and Q500 flood peaks
3) Using the URBS model (refer Section 2.4) with the flood control weir included[4], estimate how peak annual floods would have been attenuated for floods before 1985 and derive an amended annual series for post-weir conditions
4) Fit a flood frequency distribution to the adjusted series and estimate Q20, Q50, Q100, Q200 and Q500 flood peaks
5) Compare the flood frequencies for the observed flood series and the adjusted series and estimate peak discharges for Q20, Q50, Q100, Q200 and Q500

[4] Flows (discharges) have been recorded at the stream gauging station G8150127 since 1963/64 and the flood control weir was constructed upstream of the gauging station in 1985. This makes a discontinuity in the recorded annual series because the flood control weir will attenuate flood peaks. The flood control weir commands approximately two thirds of the catchment area above the gauging station so this attenuation can be significant for a particular largest flood of each year, depending on its size and nature.

Hydrologic modelling

6) Establish an URBS rainfall-runoff model
7) Calibrate the URBS model using recorded data for a selection of floods on Rapid Creek
8) Review the calibrations, compare the results with regional flood estimation methods and estimate the best parameters to use in the URBS model for design runs[5]
9) Run the URBS model in design mode using the best parameter set, together with appropriate loss values to simulate peak discharges consistent with flood frequency analysis
10) Run the URBS model is design mode with the best parameter set and using estimated probable maximum precipitation to produce hydrographs for probable maximum flood

This procedure is commonly used in flood studies and is designed to give increased confidence in the calibrated model and its parameters by adoption of parameters consistent with regionally-derived values and use of loss values consistent with observations.

2.3. Flood frequency analysis

2.3.1. Data available, assumptions and limitations

A stream gauging station (G8150127) was established on Rapid Creek in the 1960s approximately 500 m downstream of McMillans Road.

The catchment area above this gauging station is 18.7 sq km or two thirds of the total catchment area to the sea. Continuous recording of water level in the creek commenced in the early 1960s. These levels are converted to flows by a series of 10 rating curves from 1961/2 to the present. These rating curves are created by a series of current meter measurements or 'gaugings' carried out during floods. The largest gauging was at a water level of 3.23 m during the flood that occurred in January 1981. This is equivalent to a flow of 83 m^3/sec, which is approximately half the peak flow during the largest recorded flood. This gives reasonable confidence in the rating curves provided by DLRM for the study.

The highest flow in each year was extracted from the records to form a series of peak water year discharges as shown in Table 2.

[5] The regional flood estimation methods considered are based on runoff routing for a number of catchments in the Top End region or climatically similar regions. In theory, these provide increased confidence in the derivation of routing parameters, because it may average out data errors and peculiarities of individual; catchments

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- **Table 2. Annual flow series for G8150127**

Wet season year		Peak flow (m3/sec)
1963	64	7.86
1964	65	7.43
1965	66	9.98
1966	67	1.08
1967	68	0.00
1968	69	42.5
1969	70	6.93
1970	71	16.2
1971	72	36.7
1972	73	13.5
1973	74	43.0
1974	75	118
1975	76	20.0
1976	77	104
1977	78	16.0
1978	79	35.9
1979	80	71.2
1980	81	89.4
1981	82	83.0
1982	83	89.4
1983	84	45.1
1984	85	43.7
1985	86	37.6
1986	87	34.9
1987	88	23.8

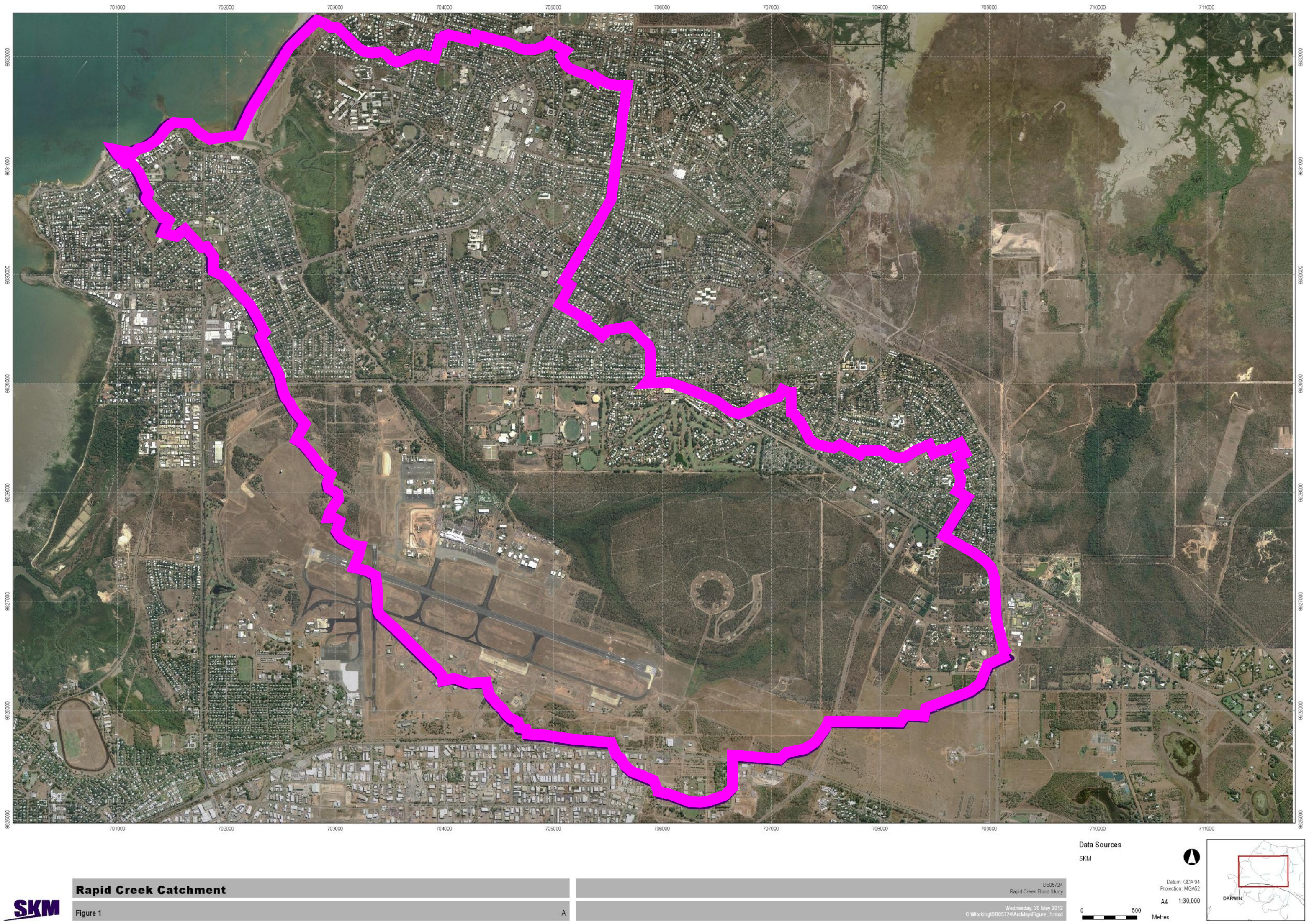
Wet season year		Peak flow (m3/sec)
1988	89	22.4
1989	90	6.51
1990	91	108
1991	92	15.6
1992	93	59.3
1993	94	62.4
1994	95	54.3
1995	96	45.0
1996	97	99.8
1997	98	89.7
1998	99	50.3
1999	2000	60.1
2000	1	27.5
2001	2	18.5
2002	3	70.7
2003	4	34.8
2004	5	23.3
2005	6	57.9
2006	7	54.6
2007	8	86.6
2008	9	27.2
2009	10	52.7
2010	11	157
2011	12	75.0

2.3.2. Results

The Generalised Extreme Value distribution has been fitted to this series in order to find the peak discharges for the probabilities of interest. Values of LH moment shift from zero to four were tried and best results were for a value of zero.

The resulting flood frequency plots are presented in the Hydrology Report (SKM, 2012).

- **Figure 1. Rapid Creek Catchment**



A discontinuity exists in the annual series because of construction of the flood control weir in 1985.

To examine the effect of this a second annual series was constructed for post-flood control weir conditions. For the largest flood in each year before construction of the weir, the URBS model was used to derive a flood peak at G8150127 as if the weir had existed from the start of the series. This series is shown in Table 3.

- **Table 3. Annual flow series for G8150127 adjusted to post-weir conditions**

Wet season year		Peak flow (m3/sec)
1963	64	5.15
1964	65	4.86
1965	66	6.56
1966	67	0.70
1967	68	0.00
1968	69	30.1
1969	70	4.53
1970	71	10.8
1971	72	26.8
1972	73	8.98
1973	74	30.4
1974	75	95.7
1975	76	13.5
1976	77	80.8
1977	78	10.7
1978	79	34.1
1979	80	50.1
1980	81	68.9
1981	82	57.8
1982	83	66.4
1983	84	31.8
1984	85	24.6
1985	86	37.6
1986	87	34.9
1987	88	23.8

Wet season year		Peak flow (m3/sec)
1988	89	22.4
1989	90	6.51
1990	91	108
1991	92	15.6
1992	93	59.3
1993	94	62.4
1994	95	54.3
1995	96	45.0
1996	97	99.8
1997	98	89.7
1998	99	50.3
1999	2000	60.1
2000	01	27.5
2001	02	18.5
2002	03	70.7
2003	04	34.8
2004	05	23.3
2005	06	57.9
2006	07	54.6
2007	08	86.6
2008	09	27.2
2009	10	52.7
2010	11	157
2011	12	75.0

Flood frequency analysis was carried out for the resulting series and the results are presented in the Hydrology Report (SKM, 2012).

- **Table 4. Results of flood frequency analysis on annual series adjusted to post-weir conditions**

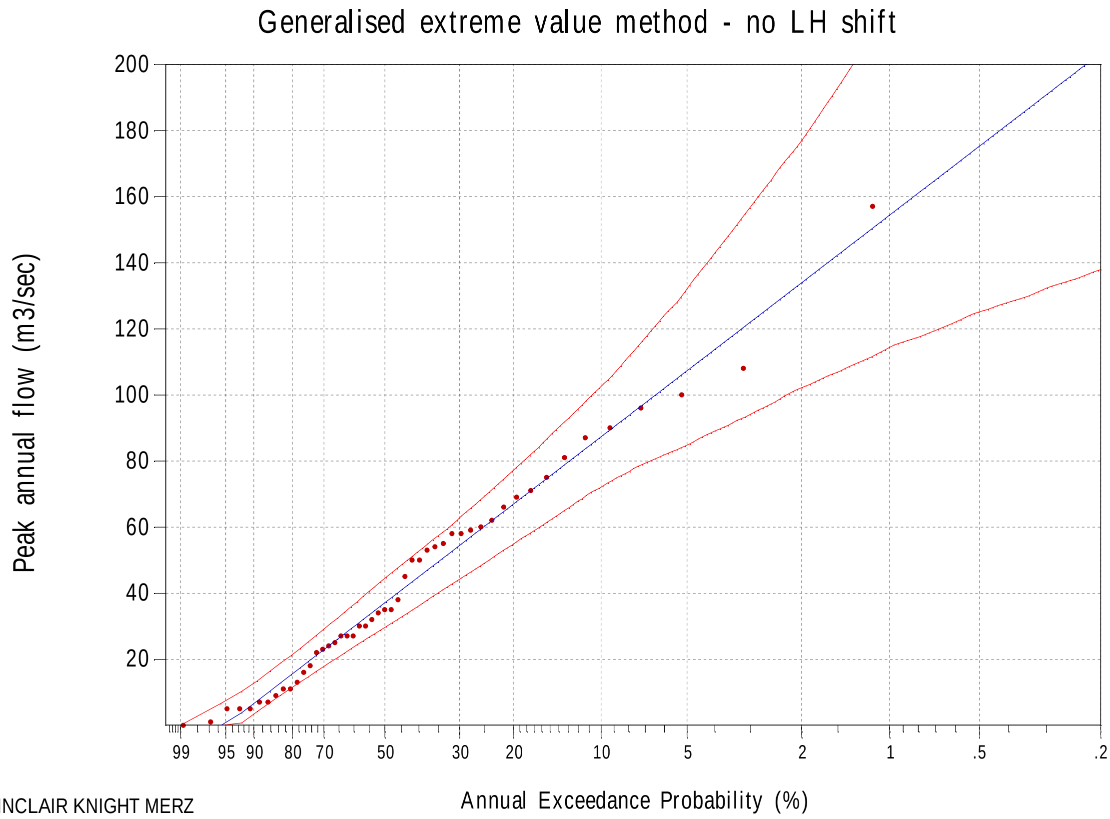
Average Recurrence Interval (yrs)	Annual Exceedance Probability (AEP)	Qpeak (m3/sec)	Lower Confidence. Interval	Upper Confidence Interval
2	50%	**38**	31	46
5	20%	**68**	56	78
10	10%	**88**	72	103
20	5%	**108**	86	131
50	2%	**134**	102	174
100	1%	**155**	114	217
200	0.5%	**176**	124	268
500	0.2%	**204**	134	341

The calculated frequencies are similar for varying LH moment shift all plot in a narrow band with the exception of the curve for LH shift zero for the raw data series.

It can be argued that the series adjusted to post-weir conditions should be adopted because it reflects current conditions. The adopted flood frequencies are those for the adjusted series with LH shift zero as bolded in Table 4.

The adopted flood frequency curve is shown in Figure 2.

- **Figure 2. Adopted flood frequency curve**



2.4. Hydrologic modelling

2.4.1. Data available, assumptions and limitations

Calibration involves running the model with observed rainfall data to produce calculated hydrographs at a location where they can be compared to recorded hydrographs. For Rapid Creek hydrographs from the stream gauging station (G8150127) located 500 m downstream of the bridge over McMillans Rd (the Kimmorley Bridge) are used.

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

- Flows at G8150127
- Sufficient rainfall data to describe how rain varied over the catchment area during the storm. Typically rainfall depths come from Bureau of Meteorology daily-read rain gauges
- Sufficient rainfall data to describe how rain varied in time over the catchment area. Typically this comes from pluviometers which are operated by the Bureau or by DLRM

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

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

Table 5 shows the dates of the storms selected, their peak discharge and the data used in the model calibration runs, where 014015 is the Bureau of Meteorology weather station at Darwin Airport, and R8150321, R8150256 and R8150257 are tipping bucket pluviometers operated by DLRM.

Table 5. Flood events selected for calibration and available rainfall data

Date of flood	Recorded peak at G8150127 (m3/sec)	Available daily rainfall station data												Available pluviometer rainfall data			
		014105	014112	014144	014164	014214	014216	014235	014246	014261	014265	014270	014280	014105	R8150231	R8150256	R8150257
5/02/69	42.6	✓	✓		✓									✓			
1/03/72	35.5	✓	✓		✓									✓			
25/12/74	120	✓												✓			

Date of flood	Recorded peak at G8150127 (m3/sec)	Available daily rainfall station data												Available pluviometer rainfall data			
		014105	014112	014144	014164	014214	014216	014235	014246	014261	014265	014270	014280	014105	R8150231	R8150256	R8150257
16/03/77	104	✓												✓			
21/01/80	71.5	✓	✓			✓								✓			
22/01.81	89.5	✓	✓			✓								✓			
22/01/82	82.7	✓	✓			✓								✓			
10/03/83	87.5	✓				✓	✓							✓			
18/02/84	45.2	✓					✓							✓			
13/04/85	43.4	✓	✓			✓	✓							✓	✓	✓	✓
10/12/90	44.5	✓					✓	✓						✓	✓	✓	✓
5/01/91	107	✓					✓	✓		✓				✓			✓
28/12/93	62.7	✓						✓		✓			✓	✓	✓	✓	✓
1/03/95	54.1	✓					✓	✓		✓		✓	✓	✓	✓		✓
23/12/96	65.5	✓					✓	✓		✓	✓	✓		✓	✓	✓	✓
28/12/96	56.3	✓					✓	✓		✓	✓	✓		✓	✓	✓	✓
3/01/97	100	✓					✓	✓		✓	✓	✓		✓	✓	✓	✓
1/03/97	76.5	✓					✓	✓		✓	✓	✓		✓	✓	✓	✓
21/01/98	68.3	✓					✓	✓		✓	✓	✓		✓	✓	✓	✓
12/03/98	45.4	✓					✓	✓		✓	✓	✓		✓	✓	✓	✓
8/04/99	49.1	✓					✓	✓		✓	✓	✓		✓	✓	✓	✓
16/11/02	41.2	✓						✓	✓	✓	✓	✓		✓	✓		
13/01/03	70.5	✓						✓	✓	✓	✓	✓		✓	✓	✓	✓
24/01/06	57.7	✓						✓	✓	✓	✓	✓		✓	✓	✓	✓
6/04/10	56.2	✓		✓					✓		✓	✓		✓	✓		
15/02/11	166	✓		✓				✓		✓	✓	✓		✓	✓		

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

The catchment area of interest is divided into a number of sub-areas based on the stream network. Storm rainfall is input to each sub-area with due regard to the intensity of rainfall and the time pattern over which rain falls. The rainfall becomes an input to the stream network and the resulting flows are routed downstream in sequence and accumulate at the catchment outlet.

The attenuation of flows during the routing process depends on the length of the stream travelled by the flood wave, the slope of the stream bed and the presence of any features that will add to the temporary storage that the flood wave encounters.

The percentage of rainfall that becomes runoff depends on the losses from pervious and impervious areas in the sub-areas. Catchments with extensive impervious areas - such as roads, roofs, car parks and formal drainage systems - produce more runoff and quicker runoff.

The data entered into the URBS model includes:

- Sub-areas (sq km)
- Stream lengths in each sub-area
- The slope of the main stream and the slope of the land contributing to the main stream in each sub-area
- Storage discharge relationships for the Marrara Swamp and the flood control weir that was constructed in 1985
- The percentage of impervious area for urbanised areas

A full description of the URSB model and the data used to establish the model for the Rapid Creek catchment is in the Hydrology Report (SKM, 2012).

2.4.2. Results

URBS can be run in BASIC mode or SPLIT mode as follows:

- In BASIC mode, the attenuation that occurs as a flood wave passes through the stream network is a function of the stream reach length (modified where necessary by an urbanisation index.[6]). There is an option to make routing also a function of reach slope.
- In SPLIT mode the attenuation in the main stream within a sub-area is modelled and the attenuation that occurs in overland flow and smaller streams that contribute to the main stream is modelled separately. The split mode requires input of a slope for those areas contributing to the main stream within each sub-area.

Full details of URBS calibration runs are given in the Hydrology Report (SKM, 2012). The most consistent calibration runs were carried out using the URBS BASIC model and average parameters were derived that fitted 23 of the 26 events listed in Table 5. For the ten largest events, the average error in peak discharge was 5.3% using these parameters.

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[6] In areas that have been urbanised, runoff is produced much quicker as a result of paved areas and constructed drainage networks so flood waves travel quicker and are attenuated less than for undeveloped areas.

Relevant regional relationships for the routing parameter of the RORB model (K_C) were also examined. RORB is another runoff routing model that has been used more extensively in the past in Australia than URBS. The RORB routing parameter K_C is theoretically related to the URBS routing parameters α. The relationships examined were:

1. Australian Rainfall and Runoff Equation 3.30 for Humid Zone NT
   $K_C = 1.8 \times (A/\sqrt{S})^{0.55}$ = 3.93 Corresponds to α = 0.58

2. Pearse et al (2002) – "Aus-wide" data set
   $K_C = 1.14 \times d_{av}$ = 7.67Corresponds to α = 1.14

3. Australian Rainfall and Runoff Equation 3.23 for Queensland
   $Kc = 0.88 \times A^{0.53}$ = 5.12Corresponds to α = 0.76

Assessing these parameters and the results of the calibration runs, together with the need to be conservative in deriving design flood peaks, it was decided to adopt the URBS Basic model with routing as a function of stream length and an alpha value of 1.12 (corresponding to K_C=7.5).

The Hydrology report (SKM. 2012) shows how the 'best fit' routing parameter α for the URBS BASIC runs and the 26 selected storms varies over time. Although there is some scatter as expected, there appears to be a clear trend for α to decrease in time.

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

Adoption of an α value of 1.12 is consistent with the increasing level of development in the catchment.

2.5. Design runs

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

The calculated design flow peaks at G8150127 are shown in Table 6. The losses from rainfall were adjusted to produce design peak discharges similar to those indicated by flood frequency analysis.

Table 6. Comparison of peak discharges from design runs and flood frequency analysis

ARI (years)	AEP (Annual exceedance probability)	Initial loss (mm)	Continuing loss rate (mm/hr)	Peak discharge calculated from URBS design runs (m3/sec)	Peak discharge adopted from flood frequency analysis (m3/sec)
5	20%	27.5	3.0	67.5	68
10	10%	27.5	2.0	88.4	88
20	5%	21.0	2.0	108	108
50	2%	19.0	2.0	134	134
100	1%	17.5	2.0	156	155
200	0.5%	16.0	1.5	177	176
500	0.2%	15.0	1.5	205	204

The resulting hydrographs are used as inputs to the hydraulic model runs for Q20, Q50, Q100, Q200 and Q500. These hydrographs are shown in an appendix of the Hydrology Report (SKM, 2012).

2.5.1. Probable maximum floods

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

In order to carry out URBS runs using PMP, and in a manner consistent with design floods, it is necessary to adopt appropriate losses. The losses that have been adopted are the same as for the Q500 design flood: viz; initial loss=15 mm and continuing loss rate = 1.5 mm per hour.

The PMPs for storm durations of 15 minutes to 6 hours were calculated using the Generalised Short Duration Method – GSDM – (Bureau of Meteorology, 2003(A)).

The Revised Generalised Tropical Storm Method – GTSMr – (Bureau of Meteorology 2003 (B)) was used to calculate a PMP for a duration of 24 hours and for storm durations of 12 hr and 18 hr the depth was interpolated graphically. However, it was found that the critical durations for the Rapid Creek catchment are generally less than 6 hours.

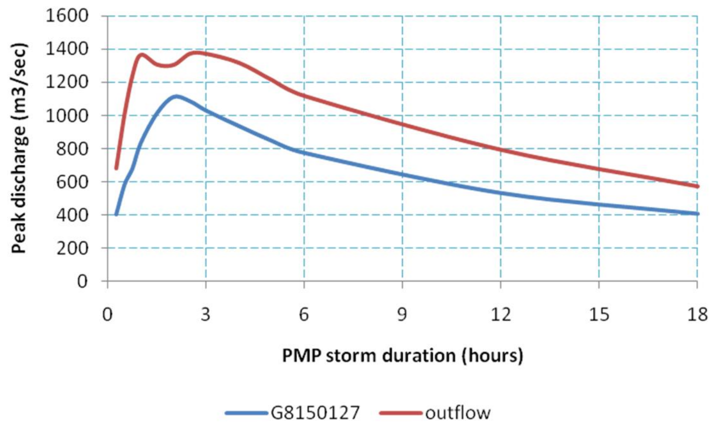
Full details are given in the Hydrology Report (SKM, 2012).

Peak discharges for the probable maximum floods calculated by URBS runs using the calculated PMPs are shown in Table 7 and Figure 3.

Table 7. Calculated PMFs

Duration (hours)	Calculated peak discharge (m3/sec) at	
	G8150127	Sea outfall
15m	405	684
30m	584	1,005
45m	687	1,244
1h	839	1,364
1.5h	1,018	1,305
2h	1,115	1,304
2.5h	1,089	1,371
3h	1,032	1,370
4h	939	1,316
5h	850	1,214
6h	777	1,118
12h	535	795
18h	410	576

Figure 3. Calculated PMF peak discharges



These show that the critical duration for the PMF is 2 hours for G8150127 and 2.5 hours for the sea outfall. PMF hydrographs are shown as an Appendix to the Hydrology Report (SKM, 2012).

2.6. Conclusions

Hydrographs of flow at locations along Rapid Creek are required for input to the hydraulic model that will be used to calculate the extent of flooding of Rapid Creek downstream of the flood control weir.

Flood frequency analysis was carried out using the Generalised Extreme Value distribution. Flood frequency curves were fitted for both the raw data series and a series adjusted to conditions that prevail after the construction of a flood control weir 500 m upstream of Henry Wrigley Drive in 1985. The Generalised Extreme Value distribution is considered to fit the ranked and plotted flood peak data reasonably well for both series and flood frequencies were adopted for floods up to Q500 for the series adjusted to post-weir conditions with an LH shift of zero.

During calibration, the URBS model successfully modelled the majority of 26 events considered. The parameters derived from the calibration runs were assessed against regional prediction equations and the URBS Basic model was used for design runs. Adjustment of loss factors was able to be used in a consistent manner to produce calculated design flows of the same order as those from the results of the flood frequency analysis.

Hydrographs of flow for design floods (Q20, Q50, Q100, Q200, and Q500) and probable maximum flood were produced for use in the hydraulic modelling.

3. Hydraulic modelling

3.1. Model selection

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

3.2. Model development

3.2.1. Model extent

The extent of the TUFLOW model was determined in order to simulate flood behaviour of the Rapid Creek main channel and floodplain from immediately downstream of the flood control weir adjacent to Darwin Airport, to the outlet of the creek into the Beagle Gulf. The extent of the model covers a 5 km reach of Rapid Creek that includes the main channel crossings of Henry Wrigley Drive, McMillans Road, and Trower Road. The model also includes the constructed open channel that enters Rapid Creek from the Charles Darwin University campus. The extent of the Rapid Creek TUFLOW model is shown in Figure 4.

3.2.2. Model terrain

Ground surface elevations of the TUFLOW model were defined using the following data sets:

- Digital Elevation Model (DEM) developed from photogrammetry (2011) of the study area. The DEM was provided by the then Department of Lands and Planning
- Field survey of the creek channel between the flood control weir and Henry Wrigley Drive. The survey was collected as part of a previous SKM project
- Field survey of the creek channel between McMillans Rd and the gauging station (G8150127). The survey was collected as part of a previous SKM project

A review of the DEM against the available field survey showed the creek invert defined in the DEM was typically 0.5m to 1.0m higher than surveyed levels. The elevation of the creek bed was defined using available creek invert levels from the field survey and design drawings of hydraulic structures.

The DEM within the tidal limit of the creek showed the channel invert was flat, indicating the DEM represents the water surface and does not capture the actual cross section of the channel below the normal water level. No bathymetric survey of the channel was available so elevations of the creek channel below the water level were assumed by estimating the depth of water from aerial photography. Depths of 0.5m, 1m and 2m were applied.

- **Figure 4. Rapid Creek TUFLOW model extent**

3.2.3. Model structure

The TUFLOW model was set up representing the creek channel and floodplain in the 2D model domain and hydraulic structures modelled as 1D elements nested within the 2D domain.

The 2D domain consists of a grid of cells at five metre spacing which contain elevation and roughness data. The grid size was selected to allow the features of the creek channel and floodplain to be represented with sufficient accuracy while maintaining efficient model run times. The grid size of 5m is considered to allow adequate representation of the creek channel and floodplain topography and is consistent with the size commonly adopted in flood studies.

Hydraulic structures such as bridges and culverts were modelled as 1D elements nested within the 2D model domain. The hydraulic structures represented in the model are:

- Culverts under Henry Wrigley Drive
- Culverts under McMillans Road
- Pedestrian footbridge in the Water Gardens (known as the Red Footbridge)
- Trower Road Bridge (modelled as an irregular shaped culvert[7])
- The footbridge and culverts along the open channel from Charles Darwin University

Dimensions and invert levels of the bridges and culverts were obtained from design drawings provided by the then Department of Construction and Infrastructure. Entry and exit loss coefficients of 0.5 and 1.0 respectively and a Mannings ‘n’ roughness value of 0.013 were applied to the culvert structures. These entry and exit loss coefficients are the standard values recommended in the TUFLOW manual. The Manning’s ‘n’ roughness value for the culverts is a typical value adopted for concrete conduits.

The footbridge at the creek outlet was excluded from the model as it would not significantly impact flood behaviour. The two piers of this bridge form only a minor obstruction to flow and the bridge is unlikely to become submerged during a flood due to the high elevation of the bridge soffit.

3.2.4. Inflows

Model inflow hydrographs were extracted from the URBS hydrologic model (refer Section 2.4) and input into the TUFLOW model. The URBS model total hydrograph from the flood control weir is entered as the upstream inflow to the TUFLOW model. A total hydrograph from the catchment

[7] The Trower Road Bridge is single span with an irregular shaped waterway opening that is concreted on the sides and invert, much like a culvert structure. Using the ‘irregular culvert’ option in TUFLOW to represent this structure was considered the most appropriate option for ensuring model stability and accuracy.

entering the open channel adjacent the university also enters the TUFLOW model. Local inflows for all other sub catchments are entered into the model proportionately along the reach of the creek.

3.2.5. Downstream boundary

A water level boundary was used at the downstream end of the model to represent the Beagle Gulf sea level. The boundary was represented as a stage hydrograph where recorded tidal levels were available and a static water level for design event model runs according to the tide level or storm surge level being modelled.

3.2.6. Land-use delineation

Regions of similar catchment land use and vegetation cover types were defined to allocate the 2D domain with Manning's 'n' roughness values. These regions included the creek channel, riparian vegetation, overbank areas, road corridors, mangroves, and residential development. The Manning's 'n' roughness values applied to the various regions were selected and refined during calibration of the model which is discussed in Section 3.3.

3.3. Model calibration

The TUFLOW model was calibrated to water levels recorded at the gauging station (G8150127) and a number of surveyed flood marks from the February 2011 flood event. Using the calibrated URBS model hydrographs as inflows to the TUFLOW model, the model was calibrated by adjusting the Manning's 'n' roughness values until a satisfactory match to the surveyed and recorded peak flood levels was achieved.

3.3.1. Selected hydraulic roughness values

The land use categories and their Manning's 'n' roughness values adopted for the model calibration are shown in Table 8.

- **Table 8. Land use categories and adopted Manning's 'n' roughness values**

Land use category	Manning's 'n'
Road corridors	0.035
Residential lots	0.500
Open space with scattered vegetation	0.045
Rural lots	0.070
Creek channel	0.060
Creek channel through mangroves	0.100
Estuary channel / open water	0.030
Mangroves	0.300
Riparian bank vegetation	0.150
Mango plantation	0.090
University campus	0.100

The Manning's n values were assigned at a block-scale, and are typically representative of the average roughness across each category and account for on-lot obstructions to flow, such as dwellings, fences and miscellaneous structures, which were not represented explicitly. The land use categories were digitised using the aerial photography. The digitised roughness polygons are then input to the model in order to assign the hydraulic roughness values to the model grid points.

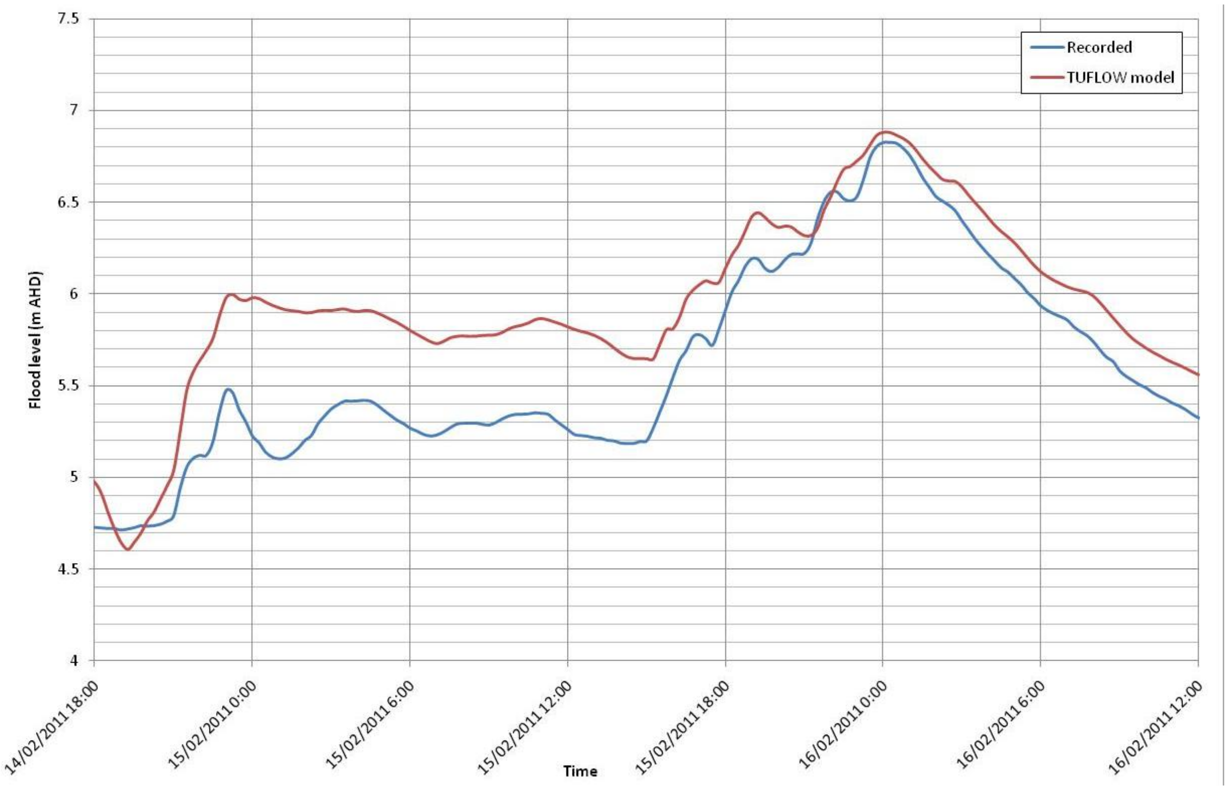
3.3.2. Calibration to gauge G8150127 records

The gauging station G8150127 on Rapid Creek is a concrete v-notch weir located approximately 400m downstream of McMillans Rd. It has a cease to flow level of 0.827m (gauge datum) which is equivalent to 3.94m AHD.

The TUFLOW model was verified using the flood event of February 2011 for which some flood plain water level height data (i.e. flood marks) was collected by survey. The TUFLOW model could not be verified to another historical event due to the lack of historical flood data. It may now be possible to use the flood event that occurred on 31 January 2012, for which only partial data was available at the commencement of this study.

During the February 2011 flood event the gauge recorded a peak flood level of 6.83m AHD at midnight on the 16th of February. A graph showing the recorded flood levels over the 15th and 16th of February compared with flood levels from the calibrated TUFLOW model is shown in Figure 5.

- **Figure 5., Recorded and modelled flood levels at gauge G8150127**

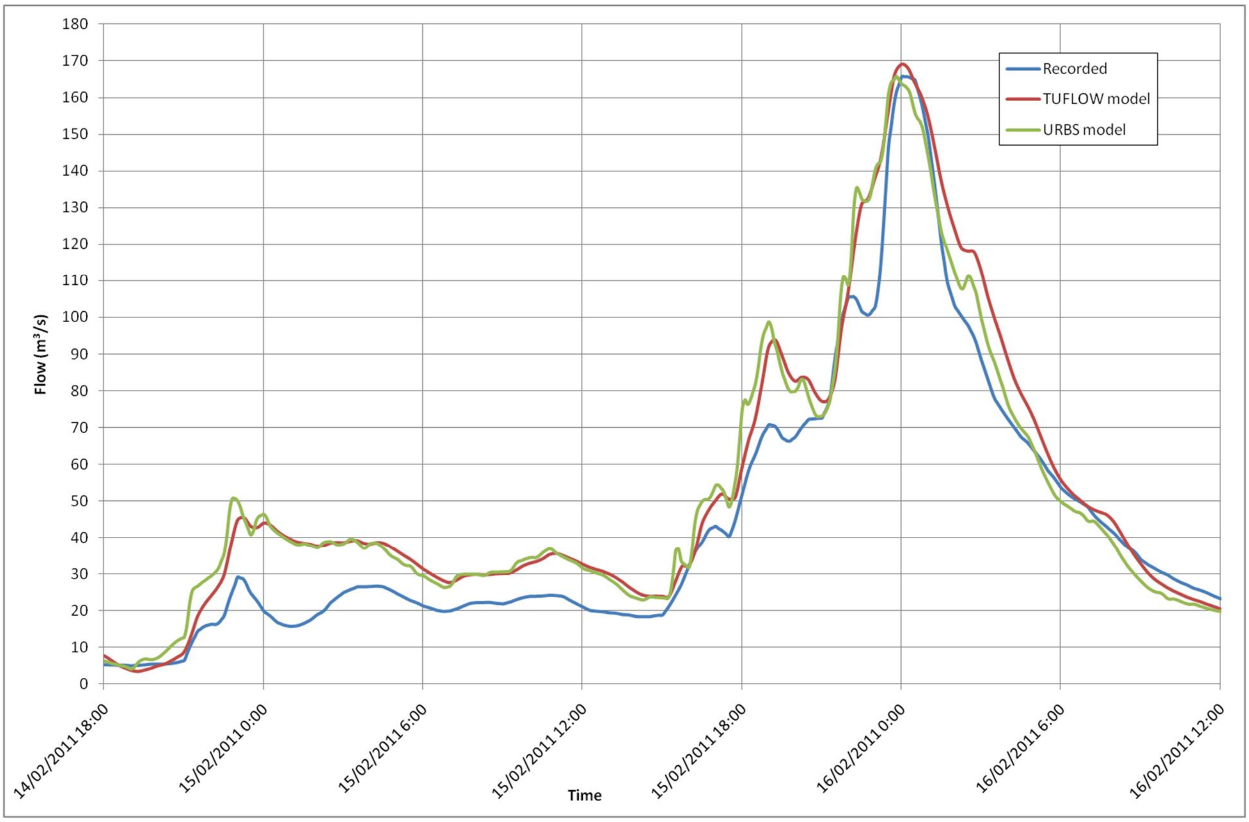


The TUFLOW model was able to reproduce the recorded peak flood level at the gauge to within 50mm, producing a peak flood level of 6.88m AHD at midnight on 16 February 2011.

However, there is a poorer fit to recorded levels over the 24 hours prior to the peak of the flood with the model producing increased levels compared to those recorded. The poorer fit is a result of the increased flows produced by the URBS hydrologic model (BASIC model a=1.4, initial loss =0.0 mm, continuing loss rate = 0.0 mm/hr) compared to the gauged flows[8], and possibly the definition of the creek channel in the 2D domain. As the main focus of the flood study is to estimate peak flood behaviour for the Q20 to Q500 design events and Probable Maximum Flood, the poorer fit of the model to these lower levels is not considered significant.

Flows at the gauge extracted from the TUFLOW model compared with the recorded gauged flows and the URBS hydrologic model flows are shown in Figure 6. The peak flow at the gauge from the TUFLOW model is 169m³/s compared to the recorded and URBS model peak flow of 166m³/s.

- **Figure 6. Recorded and modelled flow at gauge G8150127**



[8] Note that the URBS fitting process was weighted to matching the peak discharge since this was the rank 1 event and the flood peaks are of most interest to the flood study. It may have been possible to choose URBS parameters that provided a closer fit to the early rises in the February 2011 hydrograph but not as good a peak fit.

3.3.3. Calibration to recorded flood marks

After the February 2011 event, a number of flood marks between Trower Road and Darwin Airport were surveyed. Of the 21 surveyed flood marks, 18 are within the TUFLOW model extent and were used for calibration of the model. The flood mark locations along with their surveyed level and the peak flood levels produced by the TUFLOW model are shown in Figure 7 and Figure 8.

As shown in Figure 7, nine flood marks were surveyed between Trower Rd and McMillans Rd, the majority adjacent to residential areas on the left overbank of the creek. There is good agreement between the modelled and surveyed levels with 8 of the 9 modelled peak flood levels within 0.06 m or 60 mm of the surveyed flood marks. The difference at the 9th location is 0.13 m. This location is the only one on the eastern side of Rapid Creek Road. The recorded level may reflect a dip in the water surface associated with the flow accelerating as it crosses the road.

Another nine flood marks were surveyed between McMillans Road and the flood control weir, as shown in Figure 8. Six modelled levels show good agreement and are within 0.10m of the recorded levels. However, three modelled levels show a poorer fit and are lower than the recorded level by between 0.10m and 0.24m. The poorest fit is to the recorded level upstream of Henry Wrigley Drive. The poor fit could be the result of:

- Blockage of the Henry Wrigley Drive culverts during the February 2011 event causing an increase in the recorded upstream flood level
- The URBS model underestimating the peak flow from the flood control weir
- Local turbulence

3.4. Model limitations

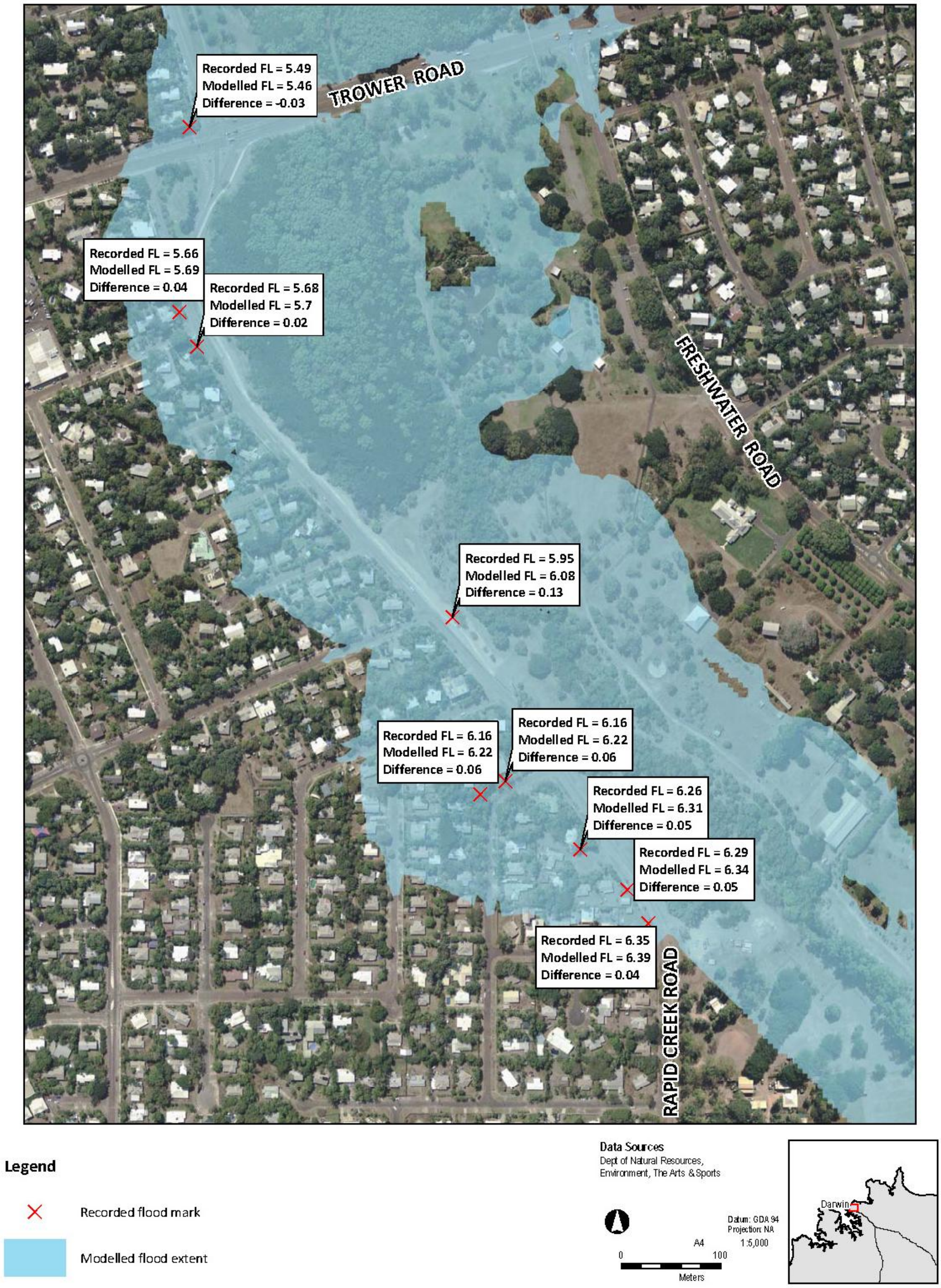
In summary, the TUFLOW model was set up representing the creek channel and floodplain in 2D and hydraulic structures as 1D elements nested in the 2D domain. The bed elevation of the creek was defined using available field survey and design drawings of hydraulic structures. Bathymetric survey of the creek channel within the tidal zone was not available so assumptions of the channel depth were made. The sensitivity of these assumptions on modelled flood levels has not been tested.

The TUFLOW model is able to reproduce the majority of surveyed peak flood levels from the February 2011 event to within 0.10m. The model is considered satisfactorily calibrated to this event and appropriate for modelling the design storm event scenarios of interest to this study i.e. the Q20 flood event and larger. It is noted, however that all of the recorded flood marks for the 2011 event were on the western side of the creek and we were unable to verify that model performance is satisfactory for the eastern flood plain.

Due to the size of the creek channel in its upper reaches and its definition in 2D, the model is not considered appropriate for simulating smaller flood events, such as the Q2 event, where a higher proportion of flow would be conveyed within the creek channel. Defining the creek channel as a 1D network nested in the 2D domain using surveyed cross sections would be required for the model to be suitable for these smaller flood events.

It is also important to note that the model has not been validated against another historical event due to the lack of historical flood data. Validation of the model to another historical event would further improve confidence in the model's ability to simulate flood behaviour of Rapid Creek.

- **Figure 7. TUFLOW calibration to recorded flood levels-1**

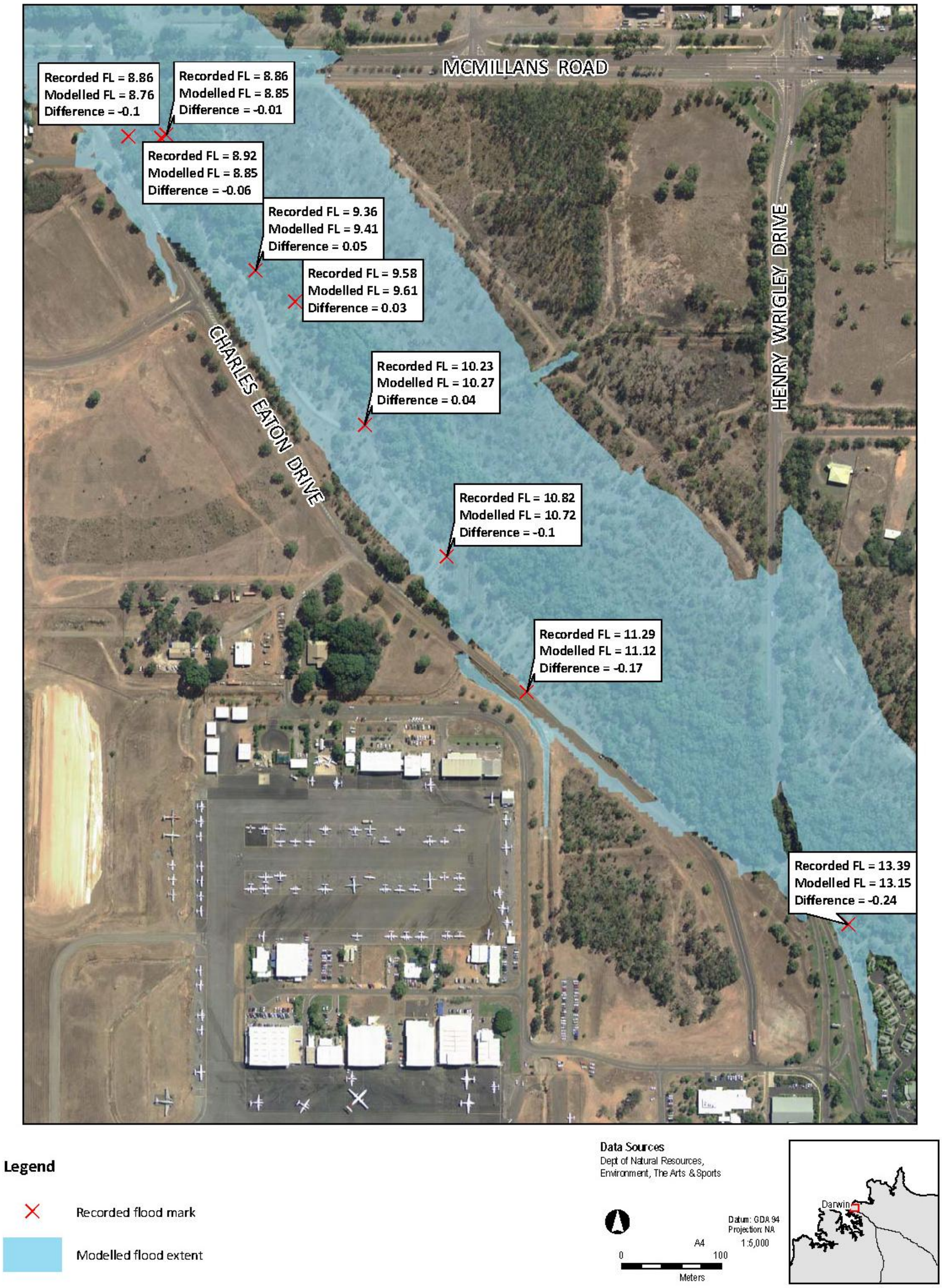


TUFLOW calibration to recorded flood levels

Figure 7

DB05724
Rapid Creek Flood Study
Thursday, 24 May 2012

Figure 8. TUFLOW calibration to recorded flood levels-2



TUFLOW calibration to recorded flood levels

Figure 8

DB05724
Rapid Creek Flood Study
Thursday, 24 May 2012
I:\DBIF\Projects\DB05724\Technical\GIS_Directory\ArcMap\Figure3-5_FloodMarkCalibration.mxd

4. Design flood modelling

4.1. Design events

The calibrated TUFLOW model was used to simulate a number of design storm events combined with various downstream sea levels. The fourteen design flood scenarios modelled for the study are displayed in Table 9.

Table 9. Design flood scenarios

Year	Sea level	Design storm
2012	Mean sea level (0.1m)	20y, 50y, 100y, 200y, 500y and PMF
	Highest astronomical tide (3.36m)	100y, 500y, and PMF
	1% AEP (1 in100 year) storm surge (4.6m)	20y and 100y
2100	Highest astronomical tide + 0.8m (4.16m)	100y, 500y, and PMF

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

4.2. Flood modelling results

The results of maximum flood height, depth, and velocity depth product were determined for each scenario and used as inputs to the floodplain mapping which is described in the following section of the report. Maximum flood heights were also used to prepare design flood profiles which are contained in Appendix A.

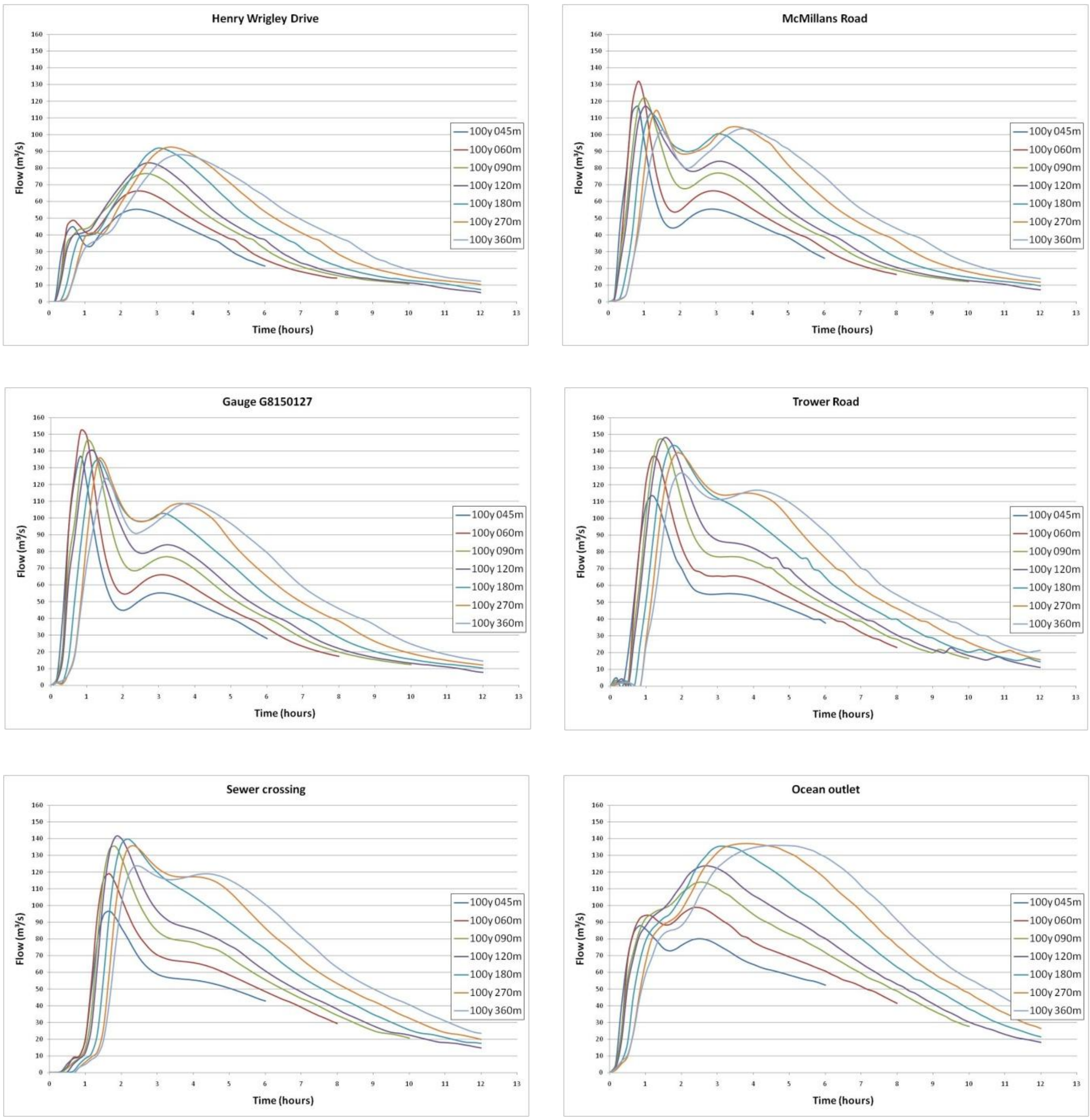
Flow and stage hydrographs at selected locations along the creek were also output from the model in order to confirm the critical duration was captured at each location. Critical durations typically ranged from the 1 hour storm to the 4.5 hour storm for all design recurrence intervals. Figure 9 shows flow hydrographs extracted from the TUFLOW model for the Q100 with mean sea level design flood scenario.

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

- At Henry Wrigley Drive the critical design flood levels result from the 4.5 hour duration storm. This is due to the flood control weir's impact of attenuating peak flows from the upstream catchment.

- At McMillans Road and the gauging station, critical flooding is from the shorter 1 hour duration storm due to inflows from the fast responding sub catchments between McMillans Road and the flood control weir. This is followed by a second flood peak of a smaller magnitude.
- Critical flooding between Trower Road and the ocean outlet is from the 2 hour to 4.5 hour duration storm events. Flood levels over this length of the creek are controlled by a constriction at the outlet and the amount of floodplain storage downstream of Trower Road.

Figure 9. TUFLOW Q100 hydrographs at selected locations - mean sea level condition



4.3. Floodplain mapping

The base data for the flood plain maps is the digital Elevation Model (DEM) supplied by the then Department of Lands and Planning developed from 2011 photogrammetry of the study area (Refer also Section 3.2.2.)

The steps involved in producing the maps were:

- All ASCII datasets from the TUFLOW hydraulic model were converted to raster using ArcGIS
- To produce flood extents, the ArcGIS raster data were converted to polygons using 'Raster to Polygon' with the 'Simplify Polygons' command
- To produce flood water surface AHD contours at 0.25 m intervals, ArcGIS 'Contour' command was used
- To derive flood depths the ArcGIS 'Reclassify Raster' command was used to classify depth values into the 0.0 to 0.5, 0.5 to 1.2, 1.2 to 2.0 and greater than 2.0 ranges. 'Raster to Polygon' with 'Simplify Polygons' was then used to create regions corresponding to these depth ranges and then 'Dissolve Polygons' was used to create a single dataset
- The floodway (the most hazardous portion of the flow) was defined by using 'Reclassify Raster' to select for values of velocity × depth larger than 1.0 and missing values converted to 'no data'. Similarly depths greater than 2.0 were selected and missing values set to 'no data'. These two sets were combined using 'Cell Statistics' with 'Maximum Value Overlay'. Then 'Raster to Polygon' with 'Simplify Polygons' was used to convert raster data into regions meeting the hazardous criteria and finally 'Dissolve Polygons' was used to create a single dataset

Flood plain maps for the scenarios required (refer Table 9) are presented in Appendix C.

4.4. Description of flood extents

4.4.1. Q20 current mean sea level

The extent of flooding during the Q20 flood event is generally characterised by the following:

- The extent of flooding between the flood control weir and the gauge is generally 150m to 200m in width
- Henry Wrigley Drive remains free from flooding but McMillans Road is overtopped by floodwaters
- Downstream of the gauge there is an expansion of flow into low-lying areas on the left and right overbank resulting in the inundation of existing residential properties on the western side of Rapid Creek Road

- Trower Road is overtopped at the intersection with Rapid Creek Road impacting existing residential properties at the north-west corner of the intersection
- Floodwaters downstream of Trower Road are confined to the creek and mangrove overbank areas and to the constriction at the outlet

4.4.2. Q100 current mean sea level

The extent of flooding during the Q100 flood event is generally characterised by the following:

- The extent of flooding between the flood control weir and the gauge is generally 180m to 230m in width
- Henry Wrigley Drive is overtopped on the northern side of the culverts and there is increased overtopping at McMillans Road
- Between the gauge and Trower Road there is further expansion of floodwaters into the low-lying areas on the left and right overbanks. A breakout of flow on the right overbank near the gauging station causes flooding of an existing rural property. While an increased number of existing residential properties on the western side of Rapid Creek Road are affected
- Trower Road is overtopped at the Rapid Creek Road intersection and a second location approximately 275m to the east (near Freshwater Road)
- Floodwaters downstream of Trower Road are confined to the creek and mangrove overbank areas and to the constriction at the outlet
- Floodwaters from the university open channel catchment threaten a number of existing structures on the university campus

4.4.3. PMF current mean sea level

The extent of flooding during the PMF event is generally characterised by the following:

- The extent of flooding between the flood control weir and the gauging station is generally 400m to 450m in width and affects a number of existing developments
- All road crossings are affected by the PMF
- Between the gauge and Trower Road the extent of flooding increases to approximately 700m in width and affects a widespread number of existing properties on the western side of Rapid Creek Road and the eastern side of Freshwater Road
- Downstream of Trower Rd there is inundation of properties adjacent Rapid Creek Road and Lakeside Drive, and increased inundation of the university campus
- At the outlet to the sea there is a breakout of flow to the north of the outlet constriction

4.4.4. Impact of sea level rises

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

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

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

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

5. Flood mitigation options

Flood mitigation options are usually classified as structural or non-structural options.

Structural flood mitigation options are measures taken to contain floods and limit the depth and extent of flooding. They typically involve placing structures or modifying existing structures in or around the stream.

Possible structural options for flood mitigation for Rapid Creek include:

- Upgrading the channel capacity:
  - Widening the channel
  - Re-grading the channel
  - Clearing vegetation from the channel
- Removing obstructions to the flow. The obstructions include:
  - The weir formed by the pipe crossing between the Rapid Creek and Lakeside Drive sewer pumping stations.
  - Trower Road bridge crossing
  - The red footbridge in the water Gardens
  - The gauging weir
  - McMillans Road bridge crossing
  - Henry Wrigley Drive bridge crossing
- Lining the channel
- Constructing levees
- Constructing flood storage to delay and reduce flood peaks.

Non-structural options are measures taken to limit the damages resulting from floods without physical measures to contain floods and limit the depth and extent of flooding. Such measures include:

- Flood warning and evacuation plans
- Flood proofing
- Land use planning and zoning measures

The relevance of each of these to Rapid Creek is discussed in the following.

5.1. Structural flood mitigation options

5.1.1. Upgrading channel capacity

Works to widen or re-grade the Rapid Creek channel are not likely to be approved. Rapid Creek is one of very few creeks that arise in the built up area of Darwin. It is in a more natural condition than the nearby Ludmilla Creek, the headwaters of which have been largely built over within RAAF Base land. Unlike Buffalo Creek and Palmerston's Myrmidon Creek, it does not receive sewage discharges.

It arises in undeveloped land largely within airport/RAAF land and although it is crossed by Henry Wrigley Drive, McMillans Road and Trower Road, it remains as a surface water feature that has not been encroached on by buildings.

It is used for passive recreation, with walking trails within the Freshwater Gardens, on the western side between the Freshwater Gardens and McMillans Road and along the southern side between the flood control weir and Henry Wrigley Drive. It is also commonly used for water play.

Rapid Creek is therefore considered a community asset and a feature that is worth preserving in its existing condition. It is very unlikely that the community will tolerate engineering works that modify the channel, leave it susceptible to erosion and deposition and involve substantial clearing of existing vegetation.

This will rule out channelisation including widening and/or re-grading of the channel and lining of the channel. Permissible clearing of the channel is likely to be limited to selective removal, such as taking away fallen trees after floods, which is unlikely to modify hydraulic roughness enough to increase channel capacity. It is more likely to be associated with maintaining existing capacity.

5.1.2. Removal of obstructions

<u>Sewer pipe across the creek</u>

A gravity sewer main has been installed connecting Alawa to the Rapid Creek sewer pump Station. A pump station has subsequently been constructed at Lakeside Drive to pump sewage from Alawa toward the Leanyer wastewater treatment ponds. We assume the creek crossing is maintained as an equalising pipe and provides some relief if the Lakeside Drive pumping stations fails.

- **Figure 10. Sewer pipe between Alawa and Rapid Creek pump station (looking upstream)**

- **Figure 11. Sewer pipe crossing between Alawa and Rapid Creek pump station (from right bank)**

This pipe is exposed a t low flow and forms a weir in the channel bed. A plan was provided of the pipeline (DC 65/968B) dated 1965 but we were not able to determine the datum for levels on that plan. However, the plan show a central section of creek 40 ft (13m) long where the pipe bridges the creek, with a clearance of up to 4.5 ft (1.37 m) under the pipe. The adjoining photos show that the pipe is acting as a weir and it is quite likely that the conveyance remaining underneath is now minimal. The plan also shows embankments extending 390 ft (120 m) out from the Rapid Creek pump station (the left bank) and 900 ft (270 m) from the Lakeside Drive pump station (right bank). These embankments are up to approximately 8 ft (2.4 m) above the natural surface. The embankments, together with the sewer pipe and any siltation of the channel below the sewer pipe, constitute an obstruction to the flow. The photos suggest the head drop across the pipe is of the order of 400 mm at low flow. (Refer Figure 10 and Figure 11.)

<u>Bridges</u>

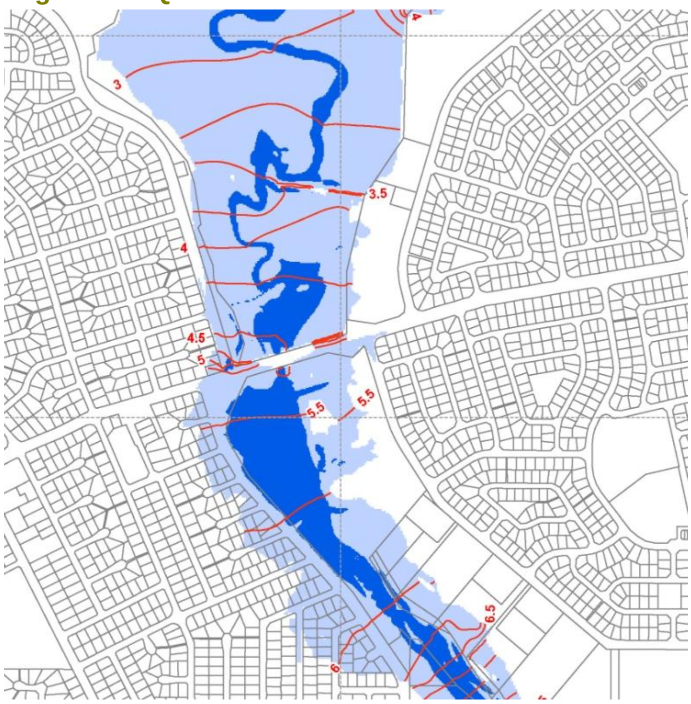
The hydraulic capacity of the creek is also constrained by bridge crossings. These are at:

1) Trower Road

Shallow floodwater has gone over Trower Road from Rapid Creek in the 1974 and 2011 floods, that is, twice in approximately 50 years. Trower Road is an arterial road and should have a reasonable standard of immunity. However, the frequency of flooding is complicated by tidal influence.

The flow pattern is complicated by the flow over the left bank and to a lesser extent the right bank, but

- **Figure 12. Q100 flood contours around Trower Road**



basically during this Q100, the level drops from 5.25 m upstream of the bridge to 4.5 m downstream - a fall of 0.75 m in just (say) 100m - as shown on Figure 12 (an extract from the flood plan map for a Q100 flood assuming present day mean sea level.) Upstream the water surface is much flatter, increasing to 6m (a rise of 0.75m) over a distance of some 600 m.

This suggests that the Trower Road crossing is contributing to the width of the floodplain in the Millner area and that enlarging the water way under the bridge could help reduce the potential for flooding. It is recommended that the TUFLOW hydraulic model be used to investigate whether modification of the Trower Road crossing will reduce water levels in the Millner area during major floods.

- **Figure 13. Red Footbridge (from left bank)**

2) The Red Footbridge in the Water Gardens

The red footbridge is located where a walking trail crosses Rapid Creek about 550 m upstream of Trower Road. It is inundated frequently during the wet season as it is submerged at low to medium flows.

The adjoining photos suggest there is a head loss across the bridge where debris builds up on the superstructure but it is not a major obstruction to large floods because water can flow around it unobstructed. We also note that the red footbridge is upstream of the Millner residential area that is subject to flooding and any modifications are unlikely to lower levels in the critical areas for flooding.

- **Figure 14. Red Footbridge (long view from left bank)**

3) McMillans Road

McMillans Road is subject to frequent inundation and the crossing has been the subject of previous reports (SKM, 2011). Problems at the crossing include service crossings downstream with concrete overlays forming low weirs in the bed of the channel, the skewed approach upstream and the approach being blocked by silt that has accumulated and supported the growth of vegetation.

The crossing is under-capacity and options for improving the flood immunity of the crossing were considered in the report (SKM, 2011). All options required raising the Charles Eaton Drive intersection. Options other than Q5 assumed the demolition and replacement of the existing culverts in order to minimise impact on the creek and vegetation by having the narrowest possible culvert. The existing culverts could be retained and supplemented by additional cells, but that would require clearing of more trees and widening of the channel.

It was considered essential to remove obstructions from the inlet and the outlet to the culverts in order to maximise culvert capacity and minimise flooding.

The report recommended that black wattles, which choke the outlet and are of no environmental significance, should be removed entirely and the outlet channel slightly widened and excavated to open it up. It was also considered highly desirable to lower the underground power line and underground communication cable that cross the culvert outlet so that they no longer act as weirs and restrict the outflow.

4) Henry Wrigley Drive

It is not known that flood flows overtop the bridge that carries Henry Wrigley Drive over Rapid Creek. This crossing is located some 500 m downstream of the flood control weir, in the area where the flood control weir will have the most impact on reducing flood flows reaching the creek. However the Q100 flood map (for present day mean sea level) shows the northern approach to the bride being overtopped.

The only existing development close to Rapid Creek between Henry Wrigley Drive and the flood control weir that is potentially affected by flooding from Rapid Creek is the Airport Resort. The resort is elevated and seems not to have flooded to floor level during the flood of February 2011 and the Q100 flood map suggest that for present day conditions the resort is unlikely to be flooded to floor level by a Q100.

<u>Gauging weir</u>

The gauging weir is designed to force a head drop to allow a stable relationship between flow and height at low flow. However it will not represent a major obstruction to flow and, because it is in a relatively steep reach of the creek, any back-up behind the weir will be limited in extent.

Removal of obstructions can be investigated using hydraulic modelling in further studies.

5.1.3. Levees

Levees are placed around streams with the intention that flood flows will be confined to the creek between levees and not allowed into areas where damage from floodwaters would otherwise result.

The application of levees to Rapid Creek may be limited because of their impact on the amenity of the creek. Levees also are problematic for drainage. There are multiple drainage outflows to Rapid Creek in the areas that are liable to flooding. To prevent floodwaters flowing 'backwards' in these drains would require flood gates. These create an ongoing requirement for maintenance to ensure they are unblocked and in good working order prior to the arrival of floods.

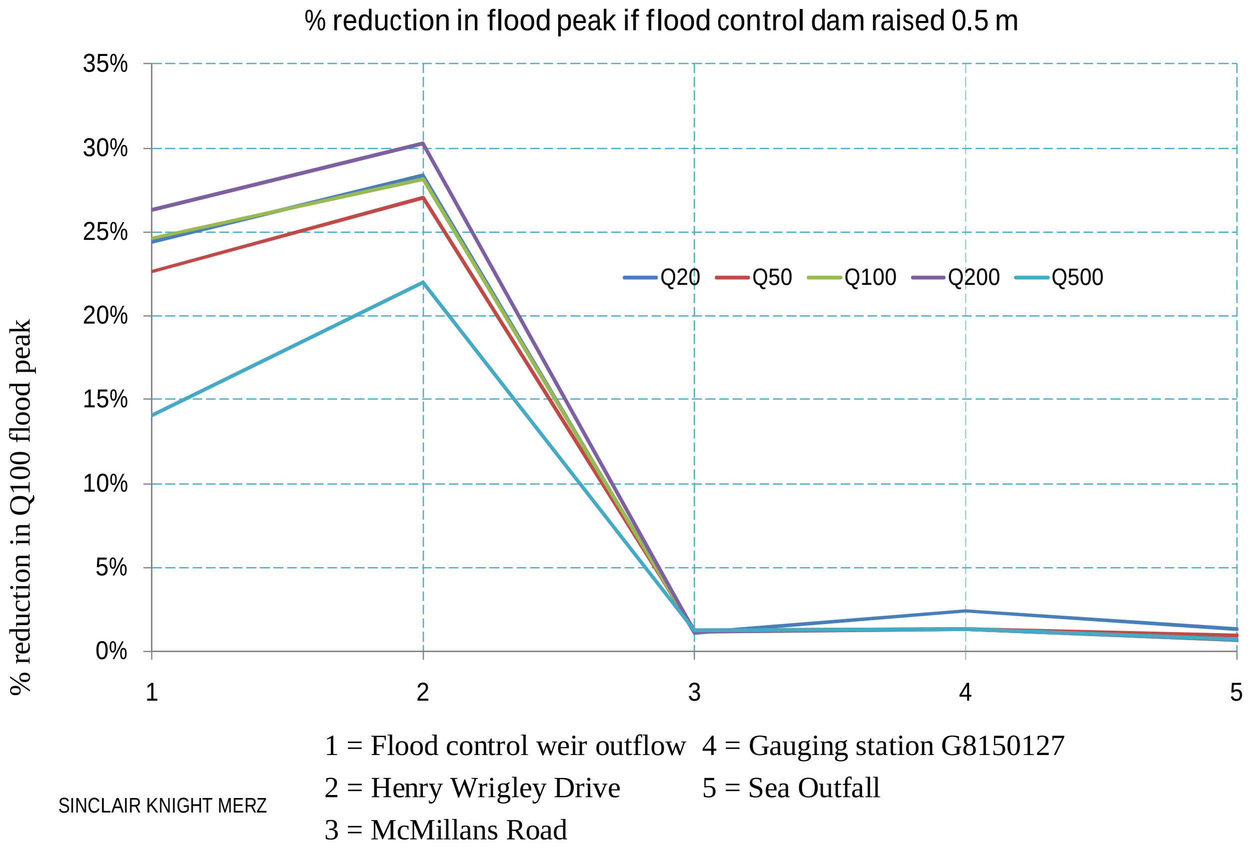
However, the open space corridor through which Rapid Creek flows is quite wide in most areas and there may be scope for small levees in particular areas to be one part of an overall flood mitigation solution.

5.1.4. Additional flood storage.

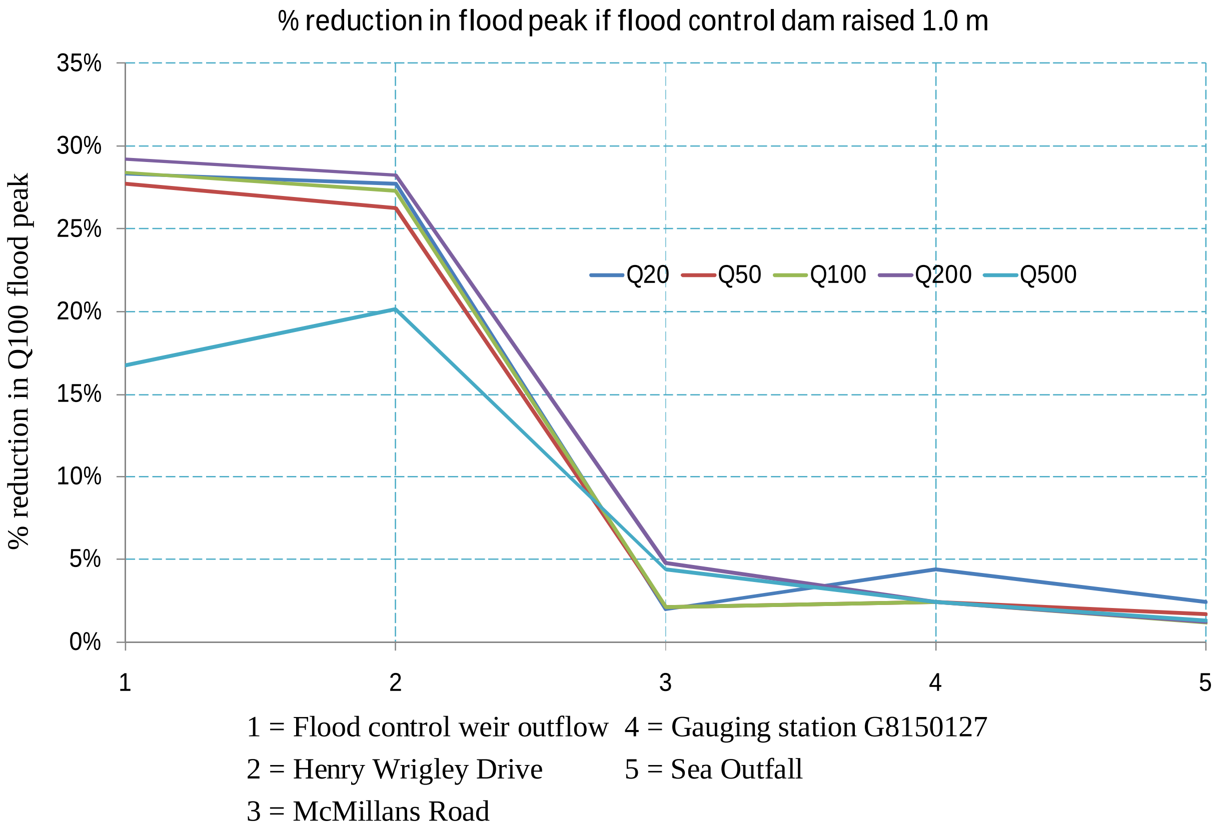
In practical terms this translates to raising of the existing flood control weir. The URBS model has been used to examine the likely mitigation potential of raising the weir.

Raising the weir by 0.5 m and 1.0 m has been investigated. This reduces the outflow from the weir as shown in Figure 15 and Figure 16.

- **Figure 15. Calculated reduction in flood peaks if flood control weir raised 0.5 m**



- **Figure 16. Calculated reduction in flood peaks if flood control weir raised 1.0 m**



The peak flows at Henry Wrigley Drive would also be reduced. However, the effectiveness of the reduction diminishes downstream and the impact on peak flows in the areas of most interest (McMillans and downstream of McMillans Road) is insignificant.

This means that raising the flood control weir will not be an effective flood mitigation measure.

5.2. Non-structural flood mitigation measures

5.2.1. Flood warning and evacuation plans

An NT floodwarning system exists that collects data from telemetered rain gauges and stream flow gauges and predicts likely river heights and flows in real time. The flood warning system is associated with larger streams such as the Katherine and Daly Rivers when many hours or even days occurs between the occurrence of heavy rain and the peak of the flood at a location of interest.

Rapid Creek is a much smaller stream and the critical storm durations are short. That is, the stream can rise to very high levels within an hour and therefore there is little warning time. This means that there is very little time available for action by emergency services groups or for individuals likely to be affected by flooding. For this reason flood warning and evacuation plans are best considered as an adjunct to other measures for flood mitigation.

5.2.2. Flood proofing

Flood proofing is the protection of buildings and structures without interruption to the flow of water around them. It typically involves sealing of structures with flood barriers so as to prevent the ingress of floodwaters. Sufficient time is required in order to deploy flood proofing measures. However, as noted above the warning time is short and Rapid Creek can rise to peak within an hour or so of the onset of extreme rain.

For most of the scenarios considered (refer Table 9) the number of buildings likely to be impacted (mostly private residences) is small and flooding will be shallow. Under these circumstances flood proofing could play a part. For example, solid fences[9] with custom made stops to be inserted in gate openings can keep buildings and yards dry while allowing floodwaters to flow though streets. It may also be possible to identify limited areas that can be provided with a ready supply of bags and sand for sandbagging, although again, there will only be a very limited time available to fill bags and place them.

It is also noted that a lot of housing in older suburbs of Darwin is elevated, such that it may be possible to allow occasional shallow floodwaters through yards and even downstairs rooms if they are of robust construction and contain furniture that can be easily and quickly moved and fixtures that can tolerate temporary inundation.

5.2.3. Land use planning and zoning

Rapid Creek arises in vacant crown land to the east of Amy Johnson Drive airport/RAAF land, flows though airport land and vacant crown land to McMillans Road. Downstream of McMillans Rd it flows through a narrower corridor with privately owned land on either side but this land is in larger allotments and has undergone limited development. Beyond these lots it flows to Trower Road through the Water Gardens on the right bank, which is a public park. On the left bank there is an open space buffer between the creek and the suburb of Millner.

Downstream of Trower Road Rapid Creek flows though undeveloped coastal reserve to the sea.

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[9] There is likely to be a practical height limit to solid fences because of cyclone loading. Typical fences in Darwin of more than a few tens of centimetres high are not solid because they allow wind to pass through.

5.3. Combinations of flood mitigation measures

The most appropriate solution for Rapid Creek is likely to be a combination of flood mitigation measures such as:

- Removal of obstructions to flow
- Minor levees
- Flood proofing
- Appropriate zoning and land use planning

It is recommended that combinations of measures should be investigated using the hydraulic model constructed for this study. An appropriate design standard should also be considered. Priority should be given to confining hazardous conditions to the creek's floodway and not allowing floodwaters of excessive depth or velocity in built up areas. However elimination of all flooding from residential areas may be cost-prohibitive.

6. Recommended additional investigations

- Improve the representation of the creek channel in the TUFLOW model.
  A review of the Digital Elevation Model (DEM) that was provided for the TUFLOW model found that it poorly defined the channel bed elevation. For example, the channel bed elevation defined in the DEM was typically 0.5m to 1.0m higher than available surveyed levels, and within the tidal zone the DEM does not capture the actual cross section of the channel below the normal water level.

  The channel bed elevation in the model is based on the limited field survey available, design invert levels of hydraulic structures, and assumptions of channel depth in the tidal zone. The impact of the channel depth assumptions on modelled flood levels is not known, so sensitivity testing of this assumption is recommended. Alternatively, the collection of bathymetric survey of the channel would assist in improving the channel representation in the model

- Validate the TUFLOW model against another historical event.
  The model has been calibrated to recorded flows and surveyed flood marks from the February 2011 event. To improve confidence in the TUFLOW model, validation to another historical event is recommended if recorded flows and flood levels are available

- Monte Carlo method in hydrology study
  The hydrology has adopted an AEP neutral approach where a 1% AEP (1 in 100 year) rainfall has “neutral” losses applied to it to produce a Q100 flood, and similarly for other design floods. It is possible to carry out Monte Carlo simulations with variable loss rates to provide a better estimate of flood peaks. The Monte Carlo method used in URBS is clumsy and we understand that the developer of the URBS program (Don Carroll) proposes to re-write it using the same approach as RORB. Because of this, and the time constraints imposed on the study, we have not used a Monte Carlo method to date but we recommend that this be carried out

- Investigation of flood mitigation measures
  The models developed for this study should be used to investigate flood mitigation measures and combinations of measures that will achieve a reasonable standard of flood protection without a detrimental impact on the amenity of Rapid Creek

7. References

"Rapid Creek Flood Study. Hydrology Report.". Sinclair Knight Merz for Department of Natural Resources, Environment, The Arts and Sports.

"Rapid Creek. Impact of proposed flood control weir upgrade and characterisation of flooding at Kimmorley Bridge. Report on investigations". Sinclair Knight Merz for Department of Construction and Infrastructure, March 2011

"The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method" Hydrometeorological Advisory Service, Commonwealth Bureau of Meteorology, June, 2003 (A)

"Revision of the Generalised Tropical Storm Method for Estimating Probable Maximum Precipitation" Hydrology Report Series HRS Report No 8, Commonwealth Bureau of Meteorology August 2003 (B)

"Rapid Creek. Impact of proposed flood control weir upgrade and characterisation of flooding at Kimmorley Bridge. Report on investigations" Sinclair Knight Merz for NT Government department of Construction and Infrastructure, March 2011

"Rapid Creek Flood Study" Revised Final Report. Connell Wagner, December 2004

"URBS (Unified River Basin Simulator) A Catchment Management & Forecasting Rainfall Runoff Routing Model." Version 4.4 by D.G. Carroll, 2009

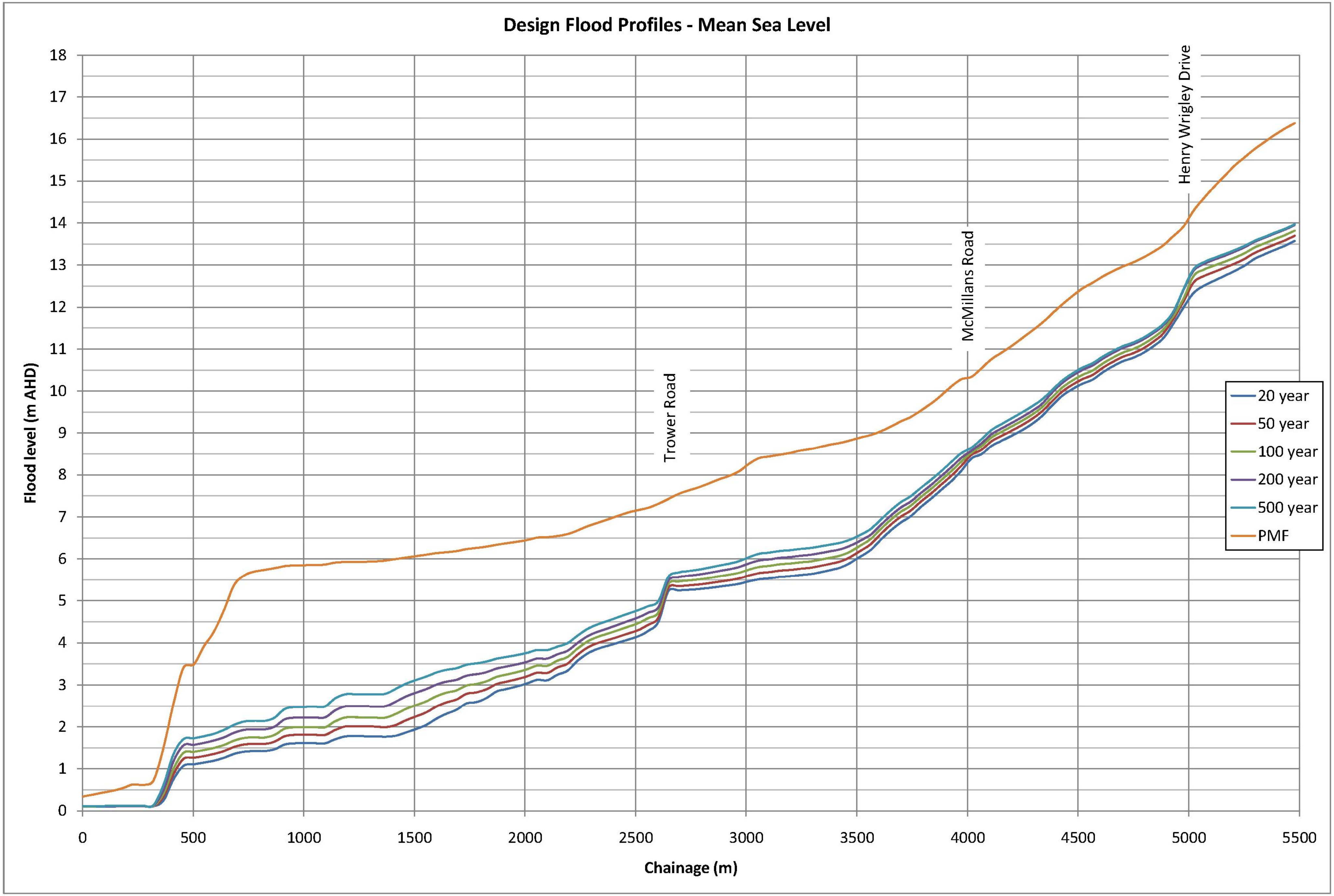
"A simple method for estimating RORB model parameters for ungauged rural catchments". Mark Pearse, Phillip Jordan and Yvette Collins, Engineers Australia Hydrology and Water Resources Symposium, 2002.

"LH moments for statistical analysis of extreme events". Q. J. Wang, Water Resources Research, Vol 33, No. 12 pp2841-2848, December 1997.

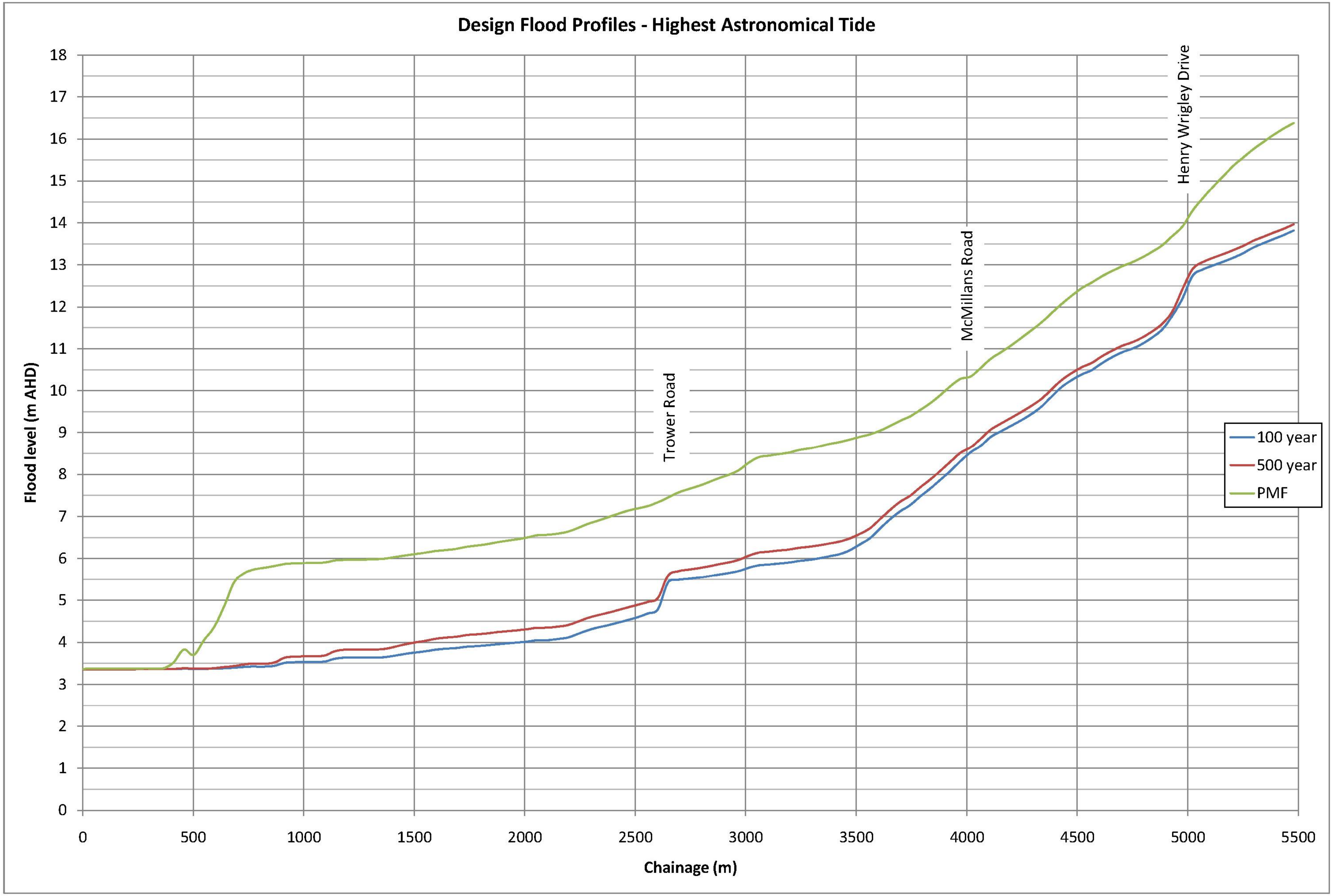
Appendix A Design flood profiles

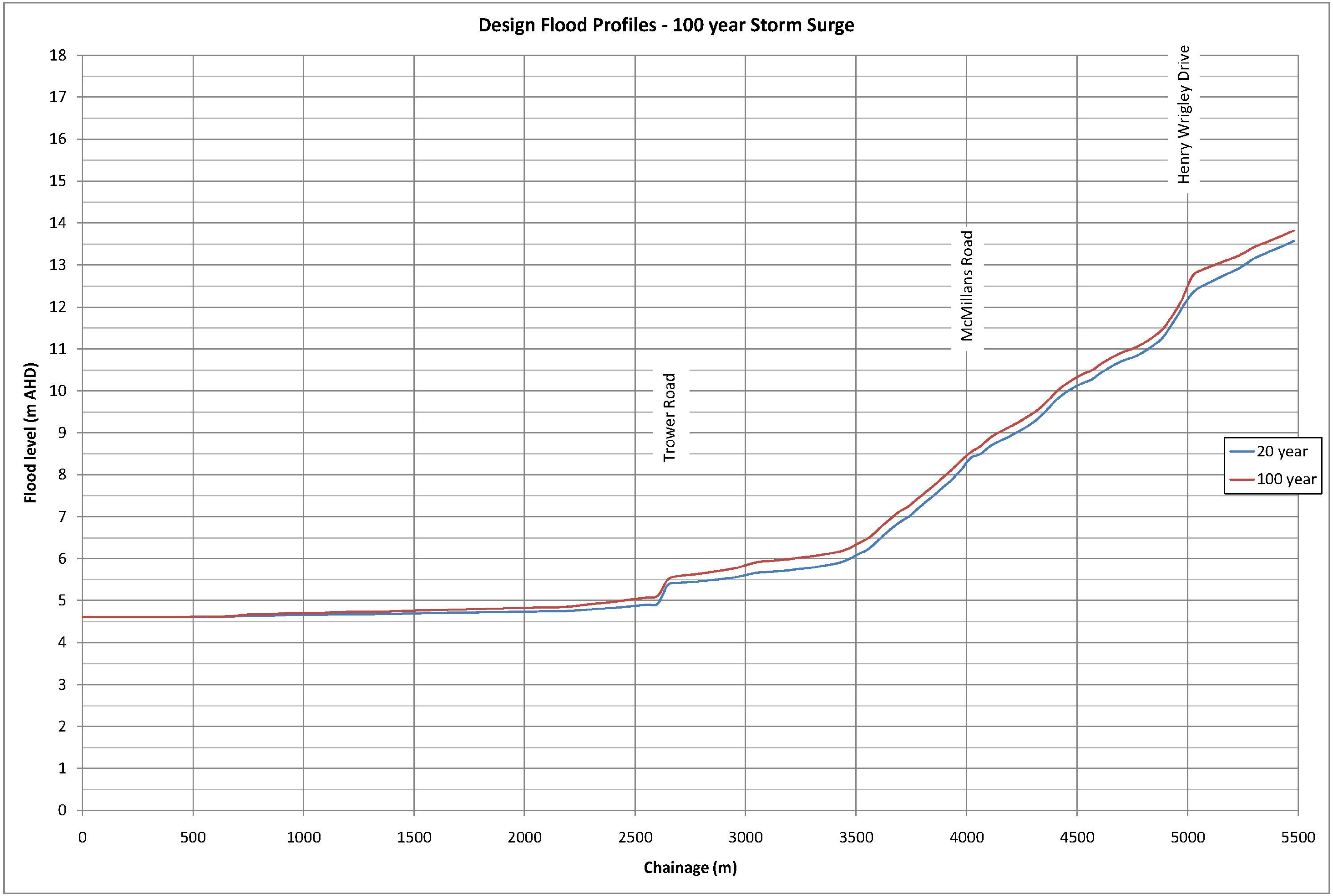
- **Figure 17. Design flood profiles**

1. **Mean sea level**
2. **Highest Astronomical Tide**
3. **100 year storm surge**
4. **2100 Highest Astronomical Tide + 0.8 m**

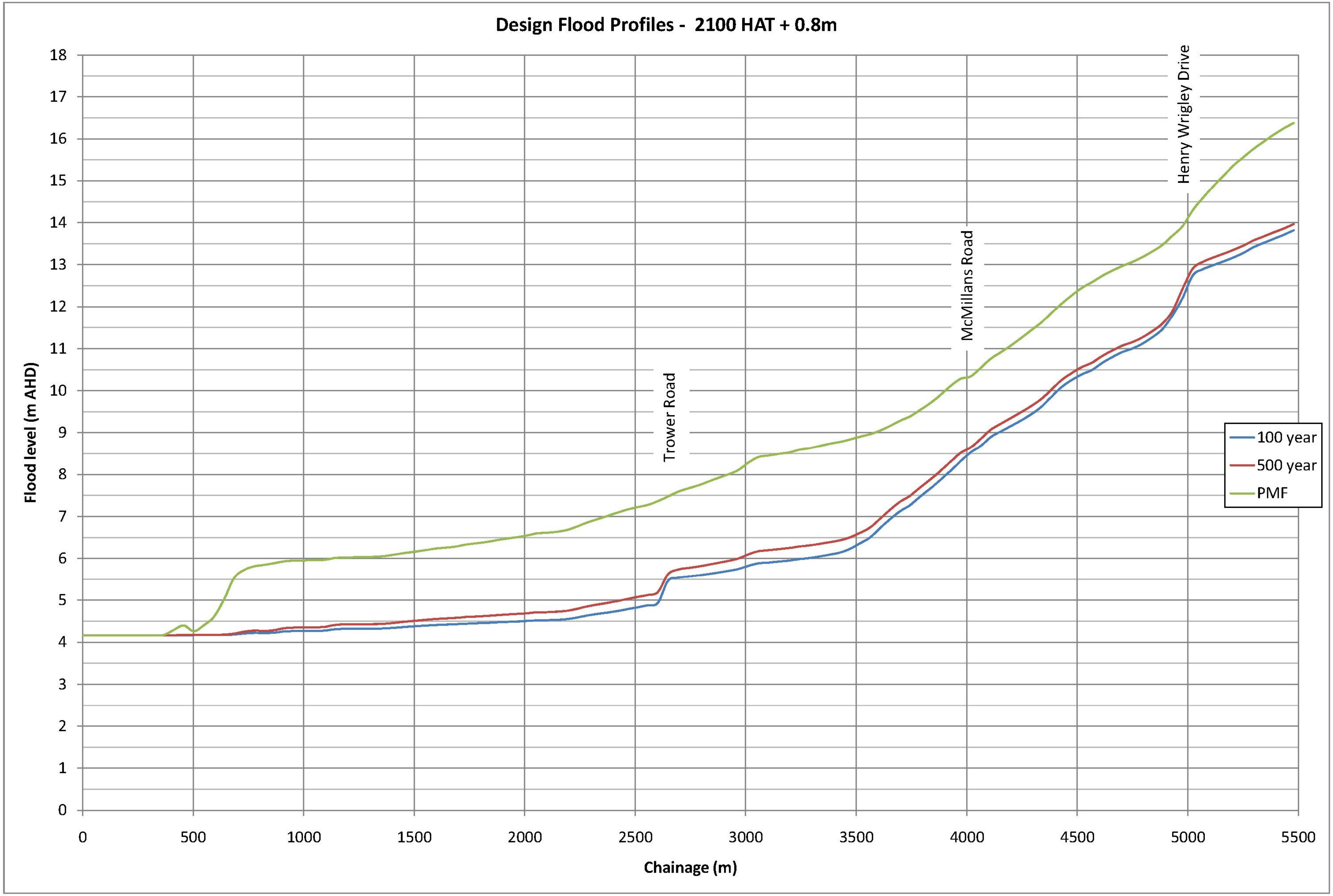


Design Flood Profiles - Mean Sea Level
Flood level (m AHD)
Chainage (m)
Trower Road
McMillans Road
Henry Wrigley Drive
20 year
50 year
100 year
200 year
500 year
PMF
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500


Design Flood Profiles - Highest Astronomical Tide
Flood level (m AHD)
Chainage (m)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
Trower Road
McMillans Road
Henry Wrigley Drive
100 year
500 year
PMF



Design Flood Profiles - 100 year Storm Surge
Flood level (m AHD)
Chainage (m)
Trower Road
McMillans Road
Henry Wrigley Drive
20 year
100 year
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500



Design Flood Profiles - 2100 HAT + 0.8m
Flood level (m AHD)
Chainage (m)
Trower Road
McMillans Road
Henry Wrigley Drive
100 year
500 year
PMF
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500

Appendix B Study technical brief

RAPID CREEK FLOOD STUDY

CONSULTANCY TECHNICAL BRIEF

Rapid Creek Flood Study

Technical Brief

General

Rapid Creek experienced unprecedented flooding caused by heavy rainfall over Darwin in February 2011, during the formation of Cyclone Carlos. A rainfall of 340 mm was recorded at Darwin Airport on 15 February, 2011 and the gauge on Rapid Creek downstream of McMillan's Road recorded a peak height of 3.74 m on the same day. The highest peak attained at this gauge before this event was in March 1977 when a height of 3.67 m was recorded. A number of houses in the suburb of Rapid Creek were affected and some were seriously damaged by flooding during this event.

Rapid Creek rises in the Marrara Swamp at the eastern end of Darwin Airport, and flows for 9.8 km discharging into Beagle Gulf at the southern end of Casuarina Beach (Figure 1). The Rapid Creek catchment covers an area of 28 sq km and includes parts of suburbs of Karama, Malak, Anula, Moil and Jingili. In these built up areas of the catchment, runoff enters the creek via underground piped drainage systems as well as unlined and lined open drains. Large parts of the catchment to the south of McMillans Road is still undeveloped. The Marrara Swamp is drained by 2 separate drainage lines, one on the north western and the other on the south western side of the swamp. Where the 2 drainage lines re-join to form Rapid Creek, a flood control weir exists which attenuates the peak discharge and delays the floodwaters

The Rapid Creek Planning Concepts and Land Use Objectives forms the framework for future development in the policy area which includes all land bordered by Trower Road, Freshwater Road, Seabright Crescent, McMillans Road and Rapid Creek Road. It is accepted planning practice to confine development to land lying below the 1.0% AEP flood level, although, as an exception elevated dwellings may be located within the 1% AEP flood provided habitable rooms are a minimum of 300mm above the flood level.

A number of earlier studies have been carried out to estimate flooding in Rapid Creek. Please see references (page 3).

The *Rapid Creek Flood Study* (Connell Wagner, 1999, 2004), estimated the flood levels associated with the 5.0% AEP (1 in 20), 2.0% AEP (1 in 50), 1.0% (1 in 100) and the probable maximum flood (PMF). The study clearly indicated that a considerable portion of the policy area would be inundated by a 1.0% AEP flood. The study indicated that any further significant development on the floodplain of Rapid Creek in the policy area has the potential to raise the flood level adjacent to and for some distance downstream of that development. This would result in flooding further into the urban residential area of Millner. An extension of the flood study also carried out by Connell Wagner (2004) looked at the possible impacts of future development of the Darwin Airport on flooding in the policy area. The study suggested that the development of airport land within the Rapid Creek catchment for commercial and aviation related purposes has the potential to aggravate flooding in the policy area, unless necessary measures are implemented to minimise this effect. The flooding investigation did not consider flooding caused by storm surge, although the study considered the issue of coincident floods and concluded that the chance of the two events coinciding in time is very small.

The inundation in the Rapid Creek area caused by cyclone Carlos in February 2011, far exceeded the 1% AEP estimated in the 1999 flood study with record heights recorded at the NRETAS stream gauge downstream of McMillans Road. Flood levels for this event were marked and consequently levelled and photographed.

Aim

(1) to carry out a flood study of Rapid Creek using a 2 dimensional hydraulic flood model preferably TUFLOW (which allows flood flow in a complex flood plain to be modelled) using currently available data now available (topographical data, realigned roads and drainage, streamflow, rainfall and available flood marks), to determine the extent and severity of flooding in the Rapid Creek environs, caused by joint storm drainage, riverine and/or tidal/storm surge influences. The model developed must be suitable for assessment of floodplain management strategies including land use planning and counter disaster management. It must be adaptable as a planning tool to predict the effect of changes on the floodplain which may be proposed in the future, including levees, upgrades to main roads and new infrastructure developments, and

(2) using the model, determine potential options to mitigate residential flooding downstream and examine the engineering feasibility of the options determined and make recommendations.

Scope of Work

(i) Hydrologic Analysis

Design flood discharges (Q20, Q50, Q100, Q200, Q500, PMF) should be derived using a hydrological runoff routing model (URBS) and calibrated to all available flood events, and by frequency analysis. The guidelines set out in Australian Rainfall and Runoff; Book 3: Choice of flood Estimation Methods and Design Standards (IE Aust 1998) should be used in the choice of the method used to estimate the design floods. The PMF discharge should also be derived using maximum precipitation estimates.

(ii) Hydraulic Modelling

The 2 dimensional hydraulic model (TUFLOW) of Rapid Creek and floodplain will be required. It should cover the areas covered by the attached map, and it must be suitably calibrated to recorded flood height data.

The model developed must be suitable for assessment of floodplain management strategies including land use planning and counter disaster management. It must be adaptable as a planning tool to predict the effect of changes on the floodplain which may be proposed in the future, including levees, upgrades to main roads and new infrastructure developments.

(iii) Flood Mapping

The calibrated hydraulic model should be used to determine the peak levels attained in the floodplain for the following scenarios;

2012

(1) Q20, Q50, Q100, Q200, Q500 and PMF with mean sea level
(2) Q100, Q500 and PMF with HAT
(3) Q20 with 100 year storm surge
(4) Q100 with 100 year storm surge

2100

Q100, Q500 and PMF with HAT + 0.8m

Flood maps including peak flood contours at appropriate intervals (in digital form compatible with client systems – Map Info preferred) should be prepared for the whole of the modelled areas. Maps showing flood surfaces for flood depth ranges 0-0.5m, 0.5-1.2m, 1.2 – 2.0 m and greater than 2.0 m are also required.

Also required are digital maps showing areas of high depth/velocity hazard (floodway, defined as areas where flood depth is greater than 2m or velocity x height is greater than 1).

(iv) Supply of Software and Provision of Training

The calibrated models should be made available to the client.

Training is also to be added in the quote but separately to provide to Water Resources professionals in the use of the models, including adaptation of the models to allow for the effect of floodplain developments and future mitigation measures.

Available Data and Information

(i) Topographic Mapping

Lidar mapping is available for whole of the Rapid Creek Catchment and environs.

(ii) Reports (some available through Water Resources Branch):

1. Willing and Partners Pty. Ltd. & Snowy Mountains Engineering Corporation July 1976 – Marrara Swamp – Rapid Creek Stormwater Drainage Study.
2. DJ. Dwyer and Associates, 1980 – Rapid Creek Recreational Project, Darwin NT.
3. Cameron McNamara (1982) – Report on Rapid Creek Hydrology Studies. Report No. 27/82D, Northern Territory of Australia. http://www.ntlis.nt.gov.au/hpa-services/techreport?report_id=WRD82027
4. Roads Division (1986) – Report on Rapid Creek Flows at the McMillans Road Crossing. Report No. 76/86D, Department of Transport and Works, Roads Division.
   http://www.ntlis.nt.gov.au/hpa-services/techreport?report_id=WRD86076
5. Sinclair Knight Merz, (1999) – Darwin Airport Development Master Plan, Rapid Creek and Coconut Grove Drainage Impact Study. Prepared for NT Airports Limited.
6. Connell Wagner (1999/2004) – Rapid Creek Flood Study Revised Final Report. Report No. 27/2000E,
   http://www.ntlis.nt.gov.au/hpa-services/techreport?report_id=WRD00411

7. Connell Wagner (2004) – Rapid Creek Flood Study: Assessment of Impact of Airport Development. Report No. 26/2000E,
http://www.ntlis.nt.gov.au/hpa-services/techreport?report_id=WRD00410
8. SKM (2007) Rapid Creek Flood Control Weir Assessment. Assessment Report. SKM, 2007
9. SKM (2011) – Rapid Creek. Impact of Proposed Flood Control Weir Upgrade and Characterisation of Flooding at Kimmorley Bridge.

Note: NRETAS reports are available through the site: http://www.nt.gov.au/landwater/index.jsp

(iii) NRETAS River and Rainfall Stations

Station		Easting	Northing	Opened	Closed

Datum MGA94

River

Current:

G8150127*	Rapid Creek Downstream McMillans Rd	703309.9	8629293.1	10/10/1960	current
G8150231	Moil urban drain opposite Airport hotel	704212.9	8628928.0	15/10/1984	

Historic:

G8150024	Rapid Creek tide gauge at Trower Rd bridge	702861.9	8630136.0	17/02/1981	27/03/2003
G8150128	Northern Suburbs drain 1 McMillans Rd	703739.9	8629080.0	12/11/1973	22/11/1974

Rainfall (In the region)

Current:

G8150231	Moil urban drain opposite Airport hotel	704212.9	8628928.0	15/10/1984	
G8150232	Karama urban drain opposite Vanderlin	708369.7	8629355.0	03/10/1984	

Historic:

R8150255	Rapid Creek at 4598 Freshwater Rd	703129.9	8629160.0	22/11/1977	21/04/1992
R8150256	Moil catchment at Primary School	704479.9	8629780.0	17/01/1985	20/02/2006
R8150257	Karama Primary School	708229.9	8628660.0	05/03/1984	05/07/2006

BOM Rainfall Stations (In the region)

Current:

DR014015	DARWIN AIRPORT	703049.9	862681631/12/1940	current	
DR014235	Northlakes	705924.8	862808431/10/1987	current	
DR014270	Nightcliff Camphor Street	701317.4	8630173.9	31/12/1966	current
DR014227	Karama	709078.7	8629600.0	31/03/1987	
DR014246	Leanyer	706880.9	8630292.0	31/10/1989	

Historic:

DR014112	NIGHTCLIFF PRIMARY SCHOOL	701262.9	8630515.9	31/12/1961	28/02/2009
DR014162	DARWIN (COCONUT GROVE)	701249.9	862867229/02/1964	31/03/2003	
DR014164	JINGILI	703062.9	862866028/02/1967	28/02/1994	
DR014214	Anula	704845.4	862923130/11/1977	30/09/1994	
DR014265	BERRIMAH C.C.N.T.	708879.9	862686731/12/1991	28/02/2009	
DR014280	Darwin RAAF Golf Club	702045.2	862693930/09/1993	30/04/1995	
DR014289	NT University	702772.5	8631425.3	31/10/1996	30/06/2007

* Station relocated in 1968

Deliverables

- Floodplain hydraulic model and hydrological model
- Draft Report and floodplain maps in digital form
- Final Report including floodplain maps in digital version in pdf format
- Digital version of the spatial data (flood surface and depth ranges) to be either in MapInfo or ESRI format, GDA94 datum and projection to be in Lat/Long (geographic) or UTM (Universal Transverse Mercator Zone 53).

Timetable, Payments and Contract Conditions

The consultant should provide a timetable relating activities and identified milestones to dates, and to progress payments if applicable.

Engagement will be in accordance with the Northern Territory Government's "Conditions: Tendering and Engineering Consultant Services – Version No. 4.1.26 of July 2010".

Project Officer and Liaison

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Jerome Paiva
Senior Engineer Water Resources Assessment

Department of Natural Resources, Environment, the Arts and Sport

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Fax: 89993666
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For Rainfall Data:

Nigel Mules
Manager Hydrology
Bureau of Meteorology
Ph: 89203838
Fax: 89203842
Email: n.mules@bom.gov.au

The consultant should be prepared to attend review meetings as required by the project officer and may also request such meetings in their own right.

Appendix C Flood plain maps

Figure 18.1 Flood Plain Map for Q20 and Present Day Mean Sea Level – Flood extent and flood surface contours
Figure 18.2 Flood Plain Map for Q20 and Present Day Mean Sea Level – Flood depth and extent

Figure 19.1 Flood Plain Maps for Q50 and Present Day Mean Sea Level – Flood extent and flood surface contours
Figure 19.2 Flood Plain Maps for Q50 and Present Day Mean Sea Level – Flood depth and extent

Figure 20.1 Flood Plain Maps for Q100 and Present Day Mean Sea Level – Flood extent and flood surface contours
Figure 20.2 Flood Plain Maps for Q100 and Present Day Mean Sea Level – Flood depth and extent

Figure 21.1 Flood Plain Maps for Q200 and Present Day Mean Sea Level – Flood extent and flood surface contours
Figure 21.2 Flood Plain Maps for Q200 and Present Day Mean Sea Level – Flood depth and extent

Figure 22.1 Flood Plain Maps for Q500 and Present Day Mean Sea Level – Flood extent and flood surface contours
Figure 22.2 Flood Plain Maps for Q500 and Present Day Mean Sea Level – Flood depth and extent

Figure 23.1 Flood Plain Maps for PMF and Present Day Mean Sea Level – Flood extent and flood surface contours
Figure 23.2 Flood Plain Maps for PMF and Present Day Mean Sea Level – Flood depth and extent

Figure 24.1 Flood Plain Maps for Q100 and Present Day Highest Astronomical Tide – Flood extent and flood surface contours
Figure 24.2 Flood Plain Maps for Q100 and Present Day Highest Astronomical Tide – Flood depth and extent

Figure 25.1 Flood Plain Maps for Q500 and Present Day Highest Astronomical Tide – Flood extent and flood surface contours
Figure 25.2 Flood Plain Maps for Q500 and Present Day Highest Astronomical Tide – Flood depth and extent

Figure 26.1 Flood Plain Maps for PMF and Present Day Highest Astronomical Tide – Flood extent and flood surface contours
Figure 26.2 Flood Plain Maps for PMF and Present Day Highest Astronomical Tide – Flood depth and extent

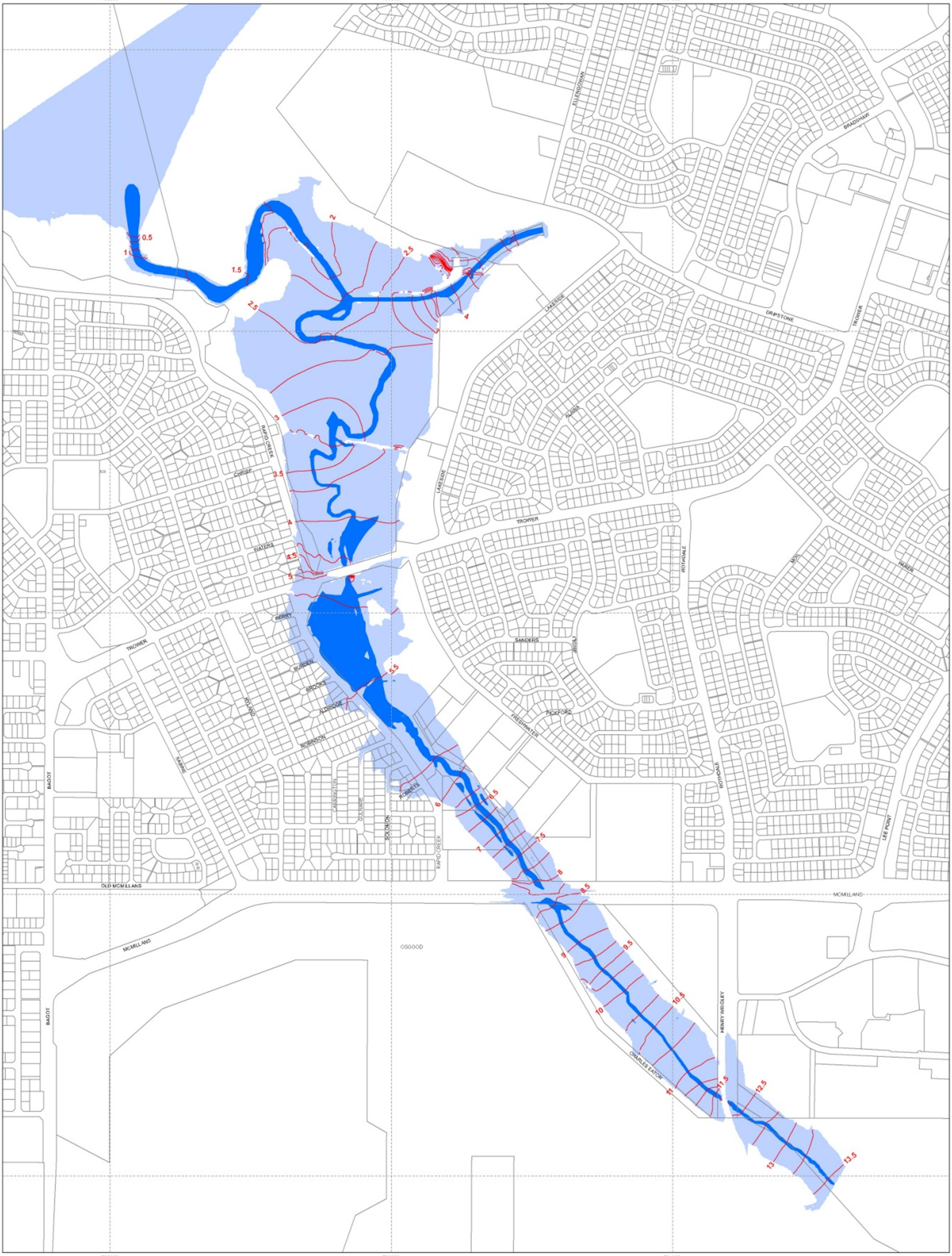
Figure 27.1 Flood Plain Maps for Q20 and Present Day 1% AEP Storm Surge – Flood extent and flood surface contours
Figure 27.2 Flood Plain Maps for Q20 and Present Day 1% AEP Storm Surge – Flood depth and extent

Figure 28.1 Flood Plain Maps for Q100 and Present Day 1% AEP Storm Surge – Flood extent and flood surface contours
Figure 28.2 Flood Plain Maps for Q100 and Present Day 1% AEP Storm Surge – Flood depth and extent

Figure 29.1 Flood Plain Maps for Q100 and year 2100 Highest Astronomical Tide – Flood extent and flood surface contours
Figure 29.2Flood Plain Maps for Q100 and year 2100 Highest Astronomical Tide – Flood depth and extent

Figure 30.1 Flood Plain Maps for Q500 and year 2100 Highest Astronomical Tide – Flood extent and flood surface contours
Figure 30.2 Flood Plain Maps for Q500 and year 2100 Highest Astronomical Tide – Flood depth and extent

Figure 31.1 Flood Plain Maps for PMF and year 2100 Highest Astronomical Tide – Flood extent and flood surface contours
Figure 31.2 Flood Plain Maps for PMF and year 2100 Highest Astronomical Tide – Flood depth and extent



Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52

A4 1:12,500

0 500

Metres

Computed Q20 with Mean Sea Level Flood Extent and Flood Surface Contours

Figure 18.1 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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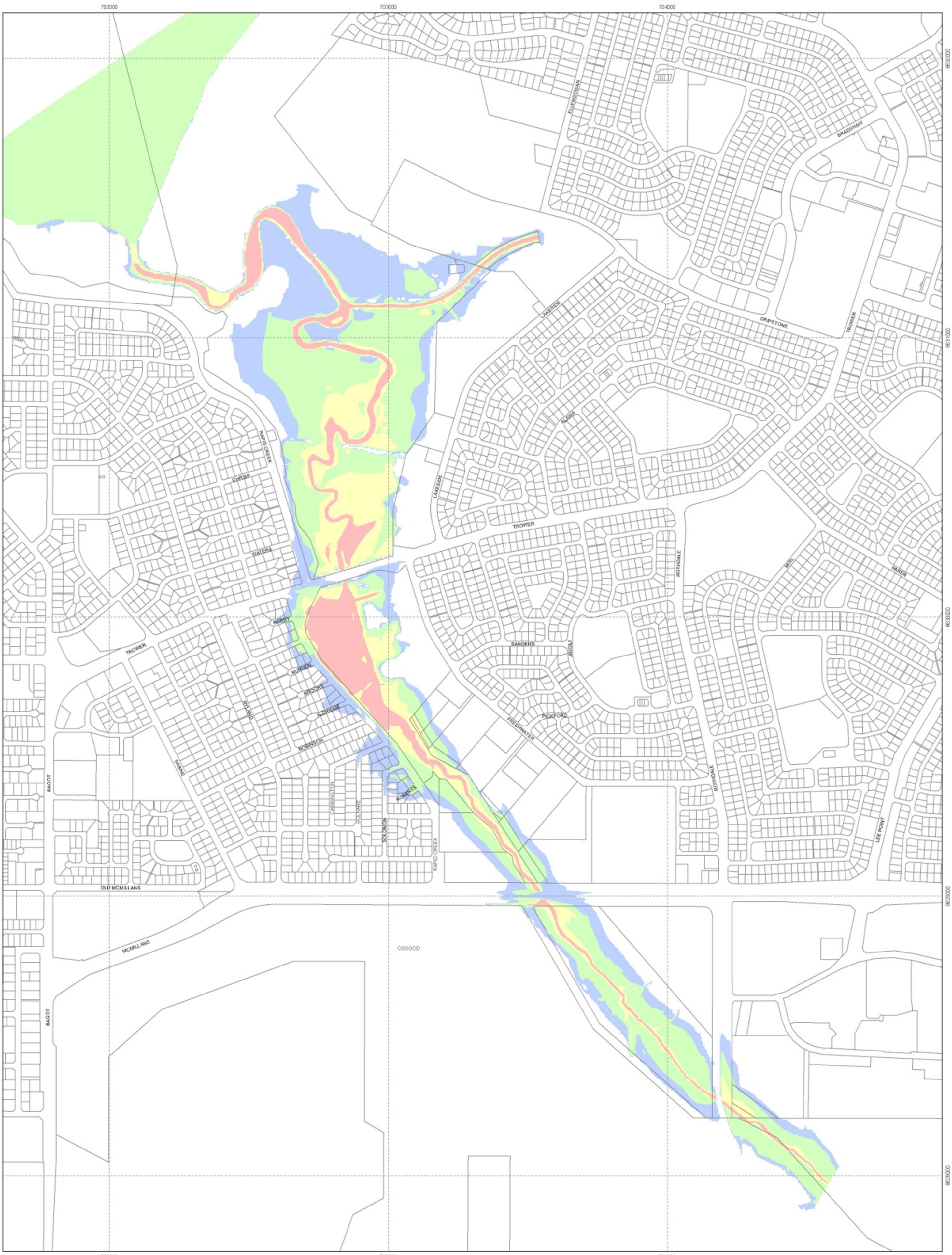
SKM



Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

Computed Q20 with Mean Sea Level Flood Depth and Extent

SKM

Figure 18.2 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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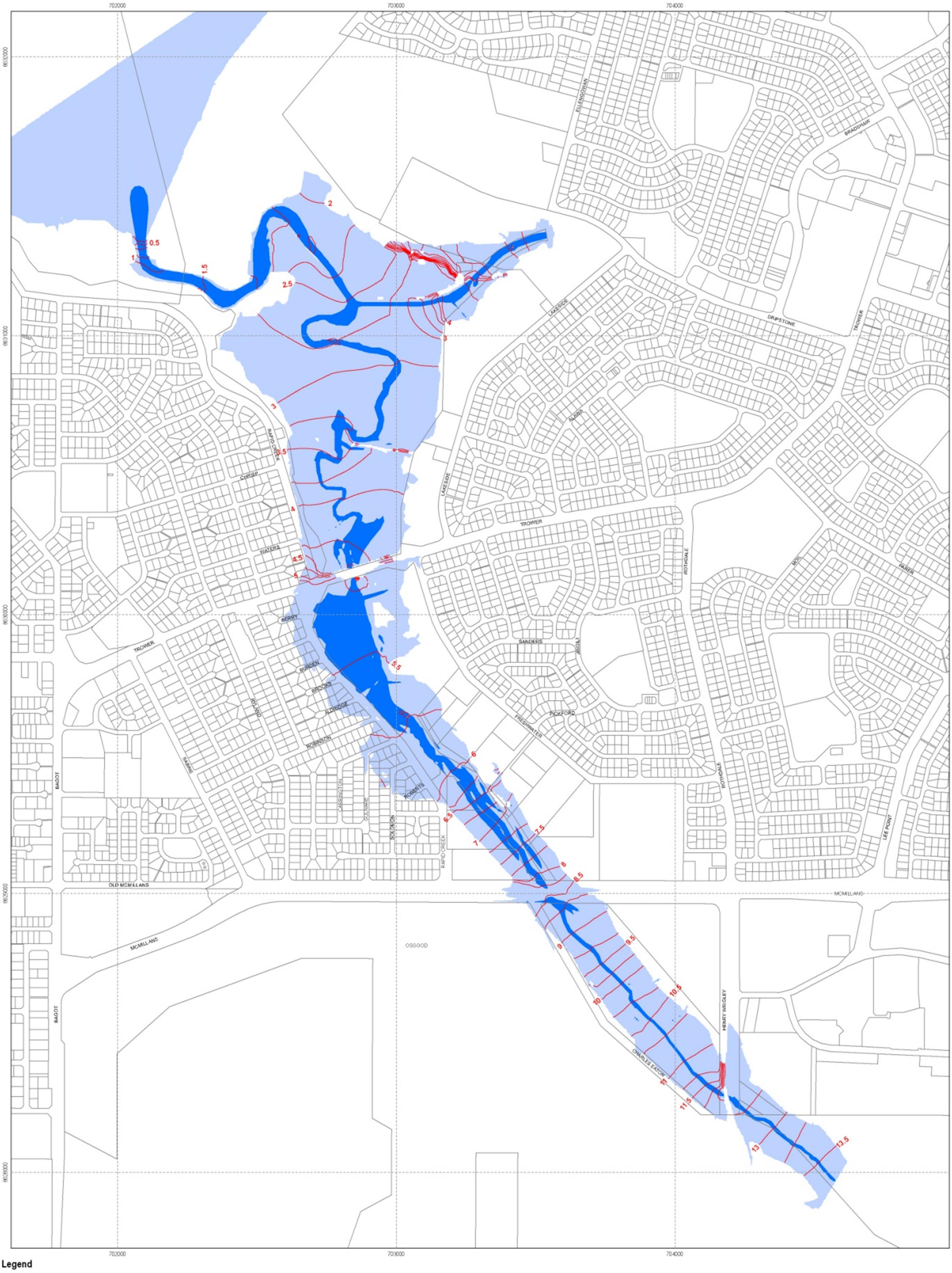

Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

Computed Q50 with Mean Sea Level Flood Extent and Flood Surface Contours

Figure 19.1 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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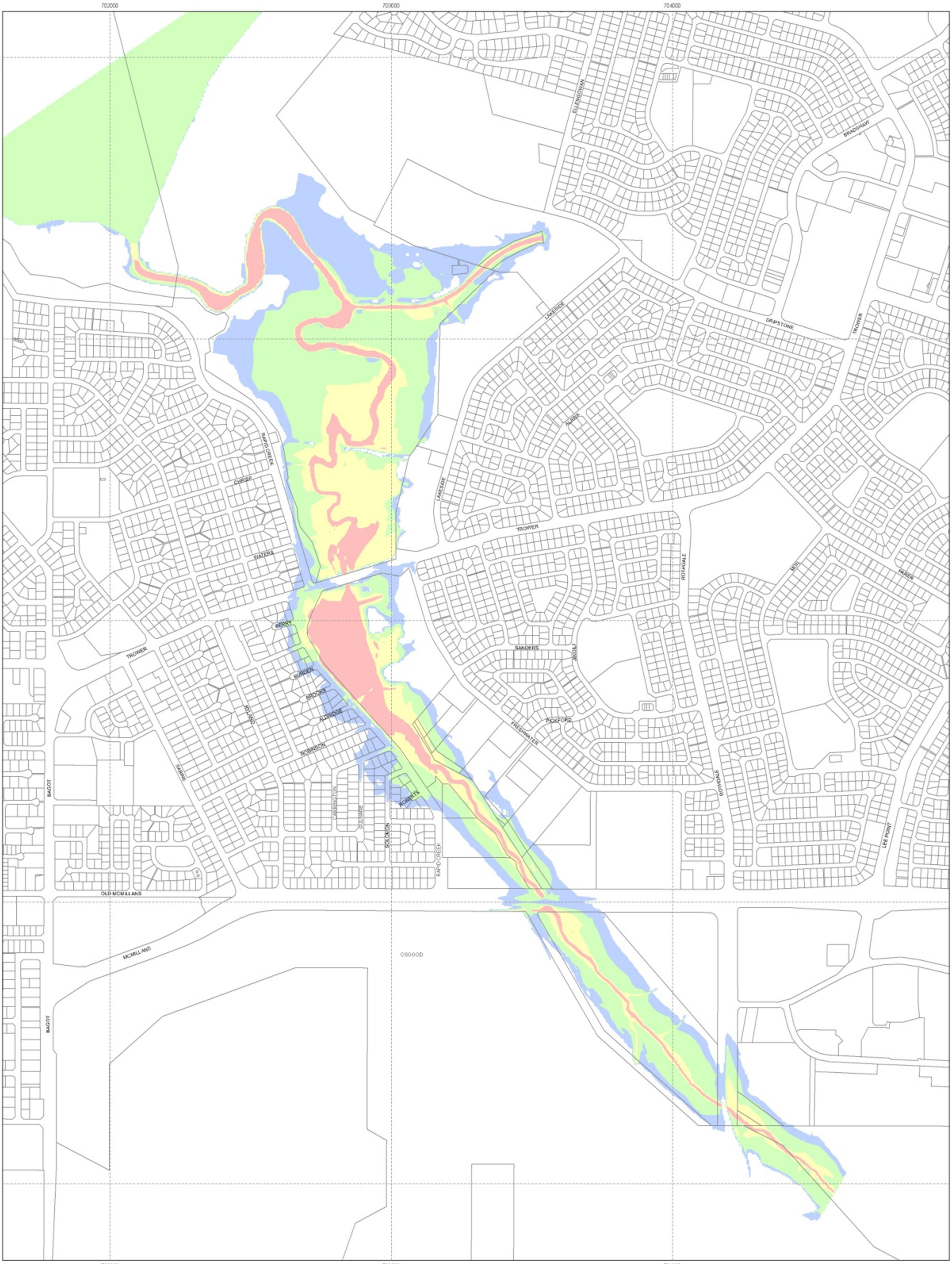

Legend

Flood Depth 0.0 - 05 metres	Property Boundary
Flood Depth 0.5 - 1.2 metre	
Flood Depth 1.2 - 2.0 metres	
Flood Depth > 2.0 metres	

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

DARWIN

Computed Q50 with Mean Sea Level Flood Depth and Extent

SKM

Figure 19.2 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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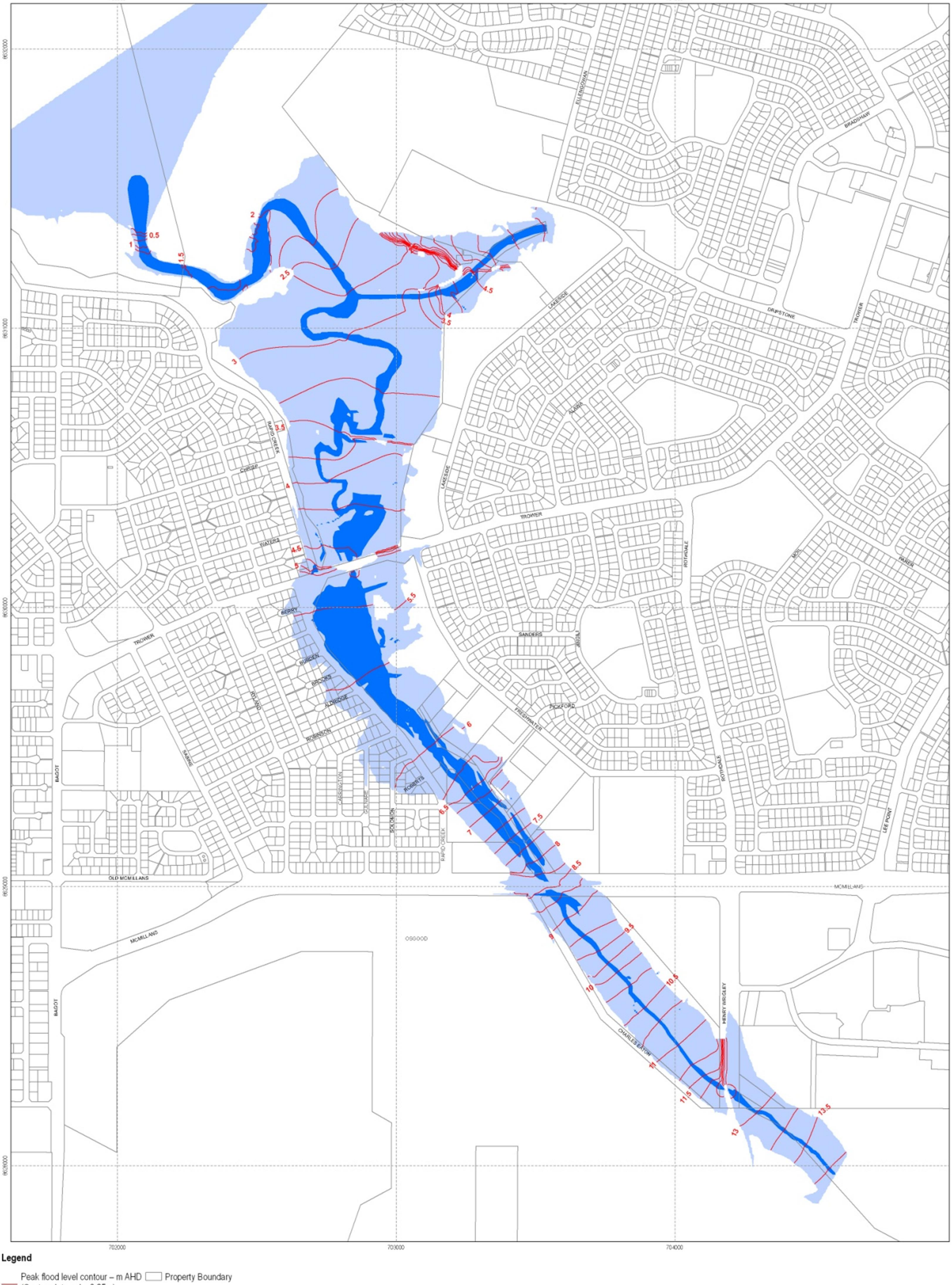

Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

DARWIN

Computed Q100 with Mean Sea Level Flood Extent and Flood Surface Contours

DB05724
Rapid Creek Flood Study

SKM

Figure 20.1 A

Wednesday, 30 May 2012
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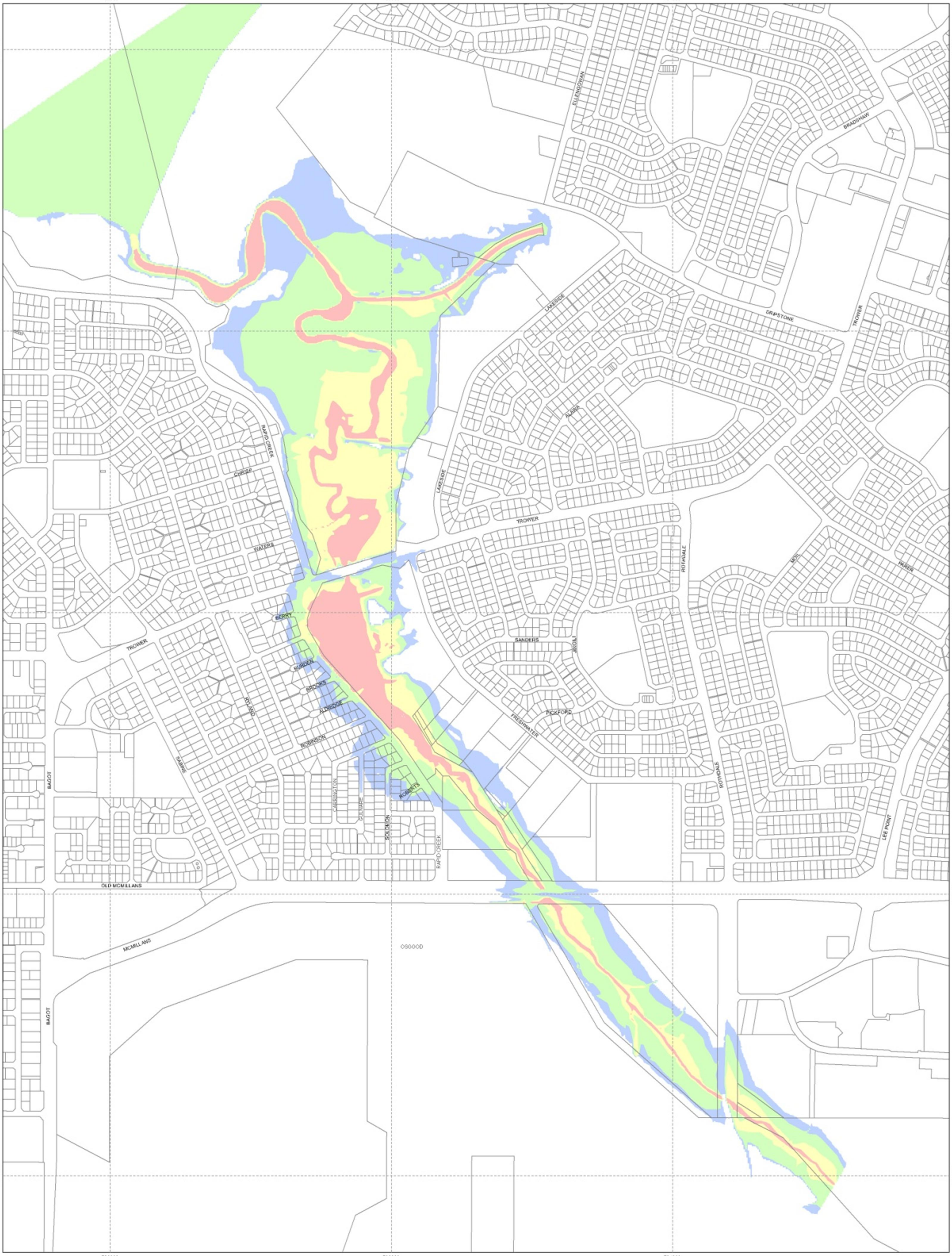

Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

Computed Q100 with Mean Sea Level Flood Depth and Extent

SKM

Figure 20.2 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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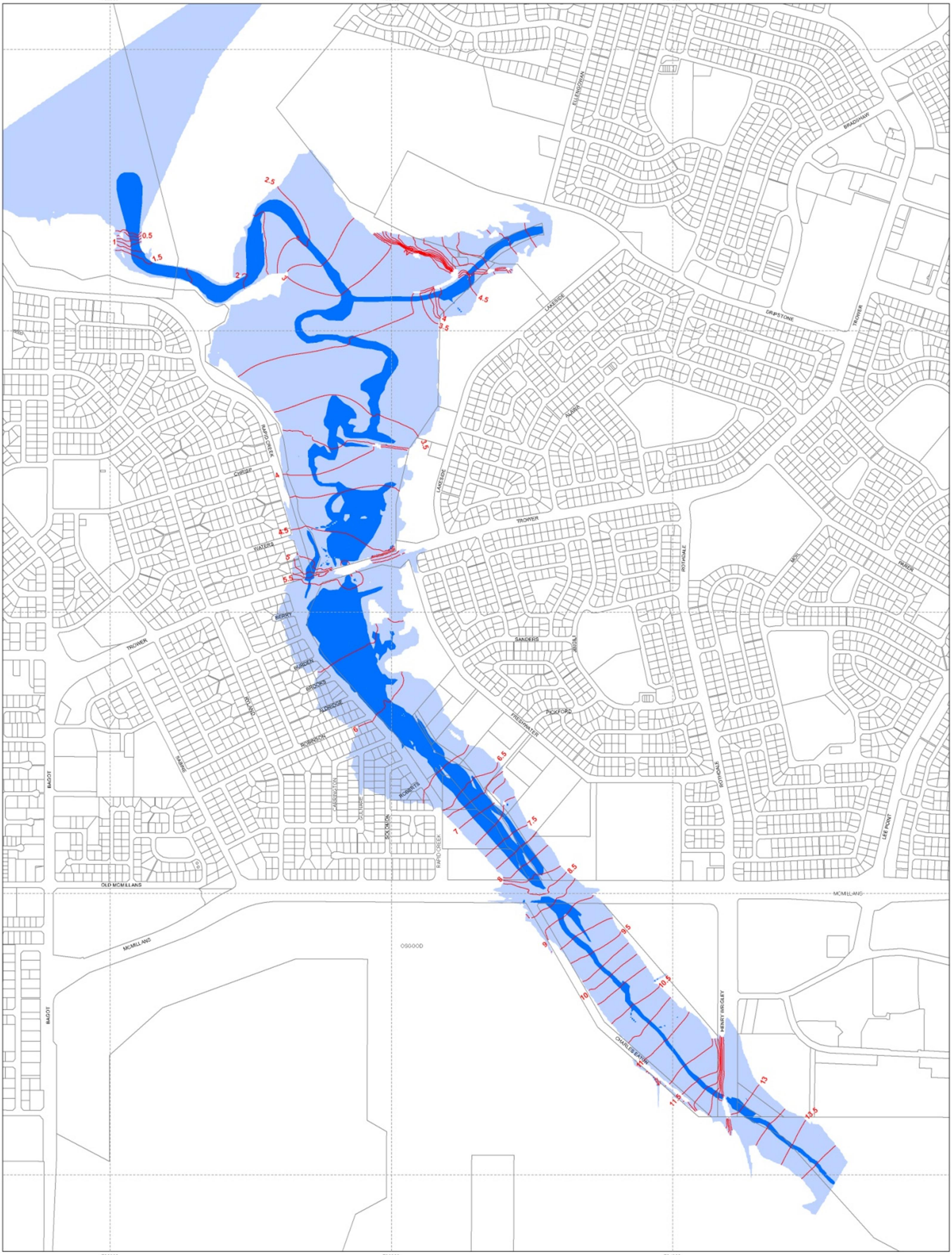

Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52

A4 1:12,500

0 500
Metres

Computed Q200 with Mean Sea Level Flood Extent and Flood Surface Contours

DB05724
Rapid Creek Flood Study

SKM

Figure 21.1 A

Wednesday, 30 May 2012
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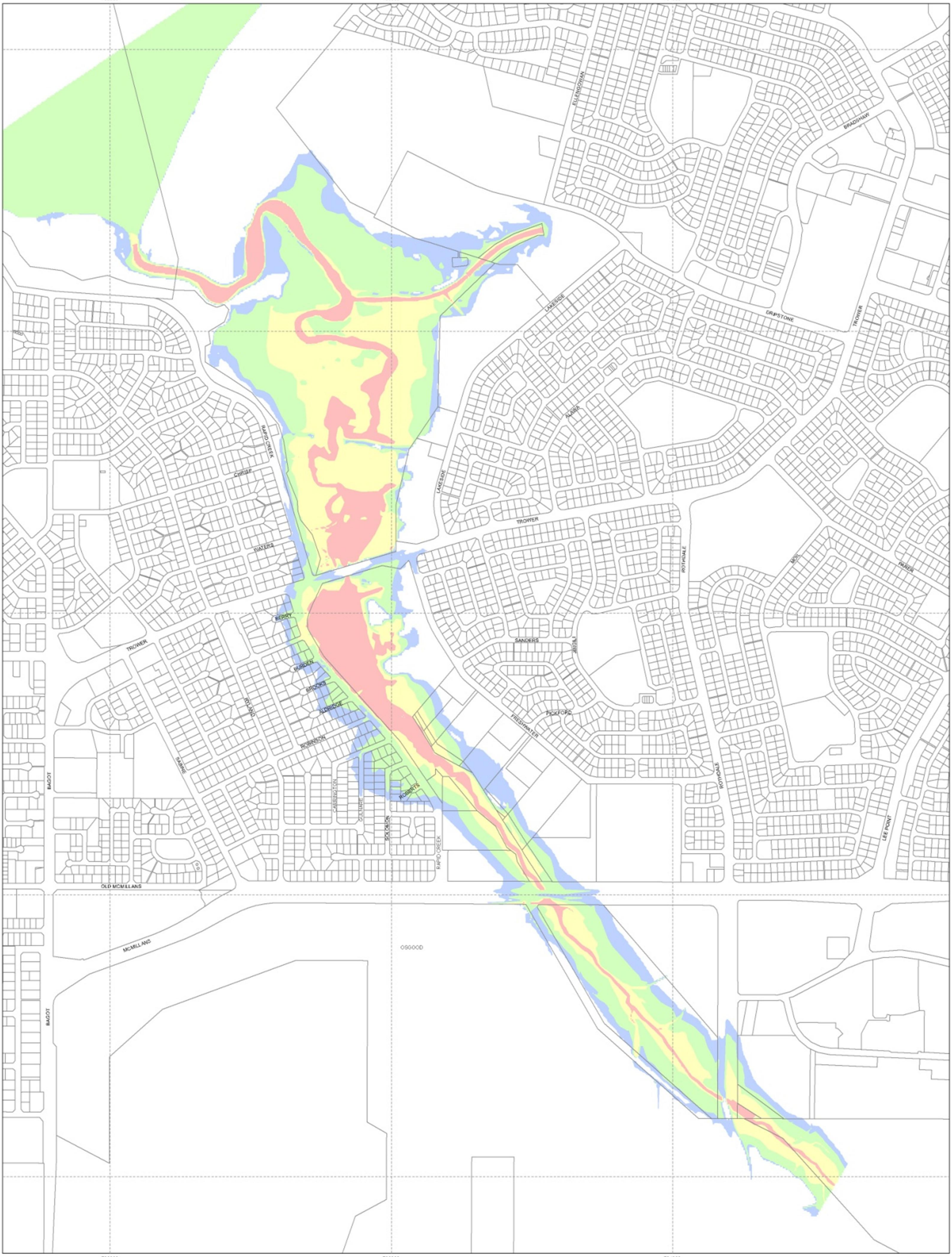

Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

Computed Q200 with Mean Sea Level Flood Depth and Extent

Figure 21.2 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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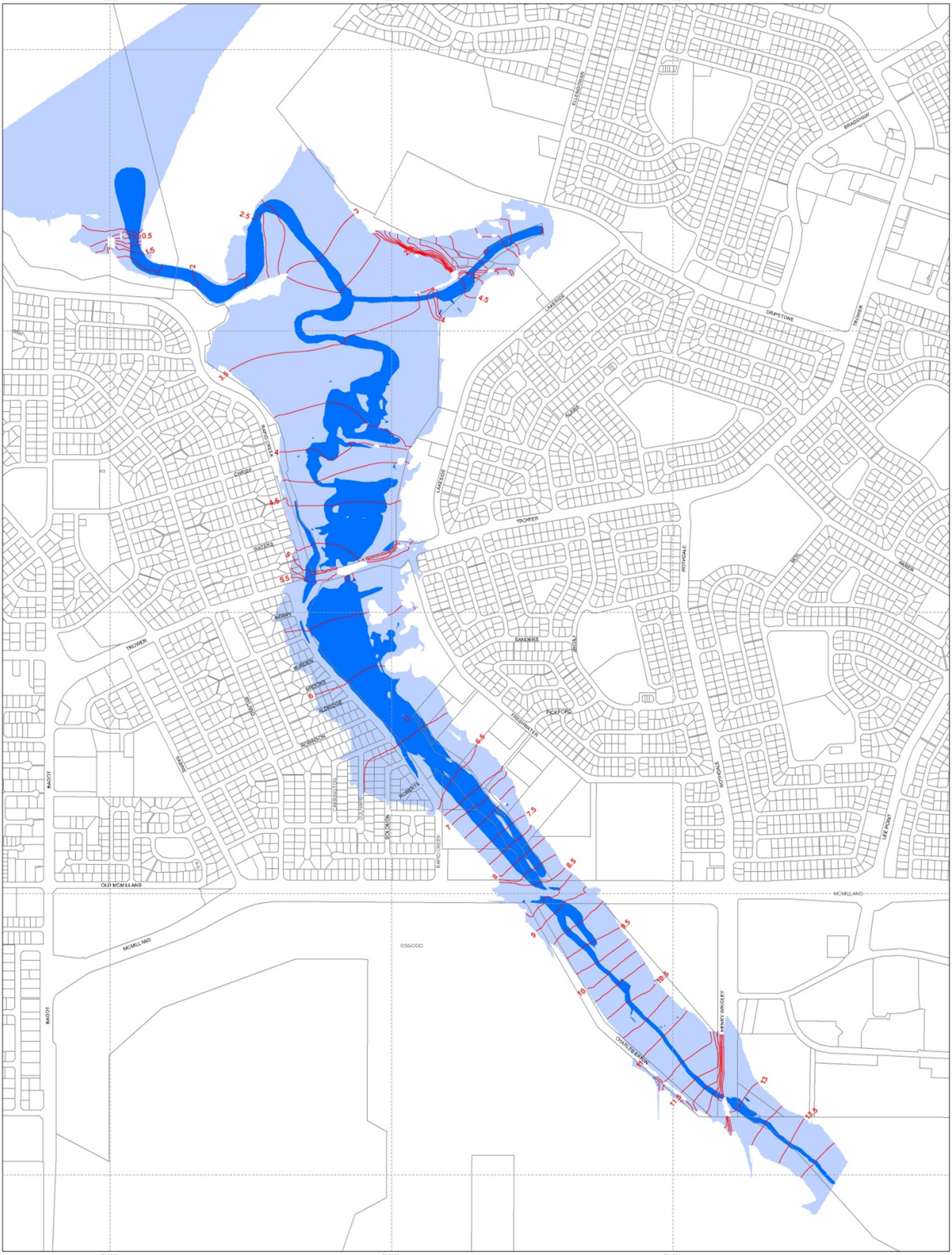

Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

SKM

Computed Q500 with Mean Sea Level Flood Extent and Flood Surface Contours

Figure 22.1 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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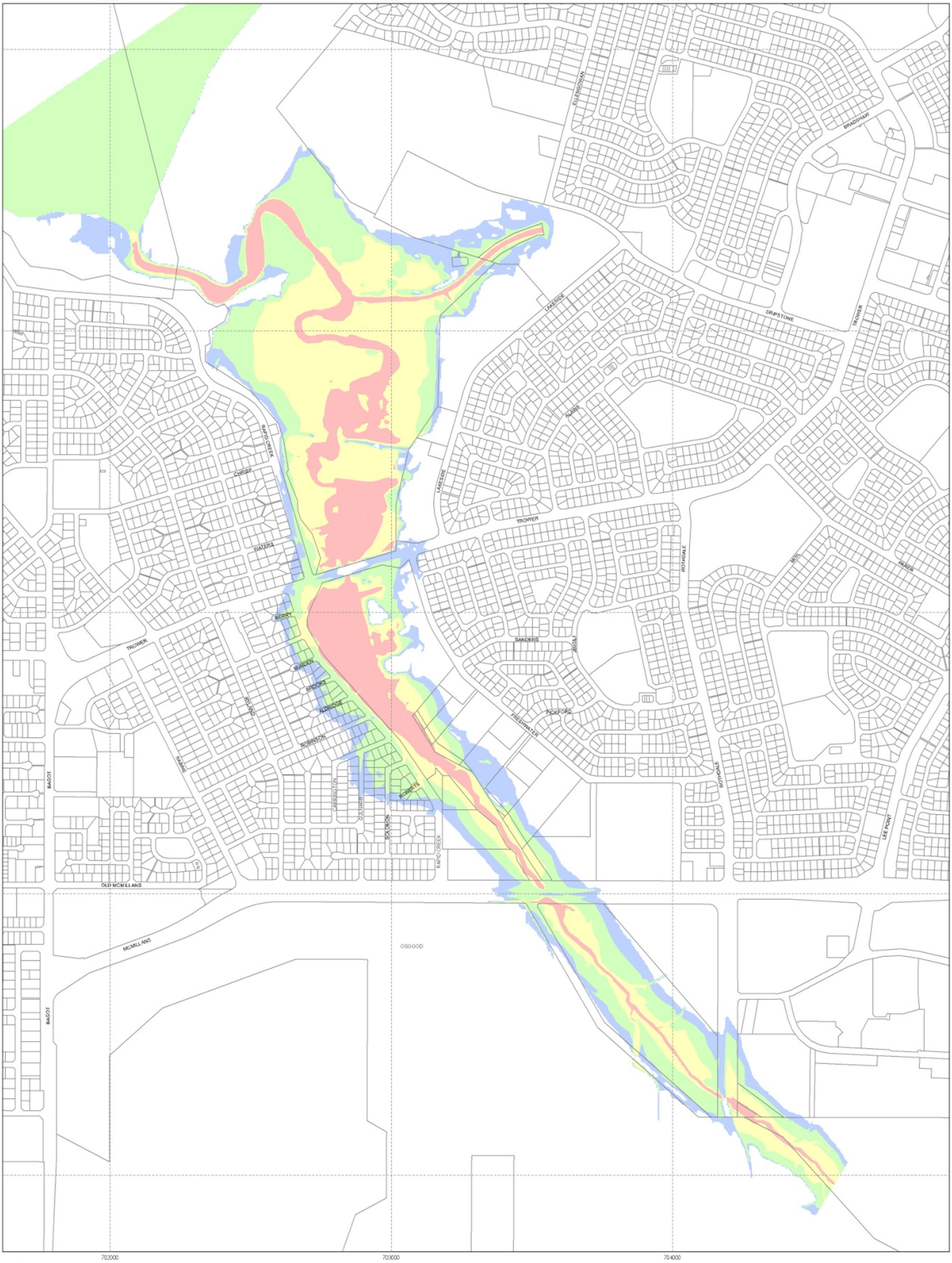

Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

Computed Q500 with Mean Sea Level Flood Depth and Extent

Figure 22.2

A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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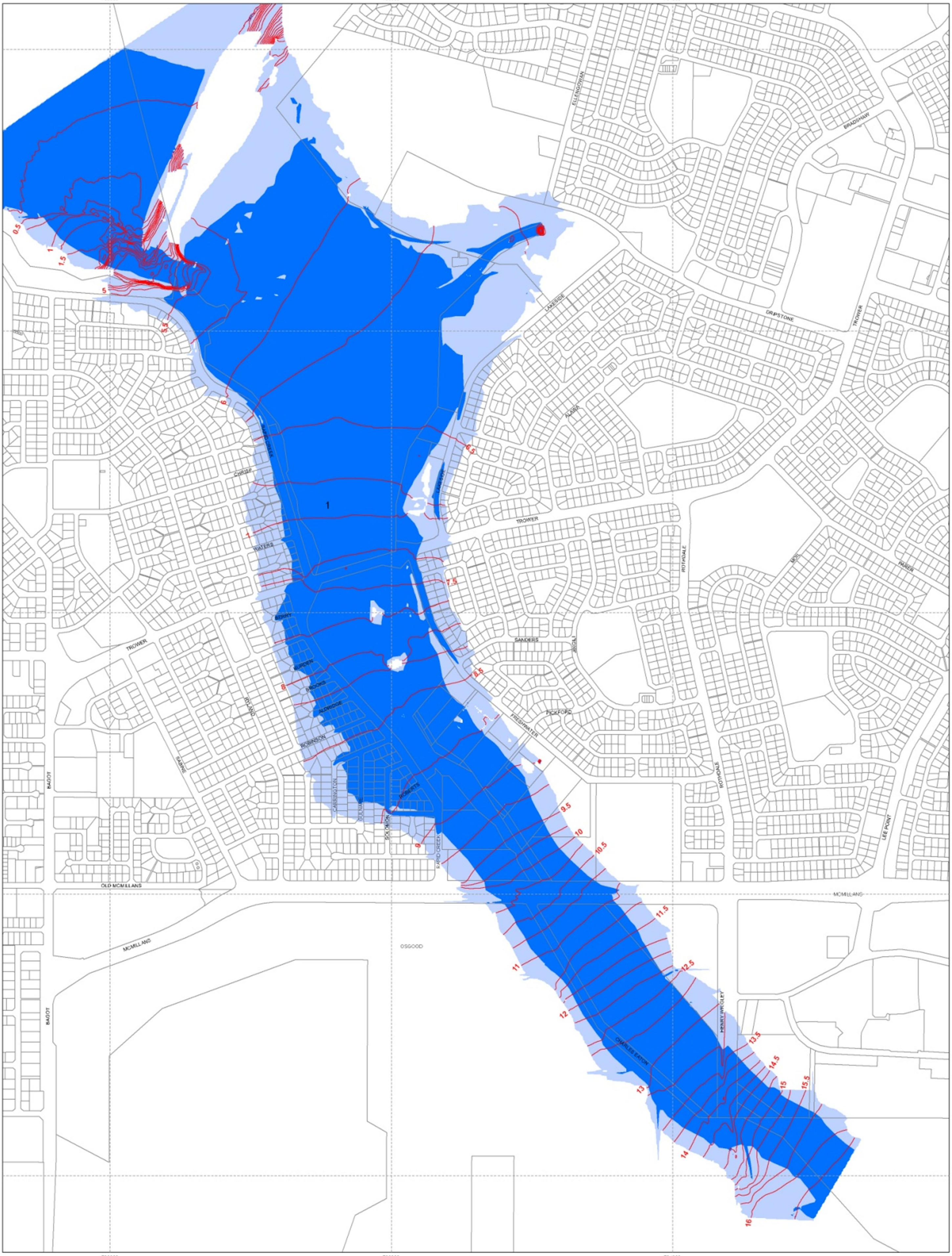
SKM



Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52

A4 1:12,500

0 500
Metres

SKM

Computed PMF with Mean Sea Level Flood Extent and Flood Surface Contours

Figure 23.1 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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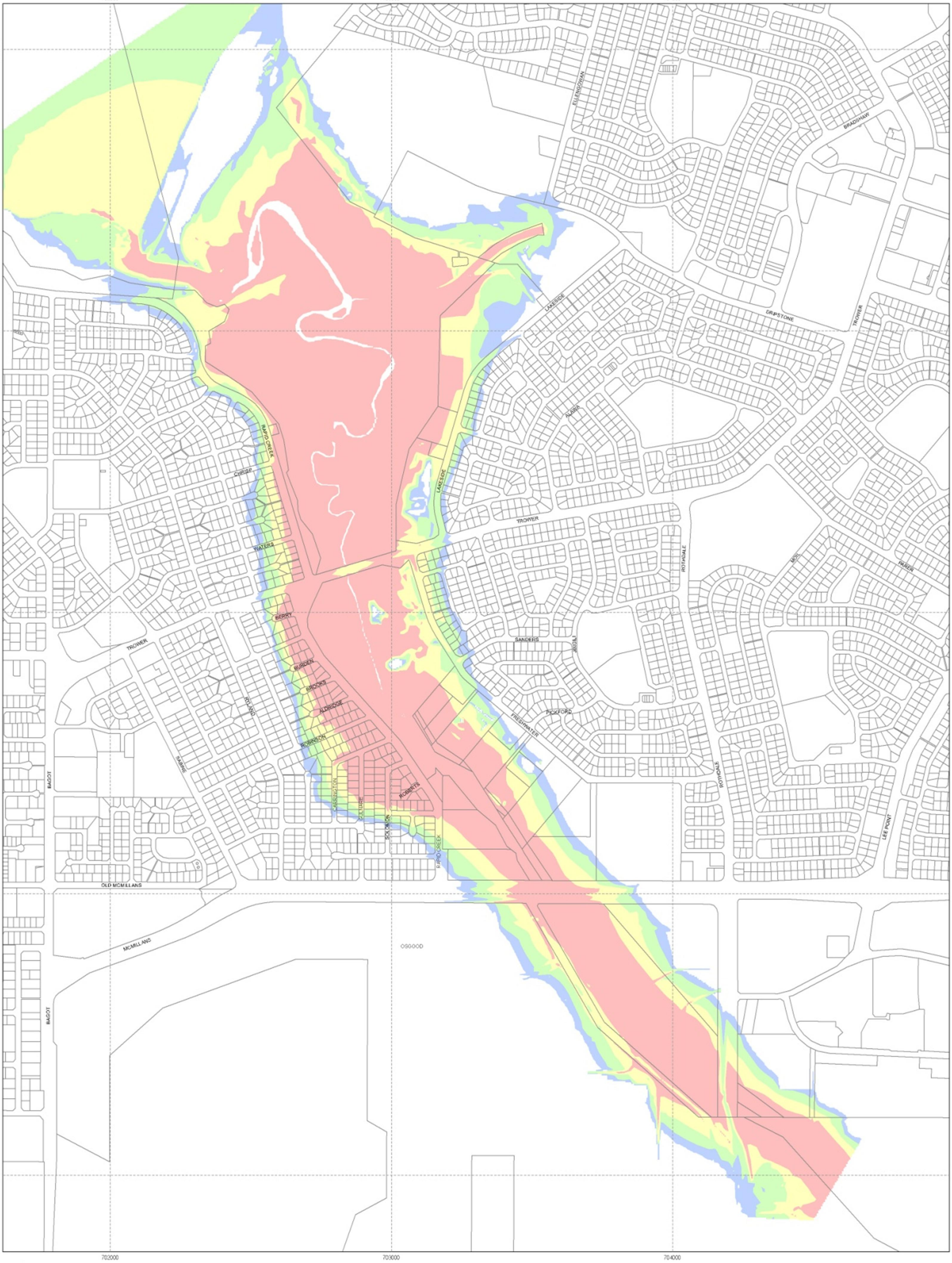

Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

SKM

Computed PMF with Mean Sea Level Flood Depth and Extent

Figure 23.2 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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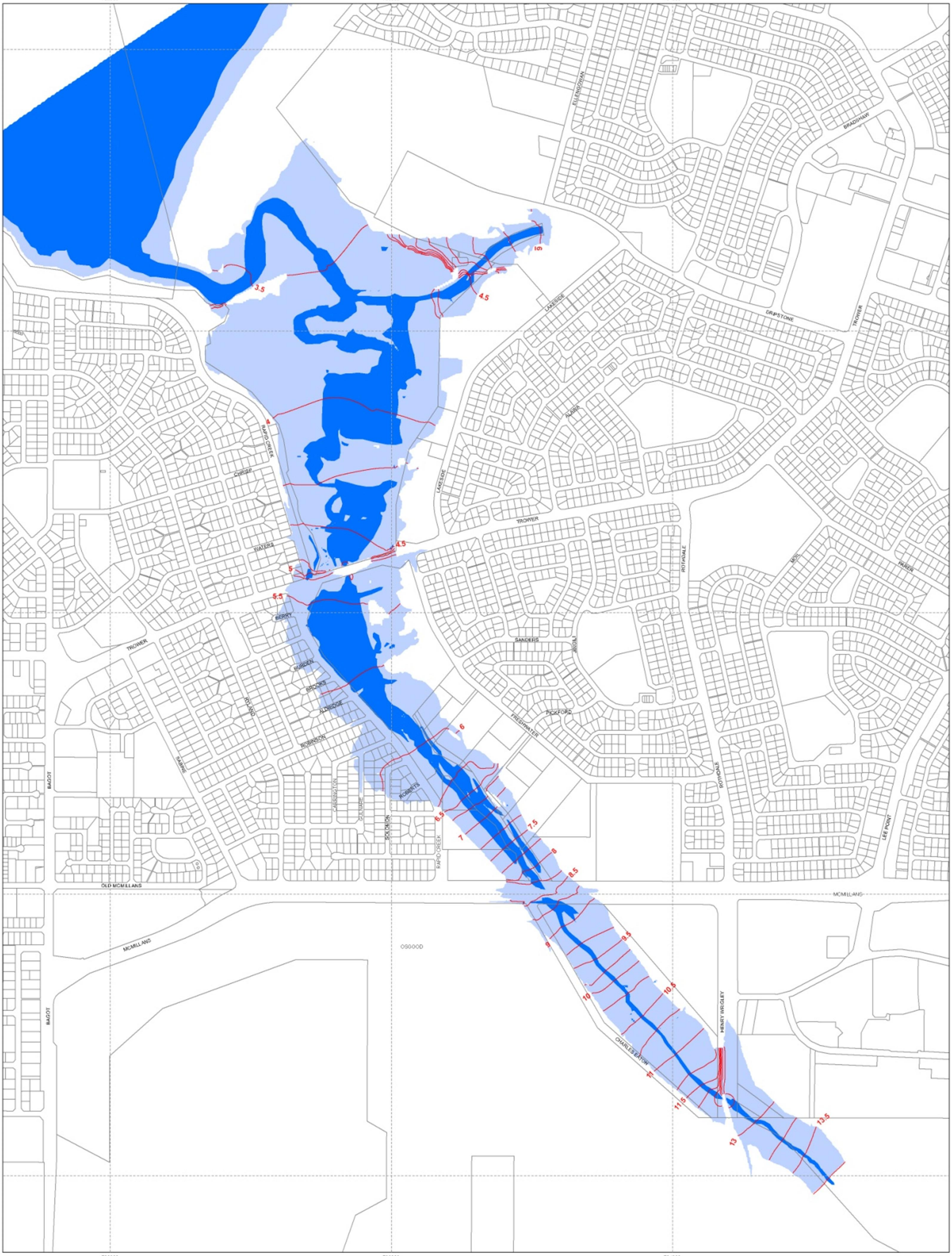

Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

DARWIN

Computed Q100 with Highest Astronomical Tide Flood Extent and Flood Surface Contours

DB05724
Rapid Creek Flood Study

SKM

Figure 24.1 A

Wednesday, 30 May 2012
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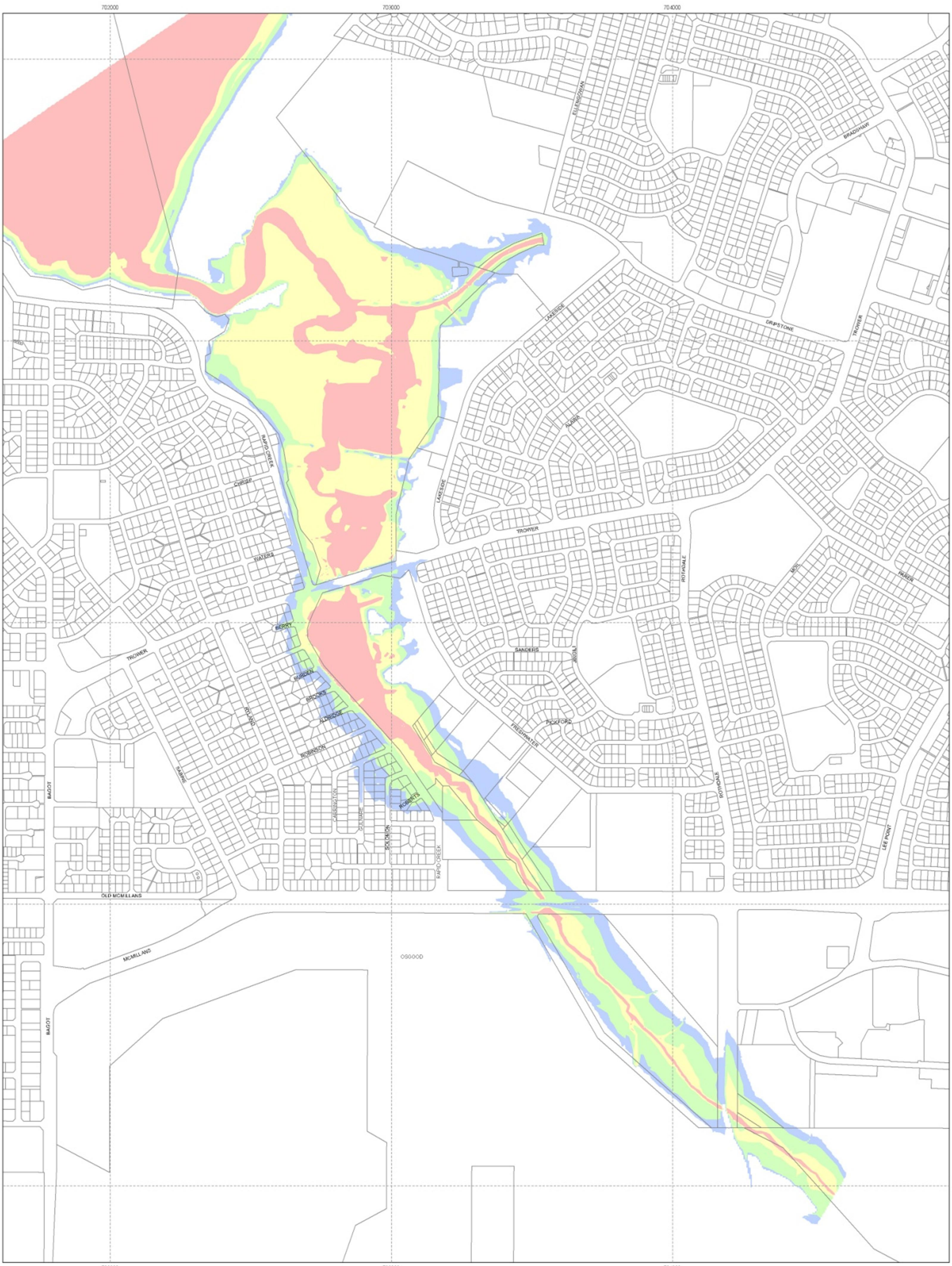

Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

DARWIN

Computed Q100 with Highest Astronomical Tide Flood Depth and Extent

Figure 24.2

A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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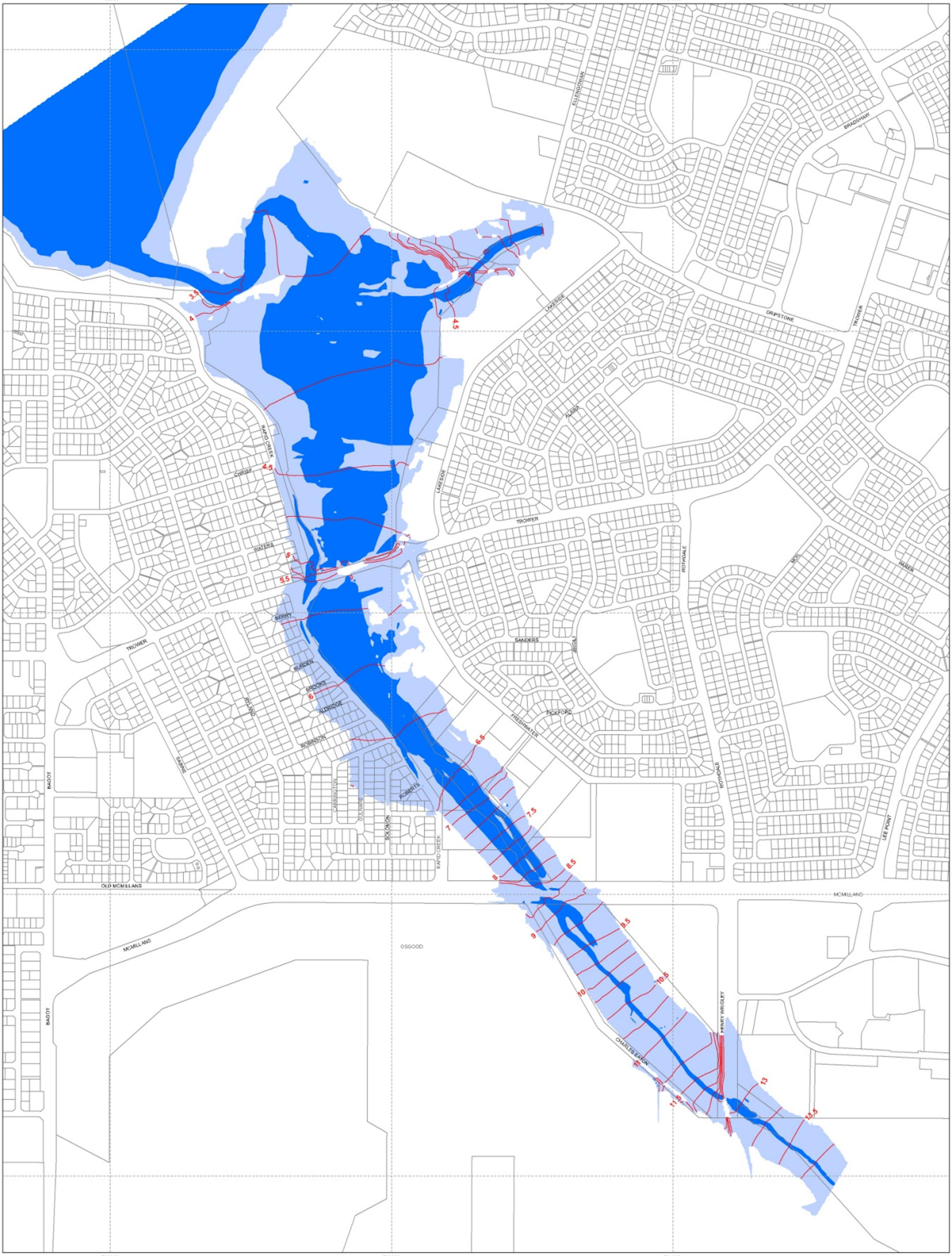
SKM



Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

DARWIN

Computed Q500 with Highest Astronomical Tide Flood Extent and Flood Surface Contours

DB05724
Rapid Creek Flood Study

SKM

Figure 25.1 A

Wednesday, 30 May 2012
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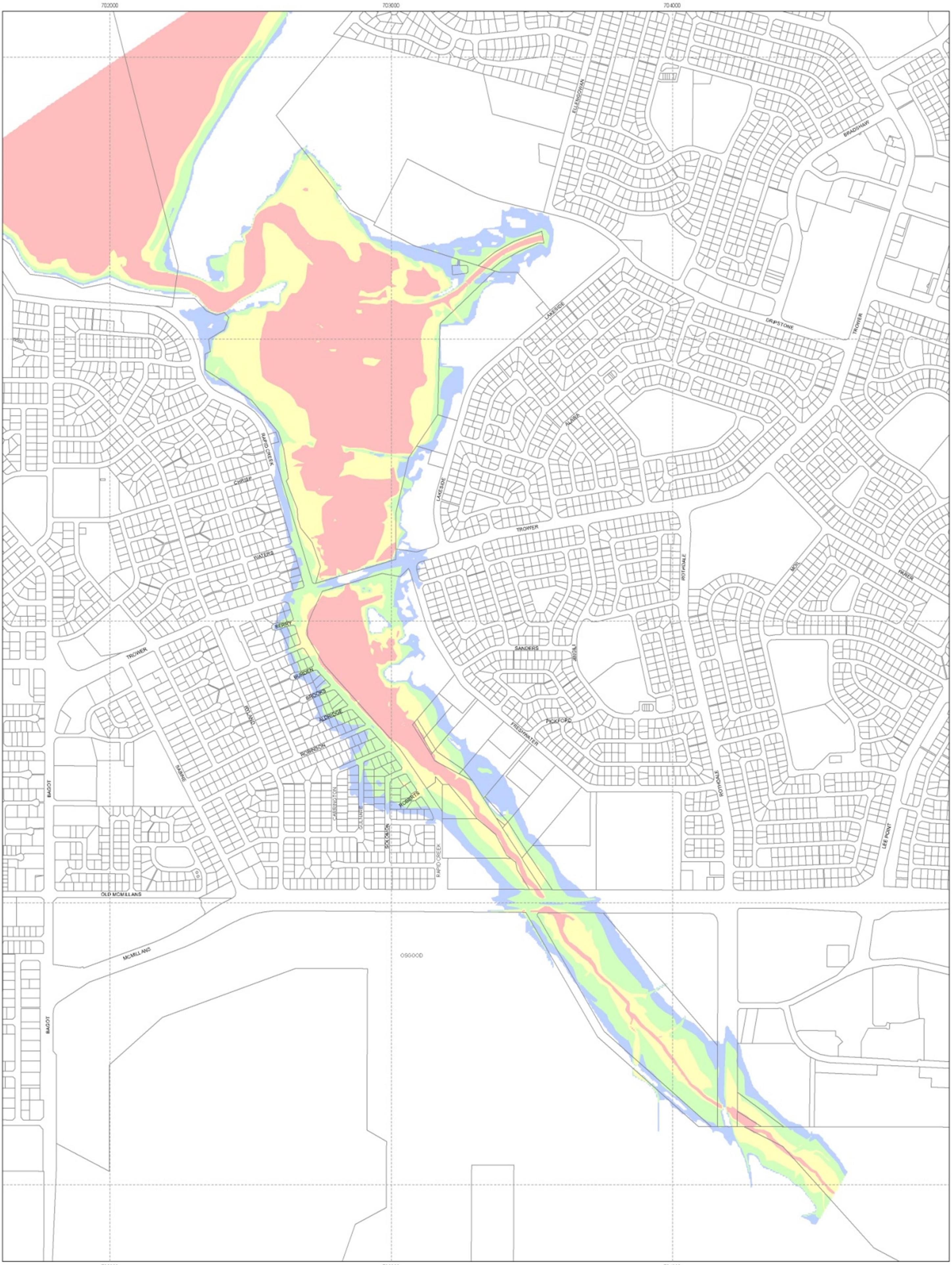

Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

DARWIN

Computed Q500 with Highest Astronomical Tide Flood Depth and Extent

DB05724
Rapid Creek Flood Study

SKM

Figure 25.2 A

Wednesday, 30 May 2012
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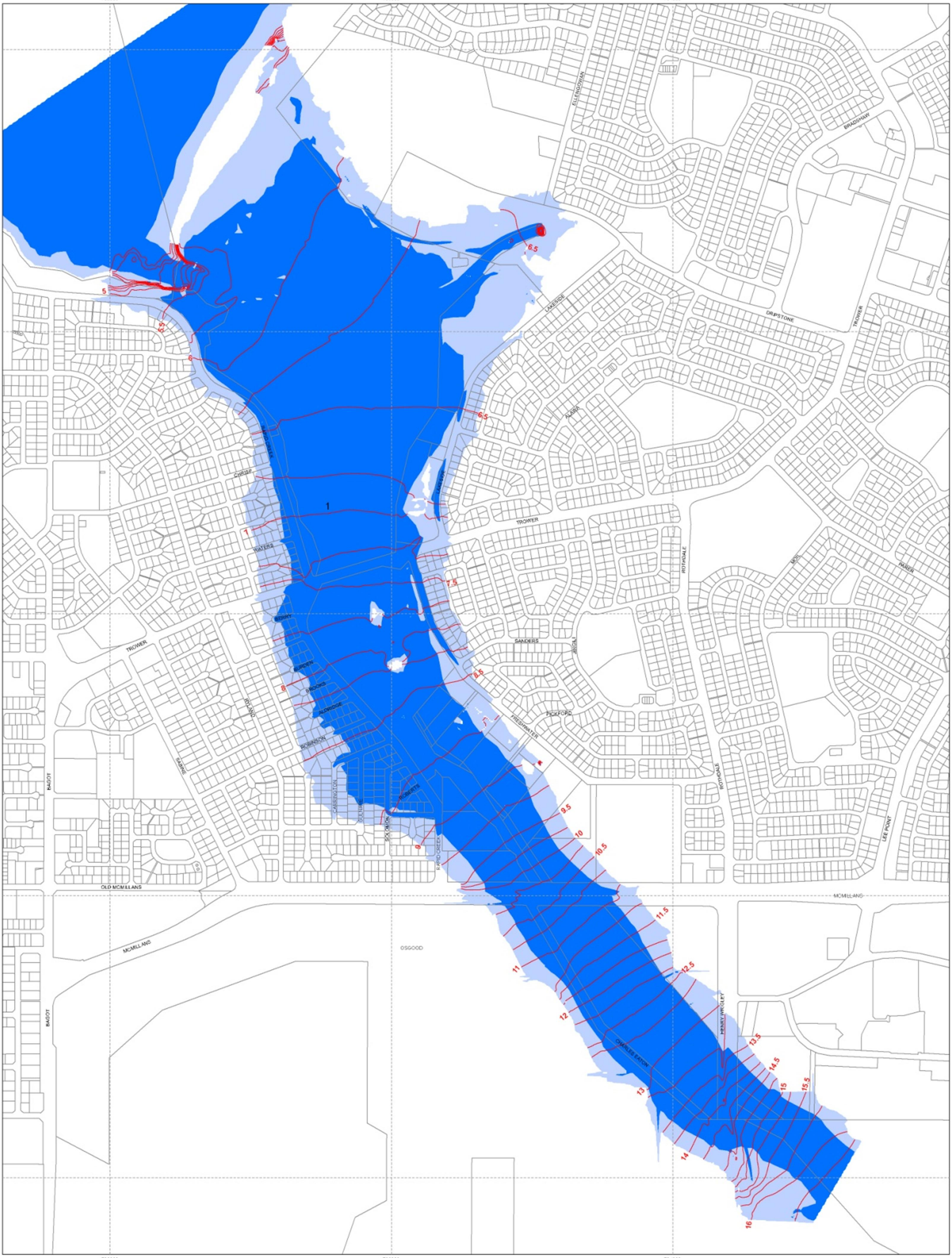

Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52

A4 1:12,500

0 500
Metres

DARWIN

Computed PMF with Highest Astronomical Tide Flood Extent and Flood Surface Contours

DB05724
Rapid Creek Flood Study

SKM

Figure 26.1

A

Wednesday, 30 May 2012
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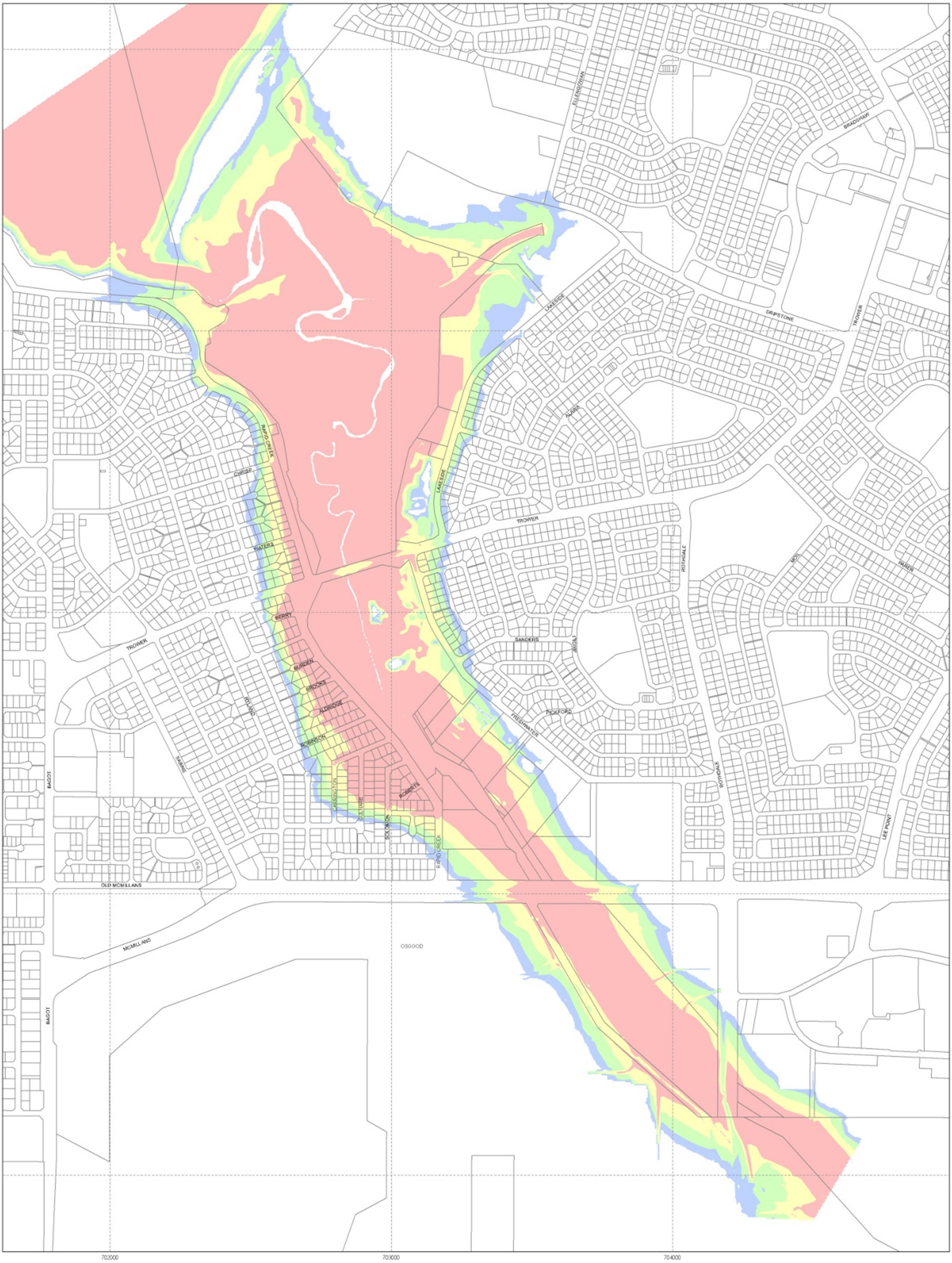

Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

Computed PMF with Highest Astronomical Tide Flood Depth and Extent

DB05724
Rapid Creek Flood Study

SKM

Figure 26.2 A

Wednesday, 30 May 2012
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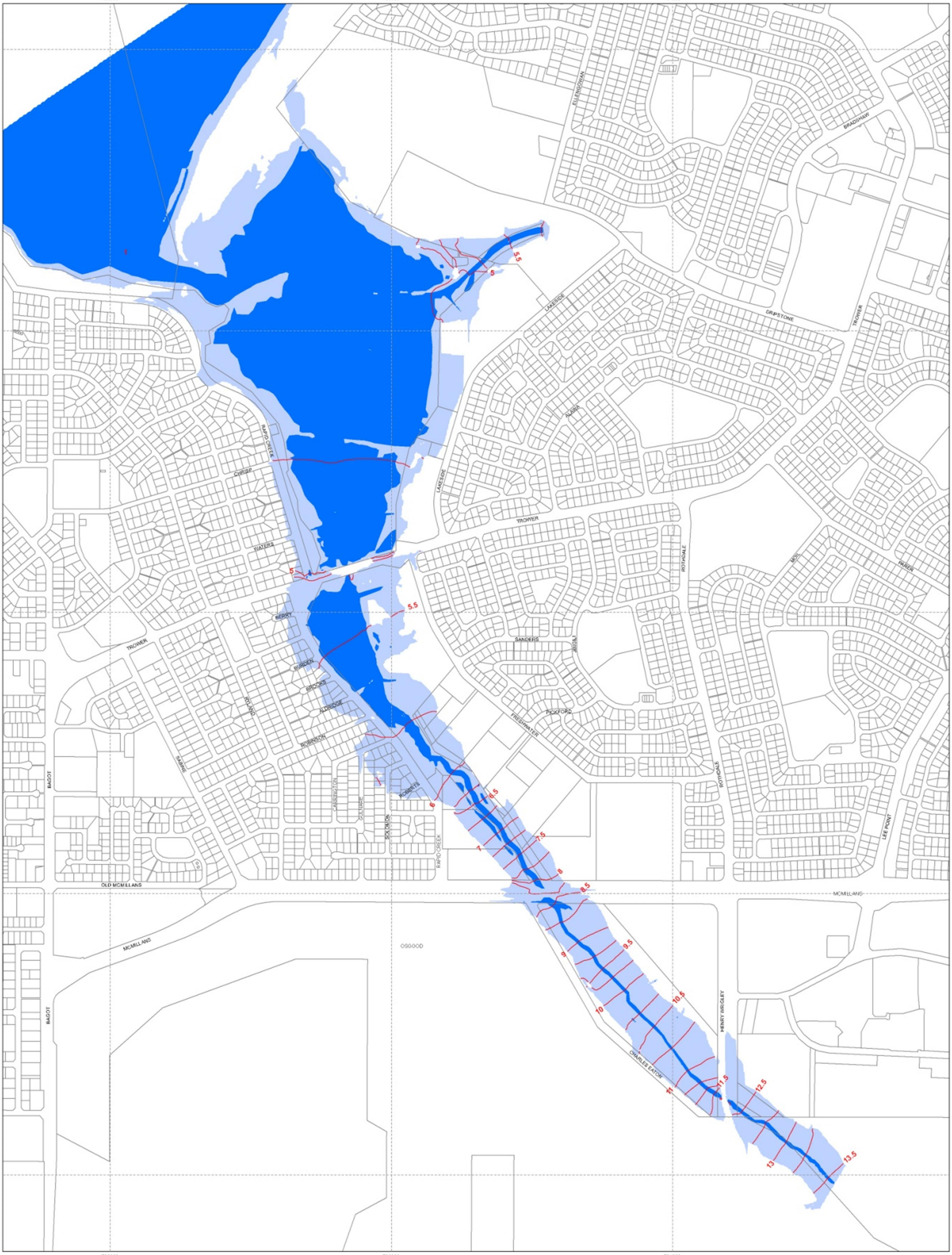

Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52

A4 1:12,500

0 500
Metres

Computed Q20 with 100yr Storm Surge Flood Extent and Flood Surface Contours

Figure 27.1

A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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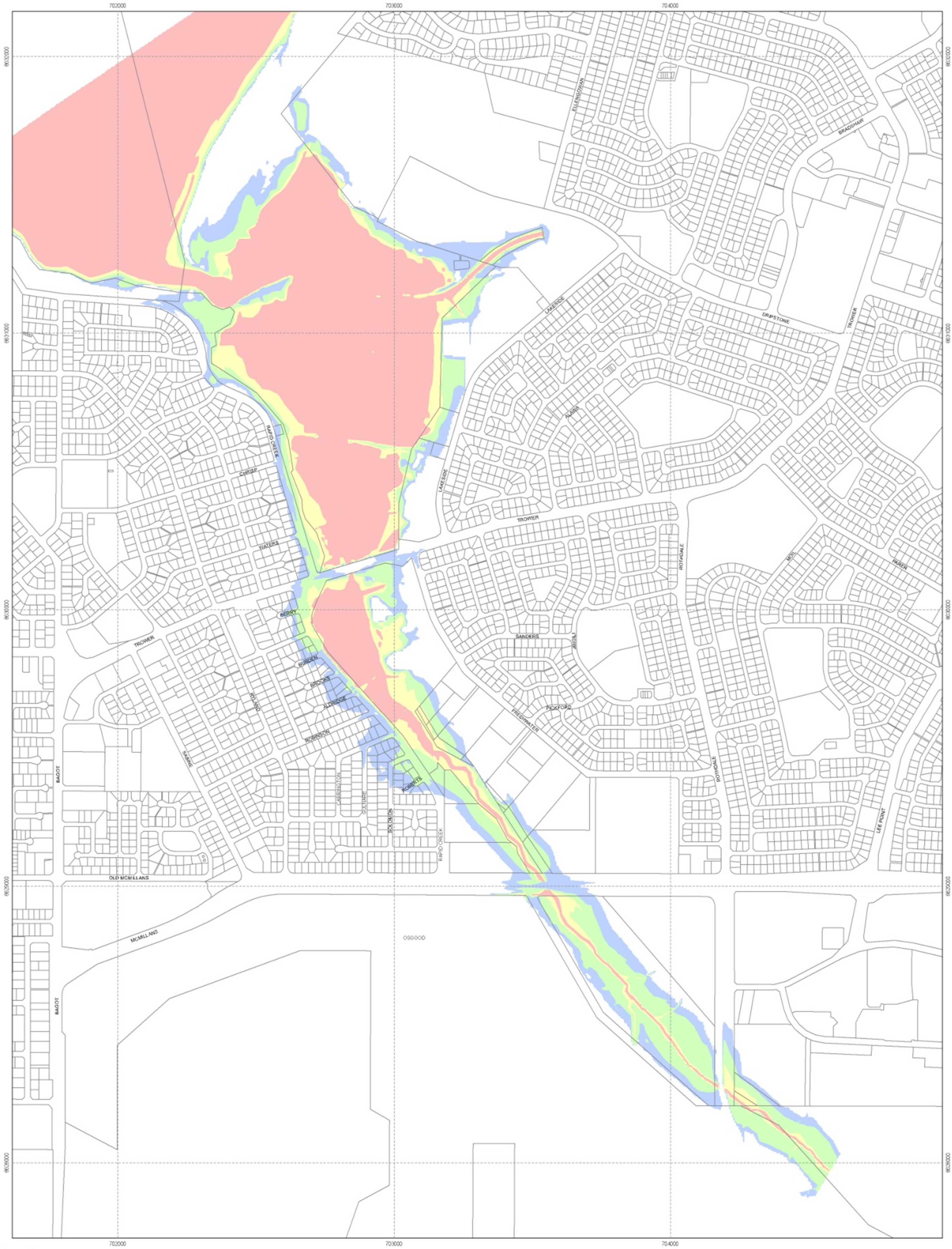
SKM



Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

Computed Q20 with 100yr Storm Surge Flood Depth and Extent

DB05724
Rapid Creek Flood Study

SKM

Figure 27.2 A

Wednesday, 30 May 2012
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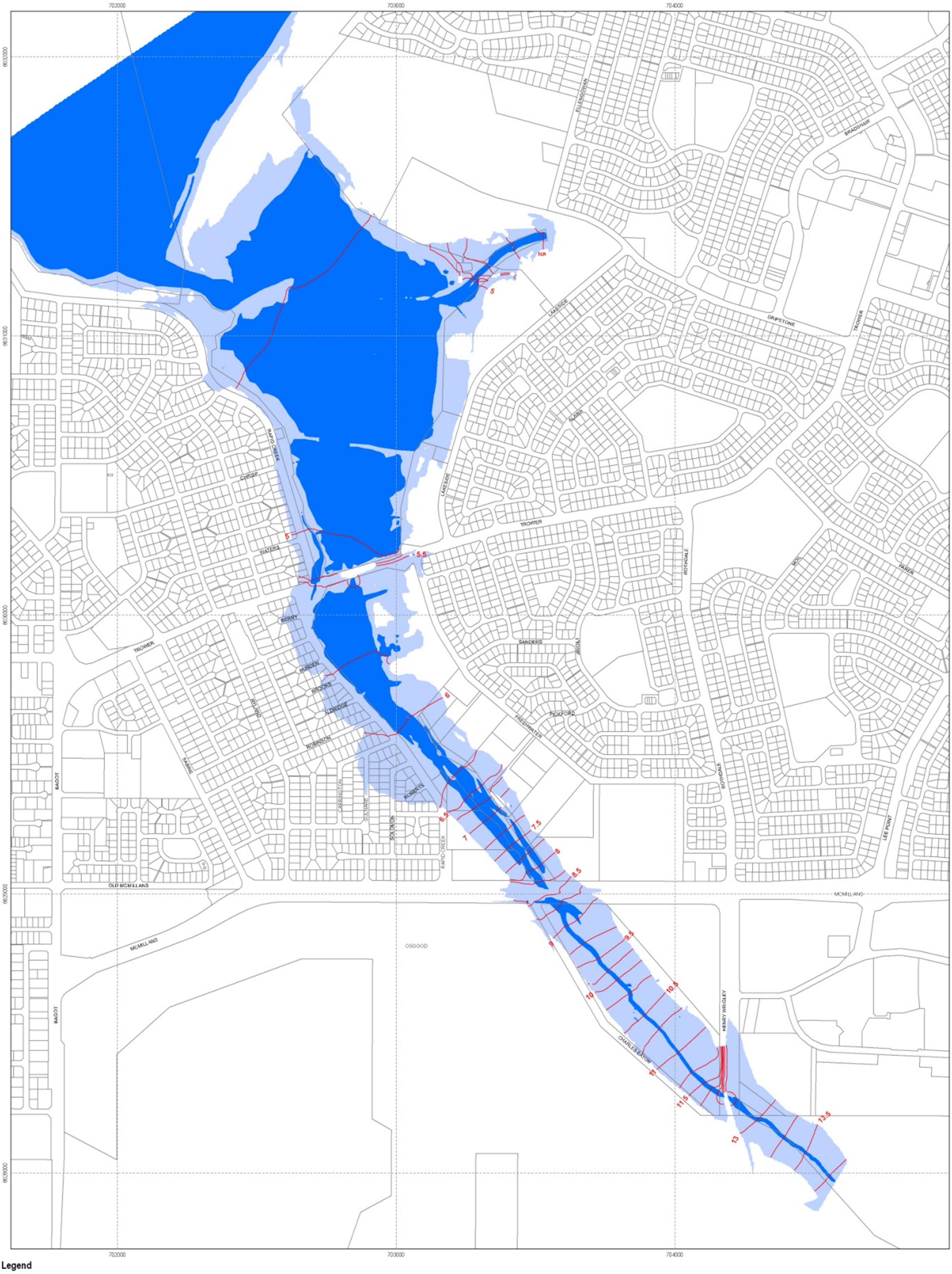

Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

SKM

Computed Q100 with 100yr Storm Surge Flood Extent and Flood Surface Contours

DB05724
Rapid Creek Flood Study

Figure 28.1 A

Wednesday, 30 May 2012
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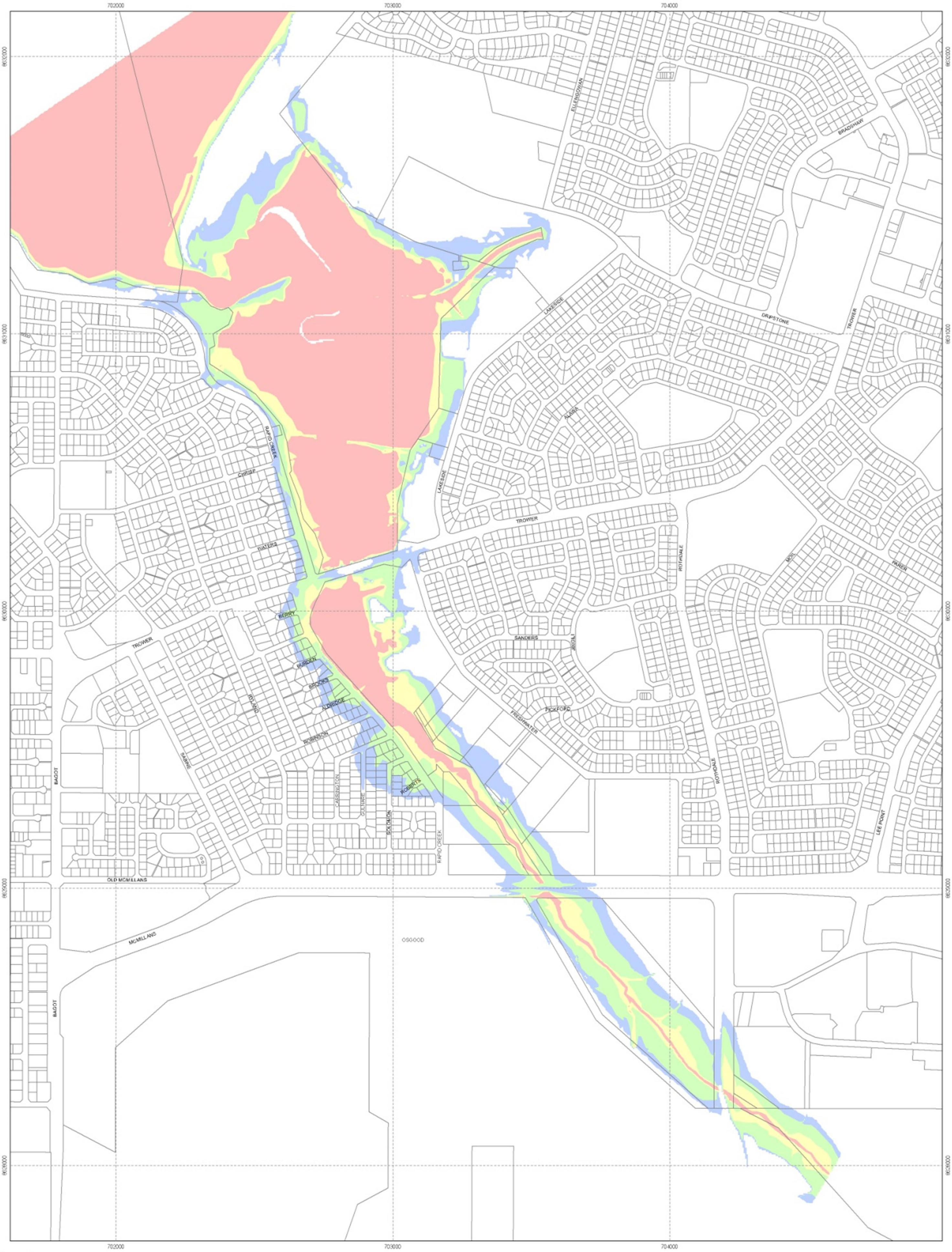

Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

Computed Q100 with 100yr Storm Surge Flood Depth and Extent

Figure 28.2 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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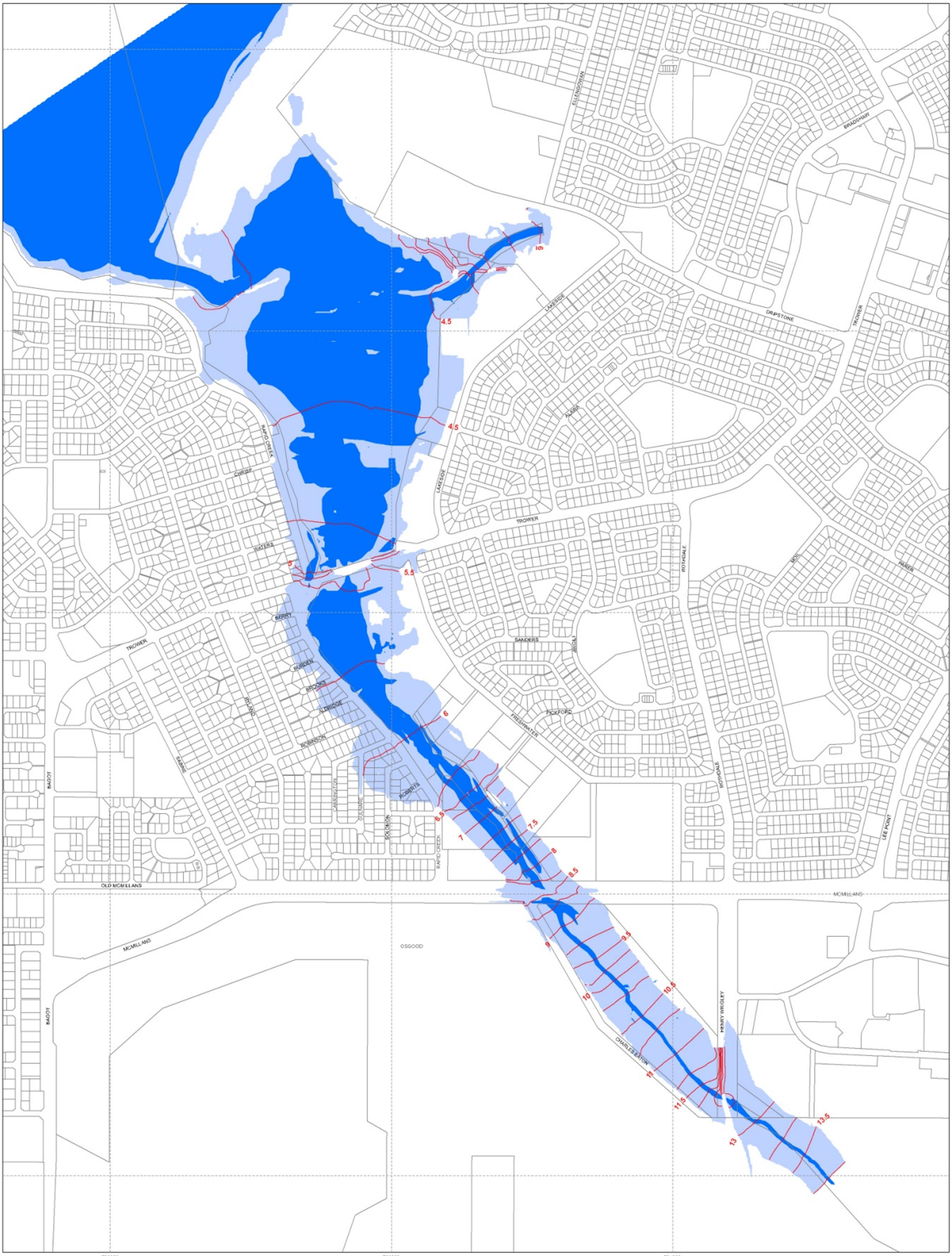
SKM



Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

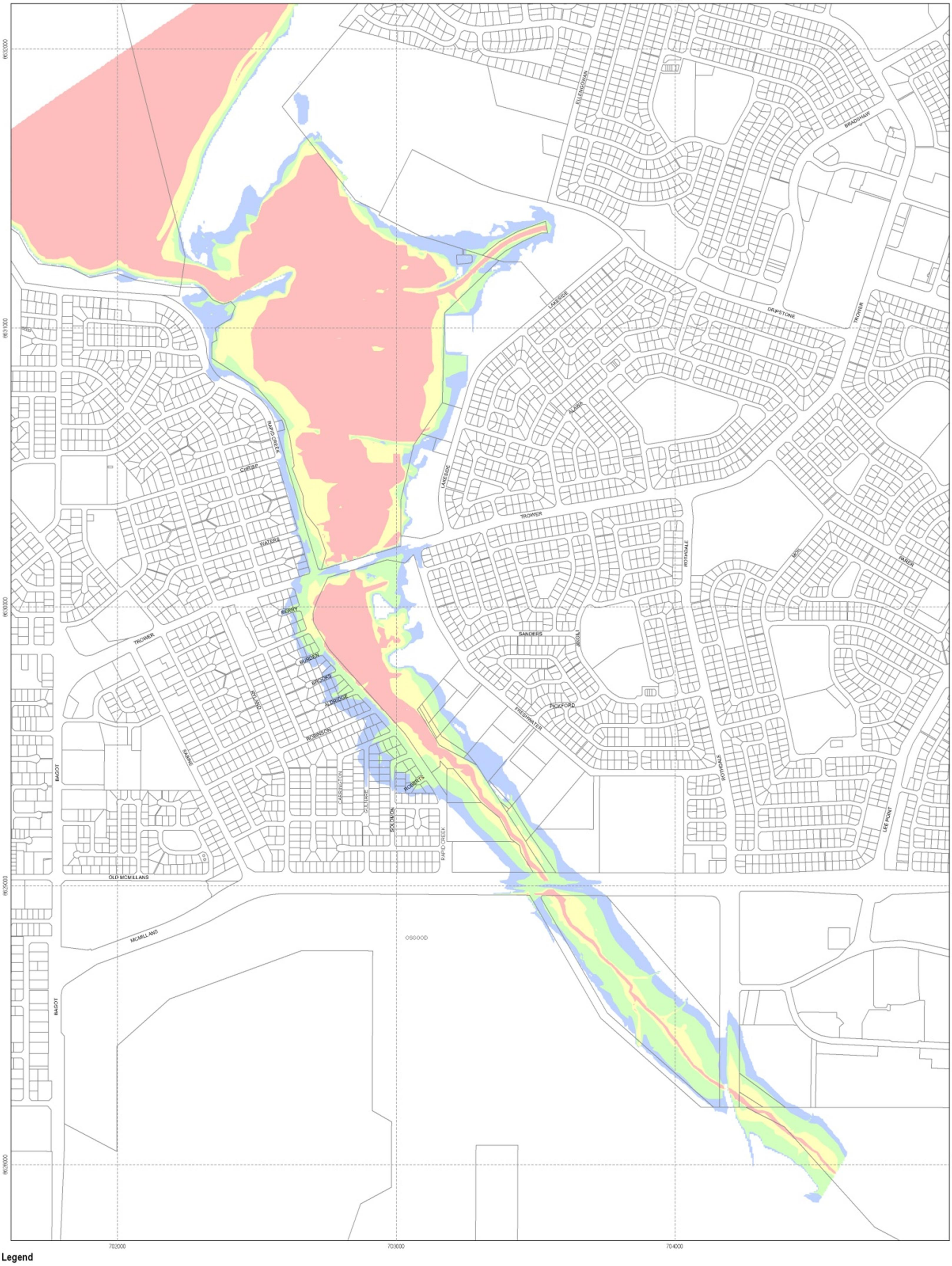
Computed Q100 with Highest Astronomical Tide +0.8m Flood Extent and Flood Surface Contours

DB05724
Rapid Creek Flood Study

SKM

Figure 29.1 A

Wednesday, 30 May 2012
I:\DBIF\Projects\DB05724\Deliverables\Spatial\ArcMap\Figure_29-1.mxd



Legend

Flood Depth 0.0 - 05 metres
Flood Depth 0.5 - 1.2 metre
Flood Depth 1.2 - 2.0 metres
Flood Depth > 2.0 metres
Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

DARWIN

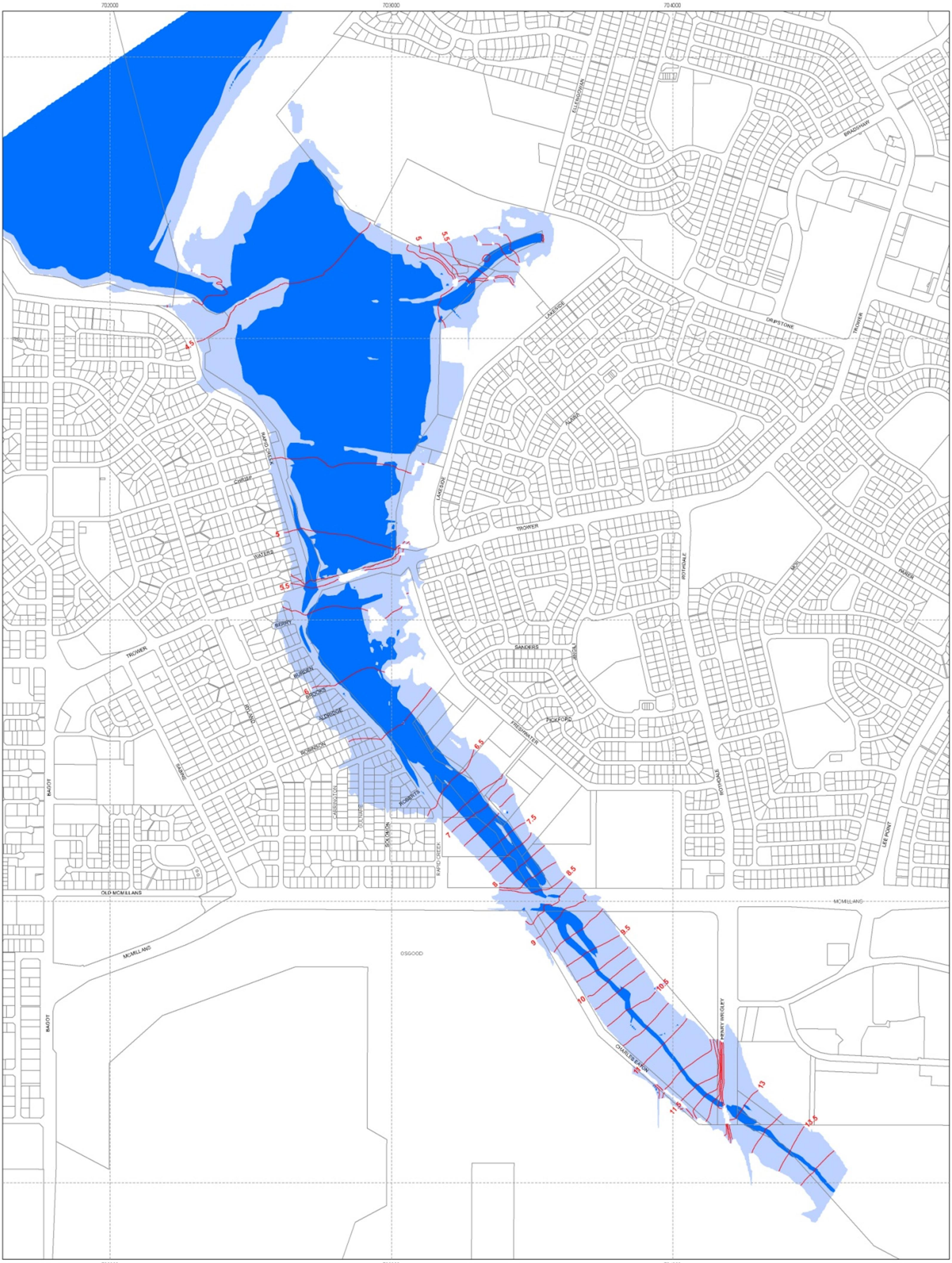
Computed Q100 with Highest Astronomical Tide + 0.8m Flood Depth and Extent

SKM

Figure 29.2 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
I:\DBIF\Projects\DB05724\Deliverables\Spatial\ArcMap\Figure_29-2.mxd



Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52

A4 1:12,500

0 500
Metres

Computed Q500 with Highest Astronomical Tide + 0.8m Flood Extent and Flood Surface Contours

DB05724
Rapid Creek Flood Study

SKM

Figure 30.1 A

Wednesday, 30 May 2012
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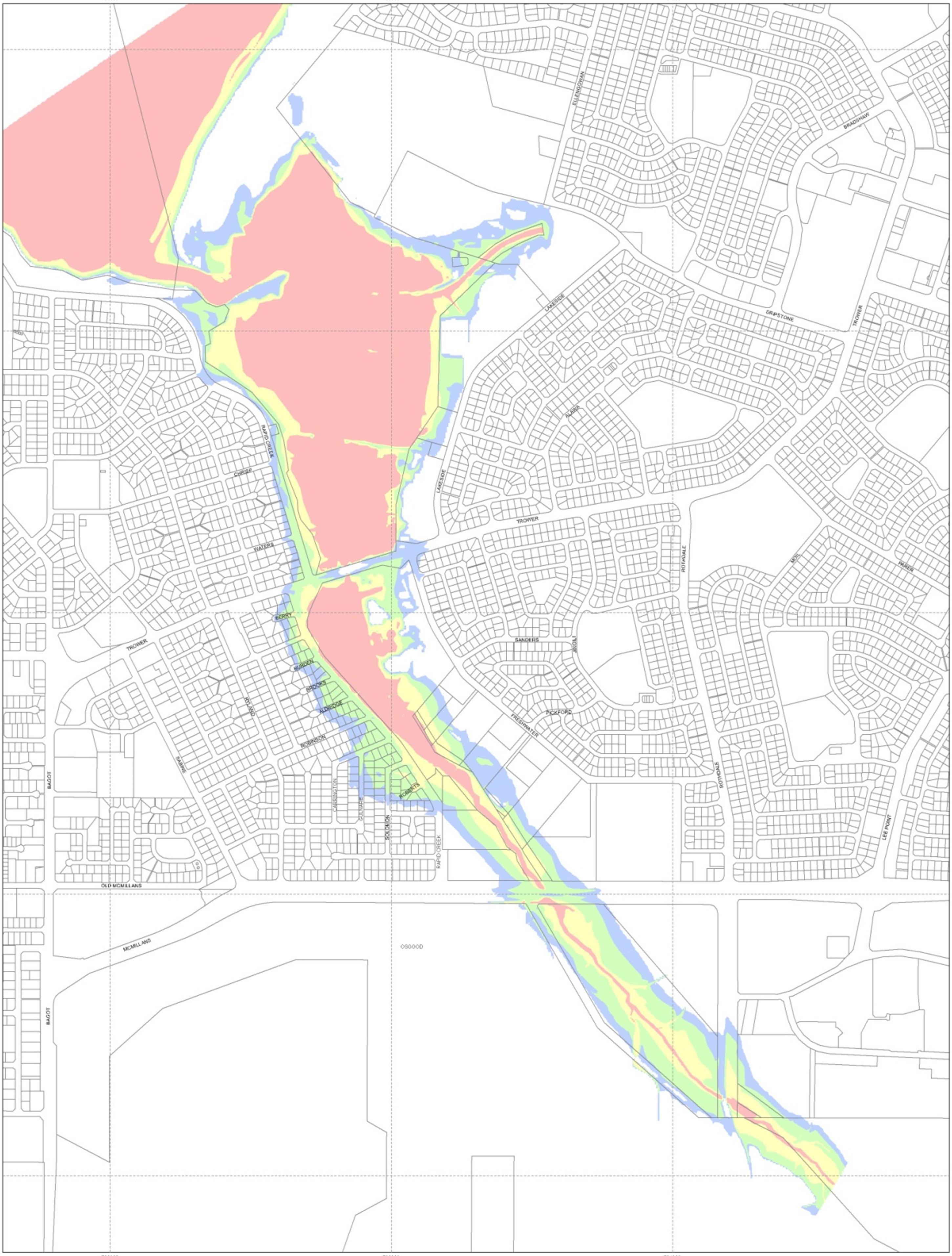

Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500
0 500
Metres

Computed Q500 with Highest Astronomical Tide + 0.8m Flood Depth and Extent

SKM

Figure 30.2 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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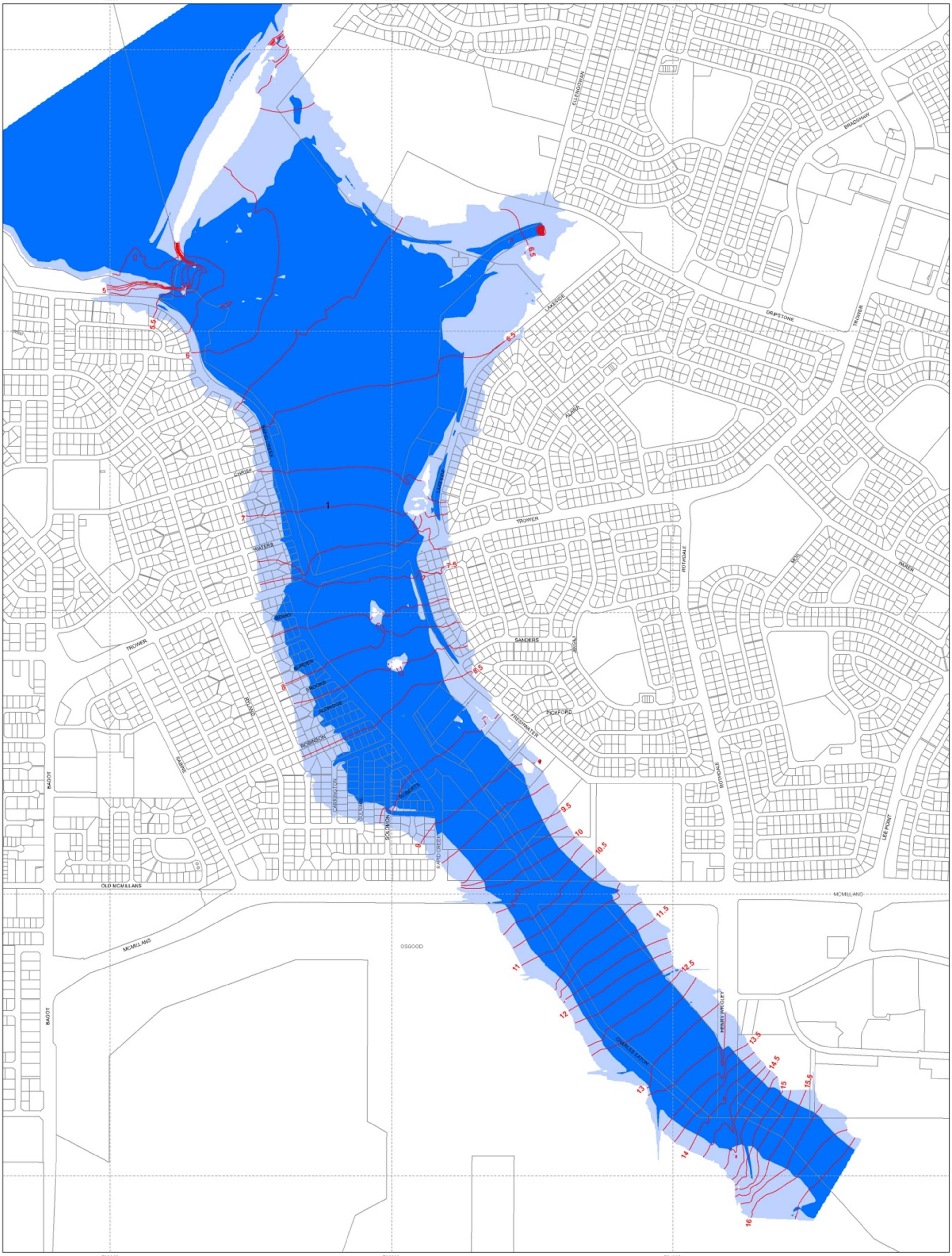

Legend

Peak flood level contour – m AHD (Contour Interval – 0.25m)

Property Boundary

Floodway
Depth > 2m
Velocity x Depth > 1m

Flood Extent

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52
A4 1:12,500

0 500
Metres

Computed PMF with Highest Astronomical Tide + 0.8m Flood Extent and Flood Surface Contours

Figure 31.1 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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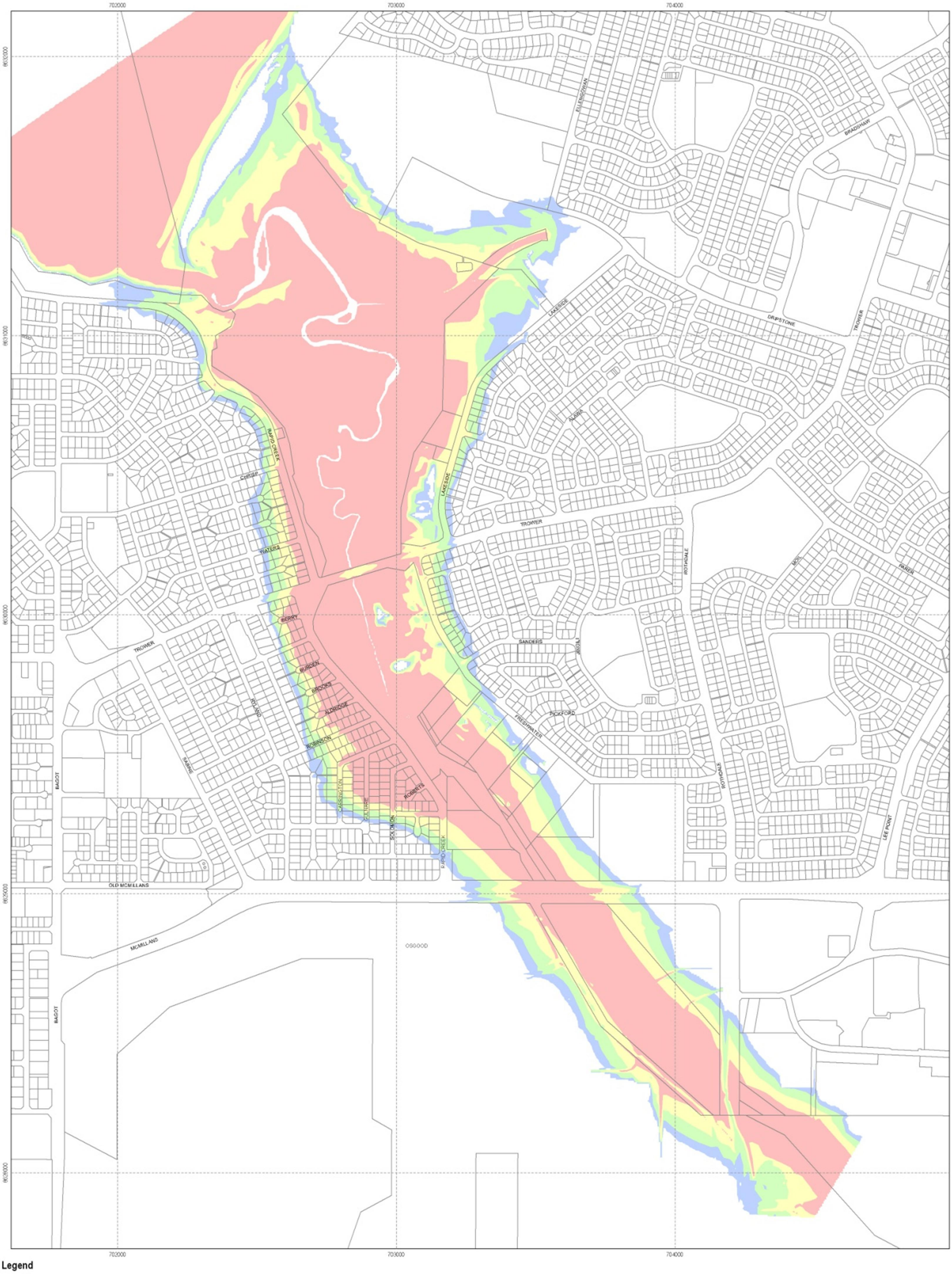
DARWIN

SKM



Legend

- Flood Depth 0.0 - 05 metres
- Flood Depth 0.5 - 1.2 metre
- Flood Depth 1.2 - 2.0 metres
- Flood Depth > 2.0 metres
- Property Boundary

Data Sources
SKM
Dept of Lands and Planning

Datum: GDA 94
Projection: MGA52

A4 1:12,500

0 500
Metres

SKM

Computed PMF with Highest Astronomical Tide + 0.8m Flood Depth and Extent

Figure 31.2 A

DB05724
Rapid Creek Flood Study

Wednesday, 30 May 2012
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