SHIRE OF BROOKTON

# Brookton Flood Study

## Hydrology Report

301012-01906

1 July 2014

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SHIRE OF BROOKTON
BROOKTON FLOOD STUDY
HYDROLOGY REPORT

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**PROJECT 301012-01906 - BROOKTON FLOOD STUDY**

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| | | | | | | | |

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# EXECUTIVE SUMMARY

The town of Brookton is situated on the western bank of the Avon River South Branch, a tributary of the Avon River, and experiences periodic flooding following large rainfall events. The Shire of Brookton engaged WorleyParsons to undertake a flood study of the Avon River South Branch to determine the nature of flooding (level, depth, velocity, extent) for a range of design events to inform the development of a Floodplain Development Strategy for the town.

As no streamflow gauging stations are located within the Avon River South catchment a variety of regional methods were employed to estimate the peak design flow rates for the 1 in 2, 5, 10, 20, 50, 100 and 500 AEP (annual exceedance probability) flood events at Brookton. The regional flood frequency estimates (RFFE) for ungauged catchments described in Australian Rainfall and Runoff (1999) and the method by David Flavell (2012) were compared. David Flavell's (2012) method for estimating peak flow rates for ungauged Wheatbelt catchments was determined to be the most appropriate method as it utilises significantly more recorded information available since the production of the ARR methods in the early 1980's . The 1 in 100 AEP event peak flow rate is estimated to be 186 $m^3$/s.

A flood frequency analysis conducted on the stream gauge data for the nearby Dale River South catchment was undertaken in an attempt to verify the estimate produced by the Flavell procedure (2012). However, this proved unsuccessful as the limited available gauge record during a relatively dry period produced results lower than considered appropriate.

A RORB rainfall-runoff model was developed for the Avon River South Branch catchment to estimate the flow rate of the river over time following the full range of design rainfall events. The RORB model was calibrated to the peak flow rates estimated by the RFFP method for each design event. The calibration parameters closely match existing regional data or local data from neighbouring catchments, giving increased confidence in both the Flavell method peak flow estimates and the hydrograph outputs of the RORB model.

Two-dimensional (2D) hydraulic modelling of the Avon River South Branch system was undertaken using the TUFLOW software package in order to estimate floodwater levels, depths and velocities and the extent of flooding within the town of Brookton. Inflow hydrographs from the RORB model were applied to the 2D model domain at three locations along the model boundary. Direct rainfall was applied to the Brookton townsite model domain, including the small catchment immediately to its west.

The modelling results show that parts of the townsite are affected by flooding during major flows. The main area of concern for flooding in Brookton is the area east of Reynolds Street where floodwaters during events equal to or greater than 1 in 20 AEP are expected to inundate some properties.

A recommended floodplain development strategy for Brookton has been prepared to manage flood risk and limit the effects of flooding on people and property. The strategy states:

- Proposed development located outside  the 1 in 100 AEP floodplain is considered acceptable with regard to major flooding;
- Proposed development that is located within the 1 in 100 AEP floodplain should be assessed based on its merit.  Some of the factors that should be considered include depth of flooding, velocity of flow, obstruction on major flows, potential flood damages and difficulties with evacuation; and
- Should proposed development be considered  acceptable a minimum habitable floor level of 0.50 metre above the appropriate 1 in 100 AEP flood level is recommended to ensure adequate flood protection.

# 1. INTRODUCTION

The town of Brookton is located in the Wheatbelt region of Western Australia, approximately 120 km south-east of Perth (Figure 1-1). The town is situated on the western bank of the Avon River South Branch, a tributary of the Avon River, and has experienced flooding following large rainfall events in the past. As part of its broader approach towards Strategic Planning, Development and Land Use Management, the Shire of Brookton engaged WorleyParsons to undertake a flood study of the Avon River South Branch.

The objective of the study was to assess the likelihood and characteristics (level, depth, velocity, extent) of flooding within the town and provide guidance on an appropriate floodplain development strategy to mitigate the impact of flooding on the community.

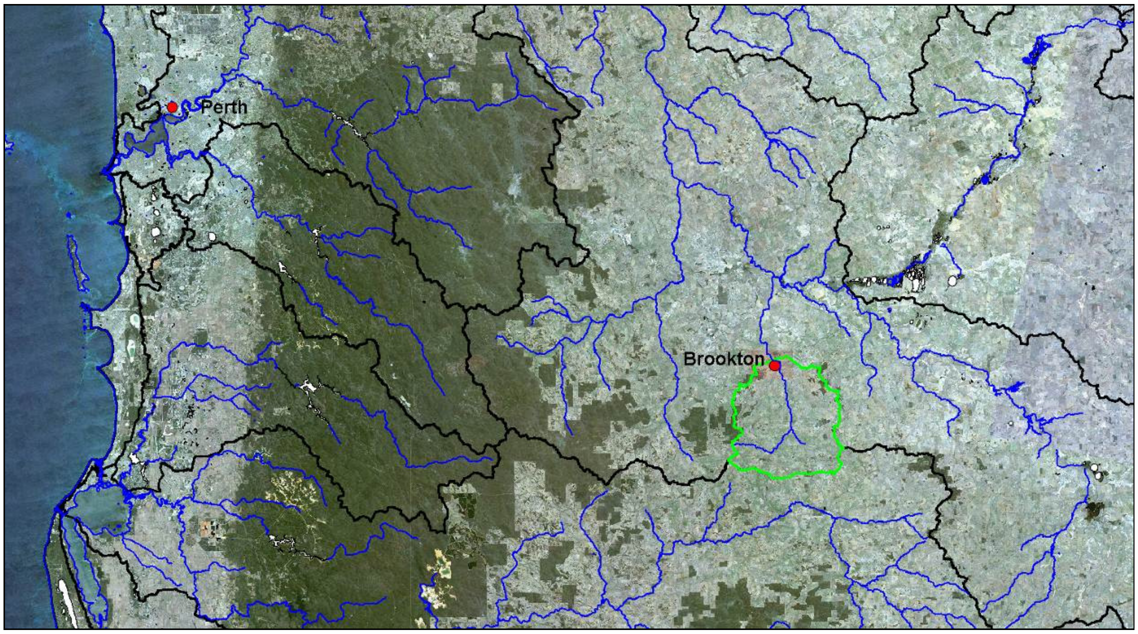


**Figure 1-1 Location map and regional hydrology**

## 1.1 Scope

WorleyParsons' scope of services included:

- Catchment hydrology to determine the 1 in 2,5,10, 20, 50, 100 and 500 annual exceedance probability (AEP) design flow rates for the Avon River South Branch;
- Detailed survey of key features relevant to the flood study, such as:
  - Bridges;

  - Drains;
  - Key infrastructure / building floor levels; and
  - Historic flood levels identified by local residents.
- Floodplain mapping of the 1 in 2,5,10, 20, 50, 100 and 500 AEP events based on the results of two-dimensional (2D) hydraulic modelling, calibrated to historic flood events where possible; and
- Preparation of a floodplain development strategy, including the identification and assessment of potential options to manage the effects of flooding.

## 1.2 Data and Resources

The datasets listed in Table 1-1 were used for the study.

**Table 1-1 – Project datasets**

| Data Type | Dataset | Provided by |
|---|---|---|
| Orthophoto | Brookton high resolution aerial photo | Shire of Brookton |
| Topographic | Digital Elevation Model (from photogrammetry) | Shire of Brookton |
| Topographic | SRTM grid (90 m) | U.S. Geological Survey |
| Infrastructure | Roads and cadastre | Landgate/Dept. of Water |
| Survey | Bridge, Drain and other survey data | AAM Vektra |
| Survey | 1955 Flood Levels | AAM Vektra |
| Hydrological | Dale River South daily stream gauging data (1967-2011) | Dept. of Water |

# 2. HYDROLOGY

The town of Brookton is located adjacent to the Avon River South Branch, with an upstream catchment area of 345 km$^2$. The surface geology consists of alluvium and colluvium in valleys and granite, adamellite, migmatite and laterite in upland areas (GSWA, 1979 and 1980). The catchment has been nearly completed cleared of native vegetation for agriculture.

## 2.1 Flood Frequency Analysis

Flood frequency analysis is a technique that makes use of information on past floods to estimate the magnitude of future floods of a selected probability of exceedance. "Flood frequency analysis is usually carried out using data at stream gauges and makes use of historical streamflow and flood data" (Ladson, 2008).

There is no streamflow gauging station on the Avon River South Branch, so data from the nearby Dale River South gauge (615222) was used to estimate design flood flow rates for that catchment using flood frequency analysis. The Dale River South catchment is similar in area and other catchment characteristics (such as slope, mainstream length, percentage cleared) to the Avon River South Branch catchment (Figure 2-1 and Table 2-1). The Dale River South gauging station recorded streamflow information for 33 years, between 1966 and 1999.

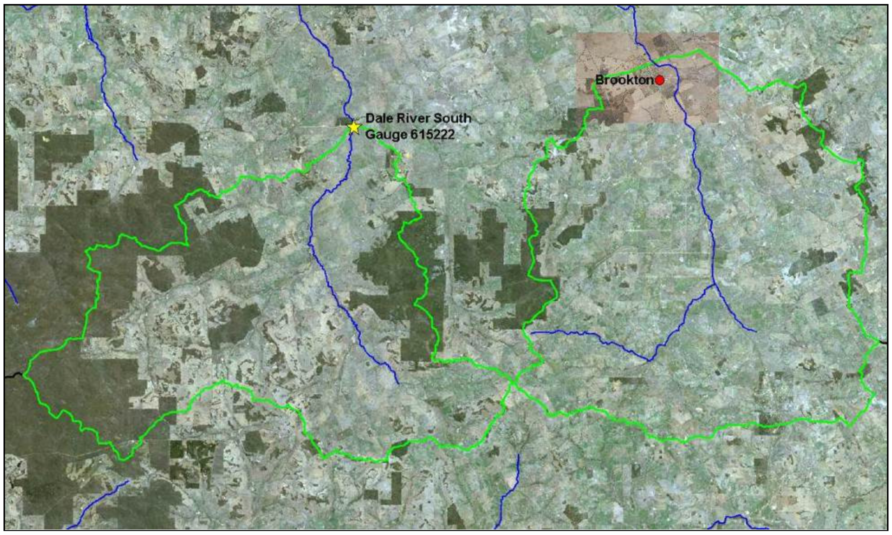


**Figure 2-1 Avon River South Branch and Dale River South catchments**

**Table 2-1 Avon River South Branch and Dale River South catchment details**

| Location | Catchment Area ($km^2$) | Centroid Latitude (°S) | Centroid Longitude (°E) | Stream Length (km) | Equal Area Slope (m/km) | Annual Rainfall (mm) |
|---|---|---|---|---|---|---|
| Avon River South Branch* | 330 | 32.46 | 117.04 | 25.7 | 2.3 | 450 |
| Dale River South | 286 | 32.48 | 116.78 | 32.0 | 3.3 | 466 |

* The catchment outlet is defined by the location of the hydrograph input to the hydraulic model, approximately 1km south of the Brookton-Kweda Road bridge.

Maximum recorded daily flows at the Dale River South streamflow gauging station were used to derive both annual and partial series for flood frequency analysis. The analysis fits a mathematical distribution to the historical flows to determine the likelihood of exceeding various magnitude flood events at the site. The likelihood is defined as the annual exceedance probability (AEP), which is the chance that a flood of a given size or larger will occur in any given year; for example a 1 in 100 AEP event has a 1% chance of occurring this year, while a 1 in 500 AEP event has a 0.2% chance of occurring. A detailed explanation of the flood frequency analysis undertaken for this study is attached in Appendix 1.

The results of the partial series flood frequency analysis for Dale River South are provided in Figure 2-2 and Table 2-2. The figure shows the partial series data as recorded at Dale River South, the fitted LPIII distribution and the 5% and 95% confidence intervals, determined from the methods in ARR Book IV. One can have confidence in the results for the 1 in 2 and 5 AEP events, but the results are unexpectedly low for all other events, particularly the less frequent events such as the 1 in 100 and 1 in 500 AEP events. This is probably due to the fact that during the record period at the Dale River South gauge there were very few medium to large size floods observed. Testing the FFA results by artificially adding in one or two large flow events (such as the expected flow from the February 1955 event) results in the FFA predicting much higher flow rates for the 1 in 100 AEP event, demonstrating that the FFA is highly sensitive to the presence of (or lack of) large flow events. If the gauged record included wetter periods prior to 1966, such as the 1950s, the results would likely have been larger (data from the nearby Murray River gauge 614006 indicates a period of significantly larger flow events from 1955 – 1965 compared to post-1965).

**Table 2-2 Design flood peak flows estimated using flood frequency analysis**

| Location | Q2 ($m^3/s$) | Q5 ($m^3/s$) | Q10 ($m^3/s$) | Q20 ($m^3/s$) | Q50 ($m^3/s$) | Q100 ($m^3/s$) | Q500 ($m^3/s$) |
|---|---|---|---|---|---|---|---|
| Dale River South | 16 | 21 | 24 | 28 | 33 | 37 | 47 |

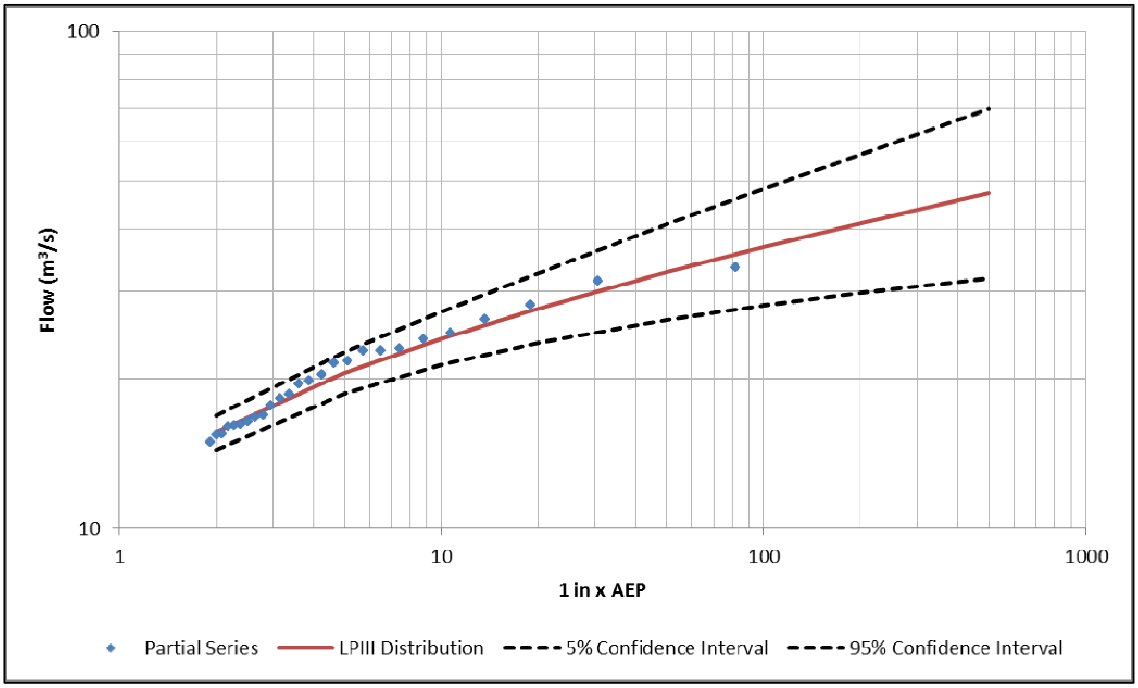


**Figure 2-2 Partial series flood frequency plot for Dale River South**

Design flow rates for the Dale River South catchment were areally scaled using the equation below from Australian Rainfall and Runoff (ARR) (IEAust, 1998) to provide estimates for the Avon River South Branch catchment:

$$\frac{Q_T}{Q_G} = \left(\frac{A_T}{A_G}\right)^{0.7}$$

*where:* *Q is peak flow rate*
*A is catchment area*
*subscript T represents the target catchment*
*subscript G represents the gauged catchment*

The estimated flow rates for the Avon River South Branch are provided in Table 2-3. The results are believed to be too low for the 1 in 10 to 500 AEP events because of the short period of available data, as discussed above.

**Table 2-3 Design flood peak flows estimated by scaling Dale River South results**

| Location | Q2 ($m^3/s$) | Q5 ($m^3/s$) | Q10 ($m^3/s$) | Q20 ($m^3/s$) | Q50 ($m^3/s$) | Q100 ($m^3/s$) | Q500 ($m^3/s$) |
|---|---|---|---|---|---|---|---|
| Avon River South Branch | 18 | 23 | 27 | 31 | 36 | 41 | 52 |

## 2.2 Regional Peak Flow Estimates

There are several methods available to estimate design flood peak flows in ungauged catchments in Australia.  In most cases these methods have been developed statistically from observed streamflow data in the particular region, and are often used in the design of drainage infrastructure in the absence of locally recorded data.

Two methods (Index Flood Method and Regional Rational Method) are recommended in ARR for estimating design floods in the Wheatbelt region. Both of these methods are based on the same streamflow data and should therefore give similar results. These methods were developed based on the streamflow data from 12 catchments, with an area of 0.03 – 20,500 $km^2$, stream length of 0.5 – 440 km, equal area slope of 0.65 – 40.3 m/km and average annual rainfall of 300 – 600 mm. The characteristics of the Avon River South Branch catchment fall within these ranges so the methods therefore should be suitable for application to the ungauged catchment.

It should be noted that these methods were developed using the limited streamflow gauging information available in the early 1980s which in many cases was less than 10 years of records.

A new Regional Flood Frequency Procedure (RFFP) has been developed for the Wheatbelt by David Flavell (2012) based on the longer (more than 20 years of additional data) streamflow records that are now available for the same catchments used to develop the existing methods.  It is expected that the new procedure should give more accurate design flood estimates as it is based on about twice the length of record from many of the gauging stations used to derive the methods recommended in ARR (David Flavell, 2012).  The RFFP method produces peak flow estimates based on catchment area, rainfall and catchment shape (a measure of how elongated or square the catchment is).

The design flood peak flows estimated for the Dale River South and Avon River South Branch catchments using each of the regional methods are shown in Table 2-4 and Table 2-5. Where there was insufficient data for some AEP's, log-log extrapolation of the available data was undertaken to estimate the missing values. This involves fitting a linear equation to the logarithm of the parameter of interest (frequency factor, rainfall etc.) plotted against the logarithm of the AEP. $R^2$ values were generally above 0.99, with the lowest value equal to 0.98, indicating that the fitted equations matched the data well.

**Table 2-4 Design flood peak flows using regional methods at Dale River South**

| Method | Q2 ($m^3/s$) | Q5 ($m^3/s$) | Q10 ($m^3/s$) | Q20 ($m^3/s$) | Q50 ($m^3/s$) | Q100 ($m^3/s$) | Q500 ($m^3/s$) |
|---|---|---|---|---|---|---|---|
| Regional Rational Method | 11 | 21 | 36 | 60 | 98 | 168 | 537[1] |
| Index Flood Method | 10 | 21 | 39 | 68 | 128 | 208[1] | 709[1] |
| RFFP | 16 | 26 | 41 | 55 | 86 | 126 | 295[1] |
| Flood Frequency Analysis | 16 | 21 | 24 | 28 | 33 | 37 | 47 |

[1] Estimate uses extrapolated (log-log) frequency factors, rainfall intensity or design flow rates

**Table 2-5 Design flood peak flows using regional methods at Avon River South Branch**

| Method | Q2 ($m^3/s$) | Q5 ($m^3/s$) | Q10 ($m^3/s$) | Q20 ($m^3/s$) | Q50 ($m^3/s$) | Q100 ($m^3/s$) | Q500 ($m^3/s$) |
|---|---|---|---|---|---|---|---|
| Regional Rational Method | 15 | 29 | 51 | 91 | 157 | 274 | 953[1] |
| Index Flood Method | 11 | 24 | 44 | 77 | 145 | 265[1] | 959[1] |
| RFFP | 16 | 27 | 57 | 82 | 128 | 186 | 565[1] |
| Areally scaled FFA | 18 | 23 | 27 | 31 | 36 | 41 | 52 |

[1] Estimate uses extrapolated (log-log) frequency factors, rainfall intensity or design flow rates

## 2.3 Rainfall-Runoff Modelling

### 2.3.1 Rainfall

Intensity-Frequency-Duration (IFD) data for the Avon River South Branch catchment was obtained from the Bureau of Meteorology website using the coordinates of the catchment centroid (32.46 °S, 117.04 °E). The IFD data is presented in terms of total rainfall in Table 2-6. Areal reduction factors were later applied to the raw depths and temporal patterns were fitted to the data as per ARR methods. The recently updated IFD data was not used in this project because many of the methods that use this data have not yet been updated for use with the new IFD data. The new IFD data is generally higher than the previous data, with the 6 and 12 hour events on average 9% higher. The 6 hour 1 in 100 AEP rainfall event is 17% higher than the previous IFD data. The sensitivity analysis conducted on the two-dimensional modelling results includes varying inflows by ±20%.

**Table 2-6 IFD data for Avon River South Branch catchment**

| | Rainfall total for each AEP (mm) | | | | | | |
|---|---|---|---|---|---|---|---|
| Storm Duration | 1 year | 2 years | 5 years | 10 years | 20 years | 50 years | 100 years |
| 5 mins | 3.4 | 4.6 | 6.6 | 8.1 | 10.1 | 13.3 | 16.2 |
| 6 mins | 3.8 | 5.2 | 7.3 | 9.0 | 11.3 | 14.8 | 17.9 |
| 10 mins | 5.1 | 6.8 | 9.5 | 11.6 | 14.3 | 18.7 | 22.5 |
| 20 mins | 7.0 | 9.4 | 12.6 | 15.1 | 18.4 | 23.4 | 27.8 |
| 30 mins | 8.3 | 11.0 | 14.6 | 17.3 | 20.9 | 26.3 | 31.0 |
| 1 hr | 10.8 | 14.2 | 18.5 | 21.5 | 25.8 | 32.0 | 37.3 |
| 2 hrs | 14.0 | 18.3 | 23.6 | 27.2 | 32.2 | 39.8 | 46.0 |
| 3 hrs | 16.3 | 21.3 | 27.2 | 31.2 | 37.2 | 45.6 | 52.8 |
| 6 hrs | 21.2 | 27.6 | 35.1 | 40.3 | 47.6 | 58.2 | 67.2 |
| 12 hrs | 27.2 | 35.5 | 45.0 | 51.6 | 61.0 | 74.4 | 85.7 |
| 24 hrs | 34.1 | 44.4 | 56.4 | 64.6 | 76.3 | 93.1 | 108 |
| 48 hrs | 40.7 | 53.3 | 67.7 | 77.8 | 92.2 | 113 | 131 |
| 72 hrs | 44.0 | 57.7 | 73.4 | 85.0 | 101 | 125 | 145 |

### 2.3.2 RORB Modelling

RORB is a general runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall-excess and routes this through catchment storage to produce runoff hydrographs at any location (RORB User Manual, 2010).

A RORB model was developed for the Avon River South Branch catchment to estimate the flow rate of the river following a range of design rainfall events. Flow hydrographs generated by the RORB model were used as inflows to the 2D TUFLOW hydraulic model for the river. Layout of the RORB network model can be seen in Figure 2-3.

Adequately detailed rainfall and streamflow information for historic flow events was not available to calibrate the RORB model. Consequently, the catchment linearity parameter (m) was set constant at 0.8 as recommended by the model developers and in ARR (1998). The catchment storage parameter (Kc) was calculated using the equation for the wheatbelt region of Western Australia from ARR (1998). The design rainfall estimates described above were applied to an initial loss / runoff coefficient loss model.

The loss parameters were varied to reproduce the design peak flows from the Flavell RFFP method (Table 2-5). The loss parameters required to reproduce the Flavell peak flow estimates (Table 2-7) are within the expected range for south west WA catchments (ARR 1998) and previous studies (SKM, 2009; Flavell, 2012) in the region. The resultant design flow hydrograph are presented in Figure 2-4.

Further information regarding the RORB modelling undertaken can be found in Appendix 2.

**Table 2-7 Avon River South Branch RORB model calibration results**

| AEP | 2 years | 5 years | 10 years | 20 years | 50 years | 100 years | 500 years |
|---|---|---|---|---|---|---|---|
| Target peak flow ($m^3/s$) | 16 | 27 | 57 | 82 | 128 | 186 | 565 |
| Modeled peak flow ($m^3/s$) | 17 | 28 | 56 | 83 | 131 | 188 | 566 |
| Critical design storm duration | 9 hr | 9 hr | 9 hr | 9 hr | 6 hr | 6 hr | 6 hr |
| Hydrograph time to peak (hrs) | 10 | 9.5 | 9 | 9 | 6.5 | 6.5 | 5.5 |
| Linearity parameter ‘m’ | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| Catchment parameter ‘$k_c$’ | 12.1 | 12.1 | 12.1 | 12.1 | 12.1 | 12.1 | 12.1 |
| Initial Loss (mm) | 10 | 10 | 10 | 10 | 5 | 5 | 5 |
| Runoff coefficient | 0.12 | 0.14 | 0.22 | 0.26 | 0.28 | 0.33 | 0.62 |

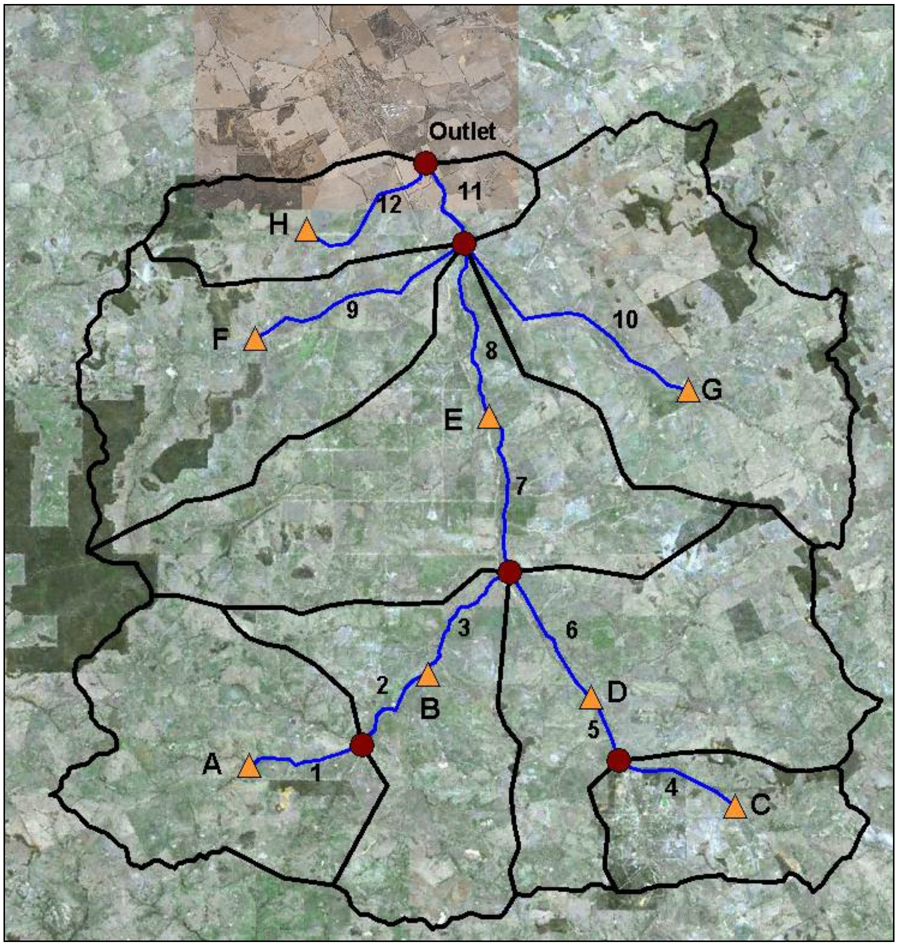


**Figure 2-3 Avon River South Branch catchment plan and RORB model**

## 2.4 Adopted Peak Flow Estimates

The Regional Flood Frequency Procedure is expected to provide better estimates of peak flows than either the Regional Rational Method or the Index Flood Method as it is based on a longer length of recorded data. The ability to reproduce these peak flows using a RORB runoff routing model of the catchment adopting parameters consistent with the existing information for the region increases the confidence in the estimates.

The results of RORB and hydraulic modelling of a large historic event that occurred in February 1955 (refer to section 3.3.1) provides additional confidence in the design flood estimates for the Avon River South Branch.

The selected design flood peak flow rates for the Avon River South Branch, at the point 1 km upstream of the Brookton-Kweda Road bridge, are presented in Table 2-8 and Figure 2-4.

### Table 2-8 Selected design flood peak flow rates for the Avon River South Branch

| | Q2 ($m^3/s$) | Q5 ($m^3/s$) | Q10 ($m^3/s$) | Q20 ($m^3/s$) | Q50 ($m^3/s$) | Q100 ($m^3/s$) | Q500 ($m^3/s$) |
|---|---|---|---|---|---|---|---|
| Avon River South Branch | 16 | 27 | 57 | 82 | 128 | 186 | 565 |

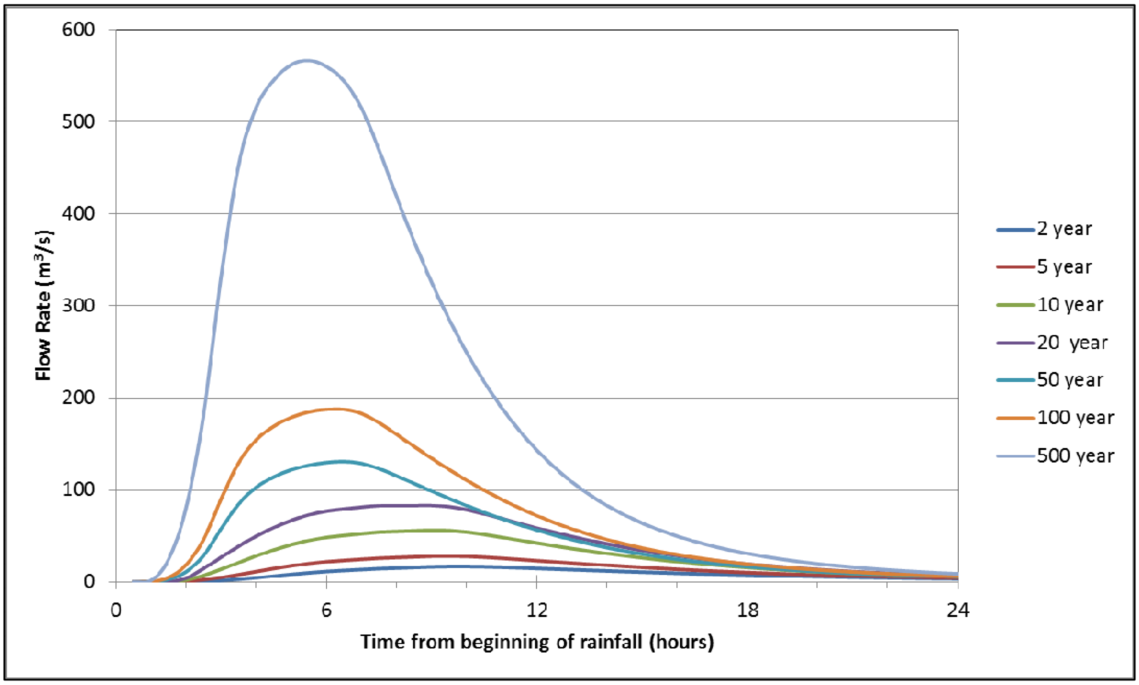


**Figure 2-4 RORB hydrographs at outlet node for design rainfall events**

# 3. HYDRAULIC MODELLING OF AVON RIVER SOUTH BRANCH

Two-dimensional (2D) hydraulic modeling of the Avon River South Branch system was undertaken using the TUFLOW software package in order to estimate design flood water levels, depths and velocities and the extent of flooding for the reach through the town of Brookton.

## 3.1 TUFLOW

TUFLOW is a computer program for simulating depth-averaged, two and one-dimensional free-surface flows such as occurs from floods and tides. TUFLOW is specifically orientated towards establishing flow and inundation patterns in coastal waters, estuaries, rivers, floodplains and urban areas where the flow behavior is essentially 2D in nature and cannot or would be awkward to represent using a 1D model (TUFLOW User Manual, 2010).

## 3.2 Hydraulic Model Setup

### 3.2.1 Spatial Domain

The extent of the Brookton hydraulic model domain is shown in Figure 3-1. The model domain extends from 1 km upstream of the Brookton-Kweda Road bridge to 2 km downstream of the Brookton Highway bridge, covering an area of approximately 16 km$^2$.

A 1 m DEM was provided by the Shire of Brookton which covered the model domain, shown in Figure 3-1 with areas of high elevation in red and low elevation in blue. A 5 metre cell size was selected for the hydraulic model grid to represent the major features and significant detail of the natural river patterns whilst achieving practical simulation times. Surface elevations for the hydraulic model cells were extracted from the underlying DEM. In the area around the Brookton Highway bridge elevations were manually changed according to the ground survey elevations where there were differences between these and the DEM. Outside of the immediate area surrounding this bridge, the DEM was within an acceptable range of the ground survey results and required no additional modification. See Table 3-1 for a comparison between the DEM and ground survey results for a series of points along the river from 100 m upstream of the Brookton Highway bridge to 100 m downstream of the bridge.

**Table 3-1 Comparison between DEM and ground survey**

| Easting (mE) | Northing (mN) | DEM (mAHD) | Survey (mAHD) | Difference (m) |
|---|---|---|---|---|
| 501303 | 6419081 | 230.94 | 230.86 | 0.08 |
| 501296 | 6419093 | 230.12 | 230.34 | 0.22 |
| 501271 | 6419119 | 230.00 | 230.19 | 0.19 |
| 501262 | 6419139 | 229.97 | 229.98 | 0.01 |
| 501229 | 6419157 | 230.00 | 230.04 | 0.04 |
| 501207 | 6419175 | 230.34 | 230.32 | 0.02 |
| 501200 | 6419187 | 230.36 | 230.50 | 0.14 |
| 501177 | 6419200 | 229.97 | 229.92 | 0.05 |
| 501165 | 6419209 | 230.20 | 230.06 | 0.14 |
| 501144 | 6419233 | 230.03 | 229.92 | 0.11 |

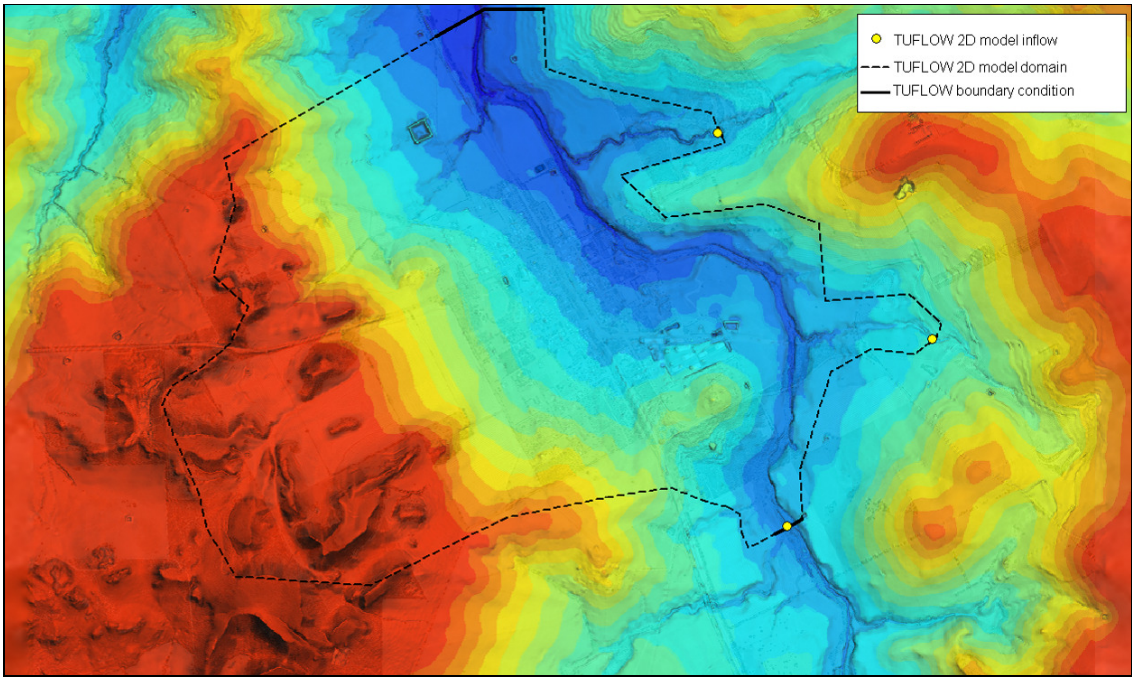


**Figure 3-1 Brookton hydraulic model domain, DEM and inflow locations**

### 3.2.2 Inflows and Scenarios

Rainfall was applied directly to the area to the west of Brookton, covering an area of 13 km$^2$, located within the TUFLOW model domain. The relevant design rainfall information and initial loss and runoff coefficients used in the RORB model were applied.

The flows generated outside of the model domain were applied as inflow hydrographs extracted from the RORB model at three locations along the TUFLOW model boundary, shown in Figure 3-1. Figure 3-2 shows the overall model layout, with the main Avon River South Branch catchment and hydrograph inflow, the two smaller catchments and inflow locations and the area where rainfall is applied directly to the model. The estimated peak flow rates of the two smaller catchments using RFFP are provided in Table 3-2. The northern catchment has an area of 16.8 km$^2$ with an estimated time of concentration of 2.2 hours and the southern catchment has an area of 6.5 km$^2$ with an estimated time of concentration of 1.6 hours.

**Table 3-2 Details of smaller Brookton catchments**

| Catchment | Q2 (m$^3$/s) | Q5 (m$^3$/s) | Q10 (m$^3$/s) | Q20 (m$^3$/s) | Q50 (m$^3$/s) | Q100 (m$^3$/s) | Q500 (m$^3$/s) |
|---|---|---|---|---|---|---|---|
| Northern Catchment | 2.2 | 4.1 | 4.7 | 6.7 | 11.6 | 18.3 | 39.6[1] |
| Southern Catchment | 1.2 | 2.2 | 3.4 | 5.1 | 9.1 | 14.7 | 40.9[1] |

[1] Estimate based on log-log extrapolation of peak flow estimates up to the 1 in 100 AEP

For the two small tributaries that enter the Avon River South Branch within the 2D model domain, the peak flow rates were estimated using the RFFP by Flavell (2012). The hydrographs for these tributaries were prepared by scaling the hydrographs for the Avon River South Branch from the RORB modelling. This relatively simplistic approach is considered acceptable as the tributaries have a minor influence on flooding in the town.

A range of event durations were modeled  to confirm the critical storm duration for the 1 in 2, 5, 10, 20, 50, 100 and 500 AEP flood levels within the townsite.

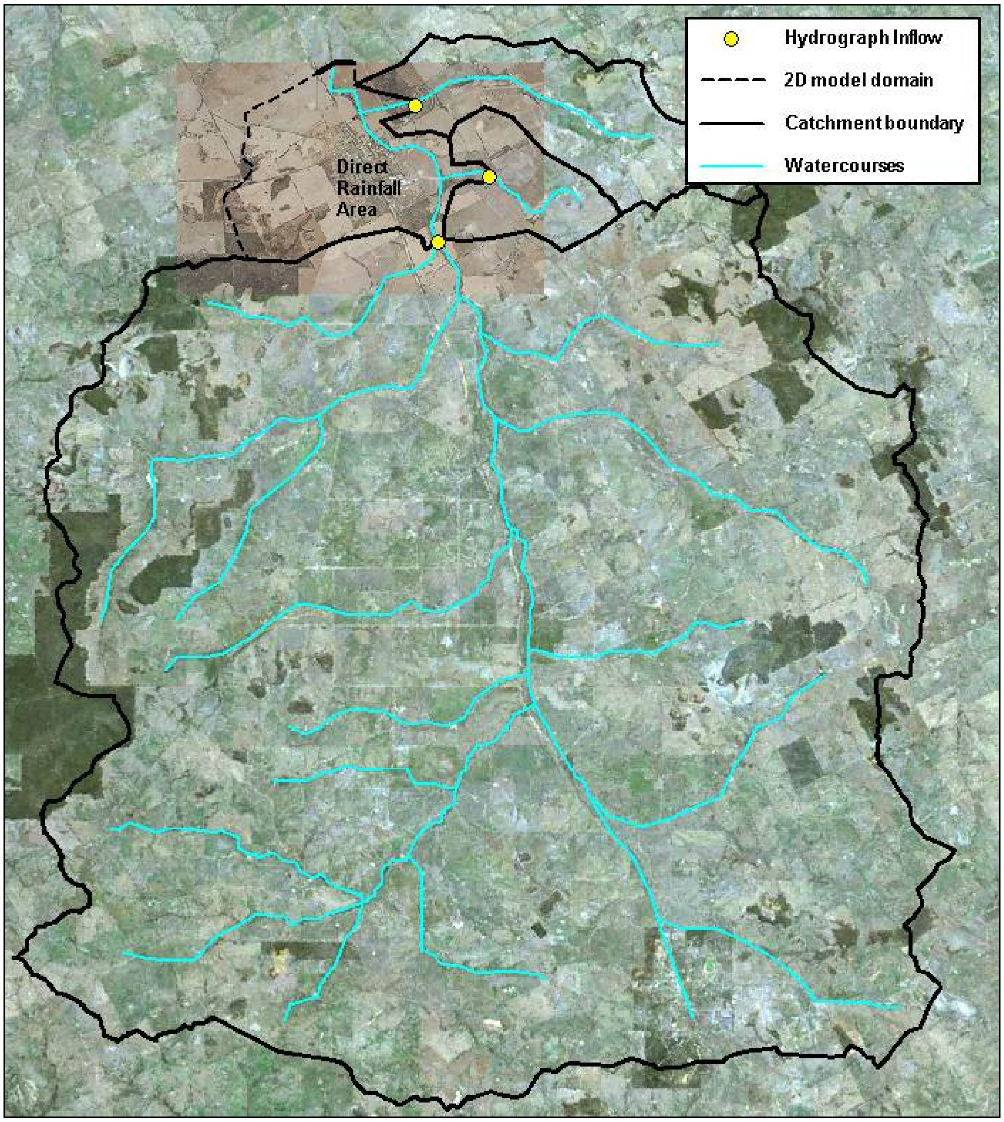


**Figure 3-2 Model layout and catchment plan**

### 3.2.3 Downstream Boundary Conditions

A water level versus flow (HQ) boundary was included 2 km downstream of the Brookton Highway bridge to allow water to exit the model. The slope for the HQ boundary was set to match the natural surface gradient at the outflow location, which allows TUFLOW to automatically generate the HQ curves.

### 3.2.4 Bridges

The following three bridges cross the Avon River South Branch within the model domain:

- the Brookton-Kweda Road bridge;
- a disused old railway bridge near the grain handling terminal; and
- the Brookton Highway bridge.

The locations of these bridges are shown in Figure 3-3. Surveyors AAM Vektra were contracted to conduct a ground survey of the Brookton-Kweda Road bridge and the Brookton Highway bridge, including abutments, soffit, upstands, headwalls, road levels and pile locations. The results of this ground survey (Appendix 3) were used to incorporate the bridges into the TUFLOW model.

The two surveyed bridges were modelled in TUFLOW using the ‘Layered Flow Constriction Shape’ facility, which models the bridge as four distinct layers:

- the first layer is for flow under the bridge deck, and some blockage and form losses would be applicable to the bridge piles;
- the second layer is for the bridge deck which would be 100% blocked and impede all flow;
- the third layer is the railing, which would be partially blocked and form losses would be applicable; and
- the fourth layer is flow over the railing and would be completely unimpeded.

The elevations of each layer were determined from the ground survey results. The TUFLOW User Manual (BMT Group, 2010) recommends calibrating models by adjusting the form loss coefficient of bridges (as well as other parameters). Form loss coefficients of 0.3 for layer one and 0.5 for layer three were selected as part of the calibration process (Section 3.3).

The disused old railway bridge near the grain terminal was not surveyed, however the DEM successfully picked up the surrounding elevations of the road and embankment. The bridge was modelled as a Flow Constriction Shape in TUFLOW, with the road level set at the elevation of the top of the approach embankment. The form loss coefficient was also varied during the calibration process and a value of 0.3 was selected for below the bridge deck, and 0.5 above the bridge deck.

**Figure 3-3 Bridge locations**

### 3.2.5 Culverts and Drain

During the project kick-off meeting WorleyParsons agreed to consider the flow of water through the town, which originates from a small catchment to the west of Brookton. To model these flows it was necessary to include one-dimensional elements in the model such as drains and culverts. The location of the cut-off drain and culverts included in the TUFLOW model are shown in Figure 3-4. The drain levels and dimensions were determined from the survey conducted by AAM Vektra. The culvert locations, types and sizes were determined form a drainage plan provided by the Shire, aerial photography and site visits. The majority of culverts with a diameter of 600 mm or greater were included in the model, however some minor culverts (300 – 450 mm diameter) marked on the drainage plan were not included. Only including the major culverts in the model provides a conservative approach to the estimation of flooding.

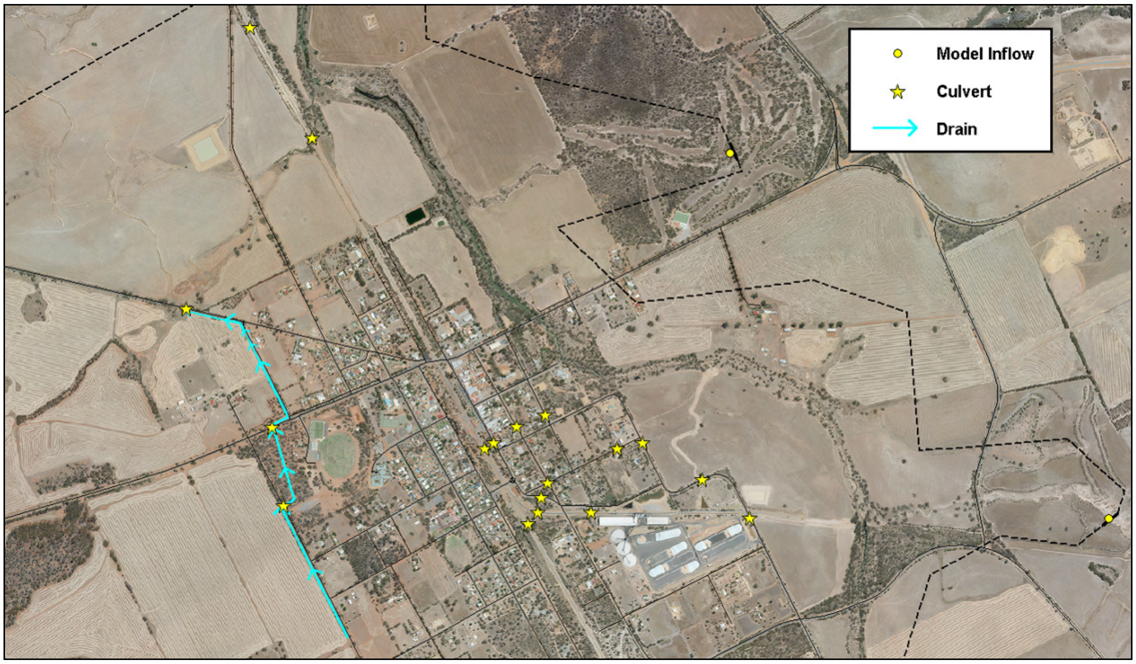


Figure 3-4 Culvert and drain locations

### 3.2.6 Bed Resistance

The bed resistance, or surface roughness, of the river and floodplain areas is required to simulate the flow of water. A smooth surface, such as a road, will allow water to flow faster while a rough surface or one with obstacles, such as trees or buildings, will cause water to flow more slowly and increase in depth.

The Brookton study area consists of urban development, pasture, vegetation and the river system. Site visits were undertaken on three occasions (27 June 2013, 5 December 2013 and 27-28 February 2014), during which the Avon River South Branch, its floodplain and surrounding areas were inspected. For the purpose of the model, ten land-use areas were defined, based on vegetation cover and expected resistance to overland flow. These areas were specified in the 2D model as pasture, vegetation, river channel, light riparian vegetation, dense riparian vegetation, roads, buildings, cleared land and water bodies, and are shown in Figure 3-5. All unshaded areas denote land classified as pasture. The land use areas were delineated from aerial imagery and familiarity with the area from site visits.

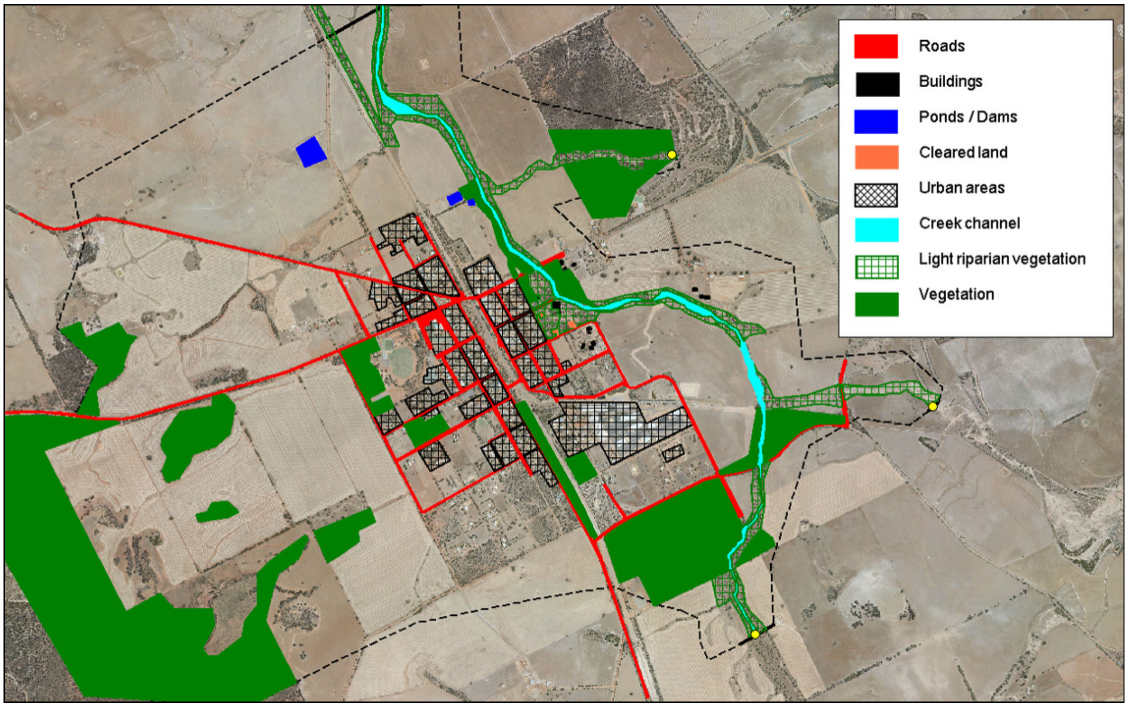


**Figure 3-5 Land use areas near Brookton**

The Brookton area outside of the town and the river system is dominated by pasture and small stands of vegetation. Photographs of typical pasture areas are shown in Figure 3-6. The Avon River South Branch consists of a low-flow channel within the broader river channel and is vegetated with grasses, weeds and trees in some areas. Photographs of typical sections of the river are provided in Figure 3-7.

Figure 3-6 Photographs of Brookton pasture areas

Figure 3-7 Photographs of Avon River South Branch channel

The Manning's *n* roughness coefficient was used in TUFLOW to set bed resistance for the model domain. The Manning's n values selected for the various land-use types are presented in Table 3-3. They are based on the site visits, typical values from literature and WorleyParsons' professional judgement. Varying Manning's n values by ± 20% formed part of the sensitivity analysis.

**Table 3-3 – Manning's *n* values for Brookton land-use types**

| Land-use | *n* | Description |
|---|---|---|
| **Pasture** | 0.04 | Pasture is generally short grains and grasses |
| **Vegetation** | 0.10 | Some trees and shrubs, moderate density |
| **River channel** | 0.05 | Some grasses, weeds and trees in channel |
| **Light riparian vegetation** | 0.06 | Sparse trees and shrubs with light understory |
| **Dense riparian vegetation** | 0.10 | Higher density of vegetation |
| **Roads** | 0.016 | Asphalt roads (Young et al, 2001) |
| **Buildings** | 1.00 | Estimate to model the effect of buildings on flow |
| **Cleared land** | 0.03 | Development sites with smoothed surface |
| **Water bodies** | 0.02 | Farm dams |
| **Urban areas** | 0.15 | Includes a mixture of buildings, cleared areas and vegetation |

## 3.3 Model Calibration

As discussed in Section 2, there is no streamflow gauging on the Avon River South Branch that can be used to calibrate the TUFLOW model. However, the model was calibrated to anecdotal information on peak flood levels for a major flood event caused by rainfall from the remnants of an ex-tropical cyclone in February 1955.

Long-term resident of Brookton, Ken Hall, witnessed the February 1955 event. Mr Hall accompanied the survey team, who surveyed peak flood levels for points that Mr Hall identified. A summary of Mr Hall's information is provided in Table 3-4. Some of the data appears contradictory, however the depths were estimates and it is possible that some points were not viewed at the peak of the flood.

Anecdotal information provided by other residents was that the main Brookton Highway bridge has not been overtopped but flow has reached the underside of the bridge deck (at an unspecified date). The anecdotal evidence also suggests that (at an unspecified date) water has flowed over Brookton Highway to the west of the bridge where the elevation of the road is lower than that of the bridge.

**Table 3-4 Anecdotal information for the February 1955 flood in Brookton**

| Location | Comment | Water Level Elevation (mAHD) |
|---|---|---|
| Vet Hospital | Flood extent was near floor level of hospital (Ken Hall most confident about this level). | 233.39 – 233.58 |
| Lennard Street, 80 m from Reynolds Street intersection | Flood depth was approximately 1 m here. | 233.6 |
| Lennard Street house | Flood depth was approximately 1.3 m here. | 234.17 |
| Lennard Street, 10 m from Reynolds Street intersection | Flood extent was to this location. | 233.1 |
| Brookton-Kweda Road bridge | Flood extent was to 40 m west of the bridge | 237.31 |

There was no pluviograph data available either at Brookton or at nearby locations, so it was not possible to determine the rainfall pattern at a sub-daily timescale. However, the daily rainfall record at Brookton (Station 10524) over the period from 14 to 18 February 1955 is provided in Table 3-5. The 48 hour period from 16 to 17 February was greater than a 1 in 100 AEP event, but as the catchment's critical duration has been determined to be 6 – 9 hours it is difficult to determine the event's AEP. Figure 3-8 presents the estimated AEP of the 1955 event compared to the Brookton IFD data for various durations.

**Table 3-5 February 1955 rainfall at Brookton**

| Date | Rainfall (mm) |
|---|---|
| 14 February 1955 | 9.7 |
| 15 February 1955 | 16.8 |
| 16 February 1955 | 61.7 |
| 17 February 1955 | 82.6 |
| 18 February 1955 | 6.4 |

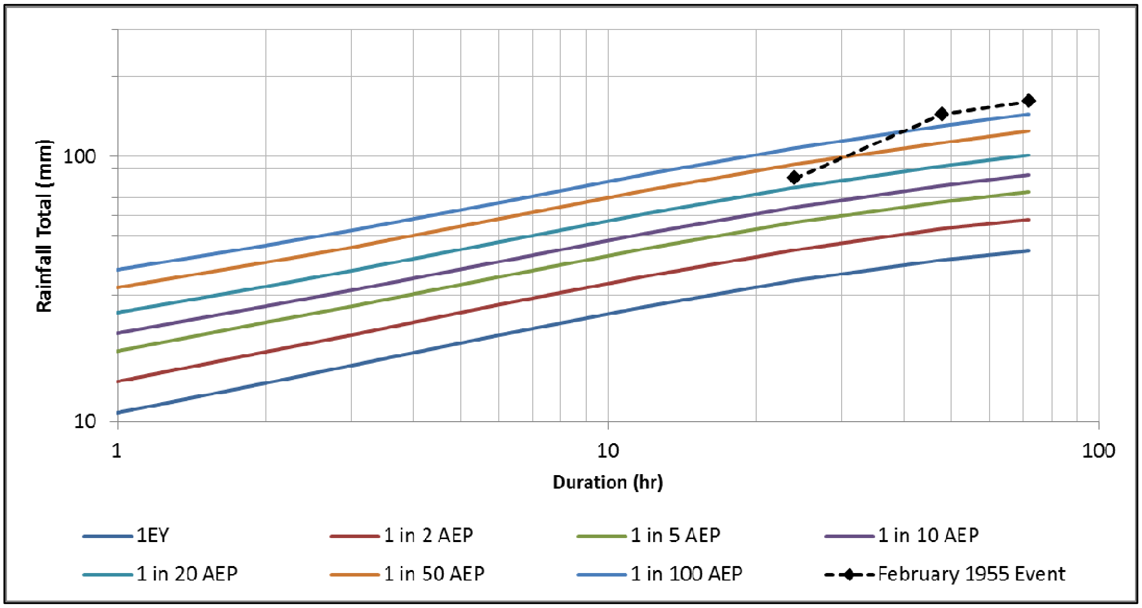


**Figure 3-8 February 1955 storm AEP**

### 3.3.1 RORB Modelling

The February 1955 rainfall event was routed with the RORB model developed previously to determine the inflow hydrograph for the TUFLOW model. In order to run the RORB model, an estimate of the rainfall pattern over the 5 day period was made. Two rainfall patterns were tested; the first is the 24 hour pattern from ARR for the South West Coast Division applied to each day of the storm; and the second is a constant rainfall intensity for each day of the storm. Figure 3-9 shows these patterns applied to the rainfall totals over the first four days of the 1955 event, at one hour intervals.

Three scenarios were tested with RORB for the 1955 event. The details of these are provided in Table 3-6. As the 1955 event occurred in summer, a high initial loss was used. Based on discussions with DoW, an initial loss of 70 mm was selected for Scenario 1 (In Figure 3-9 rainfall comprising the initial loss of 70 mm is shown in red, rainfall in excess of the initial loss is shown in blue). This was applied to the rainfall event commencing on 14 February and continuing to 18 February. Scenario 2 tested the effect of applying the same initial loss as was selected for the design rainfall events discussed in Section 2.3.2 (5 mm). The February 1955 storm event was a long duration event over several days that may not have had sharp peaks in rainfall intensity, so to determine a lower bound estimate of runoff from the catchment, Scenario 3 was run with an initial loss of 70 mm and the rainfall distributed according to the constant rainfall pattern.

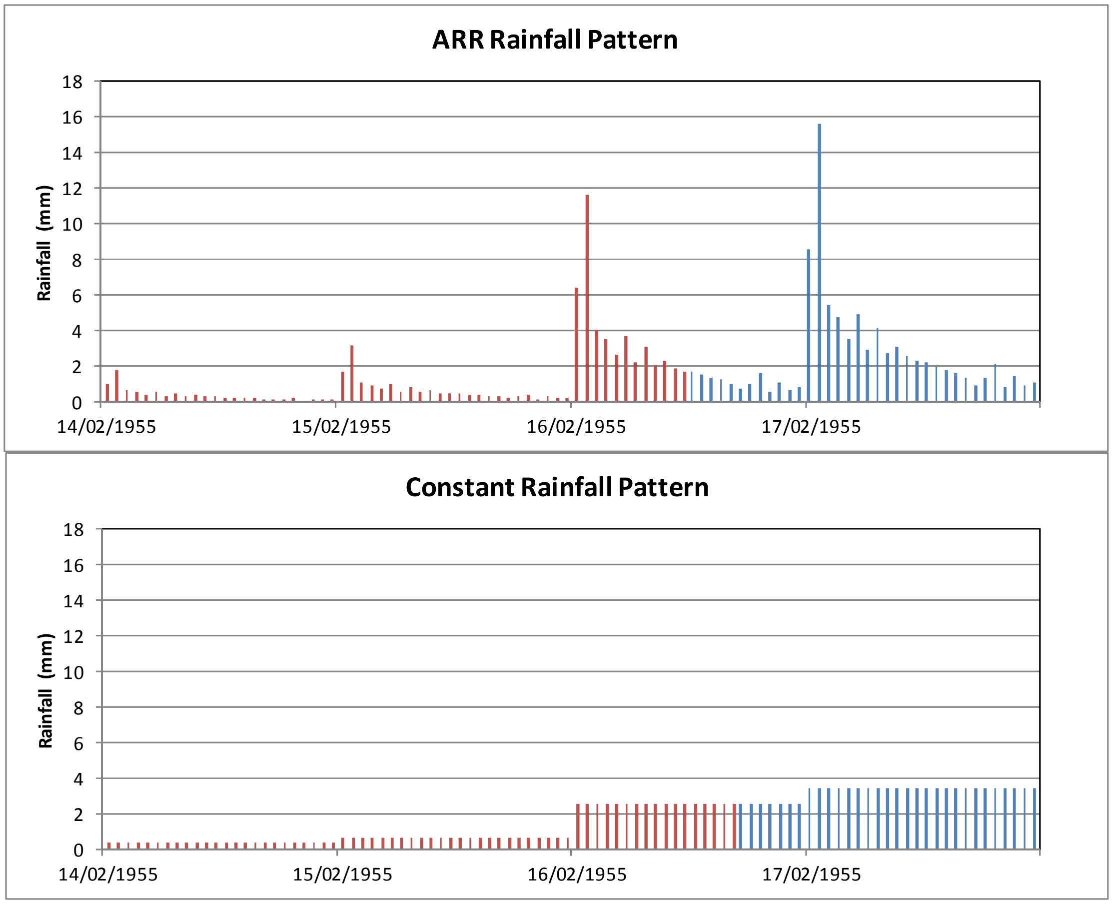


**Figure 3-9 Rainfall intensity patterns tested for the 1955 event**

**Table 3-6 Scenarios tested for the 1955 event**

| Scenario | Rainfall Pattern | Initial Loss (mm) |
|---|---|---|
| 1 | ARR Pattern | 70 |
| 2 | ARR Pattern | 5 |
| 3 | Constant Pattern | 70 |

Figure 3-10 shows the results for the three scenarios for the 1955 event. The model predicts that the minimum peak flow rate that could have been generated by the storm is 100 $m^3/s$, corresponding to the unlikely event that rainfall fell at a constant rate over the full duration of the storm (Scenario 3). The model predicts that the value selected for the initial loss has little effect on the peak flow rate for this event, with both the 70 mm (Scenario 1) and the 5 mm (Scenario 2) scenarios giving close to identical flows of ~150 $m^3/s$. Scenario 1 was adopted as the best estimate of the flow in the Avon River South Branch following the 1955 event, as it is based on a likely summer initial loss and the best estimate available of the rainfall pattern for the remainder of the event.

The hydrograph from Scenario 1 was used as the Avon River South Branch inflow in the TUFLOW model. Hydrographs for the two minor tributaries were determined by scaling the hydrograph shape and peak.

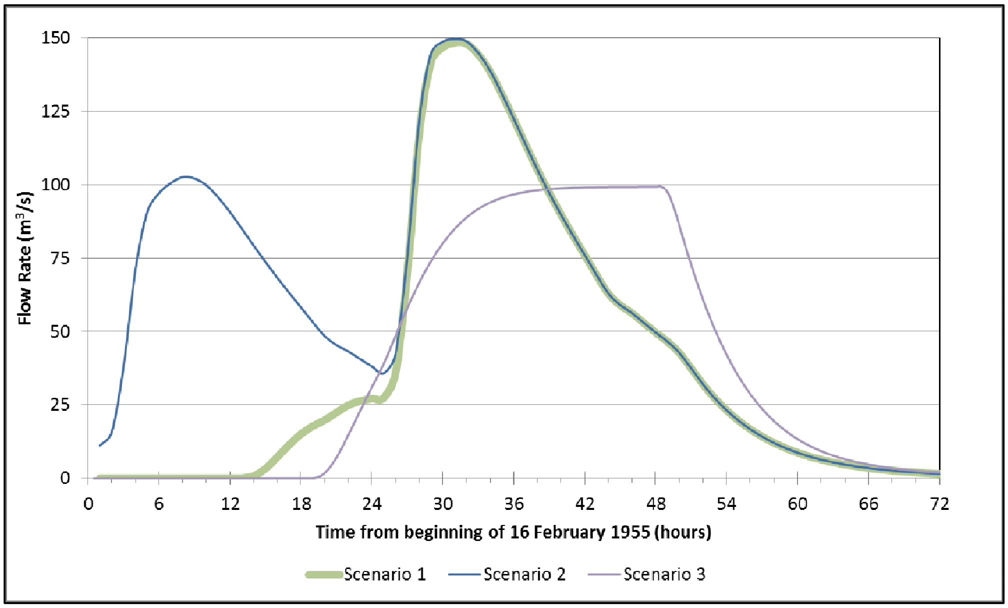


**Figure 3-10 RORB results for different rainfall patterns for the February 1955 event**

### 3.3.2 Hydraulic Modelling

Manning's n values and the form loss coefficients for the bridges were varied to calibrate the TUFLOW model to the observed levels. The vet hospital level was used as the primary calibration point as Mr Hall was most confident in this level. The model results for the February 1955 flood are compared with the anecdotal information from Mr Hall in Table 3-7 and show a reasonable match (within 0.3 metres) to observed levels.

**Table 3-7 TUFLOW model calibration results**

| Location | Observed Water Level Elevation (mAHD) | Modelled Water Level Elevation (mAHD) |
|---|---|---|
| Vet Hospital | 233.39 – 233.58 | 233.42 |
| Lennard Street, 80 m from Reynolds Street intersection | 233.6 | 233.42 |
| Lennard Street house | 234.17 | 233.42 |
| Lennard Street, 10 m from Reynolds Street intersection | 233.1 | 233.42 |
| Brookton-Kweda Road bridge | 237.31 | 237.19 |

# 4. MODEL RESULTS

The intent of the Brookton 2D hydraulic model was to characterise (i.e. depth and extent of flooding) the flooding in the vicinity of the town under existing development conditions. The modelled depth, level, velocity and hazard (See Section 4.2 for an explanation of flood hazard) of floodwaters for the 1 in 100 AEP event are shown in Figures 5.7 to 5.11. The modelling results are best viewed in the WaterRIDE software package provided to the Shire, which can be used to determine depths, levels, velocities, flow rates and hazard ratings for all design events.

The figures show that for the 1 in 100 AEP event significant flooding of some parts of Brookton is expected. The main area of concern for flooding in Brookton is the area east of Reynolds Street where floodwaters are expected to inundate some properties. The Brookton Highway bridge is not expected to be overtopped in the 1 in 100 AEP event, but the road to the west of the bridge is expected to be under water and may not be trafficable. In the 1 in 500 AEP event the bridge is expected to be overtopped and will not be trafficable. The Brookton-Kweda Road bridge is expected to be overtopped by the 1 in 20 AEP event and is not expected to be trafficable during this event and for larger events.

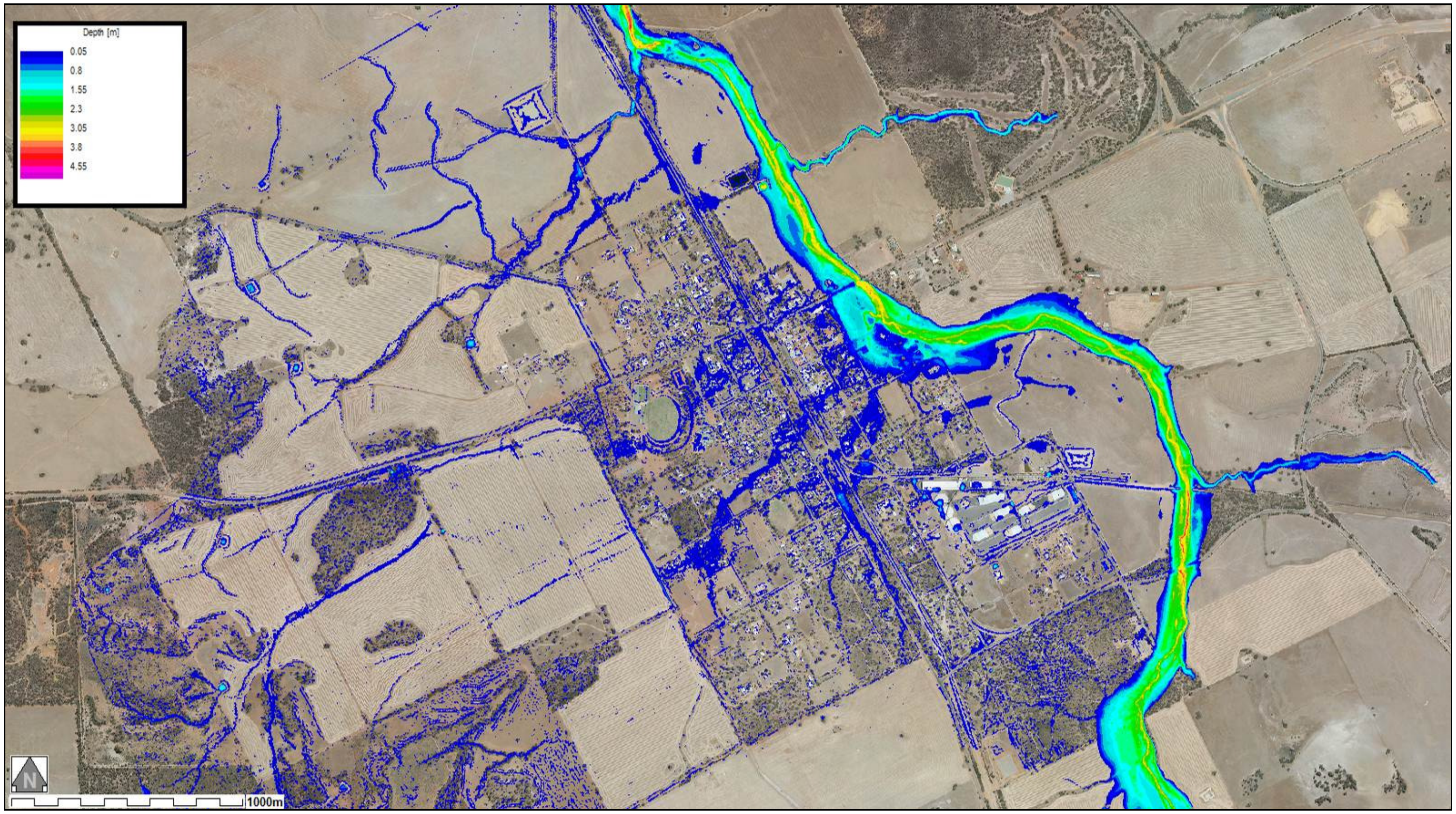


Figure 4-1 One in 100 year AEP modelled flood depths

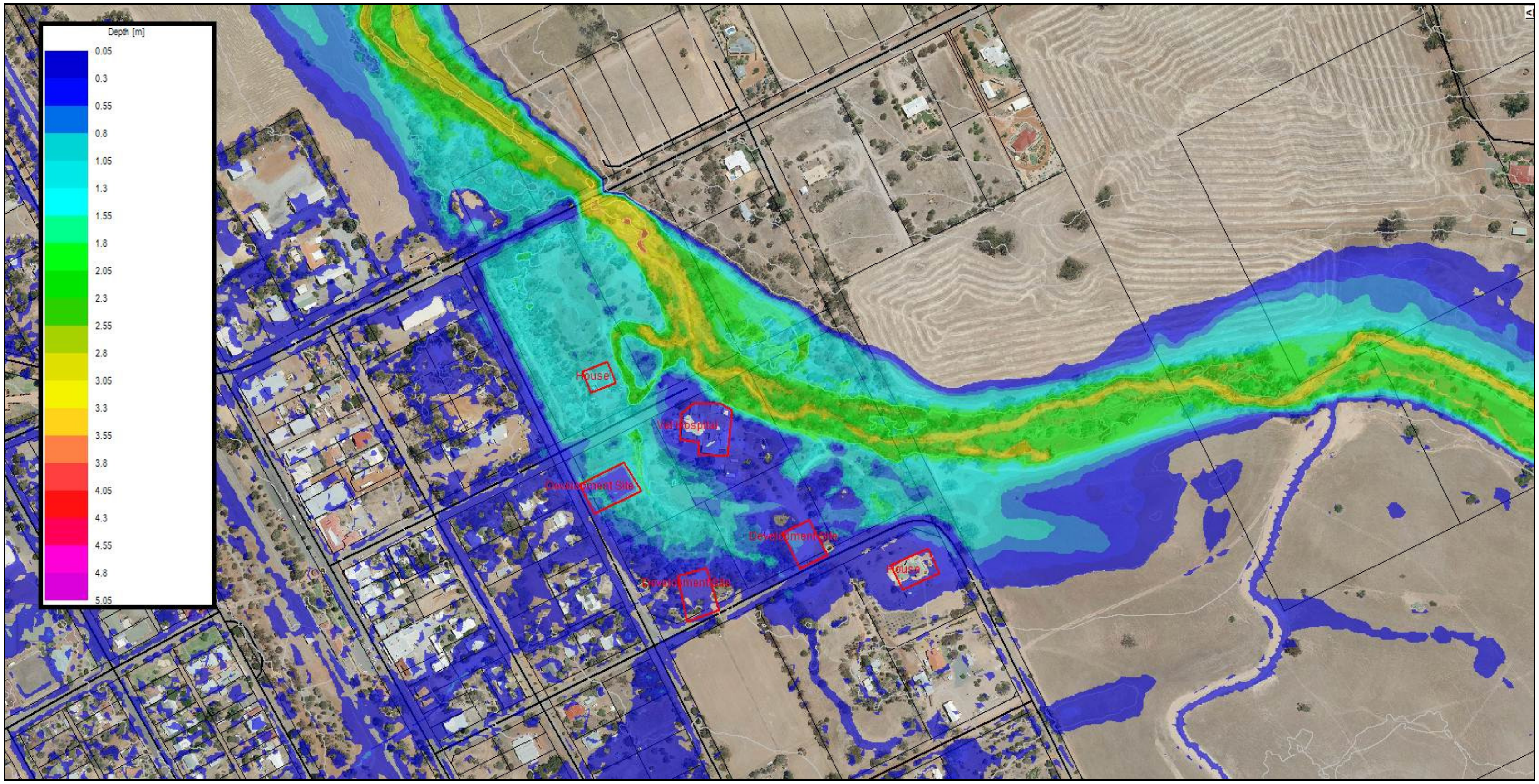


**Figure 4-2 One in 100 year AEP modelled flood depths – affected properties**

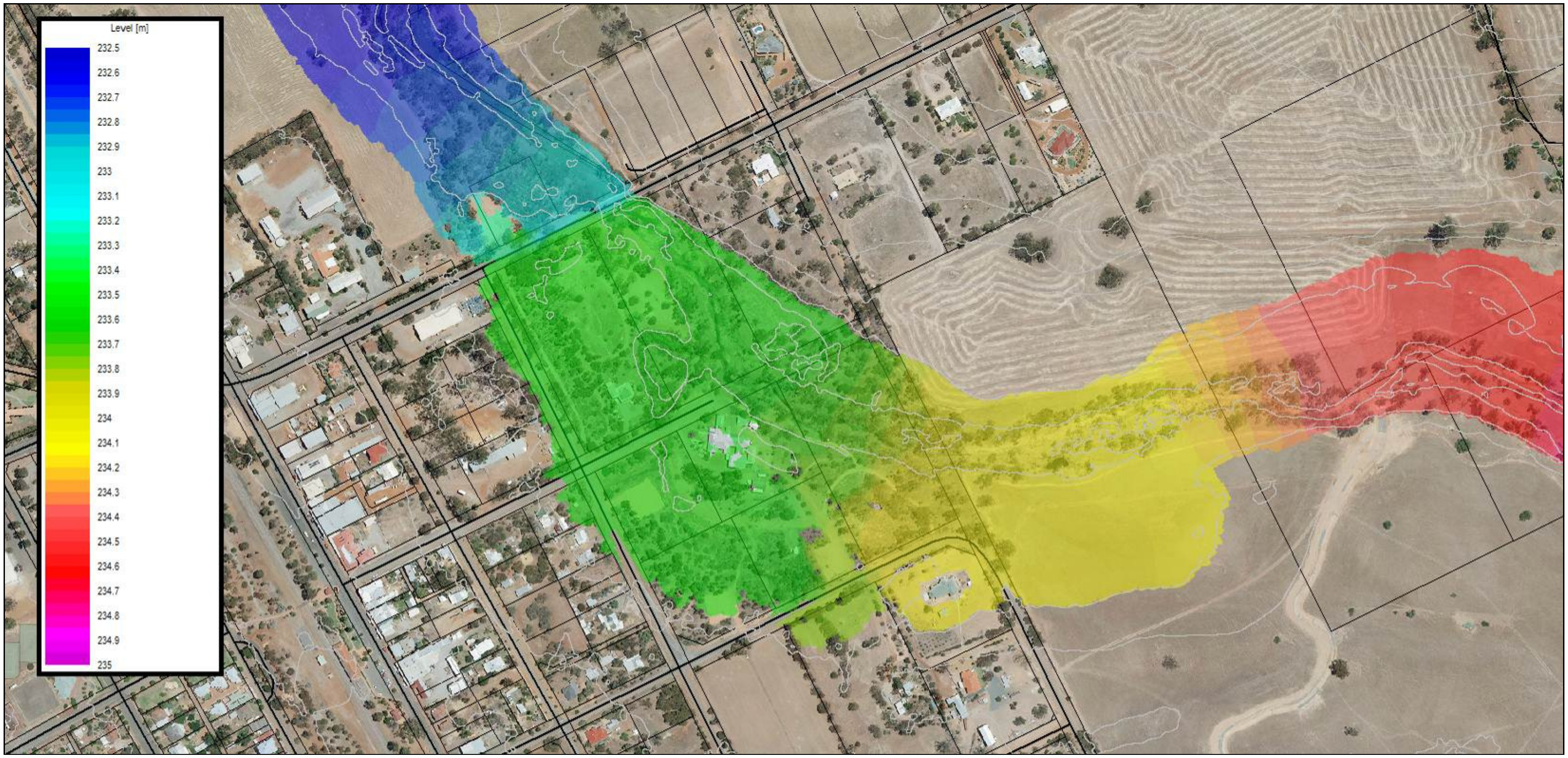


**Figure 4-3 One in 100 year modelled flood levels**

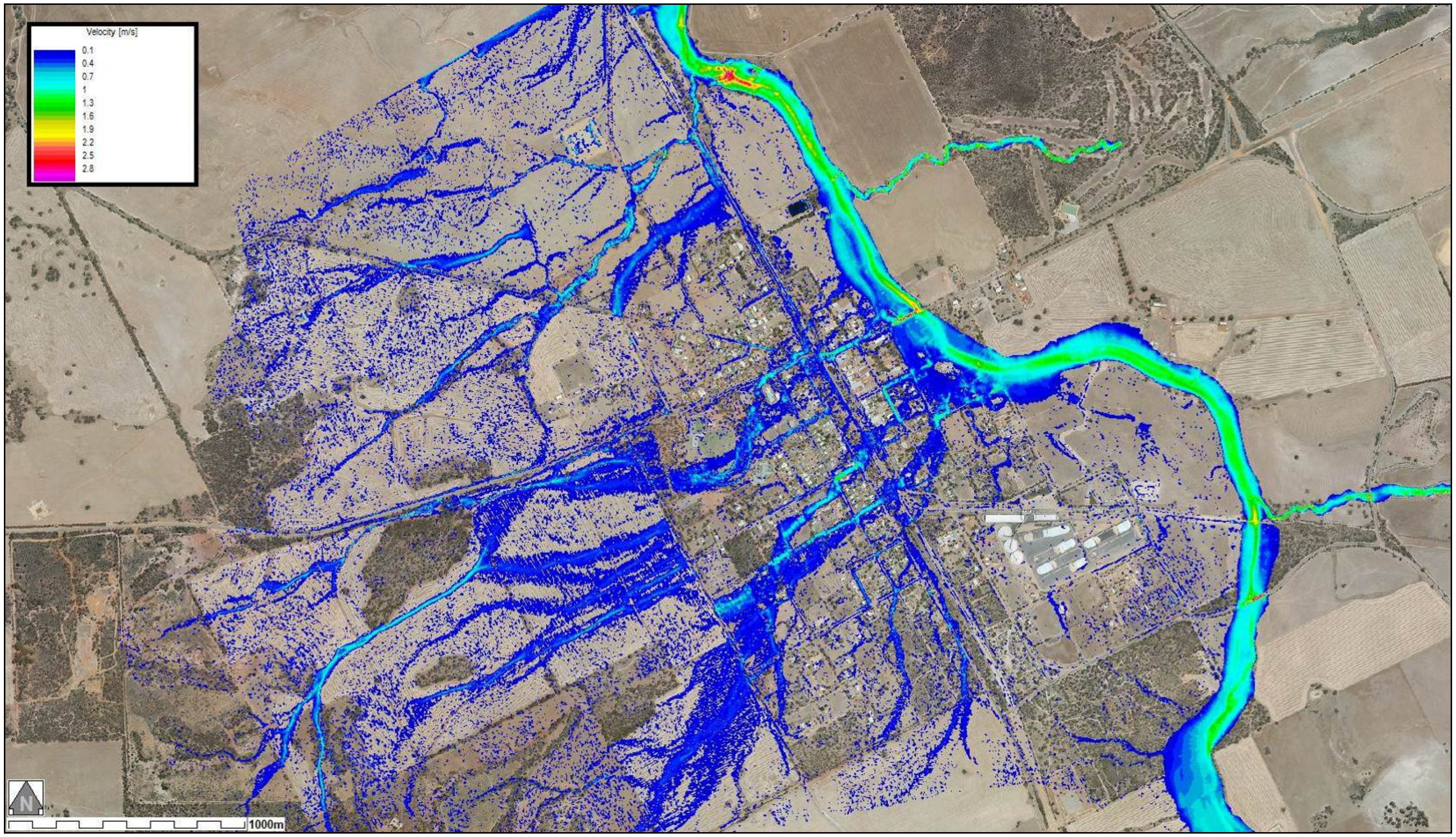


Figure 4-4 One in 100 year AEP modelled flood velocity

Figure 4-5 One in 100 year AEP modelled flood hazard

## 4.1 Townsite Drainage

During the 1 in 100 AEP event, some flows are expected to come from the hills to the west of town, but the cut-off drain is expected to intercept flows and reduce the risk to properties downstream. These flows are not expected to have a high hazard rating (See Section 4.2 for an explanation of flood hazard), with the possible exception of some roads where water velocity may be up to 1.5 m/s, but at shallow depth. It should be recognised that urban drainage is generally not designed for 1 in 100 AEP events and that areas of ponding can be expected during rare events.

For the 1 in 10 AEP event, which is more representative of the likely basis of design of the town drainage system, there are very few areas of high flood hazard in the town. One exception is a small section of Cumming Road between White and Williams streets, which has low-depth water moving at a velocity of 1 m/s. Another is the railway reserve near Noack Street, which acts as a small storage of water before it passes through a series of culverts and contains water up to 1 m deep in places. The model predicts that the town's drainage network will generally perform well for the 1 in 10 AEP event, although ponding of water in various areas is still expected.

During the 1 in 100 AEP event the model predicts that water will spill out of the western cut-off drain in some locations and flow overland through the town to the river. The ground survey results and the digital elevation model indicate that the drain is shallow in some locations (~0.4 m deep) which may be the reason that water is overflowing from the drain in these places. During the 1 in 10 AEP event the model predicts that the drain performs adequately, intercepting all of the flow.

## 4.2 Flood Hazard

Flood hazard will vary across the Brookton floodplain in time and location. Floodwaters will be deep and fast moving in some locations at a particular point in time, and shallow and slow moving in others. Factors that affect flood hazard are:

- Flood behavior (depth, velocity, rate of rise of floodwater, duration of flooding)
- Topography (whether there are evacuation routes);
- Population at risk (number of people, type of development); and
- Emergency management (the systems that are in place).

The SCARM (2000) document recognizes four degrees of flood hazard:

- **Low** – No significant evacuation problems, children and elderly can wade to safety with little difficulty, evacuation by small vehicle is possible.
- **Medium** – Fit adults can wade to safety but children and elderly may have difficulty, 4WD vehicles necessary for evacuation.
- **High** – Fit adults have difficulty wading to safety, maximum depths up to 1 m and velocities up to 1.5 m/s, evacuation by 4WD vehicles or trucks is only possible during early stages of flooding, boats or helicopters may be required.
- **Extreme** – Boats or helicopters required for evacuation, wading is not possible due to depth, velocity and rate of rise of floodwaters, maximum depths and velocities are above 1 m and 1.5 m/s respectively.

The relationship between depth and velocity of floodwaters and flood hazard is shown in Figure 4-6 (SCARM, 2000). This relationship has been adopted for determining flood hazard for the Brookton Flood Study.

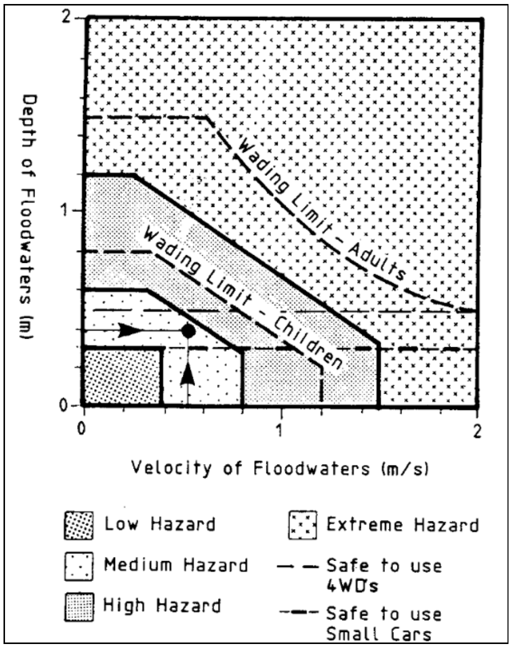


**Figure 4-6 Estimation of flood hazard**

The hydraulic modelling results presented in Section 4 show that for several of the design flood events there are areas of high and extreme flood hazard within the Brookton floodplain. The main hazards in Brookton identified for the 1 in 100 AEP event are presented in Table 4-1. The objectives of the Brookton Floodplain Development Strategy (Section 5) can be broadly thought of as managing the risks to the community posed by the flood hazard.

**Table 4-1 Main flood hazards in Brookton for the 1 in 100 AEP event**

| Location | Comment |
|---|---|
| The existing house on Lennard Street opposite the vet hospital | Located in an area of high hazard, with water up to 1 m deep. Children and elderly residents may have difficulty wading through water to safety. House likely to sustain major damage.<br>During the 1 in 500 AEP event the house is located within an area of extreme hazard with water depths of approximately 2.3 m. |
| The vet hospital on Lennard Street | Mostly in low hazard but located immediately adjacent to an extreme hazard area. Evacuation route is available through medium hazard areas to Monger Street. Flood level may go slightly above floor of building.<br>During the 1 in 500 AEP event the site is located within area of extreme hazard with an evacuation distance of 200 m to reach areas of lower hazard. |
| The three proposed development sites, one on Reynolds Street and two on Monger Street | All are located in areas of low to medium hazard with evacuation routes easily accessible. The Reynolds Street site is located adjacent to an area of extreme hazard. Depth of water based on current pad levels ranges from 0.25 – 0.50 m.<br>During the 1 in 500 AEP event sites are all located within areas of extreme hazard. |
| The existing house on the corner of Monger Street and Bennell Street | Surrounded by water of low to medium hazard with an easily accessible evacuation route to Bennell Street. House floor level is expected to be above water level for the 1 in 100 AEP event.<br>House will be inundated during the 1 in 500 AEP event and will be surrounded on all sides by extreme hazard water. |
| The Brookton Highway bridge | The bridge is expected to be dry, but the road immediately to the west is at a lower level and is expected to have up to 0.5 m of water on it, at moderate to high velocity. The hazard rating of this section of road is extreme and it may not be trafficable to vehicles. |

| Location | Comment |
|---|---|
| | During the 1 in 500 AEP event the water level could be up to 1.5 m deep to the west of the bridge and the bridge itself could be overtopped with up to 1 m of water above the road surface. The road and bridge would be an area of extreme hazard and would be impassable. |
| The Brookton-Kweda Road bridge | The bridge and approach road are expected to be in an area of extreme hazard during the 1 in 100 AEP event and will be impassable, with up to 1.3 m of water above the bridge deck.<br>During the 1 in 500 AEP event the road will be impassable, with up to 2.5 m of water above the bridge deck. |
| The disused old railway bridge adjacent to the grain terminal | The bridge and approach road are expected to be safe to drive on during the 1 in 100 AEP event.<br>During the 1 in 500 AEP event the bridge and approach road are expected to be in areas of extreme hazard, with up to 1.5 m of water above the road, and will be impassable. |

## 4.3 Sensitivity Analysis

A sensitivity analysis was conducted on the 1 in 100 AEP event to determine the influence that uncertainty in some model parameters may have on the results. The Manning’s n values were increased and then decreased by 20% whilst keeping all other parameters unchanged. Manning’s n values are selected by the modeller and as such there is a degree of inherent uncertainty in the values. Separate model runs were undertaken by varying the inflows by ± 20% to account for the uncertainty in the RFFP peak flow estimates, whilst keeping all other parameters unchanged.

The results of the sensitivity analysis show that the Manning’s n values and the RFFP peak flow estimates have an effect on the flood levels, however the overall effect of flooding on the town, the proposed development sites and other properties remains the same. The results are presented in Table 4-2, and in Figures 4.7 to 4.10. Wet-to-dry and dry-to-wet areas are indicated by blue and pink cross-hatching respectively. The results of the sensitivity analysis give confidence that the inherent uncertainty in the modelling process is not expected to significantly alter the recommended actions in the floodplain development strategy.

**Table 4-2 Sensitivity analysis results – 1 in 100 AEP event**

| Scenario | Proposed Development Sites | | Brookton Highway Bridge | |
|---|---|---|---|---|
| | Change in Levels (m) | Change in Velocity (m/s) | Change in Levels (m) | Change in Velocity (m/s) |
| Manning’s n + 20% | + 0.10 – 0.12 | ± < 0.10 | + 0.20 | - 0.20 |
| Manning’s n - 20% | - 0.12 | ± < 0.10 | - 0.18 | + 0.25 |
| Model Inflows + 20% | + 0.15 – 0.21 | + 0.15 | + 0.21 | + 0.03 |
| Model Inflows - 20% | - 0.22 | - 0.14 | - 0.22 | - 0.03 |

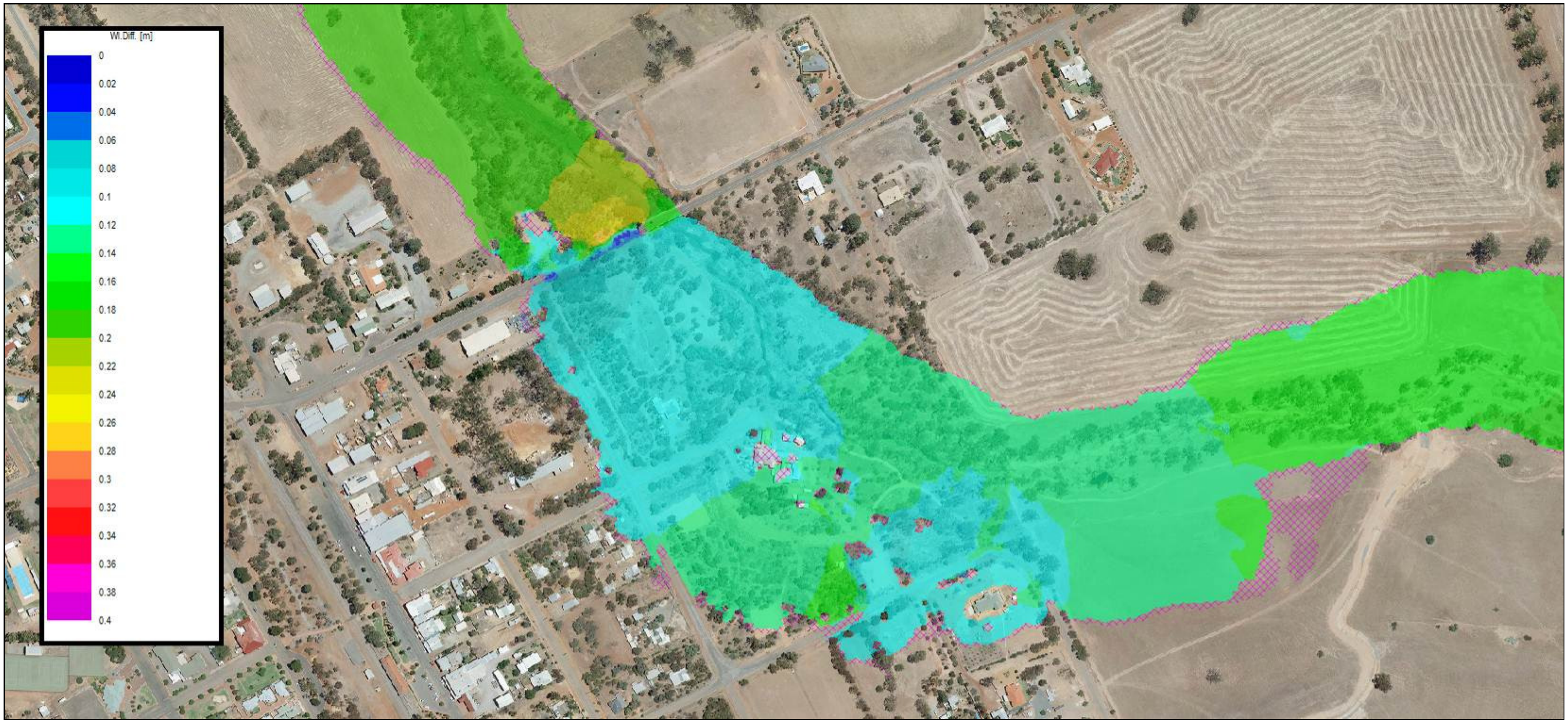


**Figure 4-7 1 in 100 AEP flood level difference from increasing Manning's n by 20%**

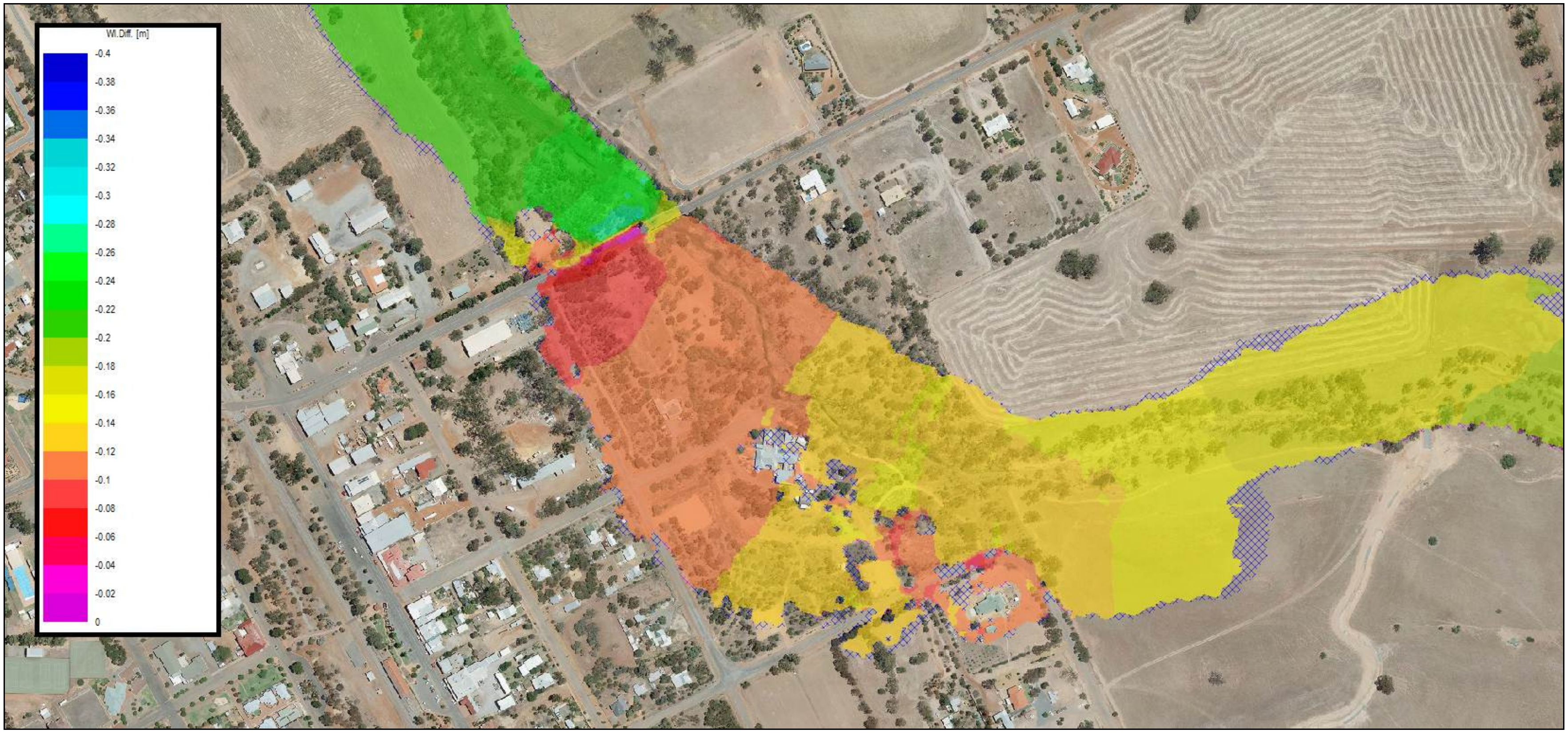


**Figure 4-8 1 in 100 AEP flood level difference from decreasing Manning's n by 20%**

Figure 4-9 1 in 100 AEP flood level difference from increasing flow rates by 20%

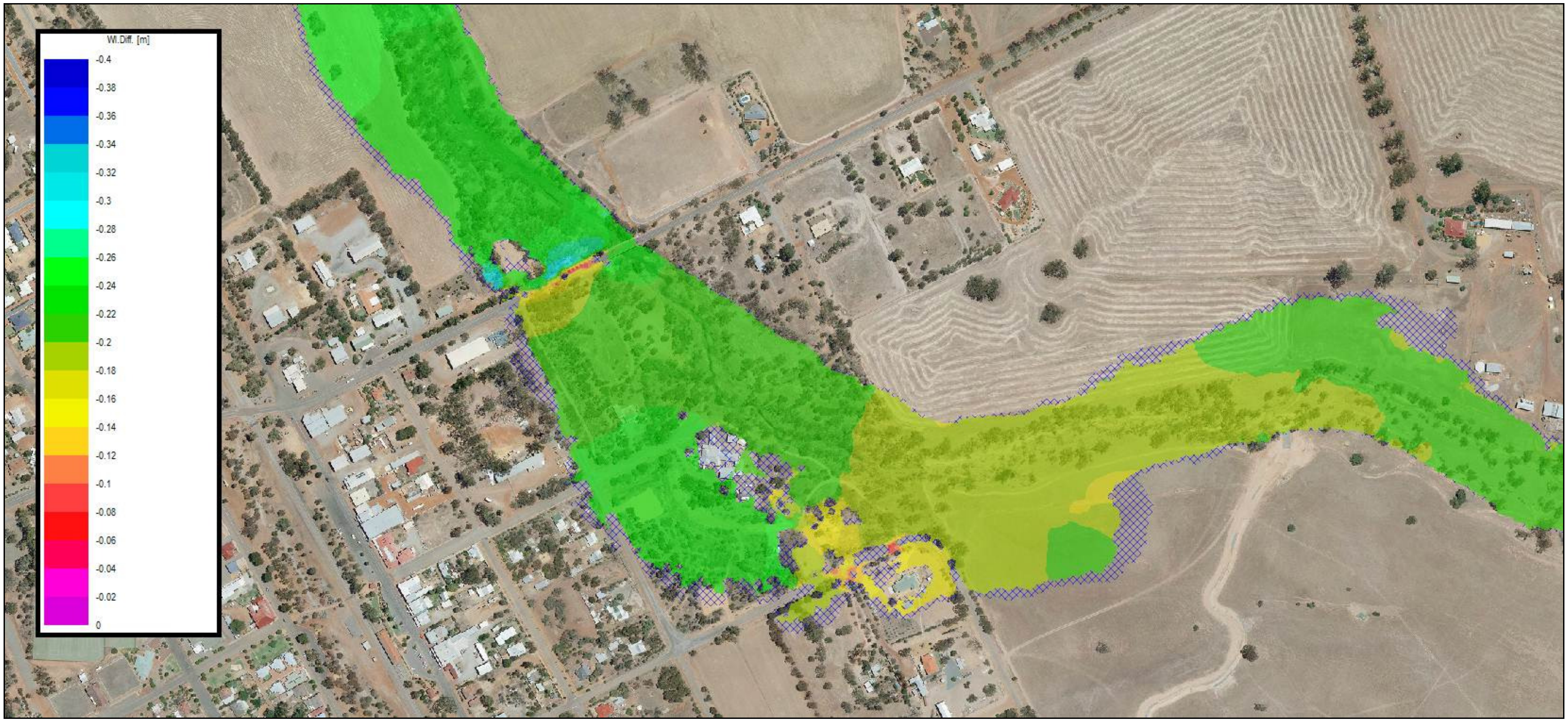


**Figure 4-10 1 in 100 AEP flood level difference from decreasing flow rates by 20%**

# 5. FLOODPLAIN DEVELOPMENT STRATEGY

## 5.1 Background

A floodplain development strategy (FDS) is a document that sets the overall strategy for development within the floodplain and identifies and compares options for managing flood risk. The recommended Brookton FDS has been developed with consideration given to the guiding document, *Floodplain management in Australia: best practice principles and guidelines* (SCARM, 2000).

The Department of Water (DoW) is the Western Australian Government's lead agency for providing floodplain management advice. DoW promotes the wise use of floodplains while minimising flood risk and damage (WRC, 2000). Figure 5-1 shows the floodplain mapping and floodplain management process in Western Australia. The floodplain development strategy (or floodplain management strategy) is a high-level document that designates the areas affected by flooding, recommends strategies for development and advises on flood mitigation options (both structural and non-structural). Following the development of the FDS, there are a number of other actions and plans that may need to be initiated or developed.

The main role of Local Government in floodplain management is the implementation of the FDS (WRC, 2000). This is done through statutory planning and building regulations, as well as the implementation of structural controls where appropriate. The Brookton FDS should be incorporated into the Brookton Town Planning Scheme (which is currently being updated) or into a Council Policy Statement. Local Government should also actively promote flood awareness and work closely with the appropriate emergency response agencies on local flood emergency planning.

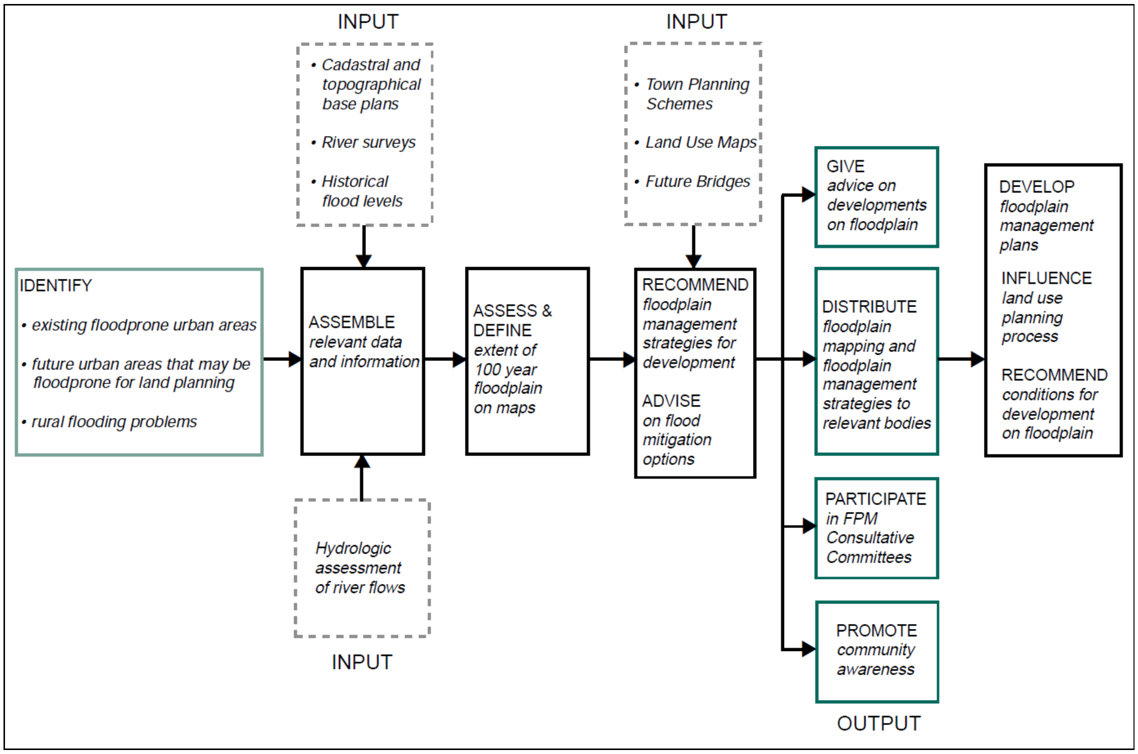


**Figure 5-1 DoW floodplain management process (WRC, 2000)**

## 5.2 Objectives

The objectives of floodplain management as set out by the SCARM (2000) document are:

- Limit the effect of flooding on the well-being, health and safety of individuals and communities to acceptable levels;
- Limit the damage caused by flooding to public and private property to acceptable levels;
- Ensure that the natural function of the floodplain and any flood-dependent ecosystems is preserved; and
- Encourage the planning and use of floodplains as a resource capable of multiple, but compatible, land uses of benefit to the community.

The Brookton FDS has been developed with these objectives in mind.

## 5.3 Principles

The principles of the Brookton FDS are based on those from the Water and Rivers Commission’s (now Department of Water) *Water Facts 14: Floodplain management* (2000) document. They are:

- Ensure land use minimises flood risk and damage costs;
- Ensure appropriate floodplain mitigation measures minimise damage and are acceptable to the local community;
- Promote the use of non-structural rather than structural mitigation measures where possible; and
- Ensure floodplain management measures have beneficial economic, social and environmental outcomes.

## 5.4 Risk Reduction Measures

There are three types of flood risk in a floodplain:

- **Existing risk** – refers to existing buildings and developments within the floodplain, which are exposed to an existing risk of flooding.
- **Future risk** – refers to future developments within the floodplain, which will only be exposed to a risk of flooding once they occur.
- **Residual risk** – refers to the risk posed by floods that exceed management measures currently in place. Unless measures are designed to withstand the PMF (Probable Maximum Flood) there will be a flood event that exceeds them at some time in the future (SCARM, 2000).

There are four types of controls to manage flood risk that be categorised into two main groups:

- **Non-structural controls**;
    - Land use planning;
    - Development and building controls;
    - Flood emergency plans;
- **Structural controls**;
    - Structural controls.

Land use planning controls are the most cost-effective means of preventing future flood damage from occurring and include zoning and voluntary purchase of existing properties.

Development and building controls are the conditions that can be attached to development approvals, which may include setting minimum floor levels, floodproofing measures and freeboard requirements.

Structural controls are physical controls that are built to prevent and minimise flooding. They include levees, bypass floodways, channel improvements, detention basins and dams.

Flood emergency plans refer to flood forecasting and warning systems, evacuation plans, plans for the relief of evacuees and plans for the recovery of the area after the flood subsides (SCARM, 2000).

Land use planning controls, development and building controls and structural controls are effective at managing existing and future flood risk, however only flood emergency plans can manage residual risk, as there will always be a flood that exceeds the other measures at some point in time.

The following measures are discussed in relation to the town of Brookton.

### 5.4.1 Structural Controls

Due to the nature of flooding at Brookton and the location of existing infrastructure, there is limited opportunity to manage flood risk through structural controls. There are some options for improving local drainage within the town, however the flood risk posed by the Avon River South Branch is unlikely to be manageable through structural controls. The structural controls available to the Shire are discussed below.

A levee could be built to protect either two or three of the proposed development sites from floodwaters from the 1 in 100 AEP event. It would run from Monger Street around two (or three) of the proposed development sites to near the corner of Lennard and Reynolds Streets, as indicated by the pink line in Figure 5-2. There are a number of disadvantages of the levee, which are summarised in Table 5-1.

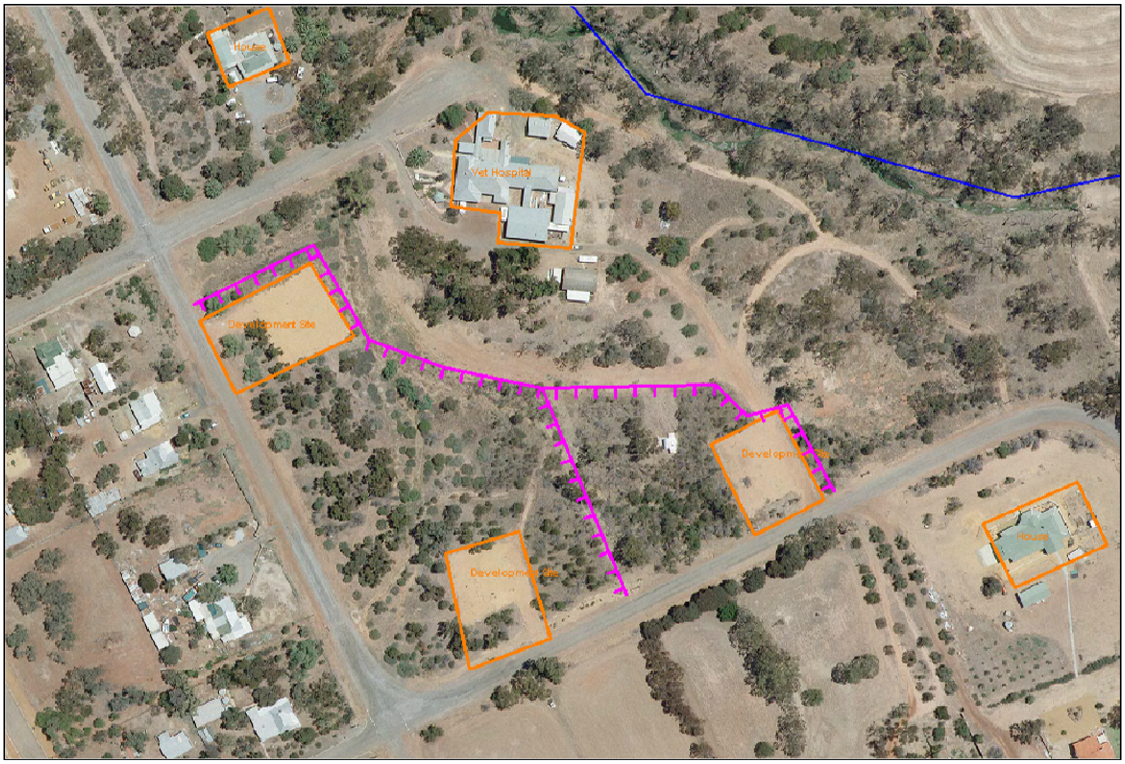


Figure 5-2 Possible levee location

Table 5-1 Cost-benefit assessment of levee

| Criteria | Assessment |
|---|---|
| **Flood risk management** | Could reduce the risk of flooding at proposed development sites (low velocity water up to 0.5 m depth) from the river up to the 1 in 100 AEP event.<br>During the 1 in 100 AEP event and larger events floodwaters from the river may enter the protected area via the adjacent roads.<br>Levee may prevent the natural drainage of local runoff from the town to the river. This may result in localised flooding within the area protected by the levee. |
| **Environmental** | Levee will prevent inundation of parts of the natural floodplain and block natural drainage lines.<br>Some vegetation will need to be cleared for the levee. |

| | |
|---|---|
| **Social** | The dirt track from the vet hospital to Monger Street would need to be realigned to ensure that evacuation routes are maintained. |
| **Economic** | The cost of the levee is expected to be high compared to its potential benefits. The 300 – 400 m long levee is expected to cost in the order of $60,000 to $90,000, but the exact cost will depend on several unknown factors. |
| **Overall** | The flood risk at the proposed development sites can be appropriately managed through other less costly measures. |

Stream improvements to increase the hydraulic capacity of the Avon River South Branch are another structural control that could be used to manage flood risk. This could be achieved by removing weeds and other invasive species that may be present in the stream channel and overbank areas. It should be stressed that native vegetation should not be removed. The advantages and disadvantages of this action are presented in Table 5-2.

**Table 5-2 Cost-benefit assessment of stream improvements**

| Criteria | Assessment |
|---|---|
| **Flood risk management** | May reduce the extent of flooding and reduce flood depths by increasing the capacity of the existing channel. However, the effects are expected to be limited, particularly for the larger events such as the 1 in 100 AEP event. |
| **Environmental** | Removing weeds and invasive species would be a positive environmental outcome, however native species should not be removed. |
| **Social** | Improved amenity of river. |
| **Economic** | Expected to be a low-cost measure that could be implemented by Shire personnel. |
| **Overall** | Likely to be a low-cost option with positive outcomes, but the effect on flood risk is expected to be minimal. |

Regularly clearing culverts of debris and sediment is another structural control that could help to manage flood risk in Brookton. The advantages and disadvantages of this action are presented in Table 5-3.

**Table 5-3 Cost-benefit assessment of clearing culverts of debris**

| Criteria | Assessment |
|---|---|
| **Flood risk management** | The capacity of existing culverts would be increased to their design rate and reduce flooding caused by backwater effects.<br>The benefits would be limited to improving drainage in Brookton's urban areas, reducing the effects of local rainfall and runoff. Flooding caused by the Avon River South Branch would not be reduced. |
| **Environmental** | No effect expected. |
| **Social** | Improved local drainage would reduce the effect of flooding on local residents and businesses. |
| **Economic** | Expected to be a low-cost measure that could be implemented by Shire personnel. |
| **Overall** | Likely to be a low-cost option with positive outcomes, but will not reduce the flood risk posed by the Avon River South Branch. |

The western cut off drain that intercepts runoff from nearby hills to the west of the town might require maintenance and/or improvements to ensure that its design capacity is not reduced. The model predicts that during larger events (such as the 1 in 50, 1 in 100 and 1 in 500 AEP events) there are several locations where water breaks out of the drain and flows overland through the town to the river. In practice this could be caused by debris blocking the drain, shallow sections of the drain where flow can break out, or by the capacity of the drain being exceeded. Within the scope of the current project it was not possible to fully investigate the effect of the drain on localised flooding within the town, but it may be prudent for the Shire to look at whether the drain's capacity could be increased through regular maintenance or small improvement works.  This could include clearing the drain of debris, deepening or widening the drain or by installing small bunds adjacent to shallow sections of the drain to limit breakout. The advantages and disadvantages of this action are presented in Table 5-4.

**Table 5-4 Cost-benefit assessment of drain maintenance**

| Criteria | Assessment |
|---|---|
| **Flood risk management** | The capacity of the existing drain that directs runoff from nearby hills around the town could be increased to its design rate and reduce flooding caused by water breaking out of the drain and flowing through the town.<br>The benefits would be limited to improving drainage in Brookton's urban areas, reducing the effects of local rainfall and runoff. Flooding caused by the Avon River South Branch would not be reduced. |
| **Environmental** | No significant costs or benefits expected. |
| **Social** | Improved local drainage would reduce the effect of flooding on local residents and businesses. |
| **Economic** | Expected to be a low-cost measure that could be implemented by Shire personnel with the assistance of contractors. |
| **Overall** | Likely to be a low to medium cost option with positive outcomes, but will not reduce the flood risk posed by the Avon River South Branch. |

### 5.4.2 Non-Structural Controls

Non-structural controls are required to manage flood risk at Brookton. There are several properties and structures that are located within the 100 year floodplain, some of which are exposed to a hazard rating of 'high'. They include:

- The existing house on Lennard Street opposite the vet hospital;
- The vet hospital on Lennard Street;
- The three proposed development sites, one on Reynolds Street and two on Monger Street;
- The existing house on the corner of Monger Street and Bennell Street;
- The Brookton Highway bridge;
- The Brookton-Kweda Road bridge; and
- The disused old railway bridge adjacent to the grain terminal.

For the 1 in 500 AEP event, all of these sites are located in areas of 'extreme' hazard and other commercial and residential areas are also exposed to hazards ranging from 'low' to 'extreme'. The focus of discussion in this section will be for events up to the 1 in 100 AEP event, however consideration should also be given to the 1 in 500 AEP event (and larger events up to the PMF), particularly in terms of emergency management planning.

The main land use planning control available to the Shire is zoning of land into appropriate land uses via the town planning scheme, which will limit flood damage to future developments by planning development to occur in appropriate locations. Land with a high or extreme hazard rating for the 1 in 100 AEP event is generally unsuitable for all purposes other than open space or recreation. Future zoning of Shire land should consider flood hazard as identified in this flood study. The advantages and disadvantages of this action are presented in Table 5-5.

**Table 5-5 Cost-benefit assessment of zoning controls**

| Criteria | Assessment |
|---|---|
| **Flood risk management** | Effective at preventing future flood risk by not allowing inappropriate developments to occur in flood prone areas. There are some areas that are currently zoned 'residential' and 'rural townsite' that are exposed to high and extreme flood hazard from the 1 in 100 AEP event. These areas would likely be zoned differently if there was no existing development in place. Zoning controls are not effective in addressing existing flood risk. |
| **Environmental** | Environmental benefits are achieved by preventing development on floodplain areas and leaving space for normal riparian and ecological processes to occur. |
| **Social** | Social amenity of riparian areas is maintained by preventing unsuitable development and maintaining ecological processes. Areas can be used for open space and recreation, allowing the community to enjoy the river and riparian environment. |
| **Economic** | Zoning controls should not incur additional expenses. |
| **Overall** | Zoning controls are highly effective in managing future flood risk, cheap to implement and result in social and environmental benefits, but are not effective in managing existing flood risk. |

For situations where there exists unsuitable development within high hazard areas of the floodplain that cannot be managed through other controls, voluntary purchase of the property can be considered. The house on Lennard Street may fall into this category, as it is located in an area of high hazard and may be difficult for children or elderly residents to evacuate safely. The house is estimated by local resident Ken Hall to have been under 1 – 1.3 m of water during the 1955 event, and the model predicts similar levels for the 1in 100 AEP event. There is currently no flood warning or forecasting system in place and the time to peak from the beginning of rainfall is in the order of 6 – 7 hours (2 hours from when the river starts to rise). If the house is to continue to be used as a residence, emergency management planning will need to consider these issues. If the risk still cannot be managed, voluntary purchase could be considered. The advantages and disadvantages of this action are presented in Table 5-6.

**Table 5-6 Cost-benefit assessment of voluntary purchase**

| Criteria | Assessment |
|---|---|
| **Flood risk management** | Effective at managing existing flood risk by removing people and property from areas of high hazard. |
| **Environmental** | No effect expected. |
| **Social** | Purchases would only ever be on a voluntary basis, but selling one's home could still result in a sense of loss or discontent. |
| **Economic** | Purchases are likely to be very expensive compared to other types of controls. |
| **Overall** | An effective last-resort option to prevent loss of life, but very expensive and potentially socially disruptive. |

The main development control available to the Shire is the setting of minimum floor levels of buildings within the floodplain (other development controls are listed in SCARM (2000) but are not suitable for Brookton). In Western Australia, the Department of Water typically recommends minimum habitable floor levels of 0.50 m above the 1 in 100 AEP flood level (WRC 2000). This action would help to manage the flood risk of the three proposed development sites. The advantages and disadvantages of this action are presented in Table 5-7.

**Table 5-7 Cost-benefit assessment of setting minimum floor levels**

| Criteria | Assessment |
|---|---|
| **Flood risk management** | Effective at managing existing and future flood risk by elevating property above the 100 year level. Consideration must still be given to evacuation routes for events larger than the 1 in 100 AEP event. |
| **Environmental** | No effect expected. |
| **Social** | No effect expected. |
| **Economic** | Costs would be incurred by developers, and are expected to be moderate. |
| **Overall** | Moderate expense for property developers, but reduces flood risk in terms of future property damage. Does not eliminate residual risk from larger flood events occurring. |

The final type of control available to the Shire to manage flood risk is emergency management planning, which includes flood forecasting and warning, planning for the evacuation of an area and for the recovery of the area after the flood subsides. Communication of emergency management plans to the community is essential, particularly with respect to evacuation. This is important for Brookton, as there is currently no formal flood warning system in place, the time to peak for the river is short and there are likely to be people located in areas of high to extreme flood hazard. Effective emergency planning will be necessary to manage the flood risk associated with the flood affected properties and bridges in Brookton mentioned previously.

Emergency management planning is the only control that can address residual flood risk, as there will always be a larger flood than the design event at some point in the future; it is a matter of when, not if. The advantages and disadvantages of this action are presented in Table 5-8.

**Table 5-8 Cost-benefit assessment of emergency management planning**

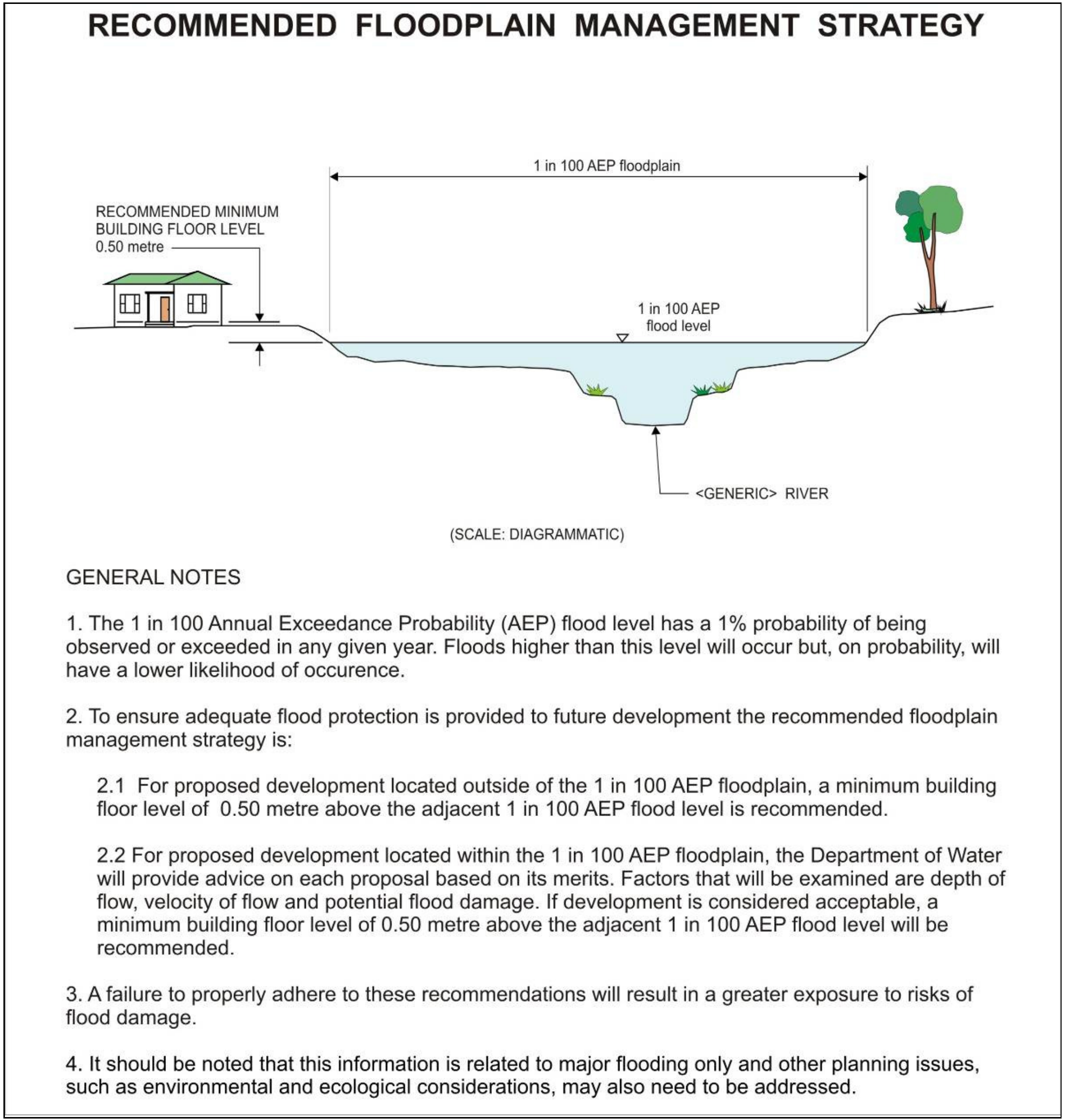
| Criteria | Assessment |
|---|---|
| **Flood risk management** | Effective at managing residual risk and aimed at preventing harm or loss of life to people. Should not be solely relied upon to manage existing or future flood risk – other controls should also be employed. |
| **Environmental** | No effect expected. |

| | |
|---|---|
| **Social** | Aims to prevent harm and loss of life to people, gives the community time to prepare for floods and minimise property damage, likely to provide positive social benefits overall. |
| **Economic** | Emergency plans take time to develop but may not involve significant additional expenses. The installation and monitoring of flood warning systems, such as stream gauges or real-time systems would require investment from the Shire of Brookton to install and monitor. Communication of flood awareness issues should be cost effective to implement. |
| **Overall** | Must be considered to manage the residual flood risk to the community. |

## 5.5 Recommended Floodplain Development Strategy

The recommended Brookton FDS incorporates the following strategies:

- Proposed development located outside of the 1 in 100 AEP floodplain (refer to Figure 4.3) is considered acceptable with regard to major flooding;
- Proposed development that is located within the 1 in 100 AEP floodplain should be assessed based on its merit. Some of the factors that should be considered include depth of flooding, velocity of flow, obstruction to major flows, potential flood damages, regional benefits and difficulties with evacuation; and
- Should development be considered acceptable a minimum habitable floor level of 0.50 metres above the appropriate 1 in 100 AEP flood level is recommended to ensure adequate flood protection.
- Culverts and drains should be regularly cleared of debris and sediment to ensure that their flow capacity is not reduced. The main drain should be inspected to ensure that there are no issues that may affect its performance.
- Flood risks should be communicated to residents of affected properties, particularly with regards to evacuation routes and the dangers associated with driving or wading through floodwaters.



## RECOMMENDED FLOODPLAIN MANAGEMENT STRATEGY

GENERAL NOTES

1. The 1 in 100 Annual Exceedance Probability (AEP) flood level has a 1% probability of being observed or exceeded in any given year. Floods higher than this level will occur but, on probability, will have a lower likelihood of occurence.

2. To ensure adequate flood protection is provided to future development the recommended floodplain management strategy is:

   2.1 For proposed development located outside of the 1 in 100 AEP floodplain, a minimum building floor level of 0.50 metre above the adjacent 1 in 100 AEP flood level is recommended.

   2.2 For proposed development located within the 1 in 100 AEP floodplain, the Department of Water will provide advice on each proposal based on its merits. Factors that will be examined are depth of flow, velocity of flow and potential flood damage. If development is considered acceptable, a minimum building floor level of 0.50 metre above the adjacent 1 in 100 AEP flood level will be recommended.

3. A failure to properly adhere to these recommendations will result in a greater exposure to risks of flood damage.

4. It should be noted that this information is related to major flooding only and other planning issues, such as environmental and ecological considerations, may also need to be addressed.

**Figure 5-3 Recommended floodplain development strategy for proposed development in Brookton**

# 6. RECOMMENDATIONS

This report outlines hydrologic and hydraulic modeling undertaken to better understand existing surface water flow regimes for the Avon River South Branch in the vicinity of the town of Brookton. It also outlines the recommended floodplain development strategy for the Shire of Brookton to manage flood risk in the town.

The following recommendations are made to the Shire of Brookton:

1. Implement the floodplain development strategy, specifically the strategies outlined in Section 5.5.
2. Give consideration to the non-structural controls available to the Shire, including:
    - Zoning controls;
    - Setting minimum floor levels; and
    - Emergency management planning.
3. Give consideration to the structural controls available to the Shire, including:
    - Clearing culverts of debris;
    - Clearing the main drain of debris and inspecting it to identify any required improvements; and
    - Removing weeds and invasive species from the river channel and floodplain.

   Note that these measures are expected to address local flooding only and make only marginal reductions to flood hazard from the Avon River South Branch.
4. Consider modelling the PMF event so that a worst-case scenario of flood hazard is understood (or accept that events larger than the 1 in 500 AEP are possible). Effective flood emergency plans can then be developed by the Shire using this information.
5. Communicate flood awareness to the community, particularly with regards to evacuation routes and the dangers associated with driving or wading through floodwaters.

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# Appendix 1 - Flood Frequency Analysis

# A1. FLOOD FREQUENCY ANALYSIS

Flood frequency analysis (FFA) is a technique that makes use of information on past floods to estimate the magnitude of future floods of a selected probability of exceedance. “Flood frequency analysis is usually carried out using data at stream gauges and makes use of historical streamflow and flood data” (Ladson, 2008).

There is no streamflow gauging station on the Avon River South Branch, so data from the nearby Dale River South gauge (615222) was used to estimate design flood flow rates for that catchment using FFA. The Dale River South catchment is similar in area and other catchment characteristics (such as slope, mainstream length, percentage cleared) to the Avon River South Branch catchment (Figure A1-1and Table A1-1). The Dale River South gauging station recorded streamflow information for 33 years, between 1966 and 1999.

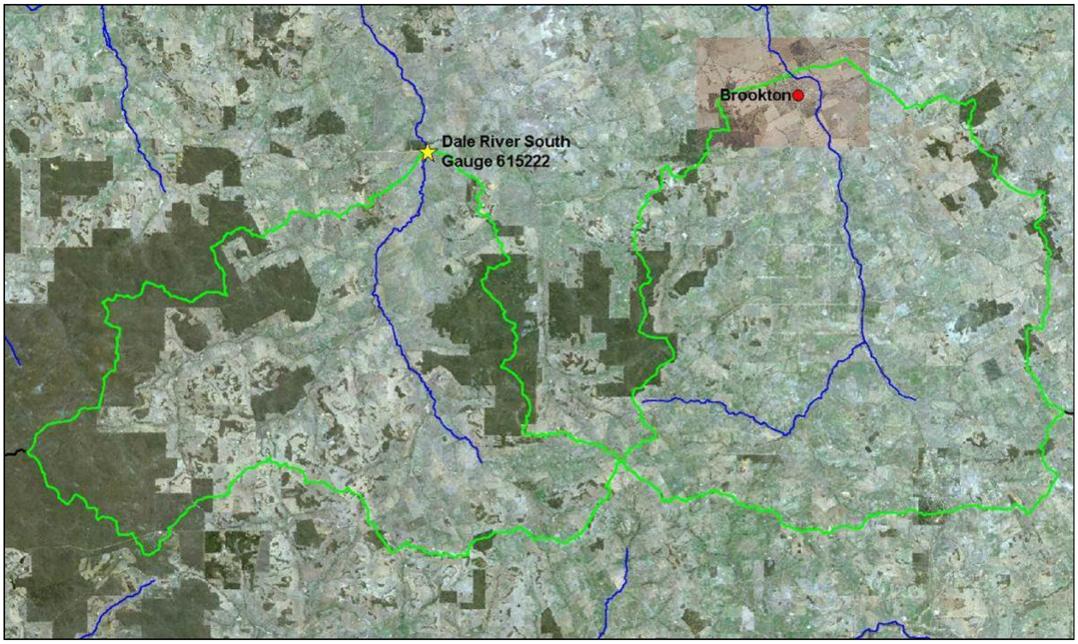

**Figure A1-1 Avon River South Branch and Dale River South catchments**

**Table A1-1 Avon River South Branch and Dale River South catchment details**

| Location | Catchment Area (km²) | Centroid Latitude (°S) | Centroid Longitude (°E) | Stream Length (km) | Equal Area Slope (m/km) | Annual Rainfall (mm) |
|---|---|---|---|---|---|---|
| Avon River South Branch* | 330 | 32.46 | 117.04 | 25.7 | 2.3 | 450 |
| Dale River South | 286 | 32.48 | 116.78 | 32.0 | 3.3 | 466 |

* The catchment outlet is defined by the location of the hydrograph input to the hydraulic model, approximately 1km south of the Brookton-Kweda Road bridge.

Maximum recorded daily flows at the Dale River South streamflow gauging station were used to derive both annual and partial series for flood frequency analysis. The analysis fits a mathematical distribution to the historical flows to determine the likelihood of exceeding various magnitude flood events at the site. The likelihood is defined as the annual exceedance probability (AEP), which is the chance that a flood of a given size or larger will occur in any given year; for example a 1 in 100 AEP event has a 1% chance of occurring this year, while a 1 in 500 AEP event has a 0.2% chance of occurring.

## A1.1 Annual Series

The annual peak flow series is a list of the largest flow events in each water year that has occurred in the period of record at a stream gauging station (Ladson, 2008). For the purposes of this study the water year has been taken as 1 January to 31 December. Annual series data for the Dale River South streamflow gauging station are summarised in Table A1-3. The Dale River South dataset has 32 full years of data from 1967 to 1998. Figure A1-2 shows the annual series in both chronological and ascending order.

### Table A1-3 Annual series data for Dale River South

| Dale River South Gauging Station 615222 | | | | | |
|---|---|---|---|---|---|
| Year | Max Annual Flow ($m^3/s$) | Year | Max Annual Flow ($m^3/s$) | Year | Max Annual Flow ($m^3/s$) |
| 1967 | 31.4 | 1978 | 31.4 | 1989 | 1.9 |
| 1968 | 11.3 | 1979 | 11.3 | 1990 | 33.4 |
| 1969 | 1.5 | 1980 | 1.5 | 1991 | 14.0 |
| 1970 | 8.0 | 1981 | 8.0 | 1992 | 8.1 |
| 1971 | 4.4 | 1982 | 4.4 | 1993 | 16.4 |
| 1972 | 1.6 | 1983 | 1.6 | 1994 | 16.9 |
| 1973 | 12.4 | 1984 | 12.4 | 1995 | 14.4 |
| 1974 | 24.0 | 1985 | 24.0 | 1996 | 20.4 |
| 1975 | 6.6 | 1986 | 6.6 | 1997 | 3.7 |
| 1976 | 3.8 | 1987 | 3.8 | 1998 | 21.7 |
| 1977 | 2.9 | 1988 | 2.9 | | |

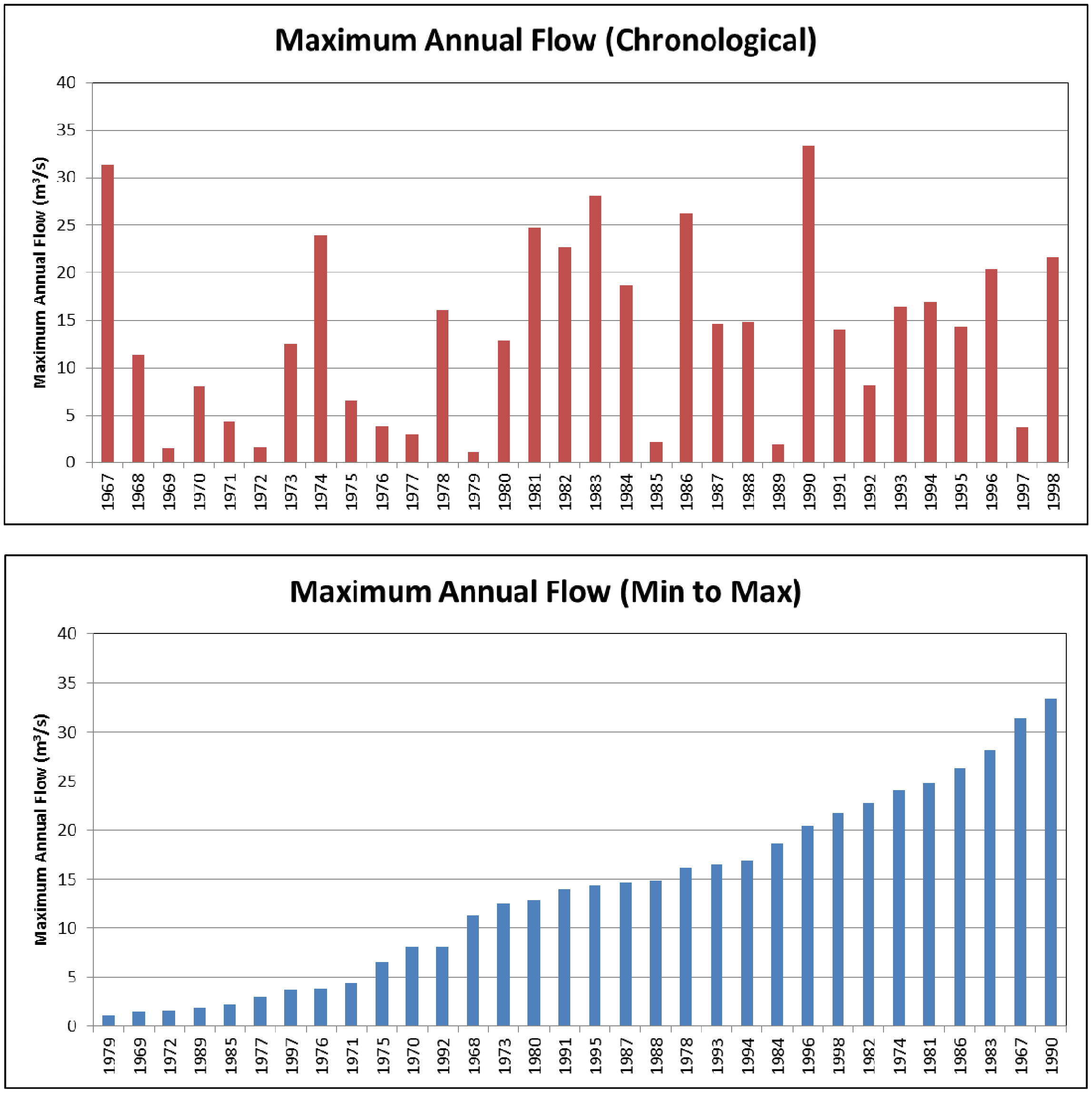

**Figure A1-2 Annual series data plot for Dale River South**

Annual series FFA for the Dale River South catchment was undertaken using the HydroFreq and Aquapak software packages, and several probability distributions were fitted to the data. The (Generalised Extreme Value) probability distribution was determined to give the best estimates for the various design flood events, which are presented in Figure A1-3 and Table A1-4.

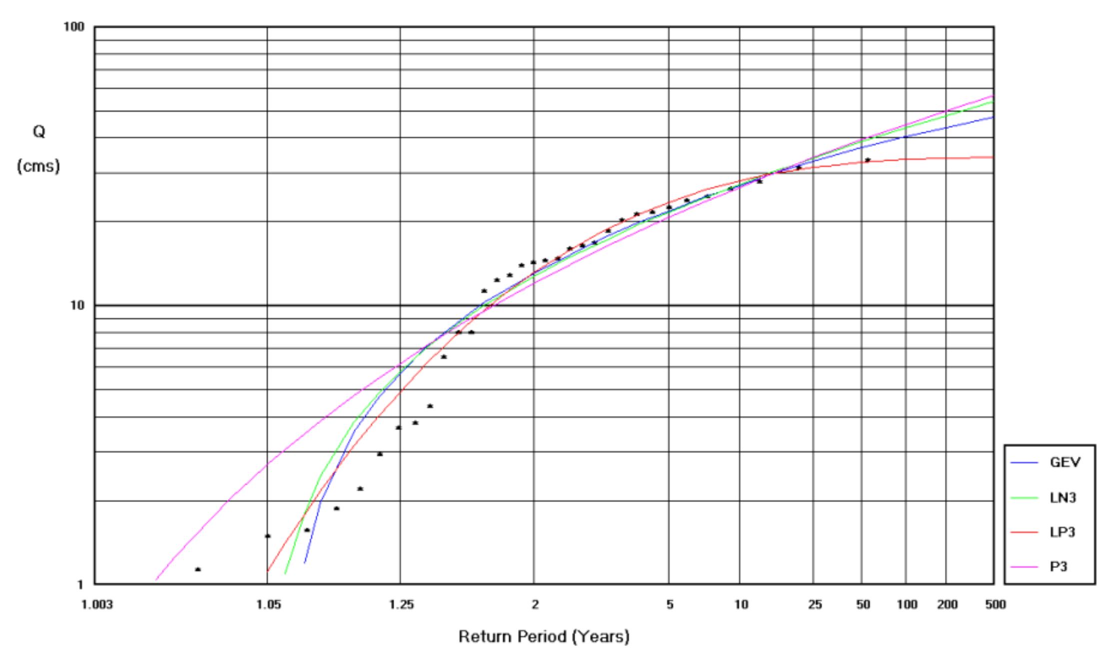

Figure A1-3 Annual series flood frequency plot for Dale River South

**Table A1-4 Annual series flood frequency analysis results for Dale River South**

| Location | Q2 ($m^3/s$) | Q5 ($m^3/s$) | Q10 ($m^3/s$) | Q20 ($m^3/s$) | Q50 ($m^3/s$) | Q100 ($m^3/s$) | Q500 ($m^3/s$) |
|---|---|---|---|---|---|---|---|
| Dale River South | 13 | 22 | 27 | 32 | 37 | 41 | 49 |

## A1.2 Partial series flood frequency analysis

"The partial flood series is a list of all the independent floods that have occurred in the period of record at a stream gauging station that are above a specified threshold value… If *n* is the number of years of record, and *k* is the number of independent flood peaks above a threshold, then the threshold should be chosen so that *k* is between *n* and *3n*" (Ladson, 2008).

Partial series flood frequency analysis for the Dale River South catchment was undertaken using the Aquapak software package. Aquapak allows the user to specify whether the partial series is selected using a threshold flow, or a nominated number of independent flood peaks, *k*.

Since partial series allows multiple flood events to be used from a single water year, it is important that only independent flood events are considered. The user is required to specify the minimum number of intervals (days) between independent events, and the receding flow value between independent events (*k%*, where the value between two independent events must be less than *k%* of the smaller peak).

The threshold flow was selected to be 10 m$^3$/s, which gave 49 independent flood peaks based on the independence criteria of a minimum of one day between events with the flow less than 50% of the adjacent peak. Figure A1.4 shows the peaks included in the partial series analysis. A Log Pearson III distribution was fitted to the data using the methods listed in Book IV of ARR (Equations 2.15 and 2.17). Figure A1.5 shows the results of the partial series flood frequency analysis with 5% and 95% confidence intervals.

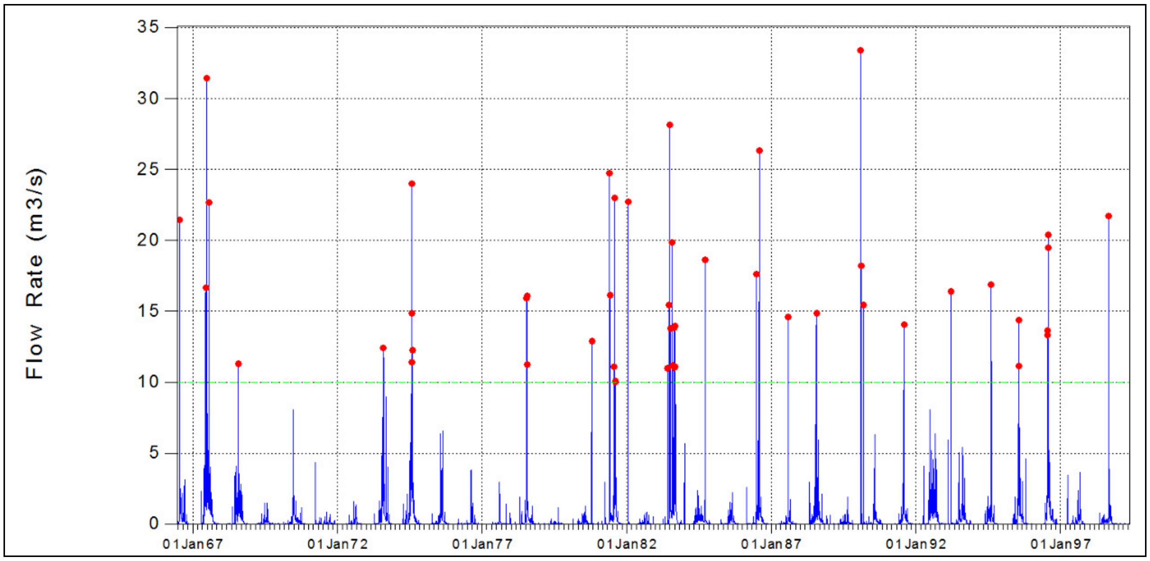

**Figure A1-4 Peaks over threshold for Dale River South partial series**

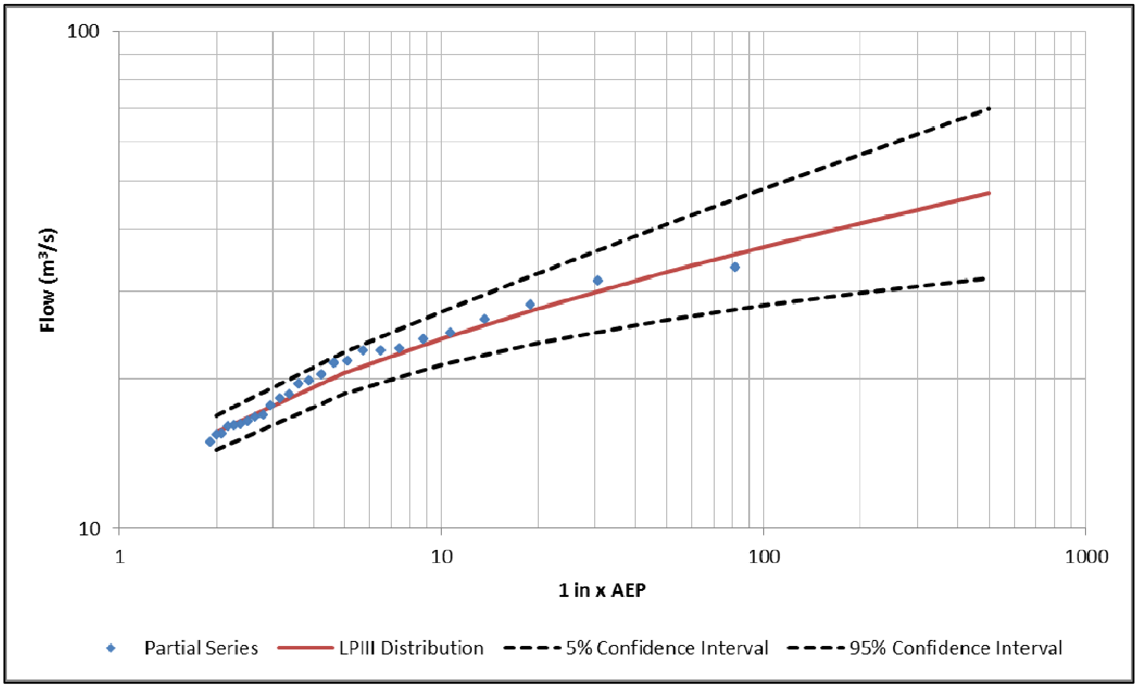

**Figure A1-5 Partial series flood frequency plot for Dale River South**

**Table A1-5 Partial series flood frequency analysis results for Dale River South**

| Location | Q2 ($m^3/s$) | Q5 ($m^3/s$) | Q10 ($m^3/s$) | Q20 ($m^3/s$) | Q50 ($m^3/s$) | Q100 ($m^3/s$) | Q500 ($m^3/s$) |
|---|---|---|---|---|---|---|---|
| Dale River South | 16 | 21 | 24 | 28 | 33 | 37 | 47 |

## A1.3 Discussion

One can have confidence in the results for the 1 in 2 and 1 in 5 AEP events due to the number of years of available data, but larger events must be determined from extrapolation of the fitted curve. The results are unexpectedly low for events greater than the 1 in 10 AEP event, particularly the less frequent events such as the 1 in 100 and 500 AEP events. This is due to the fact that, by chance, very few medium to large size floods occurred during the period that data is available for Dale River South. If the gauged record included wetter periods prior to 1966, such as the 1950s, the results would likely have been larger.

The data from nearby catchments, such as the Hotham River and the Murray River, shows that the three largest flood events since 1950 all occurred prior to data being collected at

Dale River South (1966). The estimated 1 in 25 AEP flow for Hotham River in the Boddington flood study (SKM, 2009) using flood frequency analysis was 315 $m^3/s$, which has not been exceeded at all since 1965 (The 1 in 20 AEP event is approximately 290 $m^3/s$ and has not been exceeded either over that period). It is reasonable to assume that the 1 in 20/25 AEP event has not occurred at Dale River since 1966 either. If that is the case, flood frequency analysis on the available data for Dale River is going to give greatly underestimated peak flow rates.

Little confidence can be placed in the flood frequency analysis for Dale River South due to the lack of gauge data prior to 1966. However the partial series results were determined to be the more suitable of the two series and were selected as the estimate of design flows at Dale River South based on the available data.

# Appendix 2 - RORB Rainfall Runoff Modelling

## A2. RORB RAINFALL RUNOFF MODELLING

A RORB model was developed for the Avon River South Branch catchment to estimate the flow rate of the river following a range of design rainfall events. Flow hydrographs generated by the RORB model were used as inflows to the 2-dimensional TUFLOW hydraulic model for the river.

RORB is described in the User Manual (Laurenson, Mein and Nathan, 2010) as follows:

> *"RORB is a general rainfall runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall-excess and routes this through catchment storage to produce runoff hydrographs at any location.*
>
> *The program requires a datafile to describe the particular features of the stream network being modelled and is run interactively. It can be used both for the calculation of design hydrographs and for model calibration by fitting to rainfall and runoff data of recorded events.*
>
> *The model is areally distributed, nonlinear, and applicable to both urban and rural catchments. It makes provision for temporal and areal variation of rainfall and losses and can model flows at any number of gauging stations. In addition to normal channel storage, specific modeling can be provided for retarding basins, storage reservoirs, lakes or large flood plain storages. Base flow and other channel inflow and outflow processes, both concentrated and distributed, can be modelled".*

### A2.1 Catchment Delineation

The study area modelled in RORB was mapped using GIS software (MapInfo). Initially, a model domain was selected for the 2D hydraulic model of the Avon River South Branch. This was examined to determine key flow input locations. These were selected at or near the boundaries of the 2D model domain where significant watercourses entered. A total of three inflow locations were identified; the main Avon River South Branch, a watercourse immediately downstream of the Brookton-Kweda Road bridge and a watercourse downstream of the Brookton Highway bridge. Only the main Avon River South Branch was included in the RORB model to avoid instability issues associated with including much smaller sub-catchments that would be required for the two smaller watercourses.

The main watercourses linking the notional outlets from the model to the inflow points were identified and extended through the upstream, external sub-catchments. Significant tributary watercourses were also identified and mapped. This directional tree structure network then formed a basis for further subdivision of the associated sub-catchments. A total of 8 sub-catchments were identified.

Figure A2-1 shows the RORB model network and sub-catchment delineation. Triangles denote catchment centroids and circles denote junction nodes. A summary of the model sub-catchments and reaches is included in Table A2-1. The junction of reaches 11 and 12 is the location where the Avon River South Branch enters the 2D model domain. All of the reaches in this model have been defined as natural reaches. No impervious fractions were defined except for Catchment C, which incorporates the town of Pingelly and was assigned an impervious fraction of 5%.

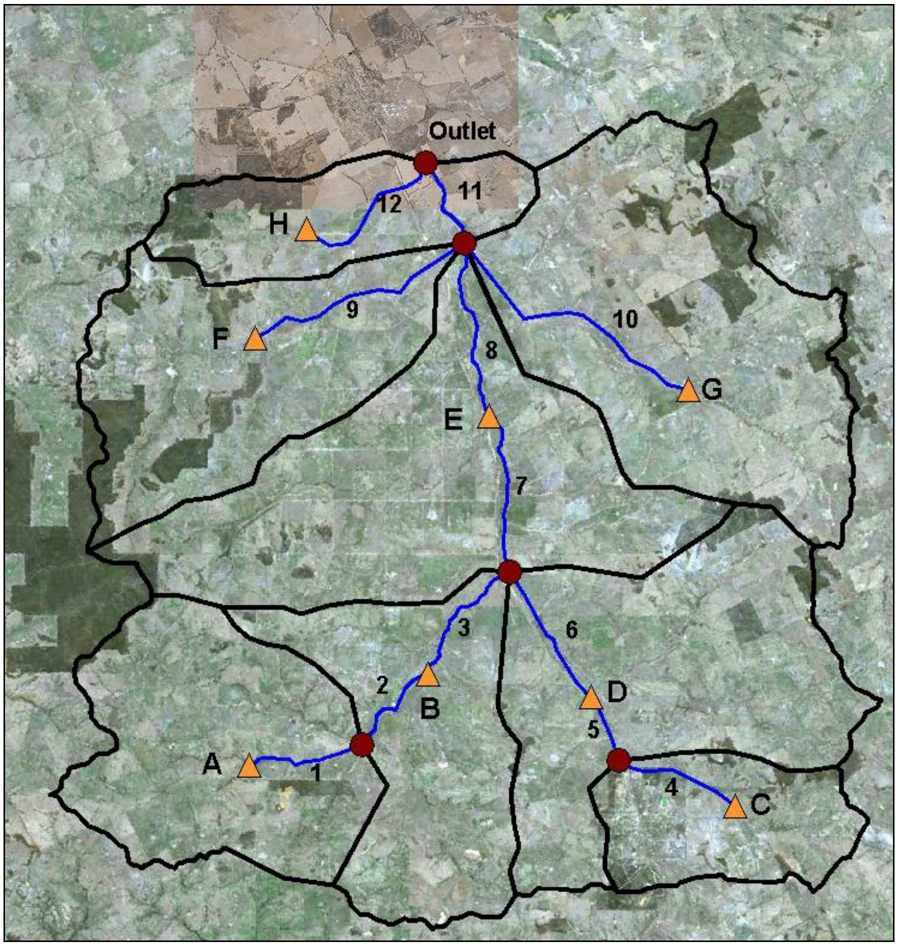

**Figure A2-1 Avon River South Branch catchment plan and RORB model**

**Table A2-1 RORB sub-catchment and reach properties**

| Sub-catchment | | | Reach | | | |
|---|---|---|---|---|---|---|
| **No.** | **Name** | **Area (km²)** | **No.** | **Length (km)** | **No.** | **Length (km)** |
| 1 | A | 40.2 | 1 | 3.05 | 9 | 6.04 |
| 2 | B | 34.9 | 2 | 2.72 | 10 | 7.27 |
| 3 | C | 19.1 | 3 | 3.66 | 11 | 2.4 |
| 4 | D | 51.3 | 4 | 3.31 | 12 | 4.2 |
| 5 | E | 61.0 | 5 | 1.74 | | |
| 6 | F | 41.1 | 6 | 3.8 | | |
| 7 | G | 60.9 | 7 | 4.19 | | |
| 8 | H | 21.5 | 8 | 4.69 | | |

## A2.2 Design Rainfall

Intensity-Frequency-Duration (IFD) data for the Avon River South Branch catchment was obtained from the Bureau of Meteorology website using the coordinates of the catchment centroid (32.46 ˚S, 117.04 ˚E). The IFD data is presented in terms of total rainfall in Table A2-2. Areal reduction factors were later applied to the raw depths and temporal patterns were fitted to the data as per ARR methods. The recently updated IFD data was not used in this project because many of the methods that use this data have not yet been updated for use with the new IFD data. The new IFD data is generally higher than the previous data, with the 6 and 12 hour events on average 9% higher. The 6 hour 1 in 100 AEP rainfall event is 17% higher than the previous IFD data. The sensitivity analysis conducted on the two-dimensional modelling results includes varying inflows by ±20%.

The design rainfall temporal patterns from ARR were applied to the model. These were generated directly by the RORB program based on the location of the catchment. RORB includes an option to filter the temporal patterns to smooth out any rare sub-durations that can be embedded in the design bursts.

**Table A2-2 IFD data for Avon River South Branch catchment**

| | Rainfall total for each Average Recurrence Interval (mm) | | | | | | |
|---|---|---|---|---|---|---|---|
| **Storm Duration** | **1 year** | **2 years** | **5 years** | **10 years** | **20 years** | **50 years** | **100 years** |
| 5 mins | 3.4 | 4.6 | 6.6 | 8.1 | 10.1 | 13.3 | 16.2 |
| 6 mins | 3.8 | 5.2 | 7.3 | 9.0 | 11.3 | 14.8 | 17.9 |
| 10 mins | 5.1 | 6.8 | 9.5 | 11.6 | 14.3 | 18.7 | 22.5 |
| 20 mins | 7.0 | 9.4 | 12.6 | 15.1 | 18.4 | 23.4 | 27.8 |
| 30 mins | 8.3 | 11.0 | 14.6 | 17.3 | 20.9 | 26.3 | 31.0 |
| 1 hr | 10.8 | 14.2 | 18.5 | 21.5 | 25.8 | 32.0 | 37.3 |
| 2 hrs | 14.0 | 18.3 | 23.6 | 27.2 | 32.2 | 39.8 | 46.0 |
| 3 hrs | 16.3 | 21.3 | 27.2 | 31.2 | 37.2 | 45.6 | 52.8 |
| 6 hrs | 21.2 | 27.6 | 35.1 | 40.3 | 47.6 | 58.2 | 67.2 |
| 12 hrs | 27.2 | 35.5 | 45.0 | 51.6 | 61.0 | 74.4 | 85.7 |
| 24 hrs | 34.1 | 44.4 | 56.4 | 64.6 | 76.3 | 93.1 | 108 |
| 48 hrs | 40.7 | 53.3 | 67.7 | 77.8 | 92.2 | 113 | 131 |
| 72 hrs | 44.0 | 57.7 | 73.4 | 85.0 | 101 | 125 | 145 |

### A2.1.1 Areal Reduction Factors

Areal reduction factors were applied to all of the RORB sub-catchments based on the total area of the catchment, the storm frequency and duration in accordance with methods detailed in ARR. The areal reduction factor ranged from 0.91 to 0.92 for the critical duration storms for the design events (1 in 2 to 1 in 500 AEP).

## A2.2 RORB Parameters

The selected peak flow estimates for the 1 in 2, 5, 10, 20, 50, 100 and 500 AEP events were used for calibration of the model. In the RORB model there are four principal parameters that are generally used for calibration:

1. The principal calibration parameter is the coefficient $k_c$, an empirical parameter applicable to the catchment and stream network. The product of $k_c$ and the relative delay coefficient $k_r$ forms the storage coefficient k.

The relative delay coefficient $k_r$ is a dimensionless ratio applicable to an individual reach storage and is calculated in the RORB program.

2. The storage exponent m is a parameter that is usually determined in fit calibration runs using observed data. The m parameter is subsidiary to $k_c$. m is a measure of the catchment's non-linearity. A value of 0.8 is recommended as a first trial value and for use on ungauged catchments as is the case with the Avon River South Branch.

The other parameters relate to the rainfall loss model selected for use in the calibration. RORB is structured to use one of two loss models to determine rainfall excess: either an initial loss (IL), constant rate continuing loss (CL) model; or an initial loss (IL), runoff coefficient (RoC) model. This latter model is equivalent to an initial loss, proportional loss (or constant fraction) model. Thus the two loss parameters used in calibration are:

3. The initial loss depth (IL), mm; and
4. Either a continuing constant loss rate (CL), mm/hr; or a runoff coefficient (RoC), which is complementary to a proportional (or constant fraction) continuing loss.

The initial values selected for these parameters were as follows:

- $k_c$ = 12.1, based on the equation below for the Wheatbelt (ARR);

$$k_c = \frac{1.06L^{0.87}}{S_e^{0.46}}$$

Where L = mainstream length (km)

$S_e$ = Equal area slope (m/km)

- m = 0.8, based on RORB Manual recommendation for ungauged catchments;
- IL = 30 mm (ARR); and
- RoC = 0.09 – 0.34 (Flavell, 2012).

To provide an initial estimate of critical duration, the time of concentration for the calibration location was estimated using several empirical formulae (Table A2-3) yielding values between 6 and 12 hours.

**Table A2-3 Time of concentration**

| Method | Time of Concentration |
|---|---|
| Wheatbelt Rational Method (ARR) | 6.9 hours |
| Bransby-Williams formula | 11.9 hours |
| Modified Friend's equation | 9.8 hours |
| US FAA method | 11.3 hours |

## A2.3 Calibration Process

Adequately detailed rainfall and streamflow information for historic flow events was not available to calibrate the RORB model. Consequently, the catchment linearity parameter (m) was set constant at 0.8 as recommended by the model developers and in ARR (1998).

**Table A2-4 Calibration flow rates for the Avon River South Branch**

| | Q2 ($m^3/s$) | Q5 ($m^3/s$) | Q10 ($m^3/s$) | Q20 ($m^3/s$) | Q50 ($m^3/s$) | Q100 ($m^3/s$) | Q500 ($m^3/s$) |
|---|---|---|---|---|---|---|---|
| Avon River South Branch | 16 | 27 | 57 | 82 | 128 | 186 | 565 |

### A2.3.1 Catchment parameter $k_c$

The catchment storage parameter (Kc) was calculated using the equation for the wheatbelt region of Western Australia from ARR (1998) as an initial estimate. The $k_c$ value was increased and decreased from the initial estimate to test its effect on the catchment response, however the initial value for $k_c$ was found to be the best fit for the catchment considering the results for all AEP events.

### A2.3.2 Initial Loss

The initial loss of 30 mm was deemed to be too high as it resulted in no flow at all from the catchment for a number of design storms for which streamflow was expected. Streamflow data from the Dale River South gauge indicates that streamflow can be generated by rainfall of 30 mm or less. The 7th highest flow event (23.0 $m^3/s$) over the 32 year record resulted from rainfall of 33 mm over a period of 4 hours. Another flow event (peak of 16.4 $m^3/s$) resulted from 24 mm of rainfall over a period of 3 hours. Rainfall occurring in winter will likely fall on a wet catchment and as such the initial loss before runoff is generated is likely to be smaller than 30 mm.

The Boddington Flood Study (SKM, 2009) found that the initial loss for pasture should be 10 mm for AEP events up to 25 years, and 5 mm for the 100 year event. During the calibration of their hydrologic model to streamflow data SKM obtained higher IL values for summer storms, however for the design events, which may occur in winter, they found that lower IL values were more appropriate.

For the Avon River Flood Study, it was found that an IL of 10 mm fitted the data for the 1 in 2, 5, 10 and 20 AEP events. For the larger storm events, the 1 in 50, 100 and 500 AEP events, it was found that the IL should be 5 mm. These values gave the best fit for the current data and are consistent with those of the SKM study.

### A2.3.3 Runoff Coefficients

Over the range of AEPs examined, the loss model based on runoff coefficient was found to give more consistent results than a continuing constant loss model. The runoff coefficients (RoC) for the RORB loss model were used to calibrate the peak flow rate for each design event to the required value. The initial values determined by Flavell (2012) in his study of streamflow data from 12 Wheatbelt catchments were selected as a starting point and minor adjustments were made to give the required peak flow rates. The final runoff coefficients chosen are presented in Table A2-5 together with Flavell's estimates and the effective overall runoff that includes the initial loss. The runoff coefficients and the overall runoff rate are presented in Figure A2-2.

**Table A2-5 Runoff coefficients**

| | Q2 ($m^3/s$) | Q5 ($m^3/s$) | Q10 ($m^3/s$) | Q20 ($m^3/s$) | Q50 ($m^3/s$) | Q100 ($m^3/s$) | Q500 ($m^3/s$) |
|---|---|---|---|---|---|---|---|
| Flavell (2012) | 0.09 | 0.11 | 0.16 | 0.21 | 0.28 | 0.34 | 0.62[1] |
| RORB model RoC | 0.12 | 0.14 | 0.22 | 0.26 | 0.28 | 0.33 | 0.62 |
| RORB model overall runoff | 0.08 | 0.10 | 0.17 | 0.21 | 0.25 | 0.30 | 0.59 |

[1] This value was estimated from log-log extrapolation of the data.

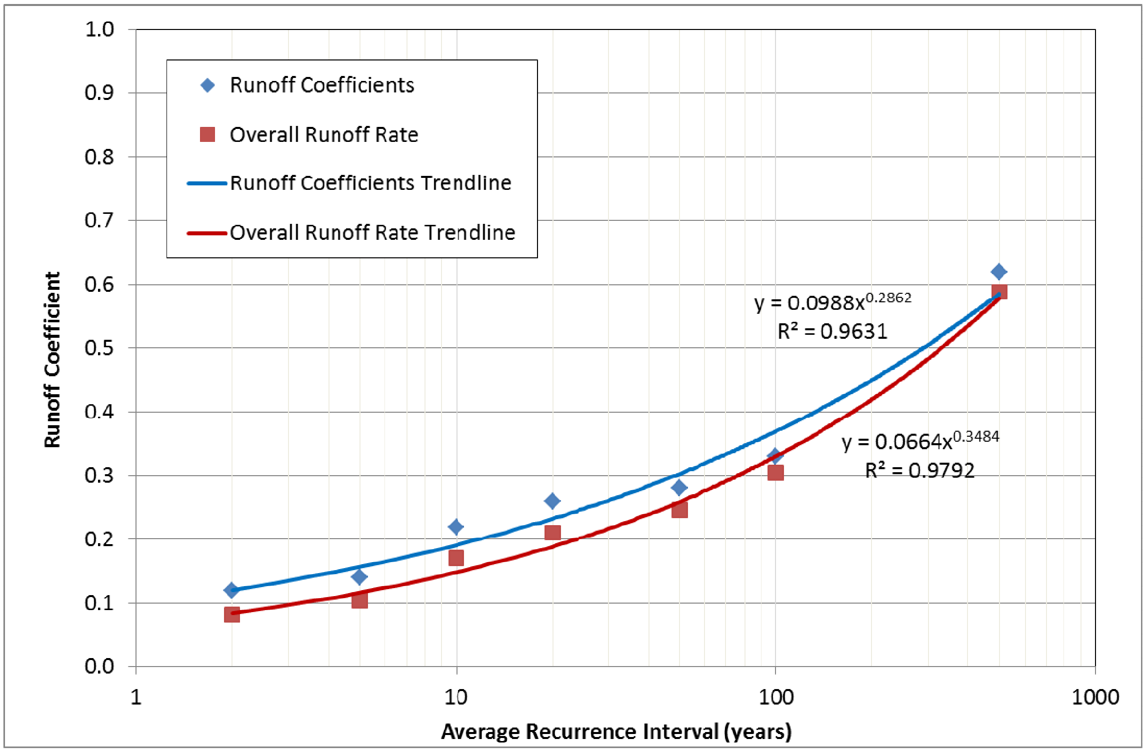

**Figure A2-2 Runoff coefficients and overall runoff rate for calibrated RORB model**

### A2.3.4 Critical Storm Duration

Once the calibration of the RORB model was complete, a check was made on the critical storm duration for each AEP event. The critical storm duration was determined to be 6 hours for the 1 in 50, 100 and 500 AEP events and 9 hours for the more frequent AEP events, for the calibration parameters selected. The results of several storm durations for the 1 in 100 AEP event are presented in Figure A2-3. For some AEP events the 30 and 36 hour events produced higher than expected peak flow rates. It is believed that these results are caused by the particular storm pattern from ARR (despite the filtering process that RORB undertakes) and that the critical storm duration lies in the 6 – 9 hour range.

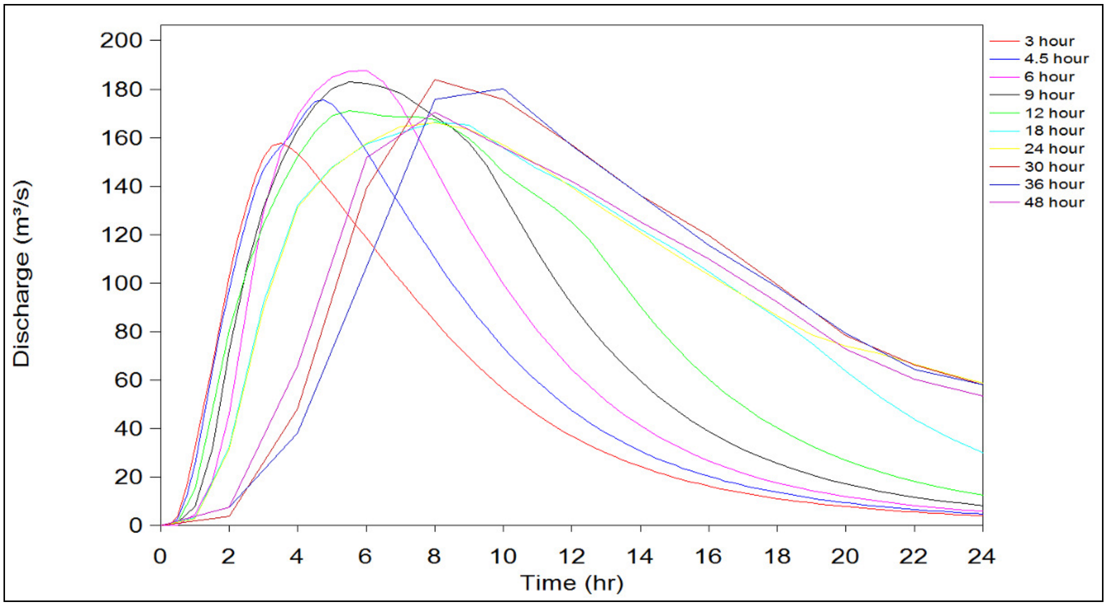

**Figure A2-3 RORB output for the 1 in 100 AEP event for various storm durations**

## A2.4 Rainfall-Runoff Model Outputs

The final calibration parameters and results of the RORB modelling are presented in Table A2-6. The hydrograph outputs for the Avon River South Branch, at the point 1 km upstream from the Brookton-Kweda Road bridge, are presented in Figure A2-4. These hydrographs were used as the inputs to the 2-dimensional hydraulic model.

Limited data were available with which to calibrate the RORB model, and the data that were available (peak flow rates for design events) were determined from equations rather than from flood frequency analysis. However, the fact that the RORB calibration parameters all closely match existing regional data or local data from neighbouring catchments, gives increased confidence in both the peak flow estimates and the hydrograph outputs of the RORB model.

**Table A2-6 – Avon River South Branch RORB model calibration results**

| **Average recurrence interval** | **2 years** | **5 years** | **10 years** | **20 years** | **50 years** | **100 years** | **500 years** |
|---|---|---|---|---|---|---|---|
| Target peak flow ($m^3/s$) | 16 | 27 | 57 | 82 | 128 | 186 | 565 |
| Modeled peak flow ($m^3/s$) | 17 | 28 | 56 | 83 | 131 | 188 | 566 |
| Critical design storm duration | 9 hr | 9 hr | 9 hr | 9 hr | 6 hr | 6 hr | 6 hr |
| Hydrograph time to peak (hrs) | 10 | 9.5 | 9 | 9 | 6.5 | 6.5 | 5.5 |
| Linearity parameter ‘m’ | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| Catchment parameter ‘$k_c$’ | 12.1 | 12.1 | 12.1 | 12.1 | 12.1 | 12.1 | 12.1 |
| Initial Loss (mm) | 10 | 10 | 10 | 10 | 5 | 5 | 5 |
| Runoff coefficient | 0.12 | 0.14 | 0.22 | 0.26 | 0.28 | 0.33 | 0.62 |

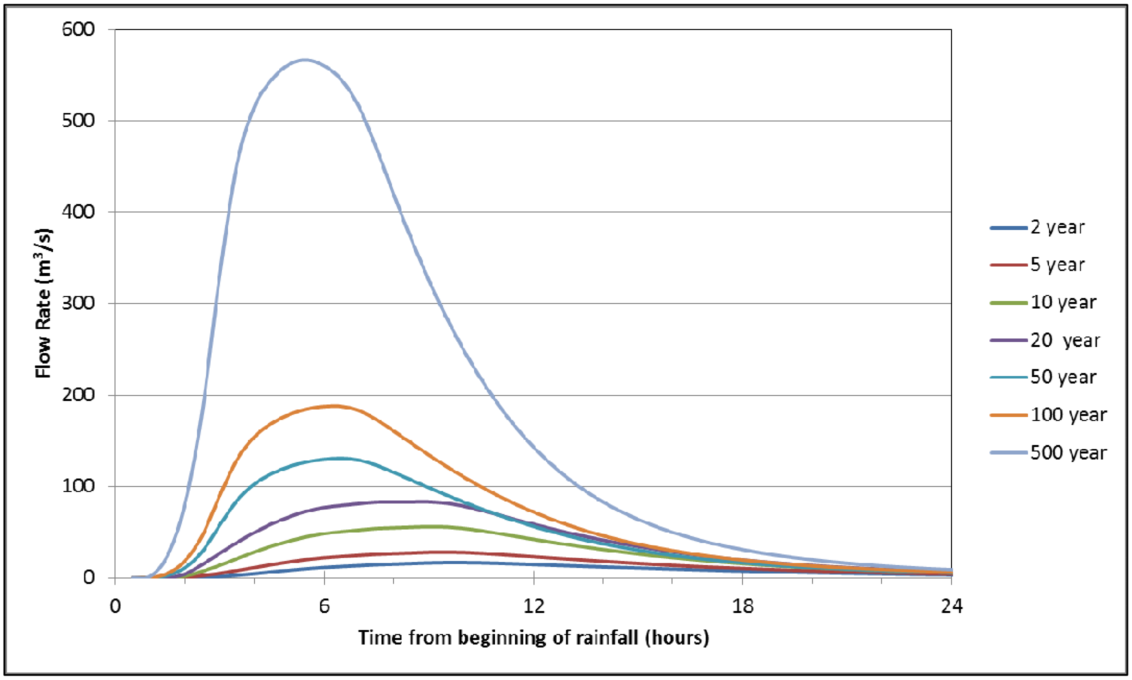

**Figure A2-4 RORB hydrograph outputs for Avon River South Branch**

# Appendix 3 - Ground Survey Results

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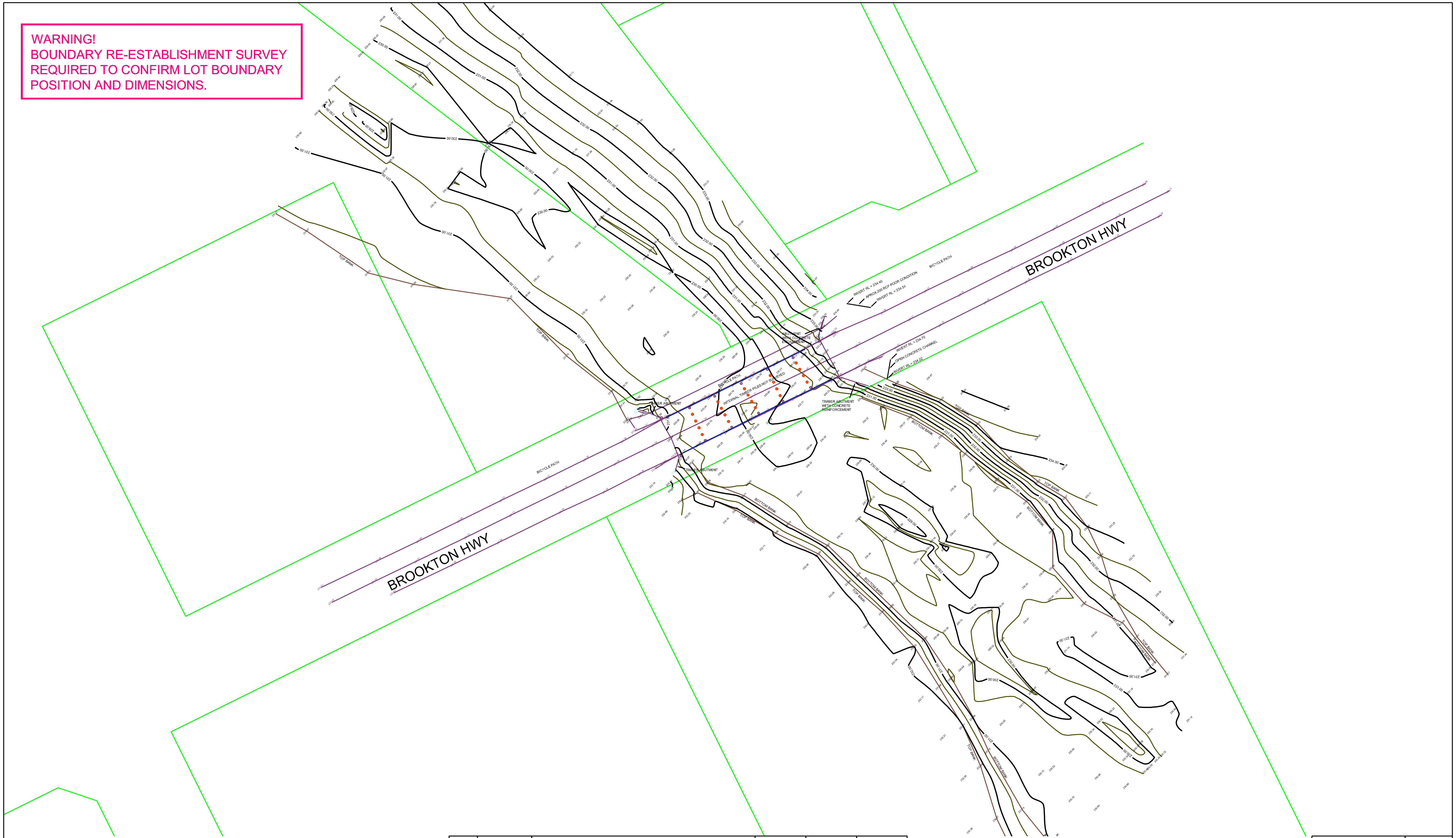

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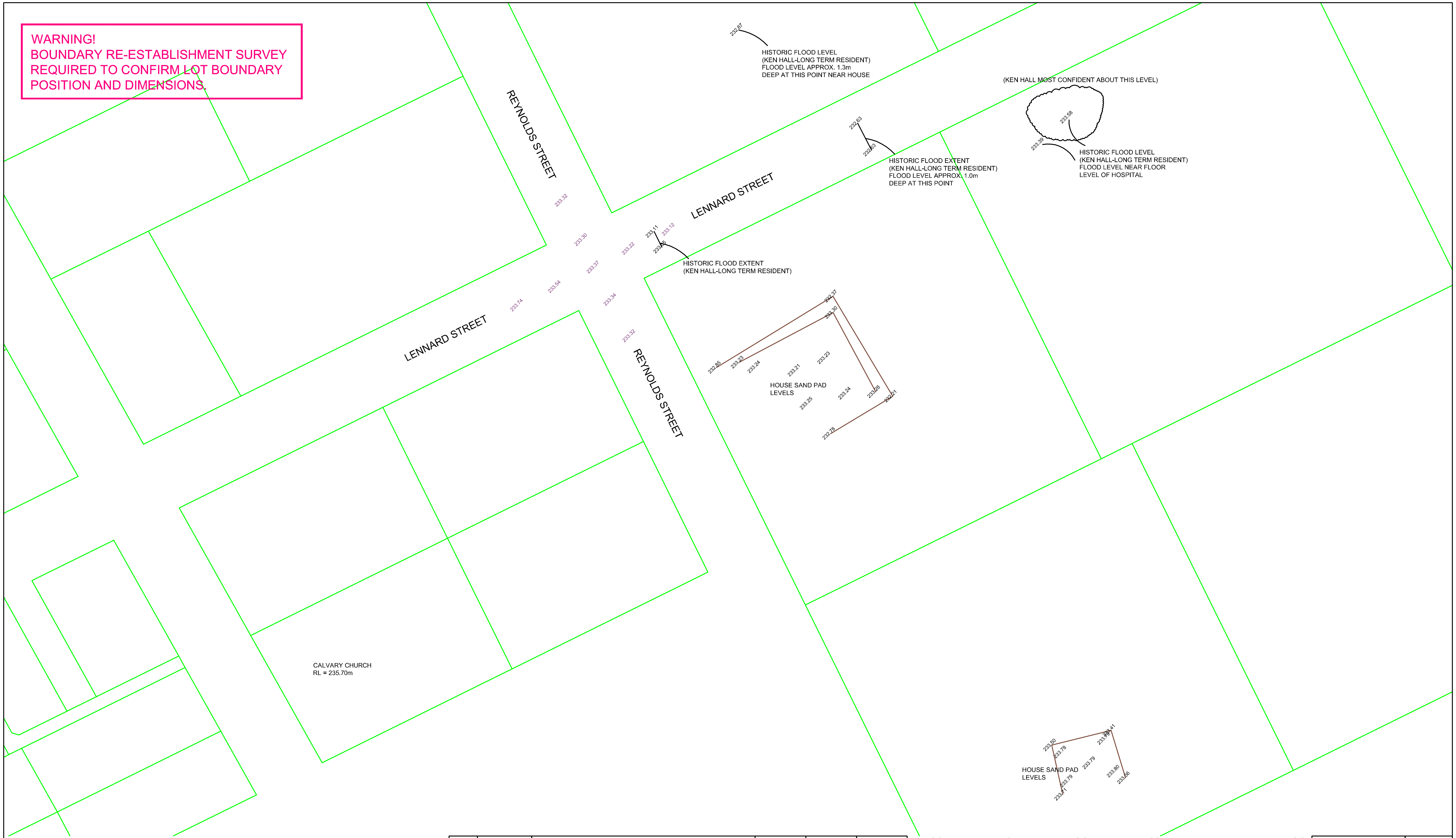

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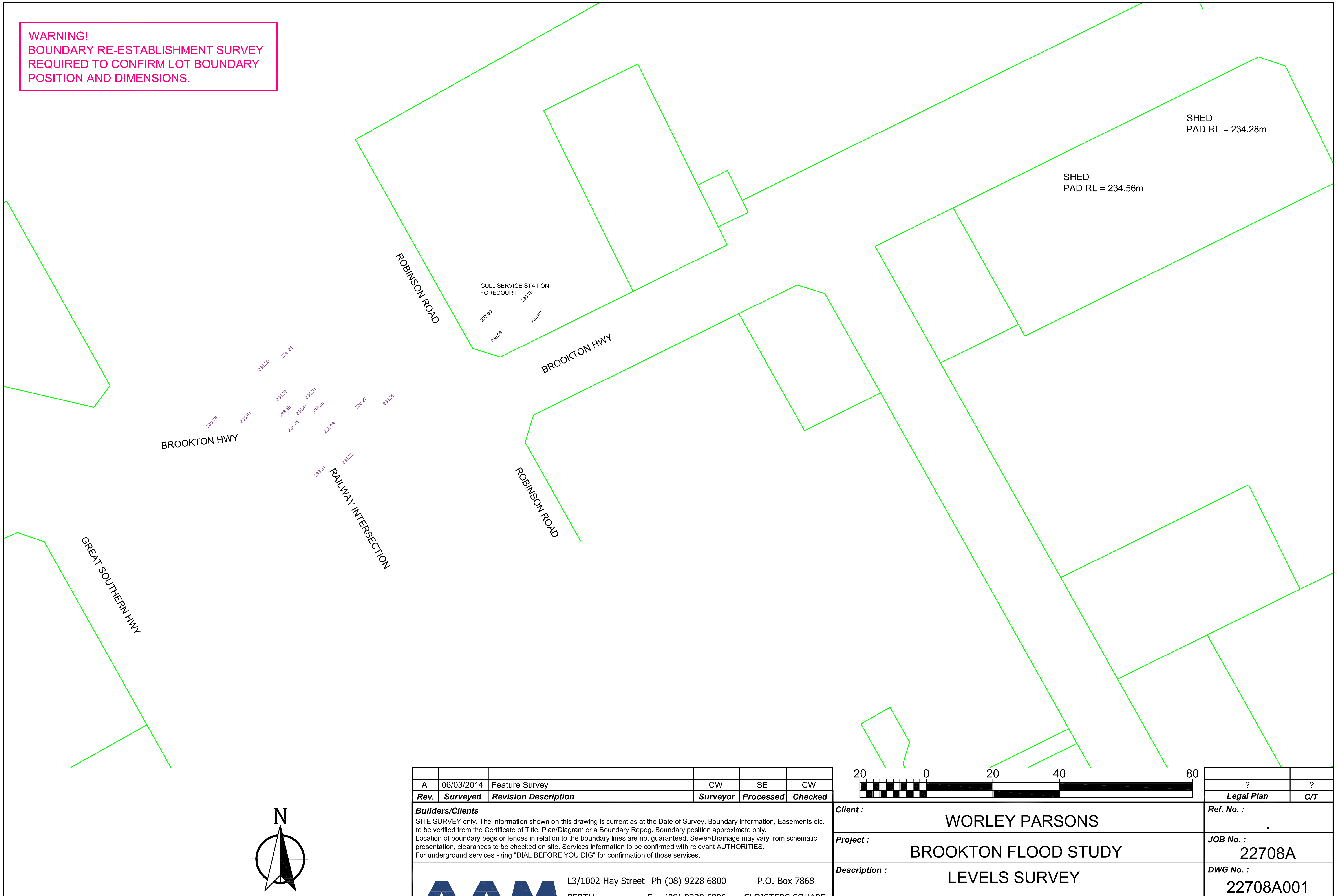

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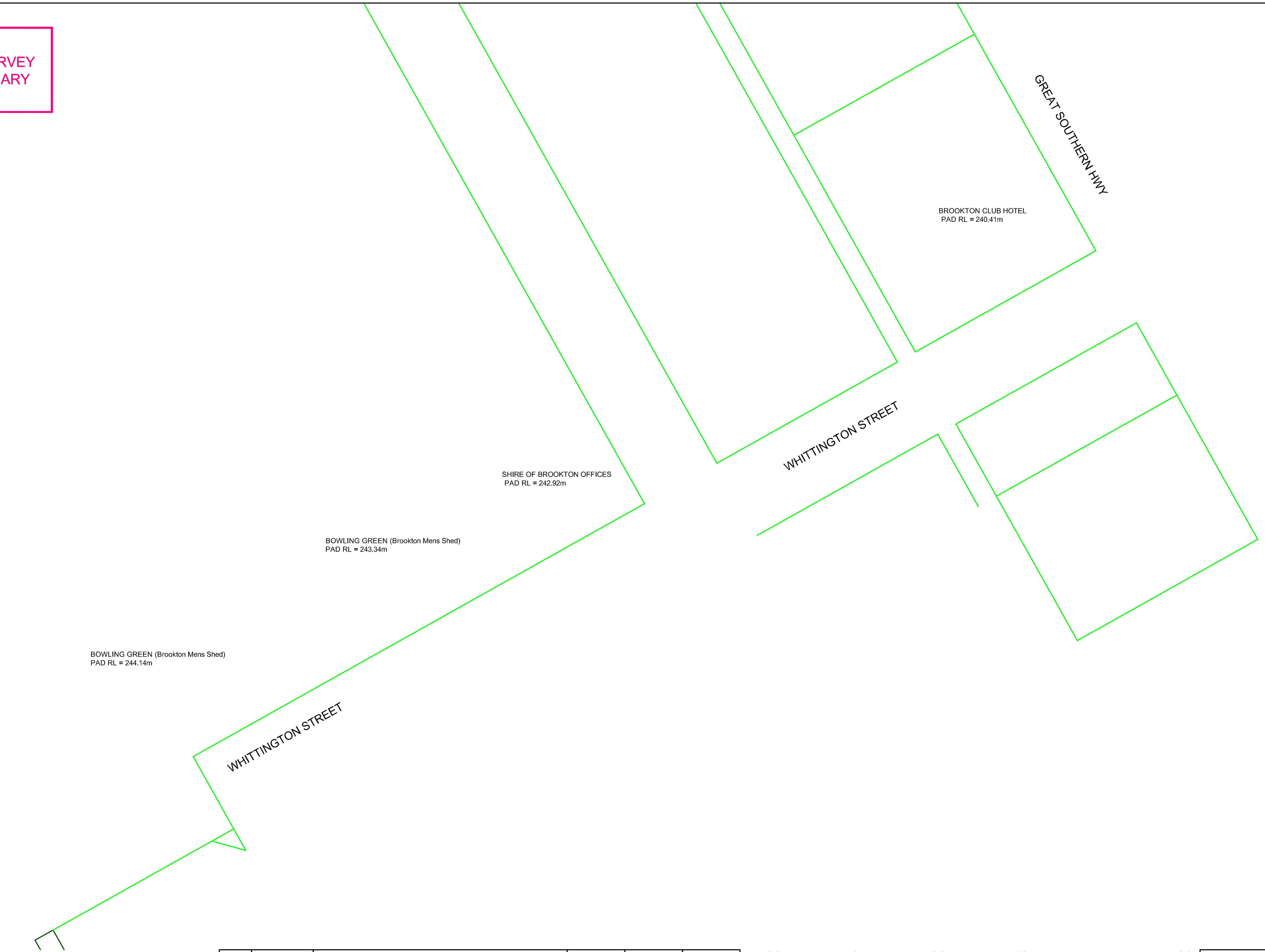

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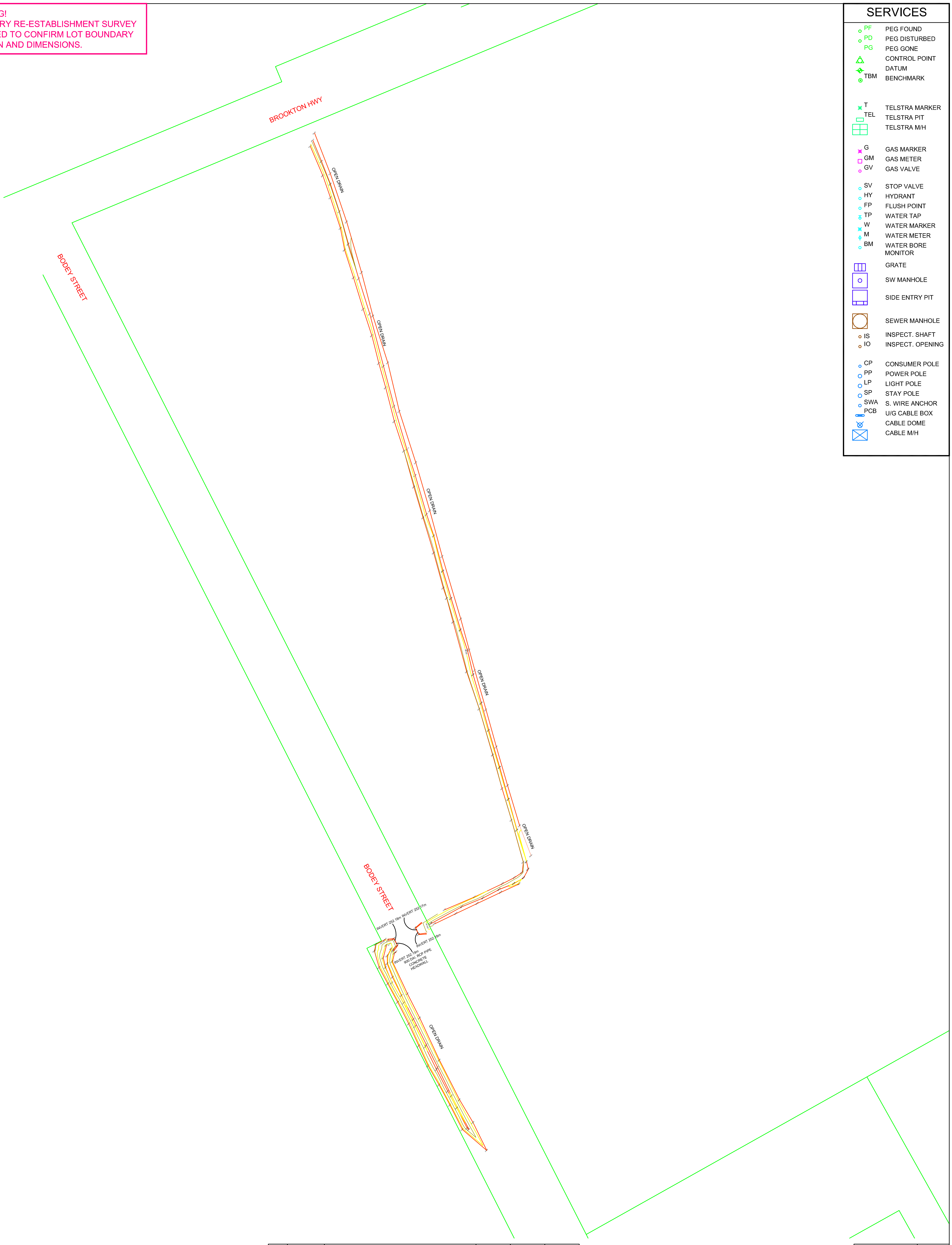
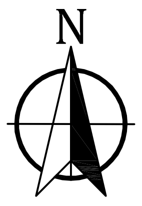


## SERVICES

| | |
|---|---|
| PF | PEG FOUND |
| PD | PEG DISTURBED |
| PG | PEG GONE |
| | CONTROL POINT |
| | DATUM |
| TBM | BENCHMARK |
| T | TELSTRA MARKER |
| TEL | TELSTRA PIT |
| | TELSTRA M/H |
| G | GAS MARKER |
| GM | GAS METER |
| GV | GAS VALVE |
| SV | STOP VALVE |
| HY | HYDRANT |
| FP | FLUSH POINT |
| TP | WATER TAP |
| W | WATER MARKER |
| M | WATER METER |
| BM | WATER BORE MONITOR |
| | GRATE |
| | SW MANHOLE |
| | SIDE ENTRY PIT |
| | SEWER MANHOLE |
| IS | INSPECT. SHAFT |
| IO | INSPECT. OPENING |
| CP | CONSUMER POLE |
| PP | POWER POLE |
| LP | LIGHT POLE |
| SP | STAY POLE |
| SWA | S. WIRE ANCHOR |
| PCB | U/G CABLE BOX |
| | CABLE DOME |
| | CABLE M/H |



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