# TAMWORTH CITY LEVEES
# INTERNAL DRAINAGE STUDY

## VOLUME 1
## MAIN REPORT AND APPENDICES

**February 2012**

DRAFT FOR CLIENT REVIEW

Prepared by:
**Lyall & Associates**
**Consulting Water Engineers**
Level 1, 26 Ridge Street
North Sydney NSW 2060

**Tel: (02) 9929 4466**
Fax: (02) 9929 4458
Email: lacewater@bigpond.com

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|---|---|---|
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## NOTE ON FLOOD FREQUENCY

The frequency of floods may be referred to in terms of their Average Recurrence Interval (ARI) or Annual Exceedance Probability (AEP). For example, for a flood having a 100 year ARI there will be a flood of equal or greater magnitude once in 100 years on the average. For a flood having a 1% AEP magnitude, there is a 1% probability that there will be floods of equal or greater magnitude each year. The approximate correspondence between these two systems is:

| ANNUAL EXCEEDANCE PROBABILITY (AEP) % | AVERAGE RECURRENCE INTERVAL (ARI) YEARS |
|---|---|
| 0.5 | 200 |
| 1 | 100 |
| 5 | 20 |
| 20 | 5 |
| 50 | 2 |

In this report floods are referred to in terms of their Average Recurrence Interval. Reference is also made in the report to the Probable Maximum Flood (PMF). This flood occurs as a result of the Probable Maximum Precipitation (PMP). The PMP is the result of the optimum combination of the available moisture in the atmosphere and the efficiency of the storm mechanism as regards rainfall production. The PMP is used to estimate PMF discharges using a model which simulates the conversion of rainfall to runoff. The PMF is defined as the limiting value of floods that could reasonably be expected to occur.

## ABBREVIATIONS

| | |
|---|---|
| AEP | Annual Exceedance Probability (%) |
| AHD | Australian Height Datum |
| ARI | Average Recurrence Interval (years) |
| BoM | Bureau of Meteorology |
| NoW | New South Wales Office of Water |
| OEH | Office of Environment and Heritage, Department of Premier and Cabinet (formerly Department of Environment, Climate Change and Water [DECCW], (formerly Department of Environment and Climate Change [DECC])) |
| TRC | Tamworth Regional Council |

**Chapter 9** of the report contains definitions of flood related terms used in the study.

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# TABLE OF CONTENTS

Page No.

**S1 SUMMARY AND RECOMMENDATIONS....S1**

**1 INTRODUCTION....1**

1.1 Background and Approach....1
1.2 Study Tasks....2
1.3 Overview of Report....3

**2 BACKGROUND INFORMATION....5**

2.1 Study Area....5
2.1.1. CBD Levee....5
2.1.2. Western Levee....6
2.1.3. Taminda Levee....6
2.2 Flood Gate Details....7
2.3 Community Consultation....10
2.4 Historic Flooding in Tamworth....11
2.4.1. Valley-Wide Flooding....11
2.4.2. Local Catchment Flooding....11
2.5 Previous Studies....12
2.5.1. Floodplain Management Study - City of Tamworth (PPK, 1993)....12
2.5.2. Hydraulic Modelling of Peel River at Tamworth (LACE, 2006)....13
2.5.3. Flood Protection Levees or Alternatives for Mathews Street to Thibault Street and Ebsworth Street West Tamworth (WP, 1998)....14

**3 HYDROLOGIC MODEL DEVELOPMENT AND TESTING....15**

3.1 Selection of Hydrologic Modelling Approach....15
3.2 Brief Review of DRAINS Modelling Approach....15
3.3 Brief Review of RAFTS Modelling Approach....17
3.4 Hydrologic Model Setup....18
3.5 Hydrologic Model Calibration....19
3.5.1. General....19
3.5.2. Hydrologic Model Parameters....19
3.5.3. Comparison of Peak Flow Estimates....20

**4 HYDRAULIC MODEL DEVELOPMENT....23**

4.1 Selection of Hydraulic Model....23
4.2 TUFLOW Modelling Approach....23
4.3 TUFLOW Model Setup....23
4.3.1. TUFLOW Model Structure....23
4.3.2. Model Boundary Conditions....24
4.3.3. Model Roughness....25

**5 DESIGN FLOOD ESTIMATION....27**

5.1 Design Storms....27
5.1.1. Rainfall Intensity....27
5.1.2. Areal Reduction Factors....27
5.1.3. Temporal Patterns....27
5.2 Probable Maximum Precipitation....27
5.3 Derivation of Design Flood Hydrographs....28

**6 HYDRAULIC MODELLING OF DESIGN FLOODS ... 31**

6.1 Presentation of Results ... 31
6.2 Discussion of Results ... 31
6.2.1. General ... 31
6.2.2. CBD Levee ... 31
6.2.3. Western Levee ... 32
6.2.4. Taminda Levee ... 33
6.3 Flood Planning Level ... 33
6.4 Provisional Flood Hazard ... 34
6.5 Climate Change Considerations ... 35
6.5.1. General ... 35
6.5.2. Sensitivity to Increased Rainfall Intensities ... 35

**7 POTENTIAL FLOOD MODIFICATION MEASURES ... 39**

7.1 General ... 39
7.2 CBD Levee Measures ... 39
7.3 Flood Damages Assessment ... 43

**8 REFERENCES ... 45**

**9 DEFINITIONS ... 47**

## APPENDICES

A Flood Questionnaire
B Historic Flood Level and Rainfall Data
C. Joint Probability Considerations of Coincident Flooding
D. Assessment of Flood Damages

# LIST OF FIGURES
# (BOUND IN VOLUME 2)

1.1 Location Plan

2.1 Plan Showing Route of CBD Levee and Location of Existing Flood Gates
2.2 Longitudinal Section along Crest of CBD Levee
2.3 Catchment Plan – CBD Levee
2.4 Plan Showing Route of Western Levee and Location of Existing Flood Gates
2.5 Longitudinal Section along Crest of Western Levee
2.6 Catchment Plan - Western and Taminda Levees
2.7 Plan Showing Route of Taminda Levee and Location of Existing Flood Gates
2.8 Longitudinal Section along Crest of Taminda Levee
2.9 Stream and Rain Gauge Data at Tamworth – November 2008 Flood
2.10 Layout of FPLAIN Hydraulic Model
2.11 Design Flood Levels on Peel River Floodplain

3.1 Sub-Catchment Layout Plan – Town Levees Hydrologic Model

4.1 TUFLOW Model Layout – CBD Levee
4.2 TUFLOW Model Layout – Western Levee
4.3 TUFLOW Model Layout – Taminda Levee
4.4 Adjusted Peel River Stage Hydrographs – Taminda Levee
4.5 TUFLOW Schematisation of Floodplain

6.1 TUFLOW Model Results – Flood Gates Fully Open – 2 year ARI (3 Sheets)
6.2 TUFLOW Model Results – Flood Gates Fully Closed – 2 year ARI (3 Sheets)
6.3 Afflux (Flood Gates Fully Closed Minus Flood Gates Fully Open) - 2 year ARI (3 Sheets)
6.4 TUFLOW Model Results – Flood Gates Fully Open – 5 year ARI (3 Sheets)
6.5 TUFLOW Model Results – Flood Gates Fully Closed – 5 year ARI (3 Sheets)
6.6 Afflux (Flood Gates Fully Closed Minus Flood Gates Fully Open) – 5 year ARI (3 Sheets)
6.7 TUFLOW Model Results – Flood Gates Fully Open – 20 year ARI (3 Sheets)
6.8 TUFLOW Model Results – Flood Gates Fully Closed – 20 year ARI (3 Sheets)
6.9 Afflux (Flood Gates Fully Closed Minus Flood Gates Fully Open) – 20 year ARI (3 Sheets)
6.10 TUFLOW Model Results – Flood Gates Fully Open – 100 year ARI (3 Sheets)
6.11 TUFLOW Model Results – Flood Gates Fully Closed – 100 year ARI (3 Sheets)
6.12 Afflux (Flood Gates Fully Closed Minus Flood Gates Fully Open) – 100 year ARI (3 Sheets)
6.13 TUFLOW Model Results – Flood Gates Fully Open – PMF (3 Sheets)
6.14 TUFLOW Model Results – Flood Gates Fully Closed – PMF (3 Sheets)
6.15 Afflux (Flood Gates Fully Closed Minus Flood Gates Fully Open) – PMF (3 Sheets)
6.16 Interim Flood Planning Areas (Ponding Areas Only) – Flood Gates Fully Open Versus Flood Gates Fully Closed
6.17 Provisional Flood Hazard – 100 year ARI – Gates Fully Open
6.18 Provisional Flood Hazard – 100 year ARI – Gates Fully Closed
6.19 Potential Impact of Climate Change – 100 Year ARI – Rainfall Increased by 10% - Flood Gates Fully Open
6.20 Potential Impact of Climate Change – 100 Year ARI – Rainfall Increased by 10% - Flood Gates Fully Closed

## LIST OF FIGURES (Cont'd)
## (BOUND IN VOLUME 2)

7.1 O'Connell Street Pressure Line Upgrade (Option 1) – CBD Levee
7.2 Impact of Option 1 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.3 Impact of Option 1 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.4 Fitzroy Street and White Street Pressure Line Upgrades (Options 2 and 3) – CBD Levee
7.5 Impact of Option 2 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.6 Impact of Option 2 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.7 Impact of Option 3 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.8 Impact of Option 3 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.9 Prince of Wales Park Trunk Drainage Upgrade (Option 4) – CBD Levee
7.10 Impact of Option 4 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.11 Impact of Option 4 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.12 Impact of Option 5 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.13 Impact of Option 5 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.14 Impact of Option 6 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.15 Impact of Option 6 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.16 Impact of Option 7A Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.17 Impact of Option 7A Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.18 Impact of Option 7B Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.19 Impact of Option 7B Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed

## SUMMARY AND RECOMMENDATIONS

### S.1 Introduction

An investigation was carried out by Lyall and Associates Consulting Water Engineers on behalf of Tamworth Regional Council (TRC) to assess local catchment flood behaviour behind three levees which have been built to protect existing development from flooding on the Peel River. **Figure 1.1** in **Volume 2** of this report shows the location of the three levees on the floodplain of the Peel River. For the purpose of the present investigation the three levees are collectively referred to as the '*town levees*' and individually as the '*Central Business District or CBD Levee'*, the '*Western Levee'* and the '*Taminda Levee'*.

### S2. Background Information

Backwater flooding from the Peel River is in the main controlled by a series of manually operated penstock type flood gates which have been fitted to the stormwater drainage lines that control local catchment flooding directly behind the town levees. Operational procedures developed by TRC dictate that the flood gates are to be fully closed when the water level in the Peel River reaches 4 m on the town gauge. This level equates to approximately bank full conditions in the river, which over the past 100 years has been reached about every 1.5 years on the average (refer **Table B1** in **Appendix B** for details).

During periods when the manually operated flood gates are closed, local catchment runoff temporarily ponds behind the town levees until such time as the flood gates are re-opened[1]. An analysis of continuous stream flow data dating back to 1993 shows that river levels have remained elevated above the critical level of 4 m on the town gauge for periods exceeding 30 hours (refer **Table B2** in **Appendix B** for details). Major flood that occurred in February 1955 remained above this level for a period of about 4 days[2].

An analysis of pluviographic rainfall records dating back to 1958 show that both short and long duration storms with average recurrence intervals (ARI's) greater than 100 years have occurred at Tamworth at or around the time water levels in the Peel River have been above the critical 4 m level on the town gauge (refer **Table B3** in **Appendix B** for details).

The most recent major storm event to have caused flooding in parts of Tamworth occurred in November 2008, when a burst of rain with an equivalent ARI of about 400 years was recorded at the Bureau of Meteorology's (BoM's) Oxley Lane rain gauge. An analysis of the available stream flow and rainfall data showed that this intense burst of rain fell only 3 hours prior to water levels in the river exceeding 4 m on the town gauge (refer **Figure 2.9** in **Volume 2** of this report for plot of available water level and rainfall data). TRC advised that whilst the very intense rain that was recorded by BoM's rain gauge was representative of rain which fell over parts of West Tamworth, it was not representative of the rain which was experienced over the catchments which drain behind the town levees. Whilst flooding experienced behind the town levees as a result of storm which occurred in November 2008 did not cause major flooding in property located directly behind the town levees, the close proximity of very intense rainfall at a time when water levels in the Peel

[1] The exception is the relatively large temporary flood storage area which is located behind the Taminda Levee south of the Tamworth Racecourse. Backwater flooding in this area is controlled by a series of hinged flap gates which have been fitted to the outlet of a large box culvert system which has been built in the levee bank.

[2] Source: Floodplain Management Study - City of Tamworth (PPK, 1993)

River were steadily rising demonstrates that under slightly different meteorologic conditions flood producing rain could have fallen over the study catchments at a time when the manually operated flood gates were closed.

Based on a review of the available data, it is concluded that whilst flood producing rainfall can and will occur over the catchments which drain behind the town levees during periods when the water level in the Peel River is below 4 m on the town gauge, there is a reasonable chance that the flood gates will be closed over the full range of ARI storms.

## S3. Hydrology and Hydraulics

The investigation required the development of a hydrologic model of the local catchments which drain behind the town levees and a hydraulic model which extended a sufficient distance behind the town levees to adequately define local drainage patterns in the study area. The hydrologic model was a runoff-routing model based on the DRAINS software, with the inbuilt RAFTS and DRAINS modelling approaches used for generating discharge hydrographs from the rural and urbanised parts of the study area, respectively. A depth-averaged, one and two-dimensional free surface flow hydraulic modelling approach was chosen as it allowed for the interaction of flow in the stormwater drainage lines which discharge directly to the Peel River floodplain and the various overland flow paths which lie directly behind the town levees. The time varying effects of elevated water levels in the Peel River were also taken into accounted. The TUFLOW hydraulic modelling program was adopted for this purpose.

Local catchment flood behaviour behind the town levees has been defined in terms of flows, levels and velocities for floods ranging between 2 and 200 years ARI, as well as for the Probable Maximum Flood (PMF). In order to understand the full range of potential flooding behind the town levees the hydraulic model was run for the cases where the manually operated flood gates are in either their fully open or fully closed position. **Figures 6.1** to **6.15** in **Volume 2** of this report show local catchment flood behaviour behind the town levees for the full range of storm events.

In the case of all three levees, the investigation found that existing development will be subject to flooding as a result of uncontrolled overland flow which approaches the ponding areas from upslope. It is recommended that the two-dimensional modelling carried out as part of this present investigation be extended into these areas as part of a future study in order to more accurately define local drainage patterns in the urbanised parts of the study catchments.

## S4. CBD Levee Flooding

It was found that existing development is at greatest risk of flooding behind the CBD Levee, where significant flood damages will be experienced in existing commercial development over the full range of storm events. It is estimated that the Present Worth Value[3] of flood damages which would be incurred for all floods up to the 100 year ARI event is around $3.6 Million for the case where the flood gates are assumed to be always open, increasing to around $42 Million for the case where the flood gates are assumed to be always closed. In reality the Present Worth Value of flood damages experienced in property located behind the CBD Levee lies somewhere between these two amounts, as there will be times when the flood gates are open during flood producing rain and times when the flood gates will be closed. Other criteria such as the duration over which the rain occurs during periods of gate closure and TRC's practice of partially opening several of the flood

[3] Assuming a discount rate of 7 per cent and an economic life of 20 years.

gates whilst river levels remain above the critical 4 m level on the town gauge will also infleunce the degree to which existing development is impacted by local catchment flooding. Based on the relatively limited number of occasions when major flooding has been experienced in the CBD of Tamworth, it is believed that the Present Worth Value of flood damages lies at the lower end of the flood damages scale (i.e. closer to $3.6 Million than $42 Million).

### S5. Western Levee Flooding

The present investigation generally confirmed the findings of an earlier study which was undertaken during the planning for the Western Levee, although the 100 year ARI flood level derived herein is 80 mm higher than that derived by the previous study. Whilst above-floor flooding in existing development will not occur as a result of local catchment runoff ponding behind the Western Levee for floods up to 100 year ARI, flood damages will still be incurred in existing development. This finding was confirmed by several respondents to the flood questionnaire which was disseminated at the commencement of the study (refer **Appendix A** for a copy), who stated that they had experienced flooding in the rear of their properties dating back to 2008.

### S6. Taminda Levee Flooding

The present investigation identified the potential for uncontrolled backwater flooding to occur behind the Taminda Levee due to the absence of flood gates on three individual drainage lines. In response to this finding, TRC advised that flood gates would be fitted to these drainage lines and that for the purpose of the present investigation.

The present investigation found that local catchment runoff that temporarily ponds behind the Taminda Levee is generally confined to the local road network and land which is presently undeveloped. Minor flooding in commercial property located to the west of Crown Street was found to occur under gate closure conditions.

### S7. Interim Flood Planning Levels

Interim Flood Planning Levels (FPL's) were derived based on the 100 year flood level plus 500 mm of freeboard for the cases where the flood gates are in their fully open and fully closed positions.[4]. **Figure 6.16** in **Volume 2** of this report shows the extent of the interim FPL's behind the town levees.

Note that whilst the interim FPL's shown on **Figure 6.16** are based on the upper envelope of potential flooding for the 100 year ARI event, they do not take account of the impact potential blockages of the existing stormwater drainage system will have on local catchment flood behaviour directly behind the town levees. It is recommended that consideration be given to the impact a potential blockage of the existing stormwater drainage system will have on ponding levels behind the town levees prior to adopting a final FPL.

[4] Note that the FPL derived as part of the present investigation is limited to the ponding areas which are located directly behind the town levees and does not include the overland flow which approaches these areas from the upslope catchment.

## S8. Provisional Flood Hazard and Potential Impacts of Climate Change

Provisional flood hazard based on the product of depth and velocity has been mapped for the cases where the flood gates are in their fully open and fully closed positions (refer **Figures 6.17** and **6.18** in **Volume 2** of this report).

The effects on flooding patterns of a potential increase in rainfall intensity associated with climate change were assessed. The increase in peak flood levels associated with a possible 10 per cent increase in 100 year ARI rainfall intensities for the cases where the flood gates are in their fully open and fully closed positions are shown on **Figures 6.19** and **6.20** in **Volume 2** of this report, respectively.

## S9. Potential Flood Modification Measures

Several potential flood modification measures aimed at reducing the impact of local catchment flooding on existing development located directly behind the CBD Levee were assessed. These measures included the upgrade of several existing pressure lines which presently convey runoff from areas which lie upslope of the CBD directly to the Peel River, as well as the installation of several major pump stations at key locations along the levee bank**. Figures 7.1** to **7.19** in **Volume 2** of this report show details of the various flood modification measures which were assessed as part of the present investigation and the impact they would have on peak 100 year ARI flood levels.

The study found that it is not possible to alleviate flooding behind the CBD Levee for all floods up to the 100 year ARI event. This is due principally to not being able to prevent uncontrolled overland flow from discharging to the CBD area from upslope areas combined with the need to evacuate large volumes of floodwater from behind the levee, especially under gate closure conditions.

The investigation found that should TRC upgrade four of the five existing pressure lines which control flooding behind the CBD Levee, the Present Worth Value[5] of damages saved for all floods up to the 100 year ARI event would be only around $1.9 Million for the case where the flood gates are assumed to be always open, increasing to around $25.4 Million for the case where the flood gates are assumed to be always closed. The installation of major pumping stations along the CBD Levee would further reduce the flood damages experienced in this area.

## S10. Recommendation for Further Investigations

Prior to adopting a final set of flood modification measures aimed at reducing the flood risk behind the CBD Levee, it is recommended that further investigations be undertaken to:

a. define flooding behaviour in the urban parts of Tamworth which lie to the north (i.e. upslope) of the Main Western Railway Line;

b. assess opportunities for implementing flood modification measures in these upslope areas which are aimed at further reducing the impact of flooding on existing development located in the Tamworth CBD; and

c. assess the impacts potential blockages in the stormwater drainage network, especially at the inlets of the pressure lines which discharge directly to the Peel River floodplain, will have on flooding behaviour in the Tamworth CBD.

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[5] Assuming a discount rate of 7 per cent and an economic life of 20 years.

The key reasons for undertaking these additional investigations are:

i. The present investigation identified the potential for inter-catchment transfers of flow to occur at several locations north of the Main Western Railway Line (i.e. in areas which lie upslope of the Tamworth CBD). The potential for stormwater to approach the Tamworth CBD via alternative routes has major implications on flood behaviour in the area and the identification of the most appropriate measures for managing the flood risk in this area.

ii. The implementation of alternative measures in the middle and upper reaches of the existing stormwater drainage network, such as flood retarding basins and channel improvement works, could reduce the flooding risk in the Tamworth CBD and possibly reduce the scope of the flood modification measures identified as part of the present investigation[6].

iii. Blockages in the existing stormwater drainage network due to the built up of debris have the potential to increase the severity of flooding behind the CBD Levee. A comprehensive understanding of the range over which peak flood levels could vary under differing blockage scenarios is required before a decision can be made on a final set of FPL's for the Tamworth CBD.

[6] It is noted that the presence of an existing flood retarding basin was identified north of the Main Western Railway Line on the drainage line which runs through Jaycees Park during the latter stages of the investigation. The attenuating effects of this basin on flows in this drainage line have therefore not been incorporated in the present study findings.

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# 1 INTRODUCTION

## 1.1 Background and Approach

This report deals with the findings of an investigation into local catchment flood behaviour behind three flood protection levees which have been built to protect parts of Tamworth. The location of the three levees, which collectively are referred to as the '*Town Levees*', are shown on **Figure 1.1**. For the purpose of the present investigation, the three levees are referred to individually as the '*Central Business District or CBD Levee'*, the '*Western Levee'* and the '*Taminda Levee'*.

Based on the findings of the above investigation, the benefits of implementing several flood modification measures aimed at reducing the impact of local catchment flooding on existing development in terms of a reduction in flood damages were assessed.

Using information provided by Tamworth City Council (TRC), photogrammetric based ground survey models of the study area, plus detailed field surveys of the existing stormwater drainage network, mathematical models were developed and interpreted to present a comprehensive picture of local catchment flood behaviour behind the town levees under current conditions.

The study objective was to define local catchment flood behaviour directly behind the town levees in terms of flows, levels and velocities for floods ranging between 2 and 200 years average recurrence interval (ARI), as well as for the Probable Maximum Flood (PMF).

One of the key features governing flood behaviour behind the town levees is the position of the thirty-two (32) manually operated penstock type flood gates which TRC has installed on the outlets of the stormwater drainage lines which control local catchment runoff behind the town levees. For the purpose of this present investigation, local catchment flood behaviour has been defined for the cases where the manually operated flood gates are in either a *fully open* or *fully closed* position.

Whilst a review and analysis of historic river levels and rainfall records has been undertaken to assess the likelihood of the flood gates being closed at the time flood producing rainfall is experienced over the study catchments, the determination of the joint probabilities of coincident gate closure conditions and local catchment runoff events of differing average recurrence interval (ARI) presently lies beyond the scope of this investigation.

Flood behaviour was defined using computer based hydrologic models of the catchments and hydraulic models of the drainage lines which control local catchment flooding directly behind the town levees. The hydrologic model was a runoff-routing model based on the DRAINS software which converts historic and design storm rainfalls to discharge hydrographs. The inbuilt RAFTS and DRAINS modelling approaches were used for generating discharge hydrographs from the rural and urbanised parts of the study area, respectively.

A dynamic hydraulic modelling approach was adopted for the analysis to account for the time varying effects of tailwater conditions and flow in the stormwater drainage lines which control local catchment runoff, the routing effects of the flood storage which is present behind the town levees and the two-dimensional effects of overland flow as it approaches the temporary flood storage areas which are located directly behind the town levees. A depth-averaged, one and two-dimensional free surface flow modelling approach was chosen as it allows for the interaction of flow in the stormwater drainage lines which discharge directly to the Peel River floodplain and the various overland flow paths which lie directly behind the town levees. The TUFLOW hydraulic modelling program was adopted for this purpose.

Design storms were applied to the hydrologic models to generate discharge hydrographs within the study area. These hydrographs constituted the upstream boundaries and internal inflow inputs to the hydraulic model. Local catchment flooding behaviour was defined for the two scenarios where the flood gates are in either their fully open or fully closed positions. Varying tailwater conditions representing Peel River floods of differing ARI were also applied to the outlet of those drainage lines for which manually operated flood gates have not been fitted[7].

The model was used to prepare plans showing the indicative extent and depth of flooding behind the town levees for the design events. The results of the modelling were used to estimate the flood damages which would be incurred in existing development as a result of local catchment floods of differing ARI.

The structure of the hydraulic model representing current conditions was altered in order to assess the impact several possible flood modification measures would have on local catchment flood behaviour.

## 1.2 Study Tasks

The study had five components:

- ***Review of available hydrologic and hydraulic data and previous investigations.*** The Bureau of Meteorology (BoM) provided rainfall data for a number of historic storms which have occurred at Tamworth, whilst the NSW Office of Water (NoW) provided continuous water level data for the Peel River at Tamworth which dates back to 1993 and peak water level data for historic flood events which dates back to 1910.

  The following data was also provided by TRC:
  - Aerial photography which was flown on 4 March 2008.
  - Design drawings showing details of the town levees, including recent works undertaken by TRC to upgrade the CBD Levee[8].
  - A database containing details of major drainage upgrades in hardcopy format.
  - A database which contained pit and pipe data in ARCVIEW format. (Note no invert levels were contained in the database).
  - Miscellaneous GIS based information such as cadastre boundary and house numbers in ARCVIEW format.
  - NSW Lands and Property Information (LPI) ground contour data at 2 m and 10 m intervals.

[7] Four pressure lines, the outlets of which are not gated, presently control runoff from parts of the catchment which drains behind the CBD Levee. Flap type gates have also been installed on a large box culvert system which controls local catchment flooding behind the Taminda Levee. Refer **Chapter 2** for further details.

[8] A limited number of work-as-executed plans were made available for the town levees and the stormwater drainage lines which control local catchment flooding behind them.

A brief was prepared for the preparation of three separate aerial based digital terrain models (DTM's) of the areas protected by the town levees. South Australian based Aerometrex subsequently prepared the DTM's to the following specifications:

- Date of Photography: January-February 2011
- Horizontal Projection: GDA94, MGA Zone 54
- Vertical Elevation: Australian Height Datum (AHD)
- Accuracy 10 cm pixel resolution

| | |
|---|---|
| Horizontal (Ortho) | +/- 0.20m RMSE |
| Horizontal (Point) | +/- 0.10m RMSE |
| Vertical | +/- 0.08m (68% confidence interval, 1$\sigma$ ) |
| | +/- 0.17m (95% confidence interval, 2$\sigma$ ) |

A brief was also prepared for the capture of pit and pipe data, as well as inbank cross sectional survey. Parkes based surveyors, Casey Surveying and Design Pty Ltd undertook the survey, with the data provided in both spreadsheet and CADD format. Photographs of each stormwater pit were also captured by the surveyors at the time of the field survey.

- ***A hydrologic component*** which included preparation of the hydrologic model of the study catchments, adoption of model parameters for design flood estimation, derivation of design storms and their application to the models to define design discharge hydrographs.

- ***A hydraulic component*** which comprised the preparation of hydraulic models of the areas which are protected by the town levees and the application of discharge hydrographs to the models to define extents and depths of inundation, flows and velocities for the design floods.

- ***A flood damages component*** which comprised the estimation of damages which could occur to existing development located directly behind the town levees as a result local catchment flooding.

- ***A flood mitigation component*** which included the assessment of measures which are aimed at reducing the impact of local catchment flooding on existing development located directly behind the town levees.

### 1.3 Overview of Report

**Chapter 2** of the Report contains background information including a brief description of the town levees and their local catchments, a review of the data base available for the study including procedures for operating the flood gates which prevent backwater flooding from the Peel River, the outcomes of the community consultation process, historic flooding in Tamworth and previous flood studies.

**Chapter 3** deals with the development of the computer based catchment model which was used to generate discharge hydrographs for input to the hydraulic model.

**Chapter 4** deals with the development of the hydraulic models which were used to analyse flood behaviour behind the town levees.

**Chapter 5** deals with the derivation of design runoff hydrographs from the catchments which drain behind the town levees. This step involved the determination of design storm rainfall depths over the study catchments for a range of storm durations, and conversion of the rainfall hyetographs to discharge hydrographs.

**Chapter 6** details the results of the hydraulic modelling of the design floods. Also contained in this section of the report are the findings of an investigation into the impacts a potential increase in rainfall intensity associated with climate change would have on local catchment dominate flood behaviour.

**Chapter 7** deals with the findings of an investigation into the impact a range of potential flood modification measures would have on local catchment flood behaviour directly behind the town levees.

**Chapter 8** contains a list of References.

**Chapter 9** contains a list of definitions of flood related terms used in the study.

A copy of the questionnaire which was distributed by TRC at the commencement of the study is contained in **Appendix A**. The results of an analysis of historic water levels in the Peel River and coincident rainfall recorded at Tamworth Airport are presented in a series of tables which are contained in **Appendix B**. Considerations given to the joint probability of coincident local catchment and Peel River flooding are contained in **Appendix C**. **Appendix D** contains details of the flood damages assessment which was undertaken as part of the present investigation.

Figures referred to in both the report are bound in a separate volume of the report (see **Volume 2**).

# 2 BACKGROUND INFORMATION

## 2.1 Study Area

### 2.1.1. CBD Levee

The CBD Levee was originally constructed in the 1930's to protect existing commercial development located along the northern overbank of the Peel River. The levee was raised on several occasions in the period 1976-78 in response to perceived flood threats, and again in 1996-97 to provide a one metre level of protection to the 100 year ARI design flood event.

**Figure 2.1** shows the route of the CBD Levee where it runs along the northern bank of the Peel River between Murray Street and Bligh Street, whilst **Figure 2.2** is a longitudinal section showing several key elements of the CBD Levee, such as the level of the levee crest, design food levels in the adjacent Peel River and details of the stormwater drainage lines that outlet to the river through the levee bank.

The CBD Levee is principally an earth embankment which is up to 4.2 metres in height. Sections of reinforced concrete wall were constructed along the top of the levee in 1996-97 at locations where the available footprint prevented the raising of the existing earth embankment. Aluminium flood barriers are also required across Brisbane Street and the pedestrian footbridge located opposite the southern end of Fitzroy Street in order to achieve the required design height for the levee (refer **Figure 2.2** for location).

**Figure 2.3** shows the extent of the 9.9 $km^2$ catchment which drains to the Peel River from behind the CBD Levee. The upper portion of the catchment lies in the hilly region to the north of Tamworth where the average sub-catchment slopes are generally in the range 10 to 20 per cent. Whilst large parts of the hilly area to the north of Tamworth are heavily wooded, the portion of the catchment that drains to the Peel River at the western end of the CBD Levee has been cleared, with vegetation cover comprising primarily pastoral grass. The watercourses that drain the hilly region are generally in a natural or semi-natural state.

Whilst several drainage channels run through the urbanised parts of North and East Tamworth, the majority of the stormwater drainage system comprises either pipe or culvert reaches (refer **Figure 2.3**).

Stormwater discharging to the Peel River from the local catchment is controlled by twenty-three individual drainage lines, the outlet level of which are shown on **Figure 2.2**. The individual drainage lines can be categorised into the following two groups:

- **Ungated Pressure Lines**: Four of the drainage lines have been designed to operate under pressure, with their inlets located generally to the north of the Main Western Railway Line above the crest level of the levee. For the purpose of the present investigation, these four lines have been denoted the O'Connell Street, Brisbane Street, Fitzroy Street and White Street pressure lines. The locations where the four pressure lines discharge to the Peel River are shown on **Figure 2.1**.

  A fifth drainage line which runs through Jaycees and Prince of Wales Park at the eastern (upstream) end of the levee controls runoff from areas which lie above the crest level of the levee. Stormwater which surcharges this drainage line discharges onto Roderick Street where it contributes to overland flooding problems behind the levee.

- **Gated Gravity Drainage Lines**: There remaining eighteen individual drainage lines control stormwater runoff west (down slope) of the Main Western Railway Line. In order to prevent backwater flooding from the Peel River each of these lines have been fitted with manually operated penstock type flood gates. Details of the eighteen penstock type flood gates, the locations of which are shown on **Figure 2.1**, are contained in **Section 2.4**.

### 2.1.2. Western Levee

The Western Levee was built in 2002-03 and protects existing residential and commercial development which is located on the eastern side of Goonoo Goonoo Road between Margaret Street and Mathews Street. The levee is of earth embankment type construction and is up to 5.6 metres in height. **Figure 2.4** shows the route of the Western Levee where it runs along the left (western) bank of Barnes Gully, whilst **Figure 2.5** is a longitudinal section along the crest of the levee.

The 1.94 $km^2$ catchment which drains to Barnes Gully from behind the Western Levee is shown in **Figure 2.6**. Average sub-catchment slopes west of the levee are generally in the range 2 to 5 per cent, however, they do increase to between 5 to 10 percent in several locations. The catchment is fully urbanised, comprising primarily residential type development.

Stormwater runoff originating from the local catchment behind the levee is controlled by four individual drainage lines which discharge to Barnes Gully on its left (western) bank. **Figure 2.5** shows the elevation of the four individual drainage lines where they outlet to Barnes Gully along the line of the Western Levee.

Six individual flood gates have been fitted to the two most northern drainage lines (refer **Figure 2.4** for location of flood gates). Details of the six individual penstock type flood gates which control backwater flooding behind the Western Levee are contained in **Section 2.4**.

The two drainage lines which discharge to Barnes Gully at the southern end of the Western Levee have been designed to operate under pressure and as a result do not have flood gates fitted to their outlets. The outlet of these two drainage lines, which for the purpose of this present investigation have been denoted the Margaret Street and Goonoo Goonoo Road pressure lines, are shown on **Figure 2.4**.

### 2.1.3. Taminda Levee

A privately built levee which once ran around the northern side of the Tamworth Racecourse was upgraded in 2008-09 to form a 2.7 km long levee which presently protects commercial and industrial property located in the Taminda area. **Figure 2.7** shows the route of the levee where is runs from the Main Northern Railway Line in West Tamworth to Jewry Street in Taminda, whilst **Figure 2.8** is a longitudinal section along the crest of the levee.

The Taminda Levee comprises two relatively short sections of reinforced concrete block wall where it runs along the northern side of Ebsworth Street between Barnes Street and Plain Street and earth embankment where it runs from Plain Street around the northern and western sides of the Tamworth Racecourse before tying into high ground at Jewry Street. Provision is also incorporated in the road reserves of Barnes Street and Ebsworth Street to facilitate the manual installation of aluminium flood barriers.

The extent of the 4.5 $km^2$ catchment which drains to either Barnes Gully or the Wallamore Anabranch from behind the Taminda Levee is shown on **Figure 2.6**. The upper portion of the catchment lies to the east of the Main Northern Railway Line and includes the Tamworth Golf Course, whilst the lower portion of the catchment comprises the commercial and industrial parts of Taminda and West Tamworth. Average sub-catchment slopes are generally in the range 2 to 5 per cent to the east, and 0 to 2 per cent to the west of the rail corridor.

Runoff from the catchment which lies behind the Taminda Levee is controlled primarily by pipe and culvert reaches, although several short sections of channel are present in the lower reaches of the drainage system immediately south of the Tamworth Racecourse.

Stormwater runoff originating from the local catchment behind the levee is controlled by eleven individual drainage lines which discharge to either Barnes Gully or the Wallamore Anabranch on the southern (left) overbank of the Peel River. Backwater flooding behind the Taminda Levee is controlled by eleven individual flood gates which have been installed on or near the outlet of nine individual drainage lines. **Figure 2.7** shows the location of the individual flood gates, whilst **Figure 2.8** shows the elevation of the individual drainage lines at their outlet. Details of the flood gates which control backwater flooding behind the Taminda Levee are contained in **Section 2.4**.

Data provided by TRC shows the presence of two minor drainage lines which control runoff from the section of Ebsworth Street which runs between Barnes Street and Plain Street. TRC confirmed that these two drainage lines, whilst located behind the reinforced block section of levee, are not fitted with flood gates[9].

There are three pipes which drain the section of Ebsworth Street which runs between Plain Street and Jewry Street. Because this section of road runs along the crest of the levee, the outlets of these three drainage lines have not been fitted with flood gates.

A 600 mm diameter pipe was observed to drain to the Wallamore Anabranch to the north of Tamworth Racecourse which also does not have a flood gate fitted to it (refer **Figure 2.8** which shows the outlet of the 600 mm diameter pipe located at about Chainage 1650). Whilst several shallow surface inlet pits were observed on the landward side of the levee immediately adjacent to the pipe outlet, TRC's database does not show the interconnecting pipework[10].

## 2.2 Flood Gate Details

**Table 2.1** gives details of the existing flood gates which control backwater flooding behind the town levees. The flood gates are all of penstock type construction requiring manual operation, with the exception of the three flap type gates which have been installed on the large box culvert system which controls flooding behind the Taminda Levee south of the Tamworth Racecourse (refer **Flood Gates 32a**, **32b** and **32c**).

Procedures for the manual operation of the 32 penstock type flood gates which control backwater flooding behind the town levees are contained in TRC's Regional Services Directorate document entitled "*Flood Standing Instructions Tamworth City Area".* Section 2 of the document states the following in relation to the procedures for closure of the flood gates:

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[9] TRC advised that for the purpose of this present investigation it is to be assumed that flood gates have been fitted to these two drainage lines.

[10] TRC advised that for the purpose of this present investigation it is to be assumed that a flood gate has been fitted to this drainage line.

"(a) *Two teams with two workers per team should be assigned to close the floodgates. Resources for road closures are in addition to this number;*

(b) *all floodgates should be half closed when the rise rises to 3.0m (as noted on height markers on bridge);*

(c) *all floodgates should be closed (completed) when the river rises to 3.0m* [sic, text should state 3.6 m] *(as noted on height markers on bridge) – if there is no rain in the area of East and North Tamworth Catchment;*

(d) *All floodgates should be closed (completed) when the river rises to 4.0m (as noted on height markers on bridge) – if there is rain in the catchment area of East and North Tamworth.*

(e) *The Field Coordinator will need to assess the rate of the rises of the River to determine when closure of the floodgates should commence. 45 minutes should be allowed for closure of the floodgates. Consideration should be given to partial closure of each gate one (1) hour prior to this time.*

**Figures 2.2**, **2.5** and **2.8** show the three critical trigger levels of 3.0, 3.6 and 4.0 m projected along the line of the town levees. Note that for plotting purposes it has been assumed that the flood slope in the Peel River and its major tributaries at these levels is identical to the 20 year ARI design flood event which has an equivalent level of 6.84 m on the town gauge (refer **Section 2.5.2** for source of design flood level data).

**TABLE 2.1**
**DETAILS OF EXISTING FLOOD GATES**

| Location | Identifier | Gate Type | Pipe/Culvert Dimensions at Outlet of Stormwater Drainage Line[11,12] | Invert Level at Outlet (m AHD) |
|---|---|---|---|---|
| CBD Levee | FG1 | Penstock | 1 off 450 RCP | 372.09 |
| | FG2 | Penstock | 1 off 375 RCP | 371.97 |
| | FG3 | Penstock | 1 off 900 RCP | 371.97 |
| | FG4 | Penstock | 900 wide by 840 high RCBC | 372.77 |
| | FG5b | Penstock | 1 off 1800 RCP | 371.53 |
| | FG6 | Penstock | 1 off 900 RCP | 372.43 |
| | FG7 | Penstock | 1 off 900 RCP | 371.68 |
| | FG8a | Penstock | 1 off 900 RCP | 372.28 |
| | FG9 | Penstock | 1 off 1350 RCP | 373.07 |
| | FG10 | Penstock | 1 off 450 RCP | 374.48 |
| | FG11 | Penstock | 1 off 1200 wide by 750 high RCBC | 374.68 |
| | FG12 | Penstock | 1 off 1200 wide by 750 high RCBC | 374.68 |
| | FG13 | Penstock | 1 off 1200 wide by 750 high RCBC | 374.68 |

[11] All dimensions are in millimetres
[12] RCP = Reinforced Concrete Pipe RCBC = Reinforced Concrete Box Culvert

## TABLE 2.1 (Cont'd)
## DETAILS OF EXISTING FLOOD GATES

| Location | Identifier | Gate Type | Pipe/Culvert Dimensions at Outlet of Stormwater Drainage Line[13,14] | Invert Level at Outlet (m AHD) |
|---|---|---|---|---|
| CBD Levee | FG14 | Penstock | 1 off 450 RCP | 374.52 |
| | FG15 | Penstock | 1 off 1120 wide by 900 high RCBC | 374.33 |
| | FG16 | Penstock | 1 off 525 RCP | 374.57 |
| | FG16-1 | Penstock | 1 off 1350 RCO | 374.14 |
| | FG17 | Penstock | 1 off 900 RCP | 375.58 |
| Goonoo Goonoo Road | FG18 | Not Relevant to Present Investigation | | |
| | FG19 | | | |
| Bicentennial Park | FG20 | | | |
| | FG21 | | | |
| Western Levee | FG22a | Penstock | 1 off 1050 RCP | 375.62 |
| | FG22b | Penstock | 1 off 1050 RCP | 375.62 |
| | FG23a | Penstock | 1 off 600 RCP | 377.25 |
| | FG23b | Penstock | 1 off 600 RCP | 377.25 |
| | FG23c | Penstock | 1 off 600 RCP | 377.25 |
| | FG23d | Penstock | 1 off 600 RCP | 377.25 |
| Taminda Levee | FG24 | Penstock | 1 off 450 RCP | 373.22 |
| | FG25 | Penstock | 1 off 450 RCP | 372.48 |
| | FG26 | Penstock | 1 off 600 RCP | 372.60[15] |
| | FG27 | Penstock | 1 off 750 RCP | 373.54 |
| | FG28 | Penstock | 1 off 750 RCP | 373.41 |
| | FG29 | Penstock | 1 off 1350 RCP | 370.66 |
| | FG30 | Penstock | 1 off 1350 RCP | 370.66 |
| | FG31 | Penstock | 1 off 900 RCP | 369.72 |
| | FG32a | Flap Gate | 1 off 2700 wide by 1200 high RCBC | 370.19 |
| | FG32b | Flap Gate | 1 off 2700 wide by 1200 high RCBC | 370.19 |
| | FG32c | Flap Gate | 1 off 2700 wide by 1200 high RCBC | 370.19 |

13 All dimensions are in millimetres
14 RCP = Reinforced Concrete Pipe RCBC = Reinforced Concrete Box Culvert
15 Based on 2011 Photogrammetric Survey

## 2.3 Community Consultation

TRC issued a press release at the commencement of the study seeking input from the community on historic flooding behind the town levees. Approximately 300 flood questionnaires were also distributed to residents and business owners of property located behind the town levees. **Appendix A** contains a copy of the flood questionnaire.

A total of forty-six questionnaires were returned by the closing date of submissions, ten of which contained information on observed flood behaviour behind the town levees. **Table 2.2** contains a summary of the comments made by the ten respondents.

Limited quantitative information, such as historic flood marks, was provided by the respondents to the flood questionnaire. The most severe flooding appears to have occurred in November 2008, when intense rainfall was experienced over parts of the Tamworth. Further discussion on historic flooding behind the town levees is contained in **Section 2.4.2**.

**TABLE 2.2**
**SUMMARY OF RESPONSES TO FLOOD QUESTIONNAIRE**

<table>
<tr><th>Town Levee</th><th>Address</th><th>Comments</th></tr>
<tr><td rowspan="6">Western Levee</td><td>116 Goonoo Goonoo Road</td><td>• Flooding occurred in property when pipes draining area behind levee became blocked by debris.<br>• Flooding was a result of stormwater which surcharged drainage pipes.</td></tr>
<tr><td>118 Goonoo Goonoo Road</td><td>• Stormwater observed to pond behind levee whenever heavy rain is experienced in Tamworth (e.g. on 28 November 2008).<br>• Stormwater ponding behind levee has impacted on the property on at least two occasions.</td></tr>
<tr><td>160 Goonoo Goonoo Road</td><td>• Stormwater observed to pond behind levee during heavy rain which fell in September 2011 and prior.<br>• Stormwater ponding behind levee has not impacted on the property.</td></tr>
<tr><td>136 Goonoo Goonoo Road</td><td>• Stormwater observed to pond behind levee during storms which occurred in November 2008, June 2011 and October 2011.<br>• During storm which occurred in November 2008, water extended into property and inundated shed located at rear of property.</td></tr>
<tr><td>168 & 172 Goonoo Goonoo Road</td><td>• Stormwater has been observed to discharge through property from Goonoo Goonoo Road.<br>• Stormwater has been observed discharging from Kent Street at a depth of between 100-150 mm. At the time, the flow extended across the full width of the road.</td></tr>
<tr><td>182 Goonoo Goonoo Road</td><td>• Stormwater observed to surcharge pit located at rear of property during storm which occurred in November 2008.<br>• During storm which occurred in November 2008, water extended into property and inundated shed located at rear of property.</td></tr>
<tr><td rowspan="4">CBD Levee</td><td>178 Peel Street</td><td>• Stormwater observed to pond in Peel Street during periods when the flood gates are closed.</td></tr>
<tr><td>184 Peel Street</td><td>• Depth of flow in the gutter has on several occasions been sufficient to inundate driveway.</td></tr>
<tr><td>365 Peel Street</td><td>• Property has been flooded due to surcharge of the pipe drainage system in Peel Street.</td></tr>
<tr><td>523 Peel Street</td><td>• Property has been flooded due to excessive gutter flow.</td></tr>
</table>

## 2.4 Historic Flooding in Tamworth

### 2.4.1. Valley-Wide Flooding

From 1840 up to the time when records commenced in January 1925, two major floods have been reported on the Peel River at Tamworth, in 1864 and 1910. The most severe of these early floods was the 1864 flood which was apparently "probably Tamworth's worst"[16]. The 1910 flood reached an equivalent level of 6.93 m on the stream gauge which is located on the Bridge Street crossing of the Peel River (denoted herein as the *town gauge*).

Since records commenced in January 1925 the water level in the Peel River has exceeded the critical 4 m trigger level on the town gauge on over 60 separate occasions. The days when the Peel River has peaked above the critical 4 m trigger level are summarised in **Table B1** in **Appendix B**).

The two largest floods since 1925 were those of February 1955 and January 1962, when the water level in the river reached 7.16 m and 6.86 m, respectively. The February 1955 flood is understood to have remained above the critical 4 m trigger level on the town gauge for around four days[17].

A telemetered stream gauge was installed by the NSW Office of Water (NoW) at the site of the town gauge on 27 July 1993 (*Peel River at Tamworth – Station No. 419009*). Using instantaneous water level data captured by the telemetered stream gauge, it is possible to determine the duration water levels have remained above the critical 4 m trigger level during historic floods dating back to 1993 (refer **Table B2** in **Appendix B** for details). Whilst the duration water levels remain above the critical 4 m trigger level on the town gauge varies, the data shows that there have been several floods when the water level in the Peel River has remained above the critical trigger level for periods exceeding 30 hours.

The most recent 'major' flood to be experienced at Tamworth occurred in November 2008, when the Peel River peaked at 6 m on the town gauge. **Figure 2.9** shows the stage hydrographs which were recorded at the town gauge and also at the NoW operated stream gauges on Goonoo Goonoo Creek at Meadows Lane (Station No. 419097) and Timbumburi (Station No. 419035). During the November 2008 flood water levels in the Peel River remained above the critical 4 m trigger level for a period of about 22 hours.

### 2.4.2. Local Catchment Flooding

Information on historic flooding behind the town levees is limited to a small number of events dating back to November 2008 (refer **Section 2.3** for details).

Based on the responses received from several business owners to the flood questionnaire, it is evident that commercial property within the Tamworth CBD is subject to flooding during storms which surcharge the local stormwater drainage system. Anecdotal evidence on flooding in commercial property as a result of stormwater ponding behind the CBD Levee is limited to property located at the southern end of Roderick Street and in Peel Street west of O'Connell Street.

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[16] PPK (1993)
[17] PPK (1993)

Several residential properties located behind the Western Levee were affected by local catchment runoff during intense rain which fell on 28 November 2008. One resident observed stormwater upwelling from an existing surcharge pit which is located directly behind the levee on the drainage line to which flood gates FG23a to FG23d have been fitted, whilst several other residents observed flooding in their yards.

Rainfall recorded at BoM's Tamworth (Oxley Lane) rain gauge (Station No. 055327) during the intense storm that occurred over parts of Tamworth on 28 November 2008 shows that a total of 166 mm of rain fell over the course of the day. This compares to a total depth of 164.2 mm which was recorded at BoM's All Weather Station (AWS) located at Tamworth Airport (Station No. 055325) on the rain day of 29 November 2008[18].

**Figure 2.9** shows the cumulative rainfall depths recorded at BoM's Tamworth (Oxley Lane) rain gauge for the three day period commencing 0000 hours on 28 November 2008. The rainfall data shows that an intense burst of rainfall occurred around 2030 hours on 28 November 2008, approximately one hour prior to the time when water levels in the Peel River reached the first critical trigger level of RL 3 m on the town gauge.

Based on rainfall recorded at BoM's Tamworth (Oxley Lane) rain gauge, the burst of rainfall which occurred around 2030 hours on 28 November 2008 had an equivalent ARI of between 350 and 400 years for periods of between 20 and 60 minutes, reducing to an equivalent ARI of about 150 years for a period of 90 minutes.

TRC advised that the very intense rainfall that was recorded at BoM's Tamworth (Oxley Lane) rain gauge was indicative of rain that fell over the western parts of Tamworth in the vicinity of Westdale and Tamworth Airport and that the rainfall was less intense over the catchments which drain behind the town levees (i.e. the recorded rainfall was not representative of that which fell over the study catchments).

## 2.5 Previous Studies

### 2.5.1. Floodplain Management Study - City of Tamworth (PPK, 1993)

A floodplain management study for Tamworth was prepared in 1993 by PPK Consultants Pty Ltd (PPK). The study contained recommendations for several flood modification measures which were aimed at reducing the impact of Peel River flooding on existing development. These measures included the upgrade of the CBD Levee and the then existing racecourse levee (now Taminda Levee) and the construction of the Western Levee (referred to previously as the Mathews to Thibault Levee).

Appendix A of the report dealt with the derivation of design inflow hydrographs for use in hydraulic modelling. Such hydrographs were required for inflows on the Peel and Cockburn Rivers, and Goonoo Goonoo and Timbumburi Creeks for frequencies of 20, 50 and 100 year ARI, as well as the PMF.

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[18] Note that due to problems with the AWS rain gauge, BOM advised that only a daily total is available for this event.

Appendix B of the report dealt with hydraulic modelling which was used to simulate the flows and water levels along the four major streams: the Peel and Cockburn Rivers, and Timbumburi and Goonoo Goonoo Creeks, as well as the part of the flow from the Peel River which is diverted into the Wallamore Anabranch near Taminda.

The hydrologic and hydraulic components of the floodplain management study were undertaken by Lyall & Macoun Consulting Engineers (LMCE). The FPLAIN hydraulic modelling software developed by LMCE was used for undertaking the hydraulic modelling component of the study.

### 2.5.2. Hydraulic Modelling of Peel River at Tamworth (LACE, 2006)

This report dealt with the upgrade of the FPLAIN hydraulic model which was originally set up as part of PPK (1993) and subsequently updated as part of several site specific studies which were undertaken by LMCE and Lyall & Associates Consulting Water Engineers (LACE) on behalf of TRC prior to 2006. The analysis modelled the floodplains of the Peel and Cockburn Rivers for about 6 km upstream of their junction to the east of Tamworth and continued to a point about 2 km downstream of the junction of the Peel River and the Wallamore Anabranch. Timbumburi Creek was also modelled for a distance of 6 km upstream of the West Tamworth Barraba Railway crossing. The modelling also included the assessment of a flood with an ARI of 200 years. The 2006 study report contains the most up to date information on flood behaviour on the Peel River and its major tributaries at Tamworth.

**Figure 2.10** shows the layout of the updated FPLAIN model which was developed as part of LACE (2006), whilst **Figure 2.11** shows peak flood levels for the 20, 100 and 200 year ARI design flood events, as well as the PMF, for those FPLAIN model nodes which border the town levees.

Design water surface profiles along the line of the town levees for the 20, 100 and 200 year ARI design flood events, as well as the PMF, are shown on **Figures 2.2**, **2.5** and **2.8**.

**Table 2.3** gives peak flows which were extracted from the FPLAIN model developed as part of LACE (2006). Included in **Table 2.3** are the corresponding heights on the town gauge for the various design floods.

**TABLE 2.3**
**DESIGN PEAK FLOWS**
**PEEL RIVER AND GOONOO GOONOO CREEK[19]**
**($m^3/s$)**

| Stream | Location | Design Flood Event | | | |
|---|---|---|---|---|---|
| | | 20 year ARI | 100 year ARI | 200 year ARI | PMF |
| Peel River | Bridge Street crossing of Peel River floodplain | 1,790 | 3,420 | 4,800 | 12,845 |
| Goonoo Goonoo Creek | Upstream boundary of FPLAIN model | 585 | 1,305 | 1,650 | 5,904 |

19 Source of Data: FPLAIN model input and output files generated during preparation of LACE (2006)

**Table 2.4** gives the duration design water levels in the Peel River remain above the critical 4 m trigger level on the town gauge. It is noted that the duration water levels remain elevated above the critical 4 m trigger level are generally consistent with historic flood data (refer **Table B2** in **Appendix B** for details), however, water levels have historically remained above the critical trigger level for periods exceeding 30 hours when the water level in the Peel River has exceeded 7 m on the town gauge (i.e. for those historic floods which have an ARI greater than 20 years).

**TABLE 2.4**
**DURATION WATER LEVEL REMAINS ABOVE CRITICAL RL 4 m TRIGGER LEVEL DESIGN FLOOD EVENTS[20]**

| Design Flood Event | Maximum Water Level on Town Gauge (m) | Duration Water Level Remains above 4 m on Town Gauge (hours) |
|---|---|---|
| 20 year ARI | 6.84 | 28.5 |
| 100 year ARI | 8.14 | 29 |
| 200 year ARI | 8.94 | 30 |

### 2.5.3. Flood Protection Levees or Alternatives for Mathews Street to Thibault Street and Ebsworth Street West Tamworth (WP, 1998)

This report presents the findings of an investigation which was undertaken into alternative routes for the Western Levee, as well as works required to protect several residential properties which are located in the Ebsworth Street area from Peel River flooding.

In regard the investigation into the Western Levee, the XP-RAFTS rainfall/runoff modelling software was used to analyse the capacity of pipes required to convey runoff from the upper reaches of the catchment directly to Barnes Gully and the volume of stormwater which would need to be temporarily stored directly behind the levee during periods of elevated tailwater conditions.

The following initial loss and continuing loss rate were adopted for generating flows in the various drainage lines which discharge to Barnes Gully:

- Pervious Area: Initial Loss = 10 mm; Continuing Loss = Between 0 and 3 mm/hr
- Impervious Area: Initial Loss = 1 mm; Continuing Loss = 0 mm/hr

The study showed that the maximum 100 year ARI ponding level behind the then proposed Western Levee would be RL 379.1 m, which is 500 mm below the floor level of No. 160 Goonoo Goonoo Road[21]. The study found that of the 80,000 $m^3$ of available flood storage behind the Western Levee, only 56,600 $m^3$ would be utilised during the occurrence of a valley-wide flood which resulted in the closure of the flood gates for a period of 24 hours (deemed to be the worst case scenario).

[20] Source of Data: FPLAIN model output files generated during preparation of LACE (2006)
[21] WP (1998) states that the floor level of this residence is the lowest of all the properties protected by the Western Levee.

# 3 HYDROLOGIC MODEL DEVELOPMENT AND TESTING

## 3.1 Selection of Hydrologic Modelling Approach

For hydrologic modelling in the present investigation, the practical choice is between the models known as DRAINS, RAFTS, RORB and WBNM, and any of these would be suitable. Whilst there is little to choose technically between these models, DRAINS has been developed primarily for use in modelling the passage of a flood wave through urban catchments, whilst RAFTS, RORB and WBNM have been widely used in the preparation of rural flood studies.

The DRAINS and RAFTS models were used in the present investigation to generate discharge hydrographs from urban and rural areas, respectively, as this combined approach was considered to provide a more accurate representation of the rainfall runoff process in the catchments which drain behind the town levees.

## 3.2 Brief Review of DRAINS Modelling Approach

The main hydrologic model in DRAINS is based on the ILSAX program (O'Loughlin, 1993), which in turn is based on the American ILLUDAS program. The ILSAX hydrological model uses time-area calculations and Horton infiltration procedures to calculate sub-area discharge hydrographs that are assumed to enter the drainage system, subject to constraints imposed by the entrance and conveyance capacity of the system. DRAINS is able to calculate hydraulic grade lines throughout a drainage network, enabling users to analyse the magnitude of overflows and stored water for established drainage systems[22].

The time-area method utilised in the ILSAX hydrological model is a form of catchment routing model in which a hyetograph of rainfall is combined with a time-area diagram to produce a flow hydrograph. The procedure effectively divides a catchment into a number of equal sub-areas, and superimposes the individual flows from these sub-areas, allowing for time lags depending on their distance from the outlet. In the ILSAX hydrological model, the time-area diagram is considered to be a triangular shape, with the increase in area per time step being constant.

The ILSAX hydrological model uses the depression storage (or initial loss) model for rainfall applied to impervious surfaces and the Horton infiltration model for rainfall applied to pervious surfaces. Horton's equation is the most common relationship for describing infiltration capacity in a soil. It describes the decrease in capacity as more water is absorbed by the soil, and has the form:

$$f = f_c + (f_0 - f_c).e^{-kt}$$

where:
- $f$ is the infiltration capacity (mm/h) at time $t$;
- $f_0$ and $f_c$ are the initial and final constant rates of infiltration (mm/h);
- $k$ is a shape factor (fixed at a value of 2 /h in ILSAX); and
- $t$ is the time from the start of rainfall (h).

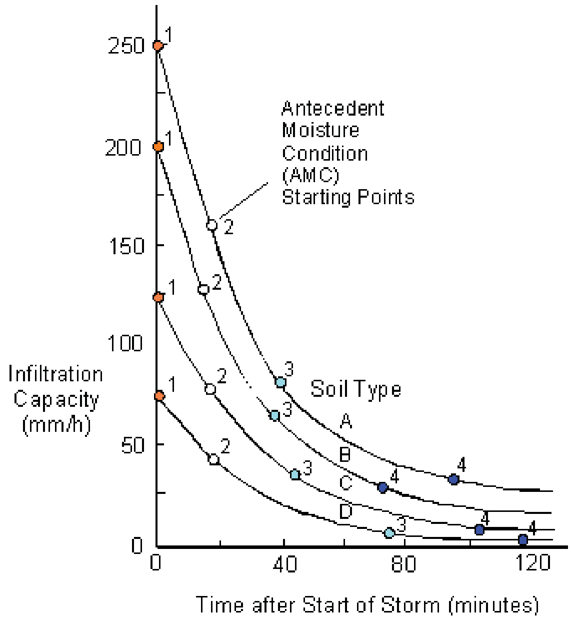
The soil type specified in ILSAX determines values for $f_0$ and $f_c$. Soil types used in the ILSAX hydrological model follow the U.S. Soil Conservation Service system adopted in the ILLUDAS

[22] Note that this capability within DRAINS was not utilised as part of this present investigation, as the TUFLOW software was used for this purpose.

model from which ILSAX was developed. There are four soil types involving different infiltration characteristics:

- Type 1 (or A) low runoff potential, high infiltration rates (sand and gravels),
- Type 2 (or B) moderate infiltration rates and moderately well-drained,
- Type 3 (or C) slow infiltration rates (may have layers that impede downward movement of water),
- Type 4 (or D) soils with high runoff potential, very slow infiltration rates (consisting of clays with a permanent high water table and a high swelling potential).

Users can specify a number between 1 and 4. ILSAX will interpolate between the standard infiltration factors applying to values of 1, 2, 3 or 4. The infiltration curves for these standard soil types are presented in the adjacent illustration.




Antecedent rainfall is the rainfall that occurs prior to the start of a storm event. It increases soil moisture levels and affects rates of infiltration into the soil.

The Antecedent Moisture Condition (AMC) is a parameter used in the loss calculations of the ILSAX hydrological model to specify the wetness or dryness of a catchment at the start of a storm. It is used to set the starting levels for infiltration relationships, and can have a significant effect on the flowrates generated by ILSAX.

An AMC number corresponds to a starting point on an infiltration curve, as shown in the above illustration. The curve defines the rate at which rainwater can penetrate into the soil. During a storm event, this will decrease, due to the soil becoming wetter, soil swelling and other effects. In research on ILSAX and related models, it has proved to be reasonably accurate to relate the AMC value of 1 to 4 to the rainfall in the previous 5 days, as follows:

| Number | Description | Total rainfall in 5 days preceding the storm (mm) |
|---|---|---|
| 1 | Completely dry | 0 |
| 2 | Rather dry | 0 to 12.5 |
| 3 | Rather wet | 12.5 to 25 |
| 4 | Saturated | Over 25 |

Details on model parameters which were adopted for the present investigation are given in **Section 3.5.2**.

### 3.3 Brief Review of RAFTS Modelling Approach

The RAFTS software converts storm rainfall to discharge hydrographs using a procedure known as runoff-routing and envisages the catchment to be comprised of a series of concentrated storages which represent sub-catchments defined on watershed lines, plus concentrated special storages which could simulate flood storage areas.

Each sub-catchment model is represented by a series of ten non-linear concentrated cascading storages. Within RAFTS each of the subareas in a sub-catchment is treated as a concentrated storage with a storage-discharge relation:

$$S = k(Q)Q \qquad (C3.1)$$

where $$k(Q) = BQ^n \qquad (C3.2)$$

The parameters n and B represent the catchment non-linearity and sub-catchment storage delay coefficient respectively and roughly equate to the parameters m and k in RORB.

The storage delay coefficient B is either directly input for each sub-catchment or estimated from equation C3.3 which was derived by Aitken (1975).

$$Bav = 0.285\ A^{0.52}\ (1+U)^{-1.97} Sc^{-0.50} \qquad (C3.3)$$

Where:

- Bav = mean value of coefficient B for each sub-catchment;
- A = sub-catchment area ($km^2$);
- U = fraction of catchment that is urbanised (where U = 1.0, the catchment is fully urbanised and when U = 0.0, the catchment is completely rural); and
- Sc = main drainage slope of the sub-catchment (%).

Reduced values of B tend to indicate steeply rising flood hydrographs where times of concentration are relatively short. **Table 3.1** summarises the effects of urbanisation and catchment slope on the storage delay parameter B for a catchment with a unit area of 1 $km^2$. Note that slopes of less than 2 per cent are characteristic of the urbanised areas which lie behind the Taminda Levee, whilst slopes of between 10 and 20 per cent are characteristic of the wooded slopes which lie to the north of Tamworth in the catchment which drains behind the CBD Levee.

**TABLE 3.1**
**VALUES OF STORAGE DELAY COEFFICIENT B[23]**
**AS A FUNCTION OF URBANISATION AND MAIN DRAINAGE SLOPE**

| Degree of Urbanisation | Main Drainage Slope (%) | | | |
|---|---|---|---|---|
| | 2 | 5 | 10 | 20 |
| 0 | 0.202 | 0.127 | 0.090 | 0.064 |
| 0.25 | 0.130 | 0.082 | 0.058 | 0.041 |
| 0.5 | 0.091 | 0.057 | 0.041 | 0.029 |
| 0.75 | 0.067 | 0.042 | 0.030 | 0.021 |
| 1 | 0.051 | 0.033 | 0.023 | 0.016 |

23 Refer equation C3.3 for derivation of the storage delay coefficient values

In order to take account of the lag effects of different surface treatments on surface water hydrology, an additional empirical parameter was added to the RAFTS code by the software developers to take pervious sub-catchment roughness into account. The parameter PERN is input as a Mannings 'n' representation of the average sub-catchment roughness. The storage delay coefficient is then modified in accordance with the following:

| PERN or Mannings 'n' value | Multiplication Factor |
|---|---|
| 0.010 | 0.4 |
| 0.015 | 0.5 |
| 0.025 | 1.0 |
| 0.1 | 3.0 |

From the above values of PERN, it can be seen that the adoption of a value equal to 0.025 will give a multiplication factor of 1.0 for B. This value of PERN is commonly adopted by practitioners when describing the "roughness" of urbanised catchments. The higher value of PERN given above (i.e. the value of 0.1) may be considered representative of densely vegetated areas and as such will result in a multiplier of 3.0 being applied to the computed value of B.

Details on model parameters which were adopted for the present investigation are given in **Section 3.5.2**.

### 3.4 Hydrologic Model Setup

There are three primary land-use types present within the study area: the urbanised parts of Tamworth, as well as the relatively steep wooded hills and cleared pastoral land which lie to its north. In order to best represent the rainfall-runoff process from these three land-use types, the RAFTS modelling approach was used for those catchment which lie upstream of the densely populated parts of Tamworth, whilst the DRAINS modelling approach was used for the remainder.

**Figure 3.1** shows the layout of the various sub-catchments which comprise the hydrologic model for the study area. Note that the individual sub-catchments have been shaded to separately identify those areas where the RAFTS and DRAINS modelling approaches were applied.

Careful consideration was given to the definition of the sub-catchments which comprise the hydrologic model to ensure peak flows at various flow control structures were properly assessed. In addition to using the available contour data, the location of kerb inlet pits in the urbanised parts of Tamworth was also taken into consideration when deriving the boundaries of the various sub-catchments. Percentages of impervious area were assessed using the aerial photography and cadastral boundary data.

Whilst the primary function of the hydrologic model was to generate discharge hydrographs for input to the TUFLOW hydraulic model, it was necessary to incorporate a number of individual reaches linking the various sub-catchments in the model. The outlets of the sub-catchments which lie outside the extent of the TUFLOW model boundary were linked as follows:

- The simple lag approach was adopted between each sub-catchment modelled in RAFTS. The lag time between the outlet of each sub-catchment was assumed to be equal to the distance along the main drainage path divided by an assumed flow velocity of 1 m/s.
- The linkages between the various sub-catchments modelled using the DRAINS approach were input as either piped or overland flow reaches. Data contained in TRC's GIS database was used to develop the piped network, whilst a simple lag approach was adopted for the overland flow reaches.

  Similar to the approach adopted for the RAFTS sub-catchments, the lag time between each sub-catchment was assumed to be equal to the distance along the assessed route of the overland path divided by an assumed flow velocity of 1 m/s.

  Note that flows that surcharge the stormwater drainage system tend to follow the prevailing grade in the road network, which does not necessarily follow the route of the stormwater drainage network.

Sub-catchment slopes used for input to the RAFTS component of the hydrologic model were derived using the vectored average slope approach, whilst the average sub-catchment slope computed using available contour data was used for input to the DRAINS component of the hydrologic model.

Aerial photography was used to assess the degree of urbanisation which is present in the study catchments.

## 3.5 Hydrologic Model Calibration

### 3.5.1. General

Quantitative information on historic flooding behind the town levees is limited to the intense storm which occurred over parts of Tamworth on 28 November 2008. This information was deemed to be of limited use in the model tuning process as the rainfall that was recorded at BoM's Tamworth (Oxley Lane) rain gauge was not considered by TRC to be representative of the rainfall which was experienced over the study catchments.

As there are no historic rainfall data which can be used to generate flows in the drainage system, the procedure adopted for the calibration of the hydrologic model involved a comparison of model results with peak flow estimates derived using the probabilistic rational method (PRM), procedures for which are set out in Australian Rainfall & Runoff (IEAust, 1998).

### 3.5.2. Hydrologic Model Parameters

As described in **Section 3.2**, the DRAINS component of the hydrologic model requires information on the soil type and losses to be applied to storm rainfall to determine the depth of runoff. Infiltration losses are of two types: initial loss arising from water which is held in depressions which must be filled before runoff commences, and a continuing loss rate which depends on the type of soil and the duration of the storm event.

The following DRAINS model parameters were adopted for generating flows from the urban portion of the study catchments:

- Soil Type = 3.0
- AMC = 3.0
- Paved area depression storage = 2.0 mm
- Grassed area depression storage = 10.0 mm
- Paved flow path roughness = 0.02
- Grassed flow path roughness = 0.07

As for the RAFTS component of the hydrologic model, a PERN value of 0.1 was applied to those sub-catchments which describe the relatively steep wooded hills which lie to the north of Tamworth, whilst a PERN value of 0.08 was applied to those sub-catchments which comprise both wooded and cleared pastoral land.

Continuing loss rates for impervious and pervious areas which were found to give good correspondence with rational method peak flow estimates were as follows:

| | Impervious Area | Pervious Area |
|---|---|---|
| Initial Loss | 2 | 15 |
| Continuing Loss | 0 | 2.5 |

It is noted that the RAFTS loss model parameters adopted as part of this present investigation are similar to those adopted for the design of the temporary storage area behind the Western Levee (refer **Section 2.5.3** for details).

### 3.5.3. Comparison of Peak Flow Estimates

**Table 3.2** gives a comparison of peak flow estimates derived using the PRM and those generated by the RAFTS component of the hydrologic model developed as part of the present investigation[24].

It is noted that the study catchments lie on the line which is used to determine the appropriate methodology for deriving the $C_{10}$ factor for use in the PRM (i.e. they lie on the line which links the townships of Ashford, Tamworth, Bathurst, Yass, Tumut and Jingellic). Depending on which methodology is used, a $C_{10}$ factor of either 0.25 or 0.4 can be derived for use in the PRM. **Table 3.2** therefore includes a comparison of the peak flows derived using both values of $C_{10}$.

By inspection of the peak flows given in **Table 3.2**, the hydrologic model developed as part of the present investigation generates flows which are higher than those derived using PRM for the 5 year ARI event, but generally fall within the range for the 20 and 100 year ARI events.

[24] Refer **Chapter 5** for background to the derivation of design storms used in the above analysis.

**TABLE 3.2**
**COMPARISON OF PEAK FLOW ESTIMATES**
**($m^3$/s)**

| Location[25] | Catchment Area ($km^2$) | 5 year ARI | | | 20 | | | 100 | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | PRM | | RAFTS | PRM | | RAFTS | PRM | | RAFTS |
| | | $C_{10}$=0.25 | $C_{10}$=0.4 | | $C_{10}$=0.25 | $C_{10}$=0.4 | | $C_{10}$=0.25 | $C_{10}$=0.4 | |
| CBD_PRM_001 | 2.05 | 3.9 | 6.25 | 8.4 | 8.1 | 12.9 | 13.5 | 18.0 | 28.7 | 20.7 |
| CBD_PRM_002 | 2.45 | 4.4 | 7.1 | 8.1 | 9.2 | 14.7 | 13.9 | 20.4 | 32.7 | 20.7 |
| CBD_PRM_003 | 0.72 | 1.7 | 2.7 | 3.4 | 3.5 | 5.5 | 5.4 | 7.7 | 12.3 | 8.2 |

[25] Refer **Figure 3.1** for location where peak flows estimates derived by the various methods are compared.

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# 4 HYDRAULIC MODEL DEVELOPMENT

## 4.1 Selection of Hydraulic Model

In the lower reaches of the drainage system which controls local catchment flooding behind the town levees, the pattern of flow required the implementation of a two dimensional model capable of analysing the time varying effects of flow in the stormwater drainage system versus water levels on the Peel River floodplain, the routing effects of flood storage which is present behind the three levees and the two-dimensional effects of flow in the urban parts of the study area.

The TUFLOW modelling software is one of only a few commercially available hydraulic models which contain all the features described above, and was therefore adopted for use in this present investigation.

## 4.2 TUFLOW Modelling Approach

TUFLOW is a true two-dimensional hydraulic model which does not rely on a prior knowledge of the pattern of flood flows in order to set up the various fluvial and weir type linkages which describe the passage of a flood wave through the system.

The basic equations of TUFLOW involve all of the terms of the St Venant equations of unsteady flow. Consequently the model is "fully dynamic" and once tuned will provide an accurate representation of existing flood behaviour in terms of depth, velocity and distribution of flow.

TUFLOW solves the equations of flow at each point of a rectangular grid system which represent overland flow on the floodplain and along streets. The choice of grid point spacing depends on the need to accurately represent features on the floodplain which influence hydraulic behaviour and flow patterns (e.g. buildings, streets, changes in channel and floodplain dimensions, hydraulic structures which influence flow patterns, etc).

Pipe drainage and channel systems can be modelled as one-dimensional elements embedded in the larger two-dimensional domain which typically represents the wider floodplain. Flows are able to move between the one and two-dimensional elements of the model depending on the capacity characteristics of the drainage system being modelled.

The TUFLOW software allows for the assessment of potential flood management measures, such as detention storage, increased channel and floodway dimensions, augmentation of culverts and bridge crossing dimensions, diversion banks and levee systems.

## 4.3 TUFLOW Model Setup

### 4.3.1. TUFLOW Model Structure

A single TUFLOW model was set up to represent the various fluvial and weir type linkages which comprise the drainage systems located directly behind the town levees. **Figures 4.1** to **4.3** show the layout of the various components which comprise the TUFLOW model for the town levees.

A 2 m grid spacing was found to provide the appropriate balance between the need to define features in the urban areas which are protected by the town levees versus model run times. Grid elevations were based on the DTM which was developed for the area directly behind the town levees.

Fourteen cross sections were used to define the inbank waterway area of the channels which drain parts of the catchment associated with the CBD Levee (3 off cross sections) and Taminda Levee (11 off cross sections).

The footprints of a large number of individual buildings located in the two-dimensional model domain were digitised and assigned an artificially high hydraulic roughness value which accounted for their blocking effect on flow whilst maintaining storage in the model.

Individual allotments where development is present were also digitised and assigned an artificially high hydraulic roughness value (although not as high as for individual buildings) to account for the reduction in conveyance capacity which will result from fences and other obstructions stored on these properties.

TRC's pit and pipe database was used to obtain details of the 937 individual pipe and culvert reaches and 934 pits which comprise the TUFLOW model. Design drawings provided by TRC were used to update the pit and pipe database where it differed from the information contained on the drawings.

Limited information was available on the invert levels of the individual pipes and culverts which comprise the local stormwater drainage system in Tamworth. An assumed cover of 700 mm was therefore assumed for those drainage elements where invert levels were not available. Further adjustments were made to the assumed invert levels where this approach resulted in a negatively graded reach of pipe or culvert.

Inlet pit capacity relationships were incorporated in the TUFLOW model based on a visual inspection of the existing stormwater drainage system.

It is noted that the presence of an existing flood retarding basin was identified north of the Main Western Railway Line on the drainage line which runs through Jaycees Park during the latter stages of the investigation. The attenuating effects of this basin on flows in this drainage line have therefore not been incorporated in the present study findings.

#### 4.3.2. Model Boundary Conditions

The locations where inflow hydrographs were input to the upstream limits of the two-dimensional model domain are shown on **Figures 4.1** to **4.3**.

Internal to the models, discharge hydrographs were input directly to a pit in the stormwater drainage system, or over individual regions called "Rain Boundaries". The Rain Boundaries act to "inject" flow into the one and two-dimensional domains of the TUFLOW model, firstly at a point which has the lowest elevation, and then progressively over the extent of the Rain Boundary as the grid in the two-dimensional model domain becomes wet as a result of overland flow. The extent of each Rain Boundary corresponds with the corresponding sub-catchment in the hydrologic model. The locations where inflow hydrographs were input to the TUFLOW model are shown on **Figure 4.1** to **4.3**.

The downstream boundary of the TUFLOW model comprised the following two types:

- **Outlet fitted with manually operated flood gate** - An option to run the model with the thirty-two manually operated flood gates in either a fully open or fully closed position was incorporated in the TUFLOW model at the locations identified by a blue inverted triangle on **Figures 4.1** to **4.3**.

- **Flap Gate or Uncontrolled Outlets** - A stage hydrograph representing varying tailwater conditions on the Peel River and its floodplain was applied to the downstream end of the drainage channel which controls flows discharging from the large box culvert system on the Taminda Levee (i.e. the system to which flood gates **FG32a** to **FG32c** are fitted). **Figure 4.4** shows the stage hydrographs which were used as input to the hydraulic model. Note that the ordinates of the stage hydrographs were extracted from Node 58 in the FPLAIN model (refer **Figure 2.10** and **2.11** for location) and then lowered by 900 mm to take account of the flood slope on the floodplain.

  Static water levels were adopted for the outlet of the various pressure lines which control flooding behind the CBD and Western Levee, as early runs of the hydraulic model showed that the hydraulic capacity of these drainage lines is not influenced by elevated tailwater conditions on the Peel River and its floodplain.

  The location of these boundary types are identified by stage hydrographs on **Figures 4.1** to **4.3**. **Section C1.3** in **Appendix C** deals with the derivation of coincident river levels which were adopted for modelling local catchment storm events of differing ARI.[26]

#### 4.3.3. Model Roughness

The main physical parameter for TUFLOW is the hydraulic roughness. Hydraulic roughness is required for each of the various types of surfaces comprising the overland flow paths, as well as for the cross sections representing the geometric characteristics of the channels. In addition to the energy lost by bed friction, obstructions to flow also dissipate energy by forcing water to change direction and velocity and by forming eddies. Hydraulic modelling traditionally represents all of these effects via the surface roughness parameter known as “Mannings n”. Flow in the piped system also requires an estimate of hydraulic roughness.

There are limited historic flood level data available to allow tuning of the model for roughness. Assessment of Mannings n values for sections of channel was relatively straightforward, as cross sections taken normal to the direction of flow have traditionally been used when modelling one-dimensional waterways. Channel roughness was estimated from site inspection, past experience and values contained in the engineering literature.

**Table 4.1** over presents the “best estimate” of hydraulic roughness values adopted for design purposes. The adoption of a value of 0.02 for the surfaces of roads, along with an adequate description of their widths and centreline and kerb elevations, allowed a reasonably accurate assessment of their conveyance capacity to be made. Similarly the high value of roughness adopted for buildings recognised that they completely blocked the flow but were capable of storing water when flooded.

[26] Note that for the case where the flood gates are assumed to be fully open (i.e. for the condition when the water level at the town gauge is below 4 m), elevated tailwater conditions were not applied to the outlet of these drainage lines.

**Figure 4.5** is a typical example of flow patterns derived from those values. This example applies for the 100 year ARI design flood and shows flows which surcharge the existing stormwater drainage network west of the Western Levee. The left hand side of the figure shows the roads and inter-allotment areas, as well as the outlines of buildings that have all been individually digitised in the model. The right hand side shows the resulting flow paths in the form of scaled velocity vectors and the depths of inundation.

The buildings with their high values of hydraulic roughness block the passage of flow, although the model recognises that they store floodwaters when inundated and therefore correctly accounts for flood storage. The flow is conveyed along the roads and through the open parts of the allotments.

Similar information to that shown on **Figure 4.5** may be presented at any location within the model domain (which is shown on **Figures 4.1** to **4.3**) and will be of assistance to TRC in assessing individual flooding problems behind the town levees.

**TABLE4.1**
**"BEST ESTIMATE" OF HYDRAULIC ROUGHNESS VALUES**
**ADOPTED FOR TUFLOW MODELLING**

| Surface Treatment | Mannings n Value |
|---|---|
| Asphalt or concrete road surface | 0.02 |
| Concrete Surfaces | 0.015 |
| Grass or Lawns | 0.045 |
| Trees | 0.08 |
| Allotments where fences and outbuildings are present. | 0.1 |
| Buildings | 10 |

# 5 DESIGN FLOOD ESTIMATION

## 5.1 Design Storms

### 5.1.1. Rainfall Intensity

The procedures used to obtain temporally and spatially accurate and consistent intensity-frequency-duration (IFD) design rainfall curves for the study area are presented in Book II of Australian Rainfall and Runoff (IEAust, 1998). Design storms for frequencies of 2, 5, 20, 100 and 200 year ARI were derived for storm durations ranging between 25 minutes and 36 hours. The procedure adopted was to generate IFD data for each catchment by using the relevant charts in Volume 2 of IEAust (1998). These charts included design rainfall isopleths, regional skewness and geographical factors.

### 5.1.2. Areal Reduction Factors

The rainfalls derived using the processes outlined in IEAust (1998) are applicable strictly to a point. In the case of a large catchment of over tens of square kilometres, it would not be realistic to assume that the same rainfall intensity can be maintained over a large area, an areal reduction factor is typically applied to obtain an intensity that is applicable over the entire area.

As the catchments which drain behind the town levees are relatively small, negligible reduction in intensity would result, thus the point values derived using the method outlined in **Section 5.1.1** were adopted.

### 5.1.3. Temporal Patterns

Temporal patterns for various zones in Australia are presented in IEAust (1998). These patterns are used in the conversion of a design rainfall depth with a specific ARI into a design flood of the same frequency. Patterns of average variability are assumed to provide the desired conversion. The patterns may be used for ARI's up to 500 years where the design rainfall data is extrapolated to this ARI.

The derivation of temporal patterns for design storms is discussed in Book II of IEAust (1998) and separate patterns are presented in Volume 2 for ARI < 30 years and ARI > 30 years. The second pattern is intended for use for rainfalls with ARI's up to 100 years, and to 500 years in those cases where the design rainfall data in Book II of IEAust (1998) are extrapolated to this ARI.

## 5.2 Probable Maximum Precipitation

Estimates of Probable Maximum Precipitation (PMP) were made using the Generalised Short Duration Method (GSDM) as described in BoM's update of Bulletin 53 (BoM, 2003). This method is appropriate for estimating extreme rainfall depths for catchments up to 1000 $km^2$ in area and storm durations up to 3 hours. The steps involved in assessing PMP for the study area are briefly as follows:

- Calculate PMP for a given duration and catchment area using depth-duration-area envelope curves derived from the highest recorded US and Australian rainfalls.

- Adjust the PMP estimate according to the percentages of the catchment which are meteorologically rough and smooth, and also according to elevation adjustment and moisture adjustment factors.
- Assess the design spatial distribution of rainfall using the distribution for convective storms based on US and world data, but modified in the light of Australian experience.
- Derive storm hyetographs using the temporal distribution contained in Bulletin 53, which is based on pluviographic traces recorded in major Australian storms.

**Figure 2.3** shows the location and orientation of the PMP ellipses which were used to derive the rainfall estimates for each individual catchment which drains to the CBD Levee[27]. Note that the PMP rainfall estimates derived for Ellipse A were applied to the sub-catchments which drain behind the Western Levee and Taminda Levee, whilst an average value of the PMP estimates derived for Ellipse A and Ellipse B were applied to the sub-catchments which drain behind the CBD Levee.

### 5.3 Derivation of Design Flood Hydrographs

The hydrologic model was run with the parameters set out in **Section 3.5.2** to obtain design hydrographs for input to the TUFLOW hydraulic model.

**Table 5.1** gives peak flows for design storms of 2, 5, 20, 100 and 200 year ARI, together with the PMF, at the locations where inflow hydrographs have been used as upstream boundary conditions in the TUFLOW model (refer **Figures 4.1** to **4.3** for location of inflow hydrographs).

The hydrologic modelling undertaken as part of the present investigation showed that there is the potential for stormwater to transfer into adjacent catchments, where it will approach the levees via alternative routes (i.e. it will follow a route which is different to that of the stormwater drainage network). This typically occurs where the prevailing grade in the local road network falls away from the line followed by a particular drainage line.

This feature is most prominent in the catchments which lie to the north (upslope) of the CBD Levee and has implications on the assessment of appropriate flood modification measures which are aimed at reducing the impact of flooding on existing development (e.g. upgrade requirements for the existing pressure lines).

As discussed in **Chapter 7**, it is recommended that before committing funds to upgrading the existing stormwater drainage system behind the town levees, the TUFLOW model developed as part of this present investigation be extended upstream into the urban parts of the catchment so that drainage patterns in these areas can be more accurately defined.

It is noted that peak flows generated by the hydrologic model for the PMF event are between 10-12 times those derived for the 100 year ARI event, after taking into account inter-catchment flow transfers which occur upstream of the TUFLOW model boundary (refer discussion above). This large multiplier is a function of the PMP estimates which are a similar multiple greater than the 100 year ARI design rainfall depths derived using the method outlined in **Section 5.1.1**.

[27] Note that the PMP ellipses for the Western Levee and Taminda Levee are not shown on their respective catchment plan as the subcatchments generally lie wholly within Ellipse A.

**TABLE 5.1**
**PEAK FLOW RATES[28]**
**($m^3/s$)**

| Catchment | Location | Identifier[29] | Hydrological Model Link Number | Flood Event Years ARI | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | 2 | 5 | 20 | 100 | 200 | PMF |
| CBD Levee | Northern Side of O'Connell Street and Marius Street Intersection | CBD_F1 | OF_CBD07 | 7.0 [30] | 11.0 [180] | 17.9 [180] | 26.6 [180] | 31.4 [180] | 307 [90] |
| | Upstream of Macquarie Street | CBD_F2 | OF_CBD11b | 6.5 [30] | 9.2 [180] | 14.8 [180] | 21.7 [120] | 25.4 [120] | 69 [30] |
| | Bourke Street | CBD_F3 | OF_CBD55 | 0.3 [25] | 0.4 [60] | 0.8 [25] | 1.2 [25] | 1.5 [25] | 222 [90] |
| | Brisbane Street | CBD_F4 | OF_CBD00a | 0.9 [25] | 1.5 [60] | 2.9 [25] | 4.4 [25] | 5.3 [25] | 31 [15] |
| | Fitzroy Street | CBD_F5 | OF_CBD04b | 3.3 [25] | 5.8 [25] | 11.5 [25] | 15.4 [25] | 17.1 [25] | 102 [30] |
| | White Street | CBD_F6 | OF_CBD23 | 3.1 [30] | 5.7 [60] | 9.8 [25] | 15 [25] | 17.6 [25] | 95 [45] |
| | Murray Street | CBD_F7 | OF_CBD02 | 0.8 [25] | 1.4 [60] | 3.2 [180] | 6.0 [180] | 7.5 [180] | 93 [90] |
| Western Levee | George Street | WES_F1 | OF_WES02 | 0.1 [60] | 0.1 [120] | 0.9 [25] | 3.5 [25] | 4.5 [25] | 50 [15] |
| | Kent Street | WES_F2 | OF_WES01 | 1.6 [60] | 3.1 [60] | 4.6 [60] | 6.2 [25] | 7.0 [25] | 45 [15] |
| | Margaret Street | WES_F3 | OF_WES09 | 4.4 [60] | 8.3 [60] | 12.6 [25] | 14.1 [25] | 14.8 [25] | 59 [30] |
| | Lydia Street | WES_F4 | OF_WES07 | 0.4 [60] | 0.7 [60] | 2.1 [60] | 7.4 [25] | 10.1 [25] | 100 [30] |
| Taminda Levee | Western Side of Tamworth Showground | TAM_F1 | OF_TAM15 | 3.9 [60] | 6.2 [60] | 10.6 [25] | 14.7 [25] | 16 [25] | 103 [45] |
| | Intersection of Gunnedah Road and Main Northern Railway | TAM_F2 | OF_TAM23 | 4.7 [60] | 7.8 [60] | 12.9 [60] | 19.2 [60] | 21.5 [60] | 57 [15] |
| | Intersection of Gunnedah Road and Showground Road | TAM_F3 | OF_TAM02 | 0.5 [60] | 0.9 [60] | 1.6 [25] | 2.6 [25] | 3.3 [60] | 145 [45] |

[28] Values in [ ] represent critical design storm duration in minutes.
[29] Refer **Figures 4.1** to **4.3** for location of inflow hydrographs.

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# 6 HYDRAULIC MODELLING OF DESIGN FLOODS

## 6.1 Presentation of Results

**Figures 6.1** to **6.15** show the indicative extents and depths of inundation behind the town levees for the full range of design storm events. Note that **Sheets 1**, **2** and **3** in the series show modelled flood behaviour behind the CBD Levee, Western Levee and Taminda Levee, respectively. For each design storm event, the results of modelling the flood gates in both their fully open and fully closed positions are shown. Also included in the set of figures is an afflux diagram showing the difference in peak flood levels between the flood gates fully open and flood gates fully closed condition.

**Tables 6.1** and **6.2** located at the end of this Chapter give the maximum elevation to which local catchment runoff ponds behind the town levees for the flood gates in their fully open and fully closed positions, respectively. The values given in **Tables 6.1** and **6.2** are shown on the respective figures described above.

Note that the impact of potential blockages in the existing stormwater drainage network, especially the pressure lines which discharge directly to the Peel River floodplain from behind the CBD and Western Levees, has not been assessed as part of this present investigation.

## 6.2 Discussion of Results

### 6.2.1. General

The depths and extents of inundation shown on **Figures 6.1** to **6.15** represent the upper envelope of flooding and incorporate the results of modelling local catchment storms of between 25 minutes and 36 hours duration, the latter being the maximum duration over which water levels in the Peel River remain above the critical trigger level of 4 m on the town gauge. In the case of the modelling undertaken for the flood gates in their fully closed position, the 36 hour storm was found to be critical for maximising peak flood levels behind the town levees, since this storm has the greatest volume of runoff associated with it. Whilst there is the potential for river levels to remain elevated above the critical trigger level of 4 m on the town gauge for this period of time (refer **Table B2** in **Appendix B** for analysis of historic flood behaviour), TRC advised that it has successfully opened several flood gates located along the CBD Levee during periods of elevated water levels in the Peel River in order to relieve flooding behind the levee. The results of the hydraulic modelling undertaken for the case where the flood gates are in their fully closed position therefore represent a worst case scenario in terms of the maximum depth and extent of inundation which could be experienced behind the levee for each ARI storm.

### 6.2.2. CBD Levee

The impact of local catchment flooding on existing development located directly behind the CBD Levee is significantly greater than that associated with either the Western Levee or Taminda Levee. The key reasons for this are:

i. The relatively high density of commercial development which is located directly behind the levee bank, especially to the east of Darling Street. The highly developed nature of this area means that there is limited space available for the temporary storage of local catchment runoff which under certain storm and gate operation conditions is forced to pond behind the levee bank.

Whilst there is a relatively large area of open space land to the south of Kable Street in the vicinity of Bicentennial Park, the modelling shows that the greatest depths of inundation will occur in the road reserve immediately east of Brisbane Street, along the frontage of existing commercial development.

ii. The relatively large catchment which lies to the north of the CBD Levee, runoff from which surcharges the existing pressure lines for storms between 5 and 20 year ARI. **Table 6.3** located at the end of this Chapter shows that major surcharge of the existing pressure lines will occur during a local catchment storm with an ARI of 20 years.

A major source of local catchment runoff behind the CBD Levee is the surcharge of the O'Connell Street pressure line. Investigations show that the eastern arm of this pressure line, the inlet of which is located in Angora Park is subject to a back pressure which forms upstream of the confluence with the western arm. The capacity of the eastern of the pressure line reduces significantly as soon as major flows are experienced in the adjoin arm of the pressure line, the inlet of which is located at a higher elevation west of O'Connell Street and north of Marius Street. This finding lead to the formulation of a flood modification measure involving the separation of the two arms of the existing pressure line (refer **Chapter 7** for further details).

The modelling also shows that in addition to contributing to the flooding problem directly behind the levee, stormwater which surcharges the existing pressure lines will impact on commercial developed located remote from the ponding areas. The extent of the hydraulic model developed as part of this present investigation was determined at the commencement of the study based on an initial assessment of drainage patterns in the area. Subsequent, more detailed investigations have shown that there is the potential for a redistribution of flow to occur in the area which lies to the north of the Main Railway Line, which has the potential to alter flooding patterns in the CBD of Tamworth. It is recommended that two-dimensional flood modelling similar to that undertaken as part of the present investigation be undertaken to more accurately define flooding patterns in the areas which lie upslope of the backwater zone behind the CBD levee.

iii. The relatively large urbanised catchment which lies downslope of the inlets to the existing pressure lines, runoff from which must be temporarily stored behind the levee bank when the flood gates are closed. The modelling carried out as part of this present investigation shows that depths of inundation behind the CBD Levee for any given ARI storm will increase the longer the flood gates remain closed (i.e. the longer the flood gates remain closed, the greater the volume of runoff which will temporarily pond behind the levee).

The modelling carried out as part of the present investigation confirmed that the capacity of the existing pressure lines is not impacted by elevated tailwater levels in the Peel River.

#### 6.2.3. Western Levee

The findings of the present investigation generally confirm those of the previous study by Willing & Partners (WP, 1998), albeit that the previous study adopted a slightly different approach to deriving the peak 100 year ARI flood level directly behind the levee.

The peak 100 year ARI flood level derived as part of the present investigation is 80 mm higher than that derived by the previous study (i.e. RL 379.18 m AHD versus RL 379.1 m AHD). The level would still provide a 420 mm freeboard to the lowest floor level in existing residential development located behind the Western Levee.[30]

Existing development is shown to be impacted by overland flow which approaches the temporary flood storage area located directly behind the levee from the west. Whilst depths of overland flow are generally less than 300 mm, greater depths of inundation were found to occur in several properties located to the north of Margaret Street. It is recommended that a more detailed overland flooding investigation be undertaken in the catchment which drains behind the Western Levee in order to more accurately define flooding patterns in this area, given its close proximity to the upstream boundary of the hydraulic model.

It is noted that the crest of the Western Levee will be overtopped by a local catchment PMF event for the case when the flood gates are in their fully open position.

#### 6.2.4. Taminda Levee

Flooding resulting from local catchment runoff ponding directly behind the Taminda Levee is generally confined to presently undeveloped land. The exception is existing development located immediately to the west of Crown Street, which is impacted by local catchment runoff which initially ponds on the road reserve and then flows through these properties when the flood gates located along Ebsworth Street between the Main Western Railway and Plain Street are closed. Depths of inundation in property located to the west of Crown Street are generally less than 50 mm, although ponding to a depth of about 300 mm is shown to occur in one property.

The hydraulic capacity of the stormwater drainage line which runs along Belmore Street is impacted by the closure of the flood gates. These impacts are shown to extend possibly as far upstream as far as Avro Street. Given that there was limited information available on the existing stormwater drainage network in the study area, it is recommended that more detailed modelling be undertaken in order to more accurately define flooding behaviour along the route of this drainage line. A similar comment applies to the whole of the area which lies upstream of the Taminda Levee in the two-dimensional model domain as the flooding patterns in this area are considered approximate only and should therefore be treated with caution.

### 6.3 Flood Planning Level

Pending the completion of a future *Flood and Floodplain Risk Management Study* for the catchments which drain behind the town levees, an interim Flood Planning Level (FPL) was derived based on the 100 year flood level plus 500 mm of freeboard. **Figure 6.16** (**Sheets 1 to 3**) shows the provisional extent of the FPL behind the town levees for the flood gates in both their fully open and fully closed positions.[31].

Note that whilst the interim FPL shown on **Figure 6.16** is based on the upper envelope of potential flooding for the 100 year ARI event, it does not take account of the impact potential blockages of the existing stormwater drainage system will have on local catchment flood behaviour directly behind the town levees. It is recommended that consideration be given to the

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[30] WP (1998) states that the floor level of No. 160 Goonoo Goonoo Road, at an elevation of RL 391.6 m AHD, is the lowest set property behind the Western Levee

[31] Note that the FPL derived as part of the present investigation is limited to the ponding areas which are located directly behind the town levees and does not include the overland flow which approaches these areas from the upslope catchment.

impact a potential blockage of the existing stormwater drainage system will have on ponding levels behind the town levees prior to adopting a final FPL.

### 6.4 Provisional Flood Hazard

Flood hazard categories may be assigned to flood affected areas in accordance with the procedures outlined in the Floodplain Development Manual, 2005.

Flood prone areas may be provisionally categorised into *Low Hazard* and *High Hazard* areas depending on the depth of inundation and flow velocity. Flood depths as high as a metre, in the absence of any significant flow velocity, could be considered to represent Low Hazard conditions. Similarly, areas of flow velocities up to 2.0 m/s, but with small flood depths could also represent Low Hazard conditions.

Provisional Flood Hazard diagrams for the 100 year ARI flood in the areas protected by the town levees based on Diagram L2 of the *Floodplain Development Manual, 2005* are presented in **Figures 6.17** and **Figure 6.18** (**Sheets 1 to 3**) for the cases where the flood gates are in their fully open and fully closed positions, respectively.

In regard flood behaviour behind the CBD Levee (refer **Sheet 1** in the series), high hazard flooding is generally confined to the Peel Street road reserve west of Darling Street and Viaduct Park for the case where the flood gates are in their fully open position. High hazard flooding conditions increase significantly behind the CBD Levee for the case where the flood gates are in their fully closed position, with commercial property located along Peel Street and Kable Street inundated to depths exceeding 1 m.

In regard flood behaviour behind the Western Levee (refer **Sheet 2** in the series), high hazard flooding is generally confined to the temporary flood storage area located north of George Street and does not extent into private property Park for the case where the flood gates are in their fully open position. In the case where the flood gates are in their fully closed position, the associated increase in the depth of ponding behind the levee results in high hazard conditions extending into the rear of several residential properties located along Goonoo Goonoo Road.

In regard flood behaviour behind the Taminda Levee (refer **Sheet 3** in the series), high hazard flooding is generally confined to the temporary flood storage area which is located to the south of the Taminda Racecourse and the drainage channels which feed into it for the case where the flood gates are in their fully open position. In the case where the flood gates are in their fully closed position, the associated increase in the depth of ponding along the northern side of the levee results in high hazard conditions forming in the temporary flood storage area located west of Jewry Street. The closure of the gates also affects the hydraulic capacity of the stormwater drainage system which controls ponding levels in Belmore Street. As a result, high hazard conditions also form at the location of the sag in the road.

The Flood Hazard assessment presented herein is based on considerations of depth and velocity of flow and is *provisional* only. As noted in the *Floodplain Development Manual, 2005,* other considerations such as rate of rise of floodwaters and access to high ground for evacuation from the floodplain should also be taken into consideration before a final determination of Flood Hazard can be made. These factors are normally taken into account in the *Floodplain Risk Management Study* for the study area.

## 6.5 Climate Change Considerations

### 6.5.1. General

The weight of scientific evidence shows that climate change will have adverse impacts on sea levels and rainfall intensities. The significance of these effects on flood behaviour will vary depending on geographic location and local topographic conditions. Climate change impacts on flood producing rainfall events show a trend for larger scale storms and resulting depths of rainfall to increase.

CSIRO prepared reports for the NSW Government on the impacts of climate change on rainfall intensities in the major river basins in the state (CSIRO, 2007). In the Namoi River catchment, the 40 year ARI, 1 day rainfall was predicted to change by about +3 per cent by 2030 and by about +10 per cent by 2070.

### 6.5.2. Sensitivity to Increased Rainfall Intensities

For the purposes of the investigation, the design flood envelopes which have been developed for the 200 year ARI events were adopted as being analogous to flooding which could be expected should present day 100 year ARI rainfall intensities increase by 10 per cent (i.e. the upper limit of potential rainfall increases predicted by CSIRO).

**Figures 6.19** and **6.20** (**Sheets 1 to 3**) show the impact an increase of 10 per cent in 100 year ARI rainfall intensities would have on flooding behaviour behind the town levees for the cases where the flood gates are in their fully open and fully closed positions, respectively..

In regard potential changes to flood behaviour behind the CBD Levee (refer **Sheet 1** in the series), depths of inundation directly behind the levee would be increased in the range 50-100 mm east of Brisbane Street and 200-300 mm west of Brisbane Street for the case where the flood gates are in their fully open position. For the case where the flood gates are in their fully closed position, the increase in ponding levels directly behind the levee would be more uniform and in the range 100-200 mm.

In regard potential changes to flood behaviour behind the Western Levee (refer **Sheet 2** in the series), depths of inundation would increase in the range 2-50 mm south of George Street and 200-300 mm north of George Street for the case where the flood gates are in their fully open position. Whilst not directly related to the Western Levee, the modelling shows that existing development west of Goonoo Goonoo Road and north of Margaret Street would be affected as a result of an increase in the rate of flow surcharging the existing stormwater drainage network. For the case where the flood gates are in their fully closed position, the increase in ponding levels directly behind the levee would be more uniform and in the range 50-100 mm.

In regard potential changes to flood behaviour behind the Taminda Levee (refer **Sheet 3** in the series), peak flood levels along Belmore Street would be increased in the range 50-100 mm and in the temporary flood storage area south of Tamworth Racecourse in the range 100-200 mm for the case where the flood gates are in their fully open position. For the case where the flood gates are in their fully closed position, the increase in ponding levels in Belmore Street would be in the range 10-20 mm. [Note, findings may be subject to change following completion of final runs of the hydraulic model].

**TABLE 6.1**
**MAXIMUM PONDING LEVELS BEHIND TOWN LEVEES**
**FLOOD GATES FULLY OPEN**

| Flood Event Years ARI | CBD Levee | | Western Levee | | Taminda Levee | | |
|---|---|---|---|---|---|---|---|
| | Peel Street at Viaduct Park | Kable Avenue immediately east of Brisbane Street | North of George Street Pedestrian Walkway | South of George Street Pedestrian Walkway | Belmore Street Cul-de-sac | Adjacent to Britten Road and Jewry Street Intersection | Between Jewry Street and Tamworth Racecourse |
| 2 | 374.97 | 375.96 | 376.51 | 377.60 | 372.80 | 370.70 | 370.80 |
| 5 | 375.17 | 376.14 | 376.85 | 377.94 | 372.97 | 370.79 | 371.06 |
| 20 | 375.81 | 376.42 | 377.30 | 378.15 | 373.29 | 370.90 | 371.41 |
| 100 | 376.27 | 376.72 | 377.81 | 378.28 | 373.53 | 371.01 | 371.83 |
| 200 | 376.44 | 376.78 | 378.03 | 378.31 | 373.62 | 371.08 | 371.99 |
| PMF | 379.19 | 379.70 | 382.06 | 382.07 | 374.63 | 373.92 | 373.85 |

**TABLE 6.2**
**MAXIMUM PONDING LEVELS BEHIND TOWN LEVEES**
**FLOOD GATES FULLY CLOSED**

| Flood Event Years ARI | CBD Levee | | Western Levee | | Taminda Levee | | |
|---|---|---|---|---|---|---|---|
| | Peel Street at Viaduct Park | Kable Avenue immediately east of Brisbane Street | North of George Street Pedestrian Walkway | South of George Street Pedestrian Walkway | Belmore Street Cul-de-sac | Adjacent to Britten Road and Jewry Street Intersection | Between Jewry Street and Tamworth Racecourse |
| 2 | 376.26 | 376.66 | 378.05 | 378.05 | 373.79 | 372.79 | 370.81 |
| 5 | 376.46 | 376.71 | 378.39 | 378.39 | 373.87 | 372.88 | 371.06 |
| 20 | 376.82 | 376.82 | 378.74 | 378.74 | 373.94 | 373.01 | 372.09 |
| 100 | 377.33 | 377.33 | 379.18 | 379.18 | 373.99 | 373.13 | 372.30 |
| 200 | 377.46 | 377.46 | 379.25 | 379.25 | 374.01 | 373.13 | 372.30 |
| PMF | 379.19 | 379.70 | 382.06 | 382.06 | 374.63 | 373.92 | 373.85 |

## TABLE 6.3
## PEAK FLOWS(1) SURCHARGING EXISTING STORMWATER DRAINAGE NETWORK NORTH OF CBD LEVEE
## ($m^3/s$)

| Location | Flood Event Years ARI | | | | | |
|---|---|---|---|---|---|---|
| | 2 | 5 | 20 | 100 | 200 | PMF |
| Marius Street immediately west of O'Connell Street | 0 | 0 | 3.6 [180] | 11.4 [180] | 15.9 [180] | 240 [90] |
| Marius Street immediately east of O'Connell Street | 0 | 0 | 4.4 [180] | 10.7 [180] | 11.5 [60] | 17 [180] |
| Macquarie Street north of Main Northern Railway Line | 0 | 0.4 [180] | 3.9 [180] | 7.2 [180] | 1.9 [120] | 47 [30] |
| Darling Street immediately south of Marius Street | 0 | 0.2 [60] | 0.5 [60] | 0.7 [60] | 0.9 [25] | 35 [90] |
| Bourke Street immediately south of Marius Street | 0 | 0 | 0 | 0 | 0 | 36 [90] |
| Brisbane Street immediately south of Marius Street | 0 | 0 | 1.4 [25] | 2.7 [25] | 3.3 [25] | 75 [90] |
| Fitzroy Street immediately south of Marius Street | 0 | 0.5 [60] | 4.0 [25] | 7.5 [60] | 10.6 [60] | 80 [30] |
| White Street immediately south of Marius Street | 0 | 0.8 [60] | 1.7 [60] | 3.1 [25] | 4.1 [25] | 55 [30] |
| Hill Street immediately north of Marius Street | 0 | 0.4 [25] | 1.4 [25] | 3.7 [25] | 4.6 [25] | 20 [15] |
| Peel Street immediately east of Roderick Street | 0 | 0.2 [25] | 1.9 [180] | 4.8 [180] | 6.3 [180] | 65 [90] |

1. Numbers in [ ] represent critical storm duration in minutes.

# 7 POTENTIAL FLOOD MODIFICATION MEASURES

## 7.1 General

The current assessment of the benefits which can be achieved through the implementation of a selected number of potential flood modification measures is limited to the CBD Levee. This is because in the case of the Western and Taminda Levees, flood related damages in existing development are generally a result of overland flow which approaches the temporary ponding areas from upslope areas. Whilst flood damages in these areas could be reduced through the upgrade of the existing stormwater drainage network, the assessment of such measures is beyond the scope of the present investigation.

Furthermore, whilst flood related damages will be incurred in existing development as a direct result of local catchment runoff ponding behind the Western Levee, this appears to have been the intent of the original design, since the peak 100 years ARI flood level derived as part of this present investigation is only 80 mm higher than that derived in WP (1998). It is noted that flood related damages in existing development will be limited to external damages and clean up costs for floods up to 100 year ARI given than no above floor inundation will occur in property protected by the levee[32].

## 7.2 CBD Levee Measures

The following flood modification measures aimed at reducing the impact of flooding on existing development located directly behind the CBD Levee were assessed. Note that the benefits of each option in terms of a reduction in peak 100 year ARI flood levels at several key locations behind the CBD Levee are summarised in **Table 7.1**.

- **Option 1 –** As previously mentioned, the existing pressure line is surcharged for design storms between 5 and 20 year ARI. The modelling also showed that the eastern arm of the pressure line is subject to a back pressure which forms upstream of the confluence with the western arm.

  The option, denoted the **O'Connell Street Pressure Line Upgrade**, would involve the separation of the two arms of the existing pressure line and the construction of a new pressure line as shown in **Figure 7.1**. It would be necessary to construct an earth embankment across Macquarie Street to intercept flow which surcharges the existing culvert under the same named street immediately east of Angora Park. A major intake structure is shown in the south western of the adjacent playing field around which an earth embankment would need to be constructed. A short section of the existing concrete lined channel to the north of the playing field would also need to be rebuilt in order to redirect flows toward the inlet of the new pressure line.

  The impact the implementation of this potential measure would have on peak 100 year ARI flood levels behind the CBD Levee are shown on **Figures 7.2** and **7.3** for the cases where the flood gates are in their fully open and fully closed positions, respectively. The modelling shows that this option would alleviate flooding in several commercial properties

[32] Note that this finding relates only to local catchment runoff ponding directly behind the levee and that the flood damages assessment undertaken as part of this present investigation (refer **Appendix D** for details) indicates that existing development may be subject to above floor inundation as a result of overland flow which approaches the levee from upslope areas.

**TABLE 7.1**
**IMPACT OF VARIOUS FLOOD MITIGATION OPTIONS ON PEAK 100 YEAR ARI FLOOD LEVELS BEHIND CBD LEVEE**

| Option | Flood Gates Fully Open | | | | Flood Gates Fully Closed | | | |
|---|---|---|---|---|---|---|---|---|
| | Peel Street at Viaduct Park | | Kable Avenue Immediately South of Brisbane Street | | Peel Street at Viaduct Park | | Kable Avenue Immediately South of Brisbane Street | |
| | Peak Flood Level (m AHD) | Reduction when Compared to Present Day Conditions (m) | Peak Flood Level (m AHD) | Reduction when Compared to Present Day Conditions (m) | Peak Flood Level (m AHD) | Reduction when Compared to Present Day Conditions (m) | Peak Flood Level (m AHD) | Reduction when Compared to Present Day Conditions (m) |
| Present Day Conditions | 376.27 | - | 376.72 | - | 377.33 | - | 377.33 | - |
| Option 1 | 375.77 | -0.50 | 376.72 | 0.00 | 377.00 | -0.33 | 377.00 | -0.33 |
| Option 2 | 376.27 | 0.00 | 376.54 | -0.17 | 377.33 | 0.00 | 377.33 | 0.00 |
| Option 3 | 376.27 | 0.00 | 376.68 | -0.04 | 377.27 | -0.06 | 377.27 | -0.06 |
| Option 4 | 376.27 | 0.00 | 376.70 | -0.02 | 373.33 | 0.00 | 373.33 | 0.00 |
| Option 5 | 375.76 | -0.50 | 376.38 | -0.34 | 376.76 | -0.57 | 376.76 | -0.57 |
| Option 6 | 375.77 | -0.50 | 376.55 | -0.17 | Final runs of TUFLOW model yet to be completed | | | |
| Option 7A | Final runs of TUFLOW model yet to be completed | | | | 376.67 | -0.66 | 376.81 | -0.52 |
| Option 7B | Final runs of TUFLOW model yet to be completed | | | | 376.43 | -0.90 | 376.77 | -0.56 |

located between Brisbane Street and Darling Street for the case where the flood gatesare in their fully open position. Whilst this potential measure would also reduce peak flood levels by about 710 mm for the case where the flood gates are in their fully closed position, the reduction in the overall extent of land impacted by the 100 year ARI flood under these conditions is not significant.

- **Option 2 –** This option, denoted the **Fitzroy Street Pressure Line Upgrade**, would involve the duplication of the existing 1500 mm diameter pressure line which runs along Fitzroy Street from the Main Western Railway to the Peel River. The works would also involve improvements to the inlet arrangement located on the northern side of the Main Western Railway. It would also be necessary to install a new length of 1500 mm diameter pipe through the existing levee bank, either by excavating a trench or by thrust boring methods. **Figure 7.4** shows the route of the new pressure line and the works which would be required to improve the inlet conditions.

  The impact the implementation of this potential measure would have on peak 100 year ARI flood levels behind the CBD Levee are shown on **Figures 7.5** and **7.6** for the cases where the flood gates are in their fully open and fully closed positions, respectively. The modelling shows that the benefits of this option in terms of a reduction in peak flood levels would generally be confined to commercial property located between Brisbane Street and Fitzroy Street for the case where the flood gates are in their fully open position. Whilst the reduction in peak flood levels is more wide spread for the case where the flood gates are in their fully closed position, the impact in terms of a reduction in peak flood levels is relatively minor, in the range 10-100 mm. [Note, findings may be subject to change following completion of final runs of the hydraulic model].

- **Option 3 –** This option, denoted the **White Street Pressure Line Upgrade**, would involve the duplication of the existing 1200 mm diameter pressure line which runs along White Street from the Main Western Railway to the Peel River. **Figure 7.4** shows the route of the new pressure line. Unlike the Fitzroy Street pressure line option, it would not be necessary to install a new length of 1200 mm diameter pipe through the existing levee bank, since an unused section of pipe of the same diameter already exists. The main concern with this option is the ability to pressurise the new drainage line given the limited opportunities available for improving the inlet capacity north of the Main Western Railway Line.

  The impact the implementation of this potential measure would have on peak 100 year ARI flood levels behind the CBD Levee are shown on **Figures 7.7** and **7.8** for the cases where the flood gates are in their fully open and fully closed positions, respectively. The modelling shows that the benefits of this option in terms of a reduction in peak flood levels are more widespread than for Option 2, but are generally confined to commercial property located between Bourke Street and White Street for the case where the flood gates are in their fully open position. Whilst the reduction in peak flood levels is more wide spread for the case where the flood gates are in their fully closed position, the impact in terms of a reduction in peak flood levels is relatively minor, in the range 10-100 mm. [Note, findings may be subject to change following completion of final runs of the hydraulic model].

- **Option 4 –** This option, **Jaycees Park and Prince of Wales Park Trunk Drainage Upgrade**, would involve improvements to the existing trunk drainage line which runs through the two aforementioned parks, as shown on **Figure 7.9**. The works would require a new major intake structure to be built in the south-west corner of land which is owned by TRC, in combination with the construction of a new reinforced block wall which would act to intercept and pond overland flow above the inlet to the system. The works would also involve the reshape of land to the west of the major intake structure in order to redirect overland flow.

  The impact the implementation of this potential measure would have on peak 100 year ARI flood levels behind the CBD Levee are shown on **Figures 7.10** and **7.11** for the cases where the flood gates are in their fully open and fully closed positions, respectively. The modelling shows that the benefits of this option in terms of a reduction in peak flood levels would generally be confined to commercial property located between Brisbane Street and Murray Street for the case where the flood gates are in their fully open position. Whilst the reduction in peak flood levels is more wide spread for the case where the flood gates are in their fully closed position, the impact in terms of a reduction in peak flood levels is relatively minor, in the range 10-100 mm.

- **Option 5**: This option comprises the four potential measures described above (i.e. Options 1 to 4 combined). The impact the implementation of all four potential measures would have on peak 100 year ARI flood levels behind the CBD Levee are shown on **Figures 7.12** and **7.13** for the cases where the flood gates are in their fully open and fully closed positions, respectively.

  The modelling shows that the implementation of all four potential measures would have a significant impact on the extent of inundation behind the CBD Levee for the case where the flood gates are in their fully open position. Whilst the benefits in terms of a reduction in peak 100 year ARI flood levels for the case where the flood gates are in their fully closed position is greater than 200 mm, the overall extent of land impacted by the 100 year ARI flood under these conditions is still not significant.

- **Option 6**: This option comprises only the O'Connell Street and Fitzroy Street pressure the line upgraded (i.e. Options 1 to 2 combined). The impact the implementation of these two potential measures would have on peak 100 year ARI flood levels behind the CBD Levee are shown on **Figures 7.14** and **7.15** for the cases where the flood gates are in their fully open and fully closed positions, respectively.

  The modelling shows that the implementation of these two measures would have a reduce the impact of flooding on existing development located to the west of Fitzroy Street for the case where the flood gates are in their fully open position. Similar to the findings of modelling the two potential measures individually, the benefits in terms of a reduction in peak 100 year ARI flood levels for the case where the flood gates are in their fully closed position, whilst greater than 200 mm, does not significantly reduce the overall extent of land impacted by the 100 year ARI flood.

- **Option 7**: This option would involve the installation of two sets of low head/high volume flood evacuation pump stations, the first of which could be located on the southern side of Peel Street immediately north of Darling Street and the second on the southern side of Kable Street immediately east of Brisbane Street.

Two runs of the hydraulic model were carried out to assess the reduction in peak flood levels which could be achieved by installing three pump stations, the first incorporating a combined lift capacity of 9 $m^3/s$ (i.e. three pump stations each with a maximum lifting capacity of 3 $m^3/s$) (denoted herein as **Option 7A**) and the second incorporating a combined lift capacity of 18 $m^3/s$ (i.e. three pump stations each with a maximum lifting capacity of 6 $m^3/s$) (denoted herein as **Option 7B**). The location of the three pumping stations is shown on **Figures 7.16** to **7.19**.

**Figures 7.16** and **7.17** show the impact the implementation of **Option 7A** would have on peak 100 year ARI flood levels behind the CBD levee for the cases where the flood gates are in their fully open and fully closed positions, respectively, whilst **Figures 7.18** and **7.19** show the impact the implementation of **Option 7B** would have on peak 100 year ARI flood levels behind the CBD levee for the cases where the flood gates are in their fully open and fully closed positions, respectively.

## 7.3 Flood Damages Assessment

Estimation of urban flood damages was carried out for property located behind the CBD and Western Levees. Urban flood damages for property located behind the Taminda Levee were not estimated because existing development in this area is not impacted by local catchment runoff which ponds directly behind the levee (i.e. the subject of this present investigation), rather it is impacted by overland flow which approaches the levee from the surrounding local catchment.

The objective of this analysis was to allow a "broad brush" economic assessment of various flood modification measures. Damages from floods ranging between the 2 year ARI and PMF events were assessed. Details of the flood damages assessment carried out as part of the present investigation are contained in **Appendix D** of the report.

**Table 7.2** summarises the _Present Worth Values_ of damages behind the CBD Levee for all flood events up to the 100 year ARI for both present day conditions and after implementation of the works described under Option 5. The benefits of each scheme in terms of the _Present Worth Value_ of damages saved by the implementation are also presented in **Table 7.2**.

**TABLE 7.2**
**PRESENT WORTH VALUE OF DAMAGES DIRECTLY BEHIND CBD LEVEE**
**ALL FLOODS UP TO 100 YEAR ARI AND FOR AN ECONOMIC LIFE OF 20 YEARS[33]**
**$\$ \times 10^6$**

| Position of Flood Gates | Present Worth Value of Damages | | Present Worth Value of Damages Saved by Implementing Option 5 Works |
|---|---|---|---|
| | Present Day Conditions | Post Implementation of Option 5 Works | |
| Fully Open | 3.58 | 1.64 | 1.94 |
| Fully Closed | 42.45 | 17.04 | 25.41 |

[33] Present Worth Values are for a discount rate of 7 per cent.

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## 8 REFERENCES

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PPK (PPK Consultants Pty Ltd), 1993, ***“Floodplain Management Study City of Tamworth”***

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## 9 DEFINITIONS

The *Floodplain Development Manual, 2005* contains a number of definitions which are relevant to the discussion of planning measures to assist in the management of development in the floodplain. These definitions include:

| TERM | DEFINITION |
|---|---|
| **Australian Height Datum (AHD)** | A common national surface level datum approximately corresponding to mean sea level. |
| **Average Recurrence Interval (ARI)** | The average period in years between the occurrence of a flood of a particular magnitude or greater. In a long period of say 1,000 years, a flood equivalent to or greater than a 100 year ARI event would occur 10 times. The 100 year ARI flood has a 1% chance (i.e. a one-in-100 chance) of occurrence in any one year.<br>In relation to the economic life of structures, there is a 23% chance of the 100 year ARI event or greater occurring in a 30 year period, a 50% change of occurrence in a 70 year period and a 60% chance within a 100 year period. |
| **Flood Liable Land** | Is synonymous with flood prone land (i.e.) land susceptible to flooding by the Probable Maximum Flood event. Note that the term flood liable land now covers the whole of the floodplain. |
| **Floodplain** | Area of land which is subject to inundation by floods up to and including the Extreme Flood event, that is, flood prone land. |
| **Floodplain Risk Management Plan** | A management plan developed in accordance with the principles and guidelines in the Floodplain Development Manual, 2005. Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives. |
| **Flood Planning Levels (FPLs)** | Are the combinations of flood levels and freeboards selected for planning purposes, as determined in floodplain risk management studies and incorporated in floodplain risk management plans. |
| **Flood Storage Areas** | Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas. |

| TERM | DEFINITION |
|---|---|
| **Floodway Areas** | Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow, or a significant increase in flood levels. |
| **Freeboard** | A factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. It is usually expressed as the difference in height between the adopted Flood Planning Level and the peak height of the flood used to determine the flood planning level. Freeboard provides a factor of safety to compensate for uncertainties in the estimation of flood levels across the floodplain, such and wave action, localised hydraulic behaviour and impacts that are specific event related, such as levee and embankment settlement, and other effects such as "greenhouse" and climate change. Freeboard is included in the flood planning level. |
| **High Hazard** | Where land in the event of a 100 year ARI flood is subject to a combination of flood water velocities and depths greater than the following combinations: 2 metres per second with shallow depth of flood water depths greater than 0.8 metres in depth with low velocity. Damage to structures is possible and wading would be unsafe for able bodied adults. |
| **Low Hazard** | Where land may be affected by floodway or flood storage subject to a combination of floodwater velocities less than 2 metres per second with shallow depth or flood water depths less than 0.8 metres with low velocity. Nuisance damage to structures is possible and able bodied adults would have little difficulty wading. |
| **Mainstream flooding** | Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam. |
| **Mathematical/computer models** | The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain. |
| **Merit approach** | The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State's rivers and floodplains. |

| TERM | DEFINITION |
|---|---|
| **Peak discharge** | The maximum discharge occurring during a flood event. |
| **Probable Maximum Flood (PMF)** | The largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with the PMF event should be addressed in a floodplain risk management study. |
| **Probability** | A statistical measure of the expected chance of flooding (see annual exceedance probability). |
| **Risk** | Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment. |
| **Runoff** | The amount of rainfall which actually ends up as stream flow, also known as rainfall excess. |

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# APPENDIX A

# FLOOD QUESTIONNAIRE

# Tamworth City Levees Internal Drainage Study

# Community Questionnaire

## FOR RESIDENTS AND BUSINESSES

This questionnaire is part of the Tamworth City Levees Internal Drainage Study which is currently being undertaken by Consultants on behalf of Tamworth Regional Council. It will help us to collect information on local catchment flooding which may have occurred behind the three main levees which have been constructed primarily to protect existing development from Peel River flooding.

Flooding behind the three main levees in Tamworth can occur when local catchment runoff cannot discharge quickly enough to the adjacent river or creek via the existing stormwater drainage system. When this occurs, stormwater will temporarily pond behind the levees where it can, under certain circumstances, cause flooding in existing development.

If you have observed stormwater ponding behind the levee which lies adjacent to your property, then you are invited to complete this questionnaire. Please return your completed questionnaire in the reply paid envelope provided before **FRIDAY 14 OCTOBER 2011**. No postage stamp is required. If you have misplaced the supplied envelope or wish to send an additional submission the address is:

Lyall & Associates Consulting Water Engineers
Reply Paid 85163
NORTH SYDNEY NSW 2060

Your Address____________________________________________________

Name of Business/Organisation (if applicable)____________________________________

### *About your property*

**1. Please tick as appropriate:**

a. I am a resident ❑
b. I operate a business ❑
c. I own the property ❑
d. I rent the property ❑
e. Other (please specify______________) ❑

**2. How long have you owned, lived at or conducted business at this address?**

a. Less than 1 year ❑
b. 1 year to 5 years ❑
c. 5 years to 20 years ❑
d. More than 20 years (… years) ❑

**3. What is your property?**

a. Shop/Retail ❑
b. Factory/Industrial ❑
c. House ❑
d. Villa/Townhouse ❑
e. Unit/Flat/Apartment ❑
f. Vacant land ❑
g. Other (__________________________) ❑

### *Your flood experience*

**4. Have you ever observed stormwater ponding behind the levee which has been constructed adjacent to your property?**

a. Yes ❑
b. No ❑

If yes, what were the dates when this occurred (day/month/year if possible)?

________________________________

________________________________

________________________________

**5. If your answer to Question 4 was yes, were water levels in the adjacent river/ creek elevated at the time stormwater was observed to pond behind the levee?**

a. Yes ❑
b. No ❑

If yes, for what events (your answer should correspond to one or more of the dates given under Question 4)?

________________________________

________________________________

________________________________

6. **If your answer to Question 4 was yes, if contacted by the Consultant could you describe over the telephone the extent to which stormwater ponded behind the levee?**
   a. Yes ❑
   b. No ❑

   If yes, please complete Question 10.

7. **Have you ever experienced flooding in your property as a result of stormwater ponding behind the levee?**
   a. Yes ❑
   b. No ❑

   If yes, what were the dates when this occurred (day/month/year if possible)?

   ______________________________

   ______________________________

8. **Have you ever experienced above-floor flooding in your property as a result of stormwater ponding behind the levee?**
   a. Yes ❑
   b. No ❑

   If yes, what were the dates when this occurred (day/month/year if possible)?

   ______________________________

   ______________________________

9. **If your answer was yes to either Question 7 or 8, do you have any identifiable flood marks which our surveyor could level?**
   a. Yes ❑
   b. No ❑

   If yes, what are the dates that these flood marks were recorded (day/month/year if possible)?

   ______________________________

   ______________________________

   If yes, please complete Question 10.

10. **If you wish us to contact you so you can provide further information, please provide your details below:**

    Name: ______________________________

    Address: ______________________________

    ______________________________

    Phone (Home) ______________________________

    Best time to call is ______________________________

    Fax No. ______________________________

    Email: ______________________________

# COMMENTS

*Please write any comments that you might have here:*

______________________________________________

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***Who can I contact for further information?***


**Lyall & Associates**
**Consulting Water Engineers**
Scott Button
Phone: 9929 4466
Email: tamworth@lyallandassociates.com.au

# APPENDIX B

# HISTORIC FLOOD LEVEL AND RAINFALL DATA

## TABLE B1
## HISTORIC WATER LEVEL AND RAINFALL DATA AT TAMWORTH

| Date | Peak Water Level (m) | Date | Peak Water Level (m) | Date | Peak Water Level (m) |
|---|---|---|---|---|---|
| 14/01/1910 | 6.93 | 24/02/1955 | 6.27 | 10/07/1978 | 5.32 |
| 19/06/1930 | 4.11 | 25/02/1955 | 7.16 | 28/01/1984 | 5.36 |
| 2/10/1933 | 4.72 | 24/10/1955 | 5.41 | 30/01/1984 | 6.63 |
| 1/09/1934 | 5.49 | 25/10/1955 | 5.79 | 22/02/1984 | 4.74 |
| 17/10/1934 | 5.18 | 10/02/1956 | 6.17 | 28/07/1984 | 5.12 |
| 15/01/1935 | 4.42 | 2/05/1956 | 4.88 | 8/11/1984 | 5.18 |
| 4/08/1936 | 4.88 | 5/05/1956 | 5.64 | 12/11/1984 | 4.39 |
| 22/08/1937 | 4.57 | 24/05/1956 | 4.42 | 10/12/1985 | 4.6 |
| 10/07/1942 | 5.61 | 25/06/1956 | 4.88 | 31/07/1989 | 4.7 |
| 11/07/1942 | 5.64 | 26/06/1956 | 4.17 | 27/07/1990 | 4.55 |
| 23/06/1945 | 5.03 | 13/07/1956 | 4.01 | 4/08/1990 | 5.5 |
| 3/09/1947 | 4.01 | 1/08/1956 | 4.17 | 1/09/1990 | 4.1 |
| 3/12/1947 | 4.19 | 4/10/1958 | 5.64 | 24/01/1991 | 4.2 |
| 26/12/1947 | 4.42 | 11/10/1958 | 4.42 | 27/01/1991 | 5.28 |
| 2/01/1948 | 5.33 | 25/12/1958 | 5.03 | 9/02/1992 | 4.95 |
| 10/09/1949 | 4.42 | 13/01/1962 | 6.86 | 25/01/1996 | 4.6 |
| 14/09/1949 | 4.19 | 19/05/1963 | 4.88 | 26/01/1996 | 4.5 |
| 5/04/1950 | 4.11 | 9/06/1963 | 4.01 | 14/02/1997 | 5.2 |
| 28/06/1950 | 5.03 | 14/01/1964 | 5.64 | 23/06/1998 | 4.2 |
| 22/07/1950 | 5.69 | 12/01/1968 | 5.79 | 21/07/1998 | 5.36 |
| 21/10/1950 | 6.1 | 2/02/1971 | 6.35 | 22/07/1998 | 5.61 |
| 26/10/1950 | 5.49 | 11/02/1971 | 5.89 | 28/07/1998 | 5.99 |
| 22/11/1950 | 5.46 | 8/01/1974 | 5.18 | 8/08/1998 | 5.28 |
| 18/06/1952 | 5.56 | 23/06/1975 | 4.6 | 6/09/1998 | 5.28 |
| 7/08/1952 | 4.27 | 24/01/1976 | 6.27 | 19/11/2000 | 5.43 |
| 13/08/1952 | 5.64 | 27/02/1976 | 4.11 | 20/11/2000 | 6.23 |
| 20/08/1952 | 5.64 | 4/03/1977 | 4.3 | 17/01/2004 | 5.74 |
| 15/09/1954 | 5.18 | 6/03/1977 | 5.1 | 29/11/2008 | 6 |
| 19/10/1954 | 4.95 | 7/04/1977 | 4.88 | | |
| 10/11/1954 | 5.18 | 15/05/1977 | 5.15 | | |

1. Refer footnotes over.

**Footnotes to Table B1**

1. NoW advised that peak water levels at the site of the current stream gauge in the Peel River at Tamworth (GS 419009) for the period 1910 to 1993 were sourced from BoM.
2. Peak water levels on the Peel River at Tamworth stream gauge (GS 419009) for the period 1993 to date were taken directly from NoW's web site.
3. Gauge zero on the Peel River at Tamworth stream gauge (GS 419009) is 371.057 m AHD.
4. Only days when the water level exceeded 4 m in the Peel River at the site of the current stream gauge are listed in the above table.

## TABLE B2
## DURATION WATER LEVEL REMAINED ABOVE CRITICAL RL 4 m TRIGGER LEVEL HISTORIC FLOOD EVENTS (1993 TO DATE)

| Date of Historic Flood | Date/Time(24hr) Water Level First Rose Above RL 4 m | Maximum Water Level Recorded on Tamworth Gauge (m) | Date/Time(24hr) Water Level First Dropped Below RL 4 m | Duration Water Level Above RL 4 m (hours) |
|---|---|---|---|---|
| January 1996 | 25/01/1996 1900 Hours | 4.6 | 26/01/1996 0515 Hours | 10.25 |
| February 1997 | 13/02/1997 1945 Hours | 5.2 | 14/02/1997 0145 Hours | 6 |
| June 1998 | 23/06/98 0330 Hours | 4.2 | 23/06/1998 0600 Hours | 2.5 |
| July 1998 | 21/07/1998 0315 Hours | 5.61 | 22/07/1998 1300 Hours | 33.75 |
| | 28/07/1998 0345 Hours | 5.99 | 29/07/1998 1230 Hours | 32.75 |
| August 1998 | 08/08/1998 0830 Hours | 5.28 | 09/08/1998 0330 Hours | 19 |
| September 1998 | 05/09/1998 1700 Hours | 5.28 | 06/09/1998 1500 Hours | 22 |
| November 2000 | 18/11/2000 2130 Hours | 5.43 | 19/11/2000 1645 Hours | 19.25 |
| | 20/11/2000 0600 Hours | 6.23 | 21/11/2000 1200 Hours | 30 |
| January 2004 | 17/01/04 0645 Hours | 5.74 | 17/01/2004 2345 Hours | 17 |
| November 2008 | 29/11/2008 0100 Hours | 6.0 | 29/11/2008 2300 Hours | 22 |

Source of Data: NoW's Tamworth at Peel River Stream Gauge (GS 419009).

## TABLE B3
## APPROXIMATE ARI OF RAINFALL RECORDED AROUND TIME OF HISTORIC FLOODS AT TAMWORTH[(1)]

| Date of Historic Flood | Peel River Flood | | Storm Duration (Hours) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Peak Water Level (m) | Approx. ARI | 0.5 | 1 | 1.5 | 2 | 3 | 4 | 6 | 12 | 24 | 36 |
| October 1958 | 5.64 | 5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | 1 - 2 | <1 |
| December 1958 | 5.03 | <5 | 2 - 5 | 2 - 5 | 2 - 5 | 2 - 5 | 2 - 5 | 1 - 2 | 1 - 2 | <1 | <1 | <1 |
| January 1962 | 6.86 | 20 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | 1 - 2 | 1 - 2 | 1 - 2 |
| May 1963 | 4.88 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | 1 - 2 | <1 | <1 | <1 |
| June 1963 | 4.01 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| January 1964 | 5.64 | 5 | <1 | <1 | <1 | 1 - 2 | 1 - 2 | 2 - 5 | 2 - 5 | 10 - 20 | 20 - 50 | 20 - 50 |
| January 1968 | 5.79 | 5 | <1 | <1 | <1 | <1 | 1 - 2 | 1 - 2 | 2 - 5 | 20 - 50 | 20 - 50 | 10 - 20 |
| February 1971 | 6.35 | 10 | <1 | <1 | 1 - 2 | 1 - 2 | 1 - 2 | 1 - 2 | <1 | 1 - 2 | 2 - 5 | 2 - 5 |
| January 1974 | 5.18 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | 1 - 2 | 2 - 5 | 2 - 5 | 2 - 5 |
| June 1975 | 4.6 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| January 1976 | 6.27 | 10 | 2 - 5 | 1 - 2 | 1 - 2 | 1 - 2 | <1 | <1 | 1 - 2 | 5 - 10 | 10 - 20 | 10 - 20 |
| February 1976 | 4.11 | <5 | <1 | <1 | <1 | <1 | 1 - 2 | 1 - 2 | 1 - 2 | 1 - 2 | <1 | <1 |
| March 1977 | 5.1 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| April 1977 | 4.88 | <5 | 2 - 5 | 1 - 2 | 2 - 5 | 5 - 10 | 10 - 20 | 20 - 50 | 20 - 50 | 5 - 10 | 2 - 5 | 1 - 2 |
| May 1977 | 5.15 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | 1 - 2 | 1 - 2 |
| July 1978 | 5.32 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| January 1984 | 6.63 | 15 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | 1 - 2 | 2 - 5 | 2 - 5 |
| February 1984 | 4.74 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |

1. Analysis relied upon rainfall data recorded at BoM's pluviographic stations located at Tamworth Airport (Station No. 055054) pre-1993 and Oxley Lane (Station No. 055327) post-1993.

**TABLE B3 (Cont'd)**
**APPROXIMATE ARI OF RAINFALL RECORDED AROUND TIME OF HISTORIC FLOODS AT TAMWORTH[(1)]**

| Date of Historic Flood | Peel River Flood | | Storm Duration (Hours) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Peak Water Level (m) | Approx. ARI | 0.5 | 1 | 1.5 | 2 | 3 | 4 | 6 | 12 | 24 | 36 |
| July 1984 | 5.12 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| November 1984 | 5.18 | <5 | 1 - 2 | 1 - 2 | <1 | 1 - 2 | 1 - 2 | 1 - 2 | 1 - 2 | <1 | <1 | <1 |
| December 1985 | 4.6 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| July 1989 | 4.7 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| July 1990 | 4.55 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| August 1990 | 5.5 | 5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| January 1991 | 5.28 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | 1 - 2 | 1 - 2 | 1 - 2 | 1 - 2 |
| February 1992 | 4.95 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| January 1996 | 4.6 | <5 | <1 | <1 | <1 | <1 | <1 | 1 - 2 | 1 - 2 | 1 - 2 | 1 - 2 | 1 - 2 |
| February 1997 | 5.2 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | 1 - 2 |
| June 1998 | 4.2 | <5 | <1 | <1 | <1 | <1 | 1 - 2 | 1 - 2 | 1 - 2 | 2 - 5 | 5 - 10 | 2 - 5 |
| July 1998 | 5.61 | 5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| August 1998 | 5.99 | 7 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| September 1998 | 5.28 | <5 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | 1 - 2 | 1 - 2 |
| November 2000 | 6.23 | 8 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 | <1 |
| January 2004 | 5.74 | 5 | 2 - 5 | 2 - 5 | 5 - 10 | 20 - 50 | 20 - 50 | 10 - 20 | 10 - 20 | 10 - 20 | 5 - 10 | 2 - 5 |
| November 2008 | 6 | 7 | >100 | >100 | >100 | >100 | >100 | 50 - 100 | 50 - 100 | >100 | >100 | >100 |

1. Analysis relied upon rainfall data recorded at BOM's pluviographic stations located at Tamworth Airport (Station No. 055054) pre-1993 and Oxley Lane (Station No. 055327) post-1993.

# APPENDIX C

# JOINT PROBABILITY CONSIDERATIONS OF COINCIDENT FLOODING

# TABLE OF CONTENTS

Page No

**C1. CONSIDERATIONS OF JOINT PROBABILITY OF COINCIDENT FLOODING ................ C1**

C1.1 General ........................................................................................................ C1
C1.2 Adopted Coincident Local Catchment and Peel River Flooding Conditions .......... C1
C1.3 Analysis of Historic River Level and Rainfall Data ................................................ C3

**C2. REFERENCES .......................................................................................................... C5**

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# C1. CONSIDERATIONS OF JOINT PROBABILITY OF COINCIDENT FLOODING

## C1.1 General

The coincident nature of local catchment and mainstream/riverine flooding is an important factor to consider in the design of flood protection levees, as elevated tailwater levels can impose a backwater effect on the local stormwater drainage system which in turn can exacerbate flooding conditions behind the levee.

In the case of the town levees, the time varying effects of elevated water levels in the Peel River on the hydraulic capacity of the local stormwater drainage system only need be considered when assessing the capacity of the large box culvert system which drains the temporary storage area located to the south of the Taminda Racecourse. This is because the outlet of this system has been fitted with a series of hinged flap gates (refer location of flood gates **FG32a**, **FG32b** and **FG32c** on **Figure 2.7** of the main report), the operation of which is dependent on the differential head across the structure. **Section C1.2** of this appendix deals with the coincident flooding conditions that were adopted for modelling flood behaviour on this drainage line[1].

The remainder of the drainage lines which control local catchment flooding behind the town levees (excluding the pressure lines) have been fitted with penstock type flood gates, the operational procedures for which dictate that they must be manually closed when the water level in the Peel River reaches 4 m on the town gauge. As a result, the hydraulic capacity of these drainage lines, and hence local catchment flood behaviour, is largely independent of water levels in the river (i.e. because runoff is forced to temporarily pond behind the levees until such time as water levels fall below the critical trigger level[2])

The joint probability of coincident local catchment and Peel River flooding is therefore primarily a function of the chance of water levels in the river being above 4 m at the time flood producing rain is experienced over the catchments which drain behind the town levees. Whilst it is beyond the scope of this present investigation to determine the joint probability of local catchment and Peel River flooding, an analysis of historic river level and rainfall data was undertaken to assess the likelihood of the flood gates being closed during local catchment storm events. The findings of this analysis are presented in **Section C1.3** of this appendix.

## C1.2 Adopted Coincident Local Catchment and Peel River Flooding Conditions

As mentioned, elevated tailwater levels in the Peel River will influence the hydraulic capacity of the large box culvert system which drains the temporary storage area located to the south of the Taminda Racecourse. IEAust (1998) suggests three alternative approaches to solving the definition of flood frequency in areas subject to combined flooding:

- Use of median values of variables other than rainfall depth;
- Derivation of design relations from comparison of values of the same probability obtained from frequency analyses of observed floods; and
- Joint probability analysis of the variables contributing to the flood.

---

[1] Note that the same approach was adopted for modelling the outlet of the various pressure lines which control runoff from behind parts of the CBD Levee and Western Levee, even though the hydraulic capacity of these drainage lines are not affected by elevated water levels in the Peel River.

[2] TRC advised that several of the flood gates located along the CBD Levee can be partially opened under certain river conditions in order to relieve flooding behind the levee.

The second method, flood frequency analysis, is not practicable in this present case due to the absence of historic flow data on the local catchments. The last method according to IEAust (1998) is theoretically superior to the others, but uncertainty at the low probabilities of interest due to the lack of definition of the tails of the several probability distributions involved limits its use.

The first method involves using an intermediate value rather than a maximum value of tailwater level when undertaking hydraulic modelling. This method requires the use of engineering judgement to produce reasonable results and was adopted in the present investigation[3].

**Table C1.1** shows the coincident flooding conditions which have been adopted for modelling local catchment flood events on those drainage lines which have not been fitted with a manually operated penstock gate[4].

Flooding conditions behind the Taminda Levee as a result of a Peel River event were also modelled using the combination of events shown in **Table C1.2**. The results of this modelling were also used in the derivation of flood envelopes for design storms of 20, 100 and 200 year ARI[5] in the case when the flood gates are assumed to be in their fully closed position (i.e. because for the case when the flood gates are in their .

**TABLE C1.1**
**ADOPTED COINCIDENT FLOODING CONDITIONS**
**FOR MODELLING LOCAL CATCHMENT FLOOD EVENTS**

| Design Rainfall on Local Catchment | Coincident Design Flood in Peel River |
|---|---|
| 2 | - |
| 5 | - |
| 20 | - |
| 100 | 20 |
| 200 | 20 |
| PMF | 100 |

[3] Whilst practical methods for estimating the joint probability of design coincident flows on paired streams have been recently developed for application in the United States (Kilgore et al (2010)), similar research has yet to be undertaken to confirm whether these methods are applicable to Australian conditions.

[4] Note that for the case where the flood gates are assumed to be fully open (i.e. for the condition when the water level at the town gauge is below 4 m), elevated tailwater conditions were not applied to the outlet of these drainage lines.

[5] Note that the PMF levels on the Peel River were not used to derive the flood envelope for this extreme event.

**TABLE C1.2**
**ADOPTED COINCIDENT FLOODING CONDITIONS**
**FOR MODELLING PEEL RIVER FLOOD EVENTS**

| Design Rainfall on Peel River Catchment | Coincident Design Rainfall on Local Catchment |
|---|---|
| 20 | 5 |
| 100 | 20 |
| 200 | 20 |

## C1.3 Analysis of Historic River Level and Rainfall Data

A review of the available survey shows that the critical trigger level of 4 m on the town gauge approximates bank full flow conditions in the Peel River at Tamworth. A review of the historic flood data for the Peel River (refer **Table B1** in **Appendix B**) shows that there has been about 70 independent flood events in the past 100 years when the water level in the river has peaked above 4 m on the town gauge, indicating that the river has reached bank full flow conditions about every 1.5 years on the average. This finding is consistent with other studies undertaken on river systems in Australia which found that bank full conditions are generally reach every 1.5 to 5 years on the average.

Whilst the data indicates that on a historical river level basis the flood gates would be closed on a relatively frequent basis, a review of the available pluviographic rainfall data which dates back to 1958 shows that there have been a limited number of occasions when intense rainfall has been recorded at Tamworth around the time of elevated water levels in the Peel River (refer **Table B3** in **Appendix B** for findings of analysis). This finding can be largely explained by the fact that water levels in the Peel River generally respond to more widespread rain rather than the very intense rainfall which results in flash flooding (and which is critical for maximising flow in the stormwater drainage lines which control flooding behind the town levees).

That said, the storm that occurred on 28 November 2008 demonstrated that very intense rainfall can occur over Tamworth in combination with elevated water levels in the Peel River (refer **Figure 2.9** in the main report which shows the recorded stage and rainfall data for this event). Whilst in this instance the intense burst of rainfall which caused flooding in parts of Tamworth occurred about 2 hours prior to water levels in the river exceeding the critical 4 m trigger level on the town gauge, flooding conditions behind the town levees may have been significantly worse had the burst occurred only a few short hours later in the storm.

Whilst isolated bursts of heavy rain can occur over Tamworth in the absence of elevated water levels in the Peel River (e.g. as a result of localised thunderstorm activity), the relatively low threshold which has been adopted for triggering a gate closure condition combined with the prolonged period over which water levels remain elevated in the river (i.e. generally between 20-30+ hours for major flood events on the Peel River), means that there is a reasonable chance in any one year that the penstock type flood gates will be closed at the time flood producing rainfall is experienced over the catchments which drain behind the town levees. That said, the severity of flooding experienced behind the town levees will be a function of the duration over which water levels remain above the critical trigger level of 4 m on the town gauge combined with the duration intense rainfall is experienced over the associated local catchment.

This finding will need to be taken into consideration when assessing potential flood modification measures which are aimed at mitigating the impact of local catchment flooding on existing development (refer **Chapter 7** of the main report for further details) and when setting appropriate Flood Planning Levels for future development on land located in the ponding zone of the town levees.

## C2. REFERENCES

Kilmore R., Thompson D. and Ford. D, September 2010, ***"Estimating Joint Probabilities of Design Coincident Flows at Stream Confluences"*** Research sponsored by the American Association of State Highway and Transportation Officials in cooperation with the Federal Highway Administration.

IEAust (The Institution of Engineers, Australia), 1998, ***"Australian Rainfall and Runoff – A Guide to Flood Estimation"***, Volumes 1 and 2.

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# APPENDIX D

# ASSESSMENT OF FLOOD DAMAGES

# TABLE OF CONTENTS

Page No

**SYNOPSIS** .......... **DS1**

**D1. INTRODUCTION AND SCOPE** .......... **D1**
- D1.1 Introduction .......... D1
- D1.2 Scope of Investigation .......... D1
- D1.3 Terminology .......... D1

**D2. DESCRIPTION OF APPROACH** .......... **D3**

**D3. SOURCES OF DATA** .......... **D5**
- D3.1 General .......... D5
- D3.2 Property Database .......... D5

**D4. RESIDENTIAL DAMAGES** .......... **D7**
- D4.1 Damage Functions .......... D7
- D4.2 Total Residential Damages .......... D8

**D5. COMMERCIAL AND INDUSTRIAL DAMAGES** .......... **D11**
- D5.1 Direct Commercial and Industrial Damages .......... D11
- D5.2 Indirect Commercial and Industrial Damages .......... D12
- D5.3 Total Commercial and Industrial Damages .......... D12

**D6. DAMAGES TO PUBLIC BUILDINGS** .......... **D15**
- D6.1 Direct Damages – Public Buildings .......... D15
- D6.2 Indirect Damages – Public Buildings .......... D15
- D6.3 Total Damages – Public Buildings .......... D15

**D7 SUMMARY OF TANGIBLE DAMAGES** .......... **D17**
- D7.1 Tangible Damages .......... D17
- D8.3 Definition of Terms .......... D17
- D8.4 Present Worth of Damages in Ponding Areas .......... D17
- D8.5 Sensitivity Analysis .......... D19
- D8.6 Potential Flood Modification Measures .......... D20

**D8. REFERENCES** .......... **D21**

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## SYNOPSIS

Estimation of urban flood damages was carried out for selected property located behind the CBD and Western Levees. Urban flood damages behind the Taminda Levee were not estimated because existing development in this area is not impacted by local catchment runoff which ponds directly behind the levee (i.e. the subject of this present investigation), rather it is impacted by overland flow which approaches the levee from the surrounding local catchment. The objective of this analysis was to allow a "broad brush" economic assessment of various flood modification measures. Damages from floods ranging between the 2 year ARI and PMF events were assessed.

There are limited data available on historic flood damages in the study area. The analysis was therefore carried out using the flood damages model attached to *“Floodplain Risk Management Guideline No 4. Residential Flood Damages”,* which was prepared by DECCW (now OEH) to allow a consistent assessment across NSW for the comparison of flood management projects. For *Guideline No 4*, damage assessments which had been undertaken after major flooding in urban centres were adjusted and used to estimate damages likely to be experienced to typical residential development in NSW. Data for the flood damages model comprised the depths of inundation over the floodplain, as well as information on the unit values of damages to residential property.

The depths of inundation were determined from the results of the hydraulic modelling undertaken as part of the present investigation, ground levels taken from the photogrammetric survey and floor level estimates based on a visual inspection. The type of structure and potential for property damage were also assessed from a visual inspection of the floodplain.

**Table DS1** summarises the number of properties which comprises the damages database for the CBD Levee and Western Levee catchments.

**TABLE DS2**
**NUMBER OF PROPERTIES IN FLOOD DAMAGES DATABASE**

| Study Area | Residential | Commercial/Industrial | Public |
|---|---|---|---|
| CBD Levee | 0 | 395 | 14 |
| Western Levee | 104 | 15 | 0 |

The estimated damages (rounded to two decimal places) which could occur for floods of differing ARI for the case where the flood gates are in either their fully open or fully closed positions are summarised in **Tables DS2**. This table also shows the number of properties which experience “above floor” inundation for the respective flood events.

By inspection of values given in **Table DS2** there is a large increase in the number of properties which will experience above floor inundation behind the CBD Levee should an intense local catchment storm occur when water levels in the Peel River exceed 4 m on the town gauge (i.e. when the flood gates will be in their fully closed position).

The same is not the case for the Western Levee, where the floor levels of residential and commercial property located along Goonoo Goonoo Road lie above the level to which local catchment runoff will pond behind the levee for storms with ARI's up to 200 years. Above floor inundation is shown to occur in several properties as a result of local catchment runoff which surcharges the existing stormwater drainage network. Whilst there are a limited number of properties which have been identified as subject to above floor inundation, flood damages for this area are still relatively high (i.e. in the order of $1M for a 100 year ARI event). The reason for this is that external damages will be incurred as a result of overland flow which will discharge through a significant number of properties during storms which surcharge the existing stormwater drainage network.

**TABLE DS2**
**FLOOD DAMAGES AS A RESULT OF LOCAL CATCHMENT FLOODING**

| Town Levee | Position of Flood Gates | Flood Event Years ARI | No. of Properties with Floors Inundated | | | Total Damage ($ x $10^6$) |
|---|---|---|---|---|---|---|
| | | | Residential | Commercial | Public | |
| CBD Levee | Fully Open | 2 | 0 | 17 | 0 | 0.14 |
| | | 5 | 0 | 27 | 0 | 0.36 |
| | | 20 | 0 | 55 | 0 | 1.26 |
| | | 100 | 0 | 160 | 0 | 4.20 |
| | | 200 | 0 | 201 | 1 | 6.75 |
| | | PMF | 0 | 377 | 14 | 141.5 |
| | Fully Closed | 2 | 0 | 120 | 0 | 3.32 |
| | | 5 | 0 | 161 | 1 | 5.54 |
| | | 20 | 0 | 207 | 3 | 10.43 |
| | | 100 | 0 | 283 | 8 | 22.01 |
| | | 200 | 0 | 299 | 10 | 27.33 |
| | | PMF | 0 | 377 | 14 | 141.5 |
| Western Levee | Fully Open | 2 | 1 | 0 | 0 | 0.29 |
| | | 5 | 2 | 1 | 0 | 0.50 |
| | | 20 | 5 | 1 | 0 | 0.78 |
| | | 100 | 7 | 3 | 0 | 1.02 |
| | | 200 | 8 | 4 | 0 | 1.14 |
| | | PMF | 71 | 10 | 0 | 7.29 |
| | Fully Closed | 2 | 1 | 0 | 0 | 0.29 |
| | | 5 | 2 | 1 | 0 | 0.49 |
| | | 20 | 5 | 1 | 0 | 0.78 |
| | | 100 | 7 | 3 | 0 | 1.03 |
| | | 200 | 8 | 4 | 0 | 1.15 |
| | | PMF | 71 | 10 | 0 | 7.31 |

# D1. INTRODUCTION AND SCOPE

## D1.1 Introduction

Damages from flooding belong to two categories:

- **Tangible Damages**
- **Intangible Damages**

**Tangible damages** are defined as those to which monetary values may be assigned, and may be subdivided into direct and indirect damages. Direct damages are those caused by physical contact of floodwater with damageable property. They include damages to commercial and residential building structures and contents as well as damages to infrastructure services such as electricity and water supply. Indirect damages result from the interruption of community activities, including traffic flows, trade, industrial production, costs to relief agencies, evacuation of people and contents and clean up after the flood.

Generally, tangible damages are estimated in dollar values using survey procedures, interpretation of data from actual floods and research of government files.

The various factors included in the **intangible damage** category may be significant. However, these effects are difficult to quantify due to lack of data and the absence of an accepted method. Such factors may include:

- inconvenience
- isolation
- disruption of family and social activities
- anxiety, pain and suffering, trauma
- physical ill-health
- psychological ill-health.

## D1.2 Scope of Investigation

In the following sections, damages to residential, commercial and industrial properties and public buildings have been estimated in the study area.

## D1.3 Terminology

Definitions of the terms used in this Appendix are presented in **Section D8**.

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## D2. DESCRIPTION OF APPROACH

The damage caused by a flood to a particular property is a function of the depth of flooding above floor level and the value of the property and its contents. The warning time available for residents to take action to lift property above floor level also influences damages actually experienced. A spreadsheet model which had been developed for previous investigations of this nature was used to estimate damages on a property by property basis according to the type of development, the location of the property and the depth of inundation.

Using the results of the present investigation, a peak flood elevation for each event was interpolated at each property. The interpolated property flood levels were input to the spreadsheet model which also contained property characteristics and depth-damage relationships. The depth of flooding was computed as the difference between the interpolated flood level and the surveyed floor elevation at each property.

The depth-damage curves for residential damages were determined using procedures described in *"Floodplain Management Guideline No 4. Residential Flood Damage Calculation"*, *2007* published by DECCW (Now OEH). Damage curves for commercial and industrial developments were derived from previous floodplain management investigations.

It should be understood that this approach is not intended to identify individual properties liable to flood damages and the values of damages in individual properties, even though it appears to be capable of doing so. The reason for this caveat lies in the various assumptions used in the procedure, the main ones being:

- the assumption that computed water levels and topographic data used to define flood extents are exact and without any error;
- the use of "average" stage-damage relationships, rather than a relationship for each property;
- the uncertainty associated with assessing an appropriate factor to convert potential damages to actual flood damages experienced for each property after residents have taken action to mitigate damages to contents; and
- the derivation of floor levels based on a visual inspection.

The consequence of these assumptions is that some individual properties may be inappropriately classified as flood liable, while others may be excluded. Nevertheless, when applied over a broad area these effects would tend to cancel, and the resulting estimates of overall damages, would be expected to be reasonably accurate.

*The information contained in the spreadsheets used to prepare the estimates of flood damages for the catchments should not therefore be used to provide information on the above-floor inundation of individual properties*.

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## D3. SOURCES OF DATA

### D3.1 General

To estimate A*verage Annual Flood Damages* for a specific area it is necessary to estimate the damages for several floods of different magnitudes, i.e. of different frequencies, and then to integrate the damages over the whole range of frequencies. To do this it is necessary to have data on the damages sustained by all types of property over the likely range of inundation. There are several ways of doing this:

- The ideal way would be to conduct specific damage surveys in the aftermath of a range of floods, preferably immediately after each. An example approaching this ideal is the case of Nyngan where surveys were conducted in May 1990 following the disastrous flood of a month earlier (DWR, 1990). This approach would not be practicable unless the damage survey was carried out shortly after the flood.
- The second best way is for experienced loss adjusters to conduct a survey to estimate likely losses that would arise due to various depths of inundation. This approach is used from time to time, but it can add significantly to the cost of a floodplain management study (LMJ, 1985). It was not used for the present investigation.
- The third way is to use generalised data such as that published by CRES (Centre for Resource & Economic Studies, Canberra) and used in the Floodplain Management Study for Forbes (SKM, 1994). These kind of data are considered to be suitable for generalised studies, such as broad regional studies. They are not considered to be suitable for use in specific areas, unless none of the other approaches can be satisfactorily applied.
- The fourth way is to adapt or transpose, data from other flood liable areas. This was the approach used for the present study. For the assessment of residential damages the *DECCW Floodplain Management Guideline No 4, 2004* procedure was adopted, which was based on data collected following major flooding in Katherine in 1998, with adjustments to account for changes in values due to inflation, and after taking into account the nature of development and flooding patterns in the study area. Commercial and industrial damages were assessed via reference to recent floodplain management investigations.

### D3.2 Property Database

As indicated above, an important source of data for this study was the inspection and levelling of floors of properties estimated to be affected by flooding. The properties were divided into three categories: residential, commercial/industrial and public buildings.

For residential properties, the floor level of each residence was determined by visual inspection, whereby the estimated height above ground was added to the adjacent ground level, the elevation of which was taken from the photogrammetric based survey. A visual inspection was also undertaken to classify each property into categories which relate to the magnitude of likely flood damages. An identical approach was adopted for estimating the floor levels of both commercial/industrial properties and public buildings, however, it was also necessary to determine the floor area for each of these building, estimates of which were taken from aerial photography in the case of the Western Levee and from a property database which was provided by TRC in the case of the CBD Levee.

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## D4. RESIDENTIAL DAMAGES

### D4.1 Damage Functions

The procedures identified in *Floodplain Management Guideline No 4* allow for the preparation of a depth versus damage relationship which incorporates structural damage to the building, damage to internals and contents, external damages and clean up costs. In addition, there is the facility for including allowance for accommodation costs and loss of rent. Separate curves are computed for three residential categories:

- Single storey slab on ground construction
- Single storey elevated floor
- Two storey residence

The level of flood awareness and available warning time are taken into account by factors which are used to reduce "potential" damages to contents to "actual" damages. "Potential" damages represent losses likely to be experienced if no action were taken by residents to mitigate impacts. A reduction in the potential damages to "actual" damages is usually made to allow for property evacuation and raising valuables above floor level, which would reduce the damages actually experienced. The ability of residents to take action to reduce flood losses is mainly limited to reductions in damages to contents, as damages to the structure and clean up costs are not usually capable of significant mitigation.

The reduction in damages to contents is site specific, being dependent on a number of factors related to the time of rise of floodwaters, the recent flood history and flood awareness of residents and emergency planning by the various Government Agencies (Bureau of Meteorology and State Emergency Service).

The study area comprises "flash flooding" catchments with a likely time of rise of floodwaters of less than an hour. There is no catchment specific flood warning system operated by the Bureau of Meteorology. Consequently, there would be limited time in advance of a flood event in which to warn residents and for them to take action to mitigate flood losses.

Provided warning were available, house contents may be raised above flood level to about 0.9 m, which corresponds with the height of a typical table/bench height. The spreadsheet provides two factors, one for above and one for below the typical bench height. The reduction in damages is also dependent on the likely duration of inundation of contents, which in the study area would be limited to no more than an hour for most flooded properties. The "Total Contents Adjustment Factor" which converts potential damages to actual damages to contents was 1.01 for depths of inundation up to 0.9 m and 1.14 for greater depths.

**Table D4.1** below shows total flood damages estimated for the three classes of residential property using the procedures identified in *Guideline No 4*. A typical ground floor area of 120 $m^2$ was adopted, representative of house floor areas in the study area.

**TABLE D4.1**
**DAMAGES TO RESIDENTIAL PROPERTIES**

| Type of Residential Construction | 0.5 m Depth of Inundation Above Floor Level | 1 m Depth of Inundation Above Floor Level |
|---|---|---|
| Single Storey Slab on Ground | $52,599 | $70,159 |
| Single Storey High Set | $60,966 | $75,977 |
| Two Storey Residence | $36,829 | $49,111 |

Note: These values include allowances for structural, contents and clean up costs. External costs, which are incurred when allotments are inundated, are added separately in the following tables.

### D4.2 Total Residential Damages

**Table D4.2** over summarises residential damages for the range of floods in the study catchments. The damage estimates were carried out for floods between the 2 year ARI and the PMF, which were modelled hydraulically in the present investigation. Note that there are no residential properties contained in the CBD Levee property database.

## TABLE D4.2
## RESIDENTIAL FLOOD DAMAGES

| Town Levee | Position of Flood Gates | Flood Event Years ARI | No. of Residences Flooded Above Floor Level | Total Damages ($ x $10^6$) |
|---|---|---|---|---|
| CBD Levee | Fully Open | 2 | 0 | 0 |
| | | 5 | 0 | 0 |
| | | 20 | 0 | 0 |
| | | 100 | 0 | 0 |
| | | 200 | 0 | 0 |
| | | PMF | 0 | 0 |
| | Fully Closed | 2 | 0 | 0 |
| | | 5 | 0 | 0 |
| | | 20 | 0 | 0 |
| | | 100 | 0 | 0 |
| | | 200 | 0 | 0 |
| | | PMF | 0 | 0 |
| Western Levee | Fully Open | 2 | 1 | 0.29 |
| | | 5 | 2 | 0.40 |
| | | 20 | 5 | 0.54 |
| | | 100 | 7 | 0.64 |
| | | 200 | 8 | 0.76 |
| | | PMF | 71 | 4.45 |
| | Fully Closed | 2 | 1 | 0.29 |
| | | 5 | 2 | 0.39 |
| | | 20 | 5 | 0.54 |
| | | 100 | 7 | 0.65 |
| | | 200 | 8 | 0.77 |
| | | PMF | 71 | 4.45 |

*Note: There are no residential properties contained in the CBD Levee property database.*

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## D5. COMMERCIAL AND INDUSTRIAL DAMAGES

### D5.1 Direct Commercial and Industrial Damages

The method used to calculate damages requires each property to be categorised in terms of the following:

- damage category
- floor area
- floor elevation.

The damage category assigned to each enterprise was either "low", "medium" or "high", depending on the nature of the enterprise and the likely effects of flooding. Damages were then determined on the basis of floor area.

It has recently been recognised following the 1998 flood in Katherine that previous investigations using stage damage curves contained in proprietary software tended to seriously underestimate true damage costs (*Floodplain Management Guideline No 4*). DECCW are currently researching appropriate damage functions which could be adopted in the estimation of commercial and industrial categories as they have already done with residential damages. However, these data were not available for the present study.

For a recent study on Towradgi Creek on the south coast of NSW, DECCW advised that damage estimates derived from the software should be increased by at least a factor of two. For a medium value commercial enterprise e.g. food shops, banks or dry cleaners, this would result in damage estimate of $201,720 for a depth of inundation above floor level of 2 m and $60,350 for 1 m. This estimate includes external and internal damages plus clean up costs and is equivalent to unit damage rates of $800 and $500/m$^2$ respectively for a 200 m$^2$ property.

The following damage rates were adopted for potential external and internal damages and clean up costs for both commercial and industrial properties. They are indexed to a depth of inundation of 2 metres. At floor level and 1.2 m inundation, zero and 70% of these values respectively were assumed to occur:

| | | |
|---|---|---|
| Low value enterprise | $500/m$^2$ | (e.g. Commercial: small shops, cafes, joinery, public halls. Industrial: auto workshop with concrete floor and minimal goods at floor level, Council or Government Depots, storage areas.) |
| Medium value enterprise | $800/m$^2$ | (e.g. Commercial: food shops, hardware, banks, professional offices, retail enterprises, with furniture/fixtures at floor level which would suffer damage if inundated. Industrial: warehouses, equipment hire. ) |
| High value enterprise | $1,100/m$^2$ | (e.g. Commercial : electrical shops, clothing stores, bookshops, newsagents, restaurants, schools, showrooms and retailers with goods and furniture, or other high value items at ground or lower floor level. Industrial: service stations, vehicle showrooms, smash repairs.) |

The factor for converting potential to actual damages depends on a range of variables such as the available warning time, flood awareness and the depth of inundation. Given sufficient warning time a well prepared business will be able to temporarily lift property above floor level. However, unless property is actually moved to flood free areas, floods which result in a large depth of inundation, will cause considerable damage to stock and contents.

For the present study, potential damages were converted to actual damages using a multiplier which ranged between 0.7 and 0.9 depending on the depth of inundation above the floor. The factors also took into consideration the limited warning time that would be available and the absence of major flooding behind the town levees in recent years.

### D5.2 Indirect Commercial and Industrial Damages

Indirect commercial and industrial damages comprise costs of removal of goods and storage, loss of trading profit and loss of business confidence.

Disruption to trade takes the following forms:

- The loss through isolation at the time of the flood when water is in the business premises or separating clients and customers. The total loss of trade is influenced by the opportunity for trade to divert to an alternative source. There may be significant local loss but due to the trade transfer this may be considerably reduced at the regional or state level.
- In the case of major flooding, a downturn in business can occur within the flood affected region due to the cancellation of contracts and loss of business confidence. This is in addition to the actual loss of trading caused by closure of the business by flooding.

Loss of trading profit is a difficult value to assess and the magnitude of damages can vary depending on whether the assessment is made at the local, regional or national level. Differences between regional and national economic effects arise because of transfers between the sectors, such as taxes, and subsidies such as flood relief returned to the region. Some investigations have lumped this loss with indirect damages and have adopted total damage as a percentage of the direct damage. In other cases, loss of profit has been related to the gross margin of the business, i.e. turnover less average wages. The former approach has been adopted in this present study. Indirect damages have been taken as 50% of direct actual damages. A clean up cost of $\$30/m^2$ has also been adopted.

### D5.3 Total Commercial and Industrial Damages

**Table D5.1** over summarises estimated commercial and industrial damages within the flood liable portion of each study catchment.

**TABLE D5.1**
**COMMERCIAL AND INDUSTRIAL DAMAGES**

| Town Levee | Position of Flood Gates | Flood Event Years ARI | Number of Properties with Floors Inundated | Direct Damages ($ x $10^6$) | Indirect Damages ($ x $10^6$) | Total Damages ($ x $10^6$) |
|---|---|---|---|---|---|---|
| CBD Levee | Fully Open | 2 | 17 | 0.00 | 0.13 | 0.14 |
| | | 5 | 27 | 0.05 | 0.30 | 0.36 |
| | | 20 | 55 | 0.24 | 1.01 | 1.26 |
| | | 100 | 160 | 1.61 | 2.57 | 4.18 |
| | | 200 | 201 | 3.08 | 3.63 | 6.71 |
| | | PMF | 377 | 87.46 | 46.80 | 134.27 |
| | Fully Closed | 2 | 120 | 1.37 | 1.95 | 3.32 |
| | | 5 | 161 | 2.63 | 2.87 | 5.50 |
| | | 20 | 207 | 5.44 | 4.86 | 10.30 |
| | | 100 | 283 | 12.61 | 8.80 | 21.41 |
| | | 200 | 299 | 15.94 | 10.55 | 26.49 |
| | | PMF | 377 | 87.48 | 46.81 | 134.30 |
| Western Levee | Fully Open | 2 | 0 | 0.00 | 0.00 | 0.00 |
| | | 5 | 1 | 0.04 | 0.07 | 0.10 |
| | | 20 | 1 | 0.12 | 0.13 | 0.25 |
| | | 100 | 3 | 0.20 | 0.18 | 0.38 |
| | | 200 | 4 | 0.20 | 0.18 | 0.38 |
| | | PMF | 10 | 1.77 | 1.06 | 2.84 |
| | Fully Closed | 2 | 0 | 0.00 | 0.00 | 0.00 |
| | | 5 | 1 | 0.04 | 0.07 | 0.10 |
| | | 20 | 1 | 0.12 | 0.13 | 0.25 |
| | | 100 | 3 | 0.20 | 0.18 | 0.38 |
| | | 200 | 4 | 0.20 | 0.18 | 0.38 |
| | | PMF | 10 | 1.79 | 1.07 | 2.86 |

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# D6. DAMAGES TO PUBLIC BUILDINGS

## D6.1 Direct Damages – Public Buildings

Included under this heading are government buildings, churches, swimming pools and parks. Damages were estimated individually on an areal basis according to the perceived value of the property. Potential internal damages were indexed to a depth of above floor inundation of 2 m as shown below. At floor level and 1.2 m depth of inundation, zero and 55% of these values respectively were assumed to occur for "Very Low Value" properties; and zero and 70% for the other categories.

| | | |
|---|---|---|
| Low value | $500/m$^2$ | (eg. park buildings) |
| Medium value | $800/m$^2$ | (eg. council buildings, churches, fire station) |
| High value | $1,100/m$^2$ | (eg. schools, police station) |

These values were based on the commercial damage data presented in **Section D5.1**. External and structural damages were taken as 4 and 10% of internal damages respectively.

## D6.2 Indirect Damages – Public Buildings

A value of $30/m$^2$ was adopted for the clean-up of each property. This value is based on results presented in the Nyngan Study and adjusted for inflation. Total "welfare and disaster" relief costs were assessed as 50% of the actual direct costs.

## D6.3 Total Damages – Public Buildings

**Table D6.1** over summarises estimated damages to public buildings in each catchment. Note that there are no public properties contained in the Western Levee property database.

**TABLE D6.1**
**DAMAGES TO PUBLIC BUILDINGS**

| Town Levee | Position of Flood Gates | Flood Event Years ARI | Number of Properties with Floors Inundated | Direct Damages ($ x $10^6$) | Indirect Damages ($ x $10^6$) | Total Damages ($ x $10^6$) |
|---|---|---|---|---|---|---|
| CBD Levee | Fully Open | 2 | 0 | 0 | 0 | 0 |
| | | 5 | 0 | 0 | 0 | 0 |
| | | 20 | 0 | 0 | 0 | 0 |
| | | 100 | 0 | 0 | 0.02 | 0.02 |
| | | 200 | 1 | 0 | 0.05 | 0.05 |
| | | PMF | 14 | 4.61 | 2.60 | 7.22 |
| | Fully Closed | 2 | 0 | 0 | 0 | 0 |
| | | 5 | 1 | 0.01 | 0.03 | 0.04 |
| | | 20 | 3 | 0.05 | 0.08 | 0.13 |
| | | 100 | 8 | 0.25 | 0.35 | 0.60 |
| | | 200 | 10 | 0.39 | 0.45 | 0.84 |
| | | PMF | 14 | 4.61 | 2.60 | 7.22 |
| Western Levee | Fully Open | 2 | 0 | 0 | 0 | 0 |
| | | 5 | 0 | 0.04 | 0.07 | 0 |
| | | 20 | 0 | 0.12 | 0.13 | 0 |
| | | 100 | 0 | 0.20 | 0.18 | 0 |
| | | 200 | 0 | 0.20 | 0.18 | 0 |
| | | PMF | 0 | 1.77 | 1.06 | 0 |
| | Fully Closed | 2 | 0 | 0 | 0 | 0 |
| | | 5 | 0 | 0 | 0 | 0 |
| | | 20 | 0 | 0 | 0 | 0 |
| | | 100 | 0 | 0 | 0 | 0 |
| | | 200 | 0 | 0 | 0 | 0 |
| | | PMF | 0 | 0 | 0 | 0 |

*Note: There are no public properties contained in the Western Levee property database.*

## D7 SUMMARY OF TANGIBLE DAMAGES

### D7.1 Tangible Damages

The total damages for each flood event are summarised in **Table D7.1**. Cumulative average annual damages (AAD) were assessed and are also shown. **Figures D7.1** to **C7.3** show damages versus frequency data for the individual catchments.

Flood damages under existing conditions have been computed for a range of flood frequencies from 2 year ARI to the PMF. The smallest flood modelled in the present investigation was the 2 year ARI event. From **Table D7.1**, flood damages would be expected at this level of flooding. Unfortunately there is no information available to allow assessment of damages for lesser floods. For the purposes of assessing average annual damages, the 100% AEP was adopted as the "threshold" flood magnitude at which significant damages would be experienced.

### D8.3 Definition of Terms

*Average Annual Damages* (also termed "expected damages") are determined by integrating the area under the damage-frequency curve. They represent the time stream of annual damages, which would be expected to occur on a year by year basis over a long duration.

Using an appropriate discount rate, average annual damages may be expressed as an equivalent "*Present Worth Value"* of damages and used in the economic analysis of potential flood management measures.

*Cumulative Annual Average Damages* may be referenced to a particular flood frequency. They represent the average damages which would be expected on an annual basis for all flood events up to and including that nominated frequency.

For example, the cumulative average annual value of damages in property protected by the CBD Levee for all local catchment floods up to the 100 year ARI level is around $4.01 Million (**Table D7.1**). Note that this value represents the worst case scenario since it represents the sum of all flood damages up to the 100 year ARI assuming the flood gates are in their fully closed position. A flood management scheme which has a 100 year ARI level of protection will eliminate damages up to this level of flooding. If the scheme has no mitigating effect on larger floods, then these damages represent the benefits of the scheme expressed on an average annual basis.

### D8.4 Present Worth of Damages in Ponding Areas

Under current NSW Treasury guidelines, economic analyses are carried out assuming a 20 year economic life for projects and discount rates of 7% pa. (best estimate) and 10% and 4% pa. (sensitivity analysis). This requires the conversion of average annual damages to an equivalent present worth value at each of the three discount rates.

The *Present Worth Values* of damages likely to be experienced in the study area for all flood events up to the 100 year ARI, a 20 year economic life and discount rates of 4, 7 and 10 per cent are shown on **Table D7.2**.

## TABLE D7.1
## TOTAL DAMAGES IN THE STUDY CATCHMENTS

| Town Levee | Position of Flood Gates | Flood Event Years ARI | No. of Properties with Floors Inundated | | | Total Damages ($ x $10^6$) | Cumulative AAD ($ x $10^6$) |
|---|---|---|---|---|---|---|---|
| | | | Residential | Commercial | Public | | |
| CBD Levee | Fully Open | 2 | 0 | 17 | 0 | 0.14 | 0.03 |
| | | 5 | 0 | 27 | 0 | 0.36 | 0.11 |
| | | 20 | 0 | 55 | 0 | 1.26 | 0.23 |
| | | 100 | 0 | 160 | 0 | 4.20 | 0.34 |
| | | 200 | 0 | 201 | 1 | 6.75 | 0.37 |
| | | PMF | 0 | 377 | 14 | 141.49 | 0.73 |
| | Fully Closed | 2 | 0 | 120 | 0 | 3.32 | 0.83 |
| | | 5 | 0 | 161 | 1 | 5.54 | 2.16 |
| | | 20 | 0 | 207 | 3 | 10.43 | 3.36 |
| | | 100 | 0 | 283 | 8 | 22.01 | 4.01 |
| | | 200 | 0 | 299 | 10 | 27.33 | 4.13 |
| | | PMF | 0 | 377 | 14 | 141.52 | 4.54 |
| Western Levee | Fully Open | 2 | 1 | 0 | 0 | 0.29 | 0.07 |
| | | 5 | 2 | 1 | 0 | 0.50 | 0.19 |
| | | 20 | 5 | 1 | 0 | 0.78 | 0.29 |
| | | 100 | 7 | 3 | 0 | 1.02 | 0.32 |
| | | 200 | 8 | 4 | 0 | 1.14 | 0.33 |
| | | PMF | 71 | 10 | 0 | 7.29 | 0.35 |
| Western Levee | Fully Closed | 2 | 1 | 0 | 0 | 0.29 | 0.07 |
| | | 5 | 2 | 1 | 0 | 0.49 | 0.19 |
| | | 20 | 5 | 1 | 0 | 0.78 | 0.29 |
| | | 100 | 7 | 3 | 0 | 1.03 | 0.32 |
| | | 200 | 8 | 4 | 0 | 1.15 | 0.33 |
| | | PMF | 71 | 10 | 0 | 7.31 | 0.35 |

_Note: There are no residential properties contained in the CBD Levee property database and no public properties contained in the Western Levee property database._

For a discount rate of 7% pa and an economic life of 20 years, the *Present Worth Value* of damages behind the CBD Levee for all flood events up to the 100 year ARI increases from $3.58 Million assuming the flood gates are always open, to $42.45 Million assuming the flood gates are always closed. Therefore schemes costing up to $42.45 Million could be economically justified if they eliminated damages for all flood events up to the 100 year ARI event (i.e. if it is assumed that the flood gates are always closed during flood producing rain).

More expensive schemes would have a benefit/cost ratio less than 1, but may still be justified according to a multi-objective approach which considers other criteria in addition to economic feasibility.

The feasibility of flood mitigation schemes is considered in **Chapter 7** of the report.

**TABLE D7.2**
**PRESENT WORTH VALUE OF DAMAGES IN STUDY AREA**
**ALL FLOODS UP TO 100 YEAR ARI AND FOR AN ECONOMIC LIFE OF 20 YEARS**
**$ x $10^6$**

| Town Levee | Position of Flood Gates | Discount Rate – per cent | | |
|---|---|---|---|---|
| | | 4 | 7 | 10 |
| CBD Levee | Fully Open | 4.59 | 3.58 | 2.88 |
| | Fully Closed | 54.45 | 42.45 | 34.11 |
| Western Levee | Fully Open | 4.42 | 3.44 | 2.77 |
| | Fully Closed | 4.39 | 3.42 | 2.75 |

### D8.5 Sensitivity Analysis

The sensitivity of flood damages in the ponding area behind the CBD Levee to the assumption of the flood gates being either fully open or full closed for the full range of ARI storms was assessed. The damages models were adjusted to reflect the following two scenarios:

- **Scenario 1** – Assumes that the flood gates are always in the open position for all storms with ARI's up to 20 year and that the flood gates are always in the fully closed position for storms with ARI's greater than 20 year ARI.
- **Scenario 2** - Assumes that the flood gates are always in the fully closed position for all storms with ARI's up to 20 year and that the flood gates are always in the open position for storms with ARI's greater than 20 year ARI.

The *Present Worth Values* of damages likely to be experienced in development located directly behind the CBD Levee for all flood events up to the 100 year ARI, a 20 year economic life and discount rates of 4, 7 and 10 per cent for the two scenarios are shown on **Table D7.3**.

**TABLE D7.3**
**PRESENT WORTH VALUE OF DAMAGES IN STUDY AREA**
**ALL FLOODS UP TO 100 YEAR ARI AND FOR AN ECONOMIC LIFE OF 20 YEARS**
**$ x $10^6$**

| Town Levee | Position of Flood Gates | Discount Rate – per cent | | |
|---|---|---|---|---|
| | | 4 | 7 | 10 |
| CBD Levee | Scenario 1 | 9.71 | 7.57 | 6.09 |
| | Scenario 2 | 49.61 | 38.67 | 31.08 |

### D8.6 Potential Flood Modification Measures

A similar assessment of flood damages to that described in the previous sections of this appendix was undertaken for the flood modifications described in **Chapter 7** of the main report. For the purpose of the present investigation the damages assessment was limited to assessing the Present Worth Value of flood damages for case where all four pressure lines are upgraded (i.e. Option 5).

The *Present Worth Values* of damages likely to be experienced in development located directly behind the CBD Levee for all flood events up to the 100 year ARI, a 20 year economic life and discount rates of 4, 7 and 10 per cent following the implementation of the works comprising Option 5 are shown on **Table D7.4**.

**TABLE D7.4**
**PRESENT WORTH VALUE OF DAMAGES FOLLOWING**
**IMPLEMNTATION OF WORKS COMPRISING OPTION 5**
**ALL FLOODS UP TO 100 YEAR ARI AND FOR AN ECONOMIC LIFE OF 20 YEARS**
**$ x $10^6$**

| Town Levee | Position of Flood Gates | Discount Rate – per cent | | |
|---|---|---|---|---|
| | | 4 | 7 | 10 |
| CBD Levee | Fully Open | 2.11 | 1.64 | 1.32 |
| | Fully Closed | 21.86 | 17.04 | 13.69 |

## D8. REFERENCES

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# TAMWORTH CITY LEVEES
# INTERNAL DRAINAGE STUDY

## VOLUME 2 - FIGURES

**February 2012**

DRAFT FOR CLIENT REVIEW

Prepared by:
**Lyall & Associates Consulting Water Engineers**
Level 1, 26 Ridge Street
North Sydney NSW 2060

**Tel:** **(02) 9929 4466**
Fax: (02) 9929 4458
Email: lacewater@bigpond.com



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| Job No: BZ297<br>File: TCL-IDS Vol 2 Figures [Rev. 1.0].doc | Date: February 2012<br>Rev No: 1.0 | Principal: SAB<br>Author: SAB |

## LIST OF FIGURES

1.1 Location Plan

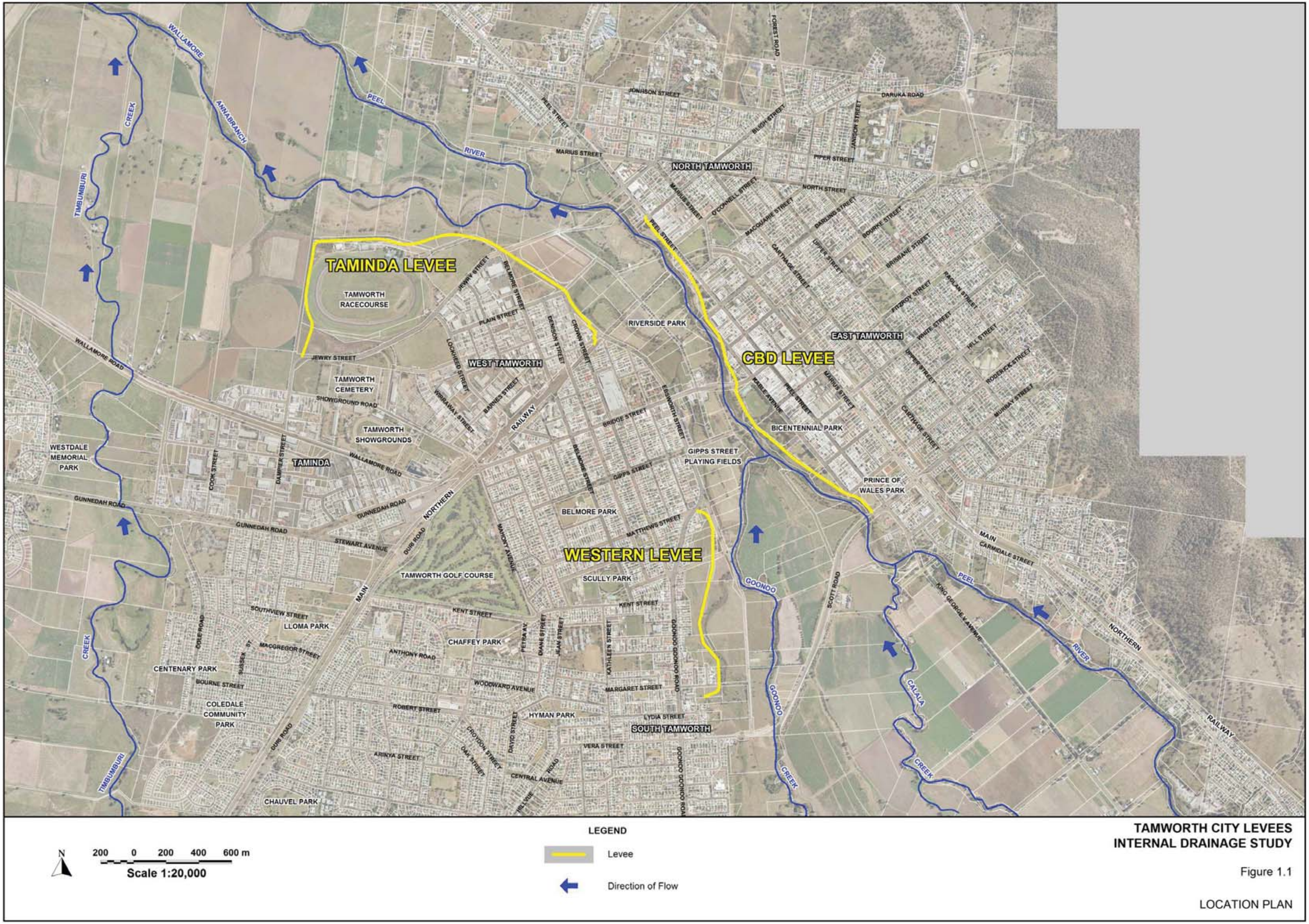
2.1 Plan Showing Route of CBD Levee and Location of Existing Flood Gates
2.2 Longitudinal Section along Crest of CBD Levee
2.3 Catchment Plan – CBD Levee
2.4 Plan Showing Route of Western Levee and Location of Existing Flood Gates
2.5 Longitudinal Section along Crest of Western Levee
2.6 Catchment Plan - Western and Taminda Levees
2.7 Plan Showing Route of Taminda Levee and Location of Existing Flood Gates
2.8 Longitudinal Section along Crest of Taminda Levee
2.9 Stream and Rain Gauge Data at Tamworth – November 2008 Flood
2.10 Layout of FPLAIN Hydraulic Model
2.11 Design Flood Levels on Peel River Floodplain

3.1 Sub-Catchment Layout Plan – Town Levees Hydrologic Model

4.1 TUFLOW Model Layout – CBD Levee
4.2 TUFLOW Model Layout – Western Levee
4.3 TUFLOW Model Layout – Taminda Levee
4.4 Adjusted Peel River Stage Hydrographs – Taminda Levee
4.5 TUFLOW Schematisation of Floodplain

6.1 TUFLOW Model Results – Flood Gates Fully Open – 2 year ARI (3 Sheets)
6.2 TUFLOW Model Results – Flood Gates Fully Closed – 2 year ARI (3 Sheets)
6.3 Afflux (Flood Gates Fully Closed Minus Flood Gates Fully Open) - 2 year ARI (3 Sheets)
6.4 TUFLOW Model Results – Flood Gates Fully Open – 5 year ARI (3 Sheets)
6.5 TUFLOW Model Results – Flood Gates Fully Closed – 5 year ARI (3 Sheets)
6.6 Afflux (Flood Gates Fully Closed Minus Flood Gates Fully Open) – 5 year ARI (3 Sheets)
6.7 TUFLOW Model Results – Flood Gates Fully Open – 20 year ARI (3 Sheets)
6.8 TUFLOW Model Results – Flood Gates Fully Closed – 20 year ARI (3 Sheets)
6.9 Afflux (Flood Gates Fully Closed Minus Flood Gates Fully Open) – 20 year ARI (3 Sheets)
6.10 TUFLOW Model Results – Flood Gates Fully Open – 100 year ARI (3 Sheets)
6.11 TUFLOW Model Results – Flood Gates Fully Closed – 100 year ARI (3 Sheets)
6.12 Afflux (Flood Gates Fully Closed Minus Flood Gates Fully Open) – 100 year ARI (3 Sheets)
6.13 TUFLOW Model Results – Flood Gates Fully Open – PMF (3 Sheets)
6.14 TUFLOW Model Results – Flood Gates Fully Closed – PMF (3 Sheets)
6.15 Afflux (Flood Gates Fully Closed Minus Flood Gates Fully Open) – PMF (3 Sheets)
6.16 Interim Flood Planning Areas (Ponding Areas Only) – Flood Gates Fully Open Versus Flood Gates Fully Closed
6.17 Provisional Flood Hazard – 100 year ARI – Gates Fully Open
6.18 Provisional Flood Hazard – 100 year ARI – Gates Fully Closed
6.19 Potential Impact of Climate Change – 100 Year ARI – Rainfall Increased by 10% - Flood Gates Fully Open
6.20 Potential Impact of Climate Change – 100 Year ARI – Rainfall Increased by 10% - Flood Gates Fully Closed

7.1 O'Connell Street Pressure Line Upgrade (Option 1) – CBD Levee
7.2 Impact of Option 1 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.3 Impact of Option 1 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.4 Fitzroy Street and White Street Pressure Line Upgrades (Options 2 and 3) – CBD Levee
7.5 Impact of Option 2 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.6 Impact of Option 2 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.7 Impact of Option 3 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.8 Impact of Option 3 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.9 Prince of Wales Park Trunk Drainage Upgrade (Option 4) – CBD Levee
7.10 Impact of Option 4 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.11 Impact of Option 4 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.12 Impact of Option 5 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.13 Impact of Option 5 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.14 Impact of Option 6 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.15 Impact of Option 6 Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.16 Impact of Option 7A Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.17 Impact of Option 7A Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed
7.18 Impact of Option 7B Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Open
7.19 Impact of Option 7B Works on Peak 100 year ARI Flood Levels – Flood Gates Fully Closed




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

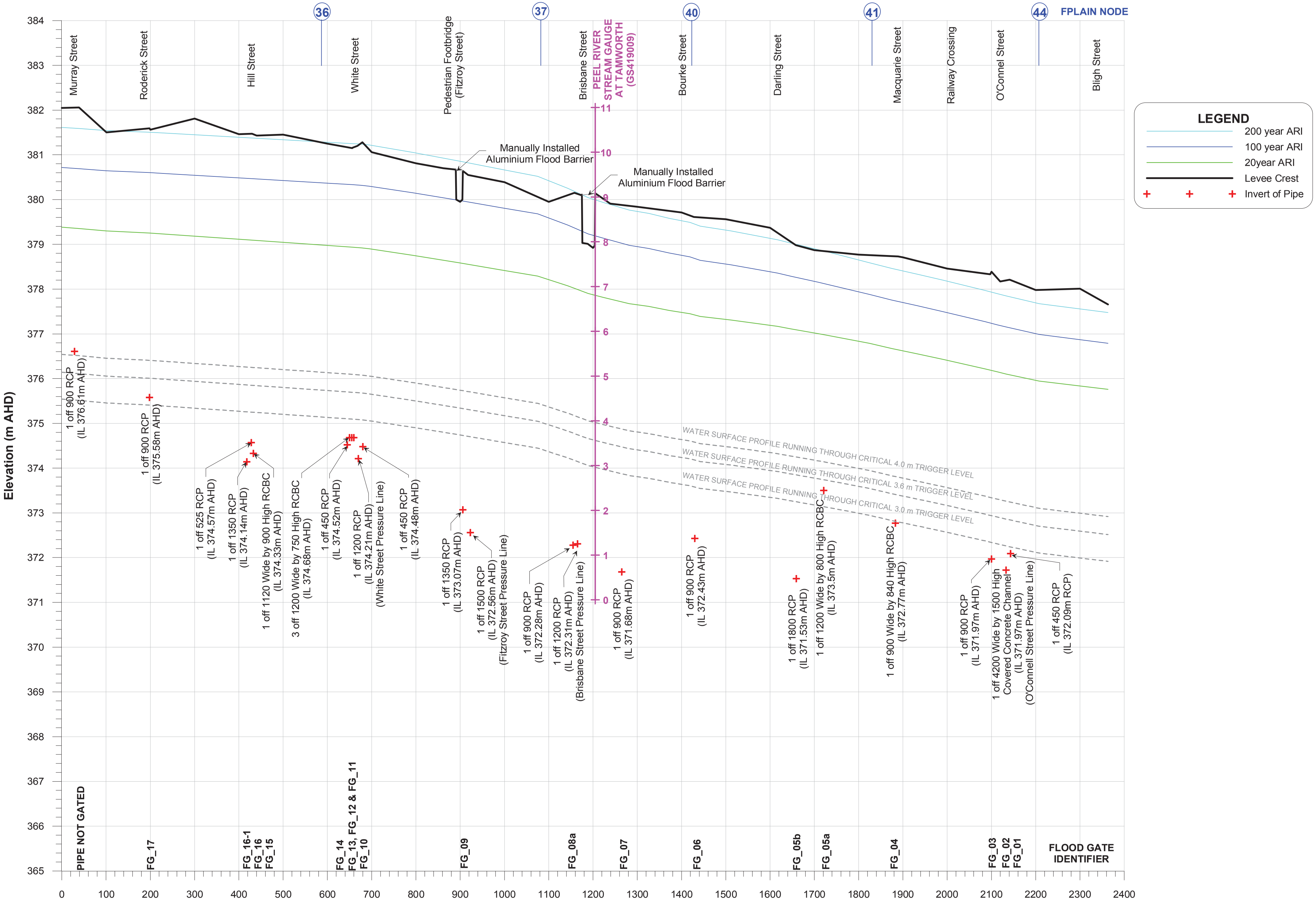
Figure 1.1

LOCATION PLAN


**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 2.1

PLAN SHOWING ROUTE OF CBD LEVEE AND LOCATION OF EXISTING FLOOD GATES



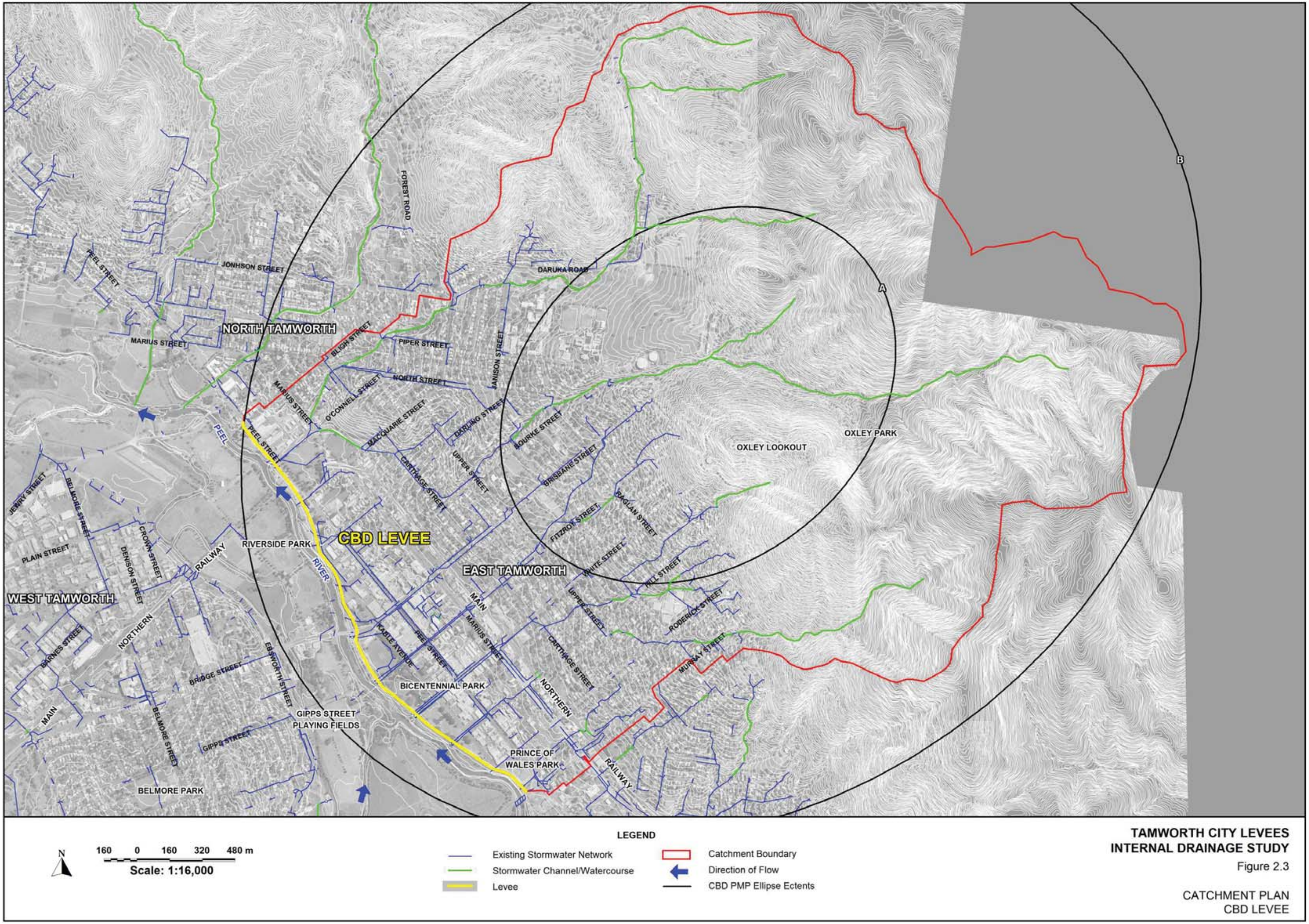

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 2.2

LONGITUDINAL SECTION ALONG CREST OF CBD LEVEE

**NOTES:**

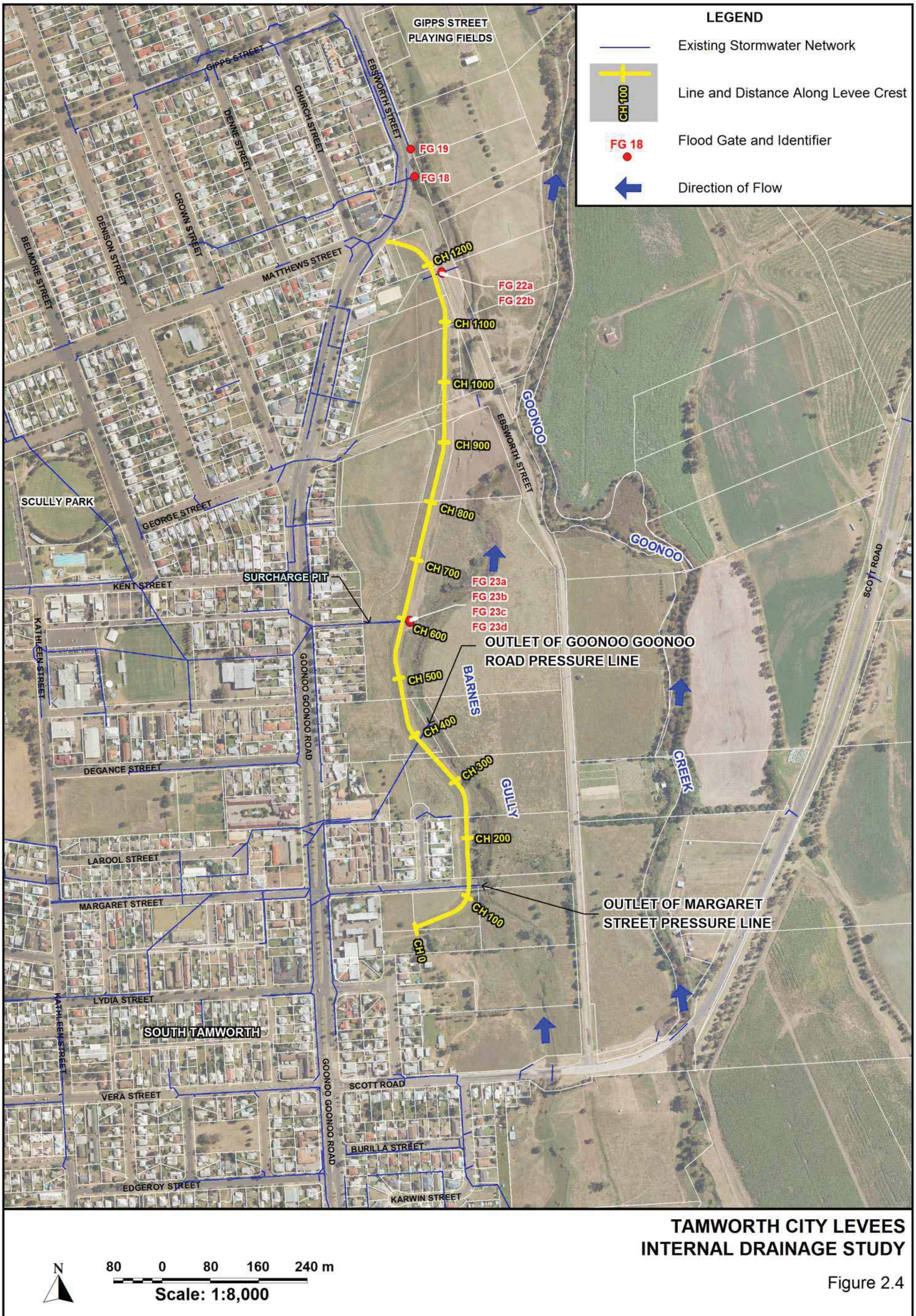
1. GAUGE ZERO ON TAMWORTH GAUGE IS 371.06 m AHD.
2. SOURCE OF CREST LEVEL DATA: 2011 PHOTOGRAMMETRIC DERIVED DTM
3. SOURCE OF DESIGN FLOOD LEVEL DATA: LACE (2006)
4. FLOOD SLOPE AT CRITICAL TRIGGER LEVELS ASSUMED TO MATCH 20 YEAR ARI WATER SURFACE PROFILE




TAMWORTH CITY LEVEES
INTERNAL DRAINAGE STUDY

Figure 2.3

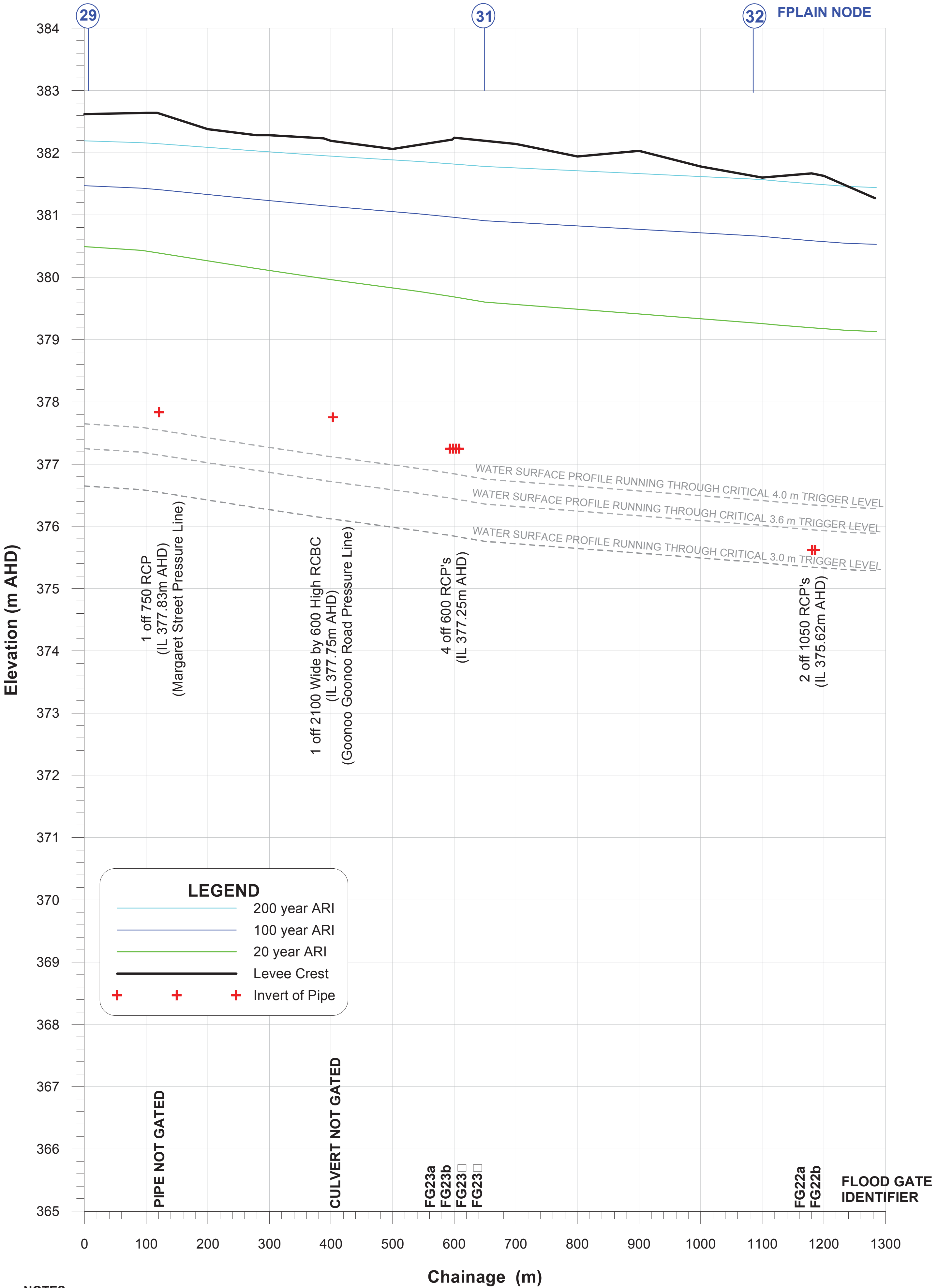
CATCHMENT PLAN
CBD LEVEE




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 2.4

PLAN SHOWING ROUTE OF WESTERN LEVEE AND LOCATION OF EXISTING FLOOD GATES



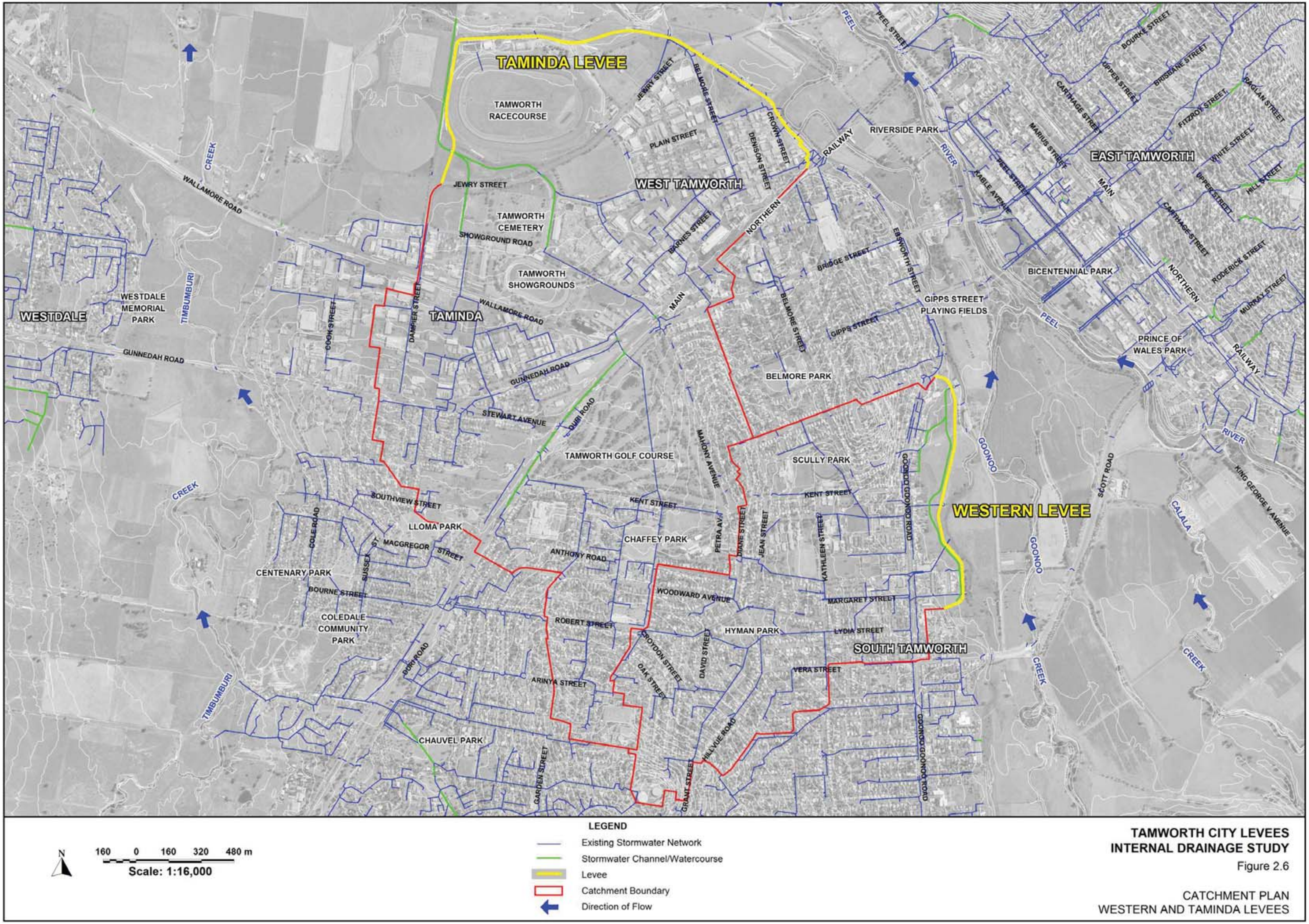

NOTES:

1. GAUGE ZERO ON TAMWORTH GAUGE IS 371.06 m AHD.
2. SOURCE OF CREST LEVEL DATA: 2011 PHOTOGRAMMETRIC DERIVED DTM.
3. SOURCE OF DESIGN FLOOD LEVEL DATA: LACE (2006).
4. FLOOD SLOPE AT CRITICAL TRIGGER LEVELS ASSUMED TO MATCH 20 YEAR ARI WATER SURFACE PROFILE.

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 2.5

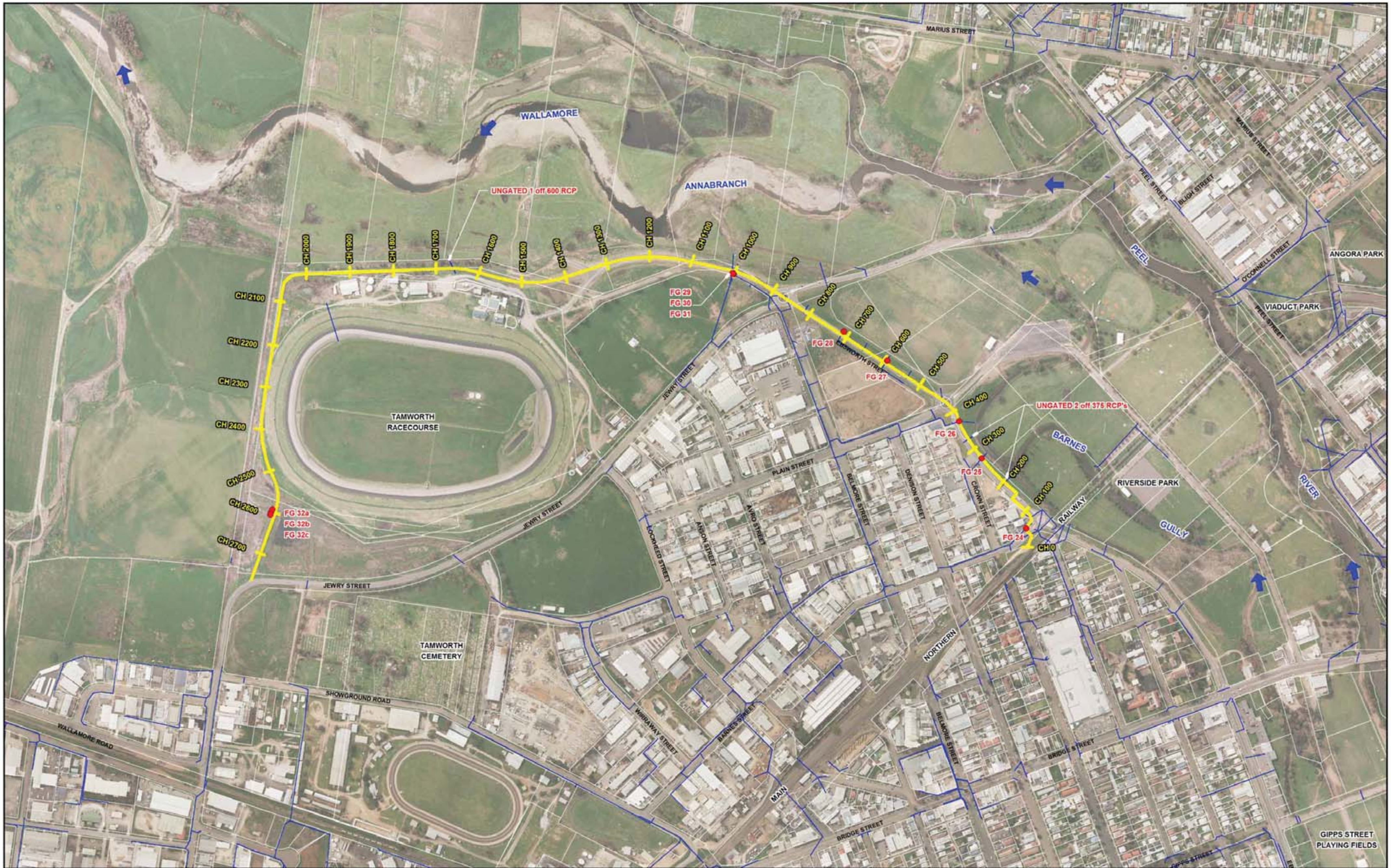
LONGITUDINAL SECTION ALONG CREST OF WESTERN LEVEE




TAMWORTH CITY LEVEES
INTERNAL DRAINAGE STUDY

Figure 2.6

CATCHMENT PLAN
WESTERN AND TAMINDA LEVEES




LEGEND

Existing Stormwater Network

CH 100 Line and Distance Along Levee Crest

FG 27 Flood Gate and Identifier

Direction of Flow

N

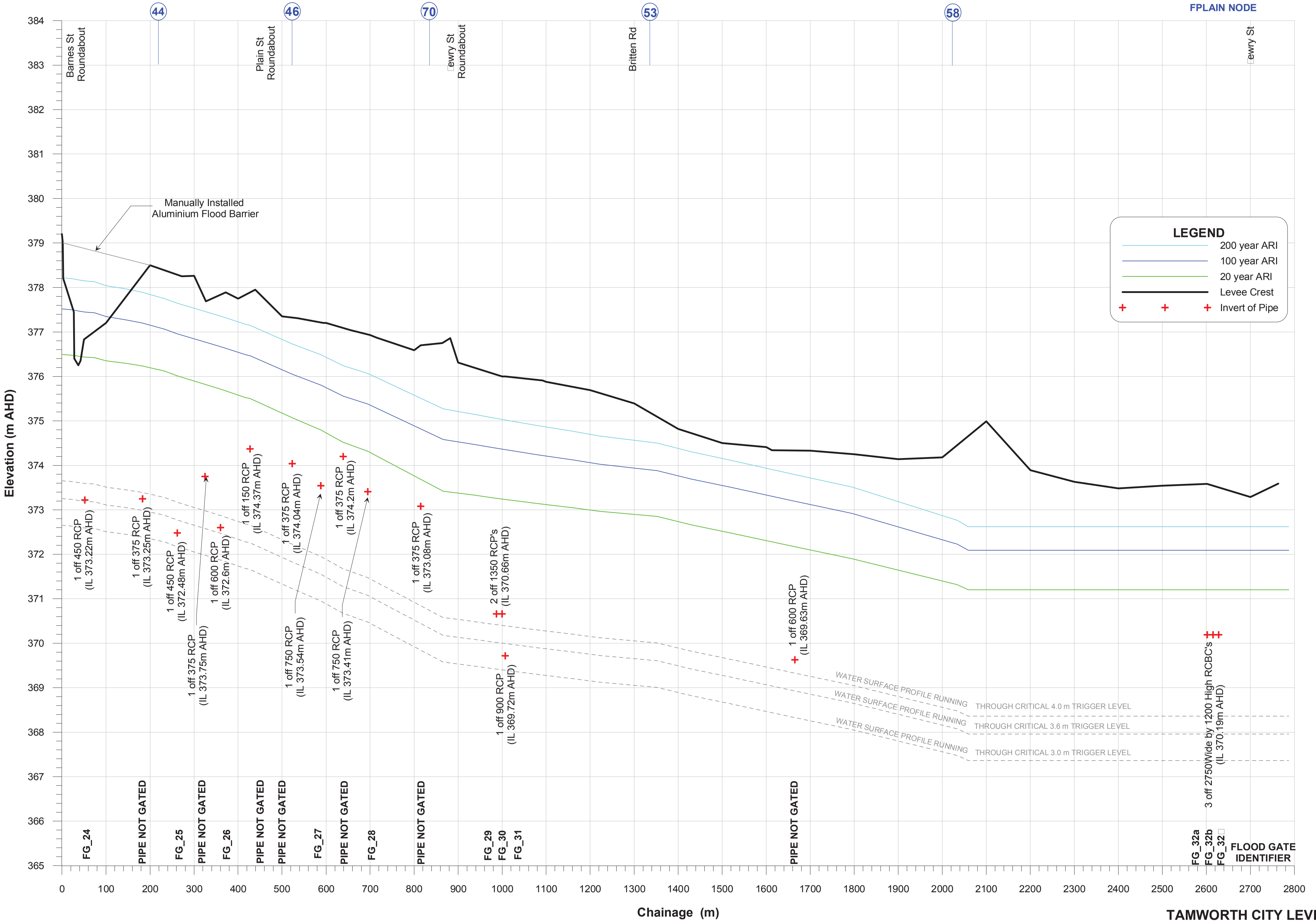
80 0 80 160 240 m

Scale: 1:8,000

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 2.7

PLAN SHOWING ROUTE OF TAMINDA LEVEE
AND LOCATION OF EXISTING FLOOD GATES



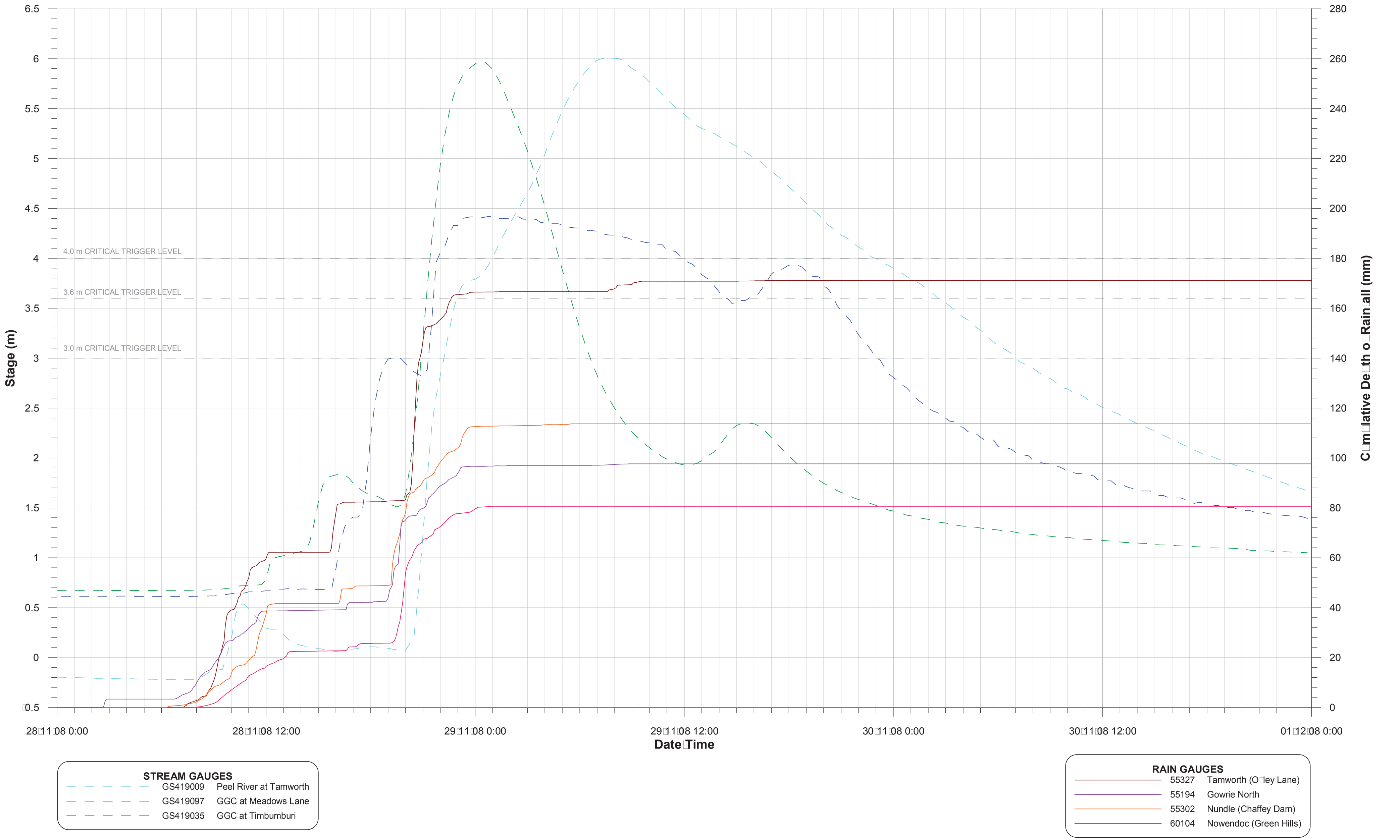

NOTES:

1. GAUGE ZERO ON TAMWORTH GAUGE IS 371.06 m AHD.
2. SOURCE OF CREST LEVEL DATA: 2011 PHOTOGRAMMETRIC DERIVED DTM.
3. SOURCE OF DESIGN FLOOD LEVEL DATA: LACE (2006).
4. FLOOD SLOPE AT CRITICAL TRIGGER LEVELS ASSUMED TO MATCH 20 YEAR ARI WATER SURFACE PROFILE.

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 2.8

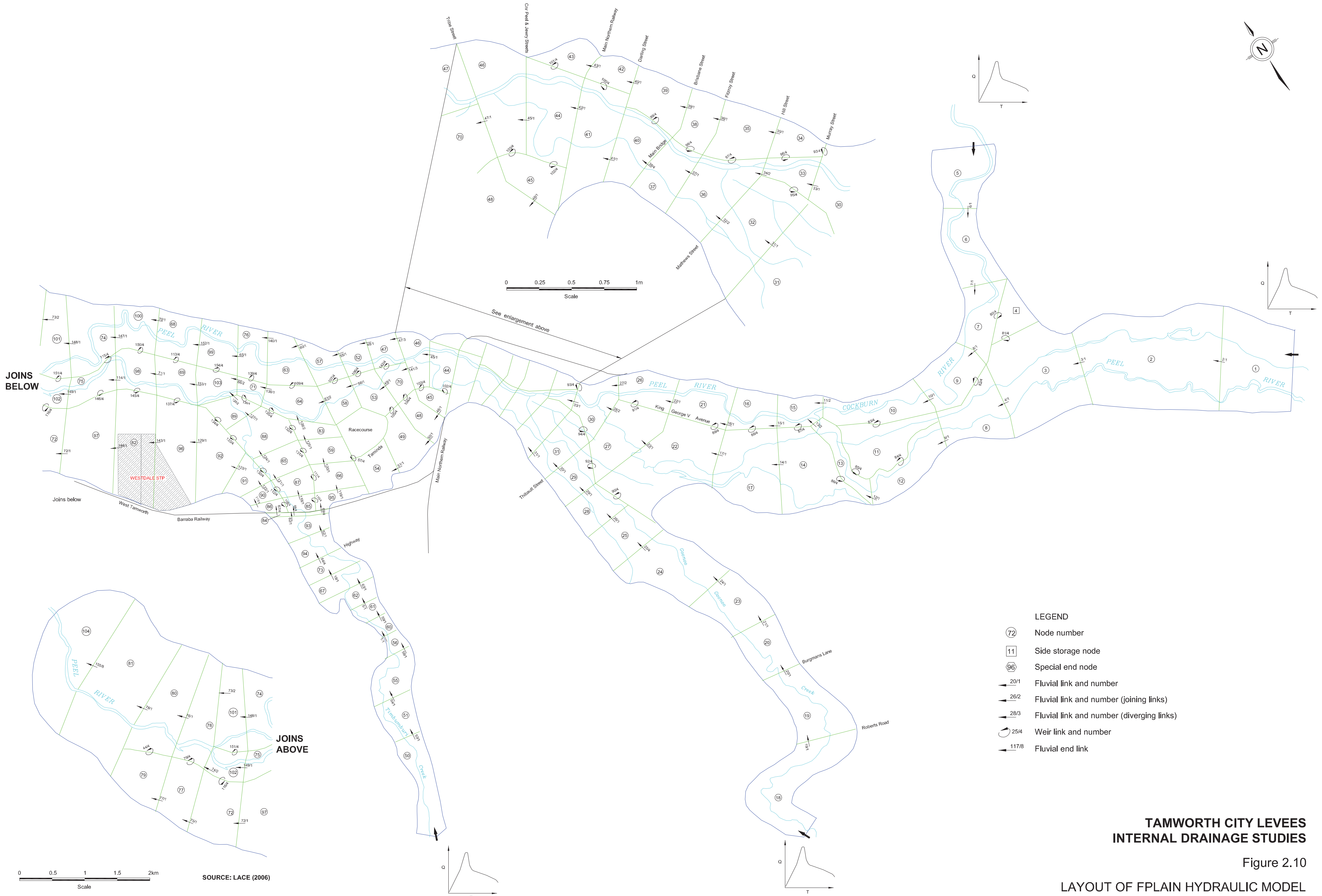
LONGITUDINAL SECTION ALONG CREST OF TAMINDA LEVEE




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 2.9

STREAM AND RAIN GAUGE DATA AT TAMWORTH
NOVEMBER 2008 FLOOD




LEGEND

- 72 Node number
- 11 Side storage node
- 96 Special end node
- 20/1 Fluvial link and number
- 26/2 Fluvial link and number (joining links)
- 28/3 Fluvial link and number (diverging links)
- 25/4 Weir link and number
- 117/8 Fluvial end link

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDIES**

Figure 2.10

LAYOUT OF FPLAIN HYDRAULIC MODEL


LEGEND

| | |
|---|---|
| N30 | FPLAIN Model Node Number |
| 379.63 | Peak 20 year ARI Flood Level (m AHD) |
| 380.93 | Peak 100 year ARI Flood Level (m AHD) |
| 381.81 | Peak 200 year ARI Flood Level (m AHD) |
| 386.32 | Peak Probable Maximum Flood Level (m AHD) |

Direction of Flow

Levee

Existing Stormwater Network

N

160 0 160 320 480 m

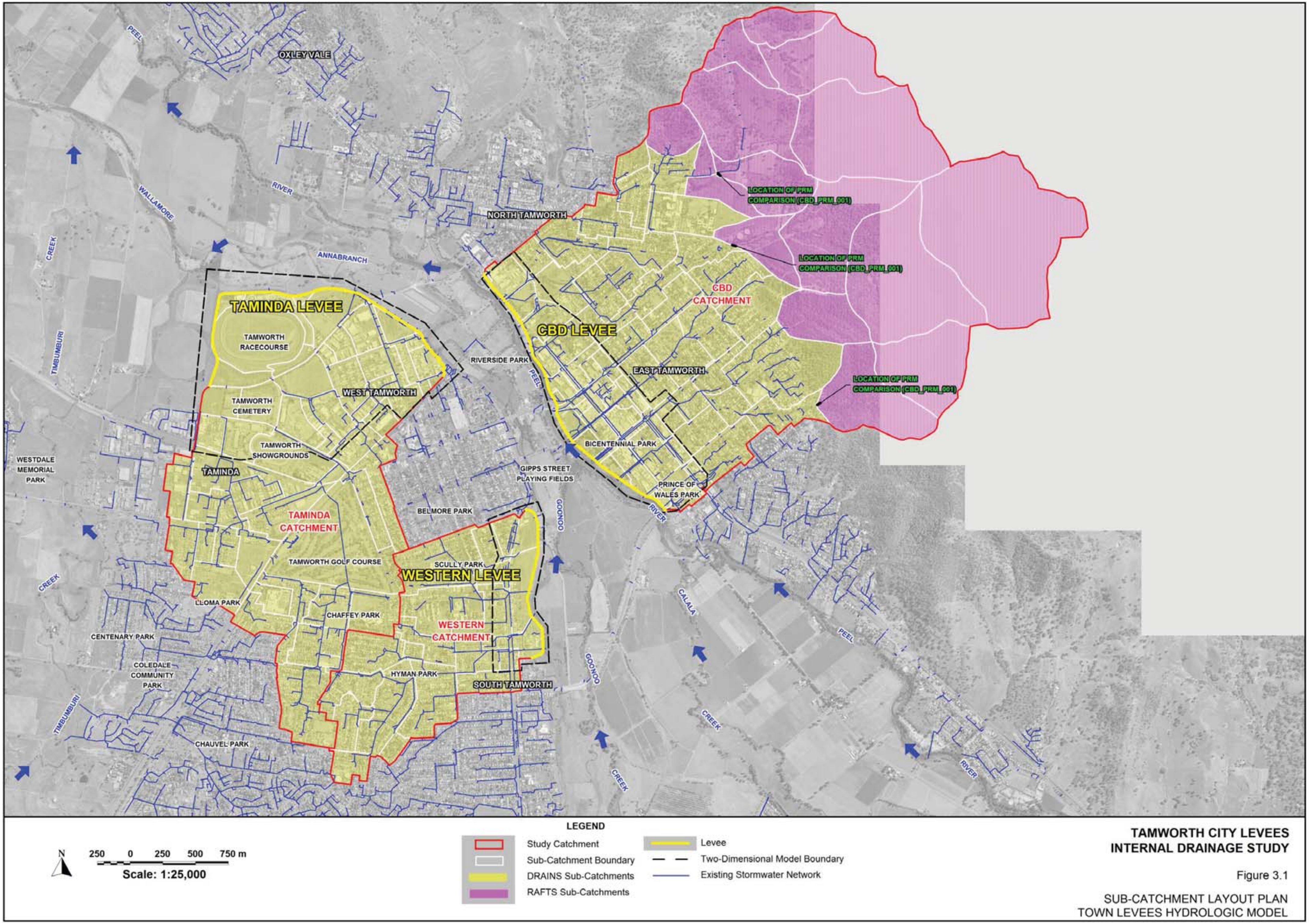
Scale 1:16,000

Source: LACE (2006)

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 2.11

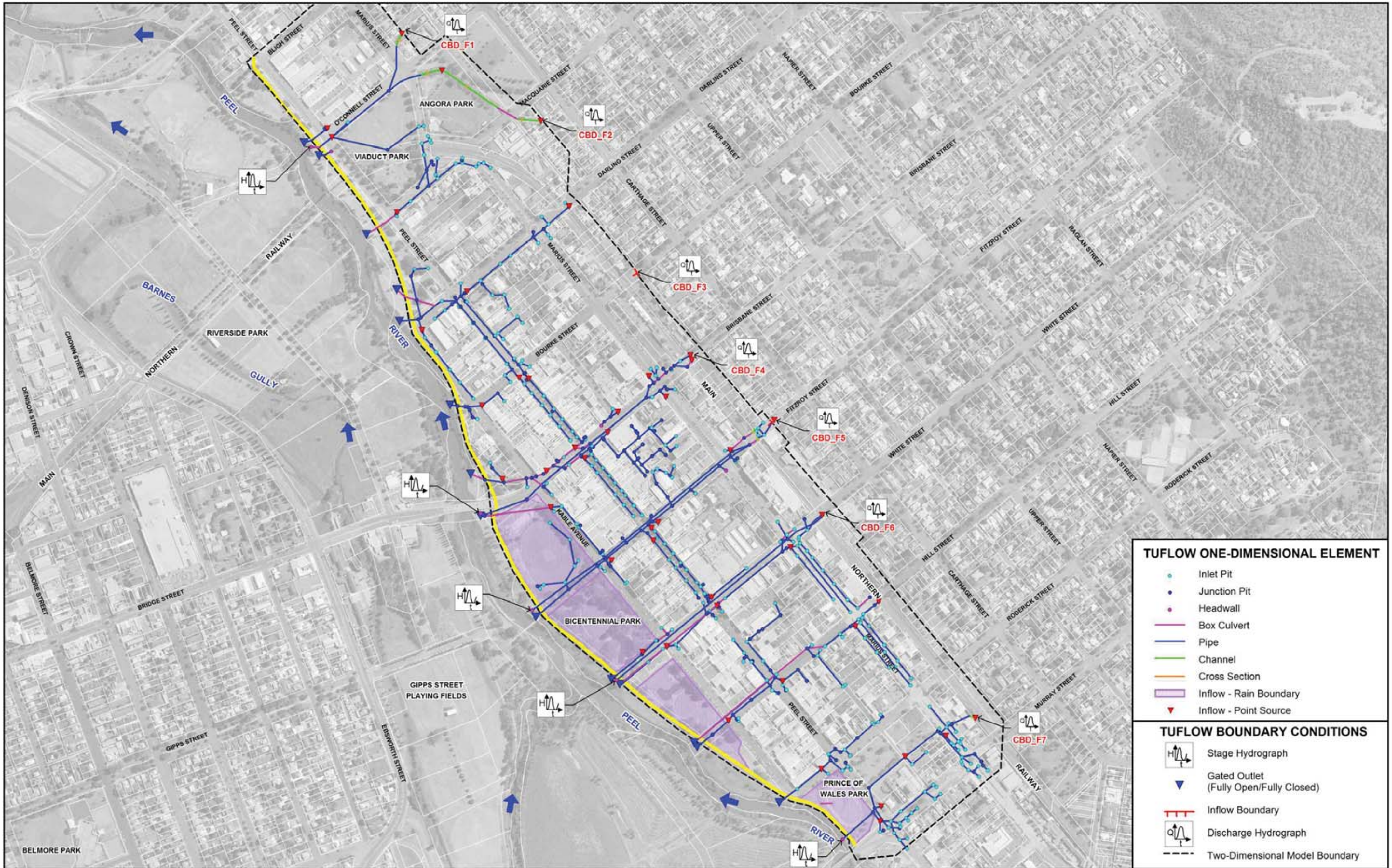
DESIGN FLOOD LEVELS ON PEEL RIVER FLOODPLAIN




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 3.1

SUB-CATCHMENT LAYOUT PLAN
TOWN LEVEES HYDROLOGIC MODEL




N

80 0 80 160 240 m

Scale: 1:8,000

LEGEND

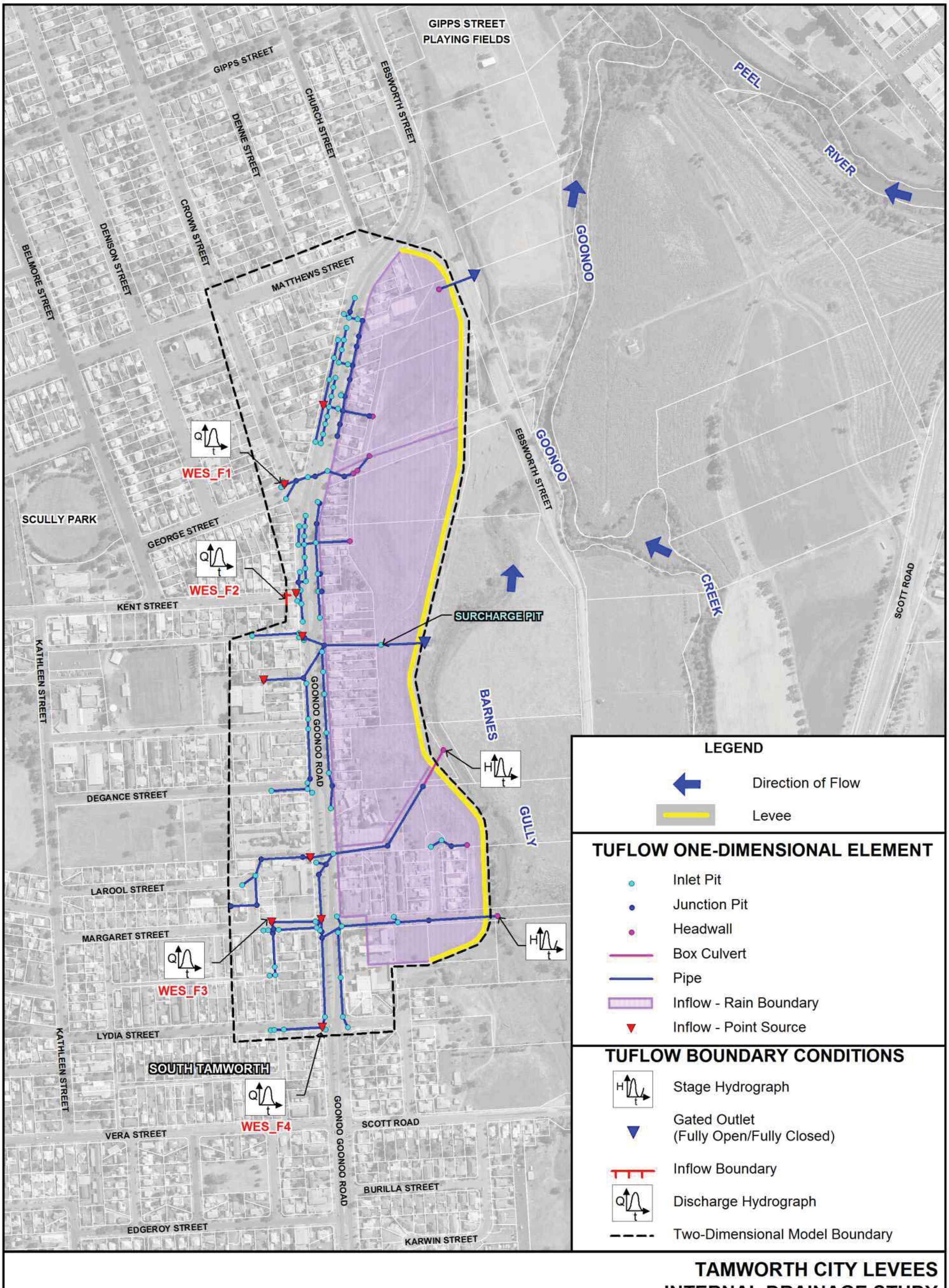
Direction of Flow

Levee

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 4.1

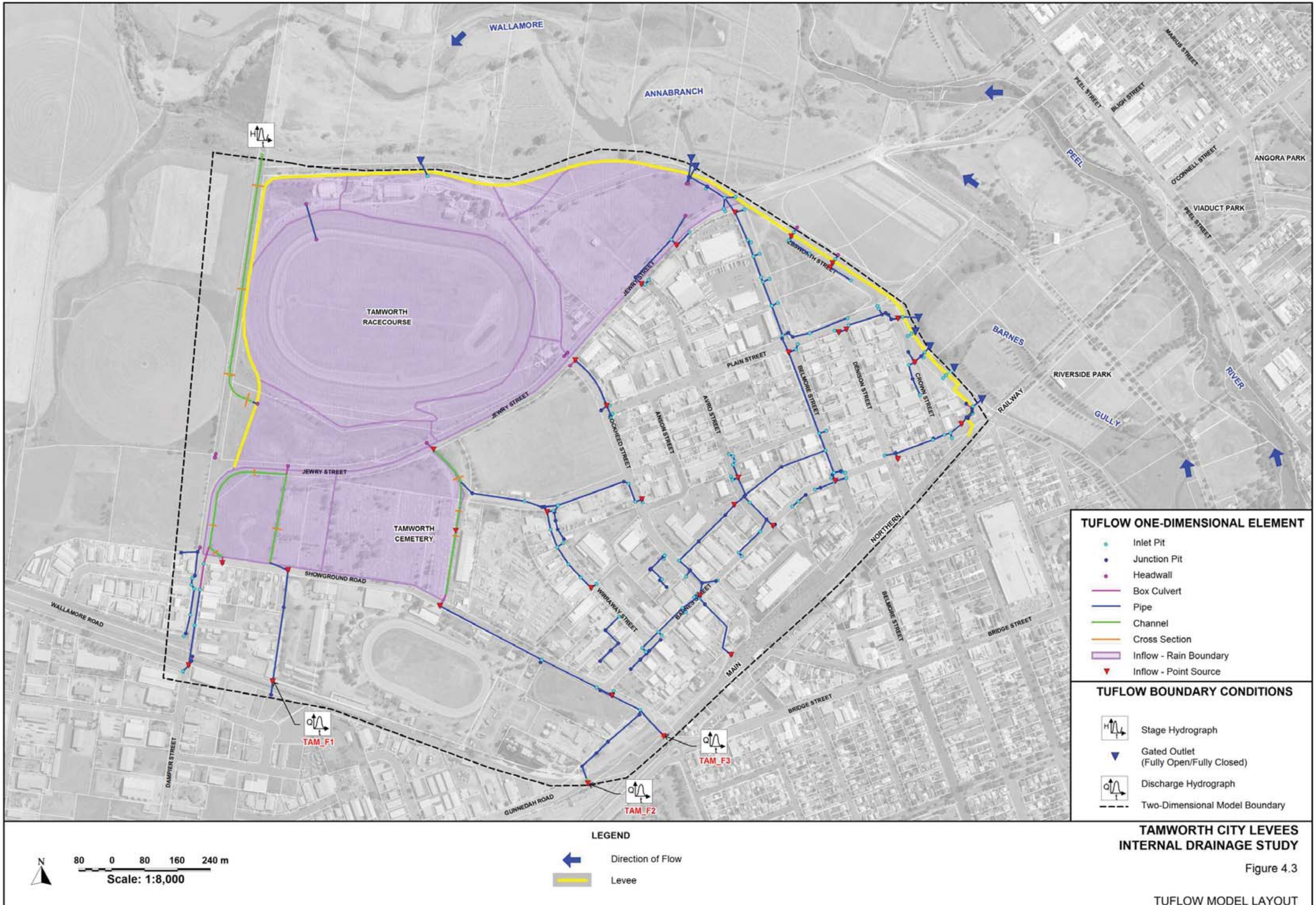
TUFLOW MODEL LAYOUT
CBD LEVEE




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 4.2

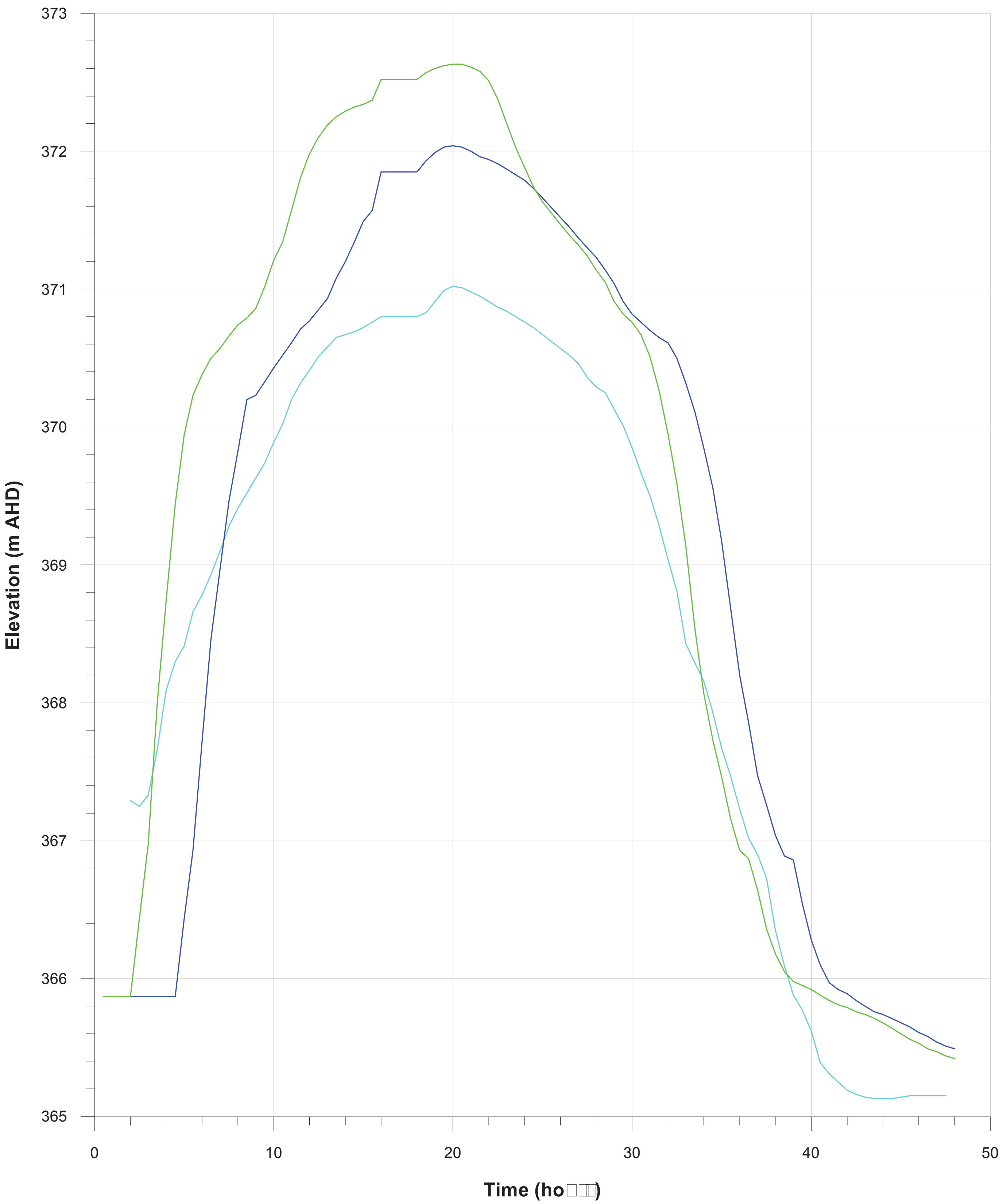
TUFLOW MODEL LAYOUT
WESTERN LEVEE




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 4.3

TUFLOW MODEL LAYOUT
TAMINDA LEVEE



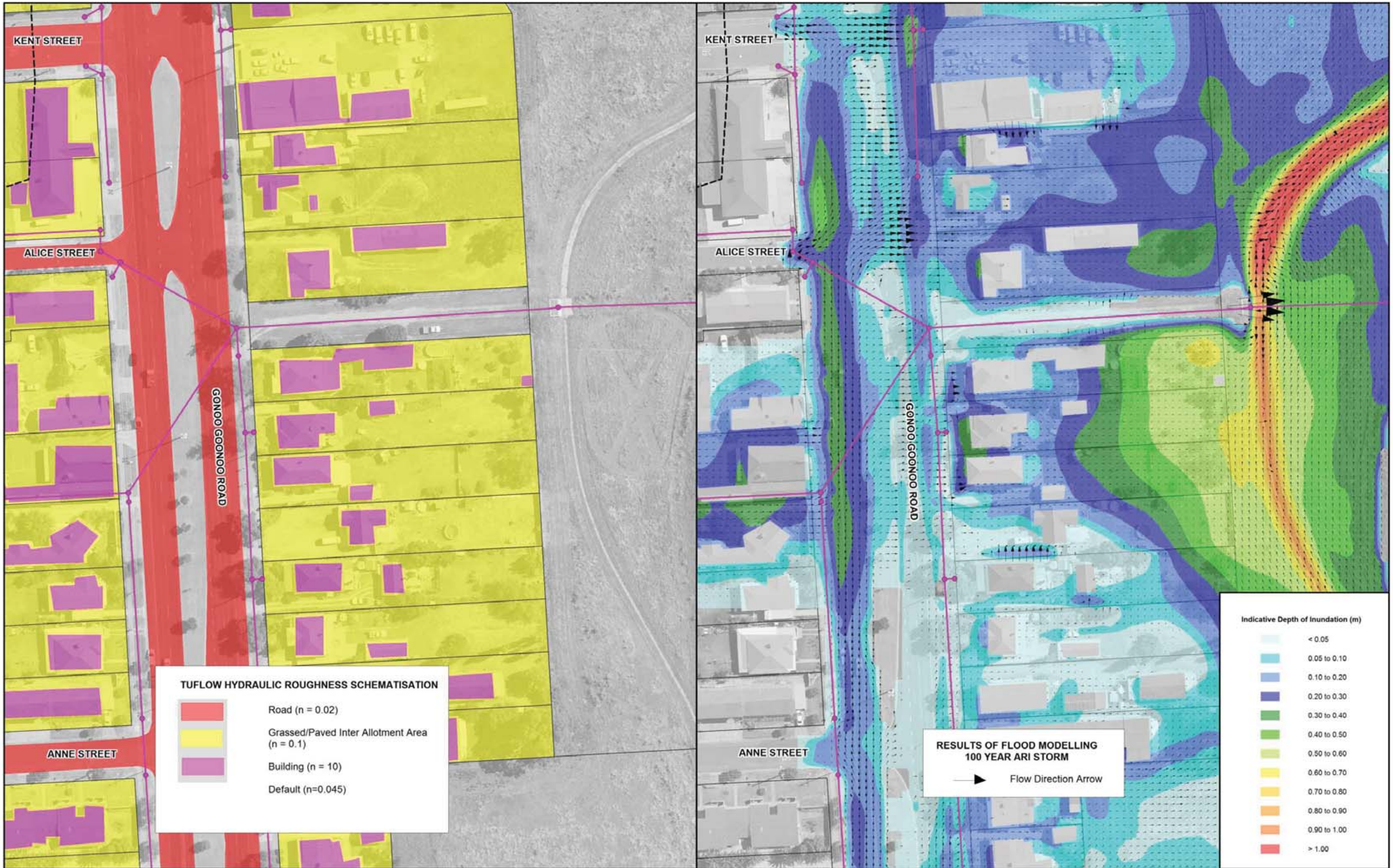

**LEGEND**

- 200 Year ARI
- 100 Year ARI
- 20 Year ARI

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 4.4

AD□USTED PEEL RIVER STAGE HYDROGRAPHS
TAMINDA LEVEE




**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

**LEGEND**

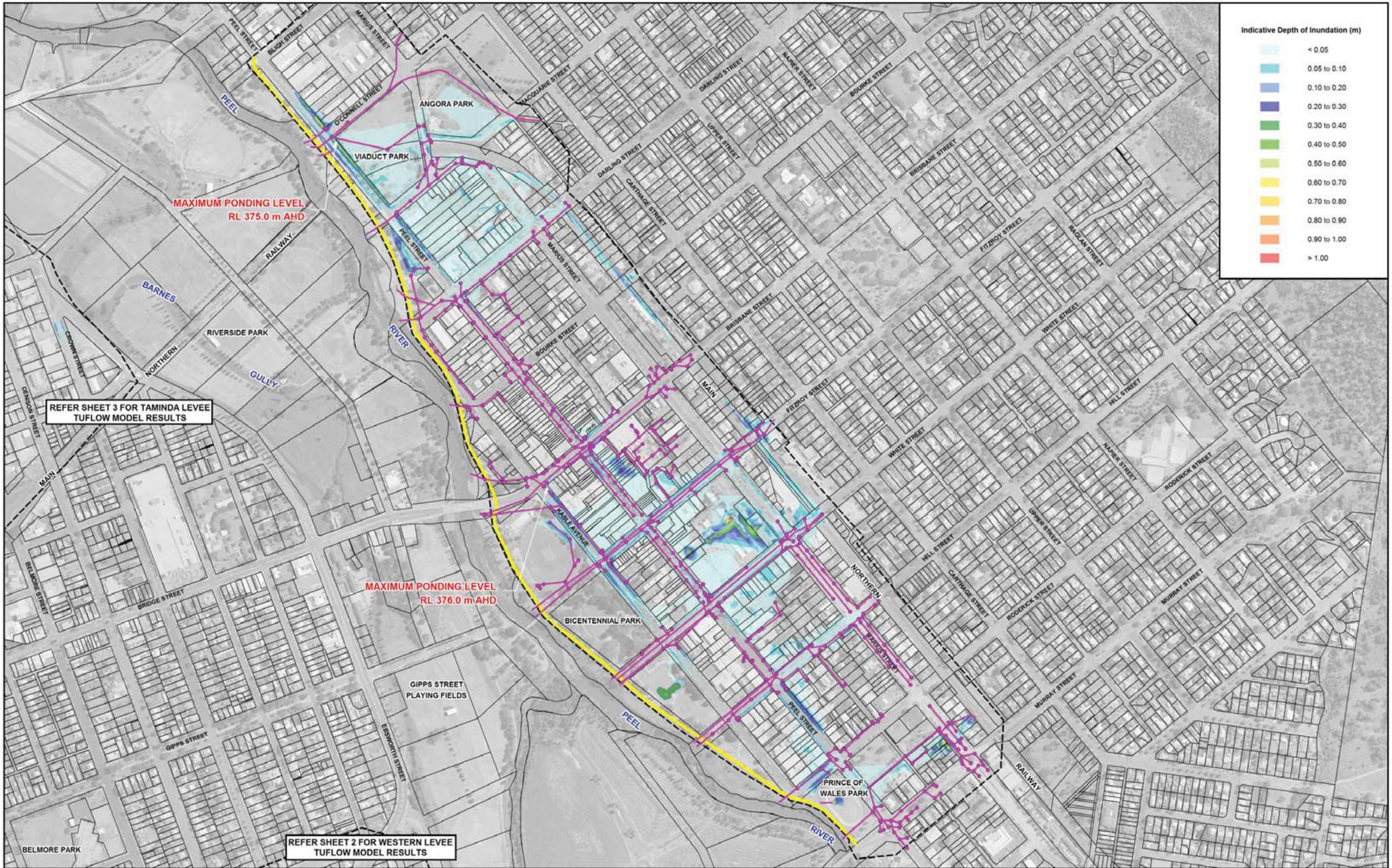
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 4.5

TUFLOW SCHEMATISATION OF FLOODPLAIN




**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

N

80 0 80 160 240 m

Scale: 1:8,000

**LEGEND**

Levee

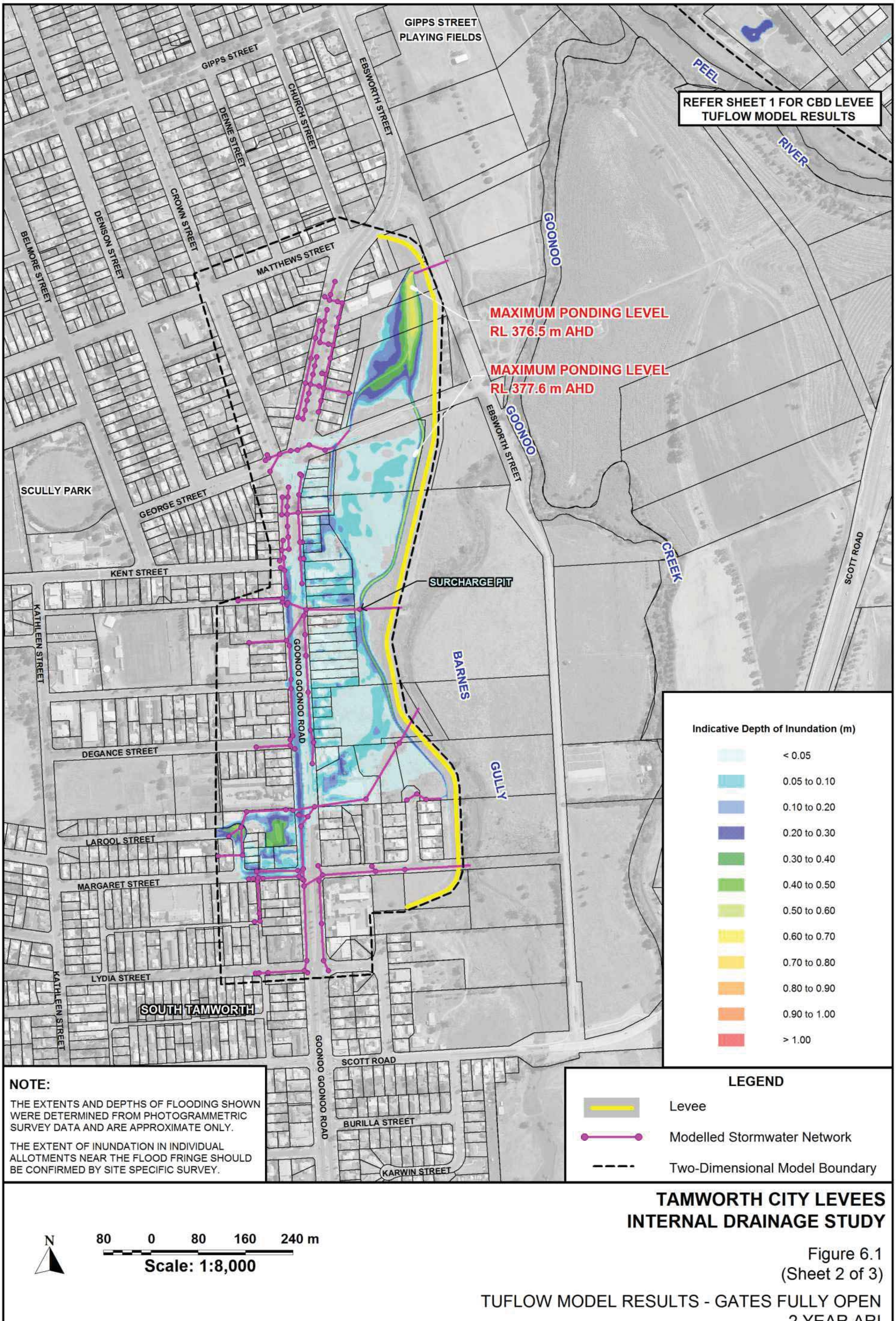
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.1
(Sheet 1 of 3)

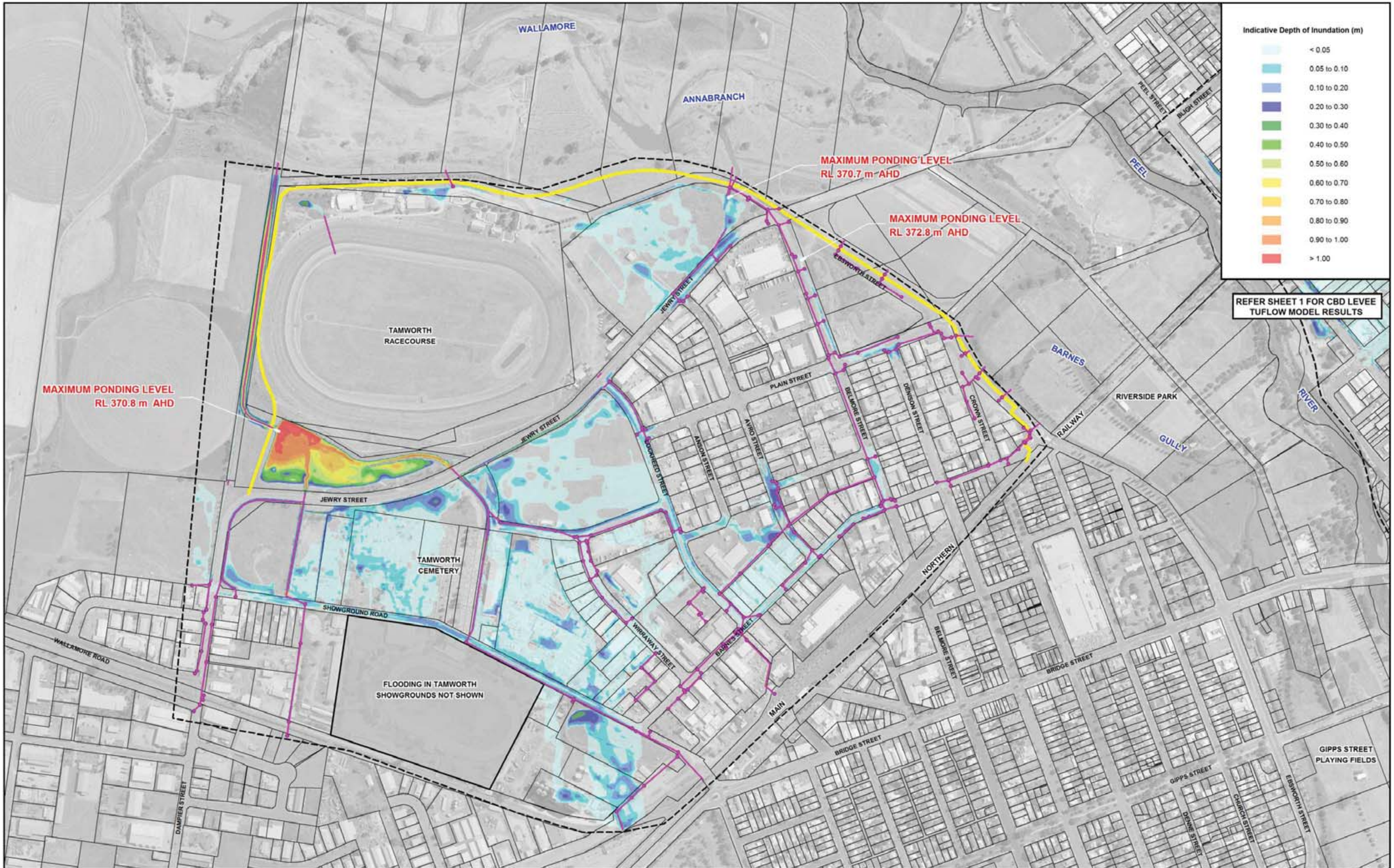
TUFLOW MODEL RESULTS - FLOOD GATES FULLY OPEN
2 YEAR ARI




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.1
(Sheet 2 of 3)

TUFLOW MODEL RESULTS - GATES FULLY OPEN
2 YEAR ARI




N

80 0 80 160 240 m

Scale: 1:8,000

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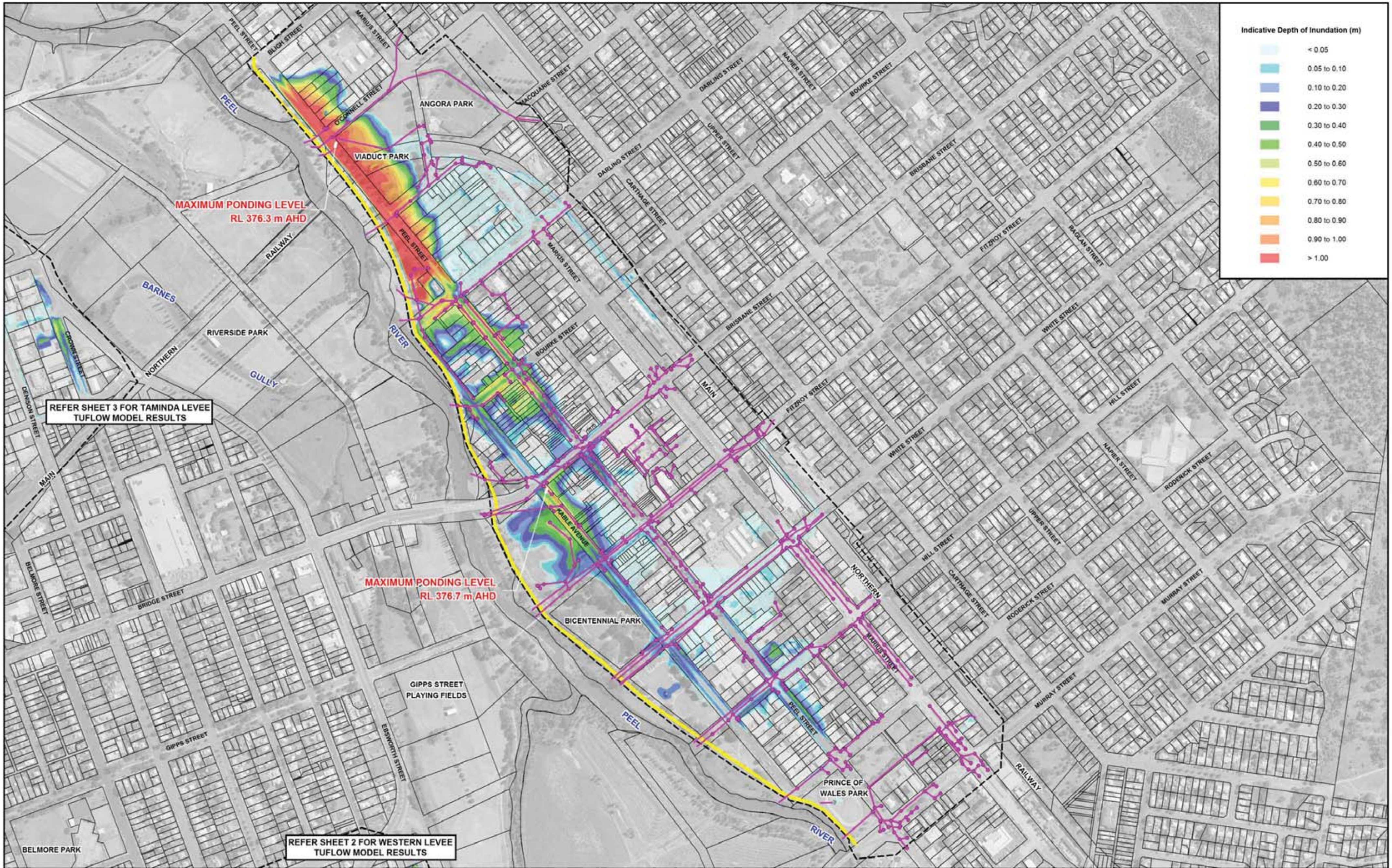
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.1
(Sheet 3 of 3)

TUFLOW MODEL RESULTS - GATES FULLY OPEN
2 YEAR ARI

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80 0 80 160 240 m

Scale: 1:8,000

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**LEGEND**

Levee

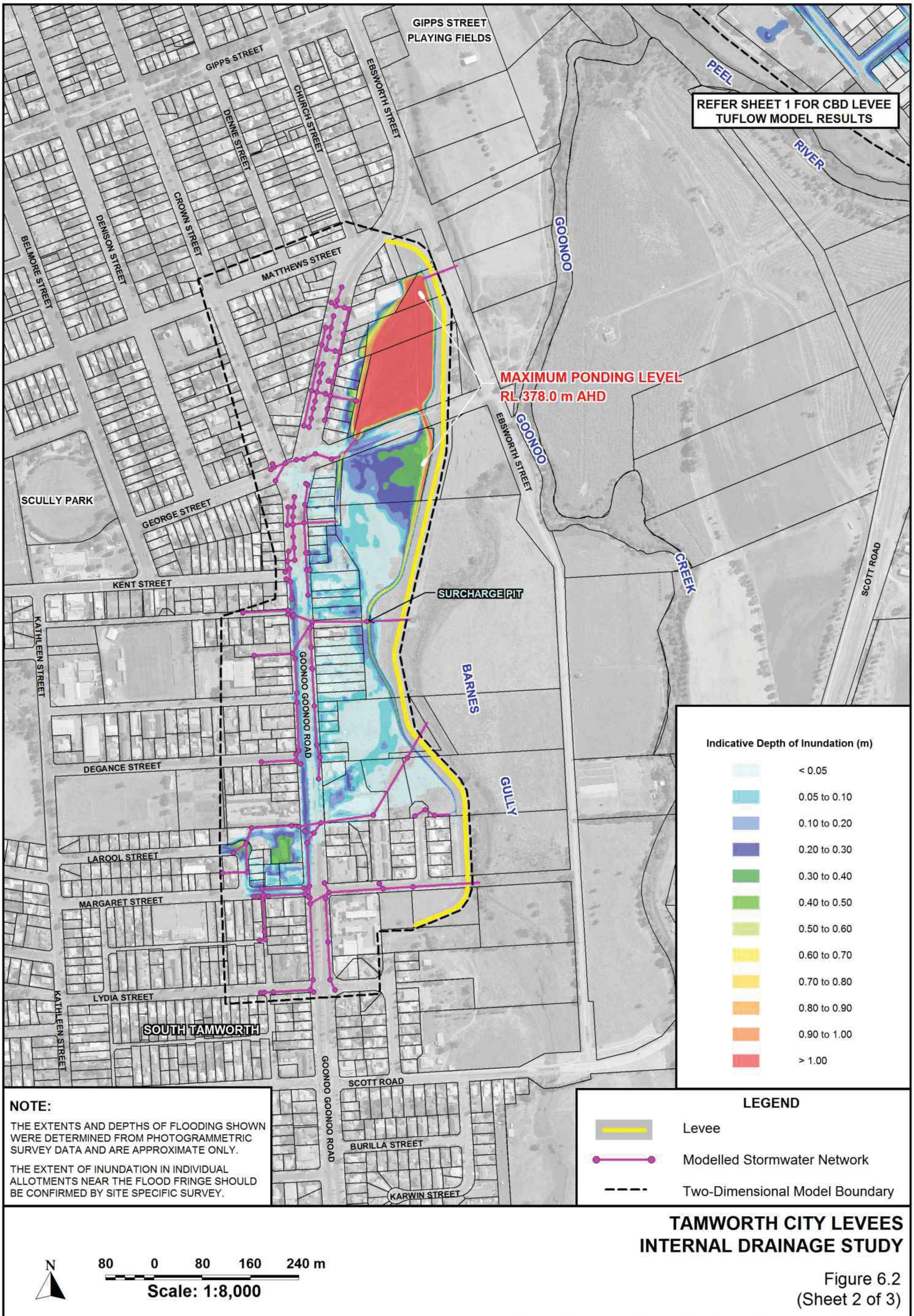
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.2
(Sheet 1 of 3)

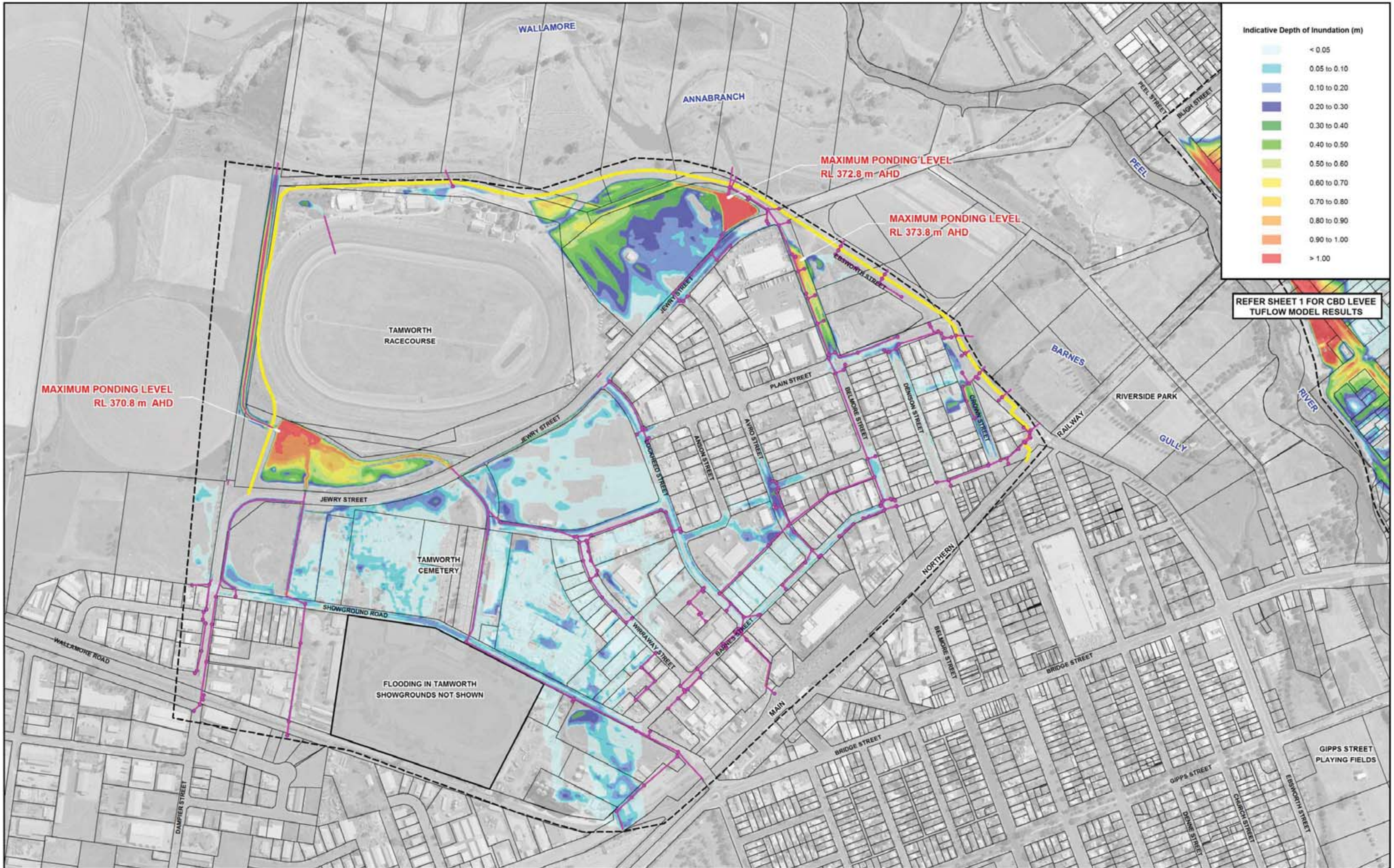
TUFLOW MODEL RESULTS - FLOOD GATES FULLY CLOSED
2 YEAR ARI




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.2
(Sheet 2 of 3)

TUFLOW MODEL RESULTS - GATES FULLY CLOSED
2 YEAR ARI

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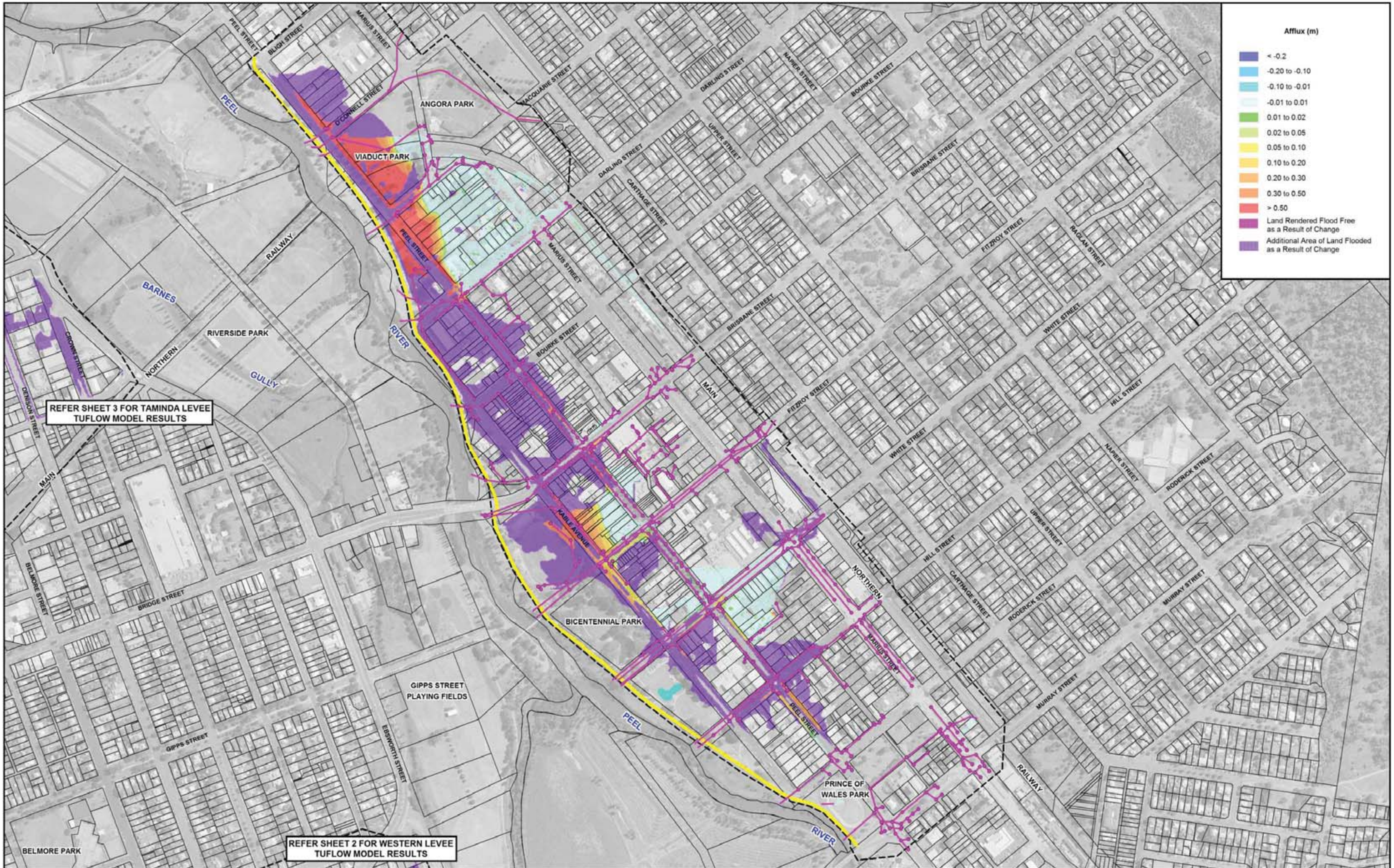
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.2
(Sheet 3 of 3)

TUFLOW MODEL RESULTS - GATES FULLY CLOSED
2 YEAR ARI




N

80 0 80 160 240 m

Scale: 1:8,000

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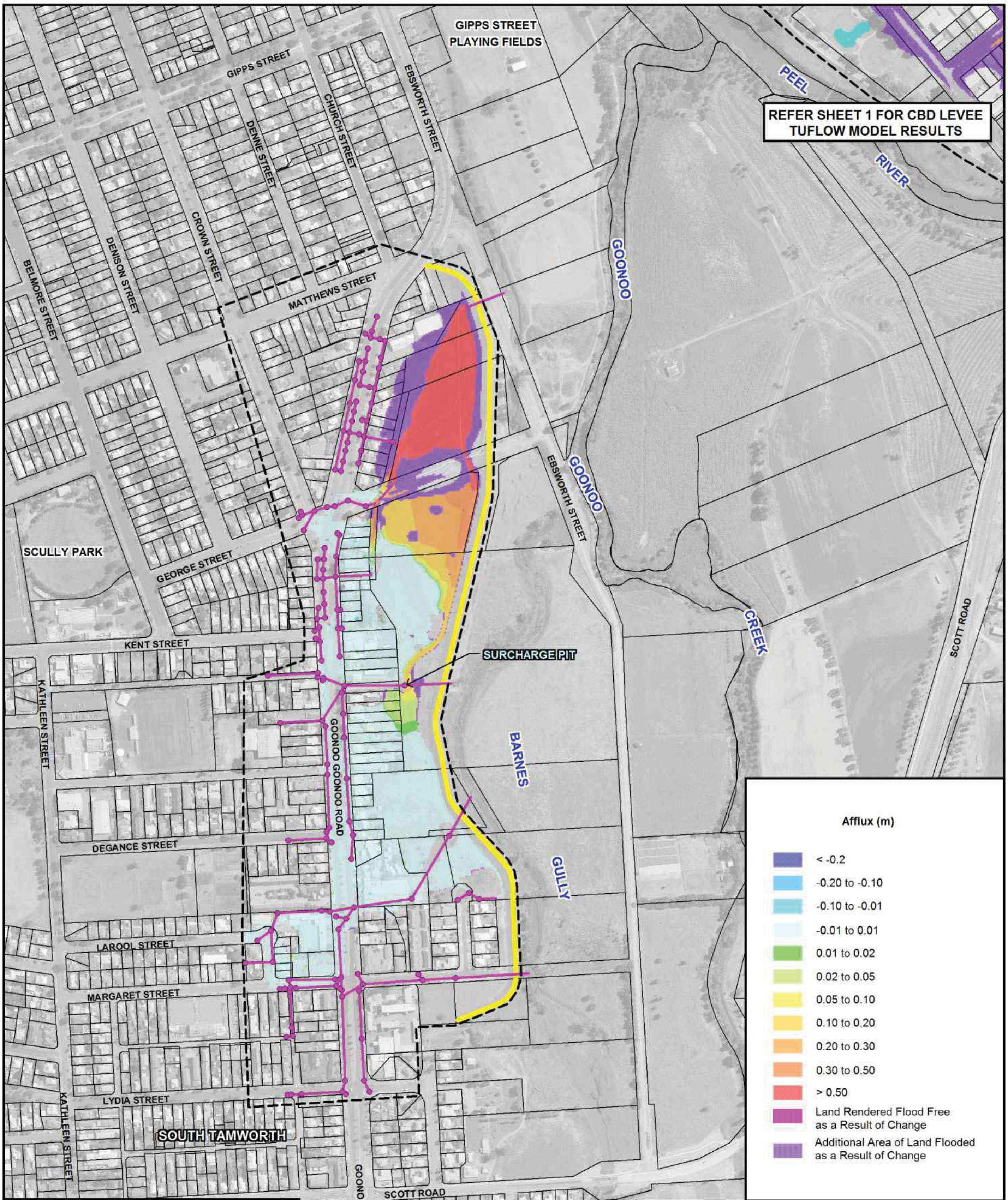
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.3
(Sheet 1 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
2 YEAR ARI

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**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

N

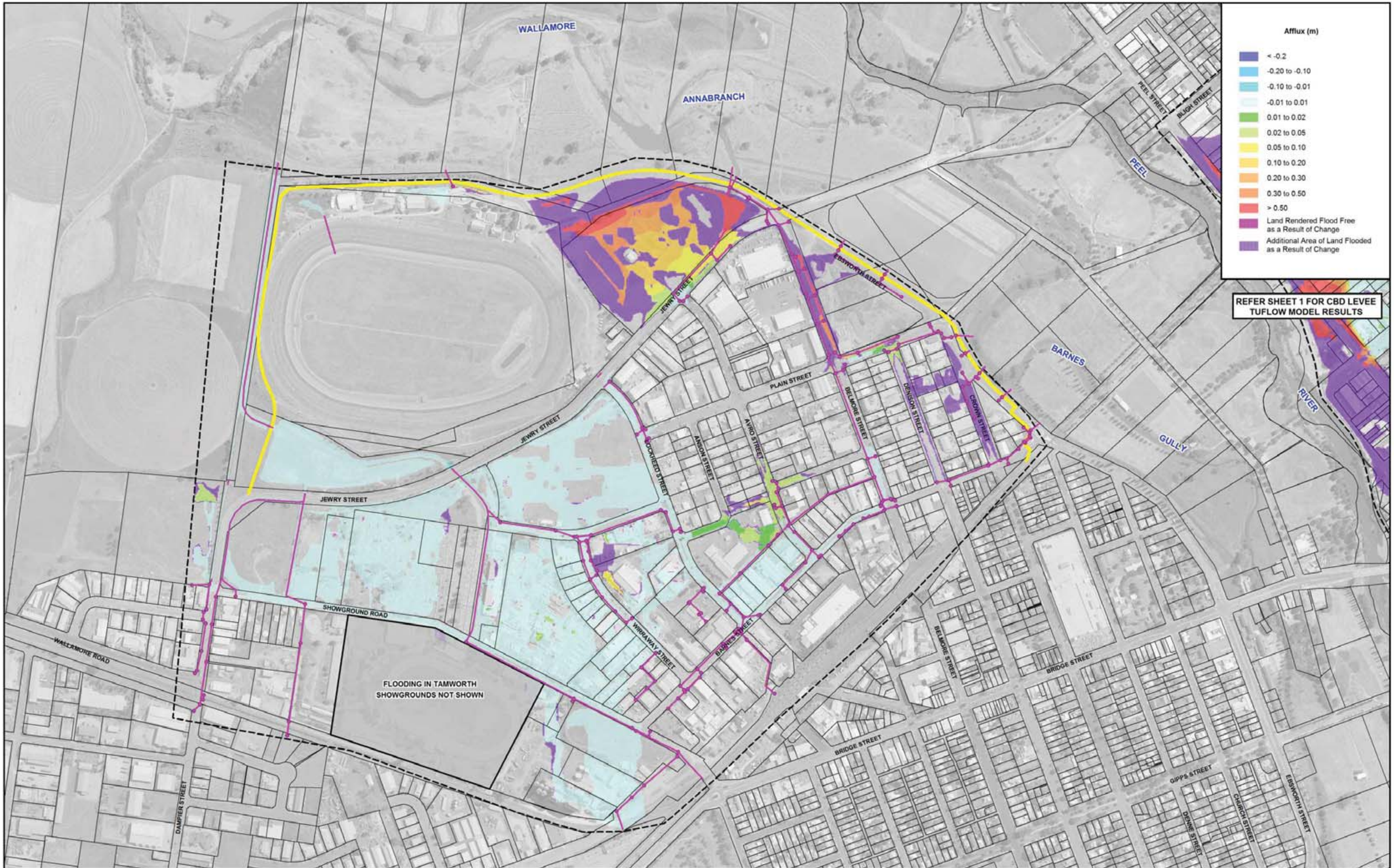
80 0 80 160 240 m

**Scale: 1:8,000**

**TAMWORTH CITY LEVEES INTERNAL DRAINAGE STUDY**

Figure 6.3
(Sheet 2 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
2 YEAR ARI

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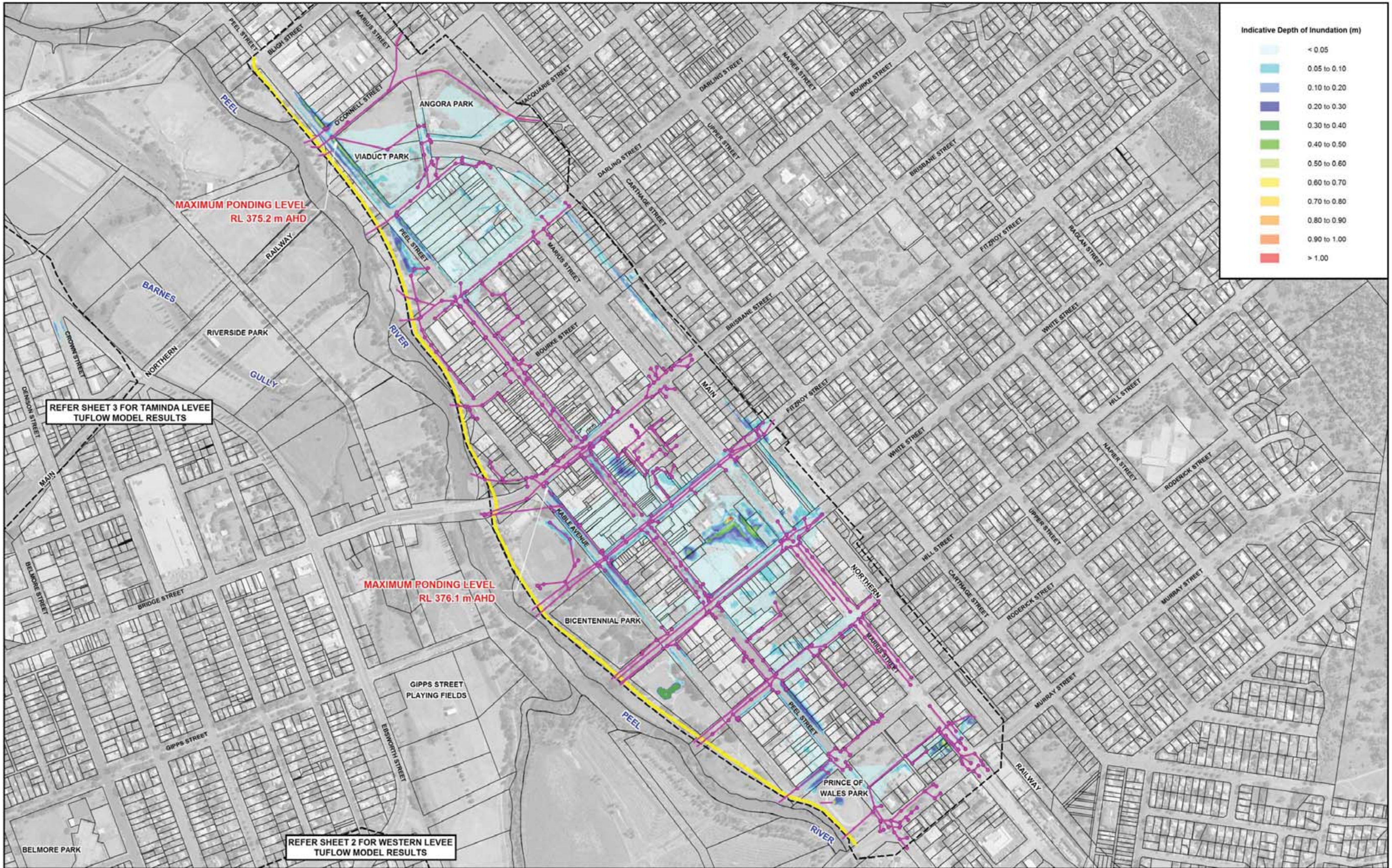
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.3
(Sheet 3 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
2 YEAR ARI

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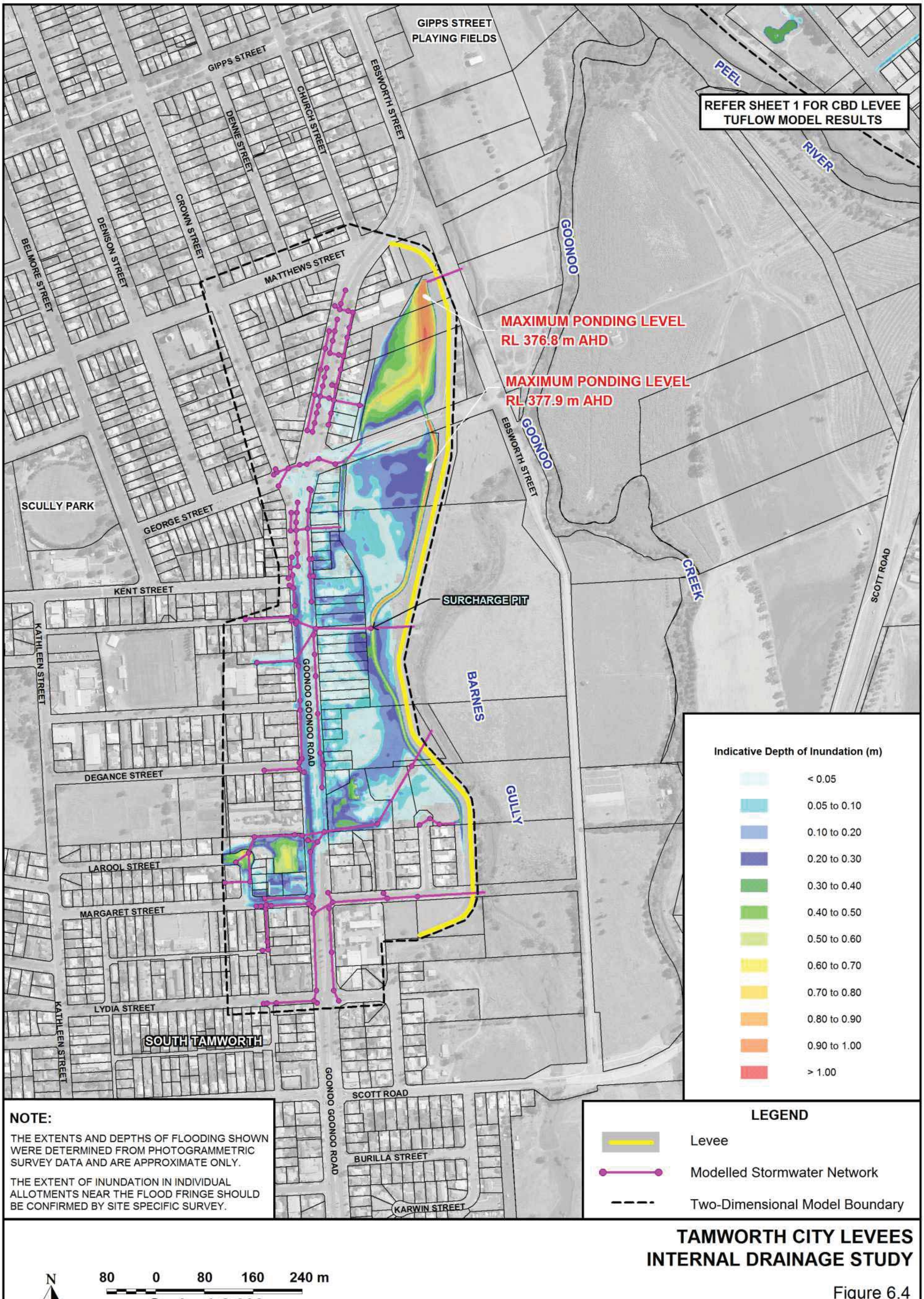
LEGEND

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.4
(Sheet 1 of 3)

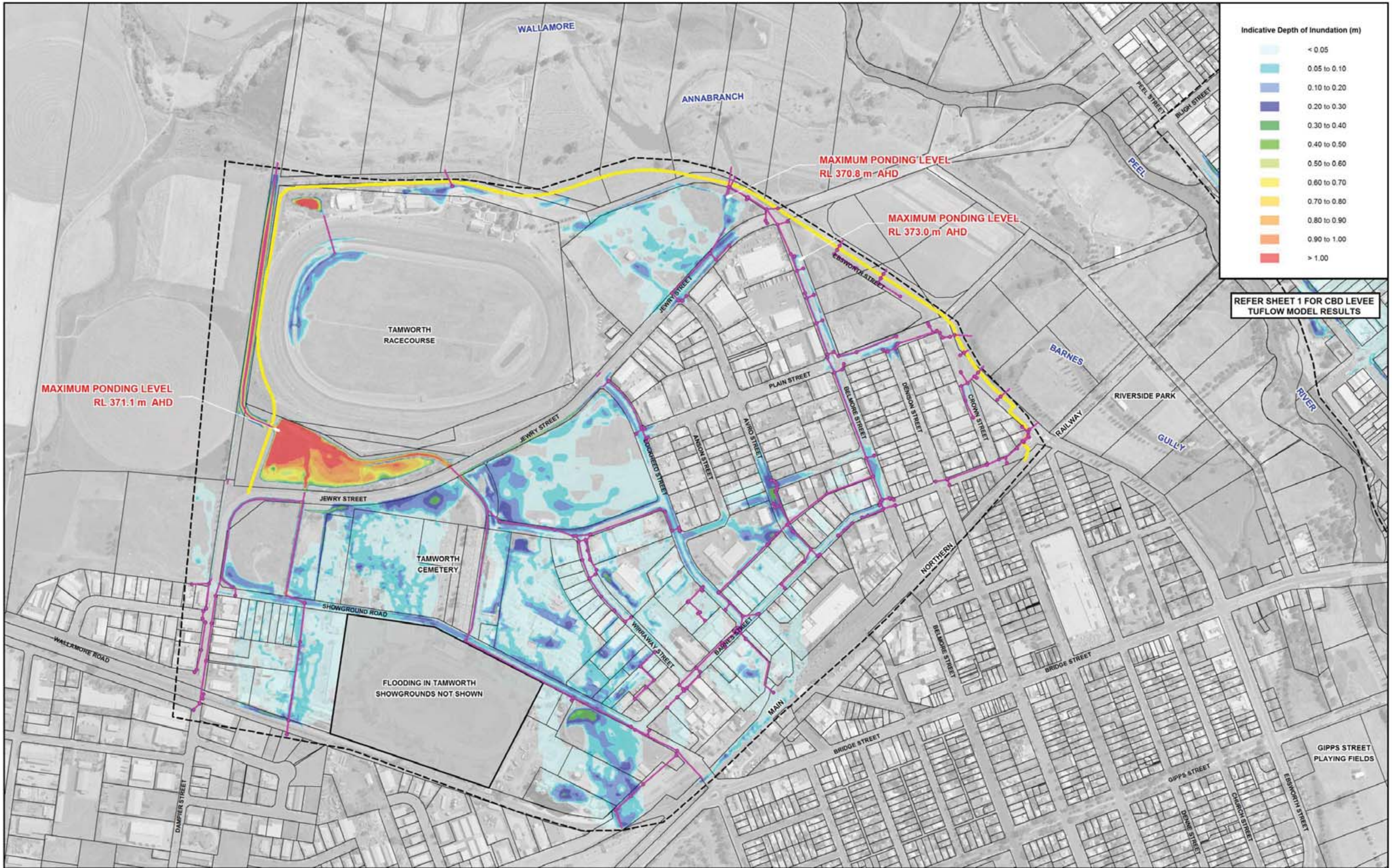
TUFLOW MODEL RESULTS - FLOOD GATES FULLY OPEN
5 YEAR ARI




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.4
(Sheet 2 of 3)

TUFLOW MODEL RESULTS - GATES FULLY OPEN
5 YEAR ARI

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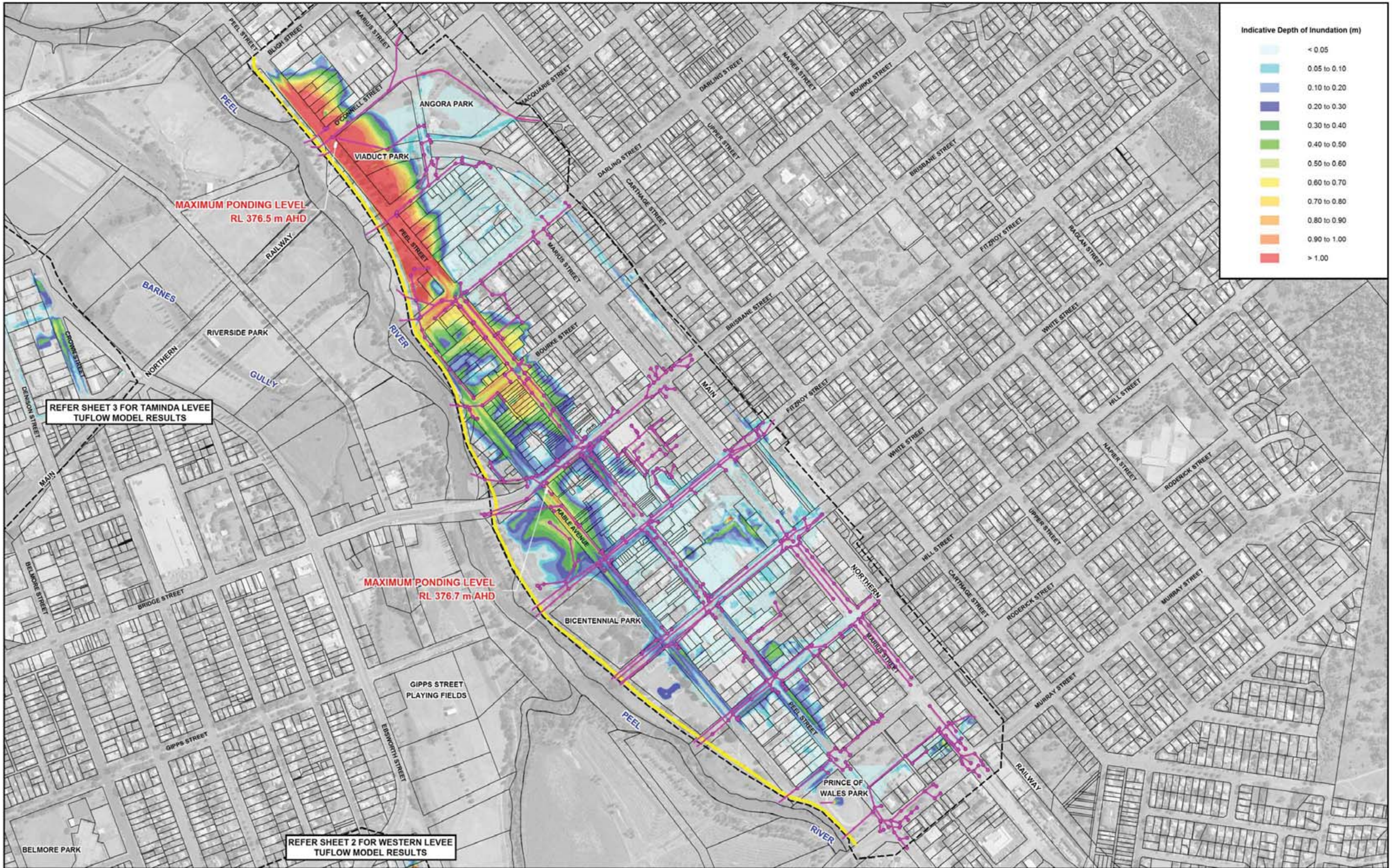
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.4
(Sheet 3 of 3)

TUFLOW MODEL RESULTS - GATES FULLY OPEN
5 YEAR ARI

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LEGEND

Levee

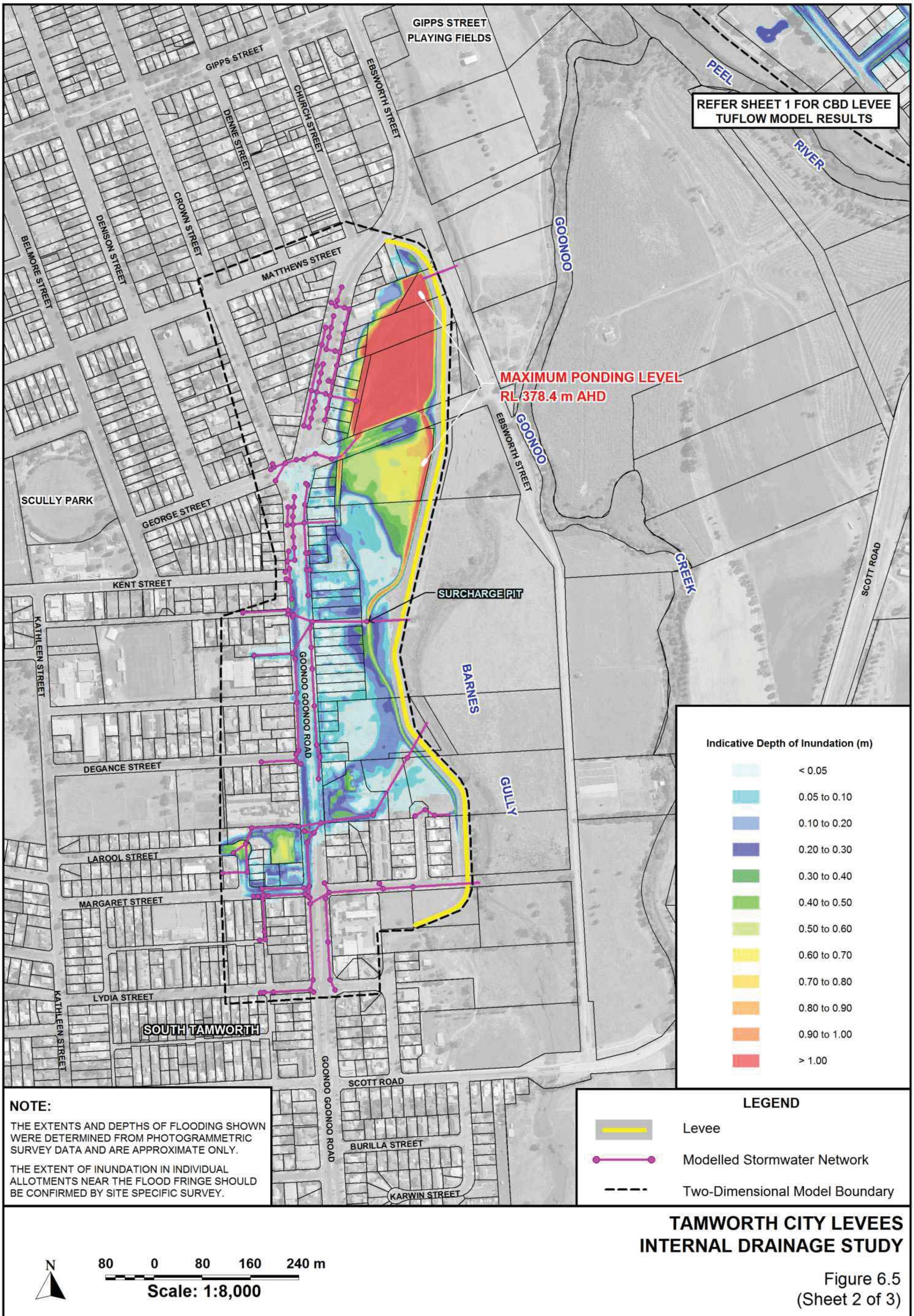
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.5
(Sheet 1 of 3)

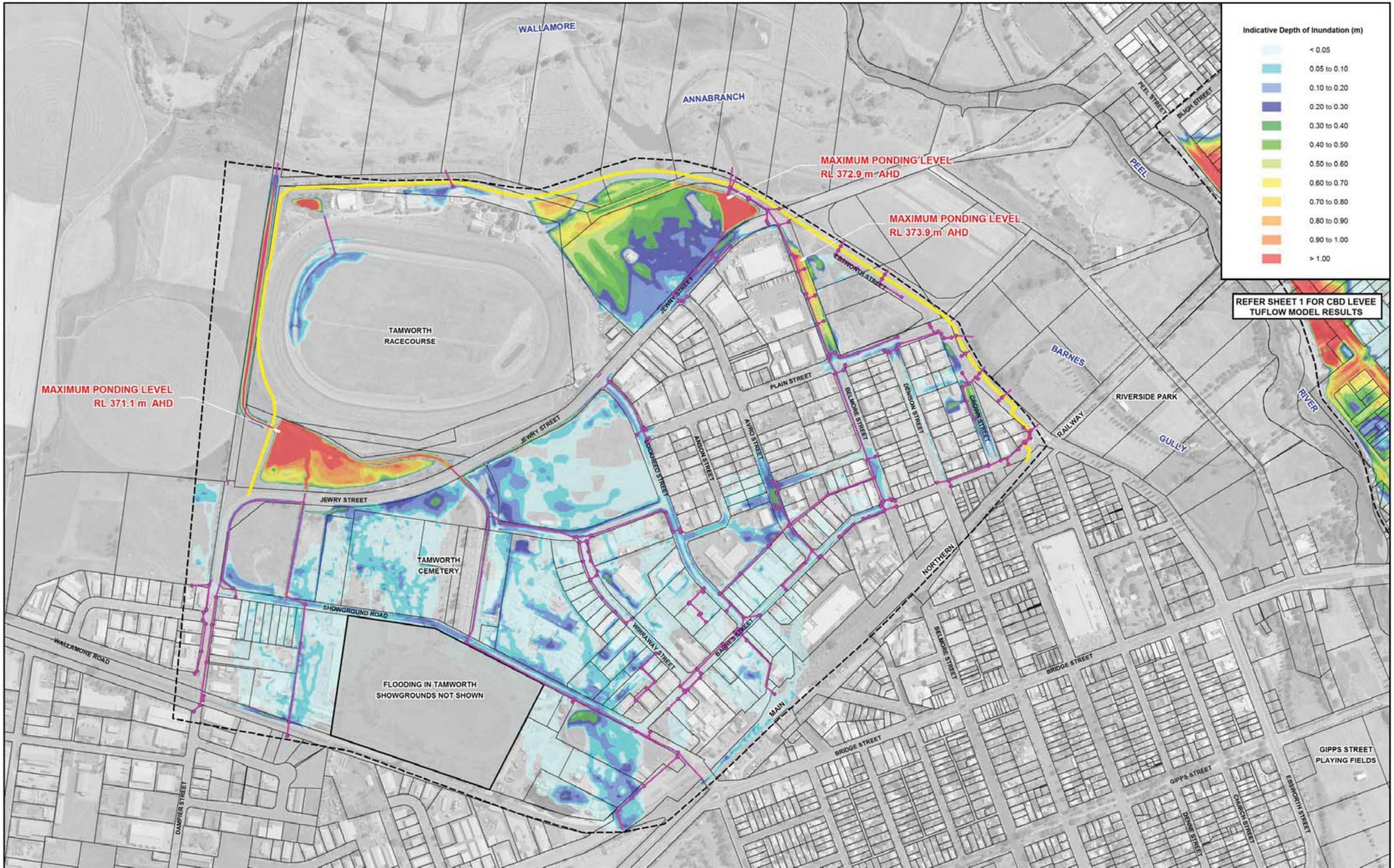
TUFLOW MODEL RESULTS - FLOOD GATES FULLY CLOSED
5 YEAR ARI




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.5
(Sheet 2 of 3)

TUFLOW MODEL RESULTS - GATES FULLY CLOSED
5 YEAR ARI

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LEGEND

Levee

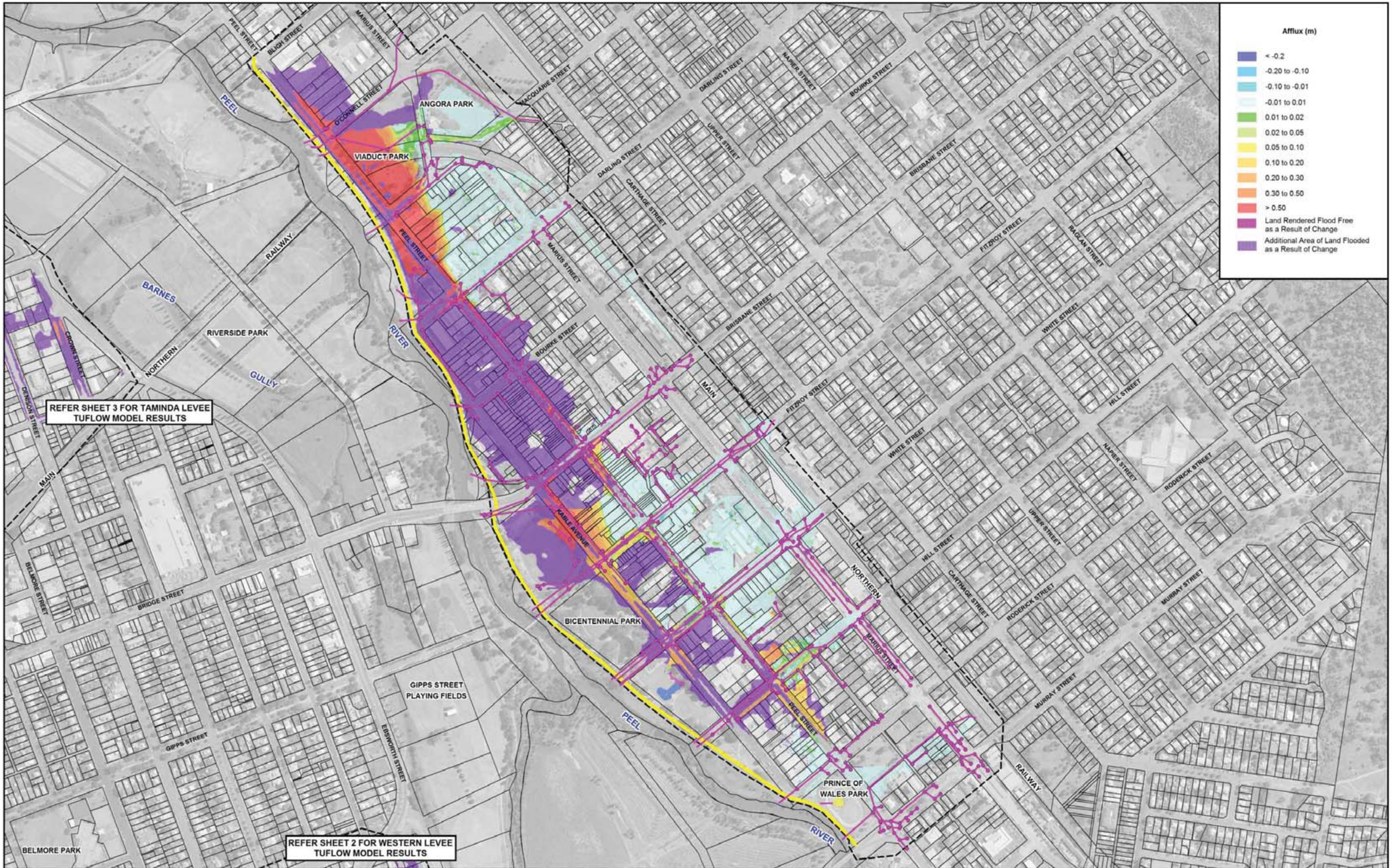
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.5
(Sheet 3 of 3)

TUFLOW MODEL RESULTS - GATES FULLY CLOSED
5 YEAR ARI

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LEGEND

Levee

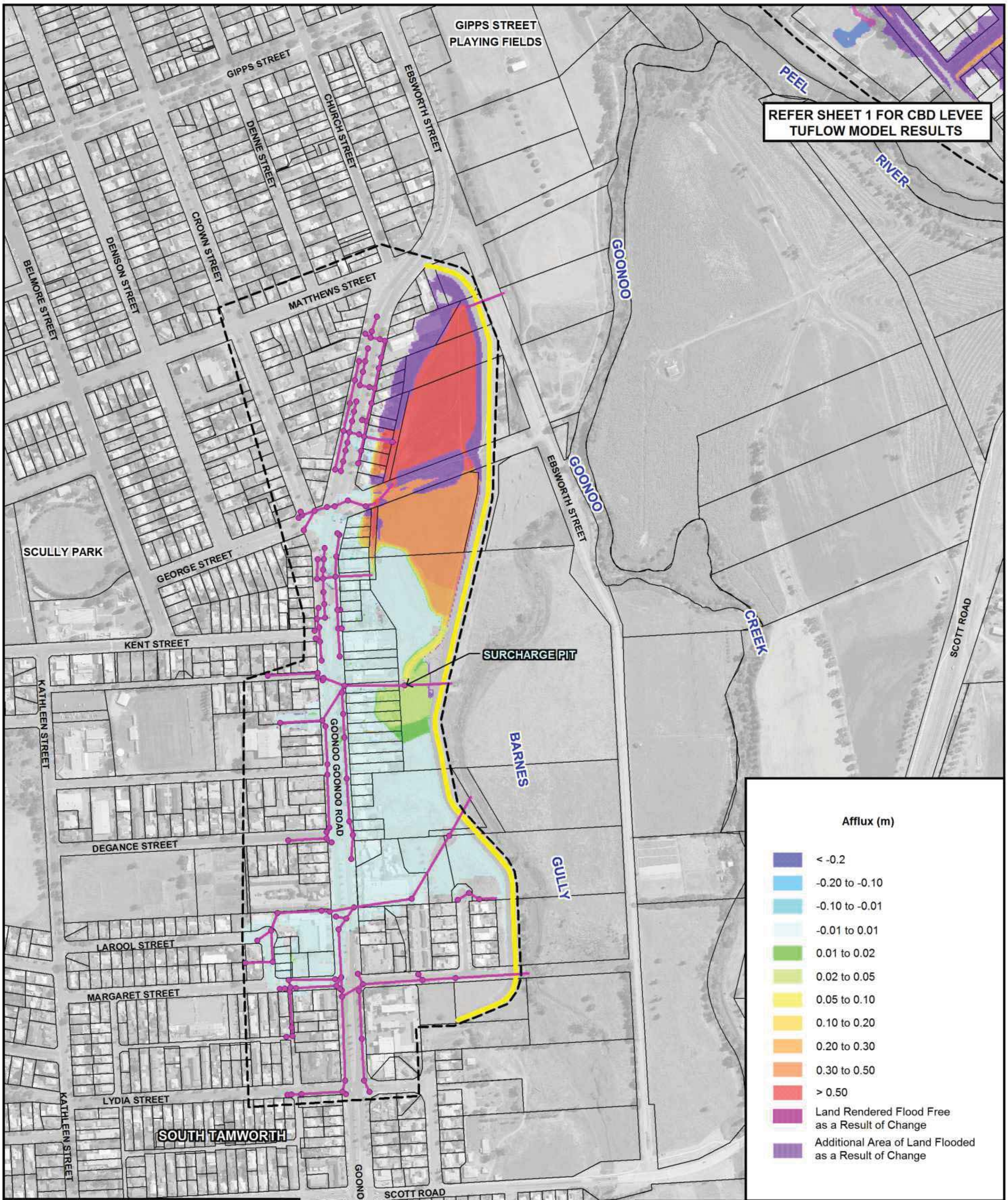
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.6
(Sheet 1 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
5 YEAR ARI

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**TAMWORTH CITY LEVEES INTERNAL DRAINAGE STUDY**

Figure 6.6
(Sheet 2 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
5 YEAR ARI


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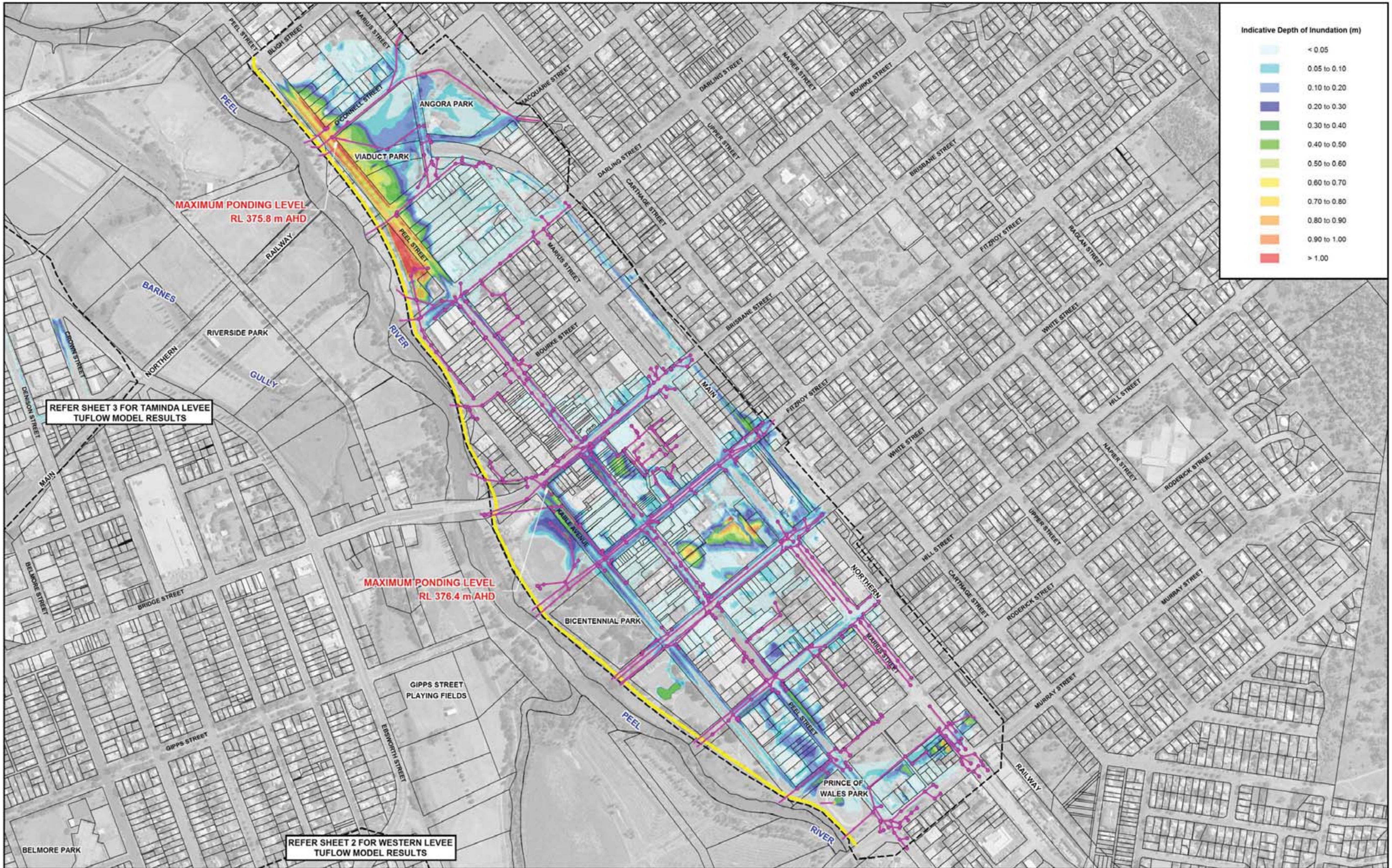
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.6
(Sheet 3 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
5 YEAR ARI

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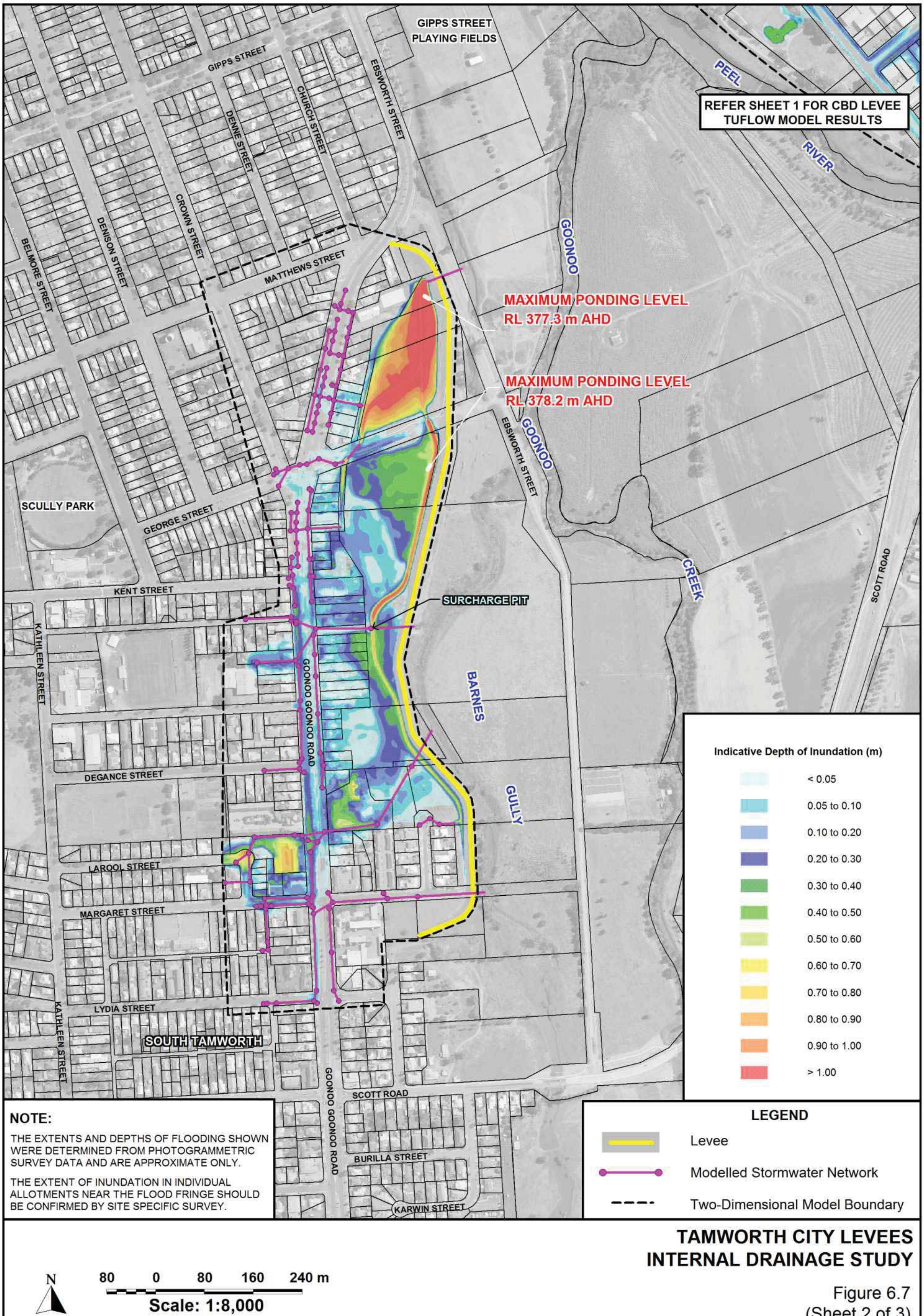
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.7
(Sheet 1 of 3)

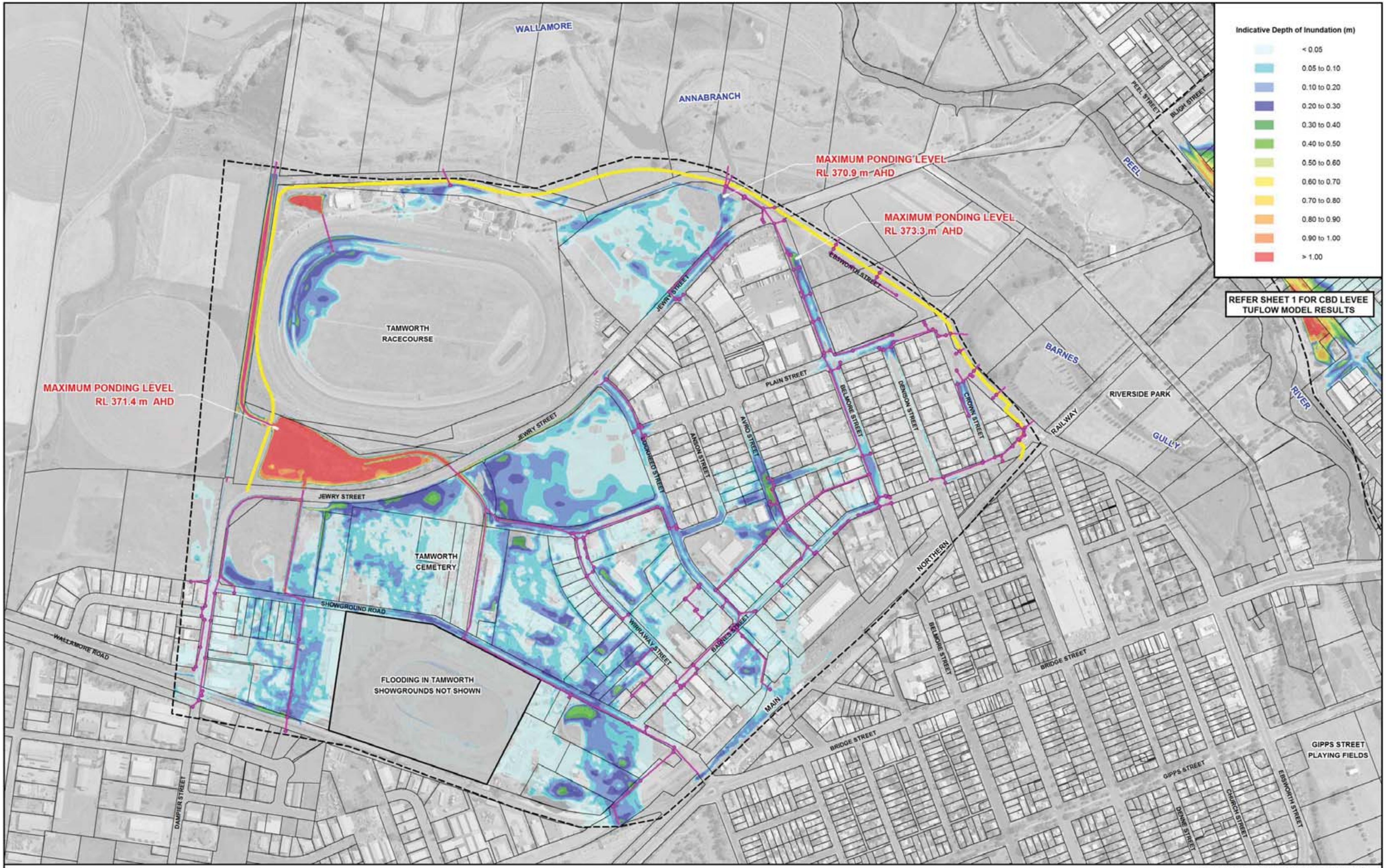
TUFLOW MODEL RESULTS - FLOOD GATES FULLY OPEN
20 YEAR ARI




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.7
(Sheet 2 of 3)

TUFLOW MODEL RESULTS - GATES FULLY OPEN
20 YEAR ARI




80 0 80 160 240 m

Scale: 1:8,000

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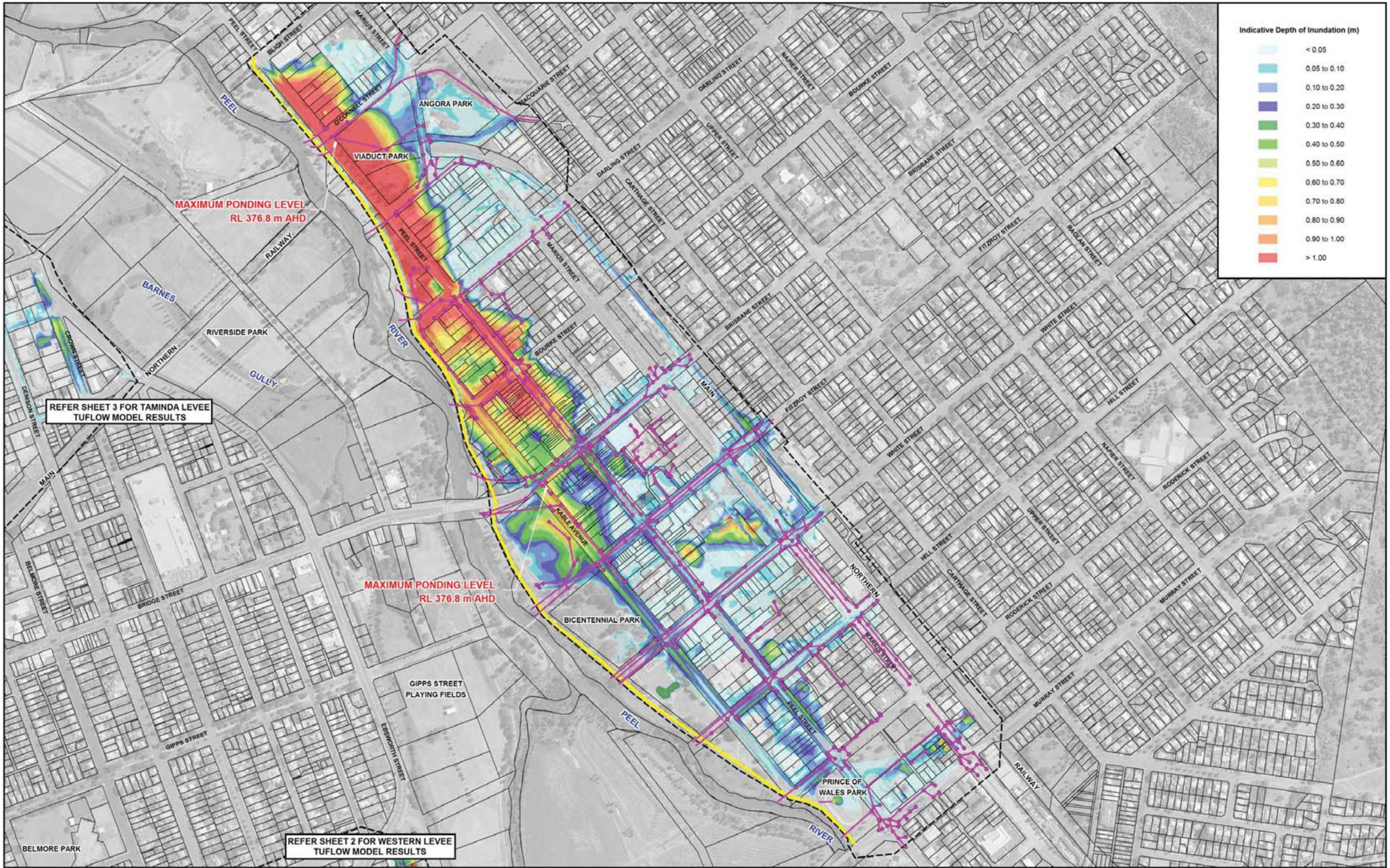
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.7
(Sheet 3 of 3)

TUFLOW MODEL RESULTS - GATES FULLY OPEN
20 YEAR ARI

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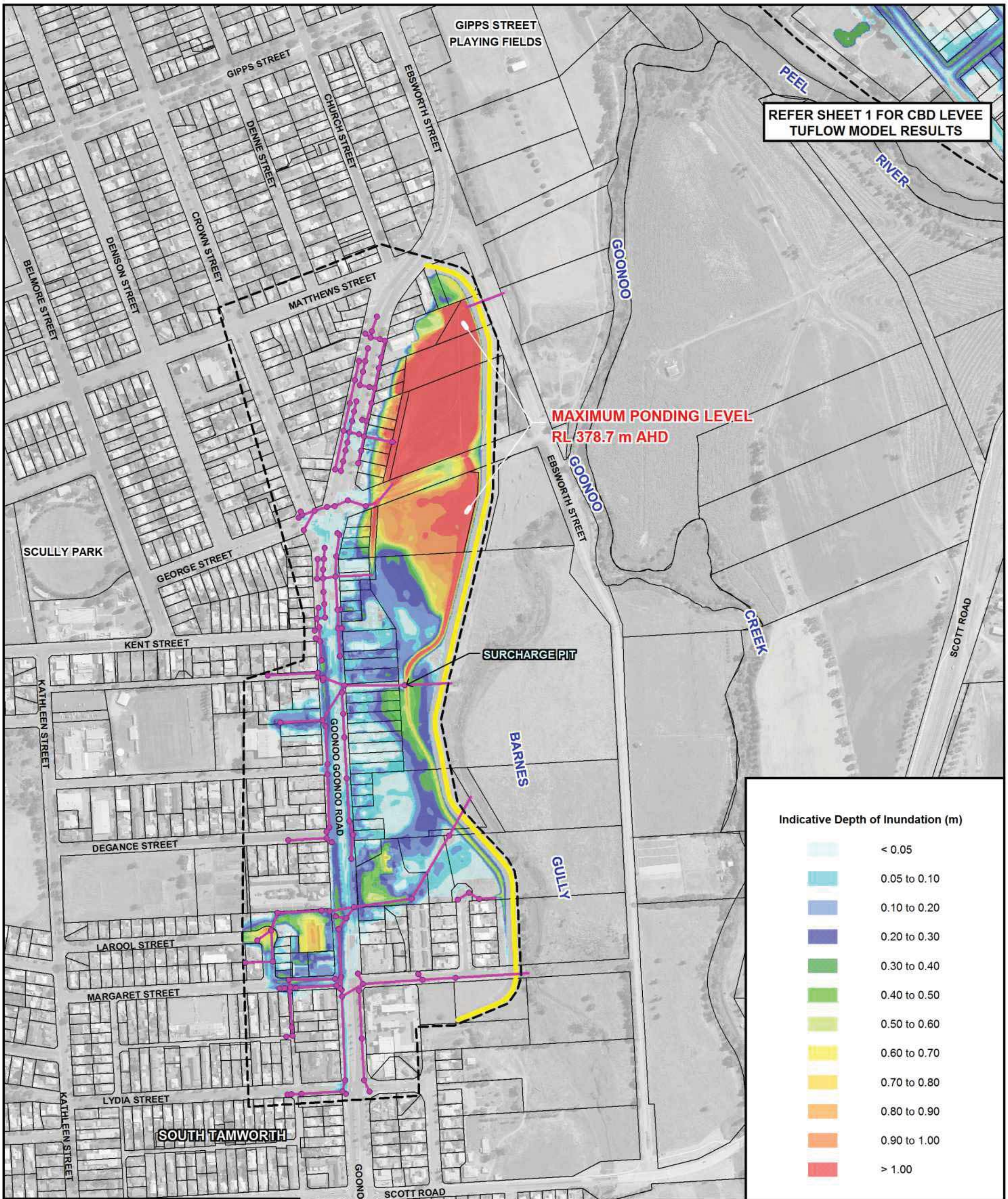
LEGEND

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.8
(Sheet 1 of 3)

TUFLOW MODEL RESULTS - FLOOD GATES FULLY CLOSED
20 YEAR ARI

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**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

N

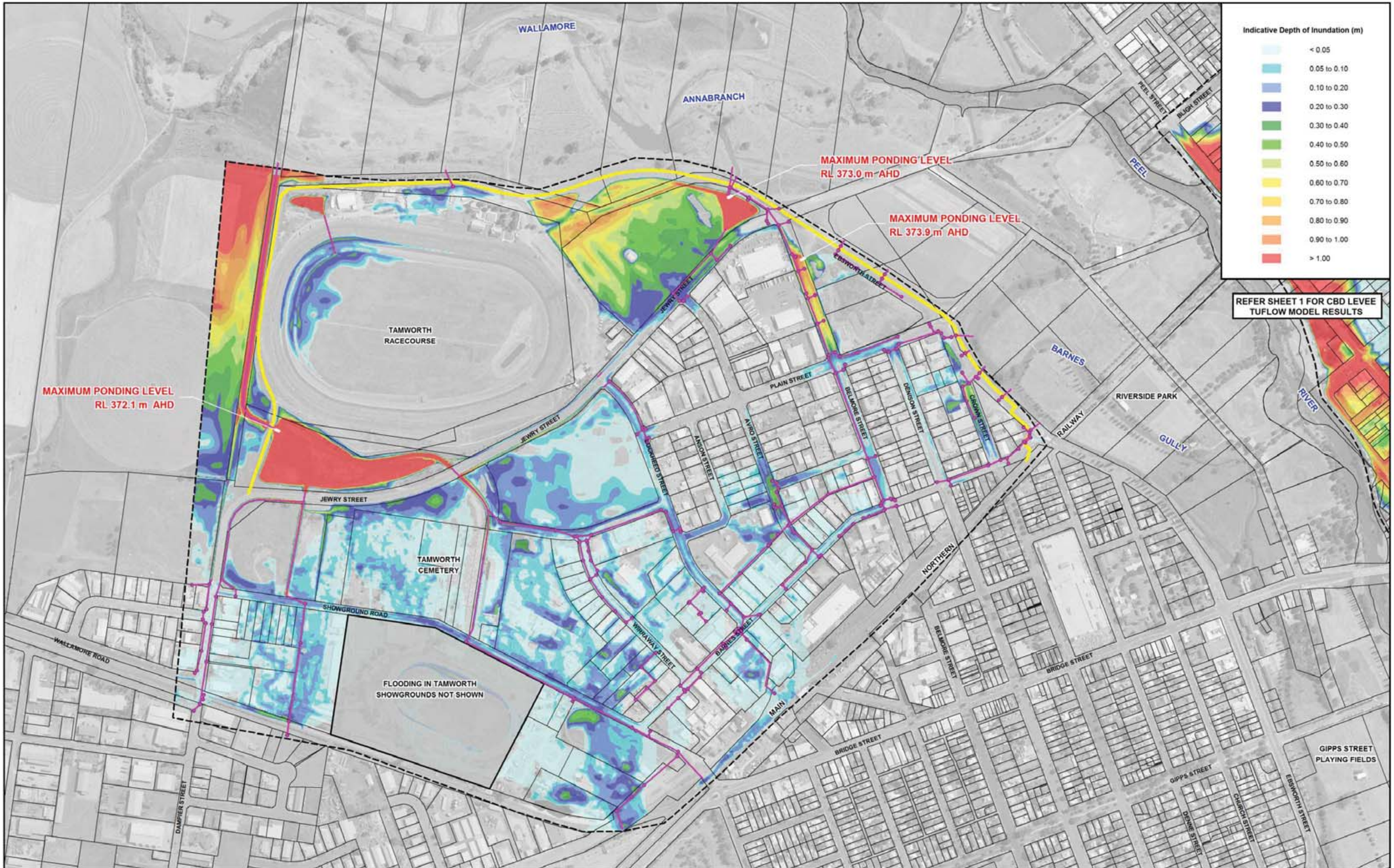
80 0 80 160 240 m

Scale: 1:8,000

**TAMWORTH CITY LEVEES INTERNAL DRAINAGE STUDY**

Figure 6.8
(Sheet 2 of 3)

TUFLOW MODEL RESULTS - GATES FULLY CLOSED
20 YEAR ARI

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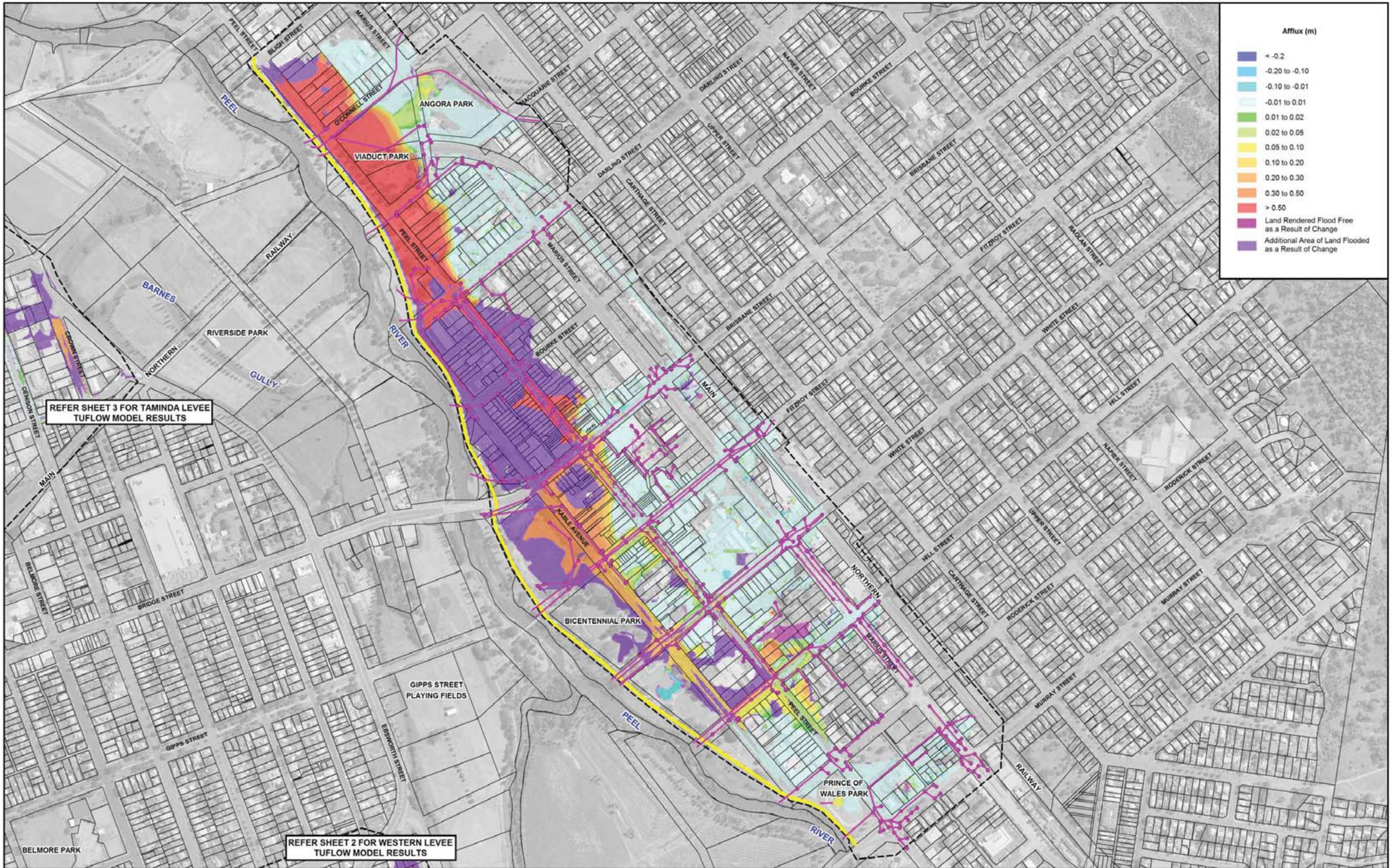
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.8
(Sheet 3 of 3)

TUFLOW MODEL RESULTS - GATES FULLY CLOSED
20 YEAR ARI

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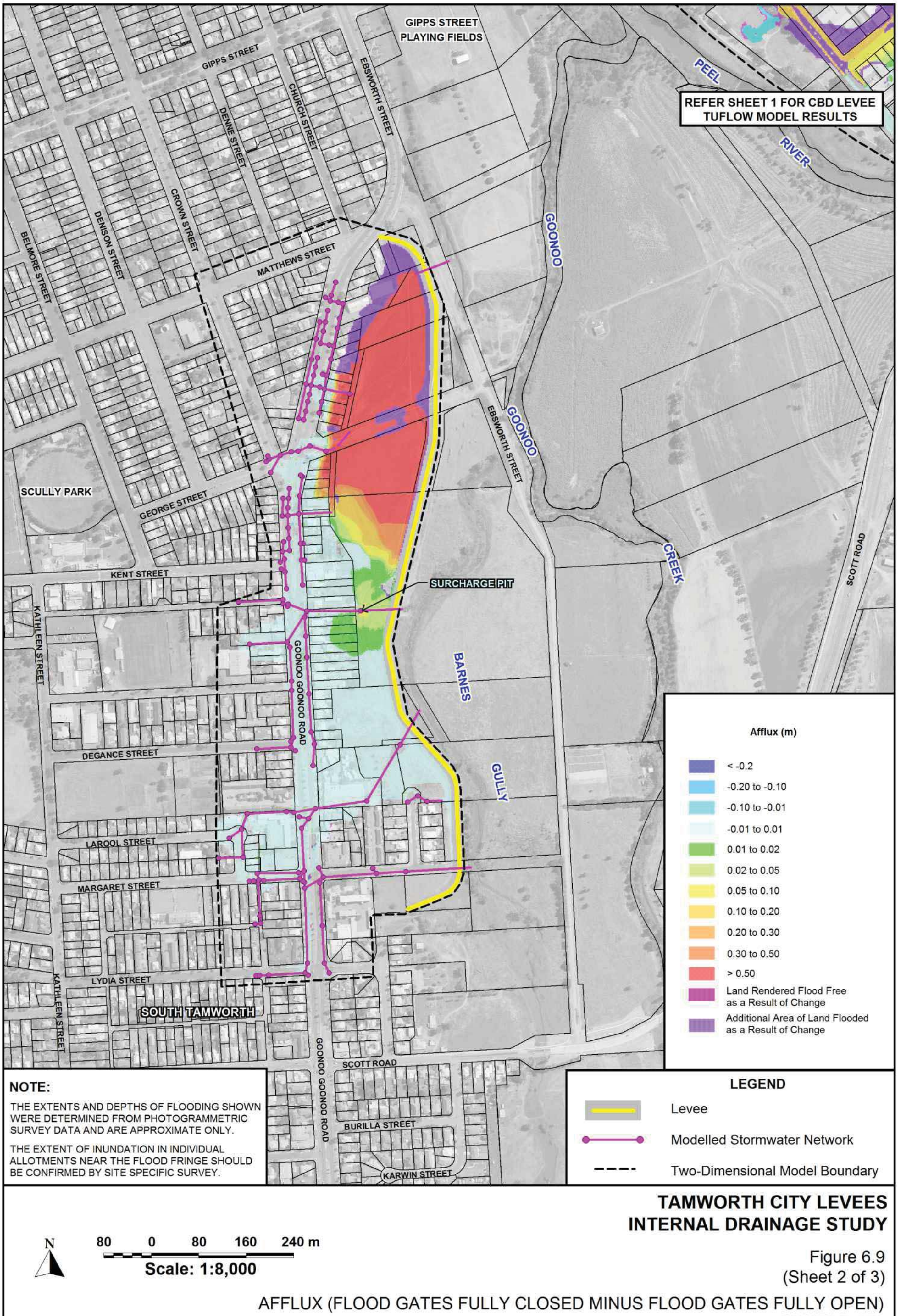
**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.9
(Sheet 1 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
20 YEAR ARI




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.9
(Sheet 2 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
20 YEAR ARI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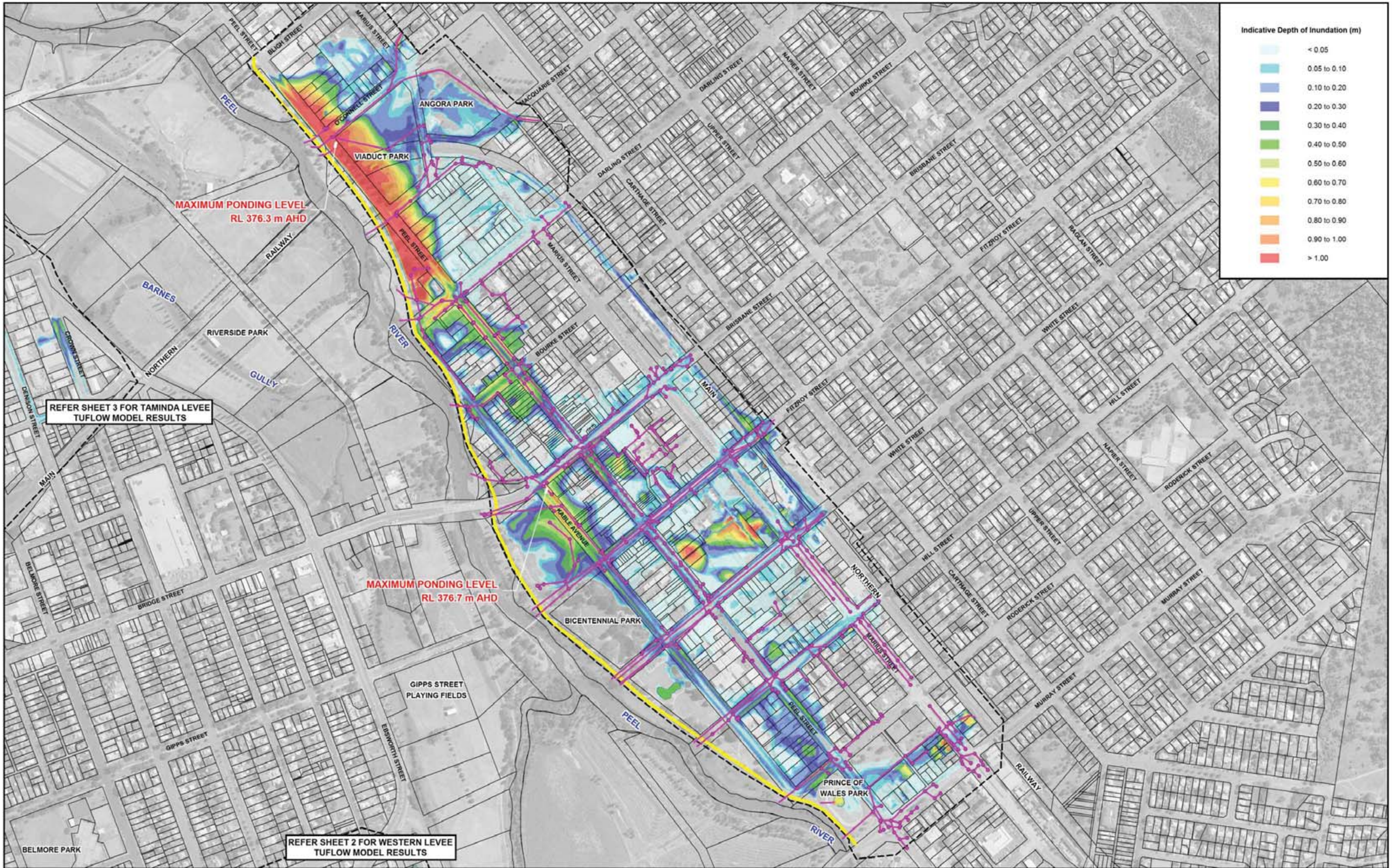
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.9
(Sheet 3 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
20 YEAR ARI

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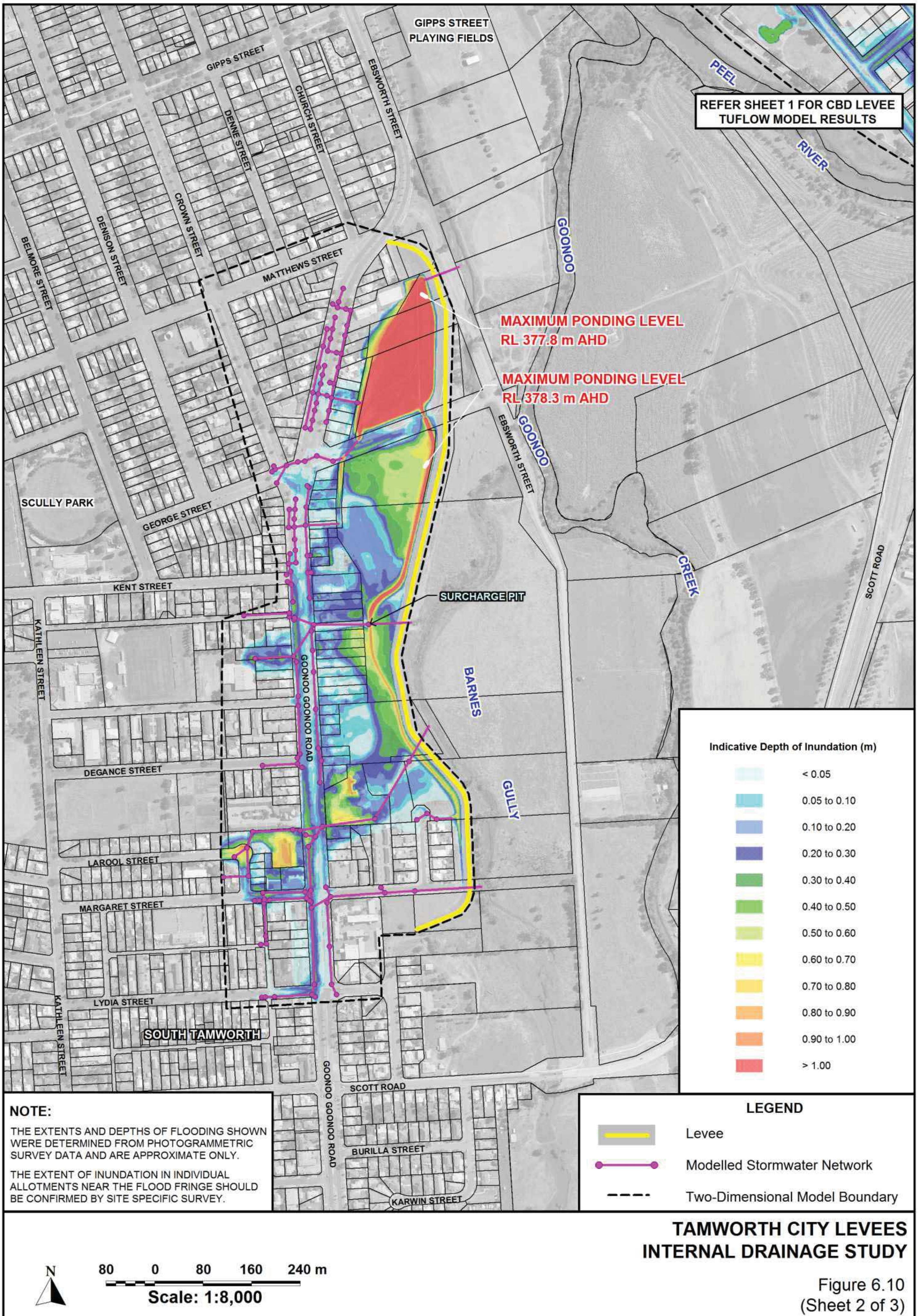
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.10
(Sheet 1 of 3)

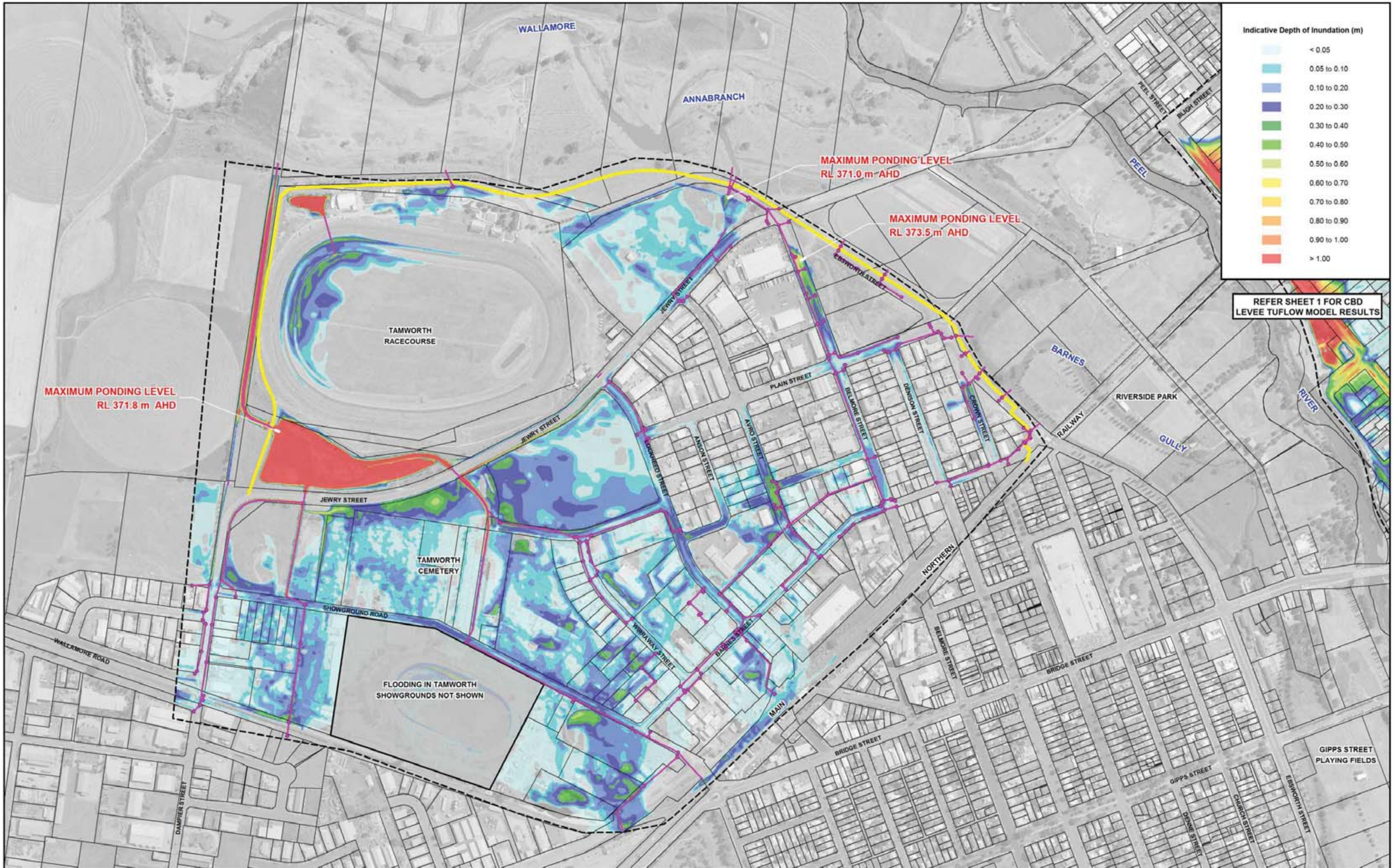
TUFLOW MODEL RESULTS - FLOOD GATES FULLY OPEN
100 YEAR ARI




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.10
(Sheet 2 of 3)

TUFLOW MODEL RESULTS - GATES FULLY OPEN
100 YEAR ARI

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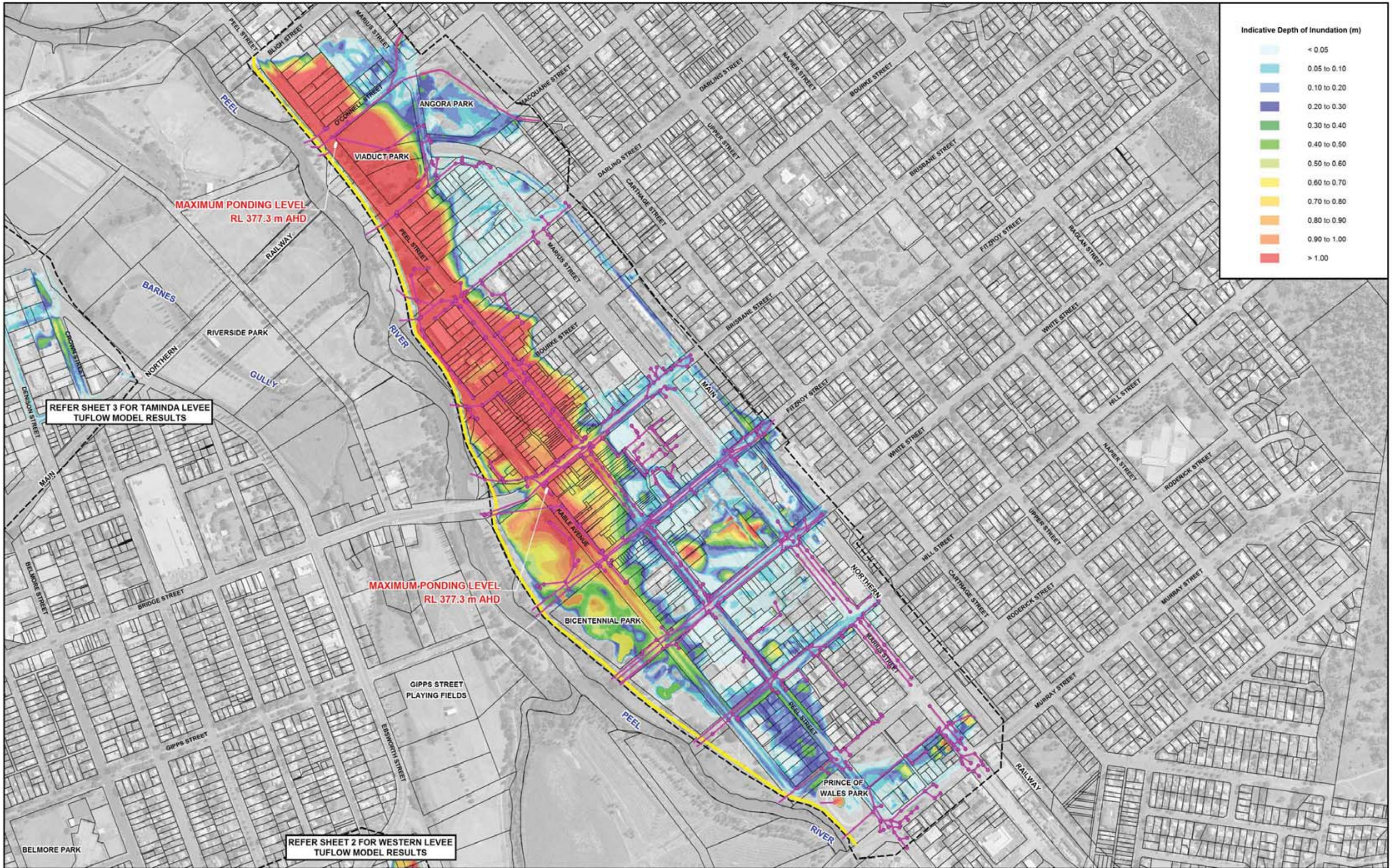
LEGEND

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.10
(Sheet 3 of 3)

TUFLOW MODEL RESULTS - GATES FULLY OPEN
100 YEAR ARI

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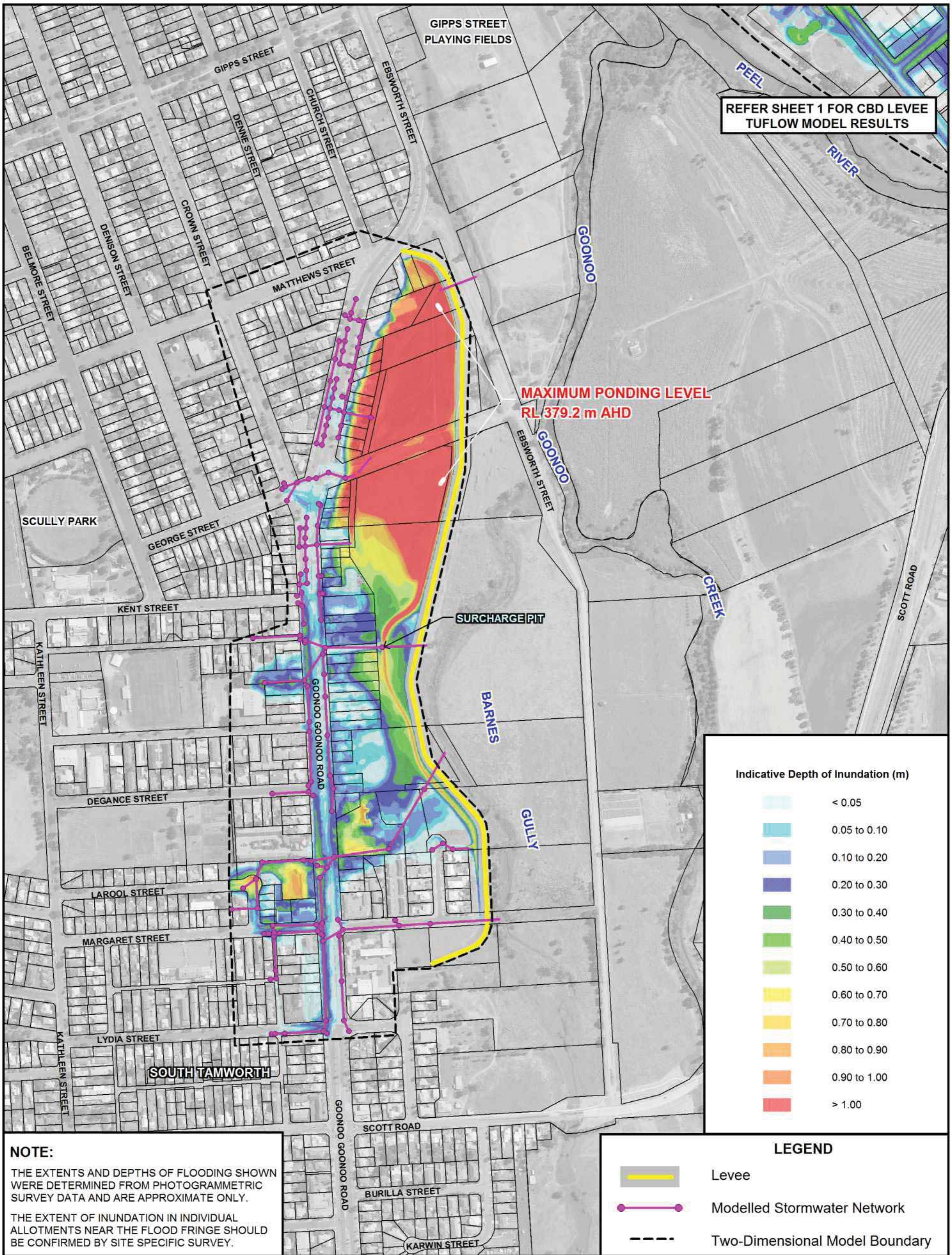
**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.11
(Sheet 1 of 3)

TUFLOW MODEL RESULTS - FLOOD GATES FULLY CLOSED
100 YEAR ARI




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

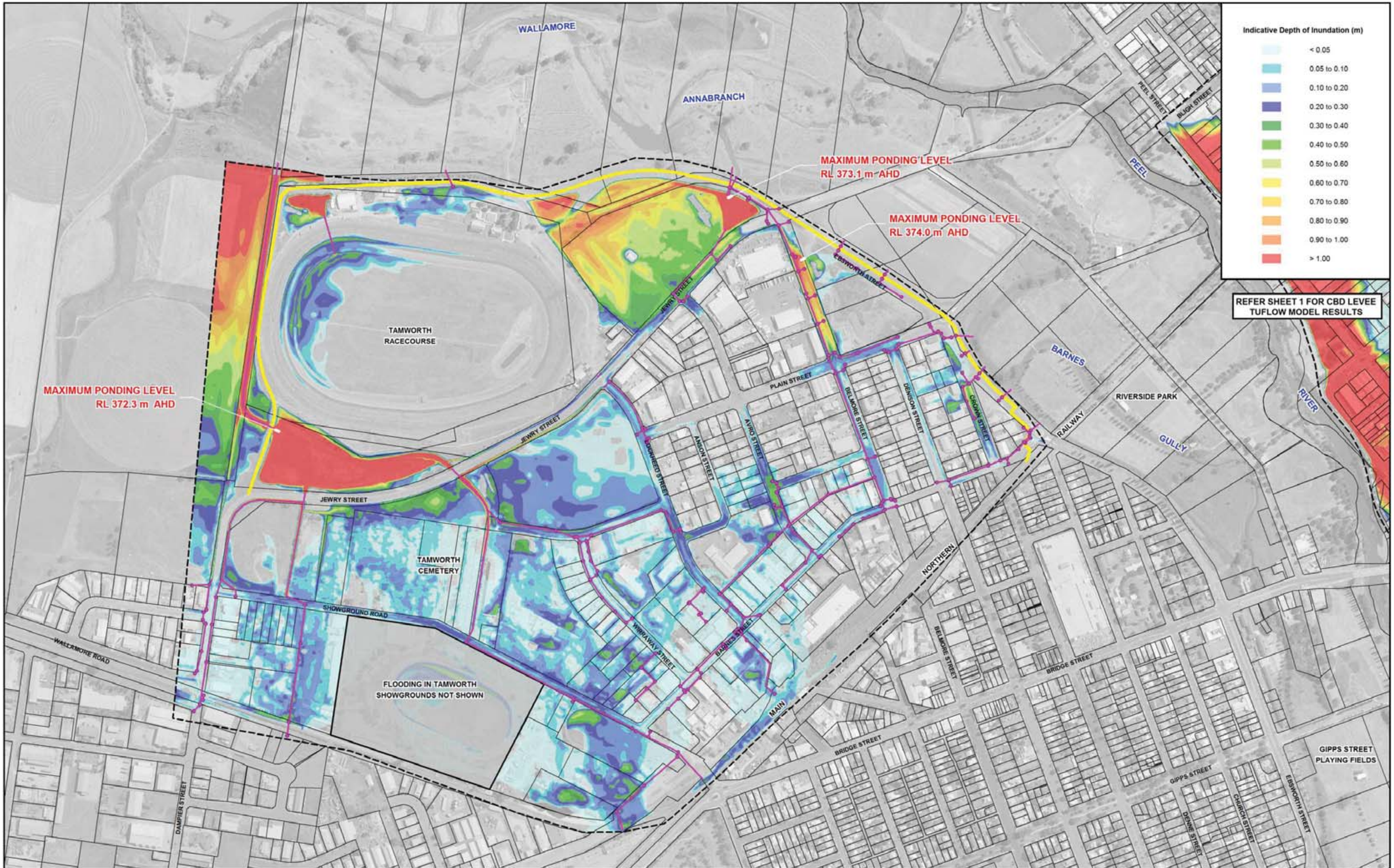
N

80 0 80 160 240 m

**Scale: 1:8,000**

Figure 6.11
(Sheet 2 of 3)

TUFLOW MODEL RESULTS - GATES FULLY CLOSED
100 YEAR ARI




N

80 0 80 160 240 m

Scale: 1:8,000

**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

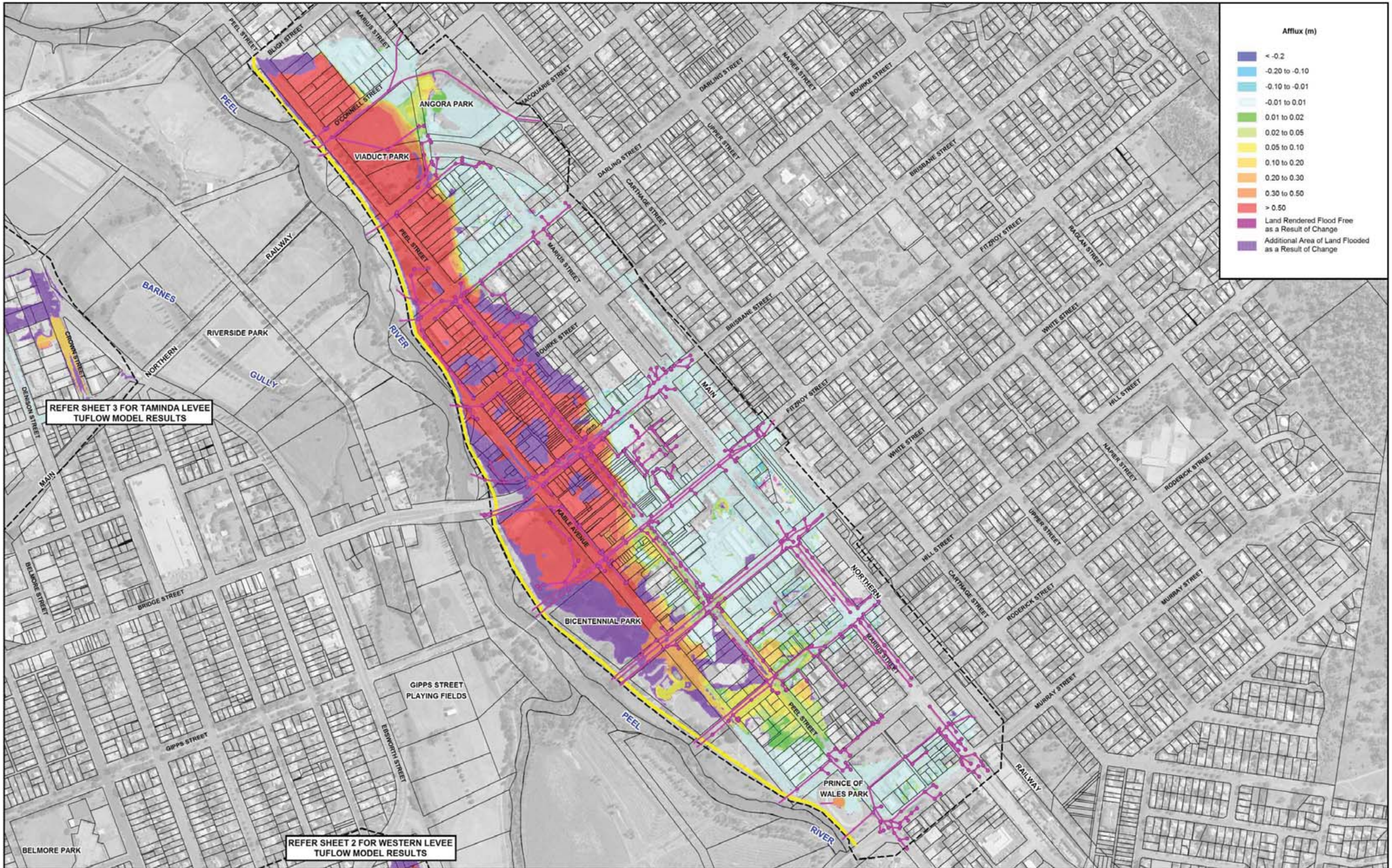
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.11
(Sheet 3 of 3)

TUFLOW MODEL RESULTS - GATES FULLY CLOSED
100 YEAR ARI




N

80 0 80 160 240 m

Scale: 1:8,000

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**LEGEND**

Levee

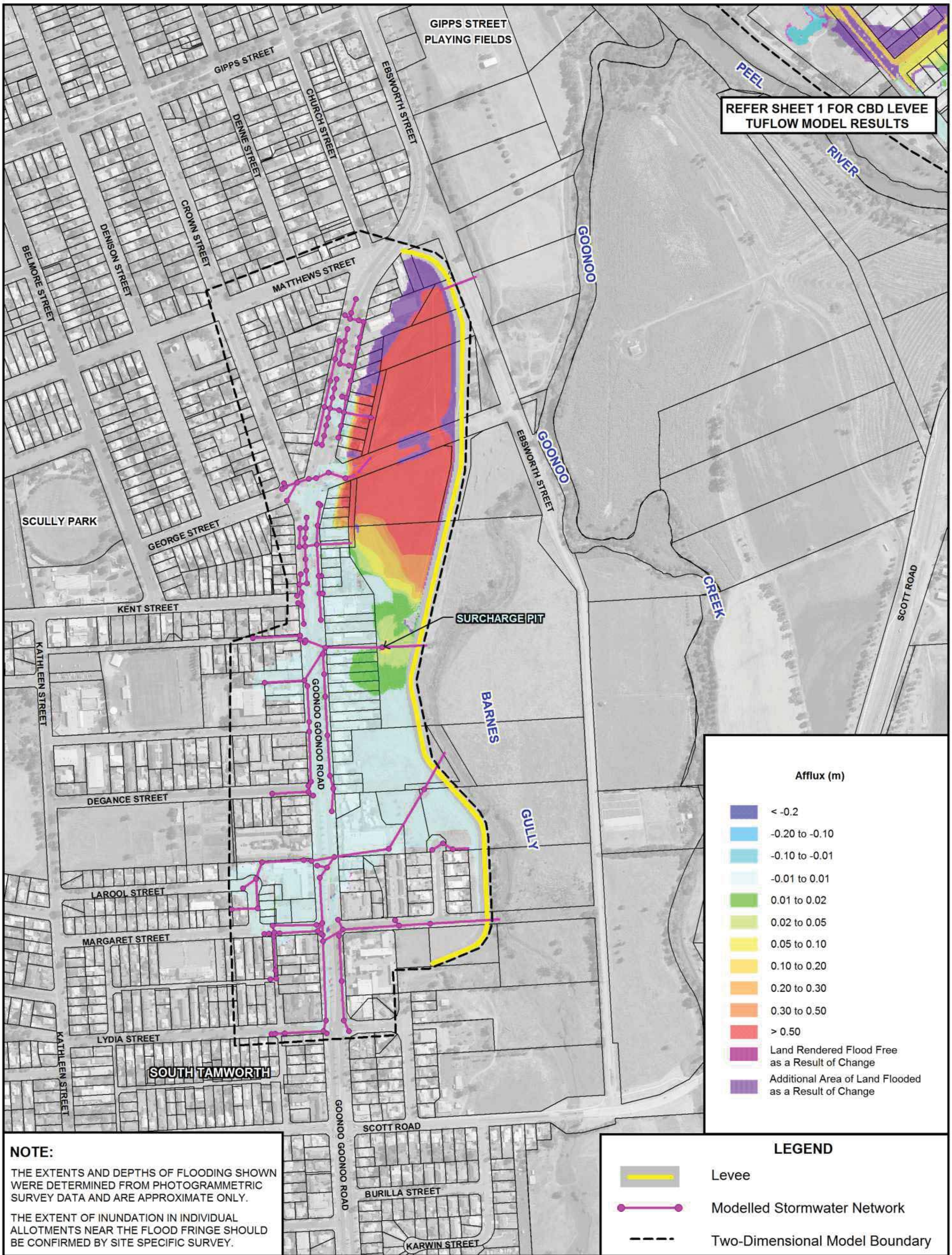
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.12
(Sheet 1 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
100 YEAR ARI




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

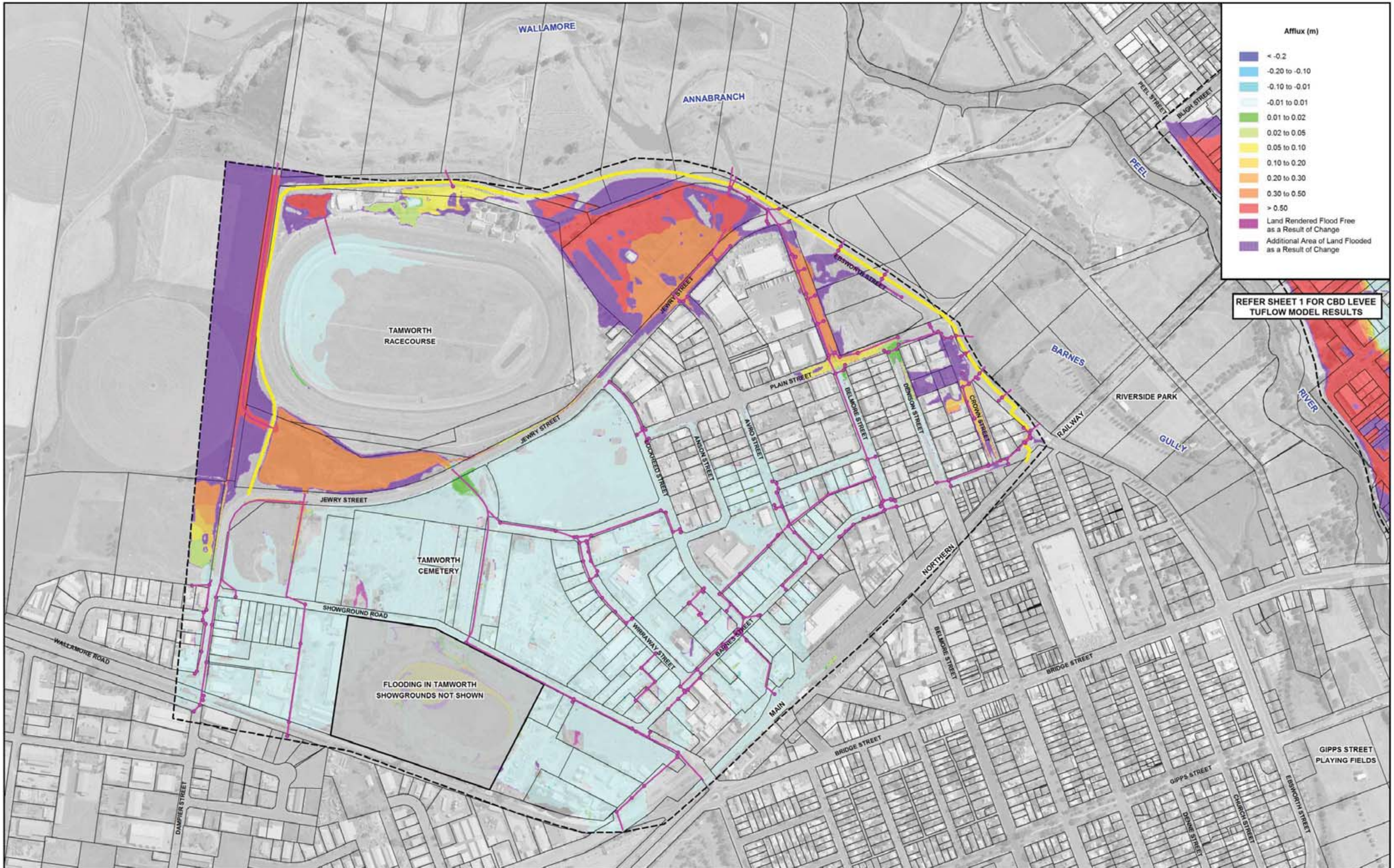
N

80 0 80 160 240 m

**Scale: 1:8,000**

Figure 6.12
(Sheet 2 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
100 YEAR ARI




N

80 0 80 160 240 m

Scale: 1:8,000

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**LEGEND**

Levee

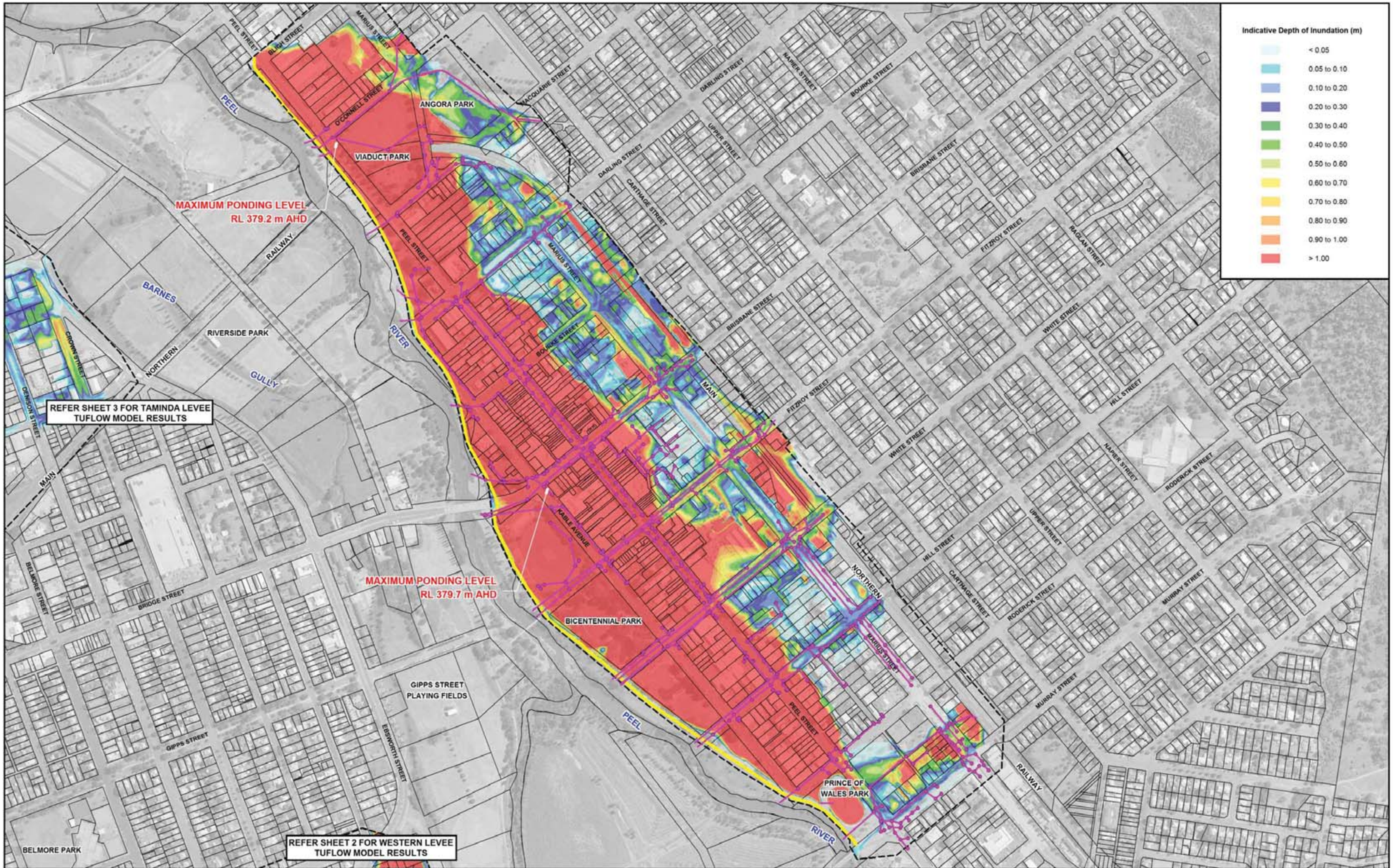
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.12
(Sheet 3 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
100 YEAR ARI

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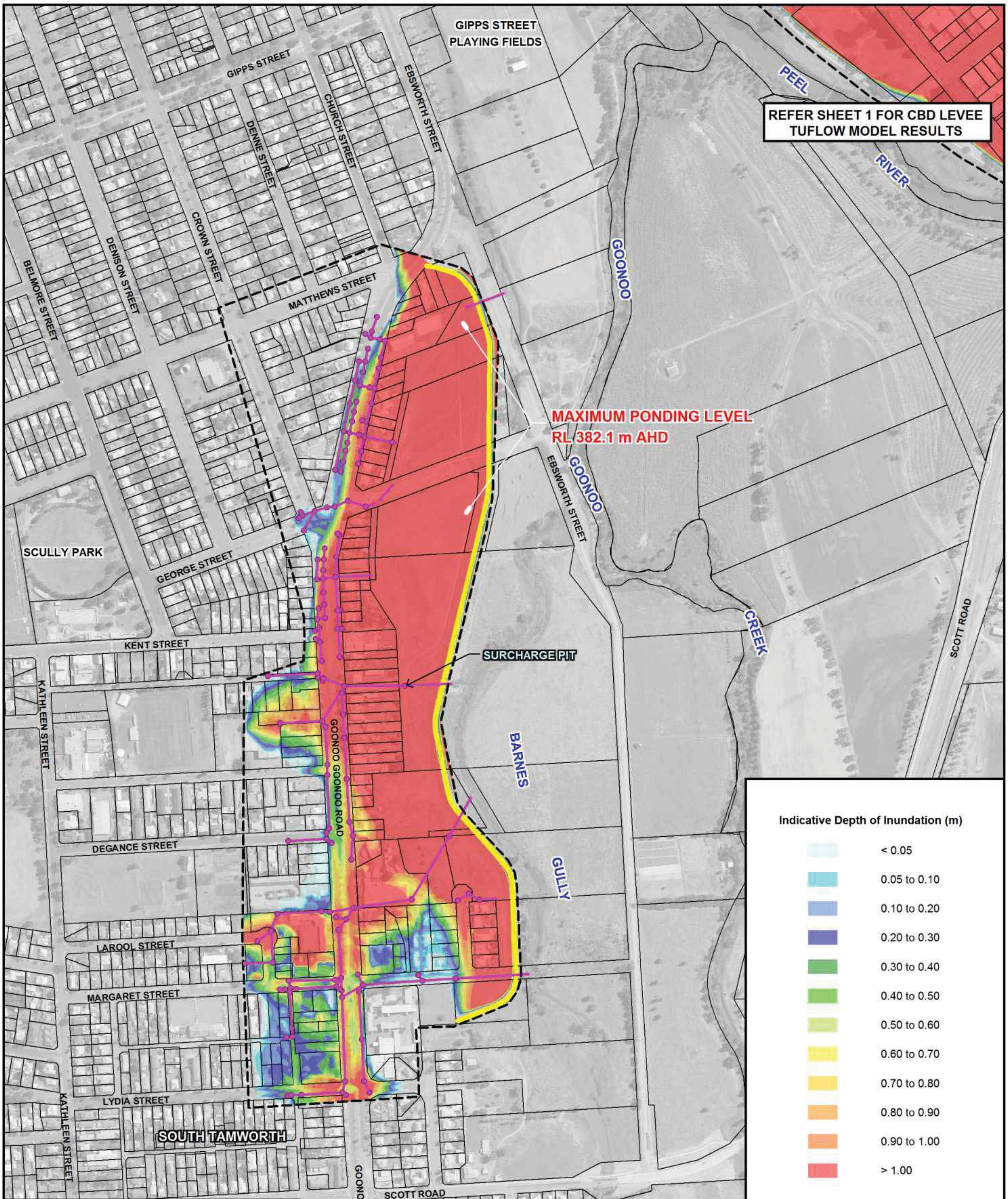
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.13
(Sheet 1 of 3)

TUFLOW MODEL RESULTS - FLOOD GATES FULLY OPEN
PMF

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**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

N

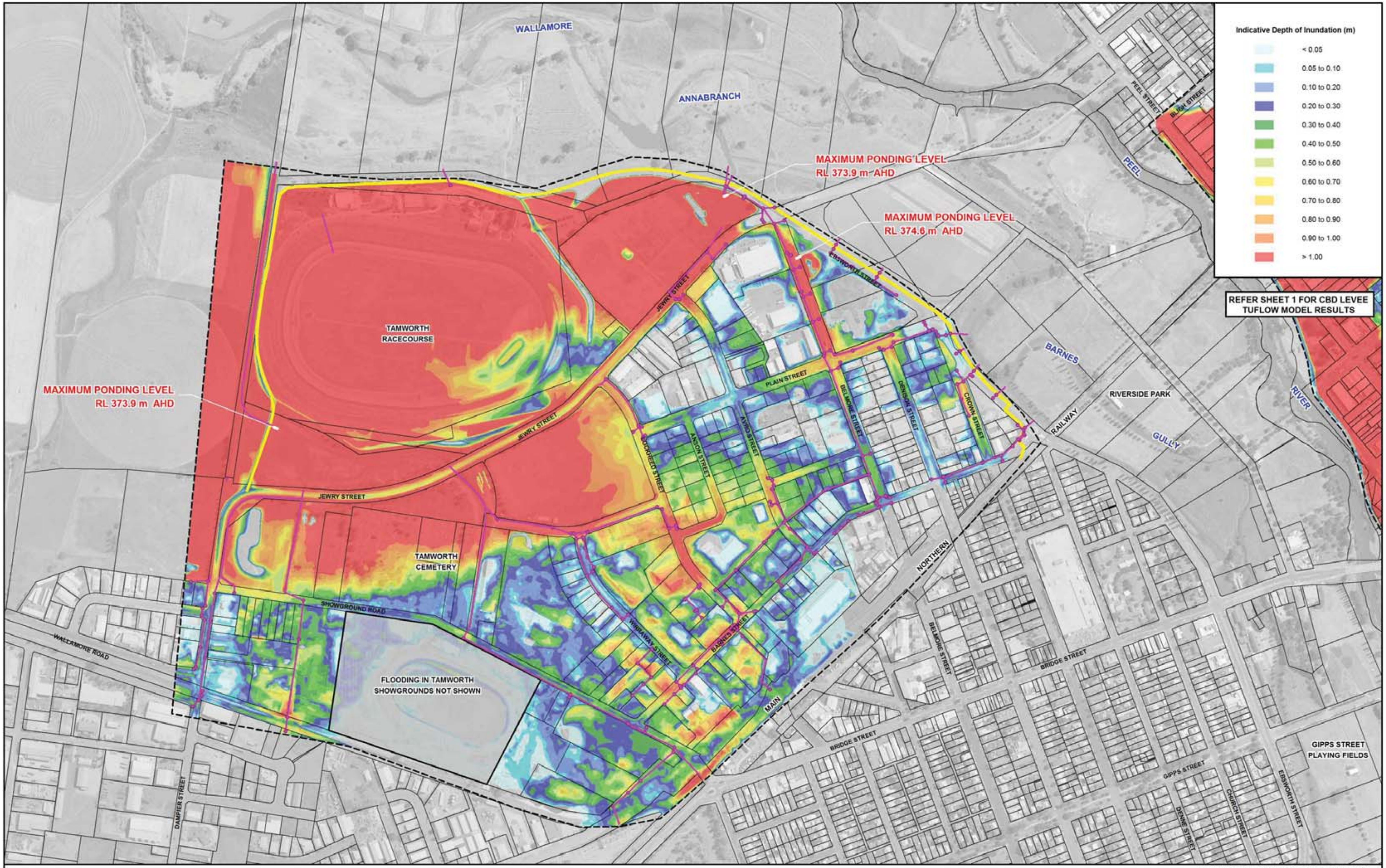
80 0 80 160 240 m

**Scale: 1:8,000**

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.13
(Sheet 2 of 3)

TUFLOW MODEL RESULTS - GATES FULLY OPEN
PMF




80 0 80 160 240 m

Scale: 1:8,000

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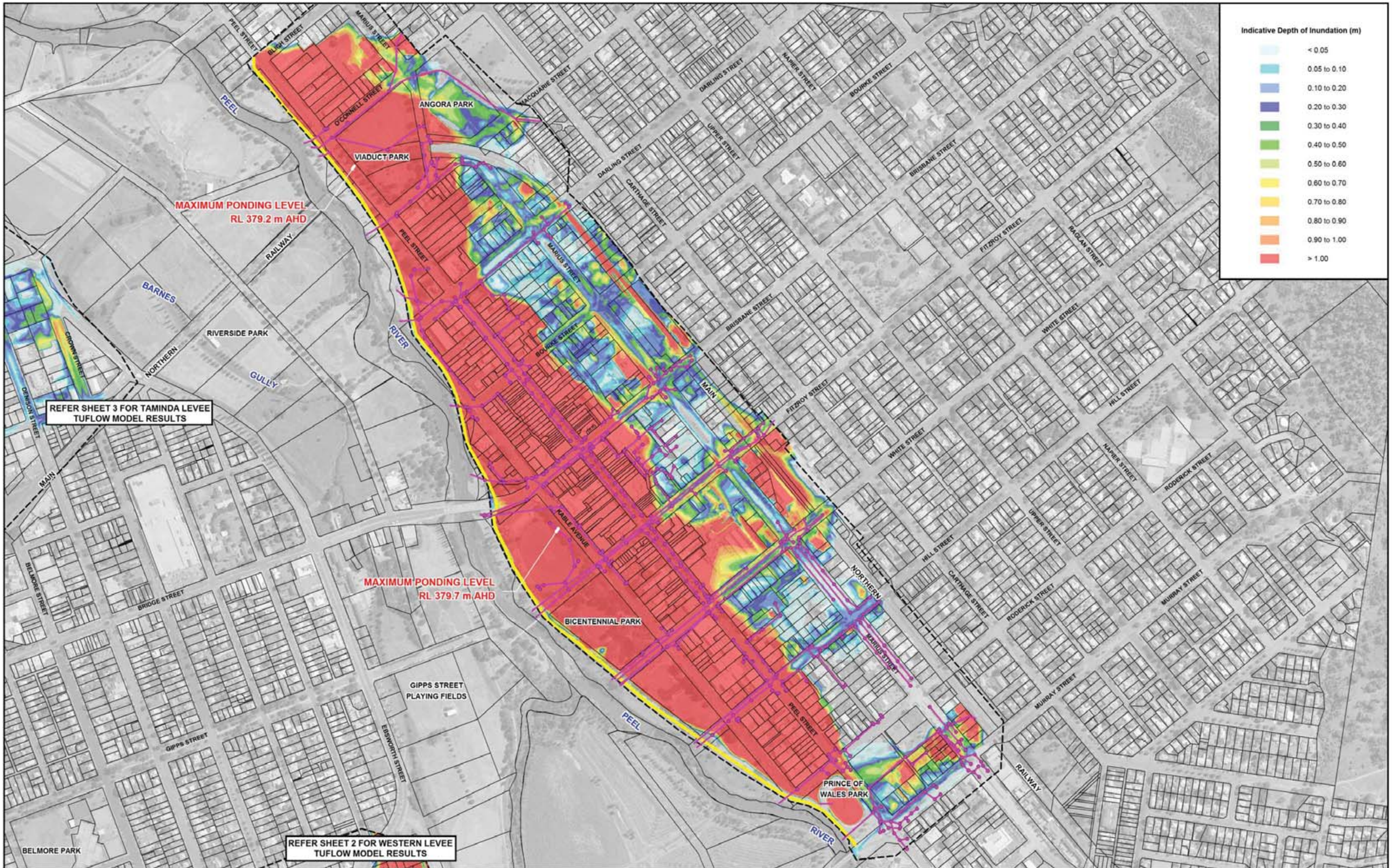
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.13
(Sheet 3 of 3)

TUFLOW MODEL RESULTS - GATES FULLY OPEN
PMF




N

80 0 80 160 240 m

Scale: 1:8,000

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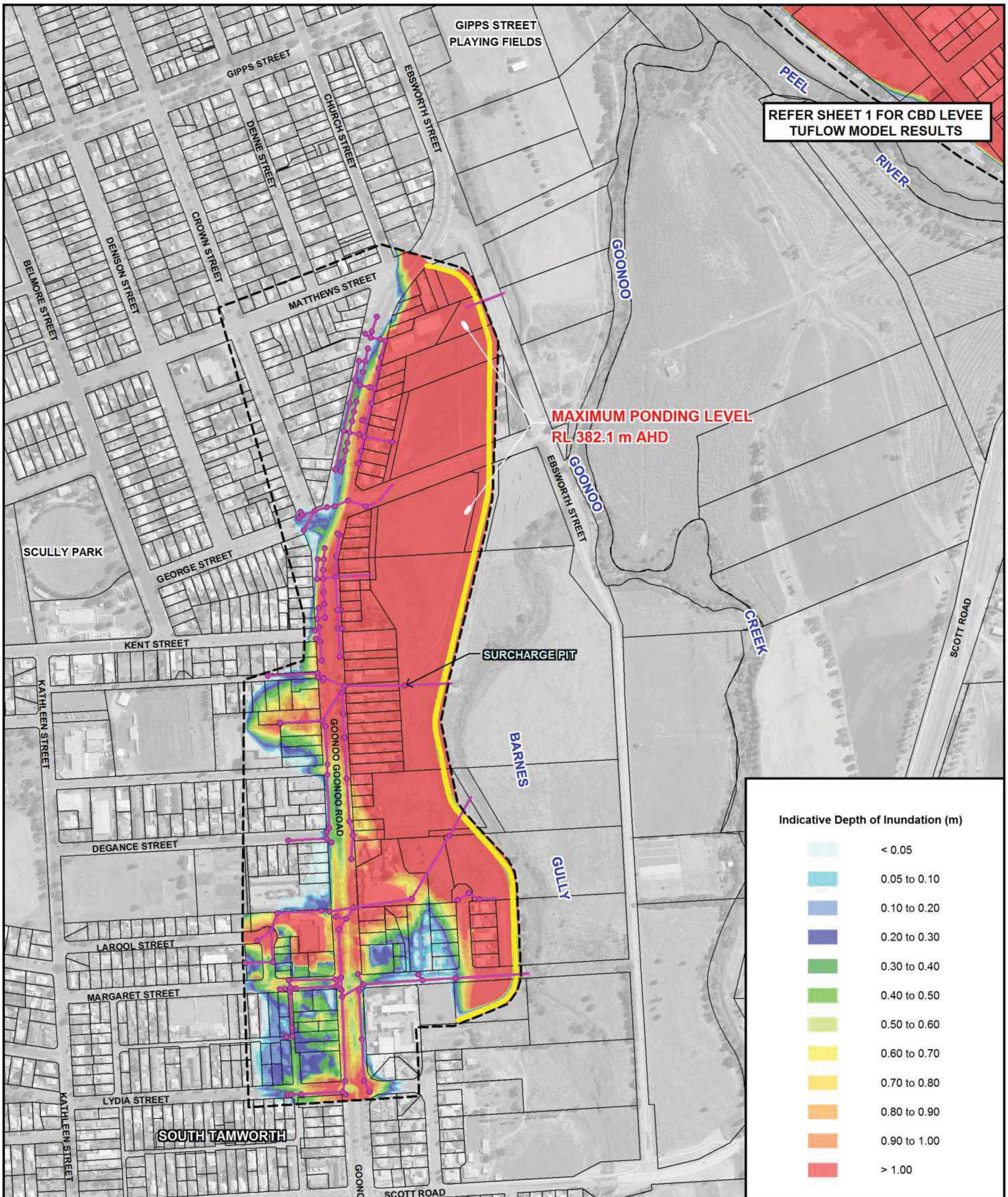
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.14
(Sheet 1 of 3)

TUFLOW MODEL RESULTS - FLOOD GATES FULLY CLOSED
PMF

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**LEGEND**

Levee

Modelled Stormwater Network

Two-Dimensional Model Boundary

N

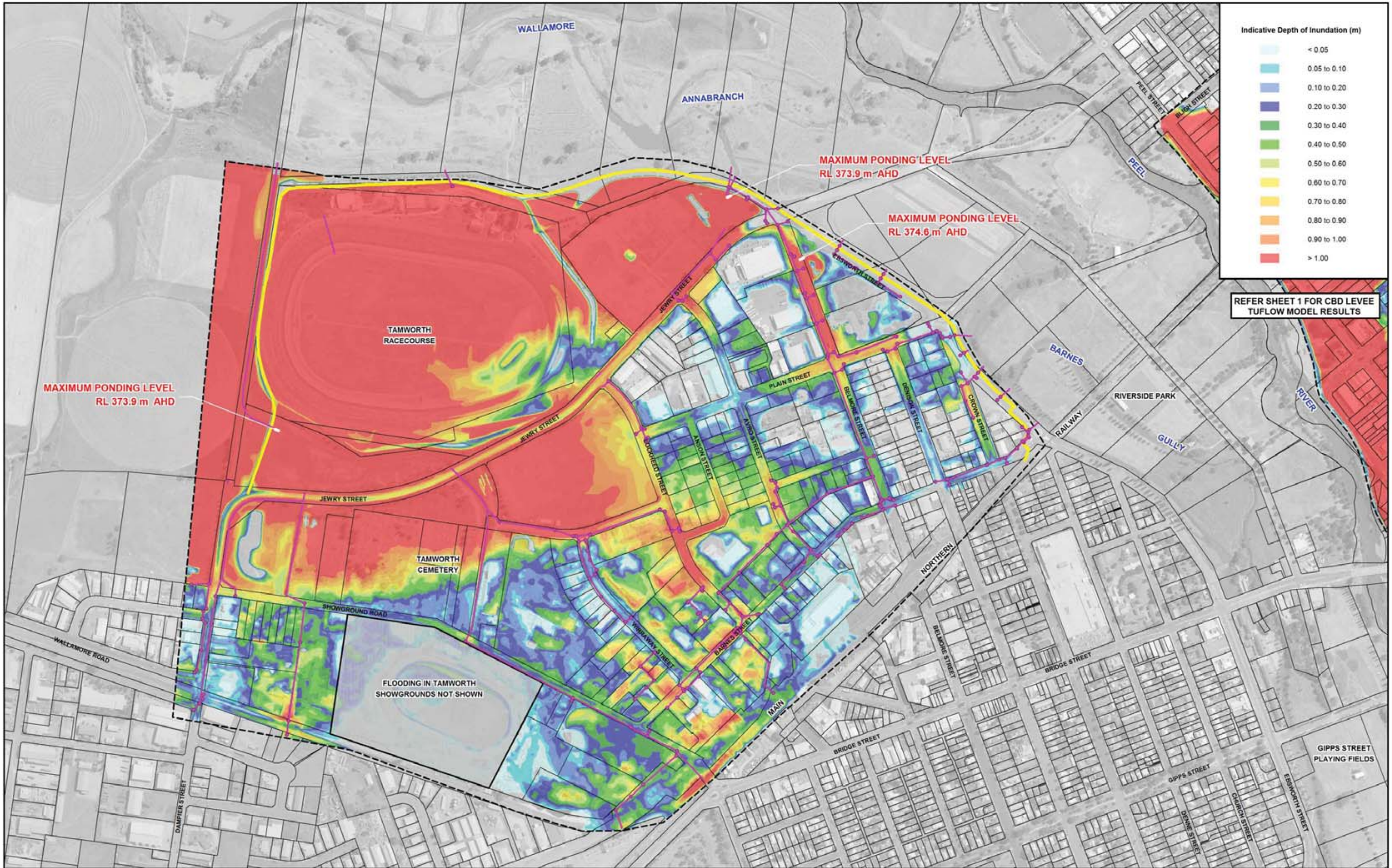
80 0 80 160 240 m

**Scale: 1:8,000**

**TAMWORTH CITY LEVEES INTERNAL DRAINAGE STUDY**

Figure 6.14
(Sheet 2 of 3)

TUFLOW MODEL RESULTS - GATES FULLY CLOSED
PMF




N

80 0 80 160 240 m

Scale: 1:8,000

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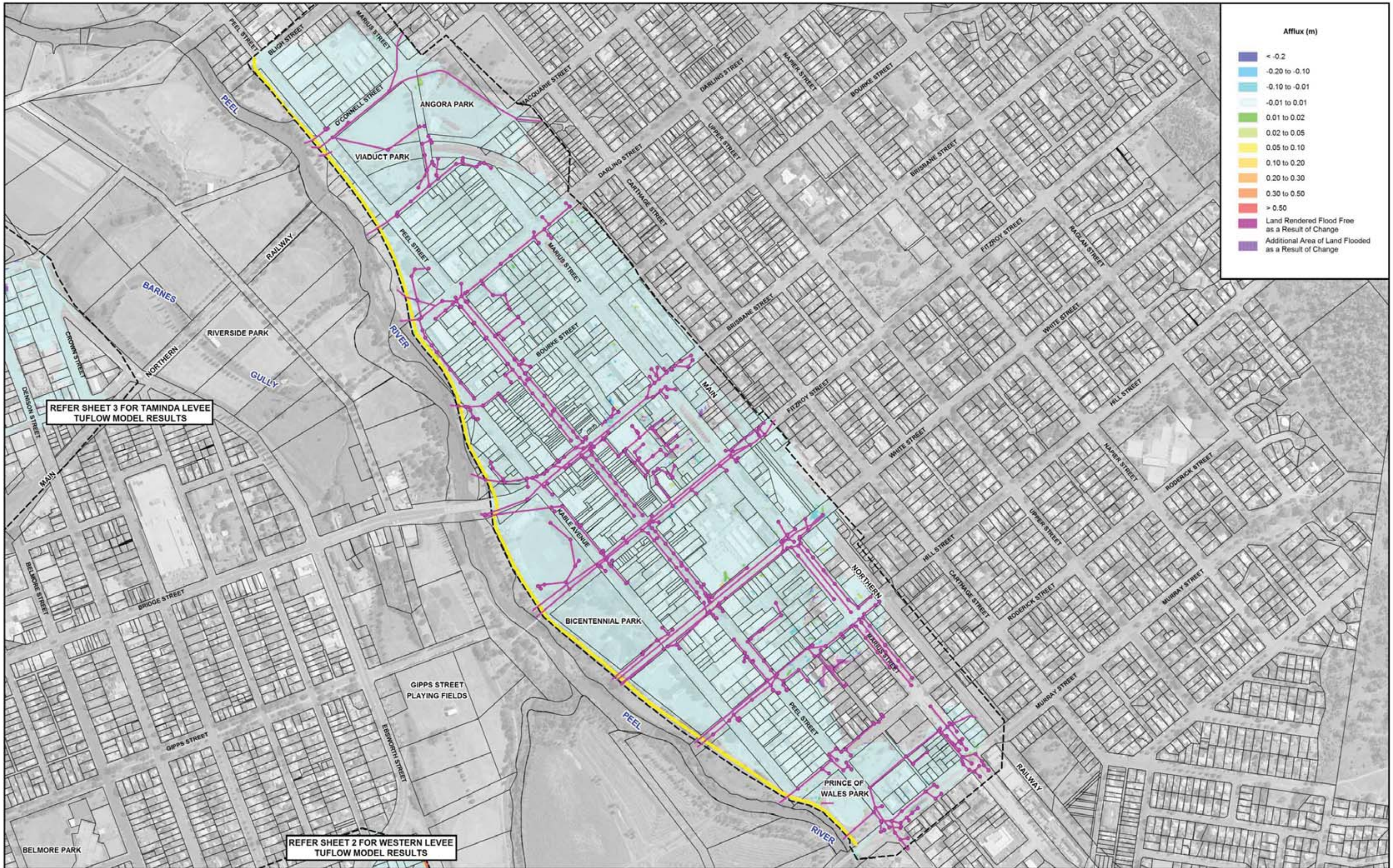
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.14
(Sheet 3 of 3)

TUFLOW MODEL RESULTS - GATES FULLY CLOSED
PMF

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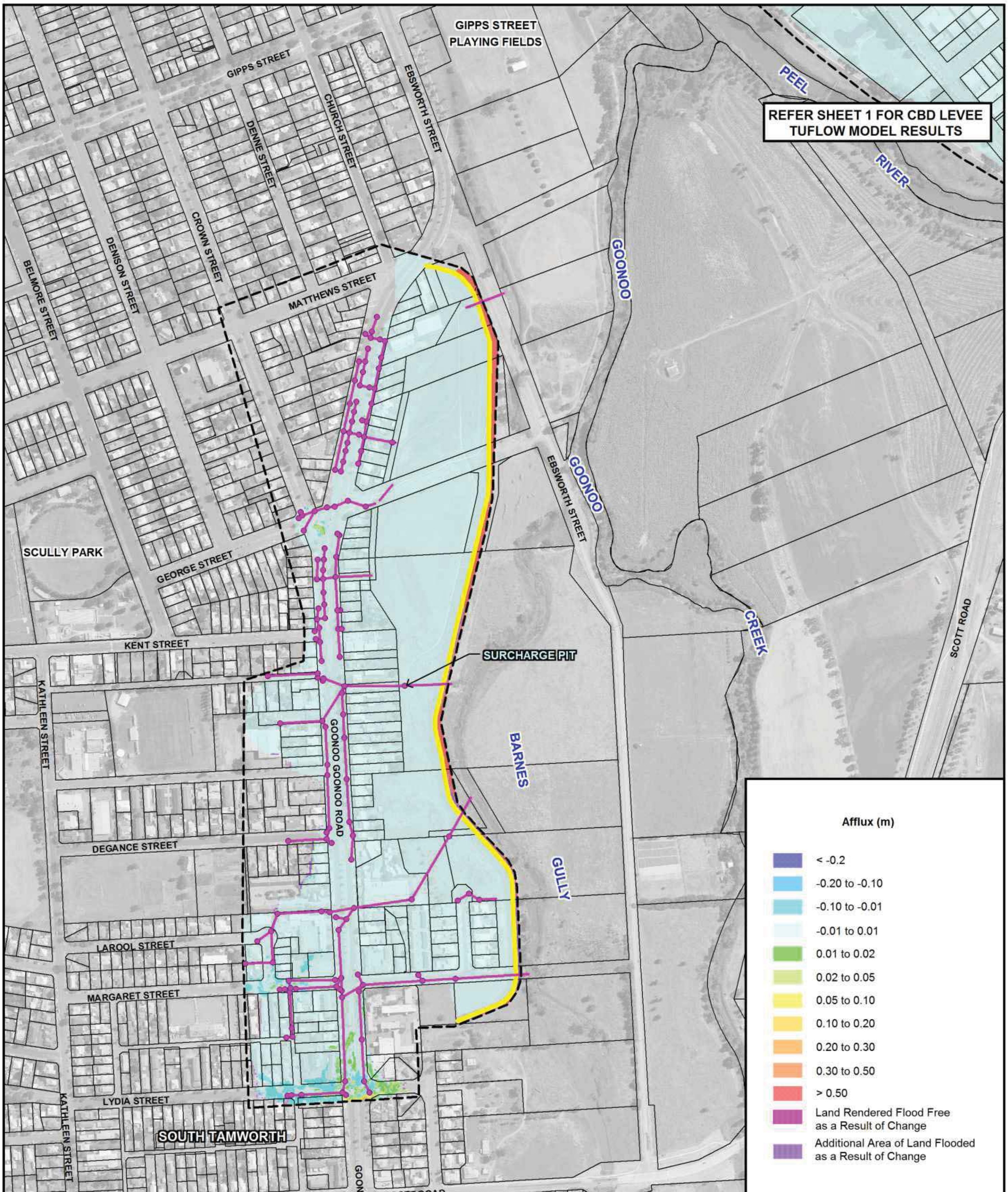
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.15
(Sheet 1 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
PMF




N

80 0 80 160 240 m

Scale: 1:8,000

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.15
(Sheet 2 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
PMF


N

80 0 80 160 240 m

Scale: 1:8,000

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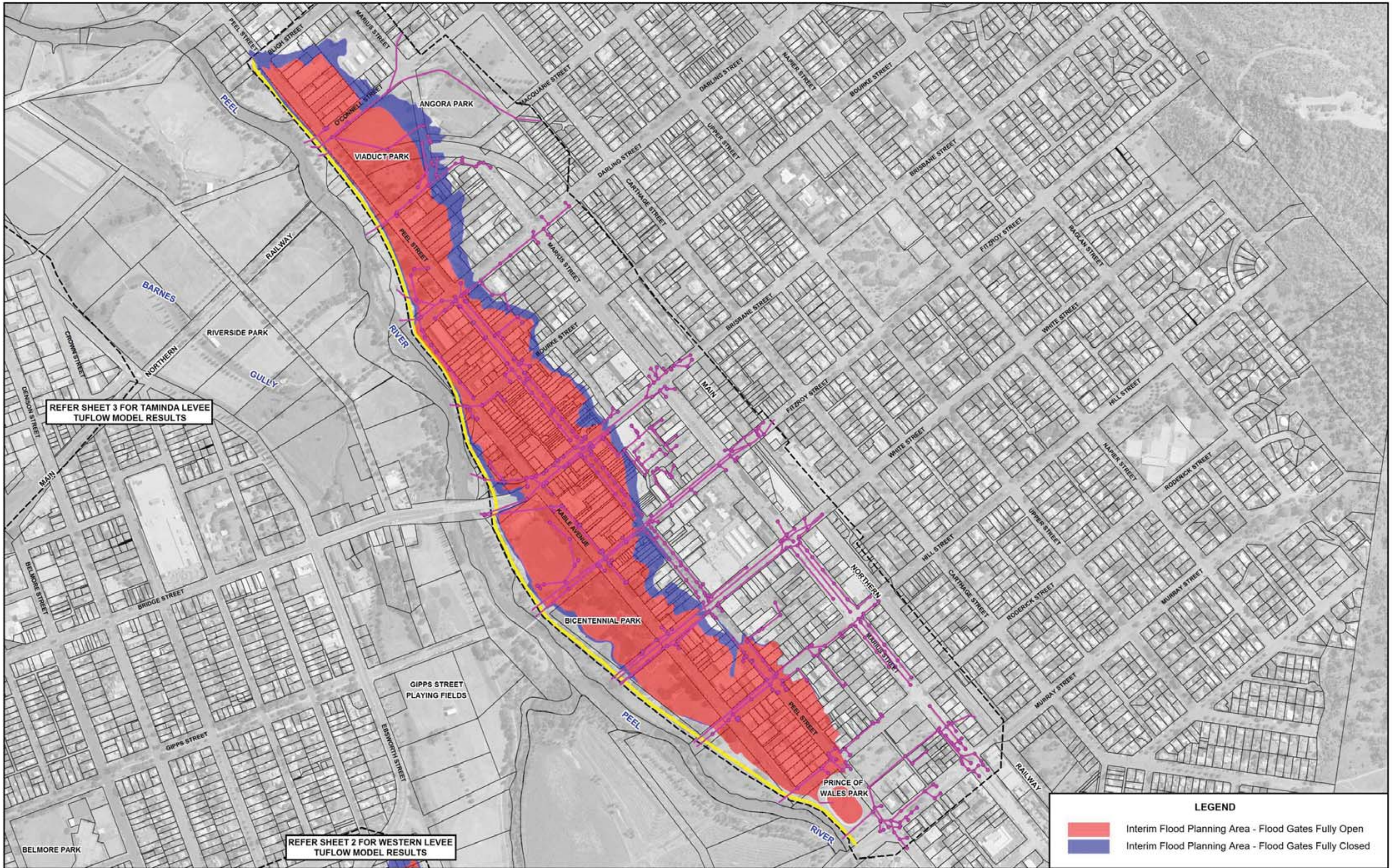
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.15
(Sheet 3 of 3)

AFFLUX (FLOOD GATES FULLY CLOSED MINUS FLOOD GATES FULLY OPEN)
PMF

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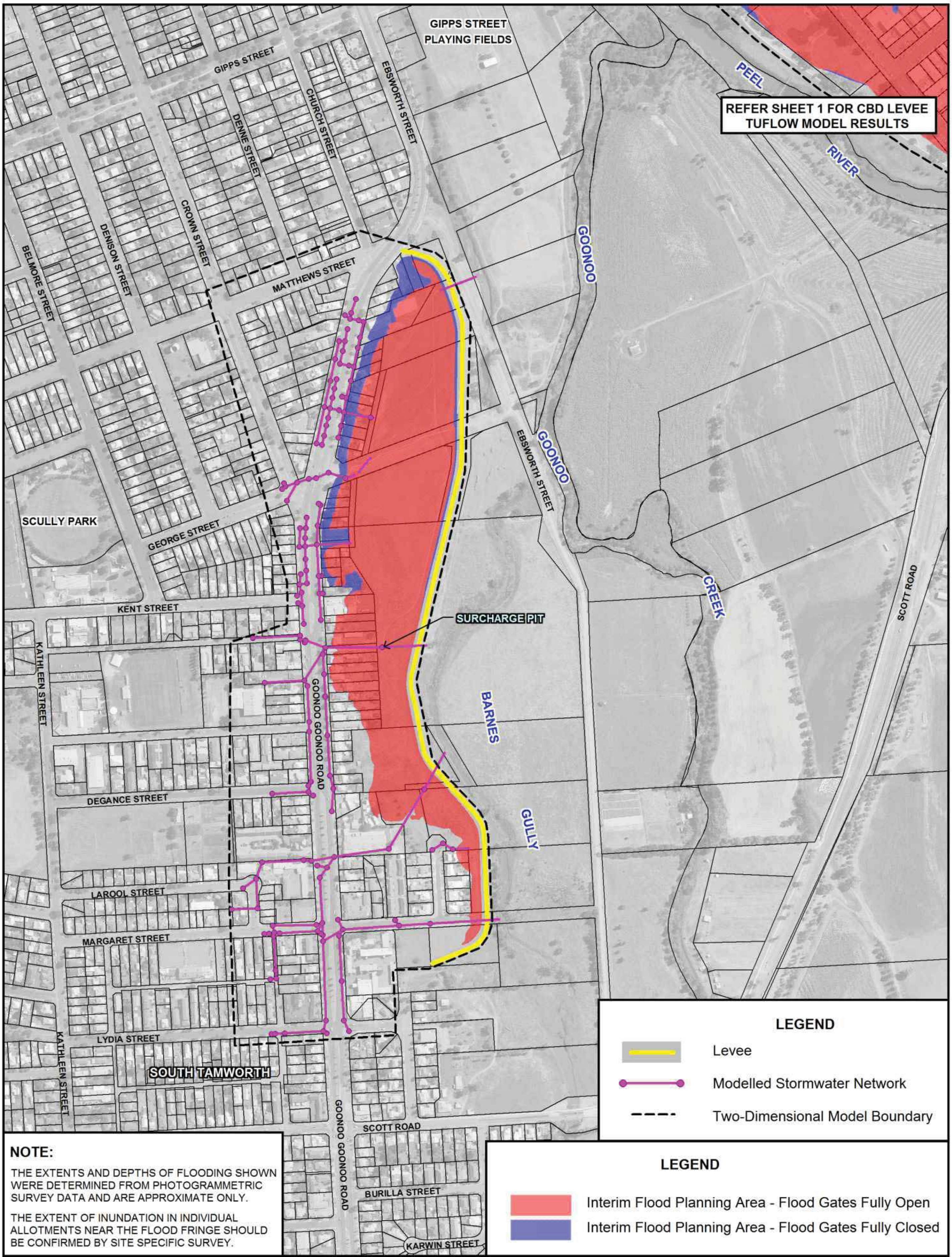
**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.16
(Sheet 1 of 3)

INTERIM FLOOD PLANNING AREAS (PONDING AREAS ONLY)
FLOOD GATES FULLY OPEN VERSUS FLOOD GATES FULLY CLOSED




N

80 0 80 160 240 m

Scale: 1:8,000

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.16
(Sheet 2 of 3)

INTERIM FLOOD PLANNING AREAS (PONDING AREAS ONLY)
FLOOD GATES FULLY OPEN VERSUS FLOOD GATES FULLY CLOSED


**NOTE:**

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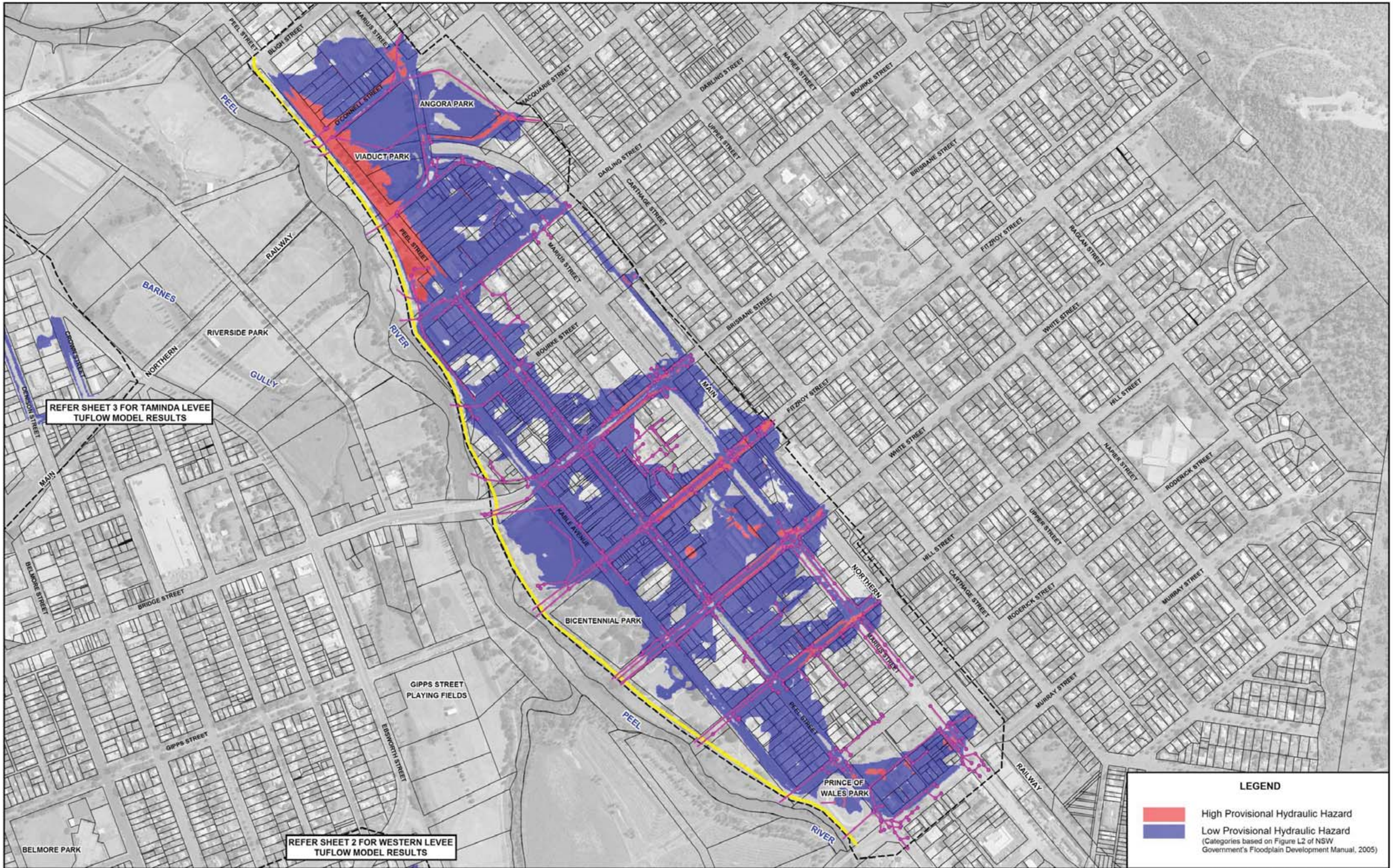
**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.16
(Sheet 3 of 3)

INTERIM FLOOD PLANNING AREAS (PONDING AREAS ONLY)
FLOOD GATES FULLY OPEN VERSUS FLOOD GATES FULLY CLOSED

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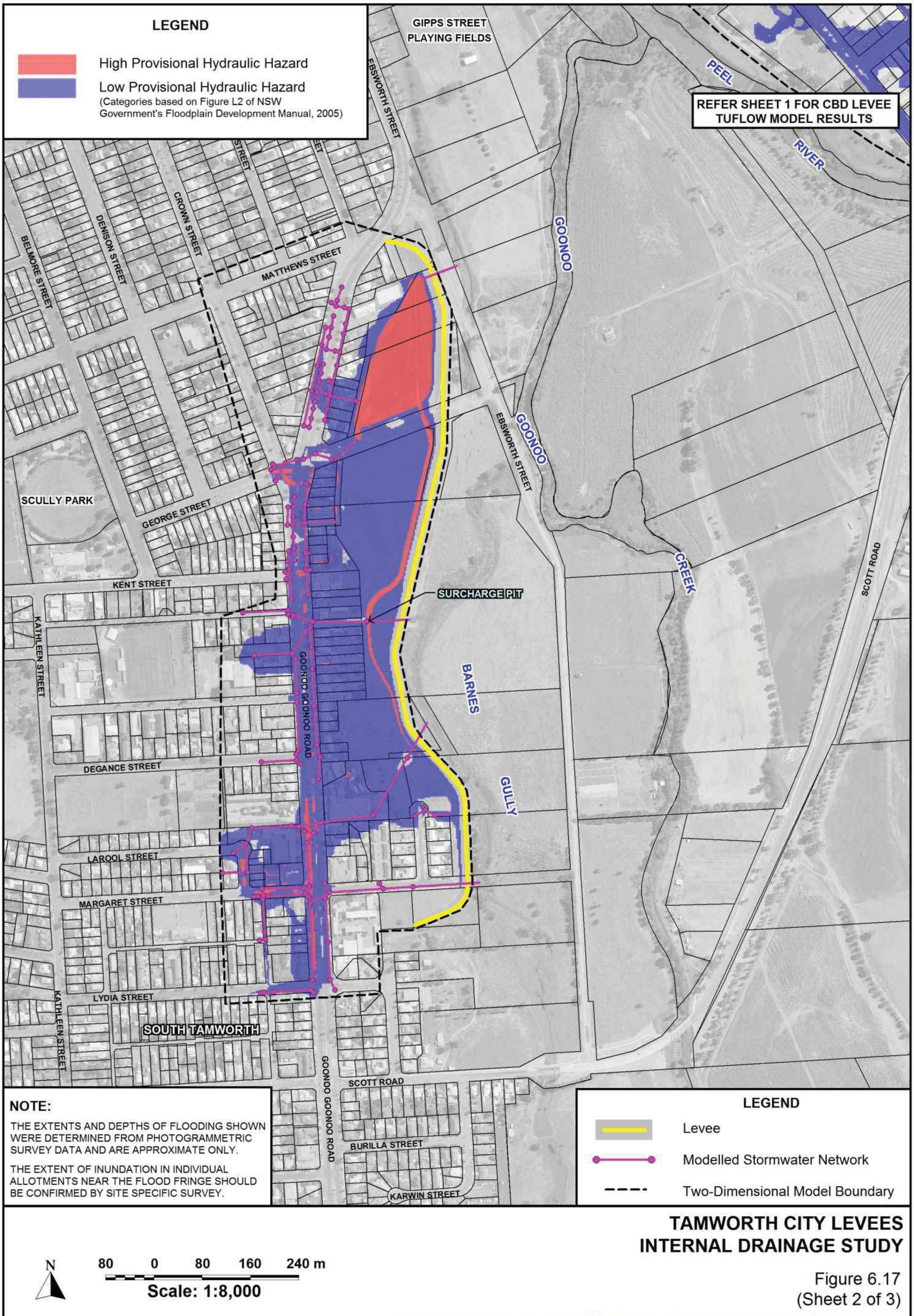
**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.17
(Sheet 1 of 3)

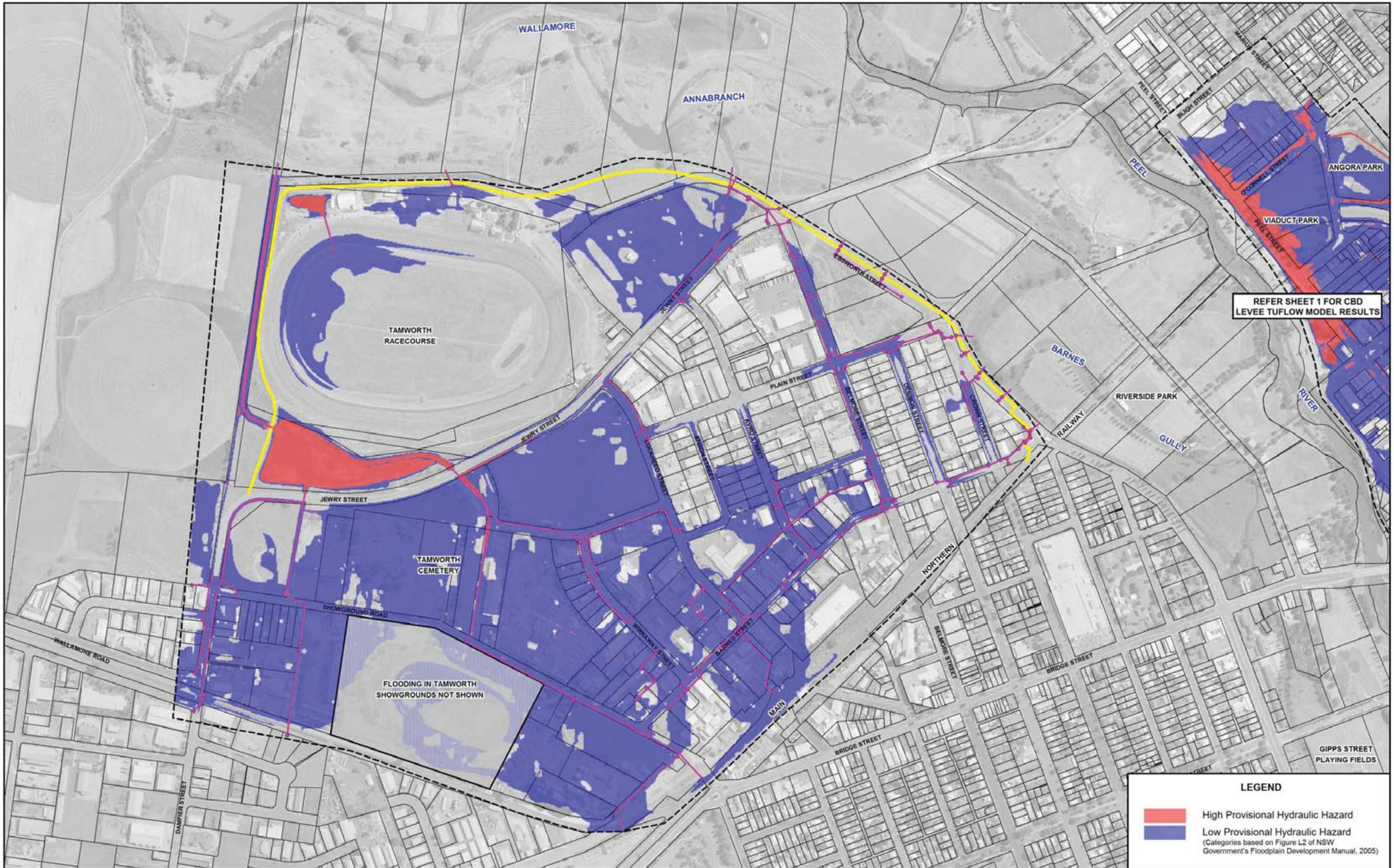
PROVISIONAL FLOOD HAZARD - FLOOD GATES FULLY OPEN
100 YEAR ARI




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.17
(Sheet 2 of 3)

PROVISIONAL FLOOD HAZARD - FLOOD GATES FULLY OPEN
100 YEAR ARI




N

80 0 80 160 240 m

Scale: 1:8,000

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**LEGEND**

Levee

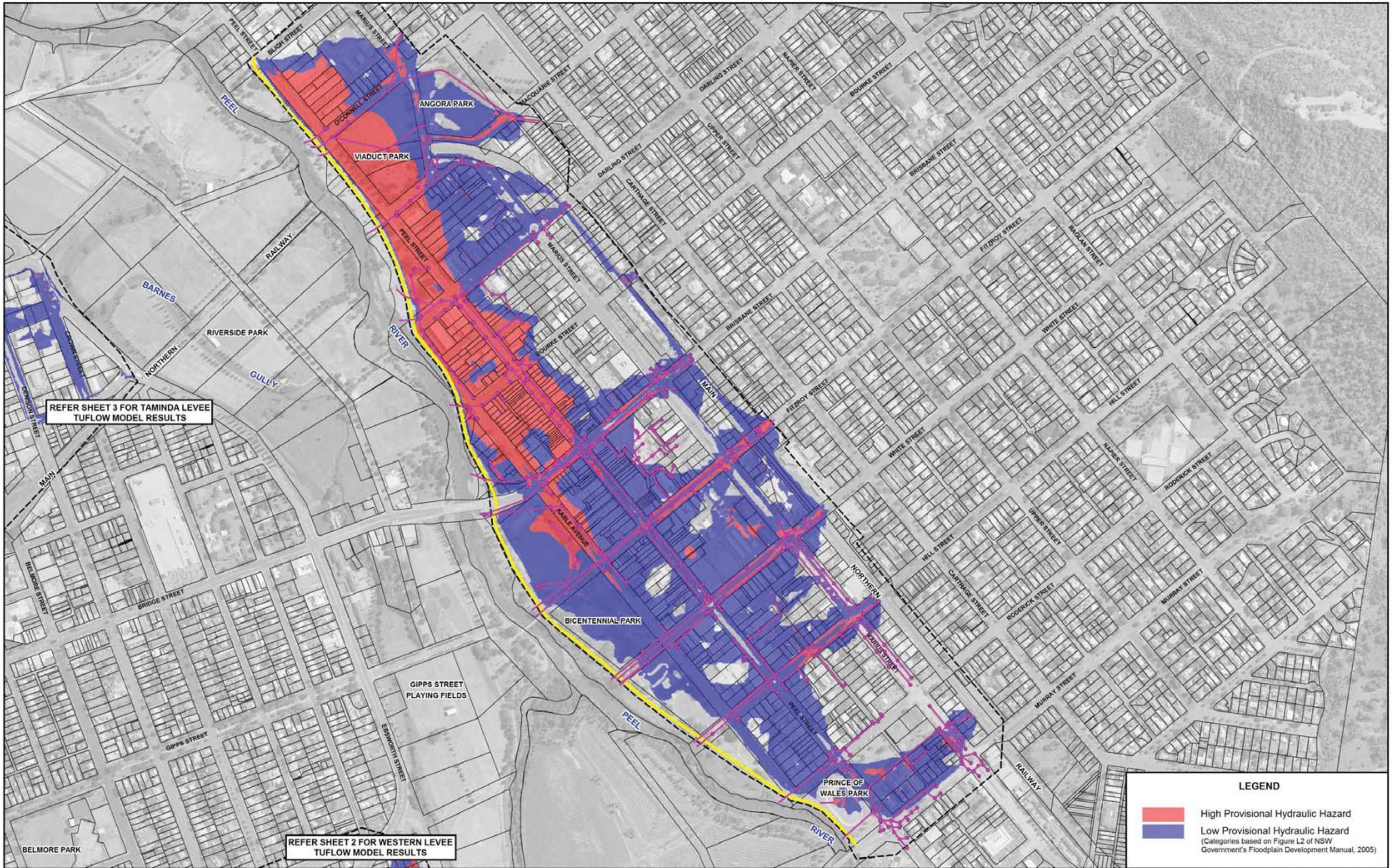
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.17
(Sheet 3 of 3)

PROVISIONAL FLOOD HAZARD - FLOOD GATES FULLY OPEN
100 YEAR ARI

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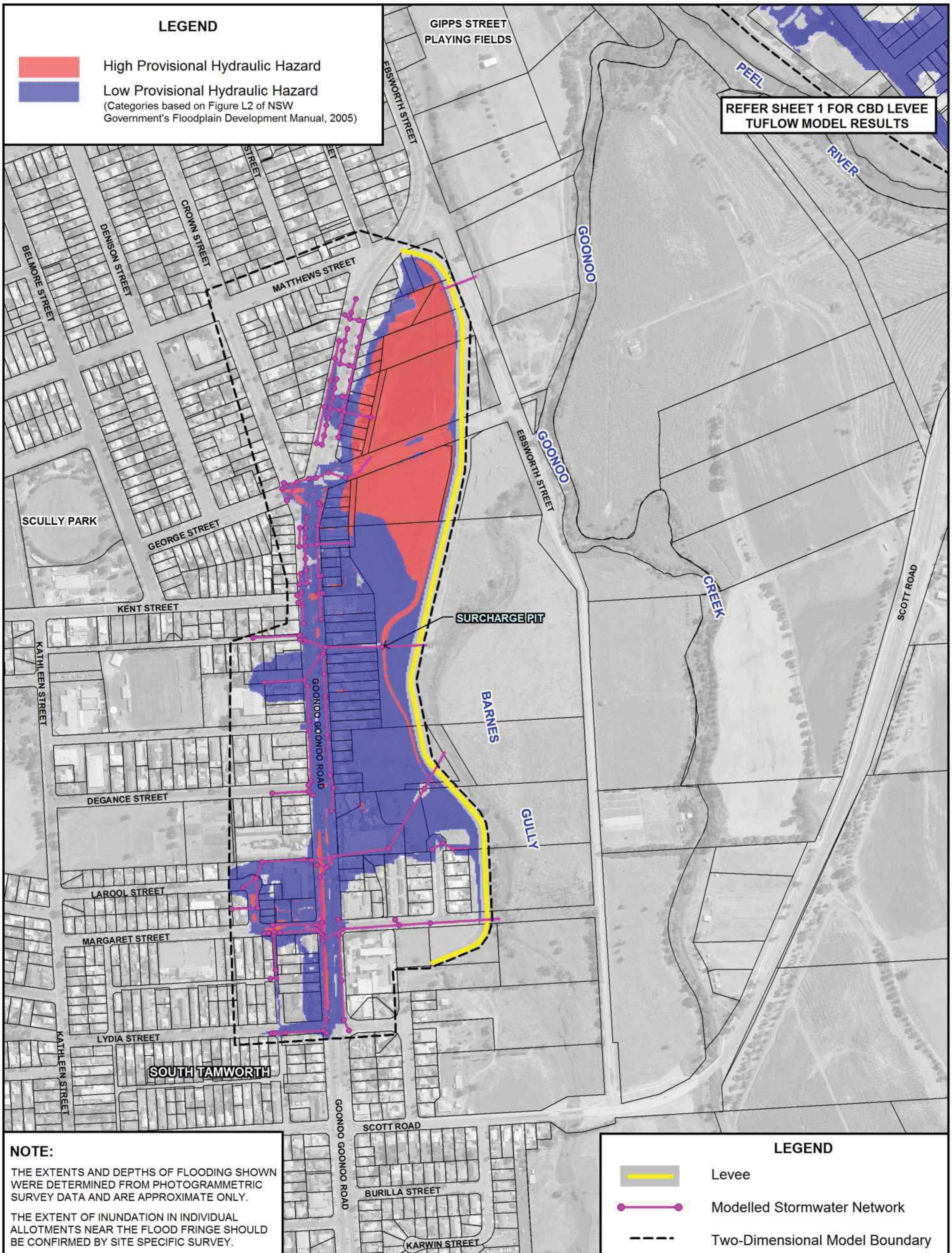
**TAMWORTH CITY LEVEES INTERNAL DRAINAGE STUDY**

Figure 6.18
(Sheet 1 of 3)

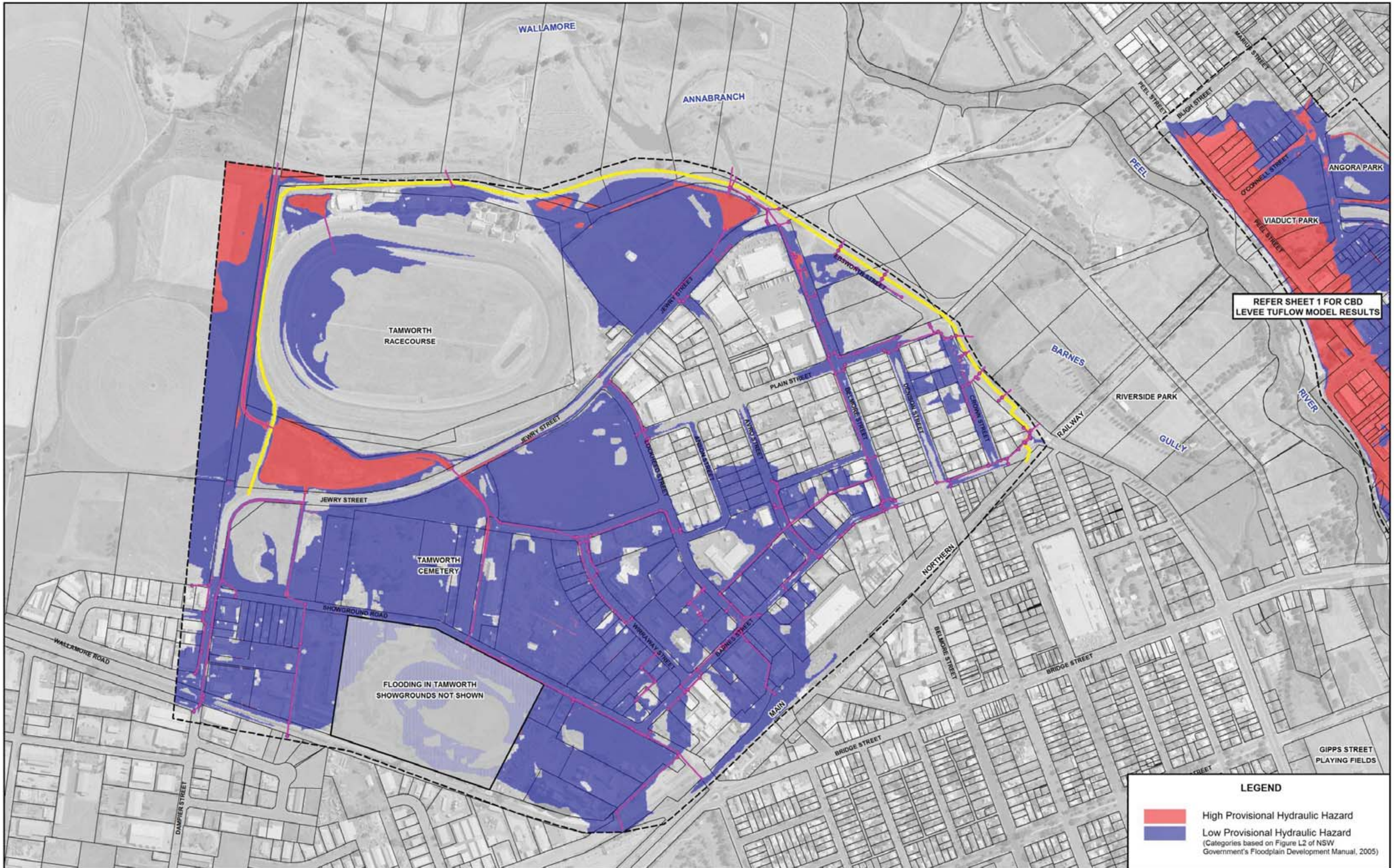
PROVISIONAL FLOOD HAZARD - FLOOD GATES FULLY CLOSED
100 YEAR ARI




N
80 0 80 160 240 m
**Scale: 1:8,000**

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.18
(Sheet 2 of 3)
PROVISIONAL FLOOD HAZARD - FLOOD GATES FULLY CLOSED
100 YEAR ARI




N

80 0 80 160 240 m

Scale: 1:8,000

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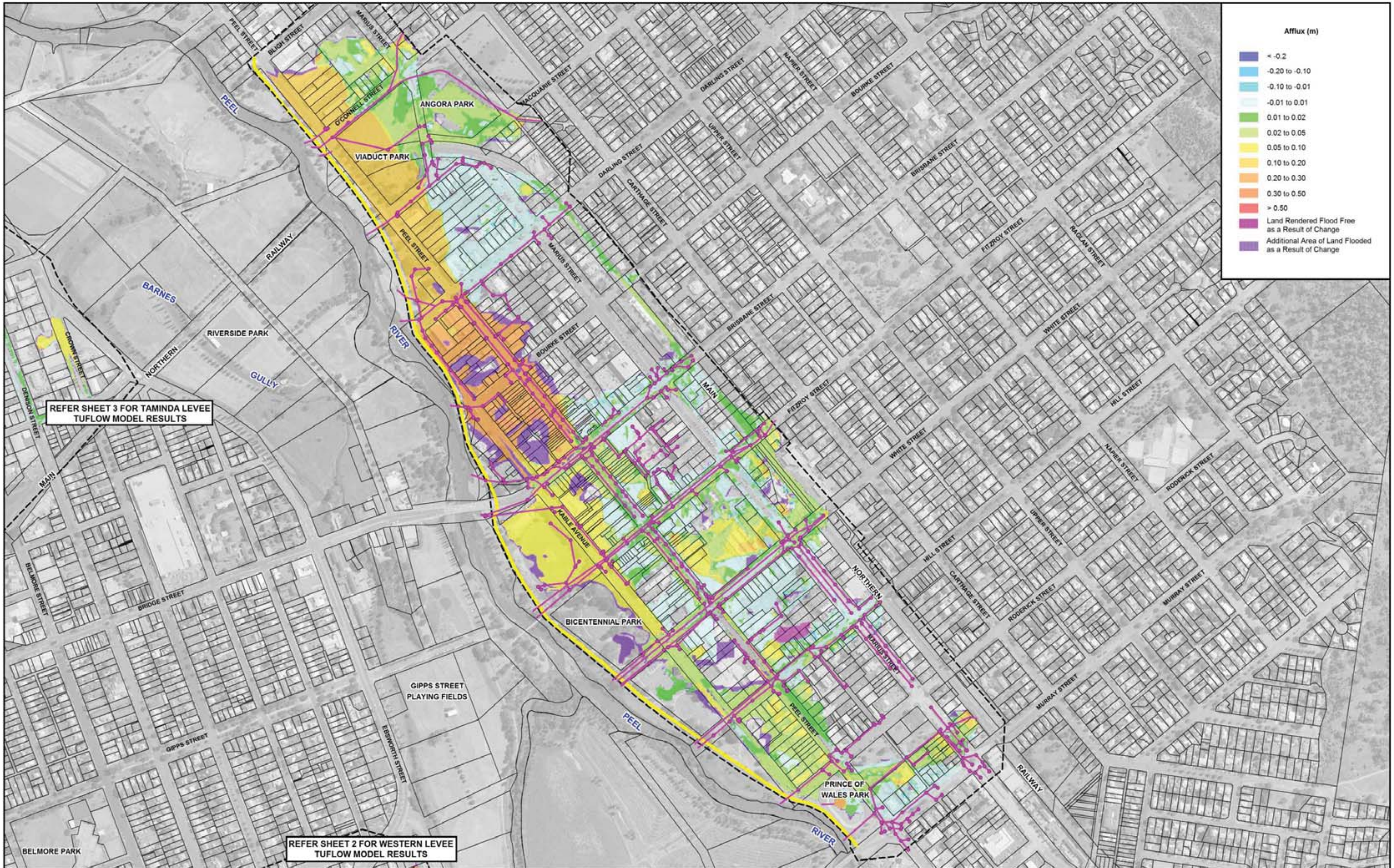
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.18
(Sheet 3 of 3)

PROVISIONAL FLOOD HAZARD - FLOOD GATES FULLY CLOSED
100 YEAR ARI




N

80 0 80 160 240 m

Scale: 1:8,000

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**LEGEND**

Levee

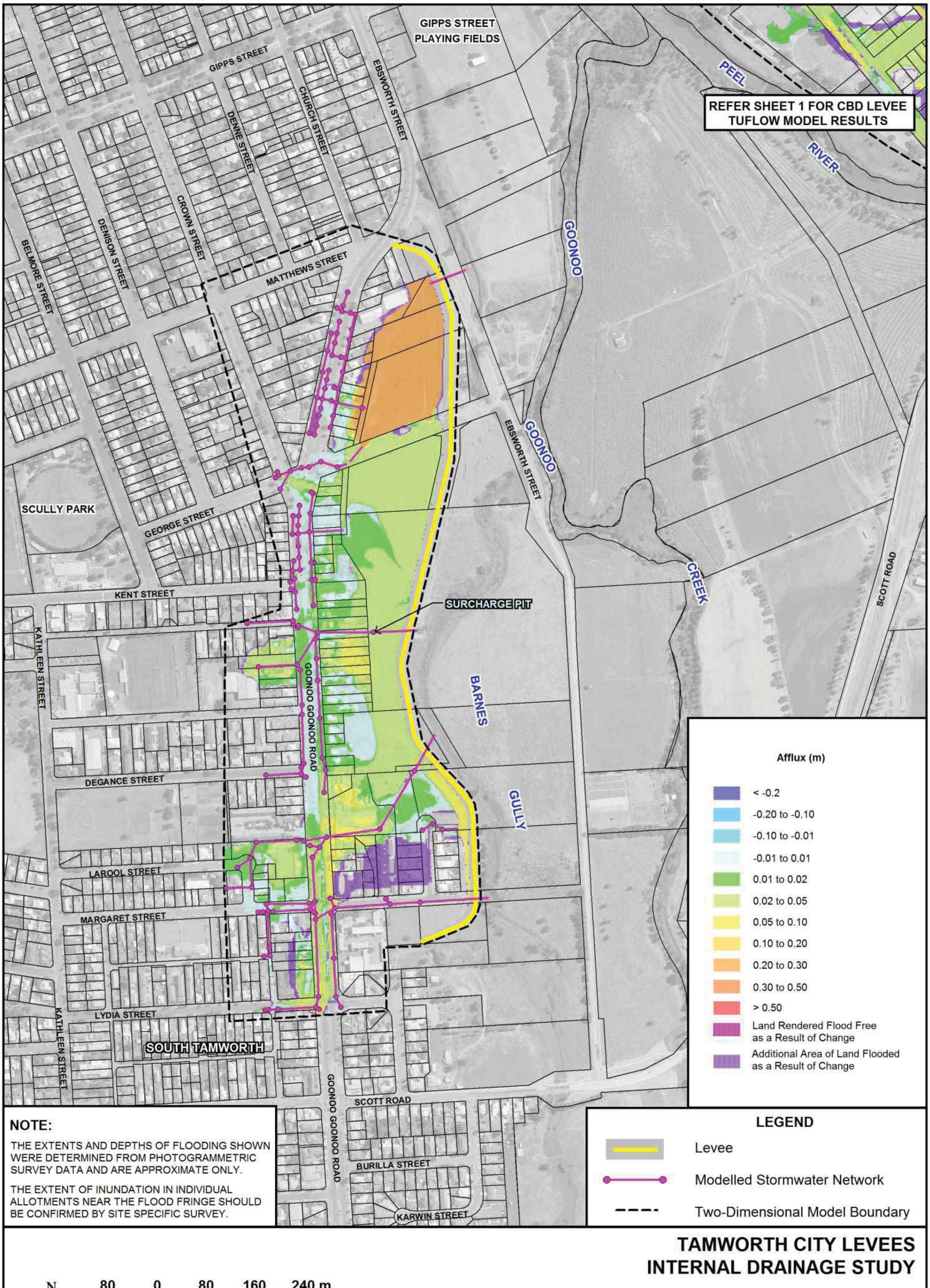
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.19
(Sheet 1 of 3)

POTENTIAL IMPACT OF CLIMATE CHANGE - 100 YEAR ARI RAINFALL INCREASED BY 10%
FLOOD GATES FULLY OPEN

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**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.19
(Sheet 2 of 3)

POTENTIAL IMPACT OF CLIMATE CHANGE - 100 YEAR ARI RAINFALL INCREASED BY 10%
FLOOD GATES FULLY OPEN


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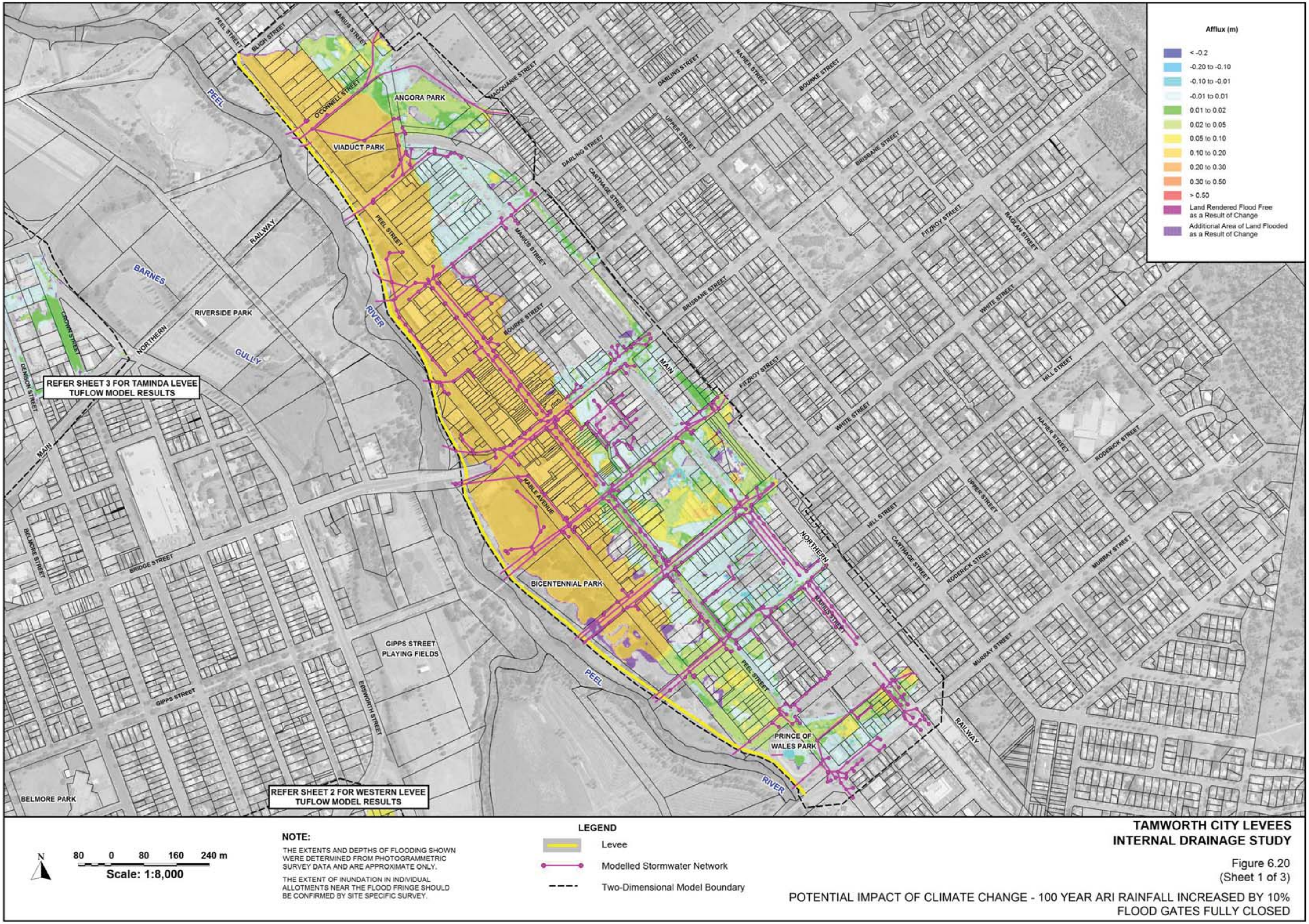
**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.19
(Sheet 3 of 3)

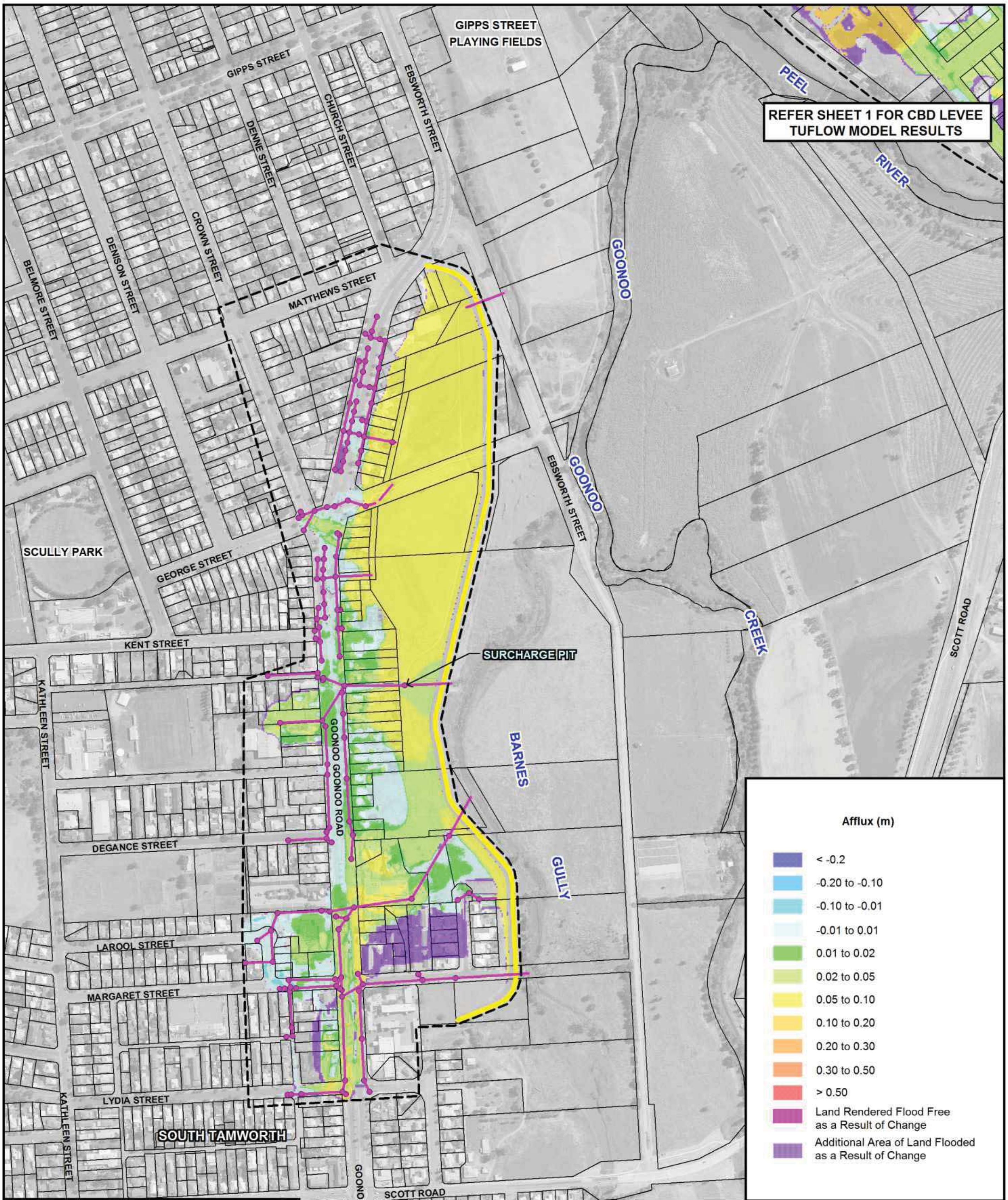
POTENTIAL IMPACT OF CLIMATE CHANGE - 100 YEAR ARI RAINFALL INCREASED BY 10%
FLOOD GATES FULLY OPEN




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.20
(Sheet 1 of 3)

POTENTIAL IMPACT OF CLIMATE CHANGE - 100 YEAR ARI RAINFALL INCREASED BY 10%
FLOOD GATES FULLY CLOSED

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**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

N

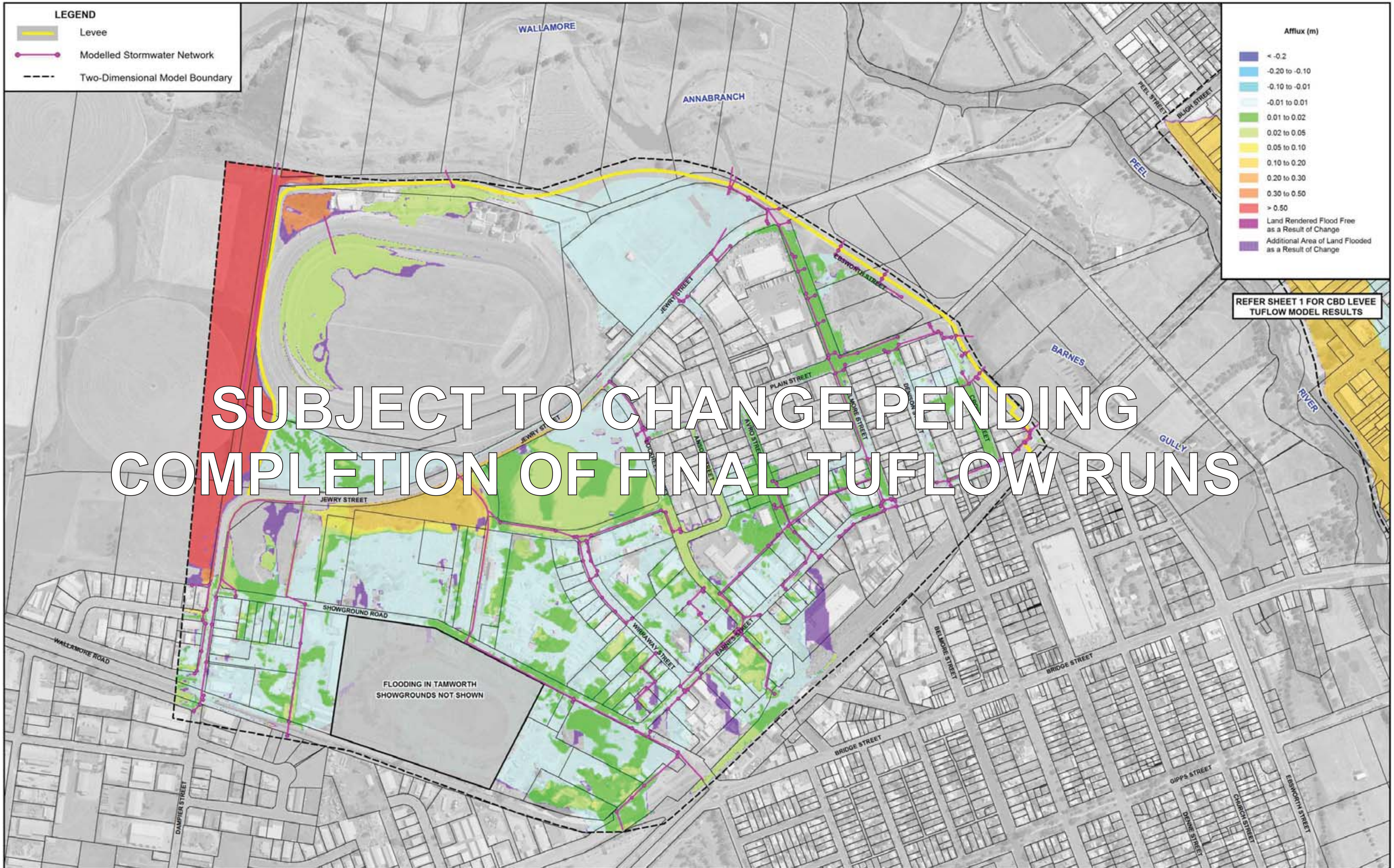
80 0 80 160 240 m

**Scale: 1:8,000**

**TAMWORTH CITY LEVEES INTERNAL DRAINAGE STUDY**

Figure 6.20
(Sheet 2 of 3)

POTENTIAL IMPACT OF CLIMATE CHANGE - 100 YEAR ARI RAINFALL INCREASED BY 10%
FLOOD GATES FULLY CLOSED

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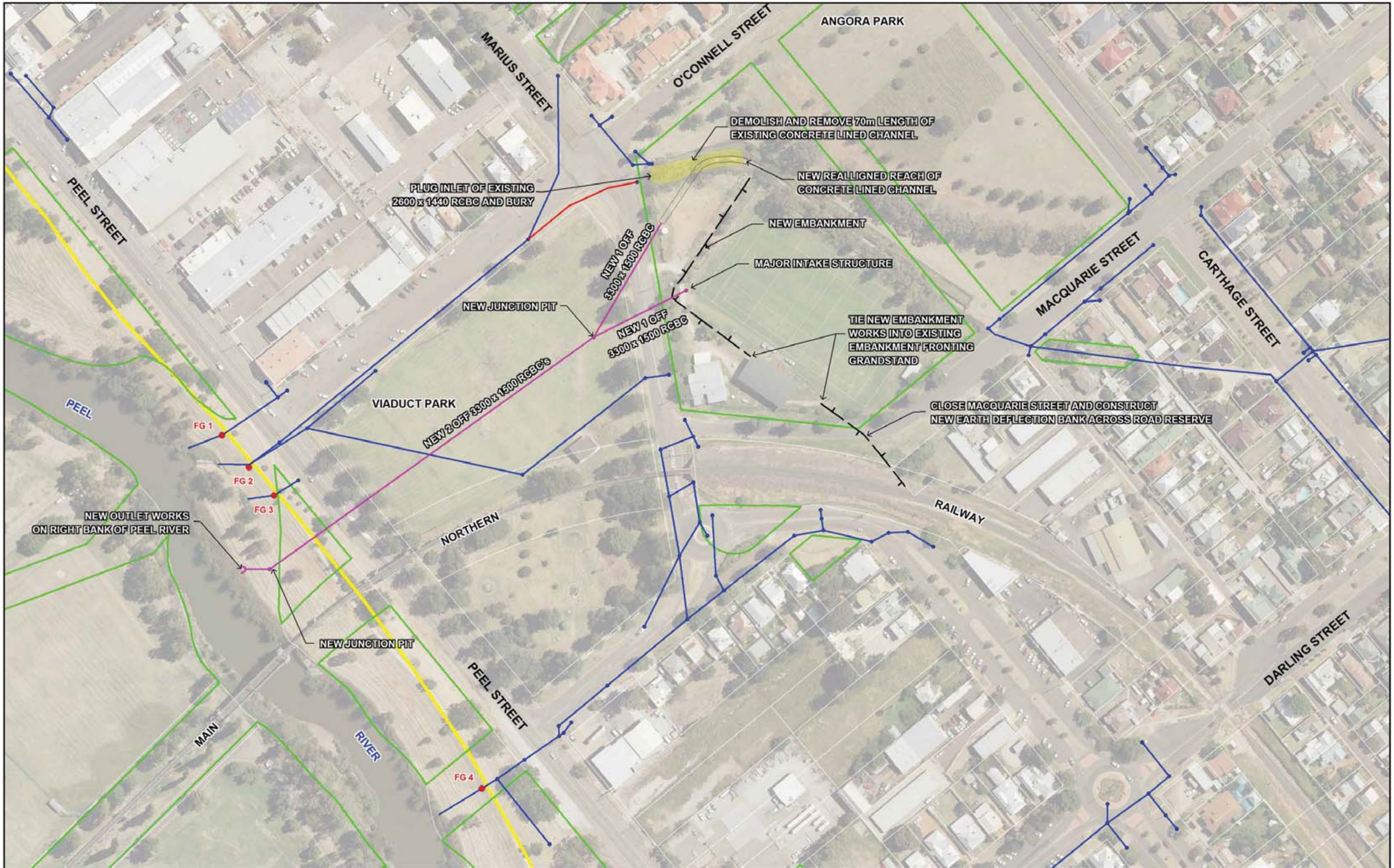
**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 6.20
(Sheet 3 of 3)

POTENTIAL IMPACT OF CLIMATE CHANGE - 100 YEAR ARI RAINFALL INCREASED BY 10%
FLOOD GATES FULLY CLOSED




LEGEND

Levee

FG 3 Flood Gate and Identifier

Existing Stormwater Network

Proposed Stormwater Network

Boundary of Council Owned Land

N

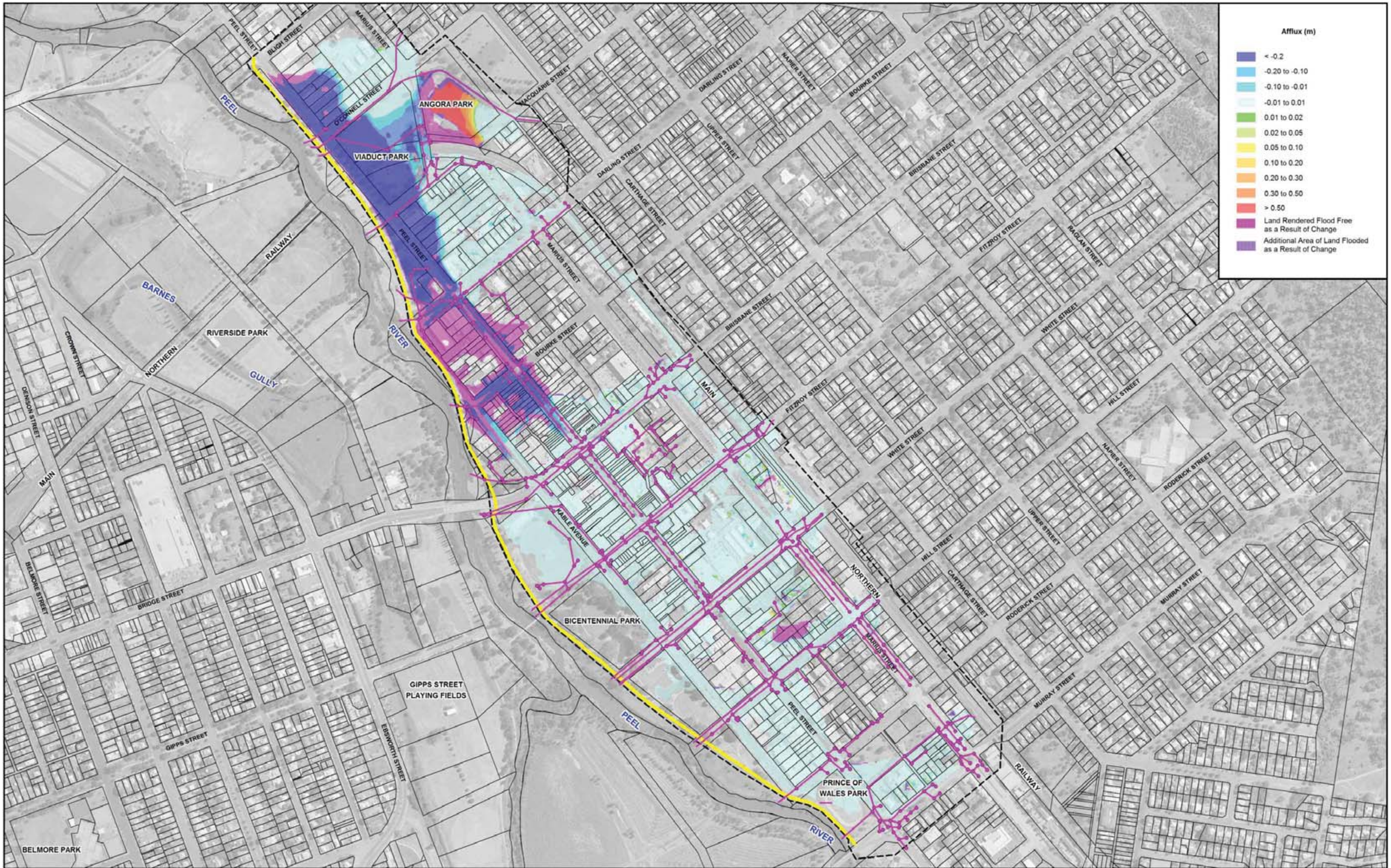
20 0 20 40 60 m

Scale: 1:2,000

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.1

O'CONNELL STREET PRESSURE LINE UPGRADE (OPTION 1)
CBD LEVEE




N

80 0 80 160 240 m

Scale: 1:8,000

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**LEGEND**

Levee

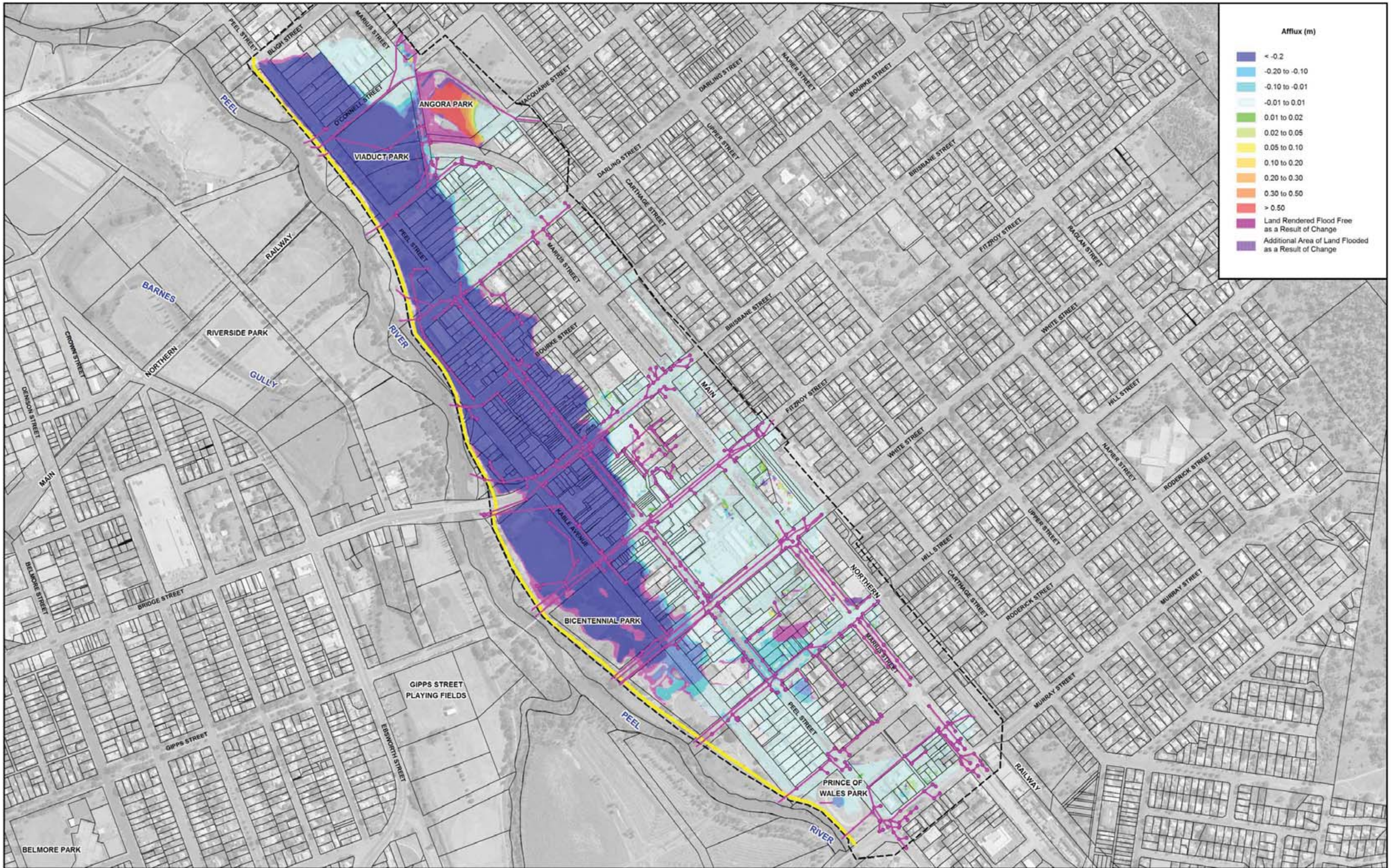
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.2

IMPACT OF OPTION 1 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY OPEN




N

80 0 80 160 240 m
Scale: 1:8,000

**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

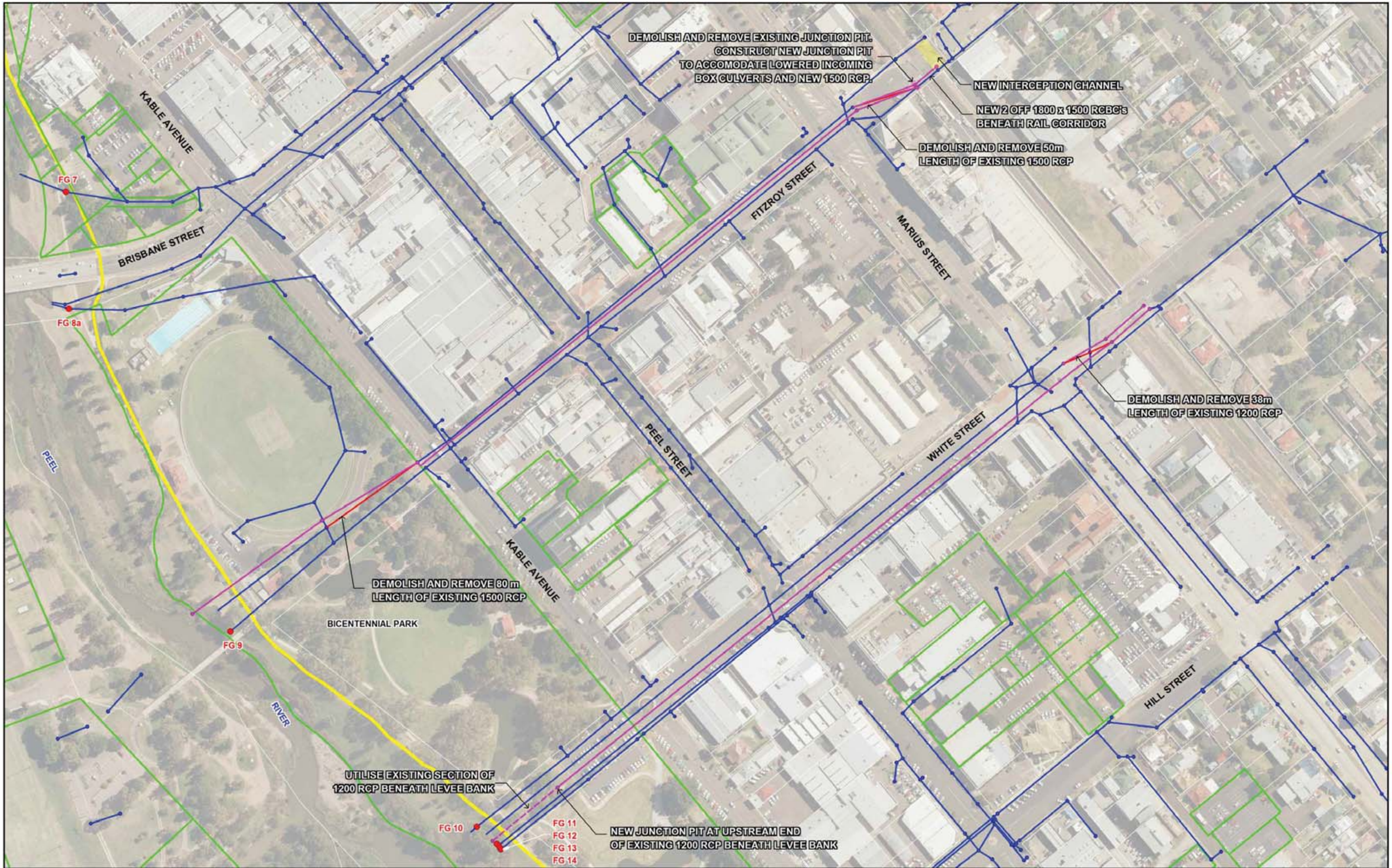
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.3

IMPACT OF OPTION 1 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY CLOSED



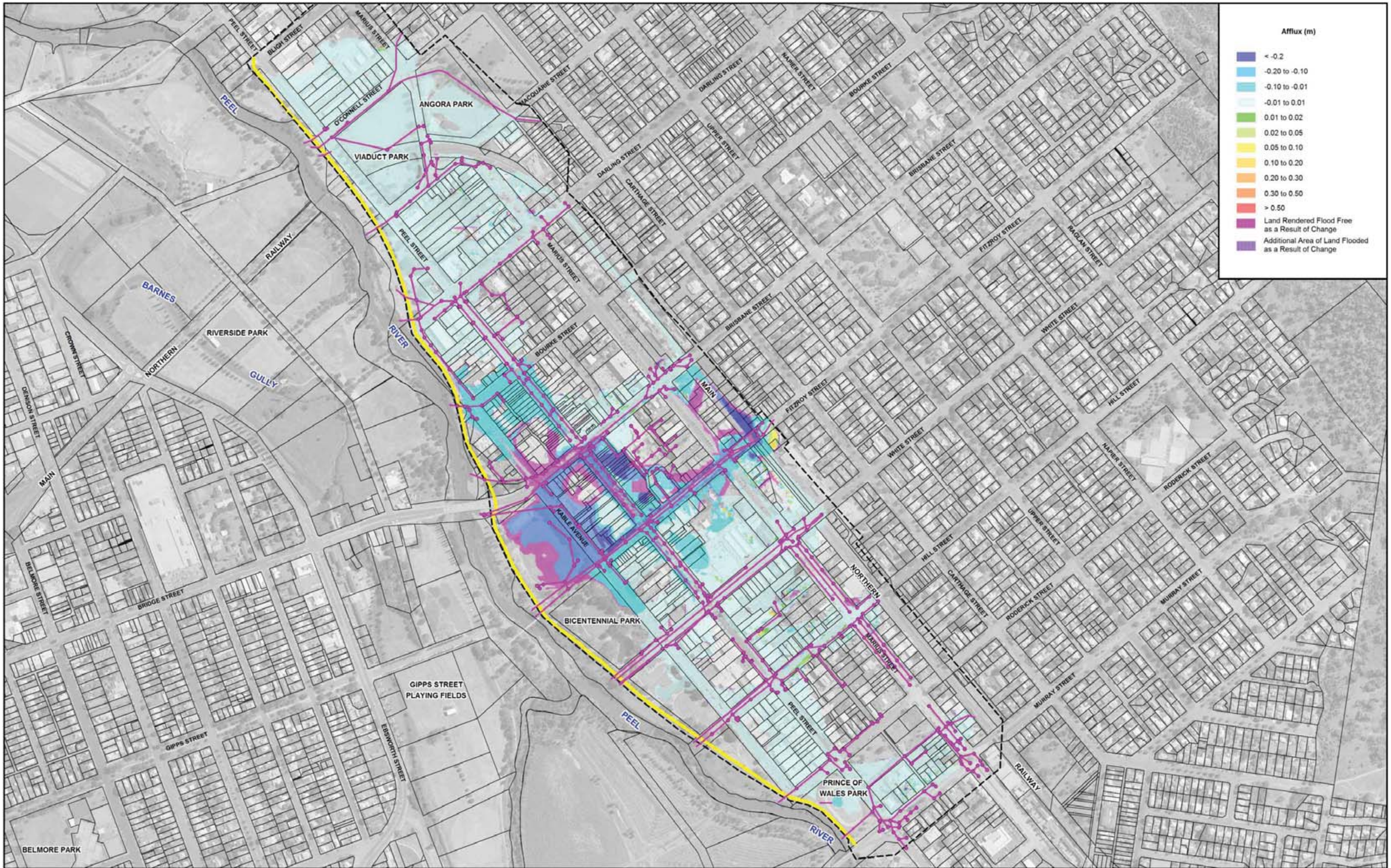

LEGEND

| | |
|---|---|
| | Levee |
| FG 3 | Flood Gate and Identifier |
| | Existing Stormwater Network |
| | Proposed Stormwater Network |
| | Boundary of Council Owned Land |

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.4

FITZROY STREET AND WHITE STREET PRESSURE LINE UGRADES (OPTIONS 2 AND 3)
CBD LEVEE




N

80 0 80 160 240 m

Scale: 1:8,000

**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

**LEGEND**

Levee

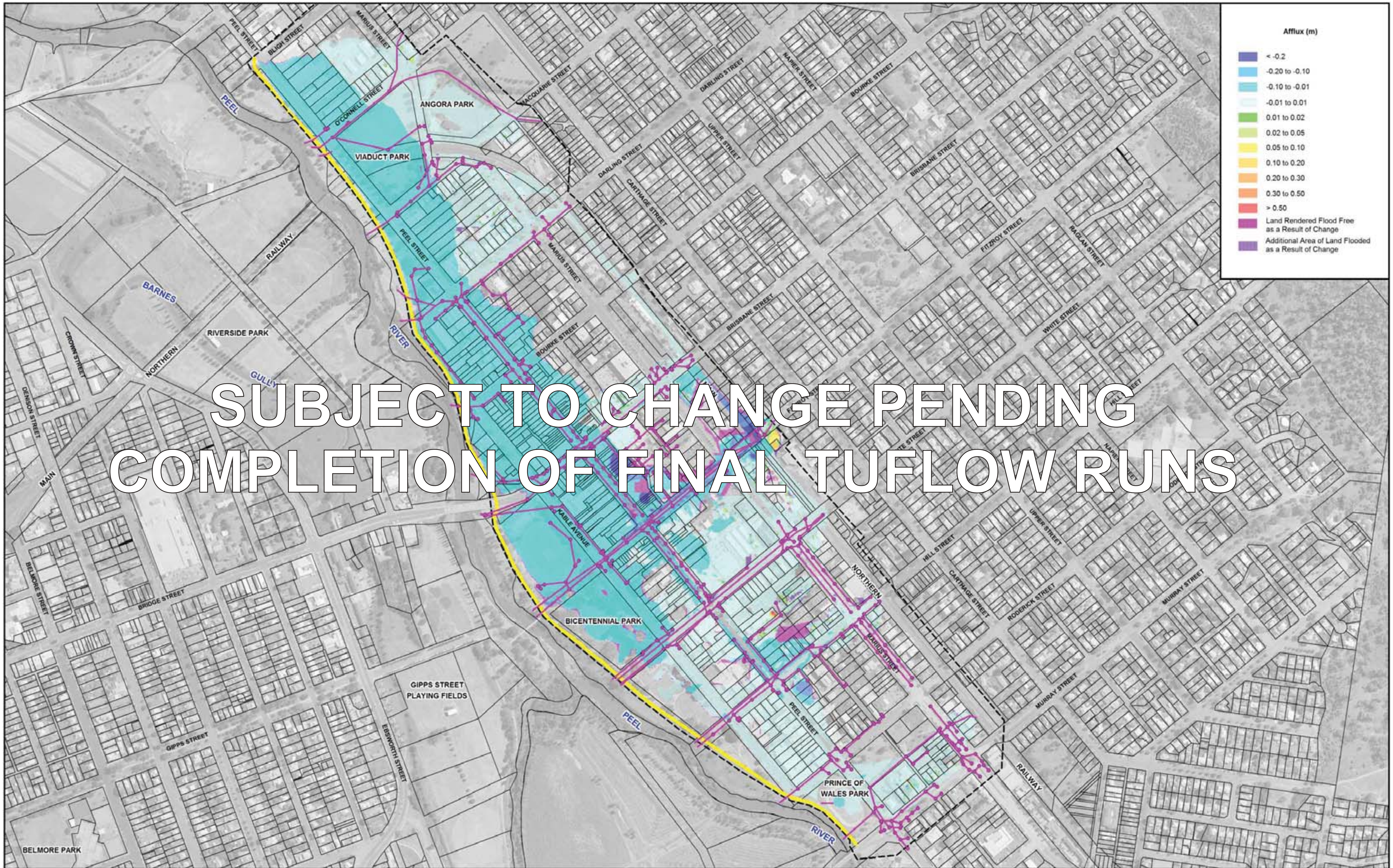
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.5

IMPACT OF OPTION 2 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY OPEN




NOTE:

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

LEGEND

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.6

IMPACT OF OPTION 2 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY CLOSED


**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

**LEGEND**

Levee

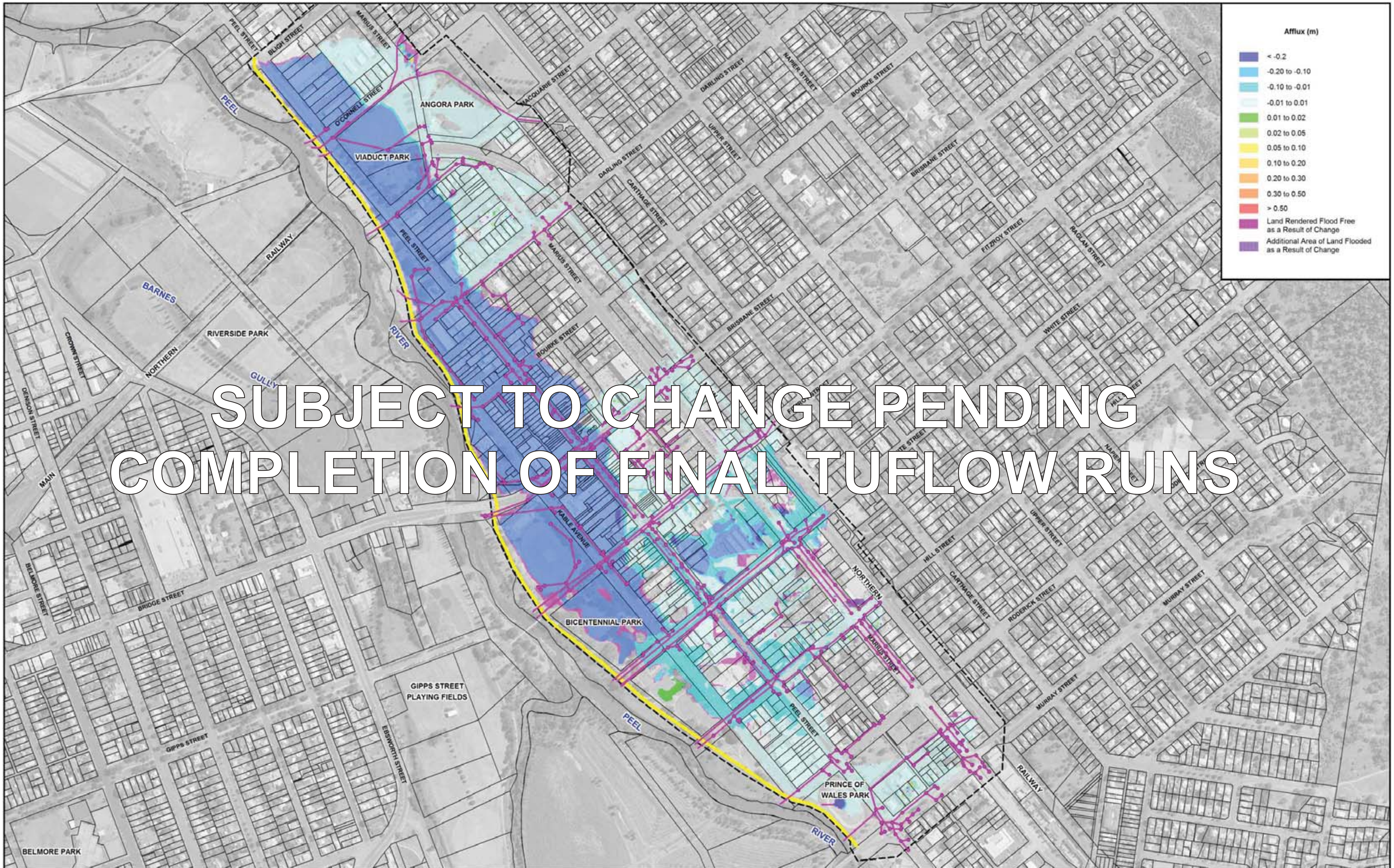
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.7

IMPACT OF OPTION 3 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY OPEN



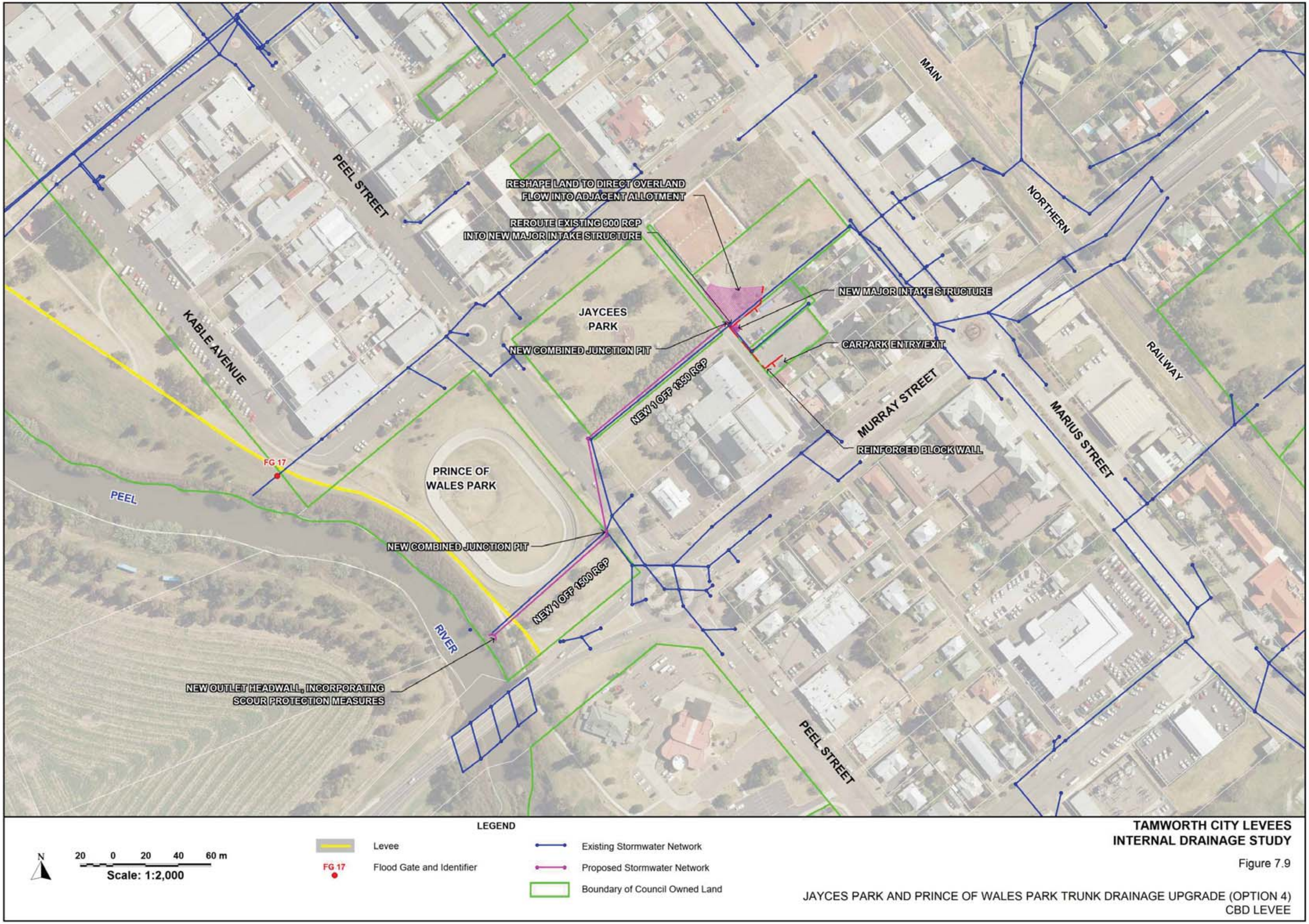

NOTE:
THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.
THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

LEGEND
Levee
Modelled Stormwater Network
Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.8

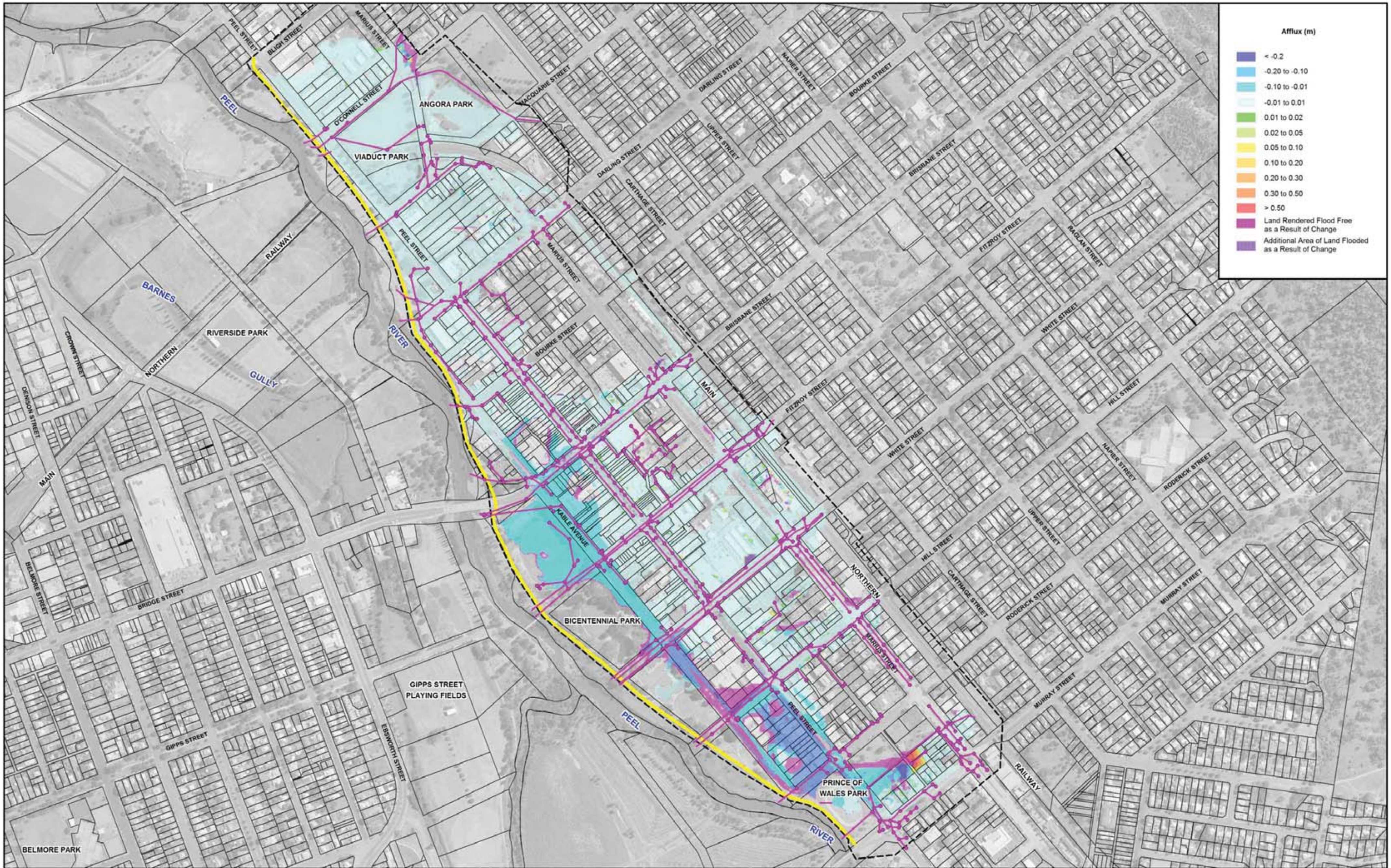
IMPACT OF OPTION 3 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY CLOSED




**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.9

JAYCES PARK AND PRINCE OF WALES PARK TRUNK DRAINAGE UPGRADE (OPTION 4)
CBD LEVEE




N

80 0 80 160 240 m

Scale: 1:8,000

**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

**LEGEND**

Levee

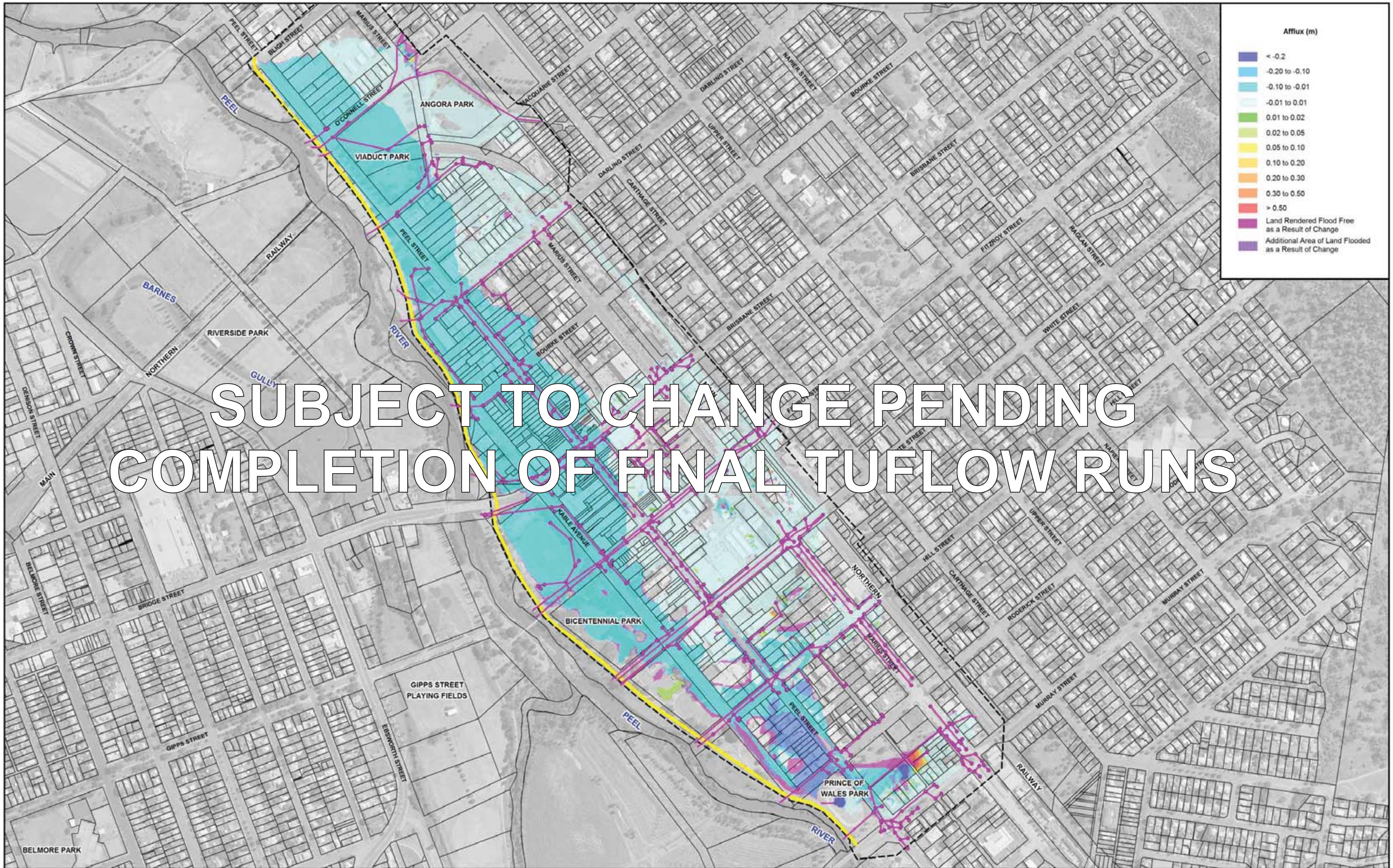
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.10

IMPACT OF OPTION 4 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY OPEN




N

80 0 80 160 240 m

Scale: 1:8,000

**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

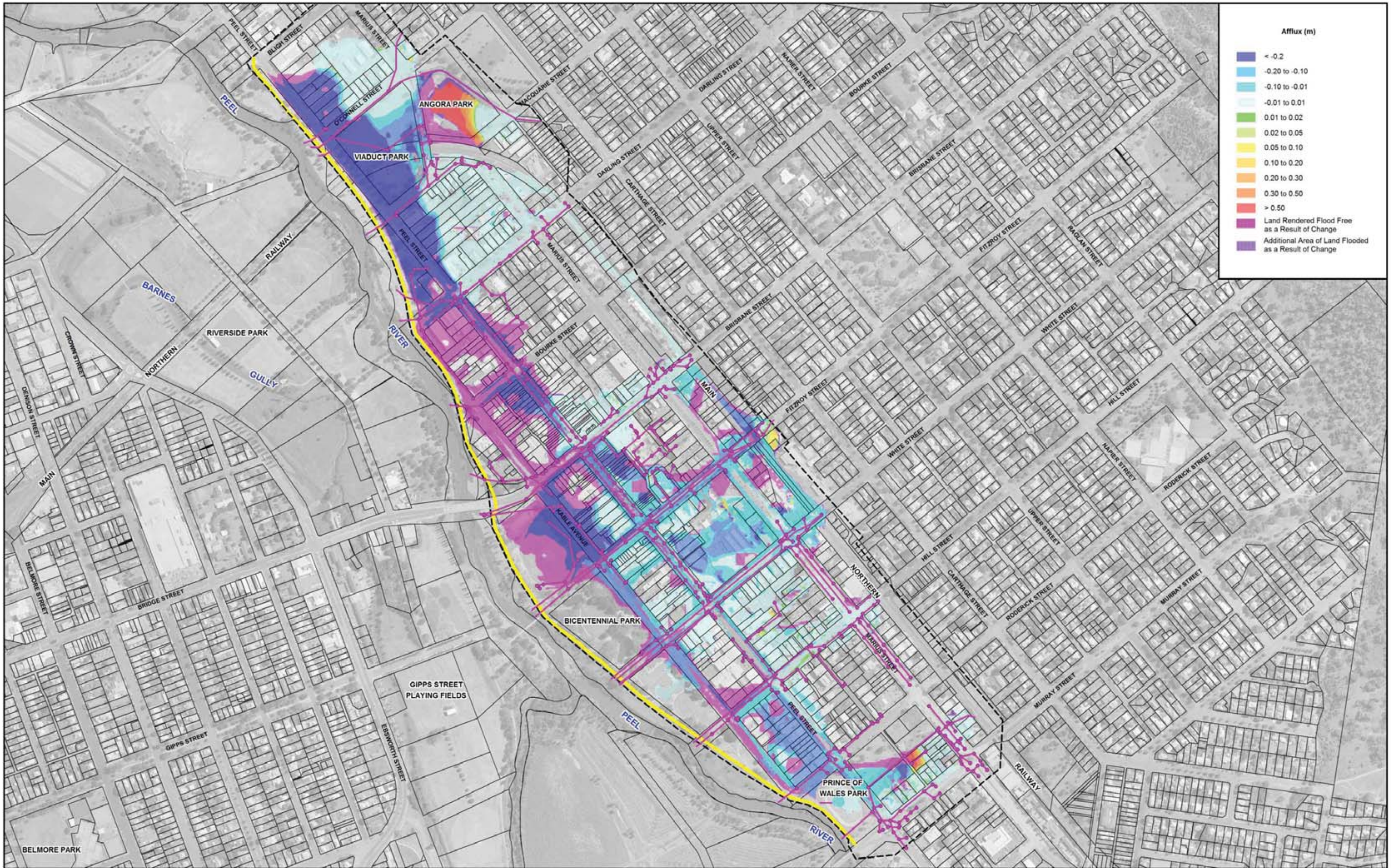
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.11

IMPACT OF OPTION 4 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY CLOSED




N

80 0 80 160 240 m

Scale: 1:8,000

**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

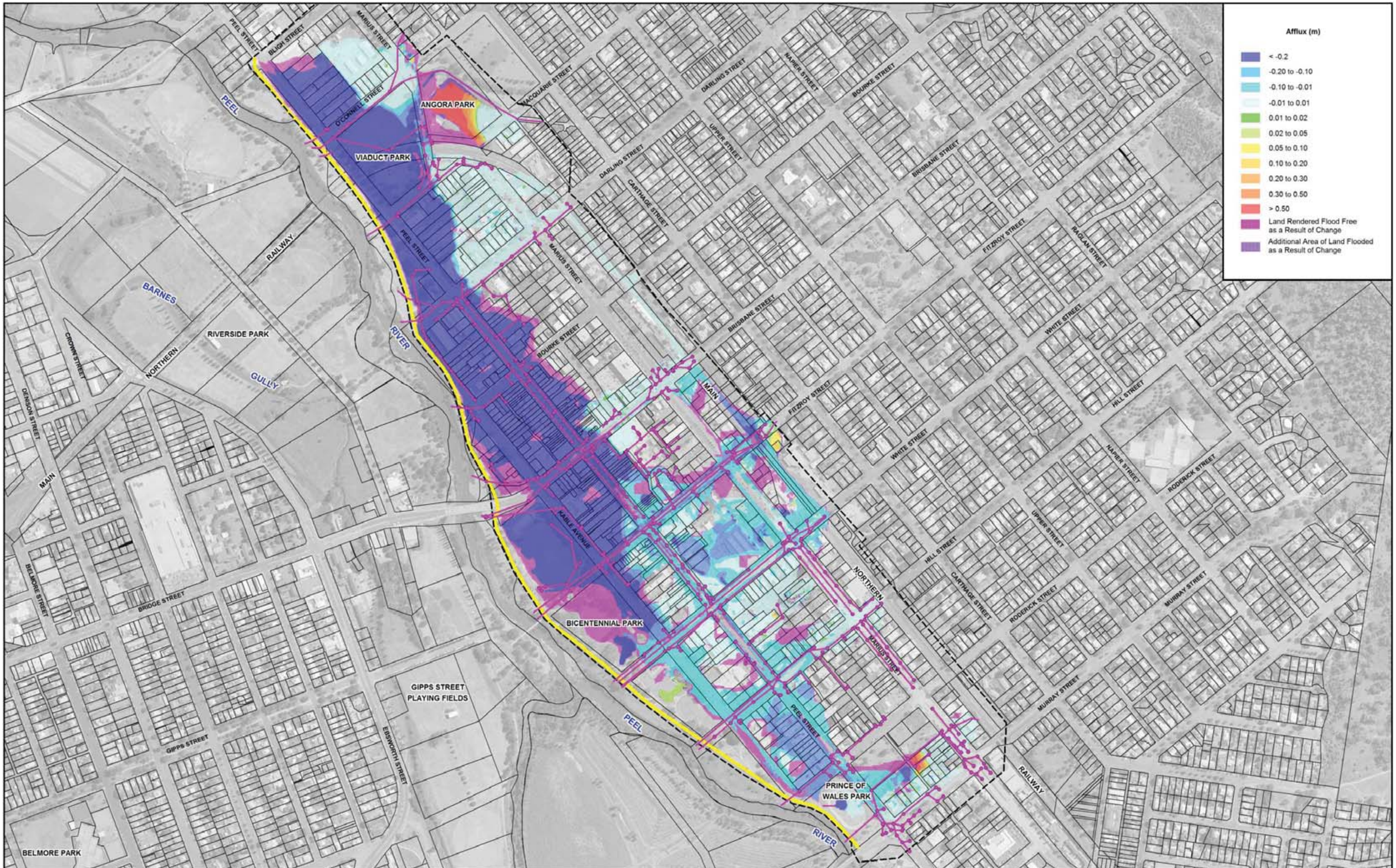
**LEGEND**

- Levee
- Modelled Stormwater Network
- Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.12

IMPACT OF OPTION 5 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY OPEN




NOTE:

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

LEGEND

Levee

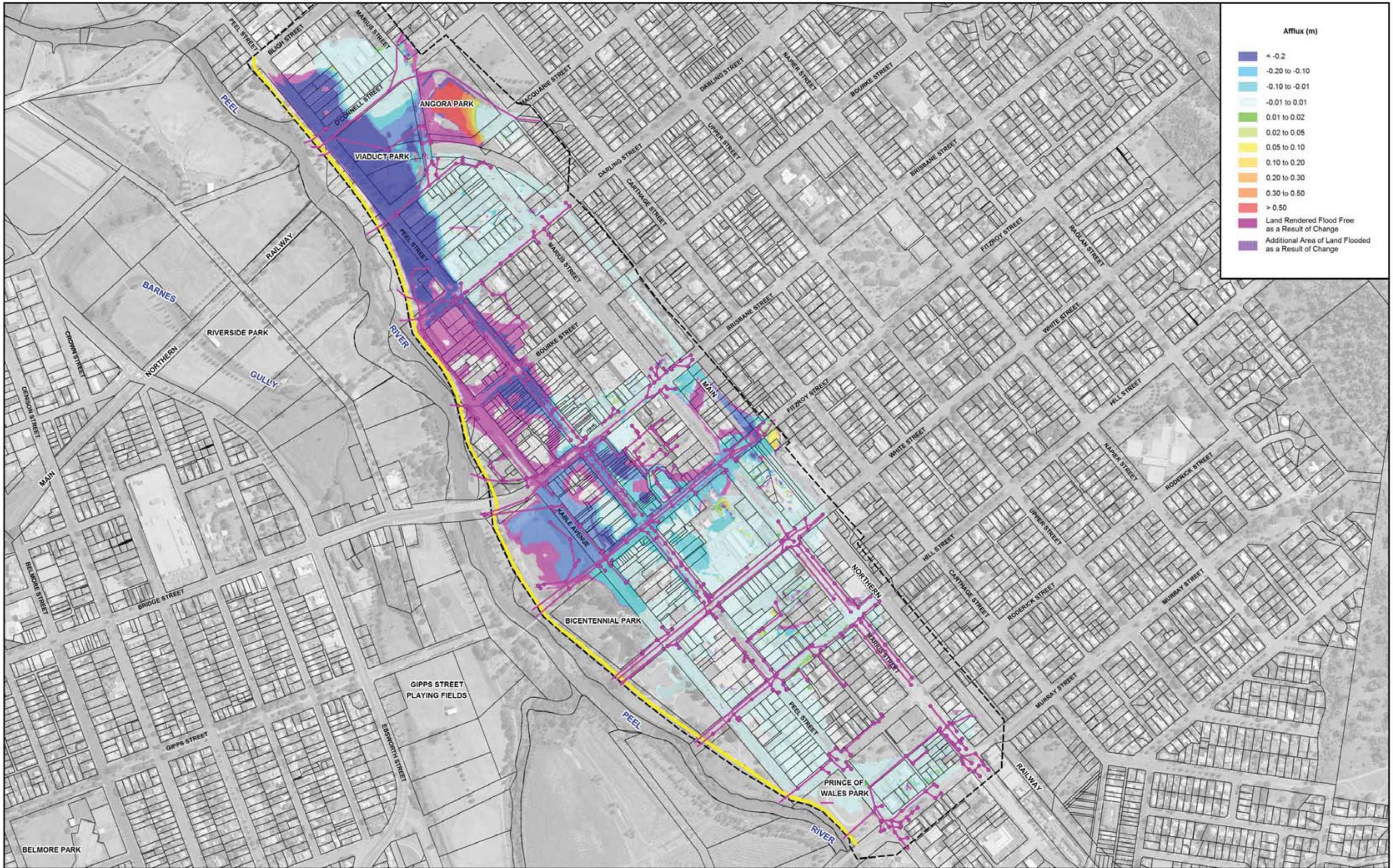
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.13

IMPACT OF OPTION 5 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY CLOSED




NOTE:

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

LEGEND

Levee

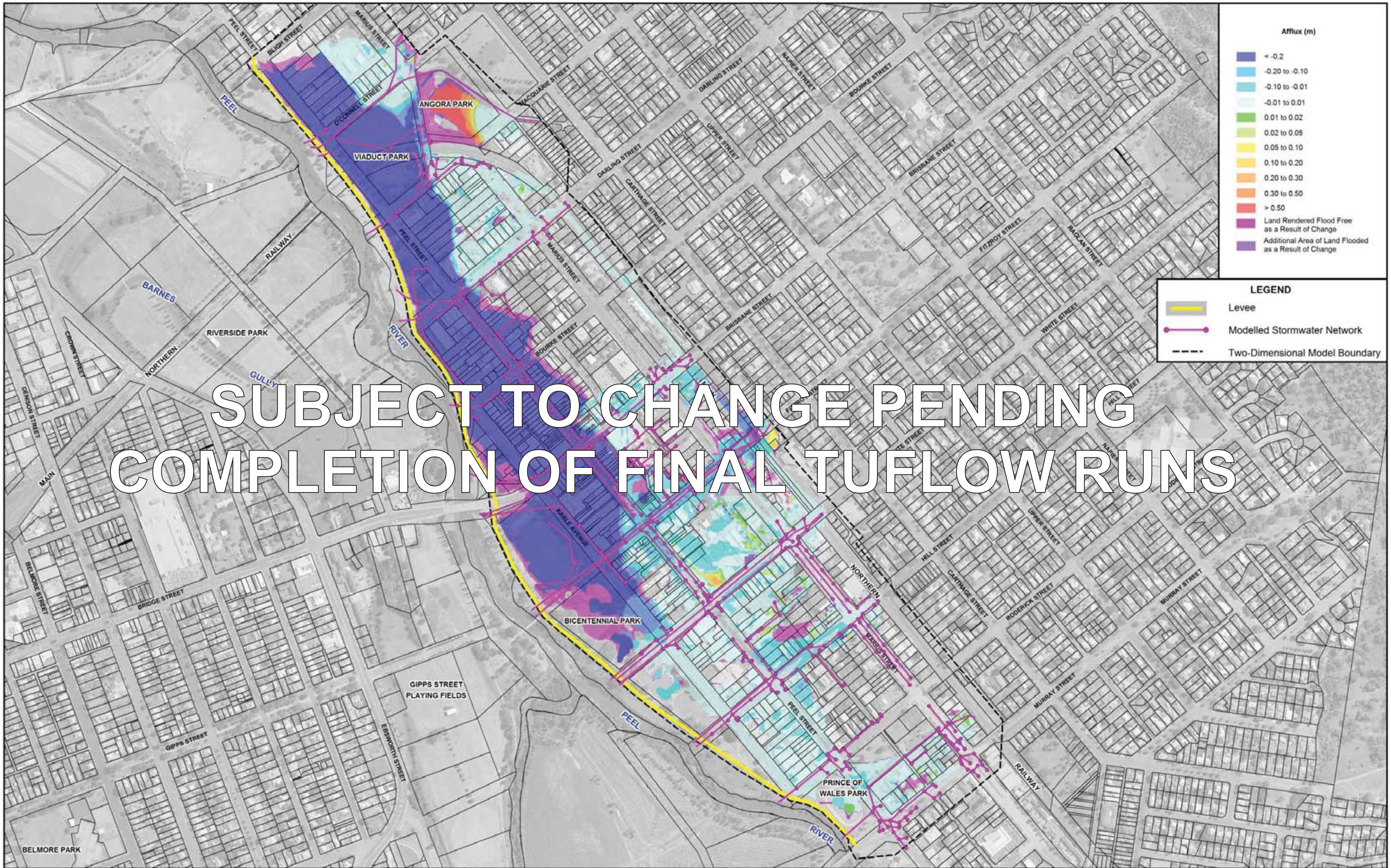
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.14

IMPACT OF OPTION 6 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY OPEN




**NOTE:**

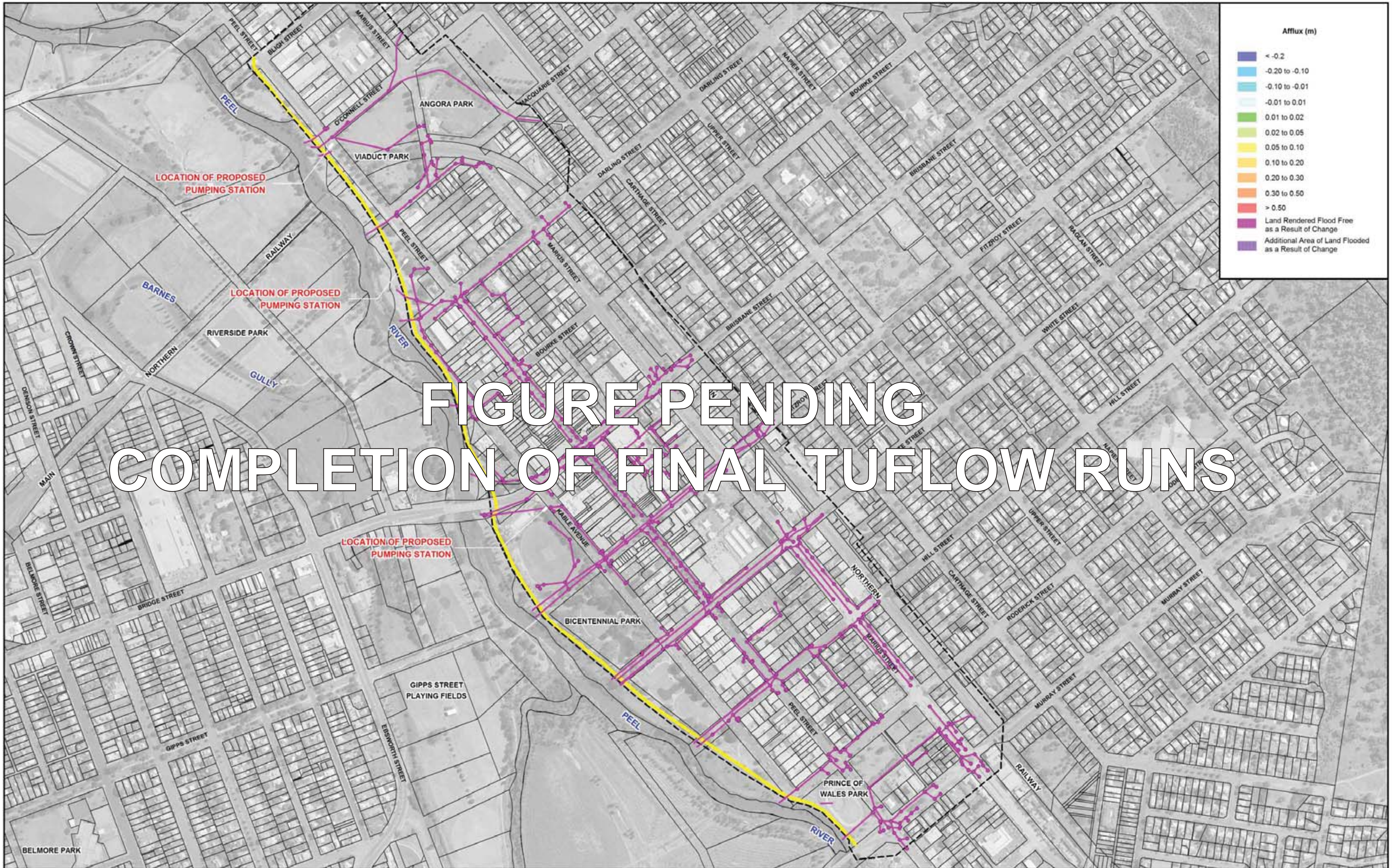
THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.15

IMPACT OF OPTION 5 WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY CLOSED




**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

**LEGEND**

Levee

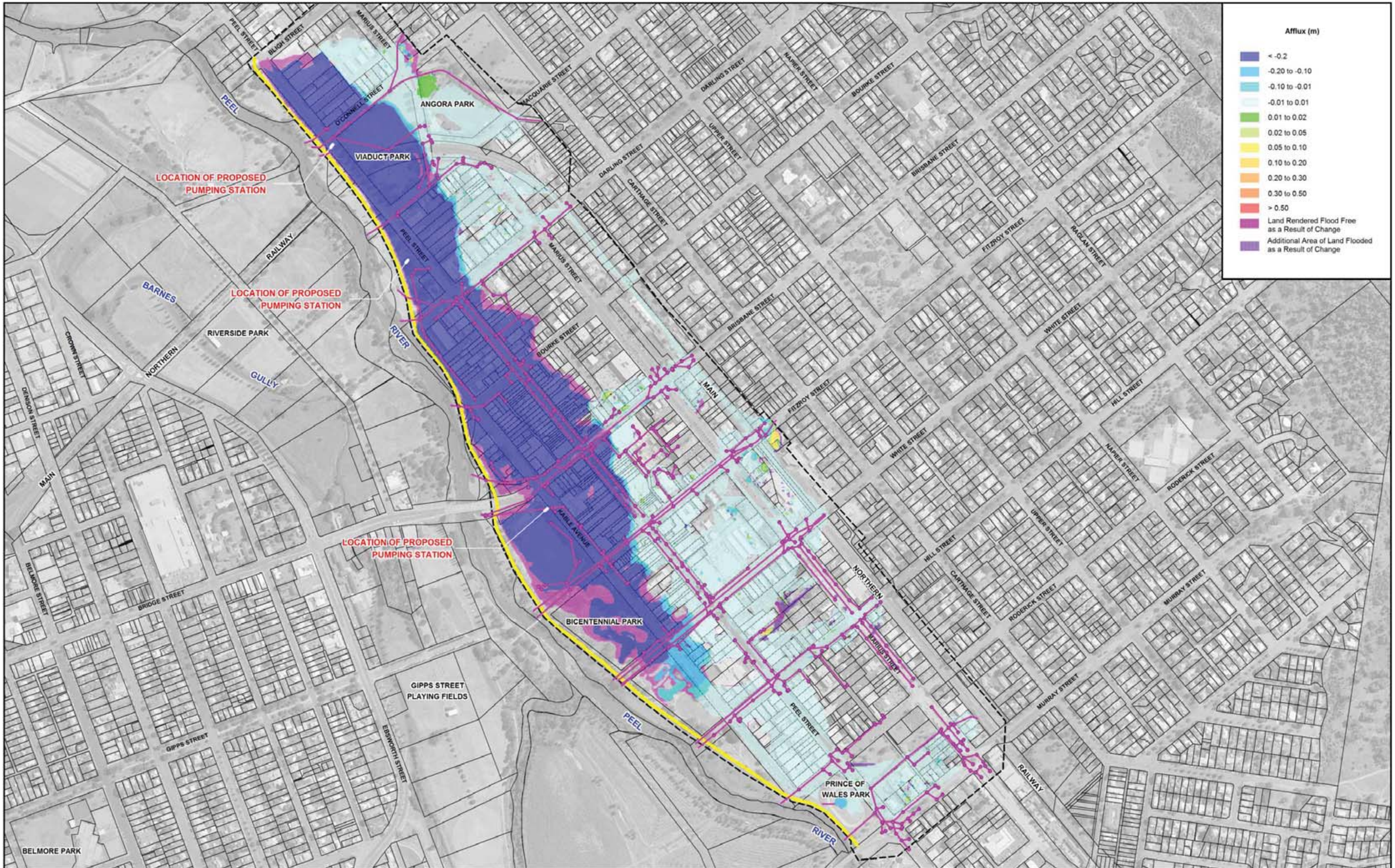
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.16

IMPACT OF OPTION 7A WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY OPEN




N

80 0 80 160 240 m

Scale: 1:8,000

**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

**LEGEND**

Levee

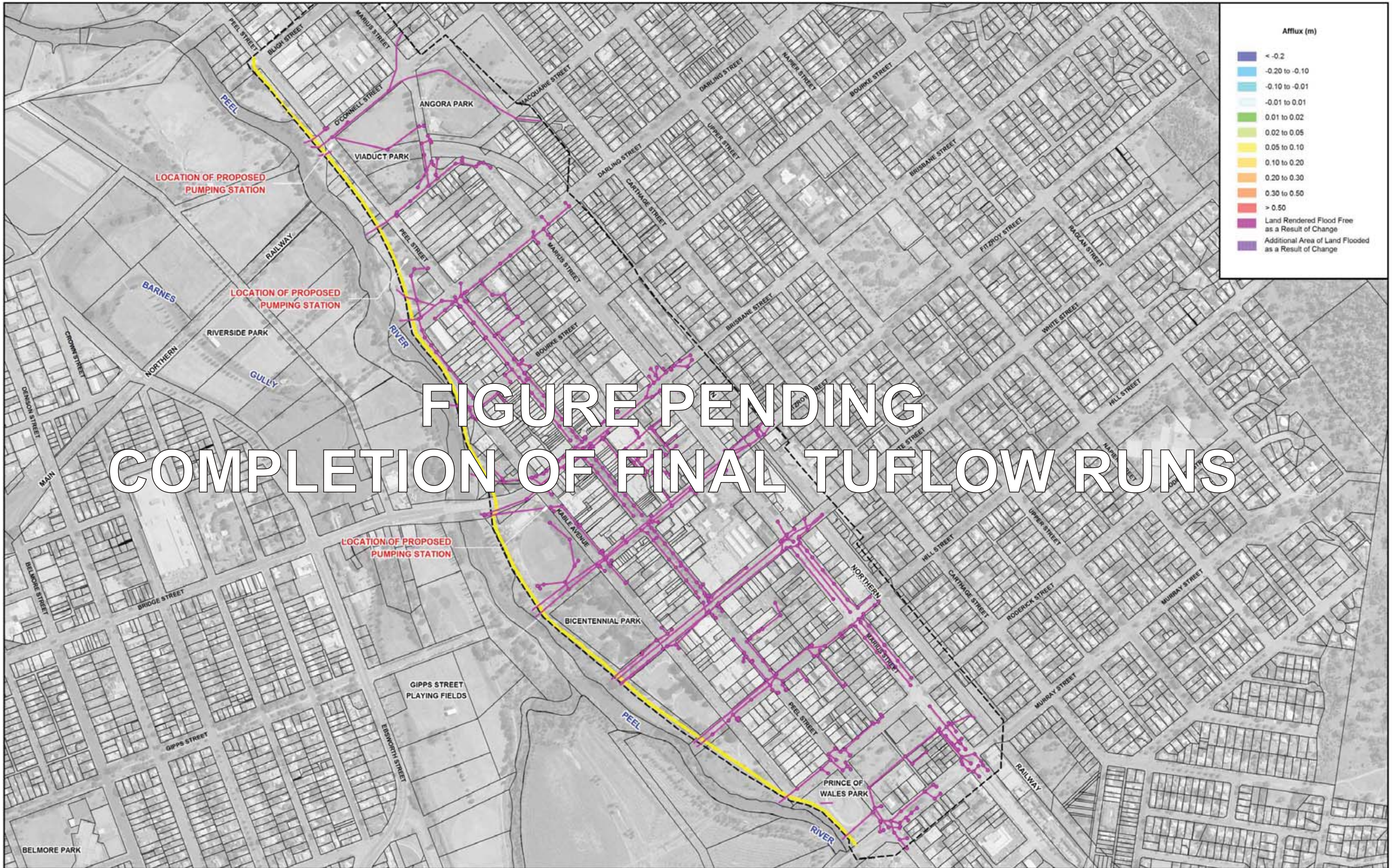
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.17

IMPACT OF OPTION 7A WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY CLOSED




N

80 0 80 160 240 m

Scale: 1:8,000

**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

**LEGEND**

Levee

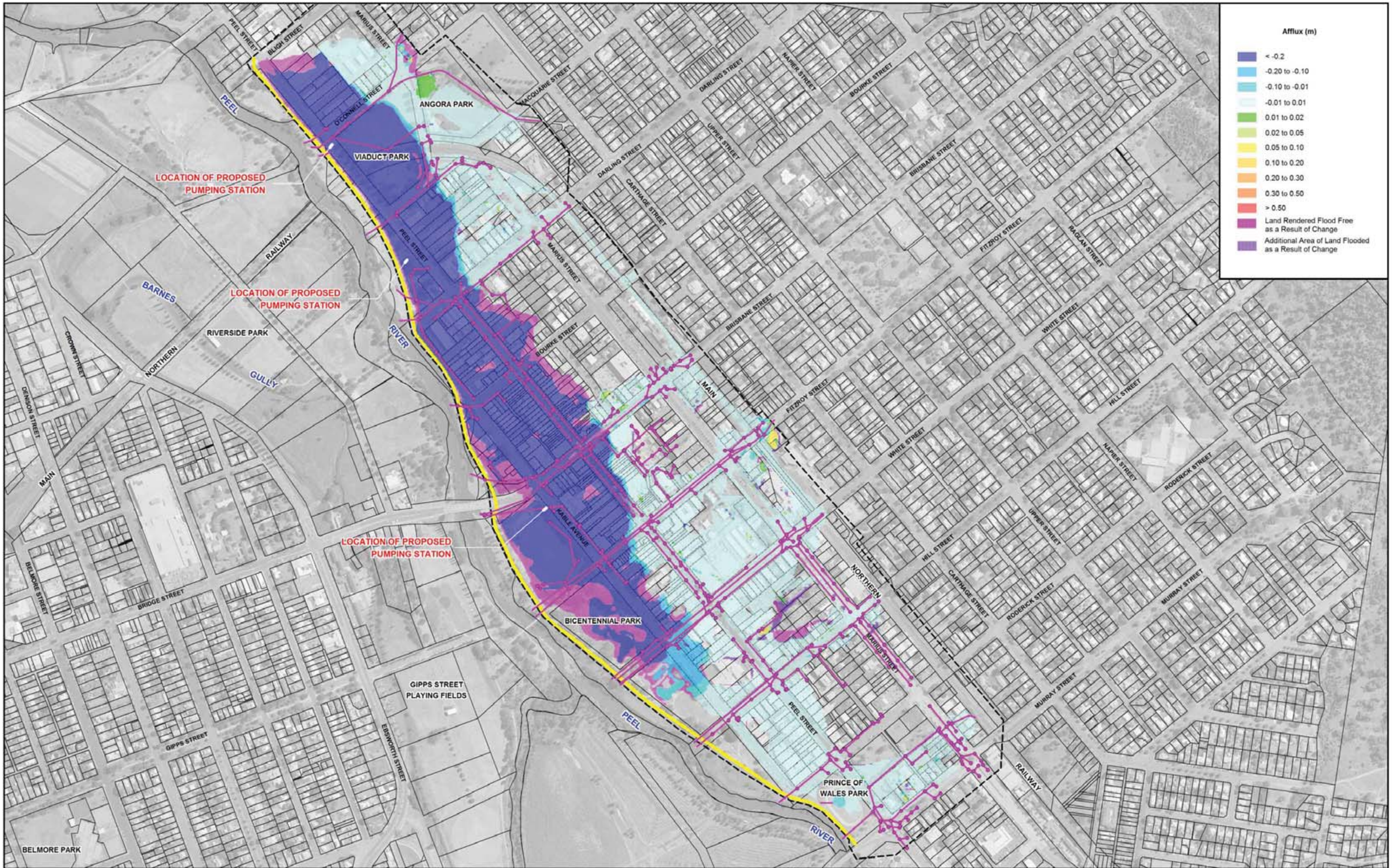
Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.18

IMPACT OF OPTION 7B WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY OPEN




N

80 0 80 160 240 m

Scale: 1:8,000

**NOTE:**

THE EXTENTS AND DEPTHS OF FLOODING SHOWN WERE DETERMINED FROM PHOTOGRAMMETRIC SURVEY DATA AND ARE APPROXIMATE ONLY.

THE EXTENT OF INUNDATION IN INDIVIDUAL ALLOTMENTS NEAR THE FLOOD FRINGE SHOULD BE CONFIRMED BY SITE SPECIFIC SURVEY.

**LEGEND**

Levee

Modelled Stormwater Network

Two-Dimensional Model Boundary

**TAMWORTH CITY LEVEES**
**INTERNAL DRAINAGE STUDY**

Figure 7.19

IMPACT OF OPTION 7B WORKS ON PEAK 100 YEAR ARI FLOOD LEVELS
FLOOD GATES FULLY CLOSED