EAST AND NORTH TAMWORTH
DRAINAGE STUDY

VOLUME 1

JUNE 2021

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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 order to be consistent with the other reports that have recently been prepared for Tamworth Regional Council on flood behaviour at Tamworth, 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
TRC	Tamworth Regional Council

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1 INTRODUCTION

1.1 Background and Approach

This report deals with the findings of an investigation which was undertaken by Lyall & Associates to define the nature of local catchment flooding in the urbanised parts of East and North Tamworth. The present investigation is an extension of a previous study which was undertaken by Lyall & Associates in 2012[1] which defined flood behaviour behind three levees which have been constructed to protect parts of Tamworth from riverine type (denoted individually therein as the "CBD Levee", "Taminda Levee" and "Western Levee", and collectively as "the town levees"). **Figure 1.1** shows the extent of the present study area, as well as the location of the town levees.

The study objective was to define local catchment flood behaviour in the study area 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 local catchment flood behaviour in the CBD of Tamworth is the position of nineteen (19) 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 CBD Levee. 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 was undertaken as part of Lyall & Associates, 2012 to assess the likelihood of the flood gates being closed at the time flood producing rainfall is experienced over the study catchment, 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 both the previous and present studies.

Flood behaviour was defined using computer based hydrologic models of the catchments and hydraulic models of the drainage lines which control local catchment flooding in the study area. 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 and the routing effects of the flood storage which is present behind the CBD Levee. 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 are present in the study area. The TUFLOW hydraulic modelling software 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

[1] *Tamworth City Levees Internal Drainage Study* (Lyall & Associates, 2012)

the manually operated flood gates that are located along the length of the CBD Levee are in either their fully open or fully closed positions. The results of the flood modelling were used to prepare plans showing the indicative extent and depth of local catchment flooding for the design events.

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 WaterNSW 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 date back to 1910.

  The following data were also provided by TRC:

  - Aerial photography which was flown on 18 July 2018.
  - Design drawings showing details of the CBD Levee.
  - A database containing details of major drainage upgrades in hardcopy format.
  - A database which contained pit and pipe data in ARCVIEW format.
  - GIS based data sets including cadastre and watercourse information that were extracted from the NSW Government's *Spatial Information Exchange* website
  - LiDAR survey data which were captured between May and July 2012.
  - Detailed ground survey of hydraulic structures and in-bank cross sectional survey data that were obtained as part of previous investigations.

  A brief setting out the requirements for the capture of pit and pipe data was also prepared at the commencement of the study. Council's surveyors undertook the survey, with the data provided in both spreadsheet and CADD format. Photographs of each stormwater pit were also taken 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 a hydraulic model of the study area and the application of discharge hydrographs to the model to define extents and depths of inundation, flows and velocities for the design floods.

- ***A flood mitigation component*** which comprised a qualitative assessment of whether the findings of the present investigation would substantially change those of Lyall & Associates, 2012, namely in regard the flood mitigation benefits that could be achieved through the implementation of a number of potential flood modification measures.

1.3 Overview of Report

Chapter 2 of the report contains background information including a brief description of the study area, including the CBD Levee; a review of the database 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 which was undertaken as part of Lyall & Associates, 2012; details of historic flooding in Tamworth and an overview of 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 study catchment. This step involved the determination of design storm rainfall depths 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 Lyall & Associates, 2012 in respect to an investigation that was undertaken into the impact a range of potential flood modification measures would have on local catchment flood behaviour directly behind the CBD Levee. A qualitative assessment of whether the findings of the present investigation would substantially alter the findings of Lyall & Associates, 2012 also forms part of this chapter of the report.

Chapter 8 contains a list of References.

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 A**. **Appendix B** bound in **Volume 2** of this report contains extracts from Lyall & Associates, 2012 showing the layout of the potential flood modification measures which were assessed as part of the previous study.

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

2 BACKGROUND INFORMATION

2.1 Study Area

The study area comprises the urbanised parts of East and North Tamworth, a portion of which is protected from riverine type flooding by the CBD Levee. **Figure 2.1** (2 sheets) shows the layout of the existing stormwater drainage system in East and North Tamworth, as well as the extent over which the nature of flood behaviour has been defined as part of the present investigation (as indicated by the extent of the two-dimensional model boundary).

The headwaters of the catchments which contribute to flow in the trunk drainage lines which run through the urbanised parts of East and North Tamworth lie to the north-east and are characterised by relatively steep and mostly wooded areas (refer **Figure 2.2** for extent).

2.2 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 (2 sheets) shows the alignment of the CBD Levee where it runs along the northern (right) bank of the Peel River between Murray Street and Bligh Street, whilst **Figure 2.3** is a longitudinal section showing several key elements of the CBD Levee, such as the elevation of its crest and details of the stormwater drainage lines that outlet to the Peel River through the levee. Design flood levels in the adjacent Peel River are also shown along the length the levee, the peak flood levels for which are based on information contained in the report entitled *"Tamworth City-Wide Flooding Investigation"* (Lyall & Associates, 2019).

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 2.3** for location).

Figure 2.2 shows the extent of the 10 km^2 catchment that 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 this 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 2.1** for extent).

Stormwater discharging to the Peel River from the local catchment is controlled by twenty-three (23) individual drainage lines, the outlet levels of which are shown on **Figure 2.3**. 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 Northern Railway Line, the elevation of which lies 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**: The remaining eighteen (18) drainage lines control stormwater runoff which is generated by the area lies to the west (downslope) of the Main Northern Railway Line. In order to prevent backwater flooding from the Peel River, TRC has fitted manually operated penstock type flood gates to the outlets of these drainage lines. Details of the eighteen (18) penstock type flood gates, the locations of which are shown on **Figure 2.1**, are contained in **Section 2.3**.

2.3 Flood Gate Details

Table 2.1 gives details of the existing flood gates which control backwater flooding behind the CBD Levee, noting that all are of penstock type construction requiring manual operation.

Procedures for the manual operation of the 18 penstock type flood gates which control backwater flooding behind the CBD Levee are contained in TRC's Regional Services Directorate document entitled "*Flood Standing Instructions Tamworth City Area".* Section 3 of the document states the following in relation to the procedures for closure of the flood gates:

"(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.

Figure 2.3 shows the three critical trigger levels of 3.0, 3.6 and 4.0 m projected along the line of the CBD Levee. 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 6.69 m on the town gauge.

TABLE 2.1
DETAILS OF EXISTING FLOOD GATES

Identifier(1)	Flood Gate Type	Pipe/Culvert Dimensions at Outlet of Stormwater Drainage Line(2,3)	Invert Level at Outlet (m AHD)
FG1	Penstock	1 off 450 RCP	372.09
FG2	Penstock	1 off 600 RCP	371.97
O'Connell Street Pressure Line	-	2 off 2100 wide by 1500 high RCBC's	371.97
FG3	Penstock	1 off 900 RCP	371.97
FG4	Penstock	900 wide by 900 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
Brisbane Street Pressure Line	-	1 off 1200 RCP	372.31
FG8a	Penstock	1 off 900 RCP	372.28
Fitzroy Street Pressure Line	-	1 off 1500 RCP	372.56
FG9	Penstock	1 off 1350 RCP	373.07
FG10	Penstock	1 off 450 RCP	374.48
White Street Pressure Line	-	1 off 1200 RCP	374.21
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
FG14	Penstock	1 off 450 RCP	374.52
FG15	Penstock	1 off 1200 wide by 900 high RCBC	374.33
FG16	Penstock	1 off 525 RCP	374.57
FG16-1	Penstock	1 off 1350 RCP	374.14
FG17	Penstock	1 off 900 RCP	375.58

1. Refer **Figure 2.1** (2 sheets) for location of pipe/culvert outlets
2. All dimensions are in millimetres
3. RCP = Reinforced Concrete Pipe RCBC = Reinforced Concrete Box Culvert

2.4 Historic Flooding in Tamworth

2.4.1. Riverine 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".[2] The 1910 flood reached an equivalent level of 6.93 m on the town gauge.

Since records commenced in January 1925, the water level in the Peel River has peaked above the critical 4 m trigger level on the town gauge on 88 separate occasions. The days when the Peel River has peaked above the critical 4 m trigger level are summarised in **Table A1** in **Appendix A**.

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 a period of about four days[3].

A telemetered stream gauge was installed by WaterNSW 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 A2** in **Appendix A** 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.

2.4.2. Local Catchment Flooding

TRC issued a press release at the commencement of Lyall & Associates, 2012 seeking input from the community on historic flooding behind the CBD Levee, as well as two other levees which protect parts of Tamworth on the opposing side of the Peel River. Approximately 300 flood questionnaires were also distributed to residents and business owners of property located directly behind the three levees.

A total of forty-six questionnaires were returned by the closing date of submissions, four of which contained the following information on observed flood behaviour behind the CBD Levee.

- 178 Peel Street - Stormwater observed to pond in Peel Street during periods when the flood gates are closed.
- 184 Peel Street - Depth of flow in the gutter has on several occasions been sufficient to inundate driveway.
- 365 Peel Street - Property has been flooded due to surcharge of the pipe drainage system in Peel Street.
- 523 Peel Street - Property has been flooded due to excessive gutter flow.

[2] PPK (1993)
[3] PPK (1993)

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 Tamworth.

Based on the responses received from several business owners to the 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 of inundation in commercial property as a result of stormwater ponding behind the CBD Levee is limited to property which is located at the southern end of Roderick Street and in Peel Street west of O'Connell Street.

2.5 Consideration of Joint Probability of Coincident Flooding

2.5.1. General

The coincident nature of local catchment and 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.

The drainage lines which control local catchment flooding behind the CBD Levee (excluding the pressure lines) have been fitted with penstock type flood gates, the operational procedures for which dictate that they must be manually closed at the time 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.[4]

The joint probability of coincident local catchment and riverine 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 catchment which drains behind the CBD Levee. Whilst it is beyond the scope of this present investigation to determine the joint probability of local catchment and riverine 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.

2.5.2. 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 A1** in **Appendix A**) shows that there has been about 65 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 that of others, that is, that the frequency of the 'bank full' flow in ARI terms is between 1 to 2 years.[5]

[4] 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 embankment.

[5] See Gippel, 2002; Wong, 2006.

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 A** 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 short duration rainfall which is critical for maximising flows in the drainage lines that control runoff from the catchments which drain 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. Whilst in this instance the intense burst of rainfall which caused flooding in parts of Tamworth occurred only 3 hours prior to water levels in the river exceeding the critical 4 m trigger level on the town gauge, flooding conditions behind the CBD Levee may have been significantly worse had the burst occurred only a few short hours later in the storm event.

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 and 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 CBD Levee.

It needs to be noted that the severity of flooding experienced behind the CBD Levee is a function of the intensity and duration of the rain which falls on the contributing catchment combined with the duration over which the flood gates remain closed.

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 and when setting appropriate Flood Planning Levels for future development on land located in the ponding zone of the CBD Levee.

3 HYDROLOGIC MODEL DEVELOPMENT AND CALIBRATION

3.1 Selection of Hydrologic Modelling Approach

The present investigation required the use of a hydrologic model which is capable of representing the rainfall-runoff processes that occur within both the non-urbanised and urbanised parts of the study catchments. For hydrologic modelling, the practical choice is between the models known as ILSAX, RAFTS, RORB and WBNM. Whilst there is little to choose technically between these models, ILSAX 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.

Both the ILSAX and RAFTS modelling approaches which are built into the DRAINS software were used to generate discharge hydrographs from urban and non-urban areas, respectively, as this combined approach was considered to provide a more accurate representation of the rainfall runoff process in the study catchments. The discharge hydrographs generated by ILSAX and RAFTS were applied to the TUFLOW hydraulic model as either point or distributed inflow sources (refer **Section 4.3.2** of this report for further details).

3.2 Hydrologic Model Setup

There are three primary land-use types present within the study area: the urbanised parts of East and North 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 East and North 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 catchments. Note that the individual sub-catchments have been shaded to separately identify those areas where the RAFTS and ILSAX 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 which lie outside the extent of the TUFLOW model boundary. A simple lag approach was adopted between each sub-catchment modelled in RAFTS, whereby 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.

Sub-catchment slopes used as 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 as 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.

3.3 Hydrologic Model Calibration

3.3.1. General

Quantitative information on historic flooding behind the CBD Levee 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, 1987).

3.3.2. Hydrologic Model Parameters

The ILSAX 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. A PERN value of 0.045 was applied to sub-catchments that are generally cleared of vegetation.

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

3.3.3. Comparison of Peak Flow Estimates

Table 3.1 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.[6]

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 3.1** therefore includes a comparison of the peak flows derived using both values of C_{10}.

By inspection of the peak flows given in **Table 3.1**, 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.

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

TABLE 3.1
COMPARISON OF DESIGN PEAK FLOW ESTIMATES
(m^3/s)

Location	Tributary	Catchment Area (km^2)	5 year ARI			20 year ARI			100 year ARI		
			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	Unnamed Gully	1.90	3.6	5.6	6.6	7.3	11.5	13.0	16.3	25.9	23.4
CBD_PRM_002	Spring Creek	6.62	9.1	14.2	17.3	18.6	29.1	26.5	40.9	65.2	45.2
CBD_PRM_003	Long Gully	2.38	4.3	6.6	7.7	8.7	13.6	14.9	19.3	30.8	27.5
CBD_PRM_004	Garrieties Gully	0.68	1.6	2.5	2.4	3.4	5.5	5.5	7.4	11.8	10.6

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

4 HYDRAULIC MODEL DEVELOPMENT

4.1 Selection of Hydraulic Model

The present investigation required the use of a hydraulic model which is capable of analysing the time varying effects of flow in the stormwater drainage system, the routing effects of flood storage which is present behind the CBD Levee and the two-dimensional nature 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 in the study area. **Figure 4.1** (2 sheets) shows the layout of the various components which comprise the TUFLOW model of the study area.

A 2 m grid spacing was found to provide the appropriate balance between the need to define features in the urban areas of East and North Tamworth versus model run times. Grid elevations were based on the available LiDAR survey data.

One-hundred and fifty cross sections were used to define the in-bank waterway area of the incised channels within the study area, the locations of which are shown on **Figure 4.1**.

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 2175 individual pipe and culvert reaches and 2052 pits which comprise the TUFLOW model. Inverts levels were taken from TRC's pit and pipe database and Work-As-Executed design drawings where available. A 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.

In order to more accurately define the conveyance capacity of the enclosed sections of concrete lined channel which are located beneath the road crossings of Spring Creek, Rifle Range Gully and Long Gully, a series of one-dimensional elements comprising a "bridge" type channel at the inlet of each structure followed by a series "S" type channels linked to "HW" type cross sections were incorporated in the TUFLOW model.

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

A short reach of the Peel River and its immediate overbank area was incorporated in the TUFLOW model to facilitate the free discharge of flow from the various piped reaches which control runoff from the study area, details of which were taken from Lyall & Associates, 2019.

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 **Figure 4.1**.

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**.

A baseflow was applied to the upstream boundary of the TUFLOW model to represent a fresh in the Peel River at the time of a local catchment flood event. The results of the modelling were then trimmed to northern (right) bank of the river, as reference should be made to Lyall & Associates, 2019 for the definition of riverine type flood behaviour at Tamworth.

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 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.

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 Lined Channels	0.035[(1)]
Grass or Lawns	0.045
Macrophytes	0.06
Lightly Vegetated Area	0.07
Heavily Vegetated Area	0.08
Vegetated Channels	0.09
Allotments where fences and outbuildings are present.	0.1
Buildings	10

1. Due to the step nature of the concrete lined channels in the study area and the resultant high flow velocities, a Mannings n value of 0.035 was required in order for the TUFLOW model to run stable .

Figure 4.2 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 system at the intersection of Hyman Street and Johnston Street, North Tamworth. 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 4.2** may be presented at any location within the model domain (which is shown on **Figures 4.1**) and will be of assistance to TRC in assessing individual flooding problems behind the town levees.

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 the 1987 edition of *Australian Rainfall and Runoff* (**ARR 1987**) (IEAust, 1987). Design storms for frequencies of 2, 5, 10, 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 ARR 1987. 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 ARR 1987 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 the study area 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 ARR 1987. 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 ARR 1987 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 ARR 1987 are extrapolated to this ARI.

5.2 Probable Maximum Precipitation

Estimates of Probable Maximum Precipitation (PMP) were made using the Generalised Short Duration Method 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 3.1 shows the location and orientation of the PMP ellipses which were used to derive the rainfall estimates for each individual catchment which drains the study area.

5.3 Derivation of Design Flood Hydrographs

The hydrologic model was run with the parameters set out in **Section 3.3.2** to obtain design hydrographs for input to the TUFLOW hydraulic model.

Table 5.1 over gives peak flows for design storms of 2, 5, 10, 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 **Figure 4.1** for location of inflow hydrographs).

It is noted that peak flows generated by the hydrologic model for the PMF event are between 7-8 times those derived for the 100 year ARI event. This large multiplier is a function of the PMP estimates which are a similar multiple greater than the 100 year ARI design excess rainfall depths derived using the method outlined in **Section 5.1.1**.

TABLE 5.1
SUMMARY OF DESIGN PEAK FLOWS
(m^3/s)

Peak Flow Identifier[(1)]	Design Storm Event						
	2 year ARI	5 year ARI	10 year ARI	20 year ARI	100 year ARI	200 year ARI	PMF
PFI-01	0.0	0.1	0.2	0.2	0.4	0.5	2.2
PFI-02	0.1	0.2	0.3	0.4	0.8	0.9	4.2
PFI-03	4.6	6.3	9.7	13.0	23.4	27.6	194.7
PFI-04	0.3	0.4	0.6	0.8	1.6	1.8	13.8
PFI-05	0.1	0.1	0.3	0.4	0.7	0.9	5.2
PFI-06	0.3	0.4	0.6	0.9	1.7	2.0	15.0
PFI-07	11.6	17.3	20.8	26.5	45.2	54.0	458.7
PFI-08	0.1	0.2	0.5	0.7	1.2	1.5	9.4
PFI-09	0.3	0.6	1.2	1.7	2.8	3.5	24.9
PFI-10	1.9	2.5	3.9	5.4	10.3	12.3	90.3
PFI-11	0.1	0.3	0.6	0.8	1.5	1.8	10.4
PFI-12	2.3	3.5	6.7	9.1	15.4	18.0	113.9
PFI-13	0.2	0.3	0.7	1.0	1.6	2.0	13.5
PFI-14	5.5	7.7	11.0	14.9	27.5	32.5	255.0
PFI-15	0.1	0.1	0.3	0.4	0.7	0.9	6.0
PFI-16	0.0	0.0	0.1	0.1	0.1	0.2	1.2
PFI-17	0.1	0.1	0.3	0.4	0.7	0.8	5.8
PFI-18	0.5	0.6	1.2	1.6	3.1	3.7	24.2
PFI-19	0.1	0.1	0.2	0.3	0.5	0.6	3.6
PFI-20	0.1	0.1	0.3	0.4	0.7	0.9	5.0
PFI-21	0.1	0.2	0.3	0.5	0.9	1.1	6.5
PFI-22	1.8	2.4	4.1	5.5	10.6	12.4	84.3
PFI-23	0.0	0.1	0.1	0.2	0.3	0.3	1.9
PFI-24	0.1	0.1	0.2	0.3	0.6	0.7	4.5
PFI-25	0.1	0.1	0.2	0.3	0.5	0.6	3.9
PFI-26	0.3	0.4	0.7	1.0	2.0	2.3	15.2
PFI-27	0.1	0.3	0.6	0.8	1.4	1.8	9.3
PFI-28	0.8	1.0	1.8	2.5	4.7	5.6	37.3
PFI-29	0.2	0.3	0.6	0.8	1.4	1.8	10.4
PFI-30	0.1	0.1	0.2	0.3	0.4	0.5	4.1
PFI-31	0.8	1.1	2.0	2.7	5.2	6.2	40.0

1. Refer **Figure 4.1** (2 sheets) for location of Peak Flow Identifiers

6 HYDRAULIC MODELLING OF DESIGN FLOODS

6.1 Presentation of Results

Figures 6.1 to **6.21** show the indicative extent and depth of inundation in the study area for each of the assessed design storm events under both flood gates fully open and flood gates fully closed conditions, as well as the difference in peak flood levels between the two conditions (referred to on the figures as "afflux').

The depths and extents of inundation shown on **Figures 6.1** to **6.21** 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 generally remain above the critical trigger level of 4 m on the town gauge.[7]

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 CBD Levee, 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 A2** in **Appendix A** 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 embankment.

The results of the hydraulic modelling undertaken for the case where the flood gates are in their fully closed position therefore represent the upper envelope of potential flooding in terms of the maximum depth and extent of inundation which could be experienced behind the levee for each ARI storm.

Table 6.1 over sets out the maximum elevation to which local catchment runoff ponds behind the CBD Levee for the flood gates in their fully open and fully closed positions, the reference location for which are shown on the figures described above, while **Table 6.2** summarises the peak flows surcharging the major trunk drainage lines which control runoff behind the CBD Levee. For comparative purposes, **Tables 6.1** and **6.2** include corresponding peak stage and discharge values that were taken from Lyall & Associates, 2012.

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 Levee has not been assessed as part of the present investigation.

[7] While water levels have not remained above 4 m on the gauge for a period exceeding 36 hours since the telemetered stream gauge was installed in 1993, there is a reference in PPK, 1993 that water levels remained above this level for a period of 4 days at the time of the February 1955 flood.

TABLE 6.1
MAXIMUM PONDING LEVELS BEHIND CBD LEVEE[(1)]
(m AHD)

Design Storm Event	Flood Gate Position	Ponding Level Identifier[(2)]	
		PL01 [Intersection of Peel Street and O'Connell Street]	**PL02 [Intersection of Brisbane Street and Kable Avenue]**
2 year ARI	Fully Open	374.71 [374.97]	- [375.96]
	Fully Closed	375.37 [376.26]	376.43 [376.66]
5 year ARI	Fully Open	374.90 [375.17]	375.95 [376.14]
	Fully Closed	375.66 [376.46]	376.67 [376.71]
10 year ARI	Fully Open	374.95 [-]	376.00 [-]
	Fully Closed	375.86 [-]	376.73 [-]
20 year ARI	Fully Open	374.99 [375.81]	376.09 [376.42]
	Fully Closed	376.06 [376.82]	376.77 [376.82]
100 year ARI	Fully Open	375.77 [376.27]	376.40 [376.72]
	Fully Closed	376.61 [377.33]	376.87 [377.33]
200 year ARI	Fully Open	376.04 [376.44]	376.64 [376.78]
	Fully Closed	377.04 [377.46]	377.04 [377.46]
PMF	Fully Open	379.18 [379.19]	379.41 [379.70]
	Fully Closed	379.09 [379.19]	379.43 [379.70]

1. Values in [ ] are taken from Lyall & Associates, 2012 and are provided for comparative purposes only. Refer **Chapter 7** of this report for further discussion.
2. Refer **Figures 6.1** to **6.21** for location of Ponding Level Identifiers

TABLE 6.2
PEAK FLOWS SURCHARGING EXISTING TRUNK DRAINAGE LINES NORTH OF CBD LEVEE[1]
(m^3/s)

Location	Surcharge Location Identifier[2]	Design Storm Event						
		2 year ARI	5 year ARI	10 year ARI	20 year ARI	100 year ARI	200 year ARI	PMF
Marius Street immediately west of O'Connell Street	S01	0 [0]	0 [0]	0.1[3] [-]	0.2 [3.6]	14.0 [11.4]	20.2 [15.9]	187 [240]
Marius Street immediately east of O'Connell Street	S02	0 [0]	0.1[3] [0]	0.1[3] [-]	0.1[3] [4.4]	1.2 [10.7]	1.9 [11.5]	53 [17]
Macquarie Street north of Main Northern Railway Line	S03	0.3[3] [0]	0.4[3] [0.4]	0.4[3] [-]	0.5 [3.9]	0.7 [7.2]	1.0 [1.9]	23 [47]
Darling Street immediately south of Marius Street	S04	0.1 [0]	0.1[3] [0.2]	0.6[3] [-]	2.9 [0.5]	11.2 [0.7]	15.2 [0.9]	92 [35]
Bourke Street immediately south of Marius Street	S05	0 [0]	0 [0]	0 [-]	0 [0]	0 [0]	0 [0]	97 [36]
Brisbane Street immediately south of Marius Street	S06	0 [0]	0.1[3] [0]	0.4[3] [-]	0.8 [1.4]	2.3 [2.7]	3.3 [3.3]	65 [75]
Fitzroy Street immediately south of Marius Street	S07	0 [0]	0 [0.5]	0.1 [-]	0.8 [4.0]	2.4 [7.5]	4.1 [10.6]	108 [80]
White Street immediately south of Marius Street	S08	0.2[3] [0]	0.5[3] [0.8]	0.8 [-]	1.8 [1.7]	5.0 [3.1]	7.0 [4.1]	104 [55]
Hill Street immediately north of Marius Street	S09	0 [0]	0 [0.4]	0.2[3] [-]	0.6 [1.4]	1.6 [3.7]	2.1 [4.6]	33 [20]
Peel Street immediately east of Roderick Street	S10	0 [0]	0 [0.2]	0.3 [-]	0.5 [1.9]	0.9 [4.8]	1.0 [6.3]	23 [65]

1. Values in [ ] are taken from Lyall & Associates, 2012 and are provided for comparative purposes only. Refer **Chapter 7** of this report for further discussion.
2. Refer **Figures 6.1** to **6.21** for location of Surcharge Location Identifiers
3. Minor surcharge flow which does not generate depths of overland flow of greater than 0.1 m at location of breakout.

6.2 Discussion of Results

The key findings of the present investigation in regards flood behaviour under gates fully open conditions are as follows:

i. Major surcharge of the enclosed reaches of the trunk drainage system occurs at the following locations:

   a. At the Johnston Street and Piper Street crossings of Unnamed Gully during storms that are more intense than about 20 year ARI.

   b. At the Johnston crossing of Spring Creek during storms that are more intense than about 20 year ARI.

   c. At the Janison Street and Victoria Street crossings of Rifle Range Gully during storms that are more intense than about 5 year ARI.

   d. At the Piper Street crossing of Rifle Range Gully during storms that are more intense than 20 year ARI.

   e. At the Carthage Street, Griffin Avenue and Marius Street crossings of Rifle Range Gully during storms that are more intense that 100 year ARI.

   f. At the Raglan Street crossing of Long Gully during storms that are more intense than about 10 year ARI.

   g. Upstream of the Napier Street crossing of Garrieties Gully during storms that are more intense than about 5 year ARI.

ii. The majority of overland flow which surcharges the existing stormwater drainage system is able to re-enter the enclosed reaches of the network prior to reaching the protected area behind the CBD Levee for storms up to 100 year ARI in intensity, with the exception of flow which surcharges Long Gully and Rifle Range Gully which contributes to major ponding which commences to occur between about Chainage 1600 and 2300 during storms that are more intense than about 20 year ARI. It is noted that floodwater which ponds behind the levee at this location extends south along Peel Street as far as Bourke Street in a 100 year ARI storm event.

iii. While flow which surcharges the trunk drainage system on Rifle Range Gully discharges directly to the pondage behind the levee (albeit through existing commercial development that is located on the western side of O'Connell Street south of Marius Street), flow which surcharges Long Gully at its crossing of Raglan Street first ponds in the Main Northern Railway corridor at the location of a trapped low point that is located between Darling Street and Brisbane Street before discharging overland to the pondage via these two streets.

iv. Floodwater would pond behind the CBD Levee along most of its length during a PMF event, with peak flood levels controlled by natural surface levels at its western end.

v. While depths of overland flow in existing residential development does not generally exceed 0.3 m in a 100 year ARI storm event, it does exceed this depth at the following locations:

   a. On Spring Creek in the area bounded by Piper Street to the south, Dean Street to the west, Johnston Street to the north and Smith Street to the east.

   b. In Rifle Range Gully catchment on the northern side of Daruka Road, west of its intersection with Janison Street.

   c. On Rifle Range Gully in the area bounded by Piper Street to the south, Bligh Street to the west, Johnston Street to the north and Victoria Street to the east.

d. On Rifle Range Gully in the area bounded by North Street to the south, Bligh Street to the west, Piper Street to the north and Cohen Street to the east.

e. On Rifle Range Gully in the area bounded by Marius Street to the south, Bligh Street to the west, Carthage Street to the north and O'Connell Street to the east.

f. On Long Gully in the area bounded by Upper Street to the south, Darling Street to the west, Raglan Street to the north and Bourke Street to the east.

g. In the Garrieties Gully catchment in the area bounded by the main Northern Railway to the south, White Street to the west, Raglan Street to the north and Murray Street to the east.

h. On the southern side of Armidale Road in the vicinity of its intersection with Hayne Street.

i. Between Armidale Road and the Main Northern Railway opposite the extension of Prentice Avenue.

j. On the southern side of Armidale Road a short distance to the east of the abovementioned properties.

The key findings of the present investigation in regards flood behaviour under <u>gates fully closed</u> conditions are as follows:

i. While major ponding behind the CBD Levee is generally confined to undeveloped land in the vicinity of Viaduct Park for storms up to 5 year ARI in intensity, it does extend into the frontage of several commercial properties that are located on the northern side of Peel Street either side of the open space area.

ii. Floodwater ponding behind the CBD Levee in the vicinity of Viaduct Park will commence to backwater along Peel Street east of Darling Street during storms that are more intense than about 5 year ARI.

iii. Major ponding also occurs at the location of the major sag in Peel Street immediately east of its intersection with the Brisbane Street. While depths of ponding are sufficient to inundate existing commercial development that is located immediately adjacent to the sag during storms as frequent as 2 year ARI, they do not exceed 1 m for storms up to 20 year ARI in intensity.

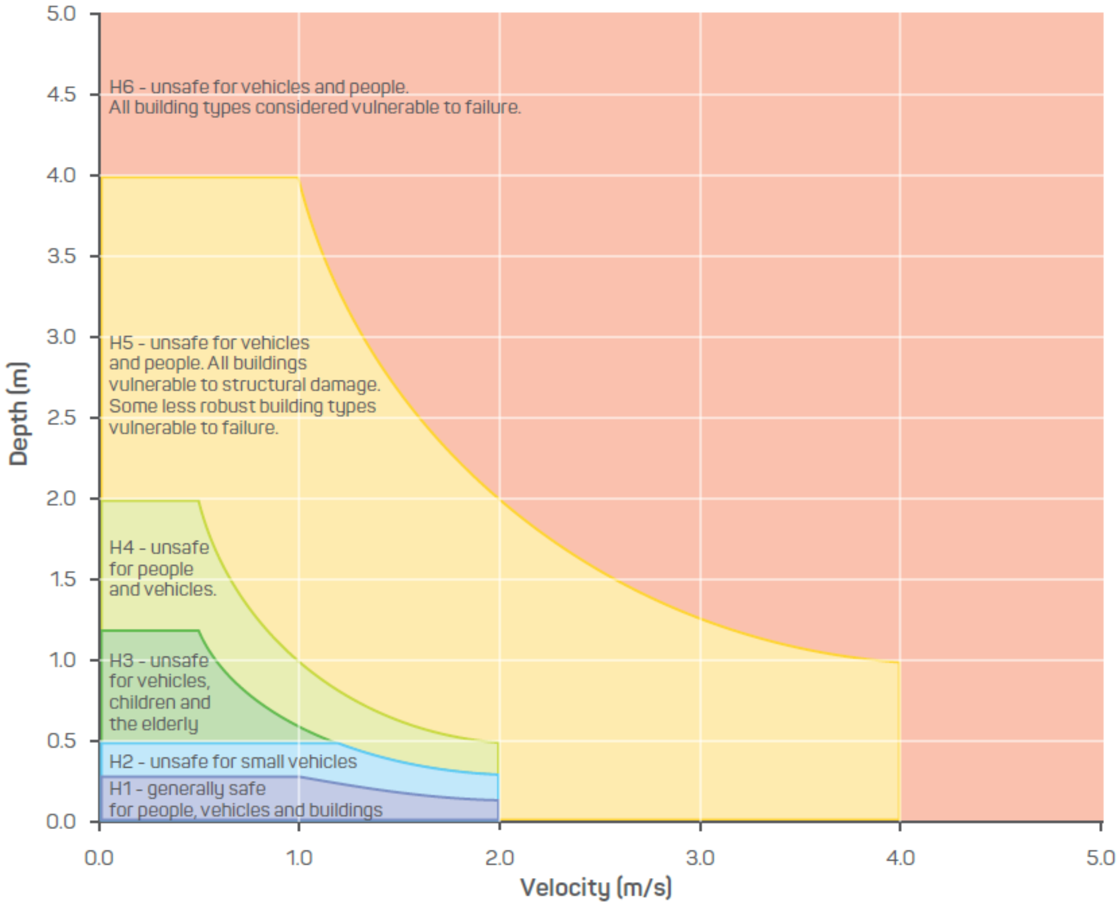
iv. Floodwater which originally commences to pond behind the CBD Levee at the two abovementioned locations effectively becomes a single body of water during storms more intense than about 100 year ARI, with only a minor head difference between the two water levels either side of the centreline of Brisbane Street north of its intersection with Peel Street during a 100 year ARI storm event.

v. Depths of inundation would exceed 1 m in a large number of commercial properties that are located to the west of Brisbane Street during a 100 year ARI storm event.

6.3 Flood Hazard and Hydraulic Categorisation

6.3.1. Flood Hazard Vulnerability Classification

Flood hazard categories may be assigned to flood affected areas in accordance with the definitions contained in the publication entitled "*Managing the Floodplain: A Guide to Best practice in Flood Risk Management in Australia*" (Australian Institute for Disaster Resilience (**AIDR**), 2017). Flood prone areas may be classified into six hazard categories based on the depth of inundation and flow velocity that relate to the vulnerability of the community when interacting with floodwater as shown in the following illustration which has been taken from AIDR, 2017:



Flood Hazard Vulnerability Classification diagrams for the 100 year ARI event under flood gates fully open and flood gates fully closed conditions based on the procedures set out in AIDR, 2017 are presented on **Figures 6.22** and **6.23**, respectively.

It was found that areas classified as H4 to H6 are generally limited to the inbank areas of the major drainage lines in a 100 year ARI event with the following exceptions:

i. In residential development that is located downstream of Victoria Street along Rifle Range Gully.

ii. In the rear of the townhouse development that is located at the intersection of O'Connell Street and Griffin Avenue in the Rifle Range Gully catchment.

iii. In residential development that is located on the western side of Bourke Street downstream of Raglan Street on Long Gully.

iv. In residential development that is located downstream of Raglan Street along Garrieties Gully.

Flooding behind the CBD Levee is generally a maximum of H3 under gates fully open conditions, generally increasing to a maximum of H4 under gates fully closed conditions, noting that the worst affected area is generally limited to the pondage which is located to the west of Darling Street.

6.3.2. Hydraulic Categorisation of the Floodplain

According to the NSW Government's *Floodplain Development Manual*, the floodplain may be subdivided into the following three hydraulic categories:

- Floodways;
- Flood storage; and
- Flood fringe.

Floodways are those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with obvious naturally defined channels. Floodways are the areas that, even if only partially blocked, would cause a significant re-distribution of flow, or a significant increase in flood level which may in turn adversely affect other areas. They are often, but not necessarily, areas with deeper flow or areas where higher velocities occur.

Flood storage areas are those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. If the capacity of a flood storage area is substantially reduced by, for example, the construction of levees or by landfill, flood levels in nearby areas may rise and the peak discharge downstream may be increased. Substantial reduction of the capacity of a flood storage area can also cause a significant redistribution of flood flows.

Flood fringe is the remaining area of land affected by flooding, after floodway and flood storage areas have been defined. Development in flood fringe areas would not have any significant effect on the pattern of flood flows and/or flood levels.

Floodplain Risk Management Guideline No. 2 Floodway Definition, offers guidance in relation to two alternative procedures for identifying floodways. They are:

- **Approach A**. Using a *qualitative approach* which is based on the judgement of an experienced hydraulic engineer. In assessing whether or not the area under consideration was a floodway, the qualitative approach would need to consider; whether obstruction would divert water to other existing flow paths; or would have a significant impact on upstream flood levels during major flood events; or would adversely re-direct flows towards existing development.
- **Approach B**. Using the hydraulic model, in this case TUFLOW, to define the floodway based on *quantitative experiments* where flows are restricted or the conveyance capacity of the flow path reduced, until there was a significant effect on upstream flood levels and/or a diversion of flows to existing or new flow paths.

One quantitative experimental procedure commonly used is to progressively encroach across either floodplain towards the channel until the designated flood level has increased by a significant amount (for example 0.1 m) above the existing (un-encroached) flood levels. This indicates the limits of the hydraulic floodway since any further encroachment will intrude into that part of the floodplain necessary for the free flow of flood waters – that is, into the floodway.

The _quantitative assessment_ associated with **Approach B** is technically difficult to implement. Restricting the flow to achieve the 0.1 m increase in flood levels can result in contradictory results, especially in unsteady flow modelling, with the restriction actually causing reductions in computed levels in some areas due to changes in the distribution of flows along the main drainage line. Accordingly the _qualitative approach_ associated with **Approach A** was adopted, together with consideration of the portion of the floodplain which conveys approximately 80% of the total flow and also the findings of _Howells et al, 2004_ who defined the floodway based on velocity of flow and depth. Howells et al suggested the following criteria for defining those areas which operate as a "floodway" in a 1% AEP event:

- Velocity x Depth greater than 0.15[8] m^2/s **and** Velocity greater than 0.25 m/s; or
- Velocity greater than 1 m/s.

Flood storage areas are identified as those areas which do not operate as floodways in a 100 year ARI event but where the depth of inundation exceeds 400 mm. The remainder of the flood affected area was classified as flood fringe.

Figures 6.24 and **6.25** show the division of the floodplain into floodway, flood storage and flood fringe areas for the 100 year ARI event under flood gates fully open and flood gates fully closed conditions, respectively.

Floodway areas are generally confined to the inbank area of the various watercourses which drain the study area, as well as the network of roads, with the following exceptions:

- In residential and commercial development that is located downstream of Victoria Street along Rifle Range Gully as far south as Viaduct Park.
- In residential development that is located on the western side of Bourke Street downstream of Raglan Street on Long Gully.
- In residential development that is bounded by Napier Street to the south, Brisbane Street to the west and Fitzroy Street to the east in the catchment which discharges to the Peel River via the Fitzroy Street Pressure Line.
- In residential development that is bounded by Carthage Street to the south, White Street to the west and Murray Street to the east in the Garrieties Gully catchment.
- In residential development that is located both to the north and south of the Main Northern Railway in the catchment which drains to the Peel River in the vicinity of Haynes Street.
- In residential development that is located both to the north and south of the Main Northern Railway in the catchments which drain to the Peel River east of Prentice Avenue.

[8] While a Velocity x Depth product of 0.25 is recommended as part of _Howells et al, 2004_, it was found that a Velocity x Depth product of 0.15 was more suitable for defining floodways in the study area.

While the extent of floodway areas does not change under gates fully closed condition, the extent of the area design as flood storage does increase significantly due to the ponding of local catchment runoff behind the CBD Levee.

6.4 Climate Change Considerations

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.

For the purposes of the present 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).

Figure 6.26 and **6.27** show the impact an increase of 10 per cent in 100 year ARI rainfall intensities would have on the extent of inundation for the cases where the flood gates are in their fully open and fully closed positions, respectively.

The increase in the extent of inundation attributable to a 10% increase in 100 year ARI rainfall intensities under both gates fully open and gates fully closed conditions would be relatively minor across the study area, with the exception of the Unnamed Gully and Spring Creek catchments where depths of overland flow of greater than 100 mm would be experienced in areas remote from the main flow paths.

6.5 Sensitivity of Flood Behaviour to Increase in Hydraulic Roughness

Figures 6.28 and **6.29** show the difference in peak flood levels (i.e. the “afflux”) for the 100 year ARI flood event resulting from an assumed 20% increase in hydraulic roughness (compared to the values given in **Table 4.2**) under gates fully open and gates fully closed conditions, respectively.

Increases in peak flood level in the channel reaches of the trunk drainage system are generally in the range 50 to 200 mm, while increases of between 10 to 50 mm are shown to occur in areas affected by overland flow.

The increase in hydraulic roughness in the upper reaches of the study area has an attenuating effect on flow in the trunk drainage lines, resulting in a minor reduction in the peak flood level in the ponding area behind the CBD Levee.

6.6 Flood Planning Level

Pending the completion of a future *Floodplain Risk Management Study and Plan* for Tamworth, an interim Flood Planning Level (FPL) was derived for the study area based on the following set of criteria:

i. peak 100 year flood level plus a freeboard allowance of 500 mm along the channelised reaches of the drainage system;

ii. peak 100 year flood level plus a freeboard allowance of 500 mm in areas where the surcharge of the enclosed reaches of the trunk drainage system will cause major flooding and where the flow is relatively confined;

iii. individual allotments that are located adjacent to the trunk drainage lines (both channelised and enclosed) of Unnamed Gully, Spring Gully and Rifle Range Creek where they run through the urbanised parts of North Tamworth; and

iv. areas affected by major overland flow where the depth of inundation exceeds 100 mm.

The criteria set out under i), ii) and iii) above were used to derive the extent of the "Interim Main Stream Flooding Flood Planning Area", while the criterion set out under iv) was used to derive the "Interim Major Overland Flow Flood Planning Area". **Figures 6.30** shows the extent of the Interim Main Stream Flooding and Major Overland Flow Flood Planning Areas (FPAs) for the flood gates in their fully closed positions.

Based on the findings of the present investigation, it is recommended that Council adopt the following minimum floor level requirements for future development in East and North Tamworth pending the completion of the Floodplain Risk Management Study and Plan:

- peak 100 year ARI flood level plus a freeboard allowance of 500 mm in areas defined as Interim Main Stream Flooding Flood Planning Area on **Figure 6.30**.
- peak 100 year ARI flood level plus a freeboard allowance of 300 mm in areas defined as Interim Major Overland Flow Flood Planning Area on **Figure 6.30**.

Note that whilst the interim FPL 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. It is therefore recommended that consideration be given to the impact a potential blockage of the existing stormwater drainage system will have on flood behaviour in East and North Tamworth prior to adopting a final FPL.

7 POTENTIAL FLOOD MODIFICATION MEASURES

7.1 Previous Investigations

Lyall & Associates, 2012 assessed the benefits which could be achieved through the implementation of the following individual flood modification measures which were aimed at reducing the severity of local catchment flooding behind the CBD Levee:

- **Option 1 – O'Connell Street Pressure Line Upgrade**. The works comprising this option include 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 in **Appendix B** which is taken from Lyall & Associates, 2012. 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 corner 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.
- **Option 2 – Fitzroy Street Pressure Line Upgrade**. This option involves the duplication of the existing 1500 mm diameter pressure line which runs along Fitzroy Street from the Main Northern Railway to the Peel River. The works would also involve improvements to the inlet arrangement located on the northern side of the Main Northern 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** in **Appendix B** which is taken from Lyall & Associates, 2012 shows the route of the new pressure line and the works which would be required to improve the inlet conditions.
- **Option 3 – White Street Pressure Line Upgrade**. This option involves the duplication of the existing 1200 mm diameter pressure line which runs along White Street from the Main Northern Railway to the Peel River. **Figure 7.4** in **Appendix B** shows the route of the new pressure line. Unlike the Fitzroy Street pressure line option, it would not be necessary to install a new section of the pipe in the levee bank, as site inspection and survey shows the presence of a redundant section of 1200 mm diameter pipe at this location. 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 Northern Railway Line.
- **Option 4 – Jaycees Park and Prince of Wales Park Trunk Drainage Upgrade**. This option involves improvements to the existing trunk drainage line which runs through the two aforementioned parks, as shown on **Figure 7.9** in **Appendix B** which is taken from Lyall & Associates, 2012. 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.
- **Option 5** comprises **Options 1** to **4** combined.
- **Option 6** comprises **Options 1** and **2** combined.
- **Option 7**: This option involves the installation of three low head/high volume flood evacuation pump stations, the locations of which are shown on **Figure 7.16** in **Appendix B** which is taken from Lyall & Associates, 2012. An assessment was undertaken assuming each pump station had a lift capacity of 3 m^3/s (denoted **Option 7A**) and 6 m^3/s (denoted **Option 7B**).

Lyall & Associates, 2012 assessed the benefits of implementing the individual flood modification measures as follows:

- **Option 1 -** This option would alleviate flooding in several commercial properties located between Brisbane Street and Darling Street for the case where the flood gates are 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 -** 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.
- **Option 3 -** 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 100-200 mm.
- **Option 4 -** 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 -** 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 -** The implementation of these two measures would 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 7A -** The installation of three pump stations with a combined lifting capacity of 9 m^3/s would reduce peak flood level behind the CBD Levee by a maximum of 160 mm in the vicinity of Bourke Street for the case where the flood gates are in their fully open position, and up to 640 mm for the case where the flood gates are in their fully closed position.
- **Option 7B -** The installation of three pump stations with double the lifting capacity than **Option 7A** (i.e. 18 m^3/s compared to 9 m^3/s) would generally limit flooding resulting from stormwater ponding behind the levee to commercial properties located along Peel Street between Darling Street and Bligh Street and along Kable Avenue between Fitzroy Street and Brisbane Street for the case where the flood gates are in their fully open position. Whilst the implementation of the **Option 7B** pump stations would reduce peak 100 year ARI flood levels directly behind the levee by up to 940 mm for the case where the flood gates are in their fully closed position, the number of commercial properties which would be rendered flood free is limited to two or three.

7.2 Present Investigation

The key findings of the present investigation in terms of the assessed distribution of flow discharging overland to the pondage behind the CBD Levee and also the effectiveness of the previously assessed options are as follows:

i. There is a significant reduction in the rate at which flow surcharges the existing trunk drainage system at Surcharge Location Identifiers S02, S03, S07, S09 and S10 when compared to the findings of Lyall & Associates, 2012 (refer peak flow comparison in **Table 6.2**).

ii. There is generally an increase in the rate at which flow surcharges the existing trunk drainage system at Surcharge Location Identifiers S01, S04 and S08 when compared to the findings of Lyall & Associates, 2012 (refer peak flow comparison in **Table 6.2**).

iii. Based on the above findings, **Option 1** should be modified to comprise the following:

    a. The upgrade of the O'Connell Street Pressure Line from its inlet at Marius Street on Rifle Range Gully to its point of discharge to the Peel River. This could comprise a new drainage line within Viaduct Park running parallel with O'Connell Street.

    b. The removal of the temporary detention system and major intake structure that was proposed in the vicinity of Angora Park, for the reason that the present investigation has demonstrated that major overland flow bypasses this location during storms which result in the surcharge of the existing trunk drainage system.

iv. Consideration should be given to constructing a new pressure line which extends from the trapped low point in the Main Northern Railway which is located between Darling Street and Brisbane Street to the Peel River via Bourke Street. This is because the majority of flow which surcharges the enclosed reach of Long Gully and discharges to the pondage behind the CBD Levee discharges to this location. There would also be merit in extending the new trunk drainage line north as far as Raglan Street as this would assist in removing the damaging flooding that is experienced in existing residential development that is located along the western side of Bourke Street.

v. **Option 3** should be given higher priority to **Option 2**, as the majority of flow which contributes to ponding behind the CBD Levee originates from the White Street crossing of the Main Northern Railway.[9] Note that consideration should be given to increasing the waterway area of the upgraded drainage line given the higher flows that the present study has identified discharging to this location.

vi. **Option 4** is no longer feasible following the construction of the Eastpoint shopping complex and commercial development in the vicinity of the proposed works.

vii. **Option 5** should be modified to comprise **Option 1** as described above and **Option 3**.

viii. **Option 6** should be abandoned.

ix. While **Options 7A** and **7B** have merit, they rely of flow first discharging to the pondage before it can be pumped to the Peel River. As a result, damages would still be experienced be existing development which is currently impacted by flow which surcharges the various trunk drainage lines.

[9] In addition to the flow which discharges south of the rail corridor via White Street, flow also discharges west along the rail corridor toward the inlet of the Fitzroy Street Pressure Line.

8 REFERENCES

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DECC (Department of Environment and Climate Change), 2007. ***"Practical Consideration of Climate Change"***. Draft Floodplain Risk Management Guideline.

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Lyall & Associates, 2012. ***"Tamworth City Levees Internal Drainage Study"***

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NSWG (New South Wales Government), 2005. ***"Floodplain Development Manual – The Management of Flood Liable Land"***.

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

APPENDIX A

HISTORIC FLOOD LEVEL AND RAINFALL DATA

TABLE A1
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. Source: Lyall & Associates, 2012
2. Gauge zero on the Peel River at Tamworth stream gauge (GS 419009) is 371.057 m AHD.
3. 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 A2
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: Lyall & Associates, 2012.

TABLE A3
APPROXIMATE ARI OF RAINFALL RECORDED AROUND TIME OF HISTORIC FLOODS AT TAMWORTH[1,2]

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

Refer footnotes over

TABLE A3 (Cont'd)
APPROXIMATE ARI OF RAINFALL RECORDED AROUND TIME OF HISTORIC FLOODS AT TAMWORTH[1,2]

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. Source: Lyall & Associates, 2012
2. 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.