Regional Floodplain Database:

Floodplain Parameterisation

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Moreton Bay Regional Council Regional Floodplain Database

FLOODPLAIN PARAMETERISATION

- Final – Revision 4
- 2 October 2012

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Contents

Tables

Figures

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1. Introduction

1.1. Study Details

Sinclair Knight Merz Pty Ltd (SKM) has been commissioned by Moreton Bay Regional Council (MBRC) to carry out an investigation into appropriate standard flood model parameters to be adopted for use in Council's Regional Floodplain Database Project (RFD Project).

The RFD Project involves a three year (three stage) program for the development of comprehensive flood mapping across the Moreton Bay Regional Council Local Government Area. A key focus for the project is the standardisation of methods and procedures so as to ensure consistency in the flood information produced. The Burpengary 'Minor Basin', incorporating Burpengary Creek, Little Burpengary Creek and Deception Bay has been selected as the Stage 1 pilot study catchment for development of these standardised methods and procedures.

This report documents the development of standard flood model parameters for the Burpengary Minor Basin. Following test application Council will consider extension of the procedures documented herein for Stage 2 of the project which will include detailed flood modelling and mapping for the region.

1.2. Background

Moreton Bay Regional Council (MBRC) was formed by the amalgamation of Caboolture Shire, Redcliffe City and Pine Rivers Shire Councils (total area of 2,070 km^2). The Moreton Bay 'Regional Floodplain Database' Project aims to comprehensively map the floodplains of the new combined region.

The key goals of the Moreton Bay 'Regional Floodplain Database' are:

- a comprehensive description of flood behaviour across the region;
- strategies for management of any flooding problems identified; and
- a system/process to store and manage this information and keep it up-to-date.

The aim of the overall project is to have a consistent and standardised approach to the hydrological and hydraulic modelling used in to determine flood behaviour in across the region. The important benefits of standardisation of flood modelling are:

- regional data consistency;
- consistency of interaction between data storage and data analysis tools;
- facilitate targeted data capture that relates specifically to the models being employed;
- enhanced understanding of changes in model behaviour due to changes in their underlying parameters, allowing Council to over time develop a more robust and accurate parameter set;
- provide an opportunity for Council to develop a stronger understanding of the modelling tools being used by their consultants (difficult when a large number of different modelling packages

are being used). This will enable a more thorough and critical assessment of the methodologies being employed; and

- achieve economies of scale when researching / deriving new approaches.

1.3. Scope

This sub-project involves the investigation and delivery of advice to support the preparation of flood models including but not limited to:

- The specification of an appropriate range of hydrologic model parameters (excluding design rainfall and infiltration loss). Including catchment lag, stream lag, impervious lag parameters
- The specification of an appropriate range of hydraulic model parameters. For example, manning's 'n', structure entry and exit loss, viscosity, wetting and drying parameters

These specifications will be provided to the sub-project team involved in the development of detailed hydrologic and hydraulic models to provide a well researched and documented understanding of the most appropriate model parameter set for our region (within which calibrated model parameters should lie).

1.4. Report Structure

This report has been developed to include the standard parameterisation of the floodplain for flooding assessment (**Sections 3** to **7**). The concept of potential additional risk of flooding due to blockage of various types of structures in the floodplain has also been addressed (**Section 8**).

It is recommended to include the recommended blockage factors into the hydraulic modelling to determine the additional risk of flooding envelope over and above the standard flooding assessment. The assessment of blockage may also be considered with other potential risks to changes in the flooding regime, for example climate change, to give an overall envelope of additional risk of flooding.

It is expected that the standard flooding assessment will predicted the highest flood levels for the downstream portions of the catchment and the application of blockage factors will predicted the highest flood levels in the upstream portion of the catchment.

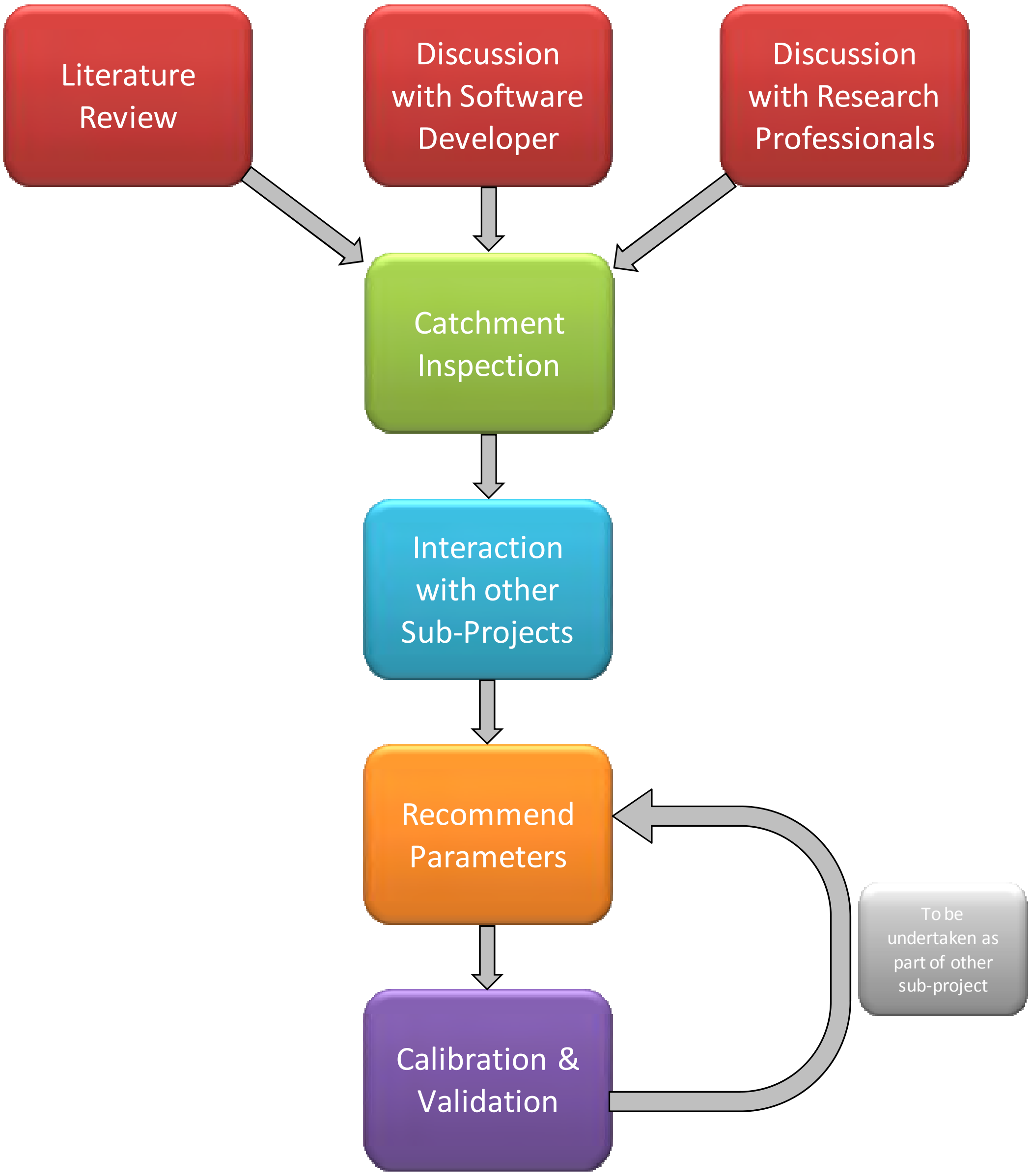
This report discusses the sub-project 2N Floodplain Parameterisation and has the following sections:

- **Section 2** – Methodology;
- **Section 3** – WBNM Parameters.
- **Section 4** – TUFLOW Parameters.
- **Section 5** – Manning's n Values;
- **Section 6** – Structure Modelling;
- **Section 7** – Buildings;

- **Section 8** – Blockage; and
- **Section 9** – References.

2. Methodology

The methodology adopted for the development of this sub-project 2N has a number of components with the aim of developing floodplain parameters which can be easily used and applicable to the other sub projects over the overall MBRC Floodplain Database Development project. The methodology is presented in **Figure 2-1**. This report also resents recommendations for a refinement and validation process for the recommended parameters.



- **Figure 2-1 Methodology for Sub-Project 2N**

3. WBNM Parameters

3.1. Parameters Assessed

Watershed Bounded Network Model (WBNM) has been selected by MBRC to be the standard hydrologic model used for flood study assessment in the region. WBNM has been developed to represent the catchment as it transforms rainfall to runoff. One of the key parameters within the model is the lag parameter.

The lag parameter is representation of the average travel time for runoff from the catchment surface. The lag parameter is used in WBNM to develop the following factors within the model:

- Overflow lag:
- Streamflow lag; and
- Impervious lag.

3.2. Lag Parameter

The lag parameter has been derived for historical floods across Australia in *Boyd and Bodhinayake (2006)*. The lag parameters have been developed from 584 storms in 54 catchments across Queensland, NSW, Victoria and South Australia. There was some variation across the states for the lag parameter. However, an average parameter of 1.6 has been developed.

It is recommended that the average value of 1.6 to be adopted in line with the recommendations of *Boyd and Bodhinayake (2006)* for ungauged catchments.

3.3. Stream Lag Factor

The stream lag factor is the average travel time for runoff in the stream or channel. As flow in streams and channels travel faster than overland flow the lag factor is reduced to account for this. The stream lag factor is also adjusted based on the type of stream or channel. The stream lag factor is further reduced if the channel has been modified from a natural channel. The stream lag factor are summarised in **Table 3-1**.

It should be noted the WBNM model applies a factor of 0.6 to reduce the stream lag time. This is automatically built into the model, which is based on stream lag factor for a natural channel compared to the overland flow lag. Therefore, the natural channel has a stream lag factor of 1.0.

It should also be noted that the hydraulic models for the MBRC RFD project will extend well up into most catchments. It is expected that the majority of WBNM sub-catchments will be local catchment inflows directly input into the hydraulic models. Hence, there will be limited influence of the stream lag parameter.

3.4. Impervious Lag Factor

The conversion of rainfall to runoff on the impervious surface uses the Impervious Lag factor to allow for faster flow velocities on these surfaces compared to overland flow. The impervious lag factor is recommended in *Boyd and Bodhinayake (2006)* is 0.1.

3.5. m Value

The 'm' value in the WBNM model is a representation of the non-linearity of the catchment lag time in relation to the discharges. As outlined in *Askew (1970)*, an 'm' value of 0.76 to 0.77 and found that this do not vary for different catchments. Therefore, an 'm' value of 0.77 recommended in that paper and also recommended for adoption in this project.

3.6. Recommended Parameters

A summary of the recommended WBNM model parameters are presented in **Table 3-1**.

- **Table 3-1 Recommended Parameters WBNM Model**

Description	Value
Lag Parameter	1.6
Impervious Lag Factor	0.1
m value	0.77
Stream Lag Factor	
a) Natural channel	1.0
b) Gravel bed with rip-rap	0.67
c) Excavated earth	0.50
d) Concrete lined	0.33

4. TUFLOW Parameters

Discussions were held with Bill Syme regarding the application of TUFLOW to the modelling required for the Regional Floodplain Database. The default parameters were seen to be applicable in all cases except the following, which are discussed below.

Bed Resistance Cell Sides == AVERAGE N

The default option for this command is “INTERROGATE”. However, it is likely that the material values will only be allocated to cell centres. This will require the model to calculate the bed resistance at the cell sides based on these cell centre values.

The “Average N” option provides a more realistic representation of the average bed resistance value of neighbouring cells than the “Average M” option.

The only other option that is not strictly standard is the use of n values that vary with depth. This is discussed in more detail in Section 5.

5. Manning's n Values

5.1. Approach to Defining Manning's n Values

The Manning's n value is used with the hydraulic model, TUFLOW, to determine the hydraulic resistance to flows for various areas with the channel and the floodplain. The Manning's n values are an important part of a flooding assessment and are used to characterise the different land use types encountered.

The Manning's n values recommended as part of this study were developed from review of the following background studies and investigations:

- *Natural Channel Design Guidelines (Brisbane City Council, 2000)*;
- previous flood studies undertaken in the region;
- Values of Manning's "n" for various degrees of vegetal retardance, RS 15326 A4 (Queensland Water Resources Commission); and
- catchment inspections undertaken as part of this project.

These sources of information have been summarised in the following sub-sections and then consolidated to provide a simplified 'shortlist' of Manning's "n" values corresponding to the simplified landuse breakdown that has been concurrently prepared by SKM as part of a separate RFD project related to floodplain landuse (refer Sub-project 1H).

5.2. Natural Channel Design Guidelines (Brisbane City Council, 2000)

The *Natural Channel Design Guidelines (Brisbane City Council, 2000)* provide details of recommended Manning's n values to be used in 1-dimensional hydraulic modelling for various stream and vegetation types within the area. The guidelines provides a number of tables with Manning's n values for difference stream types, the guidelines also provide a series of description and photographs to explain and illustrated the selected parameters. Manning's n values are given for both the channel and the floodplain.

Extracts of the *Natural Channel Design Guidelines (Brisbane City Council, 2000)* have been included in **Appendix A**.

5.3. Previous Flood Studies

The Moreton Bay Regional Council has a library of previous flood studies that have been undertaken for the various creek and rivers in the region. A review of the flood study library was undertaken as part of this project. A full list outlining the flood studies reviewed and the parameters considered is presented in Appendix B.

There have been a number of different hydrologic and hydraulic modelling approaches used for the flood studies undertaken in the region. There has been limited amount of well-calibrated studies completed in the region. The majority of the studies undertaken to date have included 1-dimensional hydraulic modelling. The Manning's n values have varied from assessment to assessment and are based on industry standard parameters.

It was not possible to draw any firm directions from the review of these studies that would constitute a regional approach as distinct from industry standard parameters.

5.4. Water Resources Commission Curves for Grass

The Queensland Water Resources Commission developed a series of Manning's n values for different types of grasses. A graph has been produced which shows the variation of Manning's n values due to the type of vegetation (which includes the average vegetation length) and the product of the velocity and the hydraulic radius. A reproduction of the original graph is presented in **Figure 5-1**. It should be noted that the graph has not been altered from the original.

Hydraulic testing was also undertaken to develop this guidance for the selection of Manning's n values for various grass types.

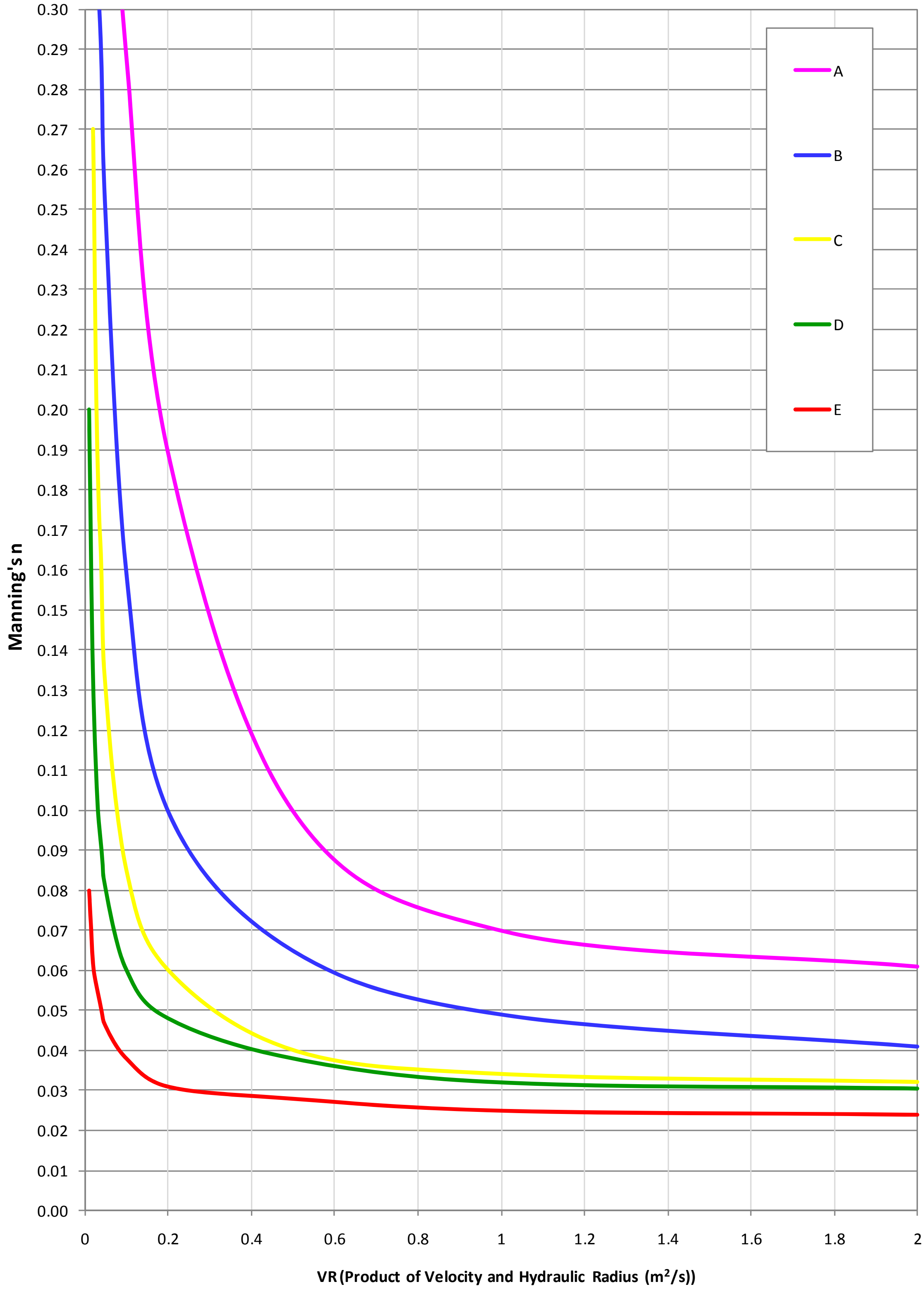


Figure 5-1 Manning's n Values for Grasses (Queensland Water Resources Commission)

Each series on the graph, denoted with a letter, represents a degree of retardance. An explanation of this degree of retardance is presented in **Table 5-1**.

Table 5-1 Vegetation Retardance Description

Stand[1]	Average Length of Vegetation (mm)	Degree of Retardance	Examples
Good	Longer than 750	A	Rhodes grass in ungrazed scrub soil waterway
Good	280 – 600	B	- Wheat[2] 650 tall in 180 rows. Rhodes grass; - Kikuyu[2] under maximum fertility conditions long and green; - African star grass; and - Lucerne[2].
Good	150 – 200	C	Most grasses can be held at this retardance with the mowing or grazing. Eg Rhodes grass, Kikuyu, African star grass, couch grass, carpet grass, native grasses.
Good	50 – 150	D	African star grass, Kikuyu[2] or couch grass all under heavy grazing
Good	Less than 50	E	Mowed lawn. Any grass burned short.
Fair	Longer than 750	B	
Fair	280 – 600	C	Rhodes grass under low fertility conditions
Fair	150 – 200	D	African star grass under low fertility condition.
Fair	50 – 150	D	
Fair	Less than 50	E	

1 – thickness of the stand has a very important bearing on the retardance, possibly more than the species.

2 – tested in experimental channels.

Based on the above data, it is possible to derive a relationship between depth of flooding and Manning's n using the average values in **Table 5-1**.

However, deriving this relationship requires an assumption to be made regarding the VxR product. For a 2D grid cell, the VxR product is equivalent to the VxD product or the q (flow intensity) value. This is true because the hydraulic radius of the cell is equivalent to the depth as the wetted perimeter is the base width of the cell (the depth is not part of the calculated wetted perimeter).

It can be assumed that areas with a velocity of less than 1.0 are not highly influential on flooding behaviour. Flood gradients are likely to be dictated by areas with higher velocities. Hence, a velocity of 1 was chosen as the value for calculation of the depth values from the above graph (i.e. VR equates to flood depth).

It was also assumed that most of the grasses on the floodplains will be in the C range (i.e. Maintained or grazed Rhodes grass, Kikuyu, African star grass, couch grass, carpet grass, native grasses).

This relationship is presented in **Table 5-2**.

- **Table 5-2 Typical Grass Mannings n Values vs Flood Depth**

Flood Depth	Mannings n
0.00	0.250
0.20	0.060
0.40	0.045
0.80	0.035
2.00	0.032

However, TUFLOW allows only two depths per landuse roughness category for depth varying Manning's n with a linear interpolation between these two depths. Hence, a further simplified two stage depth varying roughness relationship in **Table 5-3** could be used where depth varying roughness on grassed floodplain is considered important to a description of flood behaviour. This relationship will be slightly conservative for depths greater than 0.8m, but only by 10%.

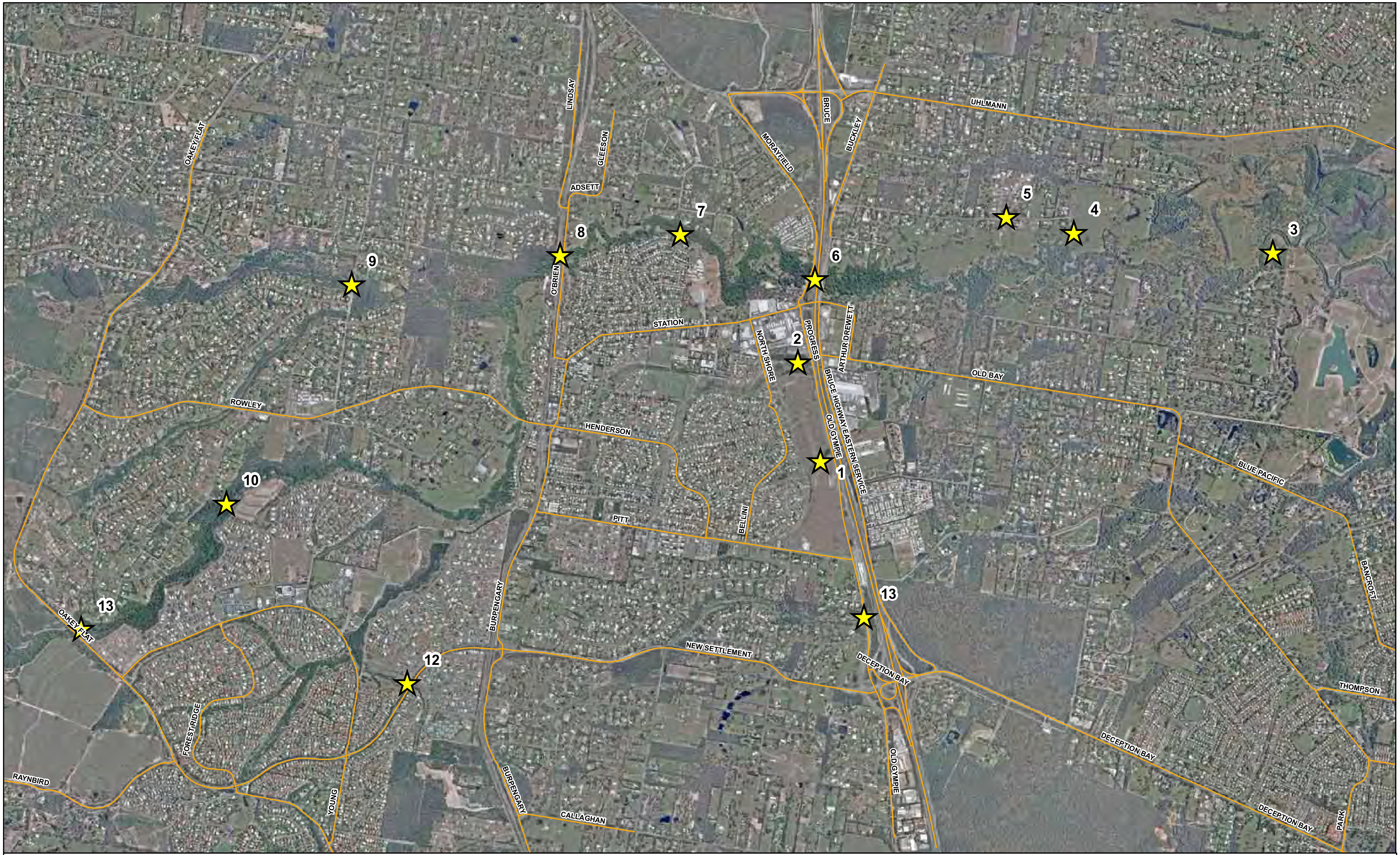
- **Table 5-3 Simplified TUFLOW Grass Mannings n Values vs Flood Depth Relationship**

TUFLOW Parameter	Flood Depth	Mannings n	TUFLOW Parameter
y_1	0.20	0.060	n_1
y_2	0.50	0.035	n_2

5.5. Catchment Inspection

A catchment inspection was undertaken to assess the differing channel and floodplain conditions for the catchment. The channel/floodplain conditions that were observed were grouped into various classifications.

The catchment inspection classification locations are shown in **Figure 5-2**. A number of photographs were taken at each location to capture to observations of the catchment inspection. The photographs taken are presented in **Appendix C**.



MBRC Floodplain Database Project

Sub-Project 2N Floodplain Parameterisation

Figure 5-2 Catchment Inspection Locations

Legend

Catchment Inspection Locations

Roads

N

0 0.25 0.5 1 1.5 2

Kilometres

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The indicative Manning's n values for each of the field classifications are presented in **Table 5-4**. This table outlines a Manning's n value for the channel and the bank (where relevant). The table also presents an average value that could be considered for use in hydraulic modelling.

Table 5-4 Manning's n Values from Catchment Inspections

Type	Description	Manning's n		
		Channel	Bank	Average
A	+2 m high dense reeds			0.080
B	1 – 1.5 m reeds in Pine/tea tree canopy	0.070	0.060	0.065
C	Slashed pasture (maintained pasture)	Refer **Table 5-1**		0.035
D	Unmaintained pasture, small trees 2 m spacings, approx. 2m tall tress with undergrowth	Refer **Table 5-1**		0.060
E	Large trees/shrubs			0.070
F	Pine forest/fern undergrowth			0.070
G	Eucalypt 8 m spacings, minimal undergrowth			0.050
H	Salt marsh			0.040
I	Tidal water course, mangrove canopy (50% to 100% canopy coverage)	0.040	0.150	0.095
J	Mowed, maintained lawn	Refer **Table 5-1**		0.025
K	Dense canopy over clear flood channel	0.050	0.100	0.075
L	Dense canopy over dense undergrowth	0.060	0.120	0.090
M	Clear channel with some snags/fallen trees, dense bank vegetation	0.060	0.120	0.090
N	Clear channel, dense bank vegetation, no snags	0.050	0.120	0.085
O	Type L with more dense vegetation	0.080	0.150	0.115
P	0.5 m vegetation sparse trees approx 5 m tall, urban creek	0.050		0.050
Q	1 – 1.5 m reeds			0.070

5.6. Pilot Study Parameters

Based on the background information described in **Sections 5.2** to **5.5**, a short-listed series of Manning's n values are recommended for use in hydraulic modelling. This shortlist corresponds to the land-use mapping developed as part of the separate floodplain land-use sub-project (Sub-Project 1H).

- **Table 5-5 Short-List of Manning's n Parameters – Floodplain and Urban**

Description	Manning's n
Dense vegetation	0.090
Swamp	0.080
Medium-dense vegetation	0.075
Crops	0.040
Low Grass/Grazing	0.035
Waterbodies	0.030
Roads/Footpaths	0.015
Buildings	1.000
Urban block	0.300

It is acknowledged that this simplified list of recommended parameters may not be sufficient to achieve calibration in some areas of the model. In some selected areas of the model a more refined definition of roughness characteristics, their spatial extent and depth varying characteristics may also be required in order to achieve calibration against historic flood level data. Therefore, the following detailed parameters are provided as an option for use where required. These have been drawn from the background sources described in Sections **5.2** to **5.5**.

- **Table 5-6 Optional Detailed Manning's n Parameters – Floodplain**

Description	Manning's n
Grass	Depth Varying See **Table 5-3**
Unmaintained pasture, small trees 2 m spacings, approx. 2m tall tress with undergrowth	0.060
Large trees/shrubs	0.070
Pine forest/fern undergrowth	0.055
Eucalypt 8 m spacings, minimal undergrowth	0.070
Salt marsh	0.040

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Table 5-7 Optional Detailed Manning's n Parameters – Riparian Vegetation

Description	Manning's n
+2 m high dense reeds	0.080
1 – 1.5 m reeds in Pine/tea tree canopy	0.065
Tidal water course, mangrove canopy (50-100 % canopy coverage)	0.095
Dense canopy over clear flood channel	0.075
Dense canopy over dense undergrowth	0.090
Clear channel with some snags/fallen trees, dense bank vegetation	0.090
Clear channel, dense bank vegetation, no snags	0.085
Dense canopy over very dense undergrowth	0.115
0.5 m vegetation sparse trees approx 5 m tall, urban creek	0.050

Table 5-8 Recommended Parameters Manning's n Parameters – Urban Areas

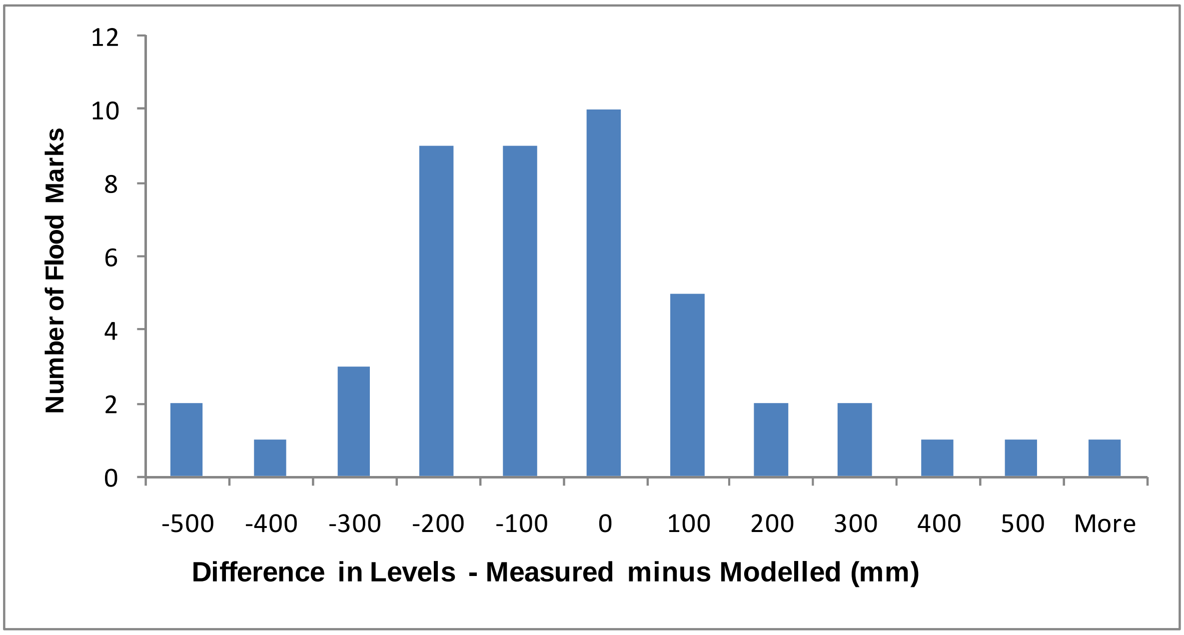
Description	Manning's n
Roads	0.015
Urban Block (excludes buildings)	0.300
Buildings (either 1 or 2)	
1) Porous + form loss = buildings represented with a blockage of 90 %, a form loss of 0.100 and a Manning's n value of 0.030	
2) Increased Manning's n value	1.000

5.7. Changes to Adopted Parameters made during Stage 2

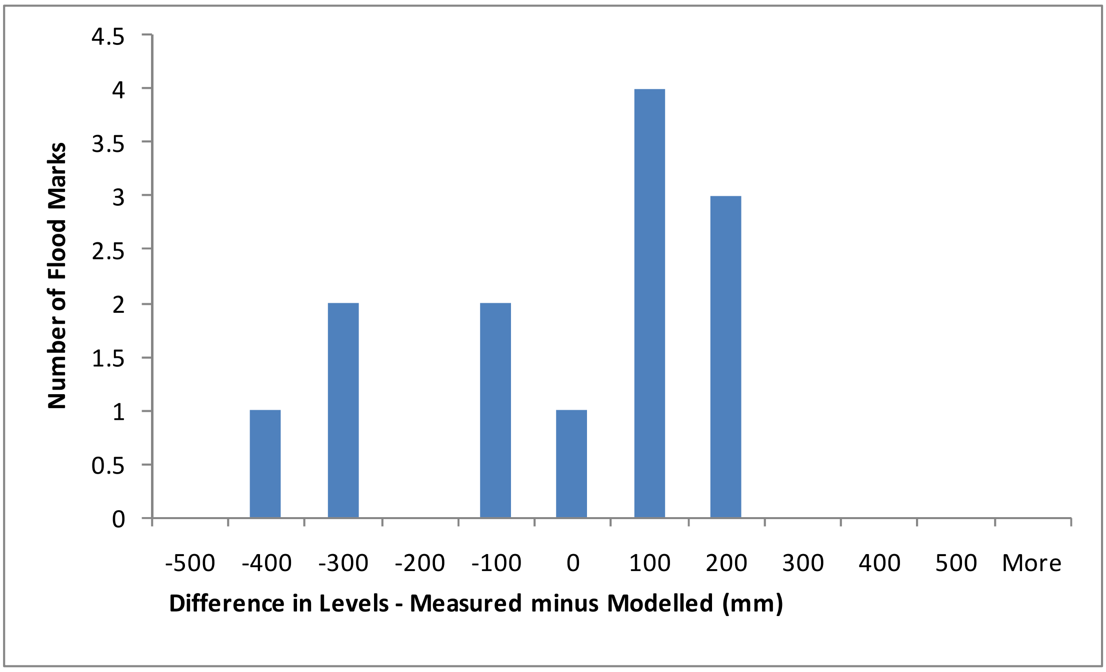
5.7.1. Calibration and Validation Outcomes from Pilot Study

The Burpengary hydraulic model was run for the May 2009 and February 1999 events with the Manning's n parameter values recommended in Section 5.6 (Table 5-5). A detailed flood survey was undertaken (sub-project 2K – Flood Information Historic Flooding, GHD, 2010) to provide a comparison between modelled and recorded flood levels.

The hydraulic model results (peak flood levels) were compared to the recorded flood levels for both the May 2009 and February 1999 events. Most of the recorded flood level marks were within ± 200mm of the modelled flood levels, which is considered a reasonable calibration. A histogram showing the number of flood marks versus the flood level difference is shown in Figure Figure 5-3 (May 2009) and Figure Figure 5-4 (February 1999).



- **Figure 5-3 May 2009 Event – Flood level difference for Burpengary Creek – Pilot Study Parameters**



- **Figure 5-4 February 1999 Event – Flood level difference for Burpengary Creek- Pilot Study Parameters**

5.7.2. Calibration and Validation Outcomes from Stage 2

Sufficient flood data was available for the January 2011 flood event in the Caboolture (CAB), Upper Pine (UPR) and Stanley (STA) Rivers minor basins to undertake further verification using the Manning's n parameter values recommended in Section 5.6 (Table 5-5). The hydraulic model results (peak flood levels) were compared to the recorded flood levels and showed some under-prediction of flood levels across all three minor basins. The Burpengary Creek (BUR) hydraulic model was also run for the January 2011 event and similar under-prediction was observed.

Following an analysis of factors that could have contributed to this under-prediction, Council chose to re-run the CAB, BUR, UPR and STA minor basin hydraulic models using the relevant depth varying Manning's n values from Tables 5-A to 5-C in order to incorporate the impact of change in vegetation density with depth, as this was determined to be the most likely contributor to the under-predictions.

The latest version of TUFLOW allows for more than two depths per landuse roughness category allowing the use of the depth varying relationships for 'Low Grass/Grazing', 'Medium-dense vegetation' and 'Dense vegetation' as shown in Tables Table 5-9 to Table 5-11 below.

- **Table 5-9 Depth varying Manning's n - Low Grass/Grazing**

Flood Depth	Manning's n
0.00	0.250
0.20	0.060
0.40	0.045
0.80	0.035
2.00	0.025

- **Table 5-10 Depth varying Manning's n – Medium-dense Vegetation**

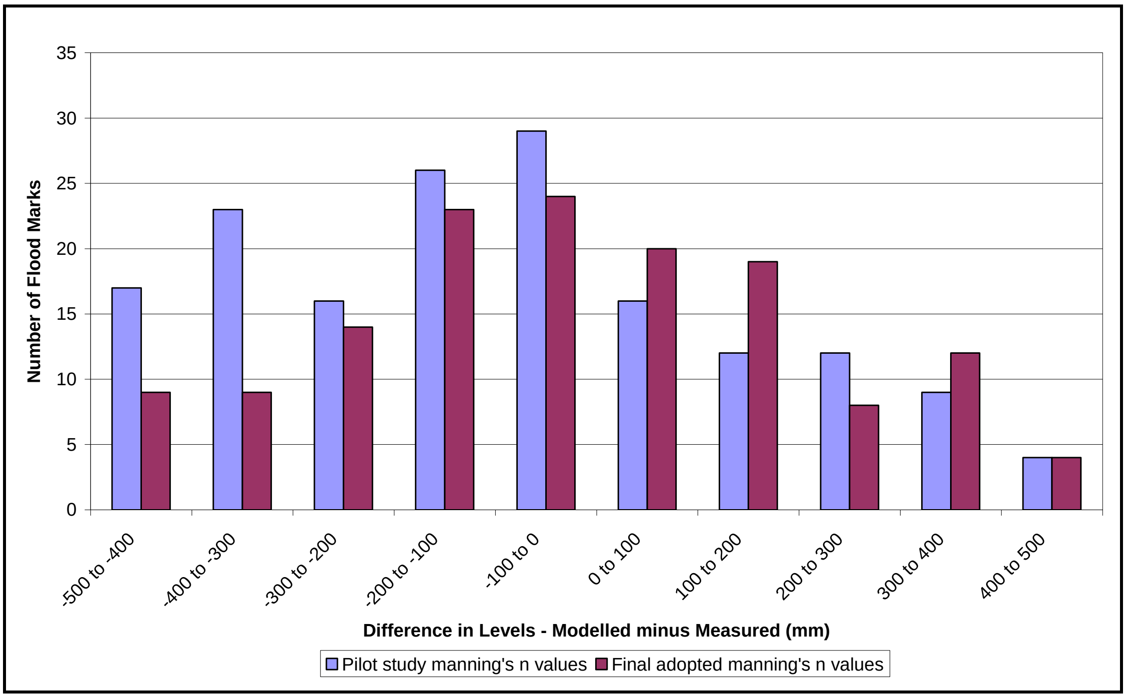
Flood Depth	Manning's n
0.00	0.075
1.50	0.075
3.50	0.15

- **Table 5-11 Depth varying Manning's n – Dense Vegetation**

Flood Depth	Manning's n
0.00	0.09
1.50	0.09
3.50	0.18

Using the parameters as detailed above, the hydraulic model results (peak flood levels) from the re-runs were again compared with the recorded flood levels. This showed an almost equal number of locations under or over-predicting the flood levels across the four minor basins. These updated model results are considered to provide a 'best fit' of the modelled versus recorded flood levels across the region. 60% of the recorded flood level marks were within ±200mm of the modelled flood levels and 76% were within ±300mm of the modelled flood levels.

A histogram showing the number of flood marks versus the flood level difference is shown in Figure 5-5 for the January event for the CAB, BUR, UPR and STA minor basins before and after the change in Manning's n values.



- **Figure 5-5 January 2011 event Flood level difference – Burpengary Creek, Caboolture, Upper Pine and Stanley Rivers minor basins - Stage 2 parameters**

The final adopted Manning's n parameters are provided in Table 5-12 below.

Table 5-12 Adopted Manning's n Parameters – Floodplain and Urban

Description	Manning's n
Dense vegetation	See Table 5-11
Reeds	0.080
Medium-dense vegetation	See Table 5-10
Crops	0.040
Low Grass/Grazing	See Table 5-9
Waterbodies	0.030
Roads/Footpaths	0.015
Buildings	1.000
Urban block	0.300

6. Structure Modelling

6.1. Culverts

6.1.1. Outlet Control Hydraulic Losses in Culverts

The three main losses to be simulated in culverts, flowing under outlet control conditions, are:

- inlet losses;
- outlet losses;
- friction losses.

The losses discussed in this section focus on inlet and outlet losses as friction losses are modelled implicitly in the hydraulic model. The losses are presented as multipliers of the velocity head within the structure.

Inlet losses are documented in Figure 7.17 of Waterway Design (AustRoads, 1994). For box culverts, the relevant values for culverts in MBRC are summarised as follows:

- square edges with wingwalls at 90^o to 75^o to barrel (i.e. headwall only) = 0.5
- square edges with wingwalls at 30^o to 75^o to barrel = 0.4
- square edges with wingwalls at 10^o to 25^o to barrel = 0.5
- square edges with wingwalls at 0^o to barrel (i.e. extension of sides) = 0.7
- any wingwall with tapered edges = 0.2

The relevant outlet control values for simulating circular culverts in MBRC are summarised as follows:

- square edges with wingwalls = 0.5
- rounded edges with wingwalls = 0.2

For pipe-arch or corrugated steel arch structures, the relevant values for culverts in MBRC are summarised as follows:

- projecting from fill = 0.9
- any headwall with square edges = 0.5
- mitred to conform to fill slope = 0.7
- end-section conforming to fill slope = 0.5

Outlet losses are usually assumed to be 1.0 $v^2/2g$. However, this is based on an assumption that the floodplain velocity into which the culvert discharges is significantly smaller than the culvert velocity. The over-estimation of outlet losses in 1D modelling is discussed further below in **Section 6.4**.

6.1.2. Inlet Control Hydraulic Losses in Culverts

The loss factors for inlet control in culverts are represented in TUFLOW as height and width contraction coefficients for orifice flow at the inlet. The recommended values for box culverts are:

Height Contraction Coefficient:

- 0.6 for square edged entrances
- 0.8 for rounded edged entrances

Width Contraction Coefficient:

- 0.9 for sharp edged entrances
- 1.0 for rounded edged entrances

For circular culverts, a 'width' contraction coefficient of 1.0 is recommended.

6.2. Bridges

6.2.1. Proposed Modelling Approach for Contraction and Expansion

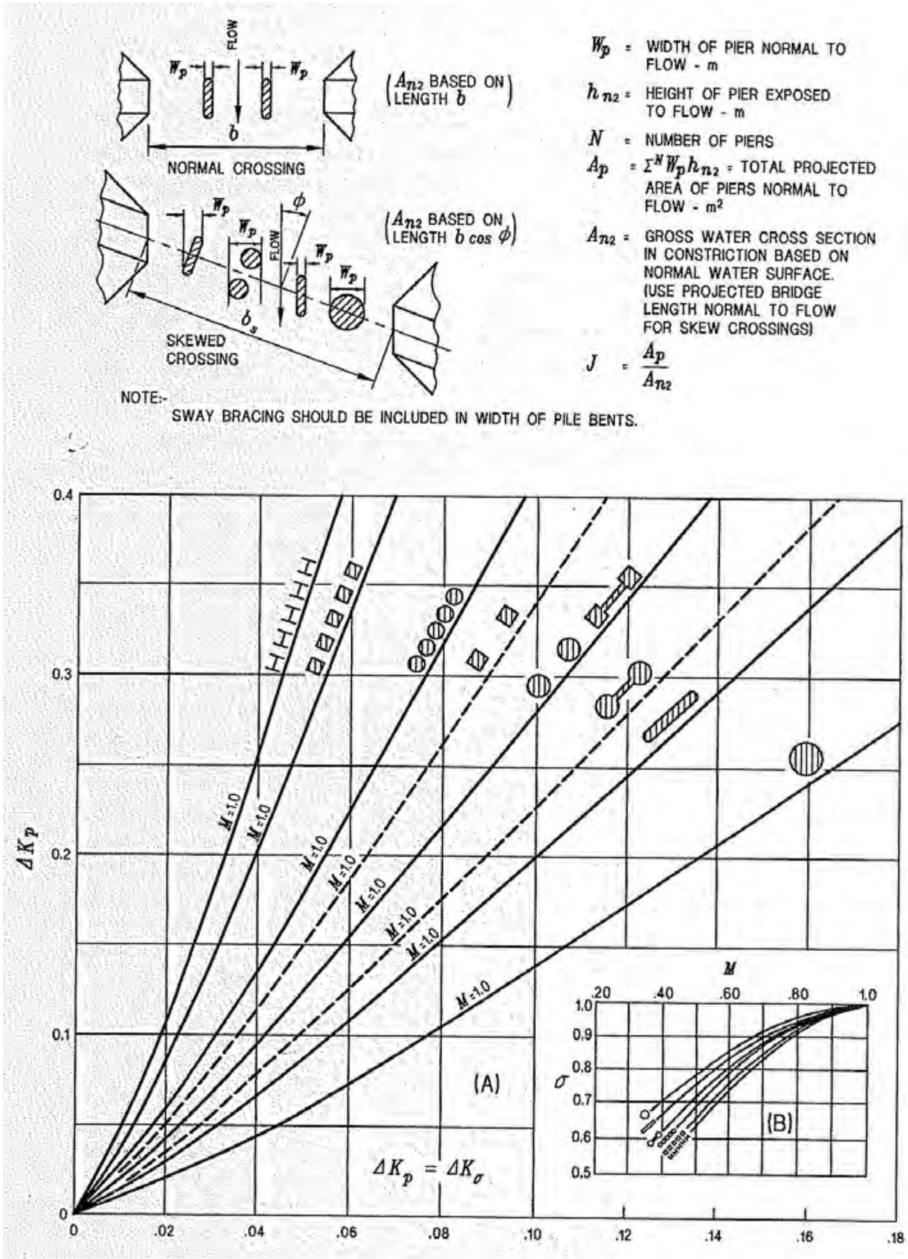
It is expected that bridges modelled in the TUFLOW model will simulated either as 1D structures or sets of 2D FC cells (or FC shape file).

Chapter 5 of Waterway Design (AustRoads 1994) presents an approach to calculating afflux across a bridge structure. This approach is based on calculation of a bridge opening ratio M. In Barton (2001), it was identified that the losses associated with the curves presented in AustRoads (1994) are very high compared to other methods. Furthermore, Barton (2001) identified that the representation of the contraction and expansion of flow in the 2D domain represents a large proportion of the expected losses across a bridge structure (see **Section 6.4** below)

Hence, minor additional loss coefficients in the order of 0.1 to 0.3 will be required to fully represent the losses associated with contraction and expansion of the flow into and out of the bridge structure in the 2D domain.

6.2.2. Pier Losses

A proposed approach is to represent the pier losses using the techniques presented in Waterway Design (AustRoads 1994). Figure 5.7 from this document is reproduced below. It is recommended for use in determining the additional losses required to represent pier losses.



- **Figure 6-1 Pier Loss Coefficients (from Waterway Design, AustRoads, 1994)**

6.3. Pipe Crossings of Waterways

Pipe crossings (e.g. water supply or wastewater pipes) across waterways result in localised turbulence and loss of energy. Where possible, these losses should be represented in the 2D domain (or 1D elements if the waterway is wholly modelled in 1D) as a form loss.

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Section 4.7.2.3 of the TUFLOW Manual (BMT WBM 2008) provides adequate guidance on how to apply layer flow constrictions to account for height varying losses. The losses for pipe crossings can be estimated by assuming that the pipe acts similar to a vertical pier and using the head loss vs J factor curves reproduced in **Figure 6-1** from AustRoads (1994).

It is recommended that a calculation of pier area against waterway area (main channel) is derived for three levels: top of pipe, 1 m above pipe, 2 m above pipe. This will enable a four value table of elevation versus loss coefficient to be developed when supplemented with the first values for bottom of pipe (i.e. with no losses).

6.4. Structure Modelling in TUFLOW Models for RFD

Research by Barton (2001) documented the losses generated by fixed grid models (specifically TUFLOW) through abrupt constrictions. The results of this research indicated that the dynamic head loss simulated by TUFLOW models "exhibit a trend of increasing dynamic head loss coefficients with increasing spatial resolution". The research presents results for a wide range of cells sizes, constriction velocities and constriction widths (with depths of 2m).

In the study area of Moreton Bay Regional Council, the TUFLOW cell sizes are expected to be in the order of 5m (up to 10 m for the broad-scale models and maybe as low as 3m for some of the detailed models). The velocities in the bridges are expected to be in the range of 2m/s for most flood events (probably between 1m/s and 3 m/s and maybe up to 4m/s).

For these general ranges of parameters, Table 3-3 of Barton (2001) indicates that the dynamic head loss represented by TUFLOW is between 1.0 and 1.2 dynamic heads. Hence, it could be concluded that the loss represented is in the order of 1.1 dynamic heads (±0.1).

These observations are generally consistent with *Syme (2001)* which considered a similar issue and stated:

> *"Based on the results of test models and numerous real-world applications, the following are typical observations of the TUFLOW software.*
>
> *(a) Box culvert structures modelled in 2D tend to require an additional form loss coefficient of from 0.1 to 0.3 to reach agreement with culvert design curves.*
>
> *(b) Dynamically nested 1D structure elements in 2D models model tend to overestimate the form losses. This is thought to be due to some duplication of losses between the 2D domain and the 1D element. These structures need to have the combined contraction and expansion loss coefficients of the 1D element reduced by amounts varying from 0.0 to 1.0. Structures with widths less than the 2D model's cell size usually require no or minimal reduction in the loss coefficients, while larger structures with high velocities may require as much as a 1.0 reduction in the loss coefficient(s).*

(c) Testing and checking of real-world applications has shown that culverts and weirs can be correctly modelled in 2D at an angle oblique to the mesh axes (TUFLOW uses a fixed grid mesh)."

This issue is further complicated by the types of linkages used between the 2D domain and the 1D structure. It is common in representing culverts under roads across floodplains as 1D structures (using TUFLOW) to spread the 2D/1D linkage (i.e. SX cells) over a number of cells. This is usually required where the conveyance capacity of the structure is much greater than the conveyance capacity of a single cell (or even a few cells). This spreading of the 2D/1D linkage effectively distorts the contraction and expansion of the flow through the structure as flow is progressively taken out of the 2D domain and then redistributed at the outlet area. The number of cells required to be linked to a 1D structure varies based on the cell size and the difference in the conveyance of the floodplain against that in the structure.

In order to draw some conclusions and guidance on this matter, Section 4.7.1 of the TUFLOW Manual (BMT WBM 2008) is reproduced below:

"It is strongly recommended that the losses through a structure be validated through:

- ***Calibration to recorded information (if available).***
- ***Cross-checked using desktop calculations based on theory and/or standard publications (e.g. Hydraulics of Bridge Waterways, US FHA 1973).***
- ***Crosschecked with results using other hydraulic software."***

However, it is outside Council's budget limitations to cross-check every culvert and bridge with desktop calculations. A more practical approach would involve prioritising the culverts and bridges based on the influence of the structure on flooding behaviour. Then, the more critical structures would be checked against desktop methods on an individual basis. Adjustments would then be made to the losses to meet the desktop values.

In order to assist in the prioritisation of structures, the following guidance is provided:

- losses for culverts and bridges where the road is significantly overtopped can have only a minor influence of the head drop across the road. The weir characteristics of the road are generally more dominant;
- given that the general focus of the RFD modelling will be on floodplain management and more specifically on development control, the 100 year ARI flood event should provide the primary focus for these prioritisation considerations. That is, it may not be worth the effort to gain very accurate modelling of a structure in a 2 year ARI event that is completely overtopped in a 100 year ARI event; and
- a focus on those structures with adequate or good quality recorded flood levels (and some confidence in the flow rate in the model) will provide guidance for expected losses across structures in the region with similar characteristics.

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7. Buildings

7.1. Discussion

The flow of floodwater through an urban area has the ability to be impacted by buildings, fences and other obstacles. The movement of water around these obstacles dissipates energy and increase flood levels in the area. When modelling urban areas in fine model resolution it is important to include considerations for changes in direction and speed of floodwaters in the urban environment.

There are a number of methodologies, which have been investigated for appropriate techniques for hydraulically modelling obstructions in the urban areas. *Syme (2008)* has undertaken model testing of a number of these techniques. The key challenges of developing a method for the hydraulic modelling of buildings is to model how water will flow around houses as well as predicting if there is flow through buildings to represent the flood hazard at the buildings.

The methodology considered to be the most representative of the buildings in the urban context is, based on *Syme (2008)*, to reduce flow widths within the building footprint with a combination of the form loss coefficient. This approach seeks to represent to following:

- water being restricted as it enters the building;
- preserving the effect of storage of the building; and
- a realistic representation of the velocity in the building to be used for determine flood hazard.

However, this methodology does require some pre-processing to build the input data for the hydraulic model. The other method that provides a similar outcome is the increasing of the Manning's n value for the buildings. This approach is simpler to apply and can allow for different Manning's n values for different building types. However, the method does produce different flow patterns particularly at the upstream corners of the building and downstream of the building.

7.2. Recommended Parameters

It is recommended to use the methodology for the treatment of buildings in the hydraulic model designated as Method 1 in **Table 7-1.** However if a simpler methodology is sort to be used then Method 2 would be appropriate.

- **Table 7-1 Recommended Parameters – Treatment of Buildings**

Description
1) Porous + form loss = buildings represented with a blockage of 90 %, a form loss of 0.1 and a Manning's n value of 0.03
2) Increased Manning's n value = 1.000

8. Blockage

8.1. Introduction

This section outlines the recommended assessment to be undertaken to incorporate the risk of blockage from various types of structures within the floodplain. It is recommended that this assessment be undertaken to develop the potential risk envelope above the standard flooding assessment. This assessment is expected to give higher peaks flood levels in the upstream portion of the catchment particularly immediately upstream of crossing structures.

8.2. Culvert Blockage – Natural Debris

Culvert blockage may occur from materialise being mobilised as a result of flooding. There is the potential for natural debris to be mobilised and this includes both:

- floating – stick, leaves, tree limbs, logs and trees; and
- non-floating – silt, sand, gravel, rocks and boulders.

Research has been undertaken into developing a method to predict the level of culvert blockage to be used as part of hydraulic modelling. The methodology recommended to be applied is based on *Barthelmess (2009)*. This research outlines a methodology which has been developed based on a study area of Wollongong City Council. This research also outlines the development of a national culvert blockage model to be developed as part of the revision of Australian Rainfall and Runoff.

There was expected to be a number of components that influence the likelihood of blockage in a catchment, these include:

- availability of debris within a catchment;
- mobility of debris within a catchment; and
- the interaction of the debris with the structure.

The factors that influence each to the above mentioned components likelihood of culvert blockage are summarised in **Table 8-1**.

Table 8-1 Factors in Culvert Blockage

Component	Factor	Description
Debris Availability	Soil Erosivity	Can vary dependant of the soil type ie weather rocks to cohesive clays. The ability of the soil in the catchment to be eroded and entrained affected the non-floating debris availability.
	Vegetation Cover	The amount and type of vegetation, this can also include crops and agricultural uses.
	Preceding rainfall	The regularity of the rainfall has the potential to affect the amount of debris available for example more regular rainfall may lead to more flushing and less debris availability.
Debris Mobility	Rainfall intensity	The rainfall intensity may affect how the debris is mobilised. It is generally considered that more intense rainfall will have a higher potential to mobilise debris.
	Slope	The slope affects the debris mobility with steep slope generally having higher debris mobility potential. Slope is highly correlated to the stream power.
Structure Interaction	Opening Diameter	This is a factor for the interaction with debris based on the opening diameter.

8.2.1. Methodology

The proposed methodology to develop a culvert blockage model is to assess the above factors for the catchment with the view to developing a debris potential risk. The debris potential risk would be based on the debris availability and mobility. This map is to be developed based on the spatial information for the catchment. *Barthelmess (2009)* outlines a methodology for determining the debris potential based on the land use and the slope, which have been found the research most significantly impact the debris potential. This methodology is recommended to be adopted for the preliminary estimates of culvert blockage.

Slope

The slope is recommended to be reclassified and normalised into 10 volume weighted classes. The lowest slopes being given a score of zero and the highest slopes assigned a value of 10.

Land Use

The land use values recommended in the report are based on *Barthelmess (2009)*. The land use values for the debris potential are presented in **Table 8-2**.

Table 8-2 Land Use Values – Debris Potential

Land Use	Value
Conservation Area	10
Mining and Quarry	8
Grazing	6
Tree and Shrub Cover	10
River and Creek System	8
Intensive Animal Production	6
Wetland	6
Horticulture	8
Cropping	8

By then adding the slope value and land use value and dividing by 10, a value for the debris potential risk can be classified into three categories:

- high;
- moderate;
- low.

The debris potential risk should be determined by an analysis of the histogram of the raster, which results for the additional of the land use and slope values. It is recommended that this approach be verified with the data which will be the output from other sub-projects.

The debris potential risk is then compared to the culverts opening size to determine the appropriate culvert blockage factor. The recommend culvert blockage factors are presented in .

Table 8-3 Culvert Blockage Factors – Natural Debris

Upstream Catchment Conditions	Culvert Blockage Conditions	
Debris Potential	**Full Blockage**	**Partial Blockage**
High	If <6.0 m diagonal	If > 6.0 m diagonal, then apply 25 %
Moderate	If <2.4 m diagonal	If > 2.4 m diagonal, then apply 15 %
Low	If <1.2 m diagonal	If > 1.2 m diagonal, then apply 10 %

Until such time as debris potential risk is carefully assessed for the region it is recommended that a 'moderate' debris potential be assumed for blockage sensitivity testing associated with any regional hydraulic modelling.

8.2.2. Validation of Culvert Blockage – Natural Debris

The recommend approach is based on research undertaken an area of the Wollongong City Council. It is strongly recommended that further assessment and validation of this methodology and the parameters be undertaken for the MBRC area. It is considered particularly important to further investigate the impact of preceding rainfall on the debris availability in the catchment. As this parameter has not been considered in the approach recommend above, the predicted culvert blockage factors may be conservative.

Through discussions with the author of *Barthelmess (2009)*, it was advised that the validation of the culvert blockage model does not require large flood events instead can be undertaken on flood of 1 year ARI magnitude.

An example of partial blockage of a culvert in tributary of Gympie Creek is presented in **Figure 8-1**.

- **Figure 8-1 Culvert Partial Blockage with Natural Debris Example (Tributary of Gympie Ck)**

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8.3. Culvert Blockage – Urban Debris

Culvert blockage in the urban areas is possible due to urban debris mobilisation, for example car, garbage bins and shipping containers. This sort of blockage is reasonably random and is therefore difficult to apply a standard factor to the structures for urban debris blockage in the hydraulic model.

In the absence of more refined information, it is therefore recommended that the 'moderate' debris potential blockage criteria developed for natural debris described in **Table 8-3** be also applied to culverts within urban areas.

8.4. Handrail Blockage

Handrails over waterway crossings have the potential to impede flows and increase water levels upstream of the crossing. From observations from previous flooding events, handrail blockage have been observed to be significant. It is recommended that handrail blockage be assumed to be 100 %.

8.5. Fence Blockage

Fence blockage is potentially caused by debris mobilisation which then accumulates on fences in the floodway as shown in **Figure 8-2**. The fence blockage factor is predicted to vary depending on the type of fence. The recommended values for the fence blockage are presented in . The fence blockages are recommended to be applied in fences, which are located in the floodway. While fences in urban areas are not explicitly modelled, however the Manning's n value selected for the urban block includes an allowance for fences (refer **Section 4**).

- **Table 8-4 Recommended Parameters – Fence Blockage Factor**

Description	Value
Solid Fence	100 %
Chain wire	90 % + $^{v^2}/_{2g}$
Wire fence (a number of signal horizontal wires)	50 %

- **Figure 8-2 Fence Blockage Example**

9. References

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