NARROMINE SHIRE COUNCIL

NARROMINE RIVER BANK LEVEE FEASIBILITY STUDY

VOLUME 1 - FINAL REPORT

DECEMBER 2013

Aerial photograph taken looking south during the February 1955 flood, with the Macquarie River in the foreground.

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FOREWORD

The State Government's Flood Policy, which is set out in the *Floodplain Development Manual, 2005*, is directed at providing solutions to existing flooding problems in developed areas and to ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

Under the Policy, the management of flood liable land remains the responsibility of local government. The State subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist councils in the discharge of their floodplain management responsibilities.

The Policy provides for technical and financial support by the Government through the following four sequential stages:

1.	Flood Study	Determines the nature and extent of flooding.
2.	Floodplain Risk Management Study	Evaluates management options for the floodplain in respect of both existing and proposed development.
3.	Floodplain Risk Management Plan	Involves formal adoption by Council of a plan of management for the floodplain.
4.	Implementation of the Plan	Construction of flood mitigation works to protect existing development. Use of Local Environmental Plans to ensure new development is compatible with the flood hazard.

Stages 1 to 3 of the Policy have been completed at Narromine. The *Narromine River Bank Levee Feasibility Study*, which was one of the high priority measures recommended in the Floodplain Risk Management Plan is jointly funded by Narromine Shire Council and the Office of Environment and Heritage as the first stage of implementing the levee which will provide protection against flooding from the Macquarie River for floods up to the 1% AEP. This report deals with hydrologic and hydraulic aspects of the concept design of the levee.

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NOTE ON FLOOD FREQUENCY

The frequency of floods is generally referred to in terms of their Annual Exceedance Probability (AEP) or Average Recurrence Interval (ARI). For example, for a flood magnitude having 5% AEP, there is a 5% probability that there will be floods of equal to or greater magnitude each year. As another example, for a flood having 5 year ARI there will be floods of equal or greater magnitude once in 5 years on average. 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

In this report the AEP terminology has been adopted to describe the frequency of flooding.

ABBREVIATIONS

AEP	Annual Exceedance Probability (%)
AHD	Australian Height Datum
ARI	Average Recurrence Interval (years)
ARR	Australian Rainfall and Runoff, 1998 Edition
BOM	Bureau of Meteorology
CMA	Central Mapping Authority
FRMS	Floodplain Risk Management Study
FRMP	Floodplain Risk Management Plan
LiDAR	Light Detection and Ranging
OEH	Office of Environment and Heritage
SES	State Emergency Service of NSW

S1 SUMMARY

S1 Background

This report deals with the findings of a flooding and drainage investigation which was undertaken to assist Narromine Shire Council in its assessment of seven possible levee routes which it has identified along the southern bank of the Macquarie River at Narromine.

The findings of the present investigation supersede those of three previous flood studies, those being BC, 1998, L&A, 2009a and L&A, 2012, and represent the most up-to-date definition of flooding and drainage patterns in Narromine under present day conditions for events with annual exceedance probabilities (AEP's) of 1% and 0.5%.

The post-Burrendong Dam flood frequency analysis which was originally undertaken as part of BC, 1998 was updated as part of the present investigation. Both the log-Pearson Type 3 and Generalised Extreme Value distributions were fitted to 45 years of post-dam annual flood peaks. The results of the flood frequency analysis were used to assign AEP's to the December 2010 and August 1990 floods, as well as update the peak design flow estimates for Narromine.

Survey was commissioned as part of the present investigation to obtain details of the stormwater drainage system in Narromine and to also confirm whether there has been any change in a) the waterway area of the Macquarie River below the level of the weir; and b) the gauge zero on the Narromine Flood Gauge (GS 421006).

A fully two-dimensional (in plan) hydraulic model of the Macquarie River and its floodplain (refer **Figure 2.1**) was developed using the TUFLOW software to provide Council with a more accurate understanding of flooding and drainage patterns in Narromine under both present day and post-levee conditions. The hydraulic model was calibrated using historic flood data which is available for the December 2010 and August 1990 events (refer **Figure 2.2** which shows the stage and discharge hydrographs which were recorded at the NSW Office of Water's Baroona stream gauge (GS 421127), which is located about 23 km upstream of the Narromine-Eumungerie Road Bridge). The calibrated model was then used to define flooding patterns in Narromine under both present day and post-levee conditions for events with AEP's of 1% and 0.5%. Sensitivity studies were also undertaken to assess the impact minor changes in hydraulic roughness along the Macquarie River would have on flooding patterns during a major flood event. Requirements for managing stormwater drainage from the protected areas of Narromine have also been assessed.

S2 Key Findings

The key findings of the present investigation were as follows:

- Based on the updated flood frequency analysis (refer **Annexure E** for details), the following conclusions were reached:
  a) the December 2010 and August 1990 floods, which were similar in magnitude (but not level), had AEP's of 3 and 3.3%, respectively;
  b) the 1% AEP peak flow in the Macquarie River at Narromine should be revised upward from its previous estimate of 3800 m^3/s to 4000 m^3/s; and
  c) the previous estimate of the 0.5% AEP peak flow in the Macquarie River at Narromine should be maintained at 5800 m^3/s.

- The calibrated hydraulic model reproduced flooding patterns which were similar to those observed during the August 1990 (refer **Figure 2.3**) and December 2010 (refer **Figure 2.4**) floods.
- Survey commissioned as part of the present investigation confirmed that:
  a) the bed profile of the Macquarie River below the level of the weir hasn't changed greatly over the past 15 years; and
  b) the gauge zero on the Narromine Flood Gauge is consistent with the level in databases held by both NSW Office of Water and State Emergency Service of NSW.
- The 590 mm difference in the peak heights which were recorded at the Narromine Flood Gauge for the December 2010 and August 1990 floods (refer **Figure 2.5** for comparative difference in water surface levels) is attributed to a minor increase in the density of riparian vegetation along the reach of the Macquarie River downstream of the Narromine-Eumungerie Road Bridge over the 20 year period leading up to the more recent event.
- The hydraulic modelling undertaken as part of this present investigation confined the findings of L&A, 2009 and L&A, 2012, namely that the southern bank of the Macquarie River at Narromine will be surcharged in a 1% AEP flood event, with parts of Narromine inundated by floodwater to depths exceeding 1 m (refer **Figures 3.1** and **3.3**).
- Flooding patterns in Narromine for the 0.5% AEP flood event are considered indicative of those that were experienced during the February 1955 flood (refer **Figure 3.2**).
- A minor increase in hydraulic roughness along the inbank area of the river upstream of the Narromine-Eumungerie Road Bridge has a significant impact on the extent and depth of inundation in the urban parts of Narromine for a 1% AEP flood event (refer **Figure 3.4**). This is due to a combination of a) the confined nature of the flow along this reach of the river; b) the relatively long distance over which the southern bank of the river is overtopped; and c) the duration over which water levels in the river remain elevated, all of which result in a relatively large influx of floodwater into the town for a relatively small increase in water level along the upstream reach of river.
- Peak 1% AEP flood levels at Narromine are not sensitive to a minor reduction in hydraulic roughness along the inbank area of the Macquarie River downstream of the Narromine-Eumungerie Road Bridge (refer **Figure 3.5**). This is due to a larger percentage of the total flow in the river being conveyed on the floodplain (when compared to the reach of river upstream of the Narromine-Eumungerie Road Bridge) when compared to the recent historic floods of August 1990 and December 2010, thereby reducing the sensitivity of flood levels to a reduction in the conveyance capacity of the channel.[1]
- Of the seven possible levee routes which were assessed as part of the present investigation, it is understood that the Council presently prefers Levee Option 2A(i). **Annexure F** contains set out details for the preferred levee route, as well as peak 1% AEP flood levels along its length.
- There are a number of properties which lie along the southern bank of the river which will not be protected by any of the assessed levee options. The floor levels of these properties were surveyed as part of the present investigation (refer **Figure A2** in **Annexure A** for their location and **Table G1** in **Annexure G** for details of their floor levels).

[1] Because the peak flow in the river for the August 1990 and December 2010 floods was about half that of the 1% AEP flow rate of 4000 m^3/s, a larger percentage of the total flow in the was conveyed in the channel, resulting in flood levels for these historic events being more sensitive to minor changes in hydraulic roughness.

- Of the 61 properties which were surveyed, 18 would presently experience above floor flooding in a 1% AEP flood event. (refer **Table G1** in **Annexure G** for details)
- **Figures 4.1** to **4.15** show the impact the assessed levee options will have on flood behaviour at Narromine. In regard Levee Option 2A(i), its impact on existing development located along the river bank would be as follows:
  - depths of above floor flooding in the 18 properties which are presently affected by a 1% AEP flood would be increased by a maximum of 140 mm;
  - above floor flooding would be experienced in an additional 9 properties; and
  - the freeboard to the floor level of buildings which do not presently experience above floor flooding would be less than 300 mm in the case of 7 properties and less than 500 mm in the case of 1 property.
- Depending on the amount of freeboard which is incorporated in the final design of the river bank levee, there may be a need to construct a short length of additional earth embankment across a natural saddle which is located to the east of High Park Road between River Drive and the Mitchell Highway. **Figure 4.9** for example shows peak flood levels at the location of the natural saddle.[2]
- In the case of Levee Option 2A(i), the existing irrigation canal which runs along the northern boundary of the aerodrome prevents a backwater from extending south toward the north-south runway in a 1% AEP event.
- Modelling of the 0.5% AEP flood event shows that backwater flooding would only extend to the western edge of the north-south runway were the embankment associated the abovementioned irrigation canal to either fail or be removed by the land owner.
- Should the Main Western Railway embankment fail at Webbs Siding during a 1% AEP flood on the Macquarie River, then the backwater which would form up the Town Cowal would not reach the urban parts of Narromine (refer **Figure 4.16**).
- Should the railway embankment fail coincident with a 1% AEP local catchment flood on the Backwater Cowal, then the resulting backwater would extend up the Town Cowal only as far as the Mitchell Highway road culverts (refer **Figure 4.17**).
- Whilst the various levee options will protect large parts of Narromine from a 1% AEP Macquarie River flood, parts of Narromine will still be affected by local catchment runoff during a 1% AEP storm event. **Figure 5.1** shows the location where depths of inundation under post-levee conditions would exceed 100 mm in a 1% AEP local catchment storm event in the absence of elevated water levels in the Macquarie River.
- Should intense rainfall be experienced over Narromine during a period when water levels in the Macquarie River are elevated, then increased depths of inundation to those shown on **Figure 5.1** would be experienced in parts of Narromine. **Figure 5.2** shows that increases of up to 300 mm in the depth of inundation would be experienced under these conditions, principally in the Crossley Drive area. Greater depths of inundation would also be experienced immediately behind the levee north of Crossley Drive.
- Reduction in the depth of ponding in the Crossley Drive area would likely require the installation of a new relief pipeline which would need to run the 1.1 km from the existing sag in Crossley Drive to the location where the Town Cowal crosses Culling Street.

[2] Note that natural surface levels shown on **Figure 4.9** are taken along the eastern boundary of several rural residential properties which are located on the eastern side of High Park Road and that the elevation of the actual saddle, which is located a short distance to the west, is about 400 mm higher that that shown on the figure.

S3 Outstanding Issues

Issues which will need to be considered by Council arising from these more detailed studies include:

i. The amount of freeboard which is to be incorporated in the design of the river bank levee. This will allow Council to determine whether a short section of earth embankment is required across the natural saddle which is located to the east of High Park Road between River Drive and the Mitchell Highway.
ii. The impact the proposed river bank levee will have on flooding patterns in existing development and the measures which could be implemented to mitigate these impacts. Consideration will need to be given not only to those properties which experience above-floor flooding, but also those in which there will be less than 500 mm freeboard under post-levee conditions.
iii. Local catchment flood behaviour in the protected parts of Narromine and requirements for flood related development controls on future development.
iv. Whether upgrades of the local stormwater drainage system should be incorporated in the design and construction of the river bank levee in order to further reduce flood related constraints on future development in Narromine.

1. INTRODUCTION

1.1 Study Background

The Floodplain Risk Management Study (FRMS) prepared for the town of Narromine by Lyall and Associates Consulting Water Engineers (L&A) in November 2009 (L&A, 2009b) recommended the feasibility study for a river bank levee as a high priority measure for inclusion in the Floodplain Risk Management Plan (FRMP).

The need for a levee to protect the town against major flooding had previously been confirmed by the results of L&A's Narromine Flood Study of March 2009. Survey undertaken for that study showed the presence of low spots in the southern river bank, particularly in the Crossley Drive area, which would allow floodwaters to overtop at around the 1% AEP level of flooding.

The flows arrived at the urban part of Narromine via a westwards flowing flow path on the southern floodplain denoted the "Manildra Floodway" and also via the Town Cowal (**Figure 1.1**). The latter flow path conveyed flows which break out of the river at a low point in River Drive and head in a south-westerly direction to the southern side of the Main Western Railway. They then travel westwards before returning to the northern side of the railway and flowing northwards through town to the river. The peak rates of inflow from these flow paths were quite small relative to the peak flow in the river. As high flows in the Macquarie River are maintained for several days, the duration of the overtopping of the river bank and the resulting volume of outflow onto the floodplain were responsible for the flooding in the town.

The hydraulic model used in L&A, 2009a was originally based on the model which had been developed in a previous flood study for Narromine prepared by Bewsher Consulting in 1998, which used the MIKE 11 one-dimensional software which was based on a geometric model comprising cross-sections of the channel of the Macquarie River and its floodplain. The Bewsher MIKE 11 model was upgraded in L&A, 2009a to incorporate additional information on the flow paths leading from the river to the southern floodplain. There are also some drainage pipes in the river bank in the Crossley Drive area which would allow back flooding into the southern floodplain prior to the bank being overtopped. These pipes were included in the upgraded MIKE 11 model at model nodes 7.05, 6.90 and 6.50 on the Macquarie River. The upgraded MIKE 11 model was adopted in the *Levee Feasibility Study* draft report which was prepared in November 2012 (L&A, 2012).

Investigations leading to the preparation of the November 2012 draft report involved the following activities:

- Updating the flood frequency analysis of the flood record at the Baroona stream gauging station for the period from the completion of Burrendong Dam in 1965 to 2012, thereby basing the assessment of design flows on the 45 year period of available post-dam flows.
- Re-calibration of the MIKE 11 hydraulic model using data collected following the December 2010 flood. The December 2010 reached a gauge height of **14.07 m** at Narromine, substantially higher than the **13.48 m** height reached by the August 1990 flood, which at the time of the previous analyses was the highest flood experienced since the construction of Burrendong Dam.
- Using the re-calibrated model to assess design flood levels and flow patterns under both present day and post-levee conditions.

- Re-assessment of flood damages likely to be experienced with the revised flood levels. The damages prevented by the levee will represent its benefits in a cost-benefit analysis.
- Consideration of requirements for managing stormwater drainage from the protected area behind the levee.

1.2 Recommendations of the November 2012 Draft Report

The November 2012 draft report recommended that a two-dimensional (in plan) hydraulic model (replacing the cross-sectional based one-dimensional MIKE 11 hydraulic model used in the previous investigations) be developed for the Macquarie River floodplain at Narromine to address the following issues:

i. To resolve the issue of whether or not significant flows leave the Macquarie River upstream of Narromine at Webbs Siding during major floods. In addition, this approach could identify the potential for these flows, in combination with flows from the local off-river catchments to the south of Narromine, including that of the Backwater Cowal, to outflank the proposed river bank levee and flood the town from the south.

ii. To allow testing of the proposition raised in the *Macquarie River (Narromine to Oxley Station) Floodplain Management Study (SKM, 2008)* that opening Webbs Siding so as to divert flows into the Bogan River system could reduce downstream flooding in the Macquarie River. **[Note that this issue was not directly addressed as part of this present investigation]**.

iii. To assess how floodwater will re-distribute across the floodplain downstream of Narromine. The spacing of the cross sections in the MIKE 11 hydraulic model downstream of the Narromine-Eumungerie Road bridge was considered to limit its use in accurately defining the two-dimensional flooding patterns downstream of the town. It was also considered important to demonstrate that floodwater would not backwater into Narromine, given that there is no high ground in the vicinity of the aerodrome into which to tie the downstream end of the levee.

The following sections of this report provide background to the development and calibration of the two-dimensional hydraulic model, which was based on the TUFLOW software, as well as the results of design flood modelling, including the impacts of the various levee options on flood behaviour at Narromine.

1.3 Scope of Study

This investigation involved the following activities:

- Updating the flood frequency analysis for the period from the completion of the dam to 2012, thereby basing the assessment of design flows on the 45 year period of available post-dam flows. This analysis was described in the November 2012 draft report and has been retained in this investigation to provide a "best estimate" of design peak discharges. The results of this analysis are presented in **Annexure E** (bound at the back of this volume of the report).
- Calibration of the TUFLOW hydraulic model for the December 2010 and August 1990 floods. The results are described in **Chapter 2**. (Figures showing the results of hydraulic modelling are bound in **Volume 2** of the report.)

- Using the calibrated model to assess design flood levels and flow patterns under both present day and post-levee conditions. The results are presented in **Chapters 3 and 4**.
- In **Chapter 4** a range of alternative routes for the flood protection levee are modelled. The differences between peak flood levels under present day and post-levee conditions are presented as "afflux" diagrams in the report.
- Assessment of flood damages likely to be experienced under design flood conditions. The damages prevented by the levee will represent its benefits in a cost-benefit analysis. This analysis was described in the November 2012 draft report.
- Consideration of requirements for managing stormwater drainage from the protected area behind the levee. The TUFLOW model was used for this phase of the investigation which are presented in **Chapter 5**.

2. MODEL DEVELOPMENT AND CALIBRATION

2.1 Survey Requirements

As the coverage of the available Light Direction and Ranging (LiDAR) survey data upon which the two-dimensional hydraulic model was to be based did not encompass all of the floodplain, additional field survey was required both to the north and south of Narromine along the Macquarie River and Backwater Cowal, respectively. This gap in the data was filled by the survey of several cross-sections of the channel and floodplain.

Following a review of the preliminary hydraulic model results, the Office of Environment and Heritage (OEH) requested that a bathymetric survey of the Macquarie River be undertaken to check whether there has been a change in waterway area below the standing water level in the river[3] since 1996 (i.e. the date of the original survey upon which the MIKE 11 hydraulic model was based). The locations of the cross sections comprising the bathymetric survey matched those of the original survey.

In accordance with one of the recommendations of the draft report of November 2012, survey of the existing stormwater drainage system in Narromine was undertaken for the present investigation.

Figure A1 in **Annexure A** shows the extent of the land and bathymetric based surveys, which were undertaken by Casey Surveying and Design (CSD) for the present investigation, while **Annexure B** contains a series of plots showing a comparison between the 1996 and 2013 bathymetric surveys. By inspection of the plots in **Annexure B**, it is apparent that the waterway area of the river has not changed greatly over the past 17 years. Additionally it was concluded that any apparent differences in the river channel are largely due to the limited number of survey points which were used to define the waterway area in the original 1996 survey rather than real changes in channel dimensions.

2.2 Hydraulic Model Layout

The TUFLOW two-dimensional (in plan) hydraulic modelling software was used to more accurately define flooding patterns at Narromine and to assess the impact of the proposed river bank levee on flood behaviour. The layout of the TUFLOW hydraulic model developed for the present investigation is shown on **Figure 2.1**. Key features of the TUFLOW hydraulic model are:

- The two-dimensional model domain comprised a 10 m grid spacing, ground levels for which were sampled from a Digital Elevation Model (DEM) that was generated from the LiDAR survey.
- Modelling of the waterway area below the standing water level in the river, as well as the steep sections of river bank, as a one-dimensional element. Cross-sections used to define this element of the model were compiled using the LiDAR survey as well as the bathymetric survey.
- Extension of the hydraulic model a distance of about 7.5 km downstream of the limit of the two-dimensional model domain using a number of cross-sections extracted from the MIKE 11 model developed as part of the rural floodplain management study prepared by SKM, 2008, supplemented by the land and bathymetric survey undertaken for the present investigation.

[3] The standing water level in the Macquarie River of about RL 226.4 m AHD is controlled by the low level weir which is located downstream of the Narromine-Eumungerie Road bridge.

- Modelling of the Backwater Cowal channel and its left (southern) overbank as a one-dimensional element. Cross-sections surveyed for the present investigation were used for this purpose.
- Modelling of the stormwater drainage system in Narromine as a series of one-dimensional elements. Details of the pit and pipe system surveyed for the present investigation were used for this purpose.
- An upstream boundary centred on the Macquarie River comprising a discharge hydrograph.
- Free draining outlets comprising conceptual weirs with sufficient capacity to convey the modelled flow in the river system.

When modelling the various levee options, it was assumed that flood gates are fitted to the piped drainage lines which discharge directly to the river.

2.3 Hydraulic Model Calibration

The TUFLOW hydraulic model was calibrated using historic data which is available for the August 1990 and December 2010 floods. **Figure 2.2** shows the stage and discharges that were recorded at the Baroona stream gauge (GS 421127) which is located about 23 km upstream the Narromine-Eumungerie Road Bridge.

Peak flood levels in the Macquarie River at Narromine are heavily dependent on the conveyance capacity of the river, with the volume contained in the hydrograph of lesser importance until levels which surcharge the river bank are experienced. The reason for this lies in the confined nature of the floodplain, with the majority of the discharge being conveyed within the channel and on its immediate overbank area. This is in contrast with flooding patterns experienced downstream of Narromine, where the flattening of the bed gradient and the reduction in channel capacity results in the volume contained in the hydrograph being of greater importance.

In view of the above and in order to reduce the run times of the hydraulic model, inflows were increased over a period of 2 hours and then run at their peak for a further 28 hours. As a check, on the accuracy of this approach the calibrated hydraulic model was run using the discharge hydrographs that were recorded at the Baroona stream gauge (GS421127) during the August 1990 and December 2010 floods (refer **Figure 2.2**). At Narromine these two flood events rose to their peak over a period of three to five days and high flows were subsequently maintained for several days. This modelling confirmed that peak flood levels in the river at Narromine are not sensitive to the time of rise.

Table 2.1 over shows Manning's n values that provided correspondence between recorded and modelled flood levels for the August 1990 and December 2010 floods, while **Table 2.2** over shows the recorded and modelled peak heights at the Narromine Flood Gauge. **Figures 2.3** and **2.4** (2 sheets each) respectively show the TUFLOW model results for the August 1990 and December 2010 floods, whilst **Figure 2.5** shows water surface profiles along the modelled reach of the Macquarie River for the two historic floods.

By inspection of the difference in recorded versus modelled peak heights shown in **Figures 2.4** (refer Sheet 2 of 2), the calibrated TUFLOW model is considered to more closely match the recorded flood slope in the river for the December 2010 flood. No flood level data are available for the August 1990 flood apart from the peak level recorded at the flood gauge where the re-calibrated model replicates the recorded peak (ref. **Table 2.2**).

TABLE 2.1
CALIBRATED HYDRAULIC ROUGHNESS VALUES
DERIVED FOR THE MACQUARIE RIVER AT NARROMINE

Surface Treatment	Manning's n Set No. 1		Manning's n Set No. 2	
	Upstream of Narromine-Eumungerie Road Bridge	Downstream of Narromine-Eumungerie Road Bridge	Upstream of Narromine-Eumungerie Road Bridge	Downstream of Narromine-Eumungerie Road Bridge
Road and Railway	0.02		0.02	
Grassed Floodplain	0.05		0.05	
River Bed	0.06	**0.044**	0.06	**0.055**
Sparsely Treed Areas	0.08		0.08	
Tree Lined River Bank	0.35	**0.24**	0.35	**0.30**
Allotments	1.0		1.0	

Note: Set No. 1 and 2 Manning's values were found to give good correspondence with August 1990 and December 2010 flood data, respectively.

TABLE 2.2
RECORDED VERSUS MODELLED PEAK FLOOD HEIGHTS
AT THE NARROMINE FLOOD GAUGE (GS421006)

Historic Flood Event	Peak Flood Height (m)		
	Recorded	Modelled	
		Manning's n Set No. 1	Manning's n Set No. 2
August 1990	13.48	**13.45**	13.92
December 2010	14.07	13.62	**14.07**

While the peak height recorded at the Narromine Flood Gauge during the December 2010 flood was 590 mm higher than that recorded during the August 1990 flood (i.e. 14.07 m versus 13.48 m, respectively),[4] the peak flow in the Macquarie River at Narromine was approximately the same (i.e. 2200 m^3/s for the December 2010 flood versus 2078 m^3/s for the August 1990 flood).

It was found that Manning's n values in the river downstream of the Narromine-Eumungerie Road Bridge needed to be reduced by 20 per cent compared to those that provided correspondence to the December 2010 flood data in order to achieve close correspondence with the recorded peak gauge height of 13.48 m for the August 1990 flood. The reduction in hydraulic roughness is attributed to the fact that the August 1990 event was the third flood in that year leading to a possible reduction in the amount of woody debris conveyed by the floodwater, combined with a reduced density of riparian vegetation compared with the December 2010 event which occurred after an extended dry period.[5]

[4] Source: NSW State Emergency Service (SES)

[5] A review of historic aerial photography taken in 1981 and 1997, and later in the 2000's showed that there has not been a major change to the density of large trees along the banks of the Macquarie River at Narromine. Changes are therefore attributed to an increase in the density of under-storey vegetation, possibly as a result of changing land management practices by land owners.

As part of the model calibration process, runs of the hydraulic model were undertaken to assess whether the presence of several existing irrigation canals (of uncertain age) which are located on the right (eastern) overbank of the Macquarie River a short distance downstream of Narromine could have affected peak flood levels at the gauge site if they had not been present in August 1990. The analyses showed that peak flood levels at the gauge site are not sensitive to the presence of the irrigation canals, as the modelled flood levels for both the August 1990 and December 2010 floods reduced by only 30 mm when these structures were removed from the floodplain.

The zero level on the Narromine Flood Gauge was also surveyed in order to check that the reference datum for the two recorded peak flood heights had not changed over the intervening 20 year period. **Annexure C** contains a copy of CSD's report which confirms the gauge zero as 224.0 m AHD (rounded to the nearest 0.1 m), which is consistent with information contained on the NSW Office of Water's (NOW's) surface water data archive system (i.e. Pinneena) and NSWSES's Narromine Flood Intelligence Card (FIC) for the flood gauge.

Drawings of the Narromine-Eumungerie Road bridge (refer **Annexure D** for a copy) were also obtained for the present investigation. The drawings of the bridge, which was upgraded in the mid-1990's, show that the location of the bridge abutments of both the new and old bridge were aligned similarly and that Eumungerie Road was not raised where it crosses the flood runner on the right (eastern) overbank of the Macquarie River. On this basis, it was concluded that the bridge upgrade works would not have caused any difference in flood behaviour.

Based on the above findings, it was concluded that the difference in observed flood behaviour for the August 1990 and December 2010 floods, which were very similar in terms of peak discharge and volume, could be attributed to differences in the hydraulic roughness of the river downstream of the Eumungerie Road bridge at the times of their occurrence. Further, it was concluded that that the Manning's n values which were found to give good correspondence with the December 2010 flood data (refer Manning's n Set No. 2 in **Table 2.1**) should be adopted as the "best estimate" for design flood modelling, with hydraulic analysis to test the sensitivity of results to variations in roughness.

3. DESIGN FLOOD MODELLING

3.1 General

Discharge hydrographs were developed based on the 1% and 0.5% AEP estimates of peak flows which were derived from the updated flood frequency analysis for Narromine (refer **Annexure E** taken from L&A, 2012 for details). In order to achieve practical run times of the TUFLOW model a similar approach to that adopted during the model calibration process was applied to the derivation of design discharge hydrographs and the assessment of the various levee options (i.e. inflows to the model were increased over a period of 2 hours and then maintained at their peak for a further 28 hours).[6]

Figures 3.1 and **3.2** (2 sheets each) respectively show the TUFLOW model results for the 1% and 0.5% AEP floods. **Figure 3.3** shows design water surface profiles along the modelled reach of the Macquarie River, and **Table 3.1** peak heights at the Narromine Flood Gauge for the two design flood events. Flooding behaviour south of the developed area of Narromine is only shown where water enters the two-dimensional model domain from the one-dimensional element which generally comprises the channel and left overbank area of the Backwater Cowal (ref. **Figure 2.1**).

TABLE 3.1
PEAK DESIGN FLOOD HEIGHTS AT THE
NARROMINE FLOOD GAUGE

Design Flood Event	Manning's n Values		
	Set No. 2 (Best Estimate/ Design Values)	20 per cent Increase in Manning's n Values in River (Full Length of Modelled Reach)	20 per cent Decrease in Manning's n Values in River (Downstream of Narromine-Eumungerie Road Bridge Only)
1% AEP (Q = 4000 m^3/s)	15.11	15.13	14.92
0.5% AEP (Q = 5800 m^3/s)	15.34	-	-

The two-dimensional modelling approach more accurately defines the nature of the breakouts which occur along the left bank of the Macquarie River at Narromine and the path floodwater takes as it flows through the urban parts of the town in a 1% AEP event. Deep areas of ponding can be observed south of Crossley Drive and west of Fifth Avenue South, while deeper flowing water also can be observed along the Town Cowal. Minor overtopping of the Main Western Railway is also shown to occur upstream (east) of Narromine at Webbs Siding in a 1% AEP event (refer breakout of flow which occurs south of the Macquarie River near MIKE 11 River Chainage 1.35).

Table 3.2 over compares peak flood levels along the Macquarie River and Town Cowal derived from L&A, 2009a and the present investigation for the 1% AEP flood. Flood levels upstream of Narromine in the vicinity of River Drive are about 300 mm higher than was derived in the *Flood Study, 2009*, and between about 0.15-0.45 m higher between MIKE 11 River Chainage 5.50 and the Narromine-Eumungerie Road bridge.

[6] The hydraulic model was run for a sufficient length of time such that steady state flow conditions developed on the overbank area of the Macquarie River, similar to that which would happen during an actual flood event.

Modelling of the 0.5% AEP flood shows that a new breakout forms east of High Park Road upstream of Narromine during this larger event. While it is only minor in nature, consideration will need to be given to the possible construction of a short length of levee across this breakout to achieve adequate freeboard for the river bank levee, given that ground levels at the location of the natural saddle are only 700 mm above the adjacent 1% AEP flood level in the river.

TABLE 3.2
COMPARISON OF 1% AEP PEAK FLOOD LEVELS
PRESENT STUDY VS LACE, 2009a

MIKE 11 Model Reach	MIKE 11 River Chainage	Location	Peak Flood Level (m AHD)		Difference in Peak Flood Levels[3] (Present vs 2009) (m)
			LACE, 2009a	Present Study	
Macquarie River[1]	0.00	Upstream limit of hydraulic model	242.50	243.2	+0.70
	1.35		242.37	243.04	+0.67
	3.25	Adjacent to eastern end of River Drive	241.93	242.24	+0.31
	5.50		240.66	240.81	+0.15
	6.50	Adjacent to eastern end of Crossley Drive	240.07	240.43	+0.36
	7.30	Adjacent to northern end of Manildra Street	239.55	239.98	+0.43
	8.15		239.08	239.55	+0.47
	8.75	Narromine-Eumungerie Road Bridge and Narromine Flood Gauge	238.67	239.12	+0.45
	9.40		238.15	238.74	+0.59

3.2 Impact of Changing Hydraulic Roughness

Figures 3.4 and **3.5** respectively show the impact of a 20 per cent increase and decrease in the design values of Manning's n in the river on flood behaviour in a 1% AEP event.[7] These figures show colour coded increments of "afflux", which represent the increase in peak flood levels compared with the levels derived from the design values of Manning's n. A negative afflux represents a reduction in flood levels. Peak heights at the Narromine Flood Gauge for these two sensitivity runs are given in **Table 3.1**.

By inspection of **Figure 3.4**, a 20 per cent increase in the 'best estimate' values of hydraulic roughness along the modelled reach of river will significantly increase the depth of inundation in Narromine, with increases in the extent of inundation greatest south of the Main Western Railway. Whilst peak 1% AEP flood levels at the Narromine Flood Gauge are not sensitive to minor increases in hydraulic roughness (i.e. because a large portion of the flow downstream of the Narromine-Eumungerie Bridge is conveyed on the overbank of the river, remote from areas of riparian vegetation), flows are more confined upstream of the bridge crossing and as a result are

[7] Note that Manning's n values in the river were increased by 20% along the full length of the modelled reach, whilst only those downstream of the Narromine-Eumungerie Road bridge were reduced by 20% (i.e. equivalent to Manning's n Set No. 1 values).

more sensitive to changes in hydraulic roughness. Minor increases in 1% AEP flood levels upstream of the existing town levee also result in a relatively large increase in the magnitude of flow surcharging onto the southern floodplain. The reason for this is a combination of the relatively long length of river bank which is overtopped and also the relatively long period over which water levels in the river remain elevated. It is also noted that floodwater approaches the height of the natural saddle which is located east of High Park Road and which as noted above was found to operate in a 0.5% AEP event.

By inspection of **Figure 3.5**, the extent of inundation within Narromine reduces only marginally in the case of a 20 per cent reduction in hydraulic roughness values in the river downstream of the Narromine-Eumungerie Road bridge (i.e. under conditions which are considered to be representative of the river and its floodplain at the time of the August 1900 flood). This finding shows that minor changes in hydraulic roughness in the river downstream of the Narromine-Eumungerie Road bridge will not have a significant impact on flooding patterns in Narromine in a 1% AEP flood event. Similar to the finding for an increase in hydraulic roughness, 1% AEP flood levels at the Narromine Flood Gauge were found not to be sensitive to a minor reduction in hydraulic roughness, again because a large portion of the flow downstream of the Narromine-Eumungerie Bridge is conveyed on the overbank of the river, remote from areas of riparian vegetation.

3.3 Comparison with February 1955 Flood

While the peak height at the Narromine Flood Gauge for the design 1% AEP event of 15.11 m is higher than was adopted in the BC, 1998 study for the February 1955 flood (i.e. 14.94 m), the modelled extent and depths of inundation in the urban parts of Narromine are not as great as occurred during that historic event. For example, O'Neil Square at the intersection of Burraway Street and Dandaloo Street is shown to be very shallow depths of inundation in a 1% AEP event, while a photograph contained in the BC, 1998 study (refer **Plate 1** over) shows that floodwater inundated this area to about knee deep in February 1955.

Depths and extents of inundation shown on **Figure 3.2** for the 0.5% AEP event are considered to be more representative of conditions which were experienced in Narromine during the February 1955 flood event (refer aerial photograph on front cover of this report). This finding is consistent with the BC, 1998 study which found that the frequency of a flood which would inundate Narromine to the same degree as the February 1955 flood but under post-Burrendong Dam conditions was about 0.5% AEP.

It is noted that there was some conjecture during the preparation of the BC, 1998 study as to the likely height reached during the February 1955 flood (and also the flow in the river), given the gauge was overtopped by that flood. For example, in a letter to the editor of the local paper, the official gauge reader at the time stated that the water level reached 51 feet 4 and a half inches, which is equivalent to a peak height of 15.66 m on the gauge. Recent discussions with long term residents indicate that this is a reliable estimate of the height actually reached at the gauge site in February 1955.

A run of the TUFLOW hydraulic model using a peak inflow of 7000 m^3/s generated a peak height on the flood gauge of 15.43 m[8]. The TUFLOW model represents present day conditions on the floodplain. Given the uncertainty surrounding the historic flood data, combined with known post-1955 changes to conditions on the floodplain (e.g. changes to the height of the town levee and the Main Western Railway Line), the Committee should not place too great a significance on the difference between the currently adopted gauge height for the February 1955 flood of 14.94 m and that derived in the present investigation for the 1% AEP flood event of 15.11 m.

□ The Dandaloo Street/Burroway Road Intersection was inundated by the flood waters of 1955.

***Plate 1** - Photograph which was taken from the rail corridor looking north along Dandaloo Street during the February 1955 flood.*

[8] A peak flow of 7000 m^3/s represents the upper bound estimate of the flow in the Macquarie River at Narromine given in the BC, 1998 study.

4. RIVER BANK LEVEE ASSESSMENT

4.1 General

A range of levee routes have been identified by the Committee and their impacts assessed for the present investigation. **Table 4.1** located at the end of this chapter provides a summary of each levee route, while **Figures 4.1** to **4.14** show the impact of each levee option on 1% AEP flood behaviour, as well as ground and water surface levels along each route. Note that the results of modelling the 0.5% AEP flood have only been shown for the currently preferred levee route (i.e. Levee Option 2A(i)). **Table 4.2** also located at the end of this chapter summarises the impact of each levee option on 1% AEP flood behaviour.

Figure F1 (2 Sheets) in **Annexure F** gives the set out details of the preferred Levee Option 2A(i) and also shows the resulting flood behaviour in the river for the 1% AEP flood event.

Modelling of the 0.5% AEP flood for Levee Option 2A(i)[9] shows that floodwater would enter Narromine via the existing natural saddle which is located between River Drive and the Mitchell Highway. Floodwater entering Narromine from this location would inundate an area along the northern side of the Main Western Railway before draining to the Town Cowal (refer blue shaded area on **Figure 4.8**).

In the case of Levee Option 2A(i), the existing irrigation canal which runs along the northern boundary of the aerodrome prevents a backwater from forming in this area, noting that minor inundation of the northern limits of the aerodrome are still likely due to back-flow in the existing transverse drainage structure which controls local catchment runoff. Modelling of the 0.5% AEP flood event shows that backwater flooding would only extend to the western edge of the north-south runway were the embankment associated the abovementioned irrigation canal to either fail or be removed by the land owner.

Floodwater which breaks out of the Macquarie River at Webbs Siding and discharges to the Backwater Cowal in a 0.5% AEP event would also back up the Town Cowal into parts of Narromine (refer blue shaded area on **Figure 4.8**). The backwater formed by this breakout of flow would extend upstream along the Town Cowal as far as the intersection of the Mitchell Highway and Warren Road. It is noted that the backwater would generally not impact existing development, with the main exception being the Peppercorn Motor Inn which is located at the intersection of the Mitchell Highway and Warren Road.

4.2 Sensitivity Studies

<u>Impact of Change in Hydraulic Roughness in the Macquarie River</u>

The height water levels would reach along the face of the currently preferred levee (i.e. Levee Option 2A(i) assuming a 20 per cent increase in the adopted 'best estimate' Manning's n values along the modelled reach of river was assessed. The resulting water surface profile along the route of the levee is shown on **Figure 4.9**.

[9] Note that this assessment assumes adequate freeboard on the river bank levee to prevent overtopping in a 0.5 % AEP flood event.

Impact of Scouring of Railway Embankment at Webbs Siding with Levee Option 2A(i)

Figure 4.16 shows that if a section of the Main Western Railway at Webbs Siding were to fail in a 1% AEP flood event, the resulting floodwave would not cause backwater flooding into Narromine.

Impact of Scouring of Railway Embankment at Webbs Siding Coincident with Backwater Cowal Flood

Figure 4.17 shows that if a section of the Main Western Railway at Webbs Siding were to fail coincident with 1% AEP rainfall on the Backwater Cowal catchment, a backwater would form extending along the Town Cowal as far upstream as the Mitchell Highway.

4.3 Impact of Levee Construction on Existing Development

While the construction of a levee along the left bank of the Macquarie River will protect a large number of properties in Narromine, there will be several which will be impacted by the increases in flood levels described in **Table 4.2**. **Table G1** in **Annexure G** gives the peak flood levels in 61 properties, the location of which are shown on **Figure A2** in **Annexure A** and the floor levels of which were surveyed as part of the present investigation.

Of the 61 properties, 18 would presently experience above floor flooding in a 1% AEP flood event (identified by the orange shaded properties in **Column M** of **Table G1**). Depths of above floor inundation in these properties would be increased as a result of one or more of the levee options (refer corresponding depths of inundation in **Columns N** to **T** of **Table G1**). An additional 9 properties which do not presently experience above floor flooding would also be flooded as a result of one or more of the levee options.[10]

The freeboard to the floor level of the 61 properties will also be reduced as a result of the levee construction. Those properties where the freeboard is less than 300 mm have been highlighted green, and those with a freeboard between 300-500 mm highlighted purple in **Columns N** to **T** of **Table G1**.

[10] No. 90 Warren Road was assumed to experience above floor inundation, even though the modelled depth of inundation was 0.00 m for Levee Option 2A(i).

TABLE 4.1
SUMMARY OF ASSESSED LEVEE ROUTES

Levee Option	Description of Levee Route	Figure No.
1	**Levee Option 1** runs from the eastern end of River Drive around the river side of the residences that are located on the northern side of River Drive. The levee follows the high point along the left bank of the Macquarie River from River Drive to Crossley Drive where it turns and runs along the river side of the residences that are located on the northern side of latter road. West of Crossley Drive, the levee follows the alignment of the existing town levee. It then continues along the river side of the residences that are located on the eastern side of Warren Road south of the Narromine-Eumungerie Road. The levee then turns west where it crosses Warren Road and follows the route of the irrigation canal along its southern side to where it crosses the Main Western Railway.	4.1 and 4.2
2	**Levee Option 2** follows the same route as **Levee Option 1** with the exception that its eastern (upstream) end is located in the road reserve of River Drive, as opposed to running along the river side of the residences that are located on its northern side.	4.3 and 4.4
2A	**Levee Option 2A** follows the same route as **Levee Option 2** with the exception that at its western (downstream) end it runs north along Warren Road before turning west at the northern limit of the Narromine Aerodrome.	4.5 and 4.6
2A(i)	**Levee Option 2A(i)** follows the same route as Levee Option 2A with the exception that rather than running along the river side of the residences that are located on the eastern side of Warren Road south of the Narromine-Eumungerie Road, it turns west at the northern end of the existing town levee and runs along the road reserve of Warren Road. *Note that the route of Levee Option 2A(i) shown on **Figures 4.7** and **4.8** should be adjusted such that it crosses Warren Road approximately 180 m north of the Mitchell Highway at the location of the existing crest in the road.*	4.7, 4.8 and 4.9
2B	**Levee Option 2B** follows the same route as **Levee Option 2A** with the exception that at its western (downstream) end it runs along the river side of residences that are located on the eastern side of Warren Road north of the Narromine-Eumungerie Road.	4.10 and 4.11
2C	**Levee Option 2B** follows the same route as **Levee Option 2** with the exception that rather than running along the river side of the residences that are located on the eastern side of Warren Road south of the Narromine-Eumungerie Road, it turns west at the northern end of the existing town levee and runs along the road reserve of Warren Road.	4.12 and 4.13
3	**Levee Option 2B** follows the same route as **Levee Option 2** with the exception that rather than run the full length of River Drive, it turns south and runs part-way along High Park Road.	4.14 and 4.15

TABLE 4.2
IMPACT OF LEVEE OPTIONS ON FLOOD BEHAVIOUR
1% AEP

Levee Option	Impact on Flood Behaviour	Figure No.
1	• Peak flood levels will generally be increased in the range 20-50 mm both upstream and downstream of Narromine, with greater increases in the range 50-100 mm shown to occur between River Drive and a location immediately downstream of the Narromine-Eumungerie Road.	4.1
2	• The impact on peak flood levels are similar to those described for **Levee Option 1**, however, increased depths of inundation will be experienced in residential properties located on the northern side of River Drive, since the levee will not prevent flooding in these allotments. • Increases in the range 100-200 mm are shown to occur in several properties located near the eastern end for River Drive.	4.3
2A	• The extension of the levee north (downstream) of the Narromine-Eumungerie Road will result in increases in the range 50-100 mm extending downstream of Narromine. • Increases generally in the range 100-200 mm will occur in the residential properties that are located on the eastern side of Warren Road north (downstream) of the Narromine-Eumungerie Road, with isolated increases in the range 200-300 mm shown to occur in their Warren Road frontages. • The extension of the levee north (downstream) of the Narromine-Eumungerie Road along Warren Road would protect existing development located in Skypark. Backwater flooding into the aerodrome is prevented by the presence of an irrigation canal which runs along its northern boundary.[11]	4.5
2A(i)	• Impacts will be similar to those described for **Levee Option 2A**, however, depths of inundation will be further increased in the residential properties that are located on the eastern side of Warren Road north (downstream) of the Narromine-Eumungerie Road.	4.7
2B	• Impacts will be similar to those described for **Levee Option 2A**, however, those residential properties which are located on the eastern side of Warren Road north (downstream) of the Narromine-Eumungerie Road would be protected by the levee.	4.10
2C	• Impacts will be similar to those described for **Levee Option 2**, however, the residential properties which are located on the eastern side of Warren Road south (downstream) of the Narromine-Eumungerie Road would not be protected by the levee. • Increased depths of inundation in the range 300-500 mm would be experienced in the residential properties which are located on the eastern side of Warren Road south (downstream) of the Narromine-Eumungerie Road	4.12
3	• Impacts will be similar to those described for **Levee Option 2**, however, increased depths and extents of inundation will be experienced in residential properties located on the eastern side of High Park Road and along River Drive east of High Park Road.	4.14

[11] Note that no data are available on the structure which would convey local catchment runoff from the aerodrome across the canal. As a result, minor backwater flooding would occur in the aerodrome via this structure during a 1% AEP flood event.

5. PROVISIONS FOR LOCAL DRAINAGE

5.1 Potential Ponding Directly Behind River Bank Levee

In general, the assessed levee routes will follow the high point along the left bank of the Macquarie River, with the exception of the following locations:

- at the eastern end of River Drive for Levee Options 2, 2A, 2A(i), 2B, 2C and 3;
- along the river side of the residences located along the northern side of Crossley Drive for all assessed levee options;
- along Warren Road south of the Narromine-Eumungerie Road for Levee Options 2A(i) and 2C; and
- along Warren Road north of the Narromine-Eumungerie Road for Levee Options 2A, 2A(i), 2B and 2C.

In regards provision for the control of local catchment runoff which could potentially pond along the upslope side of the levees, the following comments are made:

- **Eastern End of River Drive** – A stormwater pipe fitted with a flood gate could be provided through the levee bank to control local catchment runoff which would be generated by the relatively small catchment which is located between it and the left (southern) bank of the Macquarie River. The gate would need to be closed during a river flood, as water which overtops the river bank at this location would otherwise enter the town.

  Minor ponding would occur on the river side of the levee during the period the flood gate was closed and the area was not inundated by water from the river. Given the relatively small area that would contribute direct runoff to the upslope side of the levee, flooding of existing development as a result of localised rainfall will not occur.

- **Crossley Drive** – It will be necessary to install a minor piped drainage system along the protected side of the levee to prevent the ponding of stormwater in the residential properties which are located on the northern side of Crossley Drive. The outlet(s) of the minor piped drainage system will need to be fitted with a flood gate(s) to prevent backwater flooding during periods of elevated water levels in the river. Given the relatively small area that would contribute direct runoff to the upslope side of the levee, flooding of existing development as a result of localised rainfall will not occur.

- **Warren Road South of the Narromine-Eumungerie Road** – Local catchment runoff originating from within several residential properties which are located on the eastern side of Warren Road south of the Narromine-Eumungerie Road drains in a northerly direction toward the irrigation canal via an existing roadside table drain. Provided the table drain is maintained as part of the levee design, then local catchment runoff will not affect existing development.

  In the case of Levee Option 2A(i), there is a small area on the eastern side of Warren Road immediately north of its intersection with the Mitchell Highway which drains towards the Town Cowal. It will be necessary to install a minor piped drainage system along the river side of the levee to prevent the ponding of stormwater in this area. The inlet of the minor piped drainage system would need to be fitted with a flood gate to prevent water draining to the Town Cowal during periods of elevated water levels in the Macquarie River.

As stated in **Table 4.1**, the route of Levee Option 2A(i) shown on **Figures 4.7** and **4.8** should be adjusted such that it crosses Warren Road approximately 180 m north of the Mitchell Highway at the location of the existing crest in the road. This minor adjustment to the alignment of the levee route will assist in minimising requirements for the control of local catchment runoff.

In the case of Levee Option 2C, it would be necessary to install a catch drain along the eastern side of the levee to control runoff from the residential properties which lie on the eastern side of Warren Road south of the Narromine-Eumungerie Road bridge. Runoff captured by the catch drain would discharge to the existing irrigation canal which is located immediately south of the Narromine-Eumungerie Road.

- **Warren Road North of the Narromine-Eumungerie Road** – Local catchment runoff originating from the residential properties which are located on the eastern side of Warren Road north of the Narromine-Eumungerie Road drains in a northerly direction toward an existing irrigation canal via a roadside table drain. Provided the table drain is maintained as part of the levee design, then local catchment runoff will not affect existing development for Levee Options 2A, 2A(i) and 2C. In the case of Option 2B, it is envisaged that a minor piped drainage system would need to be incorporated in the levee design at the location where it crosses Warren Road. This system would need to be fitted with a flood gate to prevent backwater flooding during periods of elevated water levels in the Macquarie River.[12]

5.2 Impact of Local Stormwater Runoff

Whilst the assessed levee options will protect most of Narromine from a 1% AEP flood on the Macquarie River, heavy rain falling directly over the town will result in isolated areas being inundated by local stormwater runoff.

In order to identify areas where increased depths of inundation are likely to occur, the grid spacing comprising the two-dimensional model domain in the TUFLOW model was reduced from 10 m to 5 m. The “Direct Rainfall on Grid” approach was adopted for generating local catchment runoff, with design storms of 1, 2, 12, 24, 36 and 48 hours duration used as input to the model. Initial and continuing loss values of 15 mm and 2.5 mm/hr respectively were adopted for generating rainfall excess.

Figure 5.1 shows the envelope of indicative depths of inundation resulting from a 1% AEP local catchment storm in the absence of elevated water levels in the Macquarie River. While the coarseness of the hydraulic model may give rise to less accurate results in some areas, importantly it highlights several areas where increased depths of inundation greater than 100 mm are likely to be experienced in the protected parts of Narromine in a 1% AEP local catchment storm event (e.g. along the Town Cowal on the southern side of the Main Western Railway).

Based on the above finding, development controls principally relating to minimum floor level requirements would still need to be applied in parts of Narromine to cater for local stormwater impacts. As the Building Code of Australia requires floor levels to be set a minimum 150 mm above the adjacent ground level, controls would need to apply in areas where the depth of inundation is shown on **Figure 5.1** to exceed say 100 mm.

[12] It is envisaged that a removable barrier type arrangement would be incorporated into the design of the levee where it cross Warren Road and that the minor piped drainage system would intercept flow in the roadside table drain prior to it discharging across the footings for the barrier.

Several drainage upgrade options aimed at reducing local stormwater impacts in Narromine were identified in a study undertaken by PPK Consultant Pty Ltd in 1994. While outside of the scope of the present investigation, the TUFLOW model used to define local drainage patterns could be adjusted to assess the mitigating effects of a selected number of these drainage upgrade options, as well as others identified as part of the present investigation (e.g. a possible upgrade of the local stormwater drainage system which controls depths of ponding in Crossley Drive).

By comparison of the revised flood frequency analysis (refer L&A, 2012) with available gauged stream flows and corresponding levels on the Narromine Flood Gauge, a flood with an AEP of about 5% (or a peak flow of around 1750 m^3/s) would commence to cause backwater flooding problems in parts of Narromine.

In order to assess the impact coincident elevated water levels in the Macquarie River will have on local catchment flood behaviour in the protected parts of Narromine, the adjusted TUFLOW model was run with the flood gates in their fully closed position. The model results showed increased depths of inundation to those shown on **Figure 5.1** would be experienced in parts of Narromine under a gates fully closed scenario. **Figure 5.2** shows that increases of up to 300 mm in the depth of inundation would be experienced along Crossley Drive under these conditions. Ponding along the protected side of the levee to depths exceeding 1 m would also likely occur.[13]

In order to reduce the depth of ponding in the Crossley Drive area, a new relief pipeline would need to be installed extending from the existing sag in Crossley Drive to the location where the Town Cowal crosses Culling Street, a distance of approximately 1.1 km. The pipeline would be effectively flat and operate like a syphon during a coincident local catchment/river flood event. Given the flat gradient of the pipeline it would probably need to be cut-off from everyday operation, as otherwise it would be subject to the build-up of sediment in its invert and require continual maintenance to remain operable.

[13] Note that due to excessively long runs times of the hydraulic model, only the results for design storms with durations of 2, 12 and 36 hours have been shown.

6. REFERENCES

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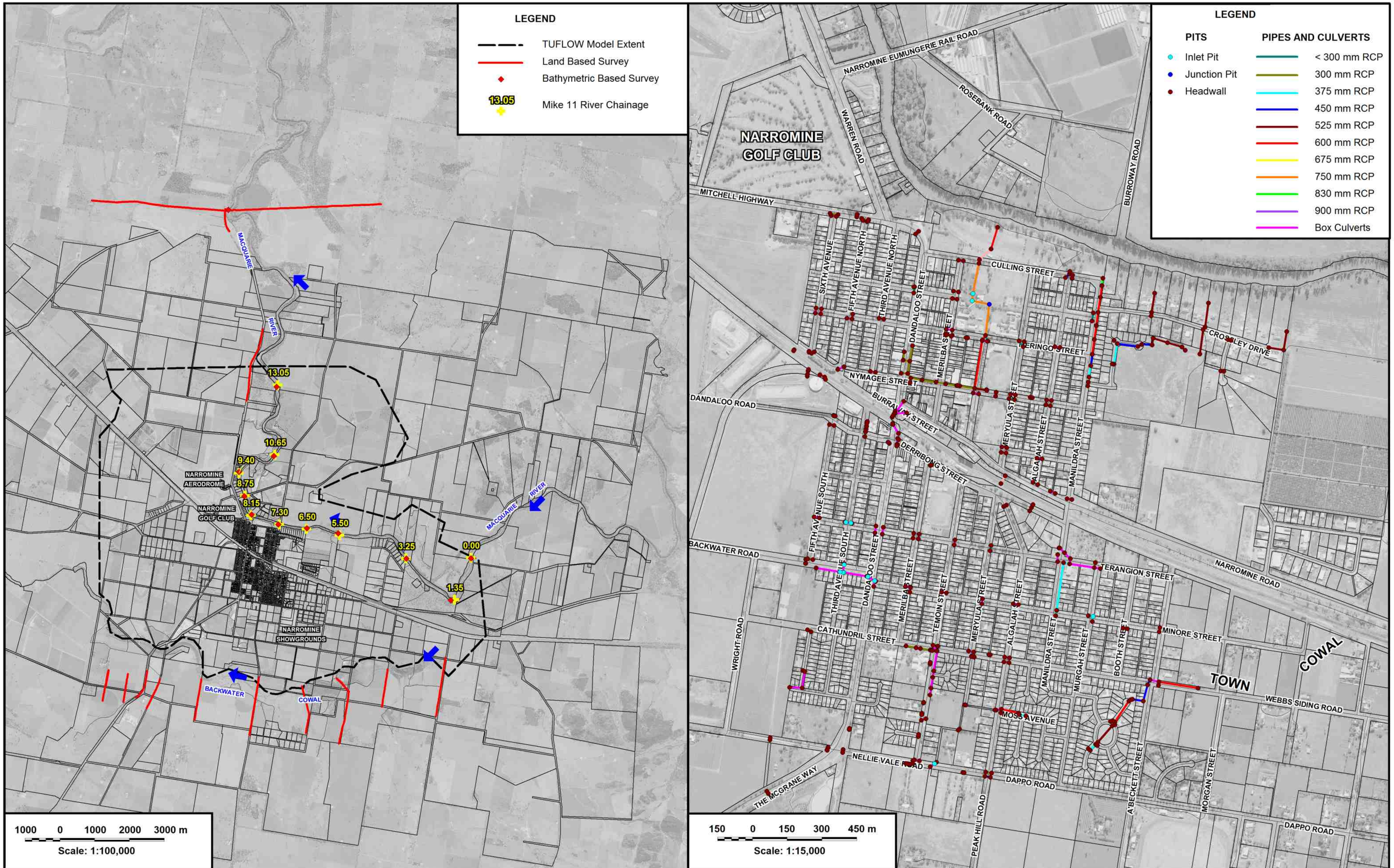
ANNEXURE A
EXTENT OF LAND AND BATHYMETRIC BASED SURVEY



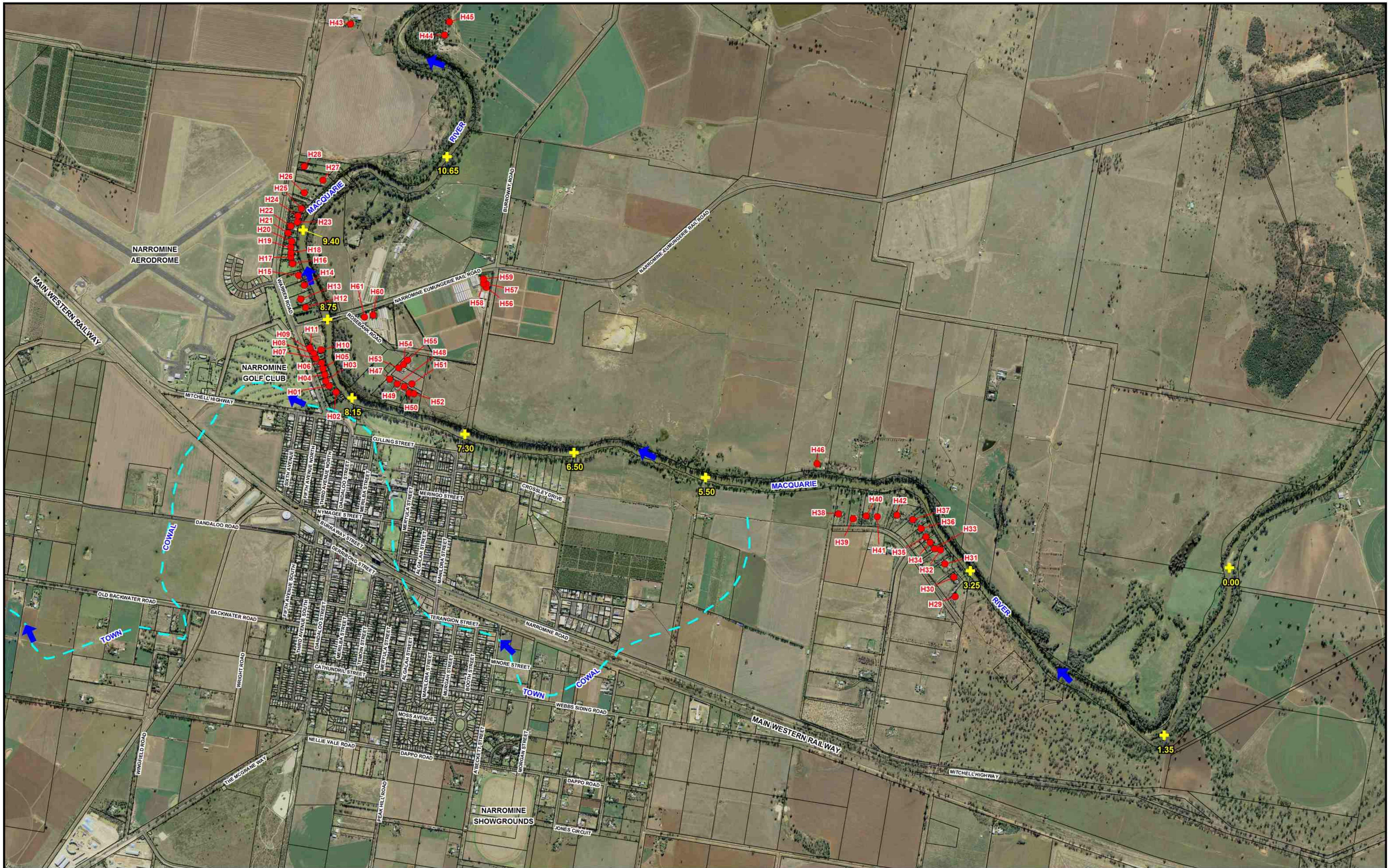
N

Lyall & Associates

NARROMINE RIVER BANK LEVEE FEASIBILITY STUDY

Figure A1

EXTENT OF LAND AND BATHYMETRIC BASED SURVEYS



LEGEND

7.30 MIKE 11 River Chainage

H30 Surveyed Property and ID

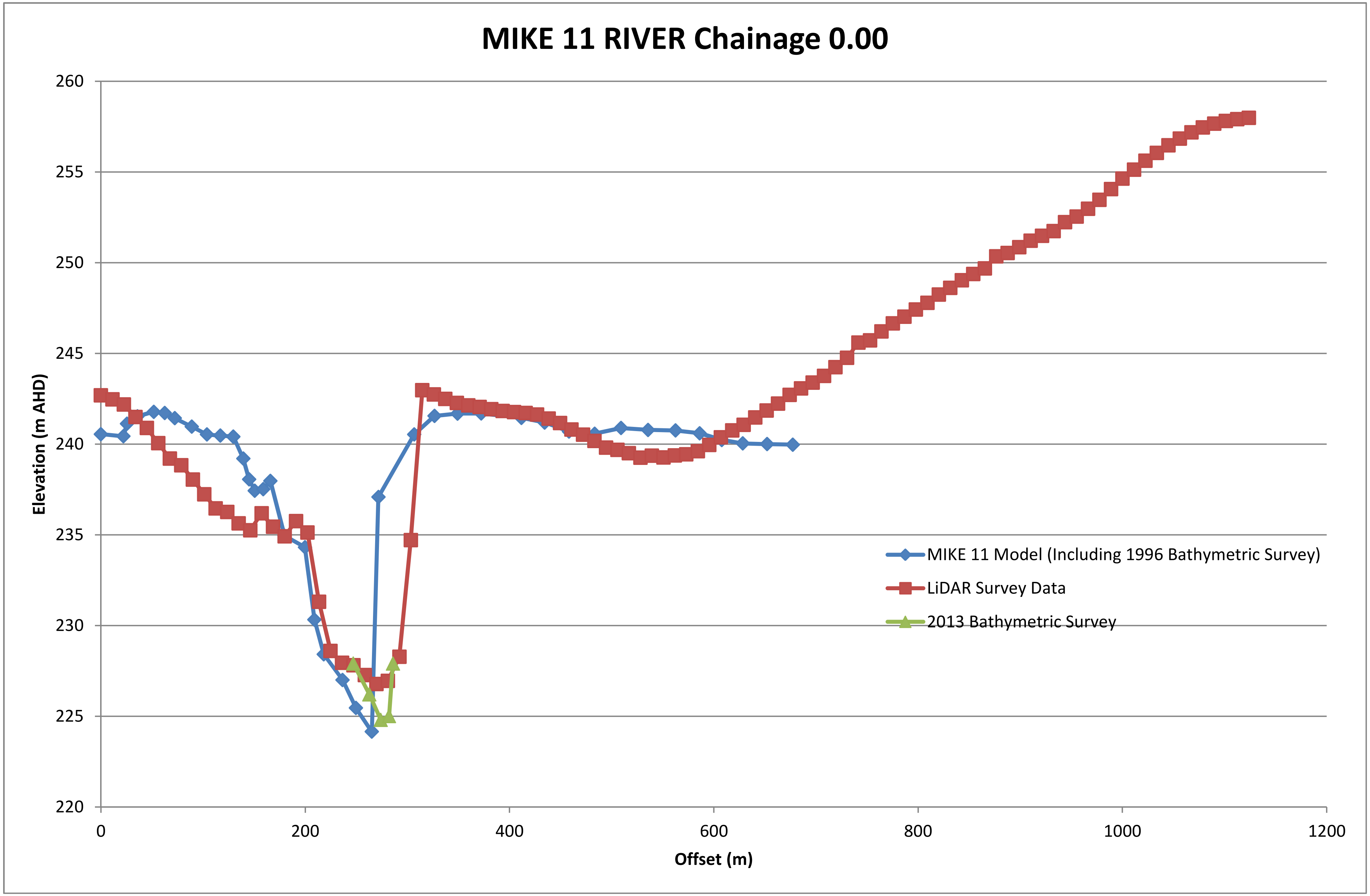
Lyall & Associates

NARROMINE RIVER BANK LEVEE FEASIBILITY STUDY

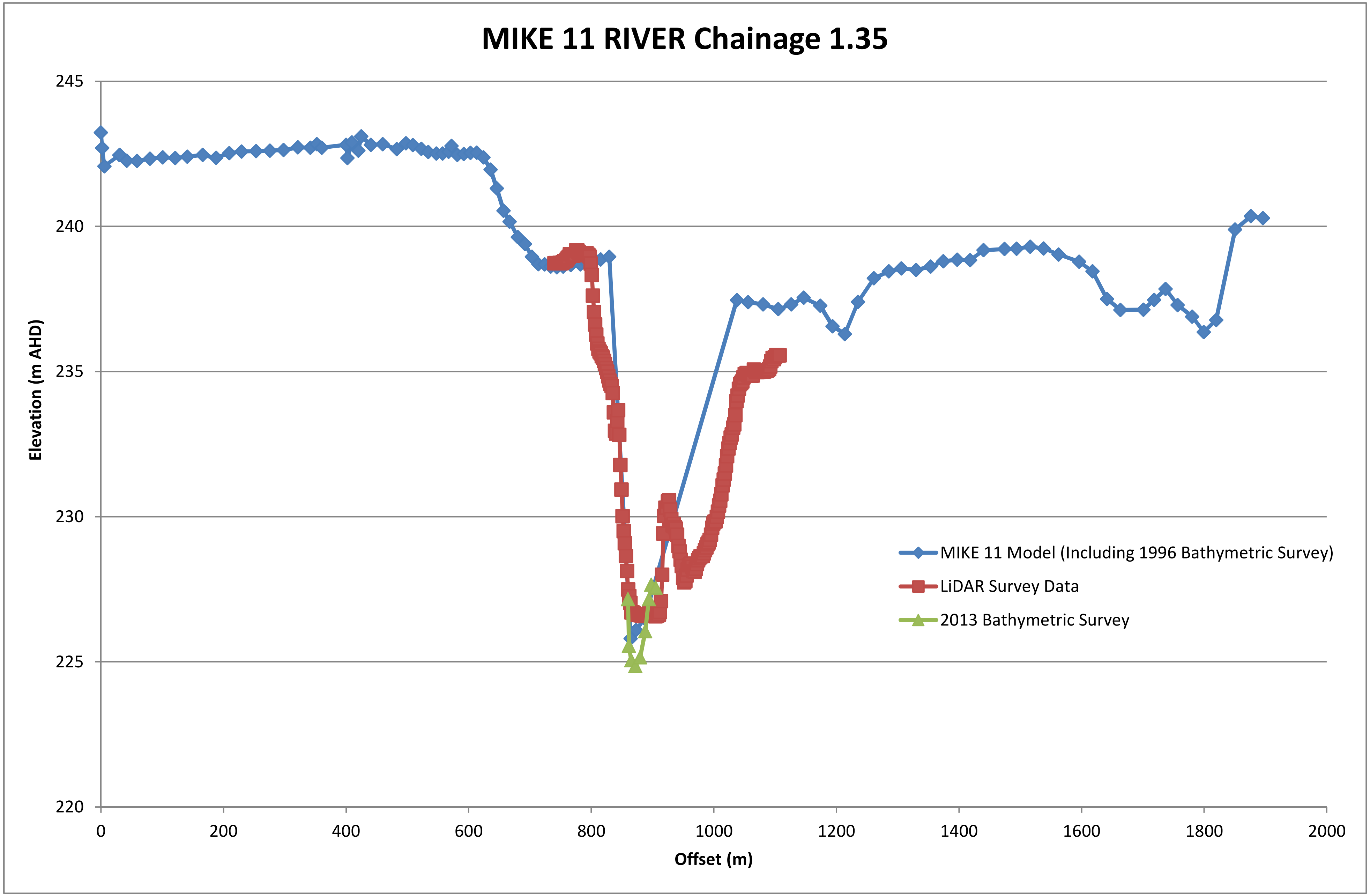
Figure A2

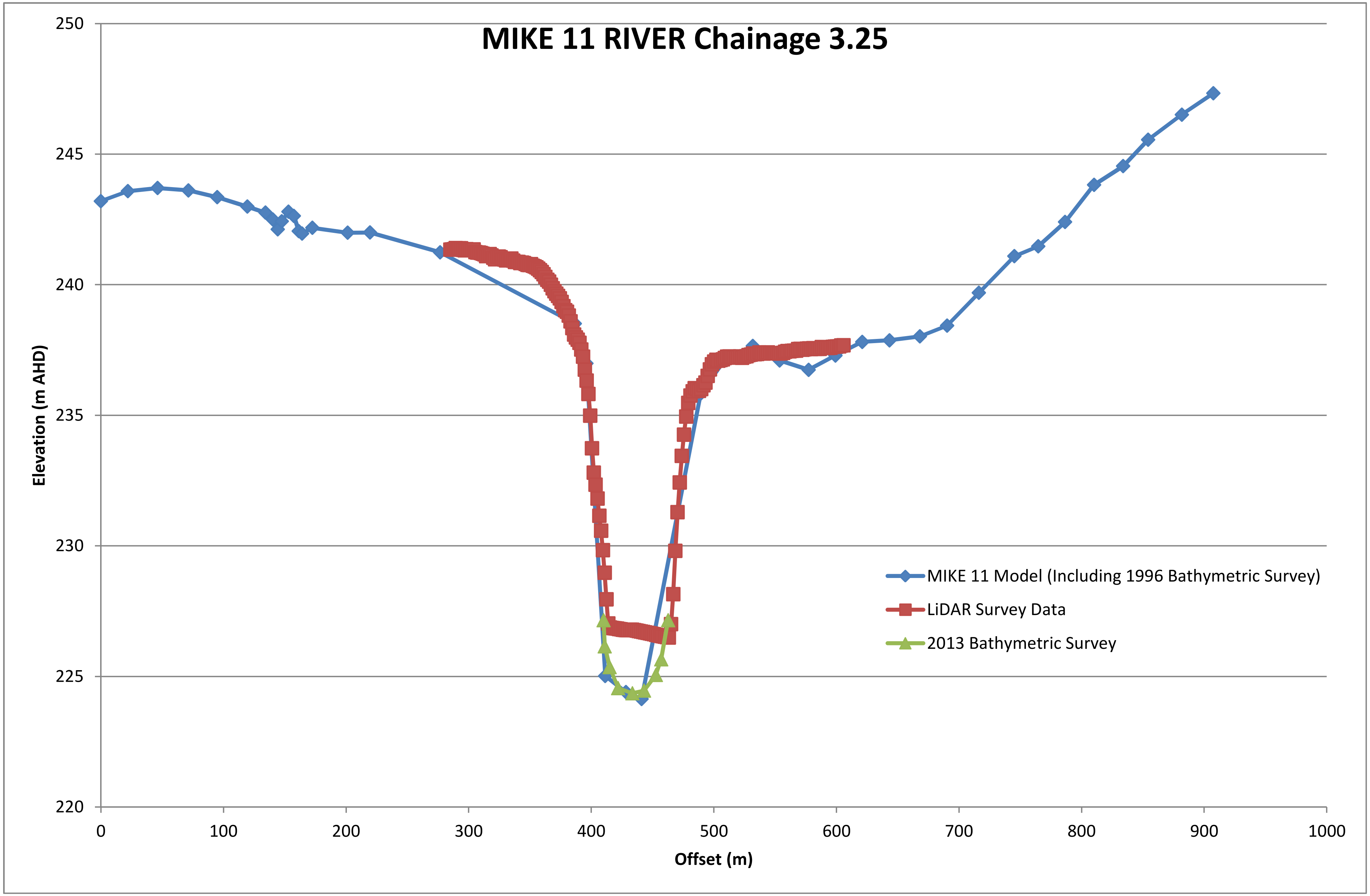
LOCATION OF SURVEYED PROPERTIES

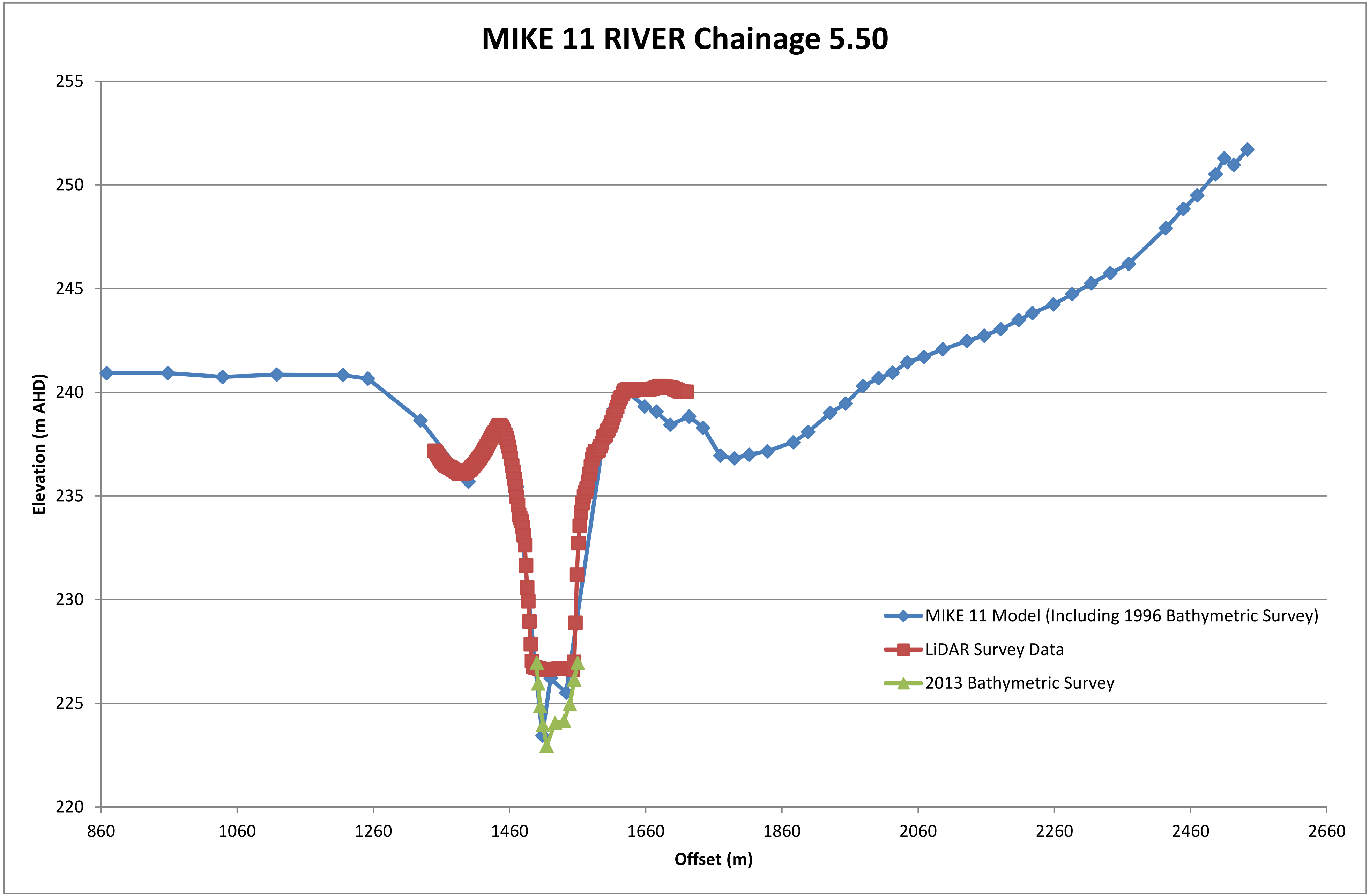
ANNEXURE B
COMPARISON OF BATHYMETRIC BASED SURVEYS



MIKE 11 RIVER Chainage 0.00
Elevation (m AHD)
260
255
250
245
240
235
230
225
220
0
200
400
600
800
1000
1200
Offset (m)
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data
2013 Bathymetric Survey


MIKE 11 RIVER Chainage 1.35
Elevation (m AHD)
Offset (m)
245
240
235
230
225
220
0
200
400
600
800
1000
1200
1400
1600
1800
2000
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data
2013 Bathymetric Survey


MIKE 11 RIVER Chainage 3.25
Elevation (m AHD)
Offset (m)
250
245
240
235
230
225
220
0
100
200
300
400
500
600
700
800
900
1000
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data
2013 Bathymetric Survey


MIKE 11 RIVER Chainage 5.50
Elevation (m AHD)
Offset (m)
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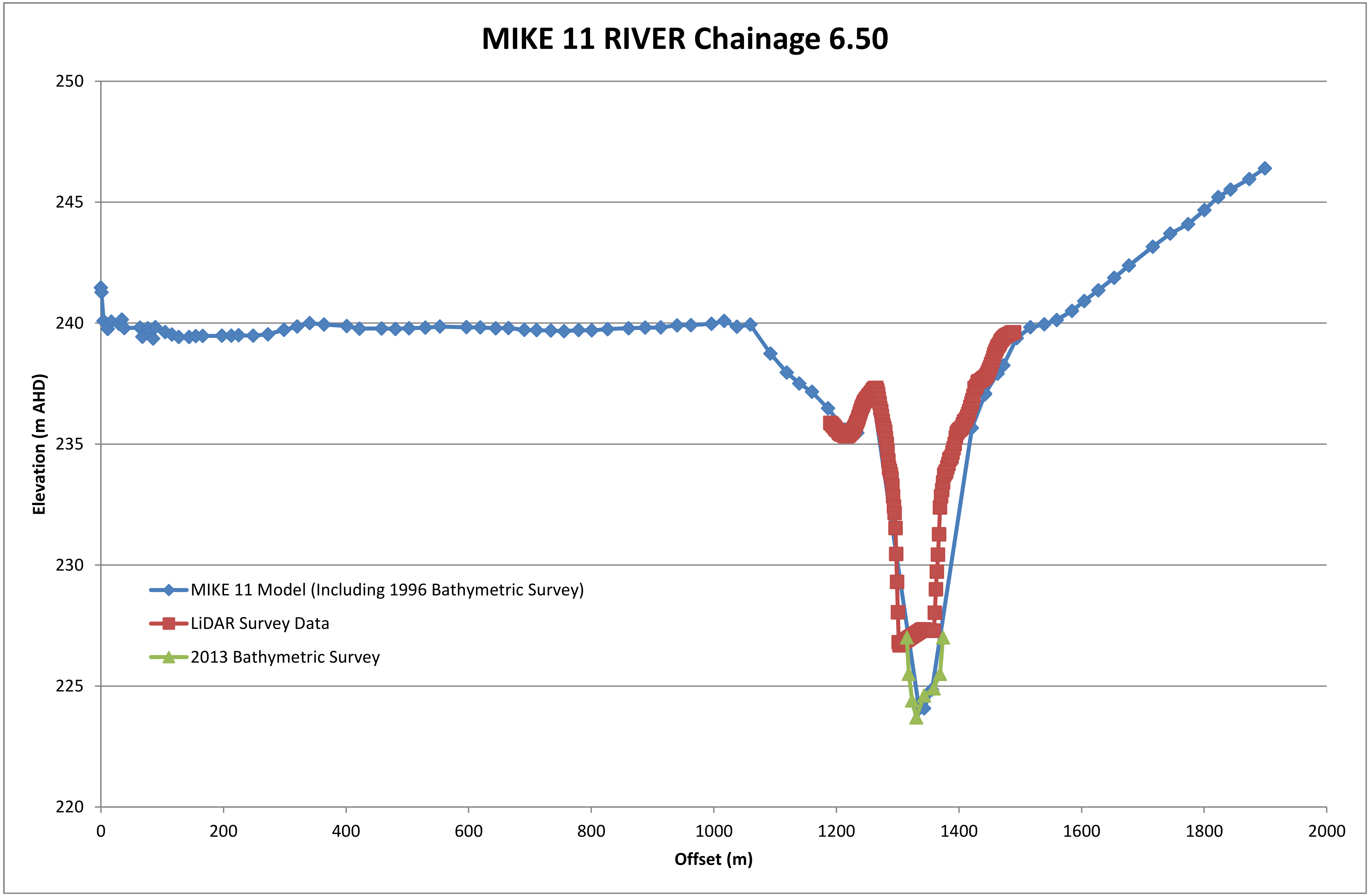
250
245
240
235
230
225
220
860
1060
1260
1460
1660
1860
2060
2260
2460
2660
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data
2013 Bathymetric Survey


MIKE 11 RIVER Chainage 6.50
250
245
240
235
230
225
220
Elevation (m AHD)
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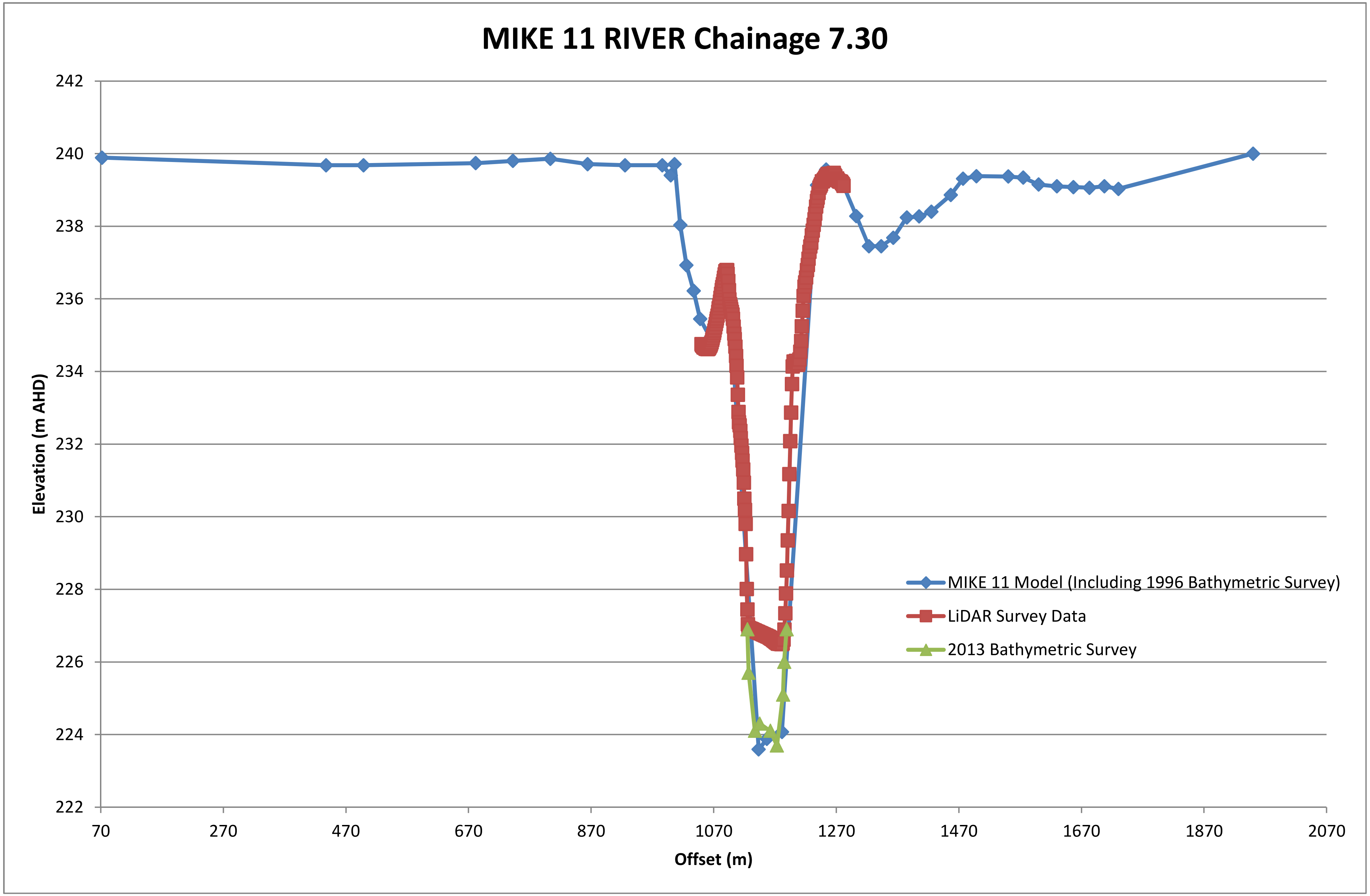
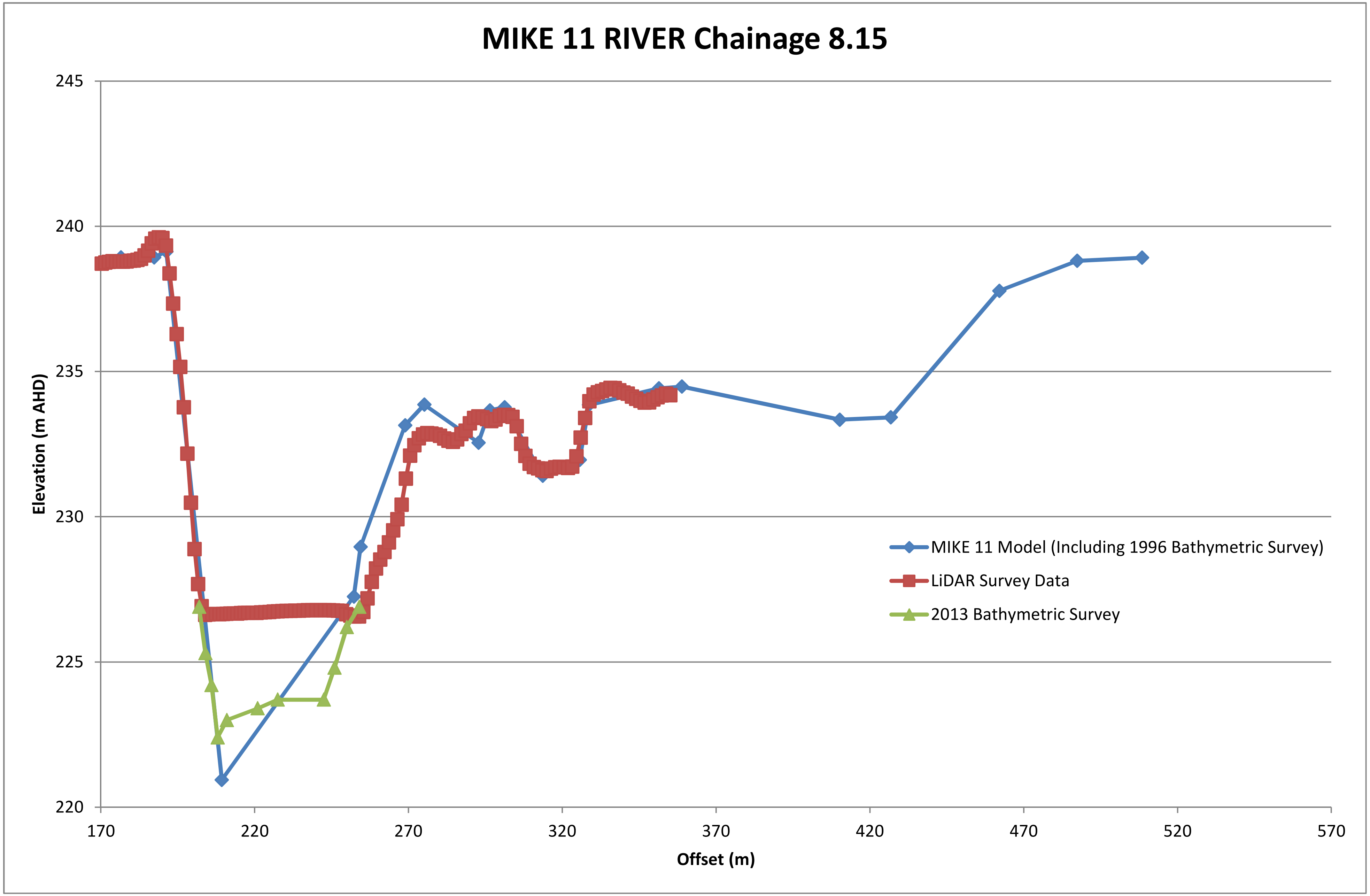
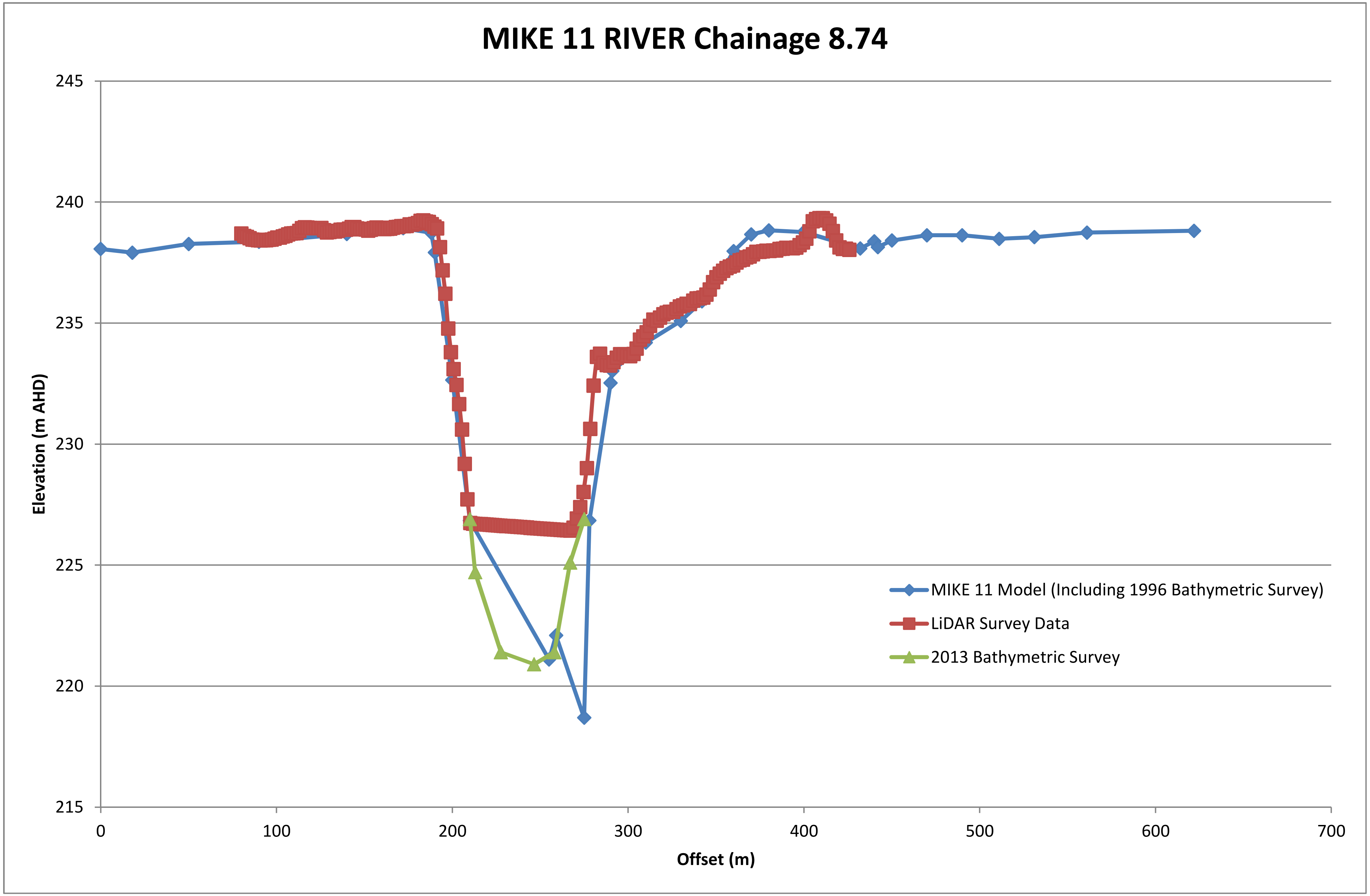
200
400
600
800
1000
1200
1400
1600
1800
2000
Offset (m)
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data
2013 Bathymetric Survey



MIKE 11 RIVER Chainage 7.30
242
240
238
236
234
232
230
228
226
224
222
Elevation (m AHD)
70
270
470
670
870
1070
1270
1470
1670
1870
2070
Offset (m)
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data
2013 Bathymetric Survey


MIKE 11 RIVER Chainage 8.15
Elevation (m AHD)
245
240
235
230
225
220
170
220
270
320
370
420
470
520
570
Offset (m)
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data
2013 Bathymetric Survey


MIKE 11 RIVER Chainage 8.74
Elevation (m AHD)
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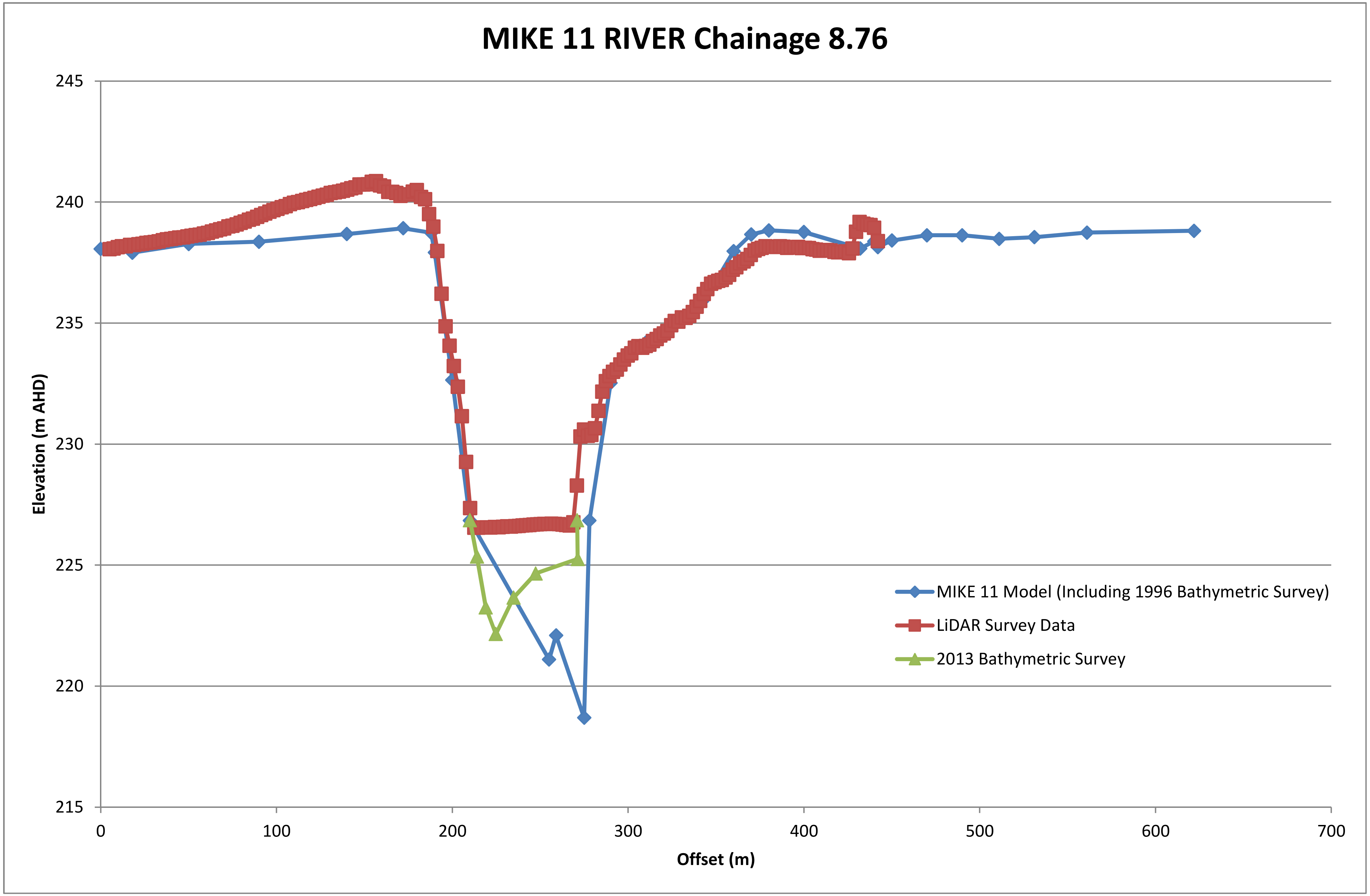
240
235
230
225
220
215
0
100
200
300
400
500
600
700
Offset (m)
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data
2013 Bathymetric Survey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Elevation (m AHD)
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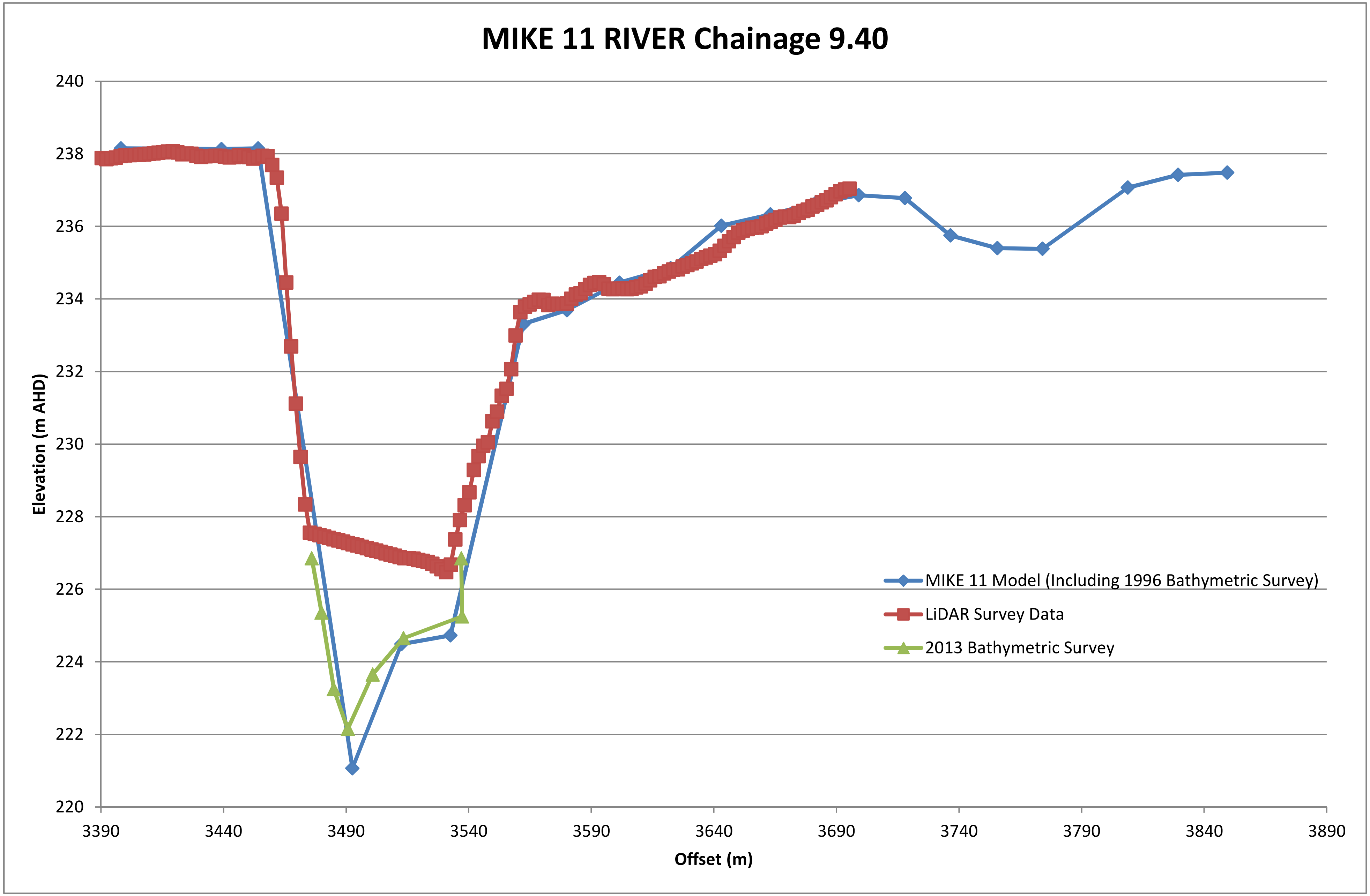
240
235
230
225
220
215
0
100
200
300
400
500
600
700
Offset (m)
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data
2013 Bathymetric Survey


MIKE 11 RIVER Chainage 9.40
Elevation (m AHD)
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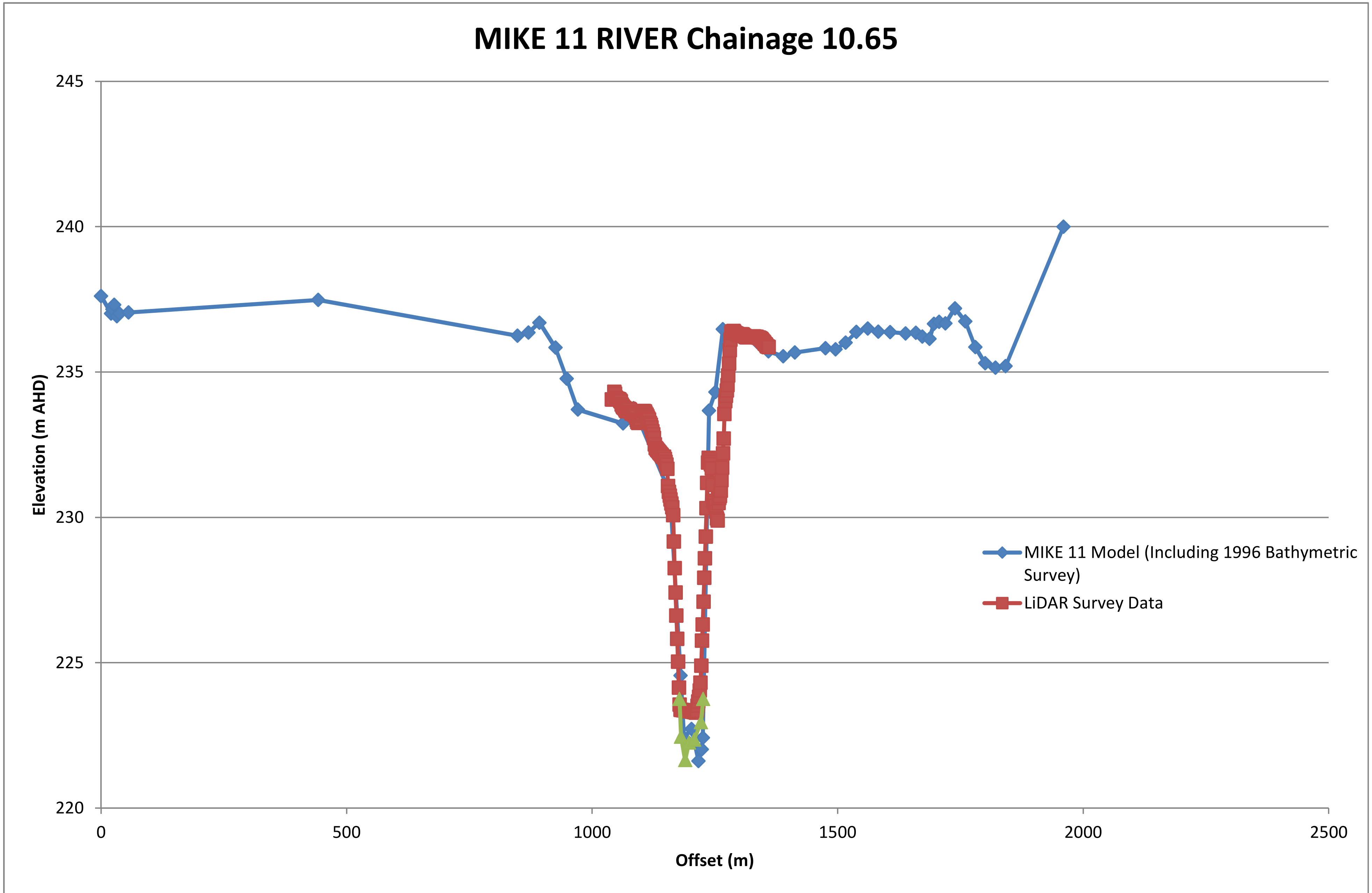
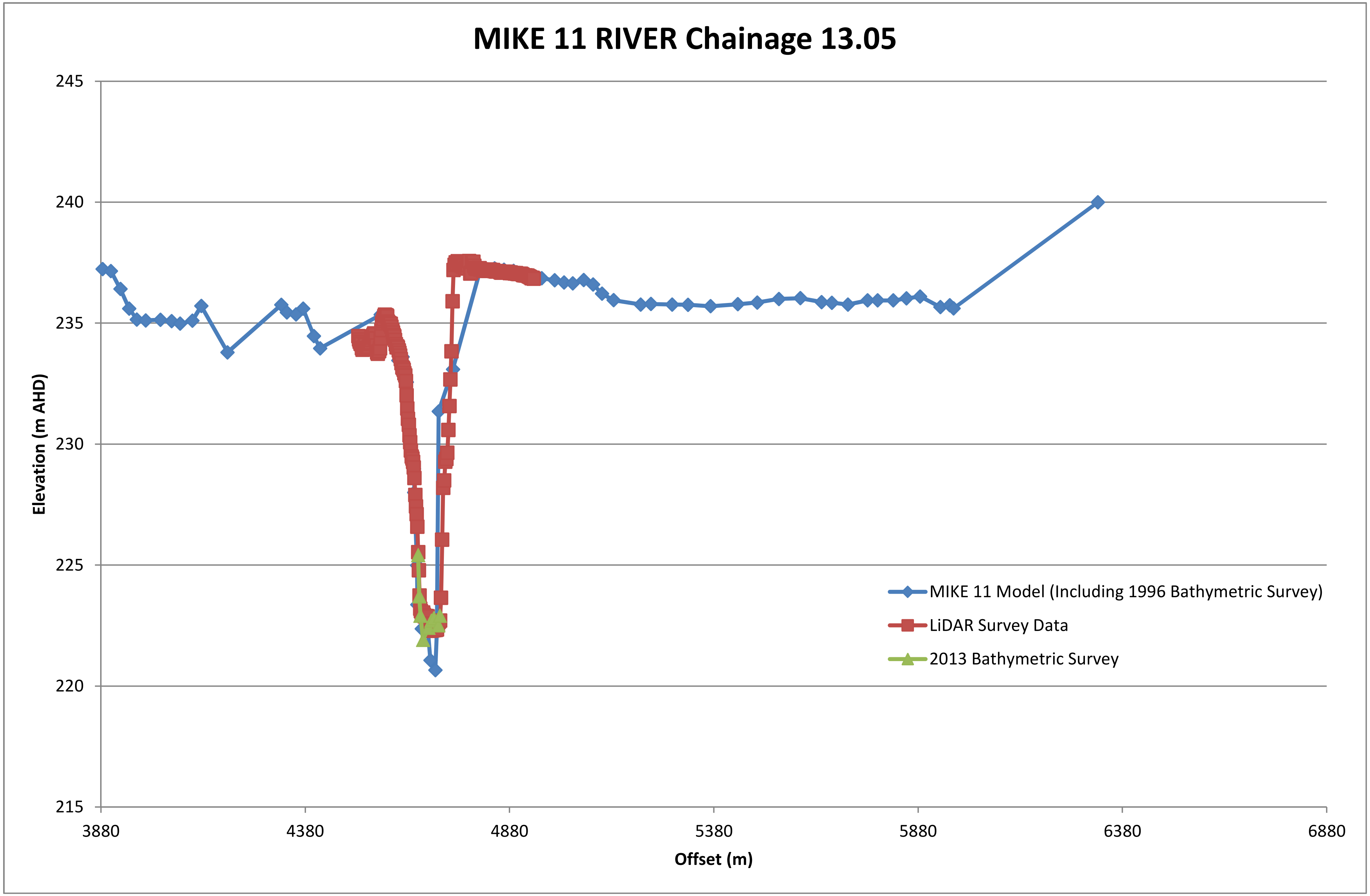
238
236
234
232
230
228
226
224
222
220
3390
3440
3490
3540
3590
3640
3690
3740
3790
3840
3890
Offset (m)
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data
2013 Bathymetric Survey


MIKE 11 RIVER Chainage 10.65
245
240
235
230
225
220
0
500
1000
1500
2000
2500
Elevation (m AHD)
Offset (m)
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data



MIKE 11 RIVER Chainage 13.05
245
240
235
230
225
220
215
Elevation (m AHD)
3880
4380
4880
5380
5880
6380
6880
Offset (m)
MIKE 11 Model (Including 1996 Bathymetric Survey)
LiDAR Survey Data
2013 Bathymetric Survey

ANNEXURE C
NARROMINE FLOOD GAUGE (GS 421006) SURVEY

13 Moulden Street
PARKES, NSW 2870
Ph: (02) 6863 5990
Fax: (02) 6862 1439
E: csad@aapt.net.au

Casey Surveying and Design Pty Ltd
ABN: 27 126 728 069

5th August 2013
Ref: 13035

Mr Scott Button
Lyall and Associates
Via Email: lacewater@bigpond.com.au

SURVEY REPORT
RE: FLOOD DEPTH GAUGE AT TIMBREBONGIE BRIDGE
OVER THE MACQUARIE RIVER AT NARROMNE

Dear Scott,

As per your instructions, I have completed a survey to determine the levels of the flood depth markers at the Timbrebongie Bridge over the Macquarie River at Narromine.

The zero mark for the gauge showing up to 9.0m attached to the pier at the northern bank of the river has been determined to be at an AHD level of 223.98m.

The freestanding gauges showing 9.0m to 10.0m, 10.0m to 11.0m and 11.0m to 12.0m each have a zero level of 224.00m.

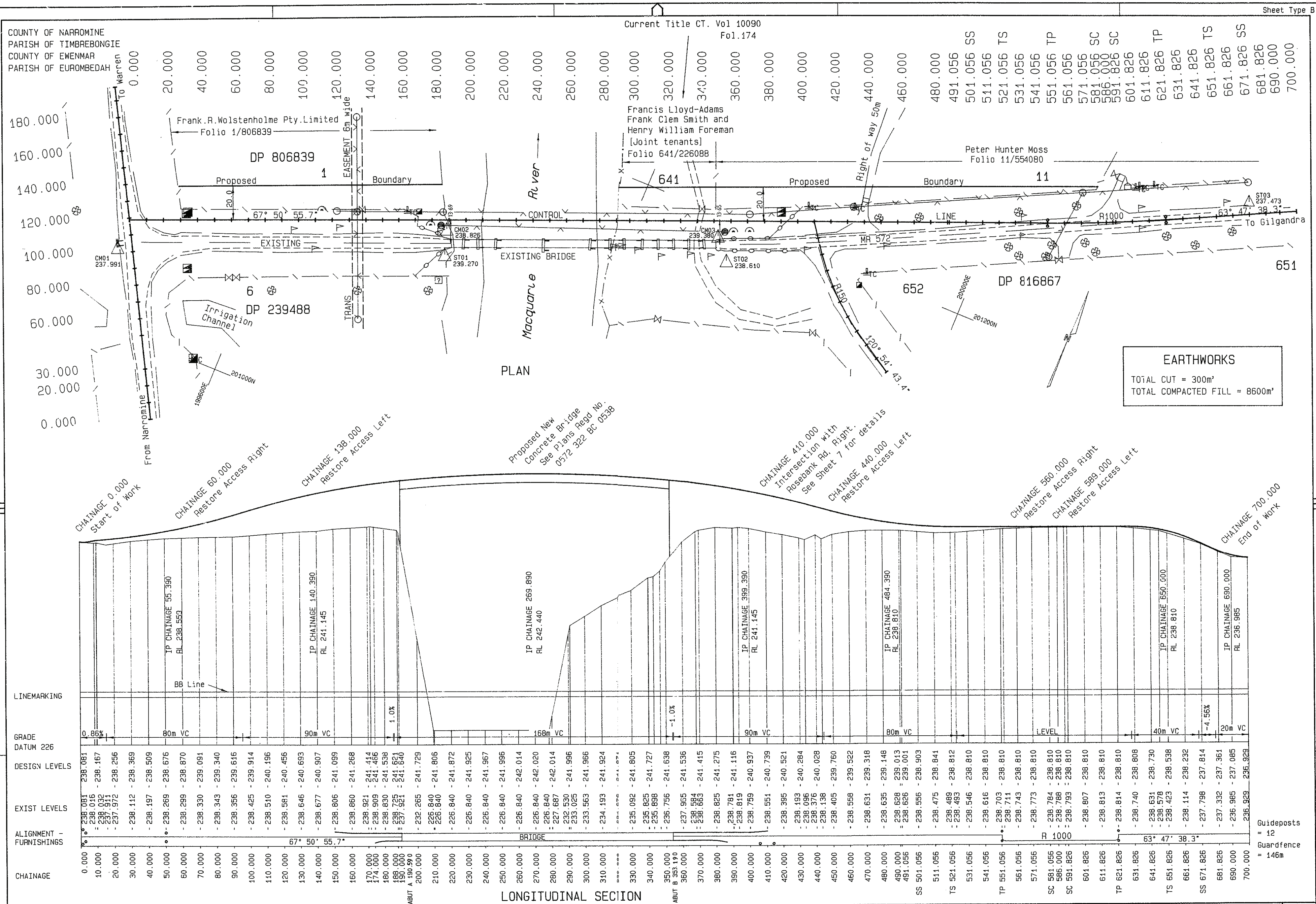
The freestanding gauge showing 12.0m to 16.0m has a zero of 223.98.

Thomas Casey

Thomas Casey
Registered Land Surveyor

ANNEXURE D
EUMUNGERIE ROAD BRIDGE DRAWINGS

Sheet Type B



EARTHWORKS

TOTAL CUT = 300m³
TOTAL COMPACTED FILL = 8600m³

No.	SHEET	AMENDMENT DETAIL	AUTH	DATE

SCALES: PLAN 20 10 0 20 40; LONG SECTION HORIZ 20 10 0 20 40; VERT 2 1 0 2 4

DATUM A H D
GRID I S G

COMPUTER FILE REFERENCES
DESIGNED USING MOSS VERSION : 9.0
ARCHIVE LOCATION :
SURVEY FILE : s54191
DESIGN FILE : d54191
DRAW FILE :
DOGS FILE : PLAN.DRG views 0,1,2
ROADLOC REF
CO-ORDS : START; R[572,10,A1,1.57] FINISH; R[572,20,A1,0.5]

PREPARED BY PARKES OPERATIONS BRANCH FOR WESTERN REGION
DESIGNED P.J.Thomas
CHECKED M.R.Pais
PASSED M.K.Hopood

THESE DRAWINGS HAVE BEEN EXAMINED AND ARE RECOMMENDED FOR APPROVAL
SIGNATURE
NAME R.T.Wagg
POSITION DESIGN MANAGER
DATE - 2 JUN 1993

APPROVED
SIGNATURE
NAME G.E.Kirchner
POSITION OPERATIONS MANAGER
DATE 3.6.'93

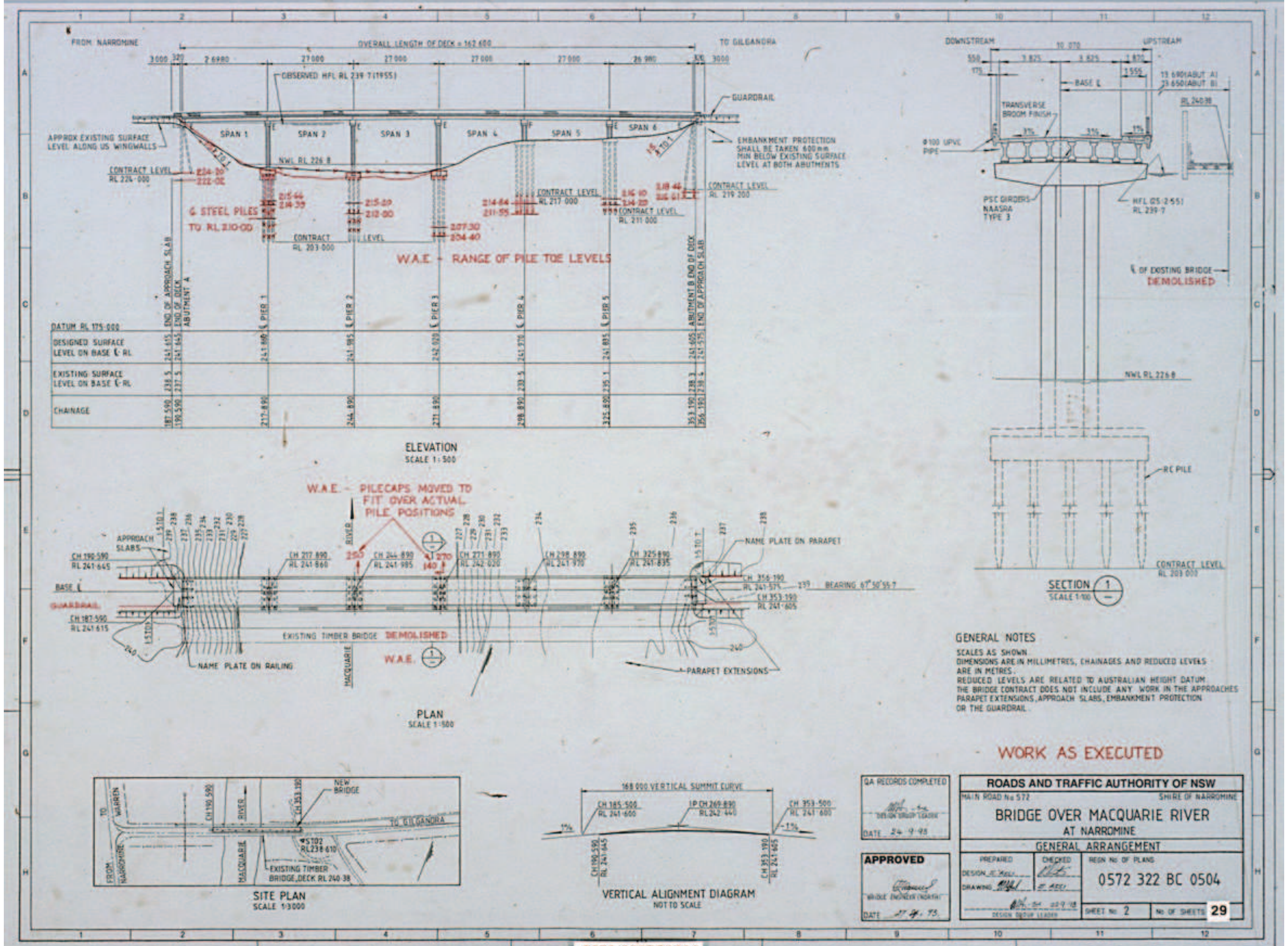
ROADS AND TRAFFIC AUTHORITY OF NSW

SHIRE OF NARROMINE, MR 572 EUMUNGERIE RD
APPROACHES TO THE NEW BRIDGE OVER THE MACQUARIE RIVER
AT NARROMINE PLAN AND LONGITUDINAL SECTIONS

FILE 322.118
REGN. 0572 322 RC 0540

SHEET 3 OF 8 SHEETS

REDUCTION RATIO


FROM NARROMINE
TO GILGANDRA
OVERALL LENGTH OF DECK = 162 600
OBSERVED HFL RL 239·7 (1955)
GUARDRAIL
SPAN 1
SPAN 2
SPAN 3
SPAN 4
SPAN 5
SPAN 6
APPROX EXISTING SURFACE LEVEL ALONG US WINGWALLS
NWL RL 226·8
EMBANKMENT PROTECTION SHALL BE TAKEN 600mm MIN BELOW EXISTING SURFACE LEVEL AT BOTH ABUTMENTS
CONTRACT LEVEL RL 224·000
CONTRACT LEVEL RL 219·200
CONTRACT LEVEL RL 217·000
CONTRACT LEVEL RL 211·000
CONTRACT LEVEL RL 203·000
G STEEL PILES TO RL 210·00
W.A.E. - RANGE OF PILE TOE LEVELS
DATUM RL 175·000
DESIGNED SURFACE LEVEL ON BASE ℄ RL
EXISTING SURFACE LEVEL ON BASE ℄ RL
CHAINAGE
ABUTMENT A
℄ PIER 1
℄ PIER 2
℄ PIER 3
℄ PIER 4
℄ PIER 5
ABUTMENT B END OF DECK
END OF APPROACH SLAB
ELEVATION
SCALE 1:500
W.A.E. - PILECAPS MOVED TO FIT OVER ACTUAL PILE POSITIONS
APPROACH SLABS
BASE ℄
GUARDRAIL
NAME PLATE ON RAILING
NAME PLATE ON PARAPET
EXISTING TIMBER BRIDGE DEMOLISHED
W.A.E.
PARAPET EXTENSIONS
BEARING 67°50'55"
RIVER
MACQUARIE
PLAN
SCALE 1:500
DOWNSTREAM
UPSTREAM
BASE ℄
TRANSVERSE BROOM FINISH
Ø100 UPVC PIPE
PSC GIRDERS NAASRA TYPE 3
HFL (05·2·55) RL 239·7
℄ OF EXISTING BRIDGE DEMOLISHED
NWL RL 226·8
RC PILE
CONTRACT LEVEL RL 203·000
SECTION 1
SCALE 1:100
GENERAL NOTES
SCALES AS SHOWN.
DIMENSIONS ARE IN MILLIMETRES, CHAINAGES AND REDUCED LEVELS ARE IN METRES.
REDUCED LEVELS ARE RELATED TO AUSTRALIAN HEIGHT DATUM.
THE BRIDGE CONTRACT DOES NOT INCLUDE ANY WORK IN THE APPROACHES PARAPET EXTENSIONS, APPROACH SLABS, EMBANKMENT PROTECTION OR THE GUARDRAIL.
WORK AS EXECUTED
NEW BRIDGE
TO GILGANDRA
TO WARREN
FROM NARROMINE
EXISTING TIMBER BRIDGE DECK RL 240·38
SITE PLAN
SCALE 1:3000
168 000 VERTICAL SUMMIT CURVE
VERTICAL ALIGNMENT DIAGRAM
NOT TO SCALE
QA RECORDS COMPLETED
APPROVED
ROADS AND TRAFFIC AUTHORITY OF NSW
MAIN ROAD No 572
SHIRE OF NARROMINE
BRIDGE OVER MACQUARIE RIVER
AT NARROMINE
GENERAL ARRANGEMENT
PREPARED
CHECKED
REGN No OF PLANS
0572 322 BC 0504
SHEET No 2
No OF SHEETS 29
0572 322BC0504
ROADS & TRAFFIC AUTHORITY N.S.W.

ANNEXURE E

FLOOD FREQUENCY ANALYSIS

E1 General

The design peak discharges adopted in the Narromine Flood Study of March 2009 and subsequently used in the FRMS report of November 2009 to assess the impacts of flooding and mitigation measures, were originally derived in the Narromine Flood Behaviour Study, 1998 by Bewsher Consulting. They were based on the results of a frequency analysis of the observed flood record following the construction of Burrendong Dam in 1965 (that is, the "post-dam" flood record).

Flood frequency analysis was selected as the method of deriving design floods in lieu of modelling the Macquarie Valley upstream of Narromine via a rainfall runoff-routing approach. The post-dam flood record includes the effects of upstream features influencing the magnitude of the flood peak at the town, namely: the storage contents of the dam at the commencement of the various floods and the procedures for operating the flood mitigation storage over the duration of the flood event, as well as tributary inflows from the uncontrolled catchment downstream of the dam (in particular from the important Talbragar and Little River tributaries). The results of the TUFLOW hydraulic model demonstrated that for floods up to the 1% AEP, peak flows leaving the Macquarie at Webbs Siding (shown on **Figure 1.1**) are not significant in comparison with the magnitude of flood peaks in the Macquarie River.

The post-dam flood frequency analysis was used to determine peak discharges for the various design floods of interest in the investigation of flooding at Narromine. They ranged between the 1% AEP and 0.5% AEP events. Design discharge hydrographs were derived by analogy with the temporal pattern of flows experienced for historic flood events, including the 1955 and 1990 floods.

E2 Flood Frequency Analysis

The hydrologic analysis for the present investigation was based on the approach adopted in the previous flood studies, namely peak flows were estimated from the post-dam frequency analysis with extension of the record to the present day. The extension gave a 45 year record of post-dam flood events for the frequency analysis.

As was the case for the previous investigations, it was concluded for the 2012 study that the effects of the various features which contribute to the Narromine flood peak, were incorporated in the end result, namely the recorded peak at the town. The relative importance of each feature clearly would vary for each individual flood event. However, a retrospective analysis of their relative magnitudes for each flood event would be a very expensive exercise requiring the development of a comprehensive catchment model of the Macquarie Valley upstream of Narromine. The data requirements for such an exercise to analyse each flood experienced over the post-dam period would be extensive and a lot of the historic flood data required to verify the catchment model may not be available.

E3 Available Streamflow Data

Post-dam streamflow data were available at the Narromine gauge (GS 421006) for the period 1966 to 1980 and at the Baroona gauge (GS 421127) from 1986 to the present. The Bureau of Meteorology and Narromine Council supplied data which allowed the estimation of flows for the intervening years from 1981 to 1985. The Baroona gauge is located about 12 km upstream of Narromine and is below the confluence with Coolbaggie Creek. There are no significant tributary inflows between the gauge and town and therefore, the discharge hydrograph at Baroona with a small lagging to allow for travel time, will represent the corresponding hydrograph at the upstream boundary of the hydraulic model.

NoW carried out gaugings near the peaks of both the August 1990 and December 2010 floods which were used to develop the high flow portion of the rating curve at Baroona and confirm the peak discharges for those events.

E4 Flood Frequency Results

A log-Pearson Type 3 distribution was fitted to the annual series of flood peaks. As the recorded flood peaks are only a small sample of peaks actually occurring over a longer duration, an expected probability adjustment was made using the procedure set out in Australian Rainfall and Runoff (ARR, 1998). ARR, 1998 recommends implementing the expected probability adjustment to remove bias from the estimate. The resulting frequency curves, along with 5% and 95% confidence limits are shown on **Figure E1** located at the end of this annexure.

Values at the low end of the observed range of flood peaks can distort the fitted probability distribution and affect the estimates of large floods. Deletion of these low values may improve the fitting of the remaining data. **Figure E2** shows the results of omitting the eight annual flows less than 100 m^3/s from the analysis.

Frequency analysis was also carried out fitting the annual peaks to the General Extreme Value (GEV) distribution using LH moments. **Figure E3** shows the results. The GEV distribution had been adopted by Sinclair Knight Merz for assigning peak flows at Narromine in its *Macquarie River (Narromine to Oxley Station) Floodplain Management Study, (SKM, 2008)*. That study of course pre-dated the occurrence of the December 2010 flood.

Table E1 shows the estimates of peak flows for various probabilities of occurrence as derived from the above analyses.

TABLE E1
ESTIMATES OF PEAK FLOWS AT NARROMINE
VALUES IN m^3/s

Annual Exceedance Probability % AEP	Annual Series*	Low Flows Removed*	GEV Distribution
5	1610	1750	1340
2	2720	2980	2135
1	3930	4390	2990
0.5	5800	5900	4100

* With the expected probability adjustment, according to ARR, 1998.

The results shown in **Table E1** show the estimated peak discharges for the post-dam 1% AEP ranging between 2990 m^3/s and 4390 m^3/s. The lowest estimate of 2990 m^3/s, as derived from the GEV distribution is considered to be on the low side. There have been two floods which have approximated a peak discharge of 2000 m^3/s over the 45 year post-dam period (August 1990 and December 2010). On this basis a flood peak of 2000 m^3/s would be expected to have an approximate probability of around 5% AEP, compared with the GEV estimate of 1340 m^3/s for that event. The annual series results for the 5% AEP peak are considerably higher and closer to the 2000 m^3/s value.

The *Floodplain Management Study for Dubbo* which was prepared by PPK and LACE in 1992 estimated the post-dam frequency of peak discharges at Dubbo at 1450 m^3/s for the 5% AEP flood and 3600 m^3/s for the 1% AEP. Both of these estimates are consistent with the peaks at Narromine derived by the log-Pearson Type 3 distribution, which incorporate contributions from the Talbragar River and would increase the peak between the two centres on a long term probability basis. Further, the multiplier of 2.5 between 5% AEP and 1% AEP flood peaks found to apply at Dubbo is the same as derived from the frequency analyses at Narromine, indicating that the 1% AEP peak discharge would be around 4000 m^3/s.

E5 Frequency of Historic Floods

E5.1 The February 1955 Flood

The February 1955 flood is the flood of record and resulted in a peak of 14.94 m on the gauge at Narromine. The *Narromine Flood Behaviour Study, 1998* carried out a “pre-dam” frequency analysis of the flood record for the years 1901-1964 and concluded that the February 1955 flood, which had an estimated peak discharge of 5800 m^3/s, had a 1% AEP frequency at the time of its occurrence. Comparison of the pre- and post- dam frequency curves presented in the FRMS, 2009 showed that the large flood mitigation storage in Burrendong Dam had an impact in reducing flood peaks as far downstream as Narromine.

The occurrence of flows in the Macquarie River approaching Narromine similar to those of February 1955 would represent a much rarer event under post-dam conditions. On the basis of the values in **Table E1** it would have a 0.5% AEP.

E5.2 The August 1990 Flood

The recorded peak discharge of the August 1990 flood at the Baroona gauge was 2078 m^3/s. A discharge of that magnitude is equivalent to a 3.3% AEP (1 in 30 year return period) according to the Log Pearson Type 3 distribution after adjustment for expected probability (refer blue line on **Figure E1**). This probability compares with the 1.5% AEP (1 in 65 years return period) assigned by SKM, 2008 to the August 1990 flood.

E5.3 The December 2010 Flood

The recorded peak discharge of the December 2010 flood at the Baroona gauge was 2200 m^3/s. A discharge of that magnitude is equivalent to a 3% AEP (1 in 33 year return period) according to the Log Pearson Type 3 distribution after adjustment for expected probability (refer blue line on **Figure E1**).

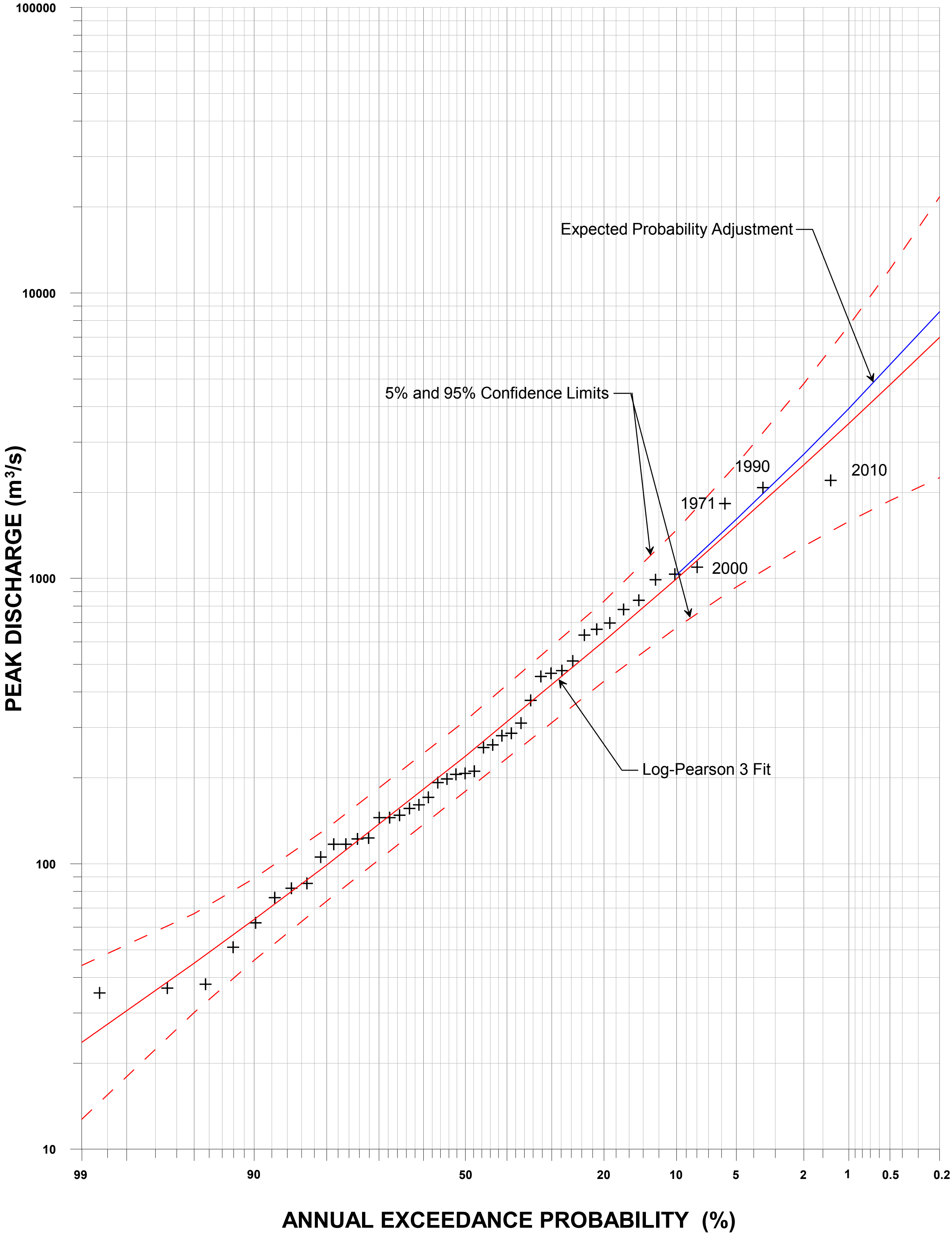
E6 Adopted Design Floods

From the above analyses it is considered that either of the Log Pearson Type 3 distributions should be adopted in preference to the GEV distribution.

Updating the frequency analysis to incorporate the December 2010 event had the effect of a small upwards revision in the estimate of the design 1% AEP discharge compared with the results of the previous studies. In both the 1998 and 2009 investigations a value of 3800 m^3/s was adopted as the peak of the 1% AEP flood. **Table E2** over shows the adopted design flood peaks. For the hydraulic modelling described in **Chapter 3**, the 1% AEP flood peak has been rounded up to 4000 m^3/s, which would also allow a small increase for future climate change.

TABLE E2
ADOPTED POST-DAM DESIGN PEAK FLOWS AT NARROMINE
VALUES IN m^3/s

Annual Exceedance Probability %AEP	Annual Series
5	1610
2	2720
1	4000
0.5	5800

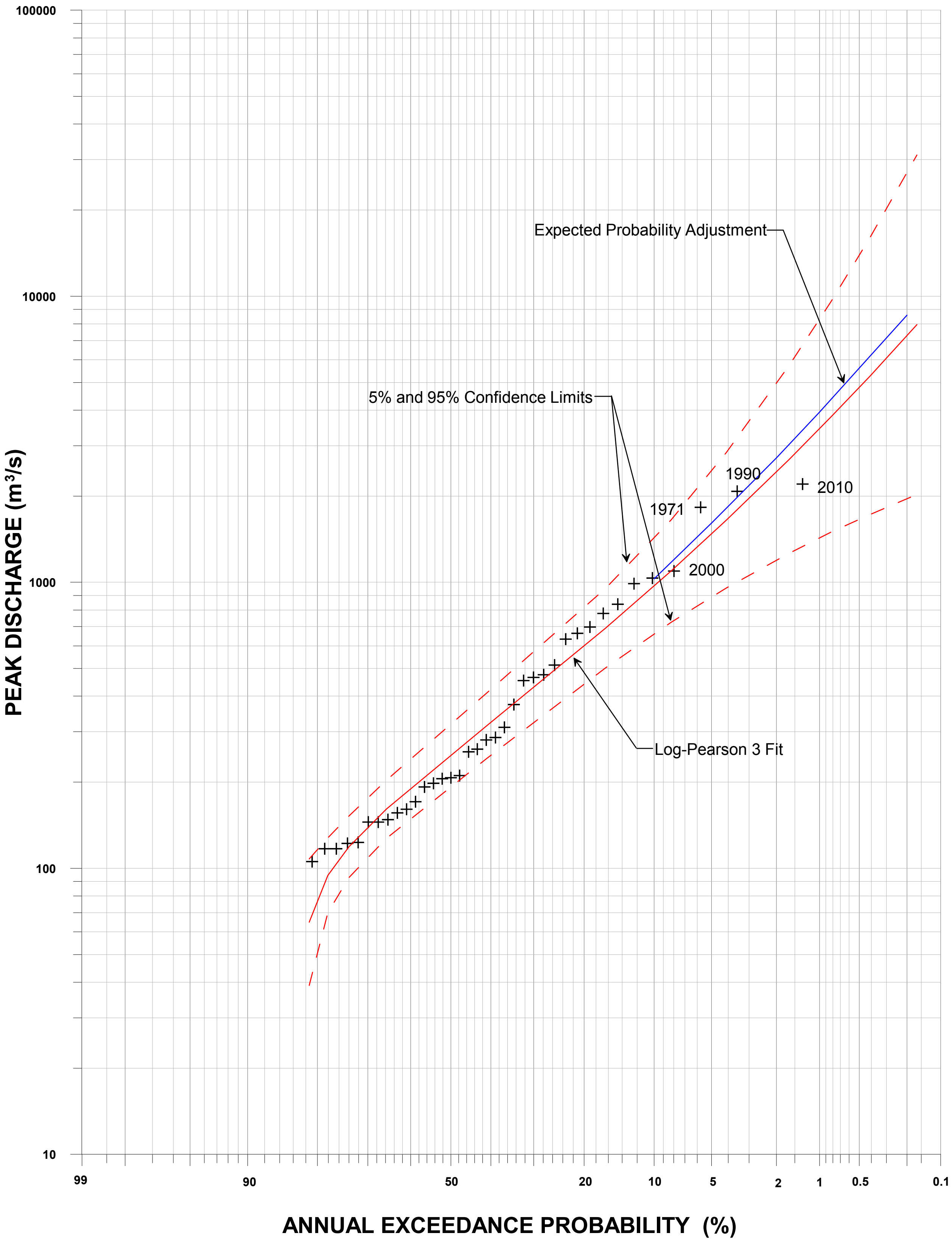


Lyall & Associates

NARROMINE RIVER BANK LEVEE FEASIBILITY STUDY

Figure E1

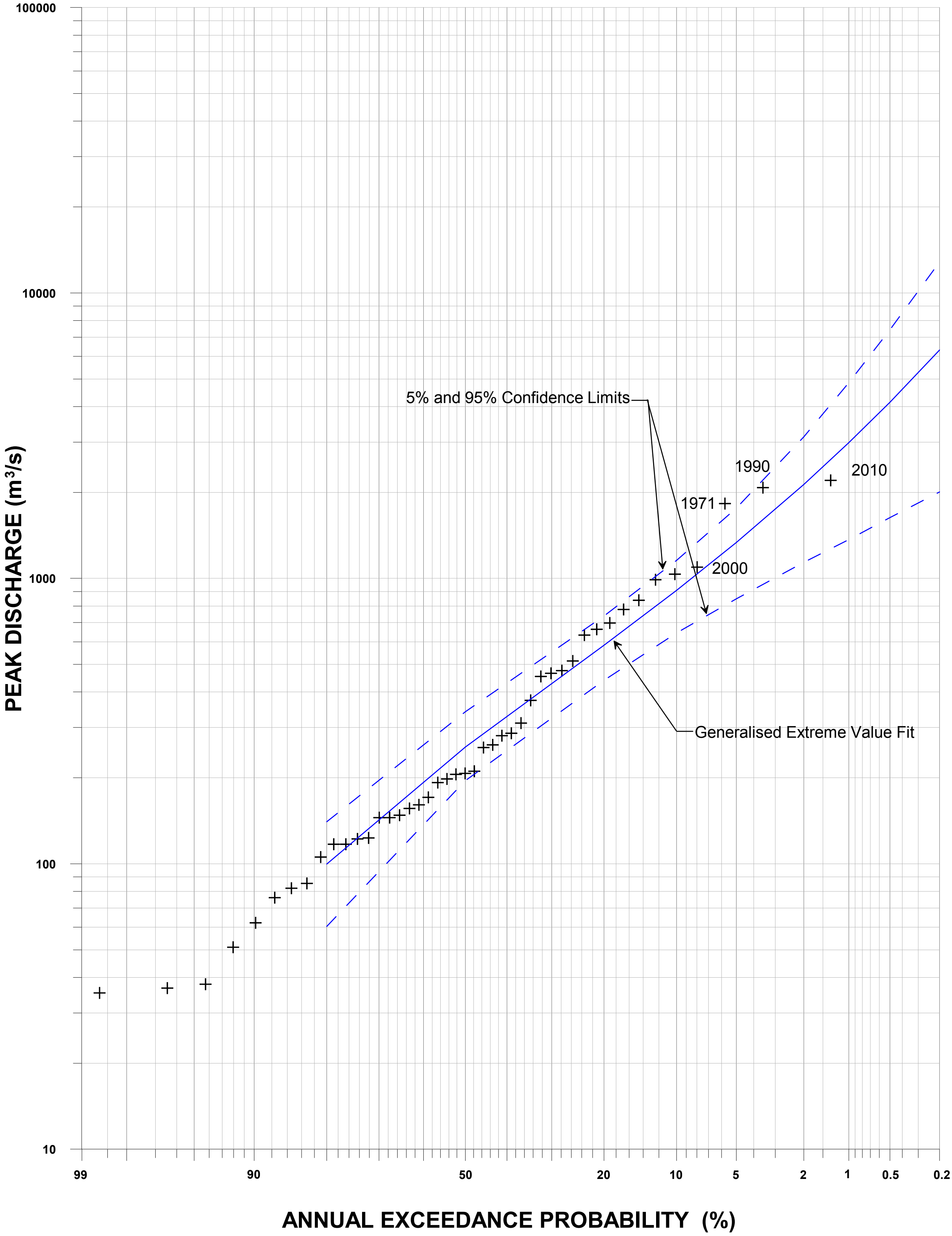
FLOOD FREQUENCY RELATIONSHIP
LOG-PEARSON 3 ANNUAL SERIES 1966-2010



NARROMINE RIVER BANK LEVEE
FEASIBILITY STUDY

Figure E2

FLOOD FREQUENCY RELATIONSHIP
LOG-PEARSON 3 ANNUAL SERIES 1966-2010
NO LOW FLOWS



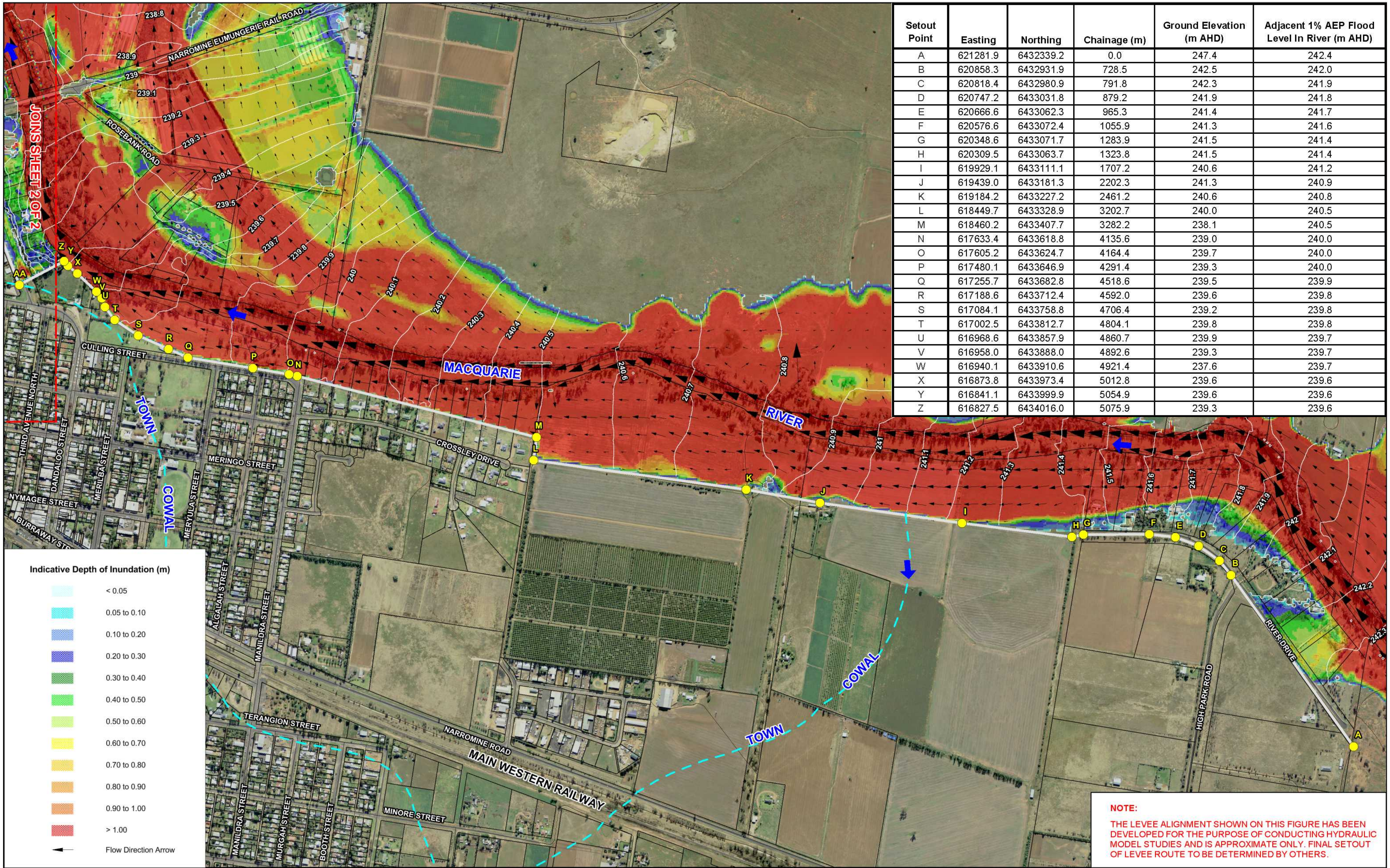
NARROMINE RIVER BANK LEVEE FEASIBILITY STUDY

Figure E3

FLOOD FREQUENCY RELATIONSHIP
GENERALISED EXTREME VALUE ANNUAL SERIES 1966-2010

ANNEXURE F
SETOUT DETAILS FOR LEVEE OPTION 2A(i)

Setout Point	Easting	Northing	Chainage (m)	Ground Elevation (m AHD)	Adjacent 1% AEP Flood Level In River (m AHD)
A	621281.9	6432339.2	0.0	247.4	242.4
B	620858.3	6432931.9	728.5	242.5	242.0
C	620818.4	6432980.9	791.8	242.3	241.9
D	620747.2	6433031.8	879.2	241.9	241.8
E	620666.6	6433062.3	965.3	241.4	241.7
F	620576.6	6433072.4	1055.9	241.3	241.6
G	620348.6	6433071.7	1283.9	241.5	241.4
H	620309.5	6433063.7	1323.8	241.5	241.4
I	619929.1	6433111.1	1707.2	240.6	241.2
J	619439.0	6433181.3	2202.3	241.3	240.9
K	619184.2	6433227.2	2461.2	240.6	240.8
L	618449.7	6433328.9	3202.7	240.0	240.5
M	618460.2	6433407.7	3282.2	238.1	240.5
N	617633.4	6433618.8	4135.6	239.0	240.0
O	617605.2	6433624.7	4164.4	239.7	240.0
P	617480.1	6433646.9	4291.4	239.3	240.0
Q	617255.7	6433682.8	4518.6	239.5	239.9
R	617188.6	6433712.4	4592.0	239.6	239.8
S	617084.1	6433758.8	4706.4	239.2	239.8
T	617002.5	6433812.7	4804.1	239.8	239.8
U	616968.6	6433857.9	4860.7	239.9	239.7
V	616958.0	6433888.0	4892.6	239.3	239.7
W	616940.1	6433910.6	4921.4	237.6	239.7
X	616873.8	6433973.4	5012.8	239.6	239.6
Y	616841.1	6433999.9	5054.9	239.6	239.6
Z	616827.5	6434016.0	5075.9	239.3	239.6



NOTE:

THE LEVEE ALIGNMENT SHOWN ON THIS FIGURE HAS BEEN DEVELOPED FOR THE PURPOSE OF CONDUCTING HYDRAULIC MODEL STUDIES AND IS APPROXIMATE ONLY. FINAL SETOUT OF LEVEE ROUTE TO BE DETERMINED BY OTHERS.

N

120 0 120 240 360 m

Scale: 1:12,000

Lyall & Associates

NOT FOR CONSTRUCTION

LEGEND

A Setout Point

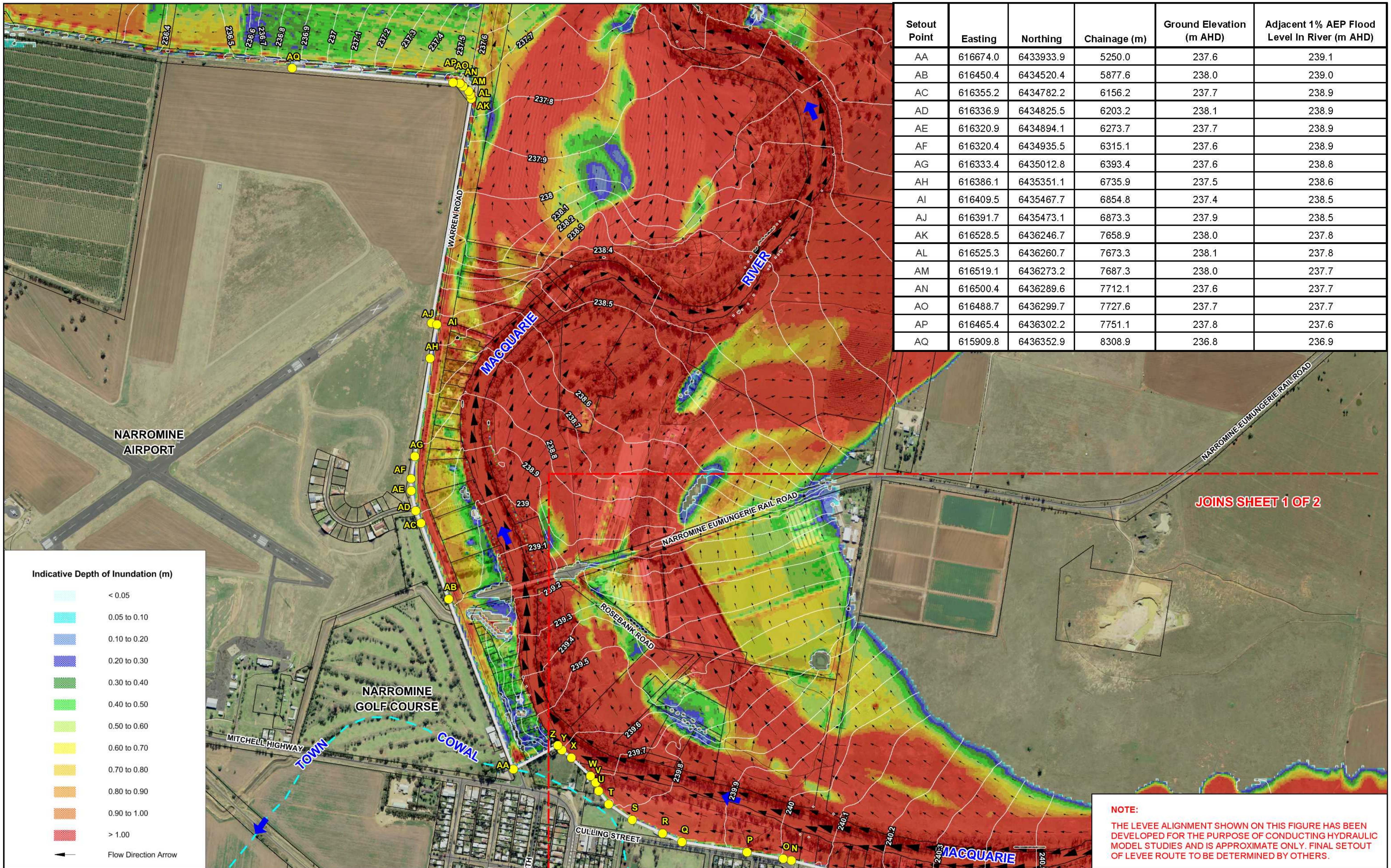
Levee Route

242.5 Water Surface Elevation (m AHD)

NARROMINE RIVER BANK LEVEE FEASIBILITY STUDY

Figure F1
(Sheet 1 of 2)

SETOUT DETAILS FOR LEVEE OPTION 2A(i)
1% AEP (4000 m³/s ESTIMATE)



Setout Point	Easting	Northing	Chainage (m)	Ground Elevation (m AHD)	Adjacent 1% AEP Flood Level In River (m AHD)
AA	616674.0	6433933.9	5250.0	237.6	239.1
AB	616450.4	6434520.4	5877.6	238.0	239.0
AC	616355.2	6434782.2	6156.2	237.7	238.9
AD	616336.9	6434825.5	6203.2	238.1	238.9
AE	616320.9	6434894.1	6273.7	237.7	238.9
AF	616320.4	6434935.5	6315.1	237.6	238.9
AG	616333.4	6435012.8	6393.4	237.6	238.8
AH	616386.1	6435351.1	6735.9	237.5	238.6
AI	616409.5	6435467.7	6854.8	237.4	238.5
AJ	616391.7	6435473.1	6873.3	237.9	238.5
AK	616528.5	6436246.7	7658.9	238.0	237.8
AL	616525.3	6436260.7	7673.3	238.1	237.8
AM	616519.1	6436273.2	7687.3	238.0	237.7
AN	616500.4	6436289.6	7712.1	237.6	237.7
AO	616488.7	6436299.7	7727.6	237.7	237.7
AP	616465.4	6436302.2	7751.1	237.8	237.6
AQ	615909.8	6436352.9	8308.9	236.8	236.9

120 0 120 240 360 m
Scale: 1:12,000

Lyall & Associates

NOTE:

LOCATION WHERE LEVEE CROSSES WARREN ROAD TO BE LOCALLY ADJUSTED TO COINCIDE WITH CREST IN ROAD.

NOT FOR CONSTRUCTION

LEGEND

AQ Setout Point

Levee Route

Water Surface Elevation (m AHD)

NARROMINE RIVER BANK LEVEE FEASIBILITY STUDY

Figure F1
(Sheet 2 of 2)

SETOUT DETAILS FOR LEVEE OPTION 2A(i)
1% AEP (4000 m^3/s ESTIMATE)

ANNEXURE G
PEAK FLOOD LEVELS AT SELECTED PROPERTIES

TABLE G1
PEAK FLOOD LEVELS AT SELECTED PROPERTIES[1]
1% AEP

Point No#	Street Name	House No	Floor Level (m AHD)	Peak Flood Level (m AHD)[2]								Depth of Above Floor Flooding (m)[3]							
				Present Day	Levee Option 1	Levee Option 2	Levee Option 2A	Levee Option 2A(i)	Levee Option 2B	Levee Option 2C	Levee Option 3	Present Day	Levee Option 1	Levee Option 2	Levee Option 2A	Levee Option 2A(i)	Levee Option 2B	Levee Option 2C	Levee Option 3
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]	[O]	[P]	[Q]	[R]	[S]	[T]
H01	Warren Road	20	239.39	239.38	NF	NF	NF	239.44	NF	239.49	NF	-0.01	-	-	-	0.05	-	0.10	-
H02	Warren Road	16	239.40	NF	NF	NF	NF	239.46	NF	239.53	NF	-	-	-	-	0.06	-	0.13	-
H03	Warren Road	24	239.83	NF	NF	NF	NF	NF	NF	239.46	NF	-	-	-	-	-	-	-0.37	-
H04	Warren Road	28	239.59	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H05	Warren Road	34	239.26	239.27	NF	NF	NF	239.33	NF	239.39	NF	0.01	-	-	-	0.07	-	0.13	-
H06	Warren Road	38	239.25	239.23	NF	NF	NF	239.30	NF	239.36	NF	-0.02	-	-	-	0.05	-	0.11	-
H07	Warren Road	44	239.29	239.09	NF	NF	NF	239.17	NF	239.32	NF	-0.20	-	-	-	-0.12	-	0.03	-
H08	Warren Road	46	238.97	239.10	NF	NF	NF	239.18	NF	239.29	NF	0.13	-	-	-	0.21	-	0.32	-
H09	Warren Road	48	238.91	239.04	NF	NF	NF	239.13	NF	239.29	NF	0.13	-	-	-	0.22	-	0.38	-
H10	Warren Road	48	238.79	239.13	NF	NF	NF	239.20	NF	239.29	NF	0.34	-	-	-	0.41	-	0.50	-
H11	Warren Road	50	238.88	239.02	NF	NF	NF	239.12	NF	239.29	NF	0.14	-	-	-	0.24	-	0.41	-
H12	Warren Road	52	239.07	238.63	238.72	238.72	238.77	238.99	NF	238.72	238.72	-0.44	-0.35	-0.35	-0.30	-0.08	-	-0.35	-0.35
H13	Warren Road	58	239.09	NF	NF	NF	NF	238.96	NF	NF	NF	-	-	-	-	-0.13	-	-	-
H14	Warren Road	62	239.20	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H15	Warren Road	66	239.31	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H16	Warren Road	70	239.13	238.64	238.69	238.69	238.76	238.88	NF	238.69	238.69	-0.49	-0.44	-0.44	-0.37	-0.25	-	-0.44	-0.44
H17	Warren Road	74	238.84	238.71	238.75	238.75	238.78	238.87	NF	238.75	238.75	-0.13	-0.09	-0.09	-0.06	0.03	-	-0.09	-0.09
H18	Warren Road	78	238.84	238.70	238.74	238.75	238.78	238.86	NF	238.74	238.74	-0.14	-0.10	-0.09	-0.06	0.02	-	-0.10	-0.10
H19	Warren Road	82	238.74	238.71	238.75	238.75	238.78	238.84	NF	238.75	238.75	-0.03	0.01	0.01	0.04	0.10	-	0.01	0.01
H20	Warren Road	86	238.50	238.71	238.75	238.75	238.78	238.82	NF	238.75	238.75	0.21	0.25	0.25	0.28	0.32	-	0.25	0.25
H21	Warren Road	90	238.75	238.55	238.58	238.58	238.66	238.75	NF	238.58	238.58	-0.20	-0.17	-0.17	-0.09	0.00	-	-0.17	-0.17
H22	Warren Road	94	238.54	238.53	238.56	238.56	238.66	238.73	NF	238.56	238.56	-0.01	0.02	0.02	0.12	0.19	-	0.02	0.02
H23	Warren Road	98	238.04	238.64	238.69	238.69	238.73	238.75	NF	238.69	238.69	0.60	0.65	0.65	0.69	0.71	-	0.65	0.65

1. Location of surveyed properties is shown on **Figure A2** in **Annexure A**.
2. NF = Not Flooded
3. A negative value indicates water level lies below the surveyed floor level.

TABLE G1 (Cont'd)
PEAK FLOOD LEVELS AT SELECTED PROPERTIES[1]
1% AEP

Point No#	Street Name	House No	Floor Level (m AHD)	Peak Flood Level (m AHD)[2]								Depth of Above Floor Flooding (m)[3]							
				Present Day	Levee Option 1	Levee Option 2	Levee Option 2A	Levee Option 2A(i)	Levee Option 2B	Levee Option 2C	Levee Option 3	Present Day	Levee Option 1	Levee Option 2	Levee Option 2A	Levee Option 2A(i)	Levee Option 2B	Levee Option 2C	Levee Option 3
H24	Warren Road	102	238.53	238.57	238.62	238.62	238.68	238.71	NF	238.62	238.62	0.04	0.09	0.09	0.15	0.18	-	0.09	0.09
H25	Warren Road	106	239.51	238.52	238.57	238.57	238.64	238.67	NF	238.57	238.57	-0.99	-0.94	-0.94	-0.87	-0.84	-	-0.94	-0.94
H26	Warren Road	114	238.65	238.40	238.43	238.44	238.55	238.57	238.49	238.43	238.44	-0.25	-0.22	-0.21	-0.10	-0.08	-0.16	-0.22	-0.21
H27	Warren Road	118	238.55	238.40	238.45	238.45	238.49	238.49	238.48	238.45	238.45	-0.15	-0.10	-0.10	-0.06	-0.06	-0.07	-0.10	-0.10
H28	Warren Road	122	238.09	238.00	238.04	238.04	238.17	238.18	238.12	238.04	238.04	-0.09	-0.05	-0.05	0.08	0.09	0.03	-0.05	-0.05
H29	River Drive	413	242.14	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H30	River Drive	391	241.78	242.23	NF	242.28	242.29	242.29	242.29	242.28	242.28	0.45	-	0.50	0.51	0.51	0.51	0.50	0.50
H31	River Drive	381	242.51	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H32	River Drive	369	241.97	242.09	NF	242.13	242.13	242.13	242.13	242.13	242.13	0.12	-	0.16	0.16	0.16	0.16	0.16	0.16
H33	River Drive	369	242.45	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H34	River Drive	361	242.59	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H35	River Drive	361	241.81	241.99	NF	242.03	242.04	242.03	242.04	242.03	242.03	0.18	-	0.22	0.23	0.22	0.23	0.22	0.22
H36	River Drive	359	241.63	241.94	NF	241.99	241.99	241.99	241.99	241.99	241.98	0.31	-	0.36	0.36	0.36	0.36	0.36	0.35
H37	River Drive	349	241.36	241.86	NF	241.91	241.91	241.91	241.91	241.91	241.91	0.50	-	0.55	0.55	0.55	0.55	0.55	0.55
H38	River Drive	303	240.22	241.40	NF	241.46	241.46	241.46	241.46	241.46	241.46	1.18	-	1.24	1.24	1.24	1.24	1.24	1.24
H39	River Drive	317	241.46	NF	NF	241.53	241.53	241.53	241.53	241.53	241.53	-	-	0.07	0.07	0.07	0.07	0.07	0.07
H40	River Drive	327	241.67	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H41	River Drive	329	241.77	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H42	River Drive	337	241.72	241.73	NF	241.77	241.77	241.77	241.77	241.77	241.77	0.01	-	0.05	0.05	0.05	0.05	0.05	0.05
H43	Warren Road	224	237.88	NF	237.48	237.48	237.54	237.54	237.53	237.48	237.48	-	-0.40	-0.40	-0.34	-0.34	-0.35	-0.40	-0.40
H44	Burroway Road	121	237.85	237.70	237.74	237.74	237.76	237.75	237.76	237.74	237.74	-0.15	-0.11	-0.11	-0.09	-0.10	-0.09	-0.11	-0.11
H45	Burroway Road	121	237.68	237.69	237.73	237.73	237.74	237.74	237.74	237.73	237.73	0.01	0.05	0.05	0.06	0.06	0.06	0.05	0.05
H46	Macquarie View Road	92	241.39	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-

1. Location of surveyed properties is shown on **Figure A2** in **Annexure A**.
2. NF = Not Flooded
3. A negative value indicates water level lies below the surveyed floor level.

TABLE G1 (Cont'd)
PEAK FLOOD LEVELS AT SELECTED PROPERTIES[1]
1% AEP

Point No#	Street Name	House No	Floor Level (m AHD)	Peak Flood Level (m AHD)[2]								Depth of Above Floor Flooding (m)[3]							
				Present Day	Levee Option 1	Levee Option 2	Levee Option 2A	Levee Option 2A(i)	Levee Option 2B	Levee Option 2C	Levee Option 3	Present Day	Levee Option 1	Levee Option 2	Levee Option 2A	Levee Option 2A(i)	Levee Option 2B	Levee Option 2C	Levee Option 3
H47	Rosebank Road	66	241.66	239.49	239.55	239.55	239.56	239.56	239.56	239.55	239.55	-2.17	-2.11	-2.11	-2.10	-2.10	-2.10	-2.11	-2.11
H48	Rosebank Road	72	242.32	239.60	239.65	239.66	239.66	239.66	239.66	239.65	239.66	-2.72	-2.67	-2.66	-2.66	-2.66	-2.66	-2.67	-2.66
H49	Rosebank Road	78	242.23	239.63	239.66	239.66	239.67	239.66	239.68	239.66	239.66	-2.60	-2.57	-2.57	-2.56	-2.57	-2.55	-2.57	-2.57
H50	Rosebank Road	82	242.07	239.75	239.81	239.81	239.82	239.81	239.82	239.81	239.81	-2.32	-2.26	-2.26	-2.26	-2.26	-2.25	-2.26	-2.26
H51	Rosebank Road	81	242.26	239.49	239.55	239.55	239.56	239.56	239.56	239.55	239.55	-2.77	-2.71	-2.71	-2.70	-2.70	-2.70	-2.71	-2.71
H53	Rosebank Road	57	241.45	239.34	239.41	239.41	239.42	239.42	239.43	239.41	239.41	-2.11	-2.04	-2.04	-2.03	-2.03	-2.02	-2.04	-2.04
H54	Rosebank Road	53	241.35	239.33	239.40	239.40	239.41	239.41	239.41	239.40	239.40	-2.02	-1.95	-1.95	-1.94	-1.94	-1.94	-1.95	-1.95
H55	Rosebank Road	49	241.54	239.32	239.39	239.39	239.40	239.39	239.40	239.39	239.39	-2.22	-2.15	-2.15	-2.14	-2.15	-2.14	-2.15	-2.15
H56	Eumungerie Road	218	240.42	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H57	Eumungerie Road	218	240.09	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H58	Eumungerie Road	218	239.84	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H59	Eumungerie Road	218	239.83	NF	NF	NF	NF	NF	NF	NF	NF	-	-	-	-	-	-	-	-
H60	Eumungerie Road	120	238.78	239.04	239.10	239.10	239.11	239.11	239.12	239.10	239.10	0.26	0.32	0.32	0.33	0.33	0.34	0.32	0.32
H61	Eumungerie Road	120	238.82	239.08	239.14	239.14	239.15	239.15	239.16	239.14	239.14	0.26	0.32	0.32	0.33	0.33	0.34	0.32	0.32

1. Location of surveyed properties is shown on **Figure A2** in **Annexure A**.
2. NF = Not Flooded
3. A negative value indicates water level lies below the surveyed floor level.