3 Hydraulic modelling

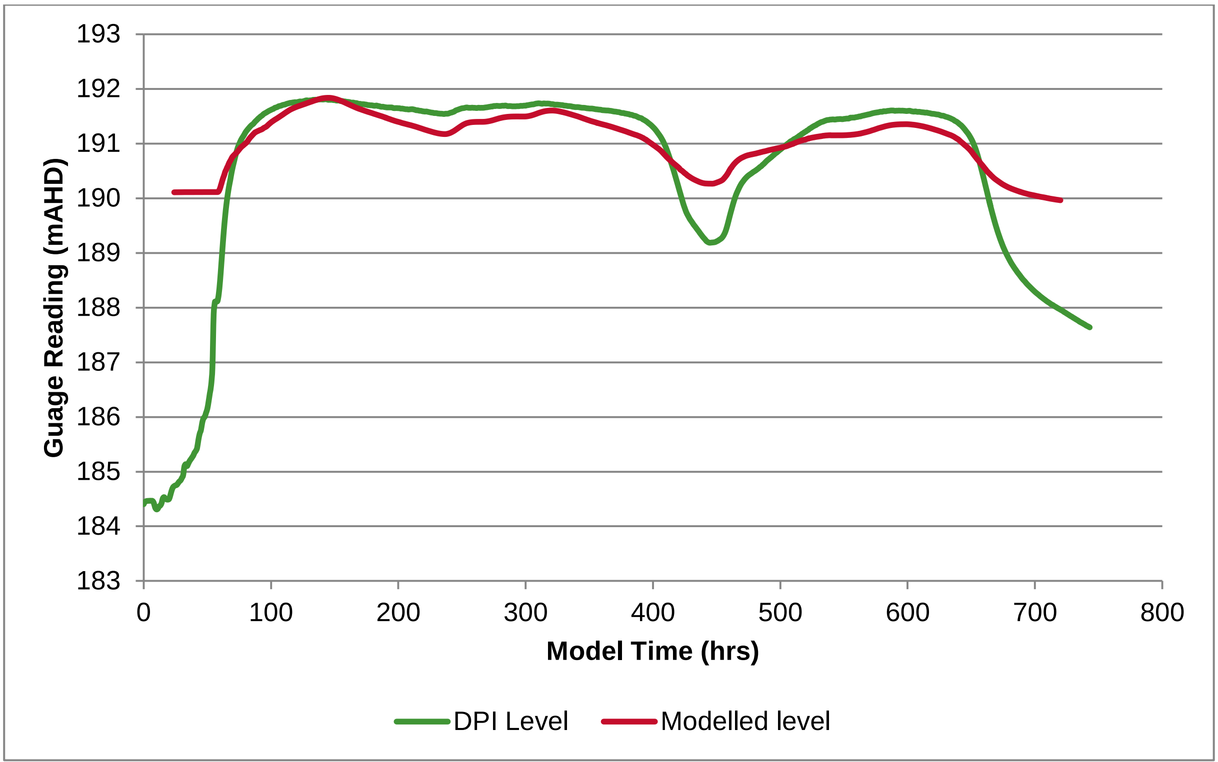


Figure 3-5 Observed vs. modelled flood levels at the Gunidgera gauge for the 1998 event

As can be seen in Figure 3-5 there is good correlation between the 1998 simulated hydrograph and observed hydrograph in terms of height and timing. The difference between the observed and modelled flood level peaks varies no more than 0.16m. No further calibration has been carried out because a lack of information means the structure and operation of the Gunidgera weir could not be improved. Currently the structure is not included in the model and therefore the effects of its operation are also not represented. If operational data were available for the structure, the model response could potentially be improved; however the limited recorded gauge data at the Gundigera gauge would prevent a more thorough verification exercise.

The other discrepancy between the modelled and observed levels is the drop in recorded water level at approximately 450 hours into the model run. It is thought that this may be caused by the operation of the Gunidgera weir. With no data on gate movements, there is no way to verify this assumption.

Further verification of the TUFLOW model was achieved by comparing the TUFLOW model results against the Mike 11 model results for the 1998 flood at Gunidgera. This is discussed in detail in Appendix C; however the comparison shows that the TUFLOW model results are approximately 0.3m lower than the Mike 11 results. It is known that 1D models can over estimate flood levels when out of bank flow occurs and therefore this is considered a successful verification.

To verify the TUFLOW model for the 1971 and 1984 floods, observed spot flood levels were compared to modelled spot flood levels at locations around the Wee Waa levee. These comparisons are presented in Figure 3-6 and Figure 3-7.

3 Hydraulic modelling

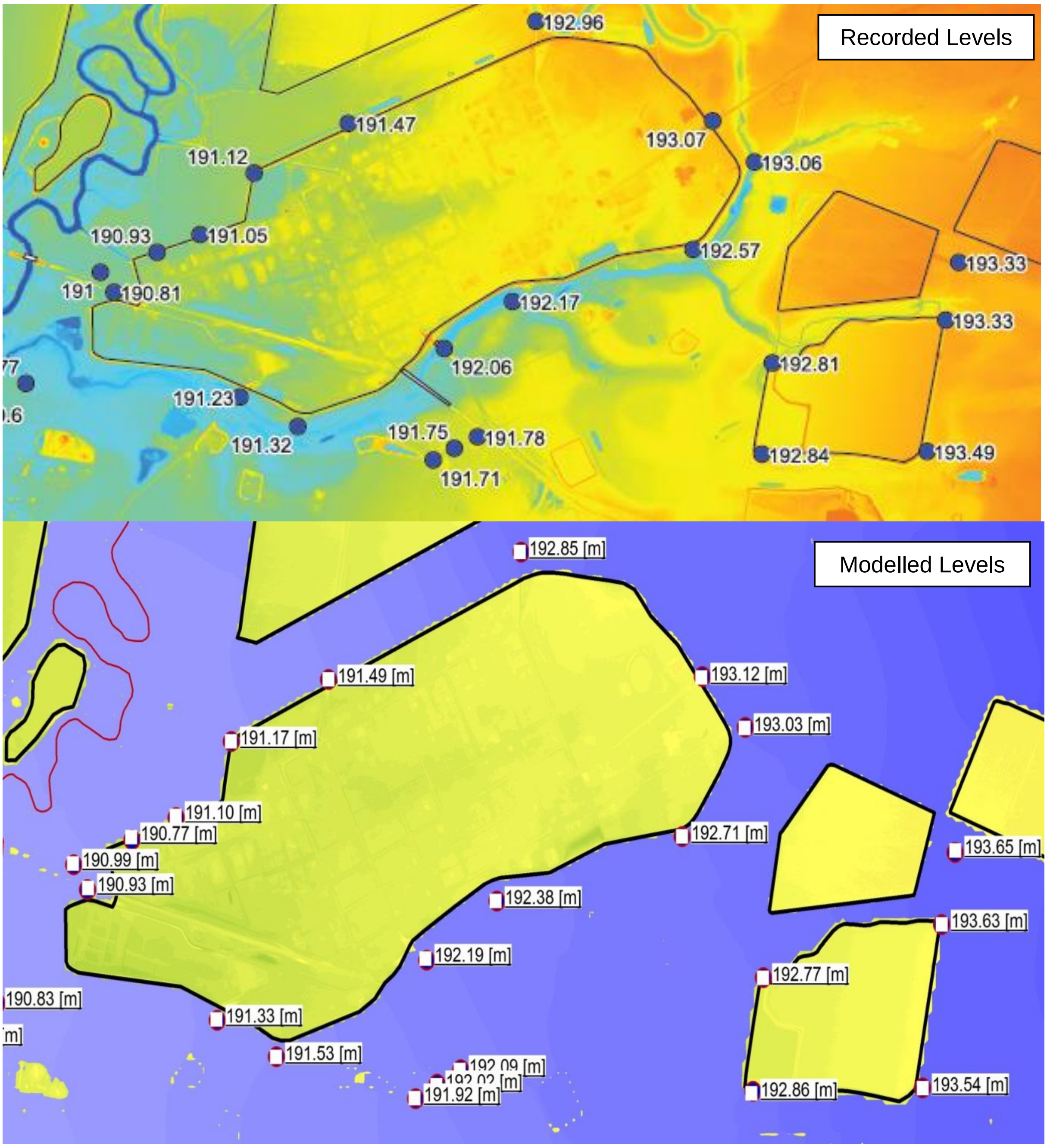


Figure 3-6 Comparison of recorded flood levels with modelled flood levels for the 1984 flood

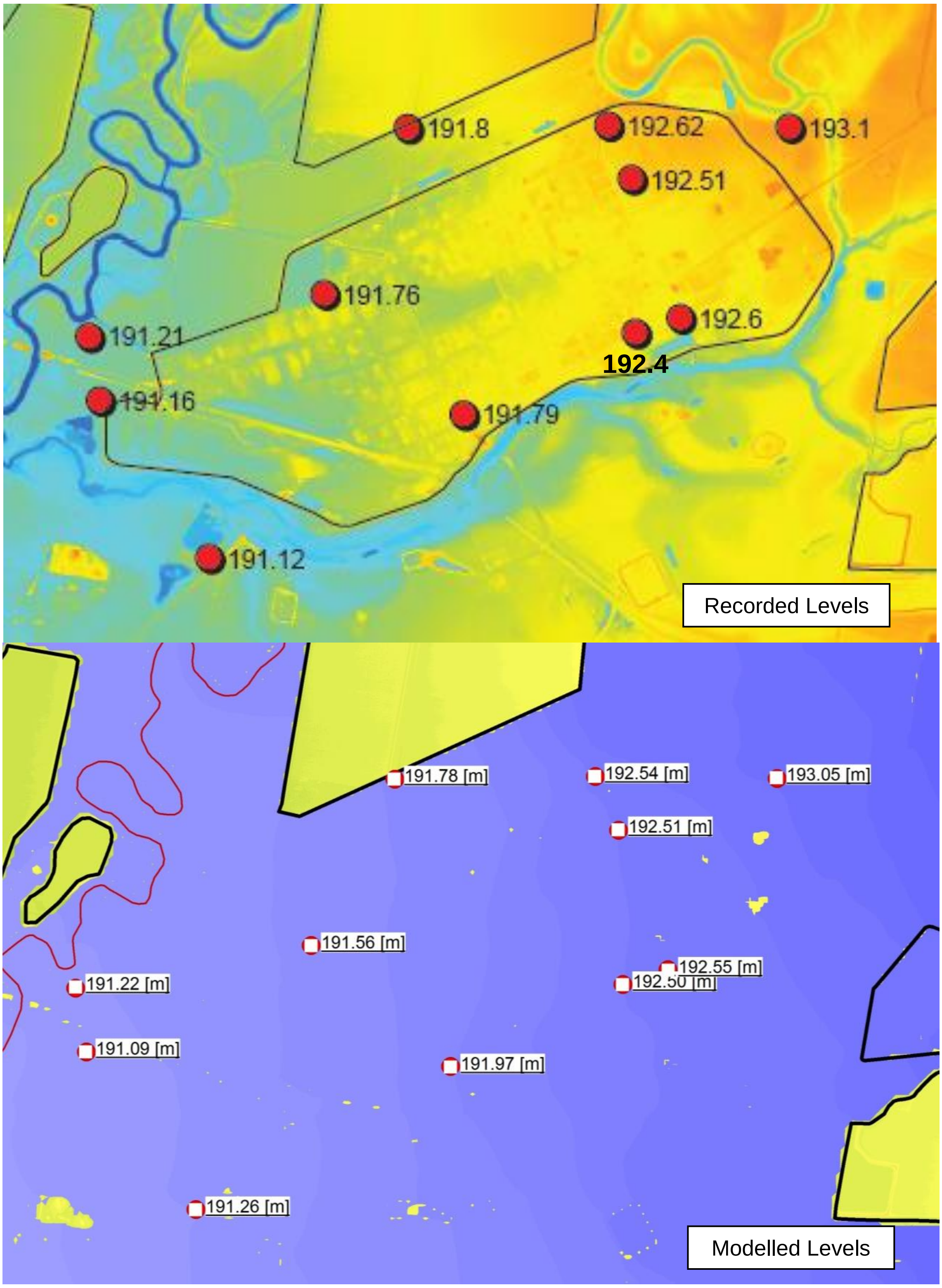


Figure 3-7 Comparison of recorded flood levels with modelled flood levels for the 1971 flood

As can be seen the TUFLOW model is providing very similar answers to observed flood peaks for both the 1971 flood and the 1984 floods. The majority of points are within 0.1m of the recorded levels, with a small number of points showing a larger difference of up to 0.3m. As described earlier, no levee existed during the 1971 event so it was removed from the model.

3.6 Sensitivity analysis

A sensitivity analysis was undertaken by running the models with all of the roughness values increased and decreased by 20%. Further sensitivity analysis was undertaken by increasing decreasing the flow in the Namoi River by 10%. Figure 3-8 shows the observed flood levels compared to the modelled flood levels at the Gunidgera gauge, with varying roughness values. Figure 3-9 shows the observed flood levels compared to the modelled flood levels with varying flow.

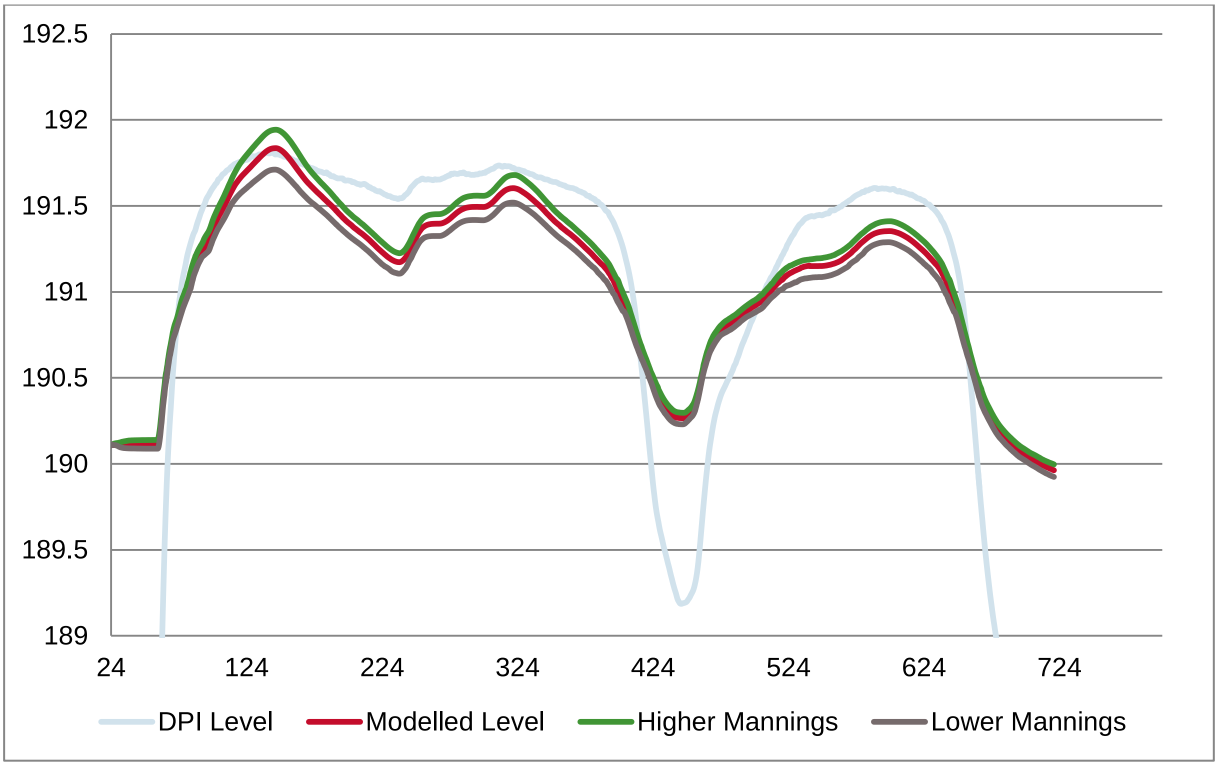


Figure 3-8 Sensitivity analysis for the 1998 flood at Gunidgera gauge by varying manning's n

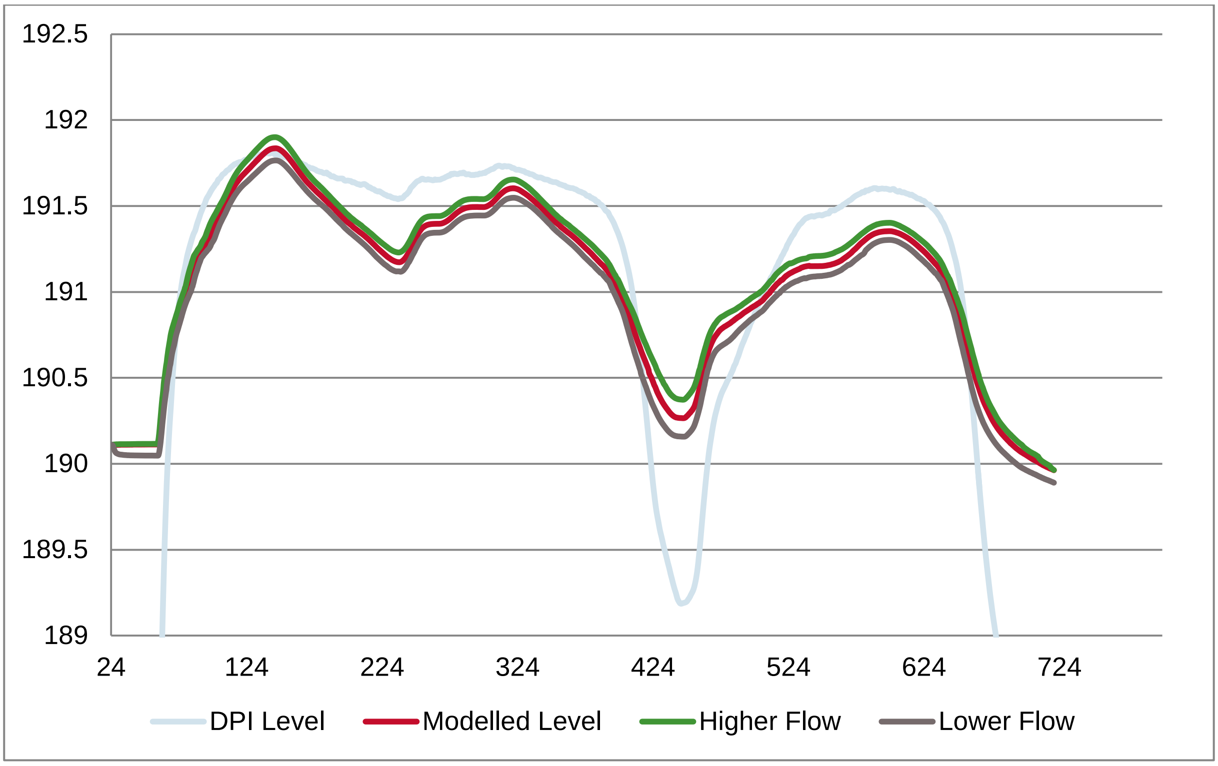


Figure 3-9 Sensitivity analysis for the 1998 flood at Gunidgera gauge by varying flow

It can be seen that the water levels across the model showed even and minor changes for the sensitivity runs. This suggests that the model is not sensitive to changes in either roughness or flow.

3.7 Pre-Development modelling scenario

OEH indicated that a significant amount of flood protection work has been undertaken on the Namoi River floodplain since the 1971 flood. These works include levees and channels, upstream of Wee Waa, which have changed the flood hydraulics/flow regime of the Namoi River floodplain. OEH was interested in determining if modelling the current Wee Waa Levee with the pre-developed floodplain would reduce flooding around the Wee Waa levee. If significant differences in flood levels were apparent OEH may consider removing some or all of the upstream flood protection works, returning the floodplain to a pre-developed state.

OEH provided URS with a Mike 11 model that represented a pre-developed Namoi River floodplain. The structure of this model was broadly similar to the developed case, but displayed differences in some key locations. The inflows for the TUFLOW model were extracted from the same location as that indicated in Section 2.1.4; however the cross section names were different.

The TUFLOW model was re run with the 1971 and 1% AEP flows. The 1971 pre-development flows were run through the TUFLOW model with the 2010 Wee Waa levee in place. The reasoning behind this is that OEH are interested in how the pre-developed flow regime influences water levels around the current Wee Waa levee. Figure 3-10 shows the water level around the Wee Waa levee for these events.

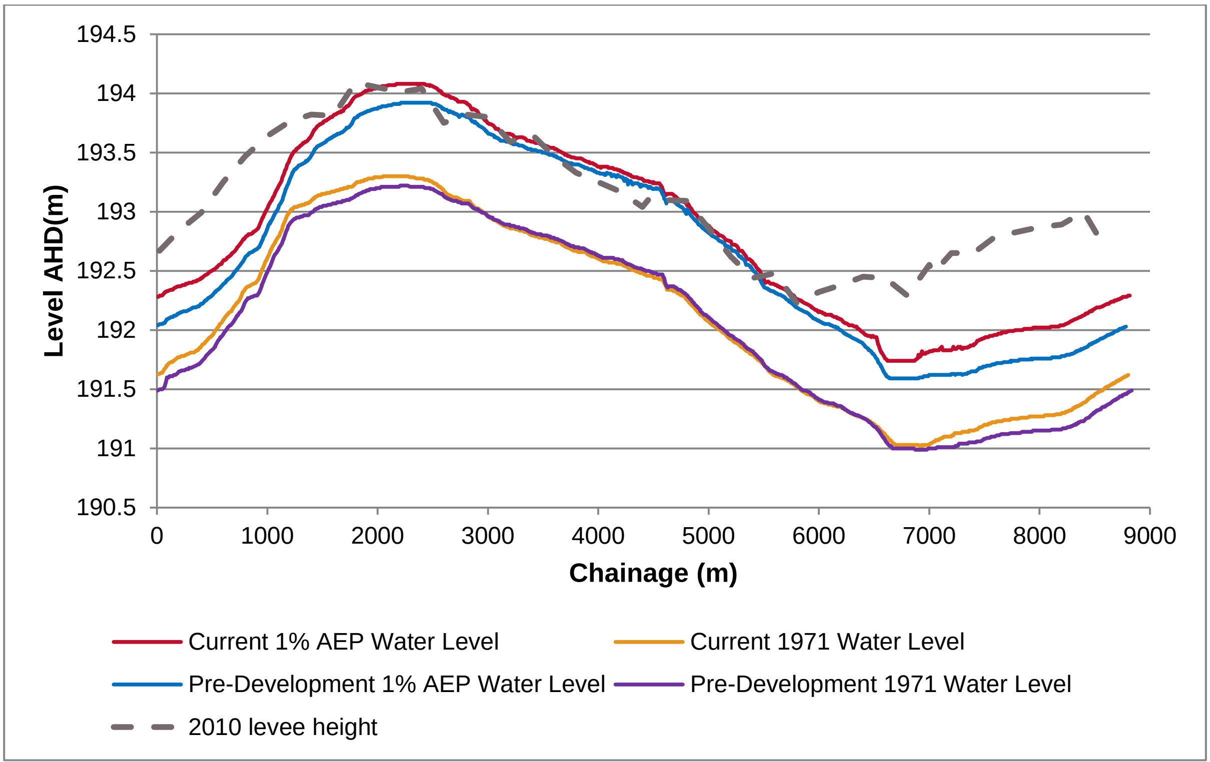


Figure 3-10 1% AEP flood level pre and post development, 1971 flood level and 2010 levee level

As can be seen the 1% AEP pre-development water level is slightly lower than the developed 1% AEP water level, however the Wee Waa levee is still overtopped. The length of levee overtopped is reduced, suggesting the volume and duration of overtopping is also reduced. This is reflected in Figure 3 11 which shows the depth and level of water at the peak of the pre-developed 1%AEP flood. It can be seen that water overtops the Wee Waa levee; however, there is a reduction in the volume of water held within the levee. The depth and extent of inundation is reduced compared to the developed 1% AEP flood (Figure 4-3).

Although there is a reduction in the flood levels between the pre-developed and developed floodplain scenarios, the Wee Waa Levee is still overtopped during the 1% AEP event. This indicates that removing some or all of the upstream flood protection works will not improve the level of service to the 1% AEP design storm.

The figure also demonstrates that an undeveloped floodplain would have only a minor change on the peak water levels during an event similar to the 1971 storm. The results shown here represent the 1971 storm with the levee in place, even though the structure was not constructed until later. By showing this data, it is possible to gain a deeper understanding of the performance of the levee.

3 Hydraulic modelling

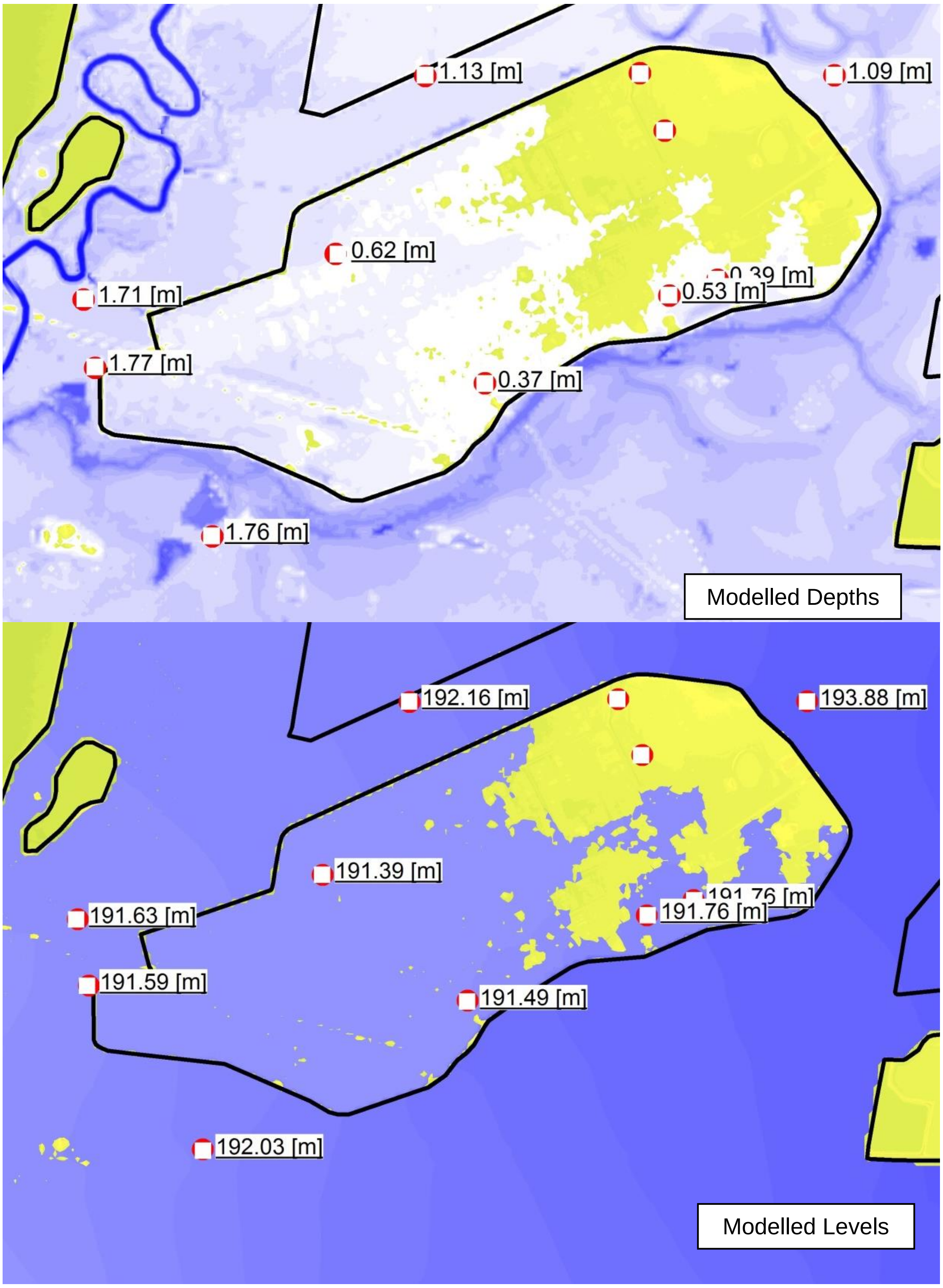


Figure 3-11 Peak 1% AEP predevelopment flood depths and levels taken at the same locations as the 1971 recorded flood levels

Hydraulic assessment

The TUFLOW model was used to determine the performance of the current Wee Waa levee for the 1971, 1% AEP and 3x1% AEP floods. The hydraulic model was also used to determine height correlations between the Wee Waa (Glencoe 419060) flood gauge and key locations around the Wee Waa levee such as the railway line openings and levee overtopping locations.

4.1 Wee Waa levee performance

The performance of the current (surveyed 2010) Wee Waa levee profile was compared to the modelled 1971 flood. As shown in Figure 4-1, the 1971 flood does not overtop the current levee (Appendix B contains the 2010 survey of the levee with chainages). Figure 4-1 also includes a line representing 1m over the modelled water level (the design criteria following the 1971 flood) and the recommended design level from the Economic Risk Analysis of Wee Waa Levee Upgrading (Department of Water Resources, 1993). It can be seen that the 2010 levee levels largely meet the recommended design level.

The model results suggest that the recommended design level was too low between chainages 4500m and 6000m. It is important to note the issues raised in Section 2.1.4 concerning the accuracy of flows to the south of Wee Waa. It is possible that the flows are over estimated and this would raise levels between chainages 3000m and 6000m. It can also be seen that the 2010 levels appear to be below the recommended design levels between chainages 0m and 800m and also 1500m and 4500m. It is possible that this is the result of settlement of the structure or inaccuracies in the measurement of the levee height. The maximum difference in the levels is 0.28m.

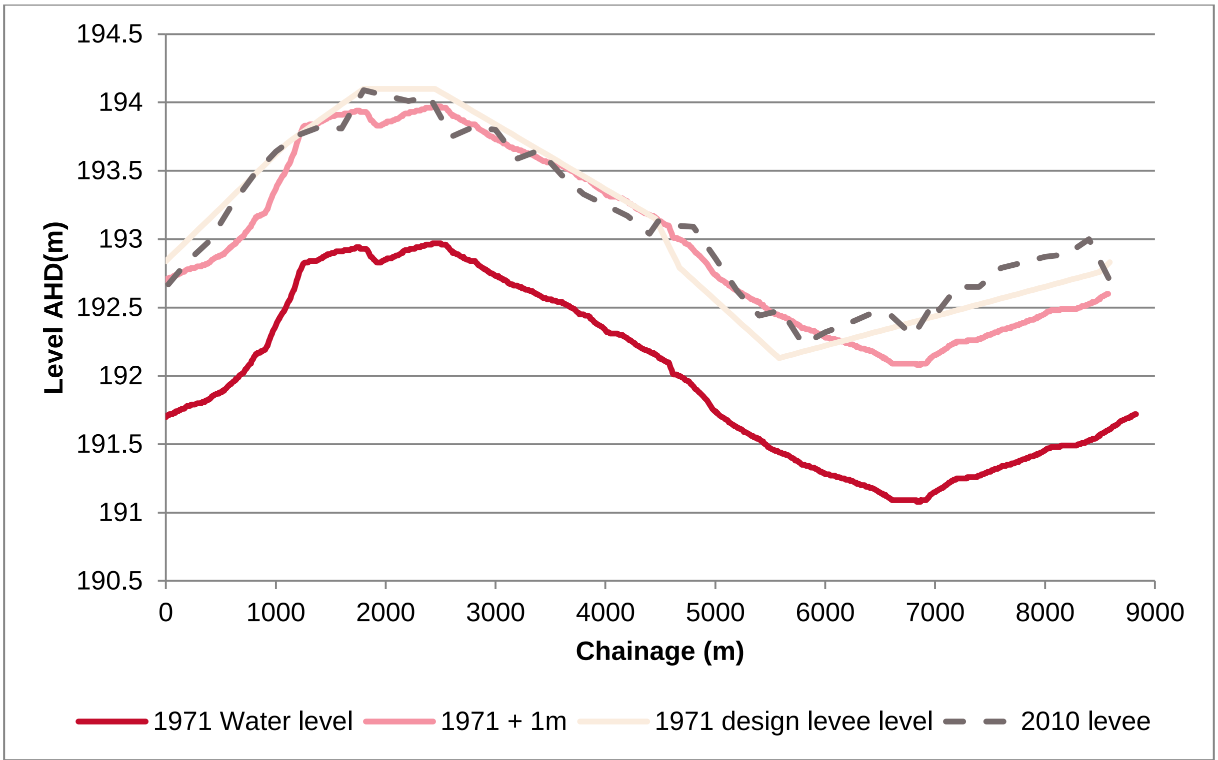


Figure 4-1 1971 flood and levee level

4 Hydraulic assessment

The performance of the current (surveyed 2010) Wee Waa levee profile was compared to the 1%AEP flood as shown in Figure 4-2.

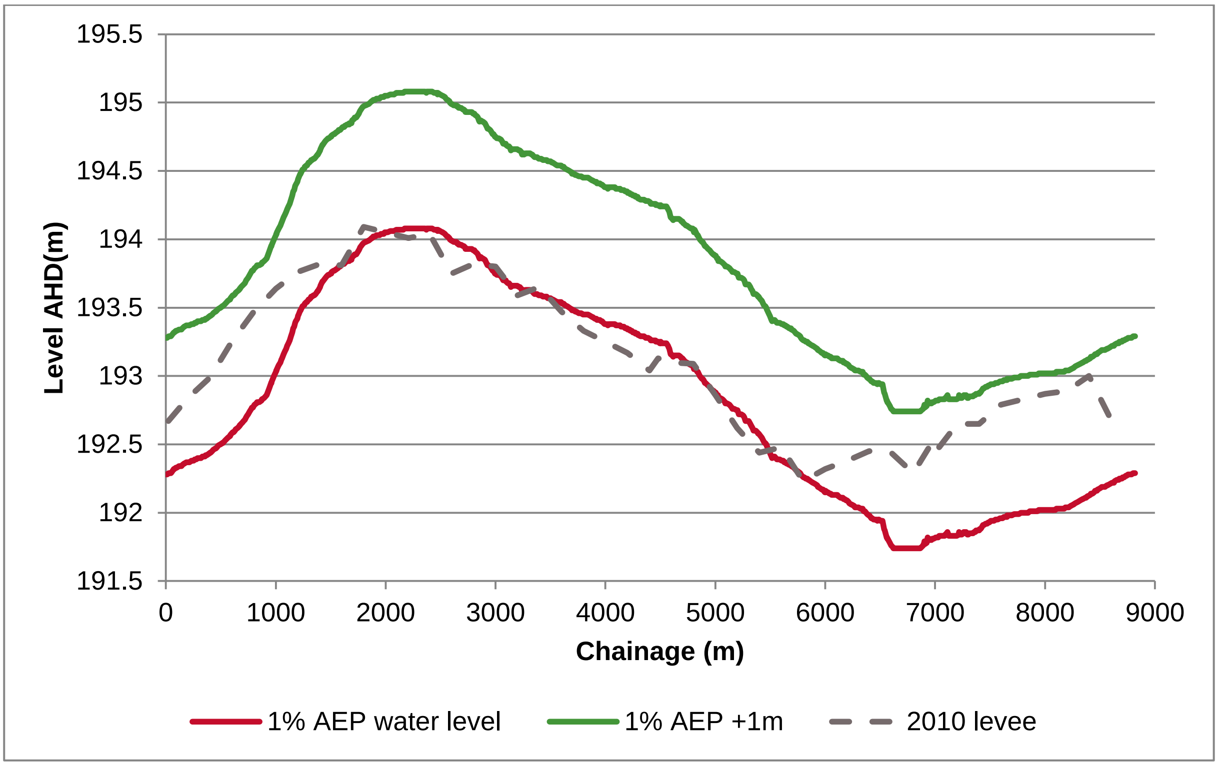


Figure 4-2 1% AEP flood and levee level

The 1%AEP flood just overtops the Wee Waa levee in a few locations resulting in water up to 1.0m deep within the town. Figure 4-3 shows the depth and level of water at the peak of the 1%AEP flood. Appendix D shows the progress of the flood inundation into town.

4 Hydraulic assessment

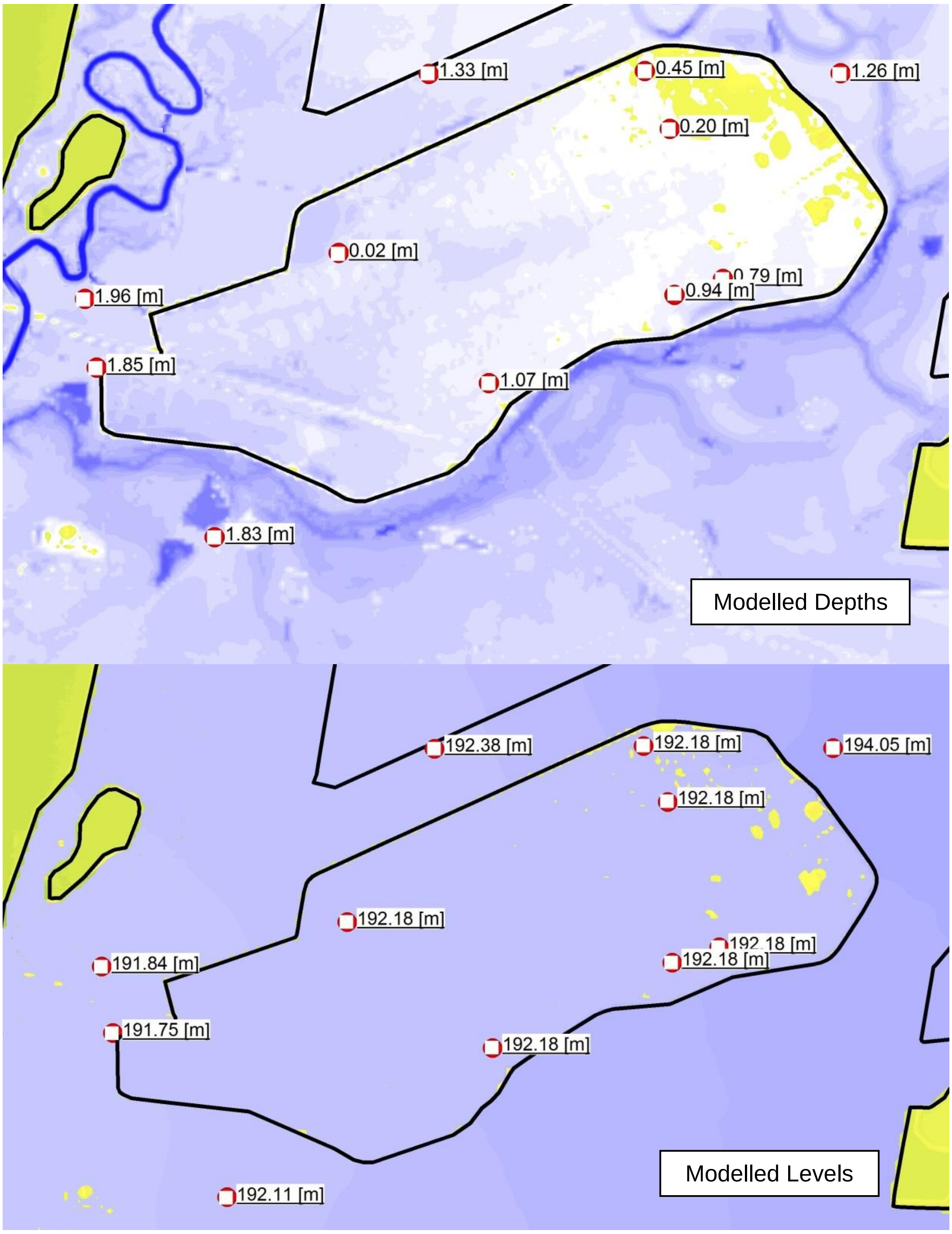


Figure 4-3 Peak 1% AEP flood depths and levels taken at the same locations as the 1971 recorded flood levels

A profile of the 3x1%AEP flood level and the current (2010 surveyed) levee is shown in Figure 4-4.

4 Hydraulic assessment

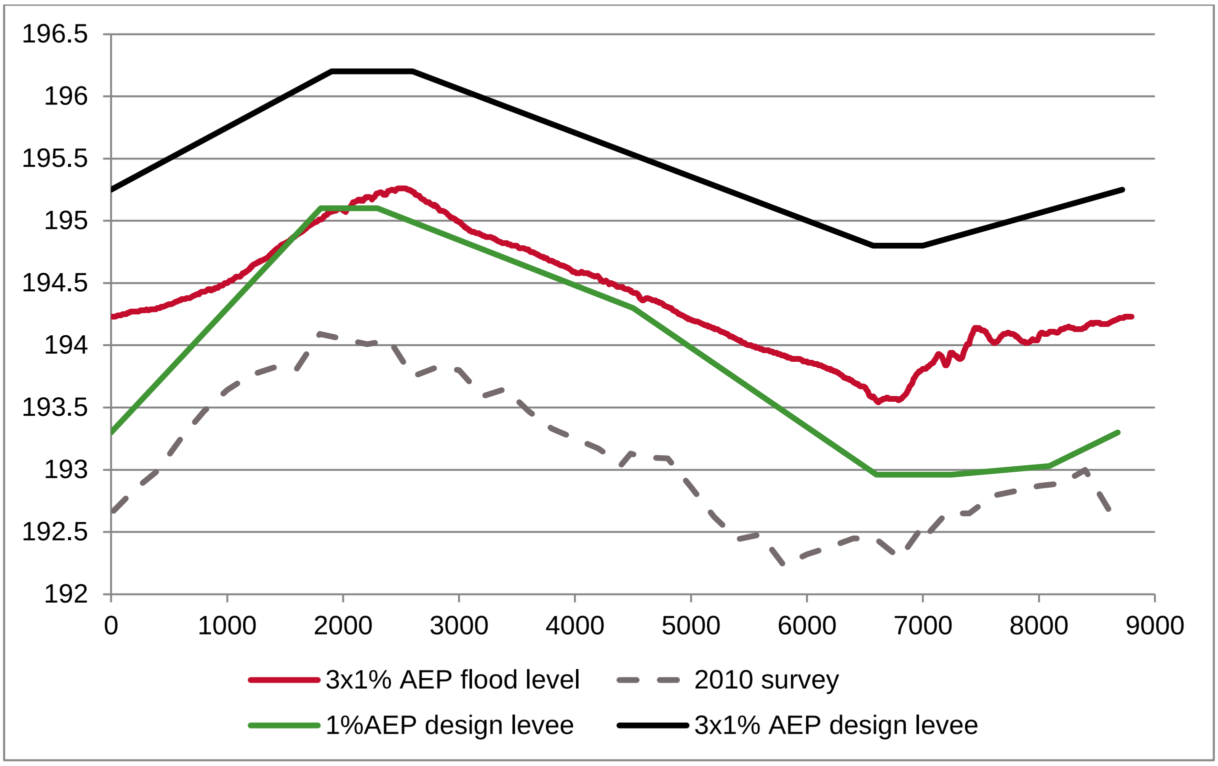


Figure 4-4 3x1% AEP flood and levee level

Figure 4-4 also demonstrates local variability in the peak water level along the levee (chainages 700m to 8000m). This suggests that there is a slight instability in water level around the levee caused by shallow water depths on top of the levee, which can be expected during such an extreme event. The instability is within acceptable limits and does not affect the interpretation of results.

The 3x1% AEP flood overtops the Wee Waa levee significantly along its entire length. This results in water up to 3.4m deep within the town as shown in Figure 4-5.

4 Hydraulic assessment

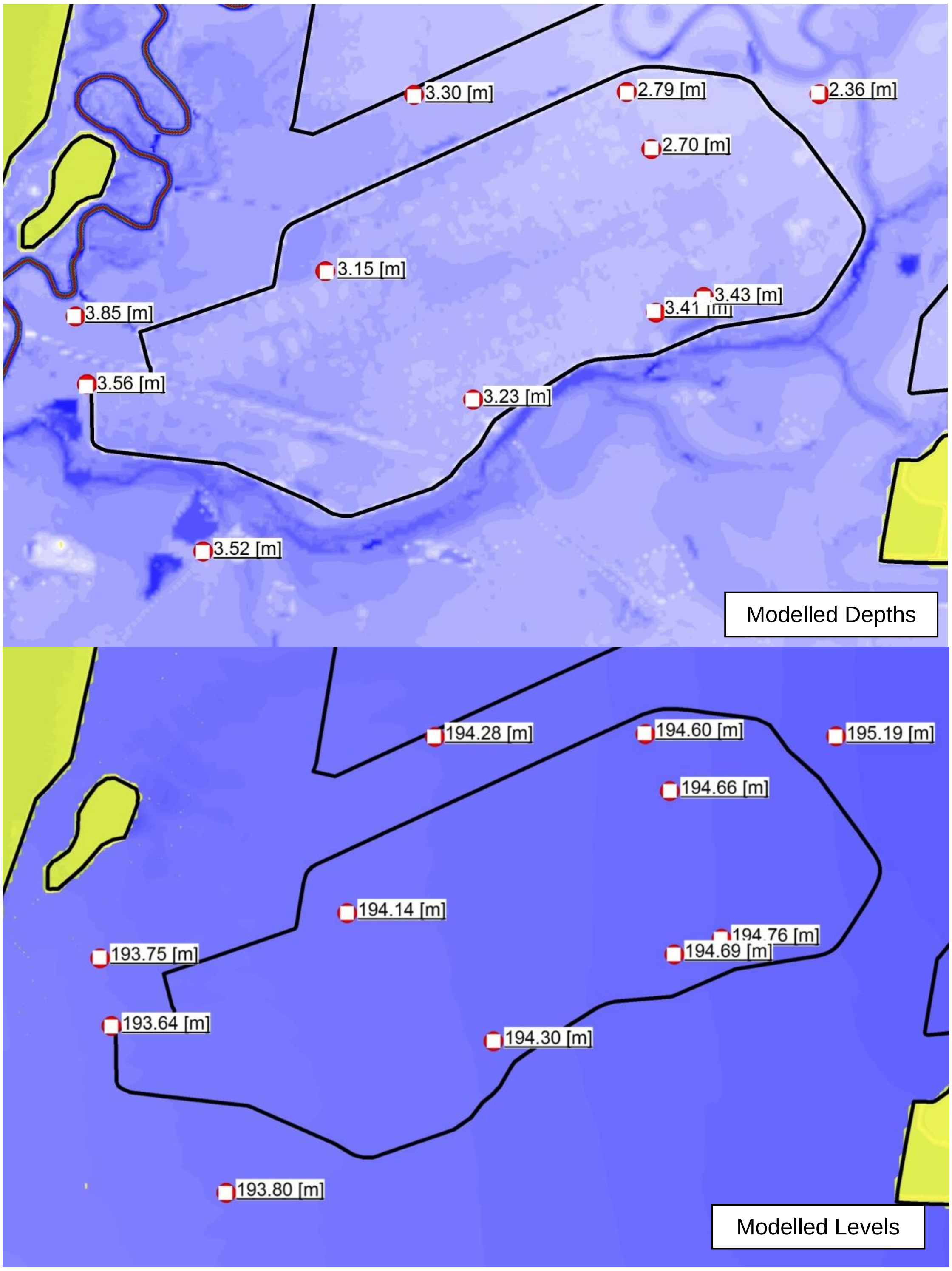


Figure 4-5 3x1% AEP flood depths and levels taken at the same locations as the 1971 recorded flood levels

4 Hydraulic assessment

Comparing the 1971 and 1% AEP flood in Figure 4-6 it can be seen that the 1% AEP flood is larger than the 1971 flood. As a result a theoretical levee design able to withstand the 1% AEP flood was delineated. The design criteria to withstand the 1% AEP flood has been taken as being the same as the design criteria stipulated after the 1971 flood, flood water level plus 1m. This would result in a levee with an approximate profile of that shown by the black line in Figure 4-6

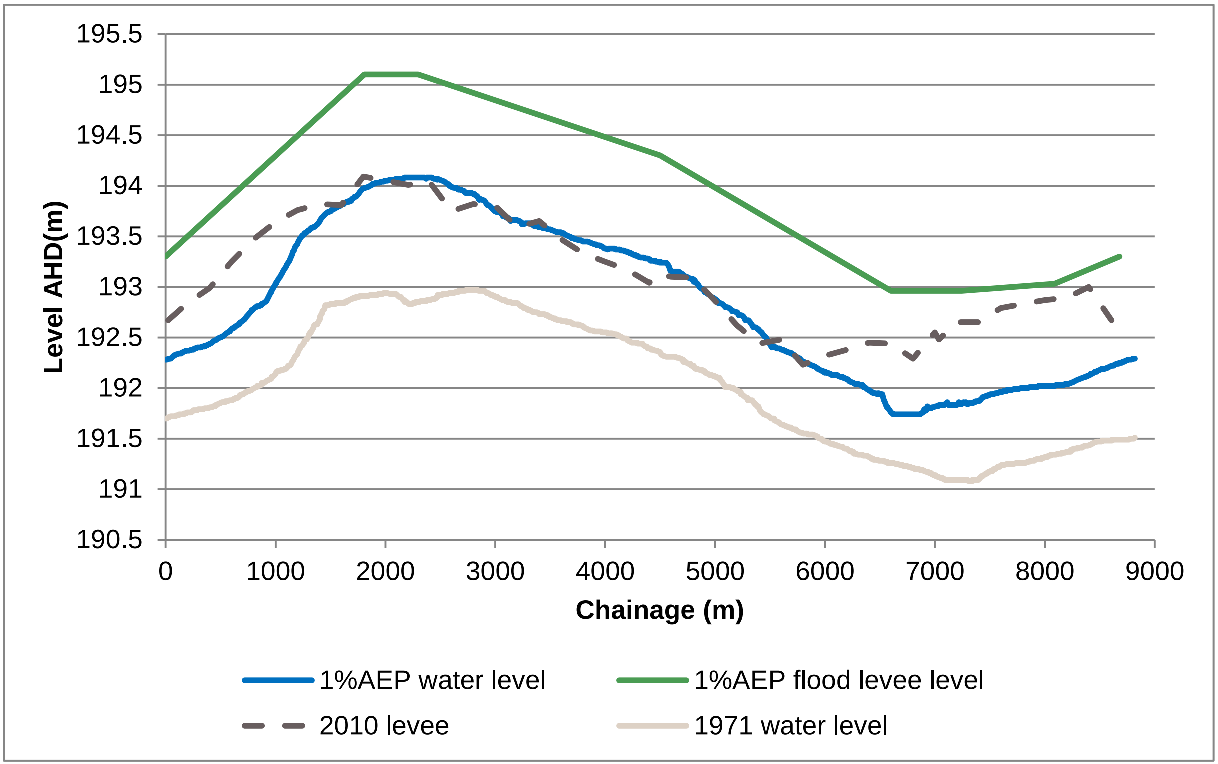


Figure 4-6 Comparison of the 1971 and 1% AEP flood, current 2010 levee and levee designed to withstand the 1% AEP flood

As shown in Figure 4-6 a levee able to withstand the 1% AEP flood with 1m freeboard would mean raising 6500m of the existing levee approximately 0.5m to 1.2m.

The requirements to defend against the “Extreme Flood” (3x1% AEP) would require a levee that has the approximate profile of that shown by the black line in Figure 4-7. To defend against this flood would mean raising the entire existing levee approximately 1.6m to 2m. The blue line in Figure 4-7 shows the design levee height to defend against the 1% AEP flood. As can be seen the 3x1% AEP event overtops the 1% AEP flood design levee.

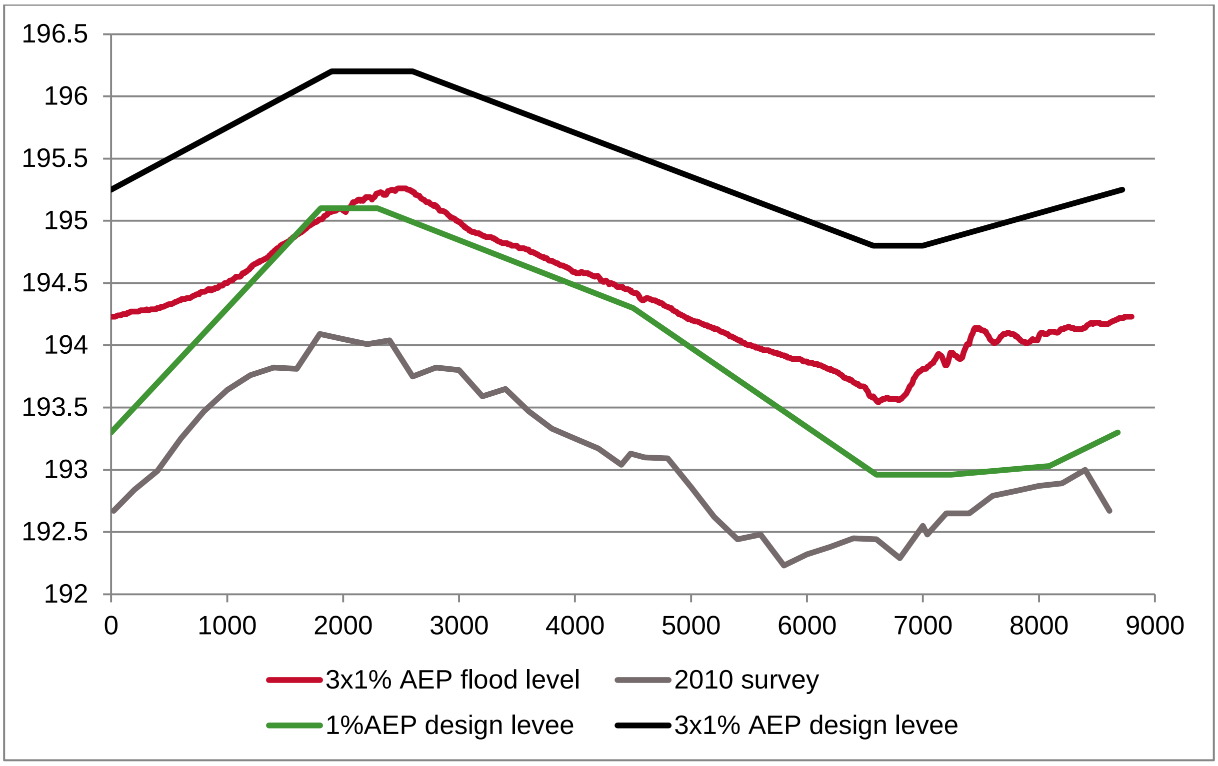


Figure 4-7 The 3x1%AEP flood, current 2010 levee and levee designed to withstand the 3x1% AEP flood

It should be noted that the levee long section and flood profiles are based on the levee being overtopped. It does not take into consideration levee failure which may be caused by other phenomenon such as seepage and erosion which may lead to a collapse of the levee during a flood.

4.2 Flood gauge correlations

A correlation between flood levels at the Mollee gauge and various locations around the township of Wee Waa was undertaken. This was done by assuming that the water levels in the Mike 11 model at Mollee could be plotted against the water levels around the township of Wee Waa from the TUFLOW model. A correlation was also undertaken between Glencoe (results from Mike 11) and the same key locations around Wee Waa.

Figure 4-8 and Figure 4-9 show the correlations between the first overtopping location (refer to Appendix D for a plan of the flood inundation extents over time) and the east and west railway line openings and the upstream flood gauges. No investigation into the accuracy of these correlations has been undertaken. It should be noted that the 1971 flood event was run both without a levee (non filled data points) in place and with a levee in place. This was done to ensure that the correlations reflected the present floodplain conditions i.e. with a levee surrounding Wee Waa.

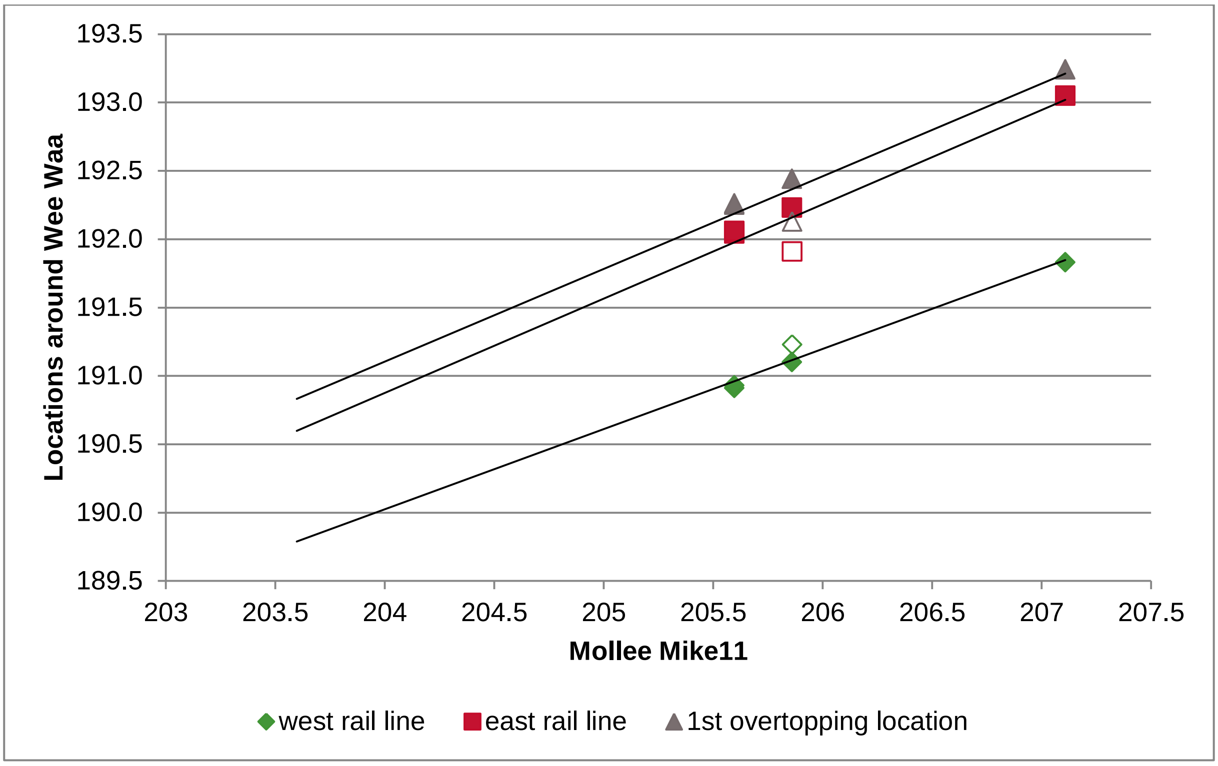


Figure 4-8 Flood correlation between the Mollee Gauge and key locations around the Wee Waa levee

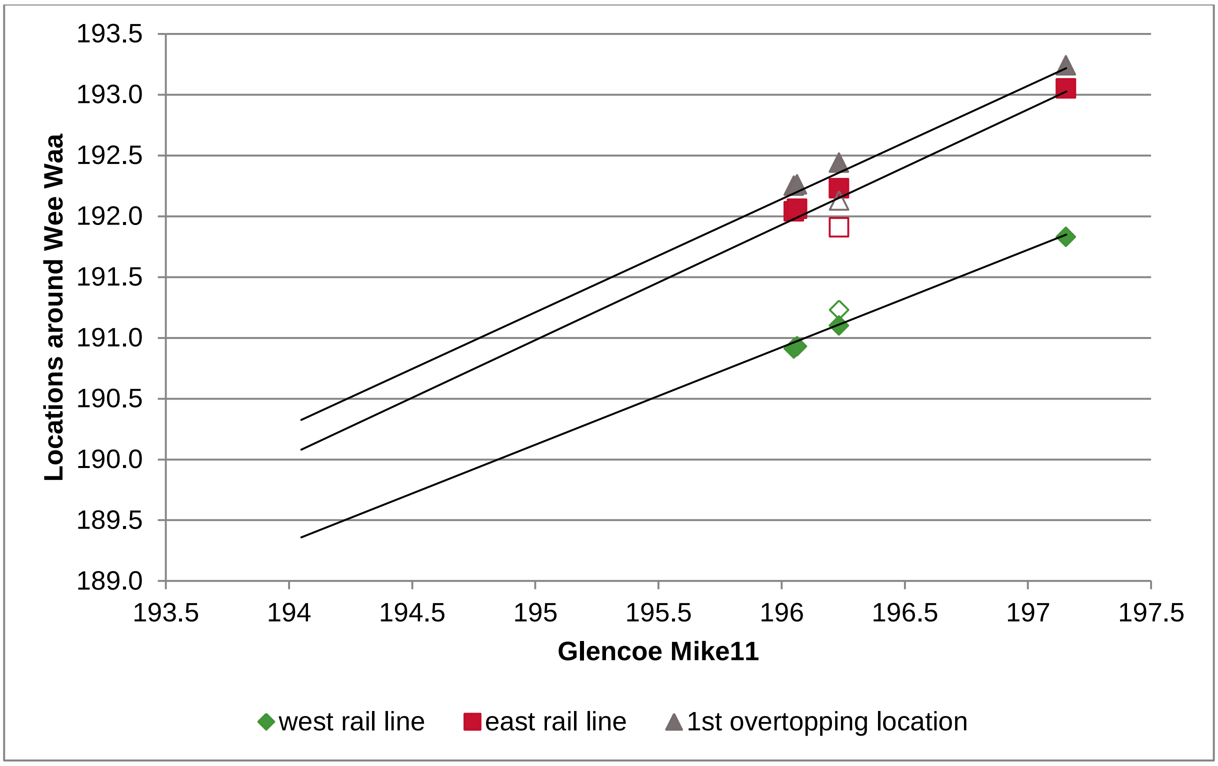


Figure 4-9 Flood correlation between the Glencoe Gauge and key locations around the Wee Waa levee

The height of the levee where it is first overtopped is 193.0m (AHD) this correlates to a level of 196.9m (AHD) at the Glencoe Gauge or approximately 8.4m on the gauge which is in good agreement with the 8.5m stated in the SES Flood Plan (1994 pg35) which represents the design levee height on the gauge.

There are two openings in the Wee Waa levee which allow the railway line to pass though the levee. During floods these openings are closed using flood gates. The minimum height of the eastern railway line opening is 191.4m AHD. This correlates to 195.4m AHD at the Glencoe gauge or a depth of 6.9m.

The minimum height of the western railway line opening is 191.0m AHD. This correlates to 196.1m AHD at the Glencoe gauge or a depth of 7.6m.

These correlations can be improved through the running of more events through the models. It would be useful to use some smaller events, so that the correlation can be improved for less extreme events. The correlations could also be verified through the measurement of water levels during flood events. It would be important to record the peak levels reached at all three locations around the levee and to compare this to the gauge record.

4.3 Hazard mapping

Following consideration of the impacts of flooding and the various flow regimes as flood waters spread across the Namoi River floodplain. The velocity and depth of water determined by TUFLOW was used to determine the flood hazard boundaries.

Flood hazard maps were produced as per the NSW flood management plan guidelines (Figure 4-10) for the 1971,1984, 1998 and 1% AEP and the 3x1% AEP floods. These are shown in Appendix E. For comparison purposes the Wee Waa levee has been shown on the 1971 hazard map.

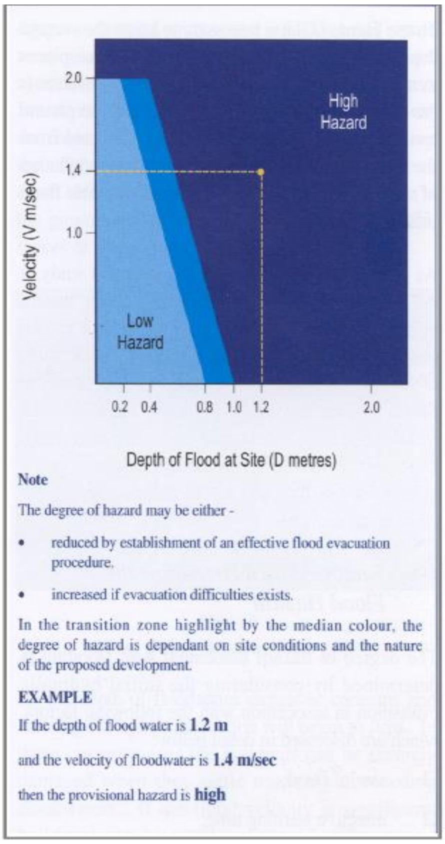
The hazard maps for all flood events modelled show similar characteristics. The major contributing factor to the hazard rating applied is the depth of water. Velocity in the river channels is high; however on the floodplains it generally remains below 0.5m/s. This is typical of a low gradient, meandering river, such as the Namoi. The river channel consists of fast deep flowing water, when the river breaks its banks the water spreads out over the floodplain and slows down.

For the 1971,1984 and 1998 modelled events, the depth of water (>1m) on the southern side of the Wee Waa levee causes a high hazard rating with the velocity not exceeding 0.5m/s. This high hazard rating reduces to the south as the depth of water decreases, caused by rising ground elevation.

North of the town the hazard rating varies from high to low generally due to the depth of water rather than the velocity. The Namoi River itself has a high hazard rating due to both the depth of water, up to 9m in some places, and a velocity of up to 1.6m/s.

The 1% AEP and the 3x1% AEP design floods cause the majority of the modelled area to register a high hazard rating. This is due to the amount of water that would be present during these floods resulting in a depth greater than 1m for the majority of the floodplain. Only areas with higher elevation have lower hazard ratings. These design events are similar to the modelled past events with a reducing hazard rating to the south where ground elevation rises.

4 Hydraulic assessment



Note

The degree of hazard may be either -

- reduced by establishment of an effective flood evacuation procedure.
- increased if evacuation difficulties exists.

In the transition zone highlight by the median colour, the degree of hazard is dependant on site conditions and the nature of the proposed development.

EXAMPLE

If the depth of flood water is **1.2 m**

and the velocity of floodwater is **1.4 m/sec**

then the provisional hazard is **high**

(source: Floodplain Development Manual 2005, NSW Government)

Figure 4-10 Extract L2, Appendix L of the Floodplain Development Manual

Conclusion and Recommendations

The 2D TUFLOW model developed for the project gave a good representation of the flooding along the Namoi River in the vicinity of Wee Waa. The previously developed 1D modelling (using Mike11) was adopted with no alteration and this was used to provide input to the 2D model developed for this study. The Mike 11 model had been calibrated and was shown to be a good representation of the Namoi system as a whole. This was confirmed by comparing the Mike 11 water level results with water level information from two gauges on the Namoi (Glencoe and Gunidgera). The 2D TUFLOW model was found to compare well with both the Mike 11 results and the data record at the Gunidgera gauge. The good comparisons between the gauge data, the Mike 11 model results and the 2D TUFLOW results, validate that the TUFLOW model adequately represents the floodplain around Wee Waa. However, it should be noted that one limitation of the model was that the Mollee inflow was a constant flow as there was no measured flows available. This may lead to a slight overestimation of modelled water levels along the southern edge of the Wee Waa Levee.

The model analysis suggested that the current levee (surveyed 2010) is adequate to protect the town of Wee Waa from a flood equivalent in size to that of the 1971 event. It is also suggested that the design levels recommended by the Department of Water Resources in 1993 may not meet the requirements of the 1971 flood + 1m.

The levee is does not protect the town from a 1% AEP flood with sections of it being overtopped with water up to 1.2m deep occurring in some areas within the levee. If the levee were to be improved to meet the criteria of the 1% AEP event + 1m, it would result in the raising a 6,500m length by between 0.5m and 1.2m.

Correlations were developed for a number of locations around the levee and recorded data at the Glencoe and Gunidgera gauges. The correlations are based on a limited data set due to the limited number of events that have been modelled; however the data is still useful. The SES have derived an informal correlation for when the levee is likely to be overtopped. It was found that the model matched the SES information, supporting the accuracy of the derived correlations. These correlations should aid the SES in determining when flood defence actions are required in Wee Waa.

References

NSW Department of Primary Industries Office of Water. Gauging data from the website http://realtimedata.water.nsw.gov.au/water.stm

Department of Infrastructure, Planning and Natural Resources. (2003). *Narrabri – Wee Waa Flood Study*

Narrabri Shire Local Emergency Management Committee. (1993). *Narrabri Shire Local Displan*

NSW Department Land and Water Conservation. (2008). *PINEENA, flow gauge information*

NSW Department of Natural Resources. (2005). *Narrabri – Wee Waa Flood Management Plan*

NSW Department of Primary Industries. (2006). *Reducing the impact of weirs on aquatic Habitat NSW detailed weir review Report to the New South Wales Environmental Trust Namoi CMA region*

NSW Public Works Department – Dams and Civil Section. (1992). *Audit of Flood Levees for New South Wales – Summary Report*

Water Resources Consulting Service. (1993). *Economic Risk Analysis of Wee Waa Levee Upgrading*

Limitations

URS Australia Pty Ltd (URS) has prepared this report in accordance with the usual care and thoroughness of the consulting profession for the use of Narrabri Council and only those third parties who have been authorised in writing by URS to rely on the report.

It is based on generally accepted practices and standards at the time it was prepared. No other warranty, expressed or implied, is made as to the professional advice included in this report.

It is prepared in accordance with the scope of work and for the purpose outlined in the Proposal dated May 2008.

Where this Report indicates that information has been provided to URS by third parties, URS has made no independent verification of this information except as expressly stated in the Report. URS assumes no liability for any inaccuracies in or omissions to that information.

This report was prepared between September 2012 and September 2013 and is based on the conditions encountered and information reviewed at the time of preparation. URS disclaims responsibility for any changes that may have occurred after this time.

This report should be read in full. No responsibility is accepted for use of any part of this report in any other context or for any other purpose or by third parties. This report does not purport to give legal advice. Legal advice can only be given by qualified legal practitioners.

Except as required by law, no third party may use or rely on, this Plan unless otherwise agreed by URS in writing. Where such agreement is provided, URS will provide a letter of reliance to the agreed third party in the form required by URS.

To the extent permitted by law, URS expressly disclaims and excludes liability for any loss, damage, cost or expenses suffered by any third party relating to or resulting from the use of, or reliance on, any information contained in this Plan. URS does not admit that any action, liability or claim may exist or be available to any third party.

Except as specifically stated in this section, URS does not authorise the use of this Plan by any third party.

It is the responsibility of third parties to independently make inquiries or seek advice in relation to their particular requirements and proposed use of the site.

Any estimates of potential costs which have been provided are presented as estimates only as at the date of the Report. Any cost estimates that have been provided may therefore vary from actual costs at the time of expenditure.

Appendix A Data Inventory

The following is a list of information used to build the TUFLOW model and compile the report.

A.1 Aerial Photography

Boggabri to Pilliga, Survey-General's Dept. July 1998

Namoi River Flooding – Narrabri-Wee Waa, Water Resources Commission February 1984

Namoi River Flooding, Water Conservation & Irrigation Commission February 1971

A.2 Reports and plans

- *Narrabri Shire Local Displan* by Narrabri Shire Local Emergency Management Committee.
- *Economic Risk Analysis of Wee Waa Levee Upgrading* by Water Resources Consulting Service 1993
- *Audit of Flood Levees for New South Wales – Summary Report* by NSW Public Works Department – Dams and Civil Section 1993
- *Narrabri – Wee Waa Flood Study* Department of Infrastructure, Planning and Natural Resources 2003
- *Narrabri – Wee Waa Flood Management Plan* by NSW Department of Natural Resources 2005.

A.3 Survey

AAMHATH LiDAR survey February 2009

DIPNAR (GPS), 2001

B

Appendix B Levee Survey

URS

Appendix B - Levee Survey

The image below shows which structures were given a representative height of 200mAHD (higher than the Wee Waa levee, and which used the raw aerial survey data. 200mAHD was chosen to make it unlikely that the levees would be overtopped. The levees were identified in consultation with OEH.