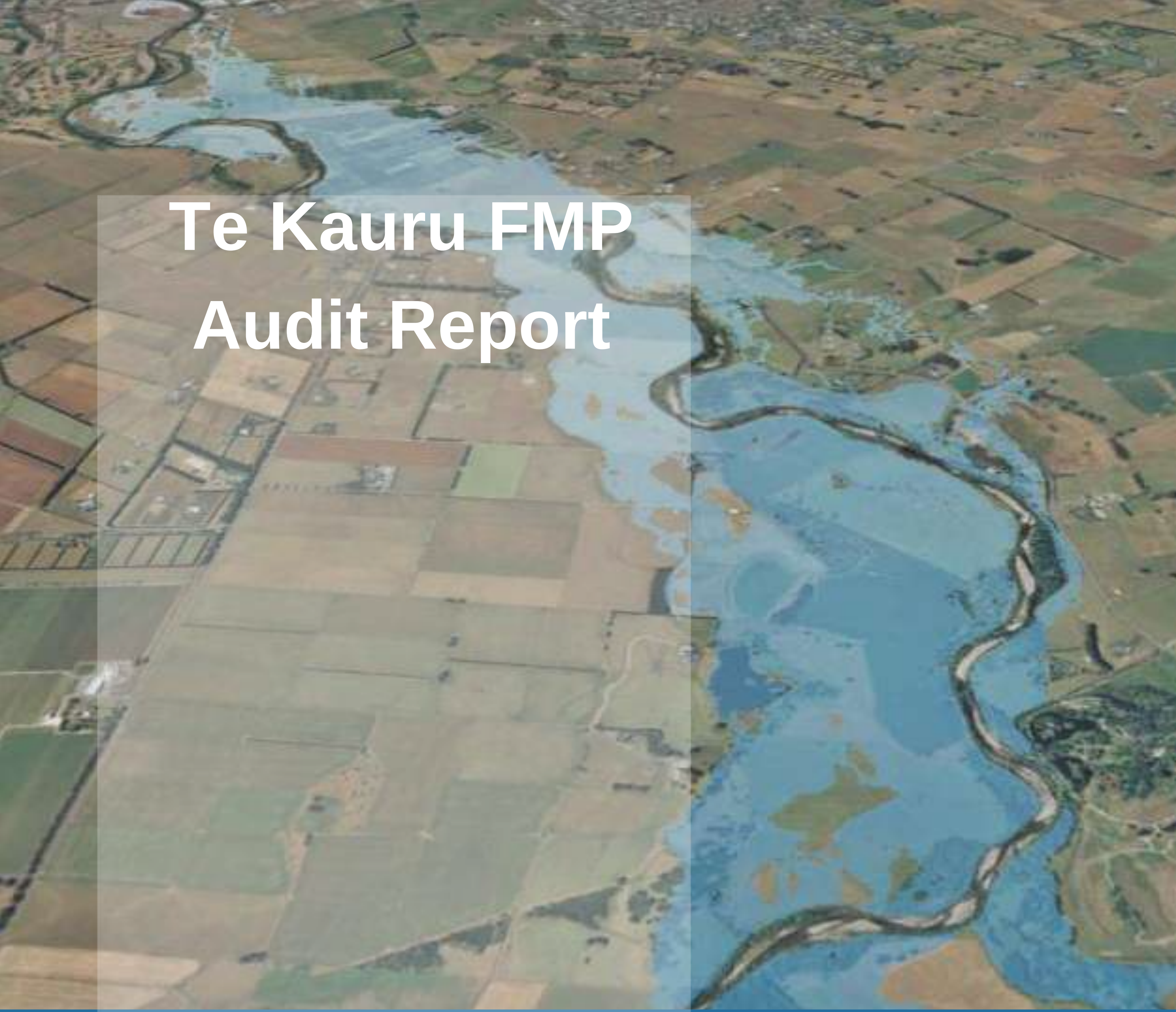

Te Kauru FMP Audit Report

JUNE 7, 2019

Client: Greater Wellington Regional Council
Report by: Matthew Gardner & Vicki Henderson
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LANDRIVERSEA CONSULTING

greater WELLINGTON REGIONAL COUNCIL Te Pane Matua Taiao

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1. EXECUTIVE SUMMARY

A detailed audit of the hydrology and hydraulic modelling components of the Te Kauru FMP has been carried out as per the scope of study provided in the RFQ document (Appendix A). The audit has ben led by Matthew Gardner who has led the audit of the hydraulic model and Vicki Henderson of Barnett and MacMurray Ltd who has led the audit of the Hydrology component of the modelling.

Statements made in regard to being fit for purpose in this report relate to the purpose detailed in the RFQ which states that the main purposes of the model are:

- Catchment-scale flood hazard assessment, including classification of hazard into different categories, for the Waipoua River and other rivers in the FMP study area.
- Development and conceptual design of different flood management options (including being used for analysis of potential flood damages)
- Providing information for use by District Councils in LIMs, land use controls and for building controls (Local Government Act, Resource Management Act and Building Act requirements)
- To provide potential update to existing flood hazard information in the District Plan

1.1 GENERAL ASSESSMENT ITEMS

1.1.1 SOFTWARE AND MODELLING METHODS

In general, the type of software used for both the assessment of the hydrology and the hydraulic model is considered appropriate and fit for purpose.

The modelling method used for the hydraulic model build is considered appropriate.

1.1.2 USE OF FREEBOARD

Freeboard is an essential component of any flood model and it is considered best practice that model freeboard is always incorporated into the model results. The provided model reports describe two separate methods for the application of freeboard within the models used to develop the floodmaps in this FMP.

The freeboard method adopted for the Waipoua Flexible Mesh Model considered the sensitivity of the model to 22 separate scenarios and combines the results from three representative scenarios, giving a dynamic freeboard over the entire model area based on actual model sensitivity.

The freeboard method used for the regular grid modelling has applied a static volume of water on top of the results and allowed the water levels to spill dynamically for a set period of time.

Whilst both methods are valid methods for the application of freeboard, it is recommended that a consistent approach for model freeboard is adopted. It is recommended that the method used for the Waipoua Flexible Mesh model is adopted for all rivers within the FMP and that consideration should be given to including more sensitivity runs in the final adopted freeboard, including bridge blockage.

1.1.3 REPRESENTATION OF THE FLOOD HAZARD

The existing flood hazard maps are easy to read and are generally considered fit for purpose. The existing maps could be improved however by using a degree of transparency on the flood layer so that key landmarks underneath the flood layer can be identified, as well as adding on road names to assist with orientation for local residents.

More information could be communicated to the public through the use of;

- Discrete colours used to categorise depth rather than a graduated colour scale
- Presentation of velocity maps which show both peak speed and flow direction
- Presentation of flood hazard maps using a categorisation such as that recommended in the Australian Rainfall and Runoff guidelines, which uses six hazard categories defined through a combination of speed and depth.
- Use of online videos to demonstrate the way that the modelled flood propagates over the floodplain.

It is also recommended that to make the interpretation of model results easier inhouse, that all model results are merged into a single file. This is particularly important for where different result files overlap.

1.1.4 INPUT DATA

In general, all of the adopted input data is considered fit for purpose. This has included an analysis of;

- Rainfall data
- Measured flood flows
- Cross section survey data
- Lidar survey data
- Representation of any structures (note: some issues with the representation of structures is presented in further detail in later sections of the report)

An assessment of the calibration data used to assess the model results against historical events is generally considered appropriate, however one calibration level on the Waipoua River immediately upstream of the railway bridge has been highlighted as likely needing adjustment for the effects of super elevation, wave run up and the backwater effects due to the protrusion of railway groynes as well as debris build up on the bridge piers.

1.1.5 GENERAL ASSUMPTIONS

General assumptions such as those listed below have all been assessed as being appropriate, unless highlighted in detail in the following sections;

- Run-off coefficients or similar hydrological parameters
- Predicted flood flows used for design events
- Climate change allowances
- Roughness coefficients of the channel and floodplain
- How the buildings and structures on the floodplain are treated through use of roughness coefficients
- Treatment of bridges, culverts and pipe crossings

1.2 RECTANGULAR GRID - UPPER HYDRAULIC MODELLING

In general, the hydraulic model setup for this model has been assessed to be fit for purpose. Technical recommendations in this report have been classified as minor (only likely to have localised effect), moderate (has potential to impact over a wider area); or major (has potential to have significant impact on overall results of the modelling). A detailed summary of all recommendations made in the audit is presented in Appendix G.

The following general minor recommendations have been made.

- Several bridges have been excluded from the model setup; due to the fact they are not considered to be a significant obstruction to flow. Consideration should be given to including the bridges into the model to allow for the localised impact and to better represent the headloss through the structure. Including the bridges will also allow for their sensitivity to blockage to be assessed.
- There are localised instabilities present within the Kopuaranga model at two separate locations. These should be investigated, and the bridge setups refined.
- The lateral links in the Whangaehu River at chainage 9452 are not functioning correctly and should be rectified.
- In areas of the model where there are significant bends, the lateral links are not correctly transferring water
- That the seamline checks are formalised and documented.
- That a comparison between the 1m grid based on the base LiDAR with the final 10m grid used in the model is carried out to ensure road crest levels are adequately captured in the model, as per the DHI peer review recommendations. A representative sample should be checked where flood waters cross roads in the final results.

The following moderate recommendations have been made

- The DHI peer review report recommended changing all lateral link elevation sources to use the M21 method. This has not been done in this model and should be implemented to ensure the model correctly represents the physical reality.
- That the apparent horizontal shift between the applied roughness file and the terrain model is investigated and the cause for the shift is rectified. The model will likely need to be rerun as a result.
- No attempt has been made to calibrate the Ruamāhanga model despite extensive debris levels being on file from 1994. Further justification would need to be made to ignore this data.
- No documented attempt has been made to verify/validate the model results with historic events. Before the results are finalised, it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;
    - Comparison with historic flood photos / debris levels
    - Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)

- Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods

A documented model verification / validation process is likely to give the community more faith in the overall model predictions and would ideally be included in the main body of the modelling report. Model validation is also useful to ensure that the model results are realistic and can ensure that errors in the underlying model data are picked up (ie missing sections of stopbank).

No major recommendations have been made for the Upper Model.

1.4 RECTANGULAR GRID - LOWER HYDRAULIC MODELLING

In general, the hydraulic model setup for this model has been assessed to be fit for purpose. The recommendations for the lower model are very similar to those from the upper sections.

The following general minor recommendations have been made.

- Several bridges have been excluded from the model setup due to the fact they are not considered to be a significant obstruction to flow. Consideration should be given to including the bridges into the model to allow for any localised impact and to better represent the headloss through the structure. Including the bridges will also allow for their sensitivity to blockage to be assessed.
- In areas of the model where there are significant sharp bends, the lateral links are not correctly transferring water between the 1D and 2D model. This is most evident on the Taueru River for this model.
- One minor instability has been noted in the MIKE11 results for the Waingawa model at chainage 6829, this should be investigated.
- It is recommended that the seamline checks are formalised and documented.

The following moderate recommendations have been made

- The DHI peer review report recommended changing all lateral link elevation sources to use the M21 method. This has not been done in the Ruamāhanga model and should be implemented to ensure the model correctly represents the physical reality.
- No attempt has been made to calibrate the Ruamāhanga model despite extensive debris levels being on file from 1994. Further justification would need to be made to ignore this data.
- No documented attempt has been made to verify/validate the model results with historic events. Before the results are finalised, it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;
  - Comparison with historic flood photos / debris levels
  - Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)
  - Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods

A documented model verification / validation process is likely to give the community more faith in the overall model predictions and would ideally be included in the main body of the modelling report. Model validation is also useful to ensure that the model results are realistic and can ensure that errors in the underlying model data are picked up (ie missing sections of stopbank).

No major recommendations have been made for the Lower Model.

1.5 WAIPOUA RIVER – FLEXIBLE MESH MODEL

In general, the approach used to model the Waipoua River is technically sound, however due to two major concerns identified in the audit, the floodmaps as they stand are not considered fit for purpose and require further investigation and refinement of the model before the floodmaps can be finalised. Once these two recommendations have been implemented, it is considered that the hydraulic models can be generally considered fit for purpose. It is recommended that the minor and moderate recommendations are implemented, or justification provided if they are ignored.

The following general minor recommendations have been made.

- That the model report is finalised with all relevant information included and the document is finalised into a single merged pdf which contains all of the Appendices.
- Consideration could be given to including individual buildings in the roughness definition file in future upgrades of the model using the recently released buildings polygon layer (LINZ)
- That the bridge piers for both the Colombo Rd and the State Highway 2 bridge are included in the model setup
- That consideration is given to lowering the soffit of the Colombo Rd bridge in order to account for the potential effects of floating debris during a large event.
- It is recommended that the lateral link on the true right bank of the river downstream from the State Highway bridge (M11 chainage 30590 to 30720) is included in the model, or otherwise its exclusion is justified.
- In some locations where the lateral links run along the top of a stopbank, the lateral link runs slightly inside the crest and may encourage water to fluctuate between the 1D and 2D models. Consideration could be given to moving the link location to be on top of the crest or defining the crest levels within the lateral links themselves using an external link file.
- It is recommended that the seamline checks are formalised and documented.
- It is recommended that the rating curve from the model and gauged rating curve at the Colombo Rd bridge are compared as per the peer review report. It is currently unclear if a rating curve is yet to be developed for the Colombo Rd site and this should be investigated.
- It is recommended that thought is given to converting the historical flood levels from the 1947 flood into the current datum and make a comparison of the historic flood levels with the 1998 model calibration results in the same location. This may assist in getting a better feel for the likely magnitude of the 1947 flood event.
- A search for historic orthophotos could also be made from the 1940's.

The following general moderate recommendation has been made.

- As per the recommendation in the T&T peer review report, the 1D bank markers are adjusted to match the 2D mesh boundary, in particular in the locations highlighted in the peer review report.

The following major recommendations have been made.

- The model shows significant spilling on the true left bank in the reach downstream from the railway bridge at MIKE11 chainage 29550. This results in significant inundation of the Mawley Holiday Park and surrounding houses. The model shows a significant gap in the stopbank in this location, however inspection of the 2013 LiDAR and visit to site has confirmed that there is a continuous bank in this location which is covered with fairly dense vegetation. It is recommended that this bank is cleared of vegetation so that the crest level can be accurately surveyed and included in the model. It is likely that the section of model in this reach will need to be recalibrated once this bank is included.
- Inspection of the setup of the railway bridge in the model shows that the headloss through the bridge is significantly greater than other bridges in the model. Based on a site visit and discussions with experienced engineers who are familiar with the bridge, it appears that the modelled headloss is greater than would be expected during a real event.
  The use of the culvert module to represent the bridge is not a standard approach for such a bridge, however, has been justified due to the apparent good calibration of the model to observed debris levels during the 1998 event. Consideration needs to be given to adjusting the level of the recorded debris level at XS7 to account for the effect of super-elevation as well as other physical phenomena. A review of the appropriateness of all debris levels used in this study may be warranted at the same time.
  It is recommended that the setup of the bridge at the railway is reconsidered, ideally using a standard methodology such as the FHWA approach, or a standard headloss factor is applied based on hand calculations using guidelines from a reputed source such as "Open-Channel Hydraulics" (Chow, 1959). Considering the sensitivity of the model results and overall floodspread to this structure, it may be appropriate to ensure that a range of experts are consulted before an appropriate headloss through this structure is agreed upon.

1.6 HYDROLOGY

The key question being evaluated is whether the hydrology used for the FMP is fit for purpose, measured against a list of intended uses for the floodplain management plan. These were outlined in the audit terms of reference. The findings for the hydrology are given for each level of use below.

- Catchment-scale flood hazard assessment, including classification of hazard into different categories, for the Waipoua River and other rivers in the FMP study area.

The current hydrology would be acceptable for high level planning and catchment scale flood assessment.

- Development and conceptual design of different flood management options (including being used for analysis of potential flood damages).

The present hydrology is considered fit for this item, if used with caution. Any conceptual design of flood management options should be high level. Once the hydrology is updated, the optimal flood management options might be different in method, location or scale. Certain specific flood management solutions might be pursued where GW is confident the hydrology refinements would have less effect.

- Providing information for use by District Councils in LIMs, land use controls and for building controls (Local Government Act, Resource Management Act and Building Act requirements).

The current hydrology is not deemed suitable for these uses, as the level of detail sought is greater and flood hazard information may change when modelling is refined.

- To provide potential update to existing flood hazard information in the District Plan.

The current hydrology is not considered appropriate for use in the District Plan.

As for the hydraulic review, technical recommendations for the hydrology have been classified as minor (only likely to have localised effect), moderate (has potential to impact over a wider area); or major (has potential to have significant impact on overall results of the modelling). General recommendations have also been made. A detailed summary of all recommendations made in the audit is presented in Appendix G. With some work to address the major recommendations below, it is believed that the hydrology could be made fit for purpose for the four levels of use described above. For general improvement in the hydrology, it is recommended that the minor and moderate recommendations also be put into effect, or suitable reasons be given for their omission.

1.6.1 HYDROLOGY SOFTWARE AND METHODOLOGY

The hydrology software used is generally fit for purpose.

The hydrology methodology in general uses industry standard techniques, but there has been some extension or simplification of these techniques which mean that the hydrology, as it stands is not entirely fit for purpose. This must be viewed in the context of the Ruamāhanga catchment, with difficult and shifting gauging sites, and often a lack of reliable hydrometric data to support flood estimates. When working with uncertain data, or where conflicting results are found, it is also important to determine a confidence interval for the output. Validation of the models is also critical, to ensure that the hydrology gives a credible representation of catchment response.

1.6.2 EARLIER PEER REVIEW

Each of the major hydrological studies provided has been peer reviewed at an earlier stage. From these peer reviews, a summary of issues raised has been made. The summary aims to show whether these issues were addressed. Because of the format and length of this summary, it is provided as a separate document, in Appendix F.

An observation on the previous peer reviews is that they did not make firm recommendations, but that the reviewers did advise improvements or query some aspects of the work in each case. Much of this advice was not followed. In the case of the Upper Wairarapa hydrology investigations conducted by PDP in 2013, there was minimal contact between the peer reviewer and PDP. One aspect of that work – the regional flood frequency parameter contours – was changed as a result of the peer review.

In summary of peer review comments made by NIWA on the PDP hydrology (2013) (generally used now for the Upper Ruamahanga catchments apart from the Waipoua), the following main points were noted:

- The broad hydrology method was supported by the NIWA peer review
- Further study of flood peak timing, river flood dynamics and weather patterns would be of benefit
- Developing scaled design hydrographs is a good technique for at-site design flood modelling.
- A study on volume and time of concentration would assist in transfer of design flood hydrographs to ungauged sites.
- More information on the chosen flood frequency distributions could have been provided, including supporting parameters and comparison plots.

In summing up the peer review comments made by Brin Williman and T+T on work focussed on the Waipoua hydrology, the following outstanding points were found:

- Potential to improve the rainfall-runoff model of the Waipoua, where the T+T reviewer questioned these aspects:

  -loss model,

  -number of catchments

  -lack of raingauges supporting hydrology

  -rainfall temporal distribution

  -rainfall weightings

  -lack of Areal Reduction Factors

- In depth hydraulic analysis of flows at Mikimiki
- More thorough collection of calibration data for historic floods like the October 1998 event, including stopbank and flood levels, bank damage, confirmation of overflows and flood extents
- Importance of benchmarking the 1998 flood to a return period, not to influence the flood frequency distribution, but to provide an understanding of the level of protection existing relative to that flood and to assist in planning.

Changes made that are aligned with previous peer reviewers' advice on the Waipoua hydrology are:

- Development of a rainfall-runoff model for the Waipoua catchment
- The Colombo Rd Bridge gauge on the Waipoua was reinstated in 2015.
- An uncertainty range was supplied for the 50 and 100 year ARI design flood estimates on the Waipoua.
- Agreement has been reached between GW and MDC on appropriate design floods and climate warming effects.

As a result of the lack of clear recommendations made, valuable and constructive critical advice from earlier peer reviews was not taken up by GW. This reviewer generally agrees with the peer review findings on the hydrology to date, and the current review builds on these.

1.6.3 GENERAL UPPER RUAMĀHANGA HYDROLOGY

For the broader Upper Ruamāhanga hydrology modelling, aspects of the methodology could be refined. The main points identified are how the regional flood frequency parameters are selected for the subcatchments to estimate design floods, that hydrograph shape should reflect catchment characteristics, such as time of concentration, and the AEP coincidence across the subcatchments.

The following minor recommendations have been made for the Upper Ruamāhanga hydrology

- That the flows from the flood frequency distribution for Kopuaranga at Palmers be corrected.
- Simplify the sub catchment representation if scaled hydrographs are to be used.
- Collect relevant regional flood frequency contours and underlying data into a single figure to assist understanding.
- Hydrological boundary conditions used as input to the hydraulic models be checked for consistency and correctness.
- Investigate the cause of the drift in the Taueru at Te Wheraiti record and decide whether any adjustment needs to be made.

The following moderate recommendations have been made for the Upper Ruamāhanga hydrology

- Estimate confidence intervals for the PE3 flood frequency distributions.
- The underlying station data and other catchment characteristics must be taken into consideration in the selection of flood frequency characteristics from contours. Used in isolation the contours can generate anomalous results.
- That GW carry out an evaluation of whether a rainfall-runoff model or the current regional flood frequency based hydrology method, or some combination, is the best way to deliver hydrology inputs to the FMP.
- The AEP coincidence tables are not proven for higher return periods, and have a linear basis which is not sound. Simplification of this approach in design flood modelling has been overconservative. Consider using the AEP coincidence tables more systematically and developing an alternative method for scaling up AEP coincident flows.
- The Ruamahanga hydrology combines flood frequency parameter contours, scaled station hydrographs, fine catchment divisions, and conservative AEP coincidence assumptions. These methods have a high cumulative uncertainty. The model ought to be validated to historic flood events, and attempts should be made to refine the hydrology input.
- Review GW climate warming impact projections for increased extreme rainfall and sea level rise in light of new MfE guidance.
- Investigate the response of Upper Ruamāhanga river flows to increased design rainfall, using rainfall-runoff modelling or similar. This would provide a stronger relationship between projected increases in climate warming and river flows.
- Validate any updated hydrology against historic events to ensure the outputs are realistic.

The following major recommendations have been made for the Upper Ruamāhanga hydrology

- Revisit application of regional flood frequency parameters to model subcatchments, paying particular attention to location of source data and catchment characteristics such as elevation, aspect and shape.

Check the resulting design flood estimates with reference to adjacent catchments, neighbouring stations and other return periods to ensure that results are sensible.

- Design hydrographs should reflect subcatchment characteristics, such as time of concentration. This can be estimated by theoretical equations or rainfall-runoff modelling.

1.6.4 WAIPOUA HYDROLOGY RECOMMENDATIONS

For the Waipoua hydrology modelling, development of a rainfall-runoff model for the catchment is to be commended, but the type of model could be improved. The aim would be a model flexible enough to represent catchment runoff behaviour, including changes in the soil infiltration capacity over time.

The following minor recommendations have been made for the Waipoua hydrology

- Review local rain records to validate the temporal rainfall distribution used for the Waipoua catchment, or consider a symmetrical temporal distribution.

The following moderate recommendations have been made for the Waipoua hydrology

- Reconsider whether Areal Reduction Factors should be applied to point rainfalls used in Waipoua catchment modelling, and update rainfall depths if required.
- Including historic flood events could improve confidence in the flood frequency distribution for the Waipoua at Mikimiki. An in-depth investigation of historic sources would be needed to turn up any useful information. This investigation would not be limited to the 1947 event.

The following major recommendations have been made for the Waipoua hydrology

- That the Waipoua rainfall-runoff model be revisited to provide a better representation of the physical catchment processes. Simplify the subcatchment delineation. Also ensure that the way the runoff hydrographs are applied in the hydraulic model allows for reasonable travel time and storage.
- That after refinement of the Waipoua rainfall-runoff model, the model be validated against at least those historic events used previously (October 1998 and September 2010). Ideally, more than two calibration events should be used.

(The aim of these major recommendations would be that hydrology linked to the Waipoua flood modelling be robust enough that it does not need to be adjusted before use in the hydraulic model).

The following general recommendations are also made

- Maintain regular gauging of sites and provide an estimate of the uncertainty in the rating.
- As soon as more data is available for the Colombo Rd site, use this to refine the estimate of flow contribution from the lower Waipoua catchment.

  A longer record at Colombo Rd could also be used to provide calibration data downstream of the ungauged catchment area. This could aid in validation of the hydrological and hydraulic models, so that these tools become more accurate.

Ideally, the site should be telemetered to facilitate access to this data and it should be gauged regularly to get a grasp on the site characteristics and stability.

1.7 IMPACT OF HYDROLOGY REVIEW ON HYDRAULIC MODEL OUTPUTS

The following general comments in regard to how the general hydrological review may impact on the overall floodmaps.

1.7.1 GENERAL UPPER RUAMĀHANGA HYDROLOGY

In general, the recommendations which have been made by the reviewer are likely to decrease the general level of uncertainty in the overall results. The review generally implies that the adopted methodologies have resulted in conservative hydrological inputs into the model. Incorporating the recommendations has the potential to reduce the overall floodspread or lower flood levels to a degree. This implies that the current floodmaps are likely to be a conservative estimate of the overall floodrisk.

The major recommendations made by the reviewer all relate to the application of the hydrology to the hydraulic model, rather than the underlying hydrology models and should be able to be addressed. It is my opinion that addressing these major recommendations would be more likely to reduce flood extents than to increase them, again implying that the current flood maps could be considered as conservative.

1.7.2 WAIPOUA HYDROLOGY RECOMMENDATIONS

Once again, it is my opinion that if the recommendations of the peer reviewer are implemented then the final flood maps will have a lesser level of associated uncertainty. Investing some more time into improving the overall setup of the rainfall runoff model would also allow for less interpretation being required from the hydraulic modeller.

Overall, it is my opinion that incorporating the major recommendations made in the hydraulic section of the review for the Waipoua model will be likely to have a greater impact on the overall flood extent for Masterton than the recommended changes to the input hydrology, however this in no way discounts the importance of the recommendations, which will impact on the overall results over a greater area of the Waipoua catchment.

1.8 OVERALL CONCLUSION

1.8.1 GENERAL UPPER RUAMĀHANGA FLOOD MAPS (EXCLUDING THE WAIPOUA RIVER)

As they stand we conclude that these maps can be considered fit for the purpose of;

- Catchment-scale flood hazard assessment, including classification of hazard into different categories, for the Waipoua River and other rivers in the FMP study area.

We also consider that as long as caution is used when interpreting the results (Any conceptual design of flood management options should be high level) then the maps are fit for the purpose of;

- Development and conceptual design of different flood management options (including being used for analysis of potential flood damages)

Further work would be required to bring the models up to standard if they are to be used for the following;

- Providing information for use by District Councils in LIMs, land use controls and for building controls (Local Government Act, Resource Management Act and Building Act requirements)
- To provide potential update to existing flood hazard information in the District Plan

1.8.2 WAIPOUA RIVER FLOOD MAPS

As they stand we do not consider the maps to be fit for purpose. If the major recommendations in the Hydraulic Modelling section of the report are implemented, then we consider the maps will be fit for the purpose of;

- Catchment-scale flood hazard assessment, including classification of hazard into different categories, for the Waipoua River and other rivers in the FMP study area.

We also consider that as long as caution is used when interpreting the results (Any conceptual design of flood management options should be high level) then the maps will be fit for the purpose of (once major hydraulic recommendations implemented);

- Development and conceptual design of different flood management options (including being used for analysis of potential flood damages)

Further work (in particular relating to flood hydrology) would be required to bring the models up to standard if they are to be used for the following;

- Providing information for use by District Councils in LIMs, land use controls and for building controls (Local Government Act, Resource Management Act and Building Act requirements)
- To provide potential update to existing flood hazard information in the District Plan

2 INTRODUCTION

2.1 OBJECTIVE

Land River Sea Consulting in conjunction with Barnett and MacMurray Ltd have been contracted by Greater Wellington Regional Council to carry out an independent audit of the Te Kauru FMP project.

The council has stated that "*it is Council policy to carry out an independent review of all flood hazard modelling. This approach is taken to improve the robustness of the information and to give confidence to decision makers and the public that the information is fit for purpose. This audit is to contain a review of the hydrology, hydraulic model, application of freeboard and flood mapping.*"

The project has essentially been split into two parts, with the hydrology component of the audit being conducted by Vicki Henderson and Hugh MacMurray from Barnett and MacMurray Ltd and the audit of the Hydraulic model build and overall approach of the project has been led by Matthew Gardner from Land River Sea Consulting Ltd

2.2 INFORMATION USED IN AUDIT

The following documents have been made available to the auditors

HYDRAULIC MODELLING DOCUMENTS

Supplied by GWRC

- TKURFMP Rectangular Grid Upper Model Hydraulic Modelling Report (Borrer, 2013)
- TKURFMP Rectangular Grid Lower Model Hydraulic Modelling Report (Borrer, 2013)
- Rectangular grid hydraulic model peer review (Macky, 2014)
- TKURFMP Waipoua Flexible Mesh Hydraulic Modelling Report (DRAFT)
- Flexible mesh hydraulic model initial review (Rix, 2018)
- Sensitivity test modelling memo (included in Flexible Mesh Report)
- Flexible mesh hydraulic model final review (Rix, 2018)
- The Waipoua River & Floodplain Investigation - Phase 1 – Issues (Ian Heslop, 1996)

The following data was supplied to the auditor by members of the community

A copy of all data supplied by the local community is presented in Appendix D.

HYDROLOGICAL MODELLING DOCUMENTS

Supplied by GWRC

- Draft hydrology report for UWVFMP from PDP (April 2013)
- NIWA review on PDP Hydrology Report (May 2013)
- PMF report by Laura Keenan (July 2013)
- NIWA review of PMF memo produced by Laura Keenan (July 2013)
- NIWA report on the 1998 event (March 2015)
- Brin Williman review of NIWA hydrology report (8 May 2015)

- Final Waipoua Rainfall Runoff Modelling report – MWH (August 2016)
- T+T peer review of MWH hydrology report (April 2016)
- 1947 flood level – supplementary information and comments, Opus
- Information on the 1947 flood, Opus (August 2018)
- Historical information about the Waipoua River, Opus (September 2018)

The reviewer requested some further GW documents and resources to provide detail and confirm how the hydrology from the source studies was used in the flood models. These documents are shown below.

- Te Kauru Upper Ruamāhanga Floodplain Management Plan: Hydraulic modelling report - Lower model (Oct 2013)
- Te Kauru Upper Ruamāhanga Floodplain Management Plan: Waipoua Flexible Mesh Hydraulic Modelling report (Dec 2018)
- Internal report hydrological statistics for surface water monitoring sites in the Wellington region 2011 summary (Aug 2012)
- Stage and flow record, annual maxima series for Waipoua at Mikimiki station
- Gaugings, slope-area calculations on the Waipoua.

2.3 RECOMMENDATIONS

Recommendations in this report have been classified as general (non-technical or needs more data), minor (only likely to have localised effect), moderate (has potential to impact over a wider area); or major (has potential to have significant impact on overall results of the modelling). A detailed summary of all recommendations made in the audit is presented in Appendix G.

2.4 FIT FOR PURPOSE

Statements made in regard to being fit for purpose in this report relate to the purpose detailed in the RFQ which states that the main purposes of the model are:

- Catchment-scale flood hazard assessment, including classification of hazard into different categories, for the Waipoua River and other rivers in the FMP study area.
- Development and conceptual design of different flood management options (including being used for analysis of potential flood damages)
- Providing information for use by District Councils in LIMs, land use controls and for building controls (Local Government Act, Resource Management Act and Building Act requirements)
- To provide potential update to existing flood hazard information in the District Plan

3 MODEL FREEBOARD

It is well acknowledged that any flood model contains a significant amount of uncertainty due to a range of factors outside of the modellers control. These factors can include but are not limited to;

- Wave action
- Wind setup
- Debris
- Data uncertainty (i.e. LiDAR levels)
- Uncertainty in discharge estimate or rainfall intensities
- Uncertainty in river behaviour in higher flows due to turbulence etc
- Potential lateral erosion, or embankment breach
- Potential physical changes in catchment (i.e. bed level changes)
- Potential blockage of structures
- Simplification of numerical representation of complex structures

In order to account for the wide range of potential uncertainties in the model results, it is considered best practice to make an allowance for model freeboard. The exact methods used to apply model freeboard in New Zealand are not consistent, however it is widely agreed that the inclusion of model freeboard is essential.

Methods used around New Zealand include;

1. Allowing for peak water levels to be increased by a fixed level (i.e. 0.5m) and extrapolated horizontally
2. Allowing for peak water levels to be increased by a fixed level and allowed to spill over the floodplain so that the results taper off (this is a less conservative approach)
3. Using the hydraulic model to simulate the wide range of potential uncertainties and using the model to assess the potential range of uncertainty across the entire model

None of the approaches will ever give a perfect representation of reality as they are simply a numerical model, however the techniques at least give an estimate of potential / likely uncertainties and are less likely to significantly underestimate the potential flood risk.

APPLIED METHOD IN TE KAURU FMP

The current modelling reports which have been reviewed as part of this audit have described two separate approaches to model freeboard / sensitivity with the regular grid modelling incorporating a different approach from the Flexible Mesh Modelling (Waipoua Model).

3..1 REGULAR GRID MODEL FREEBOARD METHOD

Both the 'Upper ' and 'Lower' modelling reports describe the same technique for the application of model freeboard. An assessment of likely uncertainty due to uncertainty in Manning's 'n' as well as discharge was determined and then applied to the model as described below;

"In order to model freeboard for this study, peak modelled channel and floodplain levels were extracted from the 'without freeboard' simulation results. The freeboard values determined…were added to these peak modelled levels and the resulting levels were used as the starting conditions for the 'with freeboard' model run. All other model inflows were set to zero for the freeboard runs. The model simulation was ended once the results showed that water levels were receding everywhere, and the peak levels reached across the floodplain are the final 'with freeboard' water surface." (Borrer, 2018)

3..2 FLEXIBLE MESH MODEL FREEBOARD METHOD

The flexible mesh modelling report has detailed a total of 22 sensitivity runs which have been simulated in the model in order to better understand the sensitivity of the flood levels to a range of potential scenarios.

In order to apply the final freeboard, the report details;

"The results of the sensitivity tests were discussed during a WOWG meeting (date) and it was agreed that the most significant sensitivity tests were:

- *Increased Manning's n*
- *Rail bridge blockage*
- *Increased flow at Mikimiki (with ungauged catchment contribution not increased)*

These three tests were re-run with the altered baseline model (e.g. including 10% rail bridge blockage) to give final sensitivity tests results for the 1% AEP with climate change event." (Borrer, 2013)

It is understood that the results of these three scenarios were then combined to make a final flood sensitivity extent which is the equivalent of model freeboard.

COMMENTARY ON THE ADOPTED FREEBOARD METHODS

It is my opinion that both of the adopted methods are suitable methods for applying freeboard. Independent to this audit, I have recently carried out a comparison of both methods using the Waiohine River model. This comparison concluded that that both techniques give very similar results for the Waiohine River. I believe it is likely that both approaches would be very likely to give fairly similar results.

My personal preference is to use the approach adopted in the flexible mesh model as the model itself is used to dynamically assess uncertainty. This allows for variable increases in water level across the floodplain, which are dependant on the actual physical features rather than a blanket assessment. It must always be acknowledged however that there will always be uncertainty in model results which cannot be easily simulated and caution should always be applied around the 'edges' of model results.

In addition to the three scenarios adopted I would recommend that a wider range of potential uncertainties are included in the final sensitivity layer which for the Waipoua model should include;

- Blockage of the State Highway Piers
- Blockage of the Colombo Road Bridge Piers
- Blockage of the Colombo Road Bridge Soffit due to floating debris (typically simulated by lowering the bridge soffit)

- Localised increases in bed levels (potentially informed by geomorphology analysis or assessment of historical bed level data)
- Effect of different temporal pattern (ie double peak hydrograph – or longer duration event_

Careful thought would need to be given as to what sensitivity scenarios should be included in the final maps to the regular grid modelling.

Discussions with Susan Borrer have indicated that some of the regular grid upper model reruns have now adopted the sensitivity approach which has been used in the Fleximesh modelling. This also has been documented in the Waipoua Fleximesh report.

GENERAL RECOMMENDATION 1

That a consistent / documented approach to model freeboard is adopted for all models in the FMP

That the sensitivity approach adopted in the flexible mesh model report is adopted for all models

That further scenarios which investigate more blockage scenarios, bed level changes and changes in temporal pattern are incorporated into the final sensitivity layer.

4 PRESENTATION OF FLOOD HAZARD INFORMATION

EXISTING FLOOD HAZARD PRESENTATION

The assessment of the manner in which the flood hazard information has been presented to the public has been made based on the following DRAFT Waipoua Flood Hazard maps:

1. DRAFT 1% Annual Chance Base Flood Spread
2. DRAFT 1% Annual Chance Flood Spread with Sensitivity Scenarios
3. DRAFT 1% Annual Chance Flood Spread with Sensitivity Scenarios and Base Flood Spread
4. DRAFT Future 1% Annual Chance Base Flood Spread
5. DRAFT Future 1% Annual Chance with Sensitivity Scenarios Flood Spread
6. DRAFT Future 1% Annual Chance with Sensitivity Scenarios and Base Climate Change Flood Spreads
7. DRAFT Future 1% Annual Chance with Sensitivity Scenarios and Base Flood Spreads
8. DRAFT Future 1% Annual Chance Flood Depth with Sensitivity Scenarios (showing depth)
9. 2014 Flood Spread and 2019 Flood Spread

These maps are included in APPENDIX B. In general, the maps only present flood extent as a single colour except for Map 8 which also includes graduated colours indicating flood depth.

COMMENTARY ON FLOOD HAZARD PRESENTATION

The provided maps are clear and easy to read and give a general understanding of the likely flood extent, with map 8 also presenting flood depth. In general, these maps are fit for purpose, however my opinion is that further information could be made available to give a more detailed representation of the nature of the flood hazard and to make better use of the detailed data available in the existing model results.

The existing maps do not show any road names, previous feedback I have received from the general public is that road names can greatly assist in reading flood maps. Some transparency on the overall flood layer also assists in seeing landmarks which are obscured by the flood layer.

GENERAL RECOMMENDATION 2

Consideration is given to adding a degree of transparency to the flood layer so that landmarks can be easily identified as well as adding road names to allow easy orientation for local landowners.

The following commentary provides some ideas on how the presentation flood hazard information could be better portray the complex hazard information.

4..1 FLOOD EXTENT MAPS

Flood extent maps on their own do not portray a significant amount of information in relation to the nature of the flood risk at any single location, however they are easy to read and are suitable for many people to easily interpret if there is a flood hazard in the location of interest or not .

4..2 FLOOD DEPTH MAPS

Flood depth maps can present both flood extent, and flood depth information at the same time. The example provided uses a graduated colour scale to portray depth. For many people a graduated colour scale is simple to read, however there is the risk that differentiating between different shades of the same colour can be difficult and can lead to confusion around likely flood depths.

My preference is for a range of colours to be used rather than a graduated colour scale. One example of a colour scheme is the colours used in the recent Waiohine flood modelling exercise. The colour scheme used in these floodmaps was developed in consultation with staff from GWRC as well as the local community. An example of these colours is presented in Figure 4-1 below.

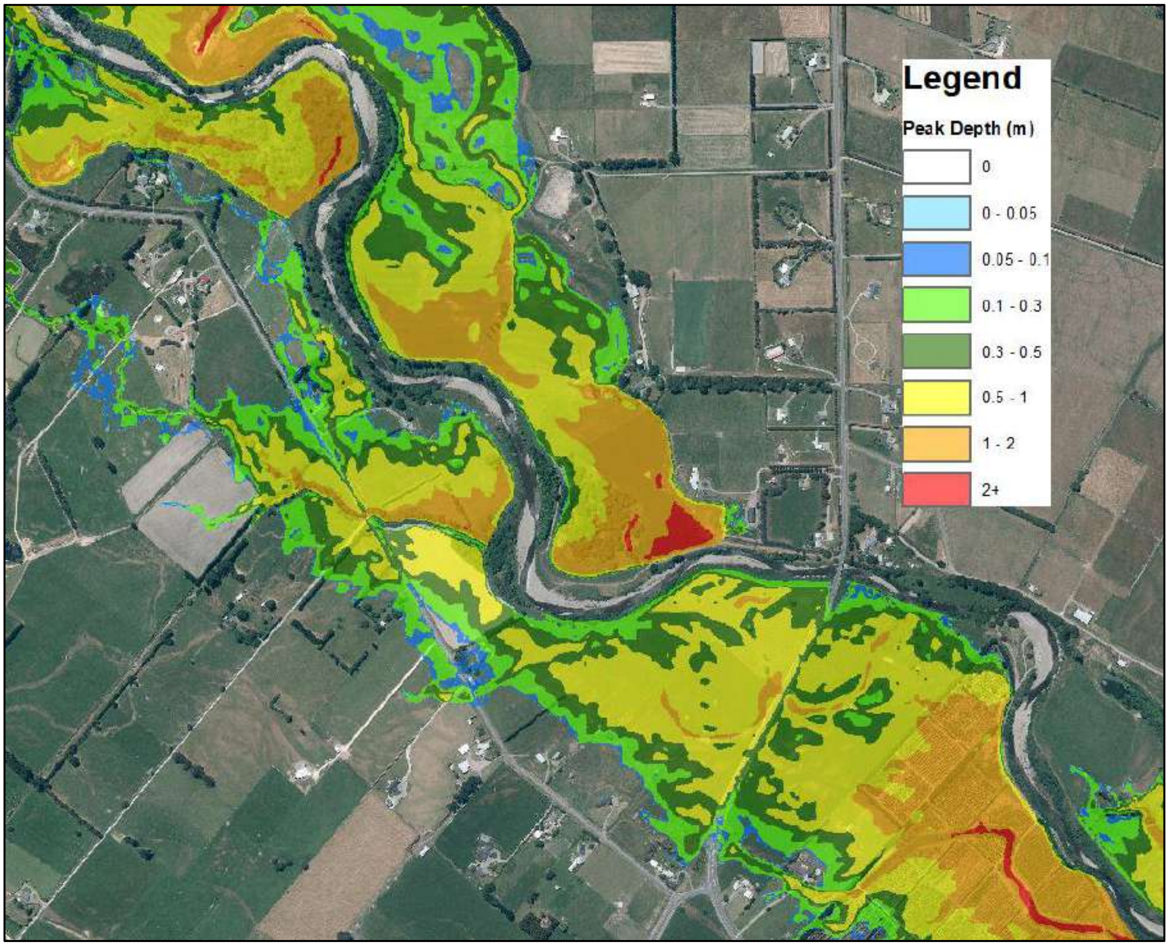


Figure 4-1 – Example of colour scheme used in recent Waihoine River modelling

GENERAL RECOMMENDATION 3
Consideration is given to presenting the flood depth maps using a range of discrete colours rather than a graduated colour scale

4..3 PEAK WATER SPEED / VELOCITY MAPS

Flood extent and depth is only part of the hazard information. The speed of the water is also essential information which helps portray the severity of the hazard.

A range of colours can be used to differentiate between speeds; further consideration can be given to also displaying vectors which also give information about the main direction of flow. The risk of showing vectors (arrows) is that the map can become cluttered. Velocity vectors are particularly useful when looking at a localised area of the model, rather than regional wide maps. They can be useful in portraying the nature of flooding to a local community during a consultation phase for example.

GENERAL RECOMMENDATION 4

Flood velocity maps are generated showing both speed and direction of flow

4..4 FLOOD HAZARD MAPS

The use of flood hazard maps which categorise the model results into specific hazard levels can be a very useful way to communicate the overall nature of the flood hazard.

There are a large number of potential hazard categorisations to use, most of which take into account flood depth and velocity with some having an allowance for debris potential also. Historically GWRC has used a simplified hazard categorisation based on dividing the flood hazard into ‘ponding’ and ‘overflow’ areas, however I believe consideration should be given to presenting flood data in a way that portrays more information about the overall hazard.

My preferred approach is taken from the Australian Rainfall and Runoff Guidelines (Cox, 2016) with hazard categories based on a combination of depth and velocity. The categories are described in easily understandable language and it is my understanding that this categorisation has used a number of real flood events in Australia in recent years to assist in the definition of the categories. The hazard categories are summarised in Table 4-1 and presented graphically in Figure 4-2 below.

Table 4-1 - Description of Hazard Categories

Hazard Vulnerability Classification	*Description*
H1	Generally safe for vehicles, people and buildings.
H2	Unsafe for small vehicles.
H3	Unsafe for vehicles, children and the elderly.
H4	Unsafe for vehicles and people.
H5	Unsafe for vehicles and people. All buildings vulnerable to structural damage. Some less robust buildings subject to failure.
H6	Unsafe for vehicles and people. All building types considered vulnerable to failure.

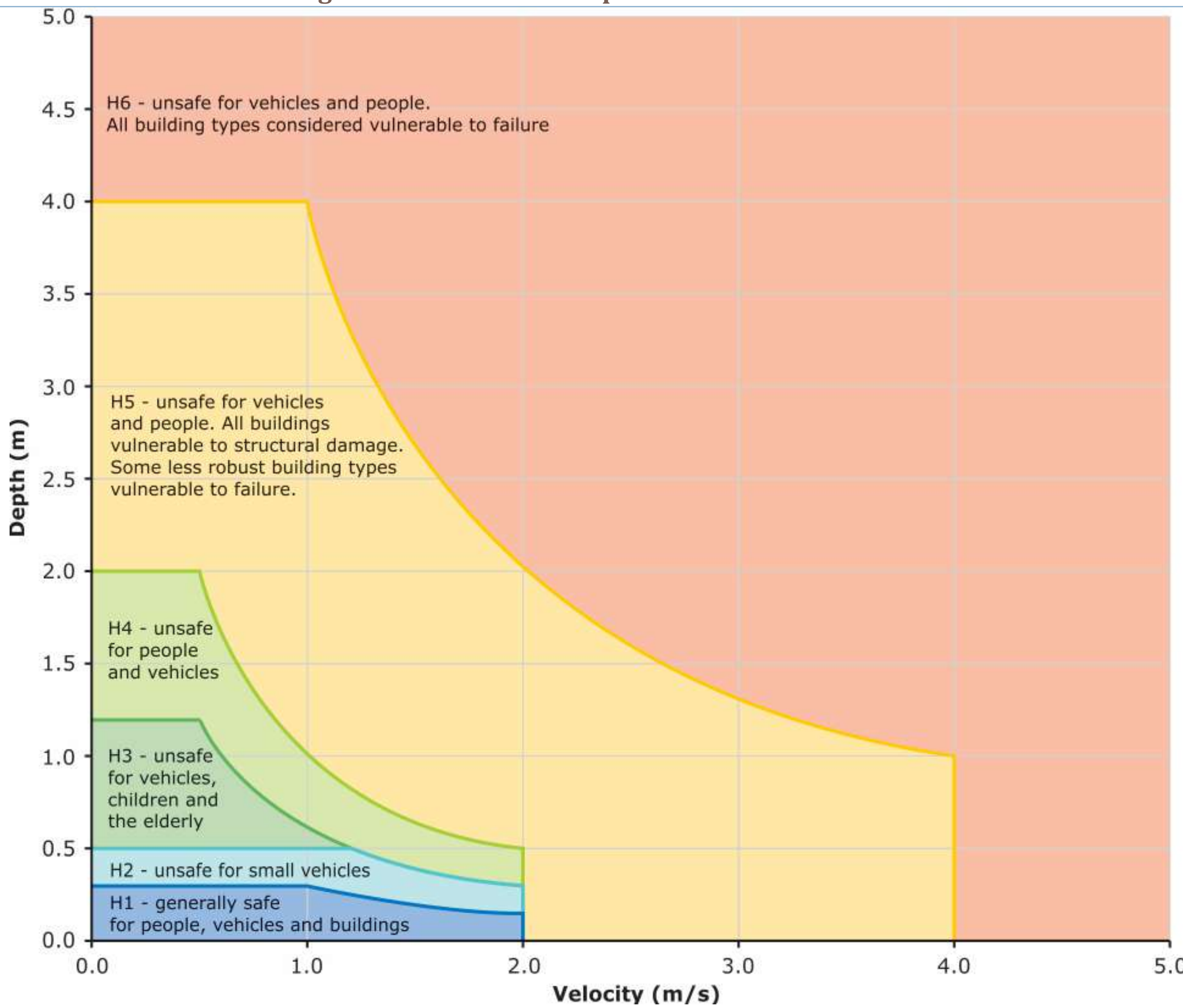


Figure 4-2 - Graphical representation of the Hazard Categories

More detailed information on the derivation of the Hazard Categories can be found in the Australian Rainfall and Runoff guidelines which can be accessed online at http://arr.ga.gov.au/arr-guideline (NB. hazard categories are discussed in Chapter 7 of Book 6 – Hydraulics).

There are a range of more specific hazard categorisations available which are more specific for evacuation planning etc, however I believe that the categories presented above is the most general, is easily understandable and is suitable for a wide range of purposes. However, I acknowledge that other categorisations may also be more suitable for specific purposes and should also be considered.

GENERAL RECOMMENDATION 5

Flood hazard maps based on a combination of speed and depth are generated in order to communicate the risk associated with the hazard.

4..5 ANIMATIONS OF FLOOD HAZARD PROPOGATION

With the increasing use of online media by the general public, I believe it would be prudent to consider hosting even more detailed content online. Videos showing the propagation of a flood over a floodplain provides even more information to the general public about the nature of the flood hazard and allows them to better visualise and understand the potential nature of the flood risk.

GENERAL RECOMMENDATION 6
Consideration is given to publishing videos online showing how the flood propagates in the model over time.

ELECTRONIC FILES FOR INHOUSE USE

It has been noted during the audit process that due to the complex nature of this project, there are a large number of model result files which look at a range of different scenarios and iterations. Many of these files overlap, and judgement is required to be made in certain areas (such as at the Waiohine Confluence) as to which result file will take preference.

Due to the large number of models included in this study, this is completely understandable, however it may be confusing for staff within GWRC who may need to rapidly respond to flood information requests in the future.

Consideration should be given to generating a single, merged flood hazard raster to be used within ArcGIS. This would require the complex decision making in areas where there are overlapping data sources to be made now and only once (rather than each time a request for information comes in).

This would reduce the need for complex decision making to be made by GWRC staff whenever requests for information come in allowing flood information to be disseminated more readily when needed.

GENERAL RECOMMENDATION 7
That all model results are merged into a single file. Decisions around which result file takes precedence in areas where models overlap could be made now, rather than relying on interpretation when requests for information come in.

5 RECTANGULAR GRID UPPER HYDRAULIC MODELLING

This model covers the following rivers:

- Upper Ruamāhanga
- Kopuaranga
- Whangaehu

The modelling report that has been provided is dated October 2013 (Borrer, 2013). In addition to the modelling report was a peer review report carried out by DHI (Macky, 2014).

The report also covers modelling for the Waipoua River, however as this model has now been superseded by the Fleximesh modelling, this section of the report has not been reviewed

The audit of this section of the project has included a detailed reading of the modelling report and peer review documents, as well as an interrogation of the actual model files themselves and a number of result files.

The model setup files for the 100 year scenario including the effects of climate change were provided to the auditor (Western_Q100_CC.couple and Eastern_Q100_CC.couple).

Both the MIKE11 and MIKE21 result files for the 50 year and 100 year CC events have been supplied for both the Eastern and Western Simulations.

SOFTWARE AND GENERAL MODELLING METHODOLOGY

The modelling has been carried out using the MIKE Flood software package by DHI. This is an industry standard software which is suitable for this type of modelling.

A general audit of the model setup has been carried out and in general the modelling methodology fits with industry best practice.

Overall the model is large and complex with a large number of structures and complexities present in the setup. Overall the model appears to perform well and give realistic flood extents.

ROUGHNESS COEFFICIENTS

The roughness values adopted within both the 1D and 2D component of the model are within reasonable ranges and appear to be sensible. A detailed comparison of the 2D roughness with the aerial imagery has not been carried out as part of this audit, however this has already been covered to a degree in the initial peer review (Macky, 2014).

The following sections comment on and make recommendations on specifics relating to each individual main river within the model

UPPER RUAMĀHANGA RIVER MODEL SETUP

5..1 REPRESENTATION OF STRUCTURES (UPPER RUAMĀHANGA RIVER)

No structures have been included in the Ruamāhanga River model. The model report states that checks have been made on the model results that the water levels are unlikely to reach the level of the soffits nor act as a significant obstruction to flow.

Whilst there is validity for ignoring the bridges, there are also reasons why including the bridges in the model could be a good idea. These include;

- Allow for the model to determine the sensitivity of water levels to the bridge piers;
- Allows for a consistent representation of all structures in the FMP;
- Allows for blockage scenarios to be investigated and included in sensitivity runs;

Considering the bridges are fairly quick and simple to setup in the model and are unlikely to have a significant impact on model stability, I believe consideration could be given to including them.

MINOR RECOMMENDATION 1

Consideration should be given to including the bridges into the model to allow for the localised impact and to better represent the headloss through the structure.

5..2 REPRESENTATION OF LATERAL LINKS (UPPER RUAMĀHANGA RIVER)

LATERAL LINK CONNECTIONS

A basic check of the setup of the lateral link connections has been carried out for the Ruamāhanga River. The lateral link connections file has been converted to a shapefile and visualised within ArcGIS. The lateral links appear to be working as anticipated for the entire length of the river. A visualisation of the lateral link connection for a section of the Ruamāhanga River branch is presented in Figure 5-1 below (The yellow lines connect the floodplain to the channel centreline, and ideally should be perpendicular to the main direction of flow as is the case in this example).

Figure 5-1 – Visualisation of Lateral Link Connections – Ruamāhanga MIKE11 Chainage - 17013 - 17446

LATERAL LINK ELEVATION SOURCE

It is noted that the current model setup generally uses the HGH method for defining the land elevation at the lateral links. This forces the model to choose the highest level of either the MIKE11 cross section or the MIKE21 terrain level. Due to the relatively wide spacing between surveyed cross sections this can results in higher or lower ground levels being estimated at the link locations between sections than the physical reality. Due to the high resolution of ground level available from the LiDAR data, the M21 method is likely to more accurately represent the physical reality and should be adopted for this model.

The peer review document (Macky, 2014) also made the recommendation that unless the MIKE11 model is known to accurately represent the berm levels, then the M21 model should be selected as the source for the link elevations rather than the HGH method.

MODERATE RECOMMENDATION 1

That the lateral link elevations for the Ruamāhanga River are changed to use the M21 method for their source as recommended in the DHI peer review.

5..3 MODEL CALIBRATION / VALIDATION (RUAMĀHANGA RIVER)

The report states that the previous modelling was calibrated to the 1994 flood event however due to changes in the river system these levels are unlikely to be helpful for model calibration.

There is no documentation that the model has been validated to historic flood photos or records, however discussions with the modeller indicate a degree of comparison with flood photos and discussions with GWRC operations staff has taken place. Documentation of this process would be very useful.

I have had discussions with staff from the GWRC Masterton office in regards to changes in the Ruamāhanga River system and the feedback I received was that there have been some localised changes however in general the system is fairly similar to what it was in 1994.

Consideration should be given to checking the model calibration against the 1994 event using the same cross section database used in the 1995 model. (It is understood that the 1994 model is unable to be opened due to changes in the software since 1994, it may be necessary to go back to the cross section survey data, rather than use the 1994 model if that cannot be easily accessed).

It would be also useful to compare the adopted Manning's 'n' with the 1995 model.

In addition to the debris levels on file, there are a number of flood photos on file from historic events as well as the flood protection shape file of estimated 50-year flood extent (which has been reported to me as being based on historic flood extents as well as other factors).

A comparison of a select area of the Ruamāhanga model results is made with the GWRC 50-year flood zone layer in Figure 5-7 below. In this location the modelled flood extent is significantly narrower than that in the 50 year flood extent on record. The reason for the reduction in flood extent may need to be investigated. This does not necessarily indicate the modelling is wrong in this location as the reason for the differences may be easily be able to be explained (ie due to more detailed data currently available, or changes in the catchment).

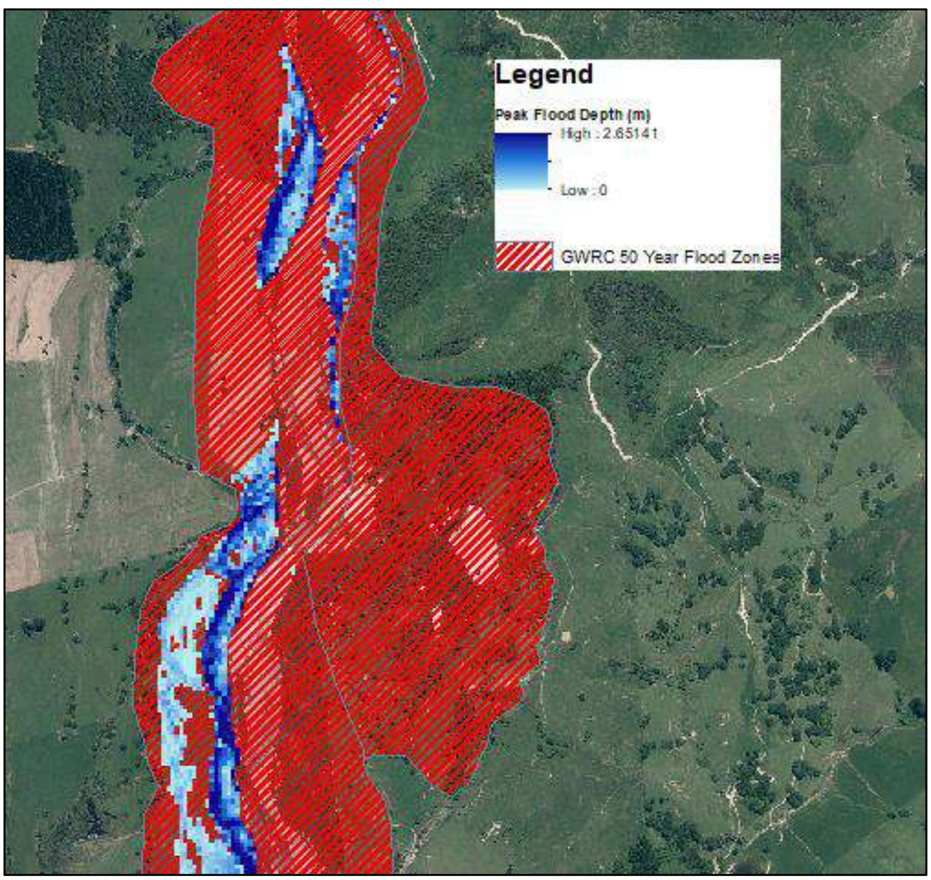


Figure 5-2 – Comparison of GWRC 50 year flood extent layer with latest 50 year model results (Upper Ruamāhanga River)

MODERATE RECOMMENDATION 2

Further justification needs to be given to ignoring the 1994 debris recordings for model calibration.

A comparison with the adopted Manning's 'n' values should be made with the 1995 MIKE11 model.

Before the results are finalised, it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;

-Comparison with historic flood photos / debris levels

-Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)

-Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods

KOPUARANGA MODEL SETUP

5..1 REPRESENTATION OF STRUCTURES (KOPUARANGA RIVER)

The model setup has a mixed setup using eight irregular shaped culverts to represent bridges, as well as 6 bridges. It is considered beyond the scope of this audit to do a detailed review of each structure in the model however a basic check of each structure has been carried out.

Each bridge/culvert has been plotted in relation to the upstream and downstream cross sections and the overall setup appears to be reasonable.

The MIKE11 results have also been checked at each structure to check for any obvious instabilities. The MIKE11 results have highlighted some issues at the culverts labelled 'KopuBridges4221 and KopuBridges 4167'. Water level results present some instabilities as shown in Figure 5-3. A closer inspection of the bridge setup indicates the following;

- The two culverts have cross sections spaced only 1m upstream of the structure – this would require a very small dt in order to compute;
- There is also no slope across the culvert. A small slope on the culvert will likely assist with stability.

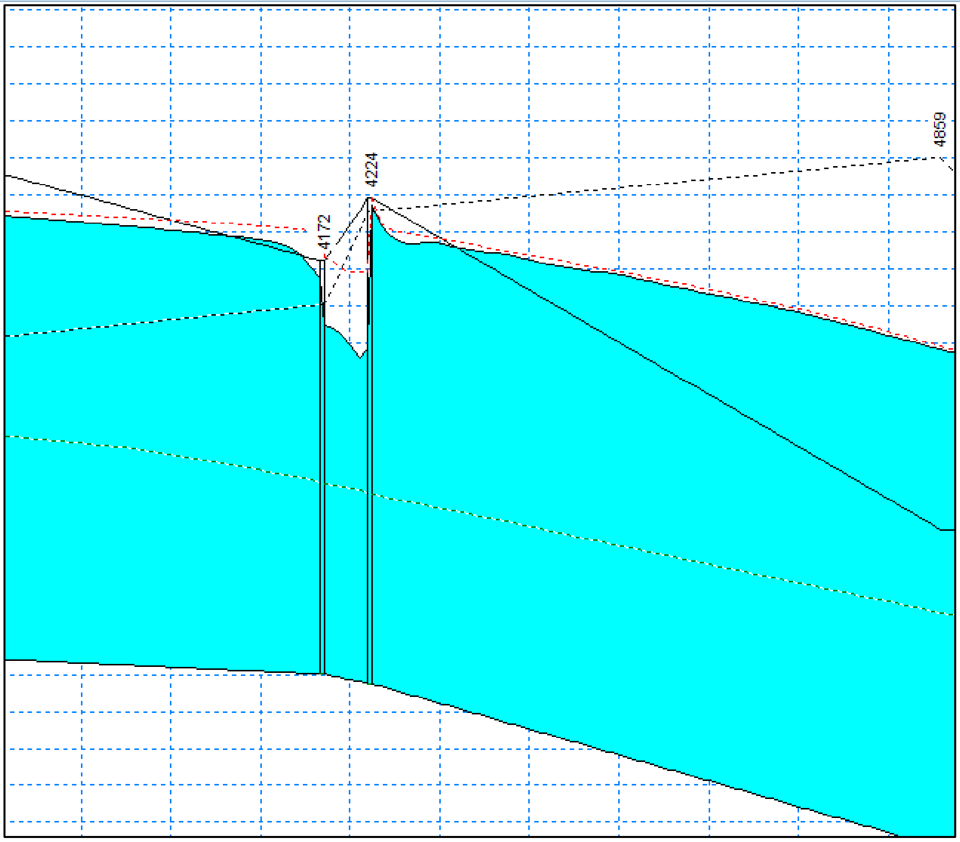
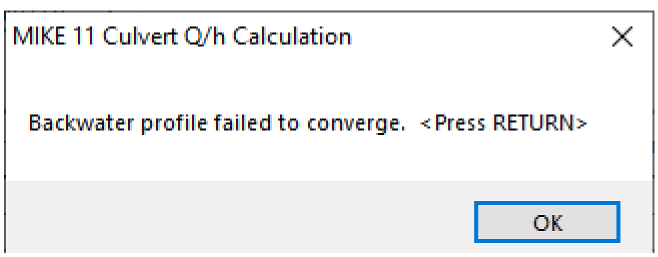


Figure 5-3 – Water level profile at KopuBridges 4221 and 4167

An attempt was also made to calculate the QH relationships in this location and the following error was encountered.



This indicates that the bridge setup in this location needs to be refined in order to function correctly. (Discussions with Susan Borrer have indicated that this cannot be replicated in house and maybe due to a version issue with the software. My checks have been carried out using the 2017 SP2 version of the software – the results still show a minor instability however).

One further instability is observed at KopuBridge 15583. Again inspection of the bridge setup shows;

- the upstream cross section is spaced only 1m upstream. The MIKE11 manual recommends that the upstream cross section is placed at least as far upstream as the opening width (see Figure 5-4 below)
- The bridge setup also has no length or slope applied. Applying a small slope may assist with stability.

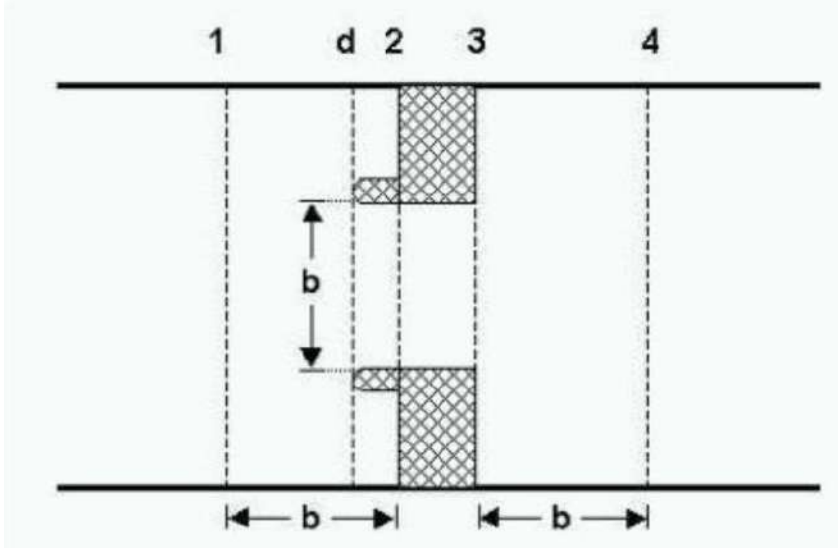


Figure 5-4 - Location of cross sections for FHWA WSPRO bridge method (M11 reference manual)

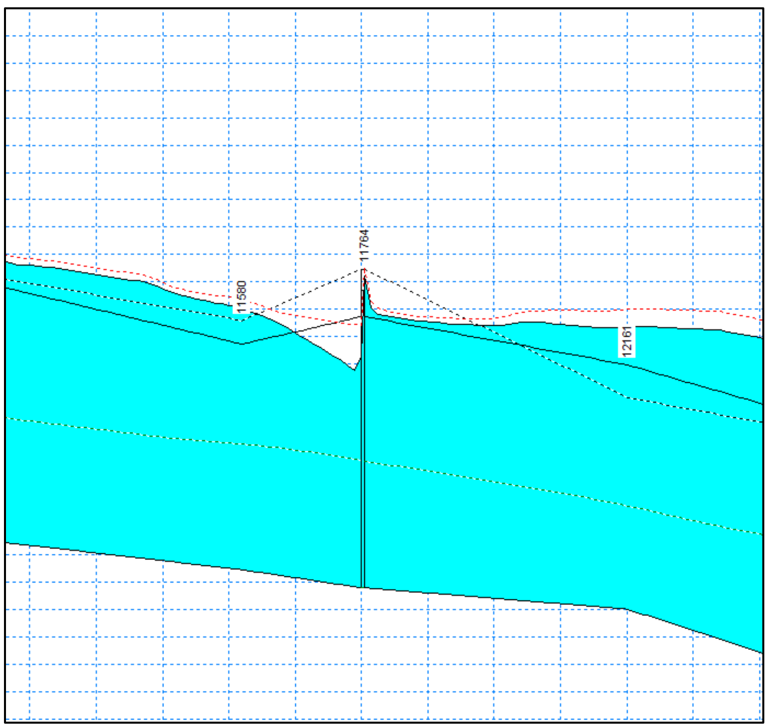


Figure 5-5 – Water level profile at KopuBridge 15583

Results of the 100yearCC run do not show any further instabilities for the Kopuaranga River. Whilst the general principles outlined above for these two bridges should ideally apply to all structures, considering the results appear to be sensible it would not warrant changing all structures in the model.

MINOR RECOMMENDATION 2

The culvert setups at KopuBridges 4221 and 4167 and the bridge setup at KopuBridge 15583 are reconsidered in order to remove the instabilities present in the 100 year CC results.

5..2 REPRESENTATION OF LATERAL LINKS (KOPUARANGA RIVER)

LATERAL LINK CONNECTIONS

A basic check of the setup of the lateral link connections has been carried out for the Kopuaranga River. The lateral link connections file has been converted to a shapefile and visualised within ArcGIS. For the majority of the length of the Kopuaranga River, the lateral links appear to be working as anticipated however in some locations, particularly in the upper reaches, the lateral links on the left and right banks are not connecting to the same location perpendicular to the channel centreline. In the worst locations, the lateral links are actually connecting approximately 75m downstream from the centrepoint. An inspection of the MIKE11 result files in this location indicates that this may result in water levels approximately 0.2m higher on the floodplain in this location than could realistically be expected. A visualisation of the lateral link connection for a section of the US_Kopuaranga branch is presented in Figure 5-6 below. The yellow lines connect the floodplain to the channel centreline, and ideally should be perpendicular to the main direction of flow.

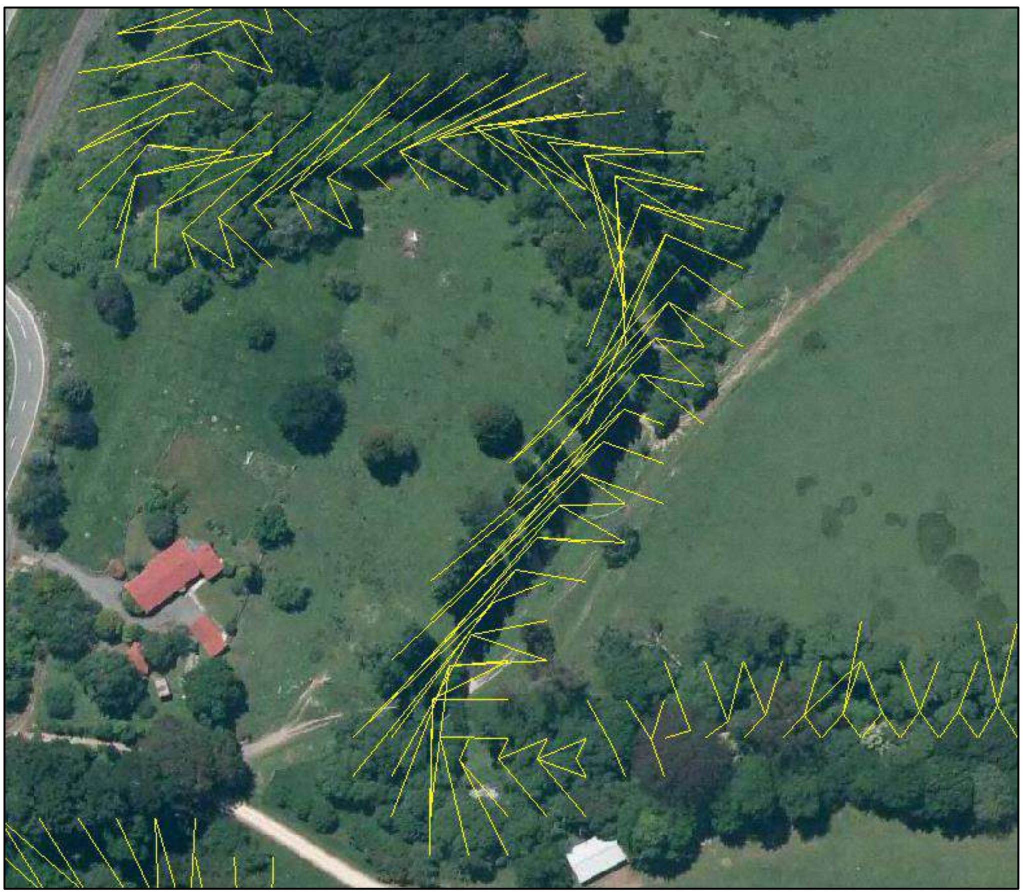


Figure 5-6 – Visualisation of Lateral Link Connections – US_Kopuaranga MIKE11 Chainage - 4422 - 4945

This is a common issue for curved sections of river and could be remedied by splitting the lateral links into shorter segments so that the link chainage can be defined more accurately. This is only expected to have a localised effect of water levels.

MINOR RECOMMENDATION 3

If the results are to be used for setting floor levels, then the lateral link connections should be broken into smaller segments, in particular around tight bends.

LATERAL LINK ELEVATION SOURCE

It is also noted that the current model setup generally uses the HGH method for defining the land elevation at the lateral links. This forces the model to choose the highest level of either the MIKE11 cross section or the MIKE21 terrain level. Due to the relatively wide spacing between surveyed cross sections this can results in higher or lower ground levels being estimated at the link locations between sections than the physical reality. Due to the high resolution of ground level available from the LiDAR data, the M21 method is likely to more accurately represent the physical reality and should be adopted for this model.

The peer review document (Macky, 2014) also made the recommendation that unless the MIKE11 model us known to accurately represent the berm levels, then the M21 model should be selected as the source for the link elevations rather than the HGH method.

MODERATE RECOMMENDATION 3

That the lateral link elevations for the Kopuaranga River are changed to use the M21 method for their source as recommended in the DHI peer review.

5..3 MODEL CALIBRATION / VALIDATION (KOPUARANGA RIVER)

The Kopuaranga model has not been calibrated to any events due to a lack of debris recordings. It is also noted that the gauge at Palmers is bypassed by high flows, meaning that the gauge record does not give an accurate reflection of flood sizes. There are however a large number of flood photos on file from historic events as well as the flood protection shape file of estimated 50-year flood extent (which is reported to be based on historic flood extents).

A comparison of a select area of the Kopuaranga model results is made with the GWRC 50 year flood zone layer in Figure 5-7 below.

It is considered that some degree of documented model validation/verification would be prudent.

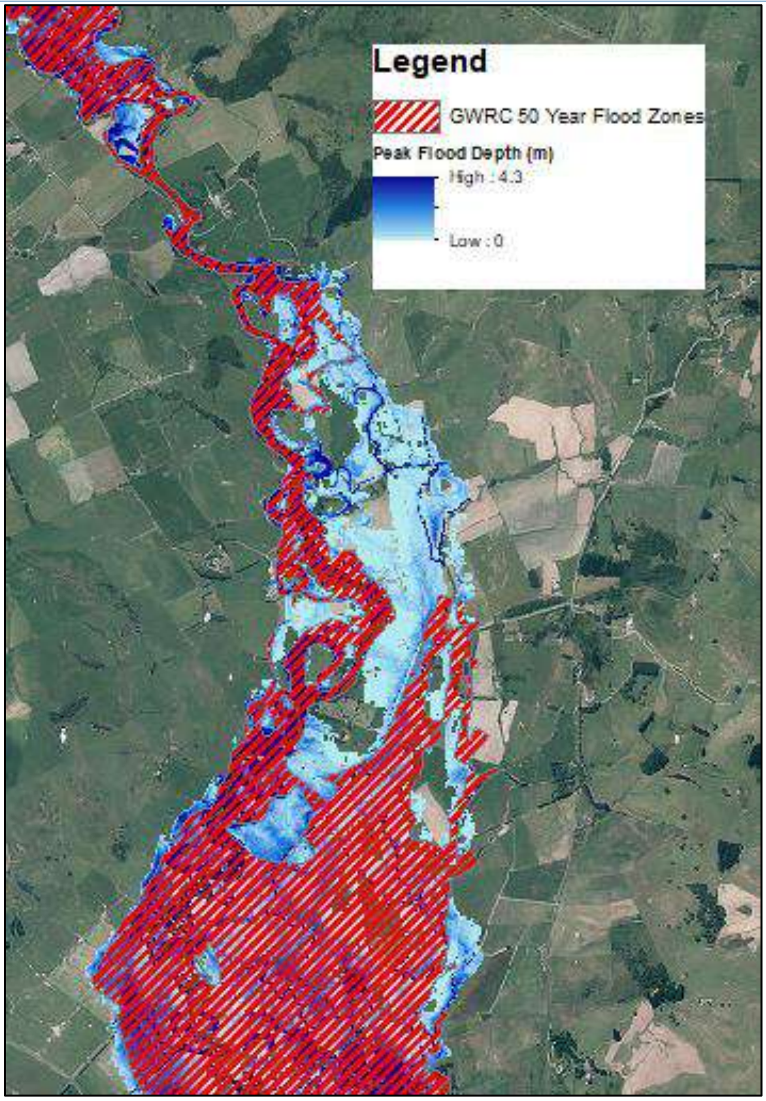


Figure 5-7 – Comparison of GWRC 50 year flood extent layer with latest 50 year model results (Kopuaranga River)

MODERATE RECOMMENDATION 4
Before the results are finalised, it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve; -Comparison with historic flood photos -Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted) - Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods

5..1 REPRESENTATION OF STRUCTURES (WHANGAEHU RIVER)

Five bridges have been modelled on the Whangaehu River using the bridge module.

Each bridge/culvert has been plotted in relation to the upstream and downstream cross sections and the overall setup appears to be reasonable.

The MIKE11 results have also been checked at each structure to check for any obvious instabilities. No obvious instabilities were present in the results files indicating that the bridge setup is sound.

5..2 REPRESENTATION OF LATERAL LINKS (WHANGAEHU RIVER)

LATERAL LINK CONNECTIONS

A basic check of the setup of the lateral link connections has been carried out for the Whangaehu River. The lateral link connections file has been converted to a shapefile and visualised within ArcGIS. For the majority of the length of the Whangaehu River, the lateral links appear to be working as anticipated however in some locations, particularly at sharp bends, the lateral links on the left and right banks are not connecting to the same location perpendicular to the channel centreline. This is the same issue as was outlined for the Kopuaranga River in section 5..2. This could be remedied by breaking the lateral links into shorter segments.

MINOR RECOMMENDATION 4

If the results are to be used for setting floor levels, then the lateral link connections should be broken into smaller segments, in particular around tight bends.

At MIKE11 chainage 9452, the links do not appear to be working correctly with the water level upstream of the bridge linking to a whole section of river downstream from here. A visualisation of the lateral link connections in this location is presented in below. The yellow lines connect the floodplain to the channel centreline, and ideally should be perpendicular to the main direction of flow.

MINOR RECOMMENDATION 5

The lateral links are remodelled for the Whangaehu River at MIKE11 Chainage 9452

Figure 5-8 - Visualisation of Lateral Link Connections – Whangaehu MIKE11 Chainage 9452

LATERAL LINK ELEVATION SOURCE

It is also noted that the current model setup generally uses the HGH method for defining the land elevation at the lateral links. This forces the model to choose the highest level of either the MIKE11 cross section or the MIKE21 terrain level. Due to the relatively wide spacing between surveyed cross sections this can results in higher or lower ground levels being estimated at the link locations between sections than the physical reality. Due to the high resolution of ground level available from the LiDAR data, the M21 method is likely to more accurately represent the physical reality and should be adopted for this model.

MODERATE RECOMMENDATION 5

That the lateral link elevations for the Whangaehu River are changed to use the M21 method for their source as recommended in the DHI peer review.

5..3 MODEL CALIBRATION / VALIDATION (WHANGAEHU RIVER)

The model report states that no historic debris information is available for the Whangaehu River. There are however a large number of flood photos on file from historic events as well as the flood protection shape file of estimated 50-year flood extent (which is reported to be based on historic flood extents).

A comparison of a select area of the Kopuaranga model results is made with the GWRC 50 year flood zone layer in Figure 5-9 below. In general the modelled flood extent for the Whangaehu River seems to match fairly well with the historic extents.

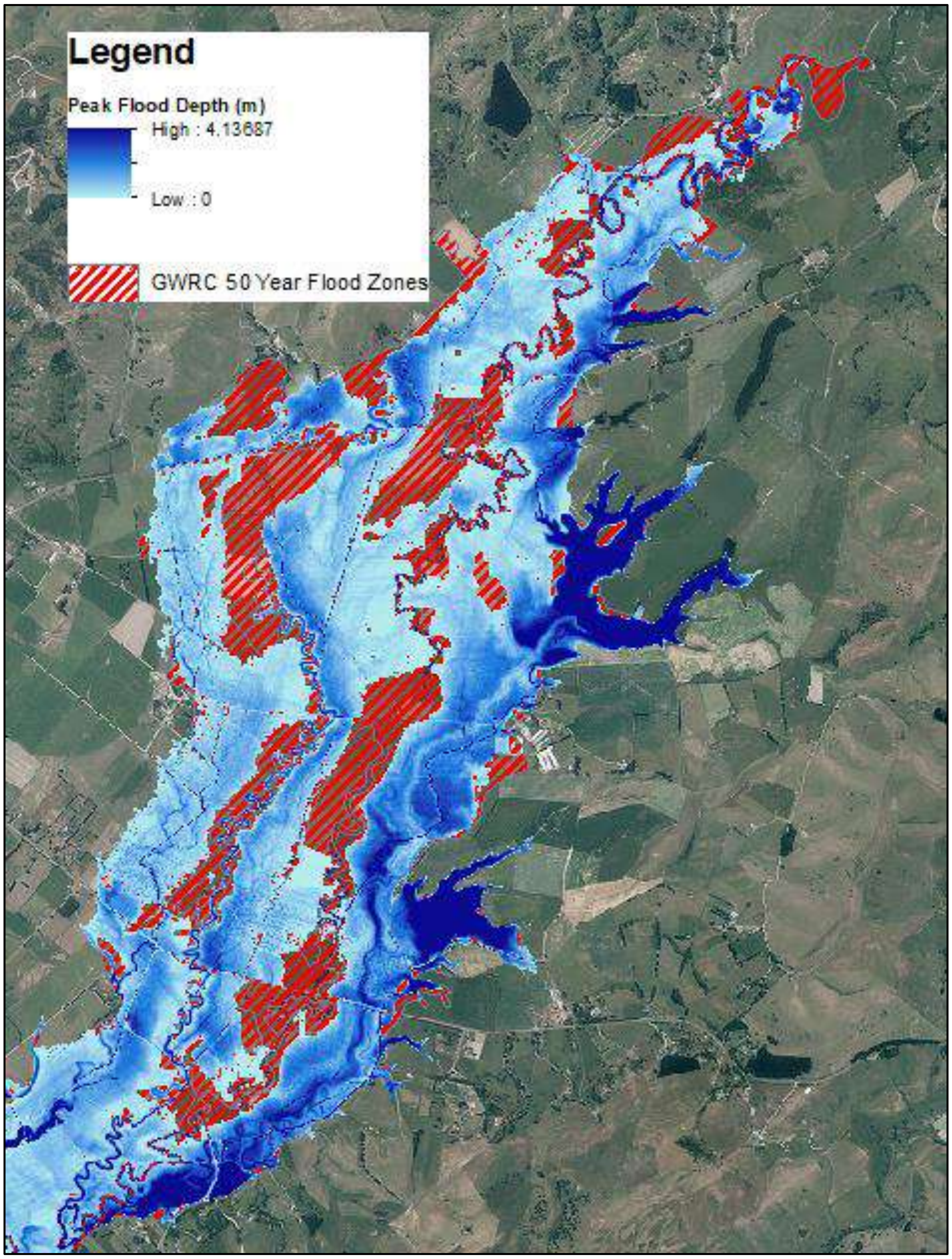


Figure 5-9 - Comparison of GWRC 50 year flood extent layer with latest 50 year model results (Whangaehu River)

It is considered that some degree of documented model validation/verification would be prudent.

MODERATE RECOMMENDATION 6

Before the results are finalised, it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;

-Comparison with historic flood photos

-Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)

- Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods

RESPONSE TO PEER REVIEW

The following recommendations were made as part of the peer review carried out by DHI (Macky, 2014). (NB The peer reviewer did not specifically use the work recommendation in his report.)

PEER REVIEW RECOMMENDATION 1

The grid size is generally small enough to capture the obstruction to flow caused by roads and vehicle tracks. However, a few locations can be expected where a road level has not been fully captured, allowing flow across the road at lower water levels than realistic. This will more often apply to narrow tracks than public roads. These locations can be quickly investigated and remedied by capturing a polyline along the road from the LiDAR data.

Response

It is not apparent that a check of the road elevations has yet been carried out, it is recommended that this is done in a range of representative areas where the 100-year CC hazard crosses roads. To do this it would be sensible to compare the 1m grid based on the base LiDAR with the final 10m grid used in the model. This is unlikely to have a significant impact on any major urban area.

RECOMMENDATION

It is recommended that a comparison between the 1m grid based on the base LiDAR with the final 10m grid used in the model is carried out to ensure road crest levels are adequately captured in the model, as per the DHI peer review recommendations. A representative sample should be checked where flood waters cross roads in the final results.

PEER REVIEW RECOMMENDATION 2

There is good justification for increased flow resistance for floodplain vegetation, and especially for heavily developed urban land, where the value of n=0.2 is a reasonable choice. This has been done only in parts of the model; if there are other urban or vegetated areas that get flooded, these should be treated similarly for consistency.

Response

A general check of the adopted floodplain Manning’s ‘n’ has highlighted an issue with a potential geographic transformation error which needs to be investigated further. In summary, the applied resistance values do not appear to align with the underlying terrain model or imagery and are shifted north by approximately 70m.

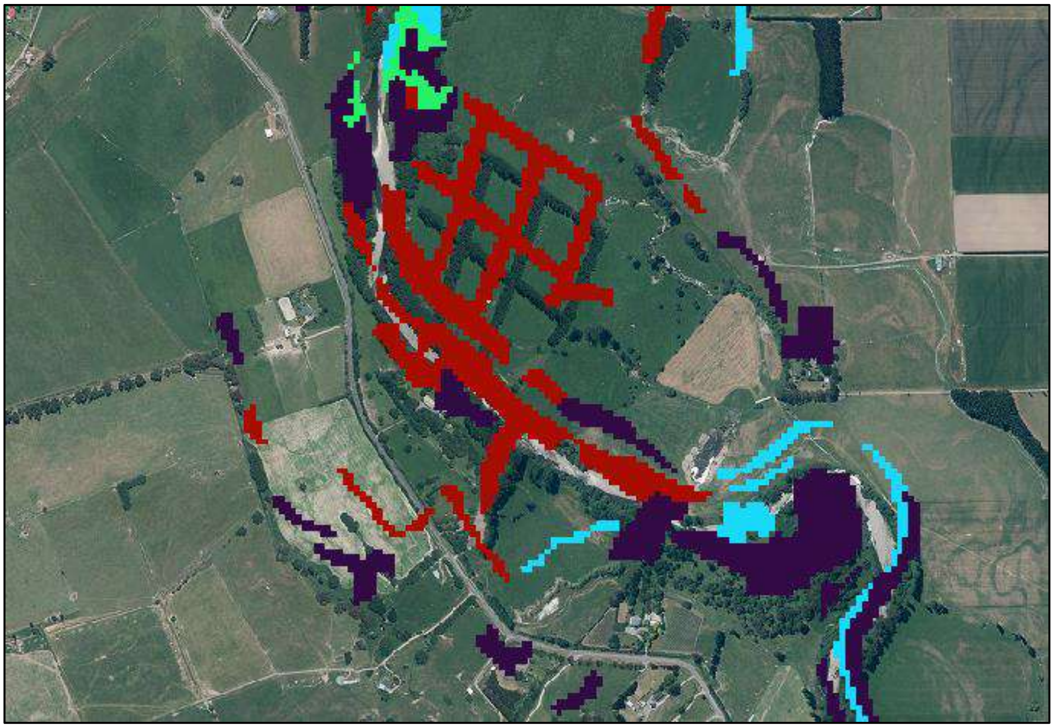


Figure 5-10 – Visualisation of shift due to possible incorrect geographic transformation

MODERATE RECOMMENDATION 7

That the apparent horizontal shift between the applied roughness file and the terrain model is investigated and the cause for the shift is rectified. The model will likely need to be rerun as a result.

PEER REVIEW RECOMMENDATION 3

The internal location of the boundaries is not official recommended practice, but appears to work without incident, and advice from DHI's support service in Denmark is that it should not adversely affect the model's function or run time. The same hydraulics apply here as in the Waingawa model: this boundary condition appears to
be a pragmatic solution for floodplain flow out of the model, but it is likely to be over-estimating this flow for the depth of water).
However, this model should be regarded as applying only at and upstream of the MIKE 11 downstream boundary. It is likely that this is far enough upstream for any effects of the boundary description to be minor.

Response

Discussions with Susan have confirmed that model results have been clipped as recommended in the peer review recommendations.

PEER REVIEW RECOMMENDATION 4

The weir settings should lead to an accurate representation of flow into and out of the channels, except for some uncertainty about levels. In particular, carrying out linking calculations at about 10 m intervals along the channel is at least as detailed as needed. The "HGH" setting is accurate except where there are places between surveyed cross-sections with berms lower than the interpolated line between berm levels at the cross-sections. This review has not included checks of this feature in the Waipoua model as it has in the Waingawa model, but with cross-sections spaced at 300 or more it is likely that the interpolated line is higher than the MIKE 21 levels in some locations.

If it can be guaranteed that the MIKE 21 levels represent the berm, then the setting for the weir crest level should be changed from HGH to MIKE21. If not, an alternative course of action is to extract the berm levels from the LiDAR data and compare them with the interpolated levels assumed in MIKE 11, with additional cross-sections added as needed to avoid assigning too high a weir crest level.

With this caveat, the linking should be satisfactory as long as numerical instabilities in water levels are avoided.

Response

The caveat in the final paragraph does not appear to hold for this model.

As has already been highlighted in section 5, it is recommended that the M21 method is adopted for all lateral links in this model, and any stability issues are rectified before the floodmaps are finalised.

PEER REVIEW RECOMMENDATION 5

(Model Instability – Ruamāhanga 23302) In both cases, the instability occurs at the flood peak. Whilst a solution is not immediately obvious, it could be worthwhile modifying the inflow hydrographs to avoid the abrupt change from a rising hydrograph to a falling one. Inflow hydrographs with more of a transition over the peak are in any case likely to be more realistic.

Response

Inspection of the final results confirms that this instability has been rectified.

PEER REVIEW RECOMMENDATION 6

The results from most parts of the model suggest that the overland flow model is basically sound. However, checks are warranted of all roadways in the floodplain, to determine whether road levels have been captured completely in the MIKE 21 terrain.

Response

This is essentially the same as recommendation 1 and has not yet been carried out.

6 RECTANGULAR GRID LOWER HYDRAULIC MODELLING

This model covers the following rivers:

- Waingawa River
- Taueru River
- Upper Ruamāhanga from the Te Whiti Road Bridge to the Waiohine Confluence

This modelling report that has been provided is dated October 2013 (Borrer, 2013). In addition to the modelling report was a peer review report carried out by DHI (Macky, 2014).

The audit of this section of the project has included a detailed reading of the modelling report and peer review documents, as well as an interrogation of the actual model files themselves and a number of result files.

The model setup files for the 100-year scenario including the effects of climate change were provided to the auditor

1D and 2D result files were also supplied for the following runs:

Taueru_Q50 and Taueru_Q100CC

Waingawa_Q50 and Waingawa_Q100CC

WAINGAWA MODEL SETUP

6..1 REPRESENTATION OF STRUCTURES (WAINGAWA RIVER)

No bridges or other structures have been modelled on the Waingawa River. The report highlights the fact that there are two bridges crossing the river which are the State Highway 2 bridge and the rail bridge.

The report explains that an assessment on the available freeboard during the 1% AEP deign event has been carried out and the bridge has been found to have significant freeboard to the soffit and therefore the exclusion of the bridges has been justified in the report.

An inspection of the final model results shows that there is water spilling out of the river upstream of both of these bridges indicating that flooding on the land immediately upstream of each bridge is sensitive to any headloss from the structure.

Analysis of the photo provided of the SH2 bridge shows debris build-up on each of the piers, indicating that there is potential for increased debris to come down the river in a large event. The inclusion of the bridge structures in the model will allow for a sensitivity analysis of debris blockage to be carried out and will also slightly elevate water levels in the vicinity of the bridges. Whilst this is unlikely to have a significant impact on the overall flood extent, it is likely to increase the flood levels for the properties immediately upstream of the bridges.

Adding these bridges into the model will be a fairly simple exercise providing the bridge dimensions are known.

MINOR RECOMMENDATION 6

That consideration is given to including the State Highway 2 and Railway bridges into the model so that the localised impact of the bridge piers is included in the model results and so that sensitivity to debris blockage can be simulated.

6..2 REPRESENTATION OF LATERAL LINKS (WAINGAWA RIVER)

LATERAL LINK CONNECTIONS

A basic check of the setup of the lateral link connections has been carried out for the Waingawa River. The lateral link connections file has been converted to a shapefile and visualised within ArcGIS. The lateral links appear to be working as anticipated for the entire length of the river. A visualisation of the lateral link connection for a section of the Waingawa River branch is presented in Figure 5-1 below (The green lines connect the floodplain to the channel centreline, and ideally should be perpendicular to the main direction of flow as is the case in this example).

Figure 6-1 – Visualisation of Lateral Link Connections – Waingawa River (SH2 and Rail bridges)

LATERAL LINK ELEVATION SOURCE

The lateral link elevation source has been checked and it can be confirmed that the source of elevation has been changed to M21 from HGH as was recommended in the peer review report (Macky, 2014).

6..3 MODEL CALIBRATION / VALIDATION (WAINGAWA RIVER)

The Waingawa model has been reported to have been calibrated to the flood event of 2006 which has an estimated return period of 2 years.

Overall the model calibration fits very well to the recorded flood levels and the overall calibration to this individual event should be considered very good.

This event is a very low return period however and debris recordings are not available for larger magnitude events. Further confidence in the results would be gained if a larger flood is able to be used for model calibration.

In the absence of debris levels, a comparison of model results with historic photos as well as the GWRC 50-year flood zone would be prudent.

Comparison for a select area of the Waingawa model results is made with the GWRC 50 year flood zone layer in Figure 6-2 below. In general the modelled flood extent for the Waingawa River seems to match fairly well with the historic extents however in some areas such as in the vicinity of the bridges, the new flood extent is significantly less than the historic 50-year flood layer. This may indeed be sensible, however some commentary around the differences would be warranted.

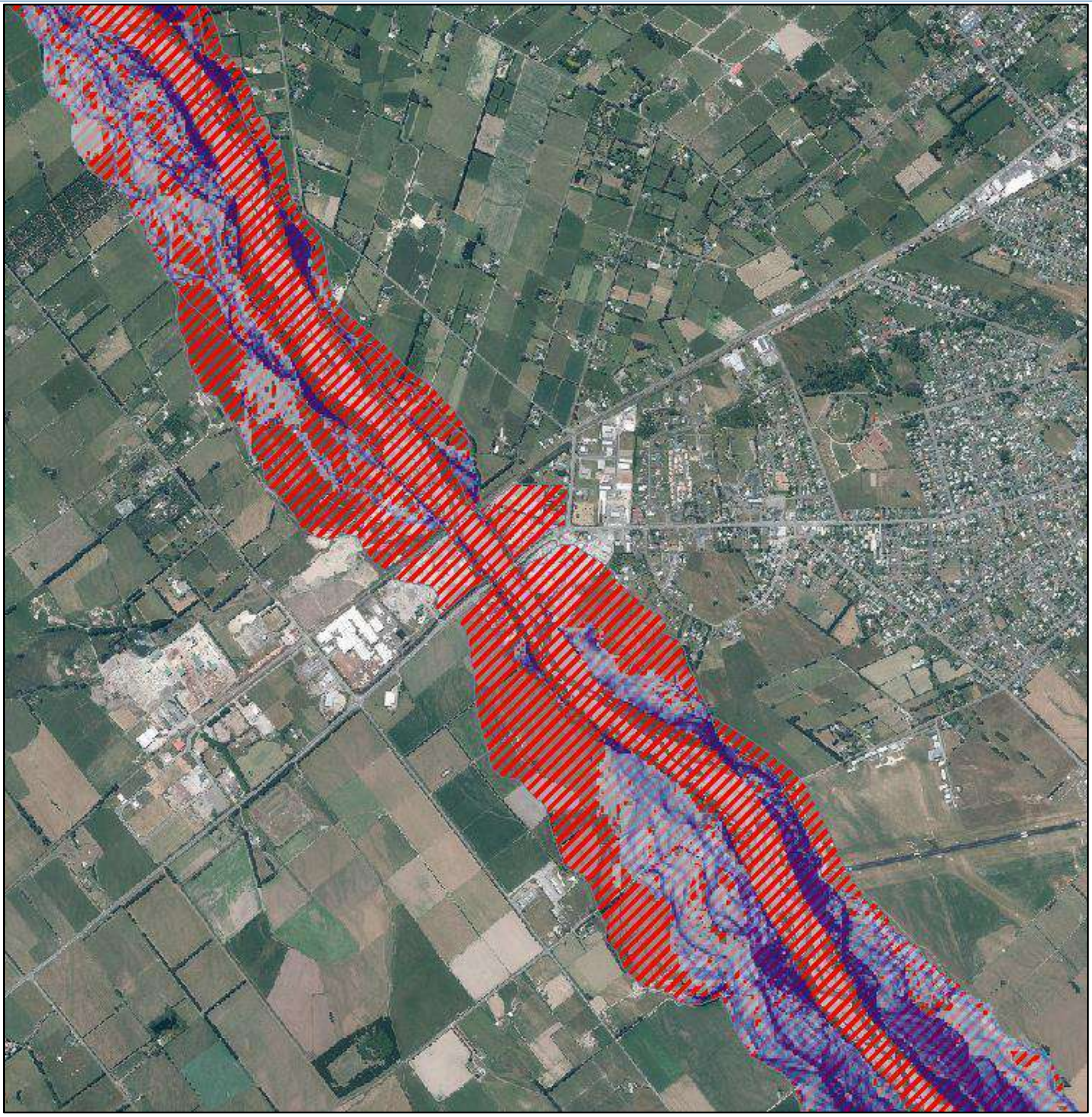


Figure 6-2 - Comparison of GWRC 50 year flood extent layer with latest 50 year model results (Waingawa River)

MODERATE RECOMMENDATION 8

Whilst the results are calibrated to a 2-year event, before the results are finalised, it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;

-Comparison with historic flood photos

-Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)

- Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods

6..4 GENERAL MODEL STABILITY (WAINGAWA RIVER)

A general inspection of the model results show that the flood spread appears to be realistic, and that the floodwaters propagate in a natural fashion.

One minor instability has been noted in the MIKE11 results at chainage 6829, this does not appear to propagate into the 2D results, however may want to be tidied up if further work is done in the model in the future.

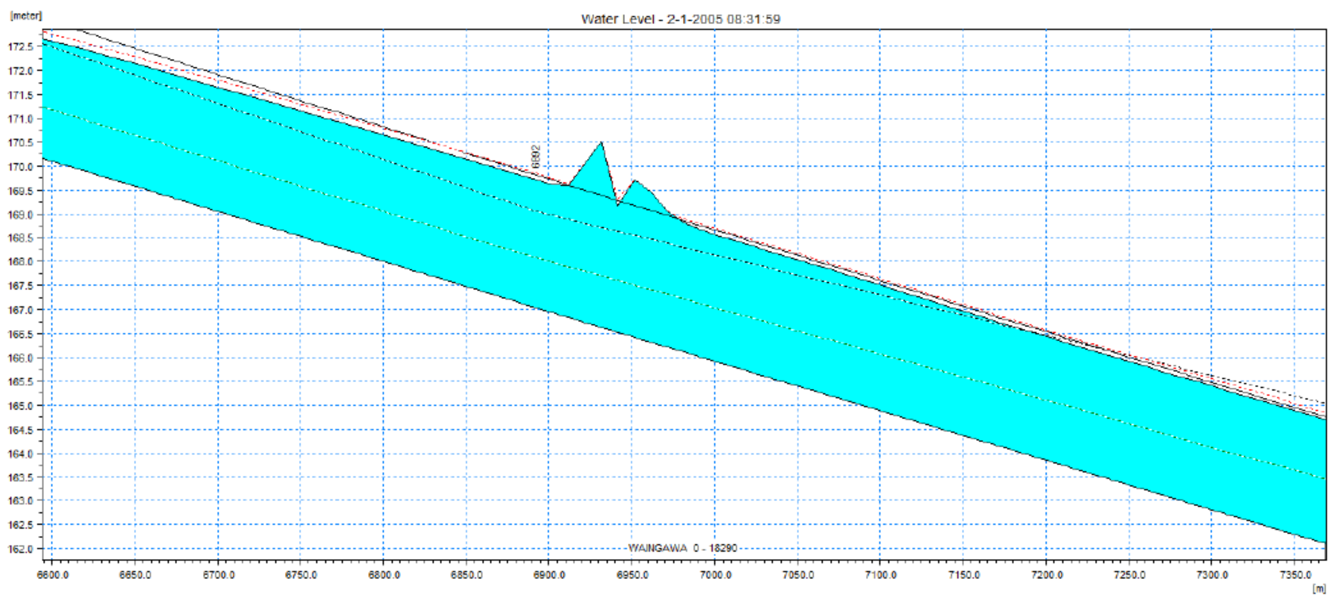


Figure 6-3 – Visualisation of MIKE11 model results in location of instability

TAUERU MODEL SETUP

6..1 REPRESENTATION OF STRUCTURES (TAUERU RIVER)

The report mentions two structures which cross the Taueru River in this reach. These are Patricks Bridge and Te Whiti Road bridge. Only the Patricks Bridge has been included in the model.

A quick check of the bridge setup has been made and the setup appears to be sensible. There are no obvious instabilities present in the MIKE11 results at this location either.

The impact of blockage does not appear to have been tested on this bridge, and it seems like an obvious location to block considering the large amount of willow etc in the catchment. Consideration may want to be given to carrying out a sensitivity test to blockage of this structure.

The exclusion of the Te Whiti Road bridge appears to be justified considering the flows spill over a much wider area on the right berm and over the road, however if sensitivity testing was to be carried out with blockage in lower flows, then the inclusion of the piers may be useful.

MINOR RECOMMENDATION 7

Consideration should be given to carrying out a sensitivity test to blockage of Patricks Bridge as well as consideration to including the piers of the Te Whiti Road bridge so that sensitivity to debris build up on the piers can also be investigated, especially for lower flow events.

6..2 REPRESENTATION OF LATERAL LINKS (TAUERU RIVER)

LATERAL LINK CONNECTIONS

A basic check of the setup of the lateral link connections has been carried out for the Taueru River. The lateral link connections file has been converted to a shapefile and visualised within ArcGIS. For the majority of the length of the Taueru River, the lateral links appear to be working as anticipated however in some locations, particularly at sharp bends, the lateral links on the left and right banks are not connecting to the same location perpendicular to the channel centreline. In some locations the model is connecting the water level to the berm almost 200m downstream from the channel centreline. This is the same issue as was outlined for the Kopuaranga River in section 5..2. This could be remedied by breaking the lateral links into shorter segments at tight bends.

A visualisation of the lateral link connections at one particularly tight bend is presented in Figure 6-4 below.

Figure 6-4 - Visualisation of Lateral Link Connections – Taueru River (M11 Chainage 1957 to 3264)

MINOR RECOMMENDATION 8 If the results are to be used for setting floor levels, then the lateral link connections should be broken into smaller segments, in particular around tight bends.

LATERAL LINK ELEVATION SOURCE

The lateral link elevation source has been checked and it can be confirmed that the source of elevation has been changed to M21 from HGH as was recommended in the peer review report (Macky, 2014).

6..3 MODEL CALIBRATION / VALIDATION (TAUERU RIVER)

No calibration has been carried out for the Taueru Model likely due to the lack of any available debris recordings. In the absence of debris levels, a comparison of model results with historic photos as well as the GWRC 50-year flood zone would be prudent.

A general comparison with the historic 50-year flood layer shows that model results appear to be fairly consistent with the historic understanding of flood risk, however there is significantly more flooding visible on the true left bank in the lower reaches. These differences would warrant some further investigation to ensure the results are sensible.

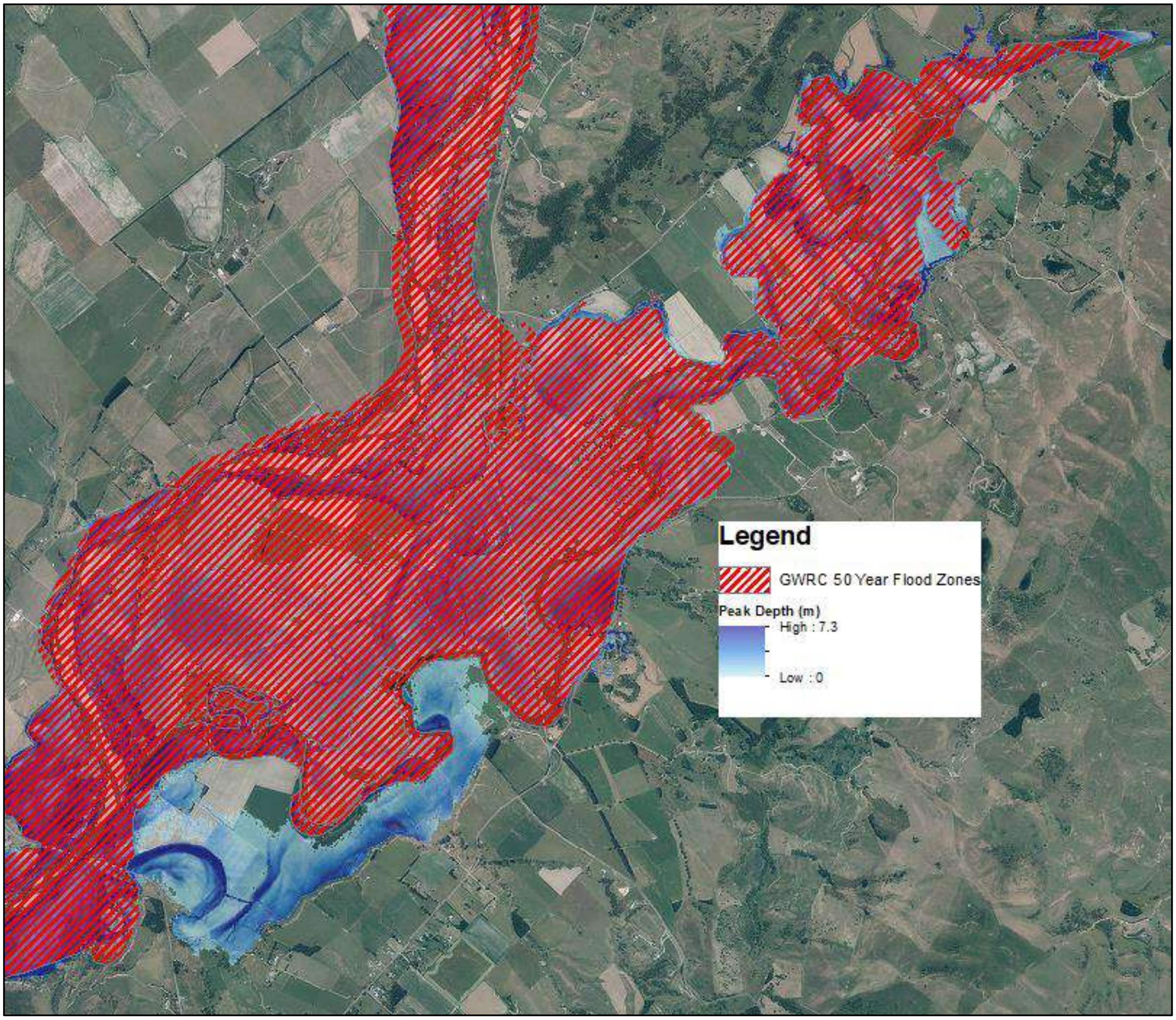


Figure 6-5 - Comparison of GWRC 50 year flood extent layer with latest 50 year model results (Taueru River)

A quick comparison of flood photos shows a fairly good fit with historic flood extents. An example comparison with a historic flood photo from 1985 (between a 10 year and 20 year event) at Te Whiti road bridge is presented in Figure 6-6 and Figure 6-7 below. This photo appears to show significantly more flooding in this location than observed in the 50-year model results, however this may be due to changes in channel capacity since 1985.

 Figure 6-6 – Flood photo Te Whiti road bridge (July 1985 – 10 to 20 year return period event)	 Figure 6-7 – 50 year model results Te Whiti road bridge

It would be prudent to carry out a more detailed comparison which is documented as well as discuss the results with people familiar with the flood behaviour of the river as part of a validation exercise.

MODERATE RECOMMENDATION 9

Before the results are finalised, it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;

-Comparison with historic flood photos

-Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)

- Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods

RUAMĀHANGA MODEL SETUP

6..1 REPRESENTATION OF STRUCTURES (RUAMĀHANGA RIVER)

The report states that there are three bridges in the modelled reach (Wardells, Gladstone and Kokatau). Wardells is at the upstream extent and has therefore been excluded. Both the Gladstone bridge and the Kokatau have not been included as they are not considered to give a significant construction to flow.

MINOR RECOMMENDATION 9

That consideration is given to including the Gladstone and Kokatau bridges into the model so that the localised impact of the bridge piers is included in the model results and so that sensitivity to debris

6..2 REPRESENTATION OF LATERAL LINKS (RUAMĀHANGA RIVER)

LATERAL LINK CONNECTIONS

A basic check of the setup of the lateral link connections has been carried out for the Ruamāhanga River. The lateral link connections file has been converted to a shapefile and visualised within ArcGIS. The lateral links appear to be working as anticipated for the entire length of the river. At the confluence of the Taueru River there appears to be a gap in links on the true left bank over a length of approximately 500m. When simulating floods which coincide with flooding on the Taueru River this will be unlikely to have any significant impact, however may impact on results which are solely focusing on flooding from the Ruamāhanga river A visualisation of the lateral link connection for the section of the Ruamāhanga River / Taueru confluence is presented in Figure 6-8 below (The green lines connect the floodplain to the channel centreline, and ideally should be perpendicular to the main direction of flow as is the case in this example).

Figure 6-8 - Visualisation of Lateral Link Connections – Ruamāhanga River

MINOR RECOMMENDATION 10

Lateral links should be included both upstream and downstream of the confluence with the Taueru River

LATERAL LINK ELEVATION SOURCE

The peer review document (Macky, 2014) also made the recommendation that unless the MIKE11 model us known to accurately represent the berm levels, then the M21 model should be selected as the source for the link elevations rather than the HGH method. The model report (Borrer, 2013) states that the model went unstable when this change was made therefore it has been left as HGH. Considering the M21 method will more accurately represent the actual physical reality of the elevations at the link location, it is considered to be important that the M21 approach is used. It is likely that the model setup parameters at the links can be adjusted slightly in order to remove any instability present at the links

MODERATE RECOMMENDATION 10

That the lateral link elevations for the Ruamāhanga River are changed to use the M21 method for their source as recommended in the DHI peer review.

6..3 MODEL CALIBRATION / VALIDATION (RUAMĀHANGA RIVER)

The report states that the previous modelling was calibrated to the 2004 flood event. A quick review of the The Te Whiti Stopbank model has shown that this was also calibrated to an event in 2009 (5-year return period).

Consideration should be given to checking the model calibration against the 1994 event using the same cross section database used in the previous modelling.

It would be also useful to make a comparison of the adopted Manning's 'n' values with the historic modelling.

In addition to the debris levels on file, there are a number of flood photos on file from historic events as well as the flood protection shape file of estimated 50-year flood extent (which is reported to be based on historic flood extents).

A comparison of a select area of the Ruamāhanga model results is made with the GWRC 50 year flood zone layer in Figure 6-9 below. In this location the modelled flood extent is significantly narrower than that in the 50 year flood extent on record. The reason for the reduction in flood extent may need to be investigated. There is a possibility that changing the lateral link elevation source to M21 from HGH as recommended in section 6..2 will allow more water to spill from the river certain locations.

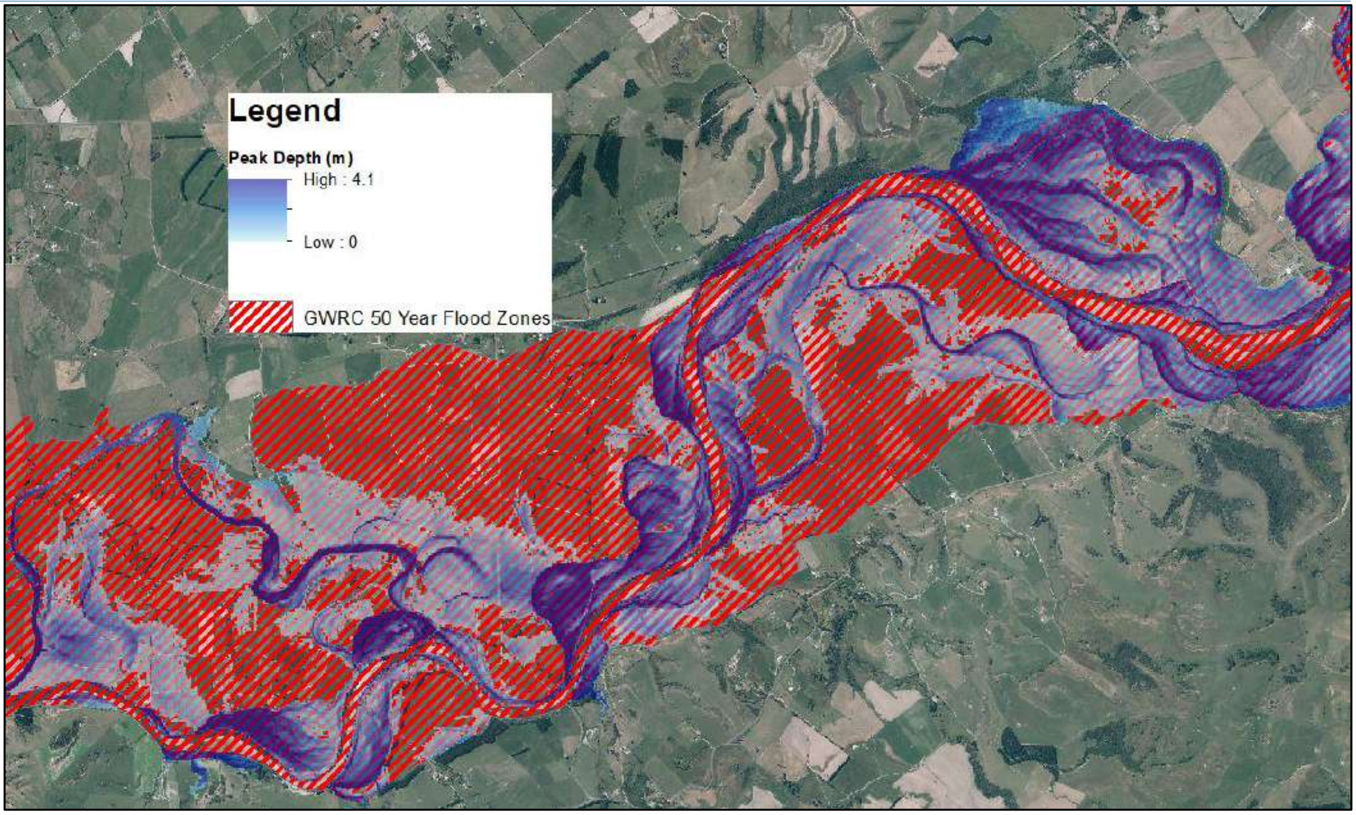


Figure 6-9 - Comparison of GWRC 50 year flood extent layer with latest 50 year model results (Ruamāhanga River)

MODERATE RECOMMENDATION 11

Further justification needs to be given to ignoring the 1994 debris recordings for model calibration as well as the more recent events such as 2009 which were used in the Te Whiti Stopbank modelling.

A comparison with the adopted Manning's 'n' values should be made with the historic models.

Before the results are finalised, it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;

-Comparison with historic flood photos / debris levels

-Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)

-Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods

6..4 GENERAL MODEL STABILITY (RUAMĀHANGA RIVER)

A general inspection of the model results show that the flood spread appears to be realistic, and that the floodwaters propagate in a natural fashion.

No obvious instabilities are present in the 100 year CC results provided.

RESPONSE TO PEER REVIEW

The following recommendations were made as part of the peer review carried out by DHI (Macky, 2014).

PEER REVIEW RECOMMENDATION 1

The MIKE 21 grid is good for purpose despite a cell size of 10m x 10m, because most of it describes fairly flat rural land without detailed topographical features. There appears to be a very few locations where a road level has not been fully captured, allowing flow across the road at lower water levels than realistic. These can be quickly investigated and remedied by capturing a polyline along the road from the LiDAR data. Farm drains are not captured by the MIKE 21 model. This should not matter as, in the context of significant river flows, their flows can be neglected.

Response

As was the case in the upper model, it is not apparent that a check of the road elevations has yet been carried out, it is recommended that this is done in a range of representative areas where the 100-year CC hazard crosses roads. To do this it would be sensible to compare the 1m grid based on the base LiDAR with the final 10m grid used in the model. This is unlikely to have a significant impact on any major urban area.

MINOR RECOMMENDATION 11

It is recommended that a comparison between the 1m grid based on the base LiDAR with the final 10m grid used in the model is carried out to ensure road crest levels are adequately captured in the model, as per the DHI peer review recommendations. A representative sample should be checked where flood waters cross roads in the final results.

PEER REVIEW RECOMMENDATION 2

The resistance values adopted seem reasonable, in the absence of the detailed calibration data that would be needed to determine values rigorously. In particular, 0.045 appears a good estimate for paddocks including the effect of fences and other obstructions. There are some places where the areas identified as vegetated need to be modified (mostly added to). This check would be worthwhile, but the effect on flood levels is likely to be fairly minor.

Response

A general check of the modelled roughness's has shown that overall the floodplain has been well represented. No shift is apparent between the resistance file and the terrain/imagery as was the case in the upper model.

PEER REVIEW RECOMMENDATION 3

The internal location of the boundary, with further terrain downstream (westwards) goes against recommended practice, but appears to work without incident. Advice from DHI's head office is that there should be no problems nor any increase in run times from including the redundant cells west of this boundary. This is a pragmatic solution to the difficulty of defining a floodplain boundary where in fact flow out of the modelled area cannot be reliably calculated. However, it is likely to result in an overestimate of the westward floodplain flow, and hence lower water levels both on the floodplain and in the Ruamāhanga River.

It is recommended that this flow be checked against hand-calculations, or against a model run with an alternative formulation of the boundary. Such an alternative formulation would be needed permanently if the floodplain flow were found to be significantly over-estimated by the present model.

One approach might then be to link the floodplain flow to an artificial MIKE 11 channel that returned the flow to the river near the downstream boundary of the MIKE 11 model. The Q-h rating boundary available in MIKE 21 is another potential solution.

Response

The modelling report states that the downstream boundary has been moved approximately 1.2 km downstream, past the Waiohine confluence, the results were then clipped to just upstream of the Waiohine confluence.

This appears to be a pragmatic solution. The report recommends that the Waiohine floodmodel is used to assess floodrisk in this area. As this model has not yet been finalised, it will be important to ensure that the appropriate flood hydrology is also run through that model to ensure that the results can be adopted.

PEER REVIEW RECOMMENDATION 4

The weir settings should lead to an accurate representation of flow into and out of the channels, except for some uncertainty about levels. In particular, carrying out linking calculations at about 10 m intervals along the channel is very satisfactory, being at least as detailed as needed. The linking should therefore be satisfactory as long as numerical instabilities in water levels are avoided.

If it can be guaranteed that the MIKE 21 levels represent the berm, then the setting for the weir crest level should be changed from HGH to MIKE21. If not, an alternative course of action is to extract the berm levels from the LiDAR data and compare them with the interpolated levels assumed in MIKE 11, and additional cross-sections added as needed to avoid assigning too high a weir crest level.

Response

This has been implemented for both the Waingawa and Taueru Rivers however has not been been implemented on the Ruamāhanga river. As has already been highlighted in section 6..2, it is recommended that the M21 method is adopted for all lateral links in this model, and any stability issues are rectified before the floodmaps are finalised.

PEER REVIEW RECOMMENDATION 5

The solution is to lower XS 18290 to be in accord with the Waingawa channel downstream slope. In addition, a narrow slot can be added to the cross-section to match the Ruamāhanga cross-section invert; this will eliminate a run-time warning message and make model stability more certain.

Response

This has been implemented by the modeller.

PEER REVIEW RECOMMENDATION 6

The results from most parts of the model suggest that the overland flow model is basically sound. However, checks are warranted of all roadways in the floodplain, to determine whether road levels have been captured completely in the MIKE 21 terrain.

Response

This is essentially the same as recommendation 1 and still needs to be carried out as previously recommended.

7 WAIPOUA FLEXIBLE MESH HYDRAULIC MODELLING

This report has been provided in draft format and is dated December 2018 (Borrer, 2018). Included in the appendices of the modelling report was a preliminary peer review report of the hydraulic model carried out by Tonkin & Taylor Ltd (Rix, 2018) as well as a detailed response to the peer review from the modeller.

The audit of this section of the project has included a detailed reading of the modelling report and peer review documents, as well as an interrogation of the actual model files themselves and a number of result files.

The model setup files for the 50-year scenario including the effects of climate change were provided to the auditor (Waipoua_Q50CC_baseline_Rua.couple and associated files)

1D and 2D result files were also supplied for the following runs:

- 1998 Calibration
- 2010 Calibration
- Waipoua_Q50cc_Baseline_Rua
- Waipoua_Q100_Baseline_Rua
- Waipoua_Q100CC_Mikimiki_upper_RuaQ100CC

SOFTWARE AND GENERAL MODELLING METHODOLOGY

The modelling has been carried out using the MIKE Flood software package by DHI. This is an industry standard software which is suitable for this type of modelling. The adoption of the flexible mesh package in this model has allowed for greater resolution of the floodplain to be represented. This allows for greater degree of confidence in the results.

A general audit of the model setup has been carried out and in general the modelling methodology fits with industry best practice.

The following sections comment on specifics relating to each individual main river within the model.

5.1 MODEL REPORT

The model report gives a reasonably thorough overview of the model build and inputs however some sections have missing information (highlighted in yellow). This includes section 9 – Design Scenarios. Appendices have been provided as separate documents.

MINOR RECOMMENDATION 12
It is recommended that the model report is finalised with all relevant information included and the document is finalised into a single merged pdf which contains all of the Appendices.

ROUGHNESS COEFFICIENTS

The roughness values adopted within both the 1D and 2D component of the model are within reasonable ranges and appear to be sensible. A comparison of the 2D roughness file with the aerial imagery has been carried out as part of this audit. The general representation of roughness is considered to be suitable.

If the model was to be updated in the future thought could be given to adding even more detail to the urban area by modelling individual buildings. Land Information New Zealand have recently released a building polygons layer which covers the majority of New Zealand. This file can be used to easily include individual buildings into the model, allowing for increased precision of water levels at individual property locations rather than a blanket roughness coefficient for urban areas.

MINOR RECOMMENDATION 13

Consideration could be given to including individual buildings in the roughness definition file in future upgrades of the model using the recently released buildings polygon layer (LINZ)

REPRESENTATION OF STRUCTURES

There are a total of six structures represented in the model setup. Four of the bridges have used the MIKE11 bridge module to represent the structure and two of the structures (Rail bridge and Paierau) have been modelled using the culvert module. The Paierau culvert represents a floodplain culvert, however the rail bridge is a bridge structure with piers which has been represented as a series of five parallel culverts.

In general, the bridge setups appear to be sensible. Of note however is the fact that no bridge piers have been modelled on any of the bridges other than the rail bridge. It has been assumed that the obstruction to flow caused by the bridge piers not be of consequence.

I believe that inclusion of the bridge piers would be warranted as a minimum for the State Highway 2 bridge (Opaki Rd) and the Colombo Rd bridge for the sake of consistency in analysis, in addition specific reasons for each bridge are;

STATE HIGHWAY 2 BRIDGE (OPAKI RD)

- Model calibration relies on a surveyed debris level immediately upstream of the bridge (XS7). Including the bridge pier will likely push the water levels up slightly and may result in a better calibration.
- The stopbank on the true left bank upstream of the bridge has a low point immediately upstream of the bridge. The model may be sensitive to overflow in this location.
- Including the bridge piers allows the structure to be tested to the sensitivity of blockage from debris. I believe that debris blockage should be investigated for each bridge, even if only a small degree of blockage is considered reasonable.

COLOMBO ROAD BRIDGE

- The Colombo Road bridge appears to have minimal capacity in relation to the other upstream bridges. This is based on observation on site and also inspection of flood photos at the bridge location (Figure 7-1). Blockage of piers may be sufficient to cause water to back up behind the soffit.

- Including the bridge piers will allow for sensitivity to debris blockage to be tested.
- Sensitivity Results (Upper Bound Mikimiki) show that peak water levels are within 0.1 to 0.2m of the soffit at this bridge. If an increase in headloss is allowed for by adding in a bridge pier allowance, then it is possible that this structure will overtop. The impact of this on the overall floodspread should be investigated, as well as the increased hazard (ie risk to evacuation routes etc acknowledged)

Figure 7-1 – Colombo Road Bridge – Nov 1994 Flood

In addition to including the bridge piers, due to the apparent lack of capacity available in the Colombo Road bridge, it is possible that the bridge soffit would be at risk of catching floating debris. Consideration should be given to lowering the soffit of this bridge to account for the potential of floating debris backing up behind the soffit.

MINOR RECOMMENDATION 14

-That the bridge piers for both the Colombo Rd and the State Highway 2 bridge are included in the model setup

-That consideration to lowering the soffit of the Colombo Rd bridge to account for potential effects of floating debris during a large event.

Model results have been interrogated at each bridge structure and no obvious instabilities or anomalies in the results were found.

7..1 RAILWAY BRIDGE

The scope included in the RFQ document contacined a separate item for the setup of the railway bridge to be investigated in detail, the following sections outlines the investigation into the modelling of this structure.

Model results show the railway embankment overtopping in a number of scenarios, which include the upper bound of hydrology (scenario 14- Mikimiki Flow Upper Bound), scenario 19 (Mikimiki Flow Upper Bound Revised) as well as the increased Manning's 'n' scenario and the 20% blockage of the rail bridge scenario.

As a result of the railway embankment overtopping, a significant number of properties within Masterton are shown as being at risk of flooding as shown in Figure 7-2 below.

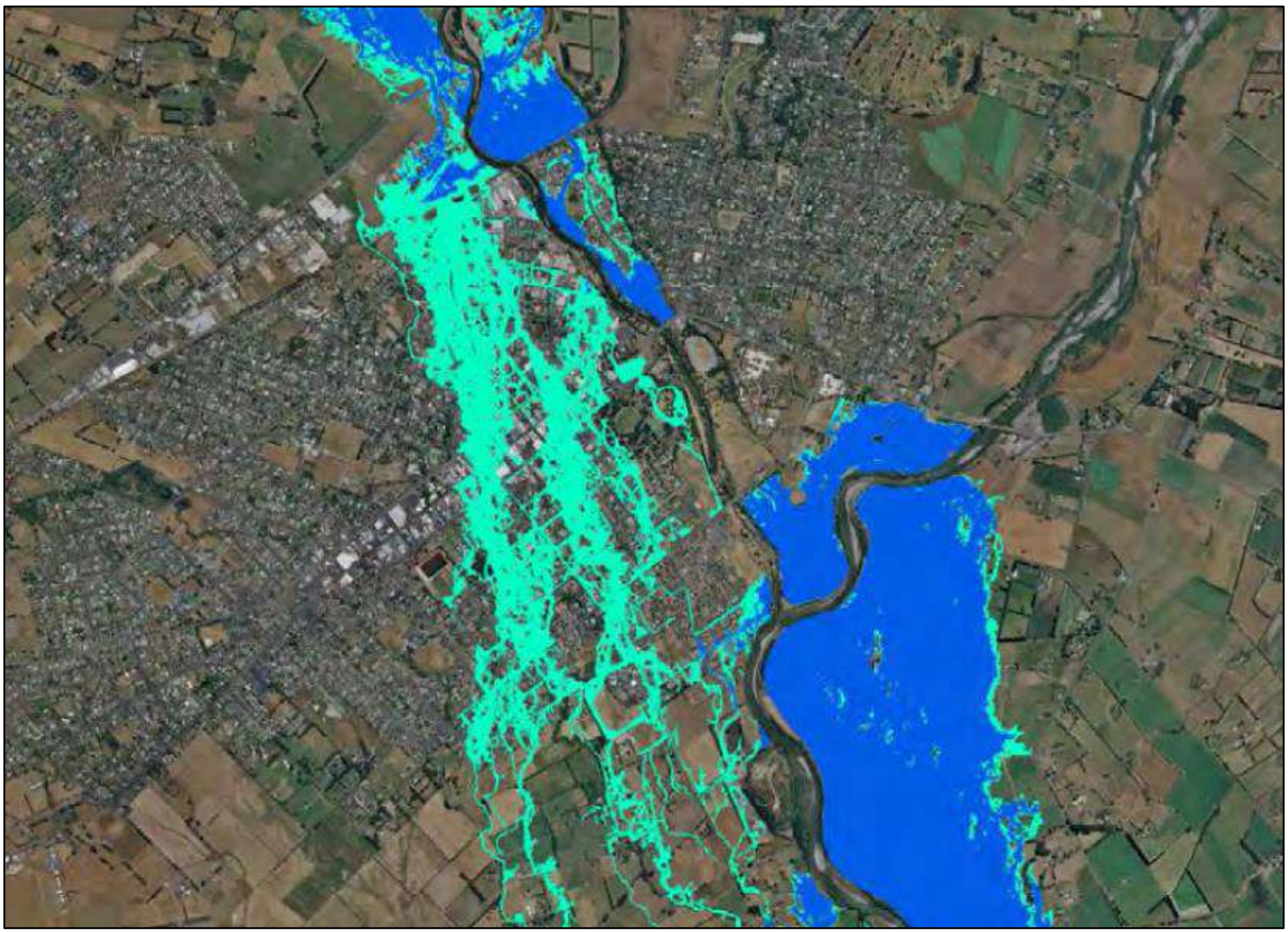


Figure 7-2 – Modelled flood extent showing flood waters overtopping the railway embankment

Analysis of the 100 year results from the Mikimiki Upper Bound model results show that the modelled embankment is only just overtopping (by about 0.1m), potentially indicating that any minor adjustment in the model in the vicinity of the bridge has the potential to make a very significant impact to the overall flood extent and therefore warrants closer investigation.

Bridge details

The railway bridge is supported by four twin cylindrical piers and spans the river on an angle to the principal direction of flow in the river. Measurement of the bridge piers confirms a diameter of 1.4m.

Bridge drawings show that the width of the piers are 1.4m (confirmed by measurement onsite) with a sloping soffit height ranging from 124.5m on the right bank to 125.65m on the true left bank. The bridge drawing in shown in Figure 7-3 below and an aerial image is presented in Figure 7-4.

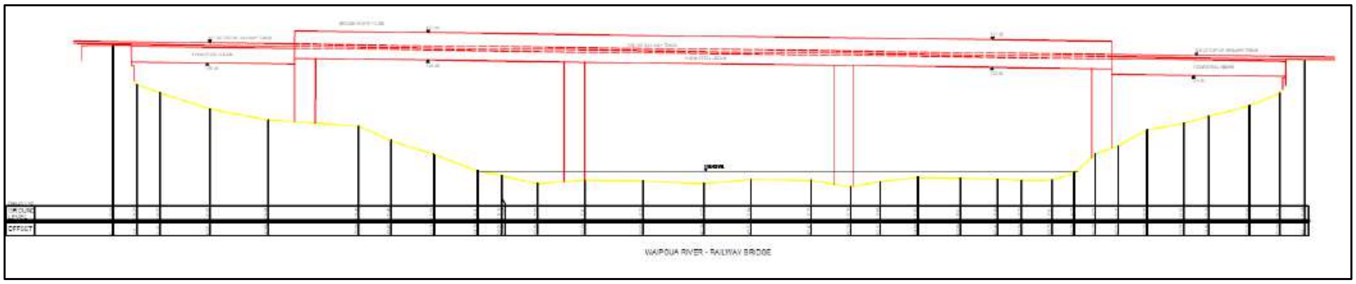


Figure 7-3 – Screen shot from CAD drawing of the Waipoua River Railway Bridge

Figure 7-4 – Aerial Image of railway bridge showing skewed alignment

MODELLED BRIDGE DETAILS

The bridge has been set up in the model as a series of five irregular shaped culverts with a length of 4m. Each culvert has been set up to represent a span of the bridge (i.e. the area between bridge piers). A visual representation of the rail bridge set up is presented in Figure 7-5 below.

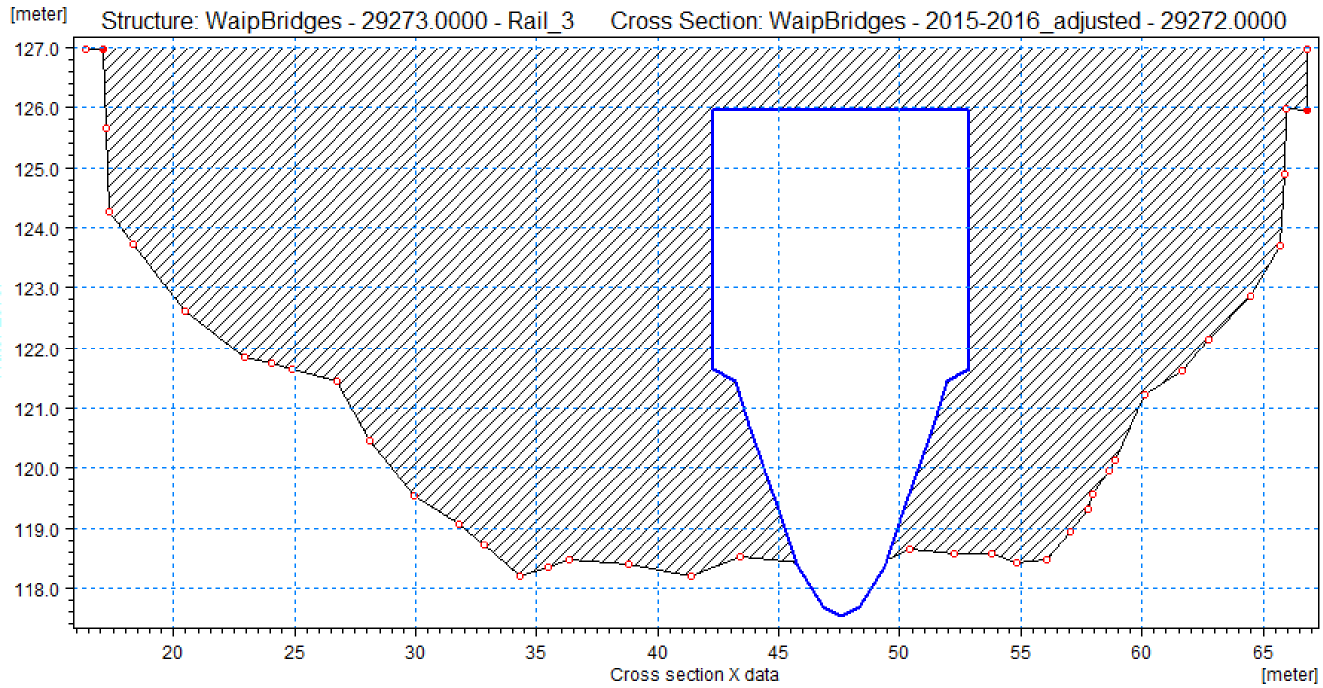


Figure 7-5 – Visualisation of modelled representation of individual rail bridge span.

Please note that the model report states, *"Overflow spill levels have been taken as the top of the fence where present, and the top of the railway track otherwise"*, however no overflow mechanism appears to be present in the version of the model provided to the auditor. A check of peak water levels in the model outputs, indicates that the model results are not reaching the level of the soffit, therefore this will have no immediate impact on the outputs from this study unless an overdesign flood is being simulated.

An analysis of the setup of the culverts within the model shows that the culvert setup appears to extend beneath the modelled cross section bed level. As a result of this, the existing bridge cannot compute a QH relationship, indicating that the bridge setup is not actually functioning as it appears. This needs to be investigated. The reason for this may simply be that the QH relationship was developed from a previous set of cross sections and the bridge setup was not updated when new cross sections were incorporated into the model.

Model Calibration at bridge

The primary event for model calibration is the 1998 flood event. Peak water levels in the model have been compared with the recorded debris levels which have been collected by staff from GWRC in the days following the flood event. These debris levels should always be treated with a degree of caution as determining a precise flood level days after a flood event based solely on debris levels can be very difficult. This is acknowledged elsewhere in the report, as two other debris levels have been excluded from the final calibration.

GWRC flood records indicate that a flood level was recorded on the true right bank approximately 30m upstream from the bridge itself and have a reported level of 122.96 m. The reported MIKE11 model results at this location give a level of 122.87 m which on the outset appears to be a very good fit with recorded flood levels.

Statement in regard to flood levels from Andrew Donald

The owner of the land (Andrew Donald, Mahunga Farm) on the true left bank at this location has reported that he walked over his land the morning after the flood event and observed that no floodwaters had inundated his farm except in two minor locations. Andrew Donald provided the following sketch indicating the extent of flooding overlaid onto a contour plan. This plan indicates that the peak flood levels immediately upstream of the bridge have an approximate level of 112.8m (Wellington MSL 1953 datum), or 122.02 m (Wairarapa Datum). This flood level is 0.85m lower than the level recorded on the true right bank by staff at GWRC. The plan is presented in Figure 7-6 below.

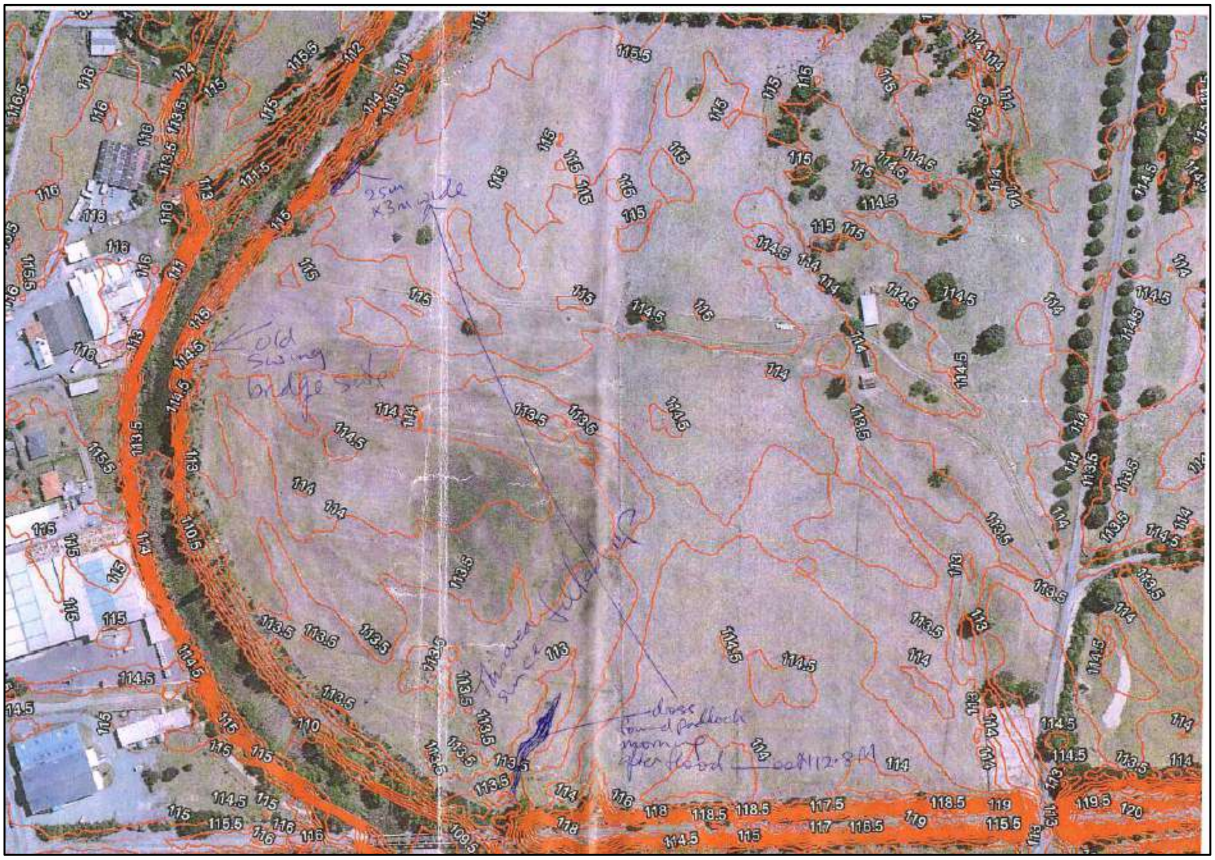


Figure 7-6 – Contour plan showing recollection of flood extent for 98 flood on Mahunga Farm (Andrew Donald)

Inspection of the only flood photo on file which covers a small portion of the Mahunga Farm does not show any visible signs that floodwaters have risen to the level shown in the model either. Whilst these flood photos were taken after the peak, there is significant ponded water and silt visible in other photos where the water has been. A comparison of the flood photo with the model results in that area is presented in Figure 7-7 and Figure 7-8 below. Unfortunately the flood photos are taken after the peak so it is difficult to draw firm conclusions, however of note in this photo is the fact that there is visible brown silt in the left of this photo indicating that flood water has inundated the area shown as flooding in the model, however there is no obvious silt present immediately upstream of the rail bridge.

Figure 7-7 – Flood photo - showing railway bridge on the right (taken after the peak)

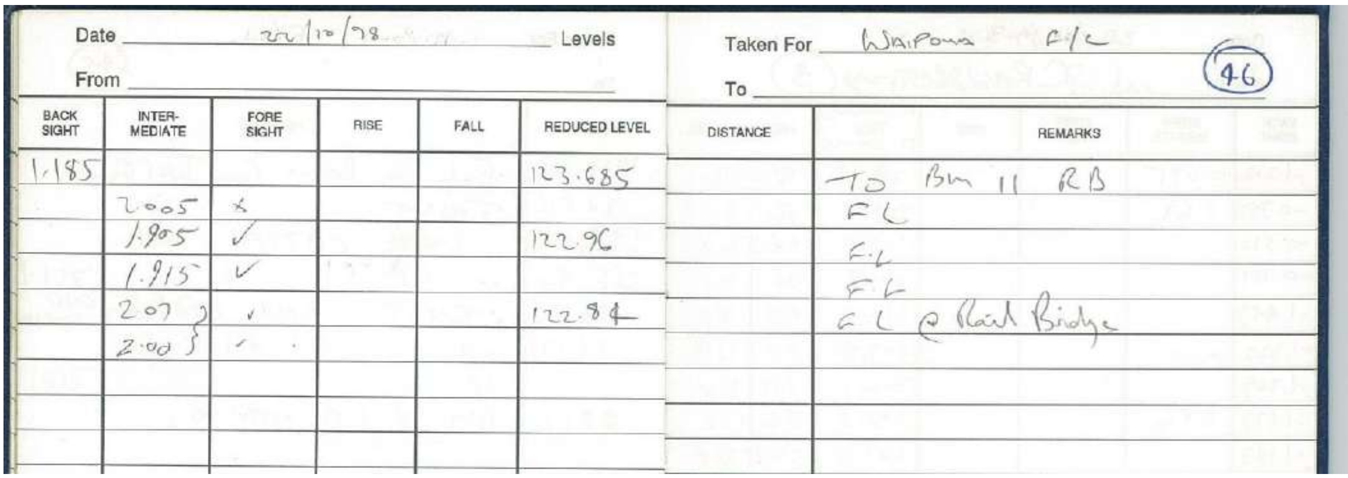
Figure 7-8 – 1998 calibration model results showing ponding on left bank upstream of railway bridge

Discussions with field staff from GWRC

Due to the significant different in flood levels between the left (as described by Andrew Donald) and the right bank (as recorded by GWRC) I have carried out some further investigations into the reliability of the recorded debris level at this location. I had a telephone conversation with Des Peterson from GWRC who is based in the Masterton Office on the 3rd of May 2019 in order to learn more information about how these levels were recorded.

Des has stated that the levels were recorded at the cross-section location approximately 2 days after the flood event. He highlighted that in this particular location the level can be tricky to accurately determine due to the steep nature of the bank in this location. He noted that there are several railway irons protruding into the river immediately downstream of this location which may have the potential to impact on the flood level. He also highlighted that there is significant vegetation growing on this bank which would have the potential to raise water levels in this location.

Des Peterson has doubled checked the recorded levels as well as confirming the benchmark levels at this location match those entered into the logbook. Three recordings were taken in the vicinity of section 11 and all three were within 0.1m of each other. A scan of the logbook is presented in Figure 7-9 below.



Date 22/10/98 Levels
From
Taken For Waipoua F/C
To
46

BACK SIGHT	INTER-MEDIATE	FORE SIGHT	RISE	FALL	REDUCED LEVEL	DISTANCE	REMARKS
1.185					123.685		TO BM 11 RB
	2.005	x					FL
	1.905	✓			122.96		FL
	1.915	✓					FL
	2.07	✓			122.84		FL @ Rail Bridge
	2.00	✓					

Figure 7-9 – GWRC logbook of debris recording after the October 1998 flood event

The logbook recordings appear to validate the reading in this location, indicating that the difference in flood levels observed on the left and right banks must be due to another reason.

Effects of Super Elevation

It is evident onsite and from aerial imagery that the water levels in this vicinity are likely to experience effects of superelevation due to being located on the outside edge of a fairly tight bend. Superelevation causes water levels on an outside bend to be raised, and conversely water levels on the inside bend to be lower. A range of formulae exist in order to estimate the likely magnitude due to the effects of super elevation.

I have carried out a check on the impacts of superelevation in this location using the following formula from the Federal Highways Administration methodology (FHWA, 1997)

$$\Delta Z = Z_O - Z_i = \frac{V^2}{g\, r_c}(r_O - r_i)$$

where

- Z = Elevation of the water surface, m
- V = Average velocity in the channel, m/s
- g = Acceleration of gravity, 9.81 m/s2
- r_c = Radius of curvature to the centreline of the channel, m
- r_0 = Radius of curvature to the outside flow line around the bend, m
- r_i = Radius of curvature to the inside flow line around the bend, m

Whilst there is a degree of uncertainty relating the precise radius of curvature of the channel at this location, my calculations give an approximate superelevation of 0.5m in this location, indicating that we could expect the water level to differ between the left and right bank in this location by 0.5m due to the effects of superelevation alone. (NB. 1-dimensional models such as that used to model this structure do not make allowances for superelevation, the effect of superelevation would be on top of the modelled water levels and need to be accounted for externally to the 1D model)

A file photo(Figure 7-10) showing water levels at the bridge during the November 1994 flood event appears to show increased water levels on the right bank due to a combination of superelevation and channel alignment. There also appears to be a fairly significant bow wave coming of the right piers which is likely to be accentuated in higher flows, particularly if blockage of the right piers is present.

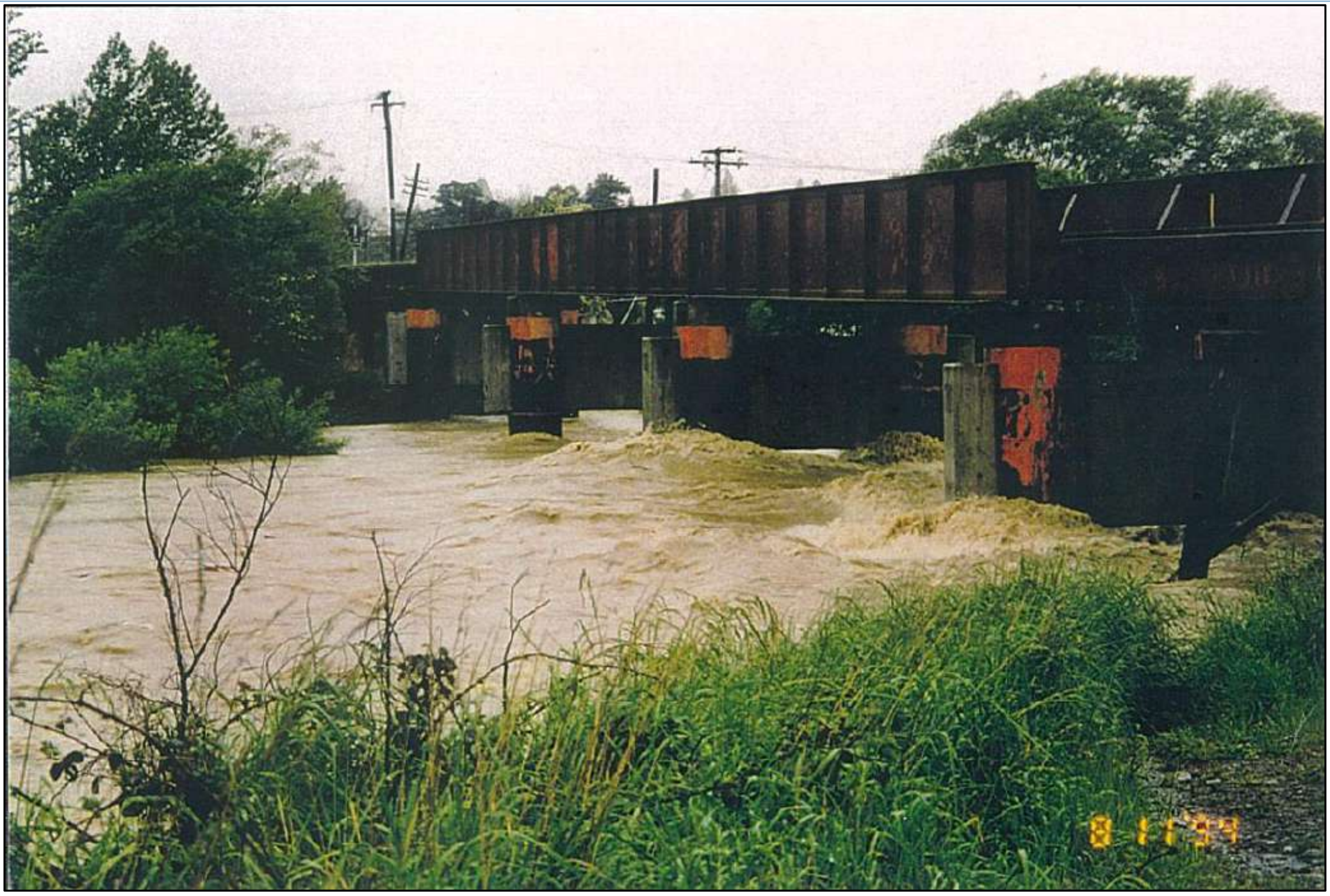


Figure 7-10 – File photo from the November 1994 flood showing elevated water levels on the true right bank

Visit to Site

I visited the site on the 7th of May 2019 and observed that in relation to the Opaki and Colombo Road bridges, there appeared to be significant capacity available through the railway bridge. The bridge piers are skewed to the principle flow direction and there are concrete blocks at the top of the piers which are likely to slightly increase the headloss once the water reaches this level. The location of the railway groynes and vegetation protruding from the true right bank was also confirmed.

Figure 7-11 – Picture showing capacity through the bridge looking downstream

Figure 7-12 – Railway groynes and vegetation on the true right bank in the vicinity of the debris recording location

Modelled Head Loss at Railway Bridge Structure

Inspection of the model results for the 1998 calibration as well as the 50 year CC run appears to be higher than I would expect for such a bridge, comparison of the headloss in relation to other bridges in the model suggests the headloss at this structure is significantly higher than any other bridge. Table 7-1 presents the modelled head loss at each structure.

Table 7-1 – Summary of modelled headloss through structures

Bridge Name	Modelled MIKE11 Chainage (m)	Headloss across structure (m)		Modelled Flow through bridge (m^3/s)	
		1998 Calibration	50 year CC	1998 Calibration	50 year CC
Lochores Road Bridge	18463	0.06	0.15	162	168
Paierau Road Bridge	25921	0.07	0.1	226	253
Rail Bridge	**29273**	**0.8**	**1.4**	**442**	**528**
Opaki Road Bridge (State Highway)	30426	0.39	0.36	433	537
Colombo Road Bridge	31398	0.26	0.36	433	537

It can be seen in Table 7-1 that the 50 year climate change results show a head loss through the railway bridge which is more than 1m greater than any other structure in the model. It also shows that the increase in headloss at the railway bridge from the 1998 calibration to the 50 year cc run is 0.6m compared to only 0.1m at the Colombo Rd bridge whereas the increase in peak flow is comparable. It is also suspicious that the headloss decreases at the Opaki Rd bridge when the flow increases.

A visual representation of the head loss at each modelled bridge is presented in the following figures for the 50 year climate change run.

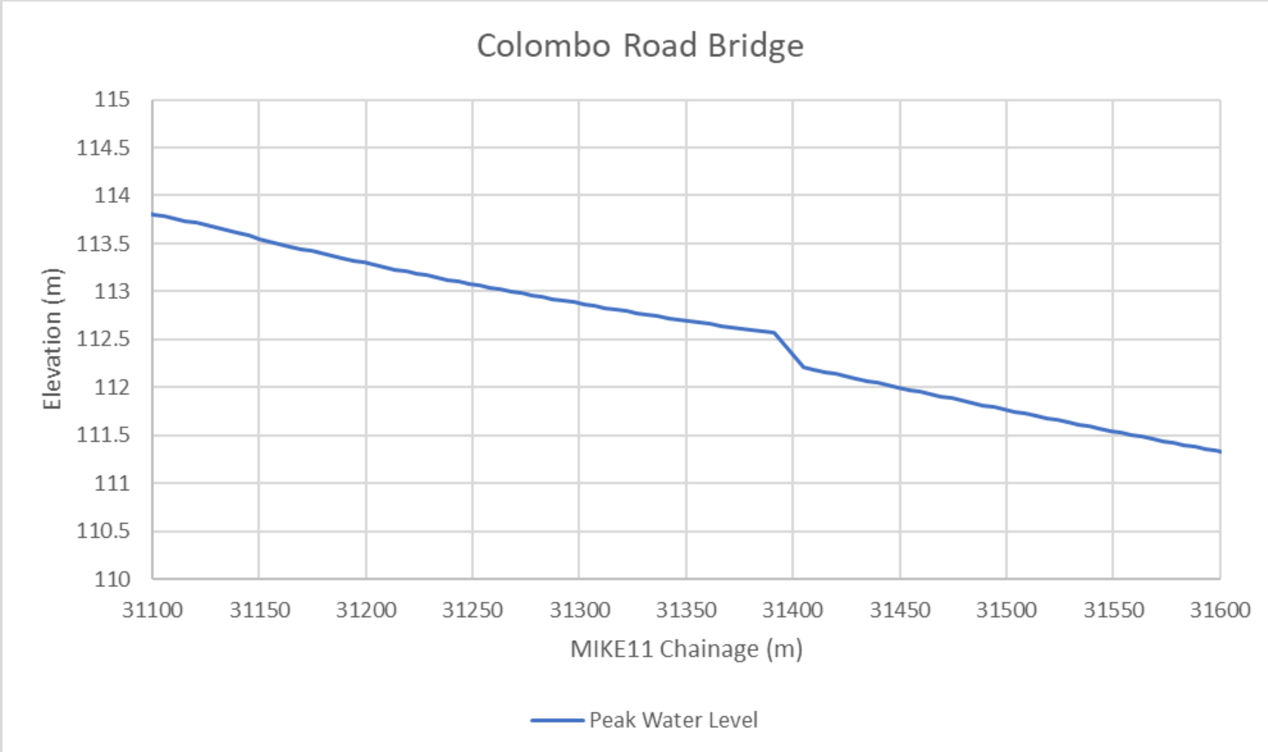


Figure 7-13 – Observed headloss through Colombo Rd Bridge (50 year CC)

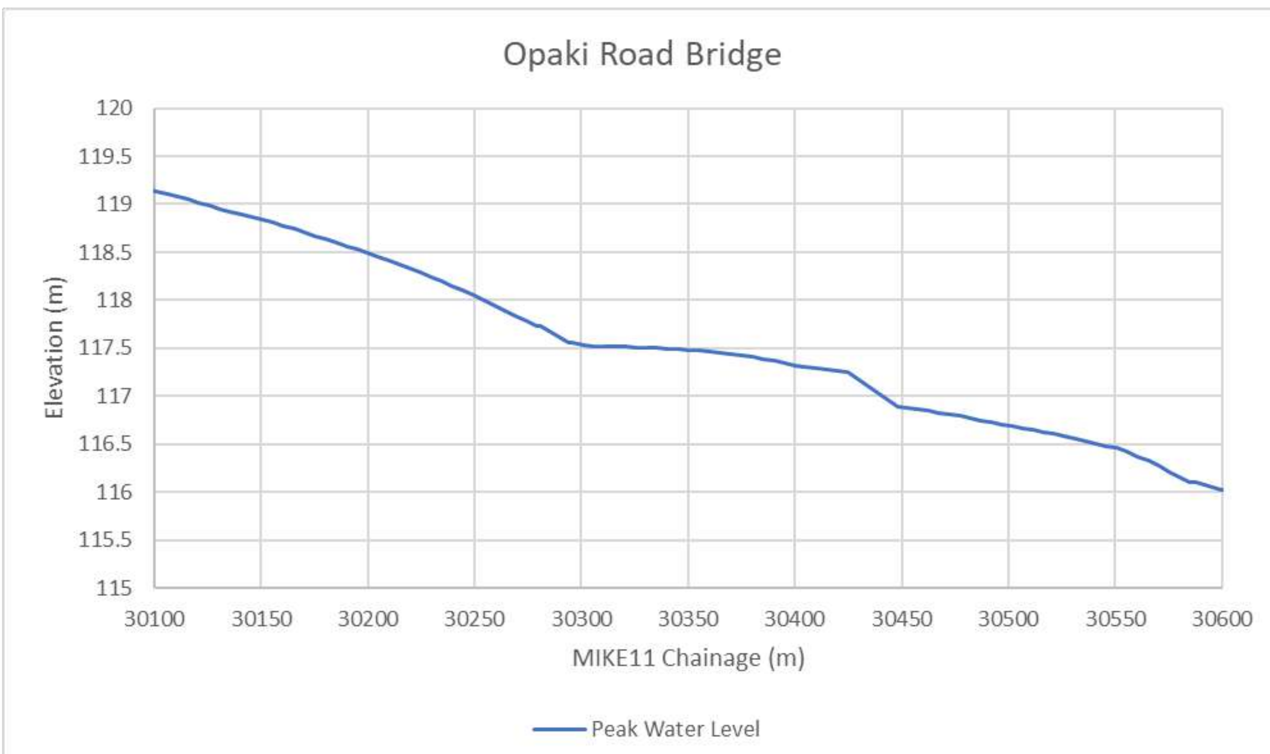


Figure 7-14 - Observed headloss through Opaki Rd Bridge (50 year CC)

Figure 7-15 - Observed headloss through Railway Bridge (50 year CC)

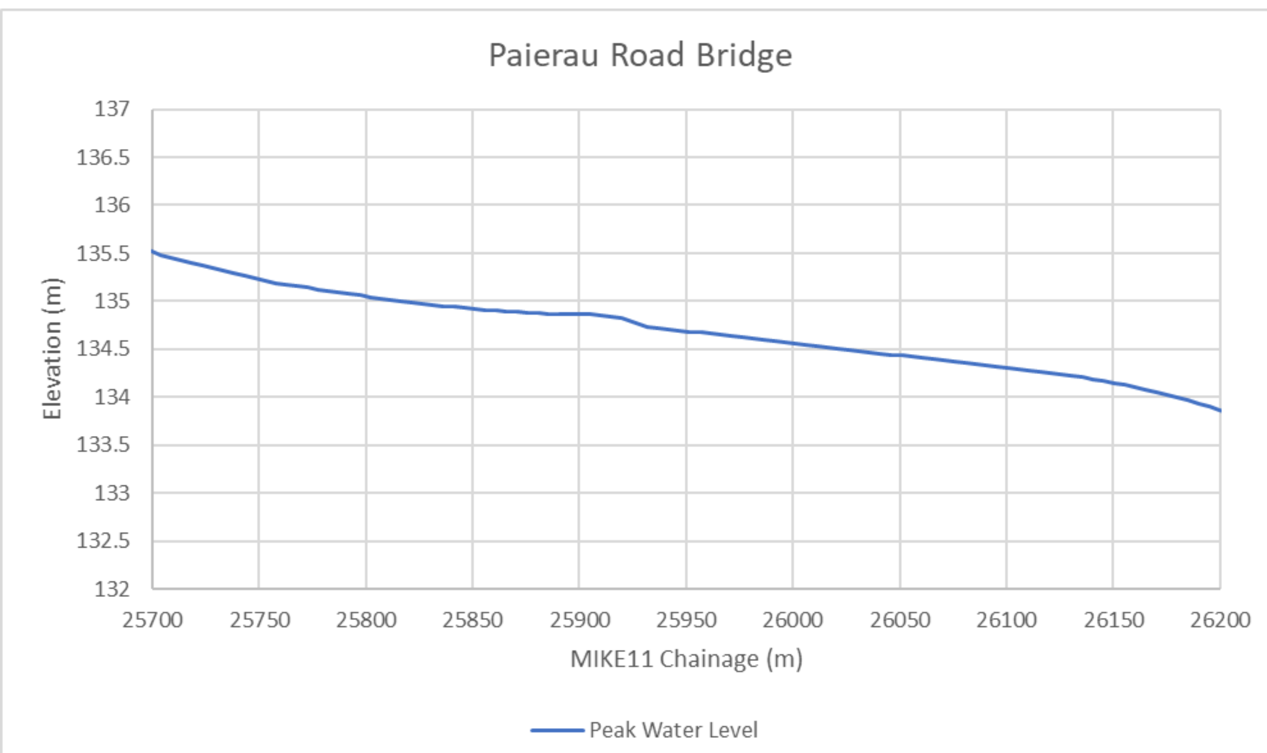


Figure 7-16 - Observed headloss through Paierau Rd Bridge (50 year CC)

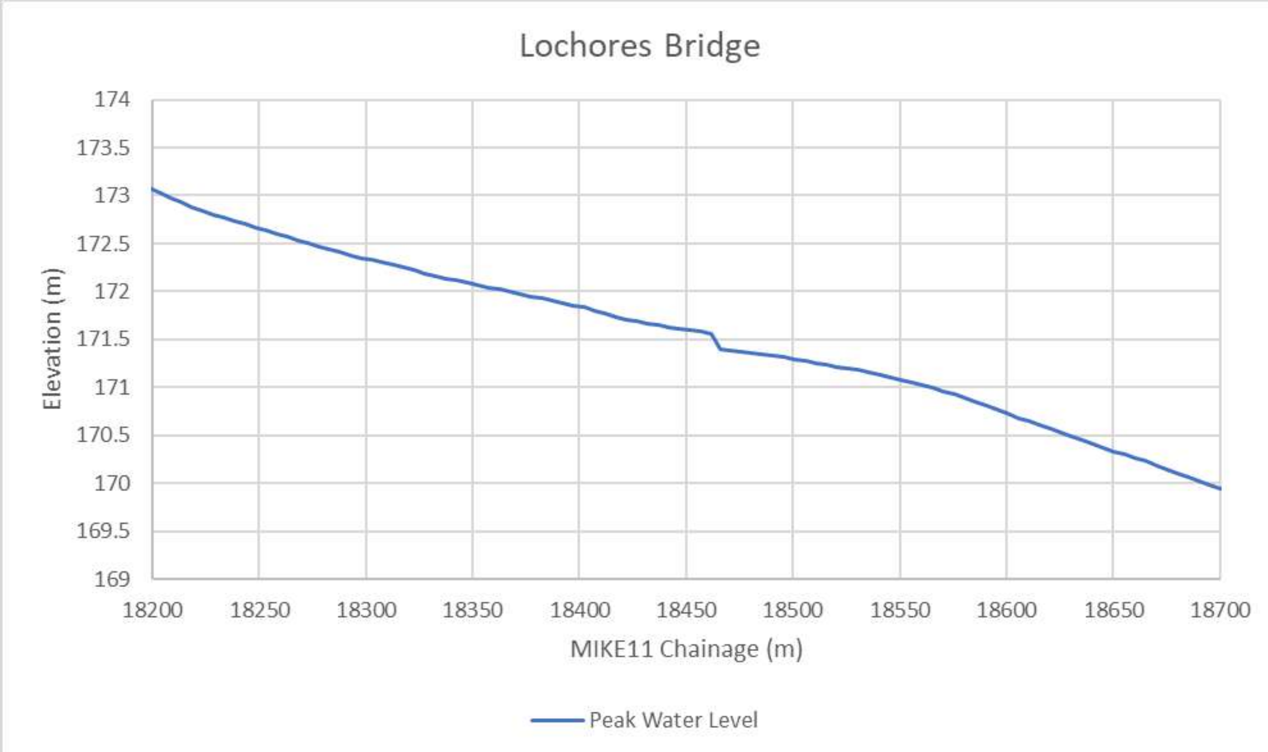


Figure 7-17 - Observed headloss through Lochores Rd Bridge (50 year CC)

It is my opinion that the observed headloss through the railway bridge is greater than can reasonably be expected, despite the fact that the bridge is skewed to the main direction of flow. Whilst the bridge is on an angle to the main direction of the flow, there appear to still be significant flow clearance on site and the observed headloss appears to be higher than I would expect.

I believe that consideration needs to be given to lower the level of the debris recording by at least 0.5m to allow for the effects of superelevation and other phenomena. Further justification could be given to lowering the level also due to the alignment of the bridge as well as for localised effects due to the trees protruding from the bank, the backwater effects from the railway irons and the localised wave impacts caused by the righthand bridge piers as observed in the 2004 flood photo (Figure 7-10).

Due to the significant impacts (i.e. flooding of Masterton) caused by the overflow of the railway embankment with the current setup, I believe further investigation into the sensitivity of the bridge setup needs to be investigated to see if reducing the headloss through the bridge will eliminate the embankment overflow which floods a large portion of Masterton.

In order to carry out a basic check on the sensitivity of the results to this bridge setup, I have remodelled the bridge using the FHWA WSPRO approach within MIKE11 and given a conservative blockage of the waterway to 25% (allowing for 8 piers and some debris blockage). The results using the increased Upper Mikimiki flows for the Q100cc run showed that the railway embankment did not overtop with this bridge setup with the total headloss through the bridge structure being in the order of 1m. I would consider this setup to be a fairly conservative representation of the bridge setup.

It is my opinion that the model setup could be more accurately represented using a standard methodology such as the FHWA approach, or a standard headloss factor is applied based on hand calculations using guidelines from a reputed source such as "Open-Channel Hydraulics" (Chow, 1959). Considering the sensitivity of the model results and overall floodspread to this structure, it may be appropriate to ensure that a range of experts are consulted with before an appropriate headloss through this structure is agreed upon. It would also be a worthwhile exercise to look into the reliability of the debris recordings for the whole network at the same time.

MAJOR RECOMMENDATION 1

Consideration needs to be given to adjusting the level of the recorded debris level at XS7 to account for the effect of superelevation as well as other physical phenomena. A review of the appropriateness of all debris levels used in this study may be warranted at the same time.

Significant thought should be given as to the most appropriate way to model the effects of the skew on the bridge. Due to the complexities involved in modelling this bridge structure, and the immense interest from the community in the impact of this structure, it may be appropriate to ensure that a range of experts are consulted with before an appropriate headloss through this structure is agreed upon.

5.2 REPRESENTATION OF LATERAL LINKS

The model report notes that instabilities were encountered due to the setup of the lateral links which transfer water between the 1D and 2D models. In order to remove the instabilities, the exponential smoothing factor has been reduced to 0.2. It has been reported that this has not had a significant impact on model results, which is likely due to the low timestep.

Where the lateral links run along the top of a stopbank, the stopbank crest levels have been included in the underlying terrain model. It is noted that in some locations the lateral link runs slightly inside the stopbank crest and may encourage water to fluctuate between the 1D and 2D models. This may be the cause for the instabilities which were observed and could be resolved by defining the crest levels within the lateral links themselves using an external link file. More information on setting up these types of links can be found within the MIKE Flood user manual. Considering the model is currently running well and not showing any instabilities at the links it is probably not justified to change the setup of the lateral links for this model, but should be kept in mind for future model setups.

LATERAL LINK CONNECTIONS

In order to assist in visualisation of the link connections, the output from the 'MFLateralLinkConnections.txt' file was converted to a shapefile. Inspection of the output shows that the link connections appear to be working as expected in most locations, however it was also evident that there were several gaps in the lateral links. The most significant of these is a 130m gap in the link on the true right bank downstream from the State Highway Bridge as shown in Figure 7-18 below. Whilst the smaller gaps (~20m wide) are unlikely to make a significant difference to the results, the large gap on the true right bank will prevent water from exiting the river in this location should the bank overtop and should ideally be included in the model.

Figure 7-18 – Visualisation of lateral link connections showing gap in right bank lateral link (red lines showing link connections to centre of river)

MINOR RECOMMENDATION 15

It is recommended that the lateral link on the true right bank of the river downstream from the State Highway bridge (M11 chainage 30590 to 30720) is included in the model, or otherwise its exclusion is justified. The inclusion of this link is unlikely to have a significant impact on the overall model conclusions.

MODEL CALIBRATION / VALIDATION

The primary event for model calibration is the 1998 flood event.

Calibration process has involved;

- Inspection of flood photos
- Liaison through MDC to receive feedback on model calibration
- Presentation of flood hazard via a range of formats including flood depth, extent and videos of the model results

Feedback received from MDC on the flood maps is included in Appendix C

Overall the model calibration is generally good. Flood extents upstream of the railway embankment appear to match the flood photos and feedback from staff at MDC is positive.

Model calibration has two issues however in two key locations which are the Railway Bridge and at Mawley Holiday Park.

7..1 RAILWAY BRIDGE

Model calibration at the railway bridge matches very well with the surveyed debris level, however does not match with the anecdotal evidence from the landowner on the true left bank. Inspection of flood photos also appears to validate the anecdotal evidence as no silt or ponded water is apparent on the land, whereas silt is clearly visible in other areas where water has been.

A detailed investigation as outlined in section 7..1 has concluded that the location of the debris recording is not ideal for use as a flood record and would likely need significant adjustment if it was to be used. It should be highlighted here that one of the recommendations in the peer review document (Rix, 2018) is that '*additional information regarding the information source for the observed water levels and discussion around the uncertainty of these levels and locations would assist further discussion about calibration confidence'.* These recommendations tie in with the recommendations and further investigations carried out in this audit report.

7..2 MAWLEY HOLIDAY PARK

Discussions with local residents as well as council staff have indicated that the extent of flooding shown in the model at Mawley Holiday Park as well as the surrounding area did not occur. Whilst it is not disputed that flooding did occur in this area, the degree of flooding has been reported to me a being considerably less.

Discussions with a range of people from residents, council staff and former staff of GWRC have given varying recollections as to the cause and exact nature of the flooding, however all agree that the flooding was not as significant as shown in the calibration results.

Inspection of flood photos also suggests that the flood extent was less than shown in the model, with silt being present in only localised areas.

Inspection of the model results show that the model is showing a significant volume of overflow on the true left bank in between sections 7 and 9. Model results show the water spilling out of the main river downstream of cross section 9 (at MIKE11 chainage 29950) and running behind the stopbank, backing up at the Opaki road bridge embankment and then spilling back into the river in the vicinity of cross section 7 (MIKE11 chainage 30425). A visualisation of the model results for the 1998 calibration run, overlaid with vectors showing the direction of flow is presented in Figure 7-19.

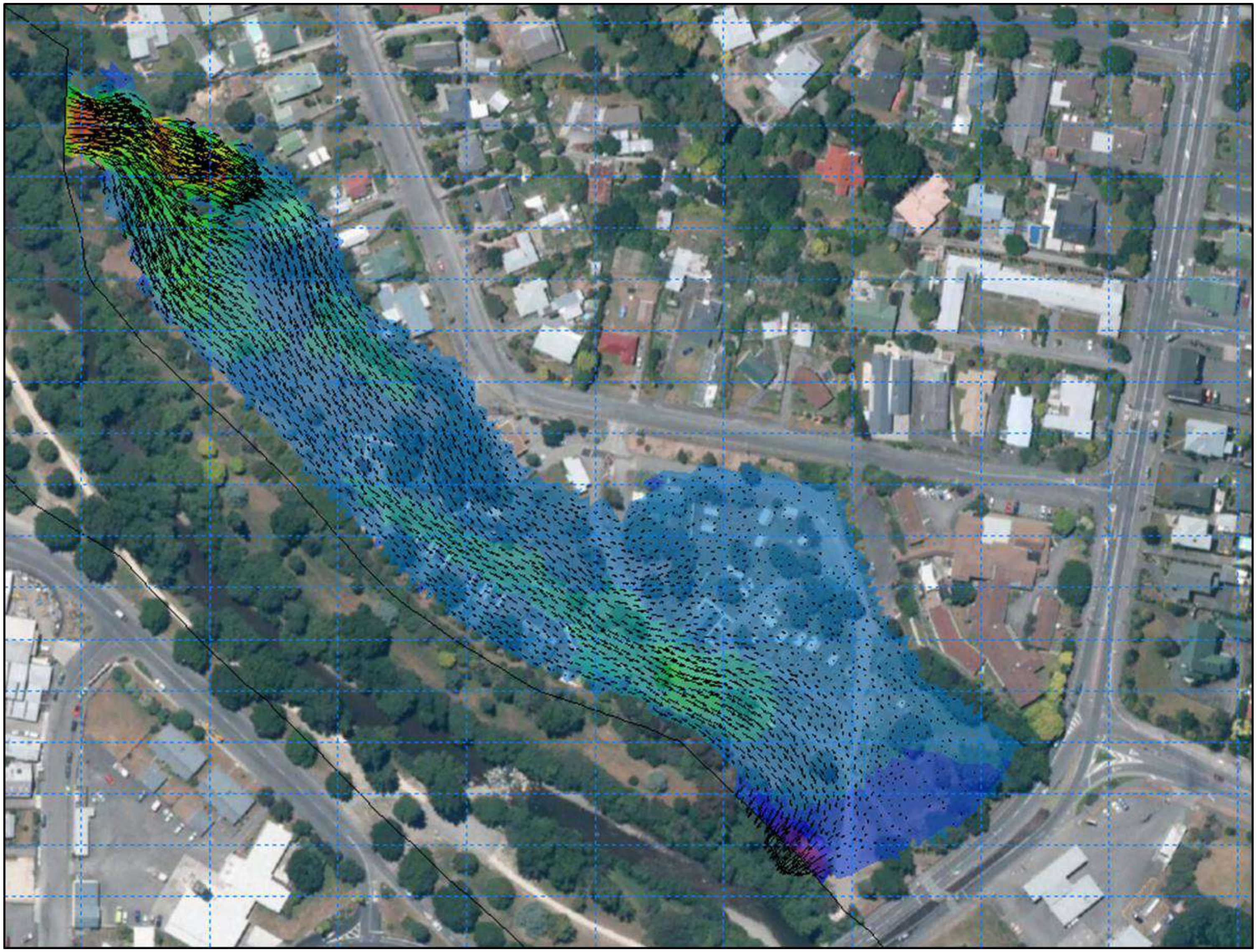


Figure 7-19 - 1998 Calibration results overlaid with vectors showing the direction of flow

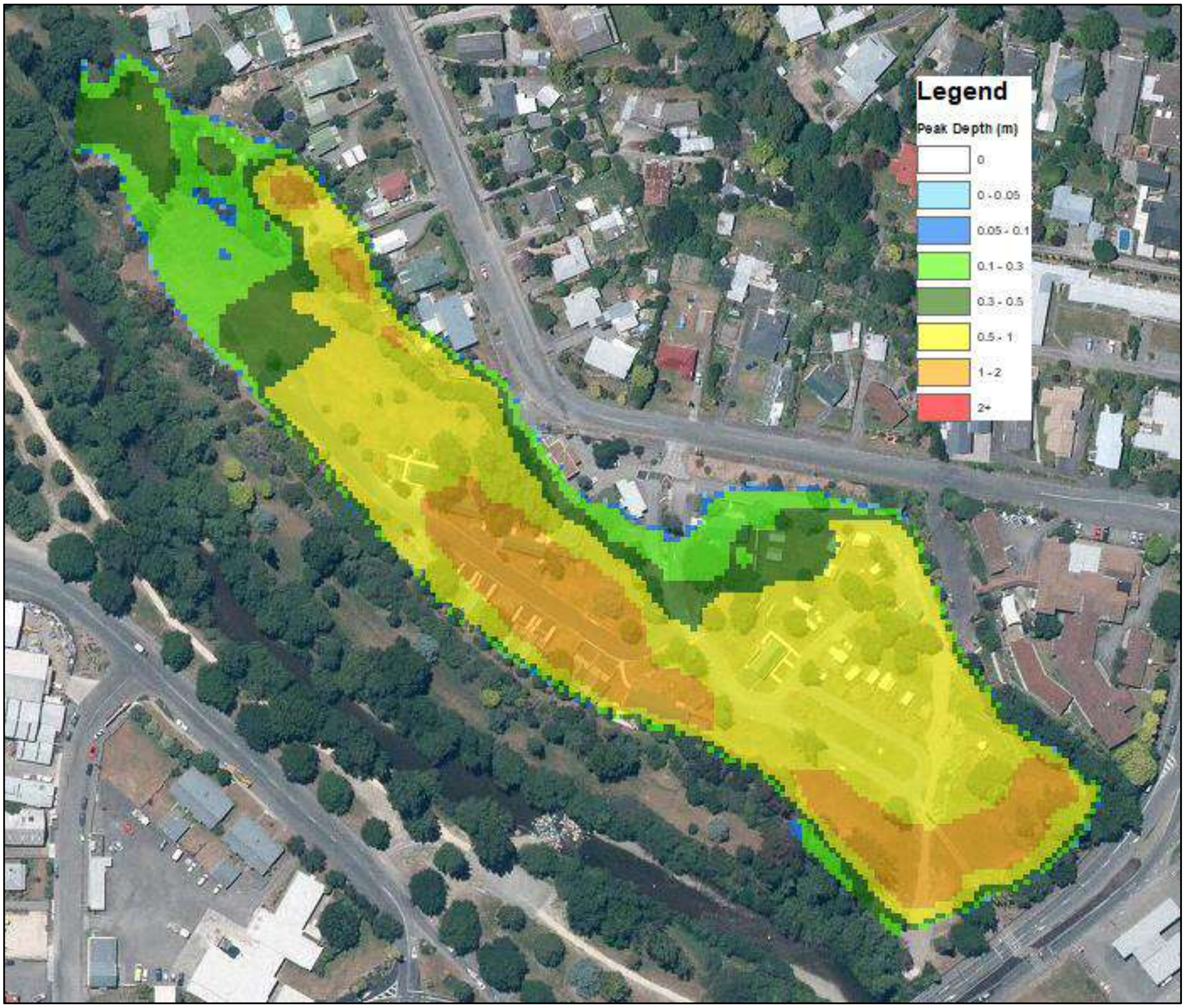


Figure 7-20 - 1998 Calibration results showing water depths

MIKE11 model results show that the bank does not actually overtop in any location however spills through the upstream gap in the stopbank.

An inspection of the model in this area shows that a significant gap in the stopbank is present in the model as shown in Figure 7-21.

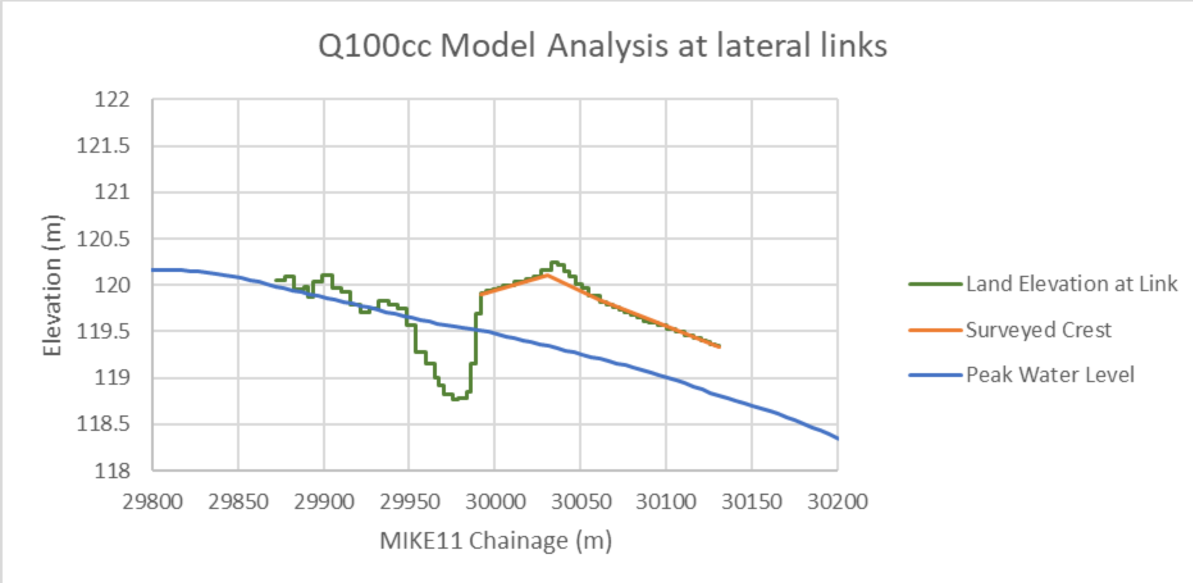


Figure 7-21 – Inspection of modelled links at left bank spill location

An inspection of the underlying LiDAR (2008) used in the model shows that there is a gap in the stopbank in this location, however when compared with the 2013 LiDAR dataset in the same location, it appears that there is actually a continuous bank in this location. Topographic survey data which was commissioned by GWRC in this area has not picked up a bank however, which explains why the modeller would have believed the gap in the bank seems realistic. The reason why the surveyors did not survey the bank should be further investigated.

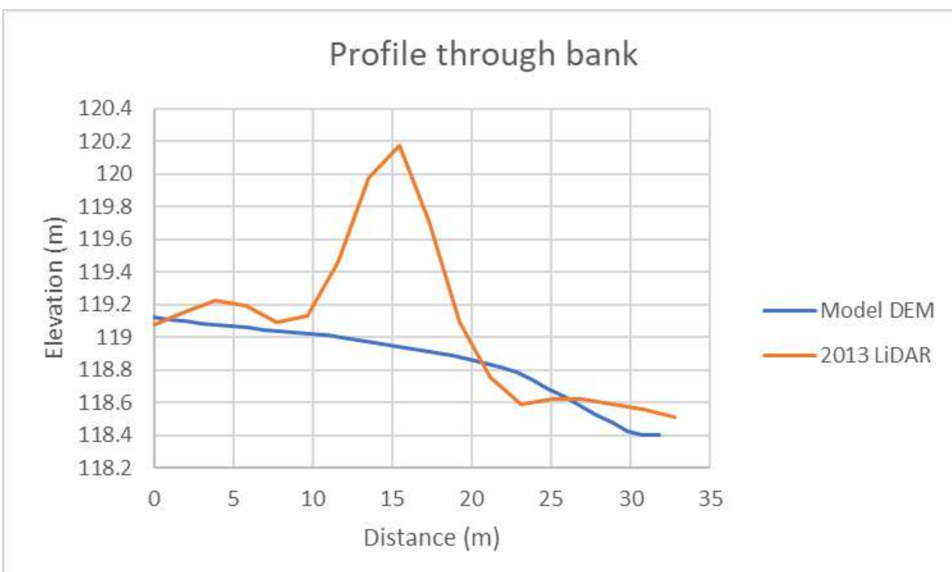


Figure 7-22 – Comparison of modelled terrain levels (2008 LiDAR) with 2013 LiDAR

I have been told that a local resident who overlooks this area (Mark Hall) insists that this behaviour did not occur during the flood event in 1998 and states that no flood waters were observed in this location. Flood photos (taken in the morning several hours after the peak) do not show any evidence that floodwaters reached this location, whilst there is evidence in the photos of silt deposits in the vicinity of Mawley Holiday Park.

Whilst there is no debate that water entered Mawley Holiday park, there are no records available to suggest the flood waters in Mawley Holiday Park were as deep as the calibration result show. Flood waters of such a depth are likely to have caused significant damage and would have been likely to have been reported on in more detail.

SITE VISIT

I went to site on the 9th of May to inspect the area and can confirm that there appears to be a solid continuous embankment in this location. The embankment is covered in thick ivy and blackberry in locations, however would appear to act as a significant barrier to flow. The following figures show the bank as observed on site.

Figure 7-23 – View of continuous stopbank on site

Figure 7-24 - View of continuous stopbank on site (2)

MAJOR RECOMMENDATION 2
That this embankment is surveyed in detail and included in the model. The model will then need to recalibrated as a result.

7..3 GENERAL CONCLUSION – MODEL CALIBRATION

A detailed comparison of the flood extents with the 1998 flood photos is presented in Appendix C. Analysis of these images in conjunction with a comparison of the model results with the recorded debris levels indicates that overall the model calibrates well to historic data however the calibration needs to be refined / reconsidered at the Rail bridge and the reach between the Rail bridge and the Opaki Rd Bridge.

RAILWAY EMBANKMENT

As has been highlighted previously, the sensitivity runs show the railway embankment overtopping and flooding a large number of properties downstream. Due to the critical nature of this flooding, particular attention has been paid to this location of the model.

Survey drawing were requested from GWRC so that the modelled terrain levels can be compared with the actual surveyed levels in this location. A profile was extracted from the base terrain which is adopted in the model and the ground levels were found to represent the ground levels well, however it was evident that the railway irons, which are marked on the survey plans had not been included.

Peak water level results for the sensitivity run were also extracted in this location and found to be overtopping the embankment by only 0.1m. Whilst the railway irons may not prevent the entirety of the flow overtopping the bank in this location, it is likely they would act as a significant barrier to the flow, and would ideally be included in the model.

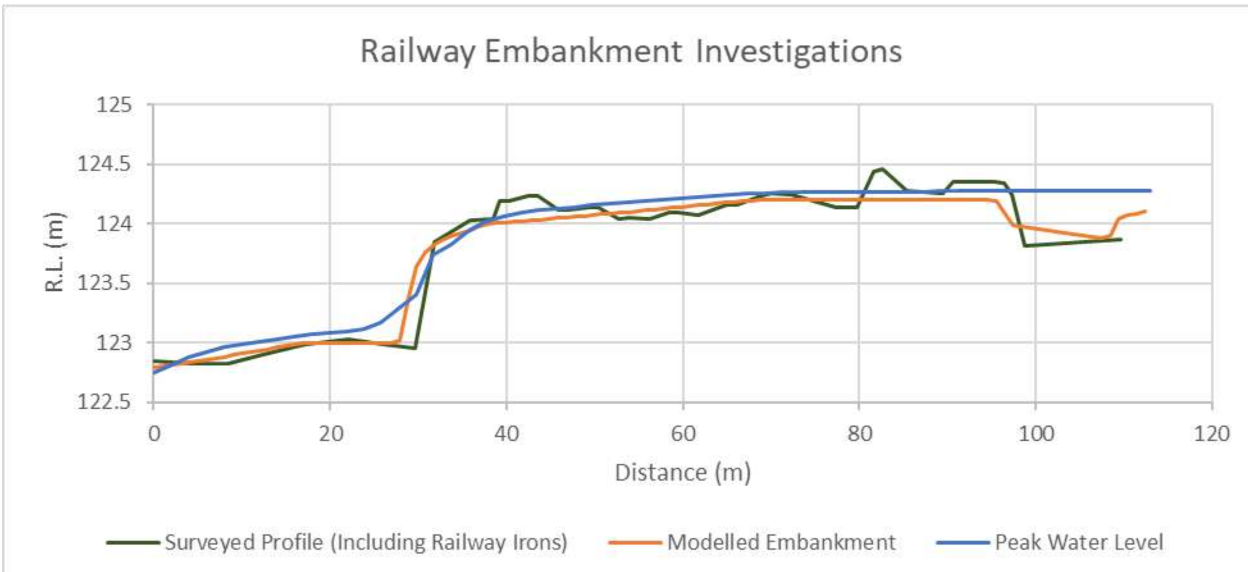


Figure 7-25 – Comparison of modelled embankment levels with survey

In order to get a feel for the sensitivity of the model to the embankment height, I have rerun the model with embankment raised to account for the railway iron. Results show that the railway irons have little impact on the overall flood extent with the floodwaters still backing up behind them and overtopping the embankment. Flood extents are reduced minimally with this sensitivity run. In reality if the embankment was to overtop, it is likely a degree of scour under the railway tracks may occur which would actually increase the flooding; therefore the exclusion of the railway tracks may be justified.

PEER REVIEW

A desktop peer review of the model build was carried out in December 2018 (Rix, 2018). The peer review document has made a number of comments / recommendations with have been categorised by the importance of recommendation as follows:

Importance	Recommendation
N/A	No action
0	No action, but items of importance for reporting
1	Minor recommendation and unlikely to affect model results. GWRC can choose whether to implement or not
2	Moderate recommendation. Unlikely to significantly affect model results but effect should be demonstrated to increase confidence in model results.
3	Major recommendation. Likely to significantly affect model results.

A basic assessment of each item given an importance level of greater than 1 is included below.

Category 1 – Minor Recommendations

PEER REVIEW MINOR RECOMMENDATION 1

Boundary Conditions - *Previous review comments that the Ruamāhanga River does not significantly affect water levels from the Waipoua River around urban Masterton have been discussed further with GWRC. Refer Item 11 for additional sensitivity assessment.*

Response

This is no longer issue as model has been run with 100 year flow in the Ruamāhanga river for the design runs as per the PDP coincidence table.

PEER REVIEW MINOR RECOMMENDATION 2

River channel topography - *As part of the review, T+T assumed that the modelled cross sections represented the surveyed cross sections. We recommend that GWRC ensure that the modelled cross sections are representative of the surveyed cross sections.*

Response

Based on a site visit and inspection of the model setup I can see no reason as to why the modelled cross sections are not representative of the surveyed cross sections.

PEER REVIEW MINOR RECOMMENDATION 3

2D topography - *The 2D model topography is based on a combination of LiDAR surveys flown between 2008 and 2013. The LiDAR based DEM has not been analysed as part of this scope of works, but we recommend that GWRC check ground levels in the model/DEM where the source LiDAR changes. This is to ensure that there are no "step changes" between LiDAR surveys. We recommend that the report provide a summary of the horizontal and vertical accuracy of the LiDAR and checks carried out.*

Response

GWRC have carried out some initial checks and have indicated that initial checks look good. I recommend that this is formalised and documented so that this point in the peer review can be checked off.

MINOR RECOMMENDATION 16

It is recommended that the seamline checks are formalised and documented.

PEER REVIEW MINOR RECOMMENDATION 4

Model Parameters *- The 2D roughness values have been set by floodplain type. The roughness values seem reasonable for the land uses referred in the report. However, we recommend that the references for roughness and land use information layers are provided in the report for transparency. We have assumed the land use adopted in the model is correct.*

Response

The modelling report has now been updated to include this and this can be checked off as complete.

PEER REVIEW MINOR RECOMMENDATION 5

Calibration - *The model results were compared with observed flood levels in the Waipoua River and the representation in the area of interest appears reasonable. Additional information regarding the information source for the observed water levels and discussion regarding uncertainty of these levels and locations would assist further discussion about calibration confidence.*

Response

It has been identified as part of this review that there is significant uncertainty at least one of the calibration debris recordings. This warrants further investigation as has already been highlighted in this report. (See section 7..1)

PEER REVIEW MINOR RECOMMENDATION 6

Calibration - Additional investigations of the September 2010 event carried out by GWRC indicate that the modelled travel time between the Mikimiki flow gauge and Colombo is similar (~2hours). This gives some confidence that the roughness between the two stations is reasonable. We also recommend that the rating curve from the model and gauged rating curve at Colombo are compared.

Response

MINOR RECOMMENDATION 17

It is recommended that the rating curve from the model and gauged rating curve at Colombo are compared as per the peer review report. It is currently unclear if a rating curve is yet to be developed for the Colombo Rd site, this should be investigated.

Category 2 – Moderate Recommendations

PEER REVIEW MODERATE RECOMMENDATION 1

1D/2D duplication and gaps - *A comparison of the 1D channel widths and the extent of the 2D model domain was carried out to ensure that there were no duplications (i.e. included in both models) or gaps (i.e. not in either model). The review identified gaps in locations where 'cross section channel markers' are used to reduce the 1D cross section. The largest differences (~50m) occur around the weir locations (ch 30296m, 30601m and 30899m) although smaller differences are observed elsewhere.*

We recommend that either the 1D model is extended to the 2D domain, or the 2D domain is reduced to the represented 1D cross section width. Once the changes have been made, we recommend developing a long section displaying peak discharge and chainage. The peak discharge should include 1D and 2D flows.

Response

The model has been checked and I have confirmed that there are significant differences in channel width between the 1D and 2D models in the locations identified. The easiest way to fix this would be to move the bank markers in the 1D model. (NB. There is a function within the MIKE Hydro river software which can automatically shift bank markers to match a shapefile polyline. This may be the most accurate way to achieve this for the entire model.) Once the bank markers have been adjusted, the calibration would need to be rerun, but this has been recommended anyway so would require very little extra work.

MODERATE RECOMMENDATION 12

That the 1D bank markers are adjusted to match the 2D mesh boundary, in particular in the locations highlighted in the peer review report.

PEER REVIEW MODERATE RECOMMENDATION 2

Bridges - *There is a bridge/culvert under Paierau Road that is located within the Waipoua floodplain and is not represented in the 2D model. The bridge/culvert should be represented in the model.*

Response

This has now been included as per the peer reviewer's recommendation.

PEER REVIEW MODERATE RECOMMENDATION 3

Sensitivity - *Previous review comments that the Ruamāhanga River does not significantly affect water levels from the Waipoua River around urban Masterton have been discussed further with GWRC. The previous conclusion was assessed on the basis that GWRC have assessed the influence of a 20 year ARI level in the*

Ruamahunga River on flood levels from the Waipoua River in urban Masterton up to 100 year ARI. Based on similarities between the upstream travel times from the Ruamahunga and Waipoua Rivers, we recommend that the influence of a 100 year ARI level in the Ruamahunga River on 100 year ARI levels in the Waipoua is assessed. This can be assessed as a sensitivity scenario and can be used to assist the options assessment process.

Response

This is no longer issue as model has been run with 100-year flow in the Ruamāhanga river for the design runs as per the PDP coincidence table.

8 COMMUNITY CONCERNS (HYDRAULIC COMPONENT)

The RFQ document outlined several community concerns to be considered which are addressed below

SENSITIVITY OF THE MODEL RESULTS TO THE WEIR OPPOSITE THE FIRE STATION

Sensitivity tests have been carried out by GWRC and included in the final model report which analyse the impact of removing this weir.

The test show that removing the weir may lower the water levels in the immediate vicinity of the weir in the order of 0.2-0.3m, however will only lower water levels upstream by an average of ~0.1m This is due to the steep nature of the river in this location and the fact that the weirs are well submerged during a flood event.

These tests do not consider the long-term impacts on bed levels however which would be likely to occur if this weir was removed. Removing the weir completely may have unpredictable effects on long term degradation of the bed. If any weir was to be removed, the potential impacts would need to be considered and analysed by someone with expert knowledge in river geomorphology.

The risk otherwise is that bed degradation may undermine important assets which are protecting the town.

Consideration could be given to lowering the weir if it was considered feasible, but this is beyond the scope of this review. It is recommended that these issues are explored in more detail in the implementation phase of the FMP process.

SETUP OF THE MODEL IN THE VICINITY OF THE RAIL BRIDGE, INCLUDING HOW BLOCKAGE OF THE BRIDGE HAS BEEN REPRESENTED

This has been addressed in detail in section 7..1

THE POTENTIAL IMPACT OF LONG-TERM BED LEVEL TRENDS ON MODEL CALIBRATION (WAIPOA RIVER)

Good model calibration relies on good input data. Critical to the calibration of a flood model is accurate cross section / bed level data, and accurate flow estimates. Unfortunately, due to the lack of reliable bed level data as well as flow estimates it is very difficult to accurately calibrate a model to the historic floods of 1947. An estimate of the approximate mean bed level could potentially be made from historic plans, however information relating to the shape of the river cross section would be impossible to determine based on these plans alone, making any conclusions on channel capacity in 1947 to be limited.

Information is on file however to indicate that the bed level has degraded since the 1940's and if more information comes to hand to allow a more reliable estimate of flow to be determined then the long-term bed degradation needs to be taken into account.

There are a number of flood levels further up the river on record from the 1947 flood event (see Appendix D). It would be a worthwhile exercise to convert these levels into the current datum and into S.I units and then a comparison could be made with the existing model results for the reach upstream of the rail bridge.

Caution would need to be taken in making firm conclusions from these levels however as little information is on hand about the state of vegetation in the river as well as channel width to compare with the current channel conditions.

Differences in roughness due to changes in vegetation on the berms can have a significant impact on flood levels. Further information on the river alignment and nature of the berm vegetation may be able to be found in historical archives. There are historic ortho images of large areas of New Zealand which were taken around the time of the second world war. It would be an interesting exercise to locate any historic aerial photos and make a comparison with the current state of the river.

MINOR RECOMMENDATION 18

It is recommended that thought is given to converting the historical flood levels from the 1947 flood into the current datum and make a comparison of the historic flood levels with the 1998 model calibration results in the same location. This may assist in getting a better feel for the likely magnitude of the 1947 flood event.

A search for historic orthophotos could also be made from the 1940's.

WHETHER THE AVAILABLE DATA FOR THE 1947 EVENT IS SUITABLE TO BE INCORPORATED INTO THE DESIGN HYDROLOGY.

This is assessed in detail in the hydrology section of the report in section **Error! Reference source not found.**

9 INPUT HYDROLOGY

A technical review of the hydrology supporting the Te Kauru FMP has been undertaken by Vicki Henderson of Barnett and MacMurray Ltd, as part of the wider Te Kauru FMP audit.

Aspects of the hydrology which have been assessed were drawn from the audit terms of reference and are:

General

- The type of software and modelling package used for the hydrology
- The modelling method used and its appropriateness for the hydrology

Input data

- Rainfall data
- Measured flood flows
- Calibration data against historical events

Assumptions

- Run-off coefficients or similar hydrological parameters
- Predicted flood flows used for design events
- Climate change allowances

Community Concerns

- Lower Waipoua catchment contribution
- Whether the available data for the 1947 event is suitable to be incorporated into the Waipoua design hydrology.

The hydrology review is divided into three parts. First, the overall conclusion to the audit question of whether the hydrology is fit for purpose is given. This is followed by the technical sections which cover the general hydrology applying to the upper Ruamāhanga River and its tributaries, followed by the hydrology for the Waipoua River, where different methodology has been applied.

HYDROLOGY CONCLUSIONS

The key question being evaluated is whether the hydrology used for the FMP is fit for purpose, measured against a list of intended uses for the floodplain management plan. These were outlined in the audit terms of reference. The findings for the hydrology are given for each level of use below.

- Catchment-scale flood hazard assessment, including classification of hazard into different categories, for the Waipoua River and other rivers in the FMP study area.

The current hydrology would be acceptable for high level planning and catchment scale flood assessment.

- Development and conceptual design of different flood management options (including being used for analysis of potential flood damages).

The present hydrology is considered fit for this item, if used with caution. Any conceptual design of flood management options should be high level. Once the hydrology is updated, the optimal flood management options might be different in method, location or scale. Certain specific flood management solutions might be pursued where GW is confident the hydrology refinements would have less effect.

- Providing information for use by District Councils in LIMs, land use controls and for building controls (Local Government Act, Resource Management Act and Building Act requirements).

The current hydrology is not deemed suitable for these uses, as the level of detail sought is greater and flood hazard information may change when modelling is refined.

- To provide potential update to existing flood hazard information in the District Plan.

The current hydrology is not considered appropriate for use in the District Plan.

In the technical hydrology sections that follow, a number of recommendations are given to improve the FMP hydrology. These are summarised in the executive summary of this report, and in Appendix G.

9.1 GENERAL UPPER RUAMAHANGA HYDROLOGY

9.1.1 HYDROLOGY SOFTWARE

Design flood flows were estimated using flood frequency distributions generated by TIDEDA. This was done initially by PDP (2013). TIDEDA is a commonly used standard system for managing and analysing hydrological data and is quite acceptable for this purpose. Pearson type 3 flood frequency distributions were selected as having the best fit for the upper Ruamāhanga stations. TIDEDA does not offer a confidence interval for this distribution. This presents the issue that a design flood flow has been estimated, without an associated uncertainty.

ArcGIS was used to process flood frequency parameter contours into a grid, delineate catchments and calculate physical catchment characteristics, such as areas. This is industry standard software.

The rest of the hydrological work is based mainly on spreadsheets, which is adequate for the purpose.

9.1.2 HYDROLOGICAL MODELLING METHODS

9.1.2.1 FLOOD FREQUENCY ANALYSIS

Flood frequency analysis of 10 at-site flow records was used to estimate flood frequency distributions. This was done for the following stations:

Ruamāhanga at Waihenga Bridge
Waiohine at Gorge
Ruamāhanga at Gladstone
Taueru at Weraiti
Waingawa at Upper Kaituna

Ruamāhanga at Wardells
Waipoua at Mikimiki
Ruamāhanga at Mt Bruce
Kopuaranga at Palmers Bridge
Whangaehu at Waihi

This work was done by PDP (2013), who selected the Pearson type 3 distribution as the best fit for all the flow series, although plots comparing the different distributions were not shown. However, a study focussing on the flood flows in the Waipoua at Mikimiki (NIWA, 2015), showed that using the alternative EV1 or Gumbel distribution for that river produced a 100 year ARI design flood estimate within 7% of the PDP estimate using the PE3 distribution. It is difficult to estimate the uncertainty for PE3 distribution flood estimates, but this should have been attempted, since the design flood flows are critical to the flood hydrology.

MODERATE RECOMMENDATION 14

Estimate confidence intervals for the PE3 flood frequency distributions.

9.1.2.2 REGIONAL FLOOD FREQUENCY CHARACTERISTIC CONTOURS

Design flows estimated from these distributions were used to generate regional flood frequency characteristic contours. Flood frequency contours were transformed into a raster grid using ArcGIS software to produce 100m grid cells of flood frequency parameters. The difficulty with rasterising contour data from sparse data points is that the grid implies that you have more information than is actually the case. Care must be taken in using this approach across ungauged catchments in particular, that parameters selected from the gridded information are consistent with the broader contour and station data that underlie them. Also an appropriate gridding technique should be selected for the type of data being processed.

In section 8 of the MWH report (2016), there was also a clear caution about using the regional frequency contours to estimate design flows. The alternative proposed by MWH for the Waipoua catchment was a rainfall–runoff model for the hill catchments, combined with a hydraulic model using rain-on-grid for the plains catchments.

MODERATE RECOMMENDATION 15

The underlying station data and other catchment characteristics must be taken into consideration in the selection of flood frequency characteristics from contours. Used in isolation the contours can generate anomalous results.

9.1.2.3 SUB CATCHMENTS

Each river catchment was divided into sub catchments down to a minimum size of 5km^2. Flood frequency parameters at a subcatchment centroid were used to calculate a peak design flood flow for each return period. The catchment division seems rather fine, given the coarseness of the available hydrology data. It seems a case of the hydraulic model channel layout dictating the scale of the catchment model.

MINOR RECOMMENDATION 19

Simplify the sub catchment representation if scaled hydrographs are to be used.

9.1.2.4 DESIGN HYDROGRAPH FORM

Design hydrograph forms were created for each of 7 river gauging sites by taking the 20 largest flow events and normalising them by dividing by the peak flow. After removing outliers, a mean hydrograph form for each of the sites was determined. Design hydrographs were created by scaling the normalised mean hydrograph to the design peak flow. These hydrographs are a good way to create a design hydrograph for a particular site, but they may not be representative of subcatchments upstream or downstream. Other subcatchments will have a different time of concentration, and the form of the hydrograph will depend on the shape, steepness and length of a subcatchment. In applying the same hydrograph form to all the subcatchments, design flood flows into the main stem of each river will tend to peak at the same time, which seems conservative. This approach would tend to overestimate peak flood levels in the Ruamahanga river network.

MAJOR RECOMMENDATION 3

Design hydrographs should reflect subcatchment characteristics, such as time of concentration. This can be estimated by theoretical equations or rainfall-runoff modelling.

9.1.2.5 AEP FLOW COINCIDENCE

This is a method employed to determine which AEP flood should be applied to different catchments for design flood modelling. It recognises that across a large catchment like the Ruamahanga with hill and flat subcatchments of different sizes and aspect and covering a large geographical area, not all the subcatchments typically experience a flood event of the same return period at the same time. A statistical analysis of high flows for 8 river stations was carried out (PDP, 2013). Using the 10 highest flows at each station, peak flows occurring during the same storm event at the other stations were identified. Peak flows at all the dependent stations were divided by the peak flow at the control station to give a percentage. For a design flow at each control station, estimated from the frequency distribution, a coincident flow at the other stations was estimated by multiplying the control station design flow by the mean percentage from the analysis for each dependent station. These flows were then converted to an AEP value based on each stations own frequency distribution. These were provided in a table which was used by GW in determining which AEP events to apply to the subcatchments for flood modelling.

There are two issues with the AEP coincidence tables approach. First, linearity is assumed in the scaling, meaning that the flow at each dependent station is assumed to be a fixed percentage of the flow at the reference station. Secondly, the flood coincidence tables are not always consistent. Examples given below are drawn from the AEP coincidence table and working provided by PDP in their appendices. Working is only shown for one control point: Ruamāhanga at Gladstone. Incidentally, the PDP worked example only shows results for 9, not 10 high flow events as indicated in the text.

LINEARITY

For all the high flows used in this analysis, the largest estimated return periods at all stations bar two were 20 to 40 years. (The exceptions were the Kopuaranga with a 60 year flow and the Whangaehu, which had an estimated 100 year flow). The largest flow on record for Ruamāhanga at Gladstone has an estimated return period of 23 years. This forms the upper limit for flows used in the statistical analysis at this control point. This particular flow event coincided with a flood flow of return period 8.5 years on Waipoua at Mikimiki and of 2 years at Waingawa respectively. When the coincident flows at these two stations were transformed to a percentage of the peak flow at Ruamāhanga at Gladstone for the 10 (or 9) events, the percentages ranged from 6-24% for the Waipoua and 6-37% for the Waingawa.

The design flood flows up to 100 year ARI for the control point are multiplied by the mean percentage to determine a coincident flow at the dependent points.

This assumes that if you step to a larger design flow at the control point, is it sensible to linearly scale up the dependent station flows. This is despite each station having its own independent frequency distribution, and there being no information about relative flood flows at higher return periods than that stated above for Gladstone. The control point will be pulling the other station flows along, in a pattern which matches its distribution. This technique also does not account for the wide variation in coincident flow response, which in the example given, had a range of 20-30% for two critical dependent stations. The mean scaling percentage is used.

LACK OF CONSISTENCY IN TABLES

For the Upper Ruamāhanga hydraulic studies the catchment has been divided in two. The lower hydraulic model starts from Ruamāhanga at Wardells, and finishes at the confluence with the Waiohine River. It includes the Waingawa and Taueru rivers and the Ruamāhanga at Gladstone station is included just downstream of the confluence with the Taueru. Looking at the two Ruamāhanga river stations as control points in the PDP table, we can see the following:

Starting from the Ruamāhanga at Wardells as a control point, with a 100 year event, the table shows that both the Waingawa and Waipoua could also have a 100 year flow. The coincident flood on the Ruamāhanga at Gladstone is predicted to be 50 year. This implies that in a model spanning the Ruamāhanga from Wardells to Gladstone, the intervening subcatchments should have a design flood return period of 50 years to coincide with a 100 year flood at Wardells.

Starting from the Ruamāhanga at Gladstone as a control point, with a 100 year event, the coincident event on both the Waingawa and Waipoua would be less than a 5 year event. The coincident event on the Ruamāhanga at Wardells would be the 100 year.

In addition, I note that with Ruamāhanga at Mt Bruce as control point and a 100 year event, the coincident event at Ruamāhanga at Wardells is a 20 year event. However, if Ruamāhanga at Wardells is the control point, for a 100 year return period event there, the coincident event for Ruamāhanga at Mt Bruce is given as a 50 year event. These results imply that these two stations don't necessarily share a severe coincident event either.

All this means that the AEP coincident events chosen are dependent on which station is chosen as the control point.

A good analysis has been done and this approach is useful to give an indication of which coincident floods have occurred in the different catchments. Extending these relationships in a linear fashion to higher return periods does not seem a robust practice. Also, use of the summary table provided by PDP is subjective, leading to different results depending on the user's interpretation. It would seem in the lower hydraulic model, that the return periods should vary as you move down the catchment.

In order to simplify the modelling, GW has assumed that all western catchments experience the same 100y return period event together, while a minor event occurs in the eastern catchments. Conversely, all eastern catchments are assumed to share the same 100y return period event while a minor event occurs in the western catchments. While this may be simpler, it assumes more than one river catchment is affected by a severe flood event simultaneously. This is not supported by the tables and may be overconservative.

In summary, the reviewer has doubts about this AEP coincidence method being extended to large return period events. Assuming that most of the Ruamahanga catchment experiences a 100 year return period event at the same time reflects an event of greater severity than 100 years, because of the variation and evolution of weather patterns as they move across a region.

MODERATE RECOMMENDATION 17

The AEP coincidence tables are not proven for higher return periods, and have a linear basis which is not sound. Simplification of this approach in design flood modelling has been overconservative. Consider using the AEP coincidence tables more systematically and developing an alternative method for scaling up AEP coincident flows.

MODERATE RECOMMENDATION 13

The Ruamahanga hydrology combines flood frequency parameter contours, scaled station hydrographs, fine catchment divisions, and conservative AEP coincidence assumptions. These methods have a high cumulative uncertainty. The model ought to be validated to historic flood events, and attempts should be made to refine the hydrology input.

It might be timely for GW to evaluate the best way forward for hydrological modelling of the Ruamahanga catchment. Earlier peer reviewers have mentioned rainfall-runoff modelling as an alternative that could provide better data on catchment storm response. There are shortcomings related to the generality of the current hydrology, but there would need to be sufficient data to build robust rainfall-runoff models.

MODERATE RECOMMENDATION 16

That GW carry out an evaluation of whether a rainfall-runoff model or the current regional flood frequency based hydrology method, or some combination is the best way to deliver hydrology inputs to the FMP.

9.1.3 UPPER RUAMAHANGA INPUT DATA

9.1.3.1 MEASURED FLOOD FLOWS

The records for 12 flow recording stations in the Upper Ruamāhanga were analysed for stationarity and quality (PDP, 2013). The stationarity tests were for absence of trend and stability of variance and mean. The analysis was a reasonably thorough check and found that 10 out of the 12 stations appeared to be stable over time. Exceptions were Taueru River at Te Weraiti and the Waiohine at Gorge, although this record was stationary post 1980. The Taueru River at Te Weraiti record exhibited a slight upward trend in flow. It is unclear whether the cause of this has been investigated further, as was suggested by PDP at the time, but the Taueru at Te Weraiti flow record data has been used to develop a flood frequency distribution and applied in the flood modelling.

Useful information on the quality of each of the flow records is provided in the PDP report, but it is not summarised and no estimate of uncertainty is provided, given that many of the gauging sites are unstable, and have recorded flood levels well in excess of their maximum gauged flow. Difficulties with, and limitations of the gaugings described in the more detailed GW hydrological sites statistics summary (Gordon, 2011), are generally not mentioned. For example, the Ruamāhanga at Wardells site is just 600m upstream of the confluence with the Waingawa River. It is noted that in high flows, backwater effect can influence the rating, and caution is advised in reading high flows from the rating curve. Since this is the primary source of information for the flood hydrology, maintaining regular site gaugings and estimating the uncertainty of the rating is important.

GENERAL RECOMMENDATION 9

Maintain regular gauging of sites and provide an estimate of the uncertainty in the rating.

MINOR RECOMMENDATION 23

Investigate the cause of the drift in the Taueru at Te Weraiti record and decide whether any adjustment needs to be made.

9.1.4 UPPER RUAMAHANGA ASSSUMPTIONS

9.1.4.1 HYDROLOGICAL PARAMETERS

USE OF REGIONAL FLOOD FREQUENCY CHARACTERISTIC CONTOURS

Using frequency distributions for the 10 stations in the Upper Ruamāhanga catchment to generate regional contours for flood frequency parameters is an established methodology. It gives an approximate design flood estimate for ungauged catchments. However, interpolating a grid of parameter values between the contours implies that a firm value is known at all points. This has implications for the current Ruamāhanga flood hydrology. A screen shot of the $\bar{Q}$ / A $^{0.8}$ contours from the GW hydraulic modelling report is shown in Figure 9-1 below.

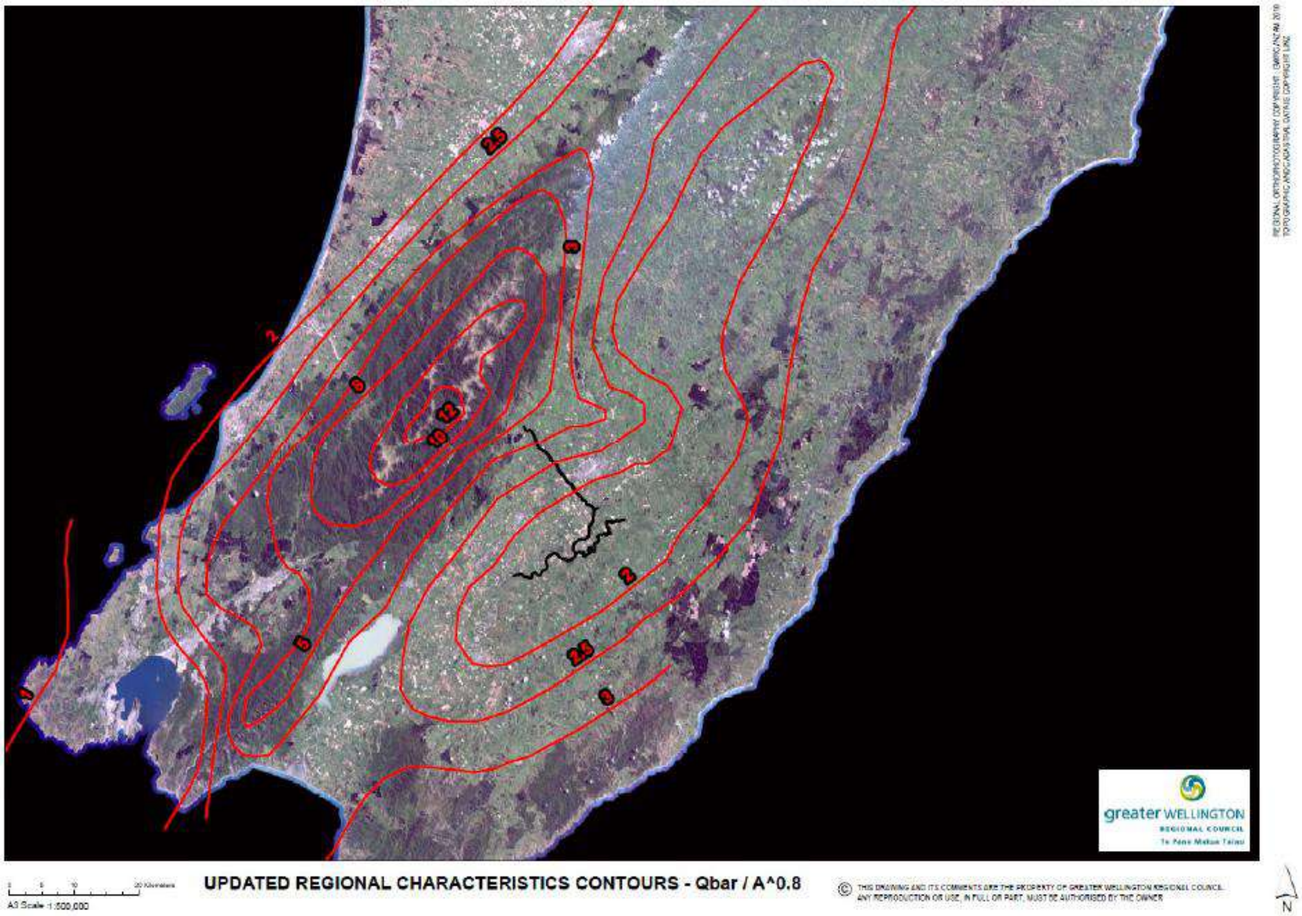


Figure 9-1 - Regional $\bar{Q}$/ A $^{0.8}$ contours

This parameter relates the size of the mean annual flood to the catchment area. A larger value means a larger mean annual flood relative to the catchment area, and hence larger design flood estimates. It can be seen that the contour values increase sharply going up into the Tararua ranges, and are less on the plains. Looking at the lower hydraulic model (GW, 2013), where the channel network is shown in black, the 2.0 contour creates a closed loop around the lower Waingawa, the Ruamāhanga catchment from Wardells to its confluence with the Waiohine and much of the Taueru catchment. The interpolated raster grid from these contours has assigned a value of 2.0 to this entire region. This is despite the fact that $\bar{Q}$ / A $^{0.8}$ values for the 4 individual stations with centroids within this contour have values ranging from 0.8 to 1.7. All of the Taueru subcatchments and the Ruamāhanga subcatchments downstream of Wardells, have been assigned a $\bar{Q}$ / A $^{0.8}$ = 2.0. This has been done for the upstream extent subcatchment of Taueru, even though it has its own at-site value of 0.8 for $\bar{Q}$ / A $^{0.8}$. It is likely that this overestimates design flows for these subcatchments by 25-50%. This may affect all of the Taueru subcatchments, and the Ruamāhanga subcatchments currently delivering almost 40% of the peak Ruamāhanga design flows in the lower hydraulic model.

It was difficult to investigate this point because the relevant data was not presented on a single figure, but had to be inferred from studying multiple plots. A figure showing the regional contours, and the values at centroid from which they were derived, together with the sub catchments, catchment centroids, river channel network and stations would have helped. This is important because it shows the application of the data to the model.

Design hydrographs for each subcatchment have been reproduced in the GW lower hydraulic modelling report appendices. These serve as a useful check of design flood inputs. Some of the catchments have clearly spaced peak flows for the different return periods, including the source catchments. Some have one return period flood hydrograph sitting atop another. Example screen shots are shown in Figure 9-2 and Figure 9-3 (GW,2013).

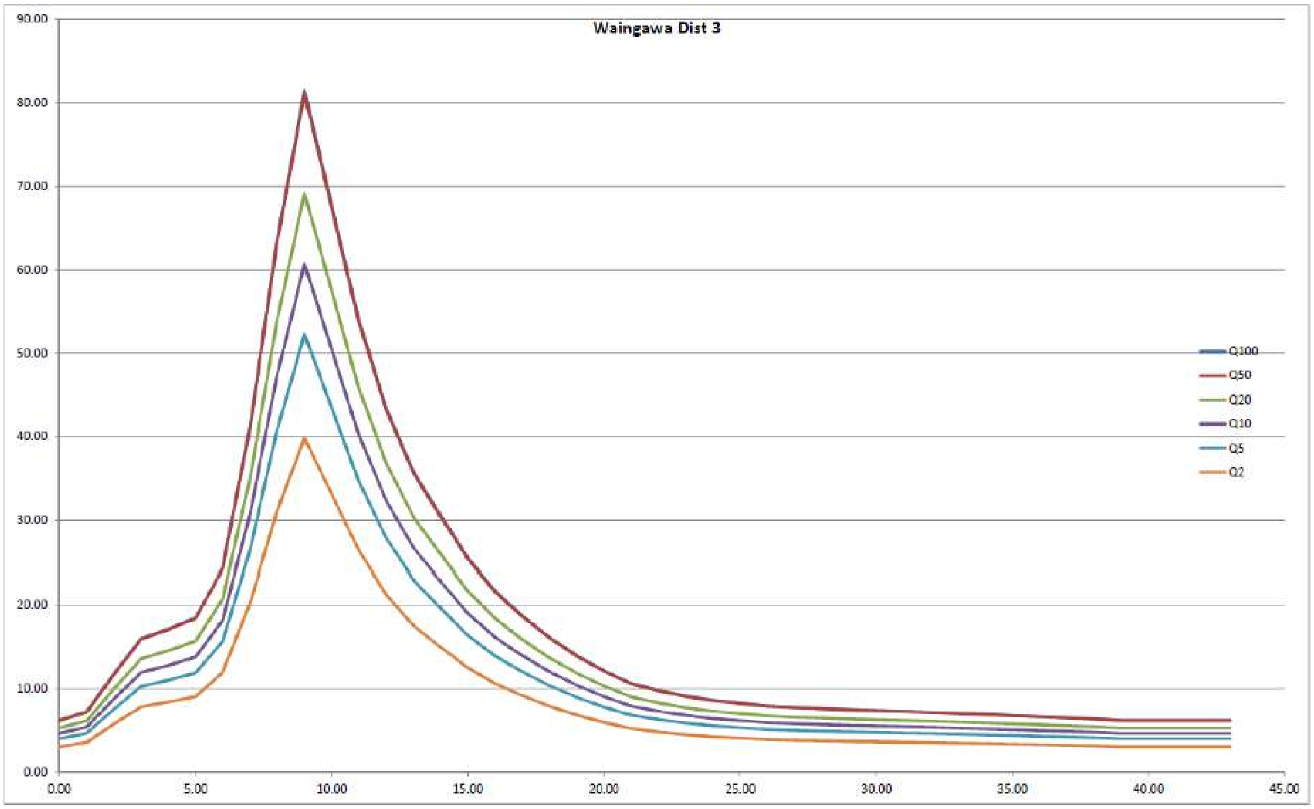


Figure 9-2 - Waingawa Dist3 subcatchment design hydrographs

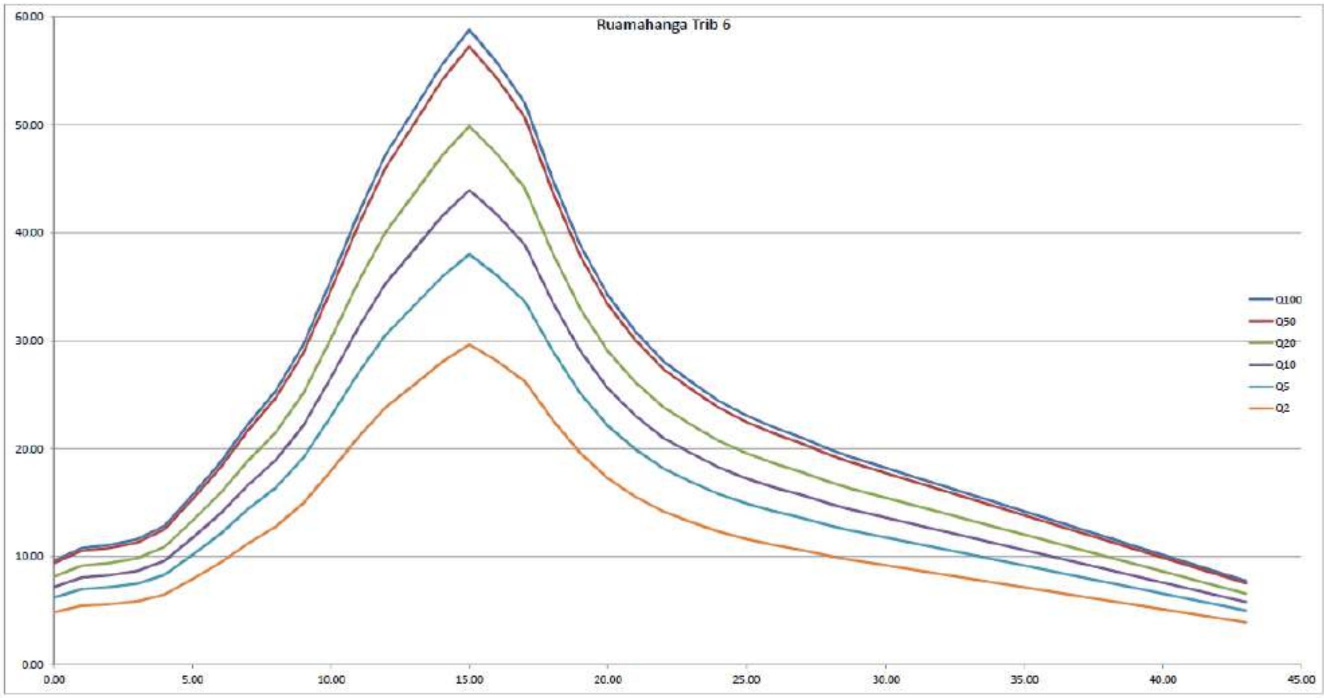


Figure 9-3 - Ruamāhanga Trib6 subcatchment design hydrographs

In these two cases, the 50-year return period peak flow is the same as, or nearly the same as, the 100 year peak flow, while the steps between the other return period flows are wider and more regular. This does not seem logical, and a more consistent estimate for these high return period events should be made for catchments where the regional flood contours give such close values for two distinct return periods.

MAJOR RECOMMENDATION 5

Revisit application of regional flood frequency parameters to ungauged subcatchments, paying particular attention to location of source data and catchment characteristics such as elevation, aspect and shape.

Check the resulting design flood estimates with reference to adjacent catchments, neighbouring stations and other return periods to ensure that results are sensible.

MINOR RECOMMENDATION 20

Collect relevant regional flood frequency contours and underlying data into a single figure to assist understanding.

9.1.4.2 PREDICTED FLOOD FLOWS USED FOR DESIGN EVENTS

FLOOD FREQUENCY VALUES

There were some discrepancies between the design flood flows given in the PDP executive summary and those in Appendix C of their report (PDP, 2013). This does not appear to have been checked with PDP. The major differences were in the 100 year flow for the Ruamāhanga at Gladstone and all the AEP flows for the Kopuaranga at Palmers. Checks by the reviewer, assuming that the PE3 frequency distribution parameters shown under the TIDEDA plots in Appendix C are correct, indicate that the value used by GW for the

Ruamāhanga at Gladstone is likely to be correct (or relatively close to), but the values used for the Kopuaranga are not. The values that agree best with the PE3 distribution parameters provided, are those in the PDP executive summary. The difference is relatively small, meaning the Kopuaranga design flows used in GW hydrology and modelling are 1-6% overestimated.

MINOR RECOMMENDATION 21

That the flows from the flood frequency distribution for Kopuaranga at Palmers be corrected.

WAINGAWA CATCHMENT

GW has previously estimated that the contribution from the Atiwhakatu River to the Waingawa was around 33% of the flow at the Kaituna gauge. This was checked by comparing historic flows at the Kaituna gauge with flows at the closed gauge Atiwhakatu at Mt Holdsworth over a coincident period of about 21 years. By comparing all flows above the mean annual flow at Kaituna, a flow increase of 33% from the Atiwhakatu River was confirmed (PDP, 2013). This seems a sound method of estimating flow for the ungauged catchment, and confirmed the previous GW value.

In the lower catchment hydraulic modelling report (GW, 2013), it is noted that design flood flows estimated from regional flood frequency contours for the Atiwhakatu River of the Waingawa catchment (downstream of the Kaituna gauge) were 66% of flows at the Kaituna gauge. This is twice as large as the at-site estimate from historic flow comparison.

The lower hydraulic model report explains that the Waingawa individual hydrology was used in preference over the flow estimates from the regional flood frequency contours. The reviewer agrees that this was a better approach, based on catchment specific data. Note that the design hydrographs used in the hydraulic model and shown in the report for the Atiwhakatu subcatchment (Trib1 of Waingawa) do not agree with a 33% scaling of the upstream flows (The presumed correct values are shown in Table 7, GW, 2013). This implies that Trib1 of the Waingawa has had a 50 year return period flow applied when the other subcatchments had a 100 year return period flow applied.

MINOR RECOMMENDATION 22

Hydrological boundary conditions used as input to the hydraulic models be checked for consistency and correctness.

9.1.4.3 CLIMATE CHANGE ALLOWANCES

For the general hydrology, a 20% increase in rainfall has been assumed as part of climate change impacts to the year 2100. Assuming an 8% increase in rainfall per degree of climate warming (a generally assumed increase in extreme rainfall per degree of atmospheric warming), this links to a 2.5°C increase in mean temperature. Based on the MfE guidance current in 2013, a mean temperature increase of 2.1°C was estimated for the Wellington region from 1990 to 2090 (MfE, 2008). The GW projected rainfall increase is consistent with the (then) current MfE guidelines.

For the Upper Ruamāhanga hydrologic catchments except the Waipoua, it has been assumed that 20% increase in rainfall will generate a 20% increase in river flow (GW, 2013). In Section 3.7 of the GW lower hydraulic model report, results from the Waiohine floodplain management plan are referenced. Investigations in this catchment showed a flow response of 17-27% to a 17% increase in rainfall. Assuming a linear relationship between rainfall and river flow increase is at the bottom end of this range. In the same section of the lower hydraulic model report, it was recommended that the flow response to increased rainfall be investigated during the next phase of the Upper Ruamāhanga floodplain management plan.

The river flow response to increased rainfall is often non-linear, and this can be explored by using rainfall-runoff models of the catchments.

MODERATE RECOMMENDATION 18

Review GW climate warming impact projections for increased extreme rainfall and sea level rise in light of new MfE guidance.

Investigate the response of Upper Ruamāhanga river flows to increased design rainfall, using rainfall-runoff modelling or similar. This would provide a stronger relationship between projected increases in climate warming and river flows.

9.2 WAIPOUA HYDROLOGY

9.2.1 WAIPOUA HYDROLOGY SOFTWARE

For the Waipoua, the adopted hydrology reported by MWH has used Kisters Time Studio to create a rainfall-runoff model, and Innovyse ICM (Integrated Catchment Modelling) to build a 2-D model of the lower Waipoua catchment. The reviewer does not know Time Studio, but understands from discussion with Tom Kerr, (who completed the Waipoua hydrological modelling for MWH) that it is a reasonable and flexible tool for catchment modelling. ICM is recognised industry standard software and an acceptable choice for 2-D modelling. The results seemed to indicate that the rain-on-grid option in the ICM model did not perform as well as the separate rainfall-runoff model, and the eventual runoff results employed the rainfall-runoff model hydrographs.

Design rainfalls were built using data from HIRDS v3 and rainfall contours produced by NIWA. This is acceptable practice.

9.2.2 WAIPOUA HYDROLOGICAL MODELLING METHODS

For the Waipoua hydrology, a rainfall- runoff model has been used to generate runoff for both historic and design events. Design events have used peak flows from the frequency distribution at Mikimiki, and design rainfall applied to the rainfall-runoff model to create hydrographs for the lower catchment.

A 2-D hydraulic model using rain-on-grid was trialled for the lower part of the Waipoua, but the results were discarded when the design flows seemed abnormally high (MWH, 2016).

Final hydrology used for flood modelling was a combination of flood frequency estimates, flow records for Mikimiki and rainfall-runoff modelling for the ungauged catchment.

9.2.2.1 CONTRIBUTION FROM UNGAUGED CATCHMENT

Design flow from the ungauged catchment has been treated as an increase in flow of the hydrograph at Mikimiki. This was assessed by MWH (2016) by comparing flows at the Mikimiki recorder with those at the Colombo Rd bridge, where a recorder was placed for just over a year. Limited by the comparison period being very short, this is otherwise a reasonable method. The results from this study could also have been compared to the ungauged catchment flow proportion found by the PDP hydrological modelling (2013) and to any gauged flow data, such as that for the 1998 flood (in MWH Tables 4-2 and 4-3, Rail bridge flow/ Mikimiki flow = 413/356 = 1.16, or a 16% flow increase). It is not clear if other sources were used to evaluate the lower Waipoua catchment flow contribution.

The comparative flow records at Mikimiki Bridge and Colombo Rd Bridge are replotted in Figure 9-4 from the MWH report to illustrate the following points.

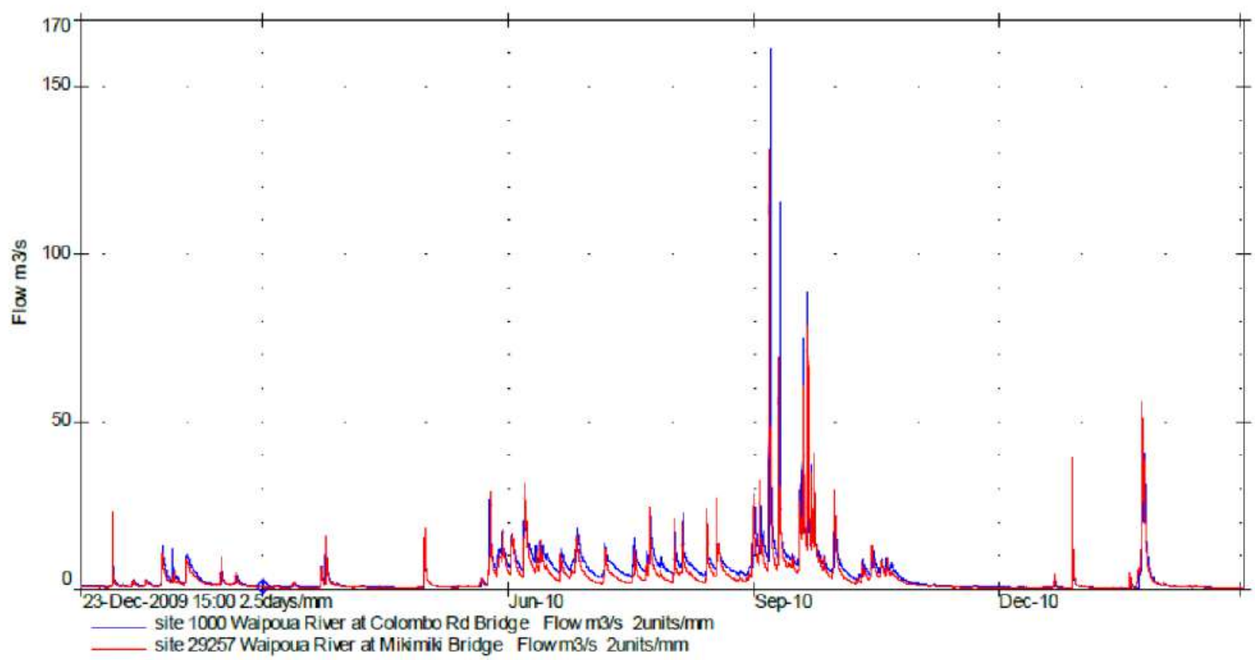


Figure 9-4 - Flow comparison, Mikimiki and Colombo Rd Bridge

Key points to recognise about this approach are:

- Comparison period available for the two gauges was just over 1 year.
- The 4 highest flow peaks used for the comparison all occurred in the same month: September 2010.
- The examined events had a maximum return period of 2 years. Findings have been extrapolated to higher magnitude events, up to 100 year ARI.
- The highest contribution from the lower catchment as a percentage of the peak flow at Mikimiki was 67% on September 10. This occurred just 4 days after the greatest fresh, on September 6. It is possible that the catchment had not fully drained from the previous event.
- In the figure displaying the flow data for both gauges, another event is visible in February 2011, with a peak flow at Mikimiki of 60m^3/s which is of a similar size to those events examined. The

flow at Columbo Rd appears to be 30% less than that at Mikimiki. This event was not mentioned in the MWH evaluation.

It would seem that very little data was used to draw these conclusions. Possibly the event with the 67% contribution should not carry as much weight as the other events, given its proximity to the previous flood. From the available data the average contribution might be between 13-25%. The 25% contribution decided by GW, MHW and T+T; the peer reviewer of that work, seems sensible.

Gathering more information on the lower catchment contribution would be very useful. The reviewer was encouraged to learn that the gauge at Colombo Rd bridge was reinstalled in September 2015, following recommendations from earlier work. More confidence in the flows at this point in the catchment would be of great assistance in flood planning.

GENERAL RECOMMENDATION 8

As soon as more data is available for the Colombo Rd site, use this to refine the estimate of flow contribution from the lower Waipoua catchment.

A longer record at Columbo Rd could also be used to provide calibration data downstream of the ungauged catchment area. This could aid in validation of the hydrological and hydraulic models, so that these tools become more accurate.

Ideally, the site should be telemetered to facilitate access to this data and gauged regularly to get a grasp on the site characteristics and stability.

9.2.2.2 RAINFALL-RUNOFF MODEL

Using a rainfall-runoff model to produce flows for the Waipoua catchment was a sensible approach. Sometimes a rainfall-runoff model on its own has only a fairly simple representation of channel routing, so combining the rainfall-runoff model with a hydraulic model to simulate the channel processes is also a good step (MWH, 2016). However, the initial and constant loss model chosen for the rainfall is too simple to represent soil infiltration processes during a storm, particularly historic storms with more than one peak. Only two other parameters are mentioned for the model: a channel storage parameter and non-linearity parameter. It would be difficult to represent catchment storage, overland processes and channel routing with just these two parameters, and it is not clear from the report how these two parameters work. Possibly there were also limitations with the representation of catchment processes of the rainfall-runoff model. From the calibration to historic events, it does seem that the model was concentrating flow too quickly and lacked storage.

The Waipoua has been divided into 13 subcatchments, which seems a high number, given that there was only a single flow gauge (there are now two) and a single rain gauge to use for calibration purposes. A simplified division of subcatchments would better reflect the scale of the available data.

After making a considerable effort to develop and calibrate these models, it appears that the hydraulic model was discarded, and the rainfall-runoff model output was manually scaled before use to achieve expected flows in the hydraulic model.

9.2.2.3 RAINFALL DEPTHS

The design rainfalls have been extracted from HIRDS v3 for the Westons site and then extended to the rest of the catchment. The rainfall depths have been related using 100 year-24 hour rainfall depth contours provided by NIWA. As the time of concentration for the catchment is closer to 6-8 hours, it is not clear whether this relationship for rainfall depths holds. It does, however, provide a way to estimate design rainfall for the Waipoua subcatchments.

Similarly, historic event rainfall depths from Westons gauge have been extended catchment wide, using the 100 year–24 hour contour relationship. This means that, ultimately, all the rainfall applied to the catchment stems from the Weston gauge (or Ruamāhanga at Mt Bruce if the Westons record does not cover the period in question). Because storms vary in space, and because Westons is in the upper catchment, this may not give a reliable picture of the historic rainfall pattern.

Point estimates of rainfall (including those from HIRDS) in standard practice have an Areal Reduction Factor (ARF) applied when they are used across a larger area. This point was raised in Section 7.7 of the T+T peer review (T+T, 2016), where it was pointed out that the effect of these for a 163km^2 catchment down to the Rail Bridge, would be to scale point rainfalls by a factor of 0.77 (2 hour duration) to 0.88 (24 hour duration). Between MWH and the T+T reviewer, it was decided not to apply ARF to point rainfalls for the Waipoua catchment modelling. This was because of the existing uncertainty in the underlying data. It is worth noting that applying ARF would have the effect of reducing rainfall depths across part of the Waipoua catchment by up to 15-20%.

MODERATE RECOMMENDATION 19

Reconsider whether Areal Reduction Factors should be applied to point rainfalls used in Waipoua catchment modelling, and update rainfall depths if required.

9.2.2.4 TEMPORAL DISTRIBUTION

Design rainfall hyetographs were formed based on a temporal distribution in a paper written by Tomlinson (1992). This was developed from accumulated rainfall for 17 annual maximum storms in the Wellington area. The form is heavily forward weighted (ie, most of the rainfall occurs in the first to middle parts of the storm). The reviewer has not seen this paper. The Wellington region covers a wide area. It might be more relevant to look at local gauges to develop representative storm temporal distributions. Alternatively, if reliable local data is not available, a more symmetric rainfall pattern could be used. This point was also raised by the previous peer reviewer of the MWH report, T+T.

MINOR RECOMMENDATION 24

Review local rain records to validate the temporal rainfall distribution used for the Waipoua catchment, or consider a symmetrical temporal distribution.

9.2.2.5 APPLICATION TO THE WAIPOUA FLEXIBLE MESH HYDRAULIC MODEL

Design hydrographs for the lower Waipoua catchment determined from the MWH study were halved to match the design target of Mikimiki flow + 25% at the Rail Bridge in the Waipoua Flexible Mesh hydraulic model (GW, 2018). This is an indication that the expected catchment response is not adequately reproduced in the flexible mesh modelling. Possible reasons for this are:

- Flows from the rainfall-runoff model are too high,
- relative timing of the design hydrographs from different subcatchments is not right,
- overland travel time is being short circuited in applying lower catchment hydrographs directly to the main channel,
- Storage and overland flow parameters in the 2d model need more tuning.

The 100 year design hydrographs are reproduced here in Figure 9-5.

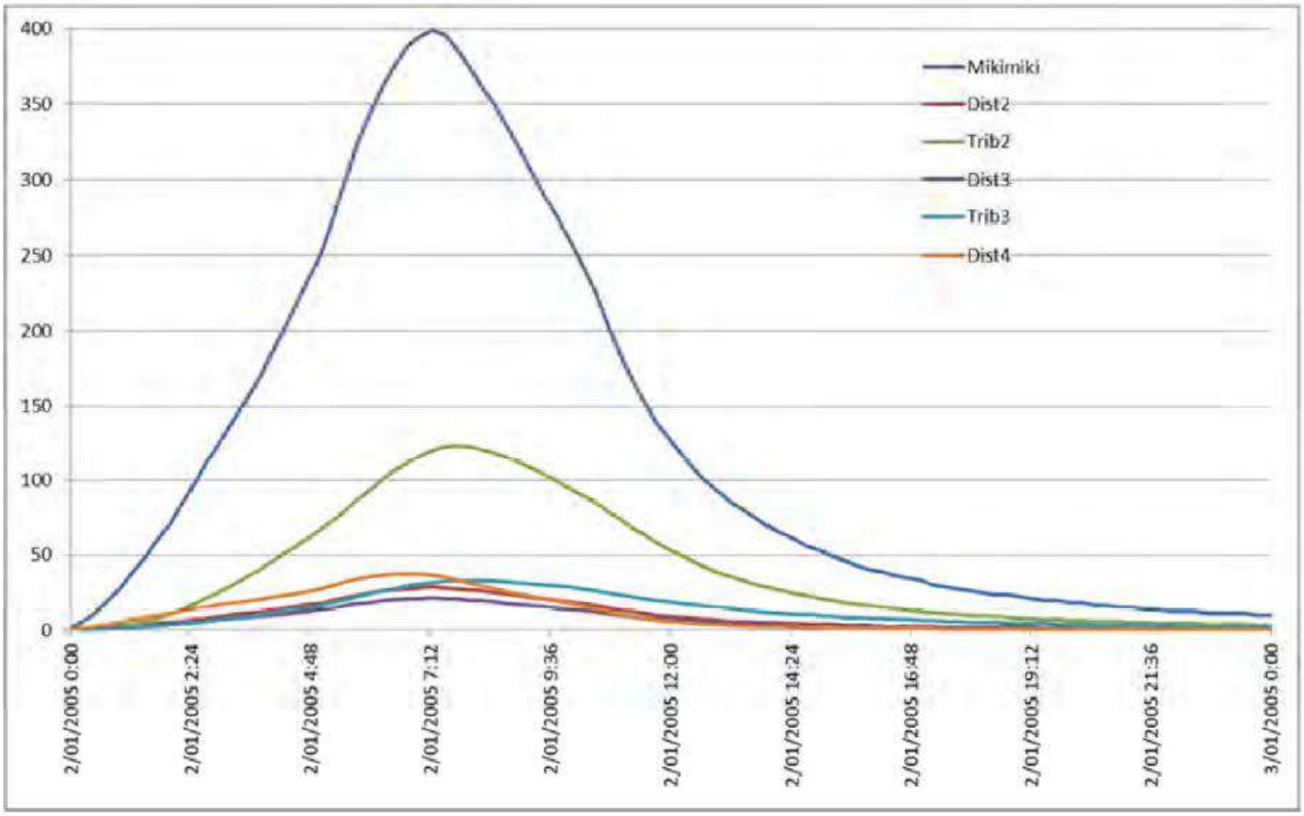


Figure 9-5 - MWH 100 year design hydrographs, Waipoua

It can be seen that the lower subcatchments, while having slightly different shapes, peak around the same time or within 2 hours of the flow at Mikimiki. Since the travel time between Mikimiki Bridge and Colombo Bridge is approximately 2 hours, the peaks from the different subcatchments probably coincide there. It seems surprising that the time to peak for an 80km2 mountain catchment is the same (within 2 hours) as that of several plains catchments with area of about 10km2. This may be a reason why the modelled flow at Colombo Bridge was initially too high. Having to make such broad adjustments to the applied hydrology is not good practice. It indicates that the design hydrology needs some refinement.

Historic events used the rainfall- runoff model hydrographs for both Mikimiki and the ungauged subcatchments. These flows were applied within, or at the boundary of the hydraulic model. The hydrograph at Mikimiki had a high first peak, which caused a high initial volume of water to fill the flood plain, and increased flood levels in the hydraulic model prematurely. The high initial flood peak was manually edited out of the hydrographs, which is not ideal.

Here, the rainfall-runoff model has not been able to duplicate the double peaked 1998 flood flow at Mikimiki, which is problematic. The usefulness and credibility of the rainfall-runoff model would be reinforced by better calibration to historic events.

MAJOR RECOMMENDATION 4

That the Waipoua rainfall-runoff model be revisited to provide a better representation of the physical catchment processes. Also ensure that the way the runoff hydrographs are applied in the hydraulic model allows for reasonable travel time and storage.

That after refinement of the Waipoua rainfall-runoff model, the model be validated against at least those historic events used previously (October 1998 and September 2010). Ideally, more than two calibration events should be used.

The aim is that hydrology linked to the Waipoua flood modelling be robust enough that it does not need to be adjusted before use in the hydraulic model.

9.2.3 WAIPOUA INPUT DATA

9.2.3.1 RAINFALL DATA

For the Waipoua, intensity rainfall data was used to feed the rainfall-runoff model to generate runoff hydrographs. The chief rain gauge used for this was Waipoua at Westons, in the upper Waipoua catchment. This gauge has been operating since November 2007. The MWH report also makes mention of Waipoua at Mikimiki and Waipoua at Wairarapa College gauges within the catchment and Mangatarere at Valley Hill and Ruamāhanga at Mt Bruce at high elevation in neighbouring catchments (MWH, 2016). The gauge locations, taken from the MWH report, are shown in Figure 9-6.

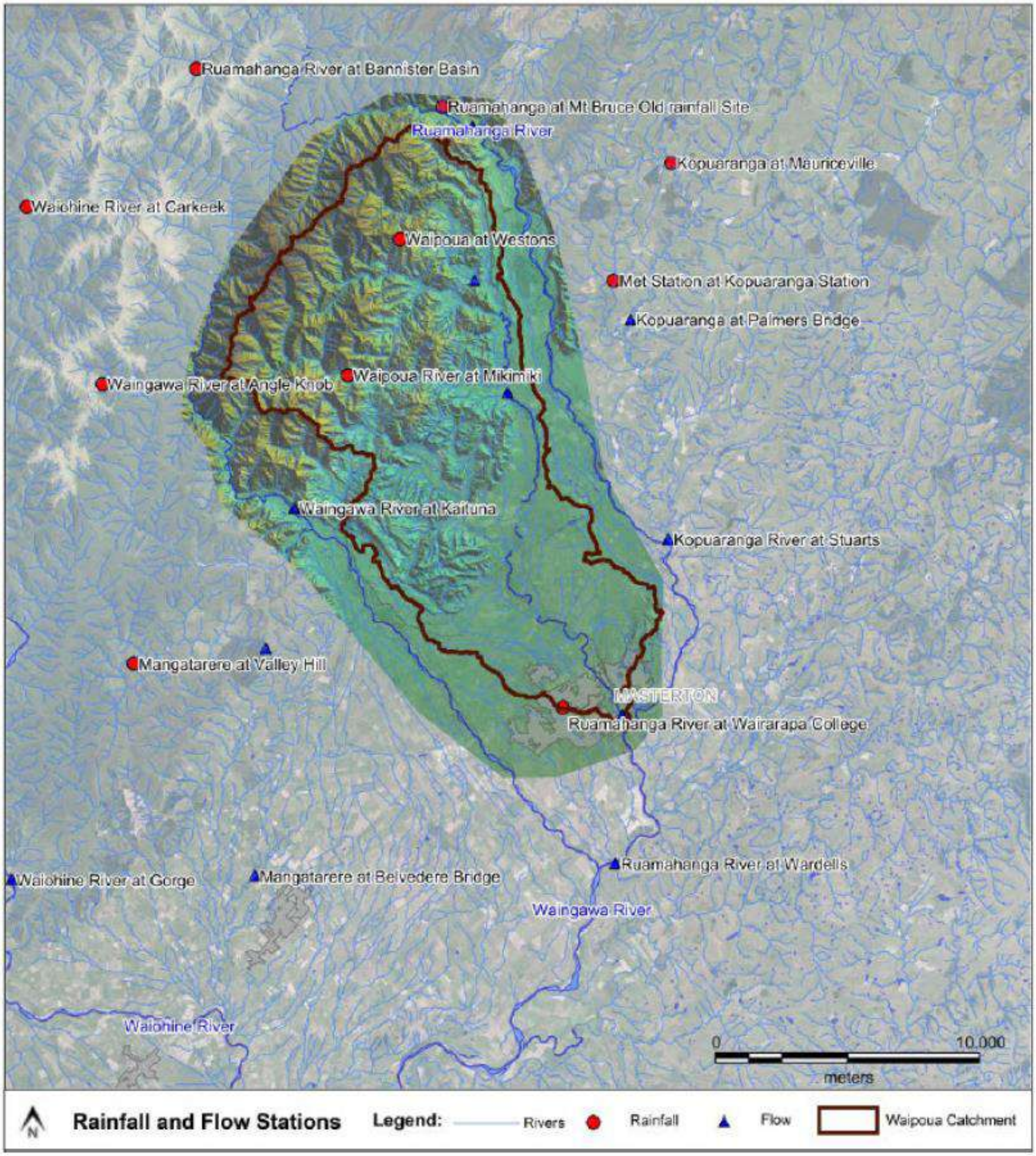


Figure 9-6 - Waipoua area rainfall and flow stations

A number of these gauges were established quite recently, and were not used in simulating historic events. It seems that the rainfall for the catchment model came mainly from the Westons gauge, and for the 1998 event, from the Mt Bruce gauge, scaled by the difference in average storm depth between the two stations. Rainfall from the Westons gauge is likely to be fairly representative for the upper catchment, and less so for the lower catchment. The lack of historic gauges local to Waipoua, particularly on the lower land, restricts the effectiveness of the rainfall runoff modelling. In the future, the spread of available gauges should make it easier to understand rainfall patterns across the catchment and model events. It is not clear if more use was made of other gauges, but it appears that the best was done with the available rainfall data at the time.

9.2.3.2 1947 FLOOD RAINFALL

In discussing the 1947 flood event, NIWA (2015) compared rainfall during that event and during the 1998 event. This could only be done for two daily rain gauges which had been operating at both times. These gauges were Bagshot in the Whangaehu Valley, about 12km ENE, and Te Ore Ore flat, about 6km ESE of the Waipoua catchment. The 1947 daily rainfall depths at these gauges were 1.5-3 times as great as those recorded in 1998. However, the gauges are in different catchments and some distance away. The Waipoua generates the greatest concentration of runoff in about 6 hours to the Mikimiki gauge and 12 hours or less to the Rail Bridge, so a daily rainfall total without more detail does not mean that there was rainfall intense enough to cause significant flooding. While daily rainfall totals provide a useful check against sub daily records, this is not enough information to say anything definite about the magnitude of the 1947 storm in the Waipoua.

9.2.4 WAIPOUA ASSUMPTIONS

9.2.4.1 1998 FLOOD ESTIMATE

The gauge site at Mikimiki is steep and unstable. It scours during floods, and the highest gauging flow was 203m^3/s. This means that all high flows are extrapolated from the rating curve. The flood in October 1998 was the biggest in recent times. Various estimates of the peak have been made. The GW rating estimate gave the flood flow as 356m^3/s. NIWA, in their in-depth study of the Mikimiki record, and this flood in particular estimated the flood flow as 400m^3/s (NIWA, 2015). This assumed some bed scour and a flow velocity of 5m/s. A review of this information by Brin Williman pointed out that the estimate is sensitive to the chosen velocity, and Froude number. From experience on a similar river, he used a velocity of 4.2m/s and a Froude number of 0.7 to give a flow of 336m^3/s. This reviewer has not investigated the rating in detail. It is clear that estimating high flows at Mikimiki carries considerable uncertainty. However, a velocity of 5m/s seems very high for a channel that would scour at these high flows, thus increasing the channel area and reducing the velocity and Froude number. Hydraulic model results from this reach in the 100 year ARI simulations, for example, find a mean channel velocity of 3-4m/s. The 1998 flood peak is very likely to lie within the range of flows suggested, but without a detailed look at the rating and related data it would be hard to confirm. The flood frequency distribution is not greatly affected by a change in the 1998 flood. The key issue is that a different flow affects the return period of the 1998 flood, and hence its standing relative to the design floods.

9.2.4.2 WAIPOUA CLIMATE CHANGE ALLOWANCES

For the Waipoua, a 20% rainfall increase to the year 2100 was also used, which is consistent with the wider catchment. A rainfall runoff model for the Waipoua catchment was developed, and design rainfalls were scaled up by 20% before applying them to the model. Although there were some shortcomings with the model, it demonstrated that the 20% increase in rainfall representing climate change produced a 25% increase in river flows. This is a reasonable approach to modelling climate change impacts.

Since this work was done in 2013, the MfE has produced updated climate guidance, and the rainfall increase due to climate change should be revisited. In particular the MfE recommend considering a range of scenarios based on stronger or weaker climate warming, which can be used to provide an envelope of likely futures.

9.2.5 COMMUNITY CONCERNS

The community has raised some questions as part of this investigation. Those that relate to the hydrology are addressed here.

9.2.5.1 INCLUSION OF 1947 EVENT IN DESIGN HYDROLOGY

Relating to the Waipoua hydrology the community has asked whether the available data for the 1947 event is suitable to be incorporated into the design hydrology.

Information provided to the reviewer relating to this particular event was:

Parts of the NIWA report on the 1998 flood
a memo from Opus to GW detailing investigations in the archives, and copies of historic Waipoua long sections through Masterton township
Further information was collected by talking to GW and MDC staff who work or have worked in the areas of river flows and records.

Key points are:

- 1947 flood appeared to have a slightly lower flood peak level at the SH2 bridge in Masterton
- Bed level in the Waipoua in this reach is degrading and was estimated to be 1.6m higher at the time of the 1998 flood than in 1947.
- If the flood levels deduced for these two events and the change in bed level are accurate then it is possible that the 1998 flood was larger than the 1947 event.
- Anecdotal evidence indicates that the 1947 flood was not a major flood at Masterton, although it was a significant flood on the lower Ruamāhanga.
- The information provided to the reviewer is not enough to reliably estimate the flow during the 1947 flood.
- The information so far indicates that the 1947 flood may, or may not have been significant. It may, or may not have been, greater than the 1998 flood.
- It appears that more information and clues on the 1947 flood might be available.

Increasing the number of years in the record back to 1939 when the stopbanks were built, (or to 1897 as done by NIWA [2015]) and using a plotting position like the Gringorton formula does not provide a realistic return period for a flood event from a series, because it depends on rank alone. It assumes that the data can be represented by an EV1 distribution and also depends on the heavy assumption that no other significant floods have occurred in the intervening period.

For example, if the record is extended back to 1939 then we have a period of about 78 years. If we assume that the 1998 flood was the largest flood during that time, the Gringorton formula gives the first ranked flood a return period of 140 years. But if the 1998 flood was second in that rank, then it is assigned a return period of 50 years. Neither indicates a true return period for this event and they are misleading when used as such.

Extending the flood record back to include large, historic storms is possible and could improve confidence in the flood frequency distribution. If this is done, there must be more certainty about the ranking of events. If a significant event has been missed out, or an event is ranked highly without firm evidence, the distribution will be affected.

MODERATE RECOMMENDATION 20

Including historic flood events could improve confidence in the flood frequency distribution for the Waipoua at Mikimiki. An in-depth investigation of historic sources would be needed to turn up any useful information. This investigation should not be limited to the 1947 event.

9.2.5.2 CONTRIBUTION FROM LOWER WAIPOUA CATCHMENT

The community has questioned the value used for the flow contribution from the lower Waipoua catchment. The method for determining this, and its limitations, are discussed in detail in Section 2.2.6. The main finding is that this contribution has been difficult to confirm due to lack of measured data. Sensitivity tests with the Waipoua hydraulic model compared the effect of changing the lower Waipoua contribution for a 100-year ARI flood event with climate change (GW, 2018). Reducing the lower catchment contribution from 25% to 15% resulted in a maximum reduction in river levels of 0.1m and minimal change on average.

Increasing the lower catchment contribution from 25% to 45% resulted in a maximum increase in river levels of 0.1m and an average increase of 0.1m through the urban reach. This implies that peak flood levels for large events are not that sensitive to the lower catchment contribution and are mainly driven by the flow at Mikimiki. Therefore, setting the lower catchment contribution at 25% is reasonable and this parameter does not appear as critical to extreme flood planning as the flow at Mikimiki.

10. REFERENCES

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APPENDIX A – RFQ DOCUMENT

Request for Quote

TKURR – Independent Model Audit

RFQ released: 06 03 19
Deadline for Quotes: 1200 15 03 19

Shed 39, 2 Fryatt Quay
Pipitea
Wellington 6011

SECTION 1: Key information

1.1 Context

a) This Request for Quote (RFQ) is an invitation to suitably qualified suppliers to submit a Quote for the TKURR – Independent Model Audit contract opportunity.

1.2 Our timeline

a) Here is our timeline for this RFQ.

Deadline for Quotes:	**1200**	**15/03/19**
Anticipated Contract start date:		20/03/19

b) All dates and times are dates and times in New Zealand.

1.3 How to contact us

a) All enquiries must be directed to our Point of Contact. We will manage all external communications through this Point of Contact.

b) **Our Point of Contact**

Name: Francie Morrow – Project manager

Email address: Francie.Morrow@gw.govt.nz

1.4 Developing and submitting your Quote

a) For helpful hints on tendering and access to a supplier resource centre go to: [www.procurement.govt.nz / for suppliers](www.procurement.govt.nz).

1.5 Address for submitting your Quote

a) Quotes must be submitted by email to the following address:

Andy.Brown@gw.govt.nz cc'd to Francie.Morrow@gw.govt.nz

b) Quotes sent by post or fax, or hard copy delivered to our office, will not be accepted.

1.6 Our RFQ Process, Terms and Conditions

a) **Offer Validity Period**: In submitting a Quote the Respondent agrees that their Quote will remain open for acceptance by the Buyer for 1 calendar months from the Deadline for Quotes.

b) The RFQ is subject to the RFQ Process, Terms and Conditions (shortened to RFQ-Terms) available at www.procurement.govt/for agencies The RFQ-Terms are incorporated into this RFQ by reference. We have not made any variation to the RFQ-Terms.

SECTION 2: Our Requirements

What we require

We require an Independent Model Audit for the Te Kauru FMP project.

Purpose of the audit

It is Council policy to carry out an independent review of all flood hazard modelling. This approach is taken to improve the robustness of the information and to give confidence to decision makers and the public that the information is fit for purpose. This audit is to contain a review of the hydrology, hydraulic model, application of freeboard and flood mapping.

Terms of reference for audit

The audit will assess whether the model and its outputs are fit for purpose. The main purposes of the model are:

- Catchment-scale flood hazard assessment, including classification of hazard into different categories, for the Waipoua River and other rivers in the FMP study area.
- Development and conceptual design of different flood management options (including being used for analysis of potential flood damages)
- Providing information for use by District Councils in LIMs, land use controls and for building controls (Local Government Act, Resource Management Act and Building Act requirements)
- To provide potential update to existing flood hazard information in the District Plan

The audit will comment on the appropriateness and fitness for purpose of the following criteria. [Note: we will give consultants the opportunity to suggest additional criteria in their proposals if they feel that these are necessary].

General

The following are *general* assessment items to be included in the audit;

- The type of software and modelling package used for the hydrology and hydraulic model
- The modelling method used and its appropriateness for both hydrology and the hydraulic model
- The use of freeboard and method by which it was applied
- Representation of the flood hazard through the way in which maps are displayed and information provided.

Input data

The assessment of the *data* used to create the flood model shall include;

- Rainfall data
- Measured flood flows
- Cross section surveys
- Lidar surveys
- Representation of any structures

- Calibration data against historical events.

Assumptions

The assessment of *assumptions* used to create the flood maps include:

- Run-off coefficients or similar hydrological parameters
- Predicted flood flows used for design events
- Climate change allowances
- Roughness coefficients of the channel and floodplain
- How the buildings and structures on the floodplain are treated through use of roughness coefficients
- Treatment of bridges, culverts and pipe crossings
- Use of freeboard to define flood hazard
- How the freeboard has been applied to the model and suitability of the freeboard values used.

Community Concerns

The *community concerns* to be assessed in the audit include:

- The sensitivity of the model results to the weir opposite the fire station
- The setup of the model in the vicinity of the Rail Bridge, including how blockage of the bridge has been represented
- The potential impact of long-term bed level trends on model calibration
- Whether the available data for the 1947 event is suitable to be incorporated into the design hydrology.

Information provided by GWRC

The following information will be made available by GWRC

Reference	Title
1202577	Draft hydrology report for UWVFMP from PDP (April 2013)
1215096	NIWA review on PDP Hydrology Report (May 2013)
1301892	PMF report by Laura Keenan (July 2013)
1236270	NIWA review of PMF memo produced by Laura Keenan (July 2013)
1474002	NIWA report on the 1998 event (March 2015)
1487993	Brin Williman review of NIWA hydrology report (8 May 2015)
FMGT-8-849	Final Waipoua Rainfall Runoff Modelling report – MWH (August 2016)
FMGT-8-1871	T+T peer review of MWH hydrology report (April 2016)
	1947 flood level – supplementary information and comments.
FMGT-8-1873	Information on the 1947 flood (August 2018)
FMGT-8-1945	Historical information about the Waipoua River (September 2018)

	TKURFMP Rectangular Grid Upper Model Hydraulic Modelling Report
	TKURFMP Rectangular Grid Lower Model Hydraulic Modelling Report
	Rectangular grid hydraulic model peer review
	TKURFMP Waipoua Flexible Mesh Hydraulic Modelling Report
	Flexible mesh hydraulic model initial review
	Sensitivity test modelling memo
	Flexible mesh hydraulic model final review
	Ruamahanga/Waipoua confluence modelling memo

Any additional information may be requested during the audit.

Deliverables

The key Deliverable is a **Single Volume Audit Report.** This should contain:

- Executive summary including comment about whether the output flood maps and the process by which these were derived makes them fit for purpose;
- An assessment against the key bullet points in section 2 above and any other information deemed relevant to the outputs.
- A summary explanation of any issue which is deemed as being not fit for purpose and what remedial work would be required to make this fit for purpose and deliver a positive audit result.

SECTION 3: Our Evaluation Approach

3.1 Evaluation model

The evaluation model that will be used is lowest price conforming. This means that all Quotes that are capable of full delivery on time will be shortlisted. The shortlisted Quote that is the lowest price over whole-of-life will likely be selected as the Successful Respondent.

The Buyer reserves the right to undertake due diligence and use the results of due diligence to inform the evaluation of Quotes.

3.2 Pre-conditions

Each Quote must meet all these pre-conditions.

#	Pre-condition
1.	**Independence –** Supplier must meet the definition of independence We have defined "independent" as being independent of the previous work done on the Te Kauru Project, ie. consultants who have not been previously involved in the hydrology, hydraulic modelling or peer review of either. A stricter definition of "independent" (ie. have never worked for GWRC or are unlikely to work for GWRC in the future on other projects) is unworkable and unnecessary.

SECTION 4: Pricing information

4.1 [Pricing information to be provided by Respondents

In submitting the Price the Respondent must meet the following:

a) Respondents are to use a pricing schedule.

b) The pricing schedule must show a breakdown of all costs, fees, expenses and charges associated with the full delivery of the Requirements over the whole of the life of the contract. It must also clearly state the total contract price exclusive of GST.

c) Where the price, or part of the price, is based on fee rates, all rates must be specified, either hourly or daily or both as required.

d) In preparing their Quote Respondents are to consider all risks, contingencies and other circumstances relating to the delivery of the Requirements and include adequate provision in the Quote and pricing information to manage such risks and contingencies.

e) Respondents are to document in their Quote all assumptions and qualifications made about the delivery of the Requirements, including in the financial pricing information. Any assumption that the Buyer or a third party will incur cost related to the delivery of the Requirements must be stated, and the cost estimated, if possible.

f) Prices should be tendered in NZ$. Unless otherwise agreed, the Buyer will arrange contractual payments in NZ$.

SECTION 5: Our Proposed Contract

5.1 Proposed contract

The following is the Proposed Contract that we intend to use for the purchase and delivery of the Requirements.

In submitting your Quote you must let us know if you wish to question and/or negotiate any of the terms or conditions in the Proposed Contract, or wish to negotiate new terms and/or conditions. The Response Form contains a section for you to state your position. If you do not state your position you will be deemed to have accepted the terms and conditions in the contract in full.

APPENDIX B – EXAMPLE FLOOD MAPS USED FOR ASSESSMENT

Waipoua River: DRAFT 1% Annual Chance Base Flood Spread

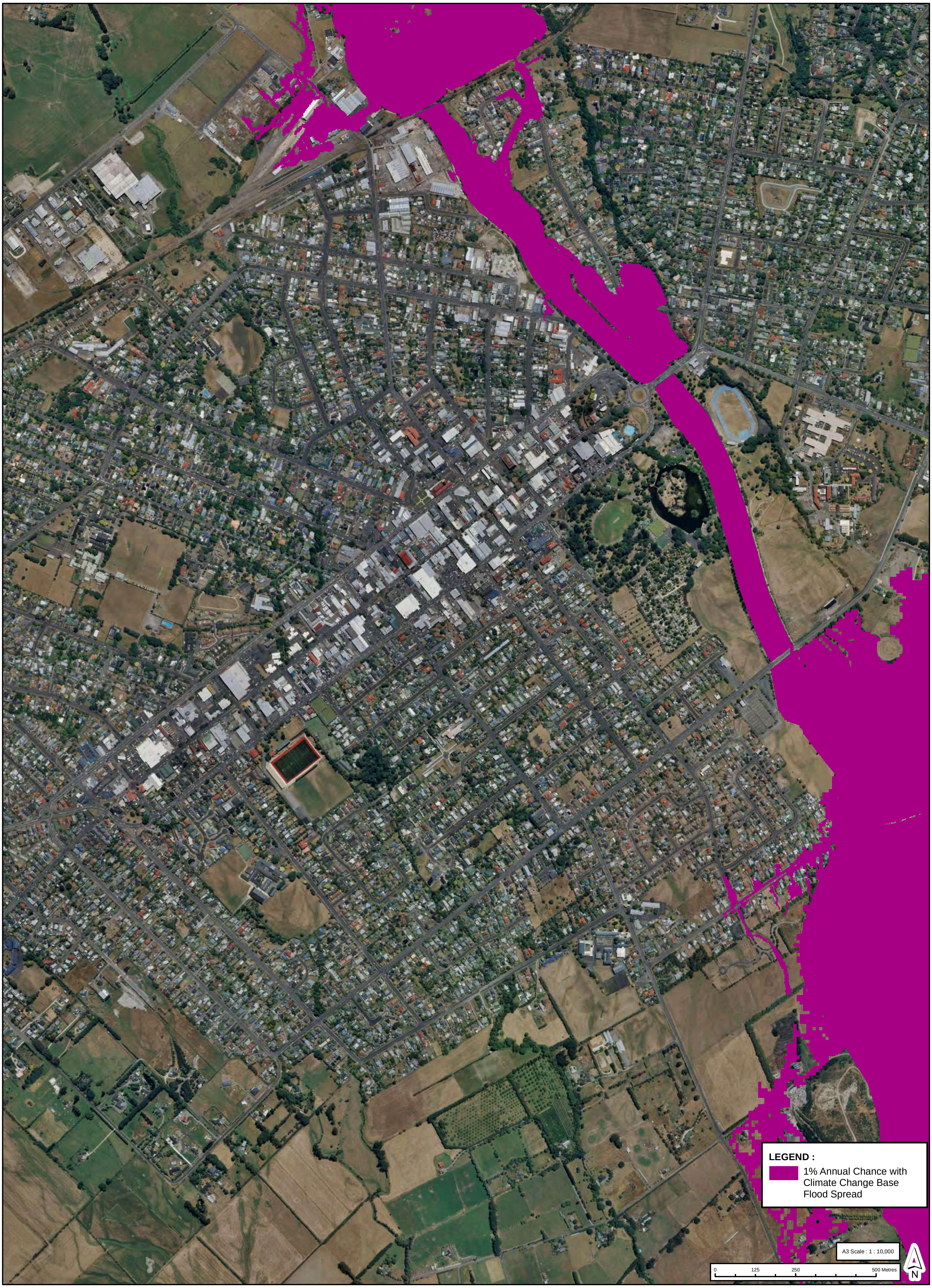
AERIALS FLOWN 2017 GWRC
Drawn : S Borrer, 30 January 2019
File Ref : A3 SIMPLE

Waipoua River: DRAFT 1% Annual Chance Flood Spread with Sensitivity Scenarios

AERIALS FLOWN 2017 GWRC
Drawn : S Borrer, 30 January 2019
File Ref : A3 SIMPLE

Waipoua River: DRAFT 1% Annual Chance Flood Spread with Sensitivity Scenarios and Base Flood Spread

AERIALS FLOWN 2017 GWRC
Drawn : S Borrer, 30 January 2019
File Ref : A3 SIMPLE

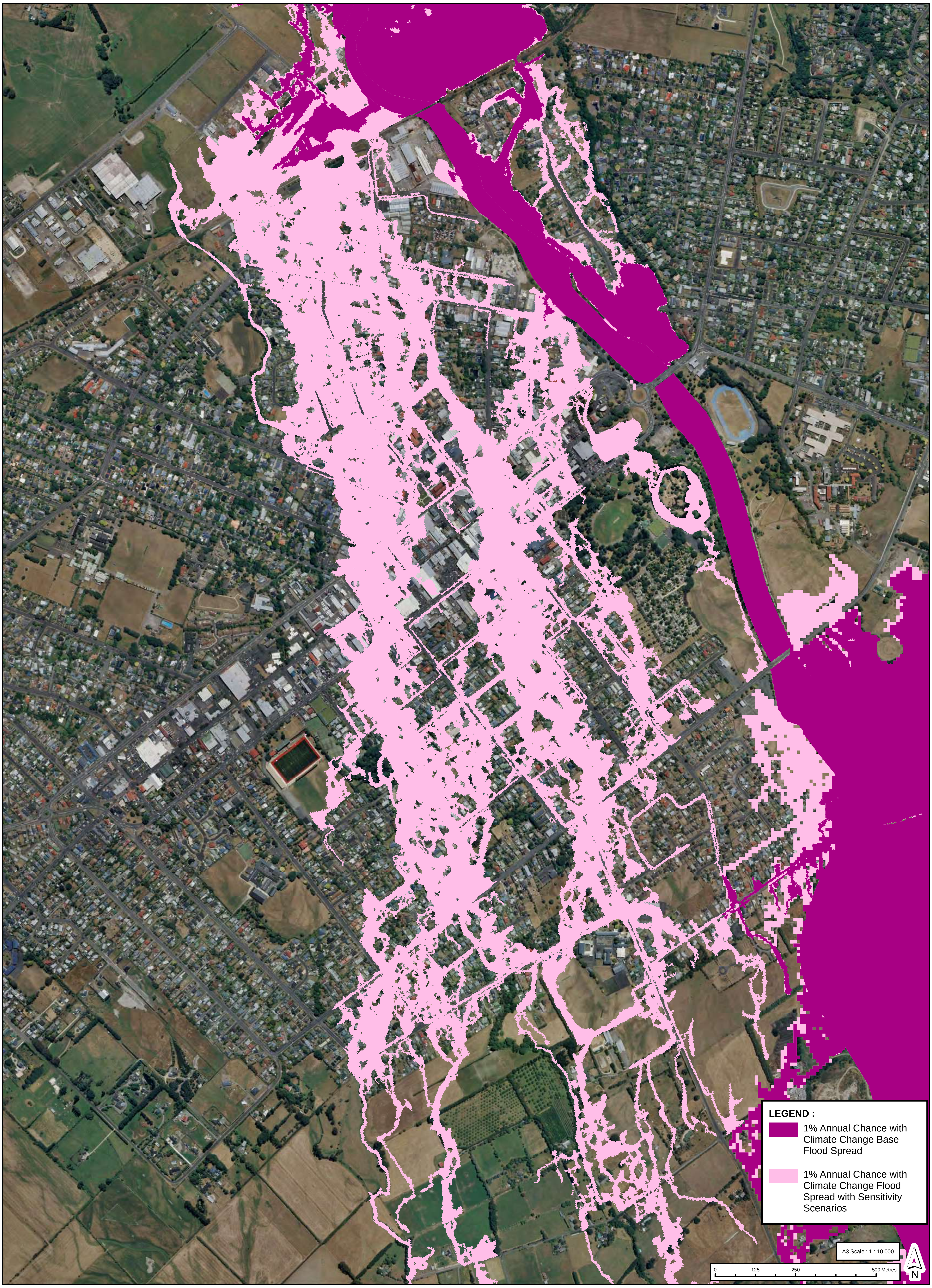


Waipoua River: DRAFT Future 1% Annual Chance Base Flood Spread

AERIALS FLOWN 2017 GWRC
Drawn : S Borrer, 30 January 2019
File Ref : A3 SIMPLE

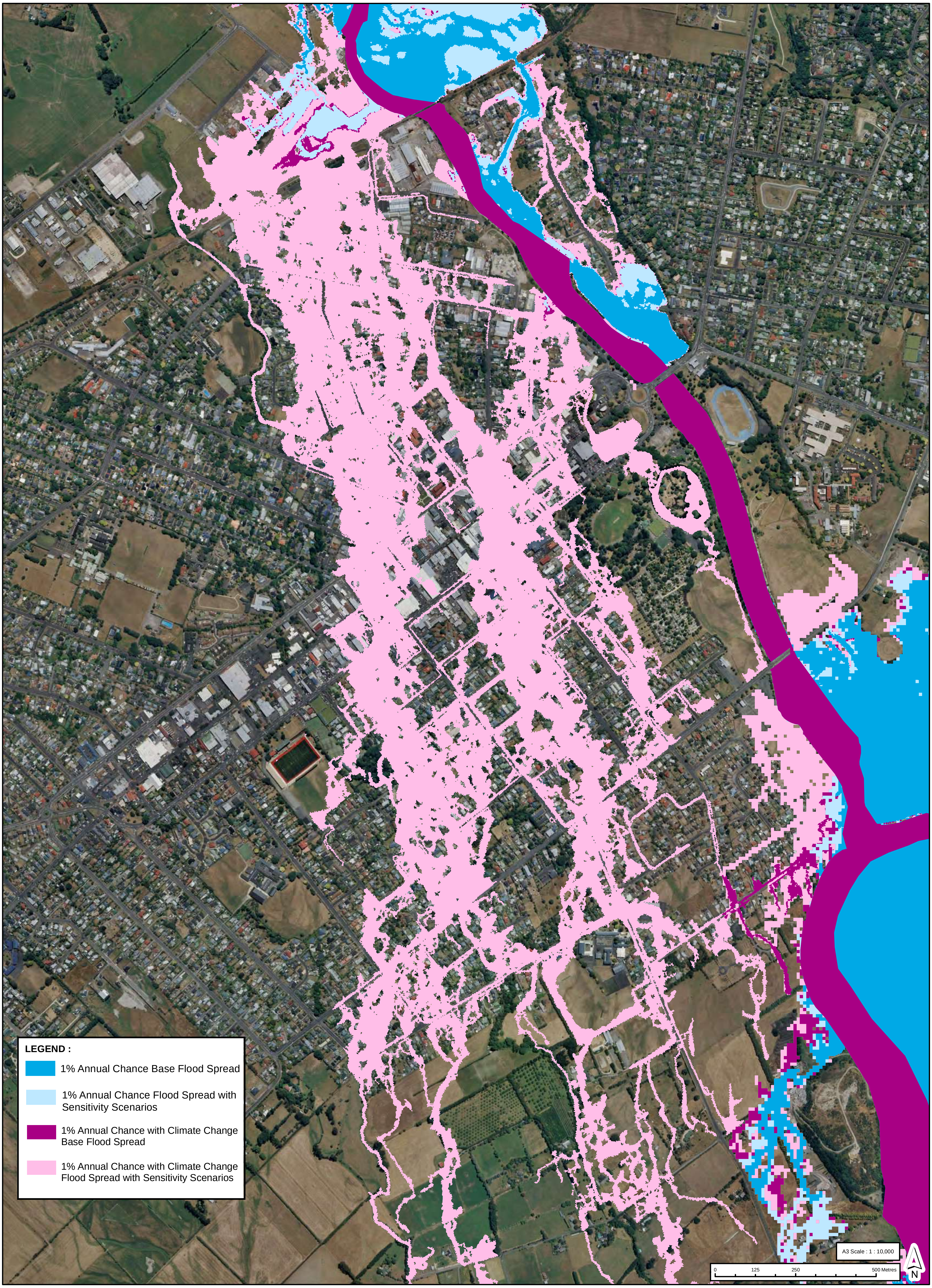
Waipoua River: DRAFT Future 1% Annual Chance with Sensitivity Scenarios Flood Spread

AERIALS FLOWN 2017 GWRC
Drawn : S Borrer, 30 January 2019
File Ref : A3 SIMPLE



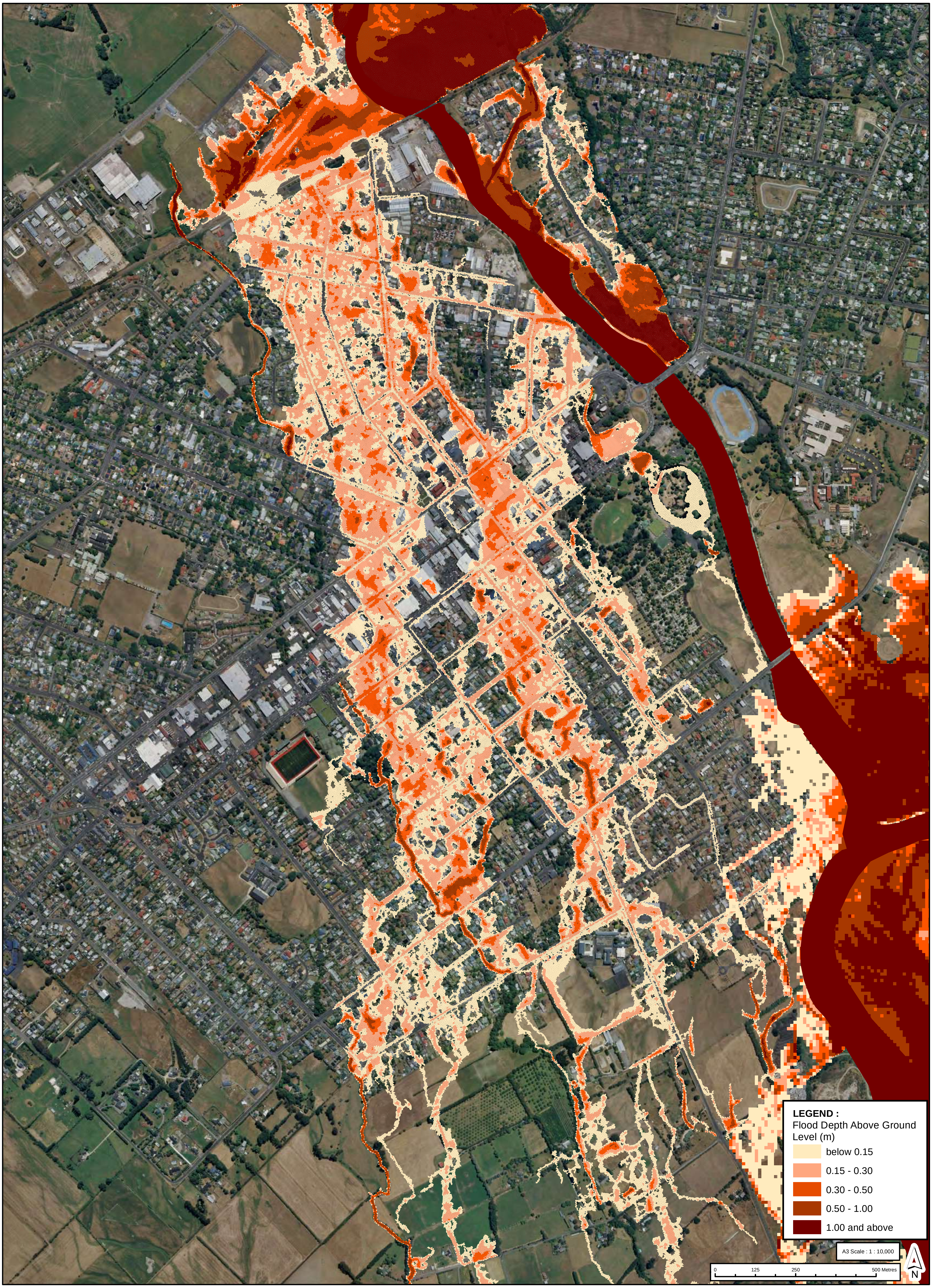
Waipoua River: DRAFT Future 1% Annual Chance with Sensitivity Scenarios and Base Flood Spreads

AERIALS FLOWN 2017 GWRC
Drawn : S Borrer, 30 January 2019
File Ref : A3 SIMPLE



Waipoua River: DRAFT Future 1% Annual Chance with Sensitivity Scenarios and Base Flood Spreads

AERIALS FLOWN 2017 GWRC
Drawn : S Borrer, 30 January 2019
File Ref : A3 SIMPLE



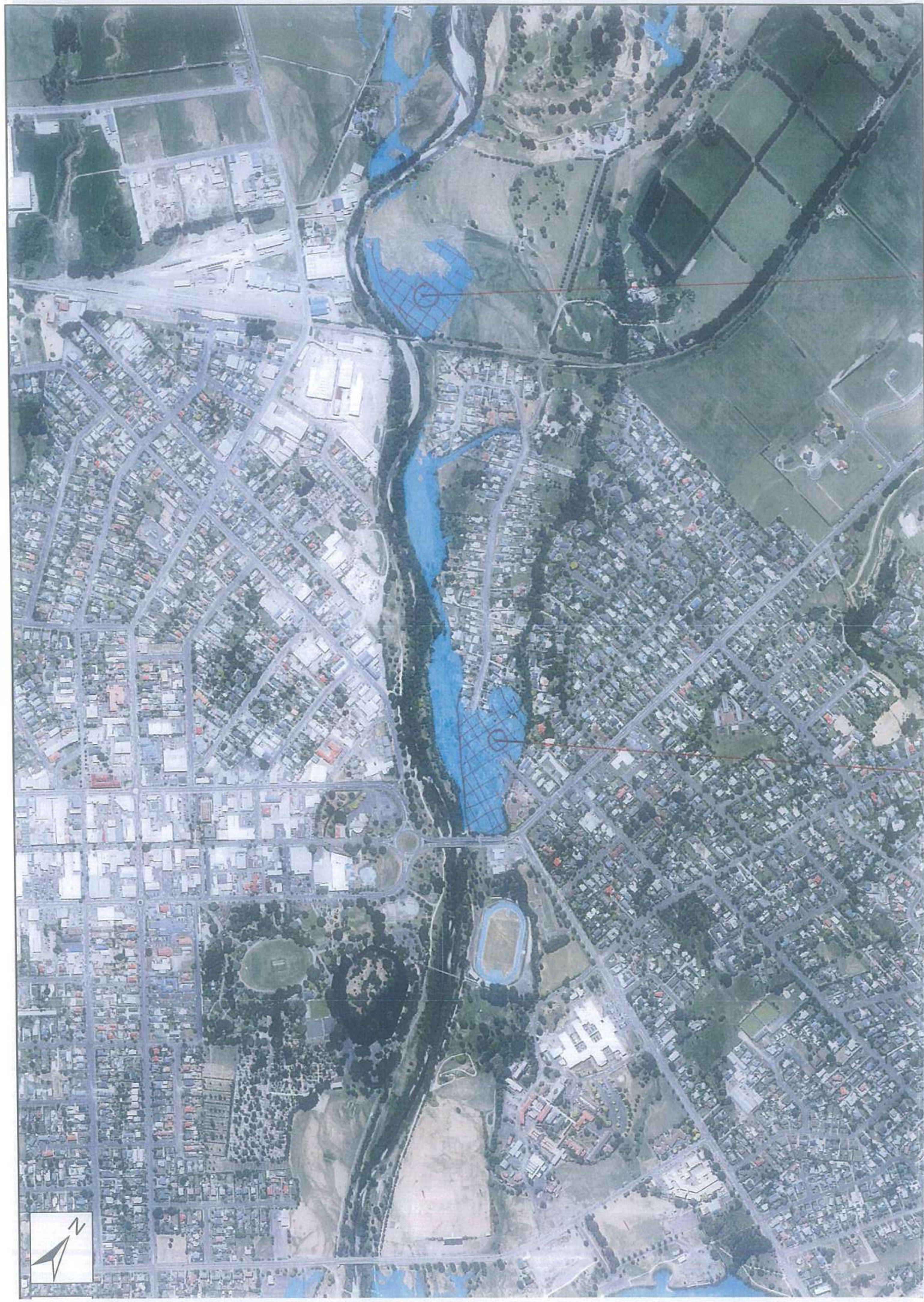
Waipoua River: DRAFT Future 1% Annual Chance Flood Depth with Sensitivity Scenarios

AERIALS FLOWN 2017 GWRC
Drawn : S Borrer, 30 January 2019
File Ref : A3 SIMPLE

Waipoua River: 2014 Flood Spread and 2019 Flood Spread

AERIALS FLOWN 2017 GWRC
Drawn : S Borrer, 30 January 2019
File Ref : A3 SIMPLE

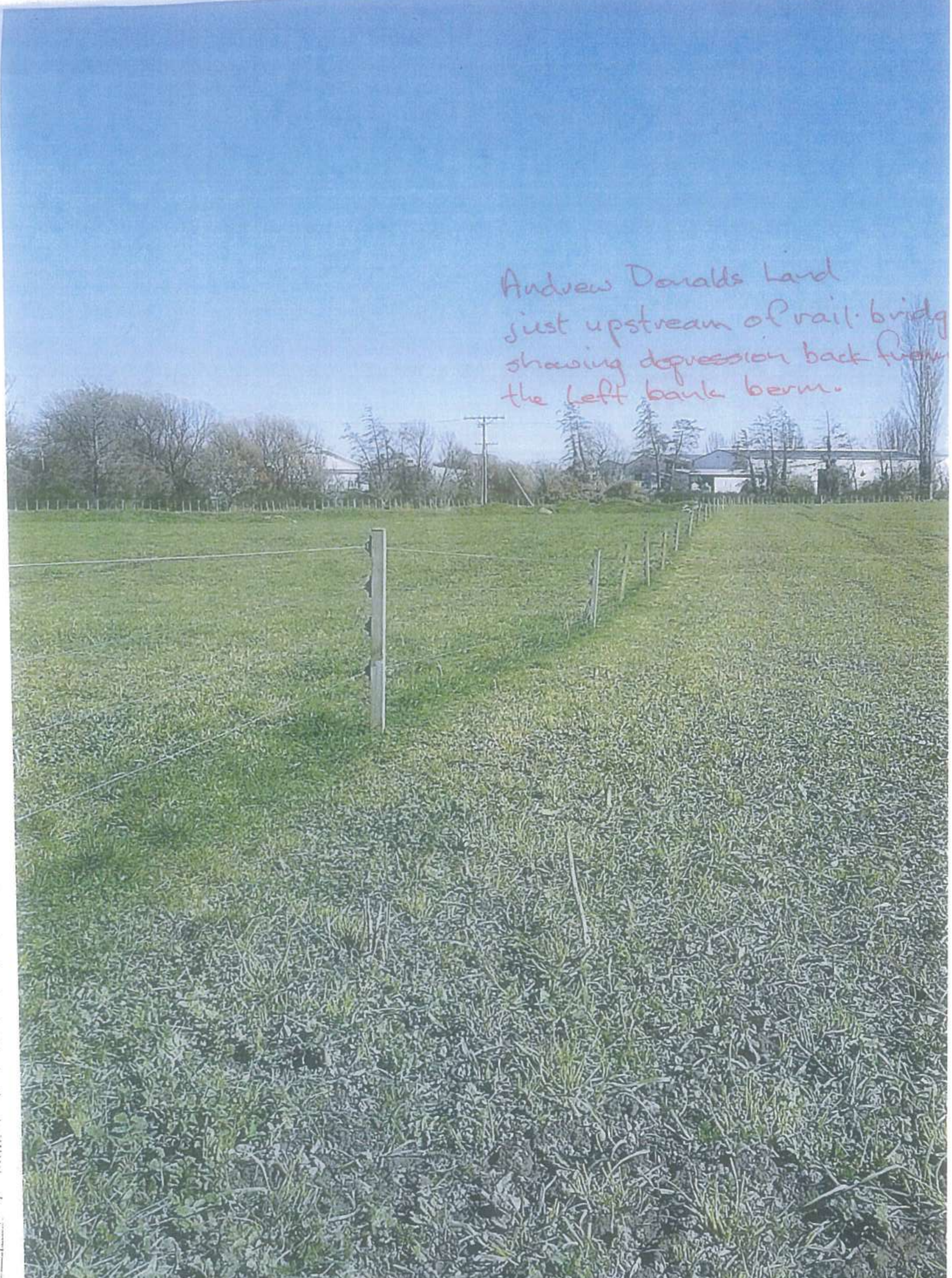
APPENDIX C – FEEDBACK FROM MDC ON 1998 CALIBRATION (WAIPOUA RIVER)

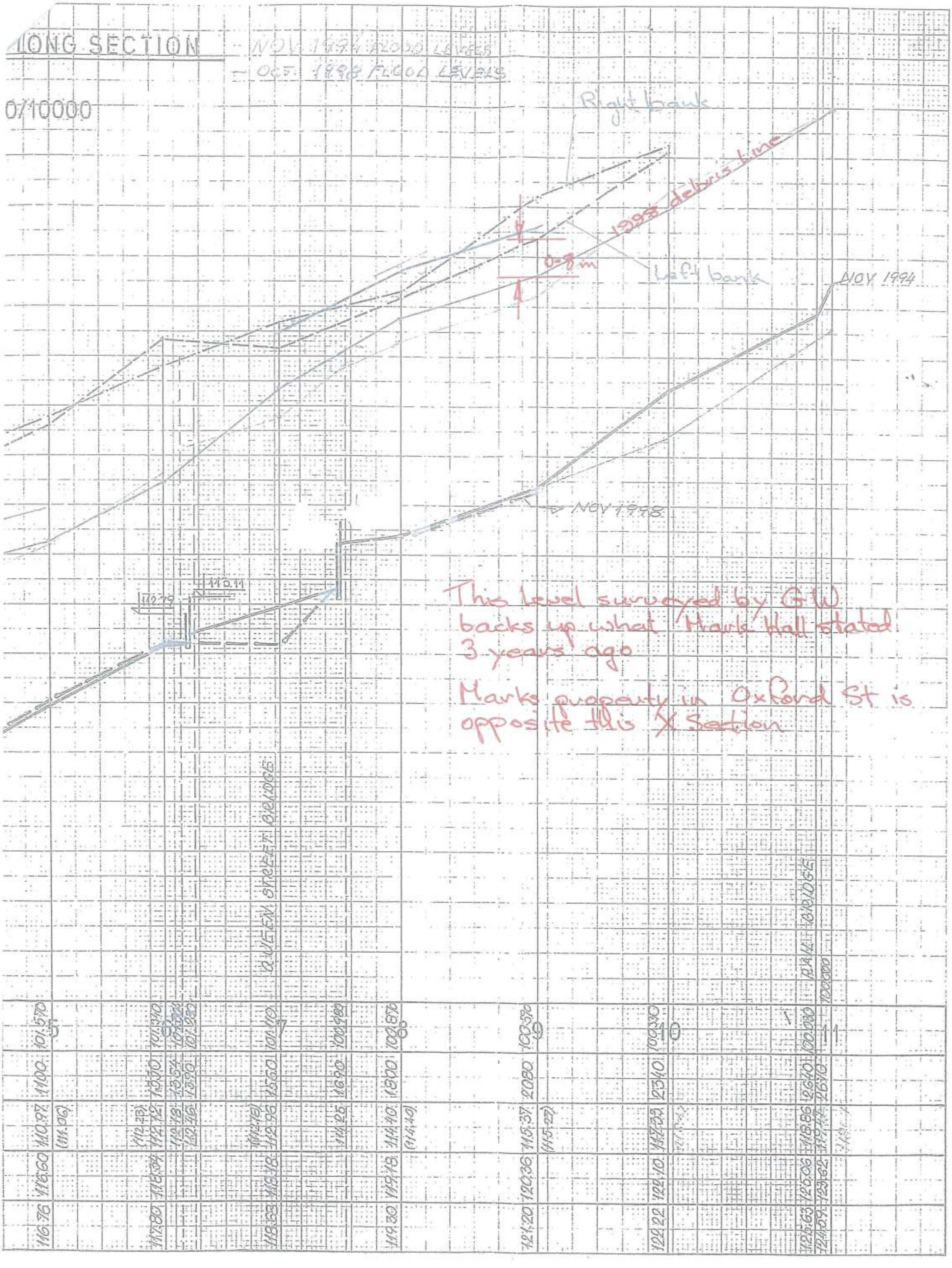


Land owner questions this level of inundation
However if the level came over the bottom wires of the fence along the river bank a small amount of water would pond in the depression shown in the photo attached.
The Land owner said there was debris on the bottom wires of the fence but no ponding.
If anything the river level as modelled is slightly high at this point and over tops the 'sill' provided by the berm.
1 span of the rail bridge was blocked with debris

flooding here caused by a cut in stopbank for storm water pipe.
This would not have happened if the pipe had been back filled at the time.

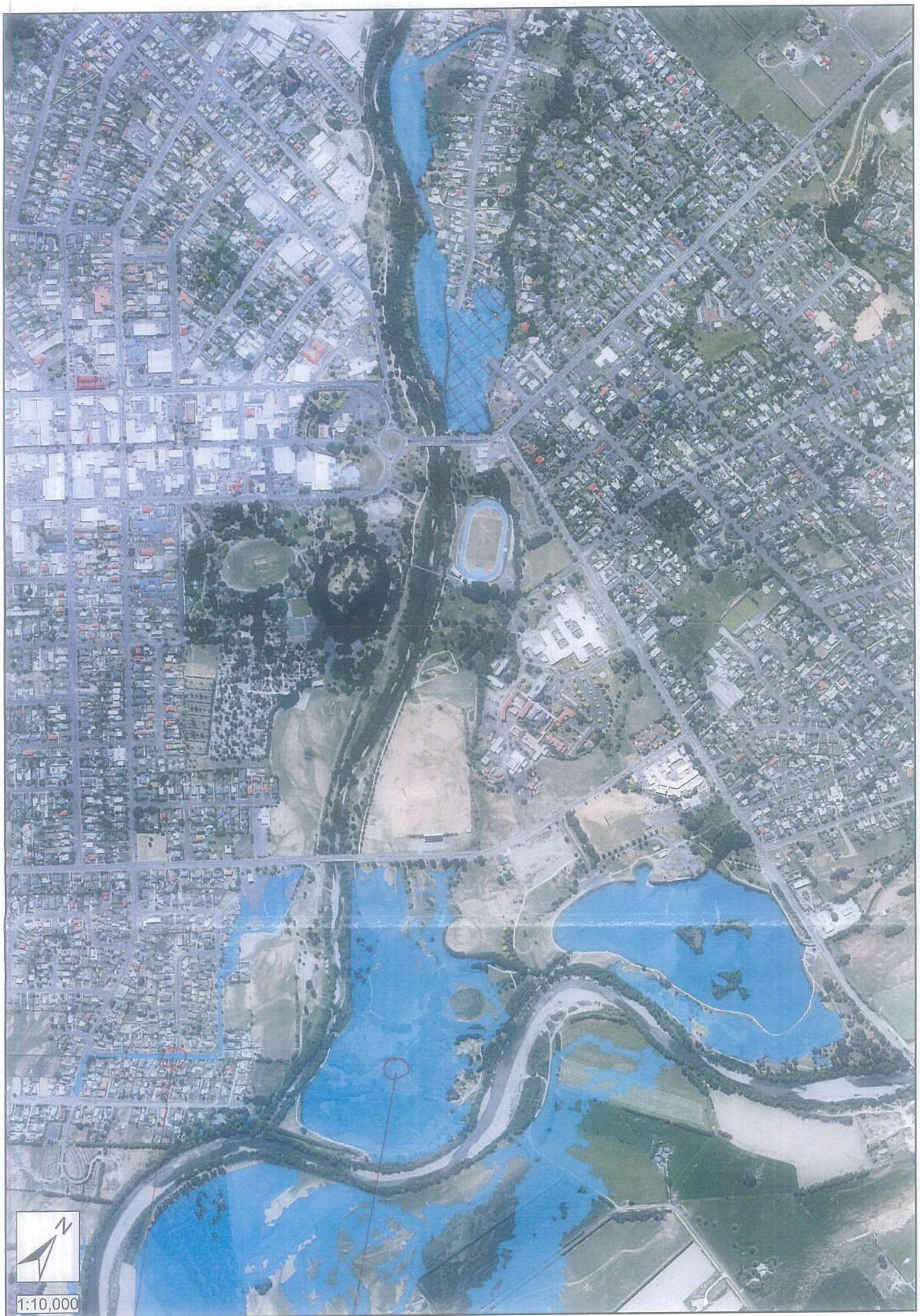
Question.
(1) Were the inundated areas shown taken from aerial photos flown by GW at the time? Answer No
(2) What was the modelled profile along the stop bank compared to the surveyed debris lines?
The debris lines represent the flood peak + wave hgt.
The attached GW flood profile is backed up by personal observation of local resident Mark Hall


Andrew Donalds Land
just upstream of rail·bridg
showing depression back from
the left bank berm.


LONG SECTION
NOV. 1994 FLOOD LEVELS
OCT. 1998 FLOOD LEVELS
0/10000
Right bank
1998 debris line
0.8m
Left bank
NOV. 1994
NOV 1998
112.79
110.11
This level surveyed by GW backs up what Mark Hall stated 3 years ago
Marks property in Oxford St is opposite this X Section
QUEEN STREET BRIDGE
RAIL BRIDGE
5
6
7
8
9
10
11
101.570
1100
110.97
116.60
116.76
(111.06)
(112.23)
101.310
1330
112.72
118.34
117.80
1354
1390
112.46
101.280
(112.78)
101.410
1560
112.96
118.53
100.980
1690
114.25
100.876
1800
114.40
119.18
119.30
(114.40)
100.570
2080
115.37
120.36
121.20
(115.27)
100.330
2340
122.10
122.22
100.080
2640
2670
118.86
125.06
123.62
100.000

Alec Birch verified this extent of flooding while inspection roads at the time.

The extent of flooding on Mahunga appears about right.
The damage to the golf course was bank erosion caused by bank full flow.

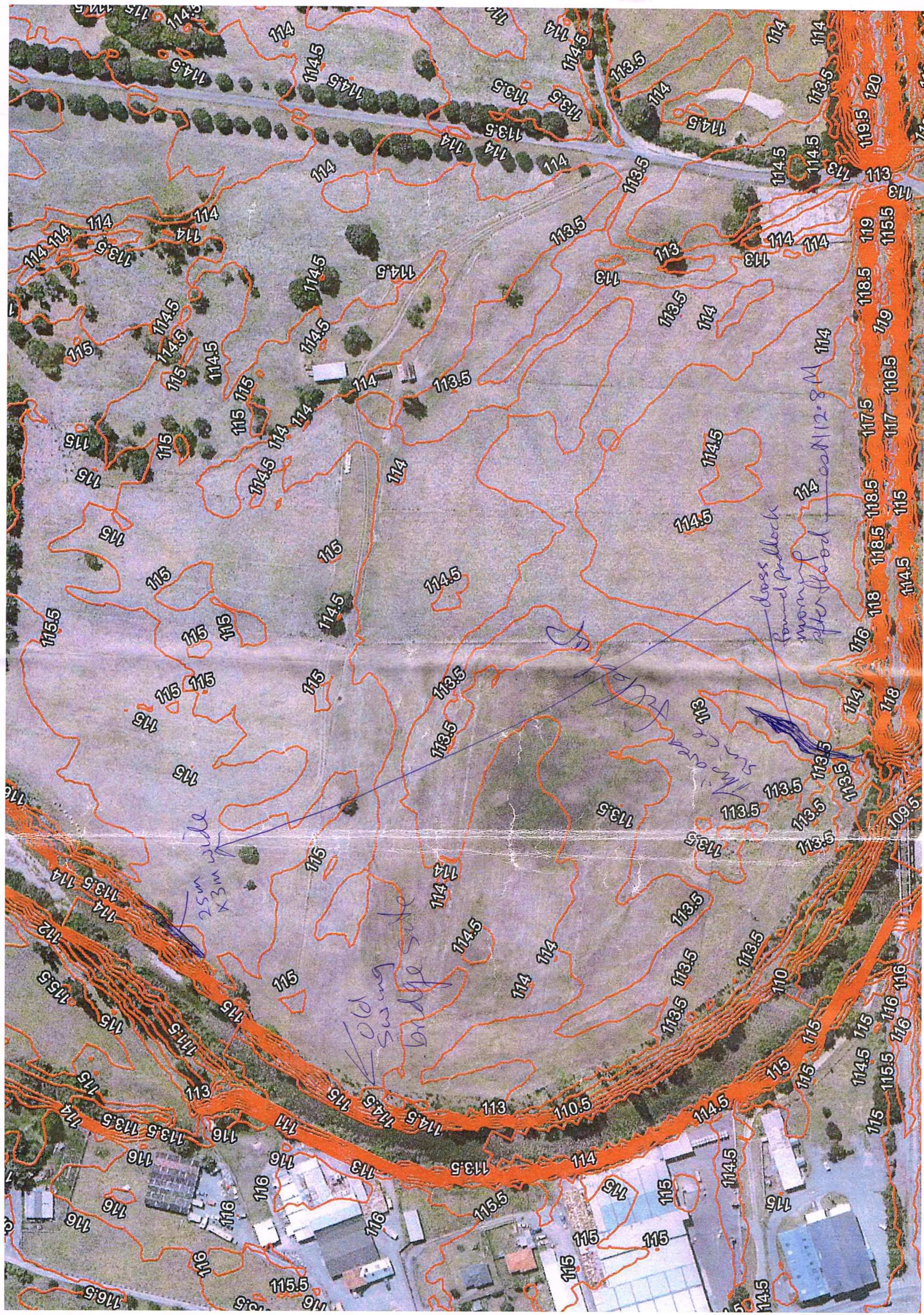


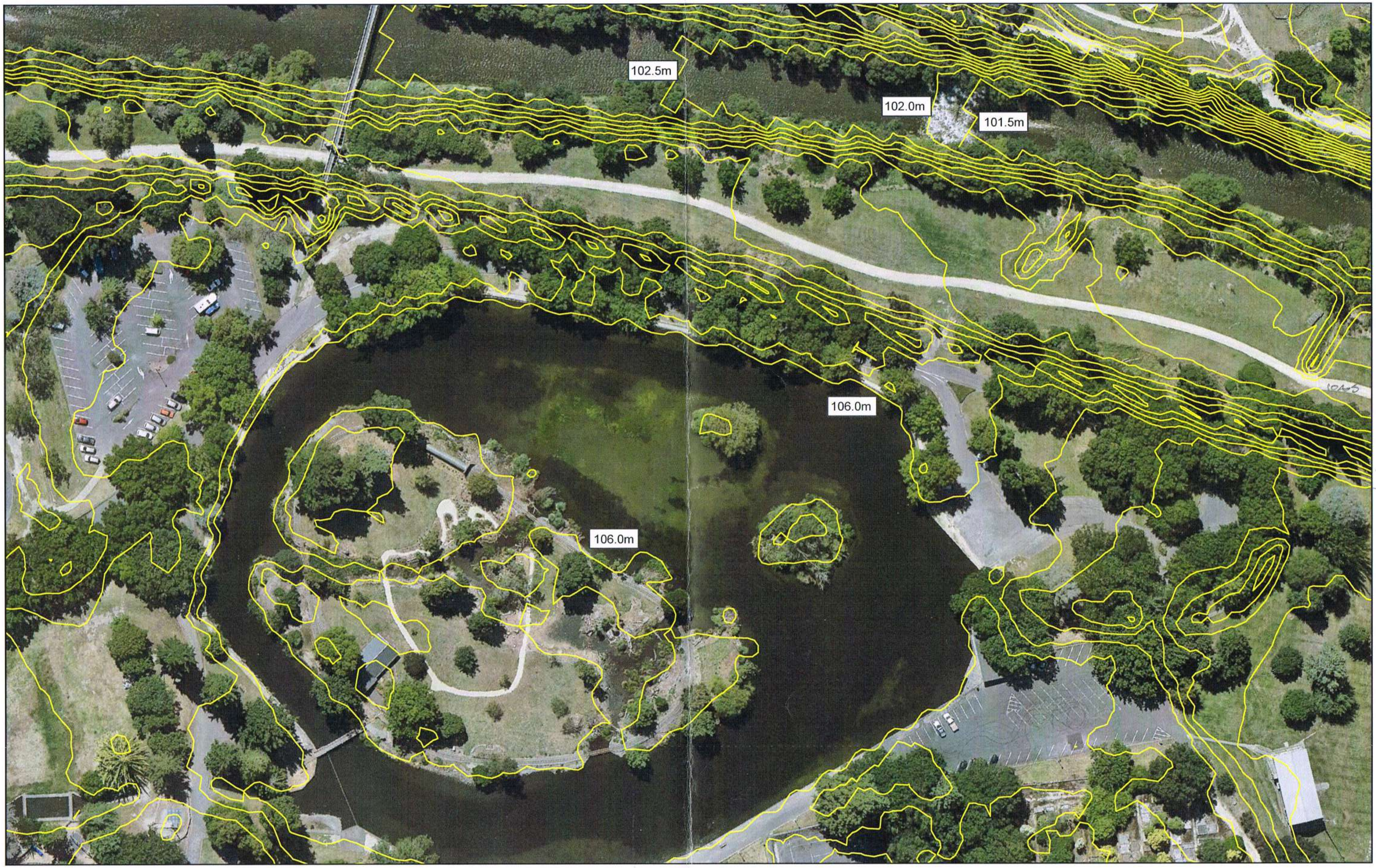
area caused by cut in stop bank during storm water pipe work.

Is this back water from the confluence? Yes. See attached photo from Oct. 2000 aerial. GW flew the 98 flood too late to record what happened in Masterton. They managed to record the Lower valley peak.

Whats this?

APPENDIX D – INFORMATION SUPPLIED BY LOCAL COMMUNITY MEMBERS


25m wide x3m
Old Swing bridge site



104.5

Masterton District Council

N E S W

Date: 29/09/2017

Queen Elizabeth Park Lake
500mm Contours

0 12.5 25 50 M

Scale = 1:1,000

Drawn By: alanf

C:\Users\alanf\Desktop\GIS\aaPROJECTZ\2017\QE Park Lake\500mm Contours.mxd

Masterton Flood Protection

Back ground.

To date there have been 4 estimates of flood probability made for our stop bank system to gauge our level of protection during peak flows.

George Butcher produced an estimate for our civil defence group and came up with a 200 yr return period before the banks would be overtopped at the low point.

Niwa in the recent study came up with 209 yr return period for the 1947 event for which there is no record of breach of the new stop banks back then.

I have come up with a 200 yr estimate for the lowest point in the stop bank system opposite the fire station by summising that the 98 flood was a 75 yr event (based on work done by NIWA) and adding the differences in height between a 75 – 100 and 100 – 200 yr flood derived from the flood probability graph fig 2 at the lowest section of stop bank.

GW came up with a 36 year return period for the 98 flood, and ignoring history said we are not protected from a 50 yr event.

I have worked backwards from actual flood levels produced during the 98 event and added the increased height caused by the difference in flow from the 75 – 100 – 200yr estimates of peak discharge at Miki Miki and included the ungauged flow estimate provided by GW.

The estimate I made is conservative in that I have used the high value of 30% for ungauged flow and used a low value for the mean velocity in the river cross sections used for the analysis. The mean velocity value was taken from historic GW slope area gauging's.

From experience, valley plains areas like the ungauged portion of the Waipoua Catchment don't contribute much to peak discharge, only total volume of water produced from a rainfall event. The reason for this is that overland flow is usually slow and follows up on the recession of a flood hydrograph in the main river channel.

The flood profile (attached) along the stop banks during the 98 flood shows the low points and freeboard above the flood level for the entire reach.

I have had an estimate of $34,000 for plugging the low point opposite the fire station which would give the town an even greater level of protection. Also we only need a low bank of say 1 meter on the Oxford Street side to give every one more than a 500 yr protection.

Ken

Return Period Estimates

Method 1

Using a record length of 76 years since the banks were built and ranking 1947 flood first and the 1998 flood second we can use the Gringorten empirical formula as follows

Return Period T= N +0.12/i-0.44 where N = length of record i = rank of flood

= 76/1-0.44

1947 = 136 yrs

T = 76/2-0.44

1998 = 49 yrs

Method 2

Using a historic record length of 117 yrs and ranking the 2 main floods in history as determined from the Masterton archive.

Return Period T = 117 + 0.12/ 1- 0.44

1947 = 209 yrs

T = 117 + 0.12 / 2 – 0.44

1998 = 75 yrs

Method 3

Using annual maxima floods supplied by GW and plotting on Gumbel graph paper with an EV fit.

The peak discharge for the 1998 flood was adjusted up by NIWA to around 400 m3/s.

The 2 plots provided by PDP and NIWA are shown in fig 1 and using the top adjusted plot the return period for the 1998 flood at 400 m3/s was 75 yrs.

A graph in fig 2 using the top curve is easier to read.

Fig 1

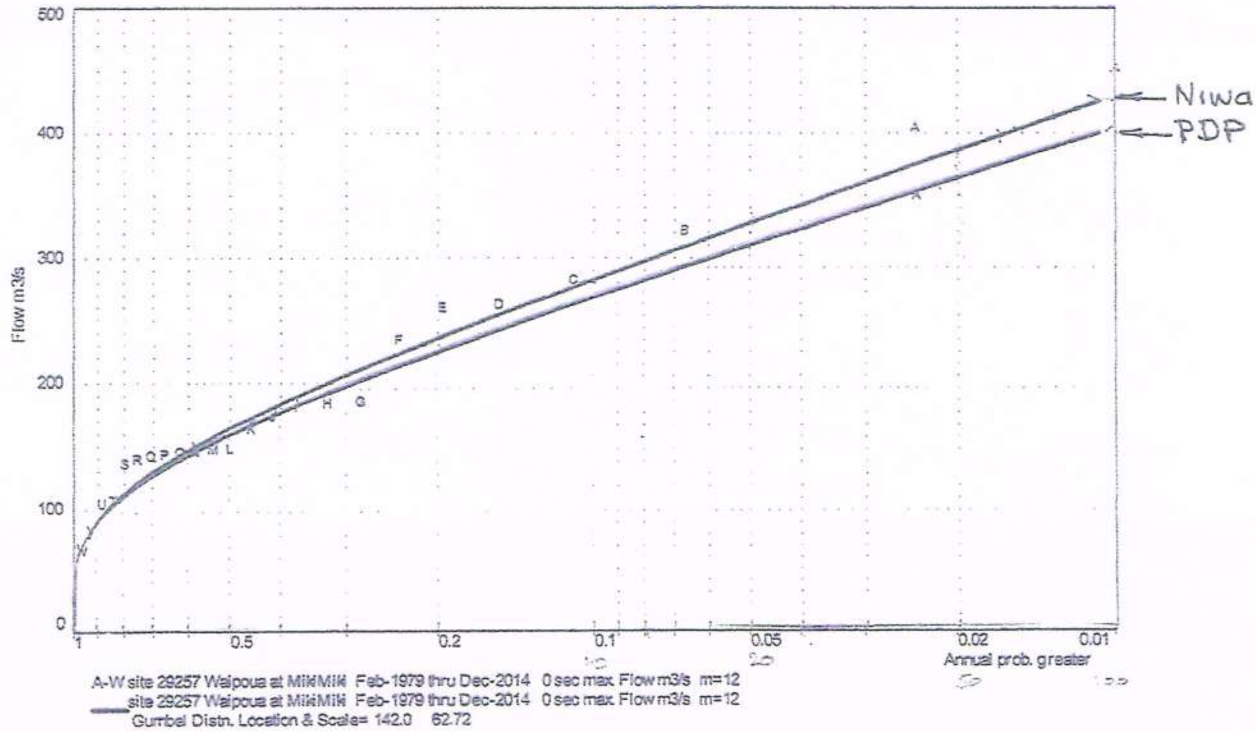


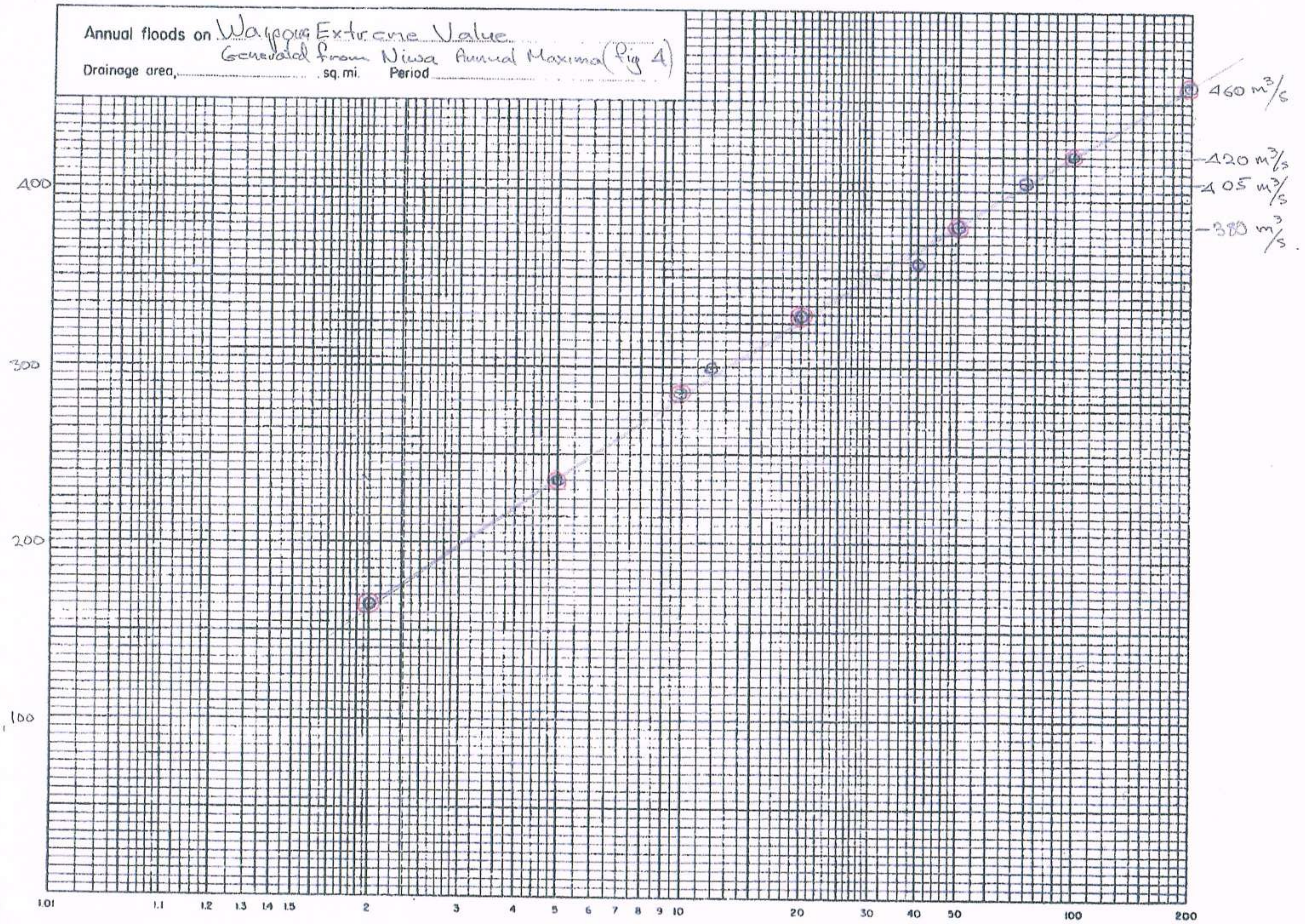
Figure 4: Frequency of annual maxima (1979-1982, 1996-2014) fitted by a Gumbel distribution (1998 flood peak = 400 m^3/s).

Fig 2

9 - 179a
Flood data plot
(March 1949)

UNITED STATES DEPARTMENT OF THE INTERIOR — GEOLOGICAL SURVEY — WATER RESOURCES DIVISION

File Number ....



Flood Height Calculation

The calculations for flood height for the 75 – 100 – 200 year return periods are done for 2 of 11 cross sections along the stop bank system. These were chosen because flow gauging's have been done at these locations and they are representative of risk to the community.

Cross section 8 is the low point in the bank opposite the fire station and cross section 4 is representative of the reach between the two bridges at SH2 and Columbo Rd.

The Cross sections were supplied by GW locally and are 2 of 11 sections that are surveyed regularly for bed change and bank erosion.

The flood heights for the 98 flood (75yr) are from a surveyed flood profile along the stop banks after the flood event at fixed bench marked points.

The bottom line across the sections represented is that from the 1998 or 75yr return period flood at the peak of the flood between the banks.

The calculations derive the change in height for the 100 and 200 yr return periods above the 98 event which is used as a bench mark for discussion.

Supplied with the cross sections is a plot of the longitudinal profile of 2 floods in 94 and 98 also obtained from GW.

Flooding fears recede for town

Masterton
Piers Fuller
piers.fuller@stuff.co.nz

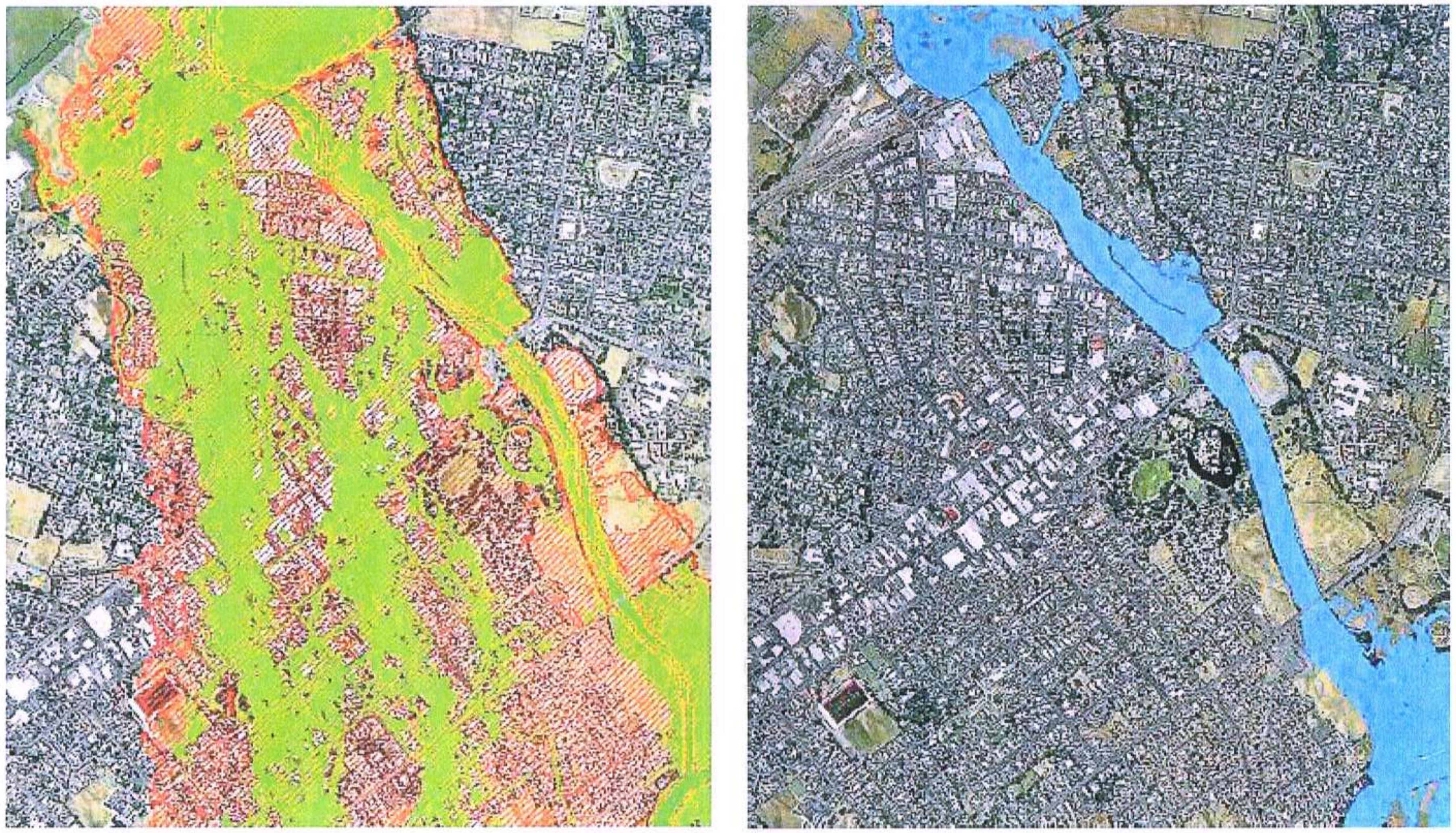
Large parts of Masterton have been shown to be safe from inundation after a long battle to correct flood modelling.

Models published in 2014 predicted a one-in-a-hundred-year flood of the Waipoua River would leave swaths of the town underwater, with protection measures estimated to cost at least $10 million. But Masterton District Council did not accept the Greater Wellington Regional Council's findings and challenged them with its own information.

Masterton Deputy Mayor Graham McClymont was a key figure in disputing the original model. He was first alerted to a problem when he contacted a resident in one of the worst-affected areas and carried out basic surveying to show the water wouldn't get as high as predicted.

McClymont was on the regional council's Te Kāuru Upper Ruamāhanga floodplain management subcommittee and has worked with Masterton District Council staff over the past four years to prove the initial model was "seriously flawed".

"It was a really long and hard process. It's taken ages and ages getting to where we are now."



Greater Wellington Regional Council's original 2014 model of one-in-hundred-year flood event showed vast swaths (in red) of Masterton would be flooded; right, the revised 2018 model showed (in blue) that most of the water in a one-in-a-hundred-year flood would be contained within the stopbanks of the Waipoua River.

The 2014 predictions would have had a severe impact on the town by stifling development, disrupting town amenities, and potentially affecting emergency response, he said. "It has created uncertainty with insurance and development."

Subcommittee chairman Bob Francis said better information improved the accuracy of the model.

"The next step is to put affordable and acceptable flood management in place and ensure inappropriate development does not create new problems."

One of the areas worst affected by a one-in-a-hundred-year flood was Oxford St, north of the Waipoua River. Resident Mark Hall originally made McClymont aware of the street's flood history and carried out some basic surveying to show water levels would not reach levels as high as predicted.

"We thought their data was a little erroneous," he said. "That modelling came from overseas and the model's only as good as the data or the assumptions."

> *"It's taken ages and ages getting to where we are now."*
> **Graham McClymont**

Greater Wellington flood protection manager Graeme Campbell said the original modelling was not wrong.

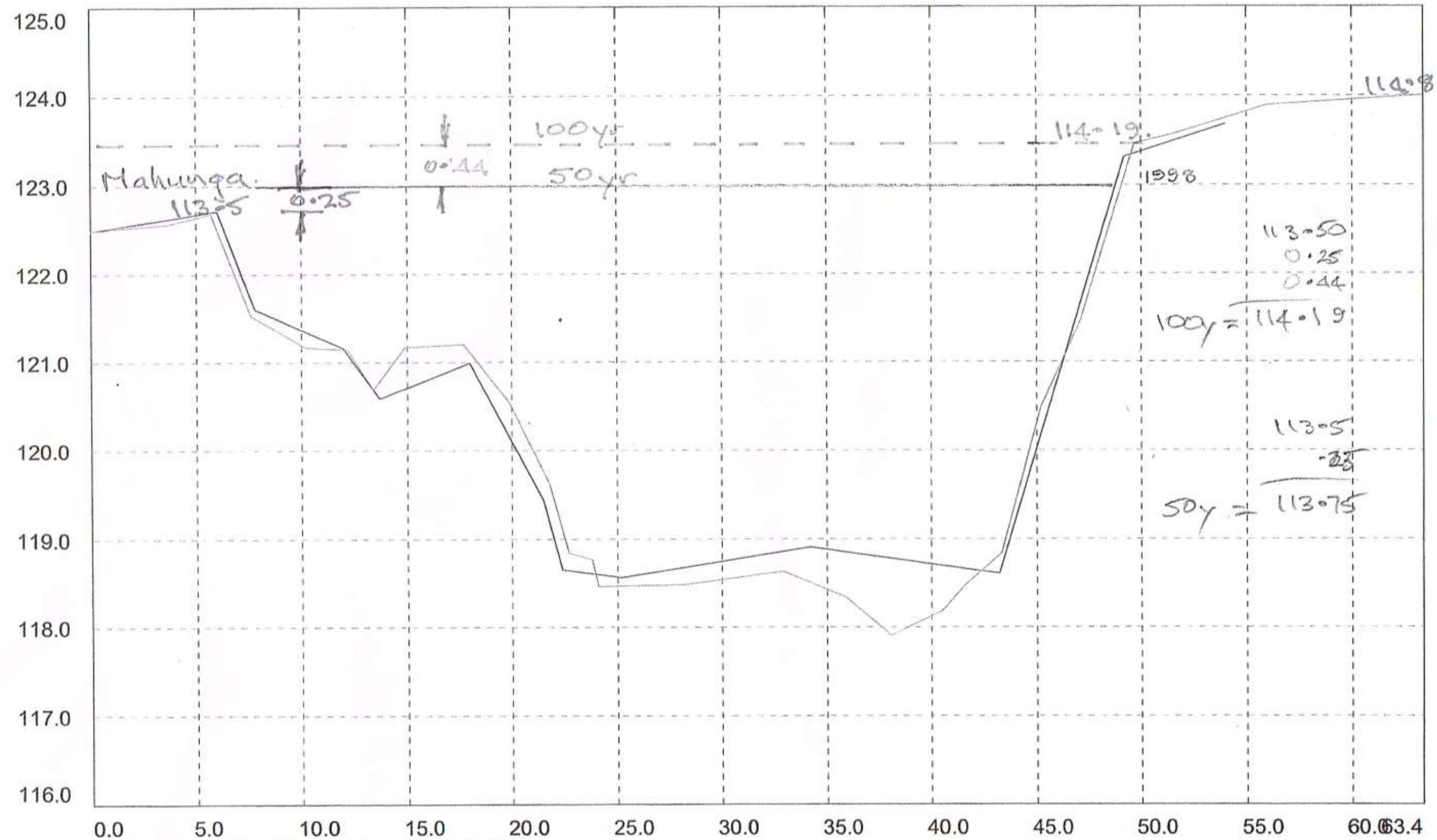
"We have used the same software but taken advantage of improved modelling techniques that have become more readily available since 2014."

A Masterton District Council source said the regional council's original model was not calibrated to an existing flood event and the data did not take into account the present levels of flood protection.

"The problem was that they never spoke to anybody and they never took any consideration of the history of the river going way back. These guys only had 20 years of bad data," the source said.

Campbell said, "We've worked hard with MDC to get the estimates as good as possible. This has included examining new information that has come to light since 2014."

ex Ken Downing



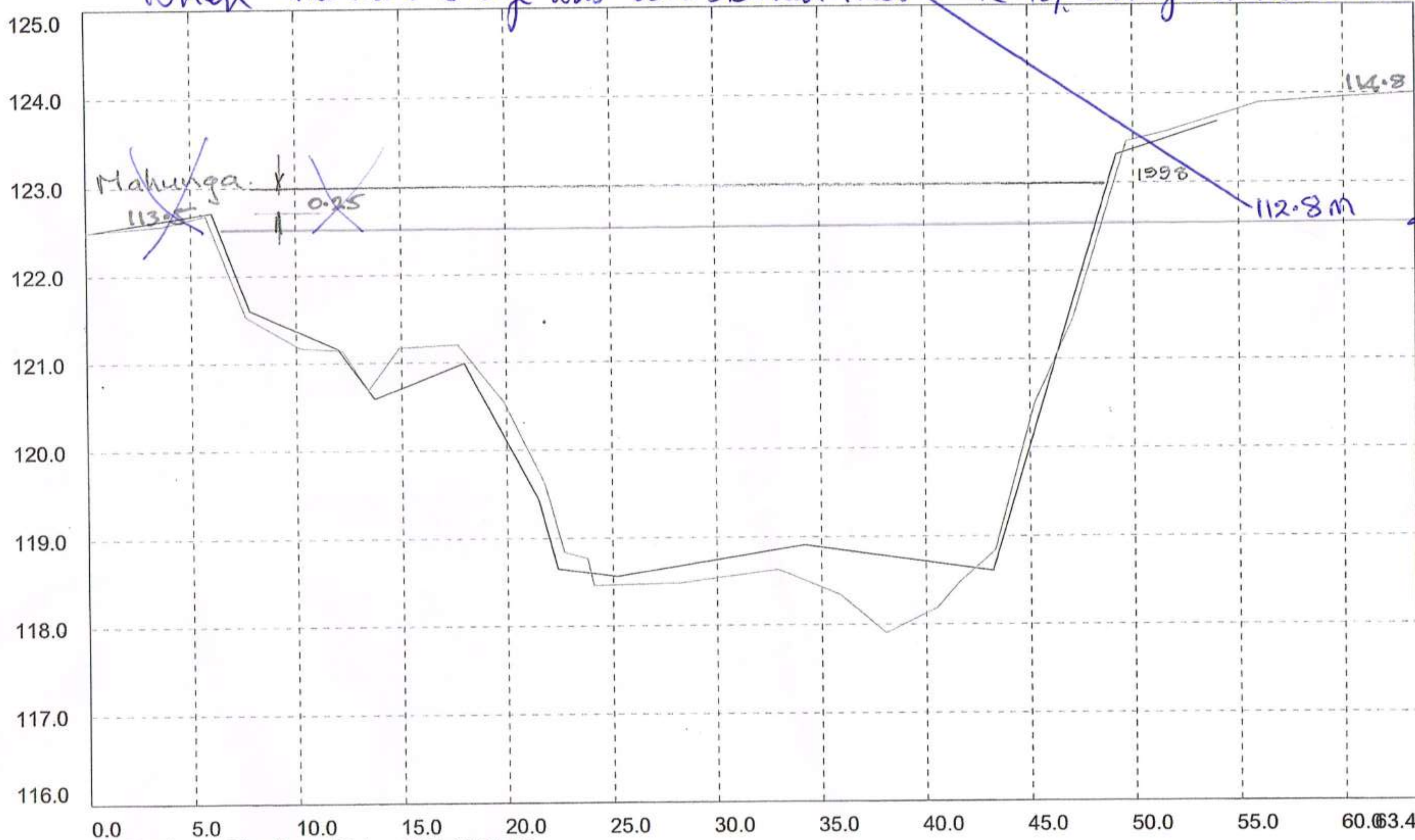
RL Section at Site 11 on 16-Jan-1995 12:00:00
RL Section at Site 11 on 24-Aug-2010 12:00:00

50 yr = 413
100 yr = 493
Diff = 80 m³

Q = W × D × 4
80 = 45 × D × 4
D = 80 / 180
= 0·44

Ex Andrew Donald

When the Rail bridge was semi blocked trees ie 10% blakage can not be added as this was actual



RL Section at Site 11 on 16-Jan-1995 12:00:00
RL Section at Site 11 on 24-Aug-2010 12:00:00

Even if you decide to be conservative and say 5 or 10% extra blockage, is it a chock point compared to SH2B or worse columbo road

Ken Downing's thoughts.

The State of The Global Warming Debate and How It Effects Masterton.

Since the United Nations funded IPCC (Intergovernmental Panel on Climate Change) was founded with a brief to investigate mans impact on global warming, the narrative has been one of manmade induced warming from CO2 emissions from fossil fuel consumption.

What has emerged from these early studies is a worldwide carbon taxation system that penalises poor and developing countries by preventing them from developing out of fossil fuelled economies as rich western countries have done.

Also, there has been a push to change from fossil fuels to renewable energy sources resulting in higher energy prices.

The IPCC research brief was narrow, measuring atmosphere, land and sea temperatures and comparing these measurements with CO2 concentrations in the atmosphere.

However, the sciences relating to Astrophysics, Paleoclimate and Geophysics have been omitted from the debate so as not to influence the narrative.

There has been much scientific dissent as new research evidence emerges from these omitted sciences questioning the manmade climate change narrative to such an extent that the 'global warming' label has been changed to 'climate change'.

The main driver for dissent has been that the IPCC climate change computer models assuming a CO2 driven climate change can't produce a prediction that is backed by empirical observations. It seems to be a common theme where computer models over estimate and are not backed up by empirical observation. This means that something else is causing the rise in CO2 and needs to be added to the model so that modelling matches actual observed data. What is emerging from new science is that we are heading into a cooling and bad weather phase.

The predicted bad weather emanates from studies from other sciences such as Astrophysics, Paleoclimate and, Geophysical sciences that show correlations between historically recorded bad weather and emerging science.

Carbon Dioxide in the atmosphere is increasing regardless of what we do. So, this means the science is not settled.

What does this mean for Masterton?

Unfortunately, Greater Wellington Regional Council has developed policy based on the old narrative of 'global warming' to increase the value of the 100yr. design flood and so reproduce the original map covering the town with water.

The flood risk on this map is based on fiction instead of estimating flood frequency from measured flow data and historic floods in the absence of flow measurements as is standard practice in the science of Hydrology.

It does appear from GWs constant campaigning in public, that they are determined to promulgate the original flood maps based on an uncalibrated computer model even though we have been safe based on empirical observation since our stop banks were constructed 80 years ago.

We need to be resilient to what may eventuate with climate change and not pour rates money into just one possible outcome of flood risk from the Waipoua river. We need to worry about our storm water systems and possibility of earth quake damage to private property and infrastructure, and not be indebted to the Regional Council with their expensive solutions backed by the consultant industry which is willing to validate their narrative.

Ken Downing – notes

Latest Finding Relating to Waipoua Flood Frequency.

Initially the 47 flood was ranked 1st since stop bank construction in 1939.

In the absence of any ground truth in town, the NIWA report ranked the 47 flood greater than 98 because of higher rainfall totals in 47.

However I recently found some ground truth reference on a Waipoua bank Profile survey (1952).

The 1947 flood levels are at points along the profile showing the level relative to the banks.

I compared the 47 flood height at the town bridge to the 98 flood at the same point using the flood profiles from the 94 and 98 floods supplied by GW at the time and comparing the distance from the top of the bank to the surveyed debris levels at the SH2 Bridge.

What I found was that the 98 flood level was higher.

This may not be much by itself but a check on bed level revealed that the 98 flood had an extra 1.6 meter of channel depth and therefore would have been considerably larger than the 47 flood as far as peak discharge goes.

So what was the return period of the 98 flood in light of this new finding?

If we believe the reference in the Soil Conservation and Rivers Control Council book (1957) that the 47 flood was the largest since 1897, and if you believe the factual evidence presented here that the 98 flood is greater, then it was the largest in 120 years.

Survey data is attached.

The Gringorten Formula would give the first ranked flood a return period of 1.8*120 = 216 yrs.

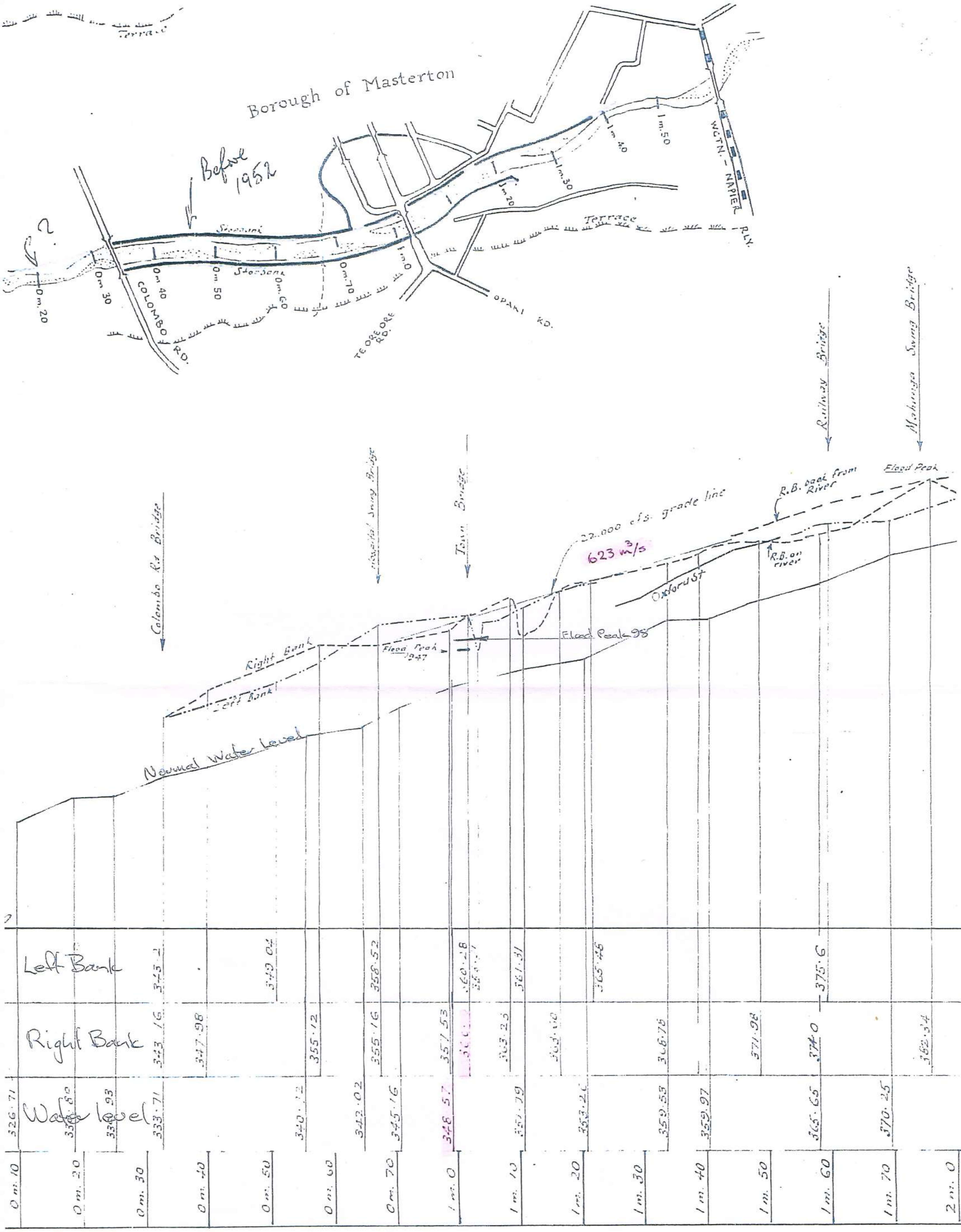
I hope this shows the error produced when using short term record as GW have done and ignoring history from a site with only 20 years of poor recorded data and a lot of theoretical flimflam to produce a model that flooded the town.

With regard to free board now that there is a model somewhere close to what might happen in a large event I have tracked bed level change from 1952 to 2010 with profiles supplied by GW.

The bed profile continues to degrade and is held high in the vicinity of the weirs but has dropped between 2-3 meters in the channel between the stop banks over time. I would not add any free board.

GW have just calibrated the model to a 200 yr flood so there is not much to do from the rail bridge to Colombo Bridge.

My concern for the town is upstream of the rail bridge and I would be a bit conservative by going for 20% bridge blockage and patch up the low spots on the right bank.



Track bed Level at town bridge

1952

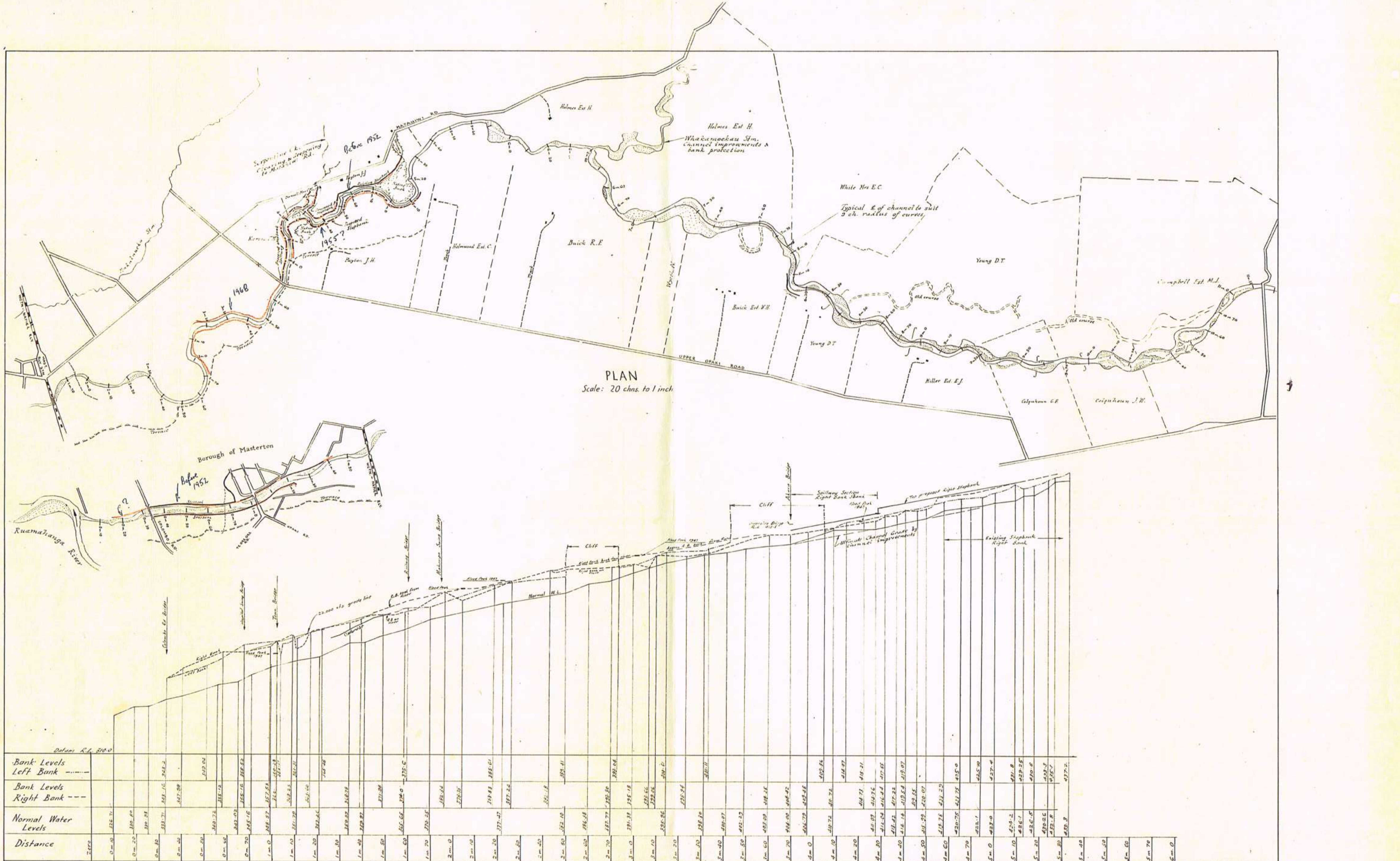
Right Bank 360·00'
WL 348·57'
+ 11·43' × 0·33 = 3·77 m add ·5m = 4·3 m (52)

1998

Right Bank 118·68
Bed Level 112·78
5·9 m (98)

1952 – 1998 bed level di

5·9 – 4·3 = 1·6 m
1998 1952



PROFILE

Scales: Horizontal: 20 chns. to 1 inch
Vertical: 20 ft. to 1 inch

WAIRARAPA CATCHMENT BOARD

WAIPOUA RIVER CONTROL
GENERAL PLAN & DETAILED
PROFILE 0 m. to 5 m. 30 ch.
ANNUAL PROGRAMME 196 - 196

Scales: As shown

Approved
Engineer

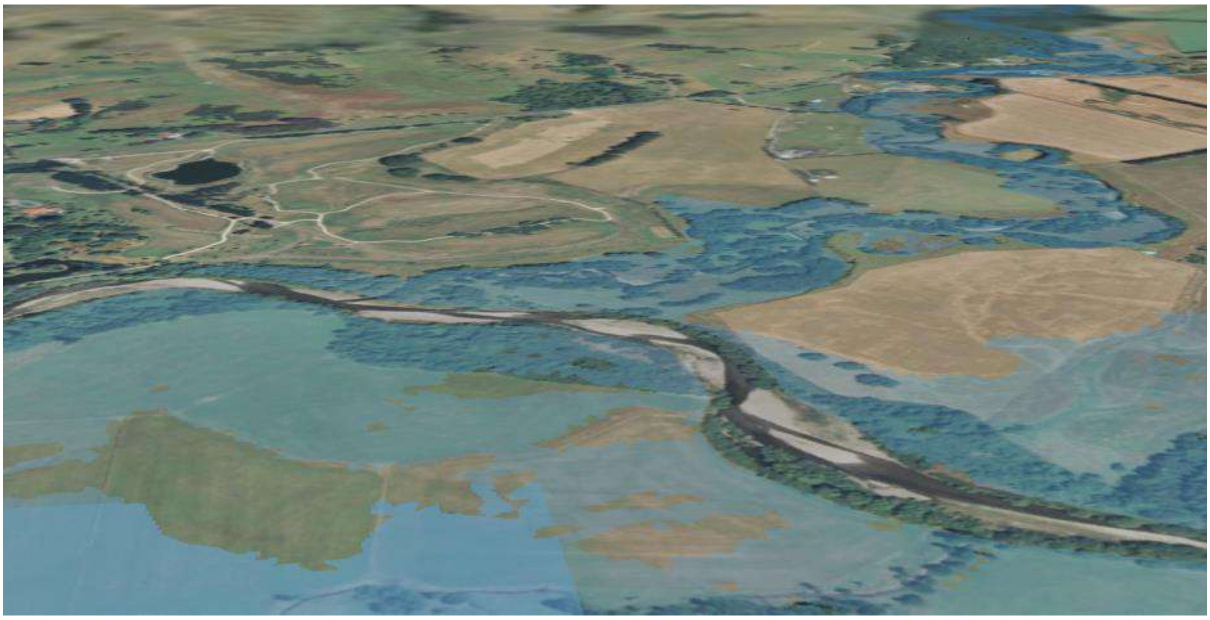
5/10

956

APPENDIX E – COMPARISON OF 1998 AERIAL PHOTOS WITH MODEL RESULTS (WAIPOUA RIVER)

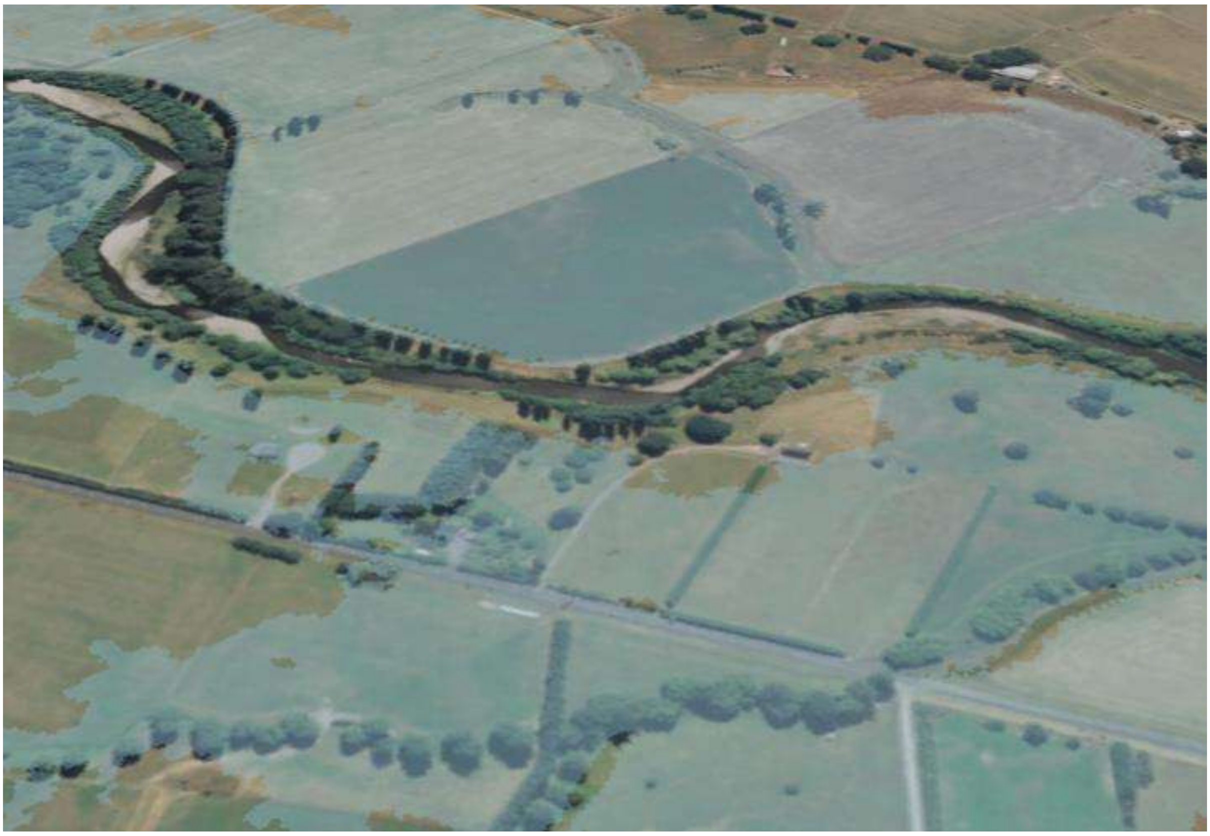
Site 1 (9810_4)

Site 2 (9810_12)

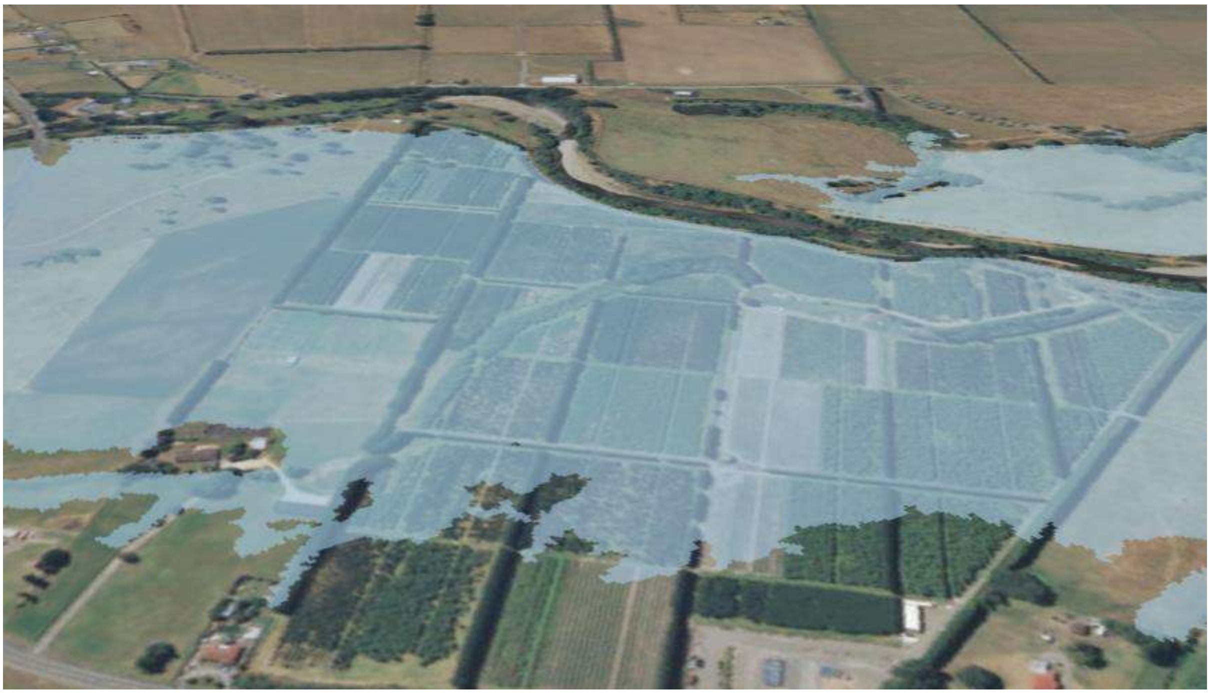


Site 3 (9810_16)

Site 4 (9810_23)

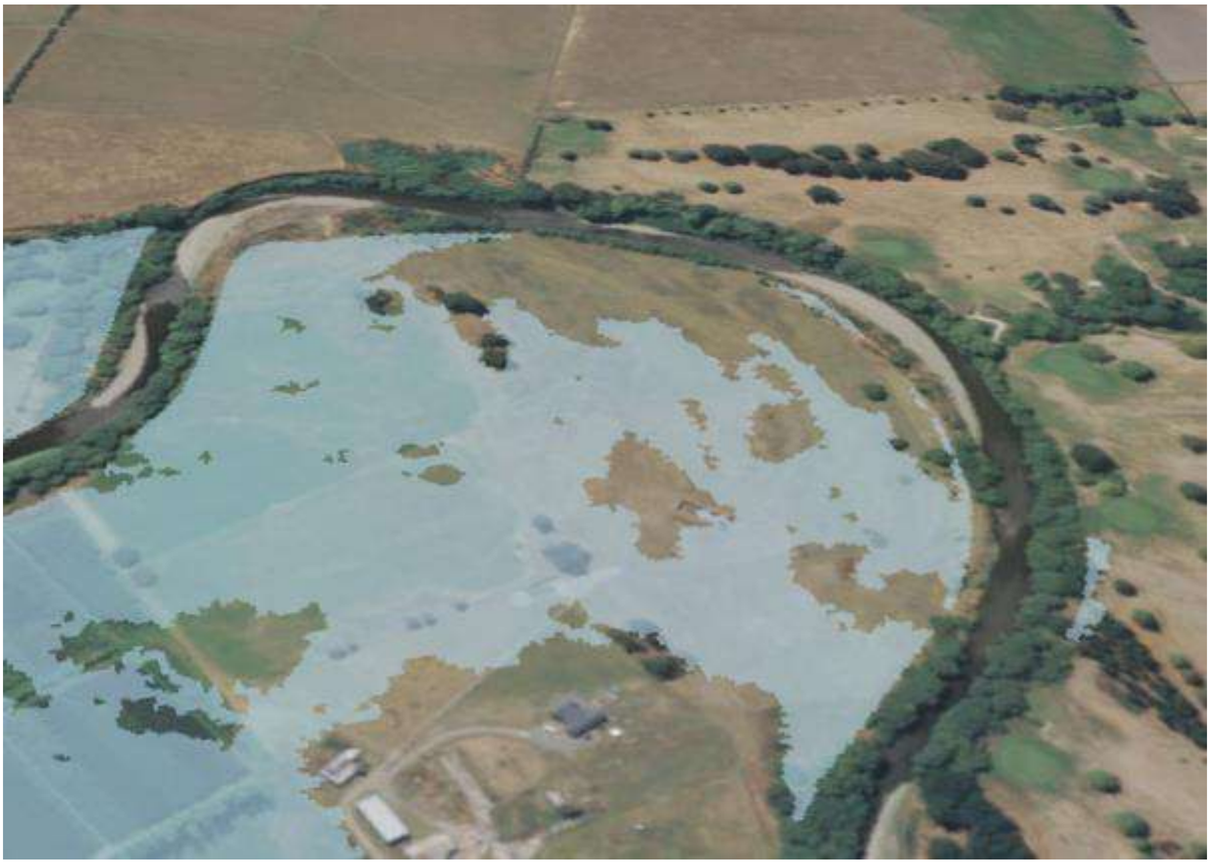


Site 5 (9810_29)



Site 6 (9810_28)

Site 7 (9810_31)



Site 8 (9810_32)

Site 9 (9810_34)

Site 10 (9810_33)

Site 11 (9810_35)

Site 12 (9810_36)

Site 13 (9810_38)

Site 14 (9810_41)

APPENDIX F – SUMMARY OF HYDROLOGICAL ISSUES IDENTIFIED IN PREVIOUS PEER REVIEWS

Summary of Te Kauru hydrology peer review points to date					
Document reviewed	By	Date	Review point	Addressed?	Notes
Wairarapa hydrological investigations, PDP 2013	NIWA	May-13	Would have been good if it provided a fuller picture of the nature of Wairarapa river flooding, weather systems and flood peak timing for tributaries	No	
			The hydraulic analysis concluding that the ungauged part of Waipoua catchment yielded 16% flow increase was discarded.	Yes	A value of 32% from the regional flood frequency contours was used instead. Based on later work, a final figure of 25% has been agreed between GW and MDC.
			Sound analysis of the impact of IPO on river flows	OK	
			No comment made on flood dynamics such as timing of flood peaks and peak travel times (in the AEP coincidence section).	No	
			Error in contour mapping of PE3 distributions using EV1 representation	Yes	Underlying Qbar was corrected and maps adjusted to show specific PE3 outputs
			Regional contour maps should be adequate for use in regional flood frequency interpolation for return periods of 100 years or less.	OK	
			Design hydrograph shapes - estimation of time of concentration on the time scale was not carried out but would have been useful for transferring design hydrographs to ungauged river locations.	No	Reviewer states that this method is acceptable to generate design hydrographs **for the flow recording sites** (my emphasis).
			Flood frequency plots were not log-scaled, and did not show the comparative distributions from which the PE3 was selected, nor the L-Moment ratios of the series. A regional flood frequency analysis was also suggested.	No	This would have justified the chosen distributions and shown more clearly the fit, which was just chosen by eye. Otherwise, this is not conclusive.
Estimation of flood peak magnitude and return period: Waipoua River at Mikimiki - 20 October 1998,	Brin Williman	May-15	Reviewer questions whether the high flow stage record was examined for all significant flood events, and also gaugings studied and their location confirmed.	Yes	Charles Pearson at NIWA said that this was done.
			Based on experience with a similar unstable river gauging site, reviewer points out that bed scour during an event limited Froude number to 0.7 and maximum Q100 velocity to 4.5m/s. NIWA used Fr = 0.99 and v=5m/s. Reviewer feels that v=4.2m/s is equally viable for the information presented, then Qp = 336m3/s	No	Charles agreed that more work on the hydraulics could be done at Mikimiki, but did not think it warranted, because the difference in flood estimates was small... 336 vs 400?
			Reivewer requested the parameters used for the regional flood estimation be provided. This yielded Q = 390ms/s	No	This is not in the NIWA report, although they said it could be provided.
			Other methods of assessing flood flows were suggested: regional flood frequency, hydraulic analysis of flood flows, rainfall runoff.	Yes	Subsequent work has involved a rainfall-runoff model.
			Historic flooding not investigated, merely focussed on the 1998 and 1947 floods. No comparison of stopbank and flood levels.	No	Charles agreed that more investigation of flood and bank levels would be useful.
			No proof was provided for the statement that the stopbanks contained both of these floods.	No	Charles suggested a rewording crediting MDC staff with this statement, which had not been checked by NIWA, but it is not included in the report I was provided.
			A clearer tabular summary of flood frequency distribution and corresponding flows would have been helpful	No	This was not provided, but can be generated from the EV1 parameters below the plot.
			No estimate of flood frequency/ flood flows for Masterton is made	No	This was not covered in the report; the work has focussed on flows at Mikimiki
			Reviewer points out that a 24hour rainfall amount is used to support the 1947 flood having a return period of about 200 years. The Waipoua will respond to a critical rainfall duration of 6-8 hours, so this is not enough information to say this rainfall caused the biggest known event on the Waipoua	No	Charles agrees that this conclusion should be toned down, but it has not been. Using a Gringorten plotting position is useful for checking a distribution shape, but it is misleading if used to estimate a single flood return period as was done for 1947.

Document reviewed	By	Date	Review point	Addressed?	Notes
			Climate change is not considered in either the NIWA or PDP reports.	Yes	Charles responds that this has been included in the GW flood modelling
			Recommends further study of the 1998 event - flood levels, bank damage, flooded land area and hydraulic sensitivity testing. Also use of the 1998 flood with added margin to be used as a 'design flood', provided there was enough information to put it in the hydraulic model.	?	Charles agrees that this would be useful
			Noted the apparent lack of cooperation between the councils and recommended that they work together as best they can for the ratepayers.	OK	This appears to be improving, but there is more to do.
			Reviewer commented that he was provided with limited information and scope.		
			Choice of design flood size, uncertainty and climate warming effects should be agreed between GW and MDC	Yes	This has been agreed, at least for the Waipoua
Waipoua River Rainfall-Runoff modelling, MWH 2016	T+T	Apr-16	Overall rainfall-runoff model development consistent with standard practice.	OK	
			Rainfall-runoff loss model (initial and continuous loss) is too simplistic to represent actual catchment runoff behaviour	No	The model output has been used, but was manually scaled or adjusted for historic and design flood simulations.
			Sub catchments over-discretised, given the single flow and single rain recorder	No	13 subcatchments were used, have not been changed.
			Calibration to 4 historic events by rainfall-runoff model was not good. Simulated flows arrived too early and volume at start of storm too high, with the falling limb receeding too quickly.	No	Only a single intensity rainfall gauge was used to construct rainfall hyetographs. Other daily gauges weren't used to provide better understanding of rainfalls.
			Reviewer questions the forward weighted temporal rainfall pattern used and asks for justification.	No	Distribution is based on rainfall from 17 annual maximum storms in the Wellington area. Intensity is high in the early to mid part of the event.
			Reviewer questions the application of 24 hour - 100 year duration relationships to construction of all rainfall events. These would generally vary with duration.	No	24 hours - 100 year depth weighting factors were applied to all rain durations and subcatchments
			Have ARF been applied to the point rainfall estimates?	No	Not mentioned in the report, but they were not applied. Reason given was that data was already uncertain, so an additional factor not warranted.
			Review recommends collecting more local hydrometric data to help with calibration and better understand effect of different storm patterns on the lower catchment	Yes	This is discussed in section 3.2.1, with respect to the four calibration floods in particular, but nothing conclusive was found.
			Different values of 356 and 400m3/s used for the 1998 flood in hydrographs and report tables 2-1 and 2-2	OK	Still a typo in Table 2-1, but 1998 flows shown as 356m3/s in Table 2-2, and is plotted at 356 in hydrograph comparison.
			Confusion over comparison of modelled design flow peaks with MWH or NIWA frequency distribution.	OK	Now modelled design flows compared to MWH EV1 flood frequency values, and 1998 flood shown as 356m3/s (Section 4.3, Table 4-2).
			Conflict between the 2d hydraulic model and the rainfall - runoff model outputs for Waipoua not resolved or explained.	No	Design flow peak estimates fall back on flood frequency estimates from the Mikimiki gauge. Design hydrograph shape is taken from the rainfall-runoff model.
			Reviewer finds wide difference between regional frequency contours 100 year flood estimates and the hydrological model output: 533 vs. 493 m3/s	No	MWH cautions against the general use of regional flood contours in Section 8, but these have been used for all Upper Ruamahanga subcatchments

Document reviewed	By	Date	Review point	Addressed?	Notes
			Assessment of runoff contribution from the lower Waipoua catchment has not converged to a definitive result, so significant uncertainty remains around the magnitude of this contribution	Yes	An intermediate value of 25% has been used to represent the contribution from the lower Waipoua catchment. In MWH report, it says that this was agreed between GW, MDC and MWH. Good reasons for the apparent low flow contribution from lower catchment given.
			Reinstate the Colombo bridge recorder to provide a measure of the lower catchment runoff contribution	Yes	This was done 22 Sept 2015. Site is not telemetered
			Shows that the 100 year ARI flow for Waipoua at Mikimiki is between 400 and 415m3/s if 1998 flood is taken as 356m3/s and between 415 and 430m3/s if 1998 flood is taken as 400m3/s	OK	A value of 406m3/s has subsequently been used. This is based on MWH flood frequency dist EV1 (u=128.1 and alpha = 60.32m3/s). The 2013 PDP PE3 distribution would give 404m3/s. Fig 4-11 still shows 100y ARI as approx. 430m3/s
			Identifies that 'determining the peak of the 1998 flood is important.. because this event can serve as a benchmark for flood management planning on the Lower Waipoua River.'	No	Waipoua floodplain hydraulic model appears to have used 356m3/s for the 1998 flood. The estimated ARI for this flow value is 45 years, based on the MWH EV1 analysis.
			Highlights that uncertainty is discussed but not all sources identified and an overall estimate of uncertainty not provided.	Yes	This is covered in Section 7. Large flood essentially +/- 23%. This seems sensible but covers a wide flow range.
					GW Waipoua flood model has used 50 and 100 year design flood peaks of 363 and 406m^3/s resp.

APPENDIX G – SUMMARY OF FINAL AUDIT RECOMMENDATIONS

GENERAL RECOMMENDATIONS:

1 That a consistent / documented approach to model freeboard is adopted for all models in the FMP
That the sensitivity approach adopted in the flexible mesh model report is adopted for all models
That further scenarios which investigate more blockage scenarios, bed level changes and changes in temporal pattern are incorporated into the final sensitivity layer.
2 Consideration is given to adding a degree of transparency to the flood layer so that landmarks can be easily identified as well as adding road names to allow easy orientation for local landowners.
3 Consideration is given to presenting the flood depth maps using a range of discrete colours rather than a graduated colour scale
4 Flood velocity maps are generated showing both speed and direction of flow
5 Flood hazard maps based on a combination of speed and depth are generated in order to communicate the risk associated with the hazard.
6 Consideration is given to publishing videos online showing how the flood propagates in the model over time.
7 That all model results are merged into a single file. Decisions around which result file takes precedence in areas where models overlap could be made now, rather than relying on interpretation when requests for information come in.
8 As soon as more data is available for the Colombo Rd site, use this to refine the estimate of flow contribution from the lower Waipoua catchment.
A longer record at Colombo Rd could also be used to improve calibration data downstream of the ungauged catchment area. This could aid in validation of the hydrological and hydraulic models, so that these tools become more accurate.
Ideally, the site should be telemetered to facilitate access to this data and gauged regularly to get a grasp on the site characteristics and stability.
9 Maintain regular gauging of sites and provide an estimate of the uncertainty in the rating.

MINOR RECOMMENDATIONS

1 Consideration should be given to including the bridges into the model to allow for the localised impact and to better represent the headloss through the structure.
2 The culvert setups at KopuBridges 4221 and 4167 and the bridge setup at KopuBridge 15583 are reconsidered in order to remove the instabilities present in the 100 year CC results.
3 If the results are to be used for setting floor levels, then the lateral link connections should be broken into smaller segments, in particular around tight bends (Kopuaranga).
4 If the results are to be used for setting floor levels, then the lateral link connections should be broken into smaller segments, in particular around tight bends (Whangaehu).
5 The lateral links are remodelled for the Whangaehu River at MIKE11 Chainage 9452
6 That consideration is given to including the State Highway 2 and Railway bridges into the model so that the localised impact of the bridge piers is included in the model results and so that sensitivity to debris blockage can be simulated.

7 Consideration should be given to carrying out a sensitivity test to blockage of Patricks Bridge as well as consideration to including the piers of the Te Whiti Road bridge so that sensitivity to debris build up on the piers can also be investigated, especially for lower flow events.

8 If the results are to be used for setting floor levels, then the lateral link connections should be broken into smaller segments, in particular around tight bends (Taueru).

9 That consideration is given to including the Gladstone and Kokatau bridges into the model so that the localised impact of the bridge piers is included in the model results and so that sensitivity to debris

10 Lateral links should be included both upstream and downstream of the confluence with the Taueru River

11 It is recommended that a comparison between the 1m grid based on the base LiDAR with the final 10m grid used in the model is carried out to ensure road crest levels are adequately captured in the model, as per the DHI peer review recommendations. A representative sample should be checked where flood waters cross roads in the final results.

12 It is recommended that the model report is finalised with all relevant information included and the document is finalised into a single merged pdf which contains all of the Appendices.

13 Consideration could be given to including individual buildings in the roughness definition file in future upgrades of the model using the recently released buildings polygon layer (LINZ)

14 That the bridge piers for both the Colombo Rd and the State Highway 2 bridge are included in the model setup

That consideration to lowering the soffit of the Colombo Rd bridge to account for potential effects of floating debris during a large event.

15 It is recommended that the lateral link on the true right bank of the river downstream from the State Highway bridge (M11 chainage 30590 to 30720) is included in the model, or otherwise its exclusion is justified. The inclusion of this link is unlikely to have a significant impact on the overall model conclusions.

16 It is recommended that the seamline checks are formalised and documented.

17 It is recommended that the rating curve from the model and gauged rating curve at Colombo are compared as per the peer review report. It is currently unclear if a rating curve is yet to be developed for the Colombo Rd site, this should be investigated.

18 It is recommended that thought is given to converting the historical flood levels from the 1947 flood into the current datum and make a comparison of the historic flood levels with the 1998 model calibration results in the same location. This may assist in getting a better feel for the likely magnitude of the 1947 flood event.

A search for historic orthophotos could also be made from the 1940's.

19 Simplify the subcatchment representation if scaled hydrographs are to be used.

20 Collect relevant regional flood frequency contours and underlying data into a single figure to assist understanding.

21 That the flows from the flood frequency distribution for Kopuaranga at Palmers be corrected.

22 Hydrological boundary conditions used as input to the hydraulic models be checked for consistency and correctness.

23 Investigate the cause of the drift in the Taueru at Te Wheraiti record and decide whether any adjustment needs to be made.

24 Review local rain records to validate the temporal rainfall distribution used for the Waipoua catchment, or consider a symmetrical temporal distribution.

MODERATE RECOMMENDATIONS

- That the lateral link elevations for the Ruamāhanga River are changed to use the M21 method for their source as recommended in the DHI peer review.
- Further justification needs to be given to ignoring the 1994 debris recordings for model calibration (Ruamāhanga River).
  A comparison with the adopted Manning's 'n' values should be made with the 1995 MIKE11 model.
  Before the results are finalised, it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;
  -Comparison with historic flood photos / debris levels
  -Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)
  -Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods
- That the lateral link elevations for the Kopuaranga River are changed to use the M21 method for their source as recommended in the DHI peer review.
- Before the results are finalised (Kopuaranga River), it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;
  -Comparison with historic flood photos
  -Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)
  - Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods
- That the lateral link elevations for the Whangaehu River are changed to use the M21 method for their source as recommended in the DHI peer review.
- Before the results are finalised (Whangaehu River), it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;
  -Comparison with historic flood photos
  -Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)
  - Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods
- That the apparent horizontal shift between the applied roughness file and the terrain model is investigated and the cause for the shift is rectified (Upper model only). The model will likely need to be rerun as a result.
- Whilst the results are calibrated to a 2-year event (Waingawa River), it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;
  -Comparison with historic flood photos
  -Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)
  - Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods

- Before the results are finalised (Taueru River), it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;
  -Comparison with historic flood photos
  -Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)
  - Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods
- That the lateral link elevations for the Ruamāhanga River are changed to use the M21 method for their source as recommended in the DHI peer review.
- Further justification needs to be given to ignoring the 1994 debris recordings for model calibration as well as the more recent events such as 2009 which were used in the Te Whiti Stopbank modelling (Ruamāhanga River).
  A comparison with the adopted Manning's 'n' values should be made with the historic models.
  Before the results are finalised, it is recommended that the model results go through a systematic / documented verification/validation process of some degree. Ideally this would involve;
  -Comparison with historic flood photos / debris levels
  -Comparison with the existing GWRC 50-year flood extent polygon (in locations where there is significant difference, a site visit, or closer inspection of the model in this location would be warranted)
  -Workshop with local residents or GWRC staff from the Masterton office with knowledge of the local catchment and behaviour of historic floods
- That the 1D bank markers are adjusted to match the 2D mesh boundary, in particular in the locations highlighted in the peer review report. (Waipoua Model)
- The Ruamahanga hydrology combines flood frequency parameter contours, scaled station hydrographs, fine catchment divisions and conservative AEP coincidence assumptions. These methods have a high cumulative uncertainty. The model ought to be validated to historic flood events, and attempts should be made to refine the hydrology input.
- Estimate confidence intervals for the PE3 flood frequency distributions.
- The underlying station data and other catchment characteristics must be taken into consideration in the selection of flood frequency characteristics from contours. Used in isolation the contours can generate anomalous results.
- That GW carry out an evaluation of whether a rainfall-runoff model or the current regional flood frequency based hydrology method, or some combination, is the best way to deliver hydrology inputs to the FMP.
- The AEP coincidence tables are not proven for higher return periods, and have a linear basis which is not sound. Simplification of this approach in design flood modelling has been overconservative. Consider using the AEP coincidence tables more systematically and developing an alternative method for scaling up AEP coincident flows.
- Reconsider whether Areal Reduction Factors should be applied to point rainfalls used in Waipoua catchment modelling, and update rainfall depths if required.
- Review GW climate warming impact projections for increased extreme rainfall and sea level rise in light of new MfE guidance.
  Investigate the response of Upper Ruamāhanga river flows to increased design rainfall, using rainfall-runoff modelling or similar. This would provide a stronger relationship between projected increases in climate warming and river flows.

- Including historic flood events could improve confidence in the flood frequency distribution for the Waipoua at Mikimiki. An in-depth investigation of historic sources would be needed to turn up any useful information. This investigation should not be limited to the 1947 event.

MAJOR RECOMMENDATIONS

1 Consideration needs to be given to adjusting the level of the recorded debris level at XS7 to account for the effect of superelevation as well as other physical phenomena. A review of the appropriateness of all debris levels used in this study may be warranted at the same time.
Significant thought should be given as to the most appropriate way to model the effects of the skew on the bridge. Due to the complexities involved in modelling this bridge structure, and the immense interest from the community in the impact of this structure, it may be appropriate to ensure that a range of experts are consulted with before an appropriate headloss through this structure is agreed upon.

2 That this embankment (upstream of Mawley Holiday Park) is surveyed in detail and included in the model. The model will then need to recalibrated as a result.

3 Design hydrographs should reflect subcatchment characteristics, such as time of concentration.

4 That the Waipoua rainfall-runoff model be revisited to provide a better representation of the physical catchment processes.
Also ensure that the way the runoff hydrographs are applied in the hydraulic model allows for reasonable travel time and storage.

That after refinement of the Waipoua rainfall-runoff model, the model be validated against at least those historic events used previously (October 1998 and September 2010). Ideally, more than two calibration events should be used.
The aim is that hydrology linked to the Waipoua flood modelling be robust enough that it does not need to be adjusted before use in the hydraulic model.

5 Revisit application of regional flood frequency parameters to model subcatchments, paying particular attention to location of source data and catchment characteristics such as elevation, aspect and shape. Check the resulting design flood estimates with reference to adjacent catchments, neighbouring stations and other return periods to ensure that results are sensible.