



Karamea floodplain investigation

**NIWA Client Report: CHC2010-107
October 2010**

NIWA Project: ELF 10234/1

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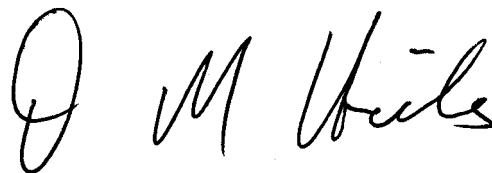
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1. Summary

A high resolution 2-dimensional numerical model of the Karamea area was produced to inform the Karamea community about the level of risk they face from flooding.

The model was tested on the October 1998 Karamea flood and the results compared favourably with residents recollections of flood depths during this event.

Flood inundation was modelled for 20, 50 and 100 year floods with and without the present overflow spillway at Umere Rd at the upstream end of the floodplain.

For the modelled 50 year flood there would be less flooding of Karamea communities if the present Umere Rd spillway was closed (provided that the present stopbanks do not breach).

For the modelled 100 year flood there is extensive flooding from water leaving the river at the Umere Road overflow and upstream of the Karamea bridge. Flood banks are overtopped by more than 0.6 m depth of water at several places. This is likely to lead to failure of the flood banks and very serious, rapid-onset flooding. All road access to the township would be cut.

WARNING: Modelled water levels given in the report do not include any residual velocity head or freeboard allowance.

2. Background

This investigation is to improve understanding of flooding in the Karamea area by using a high resolution, 2-dimensional, numerical hydrodynamic model. The modelled topography is based on LiDAR and incorporates both the Karamea river and its floodplains including Karamea township. The model results are to be used to inform the Karamea community about the level of risk they face from flooding, including more accurate information on the magnitude and path of floodwaters and the effects of physical flood mitigation measures. The model will help the community to visualise effects of large floods and to assess physical measures that could provide future flood protection for the area.

The study involves:

- 1.1 A high resolution **D**igital **E**levation **M**odel (DEM) and roughness map of the Karamea floodplain.

- 1.2 A 2-dimensional hydrodynamic model of the Karamea floodplain.
- 1.3 A map of predicted inundation from a historic flood for calibration and verification purposes.
- 1.4 Incorporation of input from Karamea residents to verify the model historical flood prediction and adjustment of the model calibration if required.
- 1.5 A series of floods run through the model (20 yr, 50 yr, 100 yr) to produce maps of inundated areas.
- 1.6 A repeat of the flood model runs with a theoretical flood bank constructed at the Umere Road spillover location, to produce maps showing levels of inundation that would occur if a flood bank were built at that location.
- 1.7 Interpretation notes for each flood situation.

3. Methodology

3.1. Topography

LiDAR scanning of the Karamea area was carried out by NZ Aerial Mapping on 18 June 2008.

Raw LiDAR survey data were pre-processed and interpolated to a 1m grid (using an algorithm that rejects non-ground points). The 24.4 million cells were then down-sampled to a 3m computational grid which preserves flow blocking features such as flood banks. Surface roughness (incorporated as the Z_0 parameter from the full log law) was mapped using raw LiDAR, photography and known ground features. Wet areas where LiDAR data were missing were patched from a ground DEM supplied by NZ Aerial Mapping and down-sampled to the 3m grid (using the ArcInfo resample command with cubic convolution).

River bathymetry was interpolated from WCRC-supplied cross-section data (2006 survey). The interpolation was aided by interpretation of aerial photography (for the Karamea River mouth area and Northern branch) and incorporated into the computational grid to form a 3m topographic DEM.

The model DEM datum is the 1937 Lyttelton datum. Mean sea level was adjusted by -0.019 m to adjust local sea level to the Lyttelton datum. Offshore bathymetry was manually approximated (it will not have any significant influence on the river model).

River mouth bathymetry was manually edited to represent the bar conditions during a flood.

The final model comprised a grid of 2.7 million, 3 metre square cells with the lower left corner of the domain positioned at NZTM grid reference: E 1523880, N 5430180.

3.2. Hydraulic model

The model used was Hydro 2de (<http://www.fluvial.ch/p/hydro.html>) which solves the depth-averaged shallow water equations on a rectangular grid using a finite volume discretisation. It allows wet and dry domains, sub- and super-critical flow conditions, variable bed topography, variable hydraulic roughness, and dynamic boundary conditions. This model has been used on a wide range of New Zealand rivers.

3.3. Model calibration

The recorded hydrograph of the 19-20 October 1998 flood (with a peak discharge of 3163 m³/s) was modelled using the hind-casted tides for the time of the flood. The October 1998 flood was assessed to have a 1/20 Annual Exceedance Probability (AEP). This flood hydrograph and the corresponding tide levels are shown in Appendix 1. The high resolution mapping of model surface roughness and results of the calibration are given in Appendix 1. The initial results were discussed with Karamea residents who had observed the actual flood and adjustments were made to correct some minor irregularities.

3.4. Simulated floods

The behaviour of 20 yr, 50 yr, and 100 yr floods was investigated with the calibrated model (note that a 20 yr flood has an annual exceedance probability of 0.05 and is also referred to as a 1/20 AEP flood). The hydrographs used to represent these floods are shown in Figure 1. The flood AEP is based on flood peak flow. Because there were actual recorded floods corresponding to the 1/20 and 1/100 AEP peak flows, the hydrographs for these historic floods were used in the model. However, the volume of water passed by the 20 year flood (October 1998) is greater than that of the 100 year flood (November 1973). The 50 year flood is interpolated between the 20 year and 100 year floods.

Tidal conditions used for the simulated floods were the same as those used for the October 1998 calibration flood event and are shown in Appendix 1, Figure A1.

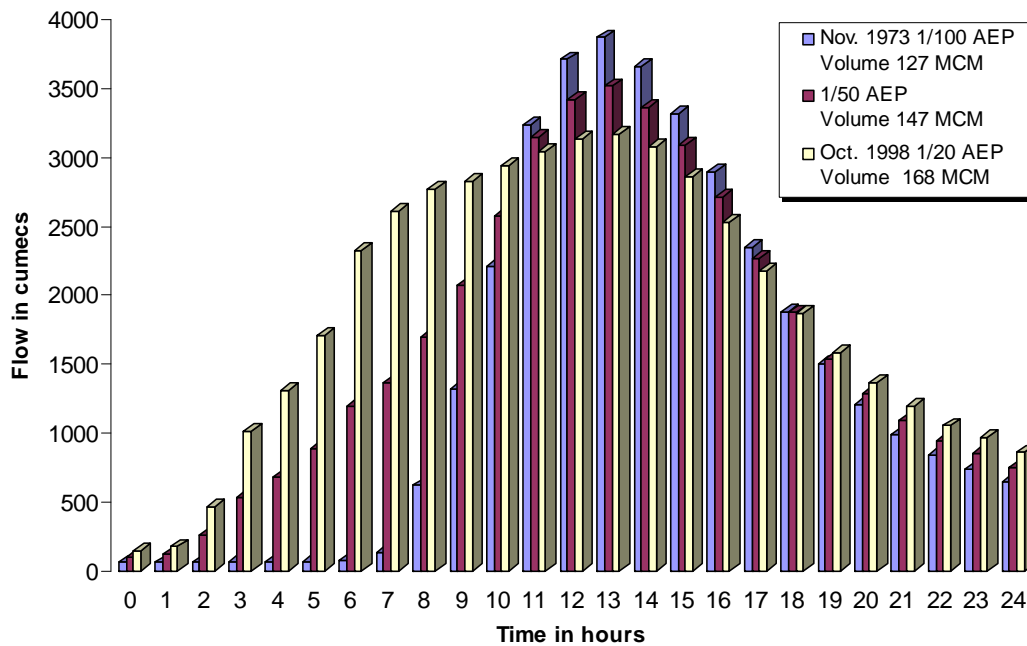


Figure 1: 100 year, 50 year and 20 year flood hydrographs used in the model. The total volume of water passing in 24 hours is indicated (MCM=million cubic metres).

3.5. Freeboard consideration

The design crest level of existing flood bank levels includes freeboard to allow for velocity head runup, for uncertainties in hydrologic and hydraulic modelling and for unmodelled effects such as waves/debris/bed movement. The model-predicted river levels are calculated levels which do not include any freeboard or safety factor. This must be born in mind when evaluating the performance of existing flood banks or when considering future flood banks.

4. Results and discussion

4.1. LiDAR

LiDAR data were compared with earlier survey information for flood bank crest levels (GEM, 2006) and it was noted that there were marked, systematic discrepancies in some places. For 1.5 km downstream of the bridge and from 1-2 km upstream of the bridge, maximum LiDAR-indicated levels for the right-hand side flood bank were typically 0.3 metre lower than the surveyed flood bank crest levels (see Figure 2). Minimum LiDAR-indicated flood bank levels at this location were up to 1 metre lower than the surveyed crest levels. Because LiDAR hits give a “shotgun” sample of ground elevations the narrowness of the flood bank crest could explain errors in the LiDAR indicated minimum flood bank levels but this cannot explain apparent errors in the maximum LiDAR-indicated levels (the LiDAR could not consistently miss

hitting the top of the flood bank for hundreds of metres along the bank). One explanation is that the flood banks have recently subsided or consolidated in these locations. Otherwise, one or both surveys may contain some systematic error. Considering that LiDAR levels are typically very accurate relative to nearby LiDAR-measured levels, that ground survey levels were not available away from the flood banks and that the LiDAR levels were lower (i.e. more conservative in terms of assessing flood inundation) than the ground survey data, it was decided to use levels derived from the 18 June 2008 LiDAR survey to represent the flood bank levels in the model runs.¹ Local minima were removed where LiDAR “hits” had missed the flood bank crest.

4.2. Flood bank levels

A comparison of near-bank water levels with the crest level of the right-hand side flood bank is given in Figure 3. This long section shows that, in addition to the upstream overflow near Umere Rd, which is not shown, there is a second significant overflow region upstream of the bridge between 800m and 1900m downstream of the Umere Rd overflow.

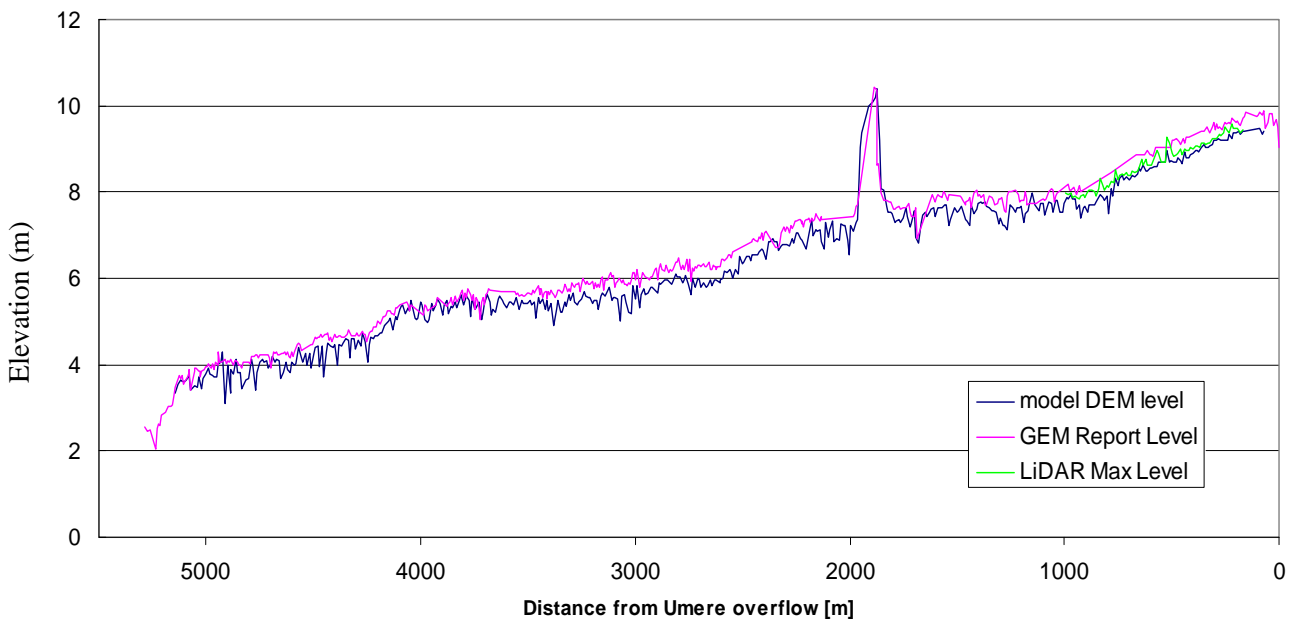


Figure 2: Flood bank crest levels from GEM (2006) compared with LiDAR-based levels used for the investigation DEM. The spike at 1900 m is the road bridge embankment. Note that the LIDAR and ground survey levels coincide on the bridge.

¹ A subsequent investigation by NZAM attributes the survey level differences to geoidal correction and recommends that the LiDAR levels should be used.

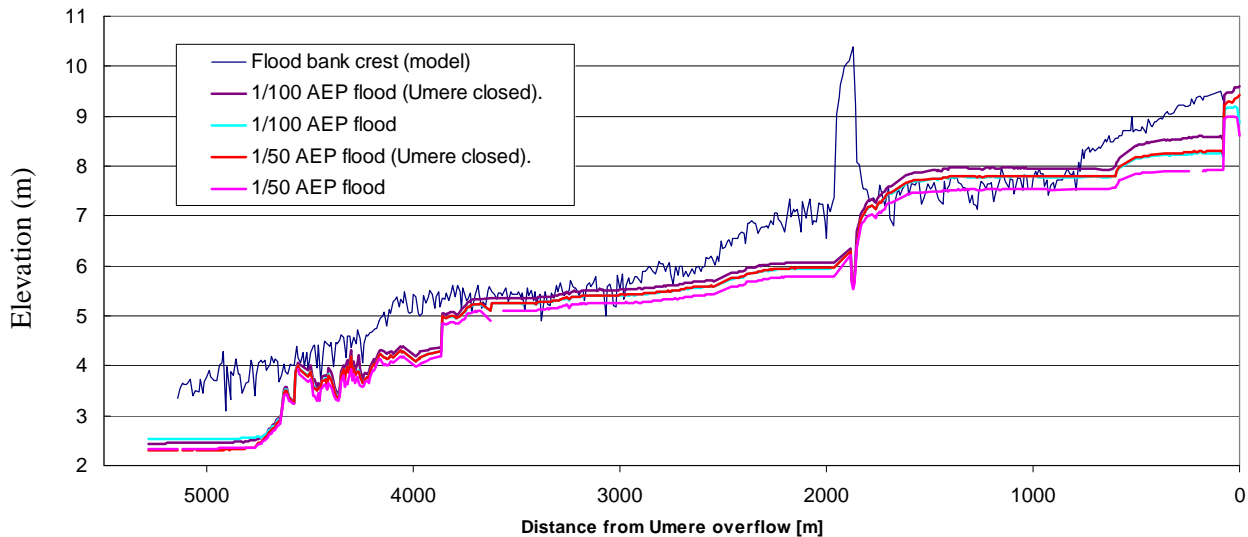


Figure 3: Peak flood levels compared with LiDAR-based, right-side, bank crest levels. Residual velocity head and freeboard are not included. In some places the flood bank is remote from the main river channel.

Four scenarios are shown on Figure 3 to illustrate water levels for 1/50 and 1/100 AEP floods with and without blocking of the existing low weir near Umere Rd which allows flood water to overflow and bypass the river by flowing to the North of the town. In addition to overflow upstream of the bridge (800m - 1900m from the overflow), the floods also overtop the flood banks downstream of Market Cross (3000m - 3500m from the overflow) where there is an old river meander channel. Closing of the Umere Rd overflow increases peak water levels in the river by 0.25 – 0.35 m upstream of the bridge (lowering to a difference of 0.1 – 0.15 m further downstream as water spills out of the river by overtopping the banks). If flood banks were also raised to prevent water spilling over them, the increase in river levels caused by blocking the Umere Rd overflow would be more than 0.25 – 0.35 m (also downstream of the bridge). Note that in Figure 3 the modelled water levels do not include any residual velocity head or allowance for modelling uncertainties. Because of such uncertainties the flood banks could spill at locations not indicated on Figure 3. In addition, the flood banks could breach due to structural failure or piping under the banks.

4.3. 20 year flood

Maximum inundation depths from the 1/20 AEP flood are shown in Figure 4. Development of inundation during the rising flood is shown by the calibration maps in Appendix 1.

The 1/20 AEP flood results show that while some downstream floodwater backs up via the Quinlands Rd channel (at the top of Figures 4 and A5), most inland inundation

comes from upstream floodwaters that spill out of the upstream river near 273 Umere Rd (right side of Figures 4 and A6). Near the flood peak, 12 hrs after the flood started, it is estimated that around 175 cubic metres per second were spilling from the right bank of the river at this point. Also at 12 hrs there was some spill over the flood banks just upstream of the bridge (as shown in detail on Figure A9). Eyewitness observations confirmed that Mr. Langford's farm track overtopped at this point in the 1998 flood. Road access to Karamea township is cut by the floodwaters.

4.4. 50 year flood

Despite a lower overall flood volume, flooding is more extensive for the modelled 1/50 AEP flood (Figure 5) than for the 1/20 AEP 1998 hydrograph. Inundation depths are slightly greater and the township is surrounded by floodwaters (Figure 5a). With a new flood bank in position at the Umere Road overflow location (Figure 6) the 1/50 AEP flooding is less than for the 1/20 AEP case and road access to the township may be feasible.

With the present Umere Road overflow, water floods into the old river channel west of Market Cross from the north. With the Umere Road overflow blocked, water flows into this river channel from river overflow upstream of the bridge.

Flood water near the main Karamea settlement also arrives from the north. In the case of closing the Umere Road overflow there is a small overflow from the river near the main settlement but it does not reach the town. There is increased overflow from the river in the vicinity of the old river meander west of Market Cross.

While the model indicates that there would be less flooding if the Umere Road overflow were closed, in places river levels are very close to the top of the present flood banks and there is no safety factor.

4.5. 100 year flood

For the modelled 1/100 AEP flood there is extensive flooding from water leaving the river at the Umere Road overflow and upstream of the bridge (Figure 7). Flood banks are also overtopped at the old Market Cross river channel and alongside the main Karamea settlement (Figure 7A). All road access to the township is cut. In places, water spilling over flood banks upstream of the bridge is over 0.6 m deep and this is likely to lead to failure of the flood banks and very serious, rapid-onset flooding.

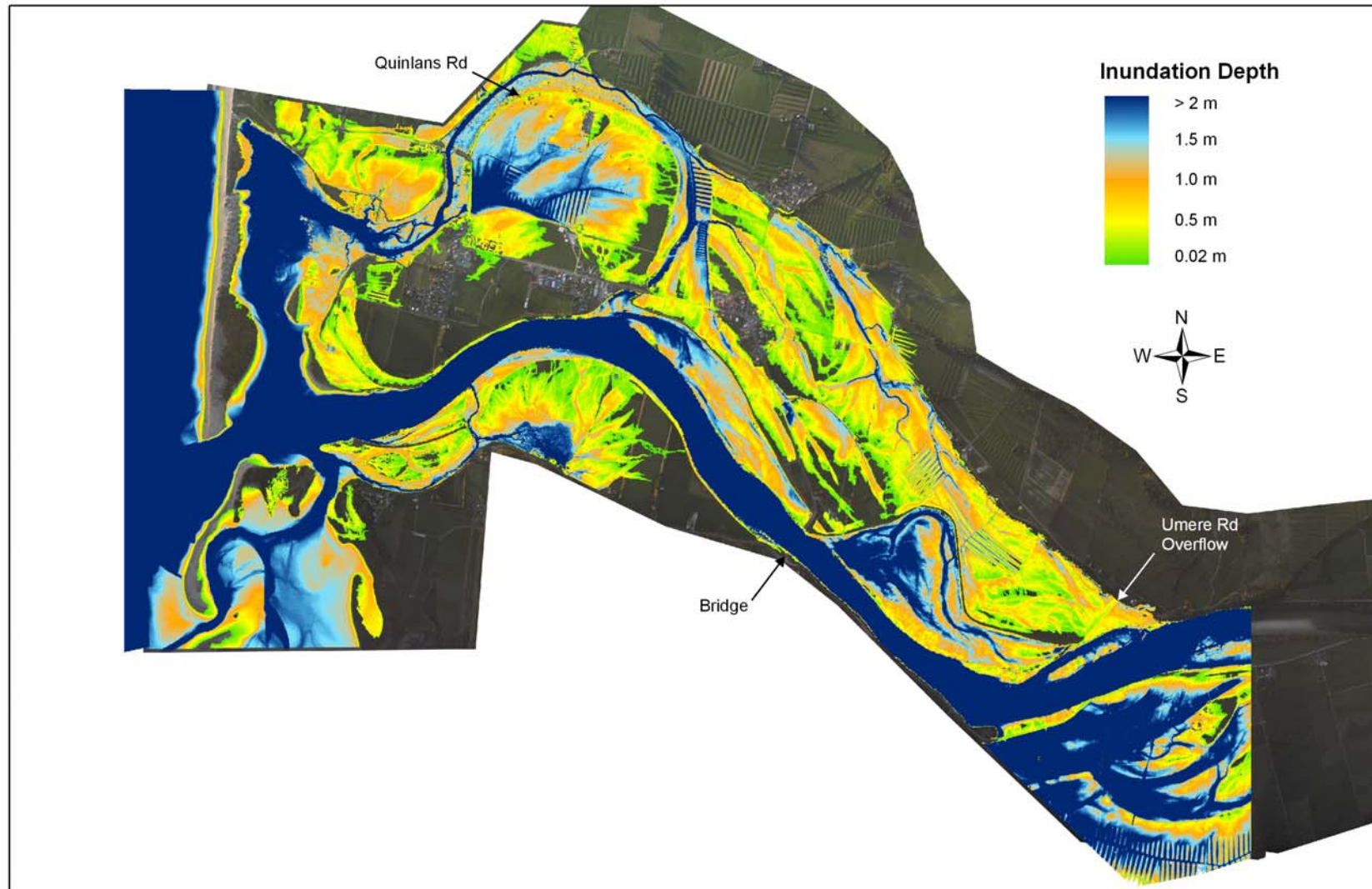


Figure 4: Maximum indicated inundation depths during a 1/20 AEP flood.

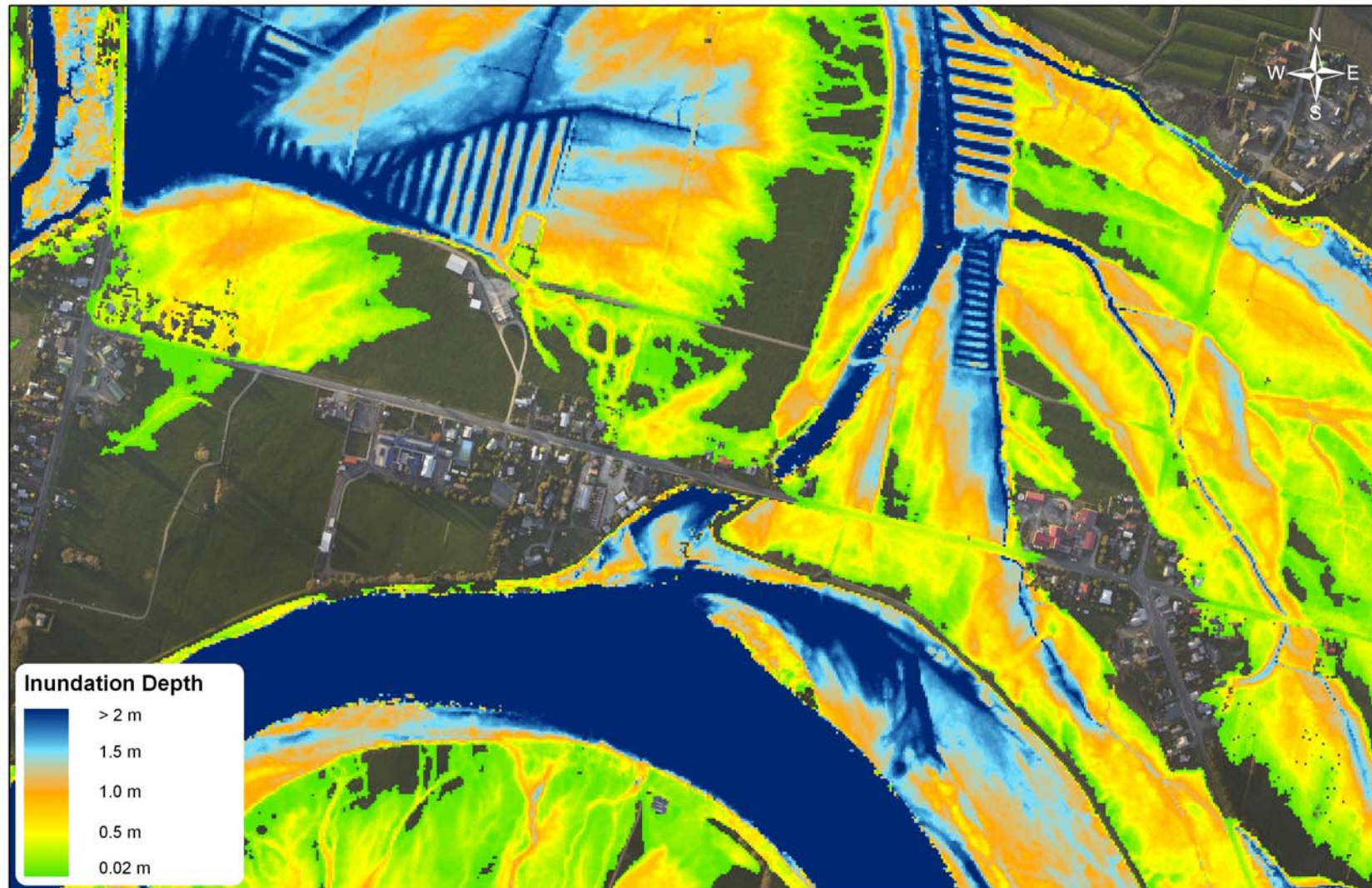


Figure 4a: Maximum inundation depths (in metres) near the township during a 1/20 AEP flood.

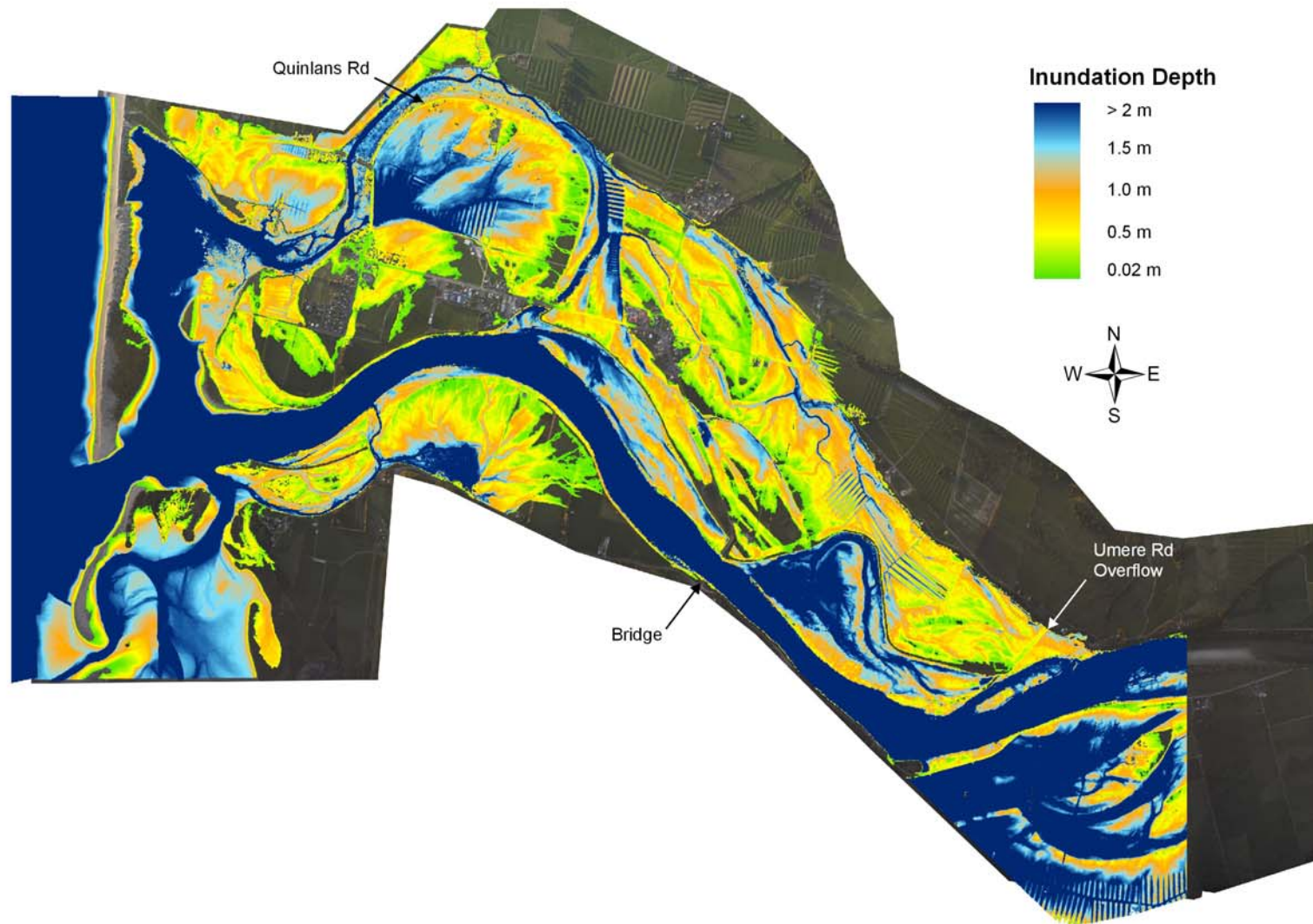


Figure 5: Maximum indicated inundation depths during a 1/50 AEP flood.

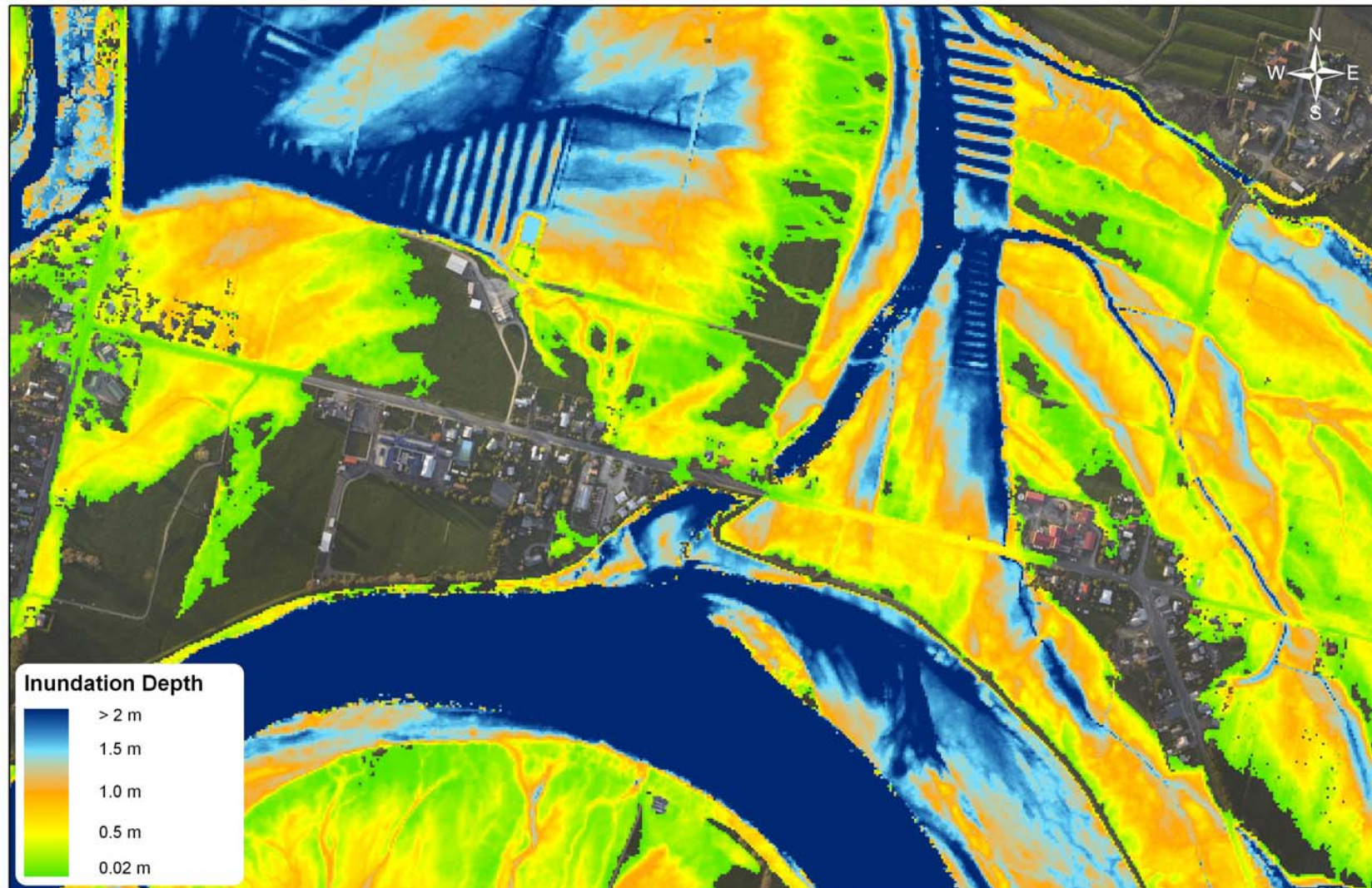


Figure 5a: Maximum indicated inundation depths (in metres) near the township during a 1/50 AEP flood.

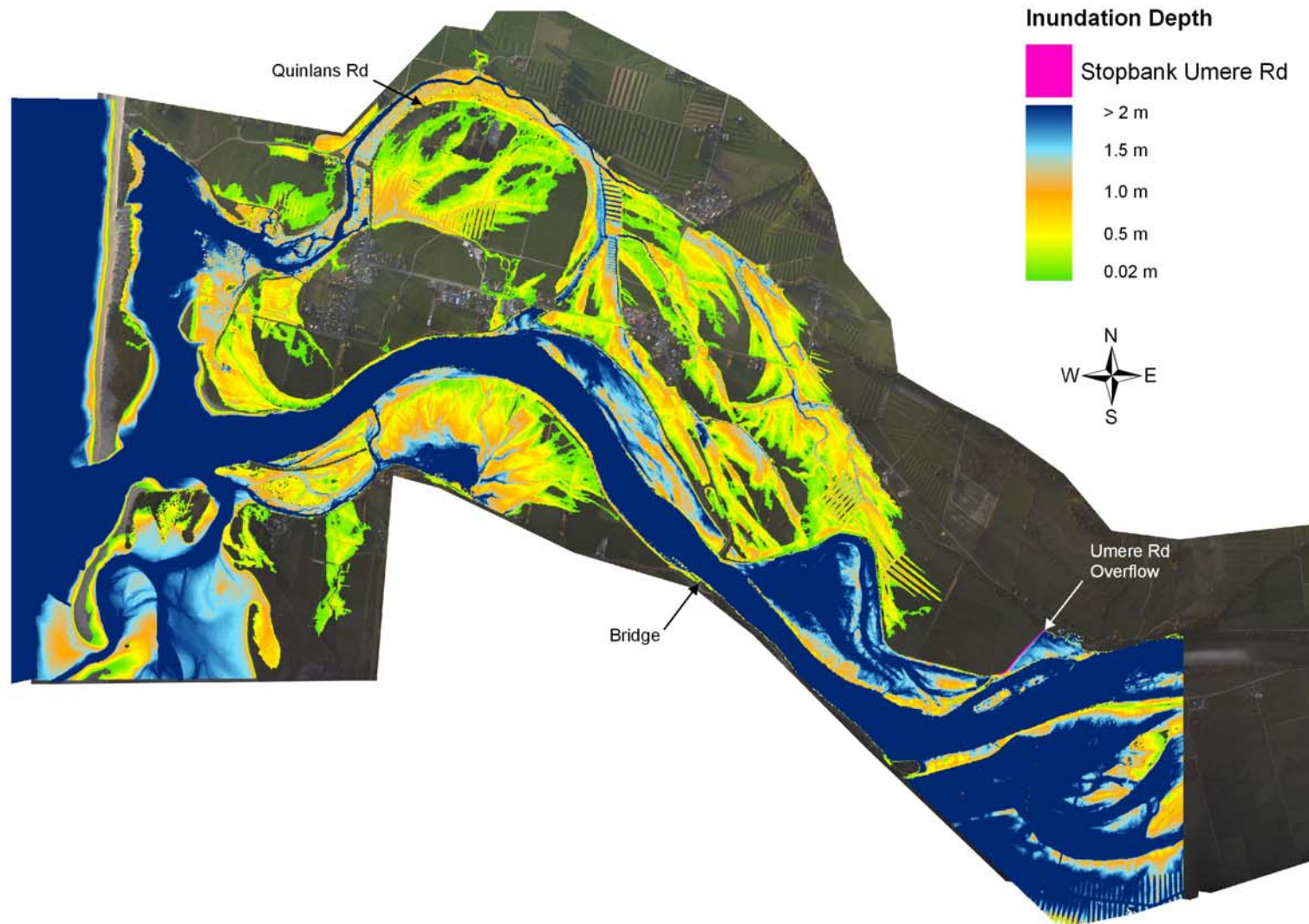


Figure 6: Maximum indicated inundation depths during a 1/50 AEP flood with a bank blocking the Umere Rd floodway.

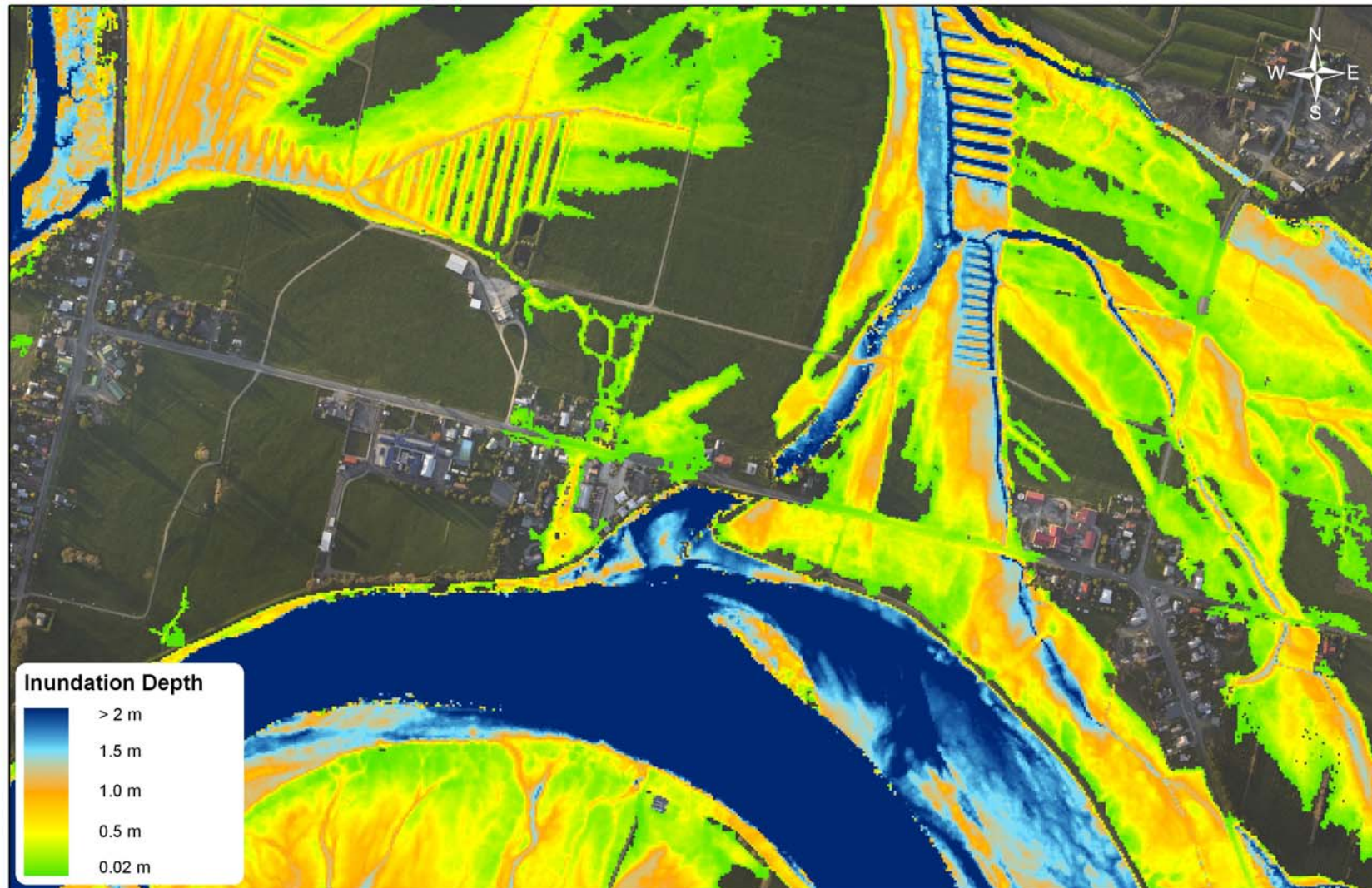


Figure 6a: Maximum indicated inundation depths (in metres) near the township during a 1/50 AEP flood with a bank blocking the Umere Rd floodway.

For the modelled hydrograph and tidal conditions the main settlements appear as dry islands. Because of the likelihood of flood bank breaching these apparently dry islands should not be regarded as safe zones.

With a new flood bank in position at the Umere Road overflow location (Figure 8) the indicated 1/100 AEP flooding appears to be significantly reduced and road access to the township appears marginal (Figure 8A). However, water spilling over flood banks upstream of the bridge is over 0.8 m deep. This will cause failure of the flood banks and very serious, rapid-onset flooding will occur so that the situation shown in Figure 8 and Figure 8a (which assumes no failure) is unlikely to represent the actual outcome in the event of such a flood.

Any proposal to raise flood bank crest levels should consider that the modelled water levels given in this report do not include any residual velocity head or freeboard allowance.

5. Recommendations for further work

A follow-up study should be undertaken to investigate effects of sea level rise (due to climate change) on flood levels. According to MfE (2008) at the very least, all assessments should consider the consequences of a mean sea-level rise of at least 0.8 m relative to the 1980–1999 average.

The systematic discrepancies that exist between the LiDAR levels and the surveyed flood bank levels (reported in the GEM, 2006 report) should be resolved if any construction or re-construction work is envisaged. The LiDAR levels (which are lower) were used in this report.²

Raising of flood bank levels upstream of the bridge will result in less spill from the river and give higher river levels downstream of the bridge. A modelling investigation should be undertaken if this is envisaged.

Further modelling could undertake sensitivity analyses to determine whether channel movements of the bar at the river mouth could affect upstream flood levels.

² A subsequent investigation by NZAM attributes the survey level differences to geoidal correction and recommends that the LiDAR levels should be used.

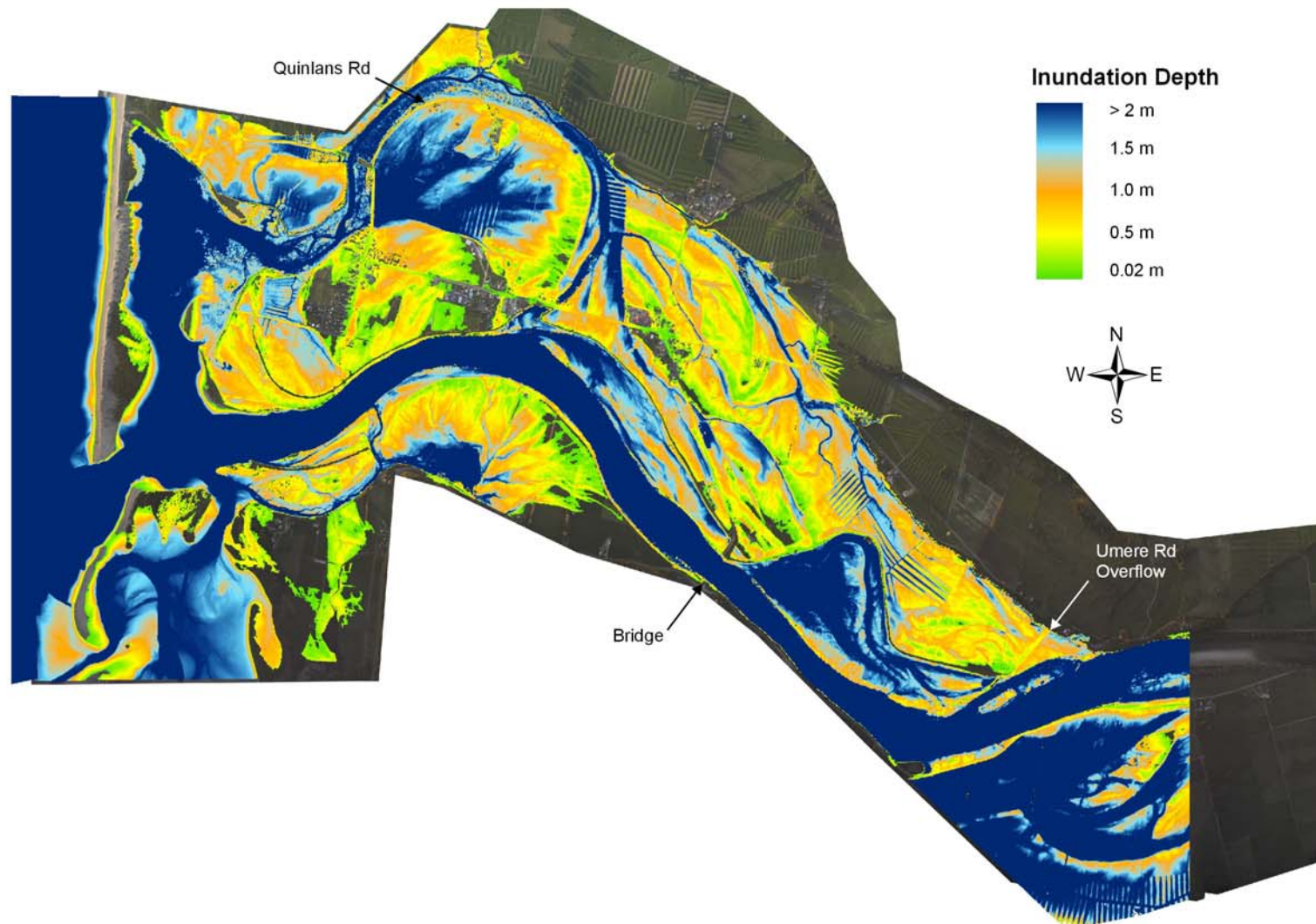


Figure 7: Maximum indicated inundation depths during a 1/100 AEP flood.

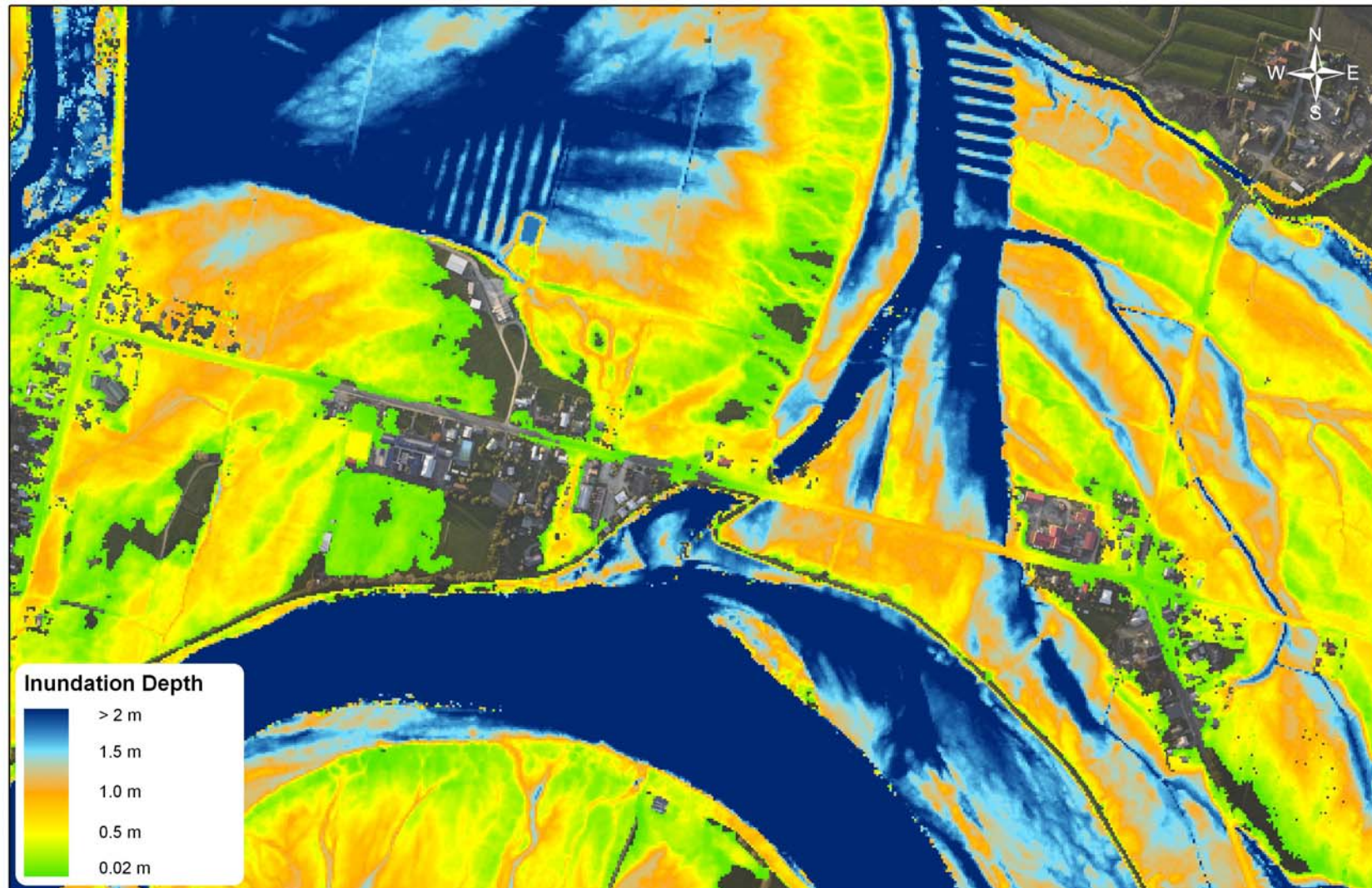


Figure 7a: Maximum indicated inundation depths (in metres) near the township during a 1/100 AEP flood.

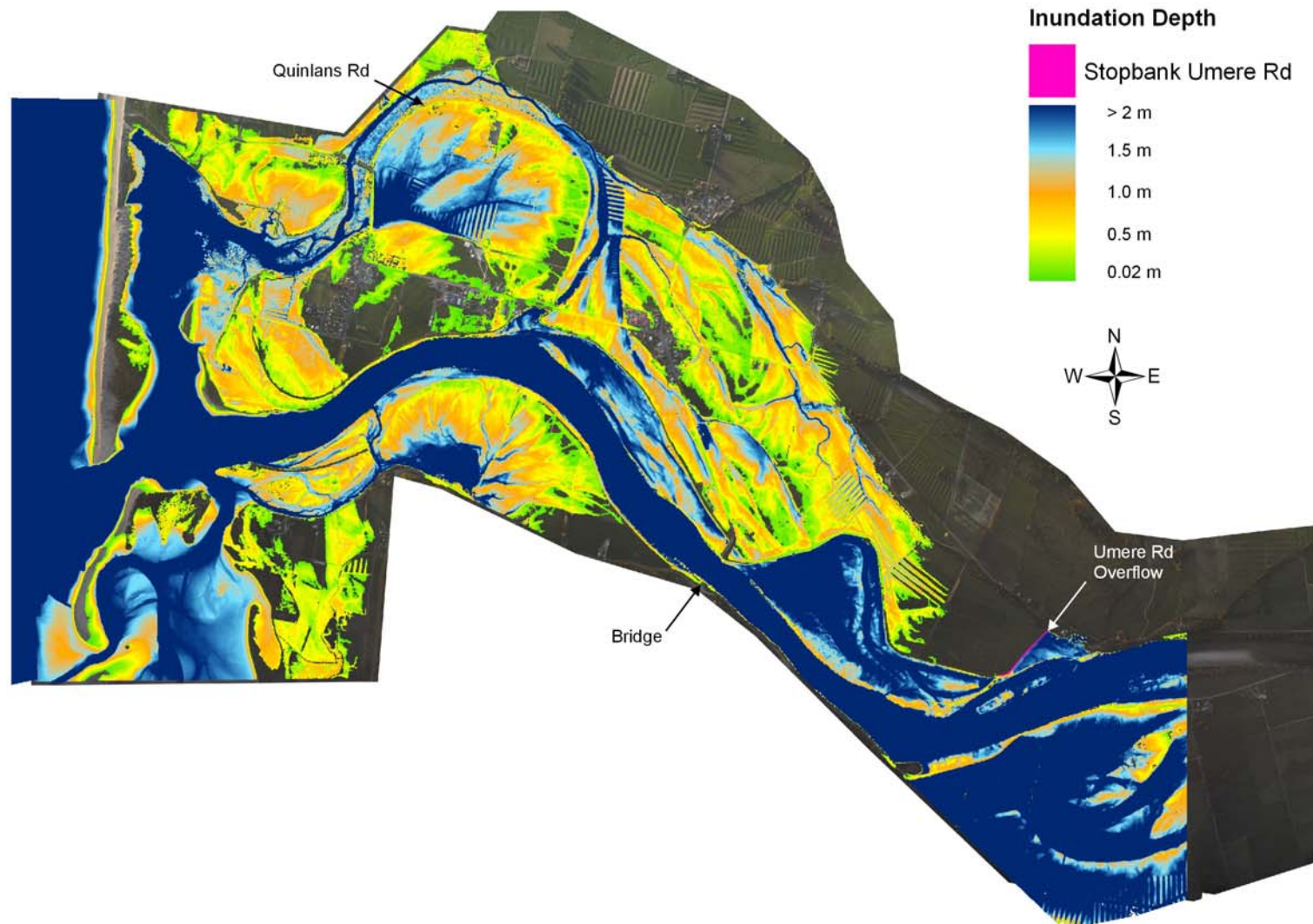


Figure 8: Maximum indicated inundation depths during a 1/100 AEP flood with a bank blocking the Umere Rd floodway. Flood bank failure will occur upstream of the bridge and cause more serious flooding than shown.

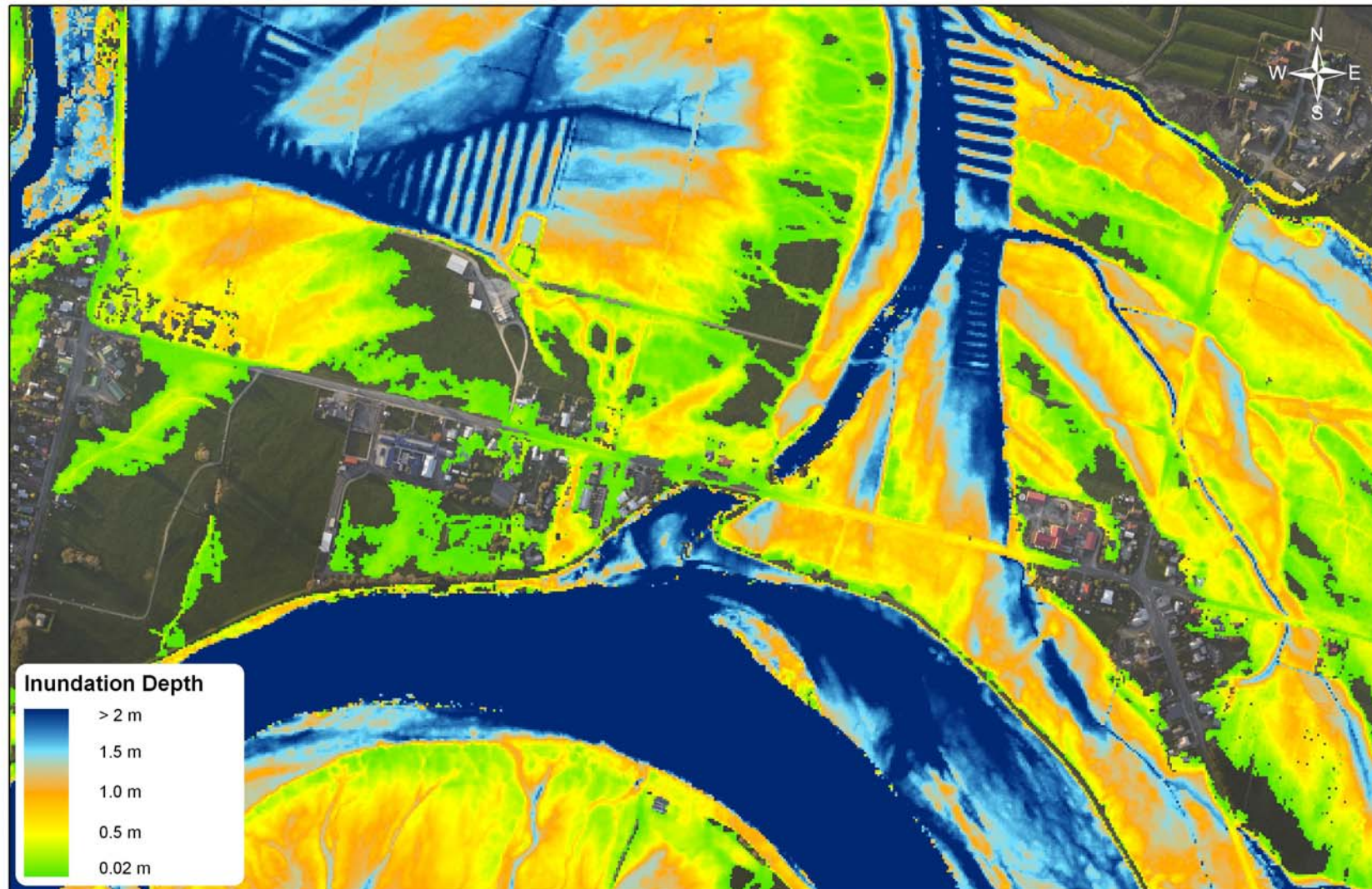


Figure 8a: Maximum indicated inundation depths (in metres) near the township during a 1/100 AEP flood with the Umere Rd floodway blocked. Bank failure upstream of the bridge will cause more serious flooding than is shown.

Flood banks to form a floodway that guides Umere Rd spillway water away from downstream properties could also be investigated.

If flood works are undertaken to give the townships a particular level of protection (say 1/100 AEP flood protection) an investigation of the effects of higher floods must also be made to determine residual hazards and risk should the design flood level be exceeded.

6. Acknowledgements

This study was instigated by the West Coast Regional Council and funded by Envirolink Medium Advice Grant 696-WCRC63.

7. References

Good Earth Matters (2006). WCRC Lower Karamea River Estimated Flood Levels.

Ministry for the Environment (2008). Coastal Hazards and Climate Change. A Guidance Manual for Local Government in New Zealand. 2nd edition. Revised by Ramsay, D.; Bell, R. (NIWA). Prepared for Ministry for the Environment. viii+127 p.

Appendix 1: Calibration flood

The calibration flood hydrograph and the corresponding tide levels are shown in Figure A1. The high resolution map of calibrated model surface roughness is given in Figure A2.

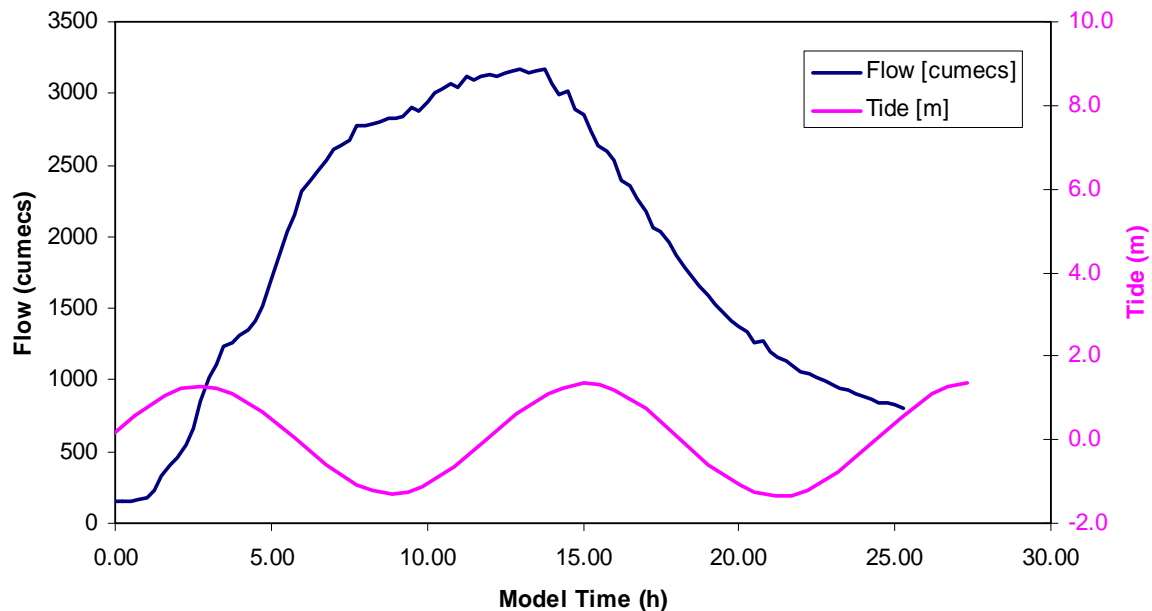


Figure A1: Modelled flood hydrograph and tides from 8 a.m. on 19 October 1998 with the flood peak at 9 p.m. on 19 Oct 1998.

Figures A3 – 11 show the modelled progression of flood waters as the October 1998 flood develops, peaks and recedes. The initial map (Figure A3) shows steady-state river conditions at a flow of 50 m³/s before input from the upstream flood hydrograph (Figure A1) has effect. The following maps are at 3-hour intervals. Enlargements at locations of river breakout are shown in Figures A7 and A9.

The inflows indicated on the maps apply at the upstream end of the model domain and these inflows take some time to propagate through the river system. Thus the inundation extent, at a given time, should be interpreted in conjunction with the flood hydrograph (Figure A1).

To verify the model, flooding results were discussed with residents who experienced the 1998 flooding. In comparing the results with what actually happened in 1998 it was borne in mind that the flood bank levels used in the modelling are those indicated at the time of the LiDAR flight in June 2008 which may differ from those that existed in October 1998 (the GEM, 2006 report mentions that flood bank crests are lower in a number of locations due to erosion by stock and vehicle crossings and there is also the

likelihood that repairs have been undertaken in places). The peak model-predicted water level under the middle of the road bridge is at RL 5.8 – 6.0 m. The observed water level under the bridge during the 1998 flood was around 7 m. The difference is likely due to river velocity head converting to static head as the water interacts with the bridge piles and embankment. Model river velocities of 4.4 m/s under the bridge indicate a velocity head of 1 m which brings the static water level to 7 m.

Flood development and peak levels corresponded very well with the recollections of residents and only minor adjustments to the calibration topography and roughness were made.

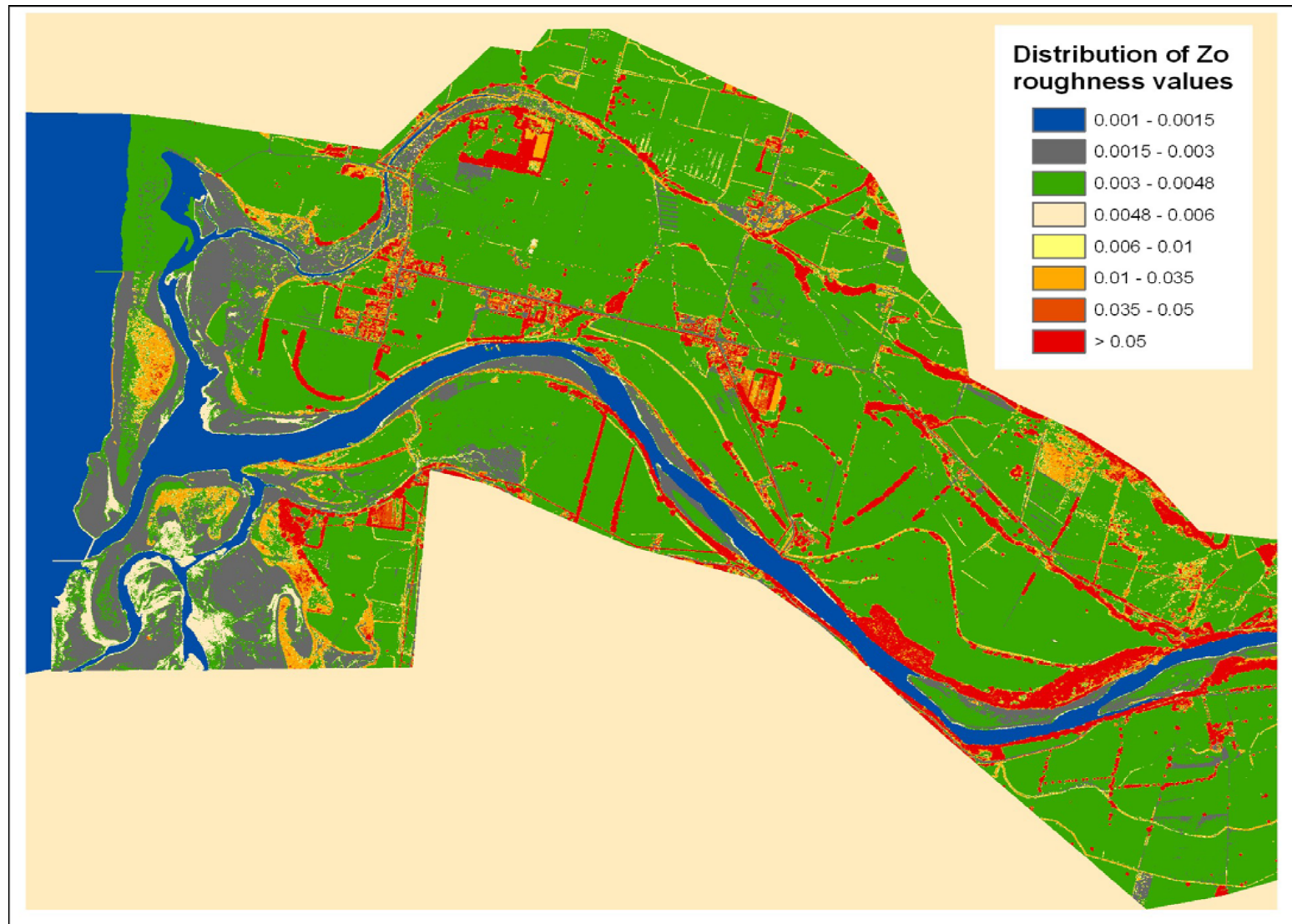


Figure A2: Modelling domain showing Z_o roughness values (in metres) used for model runs

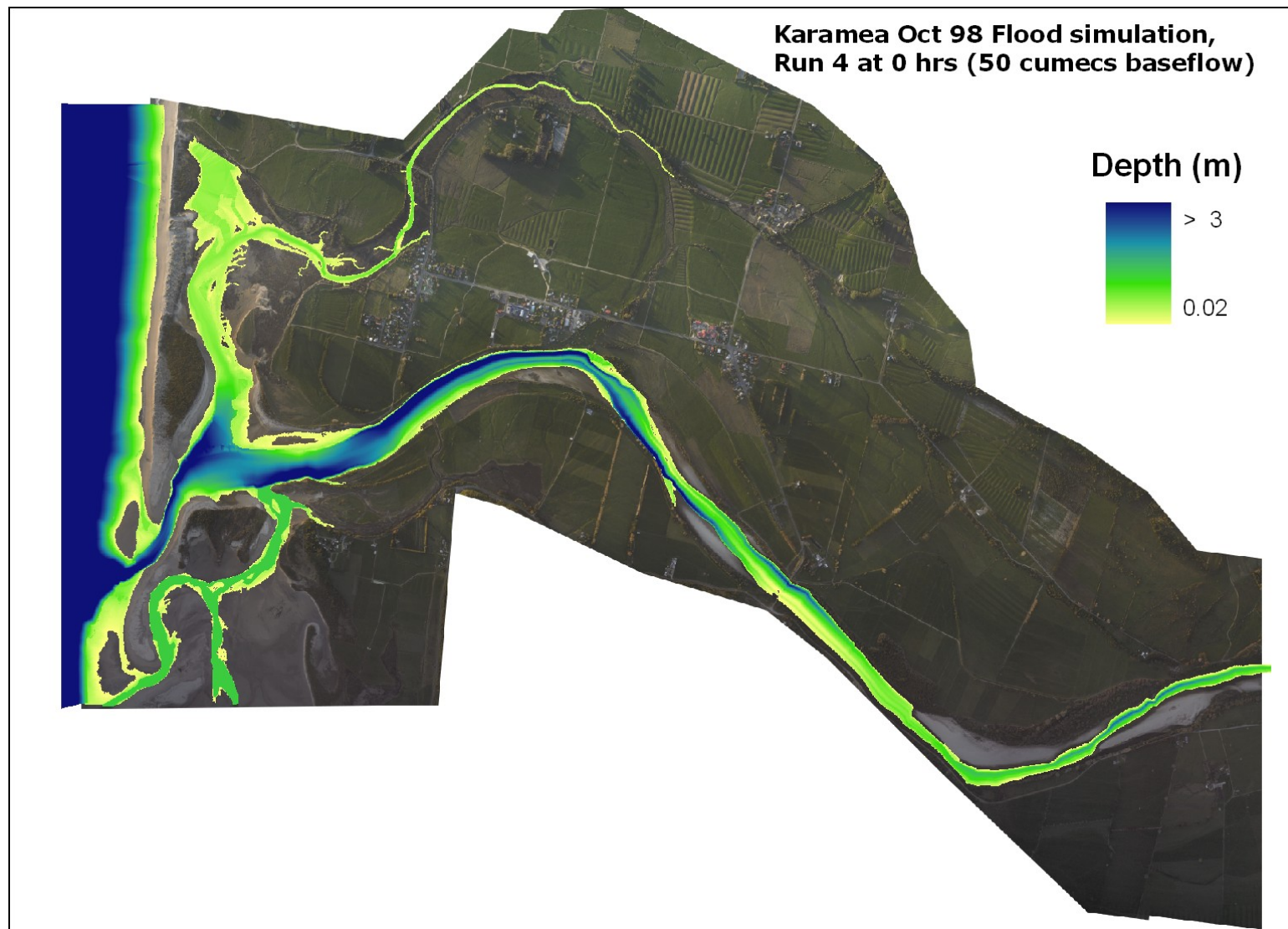


Figure A3: Flow depths with a steady inflow of 50 cumecs (calibration results, not suited for planning purposes).

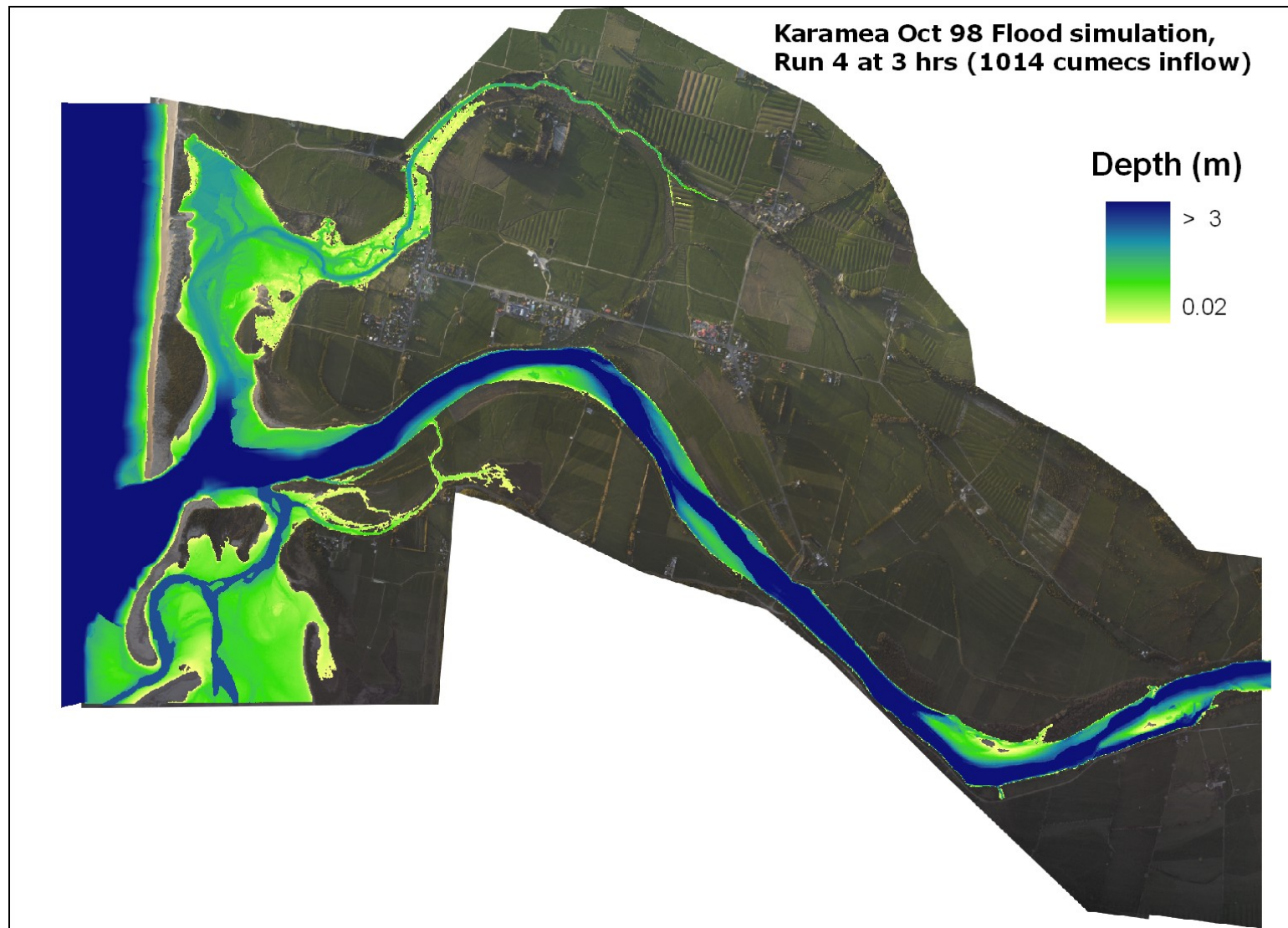


Figure A4: Flow depths 3 hrs into October 1998 hydrograph (calibration results, not suited for planning purposes).

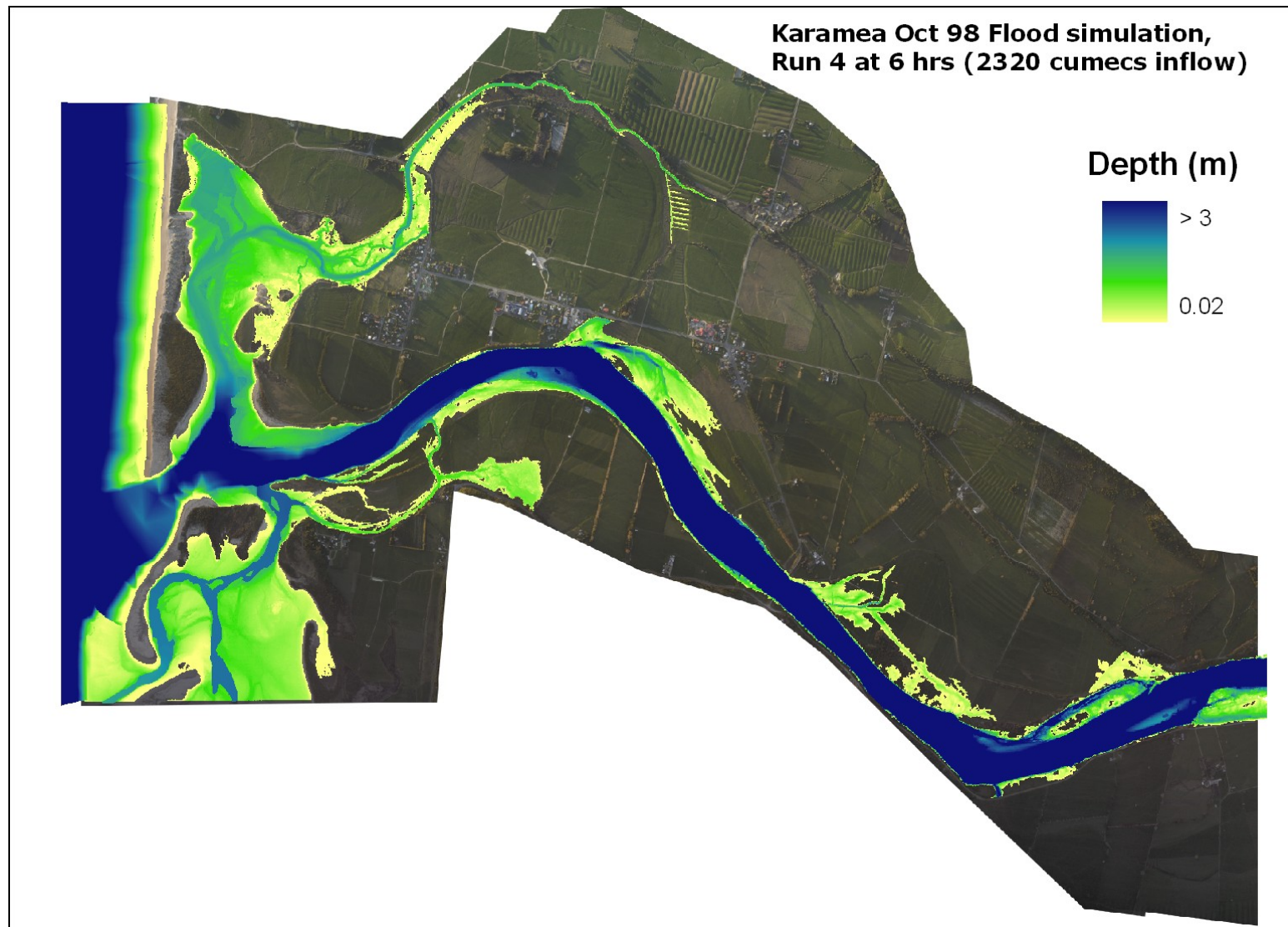


Figure A5: Flow depths 6 hrs into October 1998 hydrograph (calibration results, not suited for planning purposes).

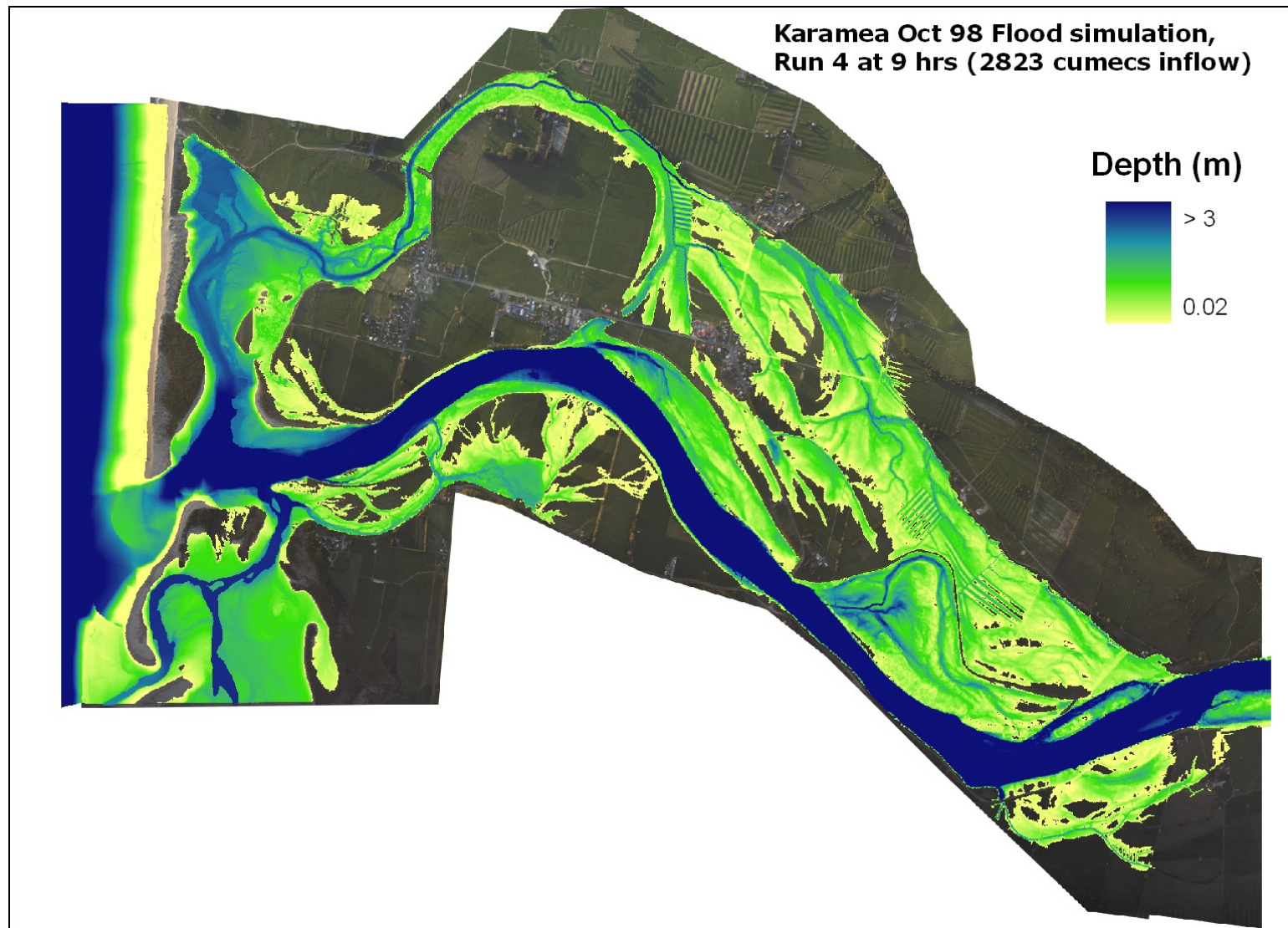


Figure A6: Flow depths 9 hrs into October 1998 hydrograph (calibration results, not suited for planning purposes).

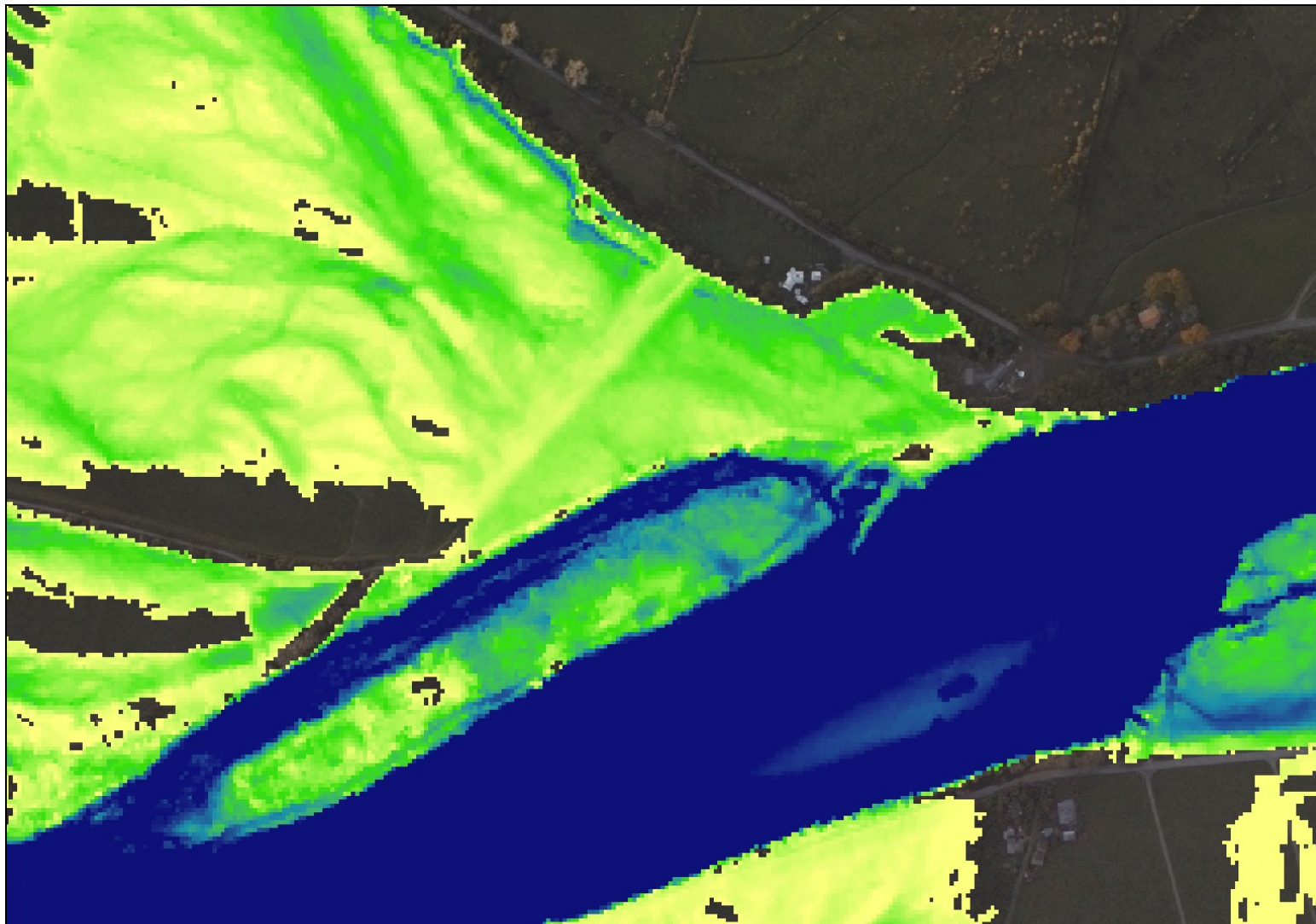


Figure A7: Detail at 9 hrs showing bank overtopping upstream near Umere Road (calibration results, not suited for planning purposes).

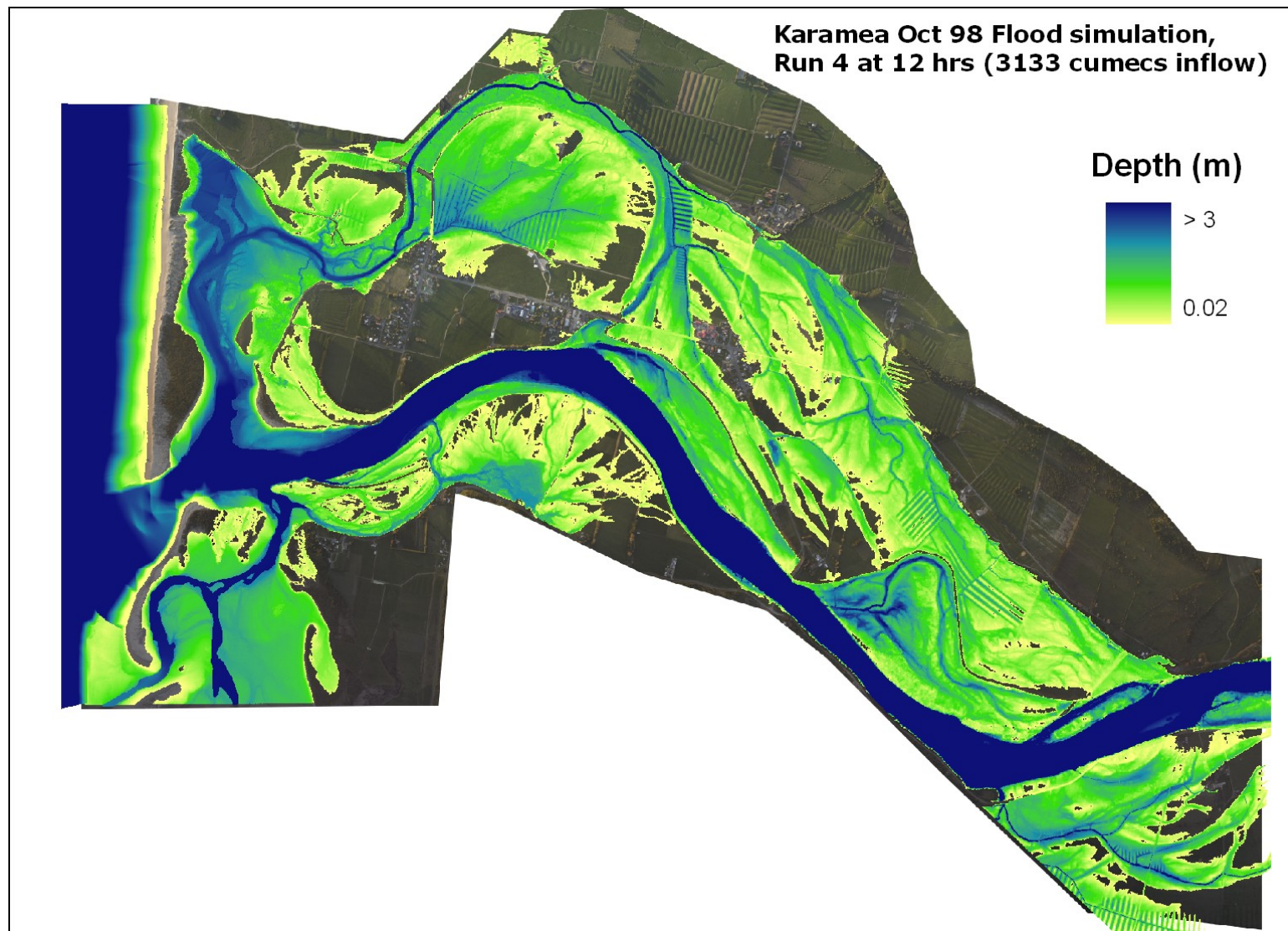


Figure A8: Flow depths 12 hrs into October 1998 hydrograph (calibration results, not suited for planning purposes).

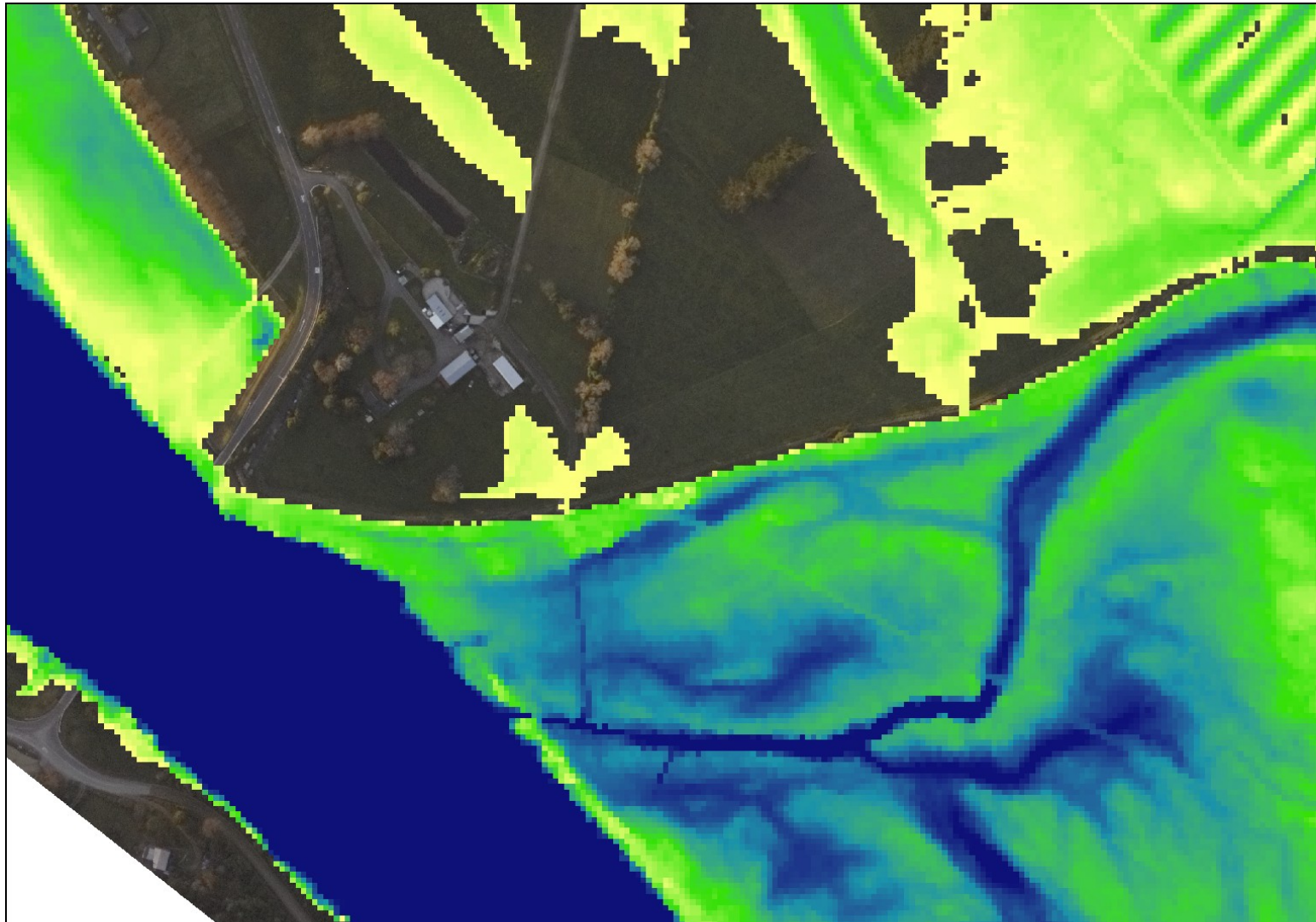


Figure A9: Detail at 12 hrs showing minor overtopping upstream of the Karamea bridge (calibration results, not suited for planning purposes).

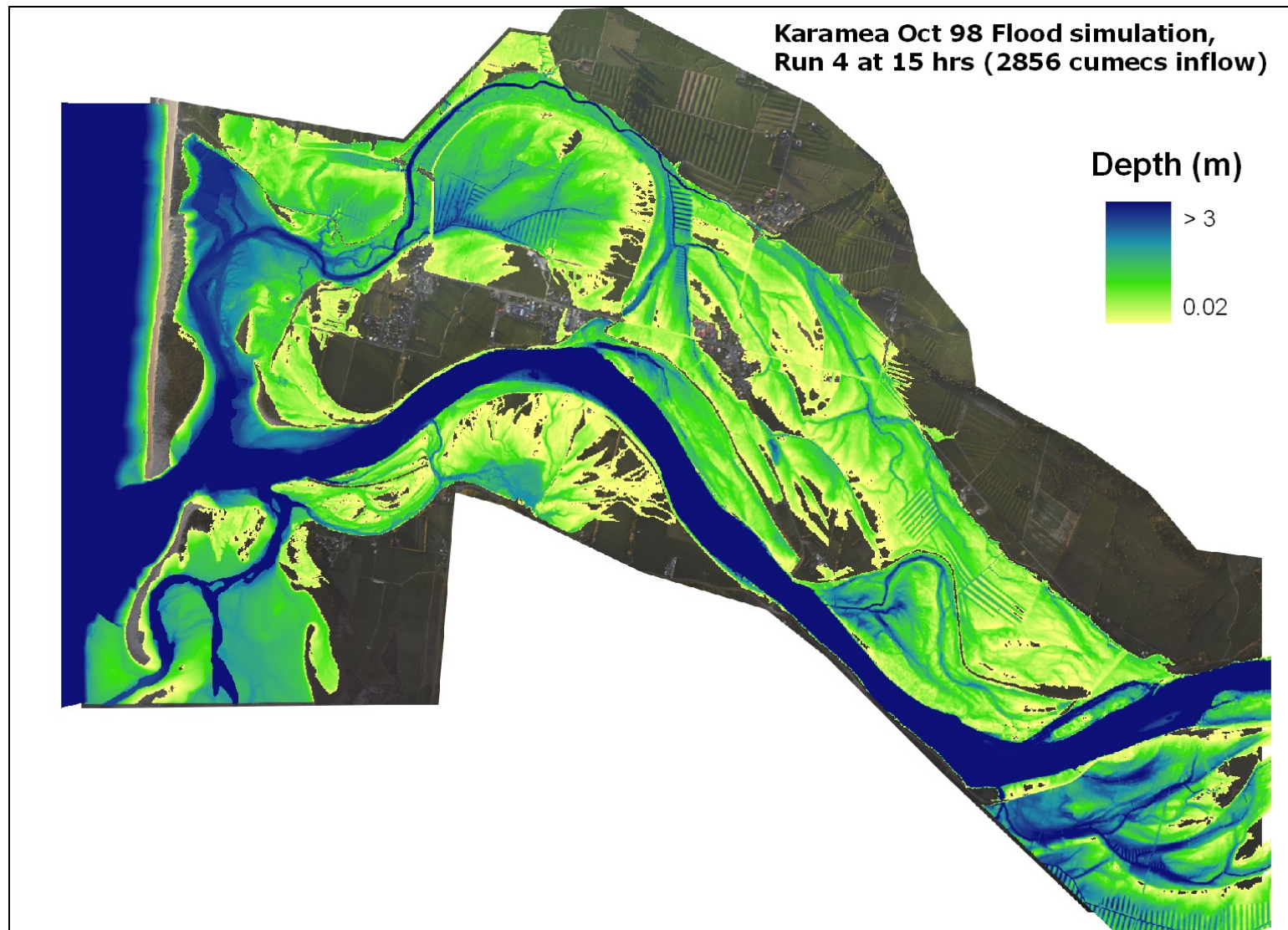


Figure A10: Flow depths 15 hrs into October 1998 hydrograph (calibration results, not suited for planning purposes).

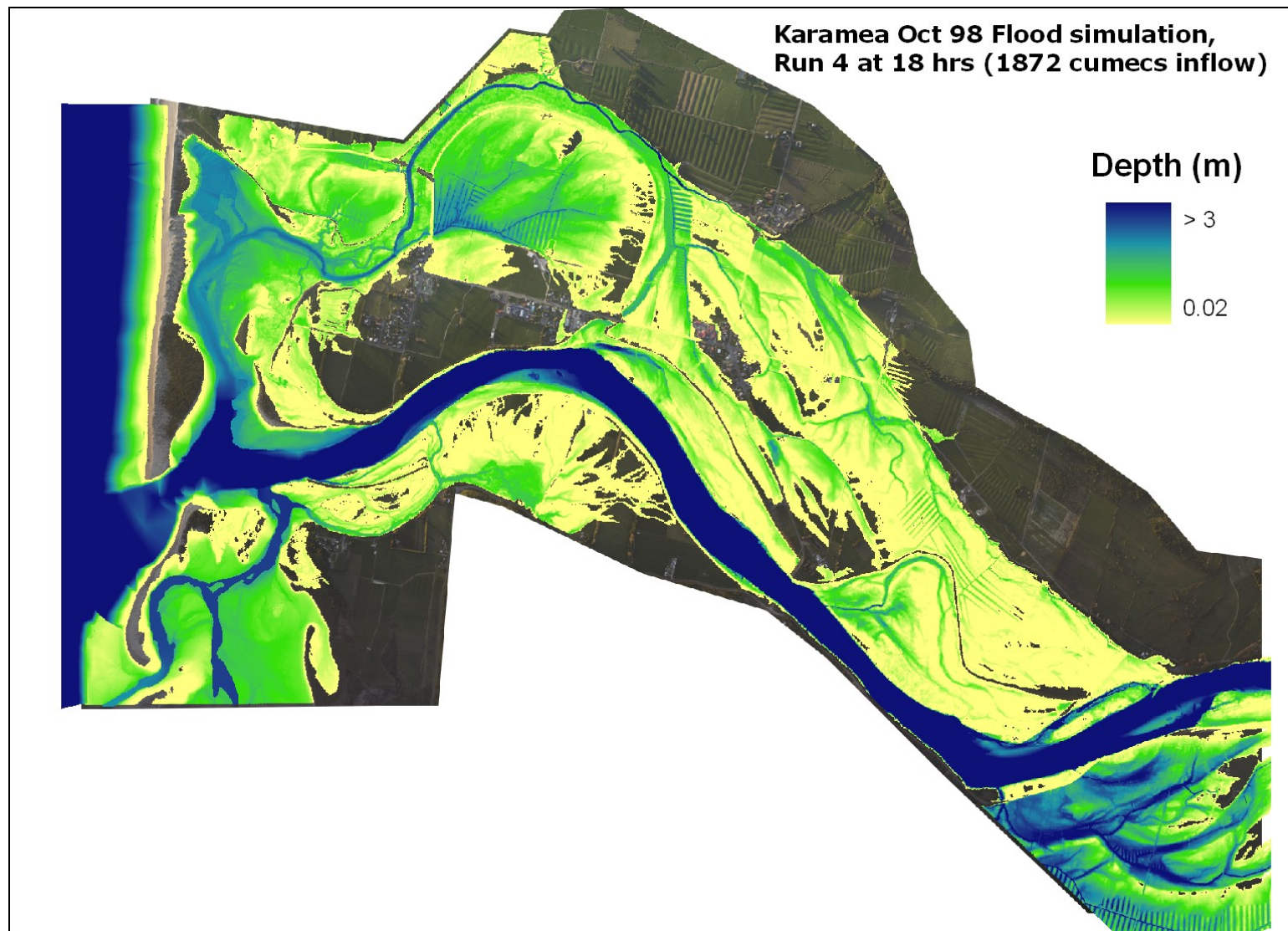


Figure A11: Flow depths 18 hrs into October 1998 hydrograph (calibration results, not suited for planning purposes).

Appendix 2: Quality Assurance procedures

1. The NZ Aerial Mapping LiDAR data were checked against ground survey points along the flood bank crest and discrepancies were found. The more conservative data were used.
2. The bathymetric survey endpoints were compared with the flood bank surveys and LiDAR to verify datum consistency.
3. The survey data and LiDAR were overlaid on a geo-referenced vertical aerial photograph to check that features were properly located on the model DEM.
4. Maps of model hydraulic resistance were reconciled with aerial photographs and land use maps.
5. The October '98 flood with a peak flow of 3163 cumecs was run through the hydraulic model. Small adjustments to model roughness were made to agree with water levels observed by local residents at the time of the flood. No major changes were required.
6. This report was internally peer reviewed.