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Ashley River floodplain investigation - 2016 update

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July 2016

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Summary

Background
A number of Ashley River floodplain investigations have been carried out in recent years, with the last being in 2008. This investigation is an update of the 2008 work.

What we did
This study of the Ashley River floodplain uses a combined one and two dimensional hydraulic computer model to estimate the extent and depth of flooding on the Ashley River floodplain. It utilises ground level (LiDAR) data acquired in 2014 and accounts for other changes to flood protection and drainage (i.e. the new stopbank adjacent to Rangiora).

Stopbank breaches and outflows onto the floodplain can potentially occur anywhere along the stopbanked reaches of the river. To handle this situation, the method employed was to study the floodplain in detail geomorphically and then divide the stopbank reaches into zones (recognising that regardless of where in the zone the breach occurs, the effects on the floodplain are virtually the same). Breaches were then assigned to each of the zones.

What we found
Modelling indicates that the current capacity of the Ashley River stopbanking system is within the range of a 50 to 100 year ARI flood event. However, breaches could occur in more frequent events due to erosion, piping or seepage mechanisms.

The modelling also illustrates significant flooding to large areas of land between the Ashley and Waimakariri Rivers. Large parts of Kaiapoi, and adjacent areas, are predicted to be flooded to depths over 1 m in a 500 year ARI event.

What does this mean?
The flow and water depth information from this modelling investigation can be used for land use planning and to provide information on minimum floor levels for new dwellings located on the floodplain.

For land use planning purposes, modelling results should only be interpreted and used by those who are familiar with the modelling.

January 1953 Flood - Looking across the Ashley River at flooding between Waikuku Beach and Woodend Beach. Floodwaters extended through to the Pines and Kairaki. Photograph by L Ernle Clark for North Canterbury Catchment Board.

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1 Introduction

The Ashley River floodplain covers ~190 km^2 and includes the two large urban areas of Rangiora and Kaiapoi.

Previous investigations into flooding on the Ashley River Floodplain include:

<u>1995 - Ashley River Flood Plain Management Plan Technical Investigation (CRC, 1995)</u>

This investigation highlighted potential breakout risk, and predicted overland flow paths and ponding areas, using a one dimensional hydraulic computer model (Mike 11).

<u>2003 - Waimakariri District Flood Hazard Management Strategy</u>

The Waimakariri District Council and the Canterbury Regional Council adopted this strategy in 2003. Mitigation measures in the strategy included maintenance of the existing stopbank system and land use management measures, such as appropriate development in flood prone areas and appropriate minimum floor levels for new dwellings.

<u>2008 - Ashley Floodplain Hazard Risks Assessment, Report U08/1 (Boyle & Surman, 2008)</u>

This report assessed the probability of Ashley River breakouts (outflows) at various locations.

<u>2008 - Waimakariri District Flood Hazard Management Strategy – Ashley River floodplain investigation, Report R08/23 (Oliver, 2008)</u>

This investigation used very detailed topographic data obtained from a 2005 LiDAR (aerial laser) survey together with a more physically realistic two dimensional hydraulic computer model (Mike 21) of the floodplain. This enabled more informed decisions to be made regarding land use management of the floodplain, and a more accurate assessment of appropriate minimum floor levels for new dwellings. The effects of various development scenarios (i.e. filling) on the floodplain in the Kaiapoi area was also able to be assessed.

<u>2015 – Waimakariri District Localised Flood Hazard Assessment – Waimakariri District Council PDU Project Number PD000362 (WDC, 2015)</u>

The purpose of this investigation was to model the effects across the District for 100, 200 and 500 year average recurrence interval (ARI) rain events and produce flood maps.

This report is essentially an update of the 2008 Ashley River floodplain investigation (Report R08/23). Improvements and updates made to the original model include:

- Use of the most recent 2014 detailed topographic data (LiDAR) for the model grids. This survey takes into account changes in ground levels due to the 2010 and 2011 earthquakes, plus recent urban developments.
- Inclusion of ‘lateral links’ in the models to represent the transfer of water between the lower Ashley River northern bank and the Saltwater Creek lagoon area (where the earlier northern floodplain model used a one-dimensional channel connection), and between the Ashley River south bank and the secondary flowpath between the existing and new stopbanks (upstream of the Rangiora railway bridge).
- Inclusion of the newly constructed Ashley River stopbank, located to the north of Rangiora, crossing Millton Avenue.
- Inclusion of many of the main culverts located on the floodplain.

Model results are provided in the form of floodplain inundation maps showing maximum water depths, elevations and velocities for the various breach locations and flows. The southern and northern Ashley River floodplain areas included in this investigation are shown in Figure 1-1. This report summarises the modelling methodology and results.

Figure 1-1: Location map of northern and southern Ashley River floodplain areas

2 Background

2.1 Study area

The Ashley River is a relatively steep braided river. Together with its major tributary, the Okuku River, the Ashley River drains a catchment of ~1200 km^2. The catchment is relatively steep with the highest point in the Puketeraki Range at an elevation of over 1900 m.

On the south bank of the Ashley River, the unconfined floodplain commences immediately downstream of the Okuku River confluence. Numerous breakouts and changes of course, over thousands of years, have resulted in alluvial deposition and the formation of the adjacent alluvial fans that extend to Kaiapoi and the Waimakariri River.

On the north side, the Ashley River is confined by a prominent 5-10 metre high terrace. This terrace extends downstream from the Okuku River as far as Ashley Township. Downstream of Ashley Township the river is confined by stopbanks, but floodwaters can break out onto the Ashley Fan.

A full description of the Ashley Floodplain geomorphology can be found in CRC (1995) and a map of the Ashley Floodplain geomorphology is shown in Appendix A.

2.2 Historic flooding

Floods in the Ashley generally result from north-easterly or south-easterly weather systems and orographic rainfall in the hill and mountain headwaters. Heavy rain from decaying tropical depressions can also penetrate the catchment from the north east.

Records from early European settlers and newspapers indicate that flooding from the Ashley River has always been a problem. The New Zealand Rivers Commission, reporting on the Ashley in 1921, stated that

"… evidence showed that in times of very high flood the water has escaped the Ashley River and run into the low country lying to the north and west of Kaiapoi…. The flooding of the adjacent lands is caused by the fact that along some portions of the river the natural banks are lower than the grade of a high flood, and furthermore by the fact that the river, in common with most other Canterbury rivers, is running on a "fan", and once the floodwaters get over the immediate river bank they tend to follow old channels that lead away from the main river and do not return lower down, as is the case with valley rivers."

Further on, in discussing the flood hazard to which Kaiapoi has been subjected, the Rivers Commission also make important observations with regard to the Ashley when they state that

"in addition to these floods (of 1868 and 1987, emanating from the Waimakariri) Kaiapoi has been inundated by local floods from the Eyre and Cust Rivers, and also by flood overflows from the Ashley River. As a rule floods in the Eyre, Cust and Ashley Rivers do not synchronise with those of the Waimakariri, but this happened in 1868 and 1905."

Numerous floods were reported in the latter half of the 19th century, but the largest appears to have been the February 1868 event. The Ashley River "*broke its banks*" flooding Rangiora and floodwaters extended all the way to Kaiapoi. Two children were drowned near Rangiora and "*Kaiapoi suffered severely with water up to 5 ft 6 inches (1.7 m) in some streets, and almost every shop and house was invaded with consequent heavy losses…*"

The flooding of Kaiapoi was due to, in the main, overflows from the Cust, Eyre and Ashley Rivers, as well as flooding from the Waimakariri River.

Further significant floods were recorded in the early 20th century, particularly March 1902 (Figure 2-1), June 1905 and May 1923. The 1923 flood was reported as "*the most disastrous since 1868… A large amount of damage was done to Kaiapoi and Rangiora was similarly affected… Between Kaiapoi and Southbrook the country is a sea of water, in some places up to the tops of the fences… the main road* *from Flaxton to Southbrook was transformed into a river of water from fence to fence*…" Kaiapoi was again flooded as a result of a breakout from the Ashley above Rangiora. "*Rangiora suffered some local (surface) flooding but Kaiapoi was inundated. Two hundred families were evacuated from their homes and the Revell family marked a new flood level at their home fifteen inches above the 1868 mark*." At Waikuku the wool works were flooded to a depth of 8 ft (2.4 m).

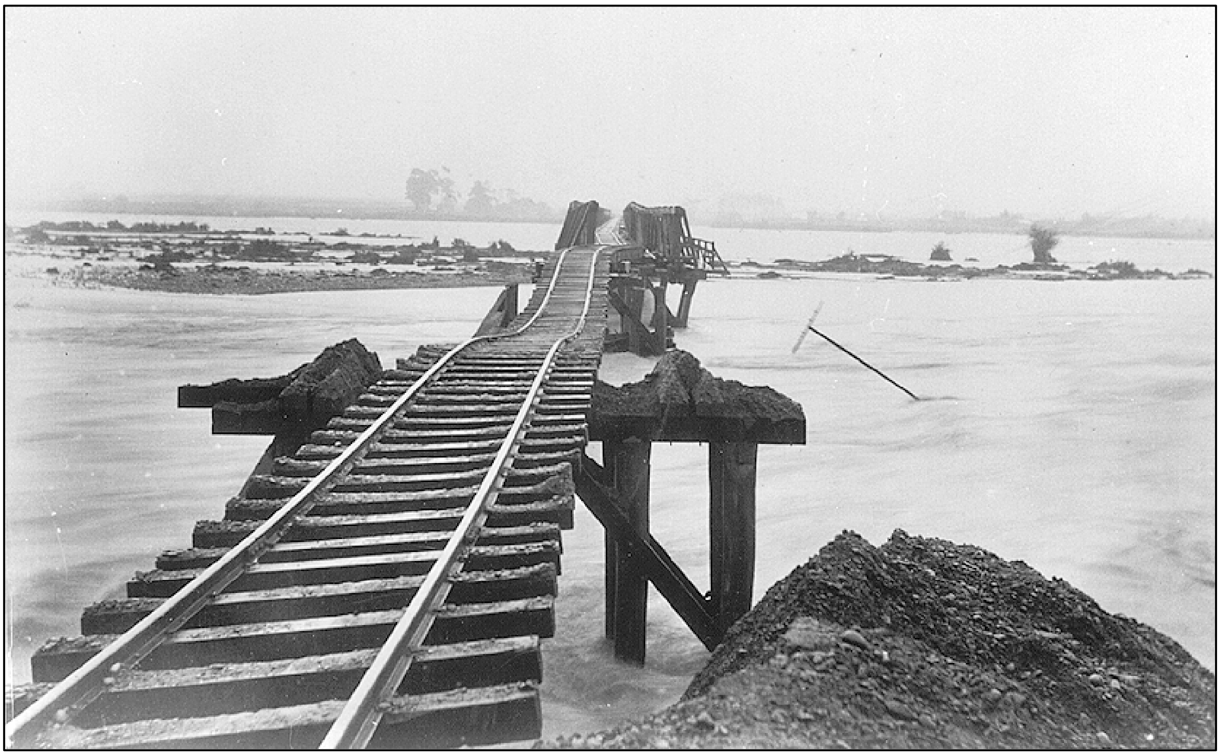


Figure 2-1: Damage to the Ashley River railway bridge in the March 1902 flood

Following this devastating flood, the original Ashley River "Flood Protection Scheme" was constructed in the 1930s. However the partially completed stopbanks were breached in at least three places by a large flood in February 1936. The most serious flood damage was at Waikuku and up to 9ft (2.7 m) at the Waikuku Wool Works. After this flood the *"height of the stopbanks was increased and belts of willows were planted as an additional defence"*.

Despite the completion of the original scheme in 1938, the flood of March 1941 resulted in the Ashley River *"breaking its banks in four or five places"*. Again this caused major inundation of farm land, stock losses, flooded houses and occupants being rescued by boat. *"Mr Wylie had to be rescued by boat, from a haystack"* and on the south bank (at the same time) an elderly woman, Miss M Leggatt, was rescued by boat from the upper story of her home. Both had been isolated for two and a half hours.

The stopbanks were again breached in 1945 (Figure 2-2 to Figure 2-4), 1951 (Figure 2-5) and 1953 (Figure 2-6). These events all caused widespread flooding and major flood damages, and evacuation of people from their homes. In the 1953 flood event, Waikuku and Woodend Beach were flooded and floodwaters flowed through the Pines and Kairaki. A Woodend farmer spent 12 hours perched 14 feet up on a pine tree, until rescued by the Lyttelton Harbour Board. Floodwaters from the Ashley also caused the Cam River to overflow its banks and threatened the low-lying Camside area of Kaiapoi.

There have been a number of reasonably large river flows since 1953, with the largest being December 1993 (~1900 m^3/s). Although the stopbanks have been close to breaching/overtopping a number of times, there have been no breakouts onto the floodplain since 1953.

In addition to flooding from the Ashley River, significant flooding has occurred from local rainfall events and from waterways such as the Cam and Cust rivers.

Figure 2-2: Surface flooding in High Street, Rangiora in the February 1945 flood

Figure 2-3: Kaiapoi River (Williams Street) in flood in the February 1945 flood

Figure 2-4: Kaiapoi in the February 1945 flood

Figure 2-5: Rangiora – Loburn traffic bridge at the height of the flood on 25 January 1951 [Donated by Mrs C Tyler to Rangiora and Districts Early Records Society]

Figure 2-6: The Coldstream area showing the major overflows, and flooding, between the Rangiora Golf Course and the sea on 26 January 1953 (looking east). Coldstream Road is on the left. [Photo: L Ernle Clark for North Canterbury Catchment Board]

2.3 Ashley River control scheme

The original stopbank scheme, built in the 1930s, was designed to contain 2000 m^3/s (with no freeboard). The present scheme was designed and constructed by the North Canterbury Catchment Board in 1976 to contain 2400 m^3/s with a 600 mm freeboard, which at the time was estimated to be a 100 year Average Recurrence Interval (ARI) flow. However, based on the latest flood frequency review for the Ashley River (Griffiths *et al.,* 2009), a flow of 2400 m^3/s is now estimated to have between a 20 to 50 year ARI.

In 2003, the Waimakariri District Floodplain Management Strategy was agreed. It included intentions to:

- Provide flood protection works for the Ashley River by increasing the channel capacity of the river to 3000 m^3/s with 600 mm freeboard (~50 year ARI).
- Ensure maintenance of appropriate channel capacity for the river (e.g. by managing gravel extraction, planting, vegetation, etc).

The latest bed level investigation (Boyle and Surman, 2009) recommended target bed levels consistent with a 3250 m^3/s flow and recommended this flow (with 600 mm freeboard) be adopted as a design flow. Therefore, for much of its length, the Ashley River control scheme has a design capacity of ~3000 to 3500 m^3/s (plus freeboard). Griffiths *et al*. (2009) estimates this to be equivalent to a 50 to 100 year ARI flow.

The Ashley River Control Scheme includes 34.7 km of stopbanks, groynes, tree planting and rock protection. The stopbanks are continuous on the north banks from the mouth to the main trunk railway at Rangiora (where it meets a natural terrace). On the south bank the stopbanks are continuous from the mouth to Sunken Road, opposite the Okuku River confluence (Figure 2-7).

Annual maintenance is undertaken to control vegetation and shingle, which could otherwise reduce the capacity of the river channel. More information about the river control scheme is available in the Canterbury Regional Council Asset Management Plan for this scheme (CRC, 2014).

A new stopbank has been constructed upstream of the railway line, on the true right bank of the Ashley River (Figure 2-7). This stopbank is designed to divert any breakout flows at this location back into the Ashley River, rather than allowing them to flow across the floodplain. At present there are still gaps in the stopbank where it crosses roads, but this is intended to be resolved in the 2017/18 financial year.

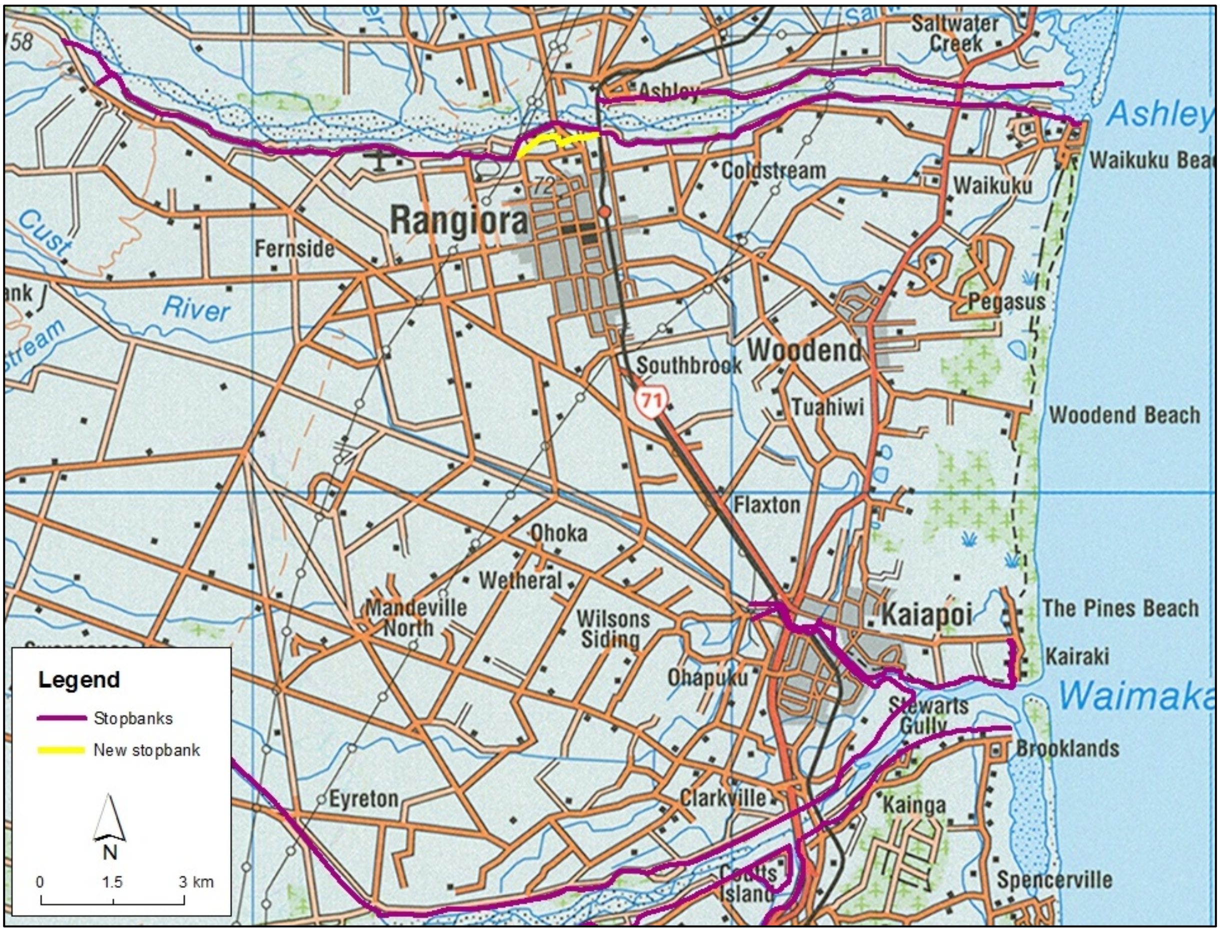


Figure 2-7: Location of Ashley, Waimakariri and Kaiapoi River stopbanks

2.4 Climate change

'Compared to 1995, temperatures are likely to be 0.7°C to 1.0°C warmer by 2040 and 0.7°C to 3.0°C warmer by 2090' (http://www.mfe.govt.nz/climate-change/how-climate-change-affects-nz/how-might-climate-change-affect-my-region/canterbury, accessed June 2016).

As the atmosphere warms, it can hold ~ 8% more moisture for every 1°C increase in temperature (MfE, 2010). This is likely to lead to more intense rainfall. For example, a 2°C increase in rainfall by 2090 could increase extreme rainfall intensity by up to 16% (MfE, 2010).

For the purposes of this investigation, the flow increases due to climate change are considered to be within the error estimates of the more conservative design flows used in Oliver (2008), compared to the lower Griffiths *et al.* (2009) design flows (see discussion in Section 3.2). In addition, there is the uncertainty of the breakout magnitude, location, and duration. Therefore, the adopted design flows do not specifically include an allowance for climate change. However, a sensitivity test with breakout flows increased by 20% has been modelled to show the sensitivity of the model to breakout flow magnitudes. This is discussed in Section 3.4.6.

Over the 20th century, relative mean sea level has increased by 0.16 m around New Zealand (MfE, 2008). Based on the Intergovernmental Panel on Climate Change (IPCC) Fourth Assessment Report, MfE (2008) recommends that '*for planning and decision timeframes out to the 2090s (2090 – 2099):*

a. *A base value sea-level rise of 0.5 m relative to the 1980-1999 average should be used, along with*
b. *An assessment of the potential consequences from a range of possible higher sea-level rises (particularly where impacts are likely to have high consequence or where additional future*

> *adaptation options are limited). 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."*

Sea level rise scenarios have been modelled, and these are discussed in Section 3.4.6.

3 Methodology

The previous Oliver (2008) floodplain investigation required information from several other studies before the modelling of floodplain inundation could be completed. Various land use impacts were also modelled. A summary of the relevant information from these studies, updated where necessary for this latest floodplain investigation, is given for:

- Historic flooding (Section 2.2)
- Risk assessment (Section 3.1)
- Flood hydrology (Section 3.2)
- Construction of computational hydraulic model and model inputs (Section 3.3)
- Modelling of design breakout flows (Section 3.4)

3.1 Risk assessment

The Ashley Floodplain Risk Assessment (Boyle and Surman, 2008) included: an examination of geomorphology; analyses and performance of the existing system; identification of potential (overtopping or lateral erosion) stopbank failure locations (Figure 3-1); and associated likelihood of failure for a range of flood scenarios.

The structural strength of the existing scheme was considered and the main conclusions were:

- The stopbanks were designed and built to a uniform standard, but they were not designed to withstand overtopping by floodwaters.

- The greatest risk of failure of the system is from erosion of the stopbanks. Such a failure could occur during a flood significantly smaller than the design flood.

- Stopbank erosion is dependent not only on discharge, but also on other variables.

- It is not possible to predict the exact point of erosion failure, although constant vigilance, and a pro-active maintenance programme, can lessen the probabilities of erosion failures.

Following an examination of the scheme performance and current theoretical scheme capacity (~ 3000 m^3/s), it was predicted that there is a 50% chance of a breakout at that magnitude. Boyle and Surman (2008) also indicated there was potential, albeit low, for breakouts to occur in flows as low as 2000 m^3/s (~ 10 year ARI). The adopted median breakout scenario indicates there is an ~ 7% chance of a breakout from the Ashley River in any one year. For Oliver (2008), breakouts in floods smaller than the 100 year ARI were not modelled, although it was recognised they could occur.

In reality, the breakout location could be anywhere along the stopbanked system, including multiple breakouts. For the purposes of the Oliver (2008) investigation (i.e. to reduce the numerous scenarios that could be considered), breakout locations were selected for each of the zones that link to the various flow paths on the floodplain. These were based on geomorphology, historical breakout locations, and locations where freeboard on the stopbanks is minimal.

Similarly, the magnitude of the peak breakout flows, and breakout flow volumes, could vary largely. A mid-range estimate was adopted for Oliver (2008) based on the premise that the residual flow in the Ashley River will be ~ 3000 m^3/s.

The greatest breakout threat, and the largest outflow, is expected to occur at the upper end of Zone A (Figure 3-1). This is where the stopbank system commences and where the freeboard is minimal.

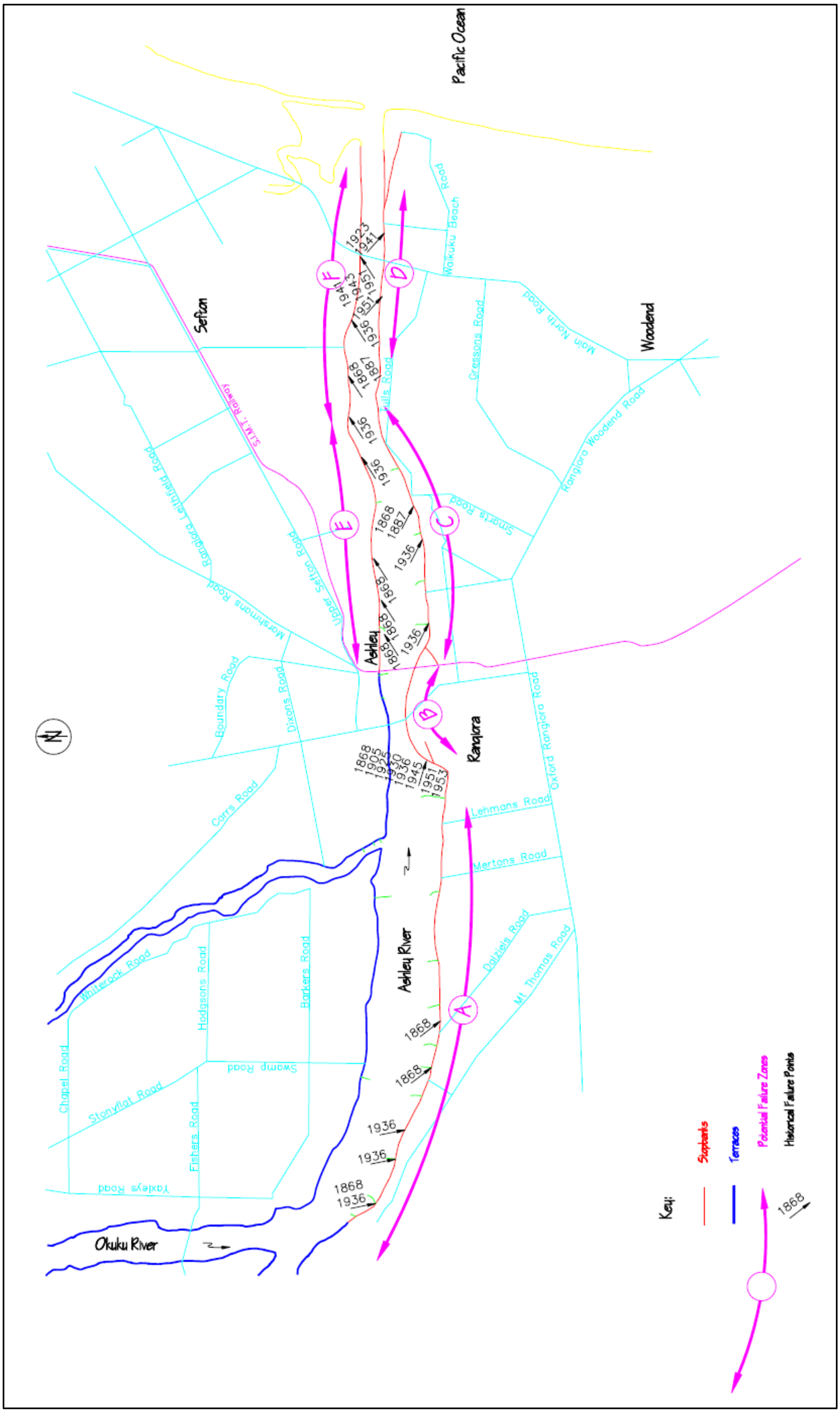


Figure 3-1: Potential failure zones and historical breakouts for the Ashley River

3.2 Flood hydrology

In 2002, flood estimates for the Ashley River catchment were prepared for Environment Canterbury, by the National Institute of Water and Atmospheric Research (NIWA). Discussions with NIWA concluded there was likely to be little change in the 2002 estimates for the 2008 modelling study (Oliver, 2008).

NIWA used two methods to estimate flood flows:

1. Analysis of the Rangiora Traffic Bridge (Site 66214) annual flood peaks.

2. The Regional Flood Estimation (RFE) method, using maximum flood data from nearby river catchments.

As there was only 12 years of continuous record for Site 66214, this analysis could not be used with much confidence. Therefore, the Regional Flood estimates were adopted as the design flows (i.e. total catchment flows). The estimated design flows at Rangiora for a range of probabilities are shown in Table 3-1.

Table 3-1: Estimated design flows at Rangiora (2002)

Event Probability	Peak Flow m^3/s
Mean Annual Flood	860
100 year ARI	3470
200 year ARI	4050
500 year ARI	5300

To test the validity of the river flow estimates, the March 1986 South Canterbury rainfall event was routed through a rainfall/runoff model. This was considered appropriate since the 1986 event (estimated to be a 150 – 200 year ARI rainfall event) was produced by weather conditions similar to those that could also occur in the Ashley catchment, and it resulted in widespread flooding and damage. The rainfall/runoff model predicted a peak flow of ~3800 m^3/s for the 1986 event, which compared well with the estimated 200 year ARI flow for the Ashley River of 4050 m^3/s.

In 2009 the Ashley River flood frequency at the Rangiora Traffic Bridge was reviewed (Griffiths *et al.,* 2009) using 18 years of continuous record for Site 66214. Three methods were used:

1. Analysis of the Rangiora Traffic Bridge (Site 66214) annual flood peaks.

2. Composite analysis of the Rangiora Traffic Bridge (Site 66214) annual flood peaks combined with synthetic record generated from the Ashley Gorge recorder.

3. The Regional Flood Estimation (RFE) method, using maximum flood data from nearby river catchments.

Griffiths *et al.* (2009) showed that, for average recurrence intervals greater than 5 years, the regional flood estimation peak flow values were larger, '*but probably not significantly different from the at site values within 95% confidence intervals*'. An EV1 distribution was also fitted to the Rangiora Traffic Bridge data since at least 30 years of data should be used to reliably fit an EV2 distribution, which is generally more typical of Canterbury rivers (Griffiths *et al.,* 2009).

Table 3-2 shows that the latest estimates for design flows, using the RFE method, are now slightly lower than the earlier 2002 values (Table 3-1), but well within the range of uncertainty. The less reliable at-site EV2 distribution also gives similar flows to the RFE, while the EV1 distribution and EV1/EV2 composite analyses all give lower design flow magnitudes.

Table 3-2: Estimated design flows at Rangiora (2009)

Event Probability	Peak Flow m³/s				
	At site EV1 (± 95% confidence limit)	At site EV2	Composite EV1 (± 95% confidence limit)	Composite EV2	RFE
Mean Annual Flood					740
100 year ARI (1% AEP)	2500±800	3200	2100±500	2600	3200
200 year ARI (0.5% AEP)	2800±950	4000	2400±500	3200	3800
500 year ARI (0.2% AEP)	3200±1100	5100	2700±600	4000	4700

Given the considerable uncertainty in the derived design flows, the original design flows in Table 3-1 have been used in this modelling study. These peak flows are 7 to 13% higher than the latest 2009 estimates so are assumed to be a conservative estimate of current design flows, and a realistic estimate of expected design flows over the next 50 to 100 years due to climate change. However, it should be noted that these flow estimates are the best available at this time, and they are subject to review.

3.3 Hydraulic model construction

Flows over a floodplain are multi-directional and thus are difficult to predict. The Mike Flood (DHI software) modelling package, combining a two-dimensional (Mike 21) model for the floodplain with a one-dimensional (Mike 11) model for the main rivers was used. This software package, linking Mike 11 and Mike 21 models, allowed flood waters to move between the river network and the floodplain (for example, where floodplain overflows are returned to the main river).

The northern and southern floodplains were modelled separately to reduce the model run times. The extent of the floodplains is shown in Figure 1-1 and model schematics are shown in Appendix F. The one-dimensional and two-dimensional models used for each floodplain model are described below.

3.3.1 Southern floodplain - one-dimensional river channel network (Mike 11)

The southern floodplain one-dimensional (Mike 11) model includes the main rivers which impact on the Ashley southern floodplain (i.e. Ashley, Kaiapoi and Waimakariri rivers).

a. Ashley River

The Mike 11 model of the Ashley River extends from the sea to just upstream of the Okuku River confluence.

<u>Cross sections</u>

The surveyed 1997 cross sections used in the model are typically 800 m apart. Approximately 25% of the cross-sections were resurveyed in 2003 to compare with the LiDAR data (which measures above water points only). A comparison of the 2003 and 1997 surveys showed that bed levels were typically 0.1 to 0.2 m lower in 2003. However, because the 2003 survey was not a full survey and there was uncertainty in the remaining cross-sections, the initial Oliver (2008) Ashley River modelling used the 1997 cross-sections.

Boyle and Surman (2009) completed a bed level investigation which compared Ashley River cross sections surveyed in 1997 to cross sections surveyed in 2008/2009. Modelled flows of 2400, 3000 and 3500 m³/s were also compared for both cross sections.

Although there were some channel reaches in Boyle and Surman (2009) where mean fairway levels had decreased by as much as 1 m between 1997 and 2008/9, the mean fairway levels near the Saltwater Creek lagoon and upstream by the Rangiora Railway bridge were only reduced by 0.15 m and 0.07 m, respectively. For an Ashley River flow of 3000 m³/s, modelled maximum water levels were within ± 0.2 m at these two locations (i.e. where the one-dimensional river channel model is connected to the floodplain and there is no stopbank separating the floodplain from the Ashley River flood waters).

As the Ashley River cross sections are now resurveyed on a 5-yearly cycle, the cross sections have most recently been surveyed in 2014. A comparison of the 1997 and 2014 cross sections shows that mean fairway levels near the Saltwater Creek lagoon, and upstream by the Rangiora Railway bridge, have remained similar or have degraded slightly (i.e. up to ~0.1 m) over this time period. Therefore, since there have been no significant changes to modelled Ashley River bed levels in these reaches, the 1997 cross sections have not been updated for this investigation.

<u>Channel roughness (surface resistance)</u>
A Mannings ‘n’ value of 0.03 has been used for the open channel bed resistance. Variations in resistance to vegetation were accounted for by using relative resistances, with Mannings ‘n’ values of up to 0.12 used for heavily vegetated berm areas.

<u>Bridge structures</u>
The two bridges, at Rangiora Traffic/Cone Road and State Highway 1, have been included in the model to take account of channel cross-section changes, submerged soffit, and pier losses.

<u>Model calibration</u>
The Ashley River one-dimensional model was calibrated with the August 1986 event (1400 m^3/s) and the December 1993 event (1900 m^3/s) for Oliver (2008).

Using an open channel Mannings ‘n’ value of 0.03 gave reasonable agreement to average observed levels at the cross-sections, especially when taking into consideration that the observed flood levels on the left and right bank are quite variable. This could be partly due to the difficulty in identifying flood marks after the event.

b. Kaiapoi River
A large portion of the Ashley floodplain overland flow drains into the Kaiapoi River. The lower 1 km of the river, upstream of the Waimakariri River confluence is included in the one-dimensional Mike 11 model.

Cross-section data were derived from the 2014 LiDAR survey and existing cross section data. The Kaiapoi River (upstream of the Mike 11 sections), is included in the two-dimensional Mike 21 model, with appropriate provision made for bed levels. The Kaiapoi River stopbank crest levels are included in both the Mike 11 cross-sections and the Mike 21 model grid.

c. Waimakariri River
The Kaiapoi River drains into the Waimakariri River. Cross-sections and channel roughness from previous modelling of the Waimakariri River were incorporated into the model. As the Waimakariri River is unlikely to be in major flood at the same time as a major event in the Ashley River, a mean annual flood (1500 m^3/s) has been assumed. Waimakariri River floods are generated by north-westerly conditions whereas Ashley River floods occur as a result of various easterly storms.

d. Other minor river branches
A number of minor branches were also included in the Mike 11 model to simulate other physical connections between the floodplain and rivers/streams. These include:

- Kairaki (Saltwater) Creek to Waimakariri River
- Courtney Stream to the Kaiapoi River, with non-return culvert to simulate tide/flood gate
- McIntosh’s Drain to the Kaiapoi River, with non-return culvert to simulate tide/flood gate
- Feldwick Drain to Kaiapoi River, with non-return culvert to simulate tide/flood gate
- Waikuku Stream to Ashley River, with non-return culvert to simulate tide/flood gate
- Taranaki Creek to Ashley River, with non-return culvert to simulate tide/flood gate
- North Drain to Ashley River, with non-return culvert to simulate tide/flood gate

e. Pegasus Bay sea levels
Sea level is a combination of tide level, storm surge and other sea level variability such as seiche, tidal residuals and other variations (Bell, 2011). The closest sea level recorder site to the Ashley and Waimakariri river mouths is located at Sumner Head (Site 66699). This site has been operating since June 1994, and is jointly funded by the National Institute of Water and Atmospheric Research (NIWA) and Environment Canterbury.

Analysis of the Sumner Head sea level record between 1995 and 2010 (16 years inclusive) shows that the mean annual maximum storm surge is 0.37 m with a maximum storm surge of 0.49 m recorded on 6 October 2005 (Bell, 2011). Between 2004 and 2010 (7 years inclusive) the mean annual maximum storm tide level is 1.60 m above mean sea level (msl) with a maximum storm tide level of 1.72 m above msl recorded on 4 January 2006.

The downstream water level for the Waimakariri River is based on a tidal cycle, with a maximum level of 1.7 m above msl (see Appendix C). This includes an allowance for storm surge, which is very likely to occur during a major rainfall event in the Ashley catchment.

The Ashley River downstream water level has been assigned a constant level of 1.7 m above msl to also represent a relatively high tide with allowance for storm surge. This downstream constant water level is contained by the Ashley River stopbanks and does not have any impact on upstream water levels in the areas of interest (e.g. where Breakout B flows return to the river).

3.3.2 Northern floodplain - one-dimensional river channel network (Mike 11)

The northern floodplain model only includes the Ashley River, as described above. For the northern floodplain model, the Ashley River downstream water level has been set to a constant level of 1.7 m above msl for all of the design breakout flow events. This is to ensure that the breakout flow reaches the Saltwater Creek lagoon area at approximately the same time as a high tide. While conservative, this could occur.

3.3.3 Two-dimensional floodplain model (Mike 21)

The two-dimensional (Mike 21) grid of the Ashley River floodplain was divided into two separate model grids – one to represent the northern floodplain and one to represent the southern floodplain. Figure 1-1 shows the extent of the model grids. The details of the floodplain topography and roughness, used in both floodplain models, are the same and are described below.

<u>Floodplain topography</u>
To realistically model floodplain flows, good topographic data (including features such as banks, terraces, overland flow channels, roads and railway embankments) are essential. This investigation uses LiDAR (Light Detection and Ranging) data obtained between March and June 2014.

This supersedes the July 2005 LiDAR used in the Oliver (2008) study. An example of the detail provided by LiDAR data are shown in Figure 3-2 for the Ashley floodplain.

In the model, water levels and flows on the floodplain are resolved on a rectangular grid. The size of the grid is based on the level of detail required, model stability, and computational efficiency (i.e. computer capacity and speed). For this model, the LiDAR data have been used to generate a grid of 10 x 10 m cells to represent the floodplain topography. The 10 m grid was chosen as it allowed a reasonable degree of topographic detail while keeping the model run time within a suitable timeframe. The 10 m grid used in this modelling study was initially produced for the Waimakariri District Council modelling work (WDC, 2015).

Unfortunately the 10 m grid does have some limitations pertaining to representation of some features such as smaller drains. Where drains are not able to be represented it is generally assumed that this is equivalent to the drain being either blocked and/or the flow from the breach is relatively high compared to any flows conveyed by the drains. This is usually a reasonable assumption – especially for the larger and less frequent breach flows across the floodplain.

Checks were made with the detailed LiDAR data to ensure important topographic features were correctly represented in the 10 m grid, and that historic flow paths were correctly represented.

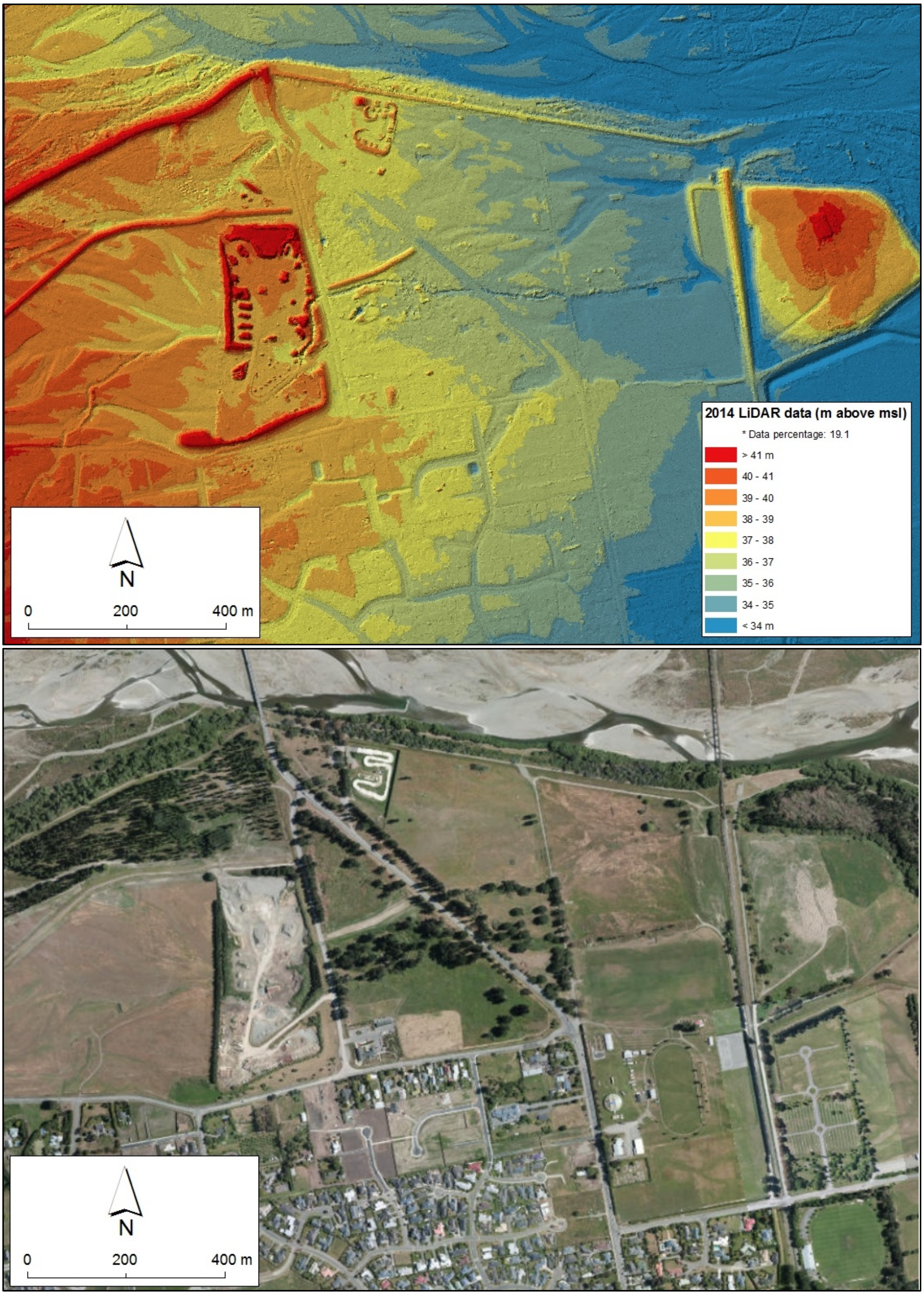


Figure 3-2: Comparison of LiDAR data and aerial photography near Breakout B, showing the quarry and southern approaches to the road and railway bridges (to the north of Rangiora) as well as part of the new stopbank under construction

Generally, there was good agreement between the full topographic data set and the 10 m model grid. Modifications to the latest 2014 model grid included:

- Lowering grid cells where crops in paddocks had artificially raised the ground level. This was able to be detected by subtracting the 2005 LiDAR data from the 2014 LiDAR data.
- Raising grid cells where stopbank heights had been reduced due to averaging over a 10 m grid cell. For example, along the eastern side of Kairaki (Saltwater) Creek and downstream along the Waimakariri River.
- The addition of the new stopbank to the south of the Ashley River (upstream of the railway bridge) that is currently under construction. Note: it has been assumed that the gaps for the roads are filled so the stopbank is continuous. A sensitivity test has also been completed to determine the effects of not closing these gaps (Section 3.4.6)
- The grid cells along the Kaiapoi River true right bank, immediately upstream of Williams Street, have been raised to represent the solid wall which exists in this location
- Kaiapoi River grid cells have been lowered to better represent the channel bed rather than the water surface.
- Grid cells have been raised to represent a wall from Allison Crescent (near Forrest Lane) to Williams Street (near Ansel Place).
- Better definition of some of the smaller drains and banks around Breakout B.

WDC have ensured that stopbanks and railway embankments have been correctly incorporated into the grid. They have also added 150 mm to building platforms in the urban areas of Rangiora, Woodend, Kaiapoi, Pegasus, Waikuku Beach and Pines Kairaki. At present some of the Kaiapoi River stopbanks have crest levels lower than the design level of 4.05 m (msl). The current stopbank levels have been used in the model grid as they are representative of the existing situation.

<u>Floodplain structures</u>
Thirty eight culverts have been incorporated into the model used in this study. All of the culverts are located on the southern floodplain with several connecting the southern floodplain to the main rivers (i.e. outlet structures).

A summary of the culvert details is given in Table 4 of WDC (2015). All of the WDC (2015) 'South Ashley' model culverts were included, except for the Mill Road culvert which was outside the modelled floodplain area. One additional culvert was also included that was not in the WDC (2015) study. This culvert structure is located immediately upstream of the Rangiora railway bridge over the Ashley River, and has been constructed to drain surface water flows back into the Ashley River (from the ponding area located to the south of the new stopbank under construction). The two pipes have a diameter of 1.05 m.

The 10 weirs in the WDC (2015) 'South Ashley' model were also included in the southern floodplain model. The weir details are given in Table 4 of WDC (2015). These weirs have been included to represent overtopping at some of the culverts. Figure 3-3 shows the culvert and weir locations.

<u>Floodplain flows (breakout flows)</u>
As described previously, stopbank breach scenarios in the Ashley River aren't necessarily a result of direct overtopping, but more likely from lateral erosion. It was, therefore, considered appropriate to introduce breakout flows (through stopbank breaches) onto the floodplain as "source" inflows distributed over 10 grid cells (for stability) at each breakout location.

As discussed in Section 3.1, these breakout locations could potentially be anywhere along the stopbanked system. The breakout flows, represented as outflow hydrographs, have, therefore, been located to ensure there is one for each zone. The timing of any breakouts is also difficult to predict so it has been assumed that the breakouts will all occur around the time of the peak Ashley River flow.

Breakouts B and E/F flows are assumed to occur 15 minutes after Breakout A flows, and the Breakout C flows occur 1 hour after Breakout A.

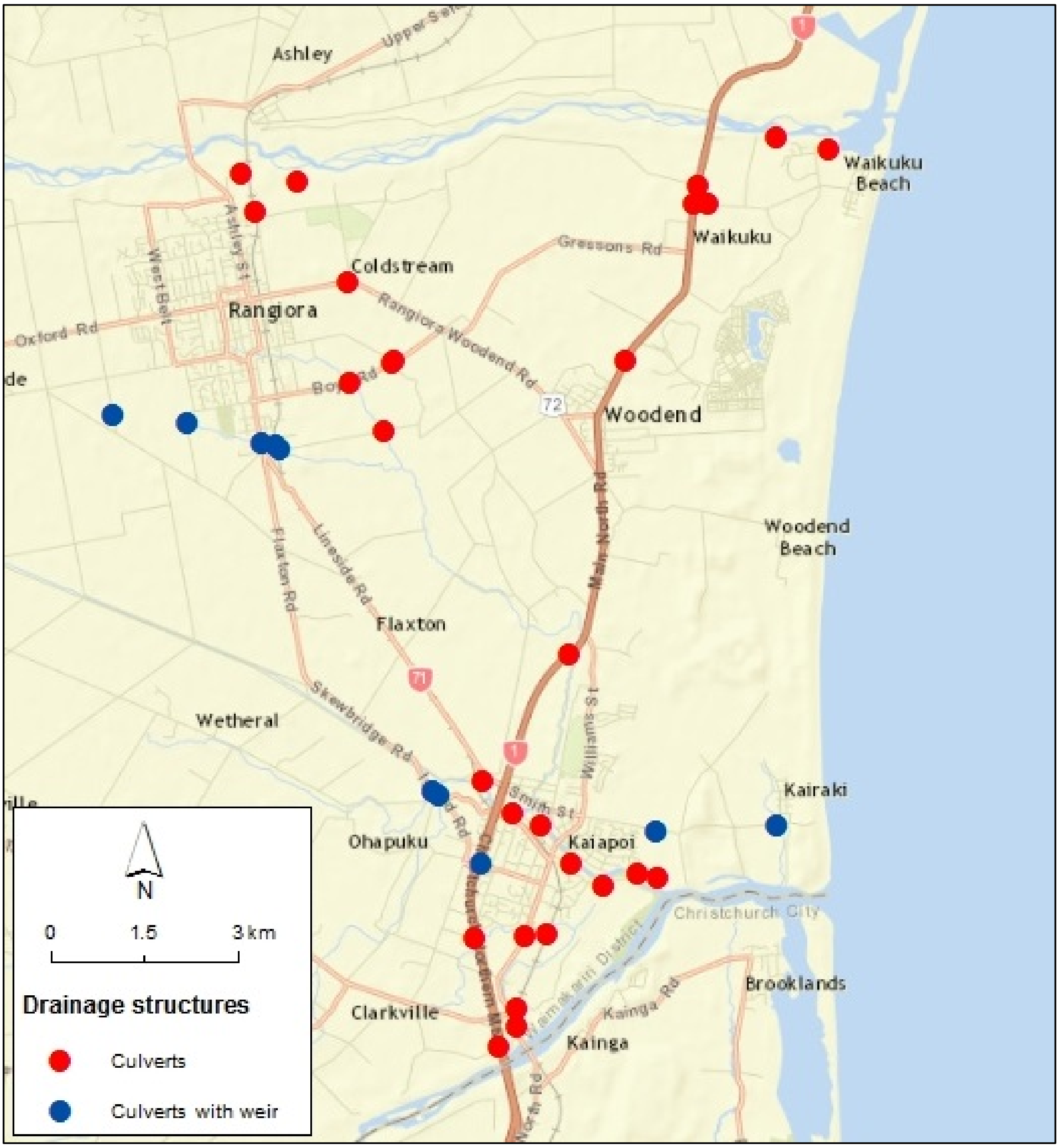


Figure 3-3: Location of drainage structures in the southern floodplain model

Floodplain roughness (surface resistance)
Floodplain flows and depths are influenced by the hydraulic resistance of the ground cover and other obstructions, such as buildings and trees on the floodplain. These resistance values (i.e. Mannings 'n' values) are usually assigned to the various surfaces of the floodplain by interpretation of aerial photographs and ground survey.

For the previous Oliver (2008) modelling study, raw data from the LiDAR survey (i.e. first and last returns and intensity) were used by NIWA to develop a very detailed surface map of floodplain resistance. This roughness detail was represented in the Mike 21 model by a 10 m grid and sensitivity analyses were undertaken for surface roughness.

For this latest study, the recently derived WDC (2015) floodplain roughness data have been used. This floodplain roughness grid was derived from the Ministry for the Environment Land Cover Database Version 4 (LCDB4) land use classification. Different land use types were assigned a corresponding Mannings 'n'/Mannings M value. Additional floodplain roughness categories were also created for roadways (for reduced roughness) and building platforms (for increased roughness).

Additional modifications to the WDC (2015) floodplain roughness grid included:

- The Mannings 'n' values along the lower reach of Kairaki (Saltwater) Creek, to the Waimakariri River confluence, were reduced to 0.03 (Manning M = 33) to better represent the channel.

- The Mannings 'n' values between the existing and new stopbank, upstream of Cones Road, were reduced from 0.125 (Manning M = 8) to 0.05 (Manning M = 20). This is to represent the reasonably well spaced, rather than densely 'clustered' grouping of the trees. A sensitivity test has also been completed assuming a Mannings 'n' value of 0.08 (M = 12.5) for this area for the 100 year Breakout B scenario (Section 3.4.6).

In both studies Mannings 'n' values typically varied from around 0.013 (roads) to 0.13 (dense vegetation). WDC (2015) also assigns a value of n = 0.333 (M = 3) for buildings. A table summarising the LCDB4 land types and corresponding Mannings 'n' and M factors is given in Appendix B and a more detailed description of the derivation of the model roughness grid is given in WDC (2015). Generally, the WDC (2015) floodplain roughness data have slightly more conservative (higher) Mannings 'n' roughness values than the NIWA roughness data used in the Oliver (2008) modelling.

3.3.4 Mike Flood model

The one-dimensional (Mike 11) and the two-dimensional (Mike 21) models are dynamically coupled together in the Mike Flood module.

<u>Southern floodplain</u>
In the southern floodplain model, standard links connect the floodplain grid cells to the Kaiapoi River, Courtney Stream, McIntosh's Drain, Kairaki (Saltwater) Creek, and several other outlet structures.

A lateral link also connects the Ashley River (between chainage 88730 and 89540) to the area of floodplain now contained by the new stopbank upstream of the Rangiora railway bridge.

<u>Northern floodplain</u>
In the northern floodplain model, a lateral link connects the Saltwater Creek lagoon to the Ashley River (between chainage 99200 and 99700). This allows northern floodplain flows to return to the Ashley River via the Saltwater Creek lagoon.

The breakout locations, river cross sections, floodplain areas and lateral link locations (connecting the river channels to the floodplain) are shown in Figure 3-4, and in more detail in Appendix F. All model run files are listed in Appendix G.

3.4 Modelling of design events

Model runs simulated a flood over 41 hours. The southern floodplain model simulations used a 1 second time step, to ensure stability; the northern floodplain simulations used a 2 second time step. Simulations on the larger southern floodplain still took ~60 hours, while the northern floodplain model could be run overnight.

Ideally any model should be calibrated with historical flood events. The calibration of the one-dimensional (Mike 11) Ashley River model was completed by Oliver (2008), as described in Section 3.3.1. Unfortunately, it was not possible to obtain any data to calibrate the two-dimensional (Mike 21) model. Even though there are historical flood maps of the 1950 and 1953 floods, the magnitude of the multiple breakouts for these events is not known.

In this investigation, simulations have been undertaken for the 100, 200 and 500 year ARI events, for the existing land use scenario. Other development scenarios for the Kaiapoi area have been previously undertaken and described in Oliver (2008).

Additional local rainfall runoff has not been considered in either investigation although it is conceded that significant local flooding will be occurring simultaneously. WDC carried out some work combining the previous modelling with their local flooding modelling and they could do so again with the new results.

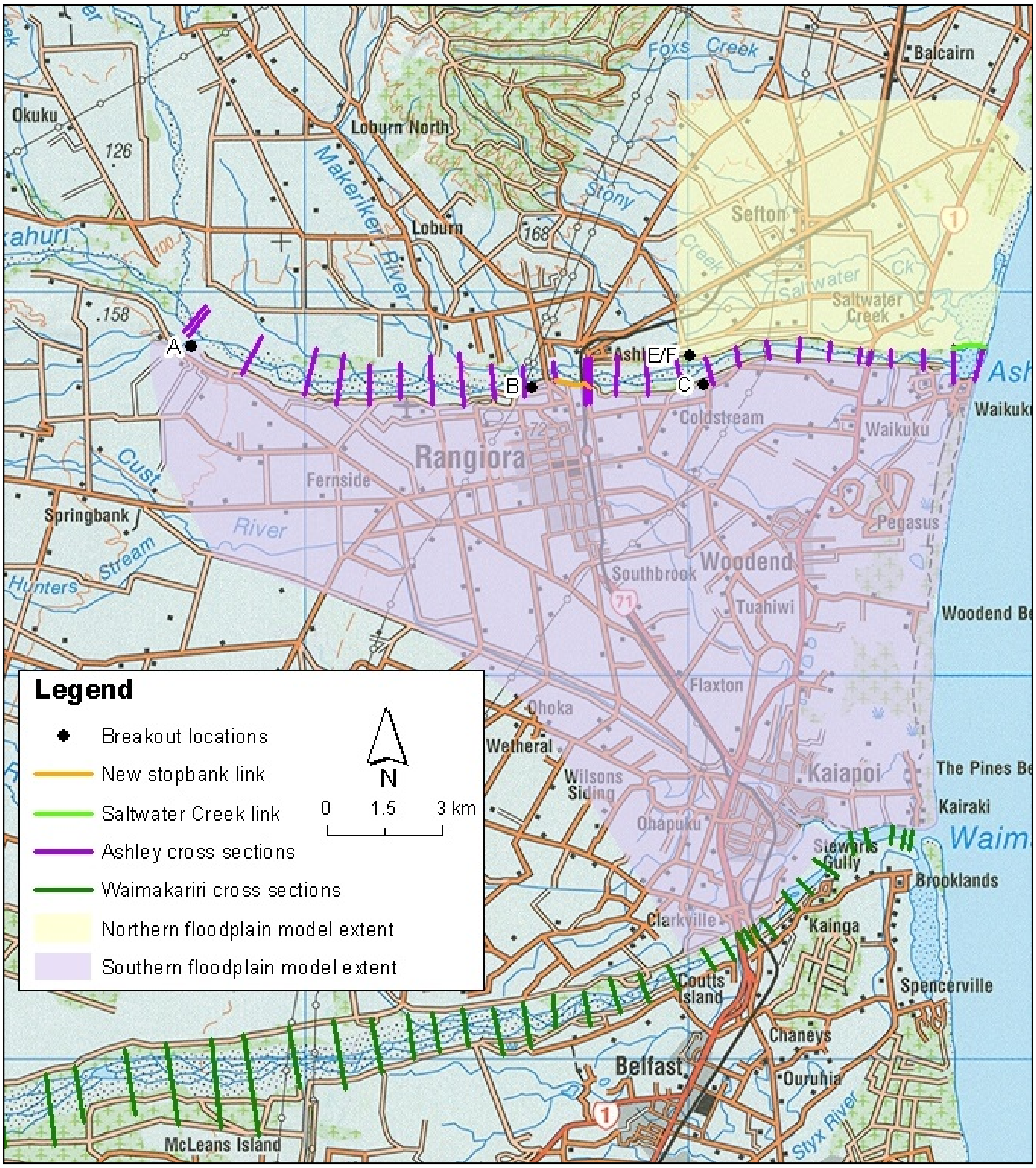


Figure 3-4: Model schematic showing cross sections, floodplain extents, links between the Ashley River and the floodplains and breakout locations

3.4.1 100 year ARI (1% AEP) flood scenarios

Based on the risk assessment analysis for a 100 year ARI Ashley River flood event, three scenarios have been modelled:

- Breakout at A – a southern floodplain breakout at the top of Zone A (downstream of the Okuku River confluence).
- Breakout at B – a southern floodplain breakout at Zone B, north of Rangiora.
- Breakout at E/F – a northern floodplain breakout at Zone E or F, downstream of Ashley township.

The 100 year ARI Ashley River flow (upstream of all the breakout locations) is estimated to be 3470 m^3/s. The mid-range estimate for a breakout flow at Zone A or B is 500 m^3/s, and for Zone E or F is 125 m^3/s. Of these scenarios, Breakout A is considered the most likely. Flow hydrographs for the Ashley River (downstream of Breakout A) and the various breakouts are shown in Appendix C.

Breakout A

Figure 3-5 shows that Breakout A leads to mainly shallow flooding (0 – 0.5 m) across a large part of the floodplain, although Rangiora is mainly free from flooding. Much of the floodwaters enter South Brook and the Cam River/Ruataniwha which drain into the Kaiapoi River, while some flood water also flows along the west side of the railway line along Lineside Road. Most of the existing urban area of Kaiapoi is free from flooding, with just isolated pockets of floodwaters in some low lying streets.

Flood waters pond in the “Flaxton Swamp” area (east of Mulcocks Road and Bramleys Road) to depths up to ~ 2.6 metres. Ground levels in this area are particularly low, at just over 1 m above mean sea level. From Flaxton Swamp, flood waters cross the northern motorway. Flooding is mainly contained on the west side of Old North Road as there is a ridge of beach deposits about 3 m high to the east of Old North Road.

Overall, flood inundation depths and extents are similar to those observed in Oliver (2008), with the exception of some additional areas of minor flooding.

At the new stopbank north of Rangiora, two 1.05 m diameter pipes are designed to drain the ponding area to the south of the stopbank. These pipes do not have sufficient capacity to pass the peak breakout flows back into the Ashley River. Instead, the excess water is designed to pond behind the new stopbank and drain over an extended period of time. This modelling study shows that the ponding area reaches maximum capacity and a very small volume of water flows to the south over Coldstream Road, before contributing to the water flowing either in a northerly direction (via the drain that flows under the railway line and towards Maria Andrew Park) or in a south-easterly direction (towards SH72).

As the overflow from the ponding area behind the new stopbank is relatively small (peak flow of ~ 2 m^3/s), and the additional overflows from Breakout A that travel into this area are also relatively small, the excess flows tend to flow into existing drains or produce shallow flooding (which is deepest where it backs up behind roads). In general, the 10 m grid does not allow the smaller drains to be well-defined. However, this is not considered an issue because these will more than likely be full with local runoff.

Given the uncertainty in the model parameters and peak breakout flows, any additional flooding for this breakout scenario, when compared to Oliver (2008) was considered negligible and within the model uncertainty (i.e. able to be explained largely by the use of the different floodplain roughness grid and changes in floodplain topography).

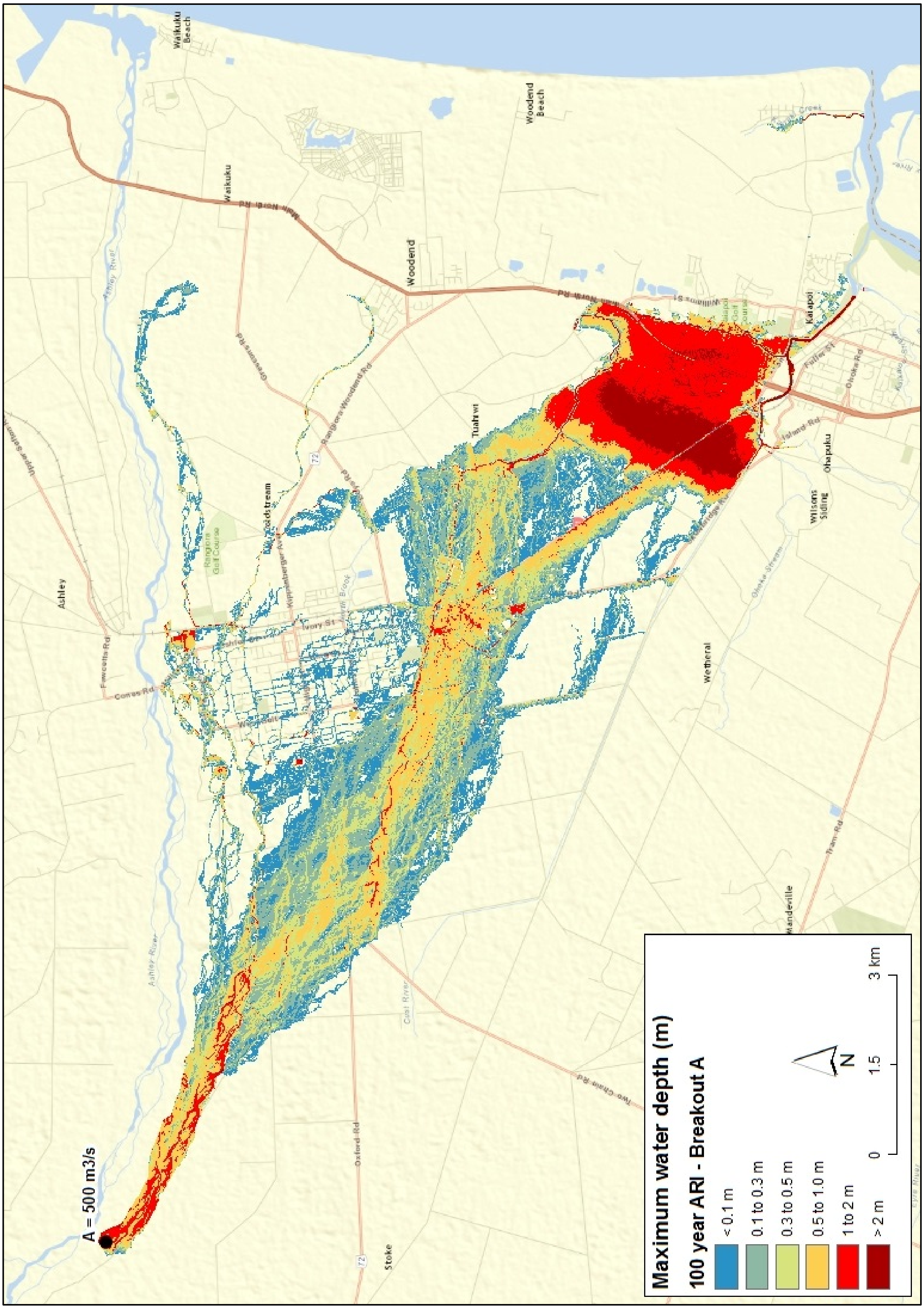


Figure 3-5: 100 year ARI (1% AEP) - Breakout A maximum modelled water depths

<u>Breakout B</u>
Figure 3-6 shows the alternative breakout, Breakout B, for the 100 year ARI scenario. Although the breakout flow magnitude is the same as Breakout A, floodwater is now expected to be contained by the new stopbank. This means breakout flows return to the Ashley River, rather than passing onto the floodplain and contributing to flooding (e.g. ponding in Flaxton Swamp).

At the time of this investigation there were still gaps in the new stopbank where Cones Road and Milton Avenue pass through the stopbank. Section 3.4.6 includes modelling (and predicted flooding) for this scenario.

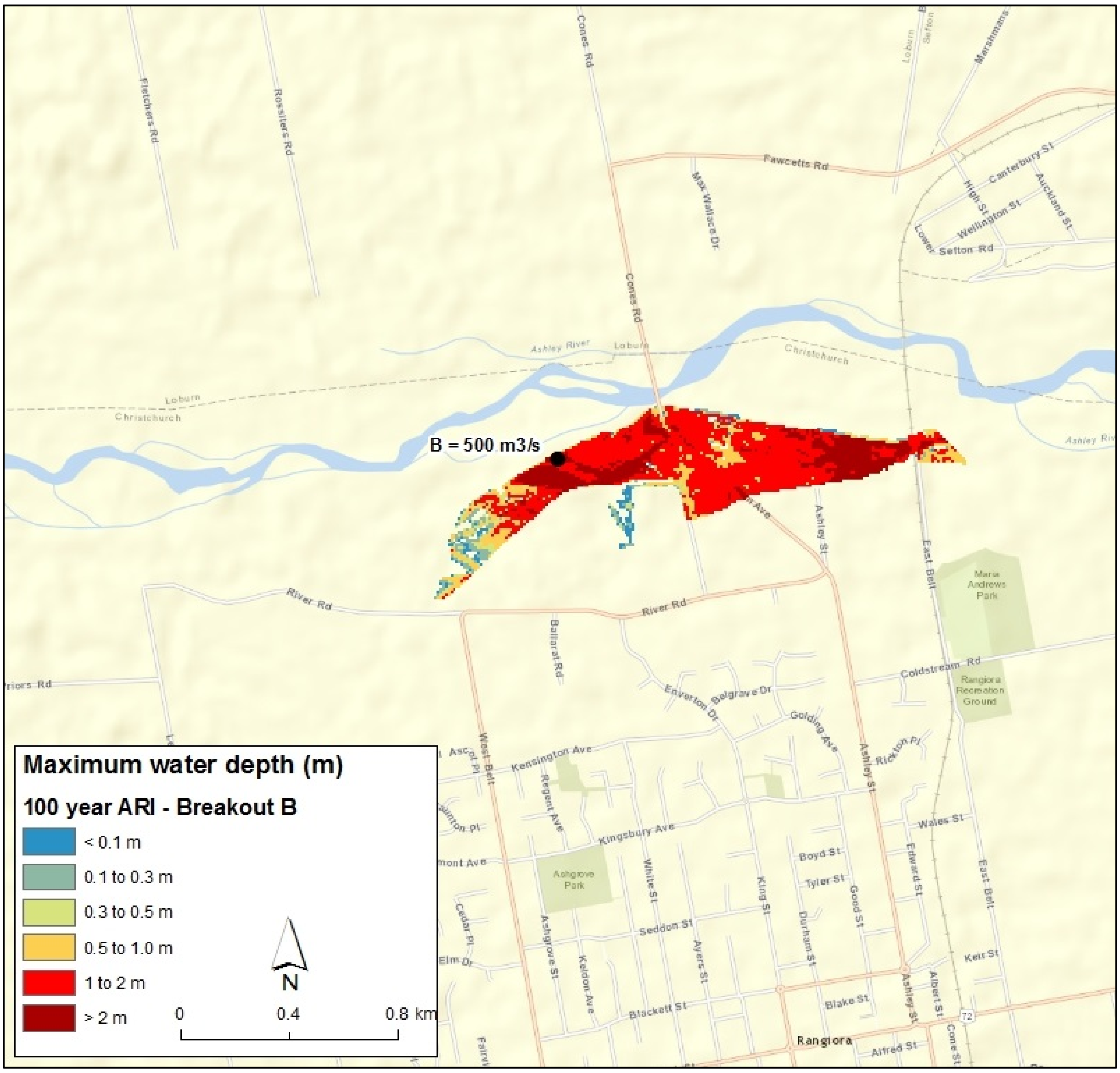


Figure 3-6: 100 year ARI (1% AEP) - Breakout B maximum modelled water depths

<u>Breakout E/F</u>
A 125 m^3/s breakout flow onto the northern floodplain (Breakout E/F) would cause flood waters to cross State Highway 1 and drain into Saltwater Creek lagoon (Figure 3-7). The inclusion of a lateral link, connecting the entire width of the Saltwater Creek lagoon area to the Ashley River has increased maximum water levels in the 'tidally-influenced' area (i.e. area to the north and east of Factory Road) by up to 0.4 m, compared to the model configuration used in Oliver (2008). All other flooding further upstream has remained similar to that observed in Oliver (2008).

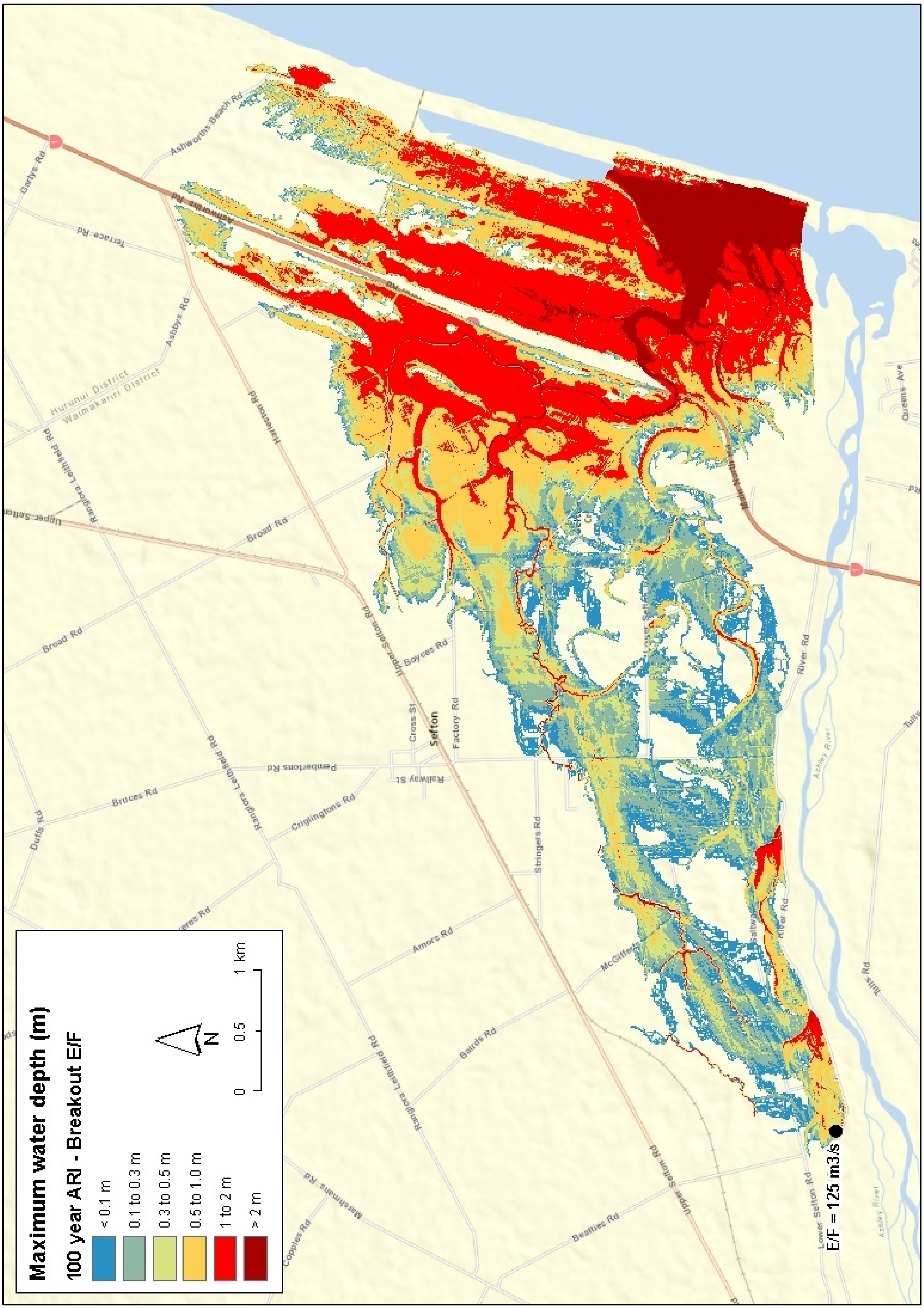


Figure 3-7: 100 year ARI (1% AEP) - Breakout E/F maximum modelled water depths

3.4.2 200 year ARI (0.5% AEP) flood scenarios

The risk assessment analysis showed that, for a 200 year ARI Ashley River flood event, the most likely breakout scenarios are either combined breakouts to the southern floodplain, or combined breakouts to the northern and southern floodplain. Three scenarios have been modelled:

- Breakouts at A and B – southern floodplain breakouts of 750 m^3/s at the top of Zone A (downstream of the Okuku River confluence) and 400 m^3/s at Zone B, north of Rangiora.
- Breakouts at A and C – southern floodplain breakouts of 750 m^3/s at the top of Zone A (downstream of the Okuku River confluence) and 200 m^3/s at Zone C, near Smarts Road.
- Breakouts at A and E/F – a southern floodplain breakout of 750 m^3/s at the top of Zone A (downstream of Okuku River confluence), and a northern floodplain breakout of 200 m^3/s at Zone E or F, downstream of Ashley township.

The 200 year ARI river flow (upstream of all the breakouts locations) is estimated to be 4050 m^3/s. Flow hydrographs for the Ashley River (downstream of Breakout A) and the various breakouts are shown in Appendix C.

Breakouts A and B
Figure 3-8 shows the combined breakout from Zones A and B. For this scenario flooding on the southern floodplain is fairly extensive, although Rangiora township is relatively flood free. Depths in the "Flaxton Swamp" area are up to ~ 3 m. Flood depths in the Kaiapoi East area (residential red-zone) are up to ~ 0.7 m.

Overflows from the ponding area behind the new stopbank (near Breakout B), remain small and follow the same flow paths as the 100 year ARI Breakout A scenario. Water depths for the shallow overflows (both overflows from the ponding area behind the new bank and other floodplain flow) are up to ~ 0.15 m higher than the 100 year ARI Breakout A scenario (mainly in areas where the flood water backs up behind roads). This increase remains small as the largest portion of the Breakout A flow continues to favour the main flow path towards Kaiapoi. Water levels on the floodplain, downstream of Breakout B and the new stopbank, remain significantly lower than modelled levels for the same scenario without the new stopbank (as modelled in Oliver, 2008).

Breakouts A and C
Maximum water depths for the other southern floodplain breakout scenario, with combined breakouts at Zones A and C are shown on Figure 3-9. Flooding in the Kaiapoi area for this scenario is the same as for the combined breakout from Zones A and B. This is because no (or very insignificant) Breakout B flows now contribute to flooding in Kaiapoi. Flood depths in the Waikuku area are typically up to 1 – 1.5 m, although Waikuku Beach settlement, on the sand hills, is only partially flooded to relatively shallow depths.

Overflows from the ponding area, behind the new stopbank (near Breakout B), remain relatively small and follow the same flow paths as the 100 year ARI Breakout A scenario. Water depths for these shallow overflows increase by a maximum of ~ 0.05 m where the flood water backs up behind roads. This small increase in water level is most likely because the largest portion of the Breakout A flow continues to favour the main flow path towards Kaiapoi, rather than flowing in an easterly direction towards the Breakout B area.

Breakout E/F
Figure 3-10 illustrates the predicted flood extent for a breakout onto the northern floodplain (Breakout Zone E & F), in conjunction with a 750 m^3/s breakout at Zone A. For this scenario flood depths on the northern floodplain are generally under 0.5 m, with greater depths adjacent to SH1. Floodwaters cross SH1 and drain into Saltwater Creek lagoon and the Ashley River. The inclusion of a lateral link, connecting the entire width of the Saltwater Creek lagoon area to the Ashley River has increased maximum water levels in the 'tidally-influenced' area (i.e. area to north and east of Factory Road) by up to 0.5 m, compared to the model configuration used in Oliver (2008). All other flooding further upstream has remained similar to that observed in Oliver (2008).

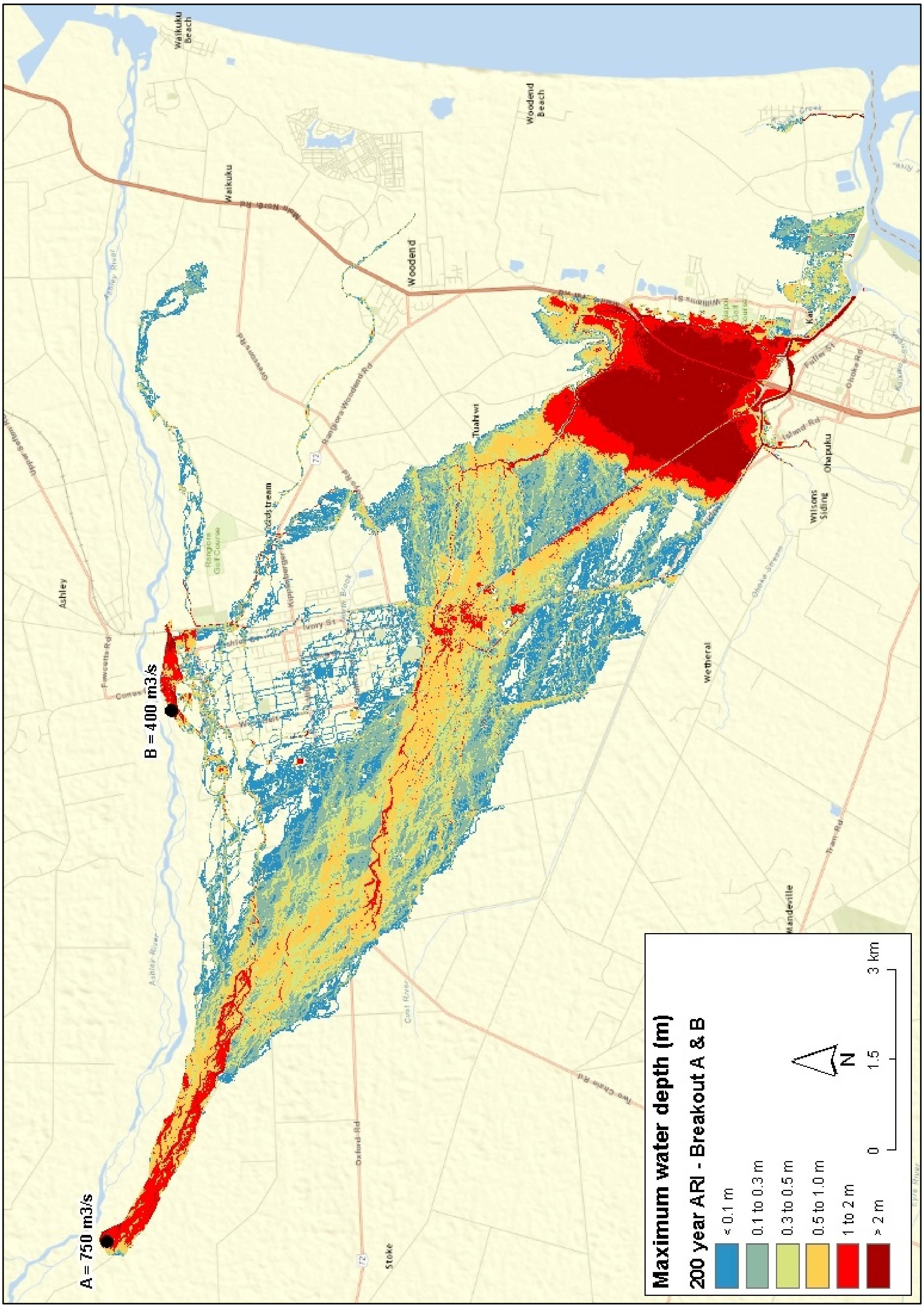


Figure 3-8: 200 year ARI (0.5% AEP) - Breakouts A & B maximum modelled water depths

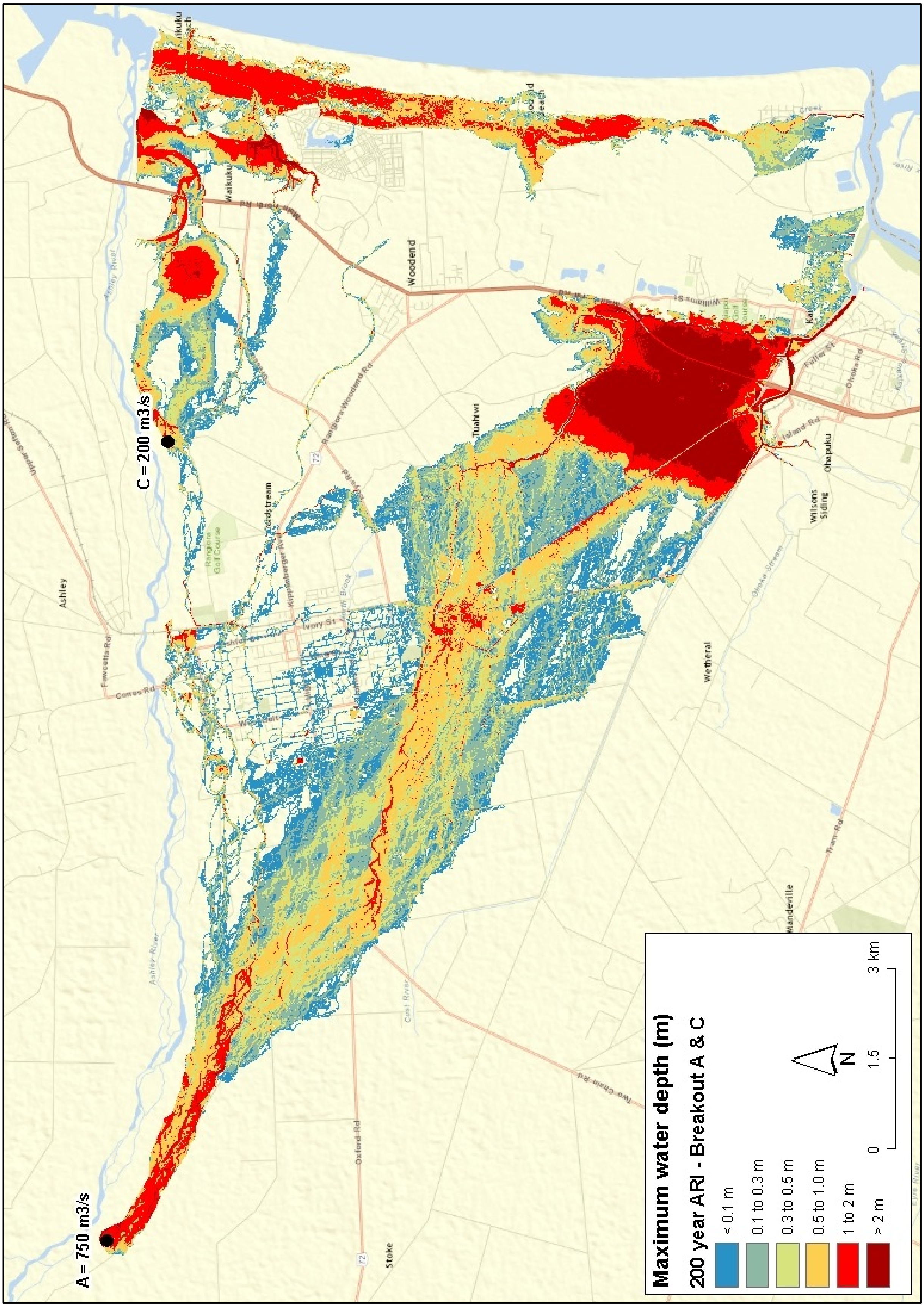


Figure 3-9: 200 year ARI (0.5% AEP) - Breakouts A & C maximum modelled water depths

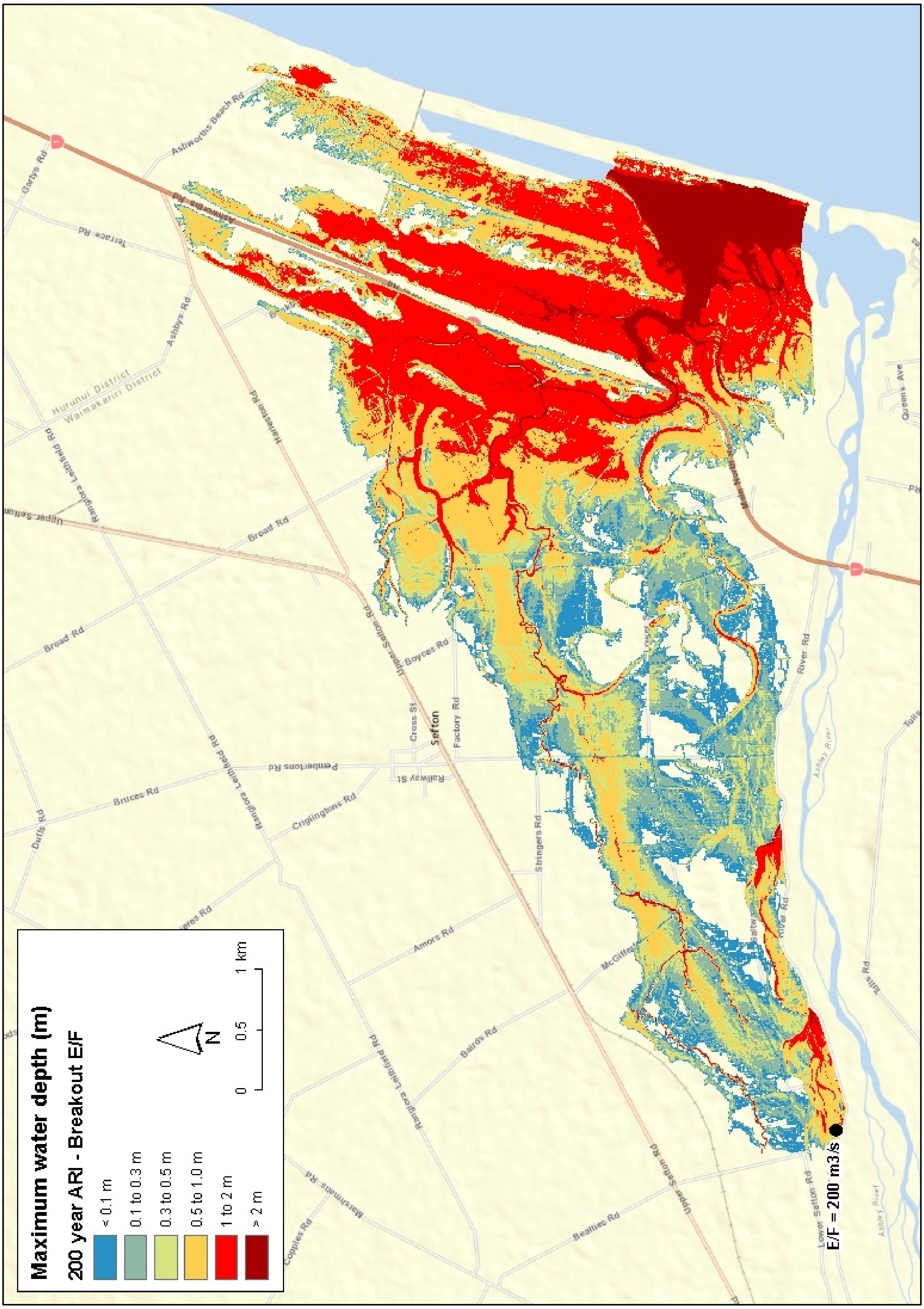


Figure 3-10: 200 year ARI (0.5% AEP) - Breakout E/F maximum modelled water depths

3.4.3 500 year ARI (0.2% AEP) flood scenarios

The risk assessment analysis showed that, for a 500 year ARI Ashley River flood event, the most likely breakout scenarios are either combined (three) breakouts to the southern floodplain, or combined (three) breakouts to the northern and southern floodplain. Two scenarios have been modelled:

- Breakouts at A, B and C – southern floodplain breakouts of 1750 m^3/s at the top of Zone A, 350 m^3/s at Zone B, and 160 m^3/s at Zone C.
- Breakouts at A, B and E/F – southern floodplain breakouts of 1750 m^3/s at the top of Zone A, 250 m^3/s at Zone B, and a northern floodplain breakout of 250 m^3/s at Zone E or F.

The 500 year ARI Ashley River flow (upstream of all the breakout locations), is estimated to be 5300 m^3/s, with an estimated 40$^+$% of the flow spilling onto the floodplain. Flow hydrographs for the Ashley River (downstream of Breakout A) and the various breakouts are shown in Appendix C.

As the stopbanks around the Waimakariri River/Kaiapoi River confluence start to overtop around the peak of this flood event, additional lateral links have been added to the model to allow for this overflow between the model grid and the 1-d channel cross sections. Figure 3-11 shows the location of these lateral links, which allow floodwaters inundating the Kaiapoi area to flow over the stopbanks and back into the Waimakariri River.

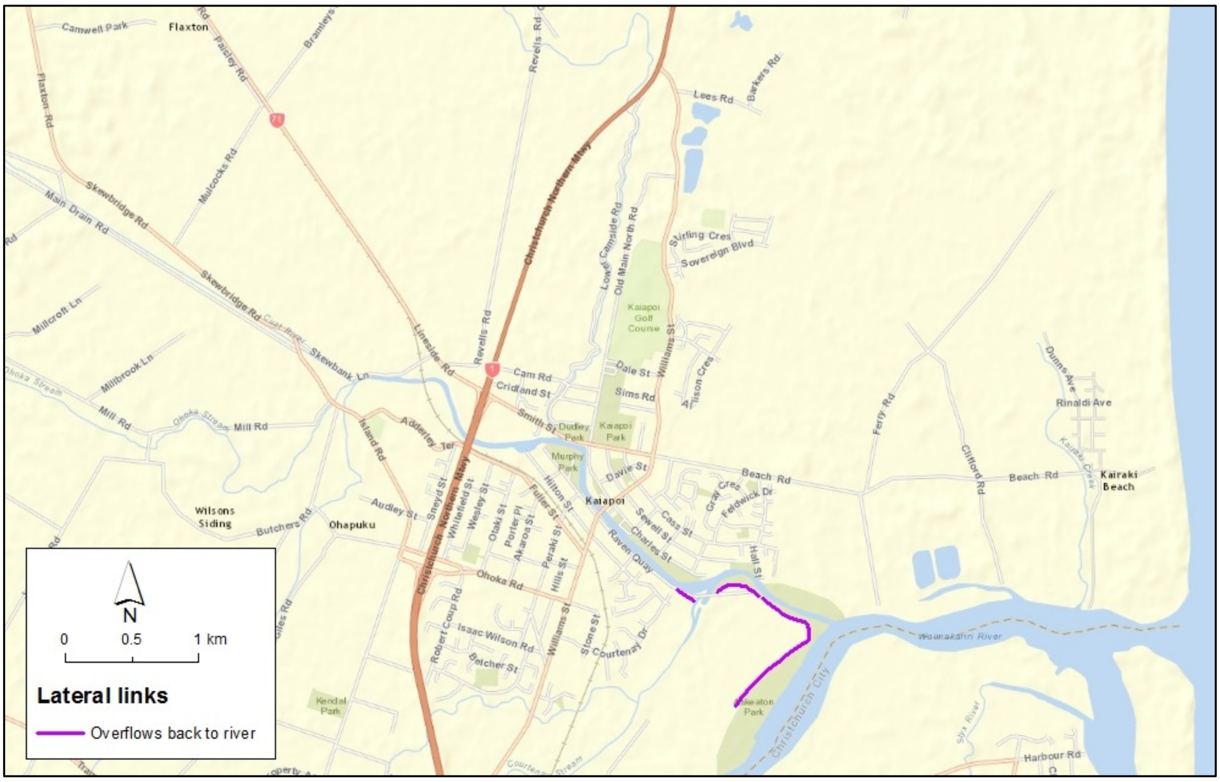


Figure 3-11: Location of lateral links allowing overflows back into the Kaiapoi and Waimakariri rivers

Figure 3-12 shows maximum water depths for the combined breakouts at A, B & C, for the southern floodplain, and maximum water depths for a breakout at E/F, for the northern floodplain.

With such large volumes of floodwater out of the river, the flooding is widespread. However, only parts of Rangiora are flooded, and to depths generally under 0.3 m. Flood depths in the Flaxton Swamp area are over 3 m. The majority of urban Kaiapoi is flooded with depths south of the Kaiapoi River, bounded by Island Road (to the west), Ohoka Road (to the south) and Williams Street (to the east), largely over 2 m deep. Flood depths in the Kaiapoi East area (residential red-zone) are up to ~ 1.5 m.

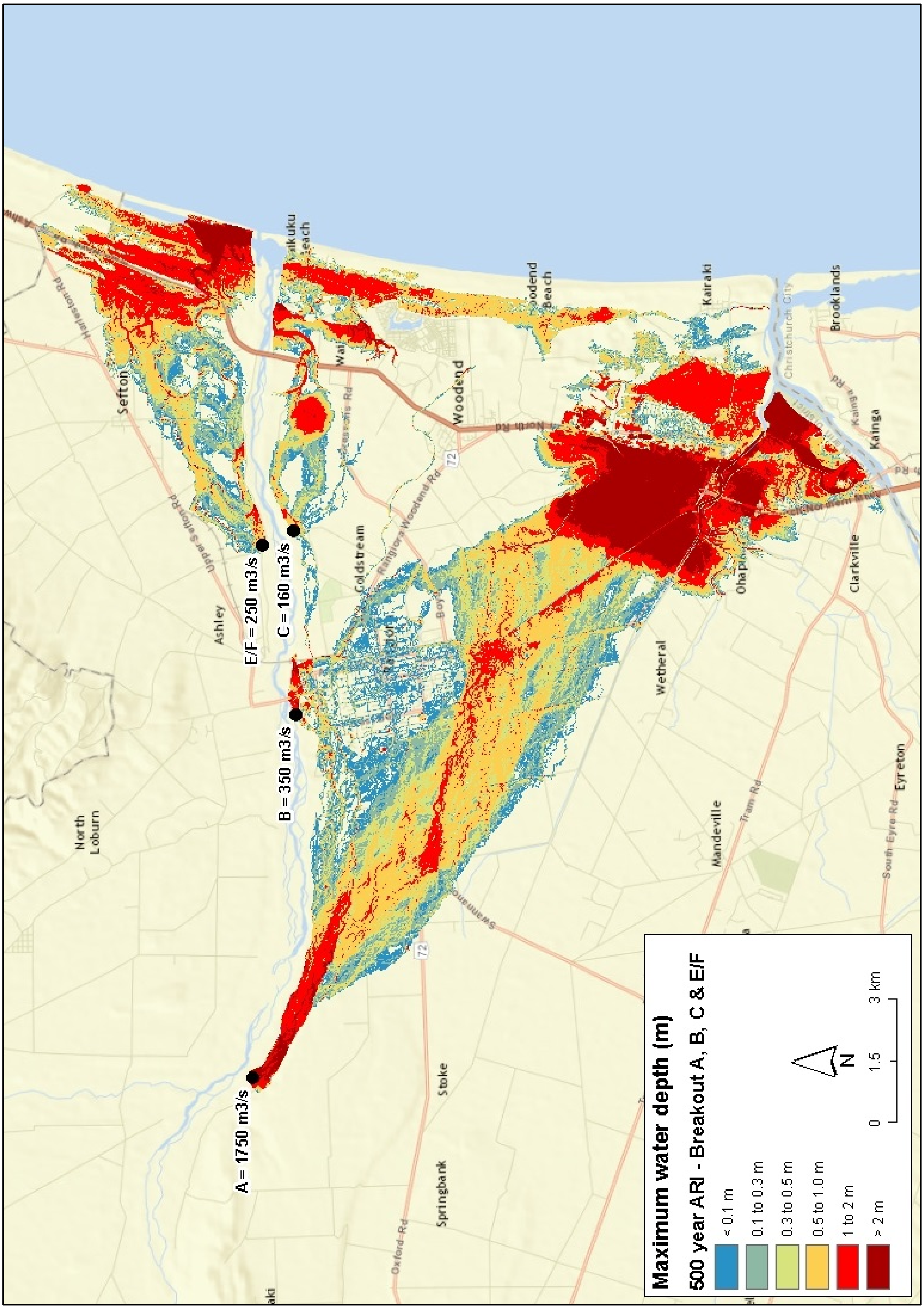


Figure 3-12: 500 year ARI (0.2% AEP) - Breakouts A, B, C and E/F maximum modelled water depths

LiDAR data indicate that some of the stopbanks are currently slightly lower than the design levels in places. Although these stopbank levels may be raised at a future date, this modelling has assumed stopbank levels are as close as possible to the existing scenario. Should stopbank levels be raised in the future, maximum modelled water depths may also increase.

Compared to the earlier Oliver (2008) model results, floodplain maximum water depths are mainly within ± 0.2 m. Along the main (deeper) flow path from Breakout A, the Oliver (2008) maximum water depths were generally at least 0.2 m lower, with water depths upstream of Lineside Road over 0.5 m lower in places. These increases in maximum water depths for this latest investigation are likely to be due to the more conservative roughness (Manning's 'n') values used.

Maximum water depths in the Waikuku and Woodend areas tend to remain similar or be up to ~0.3 m higher in Oliver (2008) – as do water depths around Kaiapoi. These decreases in maximum water depths for this latest investigation are likely to be due to the Breakout B flows being diverted back into the Ashley River, rather than contributing water to the floodplain.

3.4.4 High hazard areas

High hazard areas are defined in the Canterbury Regional Policy Statement as areas where the flood depth is greater than 1 metre or where the product of depth and velocity is greater than 1 in a 500 year ARI event (see Appendix D). Figure 3-13 shows high hazard areas for the northern and southern Ashley floodplains for the 500 year ARI events. The model results show a large proportion of Kaiapoi meets the definition of a high hazard area.

3.4.5 Summary of breakout scenarios

The modelled breakout scenarios for the various design flood events are summarised in Table 3-3.

Table 3-3: Summary of modelled breakout scenarios

Flood scenario		Ashley River peak flow (m^3/s)	Peak breakout flow (m^3/s)			
			A	B	C	E/F
100 year ARI (1% AEP)	Breakout A	3470	500	-	-	-
	Breakout B	3470	-	500	-	-
	Breakout E/F	3470	-	-	-	125
200 year ARI (0.5% AEP)	Breakout A & B	4050	750	400	-	-
	Breakout A & C	4050	750	-	200	-
	Breakout A & E/F	4050	750	-	-	200
500 year ARI (0.2% AEP)	Breakout A, B & C	5300	1750	350	160	-
	Breakout A, B & E/F	5300	1750	250	-	250

These breakouts are fairly unpredictable, but will often depend on the river flows and the location of any other breakouts along the river system. For example, as the Ashley River peak flow increases between the 100 and 200 year ARI flood events, the peak breakout flow at location B decreases. The peak Ashley River flow, at location B, is less for the 200 year ARI event than the 100 year ARI event mainly because 750 m^3/s is assumed to have already flowed out of the Ashley River at location A.

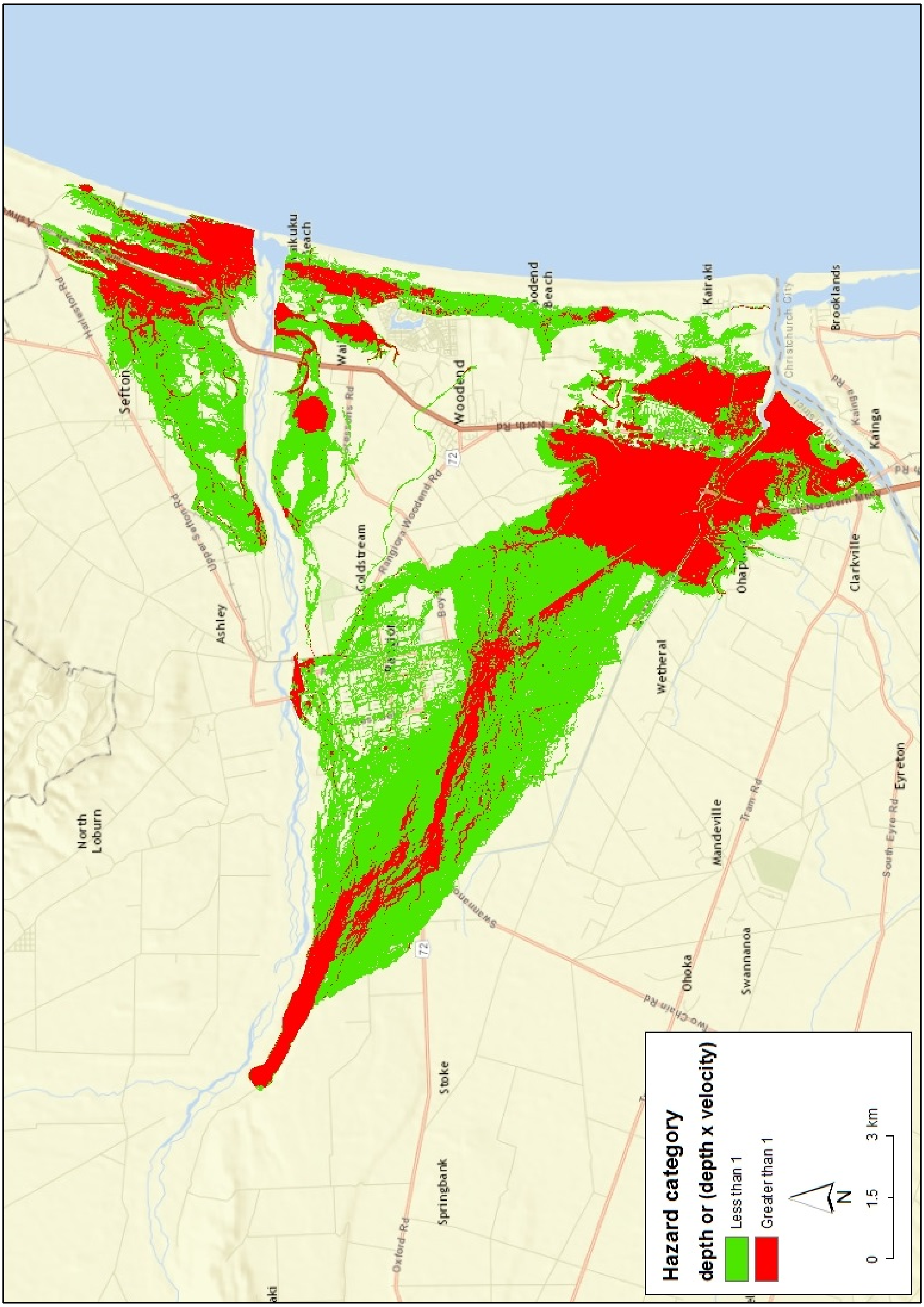


Figure 3-13: Ashley floodplain 'high hazard' areas (500 year ARI)

3.4.6 Model sensitivity analyses

As part of this study several sensitivity tests were undertaken. Oliver (2008) also modelled several additional scenarios to determine the sensitivity of maximum water depths and velocities to various model parameters. The sensitivity analyses undertaken in this investigation are summarised below.

<u>Tidal cycle versus constant downstream water level boundary</u>

As part of this study the effect of using a constant sea level versus a tidal cycle was examined using the 200 year ARI Breakout E/F scenario for the northern floodplain. The model was run using a constant downstream sea level at the Ashley River mouth of 1.7 m above msl, and using a tidal cycle with a high tide of 1.7 m above msl. In the tidal scenario, high tide occurred 2 hours before the Ashley River peak flow arrived at the river mouth. The peak floodplain breakout flow, which takes longer to travel over the floodplain, arrives at SH1 around low tide. For all scenarios modelled the outlet to the sea is assumed to be at the Ashley River mouth rather than exiting to the sea via the Saltwater Creek lagoon.

For a tidal cycle boundary, maximum water levels were up to ~0.03 m lower in Saltwater Creek lagoon and the tidal area upstream of SH1, when compared to the constant water level boundary. Therefore, the tidal boundary has a relatively small influence on maximum water levels. This is largely because the water levels in the Saltwater Creek lagoon area are strongly influenced by the maximum flood flows in the Ashley River. Larger river flows elevate the river levels, forcing river water back into the Saltwater Creek lagoon area.

<u>Mannings ‘n’ of 0.05 increased to 0.08 on the floodplain between the new stopbank and Breakout B (upstream of Cones Road)</u>

The Mannings ‘n’ floodplain roughness upstream of Cones Road (Rangiora), and between the existing and new stopbank, was initially given a value of 0.05. As a sensitivity test, this area of floodplain had the Mannings ‘n’ value increased to 0.08. Figure 3-14 shows that if the floodplain roughness is increased, overtopping of the new stopbank may occur for a 100 year ARI breakout B flow of 500 m^3/s. It is, therefore, very important that vegetation within the area between the existing and new stopbank is kept to a minimum.

<u>Effect of no fill in the road gaps in the new stopbank at Cones Road</u>

At present, the construction of the new stopbank has not included raising the roads to the same elevation as the new stopbank crest. This work to close the road gaps is scheduled to be completed in the 2017/18 financial year.

A sensitivity test was completed to determine the likely extent of flooding for a 100 year ARI breakout B flow of 500 m^3/s if the road gaps were not closed during a flood event. Figure 3-15 shows that the road gaps could potentially allow a reasonably large amount of water onto the floodplain – depending on the width and elevation of the stopbank gaps. Fortunately, the areas of inundation are those that have already been identified as being susceptible to flooding, and flood depths will be less than those predicted before the construction of the new stopbank.

<u>Floodplain roughness</u>

To assess the effect of using the WDC (2015) floodplain roughness, an additional model run was completed using the Oliver (2008) floodplain roughness (derived by NIWA) for the 100 year ARI breakout A flow of 500 m^3/s.

Figure 3-16 shows that, where floodplain water depths and/or water velocities were low, maximum water depths produced using both floodplain roughness models were within ±0.1 m (e.g. in the Flaxton swamp ponding area and the Rangiora urban area).

Along the main flow paths, where water depths and flow velocities were higher, maximum water depths for the WDC (2015) floodplain roughness model were generally up to 0.2 m higher (e.g. along the main flow path from Breakout A towards Kaiapoi).

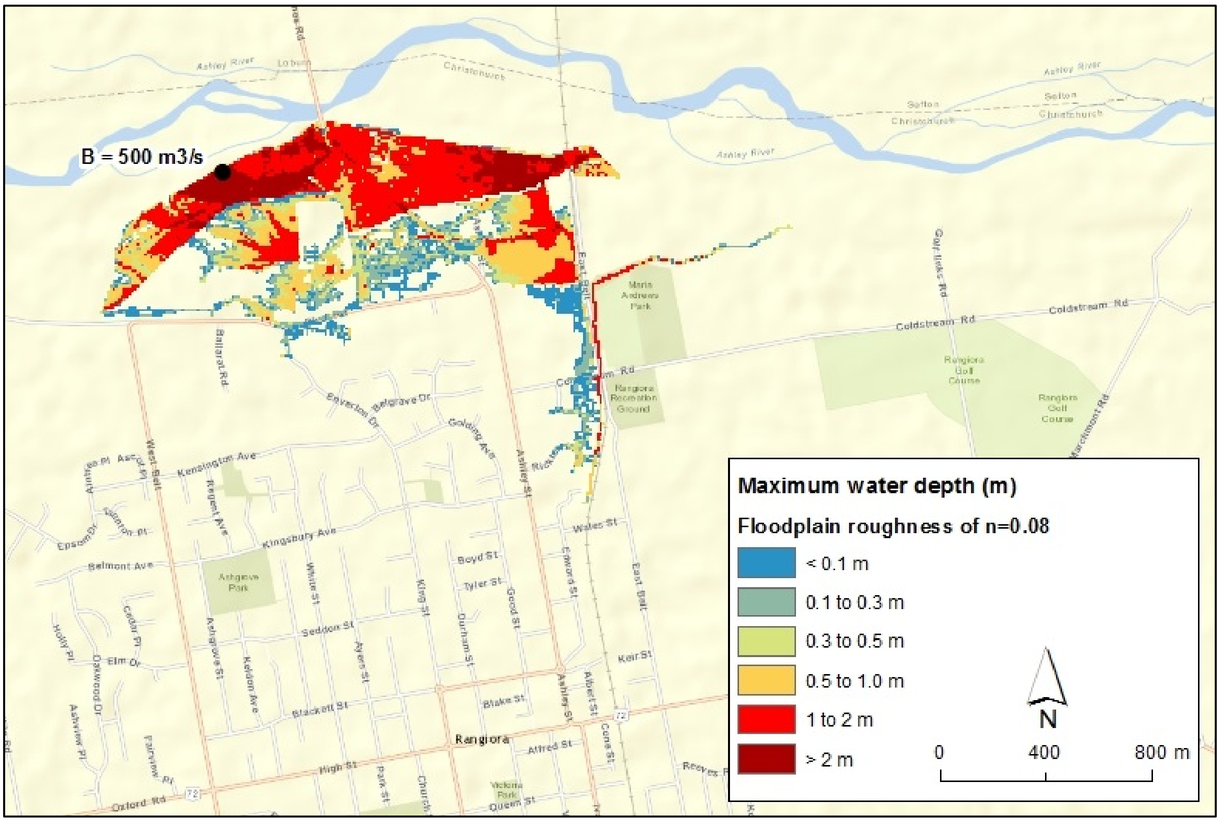


Figure 3-14: 100 year ARI (1% AEP) - Breakout B water depths when floodplain roughness increased from a Mannings 'n' of 0.05 to 0.08 upstream of Cone Road

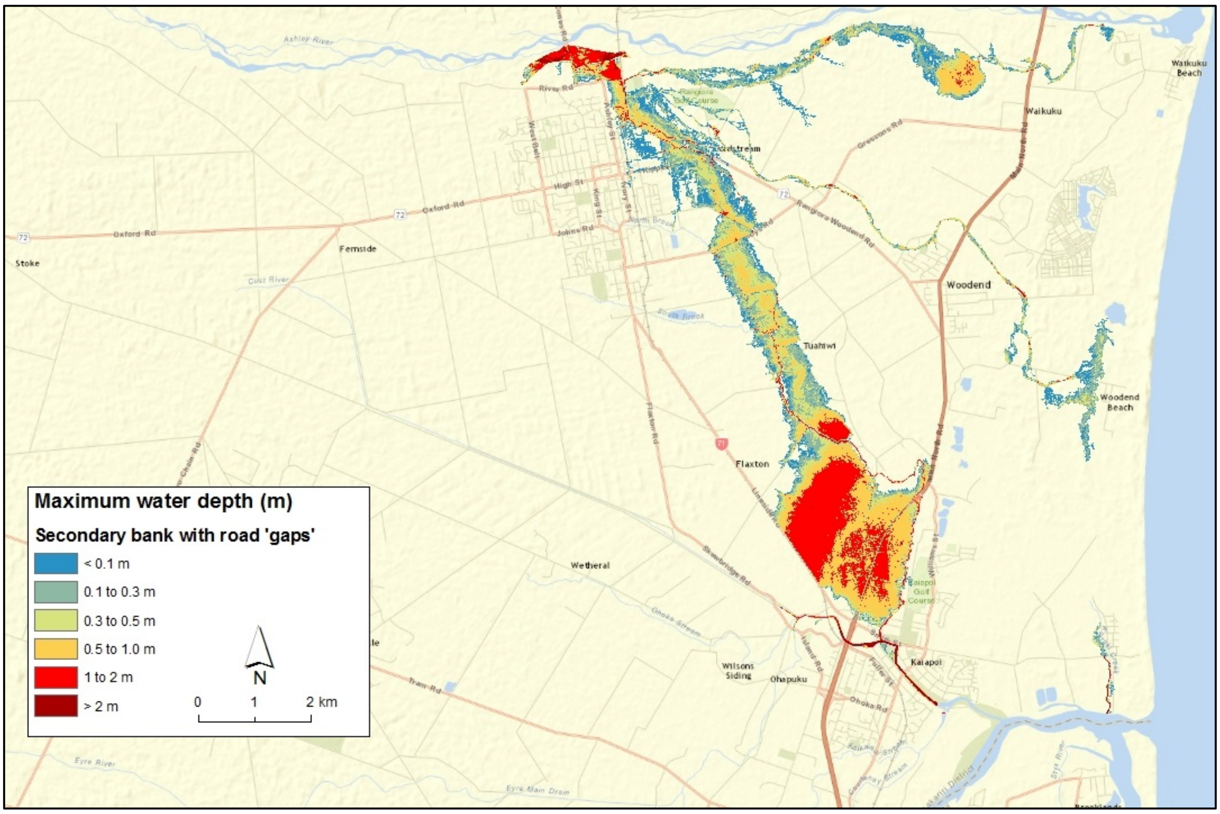


Figure 3-15: 100 year ARI (1% AEP) - Breakout B water depths for current situation with gaps in the new stopbank where there are roads

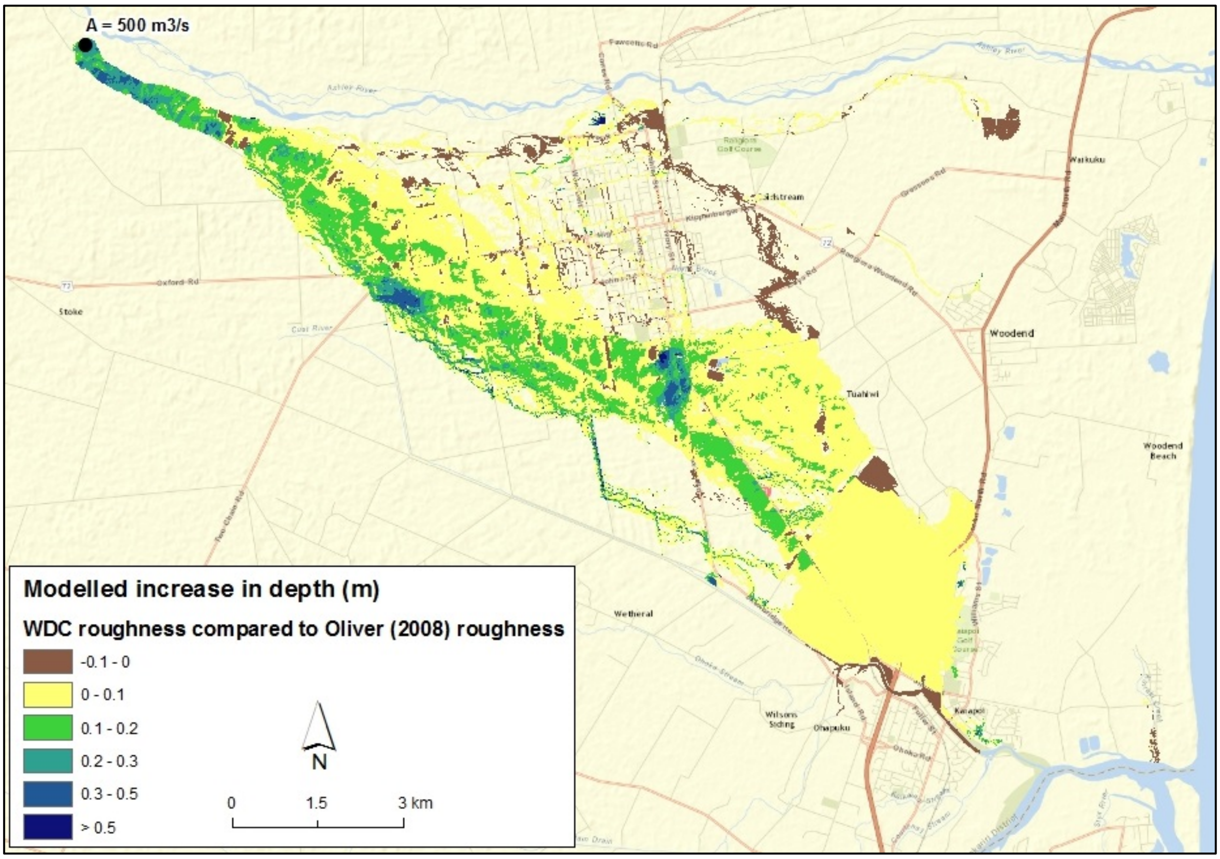


Figure 3-16: 100 year ARI (1% AEP) – Increase in water level with latest WDC (2015) floodplain roughness, compared to Oliver (2008) floodplain roughness

The extent of inundation was also greater for the WDC (2015) floodplain roughness model. For example, water backed up behind Easterbrook Road and flowed south from Mountvista Road when the WDC (2015) floodplain roughness model was used.

Increases in water depths of over 0.3 m were also noted for the WDC (2015) roughness model near the breakout source, in the Fernside area, and around Flaxton Road/Southbrook Road.

Overall, the WDC (2015) floodplain roughness model generally produces more conservative maximum water depths for the Ashley River floodplain. These floodplain roughness values are the same as those used by WDC for their latest modelling work.

<u>Breakout Flows</u>
There is considerable uncertainty in the magnitude and location of the breakout flows modelled. As a result, the analysis of breakout flows has been based on what is considered the most likely mid-range estimates.

When the 200 year ARI breakout flows were increased by 20% at locations A and B, flood depths upstream of Kaiapoi and Flaxton increased by up to 0.2 m (see Figure 3-17). In the ponding areas around Kaiapoi, the flood extent and depths are also greater.

This 20% flow increase is also an indication of the potential effects of climate change on rainfall by the end of the century.

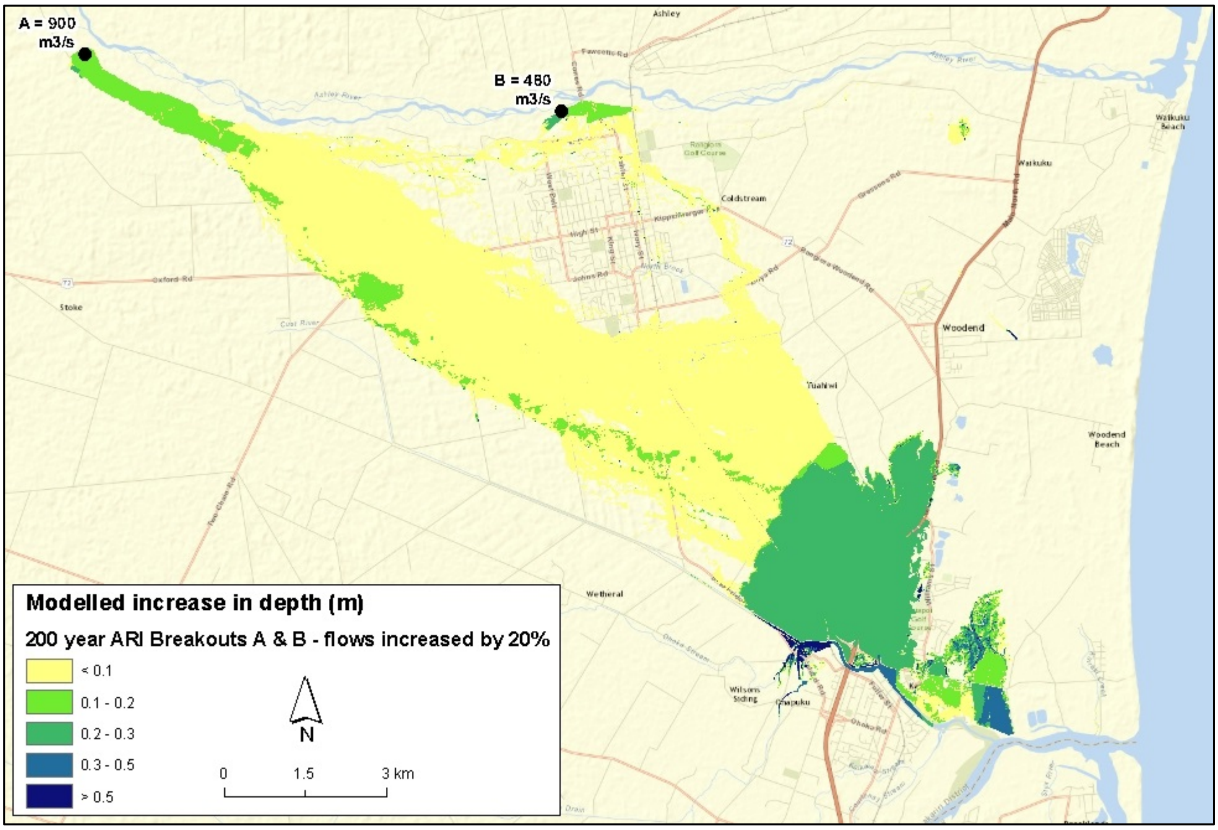


Figure 3-17: 200 year ARI (0.5% AEP) – Increase in water level when Breakout A and B flows onto the southern floodplain are increased by 20%

Sea level rise

Oliver (2008) noted that, for a 200 year ARI breakout scenario, sea level rise had a negligible impact on peak floodplain water levels as the peak levels behind the stopbanks are dominated by the large breakout flows. However, as floodwaters from the low lying areas will only be able to drain out during low tides, the time of inundation will increase. In times of local rainfall events, flooding in the Kaiapoi area (e.g. McIntosh's Drain and Courtney Stream catchments) is also likely to be worse, as floodwaters will only be able to drain when the tide level is lower.

For a 200 year ARI breakout flow onto the northern floodplain, Figure 3-18 shows that a 0.8 m increase in sea level generally increases maximum water levels by up to 0.3 m – but generally only in the area to the east of Factory Road that is already considered to be influenced by the tide.

For a 200 year ARI breakout flow (from Breakouts A and B) onto the southern floodplain, Figure 3-19 shows that a 0.8 m increase in sea level increases the maximum water levels in the Flaxton Swamp and Kaiapoi areas by less than 0.1 m (and in the Flaxton Swamp area this increase is only ~ 0.03 m). Maximum water levels in the Kaiapoi River, and Ohoka Stream, also increase as far upstream as the Island Road area.

The Kairaki (Saltwater) Creek eastern bank has been rebuilt post-earthquakes with a crest level of 2.5 m above mean sea level. This is the same height as the estimated maximum tide level when 0.8 m of sea level rise is taken into consideration. Therefore, sea level rise combined with high flows in the Waimakariri River are likely to elevate water levels in the Kairaki Creek area, overtopping the existing stopbanks. This would allow water to flow into the Kairaki settlement and surrounding area. As sea level rise takes place, stopbank heights are likely to be reviewed (by the rating areas they affect) to address the level of protection required for various sea level rise scenarios.

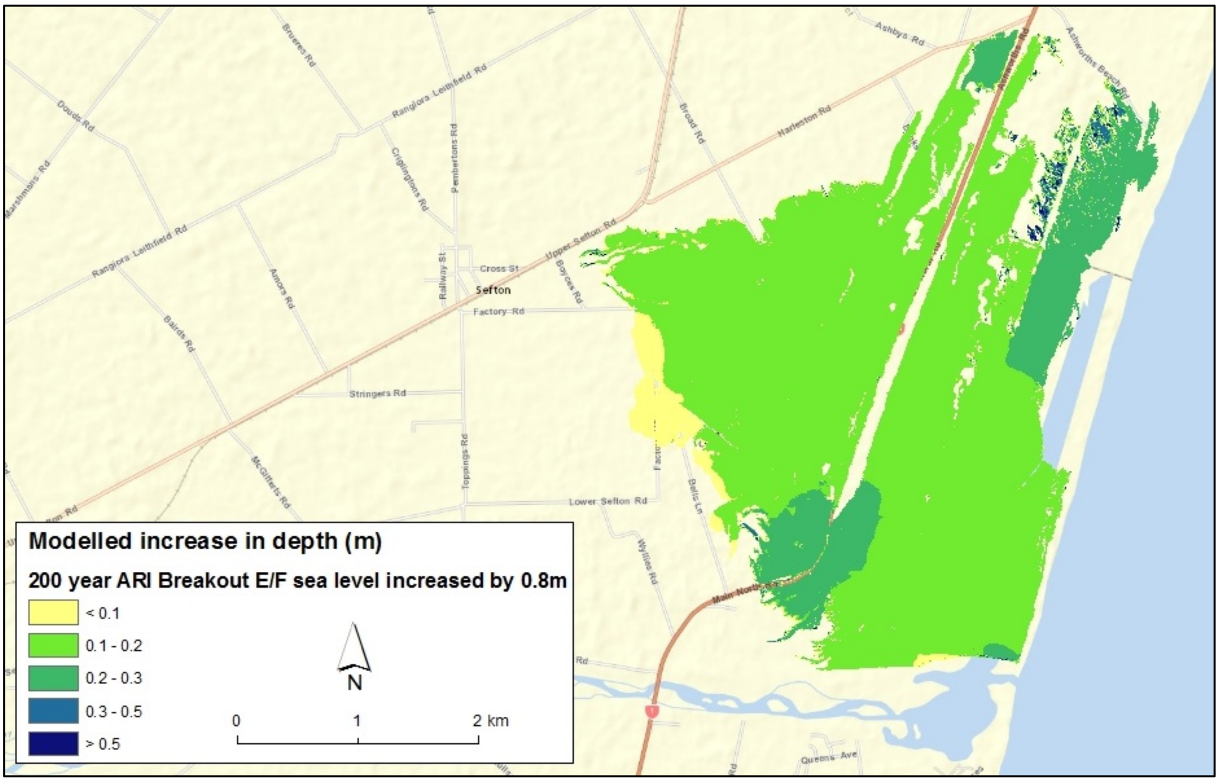


Figure 3-18: 200 year ARI (0.5% AEP) – Increase in water level when the Breakout E/F Ashley River tide boundary is increased by 0.8 m for the northern floodplain

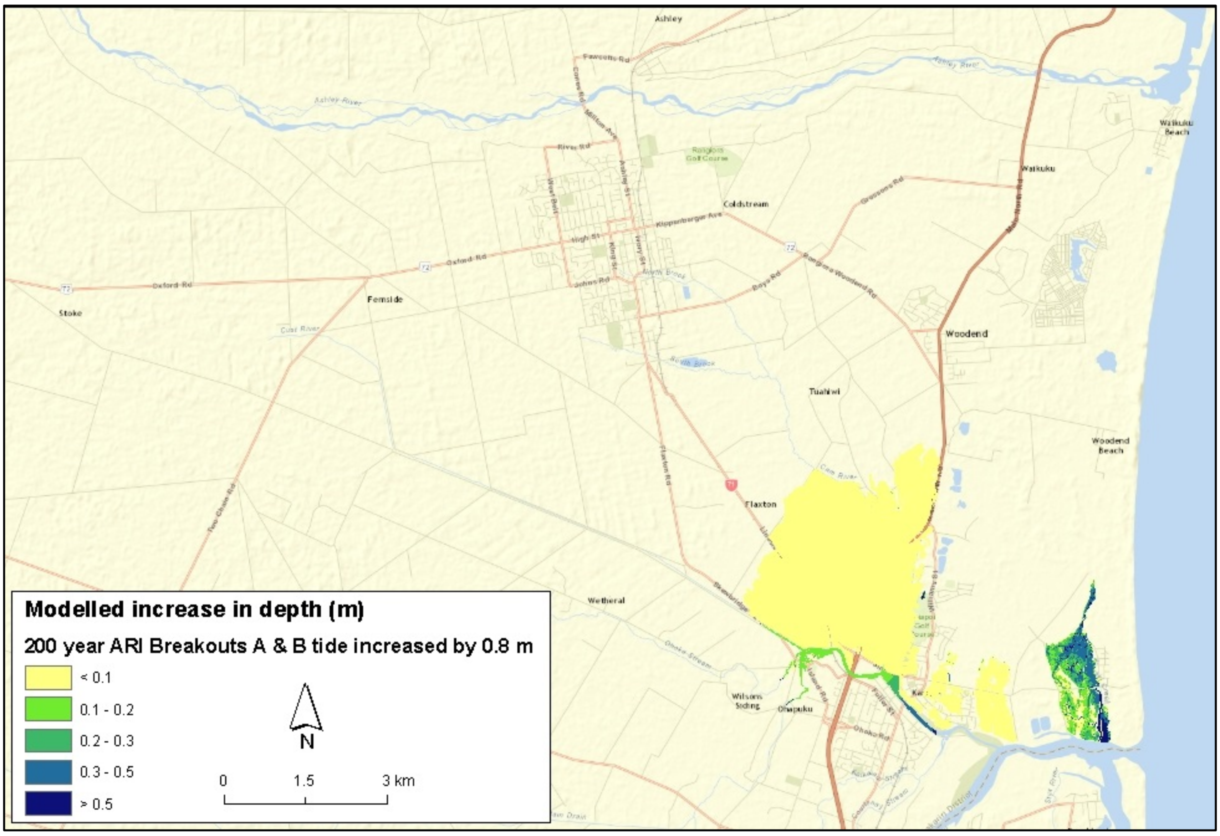


Figure 3-19: 200 year ARI (0.5% AEP) – Increase in water level when the Breakout A and B Ashley and Waimakariri River tide boundaries are increased by 0.8 m for the southern floodplain

Channel roughness - Lower Waimakariri River (Oliver, 2008)
The adopted Mannings ‘n’ channel roughness of 0.017, for the lower 5 km reach of the Waimakariri River, was obtained from calibration with observed flood levels. Increasing this relatively low value to a Mannings ‘n’ of 0.022 resulted in an increase in maximum water level at the Kaiapoi/Waimakariri River confluence of ~ 0.1 m.

The change to water depths on the floodplain is negligible since the levels are dominated by the ponding behind stopbanks and the large breakout flows/volumes. There would be some small effect on flood duration. However, this would be dependent on the relative timing of Waimakariri River peak flows, peak breakout flows from the Ashley River, and high tide.

3.4.7 Summary of model results

Southern floodplain
The difference in flood extent and damages between the 100 year ARI and the 500 year ARI events is significant. This is because the 500 year ARI scenario diverts over four times as much water onto the floodplain as the 100 year ARI Breakout A scenario.

Generally, Rangiora is relatively safe from Ashley River flooding. Waikuku and Woodend Beach settlements are likely to be flooded in some scenarios, although many of the houses at Waikuku Beach are located on higher sand hills.

Flooding to Kaiapoi and peripheral areas is potentially extensive and deep. This is because:

- Kaiapoi is at the lower end of the Ashley River fan in a location where floodwaters pond behind the Kaiapoi and Waimakariri River stopbanks,
- Drainage of the floodwaters are impeded by high tides and/or Waimakariri River levels, and
- Most of the land is very low lying and just above mean sea level.

Sensitivity analyses showed that the largest variation in modelled maximum water depths was due to breakout flows and location, although floodplain roughness can also have a significant effect on water levels along the main flow paths where velocities are higher.

For a 500 year ARI event, a large part of Kaiapoi and adjacent areas are classified as high hazard due to ponded depths over 1 metre. This includes the areas in Kaiapoi that have been recently classified as ‘red zone’ (post-earthquakes), but excludes the elevated new subdivisions.

Northern floodplain
In the Saltwater Creek lagoon area downstream of SH1, maximum water levels for the 100, 200 and 500 year ARI breakout flows are all within 0.02 m.

In the tidally-influenced area upstream of SH1, maximum water levels increase by up to 0.2 m as the breakout flow increases from a 100 year ARI flow (125 m^3/s) to a 500 year ARI flow (250 m^3/s). Further upstream, maximum water levels on the floodplain also increase as the breakout flow increases.

3.4.8 Model uncertainty

The estimates of floodplain water depths, water speeds and the extent of floodplain inundation have numerous sources of uncertainty that need to be considered when using the results. Bales and Wagner (2009) outline some of the uncertainties associated with one-dimensional hydraulic modelling. These uncertainties are also relevant for this modelling investigation and include:

- Model inputs (e.g. breakout locations and volumes, roughness values).
- Topographic data (e.g. LiDAR data).
- Hydraulic model assumptions (e.g. simplification of equations by depth-averaging, as well as averaging topography and flow behaviour over a 10 m grid cell for computational efficiency).

Oliver (2008) identifies many assumptions which need to be made in river and floodplain investigations. They include aspects summarised below.

Hydrology: with only a relatively short local record there are uncertainties, particularly when estimating extreme events.

Topography: LiDAR data provide very good detail of the ground surface. However, when converting these data to a grid for modelling purposes, there can be some loss of definition.

Breakout location and magnitude: this is based on the risk assessment. While the most likely locations and midrange estimates for magnitude have been selected, a large number of scenarios are possible.

River bed levels: bed levels (and therefore maximum modelled water levels) will vary over time in gravel-bed river channels such as the Ashley and Waimakariri rivers. For this modelling investigation, these changes in bed level (and water level) will have little effect on the floodplain model results. This is because the one-dimensional river models are only linked to the floodplain at one or two locations. For example, the Ashley River is only connected to the floodplain at the new stopbank to the north of Rangiora and near the river mouth. Should bed levels change significantly in these areas, then there may be a more noticeable change in floodplain flows and water depths.

Channel and floodplain roughness – as channel roughness increases, so too will water levels. If measured water levels are available, models can be calibrated by adjusting the roughness value until modelled water levels match measured water levels for a given flow. However, often there is little or no flow and/or water level data measured during flood events. Roughness is often, therefore, assumed based on the river channel or floodplain surface. This can be somewhat subjective, although previous investigations on similar rivers and floodplains, along with published material, can provide some confidence in any assumed values. Sensitivity tests are also able to provide an indication of the sensitivity of modelled water levels to channel and/or floodplain roughness.

Taking into consideration the above parameters, the Oliver (2008) river and floodplain model was built to represent reality as closely as possible. The model was peer reviewed by DHI Water and Environment and found to be well developed, provided a realistic description of the physical situation and was suitable for the intended purpose. Relatively minor modifications and improvements to the original Oliver (2008) model have been made for this investigation so it was not considered necessary for the model to be peer reviewed externally again.

Both Oliver (2008) and this investigation do not include localised runoff from rainfall. If required, WDC could combine this latest modelling with their latest local stormwater models.

Localised barriers to flow, such as hedges and fences, also affect water depths during a flood event as they provide resistance to flows, elevating water depths upstream of the barrier and diverting flow – more so if the barriers trap debris.

Sensitivity tests help to address the uncertainties present in the modelling results and provide a good indication of the range of water depths that may be expected on the Ashley floodplain. However, because of this uncertainty, the modelling results should only be interpreted and used by those who are familiar with all aspects of the modelling. For design purposes, freeboard is normally added to modelled values to allow for some uncertainty.

4 Conclusions and recommendations

A series of potential breakout flows, at several locations, have been modelled to determine likely maximum water depths and flows on the Ashley River floodplain. Results from the modelling are provided in Section 3.4 as flood inundation maps (Figure 3-5 to Figure 3-10 and Figure 3-12). A flood hazard map is also provided for the 500 year ARI breakout scenarios (Figure 3-13). This information will be used to assist with land use planning (e.g. to provide flood hazard advice for future development of land and minimum floor levels) and emergency management purposes (e.g. evacuation planning).

It is also recommended that:

- The model results produced in this study are reassessed when updated hydrology and topographical data become available. Additionally, if maximum flows and water levels are measured downstream of any breakout (during or immediately after a large flood event), the computational hydraulic model should be calibrated.

- No further planting of trees/vegetation should occur in the area between the existing stopbank and the newly constructed stopbank. This is because, if the resistance to floodplain flow is further increased, there will be a greater likelihood that the new stopbank will be overtopped upstream of Cones Road.

- A solution for the gaps in the new stopbank (i.e. at Cones Road and Milton Avenue) is given priority, to reduce the flood risk from breakouts at location B.

5 Acknowledgments

The following Environment Canterbury staff have reviewed this report and provided valuable input to this study:

- Tony Boyle (Principal Hazards Analyst) - provided text on the background of the river control works, reviewed report and reviewed modelling results.

- Nick Griffiths (Hazards Analyst) – reviewed the model results and report.

- Shaun McCracken (Senior River Engineer) – reviewed the report and provided information on the new stopbank adjacent to Rangiora.

6 Glossary

Aggradation: Deposition of shingle in a river, raising the river bed level.

Alluvial fan: A cone-shaped deposit of alluvium (shingle and silt deposition) made by a stream or river where it runs out onto a level plain. The fans generally form where the stream or river issue from the hills onto the lowlands.

Annual exceedance probability (AEP): The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means there is a 5% chance (i.e. a chance of one-in-twenty) of a peak flood discharge of 500 m^3/s or larger occurring in any one year. AEP is the inverse of average recurrence interval (ARI), expressed as a percentage.

Average recurrence interval (ARI): The average time period between floods, equivalent to or exceeding a given magnitude. For example, a 100 year ARI flood has a magnitude expected to be equalled or exceeded an average of once every 100 years. Such a flood has a 1% chance of being equalled or exceeded in any given year, i.e. 1% AEP. ARI is often used interchangeably with 'return period' or 'flood frequency'.

Catchment: The land area draining through the main stream and tributaries to a particular site.

Discharge: The rate of flow of water measured in terms of volume per unit time, e.g. cubic metres per second.

Fairway: The open (ideally vegetation-free) area of the riverbed that carries the majority of any flood flow. There is often a maintenance program in place for clearance of vegetation such as willows, gorse and broom from the fairways.

Floodplain: The area of relatively flat land, which is inundated by floodwaters from the upper catchment up to the probable maximum flood event.

Geomorphology: The study of the topographic and geologic form of the earth's surface and the changes that take place in the evolution of land forms.

LiDAR (Light Detection and Ranging) data: Data acquired using a laser scanner mounted on an aircraft. The scanner measures the ground level at approximately one point every square metre. This point data are used to generate very accurate and high resolution digital elevation maps which enable subtle topographic features to be identified.

One dimensional model: Uses river channel cross sections along a river. Flows and depths are calculated at discrete points along the channel. Flow is in one direction only, i.e. x direction along the channel.

Orographic Rain: Rainfall (often intense) resulting when moist air is forced to rise by hills/ranges/mountains.

Residual Risk: The hazard a community is exposed to after floodplain management measures have been put in place. For example, for stopbanked areas the residual risk is the continuing flood hazard of stopbanks being overtopped and the associated consequences.

Two dimensional model: In this study, a rectangular grid of the floodplain topography is used in the model. Flows are calculated in both the x and y direction for each grid cell. Depths are also calculated for each grid cell.

7 References

Bales, JD and Wagner, CR 2009: *Sources of uncertainty in flood inundation maps*. Journal of Flood Risk Management, Volume 2, Issue 2, p 139-147.

Bell, R.G. (2011). *Sumner Head Sea-Level Station: Annual Report for 2009-2010*. NIWA Client report HAM2011-051. Prepared for Environment Canterbury. June 2011.

Boyle, T; Surman, M 2008: *Waimakariri District flood hazard management strategy – Ashley floodplain risk assessment*. Report No. U08/1. Prepared January 2008.

Boyle, T; Surman, M 2009: *Ashley River bed level investigation*. Report No. R09/71. Prepared September 2009.

Canterbury Regional Council (CRC) 1995: *Ashley River floodplain management regional plan – Technical investigation*. Report No. 95(6). May 1995.

Canterbury Regional Council (CRC) 2014: *Canterbury Regional Council Asset Management Plan – Ashley River/Rakahuri Control Scheme*. September 2014.

Cowie. CA 1957: *Floods in New Zealand 1920-53*. The Soil Conservation and Rivers Control Council, Wellington, New Zealand.

Griffiths, G; Pearson, C; McKerchar, A. 2009: *Review of Ashley River flood frequency at Rangiora Traffic Bridge*. NIWA Client report CHC2009-103. Prepared for Environment Canterbury. July 2009.

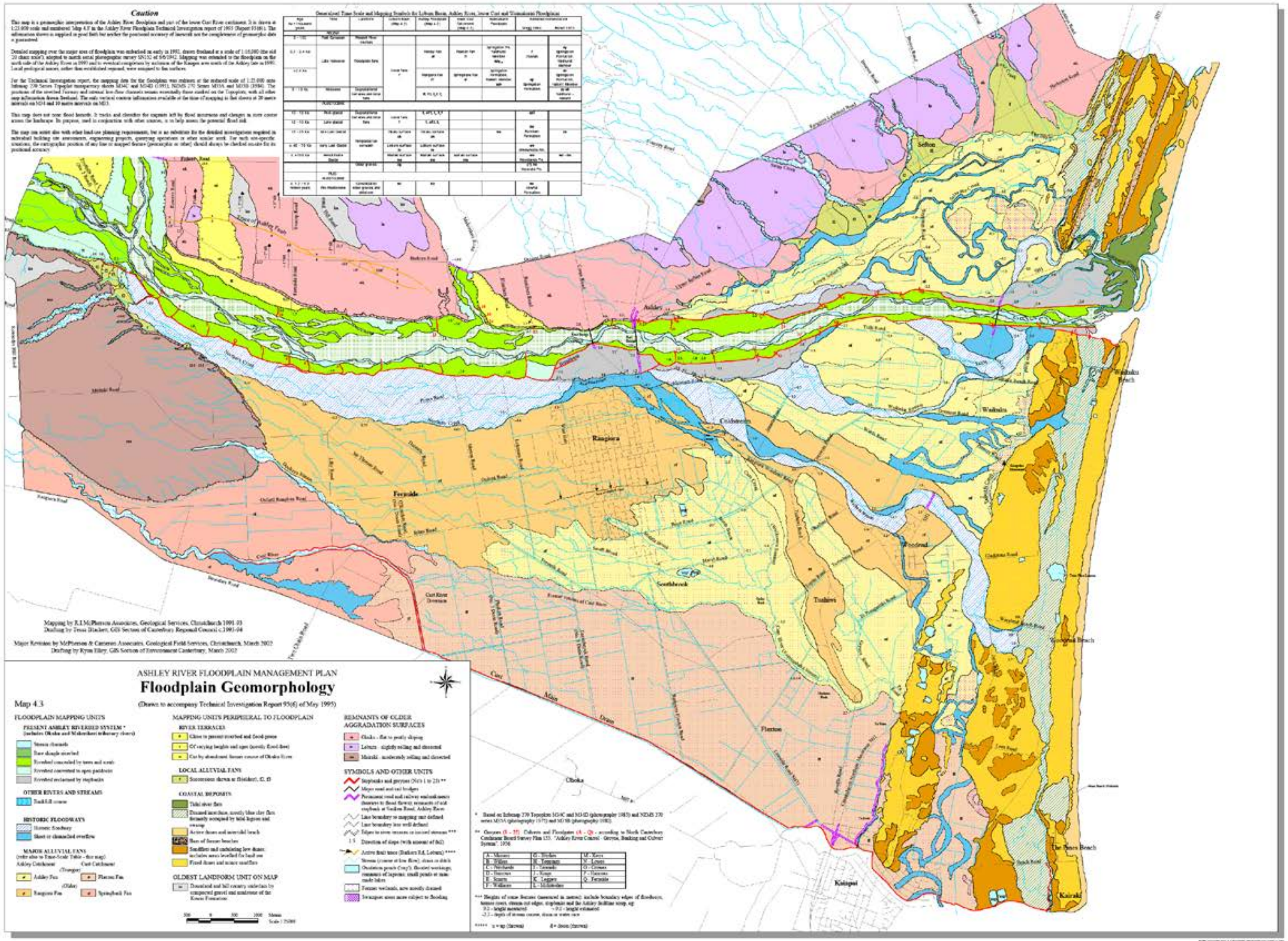
Ministry for the Environment (MfE) 2008: Preparing for coastal change: A guide for local government in New Zealand. Part One: Changing climate. Publication reference number ME 907. Released July 2008. http://www.mfe.govt.nz/publications/climate-change/preparing-coastal-change-guide-local-government-new-zealand/part-one

Ministry for the Environment (MfE) 2010: Preparing for future flooding: A guide for local government in New Zealand. Publication reference number ME 1012. May 2010. http://www.mfe.govt.nz/publications/climate-change-hazards/preparing-future-flooding-guide-local-government-new-zealand.

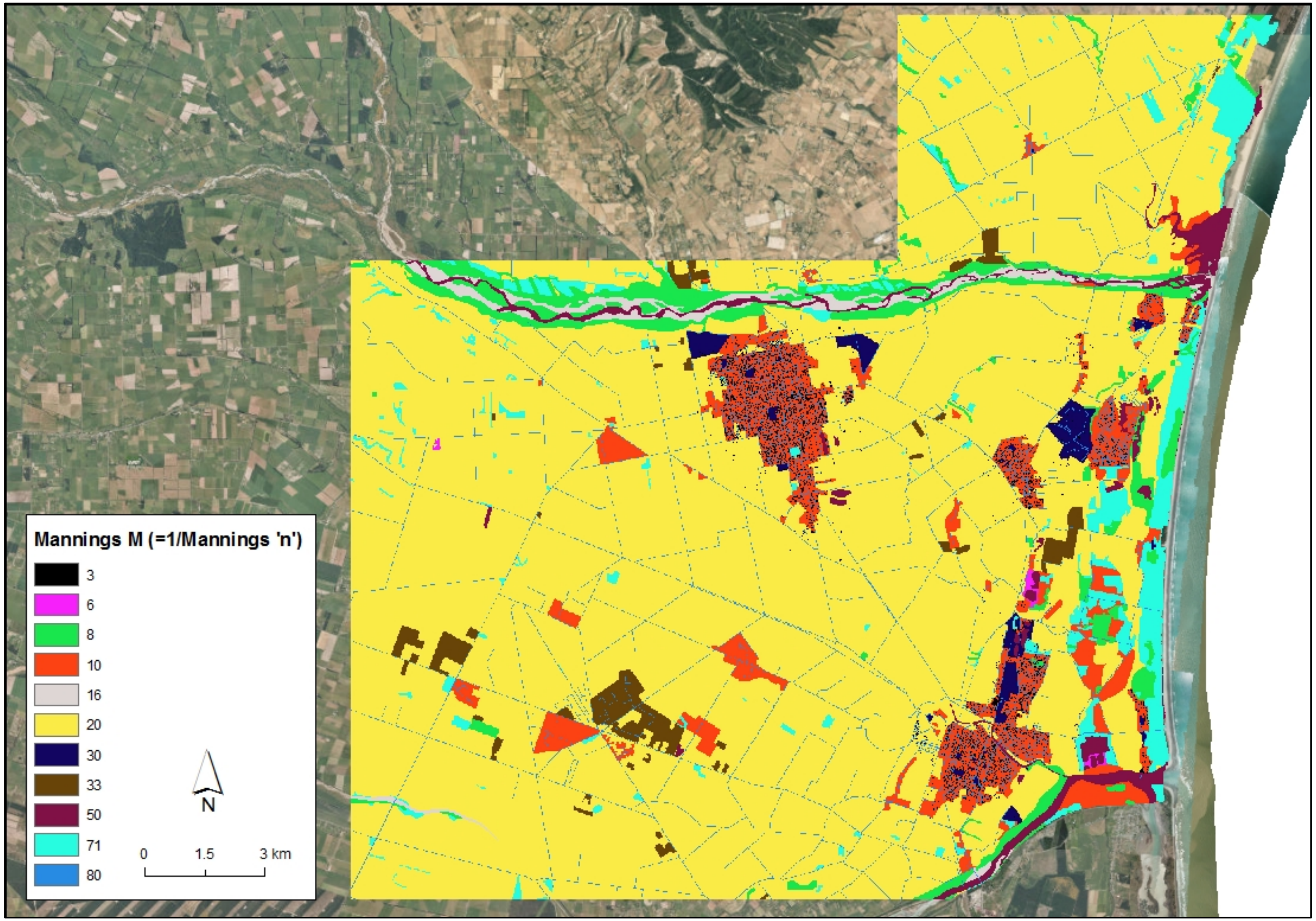
Oliver, T 2008: *Waimakariri District flood hazard management strategy – Ashley River floodplain investigation*. Report No. R08/23. Prepared June 2008.

Waimakariri District Council (WDC) 2015*: Localised flood hazard assessment 2015*. Project Delivery Unit (PDU) Project Number PD000362. Published July 2015. Document Number 150410056887.

Appendix A: Ashley floodplain geomorphology



Appendix B: Ashley floodplain roughness from WDC (2015)

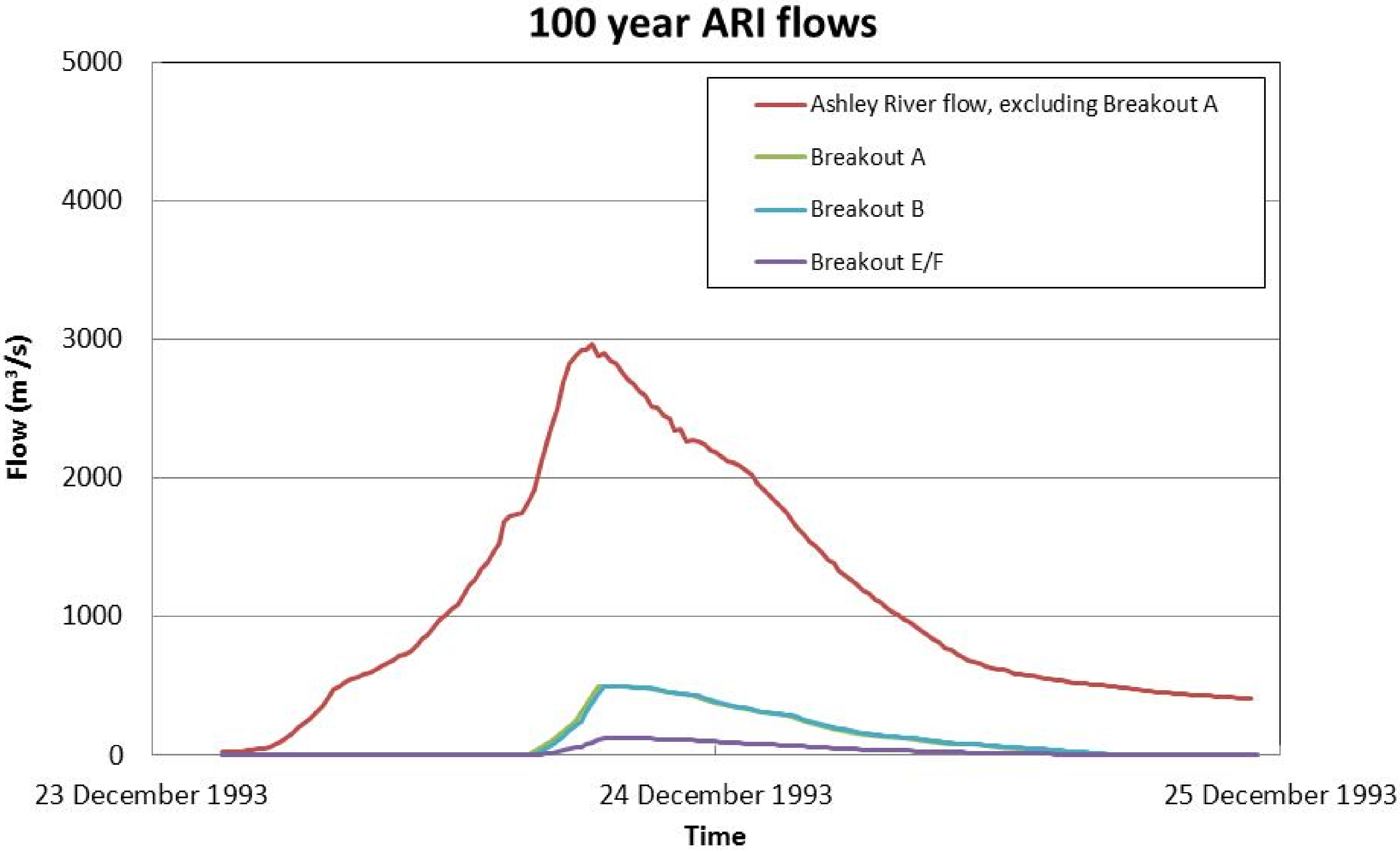


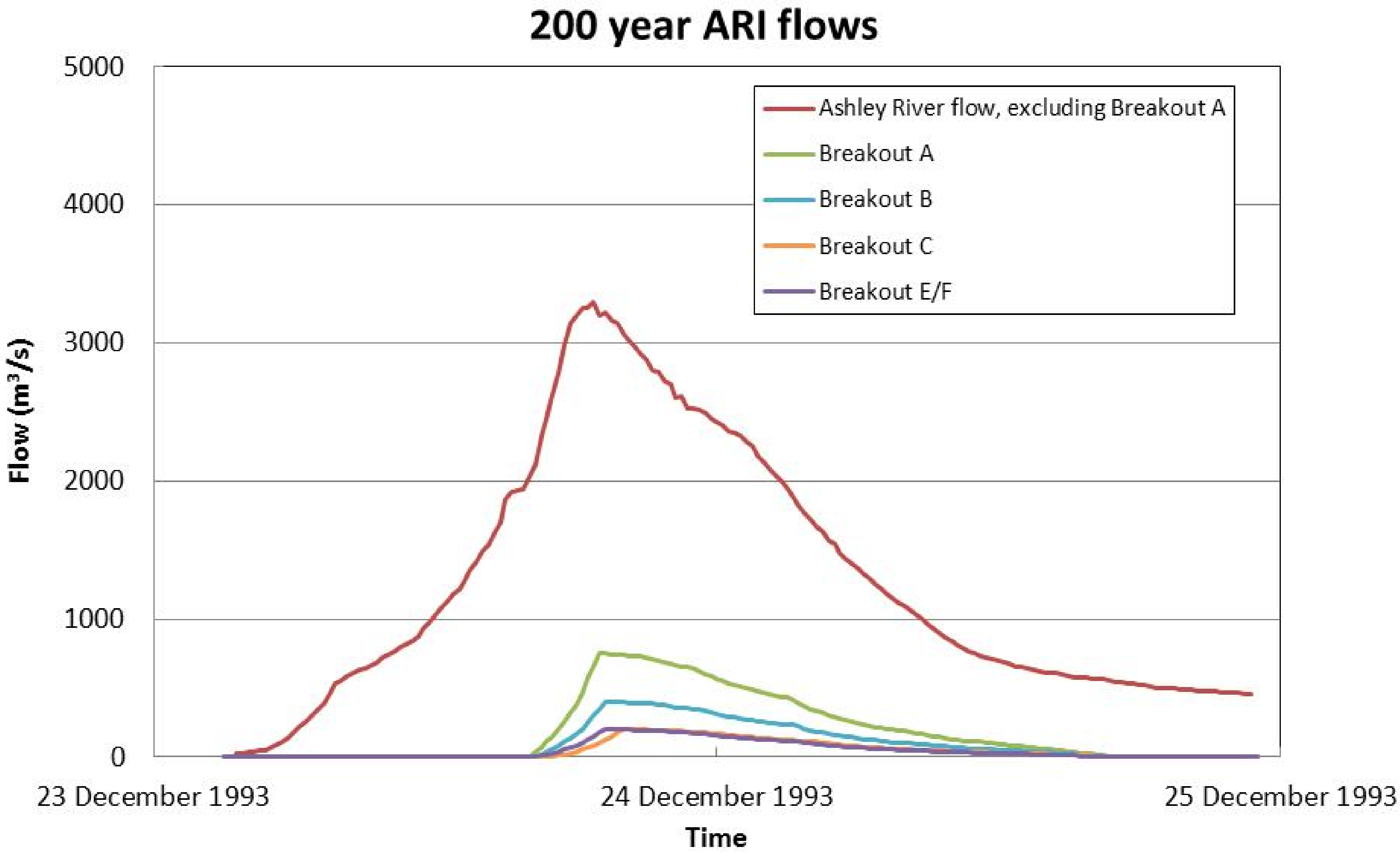
Floodplain roughness information from WDC (2015)

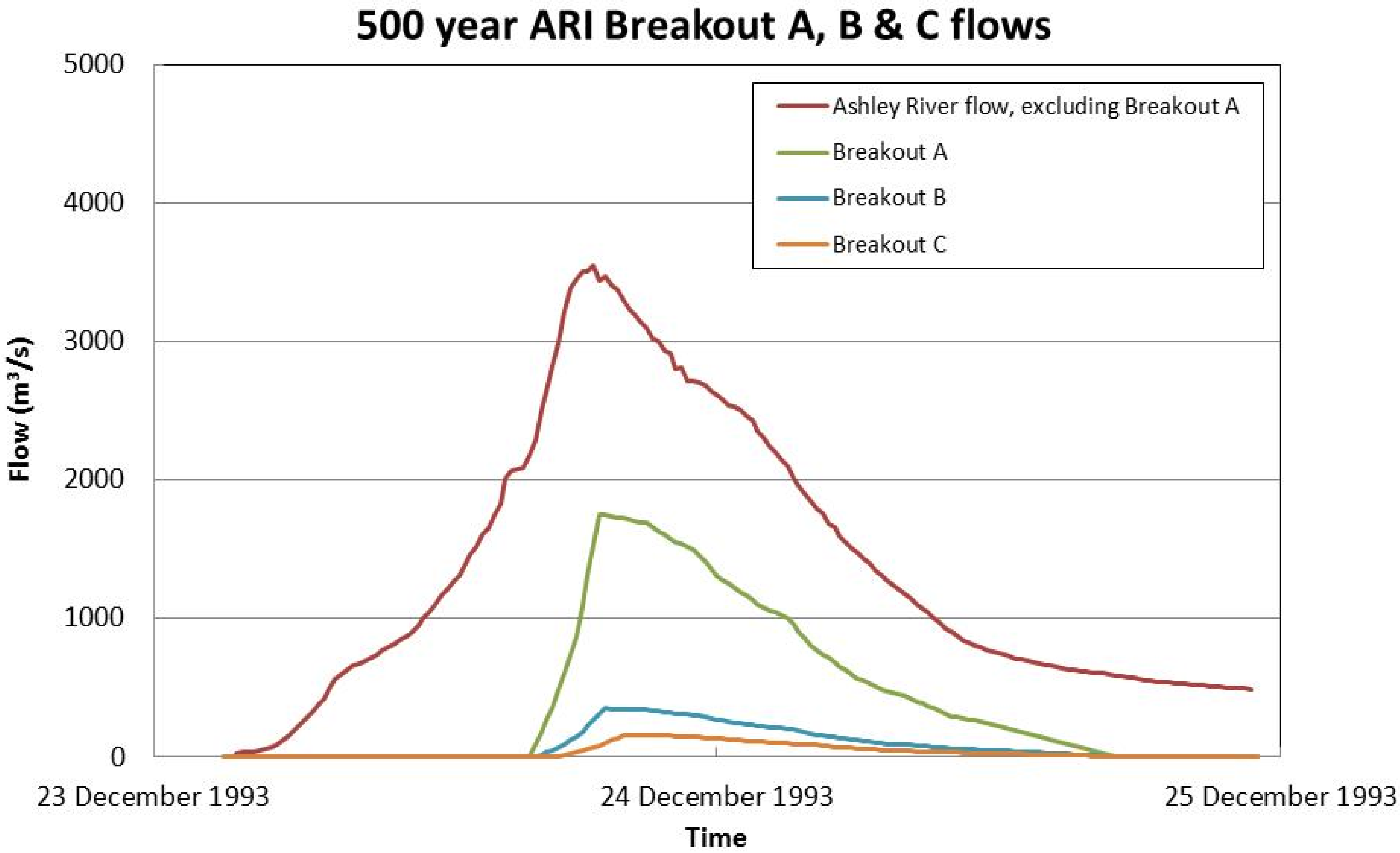
Table 5 – LCDB4 Land Types and Corresponding Mannings Factors

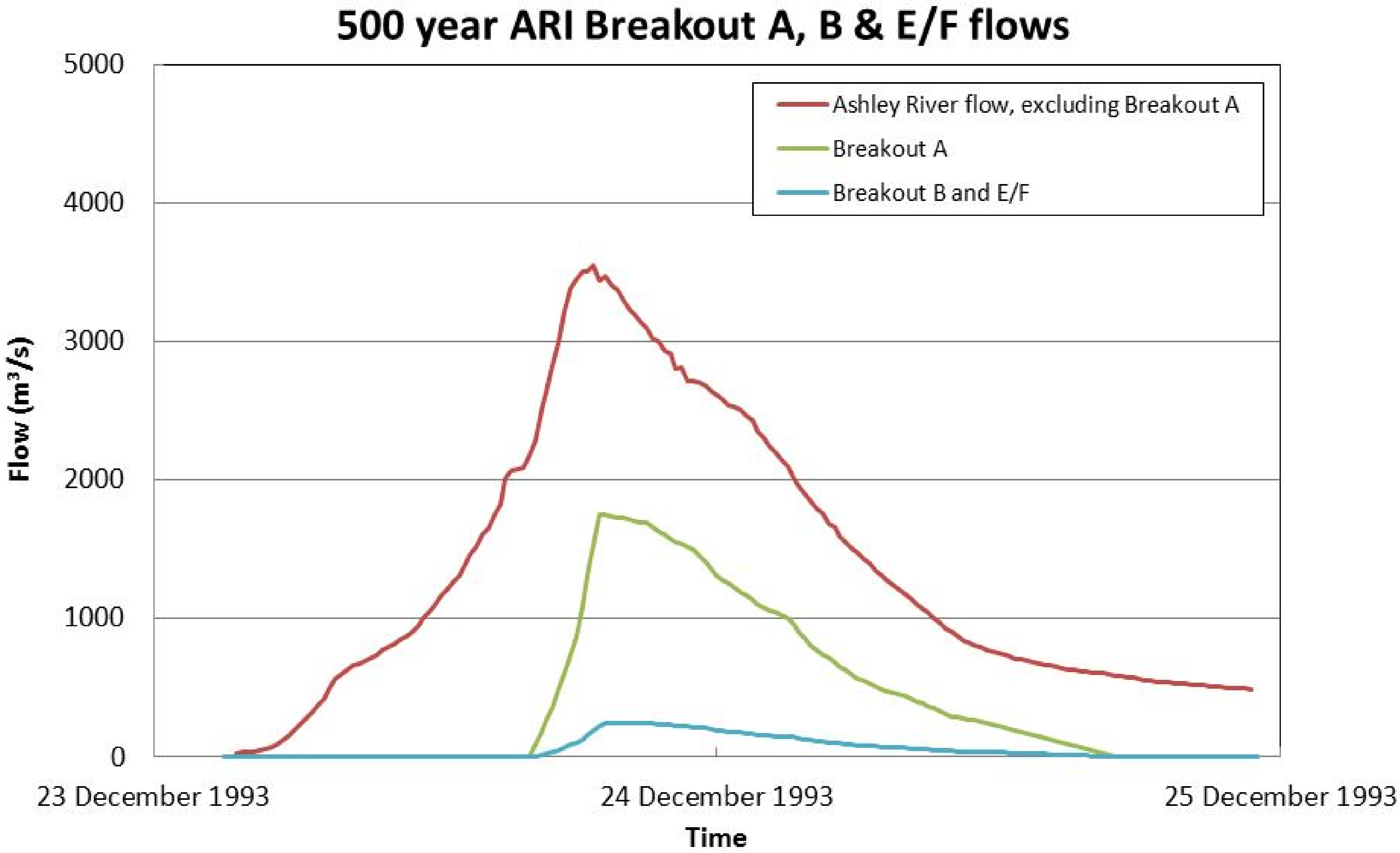
Description (based on LCDB3)	Code	n	M
Built-up Area	1	0.050	20
Urban Parkland / Open Space	2	0.033	30
Surface Mine	3	0.050	20
Dump	4	0.125	8
Transport Infrastructure	5	0.100	10
Coastal Sand and Gravel	10	0.020	50
River and Lakeshore Gravel and Rock	11	0.020	50
Landslide	12	0.020	50
Alpine Gravel and Rock	13	0.020	50
Alpine Grass-/Herbfield	15	0.050	20
Lake and Pond	20	0.020	50
Estuarine Open Water	22	0.020	50
Short-rotation Cropland	30	0.050	20
Vineyard	31	0.125	8
Orchard and Other Perennial Crops	32	0.125	8
High Producing Exotic Grassland	40	0.050	20
Low Producing Grassland	41	0.100	10
Tall Tussock Grassland	43	0.050	20
Herbaceous Freshwater Vegetation	45	0.100	10
Herbaceous Saline Vegetation	46	0.100	10
Flaxland	47	0.050	20
Gorse and Broom	51	0.125	8
Manuka and or Kanuka	52	0.125	8
Sub Alpine Shurbland	55	0.050	20
Mixed Exotic Shrubland	56	0.050	20
Major Shelterbelts	61	0.125	8
Afforestation (not imaged)	62	0.125	8
Afforestation (imaged, post LCDB 1)	63	0.125	8
Forest Harvested	64	0.125	8
Pine Forest – Open Canopy	65	0.125	8
Pine Forest – Closed Canopy	66	0.125	8
Other Exotic Forest	67	0.125	8
Deciduous Hardwoods	68	0.125	8
Indigenous Forest	69	0.125	8
Mangrove	70	0.100	10
Roadway	n/a	0.013	80
Building	n/a	0.333	3

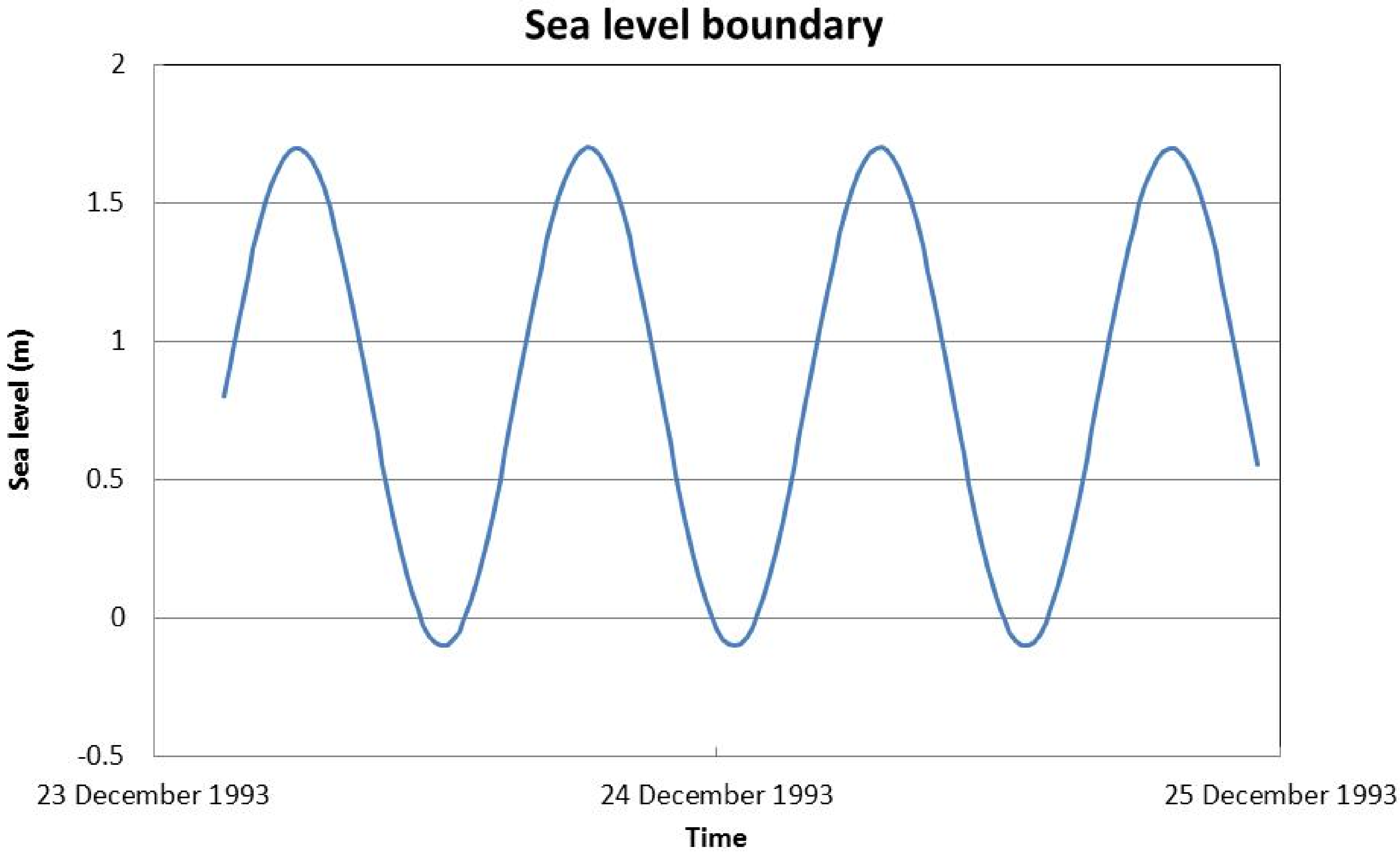
Appendix C: Ashley River & breakout flow hydrographs and tide boundary




200 year ARI flows
Ashley River flow, excluding Breakout A
Breakout A
Breakout B
Breakout C
Breakout E/F
5000
4000
3000
2000
1000
0
Flow (m³/s)
23 December 1993
24 December 1993
25 December 1993
Time

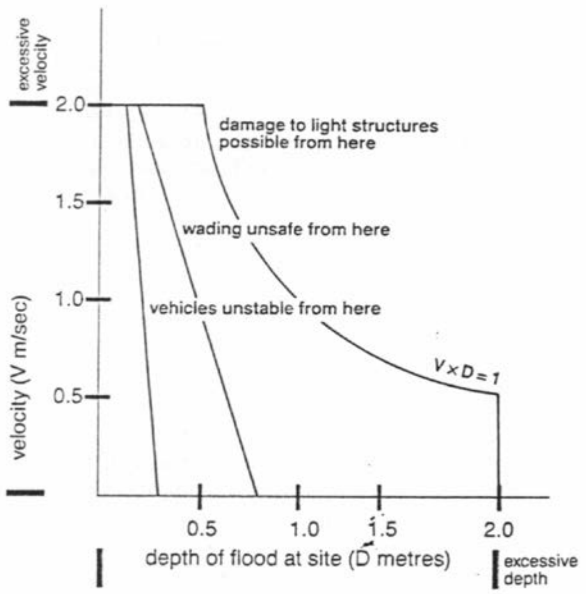

500 year ARI Breakout A, B & C flows
Ashley River flow, excluding Breakout A
Breakout A
Breakout B
Breakout C
5000
4000
3000
2000
1000
0
Flow (m³/s)
23 December 1993
24 December 1993
25 December 1993
Time


500 year ARI Breakout A, B & E/F flows
Ashley River flow, excluding Breakout A
Breakout A
Breakout B and E/F
Flow (m³/s)
5000
4000
3000
2000
1000
0
23 December 1993
24 December 1993
25 December 1993
Time


Sea level boundary
Sea level (m)
2
1.5
1
0.5
0
-0.5
23 December 1993
24 December 1993
25 December 1993
Time

Appendix D: Hazard category definition

FIGURE 6 Velocity and Depth Relationships
(Derived from laboratory testing and flood conditions which caused damage)



NOTES:

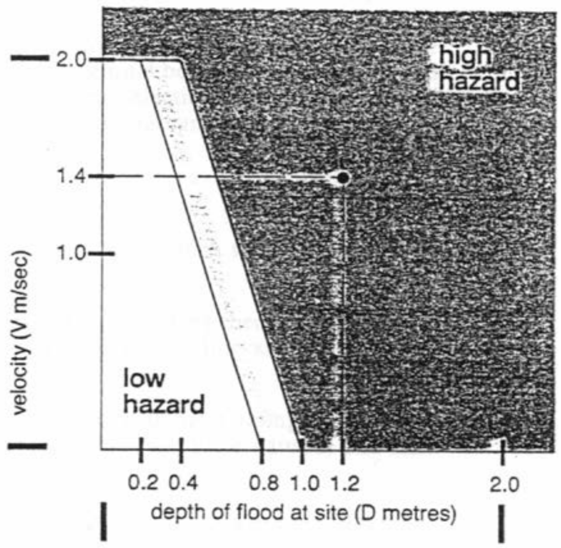
1. At velocities in excess of 2.0m/sec, the stability of foundations and poles can be affected by scour. Also, grass and earth surfaces begin to scour and can become rough and unstable.

2. The velocity of floodwaters passing between buildings can produce a hazard, which may not be apparent if only the average velocity is considered. For instance, the velocity of floodwaters in a model test has risen from an average of 1m/sec to 3m/sec between houses.

3. Vehicle instability is initiated by buoyancy.

4. At floodwater depths in excess of 2.0 metres and even at low velocities, there can be damage to light-framed buildings from water pressure, flotation and debris impact.

FIGURE 7 Provisional Hazard Categories



NOTE:
The degree of hazard may be either —

- reduced by the establishment of an effective flood evacuatic procedure.
- increased if evacuation difficulties exist.

Within area the degree of hazard is dependent on s conditions and the nature of the proposed development.

EXAMPLE:
If the depth of flood water is **1.2m**
and the velocity of floodwater is **1.4m/sec**

then the provisional flood hazard is **high**

Appendix E: Flood probability

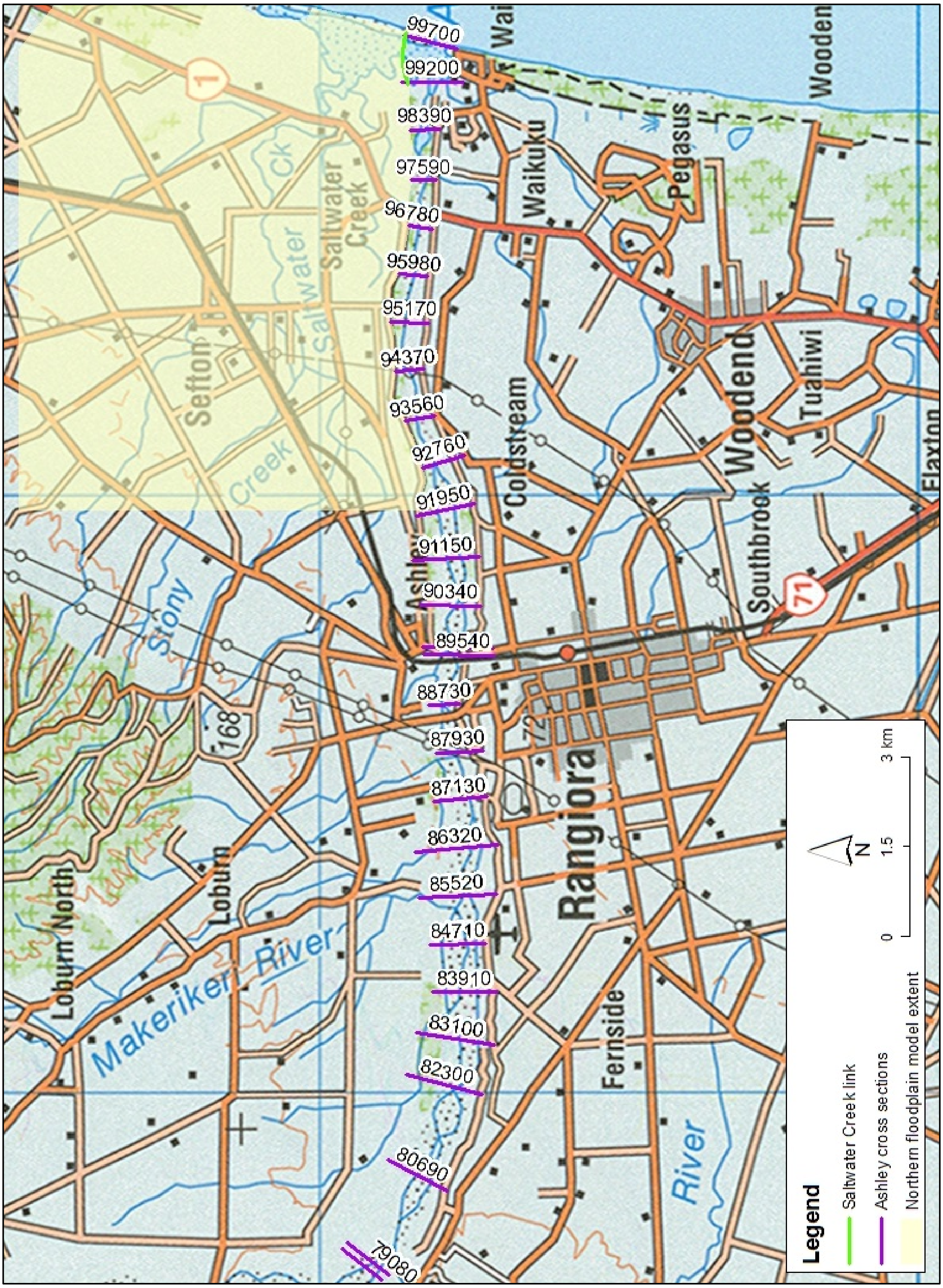
Table E-1: Summary of flood probability

Event	Probability of occurring in period		
ARI (% AEP[1])	**10 yr period**	**30 yr period**	**70 yr period**
20 year ARI (5%)	40%	80%	97%
50 year ARI (2%)	20%	50%	77%
100 year ARI (1%)	10%	25%	50%
200 year ARI (0.5%)	5%	15%	33%
500 year ARI (0.2%)	2%	6%	12%

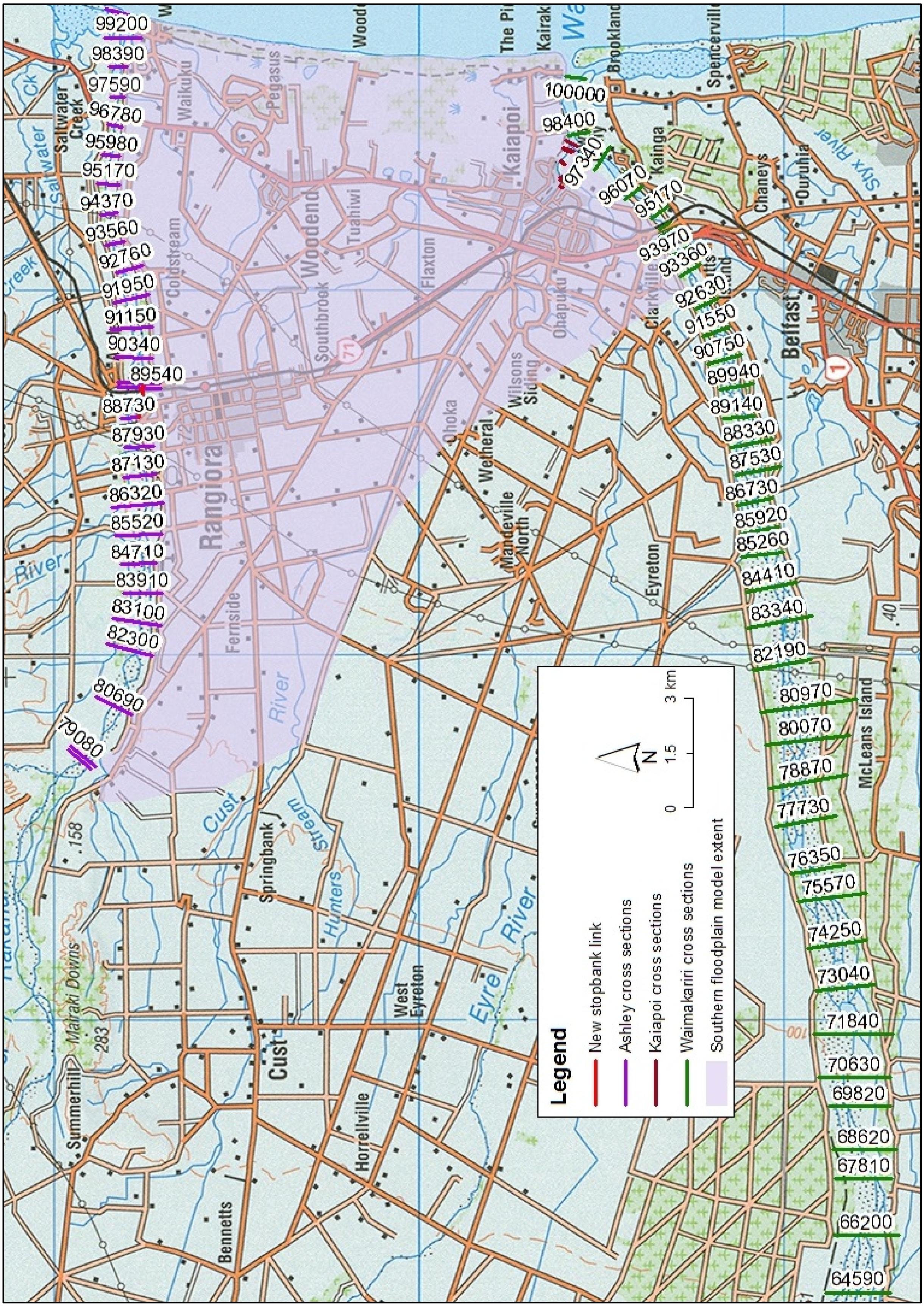
[1]AEP = Annual Exceedance Probability i.e. the chance of a flood that size occurring in any one year

For example, there is a 25% chance that a 100 year ARI (1% AEP) flood will occur within a 30 year time period.

Appendix F: Detailed model schematics



Ashley River - northern floodplain model schematic



Ashley River - southern floodplain model schematic

Ashley River Mike11 cross section information

Mike11 Chainage	Cross section reference	Location/description
78900	22.10	Upstream limit of model, downstream of the Okuku River confluence
79080	20.92	
80690	19.31	
82300	17.70	
83100	16.90	
83910	16.09	
84710	15.29	
85520	14.48	
86320	13.68	
87130	12.87	
87930	12.07	
88490	-	Upstream of road bridge
88498	-	Road bridge
88502	-	Road bridge
88510	-	Downstream of road bridge
88730	11.27	Downstream of the Cones Road bridge
89540	10.46	Downstream of railway bridge
89660	-	Downstream limit of where breakout flow returns to Ashley River at new stopbank
90340	9.66	
91150	8.85	
91950	8.05	
92760	7.24	
93560	6.44	
94370	5.63	
95170	4.83	
95980	4.02	
96780	3.22	
97040	2.96	SH1 bridge
97050	2.96	Downstream SH1 bridge
97590	2.41	
98390	1.61	
99200	0.80	Upstream of Saltwater lagoon area (downstream limit of stopbanks)
99700	0.30	Downstream boundary at outlet to sea

Kaiapoi River Mike11 cross section information

Mike11 Chainage	Cross section reference	Survey date	Location/description
-930	CS1.40km 2011	2014 LiDAR + bed level from Feb 2011	Upstream limit of 1-d model at connection with grid (near Jollie
-925	CS1.40km 2011	2014 LiDAR + bed level from Feb 2011	
-770	-	2014 LiDAR + estimated bed level	
-765	-	2014 LiDAR + estimated bed level	Upstream of Courtenay Stream
-665	CS1.135km 2011	2014 LiDAR + bed level from Feb 2011	Downstream of Courtenay Stream
-150	-	2014 LiDAR + estimated bed level	
-125	-	2014 LiDAR + estimated bed level	Downstream of Feldwicks Drain
225	-	2014 LiDAR + estimated bed level	
265	-	2014 LiDAR + estimated bed level	Downstream of McIntoshs Outfall
500	-	2014 LiDAR + estimated bed level	

Waimakariri River Mike11 cross section information

Mike11 Chainage	Cross section reference	Location/description
64590	35.41	Top of model near Intake Road
66200	33.80	
67810	32.19	
68620	31.38	
69820	30.18	
70630	29.37	
71840	28.16	
73040	26.96	
74250	25.75	
75570	24.43	
76350	23.65	
77730	22.27	
78870	21.13	
80070	19.93	
80970	19.03	
82190	17.81	
83340	16.66	
84410	15.59	
85260	14.74	
85920	14.08	
86730	13.27	
87530	12.47	
88330	11.67	
89140	10.86	
89940	10.06	
90750	9.25	
91550	8.45	
92630	7.64	
93360	6.64	
93970	6.03	
94170	Interpolated	South Branch confluence upstream of SH1 bridge
94370	5.63	Downstream of SH1 Bridge
94570	5.43	
94770	5.23	Old bridge
95170	4.83	
96070	3.93	
96640	3.36	
97340	2.66	
98270	Interpolated	Upstream opf the Kaiapoi River confluence
98400	1.60	Downstream of Kaiapoi River confluence
98820	1.18	
99470	0.53	
99990	Interpolated	Upstream of Kairaki Creek
100000	0.00	Downstream model boundary (downstream of Kairaki Creek and upstream of Brooklands Lagoon)

Appendix G: Model run files

Southern floodplain (100 year ARI)

10 m 2014 WDC grid extent: Lower left corner = 1555947.5 E, 5192709.799 N
Upper right corner = 1577967.5 E, 5208829.799 N

Breakout A	Breakout B
500 m^3/s breakout at A	500 m^3/s breakout at B

MikeFlood		
Couple file (*.mf)	**STH_Ashley_100A**	**STH_Ashley_100B**

Mike11		
Simulation file (*.sim11)	**STH_Ashley_100A**	**STH_Ashley_100B**
Network file (*.nwk11)	STH_Waimak&KaiapoiAsh_NZTM_080616	
Cross section file (*.xns11)	Waimak_Kaiapoi_Ash_Brs_LiDAR_STH_120716	
Boundary file (*.bnd11)	STH_Ashley_100_A	STH_Ashley_100_B
HD parameter (*.hd11)	Sth_Waimak&AshleyRivers_080616	
Results file (*.res11)	STH_Ashley_100A	STH_Ashley_100B

Mike21		
Simulation file (*.21)	**STH_Ashley_100A**	**STH_Ashley_100B**
Bathymetry file (*.dfs2)	s_dem2014_10m_culverts_150716	
Initial surface elevation	-0.5	
Resistance (*.dfs2)	s_rgh2014_10m_June16	
Results (*.dfs2)	STH_Ashley_100A	STH_Ashley_100B
Sources	(106,1588)→(115,1588)	(994,1480)→(1003,1480)
Sinks	-	
Drying depth (m)	0.01	
Wetting depth (m)	0.02	
Eddy viscosity	1	
Number of structures	38 culverts and 10 weirs	
Simulation start time	23/12/1993 at 6:00 am	
Simulation end time	24/12/1993 at 11:00 pm	
Time step (s)	1	
Length of run (# time steps)	147600	

Southern floodplain (100 year ARI – sensitivity tests)

10 m 2014 WDC grid extent: Lower left corner = 1555947.5 E, 5192709.799 N
Upper right corner = 1577967.5 E, 5208829.799 N

Breakout A with NIWA floodplain roughness	Breakout B with n=0.08 (M=12.5) near breakout
500 m^3/s breakout at location A. Floodplain roughness used in Oliver (2008), which is derived from LiDAR data, replaces WDC (2015) roughness grid	500 m^3/s breakout at location B. Floodplain roughness between existing and new stopbanks in vicinity of Breakout B changed from n=0.05 to n=0.08

MikeFlood		
Couple file (*.mf)	**STH_Ashley_100A_NIWA_n**	**STH_Ashley_100B_n_08_sec_bank**

Mike11		
Simulation file (*.sim11)	**STH_Ashley_100A_NIWA_n**	**STH_Ashley_100B_n_08_sec_bank**
Network file (*.nwk11)	STH_Waimak&KaiapoiAsh_NZTM_080616	
Cross section file (*.xns11)	Waimak_Kaiapoi_Ash_Brs_LiDAR_STH_120716	
Boundary file (*.bnd11)	STH_Ashley_100_A	STH_Ashley_100_B
HD parameter (*.hd11)	Sth_Waimak&AshleyRivers_080616	
Results file (*.res11)	STH_Ashley_100A_NIWA_n	STH_Ashley_100B_n_08_sec_bank

Mike21		
Simulation file (*.21)	**STH_Ashley_100A_NIWA_n**	**STH_Ashley_100B_n_08_sec_bank**
Bathymetry file (*.dfs2)	s_dem2014_10m_culverts_170616_NIWA_n_run	s_dem2014_10m_culverts_150716
Initial surface elevation	-0.5	
Resistance (*.dfs2)	rgh08_10msthcpd_NIWA_n_run	s_rgh2014_10m_June16_n_0_08_sec_bank
Results (*.dfs2)	STH_Ashley_100A_NIWA_n	STH_Ashley_100B_n_08_sec_bank
Sources	(106,1588)→(115,1588)	(994,1480)→(1003,1480)
Sinks	-	
Drying depth (m)	0.01	
Wetting depth (m)	0.02	
Eddy viscosity	1	
Number of structures	38 culverts and 10 weirs	
Simulation start time	23/12/1993 at 6:00 am	
Simulation end time	24/12/1993 at 11:00 pm	
Time step (s)	1	
Length of run (# time steps)	147600	

Southern floodplain (100 year ARI – sensitivity test)

10 m 2014 WDC grid extent:	Lower left corner =	1555947.5 E, 5192709.799 N
	Upper right corner =	1577967.5 E, 5208829.799 N

Breakout B with gaps for roads remaining in Ashley River new stopbank
500 m^3/s breakout at location B. Existing gaps in new stopbank, where roads haven't been raised.

MikeFlood	
Couple file (*.mf)	**STH_Ashley_100B_rd_gaps**

Mike11	
Simulation file (*.sim11)	**STH_Ashley_100B_rd_gaps**
Network file (*.nwk11)	STH_Waimak&KaiapoiAsh_NZTM_080616
Cross section file (*.xns11)	Waimak_Kaiapoi_Ash_Brs_LiDAR_STH_120716
Boundary file (*.bnd11)	STH_Ashley_100_B
HD parameter (*.hd11)	Sth_Waimak&AshleyRivers_080616
Results file (*.res11)	STH_Ashley_100B_rd_gaps

Mike21	
Simulation file (*.21)	**STH_Ashley_100B_rd_gaps**
Bathymetry file (*.dfs2)	s_dem2014_10m_culverts_150716_rd_gaps
Initial surface elevation	-0.5
Resistance (*.dfs2)	s_rgh2014_10m_June16
Results (*.dfs2)	STH_Ashley_100B_rd_gaps
Sources	(994,1480)→(1003,1480)
Sinks	-
Drying depth (m)	0.01
Wetting depth (m)	0.02
Eddy viscosity	1
Number of structures	38 culverts and 10 weirs
Simulation start time	23/12/1993 at 6:00 am
Simulation end time	24/12/1993 at 11:00 pm
Time step (s)	1
Length of run (# time steps)	147600

Southern floodplain (200 year ARI)

10 m 2014 WDC grid extent: Lower left corner = 1555947.5 E, 5192709.799 N
Upper right corner = 1577967.5 E, 5208829.799 N

Breakout A & B	Breakout A & C
750 m^3/s breakout at A & 400 m^3/s at B	750 m^3/s breakout at A & 200 m^3/s at C

MikeFlood		
Couple file (*.mf)	**STH_Ashley_200AB**	**STH_Ashley_200AC**

Mike11		
Simulation file (*.sim11)	**STH_Ashley_200AB**	**STH_Ashley_200AC**
Network file (*.nwk11)	STH_Waimak&KaiapoiAsh_NZTM_080616	
Cross section file (*.xns11)	Waimak_Kaiapoi_Ash_Brs_LiDAR_STH_120716	
Boundary file (*.bnd11)	STH_Ashley_200_AB	STH_Ashley_200_AC
HD parameter (*.hd11)	Sth_Waimak&AshleyRivers_080616	
Results file (*.res11)	STH_Ashley_200AB	STH_Ashley_200AC

Mike21		
Simulation file (*.21)	**STH_Ashley_200AB**	**STH_Ashley_200AC**
Bathymetry file (*.dfs2)	s_dem2014_10m_culverts_150716	
Initial surface elevation	-0.5	
Resistance (*.dfs2)	s_rgh2014_10m_June16	
Results (*.dfs2)	STH_Ashley_200AB	STH_Ashley_200AC
Sources	(106,1588)→(115,1588) & (994,1480)→(1003,1480)	(106,1588)→(115,1588) & (1447,1489)→(1447,1480)
Sinks	-	
Drying depth (m)	0.01	
Wetting depth (m)	0.02	
Eddy viscosity	1	
Number of structures	38 culverts and 10 weirs	
Simulation start time	23/12/1993 at 6:00 am	
Simulation end time	24/12/1993 at 11:00 pm	
Time step (s)	1	
Length of run (# time steps)	147600	

Southern floodplain (200 year ARI – sensitivity tests)

10 m 2014 WDC grid extent:	Lower left corner =	1555947.5 E, 5192709.799 N
	Upper right corner =	1577967.5 E, 5208829.799 N

Breakout A & B flows both increased by 20%	Breakout A & B with sea level increased by 0.8m
900 m^3/s breakout at A & 480 m^3/s at B.	750 m^3/s at A, 400 m^3/s at B, Ashley sea level is 2.5 m and Waimakariri sea level is a tidal cycle with a peak of 2.5 m.

MikeFlood		
Couple file (*.mf)	**STH_Ashley_200AB_q_plus_20perc**	**STH_Ashley_200AB_tide_plus_0_8m**

Mike11		
Simulation file (*.sim11)	**STH_Ashley_200AB_q_plus_20perc**	**STH_Ashley_200AB_tide_plus_0_8m**
Network file (*.nwk11)	STH_Waimak&KaiapoiAsh_NZTM_080616	
Cross section file (*.xns11)	Waimak_Kaiapoi_Ash_Brs_LiDAR_STH_120716	
Boundary file (*.bnd11)	STH_Ashley_200_AB_080616	STH_Ashley_200_AB_tide_plus_0_8m
HD parameter (*.hd11)	Sth_Waimak&AshleyRivers_080616	
Results file (*.res11)	STH_Ashley_200AB_q_plus_20_perc	STH_Ashley_200AB_tide_plus_0_8m

Mike21		
Simulation file (*.21)	**STH_Ashley_200AB_q_plus_20_perc**	**STH_Ashley_200AB_tide_plus_0_8m**
Bathymetry file (*.dfs2)	s_dem2014_10m_culverts_150716	
Initial surface elevation	-0.5	
Resistance (*.dfs2)	s_rgh2014_10m_June16	
Results (*.dfs2)	STH_Ashley_200AB_q_plus_20_perc	STH_Ashley_200AB_tide_plus_0_8m
Sources	(106,1588)→(115,1588) & (994,1480)→(1003,1480)	
Sinks	-	
Drying depth (m)	0.01	
Wetting depth (m)	0.02	
Eddy viscosity	1	
Number of structures	38 culverts and 10 weirs	
Simulation start time	23/12/1993 at 6:00 am	
Simulation end time	24/12/1993 at 11:00 pm	
Time step (s)	1	
Length of run (# time steps)	147600	

Southern floodplain (500 year ARI)

10 m 2014 WDC grid extent: Lower left corner = 1555947.5 E, 5192709.799 N
Upper right corner = 1577967.5 E, 5208829.799 N

Breakout A, B & C
1750 m^3/s breakout at A, 350 m^3/s breakout at B & 160 m^3/s at C

MikeFlood	
Couple file (*.mf)	**STH_Ashley_500ABC_overflows**

Mike11	
Simulation file (*.sim11)	**STH_Ashley_500ABC_overflows**
Network file (*.nwk11)	STH_Waimak&KaiapoiAsh_NZTM_080616
Cross section file (*.xns11)	Waimak_Kaiapoi_Ash_Brs_LiDAR_STH_120716
Boundary file (*.bnd11)	STH_Ashley_500_ABC
HD parameter (*.hd11)	Sth_Waimak&AshleyRivers_080616
Results file (*.res11)	STH_Ashley_500ABC_overflows

Mike21	
Simulation file (*.21)	**STH_Ashley_500ABC_overflows**
Bathymetry file (*.dfs2)	s_dem2014_10m_culverts_150716
Initial surface elevation	-0.5
Resistance (*.dfs2)	s_rgh2014_10m_June16
Results (*.dfs2)	STH_Ashley_500ABC_overflows
Sources	(106,1588)→(115,1588), (994,1480)→(1003,1480) & (1447,1489)→(1447,1480)
Sinks	-
Drying depth (m)	0.01
Wetting depth (m)	0.02
Eddy viscosity	1
Number of structures	38 culverts and 10 weirs
Simulation start time	23/12/1993 at 6:00 am
Simulation end time	24/12/1993 at 11:00 pm
Time step (s)	1
Length of run (# time steps)	147600

Northern floodplain (Breakout E/F)

10 m 2014 WDC grid extent: Lower left corner = 1569737.5 E, 5208219.799 N
Upper right corner = 1578927.5 E, 5214989.799 N

100 year ARI	200 year ARI	500 year ARI
125 m^3/s breakout at E/F	200 m^3/s breakout at E/F	250 m^3/s breakout at E/F

MikeFlood			
Couple file (*.mf)	**NTH_Ashley_100EF**	**NTH_Ashley_200EF**	**NTH_Ashley_500EF**

Mike11			
Simulation file (*.sim11)	**NTH_Ashley_100EF**	**NTH_Ashley_200EF**	**NTH_Ashley_500EF**
Network file (*.nwk11)	NTH_Ashley_only_NZTM_310516		
Cross section file (*.xns11)	Waimak_Kaiapoi_Ash_Brs_LiDAR_STH_120716		
Boundary file (*.bnd11)	AshleyNTHFP_100	AshleyNTHFP_200	AshleyNTHFP_500
HD parameter (*.hd11)	AshleyRiver		
Results file (*.res11)	NTH_Ashley_100EF	NTH_Ashley_200EF	NTH_Ashley_500EF

Mike21			
Simulation file (*.21)	**NTH_Ashley_100EF**	**NTH_Ashley_200EF**	**NTH_Ashley_500EF**
Bathymetry file (*.dfs2)	n_dem2014_10m_June16 (WDC grid with crops removed)		
Initial surface elevation	-0.5		
Resistance (*.dfs2)	n_rgh2014_10m_June16 (WDC grid)		
Results (*.dfs2)	NTH_Ashley_100EF	NTH_Ashley_200EF	NTH_Ashley_500EF
Sources	(33,6)→(33,15)		
Sinks	-		
Drying depth (m)	0.01		
Wetting depth (m)	0.02		
Eddy viscosity	1		
Number of structures	0		
Simulation start time	23/12/1993 at 6:00 am		
Simulation end time	24/12/1993 at 11:00 pm		
Time step (s)	2		
Length of run (# time steps)	73800		

Northern floodplain (200 year ARI – sensitivity tests)

10 m 2014 WDC grid extent:	Lower left corner =	1569737.5 E, 5208219.799 N
	Upper right corner =	1578927.5 E, 5214989.799 N

	Tidal cycle instead of constant water level	Constant tide level increased by 0.8m
	200 m^3/s breakout at E/F, tidal cycle for sea boundary (max level = 1.7 m above msl)	200 m^3/s breakout at E/F, Ashley River downstream boundary = 2.5 m above msl
MikeFlood		
Couple file (*.mf)	**NTH_Ashley_200EF_tide_cycle**	**NTH_Ashley_200EF_tide_plus_0_8m**
Mike11		
Simulation file (*.sim11)	**NTH_Ashley_200EF_tide_cycle**	**NTH_Ashley_200EF_tide_plus_0_8m**
Network file (*.nwk11)	NTH_Ashley_only_NZTM_310516	
Cross section file (*.xns11)	Waimak_Kaiapoi_Ash_Brs_LiDAR_STH_120716	
Boundary file (*.bnd11)	AshleyNTHFP_200_tide_cycle	AshleyNTHFP_200_tide_plus_0_8m
HD parameter (*.hd11)	AshleyRiver	
Results file (*.res11)	NTH_Ashley_200EF_tide_cycle	NTH_Ashley_200EF_tide_plus_0_8m
Mike21		
Simulation file (*.21)	**NTH_Ashley_200EF_tide_cycle**	**NTH_Ashley_200EF_tide_plus_0_8m**
Bathymetry file (*.dfs2)	n_dem2014_10m_June16	
Initial surface elevation	-0.5	
Resistance (*.dfs2)	n_rgh2014_10m_June16	
Results (*.dfs2)	NTH_Ashley_200EF_tide_cycle	NTH_Ashley_200EF_tide_plus_0_8m
Sources	(33,6)→(33,15)	
Sinks	-	
Drying depth (m)	0.01	
Wetting depth (m)	0.02	
Eddy viscosity	1	
Number of structures	0	
Simulation start time	23/12/1993 at 6:00 am	
Simulation end time	24/12/1993 at 11:00 pm	
Time step (s)	2	
Length of run (# time steps)	73800	

Environment Canterbury Regional Council
Kaunihera Taiao ki Waitaha

Facilitating sustainable development in the Canterbury region

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