Waimakariri River design standard and risk assessment (2020)

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Kaunihera Taiao ki Waitaha

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Summary

Background

The Waimakariri River flood hazard has a long history of management, dating back to the Provincial Government, followed by the South Waimakariri River Board, the Waimakariri River Trust, the North Canterbury Catchment Board, and, more latterly, the Canterbury Regional Council.

The structural protection initiatives have been progressively upgraded and strengthened, and over the past 20 years non-structural protection initiatives have been 'added to the mix'.

A risk assessment exercise was carried out during the early 2000s, to demonstrate the benefit of what was then seen as a large capital injection into structural mitigation.

Because of Rules in the Territorial Authorities' Plans (Christchurch City, Selwyn and Waimakariri Districts'), as part of the non-structural measures, this risk assessment has been re-visited, to include consideration of the relatively new structural enhancements to the primary protection system constructed as part of the Waimakariri Flood Protection Project (WFPP). These include:

- The new filter structure immediately downstream of the Old Highway Bridge on the northern side.
- Rock work:
    - -where the Otakaikino joins the main stem;
    - in the vicinity of the bridges;
    - on the north side downstream of the Eyre Diversion;
    - on the south side in the vicinity of Englebrechts.
- The overlapping bank work at Halkett.
- Enhanced mowing techniques, resulting in much improved grass cover.
- The 5 metre clearance between the riverside toe of the stopbanks and the commencement of the protection plantings.
- Enhanced targeted shingle extraction management.

Damage from flooding outside the secondary system, on the south side, is considered to be so unlikely, that, for the purposes of this assessment, it has been discounted entirely.

The design standard for the Waimakariri River primary protection system was last formally set as part of the Waimakariri River Improvement Scheme (1960) investigations.

Sixty years on, following the implementation of that scheme, and also the implementation of what effectively was the review of that scheme, the Waimakariri River Floodplain Management Plan/Strategy (1991), a re-visit is deemed appropriate.

The problem

Both the understanding of 'residual risk' and the design standards of the Waimakariri River protection system were dated, and an understanding of both were essential for on-going floodplain management.

As noted in the report,, the current design standard has been unchanged for over 50 years. Apart from the longer hydrology record, the protection system has been substantially upgraded through the Waimakariri River Improvement Scheme (1960), and, more latterly, the WFPP. The standard is an integral part of various risk assessments that have been carried out which assist with non-structural initiatives that have been introduced, such as land use controls. Professional Canterbury Regional Council staff from both the River Engineering and the Environmental Science & Hazards Sections agreed that a re-visit was both necessary and timely.

What we did

The sedimentation, hydrology, and hydraulics, were all updated to reflect the position as at 2020, and the risk assessment analyses were re-visited, taking into consideration these updates, as well as the substantial structural upgrades set out above (and implemented through the WFPP).

From the sea to the Lower Gorge, we analysed the sedimentation and updated the hydraulics, utilising the latest survey data.

What we found

Council is in ‘a very good space’ in regard to the Waimakariri Flood Hazard. Territorial Authority, and private, assets, are protected to an eminently satisfactory level (in fact the highest level of protection in New Zealand) because of the number of people, and the assets, potentially at risk.

The primary system has a 5,000 m^3/s capacity with 1 m freeboard which, without climate change adjustments represents approximately a 300 year ARI standard (using the hydrology assessment described herein). There is an assessed 1% chance of failure, in any one year, taking into account potential failures below the design standard. The secondary system, which was constructed due to the potential consequences of flooding beyond the primary system, provides protection for development and assets to a very much higher standard again (so high that it is difficult to put a number on it, because of its probable ARI value in relation to the length of record).

The gravel extraction operations have been very successful, and a new design standard is appropriate.

What this means

This means that the Christchurch City, Selwyn, and Waimakariri District Councils’ are better informed when reviewing their Plans.

Staff can make recommendations regarding a new design standard to the Waimakariri-Eyre-Cust Rating Area Liaison Committee, and to Council.

Climate Change

This project is part of a ‘three-pronged’ investigation exercise, the other two parts being:

- A hydrology re-visit, where annual recurrence intervals (ARIs) have been assigned to flows.
- Climate change modelling scenarios, where projected rainfalls are applied to flood-forecasting models.

Together, the three pieces of work enable climate change to be factored into land-use decision making.

The hydrology re-visit has been completed, however, the climate change modelling has not been possible, because the Waimakariri food-forecasting model has been decommissioned (obsolete software). A new model is being constructed.

Climate change modelling has, however, been carried out for the Ashley River/Rakahuri, immediately to the north. The results suggest that large magnitude flood flows could increase by 25-30% based on Representative Concentration Pathway 8.5 (RCP 8.5) rainfall for 2081 – 2100.

Because the Ashley River is a foothills river, compared to the alpine Waimakariri River, the results cannot be directly transferred. However, it does give interim (order of magnitude) values, pending further work.

This investigation sits to the side of climate change considerations, which are incorporated into the several and various detailed investigations that have been, and are, being carried out.

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1 Introduction

Numerous, and extensive, investigations have been carried out in relation to the Waimakariri River Floodplain Management Strategy.This report summarises a flood hazard risk assessment, as at August 2020.

2 Floodplain Management Strategies

Flood hazard is the interaction between the powerful natural processes of rainfall, run-off, and resultant floodwaters, with human settlement.

A Floodplain Management Strategy is an action plan. It consists of a number of treatments to minimise the impacts of flooding, and deposition of sediment, by reducing the risk to a determined level.

The object is to quantify the hazard so that decisions can sensibly be taken with regard to suitable responses. The responses must be effective both in terms of cost, and location, as well as integrating with social and environmental considerations.

To be effective, a Floodplain Management Strategy must have the support of the potentially adversely affected community. It also must be based on a sound appreciation of the relationship between the physical and social aspects of the flood hazard.

The above philosophies have been developed, and applied, extensively across the region.

3 History of flooding and protection initiatives

Sections 3.1 to 3.4 have been extracted from the Draft Waimakariri Floodplain Management Plan (Griffiths, 1991), to provide background, and context.

3.1 Provincial Government

"Early settlers of Christchurch were apparently unaware of the threat of flooding from the Waimakariri. But it was not long before the river manifested its presence. A number of heavy floods occurred, particularly on what was then Kaiapoi Island, and, following a flood in 1859, the Lyttelton Times commented editorially 'it would appear from unmistakable indications that the Waimakariri River, for about the first 20 miles of its course upstream, has not been in the habit of long sticking to one bed; in colonising the country we must civilise the river.'

The earliest attempts to contain the river were protection works built near Halkett, by the Provincial Government, between 1859 and 1868. In the end, they proved inadequate, because, in 1868, floodwaters from the Waimakariri poured down overflow channels into the Avon and Styx basins. Much of what is now Fendalton, Merivale, and the north of the central city area, were flooded, as well as land along the length of the Avon River. The river also broke out to the north, and parts of the Kaiapoi settlement on Kaiapoi Island were flooded to depths of nearly 2 m. Upon the petition of the people of Kaiapoi, who were unhappy with the efforts of the Provincial Government[1], the South Waimakariri River District was eventually set up by the Provincial Government in 1869.

3.2 South Waimakariri River Board

Operating through a Board of Conservators, this Board constructed a series of groynes, designed to halt any overflows from the Waimakariri into the Styx and Avon Rivers. Ratepayers were not satisfied with an appointed body, and, in 1880, the South Waimakariri River Board was elected, and continued

[1] *Others were also dissatisfied even before the 1868 flood as shown, for example, by the following quote from the Lyttelton Times of 1868: 'Engineers were riding up to Courtenay and contemplating the waters with no more influence than King Canute. If the river came down the Avon in force, Christchurch would very soon be a wild waste of shingle. I think, Sir, it behoves us all to look out in time.'*

the work of the earlier body. As a result of pressure from people on the north side of the river, who were concerned that protection works on the south were making their land more vulnerable to flooding, the Waimakariri River Improvement Act was passed, which set up the Waimakariri River Trust, giving it the responsibility of dealing with both the north and south banks of the river, from the mouth to nearly the Lower Gorge.

3.3 Waimakariri River Trust

The major work of the Trust was to design, and construct, the first comprehensive river training scheme in the lower reaches of the Waimakariri. The quaintly named Hays No. 2 Scheme provided a philosophy and methods for controlling the river that were to last 60 years. A course for the future, conceived with care and precision, but difficult to modify, or reverse, was set both in the mind, and on the ground.

Prior to the construction of Hays No. 2 Scheme, 1928, the lower Waimakariri was a complex of interlacing channels and islands. The aim of this Scheme was to prevent flood overflows, and constrict and shorten the channel, to enable the river to pass its gravel load to the sea. The design discharge was 4250 m^3/s the 'assumed maximum' (Nelson, 1928). In addition to the construction of an extensive stopbank and groyne system, a tight loop in the river was bypassed by the excavation of Wrights Cut, and the Old South Branch was closed by the Crossbank. There was also a new Highway Bridge, a stabilised river mouth, and a lower diversion between Stewarts Gully and the estuary, amongst other works. Broadly speaking, in spite of progressive aggradation, the Scheme was successful, for it kept overflows out of Christchurch and Kaiapoi, but, over the years, it proved less capable of confining major floods (Reid and Dick 1960). During the life of the Trust, the Scheme was first tested by a flood peaking at 2660 m^3/s in 1936; the protection works held, but only just. However, in 1940, a flood of 2740 m^3/s broke through the Crossbank and the stopbank near Whites Bridge. Water also came very close to topping the stopbank at Halkett.

The Waimakariri River Trust, having constructed Hays No. 2 Scheme, continued with maintenance work until it was replaced by the North Canterbury Catchment Board in 1946.

3.4 North Canterbury Catchment Board

The first major flood faced by the Board occurred in 1950; the peak flow was 2850 m^3/s. There were serious breaches on both sides of Wrights Cut, the Crossbank was breached in at least two places, and Coutts Island, Stewarts Gully, Kaiapoi, and parts of Kairaki, were flooded. This event was followed only seven years later, on 27 December 1957, by the largest flood recorded in the Waimakariri since 1930, and very probably since circa 1868. Over 120 m of the inner stopbank of Englbrechts had been eroded away in a flood on 16 December 1957, but a second loop bank prevented breaching of the protection system. Efforts to rebuild the bank were unable to contain the record flood of 27 December, which peaked at 3990 m^3/s, and breached here, as well as at Chaneys stopbank, on the true right of the South Branch. Water over 1.5 m deep flowed through Kainga, and the area from Englebrechts down to about the Belfast Hotel, and downstream to nearly Brooklands, was described by the Christchurch Star-Sun newspaper as 'one vast lake'.

The 1950 and 1957 floods, together with the 1940 event, demonstrated, unequivocally, that Hays No. 2 Scheme no longer met its objectives, mainly owing to the effects of gravel deposition in the river channel. Accordingly, a review was undertaken, which resulted in the adoption of the Waimakariri River Improvement Scheme 1960 by the Board (Henderson 1960: Reid and Dick 1960). The immediate object of this Scheme was to pass, without overflow, a design flood of 4730 m^3/s; and the longer term object was to deal with the problem of aggradation of the lower reaches, as far as appeared practical and necessary at that time (1960). Work on the scheme began in 1963, and it was completed in 1986. Dwyer and Poynter (1976), and Reid and Poynter (1982), gave progress reports. Between 1960 and 1989, all floods were contained within the protection works, including large events in 1970 (2510 m^3/s), 1979 (2910 m^3/s), and 1984 (2830 m^3/s). As far as safe conveyance of floodwaters is concerned, the Scheme was a complete success. In regard to gravel or shingle transport, the tongue of gravel that had advanced downstream to near Whites Bridge in 1927, and to about Stewarts Gully Sailing Club by 1960, remained more or less stationary, so no shingle passed to the sea, as envisaged by both Hays No. 2 Scheme, and the 1960 Scheme. However, the fallback position of bulk removal of gravel 'saved the day' because commercial extraction, within the reach below Crossbank approximately balanced the nett influx past that point reducing, but not entirely preventing, aggradation. In this respect also, the Scheme has been *successful, although the bed has built up near Crossbank. During 1989, the North Canterbury Catchment Board was abolished, as part of Local Government reforms, and its duties and responsibilities assumed by the Canterbury Regional Council."*

Figure 3-1 maps the lower Waimakariri River prior to the construction of Hays No. 2 Scheme 1928. Figure 3-2 shows river courses and protection works for the lower Waimakariri River prior to the Waimakariri River Improvement Scheme 1960 and Figure 3-3 to Figure 3-5 show Waimakariri river flooding from 27 May 1950.

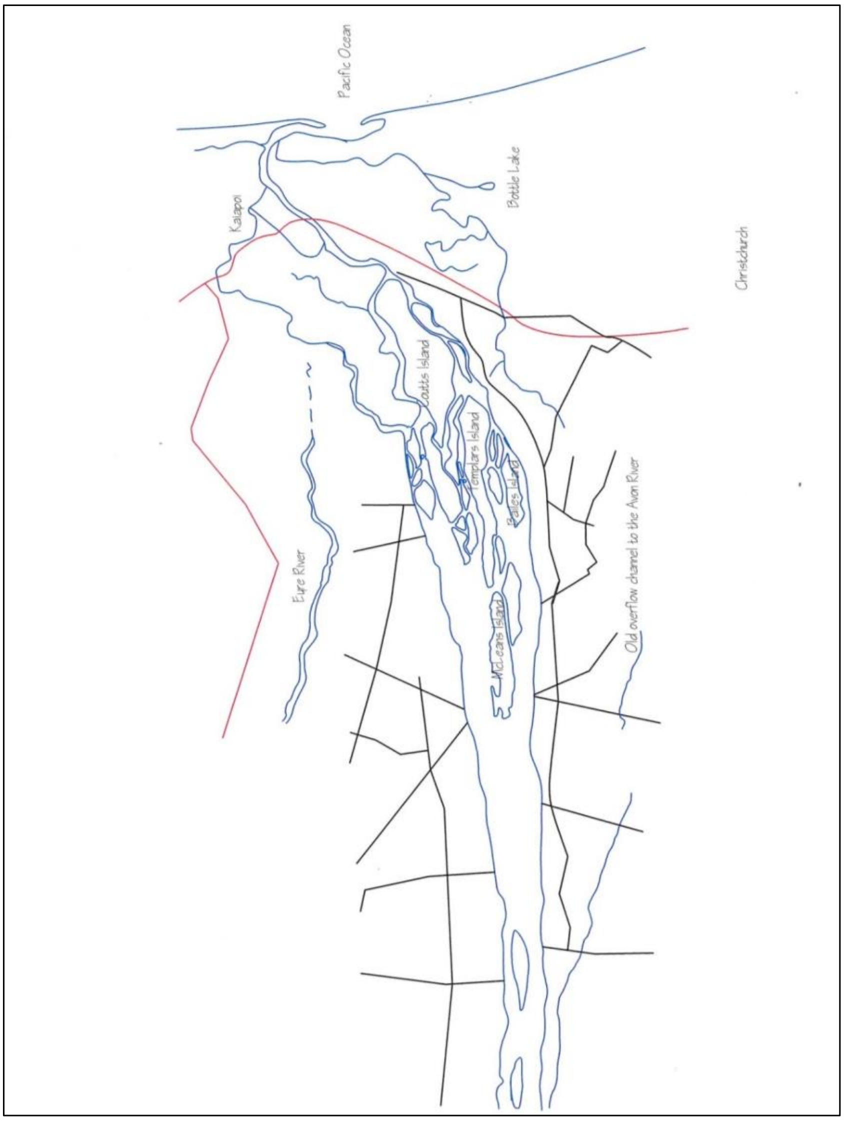


Figure 3-1: Lower Waimakariri River prior to Hays No. 2 Scheme 1928

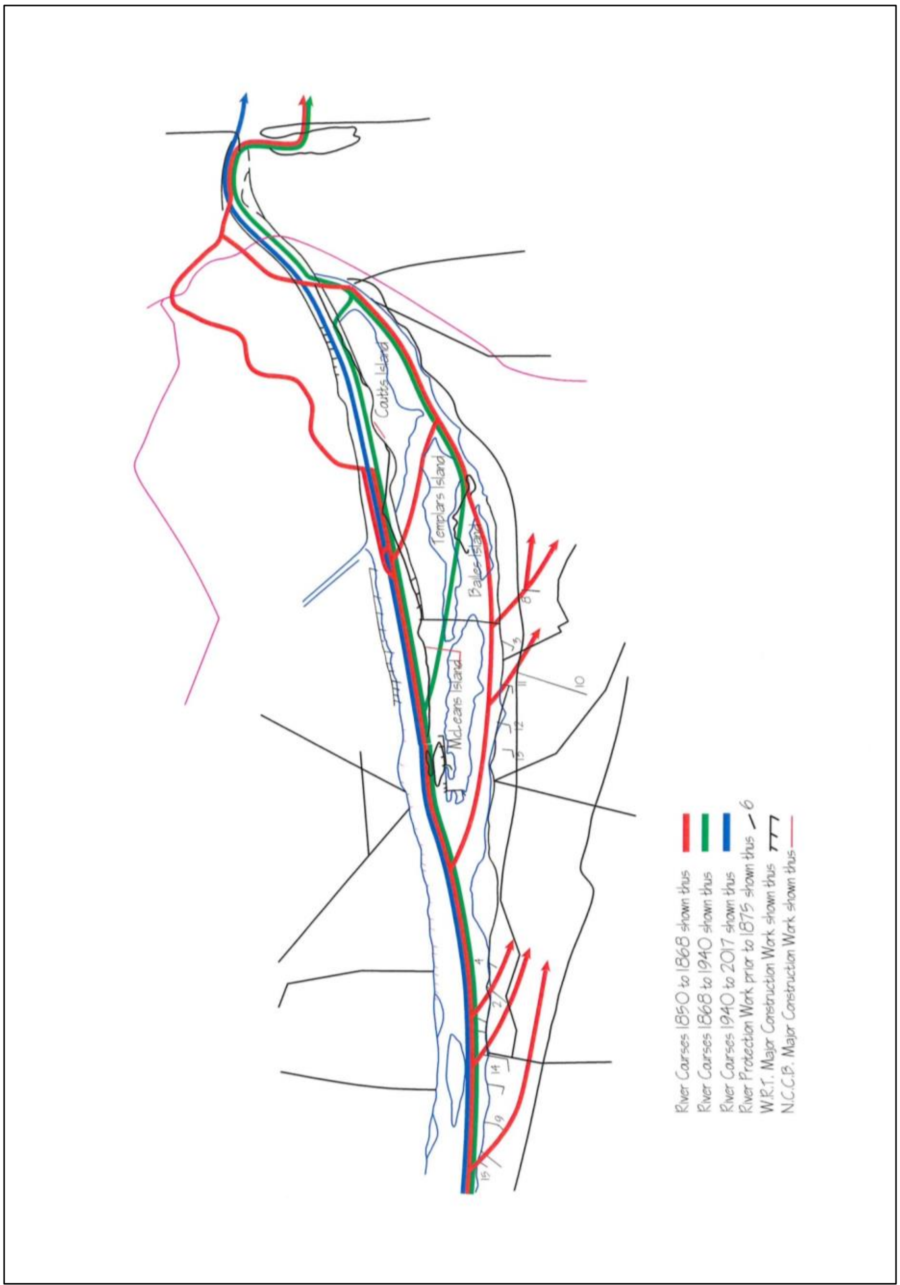


Figure 3-2: Lower Waimakariri River showing river courses and protection works prior to the Waimakariri River Improvement Scheme 1960

Figure 3-3: Lower Waimakariri River – 27 May 1950

Figure 3-4: Wrights Cut breach – 27 May 1950

Figure 3-5: Waimakariri District – 27 May 1950

3.5 Canterbury Regional Council (Environment Canterbury/ECan)

During the 1990s, the Canterbury Regional Council prepared the Waimakariri River Floodplain Management Plan, to determine what degree of protection was desirable, and justified, for the future. This re-visit was different from the three previous visits, in that it examined non-structural measures, as well as the structural ones. The plan aimed to minimise potential damage to the Christchurch City, Kaiapoi, and Selwyn District communities, for a thirty year period, and beyond. Five options, incorporating floodplain management measures ranging from dams to relief funds, were assessed for their ability to minimise social disruption and damage to life, assets, and the environment, arising from all conceivable break-outs. Twenty-seven protection measures were selected as practicable and acceptable. They were used in different combinations to form five options for the Plan. Each option was evaluated using physical, economic, social, and environmental criteria. A Community Advisory Committee unanimously agreed on a preferred option (Option 3) which has 18 measures involving river control, land-use management, community preparedness, emergency actions, and Civil Defence.

In October 1996, the Plan was withdrawn by Environment Canterbury, for reasons based on submissions on the Plan, which included:

- Lack of support by the Christchurch City Council and Waimakariri District Councils. In particular, for proposed rules in the Plan controlling land-use for building purposes.
- Public perspective that the proposed flood protection measures were too restrictive.
- Imposition of land-use controls on rural ratepayers (within the floodplain), but not on urban ratepayers, is discriminatory.

The Waimakariri River Rating District Liaison Committee recommended that Council implement the structural package of works, over a 10 year period, and the setting up of a Special Rating District to fund the capital works over a 25 year period. Council accepted those recommendations, and instructed staff to implement the works through the Annual Plan process. These works were estimated at the time to cost around $35M. Council further directed that the land use control initiatives be further pursued through Christchurch City Council, Selwyn District, and Waimakariri District Councils' planning processes. These initiatives have been implemented, with rules in those Plans regarding building set-backs from stopbanks, floor level controls, and avoidance of high hazard areas on the floodplain. A special project was set up to implement the structural package of works - the Waimakariri Flood Protection Project (WFPP).

This project has reduced the risk of flooding in Christchurch City, and in the Waimakariri and Selwyn districts. It adds strength, and resilience, to the flood protection system that was already in place, and significantly lowers the risk of break-outs during flood events.

Construction began in 2009, with a completion date of 2019. Now finished, the secondary stopbank system provides back-up flood protection for Christchurch City and Selwyn District.

Other important works to improve protection for parts of Christchurch City, and the Waimakariri and Selwyn districts, include an upgrade of primary stopbanks and rock work. The new and upgraded stopbanks, and rock work, are shown on Figure 3-6.

The primary system provides a very good level of flood protection, and is designed to protect people in Waimakariri, Selwyn and Christchurch from flooding up to what was then considered to be a 450 year ARI standard (or a 200 to 300 year ARI standard based on current flood frequency analysis).

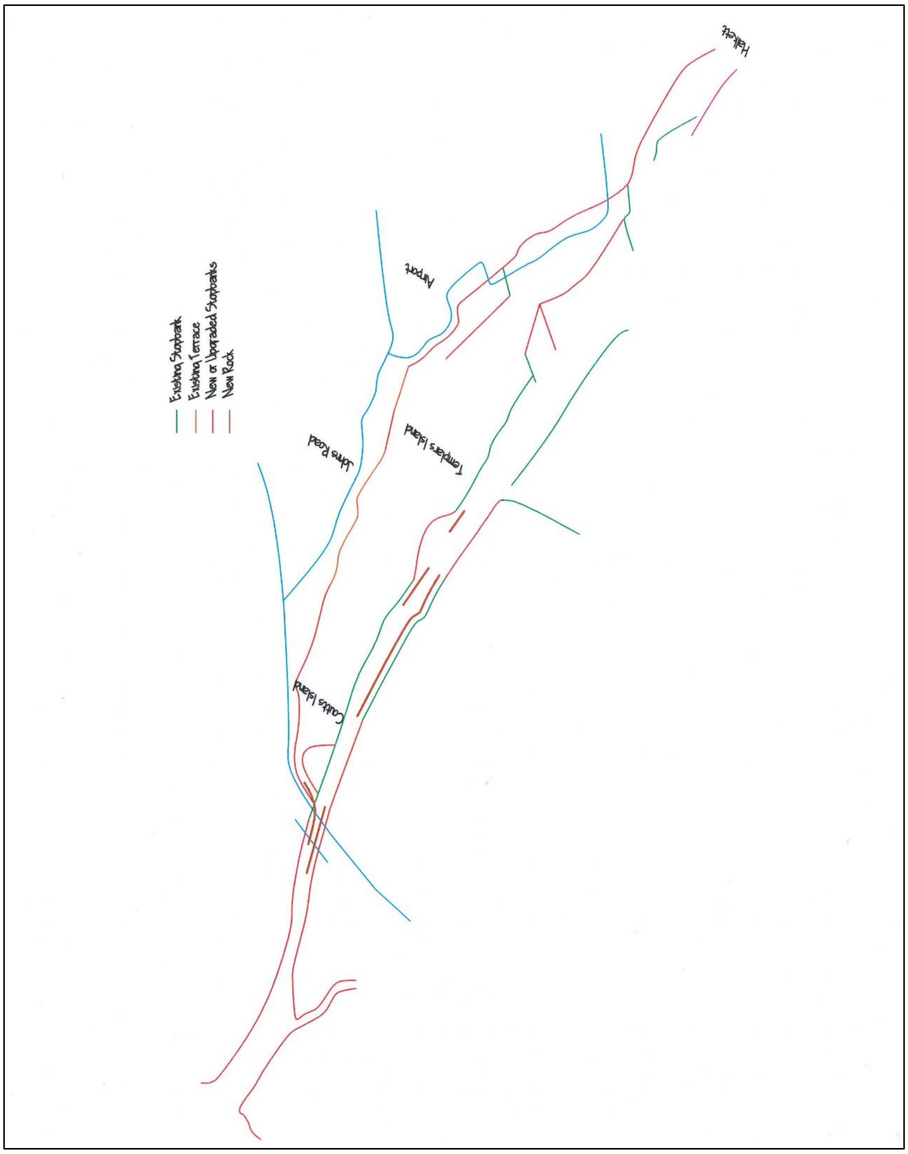


Figure 3-6: Waimakariri River new and upgraded stopbanks and rock works

4 Geomorphology

The Waimakariri floodplain, shown in Figure 4-1, is a classic alluvial fan. McSaveney & Whitehouse (1986) provides a succinct account of the geological process in a review of the flood hazard inherent on the Black Birch fan at Mount Cook village. Although written for a relatively steep fan, in a high country catchment, what it has to say on channel avulsion, and flood risk, is applicable to the coastal fans of the present day Waimakariri floodplain.

> *"A fan is a more-or-less symmetrical cone of shingle, often formed where mountain streams issue from steep side valleys, to flow over more gently sloping surfaces.*
>
> *When the slope of a stream decreases, and its channel becomes wider, the stream's ability to carry shingle decreases, and the shingle is deposited. This build-up of deposits of shingle is called aggradation. Aggradation occurs fastest near the head of the fan (at the apex of the cone). On a fan left in its natural state, the depositing of shingle, in the main channel, causes the stream, every so often, to switch abruptly from one course to another. The stream either creates a new channel, or re- occupies an old channel, where it has flowed in the past. Water then flows down an entirely different part of the fan, where it may not have flowed for decades, or even centuries. This sudden switching occurs when the stream is in flood, and is termed avulsion.*
>
> *Aggradation on a fan is not a steady, continuous, process. Most occurs during rare major storms, when massive amounts of shingle are deposited in a short time, mostly at the fan head. Then follows often long periods when the stream cuts into these deposits, at the fan head, and redistributes the material over the lower fan.*
>
> *Because of the geometry of fans, any point below the fan head is lower than the stream not far up the fan, and, so, is at risk from flooding. This may be brief inundation of areas adjacent to the channel while the stream is in flood. But the more serious risk is the formation of a new main channel, with flooding and the depositing of shingle on a different part of the fan. The risk of being overwhelmed by gravel and water generally decreases further down the fan. However, the existence of old channels will direct the course of future flooding, if water enters them, so that distance down a fan is not always a protection against flooding and aggradation.*
>
> *The particular topography of a fan will determine which areas of the fan are at greatest risk."*

Figure 4-1: Waimakariri River floodplain (fan)

4.1 Geomorphic mapping

Following on from an appreciation of the geomorphology, as a first step in the risk assessment exercise, a geomorphic mapping exercise was carried out of the entire Waimakariri floodplain.

The maps include features such as the Waimakariri River bed, historic floodways, other flood imprints, river control works, other bank structures, fluvial features, other topographical details, and coastal deposits. Across the complete set of maps, the relative ages of the floodplain, and coastal deposits, are recorded. Because of their size, the maps are not included in this report. Instead, a sketch, summarising the general layout of the major floodplain channels is incorporated (Figure 4-2). The detailed maps are available digitally.

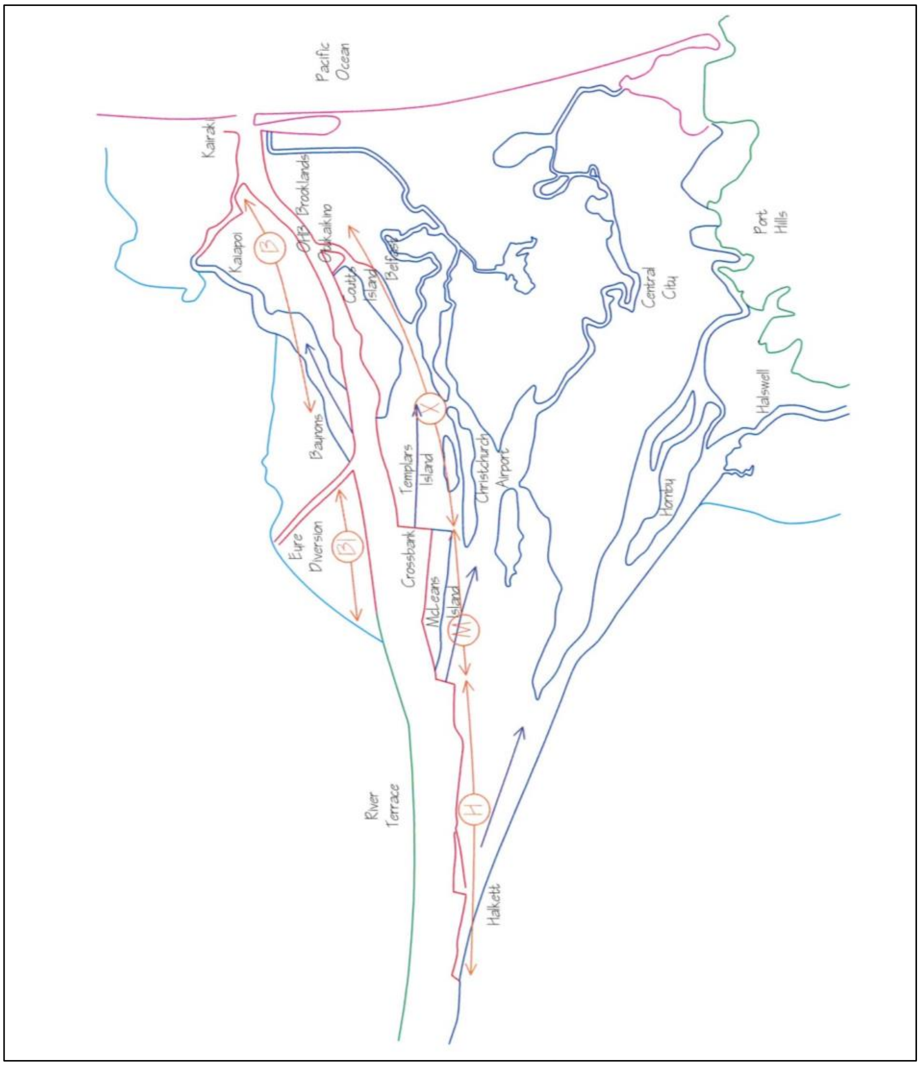


Figure 4-2: Breakout zones and geomorphic summary

5 Protection system analyses (1990)

Extracted from the Draft Waimakariri Floodplain Management Plan (Griffiths, 1991):

"The design flow for the Waimakariri River is 4730 m^3/s, from the sea to Crossbank, and 5100 m^3/s upstream of there, which was estimated to be the 100 year flood when the hydrology of the river was re-visited in the process of designing the Waimakariri River 1960 Improvement Scheme. With the benefit of additional record, this flow figure was estimated to be the 500 year flood.

The object of the current primary protection system is to pass this flow to the sea, and to prevent bank erosion, which would threaten the structural integrity of the stopbanks, and contribute to the supply of sediment, which, in turn, would lead to aggradation, and consequent reduction in flood capacity. Minimising bank and berm erosion also contributes to improved water quality.

The prime protection mechanisms are the stopbanks, which are the outer limits of the suite of measures which make up the structural system.

Inside the stopbanks are berms. Two significant protection mechanisms are employed on the berms - protection plantings and groynes. The latter are banks which are constructed out into the berms from the primary stopbanks. Their purpose is to deflect river flows into the fairway, and to keep them away from the stopbanks. The protection plantings, which are between the groynes, and also where there are no groynes, are aimed at slowing water flows, and developing a cushion of slow moving water, which, along with the plantings, protect the stopbanks from the fast moving flows. The concept is akin to the method of 'fighting fire with fire'. Protection plantings also protect the outer edges of the berms.

The system is subject to a rigorous, continuous, maintenance schedule.

However, braided river systems such as the Waimakariri are, by nature, dynamic, and, with continuing attacks from floodwaters, and continuous changes of the braided patterns, areas of weakness develop, and the protection system becomes vulnerable at various places, and at various times. The maintenance programme monitors, and remedies, the effects of these processes.

The structural strength of the existing works has been the subject of close scrutiny.

Four main conclusions have been drawn:

1. *The stopbanks are designed, and built, to a uniform standard, but they are not designed to withstand overtopping by floodwaters.*
2. *The greatest risk of failure of the system is from erosion of the stopbanks, and such a failure could occur during a flood significantly less than the design flood.*
3. *Stopbank erosion is dependent not only on discharge, but also on other variables.*
4. *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.*

5.1 Sedimentation

Extracted from the Draft Waimakariri Floodplain Management Plan (Griffiths, 1991):

'Aggradation, or the building up of the riverbed, by sediment deposition, has proved to be the single, most difficult, problem affecting the design and management of river protection schemes in the lower Waimakariri River. The reach of major concern is downstream of Weedons Ross Road, where aggradation of gravel is both consistent and persistent.[2] There is good reason for this behaviour; the lower part of an alluvial fan is, by definition, a region of deposition, because that is how fans are built up, and the lower Waimakariri is no exception (the upper end of the fan is degrading and the material is being redistributed at the toe of the fan). In pre-scheme times, aggradation occurred over a broad area as the river migrated about, often in several channels, but since confinement by Hays No. 2 Scheme

[2] *Aggradation of silt also occurs but gravel is the problem.*

1928, the deposition area has been restricted. With the 1960 Scheme, Henderson (1960) thought it physically possible to design a stable channel in the lower reaches capable of passing shingle to the sea. He presented theoretical analyses and numerical data for the design of such a channel, and, to a large extent, his recommendations were adopted in the final design. [It is important to note though the channel was not narrowed to the extent recommended by Mr. Henderson]. *The decision, in 1960, was to give this design a 10-year trial before deciding whether it was necessary to meet the problem by bulk removal of gravel from the river. This proved to be necessary, and commercial extraction was adopted as the exclusive means of removing aggraded material[3]. This has been a very successful policy.*

The quantities of gravel being transported down-river below Lower Gorge has been the subject of detailed investigation and research (Henderson, 1960, Reid and Dick, 1960, Griffiths, 1979, 1987a, Reid and Poynter, 1982, [Hudson, 2005]*). Because gravel does not pass to the sea, the system is closed. By repeated survey of channel bed levels, changes in gravel volume between surveys can be determined.[4] A budget involving this volume, the volume of bank erosion (determined by independent survey) and the volume of commercial extraction yields the average inflow of gravels to any reach in a survey period. From all these analyses, the best estimate for the medium term influx of gravel into the reach below Crossbank is 260,000 $\pm$ 10,000m^3 per year[5]. At least 65% of this gravel is derived from bank erosion below Lower Gorge (Griffiths, 1979). Although the policy of commercial extraction of gravel has been highly successful, it has not accommodated the entire influx, and between 1930 and 1987 some 3.4 million m^3 of gravel still accumulated below Crossbank.*

Griffiths (1988) assessed the long-term, as opposed to the above short-term, average rate of aggradation in the lower reaches. By calculating the volume of post-glacial entrenchment of the river into the Plains between Staircase Gorge and about Halkett, assuming it to have taken place over the past 16,000 years (Wilson, 1985), and by estimating the volume of deposition below Halkett, using data from well logs (and allowing for abrasion) Griffiths (1988) arrived at a figure of 300,000 $\pm$ 140,000m^3 per year. He also calculated that the proportion of gravel coming from bank erosion below Lower Gorge was 70% $\pm$ 11%. Two related estimates are of historical interest. Speight (1927) estimated a rate of bank erosion of 350,000 m^3 per year for the previous 40 years from Kimberley Cliffs just below Lower Gorge on the true right bank, and about 90,000 m^3 per year over the previous 55 years from the area near Two Chain Road. Together, this implies an inflow of at least 440,000 m^3 per year from the banks to the channel.[6] Nelson (1928) notes with characteristic optimism that 'the Waimakariri, when permanent conditions are established will carry 250,000 cubic yards (190,000 m^3) of shingle yearly into its lower course.' He obtained this figure by comparison of Waimakariri conditions with measurements taken on the Rhine in Switzerland, giving a total annual load of 1.9 million m^3, of which he took 90% to be sand and silt and 10% gravel. Griffiths and Glasby (1985) present an estimate for sand and silt of 4.1 million m^3 based on measurements of this material suspended in the flow. Maddock (1975) suggests that the gravel component is 2 to 8% of suspended load so an average value of 5% for gravel implies delivery of 205,000 m^3 per year. Given the limits of error, all those estimates are not really different, and the best determined value of 260,000m^3 $\pm$ 10,000m^3 per year for gravel inflow below Crossbank is almost as well founded as any estimate of this nature can be.

Sedimentation rates vary over different time scales (Carson and Griffiths, 1989; Hudson, 2005). Natural climate variability is a major contributor to the variation, but rates could potentially be affected in a number of other ways. Berm planting and stabilisation, downstream of the Lower Gorge is likely to be reducing the sediment available to the river for transport. Large scale water abstraction particularly at medium and high flows, has the potential to reduce sediment transport rates. Increased flood frequency driven by climate change (see Section 7.6), and/or seismic activity, could increase sediment transport rates in the future.

The supply rate past Crossbank was estimated to be 173,000 m^3/yr between 1995 and 2001 (Hudson, 2005) and 230,000 m^3/yr between 2001 and 2007. *'This may be a temporary lull in bedload input*

[3] *By 1988 the gravel tongue was still located near Stewarts Gully Sailing Club, the reported position in 1960.*
[4] *Seven surveys of this type have been carried out since 1930; the last was completed in 2019.*
[5] *The $\pm$ figure is a standard error i.e. by definition there is a 68% chance the true figure lies within the range 250,000 to 270,000. The standard error describes precision or how well the measurements have been made. How accurate they are is another question which has been addressed by making a number of independent estimates.*
[6] *How much of the material was deposited below Crossbank is unknown.*

associated with modest floods' (Hudson, 2005), but could equally be the start of a longer term trend in reduced sediment supply.

For the purposes of this report, the estimate of 260,000 $m^3 \pm 10,000\ m^3$ per year for gravel inflow below Crossbank (Griffiths, 1988) is regarded as the most reliable estimate, but there is some uncertainty as to how this will change over time. Figure 5-1 provides an example of sedimentation near Stewarts Gully.

Figure 5-1: Silt island near Stewarts Gully

5.2 Gravel extraction rates

Over the last 50 years, gravel extraction rates have approximately kept up with supply rates.

In the 1960s, rates were high (414,000 m^3/yr) with large scale extraction associated with the construction of the Christchurch Northern Motorway (MWH, 2006). Bed levels downstream of Heywards Road (10 km, see Figure 5-2) were lowered, on average, by about 0.9 m over this period.

Throughout the 1970s, 1980s, and early 1990s, reported extraction averaged about 200,000 m^3/yr. This largely kept up with the supply downstream of Heywards Road (10 km) where bed levels recovered by about 0.4 m. With the exception of a short period of extraction for the Christchurch Southern Motorway, little was extracted between Heywards Road and Macleans Island (10 and 26 km, see Figure 5-2 and Figure 5-3) where bed levels rose by an average of about 0.2 m over this period, or about 2.1 million cubic metres.

Below, survey dates refer to the month that the majority of cross-sections were surveyed between SH1 and Crossbank. The survey at a particular location within this reach may have been a month or two earlier (or later), while the reach downstream was generally surveyed the year earlier, and the reach upstream a year later.

Between June 1995 and May 2001 - corresponding to the estimated supply of 173,000 m^3/yr (Hudson, 2005) - reported extraction was 305,000 m^3/yr. Bed levels dropped by 0.1 m downstream of 10 km, dropped substantially in the 10-11 km reach, and rose marginally in the 11-26 km reach.

Between June 2001 and February 2007 - corresponding to the estimated supply of 230,000 m^3/yr - extraction averaged 474,000 m^3/yr. Bed levels downstream of 11km dropped by an average of about 0.35 m. In the 11-15 km reach they dropped by about 0.1 m. They increased marginally in the 15-26 km reach. The location of the drops in bed level correspond roughly to the location of extraction (extraction had started to be encouraged further upstream during this period).

For the 18 months after the last survey in March 2007, reported extraction rates averaged 729,000 m^3/yr. Anecdotal observations suggest bed levels have continued to lower; particularly in the 8-12 km reach, and to a lesser extent as far as Crossbank (18 km).

Between 1960 and 2009, there had been a net loss of sediment of about 3 million cubic metres downstream of 11 km, a net gain of 2 million cubic metres between 11 and 26 km, and a net loss of 9 million cubic metres between 26 and 56 km. The total amount of gravel extracted over this period was about 15 million cubic metres (the majority of which has been extracted between 4 and 10 km).

Between 2009 and 2020, the bed has continued to lower to around the target levels. During that time, around 6 million m^3 of material has been extracted.

The gravel extraction industry has provided, and continues to provide, a service of great benefit to Christchurch and Kaiapoi.

5.3 Hydrology

The most recent re-visit of the Waimakariri River hydrology is detailed in Steel (2016). This report derives the Waimakariri River at SH1 average recurrence interval flows summarised in Table 5-1:

Table 5-1: Estimated annual recurrence interval (ARI) flows for the Waimakariri River at SH1

Mean Annual	5 year	10 year	25 year	50 year	100 year	200 year	500 year
1405	1800	200	2800	3400	4000	4700	5800

5.4 Hydraulics

Extracted from the Draft Waimakariri Floodplain Management Plan (Griffiths, 1991):

As noted above, for the 1960 scheme, the discharge adopted for design purposes in the lower reaches of the river was 4730 m^3/s and 5100 m^3/s for the reach above Crossbank (Reid and Dick 1960). The probability of a flood peak discharge greater than 4730 m^3/s occurring in any year was estimated to be 0.01 (1/100). Put another way, it was anticipated that the design discharge would be exceeded on average once in 100 years, or in other words have a return period of 100 years (Stephen, 1958).[7] At the time this so-called '100-year flood' was the normal standard for river control schemes both in New Zealand and elsewhere. It was thought that provision for a flood of this size was a 'reasonable commitment for the taxpayers of this generation' (Henderson, 1960, p. 16). As an aside, taking the design life of the Scheme as 30 years (1960-1989), the risk of at least one 100-year flood occurring in this interval is 26%. Looked at in this way, the design standard is not as comforting. Return period estimation was based on a record of annual flood peak maxima spanning 1930 to 1957 (28 events). A repeat calculation using the record from 1930 to 1987 (58 events) yields a return period of the order of 450 years, indicating that the original estimate of 200 years (without safety factor) was conservative (Pearson, 1988; Griffiths, 1989a, 1989b).

[7] Stephen (1958) included a safety factor of 10% for flood magnitude in this calculation. His actual estimate for the 100 year event was 4300 m^3/s, which he increased to 4730 m^3/s. The return period for the latter figure according to Stephen (1958) was about 200 years.

The height of the stopbanks at any cross-section of the river was determined as the height needed to pass the design discharge plus 1m of freeboard.[8] *To check if this standard still applied in 1988, and to estimate the theoretical conveyance capacity of the stopbanked reach assuming no bank failure, Williman and Low (1988) carried out a hydraulic simulation of flood levels for the lower Waimakariri. The river reach concerned ran from near Halkett (35.4km) to the mouth, and involved calculation of water levels at 61 cross sections for the design flood. Because, in a hydraulic sense, the bed slope is steep, a backwater curve could be, and was, calculated, using the same methods, in principle, as those employed in the design of the 1960 Scheme. The main findings if the study were: (1) the design standard has been maintained; in other words the present stopbank levels are appropriate for containing a peak flow of 4730 m^3/s with 1m of freeboard*[9] [This was assuming the proposed gravel extraction would be effected. MWH (2000) and Oliver (2007) subsequently confirmed the need for this excavation]; *(2) general overtopping of the banks in the lower reaches below about Crossbank will occur with a flow approximately 50% in excess of design (7000 m^3/s)*[10]*; and (3) in the reach above Crossbank a flow of about twice the design size (9500 m^3/s) might be contained depending on the nature and pattern of flow, and the degree of afflux (building up) of floodwater against stopbanks.*

In considering these results, it is most important to remember the assumption of no failure of the stopbank system.

5.4.1 Waimakariri River design capacity

The objective of the Waimakariri River Improvement Scheme (1960), in its simplest form, was to safely pass to the sea 167,000 ft^3/s (4730 m^3/s). This was a reduction from 180,000 ft^3/s (5100 m^3/s) at the Lower Gorge, due to losses to groundwater between the Lower Gorge and 'Crossbank'.

The primary stopbank design freeboard is 0.9 m at the current design flows of 5,100 m^3/s upstream of and 4730 m^3/s downstream of Crossbank.

The recently completed Waimakariri Flood Protection Project (WFPP), including the secondary stopbank and floodplain system, has a design capacity of 6,500 m^3/s from Halkett to the Otukaikino confluence. This design allows for:

- a 1000 m^3/s breach flow onto the secondary floodplain at Halkett which is returned to the river upstream of McLeans Island.
- a 800 m^3/s breach flow at McLeans Island, and 1200 m^3/s at Templars Island (Crossbank). This produces a combined design flow of 2000 m^3/s down the Templars/Coutts Islands' floodplains. This flow is reduced to around 1000 m^3/s due to a combination of floodplain storage and hydrograph peak lag, before returning via the Otukaikino to the Waimakariri River.

The downstream peak flow, combining the 4,500 m^3/s main channel remnant flow with the 1,000 m^3/s return flow, gives a total flow of 5,500 m^3/s. This 5,500 m^3/s flow was adopted, with a 1 m freeboard downstream to the Old Highway Bridge (OHB), and 0.5 m from there to the sea.

5.4.2 Waimakariri River hydraulic modelling

A Mike11 hydraulic model has been used to determine water levels for various design flows along the Waimakariri River. Figure 5-2 and Figure 5-3 show the locations of the Waimakariri River cross sections (between the coast and 37 km upstream).

The Waimakariri River model was calibrated using flood events from 1988 (1985 m^3/s), 1984 (2825 m^3/s) and 1979 (2910 m^3/s), and cross sections surveyed in 2001. A comparison between the modelled and measured levels is provided in Table 5-2 for various locations between Crossbank and the coast. The model was also validated using the more recent 2017 flood event that had a flow of 2035 m^3/s. Table 5-3 compares modelled and measured flood levels, for the validation event, between Halkett and the Kainga area.

[8] The purpose of freeboard is to accommodate effects such as local build-up of the bed and flow during floods.
[9] This finding was expected as the standard had been checked regularly.
[10] The increase from 4730 to 7000 m^3/s is taken up by the freeboard.

Modelled water levels, and calculated freeboard, are summarised in Table 5-4 to Table 5-6 for design flows of 4000 m^3/s, 4730 m^3/s and 5000 m^3/s, respectively – covering the full study area from the coast upstream to Halkett. For a larger design flows of 5100 m^3/s and 5500 m^3/s, modelled water levels and calculated freeboard are shown in Table 5-7 (for Crossbank to Halkett) and Table 5-8 (from the coast to Otukaikino confluence), respectively.

Table 5-4 demonstrates that the 5,000 m^3/s flow is contained with adequate (1m+) freeboard for the primary stopbanks upstream as far as Halkett/Thompsons Road (cross section 32.19km), with the exception of:

- cross section 3.72km – left bank freeboard is 0.8 m
- cross section 3.36km - left and right bank freeboards are 0.7 m and 0.9 m, respectively.

Table 5-6 shows that the 5,500 m^3/s flow is contained with a 0.5 m freeboard downstream of the Otukaikino confluence to the Old Highway Bridge (OHB) with a minimum of 1 m freeboard. Downstream of the OHB there is a minimum of 0.5 m freeboard with the exception of cross section 3.72km (left bank at Woodford Glen) where the freeboard is 0.4 m.

This report recommends simplifying the primary system standard to 5,000 m^3/s with 1 m freeboard throughout. This is reasonable based on current river bed levels and modelling results. The minor lack of freeboard referred to above is not of significant concern because of the single thread nature of the channel, the low velocities and the absence of observed afflux in those vicinities.

It is further recommended that the freeboard of the 5,500 m^3/s flow be increased to 1 m between the Otukaikino confluence and the OHB because of afflux considerations related to the right angle nature of the junction.

The current and recommended design scenarios for the primary stopbanks and the WFPP secondary stopbanking system are illustrated in Figure 5-4 and Figure 5-5. A composite view is included as Figure 5-6.

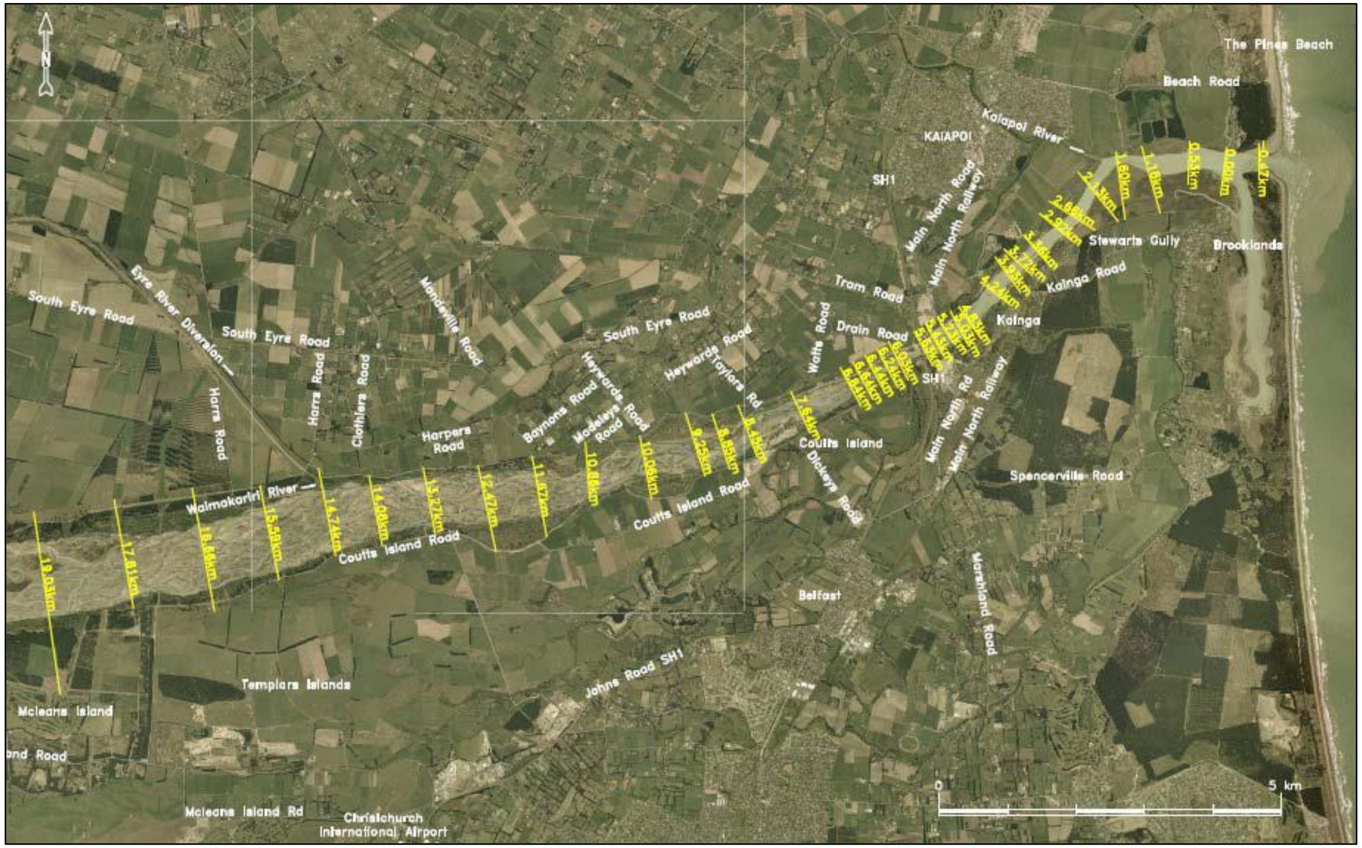


Figure 5-2: Cross-section locations (0 to 19 km)

Figure 5-3: Cross-section locations (17 to 37 km)

Table 5-2: Calibration with historical events (using 2001 cross sections)

Composite cross section roughness as per Relative Roughness & HD11; Default fairway value n = 0.04;

Specific reach values 90450 – 96640 n = 0.03; 96640 – 100000 n = 0.017; non fairway values – trees 0.125; grass 0.04

Location	Cross section distance (km)	May 1988 (1985 m^3/s)		1984 (2825 m^3/s)		Dec 1979 (2910 m^3/s)	
		Actual Level	Model	Actual Level	Model	Actual Level	Model
Crossbank	17.81	44.15 (r), 44.27	44.06	44.29	44.29		44.31
	16.66	37.70, 38.14	38.60	38.65	38.82		38.84
	15.59	33.82, 33.75	34.09	34.1	34.32	33.8	34.34
	14.74	30.02, 29.75	30.15	29.8	30.39	31.14	30.41
Eyre Diversion	14.08	26.61, 27.25	27.30	27.26	27.57	27.12	27.60
	13.27	23.53, 23.62	24.13	23.88	24.40	24.29	24.43
	12.47	19.38, 20.71	20.64	20.11	20.92		20.94
	11.67	18.24, 18.07	18.17	18.25	18.48	18.1	18.50
	10.86	16.28, 15.07	15.71	15.85	16.04	15.8	16.07
	10.06	12.16, 13.42	13.19	13.4	13.54		13.57
	9.25	11.31 (l)	10.96	11.73	11.41	11.45	11.45
	8.45	9.31 (r)	8.84	9.94	9.38	9.7	9.43
	7.64	7.64, 7.45	7.69	8.01	8.24	7.7	8.29
	6.64	6.49, 6.75	6.30	7.08	6.88	6.98	6.93
	6.03	5.65, 5.67	5.34	6.12	5.96	6.14	6.01
South Branch Confluence			4.93		5.57		5.63
	5.63	4.62, 4.94	4.45	5.24	5.13	5.19	5.19
					5.05		5.12
Old Highway Bridge	5.23	3.90 ®	4.15		4.78	4.88	4.84
	4.83	3.77, 3.59	3.94	4.22	4.58	4.31	4.64
	3.93	3.00, 3.67	3.36	3.35	3.95	3.98	4.02
	3.36	2.44, 2.30	2.68	3.24	3.20	3.40	3.26
	2.66	1.80, 1.43	2.06	2.35	2.57	2.68	2.61
	1.60	1.32, 1.57	1.72	1.81	2.22	1.98	2.25
	1.18	1.38 (l)	1.66	1.53	2.13	1.45	2.15
	0.53	1.32, 1.27	1.36	1.40	1.75	1.70	1.76
	0	1.4	1.20	1.50	1.50	1.50	1.50

NOTE	
	Comparison of mean bed levels mid 1980s – 2001 within ~ ± 0.1m on average downstream of OHB.
	Velocities in the above events in order of 2 m^3/s – silt bed downstream of Kaiapoi confluence likely to be significantly scoured during large floods.

Table 5-3: Validation with 19 January 2017 flood event

Verification Run 2035 m^3/s 19 January 2017		
Cross section (km)	Pegged Flood Level	Calculated Flood Level
3.36	2.52	2.49
6.84	6.62	6.84
10.06	13.00	13.19
14.08	27.08	27.03
16.66	38.20	38.50
21.13	60.50	60.58
22.27	65.95	65.89
26.96	89.30	89.41
32.19 (Halkett)	116.47	116.32

Note: The pegged flood marks have variations between the left and right banks, and also on either side, depending on just where the mark was pegged. The closest recorded levels to the calculated levels are the ones listed above. Given the calibration was made over 3 floods (1995, 2825 and 2910 m^3/s), the above verification results are considered more than satisfactory.

Table 5-4: Design water levels and freeboards from the coast to Halkett (flow = 4000 m^3/s)

Mike11 Distance	Cross Section (km)	Left Bank Level (m)	Right Bank Level (m)	Design Water Level 4000 m^3/s	Freeboard Left (m)	Freeboard Right (m)
100000	0.000	2.8	3.1	1.9	0.9	1.2
99470	0.530	3.3	3.2	2.0	1.3	1.2
98820	1.180	3.7	3.6	2.1	1.6	1.5
98400	1.600	3.8	4.0	2.2	1.6	1.8
97870	2.130	4.2	4.2	2.4	1.8	1.8
97340	2.660	4.4	4.4	2.7	1.7	1.7
97080	2.920	4.7	4.5	3.0	1.7	1.5
96640	3.360	5.0	5.3	3.8	1.2	1.5
96280	3.720	5.4	5.6	4.2	1.2	1.4
96070	3.930	5.8	6.1	4.4	1.4	1.7
95760	4.240	6.1	6.3	4.7	1.4	1.6
95170	4.830	6.7	6.9	5.2	1.5	1.7
94970	5.030	6.7	6.9	5.4	1.3	1.5
94765	5.235	7.0	7.2	5.5	1.5	1.7
94570	5.430	7.6	7.6	5.8	1.8	1.8
94370	5.680	8.1	8.0	6.2	1.9	1.8
93970	6.030	8.8	8.9	6.9	1.9	2.0
93760	6.240	9.1	9.1	7.2	1.9	1.9
93560	6.440	9.4	9.4	7.5	1.9	1.9
93360	6.640	9.8	9.8	7.8	2.0	1.0
93160	6.840	10.2	10.1	8.2	2.0	1.9
92360	7.640	11.1	11.1	9.4	1.7	1.7
91550	8.450	12.3	12.2	10.7	1.6	1.5
91150	8.850	12.9	12.9	11.5	2.4	1.4
90750	9.250	13.7	13.7	12.3	2.4	1.4
89940	10.060	15.4	15.3	14.1	1.3	1.2
89140	10.860	17.6	18.2	16.3	1.3	1.9
88330	11.670	20.5	19.7	18.4	2.1	1.3
87530	12.470	22.8	22.1	21.1	1.7	1.0
86730	13.270	25.8	25.7	24.3	1.5	1.4
85920	14.080	29.0	29.1	27.6	1.4	1.5
85260	14.740	33.0	32.8	30.5	2.5	2.3
84410	15.500	36.3	36.5	34.4	1.9	2.1
83340	16.660	40.7	41.3	39.0	1.7	2.3
82190	17.810	46.5	46.1	44.4	2.1	1.7
80970	19.030	52.3	51.7	50.0	2.3	1.7
80070	19.930	56.8	56.5	54.5	2.3	2.0
78870	21.130	63.6	62.1	60.7	2.9	1.4
77730	22.270	68.3	67.5	66.3	2.0	1.2
76350	23.650	75.7	74.6	73.4	2.3	1.2
75570	24.430	79.8	78.4	77.4	2.4	1.0
74250	25.750	86.9	85.6	83.8	3.1	1.8
73040	26.960	93.7	91.6	89.8	3.9	1.8
71840	28.160	99.3	98.3	96.4	2.9	1.9
706360	29.370	107.1	105.4	102.8	4.3	2.6
69820	30.150	112.5	109.5	107.0	5.5	2.5
68620	31.740	120.4	116.7	112.8	7.6	3.9
67810	32.190	125.4	120.5	117.0	8.4	3.5

Table 5-5: Design water levels and freeboards from the coast to Halkett (flow = 4730 m^3/s)

Mike11 Distance	Cross Section (km)	Left Bank Level (m)	Right Bank Level (m)	Design Water Level 4730 m^3/s	Freeboard Left (m)	Freeboard Right (m)
100000	0.000	2.8	3.1	1.8	1.0	1.3
99470	0.530	3.3	3.2	2.0	1.3	1.2
98820	1.180	3.7	3.6	2.3	1.4	1.3
98400	1.600	3.8	4.0	2.4	1.4	1.6
97870	2.130	4.2	4.2	2.6	1.6	1.6
97340	2.660	4.4	4.4	2.9	1.5	1.5
97080	2.920	4.7	4.5	3.3	1.4	1.2
96640	3.360	5.0	5.3	4.1	0.9	1.2
96280	3.720	5.4	5.6	4.6	0.8	1.0
96070	3.930	5.8	6.1	4.8	1.0	1.3
95760	4.240	6.1	6.3	5.1	1.0	1.2
95170	4.830	6.7	6.9	5.7	1.0	1.2
94970	5.030	6.7	6.9	5.8	0.9	1.1
94765	5.235	7.0	7.2	5.9	1.1	1.3
94570	5.430	7.6	7.6	6.3	1.3	1.3
94370	5.680	8.1	8.0	6.6	1.5	1.4
93970	6.030	8.8	8.9	7.3	1.5	1.6
93760	6.240	9.1	9.1	7.6	1.5	1.5
93560	6.440	9.4	9.4	8.0	1.4	1.4
93360	6.640	9.8	9.8	8.3	1.4	1.5
93160	6.840	10.2	10.1	8.6	1.6	1.5
92360	7.640	11.1	11.1	9.9	1.2	1.2
91550	8.450	12.3	12.2	11.1	1.2	1.1
91150	8.850	12.9	12.9	11.9	1.0	1.0
90750	9.250	13.7	13.7	12.7	1.0	1.0
89940	10.060	15.4	15.3	14.4	1.0	0.9
89140	10.860	17.6	18.2	16.6	1.0	1.6
88330	11.670	20.5	19.7	18.7	1.8	1.0
87530	12.470	22.8	22.1	21.3	1.5	0.8
86730	13.270	25.8	25.7	24.5	1.3	1.2
85920	14.080	29.0	29.1	27.9	1.1	1.2
85260	14.740	33.0	32.8	30.6	2.4	2.2
84410	15.500	36.3	36.5	34.6	1.7	1.9
83340	16.660	40.7	41.3	39.2	1.5	2.1
82190	17.810	46.5	46.1	44.5	2.0	1.6
80970	19.030	52.3	51.7	50.2	2.1	1.5
80070	19.930	56.8	56.5	54.6	2.2	1.9
78870	21.130	63.6	62.1	60.9	2.7	1.2
77730	22.270	68.3	67.5	66.4	1.9	1.1
76350	23.650	75.7	74.6	73.5	2.2	1.1
75570	24.430	79.8	78.4	77.5	2.3	0.9
74250	25.750	86.9	85.6	84.0	2.9	1.6
73040	26.960	93.7	91.6	90.0	2.7	1.6
71840	28.160	99.3	98.3	96.5	1.8	1.8
706360	29.370	107.1	105.4	103.0	4.1	2.4
69820	30.150	112.5	109.5	107.2	5.3	2.3
68620	31.740	120.4	116.7	113.0	7.4	3.7
67810	32.190	125.4	120.5	117.2	8.2	3.3

Table 5-6: Design water levels and freeboards from the coast to Halkett (flow = 5000 m^3/s)

Mike11 Distance	Cross Section (km)	Left Bank Level (m)	Right Bank Level (m)	Design Water Level 5000 m^3/s	Freeboard Left (m)	Freeboard Right (m)
100000	0.000	2.8	3.1	1.9	0.9	2.2
99470	0.530	3.3	3.2	2.0	1.3	1.2
98820	1.180	3.7	3.6	2.3	1.4	1.3
98400	1.600	3.8	4.0	2.4	1.4	1.6
97870	2.130	4.2	4.2	2.7	1.5	1.5
97340	2.660	4.4	4.4	3.0	1.4	1.4
97080	2.920	4.7	4.5	3.4	1.3	1.1
96640	3.360	5.0	5.3	4.2	0.8	1.1
96280	3.720	5.4	5.6	4.7	0.7	0.9
96070	3.930	5.8	6.1	5.0	1.8	1.1
95760	4.240	6.1	6.3	5.2	0.9	1.1
95170	4.830	6.7	6.9	5.8	0.9	1.1
94970	5.030	6.7	6.9	5.9	0.8	1.0
94765	5.235	7.0	7.2	6.1	0.9	1.1
94570	5.430	7.6	7.6	6.4	1.2	1.2
94370	5.680	8.1	8.0	6.8	1.3	1.2
93970	6.030	8.8	8.9	7.5	1.3	1.4
93760	6.240	9.1	9.1	7.8	1.3	1.3
93560	6.440	9.4	9.4	8.1	1.3	1.3
93360	6.640	9.8	9.8	8.4	1.4	1.4
93160	6.840	10.2	10.1	8.8	1.4	1.3
92360	7.640	11.1	11.1	10.0	1.1	1.1
91550	8.450	12.3	12.2	11.3	1.0	0.9
91150	8.850	12.9	12.9	12.0	0.9	0.9
90750	9.250	13.7	13.7	12.9	0.8	0.8
89940	10.060	15.4	15.3	14.5	0.9	0.8
89140	10.860	17.6	18.2	16.7	0.9	1.5
88330	11.670	20.5	19.7	18.8	1.7	0.9
87530	12.470	22.8	22.1	21.4	1.4	0.7
86730	13.270	25.8	25.7	24.6	1.2	1.1
85920	14.080	29.0	29.1	27.9	1.1	1.2
85260	14.740	33.0	32.8	30.7	1.3	1.1
84410	15.500	36.3	36.5	34.6	1.7	1.9
83340	16.660	40.7	41.3	39.2	1.5	2.1
82190	17.810	46.5	46.1	44.6	1.9	1.5
80970	19.030	52.3	51.7	50.2	2.1	1.5
80070	19.930	56.8	56.5	54.7	2.1	1.8
78870	21.130	63.6	62.1	60.9	2.7	1.2
77730	22.270	68.3	67.5	66.5	1.8	1.0
76350	23.650	75.7	74.6	73.6	2.1	1.0
75570	24.430	79.8	78.4	77.6	2.2	0.8
74250	25.750	86.9	85.6	84.0	2.9	1.6
73040	26.960	93.7	91.6	90.0	3.7	1.6
71840	28.160	99.3	98.3	96.6	2.7	1.7
706360	29.370	107.1	105.4	103.0	4.1	2.4
69820	30.150	112.5	109.5	107.3	5.2	2.4
68620	31.740	120.4	116.7	113.1	7.3	3.6
67810	32.190	125.4	120.5	117.3	8.1	3.2

Table 5-7: Design water levels and freeboards from Crossbank to Halkett (flow = 5100 m³/s)

Mike11 Distance	Cross Section (km)	Left Bank Level (m)	Right Bank Level (m)	Design Water Level 5100 m^3/s	Freeboard Left (m)	Freeboard Right (m)
82190	17.810	46.5	46.1	44.6	1.9	1.6
80970	19.030	52.3	51.7	50.3	2.0	1.4
80070	19.930	56.8	56.5	54.7	2.1	1.8
78870	21.130	63.6	62.1	61.0	1.6	1.1
77730	22.270	68.3	67.5	66.5	1.8	1.0
76350	23.650	75.7	74.6	73.6	2.1	1.0
75570	24.430	79.8	78.4	77.6	2.2	0.8
74250	25.750	86.9	85.6	84.1	2.8	1.5
73040	26.960	93.7	91.6	90.0	3.7	1.6
71840	28.160	99.3	98.3	96.6	2.7	1.7
706360	29.370	107.1	105.4	103.0	4.1	2.4
69820	30.150	112.5	109.5	107.3	5.2	2.2
68620	31.740	120.4	116.7	113.1	7.3	3.6
67810	32.190	125.4	120.5	117.3	8.1	3.2

Table 5-8: Design water levels and freeboards from coast to Otukaikino confluence (flow = 5500 m³/s)

Mike11 Distance	Cross Section (km)	Left Bank Level (m)	Right Bank Level (m)	Design Water Level 5500 m^3/s	Freeboard Left (m)	Freeboard Right (m)
100000	0.000	2.8	3.1	1.9	0.9	1.2
99470	0.530	3.3	3.2	2.1	1.2	1.1
98820	1.180	3.7	3.6	2.4	1.3	1.2
98400	1.600	3.8	4.0	2.5	1.3	1.5
97870	2.130	4.2	4.2	2.8	1.4	1.4
97340	2.660	4.4	4.4	3.2	1.2	1.2
97080	2.920	4.7	4.5	3.6	1.1	0.9
96640	3.360	5.0	5.3	4.5	0.5	0.8
96280	3.720	5.4	5.6	5.0	0.4	0.6
96070	3.930	5.8	6.1	5.2	0.6	0.9
95760	4.240	6.1	6.3	5.4	0.7	0.9
95170	4.830	6.7	6.9	6.1	0.6	0.8
94970	5.030	6.7	6.9	6.2	0.5	0.7
94765	5.235	7.0	7.2	6.3	0.7	0.9
94570	5.430	7.6	7.6	6.7	0.9	0.9
94370	5.680	8.1	8.0	7.0	1.1	1.0
93970	6.030	8.8	8.9	7.7	1.1	1.2
93760	6.240	9.1	9.1	8.1	1.0	1.0

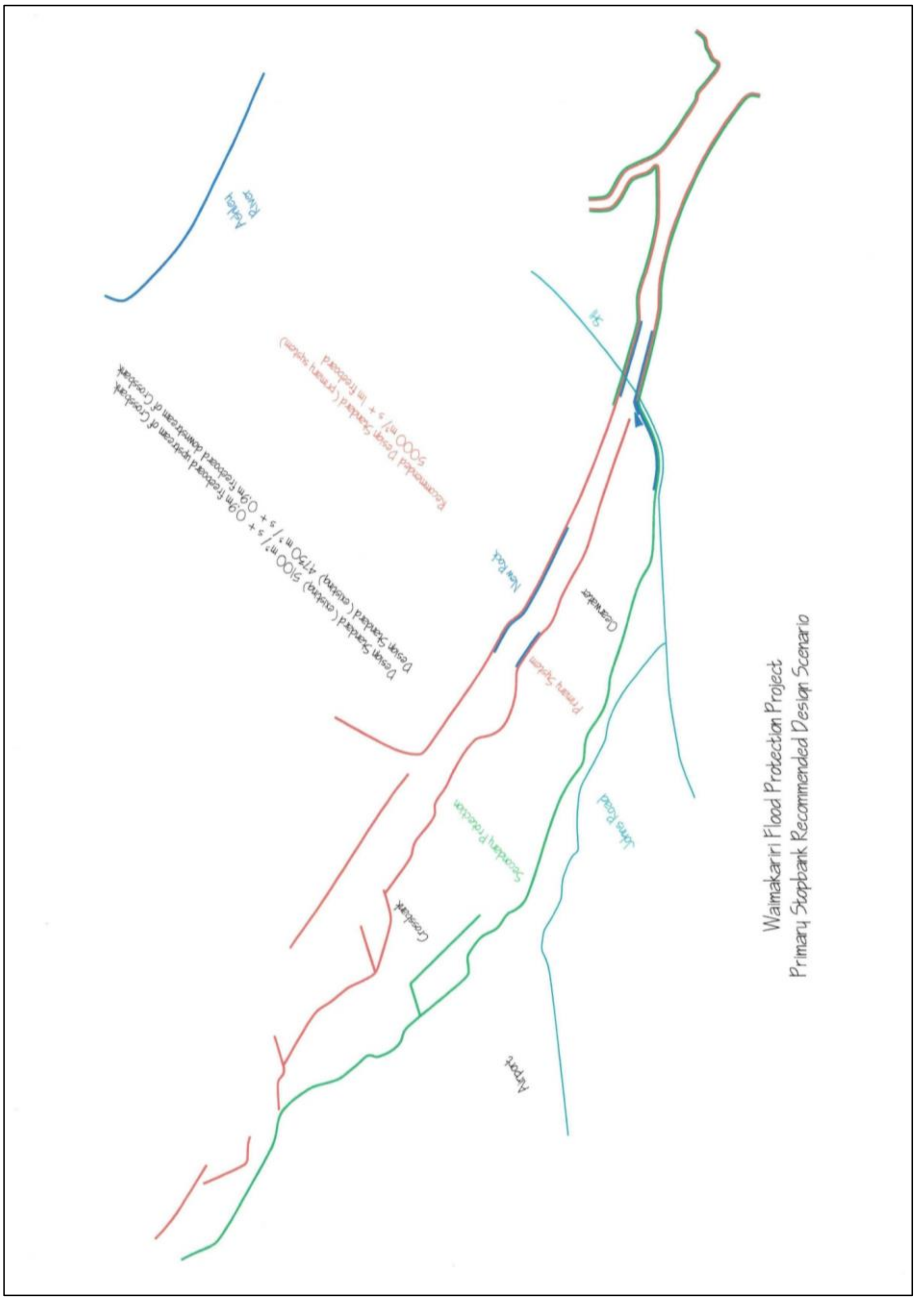


Figure 5-4: Waimakariri Flood Protection Project - primary stopbank recommended design scenario

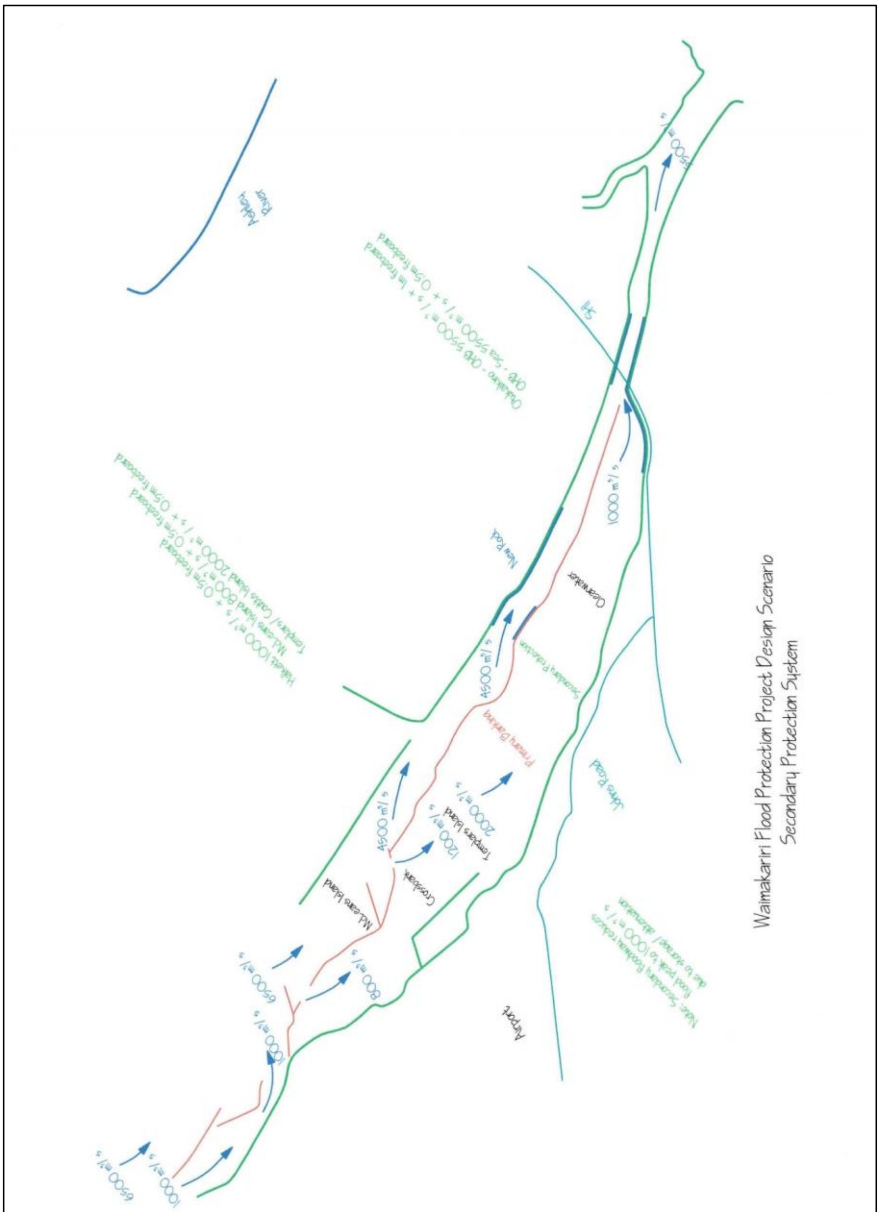


Figure 5-5: Waimakariri Flood Protection Project - secondary protection system recommended design scenario

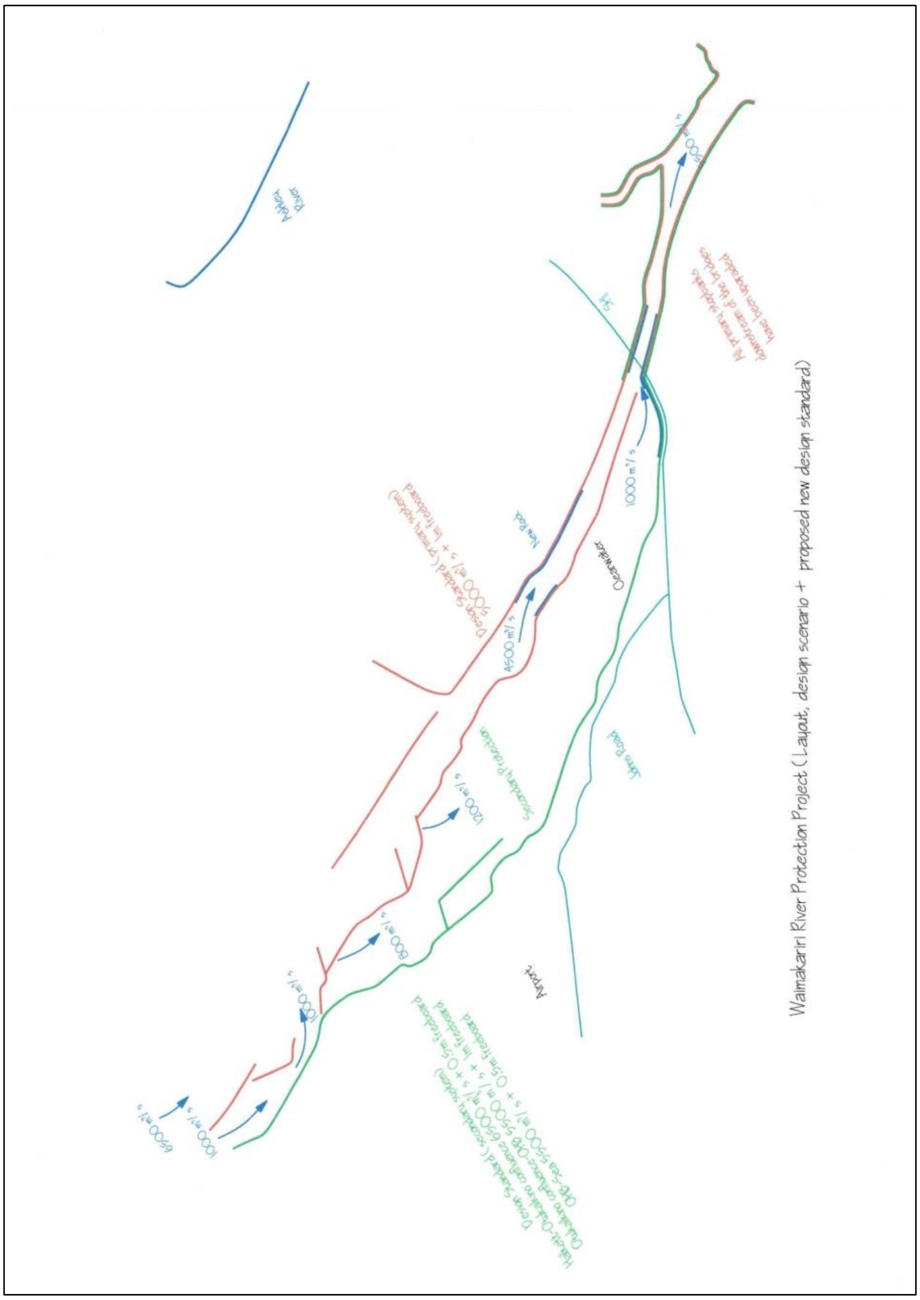


Figure 5-6: Waimakariri Flood Protection Project - layout, design scenario and proposed new design standard)

5.5 Failure locations

To deal with the continuum of points along the stopbank system, the total system was reduced to five zones, on the basis that regardless of where the failure occurred along those zones, the outflows would occupy the same floodplain channels, and result in similar adverse effects. These were deduced from the geomorphic maps.

South side:

- Zone H (Halkett) covers the river from Courtenay to West Melton (Selwyn District). This is the zone from which floodwaters inundated Christchurch during the mid to late nineteenth century.
- Zone M (McLeans) is from West Melton to Crossbank.
- Zone X (Crossbank) is from Crossbank to the sea.

Zone H impacts the Selwyn District area, and Zones M and X the Christchurch City Council area.

North side:

- Zone B1 spans from Dixon's Bay to the Eyre Diversion.
- Zone B (Baynons) from the Eyre Diversion to the sea.

Both of these zones impact the Waimakariri District area.

5.6 Flooding scenarios

Once certain threshold flood levels are reached, failures can occur at any point on the system,. For this exercise, initially two threshold flow ranges were adopted:

Conservative: the assessors were 100% confident that, below the bottom of the flood scenario flow range, there would be no possibility of failure.

Liberal: the assessors were 100% confident that, above the bottom of the flood scenario flow range, there was some chance of failure.

In other words, the assessors considered the true answer, as best as it can be assessed, lay somewhere between the two scenarios.

The scenarios all involve:

- a flood range in the river,
- a single failure, or multiple failures, of the protection system,
- for each failure, a range of outflows onto the floodplain.

5.7 River floods

The river flood flows considered ranged from 500 m^3/s (conservative assessment) and 3300 m^3/s (liberal assessment), up to a maximum of 10,000 m^3/s. As noted above, for each assessment, below the lowest flow, absolute confidence was assigned that the existing maintained system will not fail.

For the conservative assessment the floods were grouped into flow ranges, namely 500-750, 750-1000, 1000-1250, 1250-1750, 1750-2250, 2250-2750, 2750-3300, 3300-3750, 3750-4250, 4250-4750, 4750-5250, 5250-5750, 5750-6250, 6250-6750, 6750-7875, 7875-10000, and 10,000 m^3/s.

The liberal assessment commenced at 3300 m^3/s and involved the same flow ranges up to 10,000 m^3/s.

5.8 Scenario risks (Risk assessment diagrams)

Associated with each flood range, there are a number of probabilities. Firstly, there is the probability of the flood occurring. Secondly, there are the probabilities of the existing system coping with the flood event, and, if not, probabilities associated with where the failures will occur. Finally, there are probabilities associated with the range of outflows that can occur.

The final assigned probabilities are in Table 8-1 (Section 8) and the risk assessment diagrams are in Appendix 1. The diagrams have been included, as well as Table 8-1, as they include the outflow ranges which have been used for the various two-dimensional modelling exercises across the floodplain, that have been carried out since 1990, principally for consenting purposes. They were assigned having due regard to the structural integrity of the existing system investigations, and from practical knowledge of how the system has functioned in previous events.

The probabilities here relate to failures caused by erosion, overtopping, piping, and seepage.

5.9 Outflows

Outflows of floodwaters are related to the extent of the actual failure, and the magnitude of the flood in the river. For each of the scenarios, three ranges of outflows were assigned, ranging between the maximum and minimum outflows that can be expected (based off regional, national, and international experience).

6 Assessing risk

To calculate composite risk (over the full range of floods for individual sites on the floodplain), the zone(s) that would relay floodwaters to the site were identified using the geomorphic maps.

The probabilities of breakouts from these zones were then multiplied by the current estimated annual exceedance probability of the flood ranges. Across the full range of floods, these values were then summed to give the annualised risk.

7 Scheme performance

Since 1957, there have been 23 floods in the 1250-1750 m^3/s range, and no failures. The conservative assessment has a 10% chance of failure for this flood range. This suggests a better scenario would be more towards the liberal assessment, which has 0% chance of failure for this range.

Similarly, for a larger flow range of 2250-2750 m^3/s, there have been 7 floods around this size, and again no failures. The conservative assessment predicts a 35% chance of failure, so again this would suggest a scenario more towards the liberal assessment.

Higher flows of 2870 m^3/s (in 1984), 2910 m^3/s (1979) and 2400 m^3/s (2013) also resulted in no failures.

Based on this historic information, the 'most likely scenario' would be towards the liberal end of the risk envelope, that has been developed.

If we adopt the proposition of no breakouts below 2500 m^3/s, proportioning 2500 m^3/s between the liberal and conservative lower bounds (of 3300 and 500 m^3/s) suggests a weighting towards the liberal end of the assessments of 60%. For 2900 m^3/s the weighting would be 74%.

From another perspective, stopbank performance at other locations, in large floods around the 1% AEP flood level, can be used as a comparison. For example, 1% AEP floods have occurred on the Manawatu and Whakatane Rivers in the past, with one stopbank breach occurring on the Manawatu (erosion), and none on the Whakatane River. Two further overtopping failures on the Manawatu River occurred, but they were at known points of low freeboard.

Based on the above, it was decided to adopt 75% weighting towards the liberal assessment. This recognises that the current primary system, with restored bed levels, has hydraulic capacity around the 200 to 300 year ARI level at this time. It also considers its performance record to date and, most of all, the shingle management processes that are in place.

The Decision Tree diagrams were re-worked on the basis of the above weighting. A final moderating adjustment run was made in October 2019. The results of this final scenario are set out in Section 8 (Table 8-1). The diagrams (Appendix 1) include break-out flow ranges and probabilities, which have been used in the past, and can be used in the future, for floodplain hydraulic modelling purposes.

8 Adopted scenario (October 2019)

Table 8-1: Adopted scenario

Flood Range	AEP	H	X	M	B	H+M	H+X	H+B	M+X	M+B	X+B	H+M+B	H+X+B	H+M+X	M+X+B	H+M+X+B	N/B
2250-2750	0.0714		0.005	0.05													0.99
2750-3300	0.0333		0.02	0.02													0.96
3300-3750	0.0170		0.075	0.075													0.90
3750-4250	0.0093		0.075	0.075													0.85
4250-4750	0.0052		0.10	0.10													0.80
4750-5250	0.0029		0.125	0.125													0.75
5250-5750	0.0017	0.03	0.064	0.064	0.064	0.007	0.007	0.007	0.03	0.03	0.03						0.70
5750-6250	0.0011	0.12	0.12	0.12	0.12	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313						0.30
6250-6750	0.0007	0.0884	0.0884	0.0884	0.0884	0.0544	0.0544	0.0544	0.0544	0.0544	0.0544	0.0226	0.0226	0.0226	0.0226		0.20
6750-7875	0.0003	0.0848	0.0848	0.0848	0.0848	0.0758	0.0758	0.0758	0.0758	0.0758	0.0758	0.0440	0.0440	0.0440	0.0440		0.0
7875-10000	0.00008	0.0340	0.0340	0.0340	0.0340	0.0860	0.0860	0.0860	0.0860	0.0860	0.0860	0.0696	0.0696	0.0696	0.0696	0.0696	0.0
10000	0.00003	0.0300	0.0300	0.0300	0.0300	0.0854	0.0854	0.0845	0.0854	0.0854	0.0854	0.0740	0.0740	0.0740	0.0740	0.0740	0.0

9 Annualised risk (2019)

Total risk has been annualised over the full range of floods. The result is a 1% chance of failure, in any given year, which is considered eminently satisfactory by any standards (nationally or internationally). The results are shown in Table 9-1~~Table 9-1~~.

Table 9-1: Calculation of total annualised risk

Flood Range (m^3/s)	Chance of Failure	Annual Exceedance Probability (AEP)	Chance of failure x AEP
2250-2750	0.01	0.0714	0.0007
2750-3300	0.04	0.0333	0.0013
3300-3750	0.10	0.0170	0.0017
3750-4250	0.15	0.0093	0.0014
4250-4750	0.20	0.0052	0.0010
4750-5250	0.25	0.0029	0.0007
5250-5750	0.30	0.0017	0.0005
5750-6250	0.70	0.0011	0.0008
6250-6750	0.80	0.0007	0.0006
6750-7875	1.00	0.0003	0.0003
7875-10000	1.00	0.00008	0.00008
10000	1.00	0.00003	0.00003
Total			0.009; say 0.01 (1%)

10 Planning matters

The Canterbury Regional Council Policy Statement (CRPS) contains two key policies relating to flooding:

Policy 11.3.1 states that development should be avoided in 'High Hazard Areas', which are defined as areas where the water depth (metres) x velocity (metres per second) is ≥ 1, or where depths are > 1 metre in a 500 year Average Recurrence Interval (ARI) flood event. However, areas which were identified in a District Plan for residential use at the time the CRPS was notified (January 2013), are specifically exempted from this policy.

Policy 11.3.2 states that development should be avoided in areas subject to inundation in a 200 year ARI flood event, unless a range of conditions are met. These include the requirement for new buildings to have an appropriate floor level above the 200 year ARI flood level.

In regard to 'High Hazard Areas', this assessment has concluded that for a 500 year ARI flood (currently estimated to be 5,800 m^3/s) there is:

- 5% chance of flooding in Selwyn District between the Primary and Secondary banks and 0% outside the Secondary bank.
- 21% chance of flooding in Waimakariri District.
- 17%+ chance of flooding of Christchurch City between the two banking systems, and 0% outside the Secondary bank.

Commented [NG1]: Check these once confirmed other numbers are correct.

In summary, the primary system has a 200 to 300 year ARI standard, with a 1% chance of failure in any one year taking into account potential failures below the design standard.

11 Peer review

The report has been peer reviewed by Ross Vesey, who has had a long association with the rivers of Canterbury. All his peer review comments have been incorporated into the report. In addition, he noted:

"This is an excellent piece of work.

The Risk Assessment demonstrates that the standard of protection afforded by the primary and secondary systems is high by New Zealand and International standards.

It will, doubtless, be re-visited in the future, as the period of record increases.

The standard of protection relies on all components of the system being in good condition at all times. For this reason, an ongoing pro-active maintenance programme is an essential element of the protection system, along with ensuring that Regional Park developments do not increase the risk of public, or hydraulic, damage, to any element of the system."

12 References

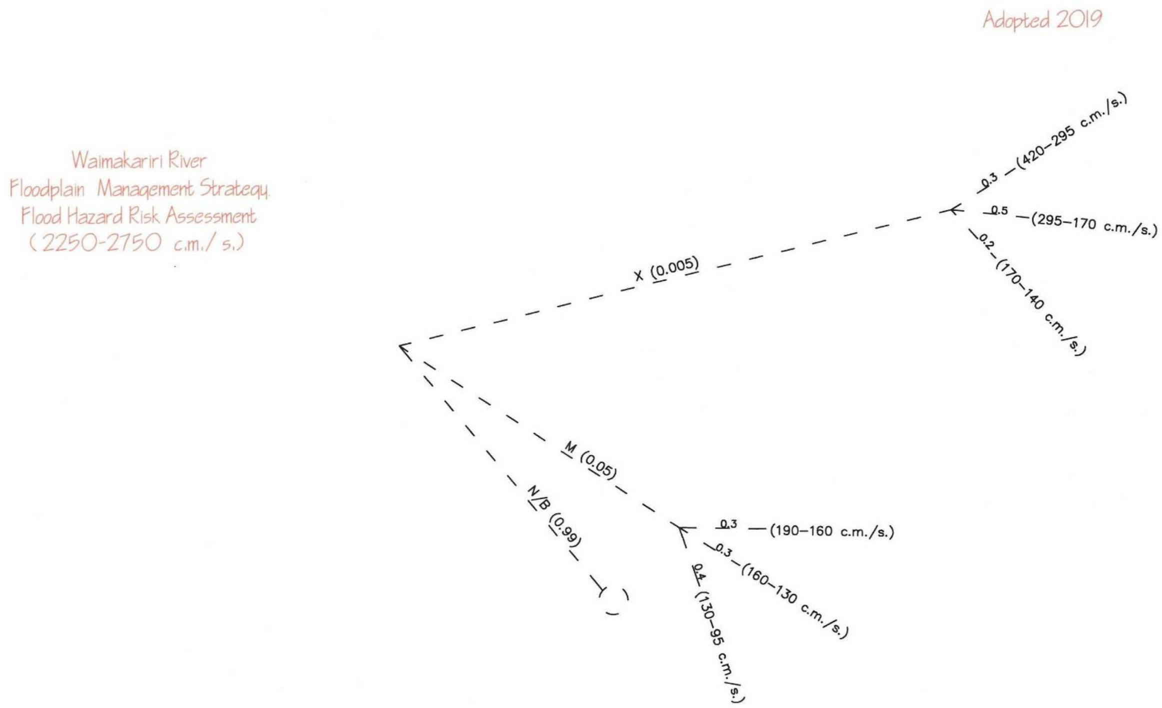
Boyle, T., Surman, M. (2007). Waimakariri River floodplain management strategy - flood hazard risk assessment, Canterbury Regional Council Report No. R07/2. Prepared January 2007. ISBNI 1-86937-628-5.

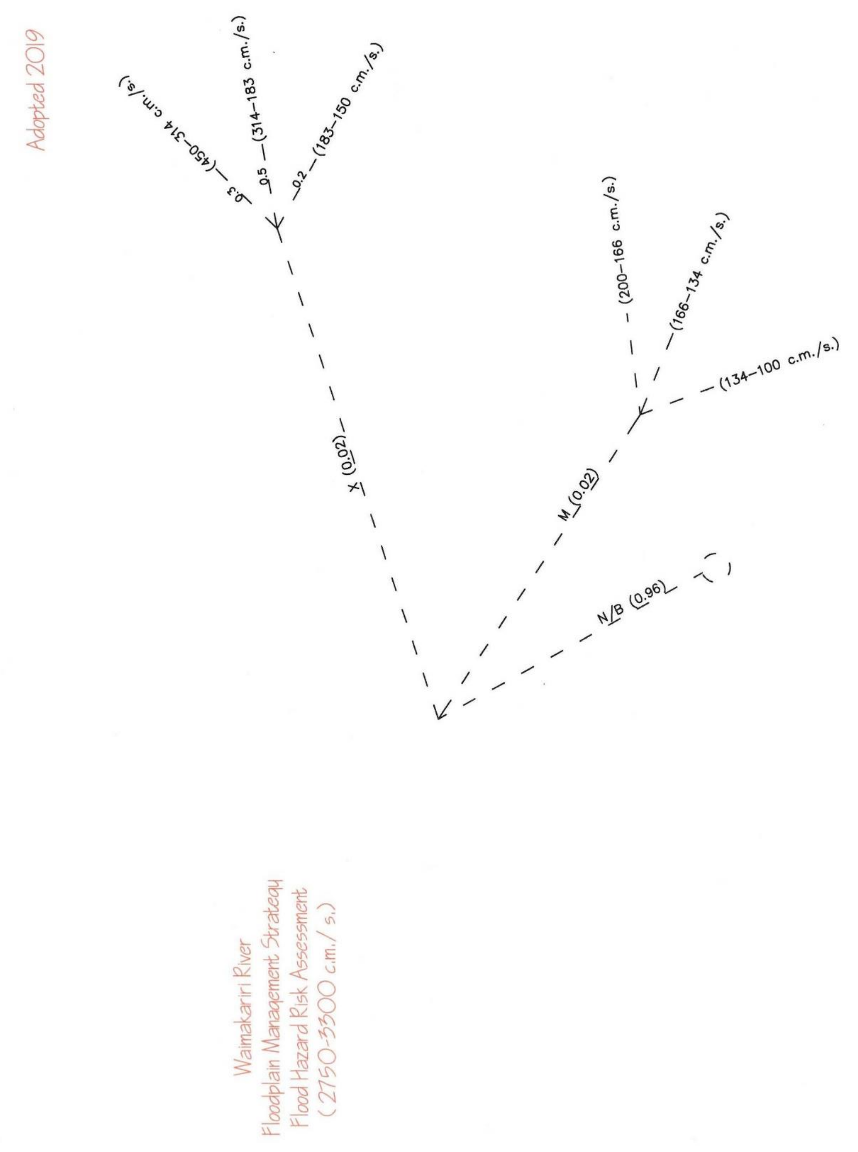
Griffiths, G.A. (1991). Draft Waimakariri River Floodplain Management Plan: Canterbury Regional Council Report R91(9). Prepared July 1991.

McSaveney, M.J., Whitehouse, I.E. (1986). A geomorphological appraisal of flood protection measures on Black Birch fan, Mount Cook. Report WS 1081, Ministry of Works and Development, Christchurch.

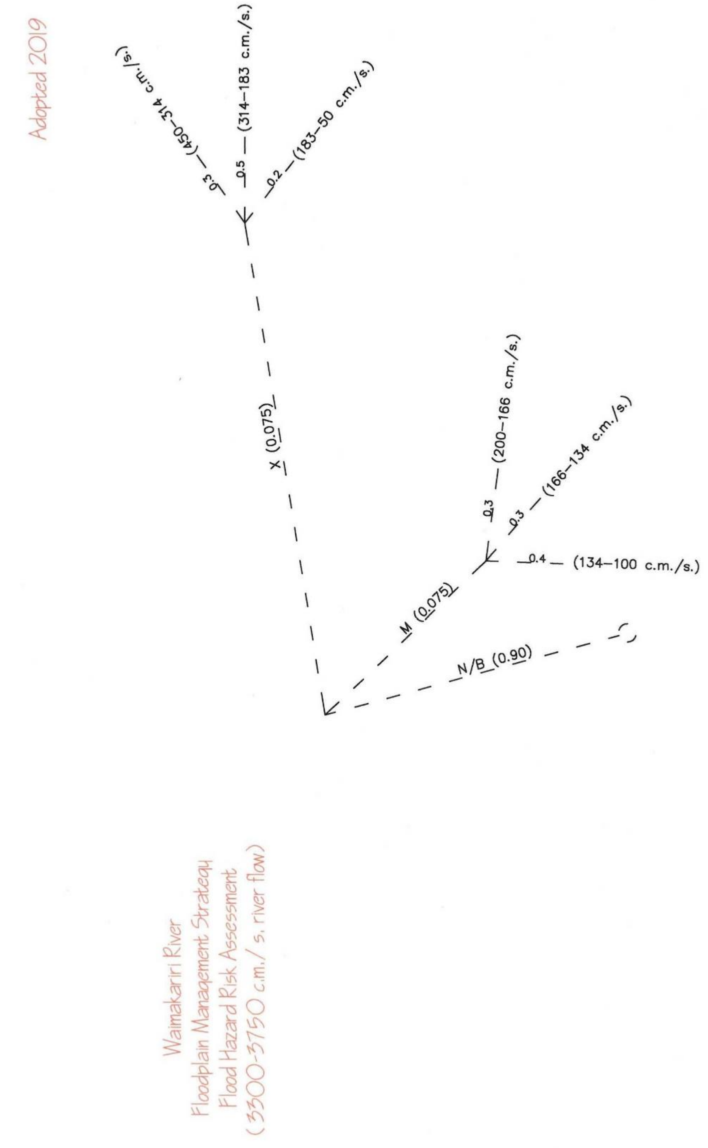
Steel, K. (2016). Flood frequency analysis pilot study: Orari, Temuka, and Waimakariri Rivers. Canterbury Regional Council Report No. R15/129. Prepared May 2016. ISBN 978-0-908316-72-4 (print) 978-0-908316-67-0 (web).

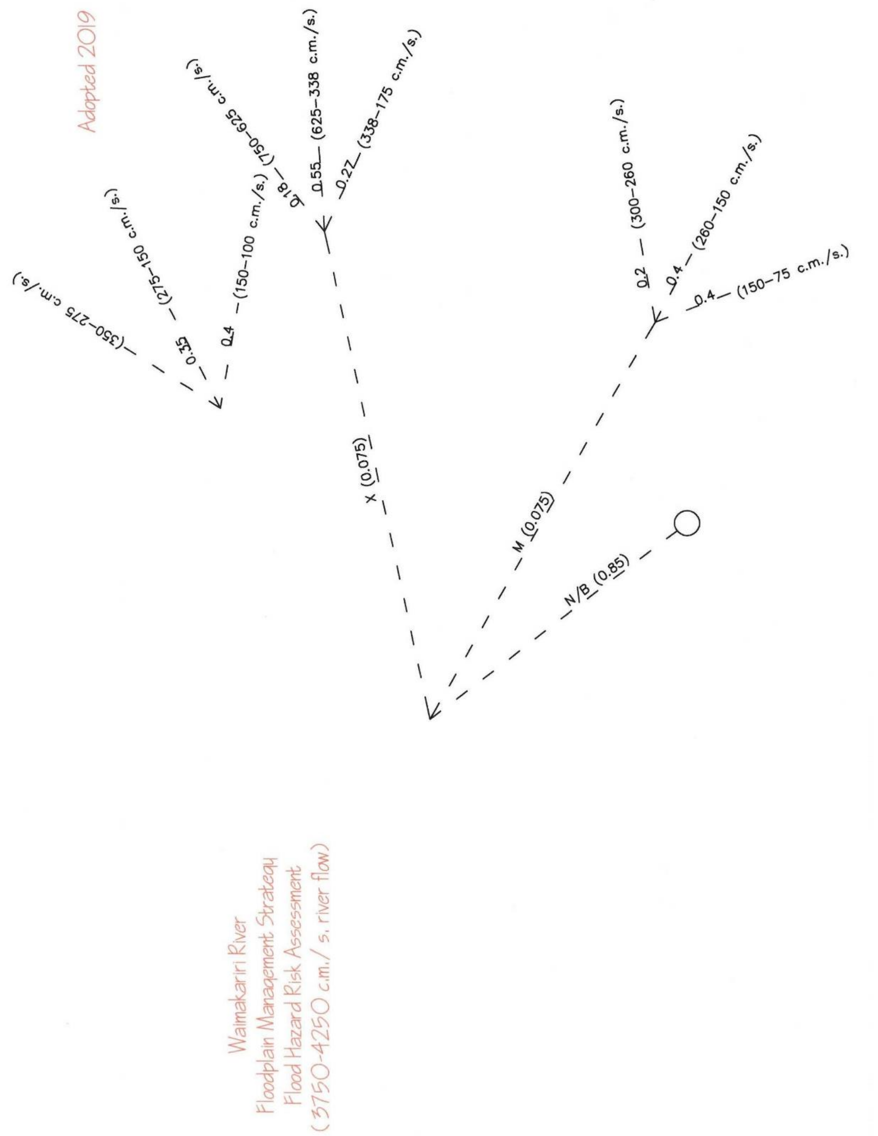
Appendix 1: Waimakariri River Floodplain Management Risk - Assessment Diagrams

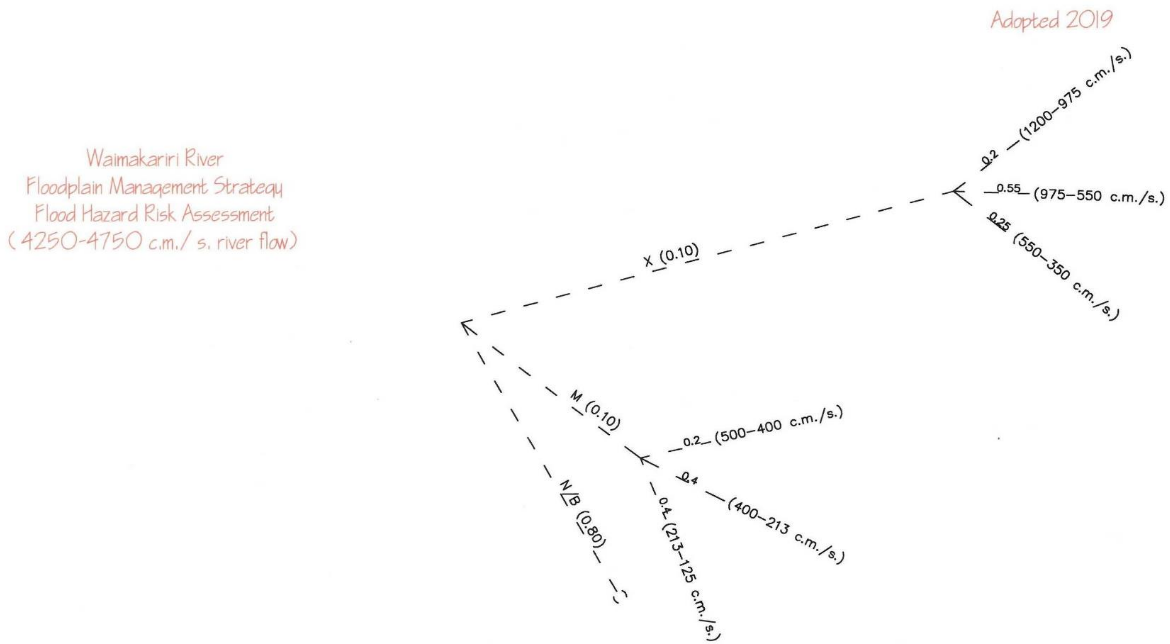

Adopted 2019
Waimakariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(2250-2750 c.m./s.)
X (0.005)
0.3 (420–295 c.m./s.)
0.5 (295–170 c.m./s.)
0.2 (170–140 c.m./s.)
M (0.05)
0.3 (190–160 c.m./s.)
0.3 (160–130 c.m./s.)
0.4 (130–95 c.m./s.)
N/B (0.99)

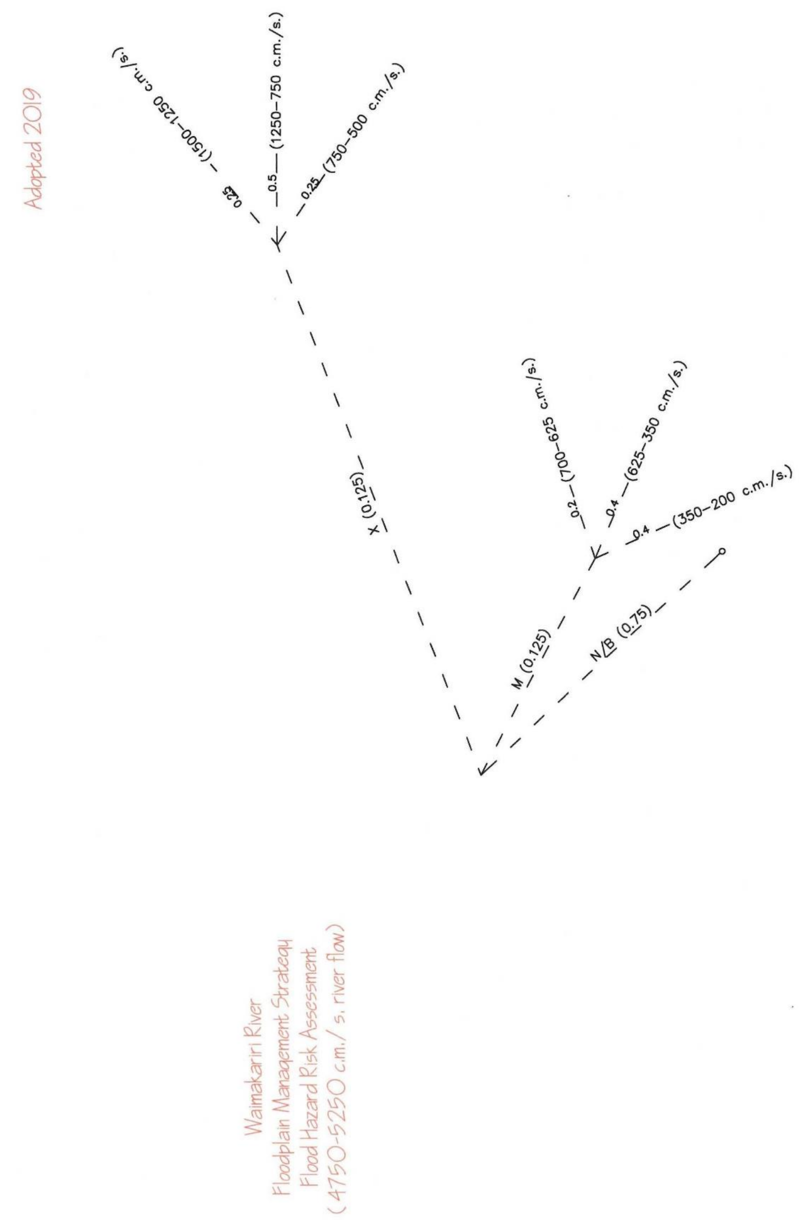


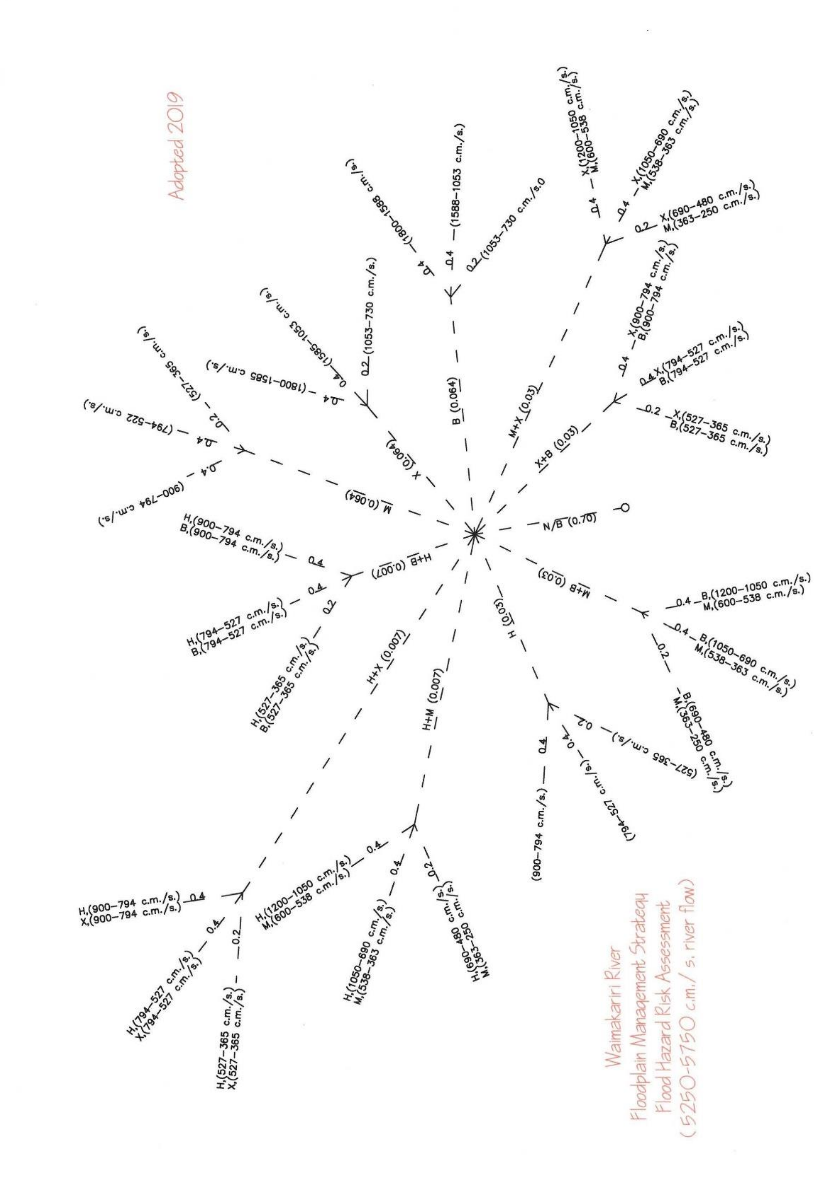
Adopted 2019
0.3 — (450–314 c.m./s.)
0.5 — (314–183 c.m./s.)
0.2 — (183–150 c.m./s.)
X (0.02)
(200–166 c.m./s.)
(166–134 c.m./s.)
(134–100 c.m./s.)
M (0.02)
N/B (0.96)
Waimakariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(2750-3300 c.m./ s.)

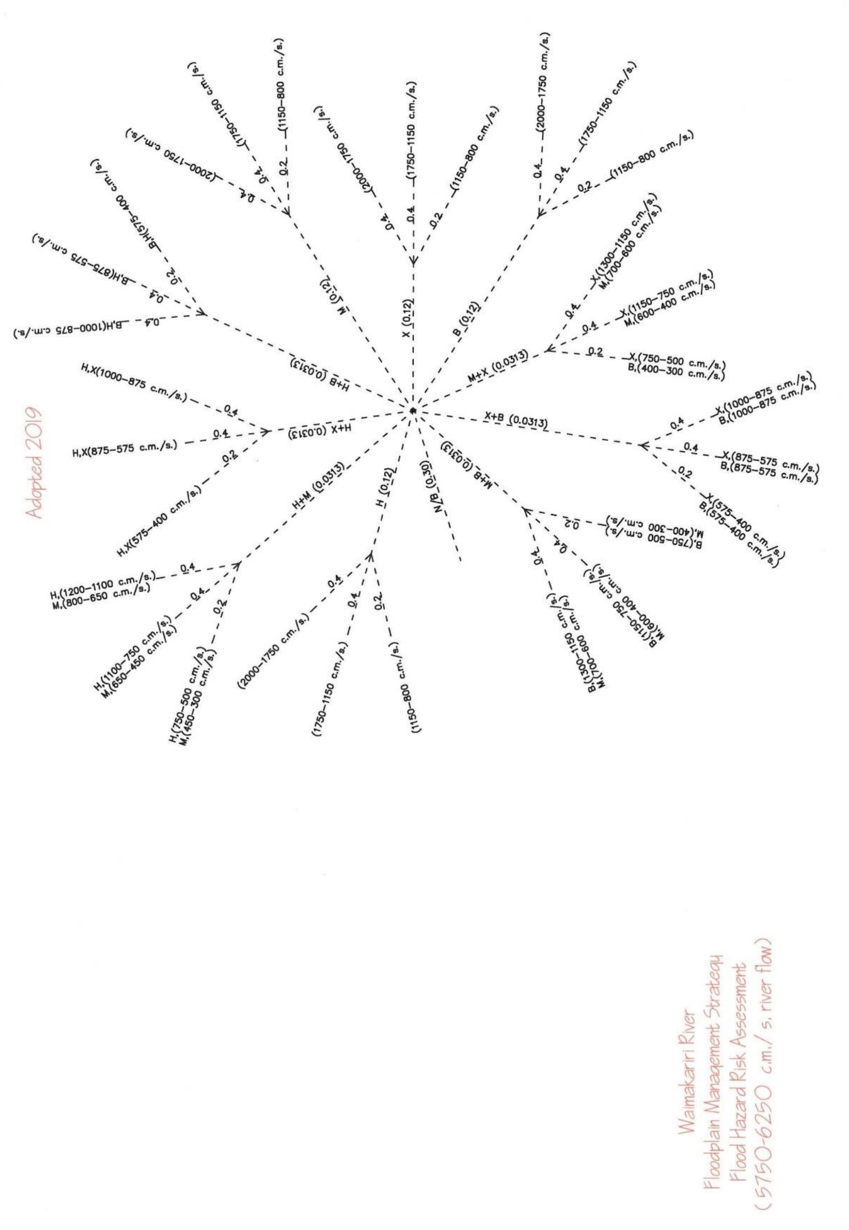

Adopted 2019
0.3 (450–314 c.m./s.)
0.5 (314–183 c.m./s.)
0.2 (183–50 c.m./s.)
X (0.075)
0.3 (200–166 c.m./s.)
0.3 (166–134 c.m./s.)
0.4 (134–100 c.m./s.)
M (0.075)
N/B (0.90)
Waimakariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(3300-3750 c.m./ s. river flow)

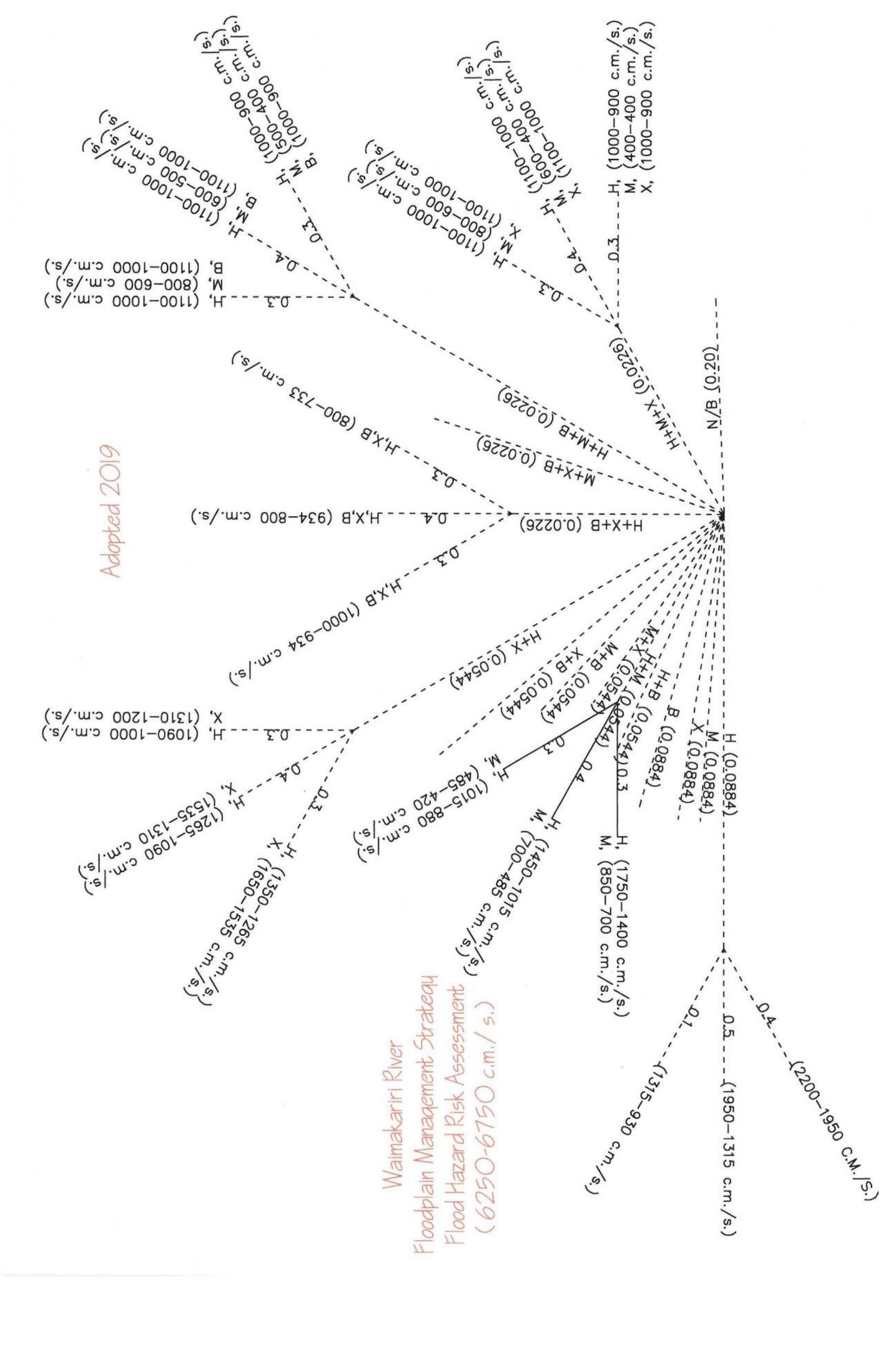

Adopted 2019
Waimakariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(3750-4250 c.m./ s. river flow)
X (0.075)
M (0.075)
N/B (0.85)
(350–275 c.m./s.)
0.35
(275–150 c.m./s.)
0.4
(150–100 c.m./s.)
0.18
(750–625 c.m./s.)
0.55
(625–338 c.m./s.)
0.27
(338–175 c.m./s.)
0.2
(300–260 c.m./s.)
0.4
(260–150 c.m./s.)
0.4
(150–75 c.m./s.)

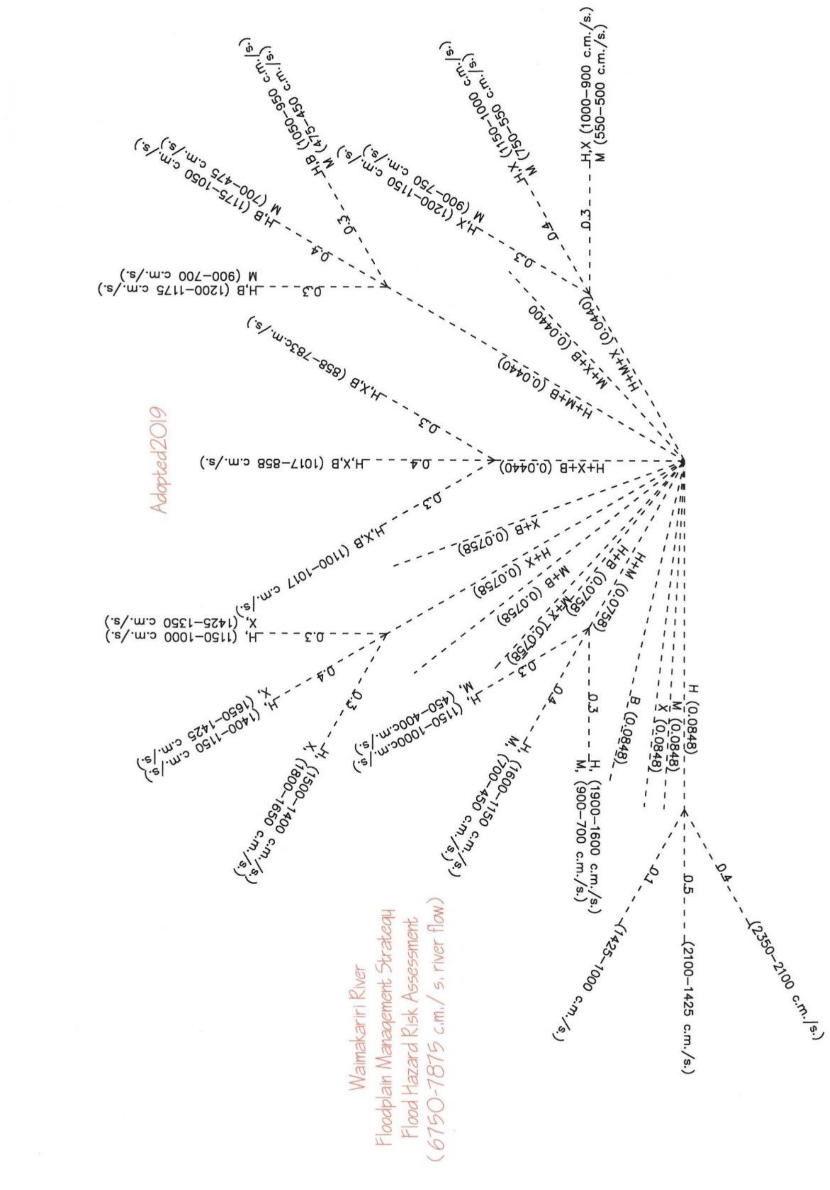

Waimakariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(4250-4750 c.m./ s. river flow)
Adopted 2019
X (0.10)
0.2 (1200–975 c.m./s.)
0.55 (975–550 c.m./s.)
0.25 (550–350 c.m./s.)
M (0.10)
0.2 (500–400 c.m./s.)
0.4 (400–213 c.m./s.)
0.4 (213–125 c.m./s.)
N/B (0.80)

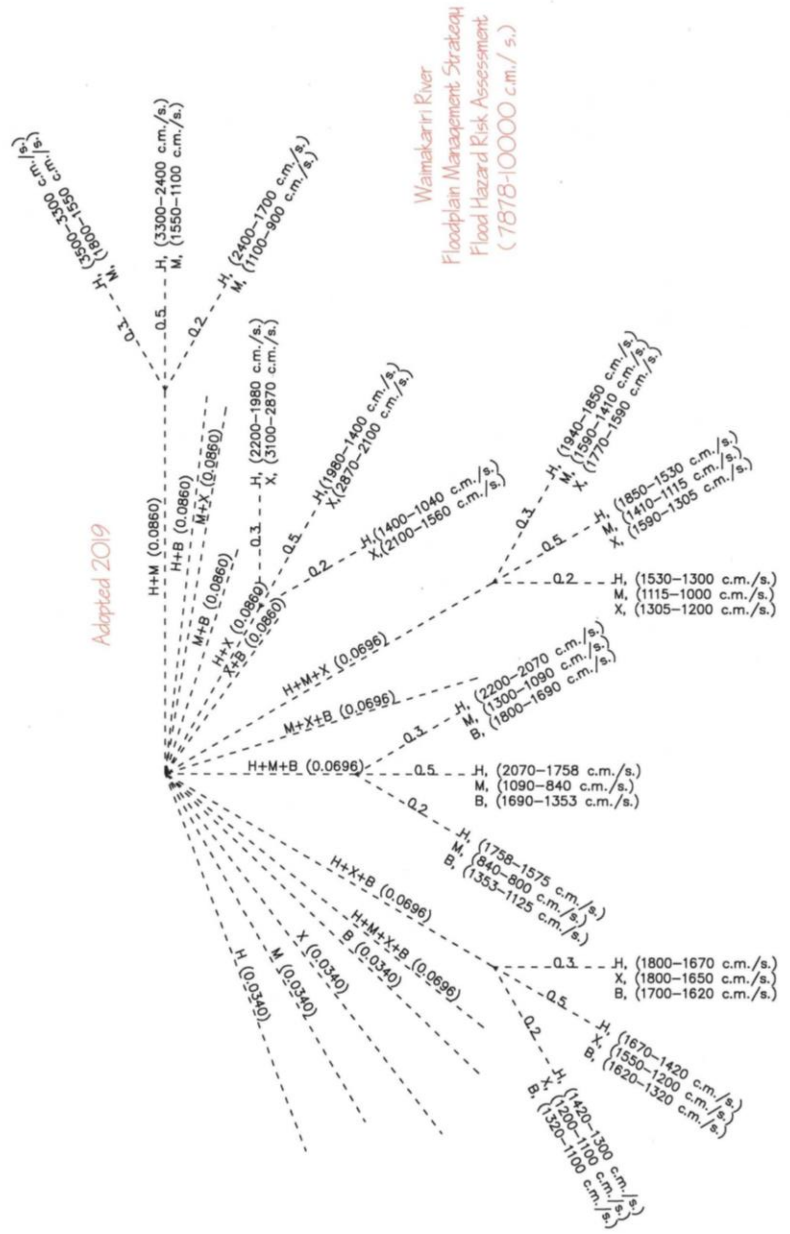

Adopted 2019
0.25 (1500–1250 c.m./s.)
0.5 (1250–750 c.m./s.)
0.25 (750–500 c.m./s.)
X (0.125)
0.2 (700–625 c.m./s.)
0.4 (625–350 c.m./s.)
0.4 (350–200 c.m./s.)
M (0.125)
N/B (0.75)
Waimakariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(4750-5250 c.m./ s. river flow)

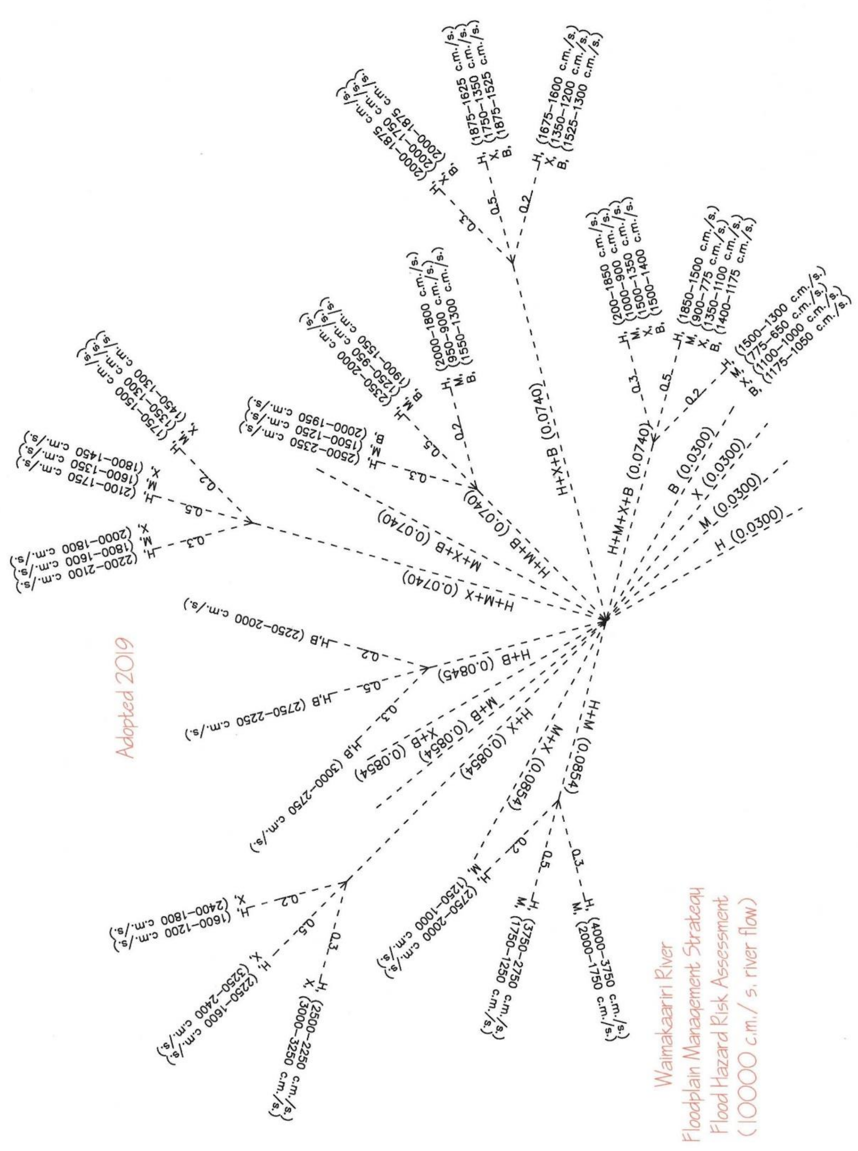

Adopted 2019
Waimakariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(5250-5750 c.m./s. river flow)


Adopted 2019
H (0.12)
M (0.12)
X (0.12)
B (0.12)
N/B (0.30)
H+X (0.0313)
H+M (0.0313)
H+B (0.0313)
M+X (0.0313)
M+B (0.0313)
X+B (0.0313)
Waimakariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(5750-6250 c.m./ s. river flow)


Waimakariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(6250-6750 c.m./ s.)
Adopted 2019
H (0.0884)
M (0.0884)
X (0.0884)
B (0.0884)
H+B (0.0544)
H+M (0.0544)
M+X (0.0544)
M+B (0.0544)
X+B (0.0544)
H+X (0.0544)
H+X+B (0.0226)
M+X+B (0.0226)
H+M+B (0.0226)
H+M+X (0.0226)
N/B (0.20)
0.5 (1950–1315 c.m./s.)
0.1 (1315–930 c.m./s.)
0.4 (2200–1950 C.M./S.)
H, (1750–1400 c.m./s.)
M, (850–700 c.m./s.)
H, (1450–1015 c.m./s.)
M, (700–485 c.m./s.)
H, (1015–880 c.m./s.)
M, (485–420 c.m./s.)
H, (1350–1265 c.m./s.)
X, (1650–1535 c.m./s.)
H, (1265–1090 c.m./s.)
X, (1535–1310 c.m./s.)
H, (1090–1000 c.m./s.)
X, (1310–1200 c.m./s.)
H,X,B (1000–934 c.m./s.)
H,X,B (934–800 c.m./s.)
H,X,B (800–733 c.m./s.)
H, (1100–1000 c.m./s.)
M, (800–600 c.m./s.)
B, (1100–1000 c.m./s.)
H, (1100–1000 c.m./s.)
M, (600–500 c.m./s.)
B, (1100–1000 c.m./s.)
H, (1000–900 c.m./s.)
M, (500–400 c.m./s.)
B, (1000–900 c.m./s.)
H, (1100–1000 c.m./s.)
M, (800–600 c.m./s.)
X, (1100–1000 c.m./s.)
H, (1100–1000 c.m./s.)
M, (600–400 c.m./s.)
X, (1100–1000 c.m./s.)
H, (1000–900 c.m./s.)
M, (400–400 c.m./s.)
X, (1000–900 c.m./s.)


Waimakariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(6750-7875 c.m./ s. river flow)
Adopted2019
(2350–2100 c.m./s.)
(2100–1425 c.m./s.)
(1425–1000 c.m./s.)
H (0.0848)
M (0.0848)
X (0.0848)
B (0.0848)
H, (1900–1600 c.m./s.)
M, (900–700 c.m./s.)
H, (1600–1150 c.m./s.)
M, (700–450 c.m./s.)
H, (1150–1000c.m./s.)
M, (450–400c.m./s.)
H+M (0.0758)
H+B (0.0758)
M+X (0.0758)
M+B (0.0758)
H+X (0.0758)
X+B (0.0758)
H, (1500–1400 c.m./s.)
X, (1800–1650 c.m./s.)
H, (1400–1150 c.m./s.)
X, (1650–1425 c.m./s.)
H, (1150–1000 c.m./s.)
X, (1425–1350 c.m./s.)
H,X,B (1100–1017 c.m./s.)
H,X,B (1017–858 c.m./s.)
H,X,B (858–783c.m./s.)
H+X+B (0.0440)
H+M+B (0.0440)
M+X+B (0.04400
H+M+X (0.0440)
H,B (1200–1175 c.m./s.)
M (900–700 c.m./s.)
H,B (1175–1050 c.m./s.)
M (700–475 c.m./s.)
H,B (1050–950 c.m./s.)
M (475–450 c.m./s.)
H,X (1200–1150 c.m./s.)
M (900–750 c.m./s.)
H,X (1150–1000 c.m./s.)
M (750–550 c.m./s.)
H,X (1000–900 c.m./s.)
M (550–500 c.m./s.)
0.1
0.3
0.4
0.5


Adopted 2019
Waimakariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(7878-10000 c.m./ s.)
H+M (0.0860)
H+B (0.0860)
M+X (0.0860)
M+B (0.0860)
H+X (0.0860)
X+B (0.0860)
H+M+B (0.0696)
M+X+B (0.0696)
H+M+X (0.0696)
H+X+B (0.0696)
H+M+X+B (0.0696)
H (0.0340)
M (0.0340)
X (0.0340)
B (0.0340)
H, (3500–3300 c.m./s.) M, (1800–1550 c.m./s.)
H, (3300–2400 c.m./s.) M, (1550–1100 c.m./s.)
H, (2400–1700 c.m./s.) M, (1100–900 c.m./s.)
H, (2200–1980 c.m./s.) X, (3100–2870 c.m./s.)
H, (1980–1400 c.m./s.) X, (2870–2100 c.m./s.)
H, (1400–1040 c.m./s.) X, (2100–1560 c.m./s.)
H, (1940–1850 c.m./s.) M, (1590–1410 c.m./s.) X, (1770–1590 c.m./s.)
H, (1850–1530 c.m./s.) M, (1410–1115 c.m./s.) X, (1590–1305 c.m./s.)
H, (1530–1300 c.m./s.) M, (1115–1000 c.m./s.) X, (1305–1200 c.m./s.)
H, (2200–2070 c.m./s.) M, (1300–1090 c.m./s.) B, (1800–1690 c.m./s.)
H, (2070–1758 c.m./s.) M, (1090–840 c.m./s.) B, (1690–1353 c.m./s.)
H, (1758–1575 c.m./s.) M, (840–800 c.m./s.) B, (1353–1125 c.m./s.)
H, (1800–1670 c.m./s.) X, (1800–1650 c.m./s.) B, (1700–1620 c.m./s.)
H, (1670–1420 c.m./s.) X, (1550–1200 c.m./s.) B, (1620–1320 c.m./s.)
H, (1420–1300 c.m./s.) X, (1200–1100 c.m./s.) B, (1320–1100 c.m./s.)
0.3
0.5
0.2


Adopted 2019
Waimakaariri River
Floodplain Management Strategy
Flood Hazard Risk Assessment
(10000 c.m./ s. river flow)
H+M (0.0854)
M+X (0.0854)
H+X (0.0854)
M+B (0.0854)
X+B (0.0854)
H+B (0.0845)
H+M+X (0.0740)
M+X+B (0.0740)
H+M+B (0.0740)
H+X+B (0.0740)
H+M+X+B (0.0740)
B (0.0300)
X (0.0300)
M (0.0300)
H (0.0300)
0.3 H, (4000–3750 c.m./s.) M, (2000–1750 c.m./s.)
0.5 H, (3750–2750 c.m./s.) M, (1750–1250 c.m./s.)
0.2 H, (2750–2000 c.m./s.) M, (1250–1000 c.m./s.)
0.3 H, (2500–2250 c.m./s.) X, (3000–3250 c.m./s.)
0.5 H, (2250–1600 c.m./s.) X, (3250–2400 c.m./s.)
0.2 H, (1600–1200 c.m./s.) X, (2400–1800 c.m./s.)
0.3 H,B (3000–2750 c.m./s.)
0.5 H,B (2750–2250 c.m./s.)
0.2 H,B (2250–2000 c.m./s.)
0.3 H, (2200–2100 c.m./s.) M, (1800–1600 c.m./s.) X, (2000–1800 c.m./s.)
0.5 H, (2100–1750 c.m./s.) M, (1600–1350 c.m./s.) X, (1800–1450 c.m./s.)
0.2 H, (1750–1500 c.m./s.) M, (1350–1300 c.m./s.) X, (1450–1300 c.m./s.)
0.3 H, (2500–2350 c.m./s.) M, (1500–1250 c.m./s.) B, (2000–1950 c.m./s.)
0.5 H, (2350–2000 c.m./s.) M, (1250–950 c.m./s.) B, (1900–1550 c.m./s.)
0.2 H, (2000–1800 c.m./s.) M, (950–900 c.m./s.) B, (1550–1300 c.m./s.)
0.3 H, (2000–1875 c.m./s.) X, (2000–1750 c.m./s.) B, (2000–1875 c.m./s.)
0.5 H, (1875–1625 c.m./s.) X, (1750–1350 c.m./s.) B, (1875–1525 c.m./s.)
0.2 H, (1675–1600 c.m./s.) X, (1350–1200 c.m./s.) B, (1525–1300 c.m./s.)
0.3 H, (200–1850 c.m./s.) M, (1000–900 c.m./s.) X, (1500–1350 c.m./s.) B, (1500–1400 c.m./s.)
0.5 H, (1850–1500 c.m./s.) M, (900–775 c.m./s.) X, (1350–1100 c.m./s.) B, (1400–1175 c.m./s.)
0.2 H, (1500–1300 c.m./s.) M, (775–650 c.m./s.) X, (1100–1000 c.m./s.) B, (1175–1050 c.m./s.)