WAIHO RIVER

14 August 2014

Hydraulic Modelling and Analysis

LANDRIVERSEA
PROFESSIONAL ENGINEERING

Report prepared for WCRC

by Matthew Gardner

Land River Sea Consulting Ltd

WAIHO RIVER

HYDRAULIC MODELLING AND ANALYSIS

TABLE OF CONTENTS

1. INTRODUCTION 4

Scope of study 4

Background 4

Available data 6

2. HYDROLOGY 9

Peak flow 9

Hydrograph Shape 10

3. MEAN BED LEVEL ANALYSIS 11

4. MODEL BUILD 15

MIKE 11 Model 15

Network set up 15

Cross section data 18

Resistance 18

MIKE Flood Model 19

Model Terrain 19

Lateral Links 20

Floodplain Resistance 20

5. MODEL VERIFICATION / CALIBRATION 20

6. MODELLED SCENARIOS 22

7. MIKE 11 MODEL RESULTS 24

8. DESIGN LEVELS 24

Stopbank Heights 24

Model Freeboard 24

Cross section plots 26

Channel longsection plot 29

Stopbank Longsection Plots 31

9. MIKE FLOOD MODEL RESULTS 39

Southside Stopbank 39

Risk to Franz Josef Township from Right Bank Overflow upstream of Sh6 42
Channel and Bridge Widening 43

10. DISCUSSIONS 45

11. CONCLUSIONS 46

11. REFERENCES 47

1. INTRODUCTION

SCOPE OF STUDY

The West Coast Regional Council has commissioned Land River Sea Consulting to build a model of the Waiho River based on the 2014 cross section survey data in order to:

- Estimate design flood heights for a theoretical 1 in 100, 200 and 400 year event
- Estimate the flow required to overtop the southside stopbank.
- Investigate the potential for inundation of the Franz Josef township due to overtopping of the right bank upstream of the SH6 Bridge.
- Determine the impacts on in channel water levels by widening the channel between the SH6 Bridge and cross section 16 as well as widening the bridge itself.

The following report outlines the background to the study, the method adopted and presents the results.

BACKGROUND

The Waiho River is located on the West Coast of the South Island of New Zealand, running from the Franz Josef Glacier in the Southern Alps to the Tasman Sea, approximately 10km southwest of Okarito. The river is crossed by the State Highway 6 (SH6) Bridge which is operated by the New Zealand Transport Authority (NZTA) and runs adjacent to the town of Franz Josef / Waiau, situated on the true right bank of the river.

The area has a high level of geologic activity, with the Alpine Fault running through the town of Franz Josef itself and crossing the river in the vicinity of the SH6 Bridge. The river is fed by meltwater from the Franz Josef Glacier in its upper reaches which is currently in a state of retreat (Mills, 2012). The river has been steadily aggrading in recent years and it is possible that the retreating of the glacier will result in an increase in the volumes of sediment entering the river and ultimately increase the rates of aggradation.

The main tributary of the Waiho River within the study area is the Callery River which enters the Waiho River immediately upstream of SH6. Figure 1-1 shows the location of the Waiho River as well as the catchment boundaries which feed the river within the study area.

Figure 1-1 – Location and catchment boundary of the Waiho River and the town of Franz Josef

The river is constrained by stopbanks in the vicinity of the township which are maintained by the regional council on behalf of the Franz Josef Rating District. The 2010 Franz Josef Rating District Asset Management Plan (WCRC, 2010) states that objectives of the Franz Josef Rating District are (WCRC, 2010):

(a) To reduce bank erosion and flooding on the left bank of the Waiho River between the State Highway Bridge and approximately 100 metres below the Franz Josef Holiday Park.

(b) To reduce bank erosion and flooding on the right bank of the Waiho River from the State Highway Bridge downstream for a distance of approximately 600 metres.

The rating district maintains stopbanks on both the left and right banks of the river. The 2010 Asset Management plan states "The Franz Josef protection works extend downstream from the State highway Bridge a distance of 450 metres on the left bank and a distance of 550 metres on the right bank. The area protected on the left bank is predominantly commercial property including a Motel complex, Backpackers, Holiday Park – Camping Ground and two private dwellings. The area protected on the right bank includes an historic church site, Franz Josef headquarters of the Department of Conservation, part of Franz Josef Township and helicopter landing sites.

Community infrastructure such as roads, power and telephone lines all derive benefit from the protection works." (WCRC, 2010) Since the publication of this Asset Management Plan, it is understood that the camping ground is no longer situated on the true left bank of the river, however the motel complex and one residential property are still occupied.

In 2011 the right bank stopbank was extended by the Hokitika Airport to provide a greater level of protection to the helipads. Figure 1-2 provides an overview of the stopbanks maintained by the Franz Josef rating district.

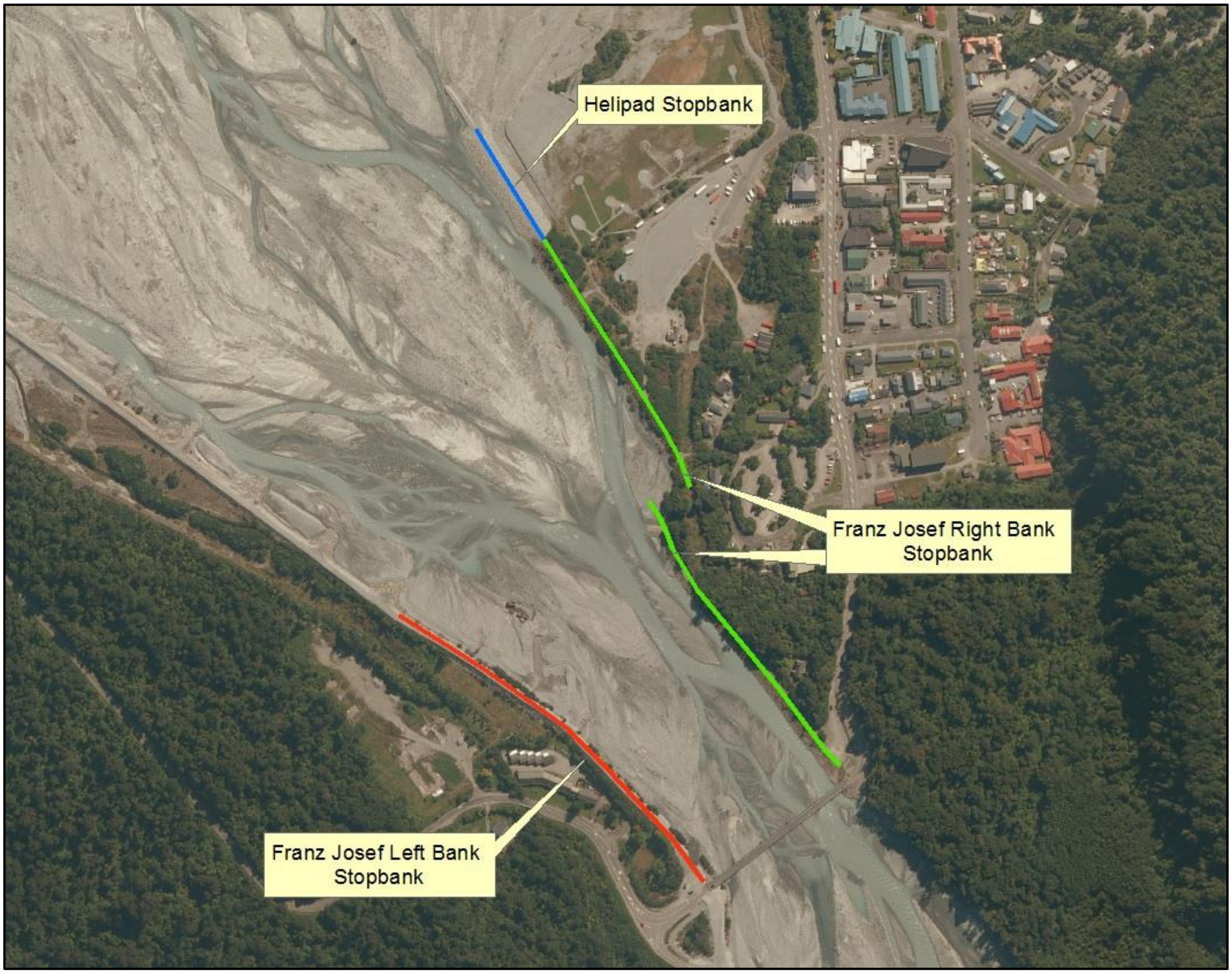


Figure 1-2 – Location of stopbanks in the Franz Josef rating district area

AVAILABLE DATA

The following data was made available for the purpose of this investigation

Cross Section Data

Regular cross section surveys are carried out by Chris J Coll Surveyors on behalf of WCRC. The most recent survey was carried out in February 2014 starting at cross section 10 at the confluence with the Callery River and going down to past the end of the terminal morraine. Surveys prior to 2008 extended approximately 5km further upstream than this up to cross section 1. All levels provided by Chris J Coll Surveying Ltd are in terms of the MSL Lyttleton Datum.

Regular monitoring of the bed levels immediately upstream of the SH6 bridge is also conducted by NIWA, who are contracted by OPUS International Consultants on behalf of NZTA. Cross sections are surveyed on average three times a year with the most recent survey taken in March 2014. A local datum has been used for this survey, in order to convert to the MSL Lyttleton datum it is necessary to add 1.44 m to these levels (Hall, 2012). Figure 1-3 shows the location of the cross section survey data utilised in this investigation.

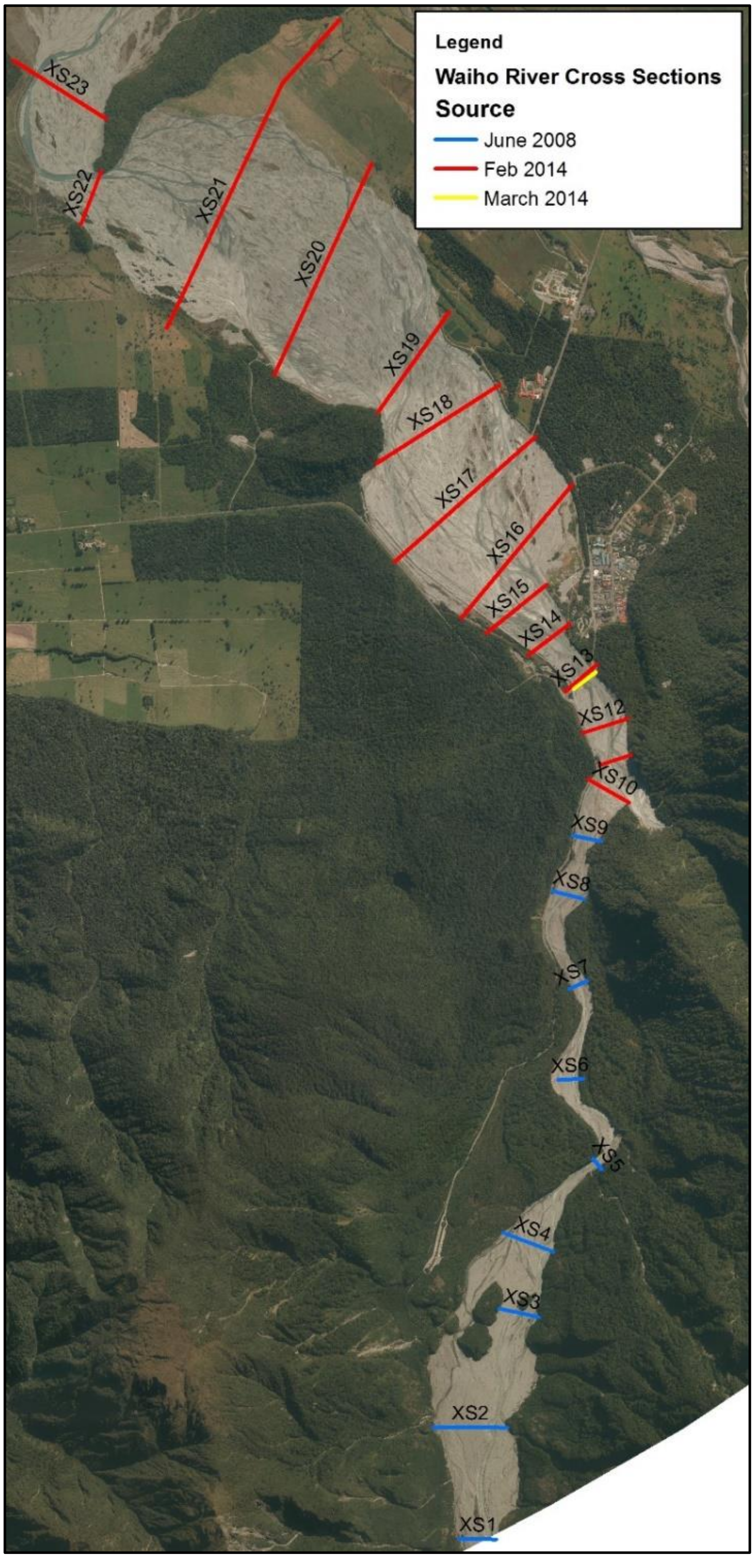


Figure 1-3 – Most recent available survey data on the Waiho River

2. HYDROLOGY

PEAK FLOW

The Waiho River has a water level gauge located at the SH6 bridge, however, due to the highly mobile nature of the bed it is not possible to accurately determine the flow of the river based on traditional gauging methods, and therefore no reliable flow information is available for this site. Inflow hydrology assumptions have therefore been made based on analysis of at-site records for the Whataroa River at the SH6 Bridge. The Whataroa catchment is considered to be appropriate due to its similar catchment characteristics and near vicinity to the Waiho Catchment. This assumption has also been adopted by NIWA (Measures, 2011) as well as Bob Hall (Hall, 2012) although both have used different methods for scaling the flows from the Whataroa catchment to the Waiho. Estimated flows for the 20, 50, 100, 200 and 500 year return period events were provided by WCRC based on data up to the end of 2013 based on an EV1 analysis (Beaumont per coms, 2014). An estimated 400 year flood peak was then interpolated from this data using an EV1 relationship.

When scaling flows from another catchment of different area, based on guidelines set out in Flood Frequency in New Zealand (McKerchar & Pearson, 1989), common practice within NZ has been to assume that flood peak magnitude (Q_p) is proportional to catchment area (A) raised to the power of 0.8.

$$\frac{Q_1}{Q_2} = (\frac{A_1}{A_2})^{0.8}$$

More recent work by George Griffiths of NIWA demonstrates using dimensional analysis as well as statistical evidence, that flood peak magnitude is proportional to catchment area raised to the power of 0.75 (Griffiths, 2008).

$$\frac{Q_1}{Q_2} = (\frac{A_1}{A_2})^{0.75}$$

Table 3-1 presents the estimated peak flows for the Waiho River at the SH6 Bridge based on both of the relationships presented above.

Table 3-1 – Estimated peak flows for a range of return period events on the Waiho River at SH6 Bridge.

Return Period (years)	Flow – based on Q_p ~ $A^{0.8}$ (m^3/s)	Flow - based on Q_p ~ $A^{0.75}$ (m^3/s)
20	1857	1953
50	2128	2238
100	2330	2451
200	2533	2664
400	2735	2876
500	2800	2945

For the purposes of this study the higher (more conservative) values have been selected, however have been rounded to 2 significant figures to reflect the uncertainty in their value. Table 3-2 presents the final adopted peak flows.

Table 3-2 – Adopted peak flows

Return Period (years)	Peak Flow (m3/s)
100	2500
200	2700
400	2900

It should be noted that these flow statistics appears to differ significantly from the flows used by NIWA staff in their 2011 study (Measures & Duncan, 2011). NIWA has adopted a significantly lower 100 year flow (1551 m^3/s) than that used in this study. It is unclear how they obtained this value, however it seems that their estimate of a 100 year event in the Whataroa River (4250 m^3/s) differs significantly from that calculated by WCRC (5230 m^3/s). Adopted flows for this study are in the same order of magnitude as those adopted by Good Earth Matters in 2011 (2300 m^3/s) but greater than those adopted by R.J. Hall (1900 m^3/s). It should also be noted that I have adopted a more conservative approach when scaling flows between catchments, based on recent research by George Griffiths of NIWA (Griffiths, 2008).

HYDROGRAPH SHAPE

Due to the fact that there is little storage available in the system the overall shape is likely to have little effect on the overall flood levels.

The shape of the hydrograph has been taken from a consideration of recorded flow hydrographs at the Whataroa @ SH6 site. Figure 3-1 shows five events, normalised to the same peak and similar time of occurrence (Gardner & Wallace, 2013). The 2010 shape has been adopted for this study.

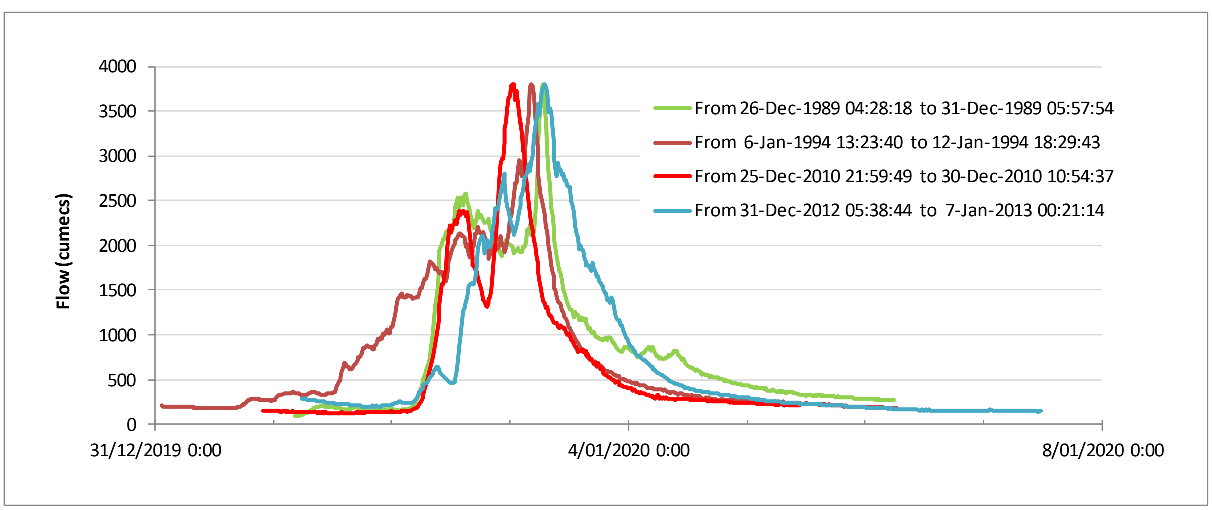


Figure 3-1 - Comparison of normalized hydrographs from a range of flood events in the Whataroa River (from Gardner & Wallace, 2013).

3. MEAN BED LEVEL ANALYSIS

The Waiho River flows in a very active geological area with large amounts of sediment entering the river upstream from the SH6 Bridge. Whilst the scope of this study does not include investigating sediment transport processes in detail, it is considered important to get a feel for potential future increases as it is possible that these will have a greater impact on flood levels than the amount of water entering the river due to rainfall in the long term.

The report, Waiho River – Future Management (Hall, 2012) discusses changes in bed levels prior to 2011 for the entire reach of the river from the Waiho / Callery confluence down to the Waiho Loop and concludes that bed levels are consistently aggrading over the entire reach. Due to the nature of this study focusing on the stopbanks in the vicinity of Franz Josef / Waiau township, I have looked at changes in bed levels in more detail for each individual cross section from the confluence with the Callery River (XS10) down to the start of the helipad stopbank on the true right bank (XS15) from 2008 to see if the aggradation has continued in recent years as well determine recent rates of aggradation. Figure 3-3 shows the location of the monitored cross sections.

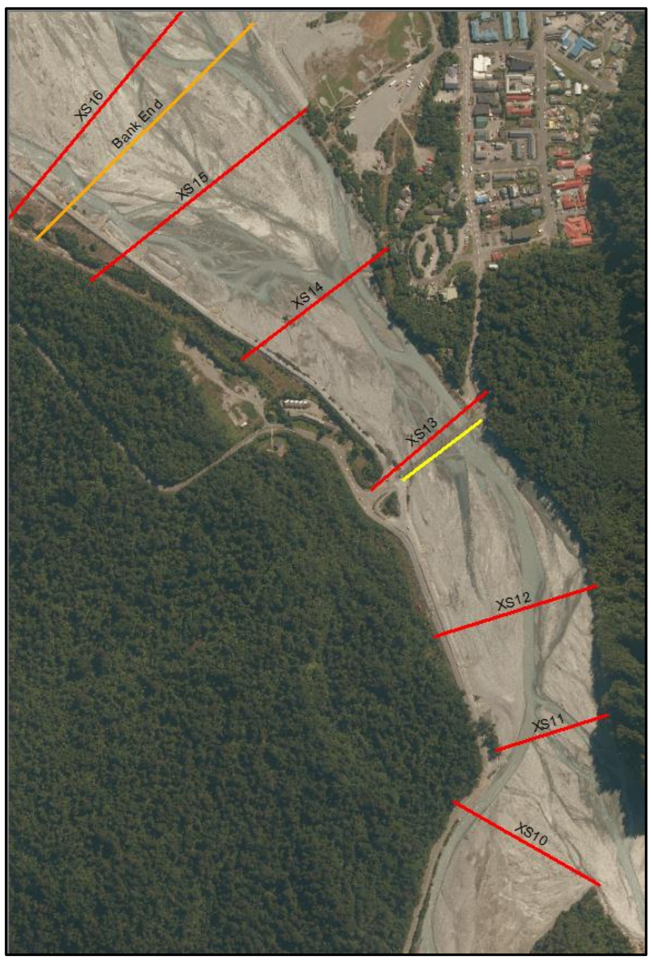
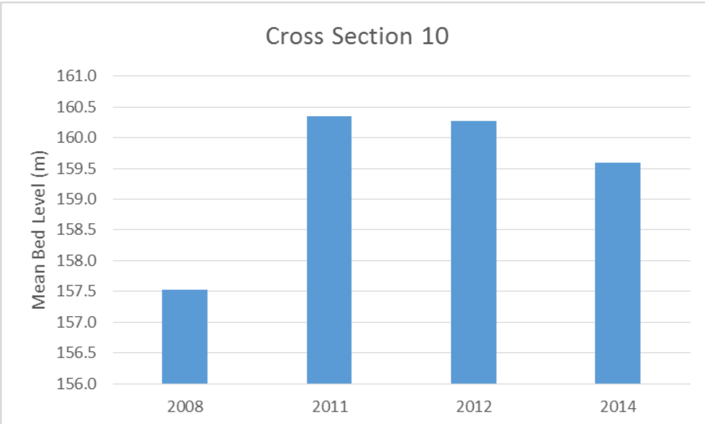
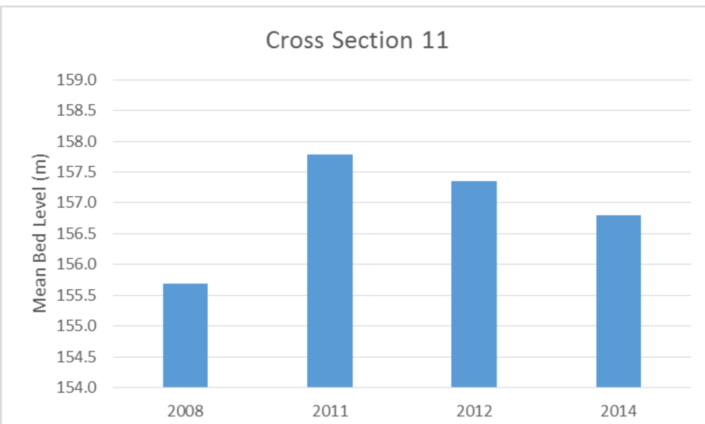


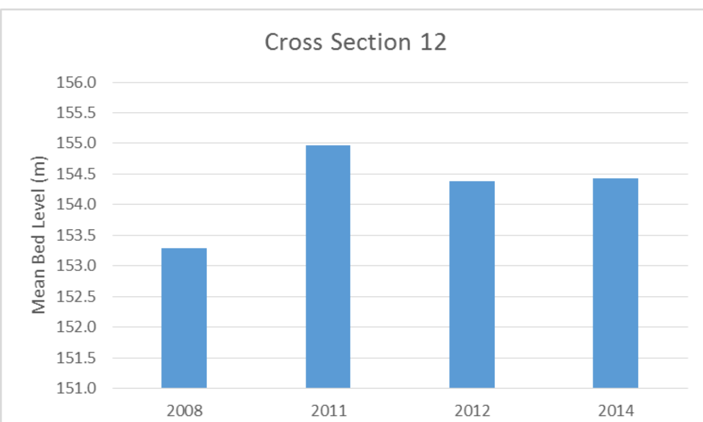
Figure 3-3 – Location of surveyed cross sections used for m.b.l analysis



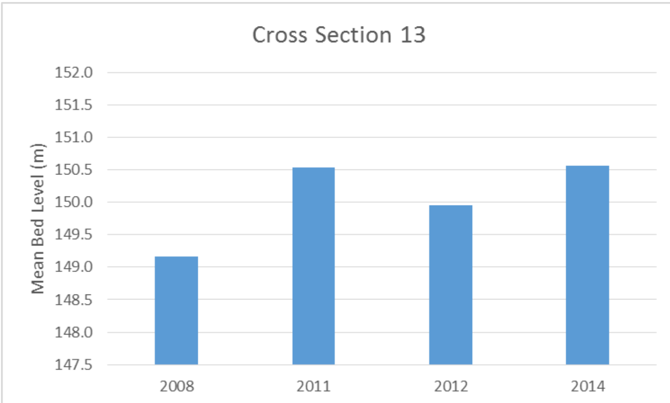
Comparison of surveyed cross sections for XS10 show that the mean bed level (m.b.l) has increased by 2.1 metres since 2008, however the majority of this increase occurred between 2008 and 2011 with slight decreases in m.b.l occurring in between 2011 and 2014. Overall, this equates to an average annual increase of 0.34 m/year.



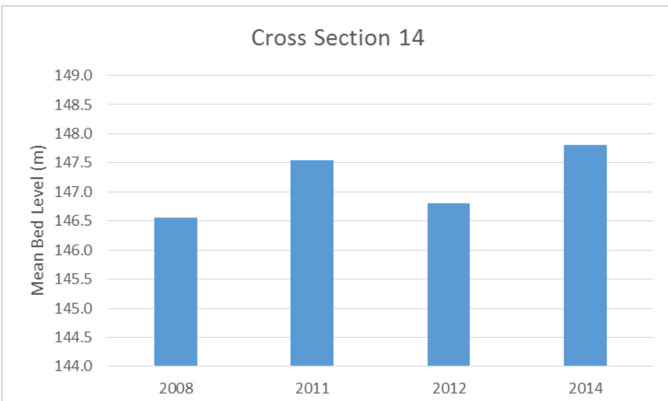
Comparison of surveyed cross sections for XS11 shows that the m.b.l. has increased by 1.1 metres since 2008. The majority of this increase again occurred between 2008 and 2011. The bed level at this location appears to have been steadily degrading since 2011 however is still significantly higher than 2008 levels. Over 6 years this increase equates to an average annual increase of 0.18 m/year.



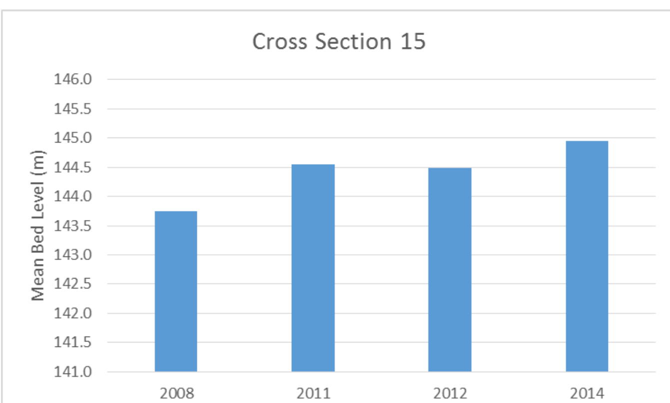
Comparison of surveyed cross sections from 2008 for XS12 shows that the m.b.l. has increased by 1.1 metres since 2008. As was the case with XS10 and 11, the majority of this increase occurred between 2008 and 2011. Over 6 years this increase equates to an average annual increase of 0.18 m/year.



XS 13 is located immediately downstream of the SH6 bridge. Comparison of available cross sections data at this location shows that the m.b.l. has increased by 1.4 metres since 2008. Over 6 years this increase equates to an average annual increase of 0.23 m/year.



Comparison of available cross sections for XS14 shows that the m.b.l. has increased by 1.3 metres since 2008. Again a significant increase between 2008 and 2011 was observed, whilst there was a drop in bed level between the 2011 and 2012 surveys, of concern is that the 2012 and 2014 data sets show an increase in m.b.l of greater than 1 metre in just 2 years. Over 6 years this increase equates to an average annual increase of 0.21 m/year.



Comparison of available cross sections for XS15 shows that the m.b.l. has increased by 1.2 metres since 2008. Bed levels in this location are showing a steadily increasing trend. Over 6 years this increase equates to an average annual increase of 0.21 m/year.

3 yearly cross sections are also surveyed by NIWA, immediately upstream of the SH6 bridge. Calculations of m.b.l for cross sections back to 2004 have been obtained and are presented in Figure 3-4.

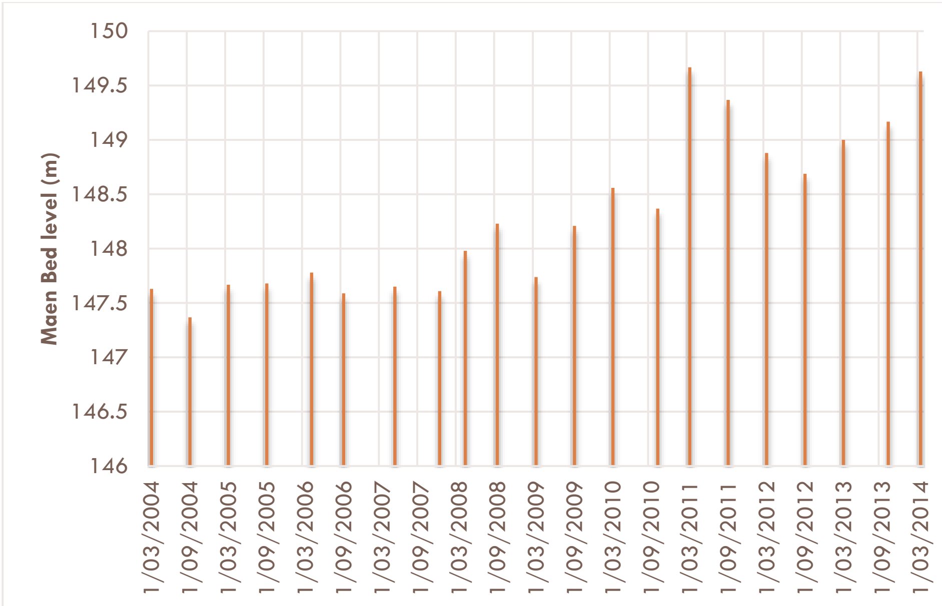


Figure 3-4 – Comparison of m.b.l for surveyed cross sections at the SH6 Bridge on the Waiho River between 2004 and 2014 (NIWA per coms, 2014)

It can be seen from this figure that m.b.l were fairly consistent from 2004 and 2007, however a significant increasing trend is visible from 2008 onwards. Over the 10 years since 2004, the m.b.l has increased by approximately 2 metres at the bridge, equating to an average increase of approximately 0.2 metres per year.

A literature review has shown that aggrading bed levels are not a new phenomenon in the Waiho River. A 2002 report states that bed levels at the SH6 Bridge have increased in the order of 10 metres since the 1940's (Optimix, 2002). This gives an average rate of increase of between 0.16 and 0.2 metres a year, consistent with recent trends identified above.

It can be concluded from this analysis that increases in bed levels are very significant and need to be taken into account when considering future design. For this modelling an increase in m.b.l of 2 metres has been simulated as a sensitivity run. Based on historic trends of bed level increase, the bed will likely be higher than this in approximately 10 years time.

4. MODEL BUILD

Two hydraulic models were essentially developed for this study, the first model being a 1-dimensional (1D) MIKE 11 model for the determination of design levels for the left and right bank stopbanks. The second model, is a more detailed model incorporating LiDAR data, which was made available later in the investigation and allows the flow paths outside of the main channel to be identified. The development on the two models is set out in the following section.

MIKE 11 MODEL

A hydraulic model of the Waiho River has been set up using MIKE 11 software by DHI. MIKE 11 is an industry standard package and is used widely both in New Zealand and internationally for the modelling of rivers and streams. MIKE 11 solves the 1D Saint-Venant equations to determine flow characteristics.

A typical MIKE 11 model consists of four major inputs, which are

1. Description of the model network including any structures
2. Cross section data describing the bed level of the river
3. Definition of applicable boundary conditions, primarily inflows and downstream conditions
4. Parameters which represent the roughness of the river bed as well as initial water conditions and a range of other computational parameters

The model set up is detailed as follows.

NETWORK SET UP

The model has been built so that it can estimate water levels from the confluence with the Callery River (XS10) to just upstream from the Hokitika Airport stopbank on the true right bank of the river. This reach of the river is well confined by the banks and the main flow of the river will run perpendicular to the banks, which is an essential assumption made when using the 1D St Venant flow equations, utilized by MIKE11. Downstream from this location (XS16), the river fans out, increasing in width in the order of three times from its width at XS15. The surveyed cross sections are no longer parallel to the direction of the main flow and the 1D St Venant Equations are not likely to be as reliable for determining the water level.

The focus of this investigation is on water levels between cross sections 10 to the end of the helipad stopbank (approximately in between cross sections 15 and 16), however in order to ensure that the boundary conditions do not have an adverse impact on the water levels within this reach, the model has been extended up to cross section 7 using survey data from 2008. To ensure that the downstream boundary water level does not adversely impact on the water levels in the study reach the model has been continued down to cross section 23. Due to the fact that the cross sections are not aligned parallel with the likely flow paths and that the difference in levels between the left and right banks are in the order of 10 metres in some locations, it was not considered appropriate to have only one main channel downstream from cross section 16. The channel has therefore been split into two channels allowing the main channel to fill up and then spill into the secondary channel. Whilst this is not a precise technique for determining downstream water levels, it is considered reasonable for ensuring that the levels upstream of this location are not adversely influenced by the water levels being too high or too low at cross section 16. Figure 4-1 shows an example of how the model set up simulates water levels in the main channel. The water first fills the primary channel (shown full of water) and once this channel reaches the peak of the right bank, it spills into the secondary channel. If this was modelled as one main channel, the section would fill from the lowest point first, which is 6 m lower than the main channel causing the water levels in the study reach to be lower than would occur in reality.

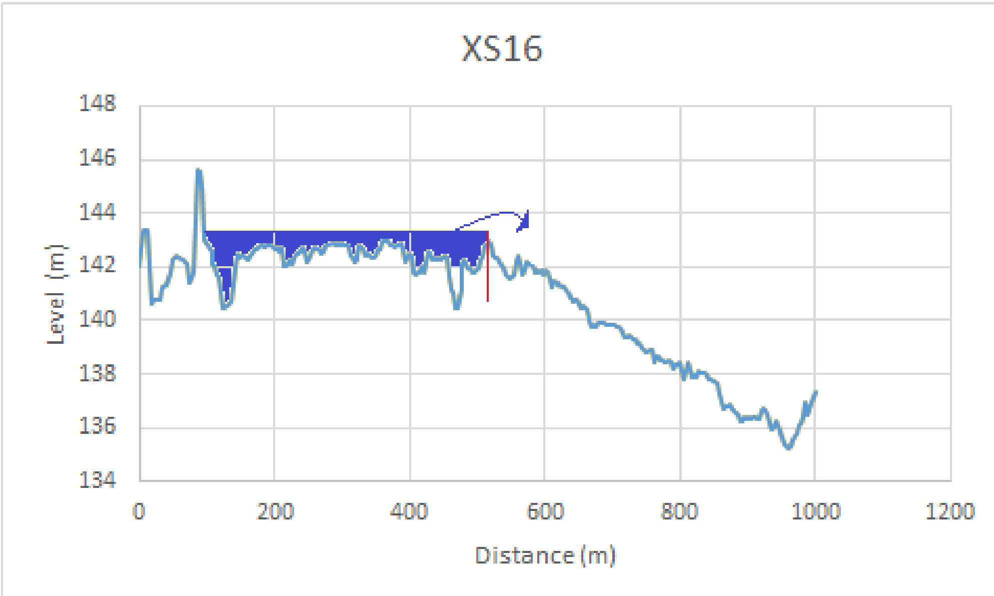


Figure 4-1 – Example of primary channel filling and spilling into the secondary channel at cross section 16

Sensitivity tests to determine how much of an influence water levels at cross section 16 were likely to have on water levels in the study reach were carried out. Sensitivity was tested by increasing the bed level at section 16. A simulation with the bed level raised by 1 metre at cross section 16, raised water levels by 0.61 metres at XS16. Due to the steep nature of the river, this equated to a water level increase of 0.01 m at XS15 and and increase of 0.17 at the end of the helipad bank.

LiDAR was obtained from GNS Science which had been flown around 2010 in order to , an attempt was made to join the model to a 2D boundary, however due to the significant difference in bed levels between the data sets, the results were no better than the existing model.

Considering the steep nature of the river, the existing model is considered to be appropriate for the purpose of this investigation, water levels downstream from XS15 should be interpreted with caution.

Bridge Set Up

Bridge details were obtained from 2011 drawings provided by Opus International Consultants on behalf of NZTA. The drawings show 6 piers in the river each having a width of approximately 0.5 m with a total opening width 158.5 m. This gives a blockage ratio of 0.02. Figure 4-2 shows a picture of a typical bridge pier.

The bridge has been modelled using the MIKE 11 bridge module using the Energy Method. This method is considered suitable in this case due to its simplicity as well the fact that there is no calibration information available. Bridge submergence and overflow have been simulated using the FHWA WSPRO routines.

The level of the soffit was not immediately clear from the supplied drawings however was supplied by John Porteous from NIWA (Porteous per coms, 2014). The provided bridge soffit level of 155.9 m in terms of the msl Lyttleton datum has been used in the model.

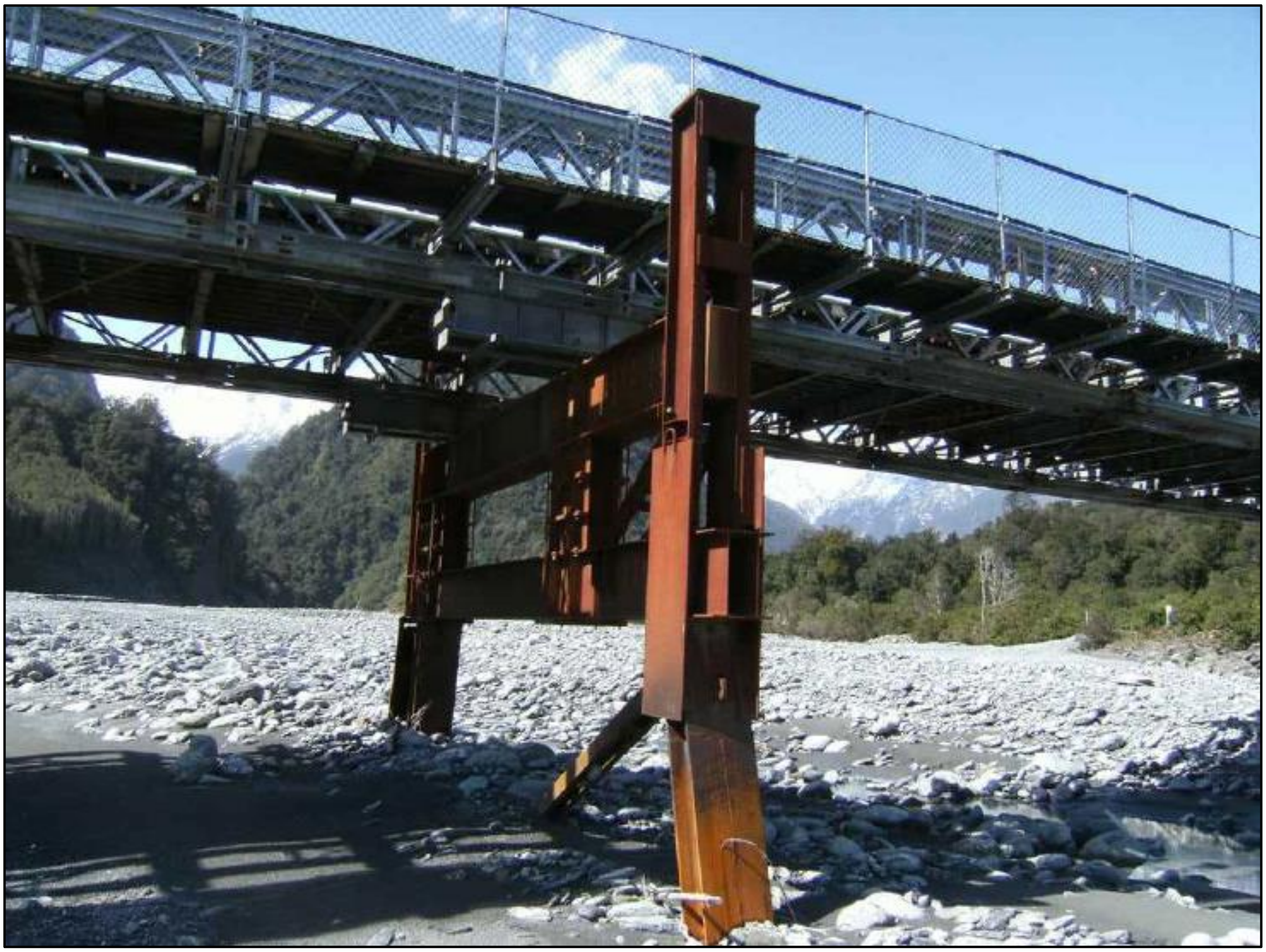


Figure 4-2 - Photo of a typical bridge pier (Opus, 2011)

Considering the forested upstream catchment, the risk of large volumes of woody debris coming down the river in a flood event is considered high in this location. The pier blockage ratio of the main channel has been increased by 20% to 0.22 to allow for debris buildup on the piers, and the soffit level has been reduced by 1m to 154.9 allow for a buildup of woody debris beneath the soffit.

During a site visit on the 26th of June 2014, several large logs were visible near the bridge piers. It is considered likely that during a significant storm event, a large amount of woody debris would be carried down the river creating a high probability for blockage.

CROSS SECTION DATA

The following survey data sets were used for the design runs.

- Cross section data from the 2008 survey was used for cross sections 7, 8 and 9.
- Cross section data from the 2014 survey was used for cross sections 10 to 15.
- The upstream bridge section uses data surveyed by NIWA in March 2014
- A cross section was interpolated at the end on the helipad stopbank. Cross section widths were manually adjusted to ensure it aligned with what was shown in the aerial photography.
- Cross section data from 2014 was used downstream from cross section 16.

RESISTANCE

Previous modelling of the Waiho River carried out by Good Earth Matters has adopted a Manning's 'n' value of 0.037 for this reach of the river based on site observations and comparison with the Roughness Characteristics of New Zealand Rivers (GEM, 2008). Bob Hall comments in his recent study "Waiho River – Future Management" (Hall, 2012) that in his opinion this coefficient is too low, he opines that a value of between 0.05 and 0.06 would be more appropriate based on his experience and engineering judgment.

In 2011, NIWA carried out an assessment into the effects of a landslide dam failure in the Callery River (Measures & Duncan, 2011). They have adopted a Manning's 'n' value of 0.05 in the Waiho River at the SH6 bridge location. They have also stated that this assessment in based on an onsite comparison with examples in 'Roughness Characteristics of New Zealand Rivers' (Hicks & Mason, 1998). They also note that this value is consistent with a check based on observed flows and velocities at the SH6 bridge site carried out by Graeme Smart in the early 1990's. (Smart, 1991).

A further check of these adopted values was made using the two 'rigid bed' formula suggested by Griffiths (Griffiths, 1981). These formula relate Manning's 'n' to the hydraulic radius (R) and the mean particle size (d_{50}). An estimate of the d_{50} was obtained from the report "Anthropic aggradation of the Waiho River, Westland, New Zealand: Microscale modelling" (Davies et al, 2002). The formulae were applied at cross sections 12 and 14, with the results summarised in Table 4-1 below.

Table 4-1 - calculated Manning's 'n' values at select locations based on the Griffiths formulae.

Location	R	d_{50}	Griffiths Formula 1	Griffiths Formula 2
XS12	5.12	0.2	0.044	0.042
XS14	2.358	0.2	0.048	0.045

These values also suggest that a value of 0.05 is in the right range.

A Manning's 'n' value of 0.05 has been adopted for this model giving particularwith a sensitivity test using a value of 0.06 also being tested.

MIKE FLOOD MODEL

A MIKE Flood model incorporating the 1D, MIKE 11 model build detailed above as a well as 2D, MIKE 21 model which has incorporated LIDAR data made available by GNS science, has been constructed.

The MIKE Flood model allows water levels and flows from the MIKE 11 model to be dynamically linked to the MIKE 21 model allowing for the mapping of detailed flow paths outside of the main river channel.

The set up of the MIKE Flood model is as follows:

MODEL TERRAIN

LiDAR data, which was flown for the purposes of mapping the main southern fault line was obtained from GNS Science. The extent of the LIDAR was limited and covers an approximately 1.5km width across the faultine. The extent of the LiDAR data is shown in Figure 4-3.

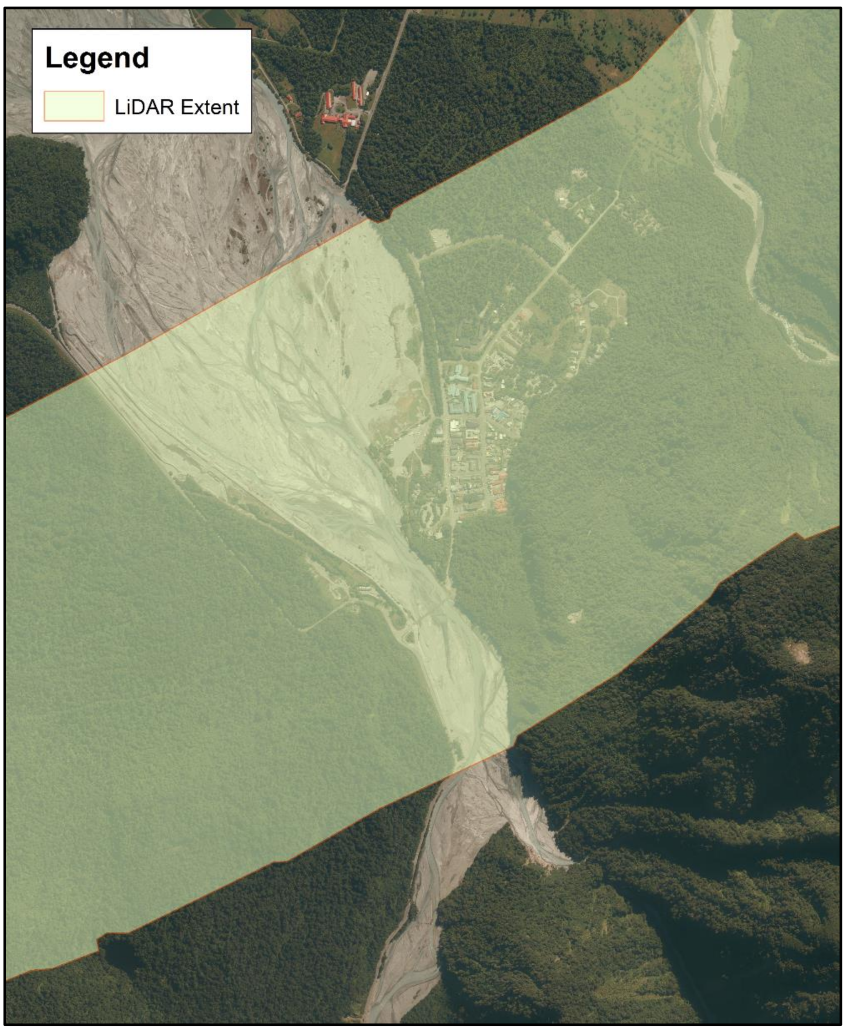


Figure 4-3 – Extent of LiDAR data made available by GNS science

This data was converted to a 5m grid using ArcGIS. The grid was then modified to include the latest surveyed levels for the stopbanks.

LATERAL LINKS

The MIKE 11 model has been linked to the MIKE21 model via a series of lateral links. The location of these links has been defined as the top of the stop banks on both the left and right banks.

FLOODPLAIN RESISTANCE

Floodplain resistance has been applied as a Manning's 'n' value. Table 4-2 outlines the adopted values for this model.

Table 4-2 – Adopted floodplain resistance values

Land Type	Manning's 'n'
Vegetation	0.1
Residential Housing	0.125
Roads	0.015
River Bed	0.05

5. MODEL VERIFICATION / CALIBRATION

Due to the lack of flow data and the constantly changing bed levels, it has not been possible to calibrate the model to any known flood events. However, reports of previous flooding have been read and several flood photos have been observed. Of particular use was an online video from January 2013 (Charleston, 2013), showing the river in flood. This event was rated as a 1 in 8 year event in the Whataroa Catchment, however it is likely due to spatial variation in rainfall, that this differed in the Waiho Catchment. This video shows the resistance created by the bridge piers can be clearly seen. There is no blockage visible at the piers in this event and only a small degree of overall headloss is visible at the bridge. Running the model with flows less than a 10 year event without any blockage also shows only a small degree of head loss at the bridge. Figures 5-1 to 5-3 show a range of useful screenshots during a flood event.

Figure 5 -1 - View of Waiho Bridge in flood (view looking downstream)

Figure 5-2 – View showing resistance at Waiho Bridge Pier during flood event

Figure 5-3 – View of Waiho River in flood downstream from SH6 Bridge

Photos of the river in flood were also obtained from the owner of the motel units (Arbuckle per coms, 2014) from a small flood event on the 23 May 2014. Civil Defence staff are reported to have said that this event equated to 75 mm of rainfall in 24 hours. Whilst it is difficult to gauge a precise level from these photos, it can be seen that the water has risen fairly high up the stopbank adjacent to the motel units. Figure 5-4 shows the water level adjacent to the motel units during this event.

Figure 5-4 – Water level adjacent to motel units on the 23rd May 2014

6. MODELLED SCENARIOS

The following table outlines the main scenarios modelled as part of this study.

Table 6-1 – Summary of model simulations used in this study.

Simulation Number	Return Period (years)	Flow	Cross Sections	Manning's 'n'
1	100	2500	Existing	0.05
2	200	2700	Existing	0.05
3	400	2900	Existing	0.05
4	100	2500	Existing	0.06
5	100	2500	Raised by 2m	0.05
6	100	2500	Widened from bridge to XS16	0.05

Run 6 simulates the channel being widened from the SH6 bridge down to cross section 16. It has assumed that the new bridge will increase in width from 150 metres to 200 metres and that the new bridge piers will block the same proportion of the waterway as the existing bridge. It also assumes that a new bank will be positioned along the approximate alignment shown in Figure 6-1 below.

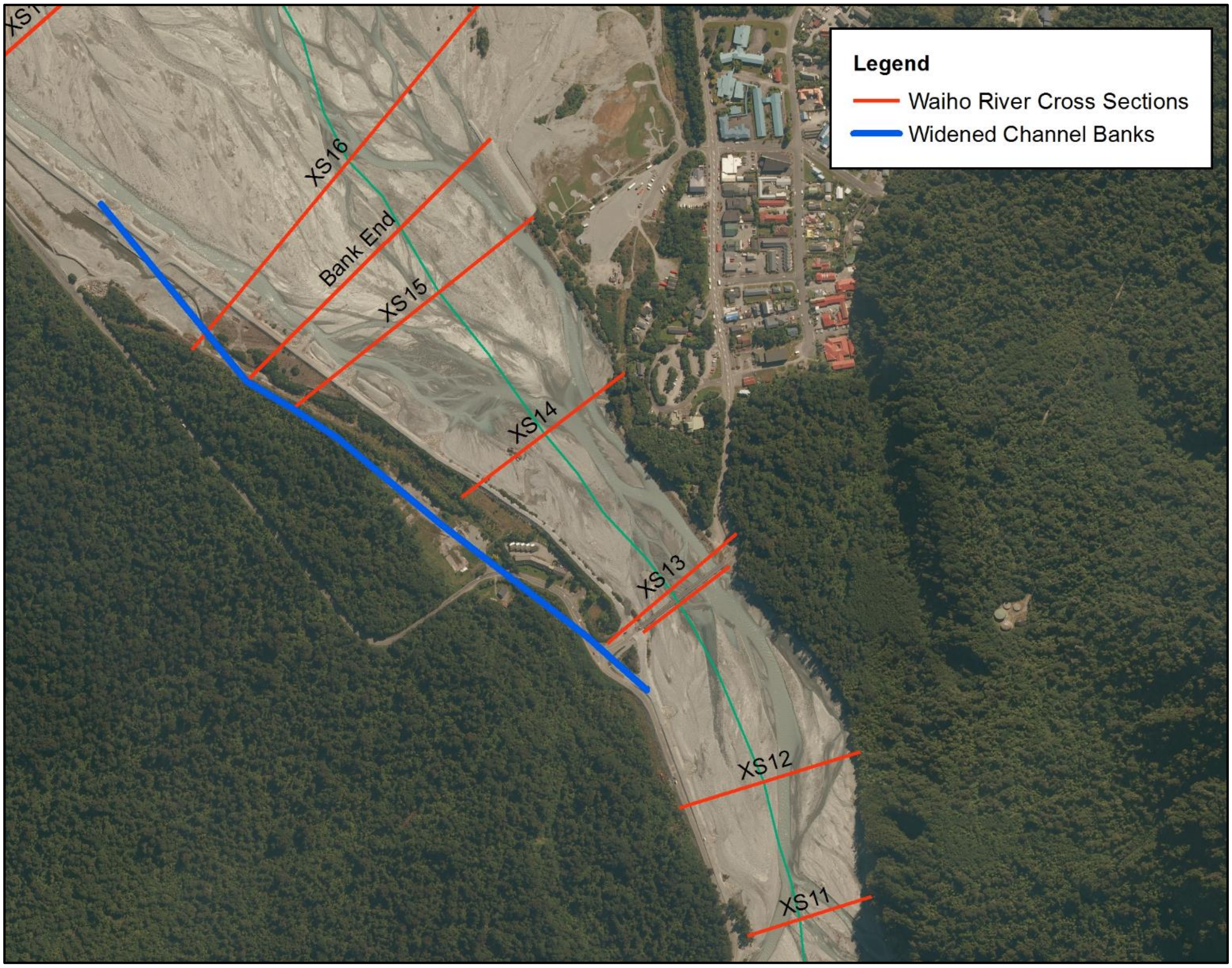


Figure 6-1 – Approximate alignment of widened left bank for run 6

Only conceptual information was provided for this scenario and therefore assumptions in cross section geometry had to be made. I have attempted to be conservative in my assumptions. Further discussions and information would be required if this option is to be explored in more detail in the future.

7. MIKE 11 MODEL RESULTS

'Raw model results without the addition of freeboard are shown below in Table 7-1. The addition of freeboard is discussed and presented in Section 8.

Table 7-1 – Raw model results for scenarios 1-6

	XS11	*XS12*	*Bridge*	*XS13*	*XS14*	*XS15*	*End of Bank*
Run 1	159.8	157.7	156.7	154.1	150.5	147.0	145.6
Run 2	159.9	157.9	156.9	154.2	150.6	147.1	145.7
Run 3	160.1	158.1	157.0	154.4	150.7	147.1	145.7
Run 4	160.1	158.0	156.8	154.4	150.8	147.2	145.8
Run 5	161.7	159.3	157.1	156.1	152.3	148.8	146.9
Run 6	159.7	157.1	155.5	155.3	153.4	149.7	146.5

8. DESIGN LEVELS

STOPBANK HEIGHTS

Design levels presented in this section of the report are the bank levels that would be required to prevent overtopping based on the existing stopbank alignments. Levels are higher than would occur with the existing stopbanks, due to the effective 'glass walling' within the model, preventing water from overtopping the banks and hence allowing design heights to be determined.

MODEL FREEBOARD

In order to decide on an appropriate level of freeboard a range of considerations were taken into account. Freeboard needs to take into account a range of factors including uncertainties in the input data, changing bed levels, wind and wave action etc. Due to the extremely active nature of the Waiho River, the degree of uncertainty in the model results is relatively high. The following uncertainties need to be emphasised in the model results.

Hydrology: There is always a significant amount of uncertainty in any hydrological data, particularly when estimates for a 1 in 100 year event are extrapolated from only 30 years or less of data. On top of this, no flow records within the study catchment were available and flows for this model have been estimated based on a nearby catchment and scaled proportionately, due to the fact that there is no reliable flow gauge in the Waiho River. Rainfall-runoff modelling is not considered appropriate in this catchment due to the complex glacial characteristics within the catchment also.

A sensitivity test increasing the flows by 20% has been carried out and has shown that water levels are likely to increase on average by 0.35 m for a 100 year event between cross sections 10 and 15.

Increases in bed level: The Waiho River bed is highly mobile with large volumes of sediment entering the river sporadically from a range of sources. The rate of input into the river is very difficult to predict due to the complex geological factors in the area. The scope of this study was to build a model based on current bed levels. The river was last surveyed in February 2014. Analysis of mean bed level changes in the study reach have shown that since 2008 bed levels have been increasing on average by 0.23 m a year. If bed levels continues to aggrade at this rate, bed levels will be on average 2.3 metres higher by 2024. Whilst an increase in bed levels by 2.3 metres in 10 years may seem to be very large, an example of actual bed level increases at cross section 14 between 2012 and 2014 is shown below in Figure 8-1.

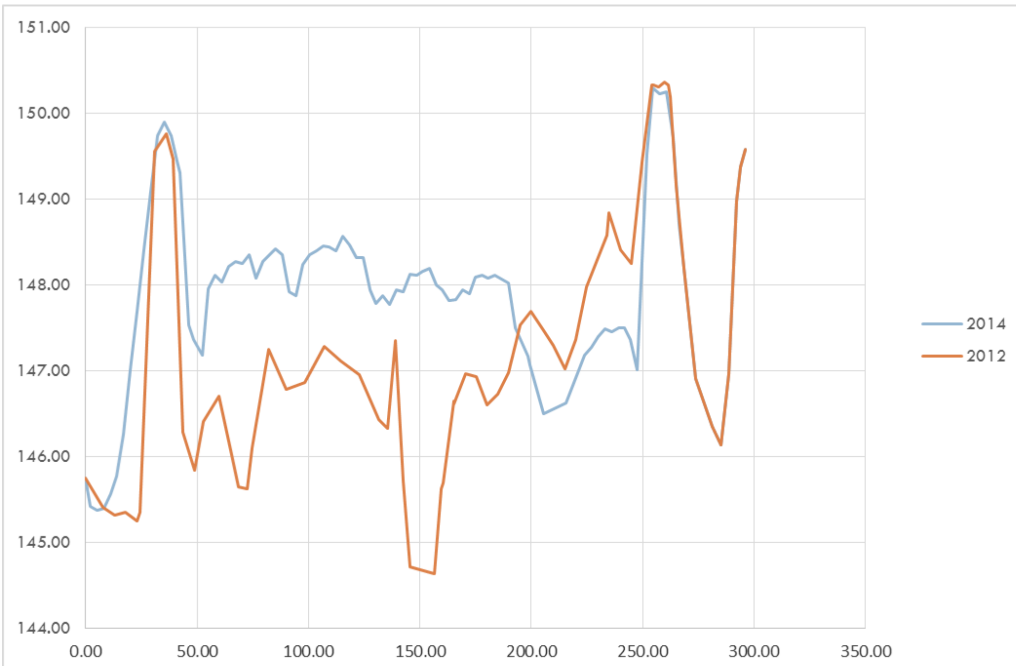


Figure 8-1 – Comparison of bed levels for XS14 between 2012 and 2014

A sensitivity run with bed levels increased by 2m over the entire length of the river has been simulated. Results (run 5) show that within the study reach, water levels increasing by an average of 1.7 m.

Uncertainty in Manning's 'n': A Manning's 'n' value of 0.05 has been selected for this model based on a desktop study. No site visit or physical measurements of particle size distribution were carried out as part of this study. Weighting has been given to the reported investigations (Smart, 1991) which achieved calibration to flow measurements using a Manning's 'n' value of 0.05. It should be noted that calibration to this Manning's 'n' value was achieved in considerably lower flows than those simulated in this study. A large degree of uncertainty in bed roughness due to bed mobility etc needs to be considered when modelling larger flows such as those used in this study. A simulation was therefore carried out using an increased Manning's 'n' value by 20% (ie 0.06). Results showed that an increase in Manning's 'n' by 20% would results in an average increase in water levels by 0.26 m within the study reach.

Adopted level of freeboard: Considering the above mentioned points as well as taking into account other potential unknown uncertainties in the river system, I have adopted a freeboard of 1 metre, keeping consistency with previous modelling studies. It should be noted that this level of freeboard is unlikely to be sufficient to account for long term increases in bed levels based on current trends, however the scope of this study has been to determine levels based on the existing bed levels.

Due to the fact that freeboard has already been partially taken into account with the inclusion of blockage at the bridge, a lower level of freeboard has been applied at the bridge of 0.6m.

CROSS SECTION PLOTS

Design water levels including freeboard for the design reach are plotted over each cross section within the study reach in Figures 8-2 to 8-8 below.

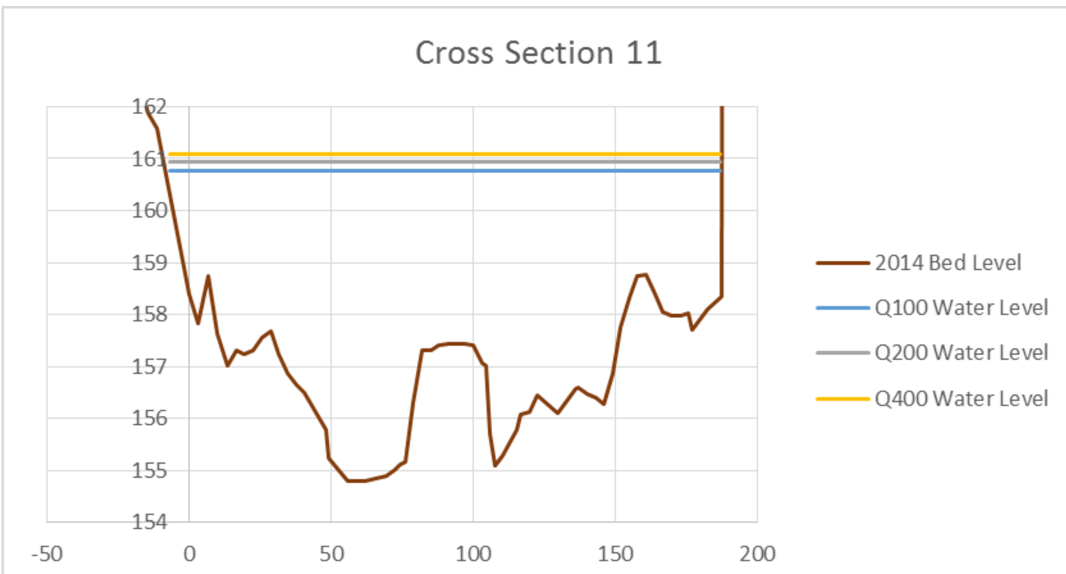


Figure 8-2 – Design water levels at cross section 11

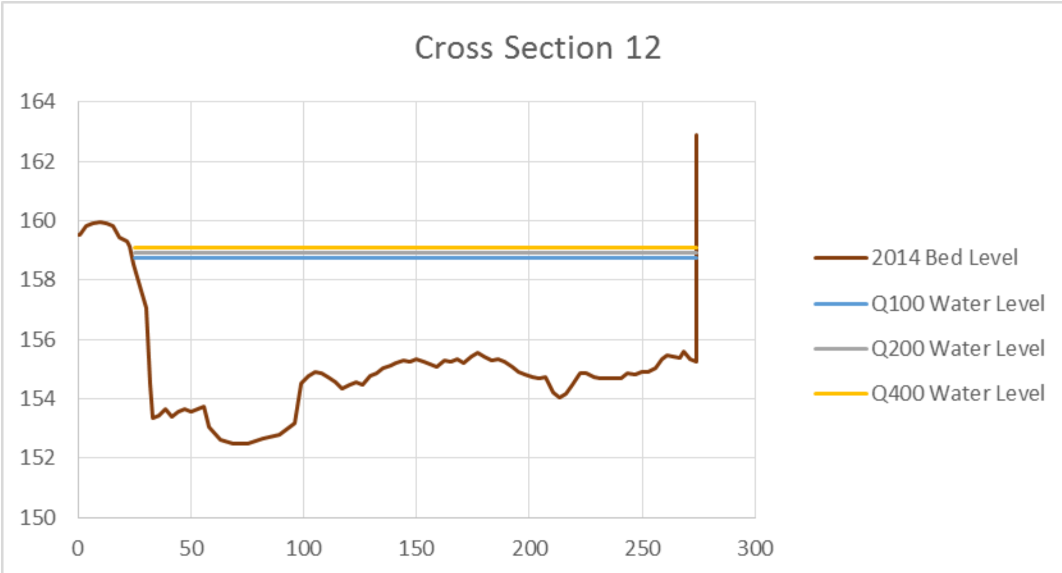


Figure 8-3 – Design water levels at cross section 12

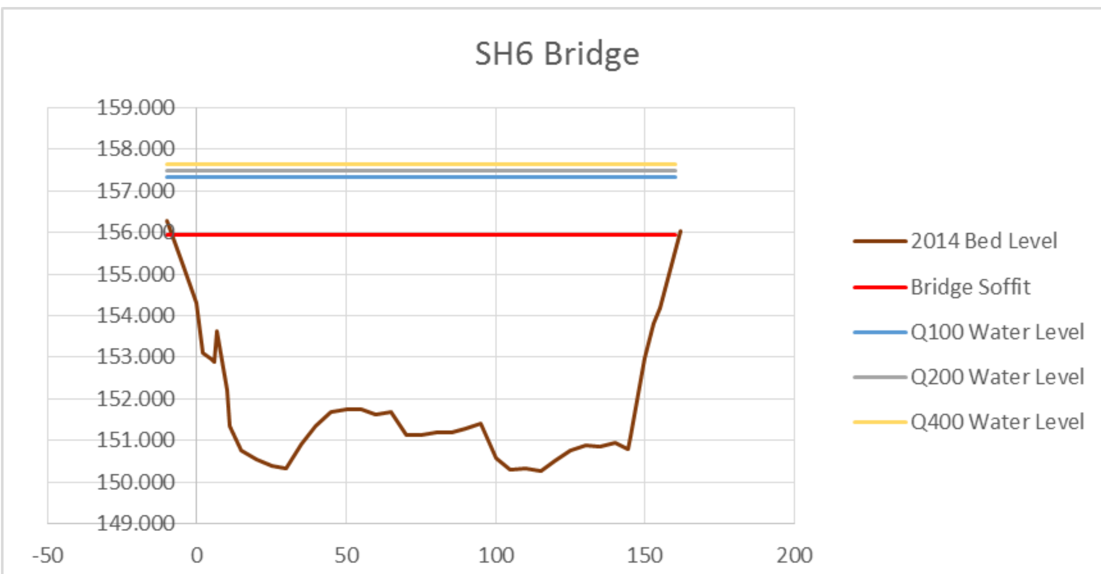


Figure 8-4 – Design water levels at SH6 bridge

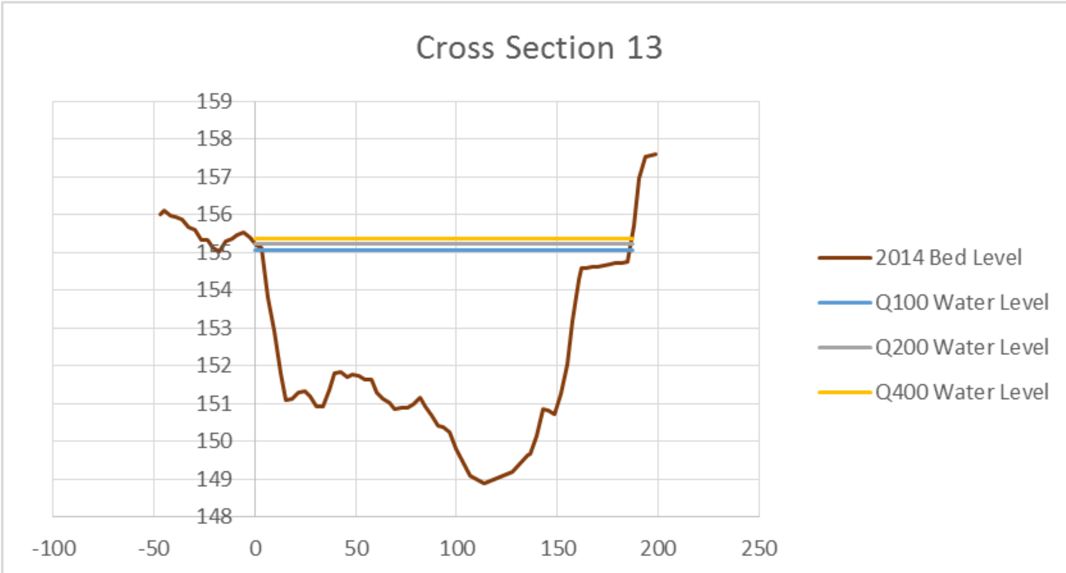


Figure 8-5 – Design water levels at cross section 13

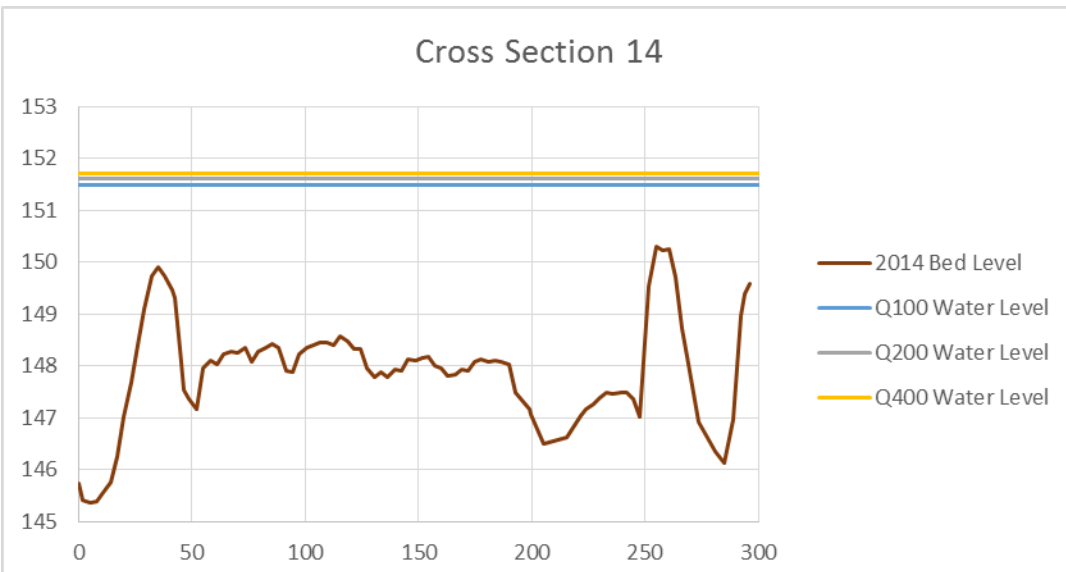


Figure 8-6 – Design water levels at cross section 14

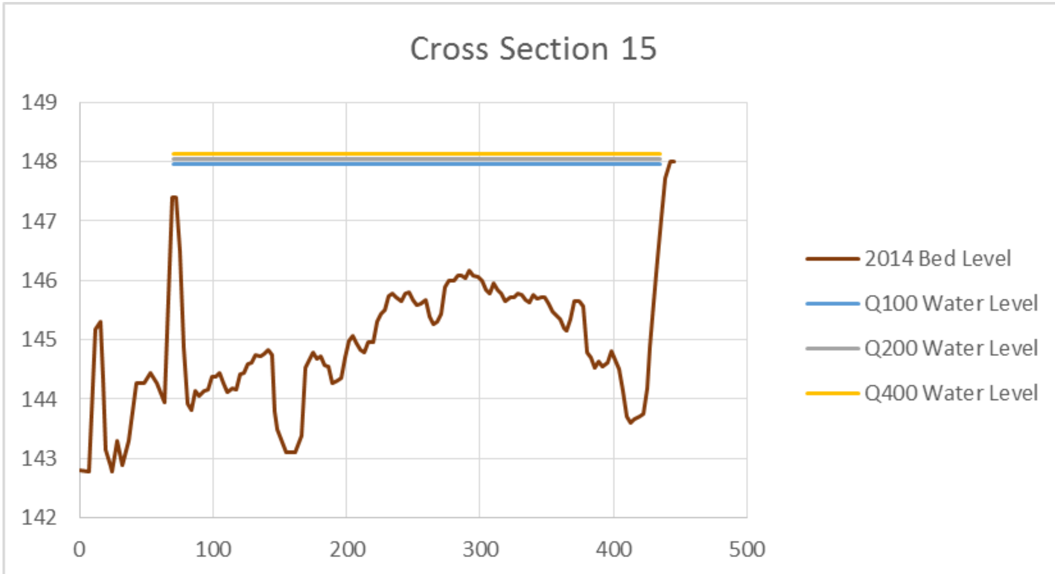


Figure 8-7 – Design water levels at cross section 15

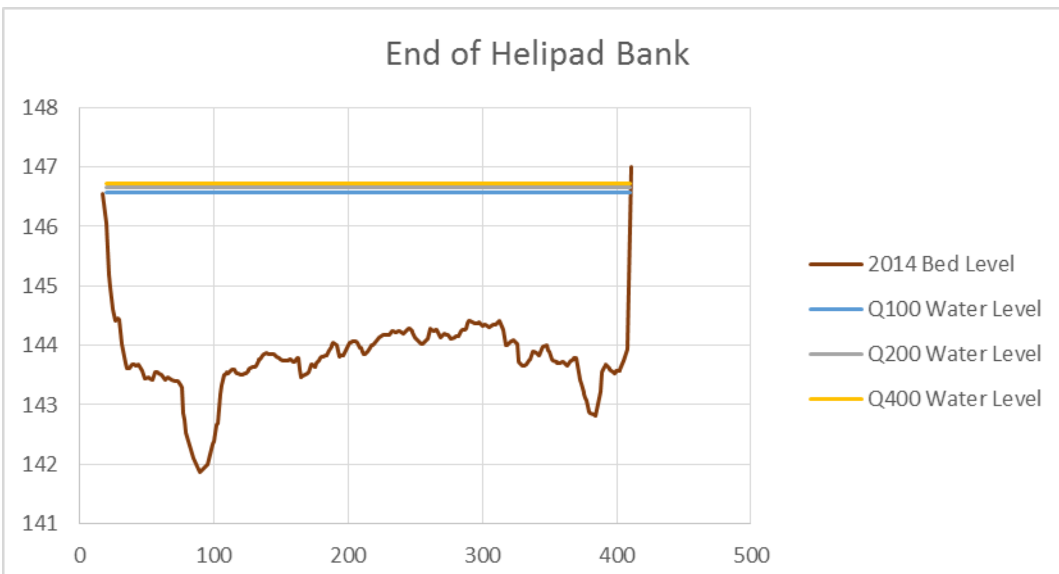


Figure 8-8 – Design water levels at end of helipad stopbank

CHANNEL LONGSECTION PLOT

A channel long section from cross section 10 to the end of the helipad stopbank has been presented in Figure 8-9 below.

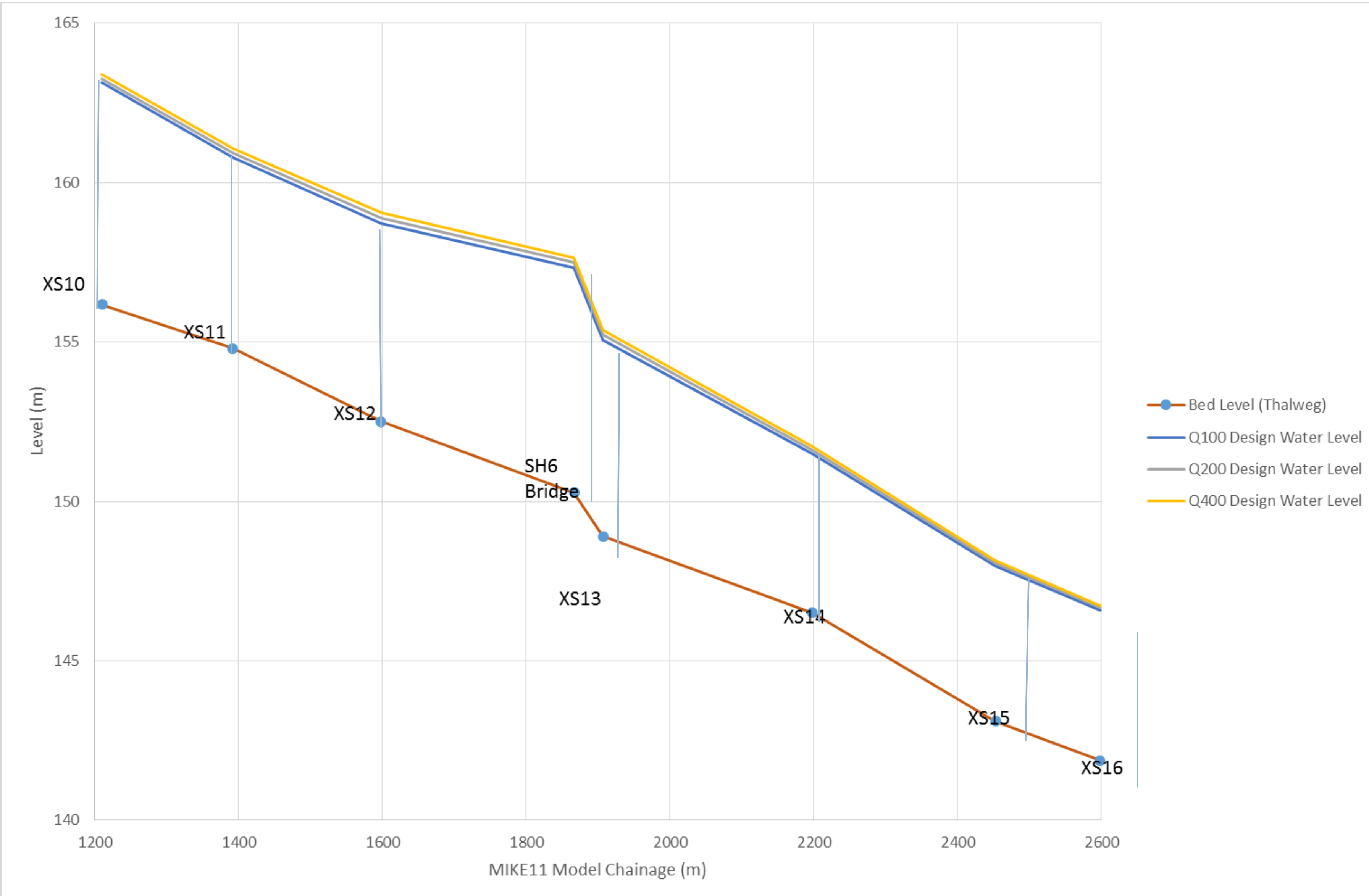


Figure 8-9 - Longsection plot showing design water levels

STOPBANK LONGSECTION PLOTS

Longsection plots showing the design water levels for the stopbanks within the study reach are shown in the following figures. Figure 8-10 below, shows the location of each longsection overlayed with the surveyed cross section locations. Figure 8-11 to 8-13 plot the design water levels along the most recently surveyed stopbank levels.

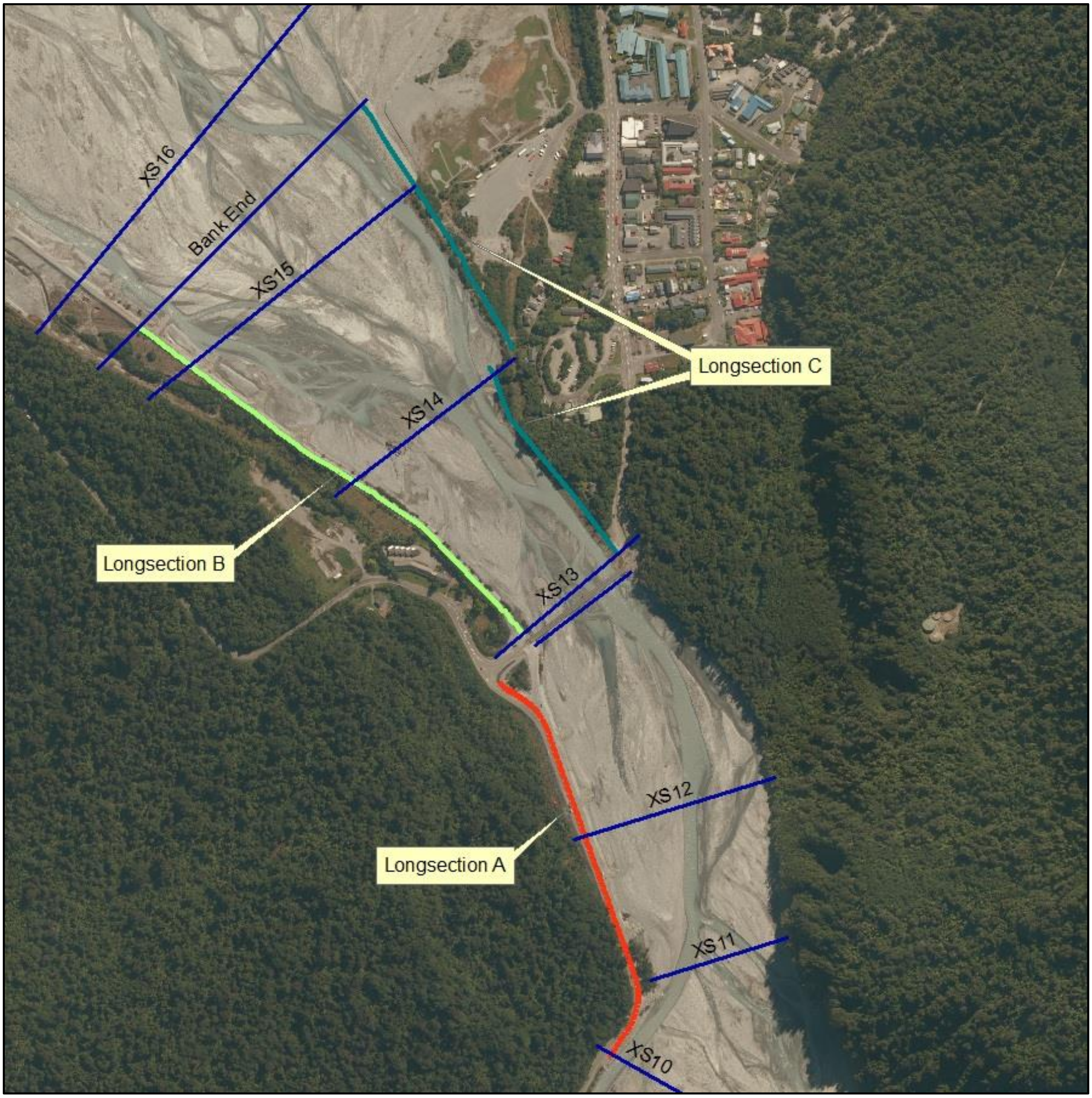
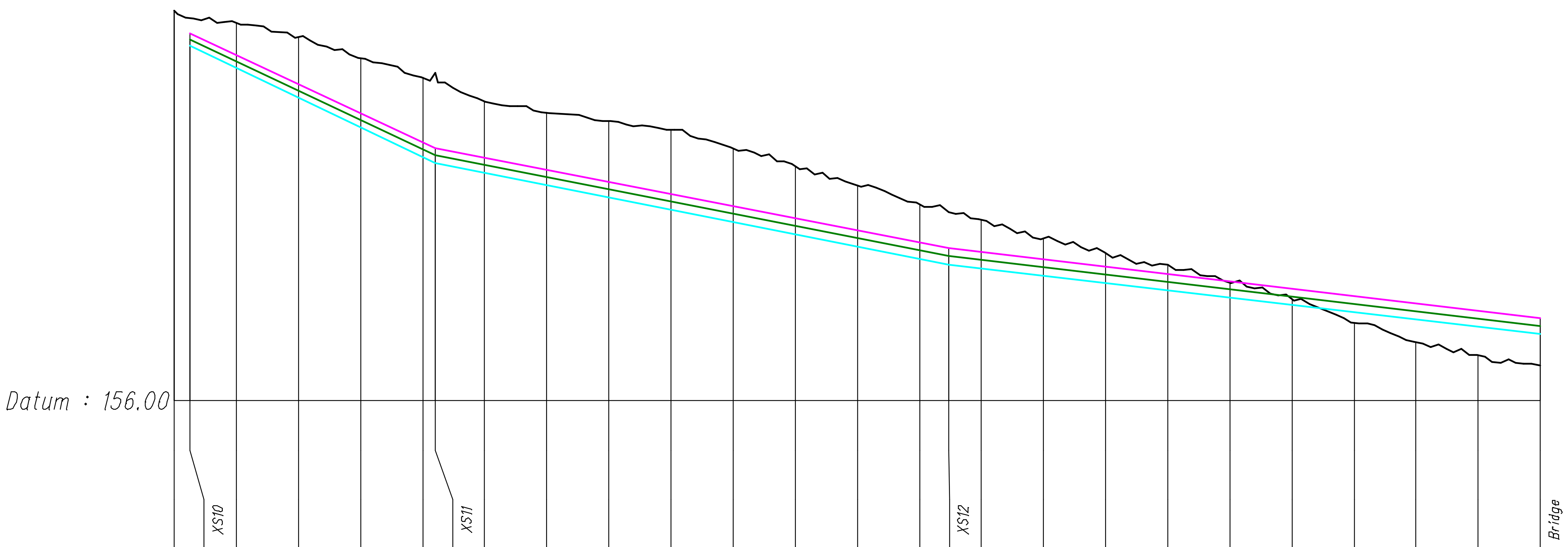


Figure 8-10 – Location of stopbank longsection plots

Figure 8-11 – Longsection showing design water levels upstream of SH6 Bridge against surveyed top of bank levels for Longsection A

2900 Cumecs
2700 Cumecs
2500 Cumecs



2900 Cumecs design level		163.38					161.08						
2700 Cumecs design level		163.26					160.93						
2500 Cumecs design level		163.14					160.77						
existing stopbank crest	163.85		163.60	163.32	162.88	162.50		162.02	161.79	161.63	161.44	161.07	160.71
Distance	0.0	6.3	25.0	50.0	75.0	100.0	105.2	125.0	150.0	175.0	200.0	225.0	250.0

2900 Cumecs design level			159.07										157.65
2700 Cumecs design level			158.90										157.49
2500 Cumecs design level			158.73										157.33
existing stopbank crest	160.33	159.95		159.64	159.26	158.96	158.73	158.37	158.03	157.56	157.17	156.90	156.70
Distance	275.0	300.0	311.9	325.0	350.0	375.0	400.0	425.0	450.0	475.0	500.0	525.0	549.9

Design
Land River Sea Consulting Limited
3/148 Canon Street
St Albans Christchurch
ph: 03 967 0549
E: matthew@landriversea.com
Design: Matthew Gardner August 2014

Sheet 1 of 3 sheets - scales horizontal 1:2000, vertical 1:100 - print size A3

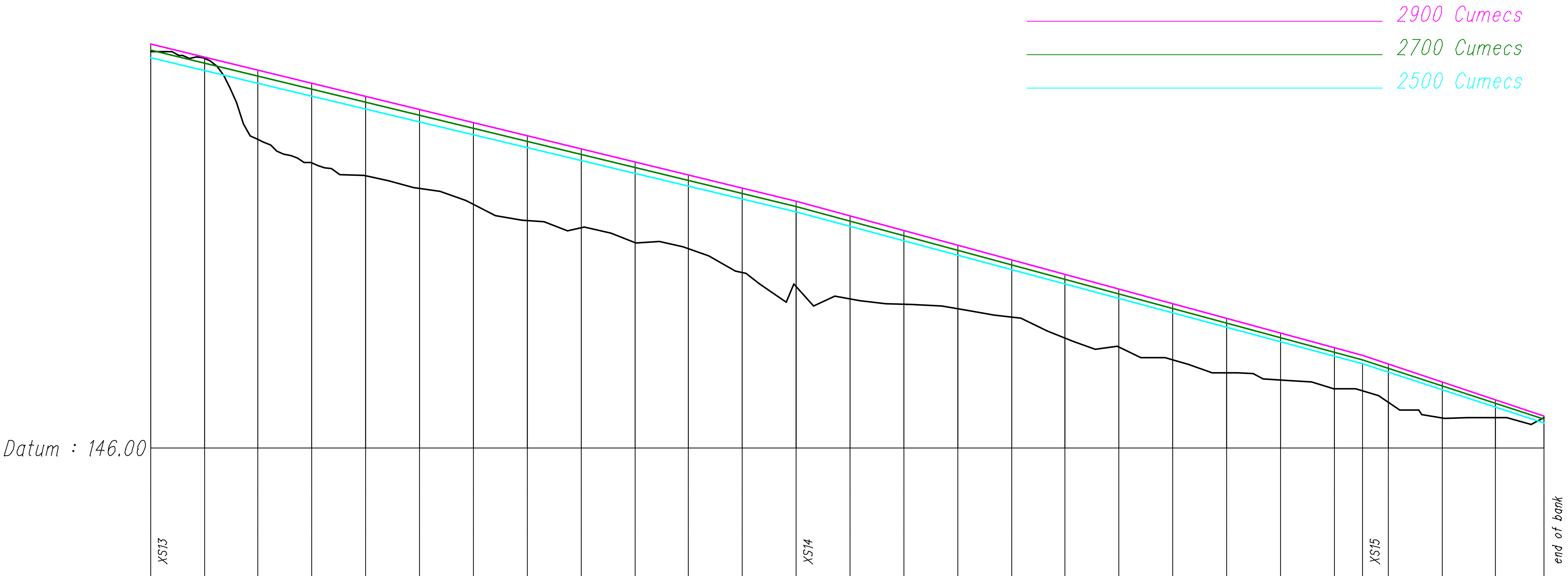
Drawn
Davis Ogilvie and Partners limited
PO Box 156 Greymouth
ph: 03 768 6299
E: greymouth@do.co.nz
Ref: GM33009
Drawn: Mike Robbins August 2014

LANDRIVERSEA PROFESSIONAL ENGINEERING

Client: West Coast Regional Council
Project: Waiho River - Design Flood Levels
Waiho River Protection
Long Section A

do Davis Ogilvie Professionals Engineers • Surveyors • Planners

Figure 8-12 – Longsection showing design water levels upstream of SH6 Bridge against surveyed top of bank levels for Longsection B on the true left bank



	XS13												XS14											XS15				end of bank
2900 Cumecs design level	155.38												151.72											148.14				146.73
2700 Cumecs design level	155.23												151.61											148.05				146.66
2500 Cumecs design level	155.07												151.49											147.96				146.58
existing stopbank crest	155.21	155.06	153.16	152.62	152.31	152.02	151.65	151.28	151.10	150.76	150.62	150.07	149.74	149.46	149.34	149.22	149.04	148.55	148.34	148.04	147.74	147.58	147.38		147.05	146.69	146.70	146.70
Distance	0.0	25.0	50.0	75.0	100.0	125.0	150.0	175.0	200.0	225.0	250.0	275.0	300.0	325.0	350.0	375.0	400.0	425.0	450.0	475.0	500.0	525.0	550.0	563.3	575.0	600.0	625.0	647.6

Design
Land River Sea Consulting Limited
3/148 Canon Street
St Albans Christchurch
ph: 03 967 0549
E: matthew@landriversea.com
Design: Matthew Gardner August 2014

Sheet 2 of 3 sheets - scales horizontal 1:2000, vertical 1:100 - print size A3

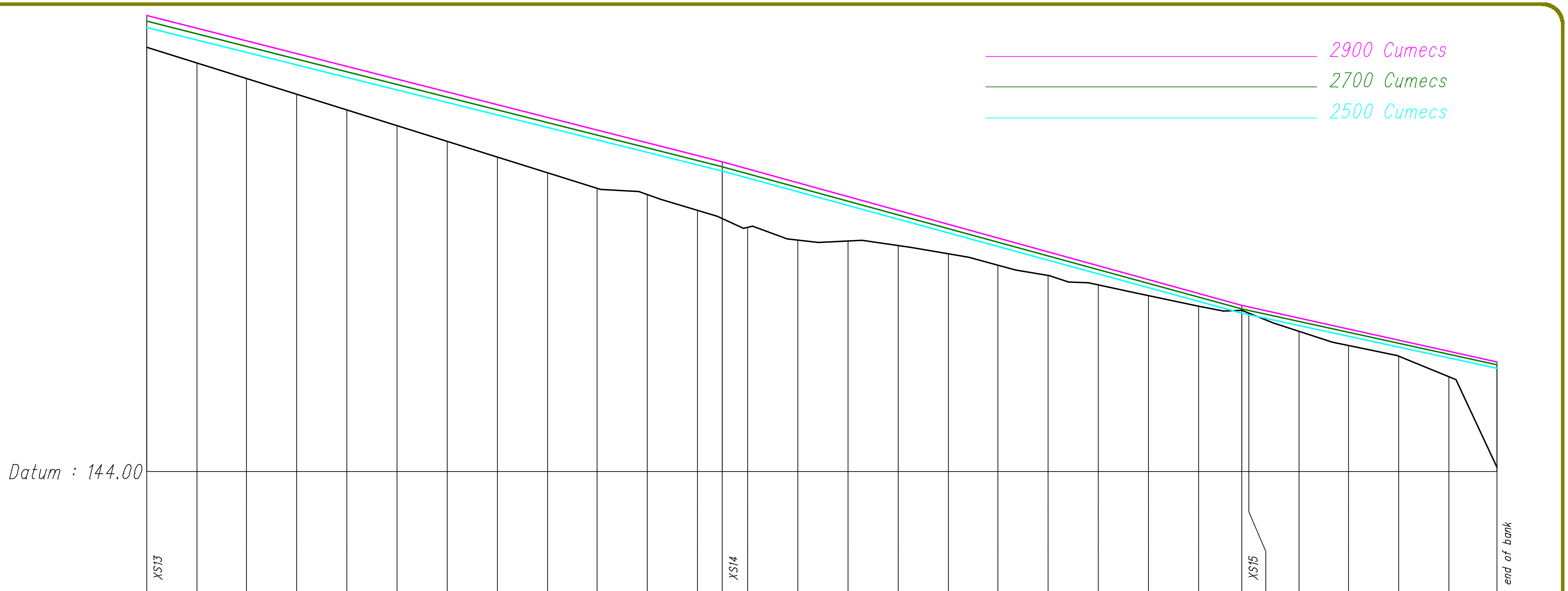
Drawn
Davis Ogilvie and Partners limited
PO Box 156 Greymouth
ph: 03 768 6299
E: greymouth@do.co.nz
Ref: GM33009
Drawn: Mike Robbins August 2014

LANDRIVERSEA
PROFESSIONAL ENGINEERING

Client: West Coast Regional Council
Project: Waiho River - Design Flood Levels
Waiho River Protection
Long Section B

do
Davis Ogilvie
Professionals
Engineers • Surveyors • Planners

Figure 8-13 – Longsection showing design water levels downstream of SH6 against surveyed top of bank levels for Longsection C on the true right bank



2900 Cumecs design level	155.38												151.72											148.14						146.73
2700 Cumecs design level	155.23												151.61											148.05						146.66
2500 Cumecs design level	155.07												151.49											147.96						146.58
existing stopbank crest	154.59	154.20	153.81	153.42	153.02	152.63	152.24	151.85	151.46	151.07	150.91	150.52		150.09	149.77	149.75	149.64	149.44	149.15	148.89	148.66	148.40	148.13		147.95	147.50	147.14	146.88	146.36	144.10
Distance	0.0	25.0	50.0	75.0	100.0	125.0	150.0	175.0	200.0	225.0	250.0	275.0	287.3	300.0	325.0	350.0	375.0	400.0	425.0	450.0	475.0	500.0	525.0	546.7	550.0	575.0	600.0	625.0	650.0	673.8

Design
Land River Sea Consulting Limited
3/148 Canon Street
St Albans Christchurch
ph: 03 967 0549
E: matthew@landriversea.com
Design: Matthew Gardner August 2014

Sheet 3 of 3 sheets - scales horizontal 1:2000, vertical 1:100 - print size A3

Drawn
Davis Ogilvie and Partners limited
PO Box 156 Greymouth
ph: 03 768 6299
E: greymouth@do.co.nz
Ref: GM33009
Drawn: Mike Robbins August 2014

LANDRIVERSEA PROFESSIONAL ENGINEERING

Client: West Coast Regional Council
Project: Waiho River - Design Flood Levels
Waiho River Protection
Long Section C

do Davis Ogilvie Professionals Engineers • Surveyors • Planners

9. MIKE FLOOD MODEL RESULTS

SOUTHSIDE STOPBANK

Model results show that the southside stopbank, downstream of the SH6 bridge is at the greatest risk of overtopping during a flood event. Interrogation of the model results shows that this section of bank is at risk of overtopping with a flow of less than 1700 m^3/s without allowing for any model freeboard. This equates to less than a 10 year return period event.

Figure 9-1 shows the extent of flooding which can be expected in a 100 year return period event in the location of the motel units. Flood depths at the motel units are predicted to be in the range of 1 to 1.3 m.

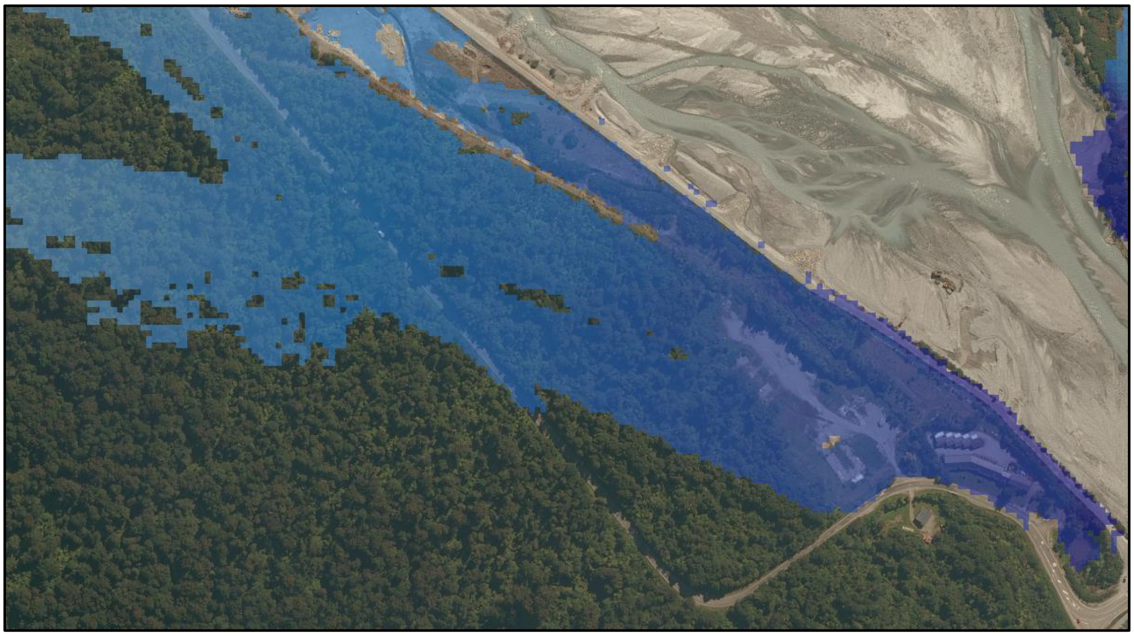


Figure 9-1- Location of overflow adjacent to the motel units

Flow velocities through the motel units are predicted to be in the order of 1.3 m/s for a 1 in 100 year flood event. Figure 9-2 presents the modelled maximum velocites for a 1 in 100 year event at this location.

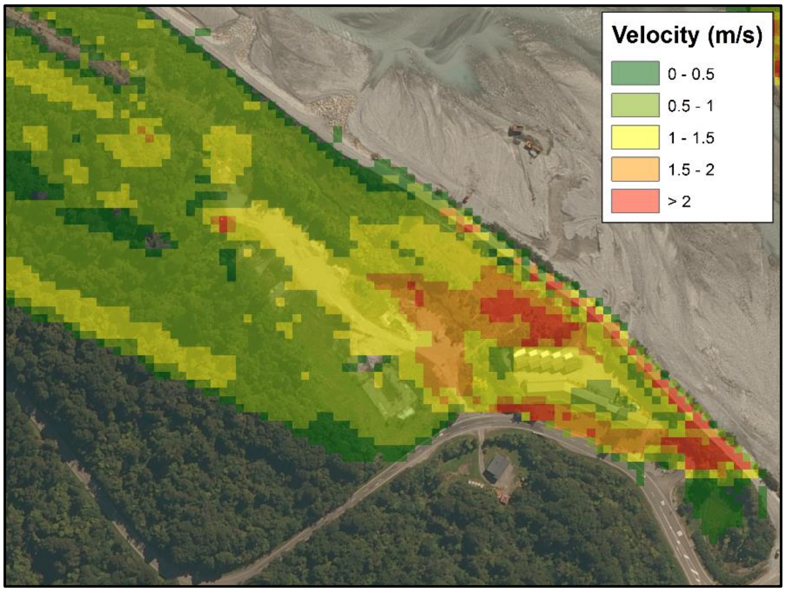


Figure 9-2 – Peak velocities at the Motel units during a 100 year flood event

Figure 9-3 shows a longsection of the south side stopbank with the modelled water level for an estimated 10 year return period event, without the addition of freeboard.

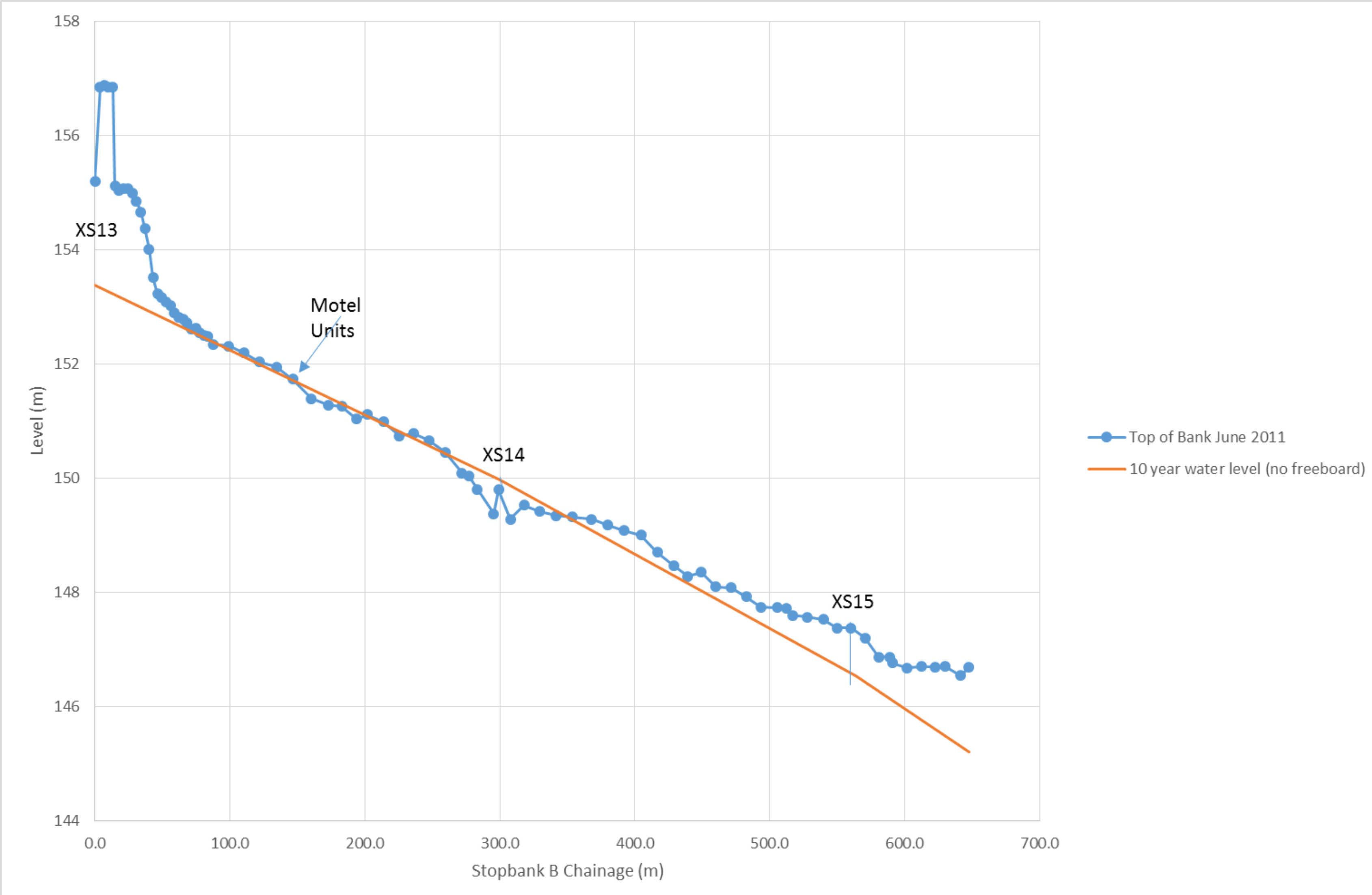


Figure 9-3 – Longsection showing water levels for a 10 year event (without freeboard) downstream of SH6 against surveyed top of bank levels for Stopbank C on the true right bank

RISK TO FRANZ JOSEF TOWNSHIP FROM RIGHT BANK OVERFLOW UPSTREAM OF SH6

Previous studies (Hall, 2012) have suggested that there is a potential overtopping risk on the true right embankment, upstream of SH6. Model results show that the risk of overtopping is low based on current river bed levels, however as bed levels rise, the risk of overtopping increases.

In order to determine how much aggradation is necessary to create an overtopping scenario, the model was run a number of times with varying degrees of aggradation. The model runs assumed as the bed levels aggrade, the current bridge will either be raised significantly to prevent overtopping or replaced and therefore the bridge deck has been removed from the model.

Model results show that bed levels would need to increase in the order of 4m on average from the 2014 survey levels before the ridge on the right bank will be overtopped with a 100 year event. The model shows the main road to Franz Josef being overtopped at approximately the same time as the ridge upstream from the bridge location. Due to the steep gradients, water depths are very shallow and unlikely to inundate floor levels, except in a small number of locations. Due to steep gradients, velocities can be expected to exceed 2 m/s down streets, however will be typically less than 0.5 m/s through properties.

Figure 9-4 shows the potential flood extent for a 100 year event with bed levels being 4m higher than the 2014 survey levels.

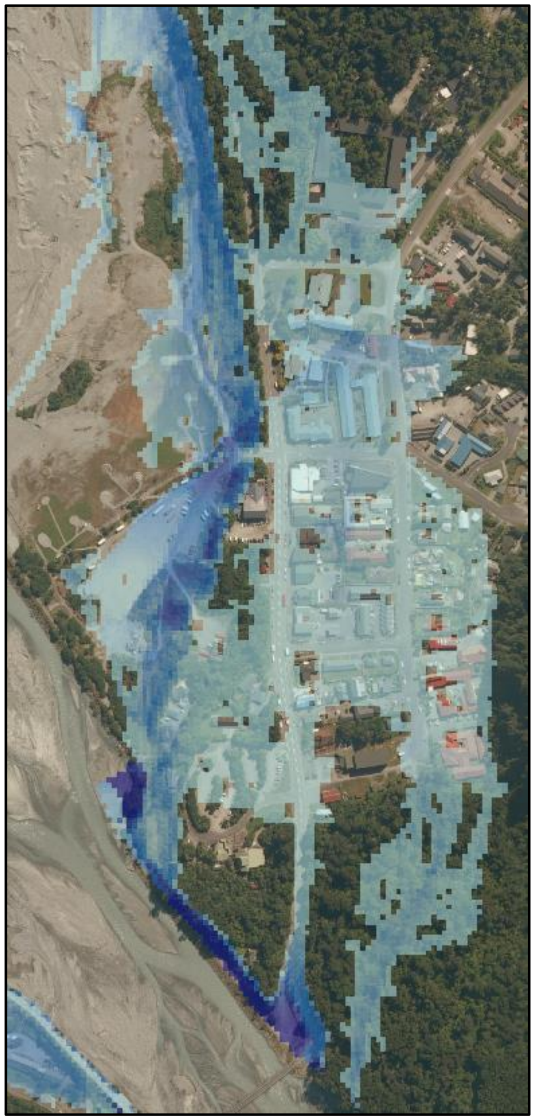


Figure 9-4 – Flood extent for a 100 year event with bed levels raised by 4m from the 2014 survey levels in the vicinity of Franz Josef town.

CHANNEL AND BRIDGE WIDENING

Results show that increasing the bridge opening width by another 1/3rd of its current width as well as allowing the downstream river channel to widen will have a significant effect on lowering water levels (see Figure 5-1 for a model schematic). Model results show that water levels will typically reduce by between 0.1 and 0.8 m, however results show a significant decrease of 1.5 metres at the SH2 Bridge due to the fact that the lowered water levels will only briefly reach the level of the soffit. A longsection of the true right stopbank is shown in Figure 9-4 comparing design levels for the existing situation as well as with the channel widening.

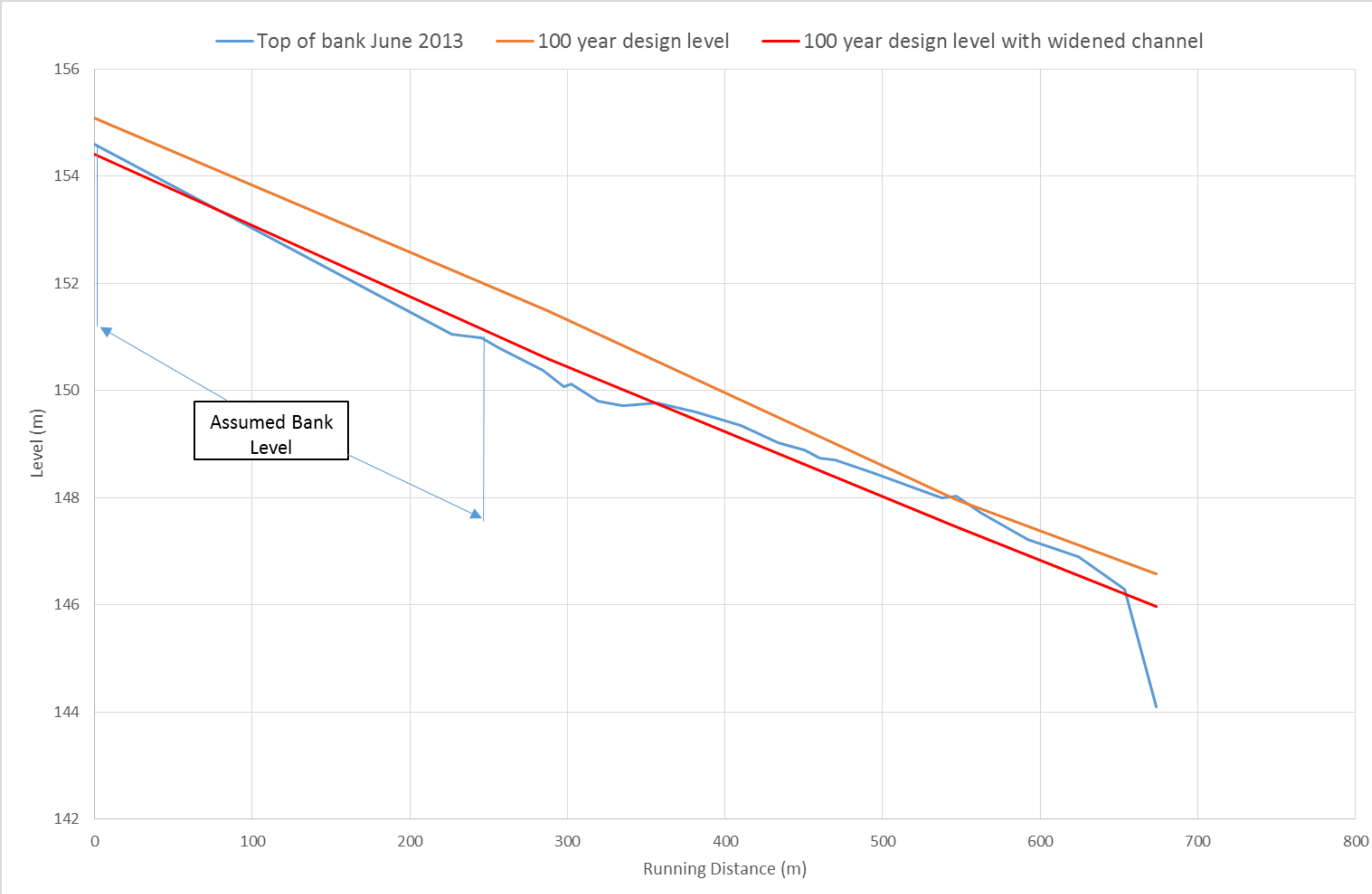


Figure 9-4 – Difference in design levels on the true right bank stopbank for a 100 year return period event, with and without channel widening downstream from SH6

10. DISCUSSIONS

There are a number of significant uncertainties in this model as outlined in Section 7. The most significant of these uncertainties is the large volumes of sediment input into the river system causing bed levels to aggrade in an unpredictable manner. There is also a large degree of uncertainty in the flow data adopted in this study. Adopted flows used in this study have been based on flow statistics from the Whataroa River provided by WCRC (Beaumont per coms, 2014). Differences between adopted flows with previous studies are detailed in Section 3.

The results show that upstream of SH6 is currently well protected. Downstream from the bridge however, in particular on the true left bank, is at risk of overtopping in events less than a 1 in 10 year return period event. Whilst the model results (Figure 7-13) show water levels overtopping the true right bank, this is only likely to occur if the southside stopbank is raised. This is due to the fact that the southside stopbank is lower than the right bank and water will overtop this bank prior to water levels reaching the top of the right bank.

Previous studies (Hall, 2012) have suggested that there is a potential overtopping risk on the true right embankment, upstream of SH6. Model results upstream of SH6 show that the right bank is currently at a low risk of overtopping, however if bed levels continue to rise, any bridge structure will likely be outflanked with potential to inundate Franz Josef via SH6 as well as overtopping the ridge on the right bank.

Current model results show higher water levels when compared with previous studies primarily due to the fact the bed levels have aggraded significantly in the study reach in recent years. Differences in adopted values for flow and Manning's 'n' will also impact on predicted water levels. Previous modelling studies have not taken into account head losses at the SH6 Bridge and are therefore likely to underestimate water levels at this location. Online video footage of recent flood events (Charleston, 2013) show the bridge piers providing significant resistance to flood flows at the bridge location and confirms the importance of the inclusion of this structure.

Widening of the bridge opening as well as the downstream river channel has been simulated and shows a significant lowering of design flood levels. Widening the river channel would likely be advantageous as it would remove some of the pressure on the true right bank reducing the risk of flooding to the Franz Josef Township. Widening the river may also have some minor impact on the rate of aggradation in this reach by widening the area over which the river can deposit sediment, therefore slowing the rate at which the m.b.l will increase. Whilst increasing the width of river in this short reach is unlikely to significantly alter aggradation rates, it should be considered as an improvement on the existing situation and a step in the right direction. If this was to occur however the motel units and residential property will be required to relocate.

Climate change has not been simulated as part of this study. Due to the already significant uncertainties in hydrological input as well as the rapidly changing nature of the bed levels, it is not considered relevant to simulate changes in rainfall 100 years into the future.

11. CONCLUSIONS

- A MIKE 11 model of the Waiho River has been built and used to produce design water levels for the estimated 100, 200 and 400 year return period events.
- Flood protection works upstream of SH6 provide a high level of protection, except for immediately upstream of the SH6 Bridge, which is at risk of overtopping if significant blockage from debris occurs during a flood event.
- Flood protection works downstream of SH6, in particular on the true left bank are not sufficient to prevent flooding in a 100 year return period event. In particular, the motel units on the true left bank are at risk of flooding in less than a 1 in 10 year return period event.
- The bed level in the river is higher than the ground level at the motel units. If the stopbank is to fail or be overtopped, risk to life and damages could be high.
- Bed levels in the river are increasing rapidly. Mean bed levels appear to have been increasing in the order of 0.16 - 0.2 metres a year since the 1940's and are likely to continue to increase.
- An increase in mean bed level of 2 metres within the study reach equates to an average increase in water level of 1.7 m. Increasing bed levels has the impact of reducing the level of protection provided by existing flood protection systems.
- The potential for an overflow of the embankment on the true right bank upstream of SH6 has been investigated. Analysis has shown that the risk will exist in the future as bed levels continue to rise, however there is potential for the river to outflank any existing bridge structure and enter the town via the main road at the same time. Bed levels will need to increase significantly before any significant risk is present.
- Widening the SH6 Bridge along with the downstream channel is likely to have significant impacts on lowering water levels within the channel. This will reduce some of the pressure on the right bank stopbanks, as well as potentially slow the current rate of bed aggradation to a small extent. This will only 'buy some more time' however as it is likely that bed levels will continue to increase in the coming years.

11. REFERENCES

Measures, R; Duncan, M. (2011). Callery River Landslide Dam - Rapid assessment of dam failure consequences. NIWA Client Report No: CHC2011-097

Hicks, D.M.; Mason P.D. (1998). Roughness Characteristics of New Zealand Rivers, NIWA.

Smart, G.M. (1991). A P.O.E.M. on the Waiho (Electronic gauging of rivers). *Journal of Hydrology (NZ) 30*: 37-44

McKerchar, AI; Pearson, CP. (1989) Flood frequency in New Zealand. Publication No. 20, Hydrology Centre, Department of Scientific and Industrial Research, Christchurch.

Griffiths GA (1981) Flow resistance in coarse gravel bed rivers. Proc. ASCE Journal of the Hydraulics Division, Vol. 107, No. 7, July 1981, pp. 899-918.

Davies, T; McSaveney, M; Clarkson, P. (2002). Anthropic aggradation of the Waiho River, Westland, New Zealand: Microscale modelling. Earth Surface Processes and Landforms 28, 209–218 (2003)

Mills, L. (2012) Franz Josef Glaciers Rapid Retreat. New Zealand Herald (http://www.nzherald.co.nz/nz/news/article.cfm?c_id=1&objectid=10818264)

Charlseworth, E. (2013) Waiho River, New Zealand, In Flood. YouTube. https://www.youtube.com/watch?v=CIGFTF6YhN8

Optimix. (2002) Waiho River Flooding Risk Assessment: for Ministry of Civil Defence and Emergency Management.