Report

Initial Assessment of Practicable Flood Management Options Lower Motueka River Local Government Act 2002 s78 – Part of Stage C

Prepared for Tasman District Council

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QUALITY STATEMENT

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Tasman District Council

Initial Assessment of Practicable Flood Management Options Lower Motueka River

CONTENTS

LIST OF TABLES

LIST OF FIGURES

APPENDICES

Executive Summary

Background
The Lower Motueka River is susceptible to flooding during significant storm events. The existing stopbanks provide some degree of flood control, but they do not meet current design standards. Geotechnical and hydraulic modelling investigations have indicated that they may not provide adequate protection to local residents or their assets. Consideration has therefore been given to assessing options which would improve the ability of the stopbanks to contain flood waters. Other options for improvement to the flood control for Motueka have also been assessed.

For reference, a location map and table of feature names are at the end of this executive summary.

Local Government Act
Section 78 (s78) of the Local Government Act (2002) provides Tasman District Council with a process to consider the community views.

The process, to be undertaken in considering the community views, follows several stages:

Stage A: definition of problems and objectives
Stage B: identification of reasonably practicable options
Stage C: assessment of reasonably practicable options and development of proposal(s)
Stage D: adoption of proposal(s).

This report addresses part of Stage C, being an initial assessment of reasonably practicable options for mitigating large flood events. It also provides comparative information of the costs and benefits of these flood mitigation options.

Public Consultation
In following the Section 78 process, Council has consulted with and informed the community through Stages A and B.

Recent community feedback to Council indicated that refurbishment of the existing stopbanks and removal of gravel (from the berm areas between stopbanks) should be assessed in more detail.

"Design Standard" Event

- Tasman District Council has selected the "design standard" to be a 1% Annual Exceedance Probability (AEP) event including allowance for 2090 climate change. The Ministry for the Environment document "Preparing for Climate Change – Guide for Local Government" has been used for guidance.
- Current 1% AEP peak flow at Alexander Bluffs Bridge is 3165 cubic metres per second.
- 2090 1% AEP peak flow at Alexander Bluffs Bridge is 4053 cubic metres per second.
- The largest recorded flow for the Motueka River at Woodstock is 2148m^3/s on 10/07/1982. Based on catchment area at Woodstock (1750km^2) and Alexander Bluffs (1968km^2) this equates to a corresponding flow of 2360m^3/s at Alexander Bluffs.

Recent Technical Work
Recent studies have been undertaken to improve understanding of the storm hydrology, the resulting river flows, and the geotechnical ability of the stopbanks to resist the effects of flood waters.

Previously, flood flow predictions have been based upon a frequency analysis of the recorded flows at Woodstock. A Rainfall-Runoff model has now been created to provide a more reliable understanding of potential storm flows into the Motueka River. Using this improved hydrological information, the river flows have been re-modelled for some key scenarios. This provides (hydraulic model) information on the likely height of flood water and the duration of flood flows. The effect of stopbank breach at a specific location (Right bank opposite Fry's Island) has also been assessed.

Further modelling "runs" have been undertaken, including 1% AEP with "banks down" and 0.5% AEP "breach" case. The outputs have been included in the economic analysis and assessments made to-date.

The flood-control stopbanks are vulnerable to a variety of failure modes. Notable modes are.

- The 24 hour storm event results in highest river levels, and therefore would give rise to the highest risk of over-topping.
- The 72 hour storm event results in the longest period of sustained high flow, and therefore creates the highest risk of saturation (stability) failure of the stopbanks.
- Erosion due to flood water velocity, which may result in stopbank breach.

The stopbanks can also fail to contain flood waters if under-seepage occurs.

Assessment of Options
The Multi Criteria Analysis Workshop assisted in the identification of reasonably practicable options for river flood management.

The practicable options are:

1) Status Quo (existing stopbanks)
2) Secondary Stopbanks
3) Spillway(s)
4) Refurbish (improve) existing stopbanks
5) Rebuild (new) stopbanks.

The practicable options are assessed in this report, and include the options of rebuilding the stopbanks, and refurbishment of all or part of the stopbanks.

The MCA workshop assessment concluded that the options to construct secondary stopbanks or spillways are not appropriate solutions to pursue further.

Mitigation of some stopbank failure modes may be achieved by raising the height of the stopbanks in critical locations, by improving the resistance to saturation (eg. by lining the river face with material of low permeability), and by appropriate river maintenance and rock protection to reduce the risk of erosion.

It is known that the existing stopbanks are not well-compacted and do not provide modern standards of resistance to flooding. Total replacement would provide the most mitigation of flood risk, although the cost would be significant. There would also be an increased level of risk during construction because the existing stopbanks would need to be removed before re-building.

Partial refurbishment may provide a more acceptable trade-off of cost versus benefit for the local community.

Removal of gravel from berm areas has also been assessed to determine whether flood capacity would be significantly improved. There is some reduction in risk of over-topping, but the stopbanks would still be vulnerable to other modes of failure.

There will still be residual flooding risks after the implementation of any of the options. The community will need to accept the (value of) residual risks of whichever scheme is chosen.

Construction Costs

The construction cost estimates for the Lower Motueka River Flood Control Scheme prepared for the Preliminary Design phase 2008 have been revised and adjusted as part of the options assessment carried out. Consideration of recent hydrology, flood hydraulics and geotechnical work has allowed for the removal of most of the subsoil seepage control measures. Landward pump stations are now not included in the proposed options.

The critical sections of the scheme have been identified as Brooklyn Stream (when concurrently affected by flood in own catchment), plus the right bank from the outlet to Woodman's Bend, and the left bank from outlet to Blue Gum Corner – both of which protect large parts of the plains and the main townships.

The Brooklyn Stream stopbanks are not expected to be overtopped during the "design standard" event in the Motueka River catchment, and there is limited space in which to implement any refurbishment works. These stopbanks will need to be addressed separately to provide protection from a flood event in the Brooklyn catchment(s).

The section of the flood control scheme below Woodman's Bend, a length of 12.6km, has been considered separately without the upgrade of stopbanks at Hurleys, Kiwifruit orchard, Peach Island ring, Brooklyn Stream and the mouth return spur on the left bank.

The "full" schemes considered in this report exclude the Peach Island ringed stopbank and the Brooklyn Stream stopbanks and are 15.6km in length. [Storms in the Brooklyn and Little Sydney catchments will still be a flooding risk to the Riwaka area that will need to be considered separately].

Construction costs are based on achieving the flood protection standard – 1% AEP rainfall event plus allowances for projected climate change impacts of increased rainfall intensity and sea level rise by 2090. Any alteration to the State Highway 60 Bridge crossing is not part of the stopbank scheme costs.

Partial refurbish options have been considered where parts of the existing stopbank that meet the design standard for height could be left in place without upgrade works. The costs for partial refurbishment are dependent upon detailed investigation and are not well defined at this stage.

The costs of the identified options are approximate only. There are many variables and unknowns, therefore the amounts should be considered useful for comparing the options. The cost estimates are not precise: for example, it is particularly difficult, at this stage, to quantify costs to address land issues; the costs of resource consents can also vary significantly. The project team will need to meet, discuss and then confirm the assessment of "whole of project" costs; assessing potential project risks against their weighted costs.

Removal of gravel is not a stand-alone option to provide flood control. The reduction in top flood water level is not sufficient to avoid the stopbanks being required. Therefore, the condition of the stopbanks must still be addressed to protect the community. Gravel removal has some merit as a concept to improve flood capacity, but there are significant issues to be overcome such as land ownership and the Water Conservation Order. It would be very expensive for Council to remove the gravel. However, it may be possible to negotiate with industry users of gravel to remove the gravel at zero cost to Council. This latter method would probably take many years, because removal would only be done to match demand for gravel.

An exploration of the uncertainties involved in the economic analysis showed that "Full" refurbishment had the greatest Benefit Cost Ratio for a wide range of conditions[1].

[1] One further refinement of the economic analysis is recommended to confirm the robustness of this conclusion.

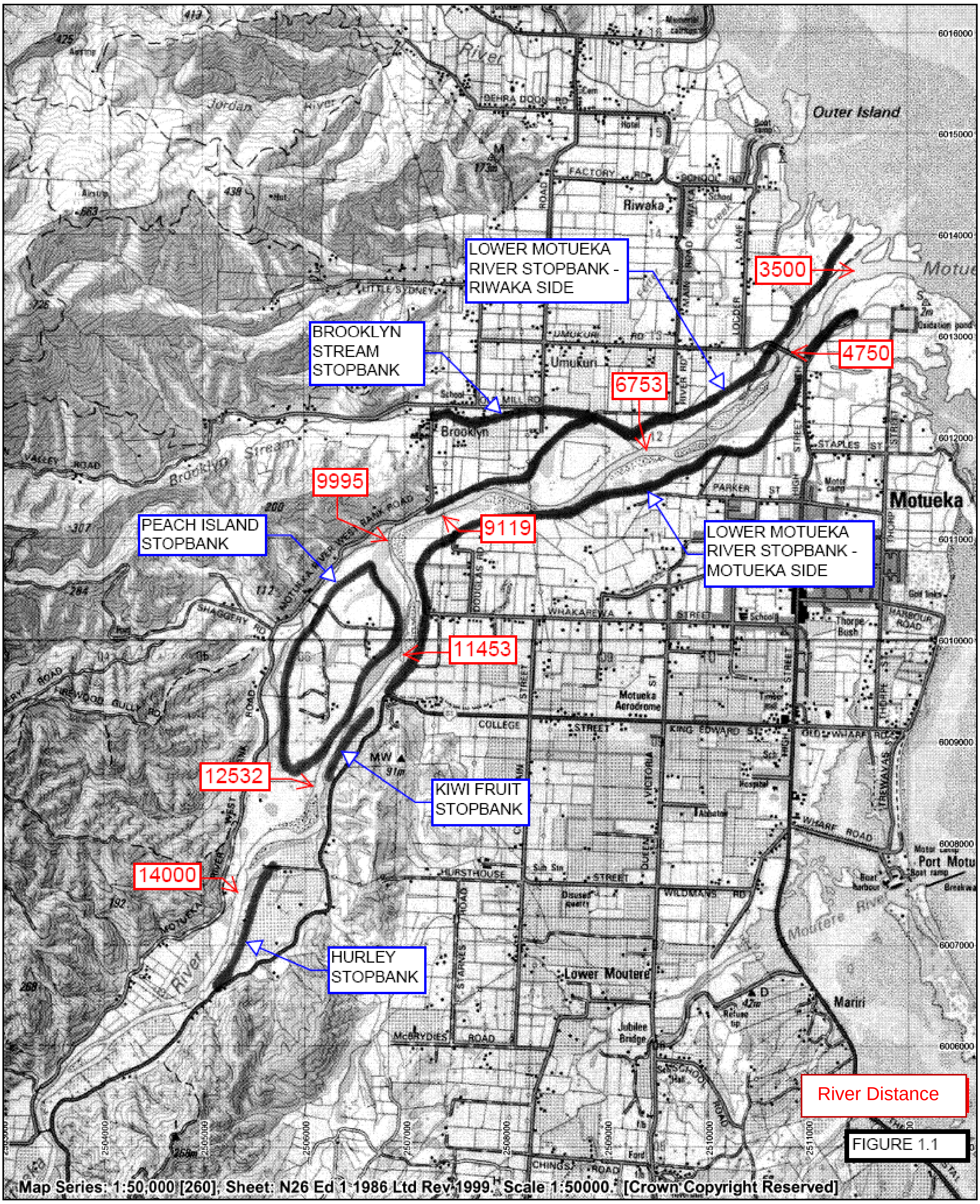
The Multi-Criteria Analysis workshop also concluded that “Full” refurbishment provides the best balance of cost and benefits. Full refurbishment has a higher benefit to cost ratio than the rebuild option because it is significantly cheaper and because it avoids the temporary increase in flood risk while the existing stopbank is removed to make way for the new stopbank.

The economic analysis allows a more quantitative assessment of scheme costs set against the reduction in risk achieved by the options. It provides a rational basis for establishing an acceptable balance between level of protection and the acceptability of the residual risks weighed against the option costs. This approach is consistent with the “Managing Flood Risk” Standard (NZS9401:2008) which puts the onus on the community to decide what residual risks it is prepared to accept.

Estimate of Total Project Costs					
Item	Rebuild Option	Refurbish Option	Secondary Stopbanks	2 Spillways only	Status Quo
Construction	$11.7m	$8.6m	$13m	$0.5m	-
LTP Consultation; Peer Review; Legal Costs; Environment Court; Land Costs; Legacy Issues	$1.45m	$1m	$1.45m	$0.65m	$0.3m
Total	$13.15m	$9.6m	$14.45m	$1.15m	$0.3m

Note that only Rebuild and Refurbish provide protection against the “design standard” event.

Estimate of Benefit/Cost Ratio (BCR)					
	Rebuild Option	Refurbish Option	Secondary Stopbanks	2 Spillways only	Status Quo
BCR	1.34	1.50	0.57	1.04	-



Map Showing locations of Stopbanks

Next Steps

Tasman District Council should consider the contents of this report and determine how and when to undertake the next phase of community consultation in line with Stage C of the s78 Local Government Act process.

The key to selecting the most appropriate option is to balance the cost of any upgrade against the (value of) residual risks that remain following the upgrade.

Lower Motueka River – Common Names and River Distance (RD in Metres)

Feature	River Distance* (m)	Bank
River mouth	3500	
Outlet Stopbank Spur	4160	Left
State Highway 60 Bridge	4500	
Brooklyn Stream confluence	6300	Left
Fry's Island	6500 to 7500	Left
Blue Gum Corner	8500	Left
Peach Island Back channel confluence	9000	Left
Peach Island stopbank	9260 - 11760	Left
Woodman's Bend	10600	Right
Kiwifruit stopbank	10900 - 11760	Right
Peach Island back channel start point	12910	Left
Hurley's stopbank	12910 – 14100	Right

*Note: River distance is based on the river mouth being at 3500m.

Left Bank/Right Bank refer to facing downstream

1 Introduction

1.1 Purpose of Report

Section 78 of the Local Government Act provides Tasman District Council with a process to consider community views.

The process, to be undertaken in considering the community views, follows several stages:

Stage A: definition of problems and objectives
Stage B: identification of reasonably practicable options
Stage C: assessment of reasonably practicable options and development of proposal(s)
Stage D: adoption of proposal(s).

This report addresses an initial part of Stage C.

Two separate (October 2010) MWH reports addressing Stage A were:

- Motueka River Flood Management Study
- Motueka River Flood Control Scheme Upgrade Phase 2 Summary Technical Report.

The January 2011 Tasman District Council report by Gary Clark (Transportation Manager), for 3 February Engineering Services Committee meeting, addressed Stage B.

1.2 History

The original Motueka stopbanks were constructed between 1951 and 1956 by the Nelson Catchment Board to accommodate a 50 year Average Return Interval (ARI) design flood (2% Annual Exceedance Probability) of 2830 cumecs with a freeboard of 600mm.

The scheme consisted of 20.4km of stopbanks, channel improvements and realignment along with bank protection including Peach Island. The scheme was primarily designed to prevent flooding of the Motueka flood plain where tobacco and hop growing yielded high returns. [Refer to figure in Executive Summary for scheme location and layout of stopbanks].

The original scheme stopbanks were designed with a top width of 2.44m, with river and land side batters of 1.5:1 and 2:1 respectively.

It should be noted that the river distances for the original scheme have a starting distance, at the mouth of the river, of 3500m.

The scheme at the time also included control of other main waterways flowing across the Motueka and Riwaka coastal plains. This included stopbanks along the Riwaka River and improvements to the Brooklyn and Little Sydney Streams.

At some time after the original scheme construction, additional banks were constructed (probably by landowners) on the eastern side of the river. These are referred to as the Kiwifruit Stopbanks and Hurley Stopbanks. The stopbanks below the State Highway 60 Bridge have also been modified and extended since the scheme was constructed.

The legal status of all the stopbanks, with respect to any scheme upgrade programme, requires further investigation by Tasman District Council.

1.3 Current Considerations

Tasman District Council's Ten Year Plan identified the need to review and reconstruct the current stopbanks on the Motueka River. Tasman District Council has more recently reviewed this intention, and continues to consult with the community on this matter. Council concluded that there was a need to determine the best practicable and affordable flood control option.

Although the stopbanks have prevented major flooding in the past, they do not meet modern standards. In past events sandbags have had to be used to prevent breaches at some weak sections of the stopbanks. It is known that the construction methods used did not provide adequate compaction of the central core of the banks. Recent investigations have shown that the current engineering fitness of the stopbanks is such that they would not hold up under sustained or repeated flooding events over a short period of time. It is therefore considered that, in their current state, they do not provide adequate protection to local residents and their assets.

The current flood control measures and stopbanks were discussed by submitters during the Ten Year Plan's consultation processes. While many submitters acknowledged the need for action, it was on the condition that adequate on-going public consultation was an integral part of the process. This was acknowledged in the July 2010 Issue No.01 Motueka Flood Control Project newsletter. At the heart of the conditional agreement by the community was the need to better understand the risks posed by the current state of flood defences. The community was concerned that any decision should take account of the balance between what is an acceptable level of risk, versus what the community is prepared to pay.

The initial review of the Motueka flood control scheme occurred in 2006 with a view to upgrading the scheme to a 1% Annual Exceedance Probability (AEP) protection standard. This is a standard for which the scheme will be capable of withstanding a flood resulting from a rainfall storm event that has an average return period of 100 years. Phase 1 of the review involved a feasibility study, and a preliminary design and costing report, for the upgrading of the scheme to the 1% AEP standard.

The 1% AEP standard of protection (with 2090 climate change allowance) has been adopted in principle, by Tasman District Council, to bring the protection of the Motueka township and the Motueka Flood Plain up to the nationally recognised minimum standard of protection provided by similar flood protection schemes in New Zealand.

As a comparison, the recent Aorere River (Golden Bay) flood has been assessed by the Tasman District Council hydrologist as a 167 year ARI event (0.6 % AEP).

Tasman District Council presented this information to the community via the LTCCP process. The community feedback from this indicated that further consideration needed to be given to affordability issues and other flood risk mitigation options.

1.4 Scope of Recent Work

To progress the identification of reasonably practicable options, it was agreed that the following activities were required:

- improved understanding of the river hydrology
- hydraulic modelling based on the improved hydrology
- hydraulic modelling to identify flood levels versus height of stopbanks
- geotechnical sampling of soils to improve understanding of stopbank permeability and to identify potential Borrow areas
- geotechnical assessment of potential stopbank failure modes
- geotechnical options development for refurbishment of the stopbanks
- cost analysis of refurbishment options
- assessment of risk mitigation of these options
- identification of residual risks for each option.

2 Overview of Flood Hazards

2.1 State of Existing Stopbanks

The Lower Motueka River stopbanks were constructed by cut and fill methods. The nature of materials in the stopbanks is variable. It is probable that the nearest available materials were sourced from borrow areas located adjacent to the stopbanks.

The stopbank materials appear to have been only moderately compacted, by tracking of the earthworks equipment used to build them.

The stopbanks vary in height and width; which are generally less than recommended by current best practice. The side slopes (batters) are moderately steep compared to accepted design parameters.

Test results show that there is considerable variability in soil strength and consistency along the stopbanks.

Permeability tests of the stopbanks also indicate significant variability. At the more sandy/gravelly sites, rapid infiltration of water into the stopbanks would be expected during high flood levels.

Generally the stopbanks are well away from the active river channel. In most areas where they are close to the active channel, the river banks are protected by large armour rocks. Additional protection to the stopbanks has been provided by the planting of willows and other trees between the river and the stopbank.

Although the stopbanks are in reasonable condition considering their age, they have suffered due to human and stock interference. The plan of “Areas of Stopbank Vulnerability” (attached in Appendix D) illustrates several types of vulnerability and the approximate location of these areas.

The types of vulnerability include:

- Existing (illegal?) open gravel pits close to the stopbanks and gravel pits backfilled with loose permeable materials
- trees growing in the stopbanks or in the toe areas of the stopbanks; as roots can cause damage
- erosion-prone areas where the stopbanks are close to the river channel or where the floodplain width between the stopbanks is constricted
- areas where water seepage beneath the stopbanks has been observed during previous floods
- areas where the stopbanks have been damaged by stock crossing points or by rabbit holes
- existing (illegal ?) pipelines through the stopbanks which can create pathways for flood waters
- permeable areas where there are old channels with gravels near surface on the landward side of stopbanks.

Maintenance of the stopbanks includes annual mowing and inspections, although the mowing may be done twice yearly at the discretion of the Rivers Engineer. The last detailed walkover survey and condition assessment of the stopbanks was carried out in 2005. Over the five year period, since this detailed condition assessment was carried out, there has been a marked improvement in the extent and quality of rock revetment along the edge of the river channel; however the majority of the vulnerable areas identified above still exist and have yet to be remediated. It should be noted that the funding available for river maintenance is limited.

2.2 Stopbank Failure Modes

The following potential failure modes have been determined as being critical for the existing stopbanks:

- overtopping of the stopbank resulting in soil erosion
- slope instability caused by the effect of soil saturation during flood conditions (water pressure on face)
- piping (the transportation of soil by water seepage) either through the stopbank itself or through the foundation resulting in uncontrolled seepage flows that erode the stopbank or its foundation
- roots from vegetation on the river bank either increasing the likelihood of piping or physically damaging the stopbanks if ripped out under flood conditions
- river erosion damage to either the stopbanks or their foundations
- interference with the stopbank and the river berm.

Each of the failure modes is discussed in detail in Appendix A and is summarised in Table 2-1.

2.2.1 Construction Period Risk of Failure

Options for improving flood protection include an option to rebuild (replace) the stopbanks and an option to refurbish the existing stopbanks.

Flooding during construction is a particular risk for the rebuild option as there will be little or no flood protection at the work front as it progresses. This temporarily raised level of flood risk may be mitigated to a degree by undertaking construction during the drier seasons, but this will significantly extend the build time. Providing a temporary, movable, impoundment would add considerably to the construction costs. The construction period is likely to span several years and therefore increases the exposure to risk of flooding.

Flooding during construction for the refurbishment option is less problematic as the existing stopbanks would remain in place. Localised down-cutting and consolidation of the stop-bank crest may be needed as part of consolidation of the existing banks prior to refurbishment. This would temporarily reduce the levels of protection at the work front.

Table 2-1: Motueka River Flood Control Scheme – Summary of Stopbank Failure Modes

Failure Mode	Key Issues	Possible Mitigation methods
Overtopping	• River flows spill over stopbanks at concentrated points. • Erosion of grass cover and then stopbank material at crest and down slope flank of stopbank. • Potential breach failure.	• Provide a crest profile that meets the design standard consistently along the river. • Provide a wide crest and a shallow flank slope of the stopbank. • Well grassed and maintained stopbank covering.
Saturation and instability	• The shear strength of the stopbank material is reduced when it is saturated. • Potential collapse of the down-slope flank of the stopbank, leading to a breach event.	• Broad cross section, compacted construction materials. • Well engineered, low permeability zone integrated with stopbank to prevent seepage of water. • Consistent construction technique and quality control on the materials.
Piping through stopbank	• Differential pressures across the stopbank force water to track through a preferential flow path (weak point) in the stopbank and carry out material which causes piping diameter to increase. • This leads to larger piping flows and eventual collapse of the stopbank, and breach event.	• Broad cross section, compacted construction materials. • Consistent construction technique and quality control on the materials. • Well engineered, low permeability zone integrated with stopbank to prevent seepage of water.
Piping through stopbank foundations	• Differential pressures across the stopbank force water to track through foundation material under the stopbank that is more permeable than the stopbank. • This causes foundation material to be carried out with the flow which causes piping diameter to increase. • This leads to larger piping flows and eventual collapse of the stopbank, and breach event.	• Investigation of foundation materials, removal of unsuitable materials. • Review historical anecdotal reports of seepage. • Increase subsurface flow-paths under stopbank through keys or curtain walls. • Minimise disturbance of the ground on either side of the structure. • Address gravel extraction practices on river berm • Broad cross section, compacted construction materials.

Failure Mode	Key Issues	Possible Mitigation methods
Piping through existing penetrations	• Differential pressures across the stopbank force water to track through a preferential flowpath in the stopbank due to concrete structures or pipe trenches that have regions of poor compaction. • Material is carried out causing piping diameter to increase. • This leads to larger piping flows and eventual collapse of the stopbank, and breach event.	• Identification of penetration structures in stopbank. • Remove structures or reduce risk of piping by use of filters and drainage layers. • Rebuild parts of the stopbank identified as poor.
Foundation "Blow up"	• Differential pressures across the stopbank force water through permeable foundation material which then acts on a weak but tight layer of foundation material near the surface on the down-slope side of the stopbank. • When hydrostatic forces exceed the restrained forces of the foundation the ground gives way suddenly. • This leads to large piping flows and eventual collapse of the stopbank, and breach event.	• Investigation of foundation materials. • Reduce risk of blow-up by installing subsoil drainage to control flows. • Review historical anecdotal evidence. • Increase soil restraining weights on the downslope side of stopbank. • Broad cross section, compacted construction materials.
River erosion attack	• High velocity flows in the bed and channel of the river erode the flood berm and undermine the stopbank foundations. • This leads to collapse of the stopbank and a breach event.	• River bed training and groynes, rock armouring of banks. • Maintain a buffer between channel and stopbank. • Maintain good grass cover on stopbanks.
Uncontrolled vegetation	• Trees and shrubs that are allowed to grow in or beside the stopbank structure develop root systems that penetrate the stopbank. • This leads to potential piping defects. • Large trees can be uprooted during a storm event causing damage to the stopbank structure.	• Annual inspection and maintenance of vegetation along the stopbank and flood berm. • Removal of identified problem vegetation.
Interference with stopbank	• Man-made development on the stopbank or close to the two sides of the structure can reduce the stability of the stopbank structure and foundations, or introduce preferential seepage paths. • Wild animal activity, such as burrowing, can damage the stopbank structure. • This can lead to weak points in the stopbank and breach development.	• Restrictions on activities near a structure. • Repair of damage to structure. • Inspection of conditions of stopbank. • Legal and town planning rules. • Active compliance monitoring.

2.2.2 Mapping of Failure Modes

Drawings have been prepared to illustrate the areas that are considered most at risk from each of the potential failure modes. See Appendix D for A3 size versions of Figure 2 and Figure 3.

- The "Stopbank Raise Crest Summary" plan illustrates those sections that are predicted to be overtopped during the "design standard" flood. All coloured areas are predicted to be overtopped in a 1% AEP rainfall event (with 2090 Climate change allowance).

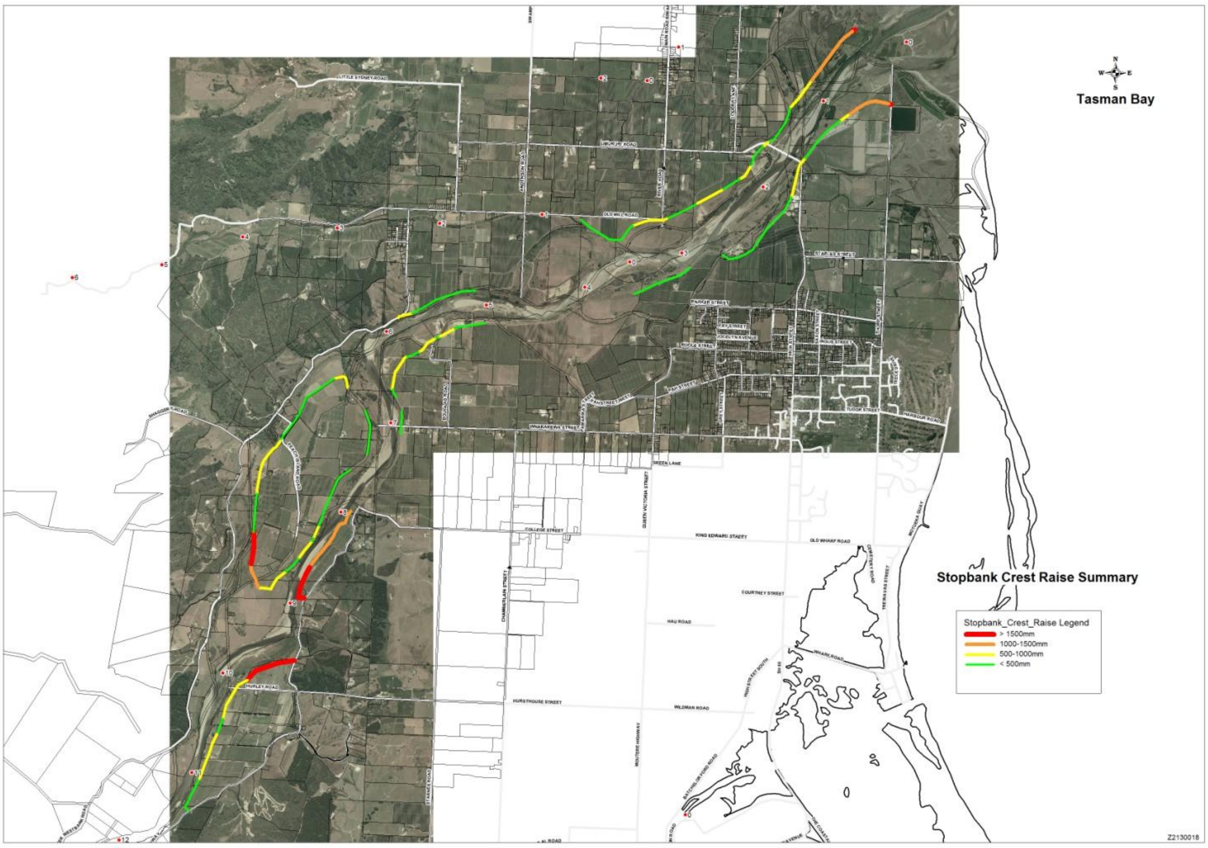


Figure 2-1: Stopbank crest raise summary

- The "Areas of Stopbank Vulnerability" plan shows a range of geotechnical information which correlates to risks around piping of the foundation. Historical areas where piping has occurred are illustrated (depths of silt that cap the underlying more permeable silts and gravels are shown in Appendix E). Areas where the silt cap is thinnest will present the highest risk of foundation piping.
- The "Areas of Stopbank Vulnerability" plan shows the location of known penetrations through the stopbank. At each of these locations there is an increased risk of seepage and piping failure.
- The "Areas of Stopbank Vulnerability" plan shows areas of high river erosion risk, areas of vegetation that affect the stopbanks and areas of gravel extraction that are adjacent to the stopbank and have some negative impact on its performance.

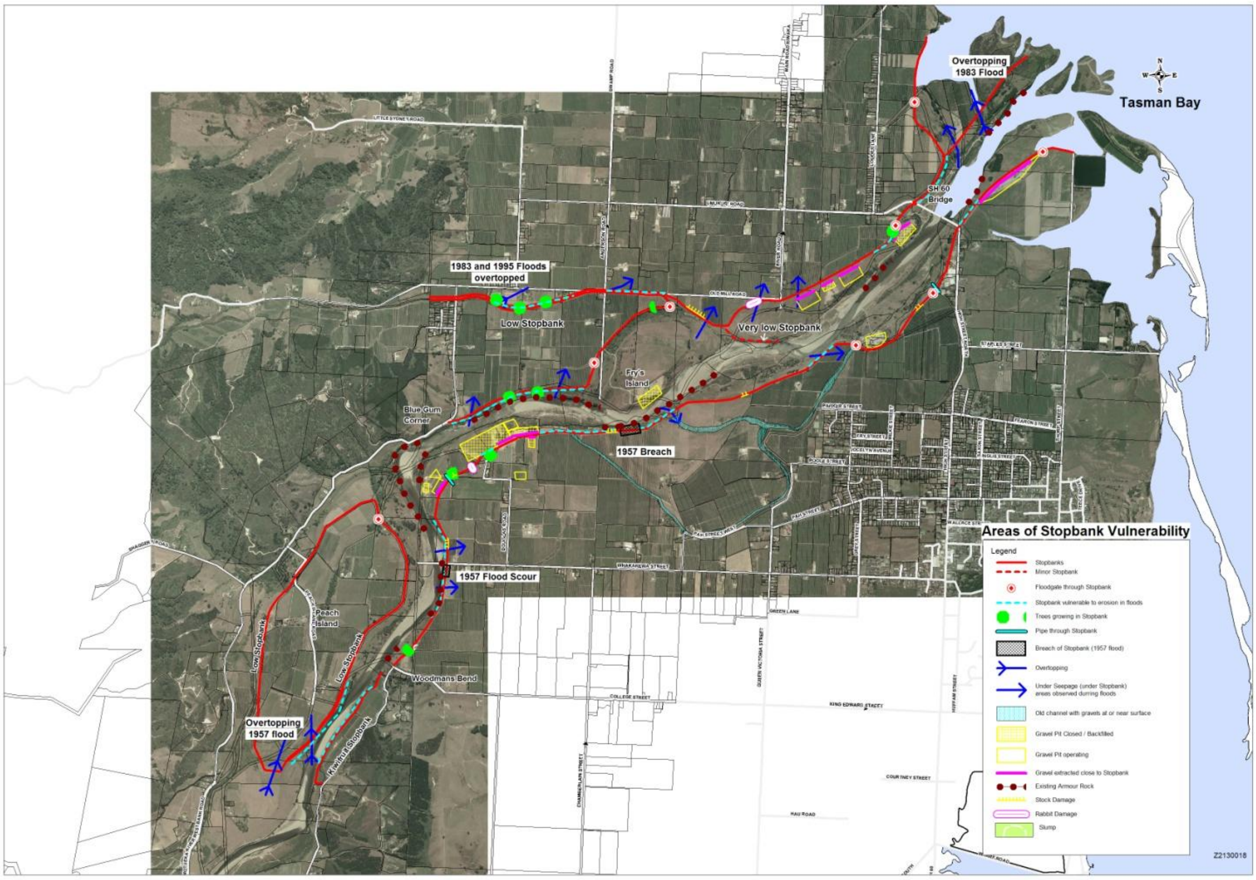


Figure 2-2: Areas of Stopbank vulnerability

3 The Short-Listing of Flood Management Options

Five options were developed and short-listed for assessment using the Multi Criteria Analysis (MCA) process. This is described in the 21 January 2011 MWH report: "*Identification of Reasonably Practicable Flood Management Options Lower Motueka River"*

Following the Stage B public consultation and the MCA workshop, it was decided to refine the rebuild and refurbishment options in this report, and to consider the potential benefits of gravel removal.

The economic analysis allows a more quantitative assessment of scheme costs set against the reduction in risk achieved by the options. It provides a rational basis for establishing an acceptable balance between level of protection and the acceptability of the residual risks weighed against the option costs. This approach is consistent with the "Managing Flood Risk" Standard (NZS9401:2008) which puts the onus on the community to decide what residual risks it is prepared to accept.

The key to selecting the most appropriate option is to balance the cost of any upgrade against the (value of) residual risks that remain following the upgrade.

4 Development of Options

4.1 Design Standard Selected for Option Comparison Purposes

The “design standard” rainfall event used for comparison purposes is derived from good industry practice.

This includes the following sources:

- At the MCA workshop, Tasman District Council decided that the options should be considered for a 1% AEP rainfall event level of protection with 500mm freeboard.
- The Tasman District Council Engineering Standard (section 7.2.2) requires “taking into account possible changes in rainfall patterns in the future”.
- The Ministry for the Environment has issued a document “Preparing for Climate Change – Guide for Local Government”.
- IPENZ has run seminar series in the past about incorporating climate change risks into best practice: *IPENZ Incorporating Climate Change into Infrastructure Planning and Design.*

It is therefore considered appropriate to allow for climate change in the adopted rainfall event for the purpose of assessing options.

The rainfall event modelled is the 1% AEP (100 year ARI), with climate change allowance to 2090 projections, and 500mm freeboard.

Cumulative Probability of a 1% AEP Flood for Varying “Life” of Stopbanks		
“Life” of Stopbank (Years)	% Probability of Flood Event During “Life”	Chance of Flood Event During “Life”
1	1%	1 in 100
10	10%	1 in 10
20	18%	1 in 5.5
30	26%	1 in 3.8
40	33%	1 in 3.0
50	39%	1 in 2.6
60	45%	1 in 2.2
70	51%	1 in 2.0
80	55%	1 in 1.8
90	60%	1 in 1.7
100	63%	1 in 1.6

4.1.1 Freeboard

Free board is a height added to the theoretically calculated flood water level. The allowance for freeboard is to account for a variety of additional effects that are not explicitly included in the hydraulic assessment of flood levels, such as:

- surface waves caused by wind or structures
- super-elevation on bends
- wear or settlement of the stopbank crest between maintenance works.

This freeboard also includes an allowance for imprecision in the estimates of flood level. It should not be thought of as surplus stopbank height above the level of the "design standard" flood. A 500mm allowance for freeboard has been made, which is considered to be minimum current industry practice. Any future detailed design must consider this carefully. The original design used a 600mm (or the imperial two feet) allowance.

4.2 Extent of Stopbanks Considered in Assessment

The community feedback was clear that more affordable flood control measures should be identified. One way to reduce costs is to reduce the extent of work. There are zones of existing stopbanks which offer the opportunity to shorten the length of stopbanks needing work.

- The ring stopbank around Peach Island. The Peach Island stopbanks are lower than the current stopbank system and the island will flood under high flow conditions to provide attenuation to downstream sections of the river. Procedures for evacuation already exist to address the risks of flooding for the residents of the island.
- The Brooklyn Stream stopbanks. Analysis indicates that these stopbanks are not expected to be overtopped during the "design standard" event in the Motueka River, and there is limited space in which to implement any refurbishment works. There will still be the risk of other failure modes for the Brooklyn banks, especially during Brooklyn stream extreme flood events. [Storms in the Brooklyn and Little Sydney catchments will still be a flooding risk to the Riwaka area that will need to be considered separately].
- Hurley and Kiwifruit stopbanks. Analysis indicates that the Hurley's stopbank height is currently at a 50 year ARI design standard and protects a single property and orchard. The Kiwifruit stopbank also protects a single property, but it is approximately a 20 year ARI design standard height and would need a crest raise of 1m to meet the overall scheme standard.

This reduction of the extent of work will need to be balanced against the chosen rating model (who benefits and who pays).

4.3 Common Maintenance Issues

The flood protection options that are considered for evaluation are described in the following sections. Implicit to the stopbank upgrade options is an assumption that Tasman District Council will continue to carry out a maintenance and inspection programme of whichever option is selected. This maintenance and inspection programme will include the following tasks to maintain the community's flood protection asset:

- create a river maintenance strategy document
- education of landowners to impart the importance of the asset and its well being
- monitoring and correction of landowner activities adjacent to the asset and across the asset
- inspection of the condition of the flood protection asset including the stopbank, flood berm and river channel
- maintenance of the grass cover over the stopbank and berms through cooperation with adjacent landowners
- maintenance of flood warning systems and lifelines emergency networks
- maintenance of existing storm-water culverts and pipelines running under the stopbank
- clarification of legal and resource management planning provisions, regulations, rules and responsibilities
- consultation with landowners and gravel extraction operations
- consideration to improve the effectiveness of the River Care group
- include berm areas outside the stopbanks in the maintenance strategy.

Any flood control scheme should include the adoption of improved management of the river channel and river berm as the initial line of protection of the stopbank. It is known that a previous failure of the stopbank (1957) was due to river erosion, highlighting the importance of river protection to the safety of the stopbank. Based on experience with the river system, the following sections have been identified as benefitting from additional erosion protection works. Extra funding will be required to address these items:

- Blue Gum Corner on left bank downstream for 1200m
- section starting 200m upstream of SH60 Bridge and then 500m upstream on left bank
- from SH 60 Bridge downstream on left and right banks over a distance of approximately 400m
- from SH60 Bridge upstream for 200m on right bank
- section in front of Motueka Metal yard on right bank over a distance of approximately 400m
- one kilometre section opposite Fry's Island on right bank (breached section 1957)
- area from Corrie – Johnson pit on right bank to Johnston Pond area (approx 600m)
- from Whakarewa Street on right bank to Woodman's corner (approx 1100m).

During inspection of the stopbank it was noted that there were a number of gravel extraction pits in close proximity to the stopbanks. These pits introduce the potential for increased river erosion of the stopbank due to a reduction in berm width and increased seepage beneath the stopbank due to increased exposure of permeable gravels to flood waters. This situation could be improved by better management of the location and backfilling of these pits. The upgrade works would involve:

- enhanced river protection works
- removal of any vegetation from the berm with roots that may grow under the stopbank
- maintenance of grass cover on the stopbanks
- better control of gravel extraction on the river berm
- better control of backfilling.

The current maintenance budget is about $200,000 per year. An additional allowance of at least $100,000 per year for maintenance is suggested by the Rivers Engineer. There will be an initial extra amount required for the creation of the strategy document.

4.4 Option Concept Drawings

A drawing (SB01) showing a conceptual cross-section of the options is included in the Appendix D.

4.5 The Rebuild Option

The rebuild option involves demolition of the existing stopbank cross section and replacement with new stopbank material sourced from the existing stopbanks, river berms and imported from further afield. The stopbank crest profile would match the design standard including 500mm freeboard and the cross section would meet current best practice side slopes and foundation treatment, and be compacted into place using modern equipment and techniques.

The entire existing scheme includes the Peach Island, Brooklyn, Kiwifruit and Hurley stopbanks as well as the stopbanks downstream of Woodman's Bend. For the comparative purposes of this report, we have considered a reduced length of stopbanks to be rebuilt, specifically excluding Peach Island and Brooklyn stopbanks (which would be left as they are).

4.6 The Refurbish Option

Compared to the rebuilding option, refurbishment concentrates on leaving as much of the existing stopbank as possible intact.

A key requirement of the stopbank system is to be of sufficient height to contain the "design standard" flood. Analysis indicates that in many locations the stopbank would currently be overtopped by the "design standard" flood.

Increasing the height of the existing stopbank to contain the "design standard" flood is not straightforward. The crest width is typically 2m or less and is only just sufficient for light vehicle access. There is insufficient width to simply place additional fill on the stopbank to increase its height. On the river side there is a wide berm, before reaching the active river channel, and raising the stopbank on this side appears comparatively straightforward (there are possibly land ownership issues that may need to be resolved). Any fill placed on the river side has the advantage of providing an additional barrier to saturation of the stopbank and enhancing stopbank stability.

Investigations have been undertaken on the river berm to identify potential sources of material for stopbank construction. A variable thickness of silt (river flood deposits) has been identified. This silt appears to be a viable source of low permeability fill; to both raise the stopbank and to achieve a low permeability blanket on the upstream face.

The refurbishment would occur along the entire length of the stopbanks except for.

- The ring stopbank around Peach Island. The Peach Island stopbanks are lower than the current stopbank system and the island will flood under high flow conditions to provide attenuation to downstream sections of the river. Procedures for evacuation already exist to address the risks of flooding for the residents of the island.
- The Brooklyn Stream stopbanks. Flood hydraulic model analysis indicates that these stopbanks are not expected to be overtopped during the "design standard" event, and there is limited space in which to implement any refurbishment works.
- Hurley and Kiwifruit stopbanks. Analysis indicates that the Hurley's stopbank height is currently at a 50 year ARI design standard and protects a single property and orchard. The Kiwifruit stopbank also protects a single property, but it is approximately a 20 year ARI design standard height and would need a crest raise of 1m to meet the overall scheme standard.

The refurbishment concept is illustrated on drawing SB02 (Appendix D) and allows the stopbank to be raised as required to contain the predicted 1% AEP rainfall event (including climate change) with 500mm of freeboard. In sections where no stopbank raising is required to achieve the “design standard” level, a blanket of silt would be placed up to the existing crest level; increasing the mass of the stopbank and reducing permeability of the river side face. The existing stopbanks are known to be poorly compacted and of variable material. Therefore, refurbishment does not offer as much long term security as the rebuild option.

4.7 Partial Refurbishment Sub Options

4.7.1 Introduction

To identify where investments in upgrades would bring the most benefit, it is necessary to understand which areas of the stopbank represent the most risk to people and property (if failure was to occur). The following Figure 4-1 illustrates the relative damage potential of different segments of the stopbank.

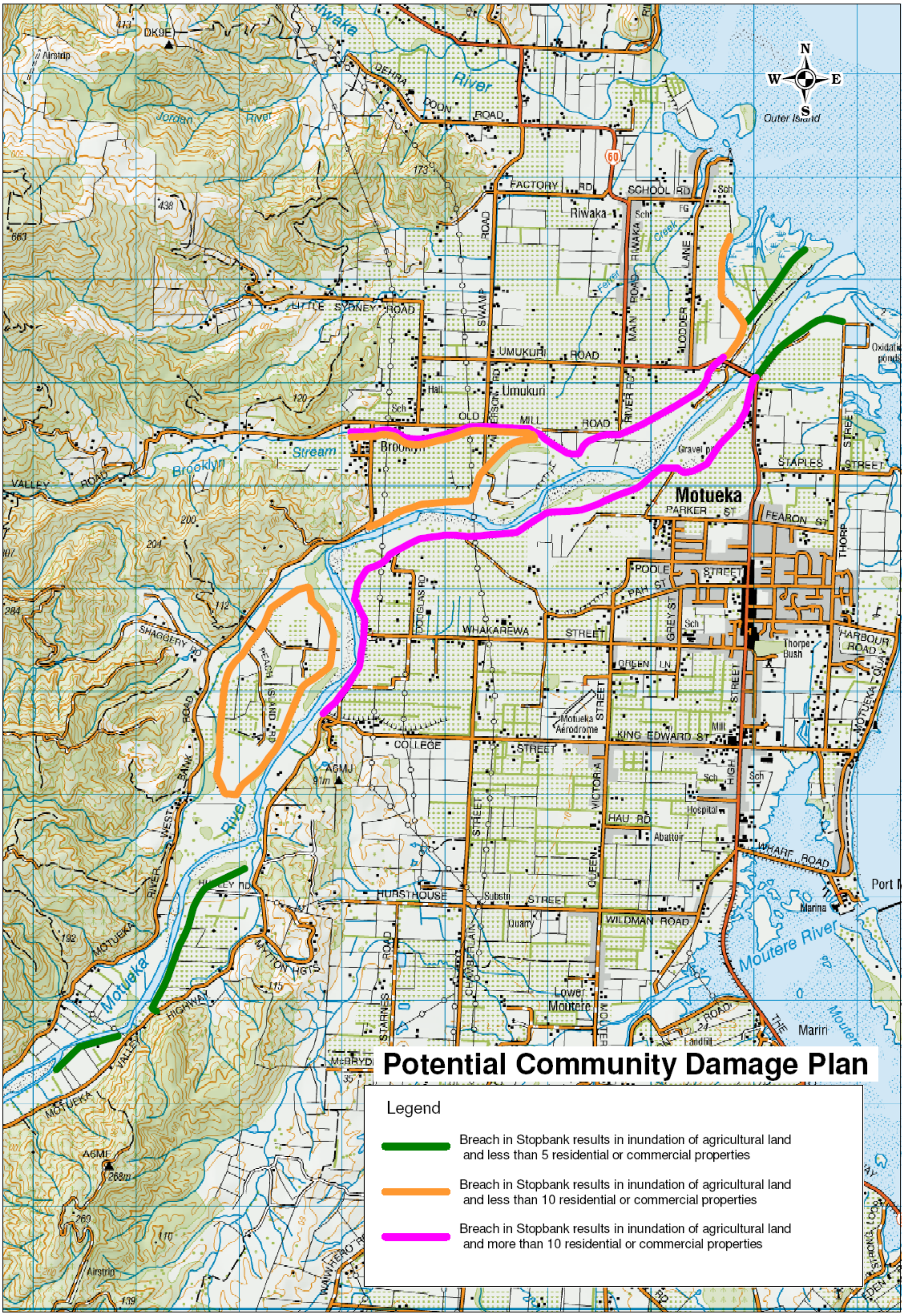


Figure 4-1: Potential Community Damage Plan

It can be seen that the stretches of stopbank that protect the highest value assets are.

- Between College Street and the State Highway bridge on the right bank where the stopbank protects parts of Motueka (a distance of 6,900m).
- Between River Road and the State Highway on the left bank where the stopbank protects Riwaka (a distance of 2,500m).
- The left bank of Brooklyn Stream. However, as noted in section 4.2, it is not intended to refurbish these stopbanks as part of the Motueka River flood control. It will need to be considered separately to address flooding in the Brooklyn catchment.

Improvements to the above-mentioned sections of stopbank will have the greatest impact on reducing the overall level of risk to people and property protected by the stopbank. These options therefore concentrate on upgrades to improve the first two bullet point sections of stopbank.

With respect to the remaining sections of the stopbank the following comments are made.

- The upper right bank stopbank between Ch 11500 and 15000 (known as the Hurley and Kiwifruit stopbanks) protect only small areas of agricultural land.
- The Peach Island stopbanks are lower than the current stopbank system and the island will flood under high flow conditions to provide attenuation to downstream sections of the river. Procedures for evacuation already exist to address the risks of flooding for the residents of the island.
- Downstream of the State Highway Bridge the stopbanks have been overtopped previously without significant damage to infrastructure.
- High tide influences river flow up to the vicinity of the State Highway Bridge.

The partial refurbishment could be configured in a number of different ways, as discussed in the following sections. Common to each of the options are the improvements to erosion protection and river management discussed in section 4.3, but restricted to the critical lengths identified above.

4.7.2 Options with Freeboard of 500mm to "Design Standard" Flood

Two sub options have been considered, namely.

- Option B1 – place fill on river side face of stopbank only where an increase in freeboard is required.
- Option B2 – place fill on river side face of stopbank along entire length of 'critical' stopbank.

Under these options the 'critical' section of stopbank would be raised as required to achieve a minimum freeboard of 500mm above the "design standard" flood. This would be achieved by the placement of fill on the river side face of the stopbank as described in section 4.6.

The advantage of Option B2 is that it addresses both the potential for overtopping and reduces the likelihood of instability of the stopbank, albeit at a higher cost than B1. Option B1 involves the upgrade of approximately 6,200 m of stopbank, while Option B2 involves 9,400 m of stopbank. Discussion of the trade off in cost versus risk is presented in section 8.

4.8 Maintaining the Status Quo

For the status quo option, the work proposed on the existing stopbanks would be minimal. The stopbank crest profile and cross section would remain the same as in the existing case. Ongoing Council maintenance and inspection operations would seek to minimise the deterioration of the condition of the stopbanks through maintenance of grass surfacing, river bed operations to prevent berms being eroded away, and the regulation of human activities around the flood protection asset.

This option has no significant capital expenditure above programmed annual asset maintenance costs.

4.9 Secondary Stopbanks Option

One way to address flood control is to accept the risk of failure of the existing stopbanks, and to build new secondary banks further away to contain any floodwater.

The further away from the existing banks, the lower the required height of secondary bank to contain floodwater. This means lower construction cost, but more land would be flooded.

To ensure that floodwater could drain away (following old river courses) there would be the need for ongoing management to ensure that structures and land forms were not changed in the secondary containment area. This could be costly and difficult to implement.

The cost of land purchase and complex negotiations required to select an alignment make this an unfavourable option.

The original purpose of the existing stopbanks was to protect adjoining horticultural land. This purpose has not changed although different crops are now grown. Allowing floodwater to inundate this land would be contrary to this purpose, and would reduce the number of beneficiaries to share the cost of the flood control works.

It is therefore considered that this option should not be progressed any further.

The Multi Criteria Analysis came to the same conclusion (Motueka River Flood Management Study report October 2010).

4.10 Spillway Option

A key requirement of the stopbank system is to be of sufficient height to contain the “design standard” flood. Analysis indicates that in many locations the stopbank would currently be overtopped by the “design standard” flood.

Consideration has been given to providing spillways that limit the flood level and thereby prevent overtopping of the stopbank, however, all of the paths for spillway involve some inundation of properties.

Allowing floodwater to inundate this land would reduce the number of beneficiaries to share the cost of the flood control works.

To ensure that floodwater could drain away (following old river courses) there would be the need for ongoing management to ensure that structures and land forms were not changed in the secondary containment area. This could be costly and difficult to implement.

It is therefore considered that this option should not be progressed any further.

The Multi Criteria Analysis came to the same conclusion (Motueka River Flood Management Study report October 2010).

4.11 Impact of Refurbishment on Failure Modes

The impact of the stopbank upgrades on each of the failure modes identified in Section 2.2 is discussed in the following sections.

4.11.1 Slope Instability

The critical consideration for stability of the stopbank is the degree of saturation during a flood event. Placement of fill on the river side of the stopbank has the potential to significantly reduce the likelihood of slope instability by preventing saturation of the existing stopbank fill, even if the existing fill has comparatively high permeability.

In sections that do not require an increase in height, alternative methodologies could be adopted to restrict the flow of water through the stopbank and therefore improve its stability. Techniques such as upstream (river side) geo-membrane liners, or in-situ mixing of grout with the existing stopbank material, could be effective and should be considered during detailed design phases, if this option is progressed.

4.11.2 Overtopping of Stopbank

The raising of the stopbank height at selected locations will directly address risks associated with overtopping the stopbank. The level of protection provided will depend on the amount of freeboard allowance included in the assessment of required stopbank crest levels. For the purposes of this report 500mm freeboard has been allowed.

4.11.3 Piping through Stopbank

The potential for piping through the stopbank is directly related to the potential for saturation of the stopbank fill. Sections of stopbank upgraded by the placement of low permeability fill on the river side will therefore have a significantly reduced risk of piping failure within the stopbank. During construction, stripping of topsoil on the existing stopbanks will allow inspection of the formed face and identification of any gravel lenses or other high permeability zones that may act as initiators of piping failures. These can be addressed on a case by case basis to further increase the resistance of the stopbank to piping.

4.11.4 Piping along Penetrations

Piping along penetrations through the stopbank can be addressed in the refurbishment by specific works at these locations to locally excavate the stopbank and place filter and drainage systems.

4.11.5 Piping through Foundations and Foundation “blow-up”

Piping and foundation blowup problems are more difficult to address. Some reduction in risk will occur as a result of being able to make sure that the new fill is tied into the silt on the river berm. This will prevent very short seepage paths directly beneath the stopbank which have a higher chance of initiating piping and foundation ‘blowup’ issues.

The earlier rebuild proposal included allowance for coarse gravel filter trenches along the landward toe of the stopbanks. These would reduce upward pressure gradients due to high foundation seepage flows.
For this report, these trenches have not been allowed for in the rebuild or refurbish options. The proposed rebuild concept should have better resistance to foundation “blow-up” than the refurbish options which leave the existing stopbanks in place.

There will always be a level of residual risk associated with seepage through the stopbank foundation. Attempts to prevent the flow of water would be very expensive over the extent of the stopbanks and may be ineffective given the thick layers of permeable gravel likely to underlie the stopbanks. The residual risks after replacement or refurbishment of the stopbanks is likely to be higher for this failure mode than the other failure modes identified in this study.

4.12 River Management

The proposed improvements to the river management regime provide a number of benefits to the security of the stopbank.

Additional river rock protection reduces the potential for erosion of the river berm during a large flood event. If the berm is sufficiently eroded then direct river attack on the stopbank is possible. The refurbishment aims to minimise these risks by ensuring the maintenance of good vegetation cover on the stopbanks. The placement of additional fill on the river side of the stopbank will also provide a greater thickness of material that would need to be eroded before failure occurred (than the existing stopbank system provides).

4.12.1 Interference with Existing Stopbank

As part of the refurbishment, it is proposed to increase annual monitoring of the stopbank system to better detect areas of interference with the stopbank. More formal maintenance requirements would be instigated to address any deficiencies identified during the annual inspections. Education of landowners along the stopbank would also be undertaken to attempt to reduce the amount of negative interference of the stopbank system. An increased annual stopbanks maintenance budget has been recommended for this work.
These items should form part of a River Maintenance Strategy document.

Table 4-1: Summary of Impacts of Upgrade Options on Failure Modes

FLOOD PROTECTION Failure Modes	Upgrade Options						
	Option A Rebuild 15.6km	Option B Refurbish 15.6km	Option B2 Partial Refurbish entire length of critical reaches 500mm freeboard	Option C Secondary Stopbanks	Option B1 Partial Refurbish in topped up areas only 500mm freeboard	Option D Spillways	Status Quo
Overtopping	Design standard	Design standard	Design Standard	Below design standard	Design Standard	Upstream of spillway as existing; downstream improved	Existing – below design standard
Stopbank stability	Highly improved	Improved	Improved	Existing	Partly improved	ditto	Existing
Piping – stopbank	Highly improved	Improved	Improved	Existing	Partly improved	ditto	Existing
Piping – foundation	Highly improved	Improved	Improved	Existing	Partly improved	ditto	Existing
Piping – penetrations	Highly improved	Improved	Improved	Existing	Partly improved	ditto	Existing
Foundation 'Blow-up'	Improved	Some improvement	Existing	Existing	Existing	ditto	Existing
River erosion attack	Improved	Improved	Improved	Controlled through asset maintenance	Improved	Controlled through asset maintenance	Controlled through asset maintenance
Uncontrolled vegetation	Improved	Improved	Controlled through asset maintenance	Controlled through asset maintenance	Controlled through asset maintenance	Controlled through asset maintenance	Controlled through asset maintenance
Interference with stopbank	Rectified	Controlled through asset maintenance	Controlled through asset maintenance	Controlled through asset maintenance	Controlled through asset maintenance	Controlled through asset maintenance	Controlled through asset maintenance

[Note: For the purposes of reducing the complexity of the cost benefit analysis, any version of partial rebuild has not been considered.]

4.13 Catchment Management

Public consultation meetings regarding the Lower Motueka River flood protection scheme options since 2009 have raised a number of questions and suggestions for Tasman District Council to consider. Two questions are considered further in this report:

<u>Item 1: Catchment Management Improvements to avoid the use of flood control measures at the mouth.</u>

A segment of the public expressed a view that if the catchment was returned to fully native species, then there would be significantly reduced flooding risk. Also, that the catchment could be managed to avoid the need for flood protection works.

We have researched and summarised information on the Motueka River catchment from the most comprehensive public domain source presented by Landcare Research. This is a research project developed by Landcare Research Ltd, Cawthron Institute and Tasman District Council.

Reference 1:
http://icm.landcareresearch.co.nz/research/research.asp?theme_id=5&research_id=52

Land use maps are available on the Landcare Motueka River Integrated Catchment Management ICM website (Reference 1), which show that the 2,170 km^2 Motueka Catchment today is largely rural.

Current vegetation in the catchment is dominated by native bush (35%) and exotic (25%) forest with pastoral grassland (19%), scrub (12%) and tussock grasslands (7%) [from Reference 1]. This indicates that approximately 54% of the catchment is currently native species (tussock, native forest and scrub).

The total percentage of native bush, exotic forestry, pastoral grassland, scrub and tussock land is 98%.

Tasman District Council Engineering Standards and Policies assign grass cover and pasture a 20% to 40% rainfall to runoff conversion; and bush and scrub cover a 15% to 35% rainfall to runoff conversion, depending upon soil properties. The remainder of the rainfall is caught in the soil, vegetation and natural storage in the land form. The Motueka River catchment is predominantly in the lower range of rainfall to runoff conversions and therefore cannot be improved much.

The speed of response to rainfall is likely to be lower on land with trees and bush, and slightly faster with grassland. The speed of response is most influenced by steepness of the topography and the speed of the flood flow along the major river reaches and through the Motueka River. The timing of peak flows from the main sub-catchments also influences the peak flow and response time at the mouth.

Artificial dams in the catchment could be constructed to modify the river response, but the costs of this approach would be very large.

In conclusion, the Motueka River catchment is already a low-response catchment due to the high proportion of green areas. Tasman District Council catchment management efforts would not be able to significantly reduce the size or speed of the flood response through changes in land usage or vegetation management.

4.14 Removal of Gravel

The second idea from recent public consultation that needs to be addressed is:

<u>Item 2: Determine the available reduction in design stopbank profile and cost savings if gravel was extracted from the berms of the Lower Motueka River and the flood way capacity between the stopbanks was improved.</u>

To assess the potential effect of removing gravel we identified zones of usually dry "beaches" of gravel downstream of Peach Island. These zones are indicated on the marked-up aerial photographs in Appendix D. We then modelled the Lower Motueka River in river calculation software (MIKE11) using altered cross sections reflecting the removal of gravel from the existing gravel berms between Woodman's Bend and the river mouth.

We have calculated the required design stopbank crest profiles along the Lower Motueka River for the existing gravel case, the 1m deep scrape case and the 2m deep scrape case. Each case provides a different flood top water profile which corresponds to the capacity of the flood channel between the stopbanks (see Appendix A). In this work, the larger the amount of gravel extracted the lower the flood top water level and the lower the required design stopbank crest profile. The lower stopbank profile reduces the volume of the stopbank required which in turn reduces the construction cost estimate.

We have carried out three tasks:

1. Run the flood hydraulic model for the three scenarios (existing river cross sections, 1m deep scrape, 2m deep scrape)
2. Calculated the volume of gravel extracted in the two scrape scenarios
3. Calculated the volume of stopbank earthworks for the three scenarios between Woodman's Bend and the river mouth, both sides of the river.

Also, we have modelled gravel removal from the area downstream of the State Highway 60 Bridge, and confirmed that any flood capacity benefit will be negated by high tide. It would therefore be a more efficient use of money to remove gravel further upstream. Also, the area below the bridge is in the Coastal Marine Area (Tasman Resource Management Plan - TRMP) which restricts activities like earthworks.

The positive effect of gravel removal is to lower the top water level by approximately 300mm for each 1m depth of gravel removed. However, this positive benefit is not achieved along the full length of stopbanks because the river is too narrow in some places to allow gravel removal. The reduction in top water level will reduce the risk of over-topping in large events. It will also reduce the pressure on the stopbanks in a large event. It would avoid or reduce the current need to quarry outside the stopbanks. It could be undertaken in conjunction with sourcing materials (eg. silt) for stopbank upgrades, if that option is implemented. Downstream of the SH60 bridge gravel removal would be ineffective both sides of high tide; therefore it would be a more efficient use of money or effort to implement upstream of the bridge.

Gravel removal does not avoid the risk of other modes of stopbank failure, and does not address the areas of stopbanks that may be weak. If undertaken too closely to the stopbanks it may increase the risk of under-seepage, or stopbank collapse. It reduces the buffer zone of natural protection to the foundations of the stopbanks. Care would need to be taken to avoid undermining existing rock protection works or the stopbanks.

It may not be a short term project as a large volume of gravel would need to be stored/stockpiled / banked up somewhere nearby. A more likely scenario is for removal to be spread over a few years; at a rate that suits gravel usage. This timeframe would delay flood protection benefits.

It will not be a permanent solution because gravels move downstream in larger river flow events; ongoing removal works will be required, meaning ongoing costs.

Gravel removal has some merit as a concept to improve flood capacity, but there are significant issues to be overcome, such as.

- Most of the land is privately owned (issues with access, royalties etc.)
- Water Conservation (Motueka River) Order 2004.
  Resource consent (New "Part 4" Tasman Resource Management Plan provisions will need to be considered).
- Avoiding the removal of trees shading the river (for water temperature control – ecology reasons).

Joseph Thomas (Tasman District Council ground water scientist) has advised that there would be no adverse effect on water table if gravel is only removed from the higher ("dry") gravel beaches. That is, gravel should not be removed from the active channel (which would affect groundwater).

It is concluded that the benefits of gravel extraction will not be sufficient avoid the need to replace or improve the existing stopbanks. It does appear that gravel removal could be investigated further as part of good river maintenance; in conjunction with the sourcing of materials for any rebuild or refurbishment of the stopbanks.

Removal of gravel is not a stand-alone option to provide flood control. The reduction in top flood water level is not sufficient to avoid the stopbanks being required. Therefore, the condition of the stopbanks must still be addressed to protect the community.

4.15 Conclusions from Hydraulic Modelling

The following conclusions from hydraulic modelling of flood flow in the Motueka River channel and berms, and in the Motueka Plains, are made.

- Computer modelling of catchment runoff and flood flow hydraulics indicate the required stopbank crest level profiles that will meet the design standard, and which will reduce the risk of overtopping failure of the banks.
- Damage as a result of stopbank breach scenario is the most severe case, compared to overtopping.
- Relocating stopbanks on new alignments is likely to be less effective (and more expensive) than stopbanks on existing alignments to meet the design standard.

It has been suggested that digging out the berm area ("gravel removal") may provide more flood capacity. To assess this, we have modelled two depths of berm earthworks modifications. The results show that the reductions in top water level reduce the risk of overtopping in some locations (but not everywhere). The reduction in top water level would not be significant enough to reduce pressure on the existing stopbanks (on average 300mm for 1m of gravel depth removed). In the same model assessment, results indicated that the channel top water level may be increased in some other parts of the river. This indicates that channel and berm changes tend to shift problems downstream and would need to be managed over long lengths of the river, involving large earthworks projects. Gravel removal downstream of the State Highway 60 Bridge only has benefit during low tide; at high tide the effort is negated. This is not a stand-alone flood control option.

- Berm borrow sites for stopbanks would provide a small benefit to hydraulics in addition to the convenience of construction materials sourced from near at hand.
- Berm gravel removal works present the issue of disposal or storage of large material volumes in suitable locations; and resource consent issues regarding the environment and groundwater recharge.
- Floods during construction phase have not been modelled.

5 Development of Options Costs

NOTE: these costs are INDICATIVE only.

5.1 Cost Estimate Assumptions

The estimates below exclude the following items:

- Environment Commissioners, Hearings and other court costs
- disruption to businesses
- consultation
- land costs
- compensations
- breaches of consent
- unknown cultural and heritage, legacy issues
- pump stations on landward side of stopbanks
- landscaping or enhancement
- importing materials from outside the river berms
- allowance for increased river maintenance (recommended by Rivers Engineer to be at least $100,000 extra per year)
- Changes to the State Highway 60 Bridge.

The estimates for construction of earthworks include for:

- vegetation removal,
- topsoil management
- reinstatement of grass, soil, fencing, drive-over access ways, utilities
- foundation preparation to 1 metre depth
- sourcing, transporting and installation of stopbank materials.

5.2 Rebuild Options

Two rebuild options are given:

1) the total scheme length of stopbank
2) the stopbank from Woodman's Bend to the mouth on both sides of the Motueka River, excluding Peach Island and Brooklyn Stream. For the (later) cost benefit analysis section of this comparative report, we have not assessed partial rebuild as an option (to avoid multiple sub options which may be confusing).

The 2011 cost estimates are based on work done in the 2008 Preliminary Design report and revised in the light of latest floodplain and river analysis. The rebuild option considered for the 2008 estimation was a stopbank with a 4m wide crest and 1 vertical to 2.0 horizontal side slopes on town side and 1:2.5 side slopes on the river side. For 2011 we have used 1 vertical to 2.5 horizontal side slopes on residential side and 1:3 side slopes on the river side.

The cost estimate for the complete rebuild of the Motueka scheme as set out in the 2008 Preliminary Design Report was $17.3 million including Peach Island, Brooklyn Stream, Hurleys and Kiwifruit stopbanks, and $300,000 for land purchase. No work was allowed for at the SH60 Bridge as part of the option upgrade.

The assessed length of the "full" scheme is 15.6km as the Peach Island and Brooklyn Stream stopbank reaches have been excluded from the scheme to focus on critical reaches. The Peach Island stopbank is currently designed to overtop prior to the 1% AEP (+2090 climate change) design event and would need to be upgraded to a sub-design level that has not yet been determined. This can be developed outside the scope of the Lower Motueka River Flood Protection Scheme works. The Brooklyn Stream stopbanks cannot physically be upgraded without a considerable land purchase on the landward sides of the stopbanks, and they cannot be built into the stream channel without reducing capacity, so these stopbanks were left out of the scheme as requiring further work to improve the definition of the scope of upgrade works.

Total length of stopbank upgraded = 15.6km. Rebuild stopbank to 4m crest, 1 to 3 wet face, 1 to 2.5 dry face over lower Motueka River left/right banks.

Table 5-1: Estimated Cost of Rebuild – "Full" Scheme

Preliminary Design Option				
Item	**Unit**	**Quantity**	**Rate**	**Cost (excl GST)**
River Erosion Protection	m	5400	340	$1,836,000
Improved River Berm Management	LS	1	250,000	$250,000
Upgrade of Stopbank Penetrations	No	8	30,000	$240,000
Under-Seepage Control	m	500	200	$100,000
Stopbank Rebuild	m^3	350,000	15	$5,250,000
Subtotal				*$7,676,000*
Preliminary and General	%	5		$383,800
Resource Consent Application	%	2		$153,520
Professional Fees	%	15		$1,151,400
Contingency	%	30		$2,302,800
Total				**$11,667,520**

The rebuild of ‘critical’ reaches of the Motueka River main banks (left and right) that provide the most protection to the townships of Motueka and Riwaka, and to the horticultural land on the Motueka and Riwaka Plains, is downstream of Woodman’s Bend and is estimated below.

The length of stopbank rebuild = 12.6km. Rebuild stopbank to 4m crest, 1 to 3 wet face, 1 to 2.5 dry face over lower Motueka River left/right banks.

Table 5-2: Estimated Cost of Rebuild –Scheme Downstream of Woodman’s Bend

Item	**Unit**	**Quantity**	**Rate**	**Cost (excl GST)**
River Erosion Protection	m	5400 (common to 15.6km option)	340	$1,836,000
Improved River Berm Management	LS	1	250,000	$250,000
Upgrade of Stopbank Penetrations	No	8	30,000	$240,000
Under-Seepage Control	m	350	200	$70,000
Stopbank Rebuild	m^3	275,000	15	$4,125,000
Subtotal				*$6,521,000*
Preliminary and General	%	5		$326,050
Resource Consent Application	%	2		$130,420
Professional Fees	%	15		$978,150
Contingency	%	30		$1,956,300
Total				**$9,911,920**

5.3 "Full" Refurbishment Option

Refurbishment is the addition of fill materials to the existing stopbank to raise the crest level and/or widen the stopbank.

The cost estimates should be considered rough order only and are intended for comparative purposes between the options. Following selection of a preferred upgrade option, it is envisaged that more detailed studies would be undertaken to refine the refurbishment cost. Costs for the refurbishment earthworks have been developed with input from Fulton Hogan and the rate is deemed inclusive of topsoil stripping/replacement.

Length of stopbank is 15.6km and includes all reaches except Peach Island and Brooklyn Stream.
Place fill on river side face of stopbank over "Full" Scheme length (Excl. Peach Is and Brooklyn Stream)
Refurbish stopbank to 4m crest, 1 to 3 wet face, 1 to 2.5 dry face over lower Motueka River left/right banks.

Table 5-3: Estimated Cost of "Full" Refurbishment

Item	Unit	Quantity	Rate	Cost (excl GST)
River Erosion Protection	m	5400	340	$1,836,000
Improved River Berm Management	LS	1	250,000	$250,000
Upgrade of Stopbank Penetrations	No	7	30,000	$210,000
Under-Seepage Control	m	500	200	$100,000
Stopbank Refurbishment	m^3	220,000	15	$3,300,000
Subtotal				*$5,696,000*
Preliminary and General	%	5		$284,800
Resource Consent Application	%	2		$113,920
Professional Fees	%	15		$854,400
Contingency	%	30		$1,708,800
Total				**$8,657,920**

5.4 Partial Refurbishment Options

Cost estimates for the two upgrade sub-options are presented in the following Tables (5-5 and 5-6) using the same basis outlined in Section 5.3.

For these sub-options, the length of stopbank includes only the left bank from Blue Gum Corner to mouth and right bank from Woodman's Bend to mouth, where improvements are considered critical (such as to address insufficient height).

Refurbish stopbank to 4m crest, 1 to 3 wet face, 1 to 2.5 dry face over lower Motueka River left/right banks.

The options are:

Table 5-4: Partial Refurbishment Options

	Only construct where stopbank needs to meet freeboard	Entire length Woodman's Bend to mouth
500mm freeboard	Option B1	Option B2

See "Stopbank Crest Raise Summary" drawing in Appendix D

5.4.1 Option B1

Option B1: Place fill on river side face of stopbank only where an increase in freeboard is required. Downstream of Woodman's Bend.

Table 5-5: Estimated Cost of Partial Refurbishment Option B1

Item	**Unit**	**Quantity**	**Rate**	**Cost (excl GST)**
River Erosion Protection	m	3800	340	$1,292,000
Improved River Berm Management	LS	1	250,000	$250,000
Upgrade of Stopbank Penetrations	No	7	30,000	$210,000
Under-Seepage Control	m	350	200	$70,000
Stopbank Refurbishment	m^3	90,000	15	$1,350,000
Subtotal				*$3,172,000*
Preliminary and General	%	5		$158,600
Resource Consent Application	%	2		$63,440
Professional Fees	%	15		$475,800
Contingency	%	30		$951,600
Total				**$4,821,440**

5.4.2 Option B2

Option B2: Place fill on river side face of stopbank along entire length of 'critical' stopbank. Downstream of Woodman's Bend.

Table 5-6: Estimated Cost of Partial Refurbishment Option B2

Item	Unit	Quantity	Rate	Cost (excl GST)
River Erosion Protection	m	3800	340	$1,292,000
Improved river berm management	LS	1	250,000	$250,000
Upgrade of stopbank penetrations	No	7	30,000	$210,000
Under-seepage control	m	350	200	$70,000
Stopbank Refurbishment	m^3	125,000	15	$1,875,000
subtotal				*$3,697,000*
Preliminary and General	%	5		$184,850
Resource Consent Application	%	2		$73,940
Professional Fees	%	15		$554,550
Contingency	%	30		$1,109,100
Total				**$5,619,440**

5.5 Indicative Options Costs – Summary

Table 5-7: Rebuild and Refurbish Options – Indicative Cost Summary Table

Status quo	Option A Rebuild "Full" 15.6km	Rebuild Downstream Woodman's Bend 12.6km	Option B "Full" Refurbish 15.6km	Option B1 only where crest level too low between Woodman's Bend and mouth	Option B2 Entire Length Woodman's Bend to Mouth
$ minimal	$11.7 million	$9.9 million	$8.6 million*	$4.8 million	$5.6 million

* Note: Place fill on river side face of the stopbank over total scheme length (excluding Peach Island and Brooklyn Stream.

5.6 The Costs of Gravel Extraction

Depth of Gravel Removal	Volume of Gravel (m3)	Cost if Industry Remove at Own Cost	Cost @ $15/m3
1m depth	717,000	$0	$10.8m
2m depth	1,878,000	$0	$28m

It may be possible that gravel extraction costs could be offset by sales of gravel to the construction industry, but would be unlikely to yield significant royalties to Tasman District Council (as most of the land is in private ownership). This would probably be done in a timeframe that suited industry demand for gravel, and may take many years. Therefore the optimistic scenario would be zero cost to Tasman District Council to have the gravel extracted from the berms.

Alternatively if the gravel was extracted, carted and stockpiled in a shorter campaign then costs could be similar to excavation and disposal operations and be $15 to $30 per cubic metre depending on the distance and the stockpile requirements. This would be a significant cost for only achieving a reduction in over-topping risk.

Other channel management may include "blading" gravels and smoothing the flood berm by pushing materials towards the stopbanks whilst increasing the capacity of the central flood berm. This could cost in the order of $1 to $2 per cubic metre, or $700,000 to $1.4 million for the 1m depth cut operation and $1.8 million to $3.5 million for the 2m depth cut operation.

5.6.1 Reduction in Stopbank Volume and Cost

As the removal of gravel will reduce the top water level, the required height of stopbank would be lower. Therefore one benefit of gravel removal would be to reduce the required size of stopbanks (lower the volume of earthworks required).

To compare the size of required earthworks, the stopbank volumes downstream of Woodman's Bend were calculated for three cases of the proposed rebuild design: with no gravel removed, with 1m deep gravel extraction, and with 2m deep extraction.

The 1m cut and 2m cuts into the river flood berms give approximately 9% and 23% stopbank volume reductions. The rate for stopbank earthworks volume is $15/m3 placed. With mark-ups for preliminary and general (5%), contingency (30%), resource consent (2%), and engineering (15%) the budgeting rate is more like $22.8/m^3 placed.

	Cost reduction based on reduced volume before percentage mark ups	Cost reduction including percentage mark ups
1m cut scenario	$378,000	$575,000
2m cut scenario	$950,000	$1,450,000

Total cost for the rebuild option from Woodman's Bend to the mouth has a budget estimate of $9.9 million including percentage mark-ups. Therefore, the stopbank budget estimate with the reduction in stopbank volumes is $9.3 million (94%) for the 1m cut scenario, and $8.5 million (85%) for the 2m cut scenario.

The refurbish option involves 220,000m^3 of stopbank fill placement for $8.6 million. The effects of gravel extraction would reduce budget estimates to $8 million (94%) for the 1m cut scenario, and $7.2 million (83%) for the 2m cut scenario.

It would be uneconomic to pay for gravel removal in one operation. The likely scenario would then be gravel removal by industry (at their cost) over several years. If the stopbanks were constructed to the lower height to account for complete gravel removal, then there would be period of time when the stopbanks were at risk of over-topping. Therefore the intended level of protection would not be attained for several years.

Option	Construction Cost (no gravel removed)	Construction Cost (1m gravel removed)	Construction Cost (2m gravel removed)	Gravel Removal Cost by Industry Over Many Years	Gravel Removal Cost 1m deep	Partial Removal and Relocation of Gravel (Capital Project vs Maintenance)
Rebuild downstream of Woodman's Bend	$9.9m	$9.3m	$8.5m	$0.1m (Tasman District Council Management cost only)	$10m	$2m
Refurbish downstream of Woodman's Bend	$8.6m	$8m	$7.2m	$0.1m (Tasman District Council Management cost only)	$10m	$2m

5.7 Development of Cost Estimates from 2008 to 2011

The construction cost estimates for the Lower Motueka River Flood Control Scheme prepared for the Preliminary Design phase 2008 have been revised and adjusted during the options assessment carried in 2010-2011. A timeline summary of the construction cost estimates is given below.

The estimates are based on the same design standard which is the 1% AEP rainfall event with climate change allowances projected to 2090 as per Ministry for the Environment climate change guidelines. The estimates are also based on the same finished stopbank cross section of 4m wide crest, 1 to 2.5 side slopes on the town side, and 1 to 3 side slopes on the river side. River erosion protection and upgrade of stormwater pipeline penetrations are included into the cost estimates.

The estimates all include the following proportional costs based on the engineering cost estimate:

Proportional Cost Item	Proportion
Preliminary and General	5%
Engineering and Administration	15%
Resource Consent Application	2% (2011) or $150,000(2008)
Contingency Allowance (options phase)	30%

Table 5-8: Timeline Revision of Construction Cost Estimates

Date	Construction Cost Estimate	Comments
2008	$17.3 million	Base case. Construction estimate based on upgrading the total scheme (24km of stopbank) to the design standard. The fill volume and rates of the work were based on the best current estimates at that time. Volume of stopbank fill estimated to be 400,000m^3 based on a cross section of 1 to 2.5 river side slope and 1 to 2 town side slope, with 4m crest width. Rebuild option implies demolition then new construct stopbank, and not relying on the existing structure.
2008	$19.5 million	Base case with sensitivity check applied to numbers: upper range value based on 25% more earthworks needed and 15% higher construction rates than 2008 base case. Volume of stopbank fill estimated to be 500,000m^3.
2008	$12.8 million	Reduced length of rebuild: 15.6km (excluding Peach Island and Brooklyn Stream stopbanks). Volume of stopbank fill estimated to be 260,000m^3.
2010	$13.6 million	Revised base case 24km Rebuild. Cost estimate has been reviewed based on new information on ground levels (from aerial survey) over the floodplain; and detailed two dimensional modelling of the stopbank system. Hydrology review of the catchment has refined the design hydrograph that determines the crest profile. Geotechnical assessment of the permeability of the stopbank scheme and its foundations has refined the foundation and cross section design. Volume of stopbank fill estimated to be 433,000m^3 based on a cross section of 1 to 3 river side slope and 1 to 2.5 town side slope, with 4m crest width.
2011	$11.7 million	Option A Rebuild 15.6km long (excluding Peach Island and Brooklyn Stopbanks). The volume of stopbank fill estimated is 350,000m^3 based on a higher design crest profile and cross section of 1 to 3 river side slope and 1 to 2.5 town side slope, with 4m crest width.
2011	$9.9 million	Rebuild 12.6km of critical stopbank length from the mouth to Woodman's Bend, and to Blue Gum Corner. Volume of stopbank fill estimated to be 275,000m^3.
2011	$8.7 million	Option B Refurbish15.6km long (excluding Peach Island and Brooklyn Stopbanks). This requires placement of fill material on the river side of the existing stopbank for the full length, in order to raise the crest level and thicken the cross section to a consistent standard. Volume of stopbank fill estimated to be 220,000m^3.
2011	$4.8 million to $5.6 million	Other refurbish sub-options (B1 and B2), only working on the parts of the stopbank scheme (12.6km reach) that require the crest to be raised. More site investigation and assessment of the existing stopbank is needed to improve confidence of estimates.

The 2008 preliminary design cost estimates were assessed for sensitivity to volumes of fill and rates of construction, and indications were that the base case estimate could be increased by 12.7% if the project required 25% more earthworks and incurred 15% higher construction rates. This sensitivity check increase could be applied to 2011 cost estimates for budgeting purposes before proceeding into more detailed design phases. However, the current assessments have refined volumes and removed some details (such as drainage); which gives more confidence in the amounts estimated.

Estimate of Total Project Costs					
Item	Rebuild Option	Refurbish Option	Secondary Stopbanks	Spillways	Status Quo
Construction	$11.7m	$8.6m	$13m	$6.5m	-
LTP Consultation; Peer Review; Legal Costs; Environment Court; Land Costs; Legacy Issues	$1.45m	$1m	$1.45	$0.65	$0.3m
Sub Total	$13.15m	$9.6m	$14.45m	$1.15m	$0.3m
Optional Gravel Removal/ Relocation	$2m	$2m	$2m	$2m	$2m
Total	$15.15m	$11.6m	$16.45m	$3.15m	$2.3m

Brooklyn Stopbanks rebuild construction cost $1.5m; other costs $0.3m

Peach Island rebuild construction cost $3m; other costs $0.6m

6 Flood Modelling Scenarios to Support Economic Analysis

6.1 Hydrology

A summary of the peak catchment discharges is shown in the Table below. The average return intervals are 100 years and 200 years with current rainfall volumes, and 100 years with projected future climate change rainfall volumes.

Rainfall storms vary in duration and intensity, therefore we have considered a range of 1% Annual Exceedance Probability (AEP) events 12, 24, 48, and 72 hours.

Table 6-1: Rainfall-Runoff Model Peak Catchment Discharges, Motueka River Catchment (m^3/s)

Return Period Rainfall	Rainfall Duration (hours)			
	12	24	48	72
100 years + climate change (2090)	3350	4050	3420	2720
200 years	2970	3550	2540	2330
100 years (2011)	2660	3165	2640	2050

The largest recorded flow for the Motueka River at Woodstock is 2148m^3/s on 10/07/1982. Based on catchment area at Woodstock (1750km^2) and Alexander Bluffs (1968km^2) this equates to a corresponding flow of 2360m^3/s at Alexander Bluffs.

6.2 Sea Level

At the mouth of the Motueka River where the stopbanks end, the sea levels were modelled as an oscillating tidal boundary based on design tidal levels in Tasman District Council Engineering Standards, with storm surge and climate change sea level rise factored into the boundary. The peak tide coincided with the peak discharge from the mouth of the Motueka River. Modelling indicates that the tidal backwater impact on flood levels in the Motueka River tends to fade out at the location of the SH60 bridge and is not a significant factor for stopbanks upstream of the bridge.

Table 6-2: Outlet Boundary Conditions

Tidal Component	Level Above Mean Sea Level (m)
Mean High Water Springs	2.0
Storm Surge	0.7
Wave Run up	0.3
Safety Margin	0.2
Global Warming Sea Level Rise	0.5
Total	**3.7**

6.3 Hydraulic Modelling

Hydraulic modelling of the Lower Motueka River and the Motueka Plains was calculated in InfoWorks RS software. The model consists of a one-dimensional (1D) main channel developed from the river cross sections, from stopbank to stopbank of the Motueka River and Brooklyn Stream (the model is held by Tasman District Council). This channel is connected within the software to a two-dimensional (2D) grid that represents the ground elevations beyond the channel (from LiDAR data obtained by Tasman District Council in 2009). The grid covers the Motueka Plains from the coast up to Peach Island and up towards Alexander Bluffs Bridge. This allows calculation of information such as the flow, water depth and velocity at each grid cell.

The modelled breach is at a nominal location to assess potential flooding. It does not indicate a known weakness, or the only weak spot in the stopbanks.

A breach failure will cause sudden flooding in that location, the effect will be more damaging than the same flood would if the stopbanks did not exist; this is because the flood waters would have built up more slowly, and spread more widely, before the stopbanks existed.

Table 6-3: Summary of Model Scenarios Completed

Stopbanks Scenario	Runoff Event	Comment
Existing stopbanks at existing crest profile	1% AEP + climate Change (design standard)	Base case run to test existing overtopping flows over the Motueka Plains. This model is in 2D and provides the flood flows across the Plains from low points of the existing stopbanks.
Existing stopbanks with a breach in the right bank, opposite Fry's Island (Chainage 8000)	1% AEP + climate Change (design standard)	Breach case to model the flooding over Motueka Plains. This model is 2D across the Plains.
Existing stopbanks at existing crest profile	0.5% AEP (overdesign event)	Overdesign case to assess residual damages from overtopping flows. This model is in 2D across the flood plains.
Upgraded stopbank crest levels to design standard	1% AEP + climate Change (design standard)	Design standard flow contained within upgraded stopbank system. This model is a 1D open channel model downstream of Woodman's Bend.
Existing stopbanks with berm modifications – various scenarios (#A1)	1% AEP + climate Change (design standard)	Increase channel capacity by lowering berm shoulders and widening active channel. This model is a 1D open channel model downstream of Woodman's Bend.
No stopbanks	1% AEP + climate Change (design standard)	To assess flood effect without banks in place.
Existing stopbanks with a breach in the right bank, opposite Fry's Island (Chainage 8000)	0.5% AEP + climate Change 2090 (design standard)	Breach case to model the flooding over Motueka Plains. This model is 2D across the Motueka Plains to give the Risk Assessment an upper bound damage estimate.

All models were run with corresponding high tides coinciding with peak discharge at river mouth, as per Section 6.2.

6.3.1 Other issues

Rebuild and refurbish options for the Lower Motueka River flood control scheme have excluded the upgrade of parts of the scheme to take account of community wishes for lower costs. One approach has been to focus upgrade works on the most critical sections of the scheme and remove less-critical scheme elements from the scope of works. The less critical sections of the stopbank scheme have been identified as including Hurley's stopbank, Kiwifruit Orchard stopbank, Peach Island stopbank loop, outlet stopbank spur on left bank – each of which protect a relatively small amount of property. The critical sections of the scheme have been identified as Brooklyn Stream (when also affected by flood in own catchment), plus the right bank from the outlet to Woodman's Bend, and the left bank from outlet to Blue Gum Corner – both of which protect large parts of the plains and the main townships.

Excluding some sections of stopbanks will mean that fewer properties are protected, and there will be fewer rate-payers to share the costs of any flood control works.

The analysis of the less-critical stopbanks indicates that the Hurley's stopbank height is currently at a 50 year Average Return Interval (ARI) design standard and protects a single property and orchard. The Kiwifruit stopbank also protects a single property, but it is approximately a 20 year ARI design standard height and would need a crest raise of 1m to meet the overall scheme standard. The Peach Island ring meets the 50 year ARI design standard height and protects fewer than 10 dwellings. The outlet stopbank return spur on the left bank is subject to tidal conditions more than river flow conditions. These stopbank sections could be upgraded in the future as standalone projects in isolation from the rest of the scheme.

The overflow channel behind Peach Island receives flows from the Motueka River after the flow exceeds $800m^3/s$ (approximately a two year ARI flood event). This is an old channel alignment that has been assessed as carrying up to $500m^3/s$ during a 100 year ARI event. Some channel improvements could be made such as to the bridge/culvert and fence lines, trees and localised contouring to increase flow and keep the water level below the Peach Island stopbank crests. Modelling indicates that the capacity is constrained by downstream water levels in the Motueka River at the Blue Gum Corner river bend, and channel improvements will not have a significant effect on the channel capacity. Small scale channel improvements are recommended as part of annual maintenance activities rather than large scale channel earthworks.

Stopbank realignments opposite Blue Gum Corner have been modelled to examine the effects on top water profiles along the Motueka River. A stopbank rebuild of the right bank that includes an inland curve and an extended flood berm width of up to 100m has shown minor reductions in top water levels for a large amount of construction. Modelling and engineering assessment shows that improving the stopbanks on the existing alignment is a more cost effective manner of improving the flood flow capacity and reducing the risk of stopbank failure.

6.3.2 Recent State Highway Modifications

The NZ Transport Agency "as built" plan of High Street shows 855m of upgraded highway between Staples Street and the Motueka River Bridge. Mostly it is a 30mm seal overlay, but some parts fill the road surface by up to 150mm (80m long). The water level change in upslope parts of Motueka township would at most be 150mm, but is likely to be 50-100mm during a large flood event that causes overland flows to cross the road centreline. It is unlikely that this road overlay has caused any additional flooding in the township during recent, normal wet weather. Ponding could be caused by other factors such as blocked drains, etc.

It is considered that the extent and type of roading works undertaken by NZ Transport Agency, namely the road overlay, would not cause a significant effect on the overland flowpaths that cross SH60 towards the coast. Formal modelling of the impacts was not carried out.

6.3.3 Modelling of Relocated Stopbanks

Model results indicated that relocating banks in isolated areas (for example Peach Island) did not provide large benefits in terms of lower peak flood levels upstream of the widening. Modelling showed that downstream cross sections that were not widened were subject to increases in peak water level. By observation, the work needed to relocate stopbanks to make a wider channel was large, but for little hydraulic benefit. It was clear that a better means of increasing flood protection is to increase the height and improve the condition of existing stopbanks in their current location.

6.3.4 Flooding Impacts

The flooding impact can be estimated using the flood modelling results in Motueka township and adding 50-100mm onto top water levels. The model accuracy in Motueka is probably in the order of +/-200mm or more due to the (in)accuracy of the LiDAR data in the urban area. The effects of fence-lines, shelterbelts, vineyard lines, orchard trees, houses, building foundations, kerbs, storm-water drainage pipelines, walls, culverts, footbridges are not accounted for in the model built (except as an increased ground roughness compared to the river channel).

7 Assessment of Flooding Risks and Costs

To assess the impact of flooding and the associated damage costs, hydraulic model outputs were mapped and combined with information such as building locations, land types, etc. (that are at risk from flooding) in a Geographical Information System (GIS). The flood damage could then be estimated from the flood depths and velocities impacting the land, building and infrastructure within the flooded area. The total flood damage cost for a range of different severity flood events was derived for the current situation, and for each of the stopbank improvement options.

Detailed explanation of the process followed is included in the Appendix B and C.

7.1 Residual Risks

Following implementation of any stopbank improvement option the flood risk will reduce, but some risk will remain. This residual risk will be less for the stopbank improvement options that are more extensive and produce more reliable resistance to flooding. The economic assessment of options has focussed on reducing flood risk assuming each scheme is built to a comparable standard in regards to the level of protection that each provides. Freeboard in the design provides an additional margin of safety against surge and wave effects over-topping banks.

In addition there were option-specific risks which needed to be addressed. For example, the rebuild option involves removing the existing stopbank to allow construction of the new stop bank on approximately the same alignment. This temporarily increases the vulnerability of property and assets behind the current stop bank to flooding. This risk could be mitigated by undertaking construction during periods when heavy is rain is not forecast; or using relocatable flood barriers around the worksite, for example, but this would add to the scheme construction costs. In regards to the refurbishment options it is more difficult to add quality to existing stopbanks rather than build it in to new structures. There will also be some residual risk that sections of the stop bank that were not refurbished will still fail. Extensive quality checks will need to be made to ensure that the refurbished stop banks will perform as intended. These option-specific risks were included in the economic analysis.

8 Discussion of Results

The results from the Benefit Cost Analysis, (assuming a stopbank design life of 100 years and 5% discount rate) are summarised in Table 8-1. These results are based on the modelling carried out to date.

The BCR figures provide a relative assessment of the options; there is no specific target amount.

Table 8-1: Benefit Cost Analysis of Options

Scheme Option	Overall Discounted Scheme Cost	Overall Discounted Residual Cost-Risk	Overall Discounted Benefit	BCR
Status Quo (with Improved Maintenance Strategy)	2,084,000	42,981,500	0	-
“Full” Rebuild	11,570,200	11,700,400	8,010,600	1.34
“Full” Refurbish	9,955,800	11,193,700	10,638,400	1.50
Partial Refurbish	7,314,700	16,267,300	3,132,200	1.13
Spillways only Option	2,572,126	19,740,780	927,851	1.04
Secondary Stopbanks	14,184,600	22,141,200	-15,485,500	0.57

The economic analysis identifies the “full” refurbishment option as offering the greatest benefit in reduced risk compared to the scheme costs. The benefit cost ratio for the “full” rebuild option (BCR= 1.34) is lower than the “full” refurbishment option (BCR=1.5) for two main reasons:

a) The cost of rebuilding the stop banks is estimated to be approximately 16% greater than the “full” refurbishment option.

b) In order to rebuild the stop banks on approximately the same alignment it is necessary to remove the existing bank in sections before construction of the new stopbank can take place. This temporarily increases the flood inundation risk at the construction site to a level equivalent to the situation when there are no stop banks at that location.

The partial refurbishment options, although approximately 36-38% less costly than the “full” refurbishment option, offer significantly less protection overall, resulting in high residual risk.

Both the spillway and secondary stopbanks options have comparatively high value of residual cost-risk; this leads to poor benefit /cost ratios.

9 Sensitivity Analysis

The following sensitivity analyses have been performed to scope the uncertainties in the analysis.

a) Different discount factors of 4%, 5% and 7%.
b) Different design life for the stopbanks of 50 years, 100 years and 150 years.
c) Including an additional social and economic benefit arising from improved protection amounting to $200,000 per annum.
d) A reduced construction time for the rebuild option of 5 years rather than 10 years. This reduces the period of increased flood vulnerability for this option during construction.

In general, the results were relatively insensitive to changes in the variables listed above. In all cases the “full” refurbishment option was shown to be the most cost effective option overall, having the greatest BCR.

10 Limitations

This study has improved knowledge of river hydrology, river hydraulics, and the geotechnical competence of the existing stopbanks. With all modelling studies there remains some uncertainty due to the assumptions of the model variables.

Any future detailed design of a chosen option will need to confirm all data and assumed variables.
For example, the height of stopbanks as assessed in this study has been based upon LIDAR data, rather than detailed topographical survey.

11 Next Steps

As per the Local Government Act 2002 Section 78 process, Tasman District Council should consider the findings of this report and work towards adopting a preferred reasonably practicable option for more detailed assessment to address the flood control of the Lower Motueka River.

Decisions will need to be made about the acceptability of residual risks versus the cost to mitigate those risks.

The adopted option should be presented to the community for consultation via the Long Term Plan (LTP).

Future growth of the township and assets should be considered.

The mechanism(s) for funding should be considered before commencing community consultation.

Tasman District Council will need to confirm land ownership and determine the legal status of the existing stopbanks before any option proceeds.

Future Issues to be Considered:

- Would there be an increase in land value if the stopbanks were improved?
- Given recent world events, will insurers view the improvements more positively, or perhaps demand a higher level of protection?
- Climate change knowledge will continue to improve; the project should be reconsidered in light of any significant changes to current knowledge.

Appendix A Details of Scheme Options

State of Existing Stopbanks

The Lower Motueka River stopbanks were constructed between 1951 and 1956. Construction methods were cut and fill earthworks using locally available riverbank materials - generally consisting of river silt with minor gravel, sand and clay content. The nature of materials in the stopbanks is variable and is probably related to the nature of the nearest available materials sourced from the borrow areas located adjacent to the stopbanks.

The stopbank materials appear to have been only moderately compacted by tracking of the earthworks equipment used to build them. Local landowners report that there was no use of specialised compaction equipment. Physical examination (of stopbank exposures and traffic and stock wear points on the stopbank batters, and also the ease of hand augering the silts in the centre of the stopbanks) confirms that there has been a relatively poor degree of compaction during construction in some places.

The stopbanks vary in height between 2m and 4.5m above the existing river flood plain. The crest varies in width between 1.8m and 2.5m, and is less than the 3m width recommended in the Tasman District Council Engineering Standards. The side slopes (batters) are moderately steep and vary from 1H: 1V to 2.2H: 1V with the gradient generally being 1.5H:1V.

The silts in the stopbanks are firm (having an undrained shear strength of 25 – 50 kPa) with some areas, particularly near the top of the stopbanks, being soft (having an undrained shear strength of 12 – 25 kPa). The batter slopes tend to be soft after wet weather and are readily “pugged” by stock movement. Scala penetrometer test results (taken in 2006) record that there is considerable variability in soil strength or consistency along the stopbanks, being soft to firm in the upper 1m and firm below 1m depth. It should be noted that results of Scala tests are dependent on moisture content at time of testing.

In-situ permeability tests of the stopbanks were carried out in 2008. The permeability tests were attempted at 10 test pit sites. Five test sites in silts were successful (they held water) and recorded permeabilities that ranged between Ksat values of 2.9×10^{-6} m/s to 6.7×10^{-6} m/s. At five test pit sites in the stopbanks, where silty sands and silty gravels were encountered, the water in the constant head cylinder seeped away so rapidly that a constant head test could not be achieved (highly permeable ground). At these more sandy/gravelly sites, rapid infiltration of water into the stopbanks would be expected during high flood levels; which means that the stopbanks would not control flood water sufficiently.

Generally the stopbanks are set 50m or more away from the active river channel. In places the stopbanks are within 20 metres of the active channel (for short distances). In these areas the river banks are well protected by large armour rocks to prevent lateral migration of the channel. Additional protection to the stopbanks has been provided by the planting of willows and other trees between the river and the stopbank. The effect of these trees would be to lower the flood velocities against the batter of the stopbank and reduce erosion.

Although the stopbanks are in reasonable condition considering their age, they have suffered due to human and stock interference. The plan of “Areas of Stopbank Vulnerability” (attached in the Appendix) illustrates several types of vulnerability and the approximate location of these areas. The types of vulnerability include:

- open gravel pits close to the stopbanks and gravel pits backfilled with loose permeable materials
- trees growing in the stopbanks or in the toe areas of the stopbanks
- erosion-prone areas where the stopbanks are close to the river channel or where the floodplain width between the stopbanks is constricted
- areas where water seepage beneath the stopbanks has been observed during previous floods
- areas where the stopbanks have been damaged by stock crossing points or by rabbit holes
- pipelines through the stopbanks

- permeable areas where there are old channels with gravels near surface on the landward side of stopbanks.

Maintenance of the stopbanks includes annual mowing and inspections, although the mowing may be done twice yearly at the discretion of the Rivers Engineer. The last detailed walkover survey and condition assessment of the stopbanks was carried out in 2005. Over the six year period, since this detailed condition assessment was carried out, there has been a marked improvement in the extent and quality of rock revetment along the edge of the river channel; however the majority of the vulnerable areas identified above still exist and have yet to be remediated.

Stopbank Failure Modes

The following potential failure modes have been determined as being critical for the existing stopbanks:

- overtopping of the stopbank resulting in soil erosion
- slope instability caused by the effect of soil saturation (from water pressure) during flood conditions
- piping (the transportation of soil by water seepage) either through the stopbank itself or through the foundation, resulting in uncontrolled seepage flows that erode the stopbank or its foundation
- roots from vegetation on the river bank either increasing the likelihood of piping or physically damaging the stopbanks if ripped out under flood conditions
- river erosion damage to either the stopbanks or their foundations
- interference with the stopbank and the river berm.

Each of the failure modes is discussed in detail in the following section.

Slope Instability of the Existing Stopbanks

Effect of Seepage through the Stopbank

Stopbanks are unique water retaining structures in that they are only exposed to their design load for very short periods of time throughout their life. The steep nature of New Zealand rivers generally results in floods that rise and fall over periods of hours rather than days, therefore the time available for water to penetrate into the stopbank is limited.

A key consideration for analysis of any stopbank is the extent of the structure that becomes saturated during the flood event. The saturation is a function of the duration of the flood event and the permeability of the soil in the stopbank. As the degree of saturation increases, the stability of the stopbank will decrease.

The permeability of the Motueka stopbanks has been estimated from a series of standpipe permeameter tests undertaken during the investigation programme. Where tests were successful the permeabilities ranged from 2.8×10^{-6} m/s to 6.5×10^{-6} m/s, consistent with the geotechnical description of the stopbank fill material as sandy silt. However, at a number of test locations the water level in the standpipe could not be successfully held at a constant level, indicating the presence of more permeable stopbank fill. It therefore is concluded that the fill material in the stopbank is variable in nature. It is important to consider the full range of soil properties that may exist along the length of the stopbank from an analysis perspective. A technical representation of this uncertainty is a normal probability distribution curve. The investigation results suggest that a ‘typical’ permeability of the stopbanks (ie. one that would sit in the middle of the distribution curve) would be in the order of 5×10^{-6} m/s. At the higher permeability end of the distribution curve a ‘reasonable upper bound’estimate of permeability would be at least an order of magnitude higher ie. 5×10^{-5} m/s.

The second consideration in assessing the degree of saturation is the duration of the flood event. The hydrology of the Motueka River has been assessed by MWH and is presented in our report *Lower Motueka River Flood Control Scheme – Hydrology Review* dated August 2010. Based on the hydrographs presented in the hydrology report the following reference flood event is considered a reasonable estimate of the time that the stopbanks may need to retain water during a design (1% AEP) flood event. The figures are based on a typical flood rise of 4m during the "design standard" flood event and a linear variation in water level has been assumed between each of the time increments in the following table.

Table A1 – Flood Rise vs Time during "design standard" Flood Event

Time (hours)	Flood Rise (m)
0	0
10	4
20	4
50	0

The stopbank system has been analysed assuming a 'typical' permeability of $5x10^{-6}$ m/s. The output below shows the phreatic surface (the line of saturation) at two hour increments during the flood event. A cross section with a height of 2m, crest width of 2m and slopes of 1.5H:1V has been adopted to give a conservative (ie. narrow) representation of the stopbanks. The 2m stopbank will only need to retain water once the flood rise reaches 2m (ie. hour five) and will not need to be retaining water once it falls below 2m (ie. hour 35). The analysis is then repeated using the higher permeability of $5x10^{-5}$m/s which may not be typical, but could exist along the length of the stopbanks. This has been referred to as a reasonable upper bound case.

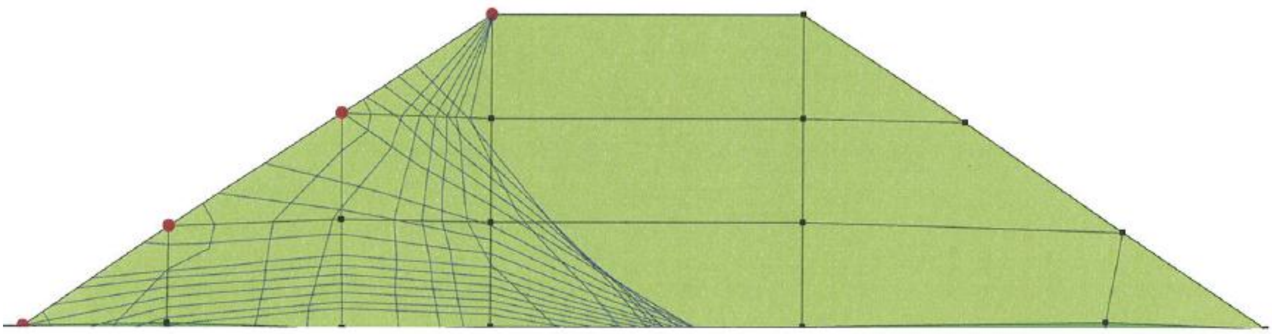


Figure A1: Line of saturation at two hour increments for 'typical' embankment permeability; represents an effective stopbank

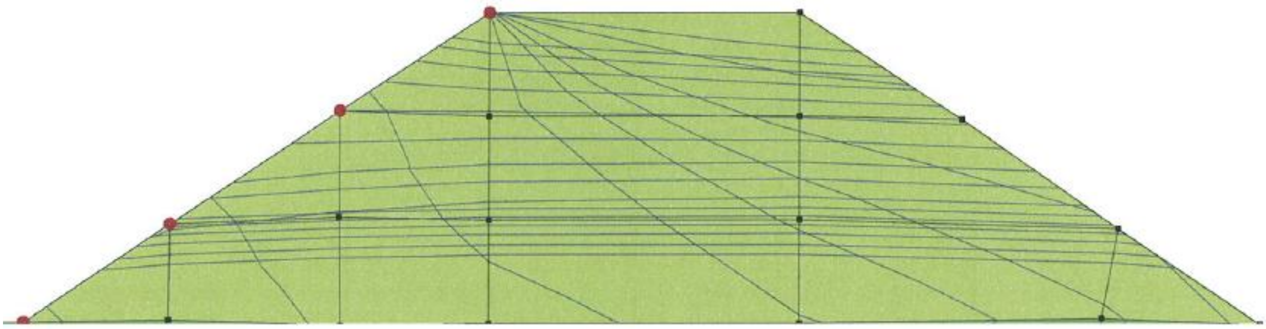


Figure A2: Line of saturation at two hour increments for ''reasonable upper bound' embankment permeability; represents a much less effective stopbank

The blue lines illustrate how far water has penetrated into the stopbank during the duration of the "design standard" flood (each line being at a two hour increment during the flood). In Figure A1 the water has only reached approximately half way across the stopbank (the land side of the stopbank being on the right of the diagram. Conversely in Figure A2 where the soil is modelled as being 10 times more permeable, the water can reach much further across the stopbank in the time that it retains water during the “design standard” flood. All of the soil below the blue lines is saturated, therefore in Figure A2 almost all of the soil is predicted to be saturated. The stability of the stopbank will be markedly different under these two scenarios as saturation acts to destabilise soil slopes.

Existing Stopbank Fill Strength

In addition to the degree of saturation, the most significant variable in the assessment of embankment stability is the strength of the fill material in the stopbanks.

MWH has undertaken preliminary investigations of the soils that form the foundation of the stopbanks and the nature of the soils that the stopbanks are constructed from. The findings are presented in the MWH report *Motueka River Flood Control Scheme Upgrade* dated June 2008.

As shown in Figure A3 below, the fill is standing vertically in the test pit walls indicating that it is displaying effective cohesion. Observation of the material in the pits and back analysis of the test pit slopes indicate that soil properties of phi = 30 degrees, c=5 kPa would represent typical soil properties of the fill material for preliminary analysis purposes.

Figure A3: Test pit in crest of existing stopbank (TP07A)

The geotechnical investigations identified areas of lower silt content (ie. sandier material) within the stopbank. For example, while the average silt content in stopbank samples was 20% the lowest silt content was 2%. To account for this potential range of materials a 'reasonable lower bound' soil strength estimate should allow for low silt content and therefore low effective cohesion. Properties of phi = 30 degrees, c=1 kPa have been adopted for this case, modelling the fill as sand with a nominal amount of cohesion.

Stability Cases to be Analysed

The US Army Corps of Engineers publication *Design and Construction of Levees* (Levee is the US term for stopbank) provides guidance on applicable factors of safety (FOS) for stability analysis of stopbanks.

Under this guideline a factor of safety of 1.4 is recommended under 'steady seepage' conditions. For the Motueka stopbanks, that infrequently retain water for short periods of time, there is unlikely to ever be a steady seepage situation that results in saturation of the stopbanks, the permanent groundwater level associated with normal river levels being well below the stopbanks. The saturation of the stopbanks shown in Figures A1 and A2 is not a steady state situation, it only occurs for a brief period of time during the "design standard" flood event. The steady state case can therefore be modelled with the stopbank slopes dry. And therefore a factor of safety of 1.4 targeted for the stopbanks in non flood conditions. It could be argued that given many years of successful service, the stability of the stopbanks is proven by precedence under non flood conditions and therefore this case has not been considered further.

The guidelines also recommend a factor of safety of 1 to 1.2 under rapid drawdown conditions. A typical rapid drawdown analysis is performed on a water retaining slope that is saturated by a pool of water and that pool of water suddenly drops leaving unbalanced water pressures in the slope. The severity of the case depends on how quickly the pool of water drops relative to the ability of the soil to drain. In the case of the Motueka Stopbanks it is unlikely that this case can exist. Given that the upstream slope only becomes saturated during a flood rise, the only time that the rapid drawdown case would apply is during the fall of the same flood. If the time of flood rise and flood fall are broadly the same the rate at which the stopbank saturates and the rate at which it drains should be approximately the same and limited differential water pressures should exist. A theoretical exception is that a breach occurs elsewhere and causes a rapid drawdown; by definition there would then be no flood water that needs to be controlled against the intact stopbank. Therefore a collapse might occur, but not cause any extra flooding. Rapid drawdown failure of the upstream (river side) slopes should therefore not be a critical consideration for these stopbanks. Some rapid drawdown failures have been experienced on other stopbank systems, indicating that higher water pressures can be trapped even though theoretically this should not be the case. It is noted that while any such failures would need to be repaired following a flood, they would occur as the river level is falling and are therefore much less likely to cause a breach of the stopbank than instability on the land side face which could occur when the river is still rising.

Stability of the downstream (ie. land side) slope under flood conditions could also be viewed as being equivalent to a rapid drawdown case in that it is a short term case of saturation of the slope under seepage conditions that are varying with time (referred to as transient conditions). A factor of safety of 1.2 appears appropriate for this case ie. at the high end recommended for rapid drawdown conditions recognising the importance of maintaining the stability of the land side slope during a flood event.

The design requirement adopted for the stopbanks is considered to be to achieve a factor of safety of 1.2 on the downstream slope under transient seepage conditions (ie. the extent of saturation shown in either Figure A1 or Figure A2 depending on typical or reasonable lower bound assumptions used).

Analysis Results

The height, crest width, upstream and downstream slopes vary along the length of the stopbanks. To analyse the variability in stability along the stopbank, three different idealised cross sections have been analysed to represent the stopbank system, as summarised in the following table.

Table A2: Representative cross sections for analysis

Stopbank Profile	Height (m)	Crest Width (m)	Upstream Slope	Downstream Slope
1	2	2	1.5H:1V	1.5H:1V
2	2	2.5	2H:1V	2H:1V
3	3	2.5	1.5H:1V	1.5H:1V

Cross section Type 1 is considered representative of the stopbank sections with the shortest seepage pathways, generally constructed at moderate heights with limited crest width. Cross section Type 2 is the most common arrangement for the stopbank. Cross section Type 3 represents the highest cross sections, where typically the slopes are variable but a number have slopes at the steep end of the range observed along the stopbank length.

The stopbank stability analysis results are summarised in the following tables. The bracketed numbers are the factor of safety (FOS) under transient seepage conditions (as per the saturation illustrated in Figures A1 and A2). Note that a factor of safety of 1.2 is the targeted value.

Table A3 – Summary of Slope Stability Results

Cross Section Type 1

	Reasonable lower bound soil strength properties	**Typical soil strength properties**
Reasonable upper bound permeability	Does not meet stability requirement (FOS= 0.6)	Meets stability requirement (FOS= 1.5)
Typical permeability	Meets stability requirement (FOS = 1.3)	Meets stability requirement (FOS=2.4)

Cross Section Type 2

	Reasonable lower bound design soil strength properties	**Typical soil strength properties**
Reasonable upper bound permeability	Does not meet stability requirement (FOS= 0.8)	Meets stability requirement (FOS= 1.9)
Typical permeability	Meets stability requirement (FOS = 1.7)	Meets stability requirement (FOS=2.8)

Cross Section Type 3

	Reasonable lower bound soil strength properties	**Typical soil strength properties**
Reasonable lower bound permeability	Does not meet stability requirement (FOS= 0.5)	Meets stability requirement (FOS= 1.2)
Typical permeability	Meets stability requirement (FOS = 1.2)	Meets stability requirement (FOS=2.0)

The results indicate that there would need to be a combination of both lower bound soil strength and higher permeability for the existing stopbanks to fall below an acceptable factor of safety. There is likely to be a strong link between these variables. For example, all of the successful permeability tests within the stopbanks (yielding values around the typical values used in the analysis above) were undertaken in locations where the silt content was high. At other test pit locations, where the silt content was lower and the tests were not successful, the permeability would be expected to be higher.

Based on the investigations undertaken to date, it is not known if this combination of adverse conditions exist along the stopbank. It would require extremely extensive investigations to prove or disprove whether they did exist along the entire length of the stopbank. Given that the stopbanks have resisted significant historical floods it is possible that this combination of conditions do not exist. However, until the stopbanks experience the "design standard" flood height and duration this could not be guaranteed.

Overtopping

In determining an acceptable level of freeboard, consideration should be given to the potential for the stopbanks to survive some overtopping for a limited period of time. Given the low plasticity nature of the fill material it could be expected to have limited capacity for overtopping, however vegetation cover on the slopes will enhance this resistance.

The proposed design standard for the upgraded stopbanks is that they should have 500mm freeboard above the predicted 1% AEP(+2090 climate change) flood event. It is understood that hydraulic modelling has indicated that the existing stopbank system will overtop in some locations during this "design standard" event.

Guidance on the erosion resistance of grass lined slopes can be taken from the CIRIA publication *Design of Reinforced Grass Waterways*: depending on the velocity of the water (a function of overtopping depth and stopbank slope), the steepness of the slope and the quality of the grass coverage. The resistance time in hours for a number of scenarios is shown in the table below.

Table A4 - Time to Erode (hours) for Overtopping Depth and Batter Slope

		100mm overtopping depth			200 mm overtopping depth		
		Grass coverage			Grass coverage		
Stopbank Profile	Batter Slope	Good	Average	Poor	Good	Average	Poor
1 & 3	34° (1.5H:1V)	8	3	0	1	0	0
2	27° (2H:1V)	20	6	2	2	0	0

With reference to Table A1, it is possible that the flood could maintain its peak stage for a period of approximately 10 hours, therefore overtopping could occur for this period of time. From the values in Table A4 it is apparent that sections of stopbank with good grass coverage may be able to survive overtopping of approximately 100mm, but are unlikely to survive greater overtopping depths. The analysis highlights the uncertainty in the overtopping resistance and therefore reinforces that overtopping of the stopbanks would introduce significant risk.

Increasing the height of the existing stopbank to increase freeboard would involve significant construction. The crest width is generally narrow and additional height cannot readily be added to the top of the stopbank. More extensive earthworks, which increase the overall width of the stopbank are generally required to achieve an increase in height.

An alternative approach to increasing available freeboard is to manage the level of the river in a flood event. This could be achieved by releasing water at controlled low points in the stopbank system, across agricultural land, which can be inundated for short periods of time without significant economic impact. However this would only address the risk of over-topping; other failure modes may still occur.

Piping

Piping is a process whereby water flowing through a soil carries soil particles. If enough material is transported, a cavity or pipe opens up carrying increasingly more soil until failure can occur. Piping could occur either through the stopbank itself or beneath the stopbank in the foundation materials. Each of the mechanisms is discussed in more detail below.

Piping through the Existing Stopbank

The fill material in the existing stopbanks is of a type that could be susceptible to piping based on published precedents. Piping will only occur if the flood acts on the stopbank long enough for water to saturate the material and flow through to the downstream face. The potential for piping is therefore strongly related to the permeability of the stopbank fill material and is generally highest where there are preferential flow paths through the stopbank that could result in a concentrated flow. During the investigations of the stopbanks the presence of gravel horizons was noted. These gravel horizons are the most likely paths for preferential seepage through the stopbank that could initiate a piping issue. The same conditions that will potentially result in instability of the downstream slope will also present the potential for piping. The risk of piping can therefore be directly related to the risk of slope instability.

Piping through Stopbank Penetrations

Special consideration must also be given to any penetrations through the stopbank. These are well known problem areas because seepage concentrates along the penetration. Types of penetrations include:

- pipelines
- bridge abutments
- historic access ramps to the river.

If identified and investigated, specific mitigation measures can be utilised at these locations to significantly reduce the likelihood of piping problems developing. These measures typically involve collecting the water seeping along a concrete to soil interface and discharging it in a controlled manner.

Piping under Existing Stopbank

When water is held against the stopbank during a flood event there is sufficient water pressure to drive water through foundation soils under the stopbanks. This water can appear as seepage on the landward side of the stopbank. Given the deep gravel deposits that underlie the stopbank, it would be prohibitively expensive to prevent this flow of water. While the flow of water represents an issue to landowners directly affected, it is unlikely to be of sufficient magnitude to cause significant downstream damage to buildings.

The flow of water could also move (by piping) soil beneath the embankment. This could undermine the stopbank and cause collapse. When assessing the risk of piping beneath the stopbanks consideration must be given to the types of soils that underlie them, as outlined below:

- Gravelly sand, which will be permeable enough to allow significant flows during a flood event, but due to the weight of the gravel particles is less likely to fail due to piping.
- Silt, which will be relatively impermeable and therefore is unlikely to be saturated during a flood event.
- Sand, which is likely to be permeable enough to allow flow during a flood event and could be susceptible to soil movement.

The risk of foundation piping would therefore be strongest where there are significant layers of sand underlying the stopbank and where the sand is exposed at the ground surface so that soil can be moved from under the stopbank. Based on the geotechnical investigation results, these ground conditions are not common and this mechanism presents a limited risk to the stopbank, albeit that it cannot be completely discounted. In locations where issues such as seepage and sand boils have been noted in previous floods, specific measures, such as providing additional weight (stability berms) and drainage, could be implemented to reduce the risk of piping.

Foundation 'blow-up'

Foundation blowup (or heaving) occurs in situations where water flows through a permeable layer beneath the stopbank and then exerts pressure on a less permeable surface layer of soil. If the pressure exceeds the weight of the surface soil it is displaced upwards. As well as the potential for physical damage to the stopbank due to ground displacement, the disruption of the confining layer of soil can lead to large concentrated flows and the potential for piping as discussed above.

Based on the results of the geotechnical investigations, the typical soil profile beneath the stopbanks consists of approximately 1m of silt overlying gravelly sand. The base of the silt layer is approximately 3m below the "design standard" flood level. Under worst case conditions with a direct hydraulic connection of the river to the gravelly sand layer, water pressures of nearly 30 kPa could be possible on the base of the silt layer. The weight of the silt layer to resist this uplift pressure is only of the order of 18 kPa, therefore the potential for a foundation blowup failure exists.

Accurately modelling this situation is not possible, without a detailed knowledge of the interconnection of the foundation layers with the river. Lower permeability zones that prevent flow through the gravely sand layer will reduce the potential uplift pressure on the silt layer. We understand that in previous floods there have been reports of flows beneath the stopbanks, but not reports of ground disturbance on the land side of the stopbank. This mechanism of failure cannot be discounted particularly in areas where the gravely sand layer has a low gravel content. The same mitigation measures as discussed earlier could be utilised to mitigate any known problem areas along the stopbanks.

River Erosion

Erosion of the existing stopbanks and their foundations can occur during flood conditions. It is understood that failure of the system occurred during a flood in 1957, due to this mechanism.

Preventing failure by this mechanism requires the river side slopes of the stopbank to have adequate protection against high velocity flows during flood events. This is normally achieved by maintaining a good coverage of grass. In areas (normally at bends in the river) where the stopbanks and its foundations are exposed to particularly high velocity or directed flows more specific protection, such as rock lining, is often required.

Areas at risk from erosion have been identified along the Motueka River stopbanks. For example, the area from Woodman's Bend through to Whakarewa Street has had substantial rock bank protection works carried out over the last three years. This has included the adding to and re shaping of the existing rock protection works to obtain a higher standard of protection than what was previously offered. Similar protection works have recently been undertaken on both the left and right banks downstream of Woodman's Bend and Blue Gum Corner.

Understanding and addressing the location of areas that will be at risk during a "design standard" flood event is a key component of managing the risk of an erosion failure along the stopbanks.

Vegetation

Vegetation cover on the stopbanks can be beneficial in that:

- it reduces river velocity and helps resistance to erosion by flood waters
- it can act to stabilise the downstream slope
- topsoil on the river face can restrict the rate of seepage into the stopbank during a flood.

However, if vegetation cover is not controlled and is allowed to take the form of plants with significant root system it can be problematic. Roots that grow into the stopbank can increase seepage and increase the likelihood of piping. If trees are ripped out of the river bank during a flood and their root system extends into the stopbank, considerable damage can be done that could ultimately result in failure of the stopbank.

The Motueka stopbanks are mowed for vegetation control at least once annually from the mouth to Blue Gum corner on the left bank and to Woodman's Bend on the right bank. The connecting Brooklyn Stream stopbank is only partially mowed on the left bank because of the difficulty in obtaining access along the narrow stopbank section. The stretches of stopbank most at risk from vegetation related failure modes have been identified by visual inspection and condition assessment carried out in 2005. This information is contained within MWH report: "Lower Motueka River Stopbank Scheme Review" dated 17 August 2006.

Interference with Existing Stopbank

There are a number of ways in which the stopbanks are interfered with, or altered, that could have a detrimental effect on their performance during a flood event. These include:

- Animal burrows (particularly rabbits) that can open up seepage paths within the stopbank increasing slope instability and piping risks. Installations of items such as power poles and fence poles can have similar effects.
- Gravel extraction in the berms close to the river side toe of the stopbanks increasing the risk of river erosion.
- Excavations into the stopbank to allow construction of structures such as farm sheds, which can reduce the overall slope stability of the stopbanks and allow shorter paths for seepage water to initiate piping.
- Cattle and vehicle crossing points on the stopbank potentially causing localised low points increasing overtopping risk.
- Stock grazing damage on the stopbanks harming vegetation cover and reducing erosion resistance.
- And also, modifications to the river bed which may cause changes in river hydraulics and reduce support to the stopbanks.

These issues need to be managed through enforcement, regular inspection and maintenance, and discussions with landowners whose properties are adjacent to the stopbank.

Mapping of Failure Modes

Drawings (see Appendix D) have been prepared to illustrate the areas that are considered most at risk from each of the potential failure modes. In particular.

- The "Stopbank Raise Crest Summary" plan illustrates those sections that are predicted to be overtopped during the "design standard" flood. All coloured areas are predicted to be overtopped in a 1% AEP rainfall event (with 2090 Climate change allowance).
- The "Areas of Stopbank Vulnerability" plan shows a range of geotechnical information which correlates to risks around piping of the foundation. Historical areas where piping has occurred are illustrated along with depths of silt that cap the underlying more permeable silts and gravels. Areas where the silt cap is thinnest will present the highest risk of foundation piping.
- The "Areas of Stopbank Vulnerability" plan shows the location of known penetrations through the stopbank. At each of these locations there is an increased risk of seepage and piping failure.
- The "Areas of Stopbank Vulnerability" plan shows areas of high river erosion risk, areas of vegetation that may affect the stopbanks and areas of gravel extraction that are adjacent to the stopbank and may have some negative impact on its performance.

An overall understanding of the geotechnical risk can be gained from these drawings. 'High' risk areas are defined as those areas that.

- Have exhibited seepage either beneath or through the stopbank during previous floods indicating an enhanced potential for piping of either the stopbank or its foundation as well as slope instability and foundation blow up issues.
- Have yielded high permeability results during previous testing indicating an enhanced potential for seepage which could result in piping or slope stability issues.
- Have penetrations through the stopbank that increase the potential for concentrated seepage and piping.
- Have significant modification of the river berm due to gravel extraction, potentially opening up higher permeability paths beneath the stopbank which may enhance piping and foundation blow up risks.
- Have significant vegetation in the vicinity of the stopbank with root systems that could provide seepage paths into the stopbank and increase the piping risk.

None of the stopbank sections are considered to have 'low' geotechnical risk due to the lack of certainty regarding quality control during construction, the often steep sided and narrow nature of the stopbank, and the anticipated presence of permeable gravels beneath all of the stopbank meaning that under-seepage is inevitable. All sections of the stopbank that are not considered high risk as defined above are therefore considered to have moderate geotechnical risk.

Hydrology – "Design Standard" Flood Estimation

Previous work completed by MWH for Tasman District Council in 2006 (*Stopbank Options – Pre-Feasibility Report, Lower Motueka River*) and 2008 (*Preliminary Design Engineering Report - Lower Motueka River Flood Control Scheme*) have derived design flood magnitudes and hydrographs based on analyses of recorded flow data on the Motueka River.

The design hydrograph shape, as derived in 2006, for the 1% and 2% AEP flood events at Alexander Bluffs Bridge was based on the flood event recorded on the Motueka River at the Woodstock recorder site on 12 May 2001. This was deemed to be a representative shape.

Before the MWH investigations, earlier analysis by Howes (1994) also derived design flood estimates for the lower Motueka River.

Table 1 details the various design flood estimates for the lower Motueka River that have been derived previously. The original design flood of 2830 m^3/s was derived in 1958 in the original stopbank design.

Table 1: Previous Flow-Record Based Design Flood Estimates for Lower Motueka River (m^3/s)

AEP	**Original (1958)**		**1994 (Howes)**		**2006 (MWH)**	
	Woodstock	Alexander Bluffs	Woodstock	Alexander Bluffs	Woodstock	Alexander Bluffs
2%	2576*	2830	2164	2225	2060	2316
1%			2384	2496	2342	2633

Calculated here by catchment area ratio ($A^{0.8}$) from Alexander Bluffs value

The 1958 estimate was based on very little available hydrological information – continuous monitoring of flow in the Motueka River did not begin until 1969.

The estimates derived in 1994 used 30 years of annual flood data from the Motueka at Woodstock site. This included continuous recorded data from 1969 and peak flow estimates of a small number of historical flood peaks prior to 1969.

The largest recorded flow for the Motueka River at Woodstock is 2148m3/s on 10/07/1982. Based on catchment area at Woodstock (1750km2) and Alexander Bluffs (1968km2) this equates to a corresponding flow of 2360m3/s at Alexander Bluffs.
The 1994 and 2006 estimates are significantly lower than the original design estimates from 1958 (approximately 20% lower) [the 2006 analysis was based on 34 years of recorded flow data at the Woodstock site]. These differences in flood magnitude can be attributed to the lack of hydrological information available at the time of the original estimates in 1958.
MWH engineers recommended that, in an effort to derive a more accurate picture of “design standard” flood flows and hydrograph shapes, a rainfall-runoff model be developed and calibrated with recorded catchment rainfall and river flow. Design rainfall totals and temporal patterns could then be input to the model to produce a “design standard” flood event for subsequent use in hydraulic modelling of the river channel and flood protection measures.

Hydstra Modelling software was used to create a rainfall-runoff model of the Motueka catchment. The model was calibrated to the Motueka at Woodstock flow recorder using seven rain gauges in the 1750 km^2 catchment area above the Woodstock site. The model is an initial-continuing loss model that assumes an initial loss of IL mm before any rainfall becomes effective runoff. After this amount is satisfied a continuing loss rate of CL mm/hour is applied to the rainfall inputs.

Runoff from the effective rainfall is moved down the drainage channels using non-linear channel routing. Two parameters; a channel lag parameter (Alpha) and, a non-linearity parameter (n) are adjusted in addition to the rainfall losses to calibrate the model to recorded flow data.

Five storm events (August 1990, December 1997, two in October 1998, and January 2004) were used as the calibration events. The model parameters were adjusted for each event until the modelled flow output at Woodstock matched the actual recorded flow at that site.

The average model parameters from the calibration events and adopted in the final design model are:

- IL = 10mm
- CL = 3mm
- Alpha = 1.4
- N = 0.78

These are expressed globally across the entire model area.

Design rainfall totals for the model – 1% and 2% AEP – were derived from the High Intensity Rainfall Design System (HIRDS) software. The predicted effects of climate change in 2090 were calculated and design rainfall totals that reflect a two degrees Celsius warming (MfE guidelines) were also derived (based on average of a range of predicted outcomes).

Investigations by NIWA (2010) have shown that climate variability (such as the Inter-decadal Pacific Oscillation and the El Nino Southern Oscillation) have imprinted no trend on the record of annual maximum floods in the area.

The rainfalls were distributed in time by adopting the standard South Island temporal pattern used in Probable Maximum Precipitation (PMP) analyses. However, feedback from MWH engineers suggested that a different temporal pattern also be used which results in flow being relatively high for a longer period of time. The May 2010 flood event is seen as representative of this and so the temporal rainfall pattern displayed through this event was also applied to the design rainfall.

Table 2 details the “design standard” flood results of the rainfall-runoff modelling.

Table 2: Rainfall-Runoff Based Design Flood Peak Flow Estimates for Motueka River at Alexander Bluffs Bridge

AEP	Standard Temporal Pattern				May 2010 Temporal Pattern			
	12 hour	24 hour	48 hour	72 hour	12 hour	24 hour	48 hour	72 hour
1%	2659	3165	2643	2076	2874	3694	3693	3452
1% + 2090 climate change (Design Standard for this report)	3354	4053	3427	2721				
0.5%	2970	3550	2547	2339				
0.5% + 2090 climate change		4598						

The 24-hour event is the critical duration for flood magnitude at Alexander Bluffs.

The May 2010 temporal pattern produces higher peak flows than the standard temporal pattern. Figure 1 shows the "design standard" 24-hour and 48-hour duration flood hydrographs with the standard and May 2010 temporal rainfall patterns.

Figure 2 shows a comparison of the standard and May 2010 temporal distribution of the 24-hour 1%AEP rainfall. The May 2010 pattern has much higher intensities for a period of time leading to a higher peak flow - 80% of the total rainfall occurs in the first half of the event. The standard temporal pattern has rainfall occurring over the entire duration and is therefore subject to a relatively higher amount of modelled losses (continuing loss to infiltration and interception etc), which explains the apparent difference in runoff volume under each of the corresponding hydrographs in Figure 1.

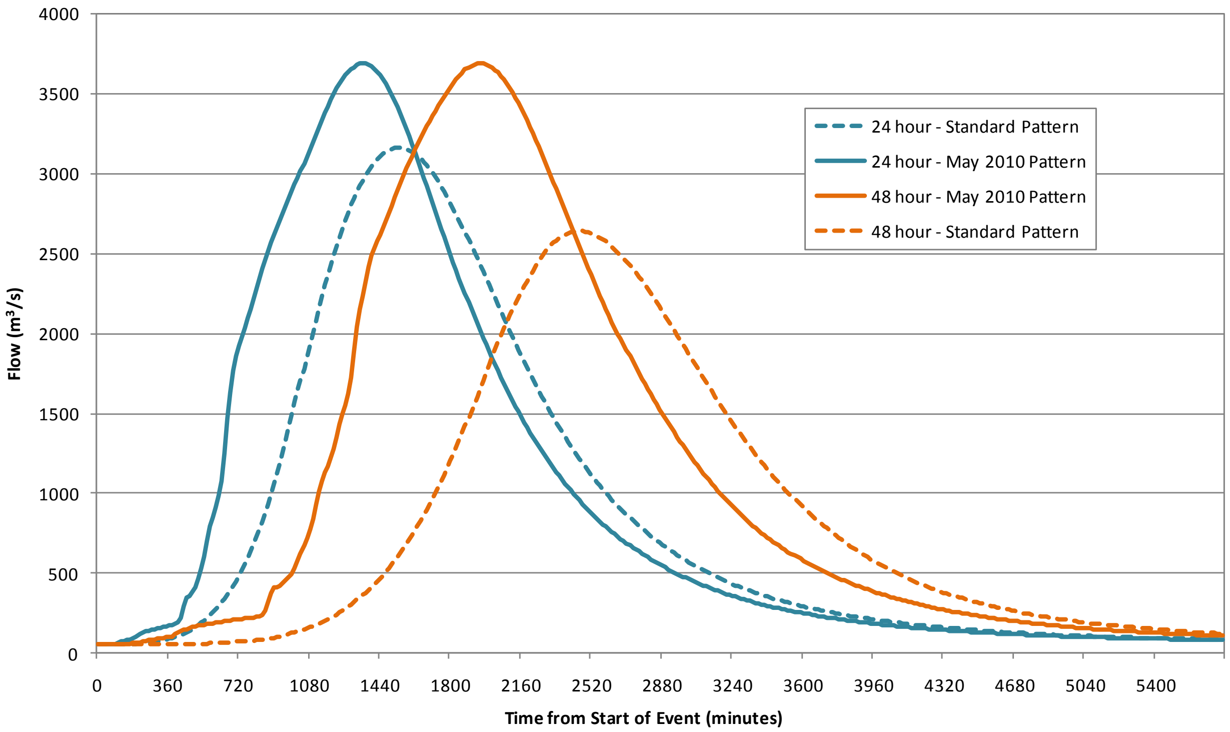


Figure 1: Hydrograph comparison for Motueka at Alexander Bluffs

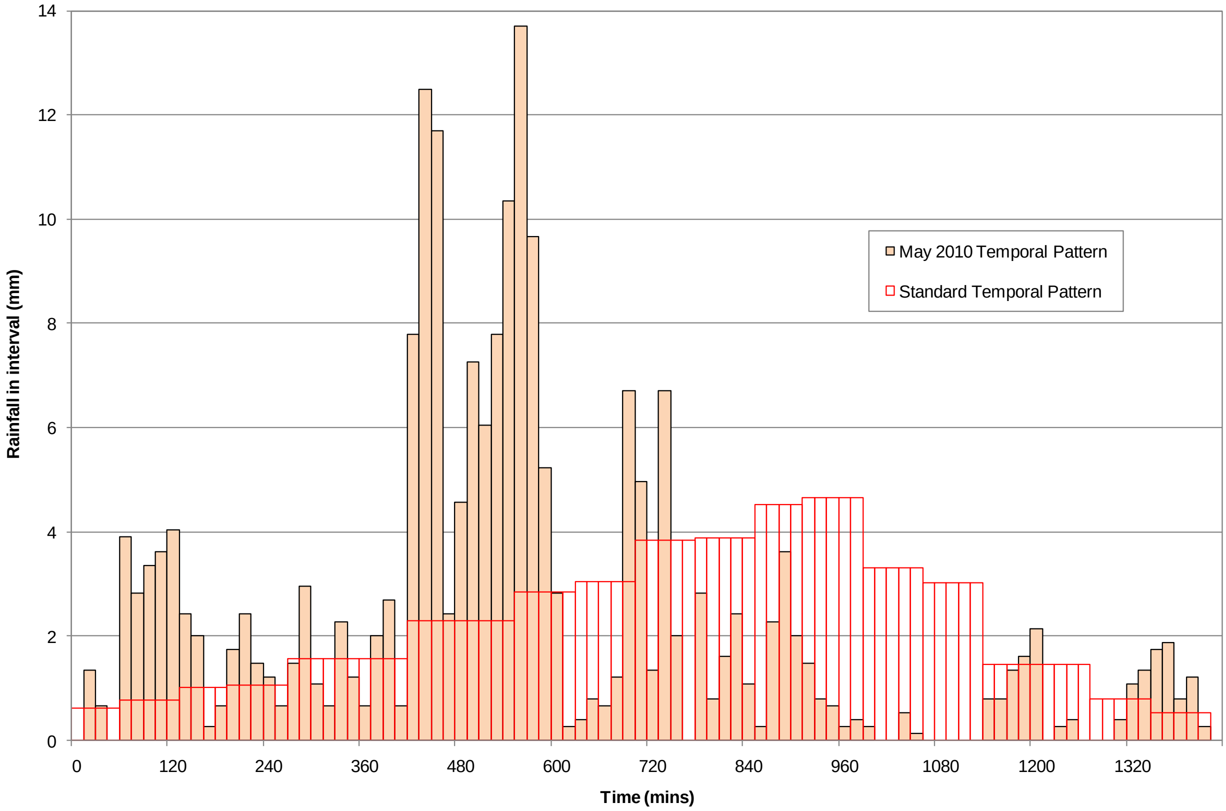


Figure 2: Comparison of the standard and May 2010 temporal distribution of the 24-hour 1%AEP rainfall (which may be indicative of climate change resulting in higher intensity events)

A preliminary frequency analysis of the recorded flow data for the Motueka River has been carried out using data from the Motueka River at the Woodstock site between 1962 and 2010. This updates the 1994 (Howes) and 2006 (MWH) flood frequency estimates.

The results are presented in Table 3, and factored by the catchment area ratio method to represent the Alexander Bluffs site. The table summarises all the "design standard" flood estimates discussed here.

Table 3: Summary of Design Flood Peak Flow Estimates for Lower Motueka River (m^3/s)

AEP	Original (1958) Probably from rainfall estimation		1994 (Howes) from river flow data		2006 (MWH) from river flow data		2010/11 from river flow data		2011 From Rainfall Run-off Model
	Wood-stock	Alexander Bluffs	Wood-stock	Alexander Bluffs	Wood-stock	Alexander Bluffs	Wood-stock	Alexander Bluffs	Alexander Bluffs
2%	2576*	2830	2164	2225	2060	2316	1978	2175	
1%			2384	2496	2342	2633	2200	2420	3165

*****_Calculated here by catchment area ratio ($A^{0.8}$) from Alexander Bluffs value_

We assessed the 1% AEP event with climate change as follows:

We have determined the predicted temperature increases for the Tasman-Nelson region to be 0.6 degrees Celsius by 2020, 1.1 degrees by 2050 and 2 degrees by 2090. These have been derived from the MfE Local Government Guidelines and from the NIWA report *Climate Change and Variability – Tasman District*.

The predicted temperature increases are the "average" change for six emissions scenarios. Temperature increases are explicitly provided in the MfE and NIWA documents for the years 2040 (0.9 degrees) and 2090 (2 degrees). NIWA advise a rate of warming of 0.2 degrees per decade which allows the increases for 2020 and 2050 to be derived.

The predicted temperature increases are input to the HIRDS software for each rain gauge location in the rainfall runoff model and design rainfall totals for each are output.

Figure 3 shows the derived 1% AEP design flood hydrographs for the Motueka River at Alexander Bluffs Bridge. All are based on a 24 hour duration rainfall event.

Table 4 details the effect of climate change on the peak flows.

Table 4: Climate Change effect on Rainfall-Runoff Model Peak Design Flow – Motueka River at Alexander Bluffs Bridge

Year	Predicted Temp Increase (°C)	1%AEP Peak Flow (m3/s)
Current	0	3165
2020	0.6	3425
2050	1.1	3645
2090	2.0	4053

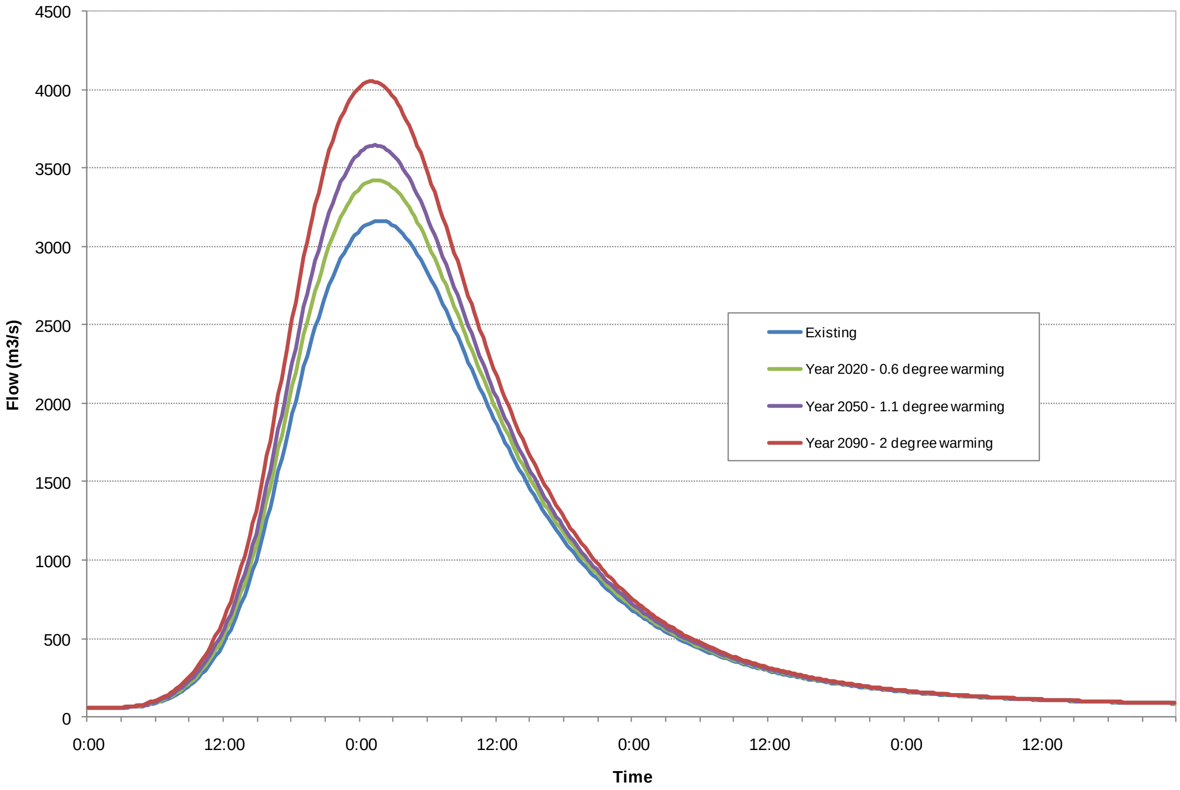


Figure 3: Rainfall-Runoff Based Design Flow 12 hour Hydrographs - Motueka River at Alexander Bluffs Bridge

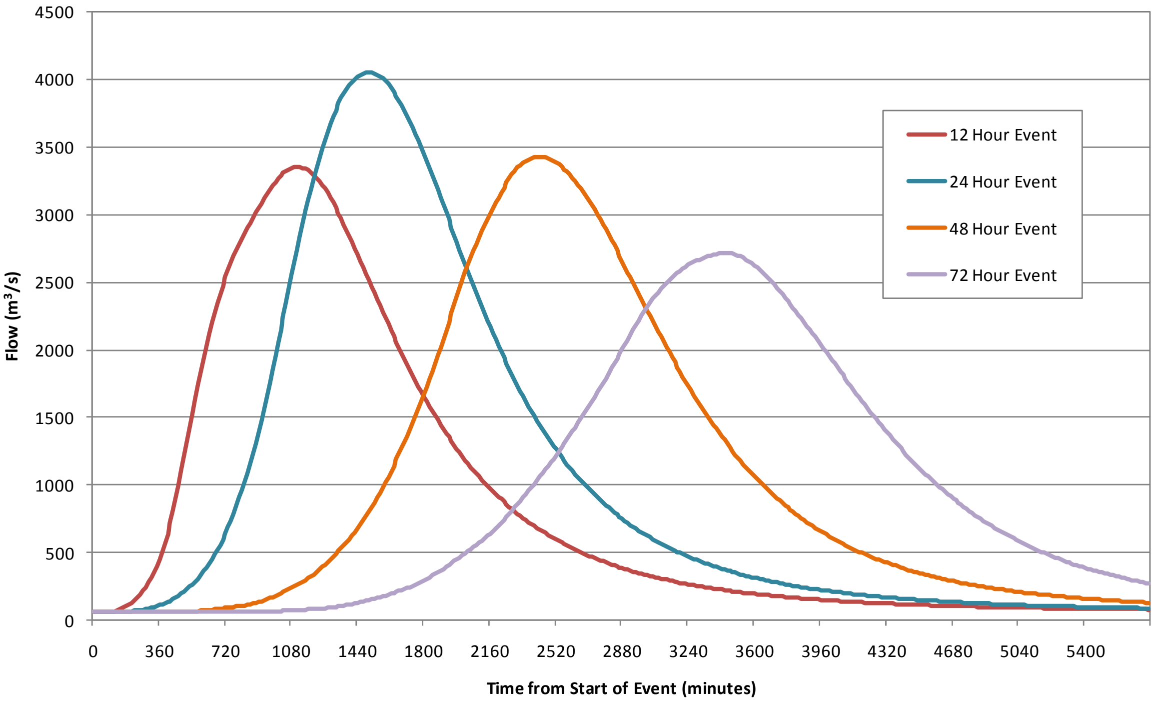


Figure 4: Motueka at Alexander Bluffs Hydrographs of Modelled Design Floods 1% AEP with Predicted Climate Change at 2090

Uncertainty Statement

The earlier hydrological analysis was carried out on flow information estimated from the Woodstock and Woodman's Bend stage recorders. Peak flood estimates at Woodstock are based on extrapolation of peak flows recorded over the period 1969 to 2010. The estimates of flows and design food peaks are subject to errors from a range of sources.

Uncertainties associated with estimating flows come from two main sources. Firstly the measurement of stage (water level) is expected to be within about 1% of actual level. Secondly, the conversion of stage to flow through the use of a rating curve is expected to be within 8% of actual flow except during flood conditions when measurement conditions are more difficult and the error band on the estimate is likely to be at least +/- 15%.

Errors associated with estimating design flood flows from the recorded flood peaks are mainly associated with sampling error which occurs because the frequency analyses are based on limited lengths of record. The table below shows the standard deviation for design flood estimates from frequency analysis of Woodstock data. It indicates that we are 95% confident that the estimated 100 year Average Return Interval (ARI) flow is within +/- 530 m^3/s of 2,308 m^3/s. That is, there is a 95% chance that the 100 year flow is between 1,778 and 2,838 m^3/s.

Uncertainty Associated with Frequency Analysis

ARI (yr)	Flow (m^3/s)	1.96 std. dev. (m^3/s)
100	2,308	530
50	2,017	440

Cumulative Probability of a 1% AEP Flood for Varying "Life" of Stopbanks		
"Life" of Stopbank (Years)	% Probability of Flood Event During "Life"	Chance of Flood Event During "Life"
1	1%	1 in 100
10	10%	1 in 10
20	18%	1 in 5.5
30	26%	1 in 3.8
40	33%	1 in 3.0
50	39%	1 in 2.6
60	45%	1 in 2.2
70	51%	1 in 2.0
80	55%	1 in 1.8
90	60%	1 in 1.7
100	63%	1 in 1.6

Development of Options for Evaluation

The flood protection options that are considered for evaluation are described in the following sections. Implicit to the stopbank upgrade options is an assumption that Tasman District Council will be carrying out a maintenance and inspection programme of whichever option is selected. This maintenance and inspection programme will include the following tasks to maintain the community's flood protection asset.

- Education of landowners to impart the importance of the asset and its well being.
- Monitoring and correction of landowner activities adjacent to the asset and across the asset.
- Inspection of the condition of the flood protection asset including the stopbank, flood berm and river channel.
- Maintenance of the grass cover over the stopbank and berms through cooperation with adjacent landowners.
- Maintenance of flood warning systems and lifelines emergency networks.
- Maintenance of existing storm-water culverts and pipelines running under the stopbank.
- Clarification of legal and town planning provisions, regulations, rules and responsibilities.
- Consultation with landowners and gravel extraction operations.

A budget allowance of $100,000 per year for maintenance is taken as a starting budget for this study.

The design standard for the Lower Motueka River flood protection scheme that sets stopbank crest longitudinal profiles is the 1% AEP flood event, with climate change allowance to 2090 projections, and 500mm freeboard.

Maintaining the Status Quo

The status quo option applies to the work proposed on the existing stopbanks which would be minimal in this case. The stopbank crest profile and cross section would remain the same as in the existing case. Ongoing Council maintenance and inspection operations would seek to minimise the deterioration of the condition of the stopbanks through maintenance of grass surfacing, river bed operations to prevent berms being eroded away, and the regulation of human activities around the flood protection asset.

This upgrade option has no significant capital expenditure above programmed, annual, asset maintenance costs.

The Rebuild Option

The rebuild option involves demolition of the existing stopbank cross section and replacement with new stopbank material sourced from the existing stopbanks, river berms and imported from further afield. The stopbank crest profile would match the design standard including 500mm freeboard and the cross section would meet current best practice side slopes and foundation treatment, and be compacted into place using modern equipment and techniques.

The Refurbish Option

Compared to the rebuilding option, refurbishment concentrates on leaving as much of the existing stopbank as possible intact.

A key requirement of the stopbank system is to be of sufficient height to contain the “design standard” flood. Analysis indicates that in many locations the stopbank would currently be overtopped by the “design standard” flood. Consideration has been given to providing spillways that limit the flood level and thereby prevent overtopping of the stopbank, however, all of the paths for spillway involve some inundation of properties and this is not a preferred option. To contain the “design standard” flood the stopbank must therefore be raised in some locations.

Raising the existing stopbank to contain the “design standard” flood is not straightforward. The crest width is typically 2m or less and is only just sufficient for light vehicle access. There is insufficient width to simply place additional fill on the stopbank to increase its height. Options to raise the stopbank therefore are to either place fill on the river side or landward side of the bank. Along almost the entire length of the stopbank there is agricultural development right up to the toe of the stopbank. Raising the stopbank by placing additional fill on the landward side would therefore be disruptive to the current land use. On the river side there is generally tens of metres of river berm before reaching the active river channel and raising the stopbank on this side appears comparatively straightforward provided any land ownership issues can be resolved. Any fill placed on the river side has the advantage of providing an additional barrier to saturation of the stopbank and enhancing its stability.

Investigations have been undertaken on the river berm to identify potential sources of material for stopbank construction. A variable thickness of silt (river flood deposits) has been identified that appears to be a viable source of low permeability fill to both raise the stopbank and to achieve a low permeability blanket on the upstream face. The properties of these materials have been estimated by undertaking an initial testing programme on the material to establish its properties. Key conclusions from the testing were that:

- The range of permeabilities from the test programme was $4x10^{-7}$ to $1.6x10^{-8}$ m/s.
- Approximately 150,000 m^3 of potential low permeability borrow material was identified on the right bank river berm in four borrow areas.
- Approximately 700,000 m^3 of potential low permeability borrow material was identified on the left bank river berm in 10 borrow areas.

The refurbishment would occur along the “full” length of the stopbanks except for:

- The ring stopbank around Peach Island. The Peach Island stopbanks are lower than the current stopbank system and the island will flood under high flow conditions to provide attenuation to downstream sections of the river. Procedures for evacuation already exist to address the risks of flooding for the residents of the island.
- The Brooklyn Stream stopbanks. Analysis indicates that these stopbanks are not expected to be overtopped during the “design standard” event and there is limited space in which to implement any refurbishment works.

The refurbishment concept is illustrated on drawing SB02 and allows the stopbank to be raised as required to contain the predicted 1%AEP (+2090 climate change) flood event with 500mm of freeboard. In sections where no stopbank raising is required to achieve the “design standard” level, a blanket of silt would be placed up to the existing crest level. Compared to the rebuilding option the re-profiled stopbank section has a reduced crest width and steeper slopes, and therefore does not offer as much security as the rebuilt option.

Initial estimates, that would need to be verified by more detailed terrain modelling, indicate the requirement for 85,000 m^3 of fill on the left bank and 135,000 m^3 of fill on the right bank. Comparison with the borrow area volumes estimated above, it can be seen that there appears to be ample borrow volume available on the left bank, but volumes available on the right bank may be marginal. There may need to be the need to transport material from the left to the right bank either via the existing road system or via a dedicated temporary crossing of the river.

The refurbishment would also include improvements to the detailing of penetrations through the stopbank. Penetrations such as pipelines through any water retaining structure are well known problem areas because of the potential for concentrated seepage along the conduit that can lead to soil erosion. A number of penetrations pass through the stopbank and these would need to be addressed as part of the upgrade. The refurbishment works would consist of local excavation of the stopbank around penetrations so that filter and drainage materials could be placed that will control the flow of water along the conduit and safely discharge it on the landward side of the stopbank.

River Protection Improvements

Any flood control scheme should include the adoption of improved management of the river channel and river berm as the initial line of protection of the stopbank. It is known that a previous failure of the stopbank (1957) was due to river erosion, highlighting the importance of river protection to the safety of the stopbank. Based on experience with the river system, the following sections have been identified as benefitting from additional erosion protection works.

- Blue Gum Corner on left bank downstream for 1200 metres
- Section starting 200 metres upstream of SH60 Bridge and then 500 metres upstream on left bank.
- From SH 60 Bridge downstream on left and right banks over a distance of approximately 400 metres.
- From SH60 Bridge upstream for 200 metres on right bank.
- Section in front of Motueka Metal yard on right bank over a distance of approximately 400 metres.
- One kilometre section opposite Fry's Island on right bank.(Breached section 1957)
- Area from Corrie – Johnson pit on right bank to Johnston Pond area.(approx 600 metres)
- From Whakarewa Street on right bank to Woodman's corner (approx 1100 metres).

During inspection of the stopbank it was noted that there were a number of gravel extraction pits in close proximity to the stopbanks. These pits introduce the potential for increased river erosion of the stopbank due to a reduction in berm width and increased seepage beneath the stopbank due to increased exposure of permeable gravels to flood waters. This situation could be improved by better management of the location and backfilling of these pits. The upgrade works would involve:

- enhanced river protection works
- removal of any vegetation from the berm with roots that may grow under the stopbank
- maintenance of grass cover on the stopbanks
- netter control of gravel extraction on the river berm
- better control of backfilling.

Partial Refurbish Sub-Options

Introduction

To focus on where investments in upgrades would bring the most benefit it is necessary to understand which areas of the stopbank represent the most risk to people and property, if failure was to occur. The following figure illustrates the relative damage potential of different segments of the stopbank.

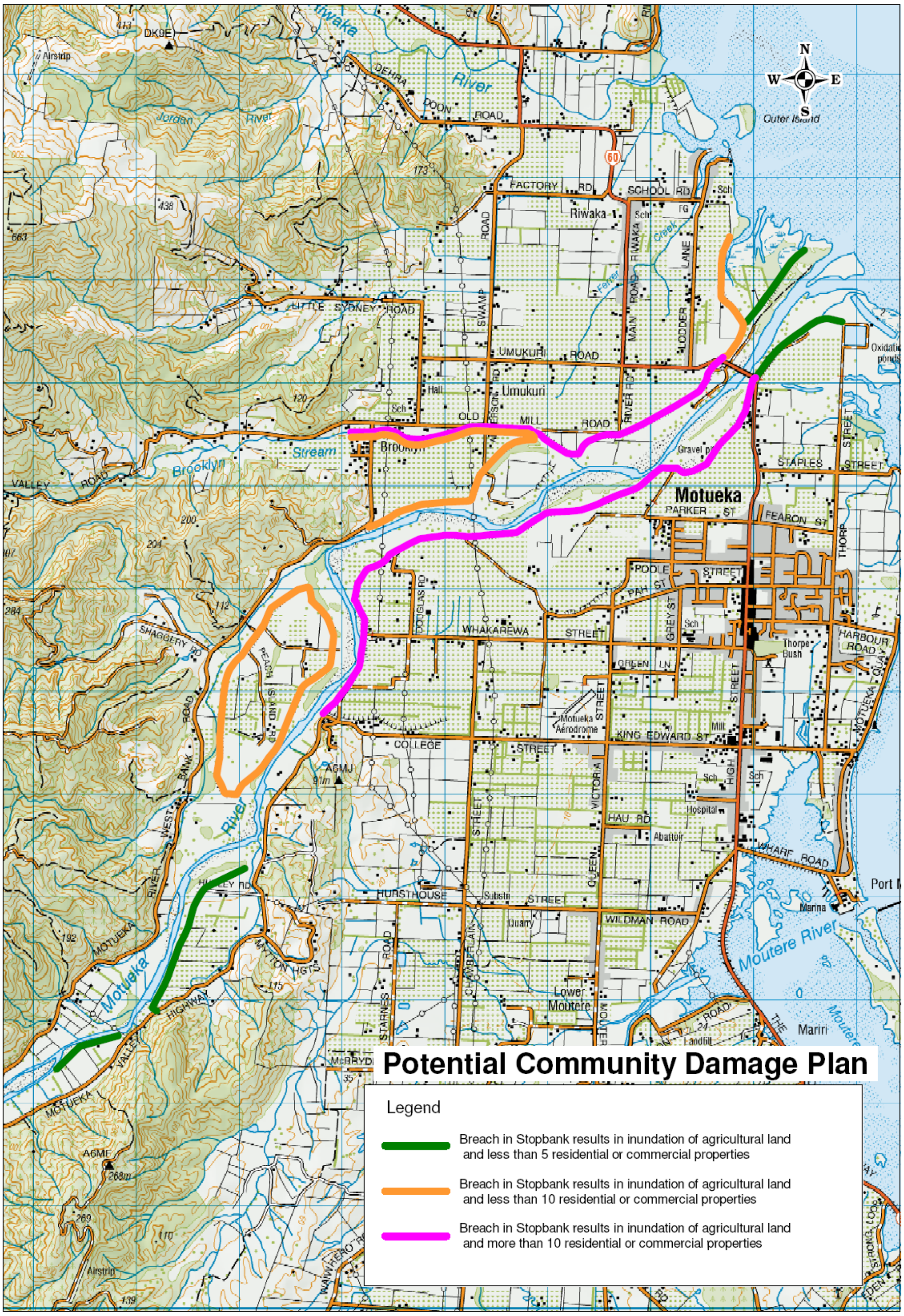

Potential Community Damage Plan
Legend
Breach in Stopbank results in inundation of agricultural land and less than 5 residential or commercial properties
Breach in Stopbank results in inundation of agricultural land and less than 10 residential or commercial properties
Breach in Stopbank results in inundation of agricultural land and more than 10 residential or commercial properties
Motueka
Riwaka
Umukuri
Brooklyn
Lower Moutere
Moutere River
Outer Island

Figure A4

It can be seen that the stretches of stopbank that protect the highest value assets are:

- between College St and the State Highway bridge on the right bank where the stopbank protects parts of Motueka (a distance of 6,900m)
- between River Rd and the State Highway on the left bank where the stopbank protects Riwaka (a distance of 2,500m)
- the left bank of Brooklyn Stream. However, as noted in section 4.3, it is not intended to refurbish these stopbanks as part of the Motueka River flood control scheme. Protection will need to be considered separately for flood events in the Brooklyn catchment.

Improvements to the above-mentioned sections of stopbank will have the greatest impact on reducing the overall level of risk to people and property protected by the stopbank. This option therefore concentrates on upgrade options to improve these sections of stopbank. With respect to the remaining sections of the stopbank the following comments are made:

- The upper right bank stopbank between Ch 11500 and 15000 (known as the Hurley and Kiwifruit stopbanks) protect only small areas of agricultural land.
- The Peach Island stopbanks are lower than the current stopbank system and the island will flood under high flow conditions to provide attenuation to downstream sections of the river. Procedures for evacuation already exist to address the risks of flooding for the residents of the island.
- Downstream of the State Highway Bridge the stopbanks have been overtopped previously without significant damage to infrastructure.

The partial refurbishment could be configured in a number of different ways, as discussed in the following sections. Common to each of the options are the improvements to erosion protection and river management discussed above, but restricted to the critical lengths identified above.

Options with Freeboard of 500mm to Design Flood

Under these options the ‘critical’ section of stopbank would be raised as required to achieve a minimum freeboard of 500mm above the design flood. This would be achieved by the placement of fill on the river side face of the stopbank. Two sub options have been considered, namely:

- Option B1 – place fill on river side face of stopbank only where an increase in freeboard is required.
- Option B2 – place fill on river side face of stopbank along entire length of ‘critical’ stopbank.

The advantage of option B2 is that it addresses both the potential for overtopping and reduces the likelihood of instability of the stopbank, albeit at a higher cost. Option B1 involves the upgrade of approximately 6,200 m of stopbank, while option B2 involves 9,400 m of stopbank. Discussion of the trade off in cost versus risk is presented in section 8.

The Effects of Gravel Extraction and Volumes

The effects of gravel extraction on the flood peak and the design stopbank profile were assessed as follows.

Removing a scrape of gravel from the Lower Motueka River flood berms between the stopbanks, over the extent as shown in the attached sketch plan in Appendix D, is summarised as:

Depth of Gravel Removal	Volume of Gravel (m3)
1m depth	717,000
2m depth	1,878,000

Cross section areas were calculated for all official Tasman District Council river cross sections downstream of Woodman's Bend. The distance between each cross section was measured off plans and multiplied by the area of the cross section. All cross section reaches were individually summed to give a total volume.

It may be possible that gravel extraction costs could be offset by sales of gravel to the construction industry, but would be unlikely to yield significant royalties to Tasman District Council (as most of the land is in private ownership). This would probably be done in a timeframe that suited industry demand for gravel, and may take many years. Therefore the optimistic scenario would be zero cost to Tasman District Council to have the gravel extracted from the berms.

Alternatively if the gravel was extracted, carted and stockpiled in a shorter campaign then costs could be similar to excavation and disposal operations and be $15 to $30 per cubic metre depending on the distance and the stockpile requirements. This would cost over $10 million for the 1m depth cut at the $15/m3 rate. This would be a significant cost for only achieving a reduction in over-topping risk.

Other channel management may include "blading" gravels and smoothing the flood berm by pushing materials towards the stopbanks whilst increasing the capacity of the central flood berm. This could cost in the order of $1 to $2 per cubic metre, or $700,000 to $1.4 million for the 1m depth cut operation and $1.8 million to $3.5 million for the 2m depth cut operation.

Potential Change in Top Water Levels

The change in the top water level profile is summarised in the following chart: a 1m cut extraction option provides an average of 0.3m crest reduction, and a 2m cut extraction option provides a 0.7m average stopbank crest reduction.

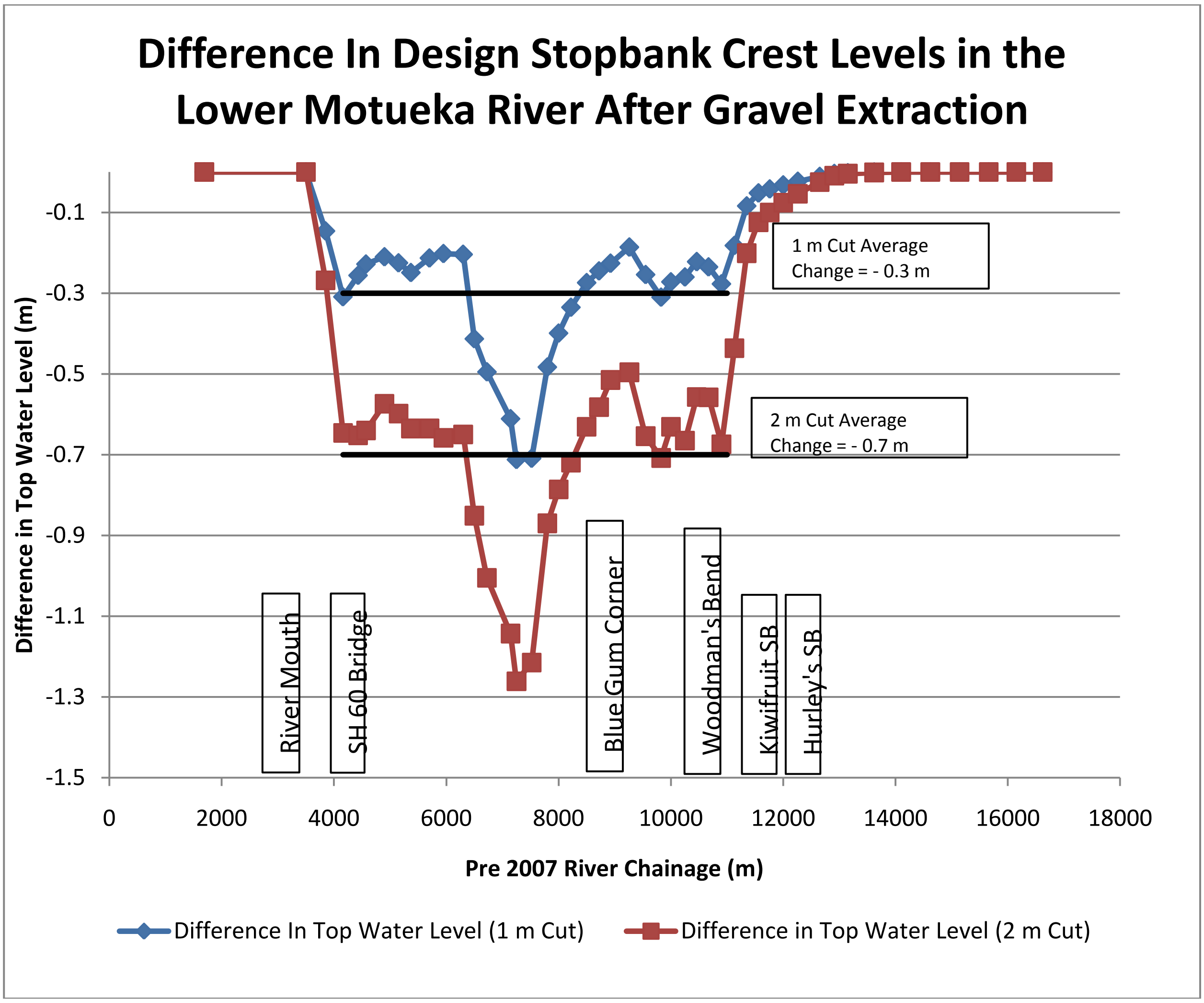


Figure 5: Reduction in Top Water Level Profile along the Lower Motueka River

The change in top water level at each cross section was taken to be the change in required stopbank crest level. The design relationship is: Required crest level of stopbank = 1% AEP (+ 2090 climate change allowance) top water level + 500mm freeboard.

Stopbank Volumes

As the removal of gravel will reduce the top water level, the required height of stopbank would be lower. Therefore one benefit of gravel removal would be to reduce the required size of stopbanks (lower volumes of earthworks).

To compare the size of required earthworks, the stopbank volumes downstream of Woodman's Bend were calculated for three cases: the proposed rebuild design, the 1m deep gravel extraction, and the 2m deep extraction.

Scenario	Right Bank Volume (m3)	Left Bank Volume (m3)	Total Banks Volume (m3)	% reduction
Rebuild stopbank to 4m crest, 1 to 3 wet face, 1 to 2.5 dry face over lower Motueka River left/right banks, downstream of Woodman's Bend			275,000	
1m cut in flood berm, reduction in stopbank volume	15,196	10,004	25,200	9%
2m cut flood berm, reduction in stopbank volume	38,150	25,239	63,389	23%

The 1m cut and 2m cuts into the river flood berms give approximately 9% and 23% stopbank volume reductions. The rate for stopbank earthworks volume is $15/m3 placed. With mark-ups for preliminary and general (5%), contingency (30%), resource consent (2%), and engineering (15%) the budgeting rate is more like $22.8/m3 placed.

	Cost Reduction based on reduced volume before percentage mark ups	Cost reduction including percentage mark ups
1m cut scenario	$378,000	$575,000
2m cut scenario	$950,000	$1,450,000

Total cost for the <u>rebuild</u> option from Woodman's Bend to the mouth has a budget estimate of $9.9 million including percentage mark-ups. Therefore, the stopbank budget estimate with the reduction in stopbank volumes is $9.3 million (94%) for the 1m cut scenario, and $8.5 million (85%) for the 2m cut scenario.

The <u>refurbish</u> option involves 220,000m3 of stopbank fill placement for $8.6 million. The effects of gravel extraction would reduce budget estimates to $8 million (94%) for the 1m cut scenario, and $7.2 million (83%) for the 2m cut scenario.

It would be uneconomic to pay for gravel removal in one operation. The likely scenario would then be gravel removal by industry (at their cost) over several years. If the stopbanks were constructed to the lower height to account for complete gravel removal, then there would be period of time when the stopbanks were at risk of over-topping. Therefore the intended level of protection would not be attained for several years.

Impact of Refurbishment on Failure Modes

The impact of the stopbank upgrades on each of the failure modes identified in section 2.2 is discussed in the following sections.

Slope Instability

The critical consideration for stability of the stopbank is the degree of saturation during a flood event. The placement of fill on the river side slope of the stopbank provides the opportunity to significantly reduce the potential for saturation during flood events.

If river berm silt is used to achieve the stopbank raise, the impact on stopbank saturation can be quantified utilising permeability data collected as part of ground investigations. Permeability testing results ranged from 4×10^{-7} m/s to 1.6×10^{-8} m/s. Given that laboratory permeability results are often difficult to replicate under field placement conditions, a permeability value of 1×10^{-6} m/s is considered to be reasonable for design purposes.

The analysis presented in section 2.2.1 is repeated below with allowance for a riverside blanket of river silt associated with the stopbank raising. The new fill is shown in blue in the figure (A5) below and the existing fill shown in green.

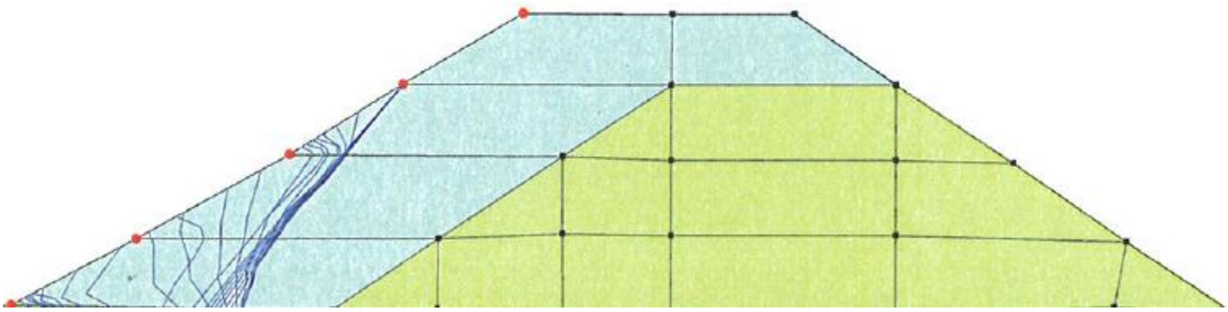


Figure A5: Predicted Saturation of Upgraded Stopbank during Design Flood

The blue lines represent the saturation of the stopbank at 2 hour time increments during the "design" flood event. It can be seen that saturation of the fill is contained within the new fill ie. the existing fill does not become saturated.

Failure of the stopbank would require a combination of reasonable upper bound permeability (allowing saturation of the fill) and reasonable lower bound soil strength to initiate a slope stability failure. Placement of fill on the river side of the stopbank has the potential to significantly reduce the likelihood of slope instability by preventing saturation of the existing stopbank fill, even if the existing fill has comparatively high permeability. The factors of safety on the landside slopes of the refurbished sections of stopbank would be expected to be similar to those presented in section 2.2.1.4 assuming 'typical permeability' conditions. Each of these cases was considered to meet stability requirements for stopbanks.

In sections that do not require an increase in height, alternative methodologies could be adopted to restrict the flow of water through the stopbank and therefore improve its stability. Techniques such as upstream (river side) geo-membrane liners or in-situ mixing of grout with the existing stopbank material could be effective and should be investigated during more detailed design phases.

Overtopping of Stopbank

Raising of the stopbank at selected locations will directly address risks associated with overtopping the stopbank. The level of protection provided will depend on whether a freeboard allowance is included in the assessment of required stopbank crest levels.

Piping

The potential for piping through the stopbank is directly related to the potential for saturation of the stopbank fill. If water cannot reach the land side face of the stopbank during the design event, piping cannot occur. Sections of stopbank upgraded by the placement of low permeability fill on the river side will therefore have a significantly reduced risk of piping failure within the stopbank. During construction, stripping of topsoil on the existing stopbanks will allow inspection of the formed face and identification of any gravel lenses or other high permeability zones that may act as initiators of piping failures. These can be addressed on a case by case basis to further increase the resistance of the stopbank to piping.

Piping along penetrations through the stopbank would be addressed in the upgrade works by specific works at these locations to locally excavate the stopbank and place filter and drainage systems.

Piping and foundation ‘blowup’ problems are more difficult to address. Some reduction in risk will occur as a result of being able to make sure that the new fill is tied into the silt on the river berm. This will prevent very short seepage paths directly beneath the stopbank which have a higher chance of initiating piping and foundation ‘blowup’ issues. In areas that have experienced historical under-seepage issues (refer to “Areas of Stopbank Vulnerability” drawing) more extensive works would be undertaken involving either the restriction of flow from the river side of the stopbank by installation of a cutoff drain, or drainage and stability berms on the landward side of the stopbank.

During inspections of the stopbank system it has been noted that a number of gravel extraction pits have been developed in the vicinity of the stopbanks. As these pits expose the permeable gravels in the foundation, they introduce the potential for short seepage paths beneath the stopbank. Part of the proposed package of upgrade works is to better control the development of gravel extraction pits on the berm. It is noted that care will need to be taken when borrowing silt from the river berms to minimise the introduction of reduced seepage paths beneath the stopbank. This would be achieved by the positioning of the borrow areas and by leaving some of the silt in place in the borrow areas to act as a low permeability cap to the underlying more permeable gravels.

There will always be a level of residual risk associated with seepage through the stopbank foundation. Attempts to prevent the flow of water and therefore potential for piping would be very expensive over the extent of the stopbanks and may be ineffective given the thick layers of permeable gravel likely to underlie the stopbanks. The residual risks after replacement or refurbishment of the stopbanks is likely to be higher for this failure mode than the other failure modes identified in this study.

River Management

The proposed improvements to the river management regime provide a number of benefits to the security of the stopbank.

Additional river erosion protection reduces the potential for erosion of the river berm during a large flood event. If the berm is sufficiently eroded then direct river attack on the stopbank is possible. It should be noted that the erosion protection is aimed at protecting the river berm rather than the stopbank itself. Once flood waters begin to flow above the level of the river berm and against the stopbank the erosion protection is provided by the vegetation cover of the stopbank. Erosion patterns under these flood conditions may be different from those experienced when the flood is contained in the river channel. The refurbishment aims to minimise these risks by ensuring the maintenance of good vegetation cover on the stopbanks. The placement of additional fill on the river side of the stopbank will also provide a greater thickness of material that would need to be eroded before failure occurred (than the existing stopbank system provides).

Interference with Existing Stopbank

As part of the refurbishment, it is proposed to increase annual monitoring of the stopbank system to better detect areas of interference with the stopbank. More formal maintenance requirements would be instigated to address any deficiencies identified during the annual inspections. Education of landowners along the stopbank would also be undertaken to attempt to reduce the amount of negative interference of the stopbank system. An increased annual maintenance budget has been allowed for the stopbanks to allow for this work.

An allowance has also been made to specifically address a number of gravel extraction areas noted along the length of the stopbanks which introduce the potential for high localised under-seepage of the stopbank. The most likely remedial action would be the placement of low permeability fill over areas of exposed gravel to better restrict the flow of water into the stopbank foundation.

Appendix B Flood Risk Mapping

Flood model outputs, and building locations, land types, etc., at risk from flooding were imported into a GIS and translated on to a common resolution to suit risk mapping. A uniform 10m by 10m mesh of cells was used in this study for the purposes of evaluating damage costs.

Having created the requisite layers of information within a GIS, risk mapping proceeded as illustrated in Figure B1- . The basic process involved the calculation of damage costs in each mesh-cell using damage functions. These damage functions are mathematical equations which relate the damage caused in dollars to water depth and velocity predicted in each mesh-cell. Damage functions are derived from statistical information and the costs incurred from past flood in New Zealand.

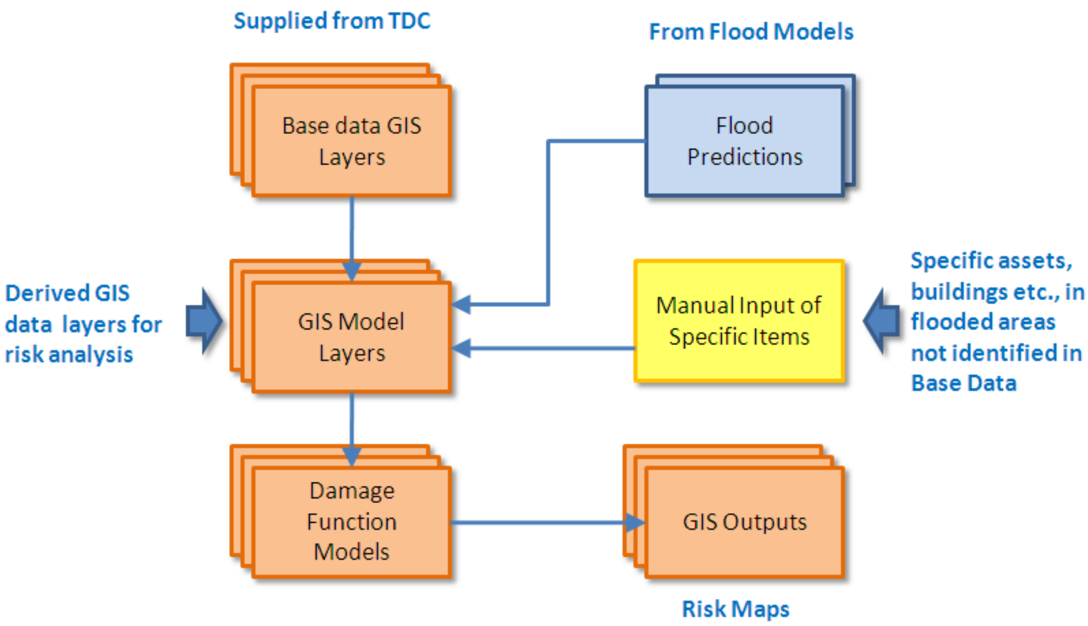


Figure B1 Basic Elements of GIS Risk Mapping Process

B1. Estimating Flood Damage Costs

Damage functions are derived from empirical information and published statistics [Agricultural Engineering Institute 1992, MAF 2003 & 2009, NZ Transport Agency 2009, Pfurtsheller et al 2008 and Statistics NZ 2008]. This information is used to derive simple mathematical expressions that predict the costs of damages caused by flood water. In most cases the amount of damage increases with the depth and velocity of flood waters. The extent of the damage caused may also depend on the time taken for the flood waters to recede and may be exacerbated by entrained debris and the deposition of sediments.

The costs accounted for are considered to be representative of the primary sources of loss from flooding. It is assumed that, because conservative cost estimates are used, the numerous other secondary sources of costs are implicitly represented. Both primary and secondary costs are assumed to lie at the location of the primary source. Thus, the primary social costs are linked to the properties that are inundated, rather than attempt to distribute them in any meaningful way to the community as a whole.

The number and complexity of damage functions used in this project is chosen to be consistent with the quantity and quality of the information available and fit for catchment-level planning purpose. More complex damage functions could be used if supported by more detailed data.
In general, damage functions are of the form;

Cost ($) = Unit Cost ($) x %Percentage Damage Caused as a function of {Flood Depth (m), Water Velocity (m/s), Flood Duration (days)}

The unit cost term represents the intrinsic value of the asset at risk. The percentage damaged caused is the empirical function derived from past studies and historical evidence and is, in general, a function of the flood depth, velocity and/or flood duration.

The cost functions are used to calculate damage costs on a spatial basis in each 10m by 10m GIS mesh-cell. The full cost of the damage caused is then calculated by summing all the damage costs across the entire flooded area. The total cost is then multiplied by the annual exceedance probability (AEP) of the flood event. This gives the annual probability weighted cost of the damage, or cost-risks for short.

Some example results are presented in Appendix C.

B2. Estimating Flood Alleviation Benefits

Modelling the status quo involves the modelling of the river for a range of flood conditions assuming the river geomorphology does not change. This gives the intrinsic flood risk against which the alternative flood management options can be compared. The effectiveness of different risk reduction measures can be explored, provided the effects of the measures can be represented in the flood modelling and GIS data. For example, modelling the effect of a new stopbank requires flood predictions to be revised with the physical effect of the stopbanks represented.

Flood damage costs were mapped for each AEP event; for the current situation (status quo) and for each scheme option. An average annual cost was then derived by integrating the flood damage costs across the catchment and multiplying by the appropriate AEP of the event. The benefit of a scheme could then be quantified from the reduction in damage costs achieved as compared to those for the existing situation (status quo). The remaining flood damage costs predicted with a particular scheme in place represents the residual cost-risk for that option.

The status quo cost-risk; the reduction in cost-risk achieved by a scheme and the residual cost-risk for the scheme, together with the costs of the scheme gives the information required to perform a benefit cost analysis as described in the main body of this report.

B3. Calculation and Mapping Flood Risk

Figure B2 illustrates how each GIS mesh grid is interrogated to establish what is at risk at that location. The damage caused is then determined based on the flood water depth (and for some risks the water velocity also) using the appropriate damage function

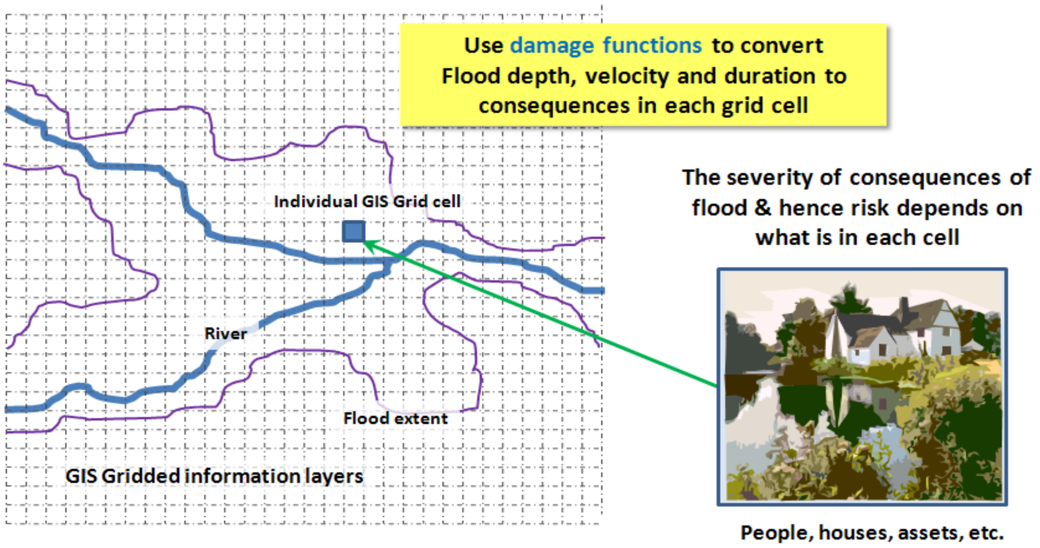


Figure B2(a): Schematic of the GIS-based Assessment of Risk

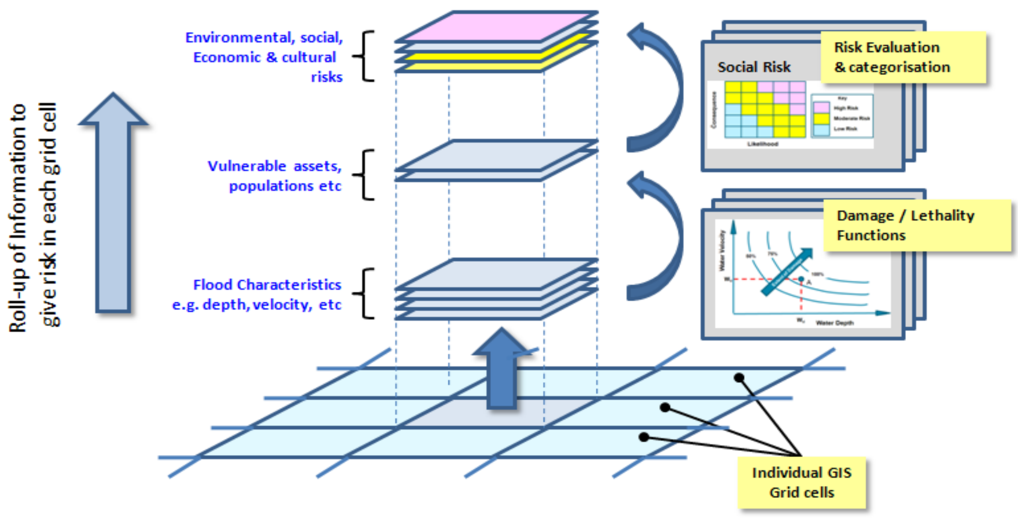


Figure B2(b): Schematic showing how GIS Layers Combine to Predict Risk

More details of the damage functions used to estimate the flood damage costs are given in Appendix C.

Appendix C GIS Risk Mapping Tool

C1. Overall Approach and Limits

Damage cost lookup tables used in risk calculations are coded in the GIS using scripts, which are Visual Basic for Applications code modules, that automate the calculations in each mesh grid overlain across the catchment.

Damage costs are evaluated separately for the following:

a) Human Life Losses

b) Social Impacts

c) Environmental Impacts

d) Cultural Impacts

e) Economic

f) Farm Losses

g) Key Assets

These various sources of damage cost are then combined to give an overall measure of the costs associated with the damage caused by a particular severity flood event. No distinction is made in the cost calculations in terms of which organisation or individual realises the cost, nor whether losses would be recoverable from insurance or other off-sets. Costs represent the overall financial burden to the Motueka region from flooding of the Motueka River.
Having determined the costs of damage caused on a spatial basis, these are then multiplied by the Annual Exceedance Probability (AEP) of the event concerned. This gives a spatially varying value of the annual average cost, or cost-risk for short. Maps of cost-risk are then produced in this way for the identified AEP events using the GIS Risk Mapping system. The results from consideration of the different AEP events are then combined to give an overall picture of the spatially varying cost-risks across the catchment.

C2. Human Life Losses

Life risk is based on the UK hazard function [UK DEFRA, 2009], determined from the population at risk and by the depth and velocity of flood waters.

Probability of fatality = Lethality Factor x (Flood Depth (m) x (Flood velocity (m/s) + 0.5) + Debris Factor

The lethality factor in the above relationship has been calibrated to past records of numbers of deaths associated with drowning due to flooding in New Zealand.

Table C1. Human Life Lethality Values

Setting	Lethality Factor	Population at Risk
Inundated dwelling	1×10^{-4}	3 per building
Inundated commercial / retail premise	1×10^{-4}	10 per building
River crossing	2×10^{-4}	2 per crossing
General urban areas	1×10^{-5}	5 per hectare
General rural areas	1×10^{-5}	1 per hectare

The debris factor in the above relationship is included to factor up the probability of a fatality when the debris load for a particular catchment is high. It is known that deaths and injuries are often caused by people being hit by debris entrained in fast moving flood water. This factor may be set to Very High (3), High (2), Moderate (1), Low (0.5) or Very Low (0) by the user according to the nature of each of the river catchment. An appropriate level for the debris factor is chosen on the basis of historical information and local knowledge of the catchment.

The dollar value associated with saving a human life has been given a typical value of NZ$4 million, in other contexts, in order to inform decisions about where risk reduction measures will have greatest benefit in regards to protecting human life. The dollar loss associated with human life safety is computed from the Probability of Fatality, calculated from the above hazard function, multiplied by the value of saving a life.

C3. Social Losses

Social risk is assumed to comprise of three components; (a) the effects of stress and minor injury on the ability of people to function normally, (b) the direct loss of income for those unable to work, and (c) disruption costs associated with flooded roads.

The stress component is evaluated using the World Health Organisation's method for quantifying the debilitating effects of disease known as the Disability Adjusted Life Year (DALY) method [Pruss-Ustun et al, 2003]. The DALY approach relates the debilitating period to the average human life expectancy and then relates this to the value of saving a life as for human life safety.

DALY = Lost Days / (Life Expectancy in days)

Social Loss (Stress) = Exposed Population x DALY x Vulnerability x Value of saving a life ($)

The vulnerability factor modifies the social loss according to the depth of flood, recognising that these impacts are likely to increase with the depth of flood waters, see the table below.

Table C2. Social Losses Vulnerability Factor Versus Flood Depth

Flood Depth (m)	Vulnerability Factor (%)
0	0
0.5	10
1.0	60
2.0	100
5.0	100

It is assumed that salaried workers would not lose pay for the duration that they are unable to work. However, for non-salaried workers, the loss of earnings is evaluated for each dwelling that is in the flooded area. The social cost per dwelling is then assumed to be the product of the following factors:

- the average number of workers per dwelling
- the duration of flood
- proportion of non-salaried workers in the population (rural or urban)
- the proportion of time at work on daily basis.

Social disruption costs associated with road closures and traffic congestion are based on traffic volumes as promulgated by NZ Transport Agency (Economic Evaluation Manual, Volume 1) in estimating the benefit from road improvement schemes. Disruption as a result of flooding on State Highways, or critical routes serving isolated communities, is calculated using specific annual average traffic volumes specified by the user. These traffic volumes can be obtained from NZ Transport Agency published statistics [NZ Transport Agency, 2010] and from the latest Tasman District Council traffic count data. The expected delay duration, and the NZ Transport Agency dollar cost of a delay per hour was obtained from NZ Transport Agency's Economic Evaluation Manual (Volume 1). Non-critical road routes are classified as "Urban", "Semi-urban" or "Rural", each given a default daily traffic volume, delay time and cost are estimated using typical values published by NZ Transport Agency for these types of road.

The Motueka River Bridge has recently been restrengthened and has been subject to a number of investigations to realign the approaches in the past however a major upgrade to the bridge or alignment has not been undertaken. It has been assumed for this model that State Highway 60 (SH60) will be closed for at least half a day during a major flooding event based on similar data from other NZ Transport Agency regions.

C4. Environmental Losses

Flood damage to the general environment is assumed to be related to the loss in value of different broad types of land depending on the frequency they are flooded, as outlined below. Typical land values are listed in Table C3.

Table C3: General Land Values

Group	Land Type	Best Estimate ($/hectare)
1a	Urban residential	250,000
1a	Urban commercial	400,000
1b	Rural residential (Townships)	120,000
1b	Life style	200,000
2	High value agricultural (horticulture)	90,000
3	Low value agricultural (grassing)	25,000
3	Forestry	14,000
4	Conservation land	300,000
4	Bare land	12,000
5	High-value cultural area	300,000
6	Low-value cultural area	150,000

The loss in land value is assumed to be related to the severity of the flood event in terms of its AEP and proportional to the overall value of the land type if it were not flood prone. These proportional losses are tabulated below.

Table C4: Table of Land Value Percentage Loss Versus Flood AEP

AEP of Flood	Group 1a & 1b	Group 2, 5 & 6	Group 3	Group 4
0.1	50%	30%	20%	10%
0.05	40%	25%	18%	8%
0.02	20%	15%	10%	5%
0.01	10%	7.5%	5%	3%
0.005	5%	2.5%	2%	1%

Hence the loss in land value per hectare is evaluated from the product of the appropriate land value, from Table C3, multiplied by the Percentage loss in value from Table C4 for the appropriate land type grouping and the AEP of the flood event being considered.

C5. Cultural Losses

Cultural impacts are represented by both general and specific losses. General losses represent the long-term effects, given the perception of poor flood management in the Tasman Region, on the area being an attractive place for tourists to visit. Specific losses are those associated with flood damage to culturally valued sites and property. Losses associated with cultural asset are based on a "willingness to pay" approach – a combination of how much people would be will to pay to visit the tourist or cultural sites and the number of people likely to visit annually or pay to replace the tourist or cultural asset if it were damaged. Thus in the case of a Marae, this could be calculated from the number of local tangata whenua multiplied by a typical acceptable donation cost for building a Marae. Alternatively a Marae could be represented as a key asset see Section C8.

General losses are based on a break-down of tourist spending according to the degree of urbanisation, namely whether the land is an urban centre, a township or it is rural. Average tourist spending power is based on figures given in the NZ Official Year Book, 2008. Default values are given for Urban, Semi-urban and rural land types. See Table C5 below.

Table C5: General Cultural Costs Associated with Land Type

Land Type	Typical Daily Spend per Hectare
Urban / major centre (Group 1(a))	$138
Semi-urban / township (Group 1(b))	$17
Rural (Groups 2 – 4)	$0.60

Specific losses are associated with direct lost income resulting from temporary closure, and from clean-up and repair costs. Lost income is derived from the product of the following factors:

- daily visitor numbers
- the duration of the flood
- average spend per visitor per day
- a cost multiplier.

The cost multiplier is included to recognise that there will be additional indirect losses, and also a drop in visitor numbers for a period after the flood event is over. Visitor numbers and average spend per visitor must be provided for each specific tourist and cultural site by the user.

Repair costs are assumed to be dependent on a worst case clean up costs, multiplied by a flood depth modifying factor which varies between zero and one, see Table C6. Clean up costs are determined from a daily disruption cost and the duration of the flood. The user needs to specify the maximum cleanup cost and the daily disruption cost for each specific tourist or cultural site impacted by flooding.

Table C6: Cultural Asset Damage Loss Factor versus Flood Depth

Flood Depth (m)	Depth Factor (-)
0.0	0.0
0.15	0.05
0.5	0.25
1.0	0.65
2.0	0.95
5.0	1.0

C6. Economic Losses

Economic losses are represented by losses associated with inundation of:

a) A Model private house.

b) Retail / commercial premise including likely stock losses at ground floor level.

In both cases the dollar value of losses is assumed to be dependent on the depth of inundation. The model house represents a typical property characterised as a 3 bedroom timber framed house with timber weather board cladding and floor area of 93m^2. It is assumed to have Gib-board internal wall linings, and no insulation and tongue and groove timber floor.

General Property Loss ($) = Loss function ($) versus flood depth (m)

Figure C1 below shows the variation in the flood damage costs used in estimating the economic losses from flooded houses depending on flood water depth.

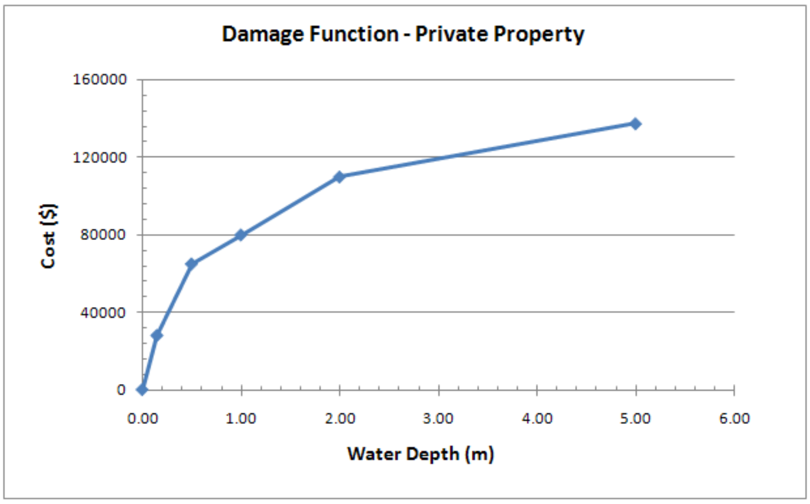


Figure C1: General Property Loss Function

Losses from retail or commercial premises is assumed to be related to the overall value of the retail or commercial premise and the value of stock held on the premise specified by the user.

User specified inputs required are:

- average area of retail or commercial premise
- typical ground floor stock values
- retail and light industrial turn-over.

Figure C2 below, shows the variation in the flood damage costs used in estimating the economic losses from flooded retail and commercial premises depending on flood water depth.

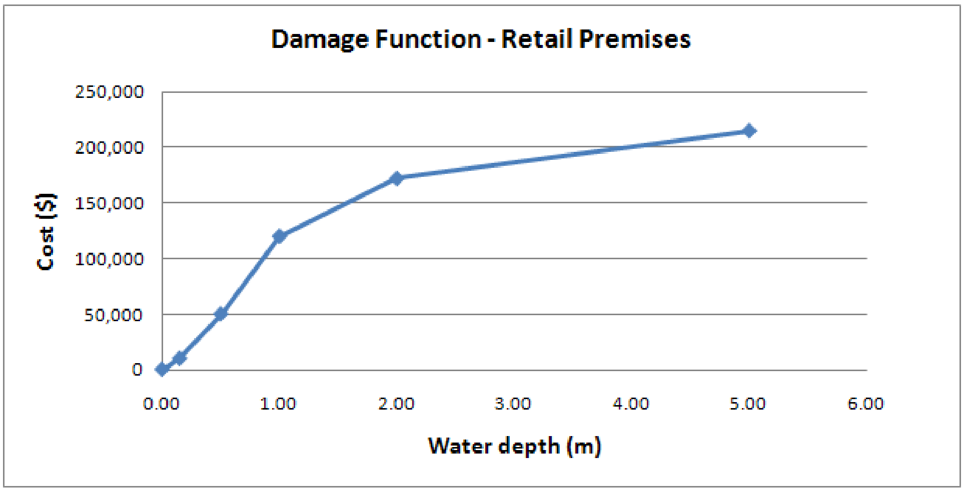


Figure C2: Retail / Commercial Loss Function with 200m^2 Floor Area

Figure C3 shows the variation in the flood damage costs used in estimating stock losses for retail and commercial premises depending on flood water depth.

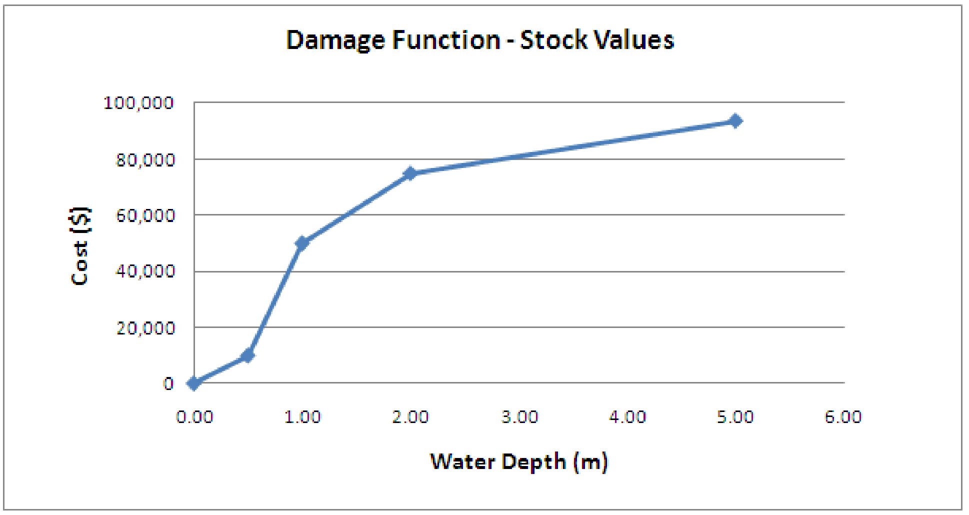


Figure C3. Retail / Commercial Stock Value Loss Function

C7. Farm Losses

Farm losses are estimated for the following categories:

a) dairy farming
b) sheep and beef farming
c) horticulture
d) forestry.

It is assumed that the most significant losses are associated with lost annual income over the duration of the flood with secondary losses represented by a cost multiplier.

Table C7: Rural Business Base Data

Rural Business Costs	Size (hectares)	Net Income ($)	Recovery Cost Multiplier	Income ($/hectare/day)
Dairy Farming	80	225,000	3	7.7
Sheep and Beef	314	203,364	2	1.8
Horticultural	100	234,444	2	6.4
Forestry	100	19,512	1	0.5

Notes:
The income figures are derived from information published by MAF.

Farm Losses ($) = Area (hectares) x Income ($/Hectare/Day) x Flood Duration (Days) x Cost multiplier (-)

Losses associated with stock farming are likely to be quite varied: as stock is generally free to roam, and farmers move stock around. The above losses are therefore based on a proportional loss of annual income rather than attempt to specify the numbers of animals involved. It is assumed that stock losses are accounted for in the cost multiplier.

C8. Key Asset

Key assets are any specific structure or high value asset, which provides a service. Damage costs arise from the temporary loss of service and from repair and clean up costs following the flood event.

The loss of service is determined from the following factors:

- number of people affected by the loss of service (-)
- service disruption costs per person per day ($/day)
- the disruption period, assumed to be the duration of the flood (days).

Key assets with a relatively small 'footprint' such as a pump station or electrical sub-station can be defined as location specific, larger assets such as a marina can be defined to cover a specified area.

Repair and clean up costs for location specific tourist and cultural assets are determined as a proportion of the user specified maximum damage caused in the worst case flood event, i.e.:

Repair and clean up costs ($) = Maximum Damage Cost ($) x Proportion Depth Factor (-)

The proportional depth factor is shown in Figure C4 below.

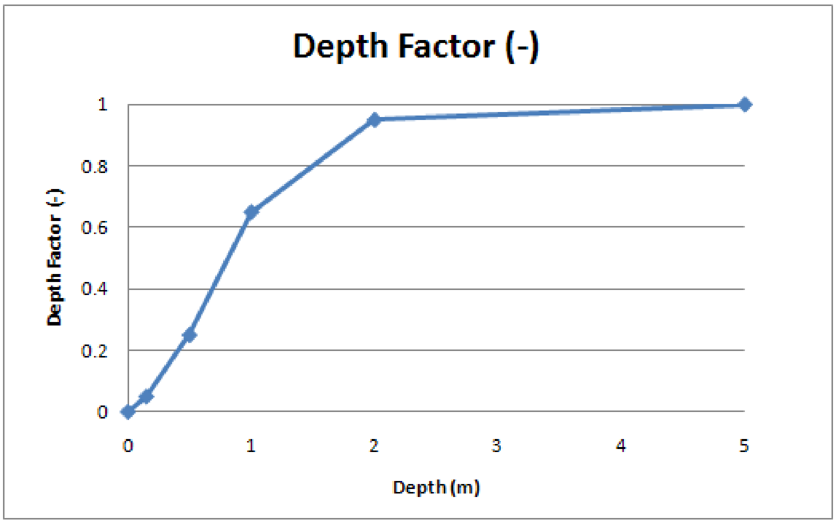


Figure C4. Key Asset Repair and Clean-up Depth Function

In the case of area key cultural assets, the following inputs are required for each key asset of this type:

a) The areal extent of the key asset (hectares)
b) The number of cultural units within the defined areal extent of the key asset
c) The unit value of what the asset contains ($/unit)
d) A vulnerability factor determined by the severity of the flood event, characterised by the AEP of the event, see Figure C5 below.

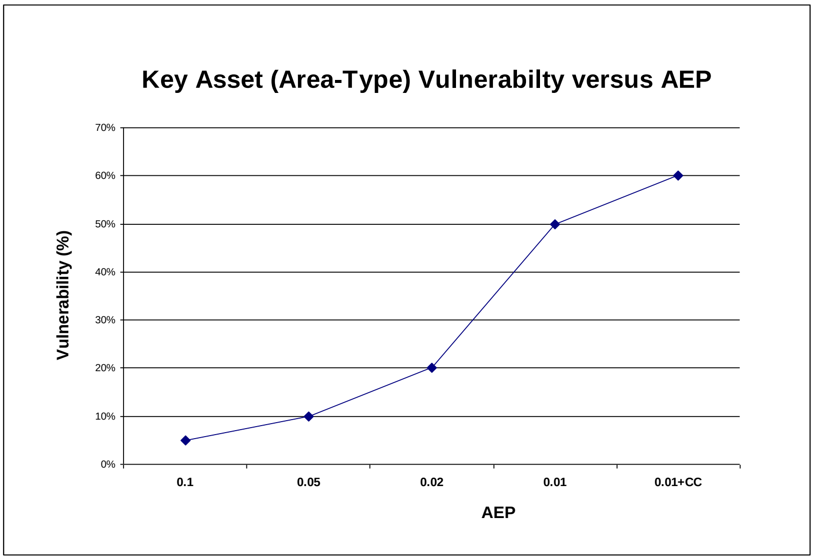


Figure C5 Area Asset Vulnerability Factor

C9. References

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Appendix D Drawings and Plans

- Areas of stopbank vulnerability
- Stopbank crest raise summary
- Drawing SB01 Stopbank options cross sections
- Drawing SB02 Indicative refurbishment cross sections
- Aerial photographs, maps 10 – 1 (excluding Map 5), marked up to indicate conceptual gravel removal areas
- Motueka Flood Risk Mapping Figure 2 (breach)
- Motueka Flood Risk Mapping Figure 3 (no breach)

Appendix E Geotechnical Information

- Test Locations
- Potential Borrow Areas
- Test Pit Logs
- Laboratory Reports

Appendix F Benefit / Cost Calculations

- Flood Modelling Cases
- Flood Event Costs
- Scheme Costs
- Scheme Assessment
- Summary