Shotover Country

Plan Change

River and Flooding Risk Assessment

Lower Shotover 21 Nov 1994

February 2010

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Shotover Country
Plan Change
River Erosion & Flood Risk Assessment

Contents

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1. Introduction

Shotover Country propose to develop a site south of State Highway 6 and on the eastern or true left bank of the Shotover River, in the location shown on Attachment A. A general site photo from 2008 is shown as Attachment B.

The Queenstown Lakes District Council has prepared Hazard Register Maps that cover the proposed area (Sheets 30 & 31 as Attachment C1 & C2).

A number of reports have been produced on the Shotover River and flooding in the Upper Clutha and Kawarau/Lake Wakatipu areas. The most significant of these are referenced and include:

(a) ORC "Shotover Delta Sedimentation" October 2002 report.

(b) "Kawarau and Shotover Rivers Sedimentation Investigation" for ORC dated January 2006 authored by Barnett & MacMurray et al.

(c) "Shotover River Sediment Management: Microscale Modelling" Final. Report to Otago Regional Council. TR Davies, Geological Sciences, University of Canterbury March 2007.

These reports were summarized in an Otago Regional Council (ORC) report of July 2007 (File EN0902, Report No. 2007/267 to Engineering & Hazards Committee "Management of the Shotover Delta."

This report looks at the risk to the proposed site from river erosion and flooding and recommends possible mitigation works where these may be necessary. It is not considered that the very low terrace with wetlands that was flooded in 1999 should be developed, and this has been excluded from further consideration in the balance of this report.

2. Shotover River

2.1 Catchment

The Shotover River has a 1,100 km^2 catchment with high rainfalls experienced as it extends back to the Main Divide at an elevation of over 2400m. It is highly erodible and recent NIWA reports assess the average annual sediment supply as about 1.6 million cubic metres per annum. Annual rainfall at the divide averages about 8000mm per year while at the proposed plan change site it is about 700mm per annum. During major storm events (generally from north-westerly air flows) there is a similar gradient from the northwest to the south-east.

2.2 Flow Records

A water level recorder has been on the Shotover River at Bowens Peak (Site Number 75276), downstream of Arthurs Point, since 1967 with a catchment area of 1,088 km^2. Because of the sediment movement the flow rating for the station is variable. There are however a number of other sites that allow a reasonable interpretation of the Shotover River flows. Mean flow is about 41 m^3/s.

2.3 Floods

Large floods have occurred in the Shotover in 1863, 1878, 1919, 1949, 1957, 1969, 1978, 1984, 1987, 1994, 1995 and 1999.

The mean annual flood is taken as 420 m^3/s from the 1967 to 2002 record (Range 150 to 918 m^3/s). Based on this and using the Regional Flood Estimation technique of DSIR Hydrology Centre Publication No.20 the ratio of the 100 year event (Q_{100}) to the mean is 2.8 that yields a flow of 1,176 m^3/s. Likewise the ratio of Q_{50} to Q_m is 2.49 yielding 1046 m^3/s. The standard error on these estimates is 26-28%. In this analysis the Bowen Peaks recorder registered a flow of 369 m^3/s for the 1999 flood. As stated in the ORC report in this event the Chard Rd recorder on the Kawarau R downstream of the confluence with the Shotover had an assessed flood flow of about 1150 m^3/s of which the Shotover contributed not less than 1000m^3/s. It states that this was the highest flow since at least 1878. Design flood levels for the purposes of the plan change site have used a flow of 1200 m^3/s. A conservative approach is considered prudent in this changing river environment.

2.4 Sedimentation

The ORC 2002 report analyses the river cross sections that have been surveyed since 1980, when they were established by the MWD. The locations of the cross–sections below the State Highway Bridge are shown on the attached plan D.

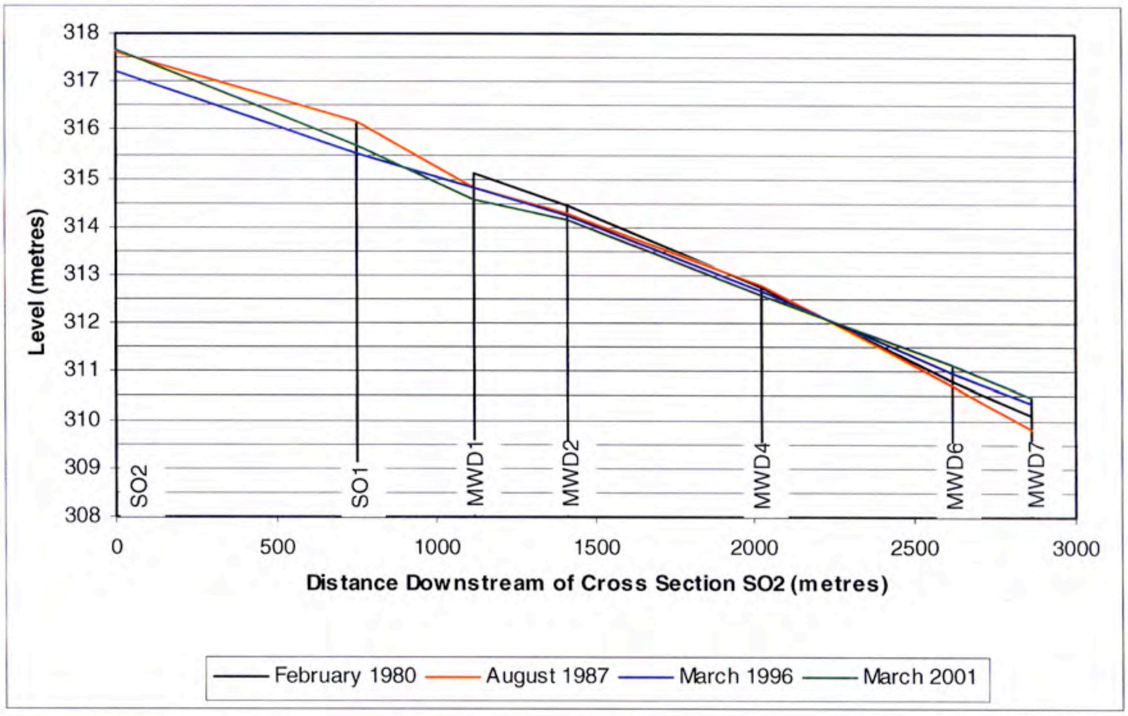

Figure 7.1 **Shotover River mean bed level profiles**

Figure 2.1: Figure from ORC 2002 report

The survey results show that the river has aggraded above the State Highway Bridge, degraded in the mid delta and the lower delta has aggraded markedly between 1980 and 2001. Figure 7.1 from the ORC report is reproduced here as Figure 2.1. Comment is made in the report that the third and downstream oxidation pond on the right bank was constructed in 1987 and would have an influence in constraining the fairway and maintaining sediment movement through this reach before the channel width widens out, although this effect is offset to some extent by the islands in the lower reaches of the delta.

Mobilisation of gravel on the delta would be expected by the time flows reach about 400 m^3/s.

From ORC Table 6.1: For cross section 7 the actual increase in MBL has been 0.33m in 21 years. For cross section 6 the increase in MBL has been 0.29m in 21 years. At cross section 4 there has been a reduction of 0.17m in 21 years and greater reductions have occurred for Cross sections 2 and 1. This is summarised in Table 2.1.

This accumulated sediment is associated with a number of large events over the 1994 to 1999 period and it is expected that the cross sections and mean bed levels as measured in 2001 will gradually lower. This is confirmed by reductions in MBL of about 0.05-0.06m as measured for cross sections 6 and 7 in March and September 2001.

Cross section MWD #	Change in MBL over 21 Years (1980-2001) m	Average annual rate m/yr	Change over 50 years at same rate m
1	- 0.5	-0. 024	- 1.2
2	- 0.29	-0.014	- 0.7
4	- 0.17	-0.008	- 0.4
6	+ 0.29	+ 0.014	+ 0.7
7	+ 0.33	+ 0.016	+ 0.8

Table 2.1 Lower Shotover Rates of Change in Mean Bed Level

In order to check on what impact a continuing rise in MBL in the lower reaches of the delta would have on flood levels adjacent to the proposed plan change area a simplified hydraulic model of the cross sections has been used. This model assumes no further reduction in the cross sections 1 to 4, an 0.3m increase in MBL for XS 5 and increases of 0.7 and 0.8m over 50 years for cross sections 6 and 7 respectively. The model is described further below.

2.5 Area Flooded in 1999 Flood

Aerial photos taken immediately after the 1999 flood show the area subject to inundation from the Shotover River and the area that was not flooded. See Figure 2.2.

Figure 2.2: Area of siltation shows area flooded in November 1999

3. Design Flood Levels

3.1 Initial (May 2005) Run

A simplified hydraulic model has been used in a computer programme HEC-RAS. The mean bed levels were used for this with channel width based on clear fairway only. The model was checked using the 1999 peak flood levels as measured at cross sections 1, 2, 4, and opposite the downstream end of the delta on the right bank of the Kawarau River. The latter was used as the control level at cross section 7. As the flows were derived rather than measured and that assessment was not less than 1000 m^3/s this figure was initially used. The model results for a range of channel roughness were used (Mannings coefficient n roughness between 0.025 and 0.04). All gave higher model water levels than measured. A good match was achieved with a flow of 750 m^3/s and a Mannings n coefficient of 0.030. The ORC flood report notes that the maximum Shotover flow may have been about 600-650 m^3/s. With the range of flows identified the figure of 750 m^3/s has been used for the 1999 flood. A Mannings n coefficient of 0.03 has been used to see what impact higher flows or changes to the mean bed level make to the design flood levels. These levels should also be conservative. The results are tabulated in Table 3.1 and shown on Figure 3.1.

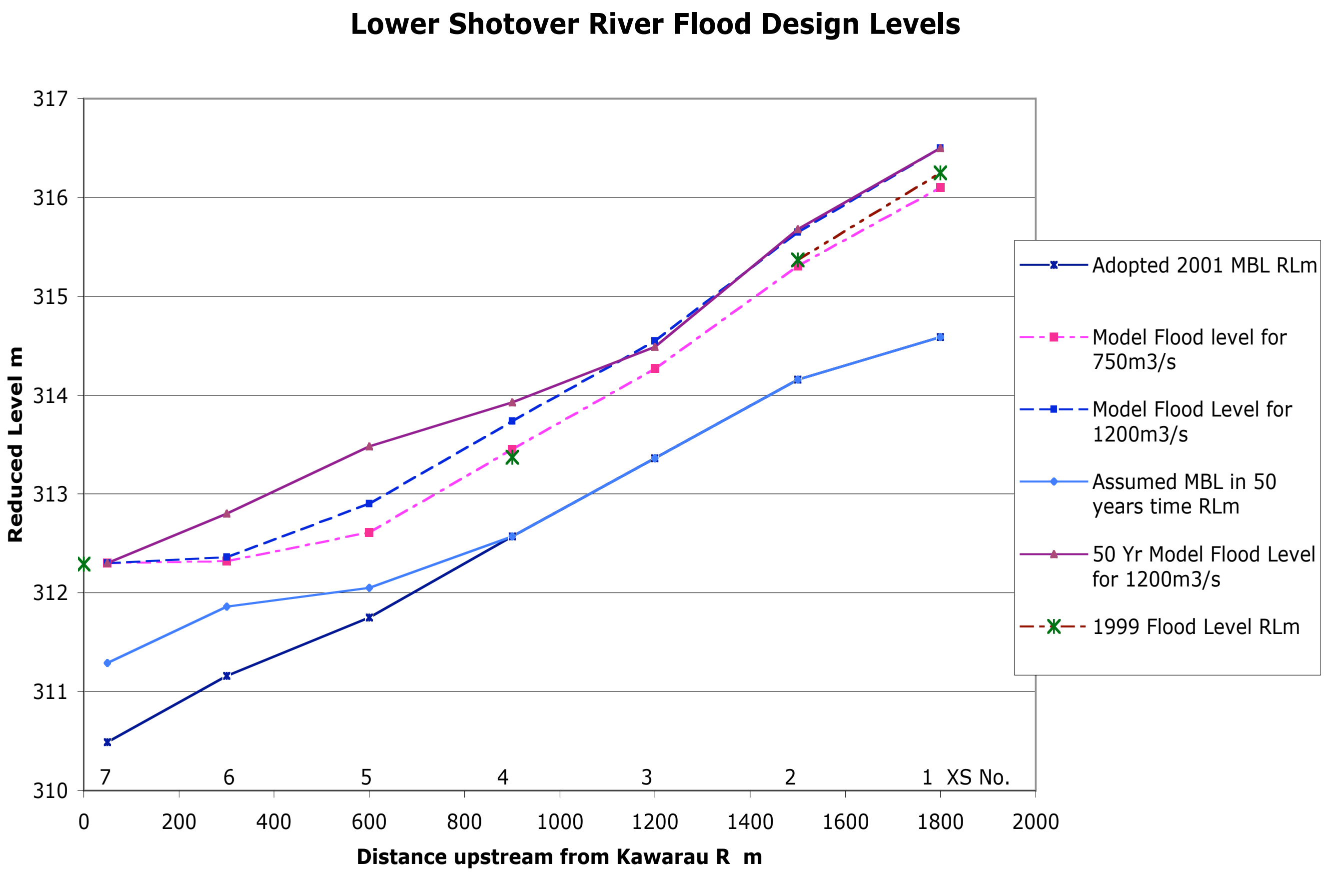

Figure 3.1

HEC RAS 3.1		Hydraulic model results			Lower Shotover River			
					Based on 2001 XS		50 Years Time XS	
MWD Cross Section No.	1999 Flood Level RLm	Running distance upstream from Kawarau m	Adopted 2001 MBL RLm	Channel width used in model	Model Flood level for 750m3/s	Model Flood Level for 1200m3/s	Assumed MBL in 50 years time RLm	50 Yr Model Flood Level for 1200m3/s
S17 Kawarau	312.29	0						
7		50	310.49	890	312.3	312.3	311.29	312.3
6		300	311.16	700	312.32	312.36	311.86	312.8
5		600	311.75	515	312.61	312.9	312.05	313.48
4	313.37	900	312.57	600	313.45	313.74	312.57	313.93
3		1200	313.36	430	314.27	314.55	313.36	314.49
2	315.37	1500	314.16	335	315.31	315.65	314.16	315.68
1	316.25	1800	314.59	240	316.1	316.5	314.59	316.5

Table 3.1: Design flood levels assuming trends continue in lower delta

A more detailed model could be prepared but as there does not appear to be recent survey of cross sections 3 and 5 it is considered that this would therefore not yield significantly different results to the simplified approach adopted.

3.2 July 2005 Model Run

Feedback from the ORC was that they considered that similar volumes of aggradation (to XS 6 & 7) may occur further upstream than XS5 and so the model has been run with a mean bed level increase of 0.75m over 50 years for XS 1 to XS 4.

At cross sections 1, 2 and 3 the channel is 340m wide while at cross section 6 the channel is 1060m wide or over three times the width. As a result the average velocities under a mean annual flood are much higher in the narrower sections and this will tend to continue to transport material through the narrower sections. The river will have been adjusting and is continuing to adjust since being narrowed by the construction of the sewage treatment ponds and parallel protection works some 30 years ago. The extent of gravel extraction is also a factor and the ORC engineering section should be endeavouring to continue the management of this gravel extraction for best river control purposes.

In order to assess what impact on flood levels a similar rise in bed levels (to those at XS 6 & 7) at cross sections 1, 2 and 3 would have the HEC-RAS simplified mean bed level model has been re-run. The mean bed level has been increased by 0.75m for all cross-sections. The same start level at the Kawarau River of RL 312.29m has been used as recorded in the 1999 flood at Kawarau XS S17.

Under this scenario the average velocities through XS 1-3 are over 2 m/s for flood flows over 1000 m^3/s and have a Froude number approaching 1 (meaning critical velocity may be exceeded). This is unlikely to occur in practice and the energy would be used in moving the sediment through this reach and picking up material from the bed under these circumstances. The design flood levels from this run are however presented in the following table and in Figure 3.2.

As would be expected the average increase in flood levels is about 0.75m as the channel width was assumed to be the same. Flood levels are approximately 1m higher than the 1999 flood.

HEC RAS 3.1		Hydraulic model results			Lower Shotover River			
All XS Modified for 50 Yr run					Based on 2001 XS		50 Years Time +0.75m	
MWD Cross Section No.	1999 Flood Level RLm	Running distance upstream from Kawarau m	Adopted 2001 MBL RLm	Channel width used in model	Model Flood level for 750m3/s	Model Flood Level for 1200m3/s	Assumed MBL in 50 years time RLm	50 Yr Model Flood Level for 1200m3/s
S17 Kawarau	312.29	0						
7		50	310.49	890	312.3	312.3	311.29	312.3
6		300	311.16	700	312.32	312.36	311.86	312.78
5		600	311.75	515	312.61	312.9	312.05	313.74
4	313.37	900	312.57	600	313.45	313.74	312.57	314.36
3		1200	313.36	430	314.27	314.55	314.11	315.05
2	315.37	1500	314.16	335	315.31	315.65	314.91	316.55
1	316.25	1800	314.59	240	316.1	316.5	315.34	317.24

Table 3.2: Flood levels if aggradation occurs in upper delta

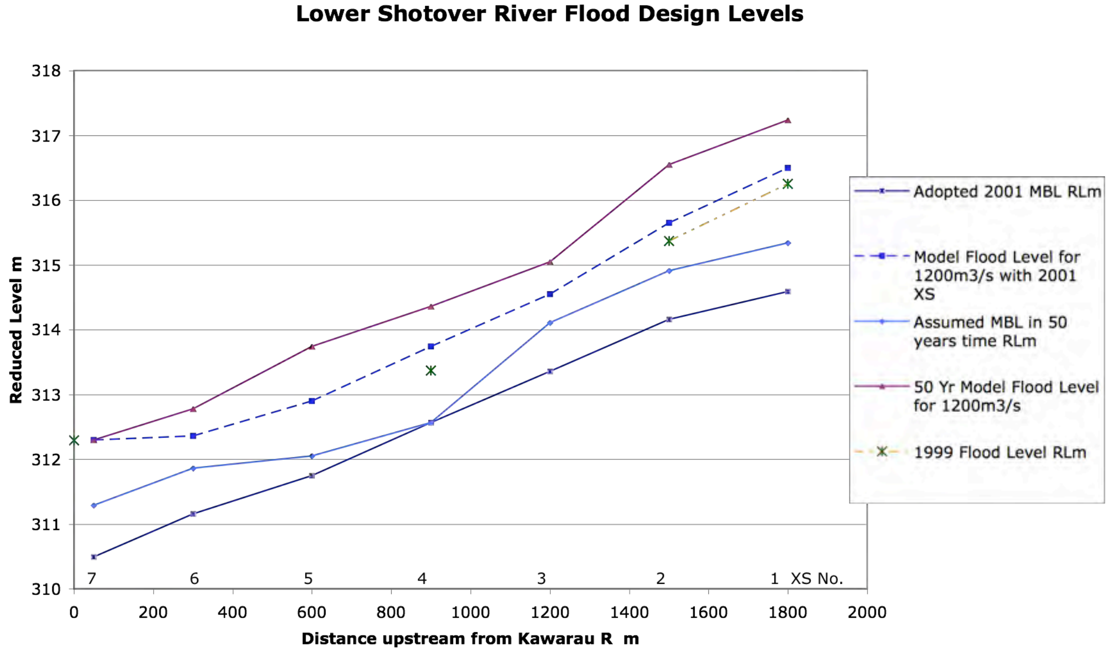

Figure 3.2: Assuming aggradation sections 1-3 by 50 years time

3.3 Reassessment of situation after 2006 survey

Survey data (December 2006) has been obtained from the ORC and the trends in mean bed level checked against the above analysis. See Figure 3.3 for a plot of the mean bed levels in 1980, 2001and 2006 with the ORC suggested levels for the 50 year projection also plotted. There continues to be degradation in the upper delta and similar levels are evident in the lower delta compared to 2001.

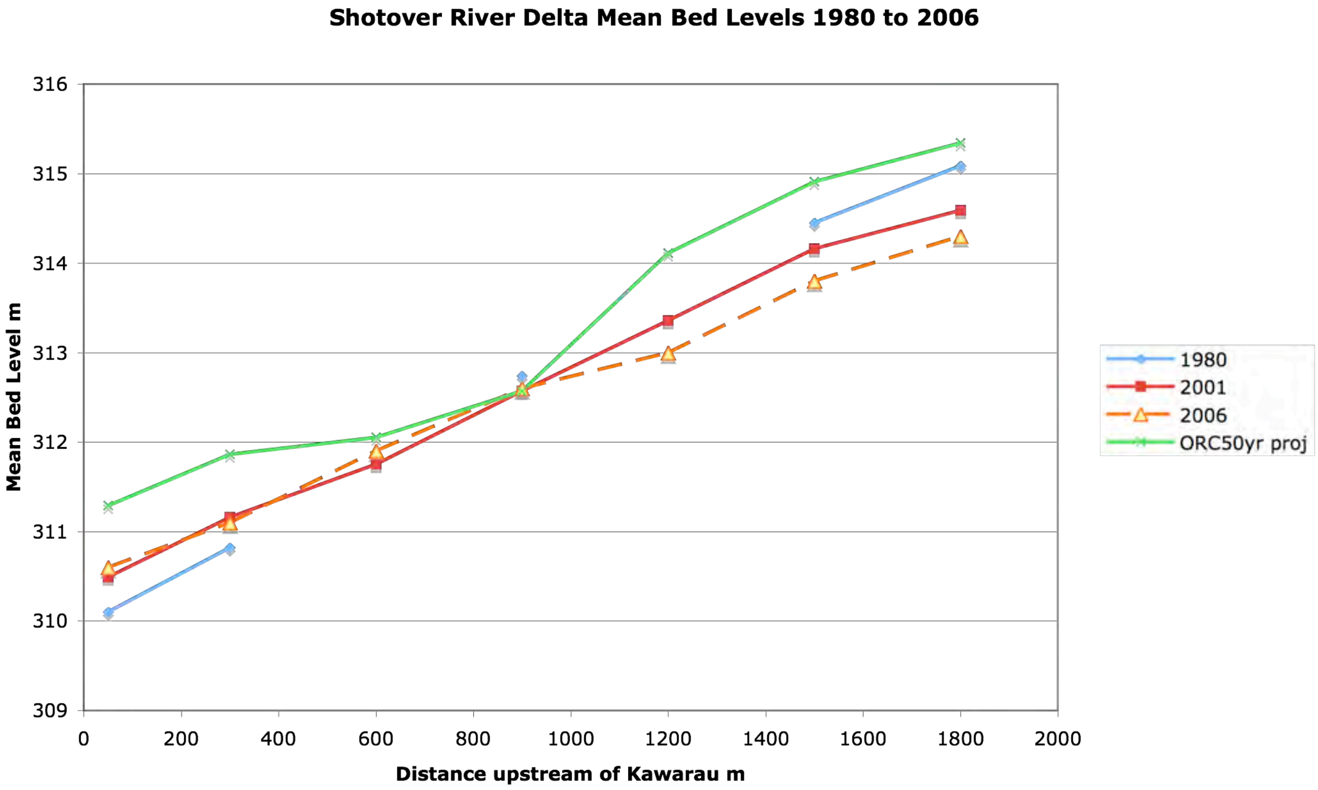

Figure 3.3: Mean Bed Levels 1980 to 2006

With the continuing trend for degradation in sections 1, 2 and 3 and the principal reasons being the narrow channel through this reach the original analysis using the 2001 mean bed levels is still considered appropriate with no provision for further degradation, and no reversal of the trend towards aggradation. No reason to change the original flood level analysis as set out in Section 3.1 above is considered necessary.

3.4 Mitigation Options

For the proposed Plan Change it is considered that while the site is unlikely to be flooded in the 1% AEP flood event it is possible within the margin of error of estimates that minor flooding could occur on the lower parts of the site given the natural variability within riverbeds.

It is therefore considered prudent to provide for mitigation works to avoid the risk of flooding to the lower parts of the site. It is considered that there are two options available for potential mitigation. The options are either:

(a) a stopbank parallel to the river from the high ground at XS1 down to the eastern corner of the wetland area (see Attached plan F1) , or

(b) Clean hard fill areas on the lower terrace that take into account estimated flood levels with appropriate freeboard.

Either the stopbank crest levels or the developed minimum floor levels should be 0.8m above the 1% AEP flood in 50 years time and using 2001 bed levels at cross sections 1-3 Figure 3.4 has been derived. The data is shown in Table 3.3.

Adopted Minimum hardfill or stopbank levels					Lower Shotover River		
					50 Years Time XS		
MWD Cross Section No.	1999 Flood Level RLm	Running distance upstream from Kawarau m	Adopted 2001 MBL RLm	Channel width used in model	Assumed MBL in 50 years time RLm	Adopted 50 Yr Model Flood Level for 1200m3/s	Adopted minimum stopbank level OR hardfill level 0.8m above flood level
S17 Kawarau	312.29	0					
7		50	310.49	890	311.29	312.3	**313.1**
6		300	311.16	700	311.86	312.8	**313.6**
5		600	311.75	515	312.05	313.48	**314.3**
4	313.37	900	312.57	600	312.57	314	**314.8**
3		1200	313.36	430	313.36	314.8	**315.6**
2	315.37	1500	314.16	335	314.16	315.68	**316.5**
1	316.25	1800	314.59	240	314.59	316.5	**317.3**

Table 3.3: Recommended minimum stopbank levels OR minimum hardfill levels at cross section locations to meet projected flood levels in 50 years time

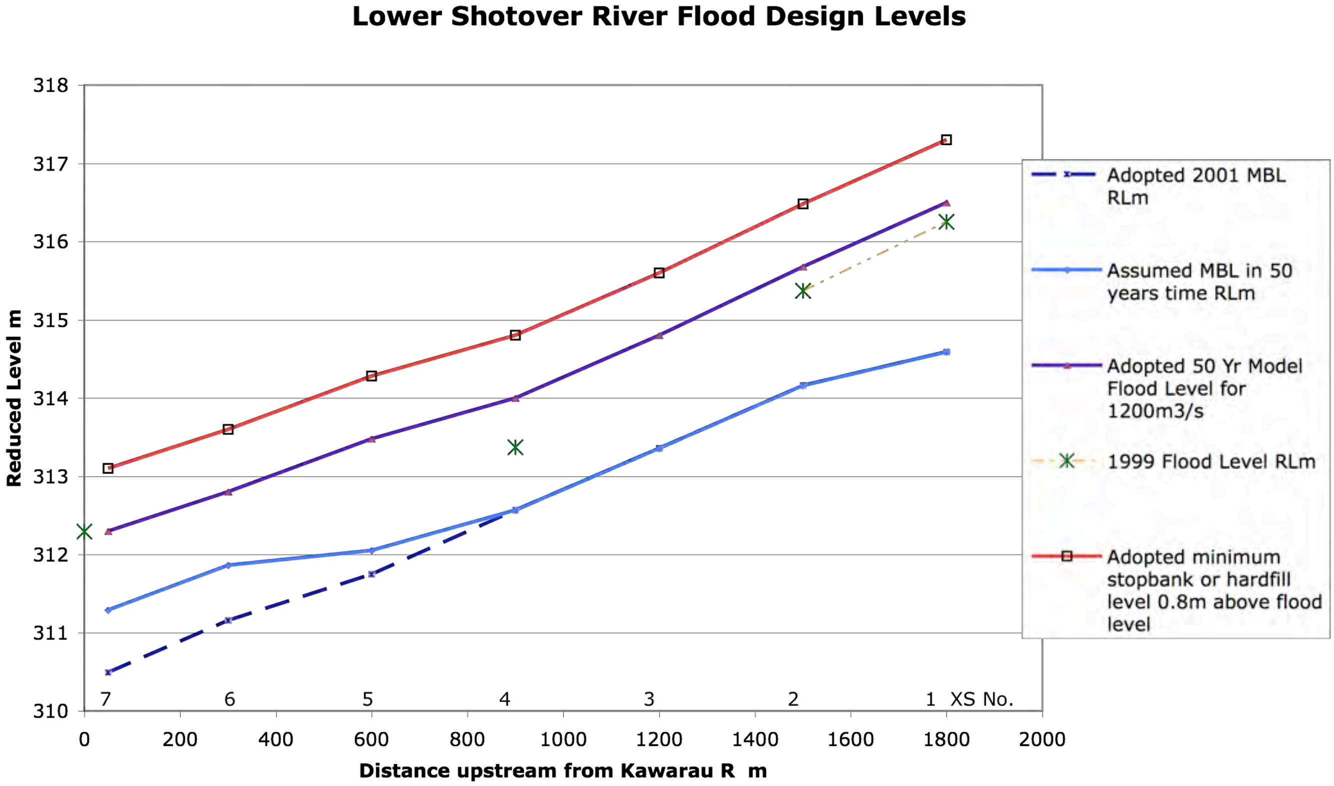

Figure 3.4: Recommended stopbank OR hardfill levels to be 0.8m above 1% AEP floods by 50 years time after smoothing of 1200m^3/s grade line

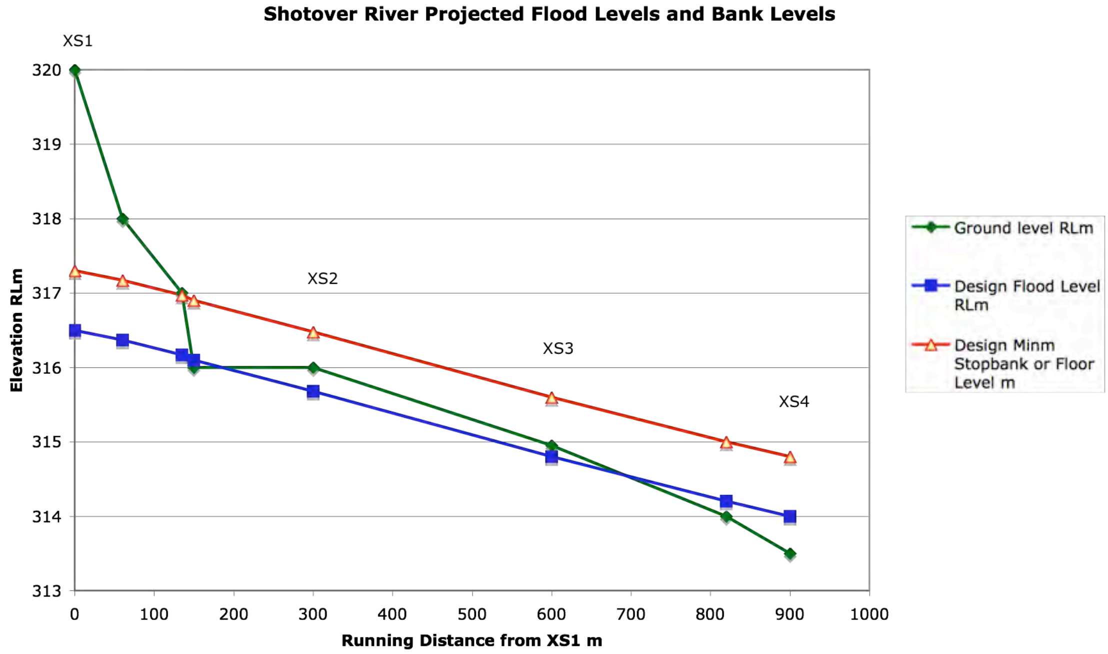

Figure 3.5: Recommended stopbank OR hardfill levels compared with existing ground level on left bank

Figure 3.5 shows the information in reverse direction from cross section one location and plots the existing ground level along the left bank with the same adopted flood levels and minimum stopbank or hardfill levels. Should the hardfill option be considered then the structures be it fill or foundations should cope with velocities of the order of 2.5 metres per second. On fill this would require light rock sizing of D_{50} of 400mm and a rip rap thickness of 0.6m on a batter slope of no steeper than 1.5:1.

Should a stopbank option be preferred then the general dimensions should be as shown in Figure 3.6. Should a more rounded or undulating landscaped bank be preferred the minimum dimensions and levels would need to be dictated by the bank design parameters. Only grasses, tussocks, small flaxes and similar sized shrubs should be permitted as vegetation on the banks. Trees should not be used on a stopbank.

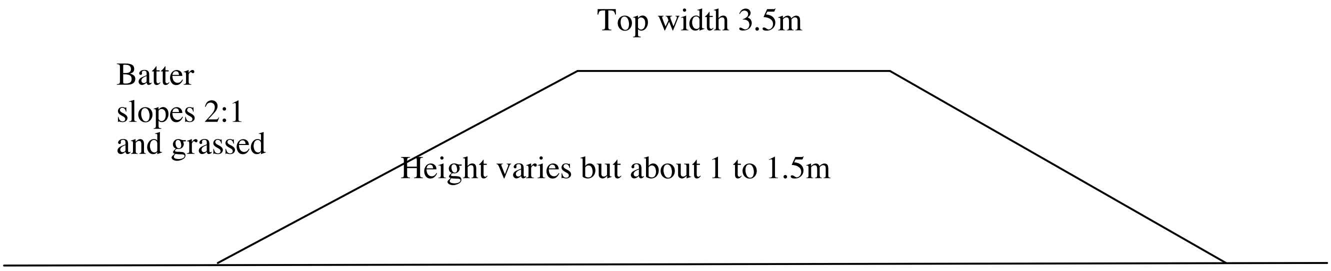

Figure 3.6: General minimum dimensions for stopbank top width and batter slopes

A possible alignment for a bank is shown on CFMA Plan attached as F1.

4. Flood Hazard Area and Bank Erosion

The flood hazard areas from the District Plan are shown as attachments. This report and the ORC report on the cross section analysis indicate that the proposed site is no longer part of the floodway for the lower Shotover River. These analyses are much more robust and site specific than the information used to originally prepare the flood hazard zone information. The extent of flooding in the large 1999 flood did not affect the proposed plan change development site (see Figure 2.2) but did affect the lower terrace wetland area.

Jeff Bryant of Geoconsulting Ltd in his geotechnical report for the Plan Change states that the site comprises several river terrace platforms successively cut down by meanders of the Shotover River over the past few thousand years. He notes that the terraces have has pronounced paleo-channel visibility on the aerial photo. Results from test pits are commented on. While the lowest terrace (T6) showed layers of silt indicating deposition during a prolonged period of submersion from two distinct flooding events, no similar layers were identified on the next higher terrace (T5).

Associated with this site is the need to maintain the longitudinal willow protection on the river side of the proposed area. As can be clearly seen on the aerial photographs the lower terraces are historic floodplains of the Shotover River. The river has adequate width to cope with the flood flows but the meander pattern could develop that could attack the edge protection. On going maintenance of this is thus essential.

The proposed developed site area is setback from the river and still provides for flood passage in a superdesign event over a wider channel than required by the river in the 1999 flood.

5. Sediment Overload

ORC has previously sought comment on the scenario where a large landslide generated by an earthquake in the Shotover catchment released large volumes of sediment.

It is generally considered that the Shotover is the main source of sediment into the Clutha River system. The sediment is derived primarily from the geological and climatic conditions and is not related to man's activities to any great degree. The average sediment load is quoted as being about 1.6 million m^3 per year. The generally accepted ratio of bed load (sand and gravel sizes) to total sediment load is of the order of 10%. So approximately 160,000 m^3 per year on average is likely to be bed load.

The ORC 2002 report notes the findings of a 1997 investigation on the Shotover River delta included that "The aggradation/ degradation of bed levels on the Shotover River is a cyclical phenomenon that is closely related to the frequency and magnitude of floods in the Shotover catchment. Aggradation occurs during significant flood events with degradation occurring during more moderate flow periods."

There is not a shortage of sediment from the Shotover catchment and the limiting factor is water energy to transport the sediment. Should an earthquake occur and provide additional sediment into the system it still requires transport to downstream sites. There are a number of wider beach areas upstream with narrower gorges or sections downstream that will settle out sediment before it reaches the State Highway Bridge and the proposed site. Big Beach and Tuckers Beach are examples. See attachment E that shows the riverbed area available for storage downstream of Arthurs Point.

The area of in-channel storage from Big Beach down to the State Highway Bridge is 286 hectares. This equates to 2.86 million m^3 per metre rise in mean bed level. The area of the delta downstream of the State Highway Bridge within the existing riverbed and on the left bank excluding the proposed developed area is about 167 ha or 1.67million m^3 per metre rise in mean bed level. The area of the proposed development is about 21 ha or 210,000m^3 per metre rise in level. It must be remembered that the proposed plan change development site is above existing river bed level and above the 1999 flood level.

In summary the total area available for storage of this potential sediment load from an earthquake derived event is thus about 453 hectares. If the proposed site was to continue to be available for sediment storage it would add about 4.6% to this figure. See table below and attached plan. As the bulk of the material would settle out upstream of the State Highway Bridge, and wedge storage with the greatest depth upstream, the areas downstream of the State Highway Bridge would be less vulnerable than those areas upstream of the Bridge.

Location	**Area hectares**	**Volume /m depth**
Channel storage Big Beach to State Highway Bridge	286	2,860,000 m^3
State Highway Bridge to Kawarau	167	1,670,000 m^3
Proposed development site	21	210,000 m^3
Total	474	4,740,000 m^3

Table 5.1: Sediment storage areas

If the sediment did arrive in large quantities it would result in new terraces in the Big Beach and Tuckers Beach area that would be likely to become at least semi-permanent features as the river would gradually cut its way down through the deposited material. The bulk of material would be trapped in these upper areas awaiting gradual reworking in future floods. The State Highway Bridge and cross sections 1, 2 and 3 would continue to pass material through with the narrower sections and higher velocities than the sections below.

The above illustrates that the impact of the proposed landfill on any future delta aggradation levels as a result of an earthquake induced mass movement in the Shotover catchment will be minimal even if:

a. the event occurs
b. there is sufficient water energy to transport the sediment in far greater volumes than occur naturally now
c. the sediment passes in large volumes through the trap areas at Big Beach and Tuckers Beach.

The ORC concerns for aggradation in sections 1 to 3 was based on the scenario of a major earthquake on the alpine fault triggering extensive slumping and additional material into the river system. For this material to affect the lower Shotover it has to be transported to the area. There are extensive areas on river flat upstream of the highway bridge that would act as temporary storage before the delta would be affected significantly. It is considered that should there be such a major increase in sediment activity that mitigation could be provided through a low floodbank on the left bank of the Shotover River. There would be time to implement such works if they ever prove necessary. It would be prudent to provide for such a bank as a contingency when planning the site layout. A 10m wide strip should be the minimum provided for.

6. Effects on Lake Wakatipu Levels

Flooding from Lake Wakatipu occurs at times of high lake level associated with heavy rain on the Main Divide. Such storm events affect both the Shotover catchment and the main rivers such as the Dart and the Rees that flow into Lake Wakatipu. There is a timing difference on the impacts of these flows due to the routing or lag effect due to the storage in Lake Wakatipu.

The interaction of sediment, lake levels and flows in the Shotover River, Lake Wakatipu and Kawarau Rivers is complex, however the following description summarises the current understanding.

There is a very flat grade in the Kawarau River between the outlet of Lake Wakatipu at Frankton and the Shotover confluence. Under heavy rainfall conditions when Lake Wakatipu is low at the start of a flood and the Shotover River is in high flow the Shotover River can flow back into Lake Wakatipu. As the flow in the Shotover recedes Lake Wakatipu continues to rise until the outflow exceeds the inflow. This may take one to two days. During a flood the Shotover moves a considerable amount of sediment and is on a steeper gradient than the Kawarau River. Until the flow in the Kawarau is more than about five times the Shotover River flow the sediment from the Shotover can infill and reduce the Kawarau channel capacity but at Kawarau flows more than 5 times the Shotover flow then the Kawarau River has sufficient energy to move this sediment faster than it arrives and the Kawarau channel will scour back to its more regular waterway area.

The major floods in Lake Wakatipu are often as a result of two or three storms where the time between events has not allowed either the Kawarau channel at the Shotover delta or Lake Wakatipu to return to their more normal conditions.

In response to the question as to how will the proposed development of Shotover Country impact on the flooding of Lake Wakatipu it is fair to state that:

a. Had developments been in place at the Plan Change site in November 1999, there would have been no impact on flood levels in Lake Wakatipu in that event.

b. It is not considered that the presence of the proposed developed site will have any measurable impact, either good or bad, on future flood levels in Lake

Wakatipu. This applies whether or not works specifically for flood control for Lake Wakatipu and Queenstown are implemented in the future.

7. Proposed ORC/QLDC Initiatives

A report was presented to the QLDC jointly by the Chief Executives of the ORC and QLDC on 29 April 2005.

In this report they acknowledge that the strategies will be best focussed on:

(a) The extent to which individuals, businesses, utilities, and the QLDC (as an owner of assets in flood-prone areas) must practise 'self-help'

(b) The extent to which the QLDC, through infrastructure design and operation, can mitigate the impact of flood events.

(c) The extent to which land development and use within flood prone areas can be managed, through the Resource Management Act, the Building Act, and bylaws, can be used to mitigate flood risk, and

(d) Physical Mitigation Works.

The latter includes a possible wall in Queenstown Bay and maybe works arising from implementation of Shotover Delta training works identified as needed from the computer and physical modelling being undertaken by the ORC and QLDC.

The proposed Plan Change site is not within the active riverbed of the Shotover delta. The low density living activity area is 500 metres or greater upstream of the confluence of the Shotover and Kawarau Rivers. It is anticipated that any training works that may be proposed would be within the current active Shotover River flood channel (see Figure 2.2 photo of the 1999 flood). The proposed Plan Change site would have no effect on any such works in the active flood channel.

The outcome of the Kawarau and Shotover Sedimentation study for ORC by Barnett & McMurray et al of January 2006 recommendation 3 is "Means of training the Shotover to ensure the confluence with the Kawarau River is near the true left or eastern side of the present Shotover delta should be investigated."

On inspection of the aerial photograph and being aware of the hill ridge on the left bank opposite MWD XS 7 towards the lower part of the delta adjacent to the Shotover River it is not considered that such river training works would be carried out on the proposed Plan Change site. There is ample room for any such training works within the currently active delta.

The conclusions in the July 2007 ORC report that incorporate the finding from Davies (2007) are that training of the Shotover River flows in the delta down the left or eastern side will reduce the peak levels and duration of flooding to properties around the shores of Lake Wakatipu. The ORC recommended training lines, vegetation clearance, gravel extraction and river flows are all proposed to be contained within the existing active delta and do not rely on use of freehold land adjacent to the river.

Proposed works for airport runway safety zone and a right bank training embankment by ORC do not significantly impact on the design flood levels at the Plan Change site.

The Otago Regional Council and local gravel extraction operators hold consents to extract gravel from the delta adjacent and downstream of the site.

8. Conclusion

The proposed development site is not currently subject to flooding in events up to the 1:100 AEP event under existing Shotover River cross sections. If the lower Shotover continues to aggrade then some minor flooding at the lower end of the Plan Change site could be experienced. The Otago Regional Council is however encouraging gravel removal in the lower Shotover for river management purposes and this will reduce prospects for aggradation in this area. It is not considered that significant aggradation will occur in the reach through the oxidation ponds. Recommended minimum stopbank levels or minimum hardfill levels have been provided. The willow edge protection should be maintained and strengthened where necessary to ensure lateral erosion is managed. Should major sediment input changes occur because of earthquake induced landslides in the upper catchment there would be adequate time to respond with mitigation measures. The proposed development will not affect flood levels in Lake Wakatipu. The overall concept is a conservative design.

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References

(a) "An Assessment of Possibilities to Reduce Flooding by Lake Wakatipu – A Joint Report Otago Regional Council & Queenstown Lakes District Council" was released in May 1996.

(b) Opus. Flood History in the Clutha Catchment. Prepared for Contact Energy April 2000

(c) Otago Regional Council. Clutha River Catchment Flood Report. March 2000

(d) Otago Regional Council. Shotover Delta Sedimentation. October 2002

(e) Barnett & MacMurray et al. "Kawarau and Shotover Rivers Sedimentation Investigation" ORC. January 2006

(f) "Shotover River Sediment Management: Microscale Modelling" Final Report to Otago Regional Council. TR Davies, Geological Sciences, University of Canterbury March 2007.

(g) Otago Regional Council (File EN0902) Report No. 2007/267 to Engineering & Hazards Committee "Management of the Shotover Delta." July 2007

(h) Geoconsulting Ltd. "Ladies Mile Partnership: Structure Plan Geotechnical Assessment". Report for CFMA. January 2008

Attachments

(A) General location map

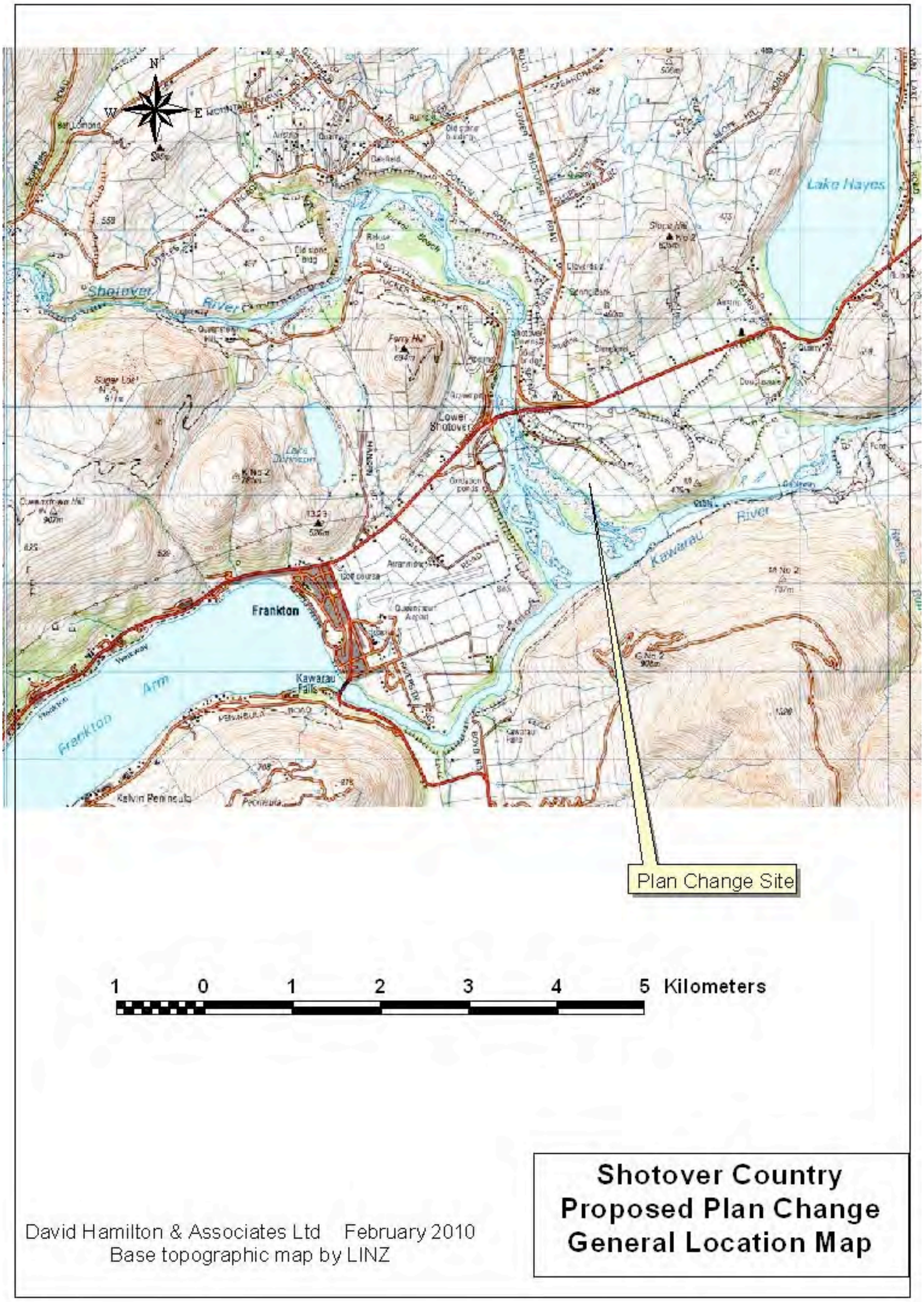
(B) Oblique photo of Lower Shotover and site lower terrace 28 January 2008

(C) QLDC Hazard Register Maps Sheets 30 and 31 (2 pages)

(D) Plan showing lower Shotover River cross section locations

(E) Plan showing Shotover sediment storage areas available downstream of Arthurs Point

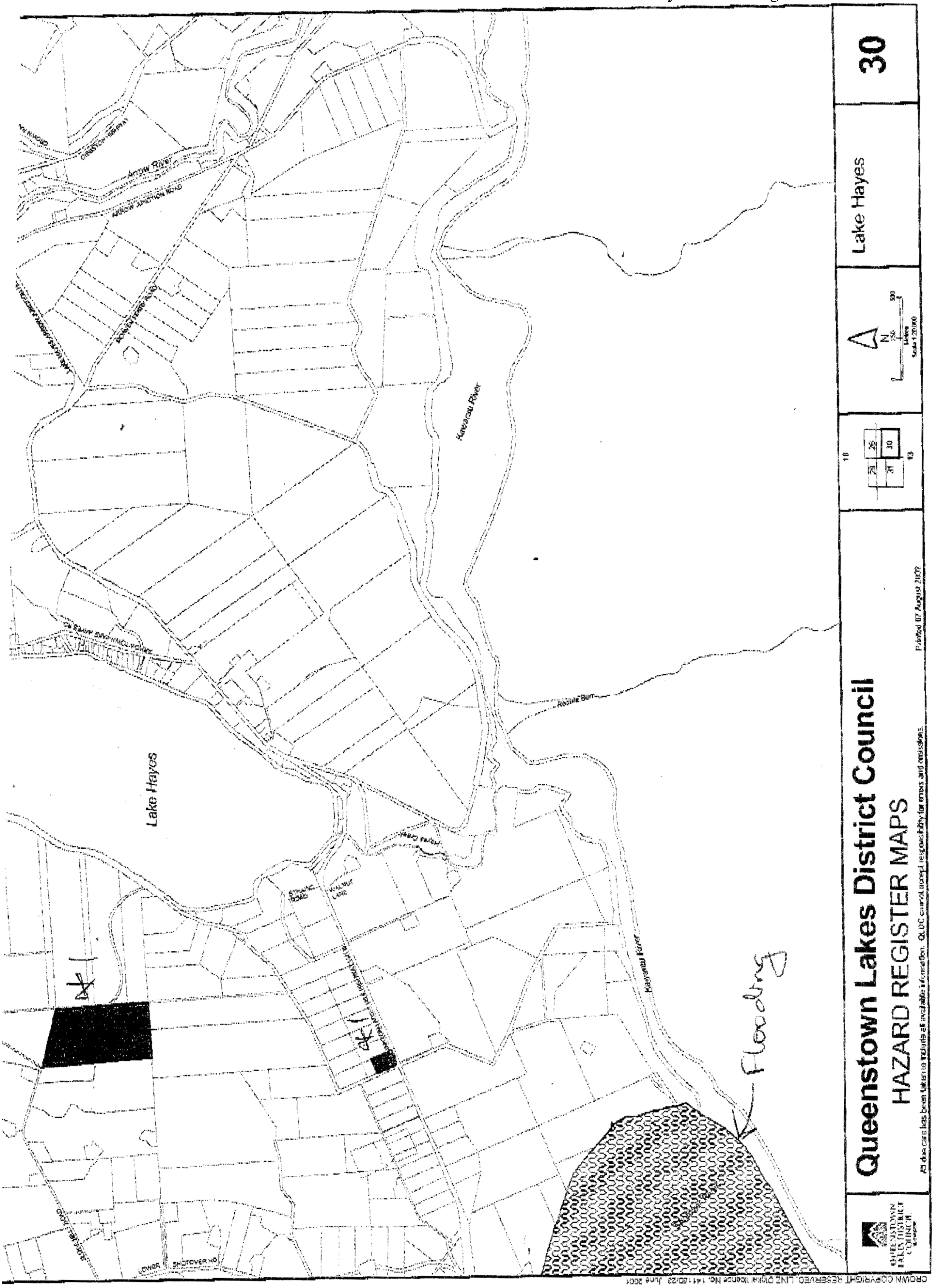
(F) Plan and Cross Sections Shotover Country (3 pages)


N
W
E
Shotover
River
Lake Hayes
Lower Shotover
Frankton
Kawarau Falls
Frankton Arm
Kawarau River
Plan Change Site
1 0 1 2 3 4 5 Kilometers
David Hamilton & Associates Ltd February 2010
Base topographic map by LINZ
Shotover Country
Proposed Plan Change
General Location Map

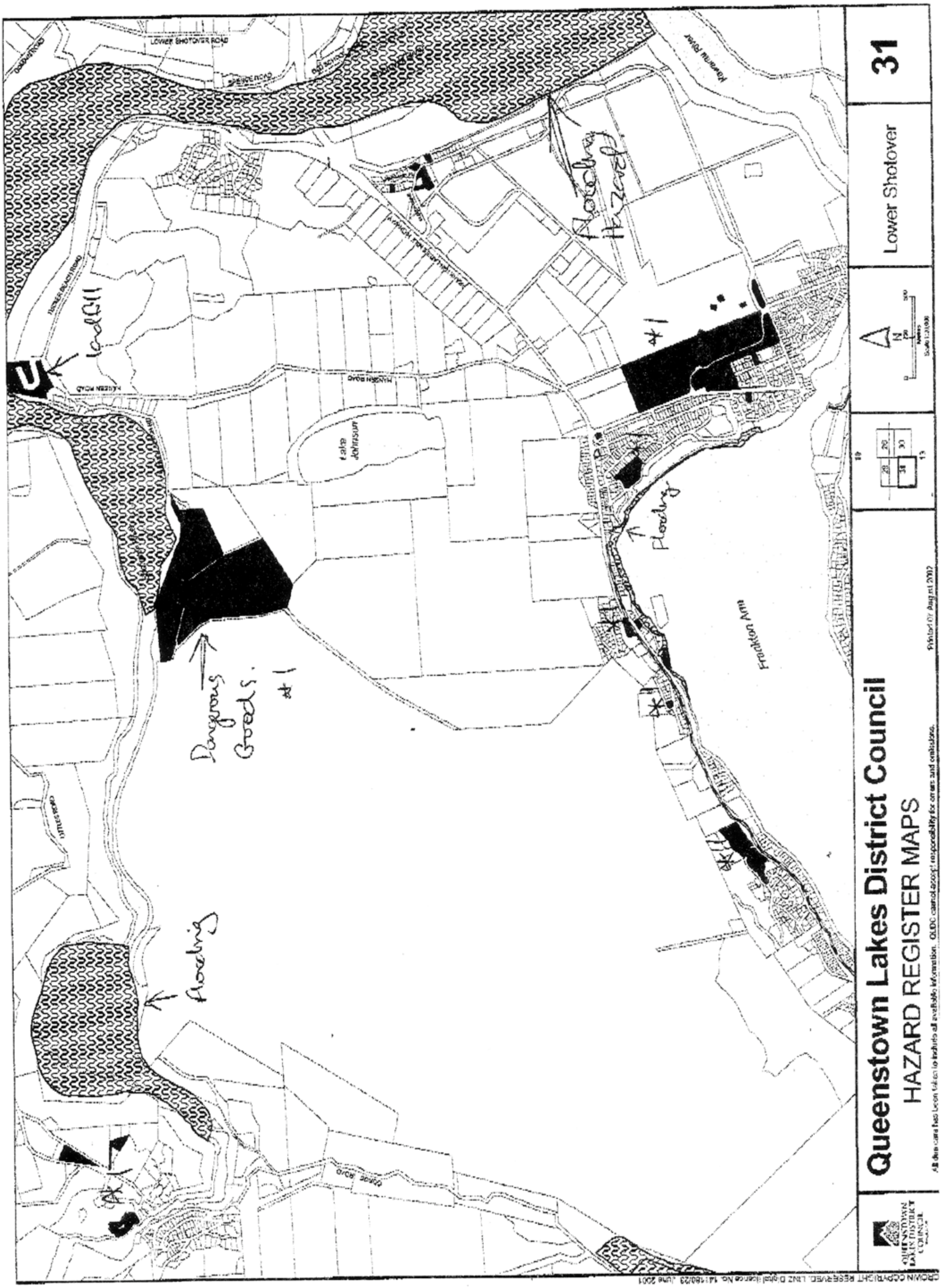
A

Site photo looking north from right bank of Kawarau River
(Credit ORC 28 Jan 2008)

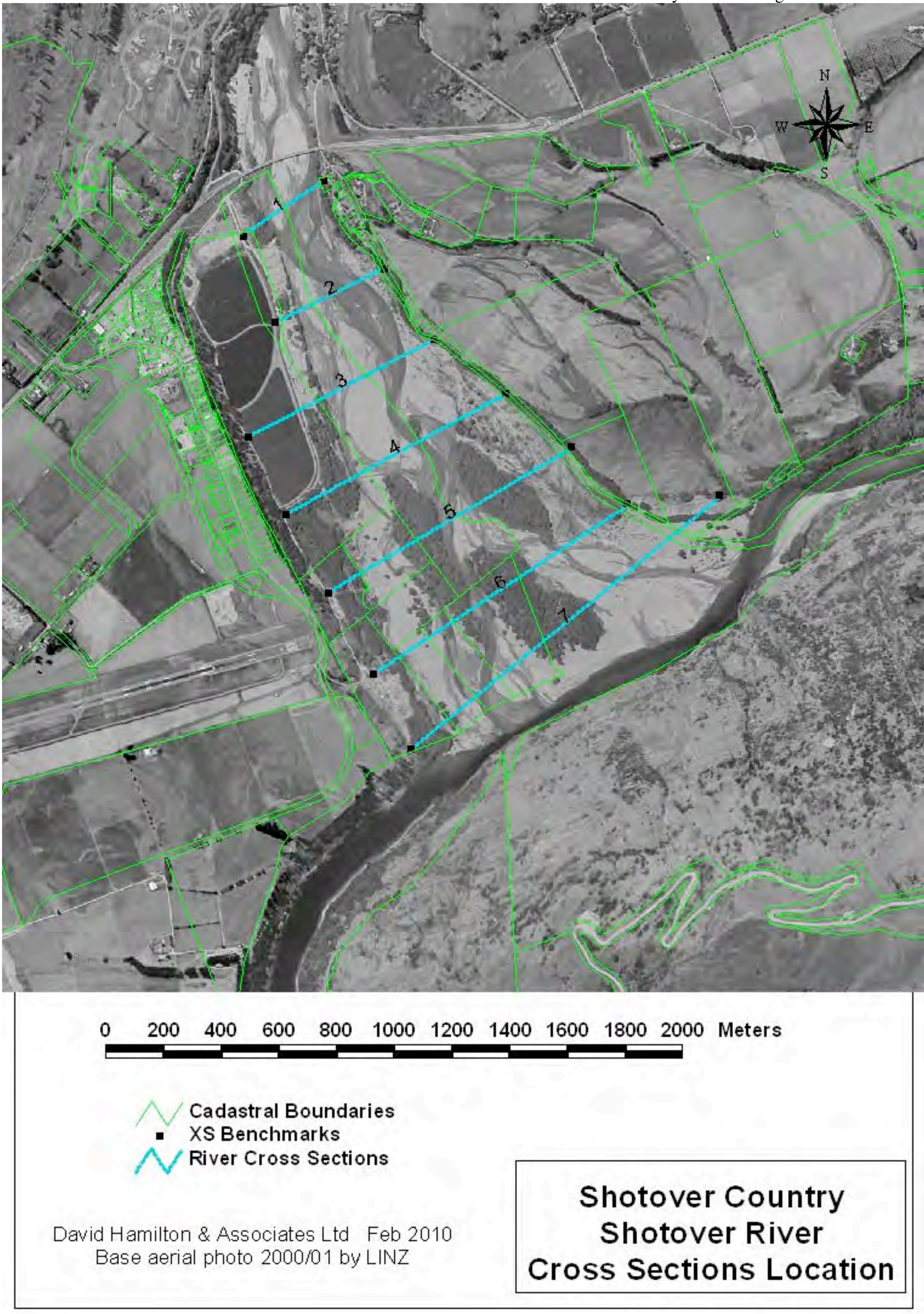
B


Queenstown Lakes District Council
HAZARD REGISTER MAPS
Lake Hayes
30
Lake Hayes
Kawarau River
Flooding

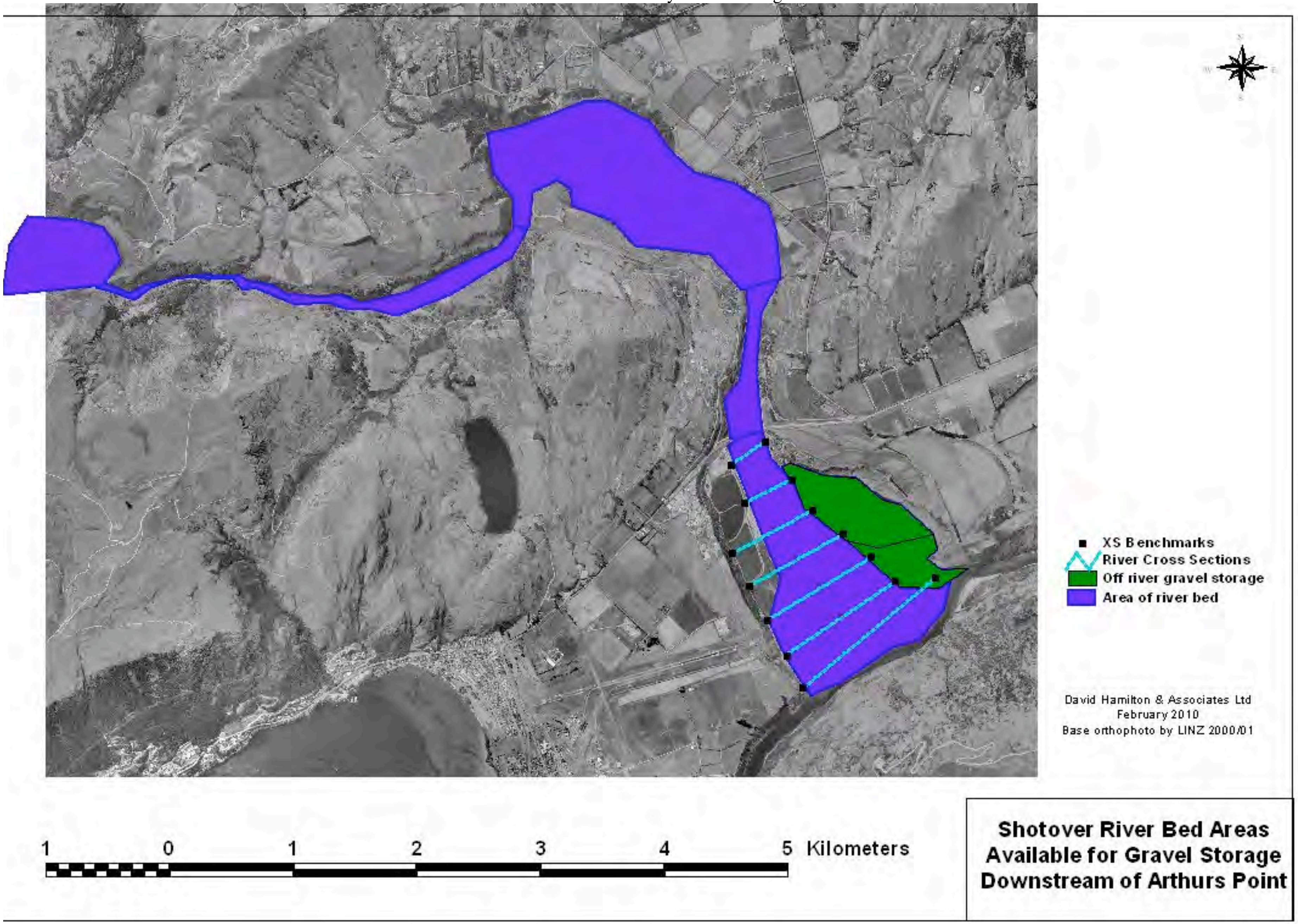
C1


Queenstown Lakes District Council
HAZARD REGISTER MAPS
Lower Shotover
31
Dangerous Goods. #1
Flooding
Flooding
Flooding Hazard
#1
Frankton Arm
Lake Johnson
HANSEN ROAD
LOWER SHOTOVER ROAD
CROWN COPYRIGHT RESERVED

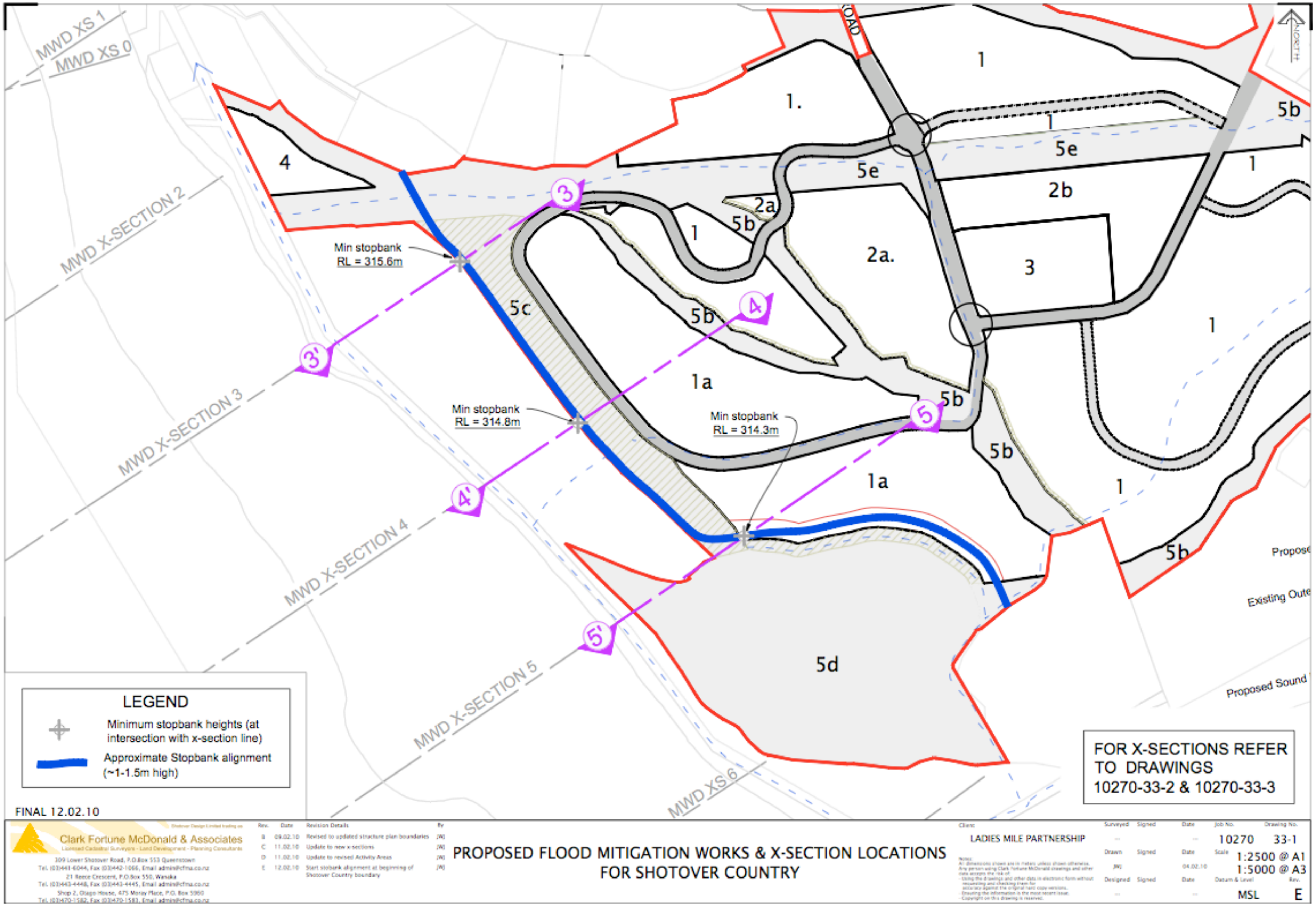
C2


N
W
E
S
1
2
3
4
5
6
7
0 200 400 600 800 1000 1200 1400 1600 1800 2000 Meters
Cadastral Boundaries
XS Benchmarks
River Cross Sections
David Hamilton & Associates Ltd Feb 2010
Base aerial photo 2000/01 by LINZ
Shotover Country
Shotover River
Cross Sections Location

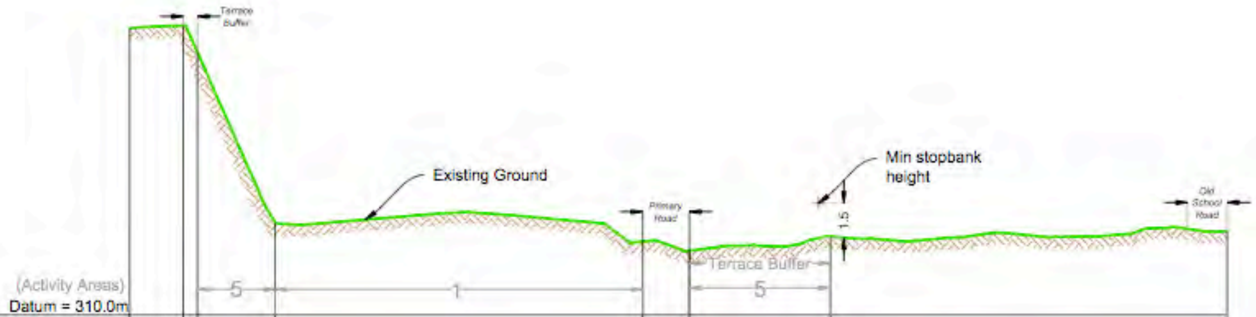
D


XS Benchmarks
River Cross Sections
Off river gravel storage
Area of river bed
David Hamilton & Associates Ltd
February 2010
Base orthophoto by LINZ 2000/01
1 0 1 2 3 4 5 Kilometers
Shotover River Bed Areas
Available for Gravel Storage
Downstream of Arthurs Point

E


MWD XS 1
MWD XS 0
MWD X-SECTION 2
MWD X-SECTION 3
MWD X-SECTION 4
MWD X-SECTION 5
MWD XS 6
NORTH
1
1.
1
5b
5e
5e
1
2b
2a
5b
1
2a.
3
5c
5b
4
1
1a
5b
5b
1a
1
5b
5d
Min stopbank
RL = 315.6m
Min stopbank
RL = 314.8m
Min stopbank
RL = 314.3m
3
3'
4
4'
5
5'
Propose
Existing Oute
Proposed Sound
LEGEND
Minimum stopbank heights (at intersection with x-section line)
Approximate Stopbank alignment (~1-1.5m high)
FINAL 12.02.10
FOR X-SECTIONS REFER TO DRAWINGS 10270-33-2 & 10270-33-3
Clark Fortune McDonald & Associates
PROPOSED FLOOD MITIGATION WORKS & X-SECTION LOCATIONS
FOR SHOTOVER COUNTRY
LADIES MILE PARTNERSHIP
10270
33-1
1:2500 @ A1
1:5000 @ A3
MSL
E

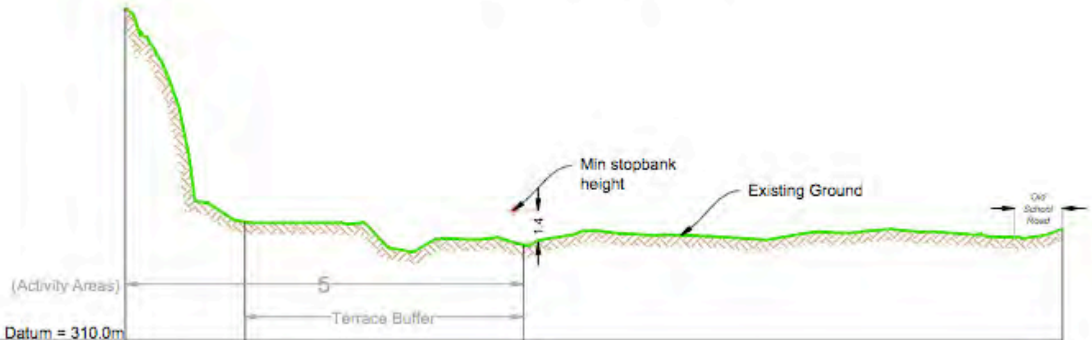
F1



HORIZONTAL DISTANCE	0.0m	23.0m	29.1m	62.6m	220.8m	240.8m	301.5m	471.9m
EXISTING GROUND LEVEL	322.3m	322.4m	321.3m	313.9m	313.2m	312.8m	313.4m	313.6m

SECTION 4-4'

SCALE 1: 2000 (Hz) 1:200(Vt) @ A3



HORIZONTAL DISTANCE	0.0m	51.4m	171.2m	402.7m
EXISTING GROUND LEVEL	324.3m	315.1m	314.1m	314.8m

SECTION 3-3'

SCALE 1: 2000 (Hz) 1:200(Vt) @ A3

FINAL 11.02.10

Clark Fortune McDonald & Associates

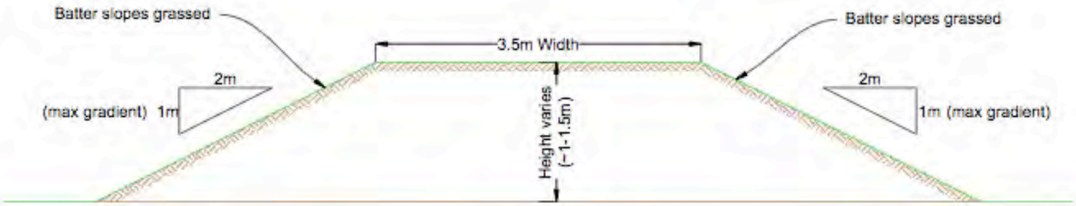
Rev.	Date	Revision Details	By
B	09.02.10	Revised to updated structure plan boundaries	JW
C	11.02.10	Update to new x-sections	JW

X-SECTIONS 3 & 4

Client: LADIES MILE PARTNERSHIP

Surveyed	Signed	Date	Job No.	Drawing No.
			10270	33-2
Drawn	**Signed**	**Date**	**Scale**	
JW		04.02.10	AS SHOWN	
Designed	**Signed**	**Date**	**Datum & Level**	**Rev.**
			MSL	C

F2



TYPICAL STOPBANK X-SECTION

SCALE 1:50 @ A3

HORIZONTAL DISTANCE	0.0m	27.6m	265.4m	317.3m	368.3m	516.2m	547.6m
EXISTING GROUND LEVEL	316.2m	316.2m	315.6m	313.3m	313.3m	313.2m	312.7m

SECTION 5-5'

SCALE 1: 2000 (Hz) 1:200(Vt) @ A3

FINAL 11.02.10

Rev.	Date	Revision Details	By
B	09.02.10	Revised to updated structure plan boundaries	JWI
C	11.02.10	Update to new x-sections	JWI

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Tel. (03)470-1582, Fax (03)470-1583, Email admin@cfma.co.nz

X-SECTION 5 AND TYPICAL STOPBANK X-SECTION

Client: LADIES MILE PARTNERSHIP

Surveyed | Signed | Date | Job No. 10270 | Drawing No. 33-3

Drawn JWI | Signed | Date 04.02.10 | Scale AS SHOWN

Designed | Signed | Date | Datum & Level MSL | Rev. C

F3