Shotover Country – Special Housing Area
Comments on ORC Letter relating to Natural Hazards

1. Introduction

These comments relate to ORC letter 27 October 2015 (ORC Reference A850487) to the QLDC. They do not cover the geotechnical liquefaction issue raised in the last paragraph of the ORC letter.

The ORC letter attaches a submission dated 9 March 2011 to the QLDC in relation to the Shotover River and private plan change 41 by Shotover Country Ltd. The QLDC Commissioners sought expert caucusing between Ramon Strong of the ORC (author of the above submission) and David Hamilton (the writer of these comments) on several matters. An agreed position was tabled for the Commissioners in June 2011. A copy of this is attached "Shotover Country Expert Caucusing_110614" (28 pages).

All of the hydraulic modeling work has relied on river cross-sections surveyed on original MWD sections first surveyed in 1980. Subsequent LIDAR survey by the ORC has also been used.

2. Expert Caucusing Fill Levels and Freeboards for QLDC Plan Change 41

The agreed minimum fill levels in the caucusing report related to design water levels as shown in Table 1 below. The current estimate for the 100-year flood (1% Annual Exceedance Probability (AEP)) event flow is 1500 m^3/s.

Level Comparisons		Levels as Reduced Level MSL Dunedin Datum		
		MWD Cross-Sections		
Item - with predicted bed levels at year shown	**Flow m3/s**	**5**	**4**	**3**
Caucusing Report 2011				
Proposed Minimum Fill Level PC 41		314.35	314.81	315.6
1999 Flood Flow Worst Case MacMurray Flood Level	1400	312.55	313.55	314.35
Freeboard		1.8	1.26	1.25
2060 1% AEP flood with Climate Change	1740	313.53	314.01	314.58
Freeboard		0.82	0.8	1.02

Table 1: Minimum Freeboards with original 2011 Design

3.Peer Review for QLDC

Subsequent to the above the QLDC requested a peer review of the flood hazard to the Plan Change 41 site. Tonkin & Taylor were engaged by the QLDC. As part of the review they sought additional modeling work, including modeling of upstream landslide/debris dam dambreak scenarios.

This work was the subject of a report "Shotover Country Plan Change 41 – Review of Shotover River Flood Risk Profiles – Supplementary Hydraulic Modelling" March 2013. This details the additional work that was undertaken. This was a Draft report that was never updated to Final. A copy can be made available should it be required.

The letter from Tonkin and Taylor dated 8 April 2013 to QLDC is attached. Table 2 attached is the summary output relating to those scenarios. Shotover Country had earlier made a decision to increase the level of the fill for Plan Change 41, and thus the available freeboard, and the impact of that is shown in Table 3 below. The freeboard increase was over 1.1m for the 2060 bed and flow of 1740 m^3/s.

Level Comparisons		Levels as Reduced Level MSL Dunedin Datum		
		MWD Cross-Sections		
Item - with predicted bed levels at year shown	**Flow m3/s**	**5**	**4**	**3**
QLDC Tonkin & Taylor Peer Review				
Proposed Minimum Fill Level Shotover Country Adopted		315.5	315.99	316.75
2010 Bed 1% AEP flood with Medium Climate Change WL	1740	313.31	313.81	314.49
Freeboard		*2.19*	*2.18*	*2.26*
2060 Bed 1% AEP flood with Medium Climate Change WL	1740	313.54	314	314.6
Freeboard		*1.96*	*1.99*	*2.15*

Table 3: Freeboard with Revised Fill Levels Adopted

The ORC comments made in their letter dated 27 October 2015 were without seeing the 2013 Supplementary Modelling Report and the Tonkin and Taylor peer review.

4. Special Housing Area Hydraulic Modelling

With the proposed Special Housing Area there is proposed to be further fill of the true left bank of the Shotover River, between the approved Plan Change 41 and the river itself.

Additional hydraulic modeling work has been carried out. This is reported on in "Shotover Country Ltd - Special Housing Area -Proposed Extension to Shotover Country Zone - Review of Proposed Development on Design Flood levels and Mitigation" David Hamilton & Associates Ltd, August 2015.
The basis for comparison adopted is the Tonkin & Taylor Scenario V. This scenario uses the 2110 projected bed levels with a flow of 2730 m^3/s that is the 1% AEP flood plus two standard deviations and High climate change factors.

Level Comparisons		Levels as Reduced Level MSL Dunedin Datum			
		MWD Cross-Sections			
Item - with predicted bed levels at year shown	**Flow m3/s**	**5**	**4**	**3**	**Top End SHA**
QLDC Tonkin & Taylor Scenario V Plan Change 41					
Proposed Minimum Fill Level Proposed RLm		315.5	315.99	316.75	
2110 Bed 1% AEP flood + 2SD High Climate Change WL	2730	314.4	314.75	315.25	
Freeboard m		*1.1*	*1.24*	*1.5*	
Special Housing Area Fill					
Proposed Minimum SHA Fill Level Proposed RLm		315.5	315.99	316.79	317.34
2110 Bed 1% AEP flood + 2SD High Climate Change WL	2730	314.4	314.75	315.36	316
Freeboard m		*1.1*	*1.24*	*1.43*	*1.34*

Table 4: Freeboard with Special Housing Area Fill

As can be seen from Table 4 the freeboard through the Special Housing Area reach is over 1.2m for the proposed fill areas for the scenario selected. The upstream end of the SHA is further upstream than the Special Zone of Plan Change 41 and the table gives the figure for the northern extremity of the SHA.

A longsection showing this detail is attached as Figure 1.

Also attached are plans and cross-sections provided by Clark Fortune McDonald & Associates, Job No. 11494 Drawing 11 Sheets 1 & 2, Revision A, Special Housing Area – Fill Extension, Client Review 17.09.15. This shows an engineered fill batter at a slope of 2H:1V with additional fill placed at a maximum slope of 15H:1V that will provide an additional 20m buffer width to the superdesign flood flows used of 2730 m^3/s. Toe protection and plantings for erosion protection are proposed.

5. Seismic Hazard

The ORC advised in the letter that a new report "Seismic hazard in the Queenstown Lakes district" August 2015 is now available. This has been reviewed in relation to the SHA flood hazard.

A matter raised is the generation of sediment from seismic shaking that will need to be transported downstream. The carrying capacity of the river system is limited by the available flows. Sediment will drop out where velocities are lower. As the Shotover River valley widens below Arthur's Point sediment would be expected to drop out as the water velocity lowers. The reach from the old State Highway bridge through past the oxidation ponds is much narrower and with the higher velocity it is not expected that sediment would accumulate. Sediment could well accumulate downstream of the oxidation ponds before being conveyed downstream to Lake Dunstan. On-going monitoring of the Shotover River delta, as is currently undertaken, would identify any trends. This would not be an immediate catastrophic event. Quote from page 66 of the report:
"Increased sediment transport in rivers following a large earthquake is anticipated to take decades to work through the river system (e.g., Robinson and Davies, 2013), meaning that delta growth and channel aggradation at the Shotover/Kawarau confluence will be a long- term issue following a large earthquake."

The Seismic Report identifies the potential for a large landslide in the narrow Kawarau Gorge downstream of the confluence with the Arrow River, in the vicinity of the suspension bridge. Should this occur water could back up into Lake Wakatipu. Lower Queenstown starts flooding at about RL 312m. The proposed SHA fill levels are RL 315.99m and above. It is expected that work to lower any landslide dam that would affect downtown Queenstown would be well in hand before flooding would be experienced at the SHA site.

The report prepared by Jeff Bryant of Geoconsulting Ltd, dated 19 April 2011, on potential landslide debris dams was specifically prepared for the Shotover Country developments and thus also applies as a site specific report for the SHA.

The T&T peer review considered the landslide debris dam dambreak scenarios provided by Jeff Bryant in 2011 as suitable, and the effects of large flows released by the failure of such debris dams were modelled. The modeling work has thus shown that the Shotover River can accommodate approximately 3 times the current estimated 1 % AEP flood event before flows would start to impact on the filled level of the proposed SHA.

6. Urban flood design standards

Three times the current 1% AEP flow is equivalent to a greater than a 0.01% AEP (1 in 10,000 year) flood and indeed similar to the estimated Probable Maximum Flood (PMF). This is a super-cautious approach for the SHA that recognizes the potential hazards in a responsible manner.

The normal urban standard of flood protection in New Zealand is to design for a 1% AEP flood (100 year return period event). Recent upgraded flood schemes for the Lower Hutt and Palmerston North have provided for 0.23% AEP (440-year return period) and 0.2% AEP (1 in 500 year) flood events respectively. These use stopbanked systems, where stopbanks themselves can be breached or suffer foundation issues.

The whole SHA area is to be filled to the design level that is above estimated debris dam dambreak flows, so the consequences of a stopbank failure, should only a stop bank be constructed, will not occur.

7. Other Comments

The modeling work as carried out in 2013 and 2015 provides for the ORC established training bank in the lower right Shotover River delta.

The modeling work did not model the effects of the substantial extraction of sand and gravel from the delta for the training bank, the airport extension, or the Shotover Country works. The removal of gravel increases the channel capacity and reduces flood water levels.

The trend over the last 30 years has been for the mean bed level to lower, i.e. degrade, through the reach adjacent to the proposed SHA. For the purposes of flood modeling the 2001 bed levels were assumed to remain, with no further degradation, even though 2010 bed levels were lower. This is a conservative approach to flood modeling levels. However on-going degradation could lead to bank instability. Hence the need for the on-going provision and maintenance for live edge protection works and toe protection of the fill batter slope for the SHA.

The proposed SHA does not reduce the active fairway width. The 1% AEP flood did not utilise the wider low terrace area to any significant extent and so no significant reduction of the floodplain is evident for floods less than the 1% AEP flood. The reduction in floodway width for very large floods is exactly what the hydraulic modeling work was carried out for to see the impact on flood water levels and velocities.

River bank erosion is addressed through the ongoing planting and maintenance and management of the live edge protection work. Depths of flow above natural ground adjacent to the proposed SHA fill are only shallow and unlikely to generate velocities against the proposed fill that would be of concern within the 2730 m^3/s flow band. It is however proposed to fill along the toe with approximately 20m width of fill up to at least the 2730 m^3/s design flood level to provide a greater buffer. Toe protection and a buffer of the live protection is required. Adequate provision for maintenance of this must be made.

8. Conclusion

The recommended minimum freeboard in the original Plan Change 41 report relating to flood hazard was 0.8 m. None of the modelled climate change flows or mean bed level increases have reduced freeboard to less than this height for this higher platform level. Even the modelled flow for a landslide dam breach scenario does not reach the platform level of the SHA. The maximum landslide dam breach flow of 4,600 m^3/s is considered to be similar to what the Probable Maximum Flood (PMF) would be. The analysis has had to make a number of assumptions about tailwater level at the Kawarau River and predictions for changes in the mean bed level of the Shotover Delta. The assumptions are considered to be realistic. Should a landslide dam develop upstream in the catchment it is considered that there would be adequate time available to consider the best options for handling the down cutting of that dam that could take months or years to realise.

Provision for toe protection of the fill and on-going maintenance and strengthening of the live edge protection on the left bank of the Shotover River is considered to be an integral part of the proposed works. Suitable maintenance arrangements should be established.

David Hamilton CPEng

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Attachments

Tonkin & Taylor Letter to QLDC 8 April 2013 re Peer Review of Supplementary Hydraulic Modelling Report

Table 2: Summary of T&T Review Flood Flow Scenarios Modelled Water Levels with Freeboards

Figure 1: Longsection of Shotover River water surface profile for 2110 bed and flow 1730 m^3/s adjacent to proposed SHA

Shotover Country Expert Caucusing_110614 Report

Clark Fortune McDonald & Associates Plans Job 11494 Drawing 11 Sheets 1 & 2, Revision A, Special Housing Area-Fill Extension, Client Review 17.09.15

Tonkin & Taylor

T&T Ref: 53094
08 April 2013

Queenstown Lakes District Council
Private Bag 50072
10 Gorge Road
Queenstown

Attention: Mr J Richards

Dear Jonathan

Shotover Country Plan Change 41

Peer Review: Supplementary Hydraulic Modelling

Following the meeting between Queenstown Lakes District Council (QLDC) and Ladies' Mile Partnership (LMP) representatives at the offices of Mactodd on 24 January 2013, we requested from David Hamilton and Associates (DHA) further computational hydraulic modelling to clarify the risks associated with flood events in the lower Shotover River. Flood risk had been identified, associated with either storm events in the catchment, or with possible flows from the breach of a possible landslide debris dam caused by slope failure along the river channel upstream.

The scope of the additional modelling was set out in Tonkin & Taylor (T&T) letter to DHA dated 11 February 2013. T&T staff discussed aspects of the scope with David Hamilton during his investigations, and T&T received from DHA a report of its modelling on 7 March 2013 (attached).

Background

The LMP development proposals include for new housing building platforms to be constructed on engineered fill to be placed in an area denoted 1b on Clark Fortune McDonald Drawings 10270-49 Sheets 1, 2 and 3 dated 29 November 2011.

QLDC wishes to understand better the significance and adequacy of the freeboard in the context of uncertainty related to the estimation of the design flood, the parameters adopted for modelling flood levels, and possible flood waves released from debris dams that may be formed upstream.

The uncertainty in the flood estimate relates principally to:

- Confidence limits of the design flow, as assessed from historic annual maximum series data
- Possible climate change effects on the flood flow.

The uncertainty in the modelling of flood levels relates principally to:

- Manning's roughness coefficients adopted in the model of the river cross sections
- Aggradation in the delta that would raise bed levels, and the planning horizon over which this process is assessed.

Scope of Supplementary Modelling

The scope of the modelling identified by T&T to clarify the uncertainty with the issues identified above was set out as a number of scenarios. The scenarios included adjustments to modelling parameters and input data to investigate the effects of:

- Confidence limits in design flow
- Higher roughness coefficients for bed resistance
- High impact climate change effects to 2090, i.e. plus 4.6 °C change in mean temperature
- Aggradation of the delta bed levels to 2110.

These scenarios are listed below.

Scenario	Design Flow Estimation	Climate Change (2090)	Manning's Coefficient	Bed Levels	
Base - a	MCE Flood (50% CL)	Medium	0.025	2010	Previously completed by DHA
Base - b	MCE Flood (50% CL)	Medium	0.025	2060	Previously completed by DHA
Base - c	MCE Flood (50% CL)	Medium	0.03/0.06	2060	Previously completed by DHA
I	MCE Flood (50% CL)	High	0.025	2060	
II	MCE Flood (50% CL)	High	0.03/0.06	2060	
III	MCE Flood (50% CL)	High	0.03/0.06	2110	
IV	1Sd Flood (84% CL)	High	0.03/0.06	2110	One standard deviation limit on estimate
V	2Sd Flood (97.5% CL)	High	0.03/0.06	2110	Two standard deviations limit on estimate
VI	Debris Dam Flow I	n/a	0.03/0.06	2110	Flow scenarios developed from Geoconsulting to identify scale of dam that would affect the property
VII	Debris Dam Flow II	n/a	0.03/0.06	2110	
VIII	Debris Dam Flow III	n/a	0.03/0.06	2110	

Supplementary Modelling Results

Boundary Conditions

For downstream boundary conditions to the model, DHA adopted as a minimum value for the water level in the Kawarau River that level which had been adopted by MacMurray and Barnett in its modelling for Otago Regional Council. This is higher than the recoded value during the November 1999 event. This level was adjusted up for higher flows in the Shotover delta, and are summarised in Table 2.3 of the DHA report. The values adopted are considered appropriate for the supplementary modelling investigations.

Bed levels

DHA has determined 2110 bed levels to allow for on ongoing aggradation in the lower Shotover delta over the next 100 years. Cross sections in the model show 2110 bed levels up to 1.4 m higher than the 2010 levels. These have been used to model flood levels for various flow scenarios.

Model roughness coefficients

Modelling scenarios identified include for roughness coefficients, determined during calibration of the model in earlier investigations (i.e. n = 0.025). The coefficients adopted for scenario modelling are 0.030 for gravel reaches, and 0.060 for vegetated reaches in the delta. These values were adopted by Barnett and MacMurray in its modelling.

Flood flow modelling

DHA has presented in its report results for the additional flood scenarios identified, and these results are summarised below.

Scenario	Design Flow Estimation	Climate Change (2090)	Peak Flow (m^3/s)	Manning's Coefficient	Bed Levels	Minimum Flood Freeboard (at Cross section 5a)
Base - a	MCE Flood	Medium	1,740	0.025	2010	2.09 m
Base - b	MCE Flood	Medium	1,740	0.025	2060	1.79 m
II	MCE Flood	High	2,052	0.03/0.06	2060	1.55 m
III	MCE Flood	High	2,052	0.03/0.06	2110	1.26 m
IV	1Sd Flood	High	2,390	0.03/0.06	2110	1.09 m
V	2Sd Flood	High	2,728	0.03/0.06	2110	0.89 m

The minimum freeboard afforded the development for the Base design case, for 2060 bed levels, has been estimated by DHA to be approximately 1.79 m.

If the delta bed levels continue to rise, with little gravel aggradation to remove material deposited, the freeboard of the scheme can be expected to be reduced by maybe 0.5 m by 2110.

The maximum flow modelled was associated with 97.5 % confidence on peak flow estimation, and the High climate change scenario. The peak flow for this scenario, 2,728 m^3/s is nearly 60 % greater (988 m^3/s) than the estimated 100 year design flow (the Base design flow, 1,740 m^3/s) used for the earlier modelling investigations by DHA. For this flow modelled with projected 2110 delta bed levels, the modelled minimum freeboard to the development is 0.89 m.

Debris dam flow modelling

As part of earlier investigation Geoconsulting Ltd had prepared a report on the potential for landslide dam formation and failure. This study considered the formation of a debris dam up to some 25 km upstream of the Shotover delta, with dam heights of 5m, 10m, 20m and 50m. The study postulated peak flows ranging from 81m^3/s to 4,600 m^3/s for breaching of the 5m and 50m high debris dams respectively.

DHA has presented results for modelling of the flows predicted to be generated by debris dam failure. These showed the proposed building platform level to remain above all of the flows generated by the range of debris dam failures considered. Modelling indicates that a peak flow of between of 5,000m^3/s and 6,000 m^3/s would be required in order for the building platform to begin to experience some level of inundation (see below).

It is estimated that the reservoir formed behind a 50 m high debris dam would require nine days to be filled under mean flow conditions.

Scenario	Debris Dam Flow (m^3/s)	Minimum Flood Freeboard (at Cross section 5)
VI	3,000	0.9 m approximately
VII	4,000	0.5 m approximately
VIII	5,000	0.1 m approximately
IX	6,000	- 0.36 m, i.e. flooded

Conclusions

The supplementary modelling undertaken by David Hamilton and Associates has provided very useful data to understand better the significance and adequacy of the freeboard of the proposed Ladies Mile Partnership development. In this respect, the uncertainty related to the estimation of the design flood, the parameters adopted for modelling flood levels, and possible flood waves released from debris dams in the upper catchment has been clarified.

The supplementary modelling results show that:

- For all scenarios identified in the original T&T brief, the minimum freeboard exceeds 800 mm.
- For high climate change impacts to 2090, projected 2110 bed levels in the delta, and 97.5 % confidence limit on the estimation of flood flows, the minimum freeboard to the proposed development, based on the DHA modelling results, would be 0.89 m

- For a debris dam beach-associated flow the proposed development will be 0.1 m higher than the flood levels associated with a flow approximately three times the estimated 100 year peak flow from the catchment
- For a dam breach flow to adversely affect the proposed LMP development, a very large debris dam, with a height in excess of 50m, would likely be required. T&T believes that such an event can be considered to be rare. Nonetheless, should such an event occur the time required to fill the reservoir behind a large debris dam should be sufficient to enable evacuation or other contingency plans to be enacted to manage the risk to downstream property owners and the wider community.

We trust that this meets your requirements, and provides the information required by your Committee. If you would like to clarify or discuss further any aspects of our review please telephone Tom Bassett on DDL 09 355 6031.

Yours sincerely

Kevin J. Hind
PROJECT DIRECTOR

8-Apr-13
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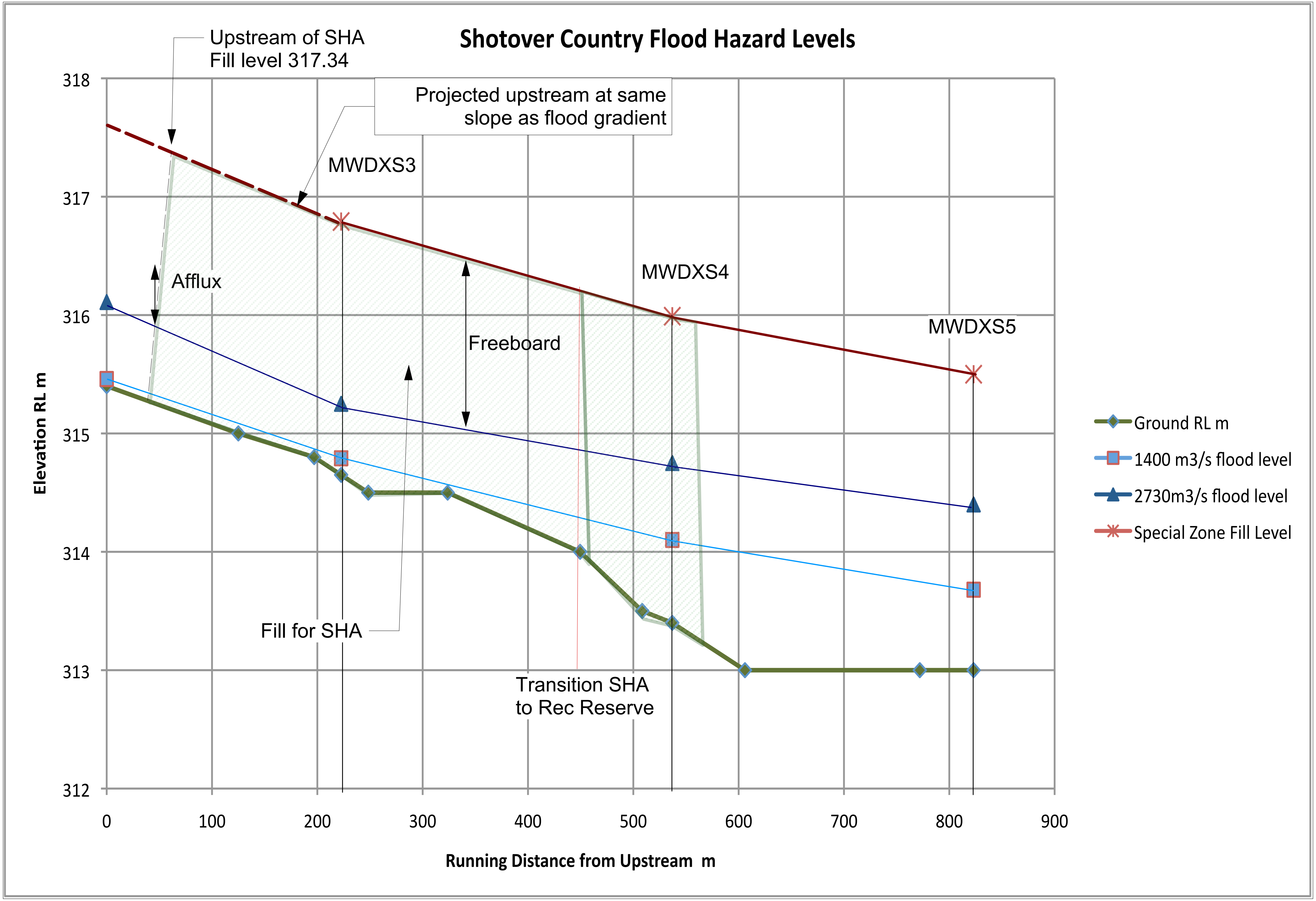

Shotover Country Flood Hazard Levels
Upstream of SHA
Fill level 317.34
Projected upstream at same
slope as flood gradient
MWDXS3
MWDXS4
MWDXS5
Afflux
Freeboard
Fill for SHA
Transition SHA
to Rec Reserve
Elevation RL m
318
317
316
315
314
313
312
0
100
200
300
400
500
600
700
800
900
Running Distance from Upstream m
Ground RL m
1400 m3/s flood level
2730m3/s flood level
Special Zone Fill Level

Shotover Country
Plan Change 41 Expert Caucusing

1. Scope

Prior to closing the Hearing on PC 41, Shotover Country Plan Change, the Commissioners seek the following further information:
In order to adequately understand the potential adverse hazard effects in respect to Activity Area 1A, the Commissioners request that expert caucusing be undertaken between Otago Regional Council, and David Hamilton and Associates, on behalf of the Applicant, in respect to determining an agreed position on the following:

1. An agreed set of flood hazard modelling figures and the resultant flood hazard modelling. It is anticipated that this will include, but not be limited to, a more detailed analysis of the peak flow versus return period relationship for the Shotover River and a more detailed analysis of the Kawarau River at Chards Road dataset to derive reliable high end return period flow estimates;

2. Assuming agreement can be reached, the level of mitigation required to avoid any potential flood event in the plan change site; such as, but not limited to, the type of protection that is appropriate, a proposed minimum ground level within Activity Area 1A and the type and form of any proposed buffer (if both are deemed appropriate by both parties);

3. The potential offsite downstream effects of reducing the flood plain area as a result of the proposed earthworks required to elevate the plan change site in order to mitigate the potential flood hazard;

4. The geotechnical consequences that may arise, in respect to future development, as a result of the fill proposed to mitigate the potential flood hazard in Activity Area 1A. This may include detail in respect to the fill type and the compaction details.

2. First Meeting

Agreement was reached that the expert caucusing should be between Ramon Strong (ORC) and David Hamilton (Shotover Country). A meeting was held at ORC Dunedin on 7 April 2011 to traverse the issues and see where agreement could be reached.

In relation to (1) it was agreed that the Kawarau River at Chards Road dataset would be utilised, in combination with Lake Wakatipu outflow data, to derive a Shotover River dataset. This in turn would be used to derive a range of Return Period flow estimates and compared with the previous analysis undertaken. The data would be made available by the ORC for David Hamilton to undertake the analysis.

In relation to (2) Ramon Strong considered that more detailed and site specific analysis was needed in regard to the impacts that landslides in the main stem of the Shotover and Kawarau Rivers may have on the Plan Change site. David

Hamilton considered the Thomson report[1] to indicate that dam break type scenarios with catastrophic flooding were not likely to result and that this was sufficient. Ramon Strong reiterated the points made in his evidence to the hearing regarding both the context within which this report was prepared and the time elapsed since it's preparation. It was agreed that further work in regard to landslide hazard specifically as it applies to the development proposal would be undertaken.

Further hydraulic analysis may be required on the likely flood profile subject to the outcome of the Shotover River flood flows in (1). It is considered that the experts may well be able to agree on a methodology for determining design flood heights. Ramon Strong reiterated that the ORC is unlikely to offer an opinion on matters such as the adequacy of freeboard to fill level. Matters of concern to the ORC are that the consideration of the plan change is based on sufficiently robust and comprehensive technical analysis in regard to the hazards that potentially affect the site.

Both items (3) and (4) are considered to be readily resolved.

Both experts agree that there will be minimal downstream effects of reducing the floodplain area as a consequence of the filling proposed as part of the Plan Change.

The ORC are of the view that liquefiable soils are unlikely to be present within the footprint of the proposed fill platform , based on the nature of the alluvium (the distribution of particle sizes) evident in the Shotover River adjacent to the proposed fill platform; the ORC is not aware of any site specific investigation having been undertaken in that regard. Although the fill platform will be elevated (and therefore groundwater levels within the fill will be relatively low) it's design will need to include appropriate consideration of the potential for elevated pore water pressures to be generated under earthquake conditions, the potential effects of those elevated pore water pressures on the integrity of the fill platform and (if necessary) how those effects are to be mitigated.

3. Second meeting

A meeting was held in Dunedin on 27 April 2011 to discuss Items (1)and (2). David Hamilton had briefed Jeff Bryant of GeoConsulting Ltd for further assessment of landslide scenarios in the Shotover catchment. GeoConsultings' report on Shotover landslides was received over Easter and had been pre circulated but with insufficient time to review it by Ramon Strong.
The methodology for the review of the hydrology/ flood frequency was agreed now that the hydrological data was to hand.

4. Third meeting

A meeting was held in Alexandra on 11 May 2011.

[1] Landslide Dam Scenarios in the Upper and Lower Reaches of the Kawarau and Shotover Rivers Respectively. Royden Thomson, 1996.

The hydrological review and updated flood frequency were agreed as being a significant improvement on the flood frequency curve derived from the Shotover at Bowens Peak certified data and through Regional Flood Frequency estimation techniques. An estimate of 1,230 m^3/s for the 1999 event at Bowens Peak was derived; this was revised to 1,400 m^3/s based on work undertaken in 2006 by Barnett and MacMurray Ltd. The use of 1,400 m^3/s in the flood frequency analysis gives a mean annual flood estimate of 500 m^3/s and 1% AEP flood estimate of 1,500 m^3/s.

The peak flood flow used for the analysis was factored to allow for climate change effects (coarse flow factoring based on MfE guidelines for increased rainfall due to climate change for the year 2060), giving a 1% AEP flood estimate of 1,740 cumecs. The experts agreed that this derivation methodology is adequate for the analysis. A flow of 1740 cumecs currently would be an 0.45% AEP flood (1 in 220 year event) based on the latest derived flood frequency curve. A short report on the method and outcomes has been prepared.

Using the 1999 flood peak flow and actual flood levels a revised calibration of Hamilton's simplified mean bed level model derived a Mannings n roughness coefficient of 0.020. However both experts were of the view that such a value was unrealistically low and the anaylsis has been based on a value of 0.025. The model was run for Year 2060 scenario using a range of bed level scenarios consistent with David's earlier analysis. The downstream control levels at the Kawarau River were increased over the 1999 level to RL 312.9m to recognise the higher Shotover flows than the 1999 flood.

These runs have not allowed for gravel removal associated with possible future large scale gravel extraction that the ORC are endeavouring to facilitate. This additional analysis has resulted in increases, albeit relatively minor, in minimum platform levels.

David's analysis has been compared with analysis undertaken by Dr. Hugh MacMurray of Barnett and MacMurray Limited, using a hydraulic model of the lower Shotover River developed for the ORC. This comparison has shown David's analysis to be sufficiently conservative.

The analysis is also based on assumptions in regard to river bed level changes in the future, assumptions that are based on data recorded over a relatively short period and that take little account of more substantial increases in bed level attributable to either short term or longer term processes, or a combination of the two.

The ORC note that the analysis presumes that the 100 year Return Period (1% AEP) event is a reasonable basis for the flood hazard assessment and consequently determining the level of any fill platform.

GeoConsulting's Report dated April 2011 on the landslides in the Shotover also references the 1996 report of Royden Thomson and considers several landslide dam scenarios. The report notes that "this exercise is to provide subjective opinions only and is not intended to be a definitive assessment of flood risk associated with landslide dam breaching".

Jeff Bryant undertook a desk exercise using topographical maps and aerial and satellite photography to identify those sites, assess the likely characteristics of those sites and estimate the dammed water volumes that could occur if the landslides occurred rapidly (landslides with slow rates of failure will add to sediment loads but are unlikely to pose any risk in regard to significant impoundment of water and sediment).

A key factor is the rate at which these landslides would fail; the report notes that apart from Lochnagar "there are no other surviving landslide dams in the Shotover Catchment". Jeff goes on to note that "any previous blockages [along the main branches of the river] have failed and left little or no evidence of having dammed the river". This contradicts a conclusion further on in the report that "landslide dams ……. seldom erode entirely down to the original base level".

The report considers landslide dam blockage scenarios of up to 50m high, noting that rapid failure would result in flood flows two to three times that of the 1999 flood. However in Jeff's view landslide dams "don't release all the impounded water instantaneously". In regard to that rate of failure, Jeff cites other studies that have found that 50% of catastrophic landslide dam failures occurred within 10 days of formation and that 85% failed within the first year.

Jeff Bryant comments that in his opinion catastrophic failure is unlikely (that is, any landslide dam is likely to last more than 12 months) although noting that this is based on "the limited experience gained from the Criterion Creek and Mother Rapid failures", events that clearly do not fall into the catastrophic category.

Jeff Bryant considers that Royden Thomson's assessment that significant landslide events in the Shotover are likely to be in the 0.25% to 0.5% AEP range is not unreasonable, although he notes that the basis for this assessment is not clear. Royden Thomson also concluded that downstream flooding would not be expected to be greater than a natural event. David Hamilton agrees with Royden Thomson's conclusions. The ORC does not agree with these conclusions.

The likely backwater effect of a landslide dam downstream of the Shotover confluence was also reviewed. If this occurred the possibility of the Shotover River flooding the Plan Change 41 Area 1A site was revisited. The probability of a landslide that may result in damming of the river with sufficient longevity to raise the level of Lake Wakatipu is considered to be low (Thomson 1996). The likelihood that after this dam had formed that there would be a 1% or larger event in the Shotover River or from Lake Wakatipu is relatively low within a year or two of the dam forming and the compounding probability of this means a lower risk profile. As stated in Hamilton's report to the hearing the Plan Change site is 0.9m or higher than the level of the 1999 flood in Lake Wakatipu that flooded the lower parts of Queenstown, or 2.4m higher than when flooding commences in lower Queenstown.

5. Outcome of Caucusing

1(a) A revised set of flood frequency figures for the Shotover River at Bowens Peak has been derived with the 1999 flood estimated to have a peak flow of 1,400 m^3/s and the 1% AEP flood estimated at 1,500 m^3/s. ORC consider the derivation of these figures to be more robust than those previously used.

1(b) Flood hazard modeling work using 1999 data for calibration has been carried out and the flood profiles for the current 1% AEP and predicted Year 2060 levels for a flow of 1,740 m^3/s incorporating a relatively crude allowance for climate change and modest allowances for bed level increases. This analytical work compares well with other analyses. The ORC note that using the estimated 1% AEP event for the analysis presupposes that it is an acceptable minimum standard in regard to flood hazard risk exposure. Although the 1% AEP event is a relatively low probability event the consequences will be significant.

2(a) Shotover Country propose a freeboard to the fill level of at least 0.8m above the flood profile for the 2060 climate change flow. The platform levels are as proposed by Shotover Country in the application for consent at the upstream end of the fill and raised by 0.1m at the downstream end of the proposed fill. The ORC make no comment on the adequacy or otherwise of this provision.

2(b) The depth and velocity of flow against the proposed fill are relatively low and a topsoiled and grassed batter slope of 2H:1V or flatter is considered acceptable. Planting of trees and shrubs for landscaping purposes between the fill area and the existing willow edge protection on Terrace T5 is considered to be acceptable. Both experts agree that the existing left bank willow protection work adjacent and upstream of the site needs to be maintained. The ORC consider this to be an absolute minimum and that more robust edge protection may well be warranted if the development proposal were to proceed.

2(c) It is agreed that significant landslides in the Shotover River catchment could result in additional sediment loads that are likely to elevate river bed levels in the lower Shotover. It is also agreed that significant landslides could result in catastrophic failures akin to dam break analysis for the site, significantly adding to river flows at a time when the river is likely to be in flood. The ORC are of the view that the risk (being function of probability <u>and</u> consequence) associated with this natural hazard to the development proposed is sufficiently high to be a major consideration. ORC consider the historic assessments of this risk and the assessments provided to date are not sufficiently robust or comprehensive to be able to accurately quantify the risk and thus allow for it to be catered for in the development proposal. The ORC consider the significance of this matter to be such that it warrants more comprehensive assessment by a recognised expert in the field of landslide dams. A decision on the Plan Change can only be made on this basis. David Hamilton concurs with the geologists view that such an occurrence does not appear to have occurred over the last several hundred years and that a catastrophic failure akin to a dambreak is unlikely. David Hamilton is of the view that further analysis is unlikely to be able to better quantify the risks.

3. It is considered that potential offsite downstream effects of reducing the flood plain area as a result of the proposed earthworks required to elevate the plan change site in order to mitigate the potential flood hazard are not significant.

4. The ORC are of the view that liquefiable soils are unlikely to be present within the footprint of the proposed fill platform, based on the nature of the alluvium (the distribution of particle sizes) evident in the Shotover River adjacent

to the proposed fill platform; the ORC is not aware of any site specific investigation having been undertaken in that regard. Although the fill platform will be elevated (and therefore groundwater levels within the fill will be relatively low) it's design will need to include appropriate consideration of the potential for elevated pore water pressures to be generated under earthquake conditions, the potential effects of those elevated pore water pressures on the integrity of the fill platform and (if necessary) how those effects are to be mitigated.

Signed

……………………………………………….. ……………………………………

David Hamilton Ramon Strong

Attachments

1. Lower Shotover River: Flood Risk Through Landslide Dam Breach, Geoconsulting Ltd, 19 April 2011 (10p)

2. Shotover River Review of Flood Hydrology-*Prepared for Shotover Country in Relation to QLDC Plan Change 41,* David Hamilton & Associates Ltd, May 2011

3. Shotover River Review of Flood Profile - *Prepared for Shotover Country in Relation to QLDC Plan Change 41,* David Hamilton & Associates Ltd, June 2011

Notes prepared by:

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19 April 2011

The Consultancy Manager
David Hamilton & Associates Ltd

376C Earnscleugh Road RD 1

Alexandra 9391

Dear David:

Lower Shotover River: Flood Risk Through Landslide Dam Breach

Introduction

The Shotover Country Partnership is currently seeking planning approval for subdivision of land on the true left of the Shotover River, between the SH 6 Bridge and the confluence with the Kawarau River. Otago Regional Council has raised concerns about the potential for flooding associated with breaches of landslide dams further up valley. Flood risk can be enhanced in one of two ways:

- Primary flooding. This can arise following a rapid breach of the dam leading to a sudden release of the entrained reservoir as a 'one-off' flood.
- Secondary flooding. This can arise following aggradation of the Lower Shotover delta area elevating the base level upon which the normal range of floods are superimposed. A rapid breach of a landslide dam would release a pulse of sediment that would accumulate in the lower reaches of the river.

To address this concern, you have briefed Geoconsulting (by email dated 15 April, 2011) to investigate a number of likely scenarios whereby the Shotover River is dammed and subsequently breached. It is stressed this exercise is to provide subjective opinions only and is not intended to be a definitive assessment of flood risk associated with landslide dam breaching.

Previous Work

As part of this desk exercise, reference was made to a similar study by R. Thomson in 1996[1]. This study was concerned with the potential for blockage of Lake Wakatipu outflow

[1] Thomson R "Landslide Dam Scenarios in the Upper and Lower Reaches of the Kawarau and Shotover Rivers Respectively" prepared for Otago Regional Council, May 1996

and consequent flooding of Queenstown. Several scenario landslide dams were considered in the section of Shotover River between Tucker Beach and the confluence with Moonlight Creek.

A review was also made of previous Geoconsulting work of relevance:

- Criterion Creek Landslide 1993. A 7-8000 m^3 landslide on the true left about 700 m downstream of Long Gully (23 km upstream of delta confluence) partially dammed the river. The debris raised the river bed level approximately 2-3 m on the opposite side and created a reservoir stretching back about 1 km. According to rafters, the dam lasted about 1-2 years and seemed to episodically erode during floods with no catastrophic release of impounded water.

- Mother Rapids Rockfall 1997. A small (50 m^3) rockfall from bluffs on the true right about 700 m upstream of the Moonlight Stream confluence (17 km upstream of delta confluence) caused a partial blockage and flooding upstream for about 700 m. Rafters blasted some of the larger blocks to ease passage for their rafts with consequent release of some impounded water. The remaining reservoir drained shortly after with the passing of several flood flows similar to the above feature. Also of interest was the presence of a very large landslide on the true left bank which extends upslope over a vertical height range of about 200 m. Active movement on this slip has led to a constriction of the valley and formation of the rapids. Following the rockfall, part of the river flow was pushed over to the left bank and the consequent erosion initiated renewed activity visible by fresh tension cracks on a localised part of the true left landslide.

- Moonlight Track Landslide 2008. A rockslide of about 10,000 m^3 developed just below the Moonlight Track some 14.7 km above the delta confluence. The failure developed about 150-200 m above the river with most of the debris accumulating in a gully leading up from the river. No actual blockage occurred as a result.

The three areas are shown on Figure 1 and images related to Criterion Creek landslide are shown on Figures 2 & 3.

Likely Landslide Scenarios

As part of the desk study, information was gained from examination of topographical maps, and online satellite (Google Earth, Ovi Maps) and aerial photograph (QLDC Mapinfo) imagery. Previous helicopter flights through the gorge have also identified signs of activity on a much wider scale, on both banks, but have not been followed up with more detailed field checks. Areas showing tension cracks, bare scarps and other signs of active landslide morphology or are being actively undercut at the toe (all on the true left) are shown outlined in red on Figure 1. In addition, there are many areas of potential first-time sliding, mostly on the true right where the banks are steeper due to the regional schistosity giving rise to a marked valley asymmetry.

Likely triggers for either existing landslide reactivation or first-time sliding are most usually extreme storm events or strong earthquake shaking. However, first-time sliding can also arise due to progressive valley-flank relaxation often preceded by oversteepening of the toe. The three examples studied by the author all failed without any obvious triggers.

For a landslide dam to occur, rapid movement must occur, i.e. at a rate faster than debris can be removed by river erosion. Thus failures giving rise to dam emplacement in less than a day are more likely to impound water than slow creep type movements. In this regard, first-time failures are considered to have a greater potential for dam creation than reactivation of existing landslides.

Dam Characteristics

Any predictions relating to longevity of potential landslide dams are highly speculative due to the myriad of unknown factors influencing their location, valley profile, magnitude, debris composition and flood breach characteristic. General comments can be made based on a limited knowledge of dams in the Shotover River and other catchments in the South Island.

The pelitic schist forming the bedrock in much of the Shotover catchment is known to be susceptible to erosion and mass movement.

In general, landslide dams comprising coarse, bouldery debris are most likely to resist rapid breaching. The coarse debris provides a natural armouring against overflow erosion and is more likely to have a high permeability and thus pass part or all of the reservoir inflow subaerially. Examples of the above include: Lakes Marion and Adelaide (west Hollyford Valley), North Young (Aspiring National Park) and Lochnagar (upper Shotover Valley). Such dams can last for millennia although not all are indestructible.

Reactivation of existing landslides will generally lead to progressive diminution of debris such that any dam formed is likely to comprise a broad range of particle size. Following overflow, the finer grained matrix is usually winnowed out leaving the coarser debris in point-to-point contact armouring the channel. The author is unaware of any dams that have formed following reactivation of an existing landslide in recent history or have formed in prehistoric times and lasted to this day.

Trial reservoir volumes have been determined for a variety of dam height scenarios. The height is measured from the flood plain (river bed) to the lowest point on the dam crest. Relatively small landslide dams are considered here (< 50 m high) as smaller magnitude failures are considered more likely than very large ones. Features like Lochnagar and North Young landslide dams have crest heights in the range 70-90 m, however, failures of this magnitude are considered to be of very long recurrence interval and of less relevance to this particular study.

A flood plain width of 40 m and 1:1 side slopes have been assumed in the following calculations to give reservoir volumes shown in Table 1. Actual volumes will be slightly less once the volume of the upstream half of the dam is subtracted.

Given a mean flow of 52 cumecs (at Bowen Peak), times to fill the reservoirs range from less than 10 minutes (5 m high dam) to 10 days (50 m high dam). However, flows can range from half the mean flow to 440 cumecs for the mean annual flow and up to 1180 cumecs for the 1% AEP flood flow so filling times could vary widely.

Table 1

Section of river	River gradient	Lake volume for 5 m high dam (m^3)	Lake volume for 10 m high dam (m^3)	Lake volume for 20 m high dam (m^3)	Lake volume for 50 m high dam (m^3)
RL 340 - 360	2.5%	22,500	100,000	480,000	11,700,000
RL 360 - 380	0.67%	84,000	375,000	1,800,000	19,687,000
RL 380 – 400	0.52%	108,000	481,000	2,280,000	43,425,000
RL 400 – 420	0.51%	110,300	490,000	2,340,000	43,875,000

Dam Breach Scenarios

Apart from the Lochnagar example, there are no other surviving landslide dams in the Shotover Catchment. This suggests that any previous blockages have all failed and left little or no evidence of having dammed the river. Two possible breach scenarios present themselves. The first involves gradual or episodic incision accompanied by concurrent sedimentation of the reservoir basin. Secondly, catastrophic failure can occur during a severe rainstorm event whereby rapid incision releases significant volumes of impounded water which in turn intensifies the erosive power of the overflow. It is the latter scenario that poses the most risk to river users and riverside communities downstream.

Costa and Schuster[2] have found that, for catastrophic failures, 50% failed 10 days after formation and 85% failed within the first year. Dams with coarse blocky debris are likely to survive longer than dams comprising earth debris. Given the limited experience gained from the Criterion Creek and Mother Rapid failures, it would be reasonable to expect any future first-time failures to last at least 12 months under normal flow conditions. Note that time to failure does not take into account any intervention by man to remove the hazard.

Very few estimates have been made of peak flood discharge from breached landslide dams. Costa and Schuster have computed or indirectly measured peak discharges from 12 case histories and developed a regression equation with potential energy as the independent variable:

$$Q = 0.0158\,(PE)^{0.41} \qquad \text{(Coefficient of determination, } r^2=0.81\text{)}$$

Where Q = peak discharge in cumecs and PE = potential energy = h.v.φ

From this equation, peak discharges can be estimated for the above dam breaches (Table 2)

[2] Costa JE & Schuster RL “The formation and failure of natural dams” Geol. Soc. America Bull. V100, July 1988

Table 2

Section of river	Peak Q for 5 m high dam (m^3/s)	Peak Q for 10 m high dam (m^3/s)	Peak Q for 20 m high dam (m^3/s)	Peak Q for 50 m high dam (m^3/s)
RL 340 - 360	80.5	197.3	498.6	2689
RL 360 - 380	138.2	339.2	857.3	3329
RL 380 – 400	153.2	375.6	944.6	4604
RL 400 – 420	154.6	378.5	954.7	4623

Flood Routing

It can be seen from Table 2 that peak flows are likely to be less than the mean annual flood (440 cumecs) for breached dams up to 10 m height but breached dams higher than 20 m can exceed the 1% AEP flood (1180 cumecs).

The main landslide dam scenarios that have been considered lie between Oxenbridge tunnel and Long Gully or between 14 km and 24 km upstream from the confluence with the Kawarau River. Flood routing through this distance will be dependent on the bed resistance, attenuation and storage characteristics of the lower Shotover Valley. The constricting effects of the canyons will cause flood heights to rise dramatically but where the valley opens out (e.g. Big Beach, Tuckers Beach, Delta area) flows and flood levels will diminish and entrained sediment will deposit out on to the flats. Actual flood levels will be influenced by the precursory flow levels in the river prior to breaching.

In reality, landslide dams don't release all the impounded water instantaneously and seldom erode entirely down to the original base level. As the overflow channel starts to incise, the lateral banks become oversteepened and collapse into the channel, possibly even blocking the outflow temporarily. The coarsest blocks resist erosion and accumulate in the channel invert to provide increasing resistance to further downcutting. Finally, as the lake is drained, the discharge decreases such that downcutting comes to a halt and an equilibrium outflow is established.

Discussion on R. Thomson's Report

The above report considers the possibility of valley blockage following reactivation of existing landslides in the section of valley between Tuckers Beach and Moonlight Creek confluence. Areas further upstream were not considered as aerial photograph coverage was not available for assessment.

This assessment extends upstream as far as Long Gully confluence and includes a number of landslides displaying much greater mobility as well as very steep canyon flanks with first-time failure potential. While ongoing activity of existing landslides is constricting the river channel and is responsible for the formation of rapids, it is considered that further constriction is constrained by the enhanced erosion of the landslide toe such that complete blockage is extremely unlikely. The rate of movement downslope is likely to be matched by the rate of removal at the toe. By contrast, dam formation is considered more likely to occur when a first-time failure is rapidly emplaced into the valley floor.

Thomson assesses the probability of failure leading to a dam blockage in the area studied to be 0.25-0.5% pa when taken over a 200 year period. It is unclear what the basis for this assessment is although the figures quoted are not unreasonable.

Closure

This study has noted the potential for valley blockage through either reactivation of existing landslides or first-time failures. Numerous areas in Shotover canyons have been identified as having such potential. Of the two possible means of dam formation, first-time failures with their rapid emplacement are considered much more likely than reactivated landslides where any movement is likely to be matched by toe erosion.

There are too many unknowns regarding location, valley profile, magnitude of failure and height of dam to allow a determination of reservoir size, longevity, breach potential and flood routing characteristics.

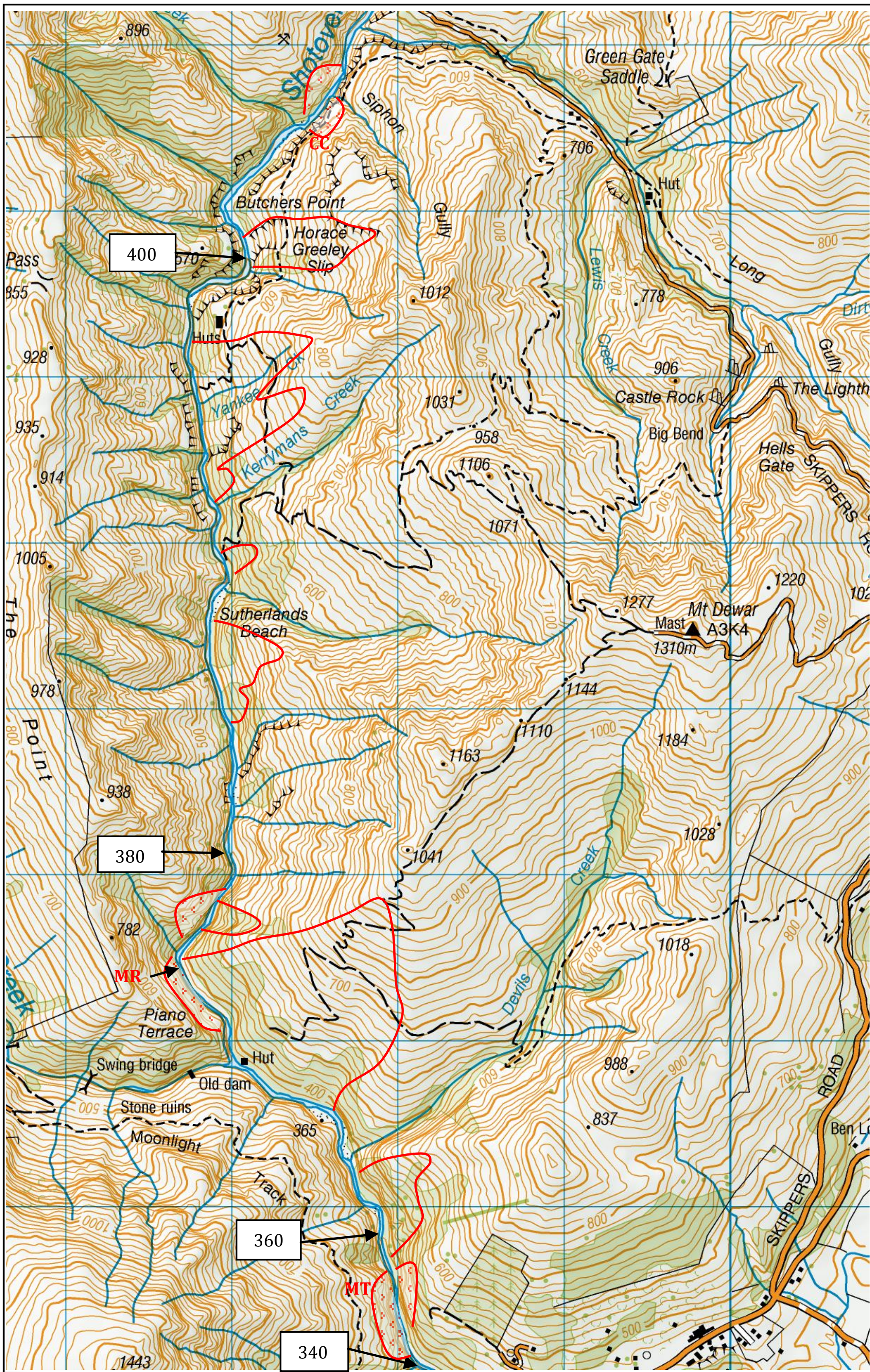
It should be noted that the economic value of the Lower Shotover River is very high with rafting and jet boating both being multi-million dollar enterprises. It is thus highly likely that some intervention measures will be initiated to ensure the risk to downstream users and communities is minimised and the river is restored back to its natural state.

Sincerely,

Geoconsulting Ltd

per J.M.Bryant

M.Sc. F.G.S.



JEFF BRYANT GEOCONSULTING Lt

PROJECT: Landslide dam scenarios, Lower Shotover River

FIGURE: 1

DESCRIPTION: Areas of active landsliding outlined in red. Previously studied landslides: CC Criterion Creek; MR Mother Rapids, MT Moonlight Track.

340 Contour level where it crosses river.

Potential first-time landslide areas.

Scale: NTS

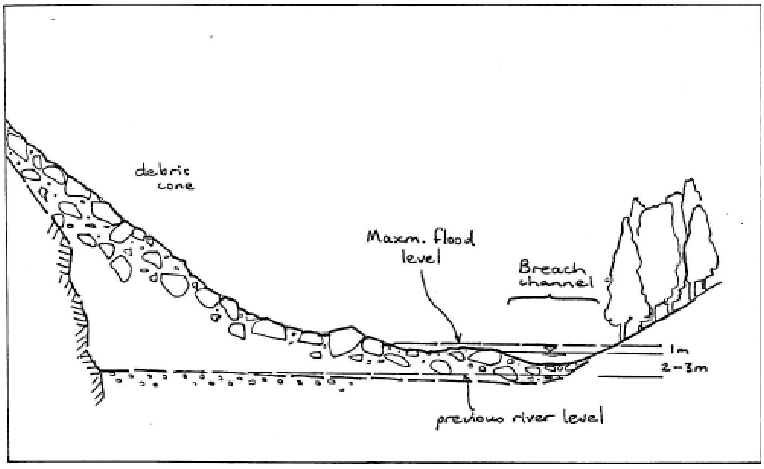
Report:

Date: 23/04/2011

Photo 1: Steep canyon walls just downstream of Moonlight Track rockslide with potential for first-time failure and river blockage.

Photo 2: True left bank just upstream of Moonlight Track rockslide also with potential for first-time failure. Note minor failures with debris cones protruding onto valley floor.

Photo 3: Criterion Creek rockslide showing dam with minimum crest level of around 3 m above valley floor.



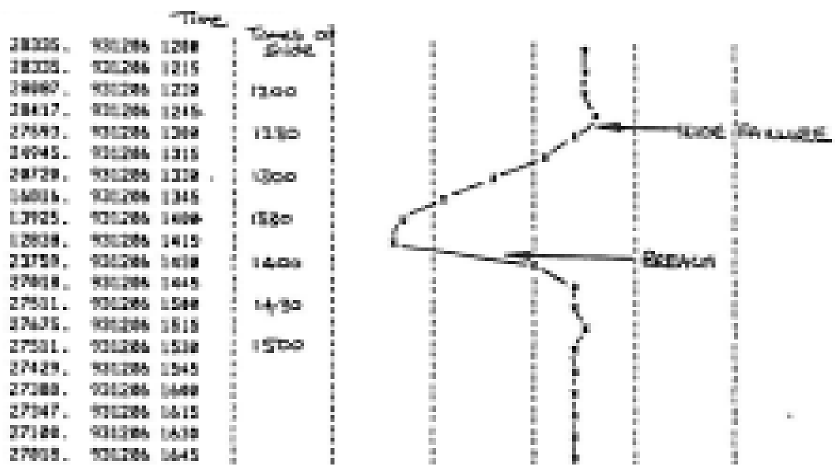


Photo 4: Bowens Peak hydrograph. Left hand column showing flow in litres/sec.

Photo 5: Existing landslide on true left bank showing actively collapsing head scarp . Failure surface lies parallel to eastward dipping foliation.

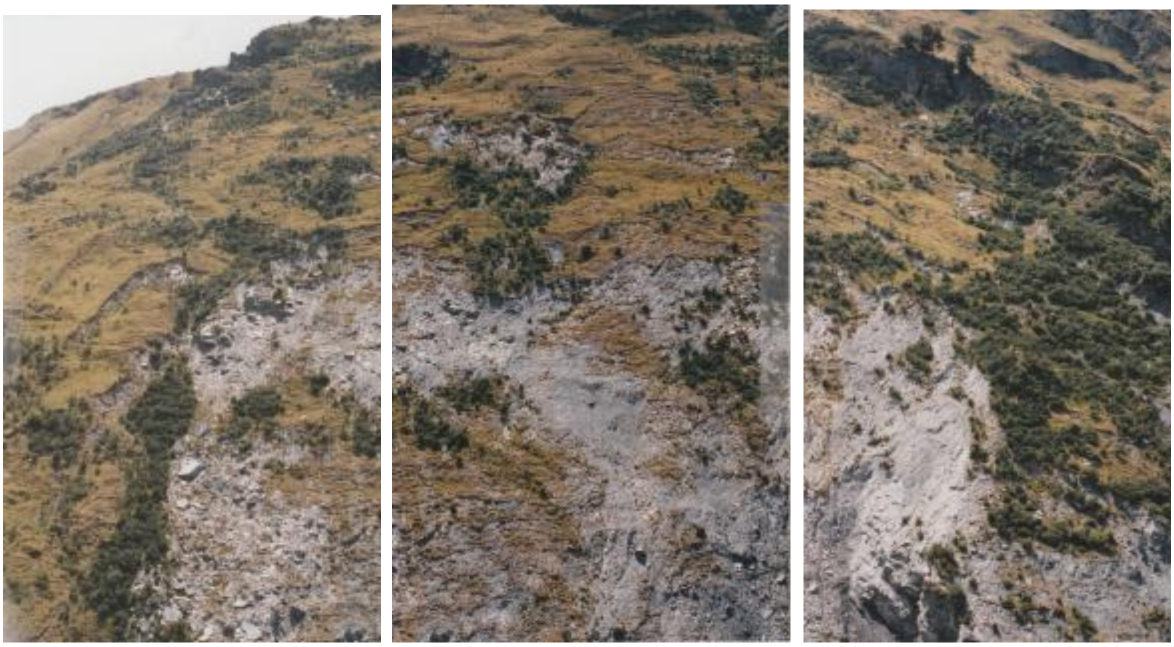


Photo 6: Existing landslide on true left bank showing numerous signs of recent activity.

Shotover River
Review of Flood Hydrology

Prepared for Shotover Country in Relation to QLDC Plan Change 41

1. Introduction

The Otago Regional Council queried the flood risk probability for the Shotover River used by the Shotover Country witness at the consent hearing for Plan Change 41.

Opus in a report for Contact Energy on the Probable Maximum Floods dated May 2000 used the November 1999 storm to check the validation of their PMF model. They believed that the Shotover recorded hydrograph was faulty and that it suffered a massive rating change due to changes in the river channel in that event. To get consistency with the model they considered that flows would have to be around 950 m^3/s or 2.6 times the recorded flow of 369 m^3/s. They also stated that the Shotover River water backflowed into the Kawarau Arm of Lake Wakatipu during the November 1999 event and reduced the Wakatipu outflow to zero on 17 Nov 1999. In fact there was an inflow to Lake Wakatipu of 130 m^3/s on that day. The 1999 flood is considered to be the largest since the 1878 flood. The Shotover Plan Change 41 used a flood flow figure of 1000 m^3/s for the 1999 flood and 1180 m^3/s for the current 1% annual exceedance probability (AEP) flood for the Shotover River.

There is an ability to cross check the reliability of flow data at the Shotover at Bowens Peak site by a simple use of the flow data at the Kawarau River at Chards Road minus the outflow from Lake Wakatipu subject to an allowance approximate travel times. Three percent of the total catchment at the Chards Road site is not covered by the Shotover or Lake Wakatipu outlet recorders but this lower catchment area is in a lower rainfall area and peak runoff would have passed before the peak from the other two sites.

This check on the reliability of the Shotover flood record needs to be carried out for more events than just the 1999 flood. This review has analysed 33 events over the 44 year record of the Shotover River at Bowens Peak. The events have been selected on the larger flows for the Chards Road site, the larger flows for the Shotover River site and to include most of the Shotover flows into Lake Wakatipu (negative outflow).

A revised flood frequency curve is provided.

2. Methodology

Water level and flow data was obtained from the ORC for the sites shown in Table 2.1. Acknowledgement is made that NIWA and Contact Energy have supplied much of this data to the ORC.

Site No.	Name	Period of record
75262	Kawarau River at Chards Road	1962 - 2010
75263	Kawarau River at Frankton	1963 - 2010
75276	Shotover River at Bowens Peak	1967 – 2010
75277	Lake Wakatipu at Willow Place	1962 – 2010
9134	Lake Wakatipu Outflow (Opus calculated)	1963-2004

Table 2.1

For this analysis only the Kawarau River at Chards Rd (75262), Lake Wakatipu Outflow (Opus calculated site 9134), and the Shotover River at Bowens Peak have been used.

The relative size of the catchments and distance between the water level recorder sites used is shown diagrammatically in Figure 2.1 attached.

Table 2.2 attached shows the list of maximum annual flow recorded at the three sites and the minimum flows at the Lake Wakatipu Outlet

For most events plots of the hydrographs for the three sites were produced and the derived Shotover flow calculated after allowing for about 2-3 hours travel time for the two upstream sites to Chards Road. This was then compared with the actual recorded flow at the Bowens Peak site. The greater of these flows was adopted for the flood series. These plots are available as an Appendix to this review report.

Barnett & MacMurray (2006) undertook a similar analysis for the 1994, 1995 and 1999 floods. Using their nomenclature they derived the Shotover flows from the relationship $S_t = C_{t+\delta t} - F_t$ where the lag time δt was estimated to be approximately 2.5 hours to Chards Road (C) with the same lag applying to both Frankton (F) and Shotover (S). They derived similar numbers for the three events as obtained through the more comprehensive series analysed for this report. They did however make other adjustments to the 1999 flood flow to accommodate errors from timing between the Frankton recorder and Chards Rd recorder that was not operating for some time at the peak and just after at Chards Road. They derived a peak flow of 1400 m^3/s for the Shotover at Bowens Peak and this higher figure has been adopted for the 1999 flood.

3. Results

Table 3.1 attached is a summary table that shows the current certified annual flood peak data for the Shotover River at Bowens Peak by year, the Shotover River peak flow as analysed for the highest flow in the year recorded at Chards Road, the Shotover River peak flow as analysed for the highest flow event recorded at the Bowens Peak site, the Barnett & MacMurray figures, and the adopted flood peak flow for the flood frequency analysis. In many instances the highest recorded flow on the Shotover and at Chards Road are in the same event but this is not always so. In some cases two events have been analysed for the same year to check that the highest Shotover flow is included.

As can be noted from the table the 1999 flood flow is now taken to be 1,400 m^3/s, or 3.8 times the recorded flow. Significant changes are also apparent for 1994, 1996 and 1998. These and the lesser changes have resulted in a revised mean annual flood of 500 m^3/s compared with the certified data mean annual flood of 434 m^3/s.

Table 3.1 also includes the flood frequency analysis based on the revised and adopted peak flood flows. The 1% AEP flood has been derived using two methods: the relationship from the Regional Flood Estimation method and a curve fitted to the data. The Regional Flood Estimation factor for the Shotover site for the 1% AEP event is 2.8 times the mean annual flood. This line with the 1% AEP as 1400 m^3/s, is shown on Figure 3.1 attached. This line and the following two lines have been overplotted on traditional Weibull and Gringorten plot points. A second line has been adopted as the flood frequency curve for this review and has been plotted with a line intermediate between two hydrological frequency analysis methods for plotting of return periods. This yields a 1% AEP event as 1500 m^3/s. This is adopted as the current figure based on the historic events.

For the purposes of incorporating an allowance for climate change the MfE guidance documents recommend an allowance of 8% increase in rainfall for every 1 degree Celsius rise in mean temperature. The Queenstown area is expected to have about a 2 degree rise in temperature by 2080. A third curve shown on the graph includes a 16% increase on the current frequency curve line. This shows a 1% AEP flood increase to 1,740 m^3/s by 2080. This flow is currently assessed in this review as an 0.45% AEP (1 in 220 year) event.

4. Conclusion

This review of the flood peak flow data for the Shotover River has proven to be valuable in reassessing the design flows for development along the Shotover River. Significant discrepancies have been found between recorded flood flows from the Shotover River at Bowens Peak recorder site and flows derived from analysis of the Kawarau River at Chards Road less the Lake Wakatipu Outlet flows. The latter two flows are considered to be more stable sites and the Shotover site to be subject to significant changes in river bed during higher flow events. The revised 1% AEP flood at 1,500 m^3/s is adopted.

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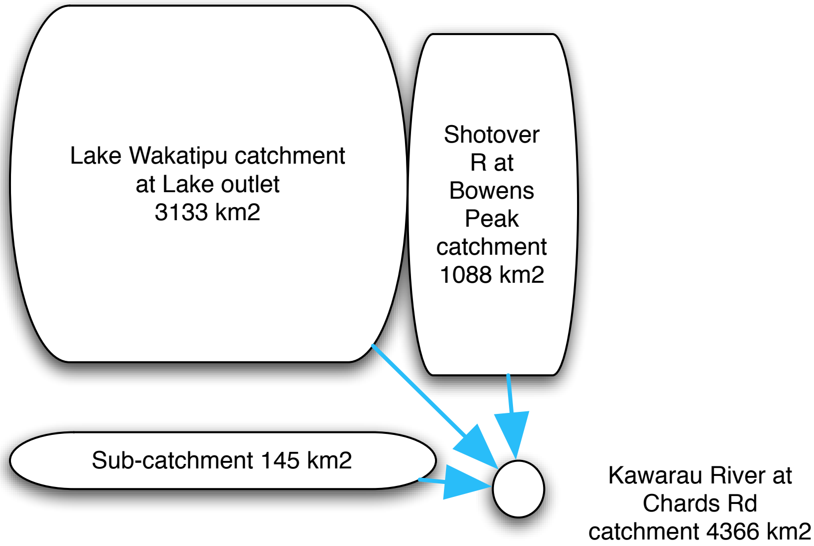
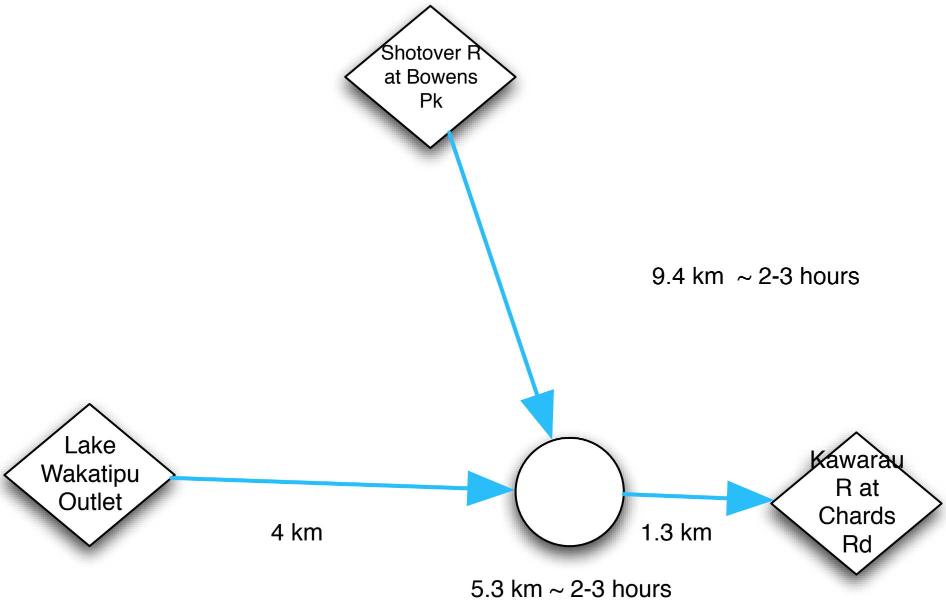


For flood flow purposes a check on the Shotover River flow can be made by subtracting the Lake Wakatipu outflow from the Kawarau R at Chards Road flow. The sub catchment area is about 3% of the catchment but is in a lower rainfall area and peak flows would pass before the other sites peaked



Distances between sites and approximate travel times

Figure 2.1: Diagram of relationships between recorder sites

Summary Table		P Extreme								
Maximums	Kawarau R at Chards Rd		Lake Wakatipu Outflow Opus		Lake Outflow Opus Minimum			Shotover R at Bowens Pk		
Year	Flow m3/s	Time	Flow Max	Time	Flow Minm	Time		Flow	Time	Travel time to Chards Rd hrs
1962	272.4	23/11/62 12:00								
1963	321.2	29/10/63 18:00	223.351	20/11/63 12:00	51.154	28/8/63 18:00				
1964	512.9	27/11/64 15:22	396.903	10/12/64 12:00	61.915	13/9/64 21:00				
1965	529.6	30/12/65 15:07	387.702	15/1/65 21:00	69.324	2-Sep-1965 12:00:00				
1966	444.1	24/1/66 10:07	358.368	3-Jan-1966 15:00:00	0	3-Aug-1966 00:00:00				
1967	757.2	14/12/67 04:00	539.962	28/4/67 09:00	49.644	24/1/67 00:00		555.834	14/12/67 02:26	1:33
1968	725	12/3/68 06:00	557.784	14/3/68 21:00	70.416	21/7/68 12:00		451.175	6-Mar-1968 05:41:45	
1969	623.6	25/12/69 04:00	474.249	27/12/69 15:00	85.646	26/6/69 09:00		639.255	24/12/69 20:48	7:11
1970	661	19/9/70 16:30	477.807	21/9/70 15:00	-59.349	28/8/70 18:00		368.992	28/8/70 13:17	
1971	435.3	28/10/71 05:54	327.891	8-Oct-1971 21:00:00	33.879	3-Jun-1971 06:00:00		328.076	11/9/71 14:08	
1972	559.3	14/11/72 01:39	377.527	17/11/72 00:00	76.272	1-Sep-1972 12:00:00		403.891	9-Sep-1972 13:41:13	
1973	492.2	6-Nov-1973 18:43:18	396.622	9-Nov-1973 21:00:00	72.268	28/8/73 15:00		278.88	3-Nov-1973 12:38:09	
1974	407.1	15/3/74 11:02	283.5	17/3/74 09:00	-82.73	15/3/74 12:00		506.307	15/3/74 08:00	3:02
1975	760.8	7-Apr-1975 17:35:12	509.677	9-Apr-1975 21:00:00	-178.356	31/3/75 03:00		600.406	31/3/75 02:00	
1976	376.6	5-Dec-1976 17:50:00	289.85	18/12/76 21:00	55.495	5-Dec-1976 18:00:00		464.346	5-Dec-1976 14:00:00	
1977	426.1	30/10/77 07:15	312.971	8-Nov-1977 21:00:00	-41.063	24/9/77 00:00		508.135	30/10/77 01:45	5:30
1978	761.8	14/10/78 15:28	430.557	17/10/78 21:00	-196.387	14/10/78 15:00		917.948	14/10/78 09:45	5:43
1979	704.9	3-Dec-1979 00:00:00	443.874	7-Dec-1979 18:00:00	-65.833	3-Dec-1979 00:00:00		478.7	3-Dec-1979 00:30:00	
1980	487.5	10/2/80 03:45	401.683	30/1/80 03:00	112.224	13/8/80 00:00		502.432	27/1/80 10:45	
1981	515	9-Mar-1981 20:15:00	337.645	12/3/81 06:00	74.909	2-Sep-1981 06:00:00		366.789	9-Mar-1981 16:30:00	
1982	702.6	26/11/82 05:53	511.517	29/11/82 15:00	80.281	31/7/82 03:00		413.908	20/5/82 13:30	
1983	884.7	13/1/83 12:15	682.558	15/1/83 21:00	-64.498	1-Aug-1983 00:00:00		612.489	13/1/83 07:57	4:17
1984	814.4	21/12/84 19:00	490.984	24/12/84 18:00	102.913	20/7/84 09:00		618.369	21/12/84 13:45	5:15
1985	628	12/1/85 20:00	449.659	8-Jan-1985 09:00:00	95.869	27/9/85 09:00		326.09	20/11/85 10:30	
1986	490.1	12/6/86 23:45	379.787	4-Jan-1986 21:00:00	90.028	26/9/86 06:00		262.643	1-Jan-1986 16:45:00	
1987	607.8	10/3/87 05:45	356.679	3-Apr-1987 03:00:00	-90.146	10/3/87 06:00		636.114	10/3/87 02:00	3:45
1988	692.6	28/10/88 15:15	546.072	1-Nov-1988 15:00:00	0	17/8/88 09:00		531.047	12/9/88 20:15	
1989	472.7	15/12/89 09:00	333.442	1-Jan-1989 03:00:00	-36.07	15/12/89 09:00		451.818	15/12/89 06:15	2:45
1990	479.4	13/5/90 21:15	372.79	17/12/90 06:00	73.26	29/9/90 00:00		292.786	13/5/90 19:45	1:30
1991	527	18/10/91 01:00	386.138	9-Jan-1991 15:00:00	51.024	9-Aug-1991 06:00:00		416.598	12/9/91 18:00	
1992	475.1	21/10/92 06:00	338.36	18/11/92 18:00	19.641	26/6/92 09:00		190.902	20/10/92 12:45	
1993	480	26/1/93 02:00	333.556	27/1/93 06:00	-140.351	5-Oct-1993 21:00:00		208.404	5-Oct-1993 17:45:00	
1994	829.2	9-Jan-1994 19:45:00	604.974	25/1/94 15:00	-207.21	9-Jan-1994 15:00:00		431.878	7-Nov-1994 08:30:00	
1995	860.5	13/12/95 21:00	582.093	30/12/95 03:00	40.229	2-Sep-1995 06:00:00		503.733	13/12/95 02:45	18:15
1996	671.6	7-Oct-1996 13:30:00	562.181	1-Jan-1996 00:00:00	-58.7	7-Oct-1996 15:00:00		317.229	7-Oct-1996 11:15:00	2:15
1997	524.2	23/12/97 19:00	391.118	30/12/97 00:00	64.731	10/8/97 21:00		149.882	23/12/97 13:45	5:15
1998	554.8	29/10/98 10:30	393.264	12/3/98 03:00	-45.275	23/7/98 00:00		246.865	8-Oct-1998 10:30:00	
1999	1197	18/11/99 12:00	907.823	19/11/99 18:00	-130.426	17/11/99 06:00		368.793	17/11/99 05:45	
2000	536.8	25/6/00 20:45	354.233	1-Jul-2000 15:00:00	91.477	25/6/00 21:00		214.671	25/6/00 14:15	6:30
2001	469	12/12/01 11:30	349.834	14/12/01 00:00	-66.967	19/11/01 18:00		193.574	19/11/01 17:45	
2002	615.2	19/9/02 09:30	373.406	24/9/02 18:00	-224.894	19/9/02 09:00		879.964	19/9/02 07:00	2:30
2003	481.8	6-Dec-2003 10:15:00	301.3	16/12/03 15:00	61.678	30/4/03 09:00		412.671	6-Dec-2003 07:45:00	2:30
2004	476	20/11/04 19:00	287.671	12/1/04 21:00	37.307	13/8/04 12:00		460.742	10/3/04 19:45	
2005	421.4	10/3/05 20:15						276.924	6-Mar-2005 19:45:00	
2006	624.7	30/11/06 05:30						525.161	30/11/06 01:45	3:45
2007	407.2	1-Nov-2007 03:45:00						429.382	11/8/07 19:00	
2008	466.1	24/9/08 05:30						375.729	24/9/08 02:45	2:45
2009	579.6	16/5/09 23:45						508.557	16/5/09 20:45	3:00
2010	474.6	3-Jan-2010 20:15:00						454.905	26/4/10 03:30	
				Colour coding	Blue	Chard & Shotover same event				
					Red	Shotover peak event				
					Green	Chard peak event				

Table 2.2

Summary Table		Shotover R at Bowens Pk				Flood Frequency Analysis			May-2011		
Maximums	A	B	C		D						
		Chard Pk Date	Shotover Pk Date			Sorted by Rank					
	Certified data	Shotover Peak Flood Estimated Flow using Chard Rd - Lake Outflow	Shotover Peak Estimated Flow using Chard Rd - Lake Outflow	Adopted Flood Series	Barnett McMurray 2006					Return Period T Yrs	Return Period T Yrs
Year	Annual Peak Flow m3/s	Flow m3/s	Flow m3/s	Flow m3/s		Year	Flood Flow m3/s	Rank	Weibull p	Weibull	Gringorten
1967	556	527		556		1999	1400	1	0.0222	45.00	78.79
1968	451		420	451		1994	1036	2	0.0444	22.50	28.28
1969	639	350		639		1978	958	3	0.0667	15.00	17.23
1970	369	236	340	369		2002	880	4	0.0889	11.25	12.39
1971	328		285	328		1975	788	5	0.1111	9.00	9.68
1972	404		270	404		1979	771	6	0.1333	7.50	7.94
1973	279			279		1996	731	7	0.1556	6.43	6.73
1974	506			506		1987	698	8	0.1778	5.63	5.84
1975	600	391	788	788		1969	639	9	0.2000	5.00	5.15
1976	464			464		1984	639	10	0.2222	4.50	4.62
1977	508	380		508		1995	620	11	0.2444	4.09	4.18
1978	918	958		958		1983	612	12	0.2667	3.75	3.82
1979	479	771		771		1967	556	13	0.2889	3.46	3.51
1980	502			502		1988	531	14	0.3111	3.21	3.25
1981	367			367		2006	525	15	0.3333	3.00	3.03
1982	414	353	365	414		1989	509	16	0.3556	2.81	2.84
1983	612	425		612		2009	509	17	0.3778	2.65	2.66
1984	618	639		639		1977	508	18	0.4000	2.50	2.51
1985	326		275	326		1974	506	19	0.4222	2.37	2.38
1986	263			263		1980	502	20	0.4444	2.25	2.26
1987	636	698		698		1976	464	21	0.4667	2.14	2.15
1988	531	223	515	531		2004	461	22	0.4889	2.05	2.05
1989	452	509		509		2010	455	23	0.5111	1.96	1.96
1990	293			293		1968	451	24	0.5333	1.88	1.87
1991	417	430		430		1991	430	25	0.5556	1.80	1.80
1992	191			191		2007	429	26	0.5778	1.73	1.73
1993	208	210		210		1982	414	27	0.6000	1.67	1.66
1994	432	1036	465	1036	1030	2003	413	28	0.6222	1.61	1.60
1995	504	590		620	620	1972	404	29	0.6444	1.55	1.54
1996	317	731		731		2008	376	30	0.6667	1.50	1.49
1997	150			150		1970	369	31	0.6889	1.45	1.44
1998	247	225	325	325		1981	367	32	0.7111	1.41	1.40
1999	369	1230		1400	1400	1971	328	33	0.7333	1.36	1.36
2000	215			215		1985	326	34	0.7556	1.32	1.31
2001	194	149		194		1998	325	35	0.7778	1.29	1.28
2002	880	840		880		1990	293	36	0.8000	1.25	1.24
2003	413			413		1973	279	37	0.8222	1.22	1.21
2004	461			461		2005	277	38	0.8444	1.18	1.17
2005	277			277		1986	263	39	0.8667	1.15	1.14
2006	525			525		2000	215	40	0.8889	1.13	1.12
2007	429			429		1993	210	41	0.9111	1.10	1.09
2008	376			376		2001	194	42	0.9333	1.07	1.06
2009	509			509		1992	191	43	0.9556	1.05	1.04
2010	455			455		1997	150	44	0.9778	1.02	1.01
Average	**434**			**500**							
1% AEP Regional Flood Multiplier				2.8							
1% AEP	1214			1400							
Climate Change Factor 16%											
1% incl Climate	1409			1624							

Table 3.1

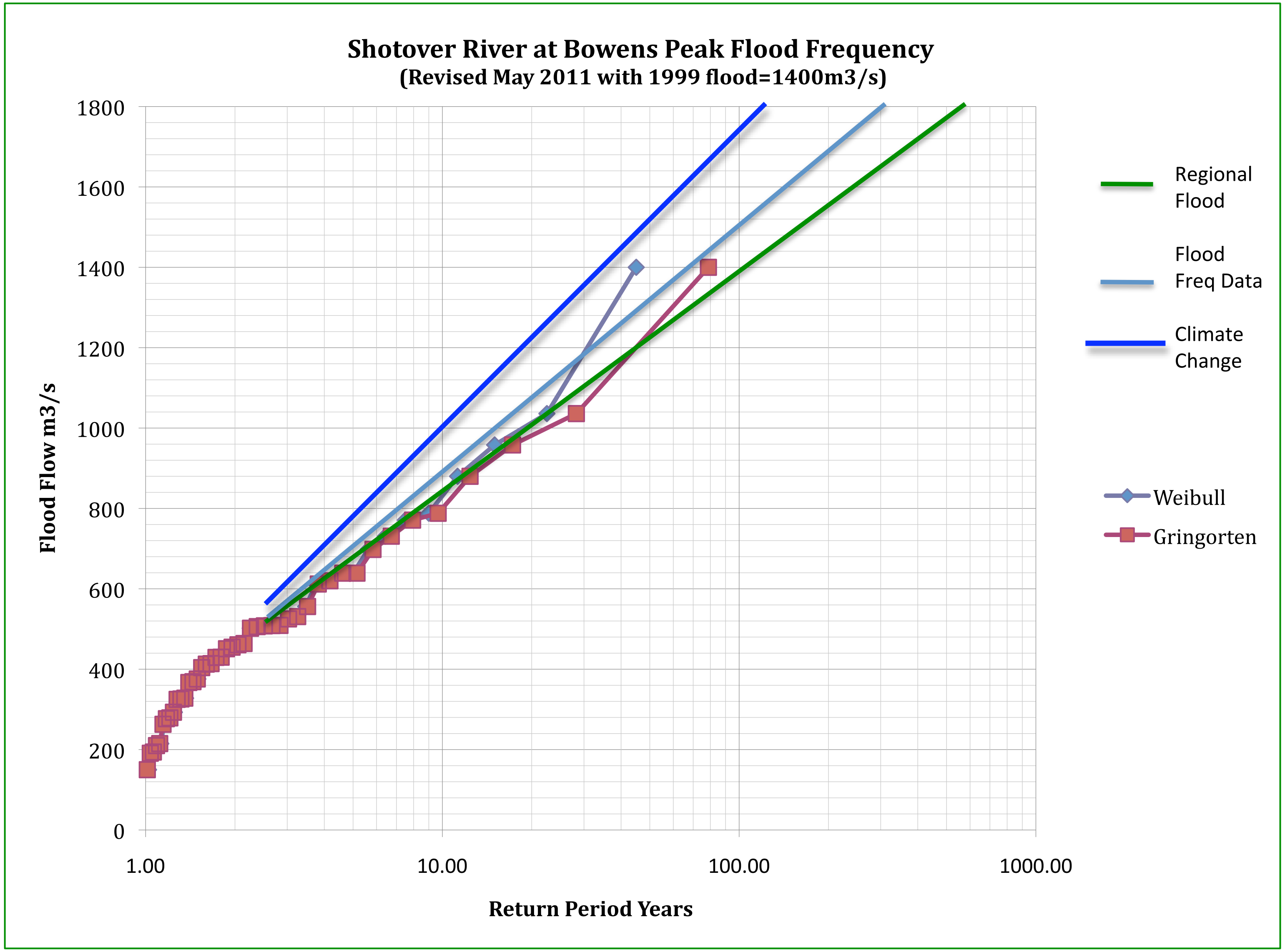

Shotover River at Bowens Peak Flood Frequency
(Revised May 2011 with 1999 flood=1400m3/s)
1800
1600
1400
1200
1000
800
600
400
200
0
Flood Flow m3/s
1.00
10.00
100.00
1000.00
Return Period Years
Regional Flood
Flood Freq Data
Climate Change
Weibull
Gringorten

Figure 3.1

Shotover River
Review of Flood Profile

Prepared for Shotover Country in Relation to QLDC Plan Change 41

1. Introduction

The Otago Regional Council queried the flood risk probability for the Shotover River used by the Shotover Country witness at the consent hearing for Plan Change 41. A review has been carried out of the flood hydrology and the outcome of that requires a review of the derived flood profiles as part of the expert caucusing sought by the Commissioners.

2. Flood hydraulic model

A software programme from the US Army Corps of Engineers HEC-RAS has been used for the purposes of estimating the flood profile for the Shotover River delta.

River cross sections were established by the MWD and surveyed a number of times since 1980. Such information is useful for estimating trends in river bed levels and can also be used for hydraulic calculations.

The large flood in November 1999 had actual peak water levels recorded at 4 sites and enables the calibration of a hydraulic model. After the 1999 flood Opus had estimated the actual peak Shotover flow as about 1000m^3/s. This figure was used in the initial calibration of the model and a Mannings n roughness coefficient of 0.025 gave a reasonable fit for a simple mean bed level model that only used the channel width clear of major vegetation and river bed levels from the 2001 survey. The outcome of the review of the hydrology has adopted the Barnett & MacMurray figure of 1400 m^3/s as the 1999 flood flow. Calibrating the same model using that flow lowers the roughness factor to n=0.020 to provide a reasonable fit.

The normal approach when using a calibrated model to predict the water level for different sized flood events is to use the roughness factor from the calibration. If a higher value of roughness is used then this is building in conservatism in the predicted flood level. This is additional to the provision of freeboard that is normally used to accommodate uncertainties and effects of local obstructions, etc.

The Otago Regional Council have sought the use of a Mannings n of 0.025 for the purposes of the flood modelling.

Three design runs have been done using the same approach as for the Hamilton report presented at the Plan Change 41 hearing but using the climate change 1% AEP flood flow as 1,740 m^3/s and the Mannings n roughness factor of 0.025. These runs are:

(a) Simplified mean bed level realistic scenario for Year 2060. This model assumes no ORC works and a continuing aggradation in the lower delta below MWD XS4. Although the trend is for degradation or lowering of the bed in the upper delta the model assumes the 2001 mean bed levels for MWD XS 1-4.

(b) Using the 2010 surveyed cross sections of the delta without any ORC works

(c) Using the 2010 surveyed cross sections and proposed training bank but without gravel removal from the delta.

This information was presented in the report dated 1 June 2011.

The ORC then considered that the Mannings n roughness characteristics may be on the low side and sought a run of the AULOS model as used by MacMurray but for the higher flood flow of 1740 m^3/s. Those results are now available and are presented here relative to the design levels in the 1 June 2011 report. The AULOS model uses the 2008 Lidar survey and the delta surface is described by 25 cross sections. Based on aerial photographs the delta surface was divided into vegetated and bare gravel areas. Mannings n for the bare gravel areas in the model was 0.03, and for the vegetated areas 0.06 was used. The downstream boundary condition used was constant water level of RL 313.0m. The model results are presented below for comparative purposes.

3. Model Results

The calculated water surface profile for a flow of 1740 m^3/s for the three modelled scenarios are shown in Table 3.1 Columns 3-5. The same downstream start level of RL 312.9m is used in all three cases. The realistic 2060 profile used for the setting of the platform fill level estimated water levels are all higher than those calculated using the current 2010 cross-sections with or without the training bank.

The MacMurray AULOS model results (8 June 2011) are shown in Column 6 of Table 3.1 and the difference between the realistic MBL model results and the MacMurray model are shown in the last column with the Realistic model producing a higher flood profile.

Shotover River Flood Profiles						
MWD XS	Distance US from Kawarau m	2010 Cross Sections flow 1740m3/s n=0.025	2010 Cross Sections flow 1740 m3/s n=0.025 w Training Bank	Realistic 2060 Flood profile MBL Model 1740 m3/s n=0.025	MacMurray AULOS Model 2008 Lidar XS 1740 m3/s n=0.03/0/06	Realistic 2060 Model - MacMurray AULOS Model
7	95	312.90	312.9	312.9	313	
6	336	312.91	312.93	313		
5	634	313.07	313.1	313.53	313.08	0.45
4	927	313.62	313.67	314.01	313.66	0.35
3	1235	314.27	314.28	314.58	314.49	0.09
2	1543	315.34	315.33	315.88	315.46	0.42
1	1842	316.42	316.43	316.66		

Table 3.1: Hec-Ras model calculated flood profiles

4. Impact on Design Fill Platform Levels

The recommended minimum freeboard in the Hamilton Plan Change 41 report was 0.8 m. Applying that freeboard to the calculated realistic 2060 flood profile in Table 3.1 requires minor adjustment in levels only. A freeboard of just over a metre has been used for XS3 as used originally. Table 4.1 shows the original design platform levels and those now proposed.

MWD XS	Distance US from Kawarau m	Realistic 2060 Flood profile 1740 m3/s n=0.025 RLm	Original Recommended Minimum Platform Level RLm	**Now recommended Platform Level RLm**	Increase in platform level compared with application m
7	95	312.9			
6	336	313	313.7	**313.8**	0.1
5	634	313.53	314.3	**314.33**	0.03
4	927	314.01	314.8	**314.81**	0.01
3	1235	314.58	315.6	**315.6**	0

Table 4.1: Recommended Minimum Platform Level

The mean bed level estimated for Year 2060, the design flood profile for the 1% AEP with climate change, and the recommended minimum platform level are all plotted in Figure 4.1.

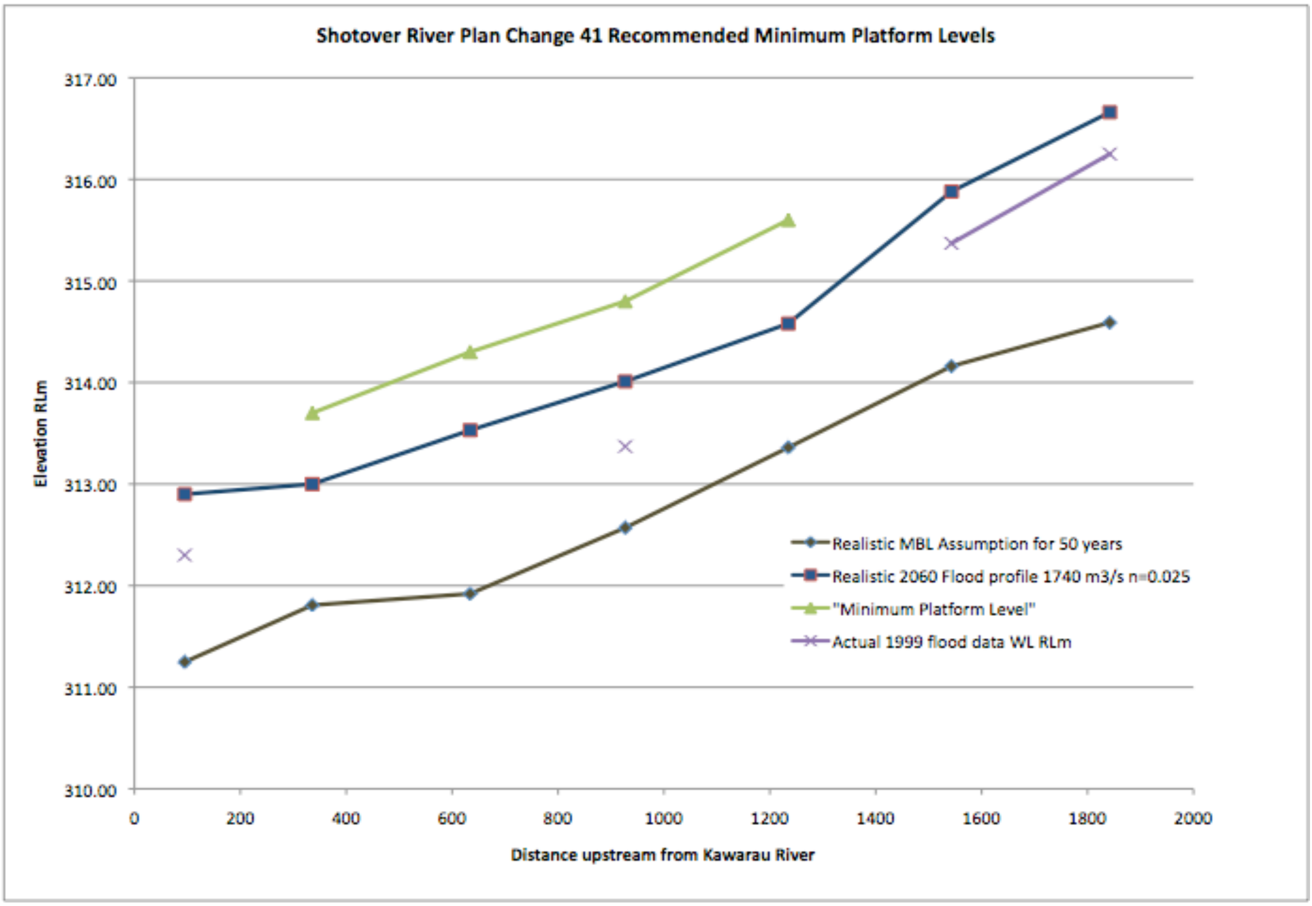


Figure 4.1: Plot showing proposed minimum platform levels compared with MBL and design flood profile

An A3 plan is attached showing the design minimum platform level, existing ground level, design flood profile (Realistic 2060 flow 1740 m^3/s), and the MacMurray worst case modelled scenario for a flow of 1400 m^3/s.

END

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RD 1
Alexandra 9391
New Zealand

Phone: 64 3 426 2850
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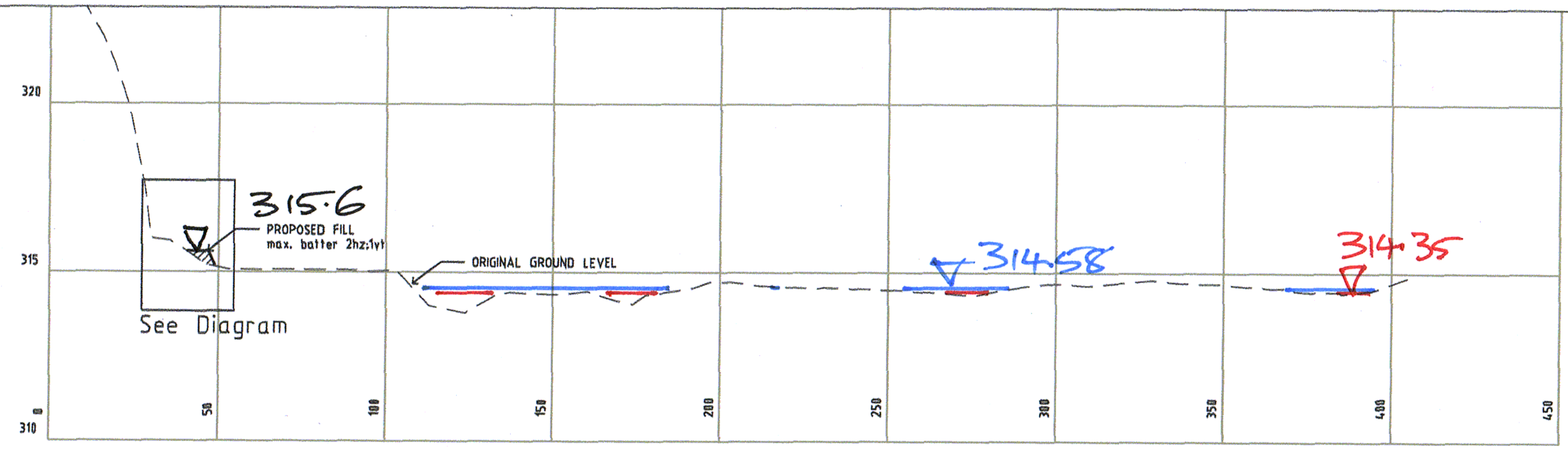

SECTION 3 - 3'

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A3 SCALE: 1:2000hz 1:200 vt
10 x VERTICAL EXAGGERATION

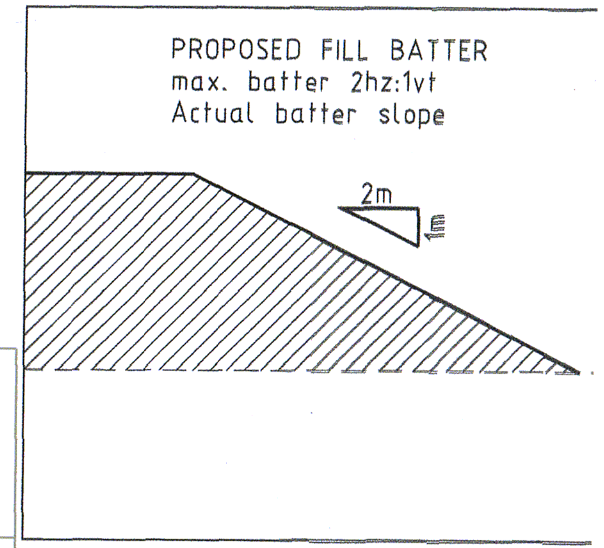
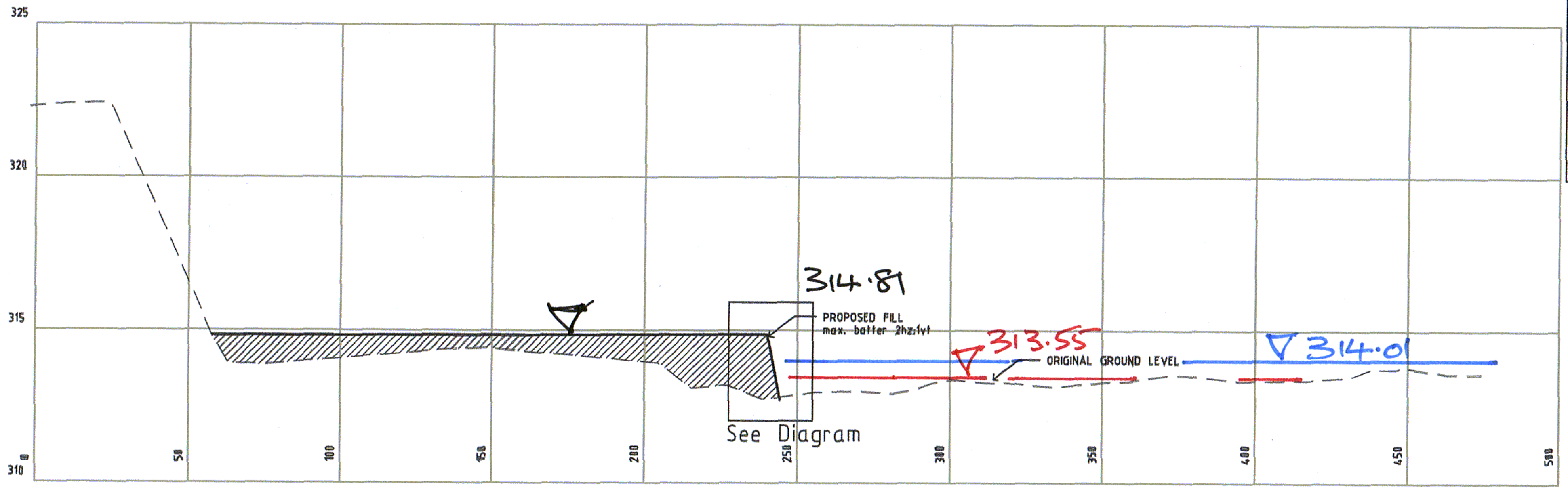


DIAGRAM - n.t.s



SECTION 4 - 4'

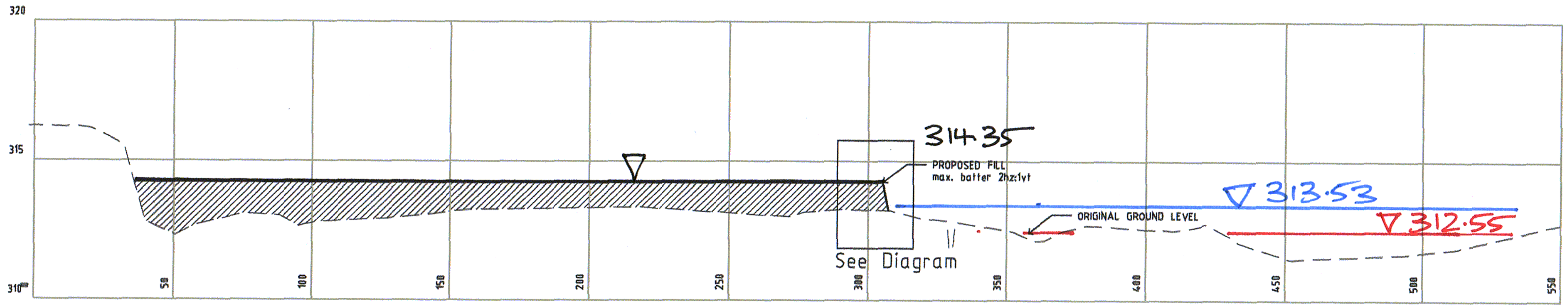
A1 SCALE: 1:1000hz 1:100 vt
A3 SCALE: 1:2000hz 1:200 vt
10 x VERTICAL EXAGGERATION

∇ Proposed Fill Level

Flow Profiles

— Realistic 2060 1740 m3/s

— MacMurray Worst Case 1400 m3/s



SECTION 5 - 5'

A1 SCALE: 1:1000hz 1:100 vt
A3 SCALE: 1:2000hz 1:200 vt
10 x VERTICAL EXAGGERATION

ISSUED 11.02.2011

ADDITIONS David Hamilton 29/5/11

Shotover Design Limited trading as
Clark Fortune McDonald & Associates
Licensed Cadastral Surveyors - Land Development - Planning Consultants
309 Lower Shotover Road, P.O.Box 553 Queenstown

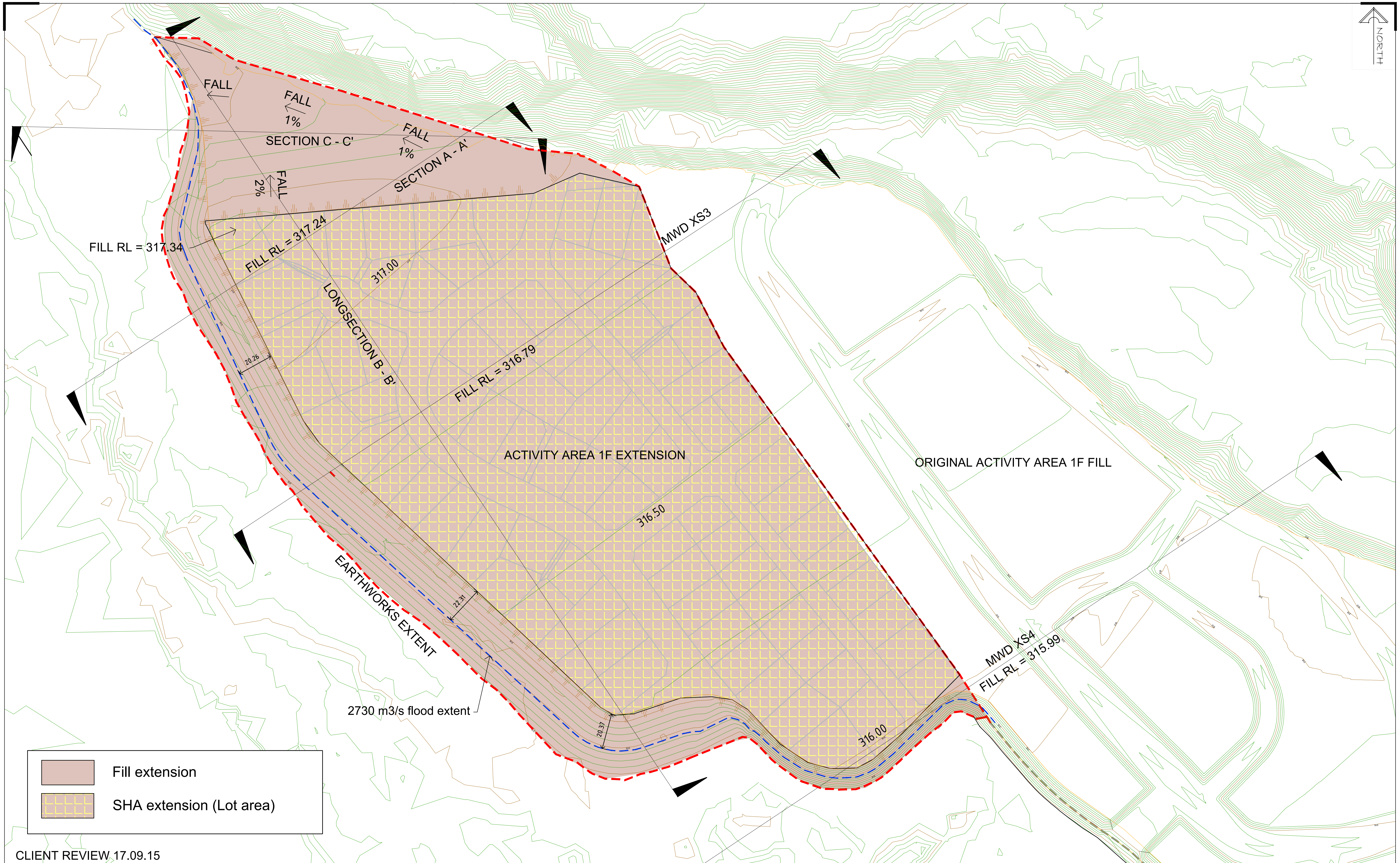
Rev.	Date	Revision Details	By
A	11.02.2011	ADD MWD SECTION LINES 3 - 5	TS

SHOTOVER COUNTRY
PROPOSED FILL

Client: QUEENSTOWN PROPERTY INFORMATION SERVICES

Notes:
All dimensions shown are in meters unless shown otherwise.

Surveyed	Signed	Date	Job No
			1
Drawn	Signed	Date	Scale
TS		02.11	



CLIENT REVIEW 17.09.15

Shotover Design Limited trading as
Clark Fortune McDonald & Associates
Licensed Cadastral Surveyors - Land Development - Planning Consultants
309 Lower Shotover Road, P.O.Box 553 Queenstown
Tel. (03)441-6044, Fax (03)442-1066, Email admin@cfma.co.nz

Shop 2, Otago House, 475 Moray Place, P.O. Box 5960
Tel. (03)470-1582, Fax (03)470-1583, Email admin@cfma.co.nz

Rev.	Date	Revision Details	By
A	15.09.15	Change batter to 1 in 8 to provide buffer	RB

SPECIAL HOUSING AREA FILL EXTENSION

Client: SHOTOVER COUNTRY LTD

Notes:
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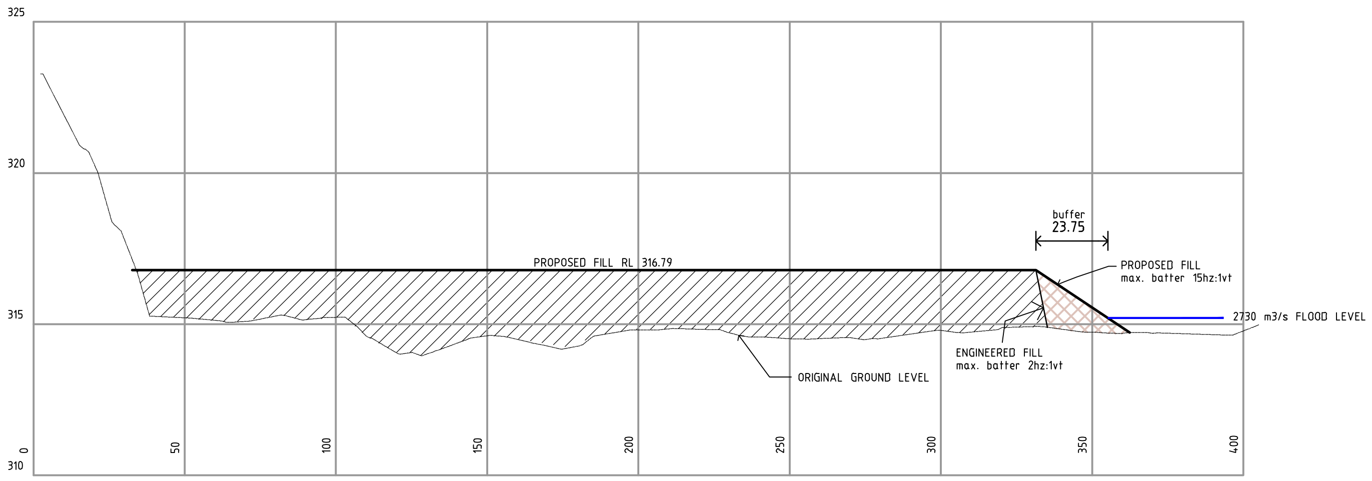
Any person using Clark Fortune McDonald drawings and other data accepts the risk of:
- Using the drawings and other data in electronic form without requesting and checking them for accuracy against the original hard copy versions.
- Ensuring the information is the most recent issue.

Surveyed	Signed	Date	Job No.	Drawing No.
-		-	11494	11 Sheet 001
Drawn	**Signed**	**Date**	**Scale**	
RWB		20.08.14	1:1000 @ A1 1:2000 @ A3	
Designed	**Signed**	**Date**	**Datum & Level**	**Rev.**
-		-	Mt Nic 2000 & MSL	A

S:\JOBS\11400\11494\acad\11494_11_SHA Fill concept_RevA.dwg Plotted: 17.09.2015



MWD SECTION 3 - 3'

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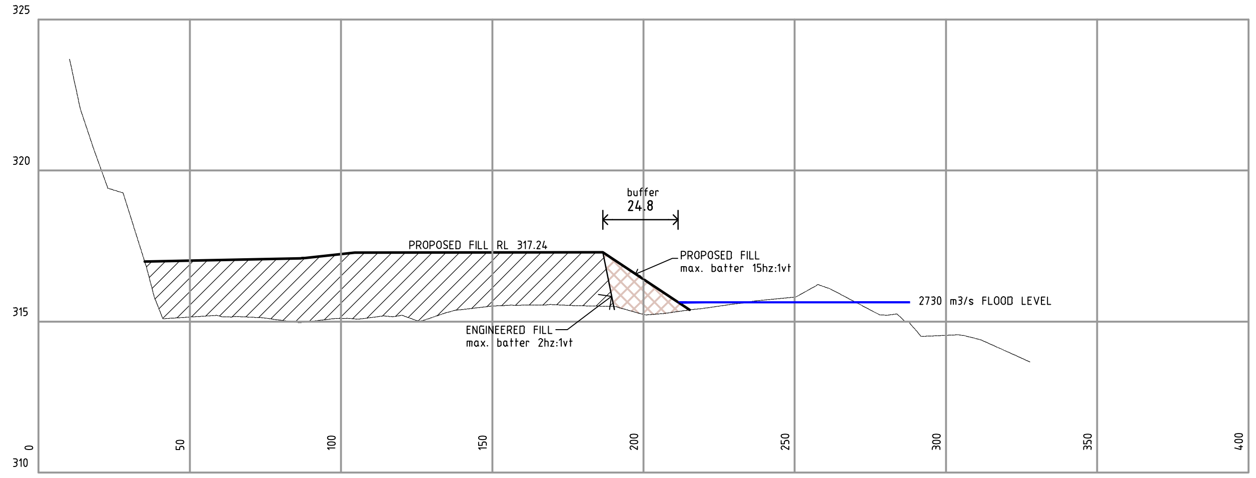
10 x VERTICAL EXAGGERATION



SECTION A - A'

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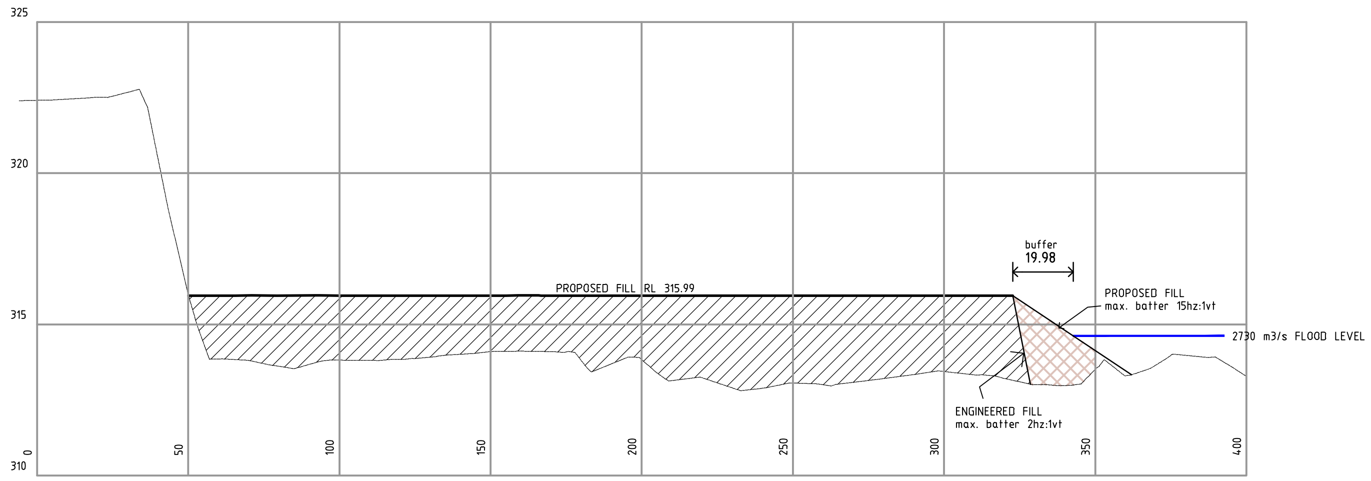
10 x VERTICAL EXAGGERATION



MWD SECTION 4 - 4'

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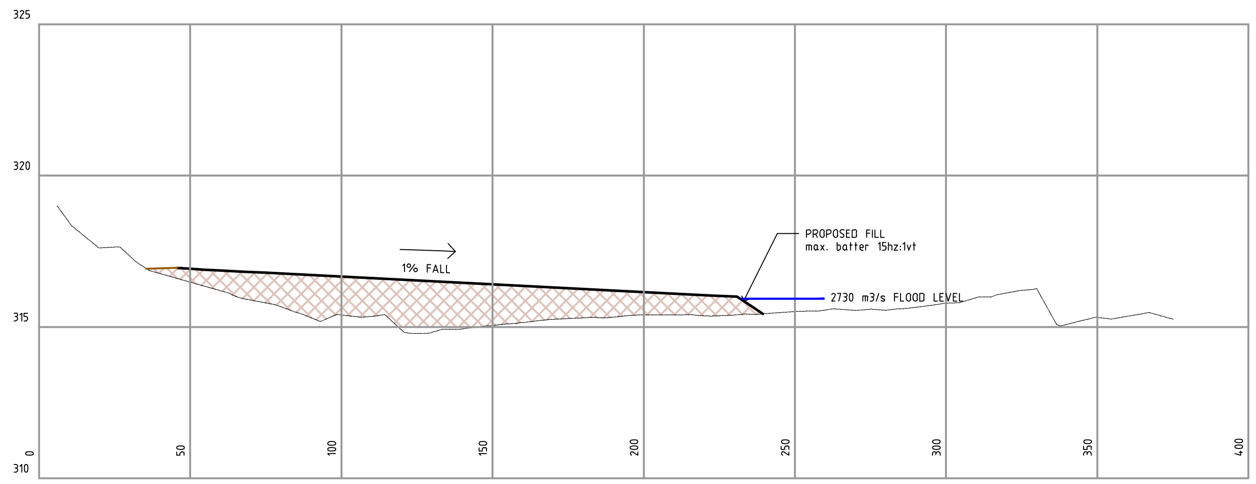
10 x VERTICAL EXAGGERATION



SECTION C - C'

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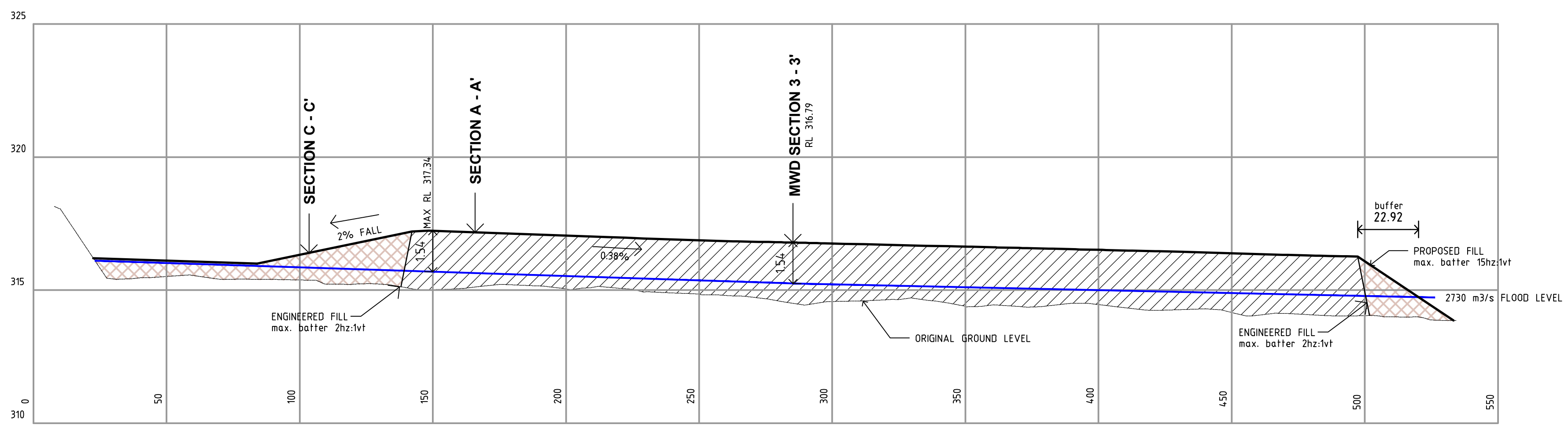
A3 SCALE: 1:2000hz 1:200 vt
10 x VERTICAL EXAGGERATION



LONGSECTION B - B'

A1 SCALE: 1:1000hz 1:100 vt
A3 SCALE: 1:2000hz 1:200 vt
10 x VERTICAL EXAGGERATION

ENGINEERED FILL

BUFFER AREA FILL

CLIENT REVIEW 17.09.15

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Rev.	Date	Revision Details	By
A	15.09.15	Change batter to 1 in 8 to provide buffer	RB

SPECIAL HOUSING AREA
FILL EXTENSION - SECTIONS

Client	Surveyed	Signed	Date	Job No.	Drawing No.
SHOTOVER COUNTRY LTD	-		-	11494	11 Sheet 002
	Drawn	Signed	Date	Scale	
	RWB		20.08.15	1:1250 @ A1 1:2500 @ A3	
	Designed	Signed	Date	Datum & Level	Rev.
	-		-	Mt Nic 2000 & MSL	A