NIWA Project: WPL13501

22 May 2014

Westpower Ltd
PO Box 375
Greymouth

Attention: Sue Cotton

Dear Sue

Effects of proposed Waitaha HEP scheme on bank stability at powerhouse

This letter responds to your request of 8 April 2014 for my opinion on how a stopbank protecting the proposed Waitaha HEP scheme powerhouse might affect erosion of the opposite bank. My response is as follows.

The problem

The scheme proposal involves building a powerhouse on the short span of low, tussock-and-scrub-covered terrace on the Waitaha right bank, immediately upstream from Alpha Creek (Figure 1). The diversion tunnel will exit at the upstream end of this terrace, with the water routed along a short span of pipe into the powerhouse. Since the terrace is relatively low above the river (5-6 m), it will be necessary to build a stopbank to protect the canal and powerhouse from floods. This stopbank will reduce the conveyance area for floods large enough to spill over the terrace, and there is the potential that this could force more of the flow along the left bank, causing it to erode. The question is: how much erosion could be caused by the stopbank?

Information sources

My analysis is based on the following information:

- Aerial and ground photographs of the proposed powerhouse site (left bank immediately upstream from Alpha Creek).
- A contour map of the low, left bank terrace that the powerhouse location, including reduced levels (RLs) of trash left by the December 2013 flood.
- Plots of three cross-sections spaced along this terrace (as located approximately on Figure 1).
- Visualisations of the powerhouse superimposed on existing imagery.
- An estimate of the peak flow at the powerhouse location for the 2013 flood event provided by Martin Doyle.

National Institute of Water & Atmospheric Research Ltd, 10 Kyle Street, Riccarton, Christchurch 8011
PO Box 8602, Riccarton, Christchurch 8440, New Zealand. Phone +64-3-348-8987, Fax +64-3-348-5548

Figure 1: The proposed powerhouse near Alpha Creek. Flow is bottom to top. Taken from Boffa Miskell Plan PH1.

Figure 2: View downstream of the Waitaha channel beside the Alpha Creek terrace (on right).

Figure 3: The Waitaha left bank opposite the powerhouse site.

Existing situation

Currently, the Waitaha River runs beside the terrace in a straight, boulder-bed channel (Figure 2). Boulder size has not been measured, but my visual estimate of the median size is about 600-700 mm (say 650 mm). The right bank terrace has a shallow flood-overflow channel running beside the slope toe and draining into Alpha Creek. The left bank is cut into a higher (10 m high) forested terrace formed of material that ranges in size grade from sand through boulders (Figure 3). The lower bank to the level of the right bank terrace is an armoured beach formed of boulders. The upper bank is near-vertical, and shows signs of recent erosion. The beach appears to have been formed as a lag as the bank has retreated. The Waitaha flow is deflected off a bedrock outcrop on the right bank at the apex of the bend immediately upstream of the Alpha Creek terrace. So the left bank opposite the powerhouse site is already prone to erosion, but its rate of erosion appears limited by natural armouring.

Analysis approach

My analysis involved applying a NIWA in-house hydraulic model that predicts water levels and calculates the lateral distribution of flow velocity. It assumes normal flow conditions in a streamwise sense, and uses the lateral transfer of momentum and an iterative solver to determine local shear stress on the channel bed and the depth-averaged velocity. The input data required is cross-section topography, energy slope, a Manning's n roughness coefficient, and a horizontal eddy viscosity coefficient. Calculations were performed for Cross-sections 2 (at the powerhouse) and 3 (midway along the stopbank). The key parameters are listed in Table 1.

The energy slope was estimated off both the surveyed flood trash RLs and the low-flow waters-edge RL's at the time of the cross-section surveys. Preliminary estimates of Manning's n were selected from Hicks and Mason (1998). The horizontal eddy viscosity was assumed to be 0.01 kg/m/s, based on detailed studies in other rivers. Calibration was achieved by adjusting the Manning's n until the modelled water level at a flow of 1400 m^3/s matched the water level indicated by the flood trash-line profile at the cross-section.

The slope estimates agreed reasonably well at Cross-section 3 (0.073 from trash-line; 0.075 from low-flow waters-edge), but diverged at Cross-section 2 (0.154 and 0.105, respectively). Accordingly, the calculations at Cross-section 2 were repeated for both slopes as a sensitivity exercise.

The modelled discharge of 1400 m^3/s was the peak flow during the December 2013 flood estimated at the powerhouse site by Martin Doyle, based on the discharge record at the top end of Kiwi Flat (with due allowance for tributary inputs, tributary lags, and peak attenuation). Matching this to the flood-frequency table in Doyle (2013) for the downstream end of Kiwi Flat (assuming the same peak flows there as at the powerhouse site), then the event return period is ~ 100 years. By contrast, the mean annual flood peak estimate was 812 m^3/s. I also modelled this mean annual flood discharge at the powerhouse site to establish whether or not relatively common floods would also be affected by the stopbank.

I located the riverside edge of the stopbank and the powerhouse wall by scaling off the photomap shown in Figure 1. I assumed the top of both had an RL of 132.5 m. I assumed the stopbank had a side-batter of 1:2.

After calibration, the model was run for the cases without and with the stopbank, to determine the increase in water level and boundary shear stress against the left bank due to the stopbank. Both the local shear stress at the left bank toe and the average stress over the bank were calculated.

The extent of erosion of the left bank due to the stopbank was estimated in two ways. First, the left bank of the cross-section was shifted laterally (i.e., eroded) until the water level fell back to the level without the wall. Second, the left bank was shifted until the shear stress at the left bank toe fell back to that without the stopbank.

The shear stresses at the left bank toe and averaged over the left bank were also compared to the threshold-of-motion stresses at those locations. The threshold stresses were estimated using the formula from Seminara et al. (2002), which adjusts the dimensionless threshold stress at low gradients (τ^*_{co}) for a steep side-slope angle (α):

$$\tau^*_{cs} = \tau^*_{co} \cos\alpha \,(1- \tan^2\alpha/\mu_c^2)^{0.5}$$

where μ_c is the tangent of the repose angle (assumed to be 0.7) and τ^*_{co} was taken as 0.06. The threshold stress (τ_{cs}) equals τ^*_{cs} x 16500 D_{50}, where D_{50} is the median boulder size (assumed to be 0.65 m).   The mobility is defined as the ratio of the actual shear stress to the threshold shear stress; a value exceeding 1 indicates mobility.

Table 1: Key parameters. Two energy slopes and matching n values were assessed at Cross-section 2.

	Cross-section 2	**Cross-section 3**
Channel energy slope	0.0105 (0.0154)	0.0075
Manning's n	0.051 (0.062)	0.046
Horizontal eddy viscosity (kg/m/s)	0.01	0.01
Water level at 1400 m^3/s (m)	130.07	131.46
D_{50} (m)	0.65	0.65
Left bank slope	0.45	0.55
D_{50} (m)	0.65	0.65
Threshold stress at left bank toe (N/m^2)	644	644
Threshold stress at mid left bank (N/m^2)	449	349

Results

The results of the hydraulic analysis are listed in Tables 2 and 3.

The effects of the stop bank are small. At Cross-section 3 midway along the stopbank (Table 2), the stopbank raises the water level by 8 cm and increases the shear stress at the left bank by 1-2%. Allowing the left bank to retreat by 1 m removes these effects. Comparing the actual to threshold shear stress indicates that the bank material should be incipiently mobile over the left bank at 1400 m^3/s in all cases (although probably not at the bank toe) - which means that the river just has the capability to erode it's banks at this flow, and explains the bank erosion observed after the December 2013 event.

Table 2. Results of hydraulic modelling at 1400 m^3/s at Cross-section 3. A mobility ≥1 indicates the bank material should move.

	No stopbank	**With stopbank**	**After 1 m left bank erosion**
Channel energy slope	0.0075	0.0075	0.0075
Manning's n	0.046	0.046	0.046
Water level (m)	131.47	131.55	131.45
Main channel mean velocity (m/s)	5.72	5.77	5.74
% flow in main channel	97.8	100	100
Maximum Froude No.	1.04	1.05	1.08
Shear stress at left bank toe (N/m^2)	506	512	501
Average shear stress over left bank (N/m^2)	340	346	339
Mobility at left bank toe	0.79	0.80	0.78
Mobility over left bank	0.97	0.99	0.97

Very similar results were found for Cross-section 2 at the powerhouse (Table 3). The increase in water level with the stopbank would be 17 cm. This would be removed by allowing the left bank to retreat by 1.9 m. A retreat of 3.3 m would restore the average left bank shear stress to the no stopbank value. These figures are not changed significantly by the choice made for the energy slope. Although the higher energy slope does render the left bank more mobile at 1400 m^3/s, the relative effects are unchanged.

Table 3. Results of hydraulic modelling at 1400 m^3/s at Cross-section 2. A mobility ≥1 indicates the bank material should move. Results in brackets are for an energy slope of 0.0154. Un-bracketed results are for a slope of 0.0105.

	No stopbank	**With stopbank**	**After 1.9 m left bank erosion**	**After 3.3 m left bank erosion**
Channel energy slope	0.0105 (0.0154)	0.0105 (0.0154)	0.0105 (0.0154)	0.0105 (0.0154)
Manning's n	0.051 (0.062)	0.051 (0.062)	0.051 (0.062)	0.051 (0.062)
Water level (m)	130.06 (130.07)	130.23 (130.24)	130.06 (130.07)	129.94 (129.95)
Main channel mean velocity (m/s)	5.83 (5.83)	5.94 (5.93)	5.88 (5.86)	5.84 (5.82)
% flow in main channel	94 (94)	100 (100)	100 (100)	100 (100)
Maximum Froude No.	0.94 (0.94)	0.95 (0.95)	0.97 (0.97)	0.98 (0.98)
Shear stress at left bank toe (N/m^2)	483 (755)	535 (790)	518 (762)	506 (744)
Average shear stress over left bank (N/m^2)	429 (630)	447 (662)	436 (642)	428 (630)
Mobility at left bank toe	0.75 (1.17)	0.83 (1.23)	0.78 (1.18)	0.76 (1.16)
Mobility over left bank	0.95 (1.40)	1.00 (1.47)	0.97 (1.43)	0.95 (1.40)

A further model run at a flow of 812 m^3/s, which is Doyle's (2013) estimate of the mean annual flood at the lower end of Kiwi Flat, showed that this flow would just be contained by the Waitaha channel and would not spill onto the Alpha Flat terrace. Thus the hydraulic effects of the stopbank would only occur with floods with return periods exceeding ~ 2.3 years.

Conclusion

My conclusion is that even with a flow of 1400 m^3/s, which is something like a 100-year return period event, the stopbank will induce only a slight reduction in the Waitaha channel conveyance and consequently cause only slight increases in water level and shear stress against the left bank. Such an event just has the competence to mobilise the boulders lining the left bank, and so some bank erosion is possible during an event of this size. However, retreat of only 1-3 m would be all that would be required to recover the hydraulic conditions without the stopbank. Thus I consider the effect of the stopbank on bank erosion would be minor.

References

Doyle, M. 2013. The hydrology of the Waitaha catchment. Consulting report for Electronet Ltd, September 2013.

Hicks, D. M.; Mason, P. 1998. Roughness characteristics of New Zealand rivers. Water Resources Publications.

Seminara, G.; Solari, L.; Parker, G., 2002. Bedload at low Shields stress on arbitrarily sloping beds: failure of the Bagnold hypothesis. *Water Resources Research*, 38(11): doi:10.1029/2001WR000681.

Yours sincerely

D M Hicks
Principal Scientist
Sediment Processes
NIWA, Christchurch

+64-3-343 7872 (022 600 4831)
m.hicks@niwa.co.nz

Peer reviewed by

R Measures
Hydraulic Engineer
Sediment Processes
NIWA, Christchurch