Technical Note

Horritt Consulting

Helebridge weir options modelling			
Project	Strategic Exe Weirs	**Date**	1 November 2023
Author	Matt Horritt	**Reference**	WRTHEL23/TN1 v2

v2 updated in response to Westcountry Rivers Trust comments

1. Introduction

Horritt Consulting have been commissioned by Westcountry Rivers Trust to investigate through hydraulic modelling options for improving fish passage at Helebridge on the River Exe (site shown in Figure 1). Currently, there is a weir, breached for approximately 25% of its width on the true right of the channel, operating as a partial blockage to flow (Figure 2). A masonry arch bridge carries the B3222 over the river 130m upstream of the weir. Within the bridge arches, the bed is formed from a concrete apron, and there is a significant pool on the downstream face of the bridge (Figure 3).

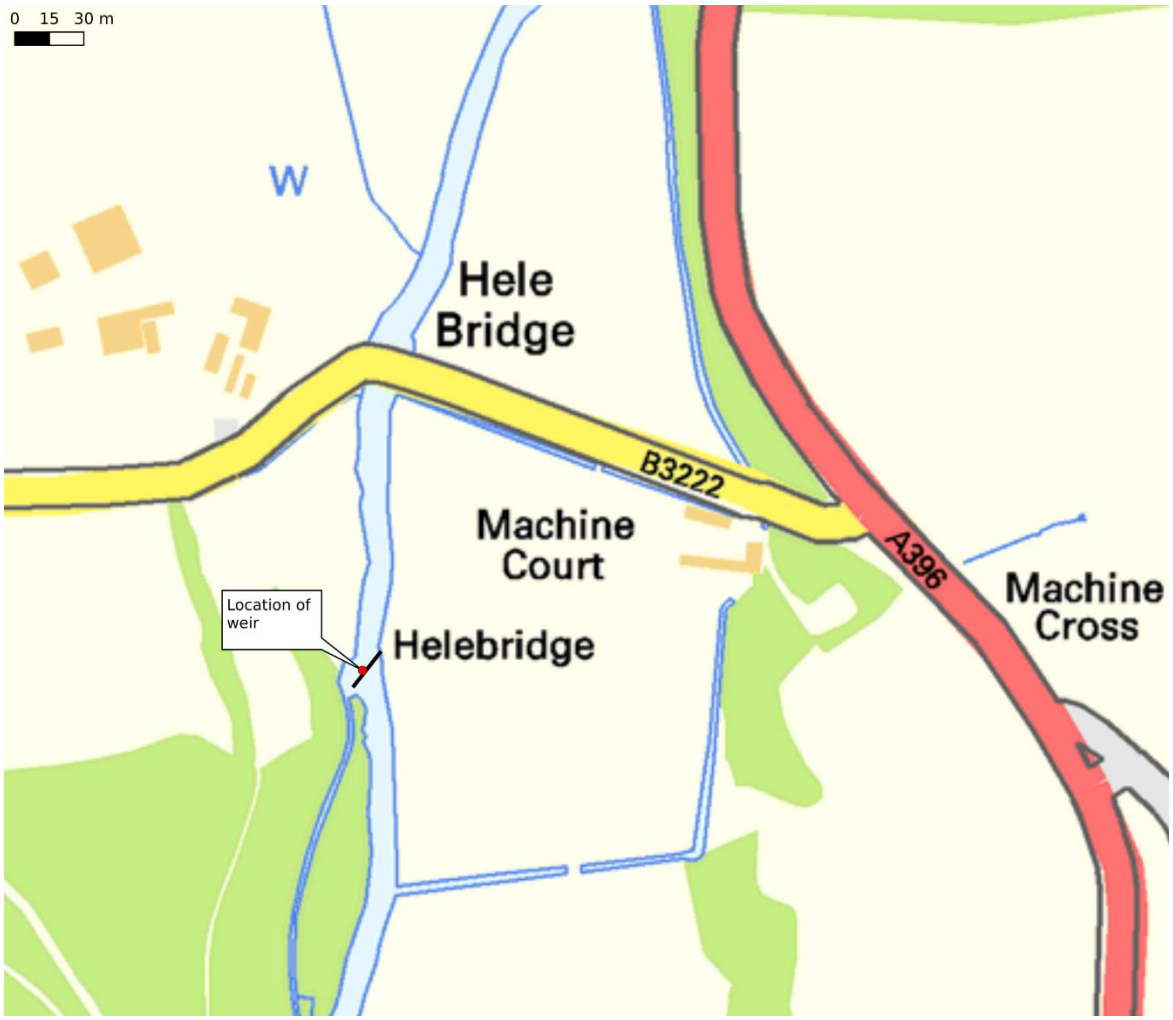

Figure 1 Site considered in the report.

Figure 2 The breached weir, looking upstream, with breach visible on left of photograph (true right of channel).

Figure 3 Central arch of the bridge, showing flow dropping off the concrete apron into pool on downstream side.

Two options are to be modelled: complete removal of the weir (referred to as Design Option 1), and increasing the breach in the weir to approximately half the channel width (Design Option 2). The aim of the modelling is to quantify potential effects of the proposed designs in terms of 3 main impacts:

- Water levels at low flows (i.e. up to the 5th percentile flow, Q5), for understanding fish passage
- Water levels at high flows (QMED, the median annual flood, and above), for understanding flood risk
- Velocities through the bridge at high flows, and potential for scour, to understand any structural impacts on the bridge

2. Hydrology

2.1 Flood flows

Flood flows (i.e. for QMED and above) have been calculated using WINFAP methods as implemented in software version 5.0.8181. There is a gauge nearby at Pixton, with 57 years of record, also on the Exe but downstream of the confluence with the Haddeo, meaning the gauge drains a catchment (160 km^2), significantly bigger than at the site (88 km^2). Nevertheless this gauge is useful for confirming WINFAP results, as well as being used in the WINFAP software as a donor and in the pooling group.

Catchment descriptors for the site were taken from the FEH web service, and used to calculate QMED as the indexing flood. Results for QMED using various methods are summarised in Table 1. For Helebridge, the QMED estimates are reasonably consistent, ranging between 42 and 53 m^{3}s^{-1}. These values tie in with those gauged at Pixton, when adjusted by catchment descriptors: the ratio between gauged and catchment descriptor QMED estimates at Pixton is 0.82; the ratio between the donor method and catchment descriptors at Helebridge is 0.78. This is to be expected as Pixton is the geographically nearest gauge and therefore is given the most weight of the 6 donors by WINFAP. Using Pixton as the sole donor catchment gives a value of 44 m^{3}s^{-1}, close to the value from 6 donors. The agreement between the various estimates gives some confidence in the QMED value of 42 m^{3}s^{-1} recommended by WINFAP.

Table 1 QMED values for the site and gauge at Pixton

Location	Method	QMED (m^{3}s^{-1})
Helebridge	Catchment descriptors	53
	Donor adjustment	42
	Pixton as sole donor	44
	Channel dimension	47
Gauge at Pixton	Catchment descriptors	57.4
	Gauged	47.3

Pooling analysis has been used along with the QMED index flood to generate flood frequency estimates for return periods up to 1000 years. The pool of 12 gauges gives a total of 512 years (as recommended for return periods up to 100 years), with no discordant sites. The pooling group was reported as heterogeneous, but the lack of discordancy indicated that no sites should be rejected. Goodness of fit measures indicate that the GEV, Pearson Type III, and Kappa 3 distributions are acceptable fits to the pooling group. As a further check, the AMAX series from the gauge at Pixton was also used for a single site analysis, and adjusted by the ratio of QMED values at Pixton and Helebridge. The results for the 3 fitted distributions from WINFAP and the single site analysis are shown in Figure 4, showing good agreement between WINFAP and flows from Pixton. This gives further confidence in the hydrological analysis. As the GEV distribution represents the middle value of the 3 fitted distributions, this is used for modelling.

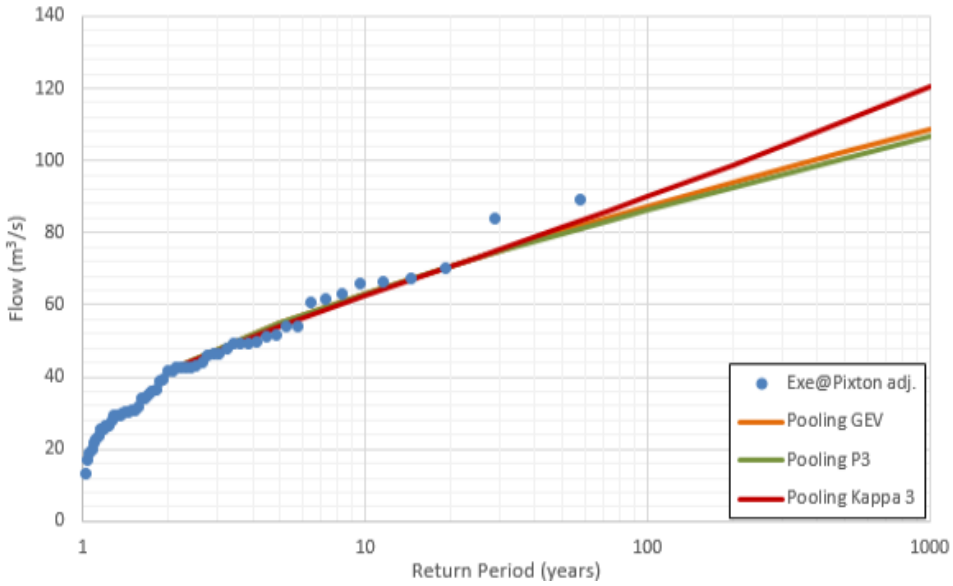

Figure 4 Flood frequency curves from pooling analysis, and flows from Pixton gauge adjusted by QMED ratios.

Climate change uplifts have been taken from the Defra climate change allowances website, for the East Devon management catchment. This give uplifts of 46%, 61% and 96% for the central, higher and upper estimates respectively. Combined with the flows derived by WINFAP, this gives a set of peak flows for modelling, shown in Table 2.

Table 2 Peak flows used in modelling

Return period/climate change scenario	Flow (m^3s^{-1})
2	42
5	55
10	63
20	71
50	80
100	87
100yr + 46%	127
100yr + 61%	141
100yr + 96%	171
200yr	94
1000yr	109

2.2 Low flows

Low flows (up to the 5th percentile flow) for the site have been calculated based on data from the gauge at Pixton, adjusted by multiplying by the ratio of catchment areas (0.55), and are summarised in Table 3.

Table 3 Low flows (m^3s^{-1}) from gauge at Pixton and adjusted for catchment areas

	Exe@Pixton	Helebridge
Q95	0.791	0.44
Q70	1.48	0.82
Q50	2.49	1.37
Q10	10.7	5.90
Q5	14.6	8.05
Qmean	4.49	2.48

3. Hydraulic model build

Hydraulic models representing current conditions and the two proposed design options have been built using Flood Modeller software version 6.1. The main input to the model is a topographic survey undertaken in March and April 2023, which comprises a dense set of spot heights for the reach between the bridge and the weir, and 8 cross sections up and down stream of the detailed area. The survey covered channel and bank areas only, and so was supplemented by LiDAR data from the Defra website at 1m resolution, captured in 2022.

The models were built as follows:

- Surveyed cross sections were imported directly into Flood Modeller.
- For the reach covered by the detailed survey, further cross sections were defined, with spacing based on standard guidance[1]. Elevations were taken from the detailed survey, and imported into Flood Modeller.
- The weir and its breach on the true right of the channel is represented as two separate spill units. Spills were used (rather than weir units) to allow variation in crest height, and two spills used to allow for different weir coefficients for the crest (1.7) and the breach (1.3, to represent the more uneven bed of the breach).
- The spill representing the breach has its lowest level set at 127.4m, corresponding to the highest bed level immediately upstream, rather than the bed level along the original line of the weir (which is significantly lower) (see Figure 5), as this is will be the level controlling water levels upstream.
- A further spill was added in parallel to represent bypassing flow over the floodplain.
- The bridge is represented as an Arch Bridge unit, with bed and arch openings taken from the survey. A spill in parallel is used to represent bypassing flow over the deck/parapet wall and floodplain areas. A further spill has been added immediately downstream of the bridge to represent the critical flow conditions likely to occur as water flows down the ~1m drop in bed height from the concrete apron into the pool immediately downstream. This spill will be drowned out at higher flows, but was necessary to represent water levels at low flows and see the potential impact on fish passage.
- Cross sections were extended to the full width of the floodplain to represent out of bank flows and avoid glass walling, with elevations taken from the LiDAR data.
- Manning's n values were specified as given in Table 4, based on photographs, satellite images and the topographic survey.
- Panel and bank markers were added at changes of roughness and where significant changes in velocity across the cross section are expected.
- An imposed flow is used as the upstream boundary condition; at the downstream end a normal depth condition is applied with a gradient of 1:200 calculated from the surveyed cross sections.

[1] Samuels, P.G. (1990) Cross-section Locations in 1-D Models – International Conference on River Flood Hydraulics.

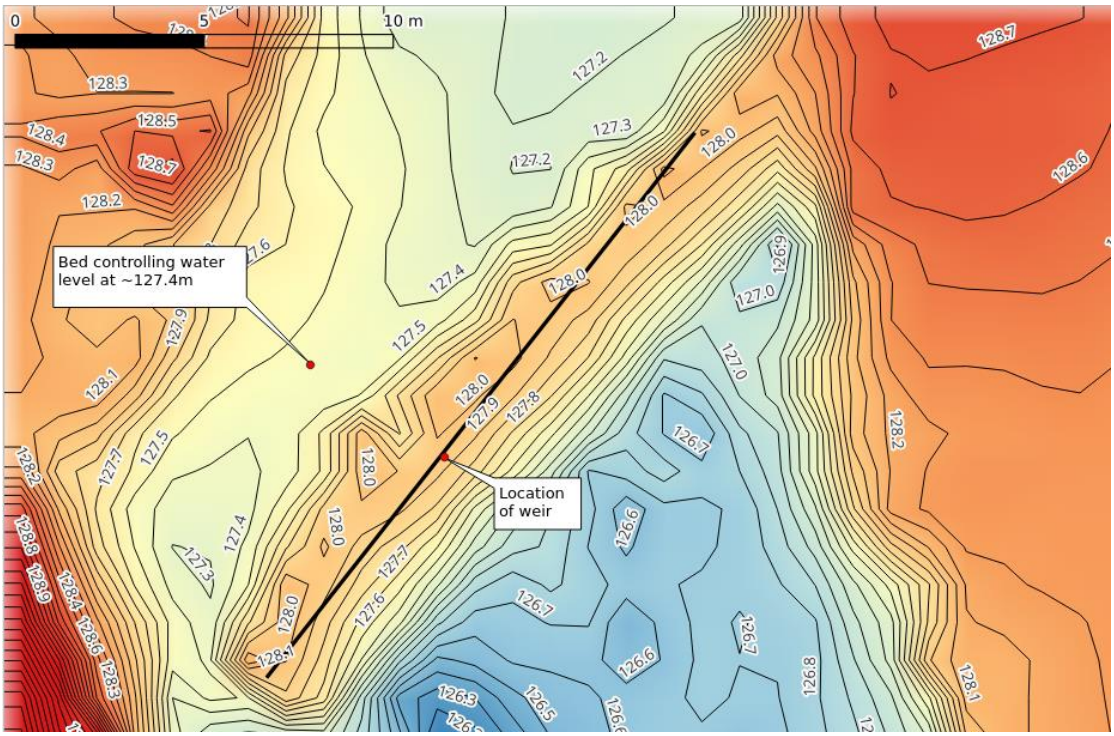

Figure 5 Contour plot (10 cm interval) of topography immediately surrounding the weir

The design options were implemented as follows:

- For design option 1 (DO1, full removal of weir), the 2 spill units representing the currently breached weir were removed entirely, along with the bypass spill. Cross sections 1215 and 1271 were adjusted downwards in the channel to represent the effects of erosion as the channel moves to geomorphological equilibrium after weir removal.
- For design option 2 (DO2, partial removal of weir), the breach in the weir was extend across a further 6.5 m (representing 25% of the weir) by extending the spill representing the breach and shortening the spill representing the weir crest, with the breach set at a level of 127.4 m, equal to the lowest level in the current breach. This level was chosen so as to represent the effect of widening the breach, rather than changing the bed level within the breach.

Table 4 Roughness values used in modelling.

Landcover	Manning's n
Channel – clean, straight, full stage, no rifts or deep pools, some weeds and stones	0.035
Floodplain – pasture, no brush, short grass	0.03
Banks – between light brush and trees and medium-dense brush and trees, in winter	0.06
Wooded floodplain – heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.10
Buildings – same as woodland but using slightly different value to allow changing independently of woodland if required later	0.101

4. Simulations

Initial stimulations for steady and unsteady flows for the 2 year flood flow and above indicated that transcritical flows occur within the reach, and therefore for accurate water levels the models should be run using Flood Modeller's direct transcritical solver, allowing hydraulic jumps etc to be represented. The direct transcritical solver was also applied to low flows, for consistency.

Dynamic simulations were also run for flood flows, with the hydrograph shape generated by an ReFH unit, using catchment descriptors from the FEH web service, with the output scaled to match the hydrological analysis described in section 0. These runs give an indication of how changes to storage within the reach caused by implementation of the design options could potentially affect water levels downstream. Given the transcritical flows seen, this runs should not be used to investigate the impact on water levels (there are some oscillations for example as flows switch between sub- and super-critical and cause problems for the approximate method used in the unsteady solver), but will give some indication of potential impacts downstream due to changes in storage.

5. Results

5.1 Low flows

The differences in water levels across the bridge and weir are summarised in

Table 5 and Table 6. The model indicates that head drop across the bridge for current conditions, while significant at low flows (e.g. ~300mm at Q50), is unlikely to be a significant barrier to fish passage. For the range of flows most important for fish passage (Q70-Q10) the head drop is below 400mm. The long section shown in Figure 6 indicates that under current conditions (blue line), water is impounded behind the weir as far as the bridge, and therefore the weir is controlling the head drop at the bridge.

The model results indicate that complete removal of the weir (DO1) will increase the head drop at the bridge, by around 500 mm, producing a head drop approaching 1m. Partial removal (DO2) also increases head drop at the bridge (by ~100-200 mm), which is less than for DO1.

Table 5 also shows depth over the concrete apron (taking the lowest point as being representative) within the bridge arches; this depth is independent of implementation of any design options. Shallow depths may be an issue for fish passage.

Table 5 Head drop at bridge and depth over apron (mm)

Flow	Current	DO1 – complete removal	DO2 – partial removal	Depth over apron (all scenarios)
Q95	435	884	536	140
Q70	370	860	517	185
Q50	291	835	490	235
Q10	229	732	334	487
Q5	225	716	313	579
Qmean	238	795	440	313
2yr	283	526	285	1487

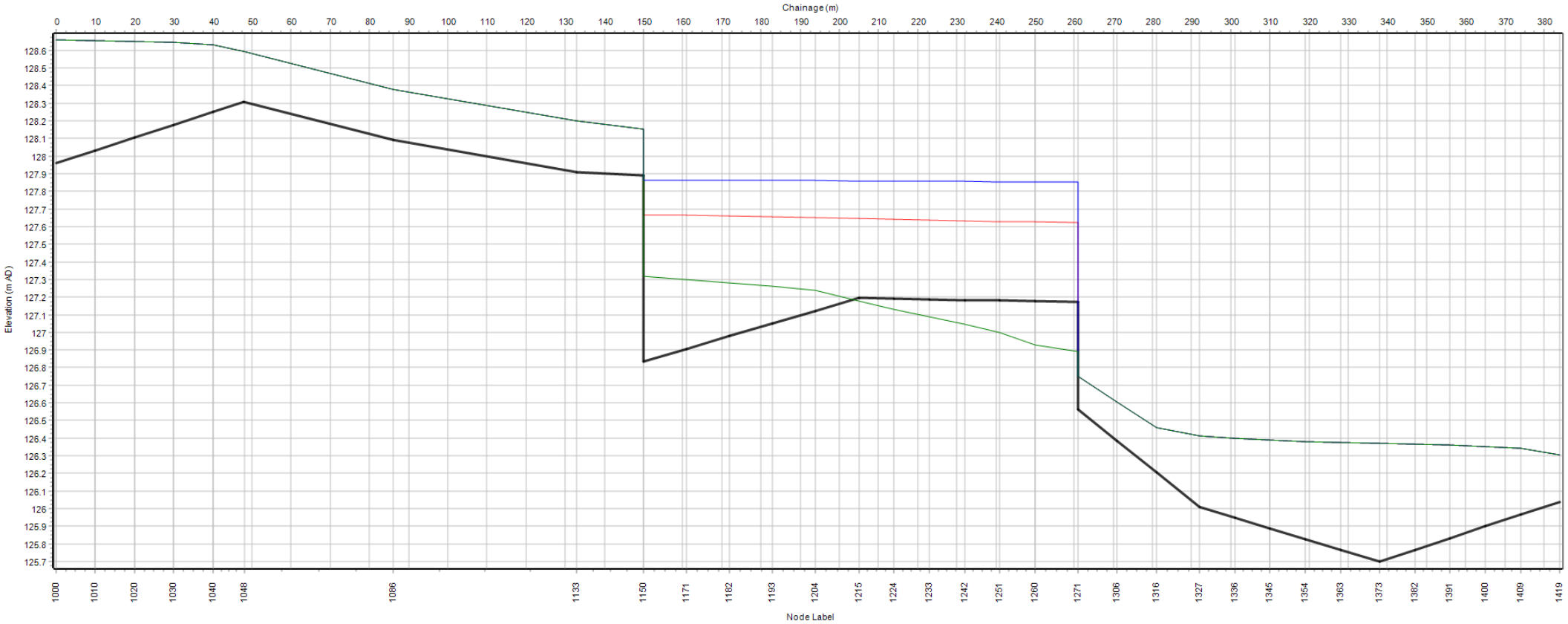

Figure 6 Long section of water levels for Q50 for current conditions (blue), design option 1 (complete removal, green), and design option 2 (partial removal, red). Current bed is shown as the black line.

Head drop at the weir (Table 6) is significant, being ~1 m for all flow values under current conditions. This head drop occurs over a distance of approximately 10m, rather than as a sudden drop (as in the case of the bridge), and therefore may not be as significant a barrier as it would seem at first. This head drop is significantly reduced in design option 1 (complete weir removal), being approximately 100-200 mm. For design option 2 (partial removal), the head drop remains large, albeit smaller than under current conditions.

Table 6 Head drop at weir (mm)

Flow	Current	DO1 - complete removal	DO2 - partial removal
Q95	962	164	847
Q70	1036	163	868
Q50	1100	140	869
Q10	1147	185	997
Q5	1142	237	1007
Qmean	1150	148	904
2yr	909	115	851

5.2 Flood flows

Table 7 shows the changes in water level caused by implementation of the design options at high flow values for key cross sections along the reach. The negative numbers indicate a general lowering of water levels at flood flows, with the only increase seen for the 1 in 20 year flood for design option 2 which arises from a hydraulic jump forming for this flow ~50m downstream of the bridge. The precise location of this jump and its size may not be well represented by the model, and given the absence of significant water level increases for other flows and cross sections, this should be viewed as an artefact of the modelling, arising from the need to use the transcritical solver.

A long section for the 1 in 100 year flood is shown in Figure 7. The decrease in water level under DO1 and DO2 affects the reach between the weir and the bridge only, and does not propagate above the bridge, indicating is acts as a control on upstream water levels rather than these being affected by hydraulics below the bridge.

Table 7 Water level changes (+ve implies increase, -ve implies decrease) in mm.

Flow	Section 1296 Downstream of current weir		Section 1271 Upstream of current weir		Section 1160 Downstream of bridge		Section 1150 Upstream of bridge		Section 1000 Upstream end of model	
	DO1	**DO2**	**DO1**	**DO2**	**DO1**	**DO2**	**DO1**	**DO2**	**DO1**	**DO2**
2yr	0	0	-794	-58	-243	-2	0	0	0	0
5yr	0	0	-676	-41	-123	-3	0	0	0	0
10yr	0	0	-182	-37	-86	-3	0	0	0	0
20yr	0	0	-187	-34	5	86	0	0	0	0
50yr	0	0	-196	-29	-78	0	0	0	0	0
100yr	0	0	-208	-28	-72	0	0	0	0	0
100yr+46pc	-488	0	-219	-17	-29	0	0	0	0	0
100yr+61pc	-440	0	-220	-16	-29	0	0	0	0	0
100yr+96pc	-340	0	-216	-14	-25	0	0	0	0	0
200yr	0	0	-217	-25	-69	0	0	0	0	0
1000yr	-665	0	-229	-21	-42	0	-1	0	0	0

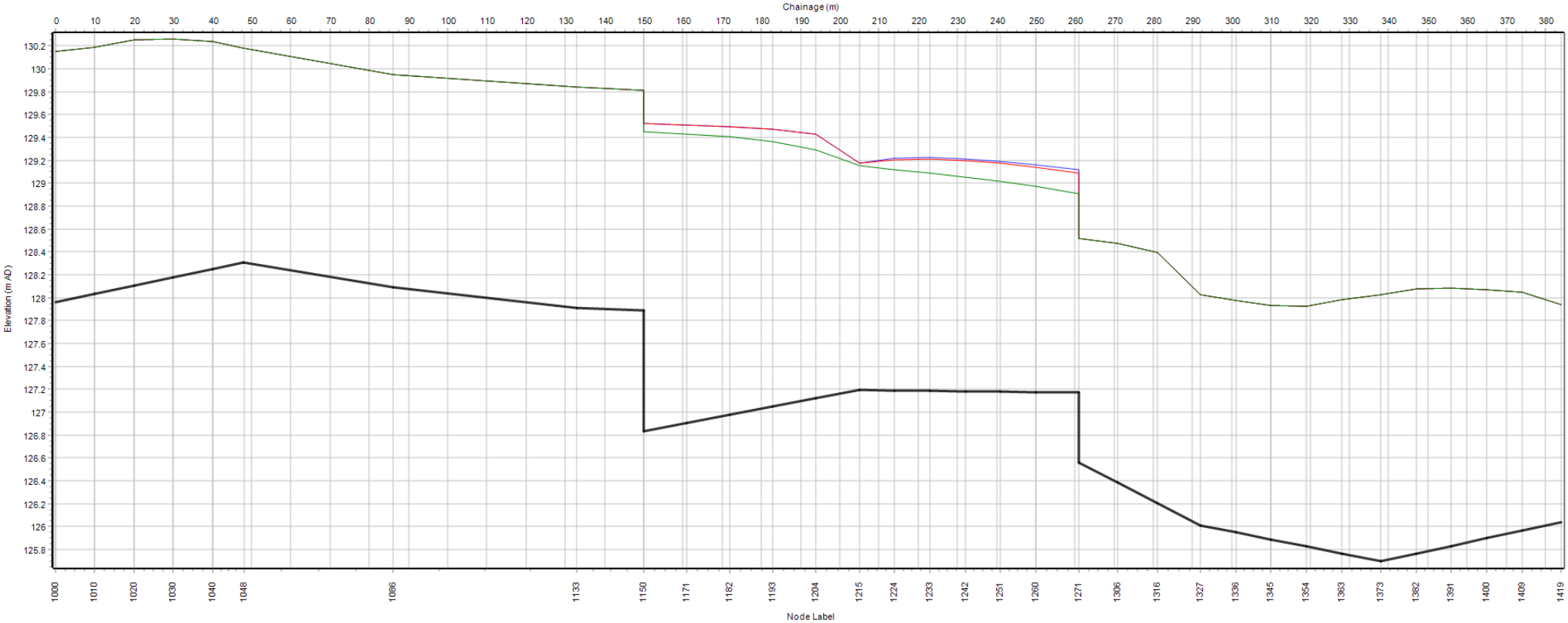

Figure 7 Long section of modelled water levels for the 1 in 100 year flood for current conditions (blue), design option 1 (complete removal, green), and design option 2 (partial removal, red). Current bed is shown as the black line.

Table 8 shows peak flows at the downstream end of the reach as predicted by the unsteady simulations. The results show no significant increases in flow caused by design options 1 or 2, because the changes in water level do not result in any significant change in storage in the reach at peak flows.

Table 8 Peak flows (m^3s^{-1}) predicted by the model at the downstream end of the reach

Flow	Inflow	Current	DO1	DO2
2yr	41.8	41.8	41.8	41.8
5yr	54.9	54.9	54.9	54.9
10yr	63.1	63.1	63.1	63.1
20yr	70.8	70.8	70.8	70.8
50yr	80.4	80.4	80.3	80.4
100yr	87.3	87.3	87.3	87.3
100yr+46pc	127.4	127.4	127.4	127.4
100yr+61pc	140.5	140.5	140.4	140.5
100yr+96pc	171.0	170.9	170.9	171.0
200yr	94.0	94.0	94.0	94.0
1000yr	108.6	108.5	108.5	108.5

5.3 Effects on bridge

There are two main possible causes of structural issues for the bridge: increased water velocity through the bridge arches, and potential for scour immediately below the downstream face.

Table 9 shows velocities in the bridge calculated for low and flood flows. These values will be representative of current and proposed design options, as the velocity is controlled by upstream water level and is unaffected by backwater effects from downstream (see Figure 7). The velocities reach a maximum of 4 ms^{-1}. Because the velocities are insensitive to downstream changes, the proposed designs will have no impact on velocities.

Table 9 also shows the head drop across the bridge for current conditions and under design options 1 and 2. The head drop is greater than currently for both design options, but these converge as flow increase: for the 1 in 10 year flow and above, the head drops are broadly the same at approximately 300-400 mm. At the flows where we might expect scour problems, the design options are therefore having little impact in terms of the head drop (and hence erosive power). Scour is not likely to be an issue here in any case, as the channel is cut into bedrock at this point, but the

modelling results indicate that scour potential is unlikely to be increases by either of the options.

Table 9 Velocities through bridge, representative of current and proposed designs, and head drop for current conditions and proposed designs.

Flow	Velocity (ms^{-1})	Head drop (mm)		
		Current	DO1 - complete removal	DO2 - partial removal
Q95	0.4	435	884	536
Q70	0.6	370	860	517
Q50	0.7	291	835	490
Q10	1.3	229	732	334
Q5	1.5	225	716	313
Qmean	0.9	238	795	440
2yr	2.9	283	526	285
5yr	3.1	301	424	303
10yr	3.2	305	391	306
20yr	3.4	388	383	302
50yr	3.5	296	374	296
100yr	3.6	295	367	295
100yr+46pc	4.0	334	363	334
100yr+61pc	4.2	348	377	348
100yr+96pc	4.5	389	414	389
200yr	3.6	296	365	296
1000yr	3.8	311	352	311

6. Conclusions and recommendations

The main conclusions from this modelling study are:

- Design option 1 (complete removal of weir) will promote fish passage at the weir itself, reducing the head drop from ~1 m to 100-200 mm at low flows, but will increase the head drop at the bridge to values approaching 1m.

- Design option 2 (partial removal of weir) will slightly reduce head drop at the weir (by 100-200 mm), with an increase in head drop at the bridge of a similar amount.
- Issues for fish passage would remain after either option were implemented, and therefore some additional mitigation measures will be required whichever option is chosen.
- Neither of the design options should affect flood risk, either within the reach, or upstream or downstream.
- Neither of the options should affect the structural integrity of the bridge.
- Shallow flows over the concrete apron under the bridge may be an additional barrier.

The main recommendations are:

- Further photos of flows at the bridge and weir, preferably with scale, would be helpful in confirming the water levels predicted by the model at low flows.
- As both options 1 and 2 would still cause issues for fish passage, intermediate options, such as complete removal of the weir with a pre-barrage to mitigate the head drop at the bridge, should be investigated.
- Some mitigation (e.g. baffles) may be required to help fish negotiate the concrete apron. Given the hydraulics of the bridge, with water levels controlled by the entrance to the arches, it should be possible to do this without affecting flood risk upstream.
- The effects of design option 1 have been modelled based on the assumption that the river returns to a geomorphological equilibrium relatively quickly, which may not be the case given the bed material. Some consideration therefore should be given to the geomorphological setting and how the bed might respond after weir removal if this option is to be pursued.