5. Hydraulic Modelling of Proposed Diversion Channel Inlet

5.1 Introduction

A number of diversion inlet options were examined in the previous section and the conclusion reached was that a flow control structure in the Deel River downstream of the diversion channel inlet is required to achieve the design requirements for the distribution of flood flow between the river and the Diversion channel.

The most suitable structure for imposing the desired flow control on the river flow and upstream water level is produced by a series of culverts that can impose inlet control and surcharging at the critical flow. The options of a number of culverts is preferred over a clear spanning structure as it provides the versatility required imposing the desired flow control to achieve the design objectives of retaining the median flood in the river and achieving an almost 50:50 split if flows at the 100year flood magnitude.

The proposed control structure comprising a series of culverts within the river channel can be sized in ope area and invert level so that the larger flood flows are throttled resulting in the upstream flood level rising which in turn improves the distribution of flow to the diversion channel. The control structure also ensures that the flood level upstream of the inlet can be maintained similar to the existing situation such that the potential for lowering of upstream flood levels and significantly increasing channel velocities and resultant river bed scouring can be avoided. Without such a structure the diversion weir setting would have to be much lower or designed as an automated movable weir / Sluice gates which would in any case drawdown the upstream flood levels by c 1 to 1.5m resulting in significant increases in channel velocities and increased local bed scouring. As the immediate upstream channel scours this drawdown could potential migrate for some distance upstream particularly in a gravel / cobble bed system.

5.2 Flow Control Structure

A schematic of the proposed flow control structure is presented in Figure 19. This structure is located on a straight section of river reach 150m downstream of the proposed diversion channel inlet. This structure is an embankment across the width of the floodplain with a reinforced concrete structure across the river channel section. Within the channel section there is a central culvert opening 6m wide by 2.4m high set at an invert of 16m OD and soffit level of 18.4m OD. Two additional culverts on either side of the central culvert, each 3m wide by 2m high are set at invert levels of 16.5m OD. These two 3m wide culverts are fitted with adjustable steel plates at the upstream face which can be lowered to reduce the section area available to flow for finer adjustment. The deck level of the structure is set at 19.7m OD and is protected from over topping by a reinforced concrete wall to level of 20.8m OD. The crest level of the earthen embankment is set at 21m OD. Rock armour rip-rap bed protection is afforded to the channel in the form of an apron and to the toe of the embankment extending both upstream and downstream to protect against local scour. A 3m wide by 2m high blind culvert with an adjustable steel plate provides the option for a further opening if required to facilitate maintenance of the other three culverts.

The base of the culverts have 500mm high concrete baffle walls to trap and retain gravel and stone within the culvert barrel for fishery requirements. The entrance and exit to and from the structure has reinforced concrete wing walls to facilitate transition of flow to and from the structure and to protect the earthen embankment tie in section. The total longitudinal width of the structure is 3.5m.

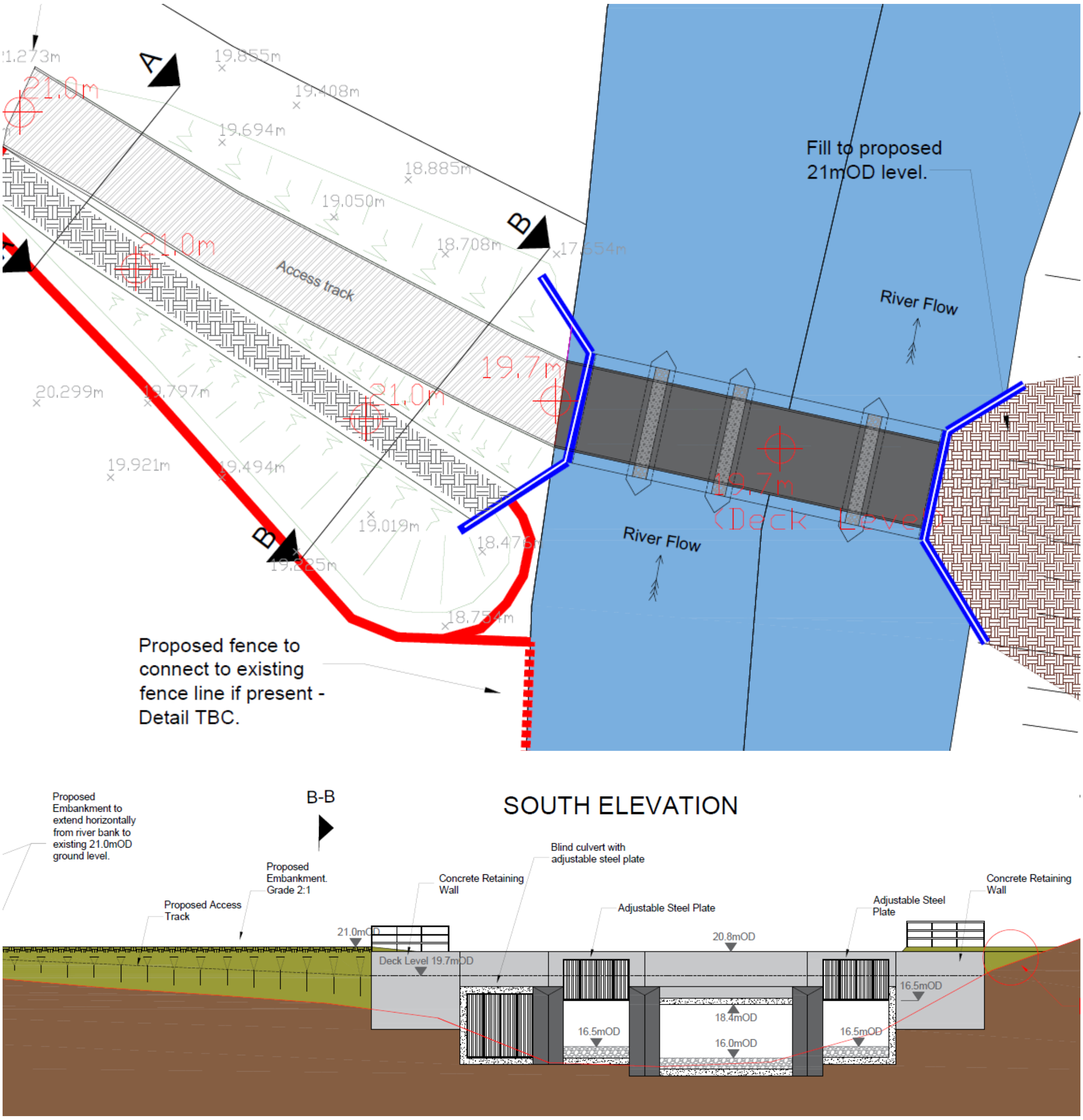

Figure 19 Plan View and Cross-Section of Flow Control structure with adjustable gates (extract from Ryan Hanley Scheme Drawings entitled River Flow Control Structure and Embankment)

5.3 Hydraulic Control Relationship of the Flow Control Structure

Hydraulic simulations were carried out for the proposed flow control site located 150m downstream of the diversion channel inlet. These simulations were performed for the current existing case and with the proposed flow control structure. Figure 20 presents the computed rating curves for the existing case and for the proposed case with the flow control structure which shows the throttling effect with increased upstream water level (Stage). At a flood level of 21m OD the downstream flow control structure commences being overtopped. However with the proposed diversion channel in place this flood level at the flow control structure is unlikely to be ever reached as increasing flood flows will spill into the diversion channel as flood levels rise downstream. The flow control culvert structure was modelled using a culvert inlet loss coefficient of 0.5 and the exist loss coefficient of 1.0 and manning n of 0.03 (not very sensitive to the manning n) but very sensitive to the inlet and outlet coefficients. The 2 number 3m wide gated culverts were modelled with an open height of 1.8m (i.e. steel gate lowered by 0.2m. The downstream channel conveyance/ roughness influences this rating relationship which is represented by the stage discharge curve predicted for the existing case.

The introduction of this downstream control structure will increase the upstream flood level from 18.85 to 19.67m OD at the median annual flood flow rate of 81cumec. For the larger flood flows the inlet weir to the diversion channel will spill and thus limit the rate of flow in the downstream channel to 95cumec. The rating relationships presented below in Figure 20 is for the channel immediately upstream of the proposed structure both existing (without and proposed). In reality for 95cumec to occur in the proposed case the actual flow in the river would be in excess of the 100year flow of 189cumec. At the proposed maximum downstream flow rate through Crossmolina of 95cumec the flood level increases from 19.14m OD immediately downstream of the control structure to 20.29m OD upstream of the control structure through the designed throttling effect in order to maintain upstream flood levels and avoid significant increases in upstream channel velocity above the diversion.

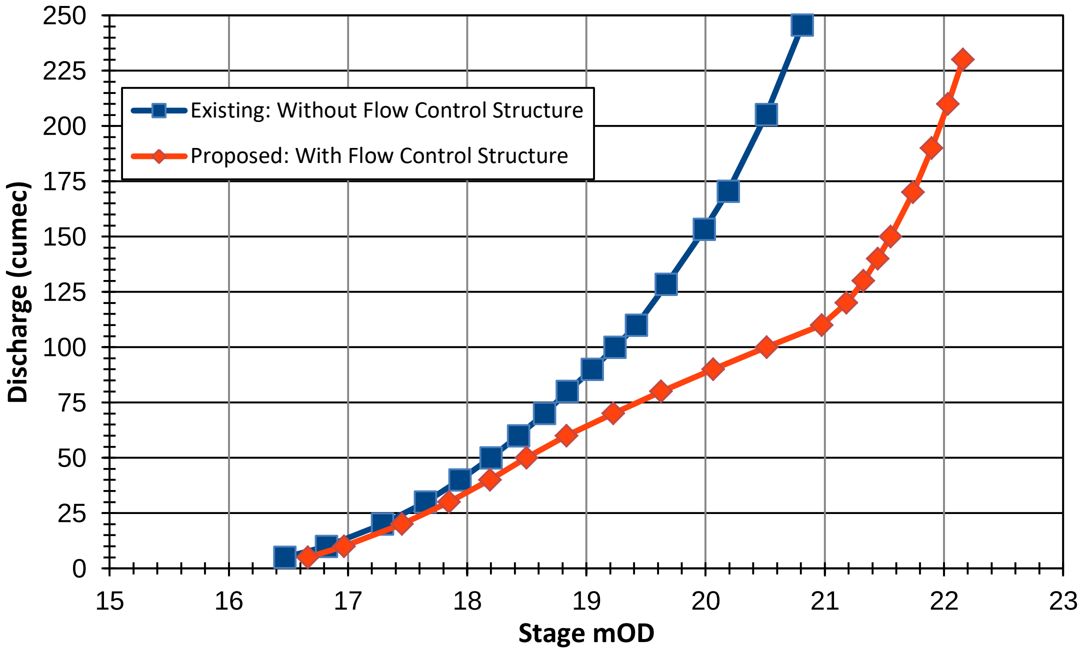

Figure 20 Computed Stage Discharge Relationships at upstream face of proposed Flow Control Structure site Existing and Proposed Cases

5.4 Diversion channel Inlet Weir

The proposed diversion channel inlet weir is an in-stream weir located at the head of the Diversion channel 50m upstream of the proposed Pollnacross Road Bridge. The proposed weir structure is a thin platted rectangular triangular plan weir with an apex angle of 90degrees and two weir legs of 35m lengths each (70m total crest length). The proposed crest level of weir is constant at 19.4m OD. The nappe of the weir should be formed using a steel plate supported on reinforced concrete wall with the potential for adjustment of the crest level up or down post construction should it be required. The proposed weir Plan layout is presented below in Figure 21.

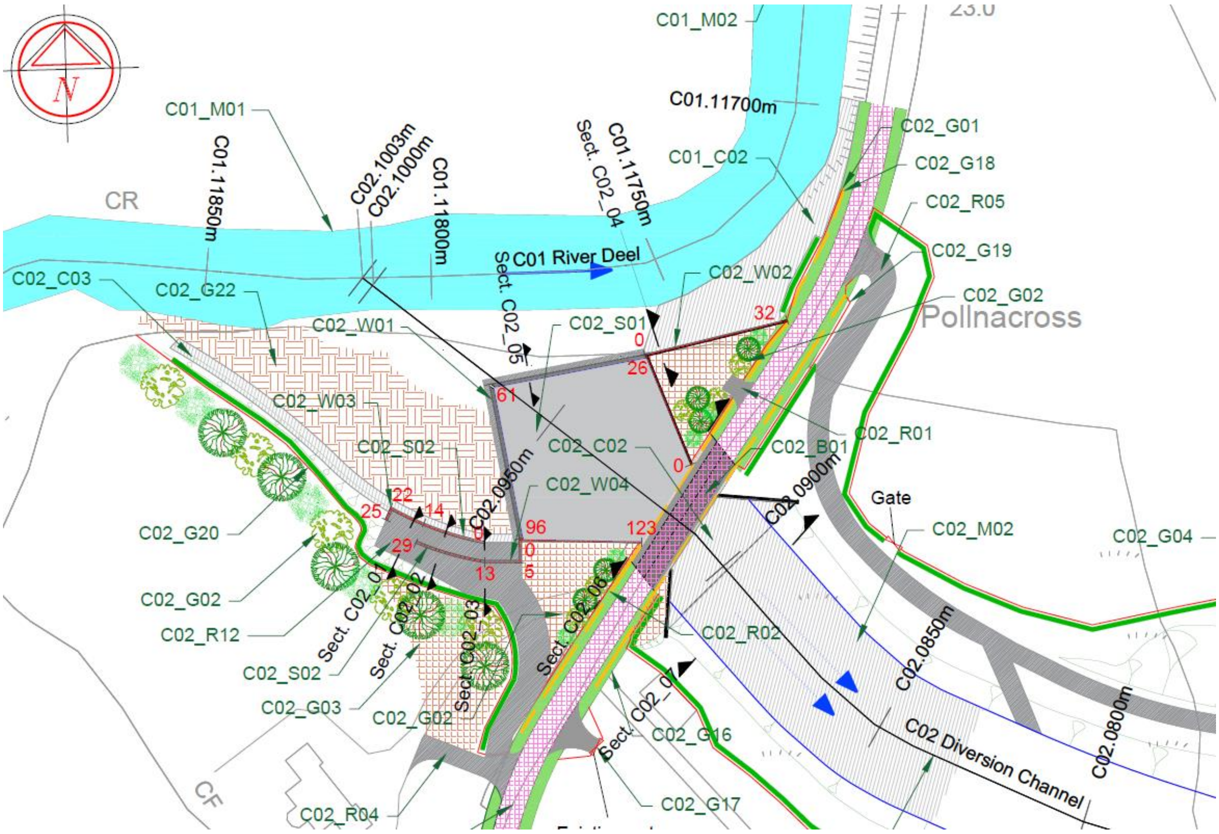

Figure 21 Plan view of diversion channel inlet weir upstream of Pollnacross Bridge

5.5 Hydraulic relationship of Diversion Inlet Weir

Figure 22 presents the proposed Weir stage discharge relationships based on Gupta et al. (2013) and Kumar et al. (2011) for a 90degree triangular sharp crested plan form weir. The Gupta et al equation includes the approach velocity and its discharge calculations are c. 10% lower than the Kumar et al(2011) weir equation. A 10% margin of error would be expected from such weir equations.

This assumes free overflow (i.e.) no drowning. Submerged weir adjustments can be made using the Villemont Equations (1947) and HR Wallingford modified equations

(HRW Report SR 564, March 2000). The impact of slight drowning (i.e. downstream flood level in diversion channel exceeds weir crest level) on the weir and diversion performance was found to be insignificant. It is important to note that drowning or the weir, partial or otherwise, is not predicted.
At the critical design flow rate of c 95cumec discharging over the weir the Gupta et al. equation requires a water level of 20.29 upstream of the weir to discharge 95 cumec whereas the Kumar et al equation would generate a flow of 105cumec for such an upstream water level (10% difference). This translates to a water level difference upstream of the weir of 0.08m.

Gupta K.K., Kumar S. and Kumar K (2013) “Flow Characteristics of Sharp-Crested Triangular Planform Contracted Weirs, Int Journal of Engineering Research and Technology, Vol 2 Issue 12 Dec 2013, ISSN: 2278-0181

Kumar S, Ahmed Z and Mansoor T ” Journal flow Measurement and Instrumentation 22 (2011)pp 175-180, Elsevier Press.

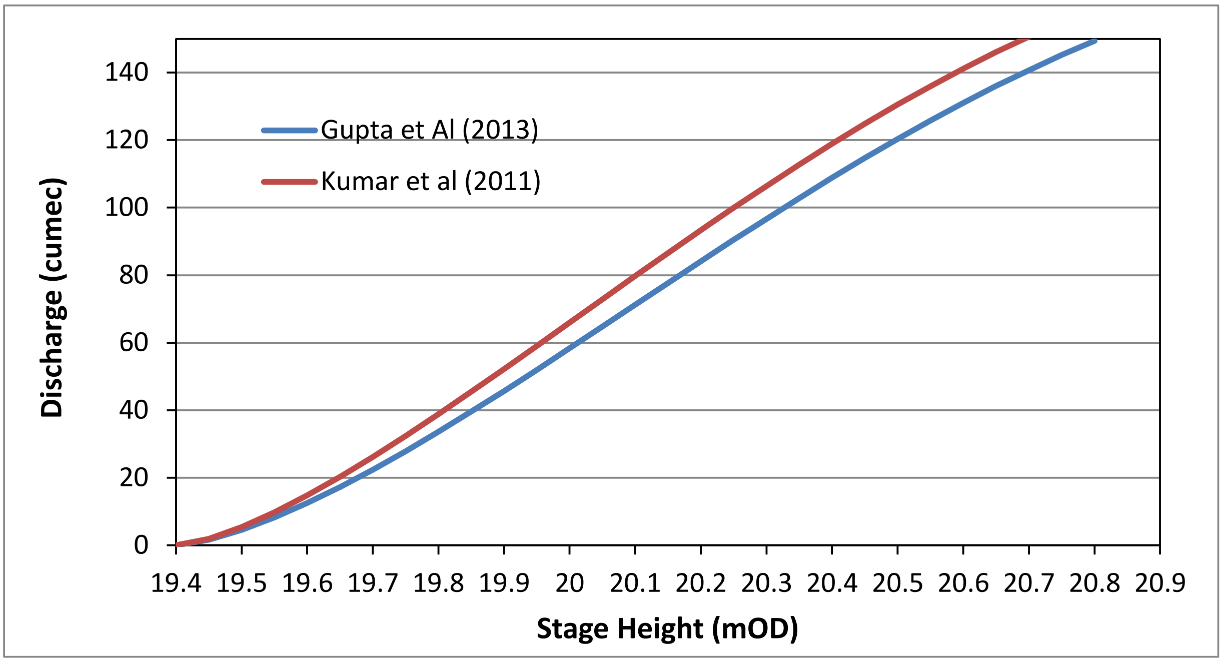

Figure 22 Computed Stage-Discharge Relationship for Diversion Weir structure

5.6 Diversion channel Stage - flow relationship

The Stage-Flow relationship for the proposed inlet weir in combination with the Flow Control Structure is presented below in Figure 23 using the Gupta et al (2013) triangular Plan Form sharp crested weir equation which includes the approach velocity. This was produced from the hydraulic modelling of a range of flows with the control structure and the weir as described above in Section 5.2 to 5.5 included in the model.

At the 100year design Flood of 187.8cumec the predicted flow distribution between the existing river and diversion channel is 94.35cumec retained in the Deel River channel through Crossmolina and 93.45cumec discharging over the weir in the diversion channel. The upstream Flood level in the river at the diversion inlet is predicted to be 20.274m OD

Using the Kumar et al. Weir equation in place of the Gupta Weir equation the predicted flow distribution for the 100year flood is 92.95cumec in the existing river and 94.85cumec over the weir and an upstream flood level in the river adjacent to the diversion inlet of 20.21m OD.

The impact of the different weir equation coefficient of discharge on flow distribution between the existing channel and the diversion channel is reasonably small with either equation achieving a favourable distribution.

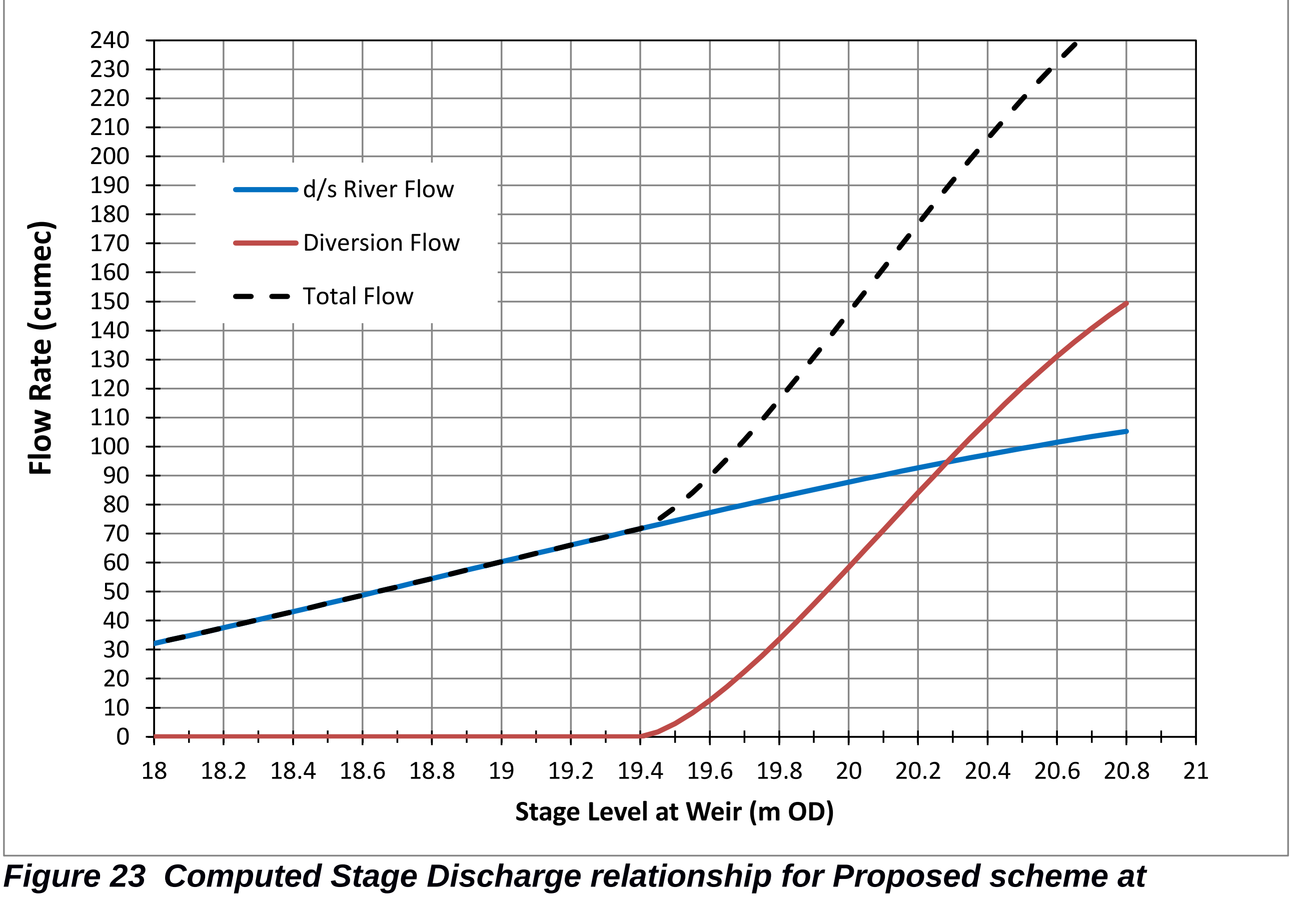

Figure 23 Computed Stage Discharge relationship for Proposed scheme at Diversion channel inlet weir

5.7 Hydraulic Model Simulation Results of Proposed Diversion Scheme

The Telemac2d Model of the upstream reach, diversion channel and washlands at Mullenmore was run for varying return period events from 2year to 1000yr under both existing and proposed cases. Figure 24 presents selected hydrograph output locations (4 locations) along the Deel river channel upstream of the flow control structure. Figures 25 to 28 present the computed water level hydrographs for selected return periods 2, 10, 100 and 1000year for both existing and proposed cases. These computed hydrographs show an increase in upstream flood level at the annual 2year flood event and a reduction in peak flood level at the 10year and greater return period with the proposed scheme. The computed flood peak velocities for the existing and proposed cases are presented in Figures 29 to 32 for selected return periods of 10 and 100year with and without the proposed scheme. These show an increase in upstream channel velocity over the existing case above the inlet weir due to a reduction in flood level and in the downstream section between the inlet weir and flow control structure a significant reduction in channel velocities is predicted due to the afflux affect from the control structure and the reduction in discharge due to the diversion.

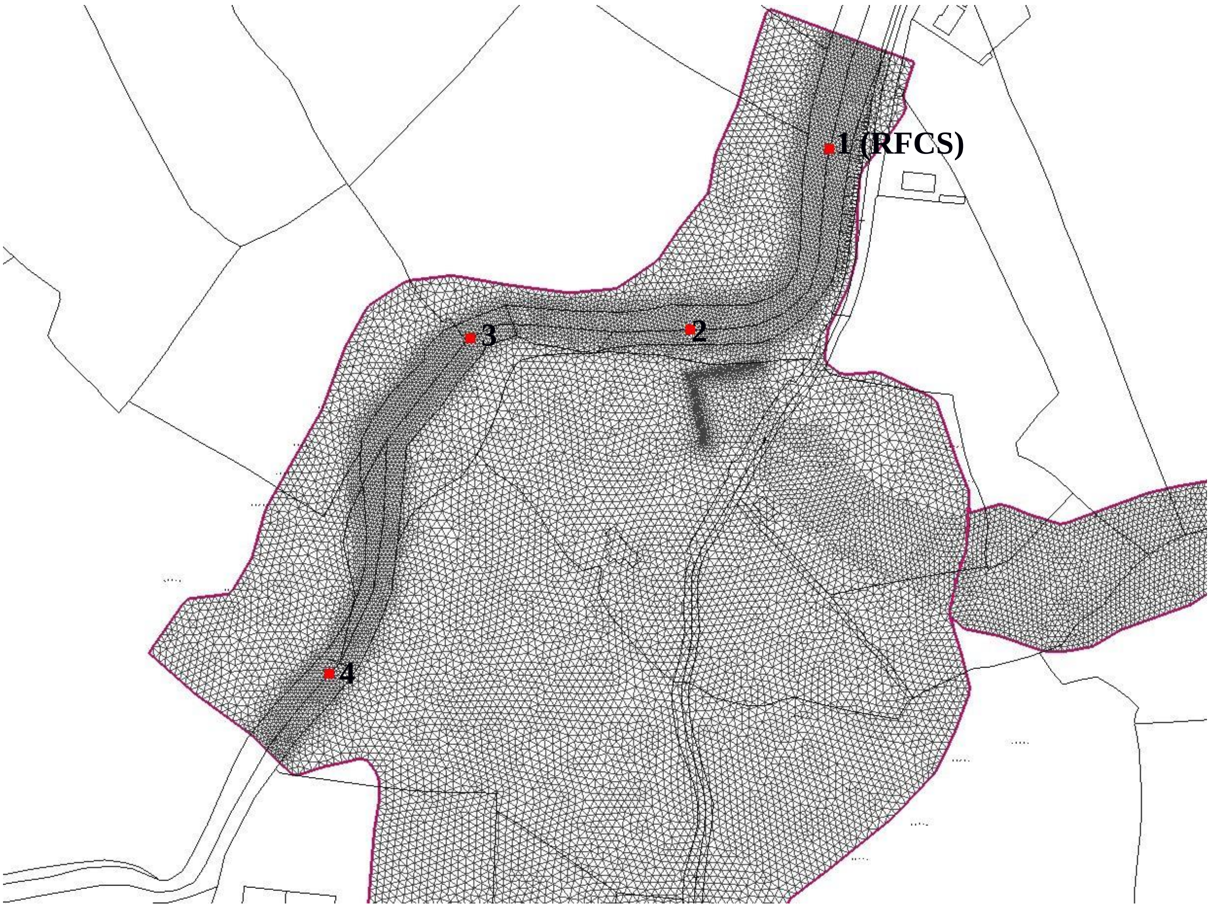

Figure 24 Upstream Reference Sites for comparing flood Hydrographs for existing and proposed cases.

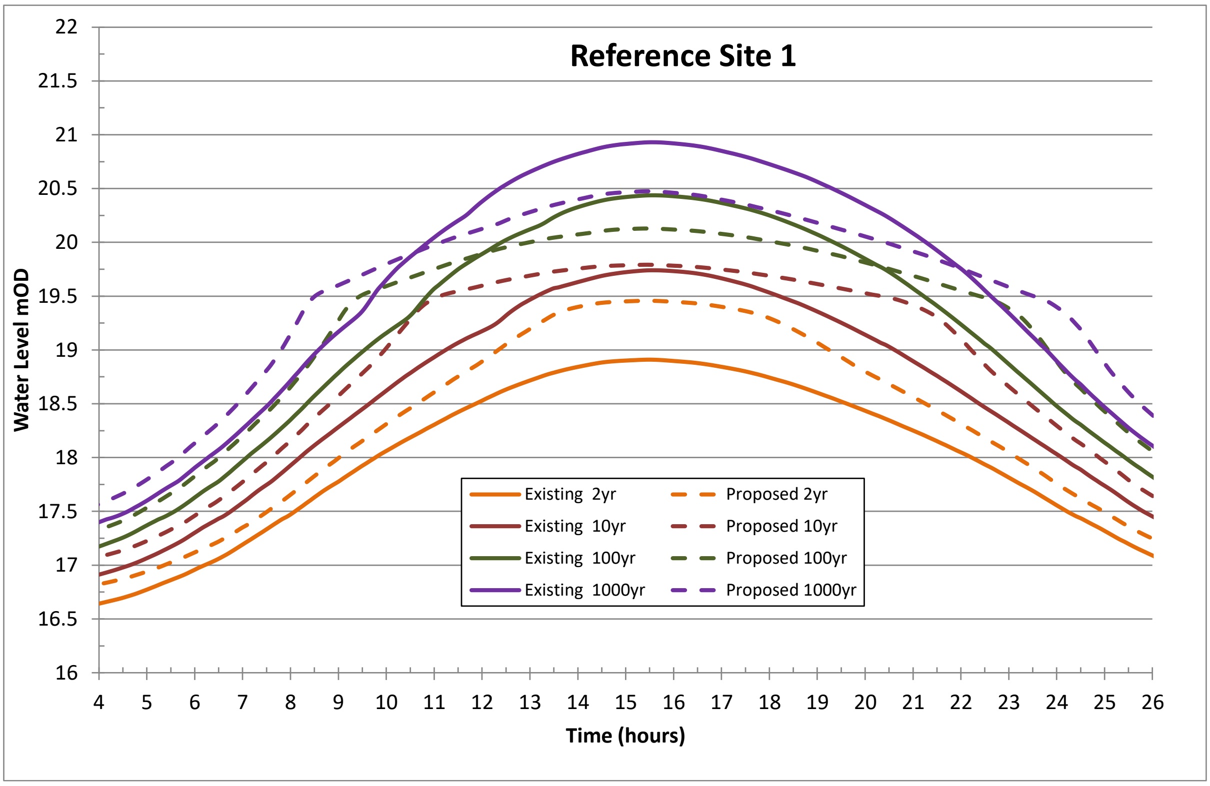

Figure 25 Computed Return Period Flood Hydrographs for proposed and existing cases at Reference Site 1

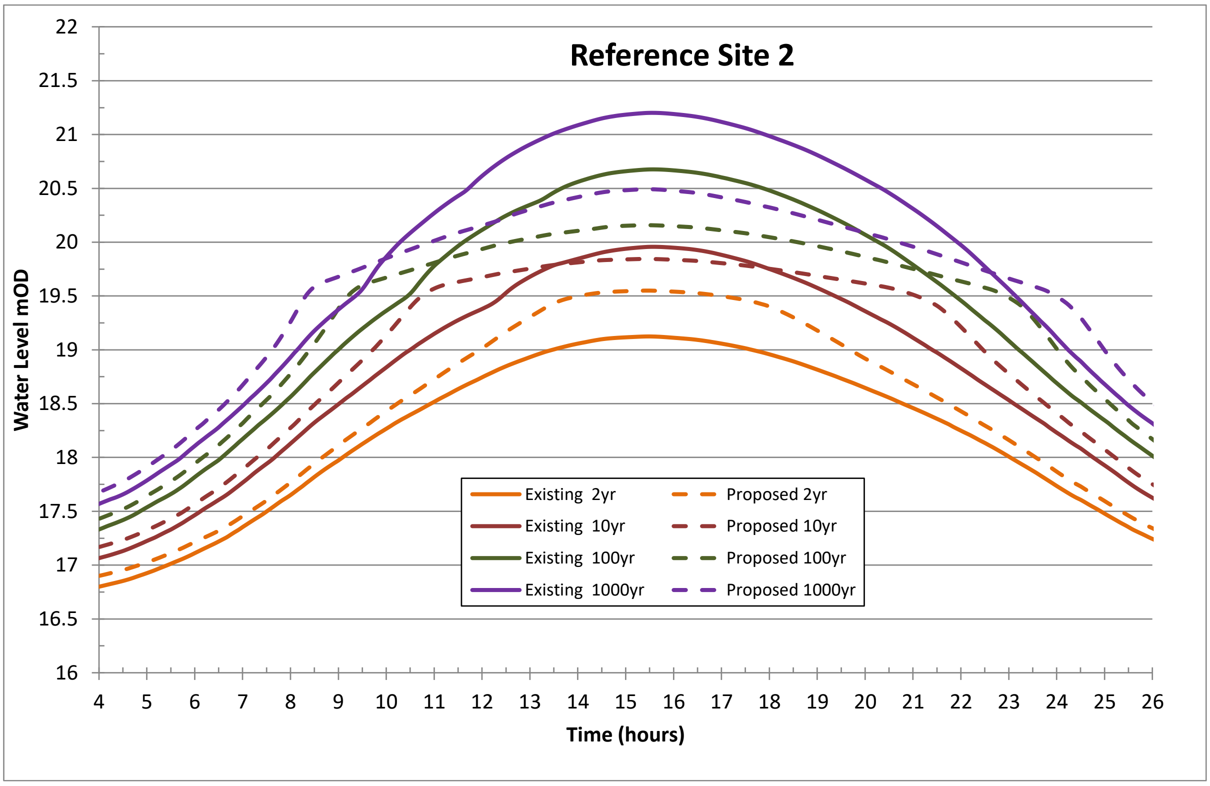

Figure 26 Computed Return Period Flood Hydrographs for proposed and existing cases at Reference Site 2

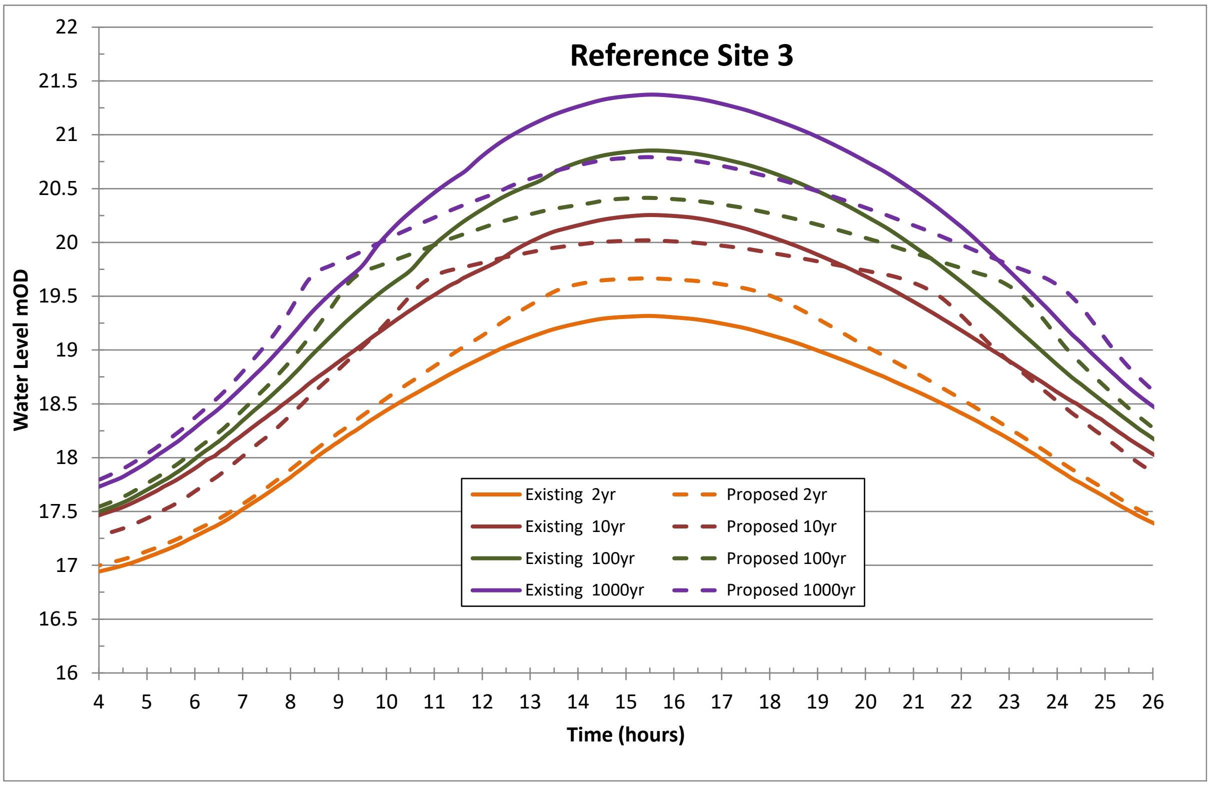

Figure 27 Computed Return Period Flood Hydrographs for proposed and existing cases at Reference Site 3

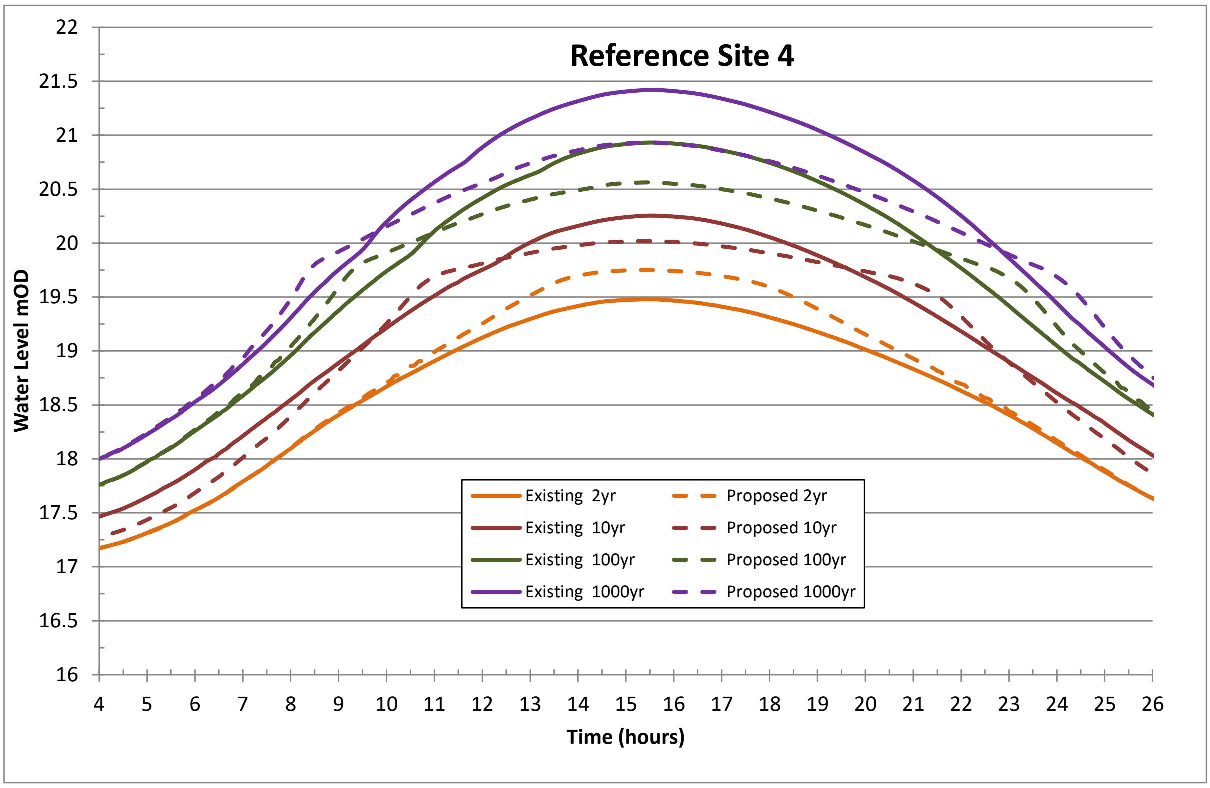

Figure 28 Computed Return Period Flood Hydrographs for proposed and existing cases at Reference Site 4

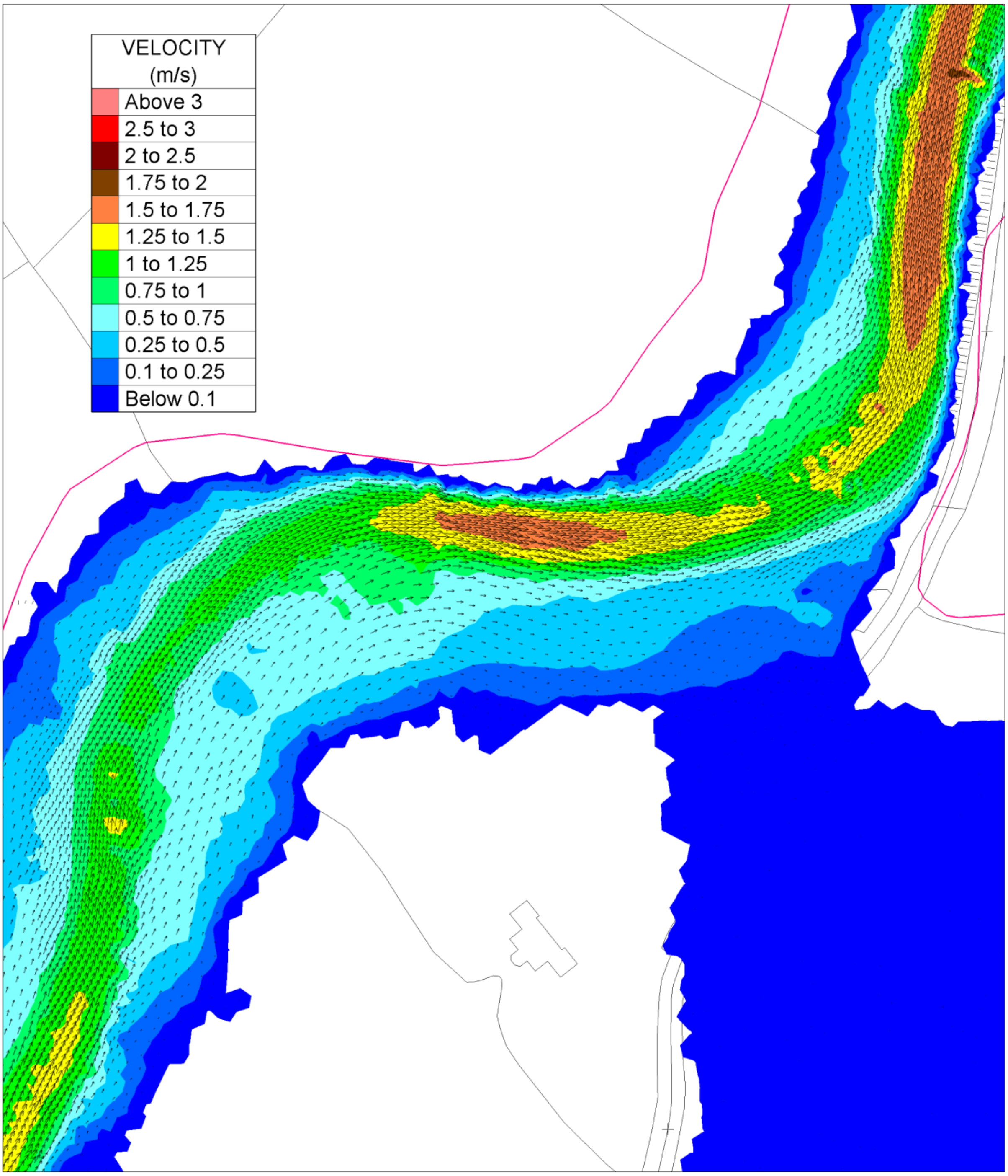

Figure 29 Computed peak flow velocities at 10year flood event under existing case.

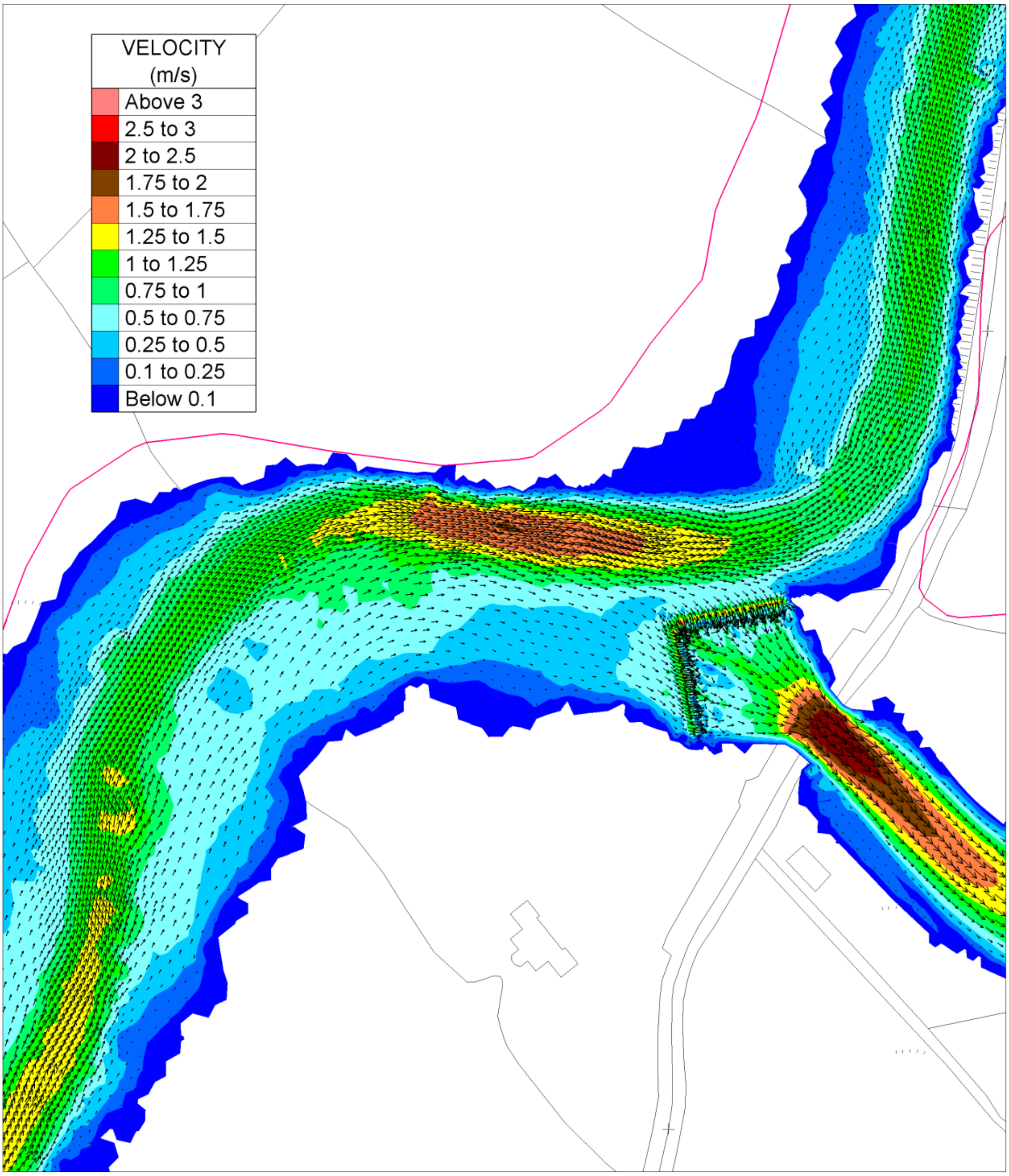

Figure 30 Computed peak flow velocities at 10year flood event under proposed case.

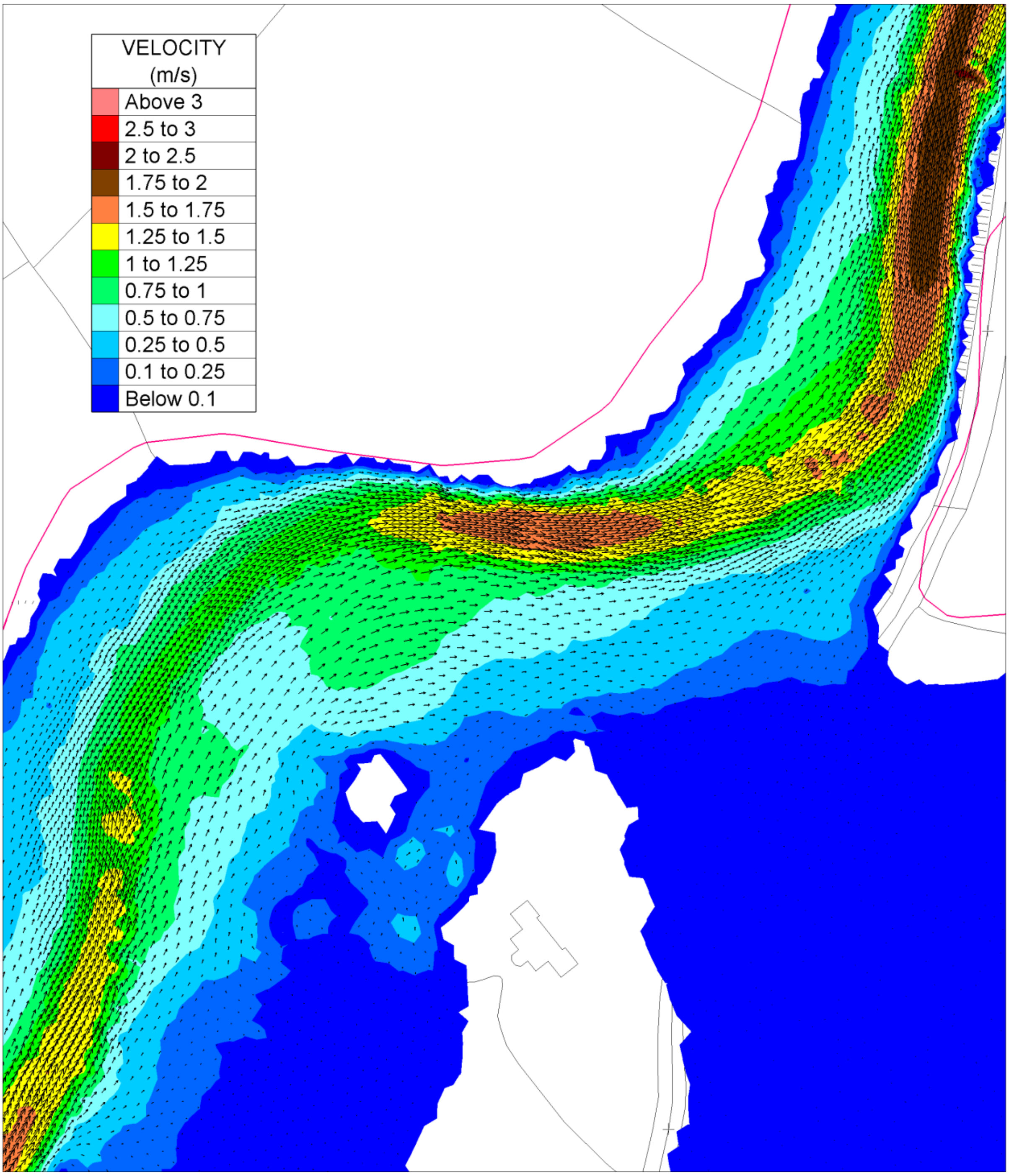

Figure 31 Computed peak flow velocities at 100year flood event under existing case.

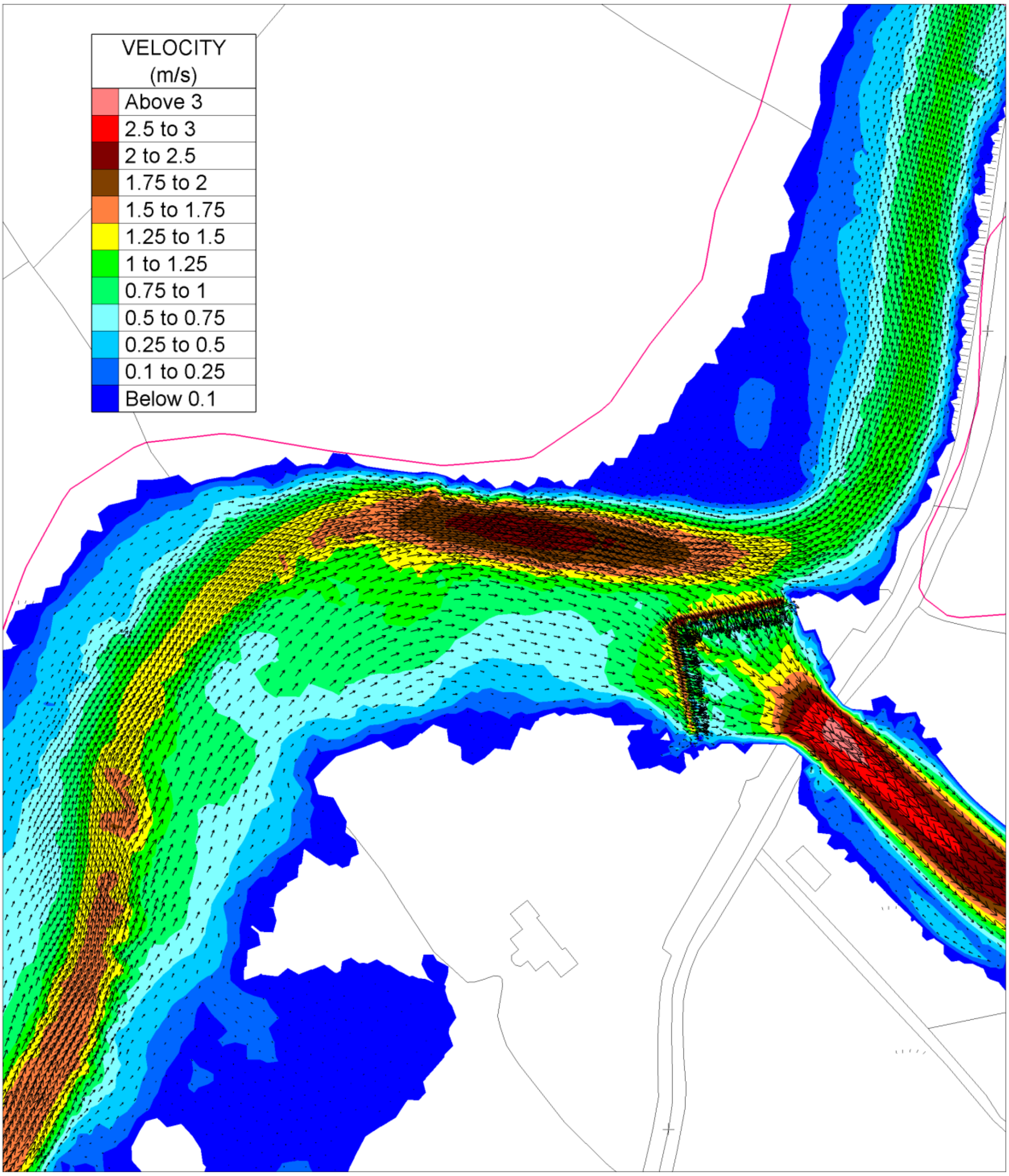

Figure 32 Computed peak flow velocities at 100year flood event under proposed case.

The upstream flood extents with and without the proposed scheme is presented in Ryan Hanley Scheme Drawing entitled "Upstream Zone of Influence of River Flow Control Structure" Drawing No. RFCS_ZOI_01 for a range of return period events (i.e. 2, 10, 25, 50, 100, 200 and 1000year floods). The 100year design flood extents is presented in Figure 33 which shows a reduction in flooding upstream of the diversion channel inlet over the existing situation.

Figure 34 and 35 show the computed flow velocities within the proposed diversion channel for the selected return periods of 10 and 100year events. These simulations show locally high velocities originating from supercritical flow at the weir and extending through the Pollnacross Bridge and extending for a distance downstream towards the first bend in the diversion channel. Locally higher velocities are also shown at the Mullenmore Bridge and in particular at the downstream drop structure due to the significant channel contraction and elevation drop. These locations will be scour protected as part of the channel works with reinforced concrete aprons at the bridges and downstream drop structure.

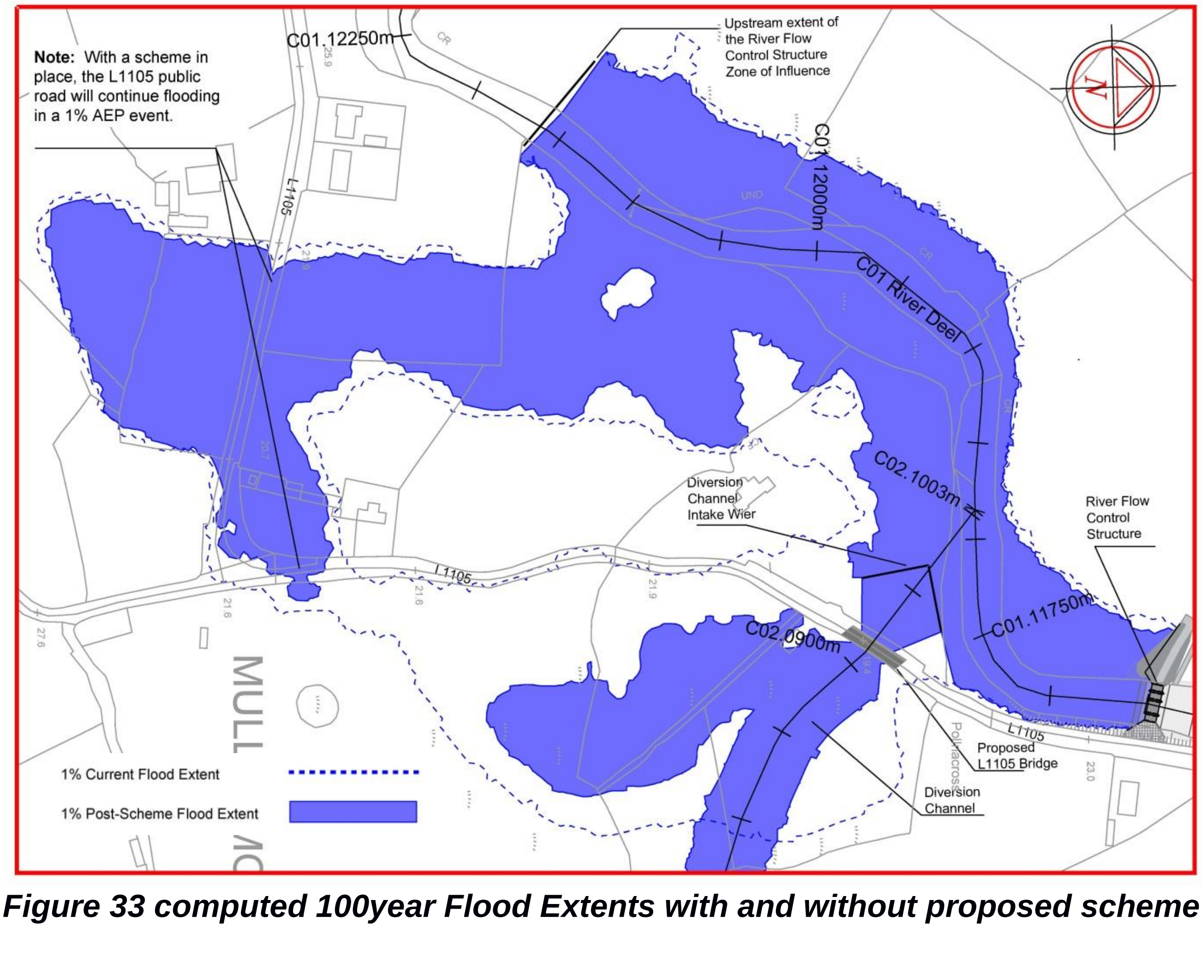

Figure 33 computed 100year Flood Extents with and without proposed scheme

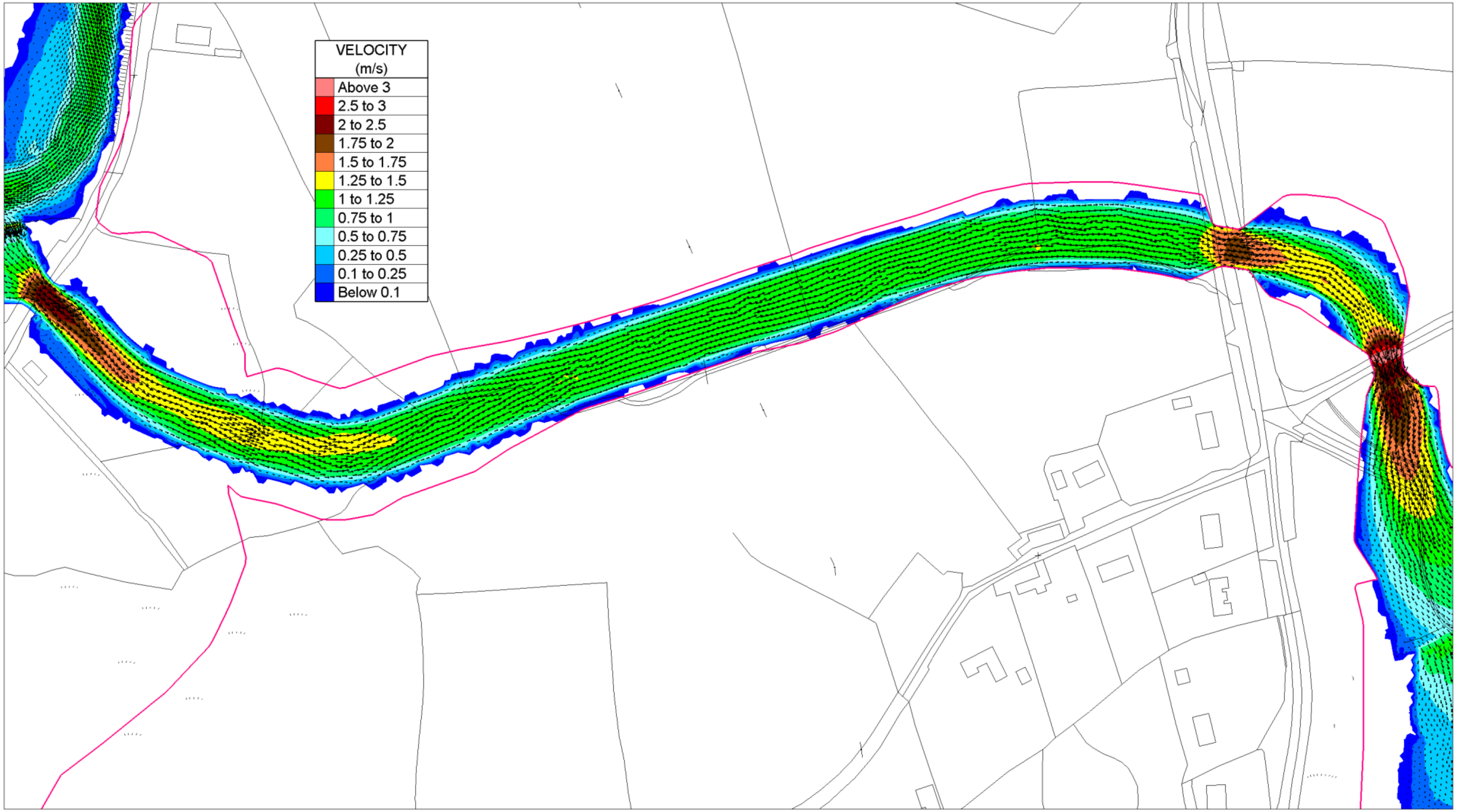

Figure 34 Computed flow velocities within diversion channel – 10year Flood Event

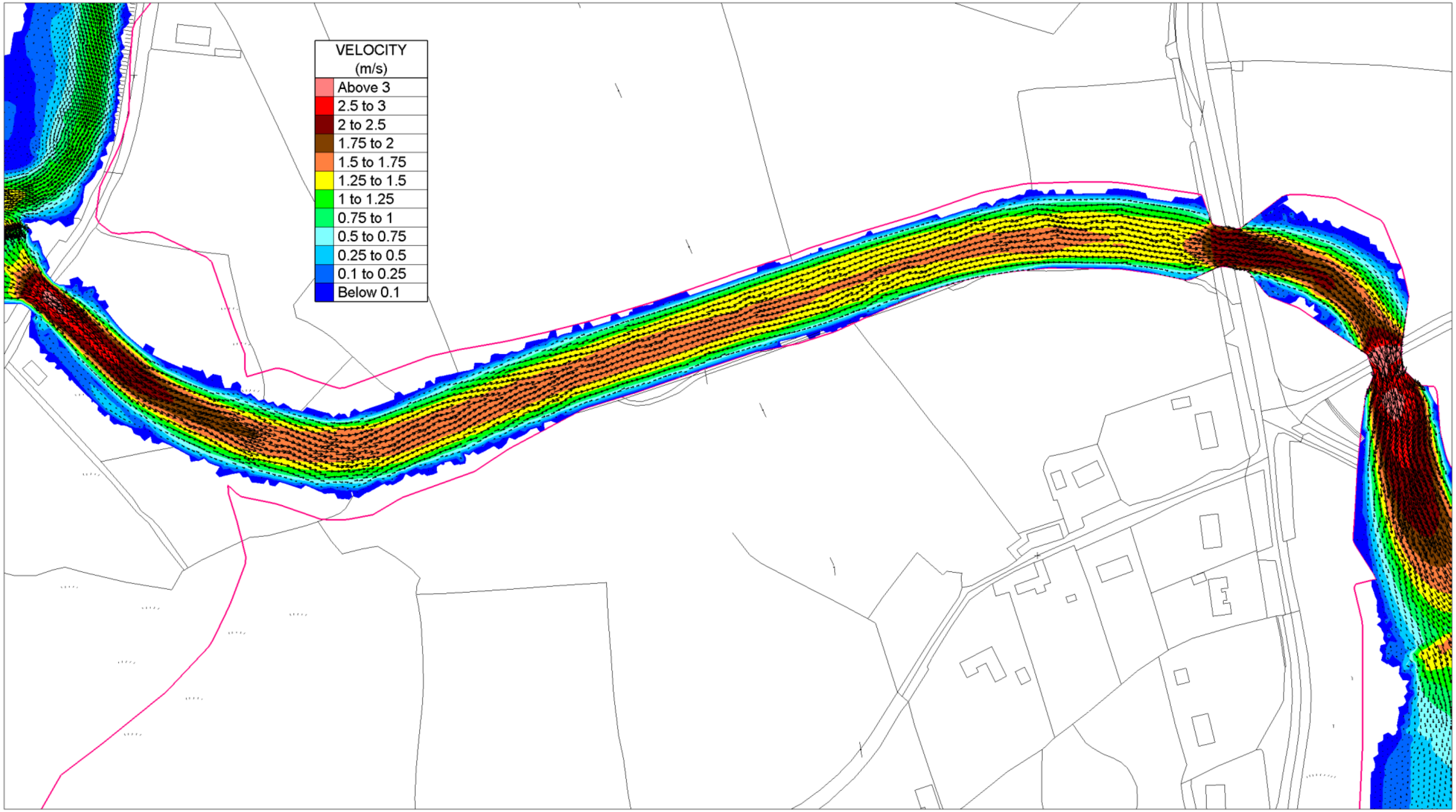

Figure 35 Computed flow velocities within diversion channel – 100year Flood Event

5.8 Downstream Mullenmore Washlands Modelling Results

The TELEMAC 2-dimensional model was extended to include the downstream washlands at Mullenmore stream and extended to Lough Conn. The diversion flow at the 100year design flood event for the relief scheme will discharge c. 95cumec to Mullenmore washlands. This flow will spread out and flood a reasonably wide washland area after exiting the diversion channel drop structure channel. In the vicinity of the former Mullenmore Mill structure and farm out buildings the topography of the basin contracts the flow width and the gradient increases locally resulting in increased velocities at this location, refer to Figure 35 and 36. Further downstream the flow again spreads out over a wide relatively flat riparian area that regularly floods from Lough Conn and flow velocities reduce significantly.

A number of flood inundation studies of the washland was performed for the 100year design flood event with different lake levels as follows:

- Lake level at normal lake extents based on the OSI mapping of 8.54m OD
- Lake Level at 95% non exceedance level (typical annual max winter flood level) of 9.84m OD
- Lake at historical maximum recorded level (December 2015) of 11.6mOD

The predicted flood extents of the 100year flood event coinciding with the above Lough Conn flood levels is presented in Figures 37 to 39. This mapping shows the extent of the lands at Mullenmore that are impacted by flooding as a result of the Diversion of the 100year flood event.

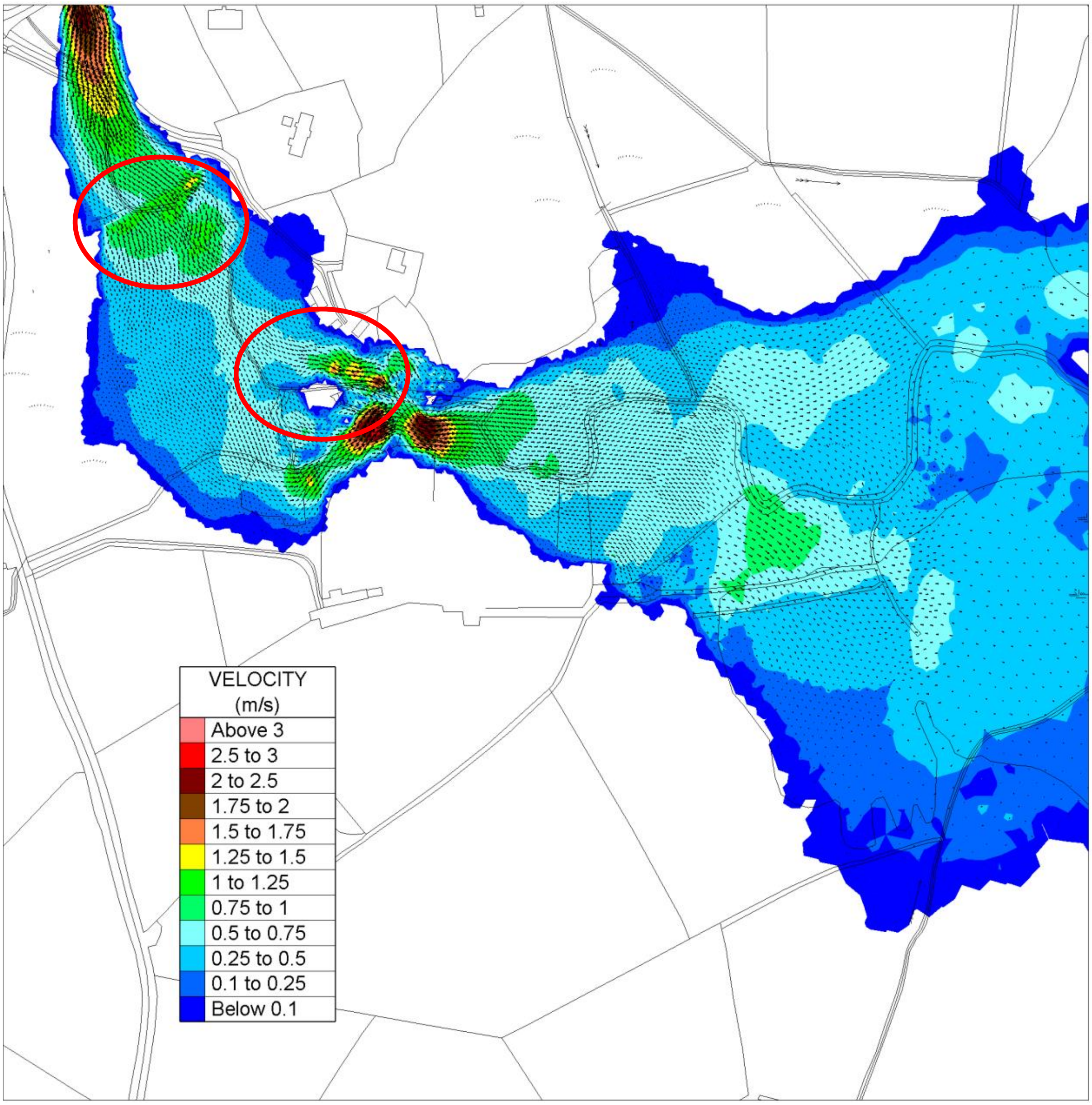

Figure 35 Computed peak flow velocities in washland at 10year design flood event

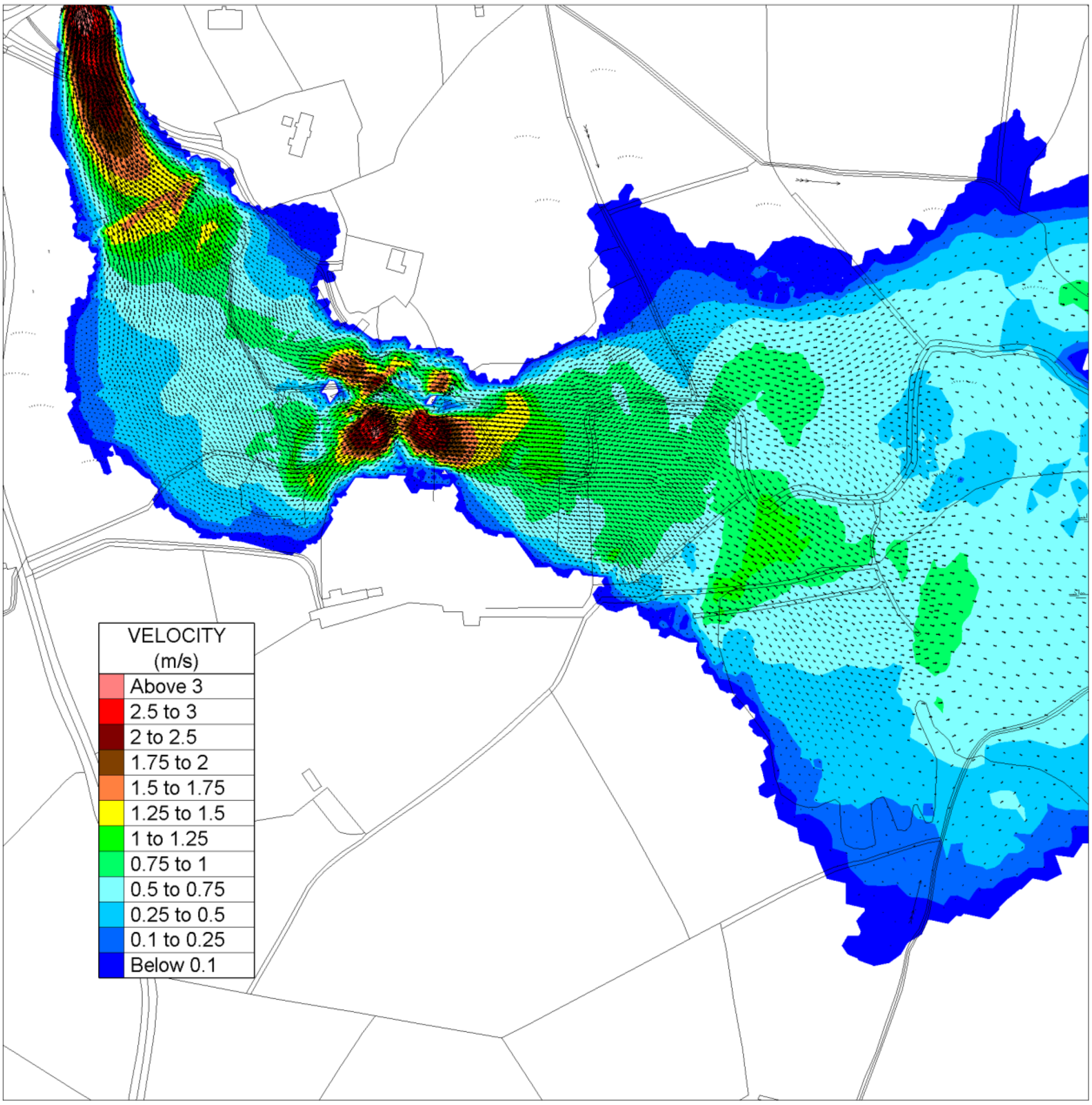

Figure 36 Computed peak flow velocities in washland at 100year design flood event

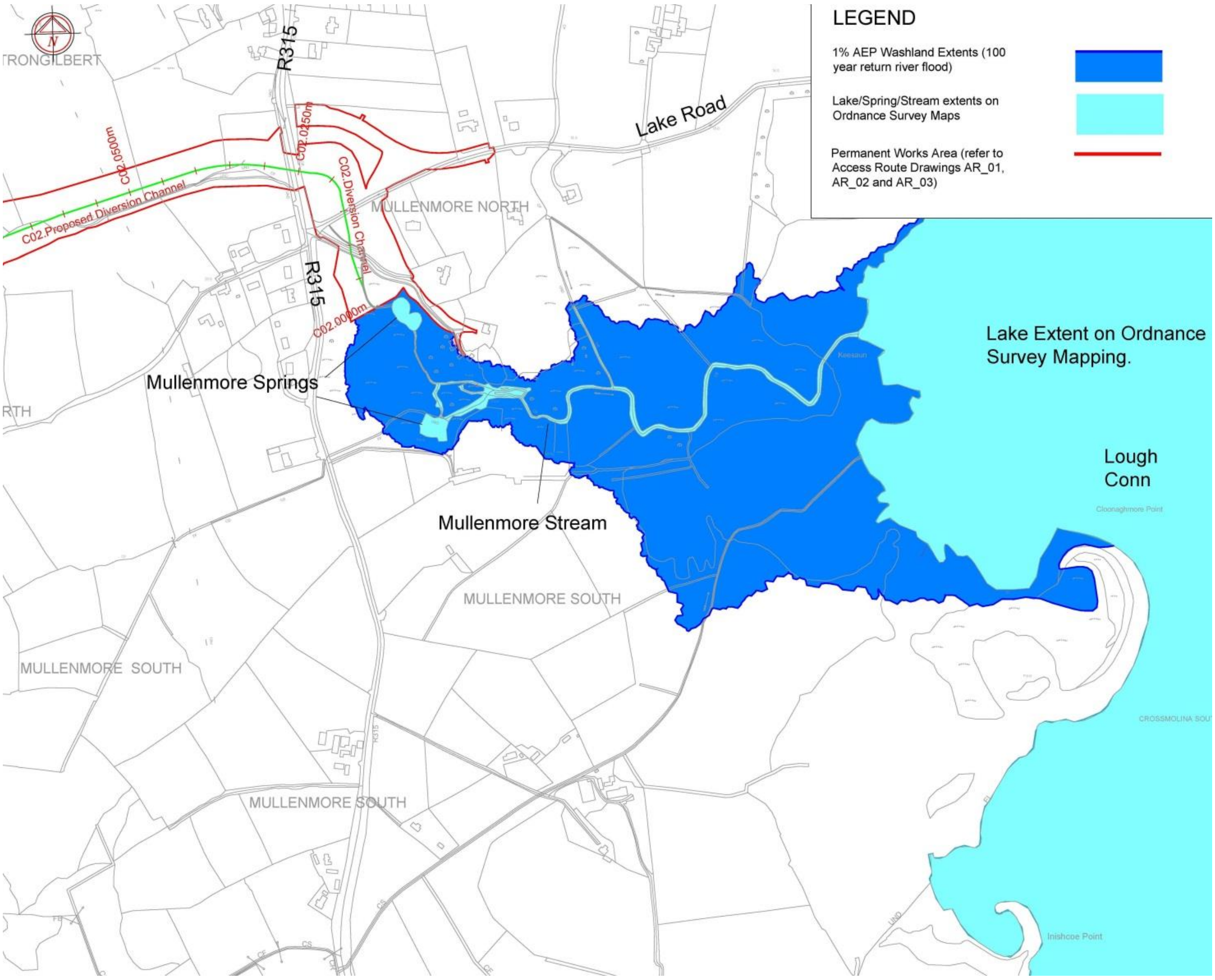

Figure 37 Computed Flood Extents in the Mullenmore for 100year design flood event with Lake level at the OSI mapped extents (8.54m OD)

Note: Lake extent represented by the cyan shading and additional lands flooded as a result of the diversion shaded in blue.

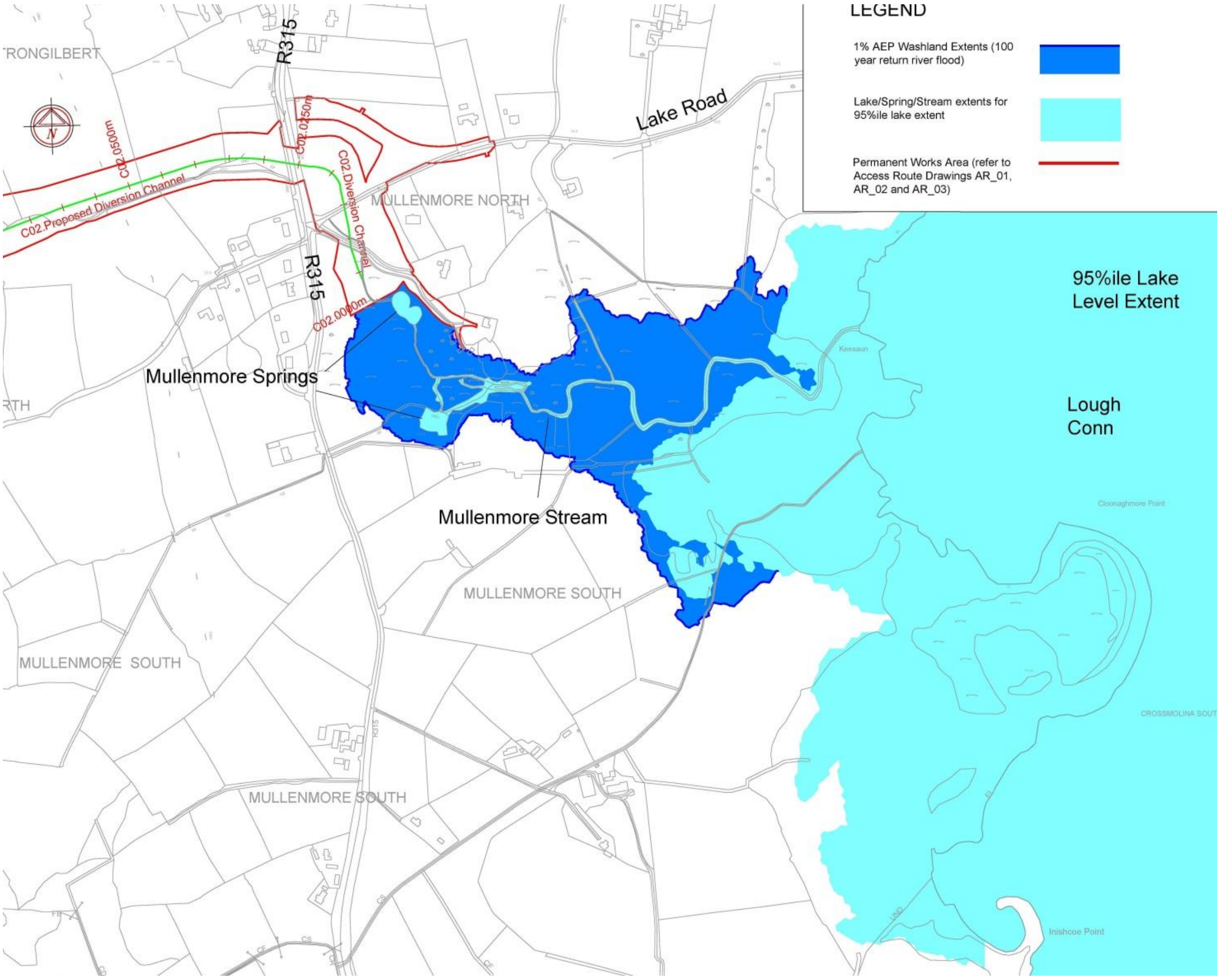

Figure 38 Computed Flood Extents in the Mullenmore for 100year design flood event with Lake level at 95% non-exceedance probability (winter lake level) (9.84mOd)

Note: The Lake extent represented by the cyan shading and additional lands flooded as a result of the diversion shaded in blue.

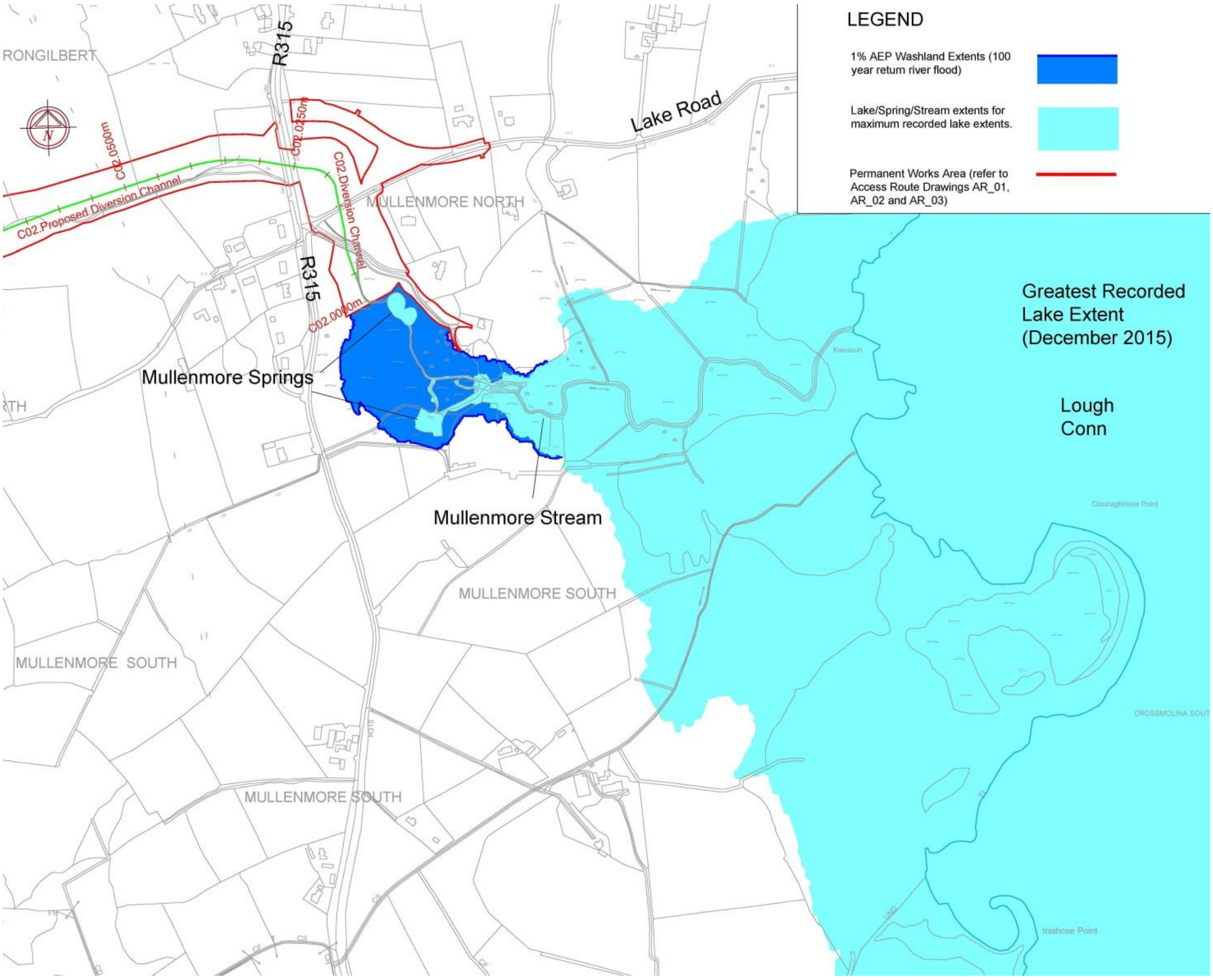

Figure 39 Computed Flood Extents in the Mullenmore for 100year design flood event with Lake level at the historical maximum flood level OSI mapped extents (11.6mOD)

Note: Lake extent represented by the cyan shading and additional lands flooded as a result of the diversion shaded in blue.

6. Conclusions

The recommended return period design flow estimates from the OPW drainage design section (OPW Memo 2018) have been reviewed with the most updated annual maxima flood flow series for the River Deel to Ballycarroon. It was concluded from this review that the OPW design flood flows fitting a Weibull distribution to the Ballycarroon Annual Maxima (AM) series provides an appropriately robust estimate of return period flood flows for the River Deel at the hydrometric gauge site and downstream at Crossmolina (located c. 2.5km downstream). The return period flood flow estimates were increased by 3% to account for the increased catchment area to Crossmolina.

It is concluded from hydraulic modelling of various diversion channel inlet options that it is not possible to satisfy the design flood flow objectives of at least retaining the 1year flood event in the Deel river and at the design 100year event not exceeding a maximum flow rate of 95cumec using a fixed inlet weir on its own.

Options involving an automated movable weir or fixed weir with flow control were found to be effective in achieving the required flood regulation. However a moving weir crest or gated inflow structure requires on going operational supervision and triggers to raise / lower the weir crest or open / close sluice gates which is not considered to be very desirable from a risk management perspective and particularly so given the very flashy nature of the Deel catchment. In any case diverting c. 50% of the 100year flood flow (i.e. 94cumec) into the diversion channel would result in lowering the flood level upstream of the inlet by almost 1.5m which would have significant undesirable consequences as it would double locally the channel velocity and give rise to significant local scouring and over time this scouring of the channel bed is likely to migrate upstream. Such scouring is likely to result is significant deposition in the channel section adjacent to the weir which would potentially alter the inlet flow relationship to the diversion channel.

The recommended inlet option is the combination of a flow control structure located in the Deel channel 150m downstream of the diversion and an in-stream, fixed crest height, 90deg, triangular plan form weir. The total length of the weir is 70m and weir crest height is set at 19.4m OD. The headwater bed level is set at 18m OD and the tailwater bed level is set at 16m OD. This configuration limits the drawdown effect on upstream flood levels and ensures through significantly restricting flows greater than the 1year flood that the additional flow favours discharging over the inlet weir to the diversion channel. As required the modelling shows that the almost 50-50 split in flows between the river and the diversion is achieved at the 100year design event. This configuration ensures that flows up to 71cumec are always retained in the river before the weir commences discharging.

The proposed flow control and diversion inlet will increase flood levels upstream of the flow control device for flood frequencies of up to the 1 in 5 year flood event. For return periods in excess of 5yr there will be a reduction in the upstream flood level with between 0.35 to 0.5m reduction predicted in the upstream reach at the 100year design flood event. Within the diversion channel some additional scour protection of the channel bed will be required in the vicinity of Pollnacross Bridge as indicated by the modelling.

The flood inundation impact of the diverted flows on the downstream Mullenmore washlands area has been evaluated through modelling with extensive lands flooded temporarily during the 100year flood event. A large portion of the lands downstream of the Mullenmore Mill building are inundated naturally by peak flooding from Lough Conn. The diversion is likely to activate most years given that spilling will commence when flow exceed 71cumec (c. 1year flood).

Importantly the inlet weir crest height can be adjusted post construction by lowering and raising a proposed steel weir crest plate that is to be attached to a reinforced concrete wall with an adjustment of c. ±200mm. The flow control structure openings are fitted with steel plates that can be raised or lowered to reduce or increase the open area for flow and thus alter the flow capacity of the structure which allows a degree of flexibility for fine tuning post construction the upstream flood level as a primary source of fine -tuning the required diversion channel inlet conditions.

These adjustments to the flow control structure and the weir crest height will allow for some hydrological uncertainty associated with channel roughness, weir equations and flow control stage-discharge relationships and also possibly the hydrological effects of future changes in channel characteristics caused by vegetation growth and sediment transport and deposition.

Channel maintenance in the form of removal of sediment deposition from the river channel between upstream of the inlet weir and the flow control structure and also in the inlet channel section to the diversion weir will be required on a intermittent basis so as to maintain the diversion inlet flow relationship.