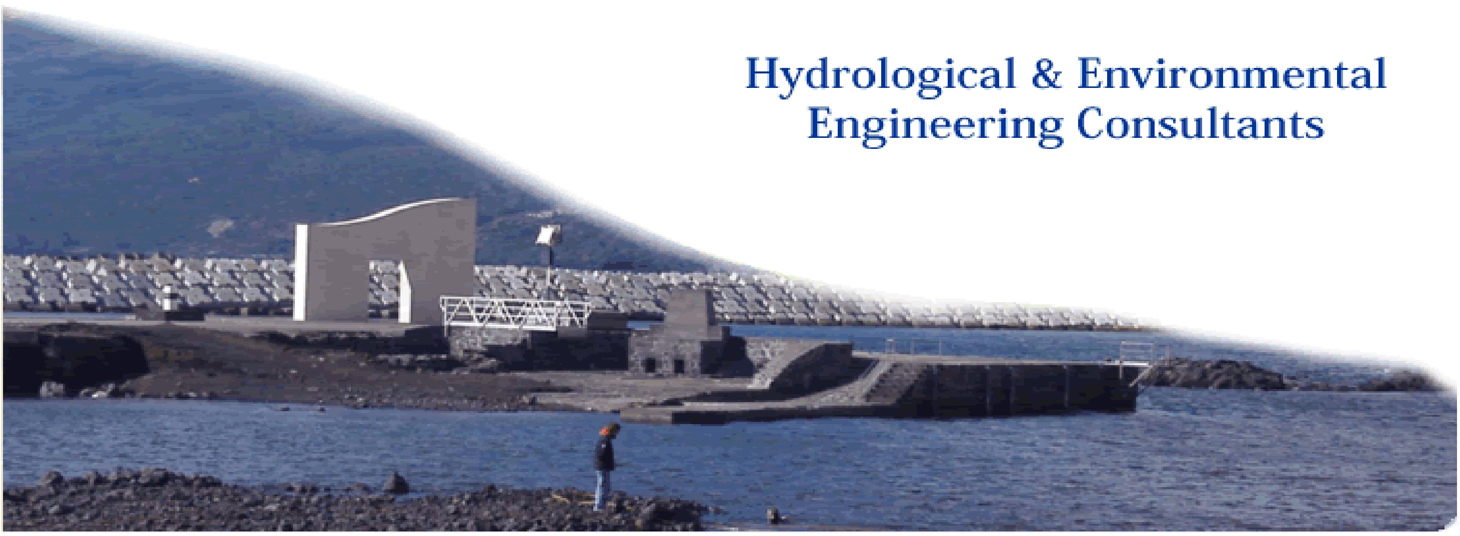
Crossmolina Flood Relief Scheme

Hydraulics Report
for the
River Deel Flood Diversion Channel

On behalf of the

OPW

September 2020



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1. Introduction

1.1 Background

Hydro Environmental Ltd was appointed by Ryan Hanley consulting engineers on behalf of the OPW in 2019 to examine in detail the flood hydraulics of the upstream reach associated with the proposed diversion channel and inlet weir structure for the River Deel (Crossmolina) Flood Relief Scheme and advise on the hydraulic design required to achieve the necessary diversion of flows under flood conditions.

1.2 Flood Diversion Channel Option

The design for the flood diversion channel is to retain in the existing Deel River channel through Crossmolina approximately flood flows up to the median annual maximum flood rate and to spill and divert remaining flow upstream of the town into a proposed engineered diversion channel to eventually reach Lough Conn a distance of 1.8km away. The proposed inlet location to the diversion channel in the townland of Cartrongilbert, 850m upstream of the Jack Garrett Bridge in Crossmolina Town as shown below in Figure 1. This represents a significant reduction in the travel distance to Lough Conn, which would otherwise involve a longitudinal distance along the existing meandering Deel River channel of c. 12Km heading firstly northwards then east and finally south to reach Lough Conn, refer to Figure 2 below.

Figure 1 Aerial View of Deel River at proposed diversion spillway upstream of Crossmolina Town

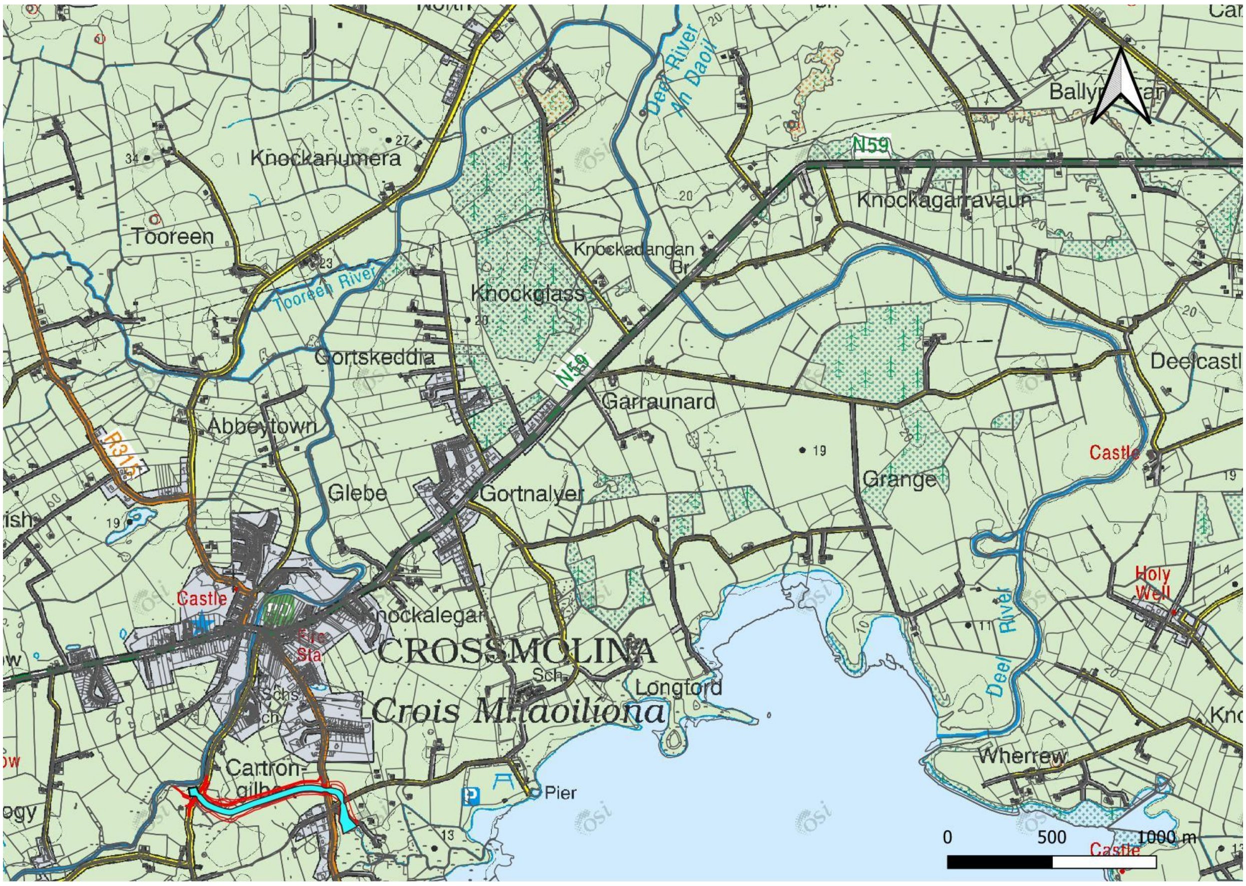

Figure 2 Proposed Diversion Channel Route to Lough Conn < 1km south southwest of Crossmolina Town Centre.

The diversion channel is designed as a grassed channel has a trapezoidal section of 28m base width, 1:2 side slopes and longitudinal gradient of 1 in 1000. producing a flow depth of 1.96m and a flow velocity of 1.50m/s for the 100year flood event with 94cumec in the diversion channel upstream of the Drop Structure. A limit of 1.6m/s average cross-sectional channel velocity was adopted to avoid erosion of the grassed channel and its banks, . Chow gives a Manning's n of 0.027 for a regular engineered channel with short grass and few weeds, a Manning's n of 0.03 for grass and some weeds and 0.035 for grass and dense weeds. For these roughness coefficients and the proposed channel geometry the normal flow depth and average cross-sectional velocity is presented below in Table 1

Table 1 Computed Normal Depths and Velocities in Proposed Diversion channel for the 100year flood event and for different Manning roughness coefficients depending on the channel vegetation conditions

Manning's N	Normal Depth	Channel Velocity
0.027 Short grass and few weeds	1.84 m	1.61m/s
0.030 Grass and some weeds	1.96 m	1.50m/s
0.035 Grass with Dense weeds	2.14	1.36m/s

The total longitudinal length of the diversion channel from the inlet weir to downstream of its drop structure at its exit to the washlands is c. 960m. The diversion channel length between the inlet weir and the drop structure (also referred to as the Energy Dissipation Structure) is 820m. The function of the drop structure located towards the downstream end of the diversion channel is to maintain sufficient flow depth in the diversion channel upstream in order to limit flow velocities and potential scouring of a grassed channel under design flood conditions. The drop structure is designed to choke and drop the water level in the channel over a relatively short length within a scour protected concrete channel with energy dissipation so as to safely take the water from the resultant level in the upstream channel to the lower level in the downstream section of the channel and protect by lowering flow velocities entering the downstream washlands. The diversion channel and drop structure was designed by Professor Micheál Bruen of UCD. ~~refer to Hydraulics Report for Diversion channel March 2020~~. Downstream of the Drop Structure, energy dissipation in the form of a series of anchored reinforced concrete dissipation blocks and a gradually increasing channel width transition was designed to protect the downstream washlands by increasing flow depth and reducing flow velocity at the exit from the diversion channel.

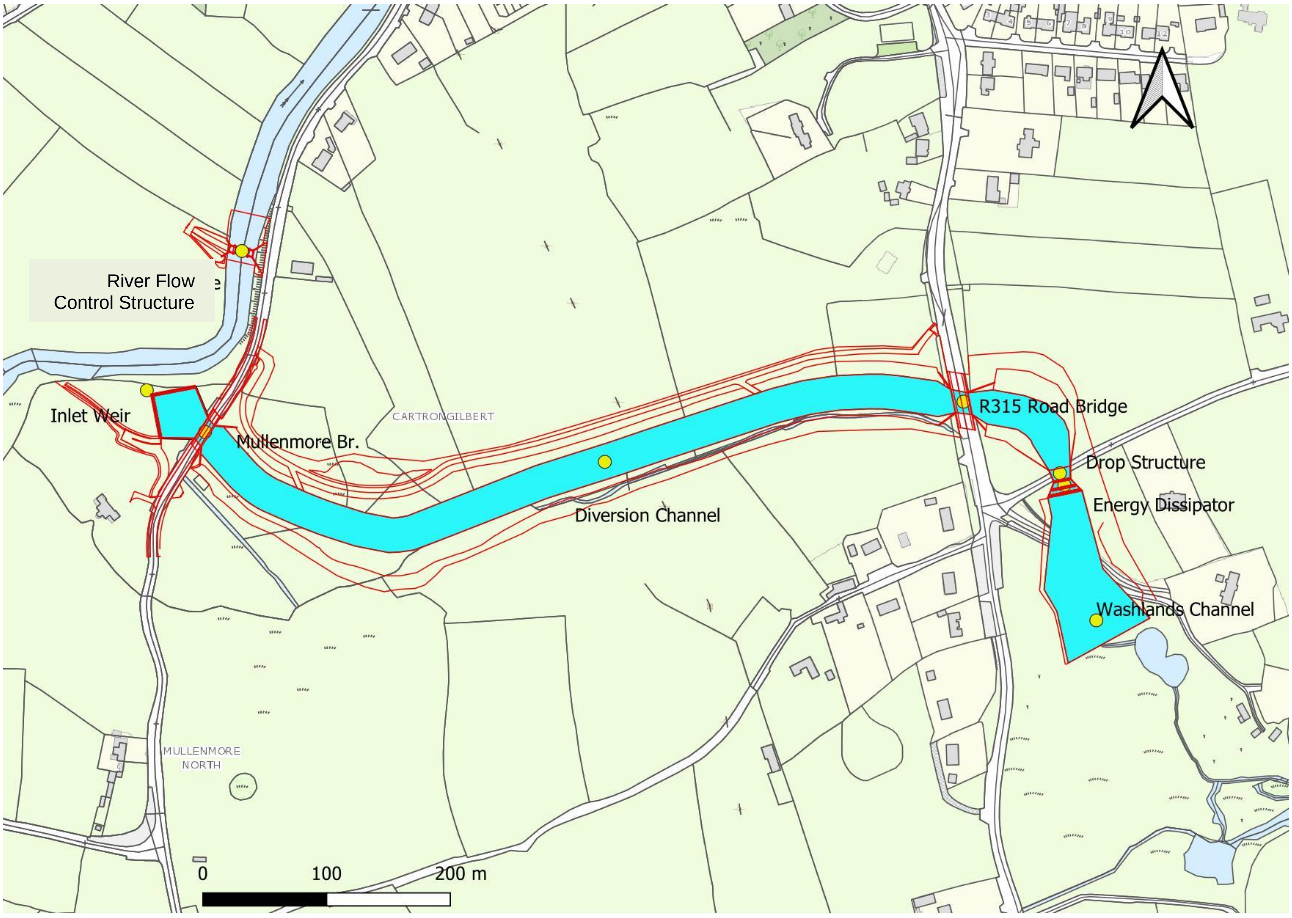

Figure 3 Diversion Channel Structures

1.3 Inlet Weir Configurations for the Diversion of Flood Flows

In the preliminary design of the proposed flood diversion channel a number of diversion inlet options were considered in respect to controlling flows between the river channel and diversion channel. In order to retain average / median flood flow in the existing river either a gated structure or a weir is required. A fully automated gated structure requires opening of gates or the lowering of a weir crest at a prescribed river stage level associated with the median flood. This option was considered not to be desirable for the following reasons:

- It generates excessive velocities in the river channel as the upstream water level is drawndown significantly in order to achieve the desirable c. 50-50 split between retained river flood flow and diverted flow
- It requires requiring on-going supervision which is considered potentially risky in the event of a failure or delay in the opening of the gates/ lowering of the weir crest.
- Such a gated scheme would require sophisticated high resolution flood forecasting capability so as to pre-empt the flood event, which is difficult given the flashy nature of the River Deel catchment.

A fixed weir structure was considered to be a safer and more robust option which would spill once water levels surpassed the weir crest. The two initial weir options considered were the construction of a long lateral (side) weir that ran along the river channel bank edge for c. 100m and a second weir option was an in-stream weir within the diversion channel combined with a wide approach entrance from the river channel edge to the weir. The crest of these weirs was fixed and the crest level set such that the median flood (i.e. 2yr flood = 81cumec) is retained in the Deel River through Crossmolina and that at the 100year return period design flood event no more than 95cumec (c. 50% of the 100year flood peak) would discharge in the Deel river channel through Crossmolina, with the diversion channel taking the remainder. This represents a very narrow window of flows above the median flood flow during spilling with only 9 to 14cumec for the existing channel and 0 to 98cumec in the diversion channel within the spilling range. Both weir proposals had a steel weir crest plate attached to a reinforced concrete weir wall that can be adjusted up and down to allow for uncertainty in inlet water level conditions.

The conclusions reached from a review of these weir options as part of this study was that there is a high degree of uncertainty associated with the conveyance characteristics of the river channel and predicted flood levels at the proposed diversion inlet site to establish the required final weir crest level that would achieve the design objectives. Hydraulic analysis showed that the spill rate of both weir options was very sensitive to the resultant headwater flood level which was in turn very sensitive to the downstream channel conveyance. A further complication was the potential for deposition in the channel adjacent to the weir and in the downstream channel below the weir due to the decreased flood flow velocities as a result of the diversion of flows. Such conditions if not maintained would alter significantly the stage flow relationship at the weir site.

The ability after a sufficient period of monitoring post construction to adjust the weir crest plate on both options allows for better refinement and may provide for the potential for further adjustment should the need arise at a later stage.

The lateral / side weir option will promote local deposition of river sediments in the river channel adjacent to the weir face during large flood events as a result of the reduced velocity from upstream to downstream along the side weir. Such deposition will require management and could during a flood alter the flow level relationship at the weir. The in-stream weir, set in from the river channel, at the head of the diversion channel provides an opportunity for introducing a sediment trap away from the main river channel that can be cleared as required.

It was also concluded that an in-stream flow control structure within the river channel downstream of the inlet weir that would throttle, as required, the flood flows in the river would provide better hydraulic certainty and better conditions for directing flow over the inlet weir and into the diversion channel and would also control flood levels upstream which would otherwise have to significantly drop in order to achieve the desired split between the river and diversion channels.

1.4 Drop Structure and Energy Dissipation

A number of options in respect to the design of the drop structure and energy dissipater were considered by Prof. Bruen investigating a vertical drop structure, a skew structure on the channel bend near the R315 Road Crossing and the final recommended structure is on a straight section of channel downstream of the Bend at the R315. A choke section in the form of a concrete flume section is associated with this drop structure / energy dissipater to maintain relatively high flood flow depths in the channel upstream. The proposed flume width at its narrowest is 14m and the opening width is 17.3m. The downstream channel from the drop structure has a gradually expanding width stilling basin to allow further energy reduction through expansion of flow and corresponding reduction in exit velocities. Within the concrete drop structure and channel which is to be rectangular in profile a series of reinforced concrete floor blocks for energy dissipation will be provided to control and return the flow regime from supercritical flow to subcritical flow within the protected concrete channel area prior to discharging to the washlands.

1.5 Proposed Bridge Structures on Diversion Channel

There are two road crossings required along the proposed diversion channel, the first is towards the upstream section of the diversion channel approximately 50m downstream of the inlet weir which facilitates the local Pollnacross Road (L1101) crossing (referred to as Pollnacross Bridge). This is to be a reinforced concrete bridge supported on vertical abutments having a 20m span width and bridge soffit levels of 20.29m OD at the Left abutment, 19.67m OD at the right abutment and 19.98m OD mid-span. The current design standard required by the OPW for Section 50 consent for a proposed watercourse bridge structure under the arterial drainage act of 1947 is the 100year flood with 20% climate change. These proposed soffit levels provide over 1m clearance at mid-span over the design flood level (100year + 20% climate change).

The second bridge structure is at the downstream end of the channel 100m upstream of the proposed drop structure at Mullenmore North and will facilitate the R315 Pontoon Road crossing (referred to as Mullenmore Bridge). This is a reinforced concrete bridge supported on vertical abutments having a 20m span width and bridge soffit levels of 19.8m OD at the Left abutment, 19.25m OD at the right abutment and 19.98m OD mid-span which provides at least 1m clearance at mid-span over the design flood level (100year + 20% climate change).

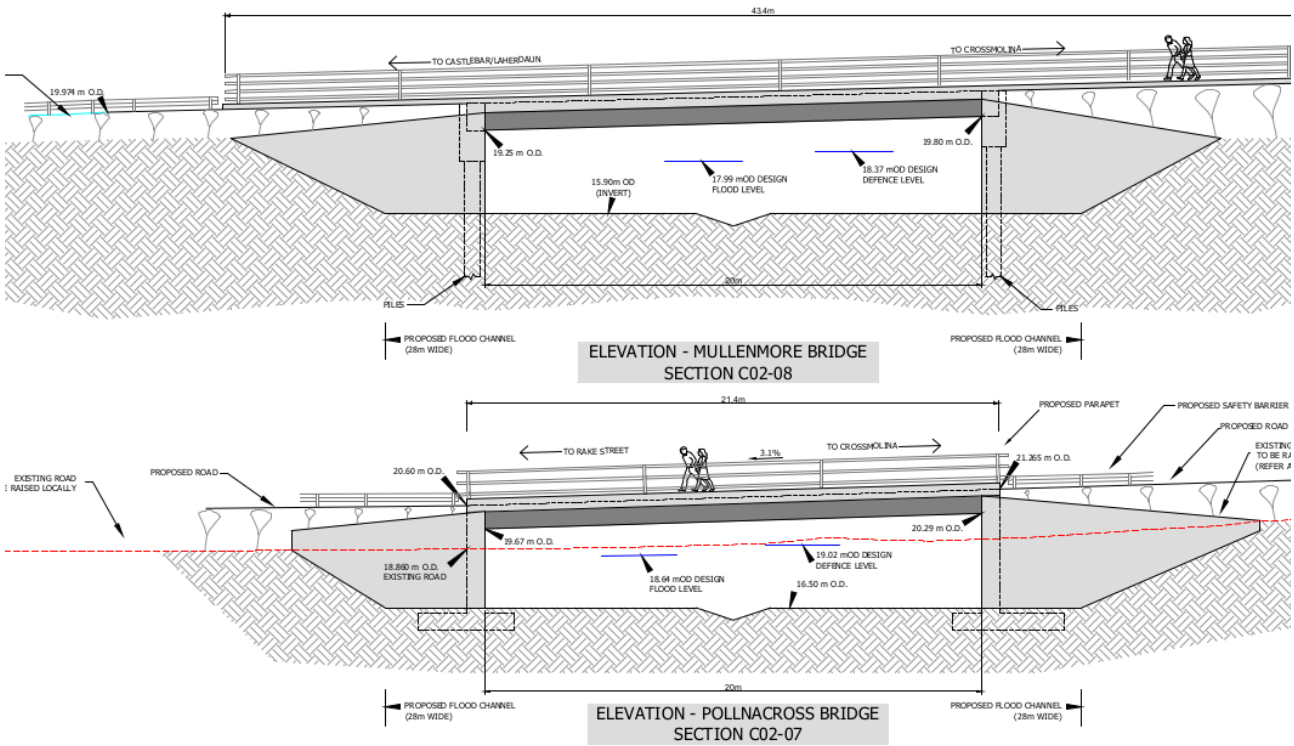

Figure 4 Proposed Diversion Channel Bridge Cross-Sections looking upstream At Mullenmore and Pollnacross Bridges

2. Design Flood Flows in River Deel to Crossmolina

2.1 Introduction

The design flood flows for the scheme were reviewed by the OPW Design Section in 2016 (refer to OPW Design section Memo - 19th December 2016, included as Appendix A to main report) and included the large flood event recorded on the 5th December 2015. This updated Annual maximum series for the River Deel at Ballycarroon Gauge (34007) located a few kilometres upstream of Crossmolina Town used an updated rating curve for the gauge based on the on the Design Section Memo " 2016-11-22 - TJ - Station 34007 Ballycarroon - Flow Measurements & Rating Curves". The frequency analysis review by the OPW concluded that the Weibull distribution (EV2) produced the best fit to the data. The OPW review also investigated trends in flood flows over the record length and concluded that the AM series was reasonably stationary over the full record length. The 66year AM Series recorded at Ballycarroon gauge is presented in Figure 6.

The recommended return period flows at Ballycarroon gauge by the OPW Design Section are based on Single Site Frequency analysis fitting a Weibull distribution to the annual maximum flow series and do not include the use of a pooling group for estimating the larger return period events.

The flow estimates for the three largest flood events recorded at Ballycarroon gauge are presented below in Table 3. The largest recorded flood in at least a 67year period is the December 2015 flood having an estimated peak flow of 178.2cumec which statically represents approximately an 85year return period event. The second largest event recorded on the 27th October 1989 produced a peak flow of 142.6cumec and is estimated to represent approximately a 20year return period flood event.

Table 2 The three largest Flood Events Recorded in the Deel River at Ballycarroon

Rank	Peak Flow Estimate (cumec)	Peak Flood Level mOD	Date
1	178.2	23.82	5th December 2015
2	142.6	23.44	27th October 1989
3	118.3	23.16	3rd December 2006

2.2 Flood Frequency Analysis

As part of this assessment the design flows were further reviewed to include the most recent hydrometric years and fitting a number of statistical distributions to the data using l-moments for comparison the OPW Weibull Fit, refer to Figure 5 and Table 3 below.

Single Site analysis fitting EV1 Extreme Value Type 1 (Gumbel), GEV General Extreme Value (EV3 – Weibull), GLO General Logistic and LO Logistic by L-moments and Gringorton EV1 fit were carried out. The adopted estimates from Tim Joyce of the OPW (GEV – Weibull distribution) was also included for comparison, refer to Figure 4 distribution fits to the 66year annual maxima series of flows at Ballycarroon gauge. As is generally the case various distribution type and fitting methods produce different results, particularly with increasing return period (i.e. increasing y-variate = -ln(-ln(1-1/T) on the X-axis. The recommended OPW Estimates represent a good fit to the AM data and agree reasonably well with the EV1 and GEV l-moment estimates (just slightly higher). The GLO distribution is more skewed and tends to curve upwards away from the observed data. For Irish catchment the Extreme Value type distributions are generally recognised as providing better fit to annual maxima flows.

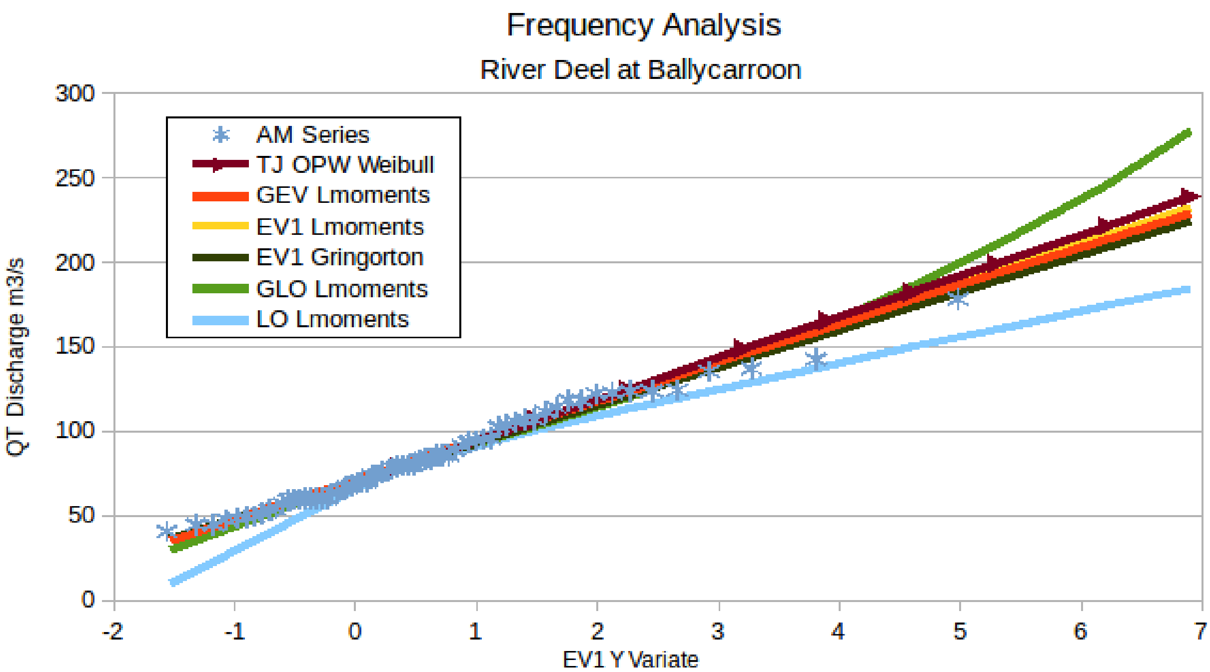

Figure 5 Updated Frequency Analysis using Annul Maximum Series of Flows 1953 to 2019 for River Deel at Ballycarroon (Stn 34007)

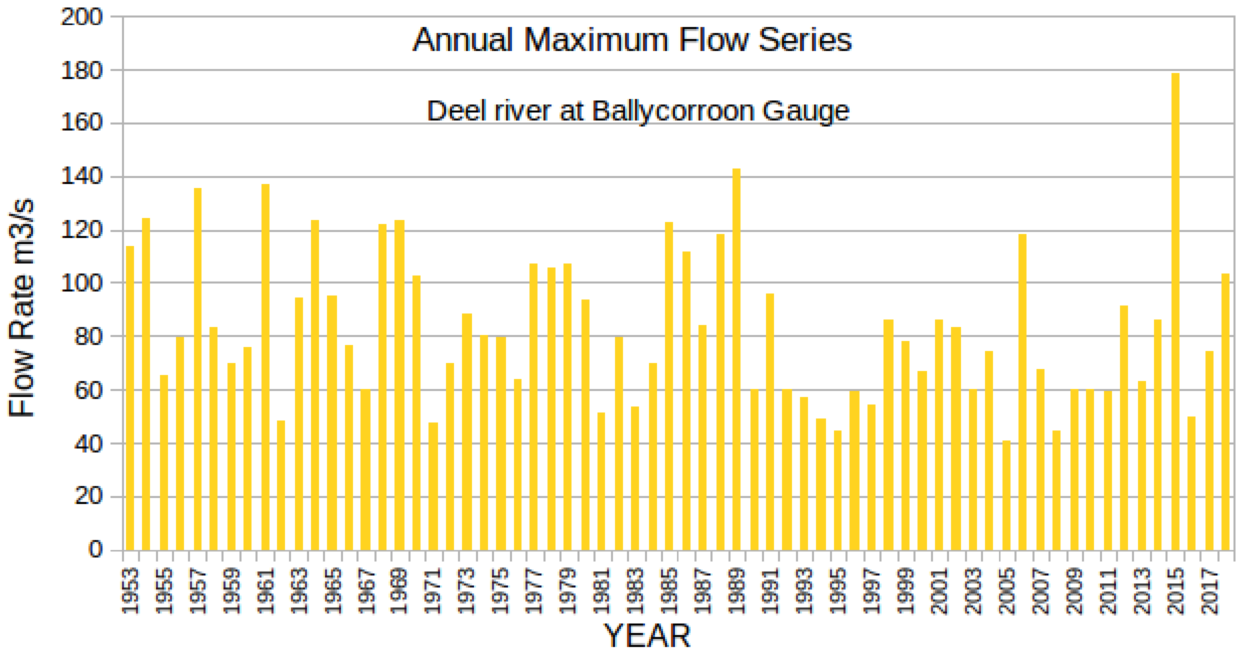

Figure 6 66 year Annual Maximum Flow Series (1953 to 2019)

Table 3 Return Period Flood Flow estimates for Ballycarroon gauge using various distributions and fitting method

Return Period	EV1 Gringorton	EV1 Lmoments	GEV Lmoments	GLO Lmoments	LO Lmoments	OPW Weibull
2	78.8	79.6	79.6	79.6	79.6	78.6
5	104.1	106.0	106.0	103.5	100.7	106.3
10	120.9	123.6	123.4	120.3	113.1	124.7
25	142.0	145.7	145.2	143.8	128.0	148.8
50	157.7	162.1	161.3	163.5	138.9	165.5
100	173.3	178.4	177.2	185.3	149.6	182.4
200	188.8	194.7	193.0	209.6	160.3	199.2
500	209.3	216.1	213.6	246.1	174.3	221.6
1000	224.8	232.3	229.2	277.7	184.9	238.5

It is concluded from this review that the OPW design flood flows fitting a Weibull distribution to the Ballycarroon AM series provides an appropriately robust estimate of return period flood flows for the River Deel at the hydrometric gauge site and at Crossmolina located c. 2.5km downstream. The return period flood flow estimates should be increased in flow magnitude by 3% to account for the increased catchment area to Crossmolina and are summarised below in Table 4.

Table 4 Return Period Flood Flows for River Deel

Return Period T Years	Flood Peak Discharge at Ballycarroon (34007) m3/s	Flood Peak Discharge at Crossmolina m3/s
2	78.6	81.0
5	106.3	109.5
10	124.7	128.4
25	148.8	153.3
50	165.5	170.5
100	182.4	187.8
200	199.2	205.2
500	221.6	228.2
1000	238.5	245.7

The OPW Section 50 design flood flow rate for bridge structures is a total flow in the Deel River of 244cumec. This includes the 100year return period flow of 187.8 cumec, the statistical standard error estimate plus a 20% Climate Change allowance. Therefore the diversion channel and bridge structures for OPW Section 50 consent should be capable of conveying a potential diverted flow of 139 to 142cumec (with 92 to 95cumec retained in the Deel River).

3. Hydraulic Modelling

3.1 Introduction

TELEMAC2D is the hydraulic software of choice for modelling the complicated hydrodynamics of the Deel River at the proposed diversion inlet requiring high refinement and variable meshing capability. TELEMAC2D is an integrated, user friendly software system for free surface waters. TELEMAC2D was originally developed by Laboratoire National d'Hydraulique of the French Electricity Board (EDF-LNHE), Paris. It is now under the directorship of a consortium of organisations including EDF-LNHE, HR Wallingford, SOGREAH, BAW and CETMEF. It is regarded as one of the leading software packages for free surface water hydraulic applications worldwide.

The TELEMAC system is a powerful integrated modelling tool for use in the field of free-surface flows. Space is discretised in the form of an unstructured grid of triangular elements, which means that it can be refined particularly in areas of special interest. This avoids the need for systematic use of embedded models, as is the case with the finite-difference method. Telemac-2D is a two-dimensional computational code describing the horizontal velocities, water depth and free surface over space and time.

3.2 Model Development

A detailed topographical survey of the Deel River channel in the vicinity of the proposed diversion inlet site was carried out by Ryan Hanley Consulting Engineers in August 2018. This survey coincided with a significant drought period which saw the entire river channel completely dry out and this allowed access to the river bed by foot for topographical surveying purposes. This survey was further augmented by a detailed overbank topographical survey carried out in Mid-October on both north and south riverbanks. The surveyed reach length presented in Figure 7 is c. 650m in length. A contour map of this survey information as represented in the model is presented in figure 8.

The OPW CFRAM cross-sectional (at c. 50m intervals) survey information and overbank lidar data (2m grid) was also used to describe the downstream river reach to Crossmolina Bridge. 1-dimensional Hec-Ras Modelling of the Deel River from 400m upstream of the Diversion channel to Lough Conn was also performed so as to predict changes in the channel hydraulics as a result of the proposed diversion of flows.

A number of 2-Dimensional models of different boundary-fitted triangular meshing were constructed. Telemac Model 1 was constructed to examine the initial weir options without any flow control structure within the river channel, refer to Figure 2 and 9. Telemac Model 2 was constructed for the final recommended inlet and diversion channel option and extended downstream of the diversion channel to include the Mullenmore North washlands to Lough Conn.

Telemac Model 1 includes the proposed 950m long diversion channel, and the 1.2km length of the existing River Deel channel from c. 400m upstream of the diversion to 15m upstream of Crossmolina Bridge, refer to Figure 9 for domain extent.

The existing River Deel channel downstream of the inlet reach is predominantly longitudinal 1-dimensional flow with little secondary and traverse flow components. This condition also applies to the diversion channel and consequently is modelled using elongated triangular elements of 15m (in the longitudinal direction) by 2m (in the transverse direction). In the 650m inlet reach of the river where 2-dimensional flow is prevalent a regular 2m triangular mesh is applied.

Figure 7 Spot levels of Deel River channel and Overbanks in the vicinity of the proposed Diversion Inlet Location

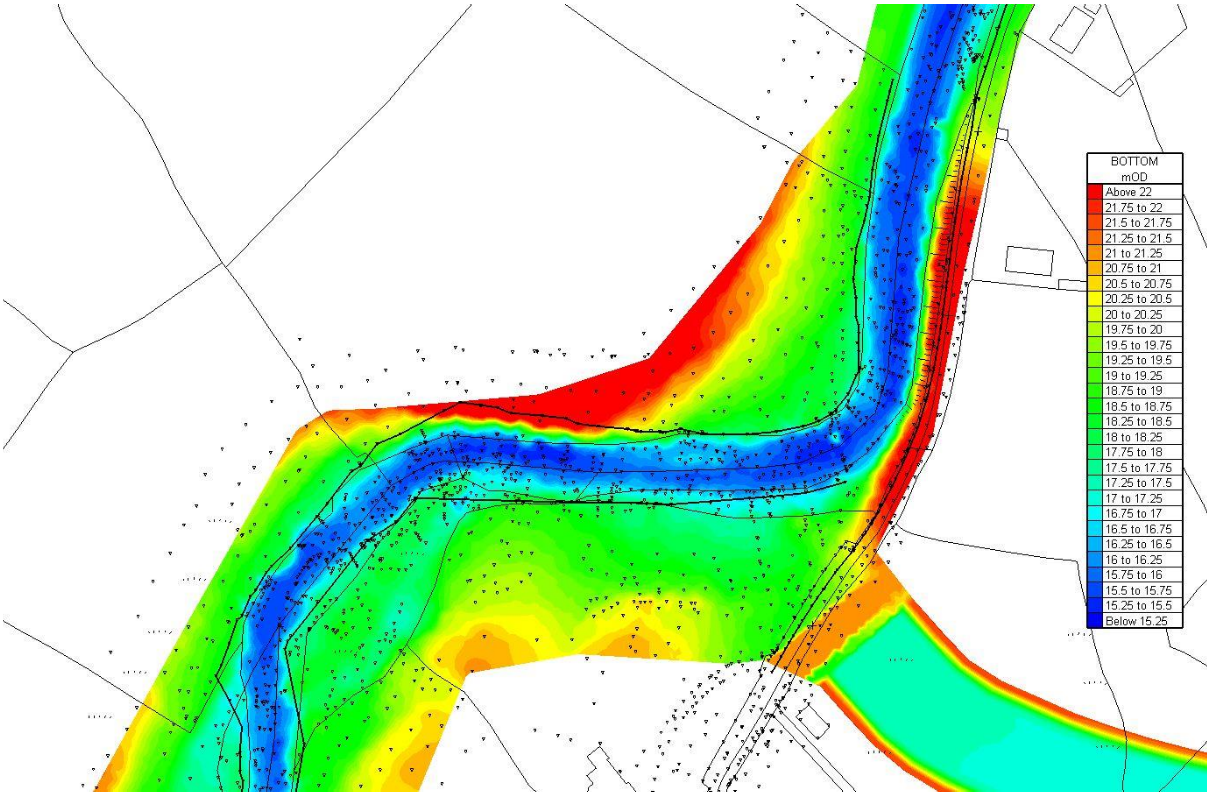

Figure 8 Contour map of channel and overbank ground elevations for the existing Case.

The downstream boundary condition for the diversion channel is water elevation which is influenced by the water level in Lough Conn. This downstream boundary condition does not affect the model results upstream of the drop structure on the diversion channel and does not influence the flow rate entering the channel from the River Deel.

3.3 Channel Roughness

The Manning's Roughness coefficient n for the main river channel was set at 0.045 and the Manning's n for the new diversion channel is set at 0.035. These roughness coefficients are considered reasonably representative coefficients.

A lower coefficient of 0.03 could be used for the diversion channel. A reduction in the roughness coefficient will influence the flow depth and velocity (reduction in depth and corresponding increase in velocity) within the diversion channel and will not influence the hydraulic control at the inlet weir and the subsequent flow distribution between the Deel River and the diversion channel, being strongly influenced by the diversion Inlet weir geometry and the roughness and conveyance capacity of the downstream reach of the River Deel. The final choice of Manning's n for the diversion channel will depend on how vegetation within the channel is maintained and cut-back, etc.

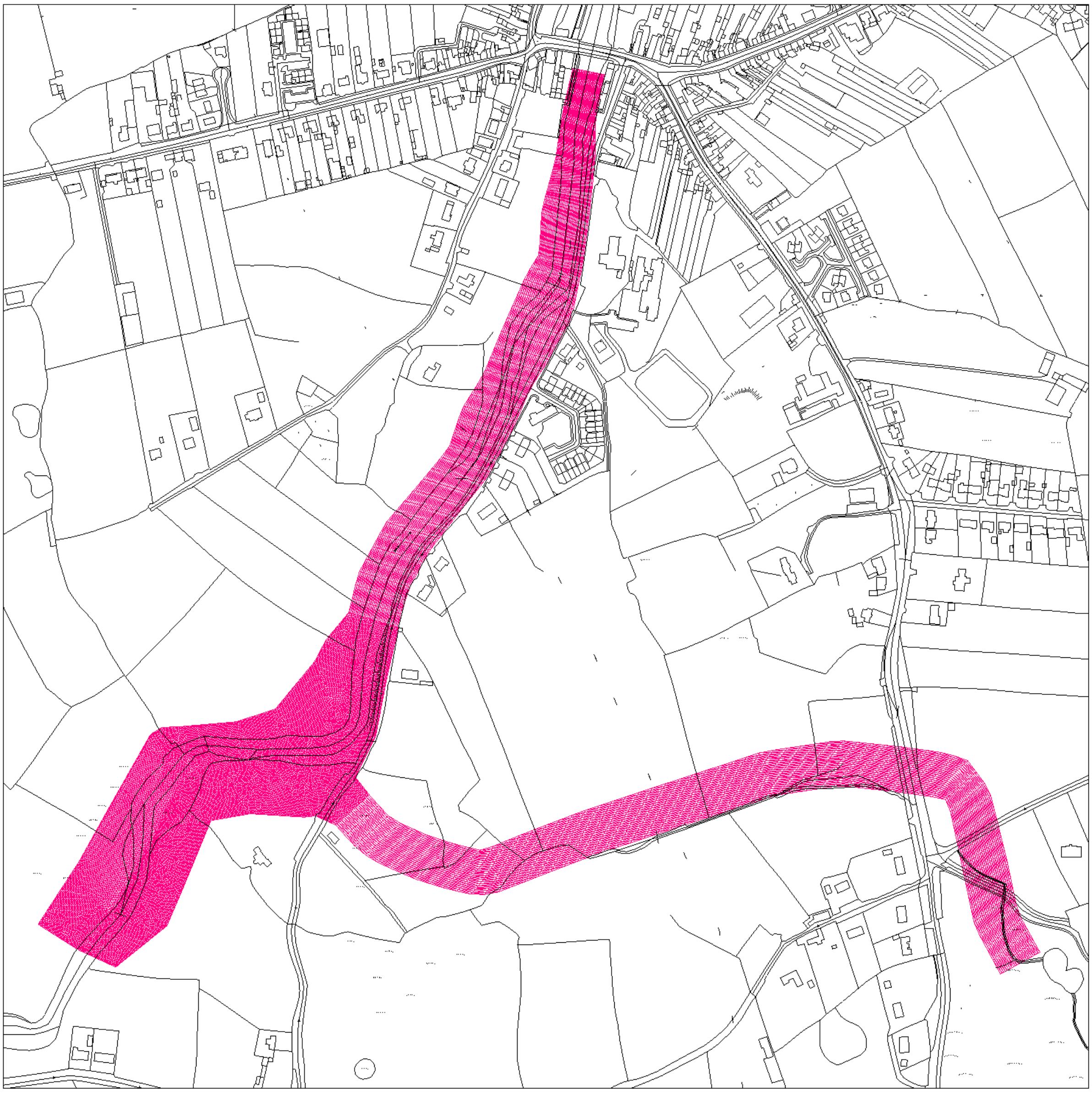

Figure 9 Telemac2D model domain Extent and Variable Mesh Structure.

3.4 Model Boundary Conditions

A flood stage-discharge rating relationship for the Deel River at Crossolina Bridge was developed using recent gauged hydrometric data (gauge 34119) for the October 2018 flood event and historical Information for the December 2015 flood event and the JBA Calibrated 1D/2D hydrodynamic model. The flood flow estimates for these events was obtained from the Ballycarroon Gauging Station (34007) and increased by 3% to allow for the slightly larger catchment area to Crossmolina.

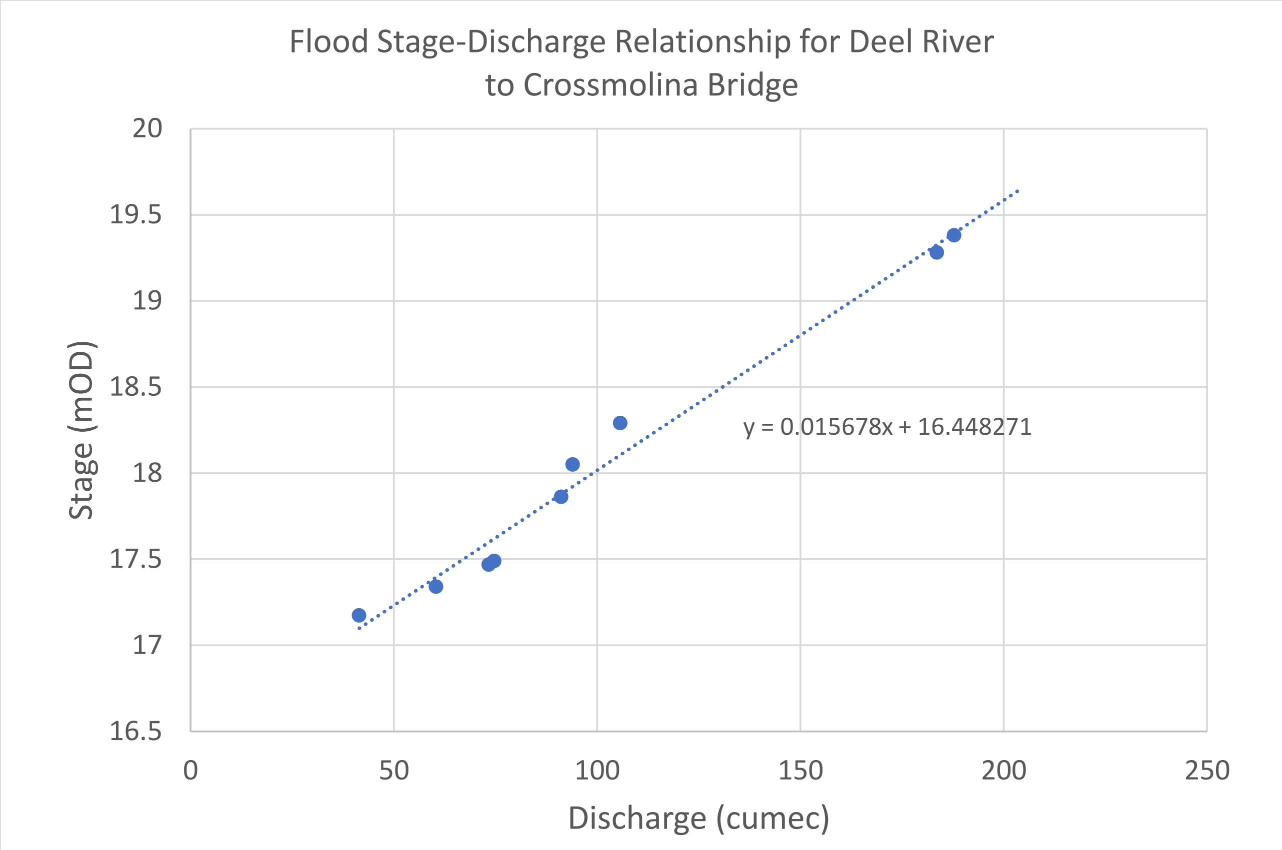

Figure 10 Rating Relationship for Deel River at Crossmolina Bridge

3.5 Model Calibration

Hydrometric gauges on the River were Installed (June 2018) to capture the River Levels at Jack Garret Bridge (34119) and also upstream and downstream (gauges 34120 and 34121) of the proposed diversion channel inlet weir site. This gauging data combined with the Ballycarroon gauged flow estimates were used for model calibration and verification purposes.

The model calibration used the recent gauged floods at Crossmolina recorded on the 9th October 2018 and on the 31st August 2019. These flood events are estimated to be 3year and 4.5year return period respectively. The computed and measured peak flood elevations are presented below in Table 1 for the two events. The calibrated flood levels are generally within 0.12m which is considered to be acceptable agreement for such modelling.

Table 1 Model Calibration Results

GAUGE	34007	34119(d/s face)		34121		34120	
Date	Discharge (cumec)	Measured mOD	Computed mOD	Measured mOD	Computed mOD	Measured mOD	Computed mOD
9 Oct 18	93.1	17.87	17.92	19.15	19.25	19.17	19.29
31 Oct 19	103.7	n/a (1)	18.09	19.44	19.48	19.46	19.51

(1) Note gauge location changed from downstream to upstream of crossmolina bridge and not available for Oct 2019.

There are no recorded peak flood levels for the extreme floods of the 5th December 2015 and 27th October 1989 at the proposed Diversion Inlet Site. The hydraulic modelling exercise predicts flood levels associated with these flood events of 20.61m OD and 20.15m OD respectively at the proposed weir inlet to the diversion channel.

4. Hydraulic Simulations of Diversion channel inlet options

4.1 Description

This exercise was carried out to determine diversion inlet requirements that would enable the median flood to remain within river channel downstream of the diversion and at the 100year design flood flow the retained flow within the Deel would not exceed 95cumec and without any downstream flow control structure within the Deel River to assist the diversion flows.

The 100year return period peak flow specified in the hydraulic simulations 187.8cumec. The objective is to retain up to the 2year flow within the existing channel and at the 100year design flood achieve an almost equal (50 : 50) split in flow magnitudes between the diversion and river channels. The model simulations included the existing case as the baseline case and various diversion channel inlet configurations. The following simulations were performed for free inlet conditions (i.e. no weir) to the diversion channel

4.2 Hydraulic Simulations

Before examining weirs, inline or lateral the following free inlet scenarios (without any weir structure) were examined in order to demonstrate proportion of flow that could enter the diversion channel at different entrance inverts and what implications the diversion would have on flood level and velocity within the river channel at the inlet.

1. Existing Case no Diversion Channel Conveyance to provide Baseline conditions
2. Wide Inlet entrance off the Deel River set at an invert of 17.3m OD and discharging to a 30m base width 1 in 1000 gradient trapezoidal diversion channel (refer to Figure 11)
3. Wide Inlet entrance off the Deel River set at an invert of 16.5m OD and discharging to a 30m base width 1 in 1000 gradient trapezoidal diversion channel (refer to Figure 12)

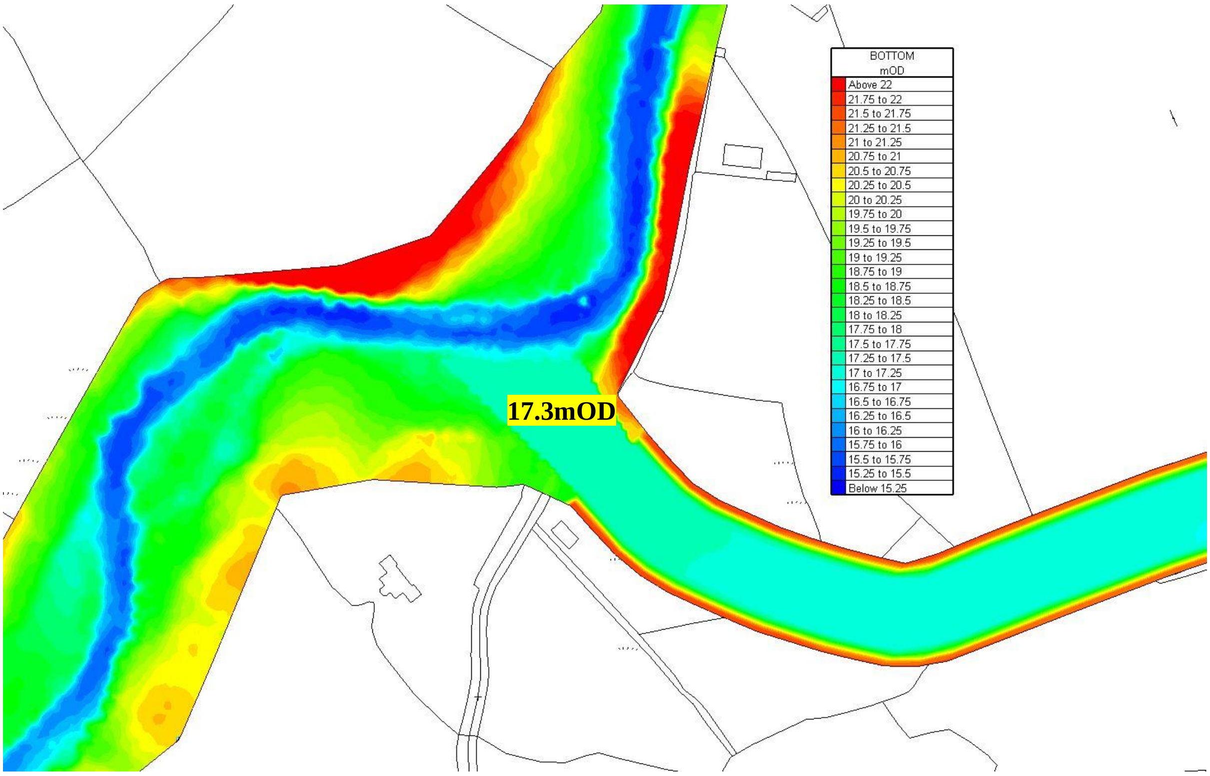

Figure 11 Configuration of 30m wide diversion channel at 1 in 1000 gradient with large inlet section at 17.3m OD invert level

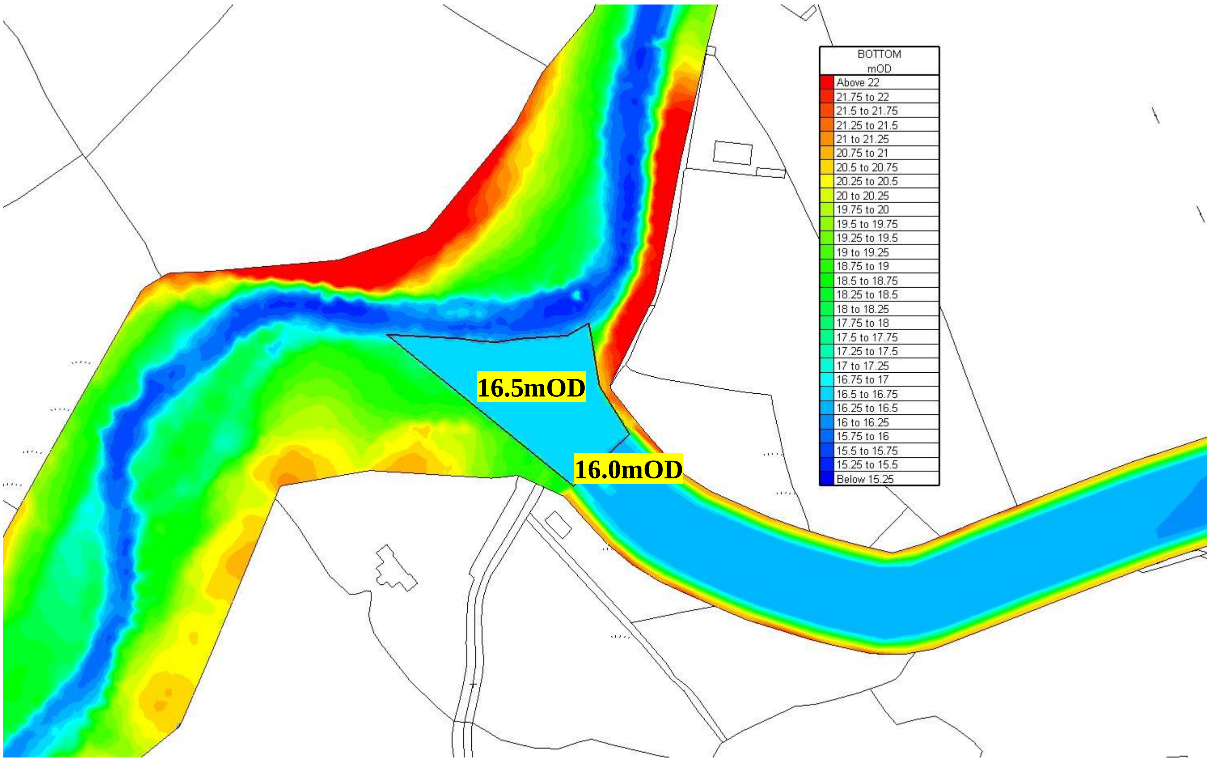

Figure 12 Configuration of 30m wide diversion channel at 1 in 1000 gradient with large inlet section at 16.5m OD invert

4.2.1 Existing Case

The first simulation to represent the baseline conditions was for the existing situation (do nothing scenario) without the diversion. Figure 13 presents the velocity plot at peak flood flow in the vicinity of the proposed diversion inlet. The predicted flood level in the river adjacent to the inlet is 20.67m OD and the computed downstream flood level at Crossmolina Bridge is 19.38m OD Malin.

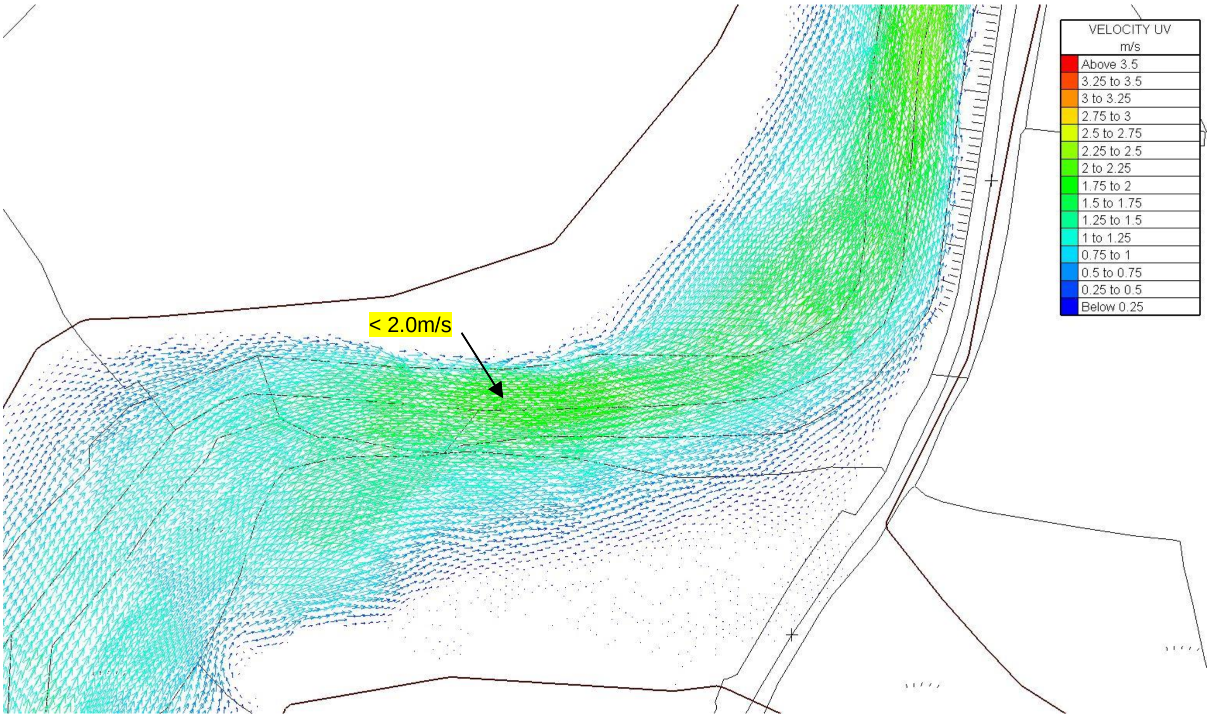

Figure 13 Predicted Flow Velocities at Peak of 100year Flood for case 1 (Existing River)

The predicted Maximum Velocity in the Main Channel just upstream of the proposed diversion inlet is approximately 1.75m/s. The velocity plot also shows the preference for overbank flow on the left bank downstream of the proposed diversion at the 90degree bend availing of the shorter path length.

4.2.2 Inlet to diversion channel at invert level of 17.3m OD

This option considers retaining a bank level of 17.3m OD (1.2 to 1.4 above talweg level of the river) and funnelling gradually to a 30m wide diversion channel laid at a 1 in 1000 gradient. Figure 14 presents the velocity plot at peak flood flow in the vicinity of the inlet. The predicted flood level in the river adjacent to the inlet is 19.22m OD. This configuration (without any weir) will divert 43.3% of the peak flow to the diversion channel and 56.7% remains in the existing river through Crossmolina.

The maximum predicted velocity in the Main Channel just upstream of the proposed diversion inlet is slightly greater 3m/s which represent a marked increase over the existing case of 1.75m/s. The velocity plot shows a drawing of the flow direction further southwards to coincide with the channel orientation and the development of an anticlockwise gyre on the north bank which limits the out of bank flow on the north side of the river and is in direct contrast to the existing case where overbank flow on the northern bank is significant.

The lowering of the Flood Level by over 1m reduces significantly the depth and magnitude of out of bank flows and increases significantly the in-channel velocity upstream of the diversion from c. 1.75m/s for the existing case to c. 3 m /s for the proposed case.

This case does not achieve the desired c. 50 : 50 split in flood flow rates between the Diversion channel and the existing channel with too little being diverted.

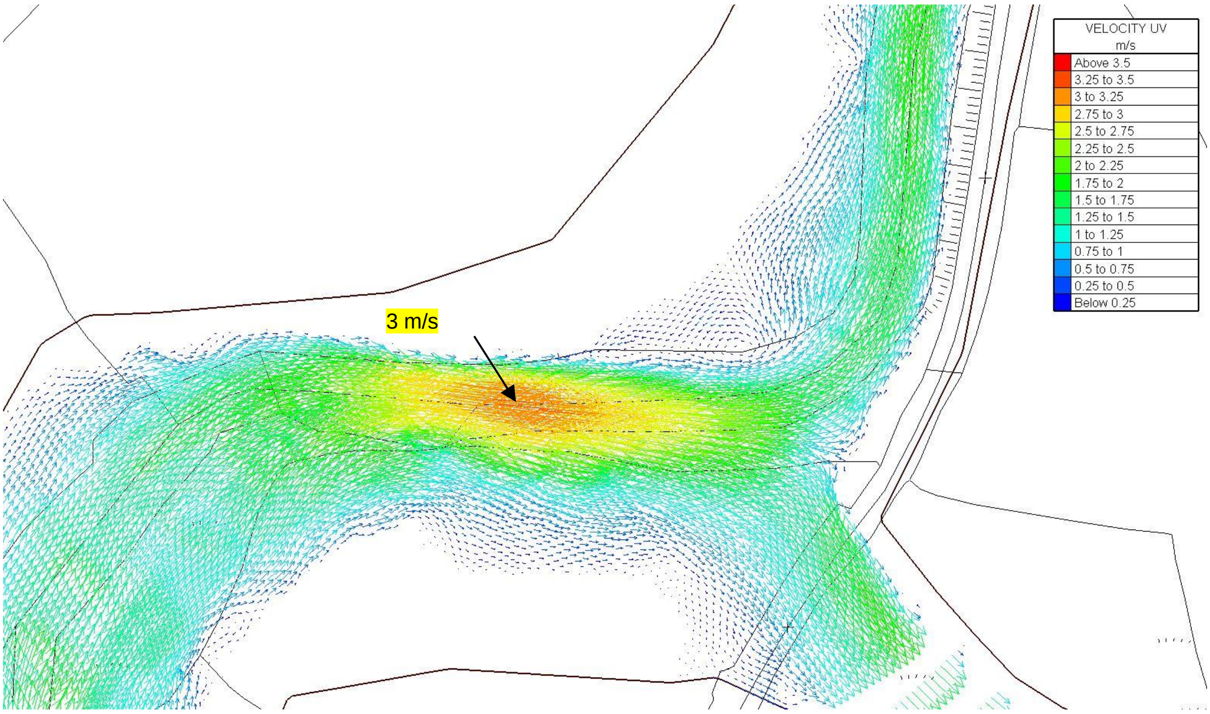

Figure 14 Predicted Flow Velocities at Peak of 100year Flood for Case 2. Wide diversion inlet at 17.3m OD and 30m wide channel at 1 in 1000.

4.2.3 Inlet to diversion channel at invert level of 16.5m OD

This option considers retaining a bank level of 16.5m OD (0.4 to 0.6m above talweg level of the adjacent river channel) and funnelling gradually to a 30m wide diversion channel. Figure 15 presents the velocity plot at peak flood flow in the vicinity of the inlet. The predicted flood level in the river adjacent to the inlet is 18.76m OD. This configuration (without any weir) will divert 58.8% of the peak flow and 41.2% remains in the existing river through Crossmolina.

The Maximum velocity in the Main Channel just upstream of the proposed diversion inlet is 4.0m/s. The velocity plot shows a drawing of the flow direction further southwards to coincide with the main channel orientation and the development of an anticlockwise gyre on the north bank with limited out of bank flow on the north side of the river.

The lowering of the flood level by over 1.5m reduces significantly the depth and magnitude of out of bank flows and increases significantly the in-channel velocity upstream of the diversion from c. 2m/s for the existing case to c. 4m /s representing a doubling of the flow velocity.

This case achieves well over the desired 50 : 50 split in flood flow rates between the Diversion channel and the existing river channel. However, this option draws too much flow and significantly lowers the flood level in the river adjacent and upstream of the weir resulting in doubling of channel velocities , refer to Figure 15 below.

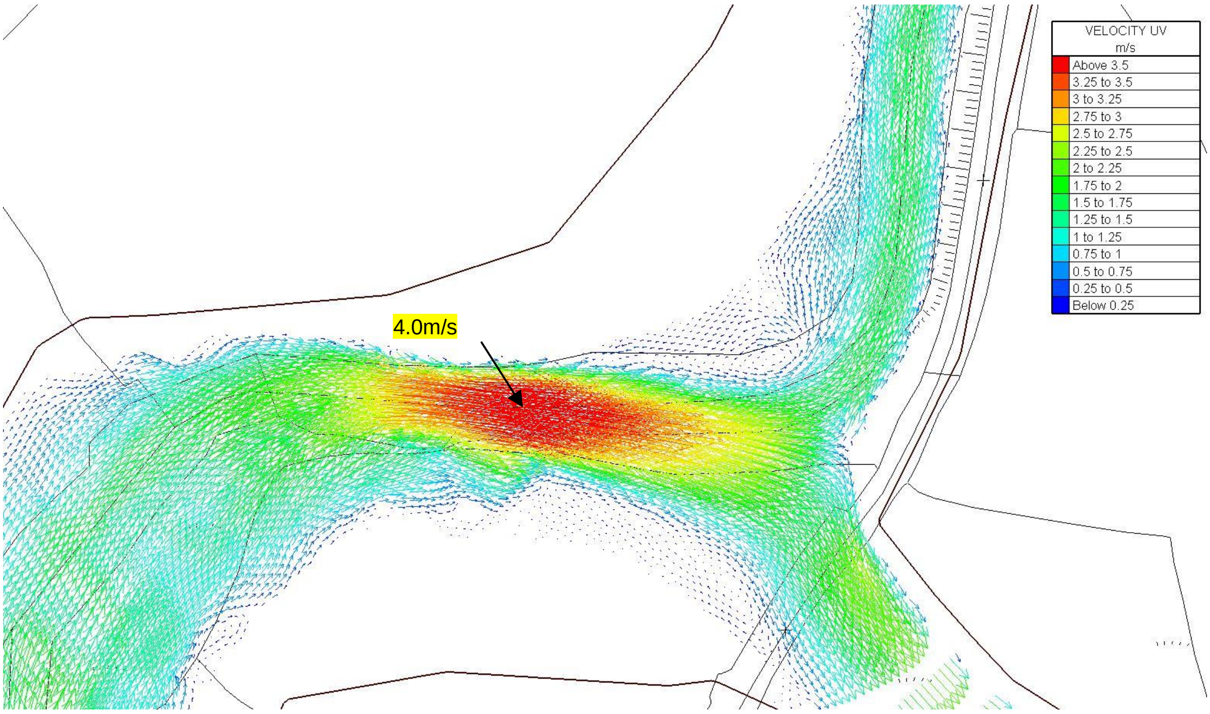

Figure 15 Predicted Flow Velocities at Peak of 100year Flood for Case 3. Wide diversion inlet at 16.5m OD and 30m wide channel at 1 in 1200.

These simulations demonstrate that to achieve the required split in flows between the river and diversion significant lowering of the entrance invert to the diversion is required which has implications for retaining the median flow within the river channel at lesser events, given that the median flood flow in the existing river at the proposed diversion inlet location has a flood level of 18.98 mOD.

4.2.4 Inline and Lateral Weir Structure Cases

Two additional simulations were performed to represent an inline weir and a lateral Weir set at a crest level of 18.25m OD and the bank lowered to 16.5m OD and a 30m wide Channel at 1 in 1200.

4. Inline Weir having crest level of 18.25m OD with wide Inlet set at an invert of 16.5m OD and discharging to a 30m base width 1 in 1000 gradient trapezoidal diversion channel.

5. Lateral Weir configuration having crest level of 18.25m OD with Inlet invert set at 16.5m OD and discharging to a 30m base width 1 in 1000 gradient trapezoidal diversion channel.

The velocity plot from these simulations are presented in Figure 11 12 and 1213.

The predicted flood level in the river adjacent to the inlet is 18.75 and 18.76 m OD for the inline and lateral weir cases. The Inline weir configuration will divert 45.8% of the peak flow and 54.2% remains in the existing river through Crossmolina. The Lateral weir configuration will divert 42.1% of the peak flow and 57.9% remains in the existing river through Crossmolina. The maximum velocity in the main channel just upstream of the proposed diversion inlet is 3.2 to 3.4m/s. These weir cases do not achieve the desired 50% split between the diversion channel and the river and significantly increases the river channel velocity upstream of the weir by almost double.

The weir crest levels are significantly below the median flood level of 18.98m and therefore the median flood will not be retained within the existing river during lesser events. Raising the weir crest level to the median flood level of 18.98m OD will significantly reduce the diverted flow.

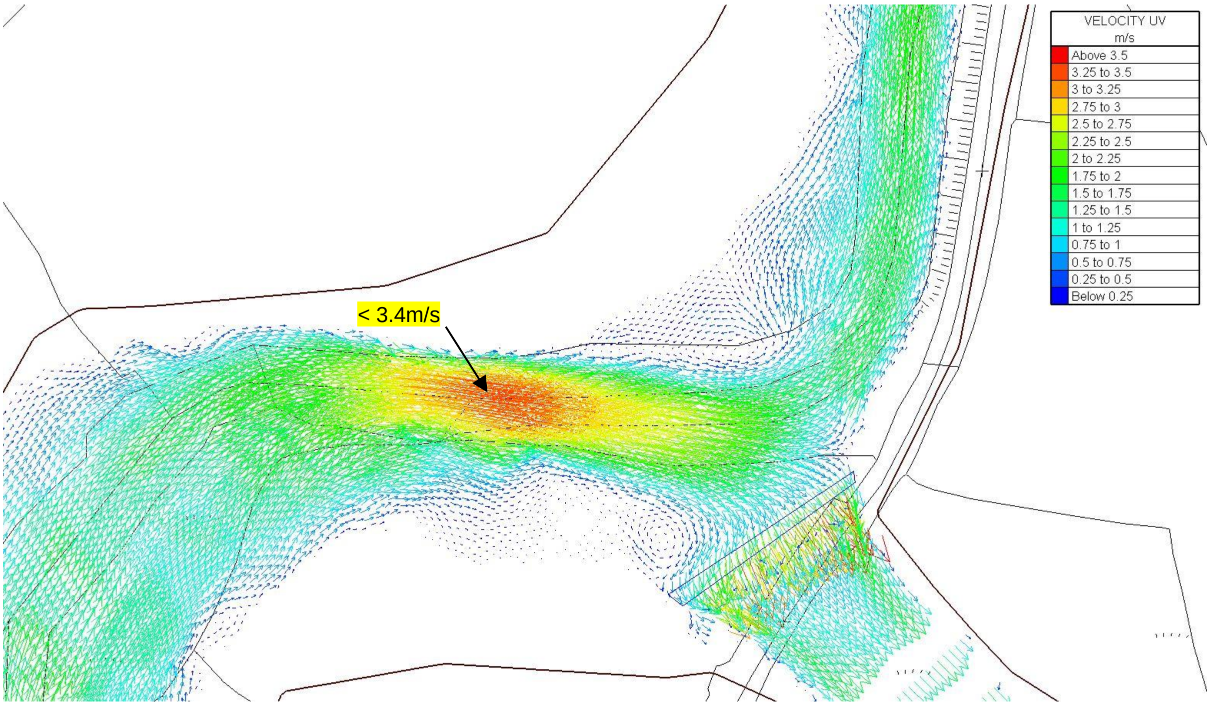

Figure 16 Predicted Flow Velocities at Peak of 100year Flood for Case 4 – Inline Weir option.

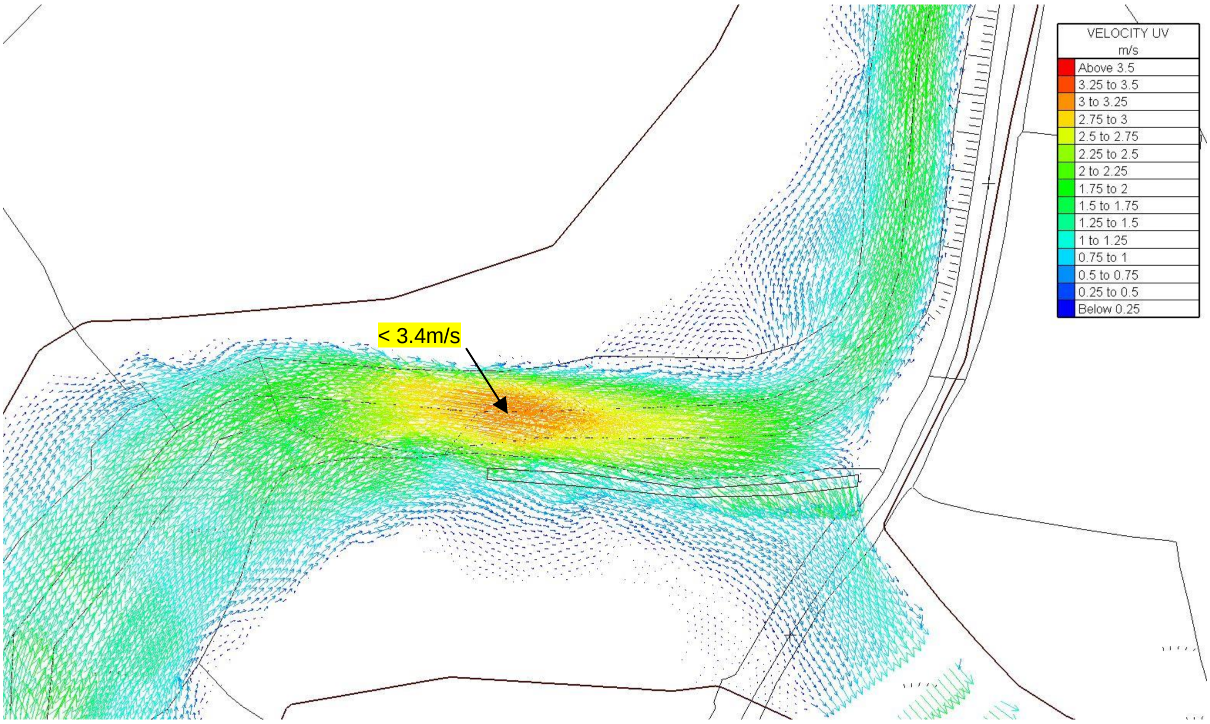

Figure 17 Predicted Flow Velocities at Peak of 100year Flood for Case 5 – Lateral Weir option.

4.2.5 Movable lateral weir at crest level 17.3

A final test simulation considered is a lateral movable gated weir that can drop over its full length to a crest level of 17.3m OD with the Invert of the channel downstream of the weir at 16m OD. The predicted flood level in the river adjacent to the inlet is 18.76m OD Malin.

The Lateral movable weir configuration will divert 58.2% of the peak flow and 41.8% remains in the existing river downstream reach to Crossmolina. This achieves well in excess of the desired split in flow between the proposed diversion channel and the river. The maximum velocity in the main channel just upstream of the proposed diversion inlet is 3.95m/s representing a substantial increase in channel velocities over the existing case due to the significant reduction in upstream flood level as a result of the diversion. The velocity plot from this simulation is presented in Figure 18.

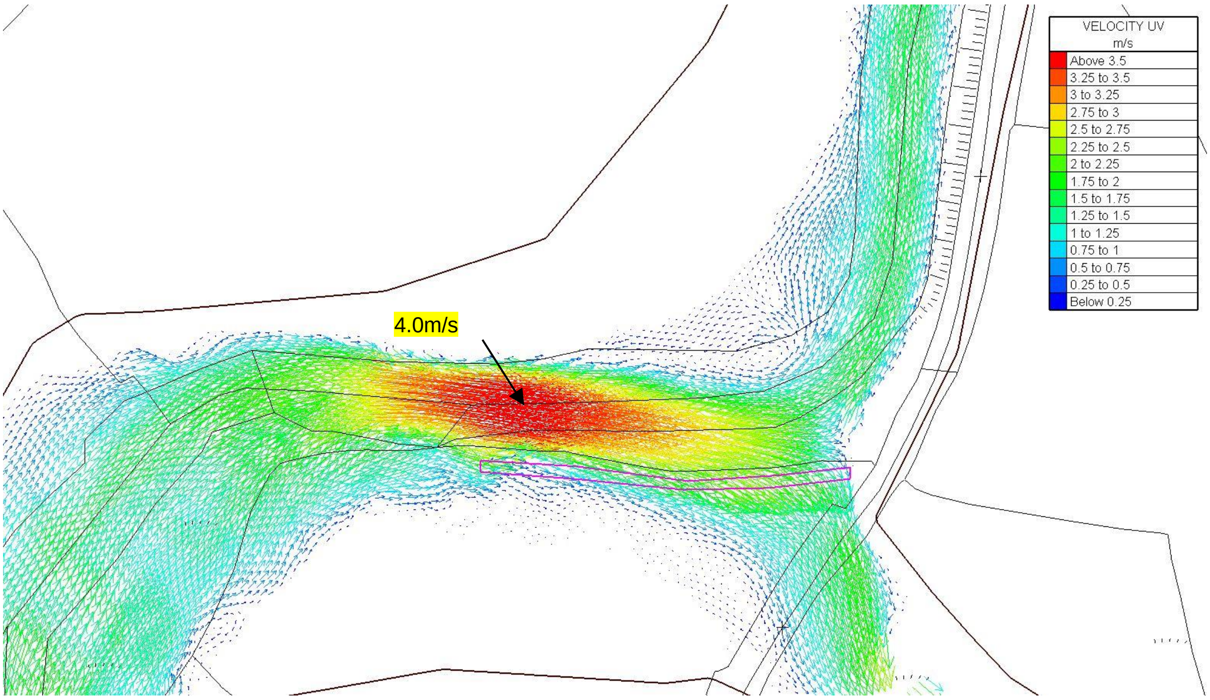

Figure 18 Predicted Flow Velocities at Peak of 100year Flood for Case 7 – Movable Lateral Weir option.

4.3 Discussion

The hydraulic simulations presented in the previous section show for the case of no controlling weir on the diversion channel that significant reduction in the bank levels to an invert of 16.5m OD (i.e. close to channel bed level) over a wide inlet length is required to achieve the necessary split in the flood flows between the diversion channel and the existing river. Achieving the desired split in flows the flood level in the river at the inlet section will be significantly lowered by c. 1 to 1.5m. Such lowering in flood level causes a speeding up of the river flow resulting in significant local increases in channel velocity at the 100year flood. The predicted increase is a doubling of peak velocity from c. 2 to 4m/s. Such velocities in a natural channel are not desirable and will cause significant local scouring and morphological changes to the river bed which has the potential to alter the hydraulic regime at the inlet.

The simulations show that including a fixed lateral or inline weir at entrance / upstream end of the diversion channel will not achieve the desired split in flows with too little spilling into the diversion channel.

A final simulation of a movable lateral Weir set with a crest level at 17.3m OD with the channel invert at 1m OD provides the desired split in flood flow of 58.2% (i.e. c 60%) to the diversion channel. This simulation reduces the flood level in river adjacent to the weir structure to 18.76m OD and increases the velocity in the river channel locally to almost 4m/s similar to Case 2 and 3.

The conclusion reached is that the weir configuration, whether it is a movable inline or lateral weir crest or a downstream inline weir that achieves a 60% diversion of flood flow, the resultant flood level in the Deel will be significantly lowered (c. 1.5m) giving rise to significant increases in the local flow velocity and serious scour implications for the Deel River channel in the vicinity of the intake.

It is recommended that such lowering / drawing down of the river level be avoided by introducing a downstream flow control structure on the main river channel that would control upstream flood levels by restricting the larger flood flows. This flow control structure can be designed to significantly throttle back flood flows and increase the flood level upstream until it can spill over a wide weir set at a prescribed crest level that maintains the median flood flow in the main river channel. Such a flow control structure can be designed to mimic existing flood levels in the River at the intake point and achieve the desired distribution of flood flow between the river and diversion channel.