LOUGH FUNSHINAGH
INTERIM FLOOD RELIEF SCHEME

Engineering Report

Roscommon County Council

September 2024

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MWP

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1. Introduction & Background

The purpose of this report is to describe the civil engineering works that are being proposed to pump water from Lough Funshinagh in the 2024/25 and 2025/26 winter seasons.

The lough reached a record peak level of 69.37mOD on the 16th of April 2024. The lough is impounded by the surrounding hills which have their lowest crest level of about 69.30mOD at a location near its southwest corner. As a consequence, the lough overflowed in 2024 and flooded a large area of land at Carrick/Lysterfield and resulted in the temporary evacuation of two houses, one of which has now been permanently vacated.

Analysis carried out by MWP in May/June 2024 indicated that the natural flow path for water overtopping Lough Funshinagh would be in a southerly direction through the village of Curraghboy. Water would then continue to flow southwards to the Cross River which is located approximately 500 metres south of the village. Fortunately, the lough level in April 2024 started to recede in time to avoid this occurrence. If it had continued to overflow, an extensive area of land and two houses adjacent to the R362 road would have been inundated before the flow continued to Curraghboy.

An analysis of the rate of change in level over the past eight years indicates that the 2024 peak water level may be exceeded in the next winter seasons, with the result that Curraghboy could be flooded. To prevent this possible occurrence, and to protect other properties at Lough Funshinagh, it is proposed to extract a sufficient volume of water from Lough Funshinagh that will negate or partly negate the increase in level prior to the developing a permanent scheme and limit the peak water level which will allow the flood risk at the properties around the lough to be successfully managed.

The more recent and historic flood level peaks at Lough Funshinagh are provided on Table 1.1 below and the recorded water levels since continuous water level monitoring commenced in 2016 are plotted on Figure 1.1.

Year	Level	Comment
1891	63.89	OSI 25" map recorded on 23rd March 1891 (218.5' Poolbeg datum)
2009	67.00	Recorded by Geological Survey Ireland
2016	68.25	First year that RCC had to raise roads and protect houses
2020	68.26	2019/2020 peak level on 28th March 2020
2021	69.03	2020/2021 peak level on 04th April 2021
2024	69.37	2023/2024 record level reached on 16th April 2024

Table 1.1: Lough Funshinagh – Recent and Historic Peak Flood Levels

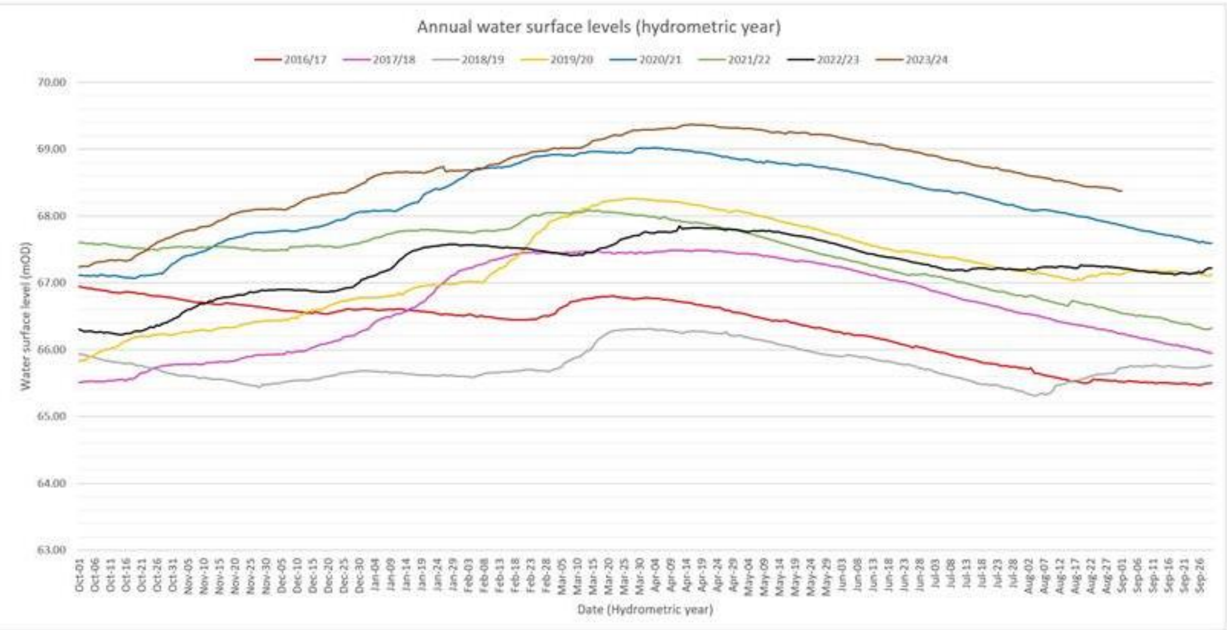


Figure 1.1: Lough Funshinagh Water Levels Since 2016

2. Pumping System Design Rationale

2.1 Existing Lough Funshinagh Overflow Mechanism

Curraghboy is a significant location at Lough Funshinagh because it is the lowest potential overland discharge route from the lough. If there is no intervention then rising water levels in the Lough will be limited by water discharging by gravity and flowing overland in a southerly direction towards Curraghboy village and then onwards to discharge into the Cross River which flows in a southeasterly direction approximately two kilometres to the south of Lough Funshinagh.

An analysis was carried out in order to provide an estimate of the flood extents in the event that Lough Funshinagh water levels were to continue to rise without intervention. The indicative flood extent map is shown on Figure 2.1 below. The analysis suggests a significant depth of water must accumulate north of Curraghboy village before the water level rises sufficiently and then spills southwards through the fields until it reaches the Cross River. It appears likely that the flood levels in Lough Funshinagh could reach close to 70.0 mOD before spilling over to Curraghboy village and flowing into the Cross River further to the south. The analysis presented in Section 2.2 of this report indicates that with no intervention this could occur before March 2025 and, based on the trend of increasing peak water levels, the likelihood of this occurring increases in subsequent years.

In order to prevent such uncontrolled overflows from Lough Funshinagh, an interim pumping system is proposed that will broadly replicate the overflow mechanism and flow path which is predicted to occur in the near future, and it will do so in a controlled manner that minimises the risk to property and people.

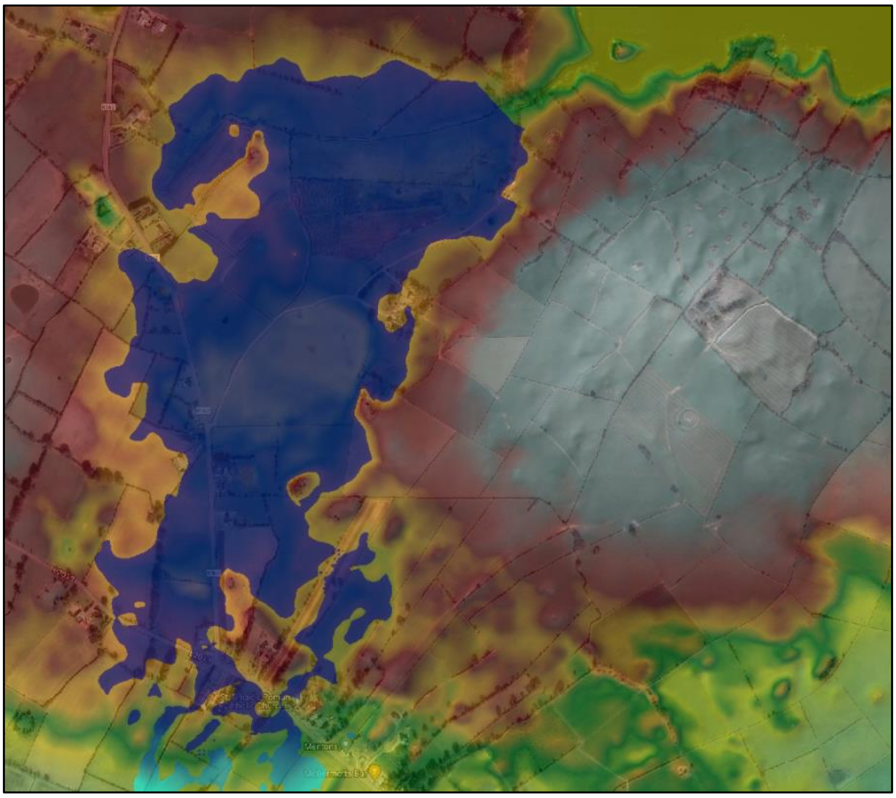


Figure 2.1 - Elevation Heat Map with Indicative Flood Extents due to Overland Flows from Lough Funshinagh

2.1.1 Average Net Inflow Rate

As discussed above, without intervention the water level in Lough Funshinagh is likely to rise, with this limited only by overland flow from the Lough to the village of Curraghboy via an area of lower elevation along the southern boundary of Lough Funshinagh. Table 2.1 presents the data used to calculate the average daily increase in water volume in the Lough of 41,139 m^3/day corresponding to an average net inflow rate of 476 litres/second over six years. The three highest values, each of which is significantly higher than the mean, are averaged to give a high-end value of 53,498 m^3/day which corresponds to an average net inflow rate of 619 litres/second.

Hydrometric year	2016/17	2017/18	2019/20	2020/21	2021/22	2022/23	2023/24
Start date	01/10/2016	01/10/2017	01/10/2019	01/10/2020	01/10/2021	01/10/2022	01/10/2023
Peak date	21/03/2017	17/04/2018	29/03/2020	04/04/2021	17/03/2022	12/04/2023	16/04/2024
Days to peak	171	198	180	185	167	193	198
Start level	66.947	65.516	65.834	67.111	67.605	66.308	67.241
Peak level	66.804	67.487	68.264	69.026	68.083	67.851	69.370
Start volume	10,683,547	5,260,535	6,377,988	11,370,827	13,536,024	8,136,209	11,927,240
Peak volume	10,094,938	13,006,070	16,655,301	20,605,380	15,769,934	14,668,237	22,516,448
Volume increase to peak		7,745,534	10,277,313	9,234,553	2,233,910	6,532,028	10,589,208
Average daily increase		39,119	57,096	49,917	13,377	33,845	53,481
Normal daily increase				41,139			
Adverse daily increase				53,498			

Table 2.1 - Analysis of historic rates of increase in water level

The hydrometric year 2016 was not included in the series because the level decreased continuously throughout the year except for a 29-day interval from the 21st of February when the level increased by 0.36 m. 2017 had a very similar profile for most of the year without the characteristic level increase in the first six months; this was also excluded. While these lower profiles could recur, they do not follow the more recent trends and were excluded from the calculations to ensure that the results are reasonably conservative.

2.1.2 Average Net Outflow Rate

In addition to calculating the average daily increase in the water volume of Lough Funshinagh, the average daily rate of net outflow was calculated. This rate was determined in a similar manner to that of the inflow rate; the results are shown in Table 2.2. This utilised data across six hydrometric years from 2016/2017 to 2022/2023.

Hydrometric year	2016/17	2017/18	2019/20	2020/21	2021/22	2022/23	2023/24
Peak date	21/03/2016	17/04/2018	29/03/2020	04/04/2021	17/03/2022	12/04/2023	16/04/2024
End date	20/08/2016	30/09/2018	17/08/2020	30/09/2021	30/09/2022	13/07/2023	
Days from peak	152	166	141	179	197	92	
Peak level	66.804	67.487	68.264	69.026	68.083	67.851	69.370
End level	65.492	65.949	67.030	67.59	66.321	67.175	
Peak volume	10,094,938	13,006,070	16,655,301	20,605,380	15,769,934	14,668,237	
End volume	5,178,370	6,794,616	11,029,652	13,468,309	8,186,218	11,643,538	
Volume decrease from peak	4,916,569	6,211,454	5,625,650	7,137,071	7,583,716	3,024,699	
Annual daily decrease	32,346	37,418	39,898	39,872	38,496	32,877	
Normal daily decrease				36,818			
Adverse daily decrease				25,752			

Table 2.2 - Analysis of historic rates of decrease in water level

Based on the analysis of the trends in the Lough, it is possible that the total outflow during the summer will be less than normal if the level stops decreasing before the end of September. In four of the seven recorded full years, the decrease in level stalled during the summer on the following dates:

- 20 August 2017 (41 days early)
- 04 August 2019 (57 days early)
- 17 August 2020 (44 days early)
- 13 July 2023 (79 days early)

The average early stalling point was 55 days before the end of summer in four of the seven years. Based on these averages, there was a greater than 50% chance that the level would stop decreasing on a probable date of 06 August. While this has not yet happened, it highlights the likelihood of an early stalling, the consequences of which would be compounded by the fact that the water level in the lough remains concerningly high for this time of year (at the time of writing, early September 2024).

The average daily increase is higher than the average daily decrease, which is consistent with the general trend of increasing peak water levels over the last eight years.

2.2 Predicted Water Levels in Lough Funshinagh

Table 2.3 shows the likely still water levels that may occur in Lough Funshinagh for different levels of risk or probability at the end of the 2024/25 and 2025/2026 winter seasons. These levels are based on average rates of rise in water surface levels since 2016 as discussed in the analysis presented in the previous section. The normal risk which has a high probability of occurrence assumes that there will be an increase in the peak level of 69.37 mOD that occurred at the end of winter 2024 and this reflects the overall trend of increasing peak levels over the past eight years. More adverse conditions could arise in the following scenarios:

- the natural decrease in level over the summer halts before the end of September as has happened in four of the eight years on record;
- the total increase in level over the winter is at the upper end of those recorded in the last eight years.

The medium and high-risk water surface levels shown in Table 2.3 relate to the occurrence of one or both, respectively, of the above adverse conditions occurring between 01 April 2024 and 31 March 2025.

Predicted Still Water Levels after Two Winters without Intervention			
Risk	Probability	March 2025 Level (mOD)	March 2026 Level (mOD)
Normal	High	69.60	69.73
Medium	Medium	69.96	70.43*
High	Low	70.30*	71.11*
** In reality these levels would not be realised as water is expected to overtop and flow towards the Cross River*			

Table 2.3 – Predicted Water Levels without Intervention

2.3 Pumping Rate and Level Limits

The aim of the interim measures is to extract a sufficient volume of water from Lough Funshinagh that will negate or partly negate the increase in level prior to the developing a permanent scheme and to limit the peak water level which will allow the flood risk at the properties around the lough to be successfully managed. To prevent an overflow at Carrick it will be necessary to limit the peak water level to below 69.30 mOD at the end of March 2025.

The proposed interim scheme is for pumping for a period of up to 24 months. Pumping will be undertaken only when the water level in the lough is above 67.50 mOD and at a flow rate not exceeding 300 litres/second.

The rationale for selecting a level of 67.50 mOD as the lower limit for pumping is that this is still above the pre-2016 normal maximum flood level indicted by the Lough Funshinagh Technical Subgroup in their reports. These include *Modelling and Analysis of Lough Funshinagh Flood Levels* (13th June 2024) and *Modelling and Analayis of Changes to Lough Funshinagh Flood Levels* (1st July 2024). The reports are included in Appendix A.

The maximum flow rate has been selected taking consideration of:

1. The expected adverse average overflow rate from Lough Funshinagh to the Cross River in the event that no intervention by pumping is made, being up to 619 litres/second as outlined in Section 2.1.1. The selected pumping rate is half of the overflow rate which provides a significant reduction to the uncontrolled overflow rates and volumes.
2. The availability of high-capacity pumps.
3. The expected volume that needs to be removed from the lough in order to have a meaningful beneficial impact and based on the flood risk assessment of the Cross River reach, as outlined in Section 4.

The pumping system will be remotely monitored and the flow rate can be changed or shut off remotely. Remote monitoring will be carried out in conjunction with monitoring the flow in the Cross River which is discussed further in Section 5.

3. Pumping System Design Rationale & Construction Methodology

3.1 Intake Compound

The intake compound is required to provide a safe access for the delivery of the pump system components and to provide for a safe and secure operation of the pumping system with appropriate protections in place to prevent contamination from fuel spillage.

The intake compound consists of:

- A stone hardstand area suitable for vehicular traffic, including loading/unloading of delivery vehicles and fuel trucks. The hardstand is sized to provide for safe truck turning within the compound.
- A concrete slab with upstand walls to contain and support the Hydraulic Power Units (HPUs) and fuel tanks. The bund walls will provide at least 110% storage for two fuel tanks (i.e. 6,600 litres) plus an additional allowance for 75mm of rainfall accumulation. A sump will be provided at the lowest corner of the slab to enable rainwater to be pumped out at regular intervals during the operational phase. The upstand walls also serve as a wheel stop to prevent vehicles accidentally impacting the fuel tanks during turning manoeuvres.
- A stock proof fence around the perimeter of the compound. An agricultural access gate will be provided at the northern end of the compound to facilitate access. An additional gate will be provided at the southern end of the compound to enable the landowner to access the agricultural lands to the south.
- A noise barrier will surround the HPUs and, for security and safety reasons, a paladin fence will be erected around the remainder of the HPU bund and access pontoon.

The compound will be constructed without excavating the existing ground. A combination of geogrid and geotextile will be placed over the vegetation on the existing surface within the footprint of the proposed access road and compound. A minimum thickness of 450 mm of imported stone (Class 6F or similar) will be placed on top of the geogrid / geotextile.

The construction of the intake compound will involve the following sequence;

I. The appointed contractor will mark out the line of the proposed compound using a GPS / total station.
II. A layer of geogrid / geotextile will be rolled out by hand along the line of the proposed road and compound.
III. The stone aggregate used to construct the compound will be imported from a local quarry using trucks. The trucks will reverse tip the stone onto the geogrid / geotextile and an excavator will be used to spread the stone before compaction. Compaction of the stone material will be completed in accordance with Transport Infrastructure Ireland (TII) Specification for Roadworks requirements. This is typically completed in layers with the use of a vibratory roller or similar.
IV. The compound will be constructed with a crossfall of 3% so that water can flow off the surfaces and reduce the risk of rutting / potholes occurring.
V. Surface water runoff from the compound will be discharged directly over the edge of the stone embankment and a continuous silt fence will be constructed on the downslope side to capture any sediment that may run off the surfaces.
VI. The timber posts in the stockproof fence will be driven into the existing soil without any excavation. Refer to the drawings for details of fences.
VII. A concrete bund measuring 11 m x 8 m will be constructed inside the compound to support the HPUs and fuel tanks and to contain any fuel in the event of a spillage. The slab will be cast directly onto the imported stone used to construct the compound. The slab will include reinforcement to prevent leakage. The upstand walls will be cast in-situ using conventional formwork.

VIII. The noise barrier fence posts will be fixed directly to the HPU bund upstand walls or slab, as shown on the drawings.

IX. The method of installation of the paladin fence posts will depend on the location. Where it surrounds the HPU bund it will be fixed directly to the upstand walls, as shown on the drawings. Outside of this, it will be connected to a 'Kelly Block'-style precast concrete block by a steel plate. Refer to the drawings for details of fences.

3.2 Pump Intake System

The pump intake system is required to facilitate safely placing pumps into the water at Lough Funshinagh and facilitate the removal of water without being harmful to fish.

The pump intake system consists of:

- A floating pump pontoon to house two high capacity hydraulically driven pumps. The container will have integrated removable fish screens.
- Two hydraulic power units (HPUs) positioned within the bund in the intake compound. The HPUs will be connected to the pumps with a pair of hydraulic hoses (flow and return) each. The HPUs will be fitted with silencers to reduce the sound to approximately 76dBA at 7 meters.
- Four 3,000 litre double skinned fuel tanks located within the bund in the intake compound. Each HPU will be connected to two fuel tanks.
- The system will include a facility for remotely monitoring the pump flow rate and for changing the flow rate remotely if necessary.
- A floating access pontoon, approximately 25.6m long, to provide safe access to the pump pontoon and to support the hydraulic hoses linking the pumps to the HPUs and also to support the flexible hoses.

From an engineering perspective, the position of the pump intake system is governed mainly by the range of possible water depths in the lough over the operational life of the pumping system as well as the need to provide safe access for inspection and maintenance of the pumps. A bathymetric survey of the southwest corner of Lough Funshinagh was undertaken which enabled the optimum position to be identified. The ground level at the proposed location of the pumps is approximately 66.25 mOD at a distance of about 25 metres out from the current position of the water edge (early September 2024).

The exact position of the Access Pontoon at the interface with dry land will be governed by the water level in the lough. As the water level reduces, the access and pump pontoons will be moved gradually eastwards.

Water from the lough will enter the pump pontoon through the removable fish screens that will have 10 mm apertures and a combined minimum net area of 2 m^2. This will ensure that the approach velocity of the water entering through the fish screen will have a maximum velocity of 150 mm/second at a flow rate of 300 litres/second. This will ensure that juvenile fish can swim away against the current and not get entrained on to the mesh.

The construction of the pump intake system will involve the following sequence:

I. The pump and access pontoons, HPUs, and fuel tanks will be manufactured in The Netherlands and will be transported to site on articulated trucks. The trucks will deliver all these components to the intake compound.

II. A mobile crane will be used to lift the pump pontoon (with the pumps already installed within it) from the truck in the compound to the lough. The pump pontoon will be floated into its final position and held there using four spud legs which will be lowered onto the ground beneath the water. A small boat will be in the water to assist with positioning.

III. The floating access pontoon will be transported to the site in five 6.4 m lengths. Each section will be lifted into position in the lough using the crane and bolted together.
IV. The same crane will lift the HPUs and fuel tanks into position within the HPU bund.
V. The hydraulic hoses and two 300 mm diameter pipes will be mounted on the sides of the floating access pontoon using brackets.
VI. The fuel tanks will be filled with diesel using a fuel truck. The refuelling methodology is outlined in Section 6.1.
VII. The pump system will be commissioned and tested following installation of the pipeline.

3.3 Pipeline

The purpose of the pipeline is to convey the water from the pump intake system to the outfall at the Cross River. The route of the pipe has been selected so that it runs along property boundaries for almost all of its length as this will minimise disruption to the landowners.

The pipeline consists of:

- Two parallel lines of 300 mm diameter flexible lay-flat hoses, each pipeline being approximately 2,130m long. The lay-flat hoses will extend from the lough to a point approximately 160 metres south of the L2013 local road.
- A 500mm diameter (outside) PE ribbed pipeline between the end of the lay-flat hoses and the outfall at the Cross River. The length of PE ribbed pipeline is 320 metres. PE ribbed pipes are used at the southern end of the pipeline because the ground at this location falls consistently towards the river which enables a gravity discharge and reduces the demand on the pumping system.
- A custom-made manifold to connect the pair of lay-flat hoses to the single PE pipe.
- A stock proof fence will be constructed on each side of the pipeline to provide a total corridor width of between 5 and 7 metres as indicated on the drawings.

The lay-flat and PE ribbed pipelines will weight 77 kg/m and 109 kg/m respectively when flowing at full capacity. These are not expected to cause significant settlement or rutting of the ground and any such settlement is not likely to exceed 50mm.

The construction of the pipeline will involve the following sequence:

I. The lay-flat hose and PE ribbed pipe system will be supplied from The Netherlands and will be transported to site on articulated trucks.
II. The lay-flat hose will be supplied in 50 m to 200 m lengths (typically 200 m) and will be housed in a container. The container will be lifted off the trucks and onto a flatbed trailer which will be attached to a tractor or excavator. The tractor or excavator will drive along the route of the pipeline and deploy the hose directly onto the ground surface. The final positioning of the hose will be done by hand.
III. The pipeline will need to pass through a number of field boundary fences/hedgerows, as shown on the drawings. At each location, the existing boundary fence/hedgerow will be taken down over a width of 5 metres which is required to allow both the pipeline and a tractor/excavator to pass through.
IV. Cross drains consisting of HDPE drainage pipes will be laid beneath the lay-flat hoses at appropriate intervals to maintain the existing drainage regime on the site. This approach eliminates the need to excavate new drainage channels or alter the existing flow regime.
V. The PE ribbed pipeline will be supplied in lengths of up to 12 metres and will be connected together with bolted collars. The pipe sections will be loaded from the articulated lorry to a flat-bed trailer attached to a tractor or excavator. The tractor or excavator will drive along the route of the PE ribbed pipe and will be followed by an excavator which will be used to lift the pipes from the trailer to the required

position on the ground. Due to the existing surface condition, which has some localised humps and depressions, the line of the 500 mm diameter PE ribbed rigid pipe will be levelled and compacted using an excavator. The maximum depth change will be 150 mm which is less that the depth of influence in conventional agricultural tilling.

VI. The two lay-flat hoses will be connected to the single PE ribbed pipe using the manifold section.

VII. The pipeline will require the construction of two public road crossings and one private access road crossing. Each crossing will include two 600 mm diameter HDPE carrier pipes with a pull ropes inside. The pull rope will be attached to the lay-flat pipe which will then be pulled through the HDPE carrier pipe. The construction methodology for road crossings is outlined in the following sub-section.

VIII. A stock proof fence with timber posts will be driven into the existing soil without any excavation. Refer to the drawings for details of fences.

IX. Provision will be made for badgers to cross the PE ribbed pipe by constructing a ramp over the pipe, as detailed on the drawings.

X. Badger access shall be facilitated through the installation of 300 mm diameter pipes through the stock proof fencing.

3.4 Road Crossings

The pipes will run overground throughout except at the road crossings which will be required at three locations:

- the private access road adjacent to the R362 road
- the R362 regional road
- the L2013 local road

The crossings will consist of two 600 mm diameter HDPE carrier pipes through which the pair of flexible pipes can be routed. The carrier pipes will be installed by open excavation followed by backfilling of the trench and reinstatement of the road. A short section of shallow open excavation will remain on both sides of each crossing.

The construction of road crossings will involve the following sequence;

I. On the public roads, in order to allow traffic to continue to use the roads, the pipe will be installed in two segments such that at least one traffic lane remains open at all times.

II. Prior to undertaking any works, a CAT scan will be undertaken to identify any services in the road. It is known that an existing Uisce Éireann watermain and a telecommunication cable are present in the road.

III. An 1,800mm wide trench will be excavated across the road to accommodate two 600 mm diameter HDPE carrier pipes. The overall trench depth will typically be 2,000mm to provide sufficient cover to the pipe and to ensure that the existing services can be avoided.

IV. The HDPE carrier pipe will be positioned onto a 100mm thick layer of pipe bedding material placed at the bottom of the trench. Once the carrier pipe is in position the trench will be backfilled and the road will be reinstated.

V. The existing hedgerow/stone walls will be removed on both sides of the road over a width of approximately 3 metres. These will be reinstated following installation of the carrier pipes.

3.5 Outfall to Cross River

The PE ribbed pipe will operate under gravity flow and will flow half full at a velocity more than 4 metres/second when the pumps are discharging at a rate of 300 litres/second. Without mitigation, the velocity of the water discharging from the pipeline has the potential to cause erosion of the riverbanks and bed. The outfall at the Cross River has been designed to prevent this.

The outfall consists of the following key features:

- A geotextile layer will cover the riverbed and extend up the riverbanks on both the sides of the river.
- Rock armour will be used to hold the geotextile in position, prevent erosion and dissipate energy from the pipeline.
- A diffuser tee fitted at the end of the PE ribbed pipe to dissipate energy and distribute the flow over a larger area of riverbank; the tee will have a series of 36 no. 80 mm diameter holes drilled at 120 mm spacing on side opposite the PE ribbed outfall pipe.
- Rock armour will be built up around the ends of the diffuser tee to further dissipate the energy from water discharging from the ends of the tee.
- A 1.60 metre width of the riverbed will be covered with natural stone flags to hold the geotextile in place and to allow unhindered fish passage, as recommended by Inland Fisheries Ireland. The flags will be laid on geotextile at bed level with the leading and trailing flags at the upstream and downstream ends sloped down so that they are embedded into and level with the existing channel.

These works will extend over a river length of 10 metres, centred on the outfall location.

Due to the significant depth of the channel relative to the maximum water level, the diffuser tee will remain well above the water level in the river when pumping is being carried out.

The construction of the outfall will be supervised by an appropriately qualified person and will involve the following sequence:

I. The geotextile will be supplied in a roll and transported to the outfall location by an excavator.
II. The rock armour and natural stone flags will be transported to the outfall location using a tipper truck or tracked dumper.
III. The geotextile will be rolled out across the full width of the channel from top of bank to top of bank.
IV. An excavator will be used to carefully position rock armour and stone flags onto the geotextile, starting at the bottom and working upwards to ensure stability is maintained.
V. The PE ribbed pipe will be laid as far as the top of the channel bank using the method outlined in the previous sub-section. A pipe bend will be used to orient the end of the pipe and diffuser down the riverbank at the correct angle to fit the riverbank slope.
VI. The diffuser tee will have been pre-fabricated and will be joined to the end of the pipe.
VII. Additional rock armour will be placed around the ends of the diffuser tee to ensure that water discharging from the ends must flow around and through the rock armour before entering the river.
VIII. As noted in drawing 24821-MWP-00-XX-DR-C-1003:
    a. All rock shall be quarried with a minimum saturated surface dry density of 2600 kg/m³.
    b. Stone should be angular and crushed from strong inert rock, which shall exclude shales and weak sandstones.
    c. All individual stones shall be dense, sound, durable rock, free from all cracks, joints and bedding planes, which could result in breakdown of the rock in a fluvial environment. It shall be capable of being handled and placed without fracture or damage.
    d. Individual pieces shall be blocky and take the basic shape of a cuboid. Armour units shall be hand selected and individually placed so each rock is securely held by its neighbours. Rocks shall not be placed so that they obtain their stability on a plane by frictional resistance alone.
    e. Armour stone is to be placed in a systematic way such that the finished construction consists of close packed layers of rock of the specified thickness for the appropriate zone. The surface of the rock shall present a close packed uneven face.
    f. The contractor shall provide details of the source of supply for approval prior to delivery to site.

3.6 General

All work will be carried out in accordance with industry best practice. A list of best practice documents is given below:

- CIRIA (2001) Control of water pollution from construction sites. Guidance for consultants and contractors (C532)
- CIRIA (2005) "Environmental good practice on site" 145
- CIRIA (2006) Control of water pollution from linear construction projects. Technical guidance (C648)
- CIRIA (2015), The SuDS Manual (C753)
- Enterprise Ireland, Best Practice Guide BPGCS005 Oil Storage Guidelines
- Environment Protection Agency (EPA), http://www.epa.ie/pubs/advice/
- Inland Fisheries Ireland (2016) Guidance on Protection of Fisheries during Construction in and adjacent to Water

4. Cross River Analysis

4.1 Introduction & Purpose

A hydrological analysis of the Cross River was undertaken and hydraulic modelling was carried out to gain an understanding of the impact of both the proposed pumping operation, and of uncontrolled overflow from Lough Funshinagh on the watercourse.

The analysis was carried out for both low and high flow situations. The high flow analysis was used to assess the flood risk along the full length of the Cross River, from the outfall to the confluence with the Shannon. Both low and high flows were used to assess the changes in water velocity and stream power which are indicators for erosion potential.

The analysis was informed by various desktop studies, site walkovers and surveys, as described in the following sub-sections.

4.2 Desktop Study

4.2.1 Geology & Soils

Along the length of the Cross River the bedrock geology predominately consists of Visean Limestones (Figure 4.1), which consists of undifferentiated limestones. A 5.5 kilometre length along the downstream end of the watercourse lies over Waulsortian Limestones which are described as massive unbedded lime-mudstones. This underlays a quaternary sediment layer, which consists primarily of Alluvium along the footprint of the Cross, with areas of gravels derived from limestones, cut over raised peat, and till derived from limestone also noted (Figure 4.2). Only 4 kilometres of the approximately 20 km length of the Cross is not underlain by alluvium quaternary sediments.

According to the Teagasc Soil Maps, the majority of the watercourse lies over Alluvium or Cut soil types, with a 4 km section towards the upstream end of the river underlain by a combination of four other soil types (Figure 4.3). These include:

- BMinDW - Deep well drained mineral derived from mainly basic parent materials;
- BMinPD - Deep poorly drained mineral derived from mainly basic parent materials;
- BMinSW - Shallow well drained mineral derived from mainly basic parent materials;
- BMinSP - Shallow poorly drained mineral derived from mainly basic parent materials.

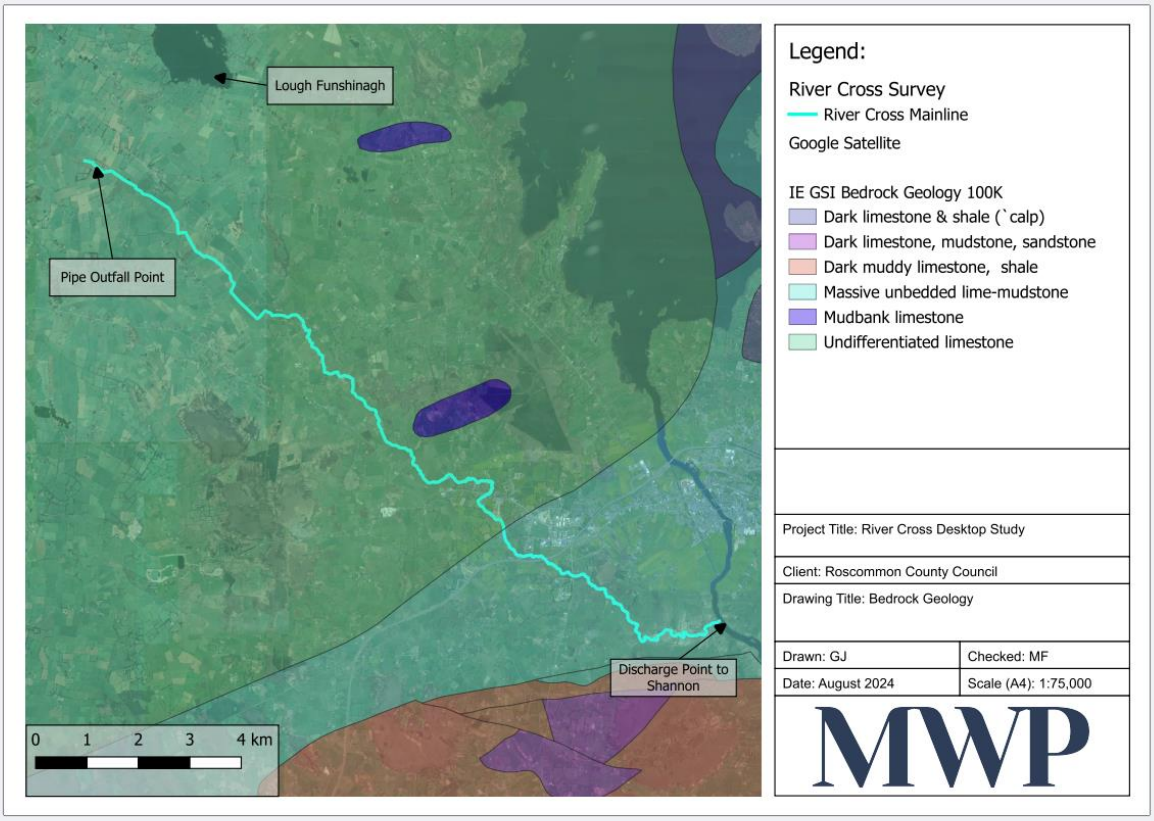


Figure 4.1 - Bedrock Geology along Footprint of Cross River

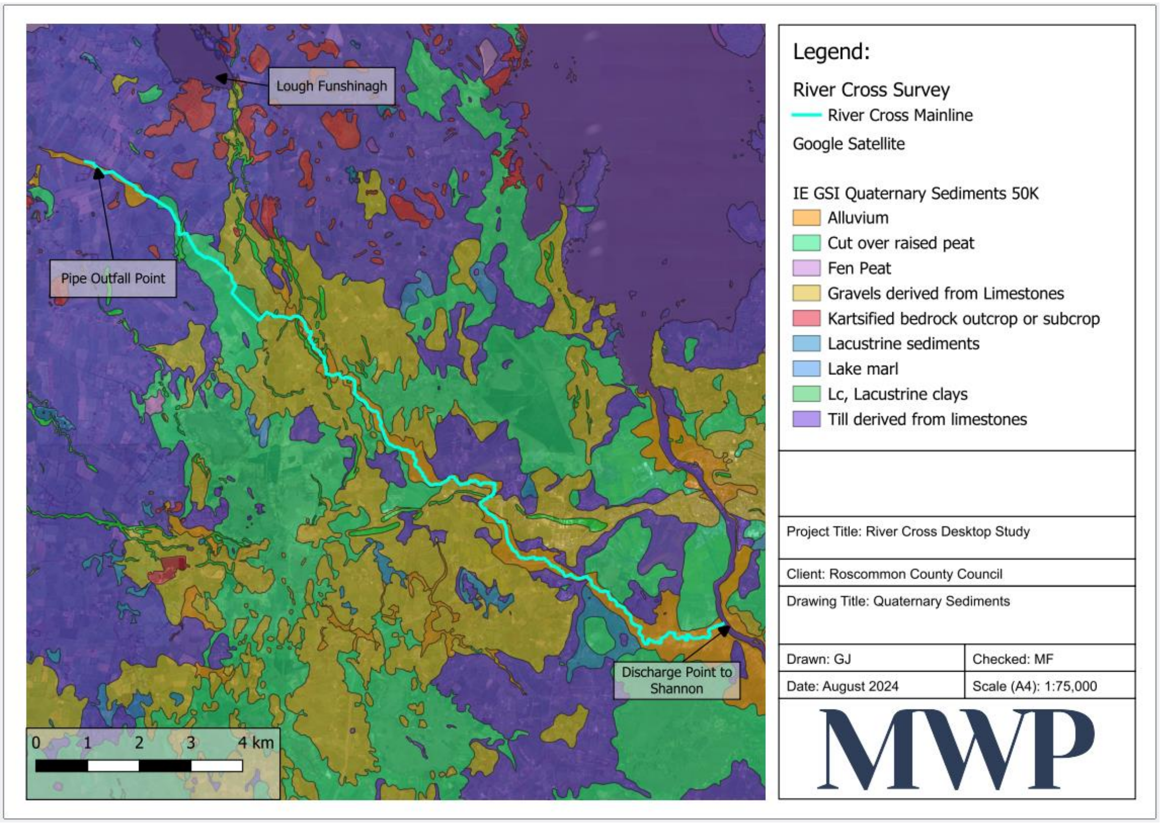


Figure 4.2 - Quaternary Sediments in Study Area

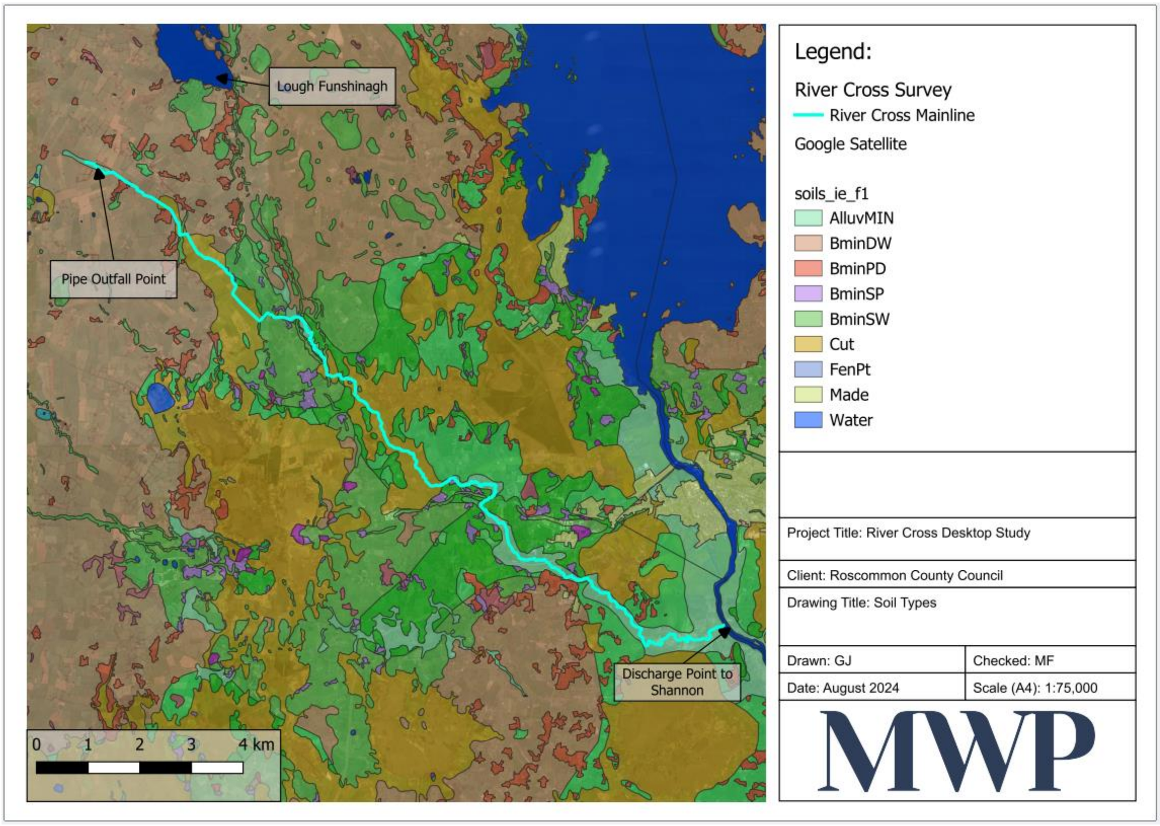


Figure 4.3 - Teagasc Soil Map for Study Area

4.2.2 Hydrology & Hydrogeology

As discussed previously, the temporary solution proposed for the flooding issues in this area involves pumping of water into the Cross River. This is a watercourse with a length of approximately 20 km which rises 2.8 km south-west of Lough Funshinagh and which discharges into the River Shannon 2.5 km south of the town of Athlone. The watercourse has a contributing catchment area of approximately 4.5 km^2 at the proposed pipe outfall point, with this increasing to over 108 km^2 as it approaches the River Shannon (Figure 4.4).

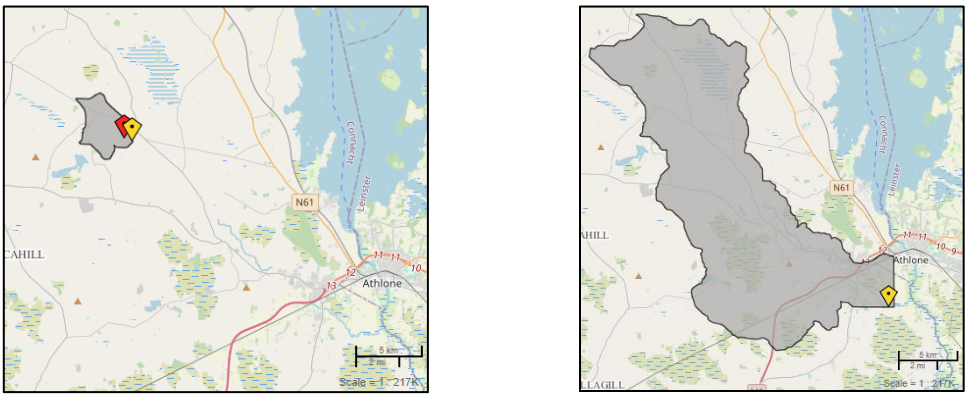


Figure 4.4 - Cross River Contributing Catchment Area at Pipe Outfall (Left) and Discharge to Shannon (Right)

The watercourse flows between areas of Moderate and High groundwater vulnerability (Figure 4.5), with the northern section of the Cross located above a regionally important karstified aquifer, and the southern section situated over a locally important and moderately productive aquifer (Figure 4.6). This data was sourced from available GSI databases which also display subsoil permeability data, which varies between high and moderate across the length of the river as shown in Figure 4.7.

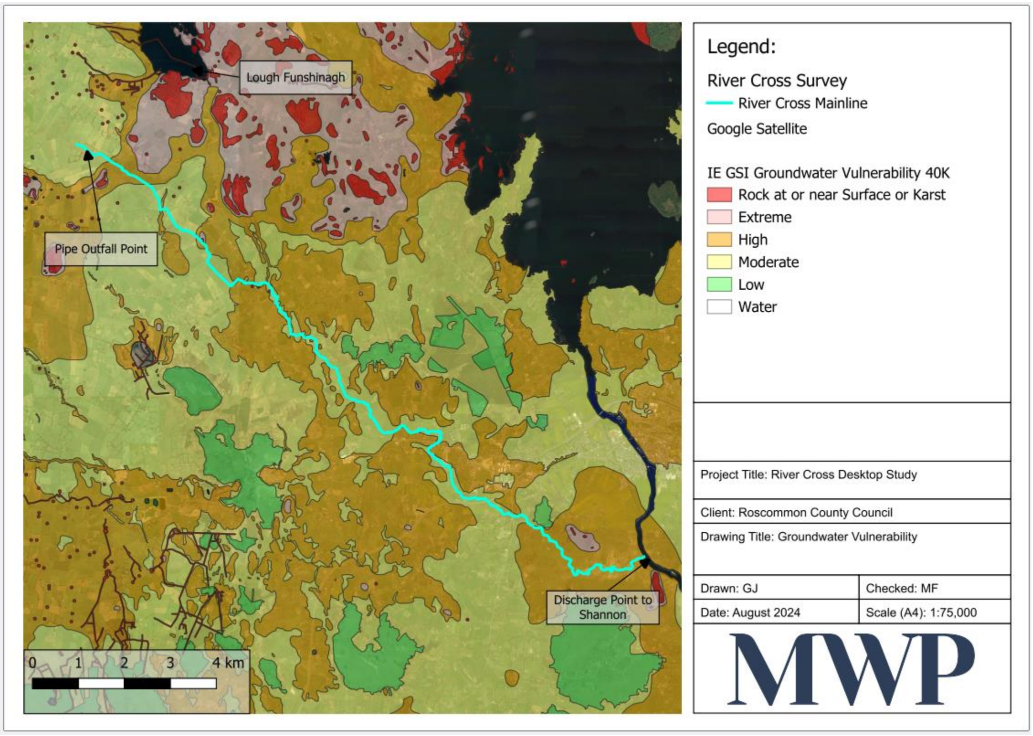


Figure 4.5 - Groundwater Vulnerability

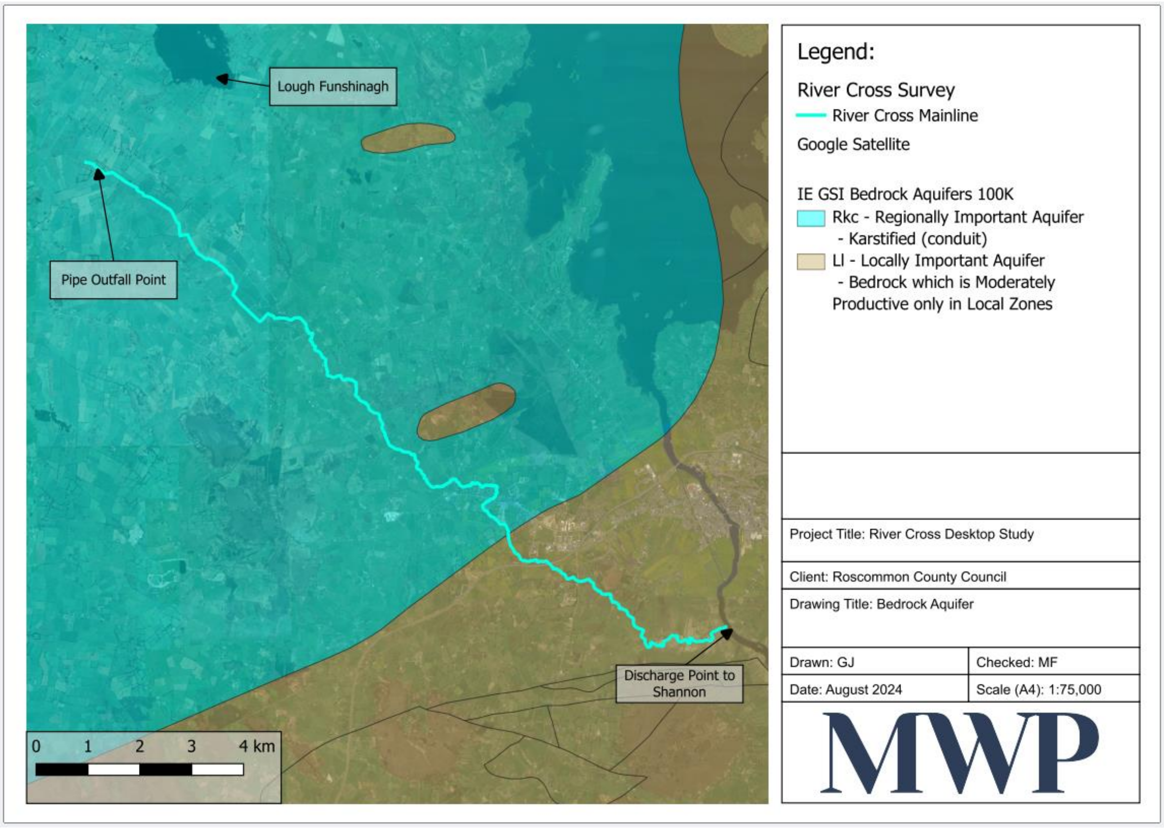


Figure 4.6 - Bedrock Aquifers

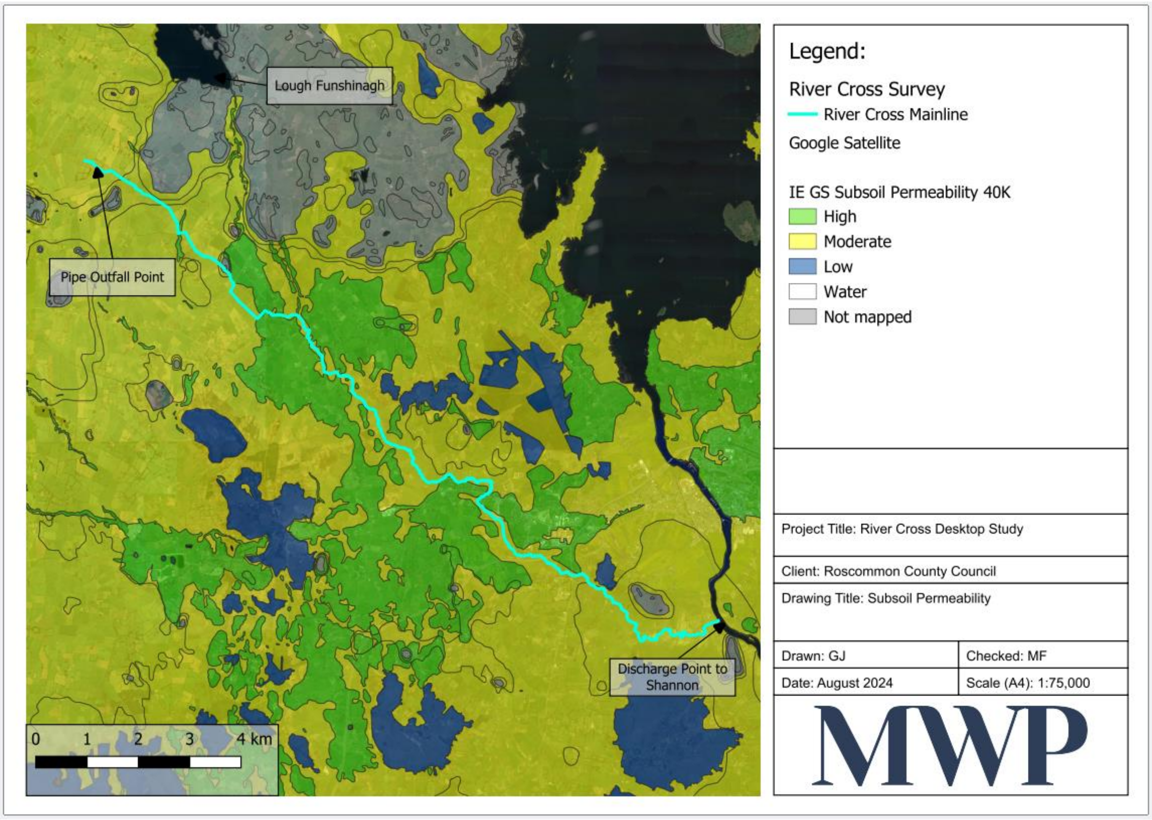


Figure 4.7 - Subsoil Permeability

4.2.3 Historic Mapping Review

To identify whether there have been any noticeable changes to the area over time, historic mapping data was used. This was sourced from the publicly available Geohive website, with the 6 Inch First Edition (1829 - 1834) maps used and compared to available Google Earth aerial mapping data. The Geohive mapping data at the outfall point is provided in Figure 4.8 below, with an overview of the historic map data for the full reach of the Cross provided in in Figure 4.9. These maps show that the area surrounding the site primarily consists of pastural agricultural land and one-off residential developments, in addition to small urban areas such as Curraghboy Village, with no significant change in these areas. A number of mills and associated mill ponds are shown along the length of the Cross River in these maps; however, these ceased to be operational in the 1900s, and the mill ponds were removed. The pond locations how function as pastural agricultural land.

More substantial changes to the area can be observed on the approach to Athlone as the town has seen significant residential expansion. Only a small percentage of this, however, is located adjacent to the watercourse, with the majority of the land surrounding the Cross River seeing no significant changes.

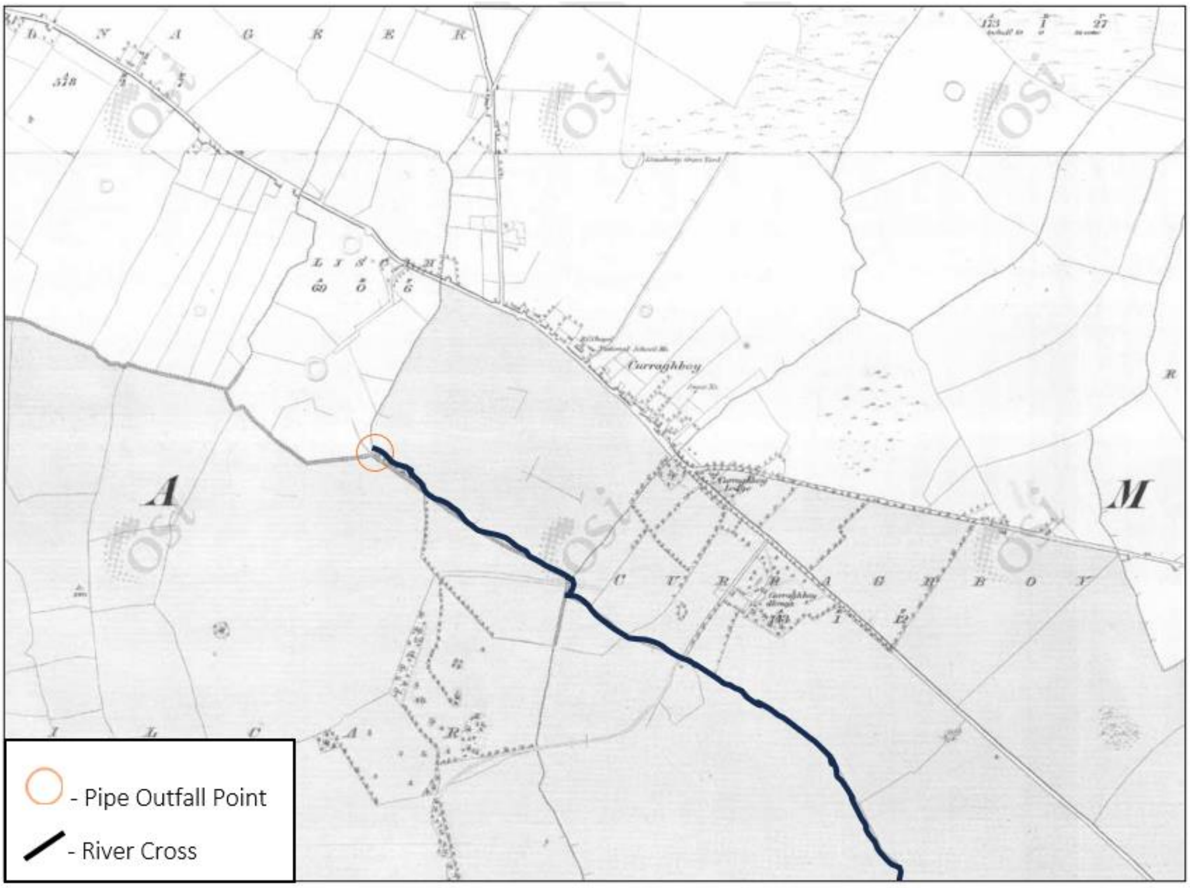


Figure 4.8 - 6 Inch First Edition Historic Mapping of Pipe Outfall Location and Curraghboy Village

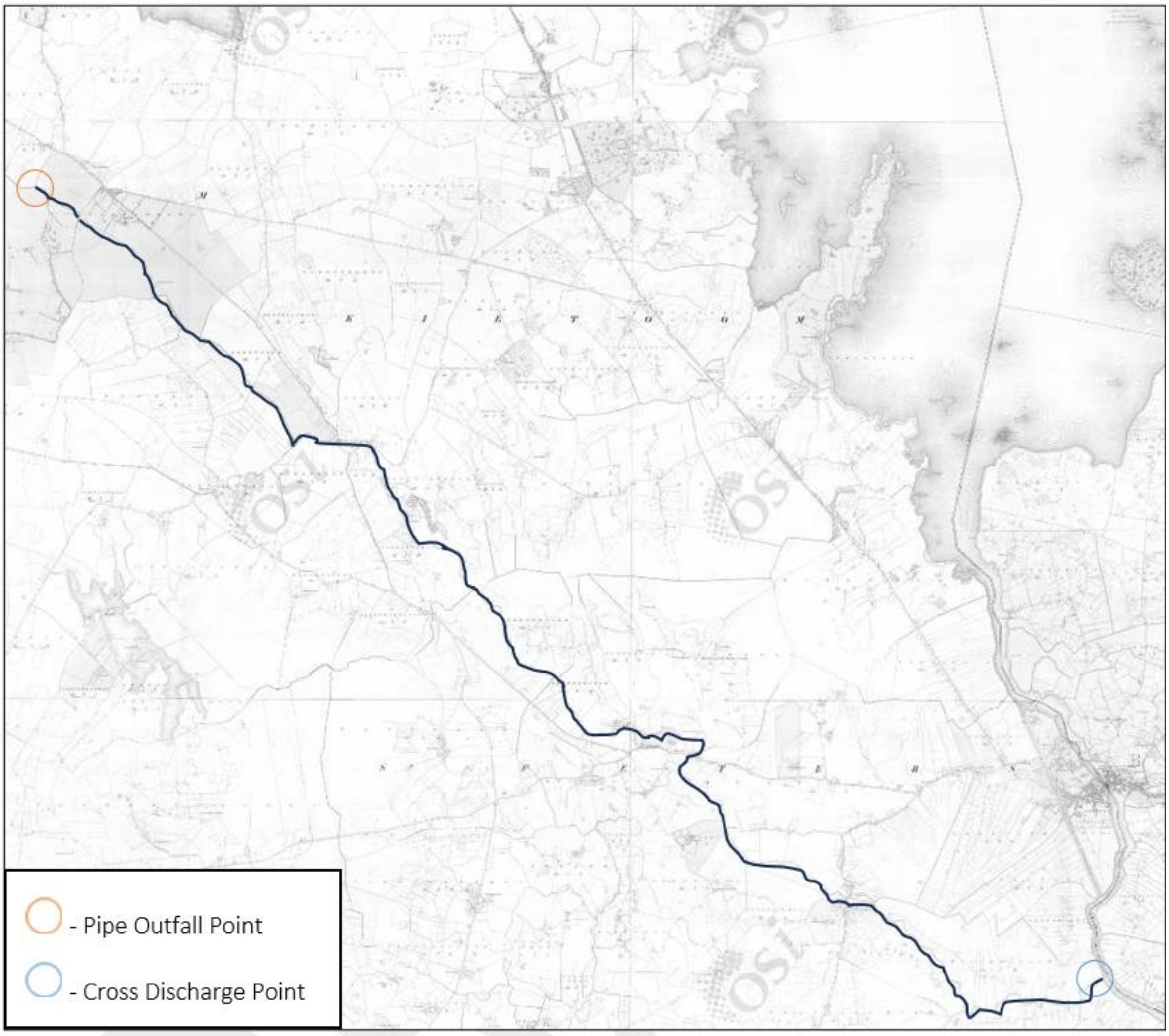


Figure 4.9 - 6 Inch First Edition Historic Mapping along Length of Cross River

4.2.4 Flood History

The Past Flood Events recorded along the Cross River were obtained from the Office of Public Works (OPW) *floodinfo.ie* website and are included on Figure 4.10 below. Over the majority of the Cross River there are no recorded flood events. There is a record of flooding at the downstream end of the watercourse, where flood risk is predominantly due to the River Shannon and does not appear to be related to the Cross River.

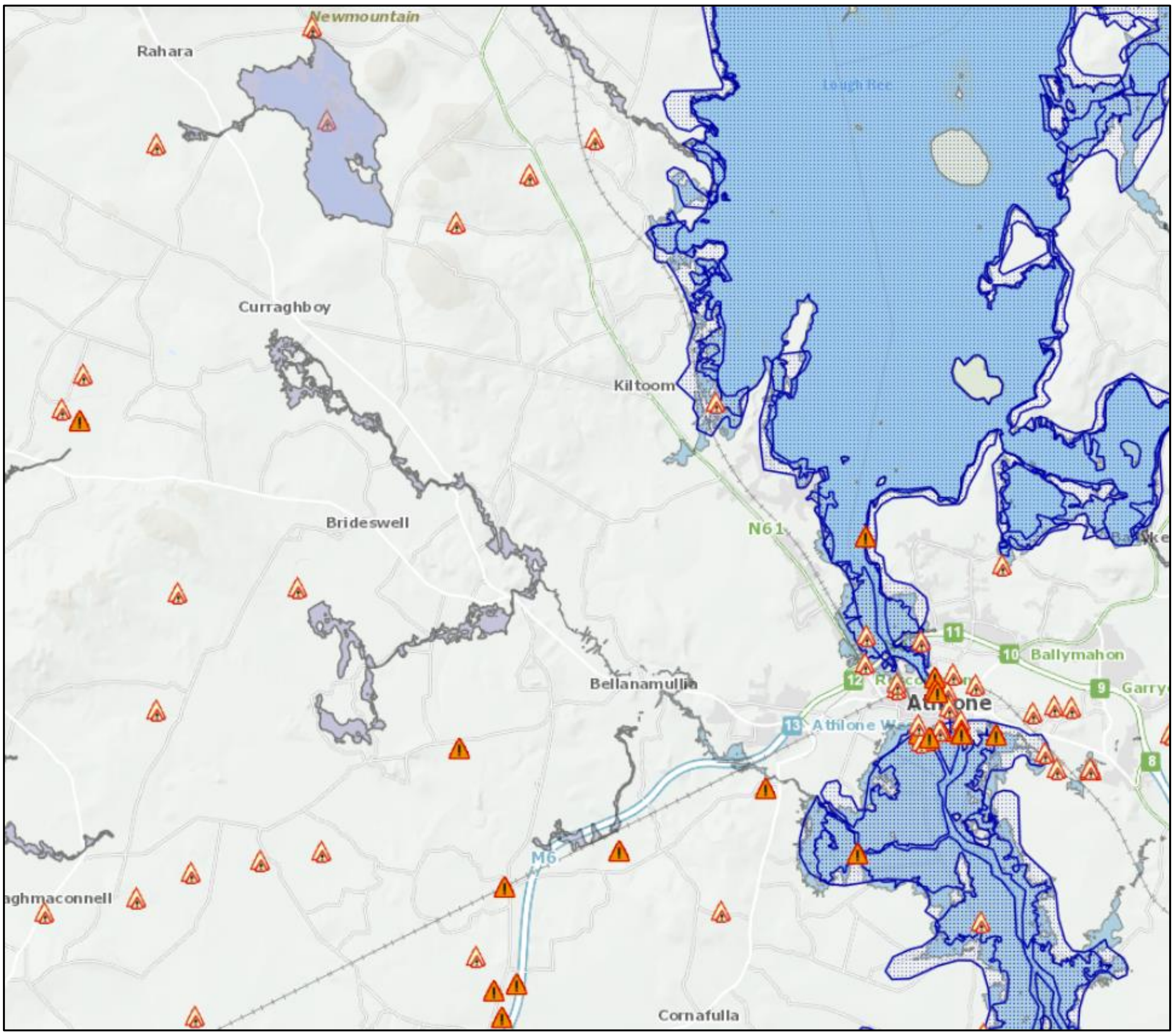


Figure 4.10 – Location of Past Flood Events (floodinfo.ie)

4.2.5 Predictive Flood Mapping

Fluvial flooding associated with the River Cross was assessed through a combination of the National Indicative Fluvial Maps (NIFM) and the Shannon Catchment Flood Risk Assessment and Management Study (Shannon CFRAMS) predictive flood map data.

The Shannon CFRAMS is an extensive study for the Shannon River Basin District which has been carried out by the OPW in conjunction with Jacobs Consulting Engineers. The OPW has subsequently published Flood Maps for the Shannon River Basin District based on this which provide data for 10, 100, and 1,000-year return period floods for both current and future scenarios, where the future scenario accounts for the effects of climate change and associated sea level rise. For the purpose of this assessment, and on the basis that this is a temporary solution, only the current scenario flood levels without the effects of climate change have been used.

The aforementioned maps are available on *floodinfo.ie*, which was previously used to identify past flood events. An extract of the fluvial flood extents map which shows 1% AEP and 0.1% AEP events in proximity to the Cross River is provided in Figure 4.11. This shows significant flooding in the Callows area of the catchment along the final 3.5 km of the Cross, it being evident that this is as a result of extreme flows in the Shannon. The flooding upstream of the Callows depicted in this map is more localised, with extreme water levels contained by the banks in a number of locations, and water level exceeding bank level and flowing onto agricultural land in certain areas to the west and south of Athlone.

The NIFM Programme provides data for catchments greater than 5 km^2, for which flood maps were not produced under the National CFRAM Programme – it is important that any data sourced for this is read under this context. In the case of the Cross River, this mapping data begins approximately 10 km upstream of the discharge point of the river as shown in Figure 4.12. As with the CFRAM Maps, only the current scenario was investigated as part of the assessment of predicted flooding, with the effects of climate change on the watercourse not applicable for the temporary solution. The flooding predicted in these maps predominately affects low-lying agricultural land, with a 0.1% AEP event remaining confined to the banks of the river at certain locations.

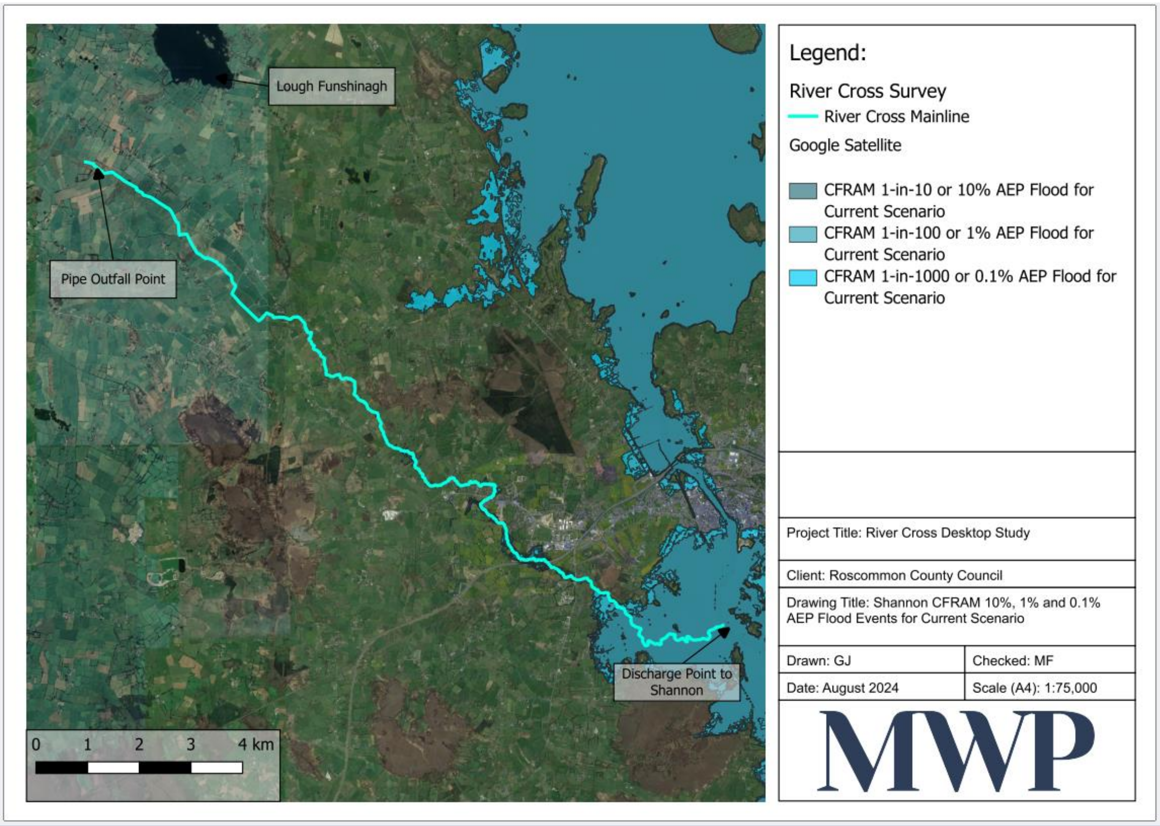


Figure 4.11 - Shannon CFRAM Predictive Flood Map for 10%, 1%, & 0.1% AEP Flood

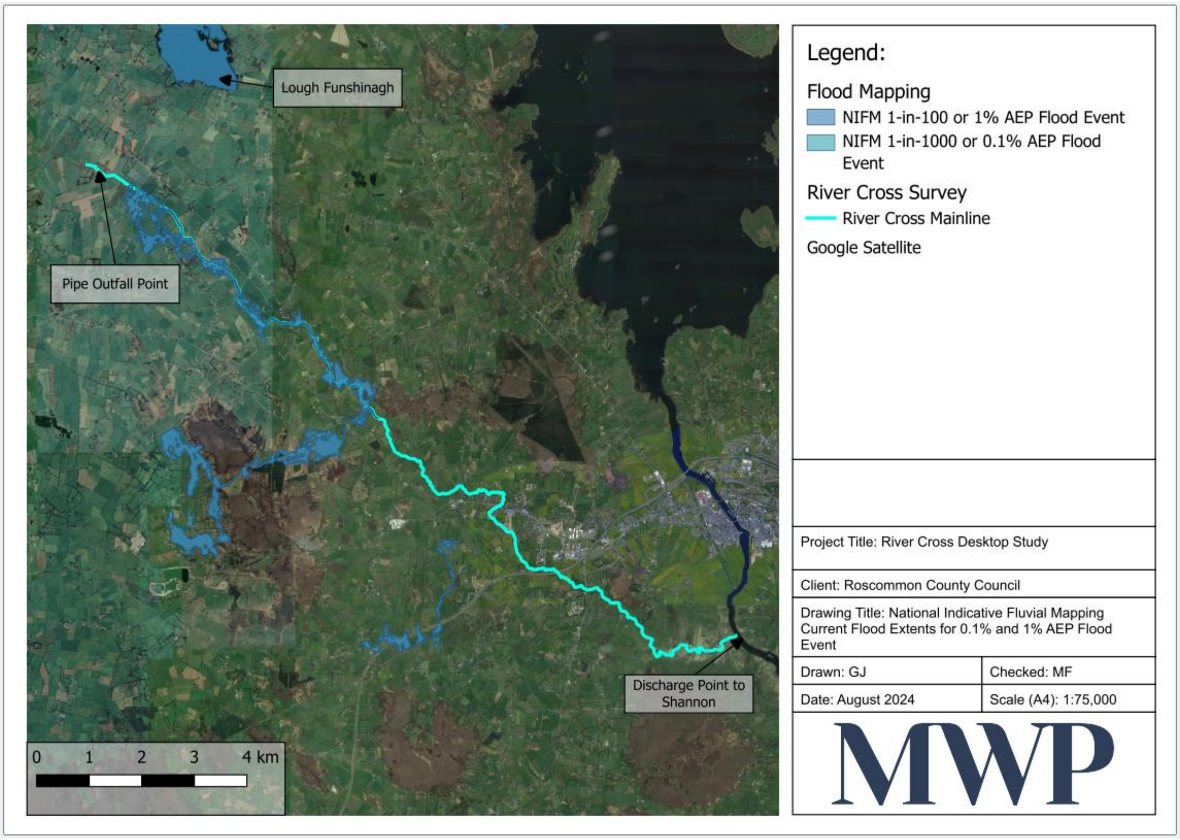


Figure 4.12 - NIFM Predictive Flood Map for 1%, & 0.1% AEP Flood

4.3 Walkover Surveys

A site walkover was undertaken by MWP alongside ARUP and Roscommon County Council on Tuesday 16th July, 2024. The intention of this was to walk the area between Lough Funshinagh and the Cross River as well as the Cross River itself to gain a greater understanding of the area and identify any elements which may influence the proposed works. As part of this walkover, photographs were taken at any point of note such as locations where the Cross intersects with the local road network. The locations of these photos are shown in Figure 4.13, and the comments noted during the survey are summarised in Table 4.1 below alongside the related images.

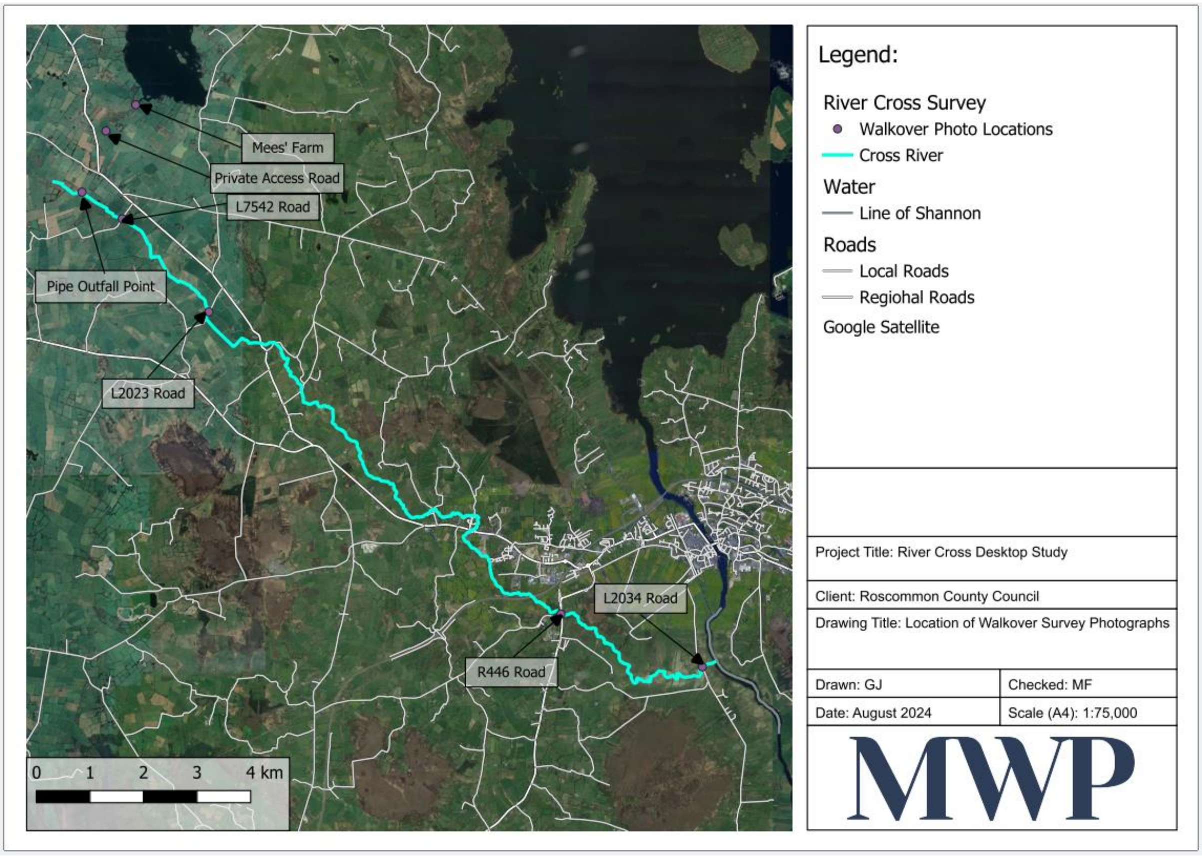
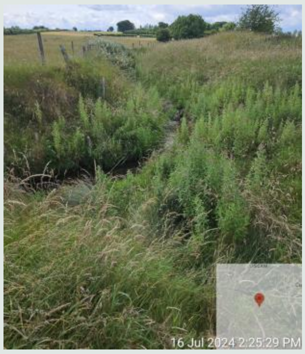
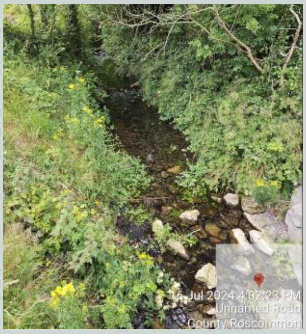


Figure 4.13 - Location of Walkover Survey Photographs

Location	Comment	Image
Mees' Farm	Photograph taken at former entrance to Mees' Farm at Lough Funshinagh along a private access road approximately 1.1 km long which connects to the R362. Entrance is no longer usable due to increased water level which has covered the roadway and partially submerged a bridge/culvert structure. An anecdotal report suggests that the water level prior to flooding was up to 1 m below the deck of the bridge/culvert structure.	
Private Access Road 750 m south of Lough Funshinagh	Photograph taken along a private access road approximately 1.1 km long which connects Lough Funshinagh to the R362. The roadway has recently been raised to ensure surface elevation is above flooding. Hedgerows which separate the roadway from neighbouring fields are covered in a white film of dried algae-like material which indicates an approximate extent of the flooding.	
Pipe Outfall Point	Photograph was taken from private agricultural land in Curraghboy Village which connects to the Cross at the approximate location of the proposed pipe outfall. At the proposed outfall point for the pump system, the channel appears to be oversized and there is evidence of artificial deepening or dredging from what appears to be deposited soil mounds adjacent to the left bank. Banks are covered in vegetation and grasses. Water level is low.	
L7542 Road Crossing	Photograph was taken from the L754 road south of Curraghboy village. Water level is low at this point as it flows beneath the road.	

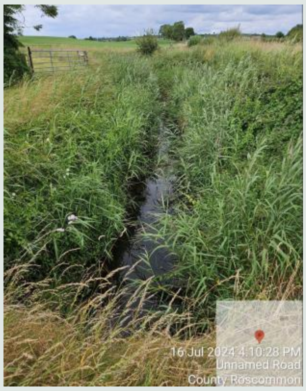
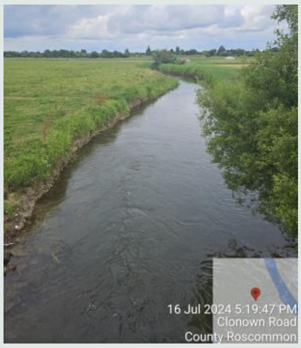
Location	Comment	Image
L2023 Road Crossing	Photograph was taken from the L2023 road south of Curraghboy village. The Cross runs parallel to the road for 160 m at this point before turning 90 degrees and crossing beneath the road. Riverbed is free from growth and straight with high banks which are covered with grass and vegetation. Channel continues to be oversized for flow.	
R446 Road Crossing & EPA Station 26221 Summerhill	Photograph was taken from the R446 regional road to the south of Athlone, at the location of the EPA Summerhill flow and water level gauge. Banks at this location are lower and flow rate in the Cross River is greater.	
L2034 Road Crossing & Callows Area on Approach to Shannon	Photograph was taken from the L2034 which is the last bridge that the Cross traverses through before discharging into the Shannon. Land becomes flat on approach to River Shannon with bank elevation only slightly higher than water level.	

Table 4.1 - Site Walkover Images and Comments

4.4 Hydrological Analysis

4.4.1 Available Flow Gauges

The EPA maintains a flow gauge with continuous flow records on the Cross River at Summerhill (Station 26221). The gauge includes 15-minute data interval stage and flow data from 2001 to present. The full flow record is included on Figure 4.14 below, with the annual maxima flows for each hydrometric year provided in Figure 4.15. The gauge is located approximately 15.3 km downstream of the pipe outfall and 4.7 km upstream of the River Shannon.

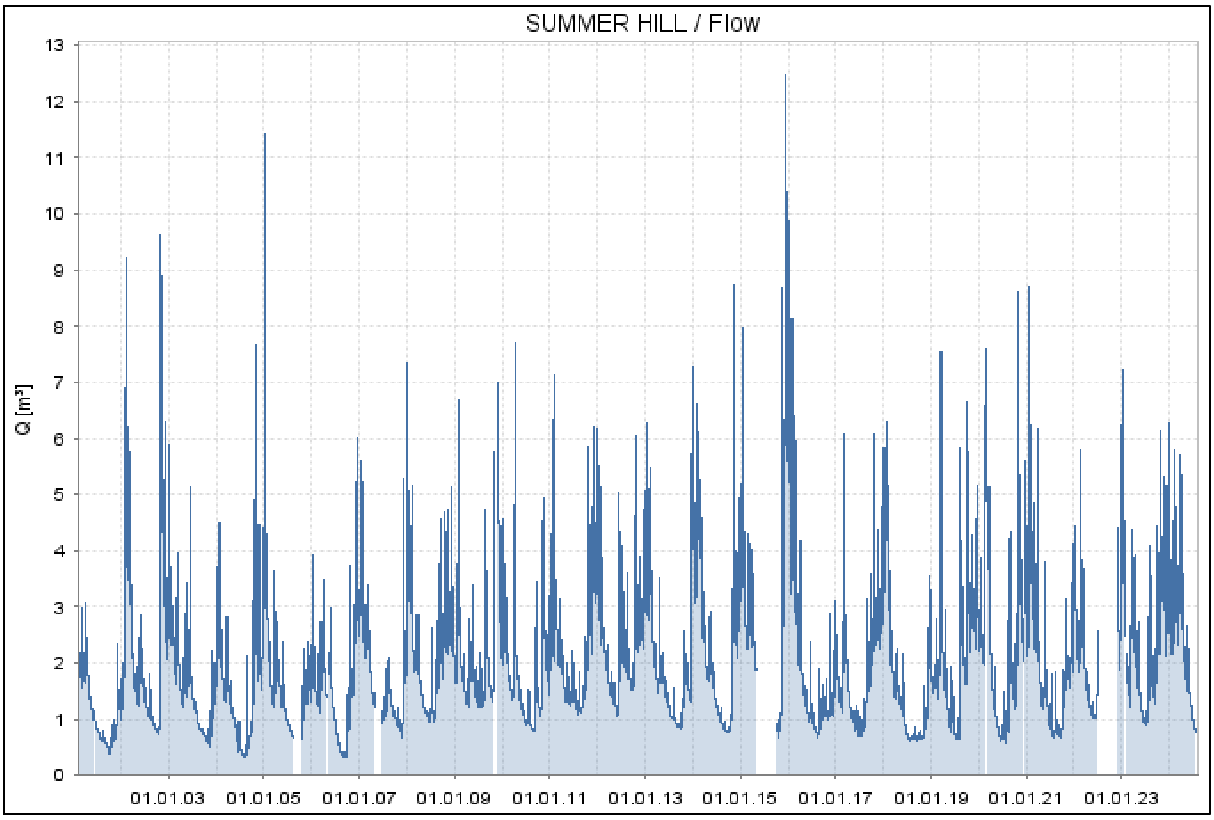


Figure 4.14 – Cross River Continuous Flow Record (EPA Gauge 26221)

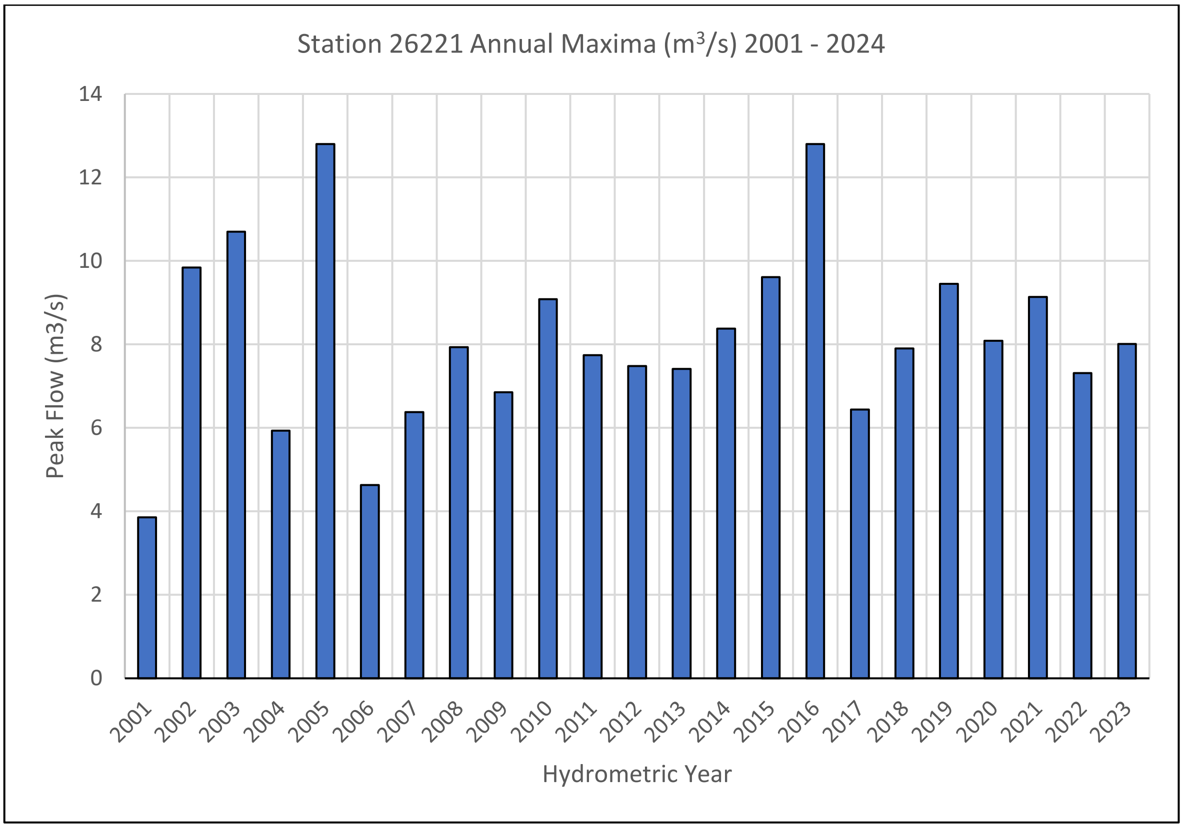


Figure 4.15 - Cross River Annual Maxima (EPA Gauge 26221)

4.4.2 Influence of River Shannon on Callows

As mentioned previously, the downstream extent of the Cross and the Callows region in particular, are believed to be under significant influence from the River Shannon, with high flows in the Shannon increasing water level in the Cross and leading to flooding in the Callows. Due to this, it is critical that the influence of the Shannon on the downstream reach of the Cross, and particularly in the Callows region, is considered as part of the hydrological analysis.

The Callows is a Special Area of Conservation (SAC) (Site Code 004096) which covers 1,318 hA along the western bank of the River Shannon from the south of Athlone to the north of Shannonbridge. SAC sites are protected under Habitats Directive (92/43/EEC) and are described as sites of community importance designated by Member States where the necessary conservation measures are applied for the maintenance or restoration, at a favourable conservation status, of the natural habitats and or the population for the species for the which the area is designated. The boundary of this is shown in Figure 4.16. This area is relatively flat with no significant elevation changes as shown in the terrain profile for the downstream reach of the Cross in Figure 4.17, making the area vulnerable to flooding in the case of a significant high flow event in the Shannon.

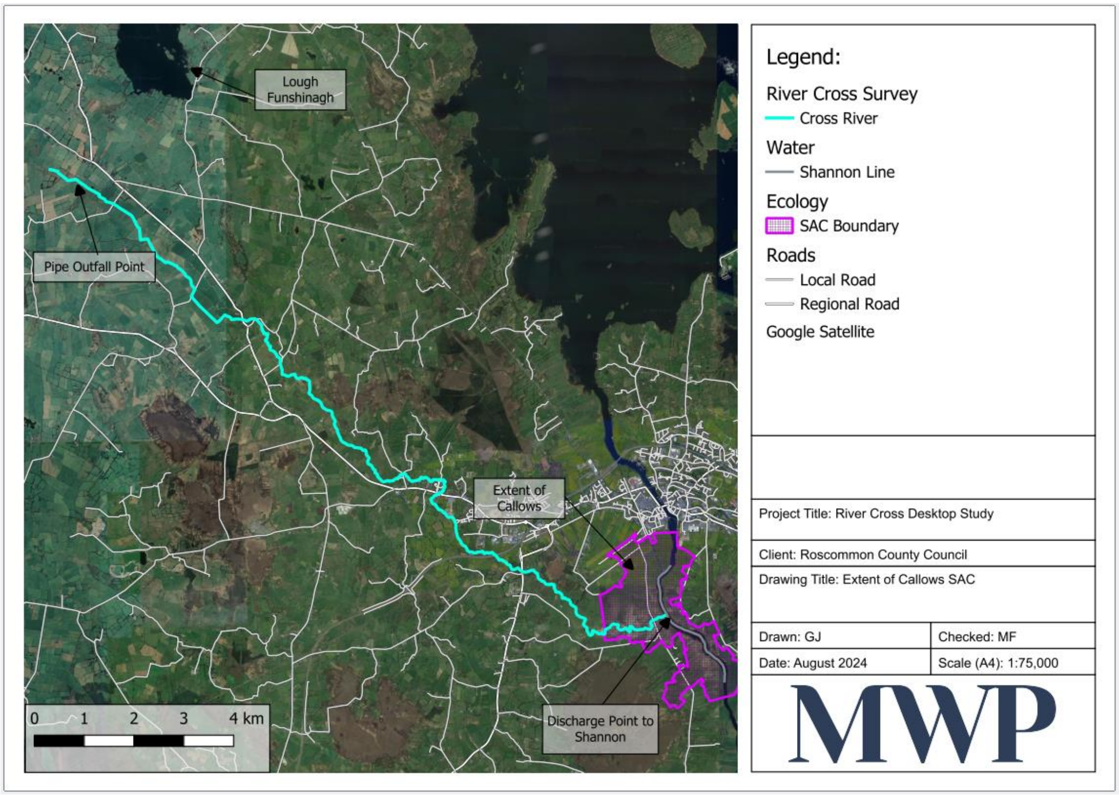


Figure 4.16 - Extent of Shannon Callows SAC

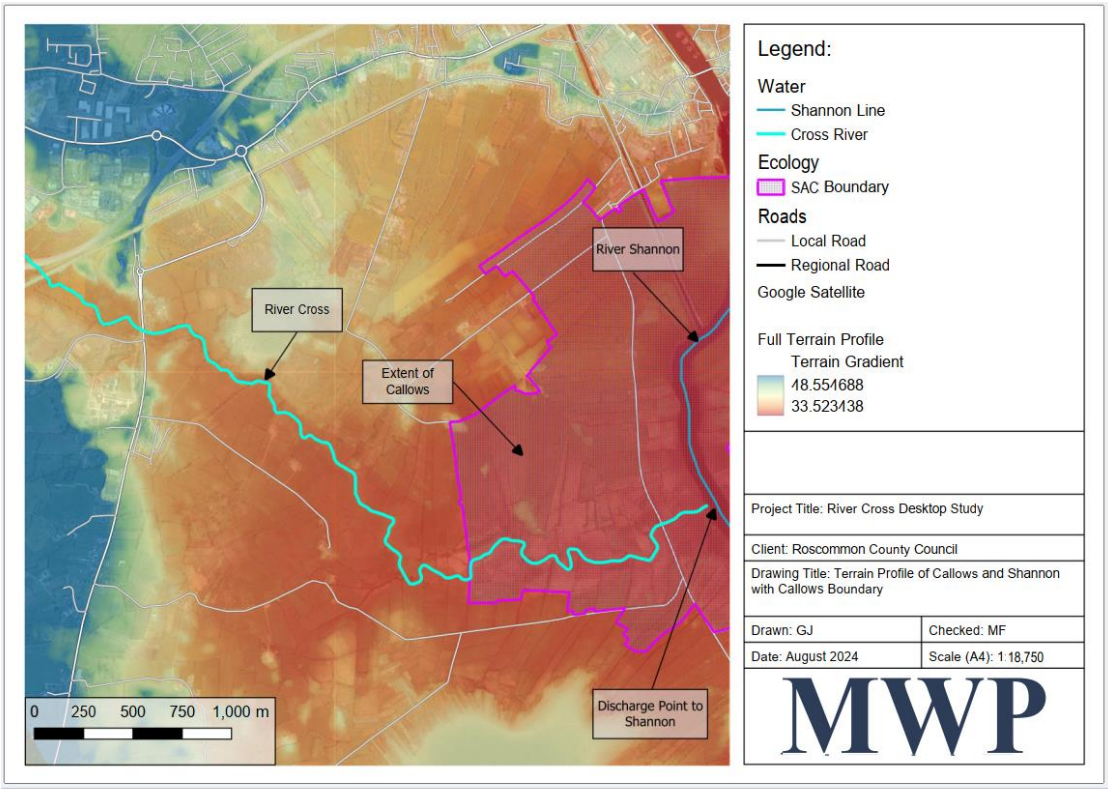


Figure 4.17 - Terrain Profile of Callows and Shannon Surface

4.4.3 High Flow Estimation

The Flood Studies Update seven variable equation with pivotal site adjustment was used to estimate peak flows along the 20 km reach considered in this assessment. Approximately 40 Hydrological Estimation Points (HEPs) were used to represent the change in peak flow along the full length of the watercourse. In addition to this, three HEP nodes were used to gain an understanding of the flow in the Shannon due to the aforementioned influence which it has on the downstream extent of the Cross. The location of each HEP is indicated on Figure 4.18 above.

The FSU Method for ungauged catchments uses Physical Catchment Descriptors (PCDs) to establish an initial estimate of the Index Flood (i.e., Q_{MED}) based on a seven variable regression equation. Ideally the application of this equation would be limited to catchments greater than 25 km^2, although it has been shown to perform reasonably well for smaller catchments, albeit with a seemingly higher Factorial Standard Error (FSE) of 1.879 rather than 1.37 (Gabre et al, 2012). The Index Flow Q_{MED} is estimated using the following seven variable regression equation which was presented in FSU WP2.3:

$$Q_{MED} = 1.237\text{x}10^{-5}\ AREA^{0.937}\ BFISoils^{-0.922}\ SAAR^{1.306}\ FARL^{2.217}\ DRAIND^{0.341}\ S_{1085}^{0.185}\ (1+ARTDRAIN2)^{0.408}$$

The initial PCD estimate can be improved by using data from a hydrologically and/or geographically similar gauged site, referred to as a Pivotal Site. In normal circumstances this approach can be considered to provide the 'best estimate' of a peak flow at a site. This utilises a comparison of PCD flow estimates and gauge data at a specific location to determine an adjustment factor which can be applied to other PCD estimates in order to improve the accuracy of calculations. For the Cross River, the pivotal site chosen was an existing EPA Flow Gauge (Station 26221) at Summerhill. This station returned an ungauged pivotal site adjustment factor of 0.975 based on a gauged Q_{MED} flow of 7.915 m^3/s and an estimated PCD Q_{MED} flow of 8.114 m^3/s.

It should be noted that the current analysis is based on the catchment area from the OPW FSU Web Portal which includes the catchment area draining to Lough Funshinagh from Node No. 26_506_5 downstream. Notwithstanding that water from Lough Funshinagh is known to discharge into the Cross River, this is considered a conservative approach because there is generally no above ground watercourse linking the areas.

As mentioned above, data relating to the Shannon was also included in the analysis to improve the accuracy of any estimations or predictions made for the downstream reach of the Cross River. Two different sources were compared to estimate flows for the Shannon: a Q_{MED} flow of 150 m^3/s as taken from an ungauged node (26_3922_4) on the OPW HydroNET Analysis WebPortal, and a lower 50% AEP flow of 98.3 m^3/s as taken from Page 7 of Annex E of *Shannon Catchment-based Flood Risk Assessment and Management (CFRAM) Study Unit of Management 25/26 Hydraulics Report* (Jacobs, 2016). Due to the large size of the catchment that contributes to the flow at the location where the Cross joins the Shannon (approximately 4,723 km^2), a conservative approach was adopted and a Q_{MED} flow of 150 m^3/s was used to ensure that the addition of pumped water in the Cross does not increase flood risk when combined with the backwater effect from the Shannon.

The above data was used to calculate the Q_{MED} Index Flow at each of the HEPs along the length of the Cross River. Example calculation data for five HEPs which represent the upstream, midpoint, downstream, and Callows extents of the Cross as well as the Shannon are provided in Table 4.2. It should be noted that while the pivotal site adjustment factor was applied to data estimated from PCDs for the Cross River, this has not been used on HEP points for the Shannon. The index flood has a return period of two years. Since it is not proposed to discharge water from the pipeline during extreme flood events, it is not necessary to estimate flows for higher return periods. Data returned for Q_{MED} flows at each cross-section location on the Cross River are summarised in Table 4.3.

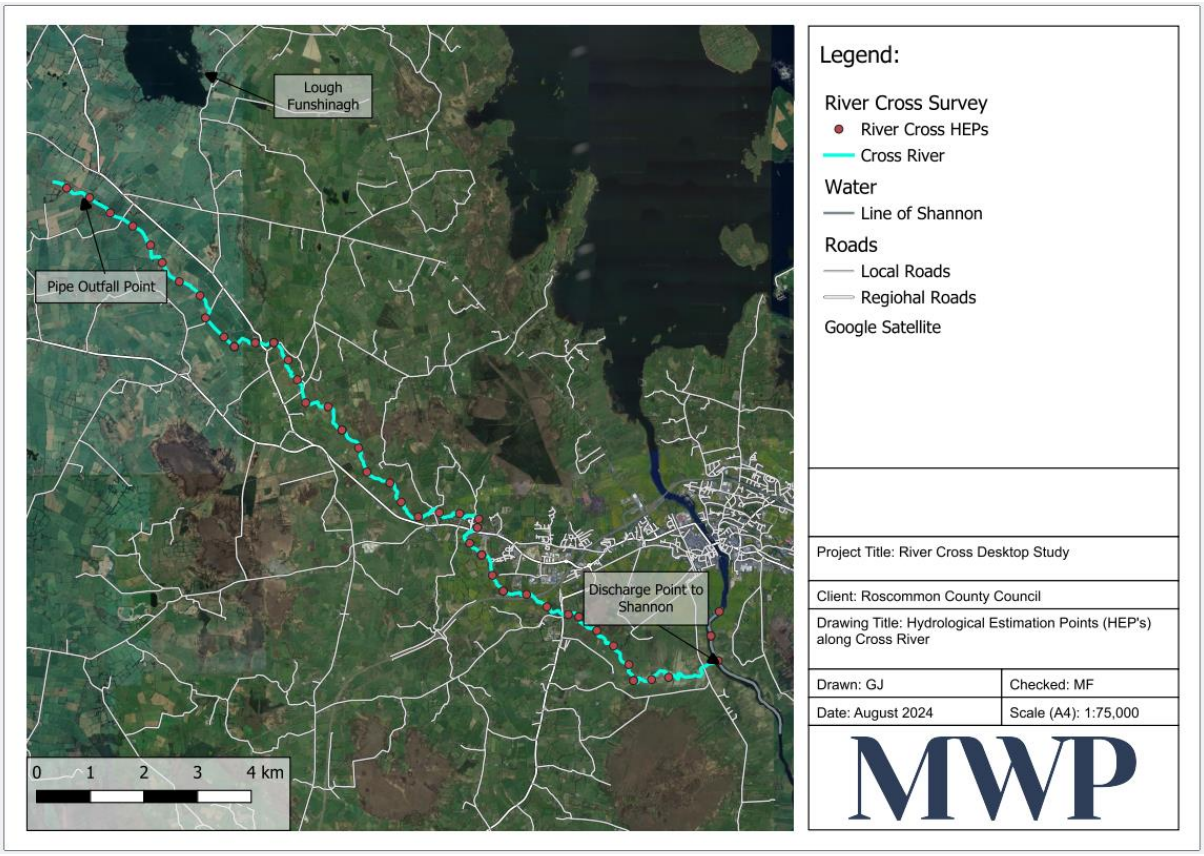


Figure 4.18 – Hydrological Estimation Points

MWP

Flow Estimation for Example HEP's Using FSU 7-Variable Equation								
	Description	Units	Upstream HEP	Midpoint HEP	Downstream HEP	Callows HEP	Shannon HEP	Source
1a	Catchment Area	km^2	4.173	50.416	107.463	108.552	4722.658	FSU Webportal
1b	Urban Catchment Area	km^2	0	0	0	0		FSU Webportal
2	Stream Slope S_{1085}	m/km	3.6454	2.5718	1.6612	1.6407	0.3091	FSU Webportal
	BFISOIL		0.7022	0.8408	0.7953	0.7952	0.693	FSU Webportal
	SAAR	mm	940.55	943.89	932.03	931.72	1055.04	FSU Webportal
	FARL		0.978	0.998	0.999	0.999	0.677	FSU Webportal
	DRAIND	km/ km^2	0.735	0.272	0.551	0.564	0.853	FSU Webportal
	ARTDRAIN2		0	0	0	0	0.2233	FSU Webportal
	URBEXT		0	0	0	0.0108	0.0064	FSU Webportal
	Q_{MED} Rural PCD Estimate	m^3/s	0.544	3.338	8.26	8.383	148.788	FSU Webportal
	Q_{MED} urban PCD Estimate	m^3/s	0.544	3.338	8.26	8.517	150.202	FSU Webportal
	Pivotal Site Adjustment Factor		0.975	0.975	0.975	0.975	N/A	FSU Webportal
	Q_{MED}		0.53	3.255	8.054	8.305	150.202	Calculate

Table 4.2 - Flow Calculation for Control Sections Using FSU

Point Information							QMED (m3/s)	
No.	Chainage (m) (If CS134 =0m	Distance (m) (From prev. nod	Node No.	Catchment (km2)	Co	Cross-Sectior	QMED PCD Onl	Pivotal Adjustec
1.00	-830.40	0.00	26_506_2	3.65		137.00	0.47	0.46
2.00	-328.70	501.70	26_506_3	4.17		134.00	0.54	0.53
3.00	173.50	502.20	26_506_4	4.55		132.00	0.59	0.58
4.00	673.50	500.00	26_506_5	32.62	•	129.00	1.81	1.77
5.00	1175.12	501.62	26_506_6	32.93		124.00	1.89	1.84
6.00	1676.10	500.98	26_506_7	33.29		118.00	1.91	1.86
7.00	2177.89	501.79	26_506_8	36.06		114.00	2.08	2.03
8.00	2678.80	500.91	26_506_9	40.85		112.00	2.37	2.31
9.00	3181.70	502.90	26_506_10	42.20		110.00	2.46	2.40
10.00	3677.40	495.70	26_506_11	42.42		108.00	2.45	2.39
11.00	4177.50	500.10	26_506_12	44.09		106.00	2.54	2.48
12.00	4438.90	261.40	26_506_13	44.19		103.00	2.56	2.49
13.00	4942.70	503.80	26_322_2	45.95		102.00	2.78	2.71
14.00	5310.40	367.70	26_322_3	48.38		100.00	2.97	2.89
15.00	5807.00	496.60	26_794_2	48.68		99.00	3.07	2.99
16.00	6309.30	502.30	26_794_3	48.87		95.00	3.12	3.04
17.00	6810.10	500.80	26_794_4	49.66		94.00	3.24	3.16
18.00	7313.50	503.40	26_794_5	50.16		93.00	3.34	3.26
19.00	7907.90	594.40	26_794_6	51.24		91.00	3.40	3.32
20.00	8403.50	495.60	26_676_2	77.53	•	90.00	5.71	5.57
21.00	8905.10	501.60	26_676_3	78.20		87.00	5.72	5.58
22.00	9405.10	500.00	26_676_4	78.91		85.00	5.75	5.60
23.00	9910.40	505.30	26_676_5	79.24		82.00	5.76	5.62
24.00	10409.40	499.00	26_676_6	80.34		80.00	5.82	5.67
25.00	10902.10	492.70	26_676_7	81.00		76.00	5.78	5.64
26.00	11400.70	498.60	26_676_8	81.41		71.00	5.83	5.69
27.00	11891.10	490.40	26_676_9			67.00		
28.00	12070.10	179.00	26_676_10	82.27		60.00	5.77	5.63
29.00	12570.10	500.00	26_1460_2			59.00		
30.00	12899.50	329.40	26_1460_3	82.77		53.00	6.00	5.85
31.00	13401.10	501.60	26_1461_2			49.00		
32.00	13773.30	372.20	26_1461_3	100.19	•	45.00	7.79	7.60
33.00	14280.50	507.20	26_4018_2			42.00		
34.00	14779.10	498.60	26_4018_3	102.73		39.00	8.13	7.93
35.00	15280.50	501.40	26_4018_4			32.00		
36.00	15521.60	241.10	26_4018_5	104.17		30.00	8.11	7.91
37.00	16023.80	502.20	26_1000_2			28.00		
38.00	16519.60	495.80	26_1000_3			26.00		
39.00	17019.10	499.50	26_1000_4	107.46		23.00	8.26	8.06
40.00	17393.70	374.60	26_1000_5	107.96		20.00	8.28	8.08
41.00	17889.60	495.90	26_1111_2			18.00		
42.00	18388.00	498.40	26_1111_3	109.04		15.00	8.36	8.15
43.00	19684.40	1296.40	26_1112_2			12.00		

Table 4.3 – Summary of Estimated QMED Values along Cross River Reach

4.4.4 Low Flow Estimation

As fluctuating flows can be anticipated during the pumping operation, an assessment of river flows between the 50th Percentile (Q50%) and 95th Percentile (Q95%) has been performed to facilitate an assessment of the impact of the pumping during low flows. In this the Q95% flow corresponds to a low flow which is exceeded 95% of the time, and similarly a Q50% describes a low flow which is exceeded 50% of the time.

The low flow estimates were examined using the HydroTool available on the EPA Water Maps. The Environmental Protection Agency (EPA) in Ireland commissioned an update to a hydrological model for estimation of annual flow duration curves (FDCs) in 2018. These curves plot river flow against percentage of time flow is exceeded and they are used in the assessment of natural flows, environmental flows and abstractions.

The Q50% and Q95% flows at available locations (map of locations shown in Figure 4.19) on the Cross River are provided on Table 4.4 below (Node Nos. relate to the nearest FSU node no.).

The nearest node to the outfall with publicly available data is at a location approximately 4.5 km downstream of the outfall point and corresponds to a value of 0.165 m^3/s. for a Q95% flow. Due to the distance between this node and the outfall, data for naturalised flows at the outfall location were requested from the EPA. Two datasets were returned; one slightly downstream of the outfall which accounts for the influence of Lough Funshinagh, and one at the outfall which does not include Lough Funshinagh within its catchment area. The location of these two additional points are shown in Figure 4.20. There is a significant difference in the values returned for these two points, with the 50% percentile flow estimated as 0.444 m^3/s when the lough is included in the catchment, and this decreasing to 0.057 m^3/s without the influence of the lough. Since there is currently no evidence that the water discharging from Lough Funshinagh enters the Cross River at the proposed outfall location the low flow analysis at the outfall was conservatively undertaken using the flow data which does not account for Lough Funshinagh in the catchment.

In addition to examining the variation in velocity and flow rate caused by the pumping operation during a low flow scenario in the Cross, a similar analysis was performed for a low flow scenario in the Shannon. As with the Cross, this utilised 95th percentile low flows which have been estimated by the EPA for a point along the watercourse. For the Shannon, the nearest available node was Node 26_3737 at Shannonbridge which returned a 95th percentile low flow of 14.153 m^3/s.

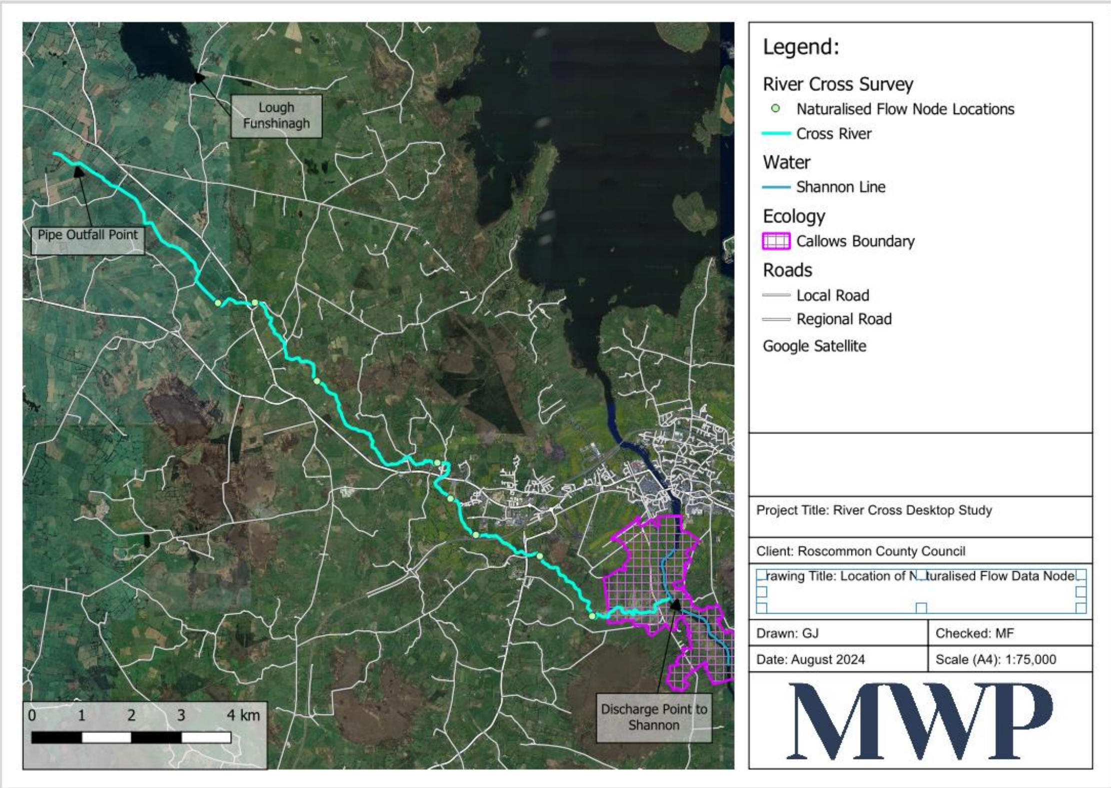


Figure 4.19 - Location of Naturalised Flow Nodes

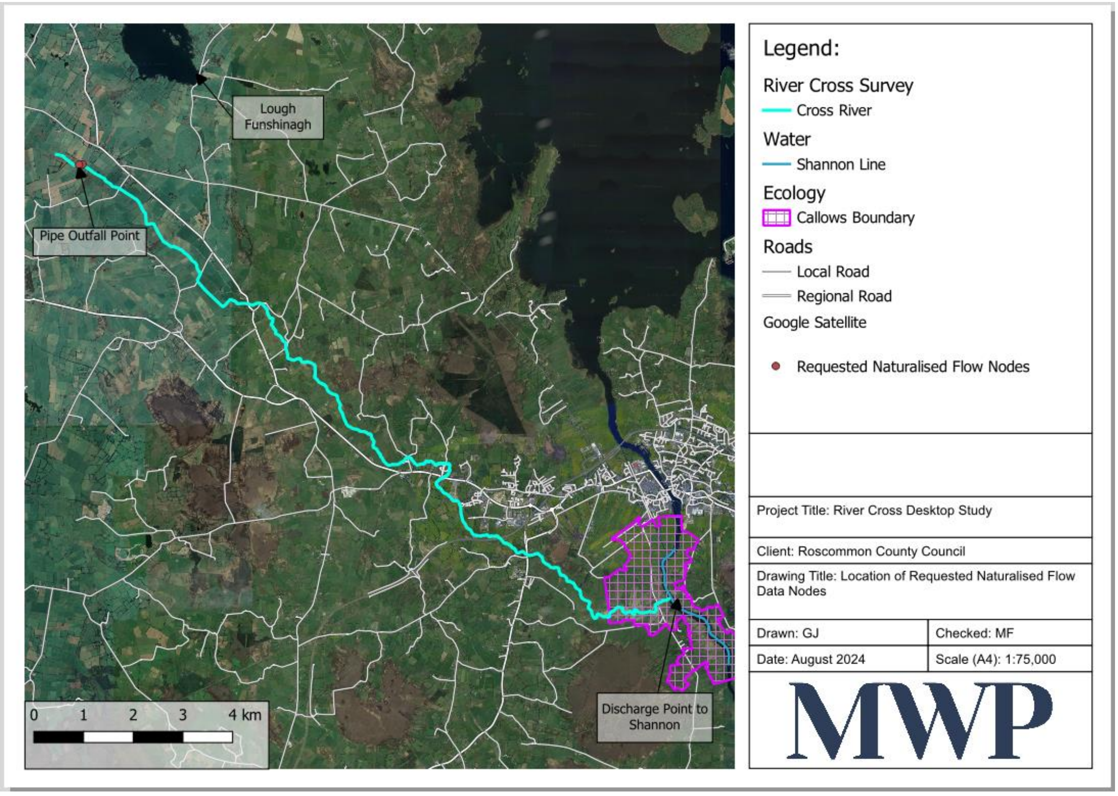


Figure 4.20 - Location of Nodes for Requested Naturalised Flow Data

Point Information							Percentile Flow (EPA)				Flow with Pump of 0.3m³/s		
No.	Chainage (m) (If CS134 =0m	Distance (m) (From prev. nod	Node No.	Catchment (km2)	Co	Cross-Sectio	Node	Q5%	Q50%	Q95%	Pump + Q95%	Pump + Q50%	Pump + Q5%
1.00	-830.40	0.00	26_506_2	3.65		137.00	REQUESTED	0.314	0.057	0.012	0.31	0.36	0.61
2.00	-328.70	501.70	26_506_3	4.17		134.00	REQUESTED	2.316	0.444	0.112	0.41	0.74	2.62
12.00	4438.90	261.40	26_506_13	44.19		103.00	26_506	3.24	0.66	0.17	0.47	0.96	3.54
13.00	4942.70	503.80	26_322_2	45.95		102.00	26_322	3.38	0.66	0.19	0.49	0.96	3.68
19.00	7907.90	594.40	26_794_6	51.24		91.00	26_794	3.76	0.75	0.21	0.51	1.05	4.06
28.00	12070.10	179.00	26_676_10	82.27		60.00	26_676	6.06	0.95	0.25	0.55	1.25	6.36
30.00	12899.50	329.40	26_1460_3	82.77		53.00	26_1460	6.05	0.96	0.25	0.55	1.26	6.35
32.00	13773.30	372.20	26_1461_3	100.19	•	45.00	26_1461	7.16	1.18	0.28	0.58	1.48	7.46
36.00	15521.60	241.10	26_4018_5	104.17		30.00	26_4018	7.34	1.23	0.29	0.59	1.53	7.64
40.00	17393.70	374.60	26_1000_5	107.96		20.00	26_1000	7.57	1.29	0.30	0.60	1.59	7.87

Table 4.4 - River Cross Low Flow Estimates at Available Locations (EPA HydroTool)

4.5 Hydraulic Modelling

4.5.1 Model Overview

The purpose of the hydraulic analysis is to confirm that the proposed pumping will not increase the risk of flooding over the length of the Cross River and also to gain an understanding of the possible impact the additional flow due to pumping may have during low flow situations. In order to assess whether an increased flow in the Cross due to the pumping operation is likely to adversely affect flood risk in the region, a 1D-Steady Flow model was created in HEC-RAS which included the full extent of the Cross River and a short portion of the River Shannon to account for the influence that the Shannon would have on the flow and water level in the downstream reach of the Cross.

This involved modelling of approximately 20 km of the watercourse from the outfall point of the pipe to the point where the Cross river discharges into the River Shannon south of Athlone (Figure 4.21). The geometry of the Cross River as well as any existing structure (e.g. bridge/culvert) along its length were input to HEC-RAS as part of the creation of the model. A 1-dimenional analysis was carried out. A schematic of the river model including a section of the River Shannon is shown in Figure 4.22 below. Figure 4.23 depicts the model at the junction of the two watercourses, as well as the terrain profile of the Callows and the locations where cross-sectional geometry has been added.

Since the Cross River flows into the River Shannon south of Athlone and water levels in the Cross are directly impacted by water levels in the River Shannon, the downstream boundary condition was set to the junction of the two watercourses, with the flow in the Shannon taken as 150 m^3/s for Q_{MED} as previously discussed. In addition to this, water level predictions in the Shannon CFRAM Study at the downstream end of the Cross River were examined. The level data provided in the CFRAMs Study is shown on Table 4.5 below; however, this was ultimately not included in the model; the Q_{MED} flow data was found to be preferable as this has a lower return period than the events depicted in the CFRAM Maps.

The channel roughness was estimated using the recommendations from Chow 1959 based on the walkover of the river reach, as summarised on Table 4.6 below.

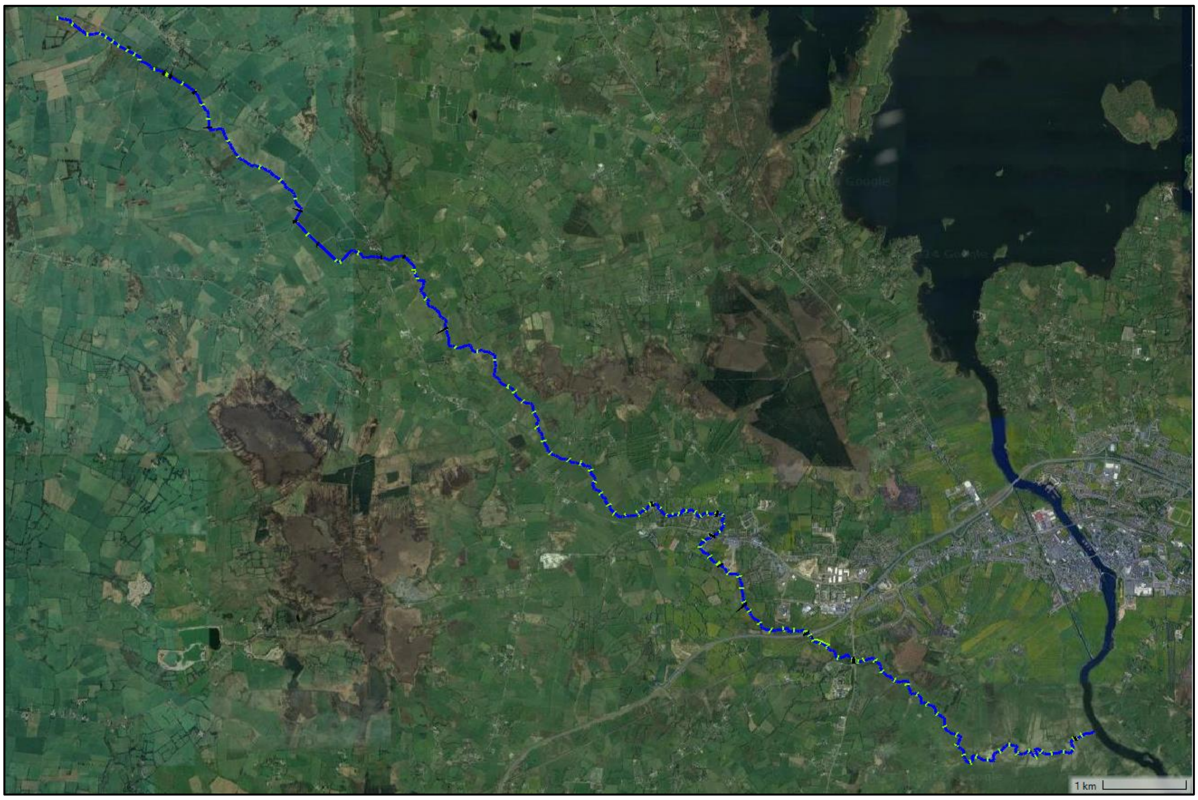


Figure 4.21 - River Cross Hydraulic Model Schematic & Extents

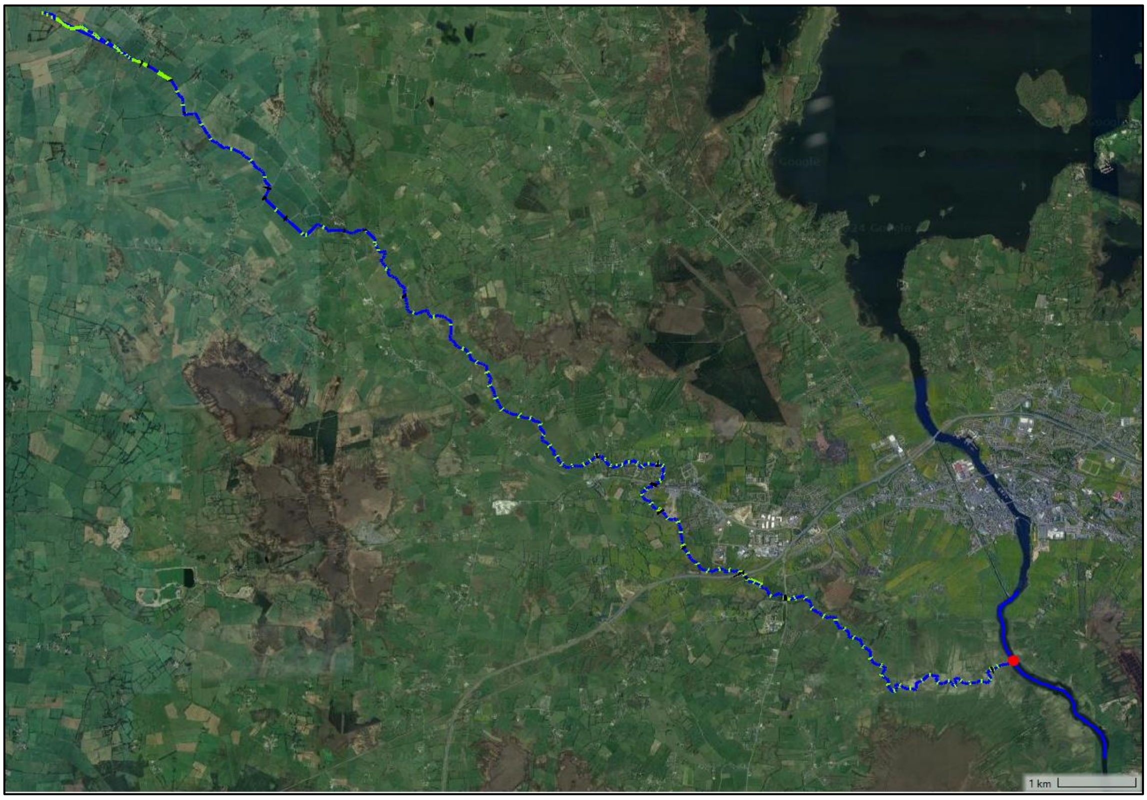


Figure 4.22 - Model with Section of Shannon

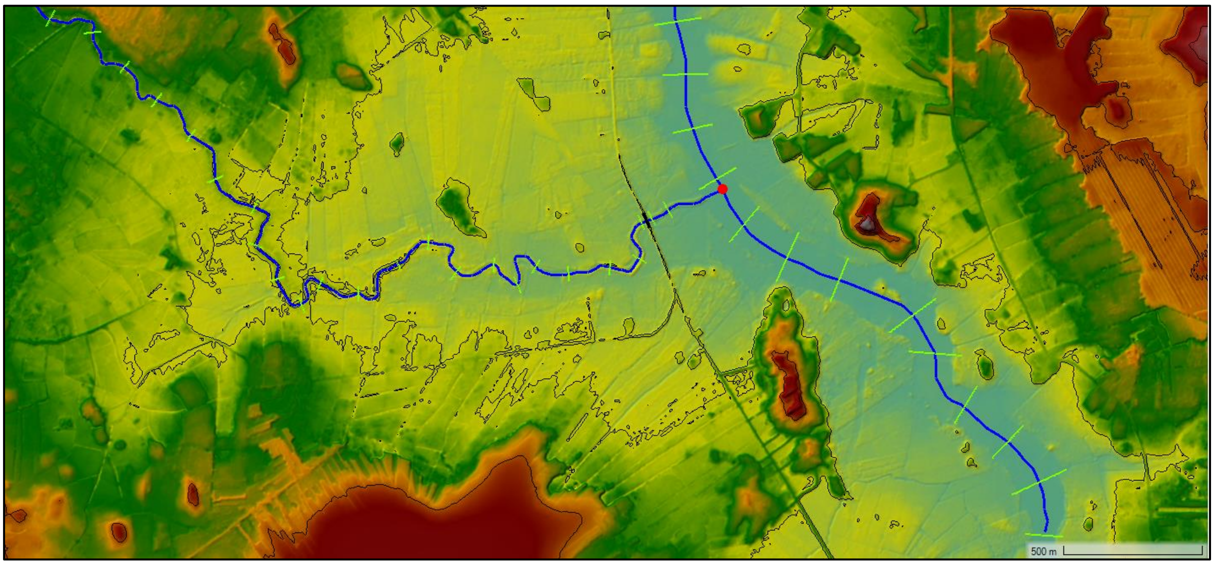


Figure 4.23 - Terrain Profile of Callows and Shannon Surface

Location	Node	10% AEP	1% AEP	0.1% AEP
Shannon South of Athlone	07MSH00682	36.01 mOD	36.43 mOD	36.74 mOD
Shannon at End of Athlone Canal	06MSH00919u	35.78 mOD	36.22 mOD	36.53 mOD
Shannon	06MSH00444	35.78 mOD	36.22 mOD	36.53 mOD
Downstream End of Cross	01CRR00060	35.25 mOD	35.25 mOD	35.25 mOD

Table 4.5 – River Shannon Flood Levels (Shannon CFRAMS Study Mapping)

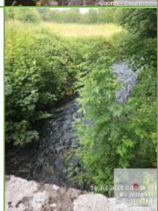
Approx Cross-Section	Location	Left Bank	Channel	Right Bank	Image	Definition (as per Chow,1959)
136	Pipe Outfall Point	0.05	0.04	0.05		Left: Scattered brush, heavy weeds. Channel: Clean, straight, full, no rifts or deep pools, but more stones and weeds. Right: Scattered brush, heavy weeds.
124	L7542	0.06	0.04	0.06		Left: Light brush and trees, in summer. Channel: Clean, straight, full, no rifts or deep pools, but more stones and weeds. Right: Light brush and trees, in summer.
112	L2023	0.05	0.04	0.05		Left: High grass. Channel: Clean, straight, full, no rifts or deep pools, but more stones and weeds. Right: High grass.
105	L7546	0.07	0.05	0.07		Left: Light brush and trees, in summer. Channel: Clean, winding, some pools and shoals, but some weeds and stones. Right: Light brush and trees, in summer.
99	L2019	0.06	0.04	0.06		Left: Light brush and trees, in summer. Channel: Clean, straight, full, no rifts or deep pools, but more stones and weeds. Right: Light brush and trees, in summer.
75	L7558	0.05	0.045	0.05		Left: Scattered brush, heavy weeds. Channel: Clean, winding, some pools and shoals. Right: Scattered brush, heavy weeds.
32	R446	0.06	0.045	0.06		Left: Light brush and trees, in summer. Channel: Clean, winding, some pools and shoals. Right: Light brush and trees, in summer.
2	L2034	0.035	0.045	0.035		Left: Short grass Channel: Clean, winding, some pools and shoals. Right: Short grass

Table 4.6 – Summary of Manning's 'n' Values Along Modelled Reach

4.5.2 Location of Control Sections

Although the full extent of the River Cross has been analysed as part of this study, results for four of the one-hundred and thirty-four total cross-sections have been presented in the main body of this report. The locations chosen are the pipe outfall point, an approximate mid-way point on the Cross River, a point on the downstream section of the Cross to the west of the Callows, and a point before the discharge point which is located within the Callows region. These four control sections portray the changes in the upper, middle, lower, and Callows sections of the Cross with the addition of the pumped flow and are representative of the changes observed throughout the watercourse. A map displaying the specific locations of the three cross-sections is shown in Figure 4.24.

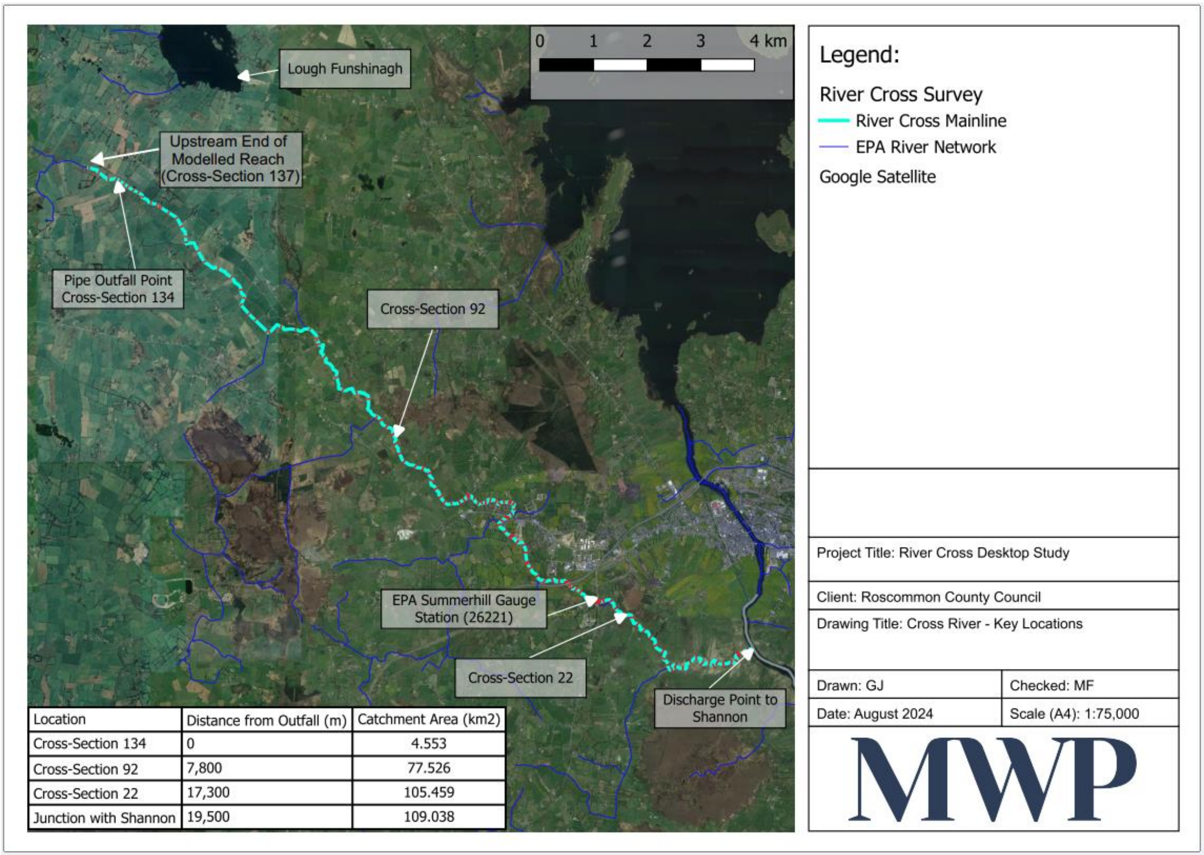


Figure 4.24 – Location of Control Sections

4.5.3 High Flow Analysis Results

A high flow analysis was undertaken to gain an understanding of the impact that both the pumping operation and uncontrolled overflows from Lough Funshinagh would have on the peak water level in the Cross River, and assess whether there was a possibility of an increase in flood risk.

This analysis shows that while the pumping operation would increase water levels by approximately 110mm at the outfall location (see Figure 4.25), the water level is still significantly below the bank level, with the channel generally oversized for the flow. However, in a do-nothing scenario where uncontrolled overflow from Lough Funshinagh reaches the Cross, a water level increase of over 200mm is predicted. A similar observation is made for the entire upper 10km of the reach, from the pipe outfall point to the control section at the approximately mid-way point, with an image of the predicted water level at the mid-point control section shown in Figure 4.26.

Towards the lower end of the reach, within the area of the Callows, the channel would be at a bank-full volume when the Shannon is experiencing a Q_{MED} flow which will tend to dissipate any energy from the Cross River flows. For this reason and, since the Q_{MED} flow in the downstream reach will be far higher than at the outfall, there would be no notable change in water level due to pumping. Typical cross sections at the downstream end of the Cross and in Callows are shown on Figure 4.27 and Figure 4.28 respectively. Following this, a similar check was performed for a case where the Shannon is not in flood, and rather has a 95th percentile low flow. A cross-section of the Cross at the Callows control section for this scenario is presented in Figure 4.29 and shows a slightly lower water level due to the lower flow in the Shannon. As with the previous scenario, there is no adverse effect on flooding risk, water level, or velocity from pumping anticipated based on the output of the computational model.

In addition to this, a longitudinal section for the full length of the Cross River has been provided. This displays the water level of the Cross and its proximity to the bank level for both the current scenario with a Q_{MED} flow (Figure 4.30), for the pumped scenario with an additional 300l/s, and with the addition of overflow from Lough Funshinagh at 600l/s. This image shows no significant change in water level with the addition of the pumped water.

The change in velocity and flow rate is summarised on Table 4.7 for key locations. It can be seen that the change in velocity for the majority of the modelled reach is negligible, as the increase in flow due to pumping is comparatively small. As would be expected, the change in velocity is largest at the upstream end of the reach where stream flows are lower. However, the velocity is still relatively low (< 1m/s). A similar table showing the depth of water at each key location for both the current and pumped scenario, as well as the corresponding depth to water from the mainline bank stations is shown in Table 4.8.

Location	Chainage from Outfall	Peak Flow (m^3/s)	Peak Flow with Pump (m^3/s)	Peak Flow with Overflow (m^3/s)	Velocity (m/s)	Velocity with Pump (m/s)	Velocity with Overflow (m/s)
Outfall Cross-Section 134	0 m	0.53	0.83	1.13	0.62	0.65	0.67
Mid-Point Cross-Section 92	7.5 km	3.26	3.56	3.86	0.39	0.42	0.42
Downstream Cross-Section 22	17 km	8.06	8.36	8.66	0.52	0.53	0.54
Callows Cross-Section 8	19 km	8.15	8.45	8.75	0.65	0.65	0.66

Table 4.7 - Velocity and Flow Rate at High Flow

Location	Chainage from Outfall	Water Depth (m)	Water Depth with Pump (m)	Water Depth with Overflow (m)	Freeboard to Bank (m)	Freeboard to Bank with Pump (m)	Freeboard to Bank with Overflow (m)
Outfall Cross-Section 134	0 m	0.357	0.469	0.56	2.07	1.96	1.87
Mid-Point Cross-Section 92	7.5 km	1.33	1.37	1.42	1.11	1.07	1.02
Downstream Cross-Section 22	17 km	2.34	2.37	2.41	0.38	0.35	0.32
Callows Cross-Section 8	19 km	2.25	2.29	2.31	0.36	0.33	0.30

Table 4.8 - Water Depth and Freeboard to Bank at High flow

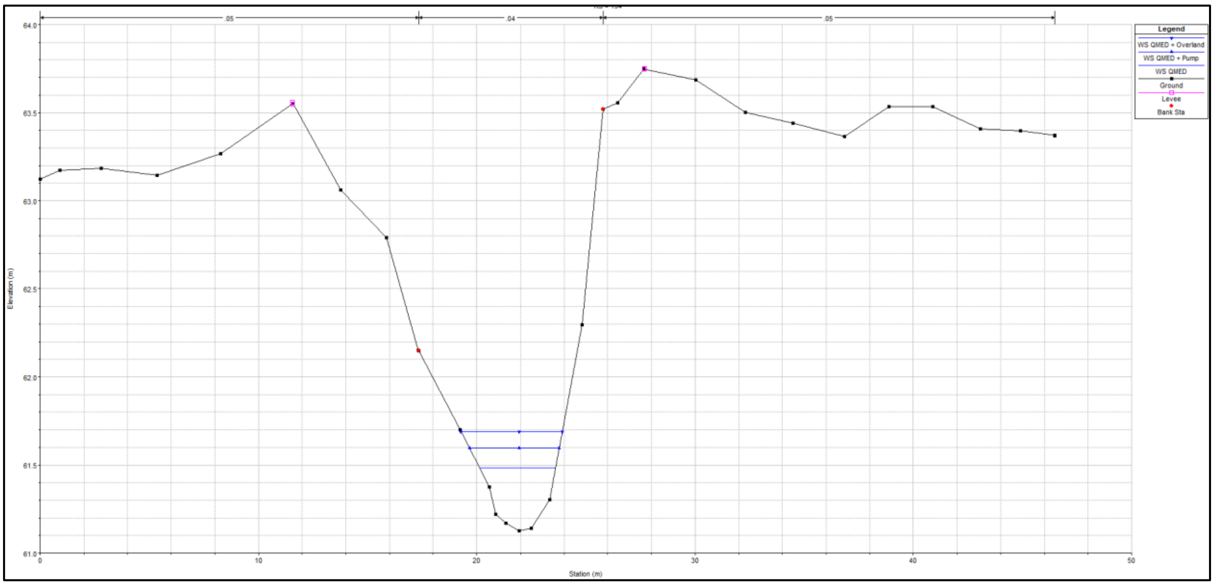


Figure 4.25 – River Cross Section at Outfall with Water Level for Q_{MED}, Pumping & Overflow

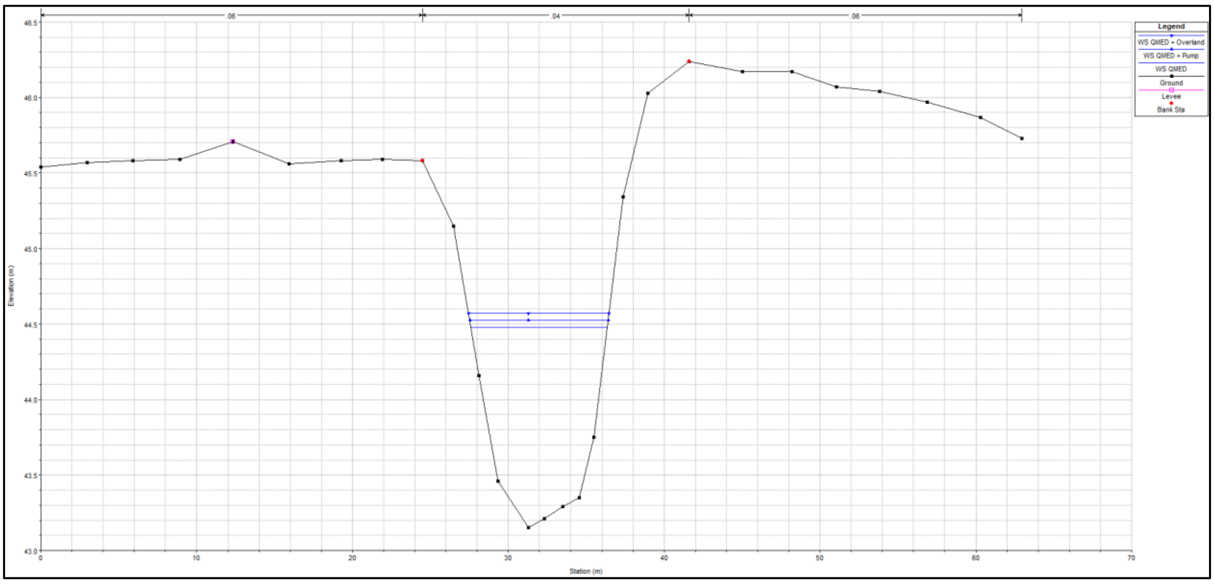


Figure 4.26 - River Cross Section at Mid-Point with Water Level for Q_{MED}, Pumping & Overflow

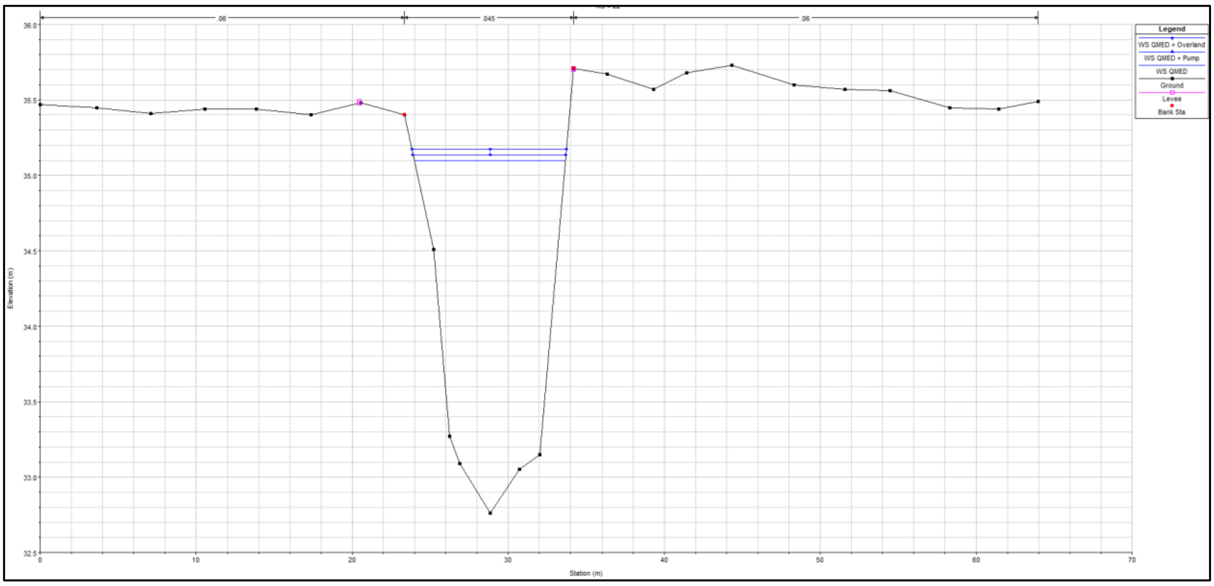


Figure 4.27 - River Cross Section at Downstream with Water Level for Q_{MED}, Pumping & Overflow

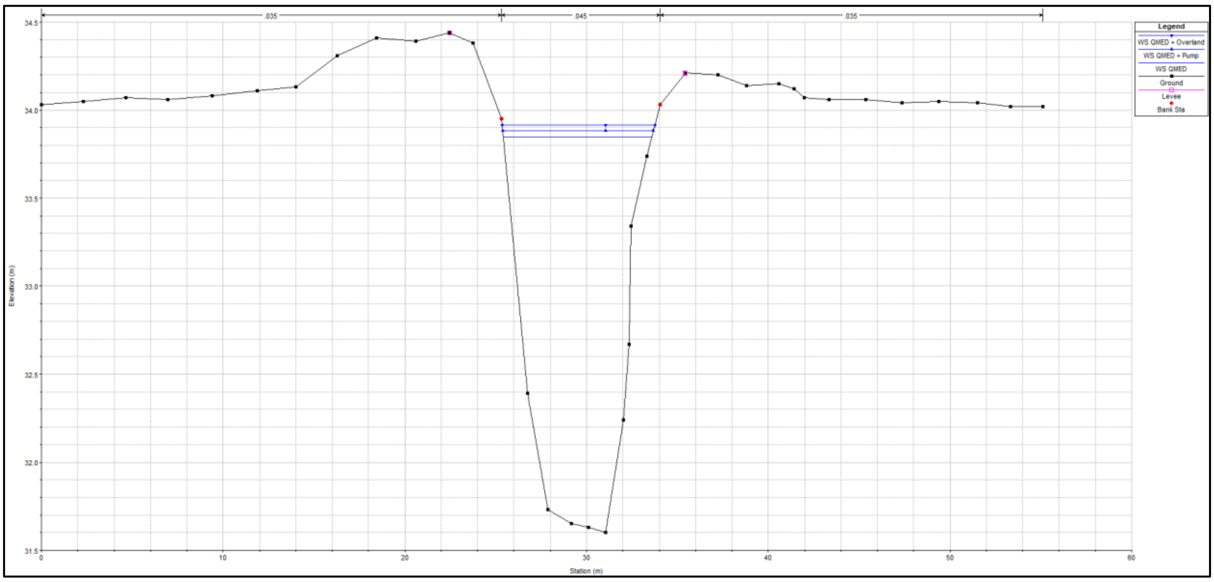


Figure 4.28 - River Cross Section at Callows with Water Level for Q_{MED}, Pumping & Overflow

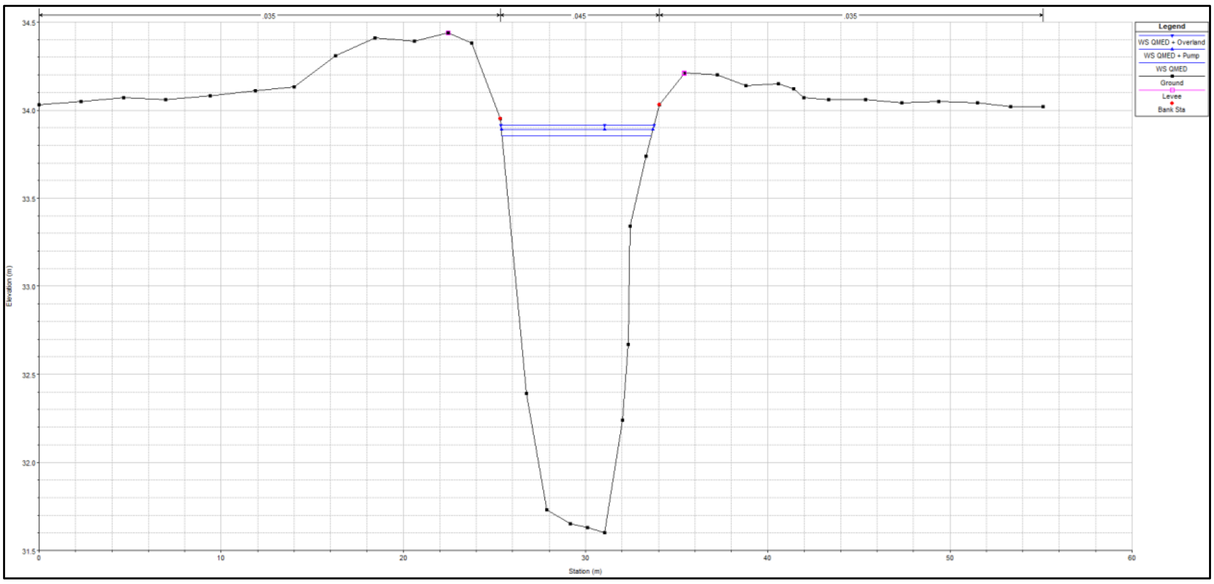


Figure 4.29 - River Cross Section at Callows with Water Level for Q_{MED}, Pumping & Overflow for Shannon Low Flow

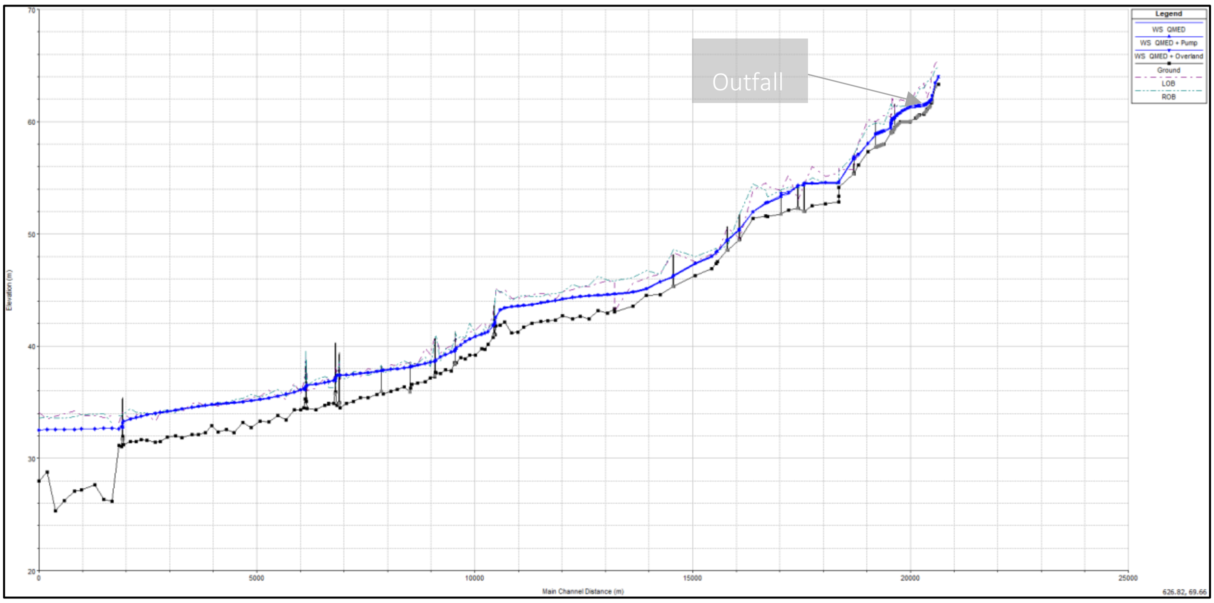


Figure 4.30 - Longitudinal Section of Cross River with Q_{MED}, Pumping & Overflow Q_{MED} Flows

4.5.4 Low Flow Analysis Results

A low flow analysis was performed for the Cross River to provide insight into the impact which the pumping operation would have on the flow and velocities in watercourse in a low flow scenario. This analysis utilised the 95th and 50th percentile naturalised flows in the Cross in combination with a Q_{MED} flow in the Shannon, and shows that, with the addition of the pump, the most significant difference in flow and velocity is observed at the outfall location, with this difference becoming less substantial further downstream. As with in the high flow analysis, the results indicate a more substantial change in flow volume and velocity at the outfall under uncontrolled overflow than during the pumping operation.

At the outfall location, the addition of the pump corresponds to an increase in water elevation of 0.141m for a 95th percentile low flow or 0.101m in the case of the 50th percentile low flow. With the addition of overflow from the Lough these increases in water level increase to 0.242m and 0.191m respectively. A significantly smaller change in water surface elevation is observed at the Callows with increases of 0.022m with pumping and 0.054m with overflow calculated for a 95th percentile flow and increases of 0.047m with pumping and 0.094m with overflow 50th for a percentile flow. The above analysis was repeated for an alternative scenario where a 95th percentile low flow is observed in the Shannon, rather than a Q_{MED} flow, however no significant difference was observed between this and the previous results.

In addition to this, a scenario where both the Cross and the Shannon experience 50th percentile low flows was also examined. In this a change in water surface elevation of 0.09m is observed at the Callows for the addition of the pumped flow, and 0.162m with overflow from the Lough. As with in the other scenarios examined, this change is more significant at the outfall point with an elevation change of 0.108m observed when the pumped water is added, and 0.191m if overflow from Lough Funshinagh is considered.

This data is displayed in the graphs provided in Figure 4.31 to Figure 4.38. The changes in flow rate and velocity associated with each scenario are presented in Table 4.9 for the 95th Percentile data in the Cross and Q_{MED} flows in the Shannon. Table 4.10 shows the results for the 95th Percentile data in the Cross and 95th percentile flows in the Shannon. Table 4.11 displays data relating to flow rates and velocities in the Cross when both watercourses are experiencing a 50th Percentile low flow. These tables display the changes in flow and velocity for five locations: a point upstream and a point downstream of the outfall location as discussed above, the approximate mid-point of the Cross River reach, a point downstream, and a point at the beginning of the Shannon Callows.

Location	Chainage from Outfall	Naturalised 95th Percentile Flow (l/s)	Naturalised Flow with Pump (l/s)	Naturalised Flow with Overflow (l/s)	Velocity (m/s)	Velocity with Pump (m/s)	Velocity with Overflow (m/s)
Outfall Cross-Section 134	0 m	12.2	312.2	612.2	0.48	1.15	1.39
Mid-Point Cross-Section 92	7.5 km	190	490	790	0.83	1.01	0.59
Downstream Cross-Section 22	17 km	290	590	890	0.10	0.16	0.21
Callows Cross-Section 8	19 km	300	600	900	0.06	0.12	0.17

Table 4.9 - Velocity and Flow Rate at 95th Percentile Low Flow in Cross with Q_{MED} in Shannon

Location	Chainage from Outfall	Naturalised 95th Percentile Flow (l/s)	Naturalised Flow with Pump (l/s)	Naturalised Flow with Overflow (l/s)	Velocity (m/s)	Velocity with Pump (m/s)	Velocity with Overflow (m/s)
Outfall Cross-Section 134	-10 m	12.2	312.2	612.2	0.48	1.15	1.39
Mid-Point Cross-Section 92	7.5 km	190	490	790	0.83	1.01	0.59
Downstream Cross-Section 22	17 km	290	590	890	0.10	0.16	0.21
Callows Cross-Section 8	19 km	300	600	900	0.06	0.12	0.17

Table 4.10 - Velocity and Flow Rate at 95th Percentile Low Flow in Cross with Q95% in Shannon

Location	Chainage from Outfall	Naturalised 50th Percentile Flow (l/s)	Naturalised Flow with Pump (l/s)	Naturalised Flow with Overflow (l/s)	Velocity (m/s)	Velocity with Pump (m/s)	Velocity with Overflow (m/s)
Outfall Cross-Section 134	-10 m	60	360	0.66	0.70	1.19	1.43
Mid-Point Cross-Section 92	7.5 km	660	960	1260	0.60	0.53	0.52
Downstream Cross-Section 22	17 km	1230	1530	1830	0.25	0.28	0.31
Callows Cross-Section 8	19 km	1290	1590	1890	0.28	0.31	0.34

Table 4.11 - Velocity and Flow Rate at 50th Percentile Low Flow

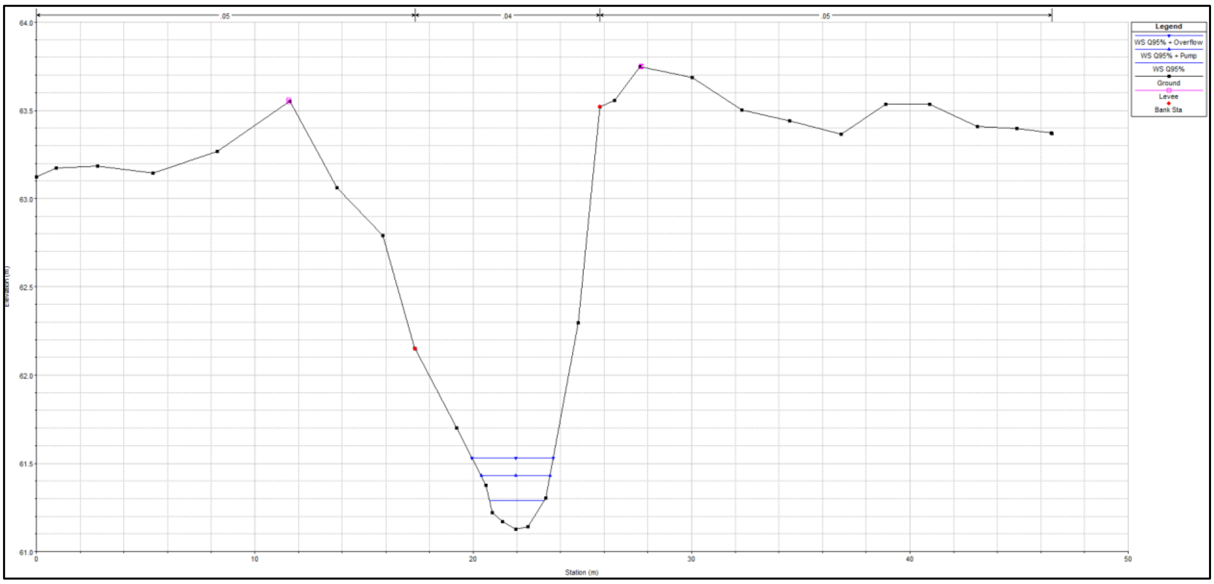


Figure 4.31 - River Cross Section at Outfall with 95th Percentile Water Level & Water Level Increase due to Pumping and Overflow

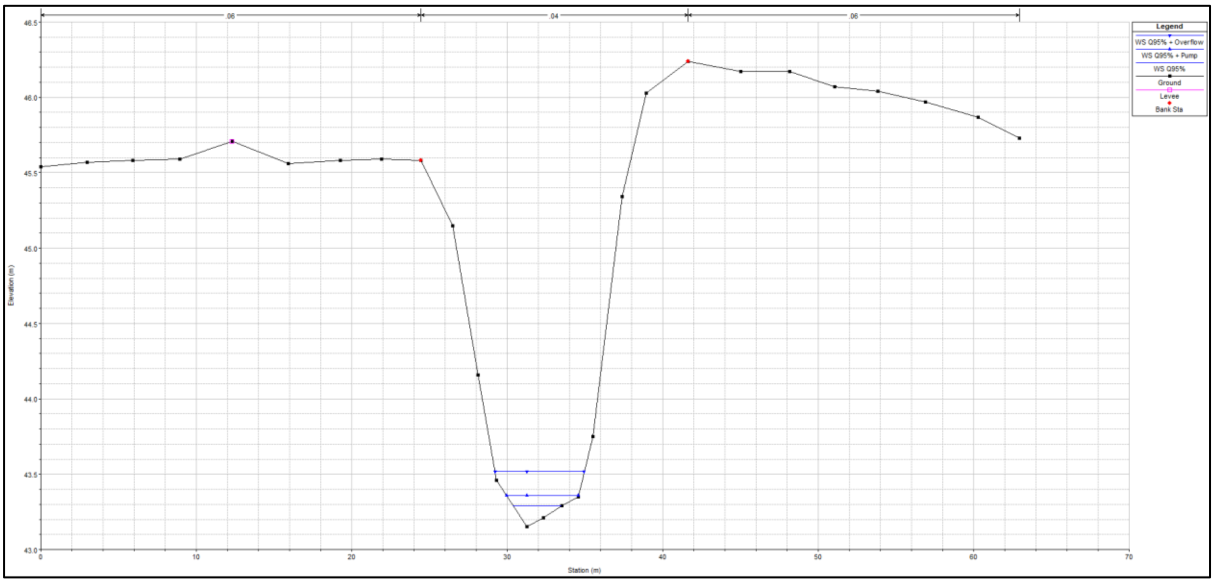


Figure 4.32 - River Cross Section at Mid-Point with 95th Percentile Water Level & Water Level Increase due to Pumping and Overflow

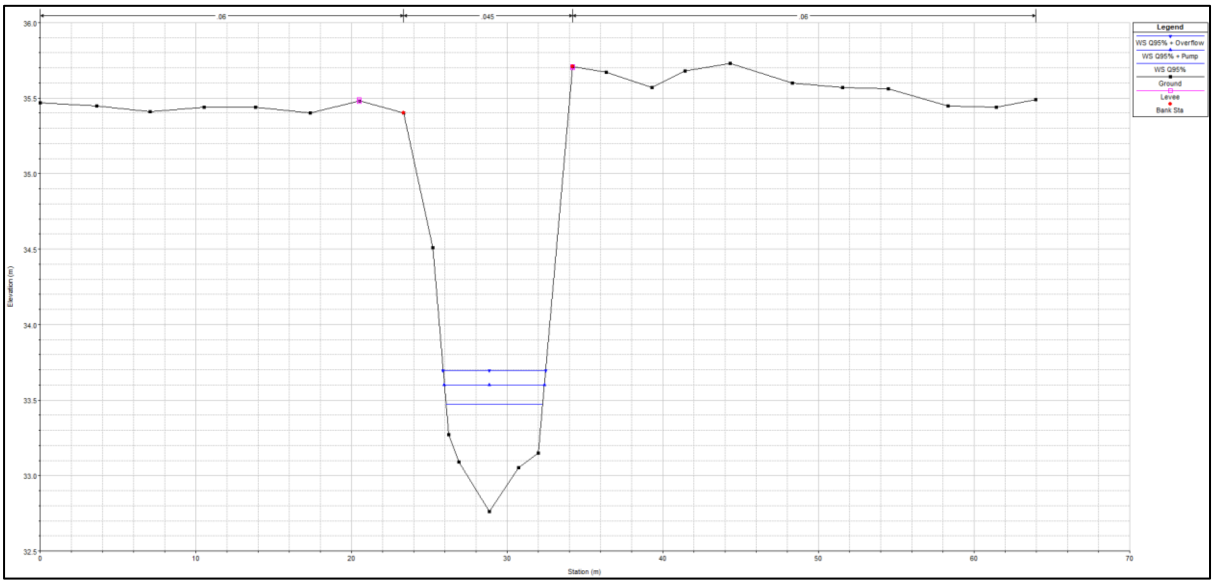


Figure 4.33 - River Cross Section at Downstream with 95th Percentile Water Level & Water Level Increase due to Pumping and Overflow

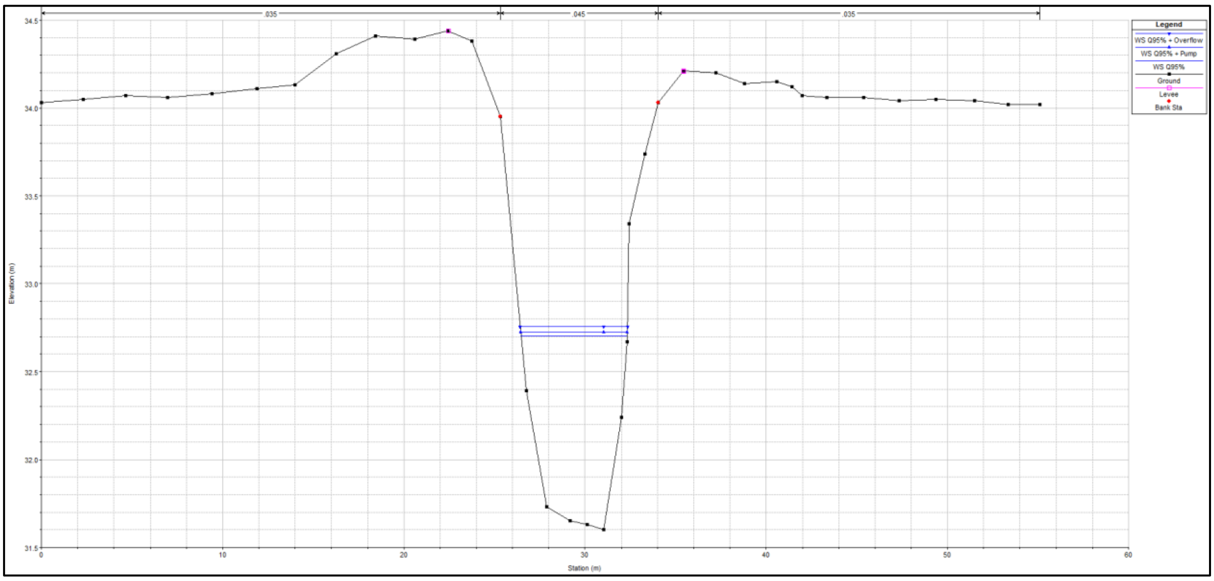


Figure 4.34 - Figure 4.33 - River Cross Section at Callows with 95th Percentile Water Level & Water Level Increase due to Pumping and Overflow

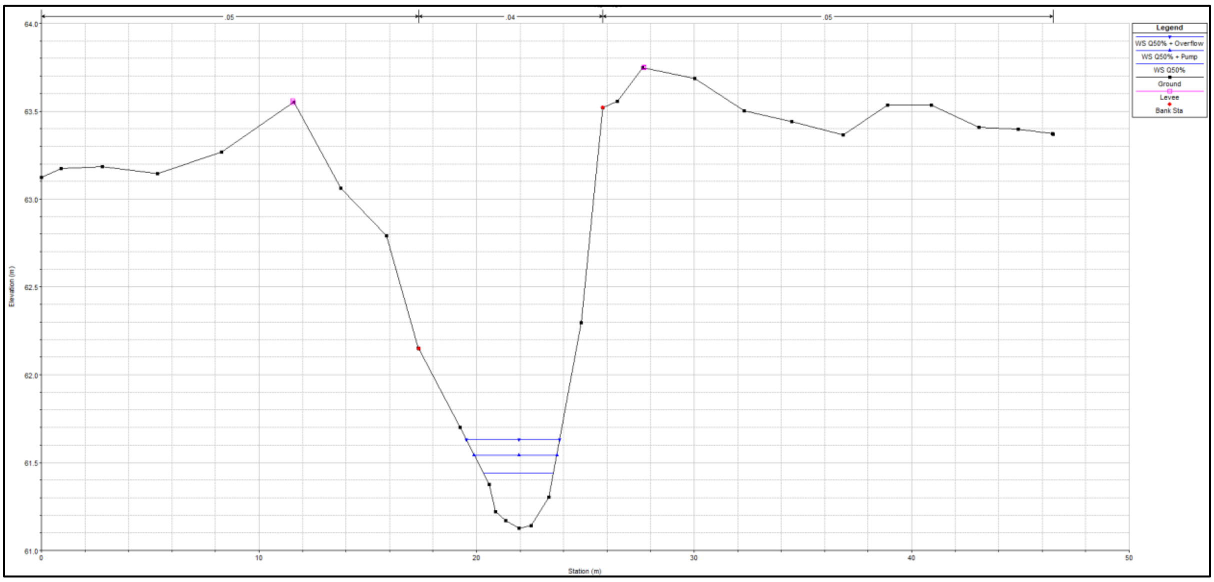


Figure 4.35 - River Cross Section at Outfall with 50th Percentile Water Level & Water Level Increase due to Pumping

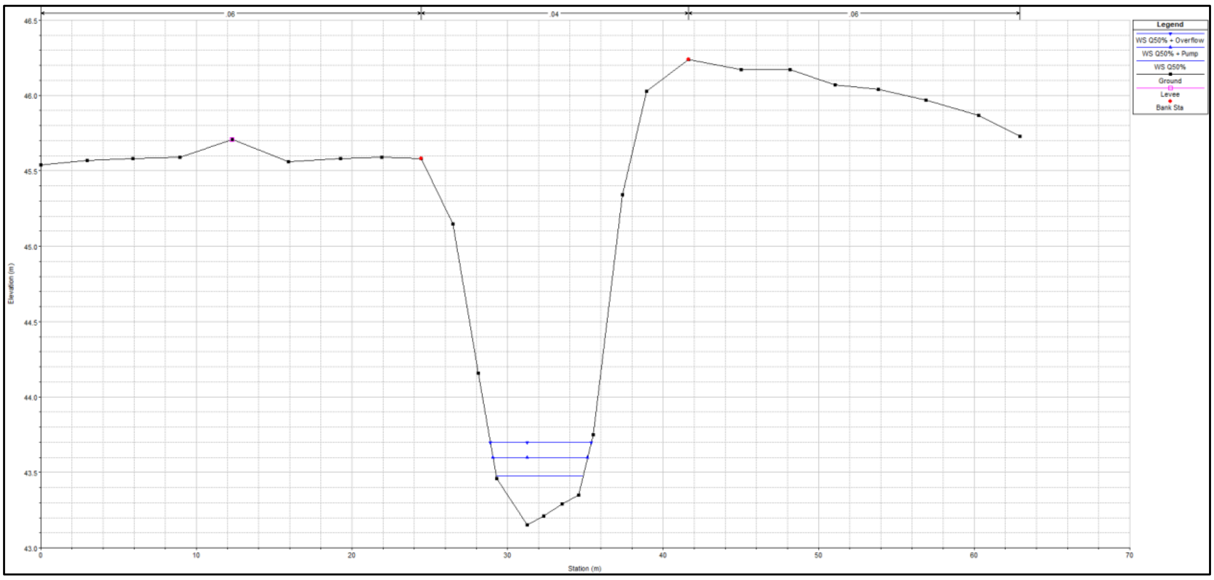


Figure 4.36 - River Cross Section at Mid-Point with 50th Percentile Water Level & Water Level Increase due to Pumping

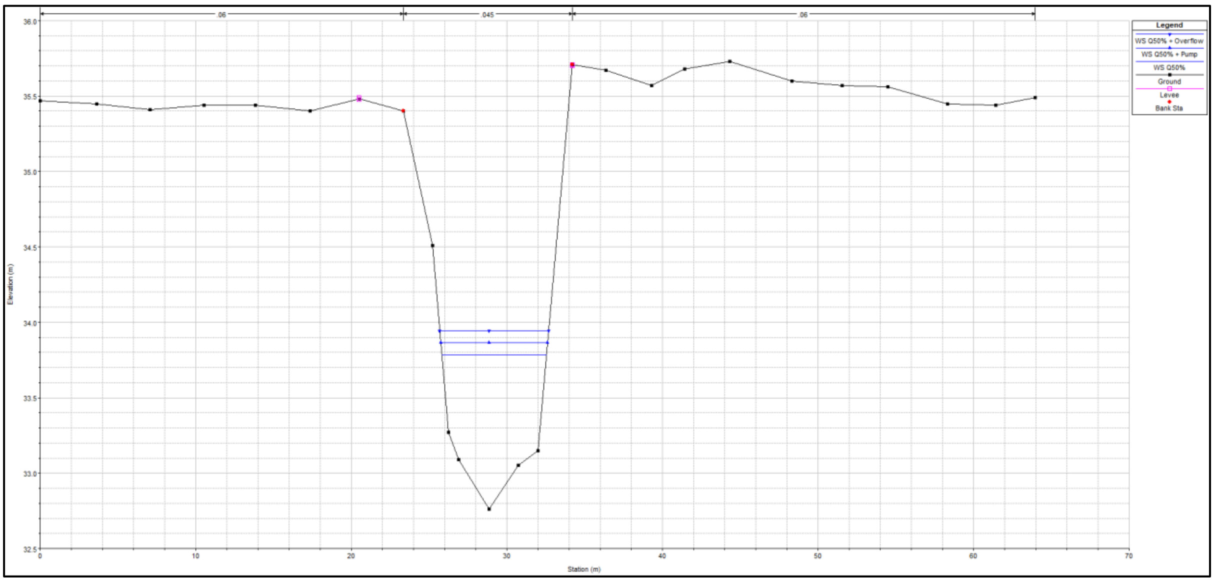


Figure 4.37 - River Cross Section at Downstream with 50th Percentile Water Level & Water Level Increase due to Pumping

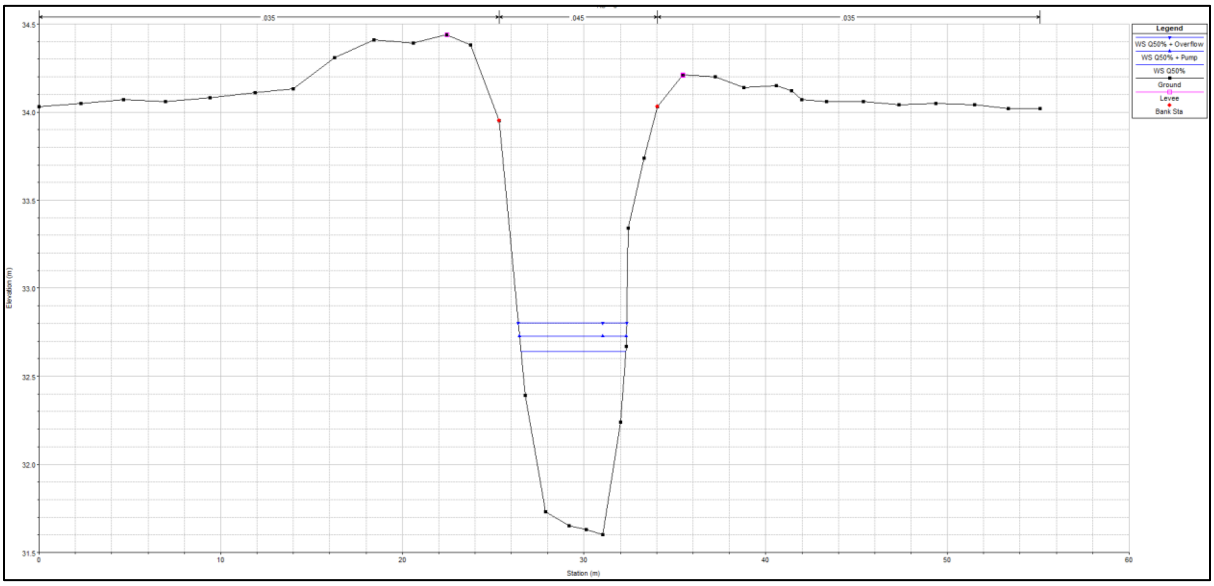


Figure 4.38 - River Cross Section at Callows with 50th Percentile Water Level & Water Level Increase due to Pumping

4.5.5 Stream Power Analysis

Stream Power for the Cross River at the location of the proposed temporary pipe outfall was assessed through use of Bagnold's Equation as taken from the Guidebook of Applied Fluvial Geomorphology (D. A. Sear, M. D. Newson, C. R. Thorne, 2003). This formula assessed stream power based on gravity, water density, discharge of the waterbody, the energy slope, and the channel bed width of the watercourse. The Q_{MED} flow, as used in the high flow analysis, was used for this assessment at the proposed outfall location.

From this, a unit stream power (ω) value of 9.62 W/m^{-1} for the stream in its current state was returned at the outfall. With the addition of the pumped flow at approximately 300 l/s, this increases to 14.47 W/m^{-1} at the outfall location. The unit stream power returned correspond to a total stream power of 18.96 W/m^2 for the existing stream and 28.50W/m^2 for the stream with the inclusion of the pumped flow. The Stream Power Analysis at the proposed outfall location is provided on Table 4.12 below.

An assessment of the available academic research on the topic such as *Channel Form and Channel Changes* (R. I. Ferguson, 1981) and *River Channel Adjustments Downstream from Channelization Works in England and Wales* (A. Brookes, 1987) suggests geomorphological inactivity in lowland rivers, with rivers which have a stream power of less than 60W/m^2 likely to lack sufficient power to erode either the stream banks or bed. Another study carried out in an *NRA Project* (NRA, 1995) and as discussed in (A. Brookes, 1987) indicates that for engineered river reaches in England and Wales streams with a power of less than 35 W/m^2 are below the high energy threshold and, as such, are unlikely to erode features. As both the current and pumped scenario return a total stream power value of less than 35 W/m^2, they fall within the parameters set out for 'low' power in this research. Therefore, it is unlikely that the addition of the pumped water will cause erosion in the watercourse.

However, while both the current and pumped flow scenarios are unlikely to reach the threshold for erosion, it is possible that the 35 W/m^2 indicative threshold could be exceeded in the case of uncontrolled overflow from Lough Funshinagh entering the Cross River if the level in the Lough continues to rise. The increased flow level in this event, with the predicted additional 600 l/s may increase flow in the Cross to 1.13 m^3/s at the outfall point which is approximately double the estimated Q_{MED} flow of 0.53 m^3/s. This would return a unit stream power of 18.87 W/m^{-1} at the outfall location – approximately twice that of the streams natural power. This corresponds to a total stream power of 37.18 W/m^2, which is marginally outside the 'low' power parameters indicated in the research suggesting a higher likelihood of erosion in a scenario where Lough Funshinagh overflows.

Following the assessment of the stream power at the location of the proposed pipeline outfall, the stream power changes with the addition of pumped flow and the Lough overflow along the full length of the watercourse were assessed. This analysis was performed at each of the three other control sections (Cross-sections 92, 22, and 08), as well as for the upstream end of the modelled reach at Cross-section 137 which is located approximately 260 metres upstream of the pipe outfall. The results are summarised on Table 4.13. The addition of the pumped flow and the Lough overflow would not impact the stream power at Cross-section-137 therefore the results are not shown on the table. The stream power analysis upstream of the pipeline outfall indicates that Q_{MED} flow at this location returns a 'low' stream power of 26W/m^2, with no change expected as a result of pumping or overflow from the Lough.

At the other three control sections, the stream power analysis shows that there is no significant difference in stream power made by the addition of pumped flow. At Cross-Section 22, due to the low slope, stream power remains below the threshold for all three flow profiles, with stream power increasing by only 9% once the pump is considered, or over 57% for the uncontrolled overflow scenario. These impacts, however, become less considerable further downstream due to the comparatively high flows predicted for a Q_{MED} flood event. Although the stream power at the downstream end of the river (both Cross-Section 22 and 08) is shown to be above the 'low' power threshold of 35W/m^2 for all scenarios, including Q_{MED}, the change in stream power due to the additional of the pumped flow is negligible and not likely to have any significant impact on erosion potential.

	Stream Power						
	Variable	Unit	Symbol	QMED at Cross-Section 134	QMED with Pumped Flow	QMED with Overland Flow	Comment
Bagnold's Equation (Guidebook of Applied Fluvial Geomorphology)	Gravity	m/s^2	g	9.81	9.81	9.81	Constant
	Water Density	kg/m^3	ρ	1,000	1,000	1,000	Constant
	Stream Discharge	m^3/s	Q	0.53	0.83	1.13	FSU 7-Variable
	Energy Slope	m/m	S	0.004	0.004	0.003	Water surface slope taken from model data
	Channel Bed Width	m	w	1.97	1.97	1.97	Taken from cross-section 134
	Index of Specfic Stream Power	N/ms	ω'	$\omega' = \frac{\rho QS}{w}$			Gravity not included as per formula - other resources checked and general confusion on addition of gravity to formula
				0.98	1.47	1.92	
	Index of Unit Stream Power	W/m^{-1}	ω	$\omega = \frac{\rho gQS}{w}$			Gravity included as per alternative formula - comparative / analysis purposes
				9.62	14.47	18.87	
	Potential Energy Expenditure	W/m^2	Ω	$\Omega = \rho gQS$			Total stream power for Cross under each scenario - gravity included
				18.96	28.50	37.18	

Table 4.12: Stream Power Analysis at Pipe Outfall (Cross Section 134)

Stream Power									
Cross-Section	Flow Profile	g (m/s^2)	ρ (kg/m^3)	Q (m^3/s)	S (m/m)	w (m)	ω' (N/ms)	ω (W/m^{-1})	Ω (W/m^2)
Cross-Section 137	QMED	9.81	1,000	0.46	0.0059	1.21	2.2	21.8	26.4
	Pump	9.81	1,000	0.46	0.0059	1.21	-	-	-
	Overflow	9.81	1,000	0.46	0.0069	1.21	-	-	-
Cross-Section 134	QMED	9.81	1,000	0.5	0.004	1.97	1.0	9.6	19.0
	Pump	9.81	1,000	0.8	0.004	1.97	1.5	14.5	28.5
	Overflow	9.81	1,000	1.1	0.003	1.97	1.9	18.9	37.2
Cross-Section 92	QMED	9.81	1,000	3.26	0.0001	5.21	0.1	0.9	4.7
	Pump	9.81	1,000	3.56	0.0001	5.21	0.1	1.0	5.1
	Overflow	9.81	1,000	3.86	0.0002	5.21	0.1	1.4	7.4
Cross-Section 22	QMED	9.81	1,000	8.06	0.0006	3.83	1.3	12.7	48.6
	Pump	9.81	1,000	8.36	0.0006	3.83	1.3	13.2	50.4
	Overflow	9.81	1,000	8.66	0.0006	3.83	1.4	13.6	52.2
Cross-Section 8	QMED	9.81	1,000	8.15	0.0010	3.25	2.6	25.4	82.5
	Pump	9.81	1,000	8.45	0.0010	3.25	2.7	26.3	85.5
	Overflow	9.81	1,000	8.75	0.0010	3.25	2.8	27.3	88.6

Table 4.13: Summary of Stream Power Analysis Results at Key Locations along Cross River

4.6 Summary of Cross River Analysis

A hydrological analysis of the Cross River was undertaken and hydraulic modelling was carried out to gain an understanding of the impact of both the proposed pumping operation, and of uncontrolled overflow from Lough Funshinagh on the watercourse. The hydraulic model utilised predicted flow estimates and the geometry of the river the water level in the Cross River for each situation considered. The analysis was carried out for both low and high flow situations.

Firstly, a high flow estimate was carried out using the Index Flood (Q_{MED}) which corresponds to a 1-in-2-year return period or a 50% Annual Exceedance Probability (AEP). Flow data for this flood was calculated through the FSU 7-Variable Equation for a number of cross-section locations across the length of the Cross. This was modelled to understand the current behaviour of the watercourse, before an additional pumped flow of 300 l/s and Lough overflow of 600 l/s were added to the model. The purpose of this was to understand whether the addition of the pump or the overflow could have an adverse effect on flood risk along the river.

Following this, a low flow analysis was undertaken based on the naturalised percentile flows in the watercourse. This provided estimates on the flow of the river for a low flow which is exceeded 95% of the time, and a low flow which is exceeded 50% of the time. This data was provided by the EPA, with additional information sought for a point at the proposed outfall to ensure the full extent of the watercourse was considered. The purpose of this assessment was to assess whether significant changes in velocities would be observed as a result of pumping during a low flow scenario, and if this is likely to have an adverse effect on the river.

Finally, a stream power analysis was conducted to determine if the pumping scenario, or the uncontrolled lough overflow scenario, have potential to lead to erosion of the banks or bed of the watercourse in a Q_{MED} event. This assessment utilised the flow, slope, and width of the stream at various locations, and assessed the power relative to a 'low' power threshold of 35 W/m^2. This was carried out at the outfall, at a location upstream, and at three other control section locations.

The analysis indicates that the majority of the Cross River Channel has ample capacity to convey the Index Flood (i.e. the 2-year return period flow). Based on the predicted bank full capacity of the channel, it appears likely that some sections have been artificially excavated or deepened, particularly at the upstream end of the modelled reach near the pipe outfall. It is clear from the analysis that pumping will not cause a flood risk in the upper part of the modelled reach.

Flooding at the downstream end of the Cross River, in the area known as the Callows, is due mainly to the River Shannon where peak Q_{MED} flows are in the order of 250 m^3/s. The analysis confirms that an additional 0.3 m^3/s pumped flows would have an insignificant impact on flood levels and would not increase the frequency or severity of flooding in the callows. Downstream of the confluence with the River Shannon, the change in flow will be insignificant.

In addition to this, the increase in velocity will be negligible over the majority of the reach (see Table 4.14). The increase will be greatest at the upstream end near the outfall. At this location it is predicted that the channel velocity would increase from c. 0.64 m/s for Q_{MED} to c.0.86 m/s when the pumped flows are added to Q_{MED}. The analysis of the low flows indicates that whilst the velocity due to pumping will increase, the water velocities are all below 1 m/s and generally significantly lower than this value (Table 4.14).

The stream power analysis was undertaken using Q_{MED} flows which indicates an increase in stream power at the outfall when pumping is included; however, the stream power for both scenarios is quite low and, based on available academic research, it appears unlikely to cause erosion in the watercourse. In the uncontrolled lough overflow scenario, however, the change is stream power is significantly higher at this location and marginally exceeds the 'low' power threshold indicated in the research.

Velocity Changes with Pumping Operation								
	Q5%		Q50%		Q95%		QMED	
Cross Section	Current Scenario	With Addition of Pump	Current Scenario	With Addition of Pump	Current Scenario	With Addition of Pump	Current Scenario	With Addition of Pump
135	0.41	0.57	0.31	0.58	0.18	0.55	0.63	0.75
134	0.71	0.7	0.63	0.69	0.4	0.62	0.63	0.68
102	1.45	1.47	0.64	0.73	0.52	0.72	0.95	0.96
100	1.54	1.59	0.74	0.86	0.51	0.73	1.29	1.33
90	0.49	0.5	0.42	0.44	0.3	0.33	0.45	0.46
59	1.18	1.19	0.89	0.9	0.82	0.89	1.16	1.17
49	0.54	0.55	0.26	0.3	0.13	0.2	0.52	0.53
42	0.36	0.36	0.11	0.13	0.03	0.06	0.39	0.39
28	0.81	0.82	0.31	0.37	0.08	0.16	0.92	0.93
22	0.39	0.41	0.07	0.09	0.02	0.03	0.43	0.44

Table 4.14 - Overall Velocity Changes with Pumping Operation

5. Flow Monitoring Points & Controls to Pumping

The proposed pumping operation will be controlled and monitored through the use of flow gauges. This system is intended to monitor the flow in the Cross River to ensure that the addition of the pumped flow does not have potential to cause a downstream flood risk.

Due to the length of watercourse and the significant change in catchment area along the reach, flows will be monitored at three locations. The monitoring locations are shown on the map in Figure 5.1 and the threshold flows to trigger a pumping shut-off are summarised on Table 5.1. The existing EPA Summerhill Flow Gauge on the Cross River is the furthest downstream flow gauge.

The flow in the Cross River will be monitored by an automated data logger or similar at 15-minute intervals for the duration of pumping. Notwithstanding that the analysis indicates that the majority of the Cross River channel has ample capacity to convey the Index Flood (i.e. the 2-year return period flow), it is proposed to stop pumping at or before this flow is reached. This will be achieved through the use of a warning system which shall be linked to the gauges and will send automatic text alerts (via mapalerter.ie or similar) once a specific water level is exceeded. Once an alert is received, the designated individual shall use the remote pump control system to shut-off pumping, with pumping only resumed when the gauge data indicates that the flow in the Cross River is once again below the specified threshold. Roscommon County Council will be responsible for these pump monitoring and control operations.

The discharge from the pumps will also be remotely monitored and the flow rate can be changed or shut off remotely. In the event that issues occur with remote operation of the pumps, and gauges indicate that flow has not stopped, a protocol will be implemented whereby the pump operator will shut-off the pumps on site.

It should be noted that these monitoring gauges are not linked to the water level in Lough Funshinagh itself – this is monitored independently through a GSI water level monitor and, due to the low rate of change in the waterbody, ample warning shall be available to shut-off the pumps in advance of the Lough reaching the desired minimum water level of 67.50 m OD.

Roscommon County Council will take responsibility for all flow monitoring for the duration of the pumping operations. The OPW will have responsibility for the operation and maintenance of gauges 1 and 2 (downstream of the pipe outfall and near the mid-point) while the EPA will continue to operate and maintain the 3rd gauge at Summerhill.

Location	Trigger	Logic	Pump Re-start
Downstream of Pipeline Outfall	Once recorded flow reaches Q_{MED} (0.53 m^3/s)	This is a precautionary approach. The water level at Q_{MED} flow is significantly lower that the main channel banks so there is no flood risk.	Pumping may restart once the flow has reduced to 0.5*Q_{MED} and there are no flood warnings applicable to the region.
Mid-Point – Near Cross-Section 92	Once recorded flow reaches Q_{MED} (5.81 m^3/s)	This is a precautionary approach. The water level at Q_{MED} flow is significantly lower that the main channel banks so there is no flood risk. This location is used as a monitoring point due to the significant length of the watercourse. Setting a trigger at this location improves the reaction time in relation to locations further downstream.	Pumping may restart once the flow has reduced to 0.5*Q_{MED} and there are no flood warnings applicable to the region.
At the existing EPA gauge at Summerhill (Station 26221)	Once recorded flow comes to within 0.6 m^3/s of Q_{MED} (7.92 m^3/s)	Although the hydraulic analysis has demonstrated that the pumped flows have insignificant impact on flood risk at the Callows, a precautionary approach has been adopted to the mitigation which seeks to avoid pumping where the Cross River has high flows. A further safety margin is provided ensure we do not	Pumping may restart once the flow has reduced to 0.5*Q_{MED} and there are no flood warnings applicable to the region.
Lough Funshinagh GSI Gauge	Before the recorded water level reaches the minimum level of 67.50 mOD	The rationale for selecting a level of 67.50 mOD as the lower limit for pumping is that this is still above the pre-2016 normal maximum flood level indicted by the Lough Funshinagh Technical Subgroup in their reports.	Pumping may restart once water level in the Lough increases and exceeds the 67.50 mOD minimum level.

Table 5.1 – Flow Monitoring Locations and Triggers to Stop Pumping

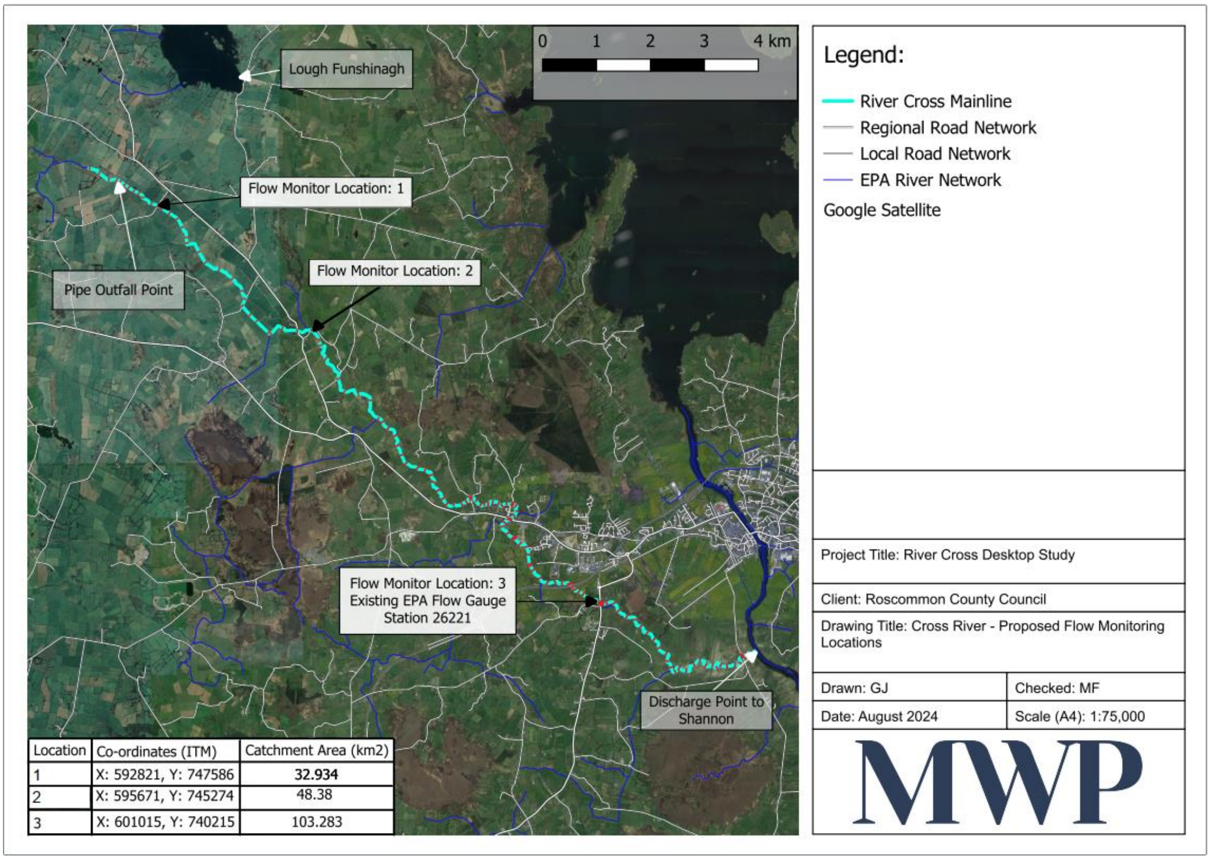


Figure 5.1 - Proposed Flow Monitoring Locations

6. Pumping System Operation and Decommissioning Phases

6.1 Operation and Maintenance

6.1.1 Description of Operation and Maintenance Activities

The following activities will be required during the operational phase of the pumping system:

- The pump pontoon will be inspected daily to ensure proper operation of the pumps and to check for any blockages or damage to the fish screens. If the fish screens become blocked they can be cleared by an operative who can safely reach the screens from the access walkways.
- The hydraulic pumps will each have a running time of about 100 hours on full fuel tanks; therefore, refuelling will be required every fourth day of pumping. This will involve a fuel tanker driving into the Intake Compound and delivering fuel to the tanks which are located within the bunds.
- Refuelling shall take place with the truck parked over a portable PVC containment bund mat. This is designed for use under vehicles and shall act as a containment system to catch any spills which may occur during refuelling. The mat is manufactured from 900 gsm PVC-coated hydrocarbon and shall be placed on top of a geotextile layer.
- Rainwater from the HPU Bund will need to be emptied daily by pumping the rainwater from a dry sump using a light duty puddle pump and discharging the water in a distributed manner onto the grassed surface at a location where the buffer distance is at least 15m to the lough edge.
- The HPU diesel engines will need to be serviced every 500 running hours, with the submersible pumps and hydraulics serviced every 2,500 hours, as per suppliers specifications. A typical service will consist of changing filters and oil. A spilt kit will be used to ensure that any spillage is contained.
- The entire pipeline route needs to be visually inspected every day by driving the route to identify any signed of damage or distress to the pipeline and to ensure all stock proof fencing remains intact.
- The Cross River Outfall will be inspected once a day. The purpose of the inspection will be to ensure that the diffuser is working properly and is not blocked, and to ensure the rock armour and geotextile have not become dislodged or unstable. The pumps will be shut down immediately in the unlikely event that there is a concern on the integrity of the outfall.
- The local drainage around the compound and the silt fence on the downslope side of the compound will be checked once a week to ensure adequate function and there are no signs of blockage.
- Remote monitoring of the flow in the pumps and in the Cross River will be carried out every day, as outlined in Sections 2, 3 and 5.

6.1.2 Specific Controls for Refuelling

As noted above, refuelling will take place over a portable bund mat. The following additional measures will be adopted during refuelling operations:

- Only designated trained and competent operatives will be authorised to carry out refuelling operations.
- Mobile measures such as the containment mat will be used during all refuelling operations.
- Tanks will only be filled from transportation tankers with automatic shut off overfill protection.
- The tanks shall not be left unattended during refuelling.

- Oil booms will be kept on site to deal with any accidental spillage.
- Strict procedures for tank and plant inspection, maintenance and repairs shall be detailed in the contractor's method statements and construction machinery shall be checked for leaks before arrival on site.
- The plant refuelling procedures described above shall be detailed in the contractor's method statements.

6.2 Decommissioning

6.2.1 Pump Intake System

Decommissioning of the pump intake system will involve the following;

I. The pumps will be shut down and disconnected from the pipeline and hydraulic hoses.
II. A mobile crane will be used to lift the Pump Pontoon (with the pumps inside) from the lough to an articulated truck parked in the intake compound. A small boat will be in the water to assist.
III. The floating access pontoon will be dismantled (unbolted) and lifted from the edge of the lough to a truck parked in the compound using the crane.
IV. The same crane will lift the HPUs and fuel tanks onto the truck.

6.2.2 Pipeline

Decommissioning of the pipeline will involve the following sequence;

I. The PE ribbed pipe will be dismantled and lifted onto a flatbed trailer using an excavator.
II. The lay-flat hose will be collected using a proprietary hose reel system and stored in a container mounted on a flatbed trailer which will be pulled by an excavator.
III. The stockproof fence will be removed by rolling up the wire and pulling the stakes from the ground.
IV. The section of pipeline corridor where the PE ribbed was located will be rotovated and tilled to reinstate it back to the existing land use.
V. A stockproof fence will be erected at all field boundary fences/hedgerows that were taken down and a native hedgerow will be planted on one site of the fence.
VI. Any stone walls that were removed will be reinstated by a suitably experienced stonemason.

6.2.3 Road Crossings

Decommissioning of the road crossings will involve the following sequence:

I. The HDPE carrier pipes will remain in place after the pipeline has been removed.
II. Each end of the pipe will be blocked up by filling in the trench at the ends. The redundant pipe beneath the road will not be of concern.
III. The existing hedgerow which was removed will be replanted using native hedge species.

6.2.4 Outfall to Cross River

Decommissioning of the outfall will involve the following sequence;

I. The PE ribbed pipe and diffuser tee will have been removed in conjunction with the remainder of the pipeline.
II. The rock armour and natural stone flags will be carefully removed from the surface of the geotextile using an excavator and placed into a tipper truck or tracked dumper.
III. The geotextile will be pulled across the river and removed by hand without entering the water.

6.2.5 Intake Compound

Decommissioning of the intake compound will involve the following sequence:

I. The stock proof fence and paladin fence will be taken up and loaded onto a flatbed truck for reuse.
II. The concrete HPU bund will be demolished using an excavator with a rock breaker and removed to a licensed facility.
III. The Class 6F stone as well as the geogrid / geotextile used to construct the compound will be taken up and brought to a licensed facility. The stone aggregate would be reused following confirmation of acceptability.
IV. The ground beneath the footprint of the compound will be rotovated and tilled to reinstate the area to agricultural usage, similar to the surrounding lands.

Appendix A
- Lough Funshinagh Technical Subgroup Reports

Modelling and Analysis of Changes to Lough Funshinagh Flood Levels

1st July 2024

Lough Funshinagh Technical Subgroup

Owen Naughton[a], Ted McCormack[b], Shane Regan[c], Paul Johnston[d]

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1. Introduction

The Lough Funshinagh Technical Subgroup was initiated by Roscommon County Council in April 2024 to examine the Lough Funshinagh flood regime in a hydrological and ecohydrological context. The group consists of turlough hydrogeology specialists from South East Technological University, Geological Survey Ireland, National Parks and Wildlife and Trinity College Dublin.

The Subgroup has previously issued a draft report to Roscommon County Council on 13th of June identifying a substantial shift in the hydrological operation at Lough Funshinagh post- the 2016 flood event towards higher flood levels. Based on this analysis, National Parks and Wildlife stated that the Lough is not functioning as one would expect or require to achieve the site-specific conservation objectives of the site. It was determined that, based on the continual standing water at the site, the qualifying interests are not in good condition and it will continue to deteriorate while the current pattern of flooding continues.

In this context, the Technical Subgroup continued its modelling work to quantify the impact of Lough Funshinagh's changed hydrological regime, and how that impact would be altered if an artificial drainage channel was implemented.

2. Methodology

The modelling work described in the first report ("Modelling and analysis of Lough Funshinagh Flood Levels", 13th June 2024) was continued and expanded. This involved firstly modelling the effects drainage would have on the turlough as it has operated post-2016. Model parameters were based on the current hydrological regime of the Lough (the 2020 calibration) with drainage installed at 65.8 mAOD. Drainage characteristics were provided by Malachy Walsh and Partners Consulting Engineers. Next, a preliminary recalibration of the model was carried out using only GSI hydrometric data from 2016 to simulate how Lough Funshinagh behaved prior to the change in flood behaviour which occurred post-2016. This gave three different modelled scenarios for comparison:

a) **Past Scenario:** Estimates how Lough Funshinagh would behave if the post-2016 change in flooding behaviour did not occur. This is taken as representative of the long-term hydrological regime of the Lough.
b) **Present Scenario**: estimates how Lough Funshinagh currently behaves (this was the model used in the previous report).
c) **Present-Altered Scenario:** estimates how Lough Funshinagh would behave if an artificial drainage channel was implemented.

Historic meteorological data was inputted into the three model scenarios to produced timeseries from 1951 to 2024, representing how Funshinagh would have behaved if a specific scenario was in place over this period. These scenarios were then compared in terms of peak floods, overall flood patterns and flood durations. The analysis focussed on three questions:

1. How has the change in flooding behaviour post-2016 altered the hydrological regime of the turlough? (i.e. a comparison between past and present scenarios).
2. How would artificial drainage impact the hydrological regime of Lough Funshinagh as it currently operates? (i.e. a comparison between present and present-altered scenarios)
3. What would be the hydrological regime of the present-day turlough with drainage compare to the turlough if the post-2016 change did not occur? (i.e. a comparison between the past and present-altered scenarios)

3. Results

3.1. Flood Levels

Timeseries of observed and simulated flooding for the three model scenarios between 2006 and 2024 are presented in Figure 1. This plot demonstrates how the Present Scenario consistently shows the highest flood levels. The Past Scenario produces peak levels approximately 1 m lower than 2018-2024 observed levels, while the Present-Altered scenario shows the greatest decrease, reducing levels between 1 and 2.2 m.

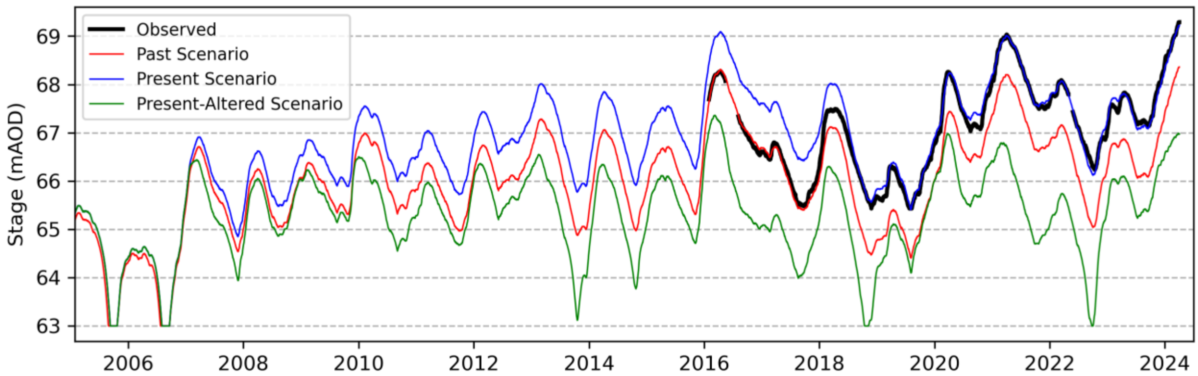


Figure 1: Observed and modelled water level timeseries in Lough Funshinagh between 2006 and 2024.

Changes in the long-term annual maxima series (largest flood in each year) between 1951 and 2024 are shown in Figure 2. which shows changes to annual maxima series based on the three main questions: 1) what is the impact of the change in flood regime post-2016 (top), 2) what would the impact of artificial drainage be (middle), and 3), what is the combined impact of the post-2016 and artificial drainage changes (bottom).

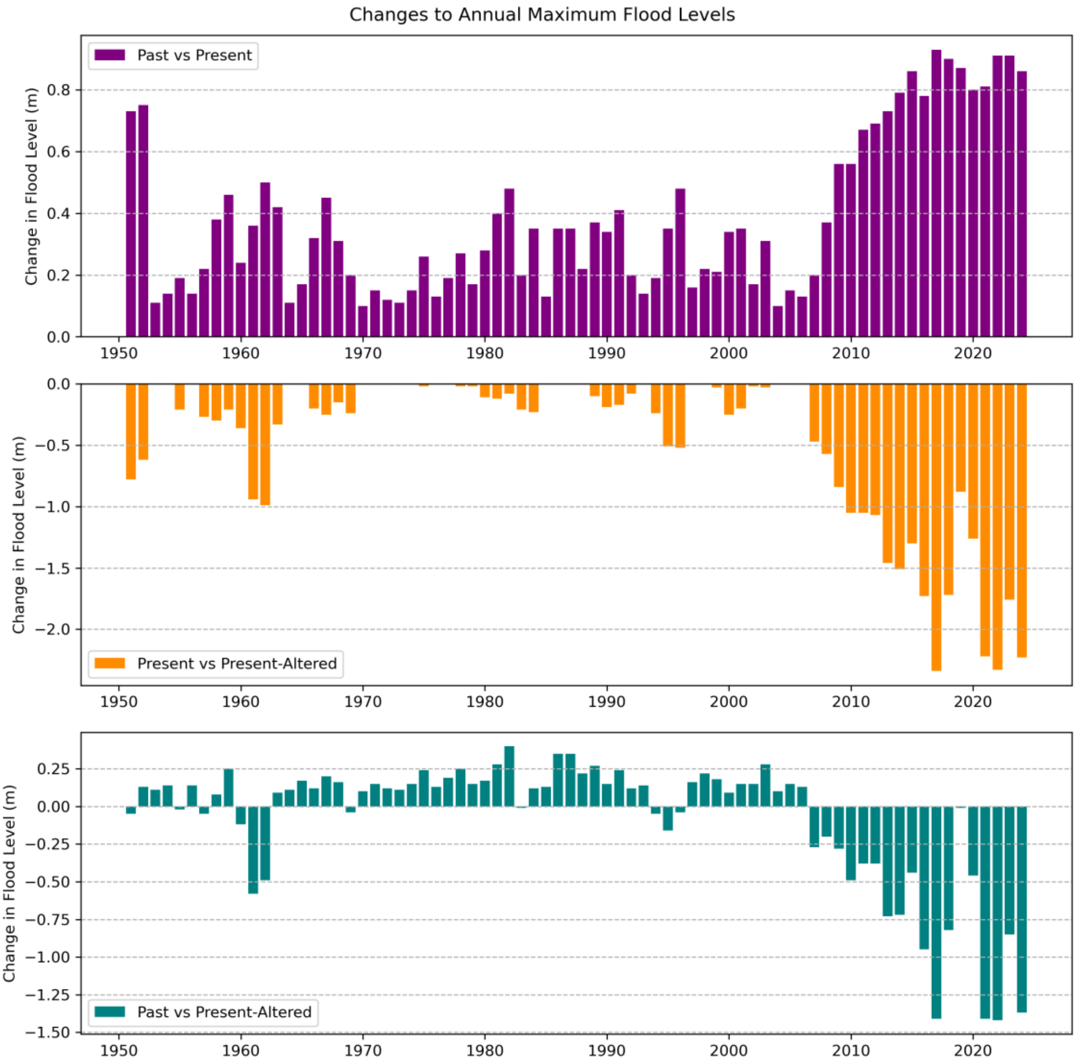


Figure 2 Changes to annual maxima series based on three comparisons: top: Past vs Present, middle: Present vs Present-altered and bottom: Past vs Past-altered

Impact of the change in flood regime post-2016 (Past vs Present)

- The present hydrological regime at Lough Funshinagh would have caused significantly higher flood levels in the past. Increases in annual maxima range from +0.1 m to +0.93 m with average change of +0.38 m.
- If the hydrological regime of Lough Funshinagh had not changed the peak flood level in 2024 would have been approximately 1 m lower than observed and close to the 2016 peak level. Conversely, if the change in flood behaviour had occurred pre-2016 then the 2016 peak would likely have been above 69 mAOD.

Impact of drainage on post-2016 flooding regime (Present vs Present-altered)

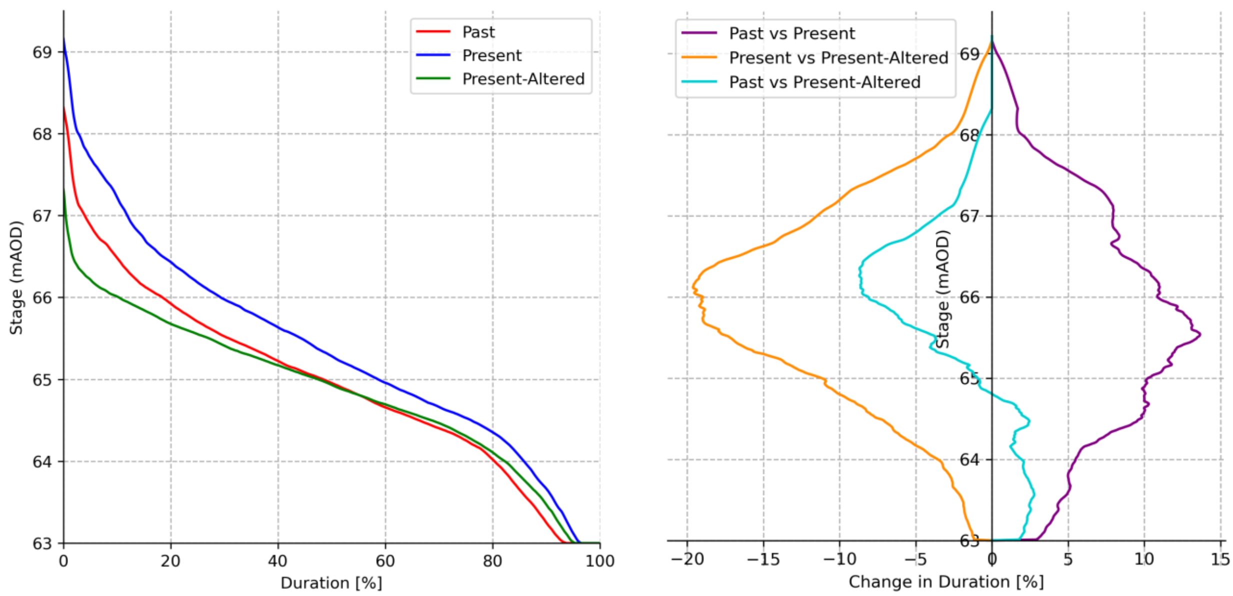
- The addition of artificial drainage would reduce annual maxima from between 2.34 m and 0.47 m, with average change of 0.53 m.
- The maximum flood level simulated to occur between 1951 and 2024 would be 67.4 mAOD. It should be noted that the model predicts that this peak flood level would have occurred in 2016 (the only year where it exceeds 67 mAOD), not 2024.

Comparison of post-2016 flooding regime with drainage to pre-2016 regime (Past vs Present-altered)

- Changes to annual maxima range from -1.4 m to +0.54 m with average change of -0.09 m. Overall, only approximately 40% of years have reduced flood levels.
- The combination of changes to the natural system and artificial drainage is mixed. While drainage reduces high flood levels, the slower natural drainage of the turlough results in higher flood levels in drier years.

3.2. Flood Durations

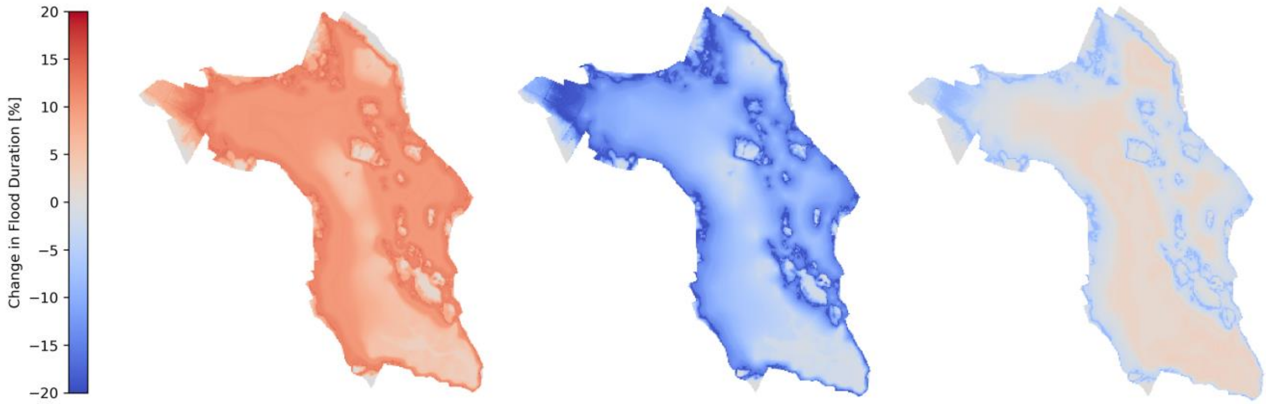
Flood duration curves were calculated for each scenario and are presented in Figure 3a. This figure shows that flood durations are greatest, across all flood levels, in the Present scenario. The Past and Present-altered scenarios have reduced flood durations, with Present-altered having lowest durations at high levels, and Past having lowest durations at low levels. The differences between the Flood Duration Curves are presented in Figure 3 which shows the changes in flood duration based on the three questions under study.



(A) **(B)**

Figure 3: A) Flood Duration Curves for Past, Present and Present-altered scenarios, B) Difference between Flood Duration Curves

The spatial coverage of changes to flood duration within the Lough Funshinagh SAC zone is presented as maps in Figure 4. The colour of these maps indicates the nature and severity of the change taking place. For example, red pixels indicate increased flood durations, and the redder the pixel, the greater the increase. Thes maps highlight: 1) the post-2016 change in flood regime has caused increased flood durations throughout Lough Funshinagh (Figure 4a), 2) the addition of a drainage channel will drop flood durations across the Lough, with greatest impacts around the edges of the SAC (Figure 4b), and 3) with a drainage channel implemented, the core of Lough Funshinagh will still experience greater flood durations than it did pre-2016 (Figure 4c).



A B C

Figure 4: Spatial coverage of altered flood durations. A) Past vs Present scenario, B) Present vs Present-altered scenario and C) Past vs Present-altered scenario.

Impact of the change in flood regime post-2016 (Past vs Present)

- Lough Funshinagh experiences higher flood durations at all flood levels under the present scenario. The greatest increase is at water levels of approx. 65.5 mAOD, whereby the duration of flooding has increased by 14%.
- Flood durations since 2016 have been significantly different to previous years. For example, between 1951 and 2015, 19.8 Ha of the Lough (4.6% of SAC area) was flooded 90% of the time. In comparison, since 2016, 365 Ha (85% of the SAC) turlough has been flooded 90% of the time.

Impact of drainage on post-2016 flooding regime (Present vs Present-altered)

- Artificial drainage would cause reduced flood durations at all flood levels. The greatest decrease is at water levels of approx. 66 mAOD, whereby the duration of flooding would reduce by almost 20%.

Comparison of post-2016 flooding regime with drainage to pre-2016 regime (Past vs Present-altered)

- Floods above 64.8 mAOD would occur less often (down to a minimum of 8% less often) whereas floods below 64.8 mAOD would occur more often (up to a maximum of 3% more often).
- In terms of area, 54% of the SAC will experience reduced flood durations, 44% of the area will experience increased flood durations, and 2% will be unchanged.

4. Conclusions

- Since 2016, the flood regime of Lough Funshinagh has changed, causing higher flood levels and durations. Based on these changes, the qualifying interests of the SAC are not in good condition, and the SAC will continue to deteriorate while the post-2016 current pattern of flooding continues.
- The implementation of a drainage channel at 65.8m AOD would reduce peak flood levels and shorten flood durations at all levels. This would diminish the impact of the 2016 flood regime change.
- In comparison to pre-2016 conditions, a post-2016 Lough Funshinagh with a drainage channel would experience: 1) a reduction in extreme flood levels, 2) a reduction in flood durations at high levels (impacting approx. 54% of the SAC), and 3) an increase in flood durations at low levels (impacting approx. 44% of the SAC).

Modelling and analysis of Lough Funshinagh Flood Levels

13th June 2024

Lough Funshinagh Technical Subgroup

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1. Introduction

The Lough Funshinagh Technical Subgroup was initiated by Roscommon County Council in April 2024 to examine the Lough Funshinagh flood regime in a hydrological and ecohydrological context. The group consists of turlough hydrogeology specialists from South East Technological University, Geological Survey Ireland, National Parks and Wildlife and Trinity College Dublin.

The current phase involves carrying out a modelling study of to examine the flooding behaviour at the turlough in response to prevailing rainfall conditions, recreating historic water levels, and assessing if any change has occurred to the flood regime.

2. Methodology

Turlough ecology is fundamentally driven by flooding regime, so the approach taken in this study is to examine the present and historic flooding regime at Lough Funshinagh using hydrological modelling. South East Technological University (SETU) has developed a modelling tool for simulating water levels at turloughs based on research previously funded by Geological Survey Ireland and National Parks and Wildlife Service (Campanya et al, 2023). The model used, UisceMod, is a Lumped Model (LM) where the turlough is represented as a single reservoir with distinct inflow and outflow mechanisms. The inflow is calculated by combining effective rainfall, a flood routing reservoir and a notional catchment area to produce an inflow time series. The outflow series is determined using a set of discharge equations based on the water depth within the turlough. The volume of water within the turlough is then computed at each time step using a water balance equation that accounts for the previous volume, inflow, outflow, together with direct rainfall and evaporation. Water level (stage) is then calculated using a stage-volume curve for the turlough developed from site topography. Calibration is performed by comparing model outputs with observed volumes and water levels and adjusting the parameters to minimize errors.

A model was calibrated for Lough Funshinagh using water level records for the period August 2016 to April 2024, obtained from the GSI Groundwater Data Viewer (gwlevel.ie). Daily rainfall, evapotranspiration and evaporation data were obtained from Met Eireann. Topography information was supplied by Geological Survey Ireland (based on OPW datasets). A three-year calibration period chosen from Jan 2019 to December 2021. Following calibration, the model successfully reproduced the flood regime at Lough Funshinagh (Figure 1). For validation, the calibrated model was run for the full period where water remainder of the available water level records (2016-2018 and 2022-2024) (see Section 3.1). Met Eireann historic rainfall from 1941 to present was also inputting into the model to predict the long-term flooding behaviour of the turlough (Section 3.2).

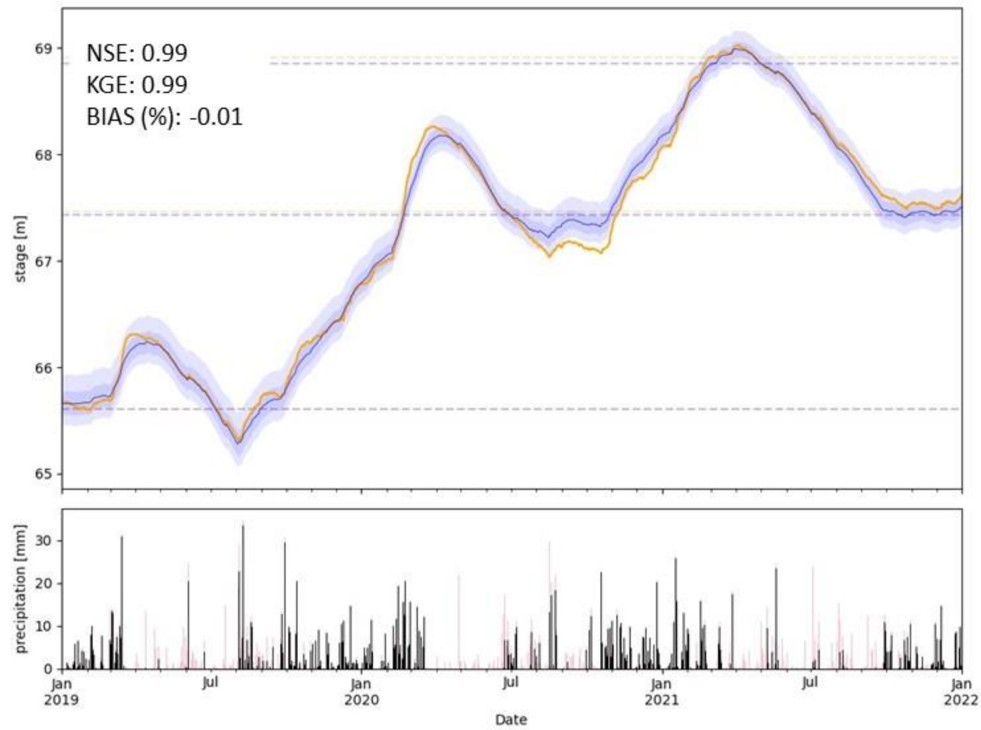

Figure 1: Calibration of Lough Funshinagh Model

3. Preliminary Results

3.1 Lough Funshinagh flooding regime: 2015-2024

The observed and modelled water level hydrographs for Lough Funshinagh for the period 2015 to 2024 are shown in Figure 2. For the period following calibration (2022-2024), the model accurately simulates both the recession in 2022 and the unprecedented peak levels reached in 2024. This would give confidence that the model well represents the current hydrological operation of Lough Funshinagh. However, the model significantly overpredicted the flood peak in 2016. The modelled peak level was 69.1 mAOD, compared to an estimated maximum of 68.25 mAOD (OPW, 2022). In terms of volume, these figures equate to peak volumes of 17 x 10^6 m^3 and 12.5 x 10^6 m^3 for levels of 69.1 and 68.25 mAOD respectively, and so the difference between modelled and observed peak levels represents an overestimate of 36% in terms of volume. Given the model performs well across a range of flood levels from 2019 to 2024 this would indicate there has been a substantial shift in the hydrological operation at Lough Funshinagh post- the 2016 flood event, with rainfall causing higher flood levels than previously would have occurred.

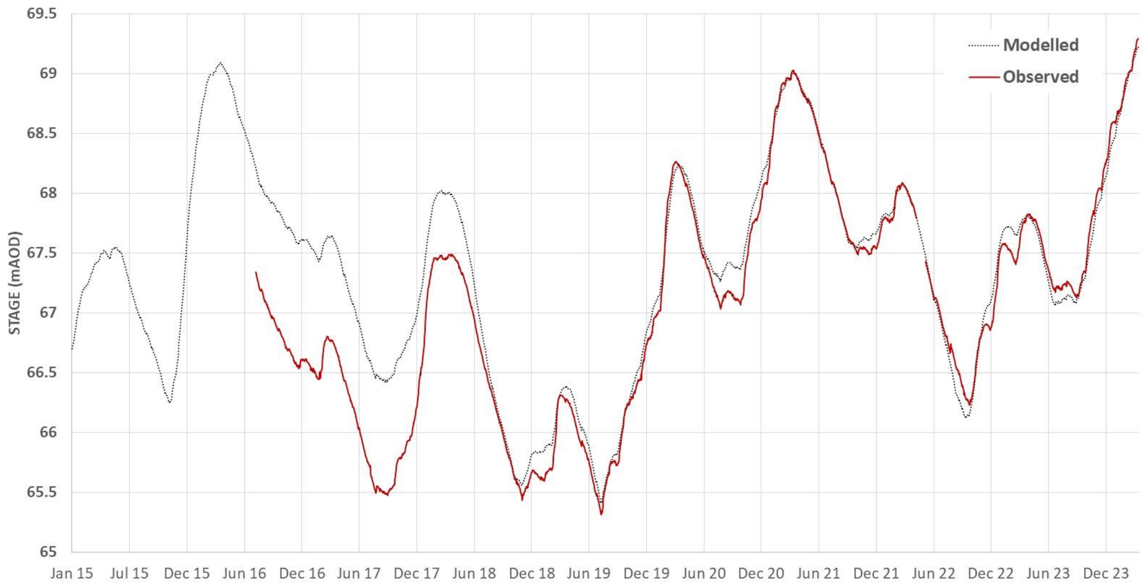

Figure 2: Modelled versus observed water levels for Lough Funshinagh, Co. Roscommon for the period 2015 – 2024.

3.2. Lough Funshinagh 1941-2024

The calibrated Lough Funshinagh model was used to reconstruct water levels between 1941 and present using historical rainfall data from Met Eireann (Figure 3). Before 2007 the peak water level in the series occurs in 1948, which is supported by anecdotal evidence of exceptionally high water levels in the Spring of that year (OPW, 2022). A shift towards higher flood levels can then be seen from 2007 onwards. This reflects the increase in long-term rainfall over this period, with records showing a 10% increase in average 5-year and 10-year cumulative rainfall post-2015 compared to the long-term average. In the post-2007 period, water levels that could pose a risk of flooding to the areas surrounding areas (taken as in excess of 67 mAOD), were predicted in 2010, 2012, 2013, 2014 and 2015 and more. Given the first recorded instance of significant flooding around Lough Funshinagh occurred in 2016, this would lend further evidence that there has been a significant shift in hydrological behaviour towards higher flood levels post-2016.

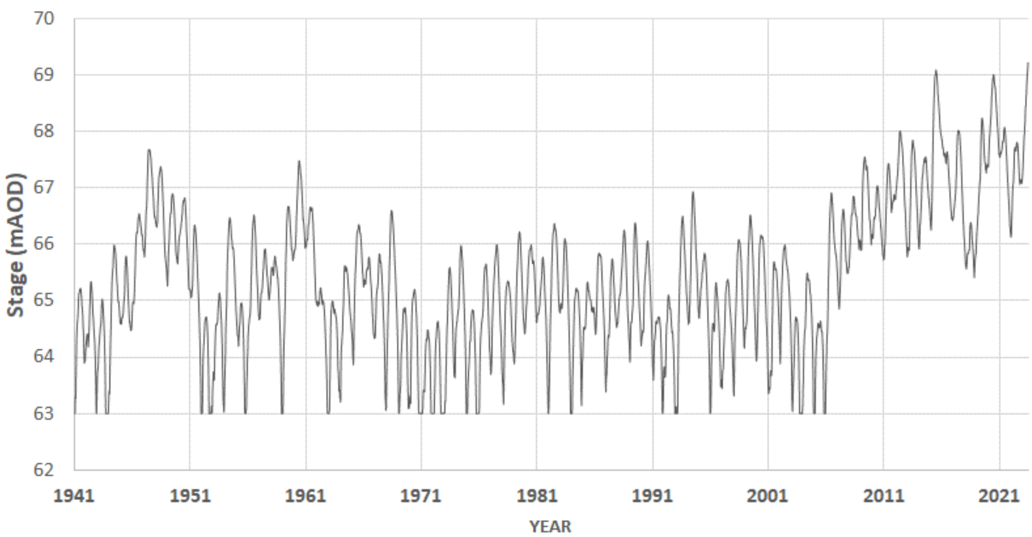

Figure 3: Long-term modelled water level hydrograph for Lough Funshinagh, Co. Roscommon. For the period 1941 to 2024.

To contextualise current flood levels in terms of flood duration, annual duration curves were calculated for each hydrological year (1st October to 30th September) from 1941 to present using modelled data (Figure 4). In Figure 4, the central black line shows the median flood duration, while the red shaded bands around the median indicate percentiles (5%iles). Duration curves for the four most recent hydrological years are also shown. In all four, flooding persisted across the site for significantly longer than historic norms.

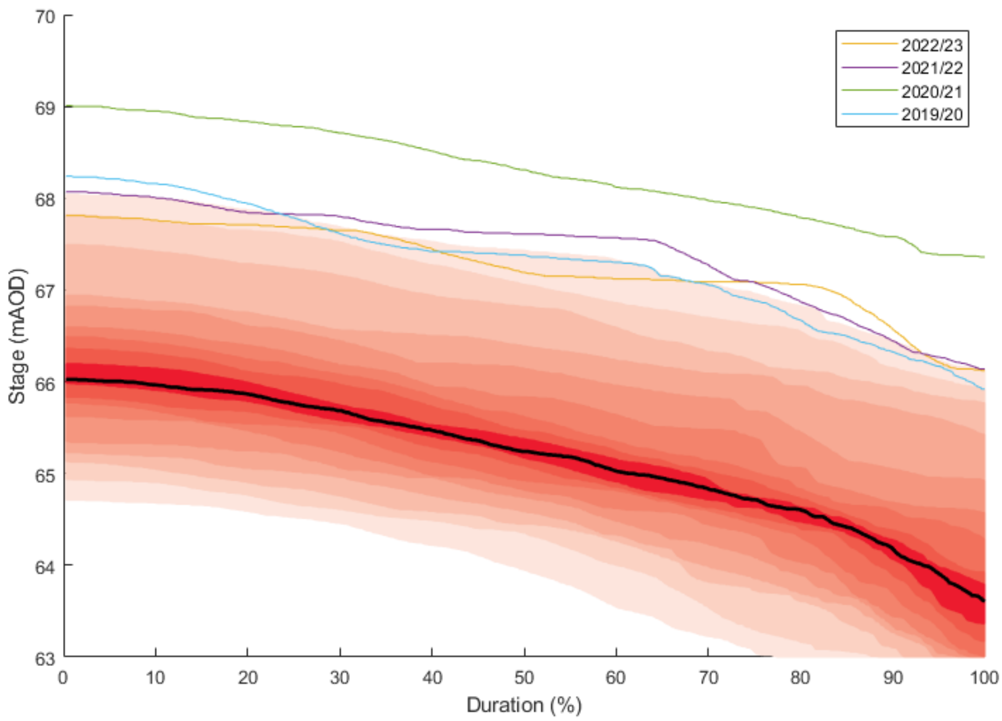

Figure 4: Duration curves for Lough Funshinagh, Co. Roscommon.

4. Preliminary Conclusions

- There has been a shift towards higher flood levels from 2007 until present, in response to rainfall levels that are consistently significantly higher over this period than long-term averages.
- The Lough Funshinagh hydrological model reliably reproduces flood levels within the turlough for the period 2018 to present. However, it significantly overpredicts the 2016 flood peak.
- Given the model performs well across a range of flood levels, this indicates there has been a substantial shift in the hydrological operation at Lough Funshinagh post- the 2016 flood event towards higher flood levels.

5. Future work

The Lough Funshinagh Technical Subgroup intends to examine the pre-2018 flood regime of Lough Funshinagh. A hydrological model will be calibrated based on 2016-2018 behaviour in order to simulate how the Lough would function today, and historically, if it still operated as it did in 2016.

References

Campanyà, J., McCormack, T., Gill, L.W., Johnston, P.M., Licciardi, A. and Naughton, O., 2023. UISCEmod: Open-source software for modelling water level time series in ephemeral karstic wetlands. *Environmental Modelling & Software*, *167*, p.105761.

Office of Public Works, 2022, Lough Funshinagh State of Knowledge: Draft Report.