N4 Collooney to Castlebaldwin, *Proposed Road Development*

APPENDIX NO. 4.2

Flood Risk Assessment

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EXECUTIVE SUMMARY

Sligo County Council Road Design Department (Sligo RDD) requested Hydro Environmental Ltd. to undertake a flood risk assessment for the N4 -Collooney to Castlebaldwin *Proposed Road Development*. The proposal involves circa 70 watercourse crossings including the construction of 61 minor watercourse culverts (contributing areas of 3 to 25ha) and 9 main watercourse structures / culverts of 5 rivers/streams. The watercourses crossed by the road development drain to the Unshin River, which itself is a large tributary of the Ballysadare River. The road development lies mostly in a karstic limestone drainage area (regionally important aquifer) with a section of poorly productive (Lisgorman Shale) bedrock to the north. Numerous karst drainage features are located close to the existing and proposed road alignments.

The main watercourses that are crossed by the Proposed Road Development are Markree Demesne Stream (Toberscanavan Loughs outflow) (1 No. crossing), Turnalaydan Stream (Lough Corran outflow) (1 No. crossing), Drumfin River (1 No. crossing), Springfield Stream (Loughymeenaghan outflow) (2 No. crossings), Lissycoyne Stream (2 No. crossings) and Tributary of the Drumderry Stream (2 No. crossings). The three largest watercourse crossings are the Markree Demesne Stream, the Turnalaydan Stream and the Drumfin River having catchment areas of 6.2km^2, 16.1 km^2 and 21.5km^2 respectively. The remaining crossings Springfield, Lissycoyne and Tributary of the Drumderry Streams have small catchments areas ranging from 1.24 to 1.6km^2. The seasonal mean average annual rainfall (SAAR) for the study area is approximately 1230mm/annum and the winter rainfall acceptance potential (WRAP) classification for the study area soils is generally classified as moderate runoff catchments (SOIL Type 3) with Greenfield flood runoff rates of c. 6.5l/s per ha.

Many of the watercourses to be crossed were subject to OPW Arterial Drainage works in the past as part of the Owenmore Drainage Scheme. Large extents of the lands adjoining and crossed by the *Proposed Road Development* are designated as benefitting lands of the OPW drainage scheme. These lands would, in general, be expected to be low lying and prone to flooding and thus benefitting from the arterial drainage works.

The flood risk assessment involved site walkovers and assessments, hydrometric and topographic survey, hydrological analysis to estimate the design flood flows (annual maximum flood, the 1 in 100year and the 1 in 1000 year events), development and application of hydraulic models of the main watercourse crossings to simulate the existing and proposed road scenarios, flood risk assessment and the production of flood risk maps in the vicinity of the road for the modelled watercourses.

All watercourses within the study area are ungauged and consequently the design flood flow hydrographs are estimated as currently recommended by Flood Study Report (FSR) catchment characteristic flood estimation methods. Standard Factorial Errors have been applied to flood flow estimations to yield the upper 66% confidence interval of the estimate and where deemed applicable are used to ensure that simulated flow hydrographs are suitably conservative in the assessment.

<u>Markree Demesne (Toberscanavan Loughs outflow) Stream Crossing</u>

The Markree Demesne Stream (Toberscanavan Loughs outflow) crossing involves the replacement of the existing 1200mm diameter pipe culvert under the N4 with a box culvert. The Toberscanavan Lakes and associated flood plain area are located 225m upstream of the existing N4. In general, the water level in lakes is controlled by the existing culvert and the study undertaken includes an impact assessment of the replacement culvert on both flood and normal / low flow conditions and water levels.

The preconstruction hydraulic model simulations for Markree Demesne Stream determined that under design flood flows the 1200mm culvert is inlet controlled and acts as the choke on higher flood flow rates discharging downstream. The model simulations demonstrate that the choke effect of the existing 1200mm culvert combined with the upstream Toberscanavan lakes provide significant flood storage and attenuation reducing significantly the downstream peak flow rates.

The impact on upstream lake levels under non flood conditions for the proposed culvert reduces the upstream lake level by 0.10m and 0.14m for annual average and 95-percentile low flows respectively and reduces the Q100 and Q1000 upstream lake flood levels by 0.10m and 0.13m respectively. The resultant decrease in upstream storage leads to an increase in the flood flow peaks downstream of the N4 with a resultant rise in peak flood levels of between 0.08m and 0.17m. These increases can be mitigated by improving the downstream channel conveyance (i.e. channel maintenance involving clearance of dense vegetation) and upsizing downstream land access culverts. The impact on the existing low flow and mean lake levels at

Toberscanavan can be mitigated by the construction of a low weir upstream of the proposed culvert to mimic the existing culvert's inlet control. The proposed weir will be fish passable.

The proposed storm drainage attenuation ponds encroaches into the 1000 year flood plain but has negligible impact on flood risk in the area. The proposed road level is 2m higher than the calculated Q1000 design flood level and therefore has a low flood risk.

Turnalaydan Stream (Lough Corran outflow) crossing

The Turnalaydan Stream (Lough Corran outflow) crossing involves the diversion of the river channel and the construction of a 20m clear span bridge. Properties adjacent to the existing N4-twin masonry arch bridge at Lackagh, located 530m downstream, are subject to recurring flood risk and are identified as having a high to moderate flood risk with at least one property immediately upstream on the right bank flooding in the recent past. The lands upstream of Lackagh Bridge form part of a large flood plain which includes two lakes namely Lough Corran and Boathole Lough. The proposed road embankment crosses through approximately 300m of the river's floodplain at a location 520m upstream of Lackagh Bridge and 340m downstream of Lough Corran. The hydraulic model simulations for Turnalaydan Stream for the pre-construction scenario determined that the existing flood plain attenuates the peak Q100+FE design flood flow from 14.0 cumec to 13.0 cumec across the study reach. It is also proposed to construct a storm drainage attenuation pond adjacent to the proposed road alignment upstream of Lackagh Bridge. The proposed pond encroaches very slightly (<10m^2) on the Q1000+FE flood plain and found to have negligible impact of flood levels.

The hydraulic model runs for the post construction scenario determined that constriction and reduction in flow conveyance due to the proposed bridge and road embankment will cause a minor afflux of 0.02m and 0.04m for the Q100+FE and Q1000+FE design flood events. The predicted impact downstream is neutral with no discernible change in flood flows and resultant flood levels. On the floodplain it is recommended that 3 no. of large diameter pipe culverts are installed at crossing of existing large open drains with matching invert levels in order to main the existing local drainage regime and connectivity and reducing the impact of the road embankment in the flood plain.

The proposed road level at the crossing is 4.7m above the 1 in 1000year flood level. The proposed road embankment encroaches on 12,900m^2 of the Q1000+FE flood plain with an associated estimated loss of storage of 4,900m^3 which is calculated to be equivalent to less than 4.5 minutes of the peak flood flow and thus has no perceptible impact on flow rate or flood level downstream. The construction of toe drains along the road embankment in the flood plain will partially negate this loss of storage. No other compensatory storage measures are proposed.

Drumfin River crossing

The Drumfin River crossing involves the construction of a 20m clear span bridge. An existing N4- three arch bridge at Behy is located 420m downstream of the proposed structure. The lands upstream of Behy Bridge form part of a large flood plain which is crossed by a network of open drains. The proposed road embankment crosses through approximately 610m of this floodplain. The hydraulic model simulations for Drumfin River for the pre-construction scenario determined that the existing flood plain attenuates the peak Q100+FE design flood flow from 20.3 cumec to 16.5 cumec across the study reach. It is also proposed to construct a storm drainage attenuation pond adjacent to the proposed road alignment upstream of the proposed bridge crossing. The proposed pond partially encroaches into Flood Zone B but will have a negligible impact on flood risk in the area.

The hydraulic model simulations for Drumfin River the post construction scenario determined that the proposed structure and embankment will cause a maximum upstream afflux of 0.14m at the peak of a 1 in 1000 year flood event and that this resulting water level rise will increase storage in the floodplain for a distance of approximately 1.4km upstream of the river crossing. The afflux is found to be due to contraction and reduction of flow conveyance across the flood plain. Increasing the span of the bridge crossing is shown to have minimal impact on flood levels. The resultant increase in upstream storage attenuates flows downstream of the floodplain proposed crossing by approximately 0.5 cumec and therefore marginally reduces flood risk downstream. It is recommended that 3 no. of large diameter pipe culverts are installed at crossing of existing large open drains with matching inverts in order to main the existing drainage regime and connectivity and reducing the impact of the road embankment in the flood plain. These culverts were shown to reduce the 1 in 1000year afflux to 0.09m.

The proposed road level at the crossing is 3.2m above the 1 in 1000year flood level. The proposed road embankment encroaches on 23,000m^2 of the Q1000+FE flood plain with an associated estimated loss of storage of 14400m^3 which is calculated to be equivalent to less than 9 minutes of the peak flood flow. The construction of toe drains along the road embankment in the flood plain will partially negate this loss of storage. No other compensatory storage is proposed.

The existing Carrownagark Spring Well pump house which is located close to the banks of the Drumfin River upstream of proposed crossing, which has a finished floor level of 51.67mOD, has been shown to be located in Flood Zone B. The post-construction Q1000+FE flood level is 0.09m lower than the surveyed finished floor level at the pump house.

Springfield Stream (Loughymeenaghan outflow) Crossing

The Springfield Stream (Loughymeenaghan outflow) crossing involves the diversion of the existing stream channel and the construction of two box culverts in sequence to accommodate the construction of the main proposed N4 road embankment and a land access road. The proposed crossings are located 340m downstream of the existing N4 Bridge at Ardloy in a minor and narrow flood plain area. An existing 900mm diameter land access culvert, located 280m downstream of the proposed crossings was shown to be overtopped during design flood events; however, the associated afflux does not extend upstream to the proposed road. Springfield Stream drains to a turlough/ swallow hole complex at Tawnagh located approximately 0.5km downstream of the proposed road.

The proposed road embankment at the Springfield Stream crossing passes through an area with little overbank flooding and conveyance and therefore has negligible impact on flood plain storage. The model confirms (by way of comparison between the upstream and downstream flows) that the road will have negligible impact on flood storage. It is proposed to construct a storm drainage attenuation pond adjacent to the proposed road alignment downstream of the proposed bridge crossing. The proposed pond does not encroach into the Flood Zone A or B.

The proposed road level at the crossing is 11m above the 1 in 1000year flood level. The proposed road embankment encroaches on 2,300m^2 of the Q1000 flood plain with an associated estimated loss of storage of 360m^3 which is calculated to be equivalent to 3.6 minutes of the peak flood flow. While no compensatory storage is proposed, the construction of toe drains will help to mitigate the storage loss.

Lissycoyne Stream Crossing

The Lissycoyne Stream crossing involves the diversion of the stream channel and the construction of two box culverts in sequence to accommodate the construction of a land access and the main proposed N4 road embankment. The proposed crossings are located immediately downstream of a local road known as Bog Road in an area designated as benefitting lands. The river models demonstrated that the local road would be overtopped during flood events.

Post construction model runs determined that the proposed structures cause a small afflux of 2cm for the Q1000 design flood which extends to Bog Road but does not impact flood levels further upstream. The replacement of the local road culvert would reduce flood storage upstream and therefore would cause an increase in peak flows downstream which would be expected to cause negligible increase in flood risk. The proposed storm drainage attenuation pond is located well out of the Lissycoyne Stream flood plain. It is recommended that a 900mm diameter culvert with matching inverts is constructed at the existing open drain at Ch12+000 to maintain existing flood plain conveyance.

The proposed road level at the crossing is 3.8m above the 1 in 1000year flood level. The proposed road embankment encroaches on 4400m^2 of the Q1000 flood plain with an associated estimated loss of storage of 616m^3 which is calculated to be equivalent to be less than 5 minute peak flood flow. While no compensatory storage is proposed, the construction of toe drains will help mitigate the storage loss.

Tributary of the Drumderry Stream Crossing

The Tributary of the Drumderry Stream crossing involves the replacement of an existing N4-culvert and the construction of a land access road culvert. The lands downstream of the N4 are designated mas benefitting lands and are drained by a network of open drains. The hydraulic models for the existing scenario confirmed that the existing 900mm pipe culvert and upstream channel are undersized and the existing N4 road would be expected to be overtopped during a 1 in 1000year flood event.

The proposed replacement culvert will reduce upstream flood risk upstream of the N4 considerably. There will be a minor increase in flood risk between the replacement N4 culvert and the proposed downstream culvert due to encroachment of the local access road embankment into the flood plain. The proposed road level at the crossing is 2.2m above the 1 in 1000year flood level.

Minor Culvert Crossings

The proposed minor culvert crossings all have been assessed using hydraulic calculations based on Manning's equation. Following the assessment it is concluded that all pipe sizes proposed are suitably sized to convey the design flood flows with minimal afflux.

TABLE OF CONTENTS

1 Introduction

1.1 Brief

Sligo County Council NRA Road Design Department (RDD) has requested Hydro-Environmental Ltd. to undertake a Flood Risk Assessment (FRA) for the proposed N4-Collooney to Castlebaldwin Realignment in Co. Sligo.

1.2 Scope of Study

The *Proposed Road Development* involves circa 63 No. watercourse crossings with catchments ranging from 21.5 km^2 to less than 1 hectare. Sligo RDD has developed the design for the road proposal including designs for bridges, culverts and stream diversions.

The flood risk assessment is required to:

- identify the existing flood risk to lands and properties with the study area;
- assess if there will any increase in flood risk to the lands and properties upstream or downstream of the Proposed Road Development due to the construction of watercourse crossings and construction in flood plains,
- propose mitigations, if required, to mitigate any increase in flood risk
- propose mitigations, if possible, to alleviate any existing flood risk
- produce flood risk mapping for the Proposed Road Development in accordance with the zoning set out in The Planning System and Flood Risk Management Guidelines for Planning Authorities (November 2009) (OPW and DEHLG).

Sligo RDD has provided topographical data for the study area including detail survey levels for the watercourse crossing channel and existing structures and Lidar (Light Detection and Ranging) and DTM (Digital terrain modelling) data for the adjoining lands and flood plain areas.

2 Flood Risk Management Planning Guidelines

2.1 Background

In September 2008 the OPW and DoEHLG jointly published for public consultation new draft Planning Guidelines on the Planning System and Flood Risk Management which are aimed at ensuring a more consistent, rigorous and systematic approach to fully incorporate flood risk assessment and management into the planning system. These Guidelines after consultation were finalised and published in November 2009.

The document sets out how to assess and manage flood risk potential and includes guidance on the preparation of flood risk assessments by developers.

The recommended stages of assessment are:

Screening Assessment – to identify whether there may be flooding or surface water management issues related to a plan area or proposed development site that may warrant further investigation;

Scoping assessment to confirm sources of flooding that may affect a plan area or proposed development site, to appraise the adequacy of existing information and to scope the extent of the risk of flooding and potential impact of a development on flooding elsewhere and of the scope of possible mitigation measures

Appropriate risk assessment: to assess flood risk issues in sufficient detail and to provide a quantitative appraisal of potential flood risk to a proposed or existing development, of its potential impact on flood risk elsewhere and of the effectiveness of any proposed mitigation measures.

2.2 Site Specific Flood Risk Assessment

Mapping:

- A location map
- A Plan that shows existing site and proposed development(s)
- Identification of any structures which may influence the hydraulics.
- Flood Inundation map showing flood zone areas on the subject site / area

Surveys:

- Site levels related to Ordnance Datum
- Appropriate cross-section(s) showing finished etc. Or other relevant levels in respect to flooding.

Assessments:

- Consideration of flood zone in which the site falls and demonstration that development meets the vulnerability criteria set out in the Guidance
- Flood alleviation measures already in place
- Information about potential sources of flooding that may affect the site;
- The impact of flooding on a site.

Design Standards

The FRA should generally be undertaken on the basis of a design event of the appropriate design standard:-

- 100 year Fluvial Flood or 1% Annual Exceedance Probability (AEP) for River Flow
- 200 year combined Return Period event or 0.5% AEP for tide affected sites
- 1000 year or 1% Annual Exceedance Probability (AEP) for River Flow (fluvial) or tidal flood Event

2.3 Decision Making Process

Management of flood hazard and potential risks in the planning system is based on

- Sequential Approach
- Justification Test

2.3.1 Sequential Approach

The aim of the sequential approach is to guide development away from areas at risk from flooding. The approach makes use of flood risk zones, ignoring presence of flood protection structures, and classifications of vulnerability of property to flooding.

Table 1 Flood Risk Zone Definitions

ZONE	DEFINITION
Zone A High Probability Highest risk of flooding	More than 1% probability of river flooding and more than 0.5% probability of tidal flooding. Development should be avoided and/or only considered through application of Justification test. Only water compatible development , such as docks and marinas, dockside activities that require a waterside location, amenity open space, outdoor sports and recreation and essential transport infrastructure that cannot be located elsewhere would be considered appropriate for this zone (i.e. not requiring application of Justification test).
Zone B Moderate Probability	Between 1 and 0.1% probability of river flooding or between 0.5 and 0.1% probability of coast flooding. Development should only be considered in this zone if adequate land or sites are not available in Zone C or if development in this zone would pass the Justification Test.
Zone C Low Probability	Less than 0.1% probability of river or coastal flooding. Development in this zone is appropriate from a flooding perspective.

2.3.2 Justification Test

Further sequentially-based decision making should be applied when undertaking the Justification Test for development that needs to be in flood risk areas for reasons of proper planning and sustainable development:

1. within Zone or site, development should be directed to areas of lower flood probability;
2. where impact of the development on adjacent lands is considered unacceptable the justification of the proposal or Zone should be reviewed
3. where the impacts are acceptable or manageable, appropriate mitigation measures within the site and if necessary elsewhere should be considered.

2.3.3 Application of the Justification Test in Development management.

Where a planning authority is considering proposals for new development in areas at a high or moderate risk of flooding that include types of development that are vulnerable to flooding and that would generally be inappropriate, the planning authority must be satisfied that the development satisfies all of the criteria of the Justification Test as it applies to development management outlined in Box 5.1

Box 5.1 Justification Test for development management (to be submitted by the applicant)

When considering proposals for development, which may be vulnerable to flooding, and that would generally be inappropriate as set out in Table 3.2, the following criteria must be satisfied:

1. The subject lands have been zoned or otherwise designated for the particular use or form of development in an operative development plan, which has been adopted or varied taking account of these Guidelines.
2. The proposal has been subject to an appropriate flood risk assessment that demonstrates:
   (i) The development proposed will not increase flood risk elsewhere and, if practicable, will reduce overall flood risk;
   (ii) The development proposal includes measures to minimise flood risk to people, property, the economy and the environment as far as reasonably possible;
   (iii) The development proposed includes measures to ensure that residual risks to the area and/or development can be managed to an acceptable level as regards the adequacy of existing flood protection measures or the design, implementation and funding of any future flood risk management measures and provisions for emergency services access; and
   (iv) The development proposed addresses the above in a manner that is also compatible with the achievement of wider planning objectives in relation to development of good urban design and vibrant and active streetscapes.

The acceptability or otherwise of levels of residual risk should be made with consideration of the type and foreseen use of the development and the local development context.

Note: See section 5.27 in relation to major development on zoned lands where sequential approach has not been applied in the operative development plan.

Refer to section 5.28 in relation to minor and infill developments.

Assessment of major proposals for development in areas of flood risk pending implementation of these Guidelines

5.27 From a flood risk management perspective, proposals fitting into this category should be considered as though the land was not zoned for development. In such situations the applicant should be required, in consultation with the planning authority, to prepare an appropriate SFRA and to meet the criteria for the Justification Test as it applies to development plan preparation. The planning authority must then assess the proposal against the Justification Test as it applies to the development management process. Where the information is not sufficient to fully assess the issues involved, the development should not be approved on the basis of flood risk and / or on the grounds of prematurity prior to addressing flood risk as part of the normal review of the development plan for the area.

Assessment of minor proposals in areas of flood risk

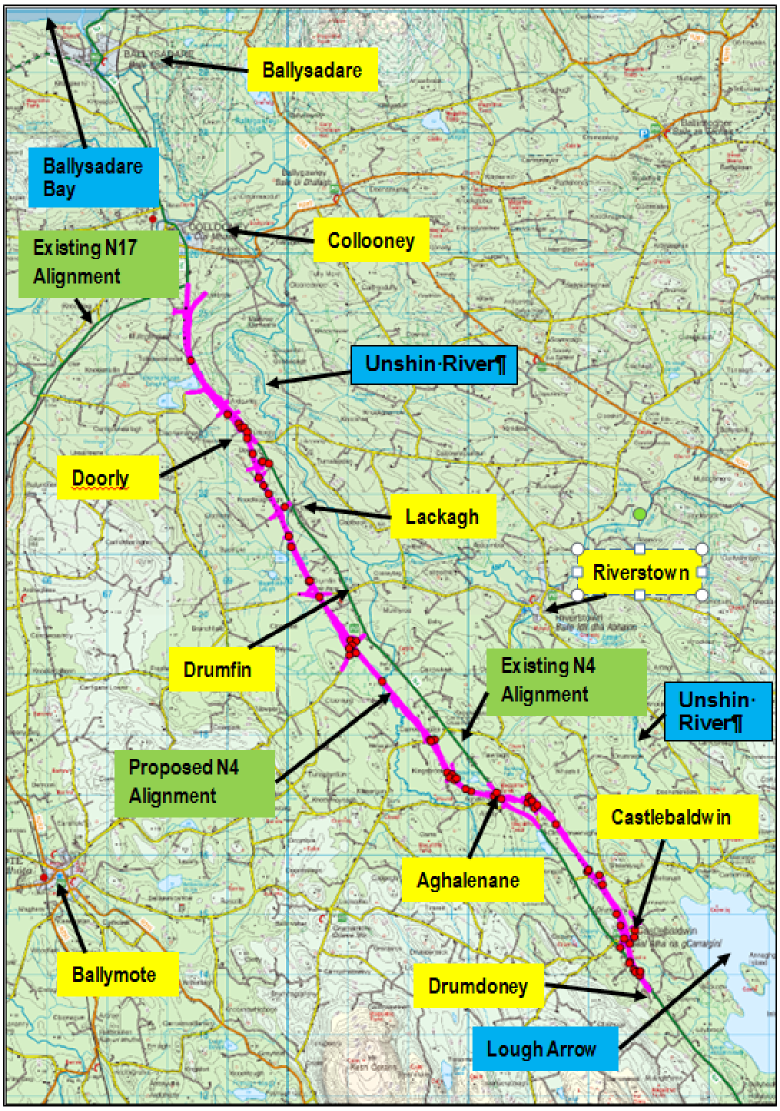
5.28 Applications for minor development, such as small extensions to houses, and most changes of use of existing buildings and or extensions and additions to existing commercial and industrial enterprises, are unlikely to raise significant flooding issues, unless they obstruct important flow paths, introduce a significant additional number of people into flood risk areas or entail the storage of hazardous substances. Since such applications concern existing buildings, the sequential approach cannot be used to locate them in lower-risk areas and the Justification Test will not apply. However, a commensurate assessment of the risks of flooding should accompany such applications to demonstrate that they would not have adverse impacts or impede access to a watercourse, floodplain or flood protection and management facilities. These proposals should follow best practice in the management of health and safety for users and residents of the proposal.

3 EXISTING ENVIRONMENT

3.1 Road Development Description

The main line length of the Proposed Road Development is approximately 14.71 km long and, while involving sections of realignment of the existing N4, comprises mostly involves off line works. The proposal extends from the junction between the N4 and N17 (approximately 0.5km south of Collooney) and follows the existing N4 alignment in a southerly direction for 2.85km. The road then diverges to the west (at Chainage 2+750) and runs parallel to the existing road alignment (by-passing Lackagh and Drumfin) for 7.5km before crossing the existing N4 at Aghalenane (Chainage 10+230). The road alignment then runs to the east of the existing N4 (by-passing Castlebaldwin) for 3.8km and re-joins the existing N4 alignment (Ch 14+000) circa 0.5km south east of Castlebaldwin. The proposal follows the existing alignment for a further 0.5km and finishes at Cloghoge Lower.

Figure 1 *Proposed N4 alignment*



The main-line finished road levels range between 32mOD and 74mOD. The road proposal includes a number of at grade and grade separated junctions, road diversions and local access roads. The road alignment passes through a rural setting of primarily agricultural and pasture lands with small areas of woodland scrub and coniferous forest.

3.2 Rainfall and Geology

The seasonal average annual rainfall (**SAAR**) for the study area (Met Éireann 1981 to 2010 - 1km grid database) is 1233mm/ annum (minimum 1200 and maximum 1348mm/annum). The average 5 year return period 1 hour and 2 day storm duration total rainfall depth (M5-1 hour and M5-2day) for the study area are 16mm and 60.53mm respectively. Table 2 presents the study area's average rainfall depths for the 100year, 150year and 200year return period for 1, 2, 6 and 12 hour and 1, 2, 4 and 25 day storm durations:

Table 2 Average Rainfall Return Period Storm Duration Rainfall depths

	1 hr	2 hr	6 hr	12 hr	1 d	2 d	4 d	25 d
100yr	34.9	43.0	60.1	74.3	92.5	104.0	123.7	258.1
150 yr	38.5	47.4	65.8	80.8	100.2	111.5	131.2	266.2
200 yr	41.3	50.6	69.9	85.7	106.0	117.0	136.7	272.1

The road alignment is mostly underlain by Sandstone and Shale Tills (TNSSs) and cutover peat subsoils with some areas of alluvium close to watercourses and lakes on predominantly limestone bedrock (Oakport, and Bricklieve formations) with an area of Shale (Lisgorman Shale) near Toberscanavan Lough).

Soil is an index of how the soil may accept infiltration and is a measure of the Winter Rainfall Acceptance Potential (WRAP). It can be determined from FSR mappings at 1 : 625,000 scale for Ireland. The SOIL index is based on only five classifications (very high, high, moderate, low and very low WRAP) and the mapping scale and number of categories are regarded as providing a very coarse measure of catchment runoff potential. The WRAP for the study area is predominantly moderate rainfall runoff (soil type 3) with a section of Low rainfall runoff to the northern end (soil type 2), refer to Figure 2.

3.3 Hydrogeology and Hydrology

3.3.1 Hydrogeology

The scheme lies mostly in a karstic drainage area (Ballygawley and Ballymote Groundwater Bodies) and also crosses a section of poorly productive bedrock (Lavagh and Ballintougher GWB) at Toberscanavan Lough. Numerous karst drainage features are located close to the existing and proposed N4 alignment. Table 3 summarises those identified on GSI-groundwater mapping:

Table 3 Karst Drainage Features

Name	Type	Chainage	Location
Toberbride	Spring	0+030	100m east
Toberscanavan	Spring	1+070	50m east
Tawnagh Swallow Hole/ Turlough complex	Swallow hole	10+560	360m north
Castlebaldwin south springs (Toberbride, Tobermuire and Toberpatrick)	Springs	13+850	<150m west

A number of other karst features (springs and swallow holes) are identified on OSI historic mapping including an additional five springs between Castlebaldwin and the end of the proposal, at least one each at Drumderry, Aghalenane and Carrownagark close to the proposed alignment and two springs and swallow hole at Drumfin. The stream outfalling from Loughymeenaghan drains to a large swallow-hole feature at Tawnagh.

The groundwater vulnerability classification for the majority of the proposed road development study area is low. The proposed alignment between Kingsbrook and Aghalenane, however, passes through an area of high to extreme vulnerability with areas of karst bedrock at the surface (GSI mapping).

Figure 2 *Soils WRAP Mapping*

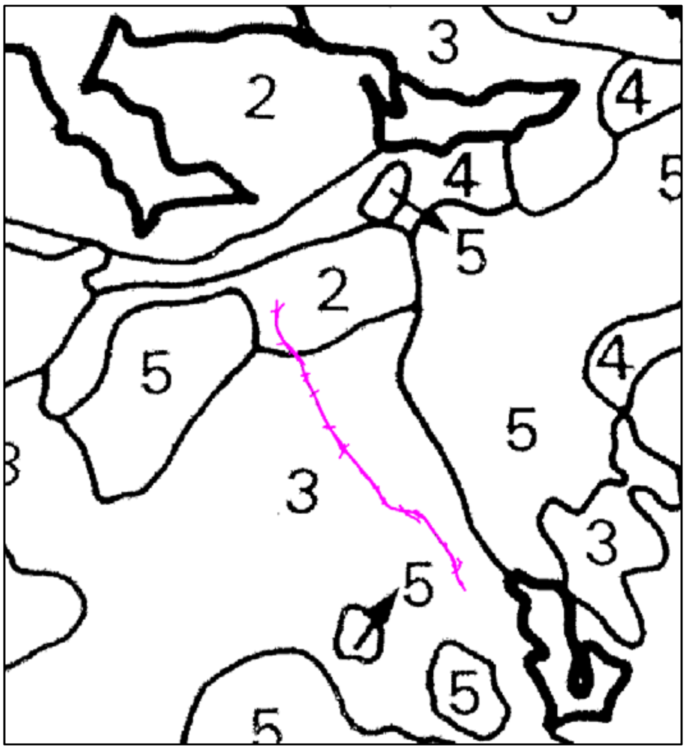
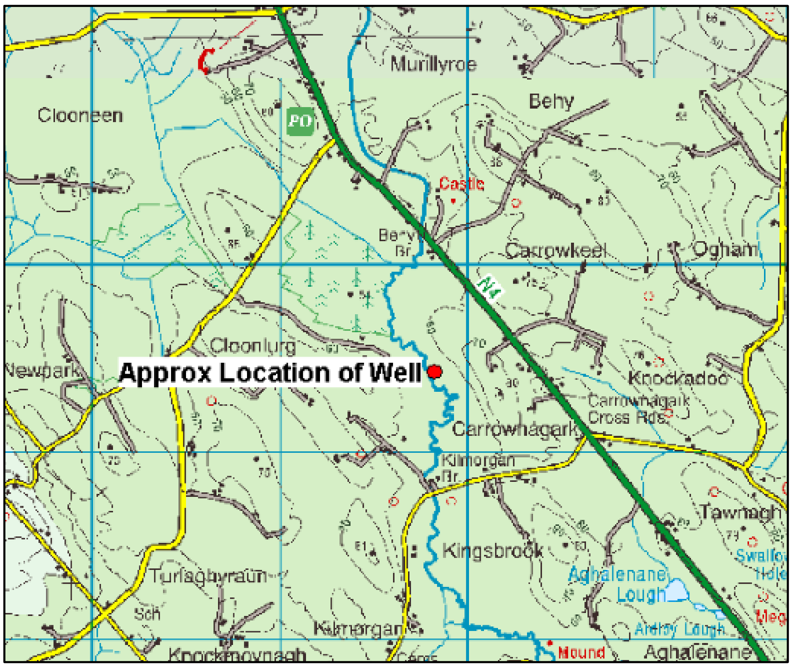
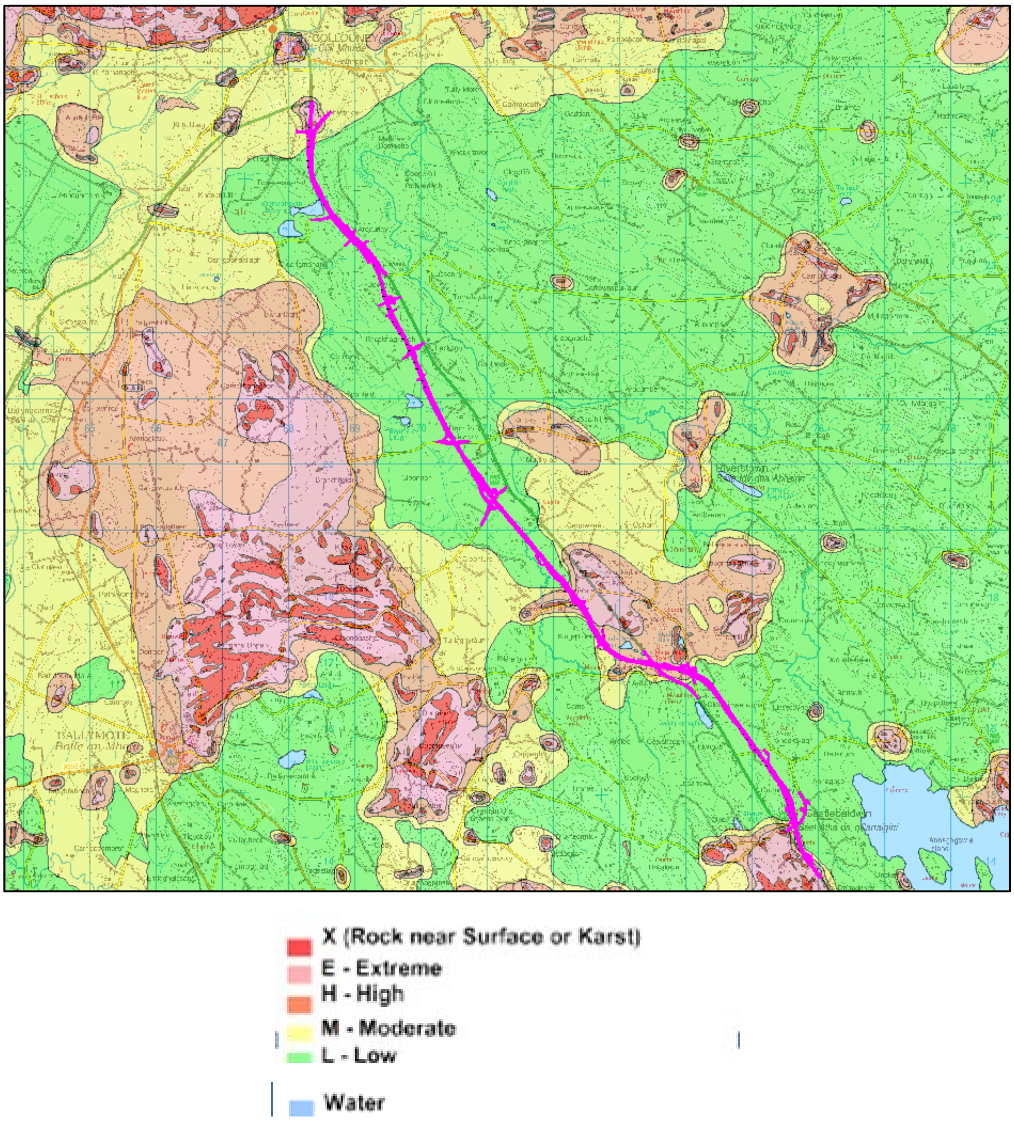


Figure 3.1 *Carrownagark Spring Well*



A spring well at Carrownagark which is used to supply a local group scheme is located 16m east of Drumfin River approximately 1.2km upstream of the existing N4-crossing at Behy Bridge. The spring location is shown in Figure 3.1 above.

Figure 3.2 Groundwater vulnerability



3.3.2 Hydrology

The road proposal is located in the Western River Basin and the Ballysdare River drainage catchment. The watercourses crossed by the proposal, in general, drain to the Unshin River. The lands adjacent to the road proposal south of Ch 13+300 at Castlebaldwin drain to Lough Arrow which in turn is drained by the Unshin River. The Unshin River joins the Owenmore River north of Collooney to form the Ballysdare River which outfalls to Ballysdare Bay at Ballysdare. The Unshin River is located to the east of the *Proposed Road Development*.

Table 4 presents the location and parameters (based on OSI 50k raster mapping) for the lakes located within the study area:

Table 4 Loughs within the study area

Name	Chainage	Recorded Water Level, mOD	Surface Area, km^2	Location
Toberscanavan Loughs	1+340	30	0.124	Immediate adjacent existing N4
Lough Corran	4+790	40	0.034	150m west of new road
Boathole Lough	4+980	40	0.016	425m west of new road
Aghalenane	9+610	-	0.014	115m north of new road
Loughymeenaghan	11+250	63	0.044	275m west of new road but immediately adjacent existing N4.
Cleavry Lough	13+100	-	0.015	840m west of road
Lough Arrow	14+500	53	12.44	1.0km east of the existing N4

70 No. watercourse crossings have been identified. The majority of these crossings are small watercourses, open drains and drainage channels. Nine watercourse crossings (six watercourses) of significant size will be crossed by the proposed road namely Markree Demesne Stream, Turnalaydan Stream, Drumfin River, Springfield Stream, Lissycoyne Stream and Tributary of the Drumderry Stream. The location, drainage catchment area, the size of existing structures immediately upstream or downstream of these larger watercourse crossings are summarised in the table below.

Table 5 Main Watercourse Crossings

Culvert Ref	Watercourse	Proposed Road level, mOD	Chainage	Catchment Area, km^2	Existing structure
Cul - 0	Markree Demesne Stream	32.08	1+090	6.16	1200mm diameter pipe culvert
Cul - 14	Turnalaydan Stream	47.10	4+480	16.11	Lackagh bridge. Twin masonry arch bridge 1.9m wide and 2.7m high
CUL - 25	Drumfin River	55.39	7+360	21.46	Behy Bridge. Three masonry arch bridge (2 No. c2.3m x c2.6m high and 1 No. 2.24m x 1.87m high)
CUL -39	Springfield stream	73.02	10+700	1.50	Ardloy Bridge located 300m upstream of the new road
CUL - 40			10+680	1.60	
CUL - 46	Lissycoyne Stream	67.56	12+230	1.54	0.625m wide by 1m high culvert under Bog Road located 30m upstream of the new road
CUL - 47		67.56			
CUL - 61	Tributary of the Drumderry Stream	65.24	14+200	1.23	32m long 900mm diameter culvert under the N4.
CUL - 62		65.24	14+160	1.25	

Many of the watercourses to be crossed were subject to OPW Arterial Drainage works as part of the Owenmore Drainage Scheme. Following the scheme benefitting land maps (definition given below) were drawn up for the rivers and their associated floodplain which include the lands crossed by the proposed road development. Benefitting Lands i.e. areas that have benefitted from OPW land drainage schemes, and typically indicate low-lying land near rivers and streams that might be expected to be prone from flooding. (www.floodmaps.ie.)

Figures 4 and 5 show the catchment areas of the larger watercourses and Figure 6 to 9 show the location of all the proposed crossings and the arterial drainage benefitting lands.

Figure 4 River Drainage Catchments Cul-0 and Cul-14

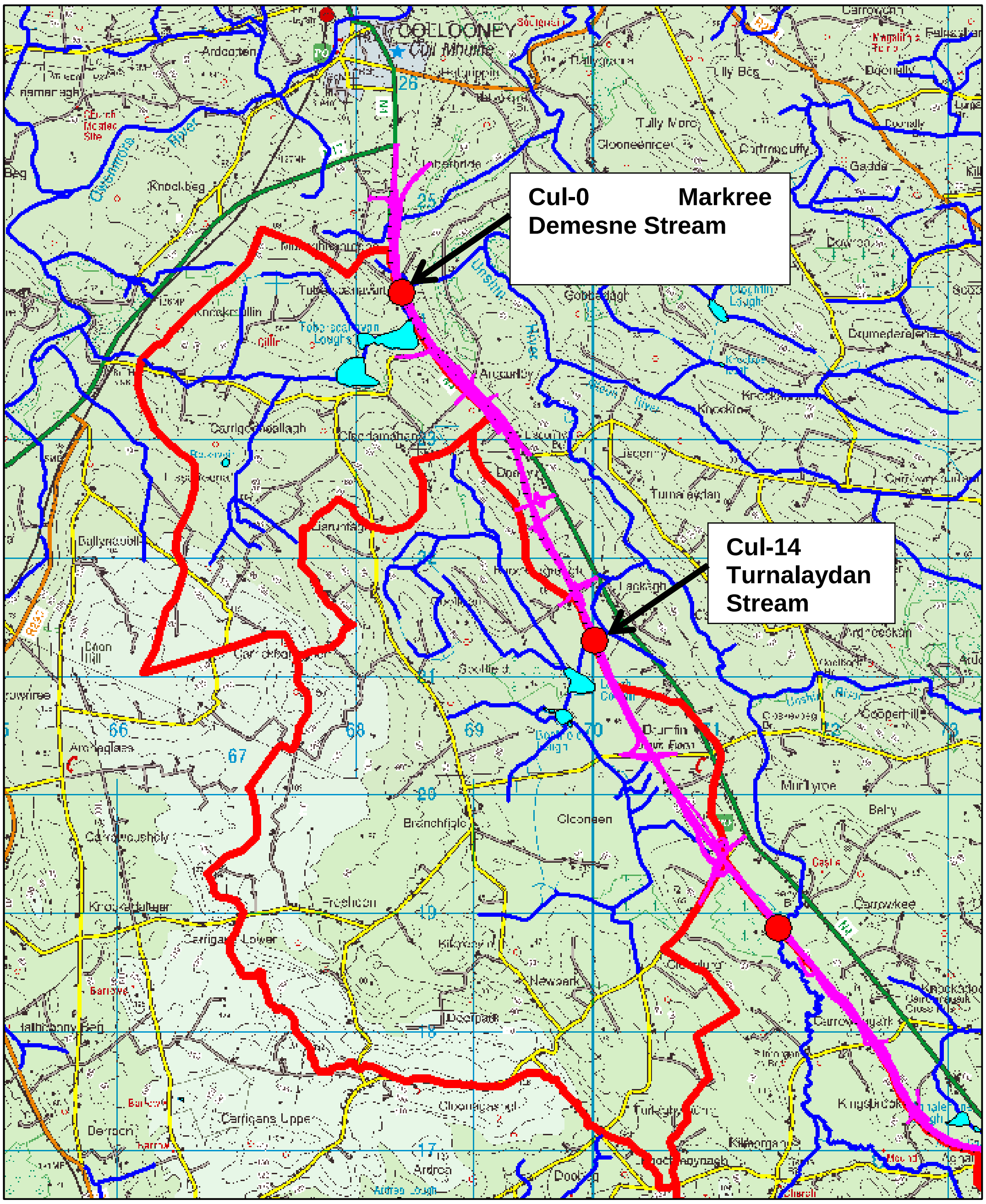


Figure 5 River Drainage Catchments Cul-25, Cul-39 & 40, Cul-46&47 and Cul-61&62

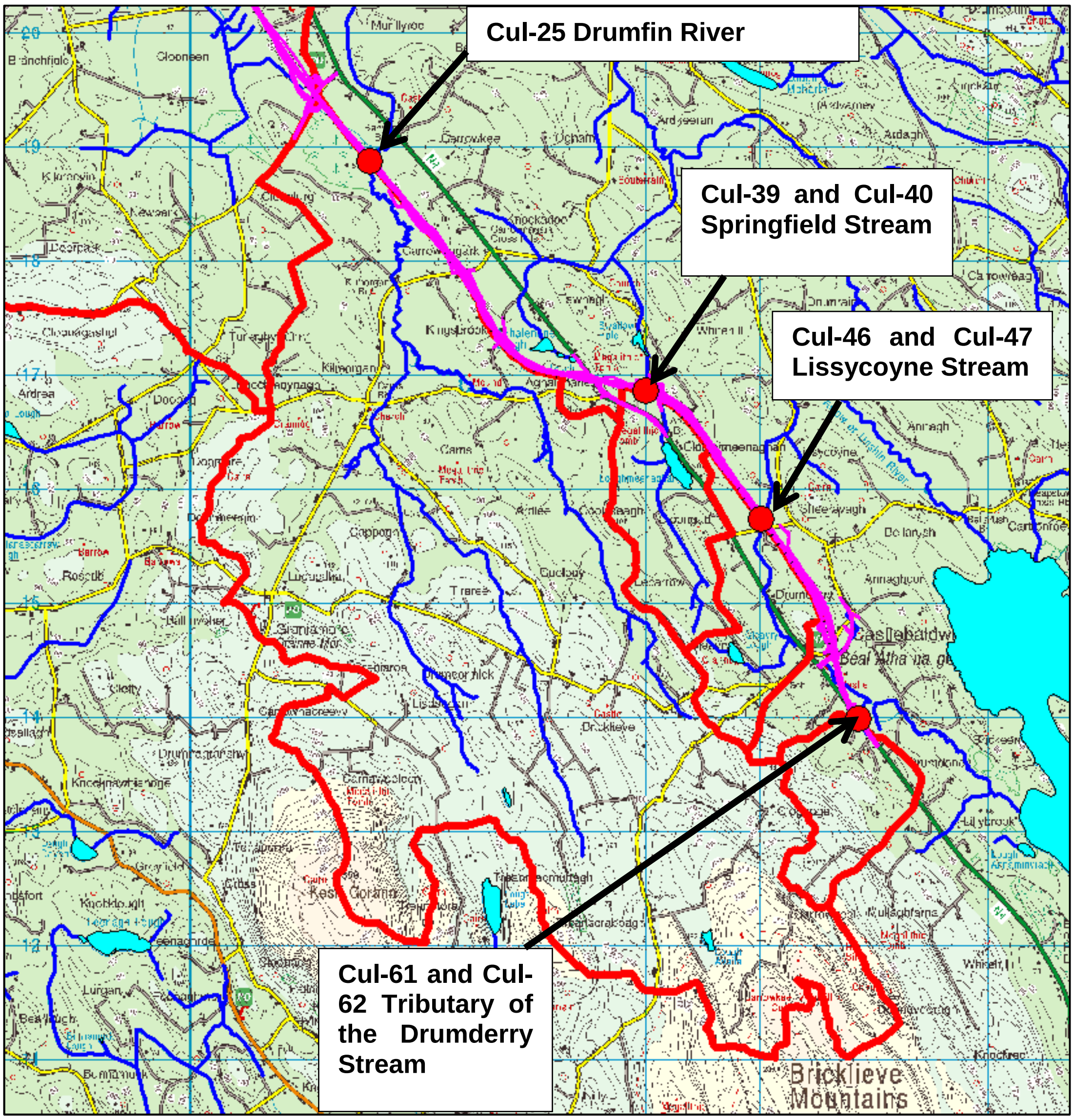


Figure 6.1 Watercourse Crossings and Arterial Drainage Benefitting Lands

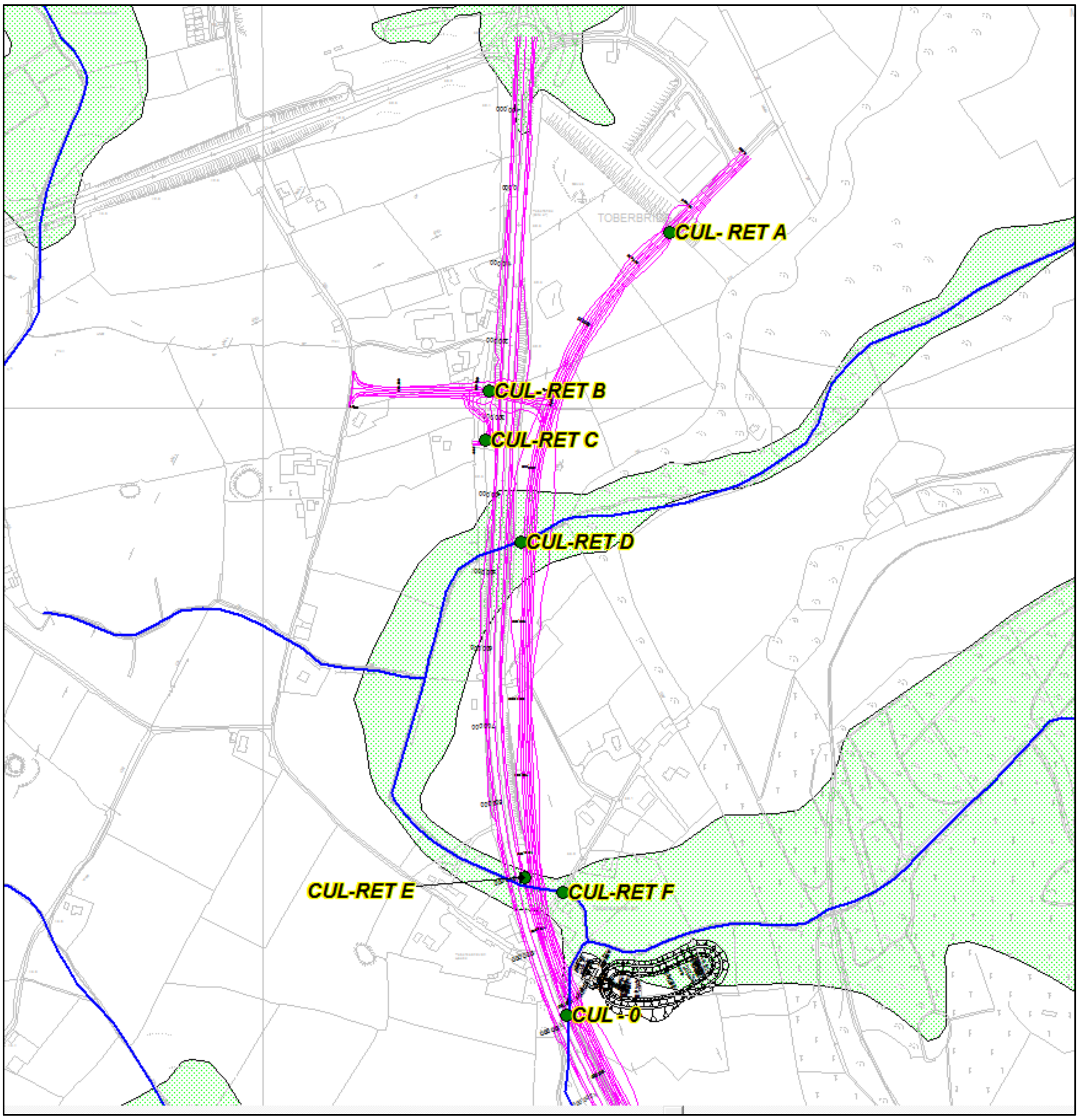
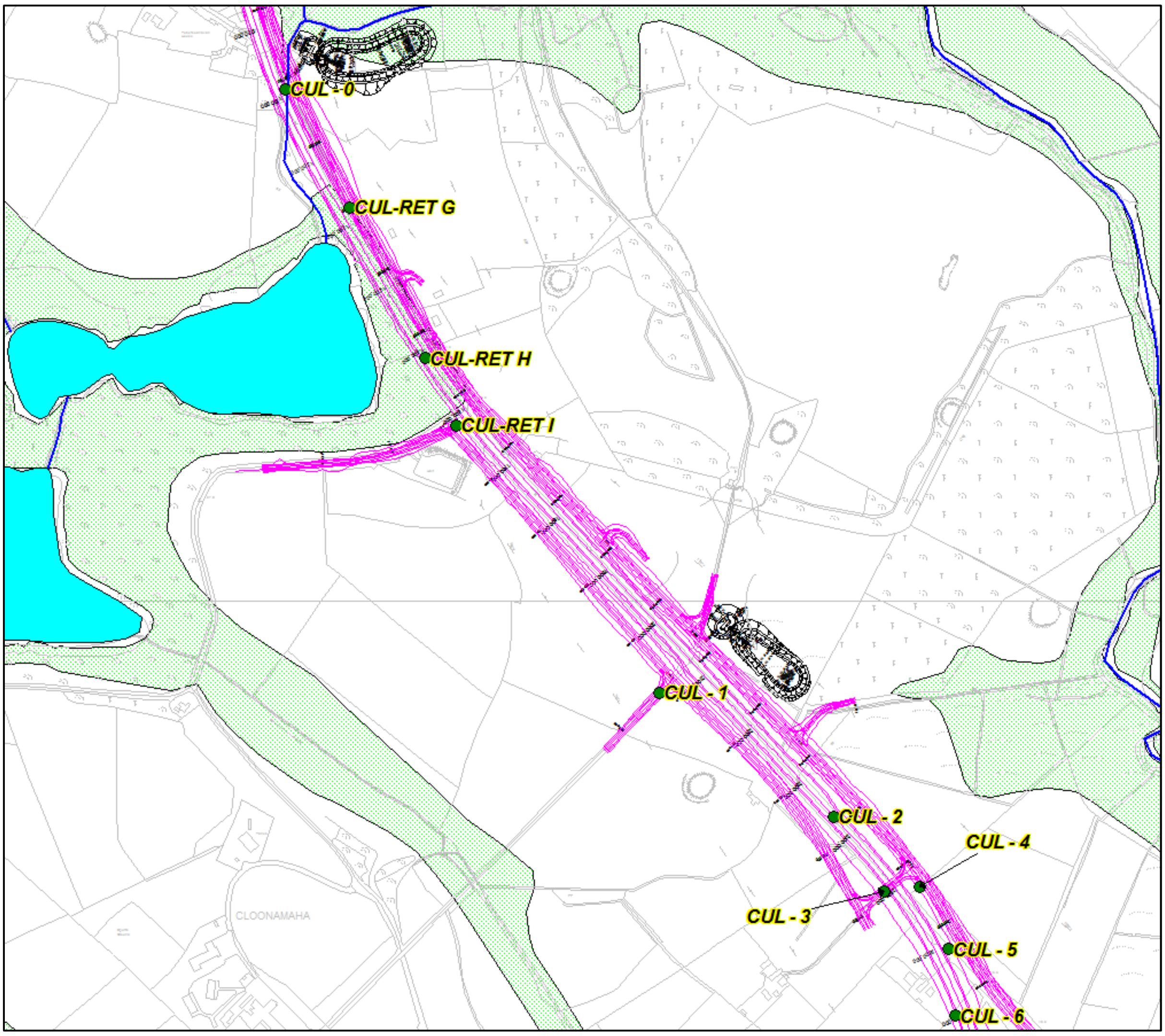


Figure 6.2 Watercourse Crossings and Arterial Drainage Benefitting Lands



Figures 7.1 Watercourse Crossings and Arterial Drainage Benefitting Lands

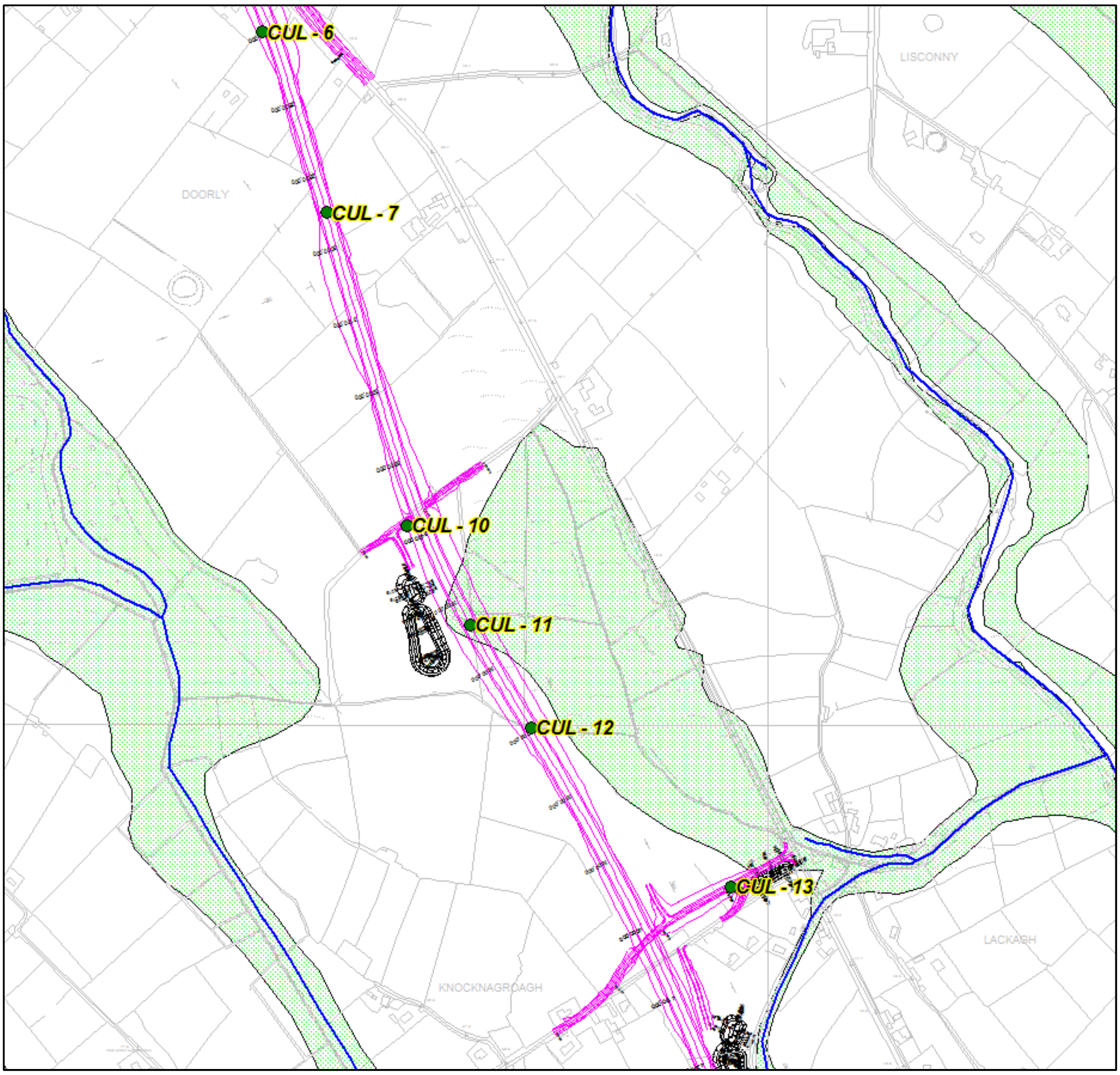


Figure 7.2 Watercourse Crossings and Arterial Drainage Benefitting Lands

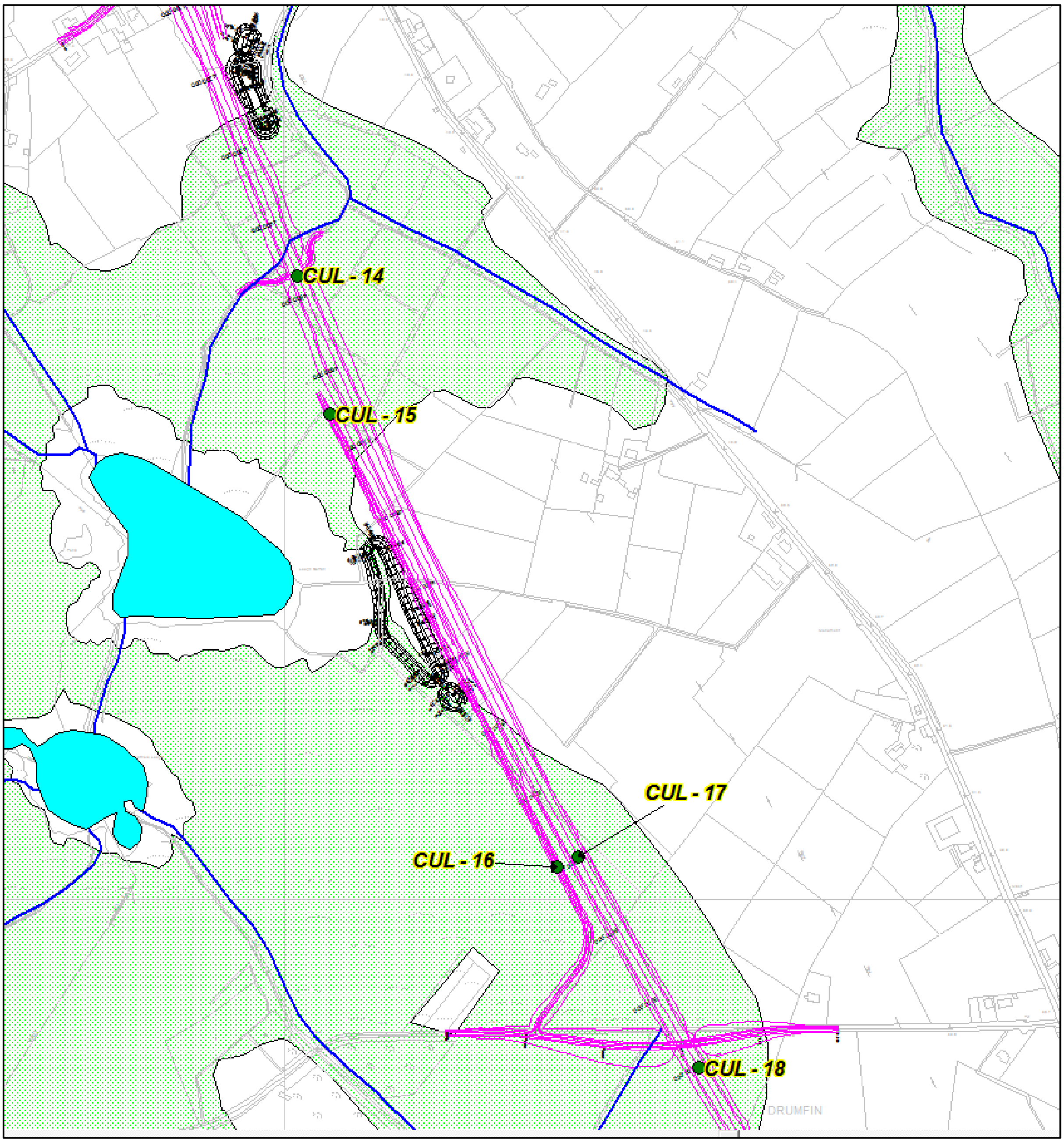


Figure 8.1 Watercourse Crossings and Arterial Drainage Benefitting Lands

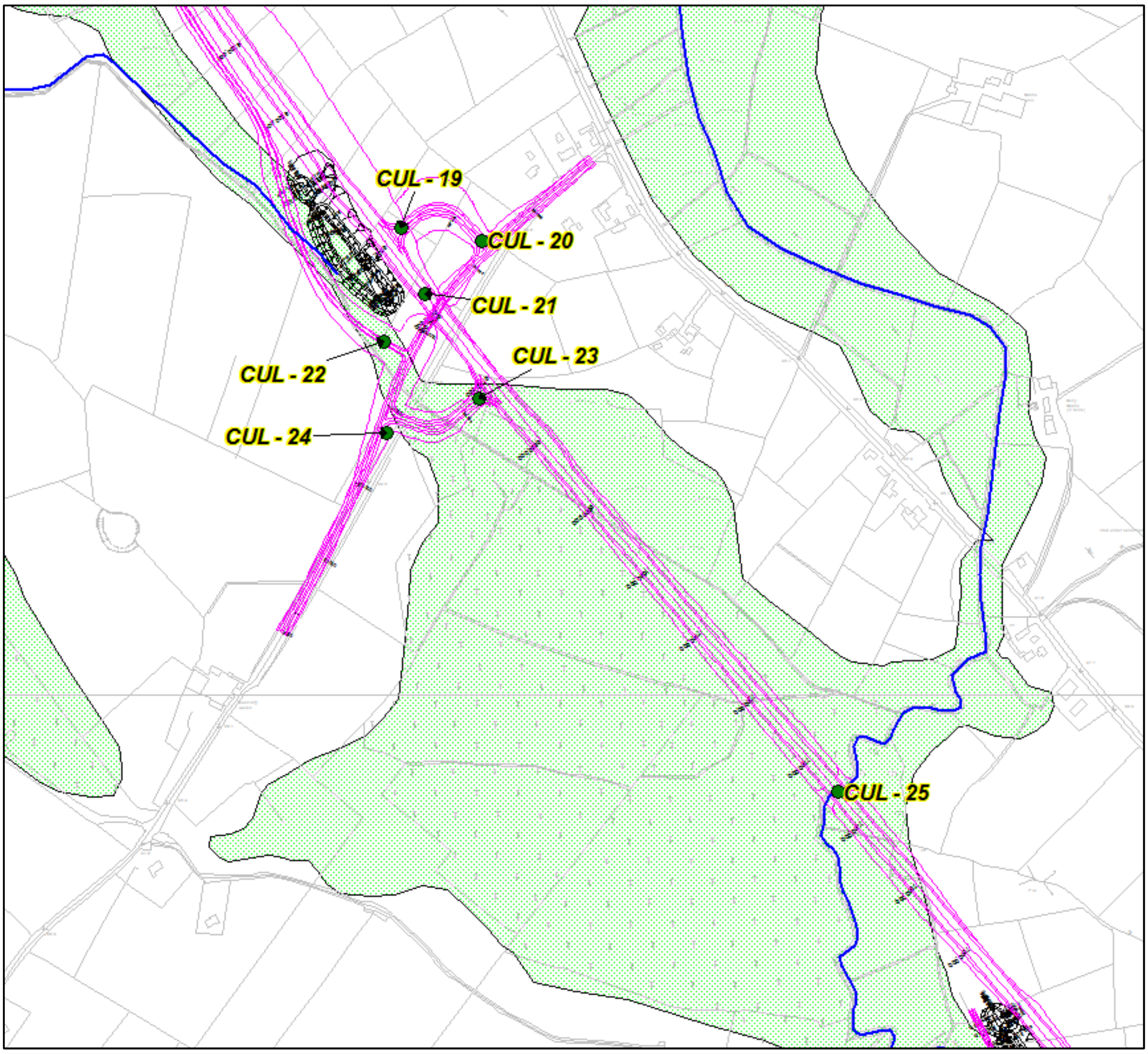


Figure 8.2 Watercourse Crossings and Arterial Drainage Benefitting Lands

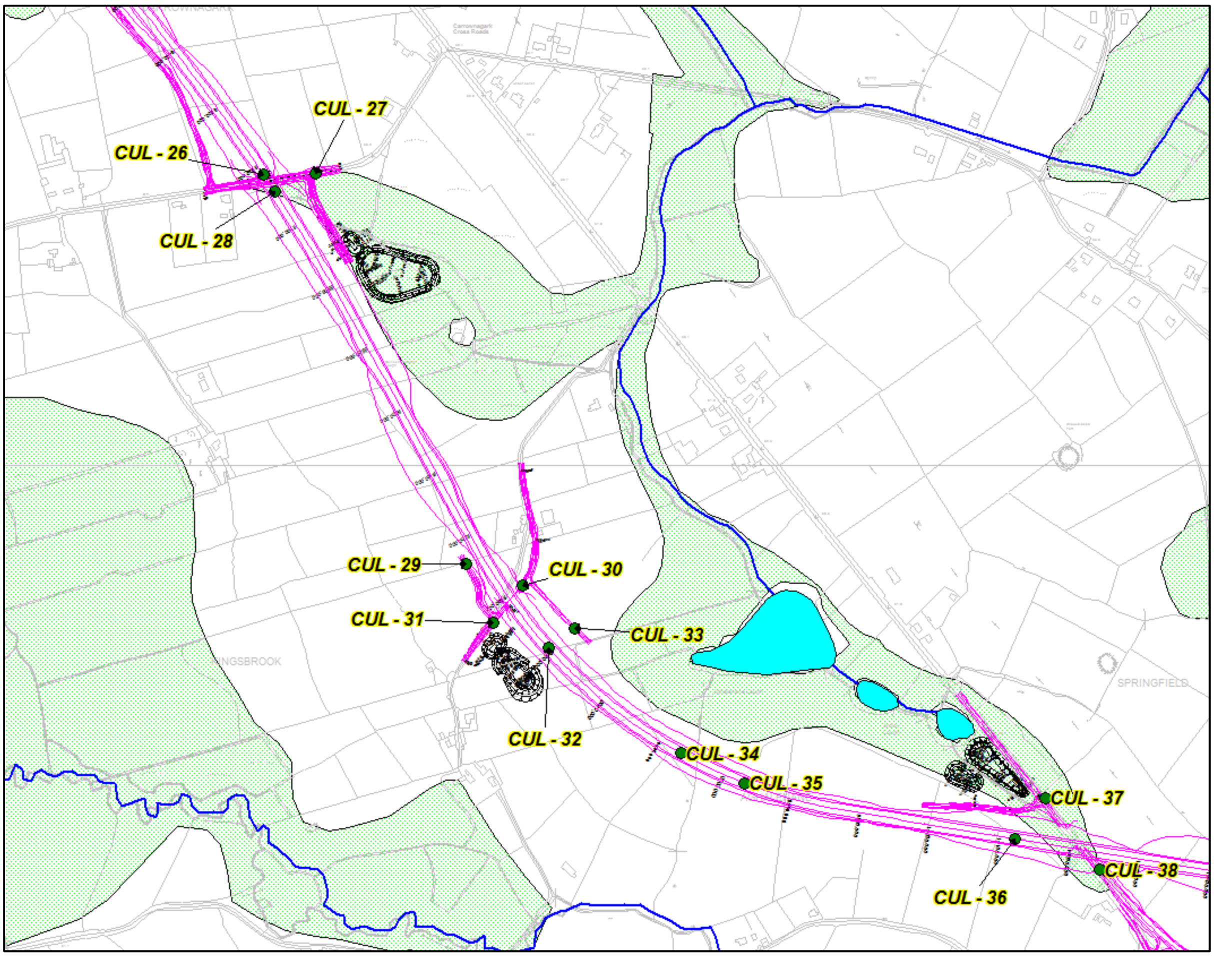


Figure 9.1 Watercourse Crossings and Arterial Drainage Benefitting Lands

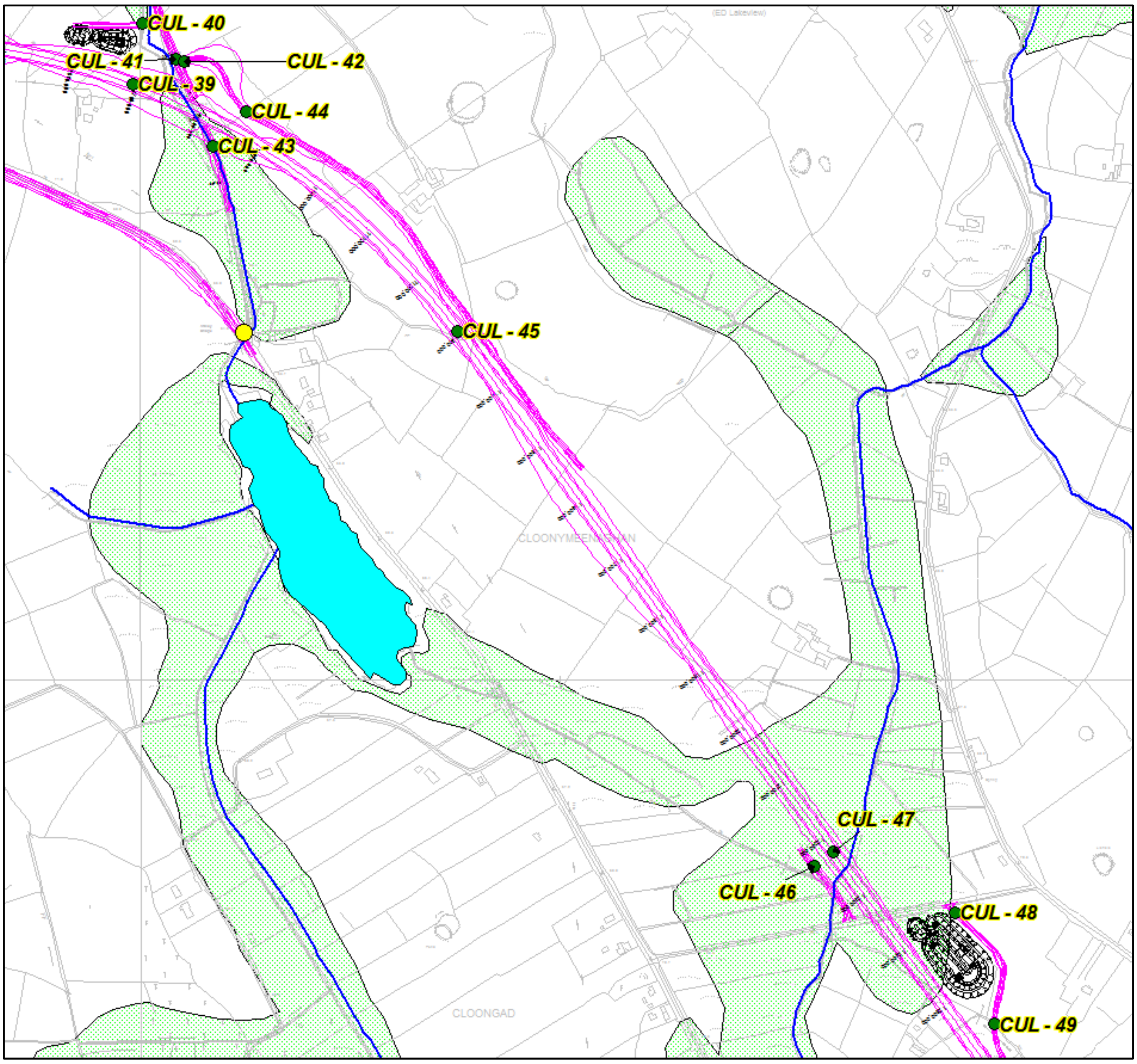
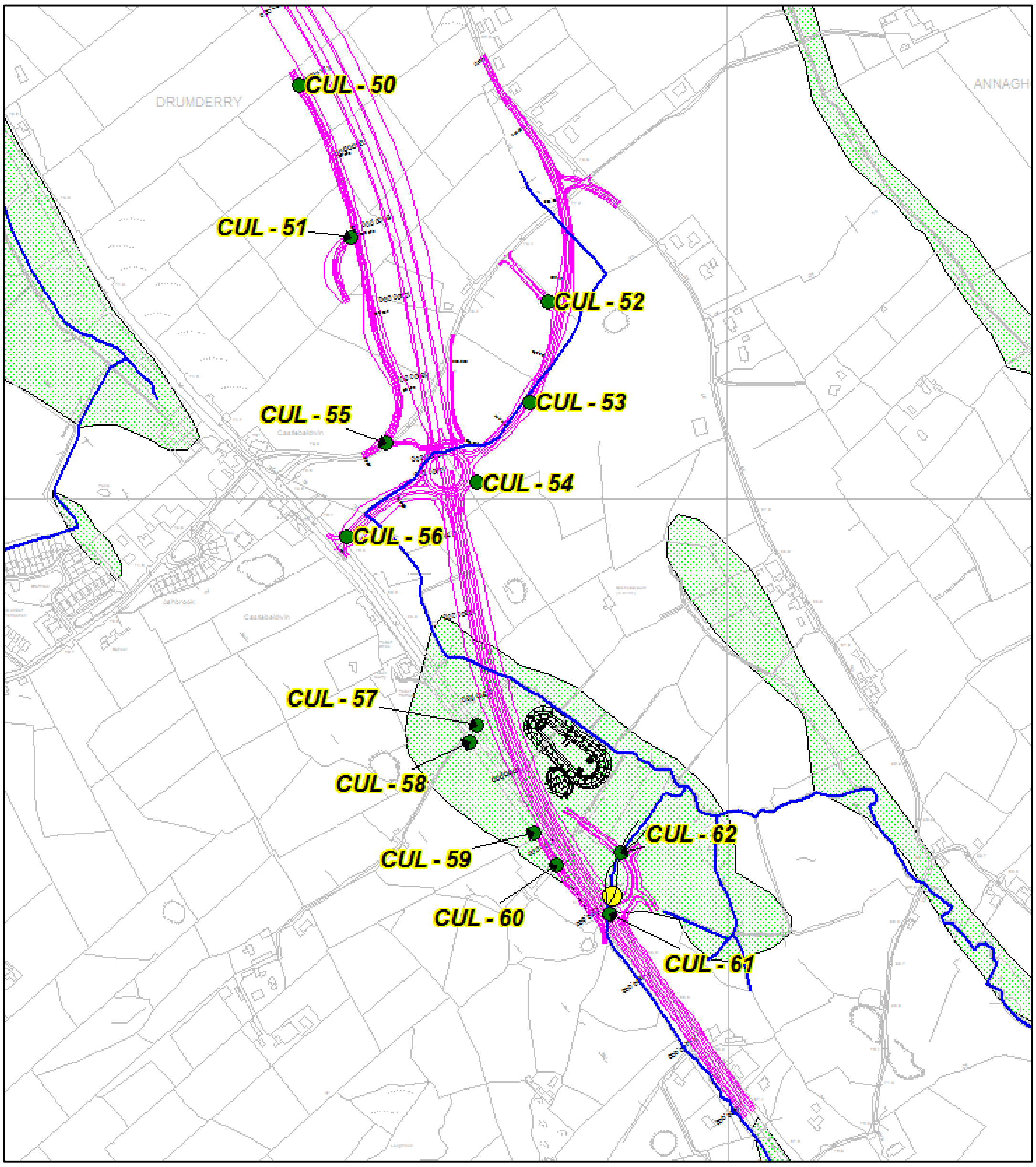


Figure 9.2 Watercourse Crossings and Arterial Drainage Benefitting Lands



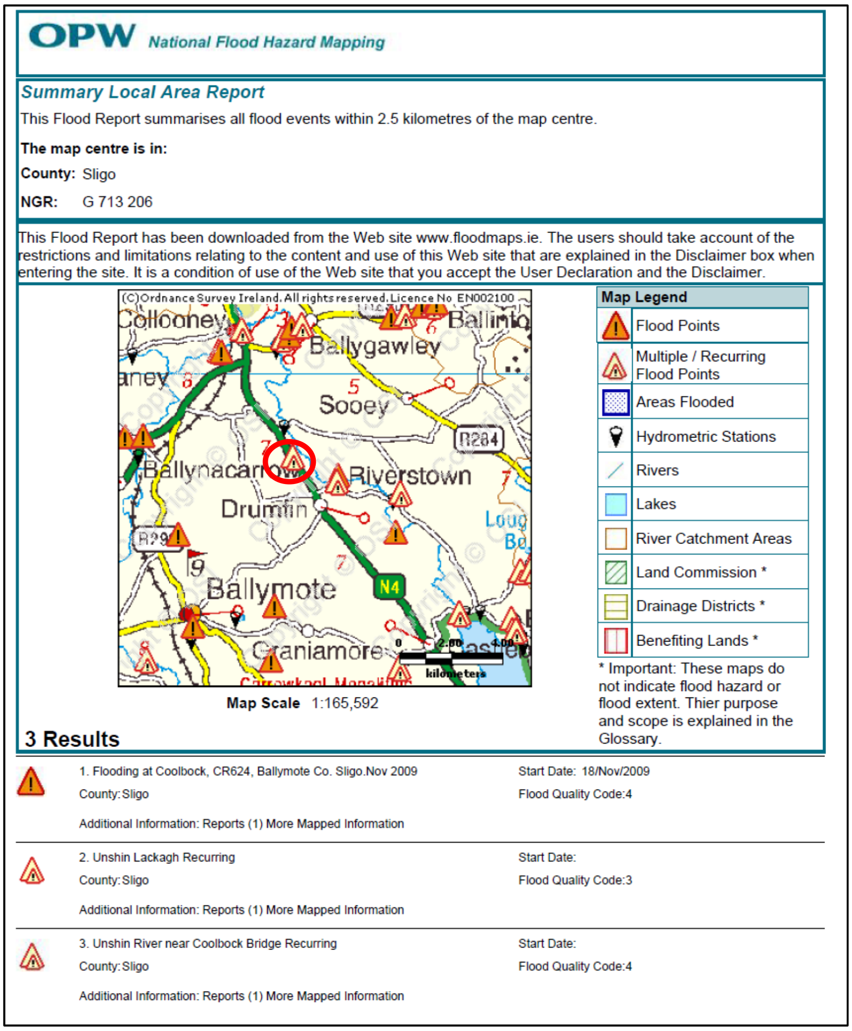
3.3.3 Flood Reports

The OPW national flood hazard mapping database www.floodmaps.ie was consulted to gather information on existing flood risk areas along the existing N4 road between Collooney and Castlebaldwin. One flood event was identified in the immediate study area associated with Turnalaydan Stream at Lackagh Bridge. It is reported in a letter sent to Sligo County Council on 6th December 2002 that an existing house located immediately on the right bank of the stream upstream of the bridge is flooded "every time there is a heavy rainfall". The house's finished floor level is surveyed at 40.57mOD whilst the river invert is 39.24mOD (See photo below). The Figure 10 below shows the extract from floodmaps.ie database showing the flood events recorded in the immediate study area.

Photo: Existing House located adjacent to Turnalaydan Stream. Note the sandbags on the steps.

The OPW have produced preliminary flood risk assessment (PFRA) mapping for Ireland including the study area between Collooney and Castlebaldwin (refer to Figure 11). By comparison of the benefitting land maps and the PFRA mapping it is concluded that the PFRA mapping underestimates the flood zones in the study area.

Figure 10 floodmaps.ie Summary map for the Study area



OPW National Flood Hazard Mapping

Summary Local Area Report

This Flood Report summarises all flood events within 2.5 kilometres of the map centre.

The map centre is in:

County: Sligo

NGR: G 713 206

This Flood Report has been downloaded from the Web site www.floodmaps.ie. The users should take account of the restrictions and limitations relating to the content and use of this Web site that are explained in the Disclaimer box when entering the site. It is a condition of use of the Web site that you accept the User Declaration and the Disclaimer.

Map Scale 1:165,592

Map Legend
Flood Points
Multiple / Recurring Flood Points
Areas Flooded
Hydrometric Stations
Rivers
Lakes
River Catchment Areas
Land Commission *
Drainage Districts *
Benefiting Lands *

* Important: These maps do not indicate flood hazard or flood extent. Thier purpose and scope is explained in the Glossary.

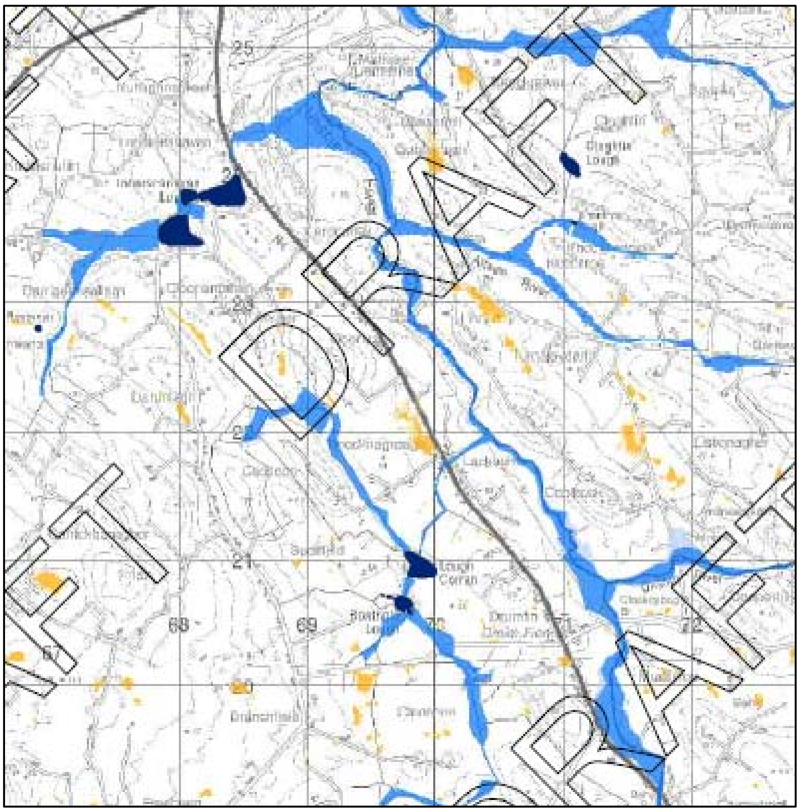
3 Results

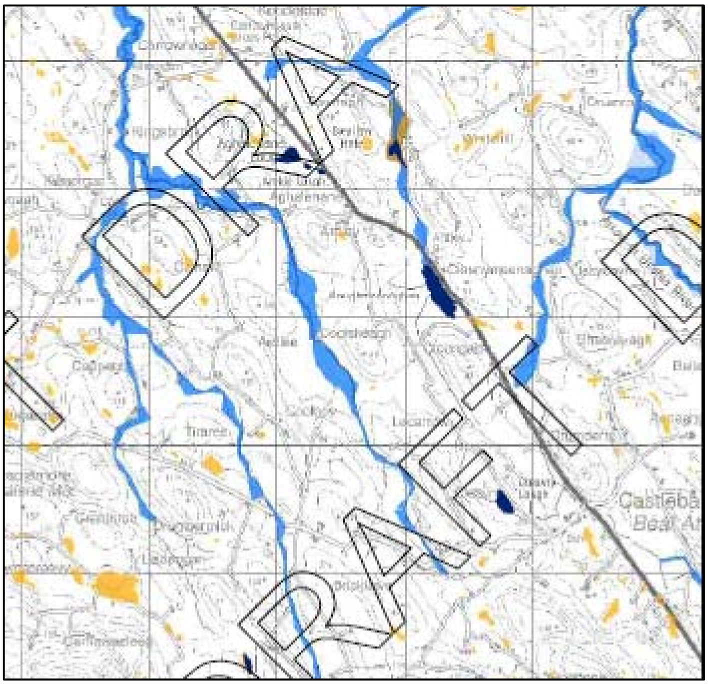
1. Flooding at Coolbock, CR624, Ballymote Co. Sligo.Nov 2009 — Start Date: 18/Nov/2009
County: Sligo — Flood Quality Code:4
Additional Information: Reports (1) More Mapped Information

2. Unshin Lackagh Recurring — Start Date:
County: Sligo — Flood Quality Code:3
Additional Information: Reports (1) More Mapped Information

3. Unshin River near Coolbock Bridge Recurring — Start Date:
County: Sligo — Flood Quality Code:4
Additional Information: Reports (1) More Mapped Information

Figure 11 PFRA mapping for the study area





4 DESIGN FLOWS

4.1 Design Flow Methodology

A design flood may be determined by either of two broad categories of methods, namely,

i. Methods based on statistical analysis of flood peak data
ii. Methods based on a design rainstorm and a rainfall-runoff model which converts the design rainstorm into a design flood.

A further distinction arises between gauged and ungauged catchment methods. The latter use formulae which relate some key component of the method, such as the mean annual flood or unit hydrograph time to peak, the catchment descriptors such as area, slope, and mean annual rainfall among others.

If catchments are gauged and there is a sufficiently long flow record (typically more than 20 years) then a relationship between peak flood flow and return period can be established by applying an Extreme Value distribution to the data series of “Annual Maximum” or “Peaks Over Threshold” (POT) flow series. The theory of extreme values states that if Z is the maximum of a number of other random variables X_1, X_2, ……X_N then the distribution of Z converges towards one of the 3 types of Extreme Value distribution(EV1, EV2 or EV3) as N becomes infinitely large. Generally in Ireland the EV1 (2 parameter) distribution fits reasonably well the majority of gauged rivers. If less data is available (e.g. typically less than 20years) then this can be used to give an approximation to the location parameter (i.e. mean Annual Maximum Flow) and a corresponding parameter for its scale deduced from regional statistics or pooled station statistics.

The statistical method may be used on a single site basis or on a pooled basis. In respect to the latter, which is recommended by the UK flood estimation handbook, 1999, the flood data from several river sites are in effect pooled together to provide an improved estimate of the required flood value. Pooled analysis is regarded as providing a more reliable estimate of the required flood, providing that catchments included in the “pooling” group are sufficiently similar in area, annual rainfall and soil/geology conditions. The latter is expressed by the descriptor BFIHOST, a quantity which is not available for Irish catchments. An earlier pooling method, based on geographical proximity of catchments rather than physical and climatic similarity was used in the Flood Studies Report. A FSR dimensionless flood growth curve is available for Ireland and will be referred to latter.

In the absence of actual data, the most appropriate methods for estimating design floods on ungauged catchments in Ireland are presented in the Flood Studies Report 1975. Three categories of ungauged flood estimation methods are available: namely:

- Mean Annual Flood by catchment characteristics plus Growth Curve Approach
- FSR Unit Hydrograph and Design Storm Method
- Rational Method Estimation (not considered in this study as catchment is large)

4.2 Ungauged Flood Estimation – Mean Annual Flood plus Growth Curve Approach

4.2.1 Introduction

There are a number of estimation methods available for estimating the mean annual Flood depending on the size of Catchment involved. These are:

i. Original 6-variable Flood Study Report (FSR) Catchment Characteristic (C.C.) Equation (typically for catchments exceeding 50 to 100km^2)
ii. Institute of Hydrology IH 124 Equation 3-variable C.C. Equation

The Mean Annual Flood plus Growth Curve method presented in the Flood Study Report (NERC, 1975) involves the following two Steps:

1. Estimation of the mean Annual Maximum Flood (QBAR) from Catchment Characteristics using a multiple regression Equation, and

2. Use of a regional flood Growth Curve to convert the QBAR estimate into the t-year Flood Flow.

4.2.2 Original 1975 Flood Study Report Flood Estimation Equation

The original 1975 FSR investigation involved flood frequency analysis of some 5500 record years from 430 British gauging Stations and 1700 record years from 112 Irish sites. The catchment areas varied from 0.05 to 9868km2 and annual maximum flows from 0.06 to 997cumec. The FSR six-variable catchment characteristic regression equation for Ireland to estimate the mean annual maximum flood can be expressed as follows:

$$Q_{BAR} = \mathit{0.0172}\ AREA^{0.94} F_S^{0.27} SOIL^{1.23} R_{SMD}^{1.03} S_{1085}^{0.16} (1+LAKE)^{-0.85}$$

The FSR six-variable catchment characteristic equation was also derived for SAAR replacing R_{SMD} :

$$Q_{BAR} = 0.00042\ AREA^{0.95} F_S^{0.22} SOIL^{1.18} SAAR^{1.05} S_{1085}^{0.19} (1+LAKE)^{-0.93}$$

AREA is the catchment area (km^2).

STMFRQ (stream frequency, Fs) is the number of stream junctions per km^2 on a 1:25,000 scale map. For Ireland this can be determined from a 1inch map and converted (using a formula given in the FSR) to an equivalent 1:25,000 (2.5 inch) number.

S1085 is the slope of the main channel between 10% and 85% of its length measured from the catchment outlet (m/km), (E85-E10)/(0.75 x MSL) where E is elevation along the channel and MSL is the main stream/ channel length.

SAAR is long term mean annual rainfall amount in mm and 1:625,000 mapping of this parameter is available for Ireland based on meteorological records from 1941 to 1970.

R_{SMD} is a measure of rainfall excess, in mm given by 1-day R5 rainfall reduced by a weighted mean of annual soil moisture deficit (SMD).

SOIL is an index of how the soil may accept infiltration and is a measure of the Winter Rainfall Acceptance Potential (WRAP). It can be determined from FSR mappings at 1 : 625,000 scale for Ireland. The SOIL index is based on only five classifications (very high, high, moderate, low and very low WRAP) and the mapping scale and number of categories are regarded as providing a very coarse measure of catchment runoff potential. The Flood Estimation Handbook in the UK have replaced the SOIL index by a more extensively classified and calibrated variable called HOST (Hydrology Of Soil Types) provided at a grid resolution of $0.5km^2$.

Lake is an index defined as the fraction of catchment draining through lakes or reservoirs and the areas contributing to lakes whose surface area exceeds 1% of the contributing area is recorded.

The FSR equation has a standard factorial error of 1.47. The factorial error applies to the middle of the data set and consequently will be significantly higher at both ends of the data set.

The Q_{BAR} estimate is multiplied by a growth factor derived either from the national, regional or pooled growth curve to arrive at the T – year flood estimate.

4.2.3 Institute of Hydrology Report No. 124 3-Variable Equation

In 1994 the Institute of Hydrology carried out further regression studies on small catchments (areas < $25km^2$). A total of 87 catchments ranging from 0.9km2 to 24.7km2 were available. 71 of these catchments were chosen as completely rural catchments having urban fractions of less than 0.025. The following 3-variable equation was derived

$$Q_{BAR} = \mathit{0.00108}\ AREA^{0.89} SAAR^{1.17}\ SOIL^{2.17}$$

Factorial Error = 1.65, N =71.

This equation is widely used in Ireland to estimate flood flows and Greenfield runoff from small catchments. Similar to the FSSR No. 6 equation the representation of lower runoff catchments (type 1 to 3) was poor being represented by only 16 of the 71 catchments. For the smaller catchments a two parameter equation (without the soil parameter performed as well as the IH 124 3-variable equation (Cawley and Cunnane, 2003).

The IH-124 has been modified, under an interim code of practice (ICP) method, to account for estimation of design floods for catchments of less than 50ha as follows:

$$Q_{BAR} = \mathit{0.00108}\ (\text{Area}/0.5)\ (0.5)^{0.89} \text{SAAR}^{1.17}\ \text{SOIL}^{2.17}$$

4.2.4 Flood Frequency Growth curve for Ireland

To estimate the T - return period flood flow the QBAR (mean annual flood flow) estimate is multiplied by a flood growth curve as follows:

$$QT = XT * QBAR$$

The FSR (1975) derived 9 regional Flood growth curves for England and a single national Flood Growth Curve for Ireland. The growth curve for Ireland is:

$$XT = -3.33 + 4.2\ e^{\,0.05Yt} \quad \textit{where } Yt = -\ln(-\ln(1-(1/T)))$$

Table 6 FSR National flood growth curve for Ireland

Return Period T years	2	5	25	50	100	200	500	1000
X_T	0.948	1.197	1.598	1.775	1.956	2.143	2.400	2.603

This equation is regressed on data for Ireland and includes catchments on the west as well as the east of the country with quite a wide variation in conditions. The sample catchments for which hydrometric gauging data was available were the larger catchments, in excess of 100km^2.

Cawley and Cunnane (2003) investigated regionally based growth curves in Ireland and concluded that the use of National Growth Curve be retained particularly for western and mid-land catchments. This assessment used only the OPW data set which did not include the EPA gauged catchments as the majority of the EPA had only a B rating classification (i.e. only reliable for estimating QBAR to 5 year return period) and of relatively short duration. This assessment therefore would have a tendency to represent OPW drainage schemes of a rural nature and would not include the Dublin and Wicklow east coast areas.

It is generally accepted that along the East Coast of Ireland the flood growth curve is likely to be steeper than the midlands and west of Ireland regions and that the curve derived for Wales (region 9) is probably more appropriate than the national growth curve.

Bruen (2005) in a recent study of mid-eastern and small Dublin catchments commissioned by Dublin City Council as part of the Greater Dublin Drainage Study concluded that the FSR national growth curve is likely to lead to an underestimation of flood flows for high return period in the Mid-eastern side of Ireland and especially in the smaller urbanised catchments of the Dublin Area. This assessment found that out of 23 catchments examined the FSR growth curve over estimated on two catchments was comparable for 7 catchments and underestimated for the remaining 14 catchments.

For the purposes of this study the National FSR growth curve will be retained but in all cases the statistical standard error will added to the QBAR estimate.

4.2.5 Flood Frequency Growth curve for Ireland

Examination of the Flood Study Report (1975) Winter Rainfall Acceptance Potential mapping for Ireland gives a type 3 WRAP class (moderate WRAP or conversely moderate surface runoff) for the majority of the N4 route, refer to Figure 2. This mapping is extremely coarse. In the majority of small catchment applications it is justifiable to perform a local catchment assessment of the WRAP using the same methodology as the FSR but with more refined local information. The WRAP Soil factor is calculated as follows:

The soil survey allocated a soil profile to the five class WRAP system by weighting four main soil and site properties. These parameters were in order of priority:

- Soil water Regime
- Depth to impermeable horizon
- Permeability above impermeable horizon
- Slope (which accentuates the effect of the above three parameters)

TABLE 7 Allocation of Soil Sites to WRAP Classes

Water regime class	Depth to impermeable horizon (cm)	Slope classes								
		<3.5%			3.5-14%			>14%		
		Permeability class (above impermeable horizon)								
		Rapid	Medium	Slow	Rapid	Medium	Slow	Rapid	Medium	Slow
1	>80	1	1	2	1	1	2	1	2	3
	80-40	1	1	2	3	3	3	3	3	4
	<40	-	-	-	-	-	-	-	-	-
2	>80	2	2	3	3	3	4	-	4	5
	80-40	2	3	3	3	4	4	4	4	5
	<40	3	4	4	4	4	4	4	5	5
3	>80	4	4	5	5	5	5	-	5	5
	80-40	4	5	5	5	5	5	-	5	5
	<40	5	5	5	5	5	5	5	5	5

NOTE: 1. Water Regime Class.
1. Rarely waterlogged within 60 cm at any time (well and moderately well drained)
2. Commonly waterlogged within 60 cm during winter (imperfect and poor)
3. Commonly waterlogged within 60 cm during winter and summer (very poorly drained)

4.2.6 FSR Unit Hydrograph and Design Storm

The unit hydrograph method most widely used in Ireland for ungauged catchments is the FSR triangular unit hydrograph and design storm method. This method estimates the design flood hydrograph, describing the timing and magnitude of flood peak and flood volume. This method requires the catchment response characteristics (time to peak, t_p), design rainstorm characteristics (return period, storm duration, rainfall depth and profile) and catchment runoff / loss characteristics (percentage runoff influenced by SOIL type, catchment wetness index and rainfall intensity).

The FSR unit hydrograph prediction equation was derived from 1631 events from 143 gauged catchments ranging in size from 3.5 to 500km^2 but only included one Irish catchment. The result was a triangular Unit Hydrograph described by the time to peak T_p of the catchment derived from Catchment Characteristics (namely S_{1085}, SAAR and MSL). The instantaneous triangular unit hydrograph is defined by a time to peak T_p a peak flow in cumecs per 100km^2 $Q_p = 220/T_p$ and a base length $T_b = 2.52\ T_p$.

Subsequent FSSR reports and in particular Report No. 16 (1985) and IH 124 (1994) (for small catchments) slightly modified the T_p equation and the calculation of the percentage runoff (PR)

$$T_p = 283MSL^{0.23}\ S1085^{-0.33}\ URBAN^{-2.2}\ SAAR^{-0.54} \quad \text{(FSSR No. 16)}$$

and the percentage runoff

$$PR = SPR + DPR_{CWI} + DPR_{RAIN}$$

Where $SPR = 10S_1 + 30S_2 + 37S_3 + 47S_4 + 53S_5$ and S_1 to S_5 are the soil classes

$DPR_{CWI} = 0.25(CWI – 125)$ and CWI = catchment wetness Index which is a function of SAAR

And $DPR_{RAIN} = 0.45(R – 40)0.7$ for storm depth > 40mm and = 0 for R < 40mm

The critical design rainstorm duration is $D = (1 + 0.001SAAR)\ Tp$

The FSR hydrograph method was tested by Bree et al. (1989) on 36 Irish catchments and it was found that the Q25 was overestimated in 30 out of the 36 catchments with 24 of the 36 catchments being overestimated by over 150% with the mean value was 164%. The factors attributed to the overestimation were:

- (i) underestimation of Tp, and storm duration D,
- (ii) overestimation of Qp caused by the factor 220 being too large for Irish catchments, derived principally for UK catchments, and
- (iii) overestimation of percentage runoff, PR.

The full procedural steps are listed in the technical annex and this method will be applied to all catchments exceeding 5km^2. Similar to the previous method the SOIL classification may be derived from local information as outlined previously.

The average non-separated flow (ANSF) or baseflow of the hydrograph is calculated by

ANSF= [33(CWI-125)+3.0SAAR+5.5]x10^{-5}, where CWI is the catchment wetness index.

4.2.7 Rational Method

The rational method remains in use, especially for small catchments despite the advent of the FSR method described above. The general form of the Rational Method equation is

$$Q_T = 0.278\, C_p\, C_v\, I_{tc,T}\, AREA$$

where:

T is the return period

C_p is a peaking factor (often taken as 1.3)

C_v the volumetric runoff coefficient (deduced from local Slope, landuse and soil permeability information

$I_{tc,T}$ i= average storm rainfall intensity (mm/hour)for duration tc and return period T.

t_c is the time of concentration defined as the travel time from the furthest point on the catchment to the outlet.

This method may be used in conjunction with the MAFF 1980 runoff method.

4.2.8 Climate Change Allowance

Climate change scenarios produced by the UK Hadley centre suggest fluvial floods in the 2080's increasing by up to 10% (low and medium low scenarios) or by up to 20% (medium high and high scenarios). Present recommendations are to include in the design flow a 20% increase in flood peaks over 50years return period as a result of climate change. This scenario based on the Irish growth curve will result in a present day 100year flood becoming a 25year flood in approximately 50 years time.

Other predicted climate change effects for the UK are:

- A 4 to 5mm per annum rise in mean sea level
- Additional intensity of rainfall of 20%
- An additional 30%winter rainfall by the 2080's
- A reduction of 35/45% Rainfall in summer
- The 1 in 100year rainfall storm to increase by 25%

Kiely (1999) published results which indicate significant changes in rainfall totals at several Irish locations since the mid 1970's. He attributes this to changes in the North Atlantic Oscillation (NAO), a quantity based on seasonal pressure difference between Iceland and the Azores. Kiely (1999) also found differences in rainfall frequencies at a number of the synoptic weather stations – for Valentia he found that, for several durations, 10 and 30year return period rainfall depths are increased by approximately 20% when calculated from the most recent data (1976 – 96) as compared to values calculated from the entire period of record (1940 –1996). While such changes in rainfall regime provide a warning it is strange that changes in flood behaviour and in particular increase in flood magnitudes were not noticed in many rivers until the 1990's.

Changes in circulation patterns across Europe have been linked to changes in flood frequency and hence increased risk of flooding in parts of southwest Germany. The circulation pattern known in Germany as "West Cyclonic" during the winter months has increased over the last century giving rise to increased risk of flooding. Caspary (2003) has shown flood magnitudes that were once deemed to be 100year return period floods, would be deemed to have much smaller return periods (5 to 30years) if judged on data of the past 25 years.

The changes observed in SW Germany may not be replicated in Ireland but it is clear that account must be made for climate change impact in view of the above findings.

The OPW have produced guidelines in respect to design considerations of possible climate change for flood risk management practice. The recommended design allowances to be used for increases in flood flows during the sensitivity and / or design process is 20%.

4.3 Design Flow Calculation

4.3.1 **Main Crossings**

As none of the study area's watercourses are gauged the design flows for each proposed crossing have been estimated using catchment characteristics.

Table 8 below summarises the catchment characteristics at each of the main crossings,

Table 8 Main Crossings Catchment Characteristics

Culvert	Area, km^2	SAAR, mm/annum	WRAP	MSL, km	S1085, m/km	Fs, Jns/km^2	Lake, %
Cul-0 Markree Demesne S.	6.2	1247	0.375	3.3	15.8	3.2	95%
Cul-14 Turnalaydan S.	16.1	1215	0.4	4.0	3.0	2.1	0
Cul-25 Drumfin R.	21.5	1242	0.41	8.1	12.4	0.9	0
Cul39/40 Springfield S.	1.59	1223	0.4	2.3	9.2	7.3	82%
Cul-46/47 Lissycoyne S.	1.46	1231	0.4	2.24	8.9	4.2	17%
Cul-61/62 Tributary of the Drumderry Stream	1.24	1259	0.4	*Note1*			0%

Note 1: The main stream channel rises from a large spring (Tobermahon) less than 0.75km upstream of Cul 61/62.

Table 9 below presents the calculated mean annual maximum flood flow or Qbar for each crossing for the FSR method (with and without the lake factor) and the IH-124 method and the Unit Hydrograph average non-separate flow (ANSF) baseflow and time to peak Tp.

Table 9 Main Crossings Qbar estimations

Culvert	FSR Qbar1, cumec	FSR Qbar2, cumec	IH-124, cumec	Comment	Baseflow, cumec	Tp, hours
Cul-0 Markree Demesne S.	1.43	2.67	2.73	IH-124 method applicable	0.23	3.2
Cul-14 Turnalaydan S.	4.83	4.83	7.13	FSR QBAR applicable	0.58	5.8
Cul-25 Drumfin R.	7.04	7.04	9.98	FSR QBAR applicable	0.80	4.3
Cul39/40 Springfield S.	0.48	0.84	0.91	FSR QBAR applicable with lake factor	0.06	3.5
Cul-46/47 Lissycoyne S.	0.61	0.71	0.85	IH-124 method applicable	0.05	3.6

Culvert	FSR Qbar1, cumec	FSR Qbar2, cumec	IH-124, cumec	Comment	Baseflow, cumec	Tp, hours
Cul-61/62 Tributary of the Drumderry Stream	-	-	0.76	IH-124 method applicable	0.05	-

Table 10 below presents the proposed design peak flows for each catchment without an allowance for lake storage attenuation. The relevant method's standard factorial error (FE) is applied appropriately.

Table 10 Main Crossings Qbar estimations

Culvert	Qbar	Q100	Q100+FE	Q1000	Q1000+FE
Cul-0 Markree Demesne S.	2.73	5.35	8.83	7.11	11.73
Cul-14 Turnalaydan S.	4.83	9.47	13.92	12.57	18.48
Cul-25 Drumfin R.	7.04	13.80	20.28	18.33	26.94
Cul39/40 Springfield S.	0.43	0.85	1.25	1.13	1.66
Cul-46/47 Lissycoyne s.	0.85	1.67	2.74	2.21	3.25
Cul-61/62 Brickeen S.	0.76	1.49	2.46	1.98	3.26

As a check of the IH-124 method used above for the Tributary of the Drumderry Stream catchment (source at Tobermahon Spring) the following two additional methods have been used to estimate the design flow based on recent studies undertaken in County Clare for karst catchments:

a. Typical mean annual flood runoff rates from karst catchments in West of Ireland QBAR = 0.09 to 0.1 cumec per km^2 =>0.62 to 0.69 cumec => Q100=**0.24 cumec**
b. Rainfall Return Data: The 6-day 2 year, 5 year and 150 year rainfall return period rainfall amount for the Tributary of the Drumderry Stream catchment is projected at 84.2mm, 99.1mm and 151.2mm (14mm/day, 16.5mm/day and 25.2mm/day) respectively. (Note: a 140 year rainfall return period is approximately equivalent to a 1 in 100 year rainfall event). Assuming that overland flow equivalent to 100% runoff of the effective rainfall would occur following a period of prolonged heavy rain (i.e. high water table), the 1 in 100 year overland flood flow is estimated at between **0.36 cumec**.

These additional methods would suggest that the IH-124 method greatly over-estimates the design flood flow for the Tributary of the Drumderry Stream catchment. Therefore the recommended design flow estimate for the Tributary of the Drumderry Stream will be the IH-124 QBAR estimate without the inclusion of the factorial error of 1.65 and combined with the FSR National Growth factor for the 100year and 1000yearreturn period flows.

Table 11 Recommended 100 and 1000year design flows for Main Crossings

Culvert	Design Q100	Design Q1000
Cul-0 Markree Demesne S.	8.83#	11.73#
Cul-14 Turnalaydan S.	13.92	18.48
Cul-25 Drumfin R.	20.28	26.94

Culvert	Design Q100	Design Q1000
Cul39/40 Springfield S.	1.25	1.66
Cul-46/47 Lissycoyne s.	1.67	2.21
Cul-61/62 Brickeen S.	2.46	3.26

Note: # = The design flow into Toberscanvan Lakes

4.3.2 Minor Crossings

The catchment areas of the minor culverts have been assessed by Sligo RDD. The average SAAR for study area is 1233mm/annum and the typical soil type relative to the WRAP classification is Type 2 to 4 (low to high run-off) is used here. The following table presents the design flow estimation for each of the minor crossings based on the IH-124 and as the areas are all less than 50 ha the ICP variation of the IH-124 is used which has a standard factorial error 1.65. The climate change (CC) factor allowance of 20% is included in calculations.

Table 12 Minor Crossings Qbar and design flow estimations

Culvert	Chainage	Area, km2	Qbar	Q100	Q100+FE+CC	Q1000	Q1000+FE+CC
Cul- Ret A	50	<0.1	0.044	0.085	0.169	0.113	0.224
Cul- Ret B	280	<0.1	0.044	0.085	0.169	0.113	0.224
Cul- Ret C	330	<0.1	0.044	0.085	0.169	0.113	0.224
Cul- Ret D	460	0.15	0.062	0.122	0.242	0.163	0.322
Cul- Ret E	900	0.64	0.445	0.445	0.882	0.591	1.171
Cul- Ret F	940	0.70	0.482	0.482	0.955	0.640	1.268
Cul- Ret G	1270	<0.1	0.044	0.085	0.169	0.113	0.224
Cul- Ret H	1500	<0.1	0.044	0.085	0.169	0.113	0.224
Cul- Ret I	1600	0.15	0.062	0.122	0.242	0.163	0.322
Cul - 1	2100	<0.1	0.044	0.085	0.169	0.113	0.224
Cul - 2	2380	0.195	0.166	0.326	0.645	0.432	0.856
Cul - 3	2500	0.112	0.095	0.187	0.370	0.248	0.492
Cul - 4	2520	0.009	0.008	0.015	0.030	0.020	0.040
Cul - 5	2610	0.009	0.008	0.015	0.030	0.020	0.040
Cul - 6	2700	0.028	0.024	0.047	0.093	0.062	0.123
Cul - 7	2980	0.029	0.025	0.048	0.096	0.064	0.127
Cul - 10	3390	0.033	0.028	0.055	0.109	0.073	0.145
Cul - 11	3540	0.062	0.053	0.104	0.205	0.137	0.272
Cul - 12	3700	0.066	0.056	0.110	0.218	0.146	0.290
Cul - 13	4000	0.004	0.003	0.007	0.013	0.009	0.018
Cul - 15	4660	0.006	0.005	0.010	0.020	0.013	0.026
Cul - 16	5300	0.25	0.213	0.417	0.827	0.554	1.098

Culvert	Chainage	Area, km2	Qbar	Q100	Q100+FE+CC	Q1000	Q1000+FE+CC
Cul - 17	5300	0.25	0.213	0.417	0.827	0.554	1.098
Cul - 18	5600	0.18	0.153	0.301	0.595	0.399	0.790
Cul - 19	6500	0.019	0.016	0.032	0.063	0.042	0.083
Cul - 20	6560	0.003	0.003	0.005	0.010	0.007	0.013
Cul - 21	6580	0.03	0.026	0.050	0.099	0.067	0.132
Cul - 22	6600	0.03	0.026	0.050	0.099	0.067	0.132
Cul - 23	6720	0.006	0.005	0.010	0.020	0.013	0.026
Cul - 24	6700	0.026	0.022	0.043	0.086	0.058	0.114
Cul - 26	8620	0.009	0.008	0.015	0.030	0.020	0.040
Cul - 27	8650	0.025	0.021	0.042	0.083	0.055	0.110
Cul - 28	8650	0.038	0.032	0.063	0.126	0.084	0.167
Cul - 29	9250	0.008	0.007	0.013	0.026	0.018	0.035
Cul - 30	9310	0.007	0.006	0.012	0.023	0.016	0.031
Cul - 31	9320	0.05	0.043	0.083	0.165	0.111	0.220
Cul - 32	9400	0.11	0.094	0.184	0.364	0.244	0.483
Cul - 33	9400	0.1	0.085	0.167	0.331	0.222	0.439
Cul - 34	9640	0.034	0.029	0.057	0.112	0.075	0.149
Cul - 35	9740	0.044	0.037	0.073	0.145	0.098	0.193
Cul - 36	10130	0.142	0.121	0.237	0.470	0.315	0.624
Cul - 37	10150	0.017	0.014	0.028	0.056	0.038	0.075
Cul - 38	10250	0.018	0.015	0.030	0.060	0.040	0.079
Cul - 41	10740	0.046	0.039	0.077	0.152	0.102	0.202
Cul - 42	10750	0.032	0.027	0.053	0.106	0.071	0.141
Cul - 43	10840	0.119	0.101	0.199	0.393	0.264	0.523
Cul - 44	10840	0.02	0.017	0.033	0.066	0.044	0.088
Cul - 45	11300	0.025	0.021	0.042	0.083	0.055	0.110
Cul - 46	12230	0.046	0.039	0.077	0.152	0.102	0.202
Cul - 48	12400	0.04	0.034	0.067	0.132	0.089	0.176
Cul - 49	12560	0.08	0.068	0.134	0.265	0.177	0.351
Cul - 50	13110	0.045	0.038	0.075	0.149	0.100	0.198
Cul - 51	13300	0.003	0.003	0.005	0.010	0.007	0.013
Cul - 52	13440	0.005	0.004	0.008	0.017	0.011	0.022
Cul - 53	13560	0.051	0.043	0.085	0.169	0.113	0.224
Cul - 54	13600	0.06	0.051	0.100	0.198	0.133	0.263
Cul - 55	13570	0.01	0.009	0.017	0.033	0.022	0.044

Culvert	Chainage	Area, km2	Qbar	Q100	Q100+FE+CC	Q1000	Q1000+FE+CC
Cul - 56	13700	0.043	0.037	0.072	0.142	0.095	0.189
Cul - 58	13650	0.091	0.078	0.152	0.301	0.202	0.400
Cul - 59	14080	0.013	0.011	0.022	0.043	0.029	0.057
Cul - 60	14120	0.007	0.006	0.012	0.023	0.016	0.031

4.4 Design Flood Hydrographs

The ungauged methods presented above provide only the peak flow rate. The shape of the flood wave or the overall flood volume was not determined (i.e. the methods did not determine the design flood hydrograph). The flood hydrograph is important when determining the attenuation effect of floodplains and quantifying the impact of the loss of flood storage by the physical encroachment of a development into a floodplain (i.e. such as the footprint of a road embankment across a floodplain area).

The shape of the Flood hydrographs for the main river / stream crossings is determined using the FSR Unit Hydrograph and Design Storm method. In order to retain consistency in respect to the recommended design flood peak magnitudes presented in Table 11 the computed flood hydrographs are scaled appropriately so as to achieve the same peak magnitudes.

Unit hydrographs have been produced for each catchment using the calculated peak design flows as the Q peak. Figures 12 to 21 present the hydrographs for each crossing bar Tributary of the Drumderry Stream. This stream because of its spring source will have a more extended and virtually constant flood magnitude that lends itself to a steady state analysis at the peak flow rate throughout. The graph's Y-axis is flow with units of cumec and X-axis is storm duration with units of hours.

Figure 12 Markree Demesne Stream Hydrograph for the Q100 and Q100+FE design flows

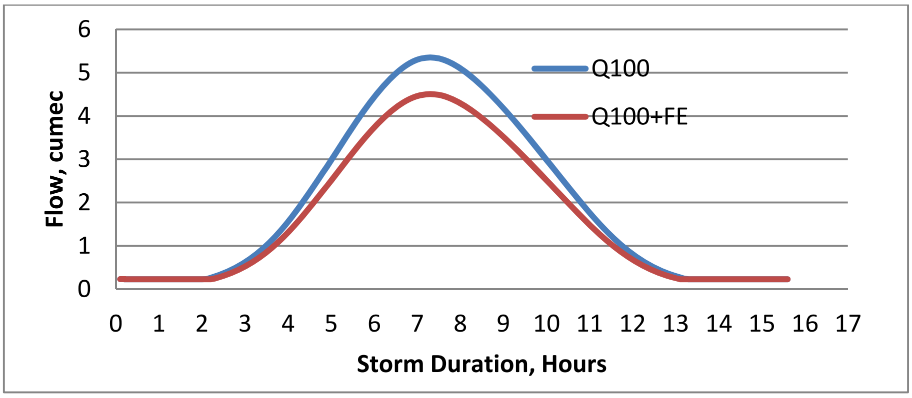


Figure 13 Markree Demesne Stream Hydrograph for the Q1000 and Q1000+FE design flows

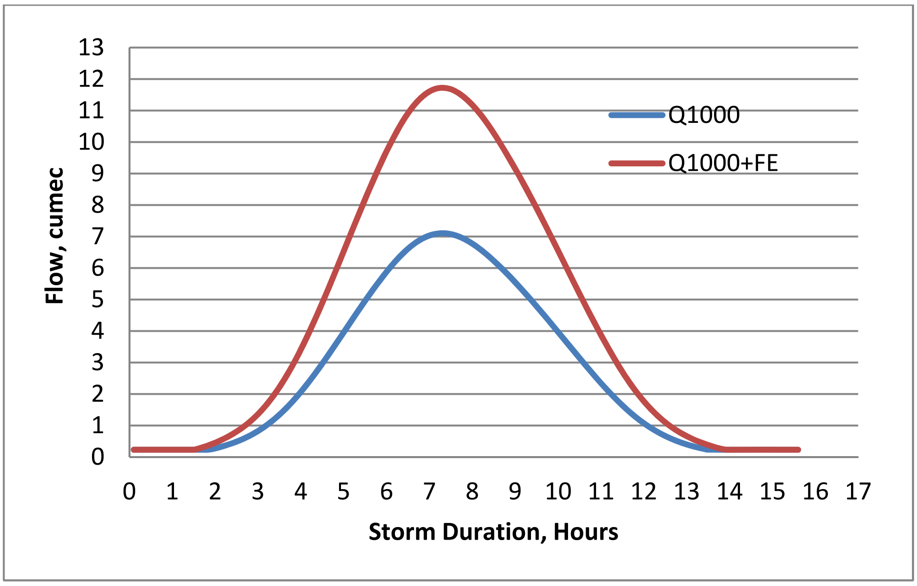


Figure 14 Turnalaydan Stream Hydrograph for the Q100 and Q100+FE design flows

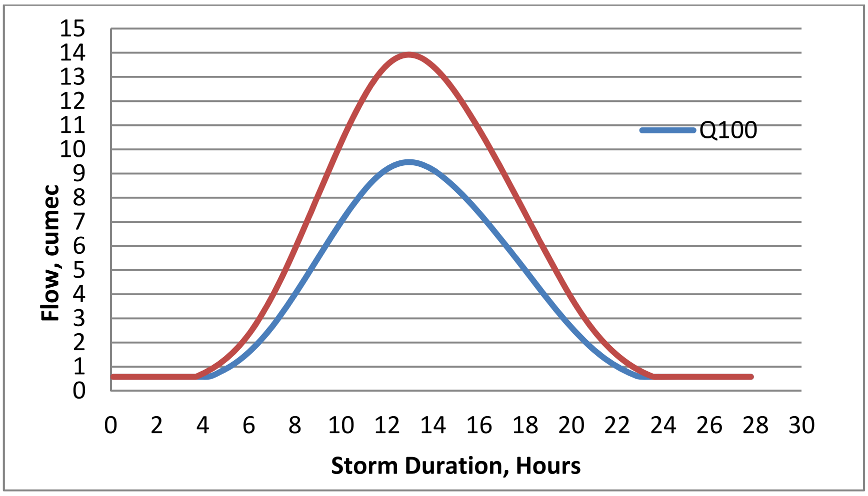


Figure 15 Turnalaydan Stream Hydrograph for the Q1000 and Q1000+FE design flows

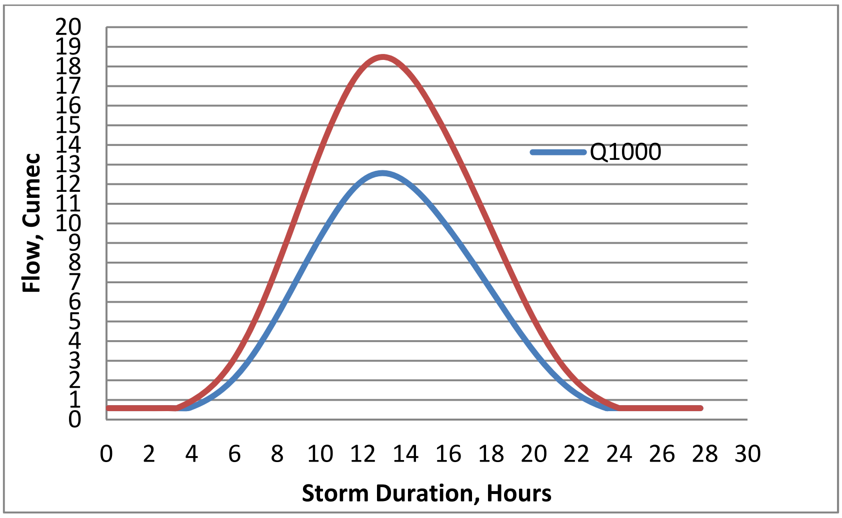


Figure 16 Drumfin River Hydrograph for the Q100 and Q100+FE design flows

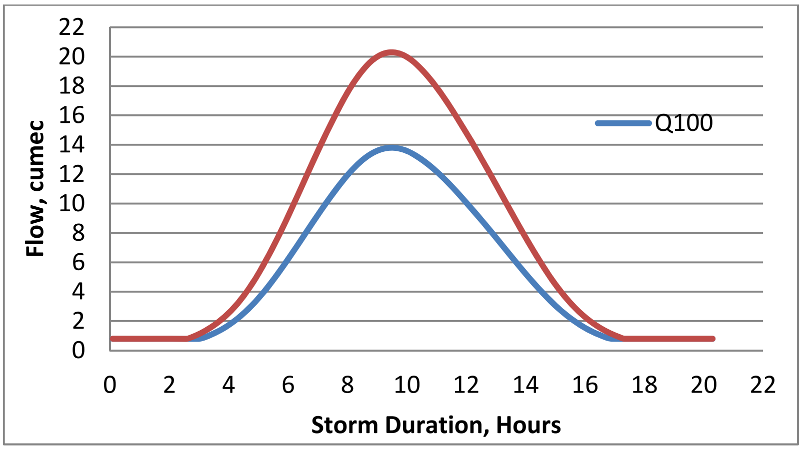


Figure 17 Drumfin River Hydrograph for the Q1000 and Q1000+FE design flows

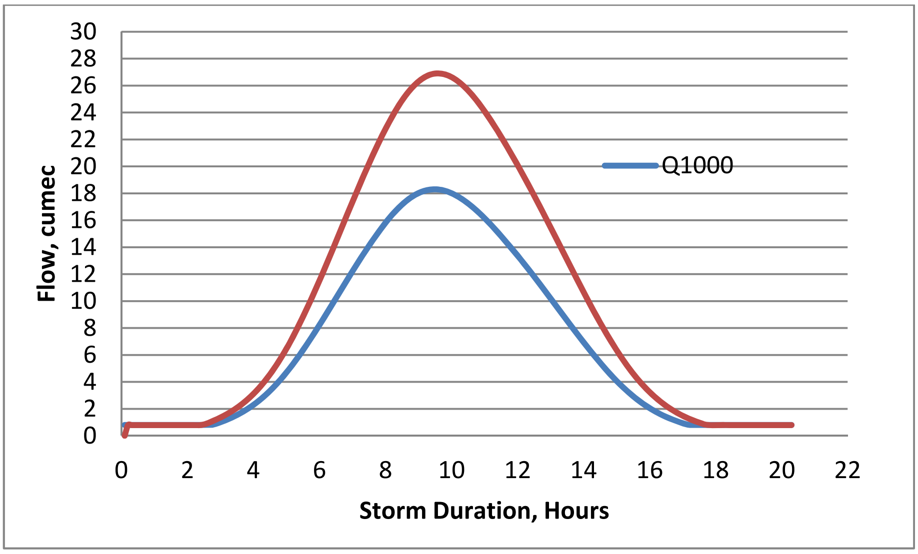


Figure 18 Springfield Stream Hydrograph for the Q100 and Q100+FE design flows

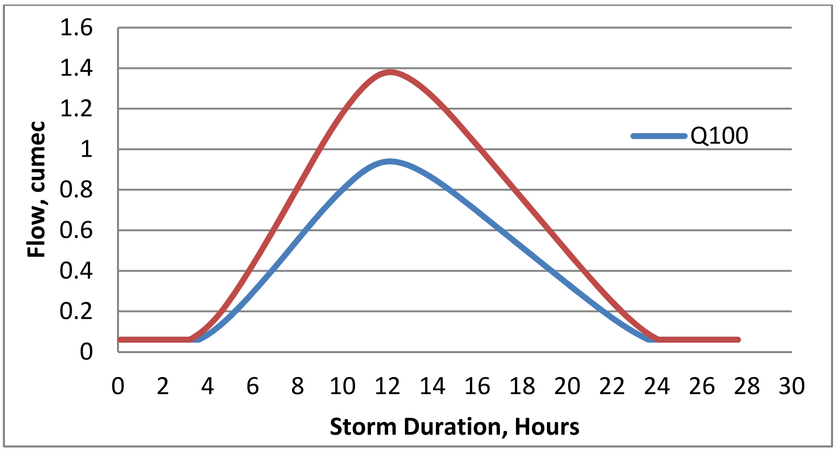


Figure 19 Springfield Stream Hydrograph for the Q1000 and Q1000+FE design flows

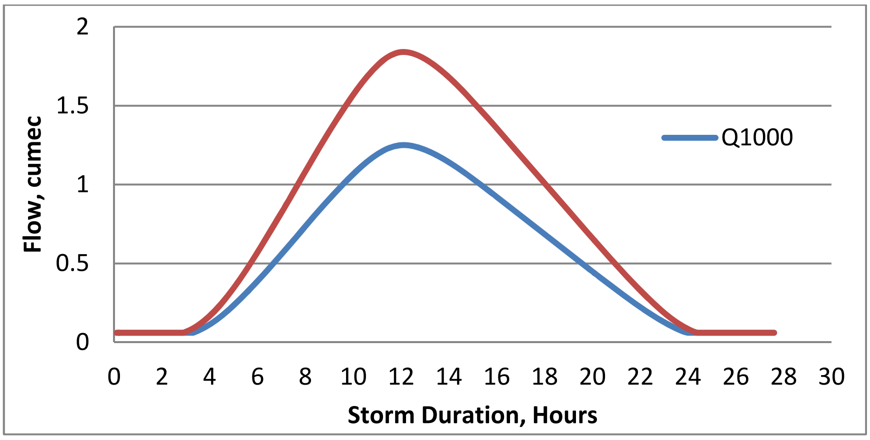


Figure 20 Lissycoyne Stream Hydrograph for the Q100 and Q100+FE design flows

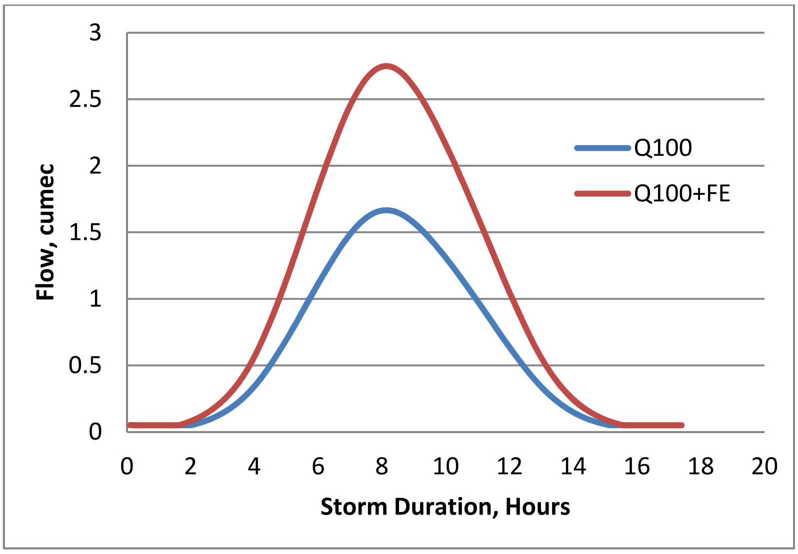
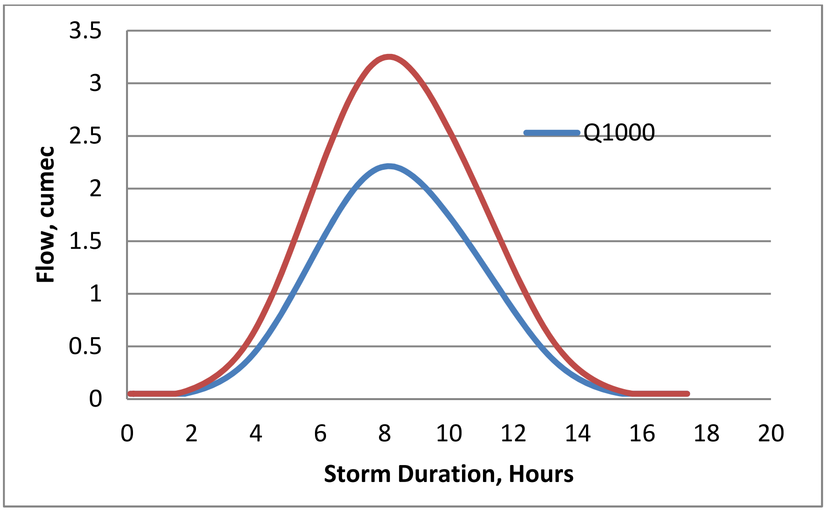


Figure 21 Lissycoyne Stream Hydrograph for the Q1000 and Q1000+FE design flows



5 Hydraulic Assessment

5.1 River Modelling Methodology

For each of the main watercourse crossings hydraulic models of the river channel and floodplain were developed so as to predict design flood levels for the existing and proposed scenarios and quantify the impact of the road crossing on flooding. These models were developed using a combination of aerial survey (Lidar and DTM surveys) and ground topographical survey data of the river channel and banks for the five main watercourse crossings. Both steady state and unsteady state simulations were carried out to examine peak flood levels and the attenuating effect of the river floodplain. By comparing the models for the existing and proposed scenarios the impact of the proposed structures and embankments and proposed storm drainage attenuation ponds and proposed accommodation roads on flood levels and flood storage can be determined. Simulations demonstrating the impact on flood risk of proposed road material deposit areas have also been undertaken.

The selected software used for this hydraulic assessment is HEC-RAS 4.1.0. HEC-RAS implements a 1-dim model of river flow and can solve for water elevation under steady conditions and in the most recent version gradually varying unsteady flows solving the full Saint Venant equations. HEC-RAS takes account of the conveyance and storage effects of floodplains in a one-dimensional manner only (i.e. in the longitudinal direction of flow). It does not resolve the possibly complex 2-dimensional aspects of floodplain flow or secondary flow caused by a structure. Hec-Ras (U.S. Army Corps of Engineers) is internationally accepted as one of the industry standard software packages for such applications and has been applied successfully by Hydro Environmental Ltd. to a range of river hydraulic studies throughout Ireland and the UK.

The model requires the following information:

- topographic survey data of river channel and flood plain (cross-section station, bed elevation and channel and overbank reach lengths),
- dimensions and elevation of relevant structures,
- upstream and internal flow boundary conditions,
- downstream water elevation boundary condition, and in the case of supercritical flow regime (Froude No. > 1) the upstream water elevation boundary condition,
- channel and flood plain roughness coefficients and
- local expansion and contraction shock loss coefficients.

The land, river bed and river structure survey data was collected by Murphy Surveys and Sligo RDD.

5.2 River Models

5.2.1 Markree Demesne (Toberscanavan outflow) Stream

5.2.1.1 Introduction

The proposed works at Markree Demesne (Toberscanavan) Stream located 1.4km south of the N17 / N4 junction at Collooney involves the replacement and extension of the existing 1200mm diameter pipe culvert with a box culvert under the proposed road embankment. A hydraulic model has been developed for the stream's reach extending from 760m upstream (including Toberscanavan Lough lower) and 685m downstream of the existing N4-crossing crossings. A total of 57 No. cross-sections were used in the model from survey data to describe the river reach. The downstream boundary condition for the model is taken as normal channel flow depth given the available gradient and distance downstream of the subject culvert replacement. The overbank and in-channel manning's n roughness coefficient were taken as 0.1 to 0.15 (dense overgrowth) and 0.06 to 0.08 (vegetated channel and fallen trees) respectively.

Toberscanavan Lakes comprises two loughs (upper and lower) and have a combined approximate area of 12.4ha (0.124km^2). The lower lough, which has an area of circa 8ha (0.08km^2), has a maximum surveyed depth (relative to a water level of 29.5mOD) of >11m.

Photo1: Existing 1200mm culvert Outlet

Photo 2: Existing Stream Channel Upstream of N4.

5.2.1.2 Pre-Construction

The existing culvert is 23.5m in longitudinal length, has a gradient of 1 in 65.3 and is 1200mm in diameter which provides an open area of 1.13m^2. Under design flood flows this culvert is inlet controlled and acts as the choke on flow rate discharging downstream. The upstream culvert invert is 29.26mOD, which is 0.36m above the channel bed invert of 28.9mOD. The survey shows the existing channel invert rises to 29.15mOD 55m upstream of the culvert before dropping towards the lake outlet (Lake outlet channel invert of 28.89mOD). The channel between the lake and the culvert is heavily overgrown with dense tree cover and vegetation on its banks and floodplain area.

The annual average flow for the stream catchment is calculated at 0.16cumec and the 95%ile low flow rate is estimated by the EPA hydronet tool to be 0.017 cumec (2.7 l/s/km^2).

The hydraulic model was run in unsteady state for the derived flood flow hydrographs, derived using the IH-124 design peak flows and FSR design hydrograph methods, to demonstrate the attenuation effect of the lower lough storage.

The model plan and calculated hydraulic profiles for the 95%ile low flow and average flow for the pre-construction / existing scenario are included in Figures 22 to 23 below. Figure 23.1 presents the hydrographs at the upstream and downstream end of the modelled reach which demonstrate graphically the attenuation of flows as a result of being routed through the lake and floodplain storage. As can be seen from this figure the inflow hydrograph is significantly reduced and attenuated by the available flood storage in the lakes and adjoining flood plain and the choke effect of the existing 1200mm culvert which limits flow converting it into upstream flood storage.

Figure 22: Pre-construction Scenario Hydraulic Model Plan

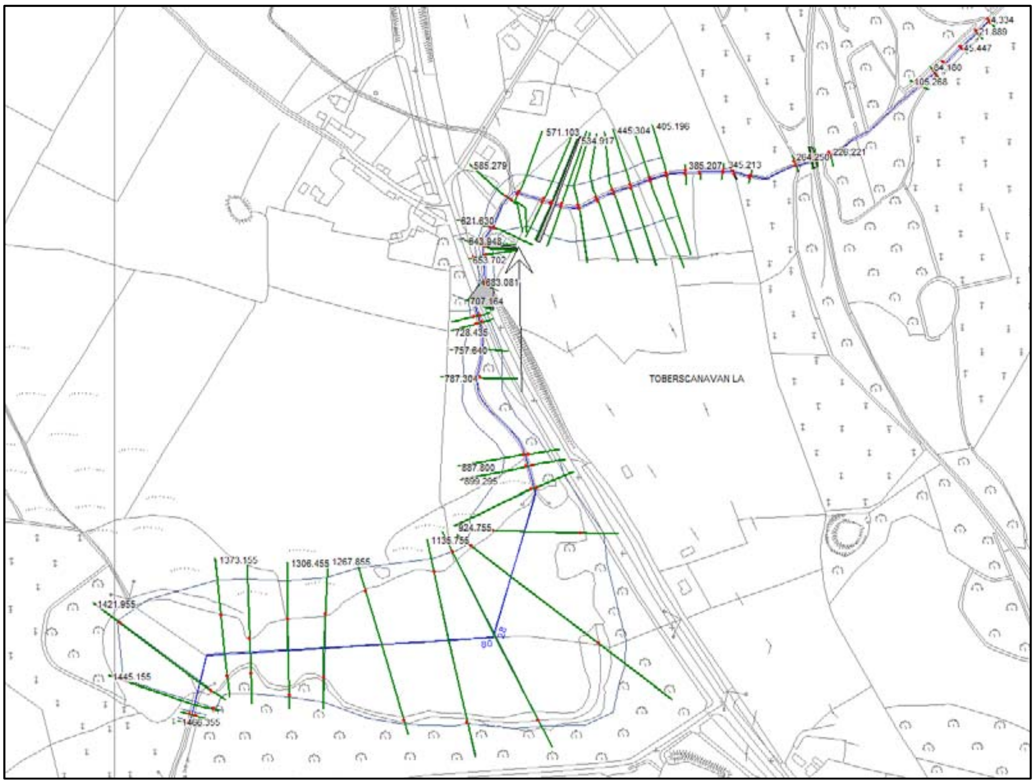


Figure 23: Pre-construction Scenario - Qlow and Qmean flow profiles

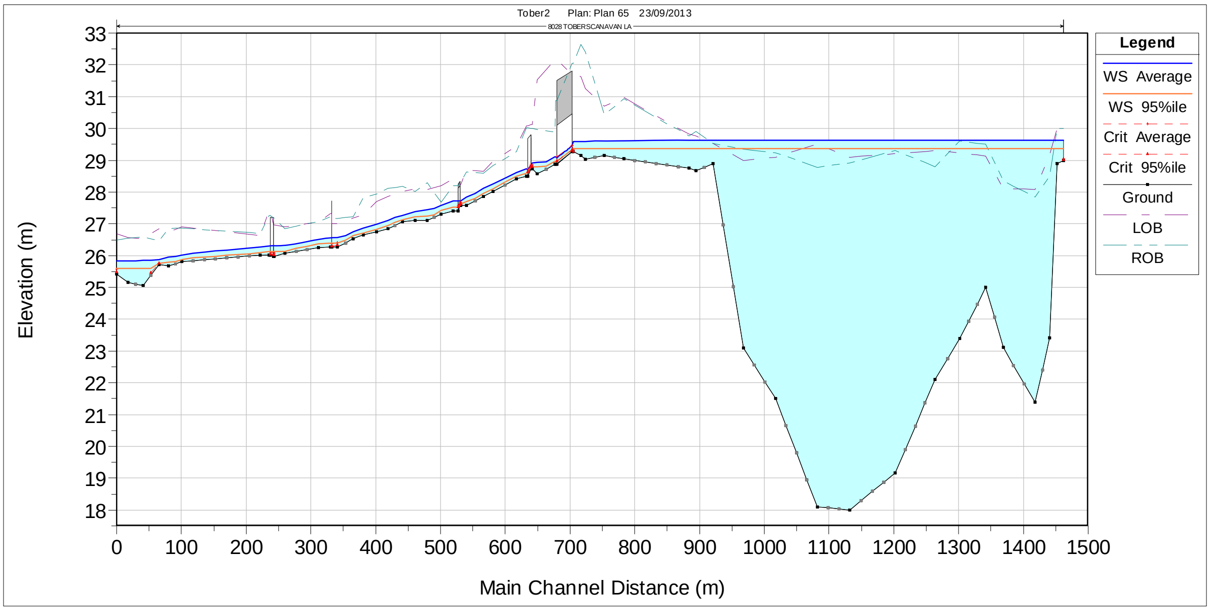
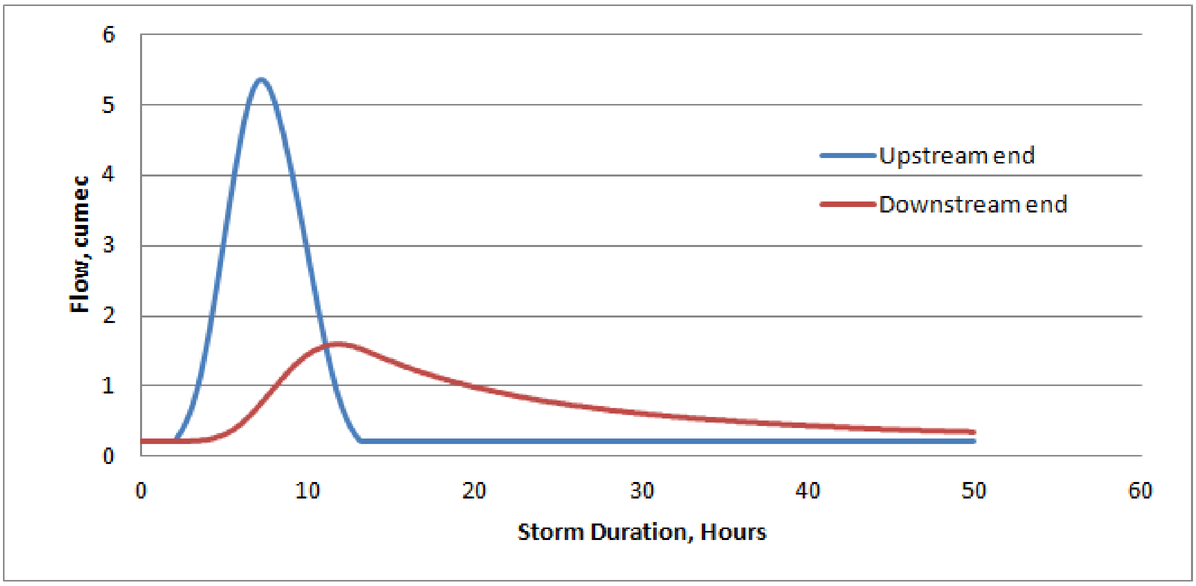


Figure 23.1: Pre-construction Scenario - Comparison of the Upstream and Downstream Hydrographs



The calculated maximum flows at the upstream and downstream end of modelled pre-construction reach for the Q100 and Q1000 events is presented in Table 13.1.

Table 13.1: Pre-construction Upstream and Downstream Max Flows

Location	Q100, cumec	Q1000, cumec
Upstream	5.35	7.11
Downstream	1.61	2.19
Attenuation of Flows	3.74	4.92

The upstream water level for the 95%ile and average flows for the existing and lowered upstream invert is given below in Table 14.

The road was shown to be overtopped when the standard factorial error was applied to the Q_{100} and Q_{1000} design flows. As overtopping of the road at this crossing has not been recorded in the past nor is there evidence of the road ever being threatened the use of the factorial error for additional conservatism is not proposed as the design flows without standard factorial error are considered suitably conservative for this study. Figure 24 and 25 present the pre-construction scenario flood profile for the Q100 and Q1000 events and Table 14 present. The hydraulic model clearly demonstrates that the existing culvert and N4 road embankment cause a considerable afflux under design flood conditions of Q100 and Q1000.

The hydraulic model was run with and without an existing private access culvert that is located 10m downstream of the existing N4 and which will be removed as part of the *Proposed Road Development*. This model run shows that this downstream small restricted culvert (twin rectangular masonry opes) does not increase flood levels upstream of N4.

Figure 24: Pre-construction Scenario (Q100) flood profile

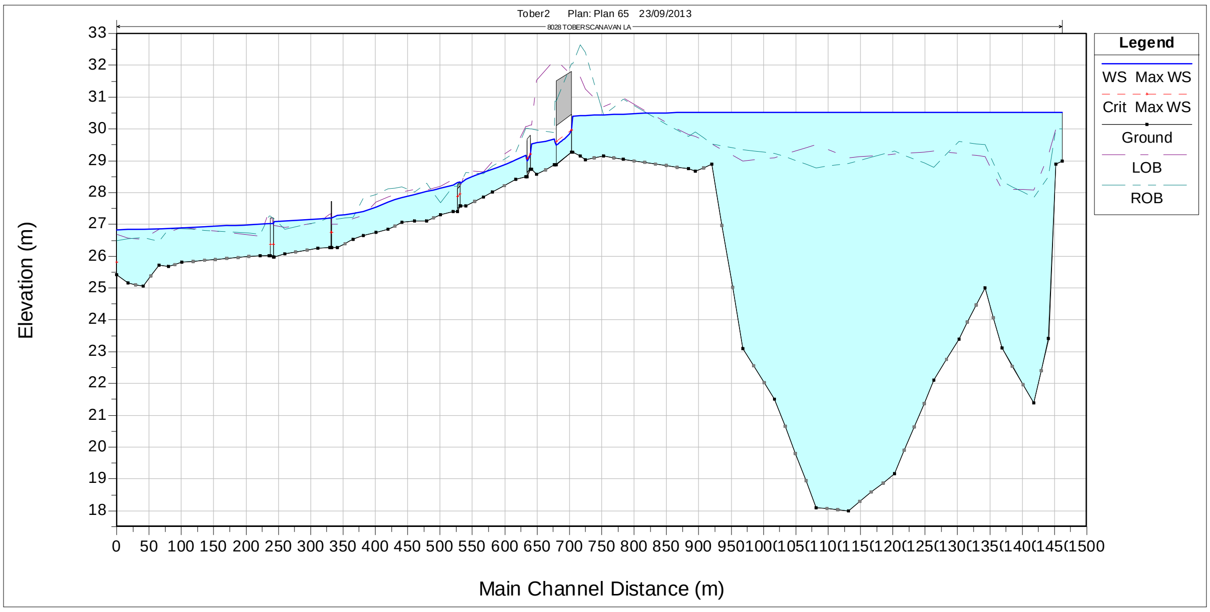


Figure 25: Pre-construction (Q1000) flood profile

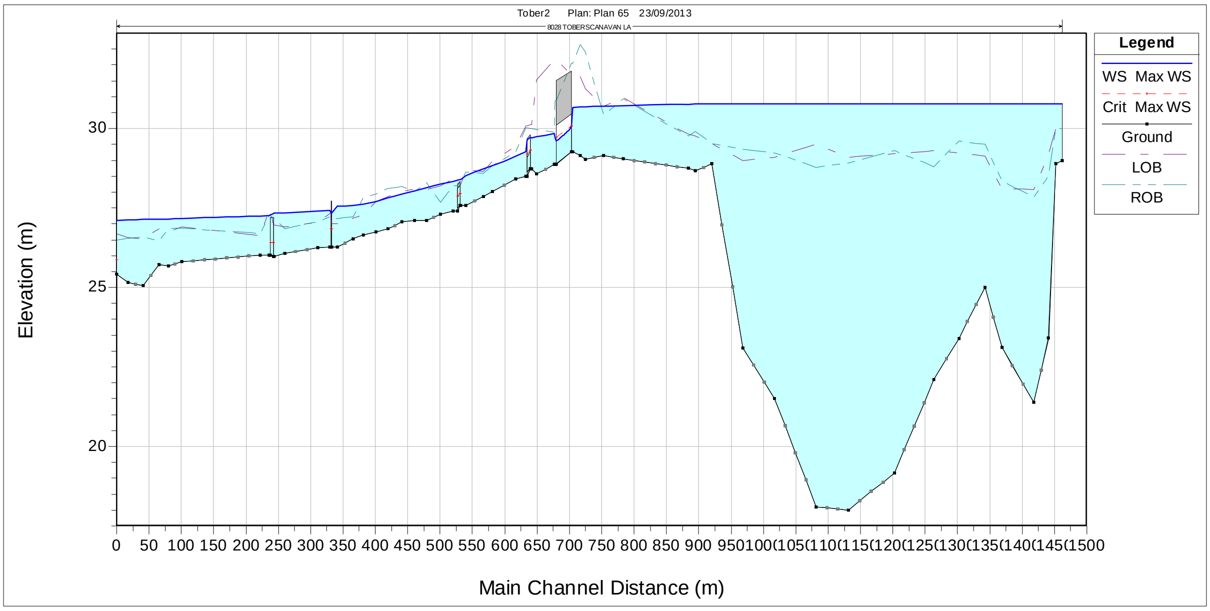


Table 14: Calculated lake levels (Pre-construction Scenario)

Flow	Qlow, mOD	Qmean, mOD	Qbar, mOD	Q100, mOD	Q1000, mOD
Toberscanavan Lakes (lower)	29.37	29.62	30.12	30.52	30.77

5.2.1.3 Post Construction

It is proposed to replace the existing 23.5m long 1200mm dia. pipe culvert at the Markree Demesne Stream crossing with a 62m long 3.0m x 2.0m box and embedded by 300mm (effective height 1.7m) to accommodate the *Proposed Road Development* which involves raising and widening of the existing N4 and the construction of a link road. The culvert will include a mammal light box located 32m downstream from the culvert inlet and will be 5m long. The proposed box culvert's upstream and downstream soffit levels are 30.25 and 29.8mOD (matching the downstream channel levels) respectively. It is also proposed to the construct two storm drainage attenuation ponds on the right bank of the stream downstream of the N4.

The *Proposed Road Development* scenario was simulated under steady constant flow conditions for low and average stream flow events and under unsteady state conditions for the design flood flow hydrographs. The simulation results for the *Proposed Road Development* are presented in Table 15 and the peak flood level profiles for the Q100 and Q1000 are presented in Figures 27 and 28 respectively.

Table 15: Calculated lake levels based (Post Construction Scenario)

Flow	Qlow, mOD	Qmean, mOD	Qbar, mOD	Q100, mOD	Q1000, mOD
Toberscanavan Lakes (lower)	29.27	29.48	30.05	30.42	30.64

Figure 26: Post Construction Scenario Hydraulic Model Plan

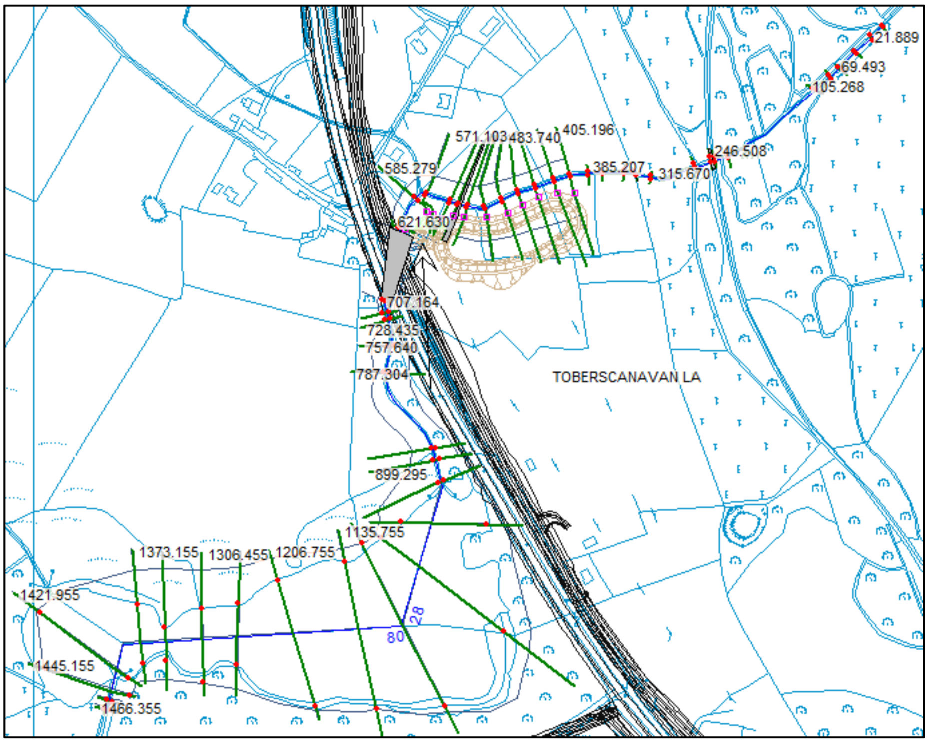


Figure 27: Post Construction Scenario (Q100) flood profile

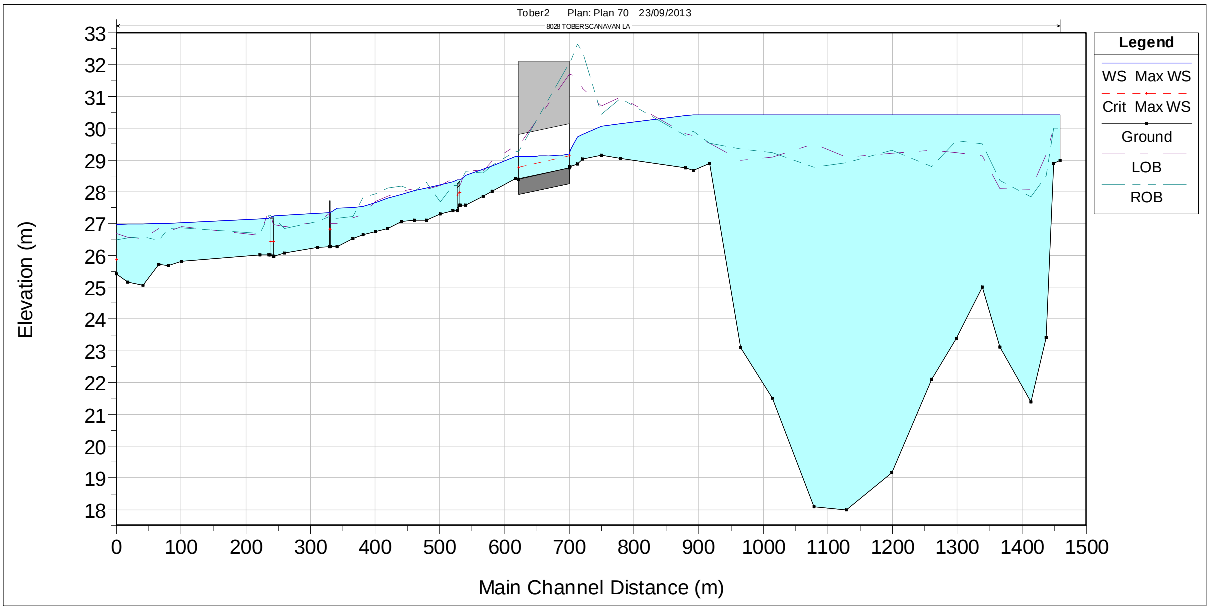
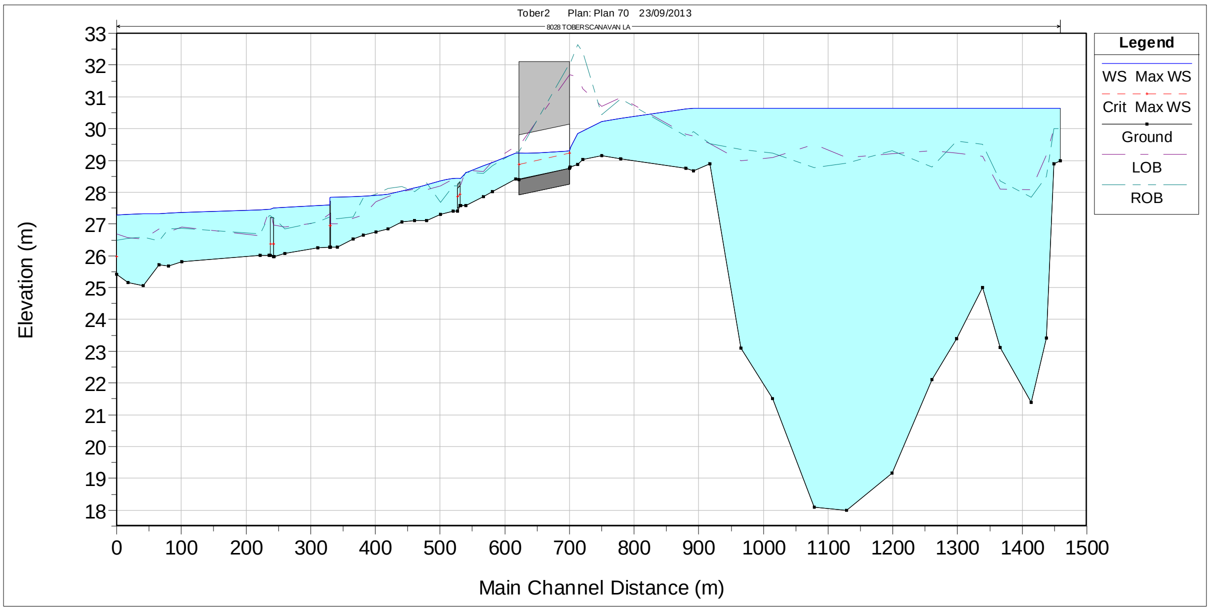


Figure 28 Post Construction Scenario (Q1000) flood profile



5.2.1.4 Discussion

The impact on upstream lake levels under non flood conditions for the proposed culvert reduces the upstream lake level by 0.10m and 0.14m for annual average and 95-percentile low flows respectively. The proposed development reduces the Q100 and Q1000 upstream lake flood levels by 0.10m and 0.13m respectively.

Table 16 below presents the peak Q100 and Q1000 flood flows under the existing and proposed scenarios at a cross-section XS571 located 112m downstream of the Existing N4. This table shows a moderate increase in peak downstream flows as a result of the proposed culvert which have significantly improved flow capacity over the existing culvert.

Table 16: Comparison of downstream maximum flood flows for the existing and proposed scenario

XS571	Q100, cumec	Q1000, cumec
Pre-Construction	1.62	2.22
Post-Construction	2.12	3.05

XS571	Q100, cumec	Q1000, cumec
Difference	*0.50*	*0.83*

This increase in downstream flood flow (i.e. due to improved culvert flow capacity resulting in reduced upstream flood attenuation) increases flood levels downstream but shortens the peak flood duration. Table 17 compares the peak flood level for the existing and proposed scenarios at three locations 112m, 338m and 436m downstream of the existing N4.

Table 17: Comparison of downstream maximum flood levels for the pre and post scenarios

XS	Q100		Q1000	
	Pre mOD	Post mOD	Pre mOD	Post mOD
571	28.63	28.71	28.73	28.83
445	27.84	27.93	27.95	28.03
264	27.10	27.26	27.35	27.52

For the Q100 and Q1000 design flows the proposed works will increase flood levels in the channel length extending 420m downstream of the existing culvert by between 0.08m and 0.17m.

5.2.2 Turnalaydan Stream

5.2.2.1 Introduction

It is proposed to cross the Turnalaydan Stream (Lough Corran outflow) at Knocknagroagh approximately 500m upstream of the existing N4 bridge at Lackagh. The existing river channel at the proposed crossing is typically 15m wide. A hydraulic model has been developed for the Turnalaydan Stream reach extending 140m downstream and 890m upstream of the existing N4 Bridge at Lackagh Bridge giving a total reach length of 1015m long and includes 59 No. cross-sections. The downstream boundary condition for the model is taken as normal flow depth in channel downstream of Lackagh Bridge for a slack gradient (s=0.002). The overbank and in-channel manning's n roughness coefficient were taken as 0.1 and 0.06 respectively.

Photo 3: Lackagh Bridge

Photo 4: Existing Building adjacent to the Turnalaydan Stream at Lackagh Bridge

5.2.2.2 Pre-Construction

Lackagh Bridge comprises twin masonry arches with typical widths and heights of 1.9m (see table 5 above). The existing bridge soffits are 41.0mOD. The pre-construction scenario was simulated under unsteady state flow conditions (Q100+FE and Q1000+FE design flows). The model plan and calculated hydraulic profiles for the pre-construction scenario are included in Figure 30 to 32 below. Figure 32.1 presents the hydrographs at the upstream and downstream end of the modelled reach to demonstrate the pre-construction attenuation of flows as a result of the upstream floodplain storage.

Figure 30: Pre-construction Scenario Hydraulic Model Plan

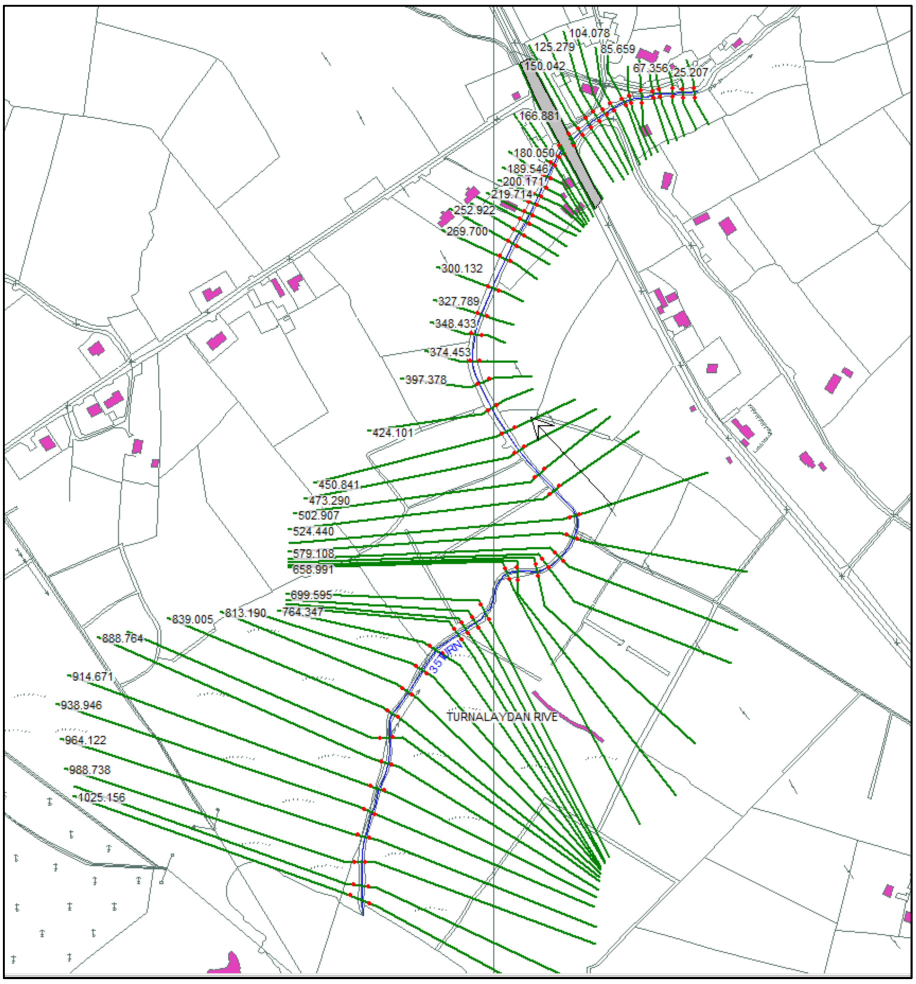


Figure 31: Pre-Construction Scenario (Q100) flood profile

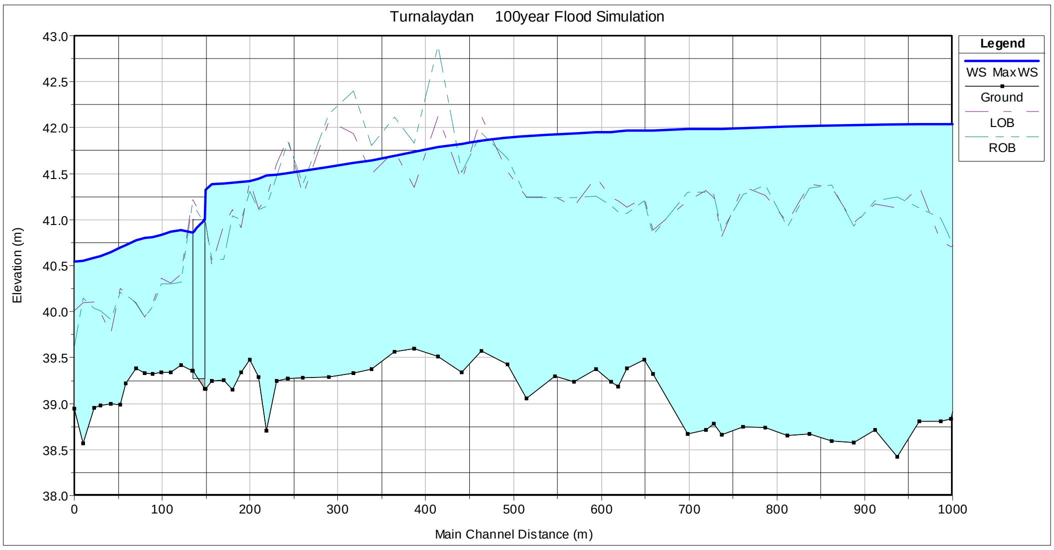
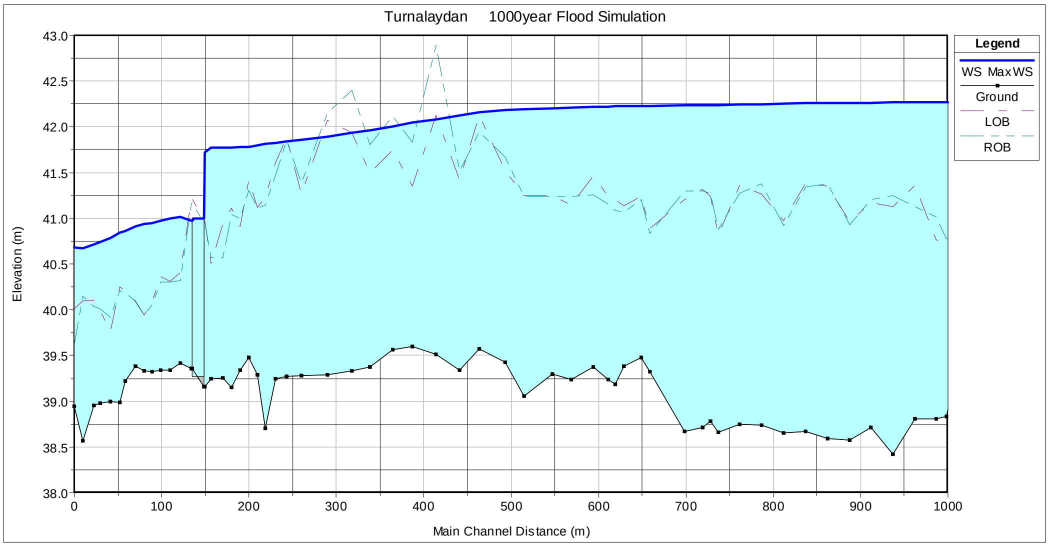
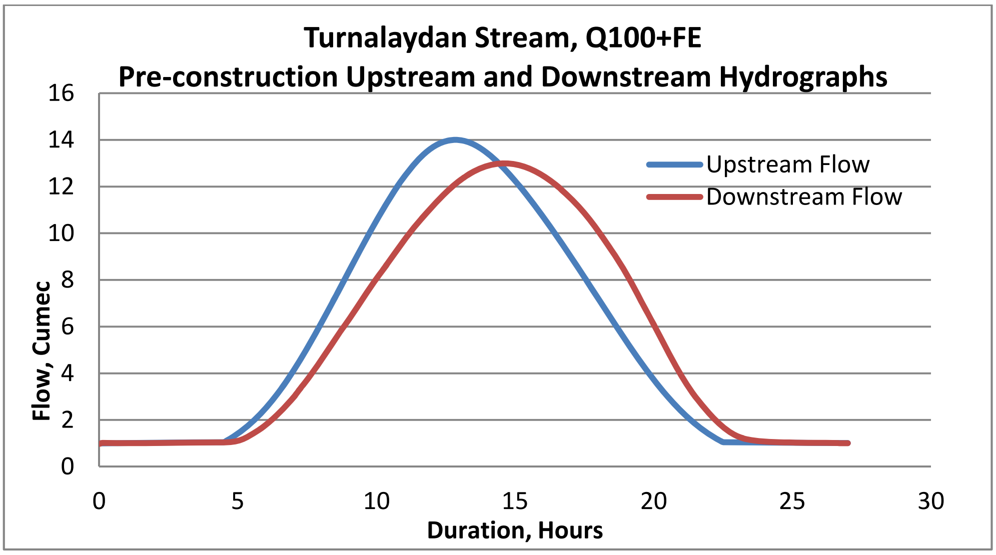


Figure 32: Pre-Construction Scenario (Q1000) flood profile





The model shows that a large afflux occurs at the Lackagh Bridge during flood conditions (c0.5m for Q100 and c0.75m for Q1000 events). The existing properties on the left and right banks immediately upstream of the existing bridge which have finished floor levels of 41.08mOD and 40.57mOD are at high flood risk.

The calculated maximum flows at the upstream and downstream end of modelled pre-construction reach for the Q100 and Q1000 events is presented in Table 18.1.

Table 18.1: Pre-construction Upstream and Downstream Max Flows

Location	Q100+FE, cumec	Q1000+FE, cumec
Upstream	14.0	18.5
Downstream	13.0	16.8
Attenuation of Flows	*1.0*	*1.7*

5.2.2.3 Proposed Scenario

The proposed bridge structure (Cul-14) is a 20m clear span by approximately 20m long bridge deck with a minimum soffit level of 42.55m. Lackagh Bridge has an ope area of circa 6m^2 while the proposed bridge, including the channel, has a capacity of approximately 38m^2. It is proposed to realign the existing channel at the proposed bridge to improve the angle of approach at the crossing.

The proposed channel scenario was simulated under unsteady state flow conditions (Q100+FE and Q1000+FE design flows). The model plan and calculated hydraulic profiles are included in Figure 33 to 35 below:

Figure 33: Post-Construction Hydraulic Model Plan

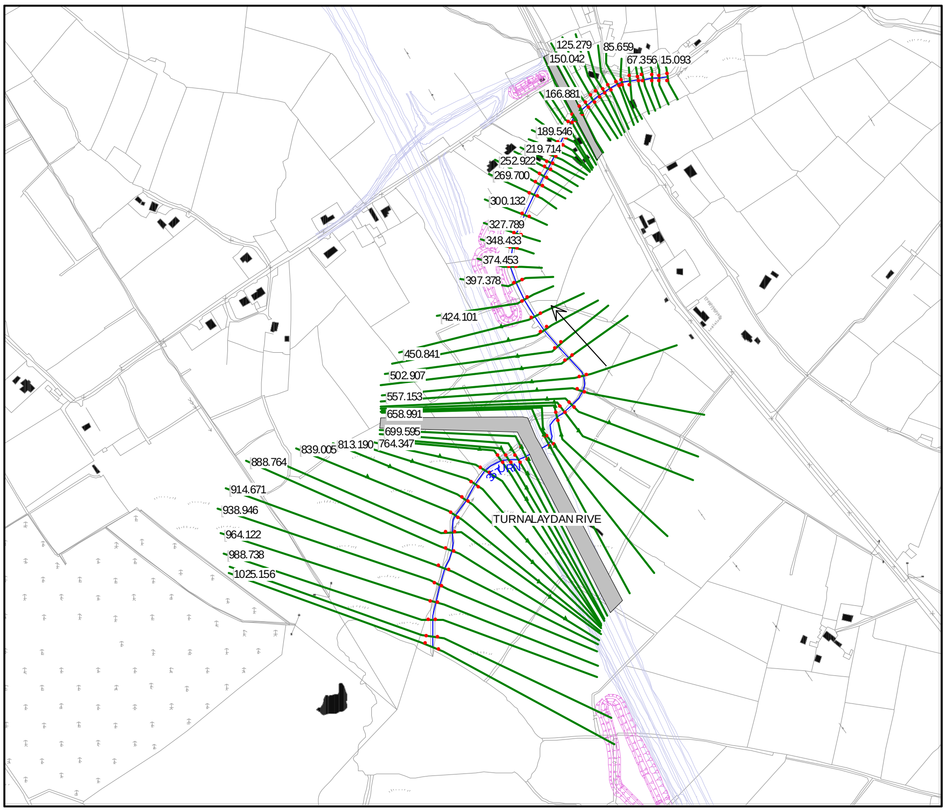


Figure 34: Post Construction Scenario (Q100+FE) flood -profile

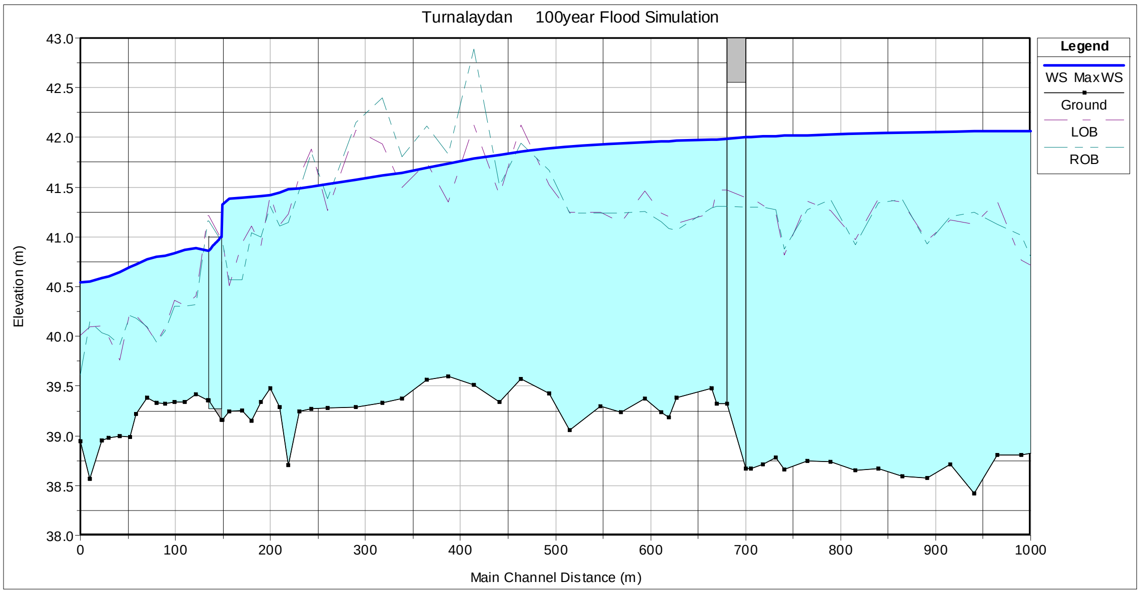


Figure 35: Post Construction Scenario (Q1000+FE) flood profile

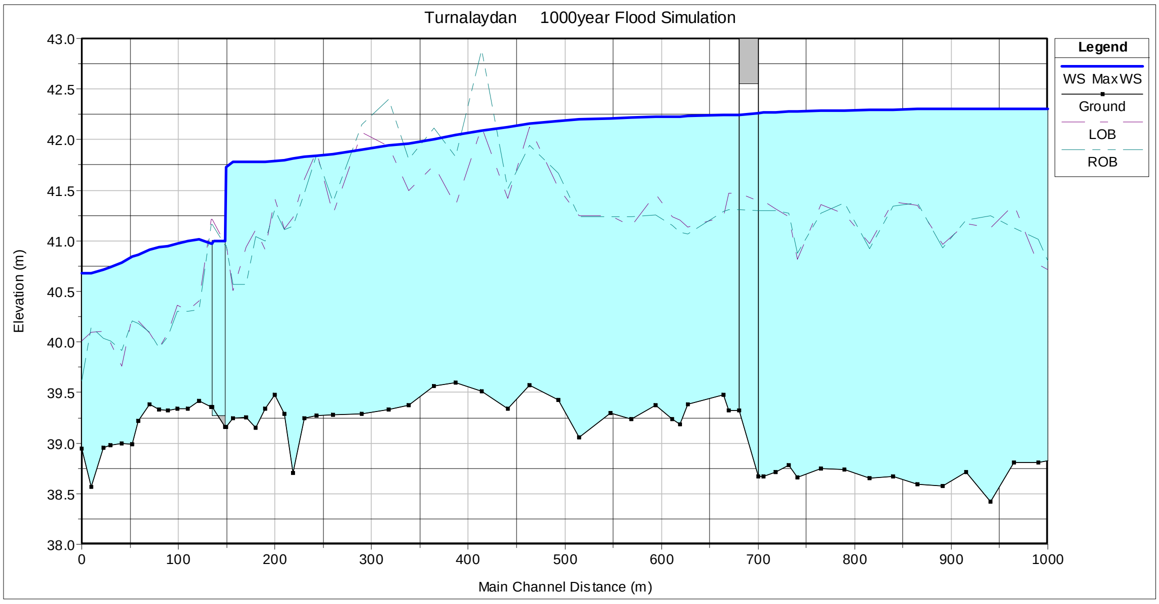
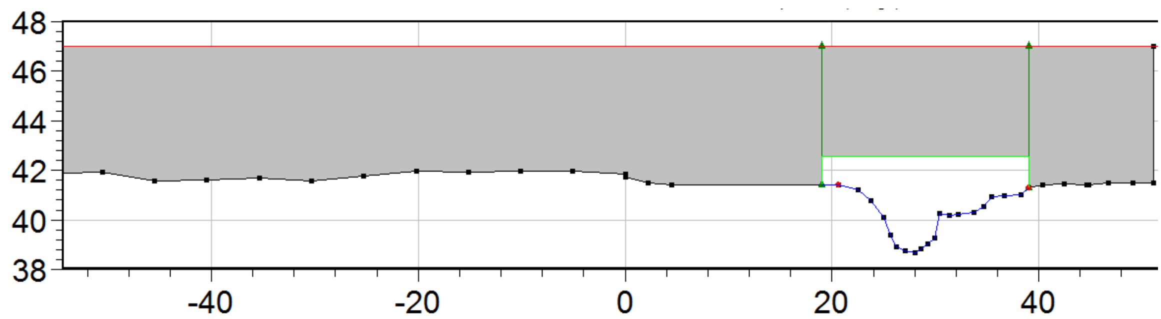


Figure 36: Proposed Bridge Crossing



The existing channel invert levels steps up by 0.8m at the bridge site and thus a minimum water depth of 0.8m will be maintained at the proposed bridge site.

5.2.2.4 Discussion

The proposed road embankment passes through approximately 0.45km of benefitting lands much of which is effective flood plain storage. The proposed bridge structure causes a negligible afflux. In addition to the main channel the road embankment crosses three large open drains at Ch 4+340, Ch 4+385 and Ch4+540. It is recommended that large pipe culverts (1500mm diameter or greater) be provided at these locations to maintain connectivity across the flood plain.

The hydraulic models were simulated for unsteady state flow conditions to determine the impact of the proposed road embankment in the river's flood plain regarding change of flood storage. The maximum flood flow and stage level a number of stations upstream and downstream of the proposed bridge for both scenarios are presented below in Table 20.

The main constriction in this channel reach is the existing twin masonry bridge at Lackagh which causes affluxes of 0.5m and 0.75m for the Q100 and Q100 design flows respectively.

Table 18: Turnalaydan Pre Construction Flows and Stage levels (unsteady state flow conditions) (mOD)

	Q100+FE		Q1000+FE	
Station	Stage, mOD	Flow, cumec	Stage, mOD	Flow, cumec
1025.156	42.04	14.00	42.27	18.48
863.928	42.02	13.15	42.26	17.12
718.006	41.99	12.94	42.24	16.49
628.881	41.97	12.85	42.23	16.28
327.789	41.64	12.74	41.96	15.97
158.316	41.38	12.73	41.77	15.95
73.681	40.77	12.73	40.91	15.95

Table 19: Turnalaydan Post Construction Flows and Stage levels (unsteady state flow conditions) (mOD)

	Q100+FE		Q1000+FE	
Station	Stage, mOD	Flow, cumec	Stage, mOD	Flow, cumec
1025.156	42.06	14.00	42.31	18.48
863.928	42.05	13.13	42.30	16.99
718.006	42.01	12.89	42.27	16.38
628.881	41.97	12.85	42.23	16.30
327.789	41.64	12.74	41.96	16.01
158.316	41.38	12.74	41.78	15.99
73.681	40.77	12.73	40.91	15.99

For the pre-construction scenario the study reach channel and flood plain attenuates the peak flow from 14.0 to 13.0 cumec and from 18.5 to 16.8 cumec at Lackagh Bridge for the Q100+FE and Q1000+FE design flows respectively. The afflux arising from the road embankment and bridge in the flood plain are shown to increase the attenuation in the flood plain, with respect to flow, by approximately 0.65 cumec for the Q1000+FE event.

The proposed bridge and road embankment and the consequential contraction of flow conveyance across the flood plain results in an afflux of 0.02m and 0.04m for the Q100+FE and Q1000+FE design flows respectively.

The proposed attenuation pond site to the east of the road alignment between Ch4+200 and Ch4+350 encroaches on a small area (<10m^2) of the Q1000+FE flood extents and therefore will have negligible impact on flood risk in the Turnalaydan Stream flood plain.

5.2.3 Drumfin River

5.2.3.1 Introduction

It is proposed to cross the Drumfin River at Cloonlurg approximately 430m upstream of Behy Bridge at a location where the existing river channel is 5 to 5.6m wide. A hydraulic model has been developed for the Drumfin River reach extending from 360m downstream of Kilmorgan Bridge to 90m downstream of Behy Bridge on the N4 at Carrowkeel. The total reach length is 1970m long and includes 71 No. cross-sections. The downstream boundary condition for the model is taken as normal flow depth in channel downstream of Behy Bridge for a slack gradient (s=0.001). The overbank and in-channel manning's n roughness coefficient were taken as 0.1 and 0.055.

Photo 5: Behy Bridge

Photo 6: Spring Well Pump Building upstream of Proposed Road Development at Carrownagart

5.2.3.2 Pre-Construction Scenario

Behy Bridge comprises three masonry arches with typical widths and heights of 2.3m and 2.6m (see table 5 above). The existing bridge soffits are 51.5mOD. The pre-construction scenario was simulated for the Q100+FE and Q1000+FE design flows. The model plan and calculated hydraulic profiles for the existing scenario are included in Figure 37 to 39.2 below. Figure 39.3 presents the hydrographs at the upstream and downstream end of the modelled reach to demonstrate the pre-construction attenuation of flows as a result of the upstream floodplain storage.

Figure 37: Pre-construction Hydraulic Model Plan

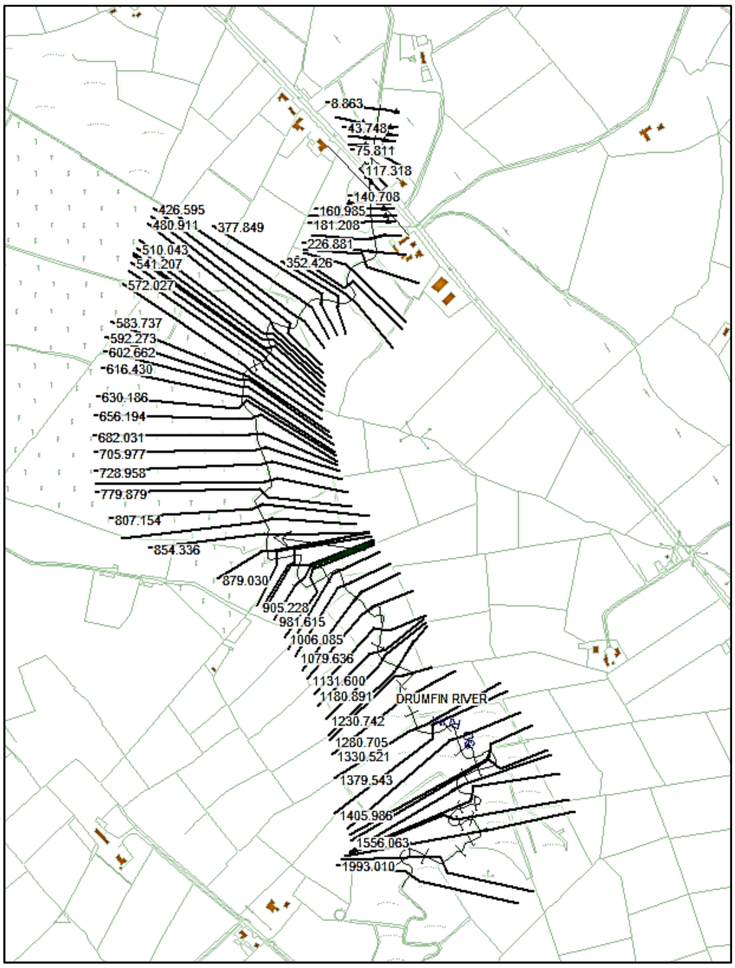


Figure 38: Behy Bridge in Hec-Ras

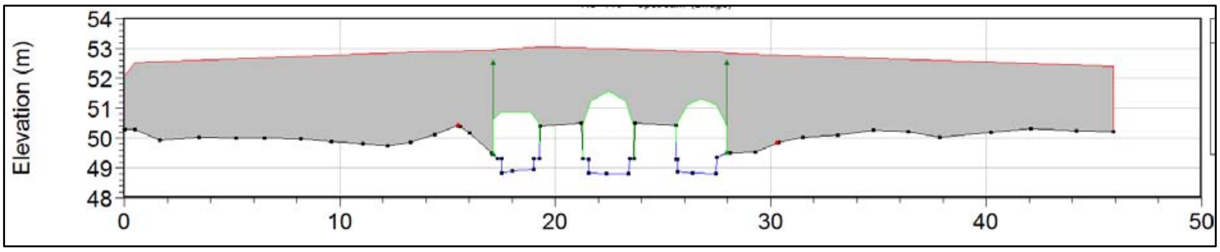


Figure 39.1: Pre-Construction Scenario Q100+FE flood profile

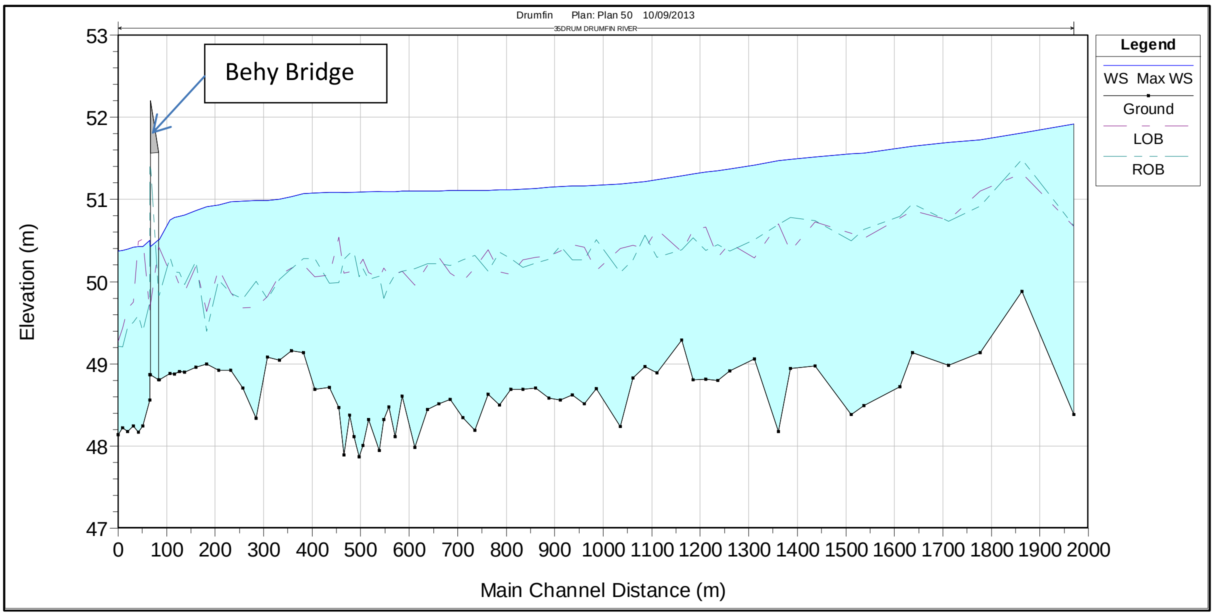
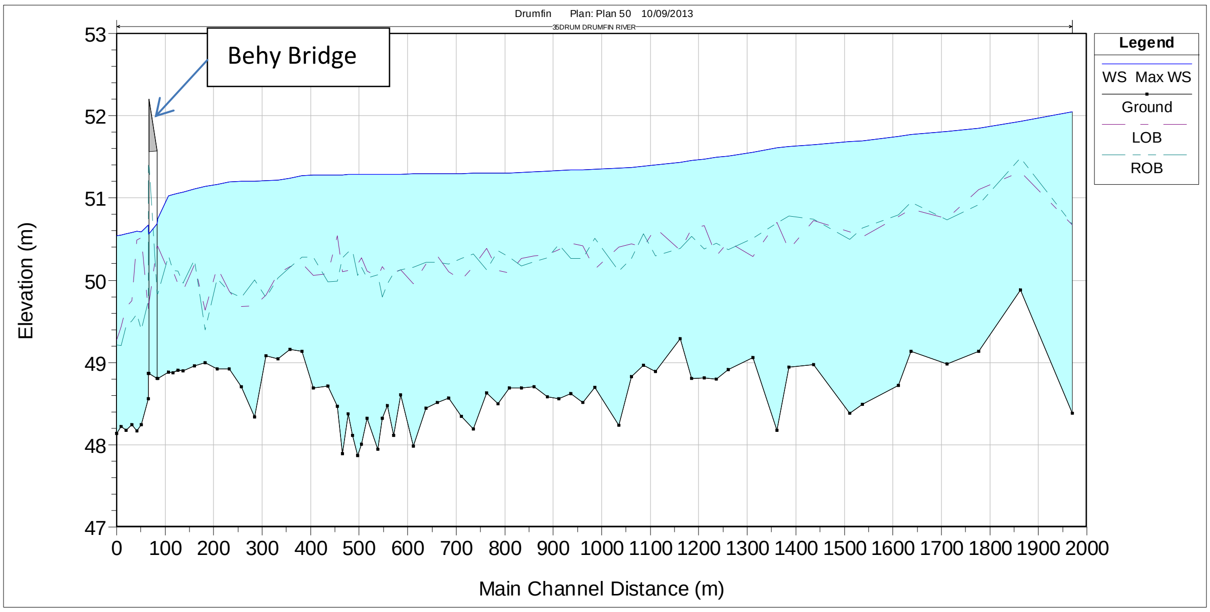
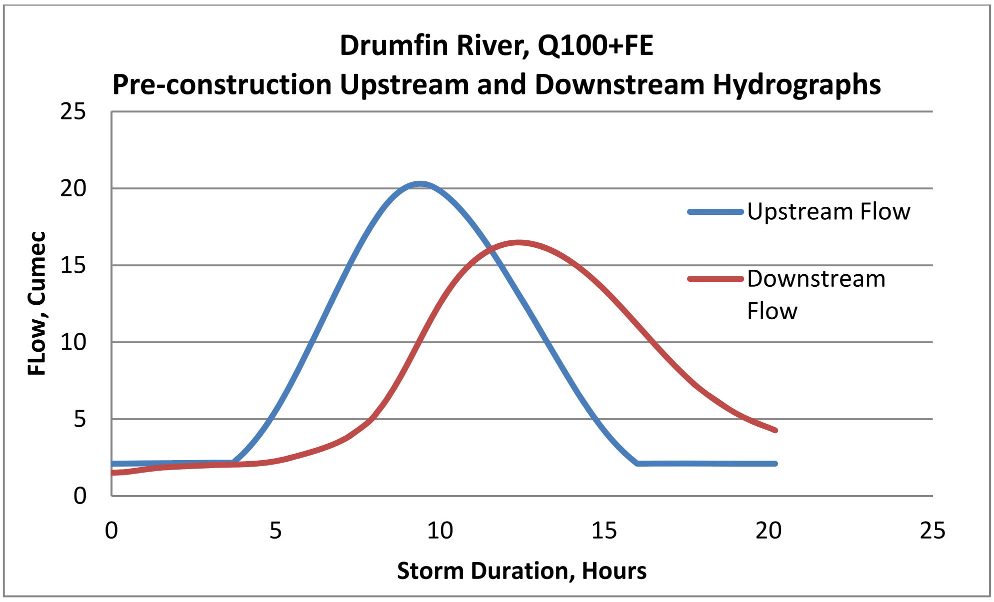


Figure 39.2: Pre-Construction Scenario Q1000+FE flood profile





The model shows that a noticeable afflux occurs at the Behy Bridge during flood conditions. The bridge appears to be located at the downstream end of a raised shelf (probably bedrock) in comparison to the upstream river bed. The river channel drops by a 0.7m step immediately downstream of the bridge.

The calculated maximum flows at the upstream and downstream end of modelled pre-construction reach for the Q100 and Q1000 events is presented in Table 20.1.

Table 20.1: Pre-construction Upstream and Downstream Max Flows

Location	Q100+FE, cumec	Q1000+FE, cumec
Upstream	20.3	26.9
Downstream	16.5	21.3
Attenuation of Flows	*3.8*	*5.6*

5.2.3.3 Post Construction Scenario

The proposed bridge structure (Cul-25) is a 20m clear span by 21.7m long bridge deck with a minimum soffit level of 52.13m. (Note: 52.13mOD allows for a freeboard for the Q1000+FE+CC design flow). Behy bridge has an ope area of circa 14.5m^2 while the proposed bridge, including the channel, has a capacity greater than 40m^2.

The proposed channel scenario was simulated for Q100+FE and Q1000+FE design flows. The model plan and calculated hydraulic profiles are included in Figure 40 to 42 below:

Figure 40: Post Construction Scenario (Q100+FE) flood profile

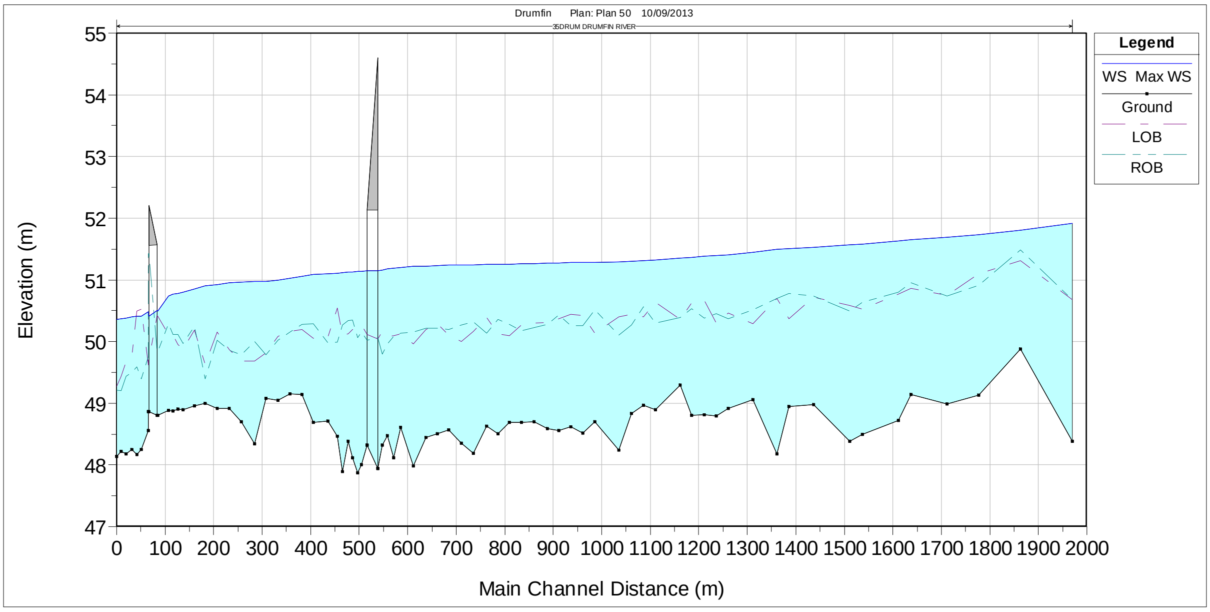


Figure 41: Post Construction Scenario (Q1000+FE) flood profile

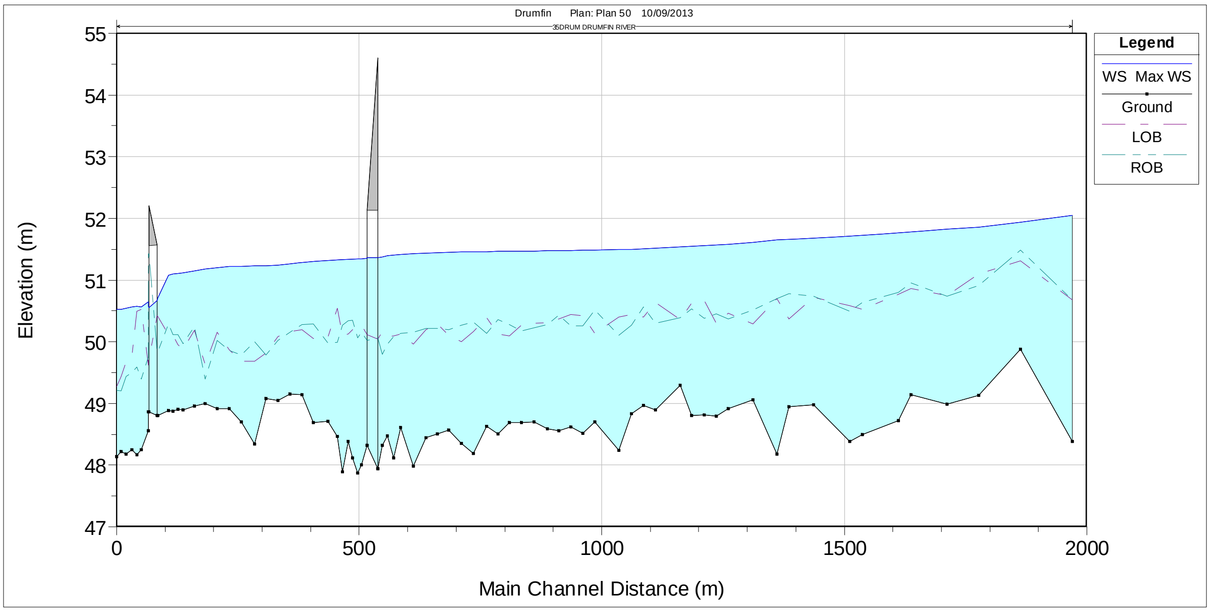
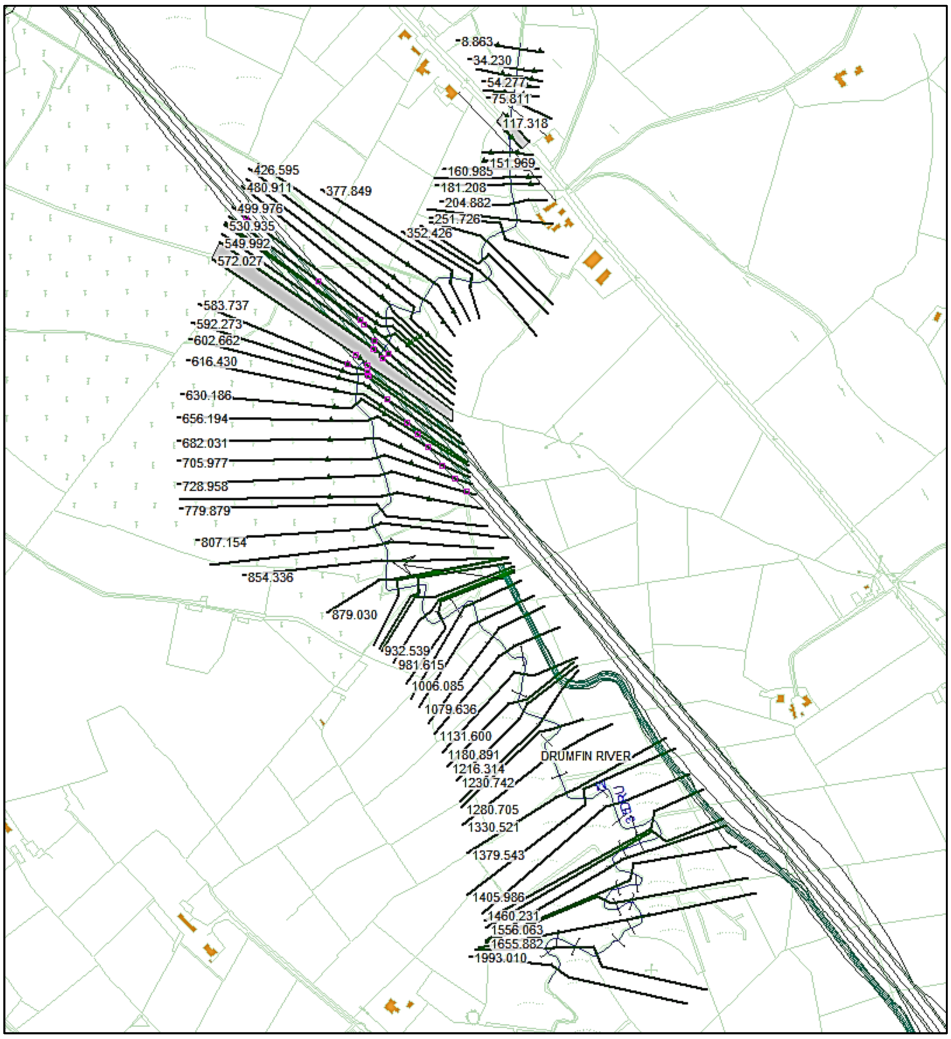
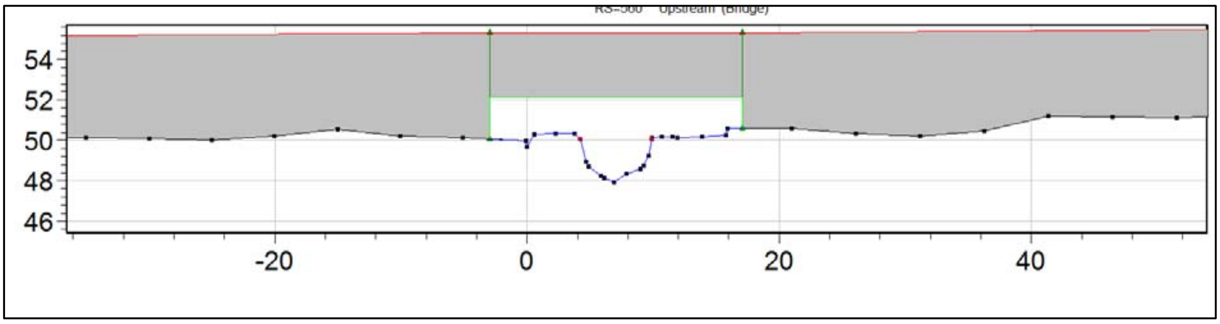


Figure 42: Post-Construction Hydraulic Model Plan (Proposed Scenario)



The proposed bridge is located upstream of a raised shelf of river bed which continues to Behy Bridge. A minimum flow depth of 1.1m will occur at the bridge due to the downstream control.

Figure 43: Proposed Drumfin Bridge in Hec-Ras



5.2.3.4 Discussion

The proposed road embankment passes through approximately 0.8km of benefitting lands much of which is effective flood plain storage. The proposed bridge structure causes an afflux which could be further reduced by increasing the bridge span; however, the magnitude of the afflux considering the inclusion of the standard factorial error, is minor and the benefit of widening the span would be negligible. In addition to the main channel the road embankment crosses three large open drains at Ch 6+880, Ch 7+210 and Ch7+460. It is recommended that large pipe culverts (1500mm diameter or greater) be provided at these locations with inverts matching the existing channel inverts to maintain connectivity across the flood plain.

The hydraulic models were simulated for unsteady state flow conditions to determine the impact of the proposed road embankment in the river's flood plain regarding change of flood storage. The maximum flood flow and stage level a number of stations upstream and downstream of the proposed bridge for both scenarios are presented below in Table 20:

Table 20.2: Drumfin Pre-Construction Scenario Flows and Stage levels (mOD)

	Q100+FE		Q1000+FE	
Station	Stage, mOD	Flow, cumec	Stage, mOD	Flow, cumec
1993.1	51.92	20.3	52.05	26.90
1180.9	51.29	19.43	51.44	25.75
602.7	51.10	17.05	51.29	22.22
592.3	51.09	16.99	51.29	22.14
521.8	51.09	16.78	51.28	21.80
140.7	50.75	16.49	51.02	21.34
43.7	50.39	16.49	50.56	21.34

Table 21: Drumfin Post Construction Scenario Flows and Stage (mOD)

	Q100+FE		Q1000+FE	
Station	Stage, mOD	Flow, cumec	Stage, mOD	Flow, cumec
1993.1	51.92	20.3	52.05	26.90
1180.9	51.35	19.09	51.54	25.09
602.7	51.19	16.52	51.41	21.44
592.27	51.18	16.49	51.39	21.39
521.8	51.13	16.44	51.34	21.28
140.7	50.74	16.16	51.08	20.62
43.7	50.38	16.16	50.54	20.62

For the pre-construction scenario the study reach channel and flood plain attenuates the peak flow from 20.3 to 16.5 cumec (3.8 cumec difference) and from 26.9 to 21.3 cumec (5.6 cumec difference) at Behy Bridge for the Q100+FE and Q1000+FE design flows respectively. This simulation demonstrates the relevance of upstream floodplain storage at Behy Bridge. The afflux from the proposed bridge and road embankment causes an increase in upstream flood plain storage and consequently further attenuates peak flows by the order of 0.5cumec at the new bridge (Q100+FE) and 0.3 cumec at Behy Bridge leading to a slight reduction in flood risk downstream and also therefore a slight increase in flood risk upstream of the proposed road.

The proposed bridge and road embankment and the consequential contraction of flow conveyance across the flood plain results in an afflux of 0.09m for the Q100+FE design flow and 0.14m for the Q1000+FE. The inclusion of flood plain culverts has been shown to reduce the afflux to 0.05m and 0.09m for the design flood flows.

The proposed attenuation pond site to the south of the road alignment between Ch7+675 and Ch7+850 lies above the Q1000+FE flood level and therefore will have negligible impact on flood risk in the Drumfin River flood plain.

5.2.4 **Springfield Stream**

5.2.4.1 Introduction

The proposed Springfield Stream (Loughymeenaghan outflow) works at Tawnagh involve realignment of an existing 173m length of channel with a 207m diversion to accommodate construction of two box culverts one each under the main road embankment and a downstream land access road (Cul-39 and Cul-40). A hydraulic model was developed for the Springfield Stream reach extending from immediately downstream of Ardloy Bridge to the inlet to the swallow hole / pond feature located at Tawnagh, for a total length of 757m and involving 28 No. cross-sections (68 to 96). The existing channel crosses under a land access bridge towards the downstream end of the model reach (0.9m diameter pipe culvert). The downstream boundary condition for the model is taken as the lake at Tawnagh being in flood with a top water level of 59.9mOD (overly conservative but appropriate for this study). The overbank and in-channel manning's n roughness coefficient were taken as 0.1 (suitably conservative).

Photo 7: Proposed Stream Crossing Location at Springfield with open drains on both sides of the road.

Photo 8: Land Access Culvert Crossing over Springfield Stream downstream of the proposed road.

5.2.4.2 Pre-Construction Scenario

The pre-construction scenario was simulated under steady state flow conditions (Q100+FE and Q1000+FE design flows). The model plan and calculated hydraulic profiles are presented below in Figure 44 and 45.

Figure 44: Pre-Construction Scenario Hydraulic Model Plan

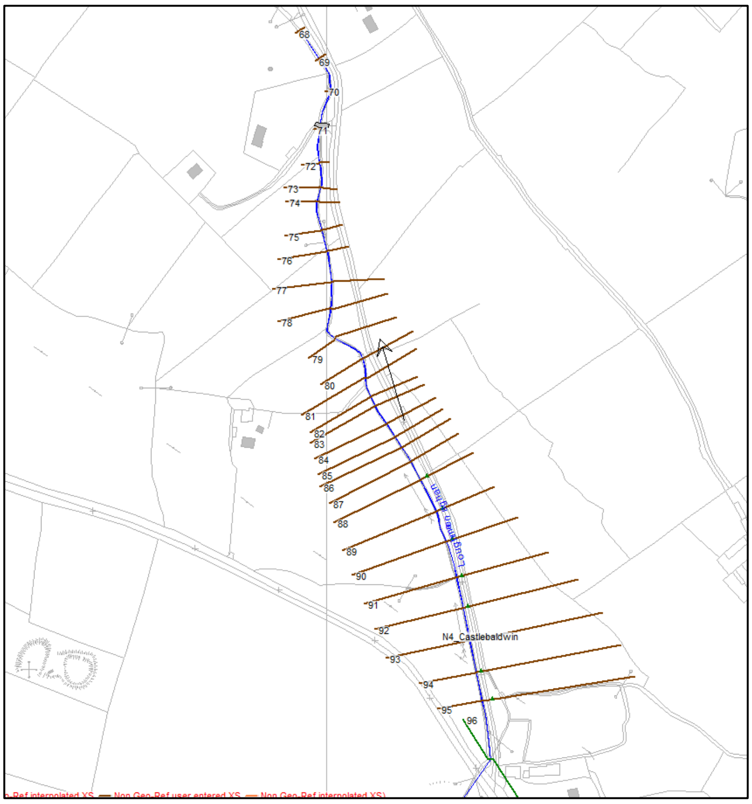
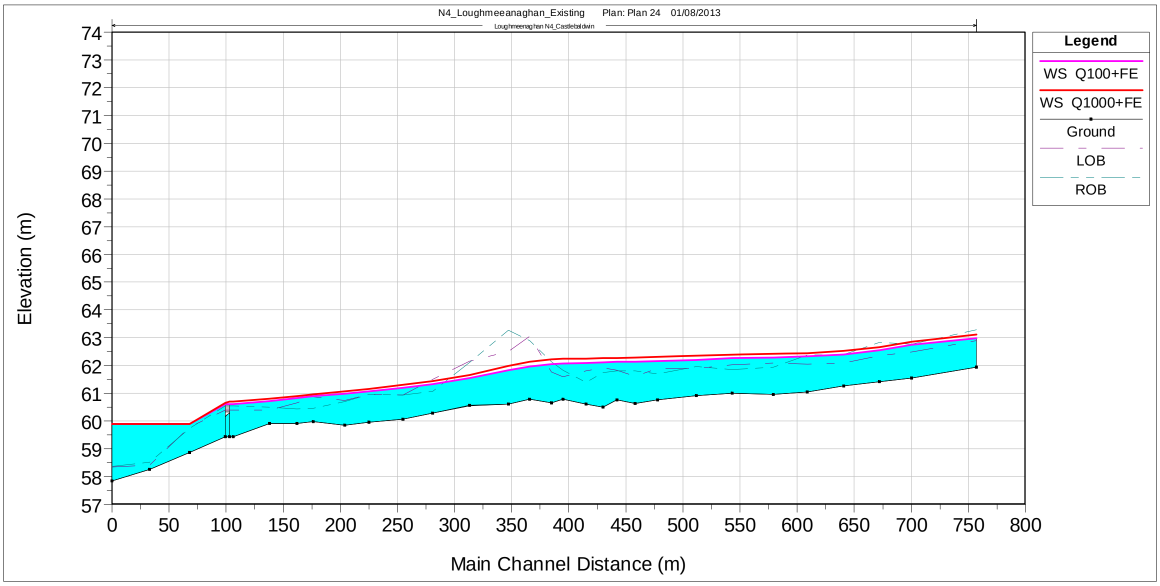


Figure 45: Pre-Construction Scenario (Q100+FE and Q1000+FE) flood profiles



5.2.4.3 Post Construction Scenario

The proposed culvert structures (Cul-39 and 40) are 2.44m x2.13m x 38.2m and 2.74m x2.13m x 7m (H:W:L) respectively. The proposed culverts dimensions include an embedment of 0.5m giving a maximum ope height 1.63m. The existing Ardloy bridge has an ope area of circa 2.0m^2 whilst the proposed culverts have effective ope areas of 4.0m^2 and 4.5m^2 respectively.

The proposed channel scenario was simulated under steady state flow conditions (Q100+FE and Q1000+FE design flows). The proposed model plan and calculated hydraulic profiles are presented in Figure 46 and 47 below:

Figure 46: Post Construction Scenario Hydraulic Model Plan

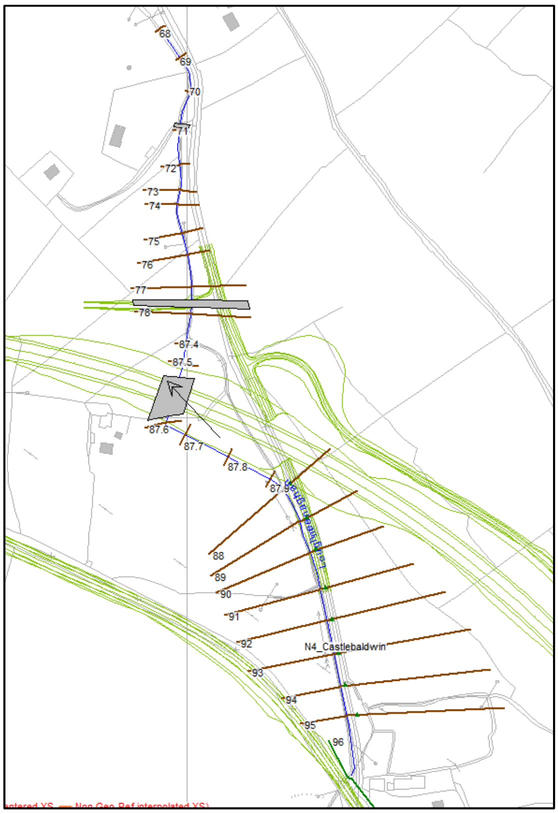
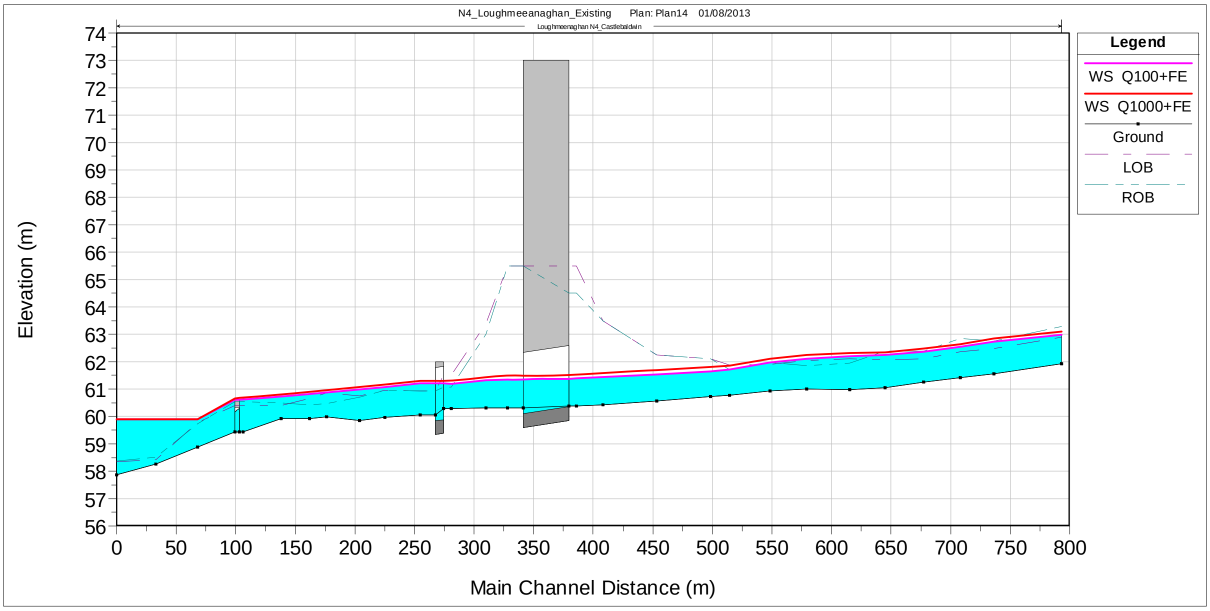


Figure 47: Post-Construction Scenario (Q100+FE and Q1000+FE)



5.2.4.4 Conclusion

Table 22 presents the calculated flood levels at the upstream end of the reach (XS-96) and upstream of the proposed channel diversion and culverts (XS-88).

Table 22: Springfield Stream Pre and Post Construction Scenario Flood levels (mOD)

Design Flow: Q100+FE		
XS	Pre-Construction Scenario	Post Construction Scenario
96	62.99	62.99
88	62.16	61.72
Design Flow: Q1000+FE		
XS	Pre-Construction Scenario	Post Construction Scenario
96	63.11	63.11
88	62.31	61.87

The proposed channel re-alignment passes through an area of high ground. The section of channel being by-passed appears to have a slacker gradient and less channel ope capacity in comparison to the proposed scenario (bed width 3m with steep side slopes 1:1). Comparison of the hydraulic models concludes that the post-construction scenario in fact reduces flood levels locally upstream of the proposed channel realignment.

The existing 900mm diameter culvert downstream of the proposed road was shown to be overtopped for both design floods; however, the associated afflux does not extend upstream to the proposed road.

The hydraulic models were simulated for unsteady state flow conditions to determine the impact of the proposed road embankment in the stream's flood plain regarding loss of flood storage. The maximum flood flow and stage level at the upstream and downstream end of the reach (XS-96 and XS-71) for both scenarios are presented below in Table 23:

Table 23: Springfield Stream Pre and Post Construction Scenario Flood levels (unsteady state flow conditions) (mOD)

Scenario	Design Flood	XS	Stage, mOD	Flow, cumec
Pre-Construction	Q100+FE	96	63.00	1.38
		71	60.60	1.36
	Q1000+FE	96	63.11	1.84
		71	60.69	1.80
Post-Construction	Q100+FE	96	63.00	1.38
		71	60.61	1.37
	Q1000+FE	96	63.11	1.84
		71	60.69	1.82

The model simulations for both design floods (unsteady state) conclude that there is a negligible impact on both stage and design flows therefore confirming that the proposed road will have a negligible impact on flood risk at Tawnagh. The degree of afflux due to the diversion will be dependent on the proposed diversion channel's capacity.

The proposed road embankment passes through an area with little overbank flooding and conveyance and therefore has negligible impact on flood plain storage. The model confirms (by way of comparison between the upstream and downstream flows) that the road will have negligible impact on flood storage.

The proposed road embankment at Tawnagh is located above the 62mOD contour and therefore lies well out of on the 'Lake at Tawnagh' flood plain.

The proposed attenuation pond site to the north of the road alignment between Ch10+585 and Ch10+690 lies above the Q1000+FE flood level and therefore will have negligible impact on flood risk in the Springfield Stream flood plain.

5.2.5 Lissycoyne Stream

5.2.5.1 Introduction

The proposed Lissycoyne Stream works at Sheerevagh involves realignment of an existing 108m length of channel with a 141m diversion to accommodate construction of two box culverts one each under the main road embankment and an upstream land access road (Cul-47 and Cul-46). A hydraulic model was developed for the Lissycoyne Stream reach extending from downstream of existing N4 culvert crossings (1 No. 900 and 2 No. 900) to a land access road culvert (Mr. Clarke'), for a total length of 685m and involving 24 No. cross-sections (177 to 201). The downstream boundary condition for the model is taken as normal channel flow depth downstream of Clark's culvert. The overbank and in-channel manning's n roughness coefficient were taken as 0.08 and 0.06 respectively.

Photo 9: Location of the proposed Lissycoyne Stream Crossing and the existing Bog Road crossing

5.2.5.2 Pre-Construction Scenario

The existing channel crosses under two culverts in the study reach namely the Bog Road culvert (a 0.625m wide by 1m high culvert located 30m upstream of the new road) and Clark's culvert at the downstream end of the model reach (900mm diameter pipe culvert).

The existing channel scenario was simulated under steady state flow conditions (Qbar, Q100 and Q1000 design flows). The calculated hydraulic profiles and existing model plan are presented below in Figure 48 and 49:

Figure 48: Pre-Construction Scenario (Qbar, Q100 and Q1000) flood profile

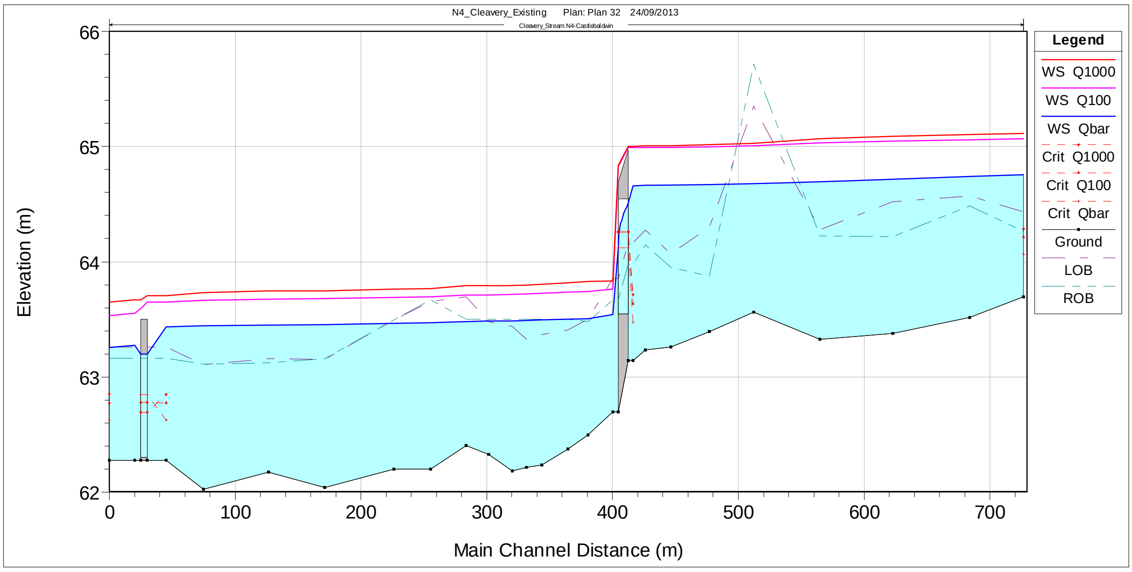
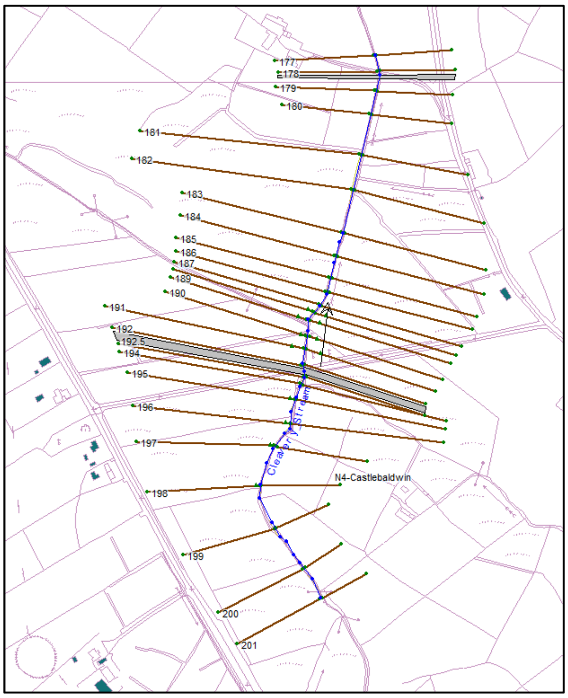


Figure 49: Pre-Construction Scenario Hydraulic Model Plan



5.2.5.3 Post Construction Scenario

The proposed culvert structures (Cul-46 and 47) are both 2.44m x 2.44m. The proposed culverts dimensions include an embedment of 0.5m giving a maximum ope height 1.94m. The existing Bog Road bridge has an ope area of less than $1.0m^2$ whilst the proposed culverts have effective ope areas of $4.73m^2$.

The proposed channel scenario was simulated under steady state flow conditions (Qbar, Q100 and Q1000 design flows). The calculated hydraulic profiles and the proposed model plan are presented below in Figures 50 and 51:

Figure 50: Post Construction Scenario (Qbar, Q100 and Q1000)

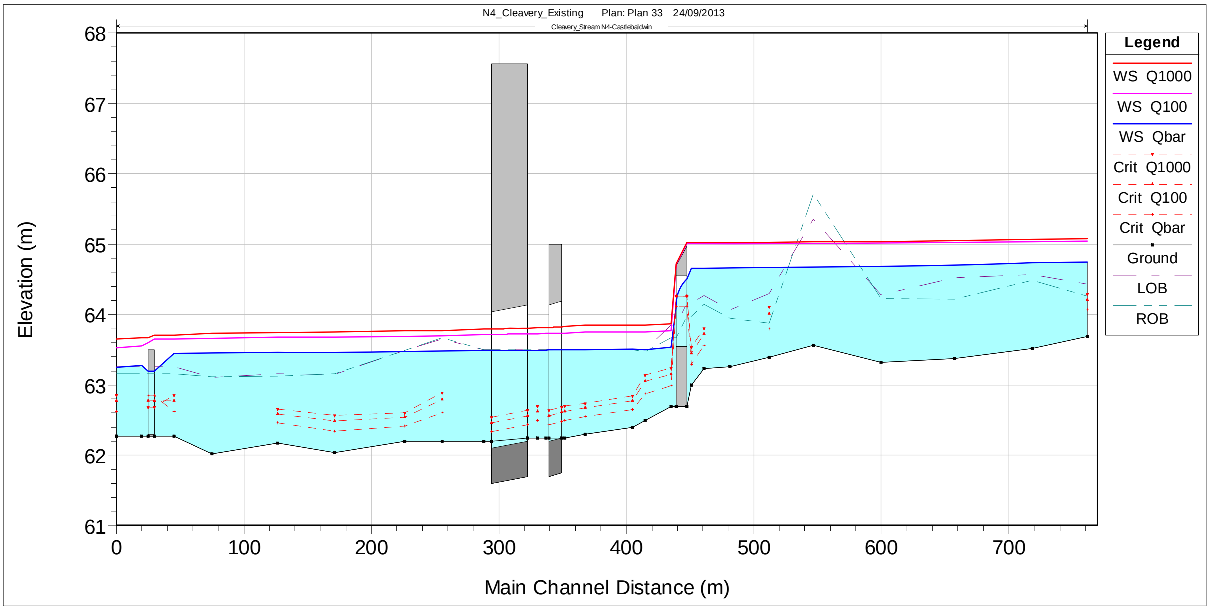
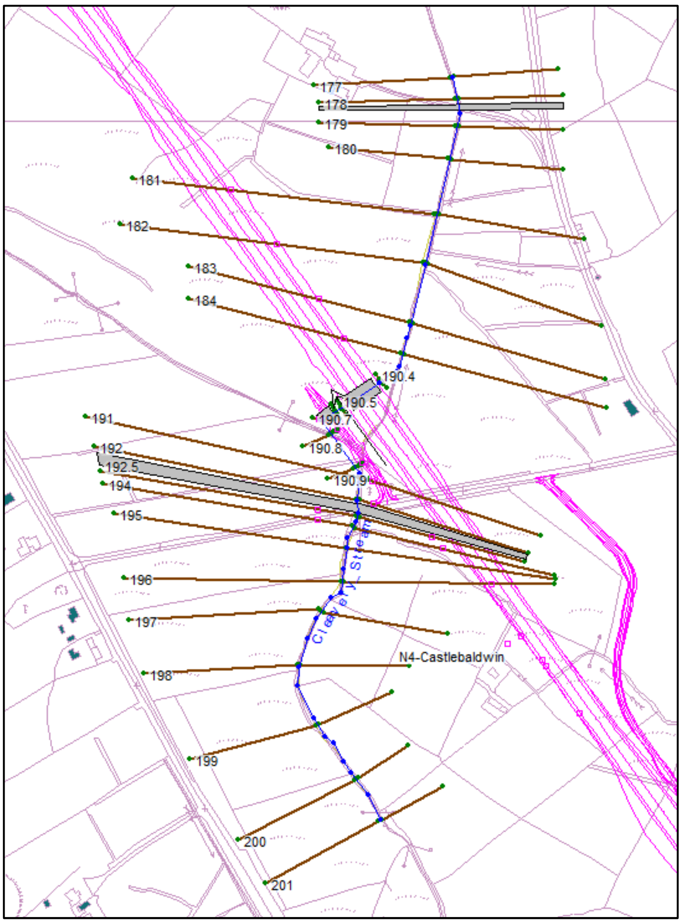


Figure 51: Post Construction Scenario Hydraulic Model Plan



5.2.5.4 Conclusion

The model clearly demonstrates for the design flows that the existing Bog Road culvert is overtopped and causes a considerable afflux during peak flood events. Significant dampening / attenuation of flood flows would be expected in the upstream catchment and consequently calculated design flows are probably overly conservative. The design flows chosen do not include a standard factorial error.

Table 24 presents the calculated flood levels at the upstream end of the reach (XS-201) and upstream of the proposed channel diversion and culverts (XS-191).

Table 24: Lissycoyne Stream Pre and Post Construction Scenario Flood levels (steady state flow conditions) (mOD)

Design Flow: Q100		
XS	Pre-Construction Scenario	Post Construction Scenario
201	65.05	65.08
191	63.74	63.83
Design Flow: Q1000		
XS	Pre-Construction Scenario	Post Construction Scenario
201	65.05	65.08
191	63.75	63.85

Comparison of the steady state hydraulic models concludes that the proposed scenario will cause a small localised impact on flood levels between the new road and the Bog Road of 2cm during a Q1000 flood event. Replacement of the Bog Road culvert would greatly decrease upstream flood risk.

The hydraulic models were simulated for unsteady state flow conditions to determine the impact of the proposed road embankment in the stream's flood plain regarding loss of flood storage. The maximum flood flow and stage level upstream of the proposed road embankment (i.e. downstream of the Bog Road) and at the downstream end of the reach (XS-191 and XS-179) for both scenarios are presented below:

Table 25: Lissycoyne Stream Pre and Post Construction Scenario Flood levels (unsteady state flow conditions) (mOD)

Scenario	Design Flood	XS	Stage, mOD	Flow, cumec
Pre-Construction	Q100	191	63.70	1.51
		179	63.60	1.37
	Q1000	191	63.82	2.20
		179	63.69	2.04
Post-Construction	Q100	191	63.71	1.42
		179	63.61	1.41
	Q1000	191	63.86	2.18
		179	63.70	2.13

The model simulations for both design floods (unsteady state) conclude that proposed culverts and road embankment across the study area's flood plain will reduce effective storage by 0.02cumec and 0.09cumec for the peak Q100 and Q1000 design flows. The simulation also confirms the moderate increase in upstream flood levels of 1cm and 4cm respectively for the peak Q100 and Q1000 design flows. The area affected by the increase in flood levels extends between the Bog Road and the new road embankment.

Replacement of the existing Bog Road culvert would be expected to significantly reduce flood risk upstream of the Bog Road, however, it may lead to a moderate increase in peak flood level downstream of the proposed road.

5.2.6 **Tributary of the Drumderry Stream**

5.2.6.1 Introduction

The proposed works at Tributary of the Drumderry Stream to the south west of Castlebaldwin involves the construction of two box culverts one each under the main road embankment and at a downstream land access road (Cul-61 and Cul-62). A hydraulic model was developed for the Tributary of the Drumderry Stream reach extending from 150m upstream and 185m downstream of the existing N4-crossing crossings and involving 17 No. cross-sections (100 to 84). The downstream boundary condition for the model is taken as normal channel flow depth. The overbank and in-channel manning's n roughness coefficient were taken as 0.07 and 0.05 respectively. The existing culvert has a steep gradient of 1 in 33.

As the main source of Tributary of the Drumderry Stream is from springs (including Tobermahon) the flood duration would be expected to be elongated with constant high flows rather than a definite peak flow. The existing and proposed channel scenario was simulated under steady state flow conditions i.e. Qbar, Q100 and Q1000 design flows (Note: simulations using design flows with standard factorial errors were undertaken. These simulations showed the existing N4 being overtopped for flood events of less than the Q100. There are no reports of overtopping events at this culvert site.)

Photo 10: Location of the existing N4- Tributary of the Drumderry Stream Crossing

5.2.6.2 Pre-Construction Scenario

The existing N4 culvert is a 32m long 900mm diameter pipe culvert. Figures 52 and 53 below present the Hec-Ras plan and flood profile for the existing scenario. The model shows that the N4 overtops for the Q1000 flood event while the Q100 is conveyed under the road with a significant localised afflux.

Figure 52: Pre-Construction Scenario (Qbar, Q100 and Q1000)

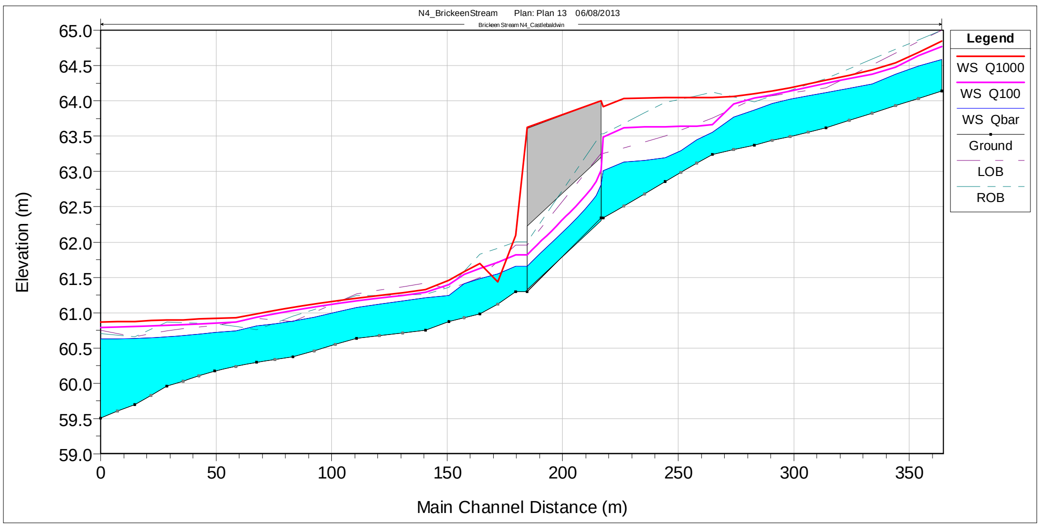
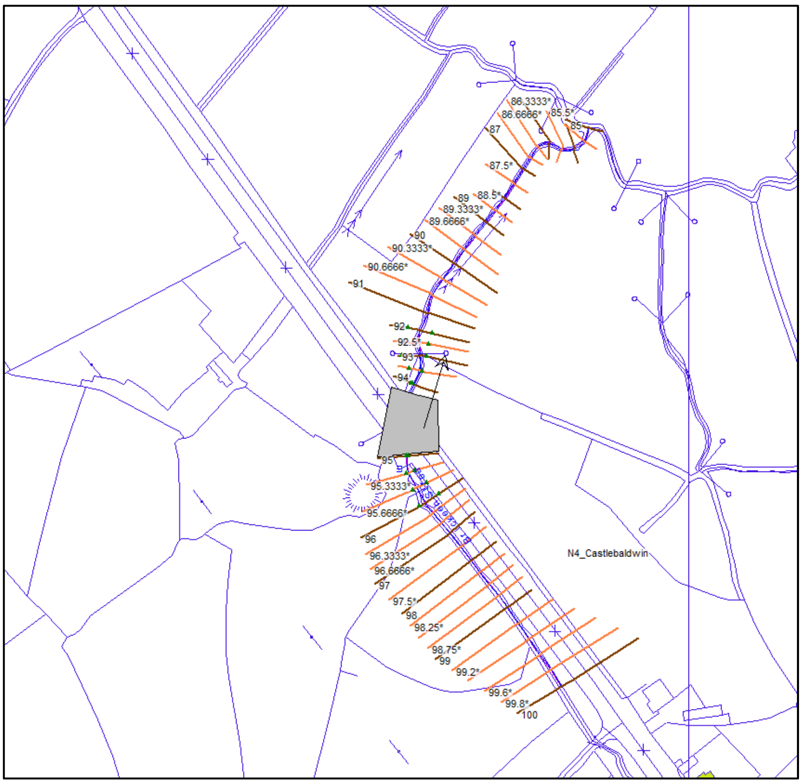


Figure 53: Pre-Construction Scenario Hydraulic Model Plan



5.2.6.3 Post Construction Scenario

The proposed culvert structures (Cul-61 and 62) are both 2.13m x 2.13m (H:W) and 36.5m and 9.5m long respectively. The proposed culverts dimensions include an embedment of 0.5m giving a maximum ope height 1.63m. In addition to the new culvert channel widening works will be required at the upstream end of the proposed N4-culvert. The existing Tributary of the Drumderry Stream culvert has an ope area of 0.64m^2 whilst the proposed culverts have effective ope areas of 3.5m^2.

Figure 54: Post Construction Scenario (Qbar, Q100 and Q1000)

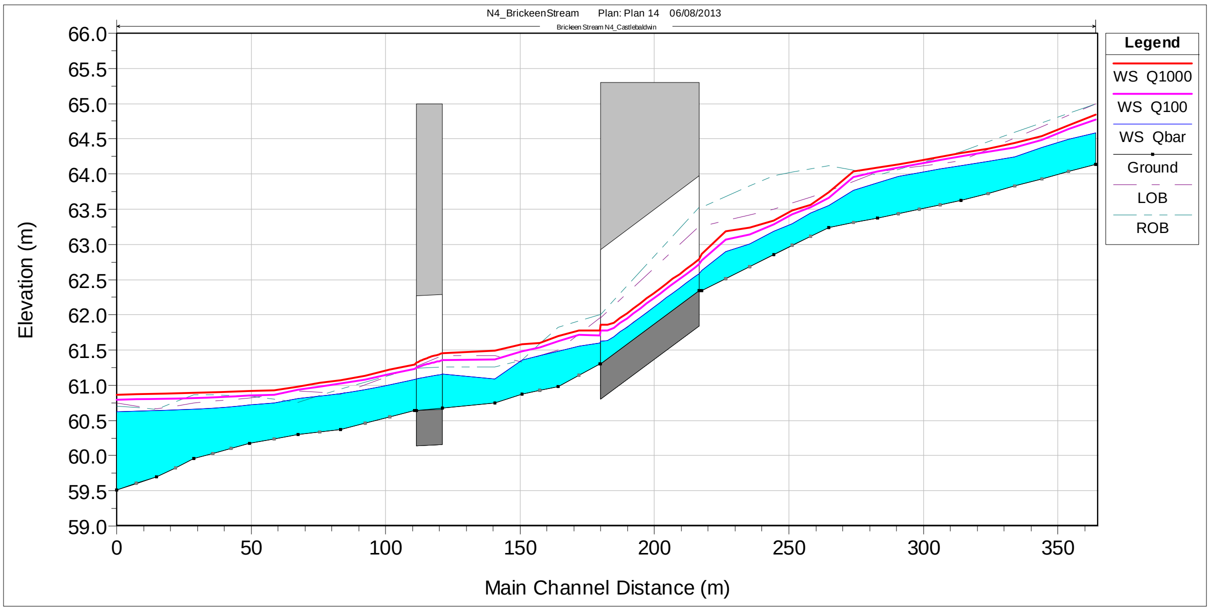
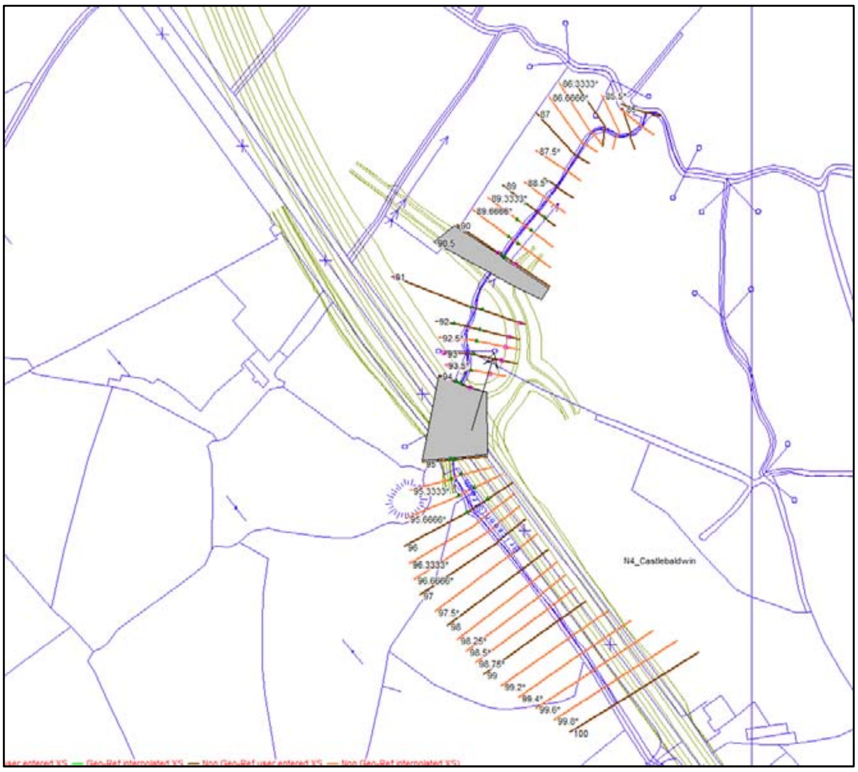


Figure 55: Post Construction Scenario Hydraulic Model Plan



5.2.6.4 Conclusion

The model clearly demonstrates that the proposed N4 culvert significantly reduces the flood risk upstream of the N4. Due to the encroachment of the proposed N4 and land access road embankments and the reduced effective flow area, a slight increase in flood level is expected between the proposed culverts.

Table 26 presents the calculated flood levels at the upstream and downstream ends of the existing culvert (and the site of the proposed downstream culvert) and the proposed culverts.

Table 26: Tributary of the Drumderry Stream Pre and Post Construction Scenario Flood levels (steady state flow conditions) (mOD)

XS	Q100	Q1000	Q100	Q1000
95	63.49	See Note	62.78	62.86
94	61.82	See Note	61.71	61.78
91	61.29	61.33	61.38	61.51
90	60.17	61.21	60.22	60.28

Note: The road is shown to overtop for the existing Q1000 design flows

The slight increase in downstream flood levels is due to the restricted right overbank flow area for the proposed scenario and is localised. No increase in flood level is projected downstream of the proposed road.

As the character of the design flood would be expected to be of long duration with a constant high flow an unsteady state simulation is deemed unnecessary.

5.3 Minor Culvert Flood Risk Analysis

All minor culverts have been assessed using hydraulic calculations based on Manning's equation. In general, all the minor culverts will be pipe culverts which will have minimum embedment of 0.15m. The minimum pipe size for culvert crossings is 900mm diameter. The culverts have been checked to ensure a minimum freeboard of 0.3m is obtained for the 1 in 100year design flow (Q100+FE+CC) and that the culvert does not become surcharged for a 1 in 1000year design flood (Q1000+FE+CC).

Following the assessment it is concluded that all pipe sizes proposed are suitably sized to convey the design flood flows with minimal afflux. Table 27 summarises the assessment and comments on minor culverts capacity. (Note: the maximum culvert gradient assessed is 1 in 150). There are opportunities to reduce a number of the culvert sizes proposed without increasing flood risk.

Table 27: Minor Culvert Assessment

Culvert Ref	Chainage	Design Flow		Proposed Culvert					Assessed Culvert Size, mm	Calculated Capacity		Comment
		Q100+FE, cumec	Q1000+FE, cumec	Type	Width, mm	Height or diameter, mm	Length, m	Gradient 1 in...		0.3m Freeboard	0m Freeboard	
Cul-Ret A	50	<0.14	<0.19	Concrete Circular Pipe		900	24.7	500	900	0.26	0.37	Pipe Size Adequate
Cul-Ret B	280	<0.14	<0.19	Concrete Circular Pipe		900	34.5	500	900	0.28	0.39	Pipe Size Adequate
Cul-Ret C	330	<0.14	<0.19	Concrete Circular Pipe		900	26	500	900	0.26	0.38	Pipe Size Adequate
Cul-Ret D	460	0.20	0.268	Concrete Circular Pipe		1050	101	500	1050	0.56	0.69	Pipe Size Adequate
Cul-Ret E	900	0.74	0.976	Concrete Circular Pipe		1800	52	500	1500	2.32	2.47	Pipe Size Adequate
Cul-Ret F	940	0.80	1.057	Concrete Circular Pipe		1800	22.4	500	1500	1.76	1.92	Pipe Size Adequate
Cul-Ret G	1270	<0.14	<0.19	Concrete Circular Pipe		900	51	500	900	0.3	0.4	Pipe Size Adequate
Cul-Ret H	1500	<0.14	<0.19	Concrete Circular Pipe		900	49	500	900	0.3	0.4	Pipe Size Adequate
Cul-Ret I	1600	0.20	0.268	Concrete Circular Pipe		1050	43	500	1050	0.29	0.4	Pipe Size Adequate
Cul - 1	2100	<0.14	<0.19	Concrete Circular Pipe		900	19	500	900	0.26	0.369	Pipe Size Adequate
Cul - 2	2380	0.645	0.856	Concrete Box	1520	2440	100	368.8	1200	0.995	1.153	Box size Adequate
Cul - 3	2500	0.370	0.492	Concrete Circular Pipe		1200	11	70.0	1200	0.873	1.074	Pipe Size Adequate
Cul - 4	2520	0.030	0.040	Concrete Circular Pipe		900	11	240.0	900	0.296	0.428	Pipe Size Adequate
Cul - 5	2610	0.030	0.040	Concrete Circular Pipe		600	60	64.9	750	0.269	0.469	Pipe Size Adequate
Cul - 6	2700	0.093	0.123	Concrete Circular Pipe		900	28	130.0	900	0.487	0.692	Pipe Size Adequate
Cul - 7	2980	0.096	0.127	Concrete Circular Pipe		900	24	442.9	900	0.273	0.389	Pipe Size Adequate
Cul - 10	3390	0.109	0.145	Concrete Circular Pipe		900	10	180.0	900	0.331	0.48	Pipe Size Adequate
Cul - 11	3540	0.205	0.272	Concrete Box	1220	1830	44	80.9	900	0.532	0.749	Box size Adequate
Cul - 12	3700	0.218	0.290	Concrete Box	1220	1520	54	65.0	900	0.55	0.771	Box size Adequate

Culvert Ref	Chainage	Design Flow		Proposed Culvert					Assessed Culvert Size, mm	Calculated Capacity		Comment
		Q100+FE, cumec	Q1000+FE, cumec	Type	Width, mm	Height or diameter, mm	Length, m	Gradient 1 in...		0.3m Freeboard	0m Freeboard	
Cul - 13	4000	0.013	0.018	Concrete Circular Pipe		600	29	195.0	750	0.214	0.374	Pipe Size Adequate
Cul - 15	4660	0.020	0.026	Concrete Circular Pipe		1050	6	150.0	1050	0.474	0.626	Pipe Size Adequate
Cul - 16	5300	0.827	1.098	Concrete Box	1520	2100	6	500.0	1800	1.007	1.123	Box size Adequate. Upsize if possib;e
Cul - 17	5300	0.827	1.098	Concrete Box	2130	2130	28	1140.0	1800	1.263	1.368	Box size Adequate
Cul - 18	5600	0.595	0.790	Concrete Box	1520	1830	29	59.1	1200	1.21	1.45	Box size Adequate
Cul - 19	6500	0.063	0.083	Concrete Circular Pipe		900	8	500.0	900	0.183	0.267	Pipe Size Adequate
Cul - 20	6560	0.010	0.013	Concrete Circular Pipe		600	29	310.0	750	0.17	0.297	Pipe Size Adequate
Cul - 21	6580	0.099	0.132	Concrete Circular Pipe		900	23	310.0	900	0.323	0.461	Pipe Size Adequate
Cul - 22	6600	0.099	0.132	Concrete Circular Pipe		1350	38	480.0	900	0.29	0.409	Pipe Size Adequate
Cul - 23	6720	0.020	0.026	Concrete Circular Pipe		900	20	120.0	900	0.501	0.717	Pipe Size Adequate
Cul - 24	6700	0.086	0.114	Concrete Circular Pipe		900	28	320.0	900	0.333	0.474	Pipe Size Adequate
Cul - 26	8620	0.030	0.040	Concrete Circular Pipe		900	64	535.7	900	0.298	0.417	Pipe Size Adequate
Cul - 27	8650	0.083	0.110	Concrete Circular Pipe		900	16	380.0	900	0.265	0.38	Pipe Size Adequate
Cul - 28	8650	0.126	0.167	Concrete Circular Pipe	300	1200	85	30.5	900			Pipe Size Adequate
Cul - 29	9250	0.026	0.035	Concrete Circular Pipe		900	6	320.0	900			Pipe Size Adequate
Cul - 30	9310	0.023	0.031	Concrete Circular Pipe	300	900	10	200.0	900			Pipe Size Adequate
Cul - 31	9320	0.165	0.220	Concrete Circular Pipe		1050	9	160.0	1050	0.54	0.708	Pipe Size Adequate
Cul - 32	9400	0.364	0.483	Concrete Box	1520	1520	48	133.3	1200	1.372	1.62	Box size Adequate
Cul - 33	9400	0.331	0.439	Concrete Circular Pipe	450	1050	6	45.5	1050	0.474	0.626	Pipe Size Adequate

Culvert Ref	Chainage	Design Flow		Proposed Culvert					Assessed Culvert Size, mm	Calculated Capacity		Comment
		Q100+FE, cumec	Q1000+FE, cumec	Type	Width, mm	Height or diameter, mm	Length, m	Gradient 1 in...		0.3m Freeboard	0m Freeboard	
Cul - 34	9640	0.112	0.149	Concrete Box	1520	1830	42	500.0	900	0.289	0.407	Box size Adequate
Cul - 35	9740	0.145	0.193	Concrete Circular Pipe	450	1350	38	35.7	900			Pipe Size Adequate
Cul - 36	10130	0.470	0.624	Concrete Box	1520	1830	50	36.3	900	0.543	0.763	Box size Adequate
Cul - 37	10150	0.056	0.075	Concrete Circular Pipe		900	5	5.5	900			Pipe Size Adequate
Cul - 38	10250	0.060	0.079	Concrete Circular Pipe		900	11	115.4	900			Pipe Size Adequate
Cul - 41	10740	0.152	0.202	Concrete Circular Pipe		1050	10	42.5	1050			Pipe Size Adequate
Cul - 42	10750	0.106	0.141	Concrete Circular Pipe		900	10	140.0	900	0.363	0.525	Pipe Size Adequate
Cul - 43	10840	0.393	0.523	Concrete Box	1520	1520	10	180.0	1050	0.53	0.694	Box size Adequate
Cul - 44	10840	0.066	0.088	Concrete Circular Pipe		900	14	83.5	900			Pipe Size Adequate
Cul - 45	11300	0.083	0.110	Concrete Circular Pipe	500	1500	40	33.2	900			Pipe Size Adequate
Cul - 46	12230	0.152	0.202	Concrete Box	1220	1520	6	340.0	1050	0.315	0.416	Box size Adequate
Cul - 48	12400	0.132	0.176	Concrete Circular Pipe		900	6	120.0	900			Pipe Size Adequate
Cul - 49	12560	0.265	0.351	Concrete Circular Pipe		1050	6	160.0	1050	0.459	0.607	Pipe Size Adequate
Cul - 50	13110	0.149	0.198	Concrete Circular Pipe		900	10	26.7	900			Pipe Size Adequate
Cul - 51	13300	0.010	0.013	Concrete Circular Pipe		900	11	320.0	900			Pipe Size Adequate
Cul - 52	13440	0.017	0.022	Concrete Circular Pipe		900	16	340.0	900			Pipe Size Adequate
Cul - 53	13560	0.169	0.224	Concrete Circular Pipe		1050	68	416.7	1050	0.579	0.724	Pipe Size Adequate
Cul - 54	13600	0.198	0.263	Concrete Circular Pipe		1050	6	437.5	1050			Pipe Size Adequate
Cul - 55	13570	0.033	0.044	Concrete Circular Pipe		900	20	433.3	900			Pipe Size Adequate
Cul - 56	13700	0.142	0.189	Concrete Circular Pipe		900	19	146.7	900			Pipe Size Adequate

Culvert Ref	Chainage	Design Flow		Proposed Culvert					Assessed Culvert Size, mm	Calculated Capacity		Comment
		Q100+FE, cumec	Q1000+FE, cumec	Type	Width, mm	Height or diameter, mm	Length, m	Gradient 1 in...		0.3m Freeboard	0m Freeboard	
Cul - 58	13650	0.301	0.400	Concrete Circular Pipe		1050	8	133.3	1050			Pipe Size Adequate
Cul - 59	14080	0.043	0.057	Concrete Circular Pipe		900	5	60.0	900			Pipe Size Adequate
Cul - 60	14120	0.023	0.031	Concrete Circular Pipe		900	10	300.0	900			Pipe Size Adequate

6 Flood Risk Assessment and Management

6.1 Introduction

Hydraulic models have been produced for the existing and proposed scenarios for the N4-Collooney to Castlebaldwin *Proposed Road Development* for both steady state and unsteady state flood flows for the six main watercourse crossings. The flood risk associated with each of these proposed crossings is discussed in the following sections.

Flood risk mapping, based on the flood zoning definitions as set out in the OPW Planning and Flood Risk Management Guidelines, have been produced for the floodplains associated with each of these crossings where Lidar data was made available. The flood risk maps were developed using Lidar topographical data for the study area and the flood profiles calculated for each of the six crossings for the 1 in 100year and 1 in 1000year design flows. These maps allow the extents of encroachment of the *Proposed Road Development* to be quantified and appropriate mitigations, if required, to be recommended.

An assessment of the proposed minor culverts has been undertaken and where significant increases in flood risk / afflux are identified recommended culvert sizes have been provided.

6.2 Main River/Stream Crossings

6.2.1 **Markree Demesne Stream**

6.2.1.1 Hydraulic Modelling Summary

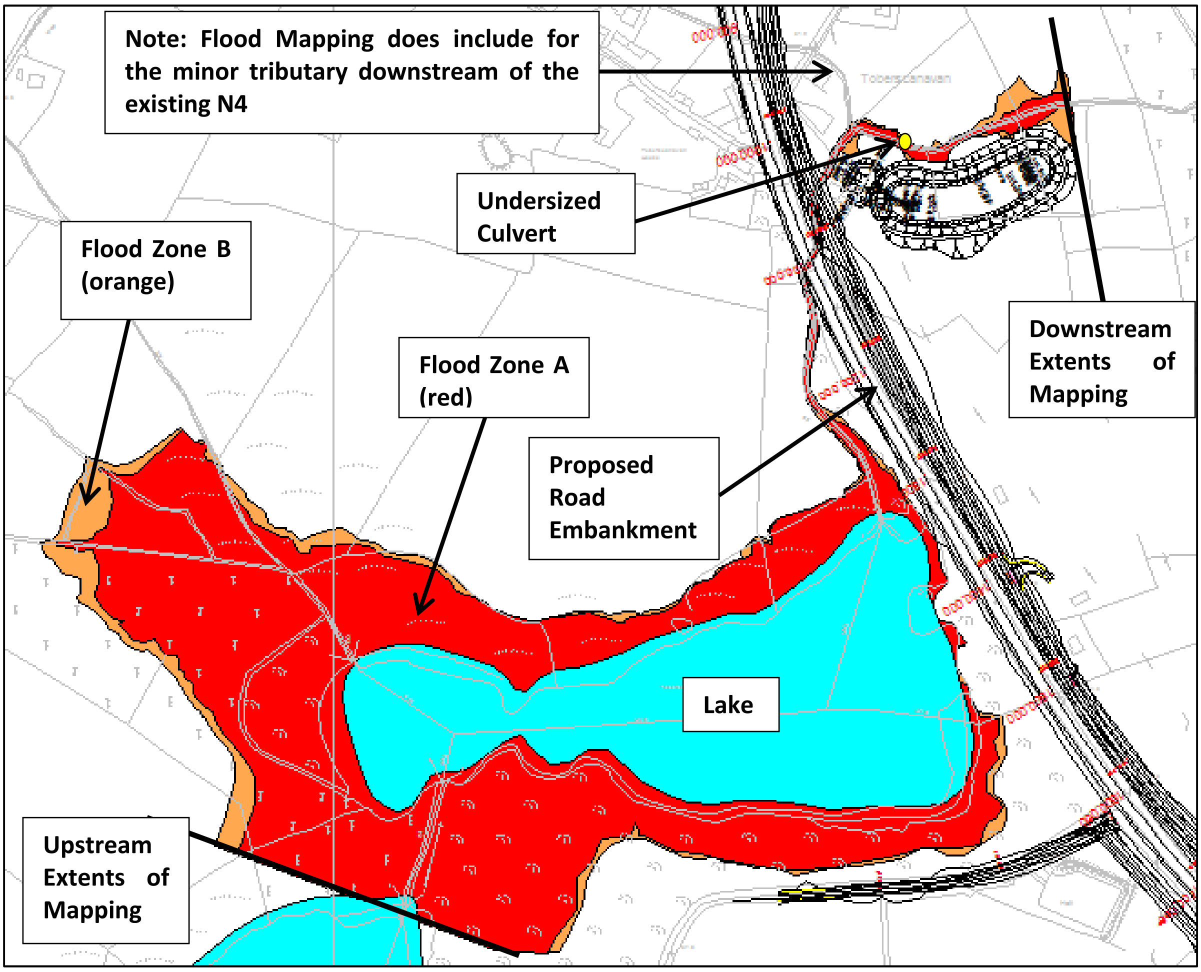
Hydraulic models were developed for the existing and proposed scenario for the Markree Demesne Stream and both steady and unsteady state design floods were simulated. The hydraulics models for the existing scenario demonstrates the afflux caused by the existing N4-pipe culvert and the attenuation provided by Toberscanavan Lough (lower) and allows the low flow and average flow lake levels to be calculated.

A comparison of the existing and proposed scenario models demonstrated that by replacing the existing pipe culvert with a larger culvert that the upstream lake levels reduce and peak downstream flood levels increase by the order of 0.13m. The capacity of the channel upstream of the N4 (both due to size and roughness) restricts the reduction in upstream water levels due to the replacement culvert.

6.2.1.2 Flood Mapping

Flood Mapping has been produced for both the 1 in 100year and 1 in 1000year design flood conditions for the existing scenario in the vicinity of the proposed Markree Demesne Stream crossing. Figure 56 below shows the Flood Zone Mapping for the lands upstream and downstream of the existing N4 crossing. The existing N4 and proposed link road (other than the stream crossing) lie outside Flood Zone B. The proposed attenuation pond downstream of the proposed link road encroaches on a small area (<0.1ha) of lands shown to be in Flood Zone B. It has been demonstrated above that this encroachment does not impact on flood levels in the stream.

Figure 56: Markree Demesne Stream Flood Zone Mapping



6.2.1.3 Flood Risk

The proposed bridge structure has been predicted to decrease the flood level upstream in the lake by 0.10m and 0.13m for the 1 in 100 and 1 in 1000year flood event. This decrease in flood levels leads to a minor decreased inundation of agricultural lands adjacent to the lake. This decrease in upstream flood storage leads to an increase in flood level peak of between 0.08 to 0.17m extending 420m downstream of the existing culvert for the Q100 and Q1000 design floods. The low flow and average flow lake levels decrease by 0.1m and 0.14m respectively due to the proposed culvert works.

6.2.1.4 Mitigations

While the proposed attenuation ponds are shown to encroach slightly into Flood Zone B, the impact on flood levels and the comparative loss of flood storage is negligible and therefore no mitigation are required. However, relocating the main attenuation pond 8m south would take it out of Flood Zone B entirely. The proposed culvert will decrease lake levels upstream and increase peak flood levels downstream. The installation of a weir upstream of the proposed culvert, with fish pass facilities, would allow the existing low flow and average flow lake levels to be retained and therefore any negative impact on the upstream riparian habitat to be avoided while also reducing upstream flood risk. Minor channel improvement works and removal of undersized culverts and other stream crossings downstream would mitigate the increase in flood risk downstream.

6.2.2 **Turnalaydan Stream**

6.2.2.1 Hydraulic Modelling Summary

Hydraulic models were developed for the existing and proposed scenario for the Turnalaydan Stream and unsteady state design floods were simulated. The existing structure at Lackagh Bridge has been shown to cause a noticeable afflux. The proposed 20m span bridge structure over the Turnalaydan Stream has been shown to cause a minor afflux at peak flood conditions (0.06m and 0.10m for the Q100 and Q1000 design flows respectively).

6.2.2.2 Flood Mapping

Flood Mapping has been produced for both the 1 in 100year and 1 in 1000year design flood conditions for the existing scenario in the vicinity of the proposed Turnalaydan Stream crossing. Figure 57 below shows the Flood Zone Mapping for the lands upstream and downstream of the existing N4 crossing. Approximately 410m of the proposed road embankment length has been shown to cross through the Turnalaydan Stream floodplain (4+300 to 4+710). The floodplain narrows significantly approximate 210m downstream of the proposed crossing. The afflux at Lackagh Bridge during design flood conditions would appear to greatly negate the beneficial storage / attenuation in flood plain.

6.2.2.3 Flood Risk

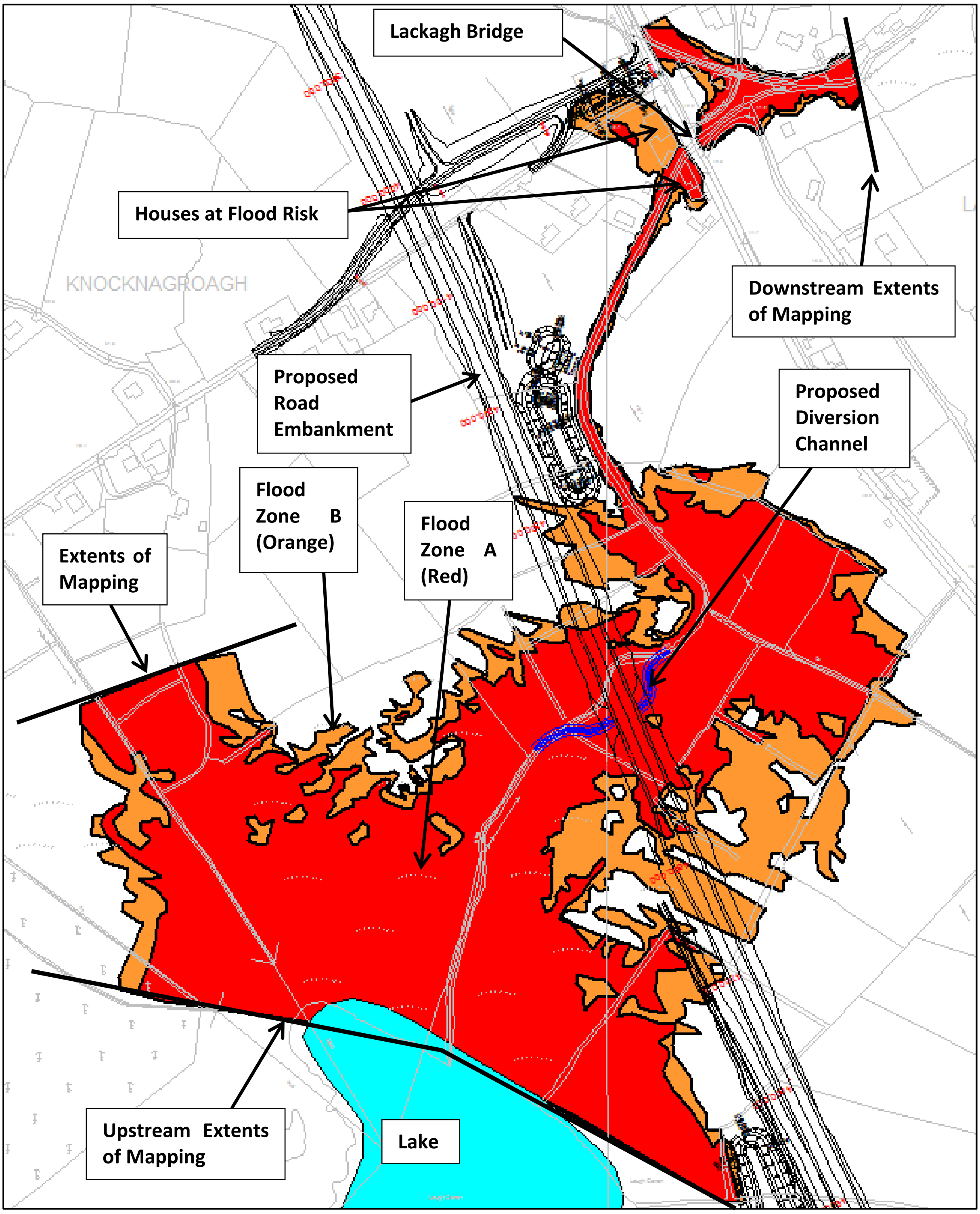
The existing Lackagh Bridge poses a high flood risk to adjacent upstream properties and the existing N4 road. The proposed bridge structure has been shown to cause minor increase in flood levels upstream (i.e. 0.01m and 0.07m for the Q100 and Q1000 design floods respectively). The construction of the road embankment in the floodplain has been shown to have minor impact on flood levels and has negligible increase flood risk downstream. Hydraulic modelling of the pre-construction scenario calculated flood levels of 41.35mOD and 41.75mOD immediately upstream of Lackagh Bridge for the Q100 and Q1000 flood flows. The existing low lying house adjacent to the bridge, which has a finished floor level of 40.57mOD, is shown to be at very high flood risk.

6.2.2.4 Mitigations

While the proposed attenuation ponds is shown to encroach slightly into Flood Zone B, the impact on flood levels and the comparative loss of flood storage is negligible and therefore no mitigation are required.

The road embankment crosses three large open drains at Ch 4+340, Ch 4+385 and Ch4+540. It is recommended that large pipe culverts (1500mm diameter or greater) be provided at these locations with inverts matching the existing channels to maintain connectivity and conveyance across the flood plain. During normal flow conditions these pipe culverts will preserve the existing drainage regime in the adjoining lands. Construction of open drains offset from the toe of the road embankment would further improve flow conveyance in the flood plain for both normal and flood conditions in addition to promoting flows away from the embankment which otherwise could lead to erosion.

Figure 57: Turnalaydan Stream (Lough Corran outflow) Flood Zone Mapping



6.2.3 Drumfin River

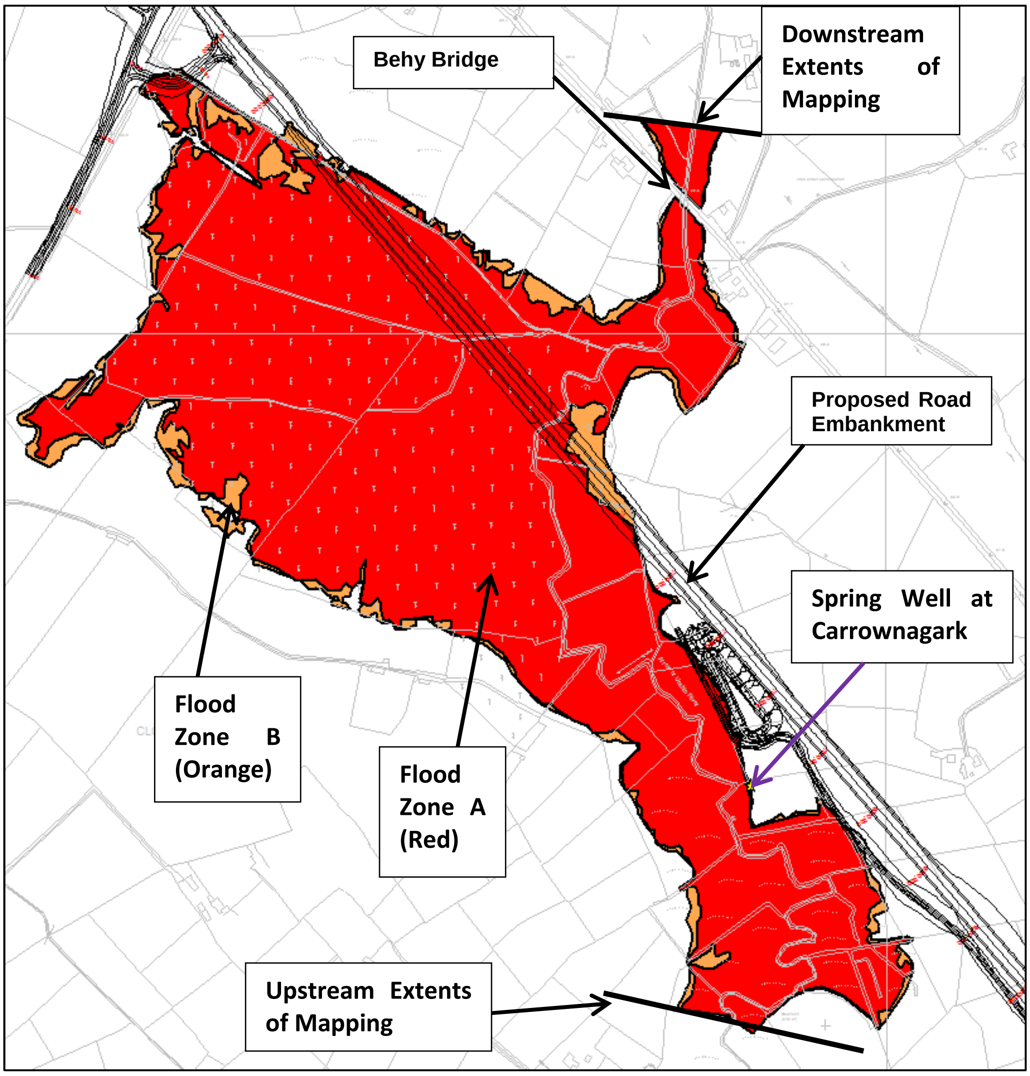
6.2.3.1 Hydraulic Modelling Summary

Hydraulic models were developed for the existing and proposed scenario for the Drumfin River and both steady and unsteady state design floods were simulated. The unsteady state models for the existing scenario demonstrate the relevance of the upstream flood plain storage regarding attenuation of flood peaks. The existing structure at Behy Bridge has been shown to cause a noticeable afflux. The channel invert levels upstream of Behy Bridge for circa 150m are higher than that within the flood plain further upstream for a considerable length of channel. The proposed 20m span bridge structure over the Drumfin River has been shown to cause a minor afflux at peak flood conditions. The increase in afflux leads to increased storage upstream which in turn attenuates peak flood flows (by less than 0.5 cumec) downstream.

6.2.3.2 Flood Mapping

Flood Mapping has been produced for both the 1 in 100year and 1 in 1000year design flood conditions for the existing scenario in the vicinity of the proposed Drumfin River crossing. Figure 58 below shows the Flood Zone Mapping for the flood plain upstream of Behy Bridge. 610m of the proposed road embankment length has been shown to cross through the Drumfin River floodplain (6+900 to 7+510). The floodplain narrows significantly approximate 100m downstream of the proposed crossing. The off / on ramp road at Ch6+700 encroaches into Flood Zone A and B. The open drain associated with this section of the flood plain is to be diverted as part of the proposed road development.

Figure 58: Drumfin River Flood Zone Mapping



6.2.3.3 Flood Risk

The proposed bridge structure has been predicted to cause an afflux of 0.1m for the 1 in 1000year flood event. This increase in flood levels leads to a minor increased inundation of agricultural lands not extending greater than 1.4km upstream of the proposed bridge. Therefore the proposed bridge causes a minor increase in flood risk in the upstream flood plain. This increase in flood levels leads to increased upstream flood storage which has been shown to further attenuate flows downstream of the proposed bridge. In effect the proposed bridge reduces flood risk downstream by a minor amount.

The proposed road level of 54.6mOD at the bridge site is greater than 3m above the predicted 1 in 1000year flood level. The proposed bridge soffit level is 0.73m higher (freeboard) than the predicted 1 in 1000year flood level. The existing road level at Behy Bridge of 52.5mOD is >1m higher than the predicted 1 in 1000year flood level.

The proposed land access road and attenuation pond site to the south of the road alignment between Ch7+675 and Ch7+850 lies partially in the Flood Zone A and B. They are located at a sufficient distant from the river to not impede overbank conveyance and the relative lose in flood storage is negligible relevant to the adjoin flood plain area. Their construction will have negligible impact on flood levels and therefore flood risk in the Drumfin River flood plain.

The existing Carrownagark Spring Well pump house, which has a finished floor level of 51.67mOD has been shown to be located in Flood Zone B. The surveyed river level on 25th June 2013 was 49.99mOD adjacent to the pump house. The table 28 below presents the predicted Q100 and Q1000 flood levels at this pump house for the pre and post construction scenarios. The post-construction Q1000+FE flood level is 0.09m lower than the surveyed finished floor level of the pump house.

Table 28: Pre and Post Construction flood levels at Carrownagark Spring Well.

6.2.3.4 Mitigations

The proposed road embankment crosses 610m of active flood plain. In addition to reducing flood storage, the embankment will reduce overbank conveyance and the connectivity of the flood plain. The road embankment crosses three large open drains at Ch 6+880, Ch 7+210 and Ch7+460. It is recommended that large pipe culverts (1500mm diameter or greater), with inverts matching the existing open drains, be provided at these locations to maintain connectivity across the flood plain. These additional culverts will increase conveyance across the flood plain and will reduce upstream flood risk marginally for the design floods, including the lands adjacent to Carrownagark spring well pump house.

During normal flow conditions these pipe culverts will preserve the existing drainage regime in the adjoining lands. Construction of open drains offset from the toe of the road embankment would further improve flow conveyance in the flood plain for both normal and flood conditions in addition to promoting flows away from the embankment which otherwise could lead to erosion.

6.2.4 **Springfield Stream**

6.2.4.1 Hydraulic Modelling Summary

Hydraulic models were developed for the existing and proposed scenario for the Springfield Stream and both steady and unsteady state design floods were simulated. The model simulations for both design floods (unsteady state) conclude that there is a negligible impact on both stage and design flows therefore confirming that the proposed road will have a negligible impact on flood risk at Tawnagh. The degree of afflux due to the diversion will be dependent on the proposed diversion channel's capacity. The existing 900mm diameter culvert downstream of the proposed bridge was shown to be overtopped during design flood conditions: however, the associated afflux does not extend upstream to the proposed road.

The proposed road embankment passes through an area with little overbank flooding and conveyance and therefore has negligible impact on flood plain storage. The model confirms (by way of comparison between the upstream and downstream flows) that the road will have negligible impact on flood storage.

6.2.4.2 Flood Mapping

Flood Mapping has been produced for both the 1 in 100year and 1 in 1000year design flood conditions for the existing scenario in the vicinity of the proposed Springfield Stream crossing. Figure 59 below shows the Flood

Zone Mapping for Springfield Stream extending from Ardloy Bridge to the lake at Tawnagh. The proposed road embankment crosses through a small area of floodplain.

6.2.4.3 Flood Risk

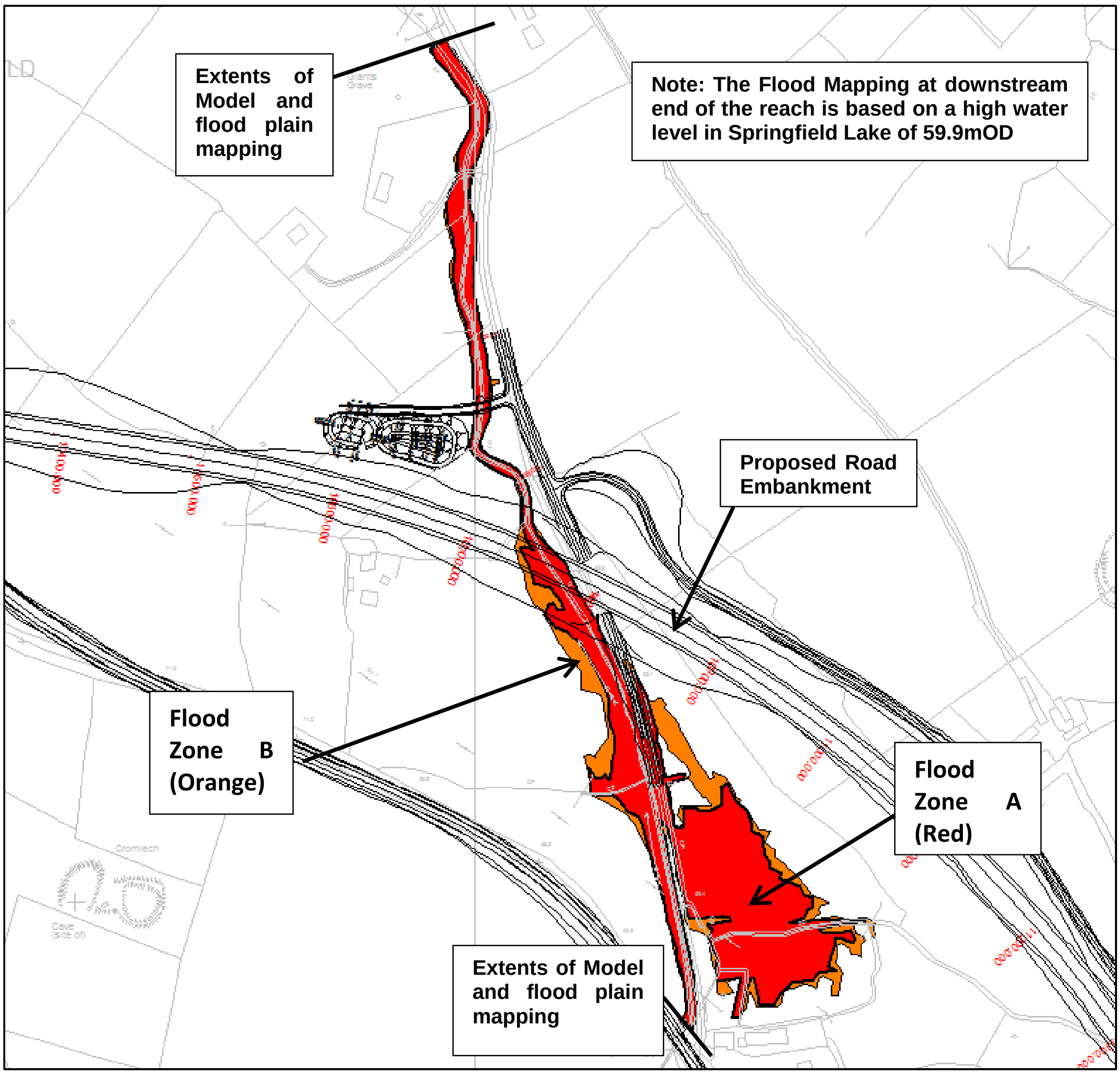
The proposed road embankment at Tawnagh is located above the 62mOD contour and therefore lies well out of on the 'Lake at Tawnagh' flood plain.

The proposed attenuation pond site to the north of the road alignment between Ch10+585 and Ch10+690 lies above the Q1000+FE flood level and therefore will have negligible impact on flood risk in the Springfield Stream flood plain.

6.2.4.4 Mitigations

It is recommended that the proposed diversion channel have a minimum width (bed level) of 3m and profiled to include a low flow channel.

Figure 59: Springfield Stream Flood Zone Mapping



6.2.5 Lissycoyne Stream

6.2.5.1 Hydraulic Modelling Summary

Hydraulic models were developed for the existing and proposed scenario for the Lissycoyne Stream and both steady and unsteady state design floods were simulated. The existing culvert under the Bog Road, located immediately upstream of the proposed culvert crossings, is clearly under capacity and has been shown to overtop and to cause a significant afflux during flood conditions.

Comparison of the steady state hydraulic models concludes that the proposed scenario will cause a small localised impact on flood levels between the new road and the Bog Road of 2cm during a Q1000 flood event.

The model confirms (by way of comparison between the upstream and downstream flows) that the road will have negligible impact on flood storage

6.2.5.2 Flood Mapping

Flood Mapping has been produced for both the 1 in 100year and 1 in 1000year design flood conditions for the existing scenario in the vicinity of the proposed Cleavry Stream crossings. Figure 60 below shows the Flood Zone Mapping for stream reach upstream and downstream of Bog Road. The proposed road embankment is shown to cross through the streams floodplain at two sections namely at Ch12+000 and between Ch12+150 to Ch12+250. The majority of the crossed floodplain areas are associated with minor open drains.

6.2.5.3 Flood Risk

The existing Bog Road Culvert has been shown to cause significant afflux during flood conditions and the local road is at high flood risk.

The proposed culverts and road embankment across the study area's flood plain will have a very minor impact of on effective storage during flood conditions. Minor increases in flood levels (2cm) are predicted to occur upstream of the proposed culverts as far as Bog Road due primarily to the increase channel length associated with the channel diversion.

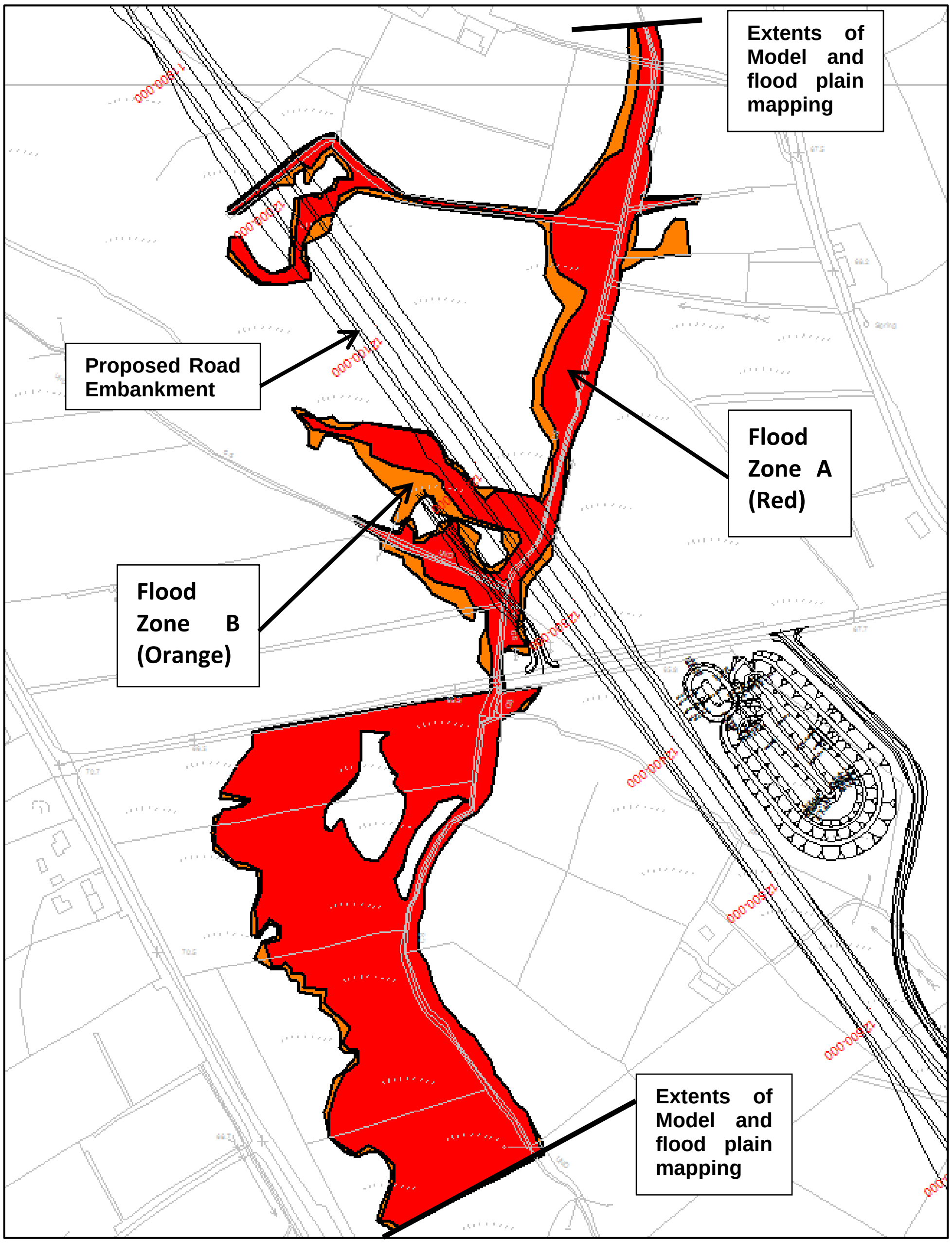
The proposed N4 road level at the crossing is located well above the 1 in 1000year flood level and therefore at low flood risk.

6.2.5.4 Mitigations

The proposed road embankment intercepts local open drains between Ch 12+000 and 12+100. It is recommended that a 900mm diameter culvert be installed at Ch 12+000, with inverts matching the existing open drains, and an open drain constructed along the upstream toe of the embankment as far as the proposed main culvert in order to preserve the existing drainage regime in the area.

Upgrading of the Bog Road culvert will reduce flood risk at and upstream of the local road while also reducing the existing effective upstream storage. If it is proposed that Bog Road culvert is upgraded then consideration should be given to upsizing the proposed culverts to further reduce the upstream flood risk.

Figure 60: Lissycoyne Stream Flood Zone Mapping



6.2.6 Tributary of the Drumderry Stream

6.2.6.1 Hydraulic Modelling Summary

Hydraulic models were developed for the existing and proposed scenario for the Tributary of the Drumderry Stream and steady state design floods were simulated. It is proposed to replace the existing culvert under the N4 and to construct a second culvert downstream under a proposed land access road. The hydraulic models for the existing scenario confirmed that the existing culvert is undersized and would be expected to be overtopped during a 1 in 1000year flood event. As the character of the design flood would be expected to be of long duration with a constant high flow an unsteady state simulation is deemed unnecessary.

The proposed scenario which includes some upstream channel improvements has been shown to significantly reduce flood levels upstream.

6.2.6.2 Flood Mapping

No flood mapping was produced for this proposed crossing as there is no Lidar information for the flood plain was available. The lands downstream of the N4 are designated as benefitting lands and are low lying. The steady state model demonstrated that the pre-construction N4-road is at a high / medium flood risk due to the inadequate existing culvert size. The proposed downstream access road is likely to lie within the 1 in 100year flood plain.

6.2.6.3 Flood Risk

The existing culvert poses a high flood risk to the existing N4. The proposed replacement culvert under the crossing will reduce upstream flood risk. There will be a minor increase in flood risk between the N4 culvert and the proposed downstream culvert due to encroachment of the local access road embankment into the flood plain.

The proposed road level at the N4 crossing is greater than 2m above the 1 in 1000year flood level and therefore at low flood risk.

6.2.6.4 Mitigations

The existing channel upstream of the proposed replacement culvert and to the west of the N4 road alignment was shown to be under capacity and will need to be widened / upgraded for at least 50m upstream of the proposed N4. For fisheries purposes the existing low flow channel is to be retained and the final channel design finalised with the IFI.

6.2.7 Loss of Storage

The loss of storage due to the road embankments crossing floodplain has been estimated for each new crossing (i.e. Toberscanavan and Tributary of the Drumderry Streams are on line improvements in general. The table 29 below summarises the approximate encroachment area and average depths at the encroachments at the crossing:

Table 29: Storage Loss due to proposed road embankment encroachment.

Crossing	Area (m2)@		Average Depth (m)*		Volume (m3)#	
	Flood Zone A	Flood Zone B	Flood Zone A	Flood Zone B	Flood Zone A	Flood Zone B
Markree Demesne Stream	0	0	0	0	0	0
Turnalaydan Stream	6650	12910	0.347	0.377	2308	4867
Drumfin Stream	18940	22900	0.507	0.627	9603	14358
Springfield Stream	1694	2332	0.074	0.153	125	357
Lissycoyne Stream	3200	4400	0.111	0.14	355	616

Notes:

1. @= includes the bridge ope, * = includes the existing channel, # = includes the culvert ope and channel volume

2. The storage loss associated with Tributary of the Drumderry Stream has not been calculated due to insufficient topographical data (Lidar).
3. The proposed N4-upgrade works and link road at the Markree Demesne Stream Crossing lies outside the streams floodplain.

The following table 30 presents the estimated storage lost due to embankment encroachment in terms of time relative to the design peak flow.

Table 30: Storage Loss due to proposed road embankment encroachment.

Crossing	Design Flood, cumec		Storage lost in terms of time relative to Peak Flow, minutes	
	1 in 100year	1 in 1000year	1 in 100year	1 in 1000year
Turnalaydan	13.92	18.48	2.8 mins	4.4 mins
Drumfin	20.28	26.94	7.9 mins	8.9 mins
Springfield Stream	1.25	1.66	1.7mins	3.6 mins
Lissycoyne Stream	1.67	2.21	3.5mins	4.6 mins

Note: the storage lost is less than shown due to the inclusion of the channel and culvert ope volume through the embankment as 'storage lost'

The impact of the storage loss for Springfield Stream and Lissycoyne stream is insignificant and, as proven by the hydraulic modelling, has a negligible impact. The 'storage time' lost at the Turnalaydan Stream and Drumfin River floodplain crossings are minor relative to the design flood flows. No compensatory storage is required for the proposed main channel crossings. The excavation of toe drains along the proposed embankments within the flood plains will increase storage and help mitigate losses due to embankment encroachment.

6.3 Minor Culvert Crossings

All pipe and box culvert sizes proposed are suitable conservatively large to convey the design flood flows with minimal afflux. The minimum culvert size is to be a 900mm diameter pipe.